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Developing Embankment & Subgrade Stabilization Regional Specifications

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DEVELOPING EMBANKMENT AND SUBGRADE STABILIZATION REGIONAL SPECIFICATIONS

Project Report

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EXECUTIVE SUMMARY

One of the major causes of poor or unstable highway conditions in Arkansas and elsewhere in the United States is due to improper placement methods of subgrade soils. Likewise, weak embankment fills and in-situ natural soils contribute to premature failure of embankments. Poor subgrade soils require stabilization to provide a stable platform during the earthwork construction operation. Engineers in Arkansas are often faced with such problematic soils, which do not possess sufficient strength to support wheel loads imposed upon them during construction. Thus, it is necessary to stabilize these soils to provide a stable subgrade or a working platform for the construction of pavements and embankments.

The main objective of this research was to establish useful guidelines for subgrade and embankment stabilization and create standard rehabilitation techniques for each geological region in the state. The research team conducted a review and evaluation of current AHTD soil stabilization procedures. Interviews of AHTD district and construction personnel were conducted to gain insight of their soil stabilization experience in their respective districted. An evaluation of the state geological conditions, in terms of surficial soil types, was conducted. The experience of neighboring states' soil stabilization techniques was researched and summarized in this report. Preliminary stabilization methods (Portland cement, Lime and Fly Ash) were evaluated. Test sites were selected in consultation with AHTD's research and district construction personnel. Soil stabilization was performed at the test site; the method of stabilization was recommended by the Primary Investigator (PI) after consultation with district construction personnel. Soil samples from the test sites were collected and transported to Arkansas State University (ASU) geotechnical engineering laboratory, where specific tests were performed. Site monitoring was conducted by AHTD field personnel and the research team. Pavement surface roughness data was collected at the test sites by AHTD's Pavement Management team and the International Roughness Index (IRI) and rutting results were then evaluated by the research team. The collected data was then analyzed and recommended guidelines for performing soil stabilization were accordingly prepared. These guidelines are presented in this report.

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1 INTRODUCTION

1.1 PROBLEM STATEMENT

The Arkansas State Highway and Transportation Department (AHTD) stabilizes embankments and subgrade soils for construction projects on routine basis, but it lacks specifications for stabilization techniques and subsequent monitoring steps. This project employed selected stabilization techniques for subgrades and embankments up to ten feet in height based on geological regions. These techniques included lime, cement and Class-C fly ash (CFA) stabilizations in various projects throughout Arkansas. Along with establishing guidelines for stabilization, this project established testing protocols for preliminary design and construction.

1.2 BACKGROUND AND MOTIVATION

One of the major causes of poor or unstable highway conditions in Arkansas and elsewhere in the United States is due to improper placement methods of subgrade soils. Likewise, weak embankment fills and in-situ natural soils contribute to premature failure of embankments. Poor subgrade soils require stabilization to provide a stable platform during the earthwork construction operation. As shown in Figure 1.1, engineers in Arkansas are often faced with such problematic soils, which do not possess sufficient strength to support wheel loads imposed upon them during construction. Thus, it is necessary to stabilize these soils to provide a stable subgrade or a working platform for the construction of pavements and embankments.

Figure 1.1 Example of Construction Operation Problem due to Soft Soil

Soil properties such as strength, compressibility, hydraulic conductivity, workability, swell potential, and volume change tendencies may be altered by various soil stabilization methods. Some of the common stabilization approaches are described below:

a) <u>Natural Drying</u>: If the construction schedule allows, the soil can be scarified and given time to naturally dry. This will require favorable weather conditions (dry periods of windy weather and warm temperature for at least a few days). Once the moisture contents are reduced to near optimum levels, the soil can then be compacted and the earthwork operation can proceed.

b) Remove and replace weak subgrade soils with granular materials: A common remedial procedure for a wet and soft subgrade is to cover it with granular material or to partially remove and replace the wet subgrade with a granular material to a pre-determined depth below the grade lines. Usually 12 to 24 inches (300 to 600mm) of granular materials are used for subgrade modification or stabilization.

c) Geosynthetics: One or more layers of geotextile fabric or geogrid can be placed above the excessively wet or weak soil to allow for the placement and compaction of subsequent layers. The inclusion of geogrid in subgrades changes the performance of the roadway in many ways. Tensile reinforcement, confinement, stress reduction in underlying subgrade layers, separation, construction uniformity and reduction in strain have been identified as primary benefits of the use of geogrids.

d) Lime Stabilization: Wet soils can be mixed with lime to reduce the moisture content, thus allowing for compaction to proceed. It is desirable that monovalent cations such as sodium and potassium which are commonly found in highly plastic clay soils, to be exchanged with cations of higher valences such as calcium, which can be found in lime [1]. This ion exchange process takes place quite rapidly, often within a few hours, resulting in a reduction in plasticity index (PI), an increase in shear strength of the clay soil and an improvement in texture from a cohesive material to a more granular one. This approach has its own limitations as it takes time, special equipment, and requires warm weather to gain desirable property [2]. Also, if the lime treatment results in soil pH of higher than 12.6, the lime treatment will have a cementitious behavior [3].

e) Cement Stabilization: Wet soils can be mixed with cement to increase their strength and stiffness and reduce their moisture contents. This approach has some limitations as well. It does not effectively work with soils that have relatively high liquid limits.

f) Fly Ash (Class C) Stabilization: Class C Fly ash (CFA) is an industrial byproduct that comes primarily from coal-fired power plants, and it has cementitious, as well as pozolanic (contains silica, alumina and calcium based minerals) properties.

g) Stone Surging: This approach requires multiple applications of relatively thin layers of stone and successively compacting them into the wet soil until a stable platform is achieved.

In the last few decades, pavement engineers have been challenged to build, repair and maintain pavement systems with enhanced longevity and reduced costs. Specifically, efforts have been made to improve the design methodology and to establish techniques for modification of highway pavement and embankment materials. Among the aforementioned stabilization techniques, cementitious stabilization is widely used in Arkansas and elsewhere as a remedial method to improve subgrade and embankment soil properties (e.g., strength, stiffness, swell potential, workability and durability) through the addition of cementitious additives. It consists of mixing stabilizing agents such as lime, cement kiln dust (CKD), lime kiln dust (LKD), and Class C fly ash (CFA) with soil. In the presence of water, these agents react with soil particles to form cementing compounds that are responsible for the improvement in engineering properties such as strength and stiffness. However, the degree of enhancement is influenced by many factors such as the stabilizing agent type, the type of soil to be stabilized, curing time, the required strength, the required durability, cost, and environmental conditions [5, 6].

1.3 SCOPE

This study included the review of existing stabilizing techniques and construction of stabilized sections in nine of the 10 AHTD districts with three additives: lime, cement and CFA. Since the natural roadbeds in the AHTD District 4 are relatively hard due to the presence of shallow rock or rock outcropping, no test section was considered in this district. Test sites were then selected to perform stabilization. The selection of test sites was finalized in consultation with the AHTD research staffs and district personnel. This study also included series of laboratory tests, which included Atterberg limits, grain size distributions, pH, standard Proctor, and strength properties (California Bearing Ratio, or CBR). The methodologies, guidelines and monitoring protocols presented in this study are expected to be useful for AHTD engineers and professionals in constructing effective soil stabilization for projects in Arkansas.

1.4 OBJECTIVES AND STUDY TASKS

The main objective of this research is to establish useful guidelines for subgrade and embankment stabilization and create standard rehabilitation techniques for each geological region in the state. The following tasks were identified in accomplishing the aforementioned objective:

1. Review and Evaluation of Current AHTD Procedures: This task includes review of current AHTD stabilization methods in all 10 AHTD districts for subgrade stabilization. A summary and evaluation of the results has been performed and taken into consideration upon making final recommendations.

2. Interview AHTD District and Construction Personnel: This task includes meeting with construction managers and inspectors in all 10 AHTD districts and discussing their past and current techniques adopted for soil and subgrade stabilization and the level of success achieved by different methods. These interviews are summarized and included in this report.

3. Evaluation of the State Geological Conditions: This task includes review of the subsurface conditions in all 10 AHTD districts using borings and subsurface explorations from recent studies conducted by AHTD, as well as available studies by other agencies such as the United States Department of Agriculture (USDA) and the US Army Corps of Engineers (USACE). Surficial materials have been classified in accordance with the American Association of State Highway and Transportation Officials (AASHTO) method

of soil classification. The information is grouped and then used to divide the state into regions in consultation with AHTD personnel.

4. Summary of Experience of Neighboring States: This task includes collecting information from the neighboring states that have similar surficial soil or rock conditions to one or more of the regions discussed above. The gathered information has been as part of this task.

5. Preliminary Stabilization Methods: Different soil types at each region are evaluated. A list of potential stabilizing methods has been prepared based on soil properties such as grain size and plasticity characteristics.

6. Site Selection: The initial plan is to select one site selected at each region, with the exception of District 4, in consultation with the AHTD personnel. However, only three test sites were available to the research team. The AHTD has provided soil investigation reports. The selected sites have been visited by the research team. The conditions of the sites have been documented. Surficial soil samples have been collected at specific locations (within the upper few feet), with help from the project contractor (if available) or using AHTD’s local equipment. These samples have been transported to Arkansas State University’s (ASU) soil laboratory. Specific tests (e.g., gradation, moisture content, plasticity, and moisture-density relationship and CBR) have been performed as previously stated.

7. Test Site Stabilization: Based on the soil information and laboratory test results included in the soil report, the Primary Investigator (PI) has made recommendations regarding soil stabilization. The recommendations included the type of additive (stabilization agent), depth of treatment and dosage. When an advanced notice was given, the research team attended scheduled fieldwork.

8. Site Monitoring: After the completion of the fieldwork at each test site, the research staff visited the test sites and made visual observations. AHTD has collected traffic, surface depressions, rutting, and cracking data, which has been provided to the research team.

9. Data Analysis: The collected field data, observations and laboratory test results will be analyzed in attempt to establish basis for subsequent recommendations and approaches regarding soil stabilization.

10. Recommendations and Guidelines for Soil Stabilization: Recommended procedures and guidelines for soil stabilization have been presented in this report. Also, recommendations for monitoring each test site beyond the initial 6-month period have been provided to the AHTD.

1.5 ORGANIZATION OF THE REPORT

This report is organized into eleven chapters and one appendix. Following the introduction, background and objectives in Chapter 1, Chapter 2 provides a review and evaluation of current AHTD procedures. Chapter 3 presents the survey results of AHTD district and construction personnel. The evaluation of the state geological (geotechnical) conditions is presented in Chapter 4. Chapter 5 presents a summary of experience of neighboring states. Preliminary stabilization methods pertinent to this study are presented in Chapter 6. Chapter 7 discusses processes involved in site selection and conditions prevailing in the selected sites. Activities and techniques involved in stabilizing selected test sites are reported in Chapter 8. Chapter 9 presents documented short-term (six months) performance related (e.g., depression, rutting, cracking, visible distress or any surface irregularities) of data of monitored test sites. Chapter 10 provides analyses of the collected field and laboratory data. Conclusions and recommendations for soil stabilization procedures and long-term monitoring of the test sites are presented in Chapter 11. Appendix A presents selected photographic pictures of post-stabilization observations.

2 REVIEW AND EVALUATION OF CURRENT AHTD PROCEDURES

A limited number of studies were conducted to evaluate the effectiveness on different stabilizing agents for Arkansas soils. At a very early stage of the AHTD research activities, Grubbs (1965) reviewed the exiting stabilizing techniques and studied possible industry by-products as stabilizing agents. The effectiveness of a stabilizing agent has been evaluated on the basis of comparative laboratory tests performed using treated and untreated soils. Test methods would determine the volumetric stability, compressive strength, freeze-thaw characteristics, plasticity changes, and absorption characteristics. It was reported that the aluminum by-product used as a soil additive in concentrations up to 10 percent by weight was ineffective as a stabilizing agent. Aluminum by-product along with cement generally showed a slight tendency to increase strengths. Lime was reported to be an effective stabilizing agent for treatment of clay of medium to high plasticity.

In a related study, Parker and Thornton (1975) studied the use of fly ash in fill and base materials in Arkansas Highways. In this study, two Arkansas soils, an organic clay (OH by the Unified Soil Classification System, or USCS) and a sand (SP-SM by USCS) were stabilized with the fly ash. It was reported that an addition of 20% fly ash to the clay reduced the fraction of less than 2 micron particles from 58% to 8%. Twenty percent (20%) fly ash increased the modified compaction unit weight by 6 pcf, and 10 pcf in the clay and sand, respectively. The unconfined compressive strengths with 20% fly ash increased by 220 psi, and 720 psi in clay and sand, respectively, when compacted immediately and cured 7 days. Researchers also reported that the unconfined compressive strengths improved with lime and cement admixtures. In a follow-up study (Thornton and Parker, 1976), these researchers reported that the permeability of soil-fly ash mixtures decreased with an increase of the percent of fly ash. In another follow up study, these researchers (Thornton and Parker, 1980) reported fly ash produced in Arkansas from burning Wyoming low sulfur coal was found to be self-hardening and could be effective as a soil stabilizing agent for clays and sands. The strength gain was reported to be rapid when fly ash was used as a stabilizing agent, with 7-day unconfined compressive strengths up to 1800 psi from 20% fly ash and 80% sand mixtures.

Annable (1986) studied the use of fly ash in road construction. In particular, this study (Annable, 1996) focused on characterizing pressure grouting using fly ash. This study blended materials together to obtain a flowable mixture which could be pumped through a 2-inch drill hole in concrete pavement to voids below the pavement. To this end, fly ash from five sources and cement from two sources, all in Arkansas, were investigated. It was reported that an admixture of 3% fly ash and 1% Portland cement or 3.5% fly ash and 1% Portland cement provided higher 7-day compressive strengths compared to other proportions.

Even though a few subgrade stabilizing techniques (lime, cement and CFA) are being utilized by engineers of different AHTD districts, limited guidelines for lime treatment of subgrade soils are detailed in **Section 301** of the **2014 AHTD Standard Specifications for Highway Construction** (AHTD, 2014). It is stated that the lime treated soil mixture must be composed of the soil in the existing subgrade, lime, and water. The mixture shall contain no more than 8% by weight of lime. Adequate quantities of soil and lime shall be supplied to the

ATHD Materials and Tests Division for determination of lime requirements at least 30 days before the beginning of lime treatment process. The AHTD engineer will specify the exact percentage of lime to be used based on laboratory tests. Either quicklime or hydrated lime can be used as shown in the contract. The lime shall comply with AASHTO M 216. The current specifications for lime treated subgrade soils are presented in Table 2.1.

Table 2.1 Guidelines for Lime Treated Subgrade Soils in Arkansas [AHTD, 2014]

Operation	Guidelines
Preparation of Subgrade	• Before beginning the treatment, the subgrade shall be shaped to the required grade and section and compacted to sufficient density to prevent permanent deformation under normal operation of construction equipment. Also, soft areas shall be corrected to provide uniform stability before the application of lime.
Preparation of Soil	• The proposed roadbed shall be scarified to the depth and width indicated on the plans for the subgrade treatment. The scarified material shall be partially pulverized, and the depth of scarification shall be carefully controlled and operations conducted in a manner to provide that the subgrade material below the depth of the proposed treatment shall remain undisturbed.
Application of Lime	• The rate of application of lime shall be as determined by laboratory design or as directed by the AHTD engineer. • Hydrated lime may be applied to the partially pulverized material either in a slurry form or in the dry condition. • Spreading equipment, including truck spreaders, shall be of a type and design capable of uniformly distributing the lime without excessive loss. • No equipment, except water trucks and that equipment used for spreading and mixing shall be permitted to pass over the spread lime until it is mixed with subgrade material. • Any procedure that results in excessive loss or displacement of the lime shall be immediately discontinued.
Addition of Water	• Water shall be applied to the spread lime immediately after placing to moisten the lime and form a dust palliative. • Water shall be added during mixing operations to moisten the mixture but the total water added to the mixture including that added to form a slurry shall not exceed the optimum by more than 5%.
Mixing	• Water shall be added to keep the moisture within ±2% of the optimum. • Mixing may be accomplished by means of rotary tillers, pulvimixers, or other mechanical equipment. • The first stage of the mixing process shall continue until the lime and moisture are thoroughly and uniformly dispersed throughout the mixture. Afterwards, the surface shall be rolled with pneumatic rollers until sealed sufficiently to shed rain. • After the first mixing stage, the mixture shall be allowed to set for a minimum of 3 days or until the mixture becomes friable. During this period, the surface shall be sprinkled as necessary to keep it moist.

	• Afterwards, the mixture shall be scarified and thoroughly and uniformly mixed with rotary tillers or pulvimixers until the soil is thoroughly pulverized and mixed with the lime.
Compaction	• After the materials have been satisfactorily mixed and pulverized, the full depth of the mixture shall be compacted to a uniform density of not less than 95% of the maximum laboratory density. • Percent coarse particles retained on the #4 (4.75 mm) sieve shall be determined according to AASHTO T 27. If P4 (% Retained #4 Sieve) is 10 or less then follow AASHTO T 99- Method A, if it is from 11 to 30 then AASHTO T 99-Method C, and if it is at least 31 then AASHTO T 180-Method D • The in-place density shall be determined by using AASHTO T 310, Direct Transmission. The moisture content shall be determined by AASHTO T 310 or AHTD Test Method 347 or 348.
Finishing	• During the final stages of the compaction, the surface of the subgrade shall be shaped to the lines, grades, and cross sections shown on the plans. • When required, the surface may be lightly scarified and bladed. Final rolling of the completed surface shall be accomplished with a pneumatic roller.

In accordance with the 2014 AHTD Standard Specifications for Highway Construction, portions of the subgrade composed of unsuitable materials shall be removed; backfilled with approved material, and the entire subgrade brought to line and grade and compacted to the required unit weight (AHTD, 2014). Section 212 of the AHTD Specifications also states that "When the subgrade is to be stabilized with lime or Portland cement, the top 8" (200 mm) shall be compacted before treatment to the extent necessary to prevent rutting under normal operation of construction equipment."

AHTD accepts lime based on the producer's/supplier's certification that the material meets all requirements of applicable Department specifications (AASHTO M 216) under the following conditions:

- The producers/suppliers shall maintain a file of their test results and methods of testing.
- Lime shall be sampled and tested by the producer as needed to maintain specification compliance (monthly composite samples will be tested as a minimum). Results of these tests covering the lime certified and shipped to AHTD projects shall be furnished to the Department's Materials Division upon request.
- Shipments to AHTD projects shall be made only from bins/silos that are certified to meet Department specifications. The producer/supplier shall furnish with each shipment a certification containing the following: consignee, date and place of production, date and time of shipment, truck or railroad car number, quantity of lime shipped, type of lime (hydrated/quicklime), bin/silo number from which shipped, a statement that "This is to certify that the lime in this shipment is from bin/silo number _ and complies to AASHTO M 216," and the signature of a responsible company official.
- AHTD will make periodic inspections of the producer's product by source sampling and by checking test results and methods of testing used by the producer.

- Destination samples will also be taken as deemed necessary to assure compliance with specifications. Failure of these samples may be considered sufficient cause to reject the lime and suspend further shipments until tests by the AHTD determine that the producer's product is in compliance with applicable specifications and requirements.

As a common practice, the AHTD requires the contractor to perform Quality Control (QC) sampling and testing and acceptance sampling and testing of construction items (AHTD, 2014). The AHTD performs verification testing to verify the contractor’s testing equipment and procedures, or both verification and acceptance testing to verify the contractor’s testing equipment and procedures and for use in the acceptance of material and to determine payment for the material. When hydrated lime is used to treat subgrade soils, one sample should be submitted to the central laboratory for each 250 tons delivered. The QC tests, which have specified frequencies, will also be verified by the AHTD engineers. In the case of lime-treated subgrade thickness, the verification rate is one per 48,000 square yards. During the verification tests, the maximum unit weight, optimum moisture content, the percent compaction, and the moisture content for lime treated subgrade should be within ±5%, ±15%, ±3%, and ±4%, respectively. The AHTD guidelines of acceptance sampling and testing of construction materials are provided below (AHTD, 2014):

- At least 30 days prior to beginning of lime treatment, submit 50 lbs. of each different soil and 10 lbs. of lime to be used on project.
- Contractor maximum laboratory density determination: one for each soil type with a minimum of one per job.
- Contractor acceptance testing of density, moisture content & thickness: one for each 12,000 yd^2.
- Resident Engineer’s verification testing of density, moisture content & thickness: one for each 48,000 $yd^{2.}$

3 SUMMARY OF DISTRICT EXPERIENCE WITH SOIL STABILIZATION

This chapter presents a summary of experiences with soil improvement projects in nine AHTD districts. Also, this chapter presents the upcoming soil improvement project at the time of interviews in Fiscal Year (FY) 12/13. The content of this chapter is based on interview results of AHTD district engineers and personnel.

3.1 District 1

As summarized in Table 3.1, five improvement projects were completed in District 1. Two projects utilized cement stabilization technique, and one (the Lee County project in S. Mariana) was reported as successful. The soil in this project site is predominately silt with a plasticity index of 6. It consists of 97% fines. This project used 14.4% cement (by volume of soil) as stabilizing agent. The other cement stabilized project was in Crittenden County which used Type 8 geotextile fabric.

District 1 engineers intended to construct three projects in the district in FY 13/14; two were supposed to be constructed in Lee County. The 11052 project (3-lane widening of Hwy.1) appeared a candidate for the current study.

Table 3.1 District Experience Summary and Upcoming Candidate Project – District 1

Dist. No.	County and Project No.	Type of Surficial Material	Past Experience with Soil Improvement		Notes	Upcoming 13/14 Projects
			Type	Level of Success		
1	Lee (S. Marianna) 110228	Silt, PI=6, 97% fines	Cement stabilization 14.4% by volume	Successful	None	110543-Bridge replacement over I-40 St. Francis Co.
	Lee (N. Marianna) 110489	3 ft. select fill A-3(0) PI=0 % fines = 3 was placed	None required	Stable subgrade	CTB was used per plans	110502-3-lane widening of Hwy 1, Lee Co., 1500 tons of lime/cement stabilization planned-8 inch CTB
	Lee (N. Marianna) 110229	Select fill was hauled to site	Blend select fill with existing soil, results A-4(0) PI=0, % fines = 44	Stable subgrade	CTB was used per plans	
	Woodruff & Cross 110423	A-6(2), PI=11, 49% fines	None	Stable subgrade	CTB was used per plans	4 Bridge replacement on Hwy 70, Monroe Co.
	Crittenden 110506	A-4 to A-7, PI=6-43	Cement stabilization + Type 8 geotextile fabric	Stable subgrade	Geotextile fabric was requested by district	

3.2 District 2

In District 2, nine soil improvement projects were constructed in recent years (see Table 3.2). Eight projects have been reported to be successful. Engineers were doubtful about one project that used a combination approach of soil replacement and lime stabilization technique on soft clay with a liquid limit of 29. Predominately soft and unstable soils in these project sites warranted soil improvement techniques. In the successful projects, 3 to 5% lime has been used to stabilize up to 24 inches of soft and unstable subgrade soils.

District engineers planned to construct two projects that could require stabilization in FY 13/14. One of them was to replace Highway 165 Bridge in Chicot County. The other upcoming project was to widen Highway 65b in Jefferson County. Since three stabilizing projects were reported to be successful in Jefferson County, the Highway 65b widening project would be a good candidate for the current study.

Table 3.2 District Experience Summary and Upcoming Candidate Project – District 2

Dist. No.	County and Project No.	Type of Surficial Material	Past Experience with Soil Improvement		Notes	Upcoming 13/14 Projects
			Type	Level of Success		
2	Lincoln 020448	Soft wet soil, no classification	4%, 18-inch lime stabilization	Successful		Hwy 165 Bridge replacement, Chicot Co.
	Arkansas 020326	Soft/fat clay, LL=29 PI=6	Combined replacement & lime stabilization	Doubtful	Some areas did not respond to lime	
	Jefferson 020487	Soft & yielding subgrade	Combined replacement & lime stabilization	Successful		Hwy 65B widening, Jefferson Co.
	Desha R20092	unknown	Lime treatment	Successful		
	Grant 020464	Soft &yielding	Combined replacement & lime treatment	Successful		
	Jefferson 020354	Yielding soil	Combined replacement & lime treatment	Successful		
	Lincoln/ Jefferson 001934	Unstable clay	3-5% lime for 24 inches	Successful		
	Lincoln/ Jefferson 012060	Unstable clay	3-5% lime for 24 inches	Successful		
	Hwy 138 Bridge Replacement 020465	Unstable clay	3-5% lime for 24 inches	Successful		

3.3 District 3

District 3 engineers have completed the highest number of soil improvement projects (see Table 3.3). A majority (13 out of 14) of the improvement projects were reported as successful. Subgrade soils in these project sites have been reported to be wet clays with high plasticity indices. Often these soils are unstable and exhibit poor drainage conditions. Of the 13 successful soil improvement projects, nine utilized lime (dosages ranging from 4 to 7% with a depth of 16 inch), three used cement, one with CFA. The Hwy 32 Phase I project in Little River County treated highly plastic, wet organic clay with 6% lime and was reported to be unsuccessful. The 5-ft. bridge lift of this project showed early success but had experienced long-term settlement.

District 3 engineers planned to complete two projects that may require stabilization in FY 13/14. Both of them are located in Miller County. The Broad St. bridge replacement project seems to be a candidate for the current study.

Table 3.3 District Experience Summary and Upcoming Candidate Project – District 3

<table>
<tr><th rowspan="2">Dist. No.</th><th rowspan="2">County and Project No.</th><th rowspan="2">Type of Surficial Material</th><th colspan="2">Past Experience with Soil Improvement</th><th rowspan="2">Notes</th><th rowspan="2">Upcoming 13/14 Projects</th></tr>
<tr><th>Type</th><th>Level of Success</th></tr>
<tr><td rowspan="8">3</td><td>Miller, HWY 245 widening 030341</td><td>High PI wet clay</td><td>Undercut& replacement with low PI soil or cement-treated sand</td><td>Successful</td><td></td><td>Broad St., 030388, Texarkana, Miller Co. Bridge replacement</td></tr>
<tr><td rowspan="3">Little River, HWY 32 Phase I 030205</td><td rowspan="3">High PI wet clay with organics</td><td>6% Lime treatment</td><td>Unsuccessful</td><td></td><td rowspan="4">La. Line – Doddridge, Miller Co., 030313 New location Future I-49</td></tr>
<tr><td>Undercut 3 ft. & replace with stone over fabric</td><td>Successful</td><td></td></tr>
<tr><td>5 ft. bridge lift</td><td>Unsuccessful</td><td>Early success but long-term settlement</td></tr>
<tr><td>Little River, HWY 32 Phase III 030268</td><td>High PI wet clay</td><td>6% Lime treatment</td><td>Successful</td><td></td></tr>
<tr><td>Pike, Co. Road 6 FA5511</td><td>High PI wet clay</td><td>6% Lime treatment</td><td>Successful</td><td></td><td></td></tr>
<tr><td>Hempstead/Lafayette / HWY 29 030348</td><td>A-2-4 wet, unstable, poor drainage</td><td>4 in. pipe drain with lime stabilization</td><td>Successful</td><td></td><td></td></tr>
<tr><td>Lafayette, HYW 160 030182</td><td>A-4(0) with some A-</td><td>4 in. pipe drain with cement stabilization</td><td>Successful</td><td></td><td></td></tr>
</table>

		7-6(27) wet & poor drainage				
	Hempstead, HWY 278&278B, 030078	Gumbo clay	Lime stabilization Undercut / Geogrid 4 in. underdrain / B Stone	Overall Successful	Fat clay did not respond to lime. Geogrid was added	
	Nevada, HWY 24, R30026	Unstable poor quality backfill	Undercut/lime stabilization	Successful		
	Nevada, HWY 371, 030322	Unstable subgrade due to poor drainage	Undercut/lime stabilization	Successful		
	Miller, AR HWY 245, 030378	Unstable, saturated high PI	16 in. of lime treatment, 5-7%	Successful	Additional 16 in. treatment in some areas	
	Miller, AR HWY 245/549, 030314	Saturated , poor stability sand	Cut / 6-8% cement stabilization	Successful		
	Miller, HWY 82 Texarkana 030349	A-24(0), A-6(5), A-6(9) saturated & unstable	16 in. of lime treatment, 4-5.5%	Successful		
	Miller, HWY 82 Texarkana 030321	Saturated & unstable subgrade	Lime treatment And fly ash at some areas	Successful	Fly ash by contractor at no additional cost	
	Miller, HWY 82 Texarkana 030261&R30095	Unstable subgrade due to moisture content	Lime treatment of varying %'s	Successful		
	Hempstead & Nevada 030322/030078/ 030056/030290	Unstable wet soil	Lime stabilization, fabric	Partially successful		070289, HWY 273 Widening, Cleveland & Dallas Co,
			Undercut and stone backfill	Successful		

3.4 District 4

Surficial soils in District 4 are predominantly rocky, which did not necessitate any soil improvement in the past. Therefore, District 4 engineers did not anticipate any soil improvement or stabilization needs. Thus, the current study did not include any soil stabilization project from District 4 for evaluation.

3.5 District 5

In recent years, District 5 engineers successfully performed subgrade soil improvement on three projects (see Table 3.4). Of which, cement stabilization of predominately wet soil in Independence County appears to be a good candidate for further consideration of the current study. The other two successful soil improvement projects in this district have received 3 to 4 ft. undercut and shot rock replacement.

Projects that would require soil stabilization in District 5 in FY 13/14 were not anticipated. Thus, a new construction project would be identified for performance monitoring of a soil stabilization technique in this district.

Table 3.4 District Experience Summary and Upcoming Candidate Project – District 5

Dist. No.	County and Project No.	Type of Surficial Material	Past Experience with Soil Improvement		Notes	Upcoming 13/14 Projects
			Type	Level of Success		
5	Independence, 050023	Wet soil	Cement stabilization	Successful		
	CRS-73-36-2A	Slope sliding Excessively wet clay & shale	3-4 ft. undercut and replacement with shot rock	Successful		
	CRS-73-167-17A	Slope sliding Excessively wet shale	3-4 ft. undercut and replacement with shot rock	Successful		
	CRS-34-367-21A	Sandy soil -Slope failure			Not subgrade-related	

3.6 DISTRICT 6

As shown in Table 3.5, several projects were unsuccessful when process drying techniques were implemented for heavy clayey soils in District 6. However, lime and cement treatments of heavy clay soils were reported to be successful. Furthermore, crushed stone backfilling improvement techniques (2 feet replacement) on unstable subgrade soils in this district were also reported to be successful.

In FY 13/14, engineers in District 6 planned to construct three projects; two of them would be Garland County and the other one was on the I-40/Highway 89 Interchange in Lonoke County.

Table 3.5 District Experience Summary and Upcoming Candidate Project – District 6

Dist. No.	County and Project No.	Type of Surficial Material	Past Experience with Soil Improvement: Type	Past Experience with Soil Improvement: Level of Success	Notes	Upcoming 13/14 Projects
6	HYW 67/167 I-40 to Kiehl Ave.	Heavy clay	Process drying / Lime treatment	Unsuccessful / Successful	18 inch treatment, dosage unknown	060897 I-40/HWY 89 Interchange, Lonoke Co.
	HYW 67/167 HWY 440 to Redmond Ave.	Mixture of SM & heavy clay	Process drying / Cement treatment	Unsuccessful / Successful	Cement does not allow delayed trimming, 18 inch treatment dosage unknown	
	HWY 67/167 HWY 440 to Redmond Ave (Ph. II)	Mixture of SM & heavy clay	Process drying / Lime treatment	Unsuccessful / Successful	Lime does allow delayed trimming, 18 inch treatment dosage unknown	061214 Printers Place-Crawford St. Widening, Hot Springs, Garland Co.
	HWY 67/167 HWY 440 to Redmond Ave. (Ph. II)	Mixture of SM & heavy clay	Process drying / Lime treatment	Unsuccessful / Successful	Lime does allow delayed trimming, 18 inch treatment dosage unknown	
	Garland RE64, 060776 HWY 5 widening	Unknown	Backfilling with stone to mitigate flowing free water	Successful	2 feet placement	0631312 Bridge construction Hot Springs & Stokes St., Garland Co.
	Garland RE64, 060686 HWY 7 – South HWY 270	Unknown (unstable subgrade)	Undercut/backfill with stone	Successful		
	Garland RE64, 061059 Hot Springs-west Passing Lane	Unknown	Backfilling with stone to mitigate flowing free water	Successful	2 feet placement	
	Garland RE64, 061259, HWY 7-Hot Springs Safety Improvement	Unknown (unstable subgrade)	Undercut/ backfill with stone	Successful		

3.7 District 7

In this district, several projects were successfully stabilized using lime in recent years (see Table 3.6). The outcomes of these projects throughout the district did not follow any particular trend: some of them were successful and others were unsuccessful or partially successful. For instance, three lime stabilization projects (4% lime) on wet gray clay soils in Union County have been reported successful, but the only lime stabilization project in Dallas County has been reported to be an unsuccessful project.

District 7 engineers planned to construct six projects in FY 13/14. Since three out of four lime stabilization projects were reported successful, the same stabilizing method could be adopted in the HWY 335 widening project in Eldorado in Union County as needed and could be utilized as a test site.

Table 3.6 District Experience Summary and Upcoming Candidate Project – District 7

<table>
<tr><th rowspan="2">Dist. No.</th><th rowspan="2">County and Project No.</th><th rowspan="2">Type of Surficial Material</th><th colspan="2">Past Experience with Soil Improvement</th><th rowspan="2">Notes</th><th rowspan="2">Upcoming 13/14 Projects</th></tr>
<tr><th>Type</th><th>Level of Success</th></tr>
<tr><td rowspan="9">7</td><td>Union, 070076</td><td>A-4, unstable wet gray clay</td><td>14 in., 4% lime stabilization</td><td>Successful</td><td></td><td rowspan="3">070280, HWY 335 Widening, Eldorado, Union Co.</td></tr>
<tr><td>Union, 070267</td><td>A-4, A-6, unstable wet gray clay</td><td>14 in., 4% lime stabilization</td><td>Successful</td><td></td></tr>
<tr><td>Union, 070297</td><td>A-4, A-6, unstable wet gray clay</td><td>14 in., 4% lime stabilization</td><td>Successful</td><td></td></tr>
<tr><td rowspan="2">Union, 070273</td><td rowspan="2">Cypress swamps, unable to provide stable platform</td><td>Soil processed with 6% cement</td><td>Unsuccessful</td><td></td><td>070281, HWY 335 Widening, Ouachita River, Union Co.</td></tr>
<tr><td>Undercut/ backfill 4 ft.</td><td>Successful</td><td>4 ft. bridge lift with sandy soil</td><td>070240, Ouachita River St. widening, Clark Co.</td></tr>
<tr><td rowspan="3">Dallas, 070288</td><td rowspan="3">Unstable wet soil</td><td>Lime stabilization</td><td>Unsuccessful</td><td rowspan="3">Bridge lift settled*, select materials were stabilized with cement</td><td rowspan="2">070344, Gurdon – Oak Grove St. Bridge, Clark Co.</td></tr>
<tr><td>Undercut, bridge lift</td><td>Unsuccessful</td></tr>
<tr><td>Select material with fabric</td><td>Unsuccessful</td><td rowspan="2">070291, Saline River – South Widening, Cleveland & Dallas Co,</td></tr>
<tr><td>Grant, 020424</td><td>Unstable wet soil</td><td>Undercut & backfill with stone</td><td>Successful</td><td></td></tr>
</table>

***:** Primary due to bad construction procedures and lack of inspection

3.8 District 8

In recent years, lime stabilization (from 3 to 5% lime) on soft clay with medium plasticity soil in Faulkner County was reported to be successful (see Table 3.7). A cement treatment (6% cement) in Pope County was reported as successful even though the other cement stabilization (4% cement) project in the same county was reported as unsuccessful. District engineers believed lime stabilization would have been a better choice instead of cement stabilization for the unsuccessful project in Pope County. There was no planned construction project requiring stabilization in FY 13/14.

Table 3.7 District Experience Summary and Upcoming Candidate Project – District 8

Dist. No.	County and Project No.	Type of Surficial Material	Past Experience with Soil Improvement		Notes	Upcoming 13/14 Projects
			Type	Level of Success		
	Vilonia Bypass, Faulkner County	Medium Plasticity; soft yielding clay (A-6 & A-7)	3-5% lime treatment	Successful		
8	RE 86 Russellville Bypass Widening, 080236; Pope County	A-4, PI<10	18 in, 6% cement treatment	Successful		None
	Weir Road Widening, 080284; Pope County	Unstable subgrade after processing	18 in., 4% cement treatment	Unsuccessful	Engineer thought lime should have been used (low PI)	
			Undercut & backfill	Successful		

3.9 District 9

In recent years, soil improvement techniques were adopted in three construction projects, which were reported as successful (see Table 3.8). Of these, the project in Benton County used a 4% cement to stabilize unstable soils. Notably, none of the highway projects in Baxter and Searcy counties had included any soil stabilization techniques in the last five years.

Table 3.8 District Experience Summary and Upcoming Candidate Project – District 9

Dist. No.	County and Project No.	Type of Surficial Material	Past Experience with Soil Improvement		Notes	Upcoming 13/14 Projects
			Type	Level of Success		
9	Benton, 090099	Unstable soil	4% cement stabilization	Successful		090065, Avoca-North Garfield, 6.29 Mile widening, Benton Co.
	Madison, HWY 412 Widening	Unstable clay	N/A stabilization was included in the Spec's	Successful		
	Various projects in Baxter, Marion & Searcy Co.	Low to high plasticity clay, clay gravel & sandy clay	Fill with granular soil	Successful	No soil stabilization has been used over the last 5 years!!	090282, Illinois River St. Bridge replacement, Benton Co.
						FA0313 HWY 341 Improvement, Baxter Co.
						090319, HWY 62-HWY 5 Widening (Mountain Home), Baxter Co.

3.10 District 10

Historically, CFA and cement stabilization were reported to be successful in District 10 (see Table 3.9). When compared with soil replacement, CFA stabilization showed better performance than the former in a widening project of Hwy 412 in Green County. Another project in Poinsett County used geosynthetics as reinforcement to improve soils of unstable slope in seismic conditions, but the level of success of the geosynthetics are still unknown. However, geotextiles used in a project in Mississippi County to improve sandy and silty clay have been reported to be successful.

Table 3.9 District Experience Summary and Upcoming Candidate Project – District 10

<table>
<tr><th rowspan="2">District No.</th><th rowspan="2">County and Project No.</th><th rowspan="2">Type of Surficial Material</th><th colspan="2">Past Experience with Soil Improvement</th><th rowspan="2">Notes</th><th rowspan="2">Upcoming 13/14 Projects</th></tr>
<tr><th>Type</th><th>Level of Success</th></tr>
<tr><td rowspan="5">10</td><td rowspan="2">Greene, widening of HWY 412, 100566</td><td rowspan="2">Unstable soil</td><td>Undercut and replace</td><td>Successful</td><td rowspan="2">Fly ash gave better results than undercut</td><td rowspan="4">10047, Widening of HWY 18 from HWY 61 to Holland St., Blytheville, Mississippi Co.</td></tr>
<tr><td>Stabilization with fly ash</td><td>Successful</td></tr>
<tr><td>Mississippi, Interstate rehabilitation, 100716</td><td>Unstable soil</td><td>12-15 in., 6% cement treatment</td><td>Successful</td><td></td></tr>
<tr><td>Poinsett, Highway 63, 100523 & 100547</td><td>Unstable slope in seismic conditions</td><td>Geosynthetic reinforcement</td><td>Unknown</td><td></td></tr>
<tr><td>Mississippi, HWY 18 widening, 100304 & 100307</td><td>Sand/silty clay</td><td>Geotextile fabric placed on top of in situ soil</td><td>Successful</td><td></td><td></td></tr>
</table>

4 EVALUATION OF THE STATE GEOLOGICAL CONDITIONS

This chapter includes review of the subsurface conditions in specific counties across the 10 AHTD districts using borings and subsurface explorations from recent studies conducted by AHTD, as well as data available through studies conducted by the USDA [7]. Surficial materials up to a depth of 18 inches have been classified in accordance with the AASHTO method of soil classification. Furthermore, texture, USCS, Liquid Limit (LL) and Plasticity Index (PI) of these soils have been summarized in this chapter for specific counties. Please note the soils below 18 inches are expected to vary, in both type and index properties. Please note there were no available information at specific counties in some districts, hence they are not included in the following tables.

4.1 DISTRICT 1

Soil types, properties and texture data in different counties vary significantly throughout the district. As shown in Table 4.1, soils in District 1 are predominately A-2, A-3, A-4, A-5, A-6, A-7, and A-7-6, in accordance with the AASHTO Classification system. The liquid limit (LL) value is lower than 94. The plasticity index (PI) value varies from 0 (non-plastic, NP) to 64. Based on the available information in District 1, Lee County seems to have the highest LL and PI values. The variations of LL and PI values of soils in Cross County are as not as high as the other countries in this district, and they range from 26 to 44 and from 6 to 25, respectively.

Table 4.1 Soil Information to a Depth of 18 inches in District 1

District	County	Unified Soil Classification	AASHTO Classification	LL Range	PI Range
1	Crittenden	CH, CL, SM, SP-SM, ML	A-2, A-3, A-4, A-6, A-7	NA	NA
	Cross	CH, CL, ML, ML-CL, SM	A-2, A-4, A-6, A-7	26- 44	6- 25
	Lee	CH, CL, SM, ML, CL-ML	A-2, A-4, A-6, A-7	<94	NP-64
	Monroe	CH, CL, CL-ML, ML, SM, SC	A-2, A-4, A-5, A-6, A-7	<85	NP-55
	Phillips	CH, CL, ML, SM, SP-SM	Varies types A-4	<70	NP-60
	Woodruff	CH, CL, CL-ML, ML, SM, SC, SC-SM,	A-2, A-4, A-6, A-7, A-7-6	<76	NP-49
	Saint Francis		No AASHTO Classification Given		

NA: Not Available

4.2 DISTRICT 2

In accordance with the AASHTO Classification System, soils in District 2 are predominately classified as A-1, A-2, A-3, A-4, A-6, and A-7 (see Table 4.2). The liquid LL value ranges from 7 to 101. The PI value varies from NP to 69. Based on the available information, Arkansas County in District 2 seems to have the highest LL and PI values. The LL and PI values of a few counties (Bradley, Ashley, Chicot, and Desha) are not available.

Table 4.2 Soil Information to a Depth of 18 inch in District 2

District	County	Unified Soil Classification	AASHTO Classification	LL Range	PI Range
2	Arkansas	CH, CL, CL-ML, SM, SC-SM, SW-SM,	A-4,A-6,A-7	7-101	NP-69
	Jefferson	CH, CL, ML, CL-ML, SC, SM, SM-SC, SP-SM	A-2, A-3, A-4, A-6, A-7	<90	NP-65
	Grant	CH, CL, CL-ML, GC, GC-GM, GM, GP-GC, ML, SC, SC-SM, SM,	A-1, A-2, A-4, A-6, A-7	15-80	NP-55
	Ashley	CH, CL, CL-ML, MH, OH, ML, SM, SM-SC, SC, SP-SM	A-2, A-2-4, A-3, A-4, A-6, A-7	NA	NA
	Chicot		A-4	NA	NA
	Lincoln	CH, CL, ML, CL-ML, SC, SM, SM-SC, SP-SM	A-2, A-3, A-4, A-6, A-7	<90	NP-65
	Bradley	CL, ML, ML-CL, SM, SM-SC, SC	A-2, A-4, A-6	NA	NA
	Drew	CL, CH, CL-ML, GC, GM, MH, ML, SC, SM,	A-1, A-2, A-4, A-6, A-7	<90	NP-55
	Desha		A-6 (9), A-7 (5)	NA	NA

4.3 DISTRICT 3

Soils in District 3 are predominately A-1, A-2, A-3, A-4, A-6, A-7 and A-7-6 (see Table 4.3). The liquid LL value is no more than 97, and the PI value ranges from NP to 65. Based on the available information in District 3, Sevier County seems to have the highest LL with PI values range from NP to 61.

Table 4.3 Soil Information to a Depth of 18 inch in District 3

District	*County*	*Unified Soil Classification*	*AASHTO Classification*	*LL Range*	*PI Range*
3	Nevada	CH, CL, CL-ML, GC, GC-GM, GM, ML, SC, SC-SM, SM	A-1, A-2, A-2-4, A-3, A-4, A-6, A-7, A-7-6	<80	NP-55
	Howard		A-4, A-7		
	Sevier	CH, CL, CL-ML, GC-GM, GM, GP-GC, GP-GM, ML, SC, SC-SM, SM	A-1, A-2, A-4, A-6, A-7, A-7-5, A-7-6	<97	NP-61
	Little River	CH, CL, CL-ML, GC, GM-GC, GP-GC, ML, SM, SM-SC, SC, SP-SM	A-1, A-2, A-4, A-6, A-7	<90	NP-60
	Hempstead	CH, CL, CN, CL-ML, GC, GM-GC, GP-GM, ML, MH, SM, SM-SC, SC, SW-SM	A-1, A-2, A-2-4, A-3, A-4, A-6, A-7, A-7-6	<90	NP-65
	Miller	CH, CL, CL-ML, GC, GM-GC, GP-GC, ML, SM, SM-SC, SC, SP-SM	A-1, A-2, A-4, A-6, A-7	<90	NP-60
	Lafayette	CH, CL, CL-ML, GC, GM-GC, GP-GC, ML, SM, SM-SC, SC, SP-SM	A-1, A-2, A-4, A-6, A-7	<90	NP-60
	Pike	CH, CL, CL-ML, ML, GC, GC-GM, GM, GP-GC, GP-GM, GW-GM, SC, SC-SM, SM,	A-1, A-1-a, A-1-b, A-2, A-2-4, A-2-6, A-4, A-6, A-7, A-7-6,	<83	NP-55

4.4 DISTRICT 4

As shown in Table 4.4, soils in District 4 are classified as A-1, A-2, A-3, A-4, A-6, and A-7. Based on the available information in this district, soils in Sebastian County appear to have the highest LL (up to 80) and PI (up to 60) values. On the other hand, soils in Washington County appear to have the lowest LL (up to 32) and PI (from NP to 12) values.

Table 4.4 Soil Information to a Depth of 18 inch in District 4

District	County	Unified Soil Classification	AASHTO Classification	LL Range	PI Range
4	Polk	CH, CL, CL-ML, GC, GP-GC, MH, ML, SC, SC-SM, SM, SP-SM,	A-1, A-2, A-4, A-6, A-7	<66	NP-39
	Scott	CH, CL, CL-ML, GC, GM, GM-GC, GP-GM, GP-GC, ML, SC, SC-SM, SM,	A-1, A-2, A-4, A-6, A-7	<65	NP-40
	Logan	Ch, CL, CL-ML, GC, GM, GM-GC, MH, ML, SM, SM-SC, SC,	A-1, A-2, A-3, A-4, A-6, A-7	<80	NP-50
	Sebastian	CH, CL, CL-ML, GC, GM, GM-GP, ML, MH, SP-SM, SM	A-1, A-2, A-4, A-6, A-7	<80	NP-60
	Franklin	NA	NA	NA	NA
	Washington	CH, CL, ML-CL, GC, GM, SM,	A-2, A-4, A-4(6), A-4(7), A-4(8), A-6, A-6(8), A-6(9), A-7	< 32	NP-12
	Crawford	CH, CL, CL-ML, GM, GM-GC, GC, MH, ML, SM, SM-SC, SC, SP-SM	A-1, A-2, A-3, A-4, A-6, A-7,	<80	NP-50

4.5 DISTRICT 5

As shown in Table 4.5, soils in District 5 are predominately classified as A-1, A-2, A-3, A-4, A-6, and A-7. Based on the available information, the LL values of soil in District is no more than 80, and the PI values range from NP to 60.

Table 4.5 Soil Information to a Depth of 18 inch in District 5

District	County	Unified Soil Classification	AASHTO Classification	LL Range	PI Range
5	Fulton	CH, CL, ML-CL, GC, GM-GC, GP-GM, GM, MH, ML, SC, SM, SM-SC, SP-SM,	A-1, A-2, A-3, A-4, A-6, A-7	<80	NP-47
	Izard	CH, CL, ML-CL, GC, GM-GC, GP-GM, GM, MH, ML, SC, SM, SM-SC, SP-SM,	A-1, A-2, A-3, A-4, A-6, A-7	<80	NP-47
	Sharp	CH, CL, CL-ML, GC, MH, ML, SM, SP-SM, SM-SC, SC, SP-SC	A-1, A-2, A-3, A-4, A-6, A-7	<80	NP-55
	Stone	CH, CL, CL-ML, GC, GM, GM-GC, SM, SM-SC, SC, SP-SM,	A-1, A-2, A-4, A-6, A-7	<80	NP-55
	Independence	CH, CL, SM, MH, ML, CL-ML, SM, SP-SM, SM-SC, GC, GM, GM-GC, GP-GC	A-1, A-2, A-4, A-6, A-7	<85	NP-55
	Jackson	CH, CL, GC, GM, ML, SC, SM	A-2, A-4, A-6, A-7	<75	NP-60
	White	CL, CL-ML, GC, GM, GM-GC, GP-GM, ML, SC, SM, SP-SM	A-1, A-2, A-4, A-6, A-7	<85	NP-55
	Cleburne	CH, CL, CL-ML, GC, GM, GM-GC, GP-GM, SC, SM, SM-SC,	A-1, A-2, A-4, A-6, A-7	<80	NP-40

4.6 DISTRICT 6

Based on the available information soils in District 6 have very high LL; up to 100 in Hot Springs County. The PI value of this district ranges from NP to 60 (see Table 4.6). The soils in this district are classified as A-1, A-2, A-3, A-4, A-6, and A-7.

Table 4.6 Soil Information to a Depth of 18 inch in District 6

District	County	Unified Soil Classification	AASHTO Classification	LL Range	PI Range
6	Garland	CH, CL, CL-ML, GC, GM, GM-GC, GP-GM, ML, SC, SM, SM-SC,	A-1, A-2, A-4, A-6, A-7	<70	NP-35
	Hot Springs	CH, CL, CL-ML, GC, GM, GM-GC, GP-GC, ML, SC, SM, SM-SC,	A-1, A-2, A-4, A-6, A-7	<100	NP-60
	Lonoke	CH, CL, CL-ML, GC, GM, GM-GC, ML, OH, SC, SM, SM-SC	A-1, A-2, A-4, A-6, A-7	<90	NP-60
	Prairie	CH, CL, CL-ML, GC, GM, GM-GC, ML, OH, SC, SM, SM-SC	A-1, A-2, A-4, A-6, A-7	<90	NP-60
	Saline	CH, CL, CL-ML, MH, GM, GC, ML, GM-GC, GP-GC, SM, SM-SC, SC	A-1, A-2, A-3, A-4, A-6, A-7	<67	NP-43

4.7 DISTRICT 7

Similar to District 6, soils in District 7 have very high LL values (up to 100 in Clark County) and the PI value ranges from NP to 60 (see Table 4.7). Surficial soils in this district are classified as A-1, A-2, A-2-4, A-3, A-4, A-6, A-7, and A-7-6.

Table 4.7 Soil Information to a Depth of 18 inch in District 7

District	County	Unified Soil Classification	AASHTO Classification	LL Range	PI Range
7	Clark	CH, CL, CL-ML, GC, GM, GM-GC, GP-GC, ML, SC, SM, SM-SC,	A-1, A-2, A-4, A-6, A-7	<100	NP-60
	Columbia	CH, CL, CL-ML, SM, SM-SC, SC, SP-SM,	A-2, A-3, A-4, A-6, A-7	<83	NP-53
	Cleveland	CH, CL, MH, MH-ML, ML, ML-CL, SM, SM-SC, SC	A-1, A-2, A-4, A-6, A-7, A-7-6	NA	NA
	Union	CH, CL, CL-ML, ML, SC, SM, SC-SM, SP-SM,	A-1, A-2, A-2-4, A-3, A-4, A-6, A-7, A-7-6	15-95	NP-40

4.8 DISTRICT 8

Based on the available information, soils of this district are predominately A-1, A-2, A-2-4, A-4, A-6, and A-7. The LL value is up to 80 and the PI values ranges from NP to 55 (see Table 4.8).

Table 4.8 Soil Information to a Depth of 18 inch in District 8

District	County	Unified Soil Classification	AASHTO Classification	LL Range	PI Range
8	Faulkner	CH, CL, CL-ML, GC, GM, GM-GC, SC, SM, OH	A-1, A-2, A-4, A-6, A-7	<80	NP-50
	Montgomery	CL, CL-ML, GC, GC-GM, GM, GP-GC, ML, SC, SC-SM	A-1-a, A-1-b, A-2, A-2-4, A-2-6, A-4, A-6, A-7-6	<70	3-45
	Perry	CH, CL, CL-ML, GC, GM, GM-GC, GP, GP-GM, ML, SM, SM-SC	A-1, A-2, A-4, A-6, A-7	<80	NP-50
	Pope	CH, CL, CL-ML, GM, GM-GC, GP-GM, SM, SP-SM, SM-SC	A-1, A-2, A-4, A-6, A-7, A-7-6	<80	NP-50
	Van Buren	CH, CL, CL-ML, GC, GM, GC-GM, GP-GM, SC, SM, SM-SC,	A-1, A-2, A-4, A-6, A-7	<80	NP-40
	Yell	CH, CL, CL-ML, ML, GC, GC-GM, GP-GM, GM, SC, SM, SM-SC, SP-SM	A-1, A-2, A-4, A-6, A-7	<80	NP-50

4.9 DISTRICT 9

Similar to District 8, soils in District 9 are predominately classified as A-1, A-2, A-4, A-6, A-7, A-7-6 with a LL value up to 80 and a PI value ranges from NP to 55 (see Table 4.9).

Table 4.9 Soil Information to a Depth of 18 inch in District 9

District	County	Unified Soil Classification	AASHTO Classification	LL Range	PI Range
9	Boone	CH, CL, CL-ML, GC, GM-GC, GP-GM, ML, SM, SM-SC, SC	A-1, A-2, A-4, A-6, A-7	<80	NP-50
	Carroll	CH, CL, CL-ML, GC, GM, GP-GC, SM, SM-SC, SC, SP-SM	A-1, A-2, A-3, A-4, A-6, A-7	<80	NP-55
	Madison	CH, CL, CL-ML, ML, GC, GM, GP-GC, SM, SM-SC, SC, SP-SC	A-1, A-2, A-4, A-4, A-6, A-7, A-7-6	<80	NP-50
	Marion	SM, SC, SM-SC, GM, GC, GP-GM, CH, CL, CL-ML, ML,	A-1, A-2, A-4, A-6, A-7	<80	NP-55
	Newton	CH, CL, CL-ML, GC, GM, GM-GC, GP-GM, SM, SM-SC, SC, SP-SM	A-1, A-2, A-4, A-6, A-7	<80	NP-50
	Baxter	SM, SC, SM-SC, GM, GC, GP-GM, CH, CL, CL-ML, ML,	A-1, A-2, A-4, A-6, A-7	<80	NP-55
	Searcy	CH, CL, CL-ML, Ml, GC, GM-GC, GP-GC, GP-GM, SM, SM-SC, SC	A-1, A-1-b, A-1-a, A-2, A-2-7, A-4, A-6, A-7	<85	NP-55

4.10 DISTRICT 10

Soils in District 10 are predominately classified as A-1, A-2, A-4, A-6, A-7, and A-7-6 with a LL value up to 95 and a PI value ranges from NP to 55 (see Table 4.10).

Table 4.10 Soil Information to a Depth of 18 inch in District 10

District	County	Unified Classification	AASHTO Classification	LL Range	PI Range
10	Greene	CH, CL, CL-ML, GC, GC-GM, GM, ML, SC, SC-SM, SM, SP-SM	A-1, A-2, A-4, A-6, A-7, A-7-6	<95	NP-45
	Craighead	CH, CL, CL-ML, GC, GM-GC, SM, SM-SC, SP-SM	A-1, A-2, A-4, A-6, A-7	<85	NP-55
	Clay	CH, CL, CL-ML, GM, GM-GC, GC, ML, SM, SM-SC, SC	A-2, A-4, A-5, A-6, A-7	<85	NP-55
	Randolph	CL, CL-ML, ML, MH, GM, GM-GC, GC, SM, SM-SC, SC	A-1, A-2, A-4, A-6, A-7	<85	NP-55

Based on the information included in Tables 4.1 through 4.10, it can be concluded that surficial soils across the state possess a wide range of properties and cannot be classified into a few categories. Each of the counties in a single district has soils of different categories with different index properties. Both of the USCS and AASHTO classification had different classes for soils in counties and the LL and PI of the soils were varying in relatively wide ranges. Accordingly, it can be concluded that the surficial soils in Arkansas vary significantly; hence, deciding a county-specific stabilization technique may not be feasible. Therefore, it is our professional opinion that feasible soil stabilization techniques for a specific project should be solely based on the index properties and conditions of the soil requiring improvement.

5 SUMMARY OF STABILIZATION EXPERIENCE OF NEIGHBORING STATES

The chapter includes a summary describes the state-of-the-practice and specifications of Arkansas' neighboring states (Texas, Louisiana, Missouri, Mississippi, Tennessee and Oklahoma). The data presented in this paper has been obtained from published materials such as state DOTs projects, technical reports, journals, etc.

5.1 TEXAS

Details of stabilization techniques of subgrade soils in Texas can be found in *Guidelines for Modification and Stabilization of Soils and Base for Use in Pavement Structure* (TXDOT, 2005). A flow chart for subgrade soil treatment is presented in Figure 5.1, which shows that specific tests (Soil Classification (Tex-142-E), Sieve Analysis (Tex-110-E), Atterberg Limits (Tex-104, 105, 106, and 107-E), and sulfate content (Tex-145-E and Tex-146-E)) should be conducted on soil samples. Specific guidelines should also be followed in the sulfate content in the soil exceeds 3000 ppm.

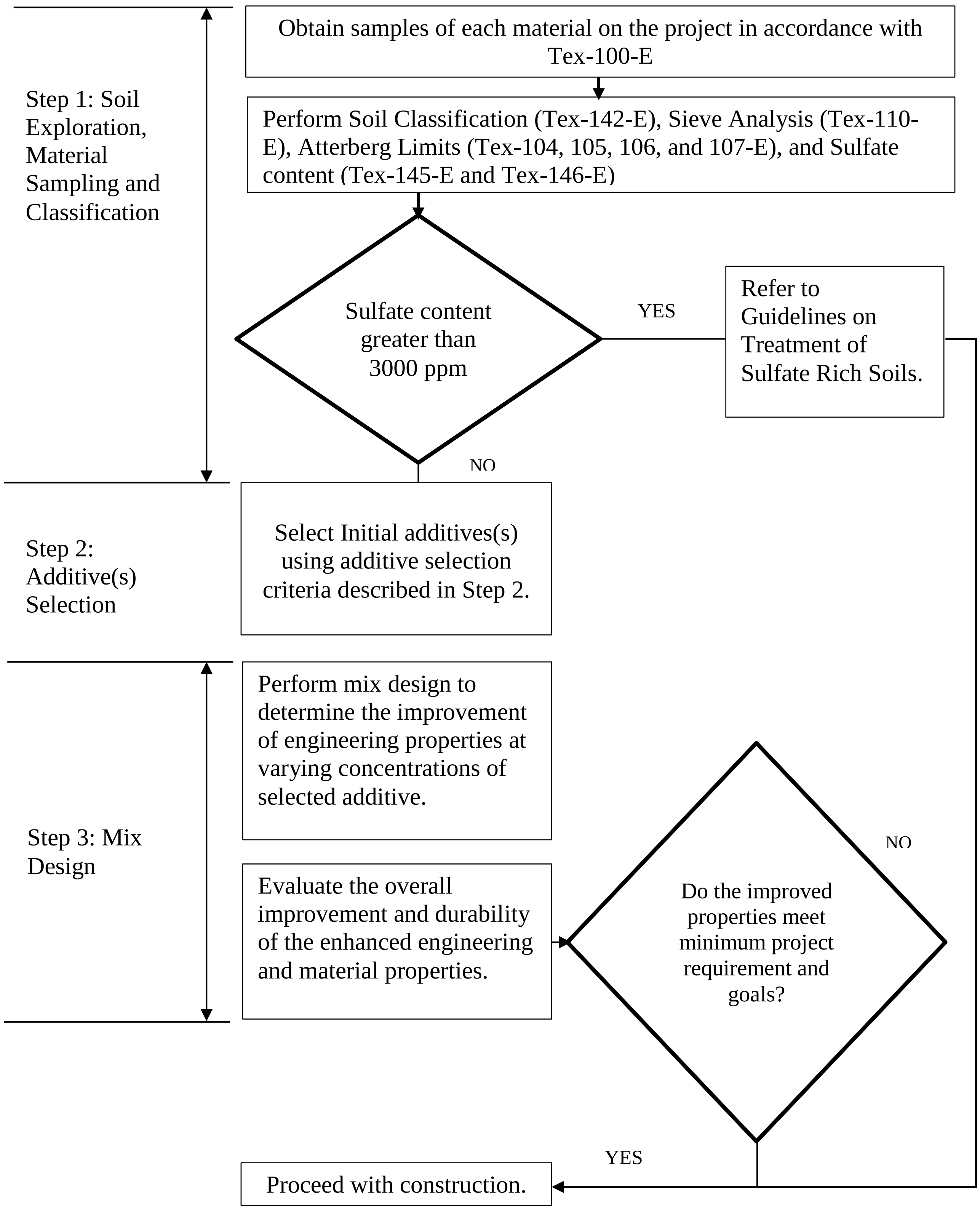

Figure 5.1 Flowchart for Subgrade Soil Treatment in Texas (TXDOT, 2005)

TXDOT guidelines base the selection of stabilizing agent for a subgrade based on the classification test results (Atterberg Limits and grain size analysis) as shown Figure 5.2. Details of the test procedures used to stabilize soils with cement, lime and fly-ash are presented in TxDOT Tex-120-E, Tex-121-E and Tex-127-E, respectively.

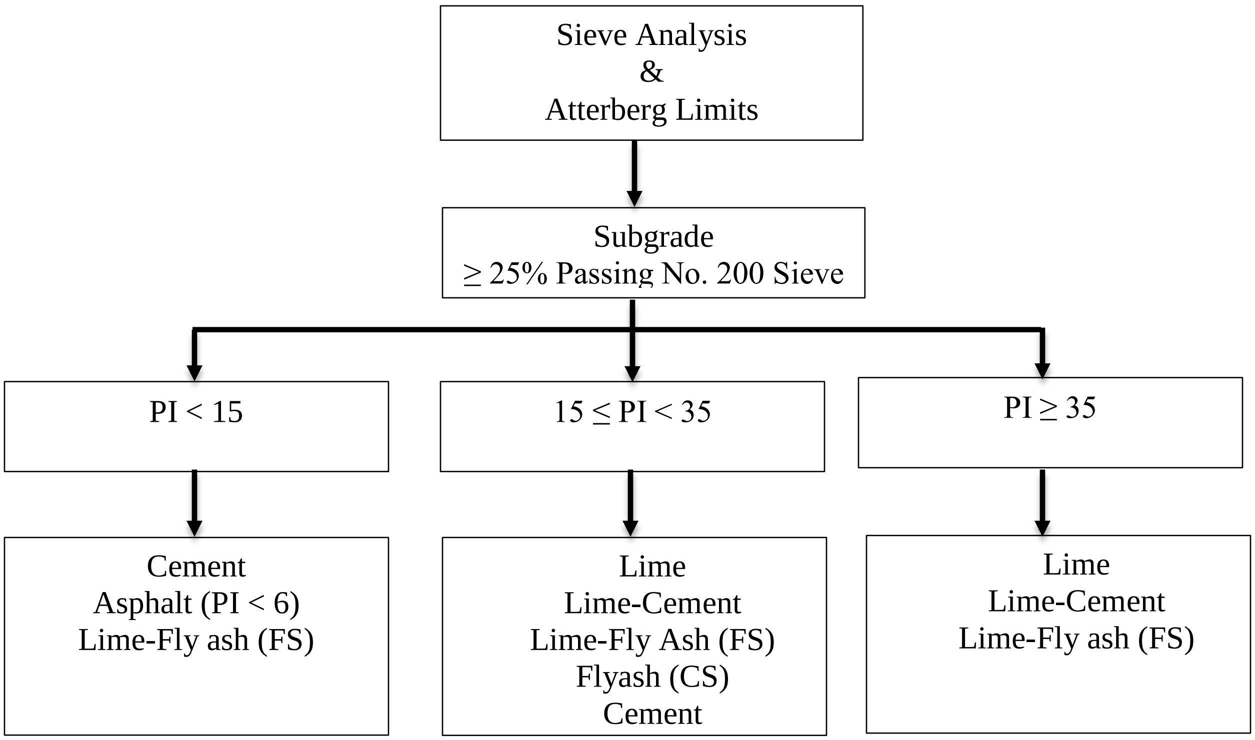

Figure 5.2 Additive Selection for Subgrade Soils in Texas (TXDOT, 2005)

Special treatment options should be followed for sulfate rich (sulfate concentration, SC, >3000 ppm) soils and the flowchart included in Figure 5.3 depicts the necessary treatment of soils based on sulfate levels where soils have been categorized in 3 levels. The lower two levels can be improved by modified treatment, which basically needs additional mellowing time and moisture addition. Level III soil, however, cannot be modified, rather to be excavated and replaced with non-plastic soils, or blended with non-plastic soils.

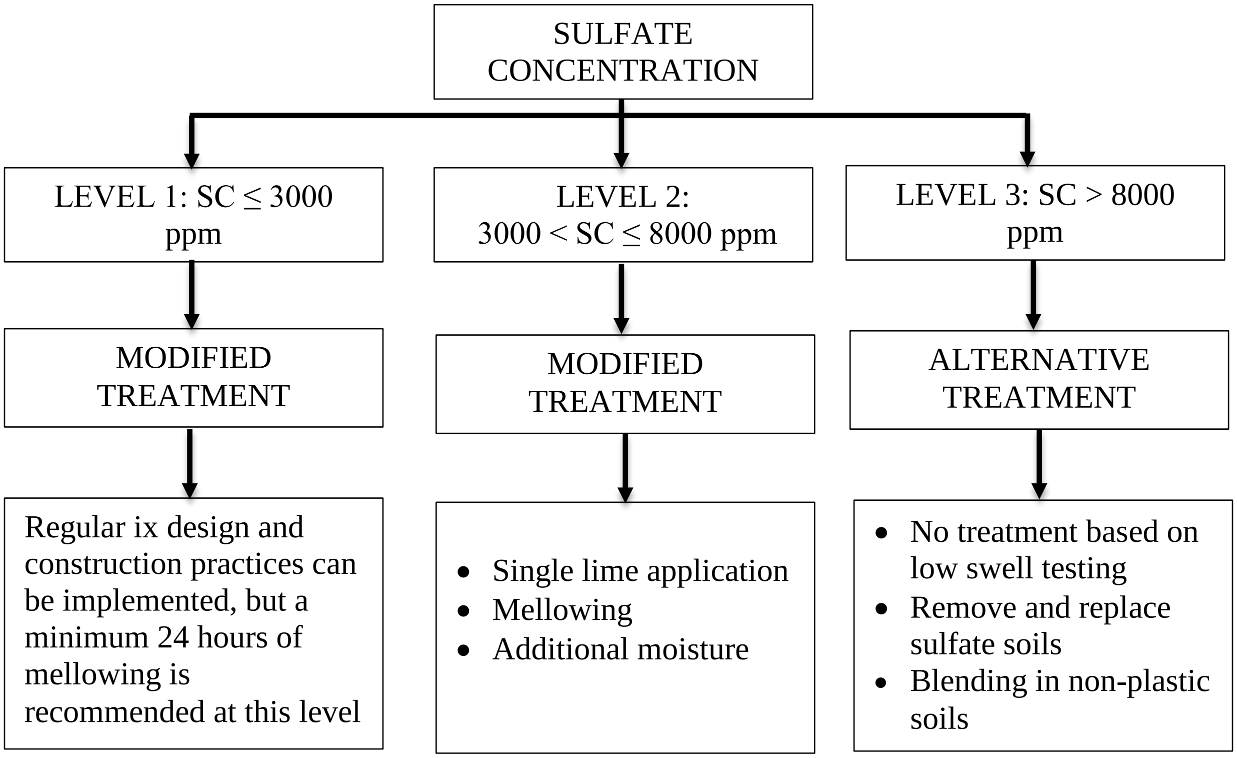

Figure 5.3 Type of Treatment for Varying Sulfate Contents in Texas (TXDOT, 2005)

In a separate study, Veisi et al. (2010) proposed revised accelerated testing methods that could potentially reduce the time required for sample preparation, curing, moisture conditioning and laboratory testing specified in the current stabilization specifications of TXDOT. The researchers estimated moisture contents, strengths, dielectric values and moduli of selected soils with different amounts of stabilizers. Four clay and two sandy materials, namely, El Paso clay, Bryan clay, Fort Worth Clay, Paris Clay, Wichita Falls sand and Bryan sand were investigated for stiffness, strength and tube suction test (TST). It was reported that pH tests were good indicators to obtain initial amount of stabilizer when lime is used. The researchers also recommended investigating clay mineralogy aspects of the raw soils rather than the PI based approach.

The unconfined compressive strength test and TST were conducted for three different amounts (6%, 8% and 10%) of lime out on the standard cured and dried samples. It was reported that a minimum strength of 50 psi was achieved for all three combinations and curing type. The highest compressive strength (132 psi) was found for 8% additives with 7 day curing. They used 4 in. by 8 in. specimen and combination of confining pressures of 0, 2, 4, and 6 psi with deviator stresses of 2, 4, 6, 8, and 10 psi for the stiffness test. El Paso dry soil with 8% lime additive exhibited highest resilient modulus (62 ksi) followed by 6% lime additive (60 ksi). On the other hand, El Paso soil with 10% lime additive and standard curing exhibited the highest resilient strain (160) and permanent strain (720). The researchers observed significant variation of dielectric constant with time for El Paso clay with the three amounts of lime stabilizers. These researchers also observed that after the first day of curing period, dielectric values were less than 10, indicating the superior performance. These researchers recommended a curing time of 2 days and a back-pressure method to complete moisture conditioning to complete the mix design in 3 days.

Bhattacharja and Bhatty (2003) compared the performance of lime and cement on three different types of soils in Texas with PI of 25%, 37% and 42%, and found that for all soils, better performance was observed when cement was used. However, there was great decrease in the strength (by more than 50%) of the cement treated soils with delay compaction of 24 hour.

5.2 TENNESSEE

Tennessee Department of Transportation (TDOT) has introduced specifications for lime-treated subgrade soils (Dam et al. 2010; TDOT, 2013). As presented in TDOT Specifications Section 302: Subgrade Treatment (Lime), treated soils should consist of in-place subgrade material and lime should be uniformly mixed, moistened, compacted and cured. The soil used in this work shall consist of the in-place subgrade material. If the in-place soil is unsuitable for stabilization, it should be replaced with suitable material. Samples of the in-place subgrade soils should tested in the laboratory to determine the percentage of lime required and the appropriate optimum moisture content of the lime-soil mixture as determined by AASHTO T 99, Method C. Lime can be used in the forms of hydrated lime or quicklime meeting ASTM C 977. Further, lime application is not permitted when the subgrade material is frozen, unless the air temperature in the shade is at least 4.0º C (40º F) and rising. Lime stabilization is permitted only if the stabilized soils are covered with the subbase or base course during the same construction season. It is also stated that lime shall be applied only to such areas that can be sealed in accordance with Subsection 302.09 Initial Mixing and Mellowing (TDOT, 2013) during the day of application. Other construction guidelines of TDOT are as follows:

- Lime that has been exposed to the open air for a period of six hours or more will be unacceptable for payment;
- To prevent excessive loss, dry lime should not be applied during periods of high winds;
- Other than equipment needed for spreading, watering, or mixing, no traffic or equipment will be permitted on the spread lime;
- Treated subgrade soil will have to be reshaped to the design grades, and cross sections and sealed with a pneumatic-tire roller, and other approved equipment, and left to mellow for a period from two hours to seven days. During this mellowing period the finish surface of the treated subgrade soil should be maintained in a moist condition.
- The entire depth of the lime-stabilized soil should be compacted by using sheepsfoot rollers to achieve the required density.

5.3 OKLAHOMA

Oklahoma Department of Transportation (ODOT) established subgrade stabilization and modification specifications for limited number local soils (ODOT, 2009). In the case of subgrade stabilization, chemical additives are incorporated in sufficient quantities to increase the shear strength of subgrade soils, and the guidelines are specified in *OHD L-50 Soil Stabilization Mix Design Procedure*. On the other hand, for subgrade modification, chemical additives are incorporated to change the PI and improve the workability of subgrade soils. The guidelines are specified in *OHD L-51 Soil Modification Mix Design Procedure.* For both modification and stabilization, the additive shall be applied at a rate based on AASHTO classification tests of the subgrade soil.

Subgrade soil modification is permitted to clayey soil of the AASHTO M 145 soil types A-4, A-5, A-6 and A-7, and the recommended percentage of stabilizing additives are presented in Table 5.1. However, the recommended percentages of additives will have to be verified for any specific soil. For verification purpose, trial percentages (2, 3 and 4% for Portland cement and CKD from prep-calciner plants; 4, 6 and 8% for CKD from other type plants; 5, 7 and 9% for CFA) of additives shall be tested to determine their optimum amounts. In case the recommended amount of lime is a concern for a specific soil, ODOT recommends following the ASTM D 6276 *Standard Test Method for Using pH to Estimate the Soil-Lime Proportion Requirement for Soil Stabilization (Withdrawn 2015)* method to determine the estimated required percentage of lime, which will have to be multiplied by 0.6 and then rounded to the nearest 0.5% to determine the recommended dosage level. Besides Liquid Limit (AASHTO T 89), Plastic Limit (AASHTO T 90) and gradation (AASHTO T 88), which are the tests needed for the AASHTO M 145 Classification System, subgrade soils should be tested for soluble sulfates according to OHD L-49. If the soluble sulfate content of any soil sample is more than 800 ppm, calcium-based additives are not recommended. The target density and optimum moisture contents of raw and treated soils shall be determined in accordance with AASHTO T 99 Method A, Method B, or Method C, as appropriate.

Table 5.1 Recommended Percentages of Modification Additives in Oklahoma (OHD, 2009a)

SOIL MODIFICATION TABLE					
ADDITIVE (Expressed as a percentage added on oven dry basis)	**SOIL GROUP CLASSIFICATION-AASHTO M145**				
	A-4	**A-5**	**A-6**	**A-7**	
				A-7-5	**A-7-6**
PORTLAND CEMENT	3	3	3		
FLY ASH	9	9	9		
CEMENT KILN DUST (Pre-Calciner Plant)	4	4	4		
CEMENT KILN DUST (Other Type Plant)	8	8			
HYDRATED LIME*			3	3**	3**
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. *: Reduce quantity by 20% when quick lime is used, i.e. 3% x 0.8 = 2.4%, 4% x 0.8 = 3.2% **: Use 4% when the liquid limit is greater than 50.					

Similar to subgrade soil modification, soil stabilization specifications of ODOT are developed for various AASHTO M 145 soil types as presented in Table 5.2. Sulfate solubility remains another predicator whether calcium-based additives would be effective for a specific soil, and they are not recommended if sulfate solubility is more than 800 ppm. Some ODOT districts (districts 2, 5 and 7) have dispersive soils, which shall be tested further using the crumb test (ASTM D 6572). Depending on the severity of dispersion test results, further testing of these soils are needed by using the Pinhole test (ASTM 4647), as special treatments may be required before using any calcium-based stabilizing agent. The recommended percentages of stabilizing additives will have to be verified for any specific soil. For verification purpose, trial percentages (2, 4 and 6% for Portland cement and CKD from prep-calciner plants; 7, 9 and 11% for CKD from other type plants; 6, 9, 12 and 15% for CFA) of stabilizing additives shall be tested to determine their optimum amounts. Further, the mix design report for stabilized soils should include the following information: (i) AASHTO soil classification of the untreated soils; (ii) soluble sulfate content; (iii) AASHTO soil classification of lime pretreated soil, if applicable; (iv) unconfined compressive strengths of cured specimens; (v) unconfined compressive strengths and moisture absorption of immersed specimens, if applicable; (vi) recommended percent of stabilizing additive and its source; (vii) density and optimum moisture content of untreated soil; and (viii) density and optimum moisture content of stabilized soil.

Table 5.2 Recommended Percentages of Stabilization Additives for Soils in Oklahoma (OHD, 2009b)

SOIL MIDOFICATION TABLE												
ADDITIVE (Expressed as a percentage added on oven dry basis)	**SOIL GROUP CLASSIFICATION-AASHTO M145**											
	A-1		**A-2**				**A-3**	**A-4**	**A-5**	**A-6**	**A-7**	
	A-1-a	**A-1-b**	**A-2-4**	**A-2-5**	**A-2-6**	**A-2-7**					**A-7-5**	**A-7-6**
PORTLAND CEMENT	4	4	4	4	4	4	5					
FLY ASH					12	12	13	14	14	14		
CEMENT KILN DUST (Pre-Calciner Plant)	5	5	5	5	5	5	6					
CEMENT KILN DUST (Other Type Plant)	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.
*: Reduce quantity by 20% when quick lime is used, i.e. 4% x 0.8 = 3.2%, 5% x 0.8 = 4.0%, 6% x 0.8 = 4.8%
**: Use 6% when the liquid limit is greater than 50.

Snethen et al. (2008) studied CKD, CFA, Portland cement, and lime stabilized subgrade soils collected from five different sites in Oklahoma. The researchers measured unconfined compressive strength (UCS), resilient modulus (M_r) and field performance parameters such as

Nuclear w-γ, stiffness gauge, portable falling weight deflectometer (FWD), dynamic cone penetration (DCP) and PANDA Penetrometer. It was reported that the type and amount of chemical additive is dependent on the purpose or function of the treated material (i.e., improved physical properties or improved strength) and selection is based on accepted or standardized procedures. The researchers reported that the UCS and M_r values for field mixed samples are 50 to 90% of the laboratory mixed samples and a general trend was the higher the PI of the soils the greater the difference between field and laboratory conditions. Further it was observed that typically 70% or more of the strength and structural improvement occurred in 7 days, and the actual rate of improvement depended on untreated soil type, additive (type, amount, quality), construction procedure, and curing environment. Among the selected additives of that study, CKD yielded higher strengths more quickly than CFA. It was also reported that the MEPDG *Level 2* correlation equations significantly under estimate M_r values for stabilized soils. Since satisfactory *Level 2* correlation could not be established, or did not exist in the literature at the time, it was suggested that the basic correlation of M_r = 1500 x CBR, with CBR defined from Dynamic Cone Index (DCI) values measured from stabilized soil layers, be used in design until better correlations are established. Additionally that study suggested that the DCP and the PANDA Penetrometer would provide very good measures of long term performance of stabilized soils layers and showed very good potential for use as quality control tools.

In another recent study (Solanki et al. 2009) determined M_r values of three stabilized (lime, CFA and CKD) soils in Oklahoma. Four different types of clayey soils (Port: A-4, Kingfisher: A-6, Vernon: A-6, and Carnasaw: A-7-6 series) were studied and effects of different dosages of these additives on M_r were identified. These researchers reported increased M_r values with the addition of any of the aforementioned stabilizing agents. It was also reported that at lower application rates (3% to 6%), the lime-stabilized soil specimens showed highest increase in the M_r values. At higher application rates (10% to 15%), however, CKD treatment provided maximum improvements. Further, it was reported that the stabilization with lime and CFA provided highest M_r values for CL soils, while CKD stabilization provided maximum improvements for the lower plasticity CL-ML soil.

In a follow-up study, Hossain et al. (2011) focused on calibrating the MEPDG for stabilized subgrade soils in Oklahoma through regression modeling of selected stress-based M_r models. The researchers reiterated significant increase in M_r values for all four soil types with lime, CKD and CFA additives, but the extent of increase in the M_r values depended on the type of soil, and type and amount of additive. A dosage of 15% CKD showed the maximum increase of the M_r value for all four soil types. The researchers established correlation equations between resilient properties of lime-, CKD- and CFA-stabilized clayey soils and routine soil properties for local calibration of the MEPDG in Oklahoma.

5.4 MISSISSIPPI

Mississippi DOT (MDOT) approves the use of lime and fly ash as stabilizing agents to improve the quality of poor subgrade soils. Lime can be in the form of hydrated or quick lime to the more plastic subgrade soils to decrease the plasticity and volume change characteristics and to increase workability, strength and durability. The mix design procedure for hydrated lime as a stabilizing agent is illustrated in the 2005 Mississippi Materials Division Inspection, Testing, and

Certification Manual (MSDOT, 2005). The following three categories of hydrated lime treatments are illustrated in the MSDOT specifications:

(1) Class A Treatment of Lime: This consists of spreading and incorporating the lime in two increments. In the first increment, spread the predetermined percentage of lime, mix with excess amount of water, seal, mellow or cure from five to twenty days. In the second increment, spread the second increment of lime, mix, compact, finish, and maintain until covered by a subsequent course.
(2) Class B Treatment of Lime: This treatment consists of spreading and incorporating the predetermined percentage of lime, mixing with excess amount of water, sealing, mellowing or curing from five to twenty days, mixing, compacting, finishing, and maintaining until covered by a subsequent course.
(3) Class C Treatment of Lime: This treatment consists of spreading and incorporating the predetermined percentage of lime, mixing, compacting, finishing, and maintaining until covered by a subsequent course.

The MDOT requires the following test methods to be accomplished in performing mix designs of subgrade soils stabilized with limes: (a) AASHTO T 87 - Dry Preparation of Disturbed Soil and Soil Aggregate Samples for Test, (b) Mississippi Test Method (MT)-23 - Particle Size Analysis of Soils, (c) AASHTO T 89 - Liquid Limit of Soils, (d) AASHTO T 90 - Plastic Limit and Plasticity Index of Soils, (e) AASHTO T 92 - Shrinkage Factors of Soils, (f) MT-8 - Moisture-Density Relations of Soils, (g) MT-9 - Moisture-Density Relations of Treated Soils, and (h) AASHTO T 193 - The California Bearing Ratio. The details of these test methods are presented in the section MT-27 Design of Soil-Lime Water Mixtures in the 2005 Mississippi Materials Division Inspection, Testing, And Certification Manual (MDOT, 2005). In accordance with the 2005 MDOT Specifications, the required hydrated lime content and class treatment should be the least percentage of lime which produces a minimum CBR of 20 and a satisfactory minimum swell. In the case quick lime is added, its percentage at 83% of the required hydrated lime content.

Specification details for fly ash modifications of subgrades soils are also explained under section MT-79 Design of Soil-Lime-Fly Ash Mixtures in the 2005 MDOT Specifications (MDOT, 2005). In accordance with the MDOT specification, soil-lime-fly ash (LFA) is defined as a mixture of pulverized soil, hydrated lime, and fly ash, which has been moistened, compacted, and permitted to harden. The LFA technique is primarily used for a base course. But, it is also used as a chemical stabilization technique for subgrade soils.

The MDOT requires the following tests to be performed required in the design of LFA mixtures: (a) MT-9 Moisture-Density Relations of Treated Soils, (b) MT-23 Methods of Testing Soils, (c) MT-26 Compressive Strength of Soil-Cement Cylinders and Cores, (d) AASHTO T 85 Specific Gravity and Absorption of Coarse Aggregate, (e) AASHTO T 87 Dry Preparation of Disturbed Soil and Soil Aggregate Samples for Test, (f) AASHTO T 89 Determining the Liquid Limit of Soils, (g) AASHTO T 90 Determining the Plastic Limit and Plasticity Index of Soils, (h) AASHTO T 92 Determining the Shrinkage Factors of Soils, (i) AASHTO T 99 The Moisture-Density Relations of Soils Using a 5.5-lb Rammer and a 12-in Drop, and (j) AASHTO T 100 Specific Gravity of Soils.

The percentages of Class "C" fly ash is dictated by the compressive strength of modified soils. In this regard, the 14-day compressive strength results are used only as an early indicator to evaluate the trial design percentages. A trial design blend of 3% hydrated lime/12% fly ash (by dry weight) as a starting point blend. The design lime and fly ash content is selected corresponding to the blend that that will produce a 28-day compressive strength of 400 psi for treated subgrade material. In regard to the sampling process the following protocols are adopted by MSDOT: (1) soil stabilization projects containing less than 2000 tons of fly ash, a sample will not be required, (2) sampling shall be at the rate of one sample for each 4000 tons for soil stabilization, or fraction thereof, received.

In a recent study, James et al. (2009) conducted laboratory evaluations to quantify the effects of compaction and moisture conditions on the strength of chemically treated soils for the typical Mississippi highways. The researchers evaluated seven typical virgin soils (four AASHTO Soil types: A-7-6, A-6, A-4, and A-2-4) of Mississippi. They conducted strength tests (CBR, UCS, and resilient modulus) of these virgin materials in order to develop baseline strength data. Afterwards, selected soils were combined with lime, cement, and/or lime/fly ash to represent typical MDOT stabilized materials. These researchers observed the followings:

- In the case of lime stabilized soils, there is an influence of increased moisture on the results of the CBR test. However, this influence was more pronounced on CL soils having a plasticity index of 8. The influence of the increased moisture content was minimal for the two soils having a plasticity index greater than 18.
- In the case of cement stabilized soils, there was an influence of density on the results of the UCS test. As the percent maximum dry density increased, the unconfined compressive strength increased. In general, the UCS values were lower for cement stabilized soils when 3% above optimum moisture was added.
- In the case of lime/fly ash stabilized soils, there was an influence of density on the results of the UCS test. The UCS increased as the percent maximum dry density increased. The UCS values were lower for lime/fly ash treated soils when 3% above optimum moisture was added.
- Resilient modulus was found to increase as the percent maximum dry density increased. The chemical treatment of the virgin soils significantly increases resilient modulus.

Based on the findings of the aforementioned study (James et al. 2009), these researchers made the following recommendations:

- Compaction requirements for the chemically treated subgrade soils should be at least 98% of standard Proctor density.
- Subgrade soils strengths can be significantly improved if compaction requirements are increased to 100% of standard Proctor density.
- Chemically treating the top 6 to 9 inches of subgrade soils should be required to improve structural strength, integrity and capacity of pavement structure.
- Field moisture contents should not exceed the optimum moisture content by 3%.

5.5 LOUISIANA

The Louisiana Department of Transportation and Development (LADOTD) recommends using either Portland cement or a combination of Portland cement and lime to improve the strength of subgrade soils (LADOTD, 2006). The in-place and the plant-mixing treatment with Portland cement should be in accordance with the LADOTD Specifications' *Section 303*, and *Section 301*, respectively. When lime is used, the treatment option should be in accordance with *Section 304*. The type of additive and its minimum quantity should be in accordance with the following guidelines presented in Table 5.3. However, the engineer is authorized to increase or decrease the percentages of Portland cement and lime based on field conditions.

Table 5.3. Stabilizer Selection Criteria for Louisiana Soils (LADOTD, 2006)

Plasticity Index (PI)	Lime/Cement (Percent by Volume)
0-15	9% Portland cement
16-25	6% lime + 9% Portland cement
26-35	9% lime + 9% Portland cement

The LADOTD TR 416 method, *Determination of the Percentage of Lime for Treatment of Soils or Soil-aggregate Mixtures*, is used to determine the minimum amount of lime required for treatment of soils based on the relationships between the percentage of lime added and reduction of plasticity (LADOTD, 2006). The minimum required lime amount, by weight, is estimated based on soils' LL and PI values with the requirements, which states that after the lime treatment, the treated soil shall have a maximum LL of 40 and a maximum PI of 10. For construction purposes, the percentage by weight is then converted to a percent by volume, either by using an additive conversion chart or formula considering the treated soil's maximum dry unit weight. Thus the LADOTD TR 416 method is a straight forward technique, but it does not consider other engineering properties of raw soils.

In an experimental study, McManis (2003) investigated instability and pumping response of non-plastic, high silt (and fine sand) soils that commonly encountered in the preparation of the subgrade for highway pavement projects in Louisiana. The researcher used lime, lime-fly-ash, Portland cement, slag cement reagents with three non-plastic soils with high silt and fine sand contents from different locations (Lake Charles District 07 U.S. Highway 171 project, the site of the LADOTD's Accelerated Load Facility ALF in West Baton Rouge, and the Natchitoches K2-1 soil acquired from the Alexandria District 08), and conducted a series of laboratory tests to simulate the moisture and loading conditions of these soils with admixtures during subgrade construction and long-term service condition. The percentages of sand, silt and clay of the US 171 soil was 14%, 75%, and 11%, respectively. For the K2-1 soil, these percentages were 24%, 60%, and 16%, and for the ALF soils they were 49%, 39% and 12%, respectively. The LL and PI values of US 171, K2-1, and ALF soils were found to be 17 and 2, 25 and 3, and 22 and 1, respectively. All three soils were classified as ML using the Unified Soil Classification and as an A-4 soil using the AASHTO classification method. The researcher observed a decrease in the percentage of the moisture content ranged from 0.6%-2.3%, with the greatest drying effects occurring with lime and lime-fly-ash. It was also reported that all three soils exhibited a tendency to creep significantly when wet of optimum and subjected to cyclic loads of 60 psf to 90 psf. Soil stabilized with lime alone still exhibited a tendency to creep. In a majority of the cases, the stabilizing agents did eliminate or retard the extent of the cyclic deformation for the test specimens. However, their performance varied depending on the type of stabilizing agent and the moisture content of the soils. This researcher reported the greatest improvements of pumping of these soils with respect to strength were achieved with cement and fly-ash mixtures. It was further added that for construction purposes, the greater drying potential of the lime and lime-fly ash performed better in eliminating the pumping potential of the predominately high-silt soils. On the other hand, the stability of the predominately fine sand in the silty-sand soil was greater with the cements and the pozzolanic mixture of lime-fly ash. Further, toward achieving the long term stability, the author concluded that the increase in strength is a key parameter, and cement exhibited the best results followed by the lime-fly ash blends.

In a recent study, Gautreau et al. (2009) reiterated that the complete lime stabilization technique for subgrade soils is not addressed in the LADOTD specifications or test methods. These researchers performed a combination of field and laboratory study for chemical stabilization of the naturally wet and problematic clayey soils typically found in south Louisiana. Soil samples modified with various amounts of three selected additives namely, cement, lime, and lime-fly ash, were subjected to index properties, UCS, tube suction, resilient modulus and permanent deformation and Accelerated Loading Facility (ALF) tests, etc. The researchers suggested that lime in the form of quicklime should be selected as a stabilization agent if PI>10 and clay content (2μm) >10%, whereas cement should be used if PI ≤ 10 and the percent passing the No. 200 sieve (P_{200}) (75 μm) < 20%. Furthermore, for modification purposes, lime should be selected If PI ≥ 5 and P_{200} > 35%, lime fly ash blends should be selected if 5 < PI < 20 and P_{200} > 35%, and cement and/or CFA should be selected if PI < 5 and P_{200} ≤ 35%. It is also suggested that LKD should not be used in the blend. To estimate the optimum amount of an additive, the researchers suggested testing of soils modified with various amounts as follows: lime: from 4% to 9%, cement: from 4% to 10%, and CFA from 10% to 25%. These researchers suggested that the UCS data to be used to gauge the effects of modification and/or stabilization, and the 28-day (or 7-day accelerated) strengths of 300 psi for stabilization and 100 psi for modification can be used as the acceptance criteria.

5.6 MISSOURI

Subgrade soil modification guidelines are presented in *Section 205 Subgrade Stabilization* of *Missouri Standard Specifications for Highway Construction* published by the Missouri Department of Transportation, or MoDOT (MoDOT, 2014). The stabilizing material shall be hydrated lime or other chemical material, a geogrid, a geotextile, or other material approved by the engineer, and it should meet *Division 1000* guidelines of the MoDOT specifications. MoDOT requires the supplier of hydrated lime to furnish certification that the product is in accordance with AASHTO M 216. 205.3.1.1. In the case subgrade modification is not specified in the contract, in consultation with the engineer, the contractor may determine the locations, amount of modifying material and depth of application, within the limits of this specification. In general, MoDOT requires that subgrade modification be done to all areas uniformly and laterally between outside shoulder points plus 18 inches on each side. In the case the chemically modified areas are stopped and started, a longitudinal transition zone at the rate of 30 feet per 6 inches of modified depth should be maintained.

A study by Petry (2001) focused on determining the effectiveness of selected stabilizing agents for Mississippi River embayment soils in the I-55 Corridor, south of Sikeston, Missouri, for overcoming long standing slope failure issues. This researcher collected large bulk samples (disturbed and remolded later on for testing purpose) of the near surface materials as well as undisturbed samples at different depths using borings from two sites near Haiti, Missouri. The bulk samples were tested for identifications, compaction characteristics, 3-D swelling potentials, and UCS. The tested bulk soils had the appearance of a silty and a clayey soil, but both were classified as A-7-6 in accordance with AASHTO M 145. The undisturbed samples were subjected to identification, UCS and direct shear tests. In order to achieve the desired stabilization level, different percentages of the following three different agents were examined in this study: lime kiln dust (LKD), Portland cement, and a mixture of quick lime and fly ash (QL-FA). These stabilized soils were also tested for the aforementioned properties for comparison purposes with their unmodified counterparts. Among the tested stabilizing agents, the most effective agent was the 12% (by the dry weight raw soil) of the QL-FA (50% QL and 50% FA) mixture, which was followed by LKD (Code L).

6 PRELIMINARY STABILIZATION METHODS

The long-term performance of any construction project depends on the soundness of the underlying soils. Unstable soils can create significant problems for pavements or structures. AHTD engineers are often faced with problems of constructing roadbeds and embankments on weak soils, which do not possess sufficient strength to support wheel loads either during construction or the pavement service life. It may be necessary at times to treat these soils to provide a working platform for the construction or a stable subgrade. These treatments are generally classified into two processes: soil modification and soil stabilization. The purpose of subgrade modification is to create a working platform for construction equipment. The purpose of subgrade stabilization is to enhance the strength of the subgrade, and this increased strength is then taken into account in the pavement design process.

The methods of subgrade modification or stabilization include physical processes such as soil densification, blends with granular material, use of geogrids, undercutting and replacement, and chemical processes such as mixing with cement, fly ash, lime, lime byproducts, and blends of some of these materials. Basic soil properties such as strength, compressibility, permeability, workability, swelling potential, and volume change tendencies may be altered by various soil modification or stabilization methods. Different modification techniques are discussed later in this chapter. The selection of a stabilizing agent is typically based on the soil grain size and plasticity characteristics.

The NCHRP Project 20-07 report titled “Recommended Practice for Stabilization of Subgrade Soils and Base Materials,” also reiterated the necessity of site-specific treatment options to be validated through testing of soil-stabilizer mixtures under simulated field conditions. In selecting an appropriate stabilizing technique and exploration plan, National Resources Conservation Service (NRCS) County Soil Surveys and geological data sources can be used to define the extent and boundaries of soil series and the depth of soil horizons that may affect chemical stabilization. The NCHRP study provides a basic framework (Figure 6.1) to select an appropriate stabilizing technique for subgrade soils as well as for base materials based on soil properties such as soil classifications, gradation and index properties. This framework also helps decide the type of additive to be selected for soils with and without high sulfate contents. It further illustrates that if the soluble sulfate content is greater than 3,000 ppm then the user should perform swell tests to verify the expected degree of expansion and take construction steps to mediate the potential expansive reactions and additional steps to be followed for stabilizing sulfate bearing soils. The National Lime Association (NLA) protocol recommends water soluble sulfate to be evaluated following AASHTO T 290 (modified).

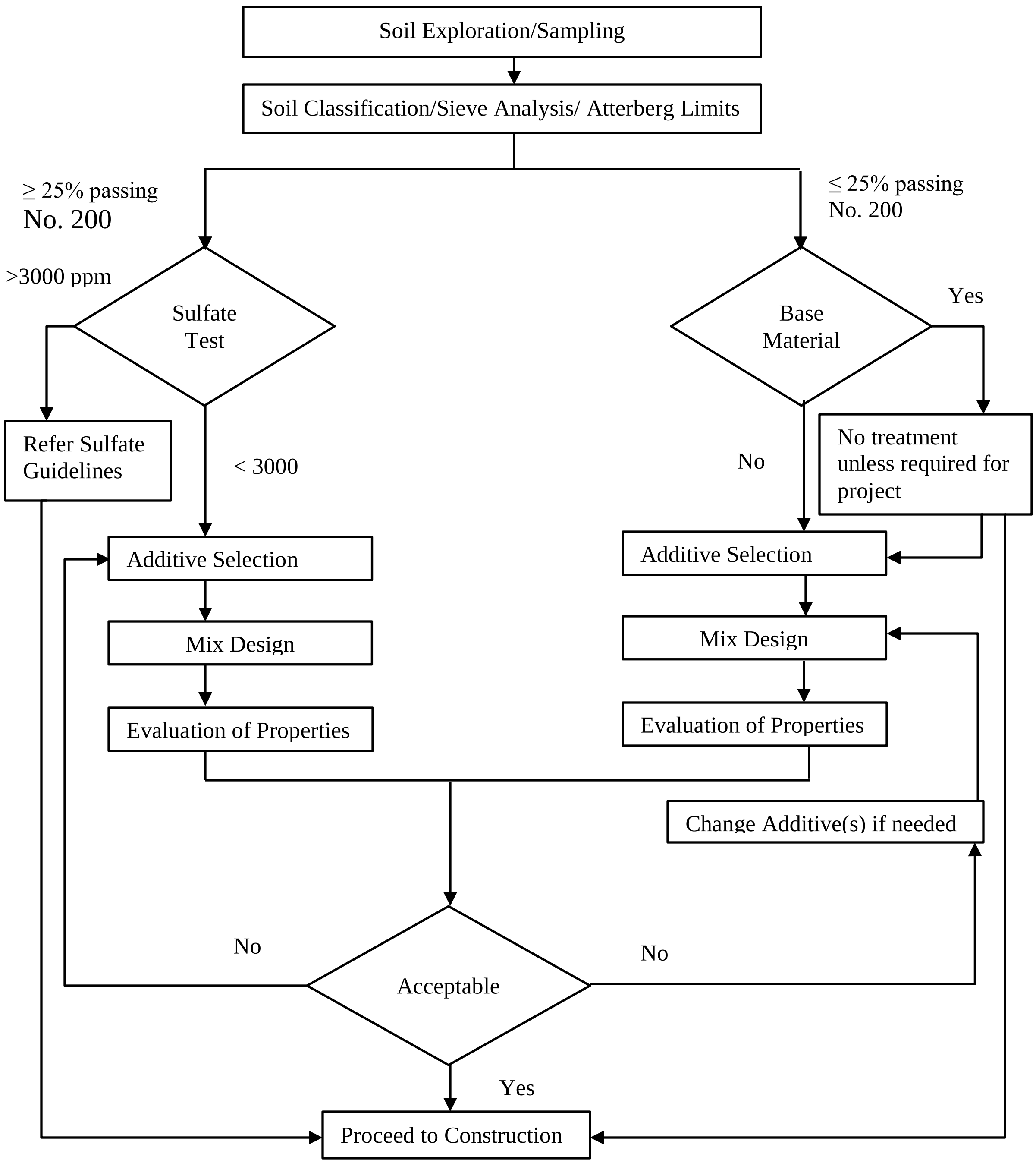

Figure 6.1 Stabilization of soils & base materials for use in pavements (NCHRP, 2008)

Figure 6.2 shows a basic framework to select an appropriate stabilizing agent for subgrade soils considering two index properties: PI and percent passing the No. 200 sieve (P_{200}). Estimating PI, liquid limit (LL) and plastic limit (LL) should be determined following AASHTO T 89 and AASHTO T-90, respectively. The P_{200} value of subgrade soil is determined following the AASHTO T-27 method. Once the stabilizing agent is selected, it is recommended to conduct detailed laboratory tests involving strength and performance characteristics to determine dosages level and mix design properties.

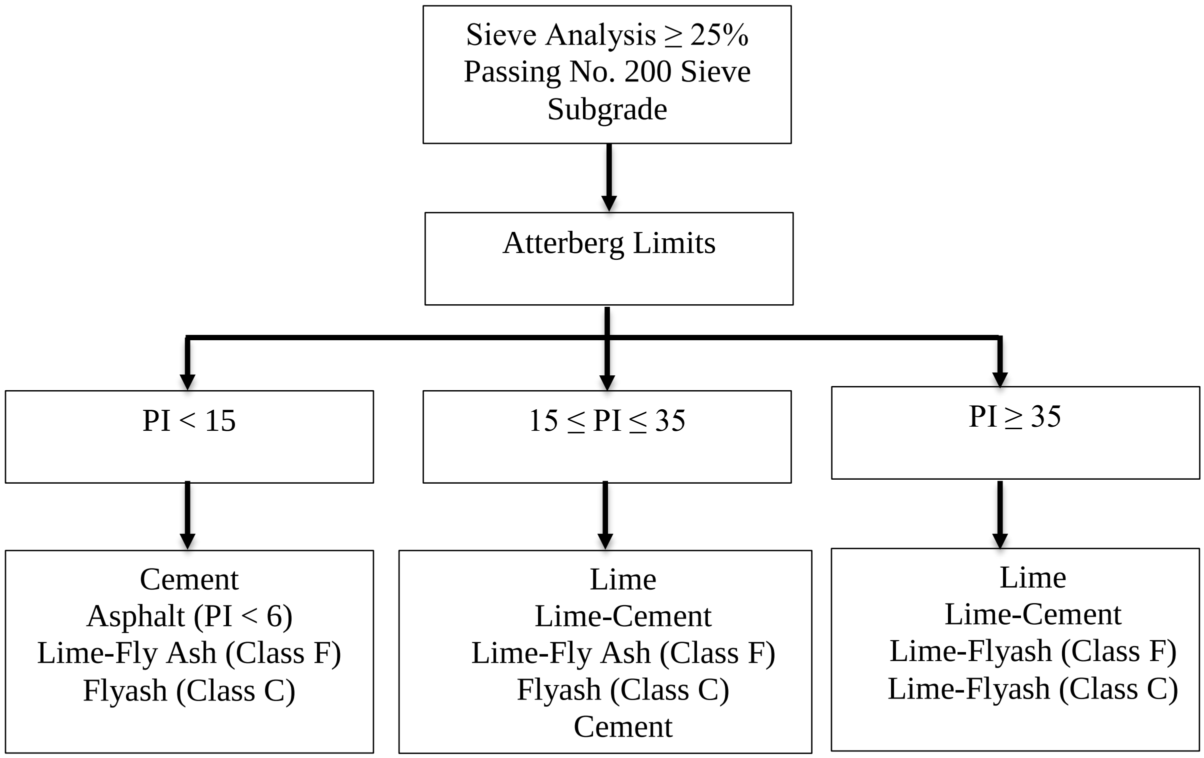

Figure 6.2 Framework for selecting an appropriate stabilizing agent (NCHRP, 2008) Mechanical Modification or Stabilization

Mechanical Modification or Stabilization

The mechanical modification or stabilization technique is the process of altering soil properties by changing the gradation through mixing with other soils, densifying the soils using compaction efforts, or undercutting the existing soils and replacing them with a granular material. A common remedial procedure for wet and soft subgrade is to cover it with granular material or to partially remove and replace the wet subgrade soil with a granular material to a pre-determined depth below the grade lines. To provide a firm-working platform with granular material, the following conditions shall be met: (a) the thickness of the granular material must be sufficient to develop acceptable pressure distribution over the wet soils, (b) the backfill material must be able to withstand the wheel load without rutting, and (c) the compaction of the backfill material should be in accordance with the standard specifications. The depth of replacement of weak subgrade soils is selected based on experience. For instance, Indiana DOT engineers usually replace 12 to 24 in. of existing subgrade soils with of granular materials. However, deeper undercut and replacement may be required in certain areas.

Chemical Modification or Stabilization

Chemical modification or stabilization is the transformation of soil index properties by adding chemicals such as cement, fly ash, lime, or a combination of two or more of these. Such treatments often alter the physical and chemical properties of the soil. There are the two primary mechanisms by which chemicals alter the soil into a stable subgrade soils: (1) increase in particle size by cementation, internal friction among the agglomerates, greater shear strength, reduction in the plasticity index, and reduced shrink/swell potential; and (2) absorption and chemical binding of moisture that will facilitate compaction.

Lime Stabilization

With proper design and construction techniques, lime treatment chemically transforms unstable soils into usable soils and increases their structural strengths. When mixed properly, lime has been found to react successfully with medium, moderately fine and fine grained soils causing a decrease in plasticity and swell potential of expansive soils, and an increase in their workability and strength properties (NCHRP, 2008). The NLA manual: “Lime-Treated Soil Construction Manual,” published in 2004, includes detailed specifications and construction mechanisms of lime treatments. As described in the NLA manual, fine-grained clay soils with a minimum of 25% passing the #200 sieve and a plasticity index (PI) greater than 10 are generally considered to be good candidates for lime (3 to 6% lime by weight of the dry soil) stabilization. Subgrade soils containing significant amounts of organic material (greater than about 1%) or sulfates (greater than 0.3%) may require additional lime and/or special construction procedures.

Basic steps of lime stabilization include scarifying or partially pulverizing soil, spreading lime, adding water and mixing, compacting to specified unit weight, and curing prior to placing the next layer or wearing course. Lime can be used in three major forms: *Dry hydrated lime, dry quick lime, slurry lime.* The type of lime stabilization technique used on a project is based on multiple considerations, such as contractor experience, equipment availability, location of project (rural or urban), and availability of an adequate nearby water source. Table 6.1 shows advantages and disadvantages of these lime stabilization techniques.

While the NLA recommends a PI of 10 or greater in order for lime to be considered as a potential stabilizer, the U.S Army Corps of Engineers recommends a PI of 12 or greater for successful lime stabilization (NLA, 2004; NCHRP, 2008). In general, lime is reported to be a suitable stabilizing agent for AASHTO soil types A-4, A-5, A-6, A-7 and some of A-2-6 and A-2-7 (NCHRP, 2008).

The type of lime (CaO or $Ca(OH)_2$) used in stabilizing subgrade soils must meet purity requirements as describe in AASHTO M 216 (ASTM C 977) or equivalent. As described in the NCHRP Report 20-07, the optimum lime content must be evaluated through strength testing. However, estimating the optimum lime content, the pH test (ASTM D 6276) is conducted to determine the amount of lime needed to achieve the design pH, which is 12.45 at 25^{o}C. The ASTM D 6276 method helps identify the amount of lime necessary to satisfy immediate lime-soil reactions and provide a sufficient quality of Ca to maintain a high residual pH and sustain significant long-term pozzolanic reactions (NCHRP, 2008). While the amount of lime is estimated from pH tests, it is necessary to verify the moisture-density relationship of lime-soil mixture in accordance with AASHTO T 99. From the moisture-density relationship, the optimum moisture content (OMC) and maximum dry density (MDD) values are obtained. Compressive strength tests (ASTM D 5102) are conducted on cylindrical samples (triplicate) prepared using a lime content determined from the pH and at OMC and OMC± 1%. Further, additional mixtures with lime contents of 1 and 2% higher than the lime content identified by the pH test should also be prepared as per ASTM D 5102 to verify the optimum lime content, which may be greater than that identified by the pH test. Test specimens are then stored and cured at 40^{o} C for 7 days (accelerated curing) before they are tested for compressive strength. Besides the accelerated curing, one set of lime soil mixture samples is cured at normal curing condition for 28 days before compression testing. The cured samples are then wrapped with a wet absorptive fabric or geotextile and placed on a porous stone for capillary soak for at least 24 hours. It is cautioned that the water used in soaking should never come in direct contact with the specimen and the water level should be maintained to the top of the porous stone and kept in contact with the fabric wrap. The capillary soaked specimens are then tested for unconfined compressive strength (ASTM D 5102 procedure B) and test results are compared with the suggested minimum requirements as shown in Table 6.2. It can be noted that the suggested compressive strength values vary depending on factors such as expected freeze thaw cycles and overlaying materials (sub-base and base) of the stabilized soils. Further, the volume change from dry to soaked UCS specimens are measured, and a volumetric expansion of up to 2% is acceptable. In the case of soil with high sulfate content, the testing protocol warrants a longer soaking period (more than 7 days), which should continue until swell comes to an end.

Table 6.1 Advantages of Major Lime Treatment Techniques (NLA, 2004)

Stabilization Technique	Advantages	Disadvantages
Dry hydrated lime	• Can be applied more rapidly than slurry. • Can be used for drying clay, but it is not as effective as quicklime.	• Since hydrated lime particles are fine materials, dust can be a problem • Generally unsuitable for populated areas.
Dry quick lime	• Economical because quicklime is a more concentrated form of lime than hydrated lime, containing 20 to 24% more "available" lime oxide content. • About 3% quicklime is equivalent to 4% hydrated lime when conditions allow full hydration of the quicklime with enough moisture. • Greater bulk density requires smaller storage facilities. • The construction season may be extended because the exothermic reaction caused with water and quicklime can warm the soil. • Dry quicklime is excellent for drying wet soils. • Larger particle sizes can reduce dust generation.	• Quicklime requires 32% of its weight in water to convert to hydrated lime and there can be significant additional evaporation loss due to the heat of hydration. • Care must be taken with the use of quicklime to ensure adequate water addition, mellowing, and mixing. • Quicklime may require more mixing than dry hydrated lime or lime slurries because the larger quicklime particles must first react with water to form hydrated lime and then be thoroughly mixed with the soil.
Slurry lime	• Dust free application. • Easier to achieve even distribution.	• Slower application rates. Higher costs due to extra equipment

	• Spreading and sprinkling applications are combined. • Less additional water is required for final mixing.	requirements. • May not be practical with wet soils. • Not practical for drying applications.

Table 6.2 Recommended Compressive Strength (psi) for Lime-Stabilized Soils (NCHRP, 2008)

Anticipated Use of Stabilized layer	Extended soaking for 8 days	Anticipated freeze-thaw cycles		
		3	7	10
Sub-base Material				
Rigid pavements/floorslabs/foundations	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
Base Material				
	130	130	170	200

Cement Stabilization:

Portland cement has been effectively used for stabilizing a wide variety of subgrade soils, including granular materials, silts, and clays (Little et al. 2000). Pozzolanic reactions between the calcium hydroxide released during hydration and soil alumina and soil silica occur in fine-grained clay soils. The reaction is an important aspect of the cement stabilization technique for clayey soils. Since the permeability of cement stabilized soil is greatly reduced, it results in a moisture-resistant soil, which is highly durable (Little et al. 2000). Due to the hydration of Portland cement, which is comprised of calcium-silicates and calcium-aluminates, forms cementitious products. Since the hydration rate of Portland cement is relatively fast, it causes immediate strength gain in stabilized subgrade soils (Little et al. 2000). Thus, a mellowing period is not allowed between mixing (soil, cement, and water) and compaction phases of stabilized soils. In general, cement stabilized soil is compacted before or shortly after initial set, which is usually within about 2 hours. However, extended mellowing periods of up to 4 hours have been used for cement stabilized soils in certain situations (Little et al. 2000).

In general, well graded sandy soils with PI of less than 30 are suitable for cement stabilization as cement can effectively fill the available void space and float the coarse aggregates (NCHRP, 2008). In the case of fine-grained soils (soils with P_{200} greater than 50%) the PI value should be less than 20 and the LL value should be less than 40 in order to ensure proper mixing with cement. Cement is also appropriate to stabilize gravel soils with not more than 45% retained on Sieve No. 4. In accordance with the Federal Highway Administration (FHWY) recommendations, soils with AASHTO classifications A-2 and A-3 are ideal for stabilization with cement, but certainly cement can be successfully used to stabilize A-4 through A-7 soils as well.

Durability and strength tests are two types of commonly performed tests for soil-cement mixture. In accordance with the Portland Cement Association (PCA) protocol, the durability of cement stabilized soils for AASHTO soil types A-1 through A-7 are determined on the basis of maximum weight losses under wet-dry (ASTM D559) and freeze-thaw (ASTM D560) tests. However, many state agencies currently require a minimum UCS (ASTM D1633) instead of the aforementioned durability tests, and the typical minimum UCS strength varies from 200 to 750 psi. As illustrated by Little et al. (2000), major objectives of cement-stabilization of subgrade soils include one or more of the followings: decrease of the PI, increase of the shrinkage limit, decrease of the volume change, decrease of clay/silt-sized particles, and improve of strength values/indices such as the California Bearing Ratio (CBR), and improve of resilient modulus.

In the mix design process of cement stabilized soils, the first step in determining the required amount of cement is to classify the soil in accordance with AASHTO M 145. Based on the estimated amount of cement (Table 6.3), the moisture-density relationship is determined for the cement-soil mixture. It can be noted that the estimated cement contents for different soil types presented in Table 6.3 is based on durability test (ASTM D 559 or ASTM D 560) results. Since the durability test results do not reflect actual field conditions such as freeze-thaw cycles, some state agencies use the UCS test as an alternative of the durability test to determine the nominal cement content. Once the nominal cement content is established, soil specimens are prepared and tested for moisture density relationship in accordance with ASTM D 558. Additional specimens are also prepared at the nominal stabilizer content and ±2% of the nominal cement content. These samples are prepared and cured (4 hours) in accordance with ASTM D 1632 and UCS tests are conducted in accordance with ASTM D 1633. The UCS test results are then compared with American Concrete Institute (ACI) recommended typical compressive strength criteria (Table 6.4) for different soil types.

Table 6.3 Cement Requirements for AASHTO Soil Groups (NCHRP, 2008)

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

Table 6.4 Typical ranges of UCS of cement stabilized soils (NCHRP, 2008)

Soil Type	AASHTO Classification	Soaked Compressive Strength (psi)	
		7 Days	28 Days
Sand and gravelly	A-1, A-2, A-3	300-600	400-1,00
Silly	A-4, A-5	250-500	300-900
Clayey	A-6, A-7	200-400	250-600

Class C Fly Ash (CFA)

Fly ash typically contains at least 70% glassy material with particle sizes varying from 1μm to greater than 1 mm. In accordance with AASHTO M 295, Class C fly ash (CFA) refers to as a self-cementing fly ash, which contains enough available calcium to react with soil in the presence of water. Prior to the determination of CFA-soil interactions, the cementitious properties of CFA should be characterized in accordance with ASTM D 5239-04. As part of the mix design process, the moisture density relationship must be established for each soil type and fly ash content in accordance with ASTM C 593 and ASTM D 1633. Then, test specimens are compacted at different moisture levels below OMC to determine the moisture content that will produce the maximum compressive strength. Before compression testing (ASTM D 1633) test specimens are cured for 7 days at 38^{o}C in accordance with ASTM C 593. The strength requirements specified by the agency should be followed in selecting the CFA soil mix design for field application (NCHRP, 2008).

7 SELECTION OF TEST SITES

The project objective was to have 10 construction sites for testing. The sites were supposed to be distributed over the geographical area of Arkansas and include geotechnical conditions in nine out of ten districts in a single report. However, due to some limitations and reasons beyond the control of the research team (such as the limited number of construction projects during the project period, the limited need for soil stabilization during construction and the hesitance of contractors to implement different stabilization techniques due to construction delay and possible liquidated damage concerns) only three test sites were available for this study. These test sites have been finalized based on the discussion with respective AHTD district construction personnel. Selected sites have been shown in Table 7.1. The geographic boundaries of the AHTD 10 districts are shown in Figure 7.1.

Table 7.1 Selected Test Sites

Project No.	Project Description	District
050260 (TS-1)	Highway 157/Highway 167, White Co.	5
060897 (TS-2)	I-40/Highway 89 Interchange, Lonoke Co.	6
100653 (TS-3)	Monette Bypass – Manila, Mississippi Co.	10

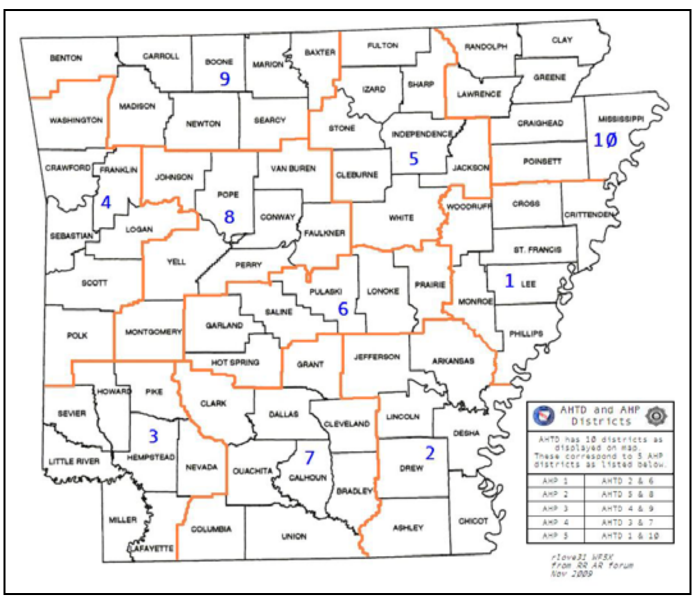

Figure 7.1 Districts of Arkansas

Surficial soil samples were collected from each project site and transported to ASU laboratory for testing. Several tests were performed to aid in the soil identification and subsequent stabilization recommendations.

Project 050260

This project consists of constructing approximately 5.8 miles of the northbound lanes of Highway 167 in White County (District 5). Laboratory test results for samples collected at specific locations of this project are presented in Tables 7.2 and 7.3.

Table 7.2 Laboratory Test Results - Project No 050260

Location	Soil Description	Proctor Results		Atterberg Limits		
		Maximum Dry Unit Weight (pcf)	Optimum Moisture Content (%)	LL (%)	PL (%)	PI (%)
STA 2145+00 CL	Red Clayey Sand	106.4	10.5	22	-	NP
STA 2031+50	Red Clayey Sand	117.5	13.5	24	21	3
STA 2042+50	Red Clayey Sand	105.0	18.5	34	27	7
STA 1972+00 Rt.	Red Clayey Sand	114.0	14.2	26	17	9
STA 2059+50 Rt.	Red Clayey Sand	113.7	13.4	29	21	8
STA 1950+00 Rt.	Red Sandy Clay	117.6	9.20	27	17	10
STA 1946+60 Rt.	Red Clayey Sand	117.6	13.0	26	16	10

Table 7.3 Laboratory Test Results - Project No 050260 cont.

Location	pH Testing		Gradation (% Passing)			AASHTO Classification
	pH	Temperature (°C)	#10	#40	#200	
STA 2145+00 CL	11.97	19.7	85.2	76.8	3.2	A-3
STA 2031+50	7.74	19.9	80.4	60.8	10.0	A-2-4
STA 2042+50	8.94	19.9	71.5	44.7	14.7	A-2-4
STA 1972+00 Rt.	8.06	19.1	96.7	94.0	55.2	A-4
STA 2059+50 Rt.	8.21	19.2	92.8	82.4	49.2	A-4
STA 1950+00 Rt.	7.48	19.5	98.9	97.3	55.4	A-6
STA 1946+60 Rt.	7.93	19.6	94.7	89.6	45.3	A-4

Based on the results, the soil primarily consists of a mixture of sand and clay. Maximum dry unit weights of the samples were in the range of 105 to 118 pcf with a corresponding range of optimum moisture contents of 9 to 19%. The soils were of low plasticity with a PI ranging from NP to 10%.

Project 060897

The project consists of bridge replacement and new interchange at the I-40 / Highway 89 intersection in Lonoke County (District 6). Laboratory test results for samples collected at specific locations of this project are presented in Tables 7.4 and 7.5.

Table 7.4 Laboratory Test Results - Project No 060897

Location	Material	Proctor Results		Atterberg Limits		
		Max Dry Unit Weight (pcf)	Optimum Moisture Content (%)	LL	PL	PI
STA 475+25	Soil	102.6	10.30	34	25	9
STA 475+00	Soil	105.0	8.60	33	23	10

Table 7.5 Laboratory Test Results - Project No 060897 cont.

Location	pH Testing		Gradation (% Passing)			AASHTO Classification
	pH	Temperature (°C)	#10	#40	#200	
STA 475+25	---	---	99.5	97.8	93.4	A-4
STA 475+00	7.14	19.9	99.4	98.1	93.4	A-6

Although the test results above indicate low plasticity soil, the project is known to have highly plastic soil, making it a candidate for lime stabilization as discussed later in this report.

Project 100653

The project consists of widening and improvement of approximately 5.8 miles of Highway 18 from 2 lanes to 5 lines between Monette and Manila in Mississippi County (District 10). The geotechnical report states that the subgrade soil on this project primarily consists of non-plastic to low plasticity clayey sands. The report included recommendation for soil stabilization using approximately 7% cement in case a stable platform is needed to support construction traffic. Laboratory test results for samples collected at specific locations of this project are presented in Tables 7.6 and 7.7.

Table 7.6 Laboratory Test Results - Project No 100653

Location	Material	Proctor Results		Atterberg Limits		
		Max Dry Unit Weight (pcf)	Optimum Moisture Content (%)	LL	PL	PI
STA 815+00	Subgrade	109.2	19.0	28	28	NP
STA 832+50	Subgrade	108.4	16.0	25	25	NP
STA 826+00	Subgrade	103.9	18.7	31	25	6
STA 819+50	Subgrade	105.1	12.6	24	23	1
STA 739+50 Lt.	Subgrade	116.6	11.8	19	20	NP
STA 748+20 Lt.	Subgrade	115.3	12.8	18	19	NP
STA 755+00 Lt.	Subgrade	117.5	12.4	18	19	NP
STA 828+00 Rt.	Subgrade	112.6	9.0	22	20	2
STA 831+50 Rt.	Subgrade	117.1	12.3	19	17	2
STA 834+00	Subgrade	120.6	11.4	21	19	2
STA 742+60	Subgrade	117.2	11.3	I	---	NP
STA 753+00	Subgrade	119.2	10.6	I	---	NP
STA 685+00 Lt.	Subgrade	116.0	17.8	I	---	NP
STA 727+00	Subgrade	108.4	12.1	28	24	4
STA 997+50 Lt.	Subgrade	113.4	14.6	27	21	6
STA 681+00 Lt.	Subgrade	110.6	15.1	25	18	7
STA 991+00	Subgrade	118.2	11.3	42	18	24
STA 732+00	Subgrade	121.0	11.8	I		NP
STA 938+75 Lt.	Subgrade	115.7	13.9	21	15	6
STA 649+10 Rt.	Subgrade	111.2	13.6	24	18	6
STA 693+00 Lt.	Subgrade	117.4	12.0	18	17	1

Table 7.7 Laboratory Test Results - Project No 100653 cont.

Location	pH Testing		Gradation (% Passing)			AASHTO Classification
	pH	Temperature (°C)	#10	#40	#200	
STA 815+00	11.20	22.1	97.5	79.9	26.7	A-2-4
STA 832+50	6.3	21.8	99.7	89.9	42.2	A-4
STA 826+00	6.50	22.1	99.6	89.7	38.1	A-2-4
STA 819+50	6.80	22.2	99.4	86.6	32.1	A-2-4
STA 739+50 Lt.	9.50	21.8	99.1	92.4	40.4	A-4
STA 748+20 Lt.	7.50	21.9	99.0	94.6	41.7	A-4
STA 755+00 Lt.	7.70	21.9	99.0	92.2	41.3	A-4
STA 828+00 Rt.	7.69	18.8	98.8	89.0	24.9	A-2-4
STA 831+50 Rt.	8.06	19.6	99.2	90.9	23.4	A-2-4
STA 834+00	8.12	19.9	99.8	95.5	40.7	A-4
STA 742+60	8.19	19.8	98.2	94.3	40.5	A-4
STA 753+00	8.09	20.1	97.1	90.7	42.2	A-4
STA 685+00 Lt.	7.38	19.4	99.1	95.2	44.9	A-4
STA 727+00	7.75	19.7	98.9	94.1	53.0	A-4
STA 997+50 Lt.	7.32	20.0	97.8	85.8	37.8	A-4
STA 681+00 Lt.	6.77	16.8	98.9	95.1	47.2	A-4
STA 991+00	7.02	16.9	99.2	93.3	42.7	A-7-6
STA 732+00	6.98	17.1	99.7	94.5	44.3	A-4
STA 938+75 Lt.	6.99	21.4	76.2	48.7	3.4	A-1-b
STA 649+10 Rt.	7.67	21.4	76.1	23.1	2.2	A-2-4
STA 693+00 Lt.	7.21	21.4	64.1	41.3	10.1	A-1-b

8 TEST SITE STABILIZATION

Soil stabilization was performed on the three test sites previously mentioned. The need for stabilization was determined by the district. The information was then communicated to the PI either directly or through the AHTD TRC 1308 project coordinator. The PI then reviewed the available soil information and made recommendations for specific stabilizing agent (additive), dosage and depth of treatment. The stabilization plan was then finalized in consultation with the district construction personnel and the contractor. Due to the urgent need for stabilization and the restrictive construction schedule, the process was expedited and there was not sufficient time for the PI to meet with the construction team prior to performing the stabilization. AHTD construction teams monitored and documented the stabilization process. Photos were taken; samples of the used stabilizing agents and soil samples were collected and later transported to ASU's laboratory. Observations were made once soil stabilization was complete and construction equipment began to operate on the stabilized soil.

Classification, compaction (Proctor) and CBR tests were performed on untreated samples from specific locations at each project where stabilization was performed. Then, soil samples were mixed with stabilizing agents in attempt to replicate the stabilized soil in the field. Additional CBR tests were then performed on the treated soil in the laboratory in an attempt to determine the magnitude of improvement in the soil strength due to a specific treatment. The following sections include a summary of the stabilization work performed and laboratory test results for each of the 3 test sites.

Project 050260

Presented in Table 8.1 is a summary of the stabilization work performed for this project. Subgrade stabilization was performed during the period of 8/27/2014 to 9/5/2014.

Table 8.1 Stabilization Summary – Project No. 050260

Station			Stabilization depth, in.	Additive	Dosage (%)	Proctor Unit Wt.	Length (ft.)	Visual Observations
From	To	Side						
2056+27	2044+74	RT	12.00	Cement	6.0%	120.1	1153.0	Satisfactory
2044+64	2029+43	RT	12.00	Cement	6.0%	120.1	1521.0	Satisfactory
2014+49	1997+57	RT	12.00	CFA	8.0%	120.1	1692.0	Marginal
1997+57	1990+00	RT	12.00	Cement	12.0%	120.1	757.0	Satisfactory
1990+00	1978+80	RT	12.00	Cement	12.0%	120.1	1120.0	Satisfactory
1978+50	1959+37	RT	12.00	Cement	6.0%	120.1	1913.0	Satisfactory
2060+15	2056+31	RT	12.00	Cement	6.0%	120.1	384.0	Satisfactory
1950+27	1949+07	RT	12.00	Cement	12.0%	120.1	120.0	Satisfactory
1949+07	1943+30	RT	12.00	Cement	6.0%	120.0	577.0	Satisfactory

Table 8.2 shows results of laboratory tests on the soils samples collected from the field.

Table 8.2 Laboratory Test Results of Stabilized Soils - Project No 050260

Location	CBR of Untreated Soil	Additive	Dosage By wt. (%)	CBR of Treated Soil	Soil Classifications	
					AASHTO	USCS
STA 2145+00	31	Cement	12.00	92	A-3	Fine Sand
STA 2031+50	3	Cement	6.00	165	A-2-4	Silty and clayey gravel and sand
STA 2042+50	3	Cement	6.00	161	A-2-4	Silty and clayey gravel and sand
STA 1972+00	2	Cement	6.00	83	A-4	Silty Soil
STA 2059+50	17	Cement	6.00	269	A-4	Silty Soil
STA 1950+00	4	Cement	12.00	42	A-6	Clayey Soil
STA 1946+60	6	Cement	6.00	99	A-4	Silty Soil

Figure 8.1 includes a relationship between the cement dosage and percent increase in CBR. As mentioned in Table 8.1, soil stabilization using cement was successful in establishing a stable platform to support construction traffic. Laboratory test results showed significant improvement of CBR values of treated subgrade soils. Improvement of CBR value was not the main target of this study, but this parameter is an indirect measure of the stability of the soil under construction vehicle loads. Higher the CBR values indicate better stability. However, Figure 8.1 does not indicate a certain trend.

The section treated with CFA ash did not perform satisfactorily. While a portion of the CFA stabilized section performed satisfactorily, the remainder of that section required additional stabilization using 2 feet of B-Stone backfill. This may be due to the relatively low application dosage of CFA or other unknown reasons.

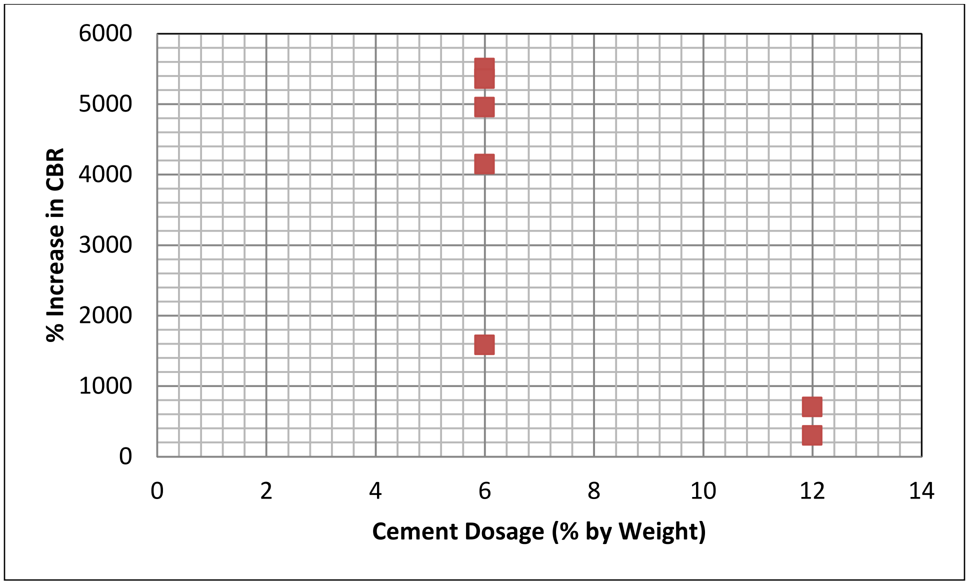

Figure 8.1 Percent Increase in CBR Due to Cement Treatment – Project 050260

Project 060897

Project No 060897 was located at the interchange of I-40 and Highway 89 in Lonoke County, in District 6. Figure 8.2 shows a photo taken during stabilization process. Lime was considered to be the only stabilizing material for this study due to the high plasticity of the in-situ soil. The lime dosage and depth of treatment were determined after a review of the project soil report and in consultation with the project construction team. Lime stabilization was performed during the period of 10/28/2013 to 11/20/2013.

Figure 8.2 Application of Lime Stabilization – Project 060897

Dosages of 4 to 8% lime were applied to the subgrade and mixed with the soil and a sufficient amount of water. The total length of the stabilized road segment was approximately 4500 feet with an average width of 25 ft. Table 8.3 shows a summary of the stabilization performed for this project.

Table 8.3 Stabilization Summary – Project No. 060897

Station			Stabilization depth, in.	Additive	Dosage (%)	Proctor Unit Wt. (pcf)	Length (ft.)	Visual Observations
From	To	Side						
474+50	487+00	CL	12.00	Lime	4.0	116.6	1250.0	Satisfactory
472+50	476+50	CL	12.00	Lime	4.0	116.6	400.0	Satisfactory
474+50	481+70	CL	12.00	Lime	6.0	116.6	550.0	Satisfactory
476+50	482+00	CL	12.00	Lime	5.0	116.6	550.0	Satisfactory
28+00	31+00	CL	12.00	Lime	5.0	116.6	300.0	Satisfactory
495+00	505+00	CL	12.00	Lime	4.0	116.6	1000.0	Satisfactory
475+00	477+00	CL	12.00	Lime	8.0	116.6	200.0	Satisfactory
28+00	30+00	CL	12.00	Lime	8.0	116.6	200.0	Satisfactory

As indicated in Table 8.3, the treated soil performed satisfactorily under the construction traffic with no notable movement observed. Table 8.4 shows results of laboratory tests on the soils samples collected from the field. For the treated soil, the soil classifications shown in Table 8.4 are for the post-treatment conditions.

Table 8.4 Laboratory Test Results of Stabilized Soils - Project No. 060897

Location	Material	CBR of Untreated Soil	Dosage (%)	CBR of Treated Soil	Soil Classifications	
					AASHTO	USCS
STA 475+25	Soil	8	6	20	A-4	ML
STA 475+00	Soil	6	4	17	A-6	CL
STA 478+50	Treated Soil	-	6	16	A-1-b	
STA 481+00	Treated Soil	-	8	20	A-1-a	
STA 480+00	Treated Soil	-	4	7	A-1-a	

Similar to what was observed in Project 0505260, the soil treatment increased the soil CBR values for this project. However, the magnitude of improvement due to lime treatment was significantly less than that measured for the cement treatment. The lime treatment also increased the soil pH. The lime-treated soil had pH values in the range 9.1 to 12.1, which is higher than the pH values of the untreated soil shown in Table 7.5.

Project 100653

Stabilization techniques for Project 100653 utilized lime, cement, and fly ash. Soil stabilization was performed during the period May 2014 to January 2016. Figures 8.3 and 8.4 show mixing and compaction during the stabilization process.

Figure 8.2 Cement Stabilization – Project 100653

Figure 8.4 Fly Ash - Project 100653

Treatment depths for Project 100653 ranged from 12 to 24 inches. Treatment dosages ranged from 3 to 6 percent. Tables 8.5 and 8.6 present stabilization details and observations.

Table 8.5 Stabilization Data - Project No. 100653

Section No.	Station			Avg. width (ft.)	Treatment depth (in.)	Additive	Dosage (%)	Length (ft.)
	From	To	Side					
1	811+00	817+00	RT	15	12.00	Cement	5.0	600.0
2	817+00	823+00	RT	17.5	12.00	Cement	3.0	600.0
3	823+00	829+00	RT	19	12.00	CFA	4.0	600.0
4	829+00	833+75	RT	28	12.00	CFA	6.0	475.0
5	833+75	835+00	RT	70	24.00	Lime	6.0	125.0
6	739+00	742+00	LT	26	24.00	Cement	6.0	300.0
7	742+00	745+75	LT	26	18.00	Cement	6.0	375.0
8	745+75	749+00	LT	26	24.00	Cement	5.0	325.0
9	749+00	753+50	LT	26	18.00	Cement	5.0	450.0
10	753+50	756+50	LT	26	18.00	Cement	4.0	300.0
11	825+50	830+50	RT	22	24.00	Cement	4.0	500.0
12	830+50	833+00	RT	22	18.00	Cement	6.0	250.0
13	833+00	834+85	RT	22	24.00	Cement	6.0	185.0
14	728+00	730+00	Lt.	38	18.00	Cement	4.0	200.0
15	724+25	728+00	Lt.	38	16.00	Cement	5.0	375.0
16	718+25	724+25	Lt.	26-45	18.00	Cement	5.0	600.0
17	675+00	682+50	Lt.	37	16.00	Lime	4.0	750.0
18	682+35	687+50	Lt.	37	16.00	Lime	4.0	515.0

Table 8.6 presents stabilization performance as documented by AHTD construction field personnel. Additional stabilization was required for some sections to bring the subgrade performance under construction traffic to a satisfactory level.

Table 8.6 Stabilization Performance Notes - Project 100653

Section No.	Performance
1	Showed signs of movement, was not 100% stable, but was good enough to support construction equipment and was covered with 3-4 feet of fill
2	Showed signs of movement, was not 100% stable, but was good enough to support construction equipment and was covered with 3-4 feet of fill
3	Showed signs of movement, was not 100% stable, but was good enough to support construction equipment and was covered with 3-4 feet of fill
4	Showed signs of movement, was not 100% stable, but was good enough to support construction equipment and was covered with 3-4 feet of fill
5	Satisfactory performance, no notable movement under construction equipment
6	Satisfactory performance, no notable movement under construction equipment
7	Satisfactory performance, no notable movement under construction equipment
8	Satisfactory performance, no notable movement under construction equipment
9	Satisfactory performance, no notable movement under construction equipment
10	Unsatisfactory performance, had to be re-stabilized with another 4% of Cement. Results were satisfactory
11	Satisfactory performance, did not need additional stabilization after 1.99” of rainfall
12	Satisfactory performance, did not need additional stabilization after 1.72” of rainfall
13	Satisfactory performance, did not need additional stabilization after 1.72” of rainfall
14	Satisfactory performance with little to no movement under construction traffic
15	Satisfactory performance with little to no movement under construction traffic
16	Satisfactory performance with little to no movement under construction traffic
17	Part of this section performed unsatisfactorily due to post-treatment rainfall. Contractor had to re-stabilize using 4.5% LKD to a depth of 18”, satisfactory performance after 2nd treatment
18	Part of this section performed unsatisfactorily due to post-treatment rainfall. Contractor had to re-stabilize using 4.5% LKD to a depth of 18”, satisfactory performance after 2nd treatment

Untreated soil samples were collected to test in the laboratory for subsequent testing. A total of 21 samples were collected and tested in the laboratory. Along with soil index parameters and other routine tests, CBR test were also performed to assess the change in subgrade strength due to specific dosage application. The test results are presented in Table 8.7.

Table 8.7 Laboratory Test Results of Stabilized Soils - Project No. 100653

Location (Station / offset)	CBR of Untreated Soil	Additive	Dosage (%)	CBR of Treated Soil	Soil Classifications	
					AASHTO	USCS
649+10 Rt.	1	---	---	---	A-2-5	Silty or clayey gravel and sand
681+00 Lt	2	Fly Ash	6.0	70	A-4	Silty Soil
685+00 Lt.	2	Fly Ash	6.0	18	A-4	Silty Soil
691+00	4	---	---	---	A-7-6	Clayey Soil
693+00 Lt.	2	Lime	5.0	9	A-1-b	gravel and sand
697+50 Lt	1	---	---	---	A-4	Silty Soil
727+00	6	---	---	---	A-4	Silty Soil
732+00	3	---	---	---	A-4	Silty Soil
739+50 Lt.	1	Cement	6.0	136	A-4	Silty Soil
742+60	7	Cement	6.0	142	A-4	Silty Soil
748+20 Lt.	1	Cement	5.0	58	A-4	Silty Soil
753+00	3	Cement	5.0	47	A-4	Silty Soil
755+00 Lt.	2	Cement	4.0	72	A-4	Silty Soil
815+00	2	Cement	5.0	27	A-2-4	Silty and Clayey Gravel and Sand
819+50 / 50 ft. Rt.	3	Cement	3.0	36	A-2-4	Silty and Clayey Gravel and Sand
826+00 / 50 ft. Rt.	1	Fly Ash	4.0	21	A-4	Silty Soil
828+00 / 25 ft. Rt.	9	Fly Ash	4.0	167	A-2-4	Silty and Clayey Gravel and Sand
831+50 / 20 ft. Rt.	4	Fly Ash	6.0	145	A-2-4	Silty and Clayey Gravel and Sand
832+50 / 30 ft. Rt.	2	Fly Ash	6.0	39	A-4	Silty Soil
834+00	6	Lime	6.0	52	A-4	Silty Soil
938+75 Lt.	2	Cement	5.0	2	A-1-b	gravel and sand

Figure 8.5 includes relationships between the additive dosage and percent increase in CBR. Laboratory test results showed significant improvement of CBR values of cement-treated subgrade soils. Fly ash treatment showed a similar trend, but the percent increase in CBR was relatively lower than those when cement was used. Lime treatment resulted in a much lower CBR improvement, although there was only one treated sample, but data presented in table 8.4 (Project 060897) showed similar increase to the single sample tested from Project 100652. Again, improvement of CBR value was not the main target of this study, but this parameter is an

indirect measure of the stability of the soil under construction vehicle loads. Again, as indicated by Figure 8.1, Figure 8.5 does not indicate a certain trend.

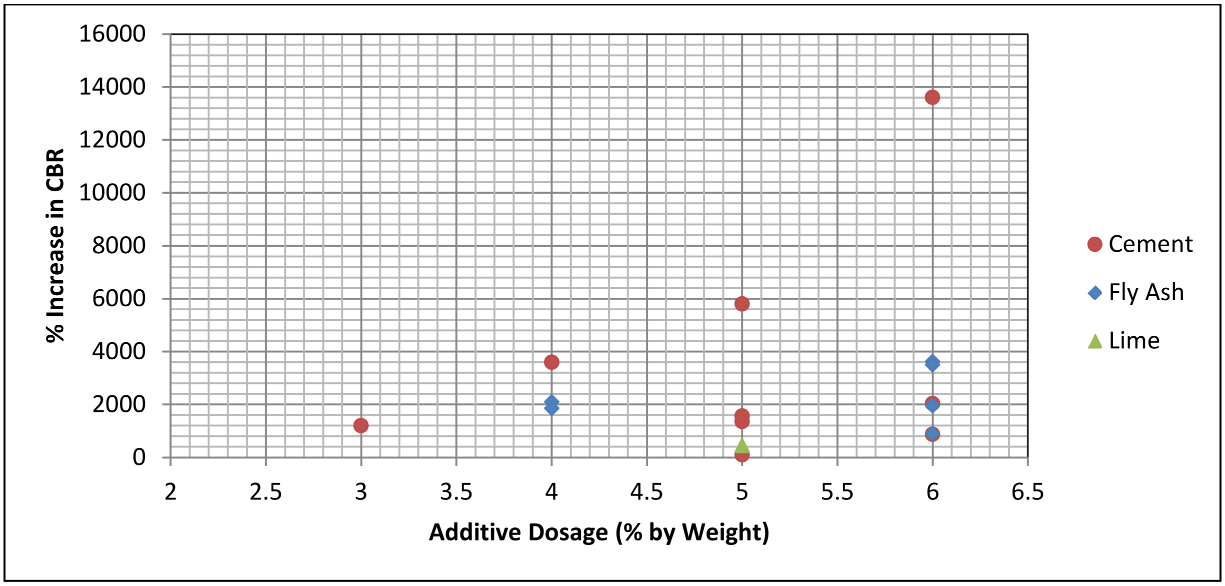

Figure 8.5 Percent Increase in CBR Due to Treatment – Project 100653

9 SITE MONITORING

Selected three test sites were stabilized using cement, lime, and fly ash with recommended dosage as described in the Chapter 8. Both untreated and treated soil samples were tested in the laboratory for all the sites. Based on the CBR test results, it appears the stabilization methods utilized in this project resulted in improvement of soil strength, with the improvement being substantial in some cases.

Several levels of site observations were made. This includes: immediate visual observations after stabilization processes were complete by assessing the subgrade deflection under the wheels of construction equipment; visual observations of the road/subgrade surface months after stabilization processes were completed, referred to herein as post-stabilization visual observations, and evaluation of roughness/rutting (pavement distress) based on measurements made by AHTD Pavement Assessment Management. These observations are described in the following sections.

Immediate Visual Observations

There observations were made and recorded by the AHTD field construction staff. The observations were presented in the previous chapter but included herein for convenience.

Project 050260

Table 9.1 Immediate Observation Summary – Project No. 050260

Station			Stabilization depth, in	Additive	Dosage (%)	Length (ft.)	Visual Observations
From	To	Side					
2056+27	2044+74	RT	12.00	Cement	6.0%	1153.0	Satisfactory
2044+64	2029+43	RT	12.00	Cement	6.0%	1521.0	Satisfactory
2014+49	1997+57	RT	12.00	CFA	8.0%	1692.0	Marginal
1997+57	1990+00	RT	12.00	Cement	12.0%	757.0	Satisfactory
1990+00	1978+80	RT	12.00	Cement	12.0%	1120.0	Satisfactory
1978+50	1959+37	RT	12.00	Cement	6.0%	1913.0	Satisfactory
2060+15	2056+31	RT	12.00	Cement	6.0%	384.0	Satisfactory
1950+27	1949+07	RT	12.00	Cement	12.0%	120.0	Satisfactory
1949+07	1943+30	RT	12.00	Cement	6.0%	577.0	Satisfactory

No notable movement was reported except from Station 2014+49 to Station 1997+57, as the section from Station 2005+07 to Station 2005+32 was not stable and required 2 feet of “B” stone backfill**.**

Project 060897

As indicated by Table 9.2, stabilization efforts for this project were satisfactory; no notable deflection was observed under construction traffic.

Table 9.2 Immediate Observation Summary – Project No. 060897

Station			Stabilization depth, in.	Additive	Dosage (%)	Length (ft.)	Visual Observations
From	To	Side					
474+50	487+00	CL	12.00	Lime	4.0	1250.0	Satisfactory
472+50	476+50	CL	12.00	Lime	4.0	400.0	Satisfactory
474+50	481+70	CL	12.00	Lime	6.0	550.0	Satisfactory
476+50	482+00	CL	12.00	Lime	5.0	550.0	Satisfactory
28+00	31+00	CL	12.00	Lime	5.0	300.0	Satisfactory
495+00	505+00	CL	12.00	Lime	4.0	1000.0	Satisfactory
475+00	477+00	CL	12.00	Lime	8.0	200.0	Satisfactory
28+00	30+00	CL	12.00	Lime	8.0	200.0	Satisfactory

Project 100653

As shown in Tables 9.3 and 9.4, sections with stabilized depth of 12 inches did not perform as well in comparison with other sections, but their performance was sufficient for construction to proceed. The reason(s) for the unsatisfactory performance of Sections 10, 17 and 18 is unknown; it may be due to field mixing or curing procedures.

Table 9.3 Stabilization Data - Project No. 100653

Section No.	Station			Treatment depth (in.)	Additive	Dosage (%)	Length (ft.)
	From	To	Side				
1	811+00	817+00	RT	12.00	Cement	5.0	600.0
2	817+00	823+00	RT	12.00	Cement	3.0	600.0
3	823+00	829+00	RT	12.00	CFA	4.0	600.0
4	829+00	833+75	RT	12.00	CFA	6.0	475.0
5	833+75	835+00	RT	24.00	Lime	6.0	125.0
6	739+00	742+00	LT	24.00	Cement	6.0	300.0
7	742+00	745+75	LT	18.00	Cement	6.0	375.0
8	745+75	749+00	LT	24.00	Cement	5.0	325.0
9	749+00	753+50	LT	18.00	Cement	5.0	450.0
10	753+50	756+50	LT	18.00	Cement	4.0	300.0
11	825+50	830+50	RT	24.00	Cement	4.0	500.0
12	830+50	833+00	RT	18.00	Cement	6.0	250.0
13	833+00	834+85	RT	24.00	Cement	6.0	185.0
14	728+00	730+00	Lt.	18.00	Cement	4.0	200.0
15	724+25	728+00	Lt.	16.00	Cement	5.0	375.0
16	718+25	724+25	Lt.	18.00	Cement	5.0	600.0
17	675+00	682+50	Lt.	16.00	Lime	4.0	750.0
18	682+35	687+50	Lt.	16.00	Lime	4.0	515.0

Table 9.4 Immediate Observation Summary - Project 100653

Section No.	Performance
1	Showed signs of movement, but was good enough to support construction equipment and was covered with 3-4 feet of fill
2	Showed signs of movement, but was good enough to support construction equipment and was covered with 3-4 feet of fill
3	Showed signs of movement, but was good enough to support construction equipment and was covered with 3-4 feet of fill
4	Showed signs of movement, but was good enough to support construction equipment and was covered with 3-4 feet of fill
5	Satisfactory performance, no notable movement under construction equipment
6	Satisfactory performance, no notable movement under construction equipment
7	Satisfactory performance, no notable movement under construction equipment
8	Satisfactory performance, no notable movement under construction equipment
9	Satisfactory performance, no notable movement under construction equipment
10	Unsatisfactory performance, section had to be re-stabilized with another 4% of Cement. Results were satisfactory
11	Satisfactory performance, did not need additional stabilization after 1.99” of rainfall
12	Satisfactory performance, did not need additional stabilization after 1.72” of rainfall
13	Satisfactory performance, did not need additional stabilization after 1.72” of rainfall
14	Satisfactory performance with little to no movement under construction traffic
15	Satisfactory performance with little to no movement under construction traffic
16	Satisfactory performance with little to no movement under construction traffic
17	Part of this section performed unsatisfactorily due to post-treatment rainfall. Contractor had to re-stabilize using 4.5% LKD to a depth of 18”, satisfactory performance after 2nd treatment
18	Part of this section performed unsatisfactorily due to post-treatment rainfall. Contractor had to re-stabilize using 4.5% LKD to a depth of 18”, satisfactory performance after 2nd treatment

Post-Stabilization Visual Observations

The ASU research staff visited the construction sites and made visual observations of the stabilization area. The site visits were made in March, 2016. Notes were made and photos were taken and are included in Appendix A. The following sections discuss the post-stabilization observations.

Project 050260

Stabilization was performed in August and September 2014, and the road was open to traffic in August, 2015. Post-stabilization observations were made in March, 2016. Photos of the stabilized areas are included in Appendix A.1. Based on the observations, it appeared the roadway surface at the stabilization areas were in good conditions at the time of the visit.

Project 060897

Stabilization was performed in October and November, 2013 and the facility was open for traffic in the fall of 2014. Post-stabilization observations were made in March, 2016. Photos of the stabilized areas are included in Appendix A.2. Based on the observations, it appeared the roadway surface at the stabilization areas were in good conditions at the time of the visit.

Project 100653

Stabilization started in June, 2014 and was intermittently performed on as-needed basis until December, 2015. The project was still under construction but completed sections were open to traffic. Post Stabilization observations were made in March, 2016. Photos of the stabilized areas are included in Appendix A.3.

Project 100653 was utilized intensively for this study in comparison with the other two projects and all stabilization additives (Portland cement, Lime and CFA) were used at specific sections. Based on the observations, it appeared the roadway surface at the stabilization areas were in good conditions at the time of the visit.

Pavement Distress (Roughness and Rutting) Evaluation

Pavement surface roughness is a major concern related to the quality of driving. Pavement roughness causes an increase in vertical stress received by pavement and exacerbation of pavement fatigue which makes the value of International Roughness Index (IRI) higher, resulting in severely deteriorated pavement (Lin et al. 2003). Furthermore, a higher pavement roughness index indicates pavement surface deformation, which may affect pavement drainage, drive safety, etc.

The IRI parameter is the extensively used quantifiable measure of roughness used by the AHTD. Moreover, rutting is the accumulation of permanent deformation resulted in distorted pavement layers, is also used by the AHTD to quantify a pavement's distress condition. Thus, the AHTD uses IRI and rutting data as the bases of rating pavements. Threshold values (Table 9.5) for pavement distress presented herein are based on the AHTD guidelines.

AHTD uses an Automated Road Analyzer (ARAN) to collect the field data of Arkansas highway network, which specifically collects IRI and rutting data. The Pavement Management Section of the AHTD is responsible for collecting, processing, analyzing and reporting of these pavement performance data on all the routes on state highway system.

Table 9.5 Roughness and rutting scoring for the state of Arkansas

	Scoring	Rating
IRI – Asphalt	000 – 060	Very Good
	060 – 095	Good
	095 – 170	Fair
	> 170	Poor
Rutting	**Scoring**	**Rating**
	0.000 – 0.125	Excellent
	0.125 – 0.350	Good
	0.350 – 0.500	Fair
	>0.500	Poor

The collected raw data is delivered to the AHTD's computer servers where these data are processed with various analytical software and tools such as, Geographic Information System (GIS). These data are then sent to Department's pavement management system to evaluate the overall pavement performance in order to determine the best and the most cost effective method of maintaining the system. In this study, IRI and rut depth data have been collected in order to study the distress of stabilized roadway sections. In addition to the IRI and rut data, AHTD also provided traffic information for each of the three projects, in terms of Average Daily Traffic (ADT).

The pavement distress for each of the three projects is discussed in the following sections.

Project 050260

The 2015 ATD data for the relevant sections of this project is provided in Table 9.6. The average IR and rut values are provided in Table 9.7 and plotted in Figure 9.1.

For this project, the average IRI value was 78.26 inch/mile, with a standard deviation of 19.21 inch/mile. The IRI value is greater than the 60 inch/mile limit. Therefore, based on Table 9.4, the roughness rating of the Project No. 050260 fits the "Good" category, with the exception of the section between LM 7.5 and 7.8.

Similarly, the average rut value for Project 050260 was 0.254 inch, with a standard deviation of 0.031 inch. Therefore, the rating is in the "Good" category.

Table 9.6 Traffic (ADT) Data - Project No. 050260

STATION	ATR	2015 ADT	STATION	ATR	2015 ADT
730045	V	12000	730066	MC	19000
730194	R	5200	730131	V	1500
730307	R	5100	730190	R	680
730082	V	14000	730196	R	670
730083	V	11000	730332	R	270
730067	MC	20000	730321	R	270
730045	C	12000	730326	R	840
730130	V	6300	730327	R	850
730129	V	7600	730105	V	1500
730441	V	1300	730077	V	980
730314	R	570	730127	V	7100
730320	R	680	730128	V	6400
730442	V	1500	730104	D	460
730315	R	200	730299	V	190
730309	R	100	2152+32		20,000

Table 9.7 Average IRI and Rutting Value for Project No. 050260

Road ID	Begin Log	End Log	Approximate Stations	Avg. IRI	Avg. Processed Rut	MTD	Collection Date
73x67x13xA	4.2	4.3		65	0.25	0.66	11/9/2015
73x67x13xA	4.3	4.4		74	0.2	0.355	11/9/2015
73x67x13xA	4.4	4.5		68	0.26	0.474	11/9/2015
73x67x13xA	4.5	4.6		63	0.28	0.422	11/9/2015
73x67x13xA	4.6	4.7		67	0.27	0.572	11/9/2015
73x67x13xA	4.7	4.8		81	0.23	0.743	11/9/2015
73x67x13xA	4.8	4.9		83	0.19	0.288	11/9/2015
73x67x13xA	4.9	5		81	0.2	0.792	11/9/2015
73x67x13xA	5	5.1		77	0.18	0.885	11/9/2015
73x67x13xA	5.1	5.2		68	0.23	0.746	11/9/2015
73x67x13xA	5.2	5.3		63	0.25	0.797	11/9/2015
73x67x13xA	5.3	5.4	1943+30 to 1949+07	73	0.23	0.675	11/9/2015
73x67x13xA	5.4	5.5	1949+07 to 1959+15	59	0.27	0.58	11/9/2015
73x67x13xA	5.5	5.6	1959+37 to 1978+50	63	0.28	0.748	11/9/2015
73x67x13xA	5.6	5.7		55	0.25	0.643	11/9/2015
73x67x13xA	5.7	5.8		63	0.23	0.652	11/9/2015
73x67x13xA	5.8	5.9	1978+80 to 1990+00	77	0.24	0.746	11/9/2015
73x67x13xA	5.9	6		72	0.26	0.5	11/9/2015
73x67x13xA	6	6.1	1990+00 to	58	0.24	0.665	11/9/2015

73x67x13xA	6.1	6.2	1997+57	79	0.28	0.643	11/9/2015
73x67x13xA	6.2	6.3	1997+57 to 2014+49	93	0.23	0.592	11/9/2015
73x67x13xA	6.3	6.4		87	0.22	0.758	11/9/2015
73x67x13xA	6.4	6.5		83	0.22	0.665	11/9/2015
73x67x13xA	6.5	6.6	2029+43 to 2044+64	71	0.27	0.736	11/9/2015
73x67x13xA	6.6	6.7		79	0.24	0.673	11/9/2015
73x67x13xA	6.7	6.8		74	0.26	0.722	11/9/2015
73x67x13xA	6.8	6.9	2044+74 to 2056+27	69	0.29	0.649	11/9/2015
73x67x13xA	6.9	7		56	0.29	0.494	11/9/2015
73x67x13xA	7	7.1		78	0.24	0.604	11/9/2015
73x67x13xA	7.1	7.2		87	0.29	0.667	11/9/2015
73x67x13xA	7.2	7.3		70	0.32	0.519	11/9/2015
73x67x13xA	7.3	7.4		74	0.31	0.459	11/9/2015
73x67x13xA	7.4	7.5		82	0.29	0.6	11/9/2015
73x67x13xA	7.5	7.6		128	0.3	0.561	11/9/2015
73x67x13xA	7.6	7.7		96	0.31	0.59	11/9/2015
73x67x13xA	7.7	7.8		105	0.23	0.628	11/9/2015
73x67x13xA	7.8	7.9		87	0.24	0.532	11/9/2015
73x67x13xA	7.9	8		82	0.26	0.676	11/9/2015
73x67x13xA	8	8.1		69	0.27	0.477	11/9/2015
73x67x13xA	8.1	8.2		93	0.29	0.41	11/9/2015
73x67x13xA	8.2	8.3		64	0.28	0.519	11/9/2015
73x67x13xA	8.3	8.4		75	0.24	0.487	11/9/2015
73x67x13xA	8.4	8.5		69	0.23	0.545	11/9/2015
73x67x13xA	8.5	8.6		91	0.22	0.736	11/9/2015
73x67x13xA	8.6	8.7		68	0.24	0.496	11/9/2015
73x67x13xA	8.7	8.8		65	0.26	0.618	11/9/2015
73x67x13xA	8.8	8.9		72	0.27	0.437	11/9/2015
73x67x13xA	8.9	9		94	0.27	0.648	11/9/2015
73x67x13xA	9	9.1		90	0.23	0.792	11/9/2015
73x67x13xA	9.1	9.2		173	0.25	0.543	11/9/2015

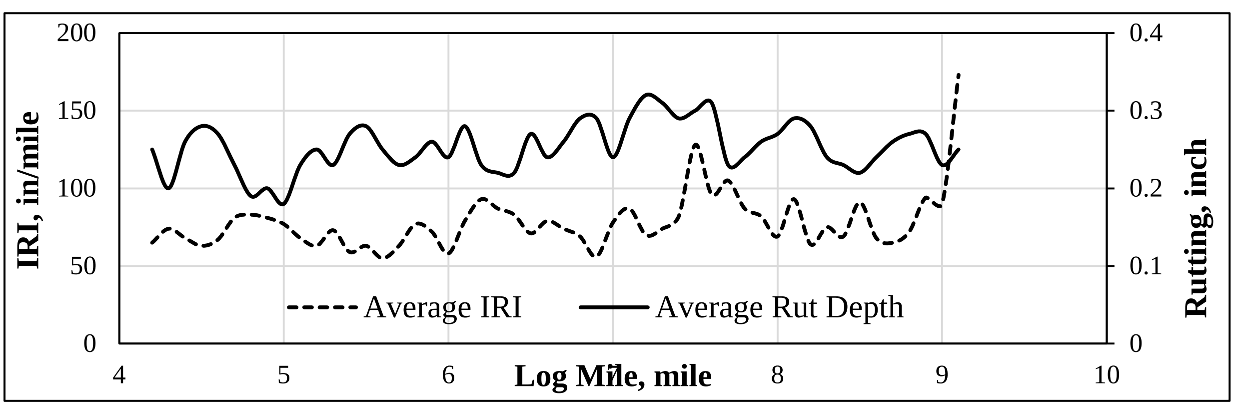

Figure 9.1 Average IRI and Rutting Value for Project No. 050260

Project 060897

The 2015 ATD data for the relevant sections of this project is provided in Table 9.8. The average IR and rut values are provided in Table 9.9 and plotted in Figure 9.2.

Table 9.8 Traffic (ADT) Data - Project No. 060897

STATION	ATR	2015 ADT
430136	R	3100
430132	R	2800
430218	V	7200
430135	R	1500
430137	R	1400
458+67.56		35,000
475+00		380
515+69.85		420
480+00		250

Table 9.9 Average IRI and Rutting Value for Project No. 060897

Road ID	Begin Log	End Log	Approximate Stations	Avg. IRI	Avg. Processed Rut	MTD	Collection Date
43x40x41xA	172.5	172.6	475+00 to 477+00	52	0.32	0.698	6/9/2015
43x40x41xA	172.6	172.7		51	0.39	0.596	6/9/2015
43x40x41xA	172.7	172.8	28+00 to 30+00	54	0.27	0.615	6/9/2015
43x40x41xA	172.8	172.9	495+00 to 505+00	57	0.28	0.671	6/9/2015
43x40x41xA	172.9	173	474+50 to 487+00	60	0.32	0.762	6/9/2015
43x40x41xA	173	173.1		61	0.35	0.607	6/9/2015
43x40x41xA	173.1	173.2		56	0.4	0.676	6/9/2015
43x40x41xA	173.2	173.3		43	0.34	0.604	6/9/2015
43x40x41xA	173.3	173.4		57	0.24	0.633	6/9/2015
43x40x41xA	173.4	173.5		45	0.24	0.685	6/9/2015
43x40x41xA	173.5	173.6		73	0.34	0.64	6/9/2015

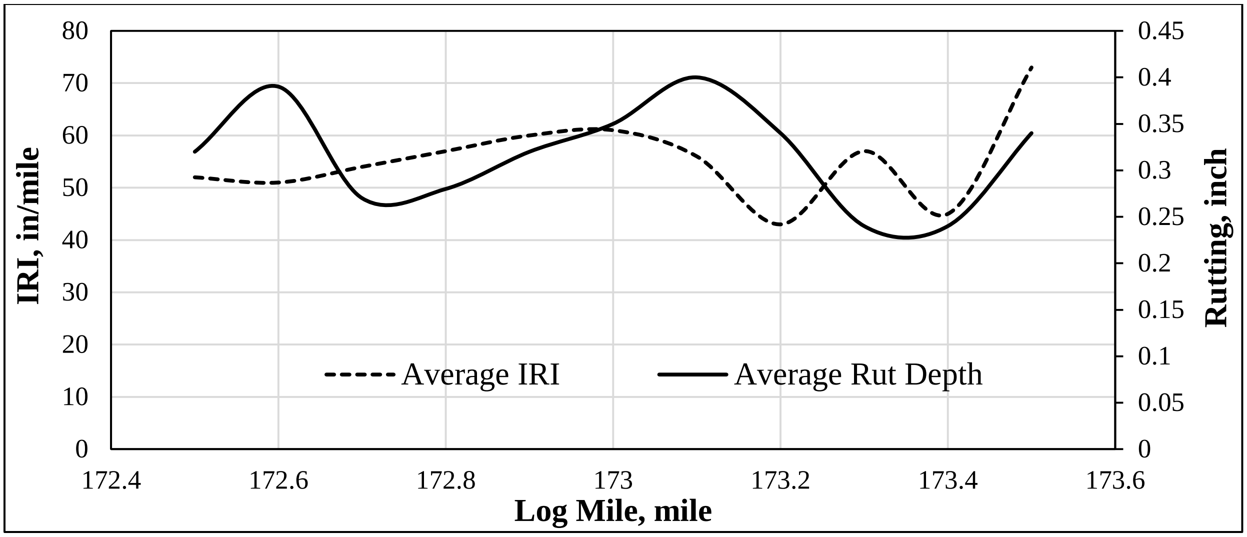

Figure 9.2 Average IRI and Rutting Value for Project No. 060897

For this project, the average IRI value was 55.36 inch/mile, with a standard deviation of 8.14 inch/mile. The IRI value is less than the 60 inch/mile limit. Therefore, based on Table 9.4, the roughness rating of the Project No. 060897 fits the “Very Good” category.

The average rut value for Project 060897 was 0.317 inch, with a standard deviation of 0.054 inch. Therefore, the rating is in the “Good” category.

Project 100653

The 2015 ATD data for the relevant sections of this project is provided in Table 9.10. The average IR and rut values are provided in Table 9.11 and plotted in Figure 9.3.

Table 9.10 Traffic (ADT) Data - Project No. 100653

STATION	ATR	2015 ADT
471969/161969	V	6200
160091	V	740
160095	A	7100
640+00		7,100
100+00		6,000

Table 9.11 Average IRI and Rutting Value for Project No. 100653

Road ID	Begin Log	End Log	Avg. IRI	Approximate Stations	Avg. Processed Rut	MTD	Collection Date
16x18x4xA	26.9	27	77		0.47	0.871	7/8/2015
16x18x4xA	27	27.1	86		0.51	0.845	7/8/2015
16x18x4xA	27.1	27.2	81		0.52	0.796	7/8/2015
16x18x4xA	27.2	27.3	72		0.47	0.845	7/8/2015
16x18x4xA	27.3	27.4	69		0.5	0.876	7/8/2015
16x18x4xA	27.4	27.5	95		0.48	0.929	7/8/2015
16x18x4xA	27.5	27.6	87		0.49	0.896	7/8/2015
16x18x4xA	27.6	27.7	131		0.39	0.809	7/8/2015
16x18x4xA	27.7	27.8	127		0.4	0.853	7/8/2015
16x18x4xA	27.8	27.9	81		0.43	0.805	7/8/2015
16x18x4xA	27.9	28	73		0.44	0.856	7/8/2015
16x18x4xA	28	28.1	83		0.45	0.735	7/8/2015
16x18x4xA	28.1	28.2	90		0.51	0.703	7/8/2015
16x18x4xA	28.2	28.3	80		0.52	0.793	7/8/2015
16x18x4xA	28.3	28.4	78	811+00 to 817+00	0.47	0.821	7/8/2015
16x18x4xA	28.4	28.449	72		0.51	0.77	7/8/2015

For this project, the average IRI value was 86.38 inch/mile, with a standard deviation of 18.06 inch/mile. The IRI value is greater than the 60 inch/mile limit. Therefore, based on Table 9.4, the roughness rating of the Project No. 100653 fits the "Good" category.

The average rut value for Project 100653 was 0.47 inch, with a standard deviation of 0.042 inch. Therefore, the rating is in the "Fair" category.

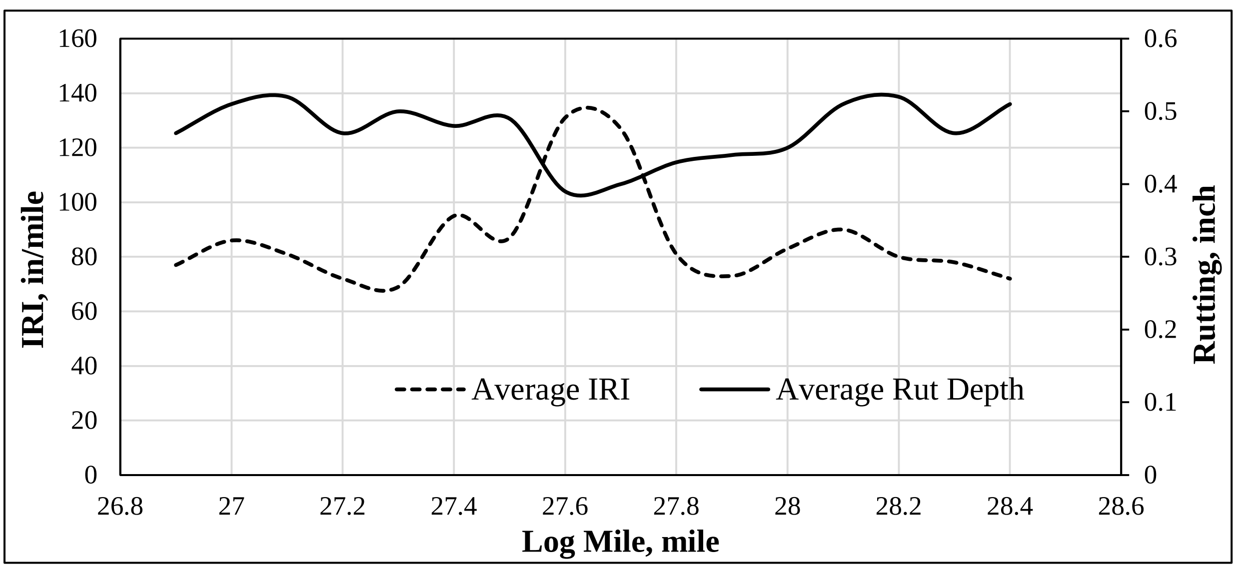

Figure 9.3 Average IRI and Rutting Value for Project No. 100653

10 DATA ANALYSIS

The collected data from the three test sites have been evaluated; the results of the evaluation are presented in this chapter. Based on the evaluation, the following was concluded.

A. Insufficient Test Sites

There is definitely a shortage of data in this study. The original plan was to have 10 test sites over a total project period of 2 years. However, over a study period of 4 years, only 3 sites were available to the research team. The lack of available sites that require subgrade stabilization was a major contributor to this shortage. Also, the unwillingness of contractors to explore with different stabilization techniques that do not have guaranteed results could be another factor. This may be due to the adverse impact on construction schedule, cost, and the potential for liability and liquidated damage issues. The results from TS-3 (Project 100653) were heavily utilized in this study and allowed the research team to explore with 3 different types of additives. However, the overall shortage of data points did not allow for repeatability and consequently, thorough statistical analyses could not be performed. Therefore, the recommendations and guidelines presented in the next chapter will be based on the results of the work performed in this project as well as established guidelines from the literature.

B. Insufficient Geotechnical Information at specific locations

The PI noted the lack of geotechnical information in some situations. For examples, in TS-3, the soil report only included a total of 3 samples taken from Station 649+00 to 809+00; an approximate distance of over 3 miles. Several sections of this stretch needed stabilization. This was not a problem for the research staff as samples were collected and classification tests were performed anyway. However, on future stabilization work, construction personnel will rely on the information included in geotechnical report to select appropriate additives and determine stabilization techniques. Therefore, it is recommended that soil samples be collected at spacing not to exceed 500 feet. Also, in-situ strength tests such as Standard Penetration Testing (SPT) or Cone Penetration Testing (CPT) should be performed to help detect soft zones that may require stabilization.

C. Impact of Treatment Depth

Stabilization utilizing treatment depths of 12 inches were successful in TS-1 (Project 050260) and TS-2 (Project 060897). However, the treatment depths of 12 inches did not result in successful stabilization in the case of TS-3. Based on the results of the limited stabilization work performed on this study, as well as the PI's professional experience with soil stabilization for both industrial and transportation projects, a treatment depth of 12 inches may not yield successful results in most cases. Treatment depth should be determined based on the soft soil type and conditions. In reality, treatment depths are sometimes dictated by the contractor's equipment limitation. Regardless, a recommendation for a minimum treatment depth of 16 inches (preferably 18 inches) will be made in the next chapter.

D. Fly Ash Stabilization Effectiveness

Fly Ash (Class C, or CFA) stabilization was only applied to one section in TS-1 (treatment depth of 12 inches and dosage of 8%) and two sections in TS-3 (treatment depth of 12 inches and dosages of 4% and 6%). All trial sections yielded marginal results. Taking into consideration that only three sections were attempted, as well as the fact that CFA has been documented in the literature as a viable soil stabilization technique, fly ash stabilization of soil should be further considered in Arkansas. In general, based on the information presented in Chapter 3, the AHTD districts have limited experience with soil stabilization using fly ash.

E. Lime Stabilization Effectiveness

Based on the limited soil stabilization performed at TS-2 and TS-3, lime stabilization was highly effective with soil classified as A-6 with relatively high plasticity (at TS-2), which had a treatment depth of 12 inches and dosage of 4 to 8%, but it was not as effective on coarser, less plastic soil (at TS-3), which had a treatment depth of 16 inches and dosage of 4%. This is in agreement with findings documented in the literature and will be taken into consideration while preparing guidelines and recommendations, which will be discussed in the next chapter.

F. Cement Stabilization Effectiveness

Cement stabilization was applied using treatment depths of 12 to 24 inches and dosages in the range of 3 to 12%. A treatment depth of 12 inches resulted in mixed success (successful results in TS-1 and marginal results in TS-3), and a treatment dosage of 3% was not successful, which may be due to the 12-inch treatment depth.

G. Lime / Cement Overlap

Based on the results of the stabilization efforts at TS-1 and TS-3, it appears there is a range of soil classification/plasticity where both cement and lime stabilization can be successful. This is supported by information and guidelines documented in the literature and will be taken into consideration upon making recommendations and preparing guidelines in the next chapter. It is important to note that several AHTD districts have successfully utilized both lime and cement to stabilize soils on numerous projects.

H. Surficial Soil Types in Arkansas

Based on the information presented in Chapter 4, it appears the surficial soil conditions in Arkansas are quite variable, hence it is the PI’s opinion that district-specific stabilization recommendations cannot be accurately established. Therefore, the recommendations presented in the next chapter will be based on the soil conditions of a specific project, not necessarily for a region.

11 CONCLUSIONS AND RECOMMENDATIONS

In this chapter, recommended guidelines for soil stabilization are presented; these guidelines will be based on specific soil conditions of a given site, regardless of the geographic location. The recommendations are made at multiple stages as explained below.

Phase I: Subsurface Exploration

To aid the process of establishing dependable soil stabilization guidelines, it is the PI's professional opinion that the following steps should be included in a given AHTD subsurface exploration program:

a. Perform field strength testing such as Standard Penetration Testing (SPT) at intervals not exceeding 1000 feet. The testing should be performed to a minimum depth of 5 feet below existing grades in fill areas and 5 feet below design finished grades in cut areas.
b. Establish zones that may require stabilization during construction as areas with very loose to loose granular soils (soil classified as A-1 to A-3) or very soft to soft fine-grained soils (soil classified as A-4 to A-7).
c. Insure soil samples are collected from these zones and perform classification tests (grain size analysis and Atterberg limits) in accordance with AASHTO T 88, T 89 and T 90 standards.
d. Perform AASHTO classification in accordance with M-145 standard.
e. If the soft soil is classified as A-4 to A-7, perform sulfate content testing in accordance with AASHTO T 290 standards.

Phase II: During Construction

To achieve successful stabilized platform that can support construction traffic, the following steps should be performed during construction.

a. In cut areas, excavate the soil to design finished grades then identify the soft soil zone requiring stabilization in the field by means of proof rolling the exposed soil. In fill areas, complete the clearing and grubbing then identify the soft soil zone requiring stabilization in the field by means of proof rolling the exposed soil. Proof rolling should be performed using a fully loaded dump truck.
b. Excavate test pits to a minimum depth of 3 feet to determine the thickness of the soft soils. The test pits should be excavated at approximate spacing of 100 feet.
c. If the soft soils are not well defined in the subsurface exploration reports, collect soil samples from the test pits and perform the classification tests listed in Steps c through e of Phase I.
d. If the thickness of the soil requiring stabilization is less than 12 inches, it is unsuitable only due to high moisture content, there is no significant organic contents, and the weather forecast shows there is expected warm, dry weather for a few days (that will depend on the construction season), attempt to stabilize the soft soil by scarifying, allowing the soil to naturally dry then compact the dry soil to the project specifications.

e. If any of the conditions listed in Item d mentioned above are not satisfied, or if the thickness of the soil requiring stabilization is more than 12 inches, then proceed with soil stabilization using the following tables.

Table 11.1 Recommended Percentages of Stabilization Additives (A-1 through A-3 Soils)

Minimum Additive Dosage (Percent Added by Weight of Oven Dry Soil)	**Soil Group Classification (AASHTO M145)**						
	A-1		**A-2**				**A-3**
	A-1-a	**A-1-b**	**A-2-4**	**A-2-5**	**A-2-6**	**A-2-7**	
Portland Cement	3	3	4	4	5	5	5
Class C Fly Ash (CFA) or CFA / Portland Cement Mixture	a	a	a	a	a	a	a
Dry Hydrate Lime*					4[b,c]	4[b,c]	

Table 11.2 Recommended Percentages of Stabilization Additives (A-4 through A-7 Soils)

Minimum Additive Dosage (Percent Added by Weight of Oven Dry Soil)	**Soil Group Classification (AASHTO M145)**						
	A-4[b]	**A-5[b]**		**A-6[b]**		**A-7[b]**	
		LL<50	LL≥50	PI<15	PI≥15	LL<50	LL≥50
Portland Cement	5	6		6			
CFA or CFA/Portland Cement Mixture	a	a	a	a	a	a	a
CFA or CFA/Hydrated Lime Mixture	a	a	a	a	a	a	a
Dry Hydrated Lime*	4	5	6	5	6	6	d

A blank in the table indicates the additive is not recommended.

[a]: Trial mixes should be performed. Perform CBR (or other strength tests as deemed appropriate by AHTD) on pre- and post-treated soil. The recommended minimum increase in strength (e.g. $CBR_{treated}$ / $CBR_{untreated}$) is 300%.

[b]: Sulfate Content (SC) is ≤ 3000 ppm. If SC>3000 ppm, do not use lime; consider stabilization by soil replacement (undercutting and backfilling)

[c]: Percent passing No. 200 sieve is more than 25%

[d]: Trial mixes should be performed using different lime percentages. Target a pH value (as determined by ASTM D 6276 standard) of 12.45 at 25ºC and a LL of less than 40.

*: Reduce quantity by 20% when Quick Lime is used (Dry Quick Lime is not recommended)

In preparation of the guidelines presented in Tables 11.1 and 11.2, it is important to note that experience with soil stabilization developed by the AHTD local District and Construction Engineers should always be taken into consideration upon making a decision to stabilize subgrade soil on a given project. These guidelines are prepared in order to streamline the process, but should not be considered an alternative to experience. Other factors to consider upon making soil stabilization decisions are construction schedule, project contractual constraints and other non-soil related project conditions or constraints, such as equipment availability.

Stabilization procedures, in terms of types of mixing equipment, pre-mixing preparation, additive application methods, mixing methods, mellowing, and curing techniques should be in general accordance with the Portland Cement Association (PCA), National Lime Association (NLA) protocols. Vendor's recommendations should also be taken into consideration for liability reasons.

Short-Term (During Construction) Monitoring

Once proper mellowing and curing time is allowed, proof rolling should be performed using a fully loaded dump truck or the construction equipment that will be operating on the stabilized subgrade, whichever is heavier. Deflection under the tires of the loaded dump truck or construction equipment of less than one inch should be considered adequate if the stabilized soil is to receive at least 5 feet of fill. If the stabilized soil is practically at the final grade elevation, then there should not be practically any deflection under the tires of the proof rolling equipment.

Once stabilized subgrades are covered with pavement structures, construction personnel should periodically observe the performance of these zones under construction equipment. Cracked areas or areas that show excessive settlement should be corrected prior to allowing actual traffic loads to operate on the facility. Observation notes should be made by the AHTD project construction personnel and forwarded to the Research Division for further evaluation.

Long-Term (After Construction) Monitoring

If actual traffic is allowed to operate on specific zones of the project where soil stabilization was performed, construction personnel should continue to visually inspect areas of the project and report areas that can be considered to perform below expectation (excessive roughness, unusual cracks, excessive settlement or noticeable rutting).

Once actual traffic load is allowed to operate on pavement areas for six months, it is recommended that the AHTD Pavement Management team be contacted to conduct pavement surface roughness and rutting evaluation. IRI and rutting data should be collected using an ARAN vehicle. IRI and rutting that does not meet or exceed the "Good" category for each criterion should be immediately reported to the AHTD Research Division. Possible reasons for pavement underperformance should be thoroughly evaluated.

Again, it is important to keep in mind that the purpose of the soil stabilization discussed in this report is to establish a suitable working platform for construction equipment to operate; it is not intended to increase the structural strength of the soil or its contribution to the pavement structure.

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APPENDIX A
POST-STABILIZAATION VISUAL OBSERVATIONS

APPENDIX A.1. POST STABILIZATION VISUAL OBSERVATION PROJECT 050260

Figure A.1 STA 1950 + 00

Figure A.2 STA 1946+00

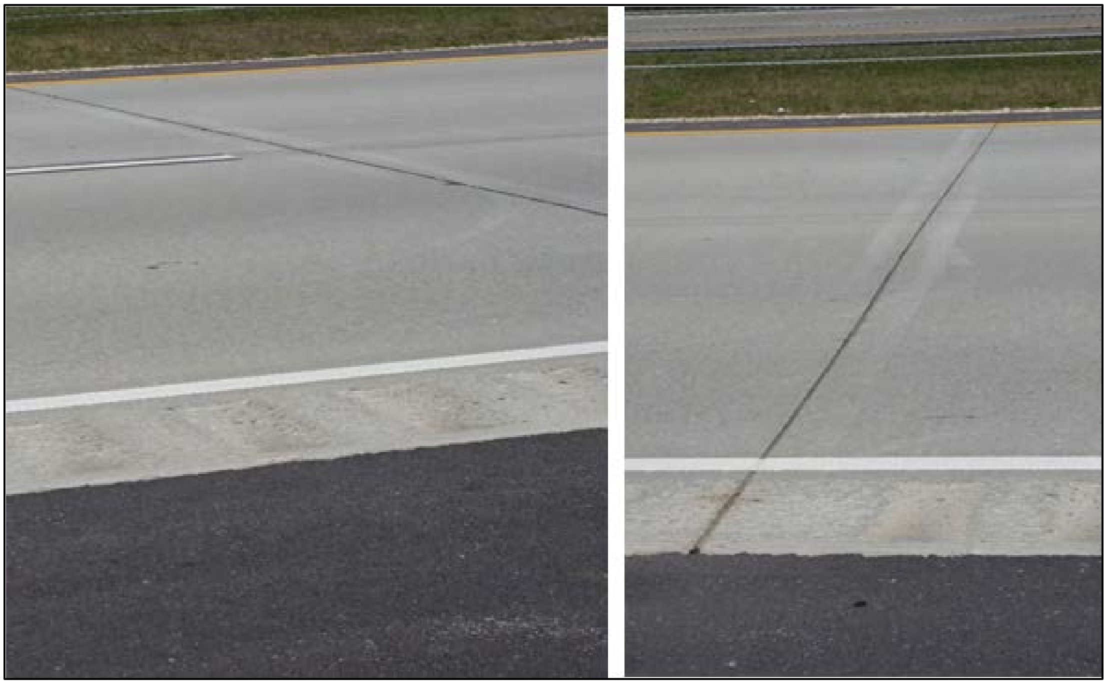

Figure A.3 STA 2042+50

Figure A.4 STA 2059+50

Figure A.5 STA 2007+50

Figure A.6 STA 1972+00

Figure A.7 STA 2145+00

APPENDIX A.2. POST STABILIZATION VISUAL OBSERVATION PROJECT 060897

Figure A.8   STA 475 + 00

Figure A.9   STA 475 + 25

Figure A.10 STA 478 + 50

Figure A.11 STA 480 + 00

Figure A.12 STA 481 + 00

APPENDIX A.3. POST STABILIZATION VISUAL OBSERVATION PROJECT 100653

Figure A.13   STA 815 + 00

Figure A.14   STA 832 + 50

Figure A.15 STA 826+00

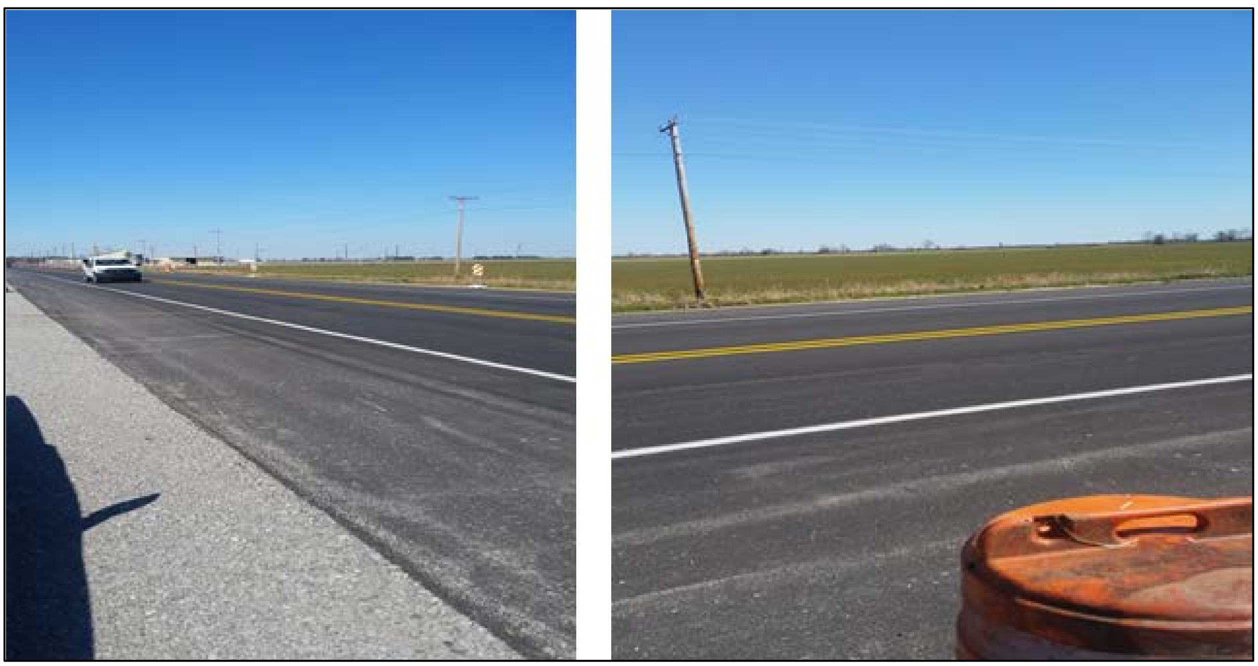

Figure A.16 STA 819 + 50

Figure A.17 STA 739 + 50 LT

Figure A.18 STA 748 + 20 LT

Figure A.19 STA 755 + 00 LT

Figure A.20 STA 828 + 00 RT

Figure A.21 STA 831 + 50 RT

Figure A.22 STA 834+00

Figure A.23 STA 742 + 60

Figure A.24 STA 753+ 00

Figure A.25 STA 685 + 00 LT

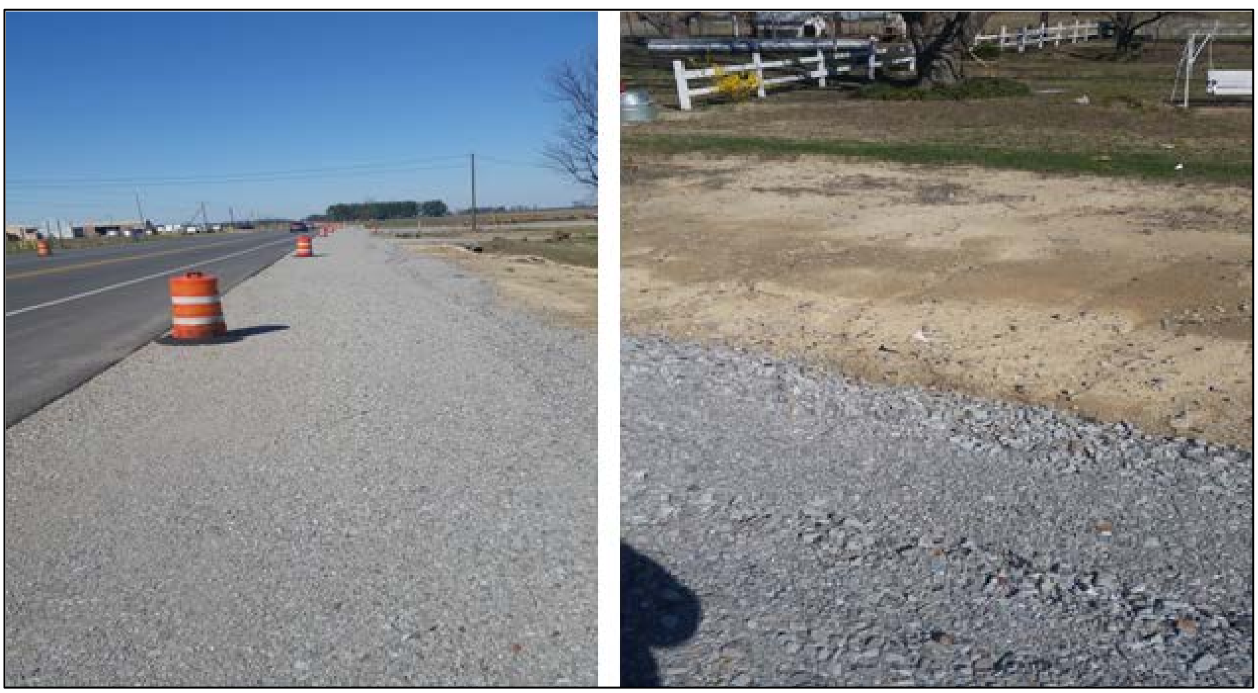

Figure A.26 STA 727 + 00

Figure A.27 Different parts of STA 681+00 LT

Figure A.28 STA 997 + 50 LT

Figure A.29 STA 991 + 00

Figure A.30 STA 732 + 00

Figure A.31 STA 938+75 LT

Figure A.32 STA 649 + 10 RT

Figure A.33 STA 693 + 00 LT