TECHNICAL REPORT STANDARD PAGE

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA/LA.09/468		

4. Title and Subtitle **Accelerated Loading Evaluation of Subbase Layers in Pavement Performance**	5. Report Date 6. Performing Organization Code **LTRC Project Number: 03-2GT** **State Project Number: 736-99-1124**
7. Author(s) Gavin Gautreau, Zhongjie "Doc" Zhang, and Zhong Wu	8. Performing Organization Report No.
9. Performing Organization Name and Address **Louisiana Transportation Research Center** **4101 Gourrier Avenue** **Baton Rouge, LA 70808**	10. Work Unit No. 11. Contract or Grant No.
12. Sponsoring Agency Name and Address **Louisiana Department of Transportation and Development** **P.O. Box 94245** **Baton Rouge, LA 70804-9245**	13. Type of Report and Period Covered Final Report – 4/1/2003 to 6/30/2008 14. Sponsoring Agency Code

15. Supplementary Notes

Conducted in Cooperation with the U.S. Department of Transportation, Federal Highway Administration

16. Abstract

This report documents the research efforts conducted at the Louisiana Transportation Research Center (LTRC) regarding chemical stabilization of the naturally wet and problematic clayey soils typically found as subgrade in south Louisiana and provides detailed information on experiment design, instrumentation, and field and laboratory tests. The objectives of the study included the exploration and development of a methodology to build reliable and conservatively achievable subgrade layers, stabilized with cementitious agents at various field moisture contents so that a treated subgrade layer would not only provide a working table for pavement construction, but could also function as a pavement subbase layer that contributes to the overall pavement structural capacity.

Three additives were studied throughout this research: cement, lime, and lime-fly ash. Testing included moisture density evaluations, various additive percentages, various molding moistures and curing times, tube suction testing, resilient modulus and permanent deformation, Eades and Grim tests, and Accelerated Loading Facility (ALF) tests on similar full-scale pavement sections with cement-stabilized and lime-treated subgrades with the magnitude of the ALF loads kept at 9,750 lb. for the first 200,000 repetitions then increased in incremental intervals of 2,300 lb.

The laboratory and field research confirmed that among subbase treatments evaluated, cement stabilized soil provided the best performance followed by lime-treated soil. Field and laboratory results also indicated that treating clays with lime and silts with cement will create stronger foundations for pavement structure, because when the appropriate additive and amount is added, the treatment modifies the soil to create consistent, drier layers with improved strength and stiffness and reduced moisture sensitivity as compared to the raw natural soil.

A life cycle cost analysis based on the field test results of this study revealed that using a 12-in. cement stabilized soil subbase in lieu of a lime-treated working table layer will create a 37 percent annualized cost savings for low-volume and 31 percent cost savings for high-volume pavement structures in Louisiana.

The primary recommendation emphasized the expanded use of treated subgrade layers with target strengths applied to all subgrades susceptible to moisture intrusion in Louisiana — rather than optional, working table, subgrade treatment. Treatment alternatives should be based on a benefit cost analysis. Additionally, updates to the Standard Specifications are paramount, which foster implementation and increased options of chemical additives to treat wet subgrade soils at competitive costs, while still producing effective subbase and treated subgrade layers.

17. Key Words	18. Distribution Statement
Subgrade Stabilization, Accelerated Loading	**Unrestricted. This document is available through the National Technical Information Service, Springfield, VA 21161.**

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
		160	

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LTRC Project No. 03-2GT
State Project No. 736-99-1124

conducted for

Louisiana Department of Transportation and Development
Louisiana Transportation Research Center

April 2009

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ACKNOWLEDGMENTS

The Louisiana Department of Transportation and Development (LADOTD) financially supported this research through Louisiana Transportation Research Center (LTRC) with State Project Number 736-99-1124 and LTRC Project Number 03-2GT. Aaron Austin, Melba Bounds, and other colleagues in the LTRC Pavement and Geotechnical group conducted the laboratory and fieldwork.

Acknowledgements are extended to the Pavement Research Facility (PRF) group especially William “Bill” King for his assistance during the construction phase; and Dr. Louay Mohammad and the Engineering Materials Characterization and Research Facility (EMCRF) for their assistance with the resilient modulus and permanent deformation testing.

The principal investigators also acknowledge the direction of the Project Review Committee, the project manager of this research, and the help and support from the LTRC’s Pavement Research Facility (PRF) for the field test activities. Student workers from Louisiana State University also played an important role in the processing of laboratory and field test data and in preparation of this report.

IMPLEMENTATION STATEMENT

The purpose of this study is to prove that appropriate stabilization techniques will result in improved performance of roadways built with high silt soils over wet subgrade. If results are favorable, special provisions and design guidelines will be developed for implementing the findings.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

INTRODUCTION

The mighty Mississippi River formed and sculpted most of south Louisiana, creating large areas of alluvial deposits consisting of soft, wet, and unconsolidated soil layers. Many Louisiana pavements were built in these areas of naturally low shear strength and minimum bearing capacity. Louisiana's wet climate (over 50 in. of rain per year), combined with the soft alluvial fine-grained soils, exacerbates the potential for moisture sensitivity and bearing capacity problems. This leads to both construction and performance problems in the long term, often exemplified by detrimental pumping of the wet subgrade under repeated traffic loads. In some cases, initial construction on wet subgrade soils is often difficult because "working table" conditions may not exist naturally.

Since the state has little natural stone or bedrock near the surface, alternatives of remove and replace are often too expensive or impractical. A more rational approach seems to be improving the existing subgrade with products like lime and cement. The modification of wet subgrades using lime is a fundamental practice and widely used among states due to its availability and cost. A study (Lambe, 1990) states, "Their (subgrade) stabilization may result in decreased plasticity, improved volume stability, increased strength/stability, decreased resilient deformation, and increased resistance to detrimental effects of moisture content fluctuations." Data (National Lime Association, 2006) indicate that in addition to drying the soil, lime modification effects include: reduction in soil plasticity, increase in optimum moisture content, decrease in maximum dry density, improvement in compactability, reduction of the soils capacity to swell and shrink, and improvement in strength and stability after compaction.

The Louisiana Department of Transportation and Development (LADOTD) historically provided a lime treatment item to be used at the discretion of project engineers when subgrade-pumping conditions were encountered. The common use of this item was to "dry-out" wet areas regardless of soil type. However, in certain soil types, the lime treatment often provided a temporary working platform for pavement construction but could not prevent further moisture intrusions and the resulting pumping condition of subgrades in wet environments during and after construction. Louisiana Transportation Research Center (LTRC) researchers were requested to evaluate the Department's methods to treat weak subgrade soils and create better methodologies for soil treatment and construction.

As part of the Louisiana solution, researchers at the University of New Orleans (UNO) were contracted by LTRC to conduct a study (McManis, 2003) on the problem. The study concluded that fine-grained soils (including fine sands) containing silt percentages of 50 percent or more, and a Plastic Index (PI) less than or equal to 10 should be considered to have a high pumping potential. Silty soils with more plasticity (PI >10) may pump under specific conditions, but are less susceptible. Although, the soils with high pumping potentials can be compacted to meet specifications and provide a firm load bearing subgrade, they will become unstable with increased moisture and traffic load. Alternate methods for improvement of such soils were also investigated in the UNO study, which evaluated the following stabilization chemicals: lime, lime-fly ash (LFA), Portland cement, and slag cement. All treatments with equal cost were in the comparison. The results indicated that cement or cement/slag treatment of silty subgrade conditions would provide greater performance benefits than lime treatment in wet conditions and greatly enhance strength characteristics necessary for both short- and long-term benefits.

Since the amount and type of cementitious additive required for modification or stabilization is dictated by the ultimate objectives of the processed material, a brief description of the process objectives after Prusinski (Prusinski, et al., 1999) is discussed as follows.

- Soil modification: Enough stabilizer is added to a soil to modify its properties enough to improve soil texture, bearing capacity, and compactability. Strength and durability are normally not criteria at this dosage level. Constructing a stable work platform is the most common use for soil modification. In Louisiana, lime-treated or cement-treated subgrade layers are typical terms used to describe this modification level.
- Soil stabilization: A higher level of stabilizer is chosen to modify properties and to ensure permanence of these properties. Pavement base courses, which assume long-term permanence of the stabilized properties with target values of strength or stiffness, are the most common use of this dosage level. These layers are constructed with strict moisture control, in contrast to subgrade layers being researched, which are constructed with the in-situ moisture conditions. Often the improvements in subgrade are quantitatively taken into account during the pavement design process. Soil cement or cement-treated base courses are the examples in Louisiana for this dosage level. The difference is that the first has a target strength value of 300 psi, while the latter has a target strength value of 150 psi.

Portland cement and lime are both calcium-based products; however, their differences may include important properties such as strength, time dependency on strength development, curing, and durability and performance of the treatment (Prusinski, et al., 1999). Briefly, the basic cement mechanisms involve the calcium silicate phases of the cement, which upon hydration produce both calcium hydroxide (provides available calcium for cation exchange and flocculation and agglomeration) and calcium silicate hydrate (C-S-H) that provides strength and structure in the soil matrix (Prusinski, et al., 1999). LADOTD experience shows sandy soils perform well when treated with cement, and lime has been effective in treating heavy clays. Both products can penetrate and modify a soil's basic properties, transforming the material through a chemical process into a less moisture sensitive material. Between the extremes of sand and clay, there are the silty soils, where a clearer determination has also been made by the UNO study to allow for a stronger, more durable treated subgrade in these wet soil types.

A recent Ohio Department of Transportation (DOT) report (Chou, et al., February 2005) also states, "Therefore, complete chemical stabilization should be considered for all new or reconstruction projects, unless boring or deflection data show very strong subgrade." A second Ohio DOT report (Chou, et al., November 2004) states, "Therefore it is recommended that lime or cement stabilized subgrade be used more systematically and be considered as part of the pavement structure when designing and constructing flexible pavements."

Other states have also moved toward the stronger stabilized structural subgrade layer. Indiana allows several different treatment types based on the application, including chemical soil modification (207.04 Subgrade Treatments). Texas DOT has established procedures to estimate additive percentages and verifies strength and durability through, Texas DOT, TEX-121-E.

The concept of cement treated silt subgrade in wet conditions has been successfully implemented on DOTD projects consistent with the "working table" concept to address the problem discussed. Both laboratory and field-testing results indicated that cement-stabilized silty soils have significantly higher modulus and strength values than lime-treated silty soils. Data (National Lime Association, 2006) also states that lime stabilization can occur in soils with a suitable amount of clay and the proper mineralogy. Questions arise: Can a cementitiously treated soil layer provide not only a working table for pavement construction but also a pavement layer that contributes to the overall structural capacity of the pavement?

A problem with this requirement is that, since most times a treated subgrade layer is called for at locations with wet weak subgrade, the field compaction at the optimum moisture content of subgrade soils is not realistic or even achievable in some cases. In other words, lime- or cement-treated subgrade layers are not the designed products. Considering various soil types and conditions in the field, very little guidance existed or was provided to guarantee the quality of treated subgrade layers so their contribution could be secured. Only after this obstacle is overcome will the attempt to search for methods to quantify the long-term benefits of a treated subgrade be meaningful.

Researchers have stated, "The addition of fly ash and lime in the stabilization of fine-grained soils increases the elastic modulus considerably, due to the formation of cementitious minerals. These minerals form at normal temperatures (field) in laboratory-compacted soils. The formation of the cementitious minerals will not be limited by the type of soil, because all of the constituents necessary to form cementitious compounds are supplied by fly ash and lime. With natural soils, higher strengths and larger changes in elastic moduli may be obtained because of the presence of non-clay constituents like quartz, which provide a mechanically stable matrix for bonding by the cementitious minerals." The authors (Ferrell, 1988) also state, "The increased rigidity of the fly ash-lime-stabilized layer should be considered in pavement design"; and "Smaller quantities of lime and fly ash may be desirable to produce more flexible soil layers." (Ferrell, 1988)

A structurally reliable treated subgrade layer would allow a reduction in thickness of more expensive upper layers (stone, asphalt, etc.). It can be achieved through an improved subgrade resilient modulus or a subgrade layer with a structural coefficient used as a subbase. Choosing a reliable structural contribution that is conservatively achievable is the key. Therefore, there is a need to address these problems to quantify the contributions of lime, LFA, and cement-stabilized subgrade soils for their strength enhancement. More specifically, there is a need to develop a guideline for determining the dosages of these additives in subgrade treatment for a target performance according to the specified function of working table and to decide whether the resulting strengths can be counted on/in the pavement design process.

OBJECTIVE

Objectives included the exploration and development of a methodology to build reliable and conservatively achievable subgrade layers stabilized with cementitious agents at various field moisture contents so that a treated subgrade layer would not only provide a working table for pavement construction but can also function as a pavement subbase layer that contributes to the overall pavement structural capacity. This included developing a guideline for selecting the dosages of chemical agents according to field soil types and moisture contents, including naturally wet subgrades and methods to quantify the resulting improvements.

This experiment also intended to study the properties of pavement layers constructed with such treatments under the Accelerated Loading Facility (ALF) loading to assess the long-term performance in an accelerated manner and to verify the laboratory findings.

SCOPE

The laboratory portion of the study explored the correlation among the moisture content of subgrade soil; the content of cement, lime, or LFA; and the strength of subgrade soil. Three types of soils were evaluated for this purpose; they were a silty clay with a low PI (silt content > 60 percent), a silty clay with a medium PI (10 < PI < 25), and heavy clay (PI > 25). Laboratory tests were conducted on the samples of these three soil types for their physical and strength properties with and without chemical stabilization.

The field portion of the study evaluated only two subbase treatments under accelerated loading conditions. It compared cement-stabilized subbase test sections against test sections with a "working table" lime treatment option. This option does not differentiate between silt or clay soils; it merely utilizes lime as a drying agent. Effects in addition to drying, if any, are not counted in the design. The current lime treatment option does not address additional strengths that may be achieved through full lime stabilization, only the current "dry out" logic as a control section.

METHODOLOGY

Soil compaction is an important operation in road construction, and the unique relationship between the optimum moisture content and maximum dry density as defined by the Proctor compaction test is fundamental and well established. LADOTD currently uses maximum dry density to control the quality of compacted material on its projects. The rationale is that higher densities mean higher strengths, and higher strengths directly relate to good performance.

In the case of subgrade preparation with high in-situ moisture, as discussed previously, densities cannot reach the maximum dry density through compaction as required to support the pavement structure construction. Since waiting for soil to dry is time-consuming and often impractical and remove and replace options are limited due to geology, high groundwater, and the high cost of replacement materials, alternative options must be considered and explored to improve soil engineering properties for support of construction and pavement structures.

As discussed previously, additives like cement, lime, and LFA have long been used to dry, modify, and stabilize soils and increase soil strength with durability. This study will develop methods for determining the appropriate amount of additives and strength targets through various combinations of additives, soil types, and molding moistures. Issues relative to the layer's design and laboratory testing will be investigated. The ultimate goal is to develop a methodology of building a subgrade layer with reliable strength and performance.

To accomplish the project goals, two major test programs were developed consisting of laboratory and field testing. Laboratory testing occurred in the LTRC Geotechnical Laboratory and in the Engineering Materials Characterization Research Facility (EMCRF), which was a factorial study on cement, lime, and LFA stabilization techniques on wet subgrade soils. Field testing occurred at the LTRC Pavement Research Facility in Port Allen, Louisiana, and utilized the ALF, which compared the field performance of cement-stabilized and lime-treated subgrade in a silty soil.

Laboratory Experiment Design

The laboratory test program was divided into three main portions, which cover the properties of soil modification and stabilization using cement, lime, and LFA. These laboratory tests were planned and conducted based on LADOTD's (and others') past experience with treatment and stabilization of soils to simulate the field construction conditions and seek a suitable stabilization scheme at various moisture contents resulting in reliable strength and durability development in stabilized wet subgrade soils. The effectiveness of each stabilization scheme was evaluated from the perspective of strength and/or durability incurred by stabilization. Some samples were prepared and tested in the laboratory; other samples were prepared from material collected from the actual field construction, then molded and tested in the laboratory.

Material

Three main soil types selected for this study were from the Baton Rouge area with low, medium, and high PI of soil. The soils used in this study were fine-grained in size and represent the wet and weak subgrade soils commonly found in many field cases in south Louisiana. These soils (Soil I, Soil II, and Soil III) were used for each different stabilizer.

The stabilization chemicals used for comparison in this study included type I portland cement, class C fly ash, and lime. The chemical constituents of the additives used in this study are listed in Table 1.

Table 1
Chemical constituents of used stabilizers

Composition (%)	Portland Cement[a]	Fly Ash[b]	Lime[c]
SiO_2	22.4	47.5	<2
Al_2O_3	4.1	4.1	0
Fe_2O_3	3.9	5.2	0
CaO	65.1	20.1	0
MgO	1.2	2.5	<5
K_2O	0.2	0.7	0
Na_2O	0.1	0.3	0
$CaCO_3$	0	0	<3
$Ca(OH)_2$	0	0	90

[a] type I Portland cement, [b]class C fly ash, and [c]high calcium hydrate lime.

Test Methods

Laboratory tests included in this study were conducted according to LADOTD testing procedures, AASHTO, and ASTM standards. Conventional tests were conducted to determine basic physical properties included sieve analysis, Atterberg limit tests, and standard Proctor compaction. These tests established baseline criteria for utilized materials. Repeating the tests on modified or stabilized material showed the effects of the additive on material properties.

Unconfined compressive strength (UCS) tests were used to determine baseline strengths of untreated soils and the subsequent improvements attained through modification or stabilization with additives. The UCS tests were conducted according to LADOTD TR 432 (ASTM D1633, Standard Test Methods for Compressive Strength of Molded Soil-Cement Cylinders.) The determination of soil strength is a conventional test related to the ultimate performance of a soil layer. For each additive, strength tests were performed for factorials of various additive content, molding moisture content, and curing time and condition to simulate different field construction conditions.

Other special tests related to soil strength included tube suction (TS), durability, and resilient modulus tests to evaluate the long-term performance of treated and untreated soils. With Louisiana's wet southern climate and low-lying areas, precipitation and groundwater accumulate on and around its pavements. The challenge is to keep moisture off and out Louisiana's constructed pavement layers. The TS tests evaluated moisture susceptibility of soil and aggregate with or without stabilization, which can be affected by wetting-drying cycles. TS tests measure the dielectric value (DV) of soil that is correlated well with the amount of free moisture content in the soil.

This study conducted the tube suction test using a modified version of the Texas Transportation Institute method (http://tti.tamu.edu/documents/5-4114-01-1.pdf) to determine the moisture intrusion properties of a soil. The modified method used a 4-in. sample versus the standard 6-in. sample, since a recent study (Barbu, 2004) indicated that for fine-grained, low plasticity soils, the diameter of a sample could be reduced without affecting the results.

The durability test according to LADOTD TR 432 (similar to ASTM D559 or AASHTO T 135), was conducted in this study evaluate a material's ability to withstand numerous wet and dry cycles. These cycles represent the wet and dry cycles soil experiences in nature.

Resilient modulus and permanent deformation testing procedures were dictated by AASHTO T 307 and were conducted to evaluate a material's ability to withstand numerous load cycles in the laboratory, representing actual traffic loads in the field.

Cement Stabilization

LTRC's past experience indicates that cement is most effective in treating or stabilizing lower PI soils. Therefore, high PI soils were not part of the cement testing program. Soil specimens with low and medium PIs were prepared at various moisture and cement contents.

UCS Test

Test Plan. Table 2 shows the factorial test plan for the cement-stabilized soils. The factorial was designed to see how cement functioned at different (especially high) moistures under different curing methods (cure time and procedure) and if higher cement percentages could compensate for the naturally wet subgrade material to produce design strengths. Therefore, moisture contents ranged from 6 percent below optimum to the wet extreme of too wet to mold to simulate wet field conditions. The cement percentages are all nominal or designed values.

Table 2
Testing factorial for cement

Soil type	Cement, A_n, %	Molding moistures, w_n %	Curing, days
Low PI (Soil I)	4, 8, 12	From at least 6% below the optimum to too wet to mold	7, 28
Medium PI (Soil II)	4, 8, 12		7, 14, 28
High PI (Soil III)	—		—

Sample Molding and Curing. The sample molding procedure for UCS is described as follows: Weighed 2000 g of the soil with hygroscopic moisture content (w_{hy}) and thoroughly mixed it with the amount of cement determined by multiplying the nominal cement content, A_n, given in Table 2 with the 2000 g soil. Therefore, the hygroscopic moisture content, w_{hy}, was based on the weight of the dry soil and the nominal cement content, A_n, was based on the weight of soil with the hygroscopic moisture content, w_{hy}.

Next, the mixture of the soil and cement was added and thoroughly mixed with the amount of water determined by multiplying the nominal moisture content (w_n), also shown in Table 2. Consequently, the nominal moisture content (w_n) is based on the total weight of cement soil mixture with the hygroscopic moisture content (w_{hy}). Specimens were immediately molded using the standard Proctor procedure (DOTD TR 434 or ASTM 698) and moisture content was taken at molding (w_m). Here, the moisture content at molding (w_m) was based on the weight of dry cement soil mixture.

Raw soil specimens (0 percent cement) were wrapped in a plastic bag and cured in laboratory room temperature for 1 day, and cement mixed specimens were wrapped in a plastic bag in a 100 percent humidity room with a temperature of 73°F for 7, 14, or 28 days. At the end of the curing period and before the loading test, specimens were submerged in water for 4 hours with sample weights taken before and after the submergence. Then specimens were loaded according to ASTM 1633 and the moisture contents were taken at/after specimen break (w_b). Here, the moisture content at break (w_b) was also based on the weight of dry hydrated cement soil mixture.

Data Analysis. In the testing data analysis, the molding quality and characteristics of specimens were checked by the comparison of their molding curves against each other. The influence factors on UCS were explored through analyzing various soil physical parameters to have a better understanding on the characteristics of soil cement.

In addition to conventional calculation of soil physical parameters, several special calculation formulas were derived to analyze the testing results in this study as follows.

Hygroscopic Moisture Content, w_{hy}, of Soil Tested. Soil will absorb a certain amount of moisture from the air during its process and storage in the laboratory, which can be described by its hygroscopic moisture content (w_{hy}). This moisture is needed to determine the water cement ratio of cement mixed soil and can be calculated as follows:

$$w_{hy} = \frac{(w_m - w_n)\cdot(100 + A_n)}{10000 + 100\cdot w_n + A_n\cdot(w_n - w_m)} \qquad (1)$$

where, (w_{hy}, w_m, w_n, and A_n) are all defined in percentages.

Because of the hygroscopic moisture content (w_{hy}), the actual cement content (A_w) used in the laboratory test has a relationship with the nominal cement content (A_n) as follows:

$$A_w = A_n \cdot \left(1 + \frac{w_{hy}}{100}\right) \qquad (2)$$

The cement content (A_w) is also as a percentage.

Moisture-Cement Ratios. The moisture-cement ratio at sample molding (C_{wm}/A_w) can be determined as follows:

$$\frac{C_{wm}}{A_w} = \frac{w_n \cdot (100 + A_n) + \dfrac{100 \cdot w_{hy}}{100 + w_{hy}}}{100 \cdot A_n} \tag{3}$$

where, (w_{hy}, w_n, and A_n) are all in percentages as before. Although the moisture-cement ratio at sample break (testing) can also be defined and calculated, only the moisture-cement ratio at sample molding has a practical implication since it can be used as a control criterion during field construction.

The formulas discussed above can improve the quality of data analysis in this study but do not have to be used in practice to calculate the moisture-cement ratio. A simple way, which ignores the hygroscopic moisture content of dry soil tested, is to calculate lab and field moisture cement ratios and can serve the guideline purpose, and the error caused by such practice are well within the variations caused by other factors.

Comparison and Other Tests

Test Plan. The TS and durability tests were planned together with an additional UCS test for Soil Type I. In this way, the testing results from different tests can be compared and analyzed together to explore and understand their possible relationships. Six different cement dosages by the dry weight of the soil were used to stabilize Soil Type I under four different moisture or water contents. Three sets of samples were molded with one set for TS tests; a second set for 7-day UCS tests; and a third set for wetting-drying durability tests. The factorial is summarized in Table 3. Due to the lack of raw soil, not all UCS specimens were prepared and tested. There is no testing factorial investigation for resilient modulus and permanent deformation tests in this study.

Table 3
Testing factorial for TS, UCS, and durability test samples

Planned Molding Moisture Content	Cement Content, %	w, %	Dry Density, pcf	Planned Tests
15.5%	2.5	14.52%	109.5	
	4.5	14.15%	109.5	Wetting-Drying
	6.5	15.70%	109.5	Durability Tests;
	8.5	15.72%	108.9	Tube Suction Tests;
	10.5	15.47%	108.9	7-Day UCS Tests;
	12.5	15.58%	110.1	
18.5%	2.5	16.83%	109.5	
	4.5	19.27%	105.0	Wetting-Drying
	6.5	18.27%	106.3	Durability Tests;
	8.5	18.75%	106.3	Tube Suction Tests;
	10.5	18.06%	106.9	7-Day UCS Tests;
	12.5	18.15%	107.6	
21.5%	2.5	19.93%	102.5	
	4.5	22.13%	99.3	Wetting-Drying
	6.5	21.46%	100.6	Durability Tests;
	8.5	21.82%	100.6	Tube Suction Tests;
	10.5	21.13%	101.9	7-Day UCS Tests;
	12.5	21.03%	102.5	
24.5%	2.5	22.81%	98.0	
	4.5	24.78%	95.9	Wetting-Drying
	6.5	24.79%	96.1	Durability Tests;
	8.5	25.20%	96.1	Tube Suction Tests;
	10.5	24.29%	97.4	7-Day UCS Tests;
	12.5	23.90%	97.4	

Sample Molding and Curing. The samples for UCS and durability tests were molded by following the same Standard Proctor procedure described previously. The samples for TS tests were molded by a similar procedure with predetermined dry densities and moisture content as follows. Measure the required amount of soil and water, or cement, according to predetermined moisture and dry density, cement content, for a mold with of 4 in. diameter and 7 in. height, and mix them thoroughly. Pour the water-cement-soil mixture into the cylindrical mold in four layers after mixing. Compact each layer with predetermined proper blows of a 10-lb. hammer, respectively. Cure raw soil specimens wrapped in a plastic bag at laboratory room temperature for 1 day or cement-stabilized specimens wrapped in a plastic bag in a 100 percent humidity room with a temperature of 73°F for 7 days. Put the cured specimens in an oven with a temperature of 104°F for 7 days until the weight of specimens are almost constant. Put the dried specimens in a pan with about 1 in. water in height and start to take DV readings with a Percometer v.3, as shown in Figure 1, periodically until the readings become constant. Figure 2 shows the specimen molding and Figure 3 shows the TS specimens during testing. The reasons for using plastic tubes were to prevent non-stabilized samples from falling apart during TS tests and to provide some lateral confinement as that in the field. Conduct UCS tests on the specimens after TS testing if possible.

(a) Percometer v.3 **(b) Reading**

Figure 1
Device for TS test

(a) Molding **(b) Molded**

Figure 2
Specimen molding

(a) Un-stabilized

(b) Stabilized

Figure 3
TS testing setup

Data Analysis. The TS test generates the variation of DV values with time (DV curves) for soil with or without stabilization. Therefore, the characteristics of DV curves were evaluated and compared among various specimens tested. In addition, the influence factors on the maximum DV of specimens tested were explored and the possible relationship between maximum DV and UCS was studied for a better understanding the TS test.

Testing data from TS, UCS, and durability tests were studied for their relationships to use one to predict the other two. The common parameter that correlates well with the testing results from all three tests was also explored.

Lime Stabilization

The modification of wet subgrades using lime is a fundamental practice and widely used among states. This research seeks to develop simple methods to determine the lime necessary to not only treat, but also stabilize various Louisiana soil types to create a stronger chemical bond than currently provided under the working table philosophy.

Lime advocates suggest that increasing the percentage of lime from the current "working table" to needed percentages for treatment and on to stabilization in clays are attainable. Additionally, when conditioning clayey soils with lime for cement stabilization (LADOTD Standard Specification Section 304), they suggest if the appropriate (higher) level of lime is added to the soil, stabilization can occur after the lime mixing event, eliminating the need for the second (cement mixing) event.

Currently, LADOTD method TR 416, *Determination of the Percentage of Lime for Treatment of Soils or Soil-aggregate Mixtures*, determines the minimum amount of lime required for treatment as identified by the relationship between the percentage of lime added and reduction of plasticity. When the plasticity index and liquid limit have met the method's requirements (PI ≤ 10, LL ≤ 40), the required lime amount, by weight, is then estimated. For construction purposes, then the percentage by weight is converted to a percent by volume, either by using an additive conversion chart or formula. Complete stabilization of subgrades with lime is not addressed in the current LADOTD specifications or test methods. There are many different methods to determine lime percentages for stabilization used by other states. This study seeks to recommend which method should be adopted by LADOTD.

Moisture-Density and UCS Tests

Test Plan. Table 4 was designed to define the relationship between the appropriate amount of lime and the moisture-density relationship. The factorial in Table 4 was also designed to examine how lime functioned at different moistures under different curing methods and if higher lime percentages could compensate for the naturally wet subgrade material to produce the required design strengths. The lime percentages are all nominal or designed values. The moisture contents ranged from 6 percent below optimum to the wet extreme of too wet to mold to simulate wet field conditions.

Table 4
Testing factorial for lime

Soil type	Lime % by weight	Molding moistures, w_n %	Curing Methods	Curing, days
High PI (Soil III)	0, 5, 18, 26	From at least 6% below the optimum to too wet to mold	100% Room	7, 28
			Oven	7
			Heated Bath	7

Sample Molding and Curing. LADOTD uses percentage by weight in its laboratories. A common translation is that an application of 3 percent hydrated lime by weight is equivalent to 9 percent hydrated lime by volume. Samples were molded using LADOTD TR 434 (ASTM D 698) as a guideline for the lime samples. The samples were formed using a 4-in. diameter mold with a volume of $^{1}/_{30}$ of a cubic-foot. The method is similar to the standard Proctor compaction test in that the material is compacted in three equal lifts each with 25 blows of a 5-lb. hammer with a 12-in. drop. Prepared samples were extruded from their molds and prepared for curing.

Curing requires proper moisture and temperature conditions so that the desired properties can develop and physical changes can occur due to chemical reactions. Design strengths requiring a 28-day cure period can slow construction. Accelerated curing techniques can expedite construction by simulating the 28-day strength (normal conditions) by producing similar results through a 7-day cure. The different accelerated curing methods are detailed below.

One technique wrapped the samples and cured them in a constant temperature heated bath (Blue M. Oven, Model NW-1140A-1) set at 104°F for 7 days. The other technique placed wrapped samples in sealed plastic containers with ½-in. of water at the bottom. The containers were then placed in a regular laboratory over set at 104°F for 7 days.

Data Analysis. The LTRC United Testing Machine, SFM-30, was used to conduct the UCS tests and its Datum software calculated the strength results. In the testing data analysis, the molding quality and characteristics of specimens were checked by the comparison of their molding curves against each other. The influence factors on UCS were explored through analyzing various soil physical parameters to have a better understanding on the characteristics of lime treatment and stabilization.

Comparison and Other Tests

Test Plan. Other methods, not currently used by the department, exist to determine the lime percentage necessary for stabilization. This study evaluated some of these methods to determine a quick and easy, yet effective way to determine the necessary lime for stabilization. The methods include pH methods, methods based on Atterberg limits, and other methods.

Eades & Grim (E&G). This study evaluated the E&G method (ASTM D 6276 - 99A, 2006), a method for determining lime percentages for stabilization based on the pH of soil-lime mixtures. The method relies on the relationship between solubility of lime and resulting pH of the soil-lime mixture to determine the optimum additive percentage. The addition of lime increases the soil-lime mixture's pH until lime coats and saturates the soil and the mixture's pH reaches saturation at the concentration of pure lime at a pH of 12.4. The percentage of lime necessary to reach this saturation level is the optimum lime additive percentage and the amount necessary to promote bonding and stabilization. The laboratory pH testing equipment was a Fisher Scientific: AB15+ meter with an Accumet Electrode.

Because this E&G method can be a bit difficult, other methods were evaluated in hopes of developing a simpler method to determine optimum lime content that would reduce or eliminate the need for any additional (or laborious) testing by the LADOTD district laboratories. Specifically, laboratory tests like Atterberg limits combined with activity (Skempton, 1953) defined as the ratio of PI to clay content, were evaluated to determine a relationship for optimum lime content determinations

$$\text{Activity}_{(Ac)} = \frac{\text{Plasticity Index}}{\%\ \text{finer than } 2\ \mu\text{m}}$$

Inactive for activities less than 0.75
Normal for activities between 0.75 and 1.25
Active for activities greater than 1.25

Soils with higher clay activity and PIs can influence construction and cause problems in construction and performance, especially at higher in-situ moisture conditions. Soils with high clay contents generally require higher lime percentages to reach saturation due to the clay's high cation exchange capacity and abundant active surfaces.

Material for E&G Testing. Soil samples for the lime portion of the laboratory portion of this current research project were retrieved from statewide projects, and consisted of fine material, 100 percent passing the #200-sieve (< 75 µm). The AASHTO (M 145) and Unified Soil Classification System (USCS) were used to classify each sample and hydrometer tests (ASTM D 422) were conducted to determine clay content (< 2 µm). To create sufficient samples of various activity classes, sodium-rich montmorillonite was added to some samples. Activity levels were calculated from the Atterberg limits and clay content; subsequently, samples were grouped and labeled according to their activity, inactive, normal, and active, respectively. Some soil samples were blended with additional montmorillonite, at quantities of 10-30 percent by weight, to create additional samples of different activity levels.

PI – Wet Method. Another method, used by the Departments of the Army, Navy, and the Air Force, documented in *Soil Stabilization for Pavements Details a Procedure for Lime Determination* (Joint Departments of the Army, 1994) utilizes the E&G procedure, but also provides a chart for determining an initial design lime content. The chart is called the PI – Wet Method and is presented as Figure 4. This method utilizes the PI and the percent binder (fine generally cohesive material) to determine the necessary percentage of hydrated lime. This method was evaluated as an option for choosing the proper amount of lime necessary for treatment and stabilization.

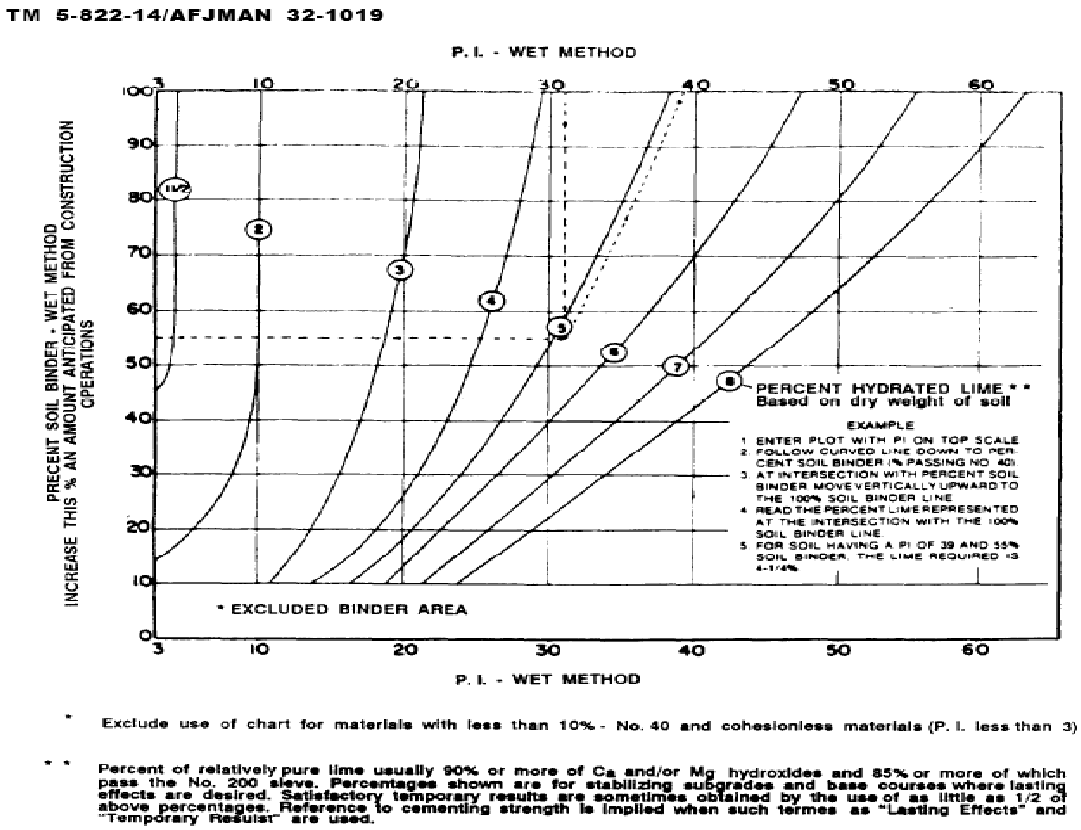


Figure 4

PI – wet method: chart for initial determination of lime content

Data Analysis. In the testing data analysis, various soil types and their required lime percentages were checked by the comparison of their pH values, Atterberg limits, and activity levels against each other. The resulting differences between required lime percentages and different activity classes were compared to determine if a correlation exists. The differences between the results for each different activity class may denote the behavior of clayey soil when stabilized with lime and define the fundamental relationship necessary for each soil type's correlation with the required optimum lime percentage. The influence factors were explored through analyzing various soil physical parameters to have a better understanding on the characteristics of lime treatments in hope of finding an easy reliable method for optimum lime determination.

Lime-Fly Ash Stabilization

As an alternative to rising cement costs and the possibilities of cement shortages, there is a need for alternative treatments and additives. Lime-fly ash (LFA) is one alternative treatment for silty soils, a role primarily occupied by cement.

The purpose of this portion of the study is to determine the blends and additive portions of LFA needed to stabilize silty soils. The use of this material is common in other states and its use may potentially reduce construction costs by providing another alternative for soil stabilization.

Other rationales for utilizing this material are the "building greener" philosophy of using recycled materials. The Department has increased and will continue to consider the use of recycled materials for use in state projects. By using proven recycled materials, the department can reduce amounts directed to landfills, while providing construction material for infrastructure projects.

Lime is not generally effective at treating silty soils because there are little or no pozzolan available for the chemical reactions. However, combining lime with fly ash, a pozzolanic material, has been effective at stabilizing silty soils. A pozzolan is defined by ASTM as "a siliceous or siliceous and aluminous material, which by itself possess little or no cementitious value, but will when in a finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties." Since the LFA blend brings its own pozzolan material (fly ash) to react with the lime's calcium hydroxide during the mixing process, the mixture has all the elements necessary to form chemical bonds of a stabilized layer.

UCS Tests

Additive Blend Test Plan. The study evaluated various different combinations of lime and fly ash to determine an optimum blend of the two LFA components. UCS tests on just the blends, no soil, would evaluate the effectiveness of the lime and fly ash reaction. The strength of the each sample was gauged using UCS testing to determine the best blend percentages of the two additives. The testing factorial is shown in Table 5.

Sample Molding and Curing. The component blends (additive only) were molded using 1500 grams of dry material and a Harvard Miniature Compaction Apparatus mold to create samples with diameters of 1-$^{5}/_{16}$ in. and heights of 2.816 in. at different moisture contents. The samples were not soaked prior to breaks.

Table 5
Testing factorial for LFA blends

Blend Ratio (Lime to Fly Ash)	Moisture Content at Molding, %	Number of Samples
100% Fly Ash	40% (Designed)	3
50 : 50	40% (Designed)	3
70 : 30	40% (Designed)	3
90 : 10	40% (Designed)	3

Soil Test Plan. A similar test plan was developed to determine the effectiveness of different blends at treating a silty soil. The study evaluated various different combinations of lime and fly ash to determine an optimum blend of the two components. The additive percentage was 6 percent and the molding moisture was roughly 22 percent. Table 6 shows the test factorial for the LFA treated silty soil (Soil I).

Sample Molding and Curing. Samples were molded using DOTD TR 434 (ASTM D 698) as a guideline for these samples. The samples were formed using a 4-in. diameter mold with a volume of 1/30 of a cubic-foot. The method is similar to the standard Proctor compaction test in that the material is compacted in three equal lifts each with 25 blows of a 5-lb. hammer with a 12-in. drop. Prepared samples were extruded from their molds and prepared for curing. The samples were cured in the 100 percent humidity room for 28 days.

Table 6
Testing factorial for LFA treated Soil I

Blend Ratio (Lime to Fly Ash)	Moisture Content at Molding, %	Number of Samples
Raw Soil	20 to 24	3
100% Fly Ash	20 to 24	3
50 : 50	20 to 24	3
70 : 30	20 to 24	3
90 : 10	20 to 24	3
100% Lime	20 to 24	3

Data Analysis. The UCS testing was conducted using our United Testing, SFM-30, compression machine. Strengths were evaluated to determine the blend performing best in treating the silty soil, based on the average UCS results. That blend was evaluated further, under various moisture contents and additive percentages in treating Soil I. Table 7 shows the various molding moistures, additive rates, and cure times.

Table 7
LFA factorial for additional UCS testing

Blend Ratio (Lime to Fly Ash)	Moisture Content at Molding, %	100% Room Cure Time, days	Number of Samples
70 : 30	14.5	7,28	2

The results of the tests were evaluated for strength changes and influence of variables. The strength gains based upon additive blend and percentage, the moisture content, and the cure times will be used to determine the effectiveness the LFA at treating wet silty soil at high molding moisture (representing naturally wet conditions).

Resilient Modulus and Permanent Deformation

Test Plan

Materials. Materials for this portion of the laboratory experiment were obtained from the field during construction. The samples were molded and tested at an in-situ condition of their respective test sections (i.e., 95 percent of the maximum dry unit weight and optimum moisture content).

The laboratory repeated load triaxial resilient modulus tests were performed at 1-day, 7-day, 14-day, and 28-day curing periods and permanent deformation tests were performed at a 28-day curing period on the compacted or cored treated subbase materials from the field. In addition, resilient modulus and permanent deformation tests on the untreated subgrade soil were performed. The test factorial is presented as Table 8. This report will focus primarily on the treated subgrade layers. Details on the full testing can be found elsewhere (Mohammad et al., 2010).

Table 8
Test factorial for resilient modulus and permanent deformation

Material	Curing, days	Moisture, w %	Dry Density, γ_d, pcf	No. of samples for M_r	No. of samples for RLC
Laboratory fabricated samples					
Subbase- Lime treated (10 %)	0	16.9	106.9	2	-
	7	16.9	106.9	2	-
	14	16.9	106.9	2	-
	28	16.9	106.9	2	2
Subbase- Cement-stabilized (8%)	0	16.9	106.9	2	-
	7	16.9	106.9	2	-
	14	16.9	106.9	2	-
	28	16.9	106.9	2	2
Cored Samples					
Subbase: Cement-stabilized soil (8%) Core -2.8in x 5.6in	28	16.9	106.9	12	12
Subbase: Lime-treated soil (10 %) Core -2.8in x 5.6in	28	16.9	106.9	12	12
Subgrade- clay- Core -2.8in x 5.6in	NA	16.9	106.9	12	12

Legend: RLC- Repeated load compression test, NA- Not applicable, w-moisture content, γ_d - Dry unit weight

Sample Preparation. For subbase and subgrade materials, cylindrical specimens with a diameter of 2.8 in. and a height of 5.6 in. were compacted for laboratory permanent deformation and resilient modulus tests. The samples for the laboratory repeated load triaxial resilient modulus and permanent deformation tests were compacted in five layers. An impact compactor was used for the compaction of subbase and subgrade materials. The compacted lime-treated and cement-stabilized samples were sealed with polythene bags and kept in a moisture-controlled room for 7-day, 14-day, and 28-day curing. The untreated subbase and subgrade samples were tested immediately after the compaction.

Repeated Load Triaxial Test (Resilient Modulus Test)
The resilient modulus is experimentally determined by applying a repeated axial load on a soil sample that is mounted inside a triaxial cell. The resilient modulus in a repeated load test is defined as the ratio of the maximum deviator stress (σ_d) over the recoverable elastic strain (ε_r).

The Standard Method of Test for Determining the Resilient Modulus of Soils and Aggregate Materials, AASHTO T 307, was used in this study to determine the resilient modulus. The resilient modulus was also estimated from the permanent deformation test that is described below.

Permanent Deformation Test
Permanent deformation tests were conducted using a repeated load triaxial compression test. The test setup consists of an MTS 810 Material Testing System with a closed-loop servo-controlled hydraulic loading system, digital controller, load unit controller, computer, and data acquisition package, TestStarII. Two vertical linear variable differential transformers (LVDT) were symmetrically placed on the top of the sample to measure the displacements. A load cell mounted inside the triaxial cell measured the applied loads. A pressure chamber was used to accommodate the material specimen during the test.

Since no standard test procedure is available for the repeated load permanent deformation of soils and aggregates, similar to the AASHTO T 307 test procedure for base (granular) materials, a haversine load pulse of a 0.1-second loading period and a 0.9-second rest period was used with 10,000 cycles. The tests for subgrade soils were conducted at a vertical stress level of 6 psi that included a cyclic stress level of 5.4 psi and a contact stress level of 0.6 psi. A confining stress level of 2 psi was also maintained during the test. These stress levels were selected based on a stress analysis conducted to compute a field representative stress condition in the subgrade layer.

The samples were conditioned before the test by applying 1,000 cyclic stress levels of 3.6 psi together with a confining stress level of 6 psi. The conditioning step removes irregularities on the top and bottom surfaces of the test specimen. The initial stage of the permanent deformation takes place under the conditioning stress.

Field Experiment Design

Pavement Research Facility

The field test program was conducted at LTRC's Pavement Research Facility (PRF) site located in Port Allen, Louisiana. Fieldwork included two major parts: building the full-scale test sections according to proposed construction specifications and evaluating the performance of those stabilized subgrade soils through in-situ tests. The test sections included in this report were constructed as part of multiple experimental lanes.

Comparison Sections

Of the six constructed at the PRF, two APT test sections, as outlined in Figure 5, were the focus of this report. Each section was 13 ft. wide by 107.5 ft. long. As shown in Figure 5, all sections had a 2.0 in. HMA (hot mix asphalt) top layer, an 8.5 in. crushed stone base layer, and a 12 in. treated soil layer. The only difference lied in the subbase layer: section 4-1B had a lime-treated soil layer (working table), while section 4-2B used a cement-stabilized soil layer as a subbase in the corresponding pavement structure.

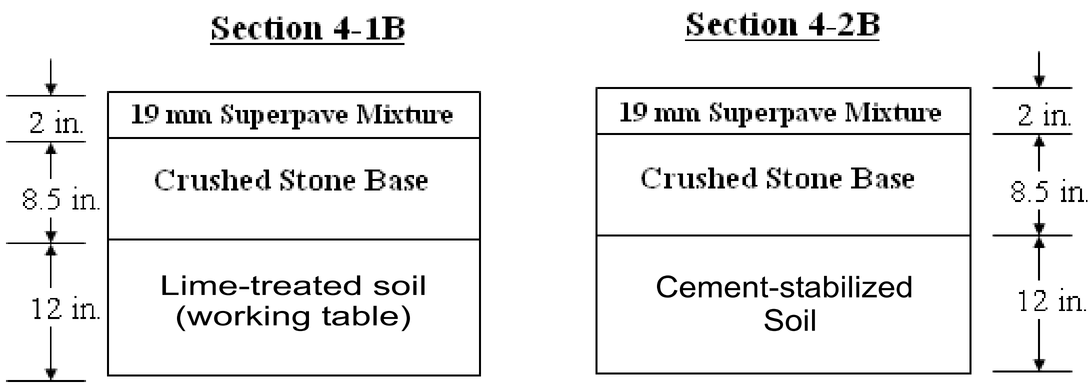


Figure 5
Pavement structures of ALF test sections

Construction Materials

Treated-Soil Layers and Subgrade. The lime-treated soil (working table) in section 4-1B was an in-place 10 percent lime-treated "working table" layer; whereas, the cement-stabilized soil (CSS) used in section 4-2B was an in-place 8 percent cement-stabilized soil. A silty-clay soil was used in the working table and CSS layers as well as the corresponding subgrade layers of this experiment. Basic properties of the silty-clay soil are presented in Table 9. It consists of 60.3 percent silt and 23.5 percent clay. The liquid limit and the plastic index (PI) are 31and 12 percent, respectively.

Table 9
Soil properties

Passing # 200 (%)	Clay (%)	Silt (%)	LL (%)	PI	W_{opt} (%)	γ_d (pcf)	Classification	
							USCS	AASHTO
91	23.5	60.3	31	12	18.5	106.8	CL-ML	A-6

Prior to 2004 the use of 10 percent lime by volume to treat wet soil subgrades for the southern Louisiana pavement condition was considered common practice. Section 4-1B represents current practice under the current lime treatment option and was not designed as a structural layer, merely a working table for wet subgrades. The comparison of its performance to the designed cement-stabilized section is not on a level playing field. The lime working table was required to improve the wet untreated subgrades to create a working table for construction of the base. The cement-stabilized section, on the other hand, was done to show the improvement over the current working table process.

The selection of 8 percent cement by volume to treat the A-6 soil in this study was based on a design criterion (proposed by this research study) of a minimum unconfined compressive strength of 150 psi under an in-situ moisture condition. The laboratory repeated load triaxial test indicated that the CSS material had a typical resilient modulus value of 115 ksi under a 6 psi deviator and 2 psi confinement stresses, followed by the working table material of 89 ksi and subgrade soil of 10.7 ksi. The percent permanent strains measured at 10,000 cycles were 0.0043, 0.0071, and 0.28, respectively, for the CSS, working table, and subgrade soils.

Sampling During Construction

LTRC technicians collected lime and cement-stabilized subgrade soil from the PRF testing sections immediately after they were thoroughly mixed by the stabilizer. The collected mixtures were then brought to LTRC and molded into samples for unconfined compression strength testing.

The UCS test was conducted according to ASTM D1633. A 4.0 in. in diameter and 4.0 in. high specimen was loaded at an axial strain rate of 0.05 in./min. using a United Compressive Testing machine. The load to failure was recorded and the UCS determined.

Crushed Stone Base. The crushed stone base used in this experiment was classified as a Class-II base course according to the LADOTD's standard specification for roads and bridges (LADOTD, 2000 Edition). Kentucky crushed limestone was used. Note that the maximum liquid limit and the maximum plasticity index of a Class-II base course shall be less than 25 and 4 percent, respectively, for the fraction of stone passing the No. 40 sieve.

HMA Mixture. The HMA mixture used was a 19 mm Level 2 Superpave mix designed at 100 gyrations using the Superpave Gyratory Compactor. An elastomeric polymer-modified asphalt binder, classified as PG 76-22m, was specified for the mixture. The aggregate blend consisted of 45.4 percent #67 coarse granite aggregate, 17.1 percent #11 crushed siliceous limestone, 10.3 percent coarse sand, 12.9 percent crushed gravel, and 14.3 percent reclaimed asphalt pavement (RAP). The optimum asphalt binder content was 4.4 percent including 3.7 percent PG 76-22 virgin binder and 0.7 percent recycled binder (estimated from the recycled asphalt pavement, RAP, materials used).

Accelerated Pavement Testing (APT)

APT Loading History

The ALF. The APT loading device used is called the Accelerated Loading Facility. The ALF device models a half single axle load with a dual-tire wheel assembly. The magnitude of an ALF wheel load per load application may be adjusted from 9,750 lb. to 18,950 lb. More details on the LTRC's ALF device may be referred to elsewhere (Metcalf, 2000).

In this study, two MICHELIN radial 11R22.5 tires were used and the target cold tire inflation pressure was 105 psi. To simulate highway traffic, the ALF loads were applied only in one direction and a traffic wander of 15 in. (normally distributed) was adopted. In addition, to minimize the environmental effects in performance comparison, the ALF device was moved alternatively from one section to another in a three-section set after every 25,000 ALF passes. During the experiment, the magnitude of the ALF loads was kept as 9,750 lb. for the first 200,000 repetitions. The loads were then adjusted in increments by adding steel plates (each weighing 2,300 lb.) at the intervals noted in Table 10.

Table 10 is the ALF loading history for the two test sections (4-1B and 4-2B) considered in this report.

Table 10
ALF loading history

Section	No. of Passes (x 1000)	Total Load lb.	ESAL Factor*	ESALs	Cumulative ESALs
4-1B	0-150	9,750	1.377	206,605	206,605
4-2B	0 - 200	9,750	1.377	275,473	275,473
	200 - 225	12,050	3.213	80,338	355,811
	225 - 300	14,350	6.463	484,729	840,540

Note: the ESAL factor is based on the ratio between ALF load and 9000 lb. raised to the 4^{th} power

Failure Criteria. For this APT experiment, a test section is considered to have failed when the pavement condition meets one of the following failure criteria: (1) the average rut depth reaches up to 0.5 in. among eight measurement stations within the trafficked area of a section or (2) 50 percent of the trafficked area develops visible cracks (e.g., longitudinal, transverse, and alligator cracks) more than 1.5 ft/ft^2.

Field Measurements

Instrumentation. The instrumentation on test sections includes the installation of devices for measuring both vertical stresses and in-depth deflection profiles. The instrumentation layout is outlined in Figure 6. For each test section, two Geokon 3500 pressure cells were embedded at two depths directly under the section's centerline: one at the bottom of the base layer and the other at top of the subgrade. One multi-depth deflectometer (MDD) with six potentiometers (deformation measurement sensors) was installed on each test section at a longitudinal distance of 4.5 ft. from the pressure cell location along the centerline (Figure 6). The MDD used is called SnapMDD, a patent product manufactured by the Construction Technology Laboratories, Inc. in Illinois. More details on those instrumentation devices may be referred to elsewhere (Wu, 2006).

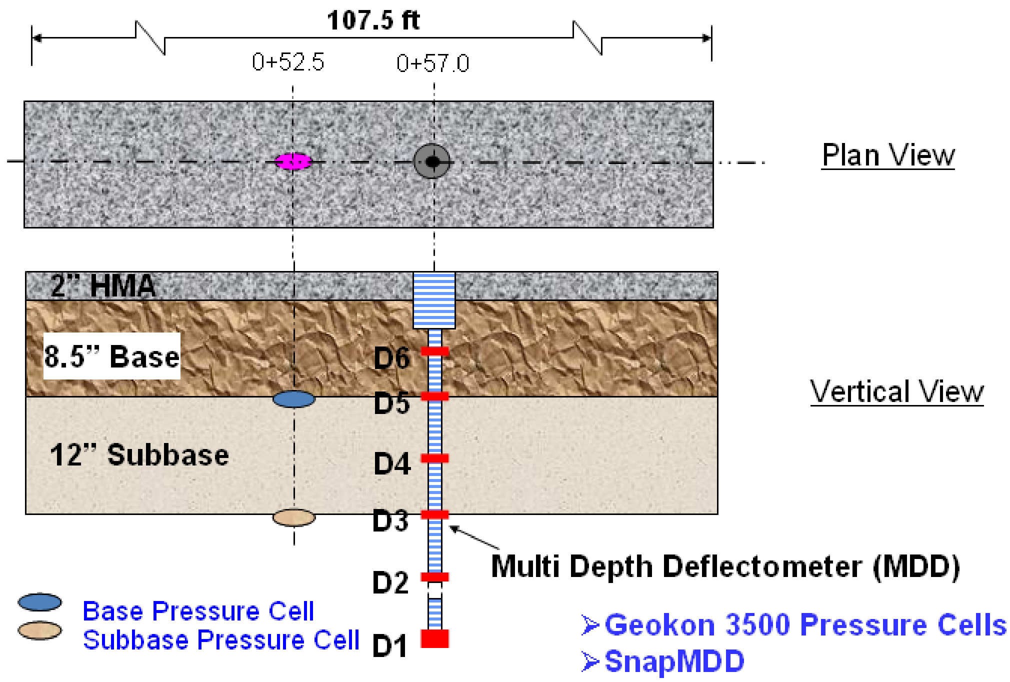


Figure 6
Field instrumentation layout

The field instrumentation data were collected at approximately every 8,500 ALF load repetitions. All pavement responses were measured under the left tire of the ALF dual tire assembly when the tire was directly positioned on the top of an instrumentation device (i.e., pressure cell and MDD).

In-Situ and Non-Destructive Testing (NDT)

NDT tests in this experiment included the falling weight deflectometer (FWD) and Dynaflect tests. The FWD device used was a Dynatest 8002 model FWD device. The surface deflections were measured with nine sensors spaced at 0, 8, 12, 18, 24, 36, 48, 60, and 72 in., respectively. The Dynaflect is another surface deflection measurement device. This device induces a dynamic load of 1,000 lb. at a frequency of 8 Hz on the pavement and measures the resulting deflections by using five geophones spaced under the trailer at approximately 1-ft. intervals from the application of the load.

Rutting and Cracking Survey. NDT tests as well as the rutting and cracking survey were performed at the end of each 25,000 load repetitions. A cracking survey was performed based on the hand-drawing method using visual observation. Rut depths were measured using a straight edge device called “A-Frame.”

Data Analysis Techniques

The data analysis of this study included the processing of NDT deflection data, evaluation of instrumentation results, modeling pavement structure, and prediction of pavement performance in terms of pavement distresses. The following analysis procedures and software were used in this study.

Louisiana Pavement Evaluation Chart. The Louisiana pavement evaluation chart (Kinchen, 1980) for the estimation of existing pavements’ structural number is based on Dynaflect measured deflection. As shown in Figure 7, an effective structural number (SN) and a design subgrade modulus of existing pavements can be determined based on a temperature-corrected Dynaflect center deflection and a percentage spread value (i.e., the average deflection in a percentage of the central deflection). This method was used in the analysis of Dynaflect deflection results for determination of the SN values of test sections under different ALF repetitions.

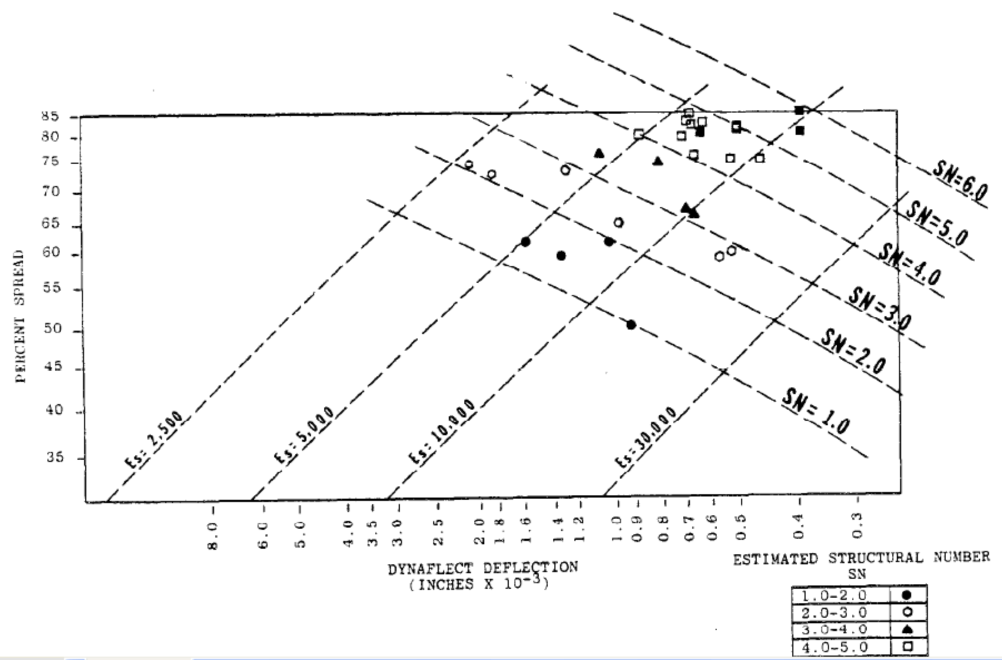


Figure 7
Louisiana pavement evaluation chart (Kinchen, 1980)

FWD-Deflection Determined Structural Number. According to the AASHTO 1993 pavement design guide, the effective structural number (SN_{eff}) may be estimated from FWD deflection results (AASHTO, 1993). Equation (4) shows the equation for the SN_{eff} estimation,

$$SN_{eff} = 0.0045 \times D \times \sqrt[3]{E_p} \tag{4}$$

where,

D = total pavement thickness above the subgrade, in.; and

E_p = effective pavement modulus of all existing pavement layers above the subgrade.

The *1993 AASHTO Guide for Design of Pavement Structures* (AASHTO, 1993) provides the following equations for backcalculation of E_p from the FWD measured deflections:

$$M_R = \frac{0.24P}{d_r r} \tag{5}$$

$$d_0 = 1.5pa\left\{\frac{1}{M_R\sqrt{1+\left(\frac{D}{a}\sqrt[3]{\frac{E_p}{M_R}}\right)^2}} + + \frac{\left[1-\frac{1}{\sqrt{1+\left(\frac{D}{a}\right)^2}}\right]}{E_p}\right\} \tag{6}$$

where,

M_R = backcalculated subgrade resilient modulus, psi;

D = total thickness of pavement layers above the subgrade, in.;

d_0 = deflection measured at the center of the FWD plate (and adjusted to 68°F), in.;

P = applied load, lb.;

p = FWD load plate pressure, psi;

d_r = deflection at a distance r from the center of FWD plate, in.;

r = distance from center of load, in.; and

a = FWD load plate radius, in.

In this study, the SN_{eff} backcalculated from the FWD tests at the beginning of the APT loading were analyzed and used in prediction of layer coefficient of CSS material investigated.

EVERCALC. The EVERCALC is a Windows-based computer program developed by Washington DOT for backcalculation of layer moduli based on FWD measured deflection basins (Pierce, 1996). EVERCALC is based on the multi-layered elastic analysis program WESLEA (provided by the Waterways Experiment Station, U.S. Army Corps of Engineers), which produces the pavement response parameters, such as stresses, strains, and deformations in the pavement system. EVERCALC was used in this study for the backcalculation of layer moduli based on FWD measured deflection bowls.

ELSYM5. ELSYM5 was originally developed by Gale Ahlborn of the Institute of Transportation and Traffic Engineering (ITTE) at the University of California at Berkeley (Wu, 2007). It is based on the multi-layer elastic computer model with the ability to consider multiple loads as well as the presence of a rigid base below the subgrade. ELSYM5 was used in computing the vertical stresses developed in the pavement section under the ALF load.

DISCUSSION OF RESULTS

Laboratory Evaluation

Soil Basic Properties

Three soil types used in this study, designated as Soil I, II, and III in the following sections, were collected from the Baton Rouge, LA area. Table 11 shows the physical properties of each soil. Specific gravities for Soil I, II, and III are 2.65, 2.72, and 2.73, respectively. Figure 8 shows their particle size distribution curves. Moisture content and dry unit weight relationships for tested soils were determined by standard Proctor procedure and are plotted in Figure 9. Their maximum dry unit weights were 108.0 pcf, 119.0 pcf, and 85.0 pcf, respectively, at the corresponding optimum moisture content of 17.5 percent, 13.5 percent, and 33.1 percent. Accordingly, the USCS and AASHTO classifications for Soil I, Soil II, and Soil III are CL/A-6, CL/A-7, and CH/A-7, respectively.

Table 11
Physical properties of tested soils

Soil Type	Silt %	Clay %	LL %	PI %	w_{opt} %	Maximum Dry Density, pcf	Specific Gravity	Classification USCS/AASHTO
Soil I	64.5	27.5	34	12	17.5	108.0	2.65	CL/A-6
Soil II	30.6	27.9	37	22	13.5	119.0	2.72	CL/A-7
Soil III	13.7	81.9	83	49	33.1	85.0	2.73	CH/A-7

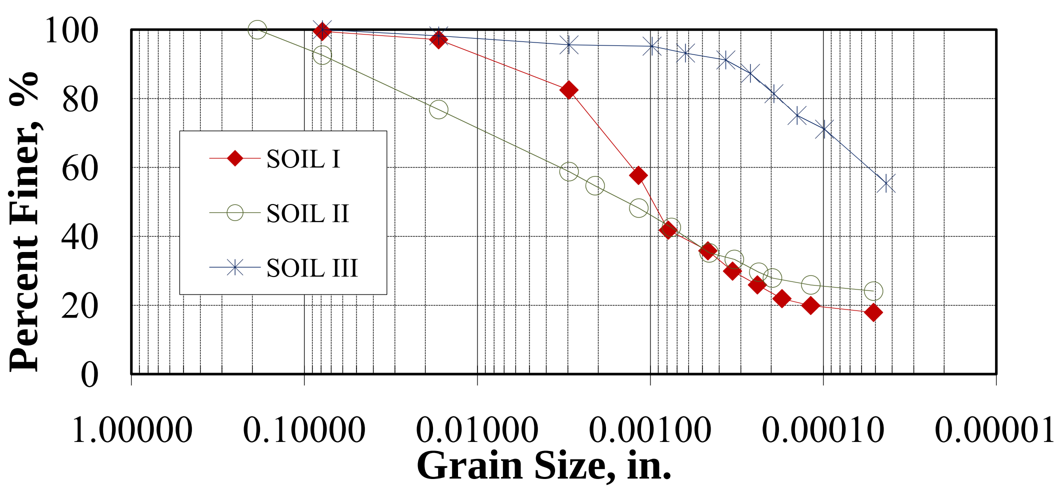


Figure 8
Particle size distribution of tested soils

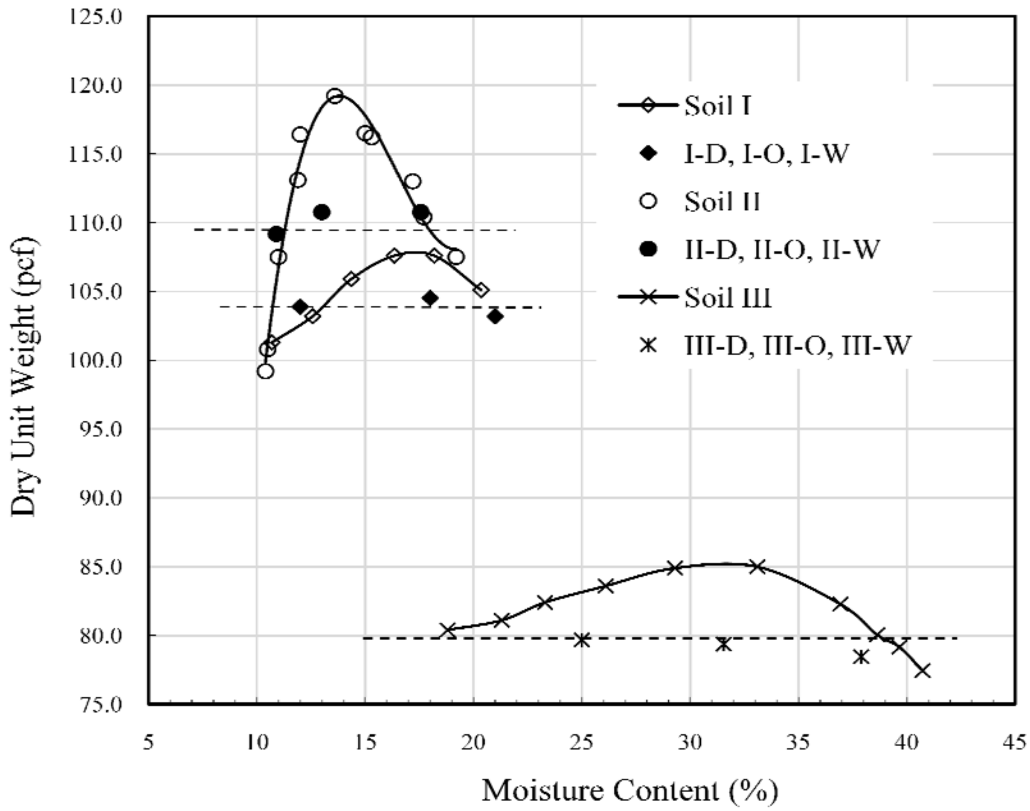


Figure 9
Standard compaction curves of tested soils

Cement Stabilization

Unconfined Compressive Strength (UCS) Test

Specimen Quality. The molding quality of specimens prepared in this study was evaluated through the relationships between their moisture content and dry unit weight with and without chemical stabilization.

Figure 10 and Figure 11 show the compaction curves with and without cement treatment. These two figures indicate that the impact of cement on the soil compaction was quite limited before any cement hydration occurred. The variation of dry unit weight for the same moisture content in the specimens prepared was mainly on the dry side of the compaction curves. On the wet side of the curves, the correlations between molding moisture content and dry unit weight were quite constant.

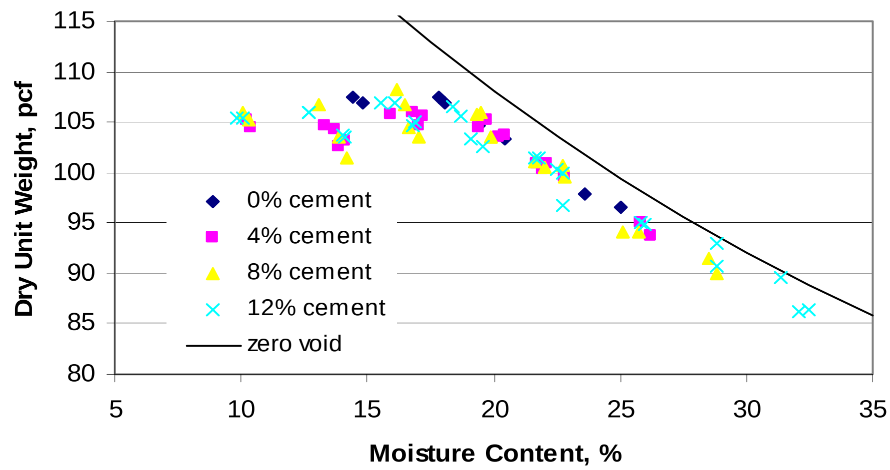


Figure 10
Compaction curves with and without cement for Soil I

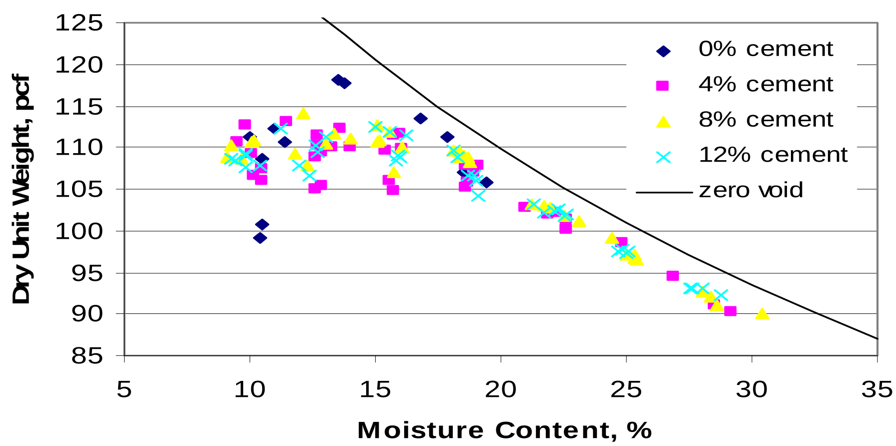


Figure 11
Compaction curves with and without cement for Soil II

UCS Tests. Figure 12 and Figure 13 show the UCS of the two soils (Soil I and II) at raw (no additive) and stabilized with 4, 8, and 12 percent cement. Samples were cured for 7, 14, and 28 days. As expected, cement content, molding moisture content, and curing period control of the cement-stabilized soils affected the UCS strength results. A higher cement content and longer curing time result in a higher strength of stabilized soils.

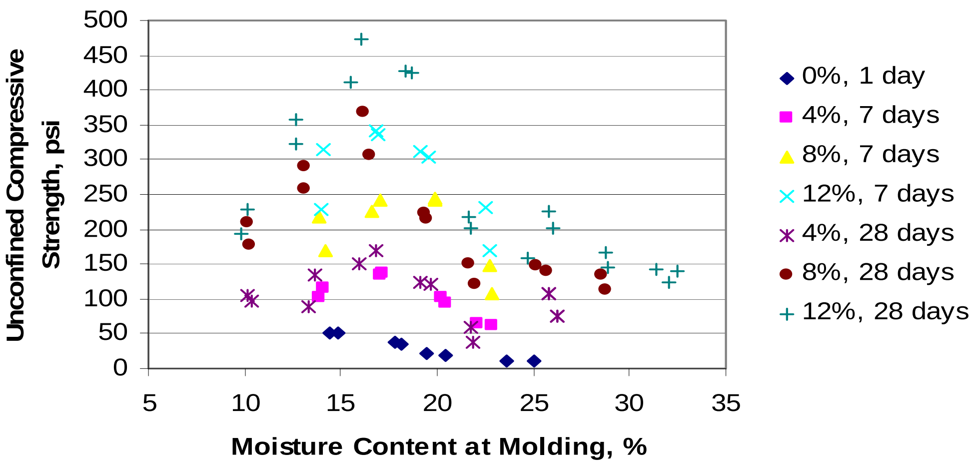


Figure 12
UCS and moisture content at molding of various conditions, Soil I

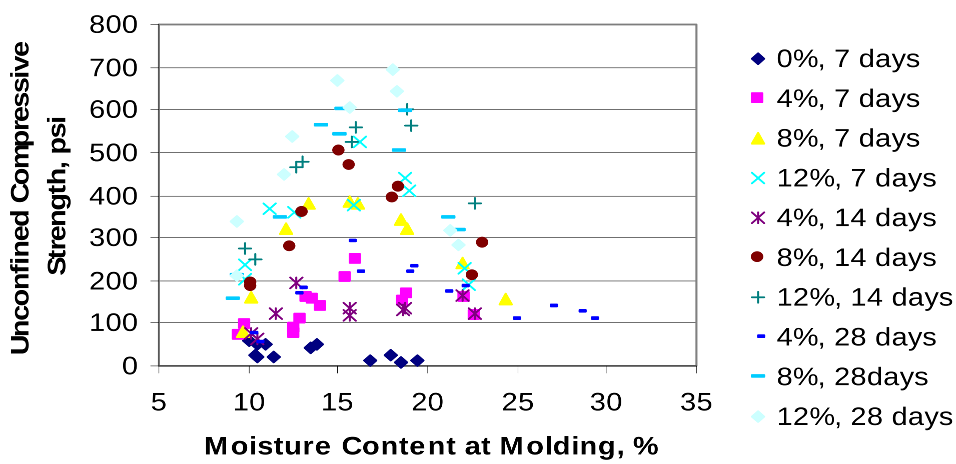


Figure 13
UCS and moisture content at molding of various conditions, Soil II

Effect of Molding Moisture Content. The effect of molding moisture content of a specimen on its UCS is quite complicated because this moisture content does not only determine how much water is available in the soil for cement hydration, but also affects the dry unit weight of the molded specimen. The researchers' experience with cement indicates that the water-cement ratio is the main controlling factor for the strength of cement-stabilized materials. If this is the case here, the UCS should decrease as the moisture content increases when same cement content and curing time are used. Instead, the comparison of Figure 14 and Figure 15 with Figure 10 through Figure 13 indicate the correlation between the UCS and moisture content of cement-stabilized soils follows the pattern of the correlation of the dry unit weight and moisture content of the material. The highest strength is not at the lowest moisture content (lowest water-cement ratio) but around the optimum moisture content of the soils. Therefore, correlations of the UCS with specimens' water-cement ratio, as shown in Figure 14 and Figure 15, are not satisfactory from both the theoretical and practical point of views. The scatter is especially large for the low range of water-cement ratio.

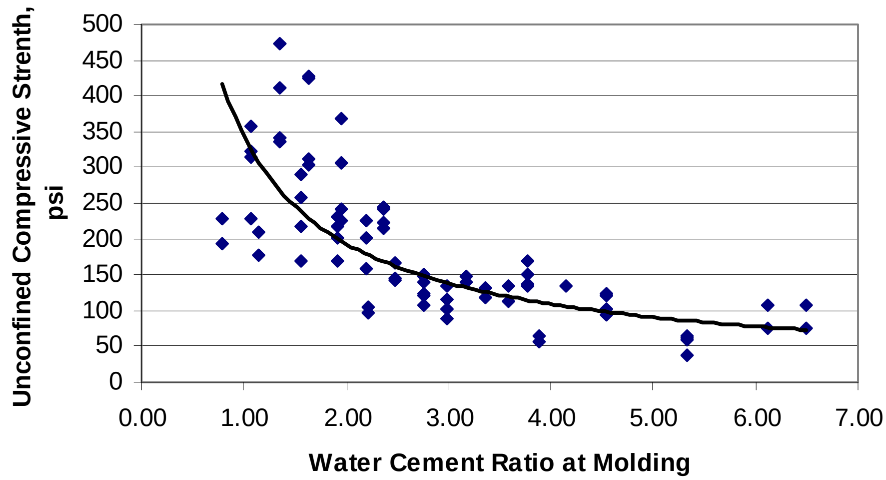


Figure 14
UCS and water-cement ratio at molding, Soil I

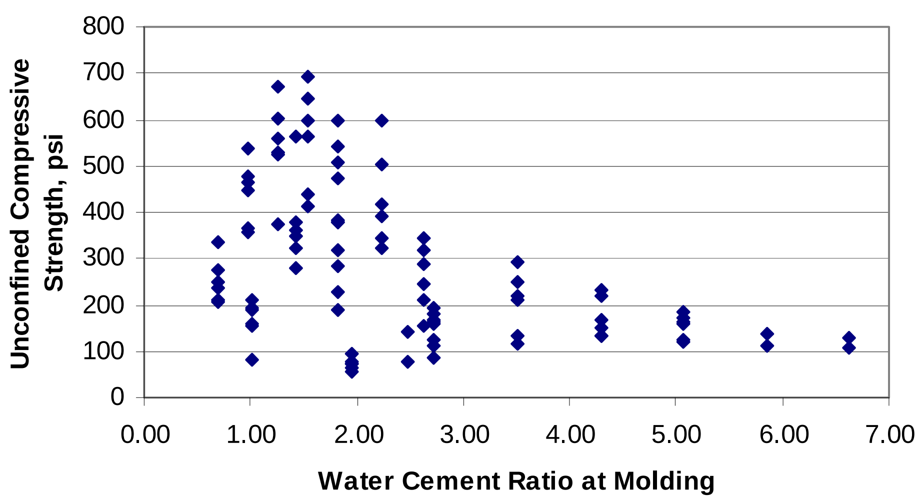


Figure 15
UCS and water-cement ratio at molding, Soil II

Table 12 explains the phenomenon shown in Figure 14 and Figure 15. If the cement content increases, the strength will increase, and the water cement ratio will decrease. For the molding moisture on the dry side of a compaction curve, as moisture content increases, both dry unit weight and strength will increase as well as the water-cement ratio; on the wet side of the compaction curve, the increase of molding moisture will reduce both dry unit weight and strength but increase the water-cement ratio. The conclusion is that on the wet side of the compaction curve, the water-cement ratio is a good indicator for the strength of cement-mixed soils. The strength will increase with the decrease of water-cement ratio. The strong correlation of water-cement ratio with UCS values on the wet side of compaction curves provides a quick means of approximating strength.

Table 12
Correlations among different factors

<table>
<tr><td>Independent Factors</td><td colspan="4">Dependent Factors</td></tr>
<tr><td rowspan="2">Cement Content
(↑)</td><td colspan="2">Dry side of compaction curve</td><td colspan="2">Wet side of compaction curve</td></tr>
<tr><td colspan="2">Water-cement ratio (↓)
Strength (↑)</td><td colspan="2">Water-cement ratio (↓)
Strength (↑)</td></tr>
<tr><td rowspan="2">Molding Moisture Content
(↑)</td><td colspan="2">Dry side of compaction curve</td><td colspan="2">Wet side of compaction curve</td></tr>
<tr><td>Water-cement ratio
(↑)
Dry unit weight(↑)</td><td>Strength
(↑)</td><td>Water-cement ratio
(↑)
Dry unit weight (↓)</td><td>Strength
(↓)</td></tr>
</table>

Note: ↑=increase; and ↓=decrease.

Different Performance along Compaction Curves

A compaction curve is the correlation of soil dry unit weight with its molding moisture content. The previous discussion implies that cement-stabilized soils perform differently on the wet and dry sides of the compaction curves. Figure 16 through Figure 19, show the water-cement ratio at molding for the two cement-stabilized soils on both dry and wet sides. They indicate that different patterns that exist between the strength and water-cement ratio on the dry and wet sides of compaction curves. This phenomenon was further studied through the data that follow.

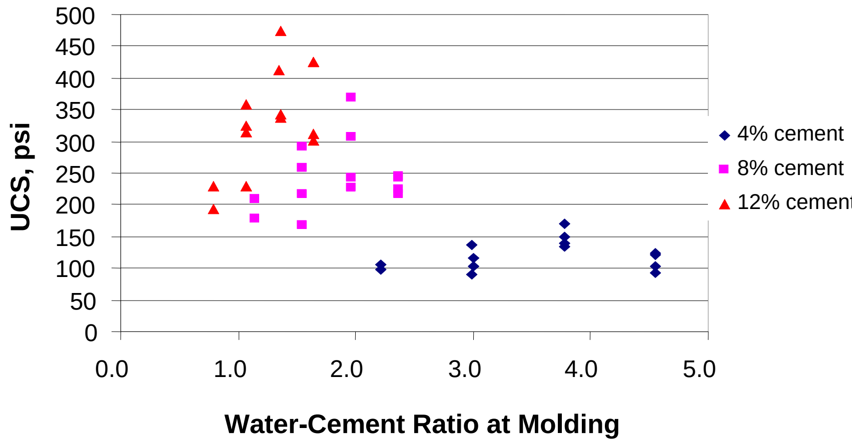


Figure 16

Performance along the compaction curve, Soil I, dry side

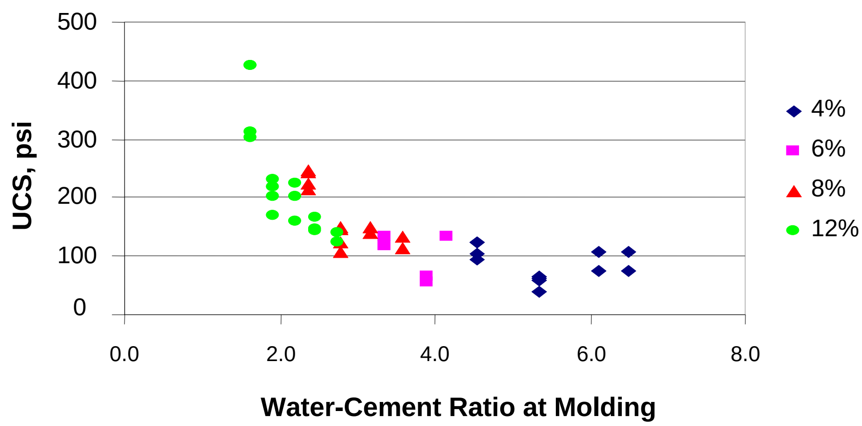


Figure 17

Performance along the compaction curve, Soil I, wet side

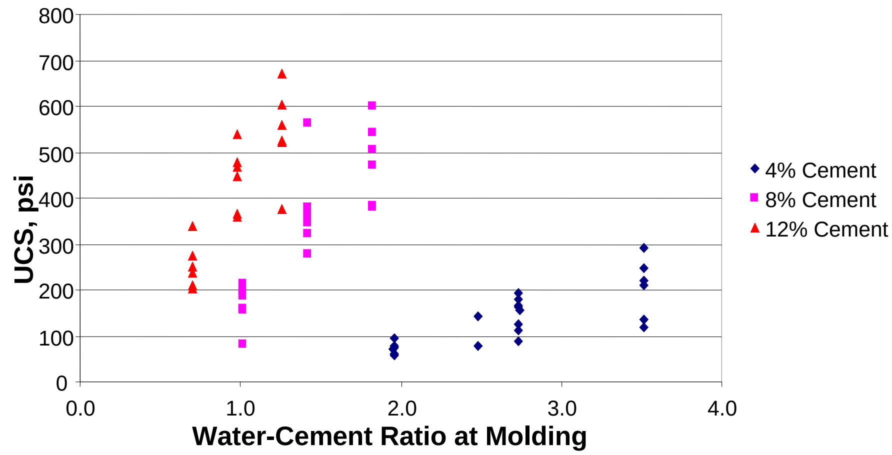


Figure 18

Performance along the compaction curve, Soil II, dry side

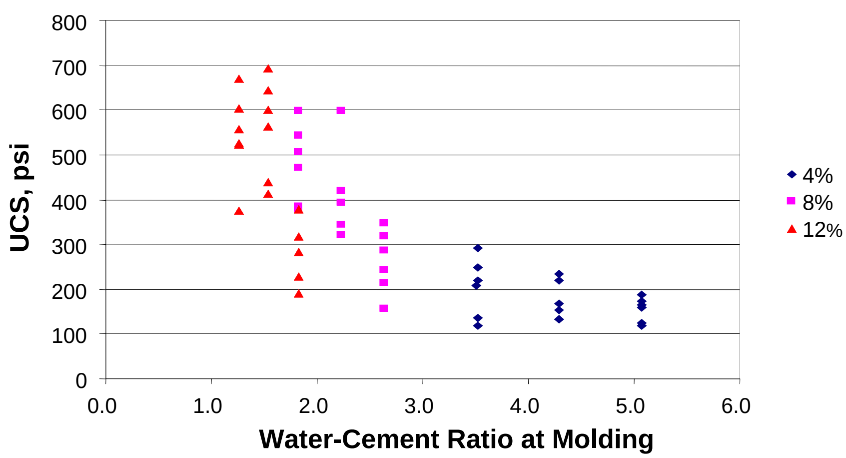
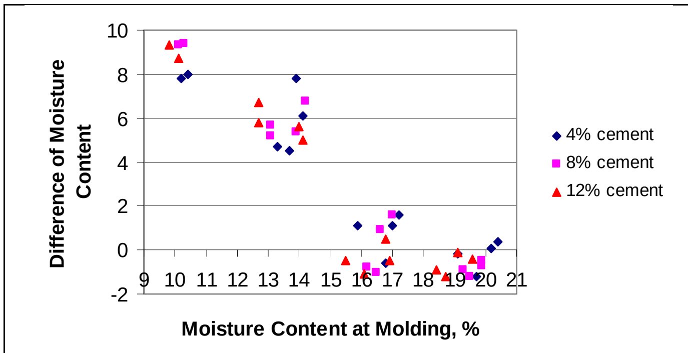


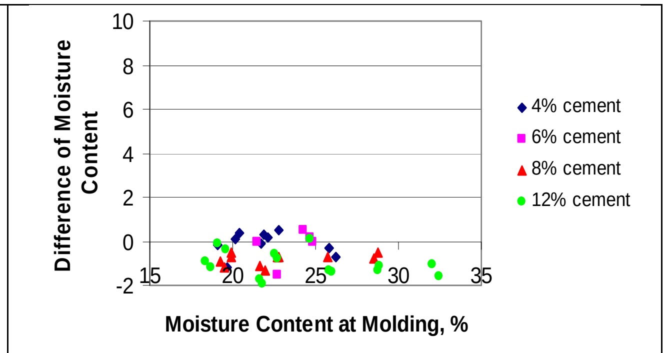
Figure 19

Performance along the compaction curve, Soil II, wet side

Figure 21 shows the correlations between the moisture content of specimens at molding and change of moisture content during specimen curing, calculated as the difference (subtraction) of the moisture content at molding from the one at break. Test data indicate that the 4-hour soak of specimens in water had little effect on the moisture content of the specimens molded on the wet side of optimum. These figures indicate that specimens molded on the dry side of the compaction curves absorbed more moisture than those on the wet side of the curves, and specimens molded at lower moisture absorbed more moisture. This is because cement-stabilized soils have a higher suction potential when they are dry. This high suction potential caused moisture movement within specimens during the curing period and hindered the strength development of the cement-stabilized soils, as shown in Figure 22 and Figure 23. These figures show the impact of moisture on strength and indicate that on the dry side of the compactions curves, higher moisture absorption during the specimen curing period resulted in a lower shear strength. The impact of moisture absorption during curing time on the strength of cement-stabilized soils depends on the plasticity (defined by PI) of raw soils. Soils with higher PIs (higher clay contents) tend to be more affected by moisture absorption when cement is used to treat soils on the dry side of compaction curves. This can lead to cracking, inconsistent distribution of cement within the soil, and weaker durability in these high PI soils.



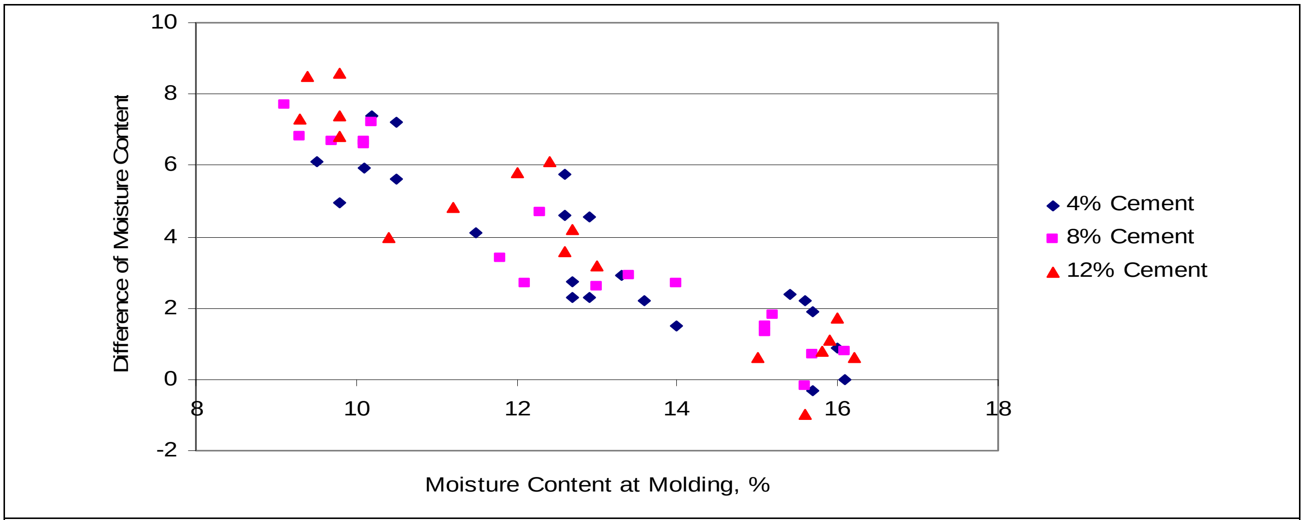
Dry Side



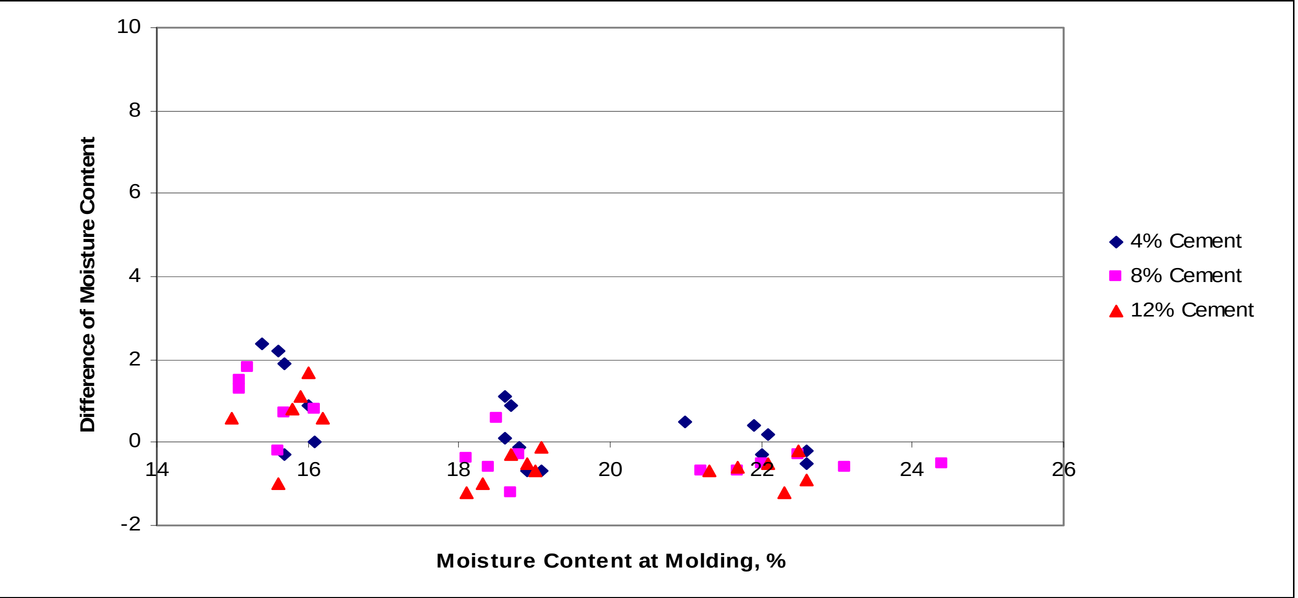
Wet Side

Figure 20

Variation of moisture content during curing, cement-stabilized Soil I



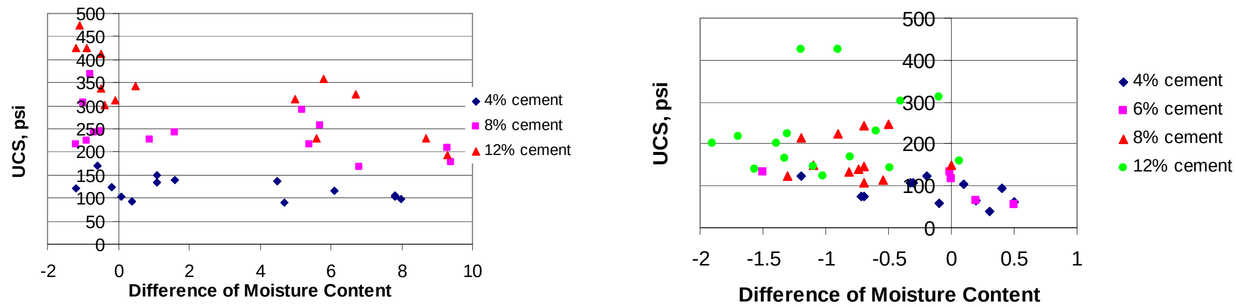
Dry Side



Wet Side

Figure 21

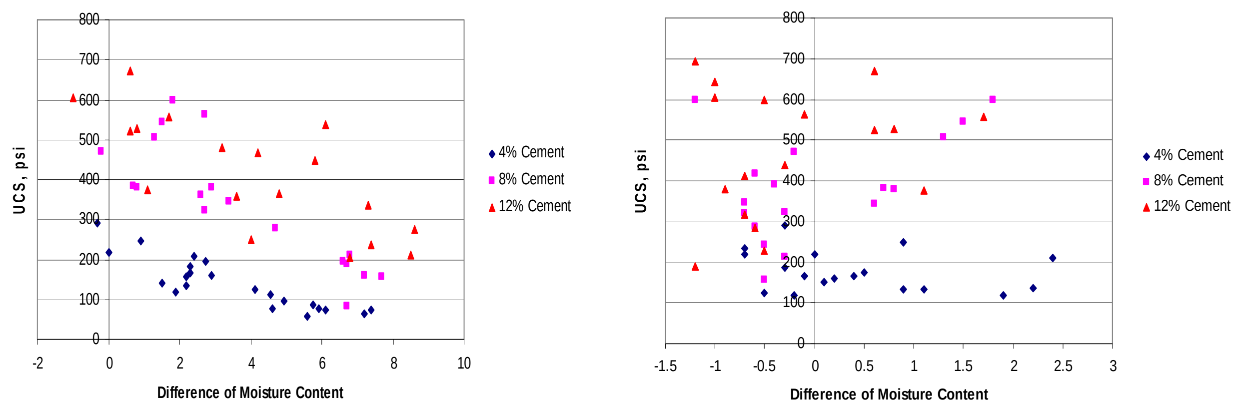
Variation of moisture content during curing, cement-stabilized Soil II



Dry Side Wet Side

Figure 22

Impact of moisture changes on strength, cement-stabilized Soil I



Dry Side Wet Side

Figure 23

Impact of moisture Changes on strength, cement-stabilized Soil II

Comparison and Other Tests

TS Results of Raw Soil. The variation of DV with time for the un-stabilized soil specimens (Soil I) is illustrated in Figure 24. The fluctuation of curves in the figure reflects the balance of moisture evaporation in the air and supply from the bottom. Apparently, the samples with the molding moisture contents of 18.5 percent, 21.5 percent, and 24.5 percent took a longer time to reach their maximum DVs than the samples molded on the dry side of the optimum moisture, possibly due to different microstructures caused by different molding moisture contents. More randomly oriented microstructures that formed on the dry side of the optimum moisture while the microstructures formed on the wet side of the optimum were more horizontally oriented. Also, the final stable DVs were all above the value of 30. Referring to the criteria proposed by (Scullion, 1997), which was mainly for coarse soils or aggregates, the soil tested was water susceptible with its maximum DV above 16. This was proved through the field experience.

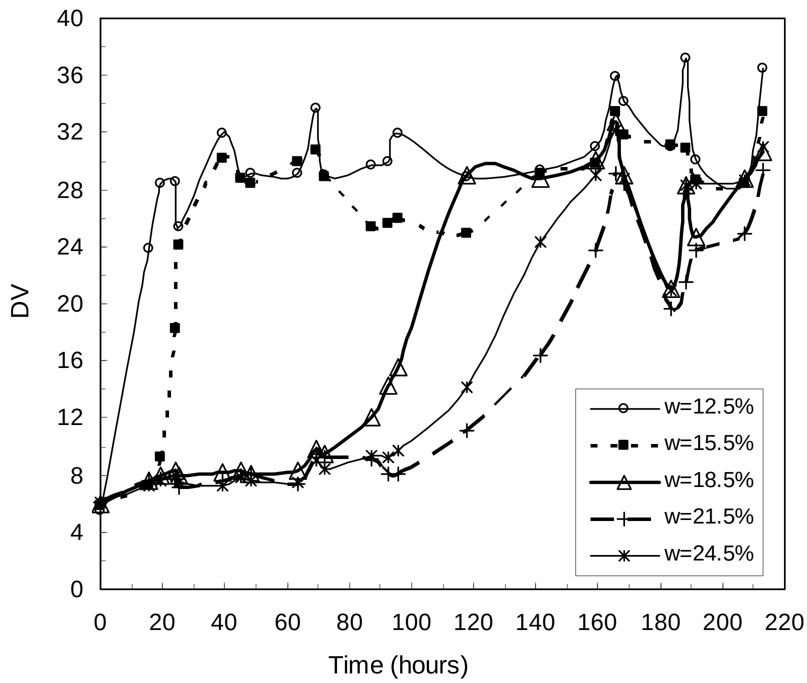


Figure 24
Variation of dielectric value (DV) with elapsed time

TS Results of Cement-Stabilized Soil. The variation of dielectric values with time for each tested sample is shown in Figure 25(a) to Figure 25(d). Compared to the non-stabilized silt, cement-stabilized soil generally yielded lower maximum dielectric values at the same molding moisture content. At each molding moisture content, the maximum dielectric value, or the asymptotic dielectric value, generally decreased with the increase in cement content. At the molding moisture content of 15.5 percent, at least 8.5 percent cement is required to reduce the maximum dielectric value down to 12.0; while at the moisture content of 18.5 is close to the optimum moisture content of the soil, cement was not effective in reducing the maximum dielectric value until a dosage of 12.5 percent was used. At higher molding moisture contents of 21.5 percent and 24.5 percent, the addition of cement was the least effective in reducing the maximum dielectric value. It follows that the ability of cement in reducing the maximum dielectric value also largely depended on the molding moisture content.

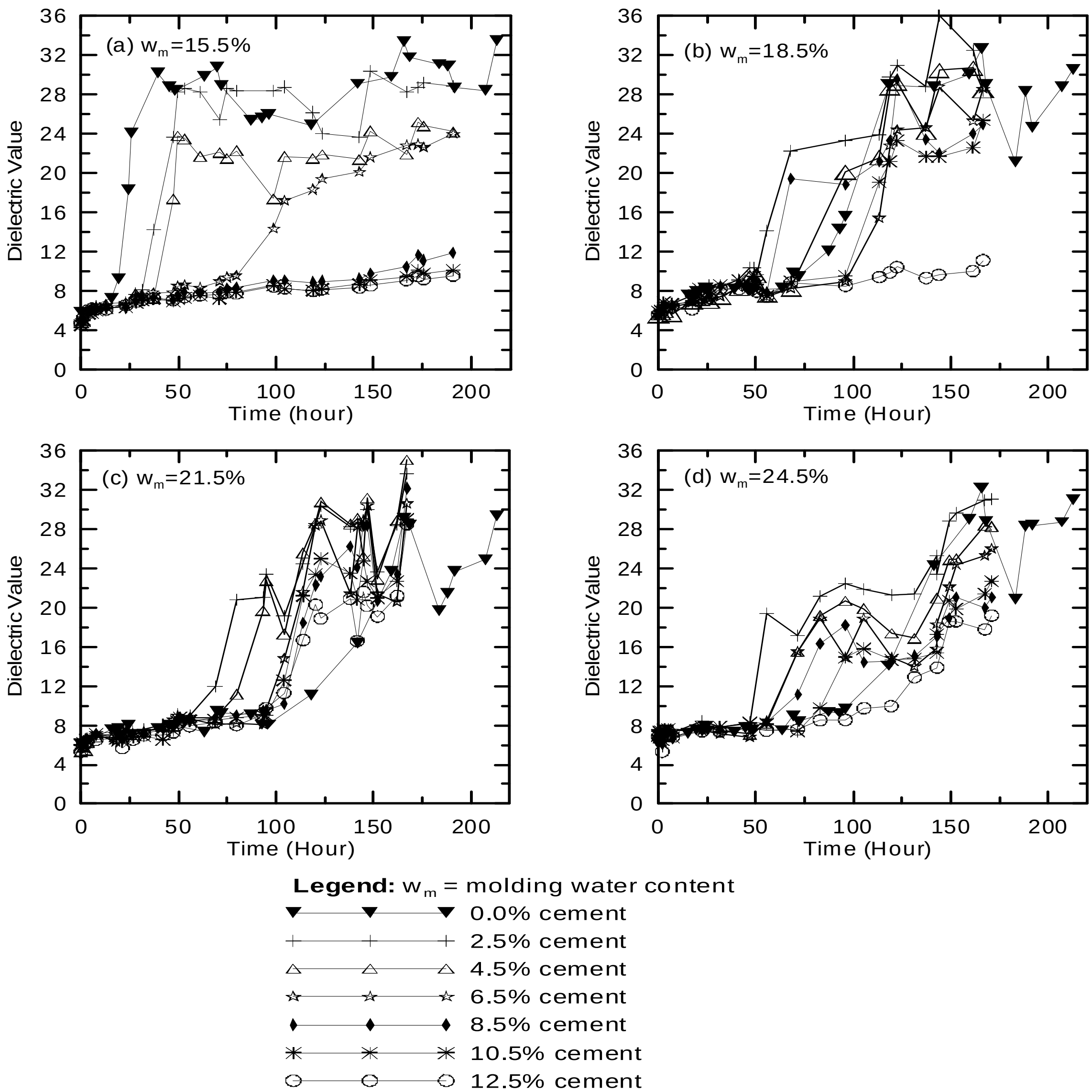


Figure 25

Variation of dielectric values with elapsed time for soil stabilized with various cement dosages: (a) molding moisture content = 15.5 percent; (b) molding moisture content = 18.5 percent; (c) molding moisture content = 21.5 percent; and (d) molding moisture content = 24.5 percent

Reorganizing the data in Figure 25 by grouping the samples with the same cement content, indicates that the increase of cement usage will delay the moisture capillarity from about 50 hours to 100 hours as shown in Figure 26. This delay may help the performance of cement stabilized soil when it is exposed to free water for a short period of time. Also at the low cement dosages, samples molded on the dry side of the compaction curve can suck in free water faster than those compacted on the wet side until enough amount of cement is used.

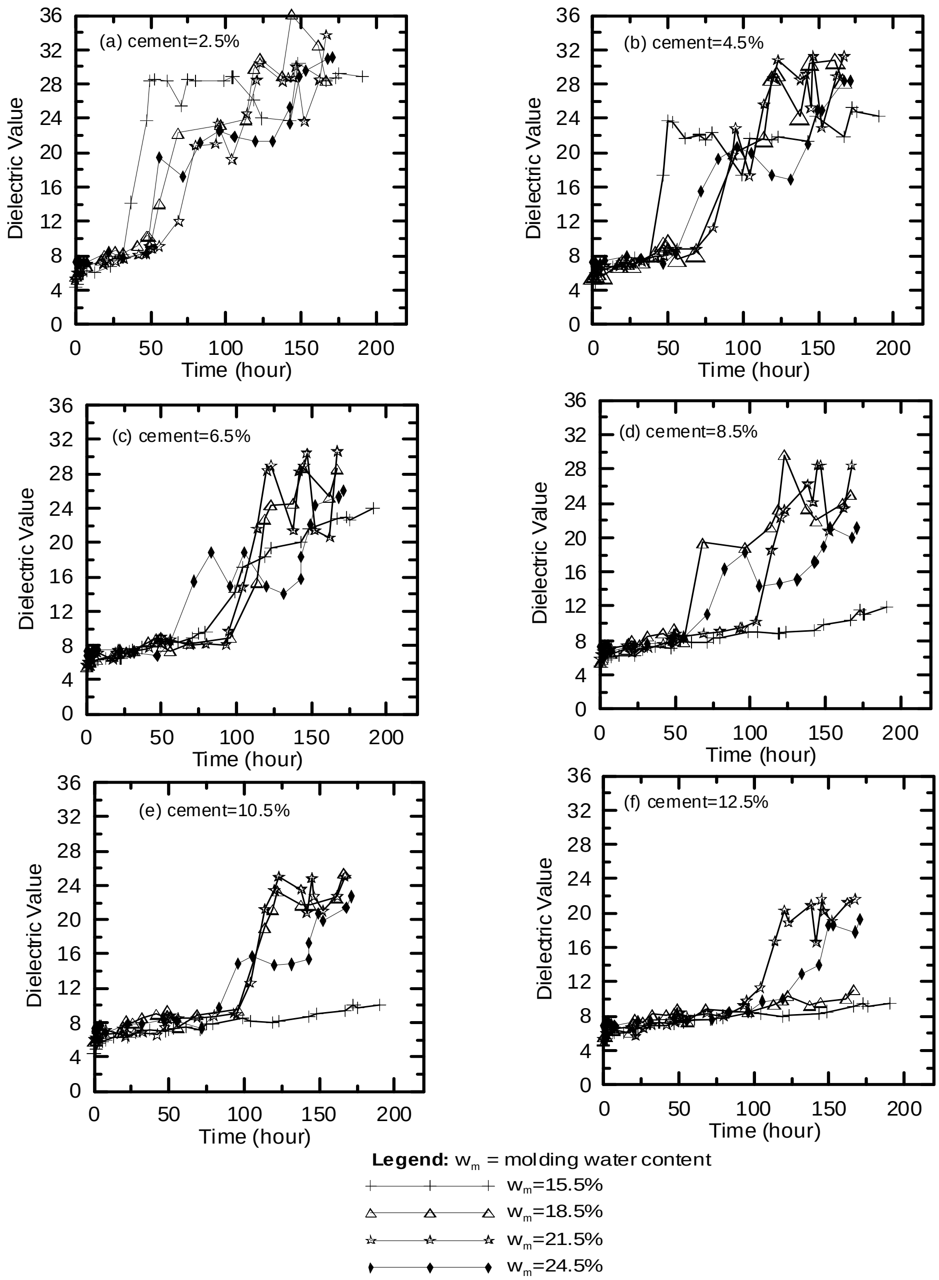


Figure 26

Variation of dielectric values with elapsed time for stabilized soil at various moisture contents: (a) cement content = 2.5 percent; (b) cement content = 4.5 percent; (c) cement content = 6.5 percent; (d) cement content = 8.5 percent; (e) cement content = 10.5 percent; (f) cement content = 12.5 percent

The influence of molding moisture and cement contents on the efficiency of cement in reducing the maximum DV of stabilized soil is further displayed in Figure 27. It follows that the water-cement ratio, *w/c* as defined before, largely controlled the maximum DV of stabilized soils, as shown in Figure 28. The maximum DV generally increased with the *w/c* ratio.

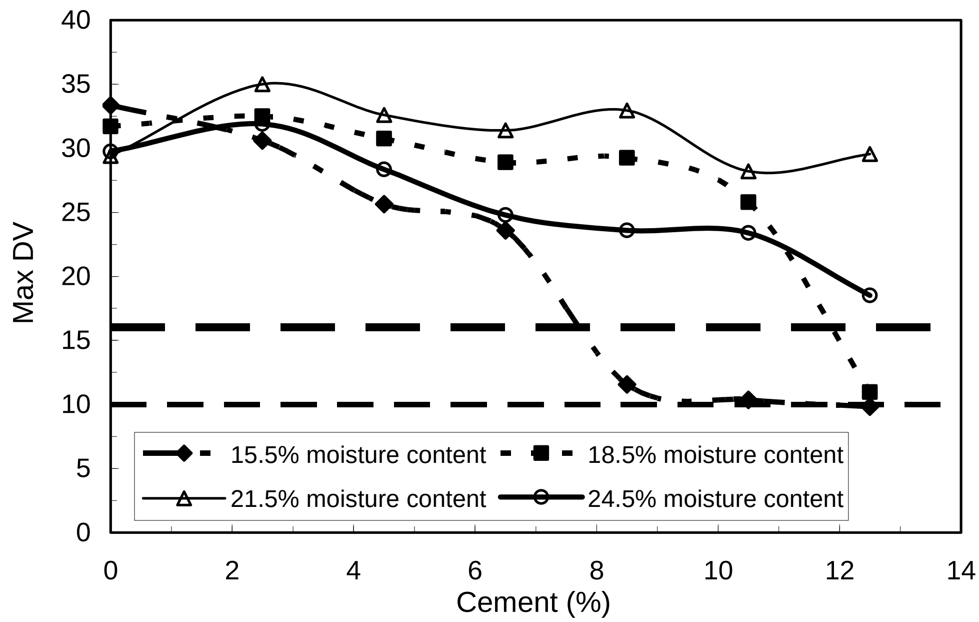


Figure 27

Variation of maximum dielectric value (DV) with cement contents for samples molded at various moisture contents

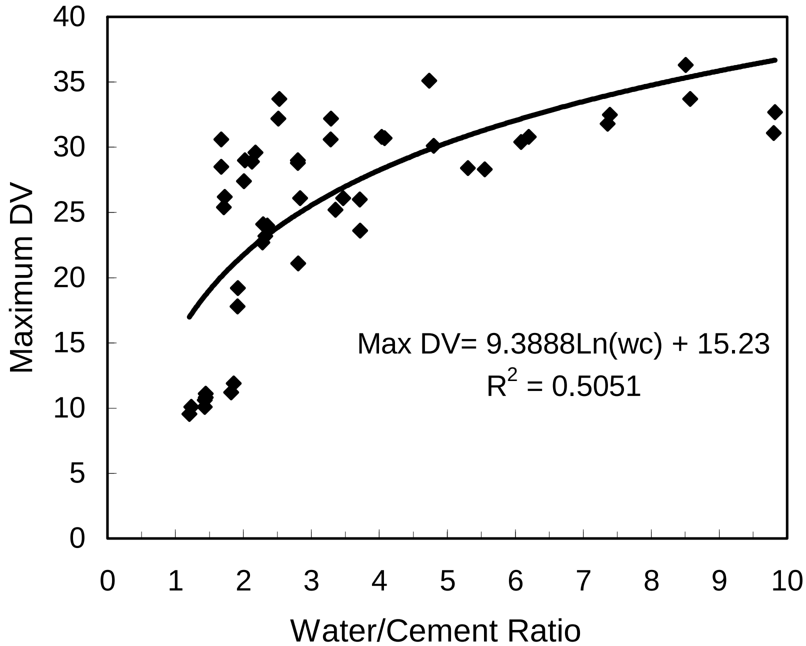


Figure 28
Maximum dielectric value versus water cement ratio

UCS of TS Sample versus Maximum DV. Figure 29 shows the correlation between the UCS of TS samples and their maximum DV. The UCS of TS samples generally decreased with the increase in the maximum DV, and this relationship was affected by molding moisture contents that controlled the dry unit weight of samples. The lower dry-unit-weight group formed the low boundary of the correlation band. The result also indicates that for a maximum DV of 20, the cement-stabilized soil had a UCS close to 150 psi.

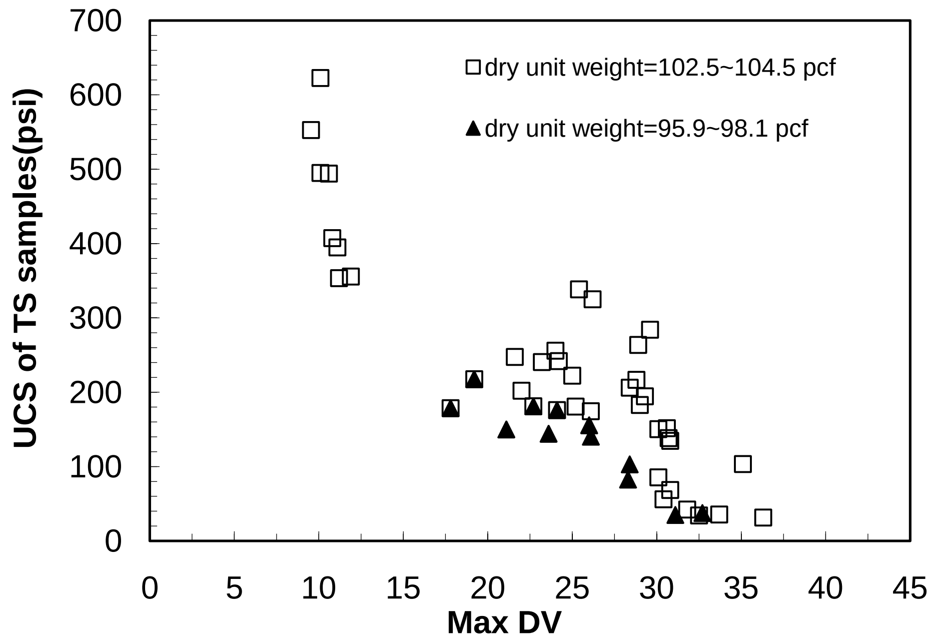


Figure 29
Relationship between UCS and maximum dielectric values

Water Cement Ratio versus UCS of TS Sample. Figure 30 shows the strong relationship between the ratio of water to cement content, *w/c*, versus the UCS of TS sample for all the tested samples. Therefore, the water-cement ratio can better predict the UCS of TS samples. In general, the UCS decreased as the water-cement ratio increased, most significantly when the water-cement ratio increased from 1.5 to 4.0. The lower dry-unit-weight group also formed the lower boundary of the correlation band that is probably attributable to their horizontally oriented microstructure. Figure 30 also suggests that the *w/c* ratio should be kept as low as possible in the mixture design of cement stabilization if water content is adequate for hydration and pozzolanic reactions between cement and soils to be stabilized.

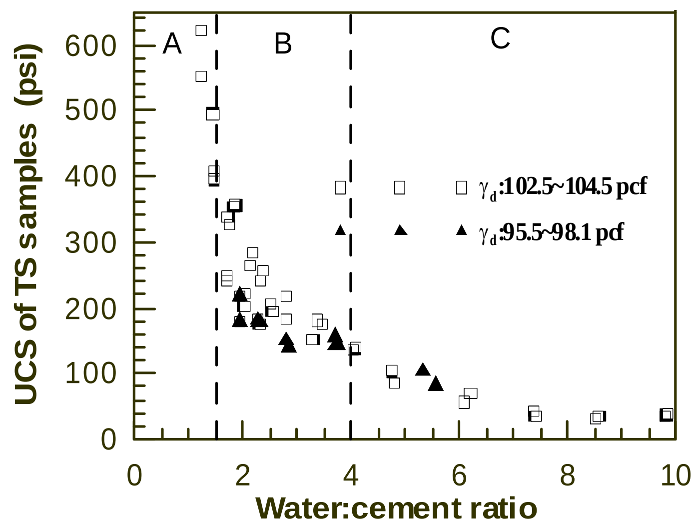


Figure 30
Residual UCS versus water/cement ratio

Seven-Day UCS. Similar to the previous discussion, cement content, molding moisture, and dry unit weight controlled the 7-day UCS of samples tested. The correlation of 7-day UCS with water-cement ratio, shown in Figure 31, has the same trend as that in Figure 30 except a wider variation for each water-cement ratio due to the wider range of dry unit weight in the prepared samples. As observed before, the data points with lower dry unit weights formed the lower boundary of the correlation band. Due to the lack of the soil, only three samples with 15.5 percent molding moisture content were tested for their 7-day UCS.

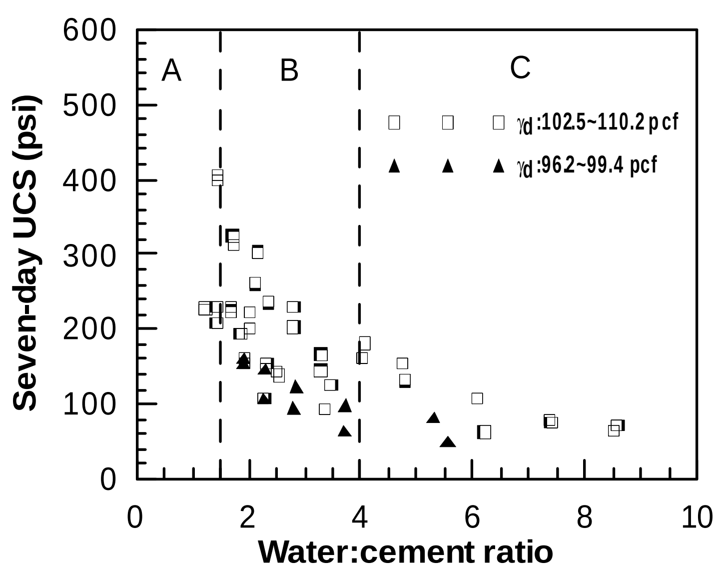


Figure 31
Correlation of 7-day UCS with water-cement ratio

TS samples and 7-day UCS samples were molded at the same moisture and cement contents but had different specimen sizes and wetting/drying history. (TS samples were oven dried and then put in 20 mm deep water for about 10 days while 7-day UCS samples were submerged in water for 4 hours before tested.) The UCS from these specimens are plotted against their water/cement ratios in Figure 32. Regardless of the difference between these two sets of samples, their UCSs generally follow a same decreasing trend with the increase in water/cement ratio.

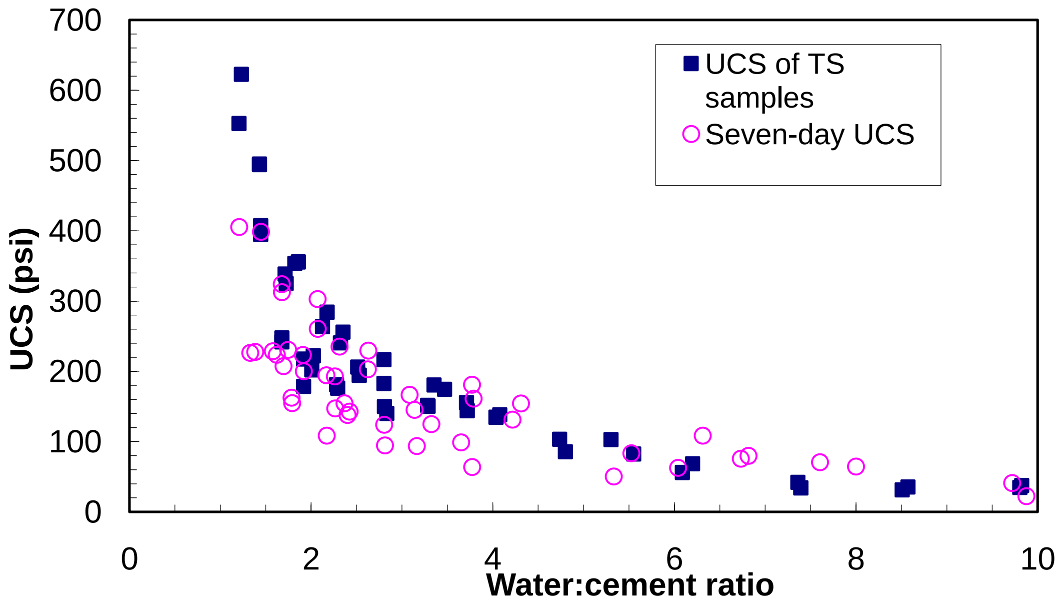


Figure 32
Correlation of UCS with water-cement ratio

Durability Test Results. Not all the specimens survived a 12-cycle wetting-drying durability test. Figure 33(a) shows that the soil-cement loss consistently decreased with the increase in cement dosages at a greater decreasing rate when cement dosages were low (from 2 percent to 4 percent). The required cement content should be at least 10.5 percent in order to meet the durability criterion of soil-cement loss (7 percent for CL soils) by [Portland Cement Association (PCA), 1992] specification. Figure 33(a) also indicates that different initial molding moisture contents (ranging from 15.5 percent to 24.5 percent) did not cause appreciable influence on the pattern of soil-cement loss.

For unbrushed samples, their maximum volumetric changes during the test, expressed as the percentage of samples' initial volumes, and volume change are plotted against cement contents, as shown in Figure 33(a) & (b). Apparently, at low cement content levels (smaller than 6 percent), the samples molded with higher moisture content experienced much higher volume changes. As cement contents are larger than 6 percent, all the samples had about the same level of volumetric strain that was smaller than 4 percent.

Figure 33(c) and (d) show the relationships among the soil-cement losses and maximum volumetric change of samples tested with water cement ratio. Both of them generally increased with the increase of water cement ratio in a either linear or non-linear pattern.

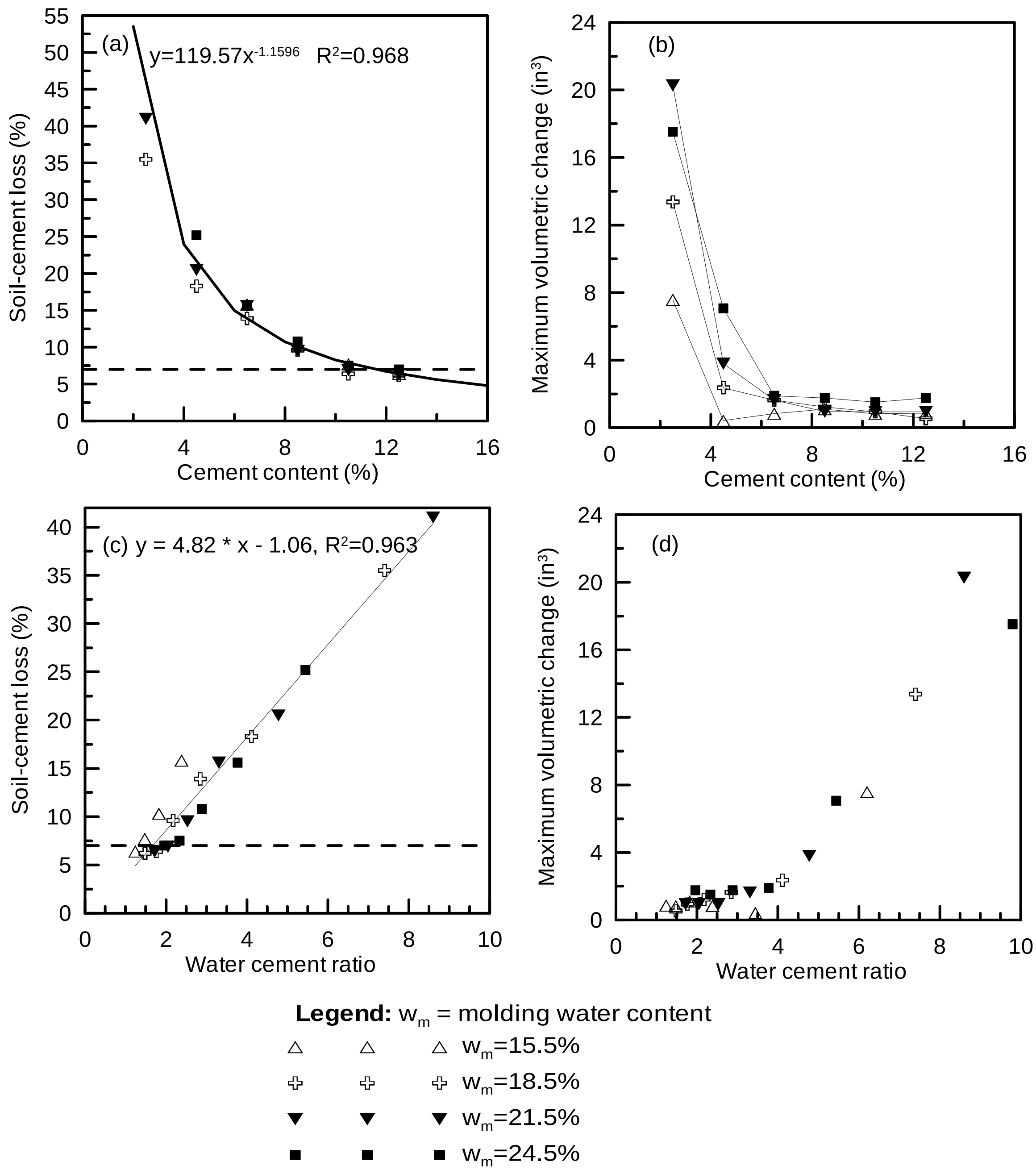


Figure 33

Results of wetting-drying durability tests: (a) soil-cement loss versus cement content; (b) maximum volumetric change versus cement content; (c) soil-cement loss versus water-cement ratio; and (d) maximum volumetric change versus water-cement ratio

Correlation among 7-Day UCS, Maximum DV, and Soil-Cement Loss

The soil-cement losses obtained from wetting-drying durability tests are plotted again in Figure 34 against the results from 7-day UCS tests. The soil-cement loss generally decreased with the increase in 7-day UCS but was also affected by the initial molding moisture contents, which is consistent with the results in Figure 33(c) and (d).

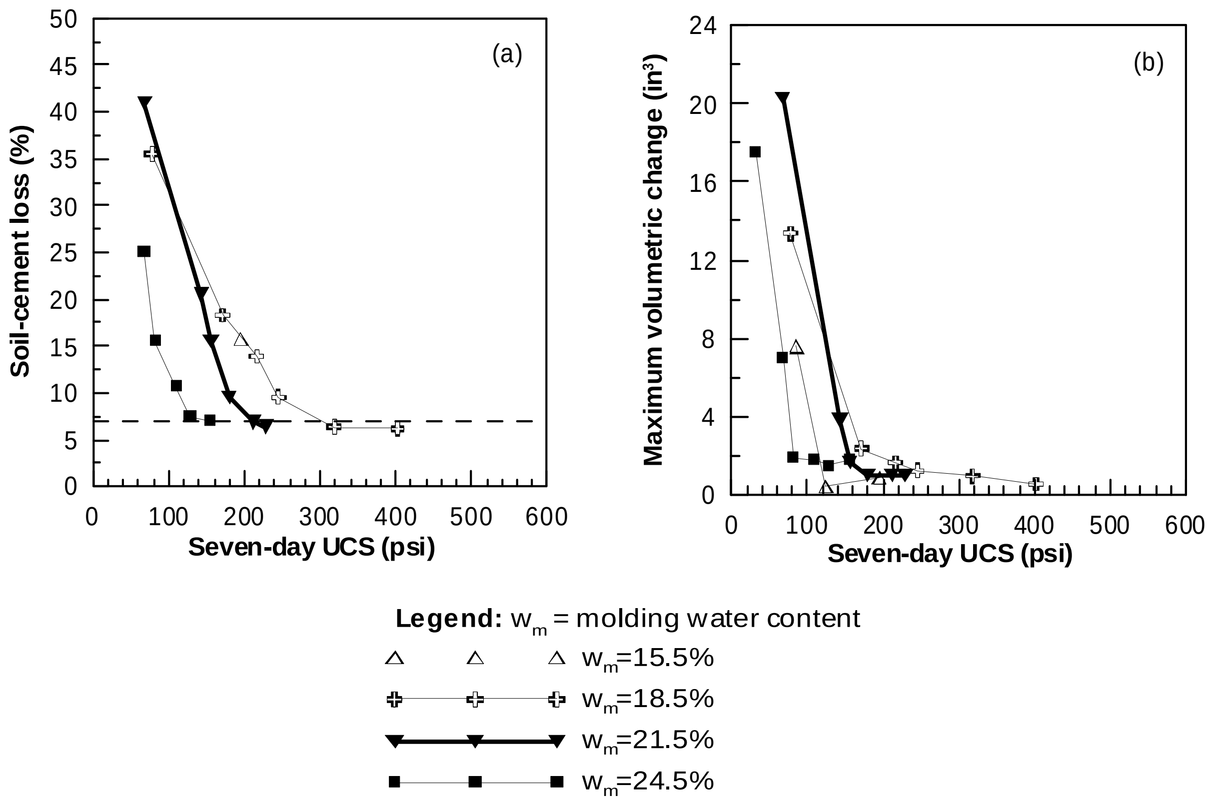


Figure 34

Seven-day UCS versus results from wetting-drying durability and tube suction tests: (a) soil-cement loss vs. 7-day UCS; and (b) maximum dielectric value vs. 7-day UCS

Figure 35(a) shows the correlations between the soil-cement loss and the maximum DV. The soil-cement loss criterion suggested by PCA (less than 7 percent) for soil-cement and the maximum dielectric value criterion (less than 10) proposed by (Scullion, 1997) for good aggregate bases are also superimposed in the figure, which divides the figure into four sub-zones (designated as i, ii, iii, and iv). For samples in Zone i, both criteria are satisfied and their long-term performance should be assured. For samples in Zone ii, only the PCA criterion is met but the DV criterion is not. For samples located within Zone iii, neither criterion is met; therefore, these samples are more likely to be water susceptible. It appears from Figure 35(a) that the DV criterion is more conservative than the soil-cement loss criterion. It should also be pointed out that the maximum DV is dependent on initial moisture content but the soil-cement losses are not. Therefore, it is desirable to mold samples for tube suction tests at appropriate molding moisture content. However, more research studies are required to determine which criterion is more accurate to predict long-term behaviors of soil-cement in the field.

Figure 35(b) shows that the maximum volumetric strains were relatively small (less than 4 percent) until the maximum DV exceeded 28. As the maximum DV was larger than 28, the maximum volumetric strain rose up to more than 30 percent, depending on initial molding moisture contents.

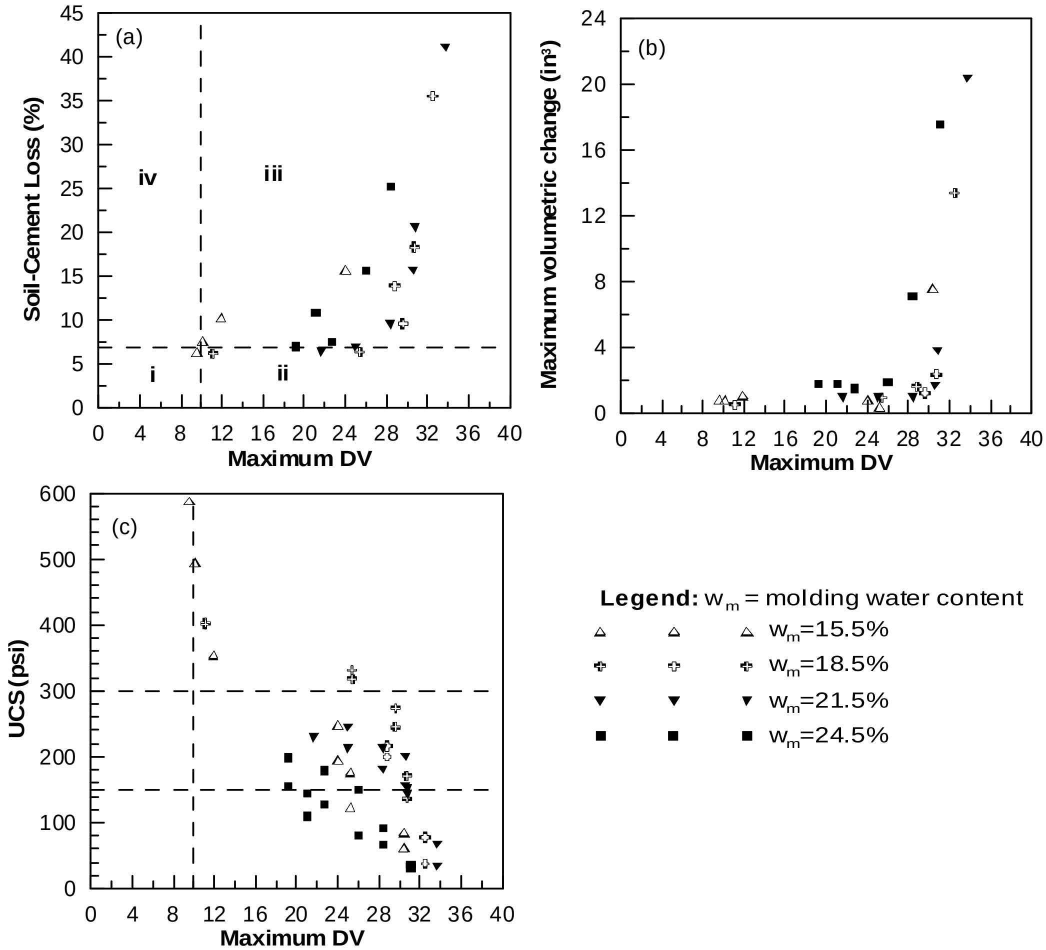


Figure 35

Maximum DV versus durability and UCS of stabilized soil: (a) maximum DV versus soil-cement loss; (b) maximum DV versus maximum volumetric change; and (c) maximum DV versus 7-day UCS

Figure 35(c) shows the correlation of the maximum DV with the 7-day UCS of the stabilized soil. The 7-day UCS DV generally declined with the increase in the maximum. The maximum DVs of 13 and 18 will secure the UCS of 300 psi and 150 psi, respectively. A 7-day UCS of 300 psi has been used by many state highway agencies to determine cement content in cement-soil stabilization mix design. The maximum DV of 10 will correspond to a much higher UCS of 406 psi, which was obtained at 18.5 percent molding moisture content and 12.5 percent cement content.

Resilient Modulus M_r of Cement-Stabilized Soils

Figure 36 shows the resilient modulus values of cement-stabilized soils in this study. They increase with the curing period from 1 day to 28 days with a majority of the hydration process complete in the initial 7 days. For example, the resilient modulus of cement-stabilized soils increased by 43 percent from 1 day to 7 days while that increased by 32 percent from 7 days to 28 days.

The resilient modulus values of cement-stabilized soils also increase with the increasing confining pressure. This is due to the decrease in dilatational properties and the increase in stiffness from the increasing confining pressure. The resilient modulus of cement-stabilized soils shows a mixed behavior, such as it remains constant, increases, or decreases with the increasing cyclic stress, as indicated in Figure 36. As the curing period increases (28 days) and the hydration process continues, the slope of the resilient modulus of the cement-stabilized soils with the cyclic stress changes towards a more positive value from a slope at the early stages of curing (7 days). This change of slope of the resilient modulus of a stabilized material with the cyclic stress may be considered as an indicator of the progress of the hydration process.

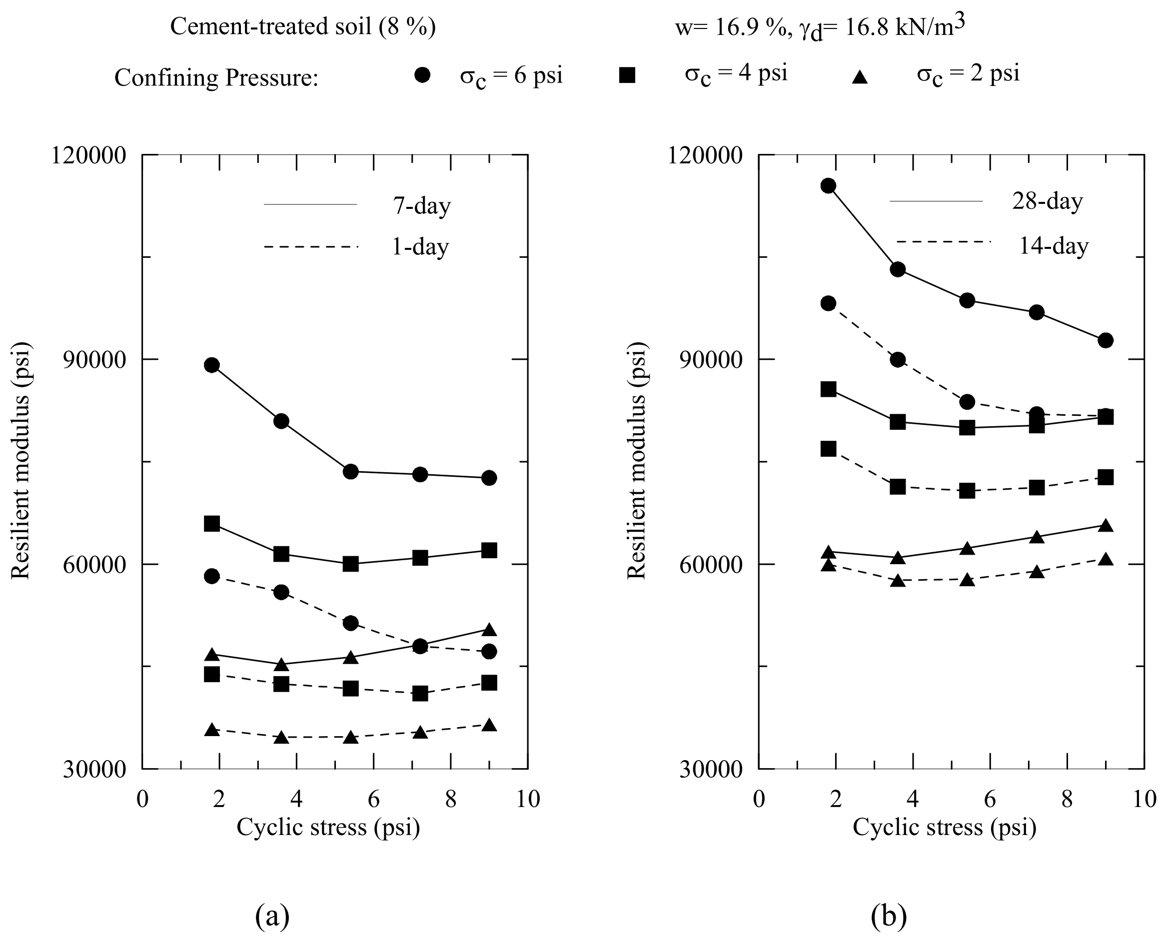


Figure 36
Resilient modulus of cement-stabilized soils (a) 1-day and 7-day curing
(b) 14-day and 28-day curing

Comparison of M_r of Treated and Untreated Soils

Figure 37 and Figure 38 compare the resilient modulus values of the cement-stabilized, lime-treated, and untreated soils. As shown in these charts, the resilient modulus values of cement-stabilized soils and lime-treated soils were higher than those of untreated soil. This implies that lime and cement treatments improve the subgrade soils.

As compared to a slight increase in resilient modulus of lime-treated soils with the curing period, the resilient modulus of cement-stabilized soils increased considerably with the curing period. This is because the hydration process in the cement-stabilized soils is faster and more dominant than the process in the lime-treated soils (within the same time period).

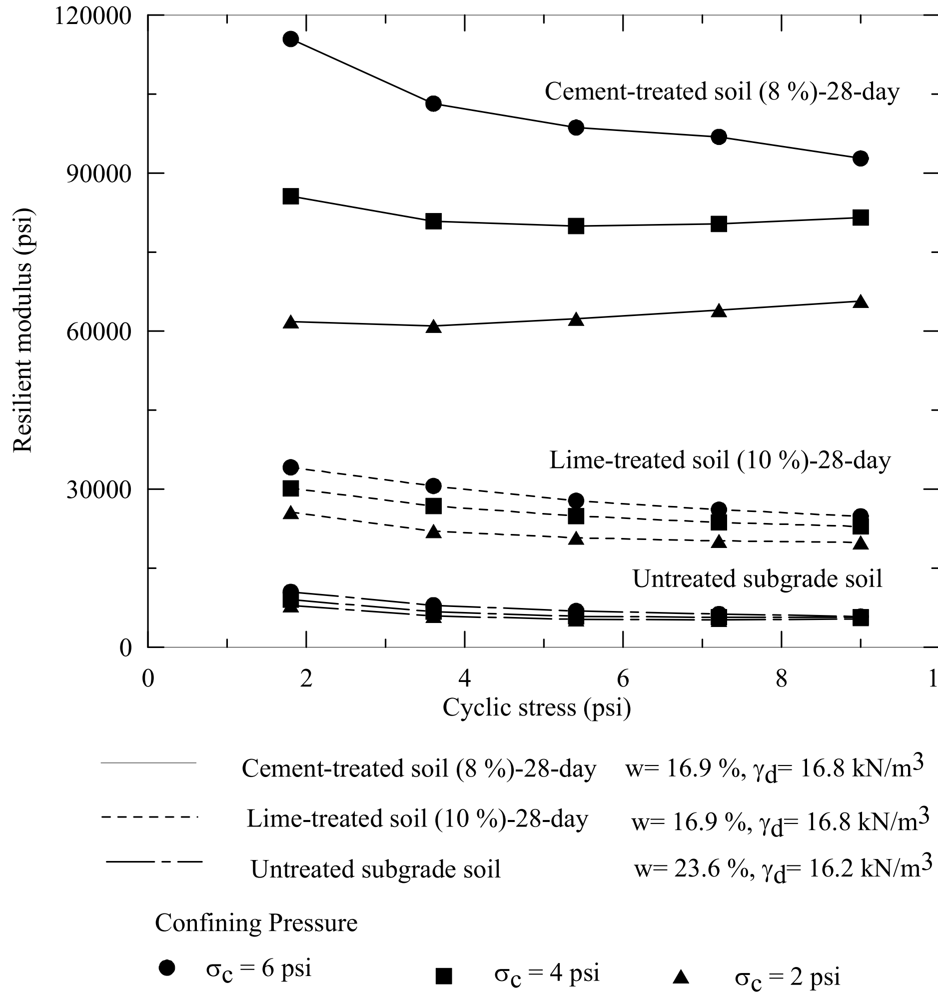


Figure 37
Resilient modulus of treated and untreated soils

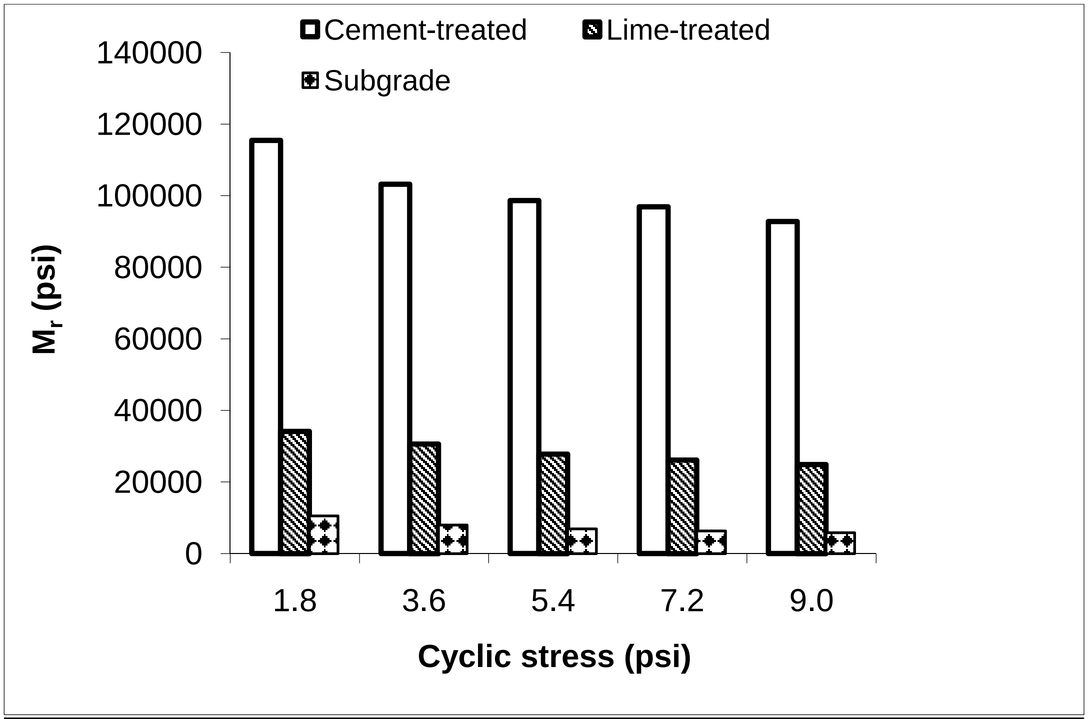


Figure 38
Resilient modulus of treated and untreated soils at confining pressure 6 psi

Permanent Deformation of Cement-Stabilized Soils

Figure 39 presents the permanent deformation of the cement-stabilized soils with the number of load cycles. The permanent deformation of cement-stabilized soils increases with the increasing number of load cycles.

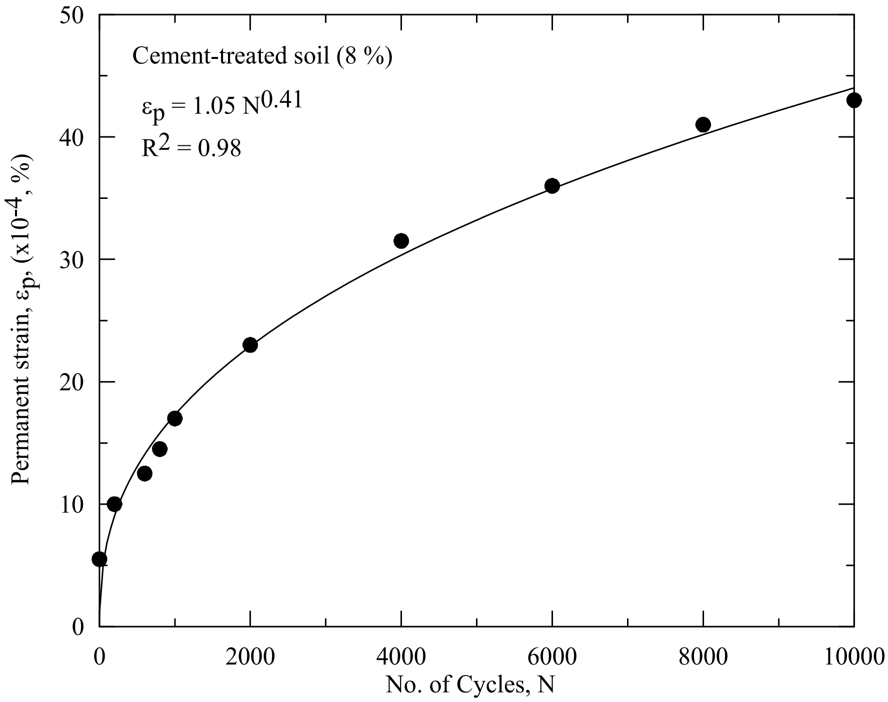


Figure 39
Permanent strains of cement-stabilized soils at 28-day curing

Lime Stabilization

Moisture-Density and UCS Tests

Soil III (heavy clay) moisture-density results for raw soil and soil with different lime percentages are shown in Figure 40. All lime percentages are shown as percentage by weight. A shifting trend can be seen as the curves' optimum moisture contents increase and dry unit weights decrease with higher and higher additive percentages. This increase in lime content causes the moisture density curves to shift down and to the right.

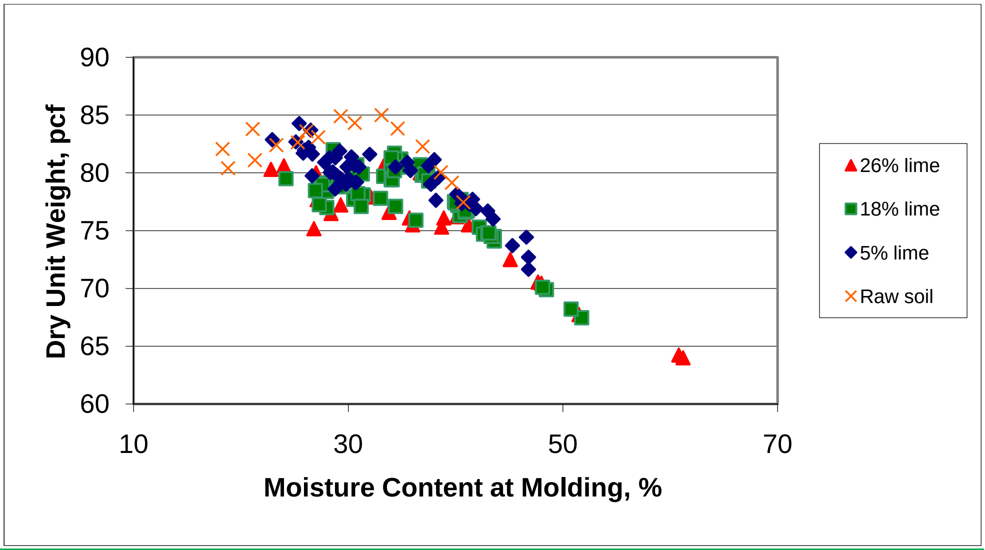


Figure 40
Dry unit weights at molding moisture

Curing Method Effects on UCS. A comparison of the various curing methods' effects on unconfined compressive strength is shown in the following figures (Figure 41 and Figure 42). For the samples mixed with 5 percent lime, the 7-day heated bath produced the highest strengths with the 28-day breaks cured in the 100 percent room, the 7-day 100 percent room, and the 7-day oven samples' strengths declining, respectively. The different curing methods resulted in roughly a difference of 40 psi from the high to low strength results at any particular molding moisture at the 5 percent additive concentration with larger exceptions at the peak strengths.

The sample strengths generally produced a curve that peaked around 37 percent molding moisture. Strengths generally declined as the molding moisture strayed from the optimum.

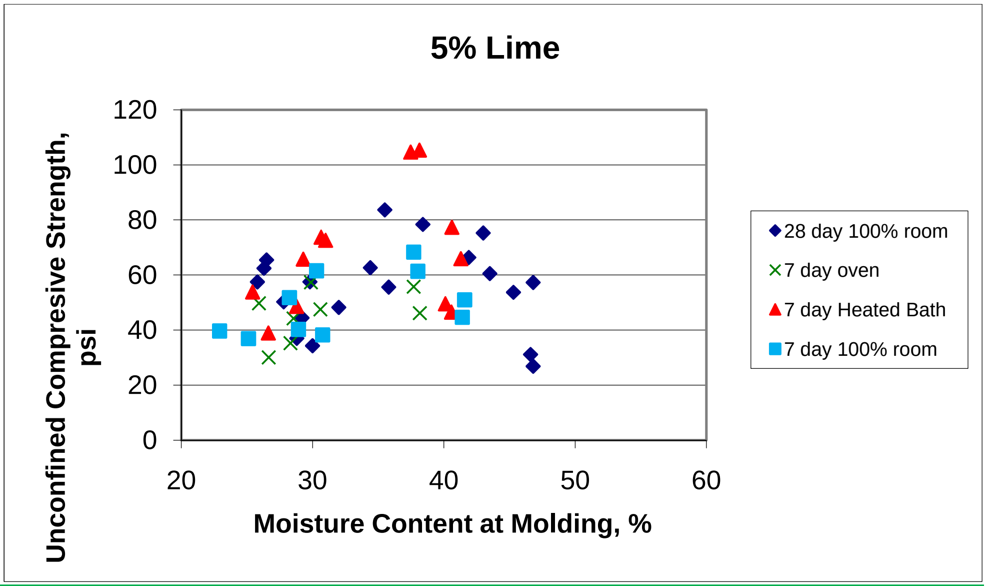


Figure 41
Strengths vs. molding moistures at 5 percent lime

For the samples mixed with 18 percent lime (Figure 42), the 7-day oven curing produced the highest peak strength, but in general the 7-day oven, 7-day heated bath, and 28-day 100 percent room curing followed similar trends. In contrast, the 7-day 100 percent curing produced the lowest strengths. The strength range at a particular molding moisture varied more at the lower moistures than at higher molding moistures. The additional lime (18 percent vs. 5 percent) produced higher strengths, especially at moisture contents nearer the optimum. Strengths generally declined as the molding moisture strayed from the optimum.

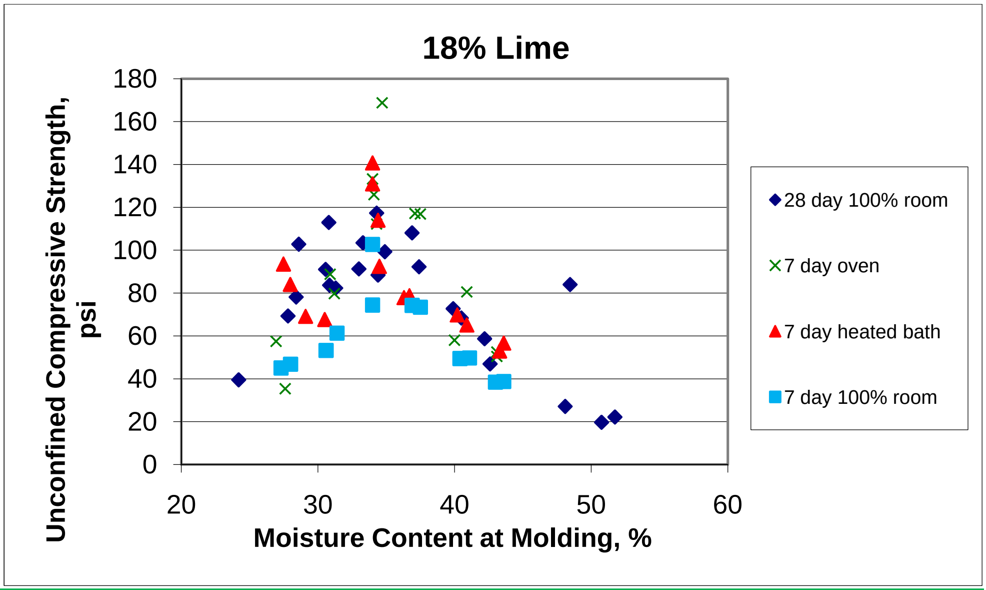


Figure 42
Strengths vs. molding moistures at 18 percent lime

Density and UCS. In Figure 43 and Figure 44, the dry unit weights are plotted against the unconfined compression strengths for different curing methods. In Figure 43, only a slight trend appears to be in those cured for 28 days in the 100 percent room, where an increase in dry unit weight offers an increase in strength.

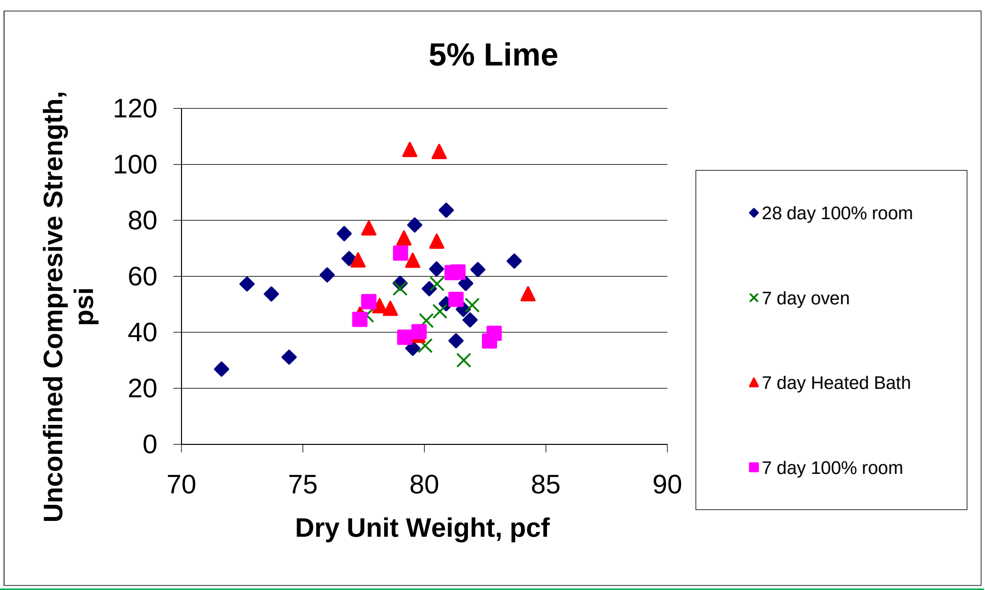


Figure 43
Strengths vs. dry unit weight at 5 percent lime

In contrast, those samples mixed with 18 percent lime (Figure 44) show a relationship of increasing dry unit weights with increasing strengths with strengths peaking at roughly 80 pcf.

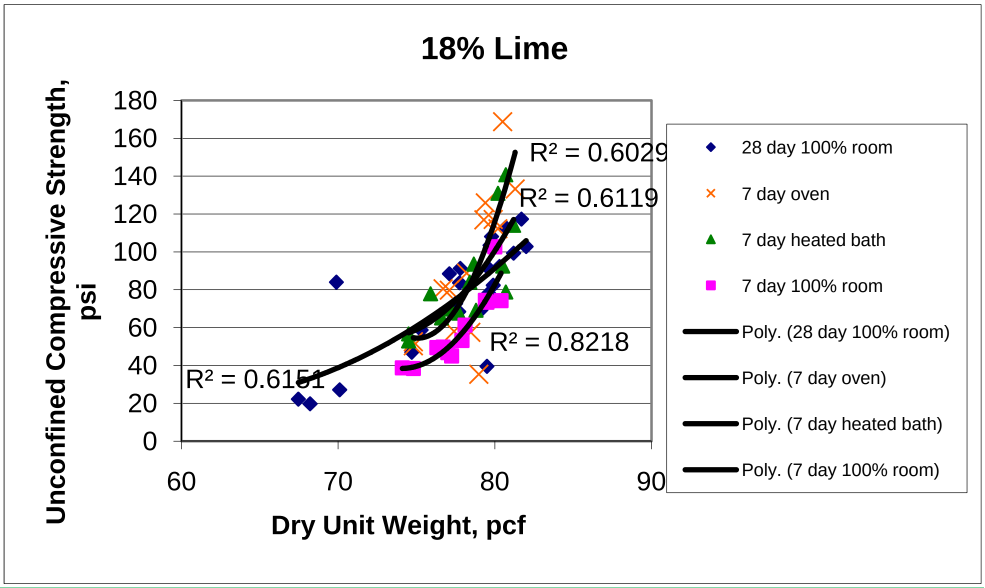


Figure 44
Strengths vs. dry unit weight at 18 percent lime

By plotting the 28-day 100 percent room curing results for the different additive percentages (5 percent, 18 percent, and 26 percent) against the raw soil (Figure 45), researchers observed an increase in strength with additive percentage. The difference, however, between the higher additive percentages is less apparent.

Effects of Molding Moisture. The effect of molding moisture is also apparent from Figure 45 in that the curves peak at the optimum moisture of the raw soil and strengths generally decline away from the optimum.

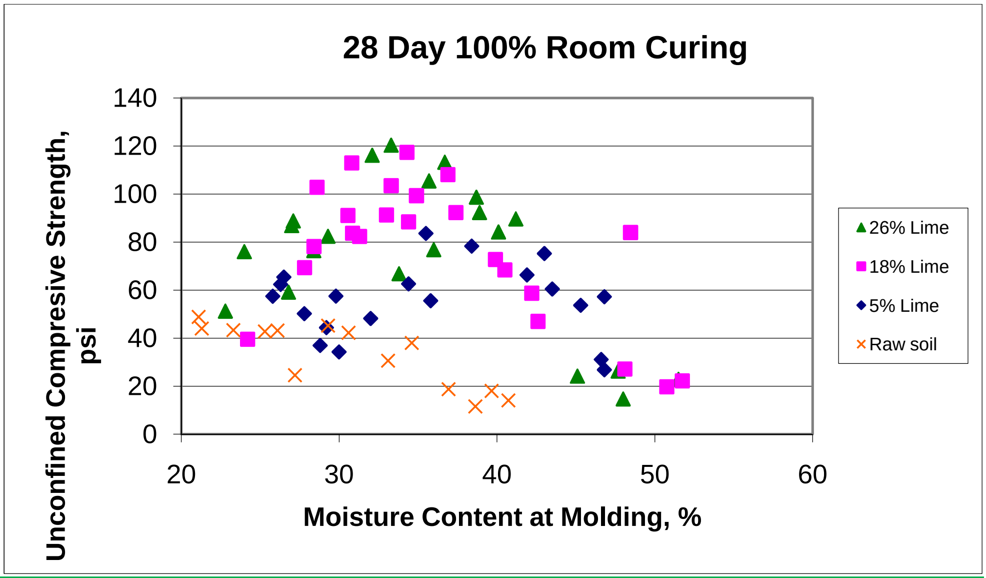


Figure 45
28-day strength vs. molding moisture

Figure 46 shows general trends like Figure 44 between the relationship of increasing dry unit weights with increasing strengths with strengths peaking at roughly 80 pcf. Higher lime percentages produced higher strengths, though the 18 percent and 26 percent followed roughly the same trend.

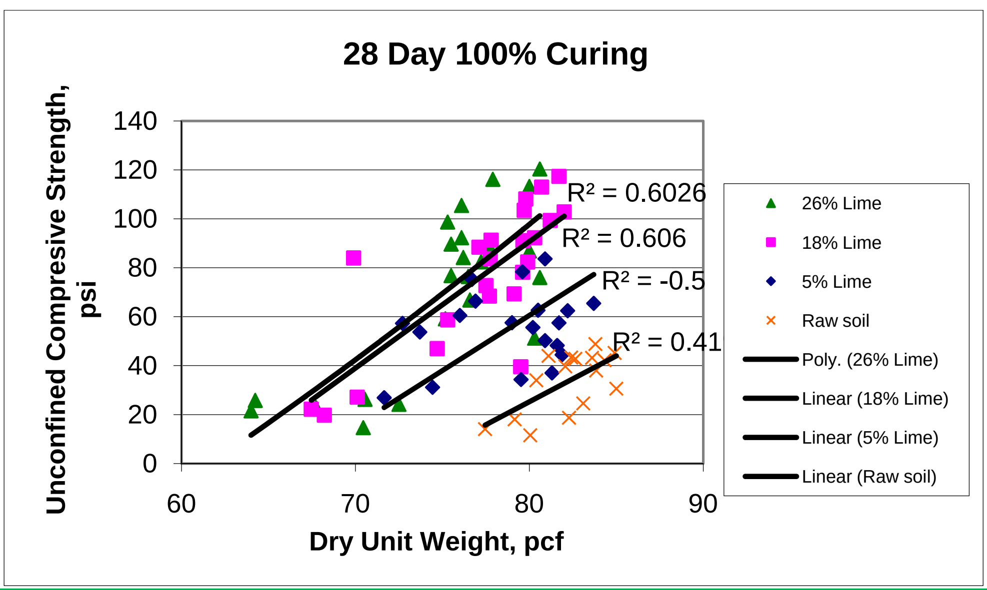


Figure 46
28-day strength vs. dry unit weight

Overall, laboratory strengths of 150 psi were attainable with the right combination of lime, water, and curing techniques.

Comparison and Other Tests

PH Values of Pure Lime based on the E&G Method. The E&G method detailed in ASTM D 6276 utilizes a pH value of 12.4 as the limit, identifying the necessary lime proportion requirement. During the LTRC laboratory testing, after calibrating the probe with pH buffers of 7, 10, and 12, the pH of the raw hydrated lime and water solution was measured. The 32 values were roughly normally distributed with values ranging from 12.500 to 12.750 with a standard deviation and median of 0.05 and 12.627, respectively. The distribution of pH measurements taken on the raw lime-water solution are shown in Figure 47. Though slightly higher than expected, the values set a calibration with the pH meter readings relative to the known pH value of the lime.

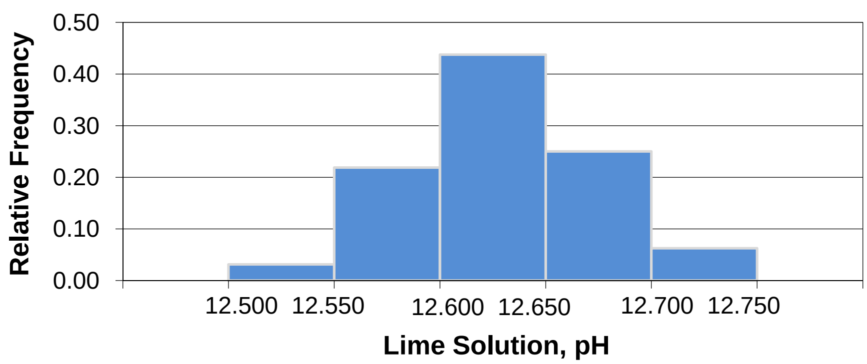


Figure 47
Lime pH histogram

The calibration process of the pH probe(s) was repeated many times with many different buffer solutions in an attempt to acquire the generally accepted lime pH value of 12.4. The average pH value of 12.6 forced an alternative method for determining the lime proportion requirement. ASTM D 6276 provides a check of the pure lime pH and allows the shift to higher pH values since the lime percentage where the pH value of the lime-soil solution levels off (at saturation) is more critical than the actual pH value. See Figure 48.

Normalized pH. Early in the study, a clayey soil was repeatedly tested with a series of lime percentages. Interestingly, different pH measurements for the soil-lime mixtures of equal lime contents were obtained for a series of tests; see Figure 48 for the typical asymptotic relationship between increasing lime percentage and the point of saturation.

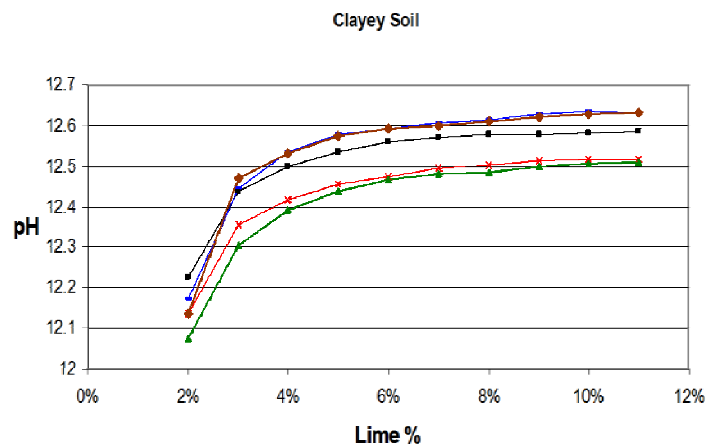


Figure 48
Eades & Grim pH results – clayey soil

As stated, the pH values with the meter were higher than anticipated. To account for this and reduce the variability in the pH measurements, the use of a normalized pH value was attempted. The normalized pH is defined as the ratio of pH the soil-lime mixture to the pH reading of the pure lime solution. ASTM D 6276 requires a “sixth specimen” representing the pure lime solution, which was collected at the beginning of each test for consistency. While the normalized value approaches 1.0, the pH of the soil-lime mixture approaches the pH of pure lime solution. Where 12.4 is the limiting value and the divisor, 12.45 is the pH of an agitated calcium hydroxide slurry (2 percent lime-water solution). The clayey soil values in Figure 48 have been re-plotted as normalized values in Figure 49.

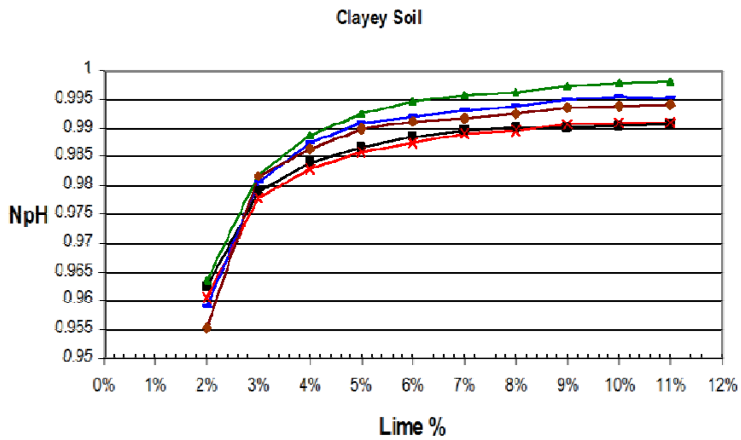


Figure 49
Eades & Grim normalized pH results – clayey soil

Comparing Figure 48 with Figure 49, it can be determined graphically that a normalized pH does control a portion of the variability as the series of plots are determined by the new vertical scale in Figure 49. Since the variance is not completely eliminated by normalizing the curve, it can be concluded that the variance is not consistent over the series of pH measurement. In addition, by normalizing the data, researchers have a comparison between the pure lime slurry and the soil-lime mixture. The normalized pH parameter can be used as an indicator in determining the lime proportion requirement since it focuses on the precision of the measurement identifying where the plot flattens to two successive pH points under increasing lime percentages.

The difficulty in determining pH values of such high concentrations is also evident by the results of the pH testing. Though common, the E&G method requires fresh lime, expensive pH equipment in top working shape, and a skilled technician.

Lime Determinations and Activity Class. A series of soil samples were collected and tested to determine their basic soil properties. Specifically, the liquid limit, plasticity index, clay content, and activity were determined. These results are shown in Table 13.

The range of LLs and PIs for the sample collection vary from 31 percent to 153 percent and 10 percent to 125 percent. Clay contents, included in Table 13, show ranges from 23 percent to 80 percent. The activity for each sample was then calculated and ranged from 0.43 to 1.98. The A, B, and C suffixes in Table 13 identify soil groups tested: A—inactive, B—normal, and C—active. Samples with M suffixes denote soils modified with sodium rich montmorillonite to create samples with the desired activity. Based on these values, the samples were labeled with their activity, so the first 14 samples in Table 13 are Inactive (A). Samples 1B through 6B were normal and the remaining samples, 1C-M through 3C-M were active.

The next column in Table 13 lists the lime saturation percentage, which represents the optimum amount (percentage) of lime necessary from the E&G test. The majority of the E&G tests were conducted with a series of lime contents ranging from 2 percent to 6 percent. The amount of lime used in samples labeled with NA, was generally less than 6 percent, and not enough to reach saturation per the E&G method. Therefore, the amount required for saturation is unknown [higher than the percentages initially setup], and not determined at the time of testing. The values ranged from 4 percent to greater than 11 percent additive.

Table 13
Clay activity & lime percentage results

Sample	Liquid Limit, %	Plasticity Index, %	Clay Content, %	Activity	Saturation, % Lime	Army PI Wet Method
1A	31	10	23.5	0.43	**6**	**2.0**
2A	58	33	75.0	0.77	**NA**	**4.5**
3A	53	29	63.0	0.46	**9**	**4.0**
4A	41	21	44.5	0.47	**>6**	**3.0**
5A	38	19	37.0	0.51	**6**	**3.0**
6A	44	21	41.0	0.51	**NA**	**3.0**
7A	62	38	62.0	0.60	**>6**	**5.0**
8A	75	48	75.0	0.64	**10**	**6.0**
9A	84	53	80.0	0.66	**>6**	**7.0**
10A	80	50	76.5	0.65	**>6**	**6.5**
11A	66	41	61.5	0.67	**>6**	**5.5**
12A	64	39	57.0	0.68	**NA**	**5.5**
13A	55	34	50.0	0.68	**>6**	**4.5**
14A	57	36	49.5	0.73	**>6**	**5.0**
1B	72	47	60.0	0.78	**>6**	**6.0**
2B	72	49	60.5	0.81	**9**	**6.5**
3B	87	62	75.0	0.83	**6**	**8.0**
4B-M	65	45	48.5	0.93	**4**	**6.0**
5B-M	95	69	74.0	0.93	**7**	**>8.0**
6B-M	104	79	77.0	1.03	**10**	**>8.0**
1C-M	139	119	60.0	1.98	**>11**	**>8.0**
2C-M	134	106	70.0	1.51	**NA**	**>8.0**
3C-M	153	125	76.0	1.64	**10**	**>8.0**

Figure 50 shows the relationship between the plasticity index, liquid limit, clay content, and activity level. A similar trend can be seen in both the liquid limit and clay activity. Yet, though the clay content roughly follows the same trend at lower PI levels, the trend falls off with higher PIs.

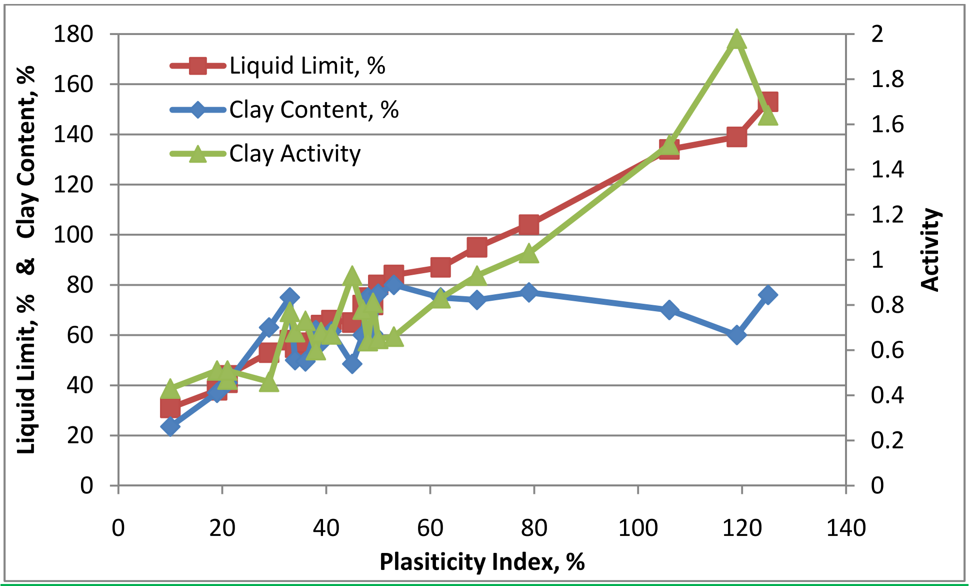


Figure 50
Clay content, activity, and Atterberg limit plot

The last column in Table 13 represents the required lime percentages for the tested soils as determined by the PI - Wet Method. The results for the E&G method were more variable but had higher recommended percentages at lower plasticity indices. The PI-Wet method mirrors the trend of the liquid limit of the soil though the method is limited by maximum a PI of 65 and a maximum percent lime of 8 percent hydrated lime based on dry weight. These results, the E&G results for the same samples, and their Liquid Limit are plotted in Figure 51. The PI-Wet method chart is only part of a method involving a series of steps to determine design lime contents. Other steps within the Army TM 5-822-14 (AFJMAN 32-1019) procedure include the E&G method, moisture density tests, and durability tests.

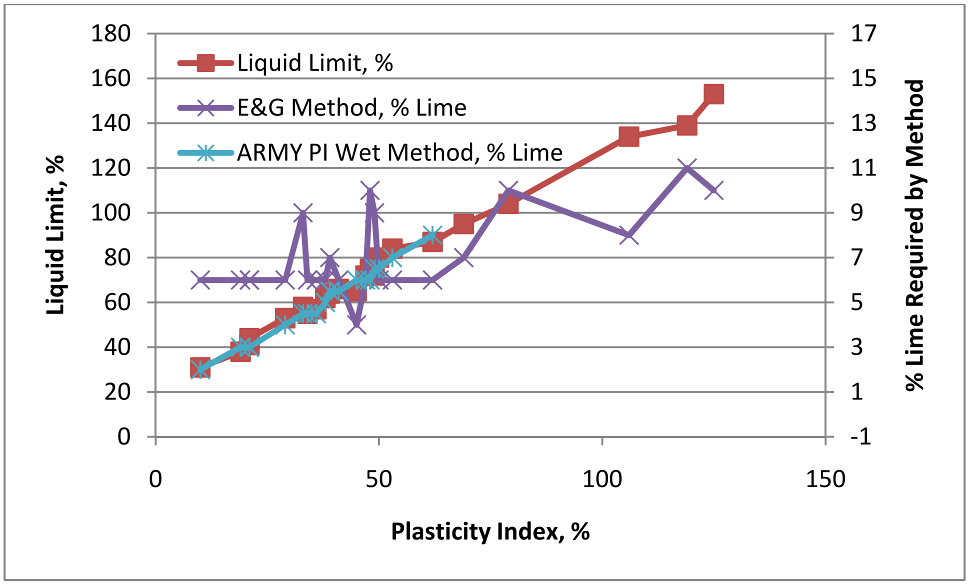


Figure 51
Lime percentage methods

The attempts to develop a new simple correlation were not completely effective. The LADOTD method, TR 416, though only for modification, as evident by the reduction in plasticity index, is simple and effective and requires minimal equipment and general soils knowledge. However, this method does not address the required lime for stabilization.

Though slightly more complicated and requiring pH equipment, the E&G method appears to be the most common method to ensure adequate additive for saturation and higher strengths. Based on the findings, the method appears rational and meets the necessary lime for saturation, which is likely higher than our current LADOTD TR method.

A combination of methods may be the best resolution, where the existing LADOTD TR 416 method can continue for the lime working table and modification applications. For stabilization (300 psi) and treatment (150 psi) applications, the PI Wet Method can be used to determine an initial lime percentage for the E&G method when strength goals are required. Indiana DOT utilizes the E&G lime determination and other methods to meet their modification and stabilization needs. A draft methodology for lime determination is attached as an appendix. LADOTD should evaluate this draft and incorporate changes into a design guide for their design needs.

Resilient Modulus of Lime-Treated Soils

Variation of M_r of Lime-Treated Soils with Curing Time. The resilient modulus values of lime-treated soils are presented in Figure 52. Similar to the cement-stabilized soils, the resilient modulus values of lime-treated soils increased with the longer curing periods (from 1 day to 28 days). The resilient modulus of lime-treated soils increased by 28 percent from 1 day to 7 days while that increased by 10 percent from 7 days to 28 days.

Variation of M_r of Lime-Treated Soils with Stress Levels. Similar to the cement-stabilized soils, the resilient modulus values of lime-treated soils increase with the increasing confining pressure (Figure 52). The resilient modulus of lime-treated soils decreases with the increasing cyclic stress (Figure 52). This is a typical behavior of cohesive soils though the on-site material has a mid to low plasticity index of 10. Due to the pozzolanic reaction process in the lime-treated soils (slower than cement hydration), the resilient modulus of this material with cyclic stress behaved like a cohesive material at early stages of the curing period. As the curing period increases (28 days), hence the pozzolanic reaction process continues, the slope of the resilient modulus of the lime-treated soils with the cyclic stress changes towards a more positive value from a negative value at the early stages of curing (7 days). As pointed out earlier, it is observed that the slope of the resilient modulus of the lime-treated soils with the cyclic stress depends on the progress of the pozzolanic reaction process.

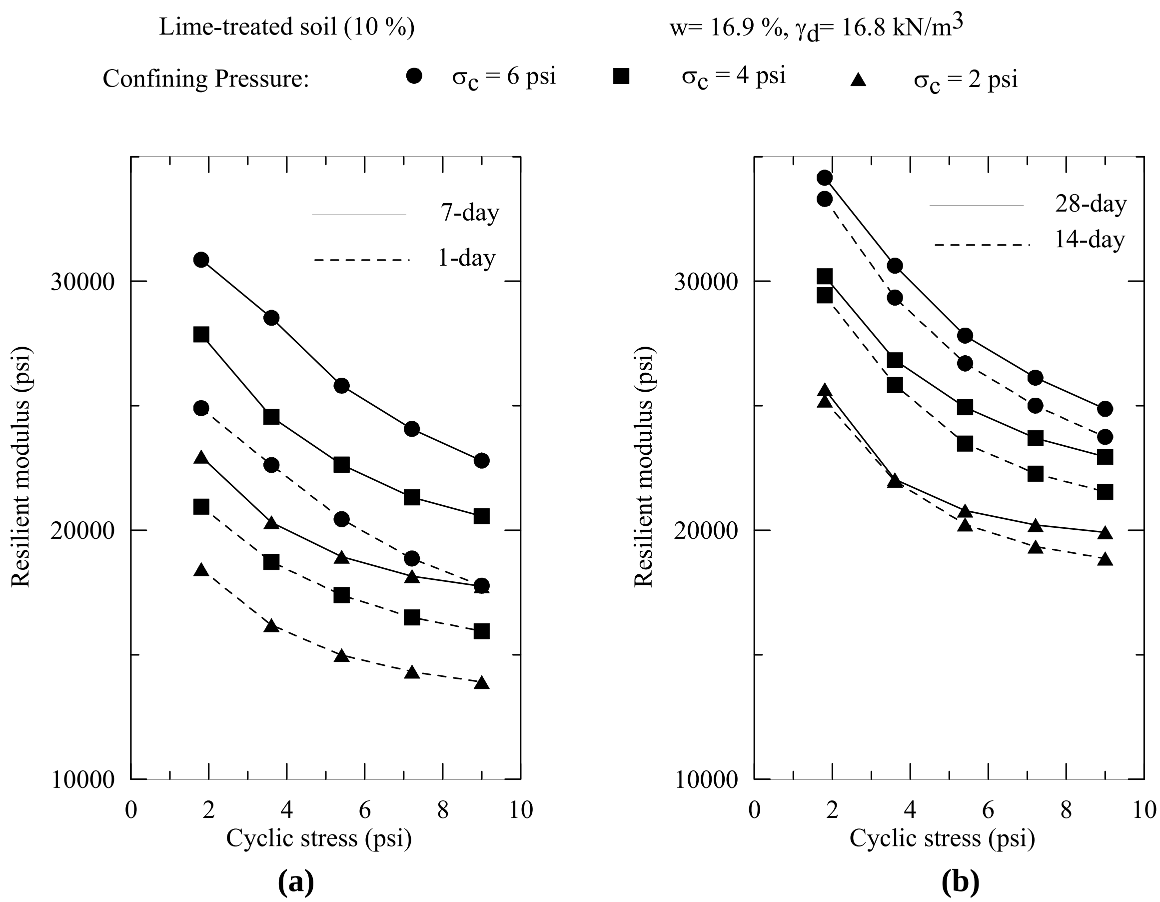


Figure 52
Resilient modulus of lime-treated soils (a) 1-day and 7-day curing
(b) 14-day and 28-day curing

Comparison of M_r of Treated and Untreated Soils. Figure 37 and Figure 38 compares the resilient modulus values of the cement-stabilized, lime-treated, and untreated soils. As shown in Figure 37 and Figure 38, the resilient modulus values of cement-stabilized soils and lime-treated soils were higher than those of untreated soils. This implies that lime and cement treatments improve the subgrade soils.

As compared to slight increment in resilient modulus of lime-treated soils with the curing period, the resilient modulus of cement-stabilized soils increased considerably with the curing period (Figure 36 and Figure 52). This is because the hydration process in the cement-stabilized soils is faster than the pozzolanic reaction process of lime-treated soils (within the same time period).

The rate of increase in resilient modulus of lime-treated soils ranged from 550 psi/day to 850 psi/day for 1 day to 7 days; whereas, that for the cement-stabilized soils ranged from 2000 psi/day to 4400 psi/day. The rate of increase in resilient modulus of lime-treated soils ranged from 103 psi/day to 157 psi/day for 7 days to 28 days; whereas, that for the cement-stabilized soils ranged from 550 psi/day to 850 psi/day. The percentage increase in the resilient modulus of cement-stabilized soils with respect to the resilient modulus of subgrade soils ranged from 1000 percent to 1500 percent; whereas, that of the lime-treated soils ranged from 225 percent to 325 percent (Figure 53). Therefore, the rate of increase in the resilient modulus of the cement-stabilized soils is far greater than that of the lime-treated soils (Figure 53). The lowest resilient modulus was observed in the untreated subgrade soils, which do not plot in the scale of the figure. The cement-stabilized soils achieved the highest resilient modulus followed by the lime-treated soils did (Figure 37). The higher the resilient modulus, the stiffer and better the material is for pavement layers. Therefore, the performance of the cement-stabilized soils is better than that of both lime-treated and untreated soils. However, shrinkage cracks in the cement-stabilized soils can be a major drawback of using it as a structural layer. The lime-treated soils perform better than untreated soils do.

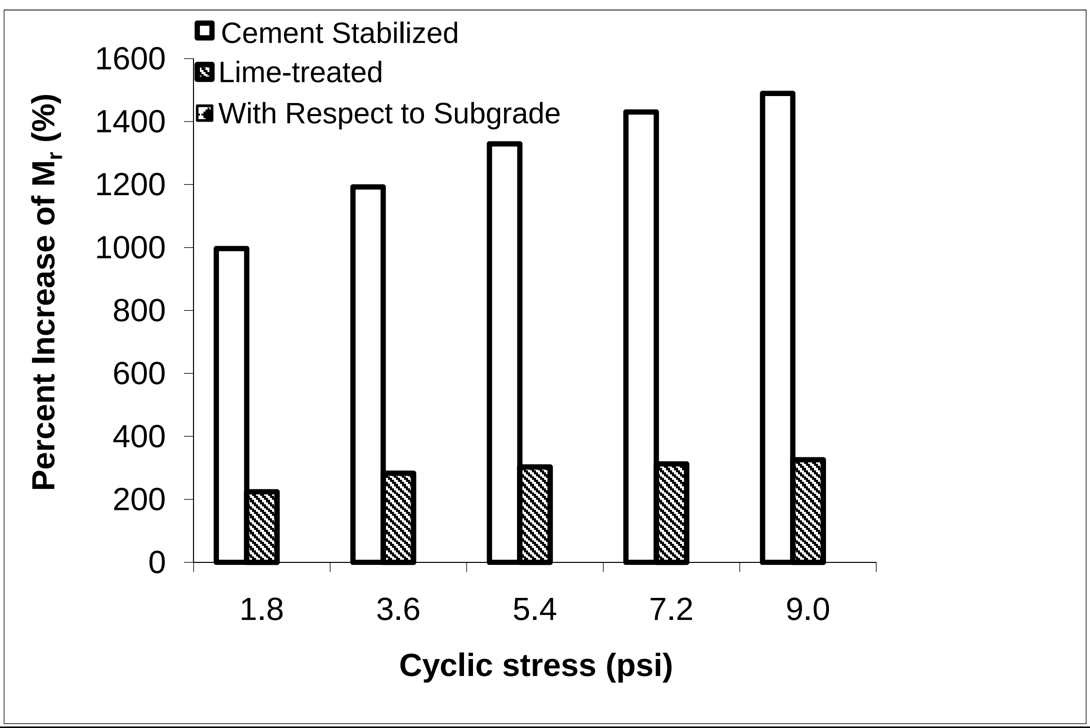


Figure 53
Increase in resilient modulus at confining pressure 6 psi

Permanent Deformation of Lime-Treated Soils

Figure 54 presents the permanent deformation of the lime-treated soils with the number of load cycles. The permanent deformation of lime-treated soils increases with the increasing number of load cycles, but with a gradually decreasing rate of increase. For lime-treated soils, the relationship between permanent strain (ε_p) and number of cycles (N) can be expressed as shown in the figure below.

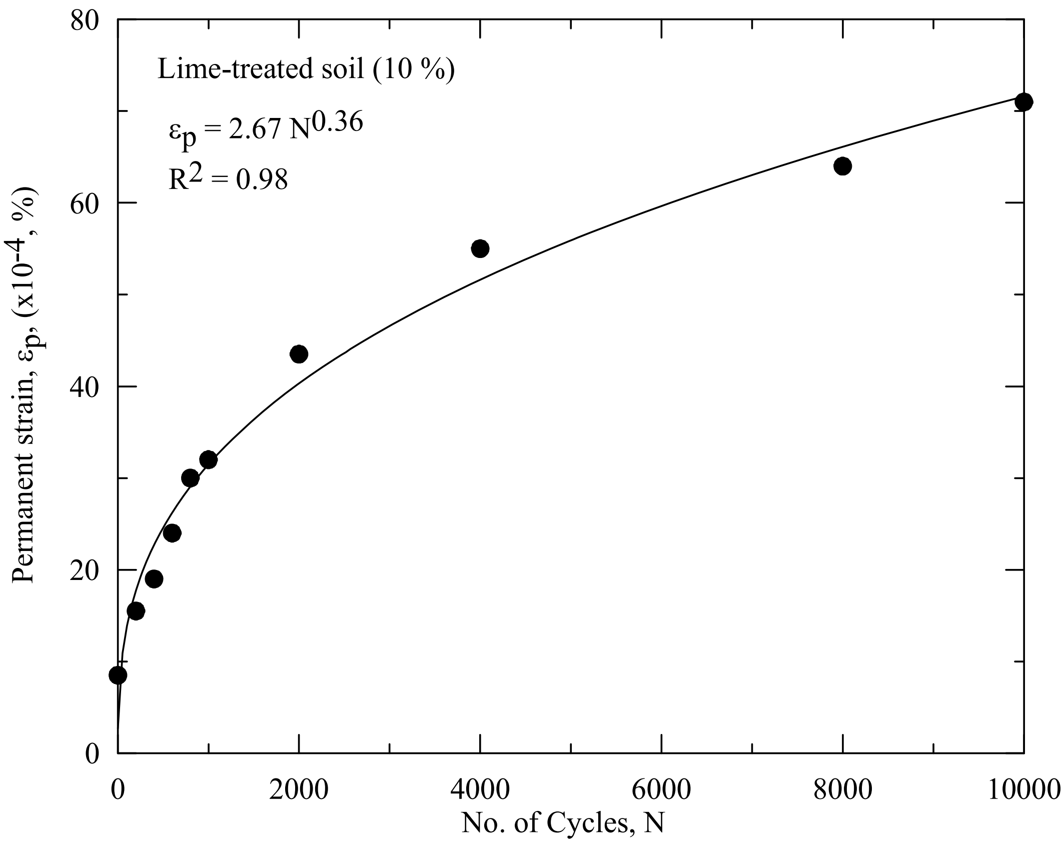


Figure 54
Permanent strains of lime-treated soils at 28-day curing

Comparison of Permanent Deformation of Untreated and Treated Soils. Figure 55 shows the permanent deformation test results of untreated and treated cohesive soils. The permanent deformation of untreated and treated soils increases with the increasing number of load cycles, but with a gradually decreasing rate of increase. The permanent deformation of the cement-stabilized soils with the number of load cycles is lower than that of lime-treated soils. The highest permanent strain was observed in the subgrade soils.

After adding cement and lime, the permanent deformation of subgrade soils decreased drastically. Therefore, both lime and cement are effective in stabilizing weak subgrade soils. The permanent strain of lime-treated soils was 1.5 to 2 times greater than that of cement-stabilized soils. As shown in Figure 55, permanent, resilient, and total strains of lime-treated soils were greater than those of cement-stabilized soils. This implies that cement-stabilized soil samples are stiffer than lime-treated soil samples. In addition, Figure 56 shows the effectiveness of the lime and cement in treating subgrade soils.

Cement-stabilized soils reduced the permanent deformation of subgrade soils by 98 percent; whereas, lime-treated soils reduced it by 97 percent at 10,000 cycles. Table 14 indicates that there is still permanent deformation at 10,000 cycles, and more cycles would be required to virtually eliminate the permanent deformation. Table 15 shows the parameters for the permanent strain model.

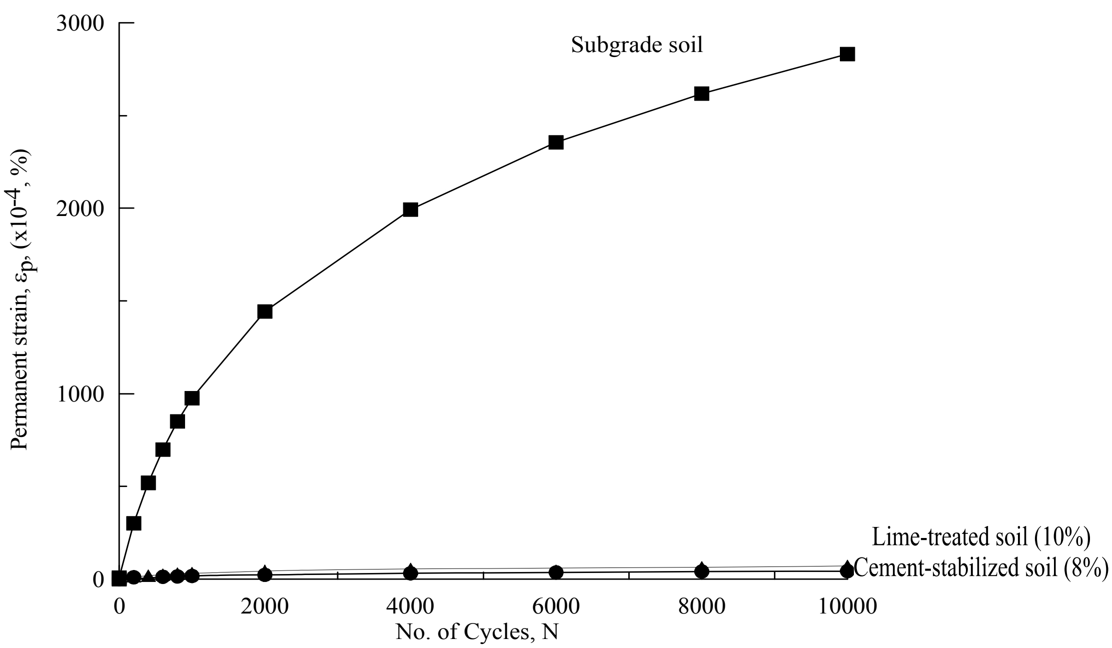


Figure 55

Permanent strains of cohesive soils at 28-day curing

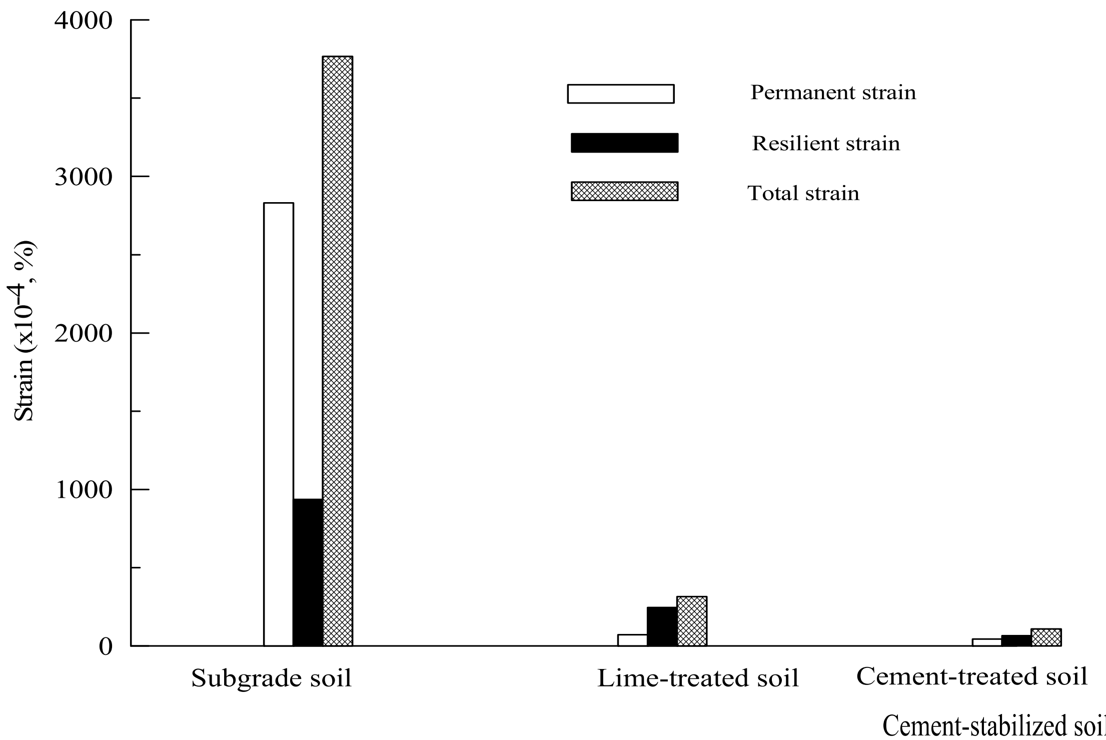


Figure 56

Strains of untreated and treated soils at the end of 10,000 cycles

Table 14

Strain ratio at 10,000 cycles for soils

	Cement- stabilized soil	Lime-treated soil
$\varepsilon_p/\varepsilon_t$	0.32	0.21
$\varepsilon_r/\varepsilon_t$	0.68	0.79

Legend: ε_p- Permanent strain (%), ε_t- Total strain (%), ε_r - Resilient strain (%)

Table 15

Parameters for the permanent strain model

Material	a	b
Subgrade soil	32.83	0.49
Lime-treated soil	2.67	0.36
Cement- stabilized soil	1.05	0.41

Legend: a- Constant of proportionality and b- Rate of change in permanent deformation

Lime-Fly Ash Stabilization

UCS Tests

Blend Only Samples. Results of tests on samples composed of various blends of different additive mixtures are presented in Table 16. Soil was not included in these samples, only blend ratios of lime and fly ash. The strongest samples were composed of fly ash and allowed to cure at 100 percent humidity for 28 days. The weakest samples were composed with a majority of lime and little fly ash. The molding moistures likely affected the samples as they were difficult to mold without adjusting the moisture to the specific additive blend.

Table 16
UCS results for LFA samples (blend only)

Blend Ratio Lime to Fly Ash	Moisture Content at Molding	Curing Days	Sample A, psi	Sample B, psi	Sample C, psi	Average, psi
0 : 100	40 (20% Actual)	1	*88.9*	781.4	455.5	441.9
		3	33.2	32.6	46.1	37.3
		7	716.0	1238.0	1412.4	1122.1
		28	1692.9	1316.6	1307.1	1438.9
50 : 50	40 (50% Actual)	1	201.0	213.6	180.7	198.4
		3	385.2	364.8	368.7	372.9
		7	299.3	325.6	*25.4*	216.8
		28	571.9	663.3	602.7	612.6
70 : 30	40 (60% Actual)	1	150.8	120.8	132.0	134.5
		3	159.5	141.0	159.5	153.3
		7	211.0	192.4	250.1	217.8
		28	389.6	417.1	311.9	372.9
90 : 10	40 (70% Actual)	1	63.0	63.2	63.0	63.1
		3	62.5	62.8	61.9	62.4
		7	64.4	64.7	64.1	64.4
		28	73.0	73.3	74.3	73.5

Notes: 1) 1500 Grams of dry material per sample, 2) Italic value represents outlier value

LFA Treated Soil (Various Blends). Table 17 shows the results of initial tests on silty soil, Soil I, with 6 percent additive. The various blend combinations were added to the silty soil, and the resulting samples were tested for strength. All moistures were within 4 percent and dry densities ranged from 95.9 and 102.7 pcf. The highest UCS results were attained from the sample treated with the 70/30 blend of LFA.

Table 17
UCS results for Soil I with 6 percent additive

Sample %Lime/%Fly Ash	Time of Mold			Time of Break			
	Moisture, %	Wet Density, pcf	Dry Density, pcf	Moisture, %	Wet Density, pcf	Dry Density, pcf	UCS, psi
Raw Soil-A	20.7	121.9	101.0	20.4	123.7	102.7	23.5
Raw Soil-B	21.4	122.8	101.2	20.9	122.3	101.2	20.2
Raw Soil-C	21.4	123.3	101.3	21.1	122.7	101.3	19.4
100% Fly Ash-A	22.4	120.9	98.8	22.4	120.5	98.4	23.76
100% Fly Ash -B	23.6	120.9	97.8	22.5	120.6	98.4	23.2
100% Fly Ash -C	21.9	120.7	99.0	21.5	120.4	99.0	21.0
50/50-A	22.9	119.2	97.0	22.8	118.7	96.7	40.8
50/50-B	23.5	119.2	96.5	22.8	118.9	96.8	43.2
50/50-C	22.7	120.3	98.0	22.5	119.9	97.9	44.7
70/30-A	22.6	119.2	97.2	22.7	119.0	97.0	46.5
70/30-B	23.1	119.9	97.4	22.5	119.5	97.6	47.5
70/30-C	22.8	119.6	97.4	22.3	119.3	97.5	45.2
90/10-A	22.8	118.9	96.8	22.4	118.6	96.9	42.4
90/10-B	23.1	119.6	97.2	22.6	119.3	97.3	46.0
90/10-C	23.4	118.3	95.9	22.5	119.3	97.4	38.8
100% Lime-A	22.9	118.5	96.4	22.8	118.2	96.3	37.0
100% Lime-B	23.1	118.4	96.2	24.0	119.6	96.5	29.8
100% Lime-C	22.8	119.1	97.0	23.6	118.9	96.2	41.6

Figure 57 plots the UCS results from Table 17. The untreated raw soil and the soil treated only with fly ash (FA), have the weakest strengths. The FA did not appear to affect the strength of the treated samples; the strengths were similar to raw soil. In contrast, the treated samples containing lime in the additive showed higher results, specifically the samples treated with the 70/30 LFA blend showed the highest strengths.

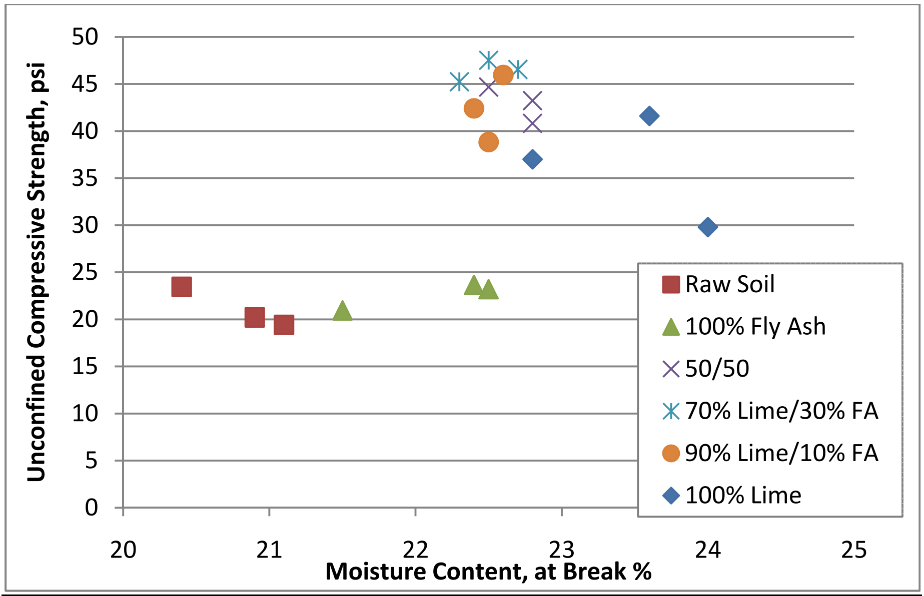


Figure 57
UCS results, various additives, and 7-day breaks

LFA Treated Soil (70 percent Lime/30 percent Fly Ash Blend). The next batch of samples used the same 6 percent additive but focused only on the 70/30 blend since it produced the highest strength results; see Figure 57. The molding moisture was varied during the creation of the samples; then each batch was allowed to cure for 7 days. The higher molding moistures produced the weakest strength results and lowest dry densities. The results are shown in Table 18.

Table 18

7-day breaks, Soil I with 6 percent total additive of 70 percent lime/30 percent fly ash

Molding Moisture - Sample	Time of Mold			Time of Break			
	Moisture	Wet Density, pcf	Dry Density, pcf	Moisture	Wet Density, pcf	Dry Density, pcf	UCS, psi
14.5-A	15.7	113.7	98.3	15.6	113.4	98.1	83.0
14.5-B	15.8	114.0	98.4	15.6	113.4	98.1	52.2
17.5-A	19.2	118.6	99.7	18.9	118.3	99.5	77.7
17.5-B	18.9	116.9	98.3	19.1	116.6	97.9	68.8
20.5-A	22.5	119.2	97.3	22.2	118.8	97.2	49.6
20.5-B	22.5	119.5	97.6	22.0	119.2	97.7	53.8
23.5-A	25.6	117.5	93.6	25.5	117.1	93.3	28.1
23.5-B	25.8	116.9	92.9	25.2	116.5	93.1	27.9
25.5-A	28.1	115.5	90.2	27.8	114.9	98.9	20.4
25.5-B	27.5	115.3	90.4	26.8	114.7	90.5	21.9

Table 19 differs from Table 18 in that the cure time was stretched from 7 to 28 days. Little improvement on strength was realized from the additional cure time. The same decrease in strength was observed in the results of the samples with higher molding moistures.

Table 19

28-day breaks, Soil I with 6 percent total additive of 70 percent lime/30 percent fly ash

Molding Moisture - Sample	Time of Mold			Time of Break			
	Moisture	Wet Density, pcf	Dry Density, pcf	Moisture	Wet Density, pcf	Dry Density, pcf	UCS, psi
14.5-A	16.1	113.2	97.5	15.7	112.7	97.4	83.7
14.5-B	16.3	112.8	97.0	15.8	112.3	97.0	67.5
17.5-A	19.8	117.0	97.7	19.5	116.6	97.6	73.7
17.5-B	19.4	117.3	98.2	21.3	118.8	97.9	51.9
20.5-A	23.2	119.7	97.2	22.6	119.3	97.3	49.3
20.5-B	22.7	119.6	97.5	22.7	119.3	97.2	48.5
23.5-A	26.6	116.2	91.8	26.1	115.7	91.8	30.1
23.5-B	26.1	116.3	92.2	26.4	116.0	91.8	26.4
25.5-A	29.0	114.4	88.7	28.0	113.8	88.9	22.1
25.5-B	28.4	114.5	89.2	29.4	114.8	88.7	14.5

Table 20 and Table 21 differ from Table 18 and Table 19 in that the additive percentage was increased from 6 percent to 8 percent. The additional additive did little to increase the overall UCS results. The strength results were roughly the same for 7- and 28-day breaks. Higher molding moistures again affected the UCS results, producing weaker values. In addition, longer cure rates showed little, if any, increase in strength.

Table 20

7-day breaks, Soil I with 8 percent total additive of 70 percent lime/30 percent fly ash

Molding Moisture - Sample	Time of Mold			Time of Break			
	Moisture	Wet Density, pcf	Dry Density, pcf	Moisture	Wet Density, pcf	Dry Density, pcf	UCS, psi
14.5-A	17.0	113.5	97.0	16.7	113.1	96.9	78.5
14.5-B	17.0	113.4	96.9	16.8	113.0	96.7	78.7
17.5-A	20.1	118.1	98.3	19.8	117.7	98.2	75.4
17.5-B	20.1	117.1	97.5	19.6	116.6	97.5	71.1
20.5-A	25.2	119.5	95.5	22.8	119.1	97.0	49.8
20.5-B	23.4	119.6	96.9	22.6	118.9	97.0	46.0
23.5-A	26.4	117.3	92.8	25.6	116.7	92.9	30.1
23.5-B	26.0	116.0	92.6	25.6	116.0	92.4	30.3
25.5-A	27.9	115.1	90.0	28.2	114.5	89.3	20.3
25.5-B	27.6	114.5	89.7	28.0	113.9	89.0	21.2

Table 21

28-day breaks, Soil I with 8 percent total additive of 70 percent lime/30 percent fly ash

Molding Moisture - Sample	Time of Mold			Time of Break			
	Moisture	Wet Density, pcf	Dry Density, pcf	Moisture	Wet Density, pcf	Dry Density, pcf	UCS, psi
14.5-A	16.5	111.9	96.1	16.0	111.3	95.9	89.7
14.5-B	15.9	111.8	96.5	15.7	111.4	96.3	87.7
17.5-A	19.4	115.9	97.1	18.9	115.6	97.2	85.0
17.5-B	19.6	116.9	97.7	19.9	117.5	98.0	73.2
20.5-A	23.1	119.1	96.8	22.1	118.6	97.1	67.0
20.5-B	23.4	118.8	96.3	22.3	118.2	96.6	59.9
23.5-A	26.1	116.7	92.5	25.5	116.2	92.6	38.3
23.5-B	26.0	116.2	92.2	25.7	115.6	92.0	35.8
25.5-A	28.1	114.7	89.4	28.1	114.0	89.0	25.5
25.5-B	28.4	115.2	89.7	27.5	114.4	89.8	26.7

The addition of LFA appears to produce increases in strength over untreated raw soil; samples with molding moistures near the optimum produced the highest strength results. In addition, results indicate that higher molding moistures produced weaker strength results than at drier molding moisture contents.

Higher additive rates, not evaluated in this study, may compensate for the wet soil and produce higher strengths. However, the additional additive may reduce its cost effectiveness. When compared to cement at the same percentage additive (8 percent), cement produced strength results ranging roughly from 100 psi to 600 psi, up to 6 times higher than the LFA strength results.

Field Evaluation Results

Laboratory Strength Tests on ALF Field Samples

During construction of the cement-stabilized subbase and the working table, material was taken from the test section after the additive (cement or lime) was blended, prior to compaction. LTRC personnel transported collected material immediately to LTRC for sample molding. The samples cured in the 100 percent room for 1, 7, 14, and 28 days prior to testing for strength. Table 22 and Table 23 present the strength test results for the treated subbase materials in this study.

Table 22
UCS results for lime treated working table

Curing Days - Sample	Time of Mold			Time of Break			
	Moisture	Wet Density pcf	Dry Density pcf	Moisture	Wet Density pcf	Dry Density pcf	UCS psi
1*-A	21.8	124.5	102.2	21.2	124.3	102.6	25.5
1-B	22.2	124.6	102.0	20.9	124.4	102.9	26.0
1-C	20.9	124.7	103.1	21.3	124.6	102.7	20.7
7-A	19.6	122.8	102.6	18.7	122.5	103.2	40.4
7-B	20.5	123.3	102.4	19.8	123.0	102.7	40.4
7-C	20.0	123.1	102.6	19.5	122.8	102.8	40.3
14-A	19.9	122.4	102.1	19.9	122.1	101.9	40.8
14-B	20.0	121.5	101.3	19.8	121.2	101.2	36.9
14-C	19.6	122.1	102.1	19.4	121.8	102.0	41.9
28-A	19.4	122.3	102.5	19.9	122.6	102.3	34.3
28-B	20.0	124.7	103.9	21.5	124.6	102.6	40.0
28-C	21.4	124.5	102.5	22.3	124.6	101.9	29.5

Note:* The number in a sample ID represents the number of curing days

Table 23
UCS results for cement-stabilized subbase

Curing Days - Sample ID*	Time of Mold			Time of Break			
	Moisture	Wet Density pcf	Dry Density pcf	Moisture	Wet Density pcf	Dry Density pcf	UCS psi
1*-A	18.0	126.1	106.9	15.7	125.7	108.6	106.7
1-B	16.8	125.9	107.8	15.2	125.7	109.1	114.7
1-C	16.6	125.9	108.0	16.6	125.8	107.9	104.2
7-A	16.6	124.8	107.0	18.8	124.6	104.9	133.0
7-B	16.2	124.3	107.0	16.3	124.3	106.9	82.6
7-C	16.7	124.1	106.3	15.8	123.9	107.0	126.2
14-A	17.3	123.2	105.0	18.3	125.0	105.7	163.5
14-B	17.4	121.5	103.5	18.6	123.7	104.3	126.8
14-C	18.1	123.0	104.1	15.7	125.0	108.0	114.2
28-A	17.7	122.6	104.1	17.2	123.9	105.7	121.1
28-B	15.1	121.4	105.5	17.7	122.7	104.3	155.8
28-C	17.7	119.6	101.6	18.5	121.2	102.2	141.2

Note: * The number in a sample ID represents the number of curing days

The following observations were made from Table 22 and Table 23:

- Lime-treated subbase strengths generally increased from 24 psi to 35 psi over 28 days with a peak value of 42 psi in sample 14-C.
- Cement-stabilized subbase strengths generally increased from 108 psi to 139 psi over 28 days with a peak value of 164 psi.
- Cement strengths were roughly four times stronger than lime at the slightly different additive percentages (percentage by volume: Cement 8 percent and Lime 10 percent).

Accelerated Loading Results

As shown in Table 10, the total number of load repetitions applied on sections 4-1B and 4-2B was 150,000 and 300,000 ALF dual tire load repetitions, respectively. Note that section 4-1B was loaded for 150,000 repetitions by the ALF self-load of 9,750 lb. only. On section 4-2B, the initial 200,000-repetitions were loaded under the 9,750-lb. self-load. Then, from 200,000 to 225,000 repetitions, the load had been raised to 12,050 lb. (i.e., one load plate of 2,300 lb. was added). Another load plate was put after 225,000 passes and the total load became 14,350 lb. until the end of ALF testing on section 4-2B (i.e., 300,000 repetitions).

The in-situ ALF results generally indicated that both test sections had a rutting failure (i.e., reaching to a rutting limit prior to a cracking limit set in this study). In fact, no visible surface cracks were found on section 4-2B at the end of ALF testing, but some medium-severe alligator cracks were observed on section 4-1B (the visible cracks began to develop approximately at end of 75,000 load repetitions).

Figure 58 presents variation of measured average rut depths along the number of equivalent single axle loads (ESALs) for the two sections tested. As shown in Figure 58, section 4-2B with a cement stabilized subbase layer performed significantly better that section 4-1B with a lime-treated working table layer. The rutting lives (i.e., the number of load repetitions for reaching to a rutting limit of 0.5-in.) of sections 4-1B and 4-2B were found to be 121,000- and 786,000-ESALs, respectively. It should be noted that the conversion between the ALF load repetitions and ESALs was based on the fourth power law (Huang, 1993) and the corresponding ALF loading repetitions were 87,800 and 282,000 on sections 4-1B and 4-2B, respectively.

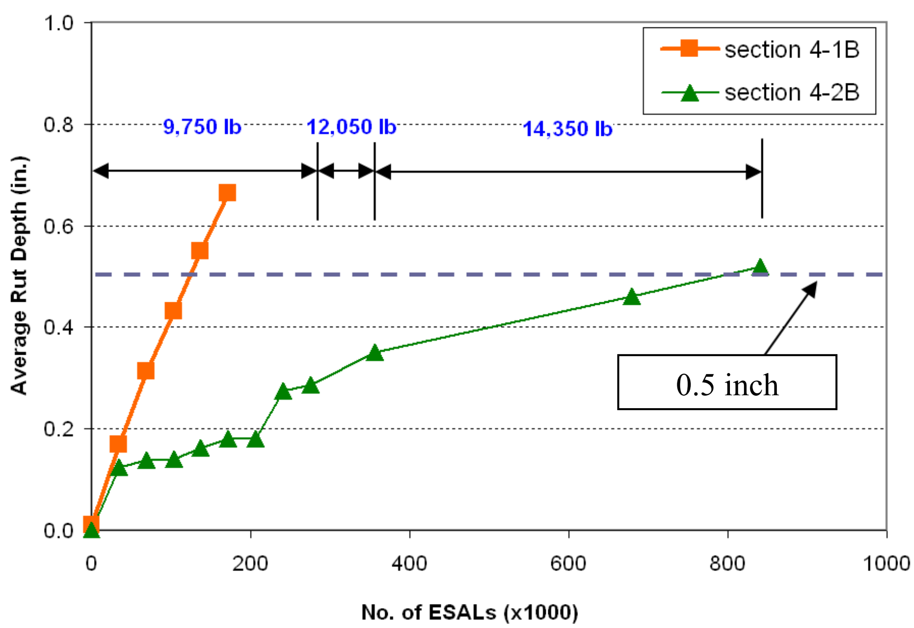
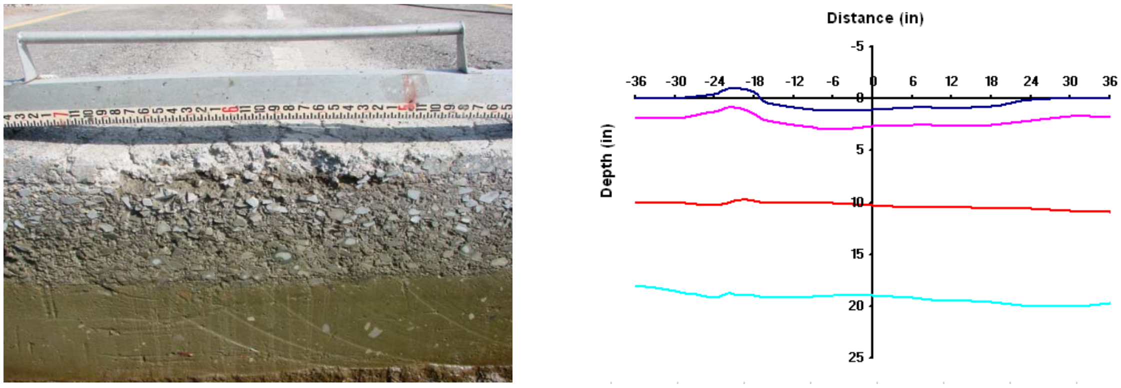


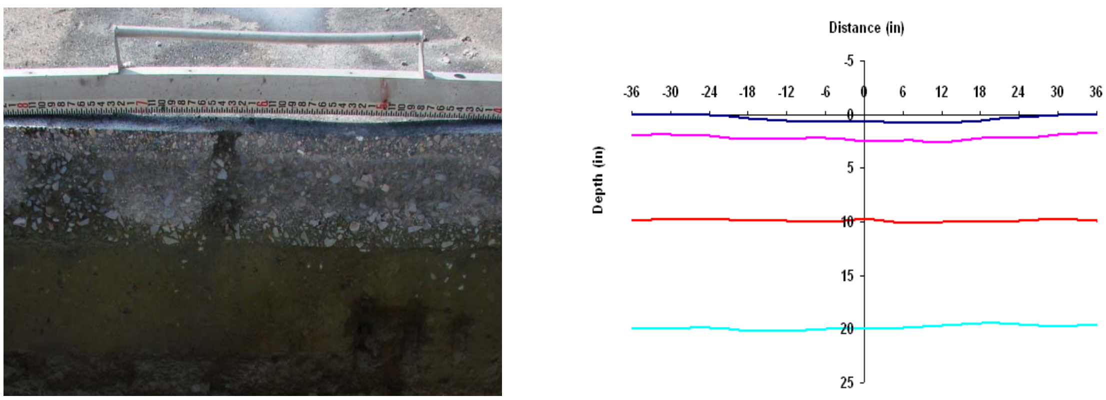
Figure 58
Rut depth development on test sections

In previous ALF research, a term of “ESAL advantage” was introduced to quantify different performance of two pavement materials under an accelerated loading (King, 2004). ESAL advantage is defined as a ratio of pavement lives for two similar pavement structures under accelerated pavement testing (APT). The two pavement structures only differ from each other in one pavement layer, e.g., different HMA mixtures, base materials, and so on. Thus, the APT pavement lives may be used to quantify the difference in field performance of the two selected pavement materials in term of an EASL advantage. The ESAL advantage implies that one material will perform how many times better than another material under an APT testing within a certain pavement structure. It is, thus, calculated that based on the APT results obtained in this study, section 4-2B with a 12-in. cement stabilized subbase would have 6.5:1 ESAL advantage over section 4-1B with a 12-in. lime treated working table layer.

Figure 59 presents post-mortem trench photos as well as corresponding measured transverse profiles for the two sections tested. Each trench was about 2 ft. wide and 10 ft. long. The measured transverse profiles reveal that section 4-1B appeared to have had a shear flow failure within its crushed limestone base layer (Figure 59a); whereas, only further densification was found within the HMA and crushed stone layers of section 4-2B and no noticeable permanent deformation could be viewed in the CSS layer (Figure 59b). As expected, the post-mortem results generally confirm that a cement stabilized subbase possesses a greater structural capacity than a “working table layer” in section 4-1B.



(a) Section 4-1B



(b) Section 4-2B

Figure 59
Post-mortem trench results

Instrument Responses to ALF Wheel Loading

Pressure Cells

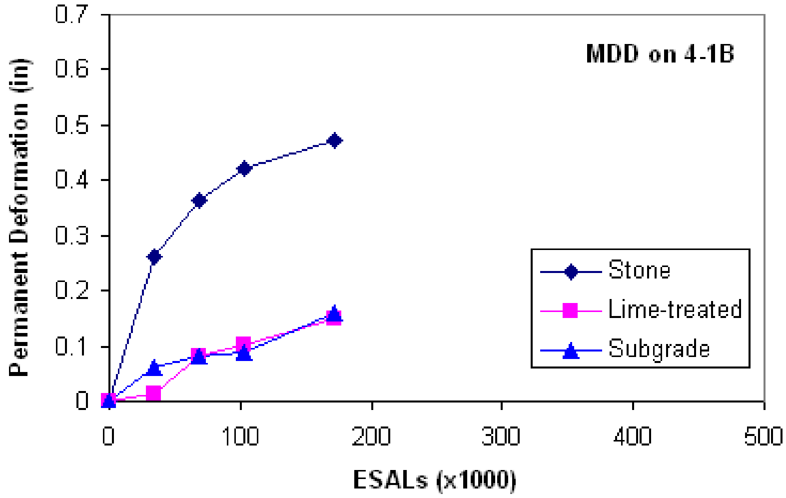
Table 24 presents a statistical summary of vertical compressive stresses measured at two vertical depths under ALF wheel loading for the two test sections considered. Note that only the stresses measured under the ALF self-load of 9,750 lb. were listed in Table 10 (due to section 4-1B was failed under this load). As shown in Table 24, under the ALF self-load of 9,750 lb., the average vertical compressive stresses at bottom of the base layers were 32.4 and 18.6 psi for sections 4-1B and 4-2B, respectively; whereas, the corresponding values on top of subgrades were 0.6, and 0.7 psi, respectively. Such results indicate that both test sections received a comparable compressive stress on top of subgrades. However, the significantly smaller stress value developed at the bottom of the stone base layer of section 4-2B reflects that this stone layer was able to distribute the load better than the stone base in section 4-1B. This further implies that the same stone layers may have different in-situ stiffnesses (or moduli) under wheel loading: the stone layer on section 4-2B probably has a larger stiffness value than that in section 4-1B. This may be explained by the so-called “stress hardening” phenomenon for stone materials. The stiffer CSS subbase may provide a stronger support (or higher confining pressure) to the stone base on section 4-2B, which results in a higher confinement stress to the stone layer, thus producing a greater in-situ stiffness value for the stone base on section 4-2B. The difference in vertical stresses also explains indirectly why section 4-2B had a long pavement life than section 4-1B.

Table 24
Results of the measured vertical compressive stresses

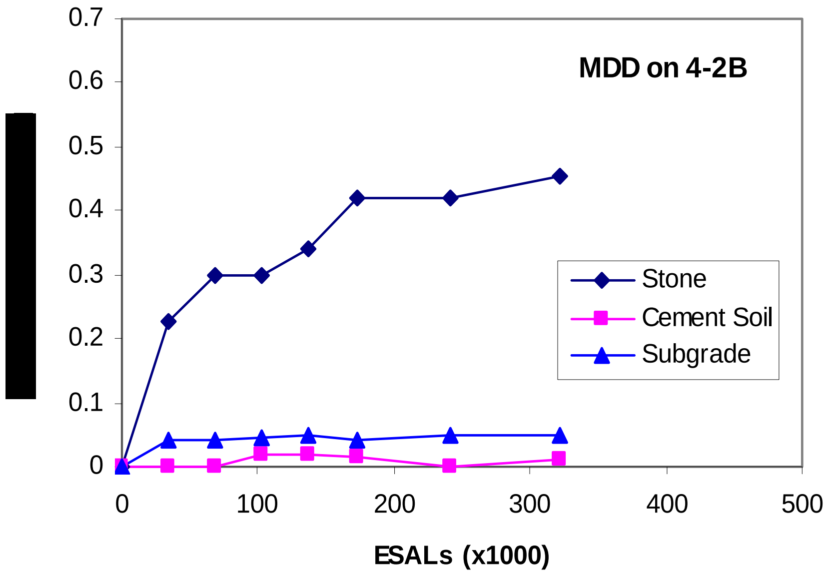
Section	Statistics	Vertical Stress, psi	
		At Bottom of Base	At Top of Subgrade
4-1B	Avg	32.4	0.6
	Std	2.2	0.2
	COV	6.8%	31%
4-2B	Avg	18.6	0.7
	Std	0.4	0.1
	COV	2.2%	7.7%

MDD Results

Figure 60 shows variation of MDD measured permanent deformations for the two test sections considered. As can be seen in the figure, a significantly large amount of permanent deformation was developed within the stone bases of both sections. For section 4-1B, the stone base, working table, and subgrade, each contributes 60 percent, 20 percent and 20 percent of the total MDD measured permanent deformation, respectively. For section 4-2B, however, the stone base, CSS, and subgrade each contributes 85 percent, 5 percent, and 10 percent of the total MDD measured permanent deformation, respectively. It shows that the permanent deformations developed on the CSS layer of section 4-2B were negligible as compared to the working table on section 4-1B. This observation further confirms the post-mortem trench results, which shows that the CSS layer had a higher strength and higher load carrying capability than the working table in this study.



(a)



(b)

Figure 60

(a) Permanent deformation at section 4-1B;
(b) permanent deformation at section 4-2B

NDT Test Results

Dynaflect Results

Figure 61 presents a variation of Dynaflect determined structural numbers (SNs) for the two test sections considered. Generally, a larger SN value is desired for a strong pavement with better structural performance. As can be seen in Figure 61, both SN curves exhibit a similar trend with an initial increasing with loads then decreasing. As expected, the SN values in section 4-2B are found much higher than those in section 4-1B, indicating better structural performance. The SN difference between the two sections is approximately equal to 0.6 (Figure 61). An initial SN increase may be related to the post compaction effect of pavement layers and possible strength gains of treated soil layers due to curing. It should be noted that some severe surface cracks developed in section 4-1B after 75,000 ALF repetitions, which may be responsible to the abnormal SN values observed at the 125,000 ALF load repetitions.

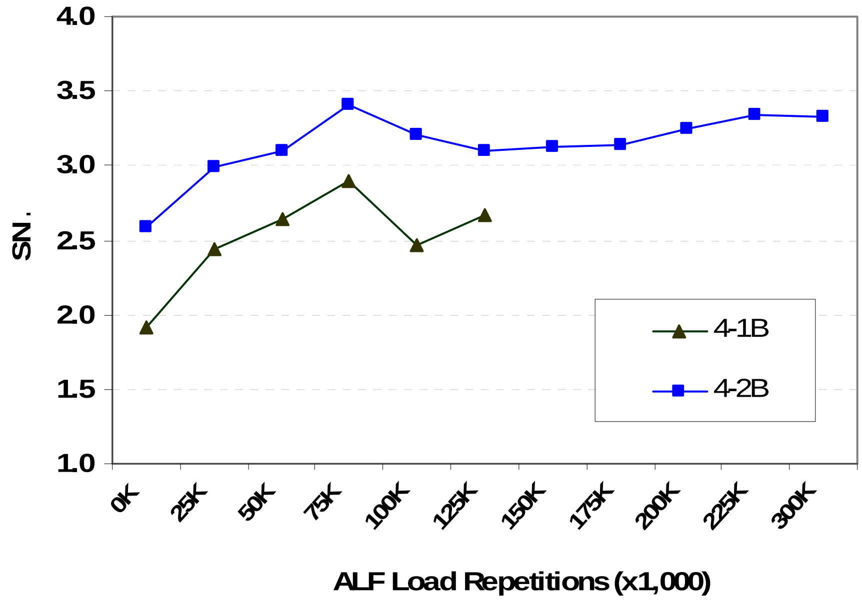


Figure 61
Dynaflect structural number results

In general, section 4-2B with a CSS layer possessed higher SN values than section 4-1B with a working table. Since the only difference between the two sections is the subbase courses, it is concluded that the in-situ structural capacity of a CSS layer in terms of SN value is found significantly better than that of a working table used in this experiment. This basically means that the subbase layer versus the working table layer can be quantified.

FWD Test Results

Figure 62 presents the average FWD center deflection (D0) results for the two sections evaluated. The deflections were first normalized to a load level of 9.0 kips and then temperature-corrected to 77°F based on a procedure developed under the Long Term Pavement Performance (LTPP) program (Lukanen, 2000). The center FWD deflection (i.e., measured directly under the FWD loading plate) is usually considered as an indicator of the composite stiffness of a pavement structure. A higher surface deflection indicates a smaller composite stiffness for a pavement structure. As shown in Figure 62, the D0s of section 4-2B were significantly smaller than those on section 4-1B, indicating an overall stronger pavement structure. The normalized D0 results basically confirm the results obtained from the DYNAFLECT SN results shown in Figure 61.

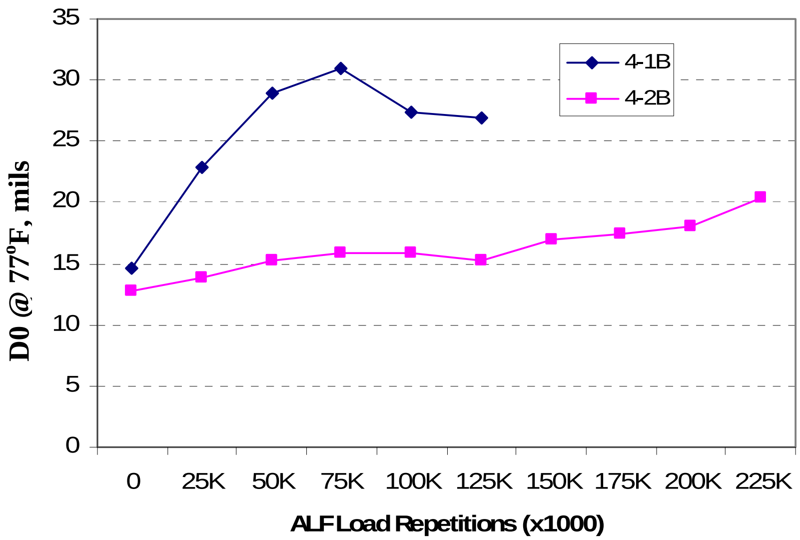


Figure 62
FWD center deflections

Table 25 and Table 26 present FWD backcalculation results for sections 4-1B and 4-2B, respectively. It is noted that in order to obtain a more realistic set of backcalculated layer moduli with acceptable RMS errors the HMA modulus in all backcalculation was assumed to be 725 ksi at 77°F during the FWD backcalculation process. In general, the FWD backcalculation RMS errors of section 4-2B were less than 3 percent, followed by section 4-1B of less than 5 percent.

Table 25
FWD backcalculation moduli for section 4-1B

No. of Passes x 1000	Cumulative ESALs x1000	Modulus, ksi			
		HMA	Crushed Limestone	Lime Soil	Subgrade
0K	0	965.9	54.4	84.9	20.1
25K	34	738.2	74.2	60.0	17.1
50K	69	581.0	82.4	65.9	16.7
75K	103	525.8	81.3	61.7	16.3
100K	138	632.2	81.1	54.6	16.7
125K	172	600.7	55.7	86.8	16.7

Table 26
FWD backcalculation moduli for section 4-2B

No. of Passes x 1000	Cumulative ESALs x1000	Modulus, ksi			
		HMA	Crushed Limestone	Cement Soil	Subgrade
0K	0	751.0	32.1	407.9	13.3
25K	34	684.5	48.8	243.9	15.8
50K	69	639.0	42.6	225.9	13.9
75K	103	542.4	40.1	264.1	13.0
100K	138	586.9	44.3	197.5	13.7
125K	172	606.0	45.0	236.3	13.1
150K	207	522.1	44.0	221.7	12.4
175K	241	556.3	48.4	223.7	13.9
200K	275	551.5	39.7	252.6	14.3
225K	356	533.4	32.0	257.9	14.5

Figure 63 presents the backcalculation results for the two subbase materials considered. As shown in the figure, the in-situ moduli of the CSS layer were observed significantly higher than those for the working table. This is generally consistent with the laboratory resilient modulus results. It may be deduced from Figure 63 that the typical in-situ moduli for the CSS and working table used are 230 ksi and 60 ksi, respectively.

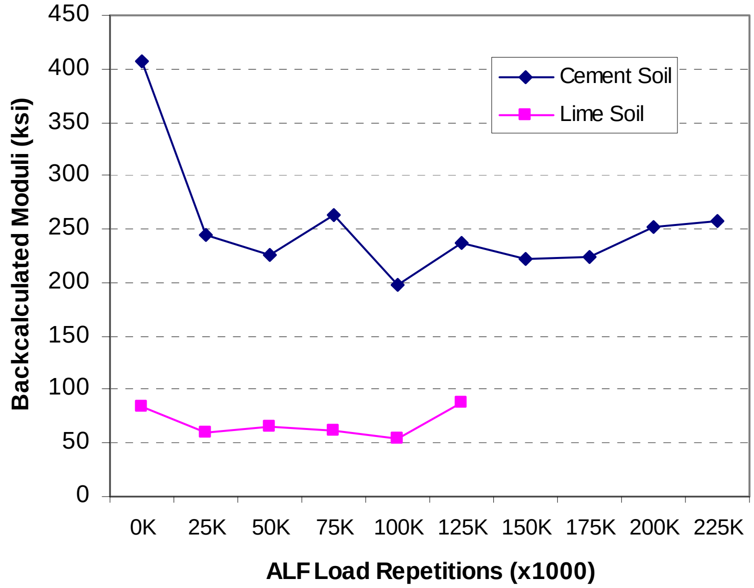


Figure 63

Backcalculated moduli of base materials

Analysis of APT Pavement Structures

Vertical Stresses

The measured vertical compressive stresses were further compared to those analytical values estimated from a multi-layer elastic analysis program, ELSYM5. Backcalculated moduli obtained from the initial FWD tests (0K in Table 25 and Table 26) were used in the analysis. Table 27 presents the measured and calculated vertical compressive stresses on the two lanes tested.

Table 27
Comparison of measured and calculated vertical compressive stresses

	Vertical Stress @ bottom of Base, psi		Cal./Meas.	Vertical Stress @ bottom of Subbase, psi		Cal./Meas.
	Measured	Calculated		Measured	Calculated	
4-1B	32.4	15.8	0.5	0.6	3.57	6.0
4-2B	18.6	19.9	1.1	0.7	2.02	2.9

As shown in Table 27, stress ratios between the calculated and the measured were generally ranged from 0.5 to 6.0, with both the highest and lowest ratios falling in test section 4-1B. The discrepancy between the predicted and the measured values is certainly due to the limitations of using the multi-layer elastic theory. However, it is interesting to notice that in Table 27 only the calculated vertical stress at bottom of the stone base in section 4-1B was found smaller than the measured one; whereas, in all other locations, the predicted values were higher. Figure 64 presents the predicted stresses and strains at the top and bottom of both base layers in sections 4-1B and 4-1B estimated using ELSYM5.

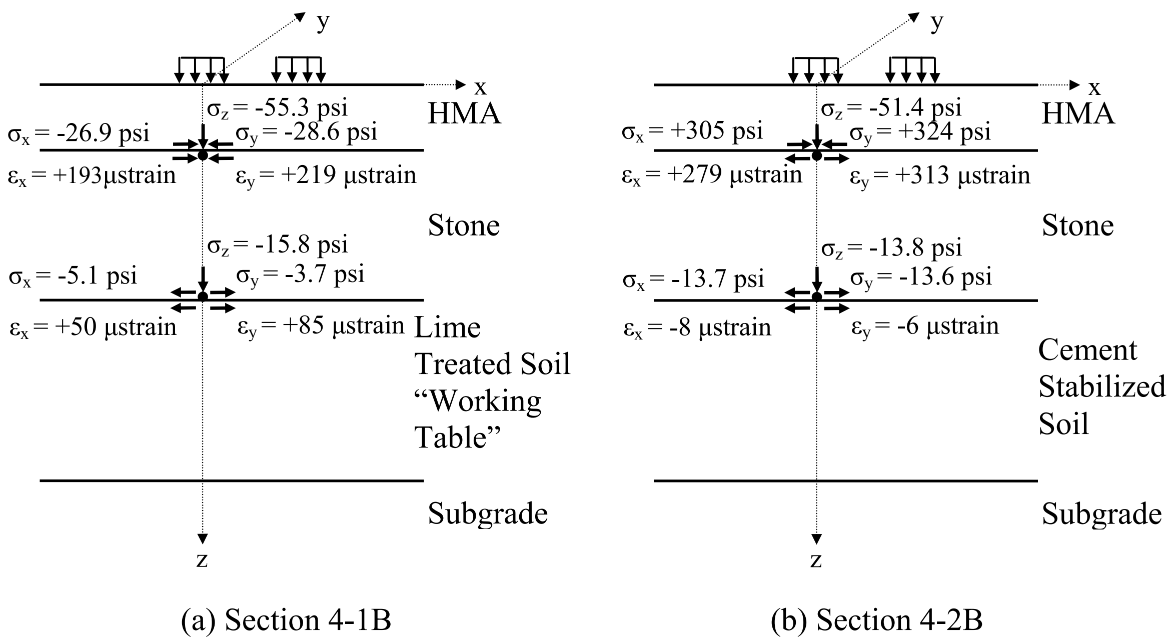


Figure 64
Stress and strain predictions ("+" tension, "-" compression)

As shown in Figure 64(a), a tension zone in section 4-1B (based on the predicted tensile strains) will start from the middle of the HMA layer and then go all the way to the bottom layers. This indicates that the entire stone base layer in section 4-1B is in tension [see ε_x and ε_y in Figure 64(a)]. However, a same tension zone in section 4-2B goes from the middle of the HMA layer and then stops somewhere inside the stone layers. The difference is that, at the bottom of the stone layers, section 4-1B is predicted under tension, while section 4-2B is predicted under compression. In reality, a stone layer cannot resist any significant tension because the material is unbound. If a stone aggregate layer received a large tension, the stone particles would begin to separate from each other. Therefore, the possible segregation of stone particles due to tension may explain why a higher than theory-predicted vertical stresses were measured at the bottom of the stone layer in section 4-1B. In addition to the limitation of the prediction theory used, another source for the discrepancy between the predicted and the measured stresses is possible from the accuracy of stress measurement device itself (Dunnicliff, 1993).

Prediction of Layer Coefficients of Stabilized/Treated Soil Materials

The 1993 AASHTO pavement design guide uses the structural layer coefficient in flexible pavement design. The structural layer coefficient (hereafter is called as "*a*-value"), originally obtained from the AASHTO road test in 1950s, reflects directly to the structural strength of a material within a flexible pavement structure. Widely accepted *a*-values for a new HMA layer, a crushed stone base and a soil cement base are 0.44, 0.14 and 0.14, respectively. However, due to lack of field performance data, the *a*-value for a chemically stabilized or treated soil material is usually not determined for a pavement design.

To fill such a gap in pavement design, the *a*-values for the two chemically stabilized/treated soil layers used in this study can be predicted based on the following three steps:

1. First, estimate the effective structural number (SN_{eff}) of a pavement section (4-1B or 4-2B) from the before-APT-loading FWD results using the following three equations (as introduced earlier in the Methodology section):

$$SN_{eff} = 0.0045 \times D \times \sqrt[3]{E_p} \tag{4}$$

$$M_R = \frac{0.24P}{d_r r} \tag{5}$$

$$d_0 = 1.5pa \left\{ \frac{1}{M_R \sqrt{1 + \left(\frac{D}{a} \sqrt[3]{\frac{E_p}{M_R}} \right)^2}} + + \frac{\left[1 - \frac{1}{\sqrt{1 + \left(\frac{D}{a} \right)^2}} \right]}{E_p} \right\} \tag{6}$$

where,

D = total pavement thickness above the subgrade, in.;

E_p = effective pavement modulus of all existing pavement layers above the subgrade;

M_R = backcalculated subgrade resilient modulus, psi;

D = total thickness of pavement layers above the subgrade, in.;

d_0 = deflection measured at the center of the FWD plate (and adjusted to 68^{o}F), in.;

P = applied load, lb.;
p = FWD load plate pressure, psi;
d_r = deflection at a distance r from the center of FWD plate, in.;
r = distance from center of load, in.; and
a = FWD load plate radius, in.

2. Second, compute the structural number of a pavement section based on the *SN* definition below:

$$SN = a_1h_1 + a_2h_2 + a_3h_3 \tag{7}$$

where, a_i is the structural layer coefficients of each pavement layer in test section and h_i is the corresponding layer thicknesses. In this study, a_1 is 0.44 for the top 2-in. HMA layer, a_2 is 0.14 for the 8.5-in. stone base, and a_3 is the 12-in. stabilized or treated soil layer, which is under determination.

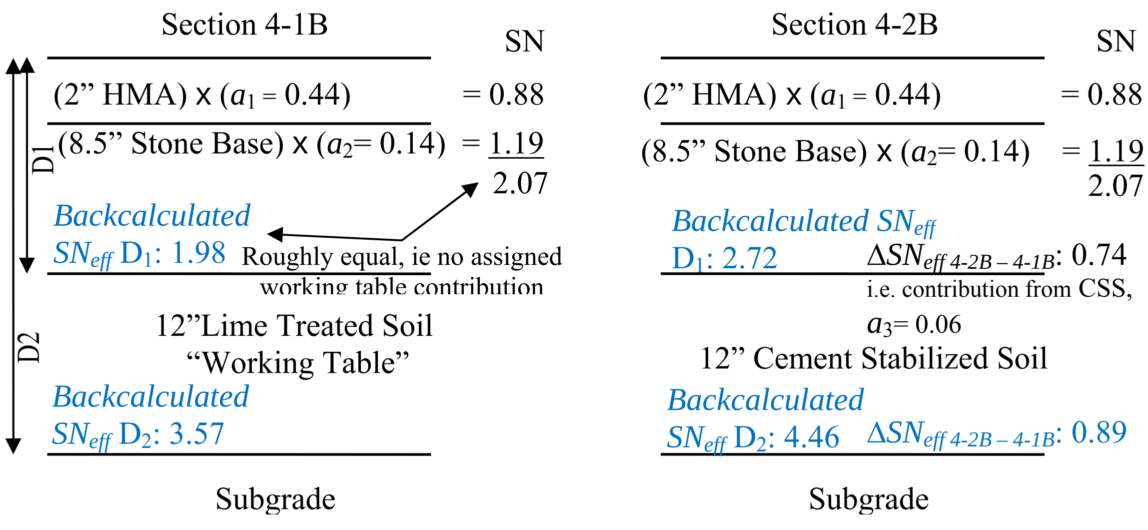
3. Let *SN* in equation (7) equal the SN_{eff} determined from Step 1, then the *a*-value for a treated soil layer can be estimated as follows:

$$a_3 = \frac{SN_{eff} - a_1h_1 - a_2h_2}{h_3} = \frac{SN_{eff} - 0.44\times 2 - 0.14\times 8.5}{12} \tag{8}$$

As can be seen in equations (4) and (6), the computation for both SN_{eff} and E_p requires an input parameter-the total pavement thickness above subgrade (D). Clearly, different total pavement thicknesses (D-values) would result in different backcalculated SN_{eff} values. In this study, two different D-values are used in the SN_{eff} estimation for test sections in which D_1 equals 10.5 in. and D_2 is 22.5 in. The SN_{eff} estimation results are presented below:

(a) If $D = D_1 = 10.5"$ → $SN_{eff}|_{4\text{-}1B} = 1.98$ and $SN_{eff}|_{4\text{-}2B} = 2.72$
(b) If $D = D_2 = 22.5"$ → $SN_{eff}|_{4\text{-}1B} = 3.57$ and $SN_{eff}|_{4\text{-}2B} = 4.46$

Figure 65 presents a graphical representation of SN_{eff} estimation results for the two APT sections considered. As shown in Figure 65, when D_1 is used, the assumption is that the overall structural strength comes from the top two layers (HMA and base) only, indicating that the 12-in. treated soil layer does not make a contribution to the overall structural strength. On the other hand, when D_2 is used, it is assumed that the 12-in. treated soil layer does make a contribution to the overall structural strength.



(a) Section 4-1B (b) Section 4-2B

Figure 65
Estimation of effective structural number

As seen from these results, due to different *D*-values, the difference of backcalculated SN_{eff} values is 3.57 – 1.98 = 1.59 for section 4-1B and 1.74 for section 4-2B (i.e.; 4.46 – 2.72 = 1.74). Clearly, a higher *D*-value would result in a larger backcalculated SN_{eff} value for a pavement structure. Therefore, a question is raised: which *D*-values should be used?

As shown in Figure 65, if only the top two layers are considered, the computed *SN* value for section 4-1B is 2 x 0.44 + 8.5 x 0.14 = 2.07. This *SN*-value is very close (roughly equal) to the backcalculation result of 1.98 (i.e. when $D= D_1$, $SN_{eff}|_{4\text{-}1B} = 1.98$). On the other hand, if *D* should be equal to D_2, then the structural number for 4-1B would be 3.57 (i.e., when $D= D_2$, $SN_{eff}|_{4\text{-}1B} = 3.57$). Such an *SN* result calls for the performance of section 4-1B to be equivalent to a pavement structure composing of a 5.4-in. HMA layer over a 8.5-in. crushed stone layer ($SN = 5.4 \times 0.44 + 8.5 \times 0.14 \approx 3.57$). Based on the APT testing results as well as engineering judgment, it is clear that section 4-1B with a pavement life of only 87,800 ALF repetitions will never perform equivalently as a pavement structure composing of a 5.4-in. HMA layer and a 8.5-in. crushed stone layer. Therefore, $D = D_1$ should be used in the backcalculation of SN_{eff} for test sections of this study. Furthermore, based on the $SN_{eff}|_{4\text{-}1B}$ backcalculation result of 1.98, it is concluded that no structural value (or *a*-value of zero) be assigned to the lime-treated working table layer used in section 4-1B. On the other hand, a *SN*-value difference of 0.74 between section 4-2B ($SN_{eff}|_{4\text{-}2B} = 2.72$) and 4-1B ($SN_{eff}|_{4\text{-}1B} =$

1.98) should come from the contribution of the cement stabilized soil subbase layer used in section 4-2B. Therefore, the *a*-value for the cement stabilized soil layer of this study may be estimated from *SN*-value of 0.74 divided by the thickness of 12-inch, that is around 0.06.

In conclusion, the predicted *a*-values for the lime-treated working table layer and cement-stabilized soil subbase layer used in this study is zero and 0.06, respectively.

Life Cycle Cost Analysis

The aforementioned APT results generally revealed that using a cement stabilized soil subbase layer in a flexible pavement structure (e.g., section 4-2B) can extend the pavement service life by several times when compared to a pavement with only a lime-treated working table layer (e.g., section 4-1B), as illustrated by a 6.5:1 ESAL advantage in rutting performance presented in Figure 58. However, the decision on whether to implement the design of a cement stabilized subbase layer should be also dependent on its economic aspects or benefits, as the initial construction cost of using a cement stabilized soil subbase layer in a flexible pavement could be higher.

To demonstrate the potential cost benefits of using a cement stabilized soil subbase in lieu of a lime-treated working table layer in a flexible pavement design, a life-cycle cost analysis (LCCA) was performed on two types of typical Louisiana flexible pavement structures: Figure 66a is for low volume roads, and Figure 67a is for high volume roads. Also shown in the two figures, two alternative structure designs are to be used in the LCCA for each pavement type: *Alternative A*-pavement structure contains a 12-in. lime-treated working table layer, and *Alternative B*-pavement structure uses a 12-in. cement stabilized soil subbase layer. According to Louisiana's Alternate Design / Alternative Bid programs, a 30-year analysis period was used in the LCCA analysis. The LCCA analysis assumed that future maintenance requirements would include the following: full reconstruction (including both base and HMA layers) at the end of pavement life for *Alternative A* (pavement with a 12-in. lime-treated working table), and a one-time 2-in. milling and 3.5-in. HMA overlay at the end of year 15 for *Alternative B* (pavement with 12-in. cement stabilized soil subbase). Additional assumption included a 5 percent inflation rate and zero salvage value at the end of year 30.

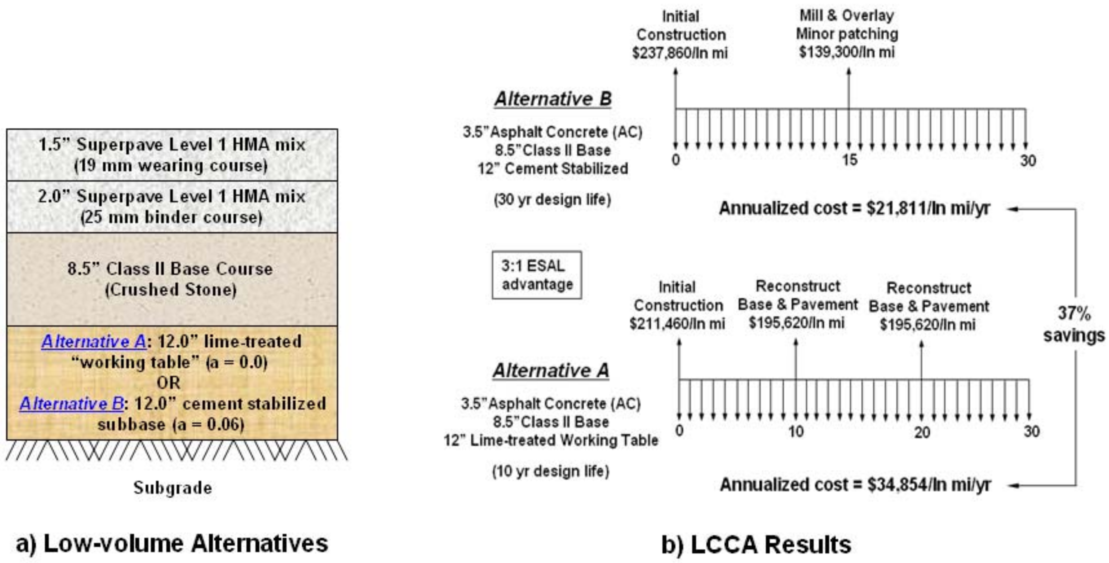


Figure 66
LCCA on low volume roads

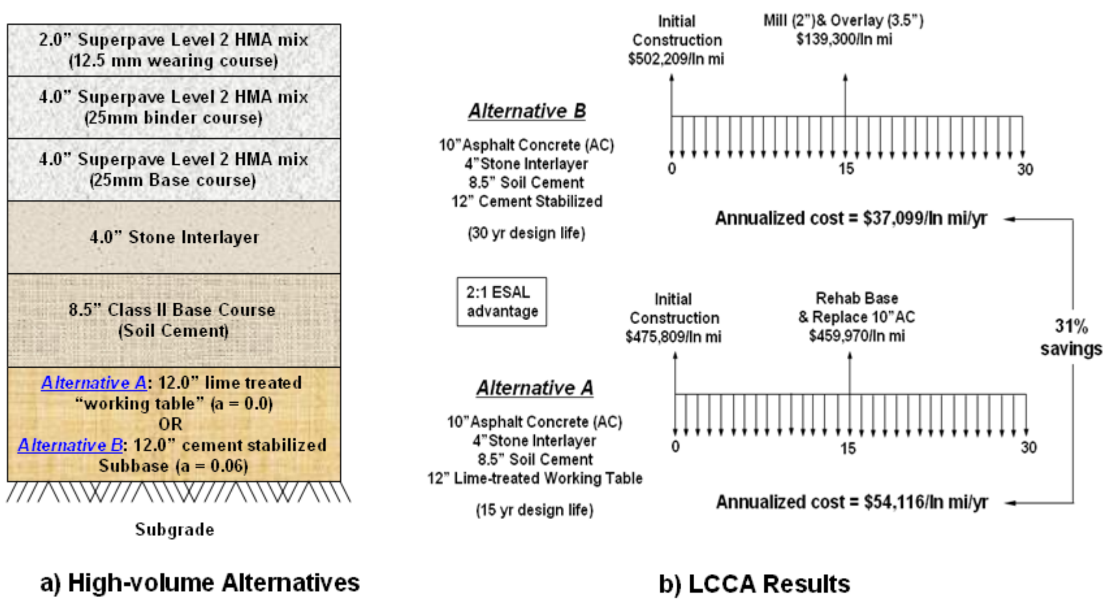


Figure 67
LCCA on high volume roads

As shown in Figure 66b, the LCCA analysis for low volume roads reveals that using a 12-in. cement-stabilized soil subbase (*Alternative B*) in lieu of a 12-in. lime-treated working table layer (*Alternative A*) creates a 37 percent annual cost savings during a 30-year analysis period. Similarly, the LCCA analysis results indicate that, for high volume roads, the cost-savings for using a 12-in. cement-stabilized soil subbase can be as high as 31 percent over using a 12-in. lime-treated working table layer during the 30-year analysis period, as shown in Figure 67b.

It should be noted that the selection of ESAL advantages of 3:1 for the low volume LCCA and 2:1 for the high volume LCCA was based on the following considerations:

1. The predicted ESAL advantage using the *flexible pavement design equation* in the 1993 AASHTO pavement design guide was 4.45:1 for the two low volume pavements (Figure 66a) and 2.48:1 for the two high volume alternatives (Figure 67b) when the *a*-values of 0.06 and zero were used as the inputs for the 12-in. cement stabilized and lime-treated working table layers, respectively.
2. The above prediction results were generally consistent with the APT results obtained in this study, where the ESAL advantage on a 2-in. HMA pavement structure was 6.5:1 between sections 4-2B and 4-1B.
3. Therefore, for safety consideration, the LCCA used an ESAL advantage of 3:1 in place of 4:45:1 for the low volume roads and 2:1 instead of 2:48:1 for the high volume alternatives.

CONCLUSIONS

Summary of Cement-Stabilized Soil

The following conclusions can be drawn from the results of this study on cement-stabilized or cement-treated mid to low-PI soils (PI < 25):

- The UCS of cement-soil mixture has a good correlation with the water-cement ratio at molding as defined in this study on the wet side of compaction curves.
- On the wet side of a compaction curve, cement-soil mixture with a water-cement ratio of 2 at molding will have a minimum UCS of 150 psi while cement-treated soils with a water-cement ratio of 3 at molding will have a minimum UCS of 100 psi.
- Cement soil mixtures compacted on the dry side of compaction curves will experience complex physical-chemical changes during specimens' curing period, which could result in a lower strength. Moisture preparation is therefore important due to its potential impact on soil strength.
- The addition of cement to soils with PI less than 25 will have a limited impact on the original compaction curves of cement-soil mixtures if the compaction is conducted before any hydration occurs.
- Using cement to treat wet subgrade has its own limitation with respect to moisture content. This limit is material dependent and beyond this limit; this approach will become uneconomical.
- There are good correlations among the soil-cement loss, the maximum dielectric values, and the 7-day UCS. These good relationships provide some support to the equivalency of wetting-drying durability, tube suction, and 7-day UCS tests.
- Initial molding moisture content significantly affected the maximum dielectric values and the 7-day UCS of stabilized samples. Consequently, the adoption of tube suction or the 7-day UCS tests as the short-cut of dry-wetting durability should consider the influence of molding moisture content. Conversely, initial molding moisture contents had no appreciable effect on the wetting-drying durability test. Therefore, tube suction tests have the potential to be a long-term performance indicator of cement soil mixtures in the field.

Summary of Lime-Treated Soil

The following conclusions can be drawn from the results of this study on lime-soil mixtures on high PI soils (PI ≥ 25):

- Lime treatment shifts optimum moisture contents to the wet side and reduces maximum dry densities from the untreated soil values.
- Lime reduces clay PIs and leads to greater workability and reduction in volume change potential. The current LADOTD method relies on this method of percentage selection for working table applications.
- The E&G method of using pH to determine the required amount of lime though complex and laborious is a logical and widely used method for obtaining higher strengths.
- A combination of methods to evaluate and determine necessary lime contents should be considered like other state departments of transportation and federal agencies (Indiana DOT, the Army, etc.).
- Accelerated curing methods are available and can simulate 28 day cure strengths.
- Treated material prepared near maximum dry densities generally produce the highest UCS test results.
- Heavy clay soil treated with 18 percent lime and 26 percent lime (by weight) generally produce similar UCS results, reflecting saturation and the limits of necessary and effective lime.
- The strength results conducted within this study on lime indicate 150-psi strengths are attainable, but should be verified with project specific laboratory results.
- A method of lime determination is included in Appendix B.

Summary of Lime-Fly Ash

The following conclusions can be drawn from the results of this study on the use of LFA on silty (low PI) soils:

- Fly ash acts as pozzolanic material for the lime, creating a more stable mixture and allowing cementitious bonds to form around and between silt particles.
- Based on the research conducted, the choice of LFA blend ratio should be a 70lime/30fly ash blend with the choice of additive percentage determined by the methods detailed in the previous lime section, specifically the E&G method.
- The benefit of LFA compared to cement or lime alone must be justified based on cost difference and effectiveness of LFA.
- The limited study produced strength results about triple the raw, untreated Soil I with a maximum strength of about 90 psi.

Resilient Modulus and Permanent Deformation

The laboratory repeated load triaxial resilient modulus tests were conducted at 1-day, 7-day, 14-day, and 28-day curing periods; permanent deformation tests were conducted at a 28-day curing period on the compacted or cored subbase or subgrade soils from the field. The findings of this portion of the study are summarized below:

- Among subbase materials, cement-stabilized soil is the best-performing soil followed by lime-treated soil relative to resilient modulus. In addition, cement-stabilized soil performed well in the permanent deformation test.
- Lime-treated soils performed better than untreated soils.
- Sample curing time and confining stresses affected the resilient modulus of the treated materials. Longer cures and higher confining stresses result in higher resilient moduli.
- The resilient modulus alone does not properly characterize the pavement materials. Current pavement design procedure should be revised to incorporate the permanent deformation in addition to the resilient modulus to properly characterize the pavement materials.
- A good correlation between the resilient moduli and permanent strains exists at cycle No. 10,000 (the test's end).
- The proposed permanent deformation test in this study is recommended for the pavement unbound material characterization.

Accelerated Loading Conclusions

The following observations and conclusions were drawn from the field loading portion of this study:

- The cement-stabilized subbase possessed a higher load-induced structural capacity than the lime-treated working table in terms of higher resilient modulus, greater effective structural number (or layer coefficient), and smaller permanent deformation.
- The layer coefficients for the cement-stabilized soil may be assigned to be 0.06; while for lime-treated soil, no structural contribution should be allowed.

The following two research recommendations may be drawn from the above APT results:

- A 12-in. thick mix-in-place cement stabilized soil (CSS) subbase layer can be used in lieu of a 12-in. thick lime-treated working table layer for a wet-subgrade pavement construction in Louisiana.

- A structural layer coefficient of 0.06 may be assigned to a 12-in. thick CSS subbase layer in pavement design if its UCS of 28 days can be achieved greater than 150 psi under an in-situ moisture condition.

The benefits of treated layers within pavement structures include:

- Strength improvements to the naturally wet and weak subgrade soil with the appropriate additive (type and percentage) can create a working platform, treated base, or stabilized base.
- Stiffer layers add to a pavement's useful life. Choosing to count a stabilized subbase may affect other pavement cross section layer design choices. If the stabilized layer is not counted, the pavement may last longer, resulting in less maintenance costs and inconvenience to the traveling public.
- Pavement sections with thinner surface layers may be realized if stabilized subgrade layers are worthy of a structural number contribution in design. Choosing to use a thicker treated layer may reduce the amount of costly stone imports necessary for base and surface layers, possibly resulting in more cost effective designs.
- Treated and stabilized subgrades may reduce the need to rebuild entire pavement cross sections to full depth. Future maintenance and rehabilitation efforts (and funds) can therefore focus on the upper wearing layers.
- The life cycle analysis of this study reveals that using a 12-in. cement stabilized soil subbase in lieu of a 12-in. lime-treated working table layer will create the annual cost savings of 37 percent and 31 percent for typical Louisiana low and high volume pavements, respectively.

RECOMMENDATIONS

LADOTD should implement the following recommendations based on the conclusions of this report:

- The philosophy of treated subgrade layer with target strengths should be expanded and applied to all subgrades susceptible to moisture intrusion in Louisiana—rather than optional, working table, subgrade treatment. Treating clays with lime and silts with cement will create stronger foundations for pavement structure because when the appropriate additive, and amount, is added, the treatment modifies the soil to create consistent, drier layers, with reduced moisture sensitivity as compared to the raw natural soil; strength is also improved.

Three alternatives are available and their selection should be based on a benefit cost analysis.

LIME

- Lime treatment should be considered as a treatment alternative since lime treated clays can attain strengths between 100 and 150 psi. These strengths will likely require higher lime percentages than working table percentages. Methods have been provided to determine these optimum lime contents. Additive percentages should be verified with strength tests prior to project applications. Care should also be taken with the different LADOTD strength terminology (treated vs. stabilized) and their respective values.

LFA

- Lime fly ash should be considered as an alternative material for the treatment of silty soils. The desired strength requirements may require higher additive percentages than cement and additional spreader passes. During elevated cement prices and cement shortages, LFA may prove an effective alternative in silty soils.

CEMENT

- Cement treatment in wet areas should be addressed using the attached method outlined in Appendix A. The method outlines a design procedure to determine cement dosage to stabilize wet subgrades.

The following chart can be used to determine applications:

Table 28
Recommended typical applications

Additive	**Soil Type**	**Strength Target**		**Testing Method**
Cement	Silty, PI < 25	Working Table 50 psi	Treated Subbase 100 psi to 150 psi	Appendix A
Lime	Clayey, PI ≥ 25	Working Table 50 psi	Treated Subbase 100 psi to 150 psi	Appendix B
LFA	Silty, PI < 25	Working Table 50 psi	NA	Appendix B

Additionally, updates to the Standard Specifications, which foster implementation and increased options of chemical additives to treat wet subgrade soils at competitive costs, while still producing effective subbase and treated subgrade layers is paramount.

LADOTD should develop a design guide for their design needs; three documents have been attached to the end of this report in the Appendix. It is recommended that the Department's pavement and geotechnical groups refine these documents and seek approval by the appropriate committees and chief engineer prior to implementation.

Appendix B: Draft - Design Procedures for Soil Modification or Stabilization
Appendix C: Draft - Updates to Standard Specification, 304, Lime Treatment
Appendix D: Draft - New Standard Specification, 304.XX, Lime Fly Ash Treatment

ACRONYMS, ABBREVIATIONS, AND SYMBOLS

°F	Degrees Fahrenheit
µm	Micrometer
a	FWD load plate radius
AASHTO	American Association of State Highway Transportation Officials
ALF	Accelerated Loading Facility
A_n	Nominal Cement Content
APT	Accelerated Pavement Testing
ASTM	American Society of Testing Materials
CSH	Calcium Silicate Hydrate
CSS	Cement Stabilized Soil
C_{wm}	Moisture-Cement Ratio at Sample Molding
D	Total Pavement thickness above the subgrade, inches
d_0	Deflection measured at the center of the FWD plate (adjusted to 68°F)
D0	Deflection Reading Zero
DOT	Department of Transportation
d_r	Deflection at a distance r from the center of FWD plate, in.;
DV	Dielectric Value
E&G	Eades & Grim
EASL	Equivalent Single Axle Load
ELSYM5	multi-layer elastic analysis program,
EMCRF	Engineering Materials Characterization Research Facility
E_p	Effective Pavement Modulus
ε_r	Recoverable Elastic Strain
FA	Fly Ash
ft.	Foot
FWD	Falling Weight Deflectometer
g	Grams
HMA	Hot Mix Asphalt
in.	Inch
ITTE	Institute of Transportation and Traffic Engineering
kip	1000 pounds
ksi	Kips per Square Inch
LADOTD	Louisiana Department of Transportation
lb.	Pounds

LCCA	Life-Cycle Cost Analysis
LFA	Lime Fly Ash
LL	Liquid Limit
LTPP	Long Term Pavement Performance
LTRC	Louisiana Transportation Research Center
LVDT	Linear Variable Differential Transformers
MDD	Multi-Depth Deflectometer
min.	Minutes
mm	Millimeters
M_r	Resilient Modulus
M_R	Backcalculated Subgrade Resilient Modulus
MTS	Material Testing System
N	Number of Cycles
NDT	Non-Destructive Testing
p	FWD load plate pressure
P	Applied Load
PCA	Portland Cement Association
pcf	Pounds per Cubic Foot
PI	Plasticity Index
PL	Plastic Limit
PRF	Pavement Research Facility
psi	Pounds per Square Inch
r	Distance from center of load
RAP	Recycled Asphalt Pavement
RMS	Root Means Squared
σ_d	Maximum Deviator Stress
SN	Structural Number
SN_{eff}	effective structural number
TS	Tube Suction
UCS	Unconfined Compressive Strength
UNO	University of New Orleans
USCS	Unified Soil Classification System
w/c	Water to Cement Ratio
w_{hy}	Hygroscopic Moisture Content
w_m	Moisture Content at Molding

w_n	Nominal Moisture Content
γ_d	Dry Density

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APPENDIX

APPENDIX A

DRAFT – Design Procedures to determine Cement Dosage to Stabilize Wet Subgrade

Description:
The procedure described here is prepared based on the results of LTRC research on cement-stabilized wet highway subgrades. This procedure is based on the laboratory finding that the unconfined compressive strength (UCS) of cement-soil mixtures has a reasonably well defined correlation with the water-cement ratio at molding of materials.

Definition:
Water-Cement Ratio: $R_{wc} = w/C$. Here, w is the molding moisture content of cement-soil mixture determined from the dry weight of soil only; C is the cement content defined as the percentage ratio of the weight of cement to the dry weight of soil.

Correlation:

Target value of UCS	Water-Cement Ratio
50 psi	5.0
100 psi	3.0
150 psi	2.0
200 psi	1.75

Laboratory:

Step 1: Select a representative subgrade sample from the roadway to be stabilized and determine its field moisture content, w_f.

Step 2: Determine its Plastic Index (PI) and optimum moisture content, w_o. If PI < 25, follow the procedure described in this document.

Step 3: Select the target value of 100 psi for the unconfined compressive strength (UCS) with the corresponding Water-Cement Ratio, R_{wc} of 3.

Step 4: Calculate the cement content in percent at the field moisture content of the soil as follows:

$$C_f = \frac{w_f}{R_{wc}}$$

Step 5: Use TR 432 to validate the target value of UCS at the field moisture content with cement contents of C_f -2, C_f, and C_f +2 in percent and cured for 3 and 7 days.

Step 6: Select field cement content to use from Step 5.

Field Construction:

Step 7: Compact the 12-in. cement-stabilized wet subgrade layer to reach 100% of dry density at the corresponding field moisture content determined by the standard proctor compaction test.

Step 8: Allow the cement-stabilized subgrade to be cured for the time duration determined by the lab test to research strength.

Step 9: (optional for emergency) Use the cement content determined in Step 4 in cases where the field soil is different from the one tested in the laboratory, the cement content determined in Step 4 can be used directly.

APPENDIX B

DRAFT - Design Procedures for Soil Modification or Stabilization

1.0 General: The following methods should be evaluated by the engineer to determine the most appropriate actions to determine which treatment is suitable given the particular and specific design situation. This guide will clarify the two treatment objectives modification and stabilization:

- 1.1 Subgrade Modification is treatment that creates a working table for construction equipment. No credit is accounted for in the modification in the pavement design. This is <u>not</u> the same as stabilization.
- 1.2 Subgrade Stabilization is stabilization that enhances the strength of the designed layer. This increase in strength is taken in to account in the pavement design process. This design requires more detailed laboratory testing.

Various subgrade guidelines are discussed below that give the contractor options on construction practices to achieve the required performance; however, it is the responsibility of the designer to use the necessary judgment to determine which methods are most applicable and cost effective, based on local environmental and project considerations.

2.0 Mechanical Modification or Stabilization: Mechanical modification and mechanical stabilization imply changing the soil properties physical or compactive efforts.

- 2.1 Add granular thickness to the pavement section sufficient to develop acceptable pressure distribution over the wet soils. A separator fabric is required.
- 2.2 Remove and replace wet soils to a predetermined depth, if suitable support is available at that depth. The backfill must be in accordance with the Standard Specifications and be able to withstand the wheel load would be covered in this

3.0 Geosynthetic Stabilization: Geogrids change the performance of the roadway through several primary mechanisms: tensile reinforcement, confinement, lateral spreading reduction, separation, construction uniformity, and reduction in strain.

- 3.1 Geogrids may allow reduced aggregate thicknesses when combined with the remove and replace options under section 2.1. Geogrids shall be in accordance with Standard Specification Section 1019.

4.0 Chemical Modification or Stabilization: The use of chemical additives like cement, lime, fly ash, or a combination of these, alters the physical and chemical properties of the soil. The two mechanisms by which chemicals alter the soil into a stable subgrade are:

4.1 Increase in particle size by cementation, internal friction among the agglomerates, greater shear strength, reduction in the plasticity index, and reduced shrink/swell potential.

4.2 Absorption and chemical binding of moisture to facilitate compaction.

Design Procedure

5.0 Criteria for Chemical Selection: If chemical stabilization is chosen as the most economical or feasible option, the following criteria should be considered for chemical selection based on the index properties of the soil. (excerpt from IDOT 2008 Design Manual)

5.1 Chemical Selection for Stabilization

a. Lime: If PI > 10 and clay content (2µm) >10%
b. Cement: If PI ≤ 10 and < 20% passing the No. 200 sieve (75µm)
Note: Lime shall be quicklime only

5.2 Chemical Selection for Modification.

a. Lime: If PI ≥ 5 and > 35% Passing No. 200 sieve
b. Lime fly ash blends: 5 < PI < 20 and > 35% Passing No. 200.
c. Cement and/or Fly ash: PI < 5 and ≤ 35% Passing No. 200.
Notes: Fly ash shall be class C only.
Lime Kiln Dust (LKD) shall not be used in blends

5.3 Suggested Starting Points for Chemical Quantities Estimates for Modification or Stabilization: The following are starting point estimates. The required additive percentage (by weight) for each soil must be verified with specific laboratory results.

a. Lime: 4% to 9%
b. Cement: 4% to 10%
c. Fly ash (Class C): 10% to 25%

5.4 Strength Requirements for Stabilization and Modification: Unconfined Compression tests will be used to gauge the effects of modification and/or stabilization. The reaction of the soil with the additive is critical to verify reactivity. The strengths within this section only indicate potential reactivity, the 28-day (or 7-day accelerated) strengths of 300 psi for stabilization and 100 psi for modification still apply.

a. Lime Stabilization: Two samples with 5% quicklime (by dry weight) prepared at the optimum moisture content and maximum dry density (AASHTO T 99). Cure the specimens for 48 hours at 120°F in the laboratory and test as per AASHTO T 208. The strength gain from the lime-soil specimens must be at least 50 psi greater than the natural soils.

b. Cement Stabilization: Two samples with 4% cement (by dry weight) prepared at the optimum moisture content and maximum dry density (AASHTO T 99). Cure the specimens for 48 hours at 120°F in the laboratory and test as per AASHTO T 208. The strength gain from the cement-soil specimens must be at least 100 psi greater than the natural soils.

c. Soil Modification: Strength improvements from soil modification on soil support are not accounted for in the pavement design process; however, approved chemical additives shall attain an increase in strength of 30 psi over the natural soils when prepared and tested in the same manner as noted in the lime and cement stabilization sections above.

6.0 Laboratory Test Requirements

6.1 Soil Sampling and Suitability: Soil from the project site shall be collected in sufficient quantity to perform the specified tests.

a. Grain size and Hydrometer test results in accordance with DOTD TR 407

b. Atterberg limits in accordance with DOTD TR 428

c. Maximum Dry Unit Weight of 92 pcf (minimum) in accordance with AASHTO T 99

d. Loss of ignition (LOI) not more than 3% by dry weight of soil in accordance with AASHTO T 267

e. Carbonates not more than 3% by dry weight of the soils, if required

f. As received moisture content in accordance with DOTD TR 403
g. pH results for the soil and lime separately

6.2 Lime Required for Modification or Stabilization: Lime reacts with medium, moderately fine, and fine-grained soils to produce decreased plasticity, increased workability, reduced swelling, and increased strength. Reactivity is based on pH, organic content, natural drainage, and clay mineralogy. The following procedure shall be utilized to determine the amount of lime required:

a. Perform mechanical and physical tests on the soil
b. Determine the separate pH of soil and lime samples
c. Determine the optimum lime content using the Eades and Grim pH test.
   - A sufficient amount of lime shall be added to soils to produce a pH of 12.4 or equal to the lime itself. The optimum lime content shall be determined corresponding to the maximum pH of the soil lime mixture.
   - Representative samples of air-dried, minus No. 40 soil, equal to 25 g of oven-dried soil are weighed to the nearest 0.1 g and poured into 150 ml (or larger) plastic bottles with screw on tops.
   - It is advisable to set up five bottles with lime percentages of 3, 4, 5, 6, and 7. This will insure, in most cases, that the percentage of lime required can be determined in one hour. Heavier clays may require higher percentages, and therefore more bottles. Weight the lime to the nearest 0.01 g and add it to the soil. Shake the bottle well to mix the soil and dry lime.
   - Add 100 ml of CO_2-free distilled water to the bottles
   - Shake the soil-lime-water mixture until there is no evidence of dry material on the bottom. Shake for a minimum of 30 seconds.
   - Shake the bottles for 30 seconds every 10 minutes.
   - After one hour, transfer part of the slurry to a plastic beaker and measure the pH. The pH meter must be equipped with a Hyalk electrode and standardized with a buffer solution having a pH of 12.00.
   - Record the pH for each of the lime-soil mixtures. If the pH

readings go to 12.40, then the lowest percent lime that gives a pH of 12.40 is the percentage required to stabilize the soil. If the pH does not go beyond 12.30 and 2 percentages of lime give the same readings, the lowest percent that gives a pH of 12.30 is the amount required to stabilize the soil. If the highest pH is 12.30 and only one pH give that value, additional test bottles should be started with the larger lime percentages.

d. Conduct Atterberg limit tests on the soil-lime mixture corresponding to the optimum lime content as determined above.

e. Conduct compaction tests shall be conducted in accordance with AASHTO T 99 on the optimum lime-soil mixture to evaluate the drop in maximum dry density in relation to time (depending on the delay between the lime-soil mixing and compaction.)

In the case of Stabilization, the Unconfined Compression test (AASHTO T208), or Resilient Modulus (AASHTO T 307) tests at 95% standard compaction shall be performed in addition to the above tests corresponding to the optimum lime-soil mixture.

6.3 Cement Required for Stabilization or Modification: The criteria for cement percentages required for stabilization shall be documented in Appendix A.

6.4 Lime Fly Ash Required for Stabilization or Modification: The criteria for percentages shall follow the subsequent guidelines, with the goal of 300 psi for stabilization and 100 psi for modification. The ratio between lime and fly ash should be in the range of 1:1 and 1:9, respectively.

7.0 Construction Considerations: Modification of soils to speed construction by drying out wet subgrades with cement, lime, and fly ash is not as critical as stabilization, which is designed to be part of the pavement structure. Thus, if and when chemically stabilized subgrades are used to reduce the overall thickness of the roadway then the stabilized layer must be built under tight construction specifications. The following considerations are provided to aid in the design of modified and stabilized subgrade soils:

7.1 Perform recommended tests on each soil to verify that the soil will react with the chosen chemicals, and then determine the appropriate amount of chemical necessary to produce the desired/required results.

7.2 More chemicals may not always give the best results.

7.3 Sulfate, when mixed with calcium, will expand. Soils having over 10% sulfate content shall not be mixed with chemicals.

7.4 Chemicals used shall meet the LADOTD Standard Specifications.

7.5 Proof rolling is required before placing the base or subbase. Pavement shall not be installed before curing is complete.

7.6 The density of modified and stabilized soils will likely be different from that of natural soils. Standard Proctor tests should be performed in the laboratory to estimate the appropriate target density.

7.7 Uniform distribution of chemicals throughout the soil is very important.

7.8 Curing takes 7 days of weather at 50°F or above for stabilization to occur. Heavy construction equipment is not allowed on the stabilized grade during the curing period.

7.9 The maximum dry density of the soil-lime mixture is lower than in untreated soils. Maximum dry density reductions of approximately 3 to 5 pcf is common for a given compactive effort. It is, therefore, important that the laboratory provide the appropriate density.

7.10 Moisture content of the modified or stabilized subgrade should be maintained above the optimum moisture content of the treated material during curing.

APPENDIX C

DRAFT - Modifications to the Existing Lime Treatment Specification

Section 304

Lime Treatment

304.01 DESCRIPTION. This work consists of constructing one or more courses of a mixture of lime and soil, or soil-aggregate, and water in accordance with these specifications, in conformity with the lines, grades, thickness, and sections shown on the plans.

Lime treatment will be designated as Type B, C, D, or E. Type B shall be used for base, subbase, or subgrade treatment (150 psi). Type C shall be used for conditioning for cement treatment or stabilization. Type D shall be used for working table treatment under an embankment. Type E shall be used for conditioning and drying of subgrades under a base course. Lime treatment shall be in accordance with these specifications and Table 304-2.

304.02 MATERIALS. Materials shall comply with the following Sections and Subsections:

Emulsified Asphalt	1002
Water	1018.01
Lime	1018.03

Quality assurance requirements shall be as specified in the latest edition of the Department's publication entitled "Application of Quality Assurance Specifications for Embankment and Base Course."

In order to meet air quality standards, the contractor may be required to use central plant mixing, lime slurry, or granular lime in dust sensitive areas at no direct pay. The Department will identify dust sensitive areas in the plans.

304.03 EQUIPMENT. Equipment necessary to produce a finished product, meeting specification requirements shall be furnished and maintained by the contractor. An approved in-place mixer meeting the requirements of Subsection 303.03 shall be used for Type B and C treatments. An approved in-place mixer meeting the requirements of Subsection 303.03 shall be used for Types D and E treatments unless the engineer approves other equipment.

The contractor shall furnish and maintain a water distribution truck or other suitable equipment with a pressure distributor capable of uniformly distributing the required amount of water.

304.04 GENERAL CONSTRUCTION REQUIREMENTS. Lime shall be protected from moisture prior to use. Water shall be added as needed during mixing and remixing operations, during the curing period, and to keep the cured material uniformly moist until covered.

When granular quicklime is applied in dry form, precautions shall be taken to prevent injury to persons, livestock, and plants. Quicklime spilled or deposited outside areas designated for treatment shall be immediately collected and buried or satisfactorily slaked.

Lime shall not be applied on a frozen foundation or when the ambient air temperature is below 35°F .

(a) Type B Treatment: Lime shall be incorporated in the following sequence: Spreading the lime, initial mixing, watering, sealing and mellowing for at least 48 hours, mixing until pulverization requirements are met, compacting, finishing, and maintaining in accordance with Subsection 304.10. The percent of lime for Type B treatment will be determined in accordance with DOTD TR 416. After lime treatment, the treated soil shall have a maximum Liquid Limit of 40 and a maximum PI of 10.

When using quicklime, the contractor shall provide special precautions to ensure that adequate water is added as needed during initial and final mixing to ensure that the quicklime becomes fully hydrated and that the lime-soil mixture is at the proper moisture content for compaction.

(b) Type C Treatment: Lime shall be incorporated in the following sequence: Spreading the lime, initial mixing, watering, sealing and mellowing for a minimum of 48 hours, mixing until pulverization requirements are met, compacting, finishing, and maintaining. The percent lime for Type C treatment will be as required by the plans or as directed.

(c) Type D Treatment: One increment of lime shall be spread and mixed with materials to be treated, watered as required and compacted to the satisfaction of the engineer. The percent of lime for Type D treatment will be as required by the plans or as directed.

(d) Type E Treatment: One increment of lime shall be spread and mixed with materials to be treated and compacted and finished in accordance with the normal embankment construction procedures of Section 203. Unless specified, the percentage of lime for Type E treatment will be determined in accordance with DOTD TR 416.

304.05 SPREADING AND MIXING. The percentage of lime to be incorporated shall be as specified. When not specified, the required percentage of lime will be determined by the laboratory in accordance with DOTD TR 416.

A unit weight of 35 pounds per cubic foot will be used to compute the required application rate of hydrated lime or granular quicklime regardless of the actual unit weight of the lime used.

Lime may be furnished in bags or bulk and distributed, in powder form, granular or in slurry, and in the required proportion. Dry lime shall be prevented from blowing by adding water or by other suitable means.

Lime shall be uniformly spread and mixed with the soil to the width and depth shown on the plans or as directed. The Department will determine lime spread rate in accordance with DOTD TR

436. Any procedure, which results in excessive loss, or displacement of lime, shall be discontinued.

Areas to which lime is applied shall be processed on the same day as application is made. Any lime not processed within 6 hours and lime lost or damaged before incorporation due to rain, wind or other cause will be rejected, deducted from measured quantities, and shall be replaced by the contractor. At no time will the contractor be paid more than once for lime treatment of a section of roadway.

(a) Type B Mixing: After the 48-hour mellowing period, the lime treated mixture shall be kept moist and be manipulated with an in-place mixer until the pulverization requirements of Subsection 304.06 have been met.

(b) Type C Mixing: Following the 48-hour mellowing period, the lime treated mixture shall be thoroughly manipulated with an in-place mixer to the satisfaction of the engineer. The mixture shall meet the pulverization requirements of Subsection 304.06 prior to subsequent stabilization or treatment with portland cement.

(c) Types D and E: Mixing shall be accomplished with an in-place mixer unless the engineer approves other equipment.

304.06 PULVERIZATION. For Types B and C treatment, the pulverized mixture, when tested in accordance with DOTD TR 431, shall meet the gradation requirements in Table 304-1 below.

Table 304-1
Gradation Requirements for Types B & C Lime Treatment

U. S. Sieve, Inches	Percent Passing By Weight (Mass)
3/4	95
No. 4	50

Pulverization requirements for Type B and C treatments shall be met prior to final compaction and finishing.

304.07 COMPACTING AND FINISHING.

(a) Type B: After meeting the pulverization requirement, the mixture shall be uniformly compacted to at least 95.0 percent of maximum dry weight density of the lime-soil mixture. The maximum dry density of the lime-soil mixture will be determined in accordance with DOTD TR 415 or TR 418 and in-place density in accordance with DOTD TR 401. Compaction and finishing operations shall be completed within 6 hours after meeting pulverization requirements. One density test will be taken per 1,000 linear feet per roadway or 2,000 linear feet per shoulder constructed separately in accordance with DOTD TR 401. At places inaccessible to rollers, such as edges adjacent to curb and gutter sections, the mixture shall be compacted using devices that will obtain uniform compaction to required density without damage to adjacent structures. Any section not meeting the required density shall be reconstructed in accordance with these specifications at no direct pay. Reconstruction shall include the addition of the specified amount of lime.

The final finish shall meet grade and cross-slope requirements and shall have a smooth, uniform, closely knit surface, free from ridges, waves, loose material or laitance.

(b) Type C: Type C lime conditioned materials shall be shaped and uniformly compacted to the required sections. The contractor shall make reasonable efforts to conform to the compaction requirements of (a) above. When conditions, such as a yielding subgrade, make this impractical or detrimental, the contractor shall establish an optimum rolling pattern.

(c) Type D: Type D lime treated materials shall be uniformly compacted and finished to the satisfaction of the engineer. The contractor shall make reasonable efforts to conform to the compaction requirements of (a) above. When conditions, such as a yielding subgrade, make this impractical or detrimental, the contractor shall establish an optimum rolling pattern.

(d) Type E: Type E lime treated materials shall be compacted and finished in accordance with the normal embankment construction procedures of Section 203.

304.08 QUALITY CONTROL. Construction methods shall prevent contamination, segregation, soft spots, wet spots, laminations, and other deficiencies. The contractor shall be responsible for taking such tests as necessary to adequately control the work.

(a) Type B Lime Treatment: The contractor shall control the grade, cross-slope, lime spread, mixing, pulverization, thickness, width, density, and curing to construct a completed course that is uniform and conforms to the acceptance requirements.

(b) Type C Lime Treatment: The contractor shall control the lime spread, mixing, and pulverization to construct a completed course that is uniform and conforms to the acceptance requirements.

(c) Type D Lime Treatment: The contractor shall control the lime spread and mixing to construct a completed course that is uniform and conforms to the acceptance requirements.

(d) Type E Lime Treatment: The contractor shall control the lime spread, mixing, and density to construct a completed layer that is uniform and conforms to the acceptance requirements.

304.09 PROTECTION AND CURING (TYPE B TREATMENT).
After finishing operations have been completed, the material shall be protected against rapid drying for 72 hours by applying an asphalt curing membrane complying with Section 506. The application shall be placed immediately following smooth rolling and shall be adequately maintained during the 72-hour curing period.

304.10 MAINTENANCE.

(a) Type B Lime Treatment: Maintenance of Type B Lime Treatment will be in accordance with Subsection 303.09.

(b) Types C, D, and E Treatments: These treatments shall be maintained by the contractor to prevent damage to the lime treated layer as directed.

304.11 DIMENSIONAL TOLERANCES (TYPE B TREATMENT).

(a) General: Thickness and width of completed lime treated courses will be checked for acceptance in accordance with DOTD TR 602.

Areas not meeting tolerances specified herein will be delineated and shall be corrected to plan dimensions by scarifying, adding lime, remixing, and recompacting deficient areas at no direct pay.

(b) Thickness Requirements: Underthickness shall not exceed 3/4 inch and overthickness shall not exceed 1 in.

(c) Width Requirements: Roadway base course width shall not vary from plan width in excess of +6 inches. Shoulder base course width shall not vary from plan width in excess of +3 in. No tolerances are provided for under widths of shoulder or roadway bases. When the base course for roadway and shoulders are constructed at the same time, the 6-in. width tolerance will be applied. Base course width deficiencies in excess of foregoing tolerances shall be corrected at the contractor's expense.

304.12 MEASUREMENT.

(a) Lime: Lime will be measured by the ton. When lime is furnished in bags, the number of bags used and the weight (mass) per bag will be used for measurement. When lime is furnished in bulk, the contractor shall furnish certified weights (mass) for each transport load.

(b) Treatment: The quantities of Type B, C, and D lime treatment for payment will be the design areas as specified on the plans and adjustments thereto. Design quantities are based on the horizontal dimensions of the completed lime treatment shown on the plans. Design quantities will be adjusted if the engineer makes changes to adjust to field conditions if design errors are proven, or if design changes are necessary.

No measurement for payment will be made for Type E lime treatment other than as specified.

(c) Water and asphalt curing materials will not be measured for payment.

304.13 PAYMENT.

(a) Lime: Payment for lime will be made at the contract unit price per ton. If quicklime is used in a slurry, payment will be made at the unit price for hydrated lime after converting the quicklime to the equivalent weight (mass) of hydrated lime by multiplying the weight (mass) of quicklime by 1.32 then multiplying that product by the purity of the lime.

(b) Treatment: Payment for Types B, C, and D lime treatment will be made at the contract unit prices per square yard (sq m). Type B lime treatment will be adjusted as specified in Section 1002 for specification deviations of asphalt materials. The Materials and Testing Section will provide the payment adjustment percentage for properties of asphalt materials. Payment for Type E Treatment will be at the contract unit price per ton of lime used.

Payment will be made under:

Item No.	Pay Item	Pay Unit
304-01	Lime	Ton
304-02	Lime Treatment (Type B) ___in. Thick	Square Yard
304-03	Lime Treatment (Type C) ___in. Thick	Square Yard
304-04	Lime Treatment (Type D) ___in. Thick	Square Yard
304-05	Lime Treatment (Type E)	Ton

Table 304-2
Types of Lime Treatment

B	Base or Subbase	1. One application of lime 2. Initial mixing 3. 48-hour mellowing or aging period 4. Pulverization[1] 5. Density control 6. Minimum thickness and width 7. 72-hour cure with asphalt curing membrane
C	Conditioning for Cement Treatment or Stabilization	1. One application of lime 2. Initial mixing 3. 48-hour mellowing or aging period 4. Pulverization[1] 5. Compact to engineer's satisfaction 6. No cure required
D	Working Table (Under Embankment)	1. One application of lime 2. Mixing[2] 3. Compact to engineer's satisfaction 4. No cure required
E	Conditioning and Drying (Subgrades Under a Base Course)	1. One application of lime per embankment lift 2. Mixing[2] 3. Embankment construction requirements including density 4. No cure required

[1] In-place mixer shall be required.

[2] In-place mixer shall be required unless the engineer approves other equipment.

APPENDIX D

DRAFT - Lime-Fly Ash Treatment Specification

Section 3XX
Lime-Fly Ash Treatment

3XX.01 DESCRIPTION. This work consists of constructing one course of a mixture of lime and fly ash and soil and water in accordance with these specifications, in conformity with the lines, grades, thickness, and sections shown on the plans.

3XX.02 MATERIALS. Materials shall comply with the following Sections and subsections:

Emulsified Asphalt	1002
Water	1018.01
Lime	1018.03
Fly Ash	1018.15

3XX.03 EQUIPMENT. Equipment necessary to produce a finished product meeting specification requirements shall be furnished and maintained by the contractor. An approved in-place mixer meeting the requirements of Subsection 303.03.

3XX.04 GENERAL CONSTRUCTION REQUIREMENTS. Lime and fly ash shall be protected from moisture prior to use. Water shall be added as needed during mixing and remixing operations during the curing period and to keep the cured material uniformly moist until covered.

When granular quicklime is applied in dry form, precautions shall be taken to prevent injury to persons, livestock, and plants. Quicklime spilled or deposited outside areas designated for treatment shall be immediately collected and buried or satisfactorily slaked.

Lime and fly ash shall not be applied on a frozen foundation or when the ambient air temperature is below 35°F.

Lime and fly ash shall be incorporated in the following sequence: Spread the lime and initially mix until pulverization requirements are met. Spread the fly ash and remix. Fly ash remixing may begin immediately after the lime is mixed and shall be complete within 48 hours of initial lime mixing. If required to facilitate construction operations, the lime treated soil may be compacted prior to fly ash spreading. After the fly ash is spread, mix, compact, finish, and maintain the lime-fly ash-soil mixture in accordance with Subsection 304.10.

3XX.05 SPREADING AND MIXING. Lime and fly ash shall be incorporated at the following rates:

Lime:	30 lb. per square yard
Fly ash:	100 lb. per square yard

A unit weight of 35 pcf will be used to compute the required application rate of hydrated lime or granular quicklime regardless of the actual unit weight of the lime used. A unit weight of 60 pcf will be used to compute the required application rate of fly ash regardless of the actual unit weight of the fly ash used.

Lime may be furnished in bags or bulk and distributed, in powder form, granular or in a slurry, and in the required proportion. Dry lime shall be prevented from blowing by adding water or by other suitable means.

Lime and fly ash shall be uniformly spread and mixed with the soil to the width and depth shown on the plans or as directed. The Department will determine lime and fly ash spread rates in accordance with DOTD TR 436. Any procedure that results in excessive loss or displacement of lime or fly ash shall be discontinued.

Lime and fly ash shall be spread in two operations. Areas to which lime and fly ash is applied shall be processed on the same day as application is made. Lime exposed to air for more than 6 hours and lime and fly ash lost or damaged before incorporation due to rain, wind or other cause will be rejected, deducted from measured quantities, and shall be replaced by the contractor at no direct pay.

3XX.06 PULVERIZATION. After the initial mixing of lime and before spreading fly ash, the pulverized mixture, when tested in accordance with DOTD TR 431, shall meet the gradation requirements in Table 3XX-1 below:

Table 3XX-1

Gradation Requirements for Lime Treated Soil Prior to Fly Ash Application

U.S. Sieve Size	Percent Passing By Weight
¾	95
No.4	50

3XX.07 COMPACTING AND FINISHING. After the lime and fly ash have been mixed into the soil, the mixture shall be uniformly compacted to at least 95.0 percent of maximum dry weight density. The maximum dry weight density will be determined in accordance with DOTD TR 415 or TR 418 and in-place density in accordance with DOTD TR 401. Compaction and finishing operations shall be completed within 6 hours after fly ash mixing. One density test will be taken per 1,000 linear feet per roadway or 2,000 linear feet per shoulder constructed separately in accordance with DOTD TR 401. At places inaccessible to rollers, such as edges adjacent to curb and gutter sections, the mixture shall be compacted using devices that will obtain uniform compaction to required density without damage to adjacent structures. Any section not meeting the required density shall be reconstructed in accordance with these specifications at no direct pay. Reconstruction shall include the addition of the specified amount of lime.

The final finish shall meet grade and cross-slope requirements and shall have a smooth, uniform, closely-knit surface, free from ridges, waves, loose material, or laitance.

304.08 QUALITY CONTROL. Construction methods shall prevent contamination, segregation, soft spots, wet spots, laminations, and other deficiencies. The contractor shall be responsible for taking such tests as necessary to adequately control the work.

The contractor shall control the grade, cross-slope, lime spread, mixing, pulverization, thickness, width, density, and curing to construct a completed course that is uniform and conforms to the acceptance requirements.

3XX.09 PROTECTION AND CURING. After finishing operations have been completed, the material shall be protected against rapid drying for 72 hours by applying an asphalt curing membrane complying with Section 506. The application shall be placed immediately following smooth rolling and shall be adequately maintained during the 72- hour curing period.

3XX.10 MAINTENANCE. Maintenance will be in accordance with Subsection 303.09.

3XX.11 DIMENSIONAL TOLERANCES.

(a) General: Thickness and width of completed lime-fly ash treated course will be checked for acceptance in accordance with DOTD TR 602. Areas not meeting tolerances specified herein will be delineated and shall be corrected to plan dimensions by scarifying, adding lime, remixing, and recompacting deficient areas at no direct pay.

(b) Thickness Requirements: Underthickness shall not exceed ¾ inch and overthickness shall not exceed 1 in.

(c) Width Requirements: The width of the completed lime-fly ash treated course will be determined in accordance with DOTD TR 602. The completed lime-fly ash treated course width shall not vary from plan width in excess of +6 in. Shoulder course width shall not vary from plan width in excess of +3 in. Base course width deficiencies in excess of foregoing tolerances shall be corrected as follows at the contractor's expense.

3XX.12 MEASUREMENT.

(a) Lime: Lime will be measured by the ton. When lime is furnished in bags, the number of bags used and the weight per bag will be used for measurement. When lime is furnished in bulk, the contractor shall furnish certified weights for each transport load.

(b) Fly Ash: Fly ash will be measured by the ton. Fly ash shall be furnished in bulk, and the contractor shall furnish certified weights for each transport load.

(c) Treatment: The quantities lime-fly ash treatment for payment will be the design areas as specified on the plans and adjustments thereto. Design quantities are based on the horizontal dimensions of the completed lime-fly ash treatment shown on the plans. Design quantities will be adjusted if the engineer makes changes to adjust to field conditions, if design errors are proven, or if design changes are necessary.

(d) Water and asphalt curing materials will not be measured for payment.

3XX.13 PAYMENT.

(a) Lime: Payment for lime will be made at the contract unit price per ton. If quicklime is used in a slurry, payment will be made at the unit price for hydrated lime after converting the quicklime to the equivalent weight of hydrated lime by multiplying the weight of quicklime by 1.32 then multiplying that product by the purity of the lime.

(b) Fly Ash: Payment for fly ash will be made at the contract unit price per ton.

(c) Treatment: Payment for lime-fly ash treatment will be made at the contract unit prices per square yard, adjusted as specified in Section 1002 for specification deviations of asphalt materials. The Materials and Testing Section will provide the payment adjustment percentage for properties of asphalt materials.

Payment will be made under:

Item No.	Pay Item	Pay Unit
3XX-01	Lime	Ton
3XX-02	Fly Ash	Ton
3XX-03	Lime-Fly Ash Treatment ___in. thick	Square Yard