



# PERFORMANCE OF SOIL STABILIZATION AGENTS

Report Number: K-TRAN: KU-01-8

By

Justin P. Milburn and Robert L. Parsons, Ph.D., P.E.

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## Introduction

Poor subgrade soil conditions can result in inadequate pavement support and reduce pavement life. Soils may be improved through the addition of chemical or cementitious additives. These chemical additives range from waste products to manufactured materials and include lime, Class C fly ash, Portland cement and proprietary chemical stabilizers. These additives can be used with a variety of soils to help improve their native engineering properties. The effectiveness of these additives depends on the soil treated and the amount of additive used.

## Project Objective

This report contains a summary of the performance of lime, cement, Class C fly ash, and Permazyme 11-X used with a wide range of soils. Each of the chemical additives tested is designed to combine with the soil to improve the texture, increase strength and reduce swell characteristics. These products were combined with a total eight different soils with classifications of CH, CL, ML, SM, and SP.

## Project Description

Durability testing procedures included freeze-thaw, wet-dry, and leach testing. Atterberg limits and strength tests were also conducted before and after selected durability tests. Changes in pH were monitored during leaching. Relative values of soil stiffness were also tracked over a 28-day curing period using the soil stiffness gauge.

## Project Results

Lime and cement stabilized soils showed the most improvement in soil performance for multiple soils, with fly ash treated soils showing substantial improvement. The results showed that for many soils more than one stabilization option may be effective for the construction of durable subgrades. The enzymatic stabilizer did not perform as well as the other stabilization alternatives.

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Report No. K-TRAN: KU-01-8  
FINAL REPORT

## **PERFORMANCE OF SOIL STABILIZATION AGENTS**

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Lawrence, Kansas



MAY 2004

### **K-TRAN**

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<b>16 Abstract</b> <p>Poor subgrade soil conditions can result in inadequate pavement support and reduce pavement life. Soils may be improved through the addition of chemical or cementitious additives. Such chemical additives range from waste products to manufactured materials and include lime, Class C fly ash, Portland cement and proprietary chemical stabilizers. These additives can be used with a variety of soils to help improve their native engineering properties.</p> <p>This report contains a summary of the performance of lime, cement, Class C fly ash, and Permazyme 11-X used with a wide range of soils. Each of the chemical additives tested is designed to combine with the soil to improve the texture, increase strength and reduce swell characteristics. These products were combined with a total eight different soils with classifications of CH, CL, ML, SM, and SP. Durability testing procedures included freeze-thaw, wet-dry, and leach testing. Atterberg limits and strength tests were also conducted before and after selected durability tests. Changes in pH were monitored during leaching. Relative values of soil stiffness were also tracked over a 28-day curing period using the soil stiffness gauge.</p> <p>Lime and cement stabilized soils showed the most improvement in soil performance for multiple soils, with fly ash treated soils showing substantial improvement. The results showed that for many soils more than one stabilization option may be effective for the construction of durable subgrades. The enzymatic stabilizer did not perform as well as the other stabilization alternatives.</p> <p>It is recommended, based on the results of this research, that some testing of the contribution of proposed stabilization agents be conducted prior to construction. For pavement designs that expect a relatively limited strength contribution from the soil, the primary anticipated benefit of stabilization is generally the control of volume change.</p>			
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Final Report

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THE KANSAS DEPARTMENT OF TRANSPORTATION  
TOPEKA, KANSAS

UNIVERSITY OF KANSAS  
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## **PREFACE**

The Kansas Department of Transportation's (KDOT) Kansas Transportation Research and New-Developments (K-TRAN) Research Program funded this research project. It is an ongoing, cooperative and comprehensive research program addressing transportation needs of the state of Kansas utilizing academic and research resources from KDOT, Kansas State University and the University of Kansas. Transportation professionals in KDOT and the universities jointly develop the projects included in the research program.

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## **ABSTRACT**

Poor subgrade soil conditions can result in inadequate pavement support and reduce pavement life. Soils may be improved through the addition of chemical or cementitious additives. Such chemical additives range from waste products to manufactured materials and include lime, Class C fly ash, Portland cement and proprietary chemical stabilizers. These additives can be used with a variety of soils to help improve their native engineering properties. The effectiveness of these additives depends on the soil treated and the amount of additive used. Specifications based on performance improvements would be ideal to help in the additive selection process and in determining the optimum additive.

This report contains a summary of the performance of lime, cement, Class C fly ash, and Permazyme 11-X used with a wide range of soils. Each of the chemical additives tested is designed to combine with the soil to improve the texture, increase strength and reduce swell characteristics. These products were combined with a total eight different soils with classifications of CH, CL, ML, SM, and SP. Durability testing procedures included freeze-thaw, wet-dry, and leach testing. Atterberg limits and strength tests were also conducted before and after selected durability tests. Changes in pH were monitored during leaching. Relative values of soil stiffness were also tracked over a 28-day curing period using the soil stiffness gauge.

Lime and cement stabilized soils showed the most improvement in soil performance for multiple soils, with fly ash treated soils showing substantial improvement. The results showed that for many soils more than one stabilization option may be effective for the construction of durable subgrades. The enzymatic stabilizer did not perform as well as the other stabilization alternatives.

It is recommended, based on the results of this research, that some testing of the contribution of proposed stabilization agents be conducted prior to construction. For pavement designs that expect a relatively limited strength contribution from the soil, the primary anticipated benefit of stabilization is generally the control of volume change. For these conditions it is recommended that KDOT consider basing selection of the additive on the ability of the additive to control shrink/swell behavior. If the design procedure is amended to account for a substantial strength contribution from the stabilized soil, then it is recommended that strength and durability testing be included as a part of developing the additive specifications.

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# **Chapter 1**

## **Introduction**

Poor subgrade soil conditions can result in inadequate pavement support and reduce pavement life. Soils may be improved through the addition of chemical or cementitious additives. These chemical additives range from waste products to manufactured materials and include lime, Class C fly ash, Portland cement and proprietary chemical stabilizers. These additives can be used with a variety of soils to help improve their native engineering properties. The effectiveness of these additives depends on the soil treated and the amount of additive used. Specifications based on performance improvements would be ideal to help in the additive selection process and in determining the optimum additive. This report contains a summary of the performance of these four different additives used with a wide range of soils.

Each of the chemical additives tested is designed to combine with the soil to improve the texture, increase strength and reduce swell characteristics. When the additives containing free calcium hydroxide are mixed with the soil, the calcium causes the clay particles to flocculate into a sand-like structure, reducing the plasticity of the soil (1). This reduction in plasticity, which is called modification, reduces the shrink/swell characteristics of the soil. Soil stabilization includes the effects from modification with a significant additional strength gain. The soil must be able to react with the chemical additives to achieve the soil stabilization or modification that is desired.

Lime, which is produced by heating limestone at elevated temperatures, is a product that is used often with highly plastic clays for subgrade improvement. In stabilizing the clay, lime performs two basic functions: flocculation and cementation. Flocculation reduces the plasticity

index of a soil, thereby improving the workability and reducing the swell potential of the soil.

The cementation process is a slow reaction after compaction, which increases the strength and durability of the soil. Cementation also creates a working platform during construction and increases the durability of the soil subgrade (1).

Class C fly ash is produced by the combustion of coal from electric generating plants. The inorganic matter that is present in coal fuses during the combustion process, solidifies from the exhaust gases and is collected by electrostatic precipitators. Class C fly ash is rich in pozzolans and calcium that eliminate the need for additional activators. With the addition of water, fly ash hydrates to form cementitious products similar to that of Portland cement. Fly ash helps reduce the plasticity index and swell and the  $\text{Ca}(\text{OH})_2$  - pozzolan combination gives the soil additional strength gains. Since fly ash begins to hydrate immediately with the addition of water, the soil strength and density are dependent on compaction time. Delays in compaction will decrease the strength and density of the soil dramatically (2).

Portland cement is reported to improve the soil in three distinct processes: cation exchange, cement hydration, and a pozzolanic reaction. The cation exchange uses the available calcium ions in the Portland cement to reduce the plasticity index (PI) of the soil. Cement hydration involves a reaction between the cement particles that stabilizes the clay particles. Hydration will continue as long as there is free moisture present and there is room for expansion products to form at the correct temperature. The pozzolanic reaction results from the calcium hydroxide and the clay particles that form cementitious compounds. Portland cement is similar to fly ash in that compaction time is critical to strength gains that occur (3).

The enzymatic stabilizer, Permazyme 11X, is an organic chemical compound that is reported to increase the wetting action of water to aid in compaction, resulting in an increase in

density. The manufacturer recommends its use in soils containing approximately 20% cohesive fines to help with internal strength, which increases the load bearing capacity of the soil (4).

The different additive types, along with the variety of soil types and conditions, can make choosing the optimum additive and correct percentage to use a difficult decision. To ease the selection process, a soil/additive performance-based specification would be of significant help in comparing the relative performance expected from each soil/additive combination based on a variety of soil testing procedures. This report contains a discussion of the relative performance of each soil/additive combination.

Each additive/soil combination was evaluated to determine its relative performance using strength, swell, stiffness, durability and Atterberg limits. To determine the strength of the soil/additive combinations, the samples were compacted and cured for a 28-day period in a moisture room and then tested to determine the unconfined compressive strength. The soil stiffness of the strength sample was tracked over the 28-day curing period using a soil stiffness gauge. The durability testing was used to evaluate the long-term performance of the soil/additive sample and included leaching, wet-dry, freeze-thaw and free swell testing. The leaching test consisted of compacting a soil sample, placing the sample in a leaching apparatus and leaching distilled water at a constant head through the sample. The leachate that flowed out of the sample was collected and monitored for flow and pH values.

The information presented in this report is organized into seven different chapters. Chapter One covers the introduction to the project. A literature review containing descriptions of the four chemical additives and the processes of improving a soil's engineering properties is covered in Chapter Two. Chapter Three describes the testing procedures used during the study. The results of the testing procedures are presented in Chapter Four and a discussion of the results



is located in Chapter Five. Conclusions and recommendations for the report are presented in Chapters Six and Seven.

## **Chapter 2**

### **Literature Review**

This chapter contains a discussion of the mechanisms of modification and stabilization and the improvement in soil properties that may be achieved. Also covered in the chapter is a discussion of selected previous works on soil stabilization with the four chemicals used in the study.

#### **2.1 Soil Structure**

The clay particles in the soil structure are arranged in sheet-like structures composed of silica tetrahedra and alumina octahedra. The sheets form many different combinations but there are three main types of formations. The first is kaolinite, which consists of alternating silica and alumina sheets bonded together. This form of clay structure is very stable and does not swell appreciably when wetted. The next form is montmorillonite, which is composed of two layers of silica and one alumina sheet creating a weak bond between the layers. This weak bonding between the layers allows water and other cations to enter between the layers, resulting in swelling in the clay particle. The last type is illite, which is very similar to montmorillonite, but has potassium ions between each layer, which help bond the layers together. Interlayer bonding in illite is therefore stronger than for montmorillonite but weaker than kaolinite (5).

Clay particles are small in size and have a large surface to mass ratio, resulting in a larger surface area available for interaction with water and cations (5). The clay particles have negatively charged surfaces that attract cations and polar molecules, including water, forming a bound water layer around the negatively charged clay particles (1). The amount of water surrounding the clay particles is related to the amount of water that is available for the clay

particle to take in and release. This moisture change around the clay particles causes expansion and swelling pressures within clays that are confined.

## **2.2 Stabilization and Modification**

The process of reducing plasticity and improving the texture of a soil is called soil modification. Monovalent cations such as sodium and potassium are commonly found in expansive clay soil and these cations can be exchanged with cations of higher valences, such as calcium, which are found in lime, fly ash and Portland cement. This ion exchange process takes place quite rapidly, often within a few hours. The calcium cations replace the sodium cations around the clay particle,s decreasing the size of the bound water layer and enabling the clay particle to flocculate. The flocculation creates a reduction in plasticity, an increase in shear strength of the clay soil and an improvement in texture from a cohesive material to a more granular, sand-like soil (3). The change in the structure causes a decrease in the moisture sensitivity and increases the workability and constructability of the soil (1).

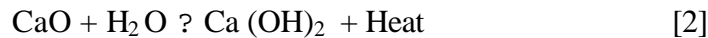
Soil stabilization includes all the effects of modification with an additional long-term strength gain. Soil conditions and mineralogical properties have a greater role for soil stabilization than modification. The magnitude of soil stabilization is usually measured by the increase in strength as determined from unconfined compression testing.

## **2.3 Lime**

The two primary types of lime used in construction today are quicklime (calcium oxide) and hydrated lime (calcium hydroxide). Heating limestone at elevated temperatures produces quicklime and the addition of water to quicklime produces hydrated lime. Equation [1] shows the reaction that occurs when limestone is heated to produce quicklime with carbon dioxide produced as a by-product.



Equation [2] shows that the addition of water to quicklime produces hydrated lime along with heat as a by-product.



Hydrated lime has a higher atomic weight of 74.1, compared to 56.1 for quicklime, because it has one additional oxygen and two additional hydrogen atoms. Since hydrated lime has a higher atomic weight than quicklime, 30% more hydrated lime is required to introduce the same amount of calcium.

For soil stabilization with lime, soil conditions and mineralogical properties have a significant effect on the long-term strength gain. Introduction of calcium hydroxide increases the pH, causing the silica and alumina in the clay particles to become soluble and interact with the calcium in a pozzolanic reaction. A pozzolanic reaction between silica or alumina in the clay particles and calcium from the lime can form a cemented structure that increases the strength of the stabilized soil. Residual calcium must remain in the system to combine with the available silica or alumina and to keep the pH high enough to maintain the pozzolanic reaction (2). A number of references contain more detailed discussions on lime modified and stabilized soils and their durability under different conditions, including (6, 7, 8, 9, 10 and 11).

The treatment of pavement subgrades with lime can significantly improve the engineering properties of a wide range of soils. There are many recommendations for evaluations of soils for lime treatment. For examples, soils that should be considered for lime treatment include soils with a PI that exceeds 10 and have more than 25 percent passing the #200 sieve (1).

## 2.4 Class C Fly Ash

Class C fly ash is an industrial by product generated at coal-fired electricity generating power plants that contains silica, alumina, and calcium-based minerals. Upon exposure to water, these calcium compounds hydrate and produce cementitious products similar to the products formed during the hydration of Portland cement. Free lime that is present or generated as a part of these reactions may also react with available pozzolans (12). The rate of hydration for fly ash is much more rapid than for Portland cement. It is, therefore, desirable to mix and compact fly ash as quickly as is practical, and a maximum delay time between mixing and compaction is often included in compaction specifications (2).

The hydration properties of fly ash are dependent on a number of factors including the coal source, boiler design and the type of ash collection system. The coal source governs the amount and type of inorganic matter present in the coal, thereby dictating the composition of the fly ash. Eastern coal sources have low calcium contents and the fly ash that is produced from these coals contain only a small amount of calcium. This Class F fly ash does not exhibit self-cementing characteristics, however the addition of lime causes a pozzolanic reaction producing the cementitious products. Western coals contain higher amounts of calcium compounds and the ashes produced through the combustion of these coals typically contain 20 to 35 percent calcium oxide. These ashes are generally classified as Class C fly ashes (2). The amount of calcium oxide that is contained in the fly ash is much lower than that of lime, and much of it is combined with silicates and aluminates (12), so the fly ash has less of an effect on the plasticity than lime does.

While coal sources dictate the chemical composition of a particular fly ash, boiler design and operation have a major influence on the hydration characteristics of a specific fly ash,

particularly the rate at which hydration occurs. During combustion the inorganic matter is fused and is transported from the combustion chamber suspended in the exhaust gases. Where rapid cooling of the fused particles occurs, the fly ash particles are generally noncrystalline. A more gradual cooling of the fused particles can result in crystallization of some compounds before the particles solidify. It is not known whether the crystallinity of the ash may be attributed to the coal source, boiler design or other factors, however the properties of a given ash source are reported to be relatively constant (2). Table 2.1 shows the typical chemical makeup of Class C fly ash from a single plant at LaCygne, Kansas. The 1996 data was originally published by Misra (13).

**TABLE 2.1: Class C Fly Ash Chemical Makeup (LaCygne)**

Compound	Feb-96 to June-96	Std. Dev.	3 month period			Mean	Std. Dev.
			Jul-01	Aug-01	Sep-01		
SiO <sub>2</sub>	34.09	2.13	37.06	36.67	37.77	37.17	0.56
Al <sub>2</sub> O <sub>3</sub>	22.67	0.98	22.48	22.63	22.34	22.48	0.15
Fe <sub>2</sub> O <sub>3</sub>	5.86	0.41	5.86	5.93	5.80	5.86	0.07
SO <sub>3</sub>	1.70	0.20	1.09	1.10	0.98	1.06	0.07
CaO	26.70	1.36	24.31	24.38	23.90	24.20	0.26
AA	1.38	0.05	1.21	1.28	1.29	1.26	0.04
MgO			4.53	4.68	4.46	4.56	0.11

Compaction time after mixing is critical to achieving the maximum density and maximum potential strength. When compaction is delayed, hydration products begin to bond particles in a loose state and disruption of these aggregations is required to densify the material. Therefore, a portion of the compactive energy is utilized in overcoming the cementation and the

maximum densities are reduced. Additionally, if the cementitious products formed prior to compaction must be disrupted to achieve effective compaction, the strength that is developed will be lower because the fly ash has already started reacting with the stabilized material (2).

While a number of researchers have looked at the effect of self-cementing fly ash on stabilizing particular soils (2, 14, 15), published work on durability testing has been more limited. Khoury and Zaman (16) looked at the effect of wet-dry cycles on the resilient modulus for fly ash-stabilized aggregate bases and McManus and Arma (17) tested vacuum saturated coarse-grained samples stabilized with fly ash, lime and cement. There has also been a significant amount of research on non-self-cementing fly ash that has been blended with lime or cement.

## **2.5 Portland Cement**

Portland cement is a multiminerale compound made up of oxides of calcium, silica, alumina and iron. When cement is mixed with water, cementing compounds of calcium-silicate-hydrate (C-S-H) and calcium-aluminate-hydrate (C-A-H) are formed and excess calcium hydroxide is released (18). Some calcium is therefore available to react with the clay particle early in the modification process when the water is added, and additional calcium becomes available later as it forms during cement hydration (1). The total amount of calcium hydroxide generated from cement is approximately 31% by weight (19).

The hydrates help to stabilize flocculated clay particles through cementation. The hydration reactions and strength increases occur for the most part between 24 hours and 28 days, although the cement will continue to hydrate at decreasing rates as long as free moisture is present (98). Prusinski and Bhattacharja (1) report that pozzolanic reactions also occur from interaction between the calcium hydroxide and the clay minerals due to the alkaline environment,

which dramatically increases the solubility of silica and alumina in clay minerals (1). Additional information and references on previous work with cement-stabilized soils can be found in (1, 19).

## **2.6 Enzymatic Stabilizer (Permazyme 11-X)**

According to the manufacturer, an enzymatic stabilizer is a natural organic compound, similar to proteins, which acts as a catalyst (4). Their large molecular structures contain active sites that assist molecular bonding and interaction. The organic formulation is designed to maximize compaction and increase the natural properties of soil to optimal conditions. The enzymatic stabilizer increases the wetting action of water to help achieve a higher density during compaction and the formulation accelerates cohesive bonding of soil particles, creating a tight permanent stratum (4).

Few peer-reviewed studies have been published on enzymatic stabilizers. Khan and Sarker reported increases in unconfined compressive strength with the addition of 5% enzymes and good performance in freeze-thaw testing (20). Rauch, et al reported no consistent, measurable improvement in soil properties with the addition of Permazyme in a more diluted form (21).

To achieve effective stabilization, the manufacturer recommends that it be used with soils containing approximately 20% cohesive fines. The soils are to contain a wide range of material sizes to provide shear strength and internal friction, which increases load-bearing capacity. Use of this material was limited to those soils meeting the manufacturer's recommendations (4).



## **Chapter 3**

### **Procedure**

This chapter contains a description of the testing procedures followed as a part of this research. Standard procedures were used where possible. Adjustments to standard procedures are noted and non-standard procedures are described in detail.

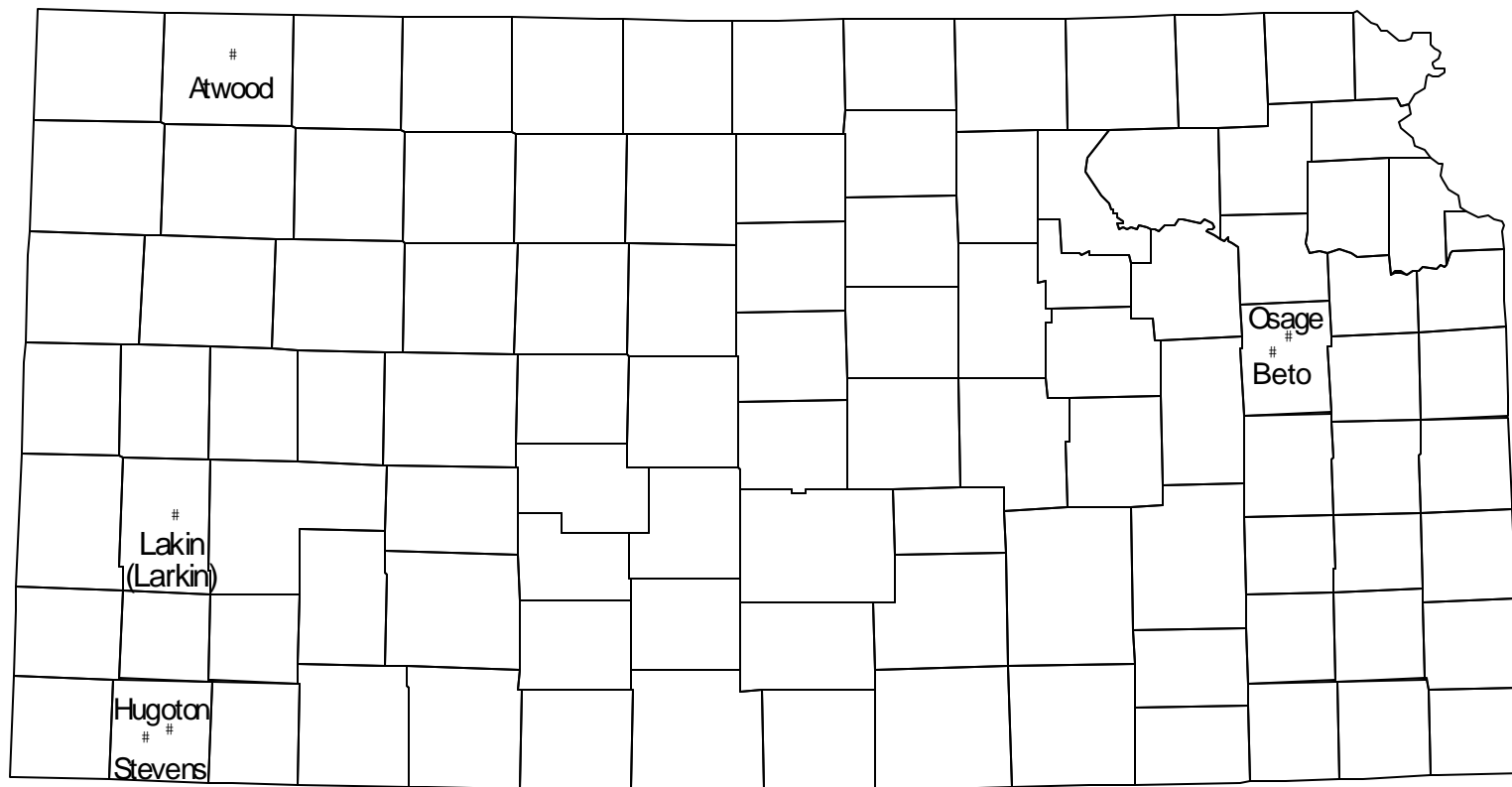
#### **3.1 Materials Used**

##### **3.1.1 Native Soil**

Eight different soils with soil types of CH, CL, ML, SM and SP were selected for use in the admixture evaluation. The native soil properties were determined according to ASTM standards listed in Table 3.1 as described in the following sections. Three CH soils from the Beto Junction area and two CL soils, one from Osage and the other from Hugoton, were tested. The silty soils came from Atwood (ML), Stevens (SM) and Lakin (SP). Lakin has been identified as “Larkin” in some earlier publications. Approximate source locations of the soils are shown in Figure 1.

##### **3.1.2 Additives**

The additives used for the stabilization and modification study included lime, Class C fly ash, Portland cement and an enzymatic stabilizer. The soils were mixed with each of the additives for which there were reasonable expectations of improved engineering properties. Due to the lack of a local knowledge base, the enzymatic stabilizer was mixed with soil types recommended by the manufacturer.



**FIGURE 3.1: Approximate Source Locations of Soils**

The amount of additive used was determined according to ASTM standards, common construction practice from around the region, or according to the manufacturer’s recommendations. The lime percentage used for stabilization was determined in accordance with ASTM D 6276. Fly ash was fixed at 16%, which is the upper end of the percentage used in Kansas. The cement content used was the amount of cement required to lower the plasticity index to 10, with a maximum limit set at 9%. For soils with a native PI below 10, the amount of cement used was determined according to the Portland Cement Association Soil-Cement Handbook (22). The enzymatic stabilizer was mixed at a dilution ratio of one ounce Permazyme to one gallon of water in accordance with the manufacturers recommendations (4).

**TABLE 3.1: Standard Testing Procedures**

Test	ASTM
Grain Size Analysis	D 422
Atterberg Limits	D 4318
Specific Gravity	D 854
pH Lime Stabilization	D 6276
Moisture-Density Relationship	D 629
Swell	KDOT in-house test method
Freeze-thaw	D 560
Wet-dry	D 559
Unconfined Compression	D 1633, D5102

### **3.2 Lab Testing**

#### **3.2.1 Soil-Preparation**

Each soil was air-dried overnight in large pans and was then broken up to pass the 3/8” sieve. Samples of the soil were wet sieved according to ASTM D 2216 over a #40 sieve to remove the larger particles. The #40 sieve was used instead of the #10 sieve because the

Atterberg limits require material smaller than the #40 sieve. The material that passed the #40 sieve was dried at 60°C and pulverized with a mortar and pestle. After the material was broken up, it was then used for hydrometer analysis, Atterberg Limits, and other durability testing.

### **3.2.2 Atterberg Limits**

#### ***3.2.2.1 Lime***

The Atterberg limits were determined on the soil-lime mixture using the KDOT Lime PI procedure. The lime was mixed with the soil and water was added to raise the moisture content of the soil-lime mixture to 5% above the native plastic limit. The soil-lime mixture was then allowed to mellow in a moisture room at 22° C for 48 hours. The sample and container were placed in an unsealed plastic bag with the opened end folded under the container. After the 48-hour moist curing period the sample was dried at 71° C overnight. The liquid limit, plastic limit and plasticity index of the soil-lime mixture were then determined in accordance with ASTM D 4318. Lime percentages are reported as percent quicklime unless otherwise noted.

#### ***3.2.2.2 Fly Ash***

A weight of fly ash equivalent to 12 or 16% of the dry weight of soil was added for Atterberg limits testing. The fly ash was mixed with the soil to a uniform consistency and water was added to raise the moisture content to 3 points below the native optimum moisture content. After complete mixing of the soil, fly ash and distilled water, the sample was covered and allowed to mellow for one hour. Atterberg limits were then determined in accordance with ASTM D 4318.

#### ***3.2.2.3 Cement***

The specification for cement modified Atterberg Limits followed the Portland Cement Association procedure (22). The soil was first wet sieved over a #40 sieve; the material

passing the sieve was dried and later broken up with a mortar and pestle. The specified amount of cement was then added to the dry weight of soil and water was mixed in with the soil-cement to a uniform consistency just above the native plastic limit. The soil-cement mixture was allowed to mellow for 24 hours and the mixture was again wet sieved over a #40 sieve and then air-dried. The soil was broken up with a mortar and pestle and then mixed with water. The Atterberg limits were then determined in accordance with ASTM D 4318.

### **3.2.3 Moisture-Density Relationships (Proctor)**

#### ***3.2.3.1 Lime***

The percent lime added was determined from ASTM D 6276. The soil and lime were mixed together dry and then water was added to bring the moisture content up to the target percent. After mixing, the soil-lime mixture was placed in an airtight container to mellow overnight. A standard 4-inch proctor mold was used and the soil was compacted with standard compaction effort in accordance with ASTM D 698.

#### ***3.2.3.2 Fly Ash***

The fly ash percentage was based on the weight of dry soil. After the fly ash was added, the soil-fly ash mixture was mixed to a uniform consistency with a mechanical mixer. Water was added to the soil-fly ash mixture to raise the moisture content to the target percent. After mixing, the soil-fly ash mixture was placed in an airtight container for 1 hour to simulate a standard construction delay. Then it was compacted in a standard 4-inch proctor mold and compacted with standard compaction in accordance with ASTM D 698.

#### ***3.2.3.3 Cement***

Cement was mixed in with the dry soil and water was added to raise the moisture content to the target moisture. The soil, cement and water were mixed to a uniform consistency

and then place in an airtight container for 1 hour prior to compaction to simulate a standard construction delay. The sample was then compacted with standard effort according to ASTM D 698.

#### ***3.2.3.4 Permazyme***

The natural moisture content of the native soil was determined and the soil was placed in the mechanical mixer. The Permazyme-water mixture was added to the soil and mixed to a uniform consistency. The sample was set aside for 1 hour prior to compaction to simulate standard construction delay. The sample was mixed at one percent below native optimum moisture, according to the manufacturers recommendations (4). After the one-hour delay period, the sample was compacted according to ASTM D 698.

### **3.2.4 Swell**

#### ***3.2.4.1 Native***

The swell test followed the KDOT in-house test method, Determination of Volume Change of Soils. For this test, two 1200 gram samples were placed in a 60°C oven overnight and a moisture content was obtained the following day. Water was added to the soil samples to bring the moisture content of one sample to 3 points below optimum and the other sample was mixed at 3 points above optimum moisture of the native soil. A moisture content was taken of the mixed samples to ensure proper mixing. The samples were then placed in an airtight container and allowed to mellow overnight. After the mellowing period, the moisture content was adjusted if necessary and sufficient soil was used to compact a sample with the required 92% of maximum density as determined by ASTM D 698. The samples were then compacted to a 2-inch height and were allowed to rebound overnight with a surcharge stress of 150 psf (7.18 kPa) in place. After compaction, a moisture content sample was taken to

determine the actual moisture content at compaction. After the rebound period, the height for each of the two samples was measured and the molds were then placed in a pan filled with water. The change in height was measured for 96 hours and the swell was determined by dividing the change in height by the original height. The swell of the two samples was plotted vs. moisture contents and the percent swell reported was the swell that corresponded to the optimum moisture content.

#### ***3.2.4.2 Lime***

The lime swell test procedure was similar to the native swell test. The soil was weighed out for both samples, the lime was mixed in and water was added to raise the moisture to 3 points below and 3 points above the lime optimum. The samples then mellowed overnight. The swell was then determined in accordance with the procedure for native soils.

#### ***3.2.4.3 Fly Ash and Cement***

The swell testing for fly ash and cement follows the native procedure with the following exceptions. The fly ash or cement was mixed with the soil samples, and water was added and the sample was mixed until a uniform consistency was achieved. The samples were then set aside to stand for one hour to simulate a standard construction delay. After the one-hour period, the moisture content of the samples was determined with a microwave according to a previously calibrated relationship and the appropriate amount soil was weighed out. The fly ash swell was then compacted and the procedure followed the native swell procedure.

### **3.2.5 Freeze-Thaw**

Freeze-thaw tests were conducted according to ASTM D 560. Two identical samples of each soil/additive combination were prepared at the optimum moisture content following moisture-density sample preparation procedures. Lime was mixed with the native soil and the treated soil mellowed overnight before compaction. Fly ash, cement and Permazyme were mixed and allowed to stand one hour prior to compaction. After compaction, the samples were cured seven days in a moisture room prior to subjecting them to freeze-thaw cycles.

Each freeze-thaw cycle consisted of placing the two soil samples in a freezer at  $-23\text{ }^{\circ}\text{C}$  for 24 hours. The samples were then moved to a moist room for 23 hours. After removal from the moist room, the first sample was measured for volume change and weighed to determine any change in moisture content. The second sample was brushed to determine the soil loss. The test was continued until 12 cycles were complete or until the sample failed.

### **3.2.6 Wet-Dry**

Wet-dry tests were conducted according to ASTM D 559. Two identical samples of each soil/additive combination were prepared at the optimum moisture content following ASTM D 698 sample preparation procedures. Lime treated samples were mixed and mellowed overnight before compaction. Fly ash, cement and Permazyme treated samples were mixed and allowed to stand one hour prior to compaction. After compaction the samples were cured for seven days in a moisture room prior to subjecting them to any wet-dry cycles. Each wet-dry cycle consisted of submerging the two soil samples in water for 5 hours and then placing them in a  $71\text{ }^{\circ}\text{C}$  oven for 42 hours. After removal from the oven, the first sample was measured for volume change and weighed to determine any change in moisture content. The second sample was brushed and



weighed to determine the soil loss. The test was continued until 12 wet-dry cycles were completed or until the sample failed.

### **3.2.7 Unconfined Compression and Soil Stiffness Testing**

The soil samples that were compacted for the moisture-density relationships (ASTM D 698) were cured for 28 days and then tested to determine their unconfined compressive strength following ASTM D 1633 and D 5102. The soil stiffness of each soil sample was monitored during the curing period using a soil stiffness gauge and an impact echo device.

The soil stiffness gauge (Figure 3.2) is a non-nuclear hand carried device manufactured by Humboldt that repeatedly generates a small dynamic vertical force on the compacted surface. The SSG measures the deflection of a known mass resulting from the application of a known vibrating force. The stiffness of the soil for a series of loading frequencies is calculated based on these deflections (23).

Measurements were obtained at 10 minutes, 4 hours, 1 day, 7 days, 14 days and 28 days after compaction. The values reported represent an average of five stiffness gauge readings. The soil stiffness gauge used a modified foot that was designed for the 4-inch proctor samples that were tested. The readings taken at 10 minutes and 4 hours were determined with the soil sample in the proctor mold. Subsequent readings were determined with the sample extruded from the mold.

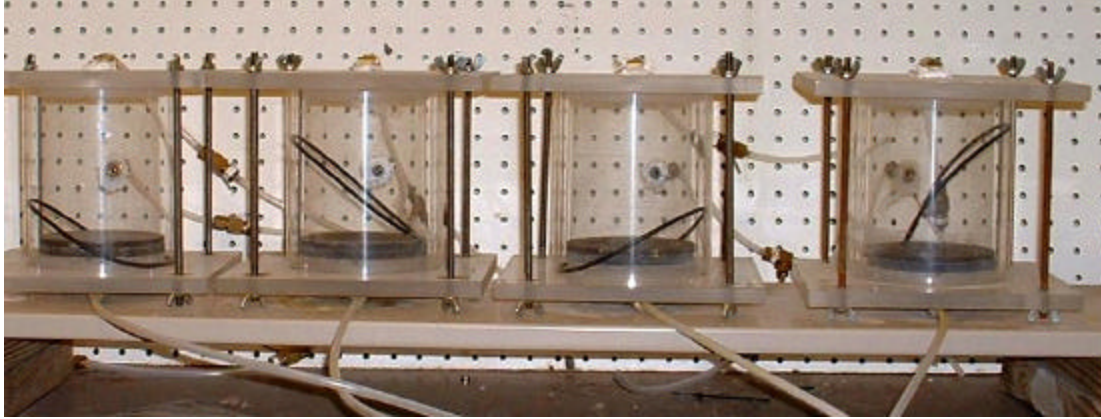


**FIGURE 3.2: Soil Stiffness Gauge**

### **3.2.8 Leaching**

The leaching test involved leaching distilled water through a soil sample under a constant head of 5.4 feet for 28 days. The leachate that flowed through the compacted soil sample was collected and used to determine pH and flow-rates over the 28-day leaching period. The leaching tank was modeled after a design from McCallister and Petry (24). The leaching apparatus consisted of a clear tank similar to a triaxial cell with flexible membrane confinement for the samples. The tank design was modified to use a four-inch proctor sample as shown in Figure 3.2. The pH and flow were monitored over regular intervals. The soil samples used were

compacted at optimum moisture content and cured for seven days in a moist room prior to leaching.



**FIGURE 3.3: Leaching Cells**

## Chapter 4

### Results

The following chapter covers the results of the testing program. The results that are presented include native soil properties, admixture percentages and the various testing results for the soil/additive combinations.

#### 4.1 Native Soil Properties and Admixture Percentages

Native soil characteristics of the soils were determined using grain size analysis, Atterberg limits, specific gravity, swell, standard proctor and unconfined compression. A summary of the test results is presented in Table 4.1. The eight soils were classified and the results showed a combination of three CH soils, (Beto “Red”, “Tan” and “Brown”) two CL soils, (Osage and Hugoton) and three silty to sandy soils that classified ML, SM and SP (Atwood, Stevens and Lakin).

The admixture percentages that were used to evaluate relative soil performance for the testing procedures are presented in Table 4.1. The amount of quicklime for soil stabilization was determined according to the Eades and Grim procedure, ASTM D 6276. The fly ash percentage was fixed at 16 percent, which is a standard percentage in the region. The percentage of cement added for clay soils was determined by the amount of cement needed to lower the PI below 10. For soils that did not have a PI of 10, a cap of 9 percent was used for economic reasons and for soils with a native PI below 10 the Portland Cement Association Handbook (8) was used to determine the cement percentages. Permazyme samples were mixed following the manufacturer’s recommendations (4).

**TABLE 4.1: Native Soil Properties and Admixture Percentages**

Soil Properties	Beto Junction							
	Red	Tan	Brown	Osage	Atwood	Hugoton	Stevens	Lakin
% Sand	5	12	5	8	12	34	70	96
% Fines	95	88	95	92	88	66	30	4
Liquid Limit	70	53	65	36	30	35	20	NP
Plasticity Index	45	31	36	16	7	16	3	NP
USCS	CH	CH	CH	CL	ML	CL	SM	SP
AASHTO	A-7-6	A-7-6	A-7-6	A-6	A-4	A-6	A-2-4	A-3
Max Unit Weight, lb/ft <sup>3</sup>	94	105.4	96.6	108	98	104	120	107
Max Density, kg/m <sup>3</sup>	1506	1689	1548	1731	1571	1667	1923	1715
Optimum Moisture, %	25.7	20.3	25.3	18.5	13.7	19.9	9.9	2
UC at Optimum, psf	6400	4600	4600	4800	6600	4415	5638	0
Max UC, psf	8600	7500	6400	7500	6600	6200	5638	0
Moisture at Max UC	18.9	18.6	23.5	17	13.7	17.6	9.9	0
Specific Gravity	2.78	2.77	2.73	2.74	2.75	2.69	2.68	2.66
Quicklime, %	5.5	3.5	6	4	1.5	2.5	1	-
Fly Ash, %	16	16	16	16	16	16	16	16
Cement, %	9	9	9	5	10	3	7	10
Permazyme	-	-	-	-	Yes	Yes	Yes	-

## 4.2 Atterberg Limits

The Atterberg limits test results with the various soil/additive combinations are presented in Table 4.2. All combinations were tested, with the exception of the silty soils with cement, and Lakin, which was non-plastic (NP) in the native state. Atterberg limit graphs are located in Appendix A.

**TABLE 4.2: Atterberg Limits Values**

	Osage		Hugoton		Stevens		Atwood		Beto "Red"		Beto "Tan"		Beto "Brown"		Lakin	
	LL	PI	LL	PI	LL	PI	LL	PI	LL	PI	LL	PI	LL	PI	LL	PI
Native	36	16	35	16	20	3	30	7	70	45	53	31	65	36	NP	NP
<b>Lime</b>																
1%	35	10	36	7	NP	NP	32	4	54	21	52	24	60	26	-	-
2%	32	5	NP	NP	-	-	NP	NP	48	9	46	11	57	16	-	-
3%	NP	NP	-	-	-	-	-	-	42	3	42	7	54	12	-	-
4%	-	-	-	-	-	-	-	-	NP	NP	NP	NP	NP	NP	-	-
<b>Fly Ash</b>																
12%	37	12	35	12	NP	NP	28	4	62	30	51	24	61	25	-	-
16%	34	9	35	9	-	-	NP	NP	60	27	51	22	58	22	-	-
<b>Cement</b>																
3%	40	13	40	12	-	-	-	-	-	-	-	-	-	-	-	-
5%	41	10	41	12	-	-	-	-	60	19	46	13	65	25	-	-
7%	42	9	42	10	-	-	-	-	62	20	48	13	59	14	-	-
9%	-	-	-	-	-	-	-	-	60	15	49	13	57	13	-	-

The native liquid limit (LL) and the plasticity index (PI) for the CL soils were 36 and 16 for Osage and 35 and 16 for Hugoton. The PI values for these soils were reduced when they were mixed with a small amount of lime and the soils became non-plastic with the addition of more lime. Osage became non-plastic with the addition of 3% quicklime and Hugoton became non-plastic at 2% lime. Fly ash had a more limited effect on the plasticity of these soils. At 12% fly ash, the PI for both soils was reduced to 12, and addition of 16% fly ash reduced the PI to 9. Cement was mixed with these soils at contents of 3, 5 and 7%. The initial amount of cement seemed to have the greatest effect on lowering the PI. At 7% cement, the LL of both CL soils increased to 42 and the PI was reduced to 9 and 10 for Osage and Hugoton.

Native CH soils had LL values that ranged from 70 to 53 and PI values that range from 45 to 31. The addition of a small amount of lime dropped the PI of the CH soils dramatically and the CH soils all became non-plastic at 4% lime. With the addition of 12% fly ash, the PI was reduced 7 to 15 points and the addition of 16% fly ash reduced the PI an additional 2 to 3 percentage points over native. Cement reduced the PI of the CH soils to a range of 13 to 15 with the addition of 9% cement.

The silty soils Stevens and Atwood had native LL values of 20 and 30, respectively and PI values of 3 and 7. With the addition of 1% lime Stevens became non-plastic and at 2% lime Atwood became non-plastic. At 12% fly ash Stevens was non-plastic and with the addition of 16% fly ash Atwood was non-plastic.

#### **4.3 Maximum Density and Optimum Moisture Content**

The optimum moisture content and maximum density for the native soils and each of the soil/additive combinations is presented in Table 4.3. A typical maximum density curve is presented in Figure 4.1. Additional maximum density figures are located in Appendix B. Each soil/additive combination was tested to determine the optimum moisture and maximum density with the exception of the combination of Permazyme and the clayey soils.

The densities of the native CL soils (Osage and Hugoton) ranged from 1729 kg/m<sup>3</sup> to 1665 kg/m<sup>3</sup> and the optimum moisture content ranged from 19 to 20%. When mixed with lime, the optimum moisture for both CL soils increased 4% and the maximum dry density decreased. The introduction of fly ash lowered the optimum moisture slightly for both CL soils. The maximum density of Osage decreased slightly, while the maximum density of Hugoton increased over native. The maximum density for the CL soils decreased with the addition of cement. The

optimum moisture for Osage decreased 1% and Hugoton experienced a 2% increase in optimum moisture when modified with cement.

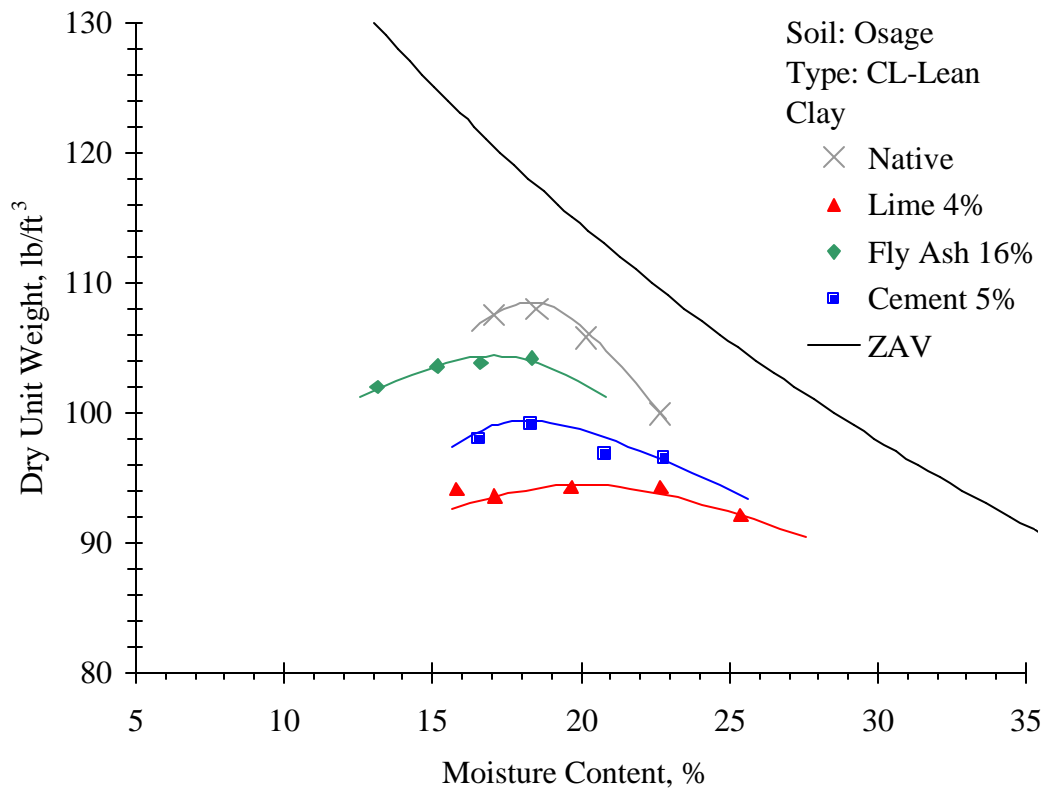
**Table 4.3: Optimum Moisture Contents and Maximum Density**

	Native		Lime		Fly Ash		Cement		Permazyme	
	w%	Density kg/m <sup>3</sup>	w%	Density kg/m <sup>3</sup>	w%	Density kg/m <sup>3</sup>	w%	Density kg/m <sup>3</sup>	w%	Density kg/m <sup>3</sup>
Osage	19	1729	23	1504	18	1681	18	1601	-	-
Hugoton	20	1665	24	1472	18	1697	22	1536	-	-
Beto "Tan"	20	1681	21	1472	22	1633	22	1520	-	-
Beto "Red"	26	1504	22	1376	24	1504	20	1424	-	-
Beto "Brown"	25	1552	20	1408	20	1601	26	1440	-	-
Stevens	10	1921	15	1825	12	1937	13	1873	9	1937
Atwood	14	1569	16	1536	18	1697	23	1472	13	1633
Lakin	2	1714	-	-	8	1925	8	1812	-	-

The CH soils (Beto “Red”, “Tan” and “Brown”) had native optimum moisture contents that ranged from 20 to 26%. The maximum dry density values for the native soils ranged from 1552 kg/m<sup>3</sup> to 1681 kg/m<sup>3</sup>. The lime optimum moisture content for the CH soils decreased for Beto “Red” and Beto “Brown”, but increased 1% for Beto “Tan” and the maximum density decreased for all the CH/lime samples. The optimum moisture for the Beto “Tan” and fly ash combination increased 2% over native while the optimum moisture of the Beto “Red” and “Brown” decreased when mixed with fly. The maximum densities varied about the native optimums for the CH/fly ash samples. Optimum moisture content for the cement modified CH soils also varied about the native optimum moisture content. Beto “Tan” and Beto “Brown” optimum moisture increased 2% and 1%, respectively, over the native optimum. Beto “Red” experienced a 6% decrease in optimum moisture when modified with cement. Each of the cement modified CH soils experienced a decrease in maximum density over native.



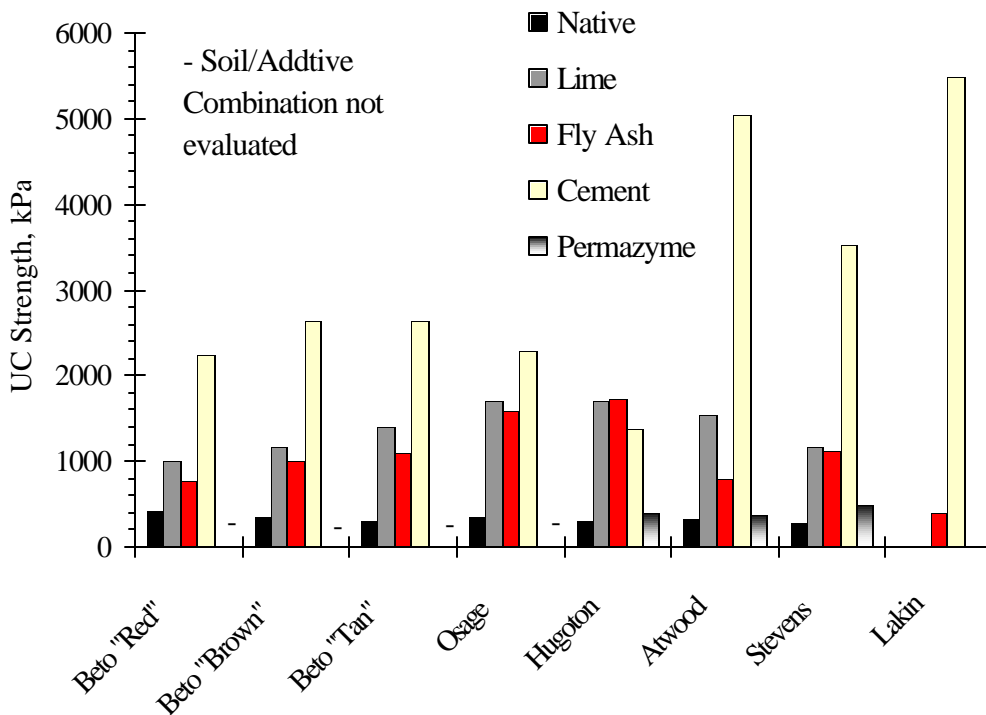
The silty soils (Stevens and Atwood) had a native maximum density of 1921 kg/m<sup>3</sup> and 1569 kg/m<sup>3</sup>, respectively and optimum moistures of 10 and 14%. The optimum moisture content increased and the dry density decreased for both soils with the addition of lime. The silty soils experienced an increase in optimum moisture and dry density when mixed with fly ash. When the silty soils were stabilized with cement, the optimum moisture contents increased and the dry density decreased slightly. The Permazyme samples were compacted at the manufacturer's recommended moisture content of 1% below native optimum and the density values were slightly higher than for the native soil.



**FIGURE 4.1: Max-Density Curves**

#### 4.4 Unconfined Compression Strength

The unconfined compression data in Figure 4.2 was determined on samples that were compacted for the determination of the moisture-density relationship. Each sample was cured for 28 days in a moist room and the strength was determined in accordance with ASTM D 5102. The values that are presented in the figure are the peak strength values of the standard proctor samples.



**FIGURE 4.2: Unconfined Compression Strengths**

Lime treated samples with CL soils (Osage and Hugoton) had a 470% to 580% increase in strength over the native soil strength and the Osage and Hugoton fly ash samples had increases of 440% and 590% over native strength. Cement mixed with Osage and Hugoton had increases of 640% and 560% over native.

The CH soils (Beto “Red”, “Tan” and “Brown”) mixed with lime had strength increases of 240% to 460% over native and fly ash increased the strength 190% to 360% for the CH soils. Cement modified CH soils had a much higher strength increase when compared with the lime and fly ash mixed samples. Cement increased the strength 540% to 872% over the native strength.

The silty soils, Atwood and Stevens, both experienced strength increases of 480% and 430%, respectively with lime. Fly ash treated samples developed a lower strength at 250% and 410% for Atwood and Stevens and cement stabilized samples developed higher strength gains of 1580% to 1310%. The Permazyme samples increased 120% and 180% for Atwood and Stevens.

#### **4.5 Soil Stiffness with Time**

Figures 4.3a and 4.3b show the results of the soil stiffness data versus time. The standard proctor samples were used to determine the soil stiffness with time using a soil stiffness gauge. Average values of five measurements were reported for curing periods of 10 minutes, 4 hours, 1 day, 7 days, 14 days and 28 days. The values that are reported are the stiffness values corresponding to the maximum unconfined compression strength sample used in Figure 4.2.

The stiffness of the CL soils (Osage and Hugoton) increased during the curing period. Lime and cement treated soils experienced the highest increases in stiffness with time. Lime treated Osage and Hugoton samples had similar initial stiffness values and experienced stiffness increases of 11 MN/m throughout the testing period. Hugoton and Osage sample treated with cement had increases in stiffness of 9 MN/m and 12 MN/m respectively. Fly ash samples had stiffness values that remained relatively constant through the testing period.

The CH soils experienced slight stiffness increases with all the additive combinations. Beto “Red” with cement had the greatest increase in stiffness compared to lime and fly ash. Beto

“Brown” had similar increases in stiffness and had greater initial stiffness values. Beto “Tan” had the greatest increase in stiffness of the CH soils. The initial stiffness values for Beto “Tan” were similar to the Beto “Brown” samples and the greatest increase in stiffness occurred in the lime treated sample. Fly ash and cement treated Beto “Tan” samples also showed increases in stiffness over the test period.

The cement-stabilized samples stiffness values for the silty soils (Atwood and Stevens) showed the greatest increase in stiffness. The Permazyme samples for both soils experienced very little change in stiffness during the curing period. Atwood with lime and fly ash also did not experience any increase in stiffness through the test. The highest stiffness gains for Atwood came from cement, which increased 20 MN/m over 28 days. Samples of Stevens had stiffness increases with lime, fly ash and cement, with the greatest gain (16 MN/m) occurring in the cement treated samples.

The impulse echo procedure was also used to estimate the soil stiffness. No stiffness improvements were observed throughout the curing period, however, the method was considered to be of limited value due to lack of repeatability of results. The stiffness values are reported in Figure 4.4.

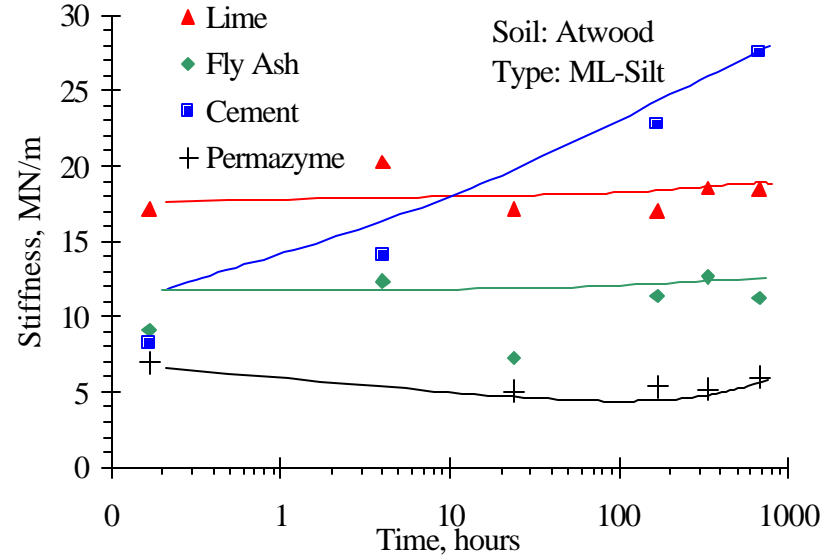
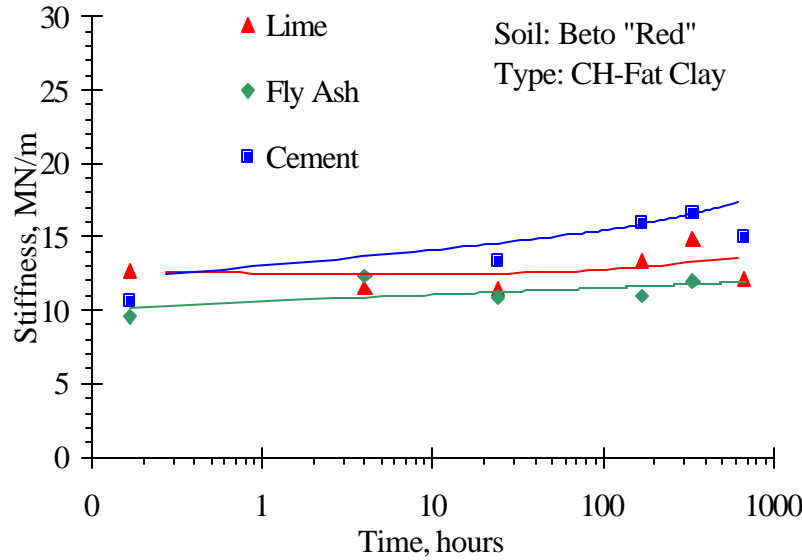
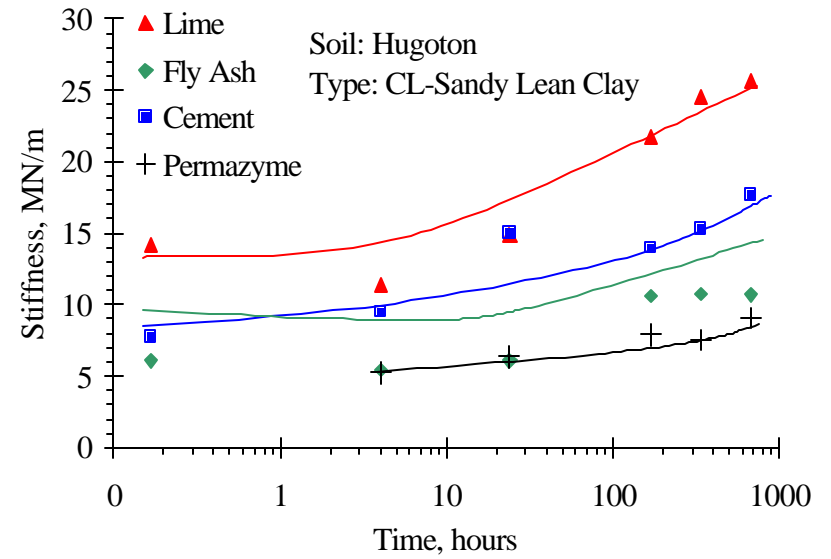
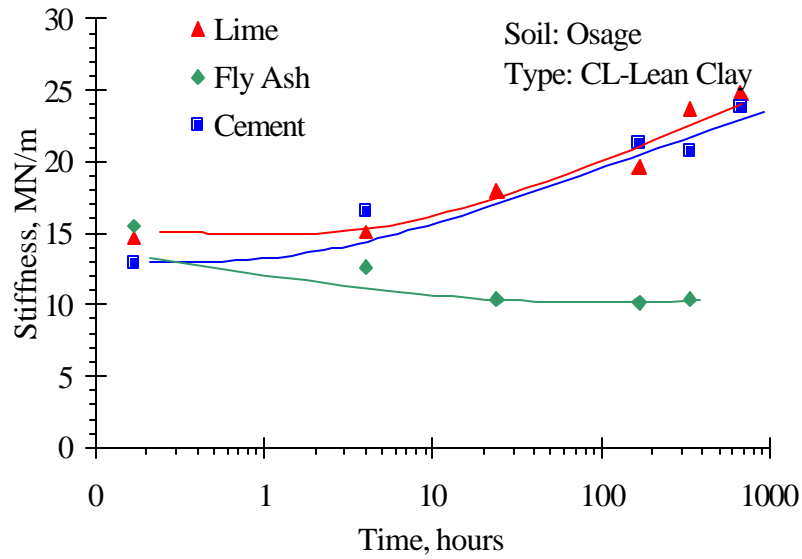


FIGURE 4.3a: Soil Stiffness with Time

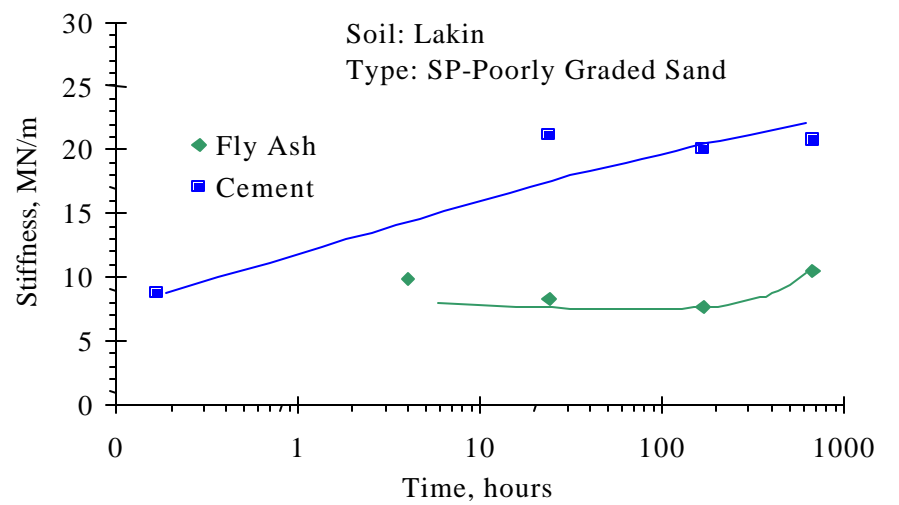
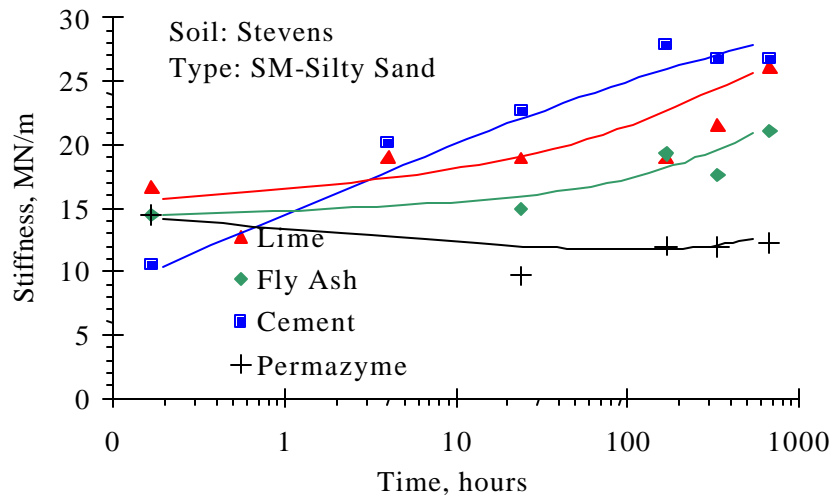
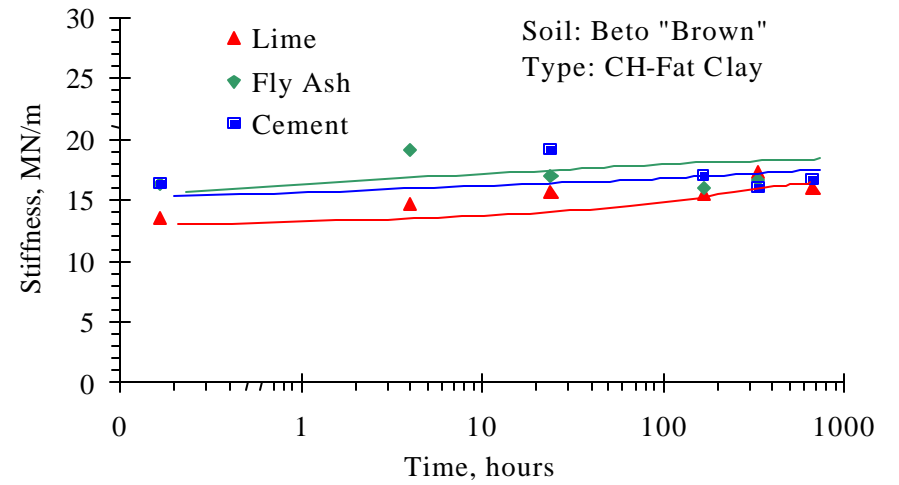
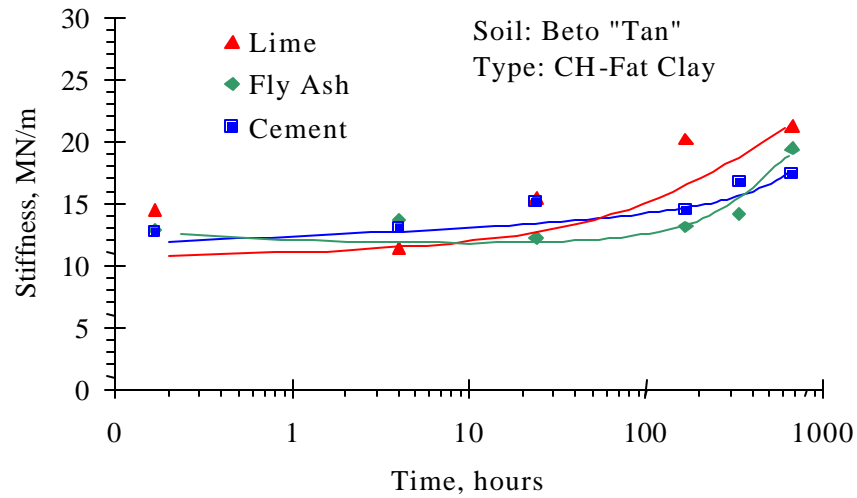
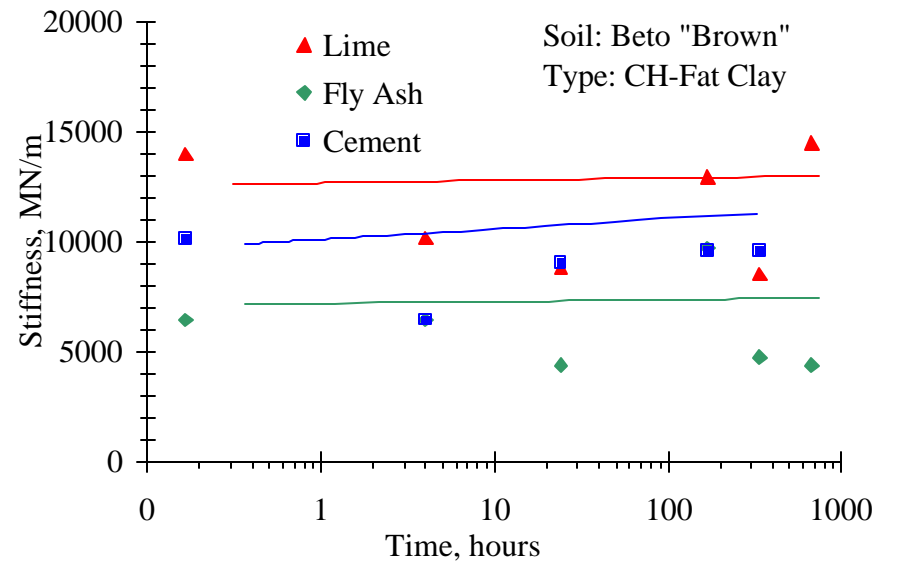
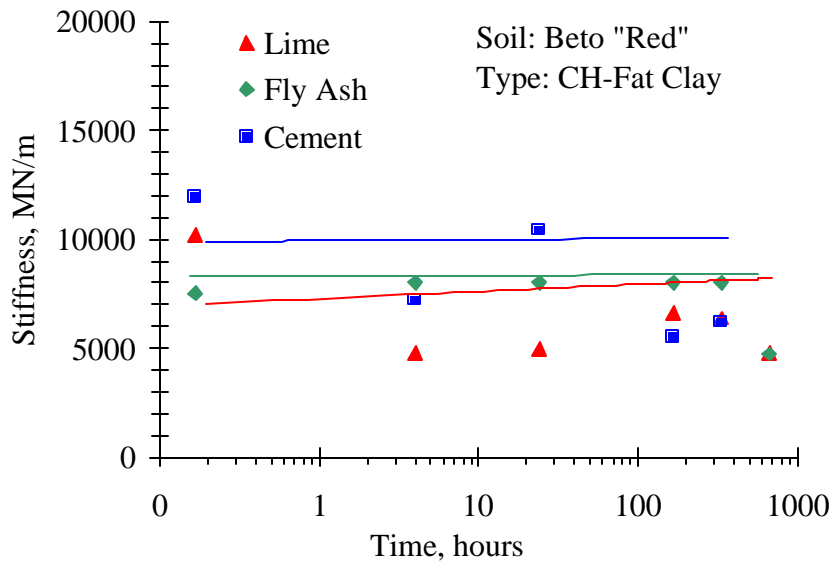
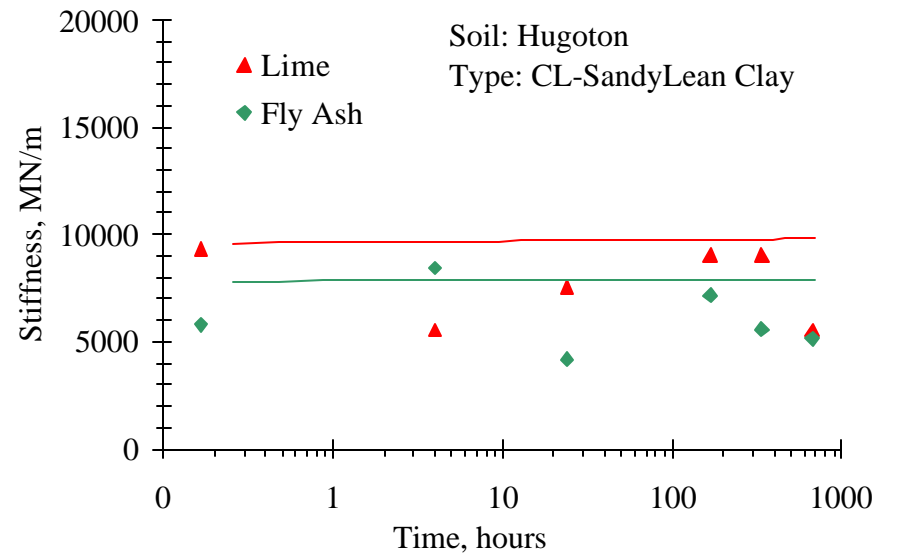
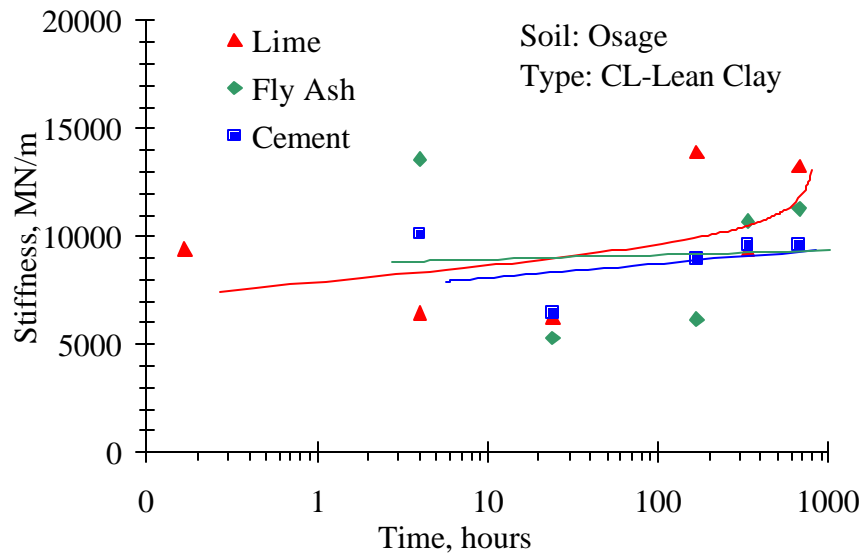


FIGURE 4.3b: Soil Stiffness with Time



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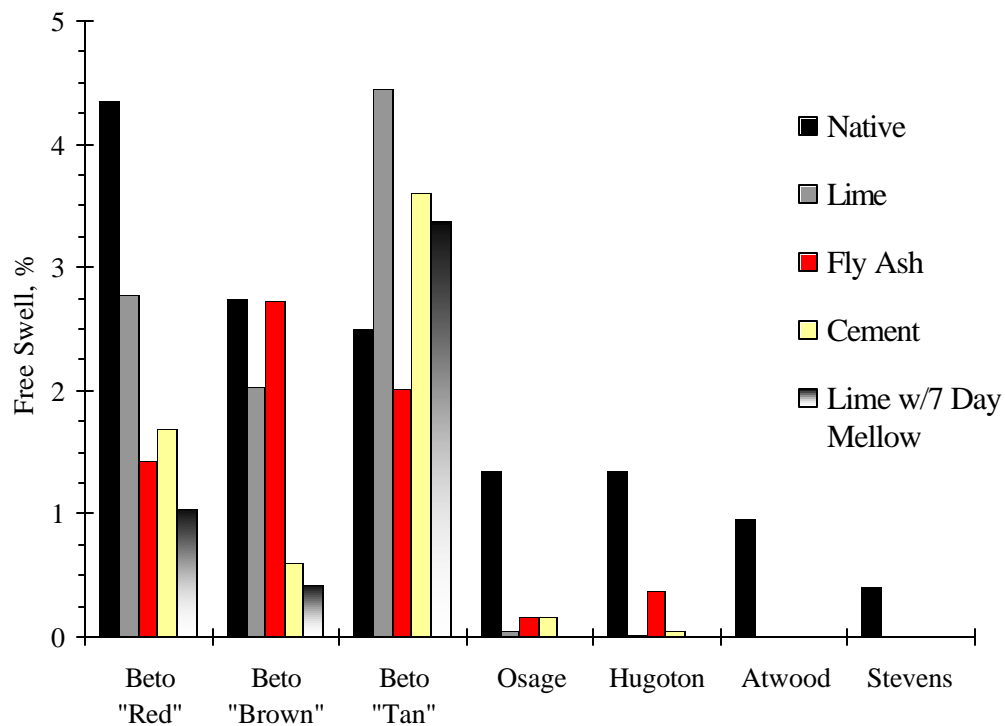
FIGURE 4.4: Impulse Echo Stiffness with Time

## 4.6 Swell

Free swell results are reported in Figure 4.5. The figure shows the amount of swell observed under a surcharge of 7.18 kPa with the various soil/additive combinations. Atwood and Stevens samples were not tested with any additives because those soils did not swell in the native state. Permazyme was also not used in this test because the manufacturer did not recommend using the product with swelling soils.

Osage and Hugoton had native swell numbers of 1.5%. When mixed with lime, fly ash or cement the CL soils had a major reduction in swell. The CH soils experienced native swells that ranged from 2.5% to 4.3%. The CH soil of Beto “Tan” was mixed with lime and the swell *increased*, even though the treated soil was non-plastic. The soil was tested for sulfates, which when mixed with a calcium based product can form expansive minerals known to cause increased swell potential. Beto “Tan” was found to have moderate sulfate content and it was assumed that the remaining Beto soils contained sulfates since the soils were from the same area and continued to swell after the addition of lime. A second swell test was conducted with the sulfate bearing soils mixed with lime and mellowed seven days prior to compaction to allow the sulfates to react with the calcium. The free swell was reduced but additional mellowing or a double treatment of lime may be required to reduce the swell further.

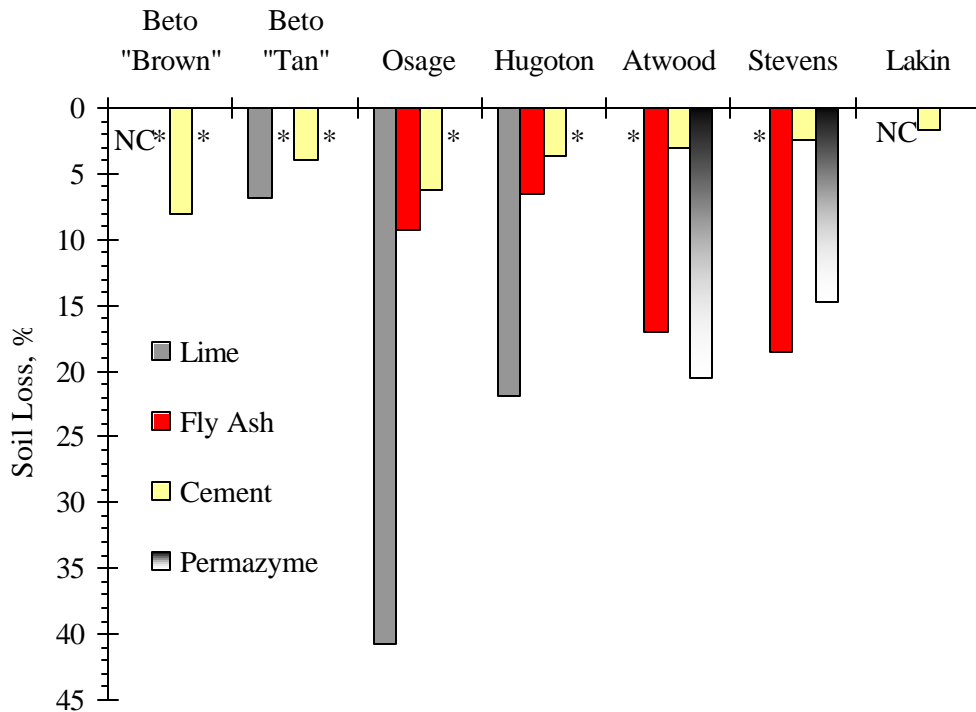




**FIGURE 4.5: Swell**

#### 4.7 Freeze-Thaw

Freeze-thaw soil loss and strength results are presented in Figures 4.6 and 4.7. The term "NC" presented in Figure 4.6 means the sample did not complete the 12 freeze-thaw cycles. The freeze-thaw soil loss showed the CL soils of Osage and Hugoton had the greatest soil loss of 41% and 22% when treated with lime. When Osage and Hugoton were mixed with fly ash, samples had soil losses of 10 and 7% over the 12 freeze-thaw cycles. The cement treated CL samples performed better than the fly ash samples with a soil loss of 6 and 4%.



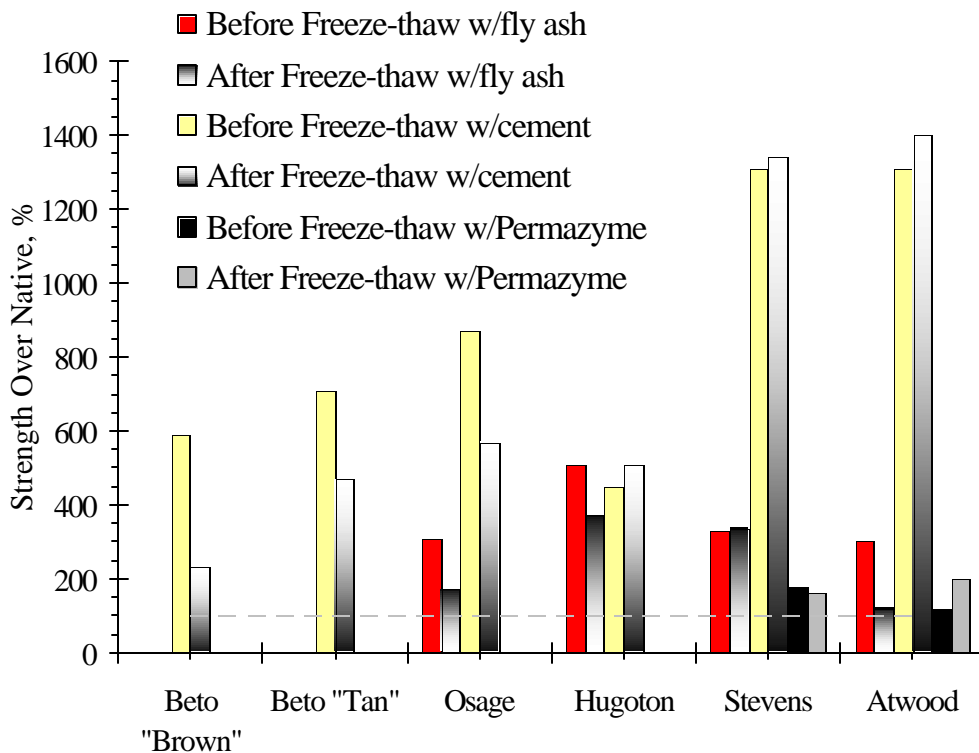
**FIGURE 4.6: Freeze-Thaw Soil Loss**

Beto "Tan" with lime lost 7% of its soil mass over the 12 cycles and the Beto "Brown" sample with lime did not complete the required 12 freeze-thaw cycles. Beto "Brown" and "Tan" with cement lost 8 and 4% of their soil over the 12 cycles.

The silty soils with fly ash and Permazyme had similar soil losses over the completed freeze-thaw cycles. Atwood and Stevens with fly ash lost 17 and 19% of the soil mass over the duration of the testing and Lakin with fly ash did not complete the testing cycle. The Permazyme samples of Atwood and Stevens lost 20 and 15%

of their mass respectively. The cement-stabilized samples performed much better for Atwood, Stevens and Lakin with losses of 3, 2 and 2%, of their soil mass after the length of the cycles, respectively.

The strengths of the non-brushed freeze-thaw samples showed that fly ash samples retained some strength after the freeze-thaw cycles. Osage and Hugoton experienced a 45 and 27% decrease in strength after the freeze-thaw cycles compared to their strengths before freeze-thaw. Both Osage and Hugoton retained a 100% of their native strengths. Stevens and Atwood retained 340% and 120% of their native strengths after completion of the 12 freeze-thaw cycles.



**FIGURE 4.7: Freeze-Thaw Strength**

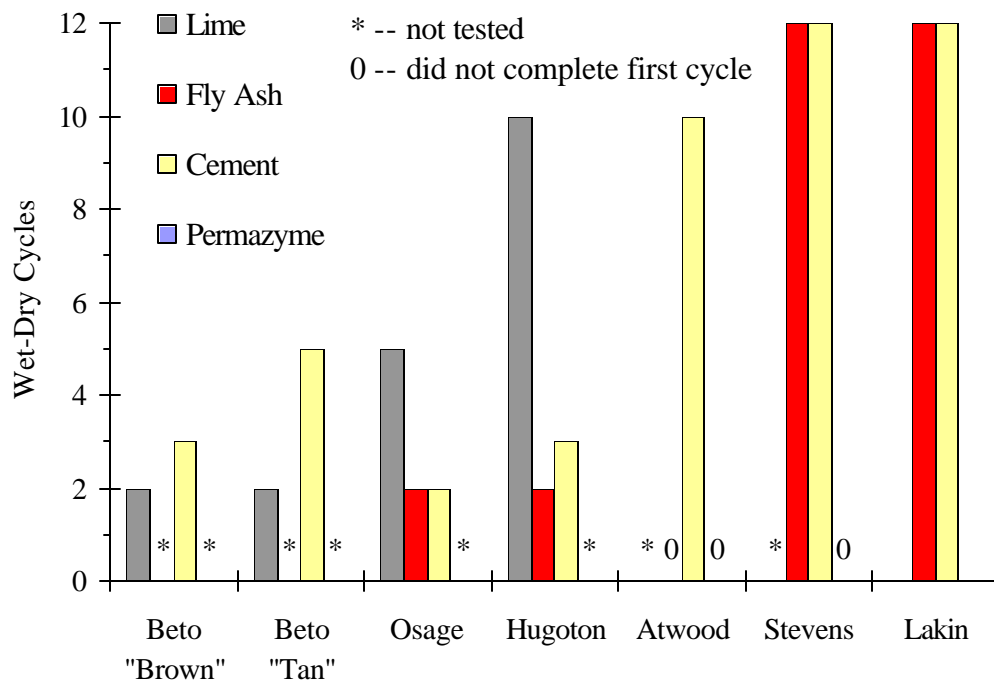
The cement modified clay soils experienced variations in strength that ranged from -61% to +13% after the 12 freeze-thaw cycles. The clay samples retained all the native strength after all the cycles. Beto “Brown” and “Tan” retained 230% and 470% of their native strength and Osage retained 570% of its native strength after the 12 cycles. Hugoton had a slight increase in strength of 13% after the 12 cycles. Stevens and Atwood both had increases in strength after the 12 freeze-thaw cycles of 3% and 7% respectively.

#### **4.8 Wet-Dry**

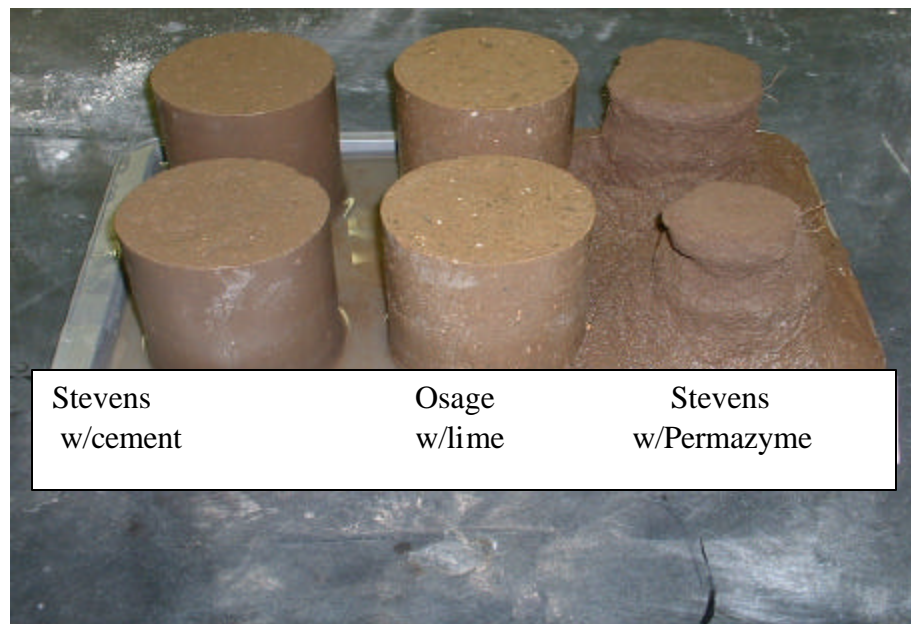
The results of wet-dry testing are shown in Figure 4.8. This is an aggressive test and most samples failed prior to completion of the test. Figure 4.8 shows the number of wet-dry cycles that were completed prior to failure. Soil/additive combinations that were not evaluated are indicated with a star and soil combinations that did not complete any of the wet-dry cycles are indicated with zero.

The CL soils with lime completed the most wet-dry cycles. Cement and fly ash with the CL soils had comparable performance by completing similar wet-dry cycles. For the CH soils, cement treated samples completed the most cycles.

The silty soils performed well with cement. Stevens performed well when treated with fly ash, while Atwood with fly ash did not complete any cycles. Permazyme performed poorly for both soils and did not complete any cycles. Figure 4.9 shows the appearance of the failed Stevens soil sample with Permazyme.



**FIGURE 4.8: Wet-Dry Cycles Completed**

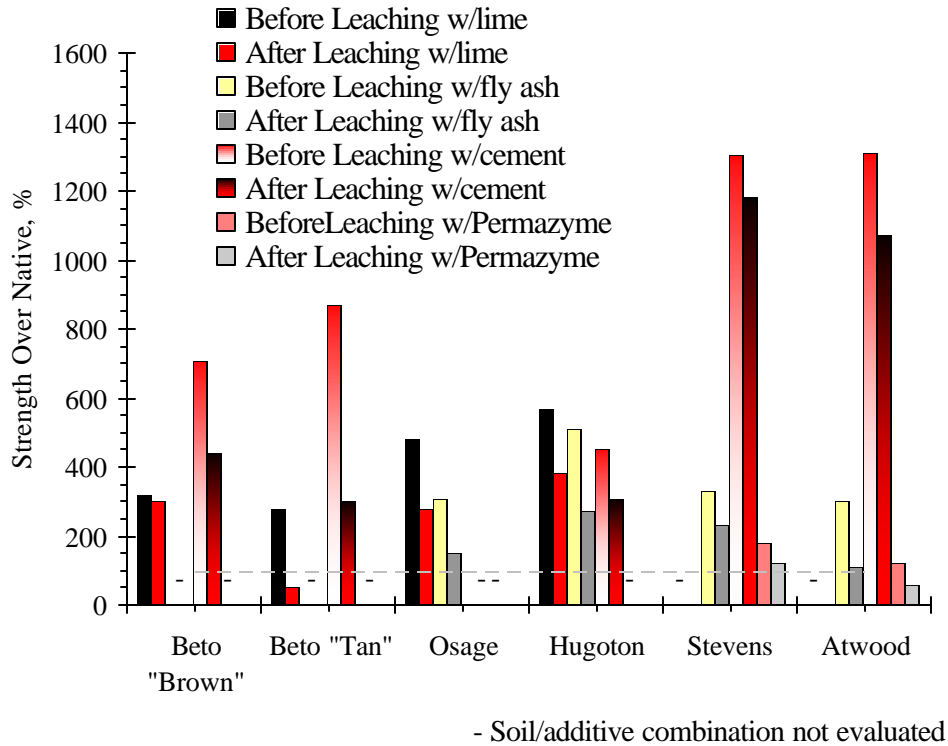


**FIGURE 4.9: Wet-Dry Samples**

#### **4.9 Leaching**

Figure 4.10 shows the comparison between the leached sample strength and the non-leached sample strength. The lime treated samples retained much of the stabilized strength after leaching. The CL soils retained 280% to 380% of the native strength after leaching and Beto “Brown” retained 300% strength where the native values represent the peak strengths of the soil without leaching. Beto “Tan” with lime experienced a large flow of water over the 28-day period because the sample was very lumpy. The large flow of water through the sample would explain the large reduction in strength. Fly ash and cement treated samples also retained most of their stabilized strengths. Fly ash samples retained 110% to 270% of their native strength after leaching. The cement-modified clays retained 300% to 440% of their native strength and the stabilized silty soils retained most of their stabilized strength after leaching. The Permazyme samples after the 28-day leaching procedure returned to the native strength values.

Table 4.4 shows the permeability and Atterberg limit values of the leached samples. The samples tended to become less permeable over the test period. Each of the lime treated samples that had flow through the samples experienced a decrease in permeability over the leaching period. Each of the cement treated samples decreased in permeability over the testing period. However, Stevens treated with fly ash experienced an increase in permeability over the leaching period and the permeability of the second Osage sample remained fairly constant through the leaching period.



**FIGURE 4.10: Strengths Before and After Leaching**

Atterberg limit values for the lime samples remained non-plastic after the leaching procedure. The fly ash samples tended to return towards the native state after the leaching procedure with the exception of Stevens, which remained non-plastic. Cement treated samples showed an improvement (reduction) in plasticity after the leaching period.

The pH of the leachate was determined at the time of sample collection and the pH readings were recorded until the leaching procedure was complete or when the flow from the samples stopped. The pH values are reported in Figure 4.11. The lime treated sample pH readings remained relatively close to 12.45 with the exception of the Beto "Brown" samples at 7 days, which dropped to a pH around 10. Fly ash pH

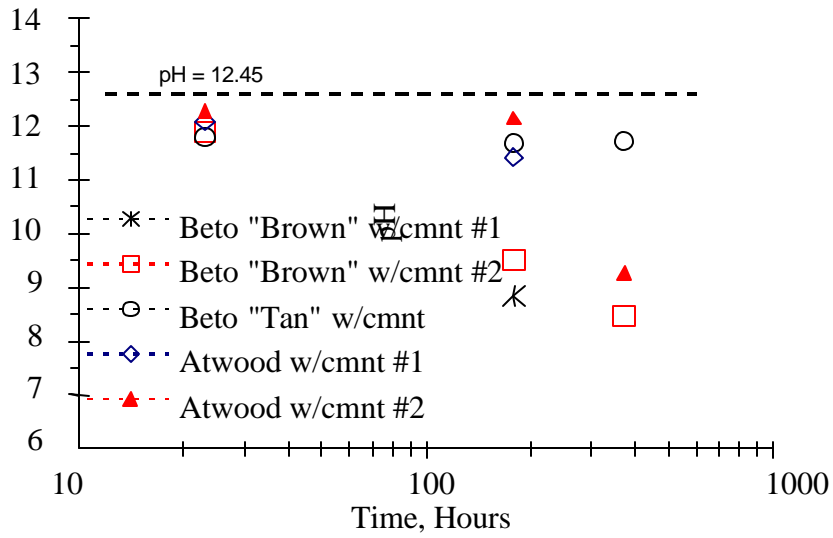
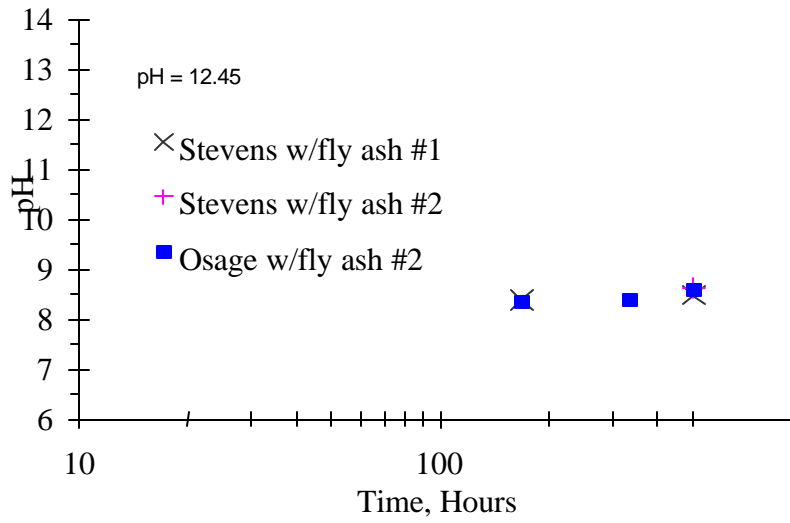
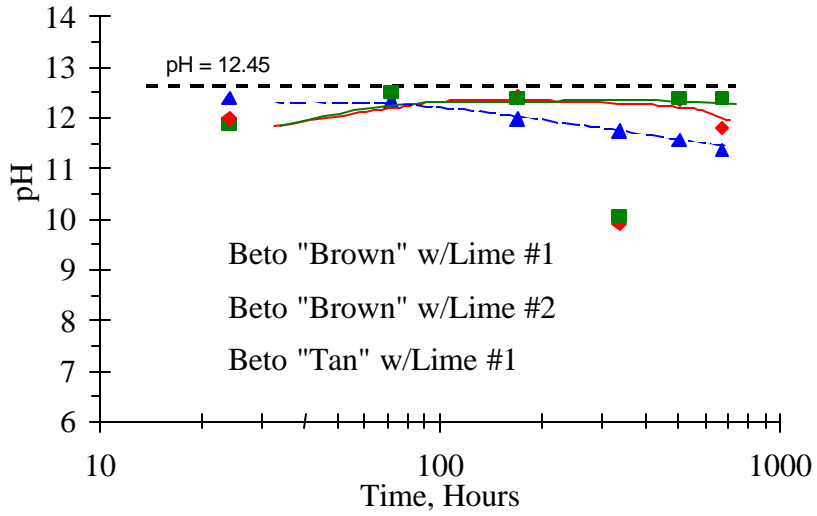
values remained fairly constant ranging in pH between 8 to 9. The pH of the cement treated samples started around 12, but then decreased as the test progressed until the flow from the samples ended.

**Table 4 4: Leaching Permeability and Atterberg Limit Values**

	* Permeability, cm/s				Atterberg Limits					
					Native		Before		After	
	0-7 Days	7-14 Days	14-21 Days	21-28 Days	LL	PI	LL	PI	LL	PI
<b>Lime</b>										
Beto "Brown" #1	4.60E-06	7.84E-07	6.87E-07	9.84E-07	65	36	NP	NP	NP	NP
Beto "Brown" #2	3.45E-06	3.61E-07	2.82E-07	2.07E-07	65	36	NP	NP	NP	NP
Beto "Tan" #1	2.27E-05	2.14E-05	1.36E-05	9.62E-06	53	31	NP	NP	NP	NP
Osage #1	No Flow	No Flow	No Flow	No Flow	36	16	NP	NP	NP	NP
Osage #2	No Flow	No Flow	No Flow	No Flow	36	16	NP	NP	NP	NP
Hugoton #1	No Flow	No Flow	No Flow	No Flow	35	16	NP	NP	NP	NP
Hugoton #2	No Flow	No Flow	No Flow	No Flow	35	16	NP	NP	NP	NP
<b>Fly Ash</b>										
Osage #1	No Flow	No Flow	No Flow	No Flow	36	16	34	9	45	15
Osage #2	1.90E-07	2.26E-07	1.68E-07	1.88E-07	36	16	34	9	43	13
Hugoton #1	No Flow	No Flow	No Flow	No Flow	35	16	35	9	38	10
Hugoton #2	No Flow	No Flow	No Flow	No Flow	35	16	35	9	38	11
Atwood #1	No Flow	No Flow	No Flow	No Flow	30	7	NP	NP	32	8
Atwood #2	No Flow	No Flow	No Flow	No Flow	30	7	NP	NP	31	6
Stevens #1	8.92E-08	1.45E-07	1.96E-07	1.14E-07	20	3	NP	NP	NP	NP
Stevens #2	2.30E-08	4.31E-08	6.18E-08	1.21E-07	20	3	NP	NP	NP	NP
<b>Cement</b>										
Beto "Brown" #1	1.09E-06	5.75E-08	2.30E-08	No Flow	65	36	57	12	NP	NP
Beto "Brown" #2	2.35E-06	1.55E-07	5.46E-08	No Flow	65	36	57	12	NP	NP
Beto "Tan" #2	2.95E-06	2.96E-06	No Flow	No Flow	53	31	48	12	NP	NP
Hugoton #1	No Flow	No Flow	No Flow	No Flow	35	16	38	10	38	6
Hugoton #2	No Flow	No Flow	No Flow	No Flow	35	16	38	10	39	6
Atwood #1	2.74E-06	9.35E-08	No Flow	No Flow	30	7	-	-	-	-
Atwood #2	7.19E-07	No Flow	No Flow	No Flow	30	7	-	-	-	-
Stevens #1	No Flow	6.50E-09	8.93E-09	No Flow	20	3	-	-	-	-
<b>Permazyme</b>										
Atwood #1	7.88E-08	No Flow	No Flow	No Flow	30	7	-	-	-	-
Stevens #1	No Flow	No Flow	No Flow	No Flow	20	3	-	-	-	-
Stevens #2	No Flow	No Flow	No Flow	No Flow	20	3	-	-	-	-

\* Constant Head level of 166 cm  
 - Soil/Additive combination not evaluated





**FIGURE 4.11: Leachate pH Readings**

## **Chapter 5**

### **Discussion**

This chapter contains a discussion of the results of the various soil/additive combinations presented in Chapter Four.

#### **5.1 Atterberg Limits and Max Density**

Atterberg limits results showed that the PI of all the soils tested was improved with the addition of a small amount of lime. Additional amounts of lime modified all soils into a non-plastic state. Fly ash and cement also improved the PI of each of the soils tested but the improvements were not as great as those of lime, with a lower limit for the PI of approximately nine when mixed with fly ash or cement.

The maximum density for the lime treated samples decreased and the optimum moisture content tended to increase over the native optimum moisture content which is consistent with the literature (1). The behavior of the fly ash treated soil was more variable. The optimum moisture for the fly ash treated clay samples experienced a variation about the native optimum and the dry density of the clay soils also varied about the native maximum density. The silty soils experienced an increase in optimum moisture and dry density when mixed with fly ash. The cement treated clay soils experienced a decrease in dry density and tended to increase in optimum moisture over native. Other research has shown that the optimum moisture tends to increase and the density decreases for cement treated clay samples (25). The silty soils, Atwood and Stevens, experienced a decrease in dry density and the optimum moisture increased slightly over native when treated with cement. Lakin experienced an increase in optimum moisture and

dry density with the addition of cement. Permazyme increased the dry density of all the soils tested.

## **5.2 Unconfined Compression Strength**

All the treated samples experienced an increase in strength over the 28-day curing period. The lime and fly ash samples had similar strength gains over the range of soils and Permazyme had modest strength gains with the soils used. Cement treated samples experienced the highest increase in strength. The cement treated CH soils had a higher increase in strength over lime and fly ash compared to the CL soils, which may have been a result of the higher percentage of cement mixed with the CH soils. The silty soil treated with cement had the greatest strength gain compared to lime and fly ash. This is probably due to the use of sufficient cement to achieve soil stabilization based on PCA guidelines (22).

## **5.3 Soil Stiffness Gauge and Impact Echo**

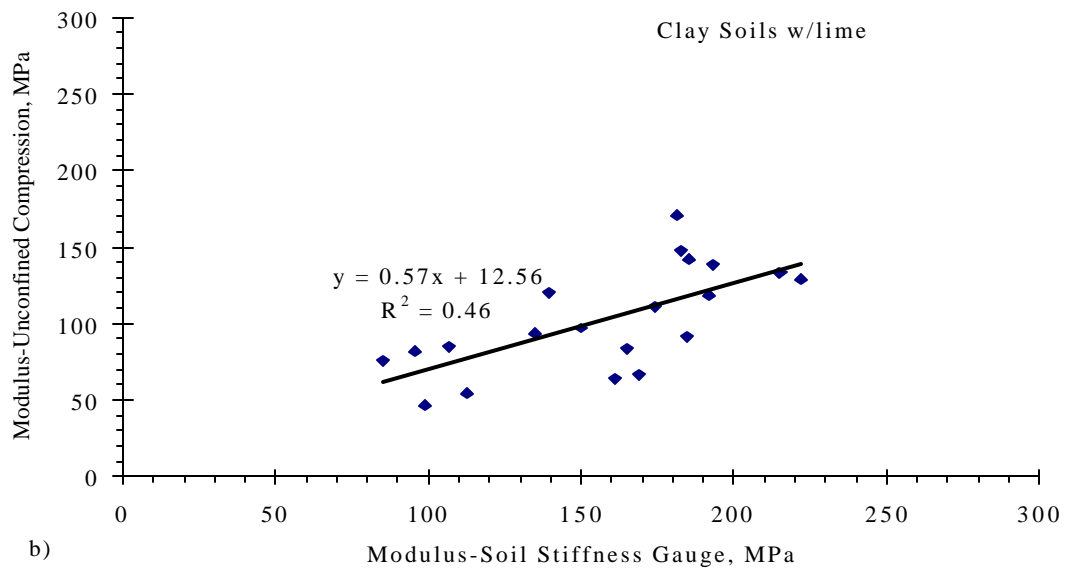
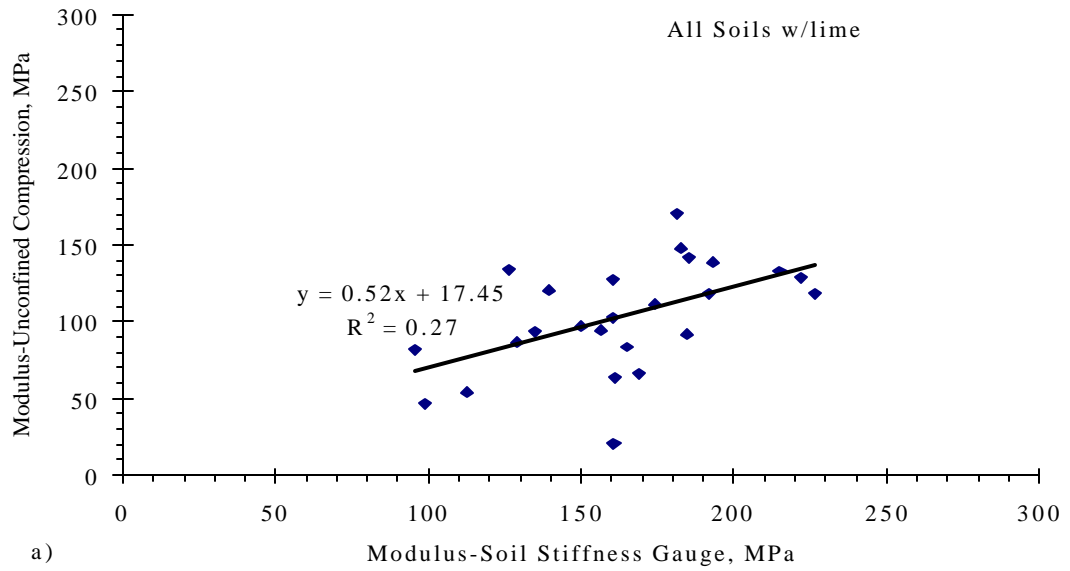
One objective of this research was to attempt to measure changes in soil stiffness with time using the soil stiffness gauge. It was determined that changes could be measured and that there was some correlation between soil stiffness and the unconfined compression strength.

Soil stiffness of the clay samples showed increases in stiffness over the 28-day curing period. The greatest increase came from the CL soils treated with lime and cement. The CH soils had modest increases in stiffness with each additive throughout the curing period. The fly ash and Permazyme treated samples developed little or no stiffness gains over the curing period.

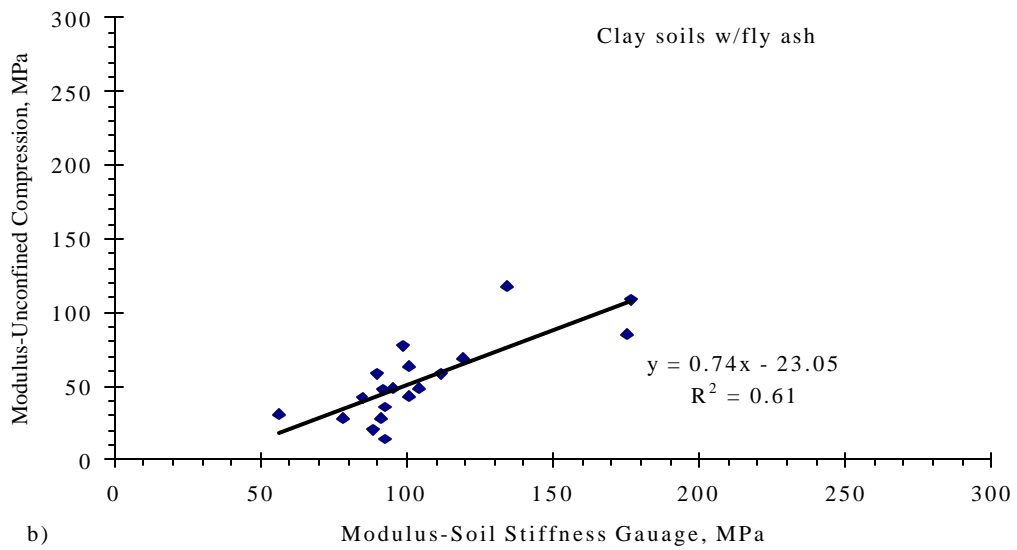
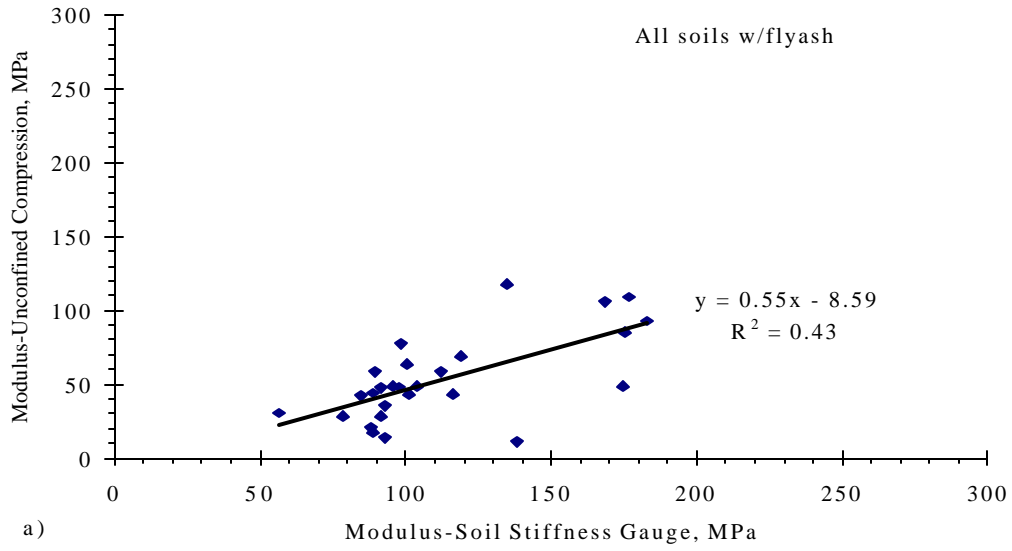
The impact echo values presented showed a wide range of stiffness values throughout the curing period. The stiffness values from the impact-echo test were probably much larger than those from the soil stiffness gauge because the stress applied to the soil sample was less than that

applied by the stiffness gauge. Values that were calculated showed no consistent increase in stiffness over the testing period. Based on the results this procedure was considered unreliable.

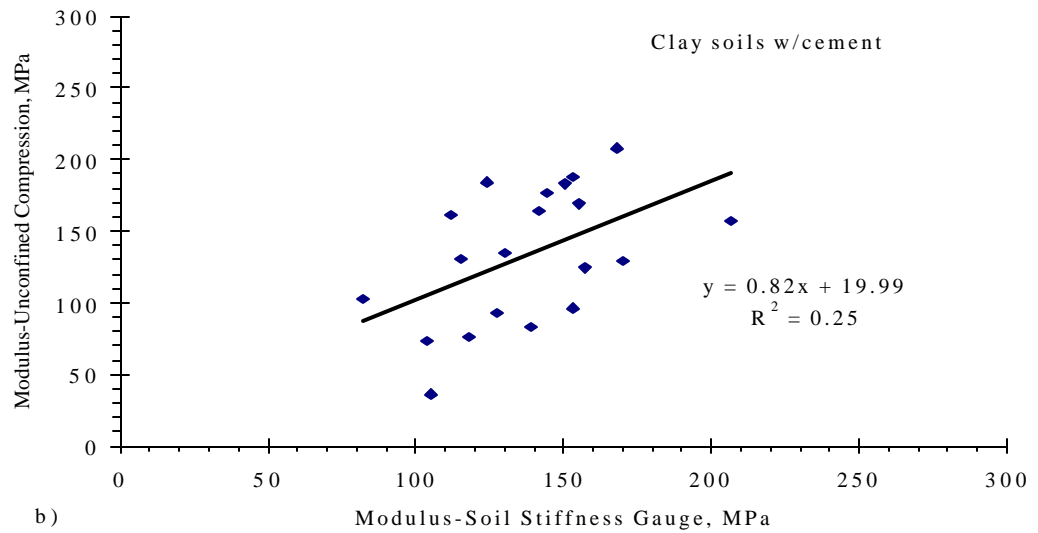
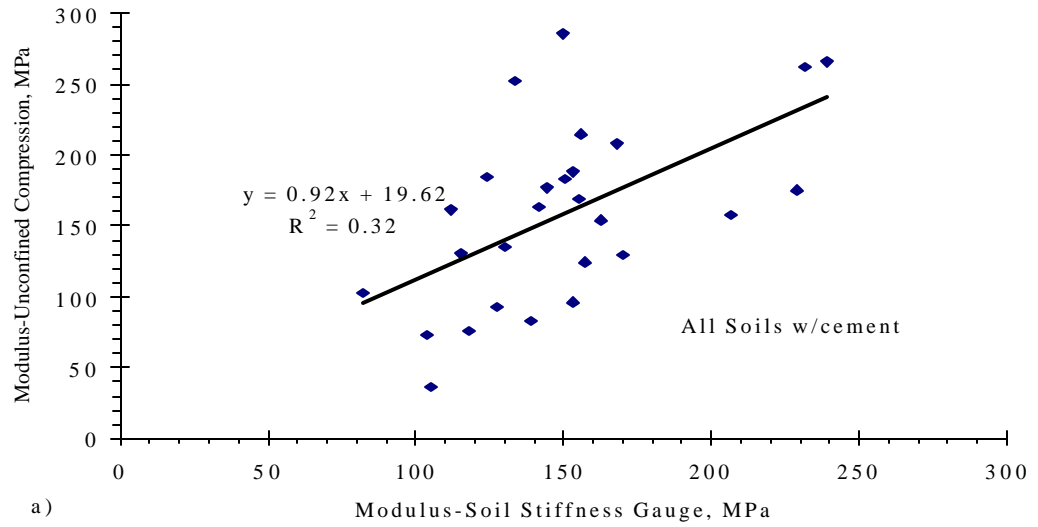
The soil stiffness readings obtained at the 28-day curing period were converted to modulus values and compared with modulus values from the unconfined compression data. The modulus values were determined from the maximum stress that the sample experienced. Figure 5.1a shows the data from the soils that were mixed with lime. The  $R^2$  value for the two moduli was 0.27. Figure 5.1b also shows the data from only the clay soils mixed w/lime and the data trend shows a better correlation ( $R^2 = 0.46$ ). Figure 5.2a shows similar correlations with the fly ash samples. The  $R^2$  value again improved by plotting only the clay soils with fly ash. The cement-stabilized samples in Figure 5.3 had a lower correlation coefficient than the lime and fly ash samples. This variation in the data points may have come from the variation of percentage used for the cement contents. The Permazyme samples from Figure 5.4 also showed weak a correlation between the modulus values.



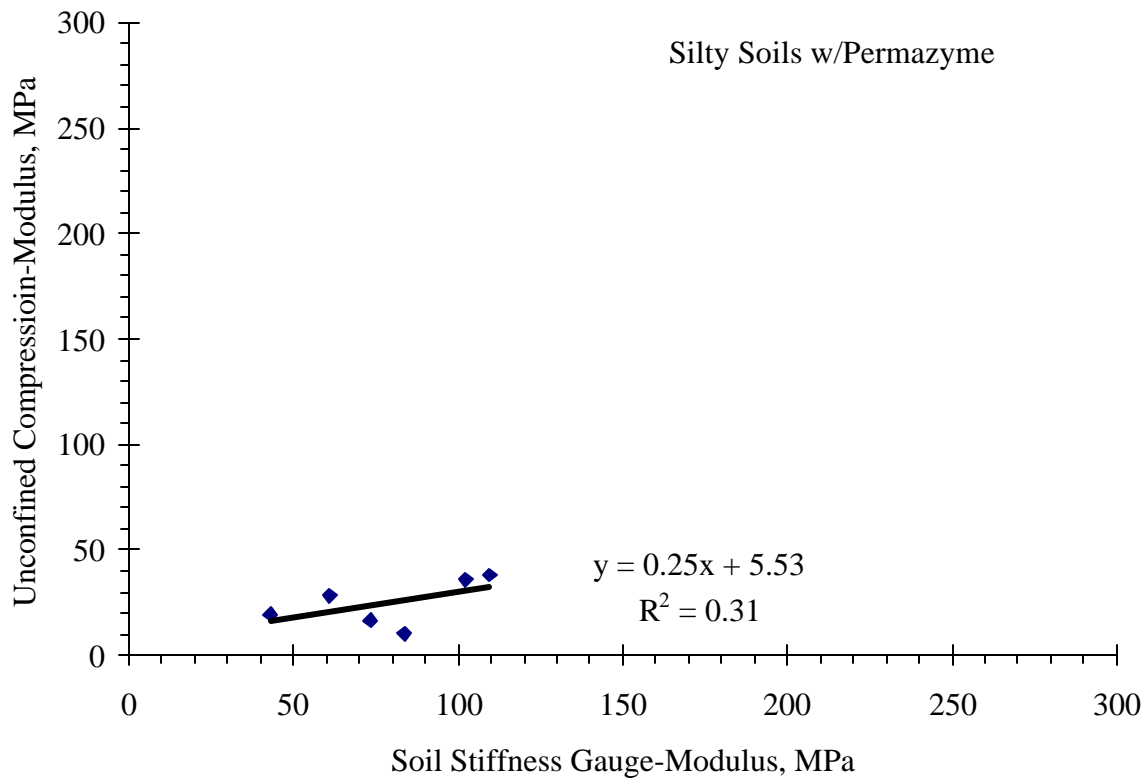
**FIGURE 5.1: Modulus-Unconfined Compression vs. Modulus-Soil Stiffness Gauge**



**FIGURE 5.2: Modulus -Unconfined Compression vs. Modulus-Soil Stiffness Gauge**



**FIGURE 5.3: Modulus -Unconfined Compression vs. Modulus -Soil Stiffness Gauge**



**FIGURE 5.4: Modulus-Unconfined Compression vs. Modulus-Soil Stiffness Gauge**

Figure 5.5 and 5.6 show the 28-day unconfined compression strength versus the 28-day soil stiffness reading. The stiffness and the unconfined compression strength of the lime and fly ash samples correlate poorly. Cement-stabilized samples show a higher correlation compared to the lime and fly samples. Permazyme samples had a correlation of 0.82 because of no cementation and modulus is a function of density.

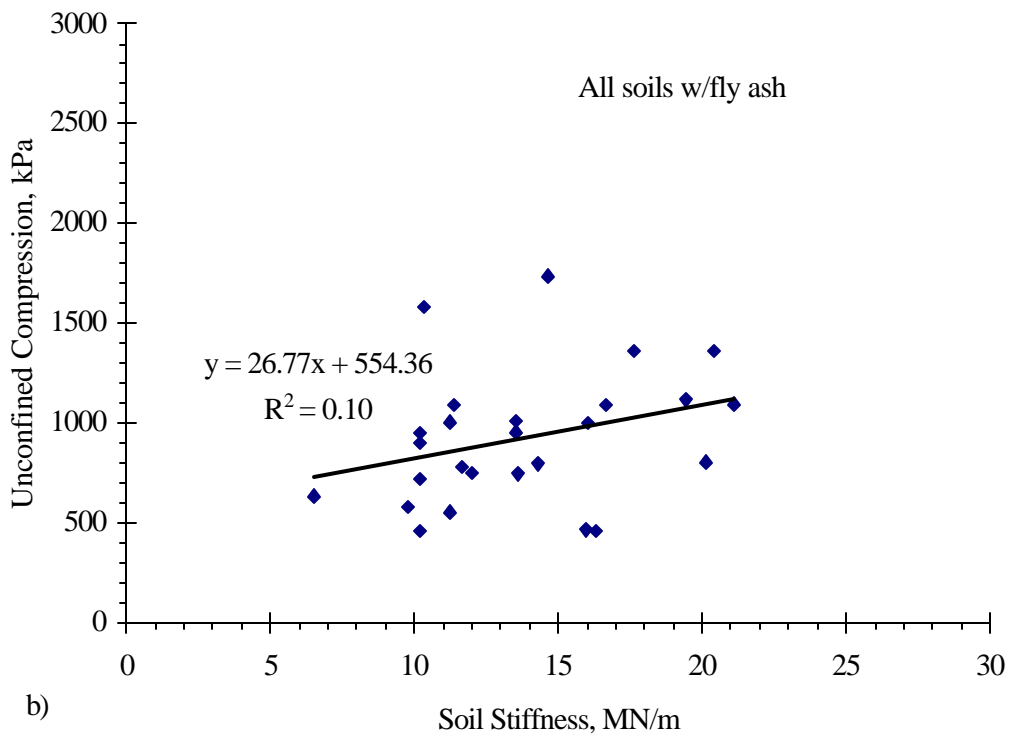
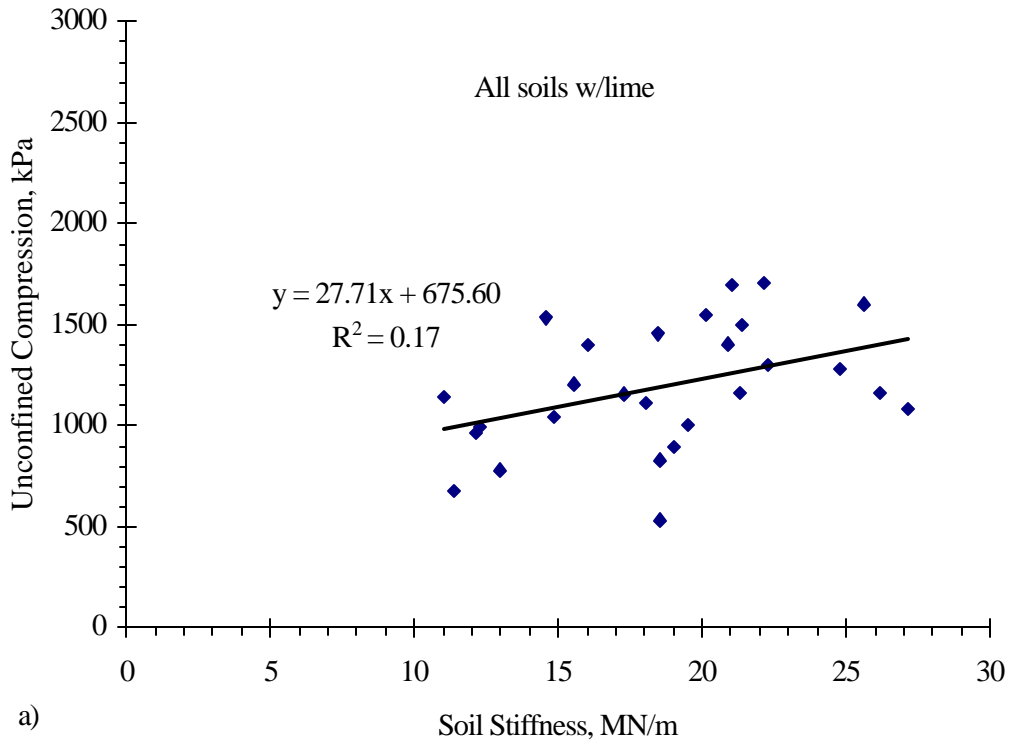
Figures 5.7-5.8 show typical soil stiffness curves vs. moisture content over the 28-day curing period. Appendix D contains the complete listing of the soil stiffness vs. moisture content curves. The two figures show the three different additives mixed with Osage. The lime- and cement-stabilized samples show a consistent increase in stiffness. Throughout the curing period fly-ash-treated samples showed little increases in stiffness over the curing period.



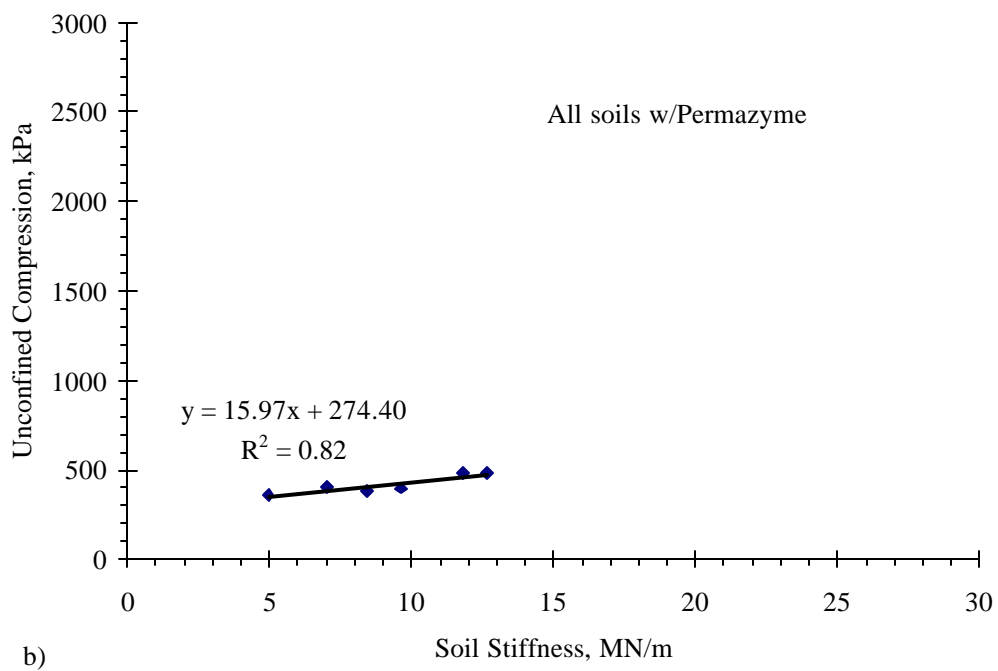
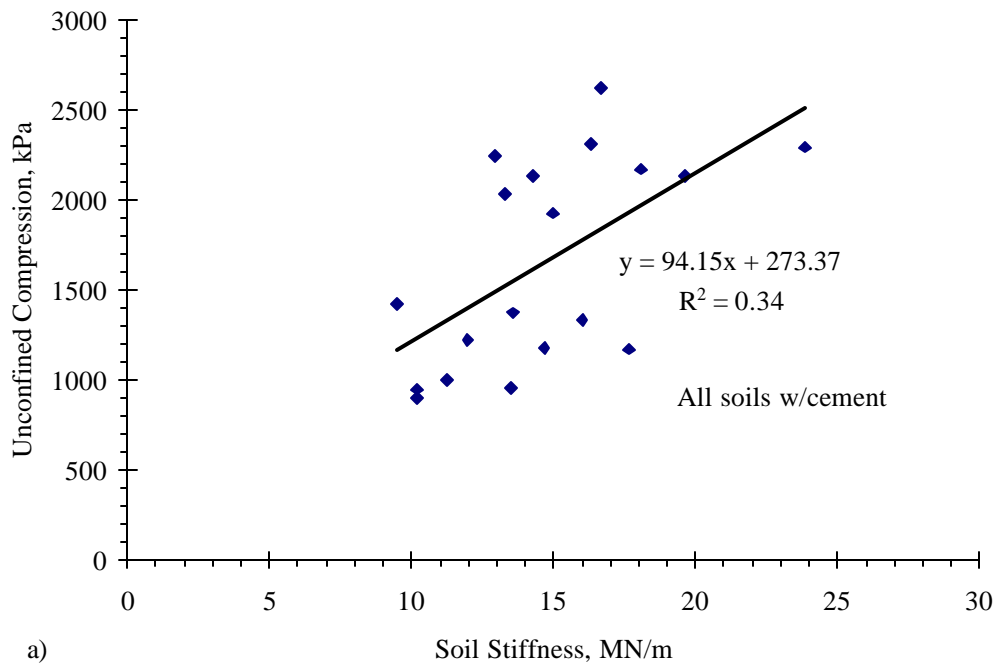
## 5.4 Swell

Swell potential was reduced for all soils when mixed with the chemical additives with the exception of the Beto soils. The Beto soils *increased* in swell over the native soil when mixed with the chemical additives. Beto “Tan” was tested to determine if any sulfates were present in the soil. Tests were conducted by KDOT according to the National Lime Association sulfate testing criteria (27) and it was determined that Beto “Tan” had a moderate sulfate content. Since the other Beto soils experienced an increase in swell with the chemical additives it was assumed that these soils contained sulfates as well.

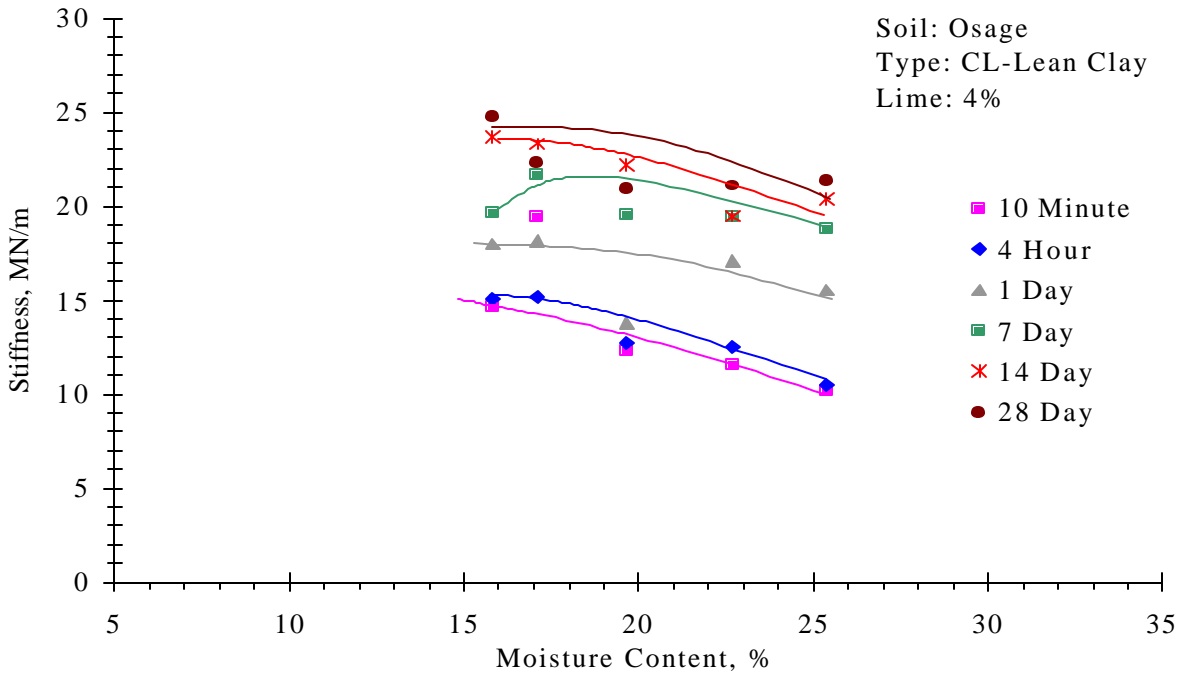
Soils that contain sulfates pose problems because of the increased amount of swell and swelling pressures that are developed when calcium based additives are mixed with sulfate bearing soils. Petry and Little proposed the following explanation for this behavior (26). When the lime is added to clay soil, the pH rises and the aluminum and siliceous pozzolans are released to form calcium silicate hydrate and calcium aluminum hydrate. The presence of the sulfate confounds this reaction and leads to the formation of ettringite, which is an expansive mineral. The formation of ettringite is favored in low alumina environments. Ettringite is stable in both wet and dry conditions and can expand to a volume equal to 227% of the total volume of the reactant solids (26).



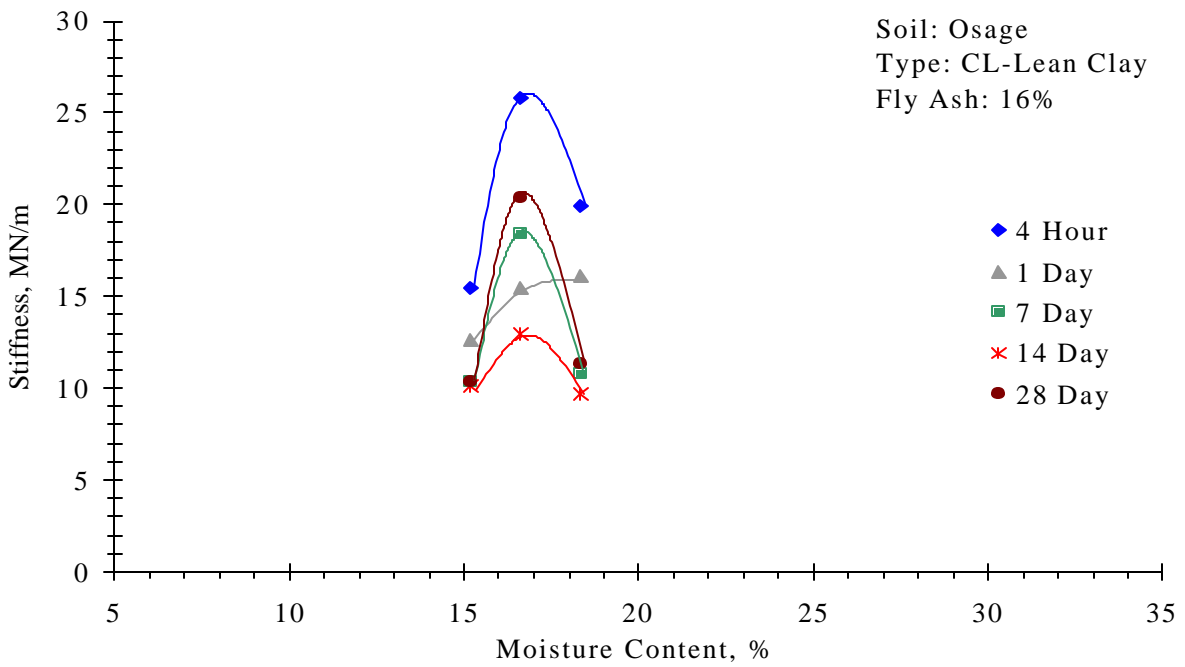
**FIGURE 5.5: Unconfined Compression vs. Soil Stiffness**



**FIGURE 5.6: Unconfined Compression vs. Soil Stiffness**

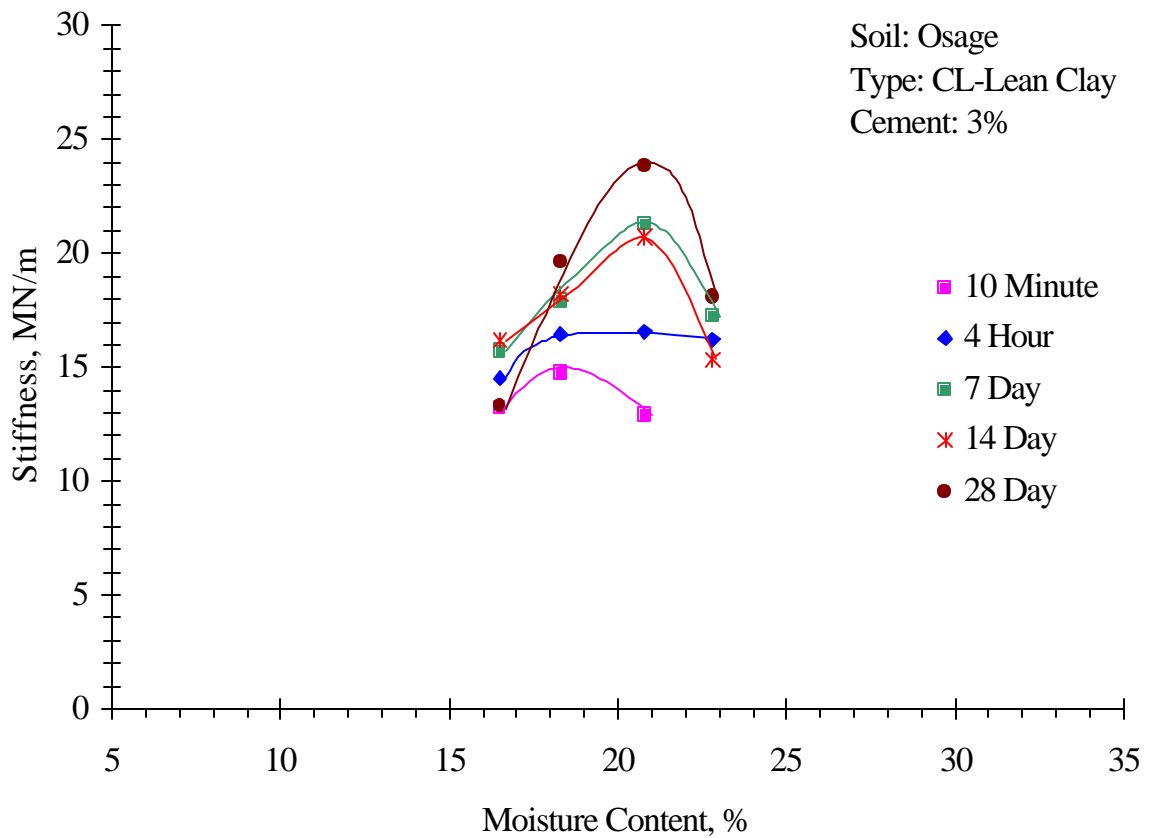


a)



b)

**FIGURE 5.7: Soil Stiffness vs. Moisture Content**



**FIGURE 5.8: Soil Stiffness vs. Moisture Content**

A double treatment of lime with an intermediate mellowing period prior to final treatment and compaction could give expansive minerals an opportunity to form prior to compaction (27).

To simulate this procedure the sulfate soils were tested again with lime and the soil/lime mixture was allowed to mellow for seven days prior to the beginning of the swell test. The swell was reduced a considerably using this procedure, however additional lime and/or mellowing may be required to reduce the swell further.

### 5.5 Freeze-Thaw and Wet-Dry Testing

The freeze-thaw clay samples experienced the most soil loss when mixed with lime. The soil losses improved for clay when mixed with fly ash and cement. Silty soils had similar soil losses with fly ash and Permazyme. The cement stabilized silty soils performed much better than the fly ash and Permazyme samples. According to the PCA requirements for soil cement (28),

the cement-stabilized samples of Atwood, Stevens, and Lakin were within the mass loss requirements for a freeze-thaw test on soil cement. The maximum soil loss set by PCA for Lakin and Stevens is 14 percent and for Atwood the maximum loss is 10 percent.

Freeze-thaw strengths showed that all the treated soils were stronger than the native soil strength after completion of the 12 freeze-thaw cycles. The cement treated samples retained a higher percentage of the native strength after the freeze-thaw cycles. Fly ash treated samples lost strength after the cycles, but still retained a large percentage of the stabilized strength. Permazyme treated samples retained similar strength values.

The wet-dry testing showed that cement treatment of the CH soils and the silty soils performed better than lime and fly ash treatments. Lime performed better with the CL soils than both fly ash and cement. The wet-dry test is a very aggressive test and some fly ash and Permazyme samples did not complete any of the testing cycles.

## **5.6 Leaching**

Strengths generally declined somewhat with leaching, however most samples maintained a significant percentage of the strength gain achieved with stabilization, as shown in Figure 4.10. For the lime stabilized soils the average strength after leaching remained 280 to 380% above the strength of the native soil for three of the four soils, with the exception of the single Beto “Tan” sample. The fly ash treated samples also experienced strength loss, but retained 110 to 270% of the native strength. For those soils where cement content was based on reducing the PI to less than 10, the soils retained 300 to 440% of the native strength; while for those soils with cement contents based on guidelines for soil cement (22) the strength retained was over 1100% of the native strength. Strengths of the two enzyme treated soils were reduced to near the native strengths.

The permeability of each sample was determined for each 7-day period and is reported in Table 4.4. Permeability values decreased with time for cement and for lime treated soils, which is consistent with results reported by McCallister and Petry (24). This reduction in permeability may be related to the formation of permanent interparticle bonds (24) or clogging of soil pores during particle movement. Permeability values for fly ash were constant or increased, while essentially no flow occurred through the enzyme-stabilized soils.

Leachate pH was measured periodically throughout the leach testing of all samples with flow until flow ceased or the test was completed. Higher pH values are considered desirable for the promotion of pozzolanic reactions. The results are shown in Figure 4.11. The lime treated samples generally maintained a leachate pH above 11.5 with the exception of Beto “Brown” at 21 days, which appears to be an anomalous occurrence as the pH returned to near 12 at 28 days for both samples. Fly ash values were generally low for stabilized materials with values between 8 and 9 for all samples tested. The pH of cement treated soils were all above 11.5 after 24 hours, but declined to as low as 8.4 prior to the cessation of flow.

Atterberg limits were determined after leaching for samples with significant native plasticity to evaluate whether reductions in plasticity were affected by leaching. All lime treated samples remained non-plastic and the plasticity of the cement treated samples decreased (improved) with time. The PI for the fly-ash-treated soils increased to near the untreated levels.

The results for the lime-treated samples are consistent with those described by McAllister and Petry (24), who found that samples treated with sufficient amounts of lime for maximum pozzolanic reaction were much less affected by leaching than samples treated with lower concentrations of lime. McAllister and Petry (24) hypothesized that the increased calcium

concentration from additional lime was sufficient to satisfy the calcium demand from the leaching water and also be available for pozzolanic reactions.

The cement-treated samples also appeared to have sufficient time and cement concentration to form permanent bonds, based on the decline in permeability with time and the relatively high strengths after leaching. The decline in performance of the fly-ash-treated samples was similar to that of the samples treated with low concentrations of lime by McAllister and Petry (24), which suggests that the bonding within the fly-ash-stabilized samples may not have been as permanent as for the lime- and cement-treated samples.



## Chapter 6

### Conclusions

The following conclusions were reached based on the interpretation of the results of this study.

#### 6.1 General Conclusions

1. Lime, fly ash and cement were effective in improving the Atterberg limits on the soils used in the study. Each soil showed improvements in plasticity with each additive tested, however fly-ash- and cement-treated samples generally retained some plasticity. Lime-treated soils had the greatest improvement with all soils becoming non-plastic with the addition of sufficient amounts of lime.
2. The native swell values for the CL soils were lowered dramatically with lime, fly ash and cement. The reaction of the calcium-based additives with the sulfate-bearing CH soils resulted in swelling similar to or higher than the native state. Additional mellowing prior to compaction to allow the sulfates to react with the calcium-based additives helped to lower the sulfate-related swelling.
3. Significant strength improvements were observed for soils treated with lime, fly ash, and cement while enzyme-treated soils showed modest strength gains. Most of these strength gains were retained after freeze-thaw and leaching. Cement- and lime-treated soils generally retained the most strength after leaching while fly-ash-treated soils retained some of their strength gains.
4. The cement treated soils had the least soil loss in freeze-thaw testing, while fly ash treated soils had lower soil losses in freeze-thaw testing than lime treated soils. Relative performance in the wet-dry cycles was mixed, with lime generally performing better on fine-grained materials and cement on coarse-grained soils, although cement performed relatively well with the CH clays. Fly ash performed well only on the SM soil, where it

survived the full 12 cycles. The enzyme-treated soils had soil losses similar to those treated with fly ash in freeze-thaw testing but did not survive the first cycle of wet-dry testing.

5. Lime- and cement-treated soils maintained higher strengths and lower plasticity values than fly ash treated soils after leaching. Higher permeability, lower pH, and higher plasticity values after leaching and relatively constant soil modulus values for cured samples suggests that fly-ash-treated soils may not be experiencing the formation of additional interparticle bonds over time, and that the improvements in soil properties with fly ash may be reversed to some degree by leaching. The pH values also declined for cement treated soils after 21 days of leaching, which may indicate a reduction in pozzolanic reactions. However, permeability values also declined during the first 21 days, suggesting that some reactions may have been occurring during that period.
6. The enzymatic stabilizer did not substantially improve soil performance when evaluated using the test regimen described.
7. The improvements in soil properties reported were observed under laboratory conditions. Less thorough mixing resulting in larger soil lump sizes, as may occur in the field, could result in less effective stabilization as shown by Petry (29).

## **6.2 Conclusions Based on the Soil Stiffness Gauge**

1. The soil stiffness gauge can be used to monitor changes in the stiffness of standard samples of stabilized soils.
2. A moisture-stiffness relationship exists for most stabilized soils and can be evaluated with the soil stiffness gauge.
3. Lime- and cement-stabilized samples resulted in samples with a higher stiffness than the fly-ash-treated samples. Permazyme-stabilized samples had the lowest recorded stiffness values of the soils that had been stabilized.

4. Lime- and cement-stabilized samples showed strength gains over the 28-day testing period, suggesting that stabilization reactions and strength gains were ongoing. Most of the fly ash stiffness gains were achieved very early in the curing process with little additional gains over time. This observation, in addition to the leaching results, suggests that for LaCygne fly ash the most significant portion of the stabilization reactions occur very quickly after mixing.

## **Chapter 7**

### **Recommendations for Implementation**

The following chapter contains recommendations for consideration by KDOT on the selection of soil additives and for further testing in the soil stabilization and modification area. It is understood that other considerations beyond the scope of this report may make the adoption of selected recommendations unfeasible.

#### **7.1 Basis for Selecting the Additive**

The procedure adopted for the selection of the most appropriate soil additive should be a function of the expected contribution of the stabilized soil to the pavement system. For pavement designs that expect a relatively limited contribution from the soil, the primary benefit of stabilization is generally the control of volume change. For these conditions it is recommended that additive selection should be based on the ability of the additive to control shrink/swell behavior.

It is recommended that the evaluation of the potential for volume change be done using swell tests. Test methods for swell evaluation could include the existing KDOT volume change method, ASTM D 4829 Standard Test Method for Expansion Index or Soils, ASTM D 3877 Standard Test Methods for One-Dimensional Expansion, Shrinkage, and Uplift Pressure of Soil-Lime Mixtures, or ASTM D 3668 Standard Test Method for Bearing Ratio of Laboratory Compacted Soil-Lime Mixtures. Other test methods have also been developed that could be used.

Use of Atterberg limits for the evaluation of the effect of additives on swell potential should be used with caution, as Atterberg limits will often not fully reflect the contribution of fly ash and cement to swell control.

Lime, fly ash, and cement were all successfully used to limit swell for soils other than CH clays. If control of volume change is the only criteria, it is recommended that selection of additives for treatment of these soils be an economic decision. Selection of the additive could be left as an option to the contractor, after treatment percentages for each additive have been established.

It is recommended that selection of additives for CH soils be done with special care because of their higher potential for significant volume change. There are alternatives to lime, particularly cement, that can be effective for these soils. However, it is recommended that testing be conducted to confirm that a sufficient percentage of additive is specified. It is possible that effective treatment options will be eliminated due to the treatment expense. It was observed that cement became economically less competitive with decreasing grain size and higher plasticity, as more cement was required.

It is likely that stabilized subgrades are providing a significant contribution to the pavement system. If the design procedure is altered to account for this contribution, then strength and durability testing should be included as a part of developing the additive specifications. For this purpose, wet-dry testing may be more appropriate than freeze-thaw testing, as it is likely to be the more common environmental condition. Leaching may also provide guidance to long-term performance, however it must be done with care to realistically model field conditions.

Permazyme is not recommended as a stabilization additive, as it did not perform as well as the other additives for the test program described.

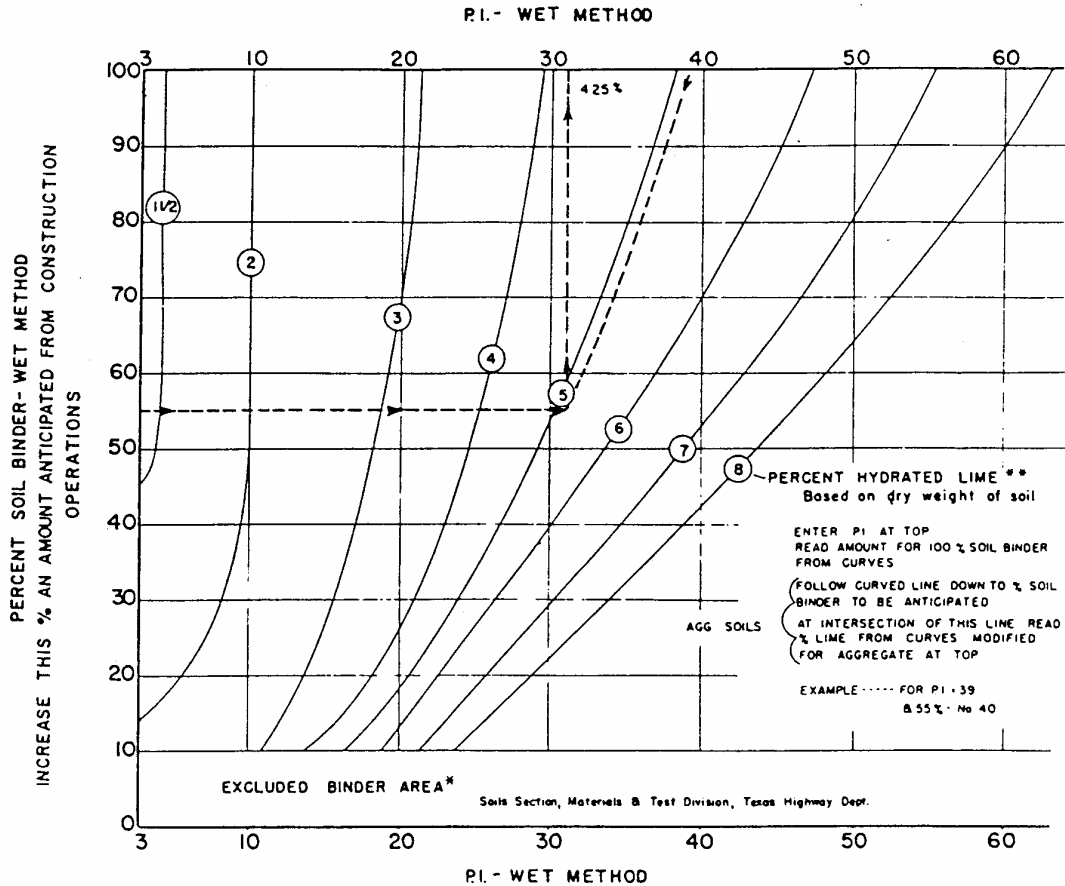
## **7.2 Determining the Amount of Additive to Use**

The results from this research show that the amount of a given additive required to stabilize a soil varies with soil type, which is consistent with previously published work. It is therefore recommended that KDOT consider implementation of a procedure for establishing the percentage of additive to use based on the soil to be compacted.

The quantity of additive to use can be estimated by comparison with similar soils from this study, by published correlations, as shown for lime in Figure 7.1, or directly by various test methods, such as ASTM D 6276 (for lime). Guidelines from the Portland Cement Association were adequate to stabilize the silty soils used in this research. The cement modified CL and CH soils performed well, but additional amounts of cement may have resulted in additional durability performance improvements. Selection of the fly ash percentages was the least sophisticated and it is recommended that the fly ash percentages be further researched to better optimize the percentage specified for use.

## **7.3 Sulfate-bearing Soils**

Sulfate-bearing soils are present in Kansas. Reactions between calcium hydroxide and sulfates can produce expansive minerals that can result in an *increase* in the swell potential as was shown with the Beto “Tan” soil. Tests for identifying the presence of sulfates both in lab samples and in the field are available and more are under development. It is recommended that KDOT investigate the feasibility of incorporating one or more methods for identifying the presence of sulfates in subgrade soils as a part of the soil characterization process.



**FIGURE 7.1: Texas DOT Lime Percentage Selection Chart (7)**

For those soils where sulfates are present, it is recommended that KDOT consider modifying subgrade construction/stabilization procedures to account for the formation of expansive minerals. Expansive minerals may form with any calcium-based stabilizer. Construction amendments could include a double treatment with lime and an extended mellowing period between treatments to allow for the formation of expansive minerals prior to compaction and trimming. The National Lime Association has published more detailed recommendations on the treatment of these soils (27).

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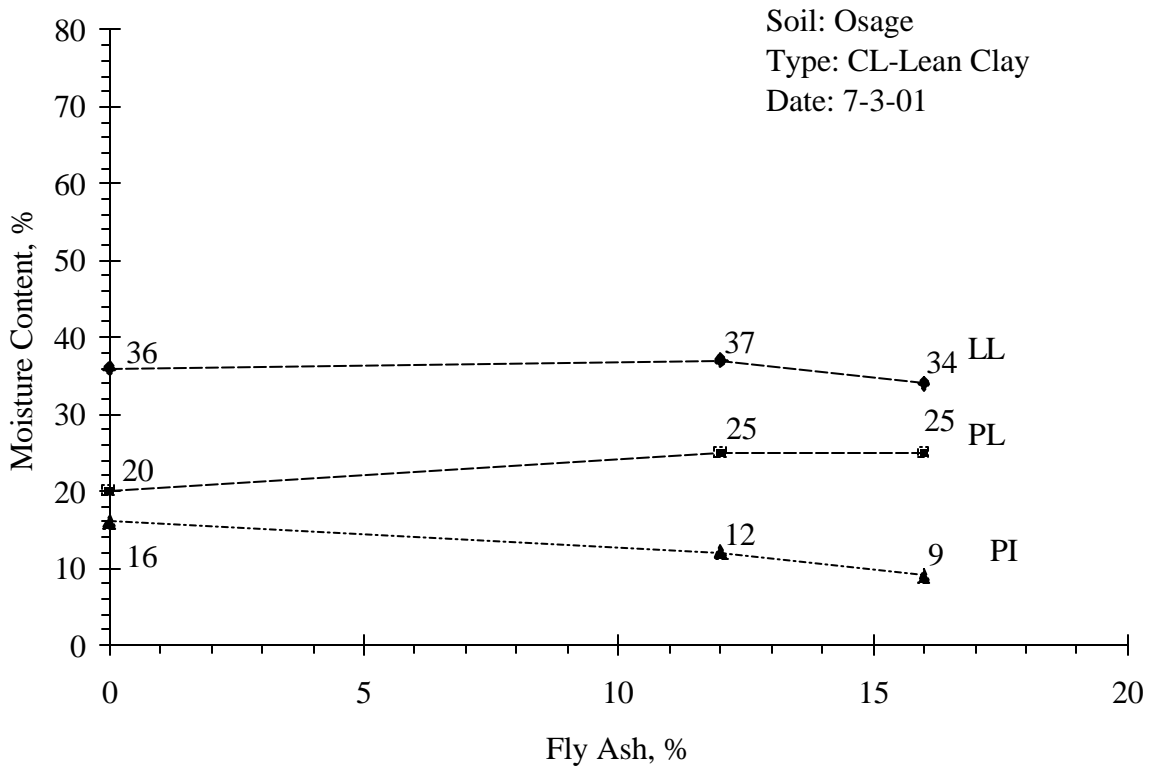
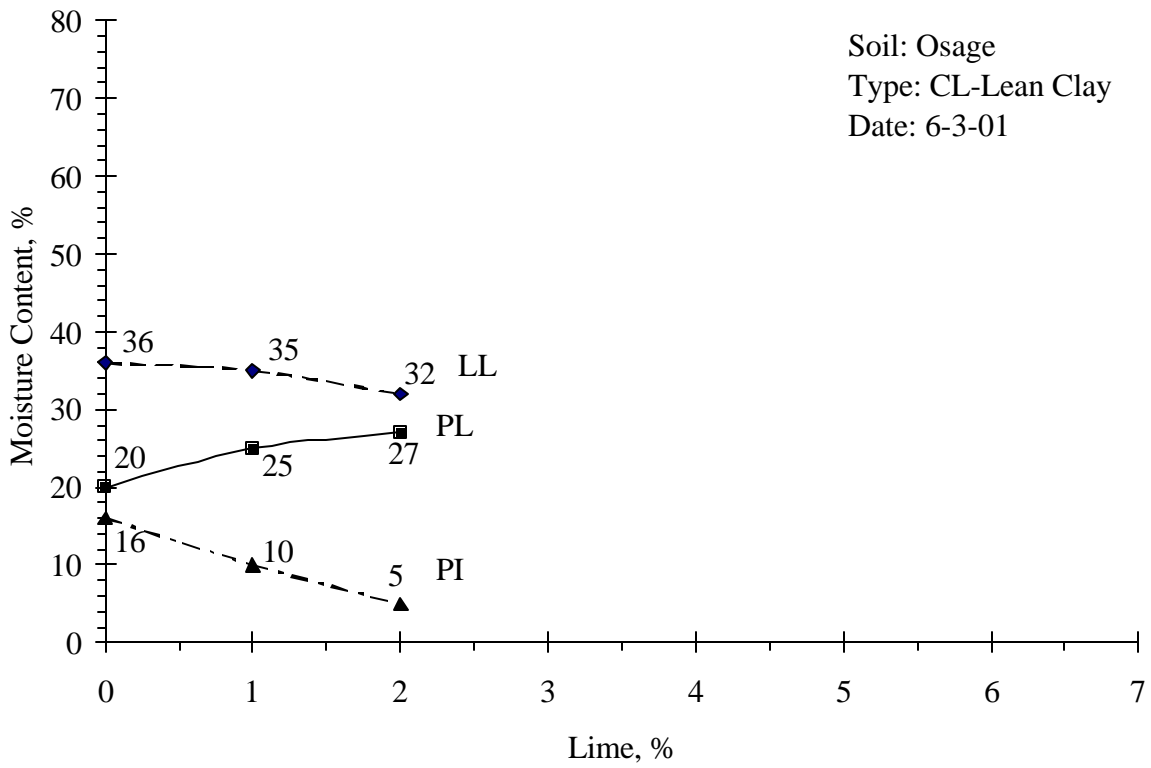


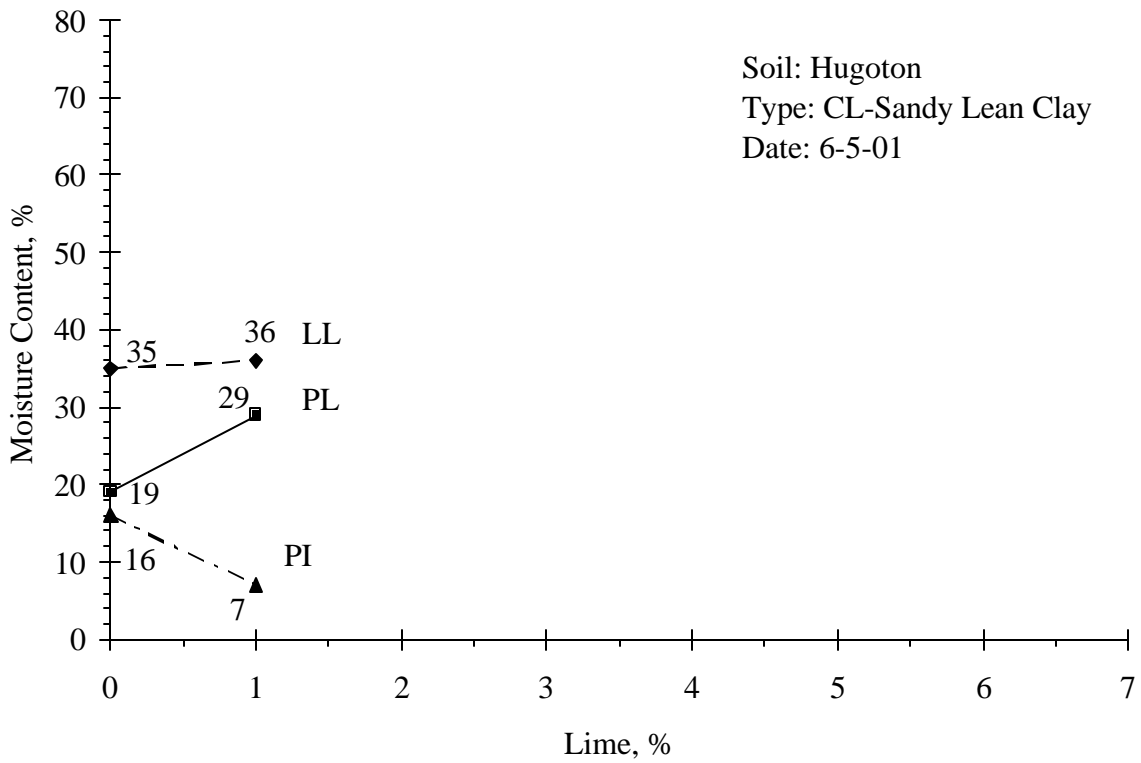
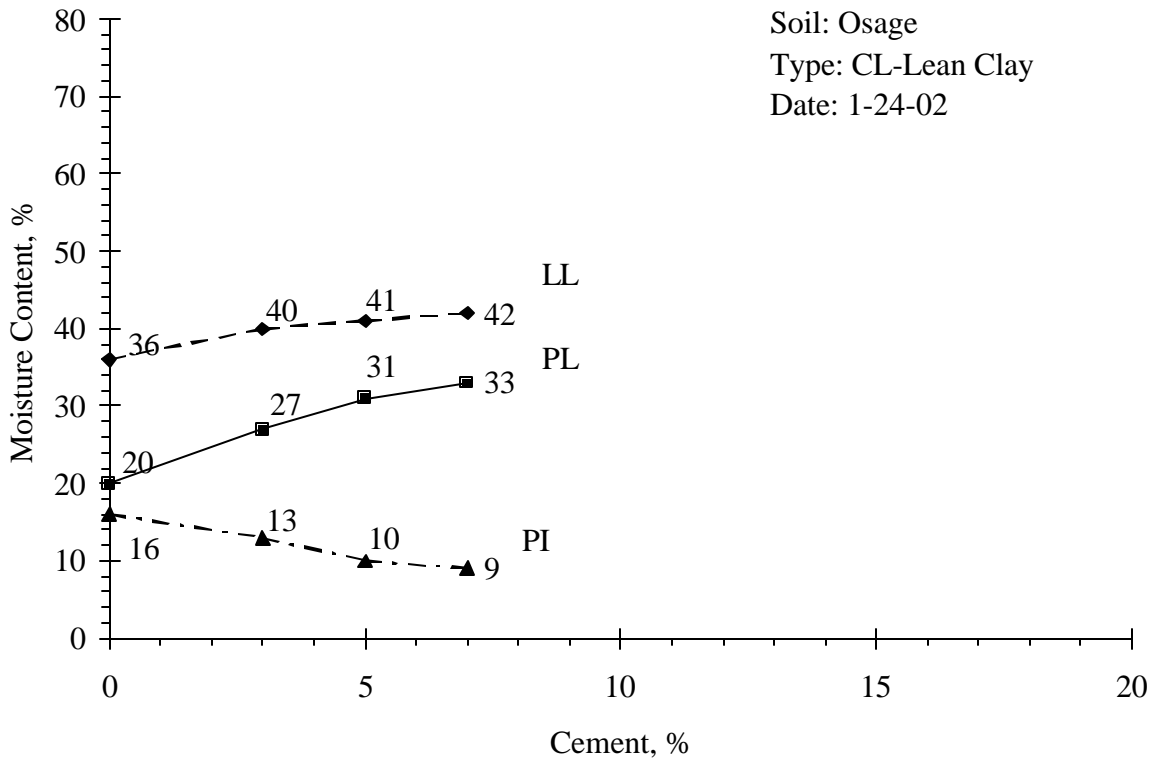
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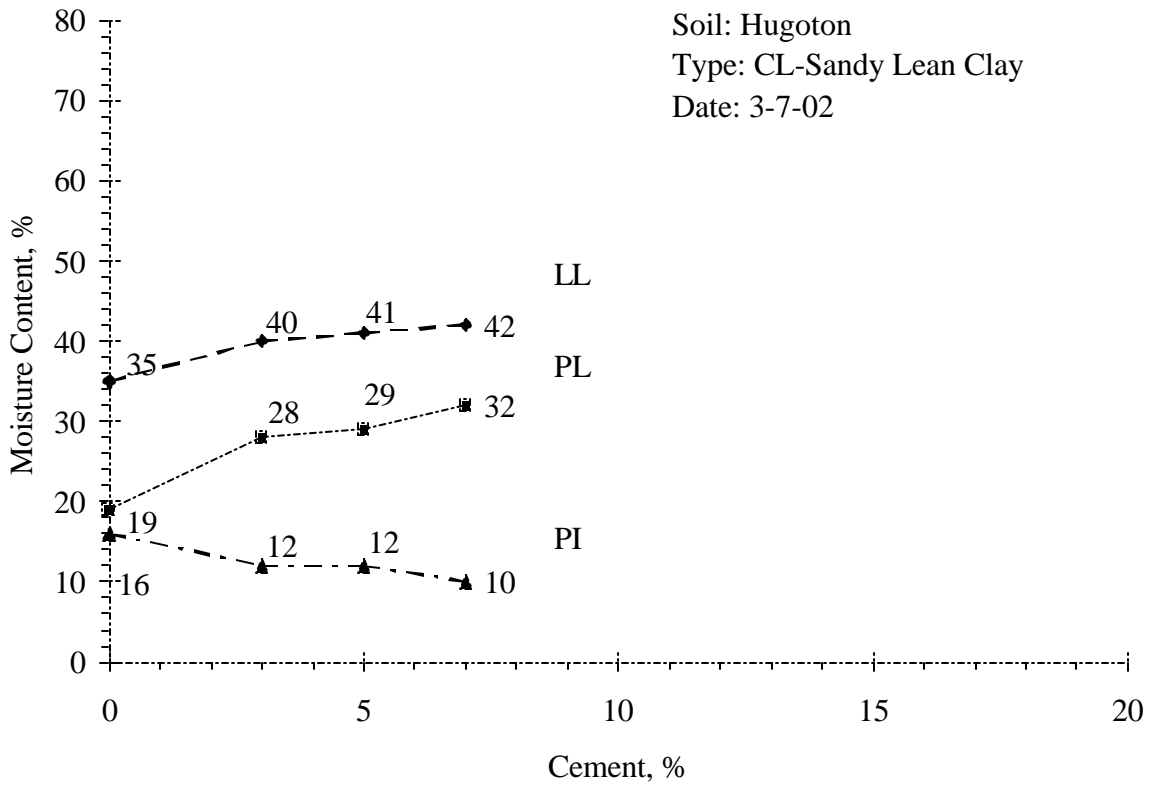
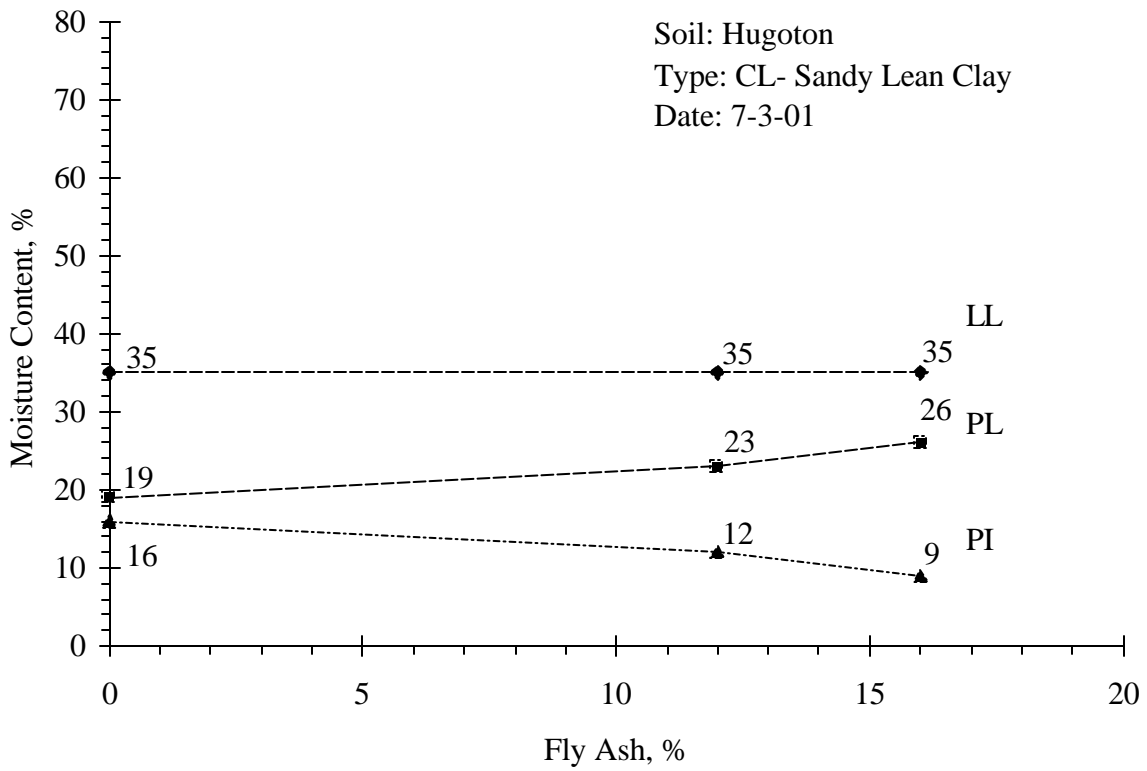
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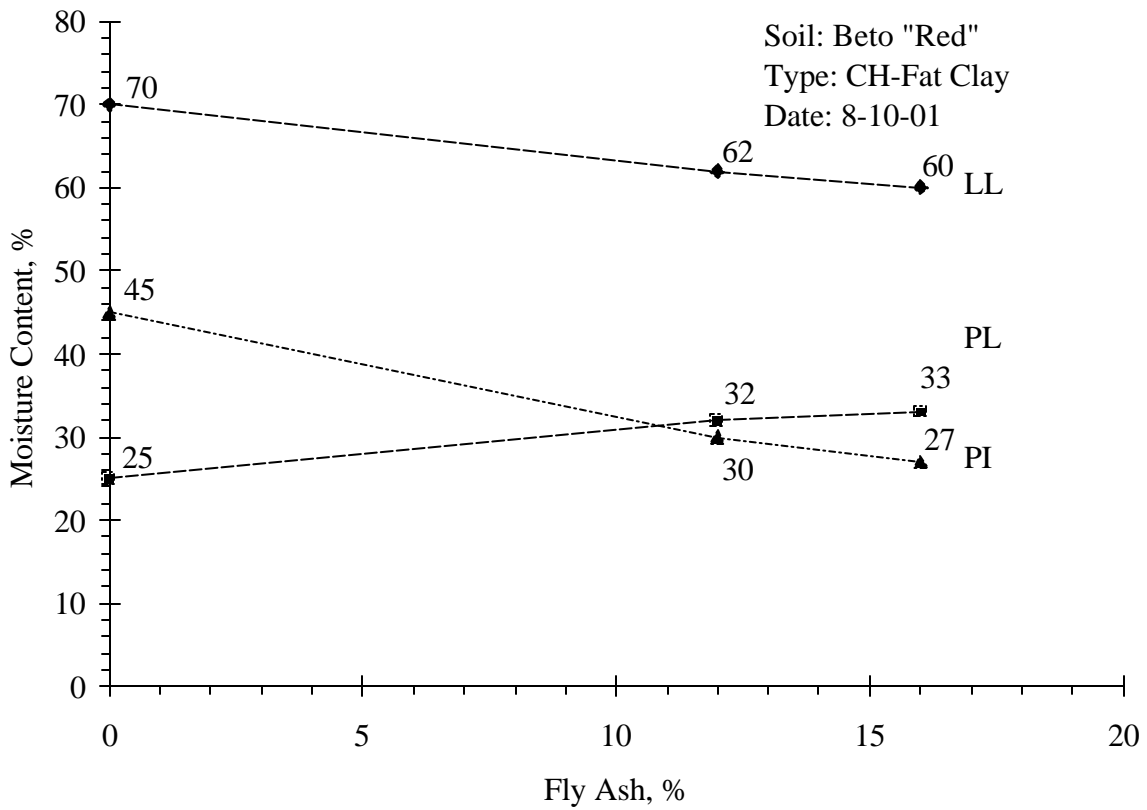
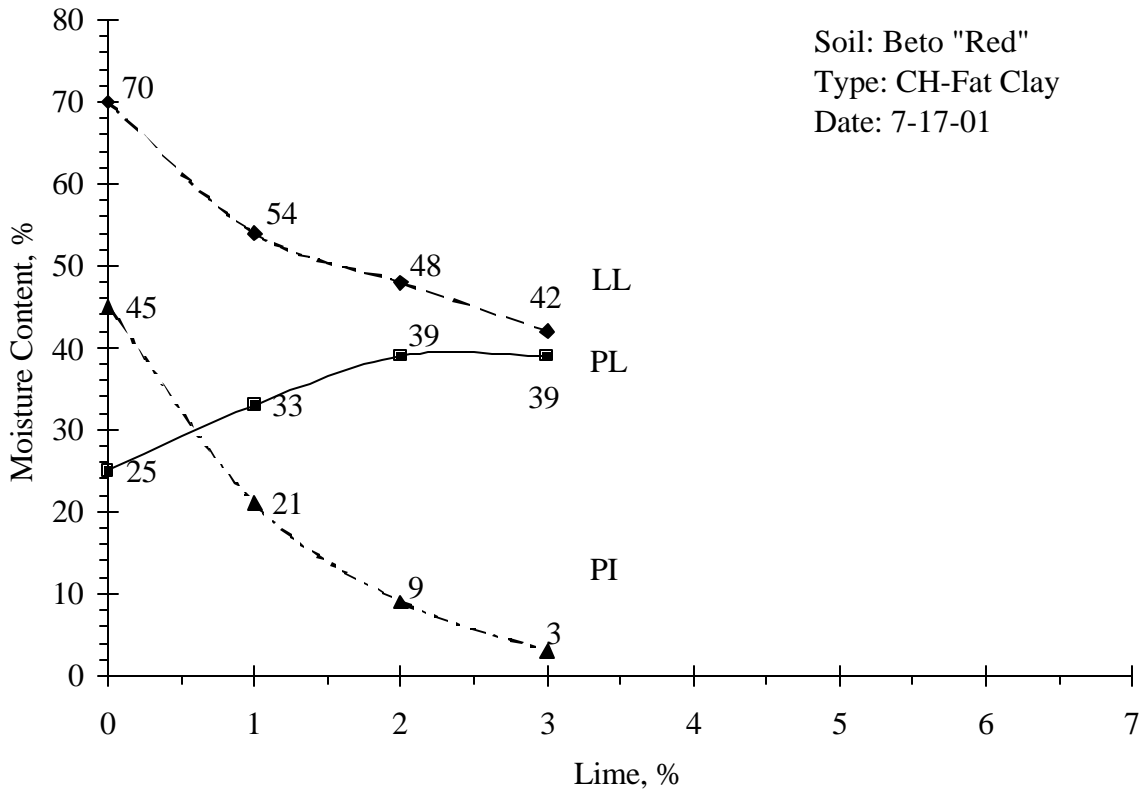
## **APPENDIX A**

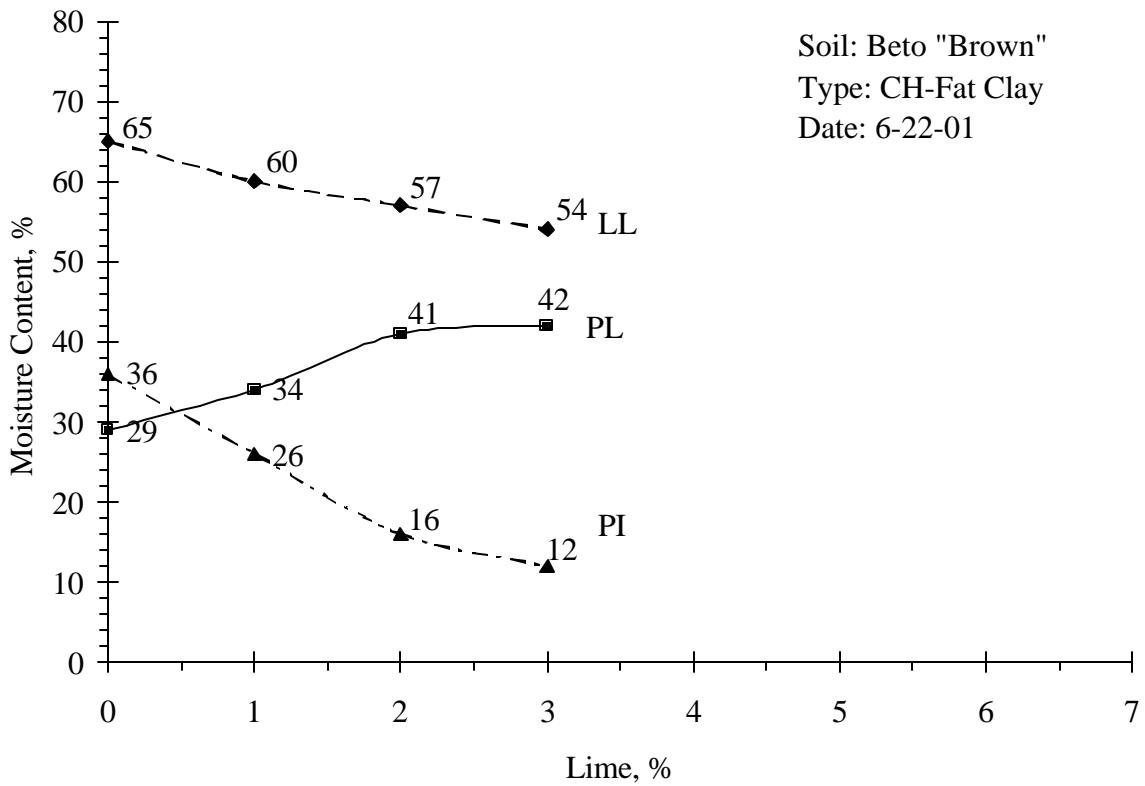
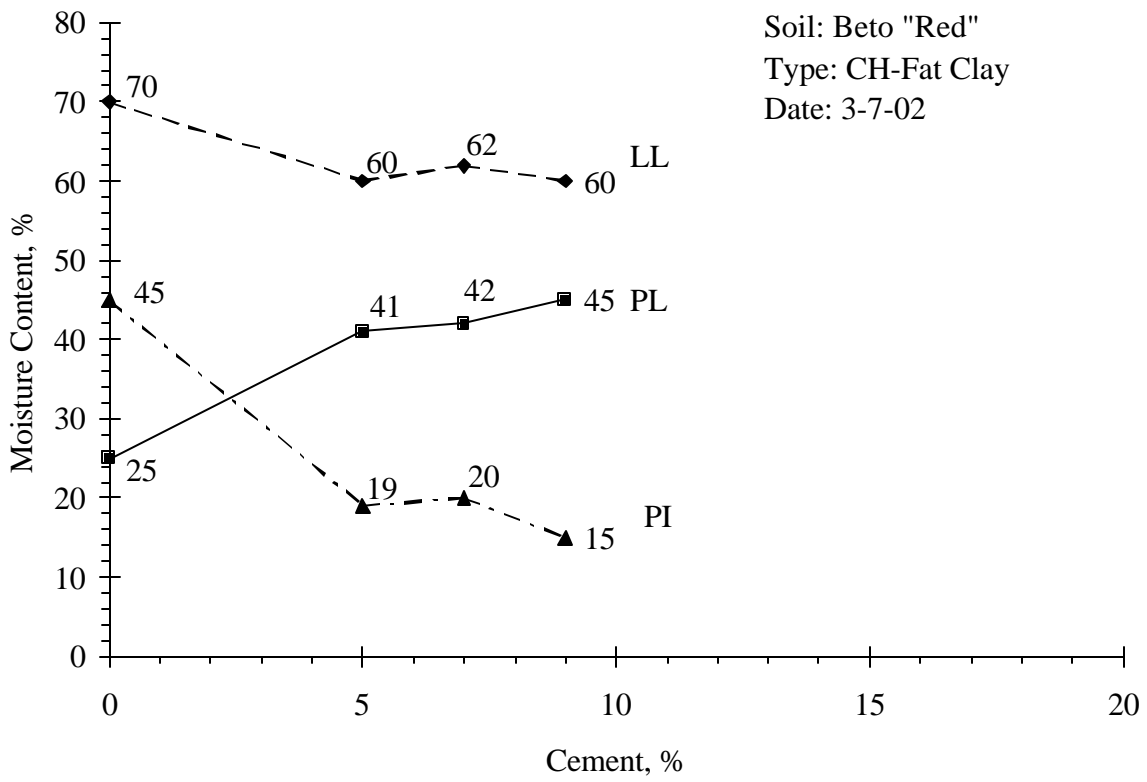
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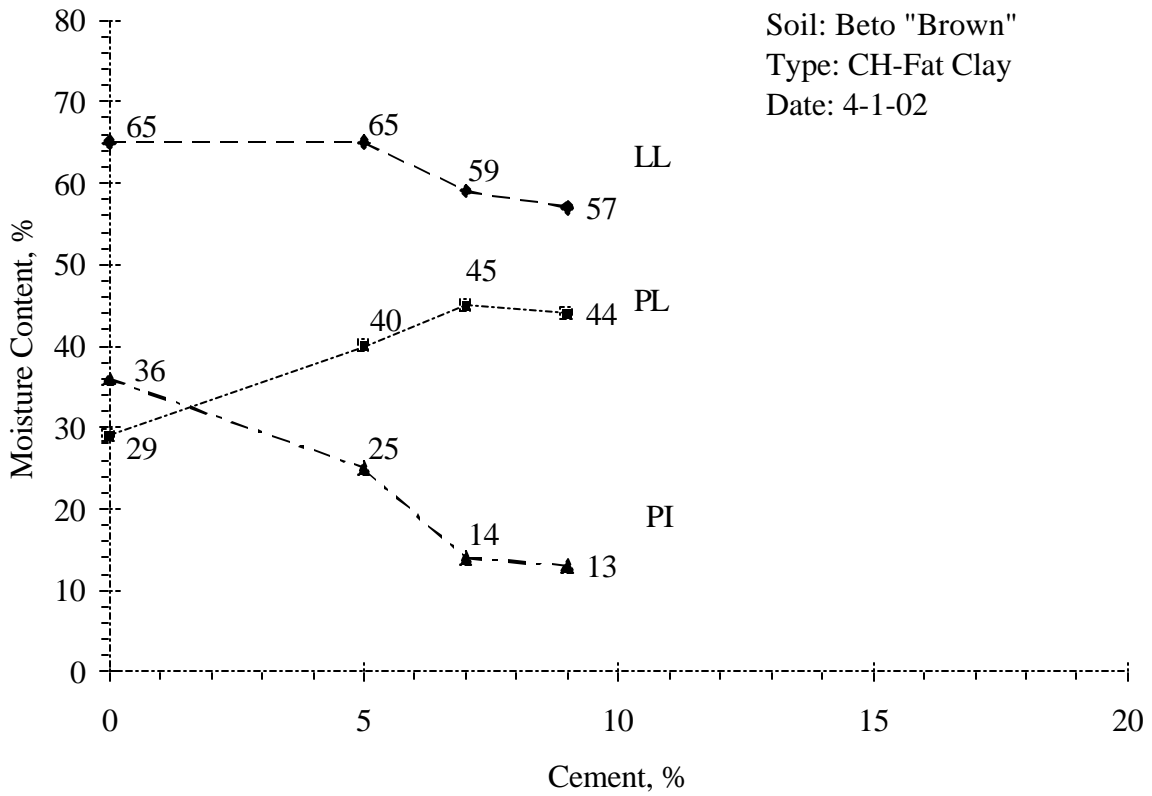
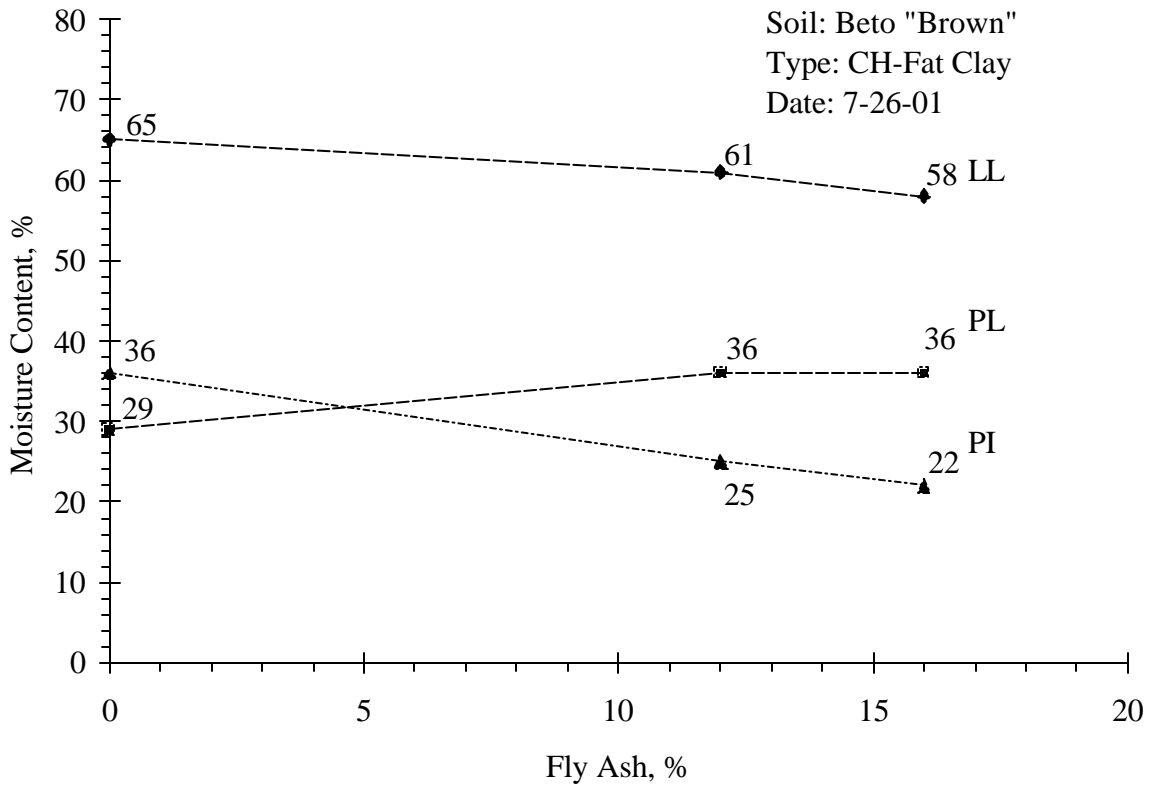


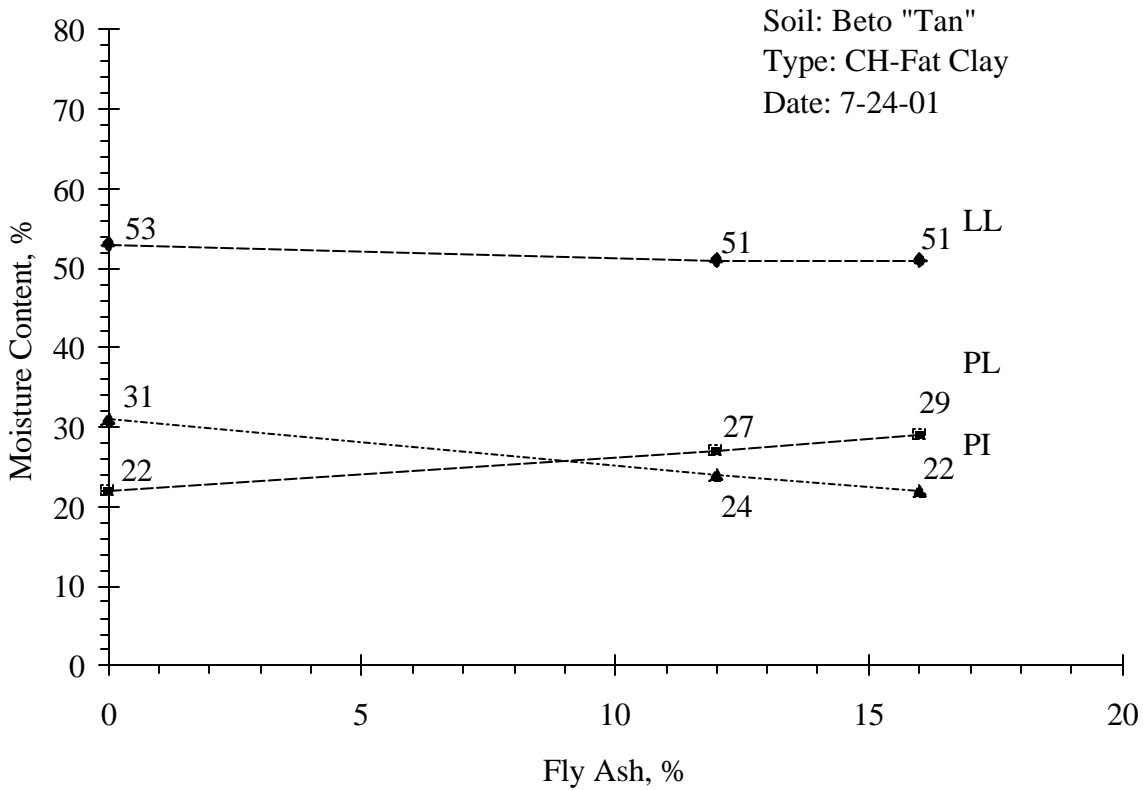
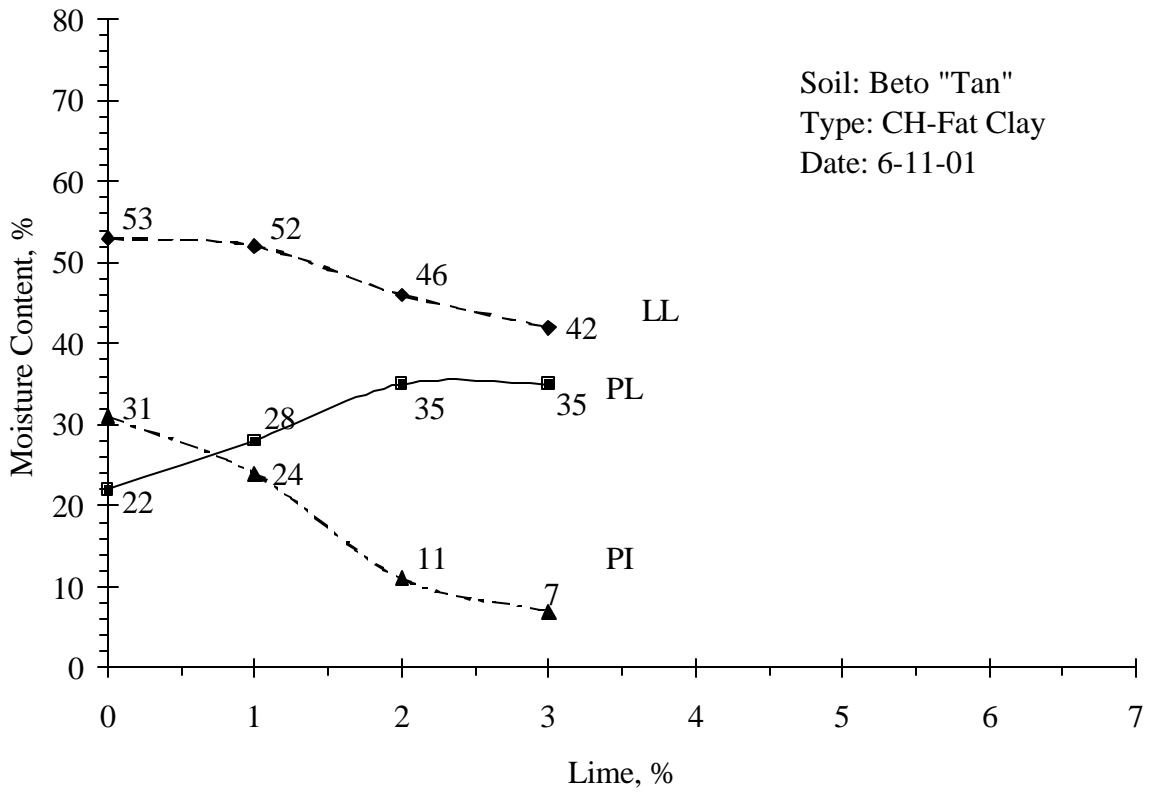


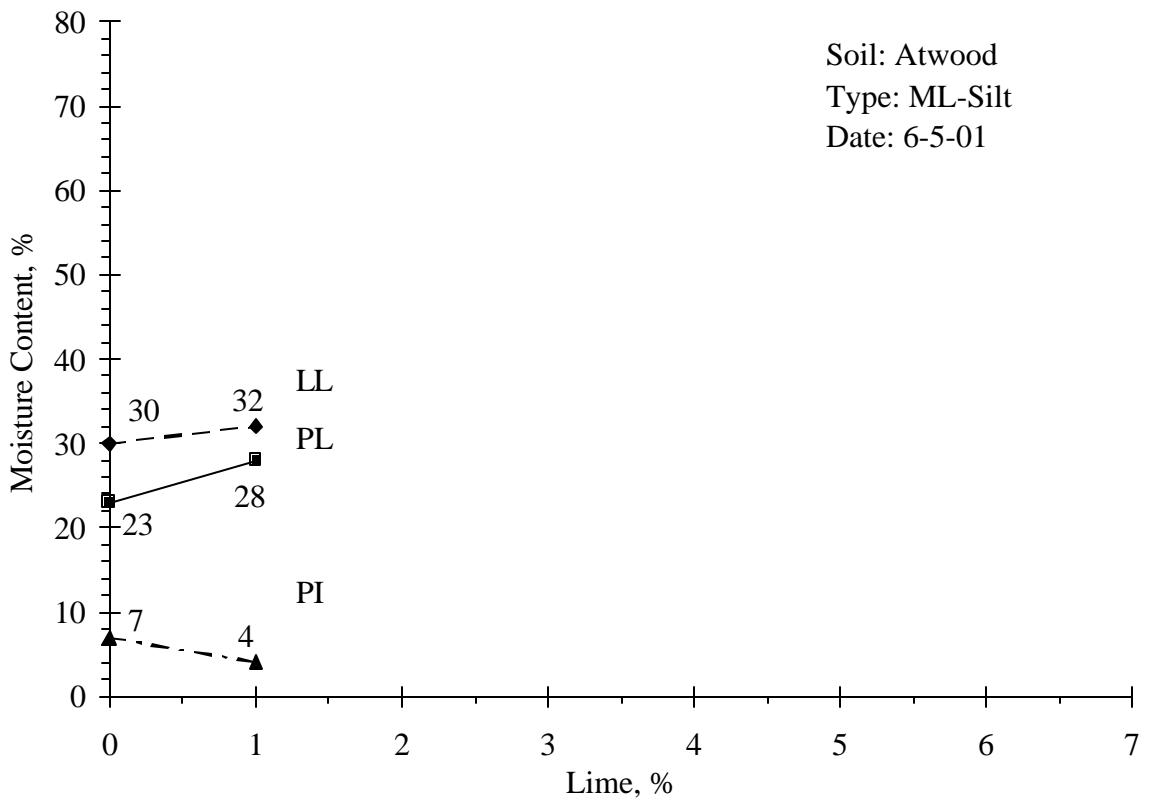
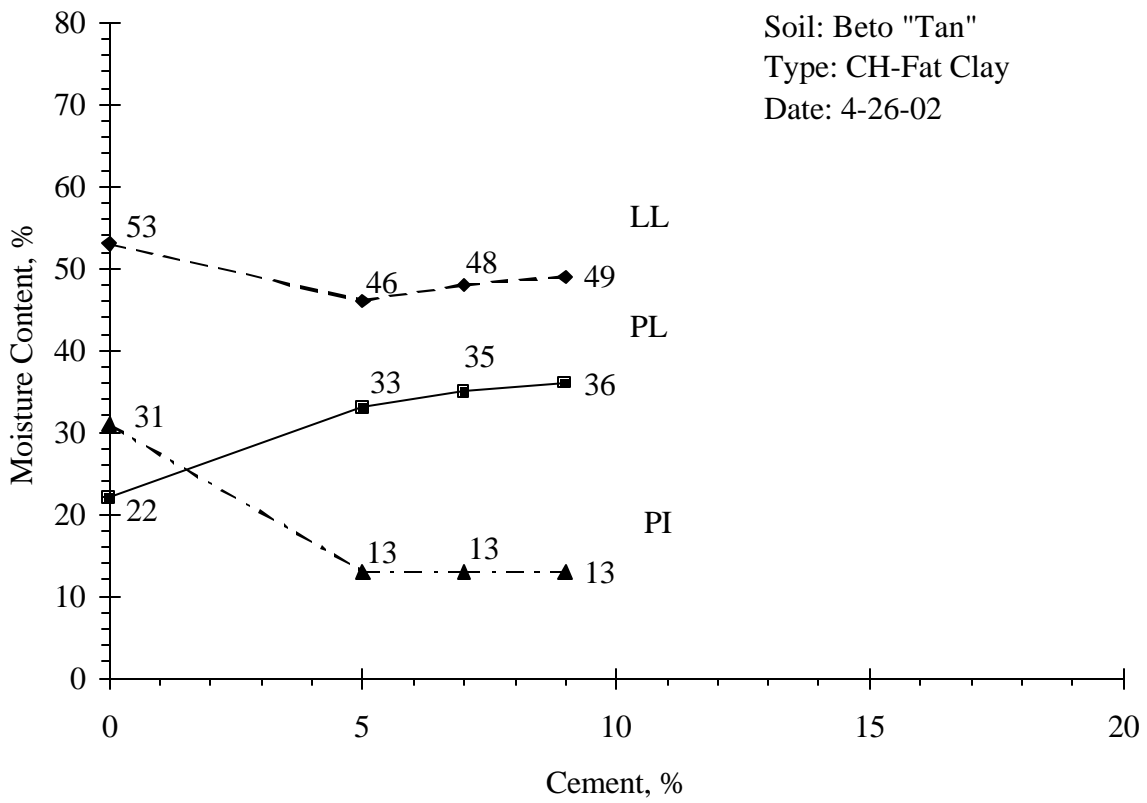




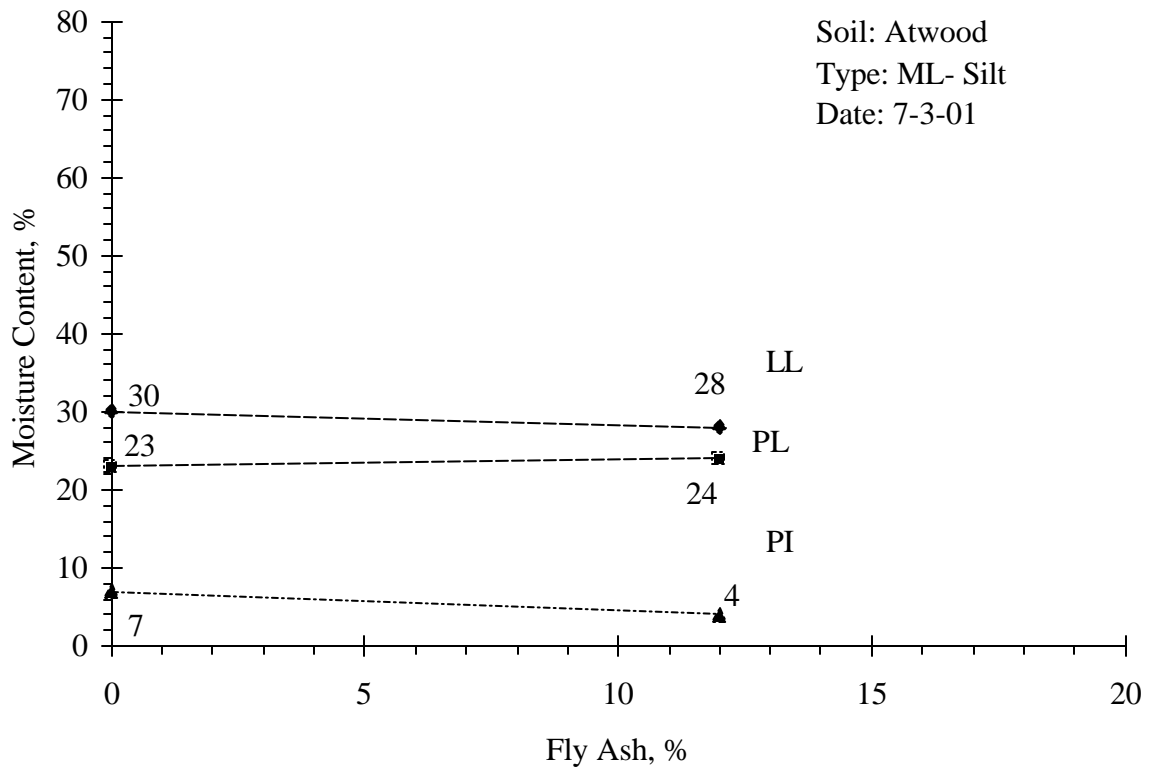






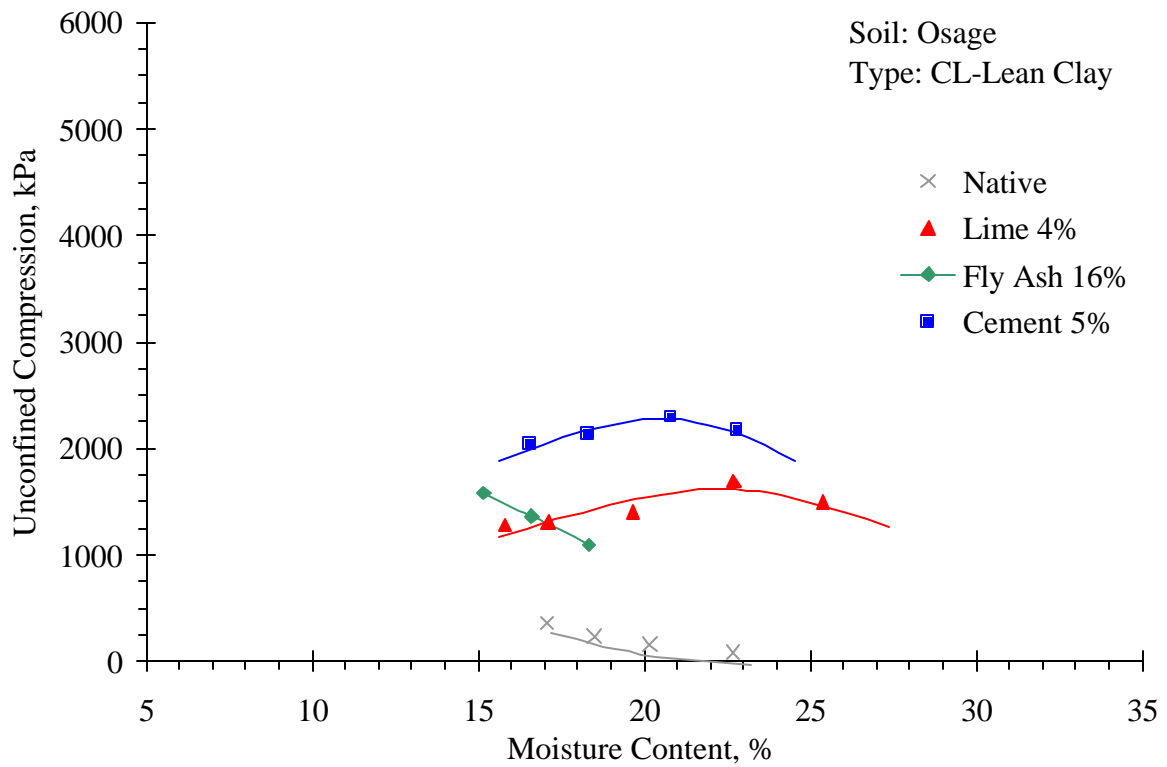
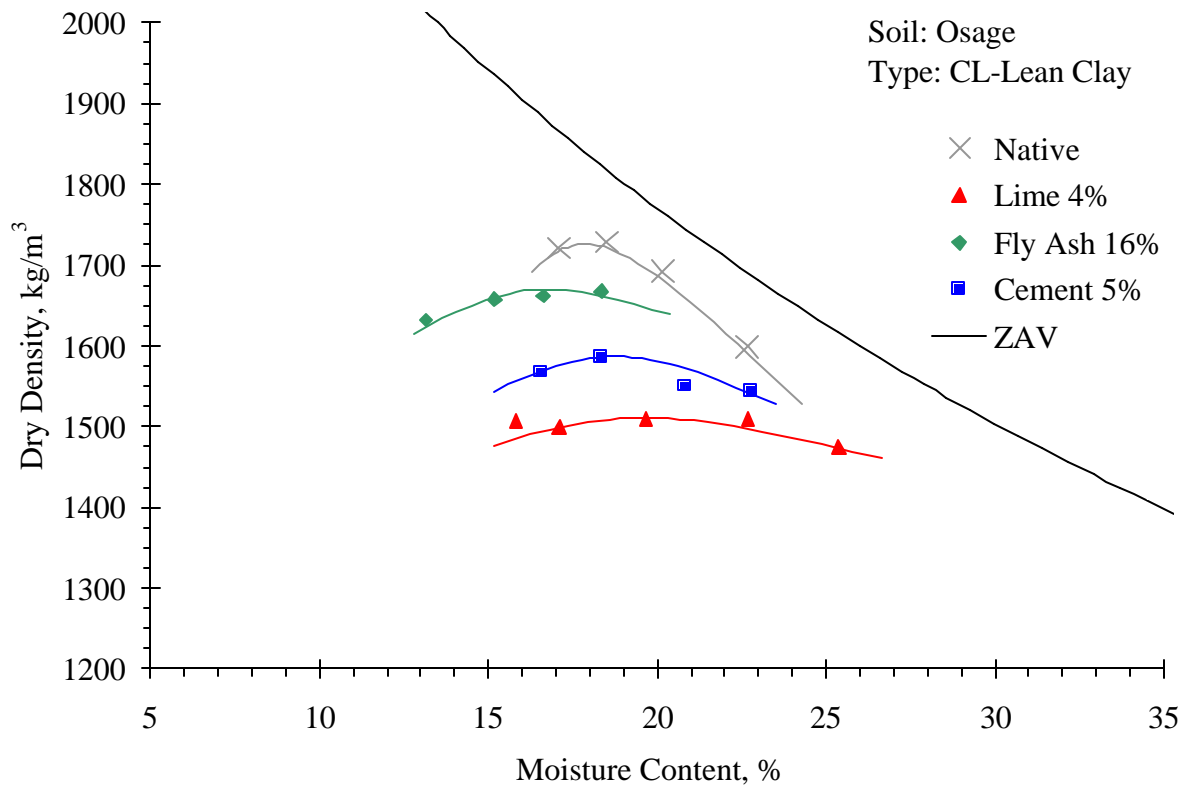


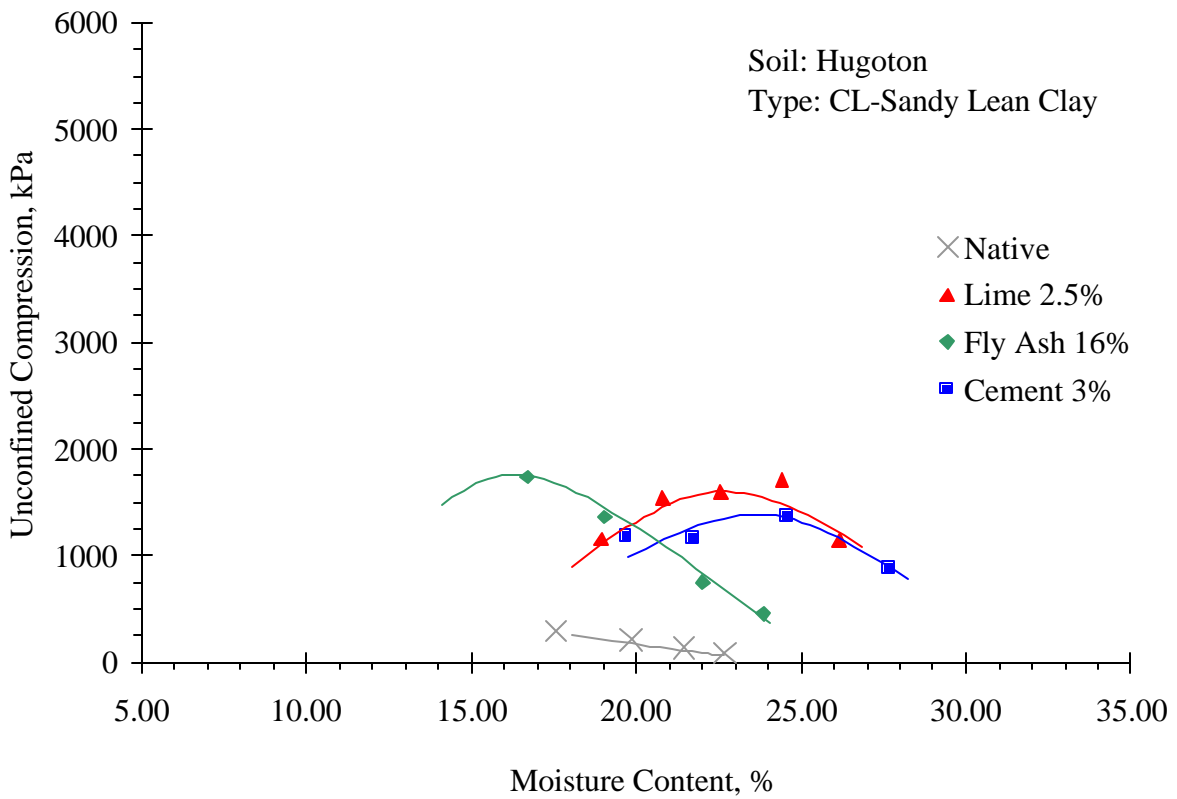
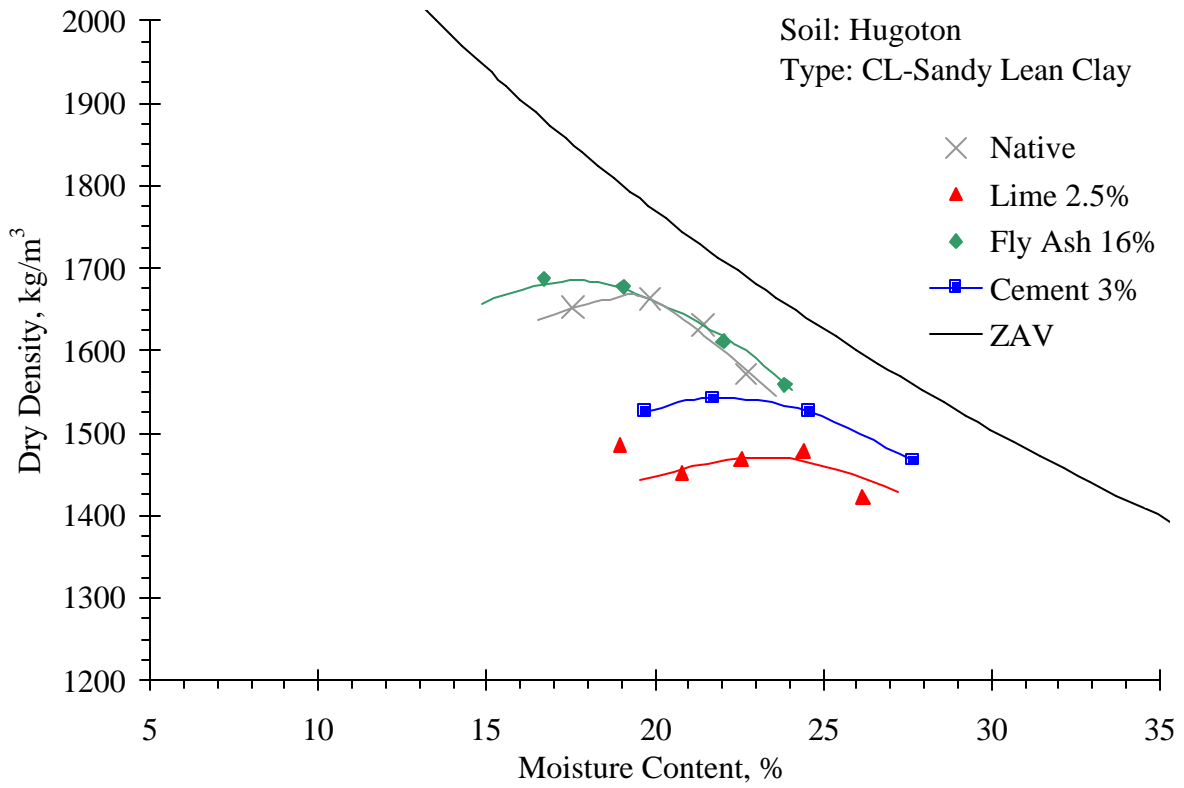
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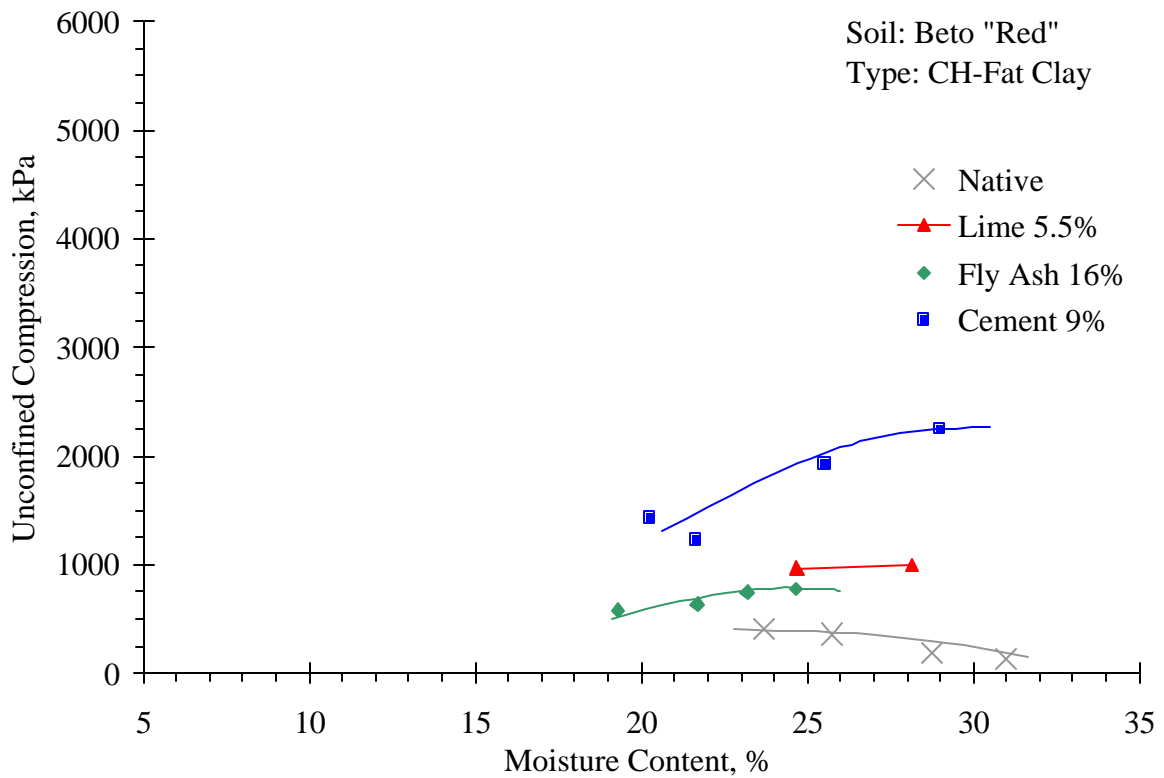
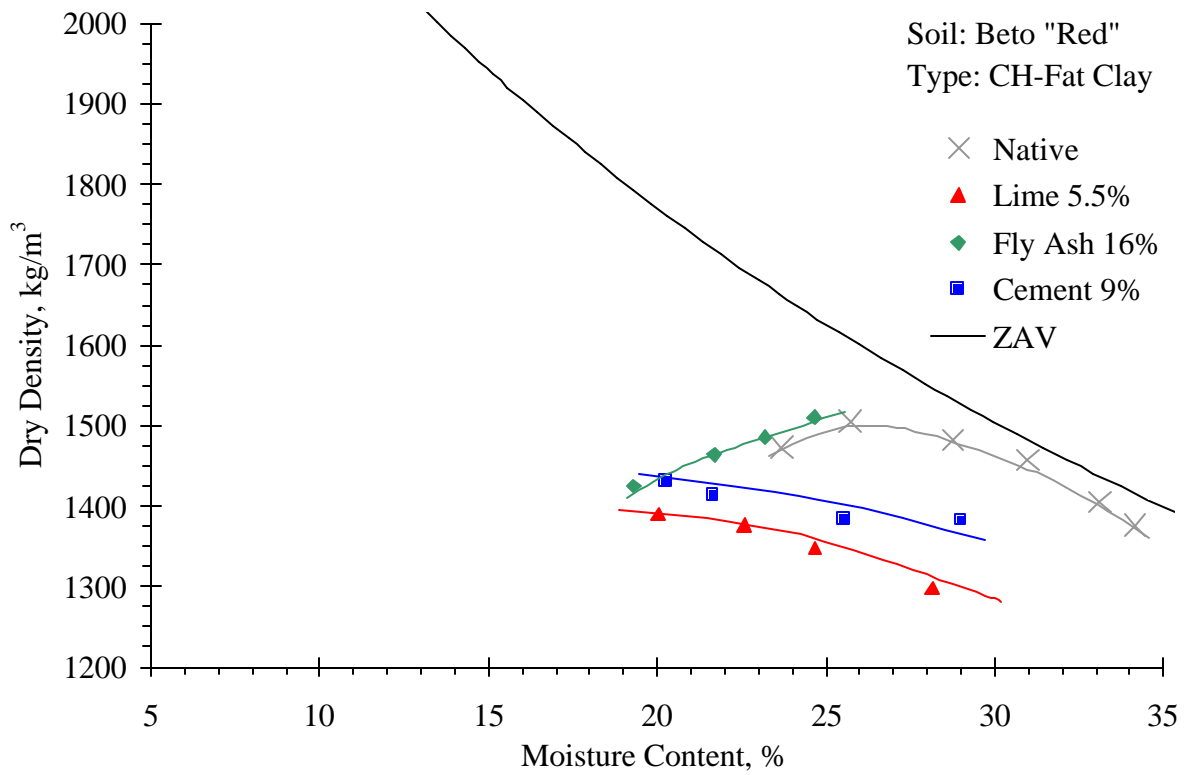


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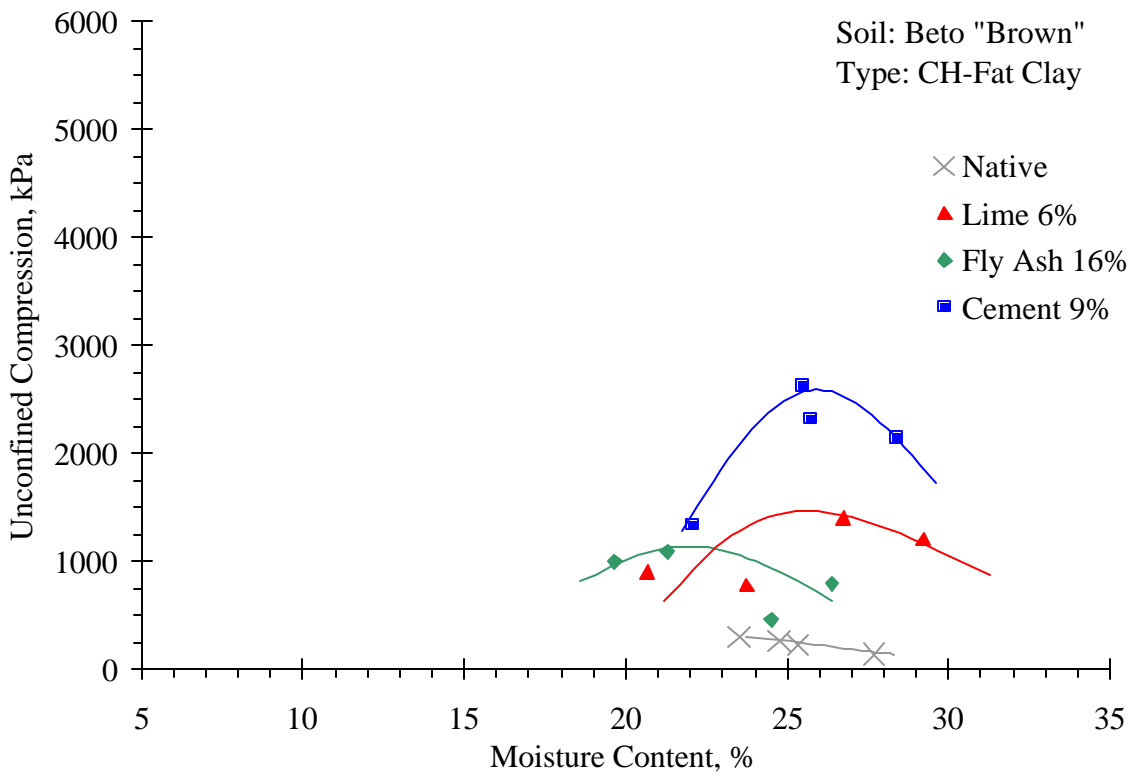
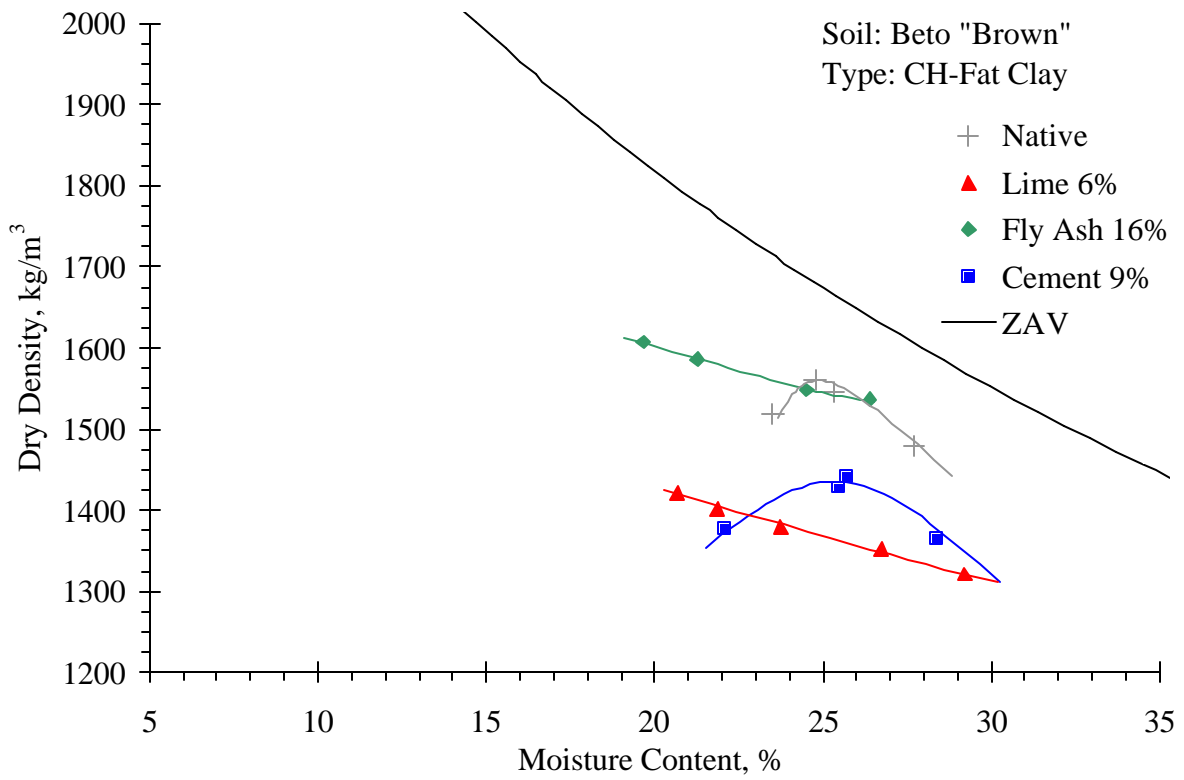
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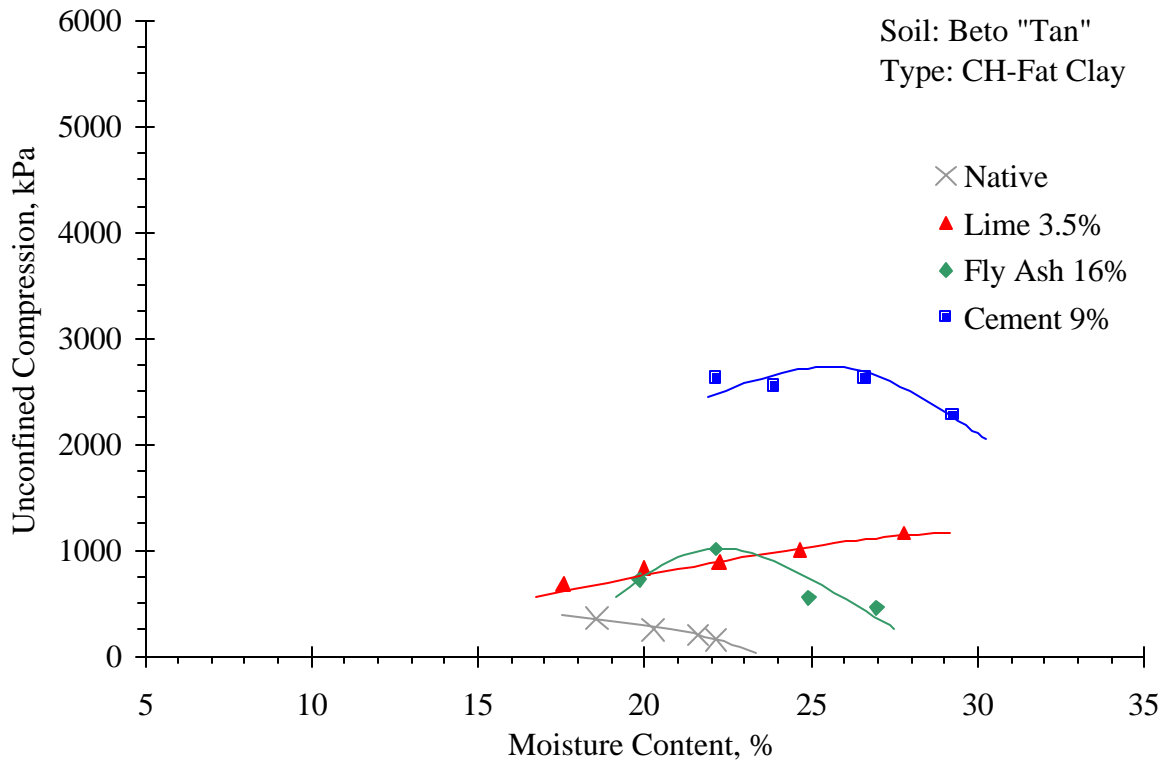
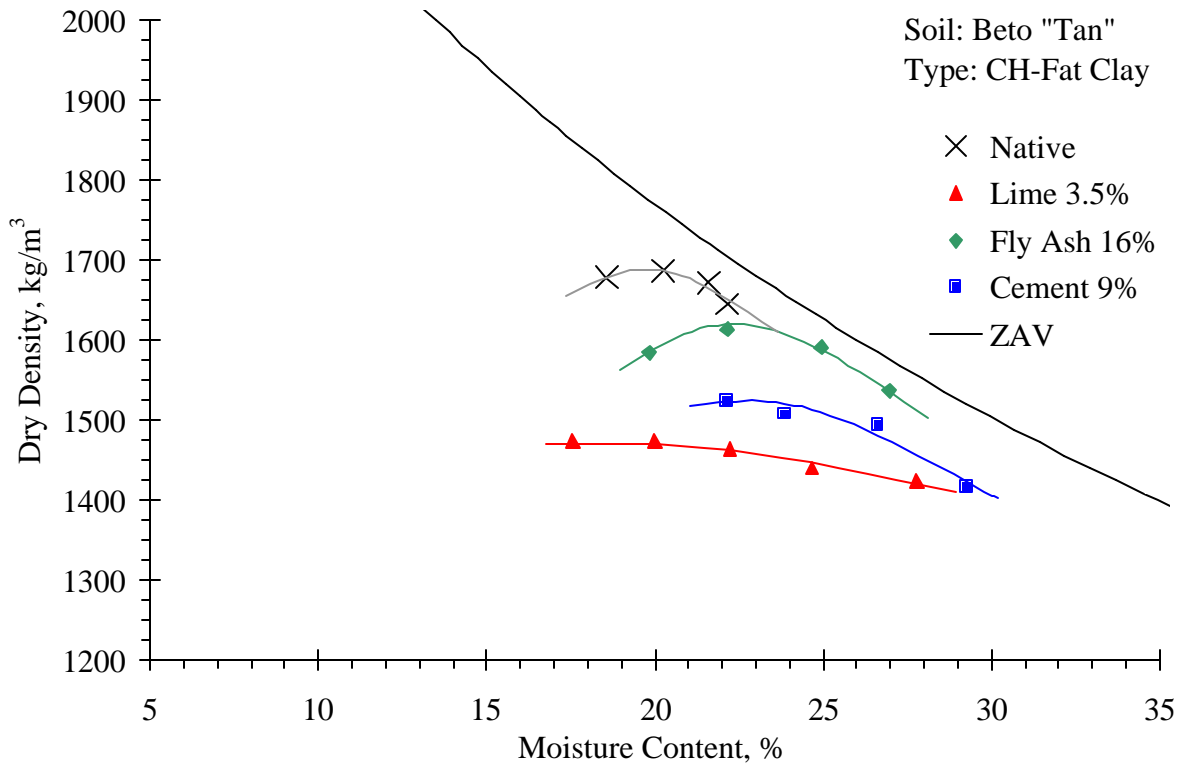


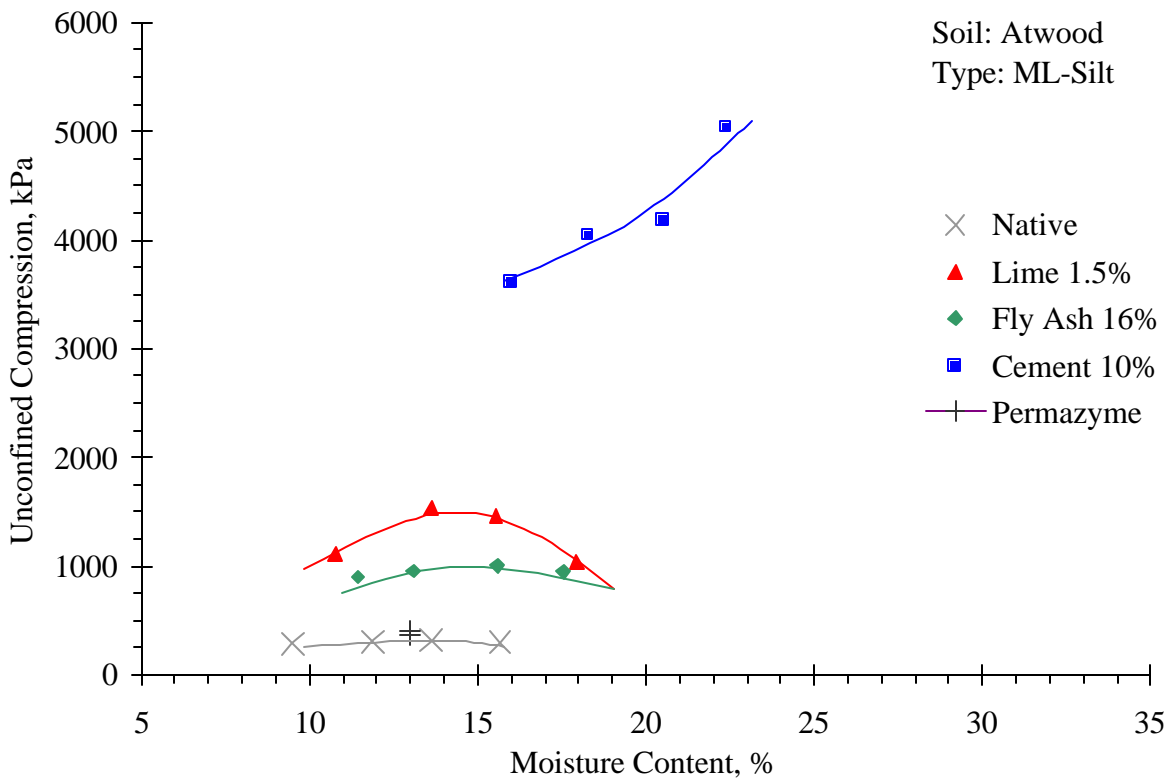
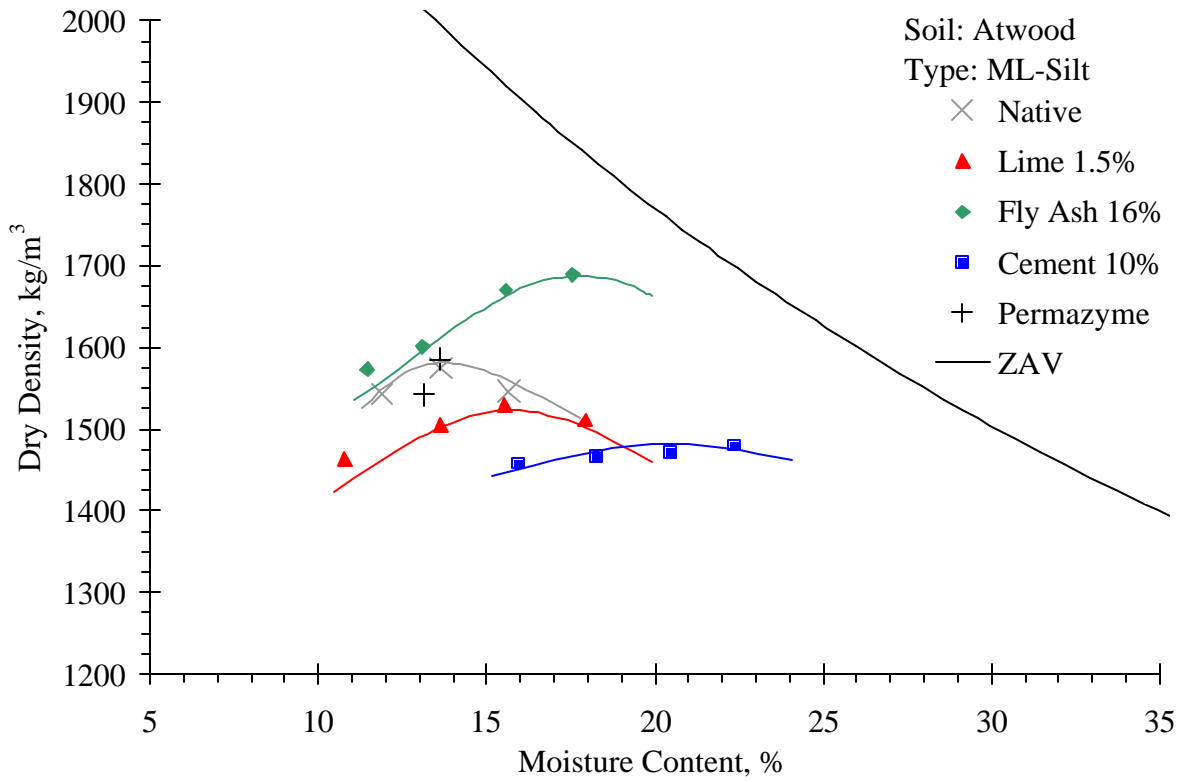


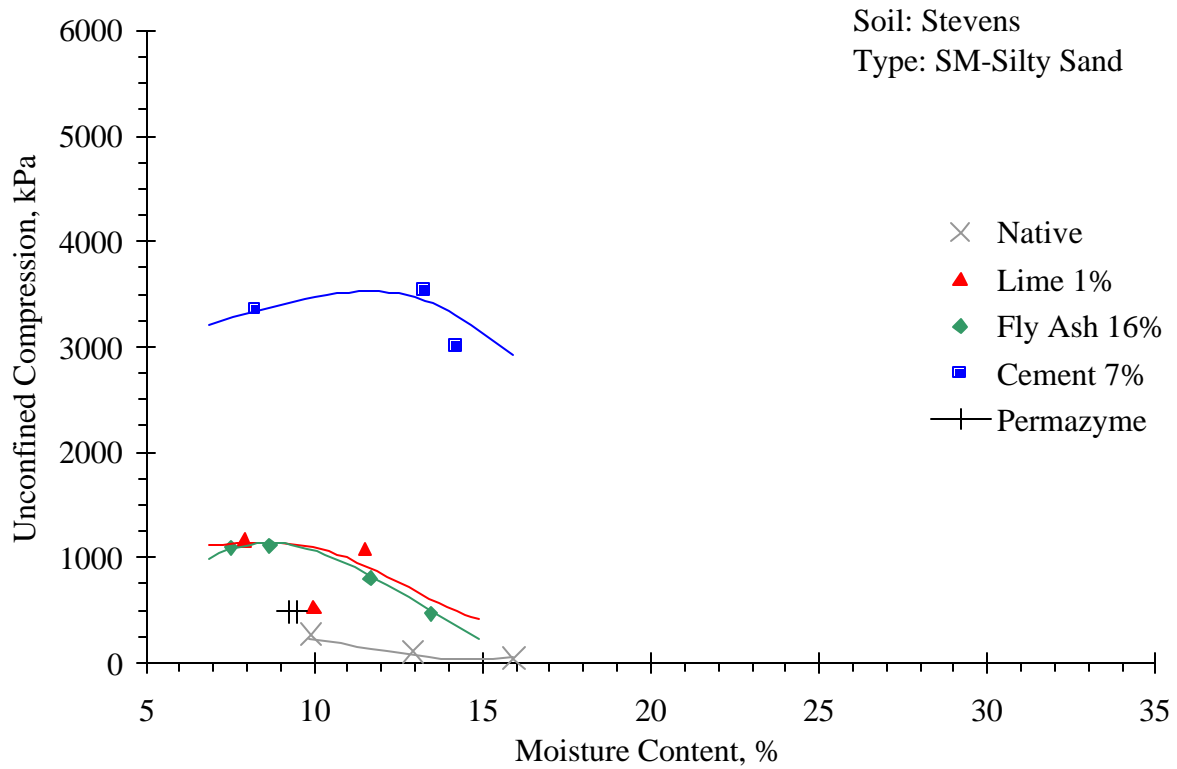
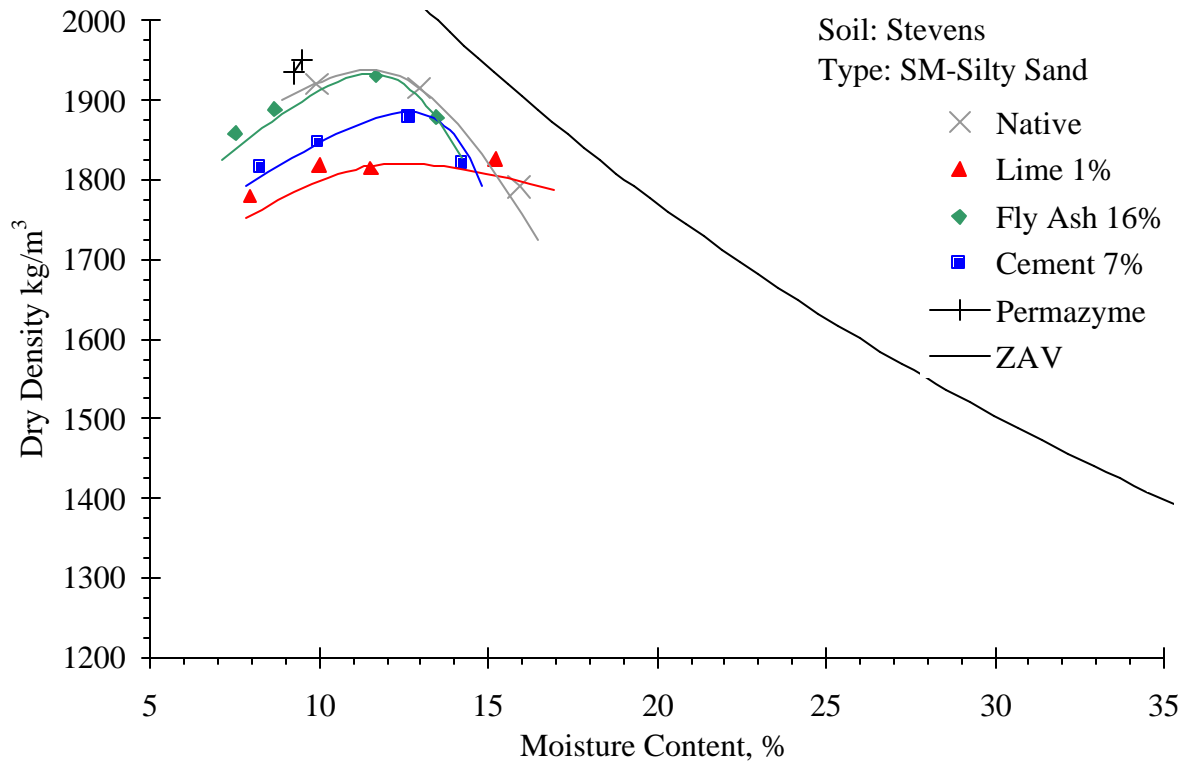


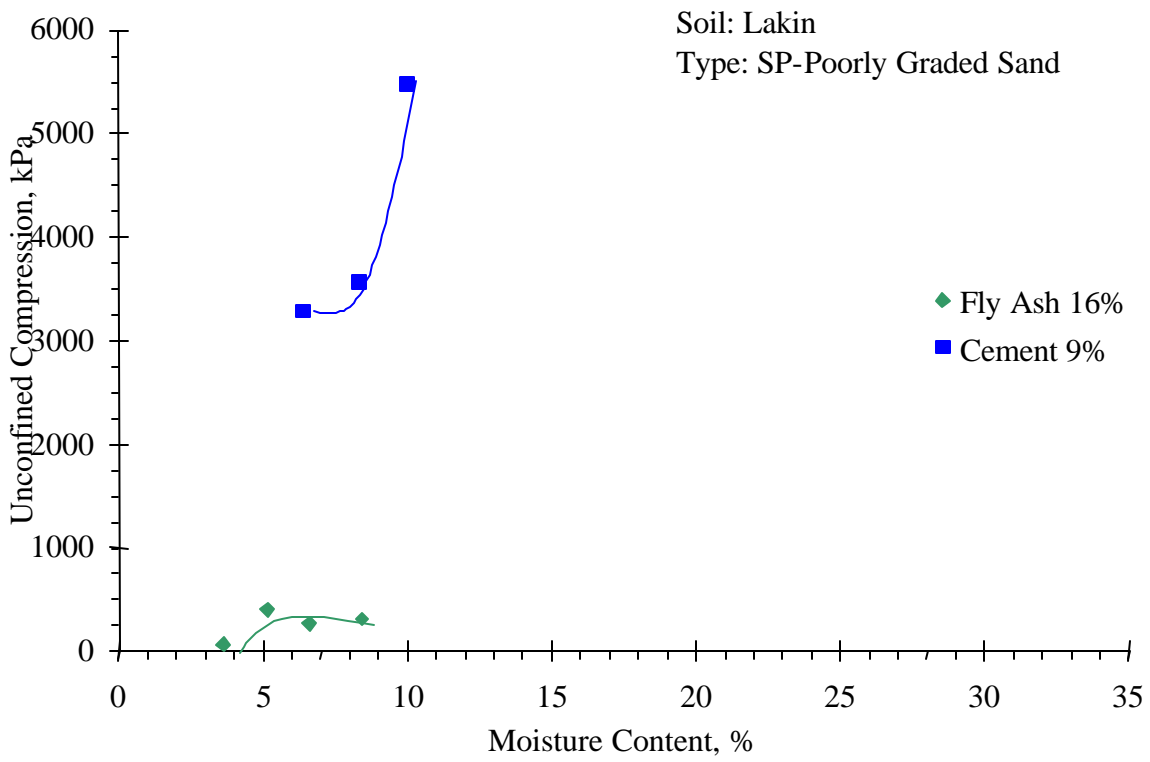
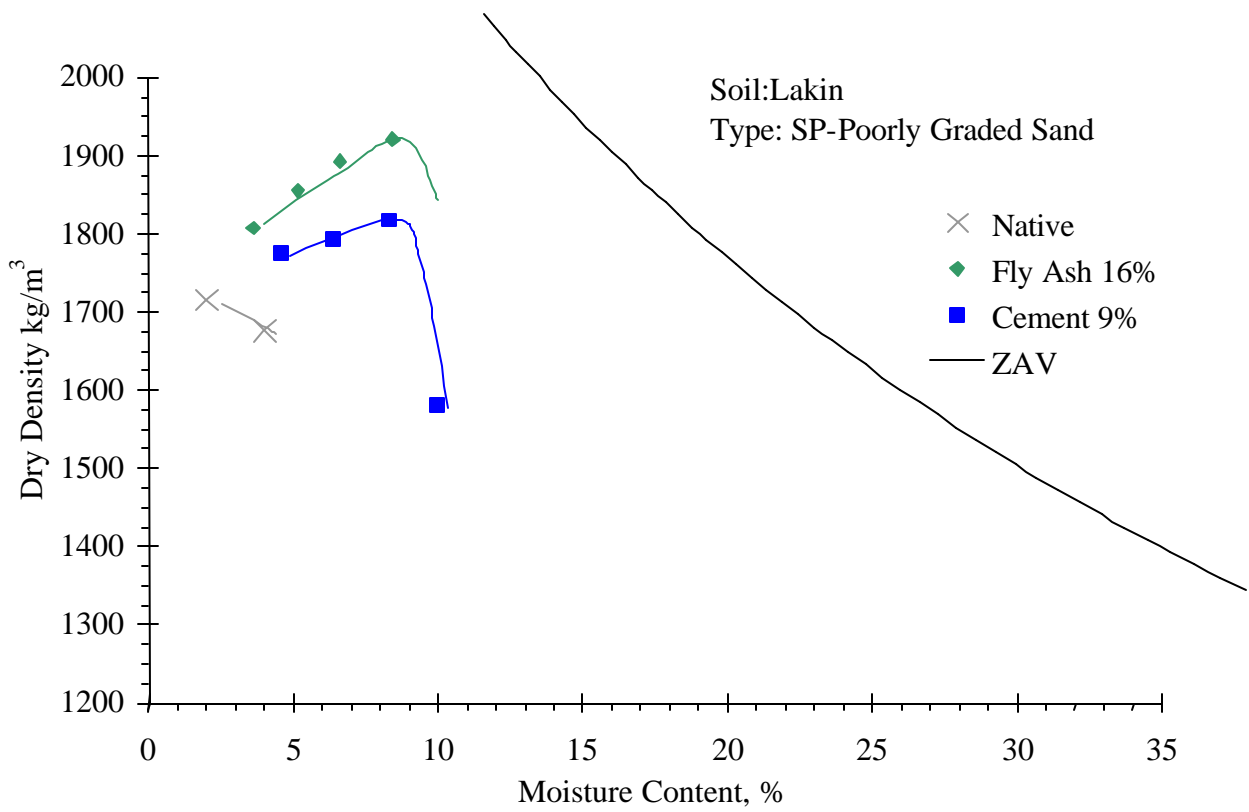




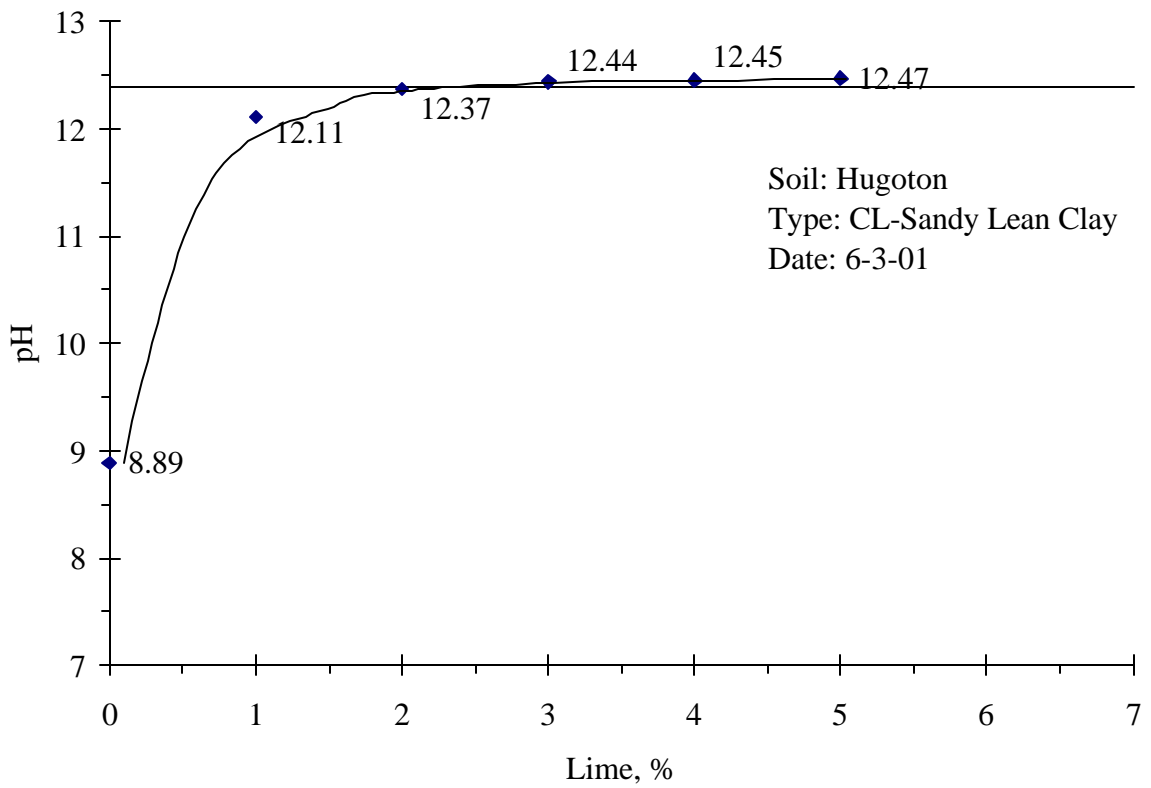
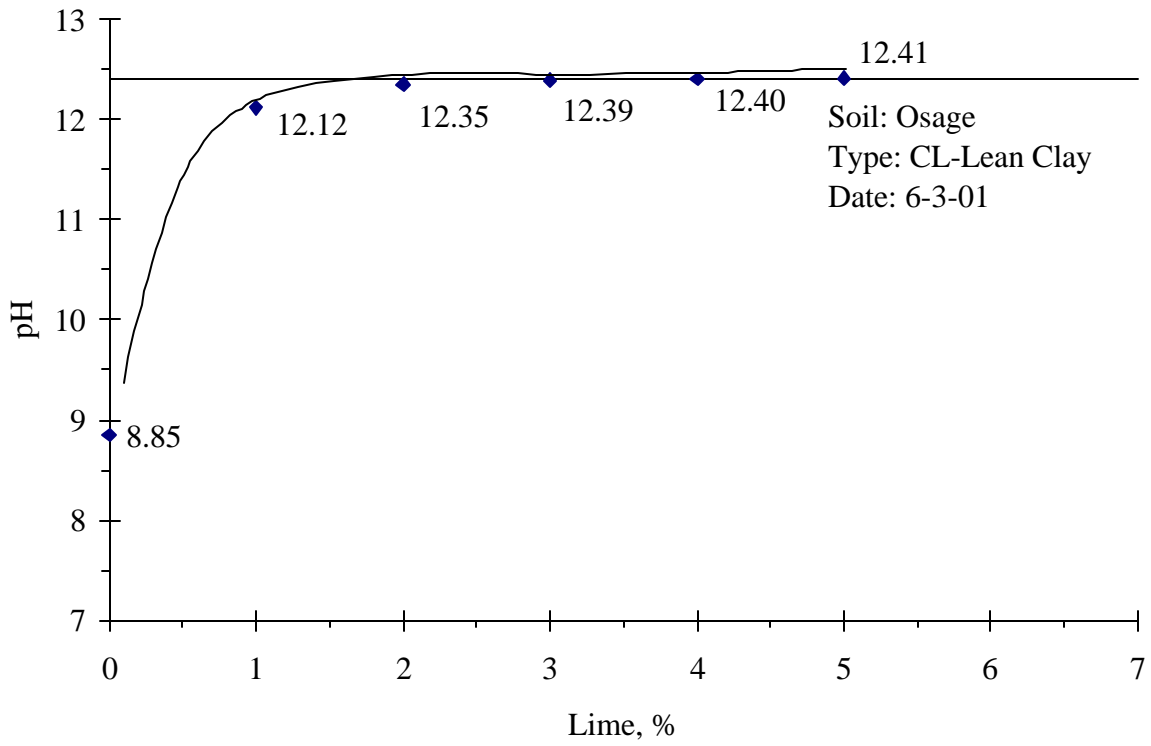


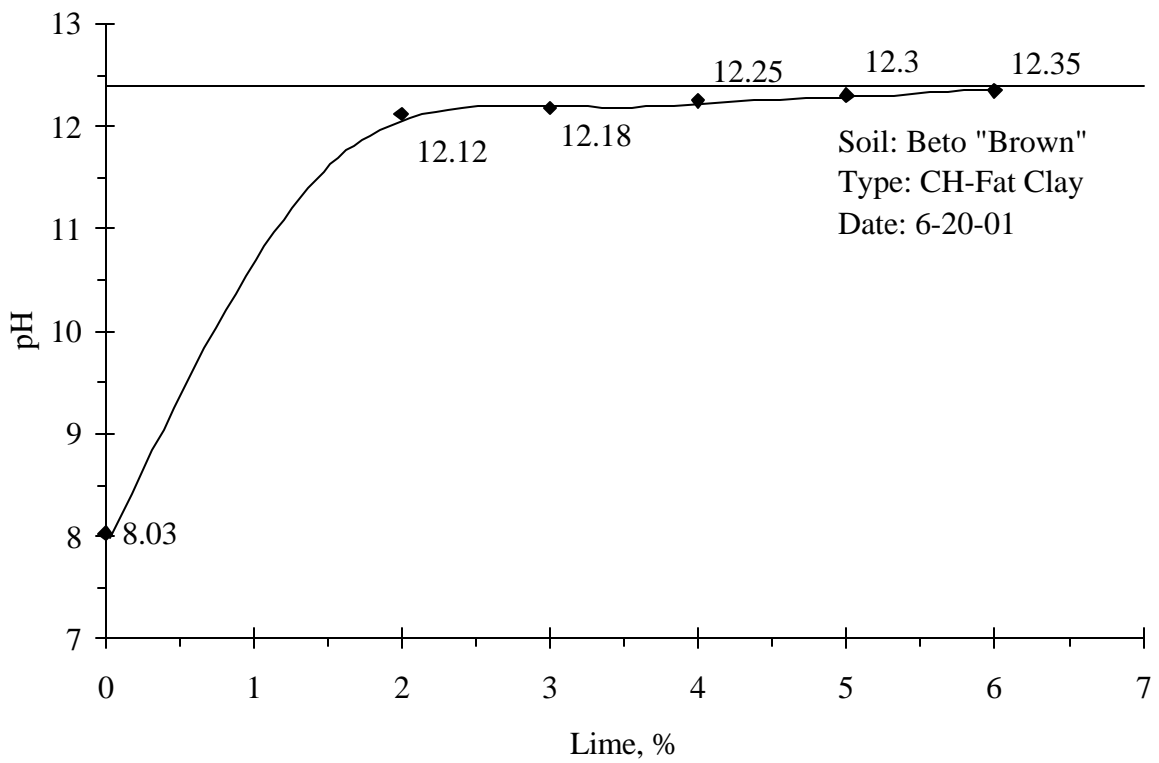
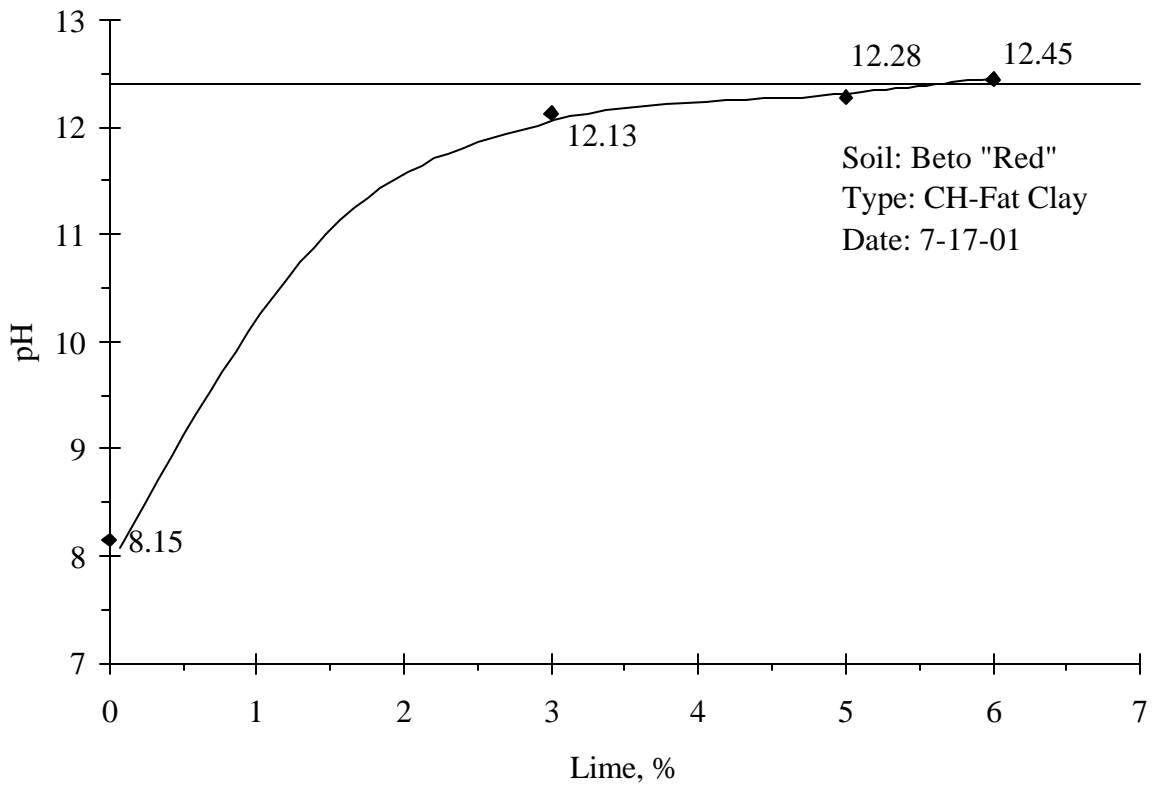




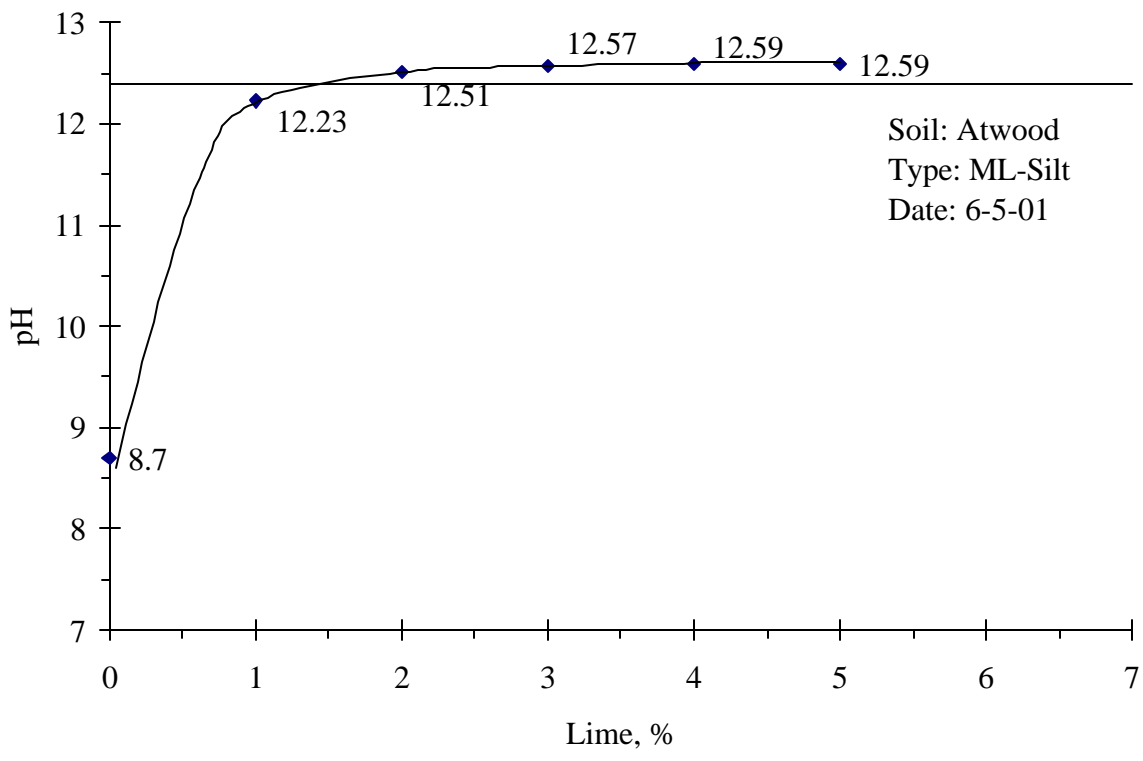
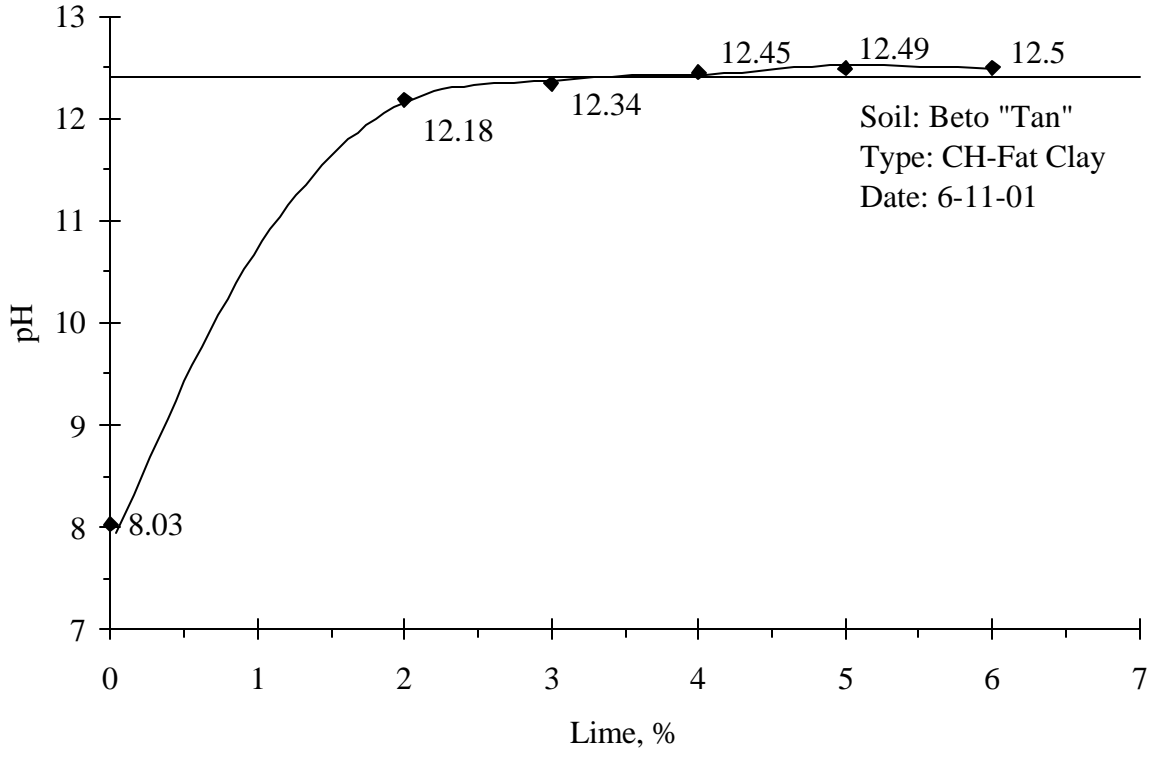


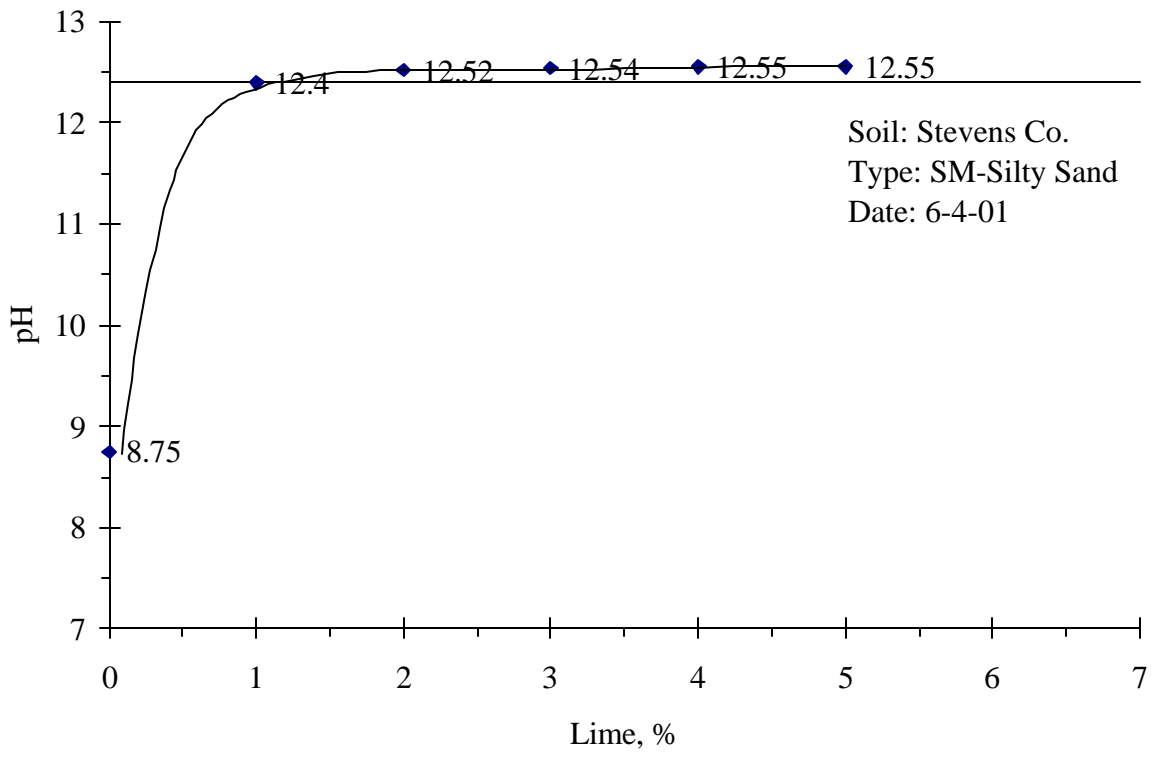
**APPENDIX C**  
**ASTM D 6276 Data**











## **APPENDIX D**

### **Stiffness vs. Moisture Content Curves**

