Effects of Calcium-Rich Additives on the Small-Strain Modulus of Representative Subgrade Soils in Missouri

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by
TAYLER J. DAY, BSCE, EIT

Dr. Brent L. Rosenblad, Thesis Supervisor

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The undersigned, appointed by the dean of the Graduate School, have examined the thesis entitled

EFFECTS OF CALCIUM-RICH ADDITIVES ON THE SMALL-STRAIN MODULUS OF REPRESENTATIVE SUBGRADE SOILS IN MISSOURI

Presented by Tayler J. Day,

A candidate for the degree of Masters of Science in Civil Engineering,

And hereby certify that, in their opinion, it is worthy of acceptance.

Professor Brent L. Rosenblad

Professor Keith W. Goyne

Professor R. David Hammer

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EFFECTS OF CALCIUM RICH ADDITIVES ON THE SMALL-STRAIN MODULUS OF REPRESENTATIVE SUBGRADE SOILS IN MISSOURI

Tayler J. Day

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ABSTRACT

Calcium rich additives, such as fly ash and lime kiln dust, can improve the mechanical properties of subgrade soils, resulting in better performance and more economical pavement design. Use of calcium-rich additives in Missouri pavement subgrades has been uncommon, due in part to a lack of quantitative data on the benefits derived from stabilization, and the parameters influencing the effectiveness of the stabilization efforts. A need also exists to identify and assess non-destructive testing methods to evaluate the quality of stabilized soils soon after they are placed. Laboratory measurements of small-strain moduli were performed in this study to: (1) quantify the effectiveness of calcium-rich additive stabilization of representative subgrade soils in Missouri, and (2) assess the viability of using stress wave-based, non-destructive testing (NDT) methods for quality assessment of stabilized subgrades.

Two representative Missouri subgrade soils, a low plasticity soil (PI=15) and a high plasticity soil (PI=33), were used in this study. These soils were mixed with fly ash (10 percent, 15 percent, and 20 percent by weight) or lime kiln dust (4 percent and 8 percent by weight) and compacted over a range of water contents. Changes in small-strain modulus with time were measured over a period of three to seven days using the free-free resonant column (FFRC) testing method. The results from this study showed large and rapid increases in modulus for most soil-additive combinations. The three-day modulus of the low-plasticity soil more than doubled with the addition of fly ash and more than tripled with the addition of lime kiln dust. However, the results also demonstrated large variability in the effectiveness of additive stabilization. In particular, modulus values of the high plasticity soil increased with the addition of lime kiln dust but showed essentially no effect from fly ash stabilization. Possible reasons for this behavior were developed from a supplemental study of the physical and chemical properties of the soil using particle size analysis, X-ray diffraction, scanning electron microscopy, pH, and cation exchange capacity measurements. Small-strain modulus measurements were also used to evaluate the viability of using stress wave-based velocity measurements in the field for quality assessment. The results of the laboratory measurements showed that the magnitude of very short term (within one hour after compaction) velocity changes of stabilized soil would be detectable using small-strain field measurements (such as seismic surface wave methods). Therefore, wave-based velocity measurements appear to be a viable method for assessing the quality of subgrade stabilization shortly after placement.

1 Introduction

1.1 Overview

The addition of calcium rich additives, such as fly ash and lime kiln dust, to soils can affect the mechanical properties (stiffness and strength) of the soil. One application of soil stabilization within engineering practice is in the construction of pavement systems. Pavement performance is greatly affected by the stiffness of the base and subgrade materials (soils). Subgrades with low stiffness values may cause excessive pavement deflections, which ultimately decrease the life of the pavement system. Increased subgrade stiffness produces a longer pavement life, and allows more economical pavement design through use of thinner pavement layers.

Many state transportation agencies use calcium-rich additives for ground improvement projects related to pavement construction to save costs and prolong pavement life. The additives have been used to create working platforms through soil drying in soft soils, and to increase the stiffness and strength of pavement subgrade materials. The Missouri Department of Transportation (MoDOT) currently does not use additives for stabilization due in part to a lack of empirical data supporting their effectiveness in Missouri soils. A need exists for an analysis of how calcium-rich additives can be applied to maximize their effectiveness in Missouri soils.

An additional issue associated with the use of calcium-rich additives is the measurement of the quality of the soil/additive mixture after placement and compaction. Traditional methods for quality control of compacted subgrades (water content and density) are insufficient because of the many factors related to additive stabilization of soils (Bergeson and Mahrt, 2000). Evaluating alternative quality control methods that can be applied in the laboratory to determine the most effective us of calcium-rich additives and also in the field to assess the quality of the placed additive/soil mixture is important. Meeting these two current needs will provide the information and tools needed for the state of Missouri to effectively apply calcium-rich soil, thus improving the performances of Missouri pavement systems.

1.2 Objectives

This thesis presents findings from two related, but distinct studies of soil stabilization using fly ash and lime kiln dust. The primary work presented in this thesis was performed as part of a collaborative project between Missouri University of Science & Technology (MST) and the University of Missouri (MU). The two project objectives were to:

1. Quantify the effectiveness of calcium-rich additives for the stabilization of subgrade soils for Missouri pavements, and;
2. Evaluate innovative non-destructive testing (NDT) methods that can be applied in the laboratory and the field to assess the quality of stabilized soils.

The primary objective of the work performed at MU was to evaluate the potential for small-strain velocity measurements to be used as a quality control method for soils stabilized with calcium-rich additives. It was hypothesized that small-strain velocity measurements performed shortly after compaction could be used to differentiate high-quality (properly mixed and hydrated with correct percentage of additive) from low-quality stabilized soils. To test this hypothesis, a series of laboratory tests of small-strain velocity was performed on two soils with differing properties while varying several experimental parameters including compaction water content, additive type, percentage of additive, and time after compaction. The data from these measurements also contributed to the objective of quantifying the effectiveness of calcium-rich additives for the stabilization of subgrade soils.

The second study resulted from observations of the small-strain velocity measurements which showed almost no effect on the stiffness properties of one of the soils when mixed with fly ash. It was hypothesized that differences in physical and chemical properties of the soil influenced the effectiveness of additive stabilization. To test this hypothesis, a series of laboratory tests was performed, including X-ray diffraction, cation exchange capacities, pH, and scanning electron microscope imaging. The results of these measurements contributed to better understanding soil parameters that can influence soil stabilization.

1.3 Organization

The thesis contains results from two distinct studies that will be discussed separately before conclusions are drawn from the collective results. The literature review in Chapter 2 discusses information pertinent to the primary small-strain velocity measurement study. The materials and methods used in both studies are described in Chapter 3. Results of the primary study of small-strain modulus are presented and discussed in Chapter 4. The secondary results are presented in Chapter 5. Chapter 6 contains the conclusions and recommended future work.

2 Background and Review of Previous Studies

2.1 Introduction

This chapter contains background on pavement subgrades, and the use of calcium-rich additives for stabilization of soils. Traditional subgrade preparation and typical quality control testing methods are discussed, followed by background on soil stabilization using calcium-rich additives. The mechanisms responsible for stabilization using fly ash and lime kiln dust are discussed. Previous studies on stabilized soils that are relevant to this research are presented and discussed. Past studies of non-destructive testing methods on subgrades and stabilized subgrades are also presented.

2.2 Traditional Subgrade Construction and Testing

Pavement systems typically contain three to four main components in a cross section, as illustrated in Figure 2.1. The wearing surface is the only part of the system that directly contacts the tires of passing traffic. Two types of wearing surface exist; flexible (asphalt) and rigid (concrete). Beneath the wearing surface is a base layer of aggregate to facilitate drainage and provide stability. Flexible pavements often include another layer of aggregate, called the subbase, which typically is not present in rigid pavements. This study focuses on the pavement subgrade, which is the material beneath the base layers. The subgrade is soil that is naturally present, or is placed as fill to meet grading requirements. The stresses imposed by vehicles on the pavement surface are carried through the pavement profile into the subgrade soils. Excessive cyclic strains will result in poor performance and shortened pavement life if the pavement system does not have sufficient stiffness.

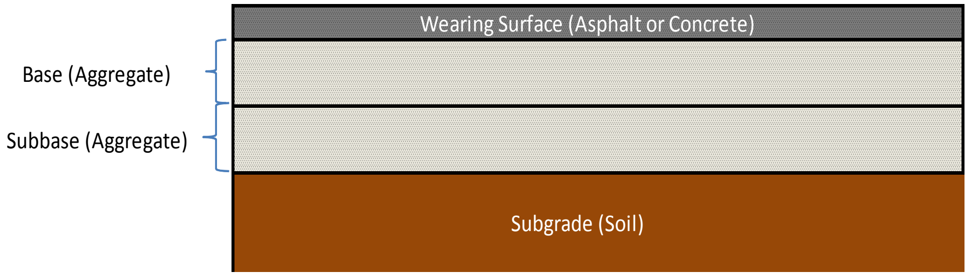


Figure 2.1 Schematic of typical pavement cross section

2.2.1 Design and Construction Techniques

Subgrade materials are characterized using traditional geotechnical parameters such as Atterberg limits, unconfined compressive strength (UCS), and compaction behavior when preparing for a pavement construction project. Soil placement can be optimized by understanding the soil-specific relationships among compaction water content, dry density, and compaction energy. The standard Proctor test (ASTM D698) is used to develop these relationships. Figure 2.2 shows the typical parabolic shape of the moisture density curve, and the peak corresponding to the maximum dry density at the optimum water content.

The Proctor test results determined in the laboratory are used to identify target water contents and densities to be achieved in the field. During construction, soil is placed in six to eight inch lifts and is compacted using field compaction equipment designed to achieve similar energy and behavior as the Proctor device used in the laboratory. After each lift has been compacted, tests are performed to compare the *in situ* water content and density to the design values. Typically, the range for dry density used in Missouri is specified by the Missouri Department of Transportation (MoDOT) as no less than 90% maximum dry density, while the water content depends on the project and soil (MoDOT Item P-152). A range of -2 to +2 percent of optimum moisture content is also suggested for compacted soils by MoDOT.

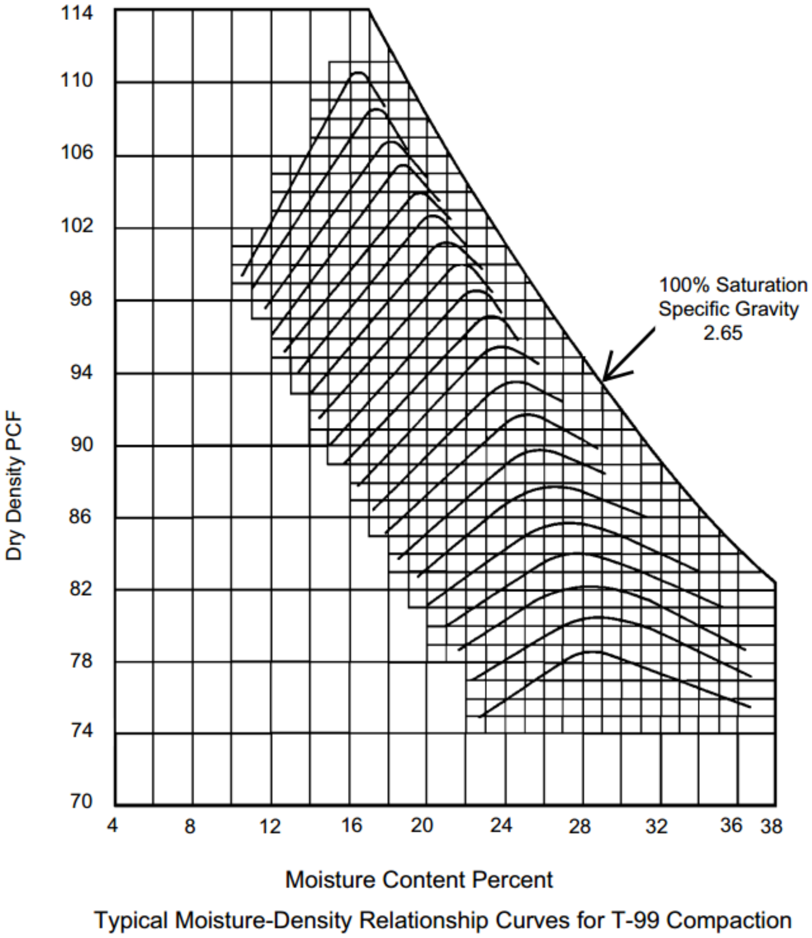


Figure 2.2 Example Proctor curves from AASHTO (2001)

Typical *in situ* measurements of water content are performed using the nuclear density gauge (ASTM, D6938), which can also be used for measurement of dry density. A more accurate method for determining density is the rubber balloon method (ASTM, D2167-08); however, the associated water content measurement from this method is more time-consuming, which may lead to delays in construction.

2.3 Non-Destructive Testing of Pavement Subgrades

Evaluation of pavement base and subgrades materials has been a focus of non-destructive testing researchers for many years. Non-destructively testing these materials to evaluate properties, while avoiding sample and site disturbance, is the preferred approach. As noted previously, soil stiffness (or modulus) is an important parameter affecting pavement performance. Several common methods provide measures of pavement stiffness. These range from empirical correlation methods to more direct measurements of pavement modulus, as discussed below.

2.3.1 Penetration Measurements

Penetration methods are commonly used field methods to assess subgrade quality. Although not truly non-destructive, these penetration methods leave most of the material intact and are largely non-intrusive. The California Bearing Ratio (CBR) test has been used extensively since before WWII when the California Department of Transportation developed the method that was standardized in ASTM D1883 for both laboratory and field use. The laboratory procedures require that soil is compacted into a 6 inch mold and a plate is placed on top of the sample, leaving a circular hole exposed. Weight can be added to the plate to prevent heaving of the peripheral soil during loading. A two-inch diameter piston is then placed in the hole, and the sample is displaced by a motor (on automated variations) toward the cylinder, allowing the cylinder to penetrate the sample while the load caused by the cylinder is measured. The loads and corresponding stresses required to produce penetrations of 0.1 and 0.2 inch are divided by 1000 and 1500 psi respectively before multiplying by 100 to calculate the CBR. The values of 1000 and

1500 psi are the stresses required to penetrate a sample of crushed California limestone (ASTM D1883). The laboratory CBR setup is illustrated in Figure 2.3.

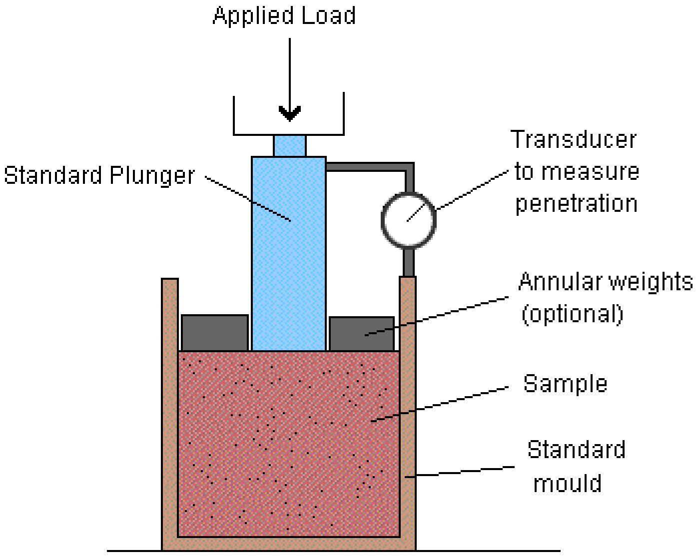


Figure 2.3 Schematic of laboratory California Bearing Ratio (CBR) test. Retrieved from http://www.testcreteconcretetesting.co.uk/construction-tests-services/cbr-test-california-bearing-ratios-tests/

The loading apparatus can also be mounted to a truck, but the compaction information (water content and density) is more difficult to measure and control compared to the cylindrical mold method. The field version of this test specifies that the plunger is aligned with the ground and the weight is added to the top. The rate of penetration is measured as the plunger enters the compacted or native subgrade. The CBR can be used as an index property of subgrade soils to provide an empirical sense of the strength for pavement systems.

Another common penetration method that has been extensively used for pavement projects is the dynamic cone penetrometer (DCP), as described in ASTM D7380. A cone-shaped tip is attached to a rod that has an anvil and a sliding weight attached to it. The weight is dropped from a predetermined height, striking the soil and causing the cone to penetrate. The penetration into the soil is measured to depths up to three feet using measurement lines cast onto the rod or a measuring stick placed on the ground surface, as illustrated in Figure 2.4. The DCP is simpler to perform in the field than the CBR measurement because of its light weight and portability; however, the local nature of the penetration can influence the results in a non-uniform stratum (Chen et al., 1999). Results of the DCP have been correlated to deflection based methods such as falling-weight deflectometer (FWD) (Chen et al., 1999), but the result is still not a value that is used directly in mechanistic design of pavements. The most glaring issue with DCP results from the penetration rate being correlated to CBR, and then CBR correlated to resilient modulus. This double correlation increases the scatter of the data (Chen et al., 1999) and can lead to unreliable results.

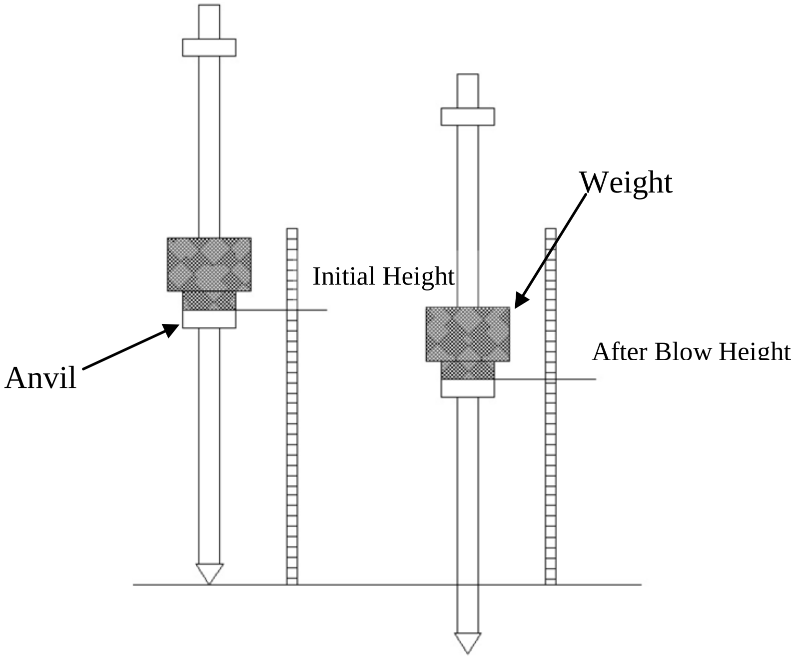


Figure 2.4 Schematic of dynamic cone penetrometer (DCP)

2.3.2 Modulus Back-calculation from Deflection Measurements

Another common approach for determining subgrade stiffness is to perform deflection measurements at the surface and then back-calculate modulus values for the underlying material. The most common non-destructive testing method of this kind in use for pavements is the falling weight deflectometer (FWD) (Chen et al., 1999). In this test, a weight is dropped from a height determined by the operator, and the load and deflection are measured by sensors placed on the ground at a predetermined spacing (ASTM, D4694-09), as shown in Figure 2.5.

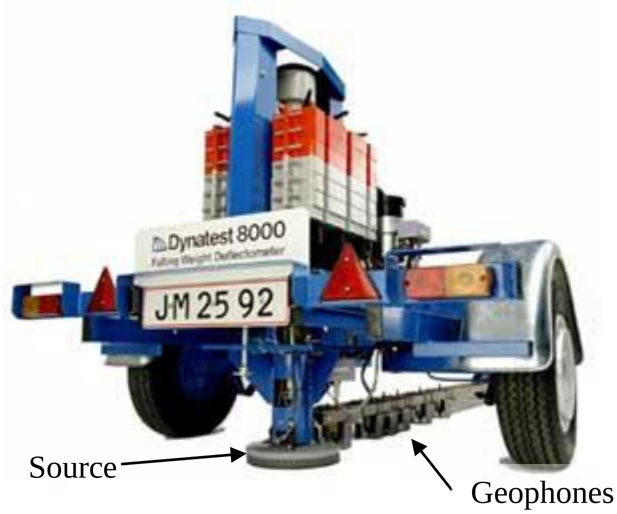


Figure 2.5 Typical Falling Weight Deflectometer (FWD) trailer-mounted setup from http://www.dynatest.com/structural-hwd-fwd.php

The measured deflection profile can be used to back-calculate pavement modulus values. Many simplifying assumptions are used in the back calculation (for example, dynamic effects are not accounted for) that can result in error (Chen et al., 1999). Proper calibration is essential for meaningful results when using FWD; however, researchers have found that even when properly calibrated, FWD models manufactured by different companies produce different results (Bentsen et al., 1989; van Gurp, 1992). Research has also indicated that the use of different shaped buffer pads can affect the pulse shape and the repeatability of the measurement, even if using the same setup (Lukanen, 1992).

The GeoGauge (Humbolt, 2010) is another device that uses deflection-based back calculation to determine the moduli of pavement systems. The portable machine (shown in Figure 2.6) measures deflection and force in a "foot" that is driven by a shaker that operates from 100 to 196 Hz in 4 Hz increments (Alshibli et al., 2005). The deflection and force are converted to stiffness and Young's modulus using an assumed Poisson’s Ratio (Humbolt, 2010). The device works well for shallow elastic modulus analysis of unreinforced compacted soils, but has an influence depth restriction of eight-to-nine inches (Alshibli et al., 2005), excluding it from being used on thicker layers or systems with multiple layers.

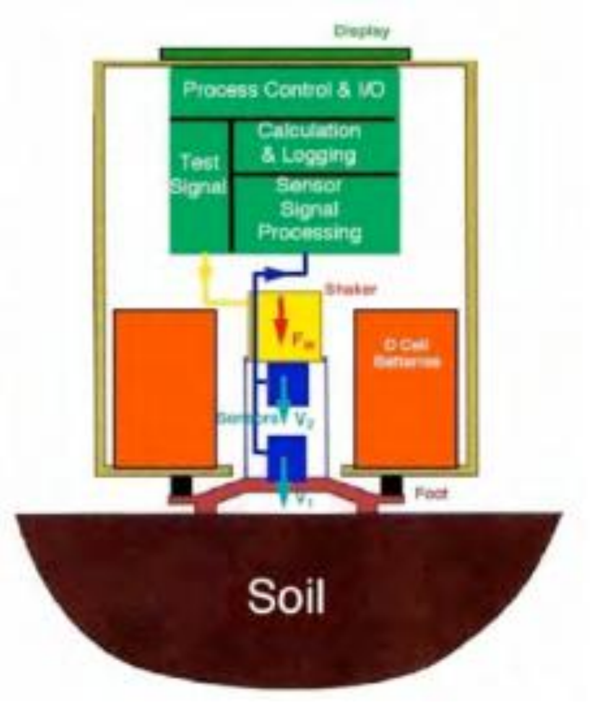


Figure 2.6 Schematic of GeoGauge system (from Humbolt, 2007)

2.3.3 Wave-based Nondestructive Testing Methods

Unlike other NDT methods, wave-based methods can be related directly through theory to the mechanistic modulus of subgrade soils. Wave-based methods allow the small-strain modulus to be calculated very quickly from velocity measurements performed in the laboratory or the field. Three velocities (shear, unconstrained compression and constrained compression) can be measured with this method using different propagation and recording techniques that will be discussed in later chapters. The general relationship between velocity and modulus is:

$$M = \rho \times V^2 \qquad (2.1)$$

where M is the modulus, ρ is the density of the material, and V is the velocity measured in the material.

One common wave-based field measurement is the Spectral-Analysis-of-Surface-Waves (SASW) method. Introduced to geotechnical engineering in the early 1980's by Nazarian et al. (1983), the SASW method uses a seismic source on the ground surface to propagate a wave which is received by geophones placed on the surface at a predetermined spacing, as shown in Figure 2.7. The spacing allows researchers to target waves of a certain wavelength and corresponding depth of influence. The small-strain elastic moduli calculated using this method are mechanics-based values that can be quantified and compared between materials.

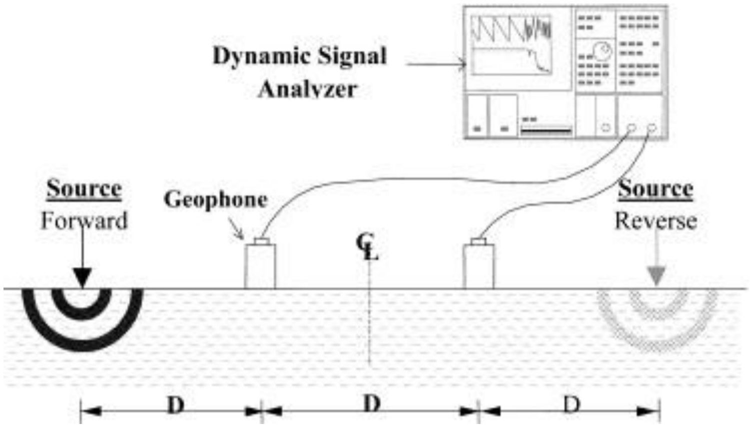


Figure 2.7 Schematic of Spectral-Analysis-of-Surface-Waves (SASW) testing. From Kim et al. (2001)

The use of SASW on pavement and subgrades is effective because, instead of a correlation from a device specific modulus, the method uses measured wave velocities and mass density to directly obtain moduli. Researchers can measure different zones of the pavement system by using specific receiver spacings, as demonstrated by Nazarian et al. (1983)

Resonance methods have been developed in the laboratory to measure wave velocities over the same small-strain range as the SASW method. The free-free resonant column (FFRC) method described by Stokoe et al. (1994), is a simple and quick method to measure the velocities of laboratory specimens. Samples are prepared by compaction to a specified water content and density to mimic field conditions. End caps and a membrane are attached to the compacted specimen to allow connection to a vacuum system. The application of vacuum pressure allows the test to be performed under small confining pressures. The specimen is suspended in a frame that isolates it from other materials. A source propagates a wave through the sample, which is detected by accelerometers on the opposite end, as shown in Figure 2.8

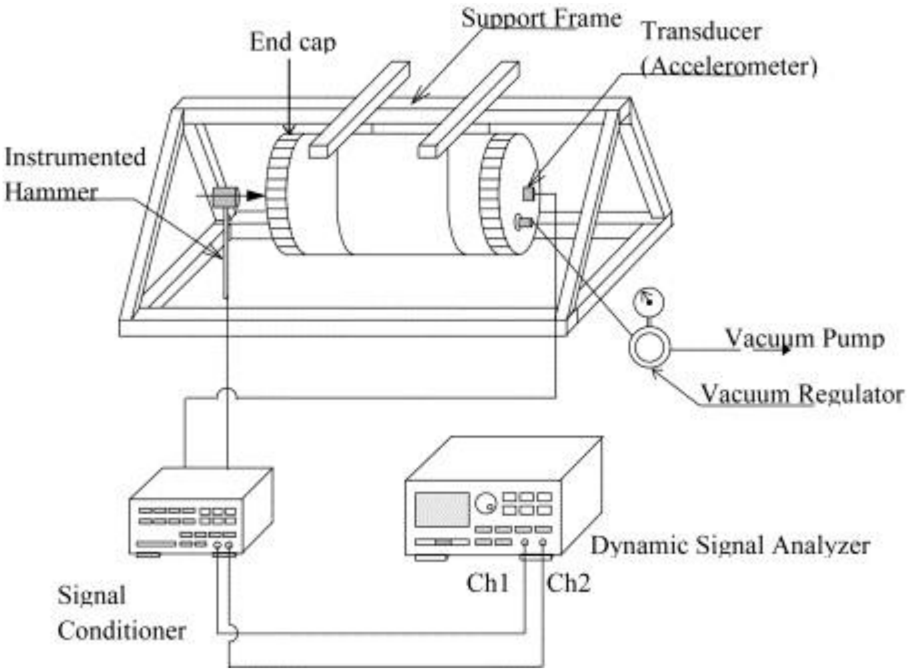


Figure 2.8 Typical Free-Free Resonant column setup. From Kim et al. (2001)

The use of the SASW method is particularly of interest to this project because of its direct relation to the Free-Free Resonant Column (FFRC) laboratory test. Both tests are performed at the same small-strains, so laboratory and field values can be directly compared.

2.4 Overview of Soil Stabilization Using Fly Ash

When coal is burned, multiple products, including fly ash, are created throughout the process, as shown in the diagram presented in Figure 2.9. Fly ash is lighter than bottom ash and consequently rises before being collected by an electrostatic precipitator. Fly ash acts as a soil stabilizer by improving the engineering properties of the soil through chemical and mechanical means. These mechanisms will be more thoroughly discussed in the upcoming section.

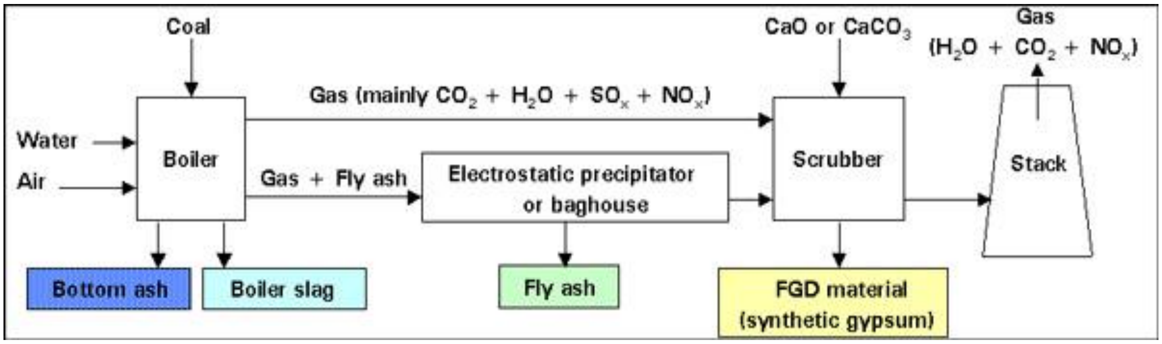


Figure 2.9 Flow diagram of the flue-gas-desulfurization process based on lime (CaO) or limestone(CaCO3), which are the sorbents used by 90 percent of FGD systems in the United States (from Kalyoncu and Olson (2001))

Fly ash has been used as a soil stabilization additive for decades all over the world (White et al., 2005). The production of coal combustion products in the United States topped 130 million tons in 2010 with more than 67 million tons in the form of fly ash, according to the American Coal Ash Association (ACAA, 2010). Only 27 million tons of the 67 million tons produced in 2010 were utilized, while the remaining tonnage was disposed of in landfills and retention ponds. It was reported by the ACAA (2010) that 785,552 tons of fly ash were used for soil stabilization projects in 2010.

2.4.1 Types of Fly Ash

Three types of fly ash are classified in ASTM C618-12: Class F, Class N, and Class C. The fly ashes are classified by an analysis of their chemical composition, with limits described in Table 2.1.

Table 2.1 Compositional requirements for fly ash classification from ASTM C618-12

	Class		
	N	F	C
SiO_2+Al_2O_3+Fe_2O_3, min (%)	70.0	70.0	50.0
SO_3, max (%)	4.0	5.0	5.0
Moisure Content, max (%)	3.0	3.0	3.0
Loss on Ignition, max (%)	10.0	6.0	6.0

Class C fly ashes are those that characteristically contain SiO_2+ Al_2O_3+Fe_2O_3 contents greater than 50%, but less than 70%. These fly ashes result from the burning of lignite and subbituminous coals (ASTM C618). Class F fly ash typically is produced from burning anthracite or bituminous coal, but may also be produced from subbituminous coal and from lignite that contain more than 70% of SiO_2+ Al_2O_3+$Fe_2O_{3.}$ Class C fly ashes typically have total calcium contents, expressed as calcium oxide (CaO), that are higher than Class F fly ashes (ASTM, D2216). The higher quicklime (CaO) content, in addition to the presence of pozzolanic compounds, such as silicon and aluminum, in Class C fly ashes causes them to react in the presence of water, while Class F requires a calcium activator to initiate the pozzolanic reactions discussed in the next section (White et al., 2005).

Class N fly ash is composed of raw or calcined natural pozzolan such as some diatomaceous earths, opaline cherts, and shales; tuffs, volcanic ashes, and pumicites; and calcined clays and shales (ASTM C618-12). Class N and Class F fly ashes are not discussed in this project.

2.4.2 Soil-Fly Ash Reactions

There are two short-term and one long-term chemical reactions that can occur between fly ash and soil resulting in stabilization (White et al., 2005). First, the soil is dehydrated via reactions of water with CaO in the fly ash, making the additive useful for soil drying applications such as construction of working platforms. Additional drying results from heat released during the initial reactions. Second, Ca^{2+} ions released from the stabilizer are adsorpted onto soil particle surfaces through electrostatic attraction, which decreases the thickness of the diffuse double layer and enhances flocculation. These two reactions provide short term benefits, such as improved workability and a decrease in the plasticity index.

The long-term benefits of soil stabilization utilizing fly ash result from pozzolanic reactions. The introduction of fly ash increases the pH in the soil-water system. The presence of high pH, in turn, enhances dissolution of soil minerals and releases silicon and aluminum into the system. These elements, in addition to the silicon and aluminum from the additive, react with Ca^{2+} released from the additive in pozzolanic reactions. Calcium silicate hydrate (CSH) and calcium aluminate hydrates (CAH), formed from the pozzolanic reactions, act as cementing agents that enhance soil stability by promoting aggregation of the clay particles and filling the void spaces to improve the mechanical properties of the soil (Little and Nair, 2009).

2.4.3 Design and Construction Procedures using Fly Ash

MoDOT (Item MO-155) specifies that, in Missouri, the engineer is responsible for determining the amount of fly ash to be mixed with the subgrade soil to achieve the desired result. Unconfined Compression Strength (UCS) testing typically is used to determine the ideal percentage of additive. Strength is not truly the engineering parameter of interest for pavement performance, but is often used as an indication for other parameters. A review of the literature finds the percentage of fly ash added to soil is typically in the range of 10 to 25 percent by weight (Manz, 1985; Misra, 1998; Parsons and Milburn, 2003; Bin-Shafique et al., 2004).

Fly ash typically is mixed in the field using a pulvamixer with a spray bar to apply the specified amount of water. The procedures outlined by MoDOT specify that the water used is to be potable or tested to make sure there are no organics, acids, or alkalis that would interrupt or affect the reactions between the additive and the soil (MoDOT, Item MO-155). The directive specifies construction time from mixing to compacting to occur within 2 hours of placement, and that the fly ash shall not be allowed to sit for longer than 30 minutes before mixing. The final compacted layer is covered with a layer of moist earth and left for no less than three days or until placement of pavement (MoDOT, Item MO-155).

2.5 Overview of Soil Stabilization Using Lime Kiln Dust

Lime kiln dust is a byproduct of the production of quicklime. Limestone is heated and crushed in a kiln, which creates quicklime and calcium hydroxide. The particulates released during this process are lime kiln dust, which is collected and utilized in many construction applications. This dust differs from “lime” or “quicklime”, which is also used in soil stabilization, because it contains quicklime and the aluminosilicates needed for pozzolanic reactions, similar to fly ash. Typically lime kiln dusts contain much higher amounts of quicklime than fly ashes, resulting in lower typical mixture ratios required to reach similar levels of stabilization (Little and Nair, 2009).

2.5.1 Soil-Lime Kiln Dust Reactions

Two reactions occur when water is introduced to a lime-soil mixture; pozzolanic interactions and carbonation. The pozzolanic reaction is similar to that which occurs with fly ash, in which the increased pH causes silicon and aluminum to release into solution where it can bond with calcium ions to form cementious materials (Diamond and Kinter, 1965; Eades and Grim, 1966; Little, 1996). Carbonation results from the lime reacting with carbon dioxide to form carbonate cement. This reaction can be harmful to soil stabilization reactions if it happens too soon because it decreases the availability of quicklime for pozzolanic reactions, thus decreasing the workability of the material (Little, 1996).

Pozzolanic materials are present in the lime kiln dust along with quicklime, which means that reactions will occur when water is added, even before the soil pH is increased sufficiently to encourage dissolution of aluminosilicates (Little, 1996). This behavior is similar to fly ash, but the amount of quicklime present generally is enough to satisfy the pozzolanic reactions, before and after the pH increase. This means that the pozzolanic reactions occur earlier, and sustain longer in soils mixed with lime kiln dust, compared with regular lime.

2.5.2 Design and Construction Procedures using Lime Kiln Dust

An ASTM standard exists for estimating the amount of lime necessary to stabilize soil (ASTM, D6276-99a). The test involves measuring the pH of different mixtures of lime with soil until the mixture no longer has a pH equal to or higher than the 12.4 pH quantity deemed conducive to reaction. However, this method is to be used cautiously with lime kiln dust, because the increased alkalinity of the material does not require as high of a pH for reactions to occur.

Another method used to estimate the amount of lime required involves preparing mixtures with different quantities of additive, and testing the unconfined compression strength, as is commonly done for fly ash stabilization. Specimens created for compaction curves would then be subjected to UCS testing to determine the optimum mixture with regards to potential strength gain in stabilized subgrade soils.

The United States Army Corps of Engineers uses a method that is based on the chart shown in Figure 2.10 (Daita et al., 2005). The chart-based method uses the plasticity index of the soil to determine the amount of hydrated lime to apply. The method develops a starting point for the amount of additive required, and can be used, in addition to the unconfined compressive strength method, for selecting the optimum mixture percentage required for the soil to reach a target stabilized strength.

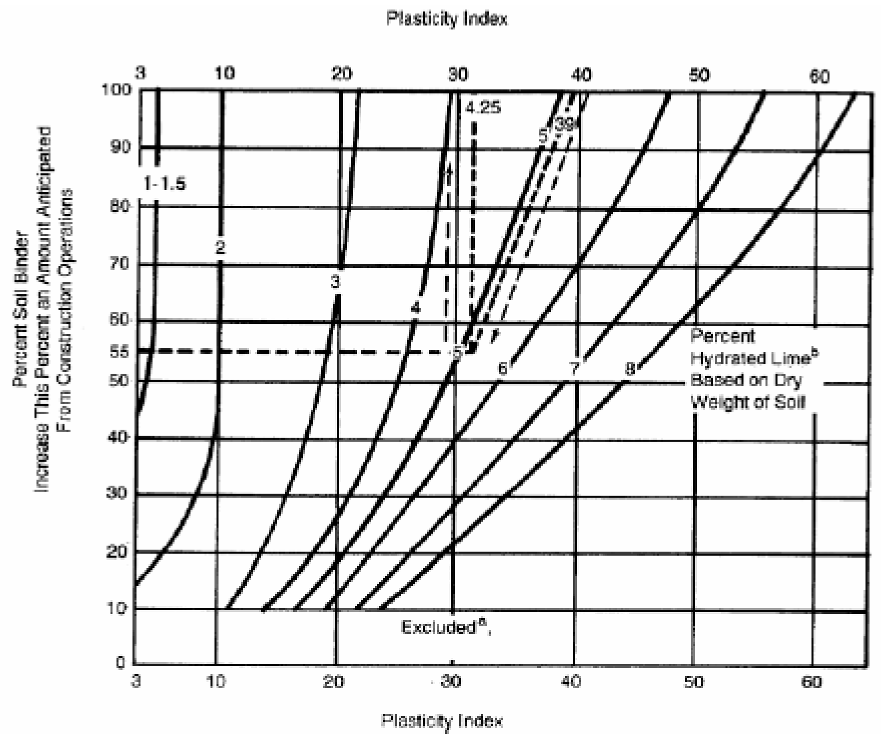


Figure 2.10 U.S. Army Corps of Engineers Procedure for Preliminary Determination of Amount of Lime (As presented in Daita et al. (2005))

Procedures used in the construction of lime and lime kiln dust stabilized subgrades are similar to those for fly ash. The lime products are mixed with the soil using a pulvamixer while water is applied through a spray bar. The main difference is lime-soil mixtures are typically allowed to mellow for one to seven days before being remixed and compacted (NLA, 2004); however, this method should not be used for lime kiln dust mixtures, because the presence of aluminosilicates will cause the reactions to occur more rapidly than traditional lime.

2.6 Previous Lab and Field Studies of Stabilized Subgrade Soils

Previous studies of additive stabilization of soils are too numerous to present. The studies mentioned below relate to the main objectives of this research, and focus on studies published over the last decades from states near Missouri, as well as studies on the application of non-destructive testing methods of stabilized and unstabilized subgrades.

2.6.1 Fly Ash Soil Stabilization

Past studies indicate that methods for evaluating the effects of fly ash stabilization of soils have included California Bearing Ratio (CBR), Resilient Modulus (Mr), Falling Weight Deflectometer (FWD), and Unconfined Compressive Strength (UCS). Mixtures of soils and fly ash have displayed measurable increases in CBR, moduli, and UCS values in Missouri (Misra, 1998) and nearby states such as Wisconsin (Bin-Shafique et al., 2004; Trzebiatowski et al., 2004), Kansas (Parsons and Milburn, 2003), and Iowa (White et al., 2005)

Misra (1998) found an increase of 289% in the UCS of laboratory-mixed clays stabilized with 20% fly ash from Labadie, MO, which is the same fly ash used for this study. The laboratory clays were prepared using calcium saturated kaolinite with 0, 2, 4, and 6 percent bentonite by weight added to provide controlled differences in soil behavior. Mixtures of 10 and 20 percent fly ash by weight were tested to analyze the effects of additive stabilization on the various clays. The use of laboratory created clays minimized the experimental variables in order to better compare relative increases in strength. Unconfined compressive strengths of 425, 875, and 1230 kPa were measured for 0 percent, 10 percent and 20 percent fly ash, respectively for one laboratory clay mixture. The addition of fly ash to the clays decreased the optimum water content and increased the maximum dry density, with the effect increasing with increased amounts of fly ash. Soil-fly ash mixtures subjected to compaction delay after mixing yielded decreased maximum density values and increased optimum water content.

In order to test the long-term effects of stabilization, Misra (1998) compacted and cured some samples for as long as 112 days. The long-term UCS study demonstrated that the increase in soil strength was variable but generally increasing with time. The strength of the clays, prepared with 0 and 4% bentonite contents, decreased over the long-term, but the 2 and 6% bentonite mixture clays showed increased UCS. Long-term decrease in strength is thought to result from tension cracks forming within the specimen because of drying from fly ash reactions (Misra, 1998). Questions have arisen about the long term viability of reinforcement because of issues such as freeze-thaw durability and increased swell potential in some soils. Misra (1998) cautions that this must be considered when selecting a water content during construction. Soils compacted wet of optimum have increased ductility compared to dry soils and also mitigate potential tension cracking.

Results from UCS tests performed in Kansas were shown to increase 190 to 480% percent over non-reinforced soils (Parsons and Milburn, 2003). Soil mineralogy and index properties were determined to be important to the validity of any prediction of strength increase as soils with lower smectite contents showing greater strength increases than higher plasticity materials (Parsons and Milburn, 2003).

Trzebiatowski et al. (2004) performed measurements on a 3.7 km stretch of fly ash stabilized subgrade soils in Wisconsin, with Plasiticity Indices (PI) between 9 and 18 and clay fractions as high as 50 percent. The subgrade was mixed with 10 percent fly ash and compacted 1 percent wet of optimum. Subgrade samples were collected immediately after mixing to be tested in the laboratory. The field CBR testing showed a ten-fold increase over unstabilized soil with the most appreciable gain occurring within seven days of compaction. Resilient modulus measurements performed on field collected samples showed a 90 percent increase over unmixed soils, indicating a substantial increase in stiffness of the soil platform.

In Wisconsin, Bin-Shafique et al. (2004) analyzed two flexible pavement systems to determine if the benefits of fly ash stabilization could be used in subgrade design. The results showed that the system designed using the predicted stabilized strength performed just as well as the control section, indicating that good laboratory testing could be trusted in the design of fly ash stabilized pavements.

White et al. (2005) tested five Iowa soils with multiple fly ashes, and determined that the gain in strength for stabilized soils was time-dependent with substantial gain in the first 7-28 days of curing. Increased compactive effort on the soil-additive mixtures resulted in higher strengths for lower water contents and less substantial effects on higher water contents. Compaction delay effects were also investigated and the results indicated that the strength was negatively affected for the soil-additive mixtures.

2.6.2 Lime Kiln Dust Soil Stabilization

The use of lime to stabilize soil is extensive, but difficult to quantify due to the commonality of the use of the term “lime” to describe different additives. For the purpose of this thesis, the term “lime” will be used to describe quicklime (CaO). Lime kiln dust contains lime as well as alkali materials that result in pozzolanic cementing reactions.

The strength of lime reinforced soils has been shown to continue increasing for more than a decade (Chou, 1987). Researchers have used UCS and field tests, such as FWD, to monitor strength. The CBR, strength, and Young's modulus were found to increase substantially with curing time and temperature, showing an influence on the magnitude of the increase (Bell, 1996). Parsons and Milburn (2003) also found 140% to 480% increases in UCS for lime reinforced Kansas soils, compared to non-reinforced samples.

Lime kiln dust-modified soils have been monitored in the laboratory, and in the field, for pavement projects in states near Missouri. In Indiana, Jung et al. (2008) and Jung et al. (2010) studied five sites that had been stabilized with lime kiln dust and had been in operation for a minimum of five years. Dynamic cone penetrometer (DCP) and falling weight deflectometer (FWD) measurements were performed to determine the long-term effects of strength stabilization. The resilient modulus calculated from FWD tests showed an increase of 190 percent to 600 percent for the sites after five to ten years of operation, indicating long-term stabilization effects. California bearing ratio (CBR) values calculated from DCP results for the treated soil were increased up to 1700 percent. The depth of mixing was shown to vary from the intended depth, indicating a problem with construction techniques (Jung et al., 2008). The fines content of the same soils in a later study showed a decrease of 20-40%, and the classification of the soils changed from a silty clay to a non-plastic silty sand after stabilization (Jung et al., 2010).

Lime-reinforced clay soils displayed a decrease in Liquid Limit (LL) and an increase in Plastic Limit (PL) resulting in increased workability of the soils in a study by Boardman et al. (2001). Bell (1996) also found a decrease in plasticity that resulted from a reduction in the amount of montmorillonitic and in increase in kaolinitic and quartzic materials, as measured using X-ray diffraction,

Lime kiln dust stabilization was also used for a high profile midwest pavement project using subgrade stabilization for a new runway at O'Hare International Airport in Chicago (Brar et al., 2006). Researchers tested four groups of soils classified by their clay content. Results of CBR testing indicated that the addition of high-calcium lime kiln dust resulted in increased CBR for all soils, at all water contents; however, soils compacted wet of optimum showed less CBR increase than those dry of or at optimum moisture contents. High-calcium lime kiln dust was chosen as the additive to use in stabilizing the subgrade soils for the new runway.

2.7 Quality Control of Subgrade Soils Using Non-Destructive Methods

Unstabilized soils have typically been subjected to standard QA/QC methods related to density and moisture content such as the nuclear density (ASTM D6938). The use of these methods on additive-stabilized soils should be questioned because the nuclear density gauge was found to provide false water content readings in stabilized soils (Bergeson and Mahrt, 2000). They postulated that the increased calcium in the soil from the ash absorbs more radiation than unmodified soil, leading to unreliable readings. Based on this information, White et al. (2005) suggests the use of sand cone, rubber balloon, and/or drive cylinder for density measurements of reinforced soils.

Non-destructive methods currently exist for monitoring pavement performance after construction, but it is clear that a non-destructive monitoring method for quality control, during and shortly after construction, would be beneficial to agencies using additive stabilization of soils. Determining if the soil is mixed properly and if the target modulus is achieved would streamline the process and result in fewer construction delays or less maintenance.

Wave-based laboratory and field tests have been evaluated on unstabilized subgrade soil to determine the compatibility of results obtained in the laboratory and field (Nazarian et al., 1983; Nazarian et al., 2002). The use of traditional Proctor compaction methods in the laboratory samples yielded FFRC moduli that were less than field values measured with SASW; however, when the density and water contents were matched to field conditions, the laboratory and field modulus values matched very well. Wave-based field and laboratory measurements yielded higher moduli than resilient modulus and FWD correlations (Nazarian et al., 1999). Nazarian et al. (1999) also found that the modulus of unstabilized soil was sensitive to variations in moisture and dry density.

Nazarian et al. (2002) determined that using FFRC, the water content that corresponds to the maximum modulus design value can be identified. Their research also suggests that after construction, field quality assurance/quality control (QA/QC) can be performed with SASW. Using SASW in the field, the study found that areas that had passed moisture-density QA/QC failed with respect to modulus (Nazarian et al., 2002). The summation of the two studies by Nazarian et al. (1999 and 2002) indicates that laboratory and field measurements of modulus are a viable method for QA/QC for subgrade soils.

Quality assessment of additive-stabilized soils typically is accomplished using measurements of UCS (Ferguson, 1993) and CBR calculated from DCP testing (Parsons and Kneebone, 2005; Jung et al., 2010). No wave-based non-destructive testing studies on additive-stabilized soils, immediately after construction, are found in the literature. Based on the results from Nazarian et al. (2002), and the current lack of use of wave-based NDT methods for stabilized soils, it was concluded that the application of wave-based methods needs to be investigated for additive-stabilized soils.

2.8 Summary

The literature review revealed a history of successful stabilization of fine-grained subgrade soils using fly ash and lime kiln dust. In addition to soil drying applications, these additives have caused a marked increase in strength and stability when mixed with soils, making construction easier and potentially cheaper for flexible pavements over soft soils if the gains are counted on in design.

Studies utilizing laboratory and field methods have shown a dramatic increase in strength in the first 7-28 days after compaction for fly ash and lime kiln dust mixtures. The magnitude of the strength increase depends on the soil and additive properties, compaction water content, compactive effort, and compaction delay among other variables (Misra, 1998; Parsons and Milburn, 2003; White et al., 2005).

Methods for evaluating subgrade construction quality were discussed as were methods for the evaluation of additive stabilized soils. Studies indicate that traditional methods may not be as effective on additive stabilized subgrades; therefore, the need exists for a nondestructive method that can be employed during the design phase in the laboratory, and also during construction in order to ensure the desired results of stabilization. The combination of Spectral Analysis of Surface Waves (SASW) field testing and Free-Free Resonant Column (FFRC) laboratory testing have shown to be viable QC method for unstabilized soils. Studies of wave-based QC on stabilized soils were not found in the literature.

3 Materials and Methods

3.1 Introduction

In this thesis, two separate but related studies were completed to gain a better understanding of the effects of calcium–rich additives on Missouri subgrade soils. Presented in this chapter are descriptions of the materials (soils and additives) used in the studies, as well as descriptions of the laboratory methods used for the small-strain modulus testing and the study of the effects of soil composition on stabilization efficacy.

3.2 Soils

The soils tested for this project were obtained from Atchison County and Putnam County, Missouri. These two soils were chosen by the Missouri Department of Transportation (MoDOT) because they represent low plasticity and high plasticity clay soils that are likely candidates for stabilization. The soils were collected in the field by removing the surface vegetation, and then placing the top three feet of soil into barrels.

3.2.1 Atchison County Soil

Atchison County, Missouri is the most northwest of the counties of Missouri located on the corner of the state, sharing a border with the state of Nebraska. The Atchison soil was chosen to represent the lower plasticity clay range of Missouri soils. The soil for this study was collected outside of Watson, Missouri along Route A. Figure 3.1 displays the grain size distribution of the Atchison soil, as determined using hydrometer (ASTM D442), indicating that the majority of the particles fall in the silt-clay range.

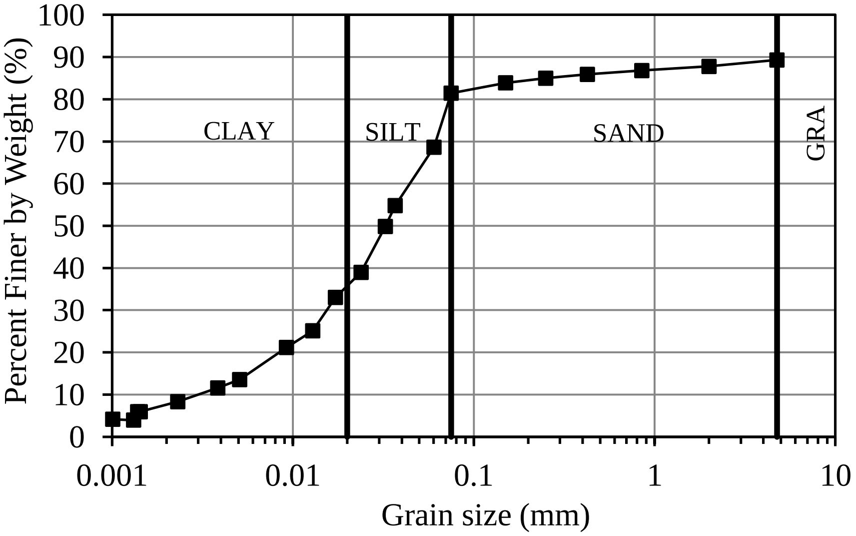


Figure 3.1 Grain size distribution curve of the Atchison soil

Table 3.1 presents the numeric values for the Atterberg limits resulting from three sets of tests, which are plotted in Figure 3.2 overlaying a USCS plasticity chart indicating its classification as a low-plasticity clay (CL).

Table 3.1 Atterberg limits for the Atchison County soil

Trial No.	Liquid Limit, LL	Plastic Limit, PL	Plasticity Index, PI
1	37	21	16
2	39	24	15
3	40	27	13

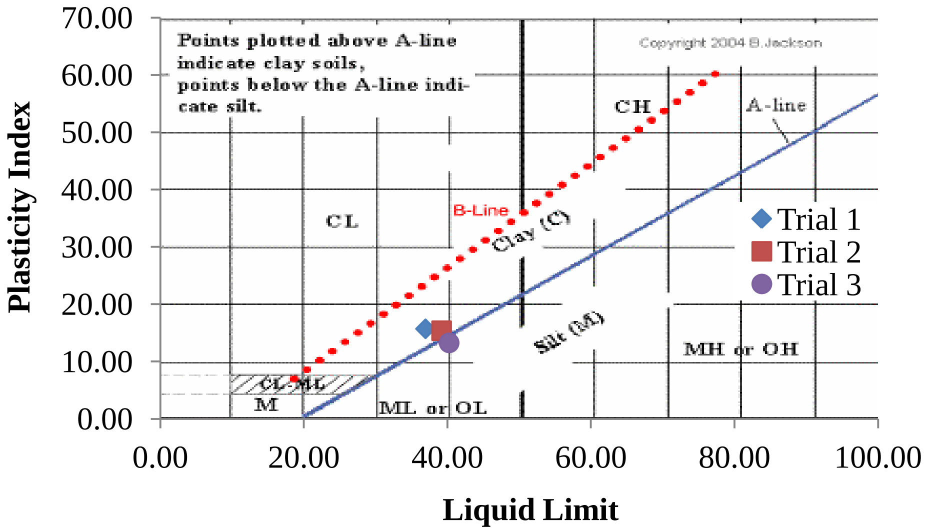


Figure 3.2 Atterberg limits on USCS plasticity chart for the Atchison County soil

The plasticity index (PI) of the Atchison County soil is between 13 and 15 which is consistent with soils used in other fly ash and lime kiln dust stabilization projects (Misra, 1998; White et al., 2005; Edil et al., 2006). The liquid limit is approximately 40 for this soil.

3.2.2 Putnam County Soil

Putnam County is near the center of the northernmost border of Missouri, sharing a border with Iowa. The Putnam County soil was chosen because of its high plasticity and difficult workability. The soil used for this study was collected near Unionville, Missouri along State Highway F. Figure 3.3 displays the grain size distribution, as determined by hydrometer (ASTM D442) of the Putnam County soil, and Figure 3.4 presents Atterberg limits for the soil on a USCS plasticity chart indicating the high plasticity clay (CH) classification. Table 3.2 contains the numeric values for the Atterberg limits for 3 sets of tests.

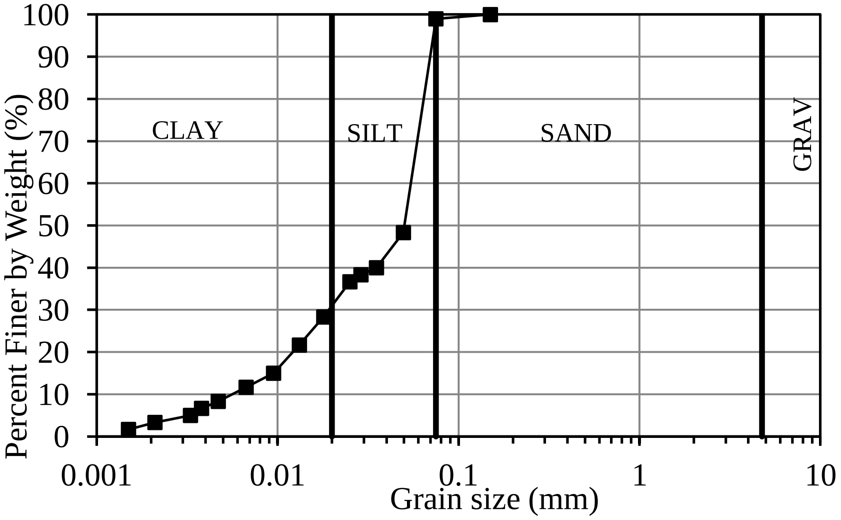


Figure 3.3 Grain size distribution curve for the Putnam County soil

Table 3.2 – Atterberg limits for the Putnam County soil

Trial No.	Liquid Limit, LL	Plastic Limit, PL	Plasticity Index, PI
1	69	31	38
2	61	31	30
3	59	28	31

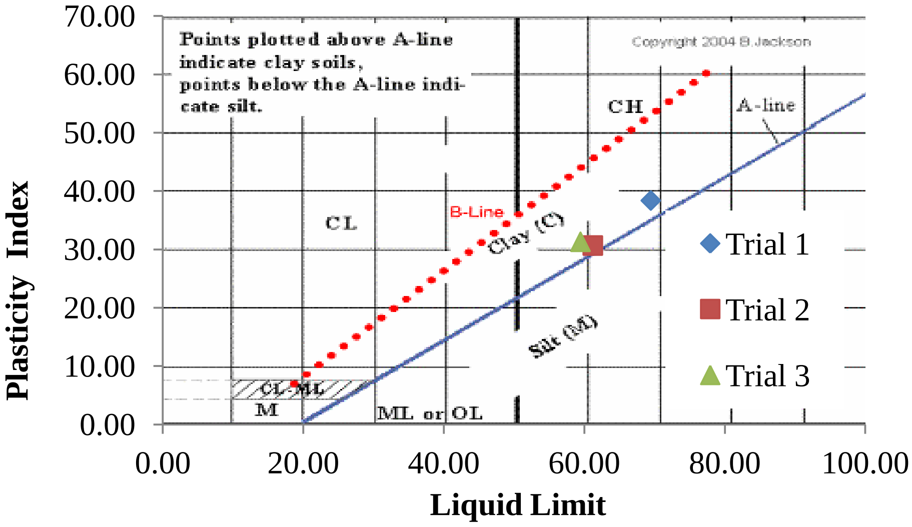


Figure 3.4 Atterberg limits on USCS plasticity chart for the Putnam County soil

The plasticity index of the Putnam County soil (between 30 and 38) is slightly higher than soils studied in other published studies using fly ash, but soils with high plasticity have been stabilized successfully in the past (Parsons and Milburn, 2003; White et al., 2005); therefore, stabilization was expected for this soil using traditional levels of fly ash additive used in previous studies. The liquid limit of 59 to 70 for Putnam County is much higher than the Atchison County soil.

3.3 Additives Used

3.3.1 Fly Ash

Initially five fly ashes were acquired from LaCygne, Nearman, Labadie, Rush Island and Meramec power plants in Missouri. Physical composition properties of the fly ashes determined from XRD and SEM-EDS are summarized in Table 3.3. All fly ashes were derived from combustion of coal and were collected using electro-static precipitators. All were high in calcium oxide and silicon dioxide content. Their CaO-to-SiO_2 ratios ranged from 0.66 to 0.93 which is an indication of how pozzolanically active the additive will be without the addition of silicon from the soil. The loss on ignition of all the fly ashes was small (<1%), with the exception of the Meramec fly ash which was still within the acceptable range of less than 4% (ASTM C618).

Table 3.3 Percentage of Chemical Compositions in Fly Ash

Chemical Compound	Percent of Chemical Compositions (%)				
	Rush Island (A)	LaCynge (B)	Nearman (C)	Meramec (D)	Labadie (E)
SiO_2	32.26	33.31	30.55	35.42	33.72
Al_2O_3	19.03	20.57	18.78	16.88	21.90
Fe_2O_3	6.24	6.15	7.48	7.97	7.15
CaO	27.94	26.34	28.43	23.21	25.31
MgO	5.55	5.27	5.09	4.87	4.48
SO_3	2.40	1.87	3.33	3.46	2.25
K_2O	0.33	0.43	0.45	0.56	0.41
P_2O_5	1.35	1.27	1.58	1.10	1.20
Ti O_2	1.30	1.59	1.60	1.56	1.30
Na_2O	2.20	1.63	1.50	1.40	1.40
Loss on Ignition	0.26	0.49	0.57	3.05	0.37
Specific Gravity	2.73	2.72	2.72	2.70	2.71
CaO/ SiO_2	0.87	0.79	0.93	0.66	0.75
Classification	C	C	C	C	C

Grain size distribution curves of all the fly ashes are presented in Figure 3.5. The Nearman, Rush Island and Labadie fly ashes are gap graded, which indicated that those three fly ashes were coarser than the other two fly ashes. Approximately 50% of the finer fly ash particles were smaller than 0.075 mm; however, for the coarser fly ashes, only 15% passed the #200 sieve (0.075 mm). The grain size distribution curve of the Atchison County soil used in this study is also shown for comparison in Figure 3.5. It was decided, based on laboratory results, that the Labadie and LaCygne fly ashes would be used; however, due to time constraints and input from MoDOT, lime kiln dust was substituted for the LaCynge fly ash.

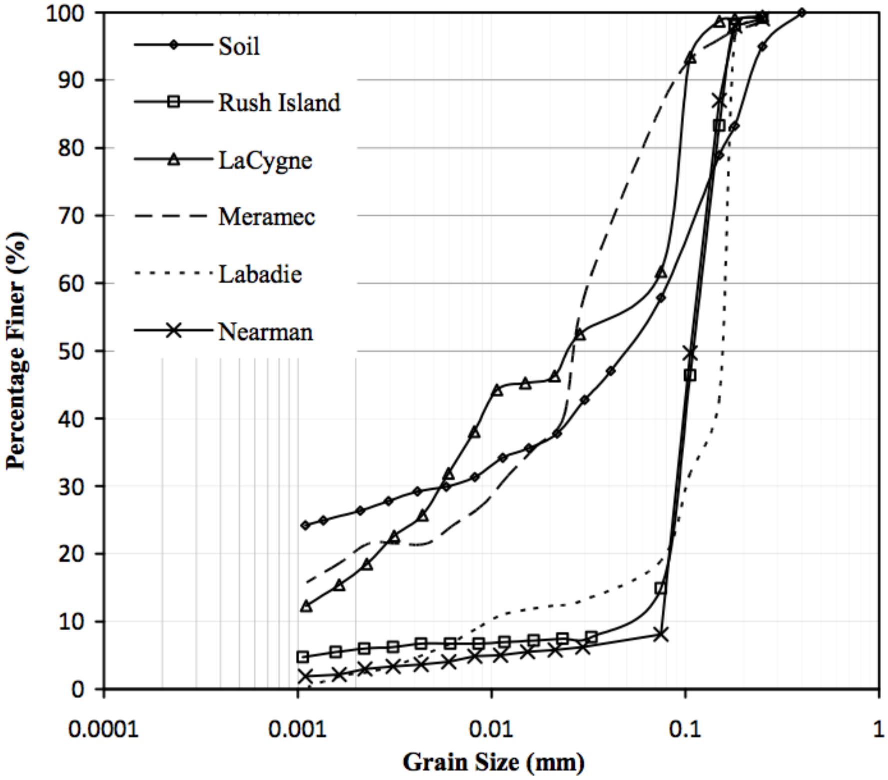


Figure 3.5 Grain size distribution of fly ashes and Atchison County soil from sieve and hydrometer analyses

3.3.2 Lime Kiln Dust

Lime kiln dust is a byproduct of lime manufacturing and consists primarily of CaO and $CaCO_3$. The lime kiln dust used in this study was supplied by Mississippi Lime Company and has an industry name of Code L. The chemical properties of Code L are listed in Table 3.4.

Table 3.4 Chemical Properties of Mississippi Lime Company Code L

Chemical Compound	Range of Values (%)
CaO	28-38
$CaCO_3$	31-38
$Ca(OH)_2$	5-8
SiO_2	4-8
Fe_2O_3	1.5-3
Al_2O_3	1-3
S	2.5-3.5

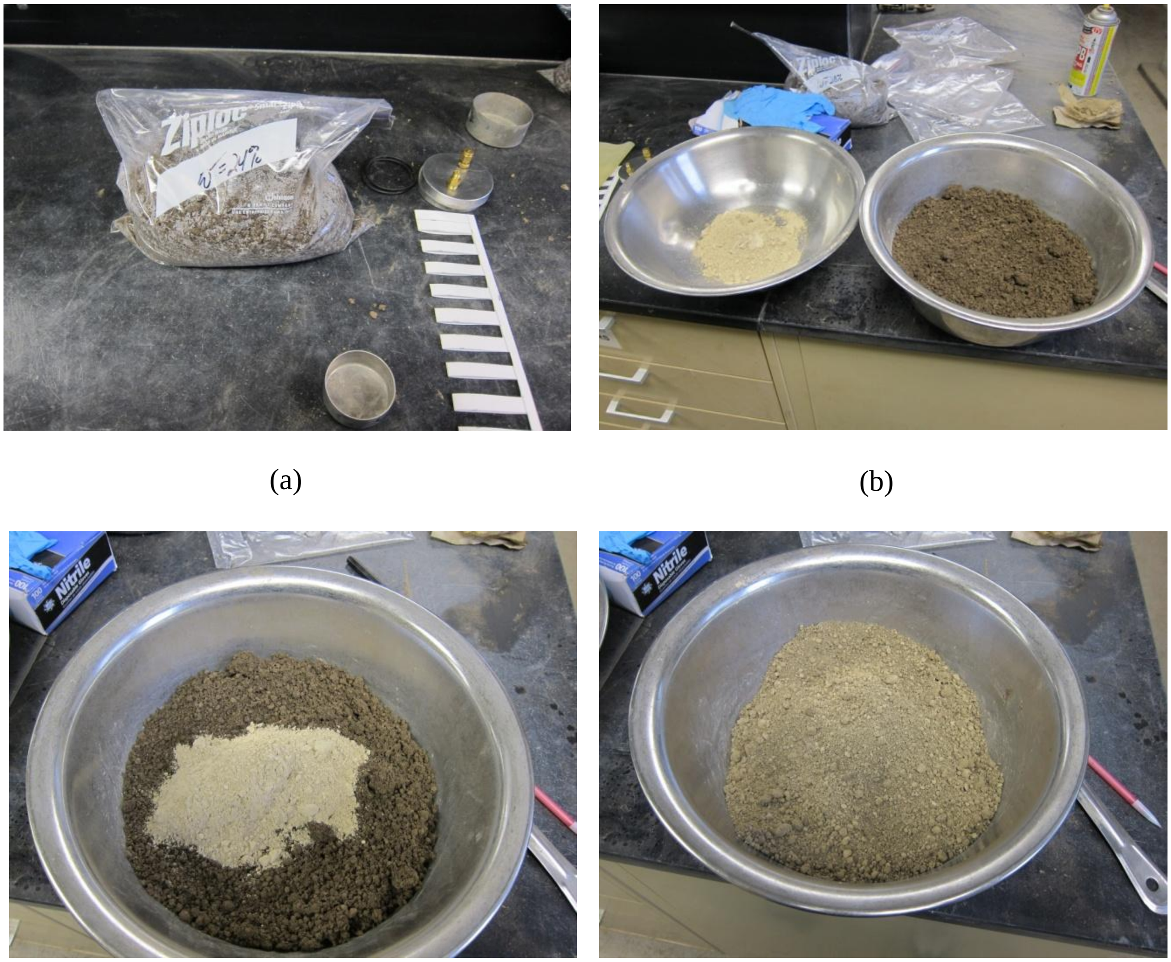
The lime kiln dust contains high concentrations of calcium in different compounds, all of which can be combined with soil to create cementious bonds. The increased concentrations of silicon and aluminum, compared to regular lime, are beneficial for the pozzolanic cementing process resulting in the composition being similar to fly ash, but with a much higher content of lime.

3.4 Sample Preparation for Small-Strain Modulus Testing

Soil gathered from two field sites required processing prior to mixing with chemical additives. This section describes the methods for soil preparation and compaction followed by background of the free-free resonant column method and a presentation of the methods used in the small strain modulus testing portion of this study.

3.4.1 Soil Preparation

The soil preparation procedure was the same for both free-free resonant column (FFRC) and physiochemical property testing. Soil from the field was placed in a large bowl that was oven-dried overnight at 110 ° C then sieved. Particles retained on the No. 4 sieve (>4.74mm) were discarded. The soil was mixed with a spatula while a spray bottle gradually added the desired amount of water. The desired water content was determined from Proctor compaction data. The amount of water added was calculated as the mass of dry solids (soil and additive) multiplied by the target water content percentage. Once mixed, the wet soil was placed in a plastic bag and sealed to temper overnight, as shown in Figure 3.6 (a). The water content was measured according to ASTM D2216 and the soil was mixed with the additive (fly ash or lime kiln dust) before compaction. The mass of soil additive was determined by multiplying the dry weight of soil by the desired percentage of additive. The mixing process is shown in Figures 3.6. The soil/additive mixture was mixed by hand to facilitate better distribution of the soil additive throughout the soil.



(a) (b)

(c) (d)

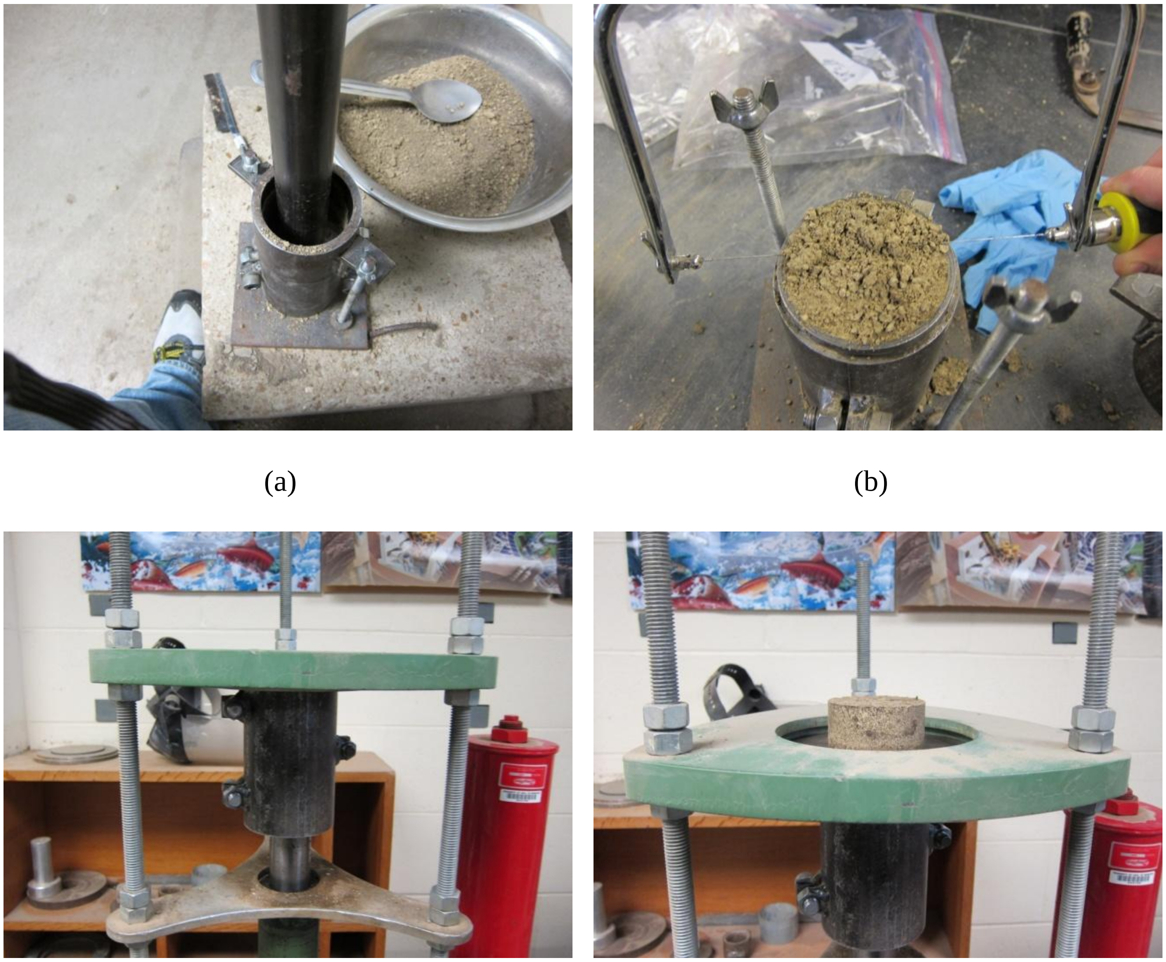
Figure 3.6 Mixing process for specimens (a) wet soil tempered overnight in plastic bag, (b) additive (left) and wet soil (right) prior to mixing, (c) additive added to wet soil ,and (d) soil and additive after mixing.

3.4.2 Compaction

Standard Proctor tests were performed by researchers at Missouri Science & Technology (MST) on six-inch diameter samples used for Briaud Compaction Device (BCD) modulus testing. Compaction was also performed on 4-inch samples using the ASTM D698 procedure to verify the consistency of the results. A comparison of values indicated good agreement between four-inch standard proctor mold curves and the curves created with six-inch samples. Using the compaction information provided by MST, samples were prepared wet, dry and near optimum. Early tests were performed using one sample dry of optimum, one near optimum, and one wet of optimum. For later tests, five water contents were selected (approximately 4% dry, 2% dry, "optimum", 2% wet , 4% wet) as targets for the FFRC testing.

Reliable FFRC measurements require samples that have length-to-diameter ratios of approximately two; therefore, it was not possible to use a standard compaction mold to prepare the samples. Instead, the compaction mold used in this project was a 2.9-inch inside-diameter steel mold with a height of 5.2 inches, producing a height-to-diameter ratio close to two. Samples were prepared using the equivalent energy per volume as the standard proctor test. A compaction hammer was constructed with a standard proctor mass and height of drop (5.5 lbs and 12 inches respectively), but with a smaller diameter to better replicate the kneading effect of the standard proctor test in a four-inch mold. This hammer produced better samples, with respect to the standard proctor curve, than a standard proctor hammer with equivalent energy in a comparative study.

The soil/additive mixture was placed in the mold in three lifts, with each lift compacted with fifteen blows in the pattern established by ASTM D698. An overview of the compaction process is shown in Figure 3.7. After compaction, the collar was removed and the excess soil was trimmed with a wire until the top of the sample was uniformly flat, as in the photo in Figure 3.7 (b). The weight of the mold and sample was then recorded, and the specimen was extruded from the mold. A split mold used, but early tests indicated the tendency of the stabilized specimens to crack upon splitting of the mold, so extrusion was used for all resonance testing and unconfined strength specimens. The extrusion process is shown in the photos of Figure 3.7 (c) and (d).

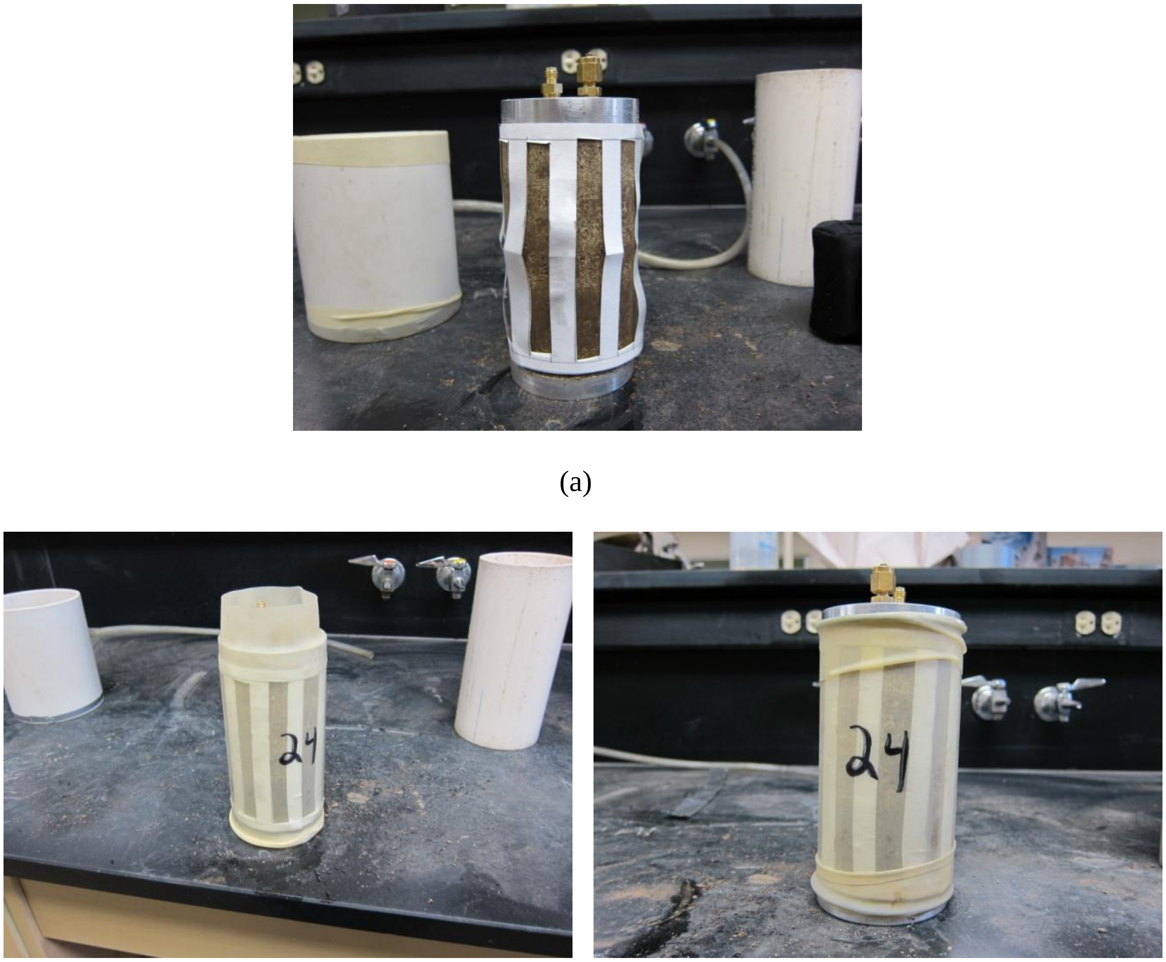


(a) (b)

(c) (d)

Figure 3.7 Compaction and extrusion procedure steps (a) soil compacted in mold, (b) excess soil trimmed from top of mold after collar removed, (c) sample placed in extruder, and (d) sample extruded from mold.

The samples designated for unconfined strength testing were wrapped in plastic following extrusion. The samples were labeled and placed in a cure room until the designated testing time. The resonance specimens were wrapped with filter paper that had been cut with 0.25-inch strips across the top and bottom and 0.5-inch vertical strips alternated with 0.5-inch spaces to facilitate uniform vacuum around the specimen. An example of the filter paper wrap is shown in Figure 3.8 (a). End caps were placed on the sample to facilitate attachment of instrumentation and application of vacuum pressure to the specimens. A rubber membrane with dimensions of 2.5 inches in diameter, 9 inches in height and 0.012 inches thick, encased the specimen, filter paper, and end caps (Figure 3.8 (b)). The membrane was secured to the end caps with rubber O-rings. The final prepared sample is shown in Figure 3.8 (c).



(a)

(b)

(c)

Figure 3.8 Final sample preparation (a) end caps and filter paper applied, (b) membrane applied, and (c) o-rings applied to end caps.

3.5 Small-Strain Modulus Testing Using Free-Free Resonant Column

3.5.1 FFRC Background

The free-free resonant column (FFRC) test proposed by Stokoe et al. (1994) allows fast and economical determination of the small-strain resonant frequency of a cylindrical soil sample under both axial and torsional excitation. The FFRC is a simple apparatus for obtaining small-strain compression and shear moduli of samples under low confinement pressures. The test was chosen for this study because it is non-destructive and allows tracking of modulus change on the same sample with time. The test can also be performed under confining pressures that are representative of pavement subgrade depths.

The specimen can be perceived as a rod of finite length that resonates at a specific frequency when excited (Stokoe et al., 1994). The direction of the excitation (longitudinal or torsional), the stiffness and length of the specimen determine the resonant frequency as governed by wave propagation theory (Richart et al., 1970). The resonant frequency for a longitudinal compression wave is related to the velocity as:

$$V_C = \frac{2\pi L f_c}{\alpha} \tag{3.1}$$

where, V_c is the unconstrained compression wave velocity, L is the length of the specimen, f_c is the resonant frequency, and α is a factor to account for the mass of the end caps attached to the specimen.

A wave excited torsionally propagates through the sample as a shear wave with a particle motion perpendicular to the wave propagation direction. The resonant frequency allows the back calculation of velocity from:

$$V_s = \frac{2\pi L f_s}{\beta} \tag{3.2}$$

where, V_s is the shear wave velocity, f_s is the resonant frequency from the torsional wave, and β accounts for the polar moment of inertia of the end caps.

The presence of accelerometers and vacuum fittings are also accounted for in the factors, α and β. The measured frequencies, f_c and f_s are the first mode resonant frequencies for unconfined compression and shear wave, respectively.

Values of V_c and V_s are calculated from frequency-domain measurements of resonance. Another type of measurement for compression wave velocity can be performed in the time domain. This measurement is performed by determining the time for a compression wave to travel from one end of the sample to the accelerometer on the other end. The constrained wave velocity, V_P can be determined by:

$$V_P = \frac{L}{t^*} \tag{3.3}$$

where L is the length of the specimen and t^* is the net time the wave travels through the sample. The net time accounts for the time the wave travels through the end caps, as shown in Eq. 3.4:

$$t^* = t - \frac{L_C}{V_{AL}} \tag{3.4}$$

where, t is the measured time of travel, L_C is the length (or thickness) of aluminum end caps, and V_{AL} is the constrained wave velocity of aluminum.

Each velocity value can be used to calculate the associated small-strain modulus values from:

$$E_{MAX} = \rho V_C^2 \tag{3.5}$$

$$G_{MAX} = \rho V_S^2 \tag{3.6}$$

$$M_{MAX} = \rho V_P^2 \tag{3.7}$$

where ρ is the mass density of the specimen, E_{MAX} is the small-strain Young's modulus, G_{MAX} is the small strain shear modulus, and M_{MAX} is the small-strain constrained modulus.

3.5.2 Free-Free Resonance Column (FFRC) Testing

Free-free resonance tests were used in this study to monitor the changes in the stabilized soil with time. An initial baseline measurement was performed for each sample immediately after the specimens were compacted and the membrane was applied. This time is considered “time zero” in this study; however, it is about 20 to 40 minutes after the additive was first introduced in the soil/water mixture. Measurements were then performed at approximately one hour, three hours, twelve hours, one day, three days, and seven days (in some cases the seven day measurement was not recorded). One objective of this portion of the research was to use the modulus-time data to evaluate the potential of wave-based non-destructive testing methods for construction quality control of stabilized subgrades. Therefore, the relevant time frame for this application is the first few hours after placement. A second objective was to study the gain in modulus achieved for soil placed at optimum or modified wet of optimum with different types and percentages of additives. An example of the application considered here is the modification of soil to develop a viable working platform in soft soils. The relevant time frame for this application is on the order of several days after compaction; therefore, this portion of the study focused on a time frame out to three to seven days.

The sample was suspended from a frame using two elastic cords and a vacuum pressure of 5 psi was applied through one of the two equally spaced fittings on one endplate, as shown in Figure 3.9. This vacuum pressure was chosen to simulate the small confining pressures at shallow depths of the subgrade. Two fittings were installed to balance the moment of inertia for the end plate. The opposite end plate was a solid disk of aluminum on which sensors were placed to detect the wave movements. Three accelerometers (Model no. 352C66 from PCB Piezotronics) were glued to the plate, two vertical for detecting shear waves (torsion) and one horizontal to detect constrained and unconstrained compression waves. The two vertical accelerometers were placed on the same plane and their outputs were summed after reversing the polarity of one of the outputs. A summation of the two accelerometers on the same channel amplifies the torsional response, and cancels out bending motions created by the propagating wave.

The accelerometers were originally attached to the end plates using super glue. This was determined to be inefficient because of the excessive time needed to clean the dried glue off of accelerometers and the end plate; therefore, the end plate was modified with the addition of three tapped holes to allow attachment of the accelerometers. Two of the holes allowed the coupling of aluminum cubes (Model no. 080B16 from PCB Piezotronics) made specifically for accelerometers. The center hole was tapped and threaded so that the axial accelerometer could be screwed directly into the end plate. The final placement of the accelerometers is shown in Figure 3.10.

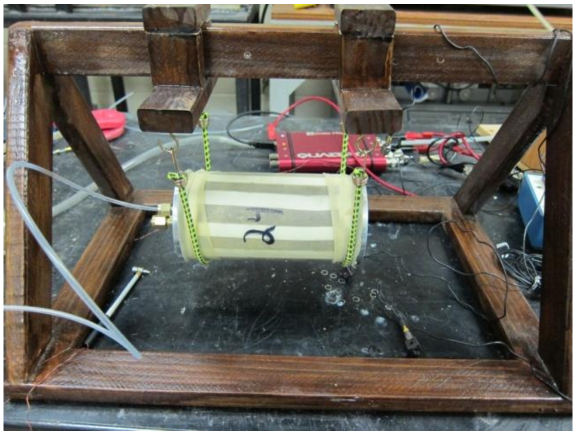


Figure 3.9 Sample in free-free resonant column

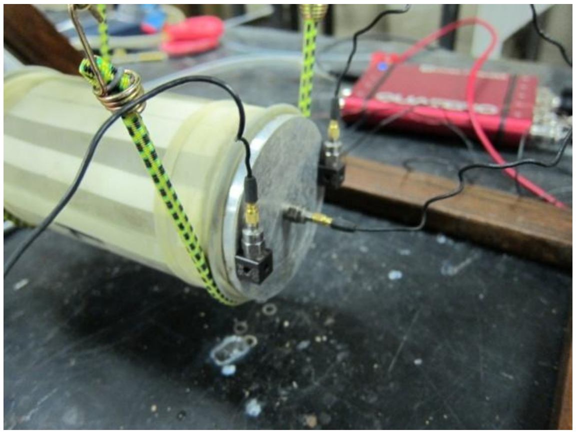


Figure 3.10 Accelerometer attached to endplate

Measurements were recorded after the accelerometers were attached to the sample and the vacuum pressure reached 5 psi. The SignalCalc software package from Data Physics, was used to record the experiments. The software is designed to work with the Data Physics Quattro signal analyzer used in this study (model no. DP240). Parameters were input to designate in which domain the measurement was to be taken. When the appropriate file was selected, the sample was struck with an instrumented hammer (model no. 086D80 from PCB Piezotronics) either axially on the center of the end plate, or vertically on the edges of the end plate to induce torsional waves, as shown in Figure 3.11. The resonant frequency was determined from the first peak of the frequency spectrum (Figure 3.12) and recorded in a spreadsheet. A spreadsheet was developed using Eq. 3.1-3.7 to calculate the modulus values for each sample from the measured frequency values. This method allowed for the tracking of the change in velocities and moduli with time.

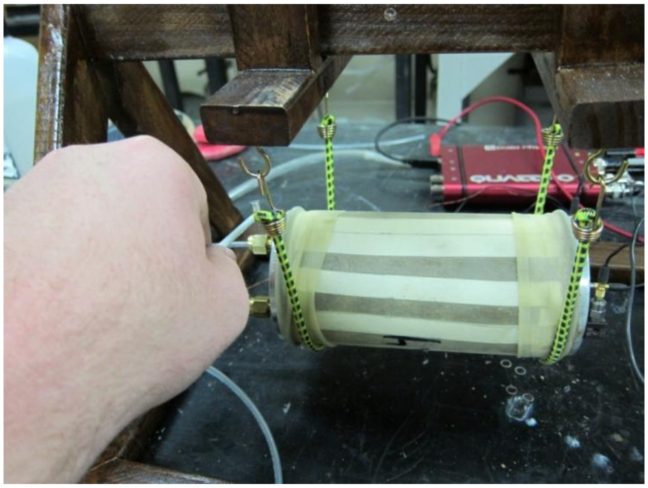


Figure 3.11 Downward strike with instrumented hammer to induce shear wave

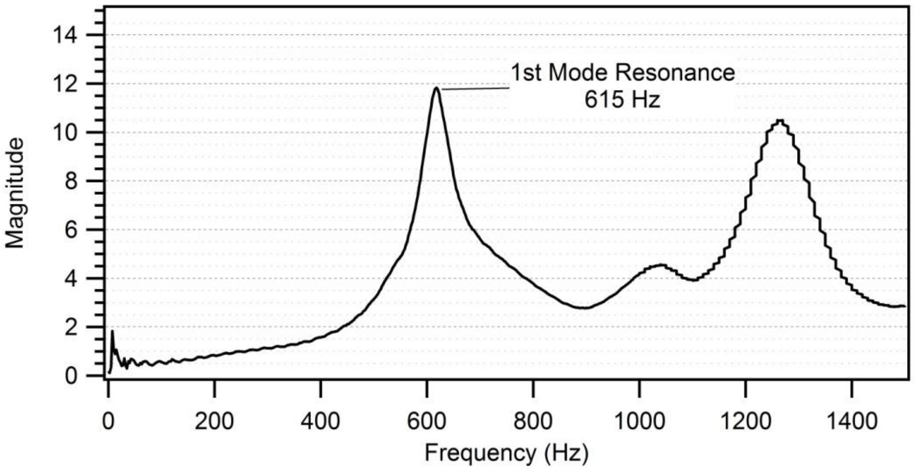


Figure 3.12 Frequency response showing first mode resonance peak at 615 Hz for torsional test of Atchison County soil with 10% fly ash tested 3-hrs after compaction

To measure the constrained modulus, the P-wave arrive time was measured directly in the time domain. The sample was struck axially in the center of the endplate to produce a longitudinal wave in the same way as the unconstrained measurement (Figure 3.13). The velocity is determined using the difference in time between the initiation of the impulse generated by the hammer and the arrival of the wave at the accelerometer (Figure 3.14). The velocity was determined using Eqs. 3.3 and 3.4, which account for the travel time through the aluminum end plates. The associated constrained modulus was calculated using the velocity and mass density values (Eq. 3.7).

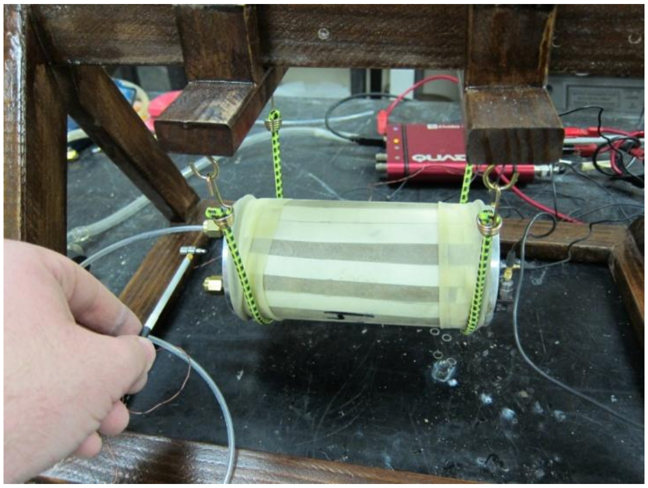


Figure 3.13 Instrumented hammer strike to induce axial waves

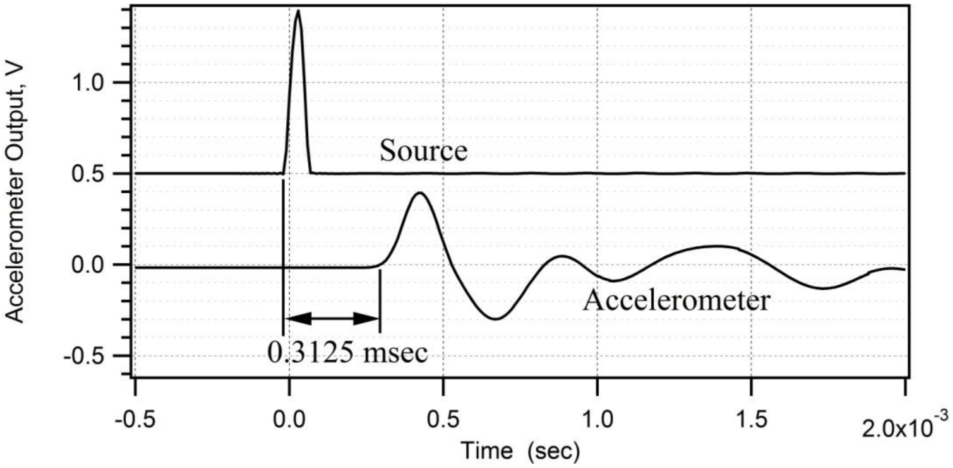


Figure 3.14 Time domain measurement of P-wave velocity measurements from testing on Atchison County soil mixed with 10% fly ash tested 3 hrs after compaction

The specimens were held under a confinement pressure of 5 psi between measurements to simulate shallow subgrade stress conditions. Specimens prepared dry of optimum were confined using vacuum pressure and specimens wet of optimum were placed in triaxial cells with a confining air pressure of 5 psi as a result of equipment quantity limitations. The applied vacuum showed a tendency to extract water from wet specimens over time, but not from those prepared dry of optimum. The pressure fittings on the top end cap of the specimens contained in the triaxial cells were attached to one of the cell pressure lines that was open to atmospheric pressure (Figure 3.15) to allow consolidation under the confining pressure applied to the cell, shown in Figure 3.16. The samples were removed from confinement, at each designated testing time and a vacuum pressure of 5 psi was immediately applied to the samples before the velocity measurements were performed.

It should be emphasized that all of these measurements were performed in the small-strain range within which the soil structure is not affected by repeated measurements. The measurement cannot be considered non-destructive if it affects the parameter of interest, immediately or long term.

Figure 3.15 Specimen after testing placed in pressurized triaxial cell

Figure 3.16 Specimen confined in pressurized cell

3.6 Methods Used for Soil Composition Study

The objectives of the second study were to examine differences in the reactions between the additives and the two soils chosen for the study in order to propose an explanation for observed responses presented in Chapter 4. The steps in the examination were:

- Determine soil composition using particle size analysis and X-ray diffraction (XRD);
- Visualize reaction products using scanning electron microscope (SEM) imaging; and
- Investigate chemical reaction effects between soil and additives through measurement of exchangeable cations and pH.

Methods for accomplishing these steps are presented in the following sections.

3.6.1 Preparation of Composition Testing Specimens

Specimens of the two soils were prepared to dry of, or near optimum water contents with no additive, with 20 percent fly ash, and with 8 percent LKD. The optimum water content was different for each mix for a particular soil as observed in Proctor testing. The Atchison County soil samples were prepared at optimum water content (15%) while the Putnam County samples were dry of optimum by about three to four percent because of an error in the calculation of target water content. Although not ideal, this difference in water content was not deemed large enough to discount the results. The soils were mixed with the desired weight of water and placed in plastic bags to temper overnight. Additives were added to the soil-water mixes after 24 hours of tempering, and the samples were immediately compacted using standard Proctor energy, before being extruded from the mold and wrapped in plastic. The specimens were then placed in a 100 percent humidity curing room for 7 days. The specimens were then unwrapped and disassembled into drying trays where they were air-dried for 24 hours. The soil-additive mixes were processed using a mortar and pestle before being passed through a number 10 sieve (2.00 mm). These final products were then distributed to the laboratories for testing.

3.6.2 X-Ray Diffraction Analysis

X-ray diffraction analysis is used to determine the minerals present in samples of unknown composition. The process is illustrated in Figure 3.17 showing that X-rays are emitted at a known angle toward a stationary sample. The rays enter the structure of the specimen and are diffracted before they leave the specimen. The rays are received by a counter that measures how many rays are received at each angle. Every mineral is associated with discrete angles; therefore the results of the counts can be used to determine what minerals are present in the specimen.

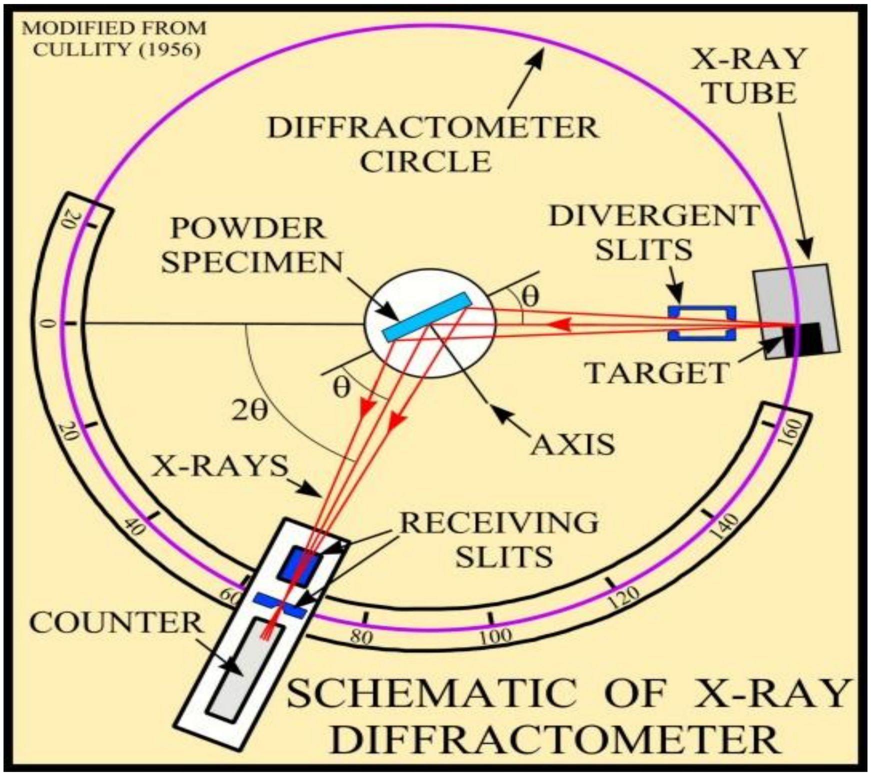


Figure 3.17 Schematic of X-Ray Diffraction (XRD) test (Retrieved from: http://pubs.usgs.gov/of/2001/of01-041/htmldocs/xrpd.htm)

Another variation of the sample preparation separates the clay sized fraction for diffraction. The results show peaks corresponding to different clay minerals. A line is drawn, and used to as a datum, to integrate under the peaks, in order to determine how much of the total clay-sized fraction is associated with each clay mineral. Calculation of the quantity of clay minerals is a semi-quantitative approach because it is difficult to ensure the accuracy of these values. The standard error is around ±10% unless a much more extensive sample preparation method is used to ensure accuracy (Środoń et al., 2001; Omotoso et al., 2006).

XRD analyses were performed on both the bulk specimens and the clay fraction, by the XRD laboratory operated within the Geology Department at the University of

Missouri. A Scintag Pad V X-ray diffractometer with CuKα radiation (1.54 Å) and Ni filter was used for all tests. The results were interpreted using MacDiff software (Petschick, 2001) to determine the composition of the samples.

Isolation of clay-size fractions consisted of air drying and crushing using a mortar and pestle. The resulting powder was then mixed in 3 percent H_2O_2 for at least 24 hours to remove any trace organic matter. Next, about 250 mL of Na hexametaphosphate solution (concentration of 4 g/1000 mL distilled H_2O) was added to the mixture and the beakers were inserted into an ultrasonic bath for several minutes to separate the clay particles. This step (and additional soaking) was repeated until visual inspection indicated that the clay particles were separated. The samples were washed with two passes through a centrifuge (8200 revolutions per minute for 25 min) and resuspended in distilled-deionized water after each pass. The suspended sediment was transferred to a 60 mL plastic bottle, and each sample was resuspended by hand-shaking. The clay-size fractions were then separated by centrifugation (1000 rpm for 2.4 min). Oriented clay aggregates were prepared using the filter-peel method (Moore and Reynolds, 1997) with 0.45 µm thick membranes. The clay aggregates were saturated with ethylene glycol vapor for at least 24 h prior to XRD analysis, using a closed vapor chamber heated to 60°C in an oven. This procedure is outlined in work by Guo and Underwood (2009).

3.6.3 Scanning Electron Microscope

A scanning electron microscope (SEM) can be used to image micron-sized specimens using the process illustrated in Figure 3.18. Electrons are emitted in the direction of the specimen. The electron beam is directed by anodes and magnetic lenses to focus at the point in space occupied by the specimen. The electrons that backscatter after entering the specimen are captured by detectors and converted into an image.

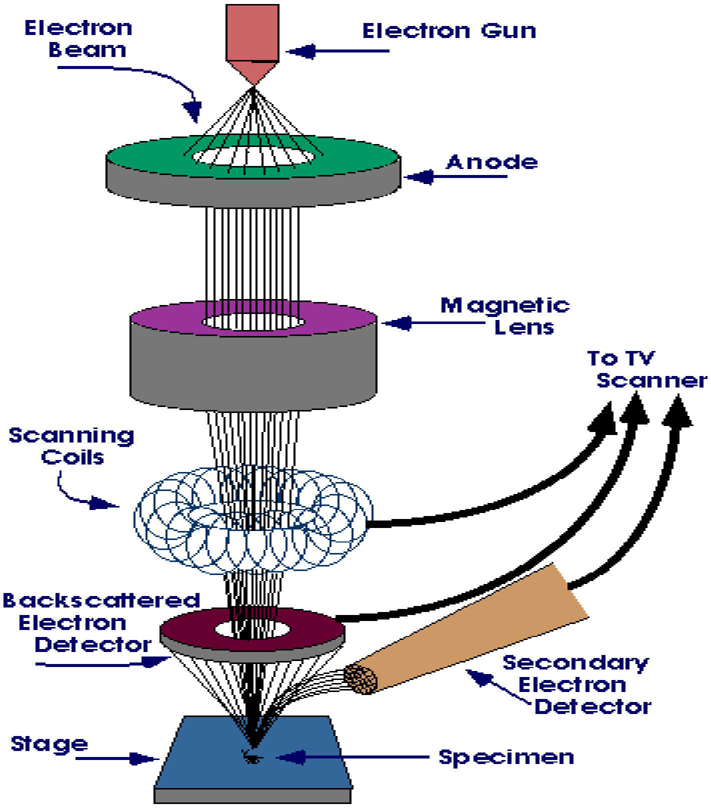


Figure 3.18 Schematic of scanning electron microscope (Retrieved from: http://www.purdue.edu/rem/rs/sem.htm)

Soil, additive, and soil-additive mixtures were imaged using the FEI Quanta 600 FEG Extended Vacuum Scanning Electron Microscope (ESEM) operated by the University of Missouri Core Microscopy Facility. Samples for imaging in the microscope were prepared by attaching them to a carbon paper. The specimens were not coated, as the images appeared clear without the additional step of adding a coating to increase the resolution.

3.6.4 Cation Exchange Capacity

The presence of cations in the interlayer of clay particles allows chemical changes to occur if another cation of equal or higher potential exchanges with the existing interlayer cation. Cation exchange is important for this study because the additives contain high concentrations of calcium, which will exchange with interlayer cations due to the bivalent charge and abundance of calcium. The composition of the exchangeable cations can be measured in the laboratory during the determination of cation exchange capacity (CEC).

The cation exchange capacity is a measure of a soil's affinity for cations. The surface charge of clays is negative, and the number of charge sites varies based on clay type and mineralogy. The three sources of cation exchange capacity in clays are derived from isomorphous substitution within the mineral structure, broken bond sites exposing exchange sites on noncleavage surfaces, and replacement of hydrogen on exposed hydroxyls in the structure (Mitchell and Soga, 2005).

Cation exchange capacities, exchangeable cations, and pH analyses were performed by the University of Missouri Soil Characterization Laboratory following procedures outlined by the USDA National Soil Survey Laboratory (USDA, 2004). Tests were performed on samples from specimens seven days after compaction Cation exchange analyses were performed using ammonium chloride (standard 4B1b1a1a1a-b1) to avoid false readings of calcium for these high-pH, calcium-enriched samples (USDA, 2004). Standard number 4C1a2 was used to test the pH of the specimens using both 1:1 water and 1:2 salt mixtures with 0.01 molar CaCl solution (USDA, 2004).

3.7 Summary

Two soils were chosen based on their differences in plasticity index and MoDOT's interest in assessing their potential for use in additive stabilization projects. The Atchison County soil is a low plasticity (PI~15) clay and the Putnam County soil classifies as a high plasticity (PI~40) clay. Two additives, a fly ash and a lime kiln dust, were selected for this study and their properties have been presented in this chapter.

Background on Free-Free Resonant Column (FFRC) testing used in this study, as well as the specific procedures used for sample preparation and testing have been presented.

Methods used to study the effect of soil composition on the effectiveness of additive stabilization were presented. Sample preparation and methods for X-ray diffraction (XRD), cation exchange capacity testing (CEC), and scanning electron microscope imaging (SEM) were also presented in this chapter.

4 Results and Discussion - Small-Strain Modulus Study

4.1 Introduction

Results from the small-strain modulus testing of stabilized and unstabilized soils are presented and discussed in this chapter. Compaction behavior and trends for both Putnam and Atchison County soils with and without additive are presented, followed by presentation and discussion of the results of the small-strain measurements performed with the free-free resonant column (FFRC). The change in modulus with time and a discussion of the factors that affect the magnitude of the change in modulus are compared and discussed for both soils. A comparison of resilient modulus measurements and small-strain Young's modulus is also presented and discussed.

4.2 Compaction Results

Compaction curves from testing performed on the soils from Atchison and Putnam Counties are presented and compared to the compaction curves of soil-additive mixtures. Mixtures using 10 percent, 15 percent and 20 percent fly ash, and 4 percent and 8 percent lime kiln dust by weight were investigated for both soils. Compaction results of each mixture are presented in Appendix A. Proctor results were obtained using methods described in Chapter 3.

Compaction curves for fly ash mixed with Atchison and Putnam County soils are presented in Figures 4.1 and 4.2, respectively. The maximum dry densities and optimum water contents for all mixtures are summarized in Tables 4.1 and 4.2 for Atchison and Putnam County soils, respectively. The compaction curves for the Atchison County soil show a clear trend of decreasing optimum water content and increasing maximum dry density with increased fly ash content. The Putnam County soil differed from the Atchison County soil in that the optimum water content exhibited only small changes (about 1 percent) in the optimum water content values.

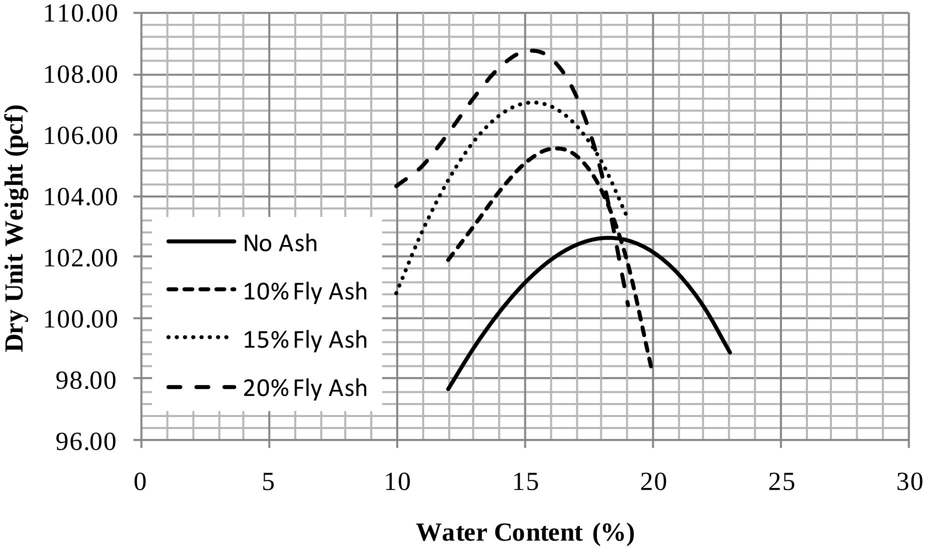


Figure 4.1 Comparison of Proctor results for Atchison County soil alone and with 10%, 15% and 20% fly ash by weight.

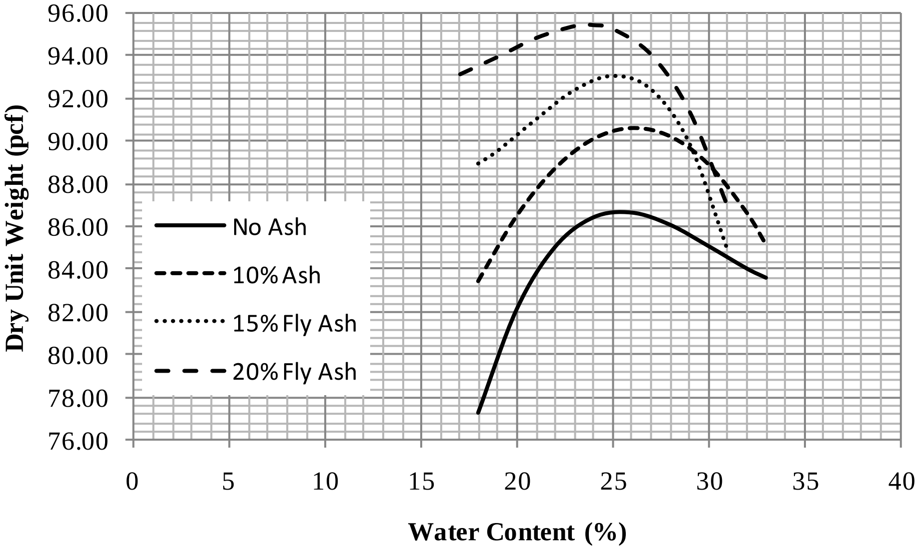


Figure 4.2 Comparison of Proctor results for Putnam County soil alone and with 10%, 15% and 20% fly ash by weight.

Table 4.1 Optimum water content and maximum dry density for Atchison County soil with and without fly ash and lime kiln dust.

Mixture	Optimum Water Content (%)	Maximum Dry Density (pcf)
Atchison No Additive	18	103
Atchison +10% FA	16	106
Atchison +15% FA	15	107
Atchison +20% FA	15	109
Atchison +4% LKD	17	102
Atchison +8% LKD	17	102

Table 4.2 Optimum water content and maximum dry density for Putnam County soil with and without fly ash and lime kiln dust.

Mixture	Optimum Water Content (%)	Maximum Dry Density (pcf)
Putnam No Additive	25	87
Putnam +10% FA	26	91
Putnam +15% FA	25	93
Putnam +20% FA	24	95
Putnam +4% LKD	27	88
Putnam +8% LKD	29	87

Compaction behavior of soils stabilized with lime kiln dust differed from the fly ash stabilized soils. The changes in compaction curve response for the Atchison County soil, with and without lime kiln dust, are presented in Figure 4.3 and Table 4.1. The optimum water content decreased by one percent compared to the soil only (from 18 percent to 17 percent) for both lime kiln dust mixtures, while the maximum dry unit weight decreased slightly from 103 pcf to 102 pcf. Percentage of lime kiln dust did not appear to have much effect on the measured compaction curves, as the optimum water content and maximum dry density were about the same for the 4 percent and 8 percent lime kiln dust mixtures.

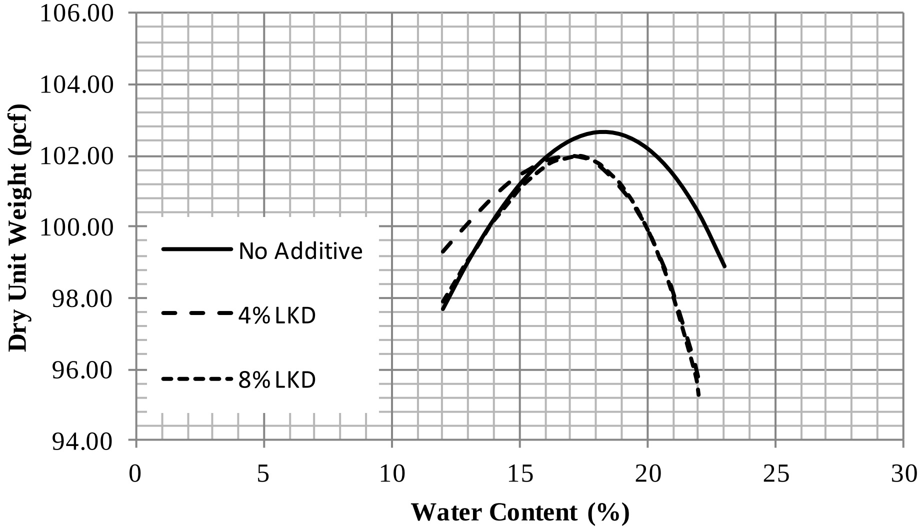


Figure 4.3 Comparison of Proctor results for Atchison County soil alone and with 4% and 8% lime kiln dust by weight.

The compaction results for the Putnam County soil, with and without the addition of lime kiln dust, are presented in Figure 4.4 and Table 4.2. The optimum water content of the lime kiln dust mixtures increased from 25 percent for the Putnam County soil alone, to 27 percent and 29 percent for the 4 percent and 8 percent lime kiln dust mixtures, respectively. The maximum dry densities of the Putnam-lime kiln dust mixtures were 88 pcf and 87 pcf for 4 percent and 8 percent lime kiln dust mixtures, respectively compared to 87 pcf for the Putnam County soil alone.

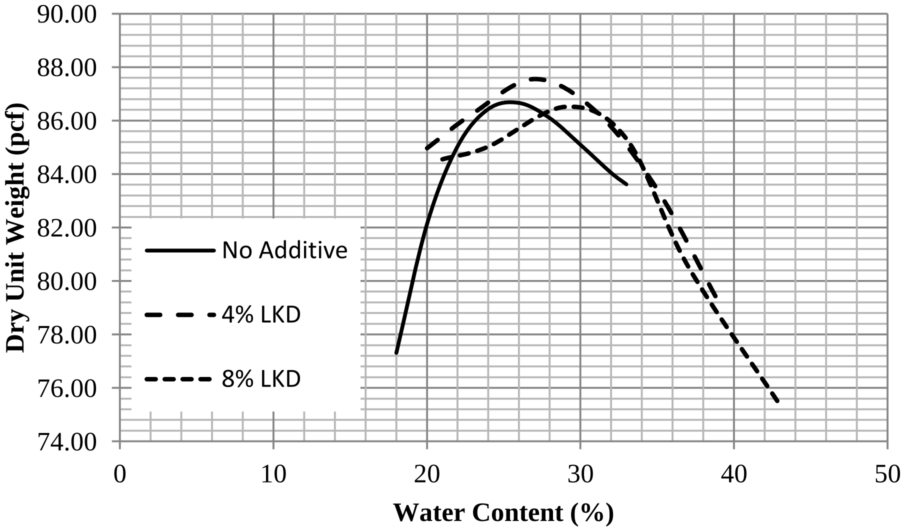


Figure 4.4 Comparison of Proctor results for lime kiln dust mixtures and Putnam County soil

The compaction results from the fly ash-soil mixtures indicate a change in the compaction properties caused by soil additives. These effects can be visualized by comparing the compaction curves to degree of saturation curves, as presented for the Atchison and Putnam County soils in Figures 4.5 and 4.6. The increased density and decreased optimum water content displayed by the Atchison County soil with increasing fly ash mixtures is similar to behavior that has also been observed when applying higher compaction energy on an unstabilized soil. The optimum water contents for the Atchison County soil-fly ash mixtures appear to fall on a line between 75 percent and 80 percent saturation. In the case of increased compactive effort on unstabilized soils, the optimum water contents fall on a line that is typically near 80 percent saturation (Coduto, 1999).

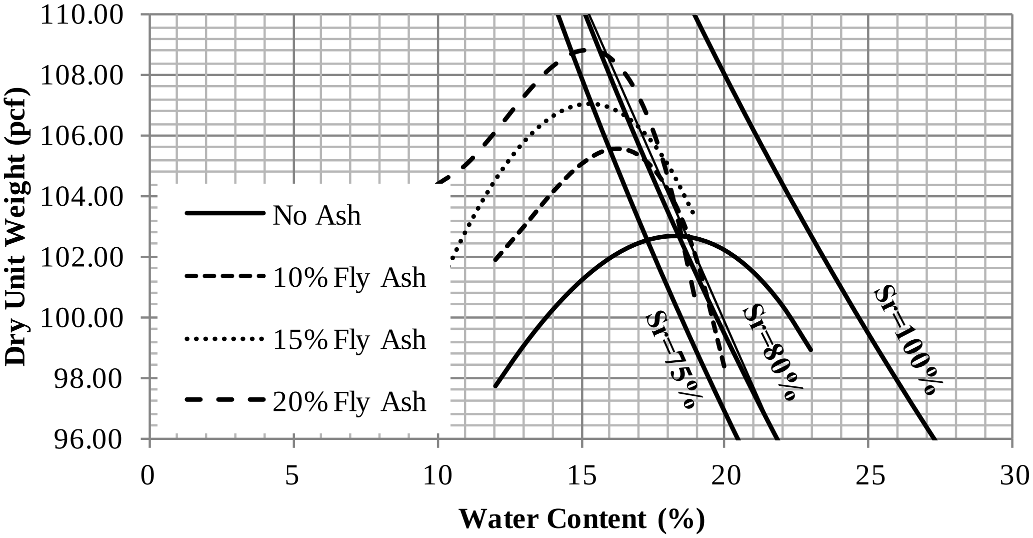


Figure 4.5 Comparison of Proctor results with lines of constant saturation lines for Atchison County soil with and without fly ash.

The compaction behavior of the Putnam County soil-fly ash mixtures is plotted with lines indicating degree of saturation in Figure 4.6. The line of optimums for the Putnam mixtures falls in between 80 percent and 90 percent saturation. While the optimum water content for Atchison County soil and soil-additives mixtures occurred near a constant line of saturation, the optimum water content of the unstabilized Putnam County soil did not fall onto the same line of optimums as the soil-additive mixtures. Identifying the specific reasons for the differences in the compaction behavior of the soils is beyond the scope of this research.

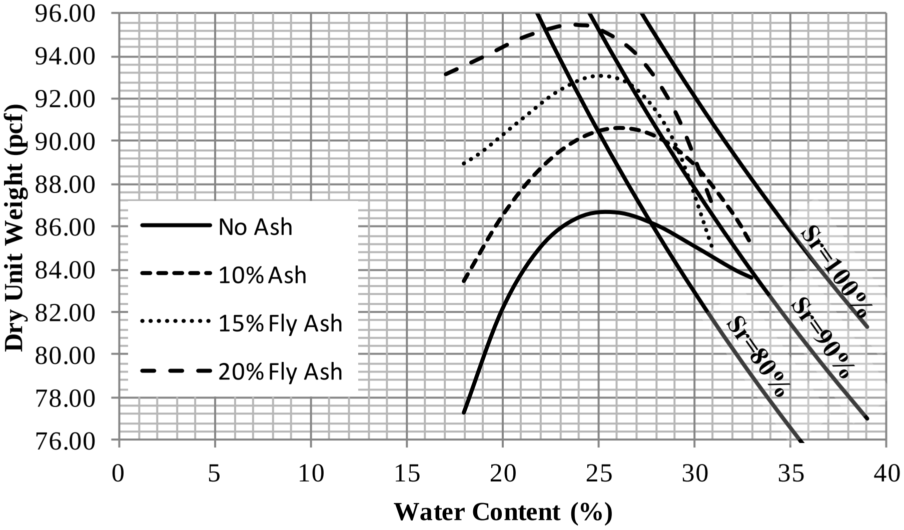


Figure 4.6 Comparison of Proctor results with lines of constant saturation for Putnam County soil with and without fly ash.

4.3 Small-Strain Young's Modulus Results

The results from free-free resonant column testing, showing modulus change as a function of time, are presented in this section. Results are presented in Section 4.3.2 from measurements performed on each soil with no additives over a range of compaction water contents. Section 4.3.3 presents results from measurements performed on soil specimens mixed with 10 percent, 15 percent and 20 percent fly ash by weight. Results from mixtures of 4 percent and 8 percent lime kiln dust by weight are presented in Section 4.3.4. Section 4.3.5 presents the modulus-time results from measurements performed after an intentional delay between soil mixing and compaction of the samples to represent possible field construction conditions.

4.3.1 Presentation of Modulus Results

Free-Free Resonant Column (FFRC) testing allows the measurement of three wave velocities (shear, constrained compression, and unconstrained compression) quickly and efficiently, as described in the previous chapter. The three velocities can be used to calculate three moduli (the shear modulus, G; the constrained modulus, M; and Young's modulus, E), as shown in Eq. 3.5 to 3.7. The three moduli are all related by Poisson's ratio in an isotropic material. The observed trends in modulus change with time are expected to be consistent among these measurements. In this study, all three velocities (and corresponding moduli) were measured, and in many cases showed good agreement in the measured trends (especially between E and G) as shown in Figure 4.7

It should be noted that due to time constraints it was not possible to perform multiple measurement for each soil/additive/water content condition. Therefore, a rigorous statistical analysis of this data was not possible. However, measurements were made on four Atchison County soil specimens prepared to the same water content and fly ash mixture. The modulus results indicated a coefficient of variation (COV) of about 10 percent using FFRC. In assessing the results, the measurement variability for all cases was assumed to be in the same range as this measured value.

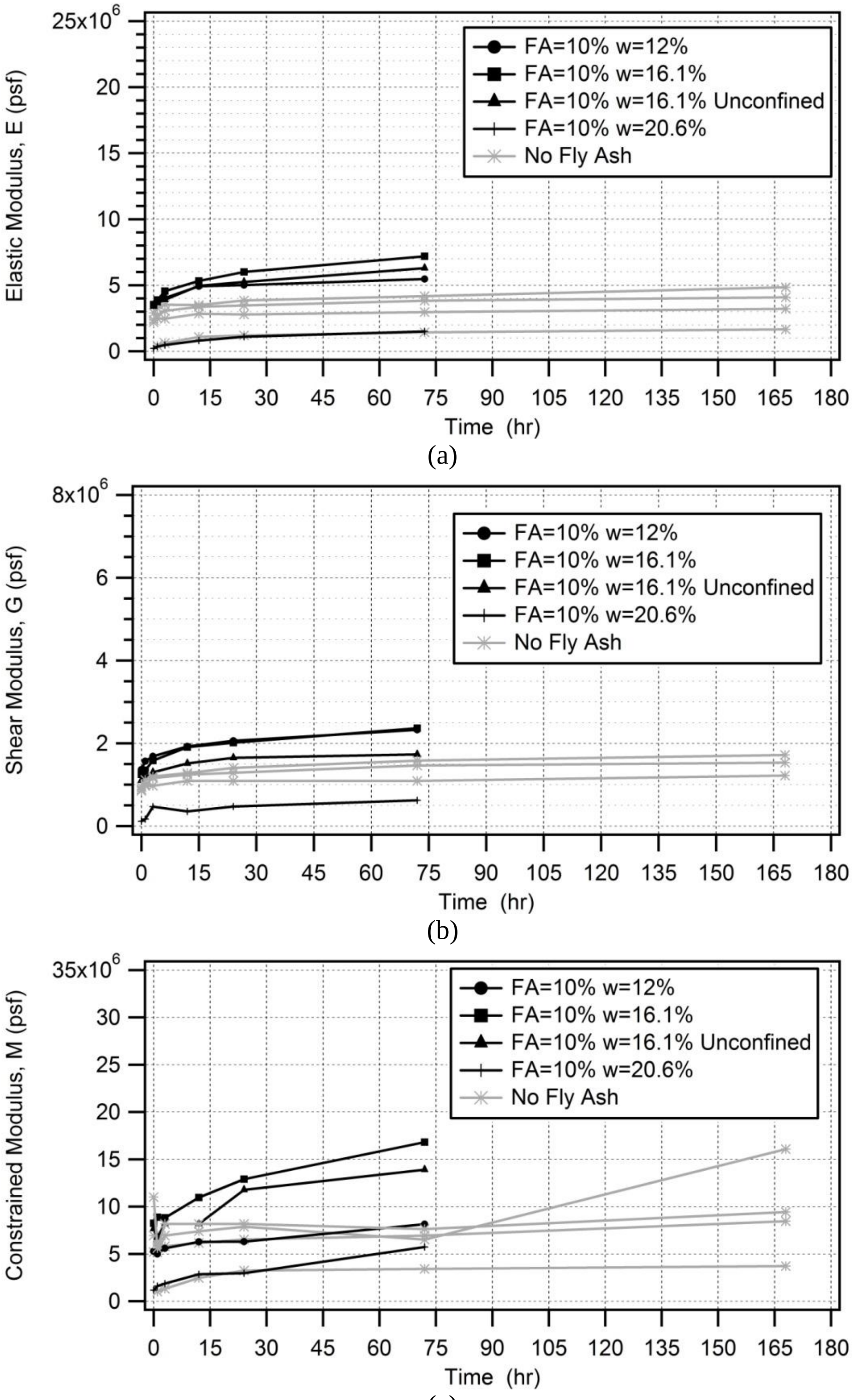


Figure 4.7 Measurement agreement among (a) Young's modulus, (b) shear modulus, and (c) constrained modulus performed for Atchison County soil with 10% fly ash

There were differences in the measured trends among the three modulus measurements for several soil-additive mixtures. These changes can most likely be attributed to problems in the measurement. The shear modulus (G), and the constrained modulus (M) measurements have the greatest potential for variability due to measurement errors. The shear modulus requires solid coupling of the end caps with the specimen to ensure that the torsional motion generated on the cap is transferred to the specimen and measured correctly on the opposite end. The constrained modulus test requires the operator to pick arrival times, which can lead to variability in results because of subjectivity and ambiguity in the arrival time picks.

To simplify the presentation and discussion of results only the Young's modulus values are used because these values appeared to be the most consistent and reliable. Although not presented in this chapter, all results of G and M values are presented in Appendix B.

4.3.2 Soils with No Additive

The measured modulus-time relationships for tests performed on the Atchison County soil are presented in Figure 4.8 Optimum water content for this case was about 18 percent, as shown in Figure 4.1 and Table 4.1, so the measurements shown in Figure 4.8 include samples compacted wet, dry and near the optimum water content. The results from measurements performed on the Putnam County soil are presented in Figure 4.9. Optimum water content for this case was about 25 percent, as shown in Figure 4.1 and Table 4.1. The lack of samples dry of optimum for the Putnam County soil was due to an oversight in not accounting for the natural water content of the soil when preparing the samples. The generally flat trend with time observed in Figures 4.8 and 4.9 is expected for these samples prepared with no additives because the samples undergo no change in confinement or water content.

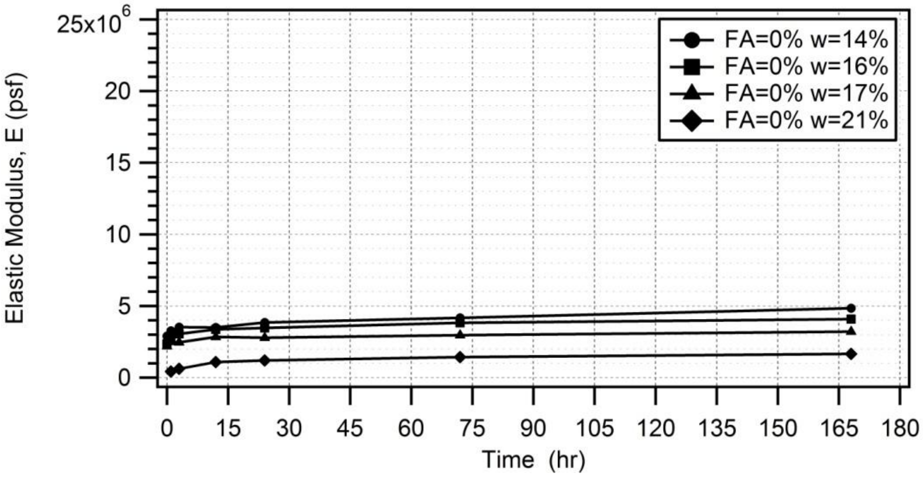


Figure 4.8 Change in Young's modulus with time for samples of Atchison County soil with no additive prepared wet, dry and near the optimum water content

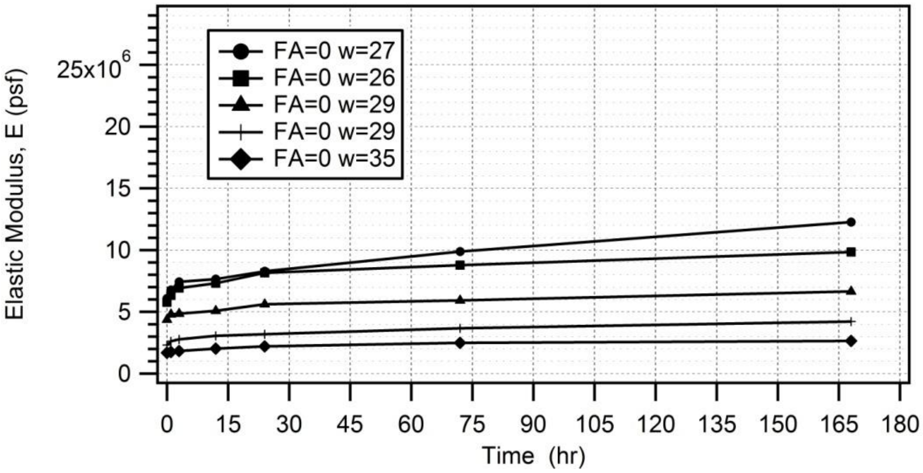


Figure 4.9 Change in Young's modulus with time for samples of Putnam County soil with no additive prepared wet of and near the optimum water content

4.3.3 Soils with Fly Ash

Modulus-time relationships for the Atchison County soil mixed with fly ash are shown in Figure 4.10 (a), (b), and (c), for 10 percent, 15 percent and 20 percent fly ash added, respectively. The results from the measurement of Atchison County soil with no additive are also presented in these figures for comparison. Note that the range of water contents for testing of each sample was chosen to sample conditions wet and dry of optimum. As previously shown in Figure 4.1 and Table 4.1, the optimum water content decreased with the addition of fly ash; dropping from 18 percent for the soil alone to 16 percent, 15 percent, and 15 percent for the 10 percent, 15 percent and 20 percent fly ash cases, respectively. The results of the soil-fly ash mixtures are only reported for times up to 3 days, as compared to 7 days for the soil-only measurements, due to problems with the seven day readings for some of the measurements.

Modulus-time relationships for the Putnam County soil mixed with fly ash are shown in Figure 4.11 (a), (b), and (c) for 10 percent, 15 percent and 20 percent fly ash added, respectively. The results from the measurement of Putnam County soil with no additive are also presented in these figures for comparison. Water content values were chosen to sample conditions wet and dry of optimum. Recall that the optimum water content remained nearly unchanged with the addition of fly ash for the Putnam County soil, changing from 25 percent for the soil alone to 26 percent, 25 percent, and 24 percent for the 10 percent, 15 percent and 20 percent fly ash cases, respectively (Figure 4.2 and Table 4.2).

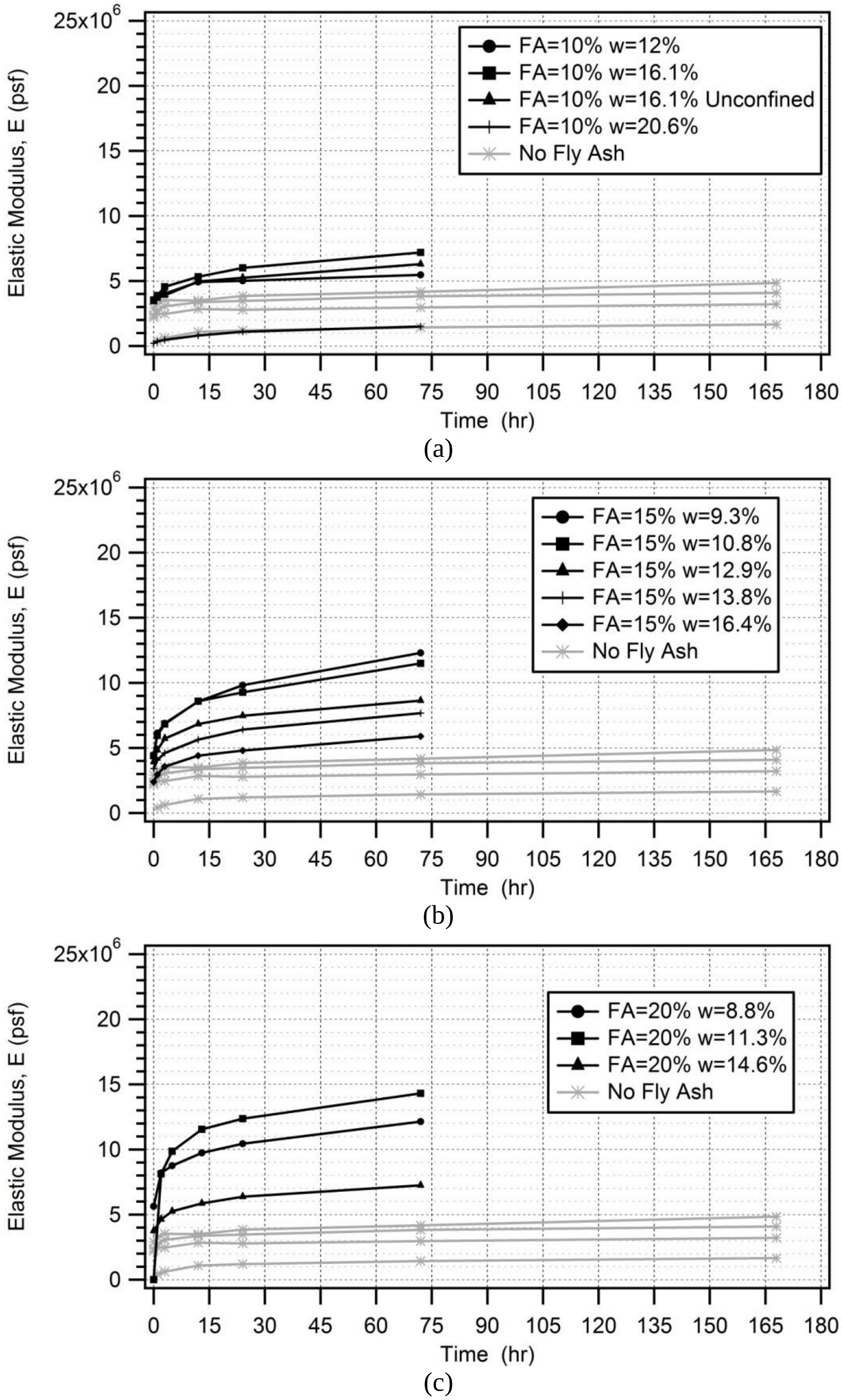
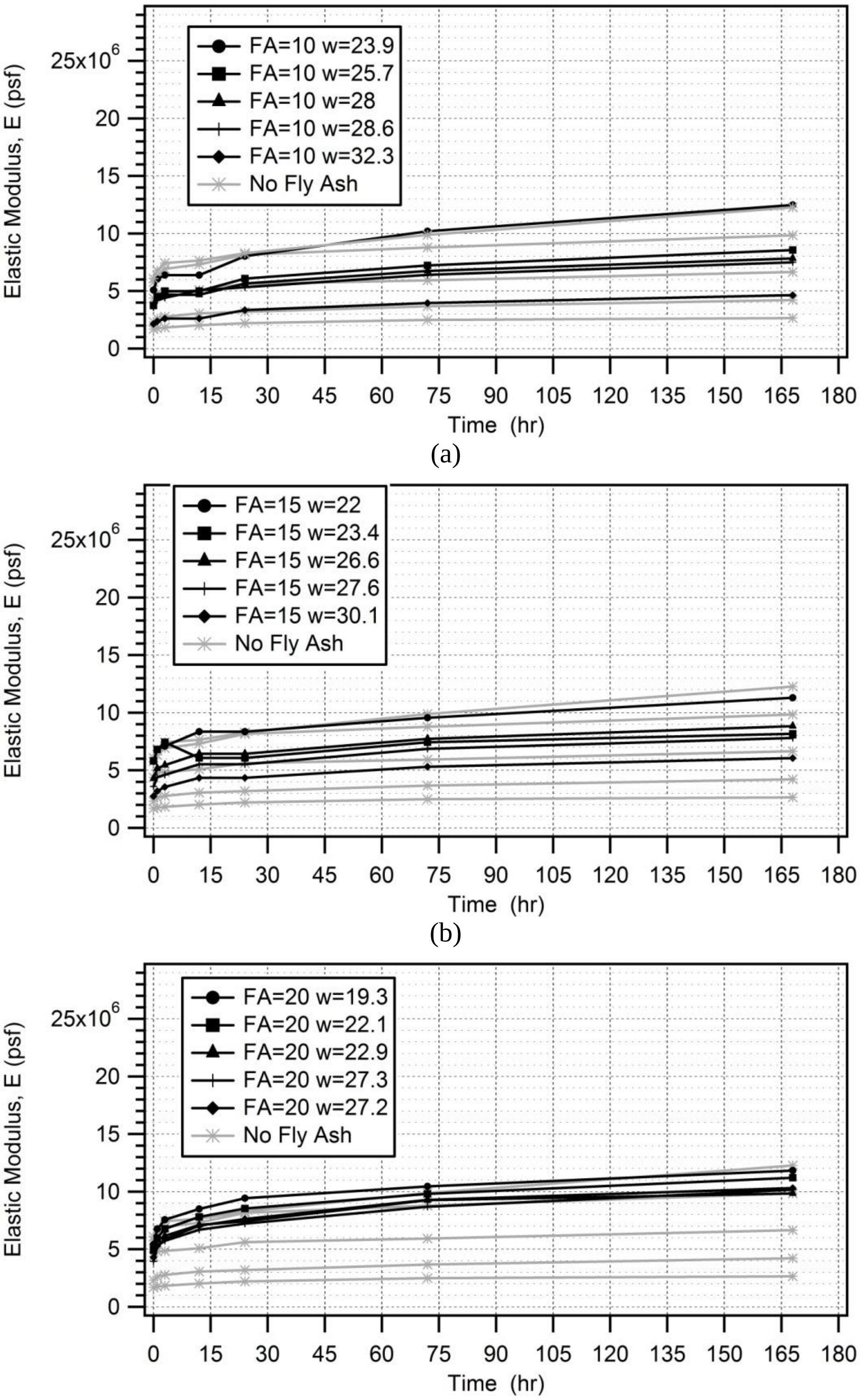


Figure 4.10 Change in Young's modulus with time for Atchison County soil with (a) 10% fly ash, (b) 15% fly ash, and (c) 20% fly ash. Results from soil with no fly ash (Figure 4.6) are also shown for comparison.



(a)

(b)

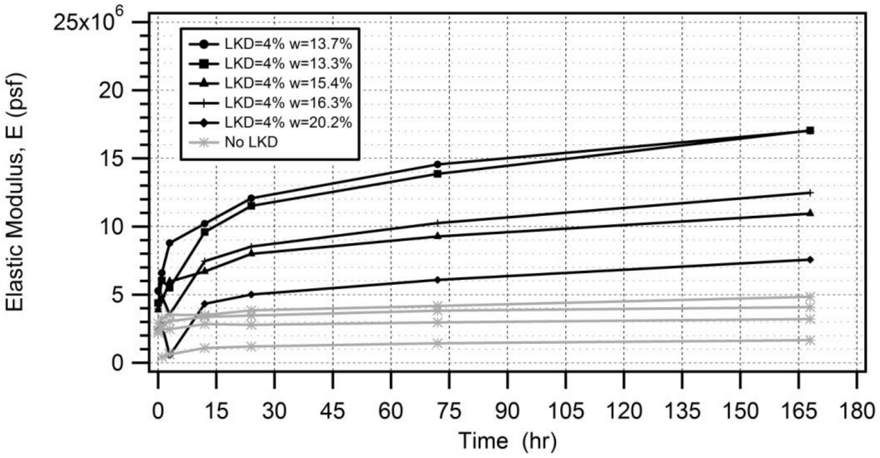
(c)

Figure 4.11 Change in Young's modulus with time for Putnam County soil with (a) 10% fly ash, (b) 15% fly ash, and (c) 20% fly ash. Results from soil with no fly ash (Figure 4.7) are also shown for comparison.

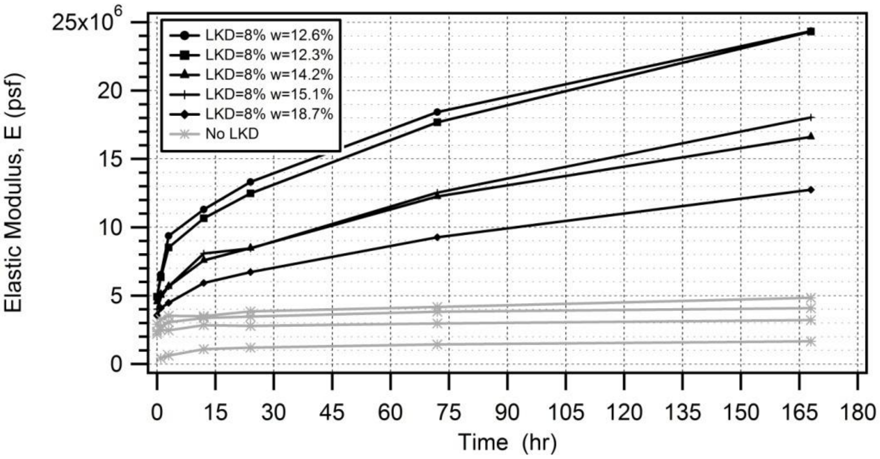
4.3.4 Soils with Lime Kiln Dust

Modulus-time relationships for the Atchison County soil mixed with lime kiln dust are shown in Figure 4.12 (a) and (b) for 4 percent and 8 percent lime kiln dust added by weight, respectively. The results from the measurement of Atchison County soil with no additive are also presented in these figures for comparison. Water contents were chosen to sample conditions from dry to wet of optimum. Recall that the change in optimum water content for the Atchison County soil was small; changing from 18 percent for the soil alone to 17 percent for both the 4 percent and 8 percent lime kiln dust mixtures, respectively (Figure 4.3 and Table 4.1).

Modulus-time relationships for the Putnam County soil mixed with lime kiln dust are shown in Figure 4.13 (a) and (b), for 4 percent and 8 percent lime kiln dust added, respectively. The results from the measurement of Putnam County soil with no additive are also presented in these figures for comparison. Water contents were chosen to sample conditions from dry to wet of optimum; however, an error in the calculation of water to add to the samples resulted in most points being wet of optimum. Recall that the optimum water content for the Putnam County soil changed from 25 percent for the soil alone to 27 percent and 29 percent for the 4 percent and 8 percent lime kiln dust mixtures, respectively as shown in Figure 4.4 and Table 4.2.

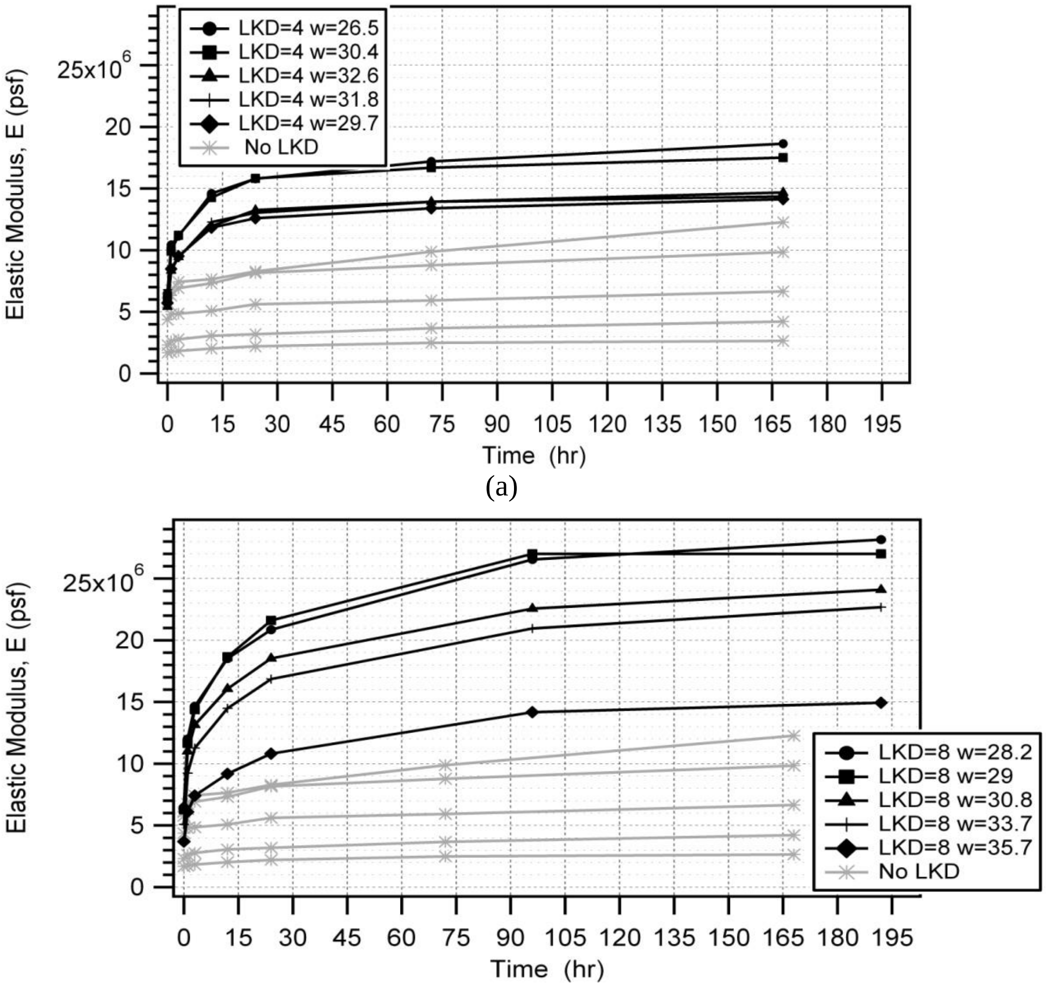


(a)



(b)

Figure 4.12 Change in Young's modulus with time for Atchison County soil with (a) 4% lime kiln dust and (b) 8% lime kiln dust. Results from soil with no fly ash (Figure 4.6) are also shown for comparison.



(a)

(b)

Figure 4.13: Change in Young's modulus with time for Putnam County soil with (a) 4% lime kiln dust and (b) 8% lime kiln dust. Results from soil with no fly ash (Figure 4.7) are also shown for comparison.

4.3.5 Compaction Delay

Additives were combined with soil-water mixtures and allowed to mellow for different time periods before compaction and FFRC testing to evaluate the effects of compaction delay. These tests were performed only on Atchison County soil with mixture ratios of 15 percent of fly ash and 8 percent lime kiln dust due to time constraints. Samples were prepared in one large batch per mixture so that the water content was at the optimum (determined from Proctor tests) and remained consistent for all specimens. For comparison, the 30 minute delay before compaction is considered to be the standard for "no delay" because it would be difficult to apply fly ash, mix with the soil, and compact the mixture in less than 30 minutes using current construction methods. The results from testing of the fly ash samples are presented in Figure 4.14.

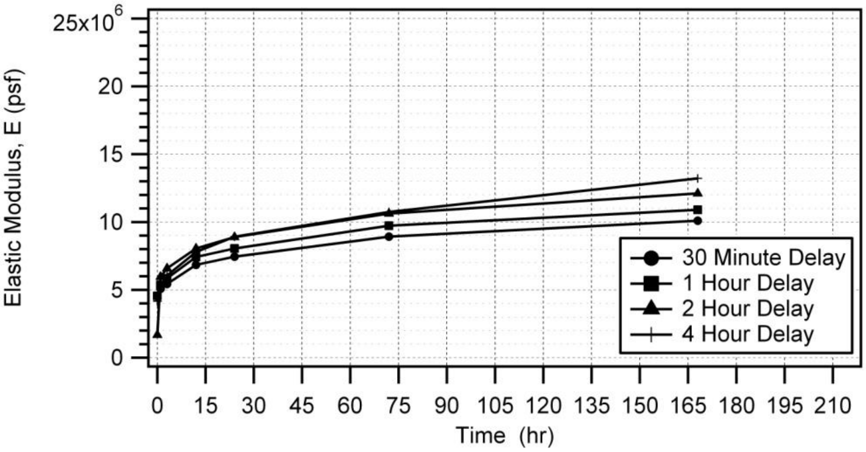


Figure 4.14 Change in Young's modulus with time for Atchison County soil prepared with intentional delay between mixing and compaction for 15% fly ash specimens prepared at optimum water content.

The Atchison County soil experienced an increase in modulus with increased compaction delay when mixed with fly ash, as compared to the 30-minute delay moduli values. While initial reactions cause an increase in modulus over unstabilized soils, the long-term reactions provide the cementation of the additive and soil structure, as discussed in Chapter 2. Taking this into account, the observed compaction delay behavior is reasonable as the pozzolanic reactions that occur require time to increase the pH and begin to stabilize the soil. The trend needs to be further evaluated with more tests at longer times and different mixtures to determine the optimum temper time.

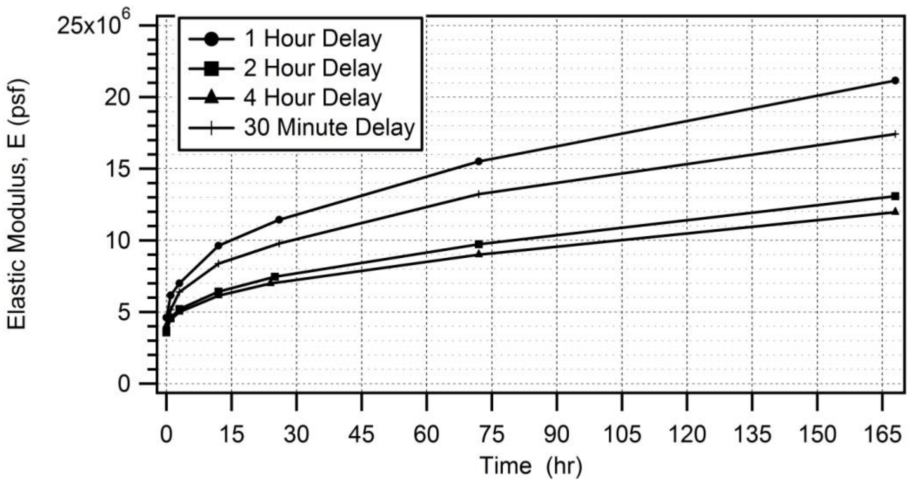


Figure 4.15 Change in Young's modulus with time for Atchison County soil prepared with intentional delay between mixing and compaction for 8% lime kiln dust specimens prepared at optimum water content.

The soil-lime kiln dust mixture, presented in Figure 4.15, showed an increase in modulus with a one hour delay, but a decrease in modulus after two and four hours of compaction delay. This seems to indicate that the lime kiln dust reacts with soil much faster than fly ash. Once reactions have occurred during tempering, the soil-additive structure could be broken during compaction resulting in lower modulus values. These results underscore the effect construction practices can have on material properties and performance.

4.4 Impact of Compaction Water Content on Small-Strain Modulus

One of the most influential parameters in construction and performance of compacted soil is the water content of the soil at the time of compaction. It is usually considered desirable to place the soil near the optimum water content to achieve the highest possible density for a given level of compaction energy. The soil may be placed wet or dry of optimum in certain applications to achieve desirable parameters, such as decreased permeability or increased strength. Water contributes to the performance of stabilized soil in two ways. First, as with unstabilized soils, the amount of water added to the soil influences the maximum density that can be achieved; higher density generally corresponds to desirable engineering properties and performance. The water also contributes to the chemical reactions between the soil and the additive, which affects the engineering properties.

An analysis of modulus results from specimens compacted over a range of different water contents was performed to better understand how soil stiffness varies with changes in compaction water content for different amounts of additive (mixture percentage) and type of additive used. The results from Section 4.3 are alternately presented in terms of modulus versus water content, along with the compaction data.

The results from measurements performed on the Atchison County soil are shown in Figures 4.16 through 4.21, and the results from measurements performed on the Putnam County soils are shown in Figures 4.22 through 4.27. Measurements on the soils with no additive showed continuously increasing modulus values with decreasing water content (Figures 4.16 and 4.22). The highest modulus values do not occur at the optimum water content, but instead continue to increase to higher values dry of optimum. This observation is consistent with findings from Nazarian et al. (2002), who showed that the optimum water content for modulus measured on unstabilized compacted fine grained soils using FFRC was typically around 4 percent dry of the optimum water content. The tests performed in this study did not extend below 4 percent dry of optimum so it is not possible to identify a peak in modulus values. Results at additional water contents, lower than those measured in this study, would be necessary to evaluate the entire shape of a modulus versus water content curve.

A review of the literature did not find any studies of small-strain modulus changes for stabilized soils compacted over a range of water contents. It was thought that the modulus trends for stabilized soils could be quite different than what was observed for unstabilized soils. Specifically, it was anticipated that there would be less change in modulus (as compared to unstabilized soil) as water content changed from dry to wet, due to the dominating influence of the cementing bonds on the measured modulus. It was also thought that the modulus may decrease dry of optimum with high percentage of additive and low water contents resulting in unreacted additive. Instead, the trends in modulus change with water content were generally quite similar to the unstabilized case. Figures 4.16 to 4.21 show measurements performed on the Atchison County soil in the water content range from ±4 percent of optimum. The trend for both fly ash and lime kiln dust was generally decreasing modulus with increasing water content. The only exception was for the 10 percent fly ash mixture where the modulus 4 percent dry of optimum was lower than the values measured at optimum. The 20 percent fly ash mixture also showed lower values but these were under very dry conditions with water contents nearly 6 percent below optimum.

The relative changes in modulus values, from wet to dry conditions, were also similar to those observed for the unstabilized specimens (factors of 2 to 3 between dry and wet values). The measurements performed on the Putnam County soil did not extend as far to the dry side of optimum as the Atchison measurements; however, similar trends of decreasing modulus with increasing water content were also observed over this smaller water content range.

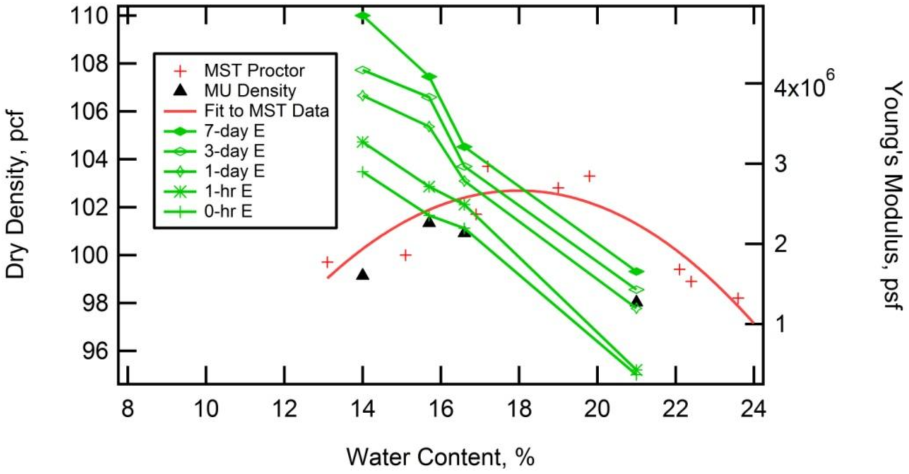


Figure 4.16 Change in Young's modulus with water content; Atchison County soil with no additive

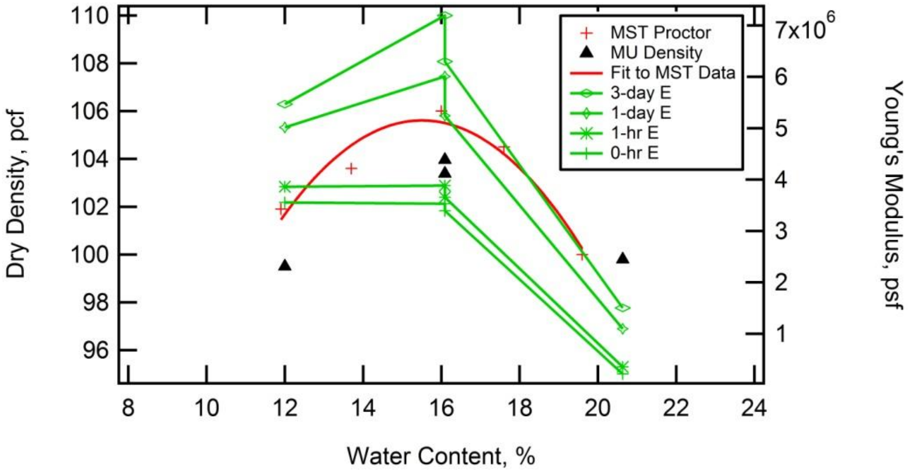


Figure 4.17 Change in Young's modulus with water content; Atchison County soil with 10 percent fly ash

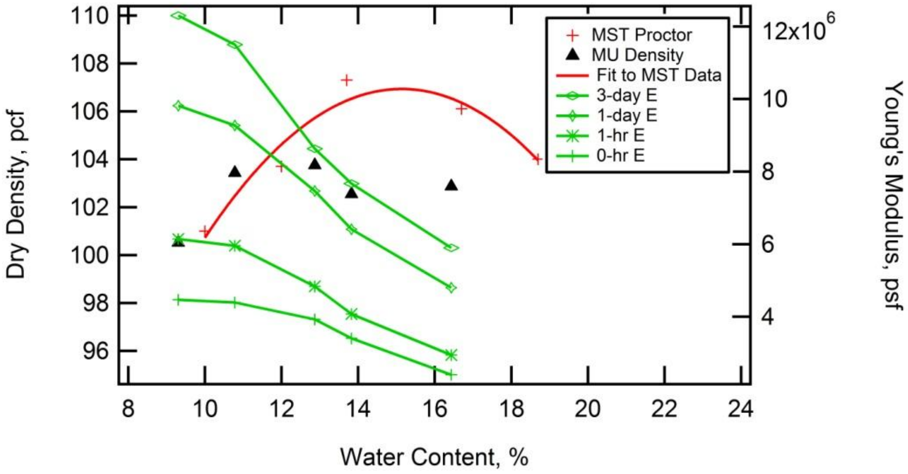


Figure 4.18 Change in Young's modulus with water content; Atchison County soil with 15 percent fly ash

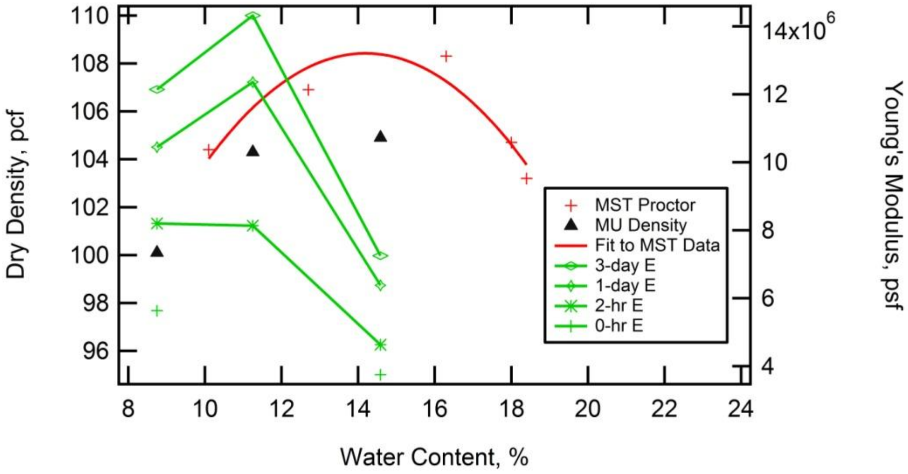


Figure 4.19 Change in Young's modulus with water content; Atchison County soil with 20 percent fly ash

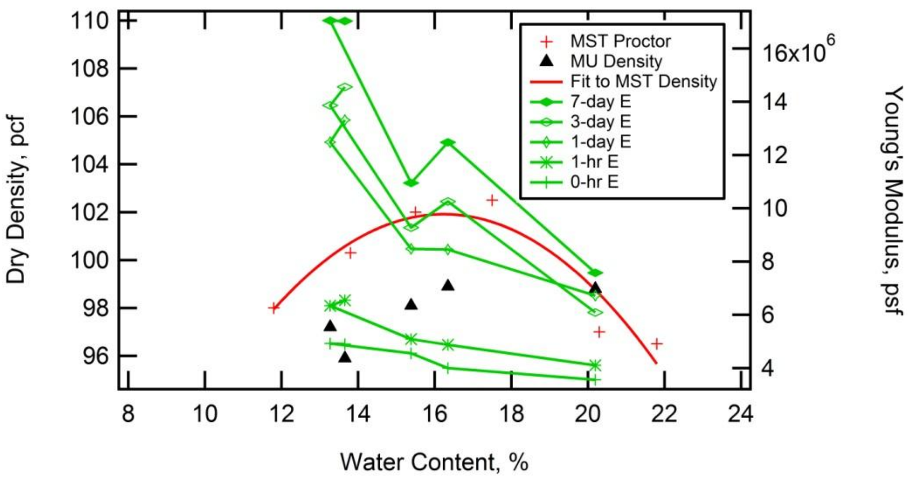


Figure 4.20 Change in Young's modulus with water content; Atchison County soil with 4 percent lime kiln dust

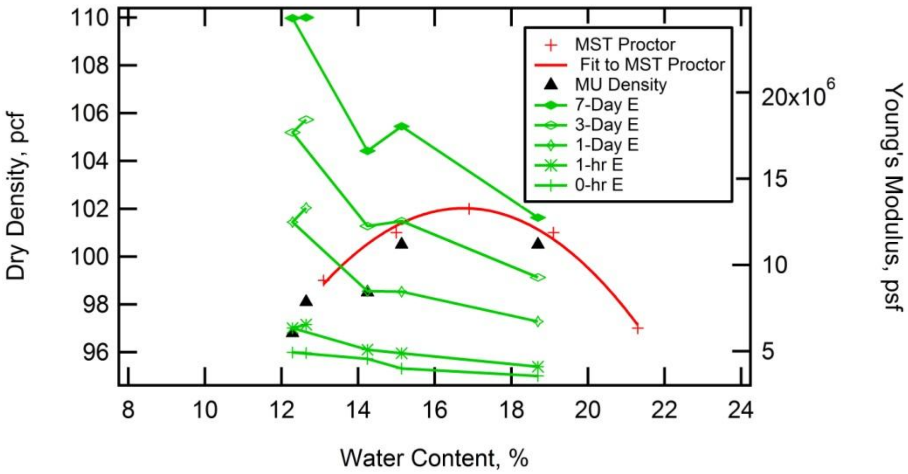


Figure 4.21 Change in Young's modulus with water content; Atchison County soil with 8 percent lime kiln dust

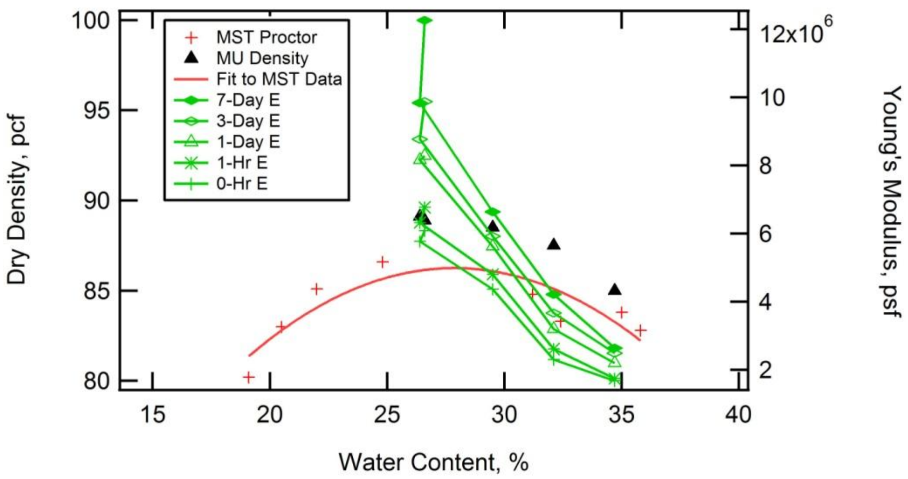


Figure 4.22 Change in Young's modulus with water content; Putnam County soil with no additive

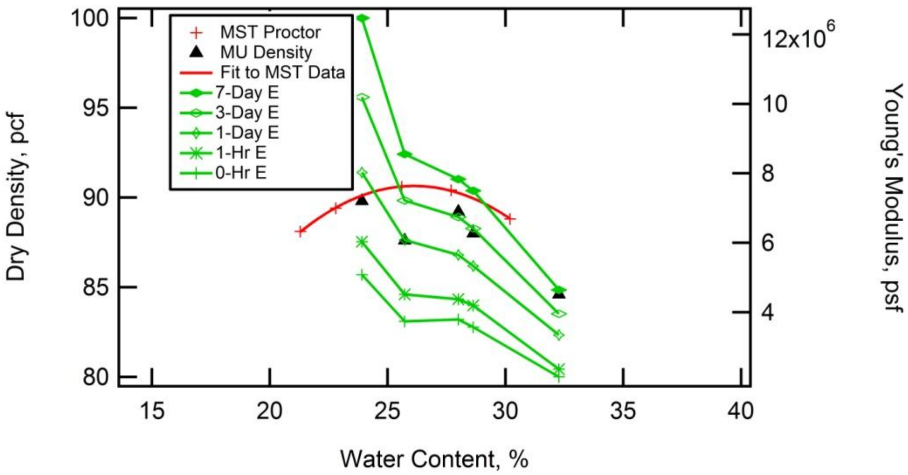


Figure 4.23 Change in Young's modulus with water content; Putnam County soil with 10 percent fly ash

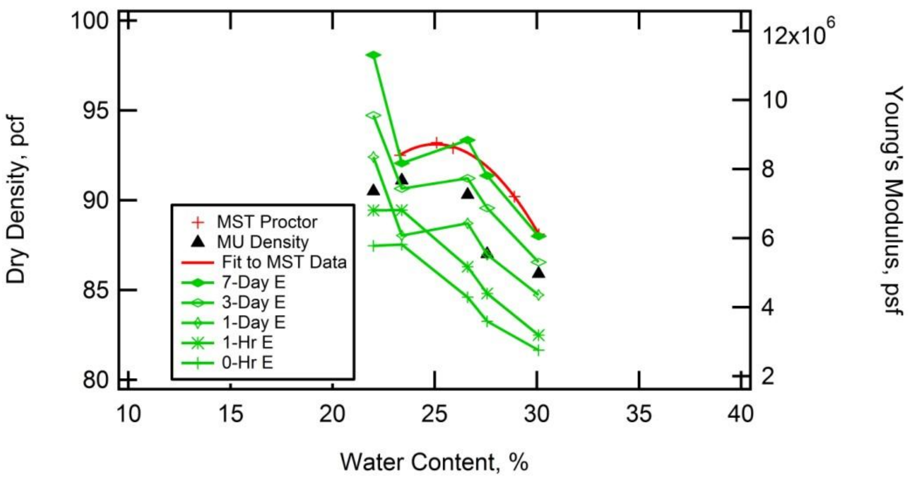


Figure 4.24 Change in Young's modulus with water content; Putnam County soil with 15 percent fly ash

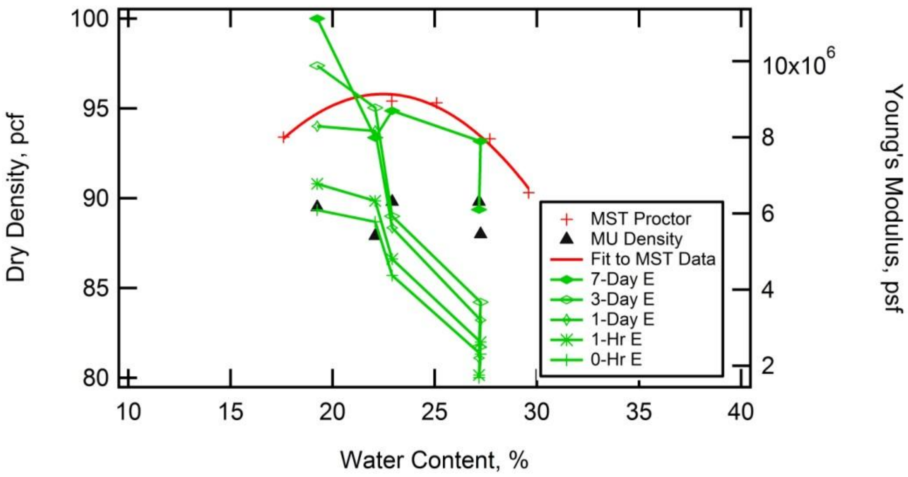


Figure 4.25 Change in Young's modulus with water content; Putnam County soil with 20 percent fly ash

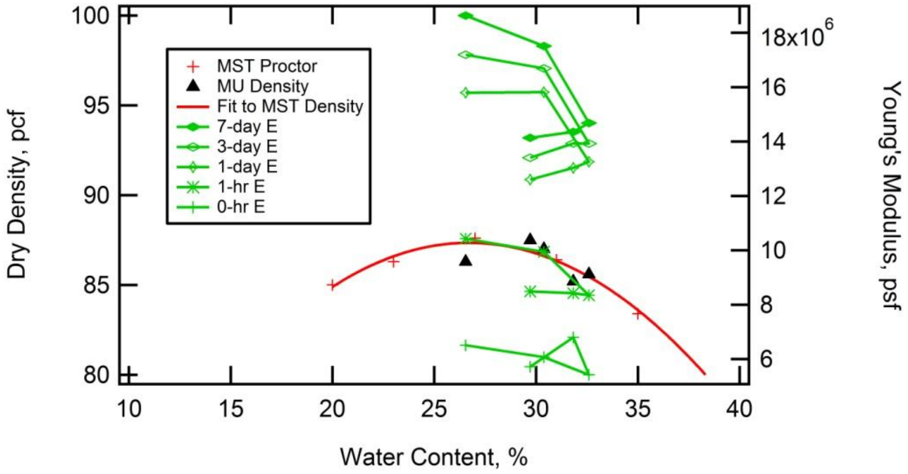


Figure 4.26 Change in Young's modulus with water content; Putnam County soil with 4 percent lime kiln dust

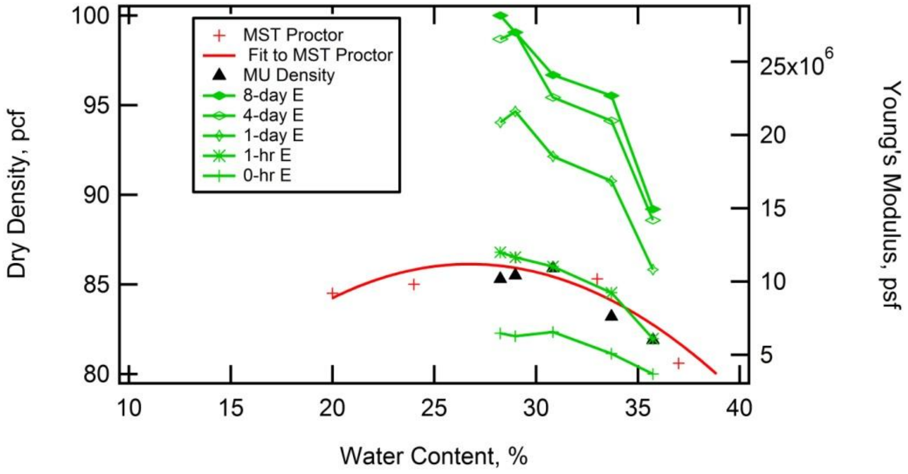


Figure 4.27 Change in Young's modulus with water content; Putnam County soil with 8 percent lime kiln dust

The results presented above illustrate a few important points. First, measurement of modulus alone cannot be used to identify the conditions under which a subgrade soil was placed. For example, similar 1-day modulus values would be measured for a soil placed with 15 percent fly-ash at a water content that was slightly wet of optimum

(Figure 4.18) as a soil placed without an additive that was 4 percent dry of optimum (Figure 4.16). Therefore, for field applications, knowledge of the compaction water content is needed to interpret the modulus measurements. Secondly, the trends for both soils (and both additives) indicate that the optimum water content that corresponds to maximum density is not the water content that produces the highest stiffness. Although a higher modulus can be achieved at lower water contents, this may not be desirable for the long-term performance of the stabilized subgrade due to problems such as tension cracks being formed due to soil drying, or decreased performance under cyclic saturation (Misra, 1998).

The question of the optimum placement water content for long-term performance of stabilized subgrade requires further study and was not within the scope of this work. Common current practice and the current specification used by MoDOT require placement within ±2 percent of optimum (MoDOT, Item P-152). The remaining analyses and discussion will focus on modulus values measured within that range of compaction water contents.

4.5 Modulus Change due to Additive Stabilization

One of the objectives of this work was to quantify the potential benefit that can be derived from using soil additives on Missouri subgrade soils. Small-strain modulus values from samples compacted near the optimum water content (± 2 percent) are presented in bar graph form to illustrate changes in small-strain modulus with both time and percentage of additive used. Measurement data points that fell within ±2 percent of optimum and ≥ 95 percent of the maximum dry density (γ_{dmax}) were used. If multiple points were in this range the average value was presented.

Young's modulus results for each soil-additive mixture are presented in Figures 4.28 and 4.30, for the Atchison and Putnam County soils respectively. Figures 4.29 and 4.31 present the same data plotted as the ratio of the stabilized soil modulus to the unstabilized case, for the Atchison and Putnam County soils respectively. For example, to calculate the ratio of the three-day modulus values, the modulus measurement from the soil/additive sample three days after the initial measurement is divided by the modulus of the unstabilized soil measured three days after the initial measurement. The 0-hr value (initial measurement time) presented in these figures represents measurements that were performed approximately 30 to 40 minutes after the soils were first mixed, as time was required for compaction and preparation of the specimens for testing.

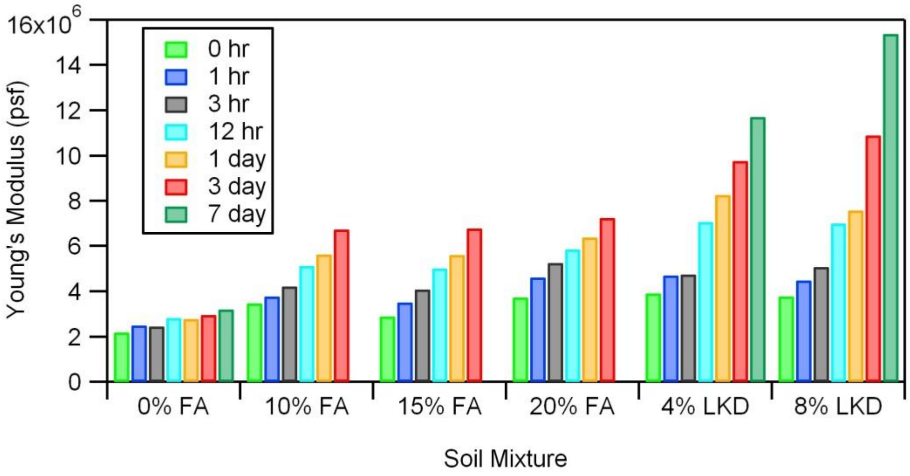


Figure 4.28 Change in Young's modulus of Atchison County soil for different soil/additive mixtures compacted near the optimum water content

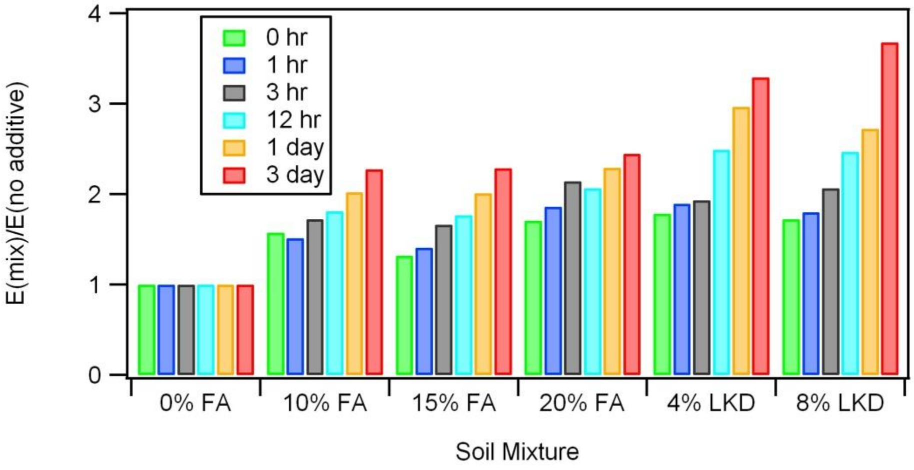


Figure 4.29 Ratio of Young's modulus from stabilized soils to Young's modulus of unstabilized soil for different soil/additive mixtures of Atchison County soil compacted near the optimum water content

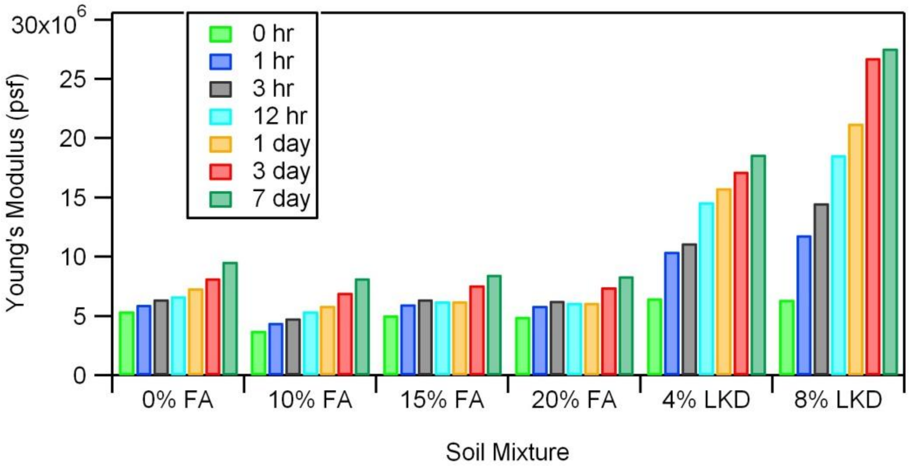


Figure 4.30 Change in Young's modulus of Putnam County soil for different soil/additive mixtures compacted near the optimum water content

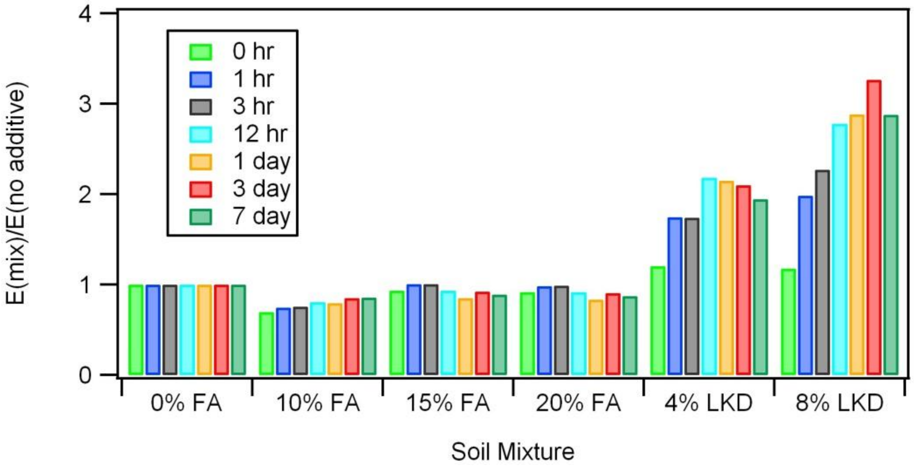


Figure 4.31 Ratio of Young's modulus from stabilized soils to Young's modulus of unstabilized soil for different soil/additive mixtures of Putnam County soil compacted near the optimum water content

The results presented in Figure 4.28 show small changes in the modulus of the unstabilized Atchison County soil over the seven day period, increasing from 2.2 x10^6 to 3.2 x10^6 psf. However, the addition of fly ash to the Atchison County soil had an immediate impact on the small-strain modulus values. The 0-hr modulus values increased from about 2.2 x10^6 psf to as much as 3.5 x10^6 to 3.9 x10^6 psf after the addition of fly ash or lime kiln dust. The modulus of the stabilized Atchison County soil continued to increase with time. At 3-days, soil with 10 percent fly ash increased by a factor of about 2.3 compared to the unstabilized soil over the same time frame (Figure 4.29). Addition of 15 percent fly ash resulted in a similar improvement in 3-day modulus of about 2.3 times the unstabilized value (Figure 4.29). The addition of 20 percent fly resulted in a slightly larger improvement in modulus with a 3-day modulus that was about 2.5 times the unstabilized value (Figure 4.29). It is interesting to note that doubling the amount of fly ash in the Atchison County soil had essentially no effect on the 3-day modulus values.

The addition of lime kiln dust likewise resulted in improvements in soil stiffness for the Atchison County soil. The immediate increase (0-hr values) in modulus was similar to the fly-ash stabilized soil with an increase of about 70 to 80 percent for both the 4 percent and 8 percent cases. The longer term improvements in modulus; however, were higher than what was achieved with the fly ash. The modulus at 3-days was about 3.3 times the unstabilized modulus for the 4 percent case and 3.7 times the unstabilized modulus for the 8 percent case. The 7-day values showed continued improvements with ratios of 3.7 and 4.8, respectively for specimens mixed with 4 percent and 8 percent lime kiln dust.

Over the 7-day period the modulus of the compacted Putnam sample with no additive increased by about 62 percent (as compared to about 45 percent for the Atchison County soil). Surprisingly, all of the fly ash mixtures exhibited modulus values that were essentially the same (or in some cases lower) as was achieved with the Putnam County soil alone. The ratios of modulus values at 7-days for the Putnam County soil were near unity for the 10 percent, 15 percent and 20 percent fly-ash mixtures. It can be concluded that the addition of fly ash had essentially no effect on the modulus of the Putnam County soil regardless of the percentage of fly ash added to the soil. This observed behavior is the focus of the second portion of this study presented in Chapter 5.

Conversely, the Putnam County soil showed increases in modulus when mixed with lime kiln dust; the three-day modulus ratios increased by factors of 2.1 and 3.3 for the 4 percent and 8 percent mixtures, respectively.

4.6.1 Comparison to Resilient Modulus Measurements

Mechanistic pavement design methods commonly used in practice currently utilize resilient modulus values. Resilient modulus testing for this project was completed by researchers at MST as part of the collaborative portion of the project. Tests were performed only on mixtures of the Atchison County soil due to time constraints for the project. Specimens for resilient modulus testing were mixed and compacted in batches and tested at 0, 1, 7, 14, and 28 days.

The resilient modulus results measured from this study are shown in Figure 4.32 and the normalized modulus values are presented in Figure 4.33. The results are from resilient modulus tests on specimens compacted near optimum (-1 to +2 percent) and tested at a standard confining stress of 41.4 kPa (close to the 5 psi confining stress used for FFRC testing) and at a deviator stress of 13.8 kPa.

It is not expected that the resilient modulus values will be the same as the small-strain values because the tests are performed at different strain levels; however, the relative changes in modulus should be similar. Table 4.3 compares the normalized modulus ratios from the small-strain measurements and resilient modulus measurements both performed 1 day after the initial readings (Figures 4.28 and 4.33). It can be observed that the normalized modulus values are in good agreement for the fly ash stabilized samples, but differ for the lime kiln dust samples (especially the 4 percent lime kiln dust mixture).

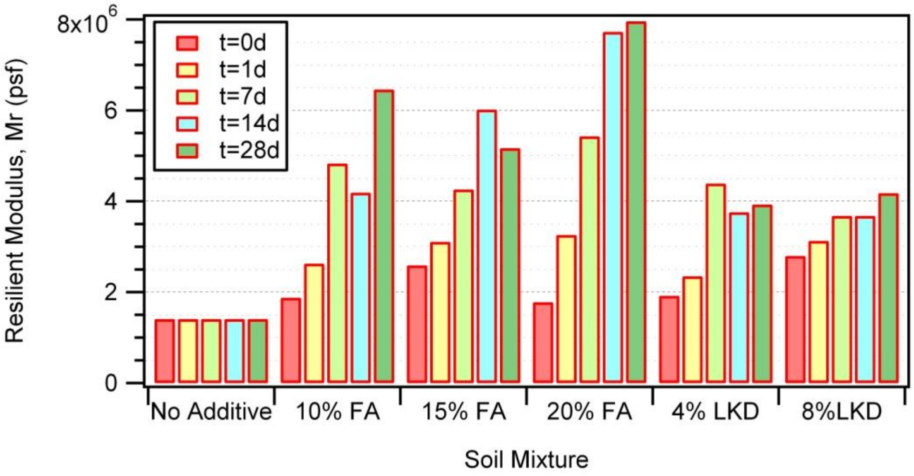


Figure 4.32 Change in resilient modulus of Atchison County soil for different soil/additive mixtures compacted near the optimum water content (tested with deviator stress of 13.8 kPa and confining pressure of 41.4 kPa)

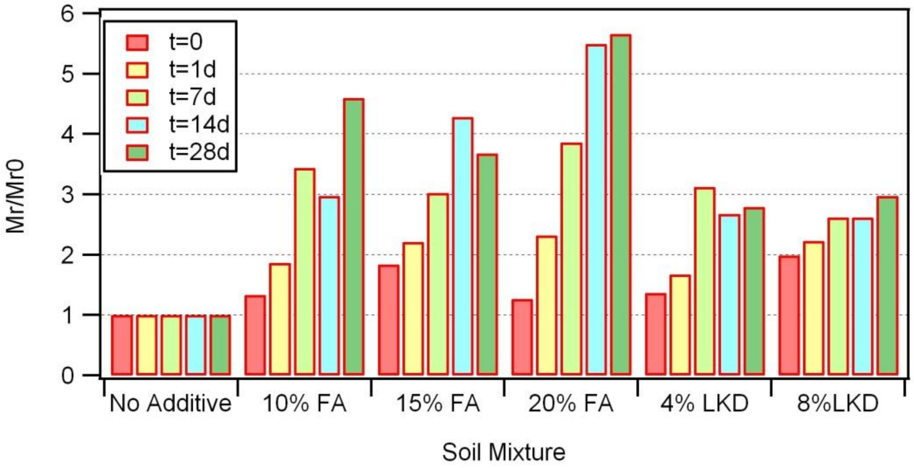


Figure 4.33 Ratio of resilient modulus from stabilized soils to resilient modulus of unstabilized soil for different soil/additive mixtures of Atchison County soil compacted near the optimum water content (tested with deviator stress of 13.8 kPa and confining pressure of 41.4 kPa)

Table 4.3 Comparison between resilient modulus and small-strain Young's modulus for Atchison County soil with and without additives performed 1-day after compaction

Mixture	Resilient Modulus Ratio	Young's Modulus Ratio
Atchison +10% FA	1.9	2.0
Atchison +15% FA	2.2	2.0
Atchison +20% FA	2.3	2.3
Atchison + 4% LKD	1.7	3.0
Atchison +8% LKD	2.2	2.7

The lower modulus ratio from the resilient modulus testing of the lime kiln dust samples could be due to the increased drying potential from the lime kiln dust causing the specimens to behave in a more brittle nature compared to fly ash specimens. Dry specimens could develop shrinkage cracks that would decrease the resilient modulus, but not the FFRC modulus, because of the small-strain nature of the FFRC measurements. The generally good agreement between the FFRC and resilient modulus measurements supports the use of small-strain velocity measurements for field quality control.

4.6 Discussion of the Use of Field Velocity Measurements for Quality Control

One of the challenges with using soil modifiers such as fly ash or lime kiln dust to improve soil properties is the need to assess the quality of the stabilized soil after it is placed. The quality of conventional compacted soils is commonly assessed using measurements of water content and dry density; however, with stabilized soils these measurements alone do not indicate the quality of the subgrade. A need exists to develop non-destructive testing (NDT) methods that can be used in the field to assess the quality of the subgrade soon after placement. One of the objectives of this project was to investigate the use of velocity-based modulus measurements for this application. Unfortunately, it was not possible to perform field studies on a stabilized soil; however, the laboratory measurements provide valuable insight into the use of velocity measurements as a means of quality control.

The modulus values presented in this study, as discussed in Chapter 3, were derived from measurements of wave velocity. Field quality control measurements would likewise use wave velocities to determine in-situ modulus values (e.g. using surface wave methods). Ideally field quality control measurements should be performed as soon after construction as possible so that remedial measures can be applied in a timely manner to avoid costly construction delays. Therefore, if velocity measurements are to be used for quality control of stabilized subgrades, the change in velocity must be measurable in the short term (ideally within hours after placement).

Figure 4.34 shows the ratio of Young's modulus values of stabilized versus unstabilized Atchison County soil over the first 3 hours after compaction. Figure 4.35 shows the same plot for the Putnam County soil. For the Atchison County soil, 1-hr modulus ratios for the fly ash samples were 1.5, 1.4 and 1.9, respectively for the 10 percent, 15 percent and 20 percent cases. The Atchison County soil mixed with lime kiln dust showed 1-hr modulus ratios of 1.9 and 1.8 for the 4 percent and 8 percent cases, respectively. Recalling that these modulus values were calculated from squaring measured velocity values (Eq 3.5), the velocity ratios for these cases are 1.22, 1.18, 1.38 for the 10 percent, 15 percent, and 20 percent fly ash samples, and 1.38 and 1.34 for the 4 percent and 8 percent lime kiln dust cases. In other words, soil with fly ash could be differentiated from soil without fly ash by measuring velocity values that were about 20 to 40 percent higher after one hour. The coefficient of variation (COV) of field SASW measurements has been shown to be between 5 percent and 10 percent (Marosi and

Hitunen, 2004). Therefore, the expected velocity changes are much greater than the measurement variability.

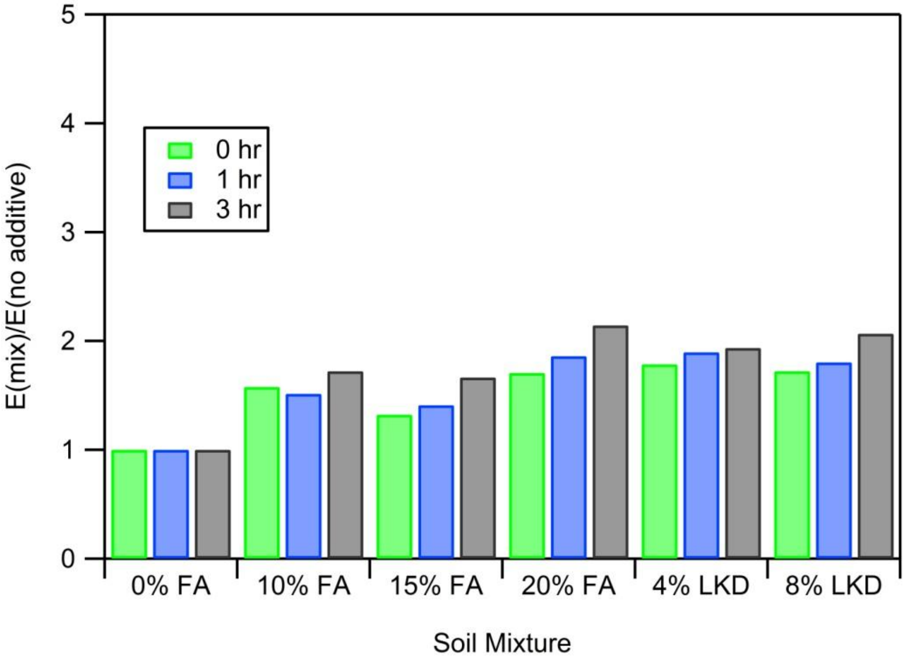


Figure 4.34 Ratio of Young's modulus from stabilized soils to Young's modulus of unstabilized soil for different soil/additive mixtures of Atchison County soil compacted near the optimum water content and measured over a time span of 3 hours

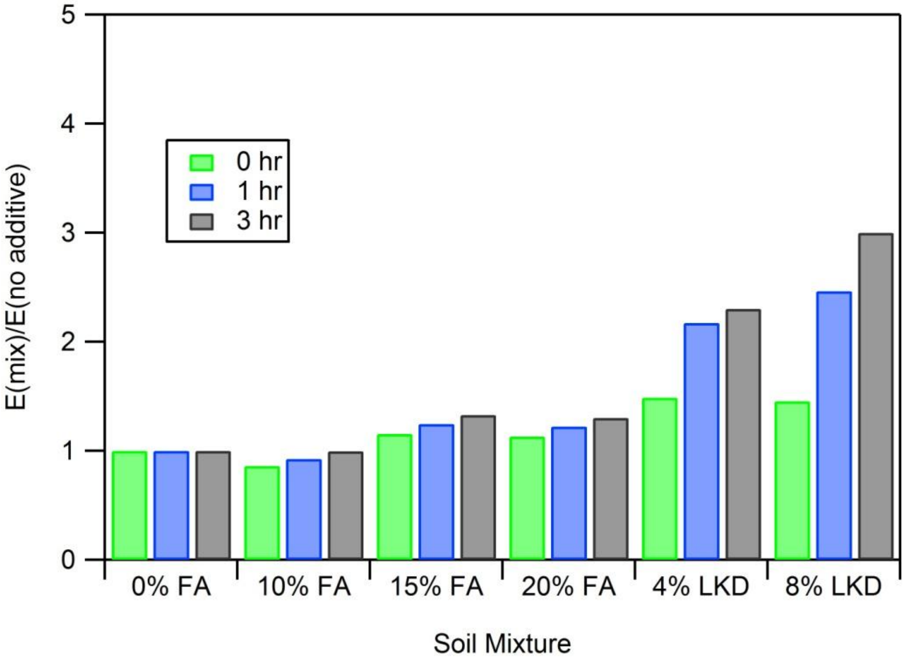


Figure 4.35 Ratio of Young's modulus from stabilized soils to Young's modulus of unstabilized soil for different soil/additive mixtures of Putnam County soil compacted near the optimum water content and measured over a time span of 3 hours

Although these results are only for a single soil/fly ash combination, the results indicate that velocity changes appear to be detectable and are greater than the variability of the field velocity measurements (although this would need to be evaluated). Based on these data, it appears that velocity could be used to differentiate soil with stabilizer from soil without stabilizer as soon as one hour after compaction. However, it is less likely that these measurements would be able to differentiate soils with different percentages of additive, as the changes in velocity are much smaller

The Putnam County soil did not react well with the fly ash so it was not possible to derive meaningful results from the velocity ratios of the samples with different fly ash percentages. However, for the lime kiln dust, the 1-hr modulus ratios were 1.9 and 2.1 for the 4 percent and 8 percent cases, which is equivalent to velocity increases of 38 percent and 45 percent. These changes, therefore, appear to be detectable using small-strain velocity measurements, such as the SASW method, in the field.

4.7 Summary

The free-free resonant column (FFRC) non-destructive testing method (NDT) was used to measure changes in small-strain Young’s modulus in the laboratory over time for a high-plasticity and a low plasticity clay. The soils were tested with and without two additives; that were fly ash and lime kiln dust. Specimens were tested at compaction water contents that were wet, dry, and near the optimum water contents determined from compaction results. The effect of compaction water content was presented and discussed for each soil and mixture combination. The effects of intentionally subjecting specimens to delays between mixture with the additive and compaction were discussed. Specimens that were within typical acceptable compaction range (±2 percent of optimum, ≥ 95

percent maximum dry density) were then analyzed to quantify the gain in modulus resulting from additive-stabilization. These results were compared to resilient modulus results for similar mixtures performed one day after compaction. Finally, a discussion of the viability of the use of wave-based NDT methods for quality control of stabilized soils was presented.

5 Results and Discussion - Influence of Soil Composition

5.1 Introduction

A study was performed to better understand the effects of soil composition on the efficacy of calcium-rich additive stabilization. Particle size analyses, X-ray diffraction, scanning electron microscopy, and cation exchange tests were performed on unmodified soils, as well as soils mixed with 15 percent fly ash and 8 percent lime kiln dust. The results are presented and discussed in this chapter. Specimens used for composition and chemical testing were compacted and tested using FFRC for seven days before being disassembled and air dried overnight. The dried mixtures were then ground, using a mortar and pestle, and were passed through a number 10 sieve (2.00 mm).

5.2 Physical Properties

An analysis of soil particle size composition was performed to better understand the relative clay content in both soils. The pipette particle size analysis, specified as 3A1a6b in the USDA Soil Survey Laboratory Methods Manual (USDA, 2004), was used to determine the relative amounts of sand, silt, and clay-sized particles. The results are presented in Tables 5.1 and 5.2.

Table 5.1 Particle-size distribution for soil and soil-additive mixtures, determined using pipette test method 3A1a6b from USDA (2004)

	% of Total			
	<.002 mm	.002 - .05	.05 - 2.00	
Sample No.	Clay	Silt	Sand	Textural Class
Atch-Water	16.4	60.6	23.0	Silt Loam
Atch-Ash	8.0	65.4	26.6	Silt Loam
Atch-LKD	7.0	50.3	42.7	Silt Loam
Putn-Water	55.2	40.8	4.0	Silty Clay
Putnam-Ash	7.9	76.0	16.1	Silt Loam
Putnam-LKD	22.1	57.4	20.5	Silt Loam
Ash	0.2	83.4	16.4	Silt
LKD	3.2	45.0	51.8	Fine Sandy Loam

Table 5.2 Additional particle-size results for soil and soil-additive mixtures, determined using pipette test method 3A1a6b from USDA (2004)

	% of Silt		% of Sand				
	.002 - .02	.02 - .05	.05 - .10	.10 - .25	.25 - .50	.50 - 1.00	1.00 - 2.00
Sample No.	Fine	Coarse	V Fine	Fine	Medium	Coarse	V Coarse
Atch-Water	20.6	40.0	17.6	2.5	1.5	0.9	0.5
Atch-Ash	27.5	37.9	17.8	3.9	2.8	1.5	0.6
Atch-LKD	19.8	30.5	16.8	6.1	7.1	8.7	4.0
Putn-Water	27.0	13.8	1.0	0.8	0.9	0.8	0.5
Putnam-Ash	45.6	30.4	7.2	4.8	2.9	1.0	0.2
Putnam-LKD	41.5	15.9	4.7	4.4	4.9	5.4	1.1
Ash	11.0	72.4	10.7	4.9	0.7	0.1	0.0
LKD	28.9	16.1	18.4	23.7	7.4	1.9	0.4

Results from pipette particle size distribution analyses show differences between the relative clay contents in the Atchison and Putnam County soil specimens, comparatively. The Atchison County soil contained 16.4 percent clay-sized particles compared to 55 percent clay-sized particles for the Putnam County soil. After mixing with additives, the soils showed a decrease in relative clay abundance and an increase in relative silt and sand-sized particle percentages. This is not unexpected because the fly ash and lime kiln dust contain more silt and sand-sized particles relative to clay-sized particles. The relative clay abundance for the Putnam County soil showed a larger decrease than the Atchison County soil when mixed with fly ash.

X-ray Diffraction (XRD) testing was also used to estimate the composition of the soils and their relative clay fractions. Results from the XRD tests performed on the bulk specimens of soils and soil-additive mixtures are presented in Table 5.3.

Table 5.3 XRD Results for bulk specimens of soil and soil-additive mixtures.

	Relative mineral abundance (wt %)			
Mix Description	Quartz	Plagioclase	Calcite	Total clay
Atchison	39.3	20.9	5.0	34.8
Atchison & Ash	37.8	28.4	5.6	28.2
Putnam	20.5	9.4	0.0	70.1
Putnam & Ash	24.5	10.8	0.0	64.7

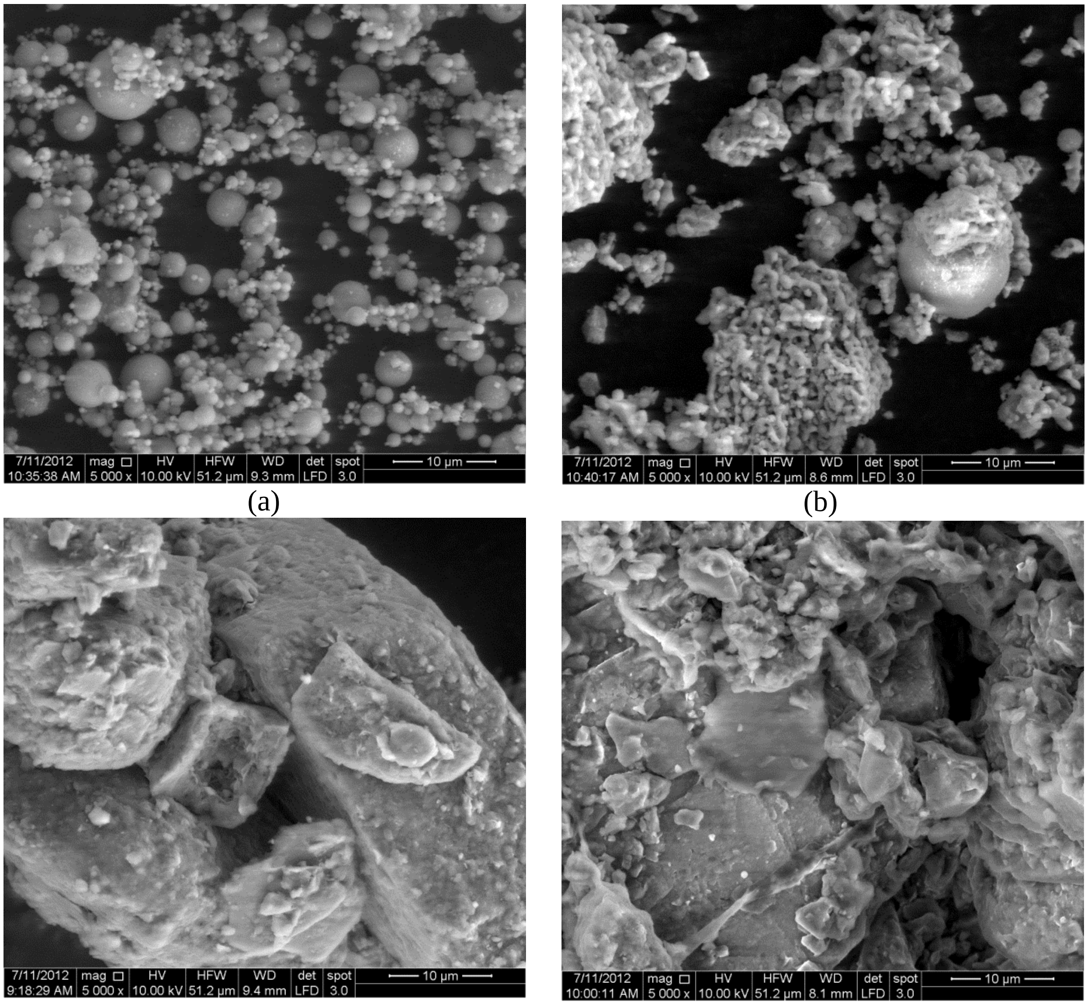
The Atchison County soil contained less clay mineral concentration by weight (28-35 percent) than the Putnam County soil (65-70 percent) as determined by XRD (Table 5.3). These results differed from the hydrometer results presented in Chapter 3, and the pipette analysis presented in Table 5.1. The relative clay contents measured from the three methods are presented in Table 5.4. According to Indorante et al. (1990), the pipette method is the most accurate and precise way to determine the relative clay content in the laboratory; therefore, the results from the pipette method will be used in further discussion.

Table 5.4 Clay content measured for Atchison County and Putnam County soil using hydrometer, pipette, and XRD methods

Soil	Clay Content (%)		
	Hydrometer	Pipette	XRD
Atchison	35	16.4	34.8
Putnam	30	55.2	70.1

Scanning electron microscope (SEM) imaging was used to visually inspect the soils and additives, before and after mixing. The images presented in Figures 5.1 and 5.2 allow visualization of the mixtures on the micro scale. The image in Figure 5.1 (c) shows that the Atchison County soil displayed a block-dominated structure with flat sheet-like particles typical of kaolinitic and illitic clays (Mitchell and Soga, 2005), and smaller particles that could be smectite. The Putnam County soil also contained block-like particles but they were covered in more of the smaller clay particle structures as shown in Figure 5.1 (d).

The spherical nature of the fly ash, as presented in Figure 5.1 (a), contrasted sharply against the flatter kaolinite and illite structures, and was consistent with the description of the particles from literature (Mitchell and Soga, 2005). The lime kiln dust particles (Figure 5.1 (b)) were much smaller and appeared jagged, with a few spheres mixed in.



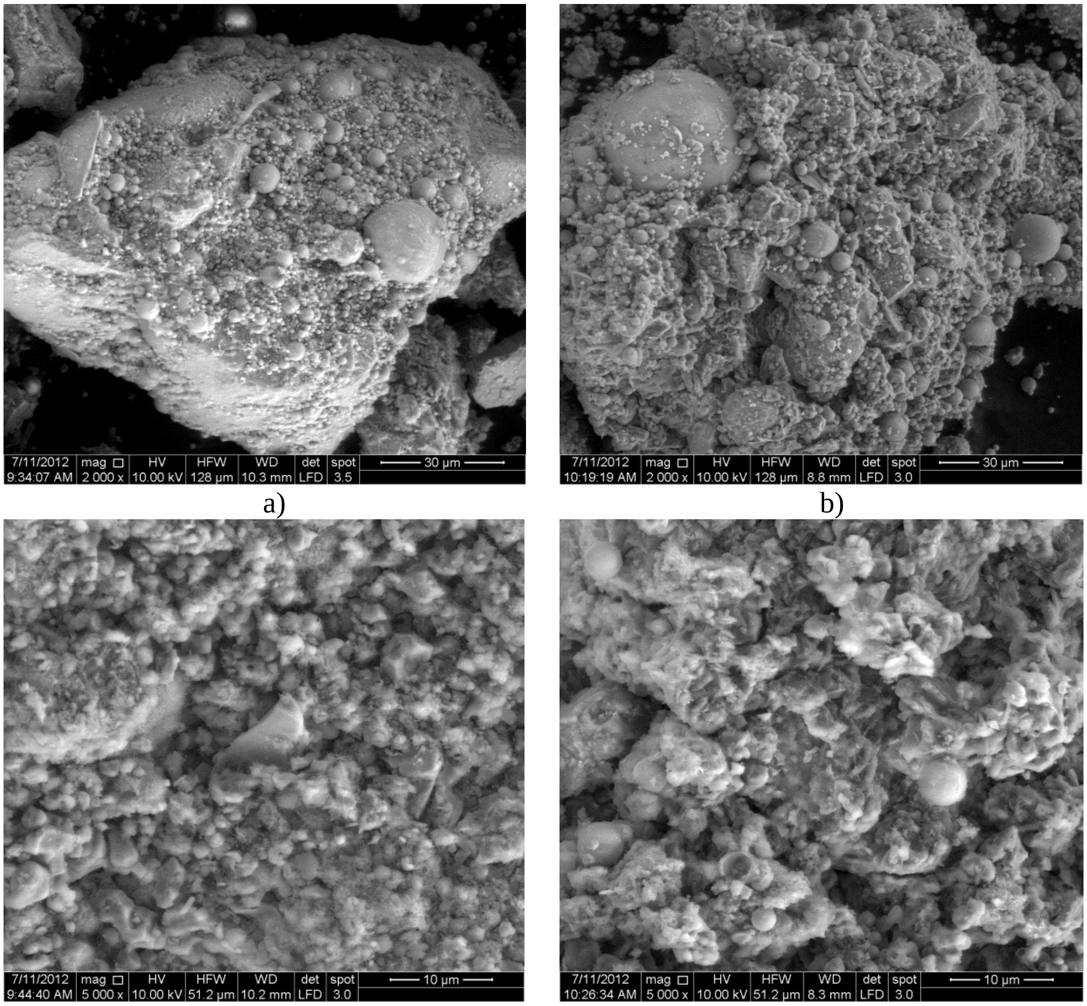
(a) (b)

(c) (d)

Figure 5.1 Scanning Electron Microscope images of (a) Labadie Fly Ash (b) Lime Kiln Dust (Code L) (c) Atchison County soil and (d) Putnam County soil

Figure 5.2-(a) and (b) shows that even though the soils are different structurally, they are both appeared to have interacted with fly ash. The spheres appeared to attach themselves to the surfaces of the clay particles, with the Putnam County soil attracting more spheres compared to the Atchison County soil. The lime kiln dust appeared to attach itself similarly to the fly ash, but the number of attachments appeared to be increased for the Putnam County soil. The soil-lime kiln dust mixture images (5.2 (c) and (d)) are on a much smaller scale relative to the fly ash images (10 microns and 30

microns, respectively) to illustrate the density of the smaller additive particles on the clay surfaces.



a) b)

c) d)

Figure 5.2: Scanning Electron Microscope images of (a) Atchison Clay with 20% Fly Ash (b) Putnam Clay with 20% Fly Ash (c) Atchison County soil with Code L and (d) Putnam County soil with Code L.

5.3 Chemical Analyses

It is possible to qualitatively assess the relative amounts of clay minerals present in the soils using XRD tests performed on the clay fraction specimens as presented in Table 5.5; however, the variability of the method must be considered as the error has been reported to be ± 10 percent by Środoń et al. (2001) and Omotoso et al. (2006). The Putnam County soil appears to contain about twice as much smectite compared to the

Atchison County soil. The two soils contained similar proportions of kaolinite but the Putnam County soil contained less illite than the Atchison County soil. The higher smectite content of Putnam County soil could explain the greater plasticity index, relative to the Atchison County soil, but it is important to reiterate that the clay mineral abundance is presented only for qualitative analysis as the method is subject to error.

Results of analyses for exchangeable cations, CEC, and pH are presented in Table 5.6. The "(soil name) & Water" designation indicates that the specimen was a mixture of soil and water that was compacted and allowed to equilibrate for seven days in the same manner as the soil-additive mixture specimens.

Table 5.5 XRD results for clay fraction specimens of soil and soil-additive mixtures.

	Biscay (1965) relative clay mineral abundance (%)		
Mix Description	Smectite	Illite	Kaolinite
Atchison	44	39	17
Atchison & Ash	42	41	16
Putnam	71	16	13
Putnam & Ash	65	17	18

Table 5.6: Exchangeable Cations and pH Measurements of soils with and without additive

	milliequivalents per 100 grams Extractable Bases						pH	
Mixture Description	Ca	Mg	Na	K	Sum	CEC	$CaCl_2$	H_2O
Atchison & Water	40.6	2.6	0.4	0.8	44.4	15.6	7.8	8.4
Atchison & Ash	60.7	8.1	1.0	0.7	70.5	14.4	9.2	9.3
Atchison & LKD	132.4	2.7	0.4	1.1	137.0	-	12.2	0.1
Putnam & Water	18.5	6.3	1.3	0.6	26.7	36.4	5.2	5.9
Putnam after Ash	58.3	11.1	1.7	0.4	71.5	28.3	8.7	8.8
Putnam & LKD	141.3	4.2	1.1	0.8	147.0	-	11.7	11.9
Ash	101.5	25.4	3.4	0.2	130.5	5.2	-	-
LKD	460.6	1.5	2.1	4.6	469.0	-	12.3	12.3

The results of exchangeable cation testing indicated that the amount of exchangeable calcium in the soils increased after mixing. The increase in concentration was expected due to the additional amount of calcium contributed by the additives, as discussed in Chapter 3. Interestingly, the relative gain in exchangeable calcium was higher for the Putnam County soil (+40 meq/100g) compared to the Atchison County soil (+20 meq/100g) when the soils were mixed with fly ash. The CEC of the Atchison County soil was ~50 percent lower than the Putnam County soil when not mixed with additives. Soil-additive mixture CECs were lower than the soil-only mixtures but the Putnam County soil mixture CECs remained larger than the Atchison County soil mixtures.

5.4 Discussion

Grain-size analysis indicated that the Putnam County soil had more clay than the Atchison County soil. The XRD results qualitatively indicated that the Putnam County soil contained a larger relative quantity of smectitic clay minerals within the clay fraction than the Atchison County soil; however, the error in the measurement must be considered. The CEC of the Atchison County soil was about 50 percent less than that of the Putnam County soil, which is consistent with increased smectite content (Mitchell and Soga, 2005).

The differences in particle sizes and surface areas were observed in the SEM images. The Putnam County soil appears to have more surface area, which is also consistent with the increased clay content shown in particle size analysis. The surface charge of clay particles facilitates the adsorption of fly ash and lime kiln dust, and increased surface areas result in increased charge sites for Ca^{2+} attraction. Research shows that the surface affinity for calcium must be satisfied before cementing reactions can occur (Hilt and Davidson, 1960; Ho and Handy, 1963; Bell, 1996). The idea of surface affinity satisfaction appears to be consistent with the SEM imaging results for the soil-fly ash mixtures. Figure 5.2 shows that the additives appeared to attach to both soils in the same manner; covering the surface of the soil particles.

Recall that in order for cementation to occur through pozzolanic reactions in a soil-fly ash system, three ions must be available. The first is calcium, which through exchangeable cation testing was proven to be available in the Putnam County soil, and the second being the presence of silicon and aluminum. Calcium and pozzolanic material (silicon and aluminum) compounds were present in both additives but more pozzolanic materials are needed to bond with the calcium. For this to occur, alkaline pH is required to enhance aluminosilicate mineral dissolution and to release aluminum and silicon into solution (Eades and Grim, 1966). Figure 5.3 illustrates that as the pH increases, silicon and aluminum mineral dissolutions are enhanced. Elevated concentrations of silicon and aluminum in solution at higher pH are essential to form the calcium silicate hydrate (CSH) and/or calcium aluminate hydrate (CAH) needed to cement the mixture (Eades and Grim, 1966). The pH for the Atchison County soil after mixing with fly ash was higher (pH 9.3) compared to the Putnam County soil after the addition of fly ash (pH 8.8) as shown in Table 5.6. While the increase in pH is small, it could affect the amount of aluminum that was released into the system (Figure 5.3 (b)). With the additive composition and quantity being the same for both soil mixtures, one explanation for the lack of modulus gain in the Putnam County soil could be that the amount of silicon and aluminum being released from the soil, due to increased pH, may have been less for the

Putnam County soil compared to the Atchison County soil. This needs to be investigated further by measuring the pH with time after mixing the soils and additives.

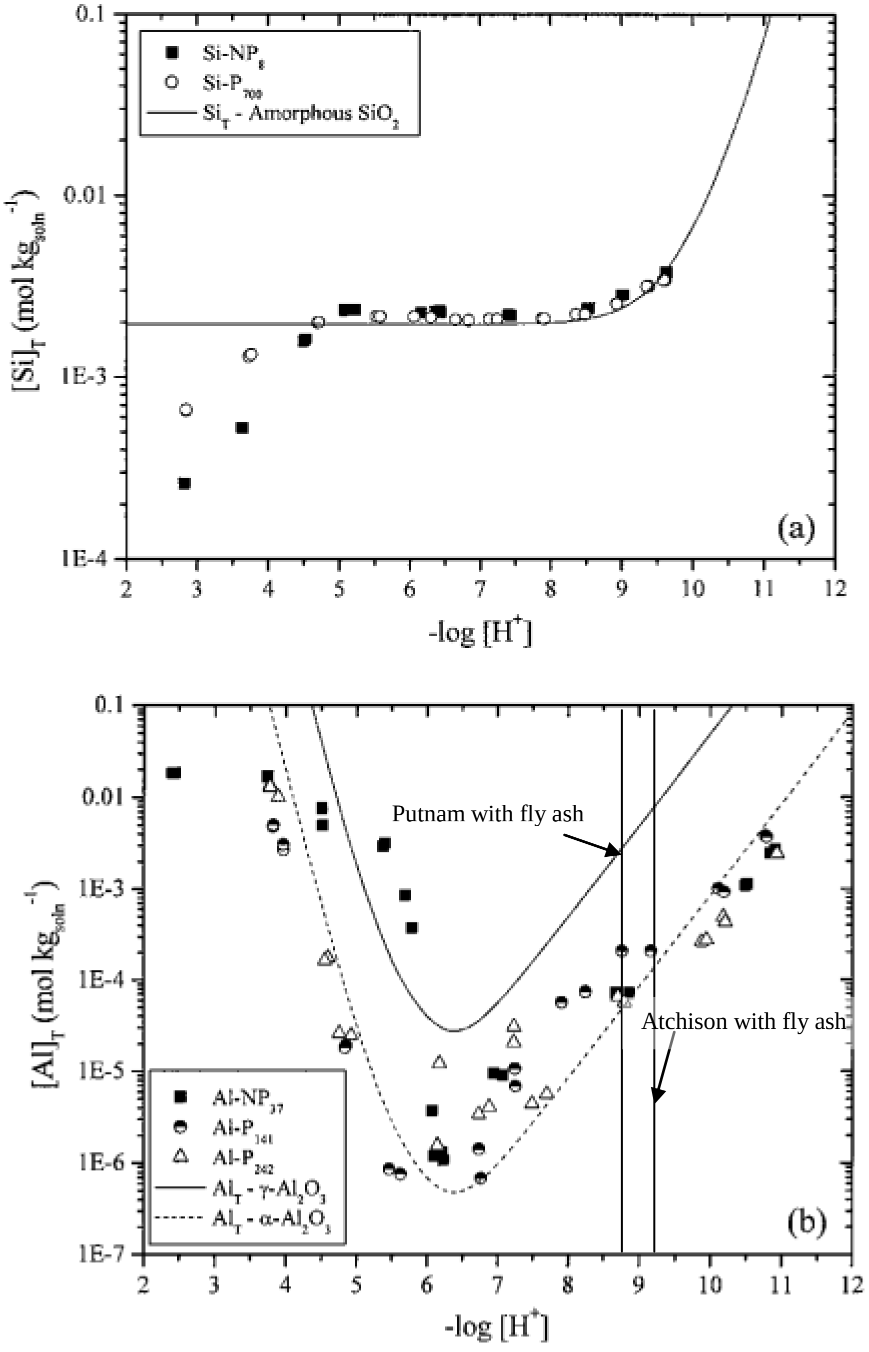


Figure 5.3 Dissolution of (a) Si-P$_{700}$ and Si-NP$_8$ with respect to SiO2 (log K$_{sp}$ = −2.71) and (b) Al-P$_{242}$, Al-P$_{141}$, Al-NP$_{37}$ with respect to γ -Al$_2$O$_3$ (log K$_{sp}$ = 11.49) and α-Al$_2$O$_3$ (log K$_{sp}$ = 9.73). Hydrolysis constants for Al and Si were obtained from Nordstrom and May and Stumm and Morgan, respectively(From Goyne et al. (2002))

The increase in exchangeable calcium for the Atchison County soil mixed with fly ash was 20 millequivalents per 100 grams less than the increase for the Putnam County soil-fly ash mixture. It is possible that the difference in exchangeable calcium could result from calcium having been utilized for cementing reactions in the Atchison County soil, but not in the Putnam County soil. Additionally, the CECs of the soils decreased when fly ash was added, which is consistent with literature (Nalbantoğlu, 2004). This decrease is because the additives have low CECs, and effectively reduce the original soil CEC by "diluting" it. These data suggest that pozzolanic reactions are occurring in the amended soils; however, it is possible that less cementing agent was formed in the Putnam County soil, or an insufficient amount was formed to sufficiently stabilize the smectitic soil. This suggests that more additive may be needed for the high plasticity soil to achieve similar levels of stabilization compared to the low plasticity soil.

The phenomenon of high plasticity clays requiring higher quantities of calcium-rich additives has been reported in the literature for lime-stabilized soils. As the soil's plasticity index increases, the amount of additive needed to satisfy the surface charges and flocculate the material also increases. Lime additions decrease the plasticity of the material until a point at which no additional change in plasticity results from additional lime. This response is called the "lime fixation point" (Hilt and Davidson, 1960; Ho and Handy, 1963; Bell, 1996). If sufficient lime is added, the lime fixation point will be passed and any excess lime will be available to form cementious products. Research on the subject has yielded a relationship between plasticity index and mix percentage by weight needed to overcome the fixation point (Daita et al., 2005), and has identified the pH-mixture percentage relationship (Eades and Grim, 1966) The concept of lime fixation point is consistent with the small-strain modulus results presented in Chapter 4, as both soils exhibited an increase in modulus when mixed with lime kiln dust. The fact that mixture ratios for lime kiln dust were chosen from the chart designed to overcome the lime fixation point based on Atterberg Limits could explain the increased stiffness in the Putnam County soil. Lime-fixation point concepts need to be applied to fly ash stabilization methodology in future research to determine effective mixture ratios.

5.5 Summary

Results of particle size analysis, XRD, SEM, and CEC testing were presented in this chapter. The Atchison County soil showed less relative clay and lower CEC than the Putnam County soil. Images from SEM indicated the fly ash appeared to attach to both soils, but appeared to cover the Putnam County soil more effectively. Both soils showed an increase in pH when mixed with additives, as is expected when introducing a large quantity of calcium to the system. Possible explanations for the lack of modulus stabilization for Putnam County-fly ash mixtures were discussed.

6 Conclusions

6.1 Summary

The two main objectives of this research were to: (1) quantify the effectiveness of calcium-rich additive stabilization of representative subgrade soils in Missouri, and (2) assess the viability of using stress wave-based, non-destructive testing (NDT) methods for quality assessment of stabilized subgrades.

Two soils from Missouri, termed Atchison County and Putnam County in reference to where they were collected, were used in this study. The Atchison County soil was classified as low plasticity clay (CL), with a Plasticity Index (PI) of around 15. The Putnam County soil represented the high plasticity clays (CH) of Missouri with a PI greater than 30.

Proctor compaction and free-free resonant column (FFRC) tests were performed on mixtures of 0, 10, 15, and 20 percent fly ash, and 4 and 8 percent lime kiln dust by weight for the two soils. Proctor curves were developed for the soils and mixtures to determine the effects of the additives on the optimum water content and maximum dry density. Trends in small-strain modulus with time were observed for the soils and mixtures using FFRC for measurements up to 7 days after compaction. The relationships between compaction water content and small-strain Young's modulus were analyzed. The effect of delay between mixing and compaction on the modulus was also measured for one soil additive combination. Additionally, the results of small-strain modulus testing were compared with resilient modulus measurements to establish the relationship between the measurements for the two soils. The small-strain modulus results from the first three hours after compaction were then analyzed to determine the viability of using wave-based measurements as a quality assurance/quality control (QA/QC) method for stabilized soils.

The results of the FFRC testing led to an analysis of soil composition and chemical behavior to better understand why the lower plasticity soil behaved differently than the higher plasticity soil. Specimens of the two soils, with and without additives, were subjected to X-ray diffraction (XRD), exchangeable cation testing, and scanning electron microscopy in an attempt to qualitatively differentiate the behavior of the two soils when mixed with additives.

6.1 Conclusions

The addition of calcium-based additives to the subgrade soils altered the compaction behavior; however, the changes in compaction behavior were different for the various soil-additive combinations. The CL soil (Atchison County)-fly ash mixtures exhibited a decrease in the optimum water content (as much as 3 percent for the 20 percent fly ash mixture) and an increase in maximum dry density (as much as 6 pcf for the 20 percent fly ash mixture), compared to the soil alone. The lime kiln dust affected the compaction behavior of the Atchison County soil differently than the fly ash. The Atchison-lime kiln dust mixtures showed a decrease in optimum water content of about 1 percent, and a slight decrease of about 1 pcf in the maximum dry density. The optimum water content increased slightly (about 1 percent for 10 percent fly ash mixture), or stayed the same (15 percent and 20 percent fly ash mixture) for the CH soil (Putnam County)-fly ash combinations, while the maximum dry density increased for all CH soil (Putnam County)-fly ash combinations, with a maximum gain of 8 pcf for the 20 percent fly ash mixture. The lime kiln dust-Putnam County soil mixtures showed a slight increase in optimum water content with increasing lime kiln dust percentage, while the maximum dry density increased by only 1 pcf for the 4 percent mixture, and showed no change for the 8 percent lime kiln dust mixture.

Results from the modulus testing of samples compacted near optimum water content showed that large increases in modulus values can be achieved with additive-stabilization of subgrade soils. Three-day modulus values of the low-plasticity clay more than doubled with the addition of fly ash. The percentage of fly ash (varied from 10% to 20%) had a negligible effect on measured modulus values for the low-plasticity soil. The addition of lime kiln dust to the low-plasticity soil resulted in modulus values that were more than three times higher than the unstabilized values.

The modulus results also indicated that the effectiveness of subgrade stabilization can be highly variable and is strongly influenced by the chemical and physical properties of the soil. The high-plasticity soil exhibited large increases in modulus (over three times the unstabilized values) when mixed with lime kiln dust, but showed essentially no effect from the addition of fly ash. The physical and chemical testing performed in the secondary study suggested that the poor performance of fly ash stabilization of the high-plasticity clay may be due to higher amounts of smectite in the Putnam County soil. In SEM images of the high-plasticity-fly ash mixture, the soil appeared to attract additives, but did not show an increase in modulus. Therefore, it is possible that the higher plasticity soil may require more additive to achieve the same degree of stabilization as the lower plasticity soil. It could be that the additive must satisfy the increased surface affinity present in the Putnam County soil before forming cementious materials, as seen in previous studies on lime-stabilized soils (Ho and Handy, 1963; Eades and Grim, 1966). Additional studies are required to confirm this hypothesis.

In regard to the second objective of this research, the results from the small-strain laboratory measurements showed that changes in velocity in the very short term (1 hr after compaction) are large enough to be detected by common wave-based methods (such as SASW) used to measure velocities in the field. Measured changes in velocity were in the range of about 20 to 45 percent higher than the velocities of the unstabilized soils (with the exception of the Putnam County-fly ash mixtures, as previously discussed). These changes in velocity are well beyond the expected measurement uncertainty (COV of 5 to 15 percent) of surface wave velocity methods (Marosi and Hitunen, 2004). Therefore, based on this limited study, it appears that surface wave velocity measurements can be used as a viable quality control/assessment technique to identify regions of subgrade that were not stabilized.

6.2 Recommendations

Several recommendations for future studies on the application of additives to Missouri soils can be made. It is recommended that the work included in this thesis be expanded to evaluate more Missouri soils (with varying PI, organic content, clay mineralogy etc.), and additional additives for a better understanding of the mechanisms and effectiveness of additive stabilization on Missouri soils.

Measurements using SASW field methods are suggested to develop relationships between laboratory results from FFRC and corresponding field velocities for stabilized soil. Field studies would also allow for an analysis of the effects of mixing methods used in the field and long term analysis of the effects of fly ash and lime kiln dust on Missouri soils after cyclic loading and exposure to weather events. Additionally, an extensive variability study of modulus is suggested with a focus on the first few hours after stabilization, to determine the limits of measurements using FFRC in the laboratory and SASW under realistic field conditions.

Additional laboratory analysis of the changes in Atterberg limits and other geotechnical index values are suggested to determine fly ash threshold percentages for effective stabilization. The isolation of the effects of clay mineralogy on additive-stabilization should also be studied. It would also be beneficial to examine how to integrate existing resources on soil classification and composition, from soil science and Geographic Information Systems (GIS), into decision making regarding additive stabilization and ground improvement. These steps would help develop consistent methodology to help practitioners select proper additives and mixture ratios for their native soils

Appendix

Appendix A documents the results of compaction tests performed on Atchison and Putnam County soils with and without fly ash and lime kiln dust Plots of small-strain modulus with time are presented in Appendix B. Modulus-water content plots are presented in Appendix C. Results from X-ray defraction measurements are presented in Appendix D.

A. Compaction Results

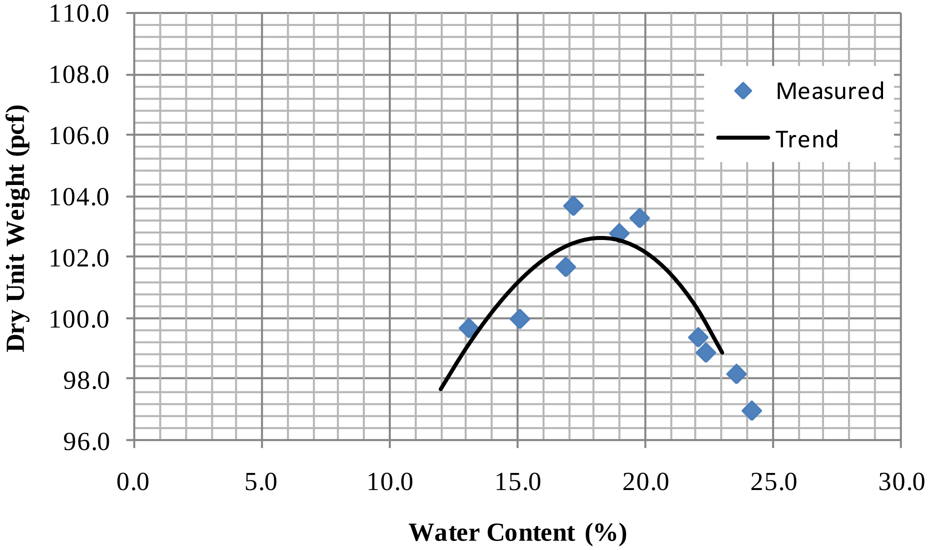


Figure A.1 Compaction results for Atchison County soil with no additive

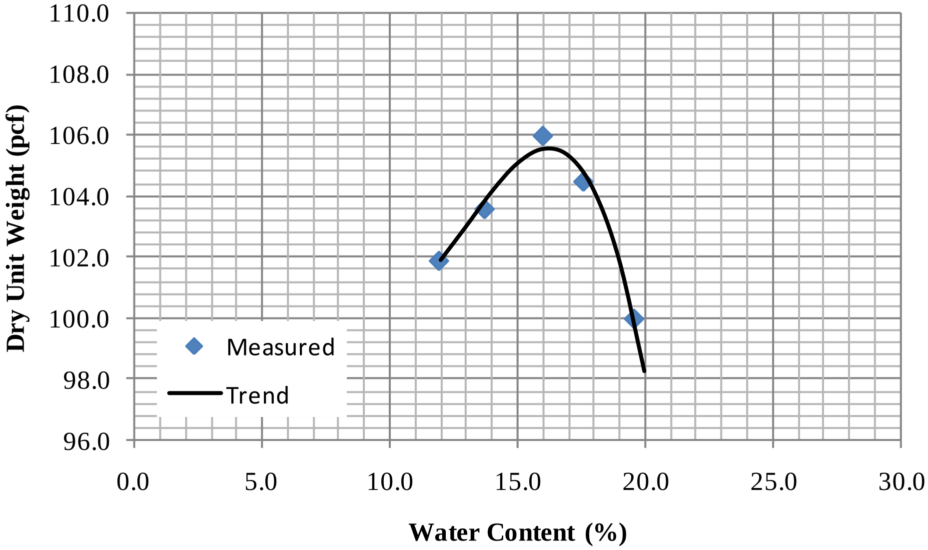


Figure A.2 Compaction results for Atchison County soil with 10% fly ash

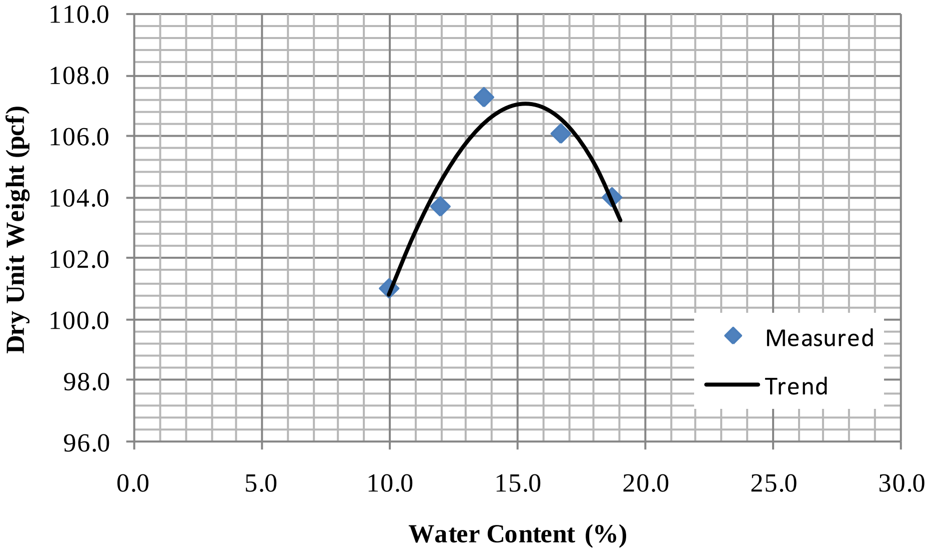


Figure A.3 Compaction results for Atchison County soil with 15% fly ash

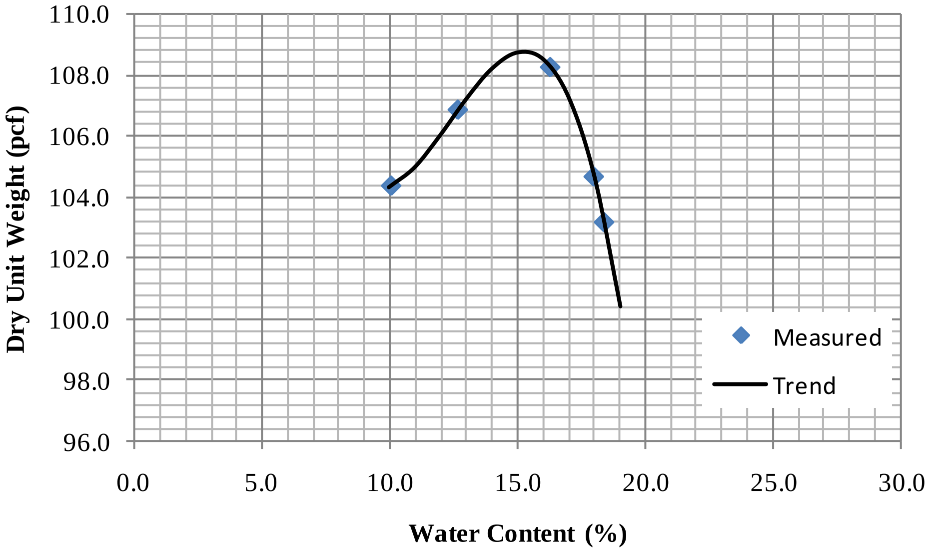


Figure A.4 Compaction results for Atchison County soil with 20% fly ash

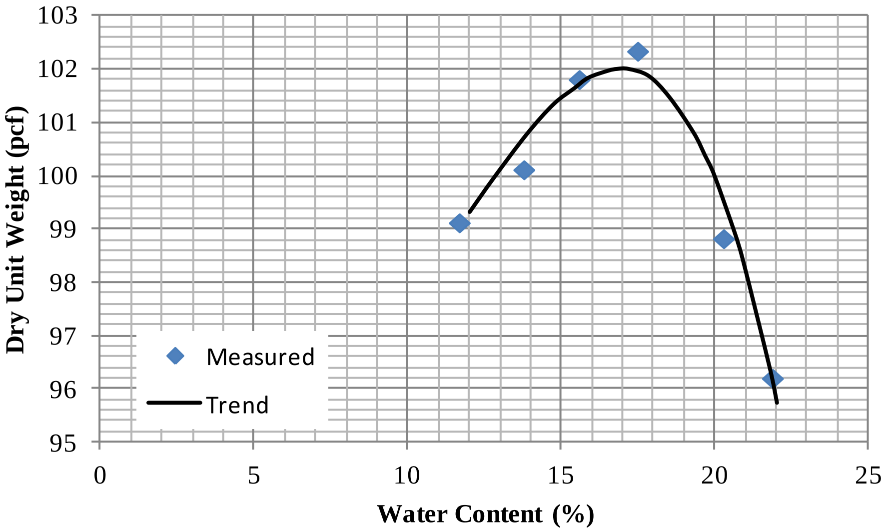


Figure A.5 Compaction results for Atchison County soil with 4% lime kiln dust

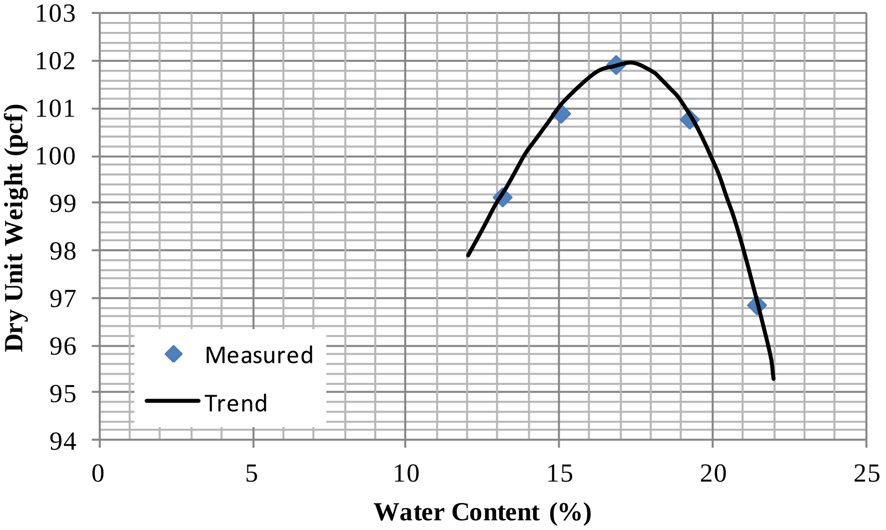


Figure A.6 Compaction results for Atchison County soil with 8% lime kiln dust

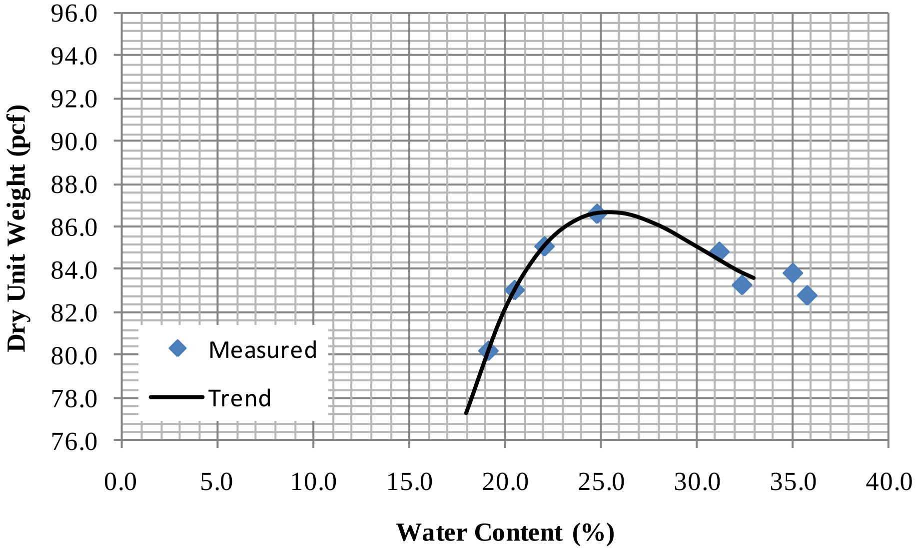


Figure A.7 Compaction results for Putnam County soil without additive

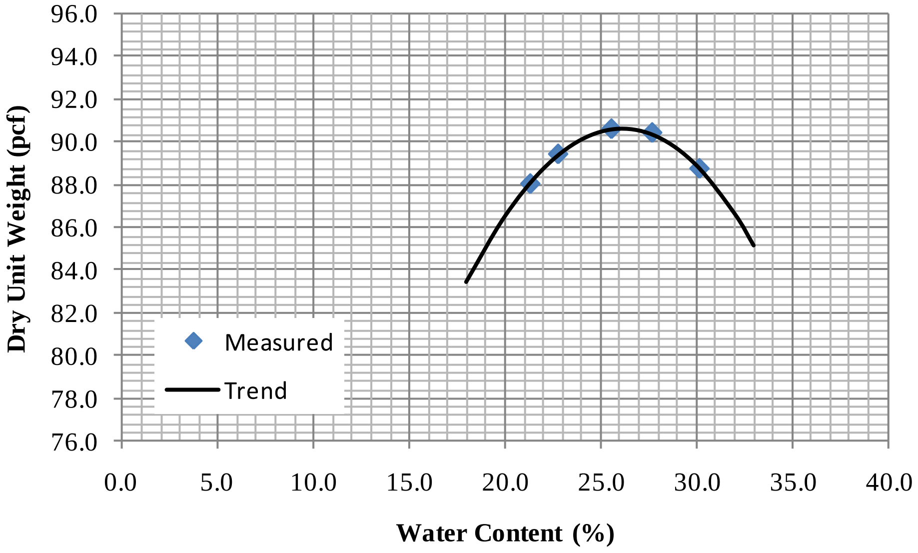


Figure A.8 Compaction results for Putnam County soil with 10% fly ash

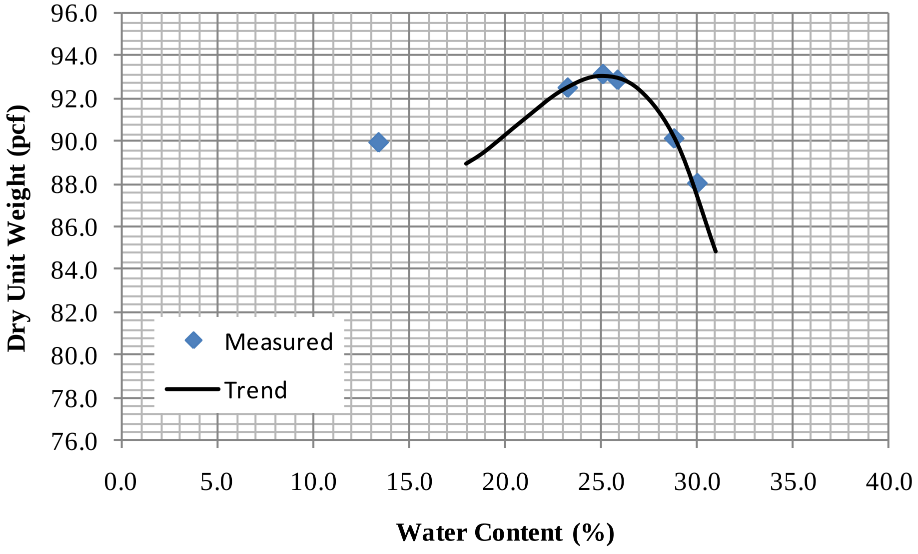


Figure A.9 Compaction results for Putnam County soil with 15% fly ash

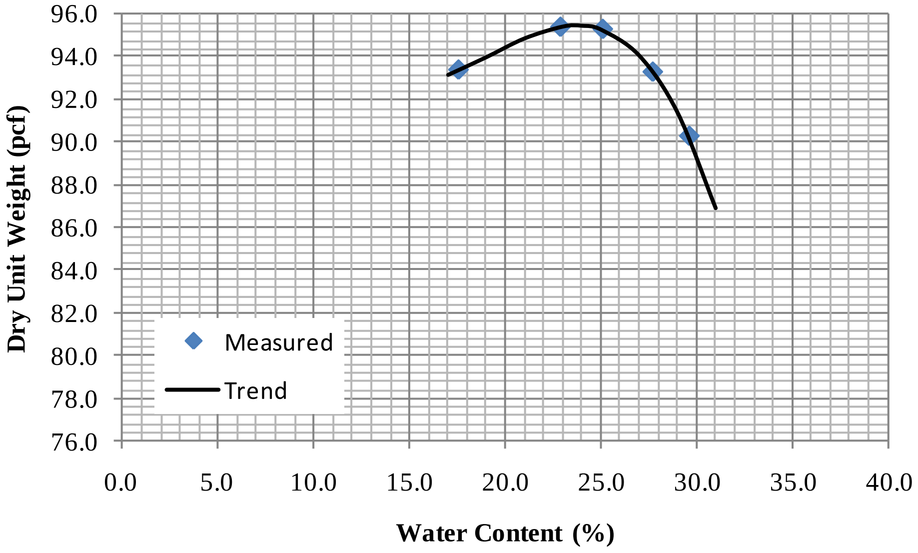


Figure A.10 Compaction results for Putnam County soil with 20% fly ash

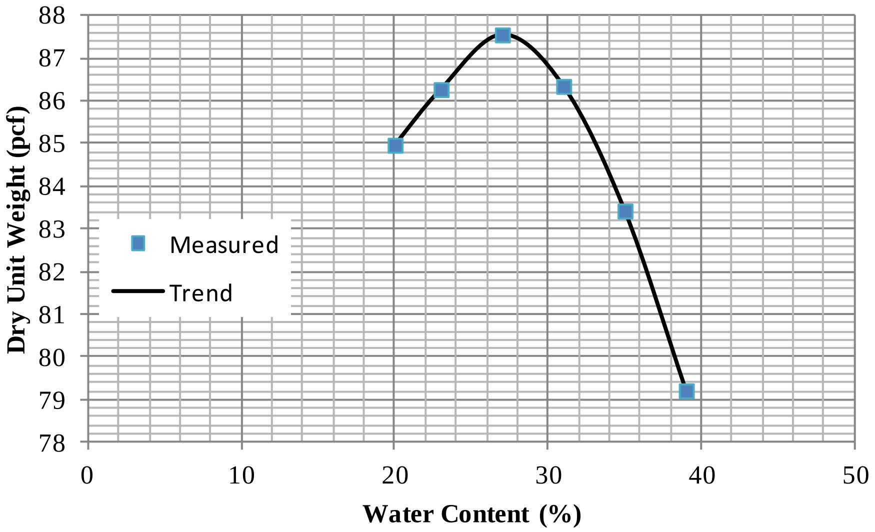


Figure A.11 Compaction results for Putnam County soil with 4% lime kiln dust

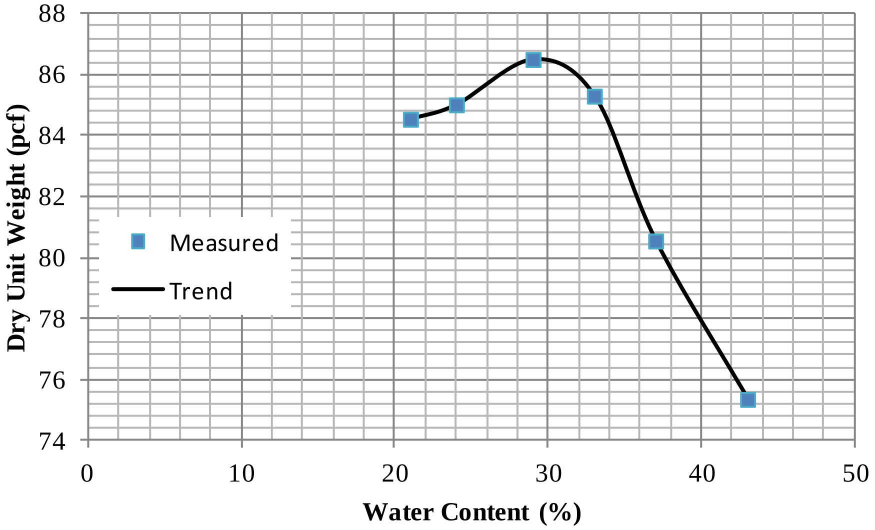


Figure A.12 Compaction results for Putnam County soil with 8% lime kiln dust

B. Small-Strain Modulus Results

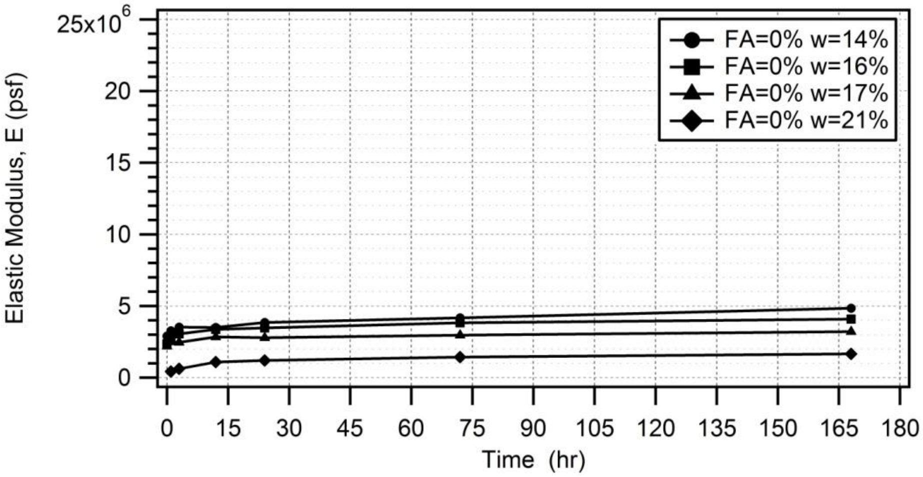


Figure B.1 Change in Young's modulus with time; Atchison County soil with no additive

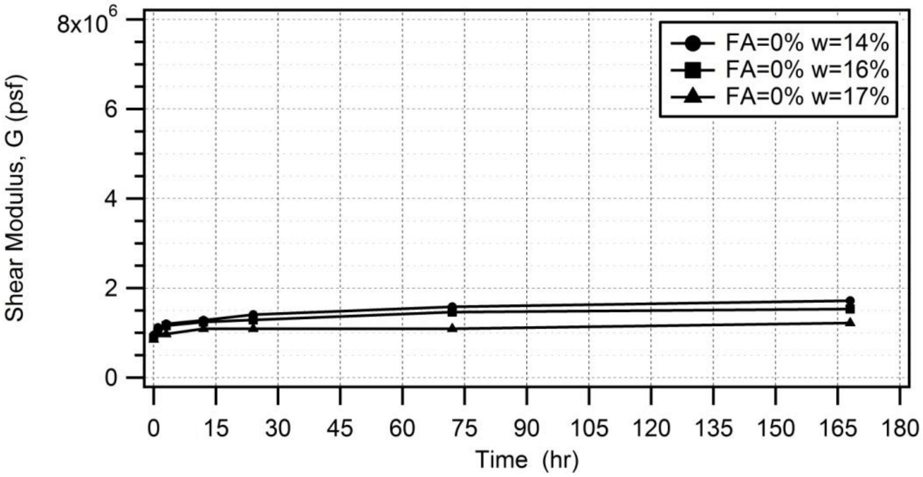


Figure B.2 Change in shear modulus with time; Atchison County soil with no additive

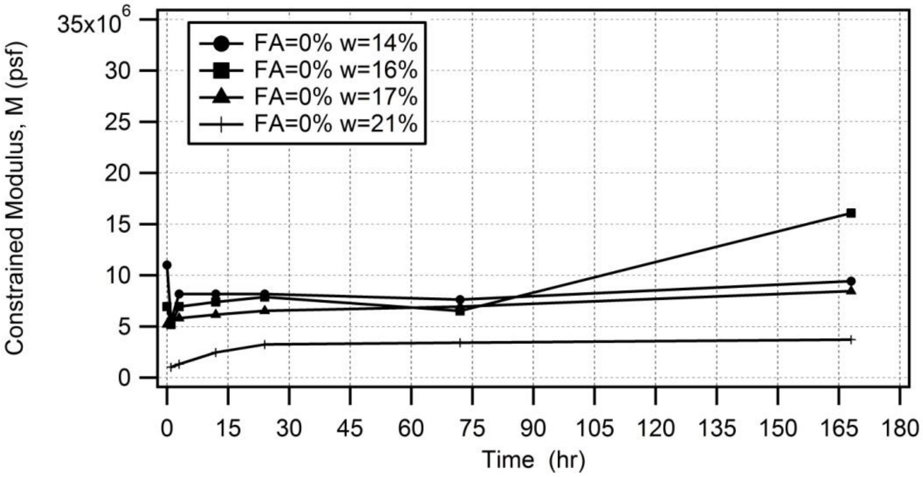


Figure B.3 Change in constrained modulus with time; Atchison County soil with no additive

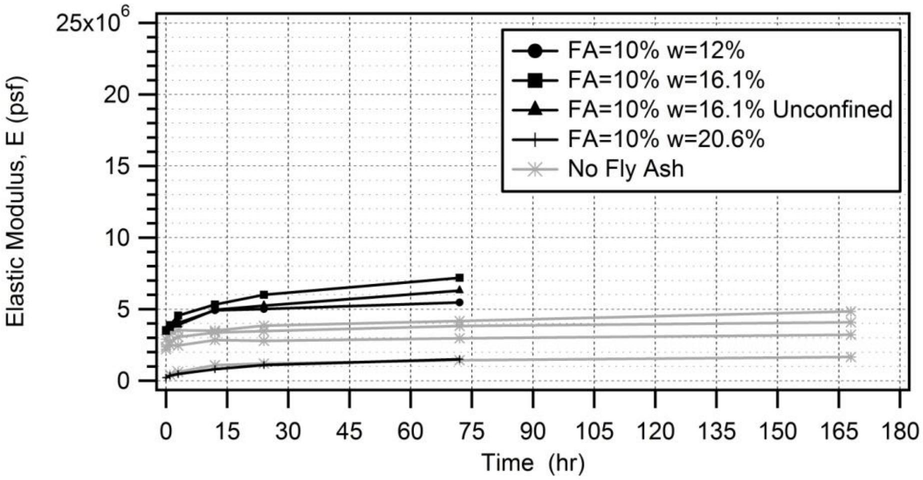


Figure B.4 Change in Young’s modulus with time; Atchison County soil with 10% fly ash

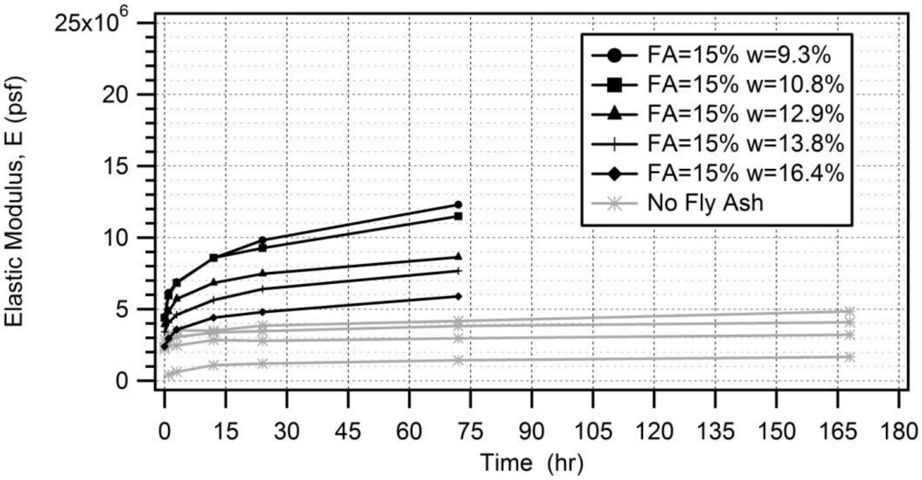


Figure B.5 Change in Young's modulus with time; Atchison County soil with 15% fly ash

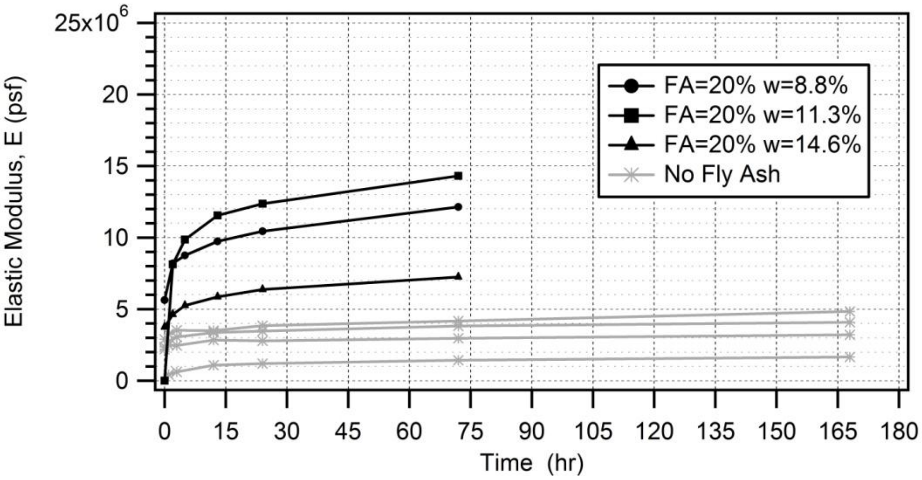


Figure B.6 Change in Young's modulus with time; Atchison County soil with 20% fly ash

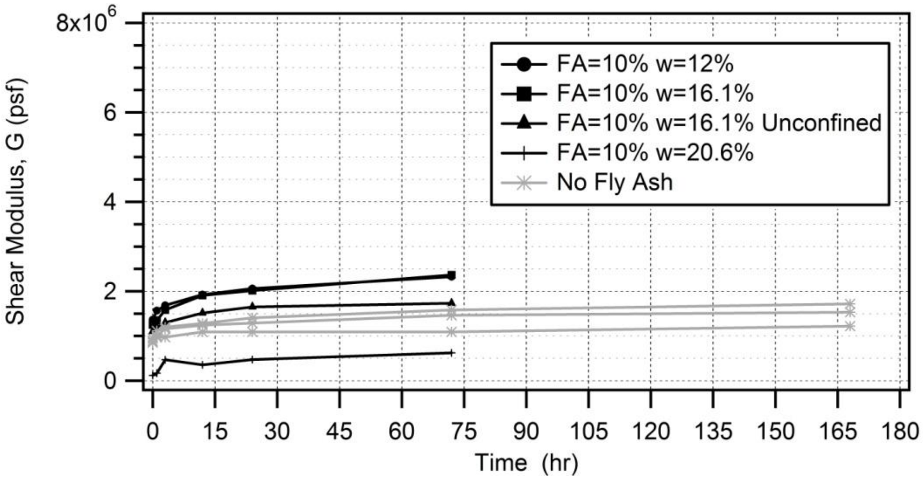


Figure B.7 Change in shear modulus with time; Atchison County soil with 10% fly ash

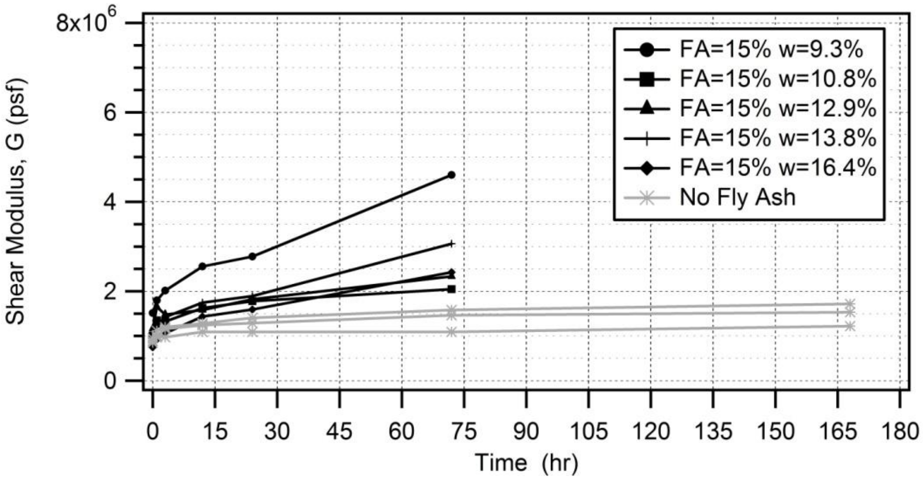


Figure B.8 Change in shear modulus with time; Atchison County soil with 15% fly ash

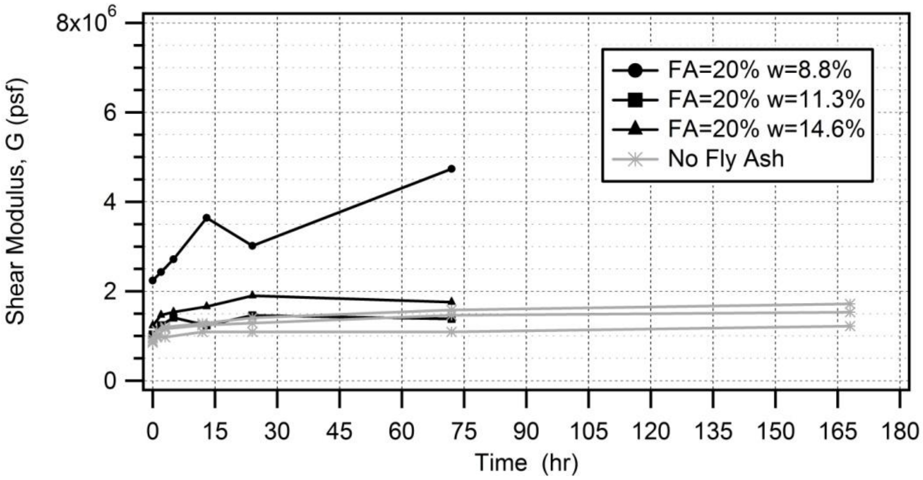


Figure B.9 Change in shear modulus with time; Atchison County soil with 20% fly ash

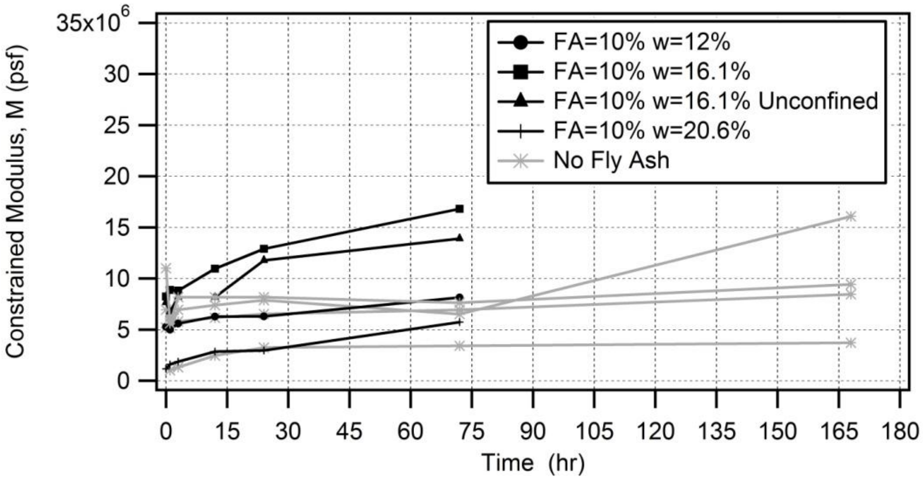


Figure B.10 Change in constrained modulus with time; Atchison County soil with 10% fly ash

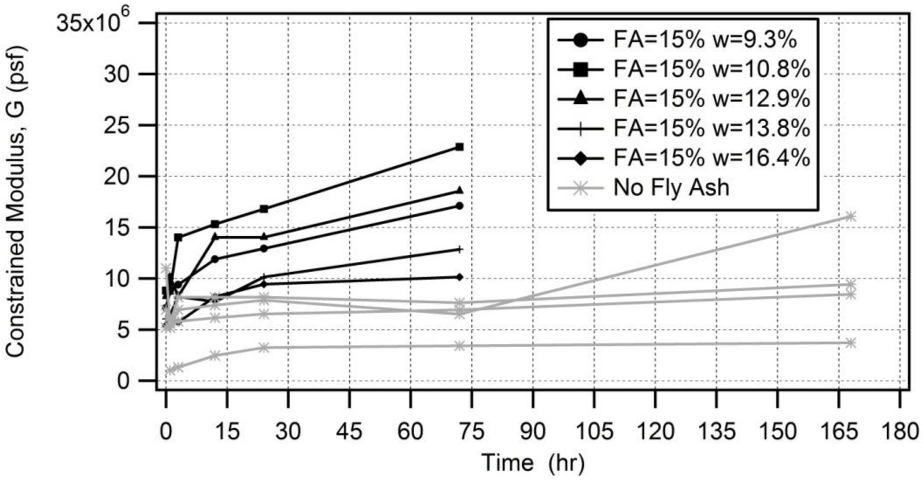


Figure B.11 Change in constrained modulus with time; Atchison County soil with 15% fly ash

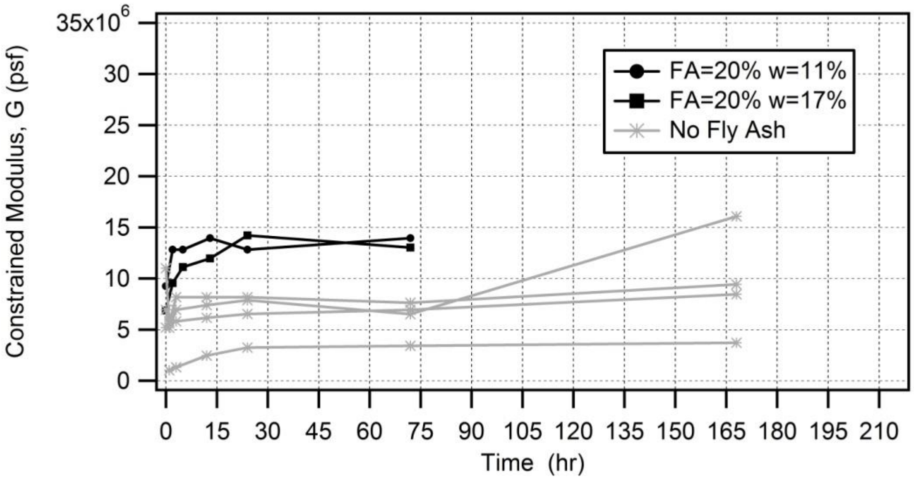


Figure B.12 Change in constrained modulus with time; Atchison County soil with 20% fly ash

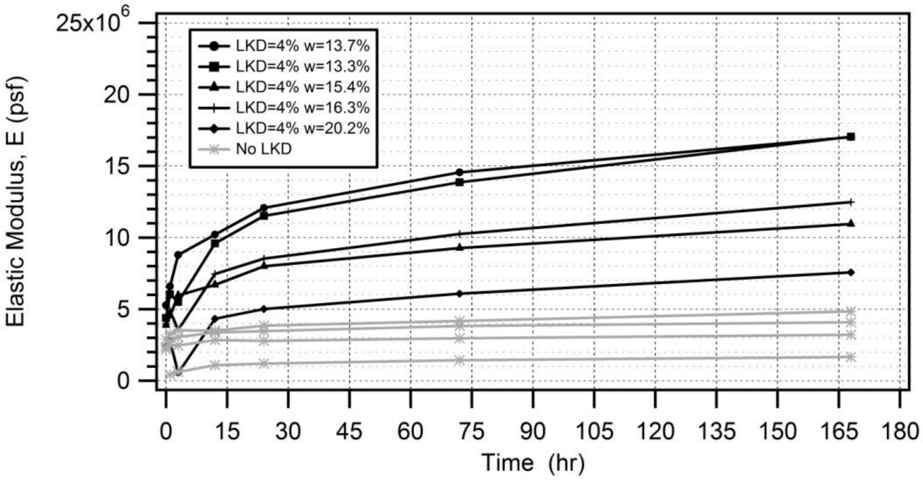


Figure B.13 Change in Young’s modulus with time; Atchison County soil with 4% lime kiln dust

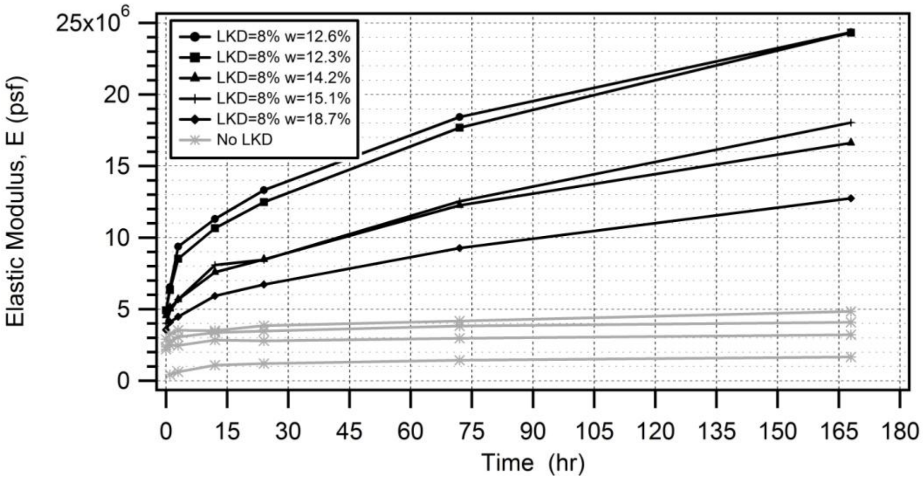


Figure B.14 Change in Young’s modulus with time; Atchison County soil with 8% lime kiln dust

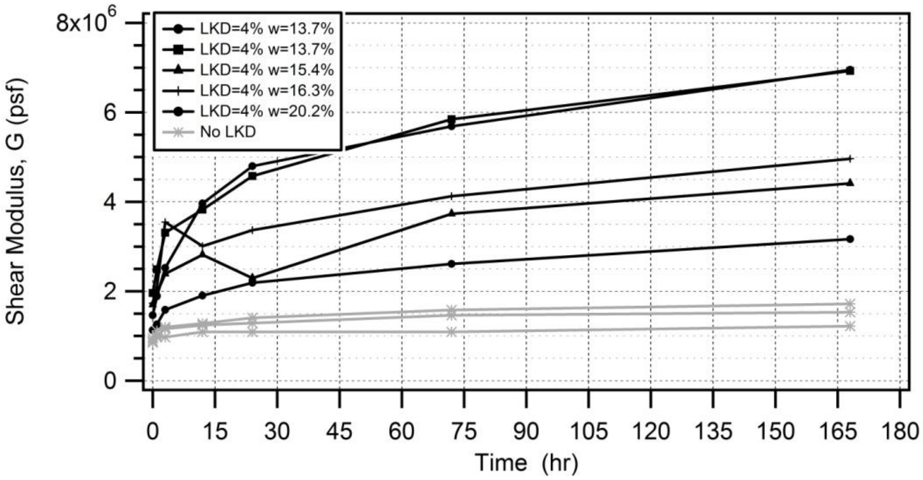


Figure B.15 Change in shear modulus with time; Atchison County soil with 4% lime kiln dust

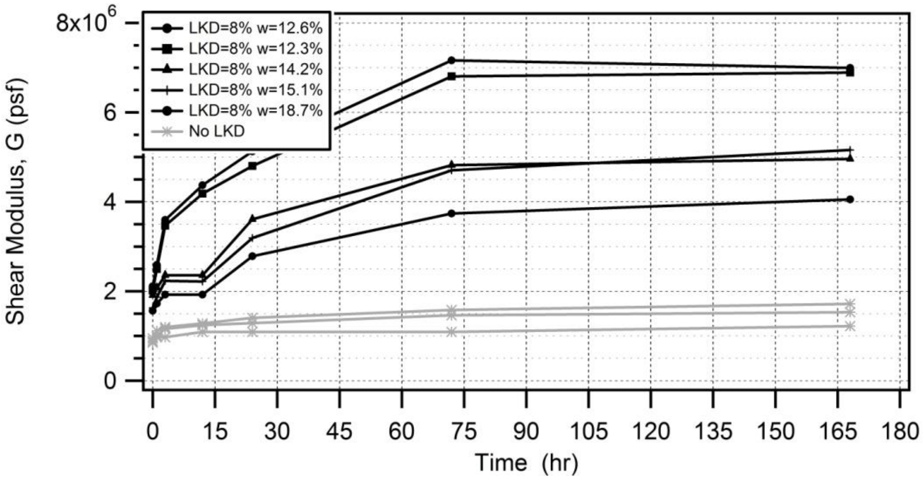


Figure B.16 Change in shear modulus with time; Atchison County soil with 8% lime kiln dust

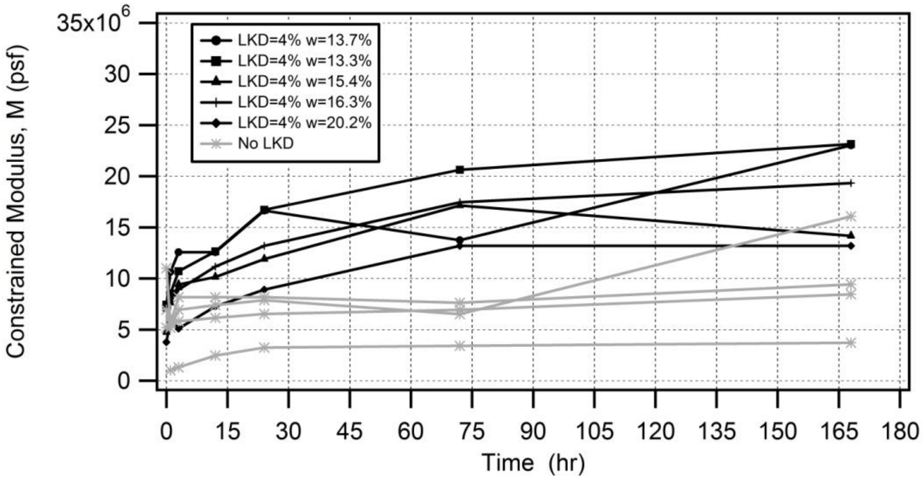


Figure B.17 Change in constrained modulus with time; Atchison County soil with 4% lime kiln dust

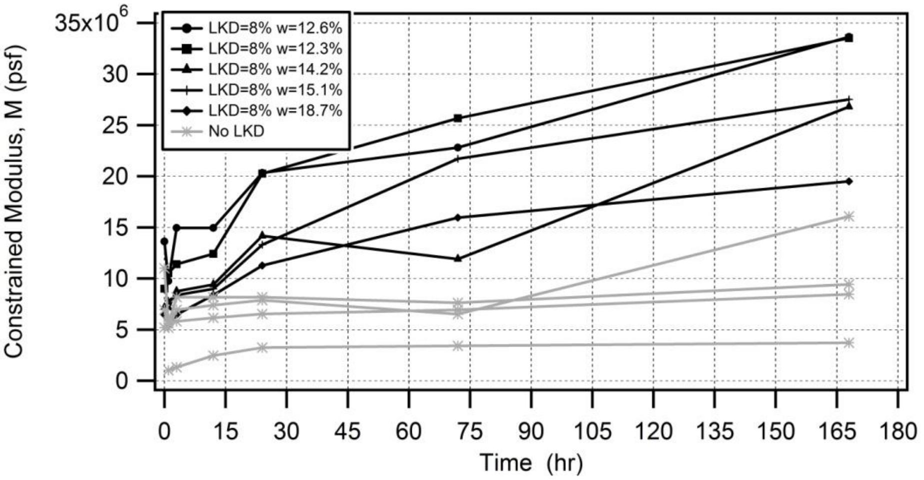


Figure B.18 Change in constrained modulus with time; Atchison County soil with 8% lime kiln dust

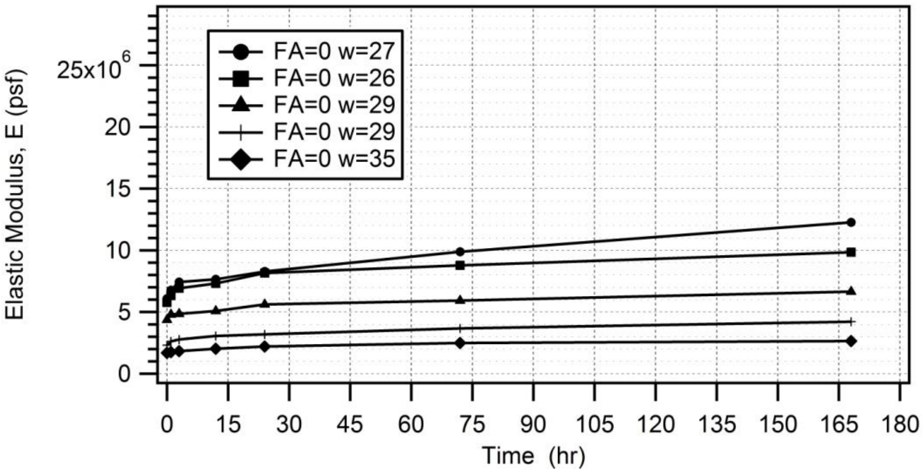


Figure B.19 Change in Young's modulus with time; Putnam County soil with no additive

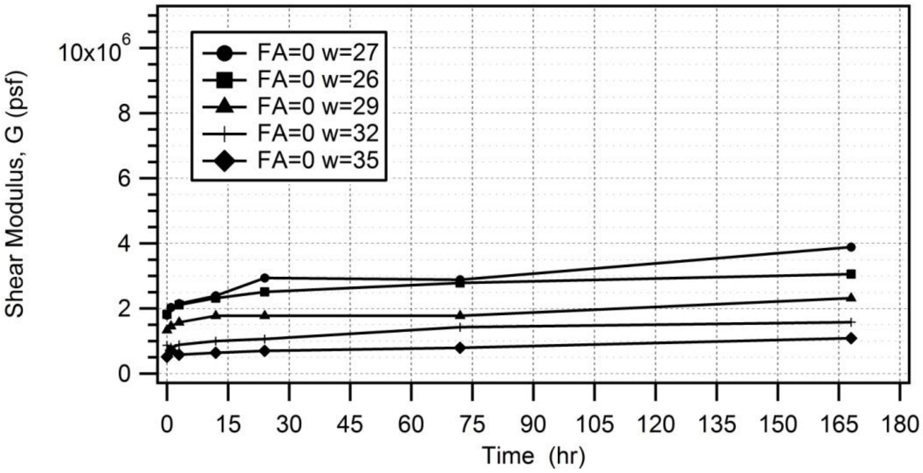


Figure B.20 Change in shear modulus with time; Putnam County soil with no additive

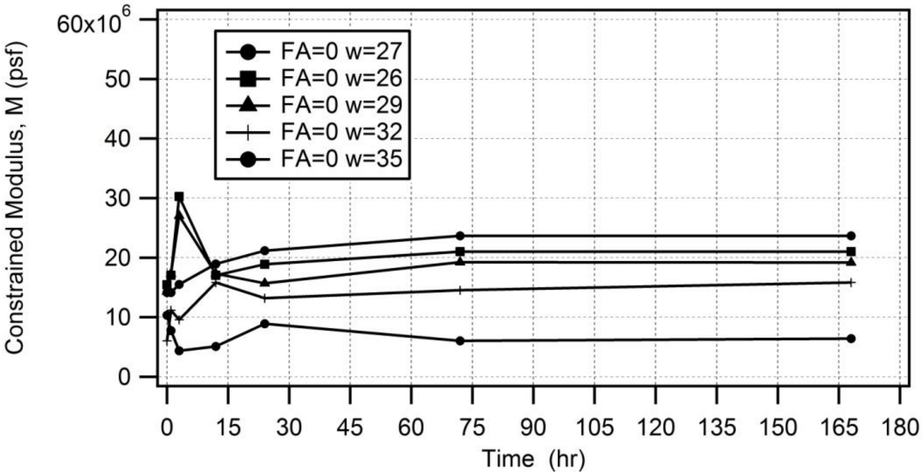


Figure B.21 Change in constrained modulus with time; Putnam County soil with no additive

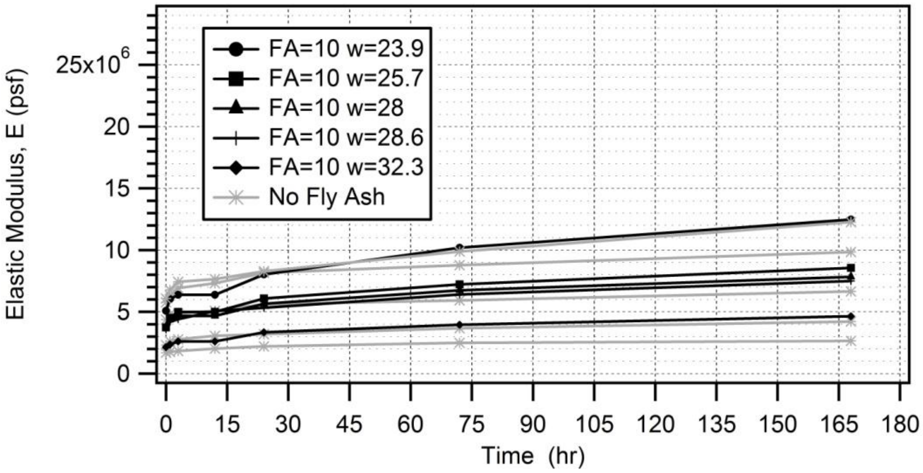


Figure B.22 Change in Young's modulus with time; Putnam County soil with 10% fly ash

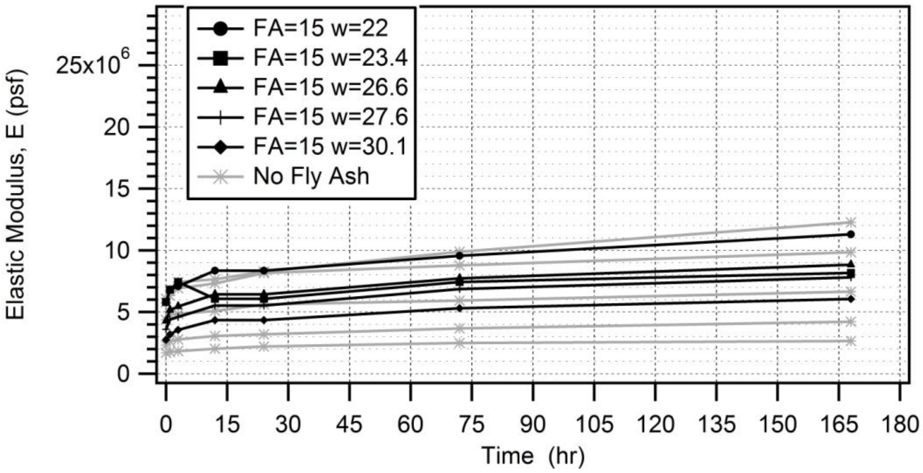


Figure B.23 Change in Young's modulus with time; Putnam County soil with 15% fly ash

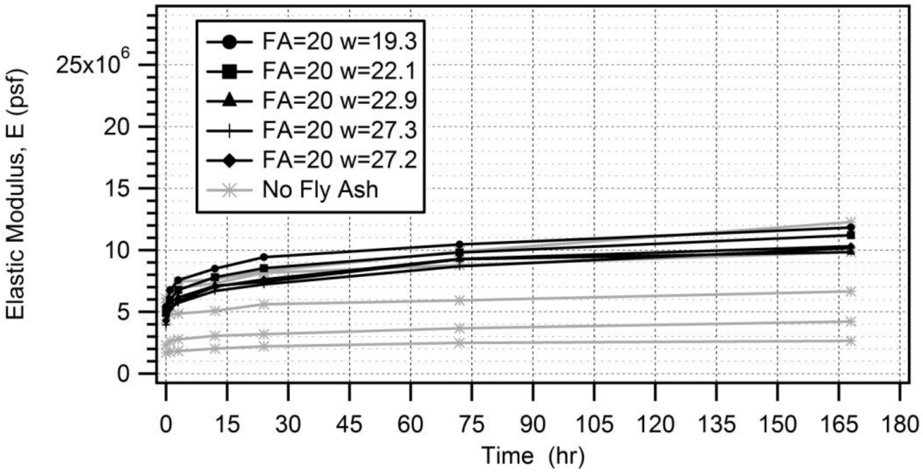


Figure B.24 Change in Young's modulus with time; Putnam County soil with 20% fly ash

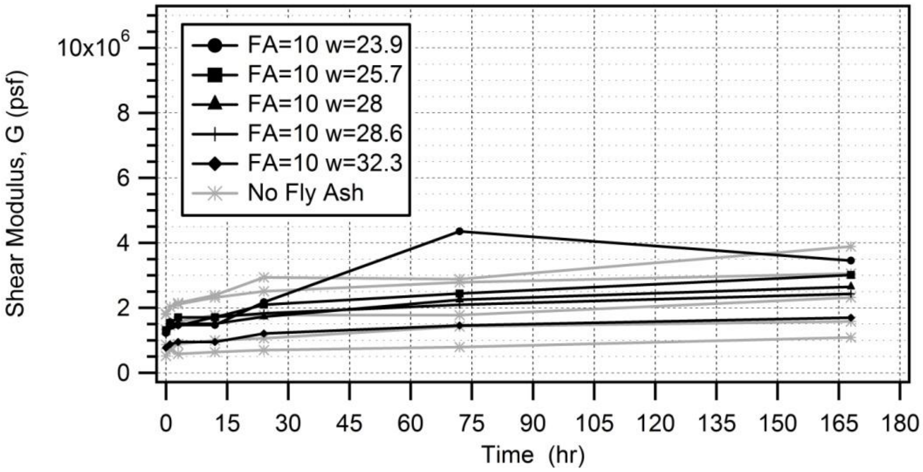


Figure B.25 Change in shear modulus with time; Putnam County soil with 10% fly ash

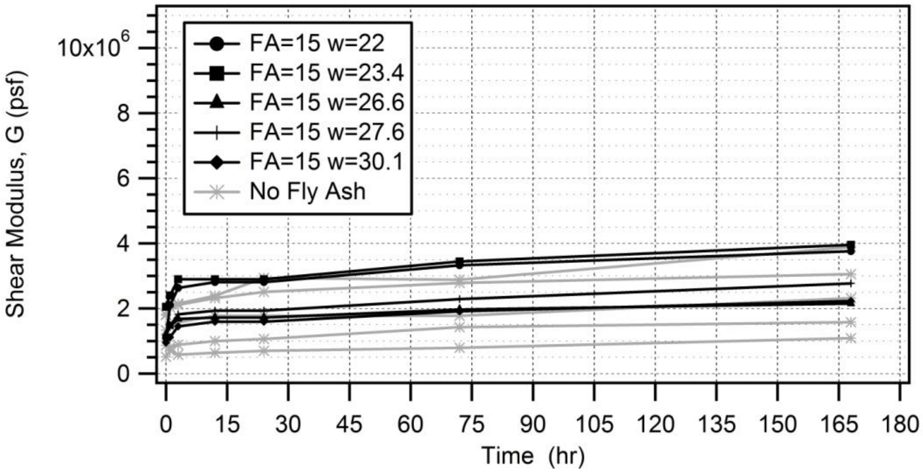


Figure B.26 Change in shear modulus with time; Putnam County soil with 15% fly ash

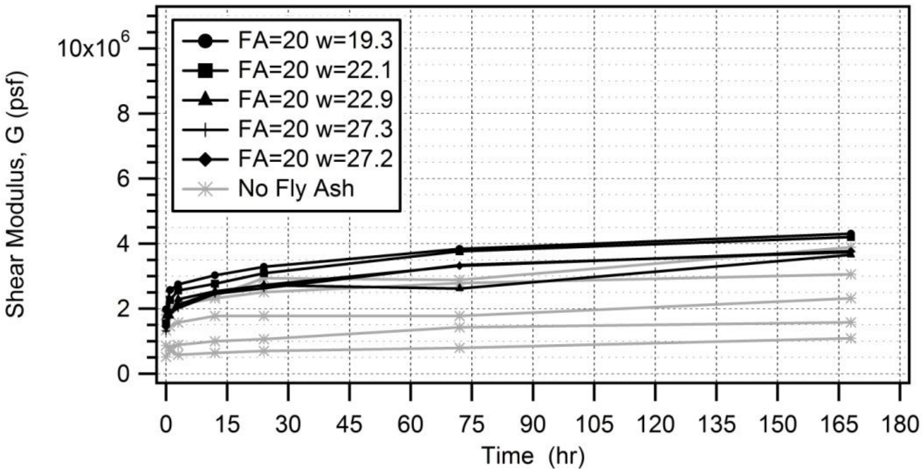


Figure B.27 Change in shear modulus with time; Putnam County soil with 20% fly ash

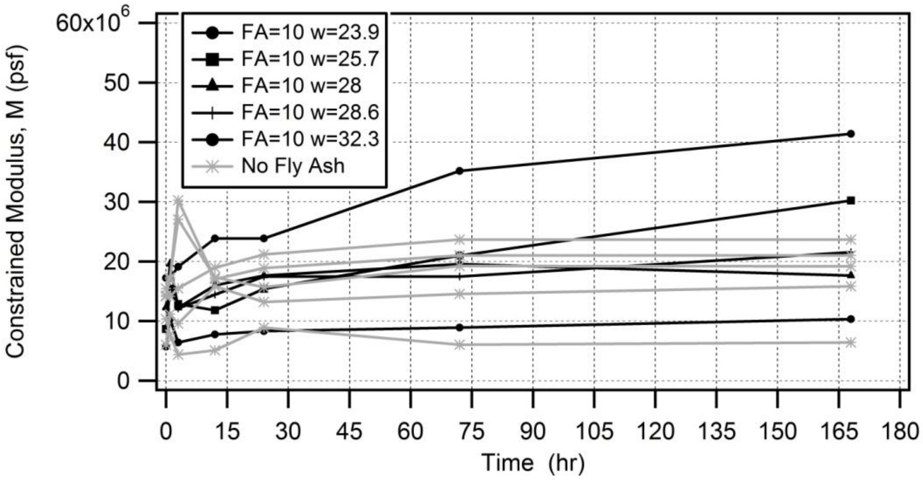


Figure B.28 Change in constrained modulus with time; Putnam County soil with 10% fly ash

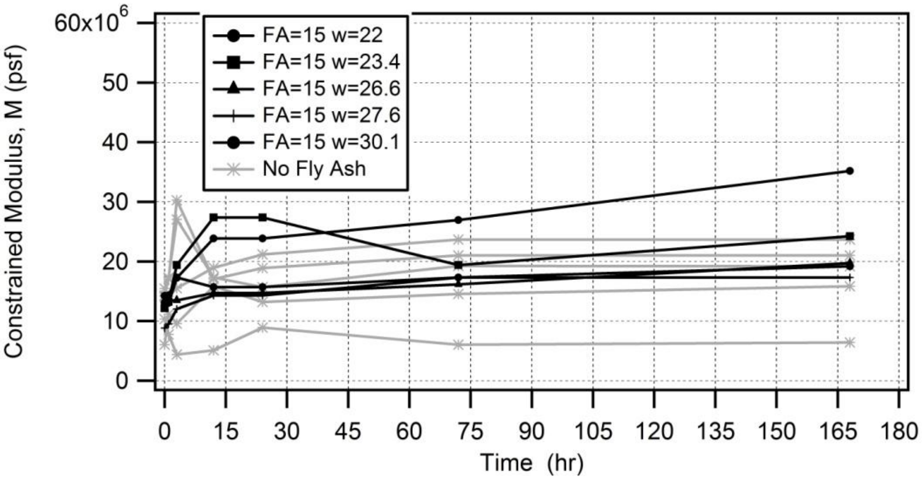


Figure B.29 Change in constrained modulus with time; Putnam County soil with 15% fly ash

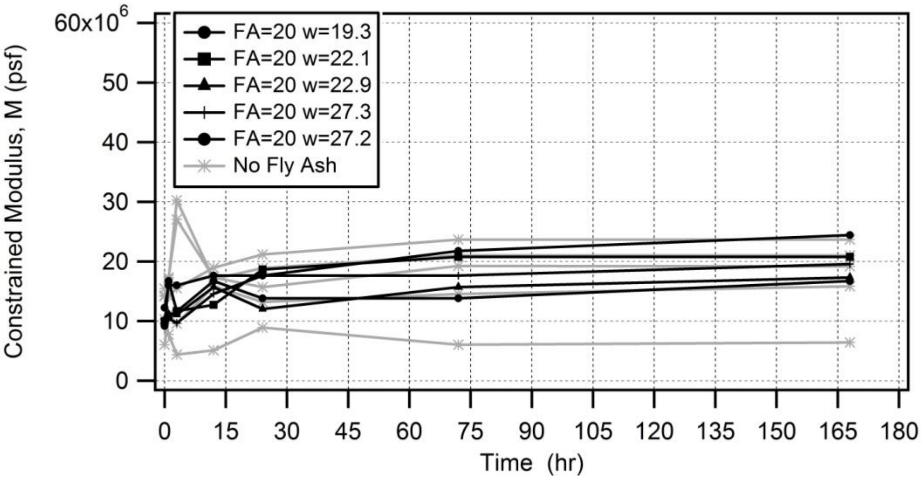


Figure B.30 Change in constrained modulus with time; Putnam County soil with 20% fly ash

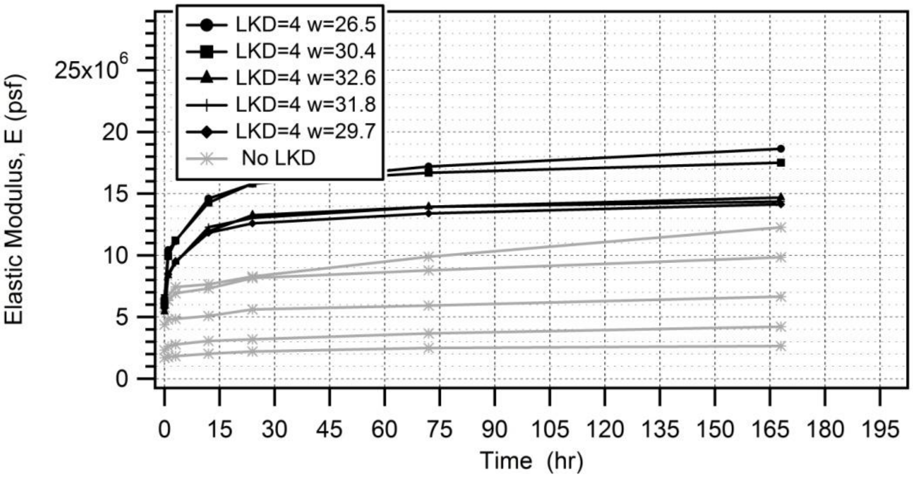


Figure B.31 Change in Young's modulus with time; Putnam County soil with 4% lime kiln dust

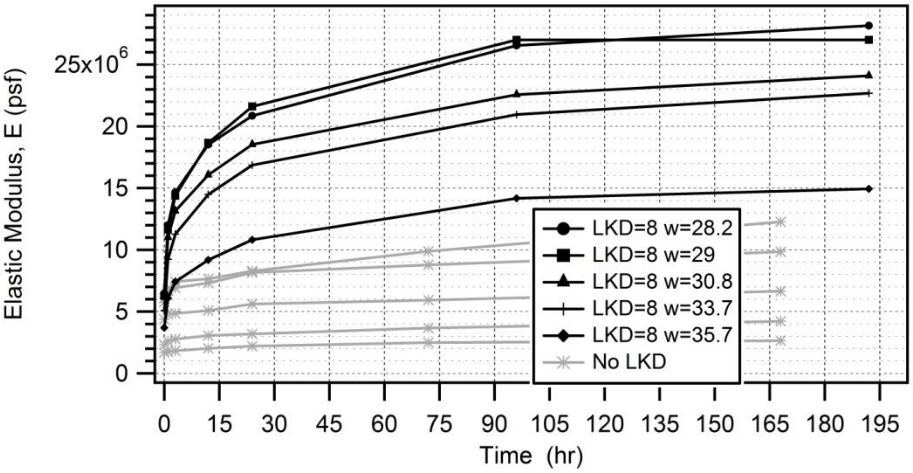


Figure B.32 Change in Young's modulus with time; Putnam County soil with 8% lime kiln dust

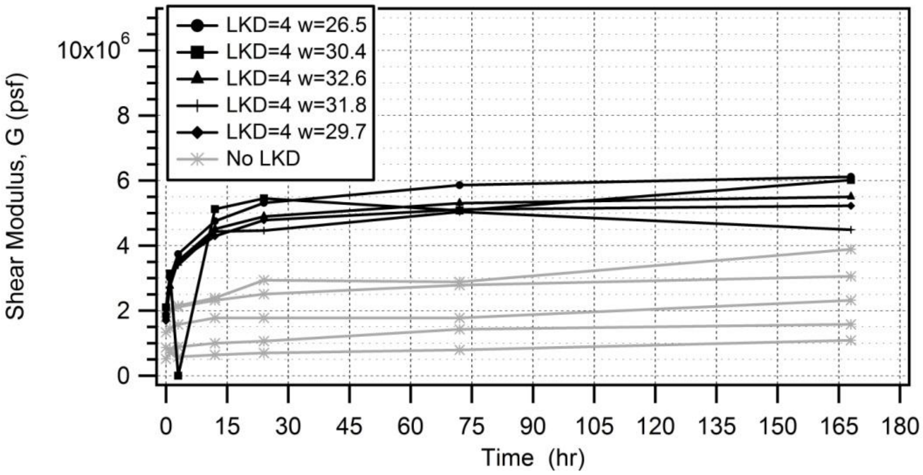


Figure B.33 Change in shear modulus with time; Putnam County soil with 4% lime kiln dust

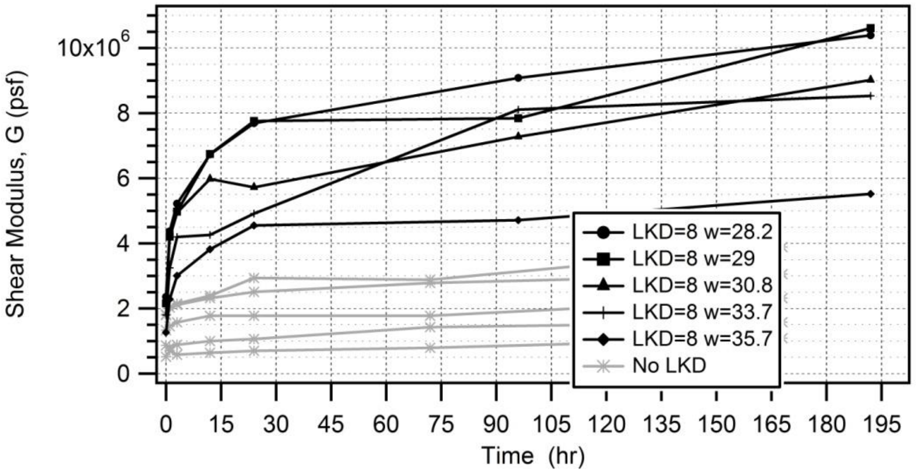


Figure B.34 Change in shear modulus with time; Putnam County soil with 8% lime kiln dust

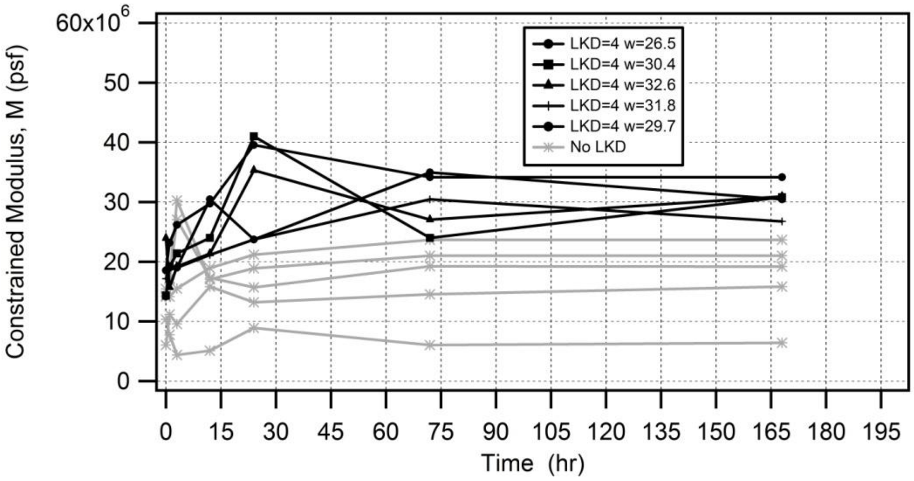


Figure B.35 Change in constrained modulus with time; Putnam County soil with 4% lime kiln dust

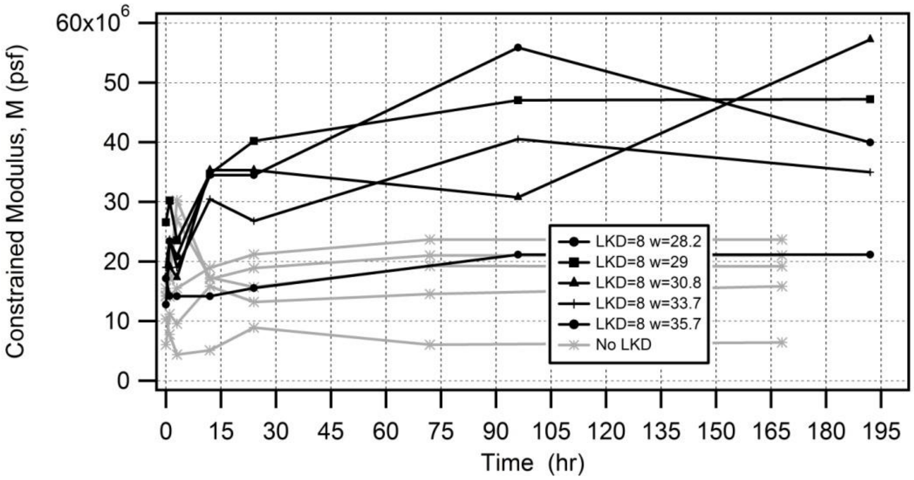


Figure B.36 Change in constrained modulus with time; Putnam County soil with 8% lime kiln dust

C. Water Content-Modulus Plots

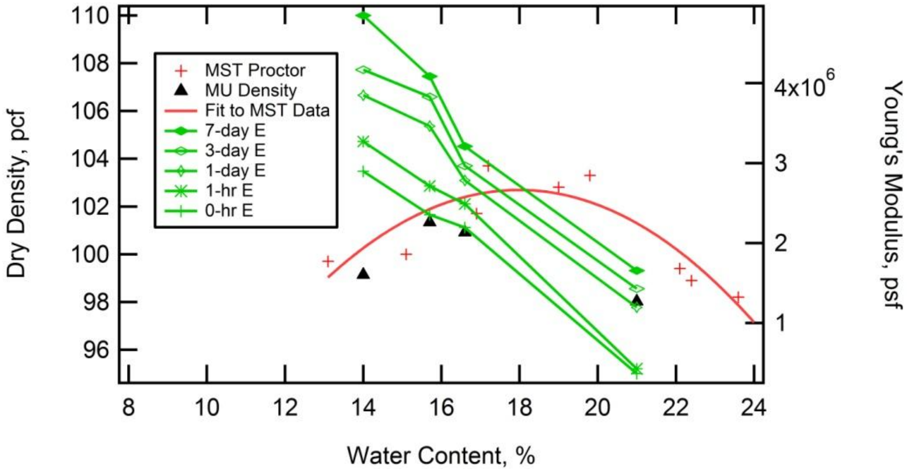


Figure C.1 Change in Young's modulus with water content; Atchison County-no additive

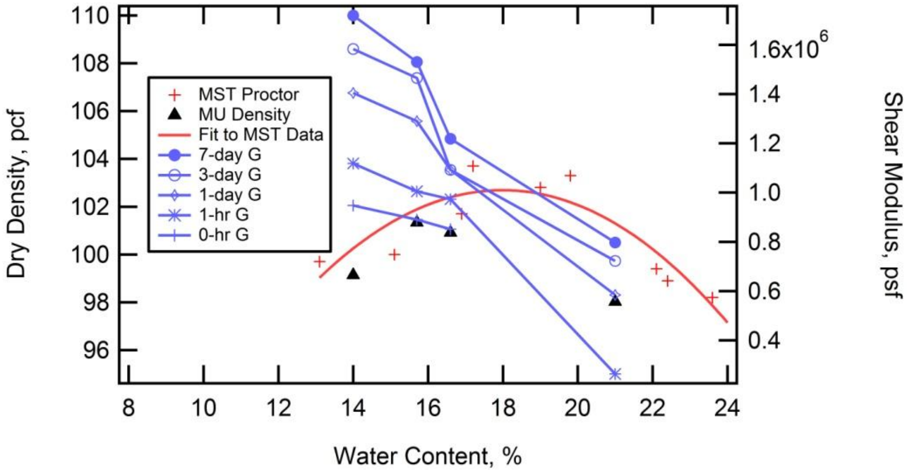


Figure C.2 Change in shear modulus with water content; Atchison County-no additive

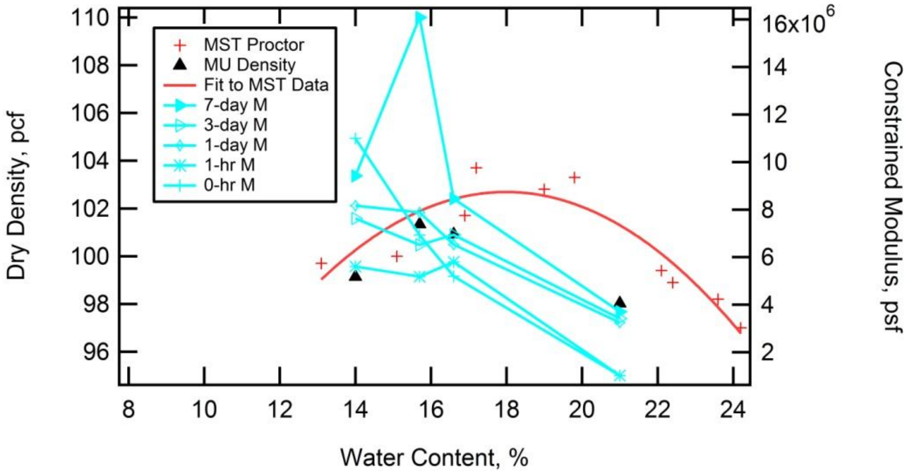


Figure C.3 Change in constrained modulus with water content; Atchison County-no additive

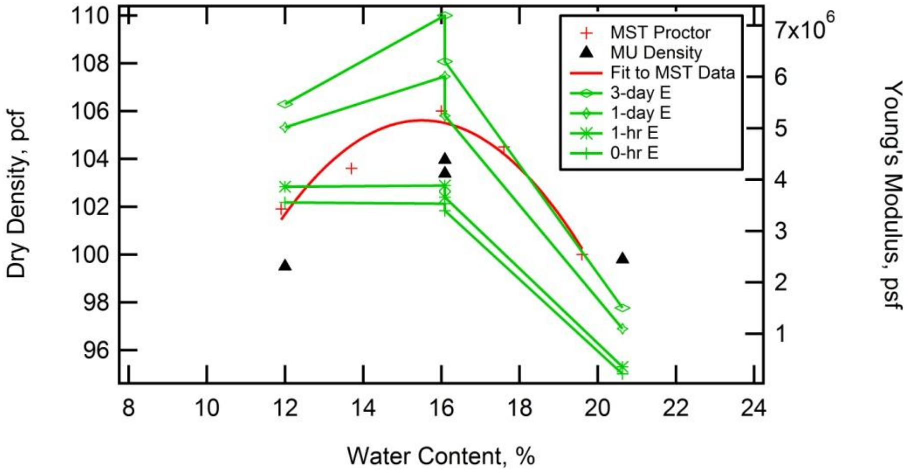


Figure C.4 Change in Young's modulus with water content; Atchison County-10% fly ash

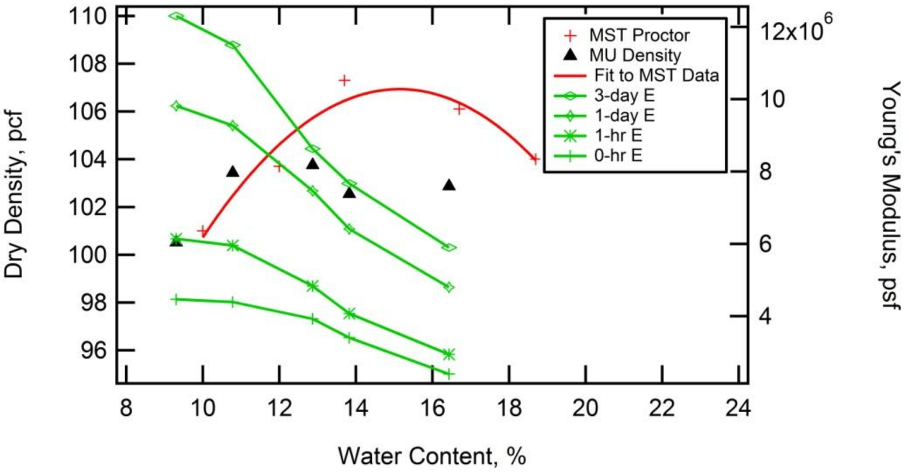


Figure C.5 Change in Young's modulus with water content; Atchison County-15% fly ash

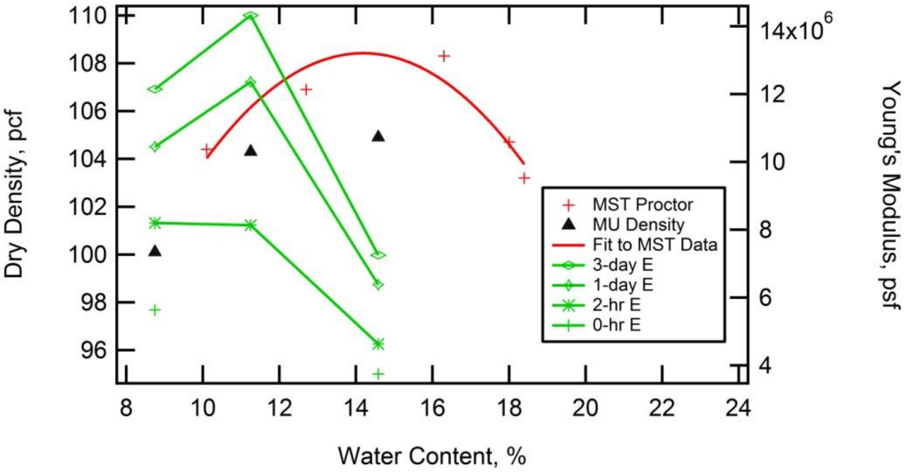


Figure C.6 Change in Young's modulus with water content; Atchison County-20 % Fly Ash

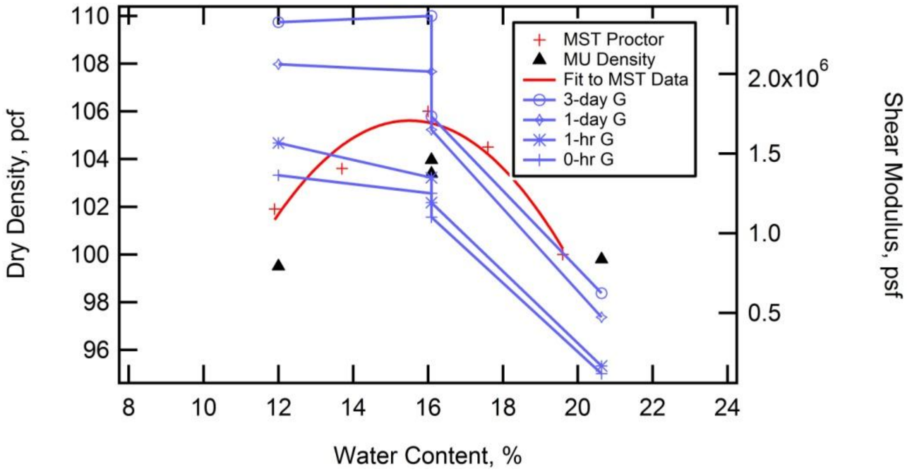


Figure C.7 Change in shear modulus with water content; Atchison County-10% fly ash

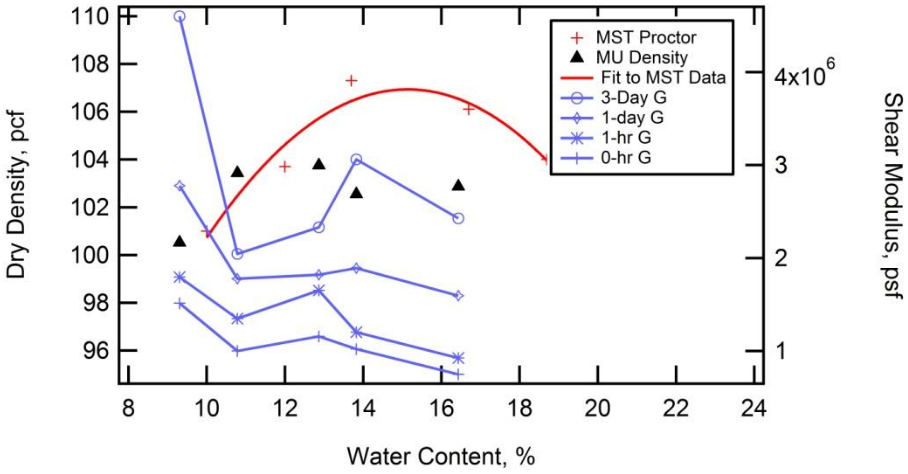


Figure C.8 Change in shear modulus with water content; Atchison County-15% fly ash

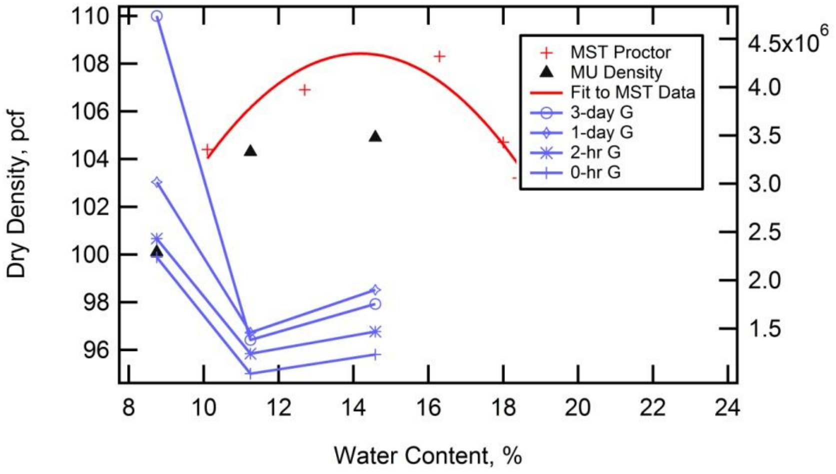


Figure C.9 Change in shear modulus with water content; Atchison County-20% fly ash

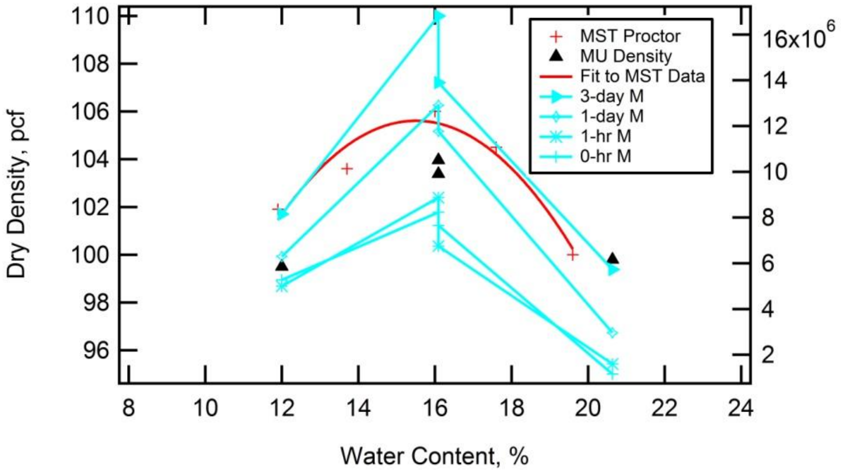


Figure C.10 Change in constrained modulus with water content; Atchison County-10% fly ash

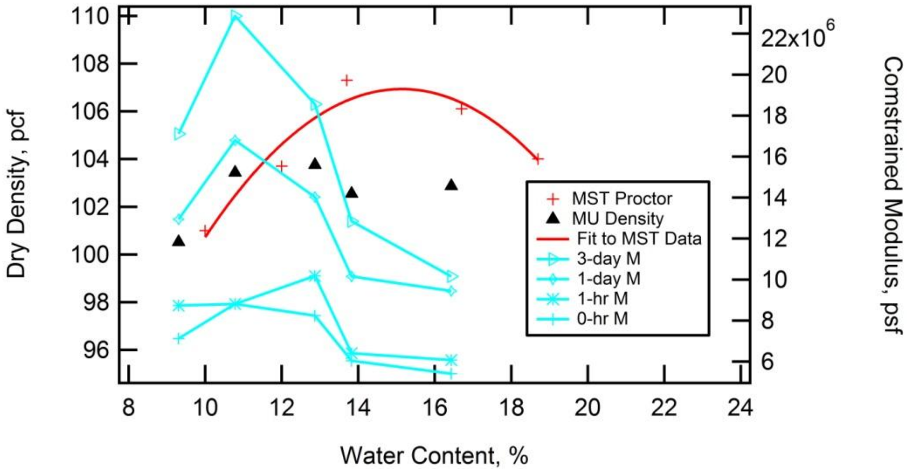


Figure C.11 Change in constrained modulus with water content; Atchison County-15% fly ash

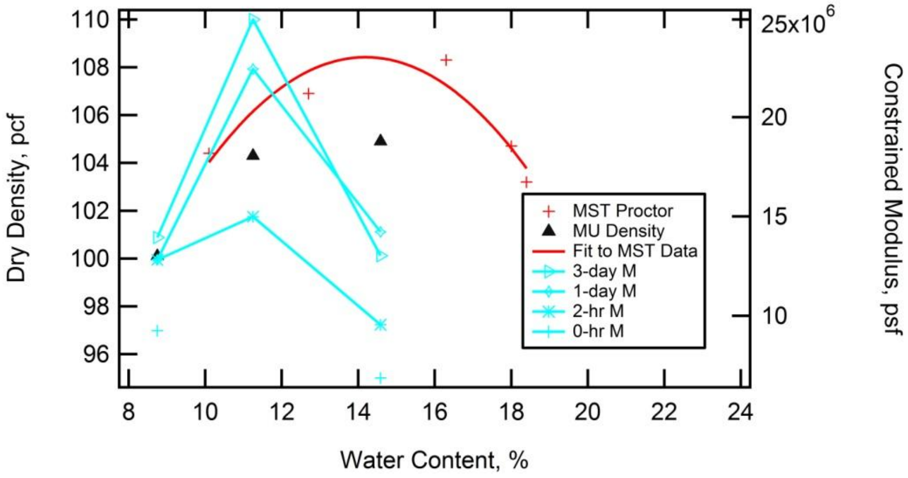


Figure C.12 Change in constrained modulus with water content; Atchison County-20 % fly ash

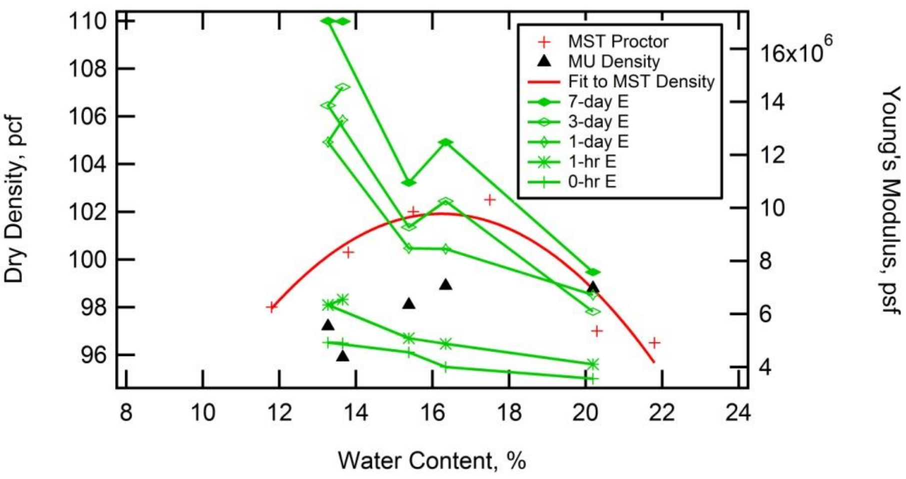


Figure C.13 Change in Young's modulus with water content; Atchison County-4% lime kiln dust

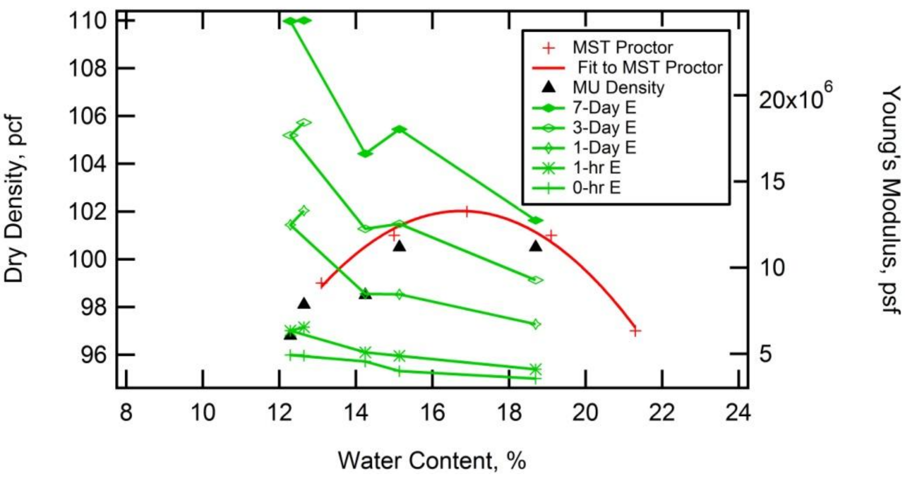


Figure C.14 Change in Young's modulus with water content; Atchison County-8% lime kiln dust

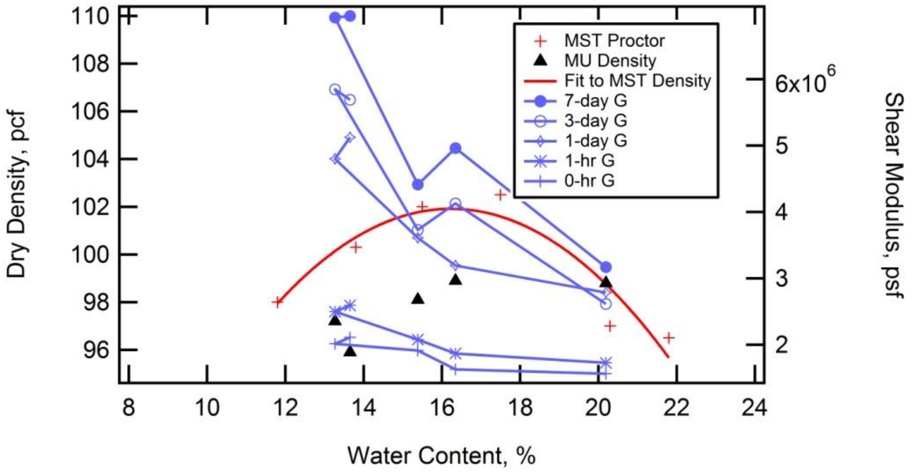


Figure C.15 Change in shear modulus with water content; Atchison County-4% lime kiln dust

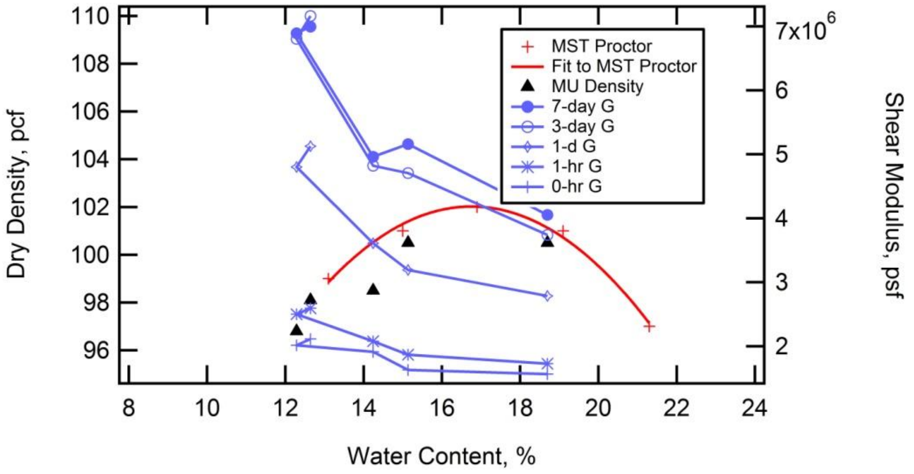


Figure C.16 Change in shear modulus with water content; Atchison County-8% lime kiln dust

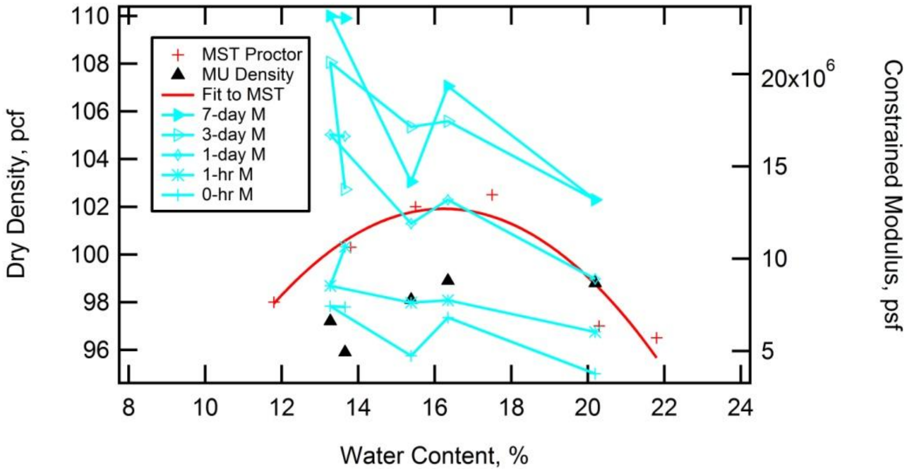


Figure C.17 Change in constrained modulus with water content; Atchison County-4% lime kiln dust

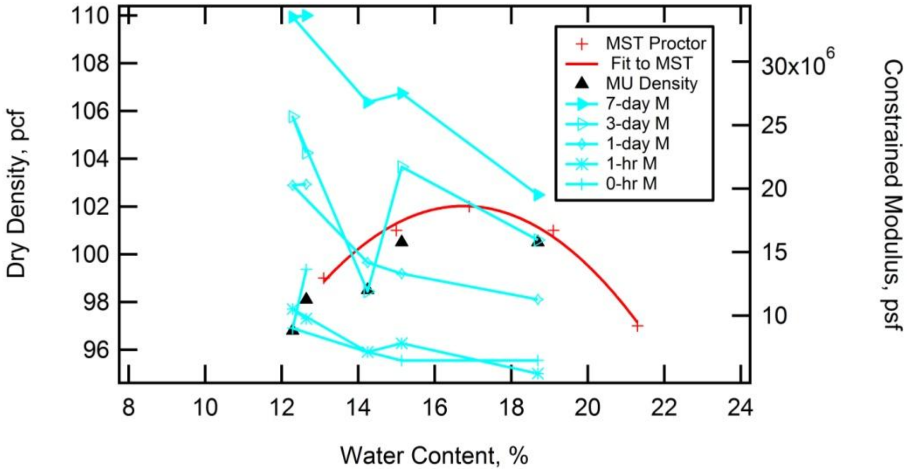


Figure C.18 Change in constrained modulus with water content; Atchison County-8% lime kiln dust

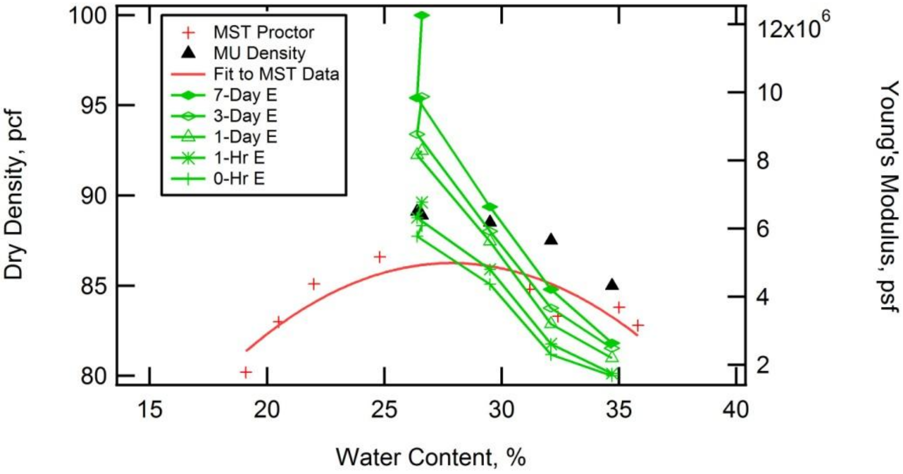


Figure C.19 Change in Young’s modulus with water content; Putnam County-no additive

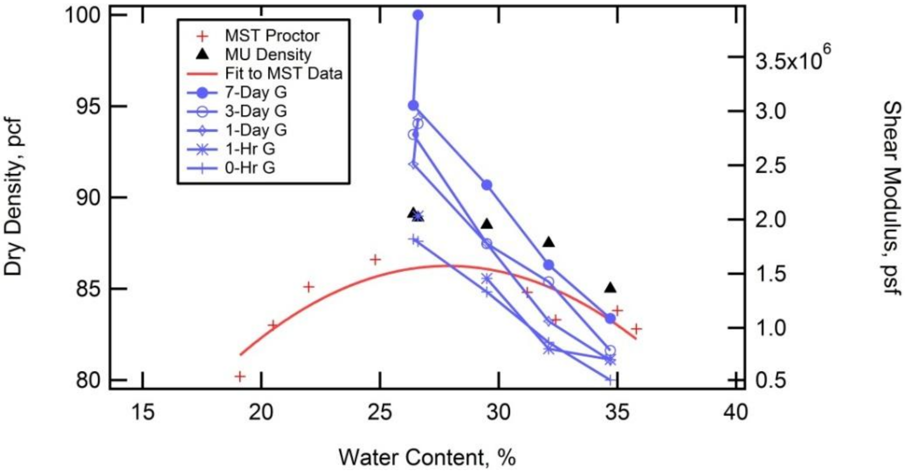


Figure C.20 Change in shear modulus with water content; Putnam County-no additive

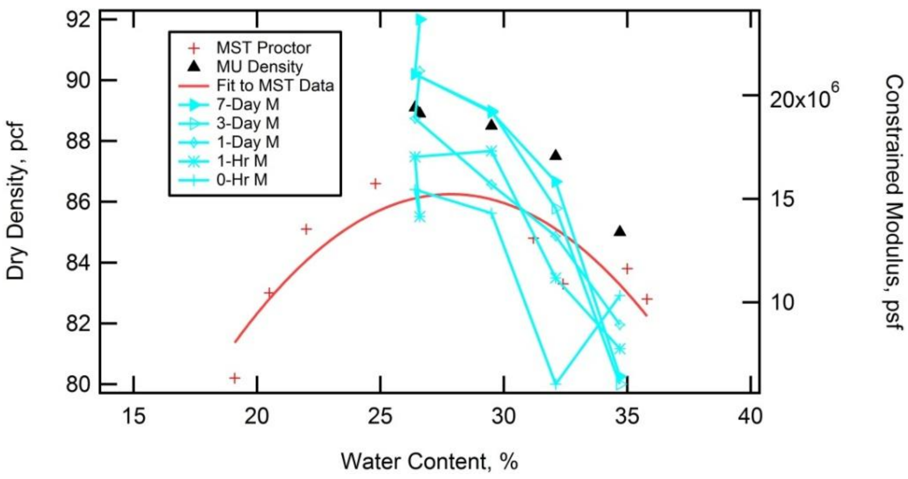


Figure C.21 Change in constrained modulus with water content; Putnam County-no additive

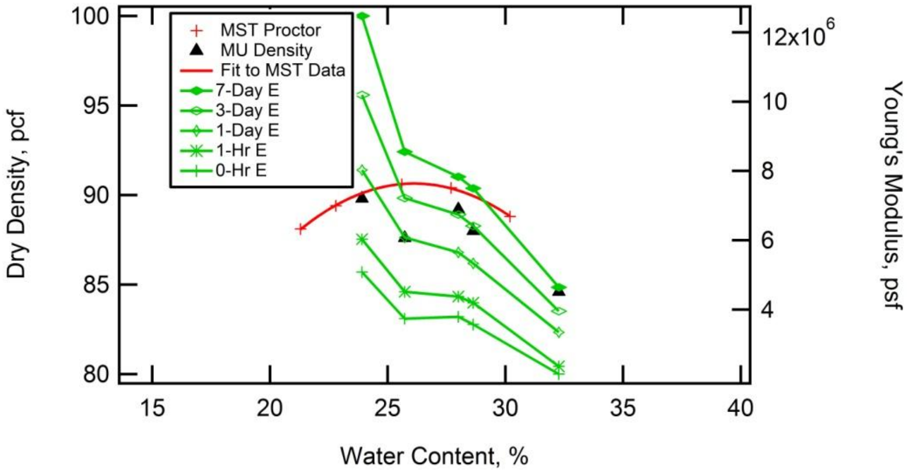


Figure C.22 Change in Young’s modulus with water content; Putnam County-10% fly ash

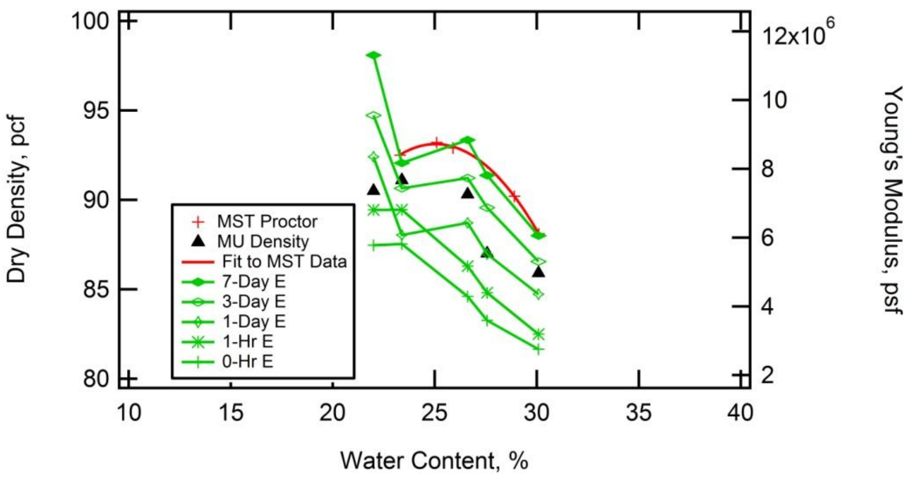


Figure C.23 Change in Young’s modulus with water content; Putnam County-15% fly ash

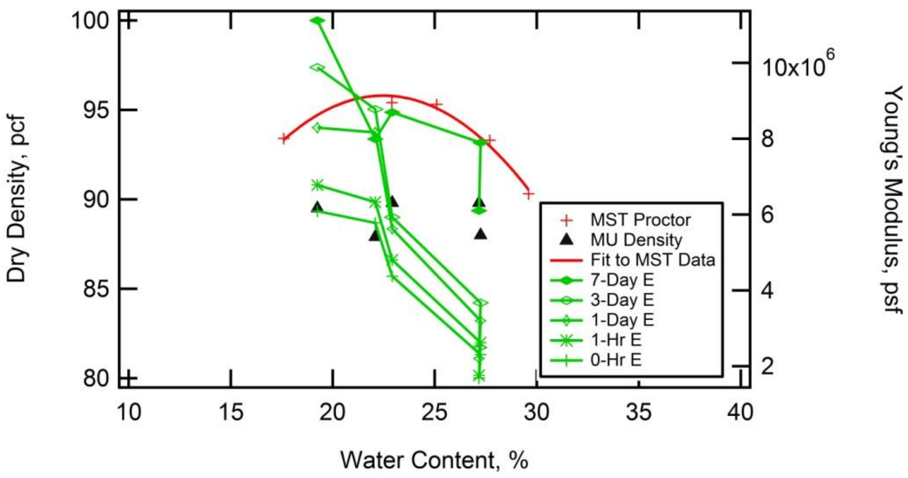


Figure C.24 Change in Young’s modulus with water content; Putnam County-20% fly ash

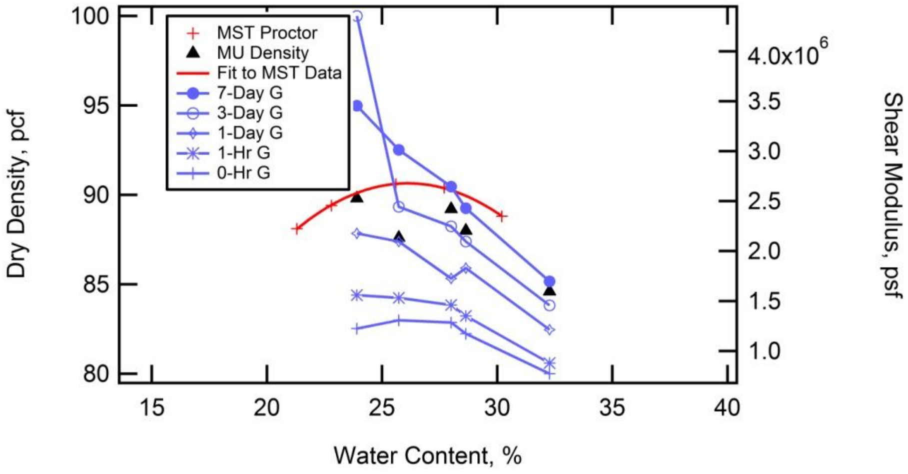


Figure C.25 Change in shear modulus with water content; Putnam County-10% fly ash

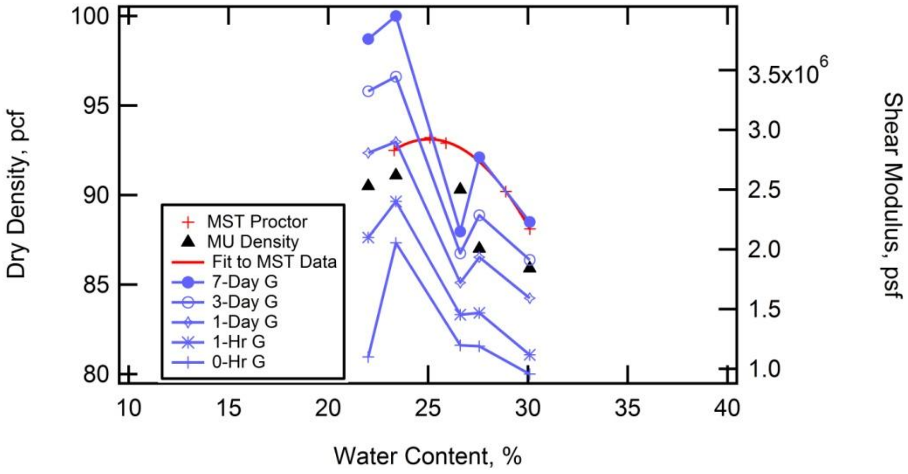


Figure C.26 Change in shear modulus with water content; Putnam County-15% fly ash

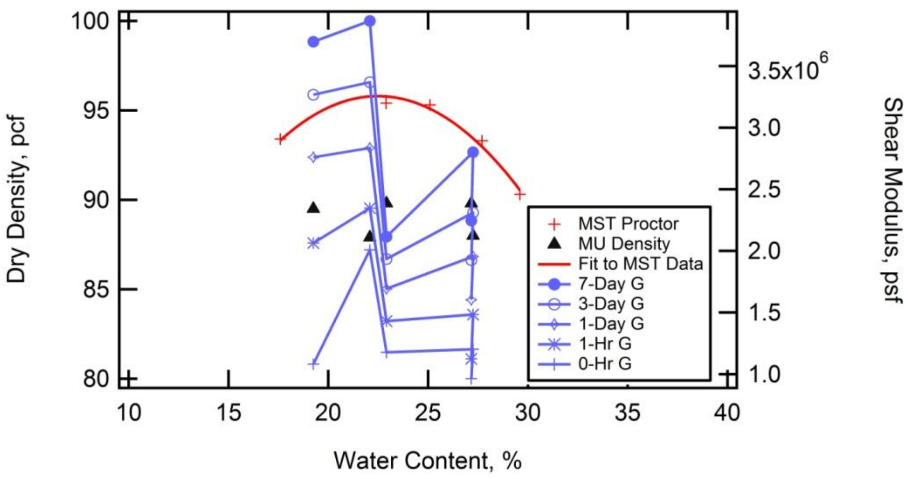


Figure C.27 Change in shear modulus with water content; Putnam County-20% fly ash

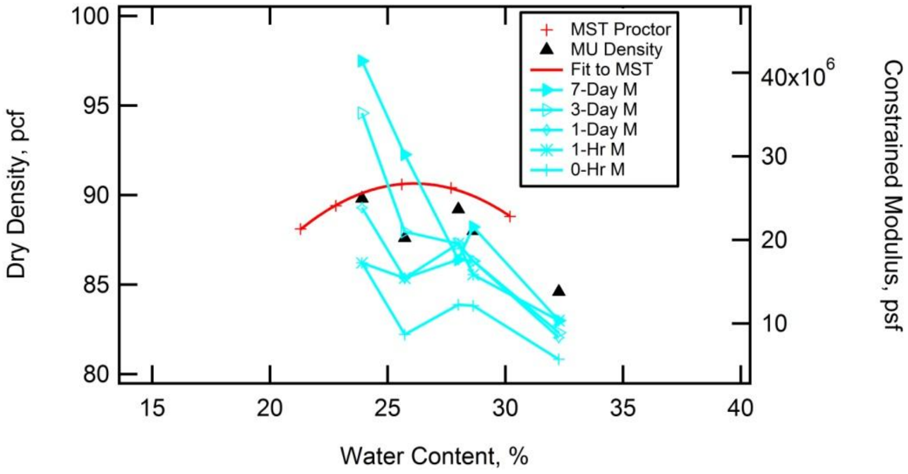


Figure C.28 Change in constrained modulus with water content; Putnam County-10% fly ash

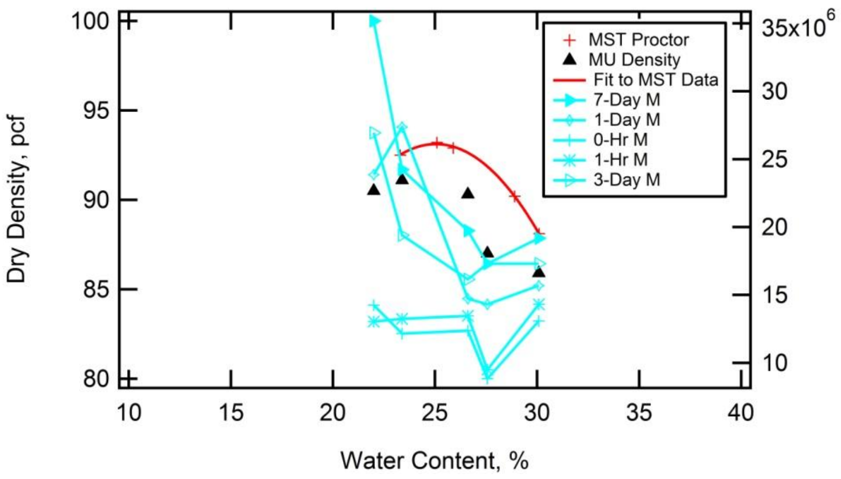


Figure C.29 Change in constrained modulus with water content; Putnam County-15% fly ash

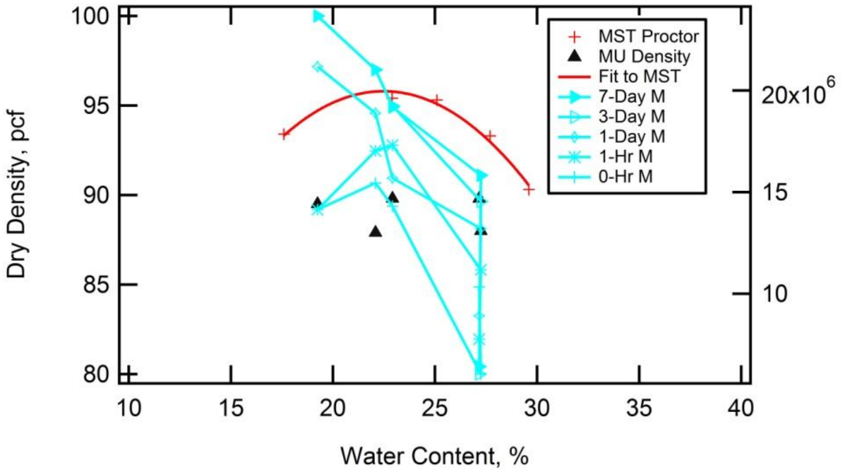


Figure C.30 Change in constrained modulus with water content; Putnam County-20% fly ash

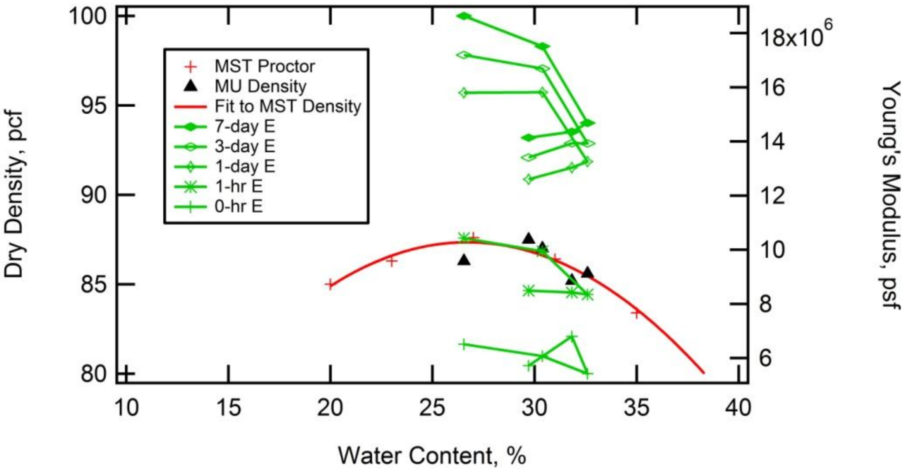


Figure C.31 Change in Young’s modulus with water content; Putnam County-4% lime kiln dust

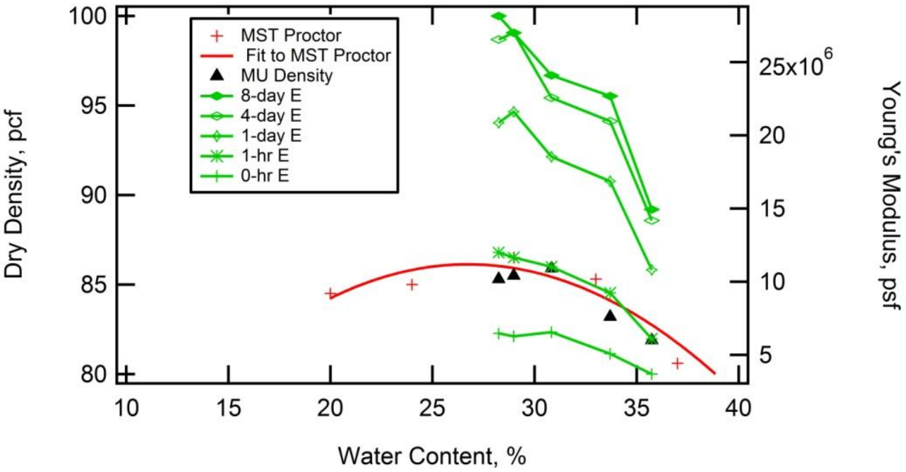


Figure C.32 Change in Young’s modulus with water content; Putnam County-8% lime kiln dust

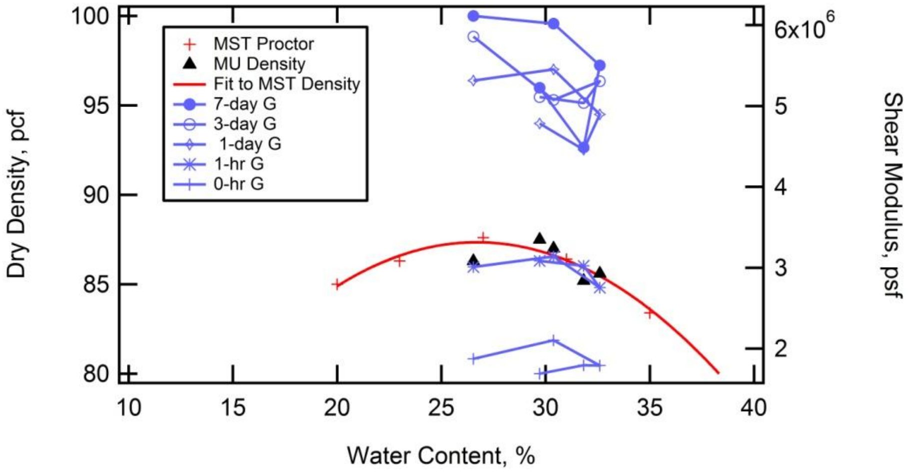


Figure C.33 Change in shear modulus with water content; Putnam County-4% lime kiln dust

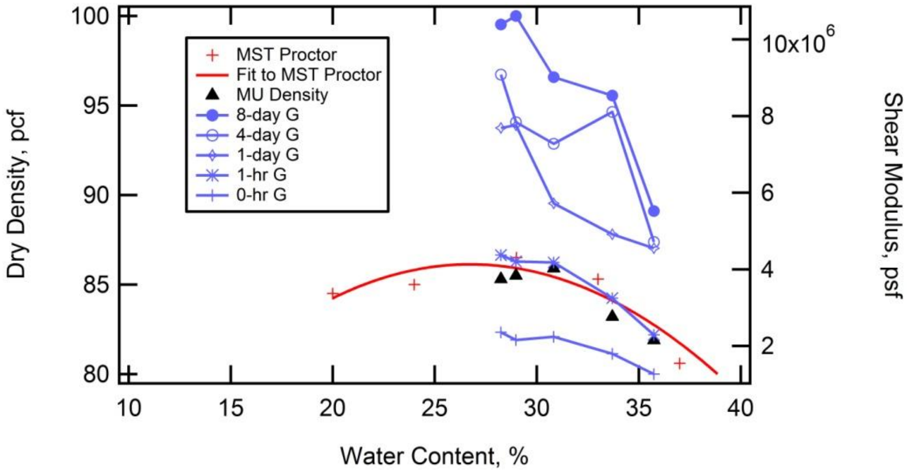


Figure C.34 Change in shear modulus with water content; Putnam County-8% lime kiln dust

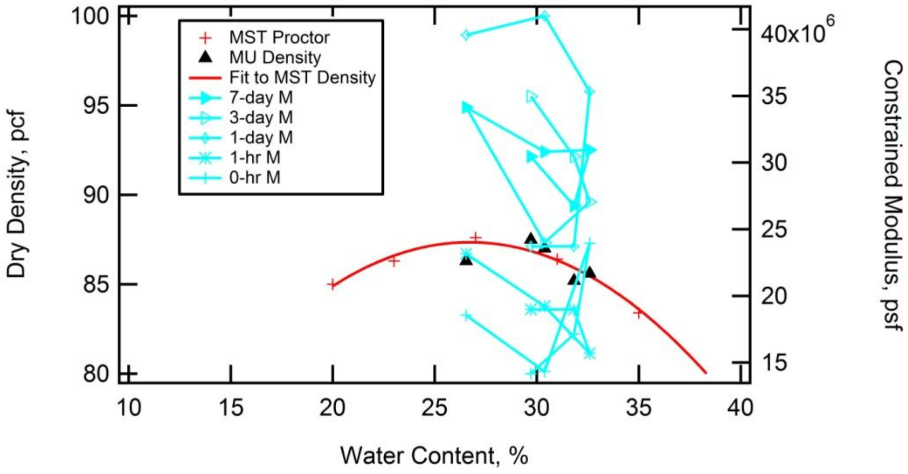


Figure C.35 Change in constrained modulus with water content; Putnam County-4% lime kiln dust

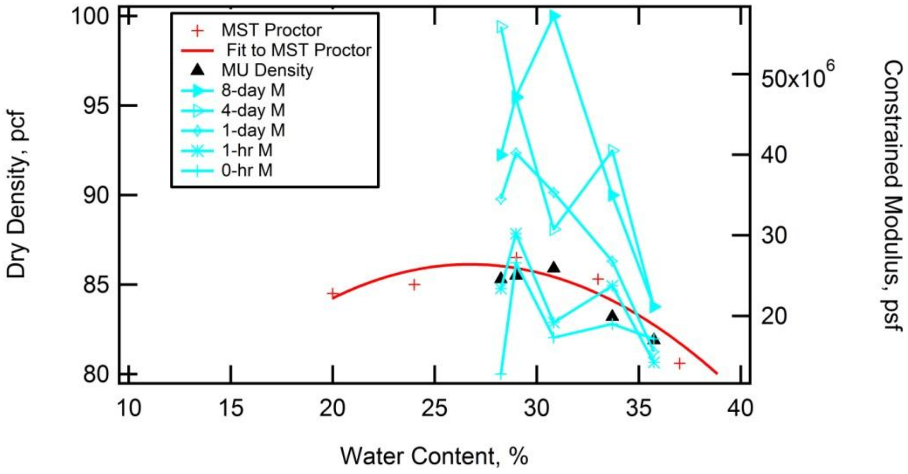


Figure C.36 Change in constrained modulus with water content; Putnam County-8% lime kiln dust

D. X-ray Diffractograms

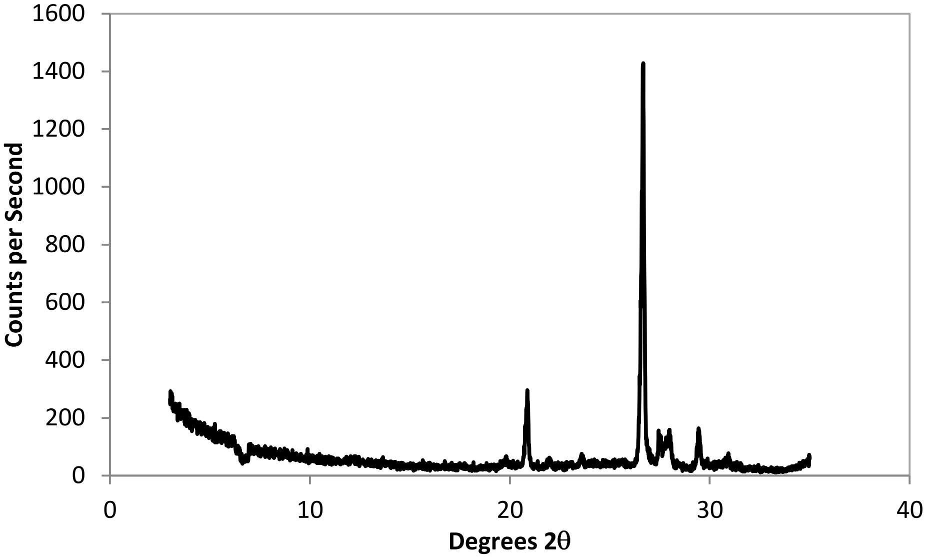


Figure D.1 X-ray diffractogram for bulk Atchison County soil specimen

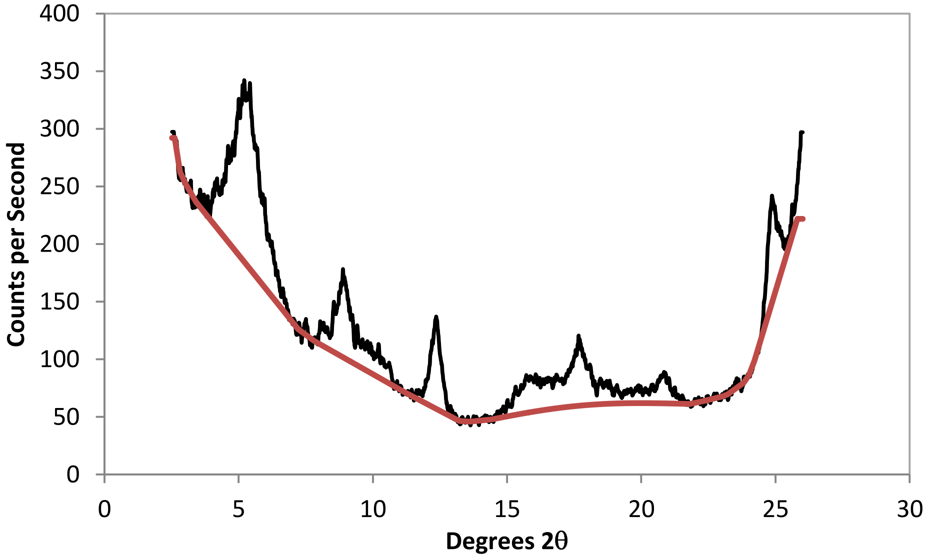


Figure D.2 X-ray diffractogram for clay-sized fraction of Atchison County soil specimen

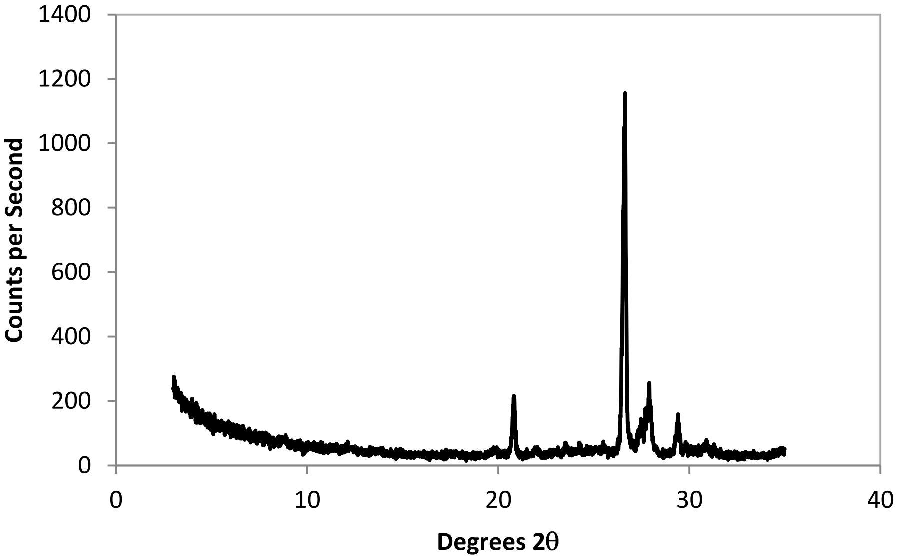


Figure D.3 X-ray diffractogram for bulk Atchison County soil with 20% fly ash specimen

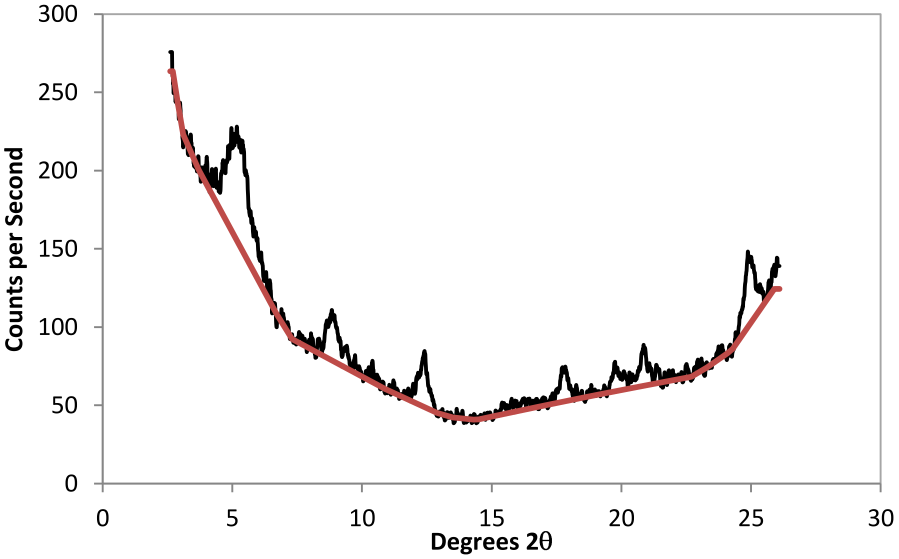


Figure D.4 X-ray diffractogram for clay-sized fraction of Atchison County soil with 20% fly ash specimen

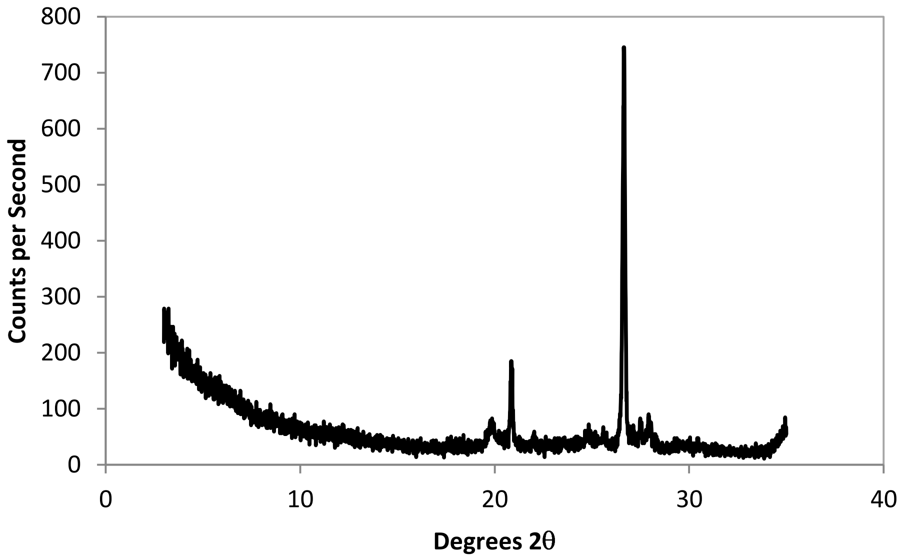


Figure D.5 X-ray diffractogram for bulk Putnam County soil specimen

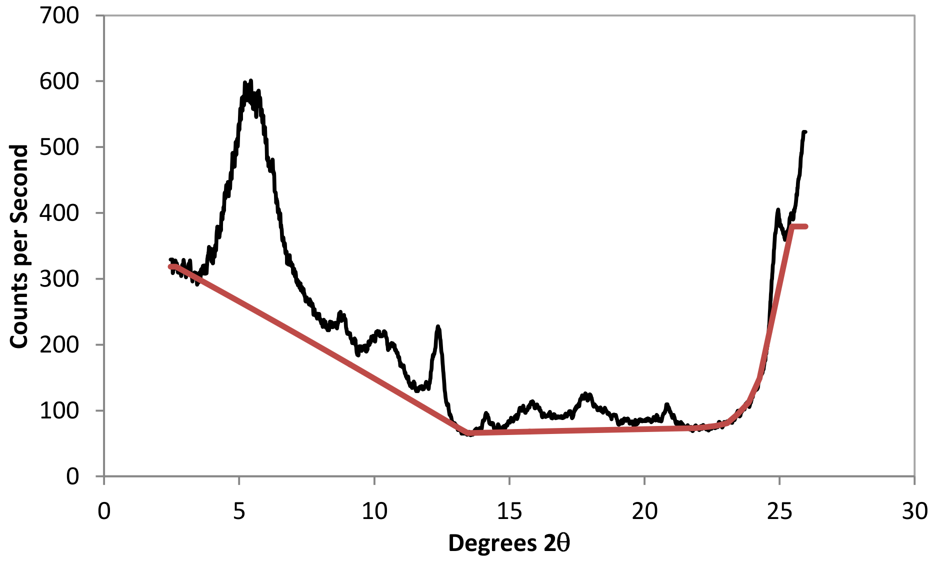


Figure D.6 X-ray diffractogram for clay-sized fraction of Putnam County soil specimen

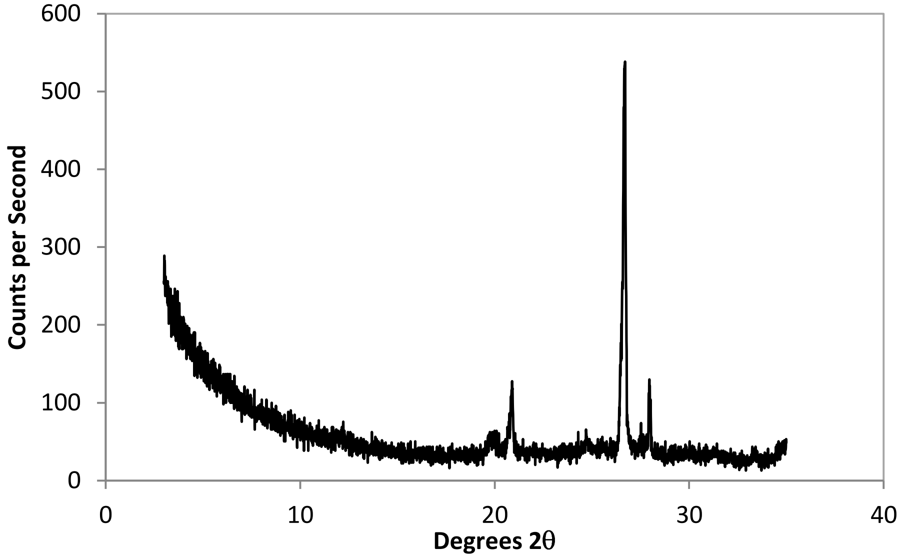


Figure D.7 X-ray diffractogram for bulk Putnam County with 20% fly ash specimen

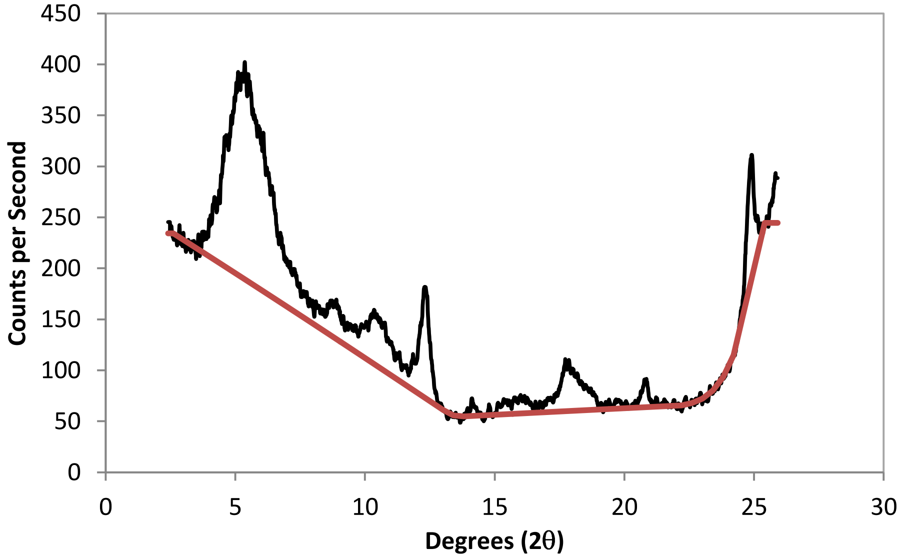


Figure D.8 X-ray diffractogram for clay-sized fraction of Putnam County with 20% fly ash specimen

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