ACCELERATED STABILIZATION DESIGN OF SUBGRADE SOILS

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16. Abstract

Chemical stabilizers are commonly used to improve the performance of problematic soils. Lime, cement, and fly ash are typically used for this purpose. To achieve effective soil stabilization, type and concentration of the stabilizer are usually calculated based on the plasticity and gradation of the material. To achieve the mix design, the optimum stabilizer content is usually based on experience or following time-consuming specifications. New accelerated testing methods are proposed that minimize the time required for soil specimen preparation, curing, and moisture conditioning to complete the design process. Proposed methods were compared to current specifications. A curing time of 2 days and a back-pressure method to complete moisture conditioning are recommended to complete the mix design in 3 days.

In addition, a more rigorous soil classification which considers the soil mineralogy is included. For that matter, simple chemical methods to determine the mineralogy of the soil fines are included. Cation Exchange Capacity, Specific Surface Area and Total Potassium could be used to substitute methods such as X-ray Diffraction and Scanning Electron Microscope.

Durability and permanency studies on the treated and untreated soils were also evaluated to address the permanency and leachability of the chemical stabilization associated with rainfall infiltration and to simulate seasonal changes. Other important issues that are evaluated are adequate mixing and curing, adequate density and moisture, adequate short-term and long tern strength and stiffness, and proper construction.

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Executive Summary

Problems encountered with subgrade failures due to the loss of stabilizer over time, or ineffectiveness or insufficient amount of stabilizer in some soils is well documented. If the selected concentration of additives is not adequate to ensure short- and long-term durability of a pavement layer, the stabilization will be ineffective, and pavement rehabilitations will be necessary, requiring costly repairs and road closures.

The proper type and concentration of stabilizers are typically determined considering the plasticity and the gradation of the subgrade. A number of other parameters such as the interaction between the mineralogy of the materials and additives in presence or absence of moisture, construction methods and curing processes significantly impact the performance of stabilized subgrades as well.

TxDOT currently requires acceptable strength in lieu of most durability tests to select the optimal additive. Strength tests can be conducted more rapidly than durability tests (7 days vs. 1 month) and require less laboratory equipment and technician training. However, achievement of a specified strength does not always ensure durability. Moreover, the current TxDOT procedures for selecting the optimum additive are time-consuming and the protocols for determining the level of moisture conditioning are not currently available. As part of this research, new accelerated testing methods are proposed that could minimize the time required for soil specimen preparation, curing, conditioning and testing time to complete the design process. Some of the shortcomings that exist in the current protocols to establish whether the stabilizer or stabilization method is deemed to be effective in the field construction projects are also addressed.

To develop new and accelerated protocols, several soils with different characteristics and different stabilizers were evaluated based on current and proposed methods of moisture conditioning under different curing and compaction methods. These soils were categorized in terms of traditional methods and their mineralogical properties. These materials were then evaluated following the current TxDOT specifications and compared with the proposed methods of moisture conditioning, using back-pressure, vacuum, or submergence of the specimens.

To select the desirable alternatives, simplicity and applicability of the test were considered as well as their correlation with methods covered in current specifications. For that matter, a protocol that could be performed on stabilized soils within one week was developed to address the effectiveness of stabilization as the main outcome of this research.

Another focus of this project was to develop several test methods to determine various chemical characteristics of the soils according to their mineralogy. Cation Exchange Capacity, Specific Surface Area, Total Potassium, Exchangeable Potassium and Reactive Alumina could be used for that matter to substitute more costly and time consuming methods such as X-ray Diffraction and Scanning Electron Microscope.

Implementation Statement

In this report some of the curing and moisture conditioning methods for stabilized specimens have been investigated in order to reduce the time required to provide a proper mix design for stabilized subgrades. The proposed methods can be readily implemented by the districts, in terms of the laboratory practices and design processes. As part of the implementation, several guidelines have been prepared for conducting lab tests to determine the effectiveness of the stabilizers based on the results from eleven soils.

To further assess of the recommended methods and to adjust the limits or criteria, the recommended methods should be implemented on a number of new and ongoing projects.

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CHAPTER ONE – INTRODUCTION

PROBLEM STATEMENT

Chemical stabilizers are commonly used to improve the performance of high plasticity soils, soils with high volumetric changes, poor workability, and low strength. Lime, cement, and fly ash or combinations of them are typically used for this purpose. To achieve effective soil stabilization, special attention needs to be given to proper type and concentration of the stabilizer. In the state of practice, this is usually based on the plasticity characteristics and gradation of the material. For more effective subgrade stabilization, a more rigorous soil classification which considers the soil mineralogy is needed for evaluating and assessing the use of certain chemical additives for enhancing soil properties. Lack of such finer soil classification often creates dilemma in users on whether or when a certain chemical treatment method could be used effectively. Hence, better and more reliable strategies are needed for screening chemical stabilizers for potential and successful use of them under field conditions. Other important issues that need to be addressed are adequate mixing and curing, adequate density and moisture, adequate short-term and long tern strength and stiffness, and proper construction.

Although many projects constructed with stabilized materials have achieved satisfactory results, sometimes the stabilized material has not responded well due to loss of stabilizer over time or in some cases the stabilizer has been ineffective due to inadequacy of its type or amount added. Challenges still remain to determine the optimal use of these materials. Extensive research has been performed with regard to the engineering properties, reliability and durability of the stabilized materials. Likewise, different testing, design, construction, and quality assurance/ quality control (QA/QC) methodologies have been developed for different stabilized materials.

Typically, there are two different methods to determine the mix design requirements depending on the objective: durability tests and strength tests. Even though strength tests can be conducted rapidly and require simple laboratory equipment and technician training, a specified strength does not always ensure durability. TxDOT currently uses strength methods in lieu of most durability tests. In addition to durability and strength, other tests should be used to ensure adequate performance, including soil and aggregate characterization, stiffness, chemical compatibility and revised durability tests. Main aspects regarding durability and strength tests are described below.

The strength development and durability of stabilized materials are highly dependent on curing time, temperature, and moisture conditioning. The curing conditions in the field commonly differ from those of the laboratory. To produce the target strength in stabilized materials it is necessary to specify the curing time, along with an allowable curing temperature range.

The optimum additives is currently carried out in TxDOT using test methods Tex-120-E, Tex-121-E, and Tex-127-E depending on the type of the additive. The specifications that govern the placement of stabilized materials fall under Items 260, 263, 265, 275 and 276. Even though the TxDOT stabilization guide is an excellent starting point for the selection of the stabilizers, it only has official sets of target strengths for lime and lime-fly ash specimens. There are no official targets for soil-cement specimens or retained target strengths for any stabilizer. Additionally, no procedure is established to examine whether the required strength has been attained. Besides, standard methods for performing moisture conditioning and procedures for determining the level of moisture conditioning are not currently available to TxDOT. Since the current procedures for selecting the optimal additive content for pavement layers are time consuming, several districts prefer to bypass the specifications and rely on their own experience to select the type and concentration of additives.

Aside from the laboratory assessment, it is essential to assess the quality of the final product and not to rely solely on laboratory results. Therefore, a comprehensive field monitoring of the sections must be conducted immediately after construction and periodically during the life of the section.

OBJECTIVE AND APPROACHES

The goal of this research project is to address some of the shortcomings that currently exist in TxDOT stabilization protocols and develop new, or revised accelerated testing methods that could potentially reduce or minimize the time required for sample preparation, curing, moisture conditioning and laboratory testing. To accomplish this, the ideal and retained strengths and moduli, as well as the results from the long-term durability obtained from the proposed alternatives were compared. In order to achieve these goals, the results from alternative methods such as back-pressure saturation, vacuum suction, submersion, and those currently practiced by TxDOT were compared.

Another important objective of this study was to address the permanency of chemical stabilizers, i.e. the additive to hold the soil particles together for a long duration. To overcome the lack of knowledge of clay mineralogy and its impact on chemical stabilization, mineralogical soil studies utilizing X-Ray Diffractometer (XRD) and Scanning Electron Microscope (SEM) with energy dispersive measurements were conducted. Since such direct methods are expensive and require advanced instrumentation, they are suitable for day-to-day use of the district personnel. For that matter, a few other indirect methods utilizing soil property measurements were considered for evaluating the clay mineralogy. Tests proposed included the Cation Exchange Capacity (CEC), Specific Surface Area (SSA), Total Potassium, Exchangeable Potassium and Reactive Alumina. The first three methods are known to provide clay mineralogy information, whereas the last two methods provide reactive chemicals in soils that could result in pozzolanic compound formation. Such methods were useful in the development of better screening strategies for stabilizer selection based on mineralogy information. These indirect methods were evaluated by performing comprehensive mineralogy investigations on the soils selected, both prior and during the chemical stabilizer treatment studies.

To develop the preliminary guidelines, the existing TxDOT specifications were considered carefully. Several variables including soil type and curing conditions were studied. To provide a preliminary guideline, the short-term and long-term stability, potential for water retention and loss of stiffness using the Tube Suction and Free-free Resonant Column tests were considered, which are not included in the current TxDOT specifications. As part of this study strength and modulus were measured after moisture conditioning to obtain the retained strength and modulus.

In this project several soils with different characteristics were selected. These soils were subjected to the developed test protocol and current specifications. The purpose of this activity was to ensure that the proposed procedures are realistic and applicable to a wide range of soils found in Texas. These results can be used to possibly develop correlations between the current and proposed guidelines.

ORGANIZATION

Chapter 2 contains a review of relevant literature regarding design of subgrade stabilization. A review of current TxDOT specifications is included, followed by a summary of other specifications adopted by several other organizations and state agencies. Some of the conventional and recently developed moisture conditioning processes to accelerate stabilization design are also described in that chapter.

The selection of proper additive according to the current TxDOT stabilization guidelines is included in Chapter 3. The objective was to evaluate the current TxDOT specifications for soil stabilization and develop a baseline database that covered a wide range of soils in terms of material properties and environmental conditions. A total of six soils were studied to complete the baseline database. In addition, the clay mineral identification was performed on all treated and untreated soils as presented in that chapter. Also, tests including Reactive Alumina and Silica measurements were conducted to further understand possible pozzolanic reactions that might have taken place in the treated materials.

The objective of Chapter 4 is to evaluate accelerated curing methods to supplant current TxDOT specifications for soil stabilization. In this chapter different methods of compaction used in this study were compared first. Since the main objective of this project is to accelerate the time needed to complete the mix design, different methods of moisture conditioning were considered to reduce it. The time needed to get similar results with different methods was evaluated and new methods of curing and moisture conditioning were proposed. The simplicity and applicability of the tests were considered for selecting the best alternative as well.

Chapter 5 mainly focuses on durability and permanency studies on the treated and untreated soils evaluated in Chapter 3. Two series of moisture conditioning tests were conducted to address the permanency and leachability of the chemical treatment. The first method looks into the leaching problems associated with rainfall infiltration and the second method addresses the permanency of stabilizer by addressing volumetric and strength changes of the soil during wetting and drying cycles. These cycles can also be used to simulate seasonal changes.

Chapter 6 presents the results obtained from the field monitoring and validation of preliminary guidelines presented in Chapter 4. Laboratory and field results of a total of six soils obtained from different construction projects within Texas are presented. For that matter, the mix design provided by TxDOT personnel was evaluated first for each material and the same laboratory tests to characterize the baseline materials from Chapter 3 were conducted. The comparison between the results from laboratory tests based on current TxDOT specifications and those proposed as new moisture conditioning protocols in Chapter 4 were also conducted. The average moduli obtained from the field were compared to the moduli measured in the lab for different moisture conditioning methods as well. In addition, long-term durability studies on these soils are also included. The same moisture conditioning tests presented in Chapter 5were conducted to address the permanency and leachability of the chemical treatment on the validation soil.

A manual that establishes evaluation criteria to select type and amount of stabilizer, methods of evaluating the effectiveness of stabilization, and methods for moisture conditioning to accelerate the mix design is included in Chapter 7. The revised procedure is an adaptation of the current TxDOT guidelines for subgrade stabilization, with the addition of chemical analysis of the soil fines to identify clay mineralogy, accelerated curing and moisture conditioning to minimize to time to achieve the mix design and wetting-drying cycles as means of durability indicators.

Chapter 8 presents the summary, conclusions and recommendations obtained from this project.

CHAPTER TWO – BACKGROUND

INTRODUCTION

An extensive literature review that documents accelerated design of subgrade stabilization is incorporated here. A review of current TxDOT specifications is introduced, followed by a summary of other specifications adopted by several other organizations and state agencies. Some of the conventional and recently developed moisture conditioning processes to accelerate stabilization design are also described in this chapter. A number of material-related, construction-related and environmental related factors play roles in accelerating the evaluation of the stabilized subgrades. Performance of stabilized subsoils depends on soil compositional variables, environmental variables and test related variables. The following items were reviewed for better understanding of the proper performance of stabilized subsoil:

- Soil characteristics
- Soil stabilization
- Strength and stiffness properties
- Curing
- Moisture conditioning
- Durability and leaching
- Field monitoring

The following sections summarize each of these topics along with an explanation on how these topics can be related or utilized in accomplishing the research objectives.

SOIL CHARACTERISTICS

The Plasticity Index (PI) of the soil has a direct impact on the selection of the additives. The determination of the optimum moisture content (OMC) and maximum dry density (MDD) is also very important when the material is mixed with the stabilizer. There is always a compaction moisture content at which the maximum strength and stiffness are achieved. This moisture content is typically on the dry of optimum or close to optimum moisture content. In certain soils, strength and stiffness properties are enhanced due to increased pozzalanic activity taking place in the presence of wet of optimum moisture condition. In few other soils, opposite trends of decrease in strength and stiffness are possible due to presence of low reactive minerals in the soils. The strength and stiffness of materials is also largely related to their compaction density, which depends significantly on the particle size distribution.

A thorough investigation of the behavior of the stabilized soil mixtures with the change in material and type of stabilizer should be considered in any study. Addition of stabilizer to soil affects the optimum moisture content and the maximum dry density. The techniques used during construction/compaction of a layer play an important role in how well the stabilized layer performs. Usually, the density and moisture content of a compacted material in the field may differ from MDD and OMC of the material.

A thorough understanding of chemical compatibility issues in soil stabilization projects requires certain background knowledge of soil or clay mineralogy and methods to identify dominating clay mineralogy in soils. The majority of the reported stabilization problems on projects built on expansive soils. Hence, mineralogical aspects of problematic soils, in particular expansive soils are discussed next.

Expansive Soils

Expansive soils, also known as swell-shrink soils, have been problematic for the civil infrastructures including roads and foundations (Nelson and Miller, 1992). Expansive soils swell and shrink with changes in moisture content. The reason for this behavior is the presence of certain type of mineral known as montmorillonite that has an expanding lattice. This clay mineral expands when it is exposed to water. Soils rich with these minerals can be found in many places all over the world especially in the arid and semi-arid regions (Hussein, 2001). Soils containing significant quantities of the minerals such as bentonite, illite, and attapulgite are characterized by strong swell or shrinkage properties. Kaolinite is relatively non-expansive (Johnson and Stroman, 1976). According to Wiseman et al. (1985), the following factors can be used to classify a soil as problematic or not:

1) Soil type that exhibits considerable volume change with changes of moisture content
2) Climatic conditions such as extended wet or dry seasons
3) Changes in moisture content (climatic, man-made or vegetation)
4) Light structures that are very sensitive to differential movement

Expansive soils can be identified by using the index tests in Table 2.1. A summary of various methods for identifying the expansive nature of soils can be found in Puppala et al. (2004). One of the soil characteristic that is less understood is the dominating clay mineralogy in a given soil system. Since clay mineralogy is directly related to the overall expansive nature of subsoil, an attempt is made to provide an overview of the clay mineralogy in the next section.

Table 2.1 – Expansive Soils Identification (from Wiseman et al., 1985)

Index Test	Usually No Problems	Almost Always Problematic
Plasticity Index	**<20**	**>32**
Shrinkage Limit	**>13**	**<10**
Free Swell (%)	**<50**	**>100**

Clay Mineralogy

Minerals are the indicators of the amount of weathering that has taken place and the presence or absence of certain minerals explains the mechanical and chemical weathering processes that result in the formations of soils (Schulze, 2002). The term clay is used as both a particle size and a type of mineral. When representing particle size, it indicates the soil particles that have their size less than 79 micro-in. (0.002 mm). As a mineral type, it represents the minerals which have a) small particle size, b) a net electrical negative charge and c) plasticity when mixed with water. Most of the clay minerals are primarily hydrous aluminum silicates. Their shape is usually platy or in few cases needle shaped or tubular (Mitchell, 1993).

Figure 2.1 presents a systematic manner in which synthesis of different clay minerals takes place. Kaolinite is a common phyllosilicate mineral in subgrades; it is most abundant in soils of warm moist climates. Halloysites are particularly interesting mineral of kaolin sub group. This exists in two different forms; one as hydrated form which consists of unit kaolinite layers separated from each other by a single layer of water molecules and having the composition $(OH)_8 Si_4Al_4O_{10} \cdot 4H_2O$, and the other, a non-hydrated form having the same unit layer structure and chemical composition as kaolinite (Mitchell, 1993). Illite is essentially a group name for non-expanding, clay-sized minerals. Smectites commonly result from the weathering of basic rocks. These minerals have a very small size and are concentrated in the fine clay fraction of soils.

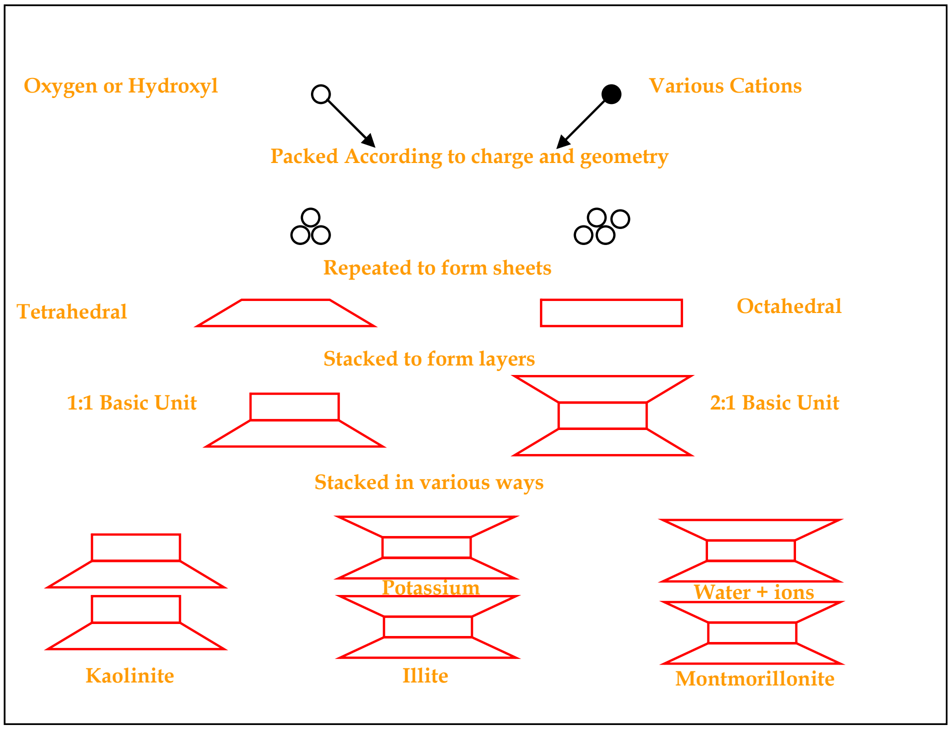


Figure 2.1 – Mineral Formations (Mitchell, 1993)

Clay Mineral Identification Methods

Soil minerals are identified primarily by their elemental composition and structure. Other distinctive properties such as color, thermal behavior, and solubility are also used. X-ray diffraction alone often provides enough information, but in many instances, two or more analytical techniques are needed to confirm mineral identity (Amonette, 2002).

Direct Methods

Two direct procedures are available for the identification of the minerals in the soil: 1) Elemental composition and 2) Structure. Elemental composition do not directly lead to the identification or quantification of mineral phases present but such knowledge will help in eliminating some of the phases from consideration. X-ray diffraction is the most widely used method for identification of fine grained soil minerals and the study of their crystal structure. The X-rays produced by the specimens are detected with the help of energy proportional detections.

Indirect Methods

Various indirect methods involving the use of chemical species measurements and physical characteristic measurements can be used to identify the dominating clay minerals in the soils and even approximate quantification of dominating clay minerals.

The cation exchange capacity (CEC) of a soil is simply a measure of the quantity of readily exchangeable cations neutralizing negative charge in the soil. According to Camberato (2001), CEC refers to the quantity of negative charges in soil existing on the surfaces of clay and organic matter. The positively charged ions or cations are attracted by negative charges, hence the name 'cation exchange capacity'. Numerous methods can be used for CEC determination. There is a significant variation in the results obtained by different methods, as there are many complicating interactions between saturating, washing, and extracting solutions. Also, CEC is not an independent and a single valued soil property (Rhoades, 1982). Camberato (2001) says the primary factor determining CEC is the clay and organic matter content of the soil. Higher quantities of clay and organic matter results in higher CEC.

The specific surface area of a soil sample is the total surface area contained in a unit mass of soil. Soils with high specific surface areas have high water holding capacities, more adsorption of contaminants, and greater swell potentials. Specific surface is therefore an important parameter. Specific surface is closely tied to particle size distribution. The smaller the particles are, the greater the surface area per unit mass of soil will be.

Potassium is an element which can be used to detect the presence of the mineral mica. Potassium belongs to the alkali metals in the periodic table that are characterized by a single electron in the outer most shell. This electron is easily lost and they readily form stable monovalent ions (Knudsen et al. 1982). There are many methods available for the determination of potassium in soils but the one proposed by Knudsen et al. (1982) is widely used.

SOIL STABILIZATION

Pavements are usually designed based on the assumption that specified levels of quality will be achieved for each soil layer in the pavement system. Each layer must resist shearing within the layer, avoid excessive elastic deformations that would result in fatigue cracking within the layer or in overlying layers, and prevent excessive permanent deformation through densification. When the quality of a soil layer is increased its ability to distribute the load over a greater area is generally increased enough to permit a reduction in the required thickness of the soil and surface layers. Generally, the soil quality improvements through stabilization include better soil gradation, reduction of plasticity index or swelling potential, and increases in durability and in strength. The tensile strength and stiffness of a soil layer can be improved through the use of additives and thereby permit a reduction in the thickness of the stabilized layer and overlying layers within the pavement system.

Soil Stabilization Methods

The two frequently used methods of stabilizing soils are stabilization by compaction or stabilization by chemical additives.

In stabilization by compaction method, soil density is increased by the application of short-term external mechanical forces, including compaction of surface layers by static, vibratory, or impact rollers and plate vibrators; and deep compaction by heavy tamping at the surface or vibration at depth. It is common experience that the stability of the soil is increased by increasing its state of compaction. The fact that a loose material may be made more stable simply by compacting is so obvious that it not really considered being a stabilization process. Nevertheless, as compaction plays a fundamental role in properties of stabilized material (Sherwood, 1995).

Mechanical stabilization is accomplished by mixing or blending soils of two or more gradations to obtain a material meeting the required specification. It is the process in which the grading of the soil is improved by the incorporation of another material which affects only the physical properties of the soil. The effectiveness of stabilization depends upon the ability to obtain uniformity in blending the various materials.

Mixing soils with stabilizing agents like lime and cement, usually in low amounts, changes both the physical and the chemical properties of the stabilized soil. This method is also referred as ‘additive method’ and ‘chemical stabilization’. Additive refers to a manufactured commercial product that is added to the soil in proper quantities to improve the quality of the soil layer. The common stabilizers used are lime and cement, and sometimes used in combination. There are many other stabilizers in use currently which can be listed as fly-ash, pozzolans, blast furnace slag and several others. The selection and determination of the percentage of additives depend upon the soil classification and the degree of improvement in soil quality desired. In general, smaller amounts of additives are required to alter soil properties, such as gradation, workability, and plasticity, than to improve the strength and durability sufficiently to permit a thickness reduction design. The method of chemical stabilization is discussed in detailed in the following sections.

Chemical Stabilization

A basic understanding of how each additive works as well as the impact of soil properties on the selection of type and concentration of these additives should be considered. The degree and speed of the mechanism depends on the composition of the additive and the material being treated. Some additives work independently, while others require water or water plus silica and alumina present in clays, to perform. The mineralogy, quantity, and particle size of fines in the soil or base can greatly impact the performance of individual additives. The goal of the soil treatment and the additive mechanism, composition, and reaction time must all be considered when selecting the best additive for a specific application. Table 2.2 compares the process, effects and applicable soil type of previously mentioned stabilizing agents. A brief description of the three most common additives used in stabilization is presented next.

Lime

Lime can be used to treat soils to varying degrees, depending upon the objective of the stabilization for a specific project. The least amount of treatment is used to dry and temporarily modify soils (Sherwood, 1995). Such treatment produces a working platform for construction or temporary roads. The highest amount can be used when it is being used to improve the soil strength properties for supporting roads (Sherwood, 1995).

1. Lime stabilization is a widely used means of chemically transforming unstable soils into structurally sound construction foundations. Lime stabilization enhances engineering properties in soils, including improved strength; improved resistance to fracture, fatigue, and permanent deformation; improved resilient properties, reduced swelling, and resistance to the damaging effects of moisture. The most substantial improvements in these properties are seen in moderately to highly plastic clays (Little, 2000).

According to Sherwood (1995) and Little (1999), lime stabilization can be used to either modify or stabilize clays. Modification, which provides substantial improvement to the performance of high plasticity clays, occurs primarily due to exchange of calcium cations supplied by the lime [$Ca(OH)_2$ or hydrated lime] for the normally present cations adsorbed on the surface of the clay mineral. Modification is also caused as the hydrated lime reacts with the clay mineral surface in the high pH environment. The results of the mechanisms are: plasticity reduction; reduction in moisture holding capacity (drying), swell reduction and stability improvement.

Stabilization differs from modification in that a significant level of long-term strength gain is developed through a long-term pozzolanic reaction. This pozzolanic reaction is the formation of calcium silicate hydrates and calcium aluminates hydrates as the calcium from the lime reacts with the aluminates and silicates solubilized from the clay mineral surface. Lime stabilization often induces a ten-fold stiffness increase over that of the untreated soil or aggregate.

Cement

Cement has been found to be effective in stabilizing a wide variety of soils, including granular materials, silts, and clays; byproducts such as slag and fly ash; and waste materials such as pulverized bituminous pavements and crushed concrete. It is generally more effective and economical to use it with granular soils due to the ease of pulverization and mixing and the

Table 2.2 – Comparison of Different Stabilizing Agents (from Hicks, 2002)

Stabilization Agent	Process	Effects	Applicable Soil Types
Cement	Cementitious inter-particle bonds are developed.	Low additive content (<2%): decreases susceptibility to moisture changes, resulting in modified or bound materials. High additive content: increases modulus and tensile strength significantly, resulting in bound materials.	Not limited apart from deleterious components (organics, sulphates, etc., which retard cement reactions). Suitable for granular soils but inefficient in predominantly one-sized materials and heavy clays.
Lime	Cementitious inter-particle bonds are developed but rate of development is slow compared to cement. Reactions are temperature dependent and require natural pozzolan to be present. If natural pozzolan is not present, a blended binder that includes pozzolan can be used.	Improves handling properties of cohesive materials. Low additive content (<2%): decreases susceptibility to moisture changes, and improves strength, resulting in modified or bound materials. High additive content: increases modulus and tensile strength, resulting in bound materials.	Suitable for cohesive soils. Requires clay components in the soil that will react with lime (i.e., contain natural pozzolan). Organic materials will retard reactions.
Blended slow-setting binders (for example: fly ash/lime)	Lime and pozzolan modifies particle size distribution and develops cementitious bonds.	Generally similar to cement but rate of gain of strength similar to lime. Also improves workability. Generally reduces shrinkage cracking problems.	Same as for cement stabilization. Can be used where soils are not reactive to lime.

smaller quantities of cement required. Fine-grained soils of low to medium plasticity can also be stabilized, but not as effectively as coarse-grained soils. If the PI exceeds about 30, cement becomes difficult to mix with the soil. In these cases, lime can be added first to reduce the PI and improve workability before adding the cement (Hicks, 2002). Cement stabilization develops from the cementitious links between the calcium silicate and aluminate hydration products and the soil particles (Croft, 1967). Addition of cement to clay soil reduces the liquid limit, plasticity index and swelling potential and increases the shrinkage limit and shear strength (Nelson and Miller, 1992).

Fly Ash

Fly ash is defined in Cement and Concrete Terminology (ACI Committee 116) as "the finely divided residue resulting from the combustion of ground or powdered coal, which is transported from the firebox through the boiler by flue gases." Fly ash is a by-product of coal-fired electric generating plants. Two main types of fly ash are being used: non self-cementing Class F and lime-fly ash self-cementing Class C. Stabilization of soils and pavement bases with coal fly ash is an increasingly popular option for design engineers. Fly ash decreases swell potential of expansive soils (Ferguson 1993, White et al. 2005). Soils can be treated with self-cementing fly ash to modify engineering properties as well as produce rapid strength gain in unstable soils. Ferguson (1993) noted that the decrease in plasticity and swell potential was generally less than that of lime because fly ash did not provide as many calcium ions that modify the surface charge of clay particles. Fly ash increases the CBR of fine-grained soils. Fly ash can also dry wet soils effectively and provide an initial rapid strength gain, which is useful during construction in wet, unstable ground conditions (White et al., 2005). Strength gain in soil-fly ash mixtures is dependent on cure time and temperature, compaction energy, and compaction delay (White et al., 2005). Sulfur contents can cause formation of expansive minerals in soil-fly ash mixtures, which severely reduces the long-term strength and durability. These negative reactions resulting from sulfur were reported by many researchers and practitioners (Puppala et al., 2004).

REVIEW OF CURRENT TXDOT STABILIZATION SPECIFICATIONS

TxDOT guidelines divide the overall process into three steps: (1) Soil Exploration, Material Sampling, Soil Classification and Acceptance Testing, (2) Additive Selection, and (3) Mix Design. The flowchart shown in Figure 2.2 provides a simplified illustration of the steps required for successful subgrade treatment.

Soil Exploration and Material Sampling

Soil exploration is vital, as it provides material for testing and also reveals conditions in underlying strata that can affect the performance of the pavement structure and treated layers, such as soil mineralogy, water table proximity, and soil strata variation. Material sampling and testing is critical and is required to characterize material and physical properties that can affect the performance of the pavement structure. It is important to obtain bulk samples large enough to perform multiple mix designs and soil classifications.

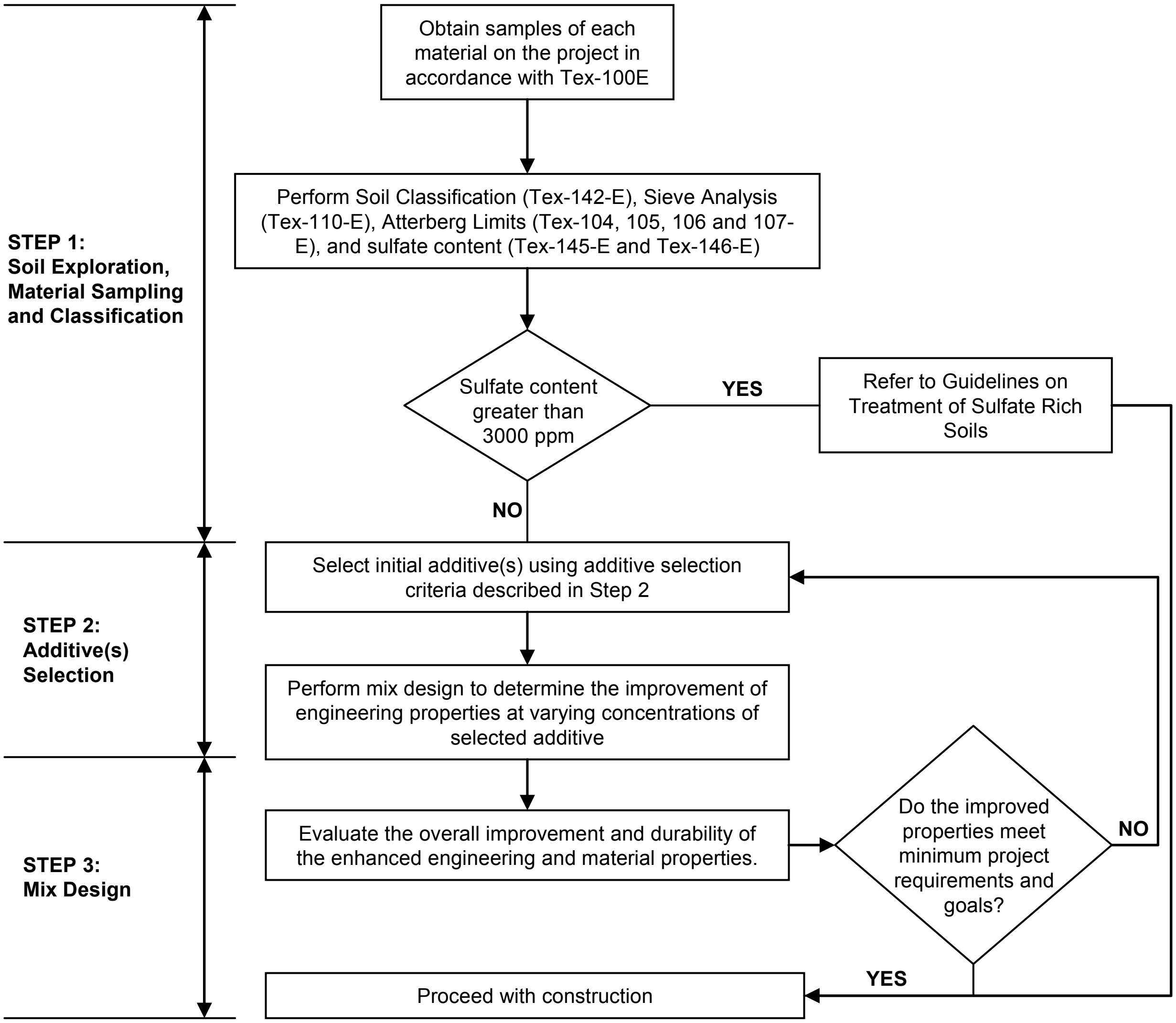


Figure 2.2 – Flowchart for Subgrade Soil Treatment (from TxDOT Guidelines)

Additive Selection Criteria

TxDOT stabilization guidelines provide a quick reference to the selection of additives. The selection of the proper additives is summarized in Figure 2.3 for subgrades. The Plasticity Index (PI) and gradation are the two most important criteria. Further validation testing must be performed to verify whether the selected additive accomplishes the goals and requirements of the treated soil.

When the sulfate content present in the soil is greater than 3000 ppm, especial studies need to be executed for subgrade stabilization as per TxDOT Guidelines on Treatment of Sulfate Rich Soils. A brief summary of this guideline is presented later.

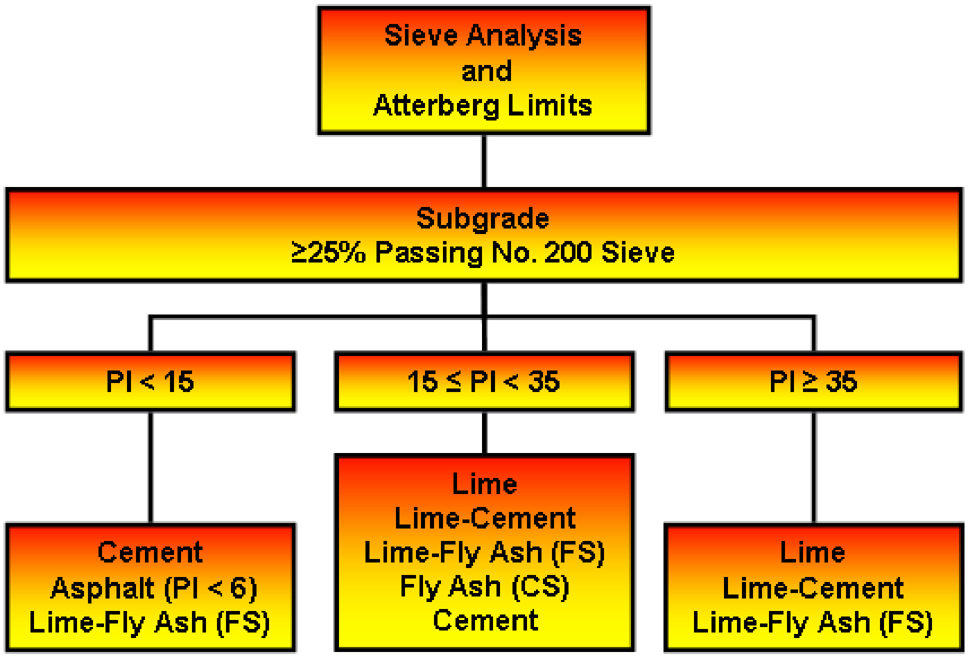


Figure 2.3 – Additive Selection Criteria for Subgrade Material Using Soil Classification

Mix Design

Mix design is essential to optimize the material properties, calculate the right percent of additive, measure effectiveness and engineering properties and provide density and moisture control parameters for construction. TxDOT guidelines denote a few steps to achieve the mix design:

- Verifying that sulfate and organic contents are within acceptable limits
- Developing moisture density curve (M/D) for field density control
- Strength testing before and after moisture conditioning, and
- Determining the lowest modifier content to satisfy strength requirements.

Determination of Additive Concentration

The procedures to select the adequate percentage necessary to stabilize base and subgrades are separated by type of additive, particularly Tex-120-E for cement, Tex-121-E for lime and Tex-127-E for fly-ash stabilization.

Cement

TxDOT guidelines to determine the amount of cement required for soil-cement stabilization are primarily based on exceeding a minimum unconfined compressive strength and attaining a minimum strength after moisture conditioning in the laboratory. Minimum strength requirements for plant-mixed stabilized mixes are based on the class specified on the plans as summarized in Table 2.3 (TxDOT Item 246). As determined by the latest TxDOT Pavement Design Guide (2006), 300 psi should be the target strength for cement stabilized bases. Higher strengths are not recommended because they can lead to early age cracking due to volume change due to shrinkage and a number of other factors.

Table 2.3 – Soil-Cement Strength Requirements

Class		***7-Day Unconfined Compressive Strength, Min. psi***
L	*Flexible pavements*	*300*
M		*175*
N	*Rigid pavements*	*As shown on the plans*

Lime
To obtain the amount of lime necessary to stabilize the soil, TxDOT specifications are based on the pH method. This method, also known as the "Eades-Grim" test (Eades and Grim, 1966) and summarized in Tex-121-E Part III. The basic objective of this method is to add sufficient lime to the soil to ensure a pH of 12.4 for sustaining the strength-producing lime-soil pozzolanic reaction. The lowest percentage of lime in soil that produces a laboratory pH of 12.4 is the minimum percentage for stabilizing the soil. A series of specimens with lime percentages ranging from 0 to 10% are tested in the lab to determine the required amount. Additional provisions for cases in which the measured laboratory pH is 12.3 or less are established. The minimum strength criterion for lime content is based on an unconfined compressive strength of 150 psi for bases and 50 psi for soils.

Fly Ash (FA) and Lime-Fly Ash (LFA)
Like cement, the unconfined compressive strength is used as an index to determine the suitable amount of additive. A minimum unconfined compressive strength of 150 psi is suggested as adequate for FA or LFA stabilized soils. Unconfined compressive strengths for FA or LFA base courses should approach the strength requirements of soil cement presented in Table 2.3.

Curing and Moisture Conditioning Requirements

TxDOT specifies the amount of time needed prior to performing unconfined compressive strength tests on stabilized materials. These minimum time requirements are affected by the time required to allow wetted specimens to stand before compaction, the time needed to cure the specimens and also the time needed for moisture conditioning. The minimum required time before completing the strength tests with soil-cement specimens is 8 days, and with lime-soil, fly-ash and lime-fly-ash specimens is 18 days.

Special Provisions for Treatment of Sulfate-Rich Soils and Bases

TxDOT has developed a series of guidelines to asses the risk for potential sulfate heave, to treat this type of soil, and to minimize the heave based on construction techniques and quality management practices. Sulfate-induced heave can occur in any type of soil regardless of the texture or plasticity, but typically, there is a higher probability for sulfates to occur in soils with high plasticity. Sulfates can also exist in granular soils, especially in arid regions or in strata that experience little water migration. TxDOT guidelines for soil stabilization address all types of soils, depending on the PI and the sulfate content of the soil. Two main cases are for moderate to highly expansive soils and one for minimally expansive soils. The flowchart depicted in Figure 2.3 summarizes the necessary treatment based on sulfate levels.

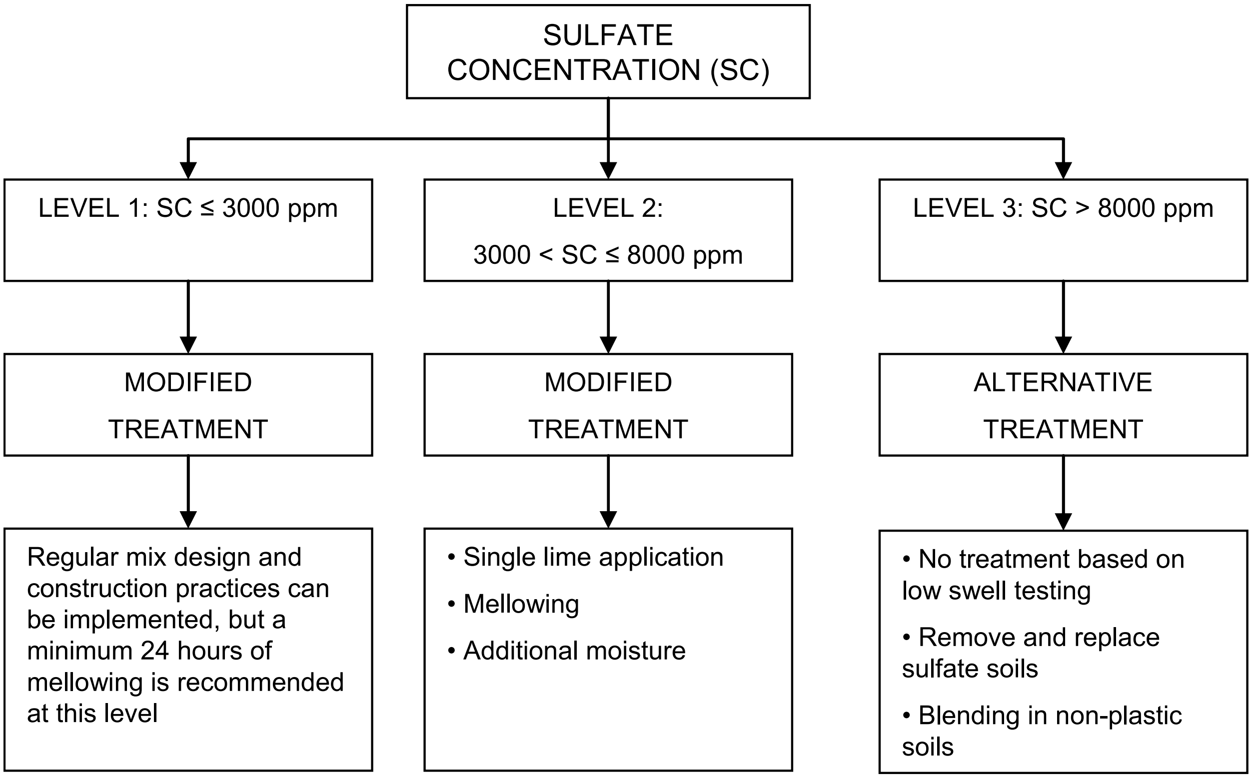


Figure 2.3 – Decision Tree to Determine Type of Treatment for Varying Sulfate Contents on Moderately to Highly Expansive Soils

REVIEW OF OTHER SPECIFICATIONS

A review of the literature was carried out in order to compare current TxDOT state of practice and stabilization guidelines with other organizations, such as the US Army and Air Force, Portland Cement Association, National Lime Association, ASTM Standards, and other relevant researches available, either nationally or worldwide. The most significant studies found are summarized in the next sections.

Army/Air Force Soil Stabilization Guidelines

The Army/Air Force manual for soil stabilization (1994) establishes criteria for improving the engineering properties of soils used for pavement base, subbase, and subgrades by the use of additives. The manual also prescribes the appropriate type or types of additive to be used with different soil types, procedures for determining a design treatment level for each type of additive, and recommended construction practices for incorporating the additive into the soil.

In the selection of a stabilizer, the factors that must be considered are the type of soil to be stabilized, the purpose for which the stabilized layer will be used, the type of soil improvement desired, required strength and durability, and cost and environmental conditions. The selection of candidate stabilizers is made using Figure 2.4 and Table 2.4. The soil gradation triangle in Figure 2.4 is based upon the soil grain size and Atterberg limits characteristics, and the triangle is

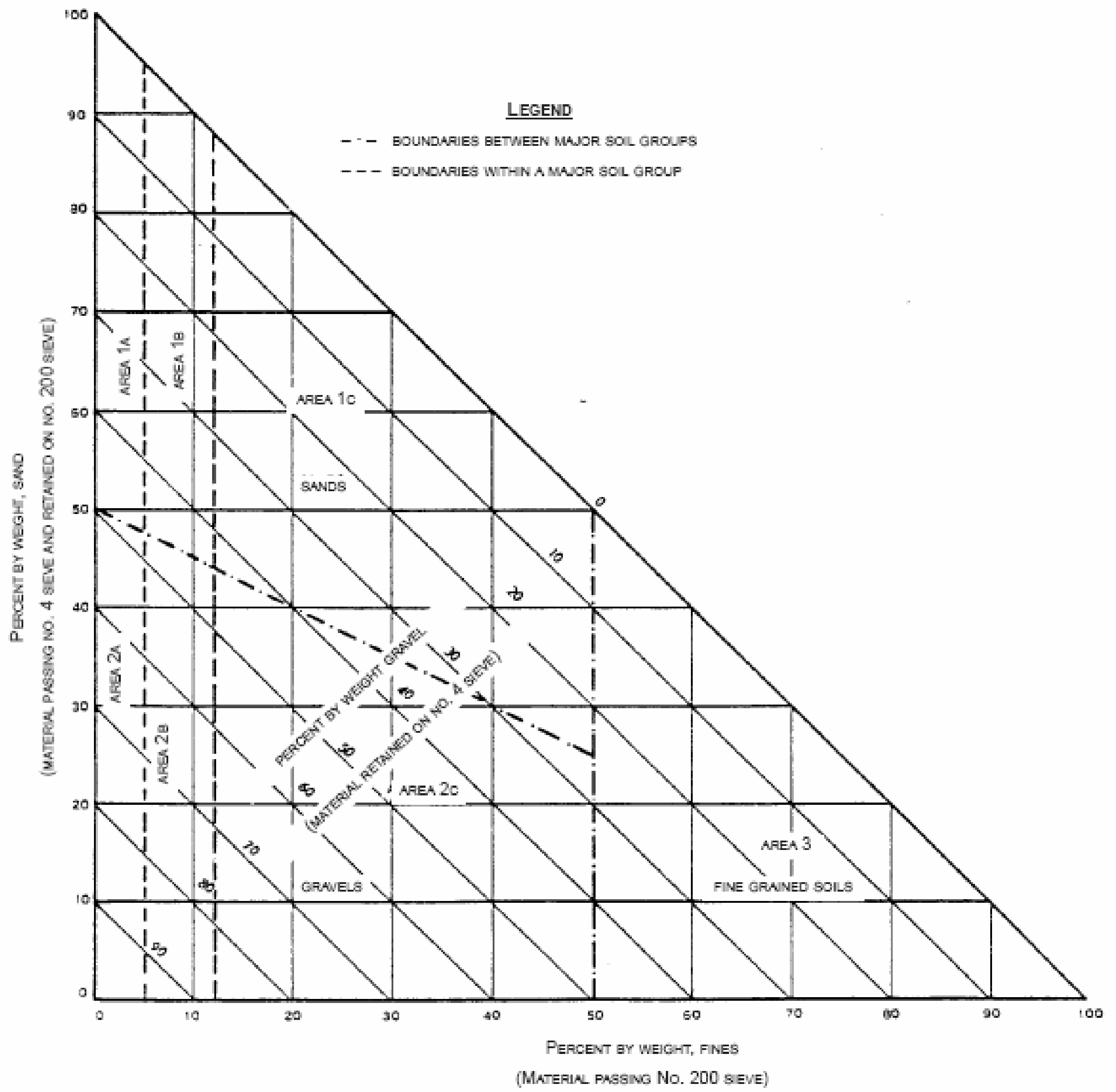


Figure 2.4 – Gradation Triangle to Select a Commercial Stabilizing Agent

Table 2.4 – Guide for Selecting a Stabilizing Agent

Area	*Soil Classification*	*Type of Stabilizing Additive Recommended*	*Restriction on LL and PI of Soil*	*Restriction of % Passing No. 200 Sieve*	*Remarks*
1A	*SW or SP*	*(1) Bituminous* *(2) Portland -Cement*			
		(3) Lime-Cement-Fly Ash	*PI not to exceed 25*		
1B	*SW-SM or SP-SM or SW-SC or SP-SC*	*(1) Bituminous*	*PI not to exceed 10*		
		(2) Portland -Cement	*PI not to exceed 30*		
		(3) Lime	*PI not to exceed 12*		
		(4) Lime-Cement-Fly Ash	*PI not to exceed 25*		
1C	*SM or SC or SM-SC*	*(1) Bituminous*	*PI not to exceed 10*	*Not to exceed 30% by weight*	
		(2) Portland -Cement	***		
		(3) Lime	*PI not less than 12*		
		(4) Lime-Cement-Fly Ash	*PI not to exceed 25*		
2A	*GW or GP*	*(1) Bituminous*			*Well-graded material only*
		(2) Portland -Cement			*Material should contain at least 45% by weight of material passing No.4 sieve*
		(3) Lime-Cement-Fly Ash	*PI not to exceed 25*		
2B	*GW-GM or GP-GM or GW-GC or GP-GC*	*(1) Bituminous*	*PI not to exceed 10*		*Well-graded material only*
		(2) Portland -Cement	*PI not to exceed 30*		*Material should contain at least 45% by weight of material passing No.4 sieve*
		(3) Lime	*PI not less than 12*		
		(4) Lime-Cement-Fly Ash	*PI not to exceed 25*		
2C	*GM or GC or GM-GC*	*(1) Bituminous*	*PI not to exceed 10*	*Not to exceed 30% by weight*	*Well-graded material only*
		(2) Portland -Cement	***		*Material should contain at least 45% by weight of material passing No.4 sieve*
		(3) Lime	*PI not less than 12*		
		(4) Lime-Cement-Fly Ash	*PI not to exceed 25*		
3	*GH or CL or MH or ML or OH or OL or ML-CL*	*(1) Portland*	*LL less than 40 and PI less than 20*		*Organic and strongly acid soils falling within this area are not susceptible to stabilization by ordinary means*
		(2) Lime	*PI not less than 12*		

* $PI \leq 20 + [(50 - \text{percent passing No. 200 sieve}) / 4]$

divided into areas of soils with similar grain size. The selection process continues with Table 2.4 which indicates for each area shown in Figure 2.3 candidate stabilizers and restrictions based on grain size and/or PI.

Gradation requirements are presented for each type of stabilizer in Table 2.5. Unconfined compressive strength and durability requirements for bases and subbase treated with cement, lime, lime-fly-ash, and lime-cement-fly-ash are indicated in Tables 2.6 and 2.7. Strength tests are determined at 7 days for cement stabilization and 28 days for lime, lime-fly-ash, or lime-cement-fly-ash stabilization and results are also separated for flexible and rigid pavements. All stabilized materials must meet minimum durability criteria to be used in pavement structures. The Army stabilization guideline establishes a maximum allowable weight loss after 12 wet-dry or freeze-thaw cycles as durability requirement.

Table 2.5 – Gradation Requirements for Stabilized Base and Subbase Courses

Type Course	***Cement or Lime***		***Fly-Ash***	
	Sieve #	***% Passing***	***Sieve #***	***% Passing***
Base	*1 ½ in.*	*100*	*2 in.*	*100*
	3/4 in.	*70-100*	*3/4 in.*	*70-100*
	No. 4	*45-70*	*3/8 in.*	*50-80*
	No. 40	*10-40*	*No. 4*	*35-70*
	No. 200	*0-20*	*No. 8*	*25-55*
	---	---	*No. 16*	*10-45*
	---	---	*No. 200*	*0-15*
Subbase	*1 ½ in.*	*100*	*1 ½ in.*	*100*
	No. 4	*45-100*	*No. 4*	*45-100*
	No. 40	*10-50*	*No. 40*	*10-50*
	No. 200	*0-20*	*No. 200*	*0-15*

Table 2.6 – Minimum Unconfined Compressive Strength (psi) for Stabilized Soils

Stabilized Soil Layer	***Flexible Pavement***	***Rigid Pavement***
Base course	*750*	*500*
Subbase course, select material or subgrade	*250*	*200*

Table 2.7 – Durability Requirements

Type of Soil Stabilized	***Maximum Allowable Weight Loss After 12 Wet-Dry or Freeze-Thaw Cycles***
Granular, PI < 10	*11*
Granular, PI > 10	*8*
Silt	*8*
Clay	*6*

The amount of additive necessary to provide an effective stabilization is dependent on the type of stabilizer. The cement content is initially estimated based on the soil classification (see Table 2.8). Using this cement content, maximum dry density and optimum water content of the soil-

Table 2.8 – Cement Requirements for Various Soils

Soil Classification	*Initial Estimated Cement Content (% dry weight)*
GW, SW	*5*
GP, GW-GC, GW-GM, SW-SC, SW-SM	*6*
GC, GM, GP-GC, GP-GM, GM-GC, SC, SM, SP-SC, SP-SM, SM-SC, SP	*7*
CL. ML, MH	*9*
CH	*11*

cement mixture is calculated. Triplicate specimens are prepared at recommended cement contents and also at ±2% cement content. Unconfined compressive strength and durability tests are performed on these specimens and the lowest cement content which meets the strength requirement and demonstrates the required durability is the design cement content.

The preferred method for determining initial design lime content is the pH test or "Eades-Grim" test, same as used in current TxDOT specifications. An alternate method of determining the initial design lime content is by the PI wet method (AASHTO T-220), as shown in Figure 2.5. Other than determination of lime content, unconfined compressive strength and durability tests are also performed to assure strength and durability requirements previously discussed. If results of the specimens tested do not meet both the strength and durability requirements, higher lime content may be selected and the mix design is evaluated again.

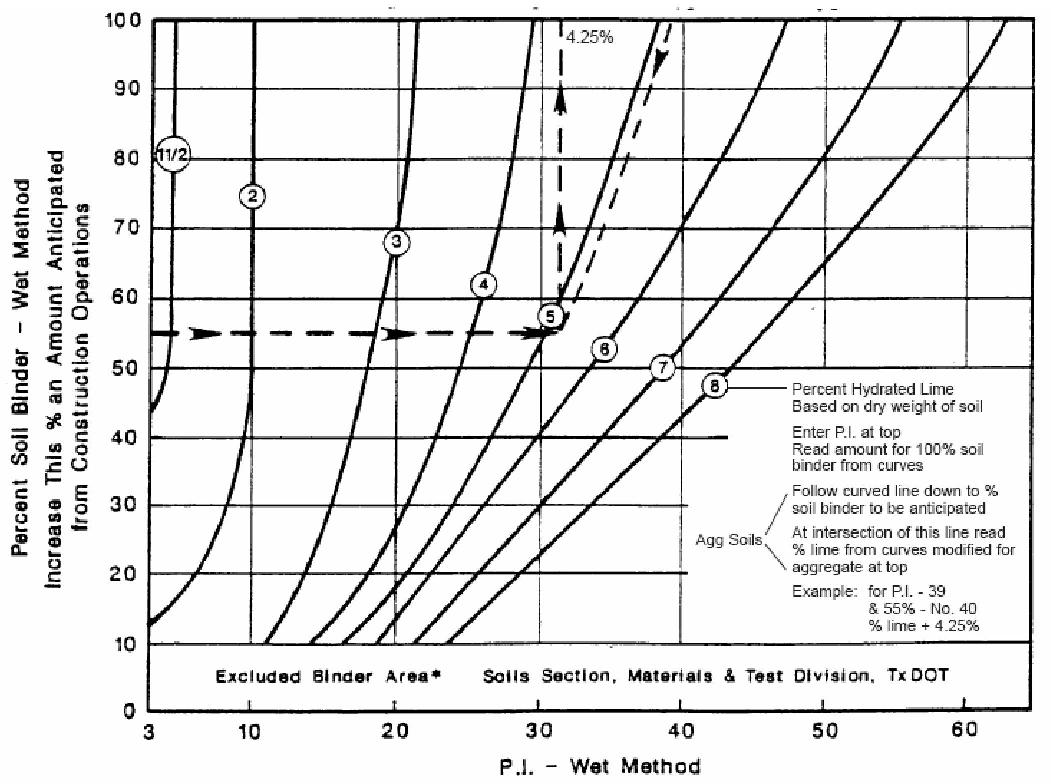


Figure 2.5 – PI Wet Method to Calculate Amount of Lime for Stabilization

Design with lime-fly ash (LF) is somewhat different from stabilization with lime or cement. For a given combination of materials (aggregate, fly ash, lime), a number of factors can be varied in the mix design process such as percentage of lime-fly ash, the moisture content, and the ratio of lime to fly ash. The matrix material, defined as the content of fly ash, lime, and minus No. 4 aggregate fines of the total mixture, is another aspect to consider in the mix design. To establish the amount of additives, the first step is to determine the optimum fines content that will give the maximum density. The initial fly ash content should be about 10% based on dry weight of the mix. Tests are run at increasing increments of fly ash, e.g. 2%, up to a total of about 20%. Moisture density tests are conducted afterward and the design fly ash content is then selected at 2% above that yielding maximum density. The ratio of lime to fly ash that will yield the highest strength and durability is determined, by using lime to fly ash ratios of 1:3, 1:4, and 1:5. Three specimens are prepared and tested for unconfined compression strength and wet-dry or free-thaw cycles (if applicable), and the lowest LF ratio content, i.e., ratio with the lowest lime content which meets the strength and durability requirements from Tables 2.6 and 2.7 is the design LF content.

When cement is added to the LF mix, the same design procedure is followed, but generally speaking, about 1 to 2% cement is used. Strength and durability tests must be conducted on samples at various LCF ratios to determine the combination that gives best results. Cement may also be used instead of lime; however, the total fines content should be maintained.

OTHER AGENCIES STABILIZATION GUIDELINES

A good summary from several highway agencies was found in TENSAR technical note (1998), including examination of engineering properties, discussions of design, construction and economics for lime, cement and fly-ash stabilization, where soft subgrades are encountered in construction. The TENSAR technical note is summarized next.

Selection of Additive

The gradation triangle illustrated in Figure 2.4 and selection criteria presented in Table 2.4 is also used as guide for selection of stabilizer agent.

Mix Design Procedures

Unconfined compressive strength (UCS) is the most widely referenced property of soil cement. Typical ranges of UCS after 7 and 28 days of curing for soaked soil-cement mixtures are presented in Table 2.9, classified by several soil groups (ACI 230.1R-90). The amount of laboratory testing to determine the amount of cement required for stabilization depends on many factors, including the constructing agency policy, the number of soil types encountered and the size of the project. The Portland Cement Association has summarized all these factors, as depicted in Figure 2.6.

The PCA soil-cement mix design procedure uses a complete series of detailed laboratory tests, summarized in Table 2.10, with typical cement requirements for different soil groups included in Table 2.11.

Table 2.9 – Ranges of UCS for Soil-Cement

Soil Type	*7-Day Soaked Compressive Strength, psi*	*28-Day Soaked Compressive Strength, psi*
Sandy and gravelly soils	*300-600*	*400-1000*
Silty soils	*250-500*	*300-900*
Clayey soils	*200-400*	*250-600*

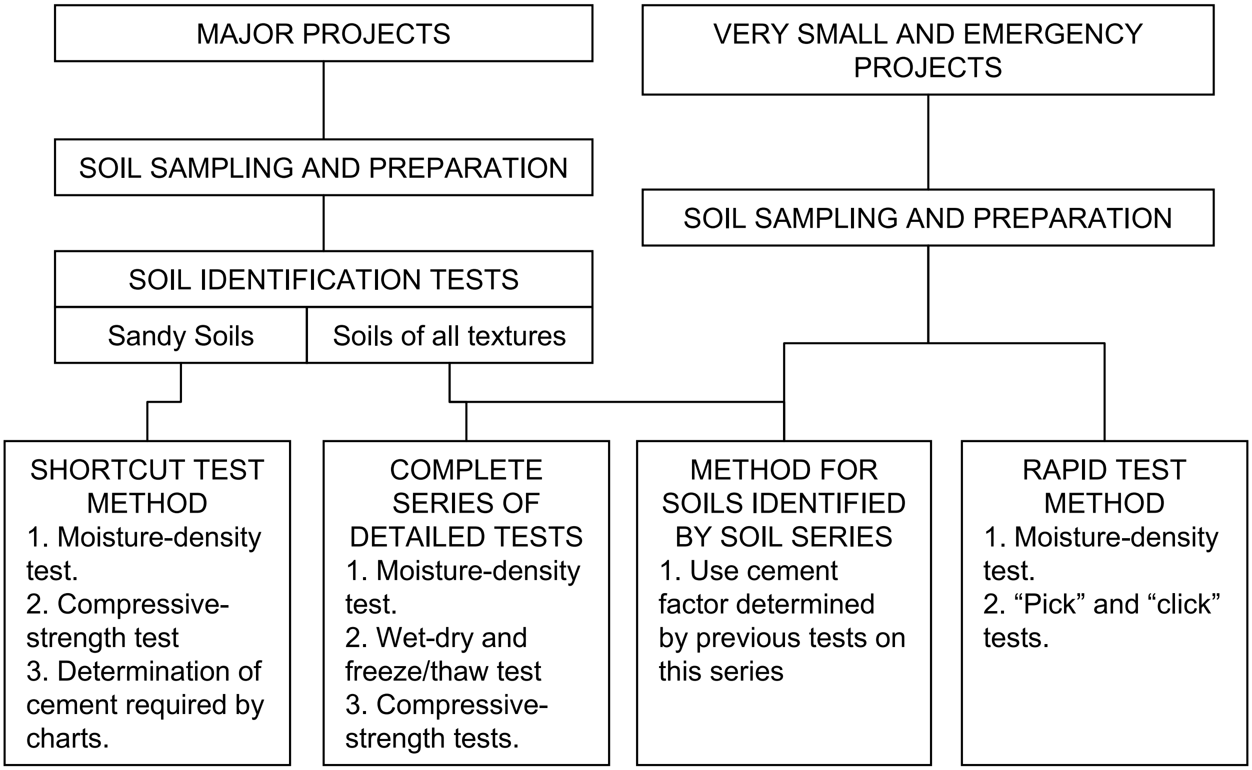


Figure 2.6 – Flowchart for Soil-Cement Laboratory Tests (after PCA, 1995)

Table 2.10 – Summary of PCA Mix Design Procedure

Procedural Step	*AASHTO Standard Method*	*ASTM Standard Method*
Chose initial cement content.	*Based on M 145-49. Estimated initial cement contents are listed in Table 2.12.*	*Based on UCS. Estimated initial cement contents are listed in Table 2.12.*
Perform moisture-density tests	*T-134*	*D 558*
Verify the initial cement content	*Table 2.12. Other references available for B and C horizons.*	*Table 2.12*
Mold and test wet-dry and freeze-thaw specimens	*Wet-dry: T-135* *Freeze-thaw: T-136*	*Wet-dry: D 559* *Freeze-thaw: D 560*

Table 2.11 – Typical Cement Requirements for Various Soil Groups (after ACI 230.1R-90)

AASHTO soil classification	*ASTM soil classification*	*Typical range of cement, % by weight*	*Typical cement content for moisture-density test, % by weight*	*Typical cement contents for durability tests, % by weight*
A-1-a	*GW, GP, GM, SW, SP, SM*	*3-5*	*5*	*3-5-7*
A-1-b	*GM, GP, SM, SP*	*5-8*	*6*	*4-6-8*
A-2	*GM, GC, SM, SC*	*5-9*	*7*	*5-7-9-*
A-3	*SP*	*7-11*	*9*	*7-9-11*
A-4	*CL, ML*	*7-12*	*10*	*8-10-12*
A-5	*ML, MH, CH*	*8-13*	*10*	*8-10-12*
A-6	*CL, CH*	*9-15*	*12*	*10-12-14*
A-7	*MH, CH*	*10-16*	*13*	*11-13-15*

The PCA soil-cement mix design criteria based on durability tests is presented in Table 2.12. Sufficient cement contents to prevent weight losses greater than the values presented in the table after 12 wet-dry or freeze-thaw cycles are considered to be adequate to provide a durable mix.

Recent studies have shown that many highway agencies typically base their design criteria on the PCA guidelines (Scullion et al., 2005). However, most focus on compressive strength alone. The rationale is that if sufficient strength is obtained, then durability as measured by abrasion resistance will not be a problem. Included in the same study is a brief summary of latest adopted requirements for compressive strengths on several DOTs, US Air Force and United Kingdom, as shown in Table 2.13. Some high strength requirements have been revised and reduced in some highway agencies after severe problems caused by shrinkage cracking have been found.

Design lime contents are usually based on the effect of lime percentages on engineering properties of the soil mixture. Different design lime contents may be selected depending on the objectives of the lime treatment. A brief summary of several lime design procedures and criteria is presented in Table 2.14.

The general mixture design procedure for lime-fly ash is to add the amount of fly ash that will fill the voids of the mixture and provide the maximum density. The next step is to add enough lime to maximize the pozzolanic reaction between the lime and the fly ash pozzolans. ASTM C 593 provides two mixture design criteria to select amount of lime-fly ash. The mix must reach a minimum UCS of 400 psi following vacuum saturation, or a maximum of 14% weight loss after 12 cycles of freeze-thaw. For more plastic clays, the approach may be to add sufficient lime initially to the soil to reduce plasticity and increase workability and then to add sufficient fly ash to the mix to support the pozzolanic strength gain

Table 2.12 – PCA Criteria for Soil-Cement as per Durability Tests

AASHTO soil classification	*USCS soil classification*	*Maximum allowable weight loss, %*
A-1-a	*GW, GP, GM, SW, SP, SM*	*14*
A-1-b	*GM, GP, SM, SP*	*14*
A-2	*GM, GC, SM, SC*	*14**
A-3	*SP*	*14*
A-4	*CL, ML*	*10*
A-5	*ML, MH, CH*	*10*
A-6	*CL, CH*	*7*
A-7	*OH, MH, CH*	*7*

**10 percent is the maximum allowable weight loss for A-2-6 and A-2-7 soils.*
Additional criteria:
1. Maximum volume changes during durability test should be less than 2% of the initial volume
2. Maximum water content during the test should be less than the quantity required to saturate the sample at the time of molding.
3. Compressive strength should increase with age of specimen.

Table 2.13 – Several Criteria for Soil-Cement Required Strengths

Organization	*7-Day Soaked Compressive Strength, psi*
California Division of Highway	*750*
Road Research Laboratory (UK)	*250-400*
US Air Force	*300*
Iowa DOT	*450*

Stabilization of coarse-grained soils having little or no fines can be accomplished using a LCF combination. Materials suitable for LCF stabilization are coarse-grained soils having no more than 12% of the material passing the No. 200 sieve. In addition, the PI of the minus 40 fraction should not exceed 25. ASTM C 593 may be used to determine the mix proportions of the LCF additive, adding about 1% of Portland cement for strength requirements, in addition to the lime and fly ash contents indicated in the procedure. Minimum UCS values are the same as in Table 2.6. If test specimens do not meet these requirements, cement is added in increments of 0.5% until strengths are adequate. The total quantity of the additives should not exceed 15% by weight. In addition, the LCF mixture should meet weight loss criteria for cement stabilized soils in frost areas, following ASTM D 560.

Table 2.14 – Summary of Soil-Lime Mixture Design Procedures

Mixture Design Procedure	*Summary of Methods*	*Mixture Design Criteria*
Eades and Grim (Figure 2.7)	*Based on pH*	*Design lime content is lime required to insure a pH of 12.4*
Thompson (Figure 2.8)	*Based on Unconfined Compressive Strength (UCS)*	*Increase of UCS of soil-lime over soil after 48 hour cure at 120°F must be at least 50 psi*
California	*California Test 373. Based on UCS and optimum moisture content*	*Highest UCS at optimum moisture content using 4-in. diameter by 4-in. height specimens.*
Illinois	*Based on UCS, optimum moisture content and maximum dry density*	*Achieve a 50 psi increase in UCS in 48 hours at 120°F. Design lime content is % above which there is no added strength gain*
Oklahoma	*Eades and Grim (ASTM D 6276 or ASTM C 977)*	*Design lime content is lime required to insure a pH of 12.4*
South Dakota	*South Dakota Test SD-107, similar to AASHTO T-193. Based on 96-hour soaked CBR and freeze-thaw cycles*	*CBR of soil-lime is 3-4 times of CBR natural soil. Maximum 0.5% vertical expansion after 30 freeze-thaw cycles. UCS after 30 freeze-thaw cycles is at least 75% of initial UCS*
Virginia	*Based on UCS*	*Design lime content based on cost effectiveness and benefit derived*

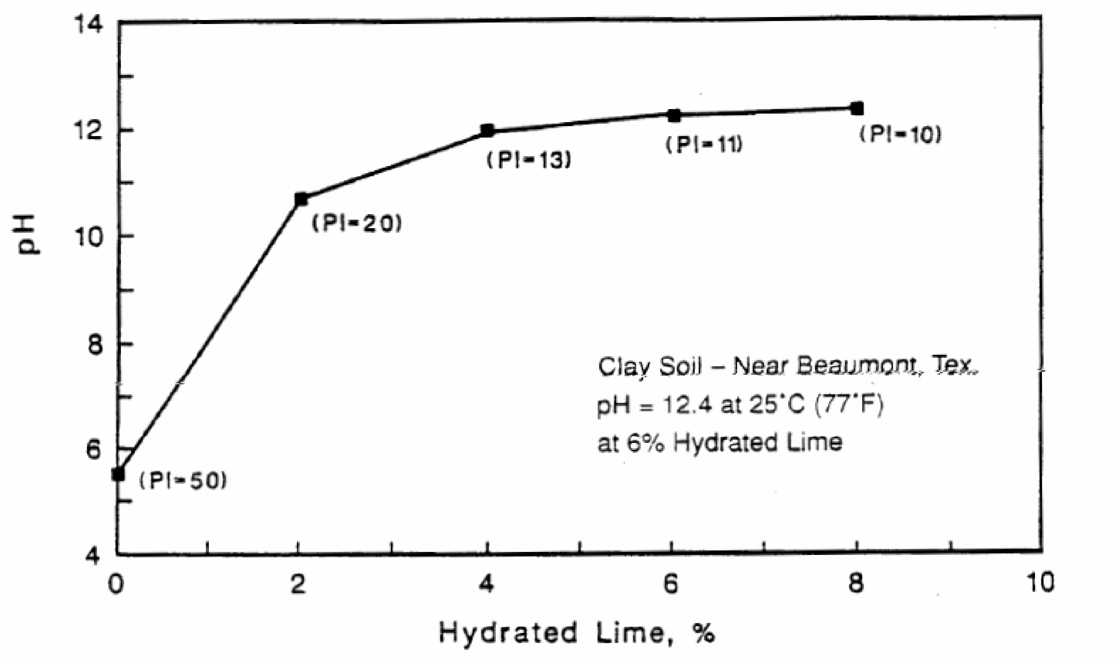
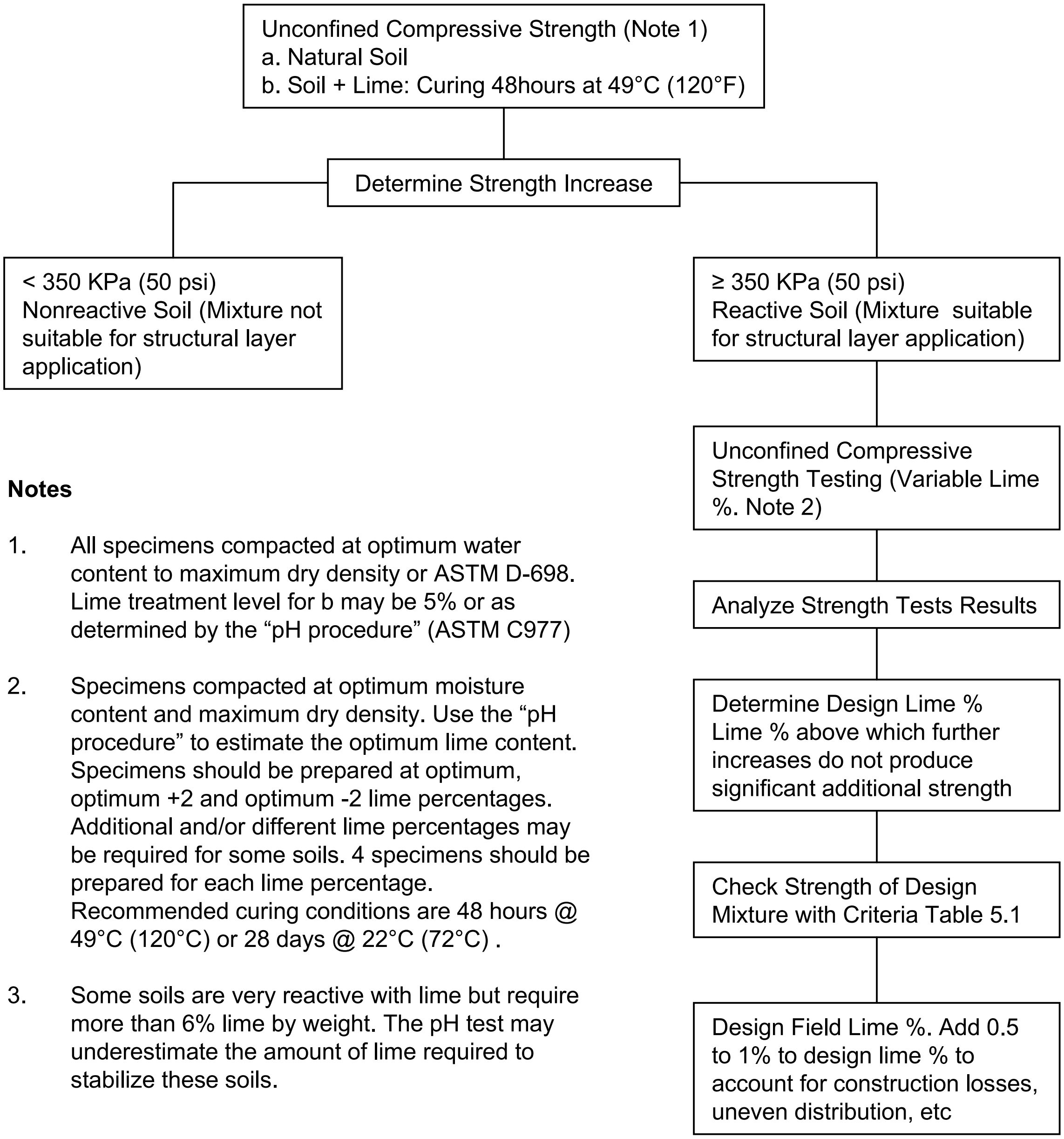


Figure 2.7 – Example of Eades and Grim Test for Different Lime Contents (Little, 1995)



Notes

1. All specimens compacted at optimum water content to maximum dry density or ASTM D-698. Lime treatment level for b may be 5% or as determined by the “pH procedure” (ASTM C977)
2. Specimens compacted at optimum moisture content and maximum dry density. Use the “pH procedure” to estimate the optimum lime content. Specimens should be prepared at optimum, optimum +2 and optimum -2 lime percentages. Additional and/or different lime percentages may be required for some soils. 4 specimens should be prepared for each lime percentage. Recommended curing conditions are 48 hours @ 49°C (120°C) or 28 days @ 22°C (72°C) .
3. Some soils are very reactive with lime but require more than 6% lime by weight. The pH test may underestimate the amount of lime required to stabilize these soils.

Figure 2.8 – Thompson Mixture Design Flow Chart (after Little et al., 1987)

NATIONAL LIME ASSOCIATION

The procedures outlined in latest recommendations from the National Lime Association (Technical Brief, 2006), are presented next. These procedures determine the minimum amount of lime required for long-term strength, durability and the other desired properties of the stabilized soil. The criteria to select lime as stabilizer agent are to have a soil with at least 25% passing a No. 200 sieve and a PI of 10 or greater. Some soils with lower PI can be successfully stabilized with lime, provided the pH and strength criteria can be satisfied. Soil with organics content above 1-2% by weight may be incapable of achieving the desired unconfined compressive strength for lime stabilized soil. Soils containing soluble sulfates greater than 0.3% can be successfully stabilized with lime, but may require special precautions.

To determine the minimum amount of lime required for stabilization, the Eades-Grim test or pH test (previously described) is followed. Immediately following the fabrication of specimens, they are wrapped and sealed in moisture proof bags. The specimens are cured for 7 days at 40°C and then subjected to capillarity action for 24 hours. For this matter, the specimens are wrapped with wet absorptive fabric and placed on a porous stone. ASTM D 5102 is the procedure followed to determine the UCS on cured and moisture conditioned specimens. The minimum desired UCS depends on the use of the soil, amount of cover material over the stabilized soil, exposure to soaking conditions, and the expected number of freeze-thaw cycles during the first winter of exposure. Suggested minimum UCS is shown in Table 2.15.

Table 2.15 – NLA Recommended Unconfined Compressive Strengths

Anticipated Use		*Extended Soaking for 8 Days (psi)*	*Cyclic Freeze-Thaw*[a]		
			3 Cycles (psi)	*7 Cycles (psi)*	*10 Cycles (psi)*
Subbase	*Rigid Pavements/ Floor Slabs/ Foundations*	*50*	*50*	*90*	*120*
	Flexible Pavements (>10 in.)[b]	*60*	*60*	*100*	*130*
	Flexible Pavements (8 - 10 in.)[b]	*70*	*70*	*100*	*140*
	Flexible Pavements (5 - 8 in.)[b]	*90*	*90*	*130*	*160*
Base		*130*	*130*	*170*	*200*

[a] *Number of freeze-thaw cycles during 1st winter of exposure*
[b] *Total pavement thickness overlying the subbase*

FEDERAL HIGHWAY ADMINISTRATION (FWHA) FLY ASH SPECIFICATIONS

The stabilization with fly ash uses several materials and material combinations to construct stabilized aggregate bases. Class C fly ash can be used as a stand-alone material or it can be blended with lime, portland cement or cement kiln dust (CKD). Typical proportions for the Class F fly ash lime blends are 2% to 8% lime blended with 10% to 15% Class F fly ash. Also, 0.5% to 1.5% Type I portland cement can be blended with Class F fly ash to produce the stabilizing agent.

The following steps summarize the process for the laboratory determination of mix proportions:

- Fly ash is added to the aggregate in five different proportions, starting at the lower limit (10% for coarse aggregate) and increment to the upper limit (20% for coarse aggregate)
- Mold test specimens at each fly ash content in accordance with ASTM C 593 at an estimated optimum moisture content (OMC)
- Identify maximum dry density (MDD)

- Select an OMC at least 2% above the matrix content found at the MDD and determine OMC and MDD for that blend
- Determine the most suitable proportions of activator to fly ash. Use five different activator-to-fly ash combinations at the optimum matrix content. Typical ranges of activator to fly ash ratio are 1:3 to 1:4 using lime or portland cement; with either lime kiln dust or cement kiln dust as an activator, the typical range is 1:1 to 1:2
- Prepare six proctor-size specimens for each combination (ASTM C 593). Cure all six test specimens for 7 days in sealed containers. For lime or kiln dust activators, cure at 37.8°C (100°F). For Portland cement as the activator, cure in a moist room at ambient conditions of 22.8°C (73°F) and 100% relative humidity
- Test 3 specimens for compressive strength and test the other 3 specimens for durability after the 7 day curing period (ASTM C 593) or after freeze-thaw cycles (ASTM D 560)
- Plot compressive strength vs. activator percentage for each combination. Only mixtures with a 7 day compressive strength exceeding 2,760 kPa (400 psi) and acceptable durability are considered
- The lowest percentage activator that exceeds the compressive strength and durability requirements is selected

METHODS FOR ACCELERATED CURING AND MOISTURE CONDITIONING

The goal of this section is to summarize potential methods that can be implemented to accelerate curing and moisture conditioning activities.

Moisture Conditioning

The effects of moisture on the performance of the base and subgrade layers in terms of strength and modulus should be estimated. Loss of strength and stiffness due to intrusion of moisture has significant negative impact on the performance of the pavement, if not addressed. The impacts of moisture on leaching the additives out of the host materials through moisture movements which result in variation of pH and Calcium and Magnesium ratios will have serious implications on the durability and sustainability of the chemical treatment. The effects of moisture fluctuations as a part of durability studies on chemical reaction of host materials and the stabilizer from seasonal changes and their impact on the performance of these soils is another factor that must be considered. A series of wet-dry tests on different types of chemically-treated soils are used in order to evaluate the efficacy of the chemical treatment in the term of strength and, stiffness and volume change property variations.

There are several concerns with this method of moisture-conditioning. First, the time to conduct these tests may be too long (about 7 to 8 days) after curing. Second, since this saturation method relies on the capillary action, the conditioning is more severe for materials with moderate amount of fines. Our work on Projects 0-4519 (Fernando et al, 2008) and 0-5430 (Sabnis et al., 2008) has clearly demonstrated to us that this method of conditioning is quite ineffective for materials with low-permeability such as clays. The review of several alternatives to the capillary suction is presented next.

Backpressure Saturation

In this method, the specimen is covered with a membrane, and placed in a triaxial chamber. The confining pressure and internal pressure through a water reservoir of the specimen are independently controlled. Both pressures are increased by the same amount first, and then the internal pressure is increased some so that the water can be pushed through the specimen. The amount of the confining pressure and the difference between the confining pressure and the internal pressure dictate how fast the water can penetrate through the specimen and its ultimate saturation. Another advantage of this method is that the degree of saturation of the specimen can be measured.

Khoury and Zaman (2004) studied the correlation of Resilient Modulus (Mr) with moisture variation, and soil suction for treated subgrade soils. The influence of moisture changes on the characteristic of soils was examined by applying several backpressure saturation approaches on treated subgrade soils. Three different time-efficient procedures for wetting sand soils and one for clay soils were presented in the study. Basically, in the proposed procedure specimens are compacted and a membrane is added to them to prevent moisture loss and placed in triaxial cells with a 2 to 4 psi confining pressure; water is injected from the bottom at pressure of 2 psi while applying a vacuum of 2 psi from the top of the specimen for about 1 hour; then the specimen is placed in a humidity room for 24 (sand soils) or 48 hours (clay soils) before testing. They also examined whether the proposed methods would induce a moisture gradient within the specimen, by measuring moisture content at 5 different heights throughout the specimen. Specimens were then dried by placing them in the oven at temperature of 40°C (105°F). The three methods used for wetting clay soils were significantly more time efficient than other existing methods with a total required time of about 48 hours. The moisture gradient in the specimens was reasonably uniform, and did not substantially influence the resilient modulus.

CO2 Injection

Another approach advocated and utilized by several research organizations with success is to replace the air in the specimen voids with an inert gas such as CO2 before moisture-conditioning. This process has been shown to significantly reduce the suction within the specimen, and accelerating the saturation of the specimens. For example, Murthy (1990) used this method to saturate naturally-cemented sands. In his experience, the time to saturation was reduced by a factor of up to 4, from about 2 weeks to less than 4 days under identical back-pressure saturation conditions.

The Army Manual (1986) describes a set up to incorporate CO2 injection, if complete saturation cannot be achieved with reasonable backpressure. CO2 can be allowed to slowly seep upward from the bottom of the specimen, while the specimen is being formed or after it has been confined in the triaxial chamber. The CO2 is able to displace the air in the specimen and, because it is much more soluble in water than air, will enable saturation of the material.

Reducing Size of Specimens

One way of accelerating the moisture conditioning is to reduce the size of the specimen. Some approaches have been oriented to reduce the height of the specimen without changing the diameter. This might be effective in terms of accelerating moisture conditioning; however, changing from the 2 to 1 length-to-diameter ratio is not recommended in terms of strength and stiffness determination. Current TxDOT specimen size for UCS (e.g., 4 in. by 6 in. for fine-grained materials) already deviates the length-to-diameter ratio (L/D).

Guthrie et al. (2001) performed resilient modulus tests on limestone samples as per AASHTO T 292-91. Cement stabilized samples of 6-in. and 8-in. gauge lengths were tested under unconfined stress conditions and axial deformations were measured across the samples height at different locations. Resilient modulus for the two types of specimens and at different cement contents with a deviator stress of 100 psi were compared. Results differed by factors of 6, 13, and 20 at cement levels of 1.5, 3.0, and 4.5%, respectively. This lack of correspondence is attributed to end effects, suggesting that the compaction and deformation characteristics of the ends vary considerably from the matrix within the midsection of a given sample. Work by Moore et al. (1970) and Pezo et al. (1998) support this matter.

White et al. (2005) attempted the reduction of size specimens to L/D ratios of 1.15 to 1 (4 x 4.5 inch sample) and 1 to 1 (2 x 2 inches) and its comparison with "true" UCS specimens with 2 to 1 L/D ratio (2.8 x 5.6 inches). Specimens with 2 to 1 and 1.15 to 1 were compacted following standard Proctor compaction method and the other with higher compaction effort. Increasing the compaction energy did increase the strength for some samples, but the increase was limited to the lower moisture contents evaluated, showing that at the highest water contents no measurable effect of the compaction energy was found. The specimens were cured at different conditions with several fly ash contents, wetted by soaking in a water bath, and subjected to UCS tests. Table 2.16 contains a comparison of strengths with "true" UCS samples. Even though curing and moisture conditioning methods were different, the 2 x 2 inch samples generally yielded higher strengths (16% and 12% more) than the other samples, and soaking the samples produced lower strength than that of testing without soaking.

Table 2.16 – UCS Comparison for Different Specimen Sizes (White et al., 2005)

Specimen Sizes (in.) and Ratios (L/D)	***Curing***	***Moisture Conditioning***	***UCS Comparison to "true" specimens (L/D ratio of 2/1)***
2.8x5.6 (2/1)	*7 days at 70°F with 100% humidity*	*Soaked for 4 hours*	*1.00*
4x4.58 (1.15/1)	*7 days at 100°F*	*Soaked for 4 hours*	*0.87*
2x2 (1/1)	*7 days at 100°F*	*Soaked for 1 hour*	*1.12*

Vacuum Suction

This method is very similar to the capillary suction method currently utilized by TxDOT. In this method, the specimen is placed inside a membrane or an air-tight container and stored on a shallow pan filled with water and a vacuum is applied to the top of the specimen. The higher the vacuum level is, the faster the specimen will be saturated. This method can be considered a simplified version of the back-pressure saturation method, without applying confining pressure. However, the degree saturation may not be measured with this method and the level of vacuum pressure has to be optimized to accelerate the moisture conditioning without causing internal damage to the specimen.

McManis (2003) used a similar approach to evaluate the long term durability of several silt soil specimens stabilized with lime, cement and lime-fly ash. Several specimens were cured in the humidity room under ambient conditions for 28 days. A set of specimens was placed in a vacuum saturation chamber, subjected to 30 minutes of de-airing, followed by immersion in water for one hour. The UCS results of these specimens are compared to the results of specimens not subjected to vacuum saturation in Table 2.17. The degree of saturation on all specimens after applying the vacuum saturation is greater than 94%. Moreover, unconfined compressive strengths exceeded or equaled the value for the unsaturated, natural soil and the strength loss after inundation varied from 50 to 25% depending on the type of additive, showing that the vacuum saturation was appropriate to moisture-condition the specimens.

Table 2.17 – UCS Comparison of Cured Specimens with and without Vacuum Saturation

Soil Mixture	***28-Day Cure***		***Vacuum Saturation***	
	UCS Strength (psi)	***Moisture (%)***	***UCS Strength (psi)***	***Saturation (%)***
Soil	*24.50*	*12.2*	*--*	*--*
Soil + 3.2% Cement	*44.52*	*11.8*	*23.50*	*95.0*
Soil + 10% Lime	*33.12*	*12.4*	*24.75*	*94.5*
Soil + 4% Lime + 7% FA	*50.00*	*11.9*	*32.80*	*94.0*

Submersion

Several organizations have advocated the submerging of specimens as a way to accelerate moisture conditioning. Harris et al. (2005) used submersion as a moisture conditioning method on stabilized materials. The effectiveness of the stabilizers was determined by the measurement of three-dimensional (3-D) swell reduction and unconfined compressive strength. For that instance laboratory specimens with sulfate concentrations of 0, 10,000, and 20,000 ppm were prepared with two different types of additives: lime and a combination of lime with ground-granulated blast furnace slag (GGBFS). The curing methods included: a 7-day moist cure, followed by immersion in water for 4 hours. Similar specimens were subjected to the same 7-day moist cure and subjected to a 10-day capillary rise. Results showed that submerging the specimens for 4 hours presented similar UCS values than the ones obtained for 10-capillary rise for soils with no sulfates. In the case of high sulfate content and combination of lime and GGBFS, 4-hour soak curing presented better results in terms of UCS on treated samples.

Kaniraj and Gayathri (2003) also studied the difference between UCS strengths on submersed specimens and specimens subjected to standard curing. Standard curing (STD) was defined as specimens wrapped in bags to prevent moisture loss and placed in a desiccator at 21°C and 100% constant humidity. Immersion comprised of placing the specimens in distilled water for 8 to 10 hours at room temperature. The average loss due to immersion was about 11% (see Table 2.18).

Geiger et al. (2006) compared the effect of water absorption through capillary and the soaking of specimens on pulverized and stabilized bases. The retained strength based on UCS results and retained moduli from FFRC were compared with three stabilizers before and after moisture

Table 2.18 – Retained Strength Ratio for Different Types of Curing (Kaniraj and Gayathri)

Type of Curing	*IMM*	*HYD*	*ASTM*	*ATM*
*Retained Strength**	*0.887*	*0.754*	*0.929*	*0.235*

*average UCS/average UCS for STD curing

conditioning. The moisture conditioning process consisted on applying 10 days of capillary action after 7 days of curing following TxDOT specifications, while on the soaking method specimens were immersed in water for 4 hours. However, the latter method was only feasible on soil cement specimens, because with other stabilizers most specimens would disintegrate. The four-hour soak method typically showed greater retained strengths as compared to 10-day capillary moisture for soil-cements. However, the 10 day capillary condition procedure seemed to be a more robust testing method and more conservative than the 4-hour soaking, for mixes with 4% or more cement content.

Curing

Curing procedures, adjusting temperature or access to moisture or both, have an important influence in the strength development, durability, and performance of stabilized soils. Another important aspect is the effect of different curing conditions that exist between laboratory and field. Kaniraj and Gayathri (2003) studied the influence of curing conditions on the performance of cement and fly ash base materials in the field, compared to laboratory controlled conditions. More than 900 specimens were subjected to compaction and UCS tests to compare different curing methods. The following six curing methods were investigated for a period of 3 years:

1. STD. In this method, specimens extruded from molds were wrapped in plastic bags to prevent moisture loss and placed in desiccators at 100% humidity and 21°C.
2. IMM. Specimens were first cured following the STD method and then removed from bags and immersed in distilled water for 8 to 10 hours at room temperature before carrying out UCS tests.
3. HYD. This method was used to simulate conditions where pavements are subjected to prolonged flooding. Specimens were first cured following the STD method for seven days and then removed from bags and immersed in running tap water at room temperature for the remainder of the curing period.
4. ASTM. This method is similar to STD, but the specimens were not wrapped in bags and are exposed to ambient humidity within the desiccators.
5. ATM. In this method, the specimens were first cured for 7 days following STD method and then removed from bags and exposed to natural environmental elements for the remainder curing period (heat, light, rain, air, etc.). After the curing period specimens were immersed in distilled water for 8 to 10 hours before UCS tests.
6. NAT. Specimens were first cured for 7 days by the STD method and then the desiccators containing the specimens were kept outside the lab in the open for the rest of the curing period.

Kaniraj and Gayathri concluded that climatic conditions, such as light, temperature and humidity influenced the most in the reduction of strength. Table 2.18 summarizes the average retained strength for four of the six different curing methods attempted. When curing method ATM was used, the specimens only retained 24% of the original strength. In terms of UCS strengths, differences between the STD and ASTM methods were minimal, while the HYD produced lower values than STD. The least strength development was for the case of the ATM curing method, proving the strong influence of atmospheric elements. For the case of the NAT method, specimens presented higher UCS strengths than standard STD method during summer season curing and smaller strengths developed when the NAT method was applied during winter season.

Howard (1966) studied the relationship between the unconfined compressive strengths of normal laboratory and accelerated cured lime-stabilized kaolinite clay specimens. He concluded that accelerated curing periods of 24, 40, and 72 hours at 120°F produced unconfined compressive strengths approximately corresponding to 20, 40 and 90 days of laboratory curing at 72°F.

Drake et al. (1972) investigated the effect of elevated temperature and reduced time required for accelerated curing of 2 cohesive soils stabilized with lime and a combination of salt-lime. The objective was to obtain similar products after 28-day curing at 80°F and 100% humidity in terms of strength gain and chemical products generated. Studies at 95% humidity and temperatures of 120, 110, 105, and 100°F revealed that a temperature of 105°F best simulated conventional strength time curing conditions. For the two soils subject of the study, 30 hours of 105°F curing produced equivalent 28-day strength for stabilization with salt-lime. For lime-only treatment, accelerated curing time increased to 72 hours. The same study revealed that similar mineralogical characteristics existed among specimens when equivalent strengths were obtained. A tentative-strength mix design procedure using the technique for accelerated curing was also suggested.

Similar studies were conducted by Townsend et al. (1976). Unconfined compression tests were conducted on silt, clay, clay gravel and sandy gravel with different percentages of lime and fly ash. A few specimens were normally cured at 50°F and 72°F each for 7, 28, and 56 days, while others were accelerated cured 1, 3, and 5 days at 90°F, 105°F, and 120°F each. No universal equivalent 28-day curing for accelerated design of soil-lime or lime-fly ash mixtures was found, because 28-day normal-cure strengths were found to depend upon curing temperature, soil type, and to a lesser extent on lime content or lime-fly ash ratio. Nevertheless, mix design procedures were developed based upon normal-cure 7-day and accelerated-cured 105°F strengths for estimating 28-day strengths of soil-lime mixtures.

A higher curing temperature accelerates the progress of lime-soil reactions when lime additions are above the optimum (Rao and Shivananda, 2005). In that work, the pozzolanic activity commenced after 1 day of curing at 77°F compared to 7 days curing at a temperature of 53°F, suggesting that strength development from pozzolanic activity will occur faster in hot semi-arid climatic zones than in cool temperature zones (Rao and Shivananda, 2005).

Beeghly (2003) examined the influence of temperature on strength of lime and class C and F fly ash stabilized soils. For most cases a dramatic improvement of strength was found when lime was added to the fly ash. To measure strength the mortar cube procedure (ASTM C-593) and UCS tests (ASTM D-5102) were employed on several mixes. One set of tests included the strength gain and moisture sensitivity using UCS tests (ASTM D-5102) for three untreated and treated samples of moderate plasticity (PI < 20) and high silt content and for different concentration of additives. Specimens were prepared at optimum moisture content and three types of curing conditions that yielded equivalent strengths were used:

- Accelerated curing for 3 days at 120ºF
- Accelerated curing for 7 days at 104ºF
- Ambient curing for 28 days at 72ºF

From these tests Beeghly also concluded that lime-fly ash mixes are able to achieve greater strength that lime soil alone, even though more lime was used in the lime only tests. In addition, he applied a simple method to test for moisture susceptibility by exposing molded cylinders to capillary soak after curing for 24 hours. The method consisted on placing the 4-inch diameter cylinders, wrapped in a wet absorptive fabric on a porous stone sitting in water. Untreated clayey soils typically degraded to compressive strengths of less than 10 psi after capillary soak, but for stabilized soil specimens the loss of strength for UCS tests was about 15 to 25%. All specimens subjected to the three types of curing showed satisfactory results in terms of UCS strength considering a minimum of 100 psi. Beeghly concluded that similar results should be obtained for both lime and lime-fly ash on moderate plastic soils when curing for 3 days at 120°F and applying a capillary soak for 24 hours, compared to 28 days of standard curing at room temperature.

Army Stabilization Guide (1994) points out that the preferred method of curing for lime-soil specimens is 73°F for 28 days. An accelerated curing method that gives satisfactory results is to apply 120°F for 48 hours; however, tests at 73°F for 28 days should also be conducted and compare with the accelerated method, because if accelerated curing temperatures are too high, the pozzolanic compounds formed during laboratory curing could differ substantially from those that would develop in the field.

Similar recommendations are found in the TRB Lime Stabilization Report (1987). Mixture design procedures that require four or more weeks are not feasible and elevated temperatures were used in the laboratory to accelerate formation of pozzolanic reactive products and therefore speed up the mix design of lime-soil specimens. It is recommended that temperatures above 120°F should be avoided during laboratory curing. A temperature of 105°F was chosen as appropriate temperature without introducing reactive products from the pozzolanic activity different from the expected field curing. It is also indicated in the report that field evidence shows some soil-lime mixtures can continue to gain strength for in excess of 10-years.

Little (2000) evaluated the structural properties of lime stabilized soils. He recommended a curing method of 7 days at 105°F, with specimens placed in plastic bags to retain sufficient moisture, equivalent to one month at typical annual average temperatures of 75°F. In terms of moisture conditioning, samples were subjected to capillary soak for 24 for low to moderate plasticity (PI < 25) to 48 hours for high plasticity (PI > 25) prior to UCS testing to represent reasonable pavement conditions. Soak protocol consisted of moisture absorption through a porous stone at the base of each sample and a wrap made of absorptive fabric placed around the circumference of the sample. This approach is fast enough to make the mixture design feasible yet long enough, and at a sufficient temperature, to provide reasonable approximations of long-term cure (ultimate strength) under ambient field conditions. Three soils with PI greater than 10

(24, 29 and 38) were evaluated for lime stabilization. Unconfined compressive strength and resilient modulus tests were performed on specimens after a capillary soak. For all samples, UCS was above 300 psi and resilient modulus over 30 ksi on the three specimens. He also demonstrated that compaction energy and moisture during molding have a large effect on strength of both untreated and stabilized fine-grained soils. The modified Proctor compaction (AASHTO T 180 or ASTM D-1557) was compared to standard Proctor (AASHTO T 99 or ASTM D-698). UCS results were highly sensitive to moisture and compaction energy, showing that UCS was approximately 2 times greater using the modified compaction, with maximum values at optimum moisture content (OMC).

TUBE SUCTION AND FREE-FREE RESONANT COLUMN TESTS

Two inter-related methods can be used to assess the impact of moisture on the performance of the base and subgrade materials in terms of strength and modulus: Tube Suction Test (Tex-145) and the Free-Free Resonant Column (Tex-149). The Tube Suction Test (TST) qualitatively provides an estimate of the water-retention of the material that can be correlated to the potential of damage to the base due to softening. The Free-Free Resonant Column (FFRC) test is a quantitative nondestructive lab method that can be performed on a specimen for its modulus. Currently the process of moisture conditioning is similar for both tests. A specimen prepared at the optimum moisture content is oven-dried for two days and then allowed to soak moisture through capillary saturation. The modulus of the specimen and the dielectric constant of the specimen are measured every day using the FFRC and TST tests, respectively. Based on evaluation of more than a dozen soils throughout the state, it has been shown that the modulus from FFRC test along with the dielectric value from the TST tests are good indicators of the long -term performance of some materials subjected to moisture. Since both tests are nondestructive, the retained strength can be obtained by conducting the unconfined compressive strength on the specimen.

Scullion and Saarenketo (1997) proposed the TST to identify poorly performing unstabilized base materials by measuring their capillary rise and surface dielectric values of prepared specimens. The setup of the TST (Figure 2.9) consists of measuring the dielectric values of compacted or stabilized aggregate samples with a 50 MHz surface dielectric probe during a monitored exposure to capillary rise conditions in the laboratory. Through an empirical relationship of the dielectric value the expected performance of the base or subgrade can be predicted. Based on studies carried out on Texas and Finish soils (Scullion et al., 2005), unstabilized materials with dielectric values above 10 may not perform well under heavy traffic loads on areas subject to freeze-thaw or wet-dry cycles. Materials with values above 16 are considered to fail, and if the dielectric value is greater than 16, the material should be chemically stabilized.

The Free-free Resonant Column test is a simple laboratory test for determining the modulus and possibly Poisson’s ratio of pavement materials (Yuan et al., 2005). The modulus measured with this method is the low strain seismic modulus. This method applies not only to stabilized or compacted bases and subgrades, provided the length is greater than the diameter, but also applies to Portland cement concrete and asphalt concrete specimens. Due to the nondestructive nature of the test, specimens can be tested later for strength or stiffness (resilient modulus or UCS tests).

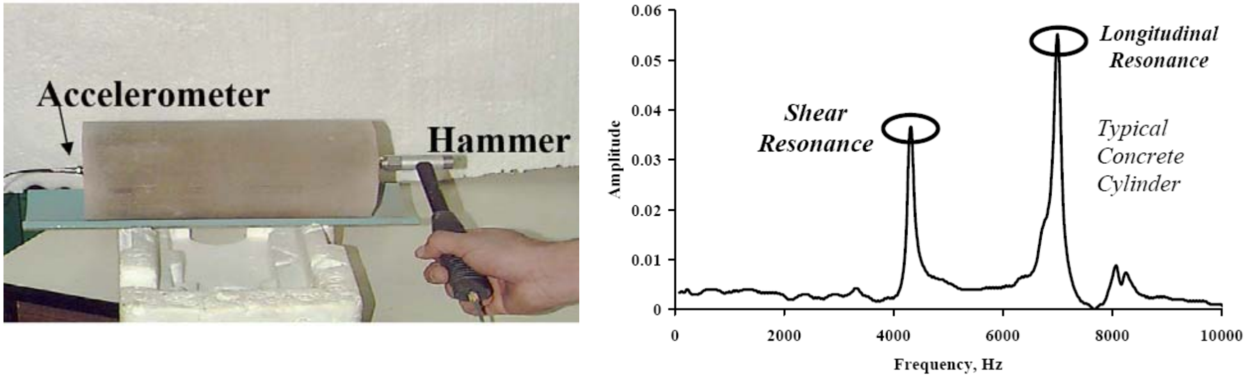
Figure 2.9 – TST Test Setup

A schematic of the test set-up is shown in Figure 2.10a. A hammer connected to a load cell impacts on one side of the specimen and an accelerometer attached on the other side reads the signal generated. Signals from the accelerometer and the load cell are used to determine the resonant frequencies of the given specimen (Figure 2.10b). These frequencies plotted in an amplitude spectrum graph appear as peaks, and usually two predominant peaks can be identified, corresponding to the longitudinal propagation of waves (higher frequency) and the shear mode of vibration.

Once the longitudinal resonant frequency, f_L, and the length of the specimen, L, are known, Young's modulus, E, can be found from the following relation:

$$E = \rho \, (2 \, f_L \, L)^2, \tag{2.1}$$

where ρ is mass density.



a) FFRC Test Setup b) Typical Resonant Frequencies

Figure 2.10 – Schematic of FFRC and Typical Results

EVALUATION OF LONG-TERM PERFORMANCE PARAMETERS

Another important objective of the stabilization is to address the permanency of chemical stabilizer, i.e. the additive to hold the soil particles for a long duration. Leaching of a chemical stabilizer through moisture movements will have serious implications on the durability and sustainability of the chemical treatment. One of the detrimental effects that a chemically-treated soil may experience is the loss of the chemical stabilizer through leaching. Previous studies report that the leaching through moisture flows in soils result in variations of pH and Calcium and Magnesium ratios, which can influence the permanency of the chemical modifiers (McCallister, 1990).

Few studies have been conducted on the leach test of chemically-treated soils to understand the leaching of chemicals from moisture flows. Barenberg (1970) reported leach tests on lime, cement and fly ash-treated soil samples compacted at optimum moisture contents. Leach tubes of 2 ft long and 4 in. diameter were filled with chemically-treated soils that were subjected to water leaching at a rate comparable to the estimated local rainfall. The process was performed for ten days and the leachate and soil samples were then chemically analyzed. This analysis showed that small amounts of chemical stabilizer leached out during these tests.

McCallister (1990) performed several leachate tests on lime-treated clays in specially-fabricated flexible cells (see Figure 2.11) for 45 to 90 days. Several variables including soil types, curing conditions and flow pressures were studied. By chemically analyzing the leachates collected, he stated that leaching had a detrimental effect on lime-treated clay properties. Maximum detrimental changes were observed when the lime content was less than the optimum needed.

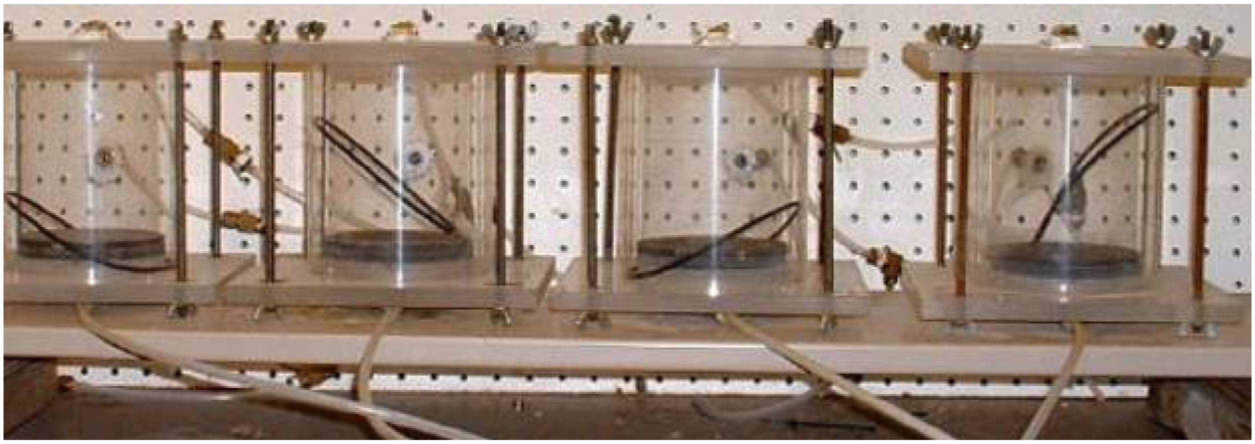


Figure 2.11 – Cyclic Wetting-Drying Setup

Another form of moisture conditioning effects on chemically-treated soils is related to moisture fluctuations from seasonal changes and their impact on the performance of these soils. This aspect is often studied in soil stabilization projects as a part of the durability studies. Wet-dry tests are typically conducted according to ASTM D 559 methods. Two similar samples of each soil/additive combinations are prepared at the optimum moisture content. The lime-treated soil specimens are prepared after mellowing, whereas the cement and other chemically treated soil specimens are prepared within an hour of mixing. Soil specimens are then cured for seven days in a moisture room prior to subjecting them to wet-dry cycles. Each wet-dry cycle consists of submerging the two soil samples in water for 5 hours and then placing them in a 70°C oven for 42 hours. After removal from the oven, one specimen is subjected to volume change and moisture content measurements. The second specimen is subjected to tests to determine the soil loss. The test is then continued until 12 wet-dry cycles are completed or until the sample failed.

Several studies have been performed on the wet-dry cycle related tests to address the durability issues to address the performance of stabilizers in arid conditions where such moisture fluctuations occur. Rogers and Wright (1986) studied natural Beaumont clay which had been used to construct road side embankments. These embankments suffered extensive slope failures, thus design strengths have been overestimated. They subjected the clay to thirty wetting and drying cycles, and shape of direct shear stresses. Cast acrylic chambers were made to maintain size and shape of direct shear and triaxial specimens for use in those devices respectively. To simulate in-field conditions, the soil was exposed to twenty-four hours of saturated conditions and twenty-four hours of drying conditions in an oven at 140ºF, creating a "quick aging" environment. The results show that repeated wetting-drying produce significant reduction in effective-stress shear strength parameters. Rogers and Wright (1986) found design factors of safety for failed embankments had been significantly reduced, but were still higher than unity. They concluded that the uncertainty in the results is due to the small amounts of scatter and uncertainty in the experimental data and recommendations were made for further laboratory testing to understand the effects of wetting-drying on natural high-PI clays.

Hoyos et al. (2005) performed a series of wet and dry cyclic tests (see Figure 2.12) on different types of chemically-treated sulfate soils to evaluate the strength, stiffness and volume change property variations with respect to these cycles. An attempt will be made to review these studies and develop a test protocol that could best simulate Texas field environmental conditions.

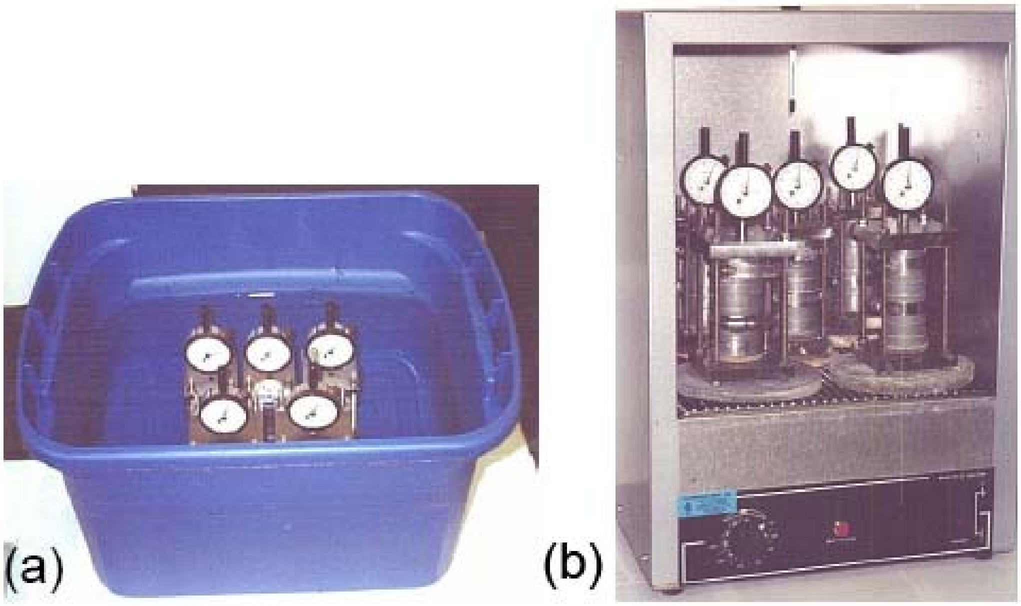


Figure 2.12 – Wet (a) and Dry Cycles Setup Used by Hoyos et al. (2005)

CHAPTER THREE – BASELINE DATABASE

INTRODUCTION

The objective of this chapter is to evaluate the current TxDOT specifications for soil stabilization and develop a baseline database covering a wide range of soils in terms of material properties and environmental conditions. A total of six soils are included in this chapter: four clays and two sandy materials. All clays were stabilized with lime and Tex-121-E specifications were used as reference. For the case of Wichita Falls sand, cement was used as stabilizer and fly-ash was applied on the Bryan sand. Tex-120-E and Tex-127-E specifications were followed on these materials respectively. In addition to TxDOT specifications, X-Ray diffraction, reactive alumina and silica tests before and after treatment were performed to study the chemical composition on these soils and results are also included in this chapter.

SOILS SELECTED FOR STUDY

Based on the interaction with the districts and the PMC of the project, six materials were selected to generate a baseline for verification of the outcome of this project. These materials were selected after carefully reviewing the responses from the questionnaires distributed among TxDOT Districts (see Appendix A). The six soils used in the study covered by this chapter are:

- El Paso Clay
- Bryan Clay
- Fort Worth Clay
- Paris Clay
- Wichita Falls Sand
- Bryan Sand

Table 3.1 summarizes the index parameters utilized for the selection of additives for the soils tested, including gradation and soil classification. The clayey soils were selected considering their clay mineralogy to meet one of the project objectives of developing a stabilizer selection process based on the clay types rather than the Atterberg limits. For this purpose, several test methods to determine various chemical characteristics of the soils were used. These methods were chosen such that they can be conducted in the laboratory with simple apparatus using gravimetric principles. Tests proposed included the Cation Exchange Capacity (CEC), Specific

Table 3.1 – Soil Classification and Plasticity Index for Soils under Study

Soil	Gradation, %				Classification		Atterberg Limits		
	Gravel	Sand	Silt	Clay	USCS	AASHTO	LL	PL	PI
El Paso	0	37	42	21	CL	A-6	30	14	16
Bryan Clay	0	13	40	47	CL	A-7-6	45	14	31
Fort Worth	0	11	37	52	CH	A-7-5	61	32	29
Paris	0	9	45	46	CH	A-7-6	60	24	36
Wichita Falls	0	93	6	1	SM	A-4	--	--	N/P
Bryan Sand	1	53	18	28	SC	A-1-b	--	--	N/P

Surface Area (SSA), Total Potassium, Exchangeable Potassium and Reactive Alumina. The first three methods are known to provide clay mineralogy information, whereas the last two methods provide reactive chemicals in soils that could result in pozzalanic compound formation. The total potassium in the soil as opposed to the exchangeable potassium can be used to better quantify the Illite in a clayey soil. A modified methodology was formulated to obtain that property (see Appendix C). This method was used to determine total potassium, which in turn can be used for final quantification of clay minerals.

Chemical analysis results are presented in Table 3.2, and the percent clay minerals in the soils selected are presented in Table 3.3. Overall, these soils represent the ranges of clay mineral properties needed for this research. Based on the chemical test results, the three soils with the highest CEC and SSA were proposed as test soils (Bryan, Fort Worth and Paris materials). On

Table 3.2 – Chemical Analysis Results for Soils under Study

Soil Region	Sulfate Content*, ppm.	CEC meq/100g	SSA m^2/g	Reactive Potassium, %	Reactive Aluminum, %
El Paso	1202	57	161	0.068	0.060
Bryan Clay	498	77	205	0.069	0.200
Fort Worth	358	117	314	0.030	0.154
Paris	136	133	431	0.032	0.180
Wichita Falls	169	58	158	0.072	0.056
Bryan Sand	256	89	210	1.12	.106

** Measurements based on fine fractions in the soil*

Table 3.3 – Percent Clay Mineral Observed for Soils under Study

Soil Region	% Illite	% Kaolinite	% Montmorillonite
El Paso	62.5	14.2	23.3
Bryan Clay	22.7	40.0	37.3
Fort Worth	16.3	23.4	60.3
Paris	13.2	16.7	70.1
Wichita Falls	58.8	22.0	19.3
Bryan Sand	18.7	42.4	38.9

the contrary, a soil with low CEC and SSA values was selected for comparison (El Paso material). Two sandy materials from Wichita Falls and Bryan were also included in the study. High values of CEC and SSA indicate the presence of large amounts of montmorillonite and illite minerals. The sand and silt materials represent those with small amounts of clay minerals. The finer fractions of the El Paso and Wichita Falls soils contained Illite as the dominant clay mineral whereas the rest of the soils possessed Montmorillonite as the dominant clay mineral.

MIX DESIGN

The objective of the mix design was to determine the type and concentration of additive and evaluate the improvement of engineering properties with varying concentrations of the selected additive. As an illustrative example, the steps carried out to obtain the optimum additive content for the El Paso Clay are shown below to ensure the long-term strength and durability of the stabilized soils.

Selection of Stabilizing Agent

According to the soil properties measured for the El Paso material, the stabilizer selected is lime (refer to Figure 2.3).

Calculation of Stabilizer Content

To determine the minimum amount of stabilizer that is necessary, the pH or Eades-Grim test was performed as outlined in the Tex-121-E method. The variation in pH versus lime content for the El Paso material is shown in Figure 3.1.

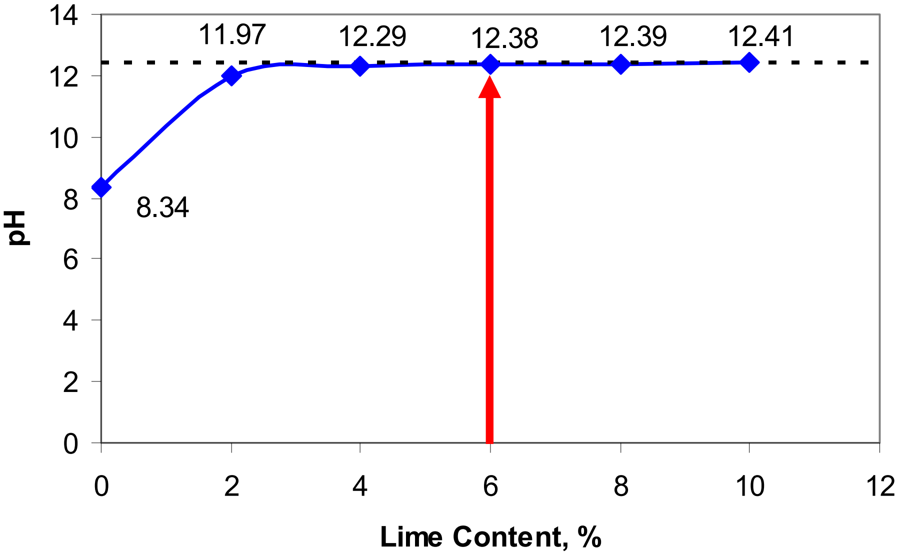


Figure 3.1 – Variation in pH with Lime Content for El Paso Clay

To study the influence of lime content on the strength, three lime contents of 6%, 8% and 10% were considered. The optimum moisture contents (OMC) and maximum dry unit weights (MDUW) for the three soil-lime combinations are shown in Table 3.4. As the lime content increases, the OMC decreases, while the MDUW fluctuates by about 3%.

Table 3.4 – El Paso Clay Moisture Density Characteristics

Lime Content, %	Optimum Moisture Content, %	Max. Dry Unit Weight (lb/ft^3)
0	16.5	112.1
6	15.8	109.2
8	15.1	113.0
10	14.9	109.9

Summary of Tests Performed

A series of tests were carried out to determine the appropriateness of the stabilizer and its content. The different tests performed were:

- Unconfined Compressive Strength Standard Curing (UCS STD)
- Unconfined Compressive Strength Dry Curing (UCS Dry)
- Resilient Modulus Standard Curing (RM STD)
- Resilient Modulus Dry Curing (RM Dry)
- Tube Suction Test (TST)
- Free-Free Resonant Column (FFRC) Tests
- Variation in Moisture Content

The time needed and specifications for curing to complete each of these tests are detailed in Table 3.5. These results are divided in terms of strength, stiffness, dielectric constant and moisture content tests.

Table 3.5 – Curing Methods for Strength and Stiffness Tests on Clay

Curing Type	Standard (STD)	Dry	Tube Suction Test (TST)
Moist Curing	7 days on counter	7 days on counter	N/A
Drying	6 hrs in oven at a temperature not to exceed 60°C	6 hrs in oven at a temperature not to exceed 60°C	2 days in oven at 40°C
Moisture Conditioning	10 days of capillary saturation	N/A	8 days of capillary saturation
Total Number of Days	17 days	7 days	10 days

Strength Tests

Unconfined compressive strength (UCS) tests carried out on the standard cured, dry and TST specimens for the three soil-lime combinations are summarized on Figure 3.2. For the three soil-lime combinations the minimum strength is achieved for all lime combinations and for all curing types. The strengths at OMC for the El Paso material in its natural state are also included after 24 hrs of compaction with no curing applied and after drying to constant moisture content. The final moisture content of the dried specimen with no additives was 2.6%, which is substantially less than those with additive (about 8%).

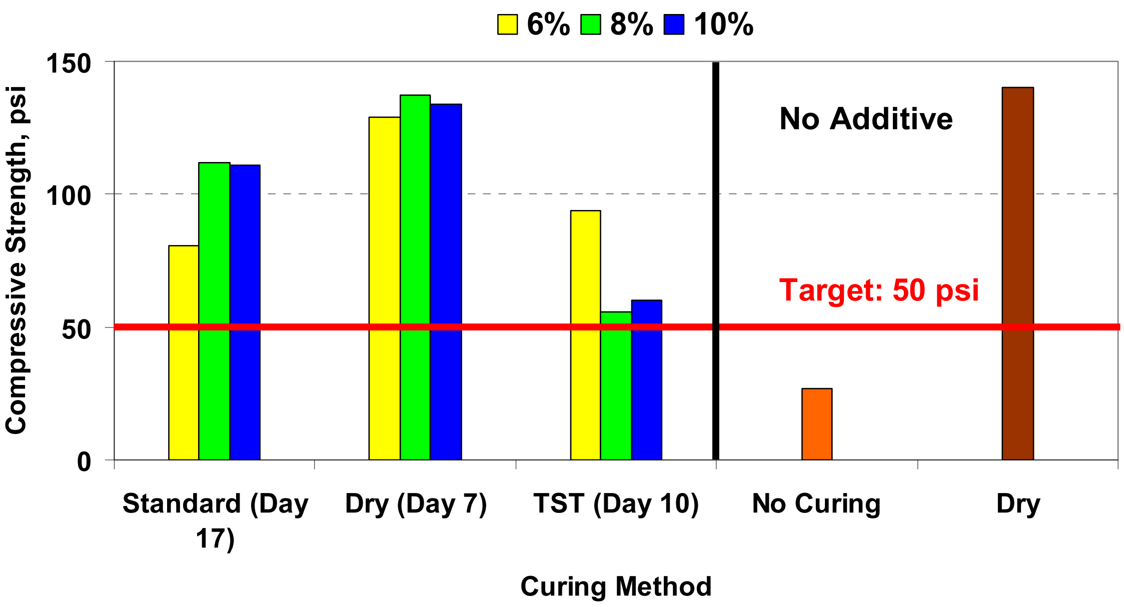


Figure 3.2 – Unconfined Compressive Strength Results for El Paso Soil-Lime Specimens

In addition to compressive strength tests, indirect tensile tests (IDT) were performed as supplementary tests on one specimen subjected to the standard, dry curing methods and, with no curing after 24 hours of compaction. Results are shown in Figure 3.3.

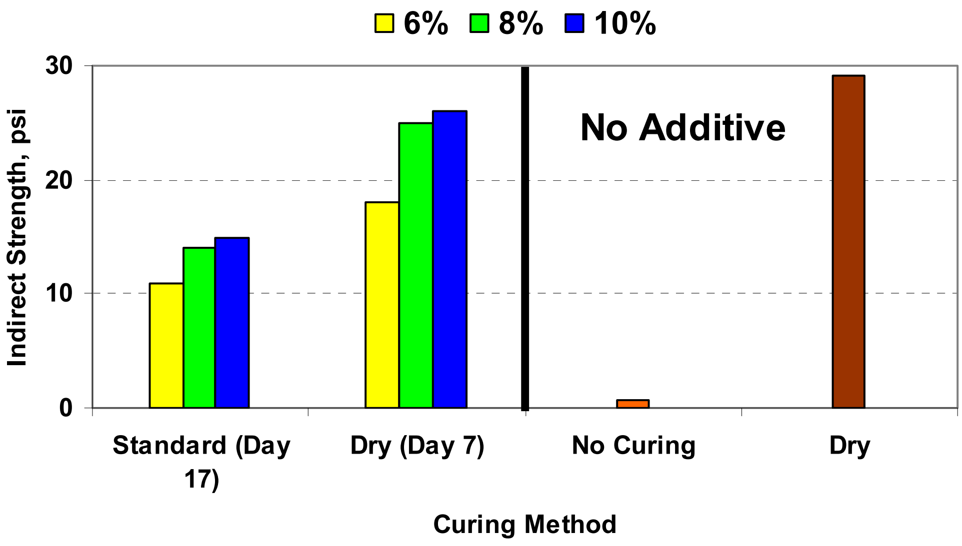


Figure 3.3 – Indirect Tensile Strength Results for El Paso Soil-Lime Specimens

Stiffness Tests
The step-by-step procedure to determine the resilient moduli of different materials can be found in Nazarian et al. (1999). For subgrade materials, 4 in. by 8 in. specimens were prepared. To complete these tests various deviatoric stresses at different confining pressures are applied. The sequence used in this project, includes a combination of confining pressures of 0, 2, 4, and 6 psi

and deviatoric stresses of 2, 4, 6, 8, and 10 psi which is a modified version of the sequence found in AASTHO T-307. The following nonlinear constitutive model was adopted:

$$E = k_1 \ \sigma_c^{\ k_2} \ \sigma_d^{\ k_3} \tag{3.1}$$

where k_1, k_2 and k_3 are coefficients determined from laboratory resilient modulus tests and σ_c and σ_d are the confining pressure and deviatoric stress, respectively.

Figure 3.4 shows the resilient modulus results for the El Paso Clay with lime content of 8% after 17 days of standard curing and after 7 days of dry curing. As anticipated, confining pressure does not influence the final resilient moduli, indicating that only tests at zero confining pressure are necessary to obtain resilient modulus. Thus, tests on other materials were conducted only at zero confining pressure to simplify the procedure. An unanticipated trend is the increase in the resilient modulus with the increase in deviatoric stress. Typically, the resilient modulus should not be impacted by the deviatoric stress as somewhat reflected in the results at higher deviatoric stresses in Figure 3.4. This problem might be associated with the test configuration proposed in the AASHTO T-307 for stabilized materials.

The average resilient moduli from all tests are summarized in Table 3.6 for each specimen and for each curing method and the average modulus is regarded as the representative resilient modulus. The FFRC seismic modulus and unconfined compressive strength of each specimen (after subjected to the resilient modulus tests) are also included in the same table for comparison purposes.

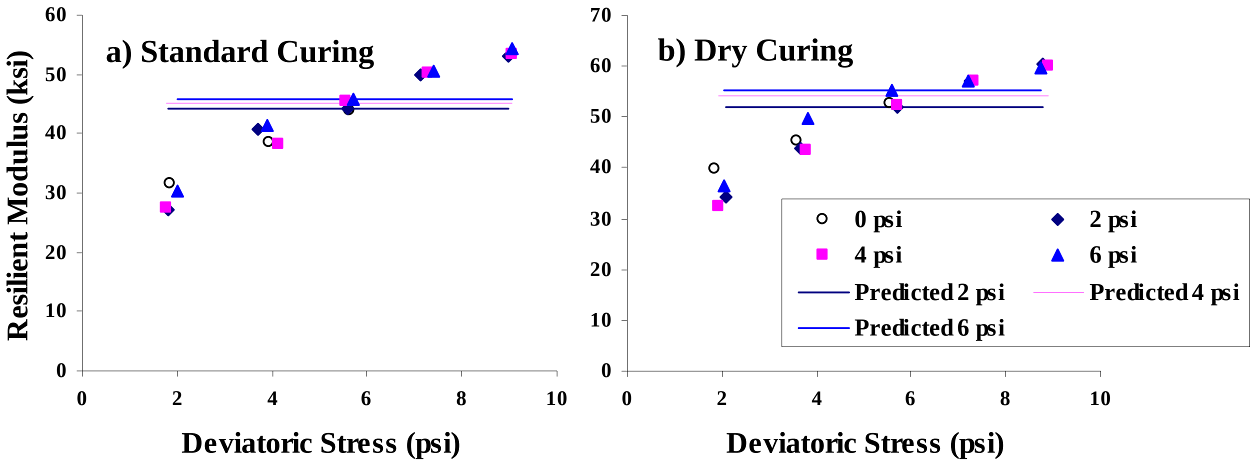


Figure 3.4 – Resilient Modulus Results for El Paso Specimens with 8% Lime

Table 3.6 – Resilient Modulus Results for El Paso Clay

Parameter	Lime Content and Curing Method						
	0%	6%		8%		10%	
	No Curing	Dry	Standard	Dry	Standard	Dry	Standard
Representative Resilient Modulus, ksi	9	60	44	62	43	65	37
FFRC Seismic Modulus, ksi	15	333	214	396	284	334	203
UCS, psi	27	170	109	282	N/A*	132	83

** Specimen broke while testing was conducted*

In addition to resilient modulus, the permanent deformation properties of the stabilized material were also determined from a series of repeated load tests. Specimens prepared for resilient modulus tests were used on these tests. Once the specimen is placed in the testing chamber, 100 conditioning cycles followed by 10,000 repetition cycles are applied to the specimen and the deformations are recorded by two LVDTs and then averaged. With this information the rutting parameters α and μ are determined by following steps (Gandara et. al, 2005):

- Determine cumulative axial permanent strain and resilient strain (ε_r), at the 200th repetition cycle
- Graph cumulative axial strain versus the number of cycles in a log-log plot to determine permanent deformation parameters (i.e., intercept, *a*, and slope, *b*)
- Rutting parameters α and μ are calculated from parameters *a*, and *b* with equations

$$\alpha = 1 - b \tag{3.2}$$

$$\mu = \frac{a \times b}{\varepsilon_r} \tag{3.3}$$

Figure 3.5 shows typical permanent deformation test results. The resilient and permanent strains for each specimen are shown in Table 3.7. These values are very small, indicating that excess permanent deformation for these materials is small and practically negligible.

Table 3.7 – Permanent Deformation Parameters for El Paso Soil-Lime Specimens

Parameter	Lime Content and Curing Method					
	6%		8%		10%	
	Dry	Standard	Dry	Standard	Dry	Standard
Resilient Strain, μ-strains	8	21	47	N/A*	30	160
Permanent Strain, μ-strains	26	67	80		49	720

** Specimen broke while testing was conducted*

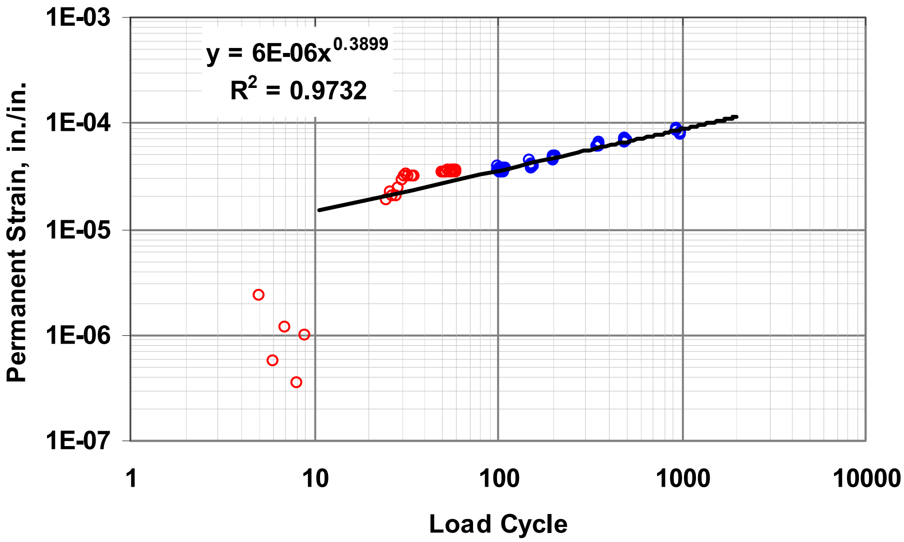


Figure 3.5 – Permanent Deformation Results for El Paso Lime Specimens with 8% Lime

Dielectric Constant Tests (Tube Suction Tests)

An Adek Percometer^{TM} with a 50 MHz surface dielectric probe is employed to measure the dielectric values of the TST. Figure 3.6 shows the variations in dielectric constant with time for El Paso Clay with the three amounts of stabilizer used (6%, 8% and 10%). The dielectric values for each day are similar for all specimens measured, with a final dielectric value of about 40. For base materials, aggregates whose average dielectric values are less than 10 are expected to provide superior performance (Scullion and Saarenketo, 1997), while those with dielectric values above 16 may result in poor performance acceptance criteria for subgrade materials have not been developed.

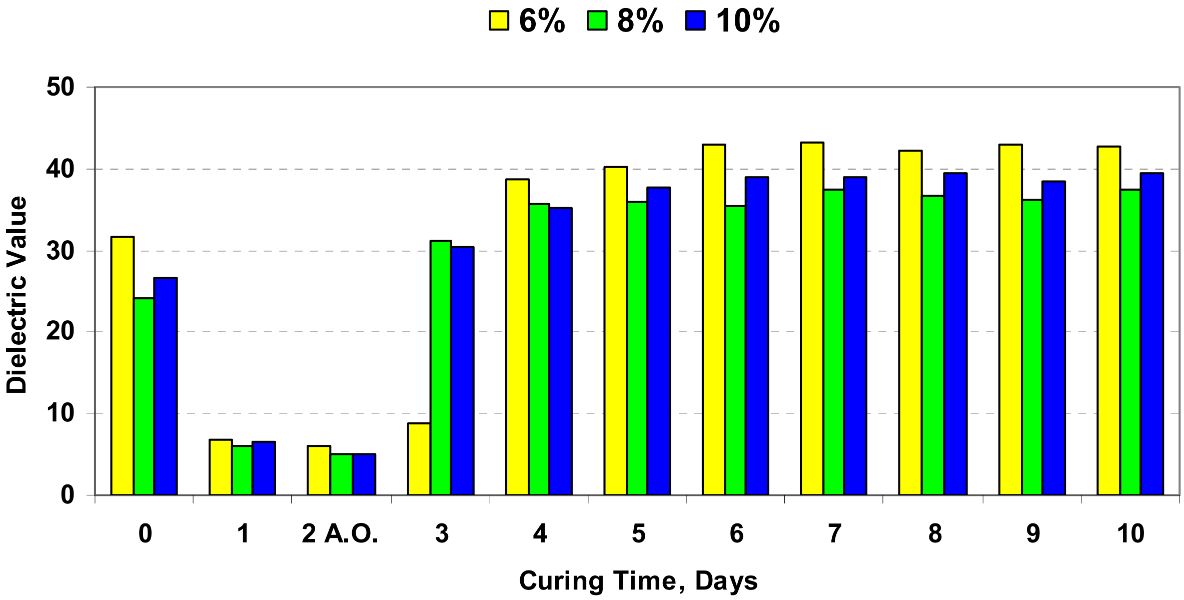


Figure 3.6 – Dielectric Results for El Paso TST Soil-Lime Specimens
(A.O. = after specimen was removed from the oven)

Variation of Moisture Content

The moisture content was monitored everyday on all specimens and for each curing method investigated in the baseline. Figure 3.7 depicts the average variations of moisture content for El Paso Clay specimens for TST and standard cured specimens. Similar trends are observed for the three lime contents used in the study and for every curing method. Based on these results TST curing method seems to apply wider moisture fluctuations to the soil-lime specimens, especially when 10% lime is added to the material. For the standard and dry curing conditions, the only significant changes in moisture occur during the six hours that the specimens are placed in the oven. Due to low permeability and layering because of compaction, the absorption of moisture in most cases is limited to the bottom half of the specimens for the standard and TST moisture conditioned specimens.

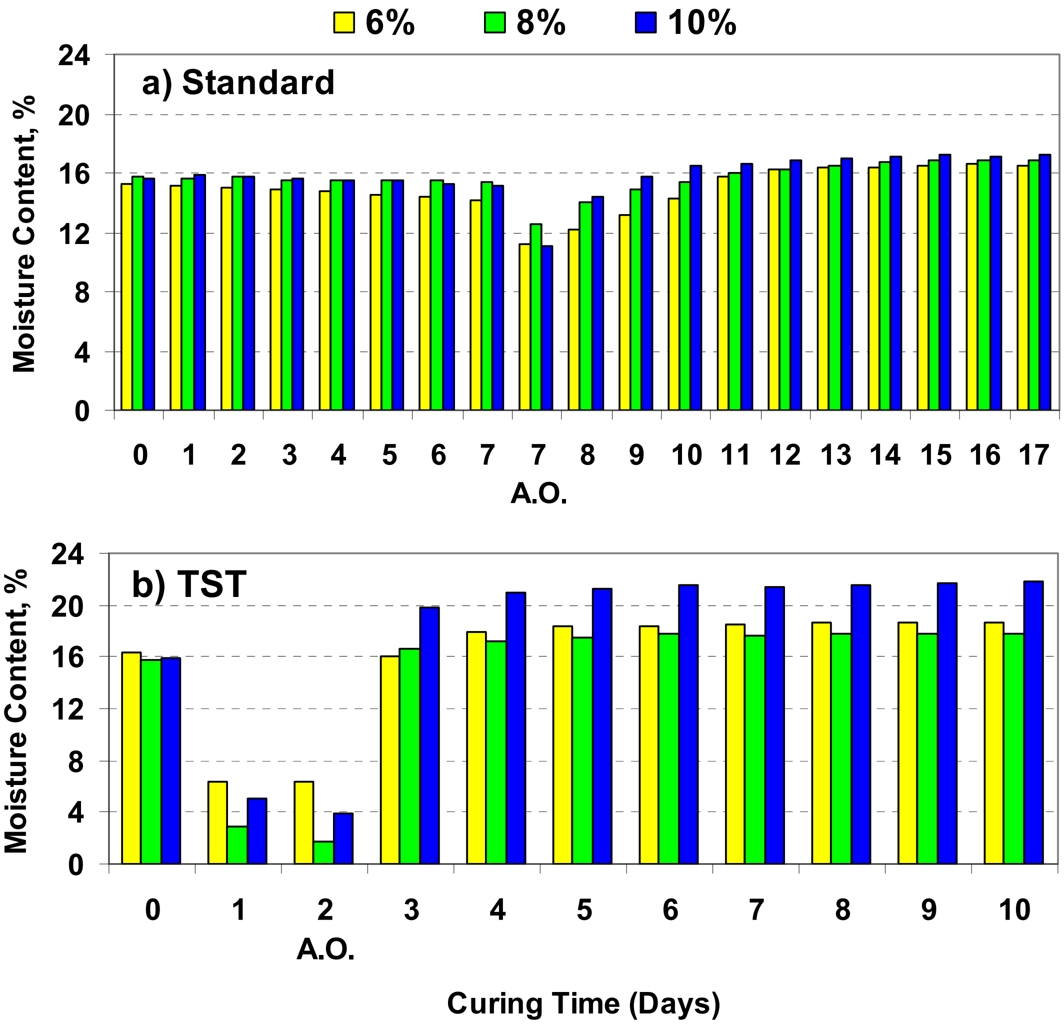


Figure 3.7 – Variations in Moisture Content for El Paso Clay Specimens
(B.O. = before specimen placed in the oven,
A.O. = after specimen was removed from the oven)

Variation of Modulus

The FFRC moduli for the two curing methods used in this study are shown in Figure 3.8. The moduli generally increase with time during the first seven days of curing, after which the modulus increases or stays constant with moisture conditioning.

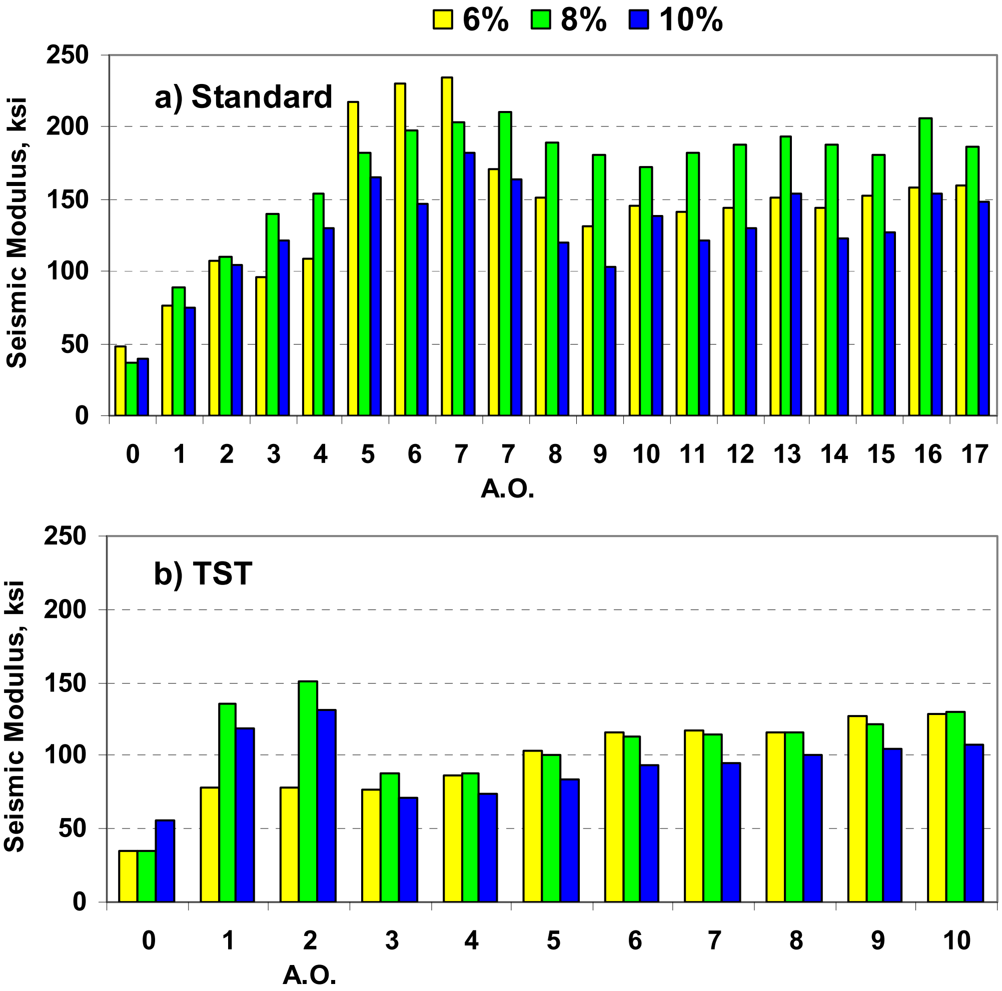


Figure 3.8 – FFRC Seismic Modulus Results for El Paso Soil-Lime Specimens

Long-Term Performance of Mixes

The dielectric values from TST, the moduli from FFRC tests, and the strengths from UCS tests can be used as indicators of the long-term performance of the mixes during the curing process. Using a single moisture-conditioning protocol, the change in moisture content, the variation in dielectric constant, and the variation in modulus with time can be readily obtained. The results are shown in Table 3.8. Final dielectric constants for all lime contents are close to 40. For each mix, the modulus increases for the first two days in the oven. For the specimens with 6% lime, the modulus further increases, whereas for the specimens with higher lime concentration, the residual moduli are less than the maximum moduli. This trend has to do with the efficiency of the movement of water within the specimens during capillary saturation. As reflected in Table 3.8, as the lime concentration increases, the change in moisture content between day 2 and day

Table 3.8 – Variations in Parameters Measured in TST Tests with Time

Lime Content, %	Final Dielectric Constant	Initial Seismic Modulus, ksi,	Maximum Modulus, ksi*	Residual Modulus, ksi**	Residual/ Maximum Modulus	Change in Moisture Content, %***
6	43	35	79	124	1.57	12.3
8	38	35	150	122	0.81	16.1
10	39	56	132	104	0.79	17.8

** Maximum modulus is defined as the seismic modulus after two days of oven curing*

*** Residual modulus is defined as the average seismic modulus of the last 3 days of TST curing*

**** Change in moisture content is defined as the difference in moisture content between day 10 and 2(after oven)*

10 increases. Our preliminary work on the permeability of the lime-stabilized mixtures for this project indicated that as the lime content increases the permeability of the mix increases due to ion exchange and flocculation in the compacted clayey specimens.

Another way to verify the long-term durability of the mixes is to compare the retained strength and retained modulus of the specimens. For this purpose, the ratios of the unconfined compressive strengths and seismic moduli between the standard and dry curing methods were used. Results for the El Paso Clay are shown in Table 3.9. The retained strengths increased from 0.63 to 0.83 as the lime content increased. Considering a threshold value of 0.8 as acceptable, at least 8% lime is required. The retained moduli were however varying between 0.73 and 1.09. Once again considering a threshold value of 0.8, the mixture with 6% lime is not adequate for stabilization.

Table 3.9 – El Paso Clay Retained Strengths and Moduli

Lime Content, %	Dry Curing Strength, psi	Standard Curing Strength, psi	Retained Strength	Dry Curing Modulus, ksi	Standard Curing Modulus, ksi	Retained Modulus
6	129	81	0.63	219	159	0.73
8	137	112	0.81	182	187	1.03
10	134	111	0.83	136	148	1.09

Final Mix Design

The minimum amount of stabilizer that is necessary to stabilize El Paso material based on the Eades-Grim test was 6% lime (see Figure 3.1). Based on the results from Figure 3.2, compressive strengths were greater than 50 psi but did not significantly improve with the increase in lime. Retained strengths increased from 63% for a lime content of 6% to 81% with a lime content of 8%, indicating that this amount is more appropriate for providing long term durability of the soil-lime mix. Similarly, the TST strength results had comparable values as compared to the standard and dry-cured specimens. FFRC results for all curing methods showed minimum values of 100 ksi for all specimens after the curing process. As a result, 8% lime appears to be more appropriate.

RESULTS FROM CLAY SPECIMENS

Stabilizer Content

Based on the variations in pH versus lime content for the Bryan, Fort Worth and Paris, the minimum amount of lime necessary to stabilize these materials is 10%, 8% and 6%, respectively. As with the El Paso Clay, the appropriateness of these soil-mixes is established by strength requirements. The optimum moisture contents and maximum dry unit weights for different soil-lime combinations are shown in Table 3.10.

Table 3.10 – Moisture Density Characteristics

Soil Type	Lime Content, %	Optimum Moisture Content, %	Max. Dry Unit Weight (lb/ft^3)
Bryan	0	19.5	97.7
	6	20.6	98.4
	8	22.7	95.3
	10	21.8	99.8
Fort Worth	0	24.0	91.5
	6	23.1	90.8
	8	23.8	91.2
	10	22.2	89.0
Paris	0	23.0	92.1
	6	24.0	88.8
	8	24.3	85.3
	10	24.5	88.1
El Paso	0	16.5	112.1
	6	15.8	109.2
	8	15.1	113.0
	10	14.9	109.9

Summary of Tests Performed

Unconfined compressive strength (UCS) tests were carried out on standard cured, dry and TST specimens for the three soil-lime combinations. Results are summarized in Table 3.11. The strength tests for the Bryan and especially Paris clays were somewhat problematic because the cracking of specimens at the interface of the compaction layers during the seven days of drying, or the slipping failure of the specimens at the interface of layers during the UCS tests (see Figure 3.9 as an example). For the three soil-lime combinations the minimum strength of 50 psi was obtained for the Bryan and Fort Worth clays under standard curing condition. As indicated above, tests on the Paris clay, despite numerous attempts, were problematic and the numbers may not be accurate. As such, the compaction method should be addressed to obtain reliable results. The indirect tensile strength (IDTS) test results also show substantial drop in strength when moisture conditioned. Reliable indirect tensile tests on the Paris clay could not be carried out as discussed before.

Table 3.11 – Soil-Lime Strength Results

Soil Type	Lime Content, %	UCS, psi			IDTS, psi	
		Curing Method			Curing Method	
		Standard	Dry	TST	Standard	Dry
Bryan	0	26*	269**	N/A	4*	34**
	6	85	171	206	16	26
	8	184	96	227	23	37
	10	171	107	208	16	34
Fort Worth	0	36*	206**	N/A	3*	15**
	6	60	86	39	10	12
	8	63	130	61	4	11
	10	65	168	81	11	24
Paris	0	28*	279**	N/A	4*	18**
	6	N/A	36	31	N/A	N/A
	8	69	75	26	10	10
	10	56.5	116	25	12	18
El Paso	0	27*	140**	N/A	1*	29*
	6	81	129	94	11	18
	8	112	137	56	14	25
	10	111	134	60	15	26

* Cured for 24 hours at room temperature

** Dry Cured until a constant moisture content reached (these specimens were much drier than those with lime)

Figure 3.9 – Typical Paris Specimens

Stiffness Tests

The representative resilient modulus test results for Bryan, Fort Worth and Paris clays are summarized in Table 3.12. Only tests at zero confining pressure were performed, since confining pressure should not affect the outcome of the tests as demonstrated with the El Paso results. The FFRC moduli and UCS results for specimens used in resilient modulus tests are also included in the table.

Table 3.12 – Resilient Modulus Results for Soil-Lime Specimens

Soil Type	Parameter	Lime Content and Curing Method						
		0%	6%		8%		10%	
		No Curing	Dry	Standard	Dry	Standard	Dry	Standard
Bryan	Representative Resilient Modulus, ksi	4	33	40	33	47	N/A	43
	FFRC Seismic Modulus, ksi	7	257	299	300	330		120
	UCS, psi	26	272	132	242	213		184
Fort Worth	Representative Resilient Modulus, ksi	9	40	30	27	14	53	37
	FFRC Seismic Modulus, ksi	15	181	127	161	112	241	149
	UCS, psi	36	159	161	172	31	164	71
Paris	Representative Resilient Modulus, ksi	13	N/A	23	15	17	10	50
	FFRC Seismic Modulus, ksi	28		120	53	218	52	256
	UCS , psi	28		81	112	69	73	92
El Paso	Representative Resilient Modulus, ksi	9	60	44	62	43	65	37
	FFRC Seismic Modulus, ksi	15	333	214	396	284	334	203
	UCS, psi	27	170	109	282	N/A	132	83

The resilient and permanent strains for each specimen are shown in Table 3.13. As for the El Paso clay, these values were rather small indicating that the rutting potentials of these materials are not significant.

Durability of Soil–Lime Mixes

Results obtained from tests on the TST specimens are shown in Table 3.14. The dielectric constants for the Bryan and Fort Worth clays were about 10, whereas the dielectric constant for the Paris clay was as high as 21. As shown in Table 3.14, the changes in moisture content were rather low (less than 3%) for the Bryan Clay, and between 5% and 12.5% for the Fort Worth and Paris clays.

Table 3.13 – Permanent Deformation Parameters for Soil-Lime Specimens

Soil Type	Nonlinear Model Parameters	Lime Content and Curing Method					
		6%		8%		10%	
		Dry	Standard	Dry	Standard	Dry	Standard
Bryan	Resilient Strain, µstrains	450	150	100	120	N/A	48
	Permanent Strain, µstrains	310	150	360	110		40
Fort Worth	Resilient Strain, µstrains	180	160	14	750	42	130
	Permanent Strain, µstrains	320	230	33	820	110	140
Paris	Resilient Strain, µstrains	N/A	68	53	130	290	65
	Permanent Strain, µstrains		110	180	97	480	49
El Paso	Resilient Strain, µstrains	8	21	47	N/A	30	160
	Permanent Strain, µstrains	26	67	80		49	720

Table 3.14 – Variations in Parameters Measured in TST Tests with Time

Soil Type	Lime Content, %	Final Dielectric Constant	Initial Seismic Modulus, ksi,	Maximum Modulus, ksi*	Residual Modulus, ksi**	Residual/ Maximum Modulus	Change in Moisture Content, % ***
Bryan	6	9	57	41	51	1.23	1.2
	8	10	55	64	89	1.40	1.8
	10	9	54	82	102	1.45	2.7
Fort Worth	6	8	47	65	71	1.09	4.6
	8	11	51	34	29	0.84	12.0
	10	9	40	49	31	0.64	8.8
Paris	6	7	16	15	30	2.01	7.2
	8	17	14	17	37	2.21	8.8
	10	21	19	17	29	1.72	12.5
El Paso	6	43	35	79	124	1.57	12.3
	8	38	35	150	122	0.81	16.1
	10	39	56	132	104	0.79	17.8

** Maximum modulus is defined as the seismic modulus after two days of oven curing*

*** Residual modulus is defined as the average seismic modulus of the last 3 days of TST curing*

****Change in moisture content is defined as the difference in moisture content between day 10 and 2(after oven)*

The FFRC moduli for every curing method and for the three clays are also shown in Table 3.14. The initial moduli were the highest for the Bryan clay and the lowest for the Paris clay. For the

Bryan clay, the residual moduli were greater than the maximum simply because the specimens did not soak much water.

Retained compressive strengths and moduli between the dry and standard curing methods are presented in Table 3.15. For the Bryan clay, the retained strength for the 6% lime was below 0.8 and the corresponding retained modulus about 0.9. For other lime contents, the retained strengths and moduli were way above 1. For the Fort Worth clay, the retained strengths were lower than 0.8 but the retained moduli were greater than 1.2. This occurred because the material was becoming brittle with time, and the UCS failure is more occurring at the interface of the layers. The results from the Paris clay should be interpreted with caution because of difficulties in conditioning the specimens.

Table 3.15 – Soil-Lime Mixes Retained Strengths and Moduli

Soil Type	Additive Content, %	Dry Curing Strength, psi	Standard Curing Strength, psi	Retained Strength	Dry Curing Modulus, ksi	Standard Curing Modulus, ksi	Retained Modulus
Bryan	6 %	171	85	0.50	98	87	0.89
	8 %	96	184	1.92	57	156	2.76
	10 %	107	171	1.60	78	163	2.11
Fort Worth	6 %	86	60	0.70	60	102	1.69
	8 %	130	63	0.48	81	100	1.23
	10 %	168	65	0.39	62	93	1.49
Paris	6 %	36	N/A	N/A	12	160	13.10
	8 %	75	69	0.92	33	104	3.14
	10 %	118	66	0.56	46	159	3.48
El Paso	6 %	129	81	0.63	219	159	0.73
	8 %	137	112	0.81	182	187	1.03
	10 %	134	111	0.83	136	148	1.09

Final Mix Design

Final additive contents for the four clays of the baseline are shown in Table 3.16 along with dominating clay mineral in the original clay. Results from the pH tests are included in the table for comparison as well. Based on the overall results of the laboratory tests, the amount of stabilizer for Bryan that provides adequate strength is 6% lime. However, retained strengths increased considerably from 50 to 192% with a lime content of 8%, indicating that this amount is better for providing long term durability of the soil-lime mix. In the case of Fort Worth, the addition of 6% lime provides minimum strengths above the target value and also retains 70% of the compressive strength. In light of high retained modulus, 6% lime may be sufficient for stabilization. In the same manner, the amount of lime that is enough to provide stabilization for Paris Clay is 8%.

Table 3.16 – Soil–Lime Final Mix Designs

Soil Type	Dominant Clay Mineral	Amount of Additive	Amount Needed from pH Test
El Paso	Illite	8%	6%
Bryan	Kaolinite	8%	10%
Fort Worth	Montmorillonite	6%	8%
Paris	Montmorillonite	8%	6%

RESULTS FROM WICHITA FALLS SAND SPECIMENS

To determine the minimum amount of cement necessary for stabilization, the criteria in Tex-120-E was used. The OMC and MDD for several cement contents are shown in Table 3.17.

Table 3.17 – Moisture-Density Characteristics

Soil Type	Cement Content, %	Optimum Moisture Content, %	Max. Dry Unit Weight (pcf)
Wichita Falls	0	11.3	115.6
	2	8.7	123.3
	3	9.0	123.6
	4	9.2	122.9
	6	9.7	120.9
	8	10.2	123.6
	10	10.7	122.7

The tests performed to determine the appropriateness of stabilizer and its content is the same as those performed on soil-lime mixes. However, the curing methods differed because of the nature of the additive. Drying in the oven was only for the TST specimens, and the curing was done in a damp room instead of placing the specimens on the counter. The three curing methods applied are summarized in Table 3.18.

Table 3.18 – Curing Methods for Strength and Stiffness Tests for Soil-Cement Specimens

Curing Type	Standard (STD)	Moisture-Conditioned	Tube Suction Test (TST)
Moist Curing	7 days in damp room	7 days in damp room	N/A
Drying	N/A	N/A	2 days in oven at 40°C
Moisture Conditioning	N/A	10 days in capillary	8 days in capillary
Total Number of Days	7 days	17 days	10 days

Unconfined compressive strengths of the specimens after the three curing conditions are summarized in Table 3.19. All strengths increased with the increase in cement content. The retained strengths varied from 0.47 for 2% cement to 0.83 for 8% cement. This indicates that all specimens lost strength due to the introduction of moisture. The retained moduli on the same specimens were also all below 0.92.

Table 3.19 – Retained Strengths and Moduli for Soil-Cement Mixes

Additive Content, %	Standard Curing Strength, psi	Moisture-Conditioned Strength, psi	Retained Strength	Standard Curing Modulus, ksi	Moisture-conditioned Modulus, ksi	Retained Modulus
2	186	88	0.47	242	223	0.92
3	508	210	0.41	964	560	0.58
4	528	351	0.66	1087	674	0.62
6	807	541	0.67	1328	932	0.70
8	863	713	0.83	1347	909	0.68

As shown in Figure 3.10, the indirect tensile strength after standard curing initially increases. Passed 6% cement, the indirect tensile strength is constant.

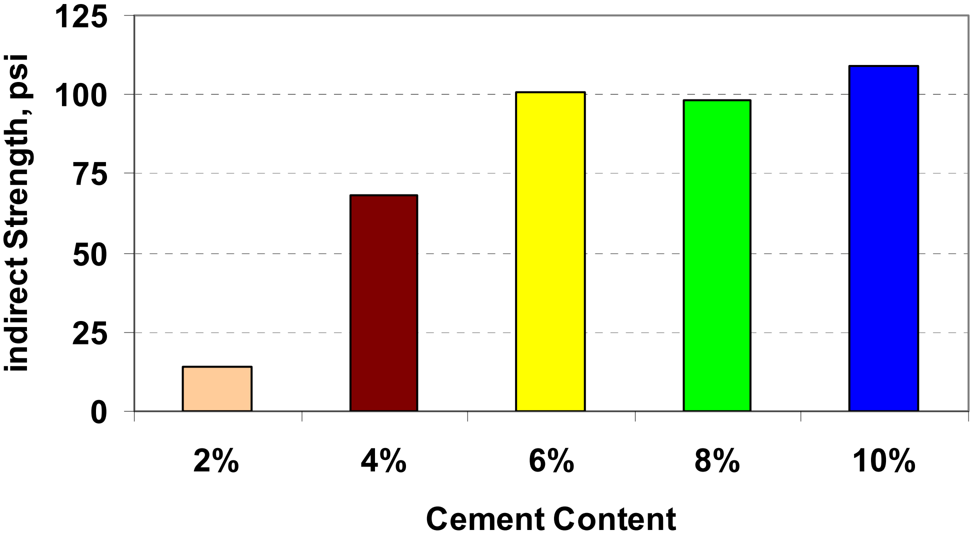


Figure 3.10 – Indirect Tensile Strengths for Wichita Falls Sand with Standard Curing

Table 3.20 summarizes the results obtained from resilient modulus tests as well as the FFRC seismic moduli and UCS for the same specimens. This data again confirms that perhaps resilient modulus tests as per AASHTO T-307 on stabilized materials may not be the best process. The AASHTO T-307 equipment and test protocol were recommended for untreated materials A more rigid test frame and more sensitive displacement sensors are needed to obtain accurate resilient moduli for stabilized materials.

Rutting parameters from permanent deformation tests for specimens after standard curing and moisture conditioning are shown in Table 3.21 for the same cement contents attempted for the resilient modulus tests. The resilient and permanent strains were quite small.

Results from tests conducted on the TST specimens are shown in Table 3.22. Final dielectric constants were below 10 for all Wichita Falls specimens except for specimen with 2% cement. The initial modulus was similar for all mixes since the hydration had not begun. After two days of curing the specimen in the oven, the maximum modulus was much higher than the initial ones

Table 3.20 – Resilient Modulus Results for Soil-Cement Specimens

Parameter	Cement Content and Curing Method							
	2%		4%		6%		8%	
	STD	Moist. Cond.	STD	Moist. Cond.	STD	Moist. Cond.	STD	Moist. Cond.
Representative Resilient Modulus, ksi	74	40	40	184	226	92	87	85
FFRC Seismic Modulus, ksi	292	217	731	654	978	790	1157	841
UCS, psi	208	163	362	319	280	316	234	339

Table 3.21 – Permanent Deformation Parameters for Soil-Cement Specimens

Nonlinear Model Parameters	Cement Content and Curing Method							
	2%		4%		6%		8%	
	STD	Moist. Cond.	STD	Moist. Cond.	STD	Moist. Cond.	STD	Moist. Cond.
Resilient Strain, µstrains	130	38	31	530	190	210	17	96
Permanent Strain, µstrains	110	100	31	630	520	210	17	88

Table 3.22 – Summary of Results from TST Soil-Cement Mixes

Cement Content, %	Final Dielectric Constant	Initial Seismic Modulus, ksi,	Maximum Modulus, ksi*	Residual Modulus, ksi**	Residual/Max. Modulus	Change in Moisture Content, %***
2	12	126	216	34	0.16	8.8
3	11	51	417	323	0.77	6.1
4	4	73	455	211	0.46	5.6
6	5	128	734	305	0.42	4.3
8	4	96	831	789	0.95	1.9

** Maximum modulus is defined as the seismic modulus after two days of oven curing*
*** Residual modulus is defined as the average seismic modulus of the last 3 days of TST curing*
****Change in moisture content is defined as the difference in moisture content between day 10 and 2(after oven)*

for the 2% cement. For the other cement concentrations the maximum moduli were similar to the initial moduli, perhaps due to loss of moisture that could have been used for hydration. The residual modulus after eight days of saturation was significantly lower than maximum modulus for the mix with 2% cement. But it increased for the higher cement contents.

Based on the initial results of the laboratory tests, the amount of stabilizer for Wichita Falls that provides adequate strength was somewhere between 2% and 4% cement content to achieve the desired compressive strength of 300 psi with a seismic modulus of about 200 ksi. A 3% cement seemed to be adequate to provide stabilization for this material. For that reason, more tests were conducted for soil-cement specimens with 3% cement. At this cement content, the retained strength and retained moduli were lower than desirable but the strength and modulus after moisture conditioning were still adequate for a subgrade.

The average variations in moisture content with time for the moisture-conditioned and TST specimens are presented on Figure 3.11. During the first seven days, all moisture-cured specimens lost some moisture, even though they were placed in a 100% humidity room. As the cement content decreased, the loss of moisture was more evident. As a reminder, the first seven days of the moisture content for the moisture conditioned specimens were similar to the standard cured specimens as per Tex-120-E. For the last ten days that these specimens were subjected to capillary moisture, a further loss of moisture was observed in all cases except for specimens with 3% cement. The two-day drying associated with the TST method seemed to allow more absorption of moisture during capillary saturation, especially for the low concentrations of cement.

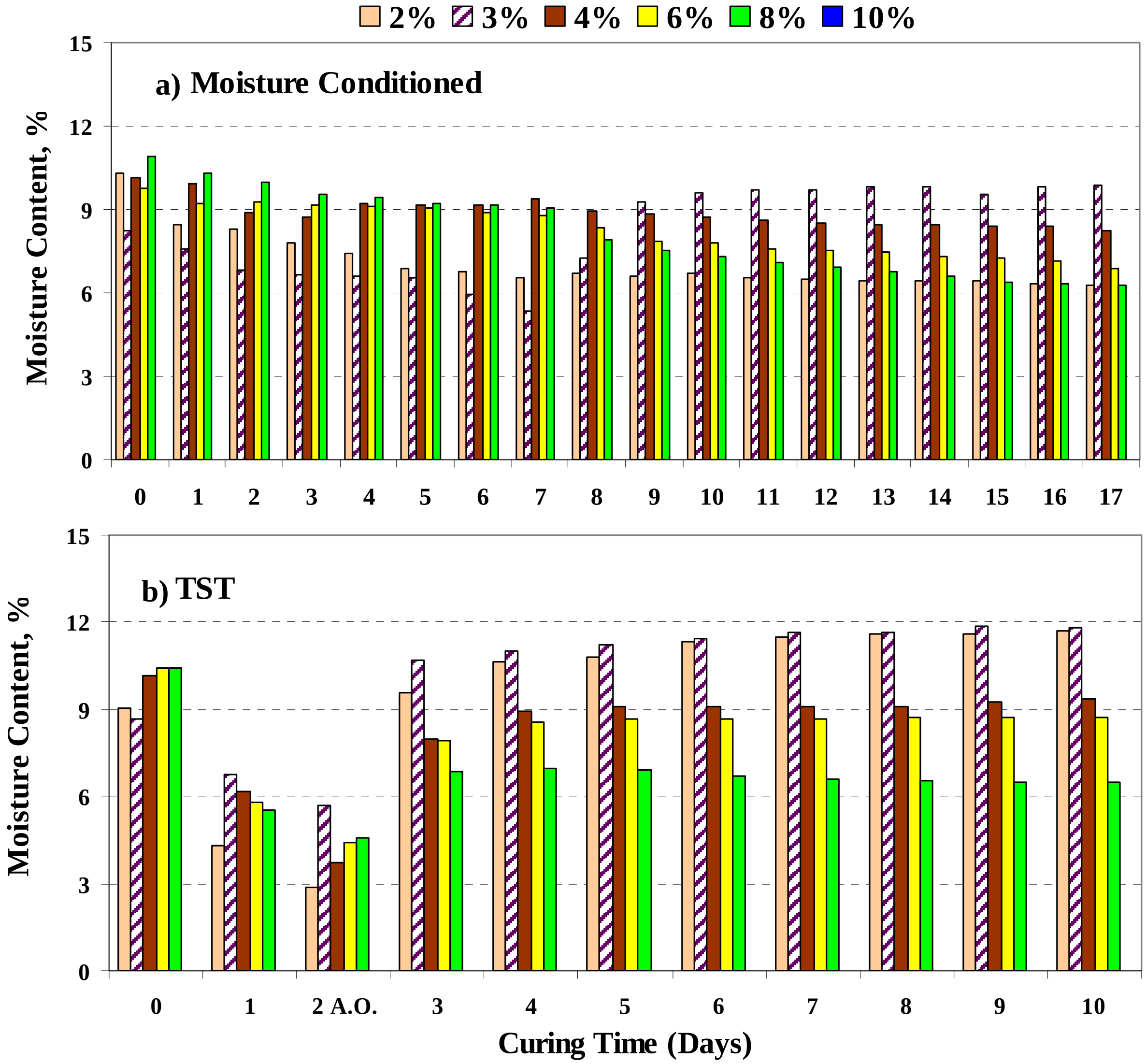


Figure 3.11 – Variation in Moisture Content for Wichita Falls Sand Specimens

The FFRC moduli with time for the same specimens are shown in Figure 3.12. Moduli for cement contents above 4% do not seem to be impacted by the moisture conditioning. In general, the moduli from the moisture-conditioned specimens are greater than those than the TST specimens at the same cement content, simply because of the lack of moisture in the first two days of curing for the TST specimens.

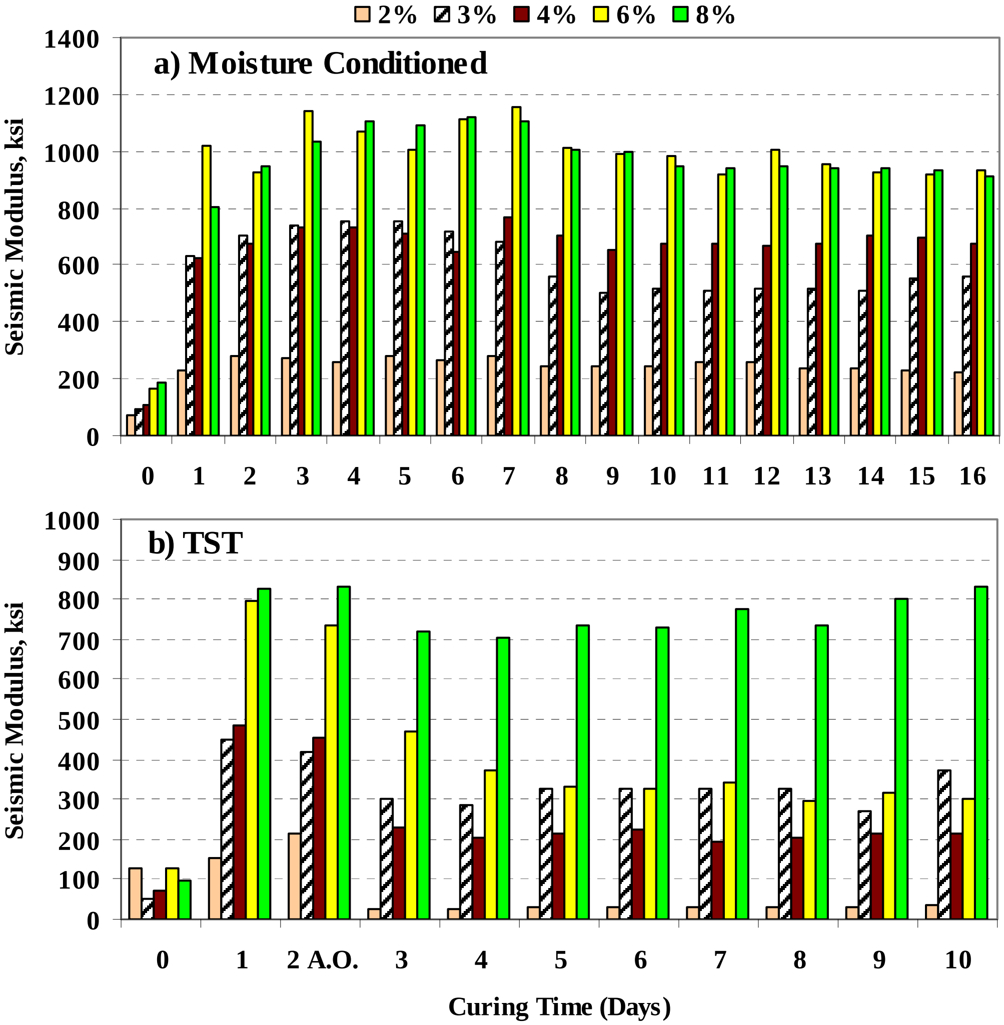


Figure 3.12 – FFRC Seismic Modulus Results for Wichita Falls Soil–Cement Specimens

RESULTS FROM BRYAN SAND SPECIMENS

To determine the minimum amount of Type C fly-ash necessary for stabilization, the Bryan sand material was tested as per Tex-127-E. Due to limited amount of material that was available, only 3% and 6% fly ash contents were studied. The optimum moisture contents for the raw material and for fly-ash content of 6% were determined. The optimum moisture and the maximum dry unit weight for these stabilizer contents and for raw material are shown in Table 3.23. Since these values were very similar, the values for 3% were assumed as equal as those obtained for 6%. The tests performed to determine the appropriateness of stabilizer and its content three different curing methods were applied: Standard (STD), Dry, and Tube Suction Test (TST), as summarized in Table 3.5.

Table 3.23 – Moisture-Density Characteristics

Soil Type	Fly Ash Content, %	Optimum Moisture Content, %	Max. Dry Unit Weight (pcf)
Bryan Sand	0	10.0	121.8
	3*	9.5	125.1
	6	9.5	125.1

** Values from 6% Fly Ash adopted*

The unconfined compressive strengths of the specimens after standard and dry curing processes are summarized in Table 3.24. All strengths increased with the increase in fly-ash content. However, UCS values under standard curing were much smaller than the recommended value of 150 psi. Significant differences between the dry and standard curing strengths exhibits the negative impact of moisture conditioning on this material. The retained strengths and moduli were less than 20%.

Table 3.24 – Retained Strengths and Moduli for Soil-Fly Ash Mixes

Additive Content, %	Dry Curing Strength, psi	Standard Curing Strength, psi	Retained Strength	Dry Curing Modulus, ksi	Standard Curing Modulus, ksi	Retained Modulus
3	209	18	0.09	271	3	0.01
6	248	48	0.19	317	36	0.11

According to the outcome of the previous sections, where the resilient and permanent strains were quite small on all stabilized specimens, permanent deformation tests were not carried out to conserve the raw materials. The resilient modulus tests were also not carried out since previous tests indicated that the resilient modulus tests are not the most appropriate tests for stabilized materials.

The average variations in moisture content for all Bryan sand specimens for the standard and the TST process are presented in Figure 3.13. Under the standard-cured condition, all specimens lost little moisture during the seven day curing process. After moisture conditioning, the final moisture contents were greater than initial values, especially for specimens with 3% fly-ash, where the final moisture content was 5% greater than initial. For the case of the TST specimens, both stabilizer contents presented similar results. A drop in final moisture content of 2.7% and 2.4% was observed for dry-cured specimens and an increase in final moisture content of 1.3% and 1.6% was identified for TST specimens when compared to initial and for 3% and 6% fly-ash contents, correspondingly. In the latter case, the two-day drying associated with the TST method seemed to allow less absorption of moisture during capillary saturation, when compared to standard curing.

The FFRC moduli with time for the two curing processes are shown in Figure 3.14. The significant decline in modulus for both Standard and TST curing methods were observed.

Results from tests conducted on the TST specimens are summarized in Table 3.25. Final dielectric constants were 14 and 7 for all Bryan sand specimens stabilized with 3% and 6% fly-

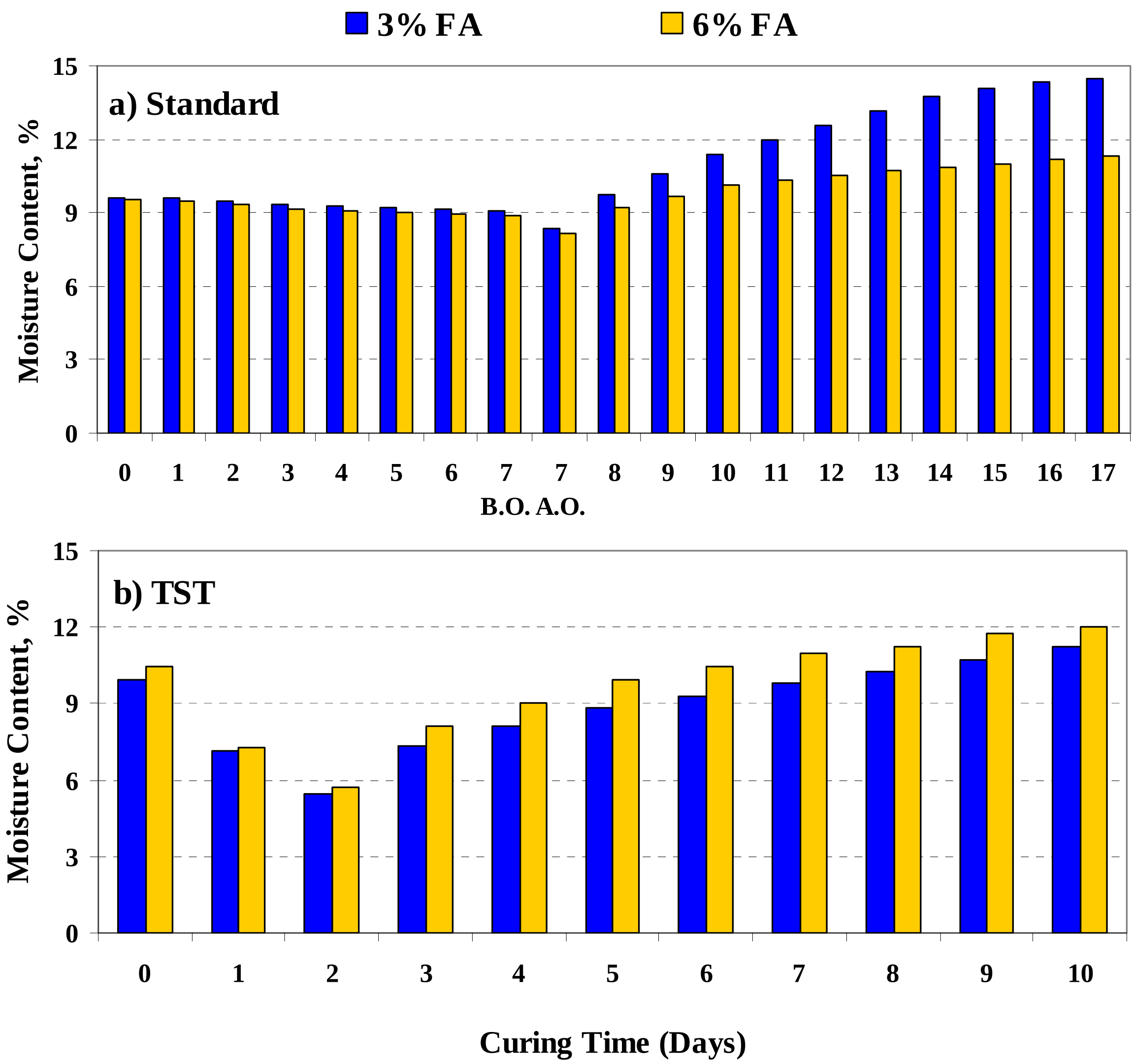


Figure 3.13 – Variation in Moisture Content for Bryan Sand-Fly Ash Specimens
(B.O. = before specimen placed in the oven,
A.O. = after specimen was removed from the oven)

ash, respectively. The initial modulus was similar for all mixes since the hydration had not begun. After two days of curing the specimen in the oven, the maximum modulus was in both cases close to 300 ksi. The residual modulus after eight days of saturation was significantly lower than maximum modulus and in both cases was around 10 ksi. This drop in modulus results in a very small ratio of residual vs. maximum modulus, demonstrating the big impact of moisture conditioning for this material. The change in moisture content between moisture conditioning and curing was in both cases close to 6%.

Based on the initial results of the laboratory tests, fly-ash does not seem to be an appropriate stabilizer for Bryan sand to provide adequate strength under current specifications. Nevertheless, a fly-ash content of 6%was selected for the further evaluation of this mix design of this material.

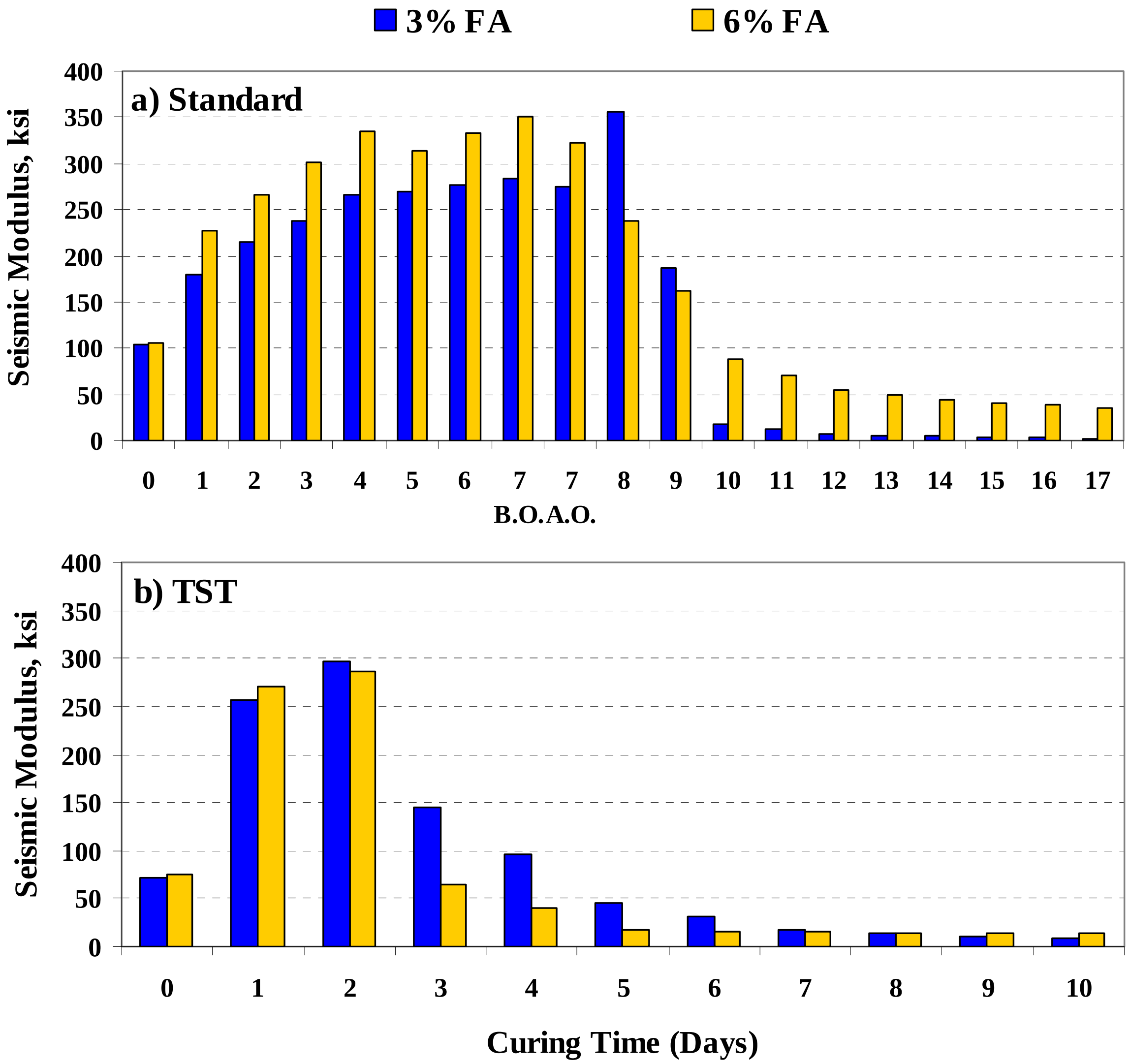


Figure 3.14 – FFRC Seismic Modulus Results for Bryan Sand-Fly Ash Specimens
(B.O. = before specimen placed in the oven,
A.O. = after specimen was removed from the oven)

Table 3.25 – Summary of Results from TST Soil-Fly Ash Mixes

Fly Ash Content, %	Final Dielectric Constant	Initial Seismic Modulus, ksi,	Maximum Modulus, ksi*	Residual Modulus, ksi**	Residual/Max. Modulus	Change in Moisture Content, %***
3	14	71	297	10	0.03	5.8
6	7	75	287	14	0.05	6.3

** Maximum modulus is defined as the seismic modulus after two days of oven curing*
*** Residual modulus is defined as the average seismic modulus of the last 3 days of TST curing*
****Change in moisture content is defined as the difference in moisture content between day 10 and 2(after oven)*

X-RAY DIFFRACTION STUDIES

In order to confirm the stabilization mechanism due to the formation of pozzalonic cementing compounds including Tobermorite, Scolecite and Prehnite, several powder samples were collected from the stabilized soil mixtures and these samples samples subjected to mineralogical analyses by X-Ray Diffraction (XRD). Presence of stabilization compounds support the successful application of chemical treatment of the present soils.

A Siemens D-500 X-Ray diffractometer with Cu Kα radiation (see Figure 3.15) was used. The tests were performed from 5˚ to 80˚ 2θ, with a step size of 0.02˚ and 2 s counting at each step. Typical results for Paris and El Paso clays are shown in Figures 3.16 and 3.17 as examples. The peak corresponding to a certain mineral has been highlighted by the name of the mineral on top of it. New peaks of CSH (Tobermorite), CASH (Scolecite) and CASH (Prehnite) can be observed for the treated material in Figure 3.16b. Similar peaks can be observed in Figures 3.17a and 3.17b for the untreated and treated soil from Paris. Overall, the peaks associated with the natural minerals such as montmorillonite, kaolinite and Illite in the untreated soil have reduced and the new peaks of the CSH and CASH compounds have been formed for the treated materials.

Test results performed on other soils are summarized in Appendix B for various clay minerals and pozzalanic compounds such as CSH and CASH.

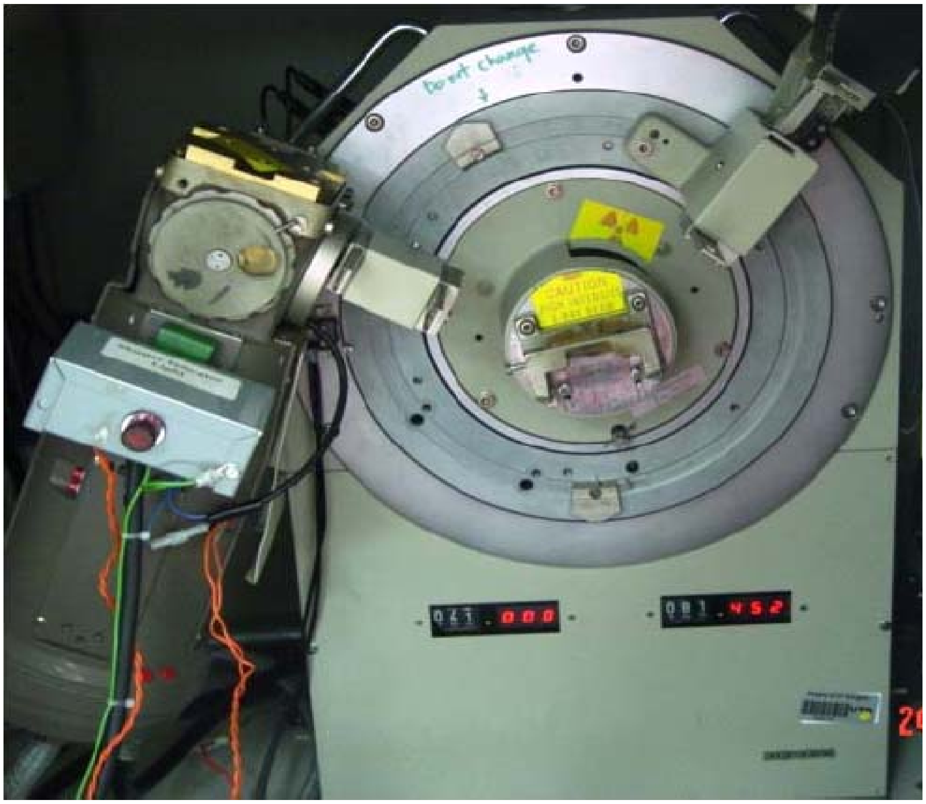


Figure 3.15 – Siemens D-500 X-Ray Diffractometer

CHEMICAL TESTS

Tests performed to quantify the reactive alumina and silica before and after treatment. Test procedures followed are given in Appendix C. The main intent of these tests is to assess the availability of reactive alumina and silica in stabilized soils, whose presence would lead to pozzalonic reactions and formation of pozzalonic compounds.

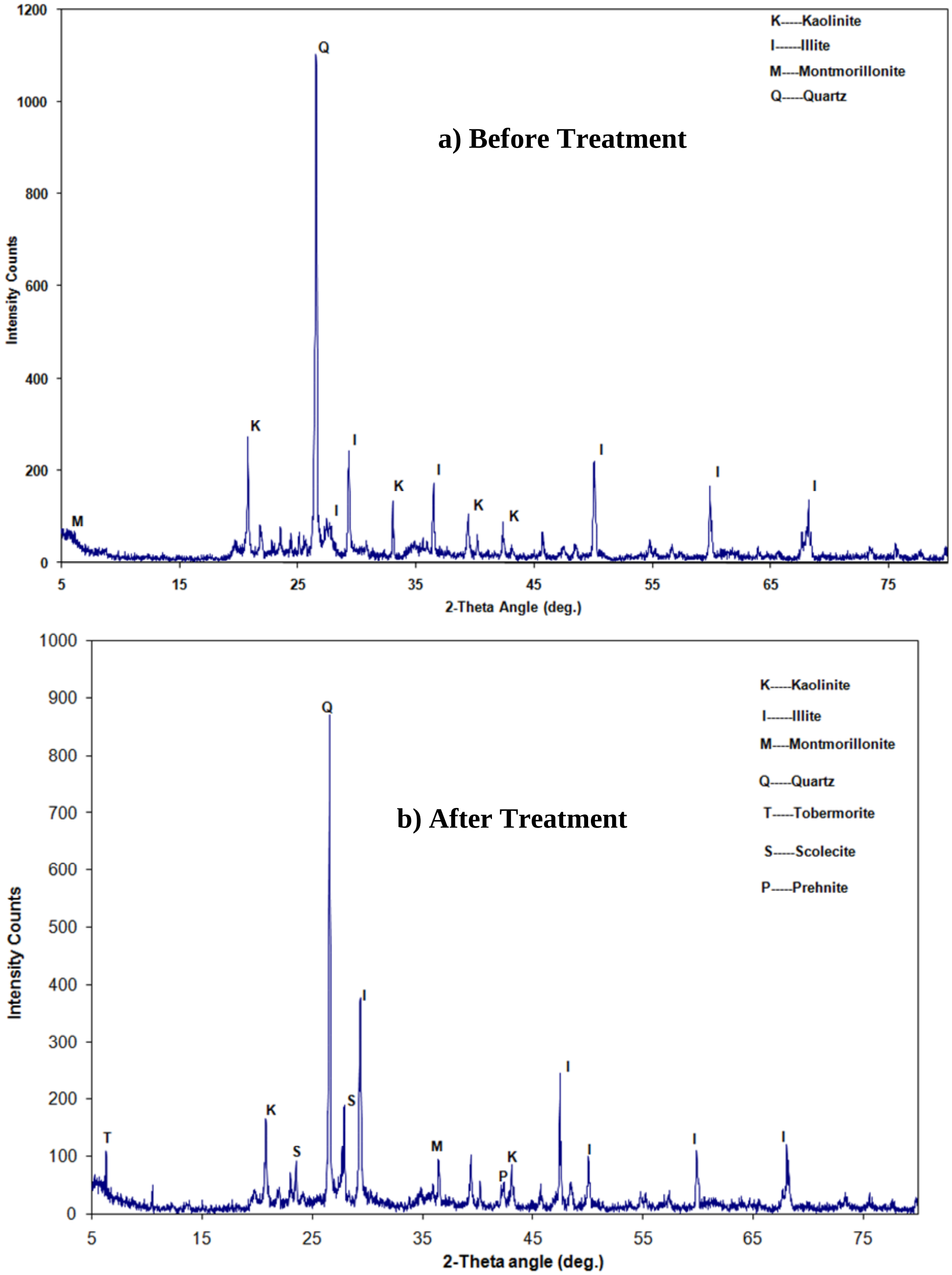


Figure 3.16 – XRD Results of El Paso Soil

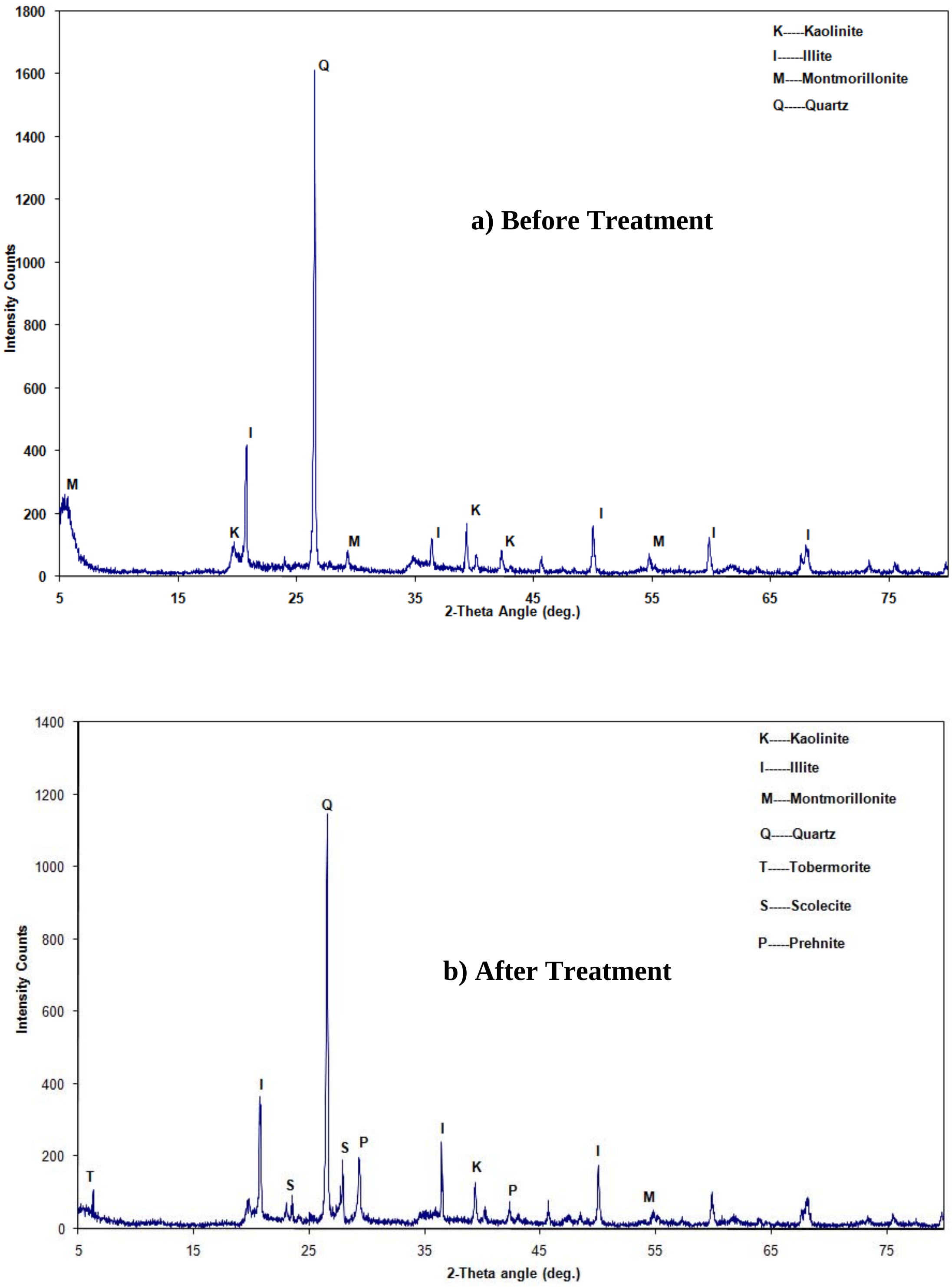


Figure 3.17 – XRD Results of Paris Soil

Table 3.26 summarizes the results for reactive alumina of different soil samples before and after treatment and Table 3.27 presents the same for reactive silica. Increases in both the reactive alumina and silica were observed after the addition of additives. This kind of behavior is expected as alumina and silica of soils dissociates with an increase in pH (Wattanasanticharoen, 2005), which typically occurs during treatment.

Table 3.26 – Changes in Reactive Alumina between Untreated and Treated Soils

Soil Type	Reactive Alumina, ppm	
	Before Treatment	After Treatment
El Paso	60 (8.2)	225 (12.3)
Bryan	200 (8.4)	550 (12.2)
Fort Worth	154 (8.4)	700 (12.1)
Paris	180 (8.4)*	525 (12.3)
Wichita Falls	65 (8.3)	570 (12.4)

** Numbers in parentheses are the pH of the soils*

Table 3.27 – Changes in Reactive Silica between Control and Treated Soils

Soil Type	Reactive Silica, ppm	
	Before Treatment	After Treatment
El Paso	880 (8.2)	765 (12.3)
Bryan	500 (8.4)	497 (12.2)
Fort Worth	540 (8.4)	543 (12.1)
Paris	580 (8.4)	1168(12.3)
Wichita Falls	840 (8.3)	846 (12.4)

** Numbers in parentheses are the pH of the soils*

MODELS FOR DETERMINING DOMINATING CLAY MINERALS

Table 3.28 presents various results of the chemical tests such as *Cation Exchange Capacity (CEC), Specific Surface Area (SSA) and Total Potassium (TP)* conducted on 20 different soils from various districts in Texas. Once these three properties were obtained for each soil, the clay minerals present in the soil were quantified by assuming that a given chemical property of a soil is contributed by the summation of the percentage of each clay mineral and the related same property of that particular mineral. Based on this, all three major clay minerals are determined which are presented in Table 3.29.

An attempt is made to develop a regression model between percent montmorillonite mineral (%M) as functions of chemical properties (SSA and CEC) measured in this research. This expression is presented in the following equations:

$$\%M = -2.87 + 0.08 \times SSA + 0.26 \times CEC \tag{3.4}$$

The percent Illite (%I) can be measured using the following theoretical equation:

$$\%I = \left[\frac{TP}{6}\right] \times 100 \tag{3.5}$$

where TP is total potassium.

Once the percent values of montmorillonite and illite are determined, the percent kaolinite (%k) can be determined using the following expression:

$$\%K = 100 - \%I - \%M \tag{3.6}$$

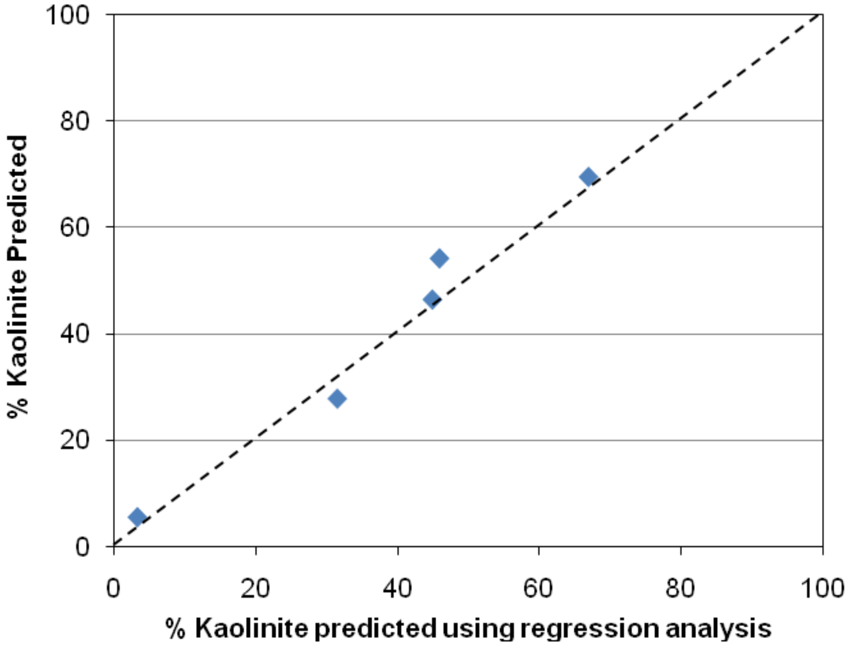
Table 3.28 – Values of SSA, CEC and TP for Several Texas Soil Samples

Soil No.	Soil Type	CEC, meq/100 gm	SSA, m^2/gm	TP, %
1	Amarillo	66	175	0.97
2	Arlington	121	324	0.77
3	Atlanta	134	460	1.22
4	Austin	101	288	1.74
5	Bryan	77	205	1.36
6	Bryan Silt	88.5	210	1.12
7	Dewitt County	63	295	1.38
8	El Paso	57	161	3.75
9	Fort Worth	117	314	0.98
10	Gate	117.8	265	0.80
11	Houston	76	236	1.76
12	Jackson Ct. # 1	125	355	0.83
13	Jackson Ct. # 3	75	240	0.95
14	Keller	71	133	1.10
15	Paris	133	431	0.79
16	Pharr A	104.0	306	1.55
17	Pharr B	76.1	132	1.65
18	San Antonio	96	269	1.10
19	Seymour	58	158	3.53
20	Victoria Ct.	109	303	1.50

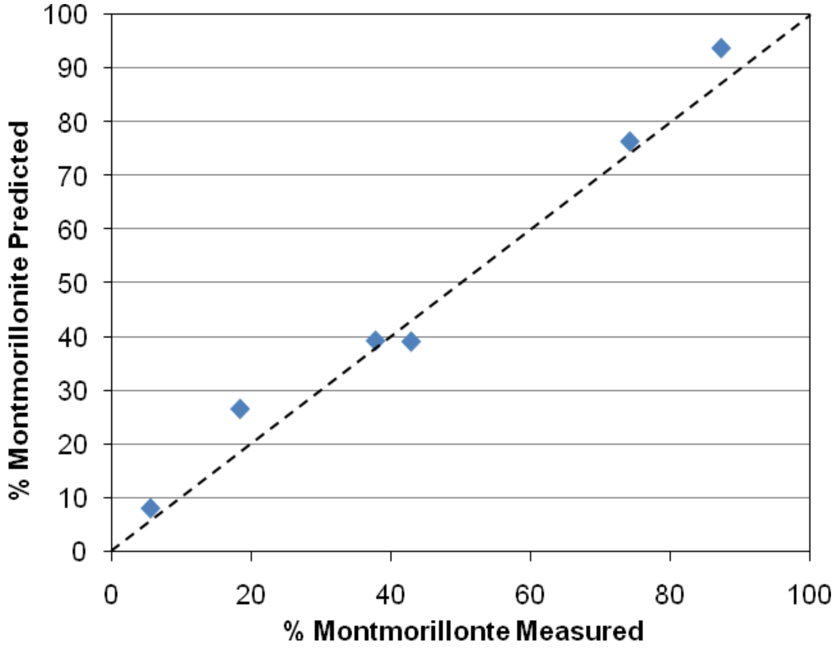
Table 3.29 – Percent Mineral Information of Several Texas Soil Samples

Soil No.	Soil Type	% Illite	% Kaolinite	% Montmorillonite
1	Amarillo	6.17	51.61	32.22
2	Arlington	2.83	24.41	62.76
3	Atlanta	0.33	5.42	74.25
4	Austin	28.96	17.67	53.37
5	Bryan	22.72	39.93	37.35
6	Bryan Silt	18.67	42.37	38.96
7	Dewitt Ct.	22.92	21.51	55.58
8	El Paso	62.50	14.18	23.32
9	Fort Worth	16.25	23.38	60.37
10	Gate	13.33	35.89	50.77
11	Houston	29.38	27.74	42.88
12	Jackson Ct. # 1	13.75	28.93	57.32
13	Jackson Ct. # 3	15.83	46.40	37.77
14	Keller	18.33	62.16	19.51
15	Paris	13.13	16.65	70.22
16	Pharr A	25.83	26.40	47.77
17	Pharr B	27.50	54.16	18.34
18	San Antonio	18.33	39.38	42.28
19	Seymour	58.75	21.92	19.33
20	Victoria Ct.	25.00	27.65	47.35

An attempt was made to validate the present correlations by using them to predict the clay minerals of six different artificial clay mixtures with known clay mineralogy. These comparisons can be seen in Figure 3.18. The predicted values compared well with the measured clay mineral fractions. Hence, the above correlations are recommended for simple and practical use of estimating the mineral fractions in a soil via three chemical property measurements.



a) Kaolinite Mineral



b) Montmorillonite Mineral

Figure 3.18 – Comparisons between Added and Predicted % of the Minerals Using Regression Analysis

CHAPTER FOUR – ACCELERATED CURING AND MOISTURE CONDITIONING

INTRODUCTION

Since the main objective of this project is to accelerate the laboratory mix design, different methods of moisture conditioning were considered. Comparing the time needed to get similar results with different methods can direct this research to propose a new method of curing that supplants the current time consuming methods. In addition, the simplicity and applicability of the tests was considered for selecting the best alternative. In this chapter different methods of compaction used in this study are also discussed.

COMPACTION METHODS

The method used to compact a specimen affects the characteristic of the specimen. Aside from representing the field conditions, a proper specimen is the one which can be cured and moisture conditioned in optimum time and without experiencing severe cracking. Variation in density is another concern with the method of compaction. The more constant the density along the length of specimen is, the more reliable the result will be.

For the purpose of this project, two compaction techniques were used in the laboratory to prepare the clay specimens. The compacted specimens were then subjected to curing and moisture conditioning. A brief description of the two compaction techniques attempted and problems encountered with each of them follows.

The current TxDOT practice to prepare subgrade specimens is based on test specification Tex-114-E. A kneading compactor is used to compact specimen in this method. The specimens are nominally 4 in. in diameter by 6 in. in height, compacted in four lifts. The number of blows per lift is specified in Tex-114-E.

Several problems were observed during the curing and moisture conditioning processes for kneading compacted specimens. During curing process, many cracks were observed for the high-PI clay specimens, as illustrated in Figure 3.9. Some specimens separated at the interface of the compaction lifts. When the lifts were not well bonded, the capillary moisture conditioning was interrupted at the interface of the lifts. Since the density is not explicitly controlled, several specimens prepared under the same compactive effort resulted in somewhat different densities. To overcome these shortcomings, the static compaction method was investigated.

Static Compaction

A static compactor, suggested in the AASHTO T-307 for preparing fine-grained soil specimens, was used. Figure 4.1 illustrates the step-by-step procedure for static compaction. Detailed protocol is provided in Appendix C. In this method, the exact amount of required material is weighed to prepare a specimen (Step 1). This amount is calculated based on desired dry density, degree of compaction and the moisture content. A metal mold is filled with the desired material. Two cylindrical blocks are placed on the top and bottom is used to compact the specimen (Steps 2, 3, and 4). The specimen is compacted when the top block becomes flush within the mold (Step 5). After waiting 1 minute in this condition, the mold is rotated and the top block is replaced with a thicker one (Step 6). To get the desired height the load is applied again on top of this block (Step 7). At the end the specimen is extruded from the mold by using a hydraulic jack (Step 8).

With this method, it is easy to make specimens that are constant in height and density. One of the concerns with this method is the variation of density along the length of specimen. A comprehensive study was carried out in Project 0-5430 (see Wanyan et al., 2008) to develop a process for static compaction of clays in one layer with small variation in density along the length of specimen. In that study, density changes were monitored in several layers along the length of clay specimens by cutting the compacted specimens in five different layers of similar heights. Three different solid blocks were used to prepare the specimens. The order in which these solid blocks are used impacts the uniformity of density along the length. Figure 4.2 illustrates the variations in density for different compaction sequences. In that figure, Block 1 is 0.5 in. thick, Block 2 is 2.45 in. thick, and Block 3 is 5.6 in. thick. Trials 5, 6 and 7 resulted in the least variation in density along different layers. Compaction process from trials 5 and 6 was recommended in Project 0-5430 and as a result, specimens for this project were prepared according to that process.

ALTERNATIVE MOISTURE CONDITIONING METHODS

In Tex-121-E protocol, the prepared specimen is cured at room temperature for seven days and is dried for at least 6 hrs in an oven at a temperature not to exceed 60°C (140°F) or until one-third to one-half of the molding moisture has been removed. The specimen is then subjected to ten days of capillary moisture conditioning. The main concerns with this method are the seventeen days required to cure the specimen, and non-uniform distribution of moisture within the specimen. In many instances the middle of the clayey specimens is not moisture conditioned. The height of the penetration of moisture within the specimen is typically dependent on the permeability of the material. Often the moisture will not pass the interface of the compaction lifts of the specimen. Many specimens were also debonded at the interface of the lifts, impacting the results from the strength tests.

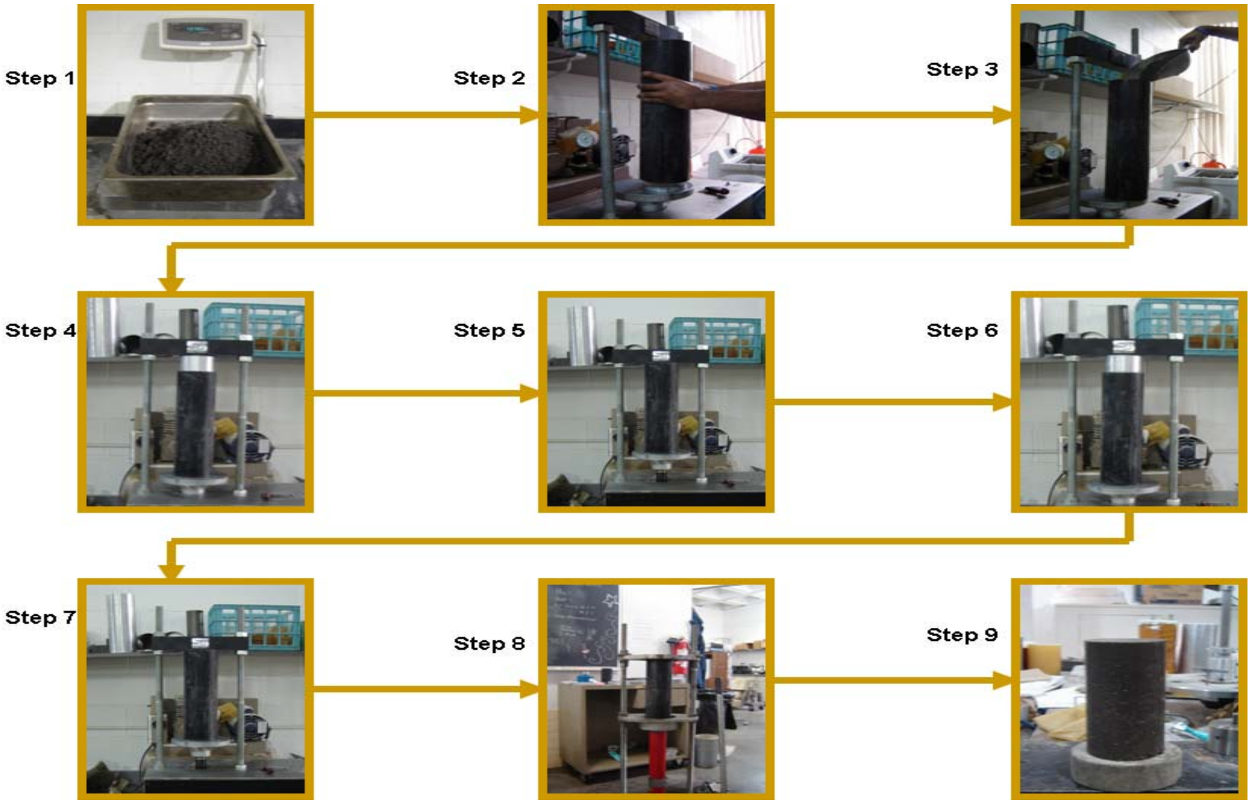


Figure 4.1 – Static Compaction Procedure

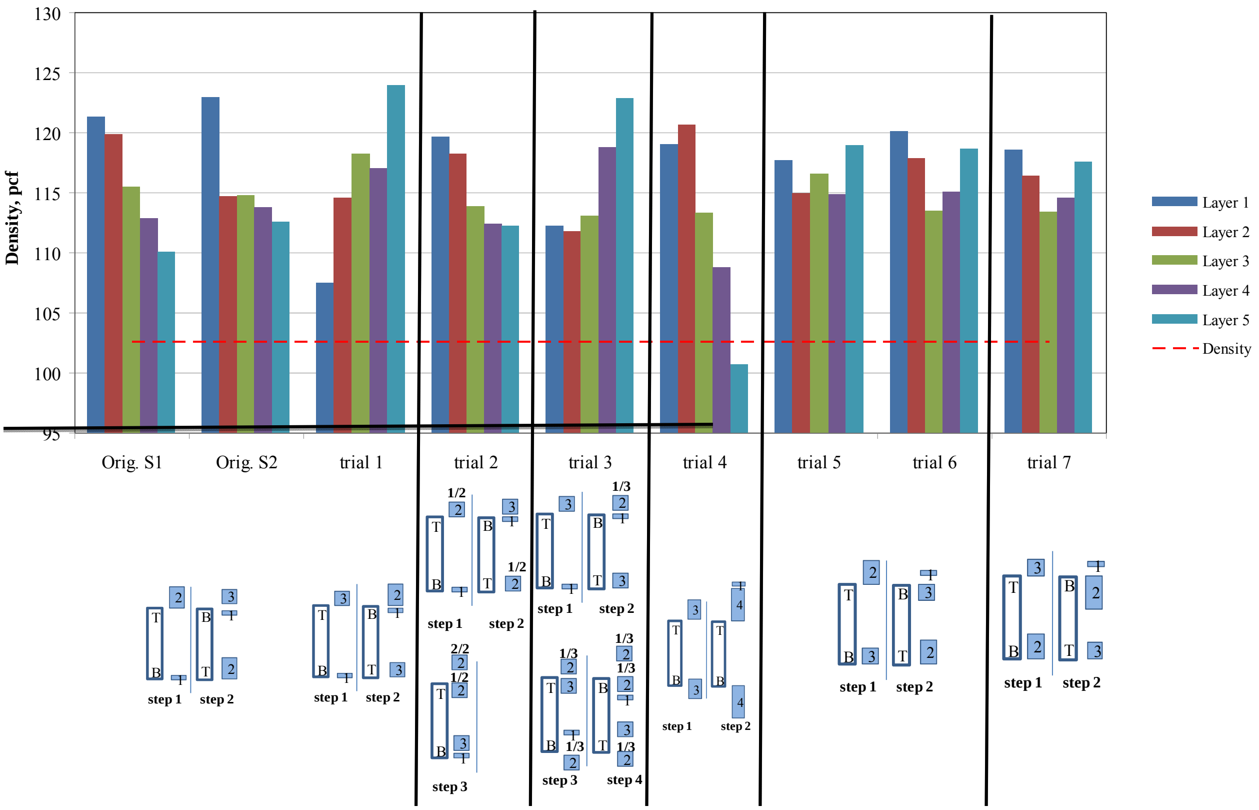


Figure 4.2 – Variation in Density Using Different Compaction Sequences

Tube Suction Protocol

The moisture conditioning associated with the Tube Suction Test (TST) is described in proposed test specification Tex-144-E. In this test, a specimen is prepared at the OMC and placed in a 140ºF (40ºC) oven for 2 days. The specimen is allowed to cool down at room temperature, and then placed under a capillary condition with porous stones at the top and the bottom for 8 days. The concerns with the TST moisture conditioning of fine-grained soil is similar to those for Tex-121-E, except that the duration of the specimen preparation is ten days as opposed to 17 days.

Backpressure Conditioning Protocol

In this process, a specimen prepared at the OMC is subjected to a confining pressure and water is forced through the specimen with pressure. The confining pressure is applied to prevent the water flow from the side of the specimen and to maintain the integrity of the specimen during backpressure. The greater the difference between the confining pressure and the backpressure is, the faster the specimen will saturate. However, significant difference between the confining and backpressure results in excessive hydraulic gradient which may damage the specimen. Since this test provides a means to accelerate the moisture conditioning and provides information about the hydraulic conductivity of the specimen, it is considered as one of the alternative moisture conditioning tests. A detailed protocol is included in Appendix C.

Figure 4.3 illustrates the set up required for this test. The test device, which is very similar to a permeameter, consists of several components. A pedestal that supports the specimen contains ports for applying either vacuum or pressure to the specimen. The middle of the pedestal contains a hole that allows for the drainage of water from the specimen. A valve controls the flow of the water out of the specimen.

The second component of the device is a confining chamber retrofitted with a membrane. The confining pressure is applied to the specimen through this membrane. The confining chamber is also retrofitted with a port where the vacuum or pressure can be applied. Vacuum is used to evacuate air between the membrane and cell during specimen assembly.

The third component consists of a top cap retrofitted with a graduated cylinder. The top cap assembly has two functions, first to secure the specimen assembly in place and second to introduce moisture to the specimen through the graduated cylinder. Finally, a cap that is connected to a pressure source is needed to apply the backpressure to the specimen.

Two porous stones and filter papers are placed on top and bottom of the specimen during the assembly. A 4 in. (100 mm) graduated cylinder is used to monitor the amount of discharged water through the specimen. A specimen (4 in. in diameter and 6 in. in height) is prepared at the OMC associated with the stabilized material. The specimen is dried in a 104ºF (40ºC) oven for up to two days. In the case of the high-PI clayey materials, it is recommended to wrap the specimen in cellophane wrap. Several holes are pricked in the wrap to allow the evaporated moisture to migrate away from the specimen.

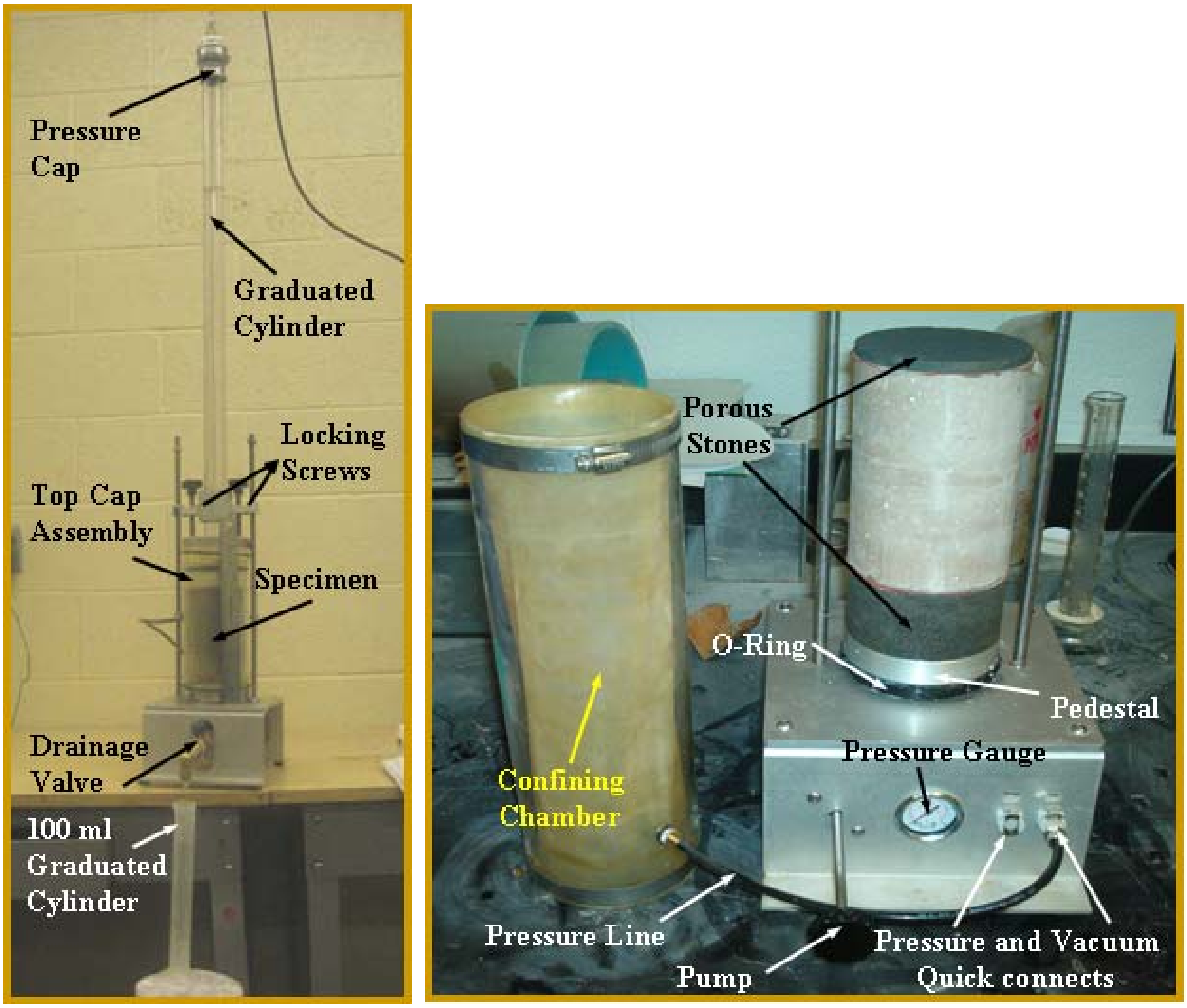
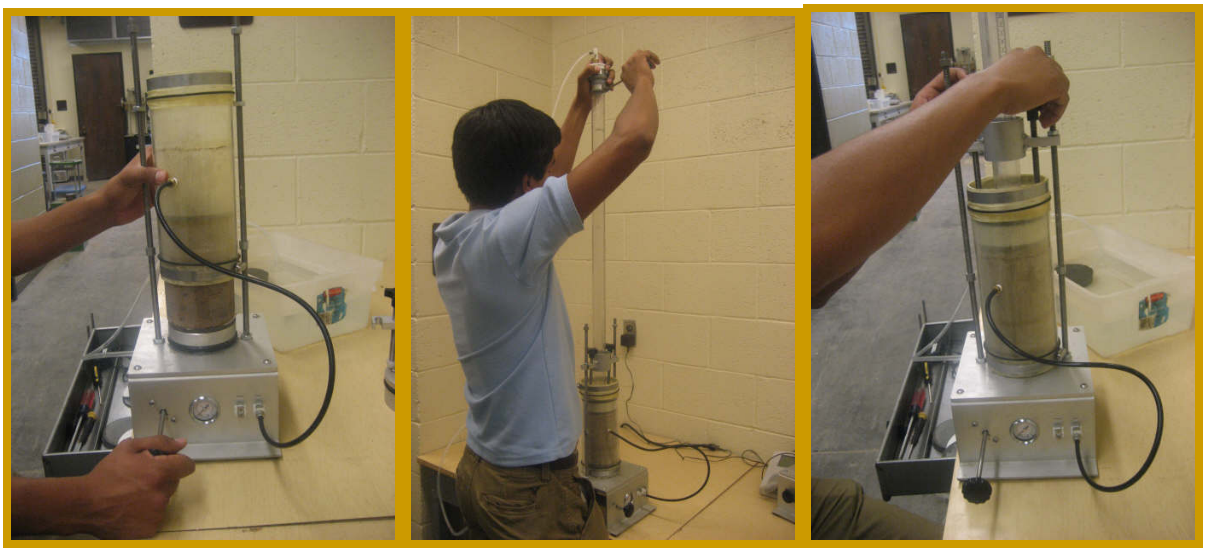


Figure 4.3 – Back Pressure Test Apparatus

Figure 4.4 shows the set-up process. The specimen is placed on top of a porous stone that is centered on the pedestal plate described above. To prevent clogging the porous stone, a filter paper is placed between the bottom porous stones and specimen. Once the specimen is centered, a porous stone and filter paper are placed on top of the specimen. The acrylic confining chamber retrofitted with a membrane is then subjected to vacuum to evacuate the air between the membrane and the cell (Step 1). The cell is securely placed around the o-ring in the base pedestal. Next, the upper cap and graduated cylinder assembly is securely placed on top of top porous stone and clamped tightly to hold the specimen assembly together (Step 2). The membrane is pressurized with the pump incorporated in the base assembly. The graduated cylinder is filled with water, retrofitted with the plastic pressure cap, and backpressure is applied and maintained from the top (Step 3).



Step 1 Step 2 Step 3

Figure 4.4 – Back Pressure Test Setup

The duration of the moisture conditioning depends on the permeability of the material. The higher the permeability is, the faster the specimen will saturate. The moisture content at the saturated state can be estimated from:

$$MC_{sat} = \frac{\gamma_w (1 + MC_i)}{\gamma_{Ti}} - \frac{1}{G_S} \tag{4.1}$$

where:

MC_{sat} = Theoretical moisture content of specimen at saturation
MC_i = Initial moisture content of specimen
γ_{Ti} = Initial total (wet) unit weight of soil
γ_w = Unit weight of water
G_S = Specific Gravity of soil

Figure 4.5 shows the progression of moisture within the specimen, and Figure 4.6 demonstrates the typical variation in moisture content with time. The amount of water absorbed by the specimen can be readily determined by periodically monitoring the amount of water loss in the graduated cylinder. The water discharged through the specimen is collected in a graduated pipette. The specimen is saturated when the water inflow through the specimen becomes equal to the water discharged from the specimen. At this time, the permeability of the specimen can be measured by measuring the volume of the outflow for a certain time (say 1 hr). The pH of the outflow can be also measured to assess the leachability of the stabilizing agent. Upon completion of the test (usually 24 hours), the drainage valve is closed, the pressures are released, and the top assembly is removed. The remaining water in the cylinder is collected and weighed. The specimen is removed, and the modulus, dielectric constant and the UCS of the conditioned specimen are measured. These parameters are used to compare different methods of moisture conditioning.

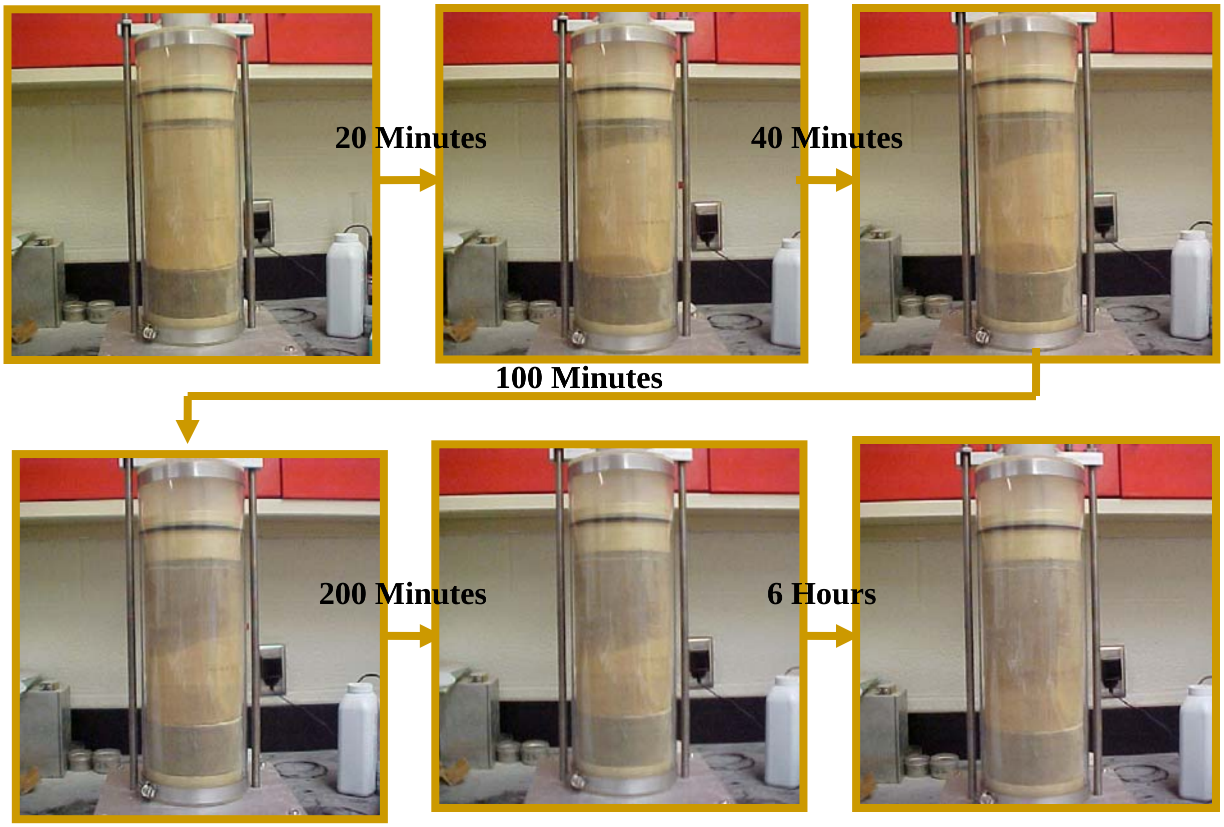


Figure 4.5 – Example of Moisture Conditioning Progress for Back Pressure Test

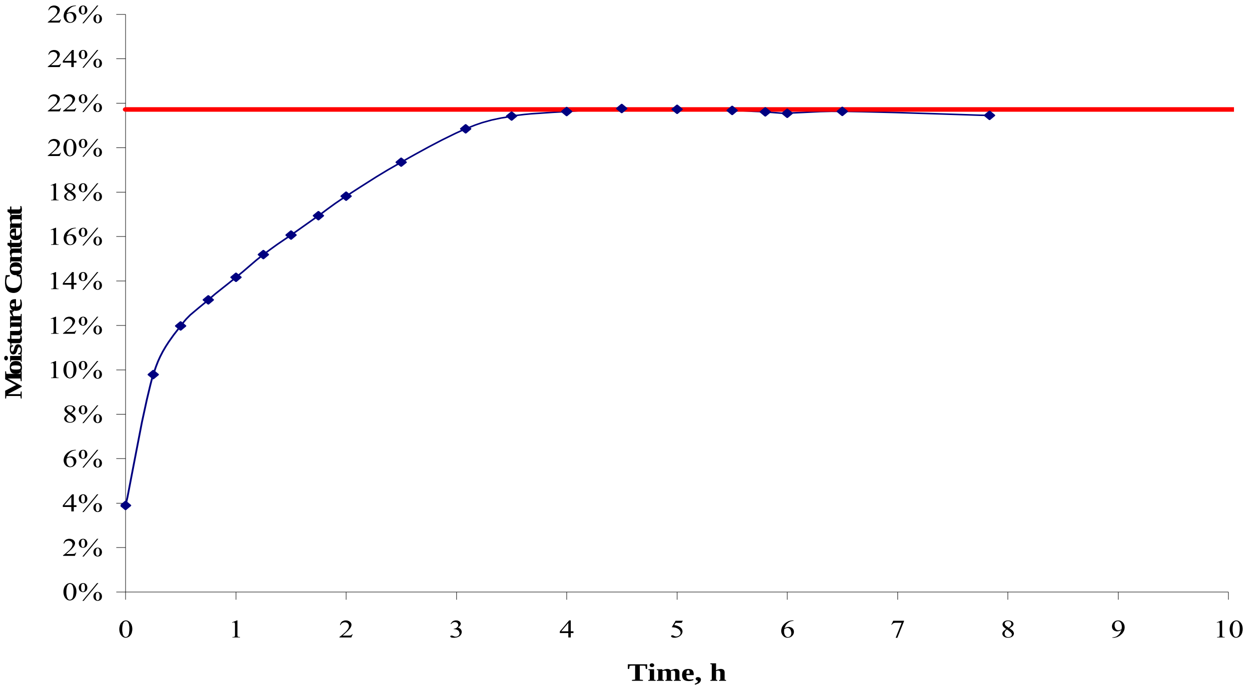


Figure 4.6 – Variation in Moisture Content with Time for Back Pressure Moisture Conditioning

Submergence Protocol

Submergence moisture conditioning is applied on a specimen prepared at OMC. After the specimen is dried up for two days in an oven at 104ºF (40ºC), it is submerged. Submergence periods of 4 hrs and 24 hrs were studied. The equipment necessary is very simple, essentially a standard plastic 5-gallon bucket filled with distilled water to a height 8 in. from the bottom. The amount of water in the bucket was arbitrarily standardized so that the change in pH of water between different specimens can be measured in a relative term.

Figure 4.7 illustrates the submerge test for two specimens. The pH change of the water and the change in the weight of specimens are monitored with time until the theoretical saturation as reflected in Equation 4.1 is reached. At the end of moisture conditioning, the modulus, dielectric constant and the UCS of the specimen are measured, if possible. Since the specimen is absorbing water from all its sides, the moisture conditioning process is rather rapid. However, in many cases, the specimen disintegrates during the process as shown in Figure 4.8. This test does not seem to be appropriate for clays stabilized with lime, as judged by the damage produced in many of the clay specimens.

Vacuum Conditioning Protocol

The vacuum conditioning method is similar to the backpressure conditioning explained above. The main difference between these two methods is the direction of water flow. In this method the moisture conditioning occurs from the bottom to the top. The higher the vacuum pressure is, the faster the moisture conditioning will take place. However, as mentioned in the backpressure protocol, excessive vacuum pressure may generate excessive hydraulic gradient which may damage the specimen during conditioning.

Figure 4.7 – Submerge Tests

Figure 4.8 – Submerge Test Specimens after Test

A detailed protocol is provided in Appendix C. The sample preparation and specimen assembly for this method are identical to the backpressure protocol. Figure 4.9 shows the vacuum conditioning setup. The setup is also very similar to the back pressure method. The major differences are that the top cap assembly is connected to a vacuum line instead of a pressure line, and that the water is introduced to the specimen through the drainage port in the bottom assembly. In this research water was allowed to rise up to 40 mm in the graduated cylinder of the top cap.

EVALUATION OF MOISTURE CONDITIONING PROTOCOLS

The backpressure and confining pressure play an important role in the outcome of the specimens conditioned with the backpressure protocol. A large number of tests were performed on the Bryan and El Paso clayey materials to propose the optimum values of backpressure and confining pressure. After numerous tests, a confining pressure of 10 psi was selected to eliminate the migration of moisture along the sides of the specimen.

Figure 4.10 illustrates the variation in absorbed moisture with time for three different backpressures for El Paso material with 6% additive. Similar results were obtained for the Bryan clayey material with 8% additive. The higher the backpressure is, the faster the specimen reaches the full saturation. The relevant information about the time to saturation for different confining pressures is summarized in Table 4.1. The time to saturation varies from 30 hrs for no backpressure to 4 hrs for a backpressure of 6 psi. By way of comparison, about 72 hrs required to saturate the specimen using the Tube Suction protocol. The pH of the specimens after

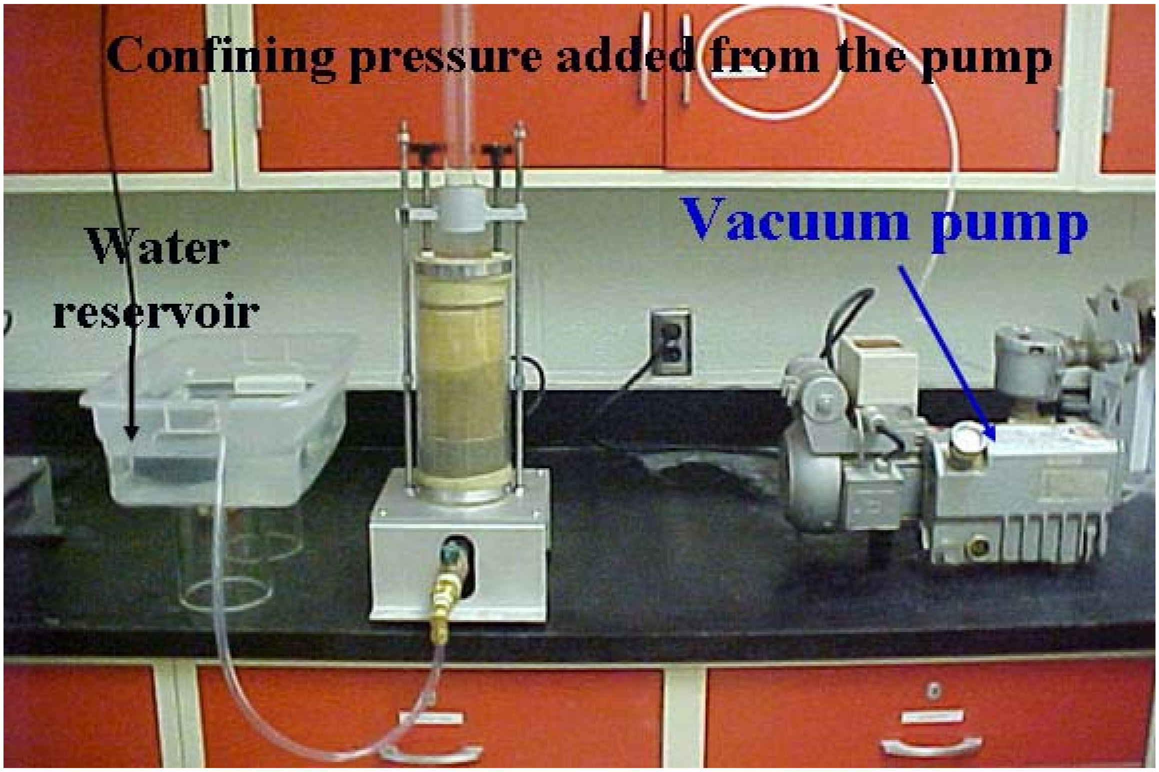


Figure 4.9 – Vacuum Test Setup

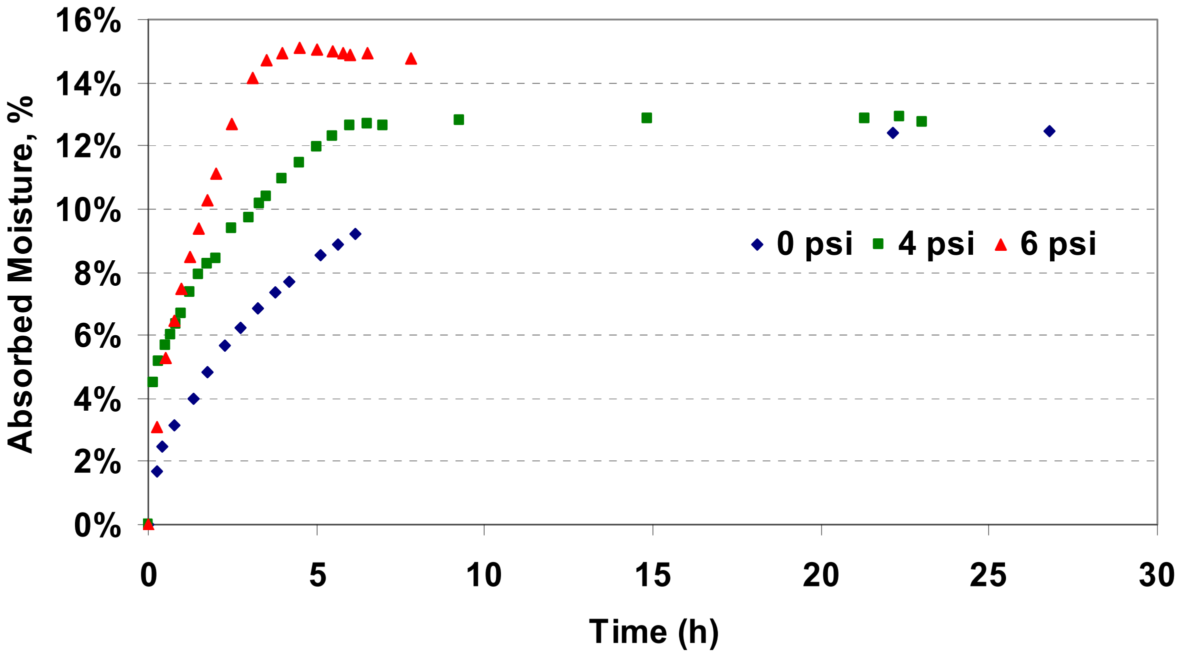


Figure 4.10 – Variations in Moisture Content with Time for Different Back Pressures

Table 4.1 – Parameters Associated with Back Pressure Moisture Conditioning

Backpressure, psi	Initial MC*, %	Final MC,%	Final pH	Time to Saturation, hr
0	3.3	16.1	8.4	30
4	4.6	16.8	12.1	7
6	3.9	18.1	12.3	4
TST	1.7	17.8	-	72

** Initial MC refers to the specimen's moisture content after 48 hour of oven drying*

moisture conditioning, when available, is shown in Table 4.1 as well. At the end of moisture conditioning, the pH values are close to 12.6 considered as saturation with lime. For the back-pressure of 6 psi, significant amount of lime was accumulated on the device pedestal, indicating that such a high backpressure maybe excessive. To minimize the damage to the specimens during the moisture conditioning, a backpressure of 4 psi or less is recommended.

For the vacuum conditioned protocol, a confining pressure of 10 psi was selected to eliminate the penetration of water along the side wall of the specimens. Since it is difficult to inexpensively control the vacuum applied to the specimens, a vacuum level of 1 atmosphere was selected.

Impact of Additive Concentration

Moisture conditioning with the backpressure protocol was carried out on specimens with different concentrations of lime. As shown in Figure 4.11, the specimens with no lime absorbed water at much lower rate. This occurred because the permeability of the clay without additives was much less than those with lime. The three specimens with lime exhibited a similar pattern. This occurred because of the flocculation of clay in the presence of lime. Similar results were obtained for the Bryan clayey material.

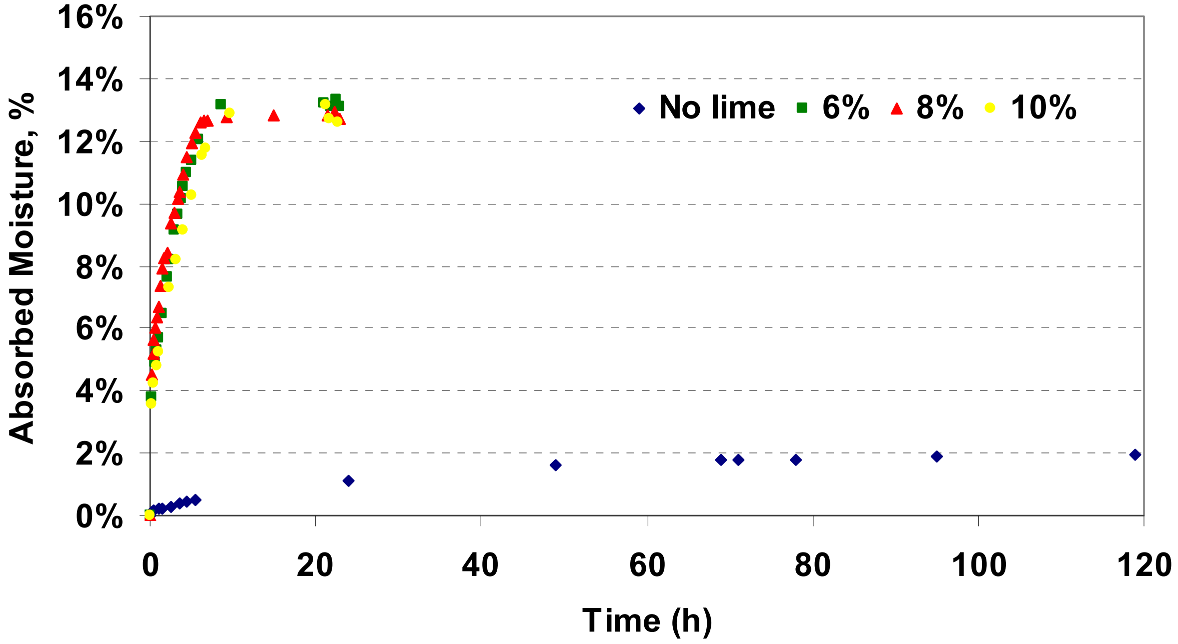


Figure 4.11 – Variations in Moisture Content with Time for Different Additive Contents

The relevant information about the impact of the lime concentration on the moisture conditioning is provided in Table 4.2. The time to saturation for the stabilized materials was about 7 to 10 hrs, whereas the untreated clay was not fully saturated after 36 hrs. Similar results were observed with the vacuum moisture conditioning protocol.

Table 4.2 – Impact of Lime Content on Moisture Conditioning

Lime Content, %	Initial MC*, %	Final MC,%	Final pH	Time to Saturation, hr
0	13.0	15.1	--	> 36
6	2.9	16.2	12.1	9
8	4.6	16.8	12.1	7
10	4.2	16.9	12.3	10

* Initial MC refers to the specimen's moisture content after 48 hour of oven drying

Comparison of Different Compaction Methods

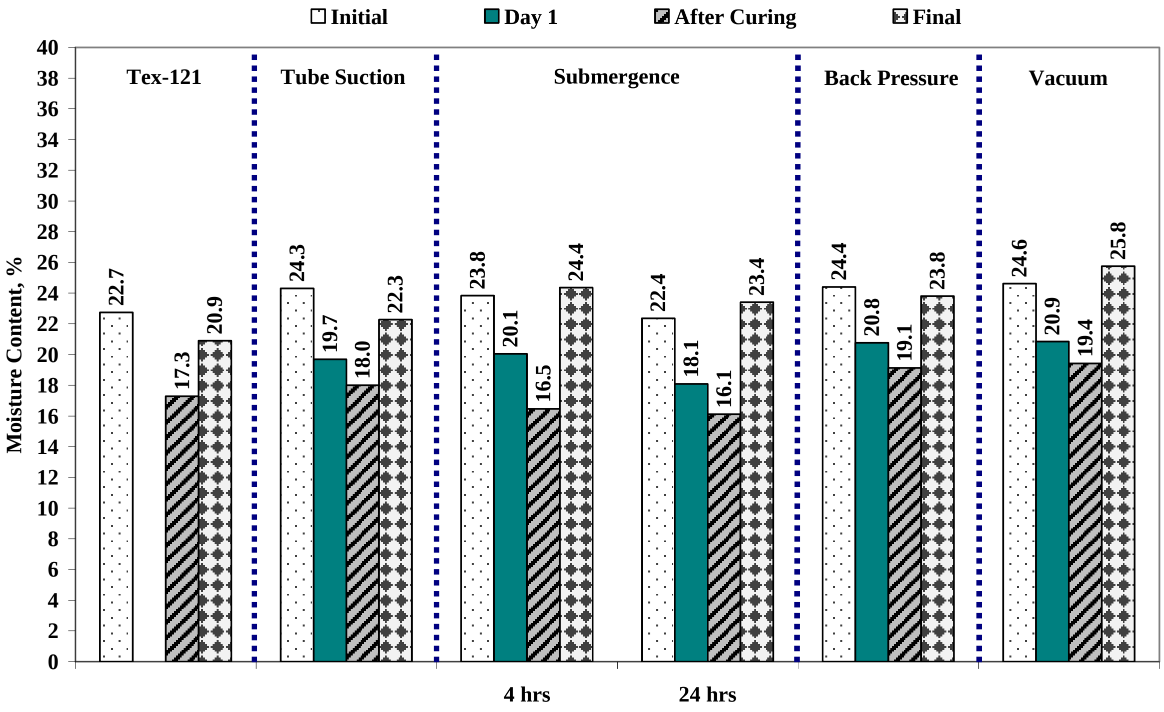
As explained above, some problems with the integrity of the specimens during curing and difficulties with the moisture conditioning of the specimens prepared with the kneading compaction were encountered. The backpressure and vacuum moisture conditioning procedures overcame some of these problems. However, the statically compacted specimens were perceived as more appropriate. To consider the effects of compaction method on moisture conditioning tests, alternative tests were performed on both static and kneading compacted specimens for all clayey materials.

As shown in Table 4.3, the moisture conditioning for the static compacted specimens was more rapid. Moisture conditioning with backpressure and vacuum methods mostly took 4 hrs or less for static compacted specimens while it took 4 hrs or more for specimens prepared with the kneading compactor. The tube suction protocol required more than 48 hrs to achieve saturation in both cases. It was also observed that in most cases the TST specimens became super-saturated at the bottom but did not absorb moisture above the interface of one of the lifts.

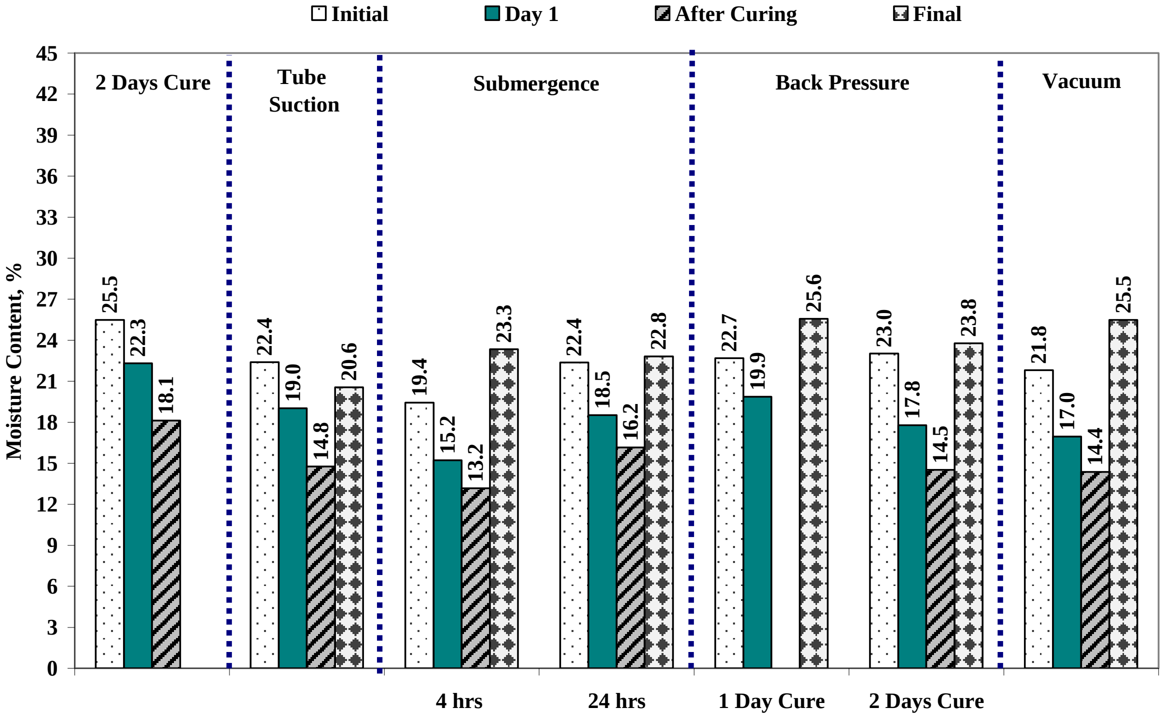
Table 4.3 – Comparison of Time to Saturation for Static and Kneading Compacted Specimens

Method	Time to Saturation, hr	
	Static	Kneading
Back Pressure	2	5
Vacuum	4	4
Tube Suction	48-72	48-72

The variations in moisture content with time for all moisture conditioning methods, and kneading and static compaction are compared in Figure 4.12 for the case of Bryan clay specimens. In addition, for statically compacted specimens, results for 2 days cure are also included for comparison. The initial moisture contents after the completion of compaction was reasonably close for all specimens. For uniformity, all specimens were first cured in a 104ºF oven for 48 hours except for Tex-121-E, where the specimens were cured for 7 days and placed in the oven for 6 hours. For the kneading compacted specimens, the moisture contents after curing were



a) Kneading Compacted Specimens



b) Static Compacted Specimens

Figure 4.12 – Moisture Variations of Specimens during Curing and Moisture Conditioning Processes

about 5 to 7% lower than the initial moisture content for all moisture conditioning methods. The final moisture contents, after completion of moisture conditioning, were around 1% above the initial moisture content for all alternative methods, except for backpressure, where the final moisture content was 0.6% lower than the initial. For the Tex-121-E and Tube Suction methods, final moisture contents were around 2% less than the initial. For the case of the static compacted specimens, moisture contents after curing were about 6 to 8% lower than the initial moisture content for all moisture conditioning methods. Final moisture contents for all alternative methods were above the initial by 0.4 to 4%. Tex-121-E was not performed on static compacted specimens, but for Tube Suction the trend was similar than kneading compaction case and final moisture content was 2% lower than initial. It should be emphasized that several of the specimens subjected to Tube Suction protocol and Tex-121-E did not become saturated to the top.

The strengths of the specimens after curing and moisture conditioning are compared in Figure 4.13. The highest strength is associated with the Tex-121-E and the lowest with the 4 hrs of submerging the specimens for kneading compacted specimens. For static compacted specimens the highest strength was associated with 2 day curing in the oven. The strengths from the other protocols varied between 80 psi and 130 psi and 70 psi and 110 psi, for kneading and static compaction, respectively. As reflected in Chapter Three, the specimens that were moisture conditioned as per Tex-121-E did not allow moisture migration to the top of the specimen. The strength of the materials after 7 days of curing but before moisture conditioning was 96 psi. The trends from the other soils were rather different than those shown in Figure 4.13 and will be discussed further later.

The typical variations in moduli of the specimens after compaction, after curing and after moisture conditioning measured using the FFRC device are shown in Figure 4.14. In all cases,

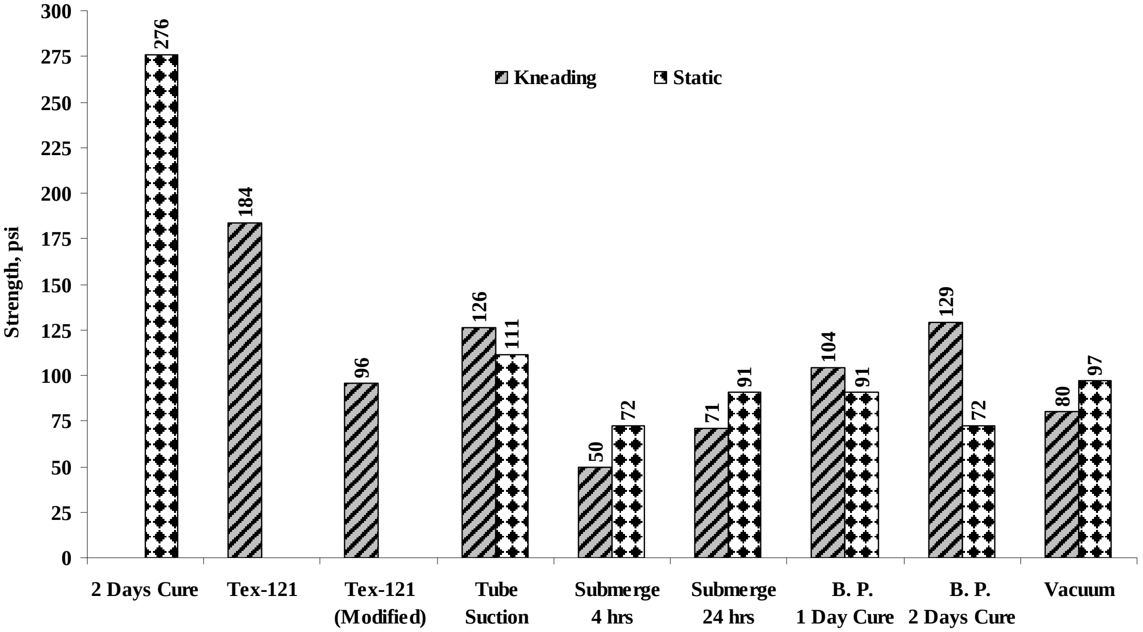
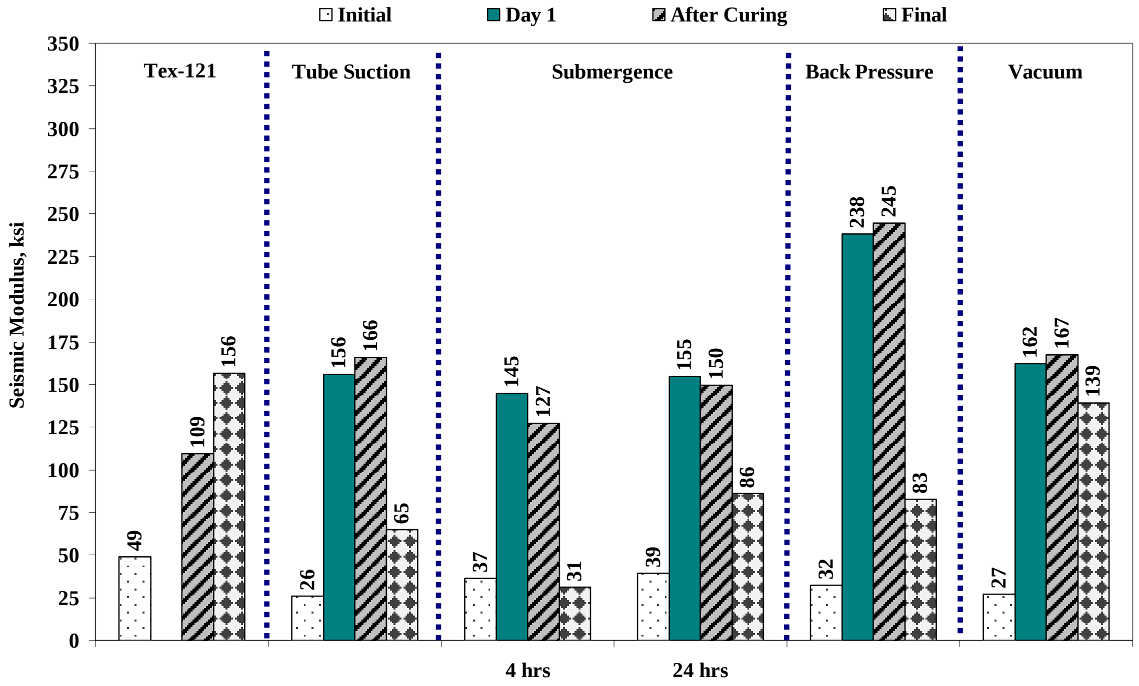
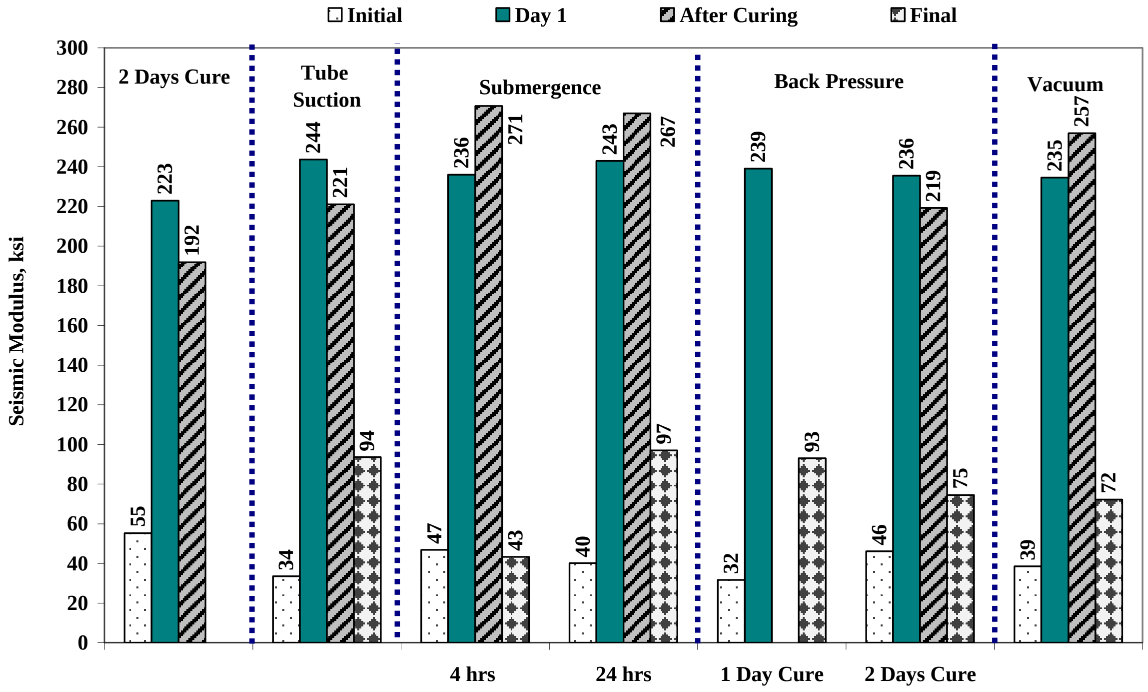


Figure 4.13 – Variations in Conditioned Unconfined Compressive Strengths for Different Moisture Conditioning Processes



a) Kneading Compacted Specimens



b) Static Compacted Specimens

Figure 4.14 – Modulus vs. Time of Moisture Conditioning Test for Bryan Specimens

the initial modulus after compaction was fairly similar for all specimens and for both compaction methods. During the curing period, the moduli were consistently higher for all static compacted specimens. After moisture conditioning, the trends in moduli for all specimens was comparable between kneading and static compacted specimens for the same moisture conditioning methods. The moisture conditioned moduli ranged from 31 ksi to 156 ksi and from 43 ksi to 97 ksi, for kneading and static compaction, respectively.

The retained modulus, defined as the ratio of the modulus after the moisture conditioning and modulus after curing for each condition is summarized in Figure 4.15. For Tex-121-E specimens the retained modulus was greater than one for this particular clay (others are discussed in next section). The retained moduli were usually similar for the kneading compacted and static specimens. The results from the static compacted specimens were more uniform since the moisture conditioning was more uniform.

Dielectric constants for all specimens and for both compaction methods are presented in Figure 4.16. Initial dielectric values were similar for all cases and for both methods. The final dielectric results were consistently smaller than initial values except for submerged specimens. Aside from the chemical interaction, the reason for this unanticipated trend cannot be speculated.

Changes in the pH for different alternative moisture conditioning methods were measured to evaluate the leachability of the soil-lime mixes. The results for the back pressure and for vacuum conditioning are presented in Table 4.4 for the Bryan material. Final pH values after moisture conditioning and after nearly 150 ml water was allowed to flush through the specimens were close to 12.

The results for the submerged specimens are shown in Table 4.5. For uniformity in measurements, three gallons of water was used to submerge each specimen. In this case, the pH

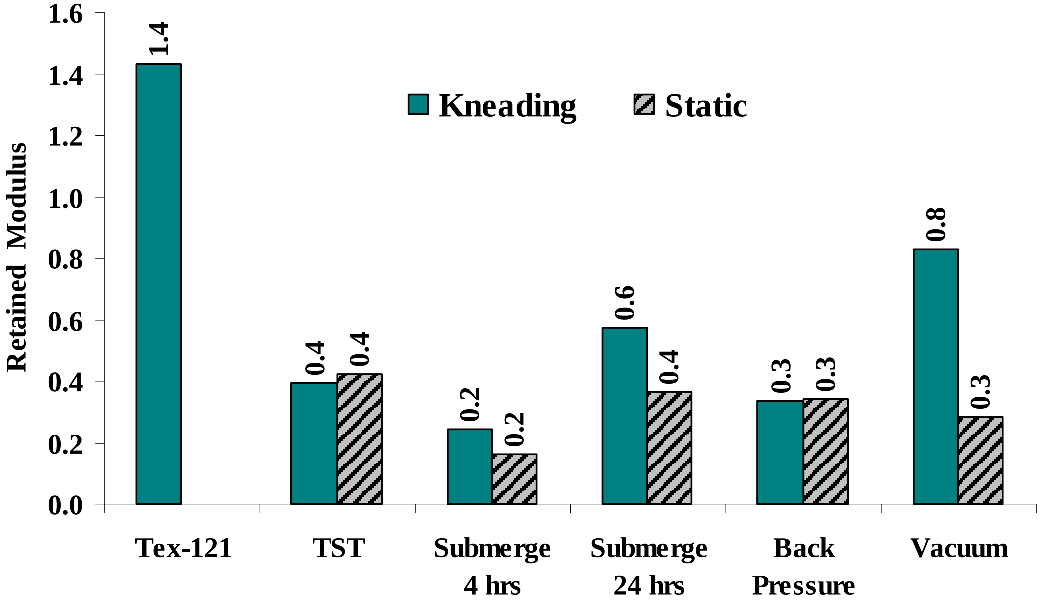
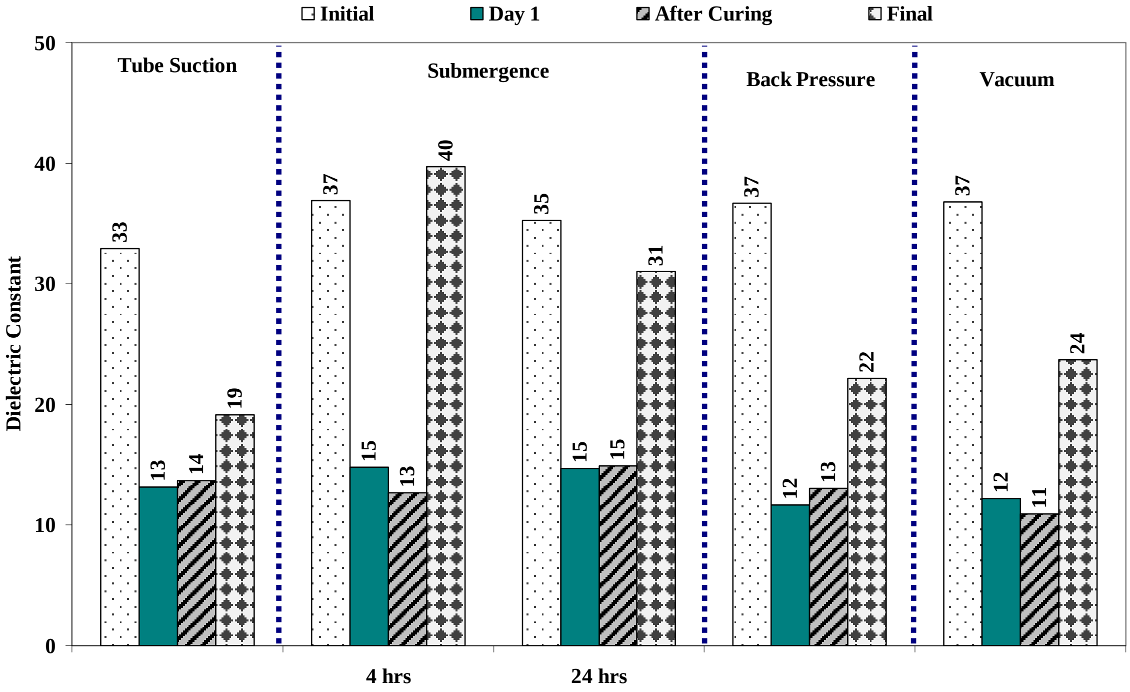
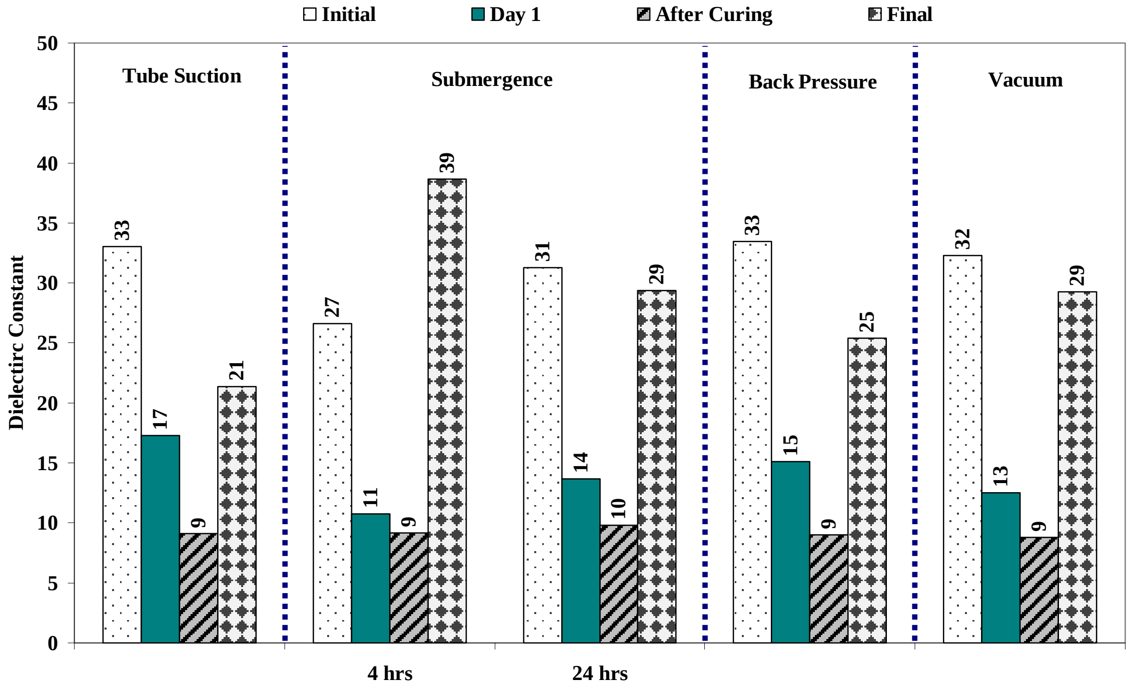


Figure 4.15 – Retained Moduli between Specimens after Moisture Conditioning and after Curing Period



a) Kneading Compacted Specimens



b) Static Compacted Specimens

Figure 4.16 – Dielectric Results of Moisture Conditioning Specimens for Bryan Material

Table 4.4 – Final pH Measurements for Alternative Moisture Conditioning Methods

Moisture conditioning Method	Final pH	
	Kneading	Static
Back Pressure	12.0	12.1
Vacuum	12.1	12.0

Table 4.5 – pH Measurements and Change with Time for Submerged Specimens

Time	pH Value	
	Kneading	Static
0	7.8	7.9
20 min	8.7	8.7
60 min	9.1	9.0
120 min	9.3	9.1
180 min	9.6	9.3
240 min	9.7	9.3
24 hr	11.0	10.5

was measured every 30 minutes for the first 4 hrs and after 24 hrs. The highest pH obtained with this method was 11 for the kneading compacted specimens.

Impact of Curing Time on Moisture Conditioning Results

To study the impact of curing time on moisture conditioning, specimens were allowed to cure either 24 hrs or 48 hrs in a 104°F oven before moisture conditioning. As an illustrative example, the strengths after moisture conditioning are shown in Figure 4.17. For the kneading compacted specimens, the two days of curing provided slightly higher moisture-conditioned strength; whereas for the static compacted specimens the trend was reversed.

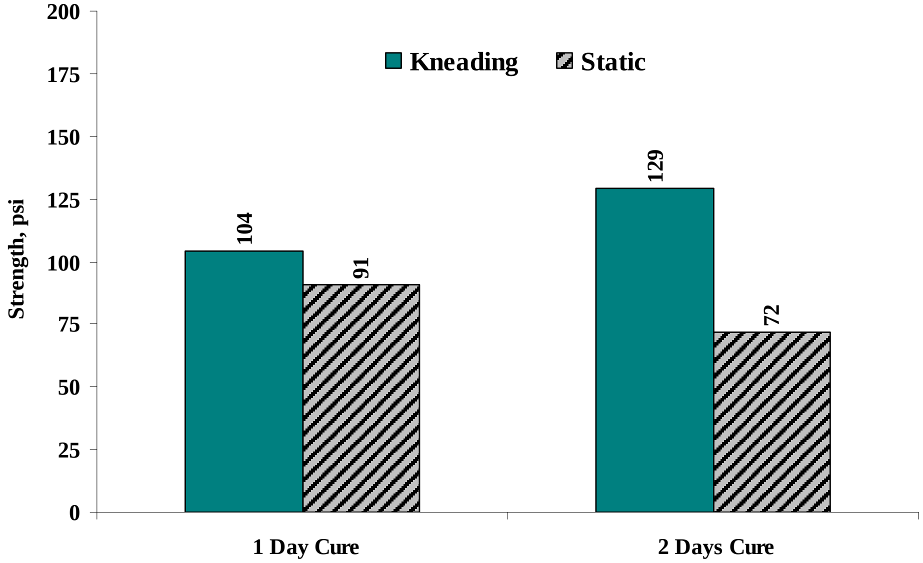


Figure 4.17 – Strengths for Different Curing Times after Moisture Conditioning

The variations in moisture content with time for kneading and static compaction specimens are compared in Figure 4.18. For the 1-day cured specimens, the moisture contents after curing were about 3 to 4% lower than the initial moisture content for both compaction methods, while for the 2-day cured specimens these values ranged from 5% to 8%. The final moisture contents, after completion of moisture conditioning, were around 1% to 3% above the initial moisture contents for static compaction, where the final moisture contents for the kneading compacted specimens were within 1% of the initial moisture contents for 1 day and 2 day cured specimens, respectively.

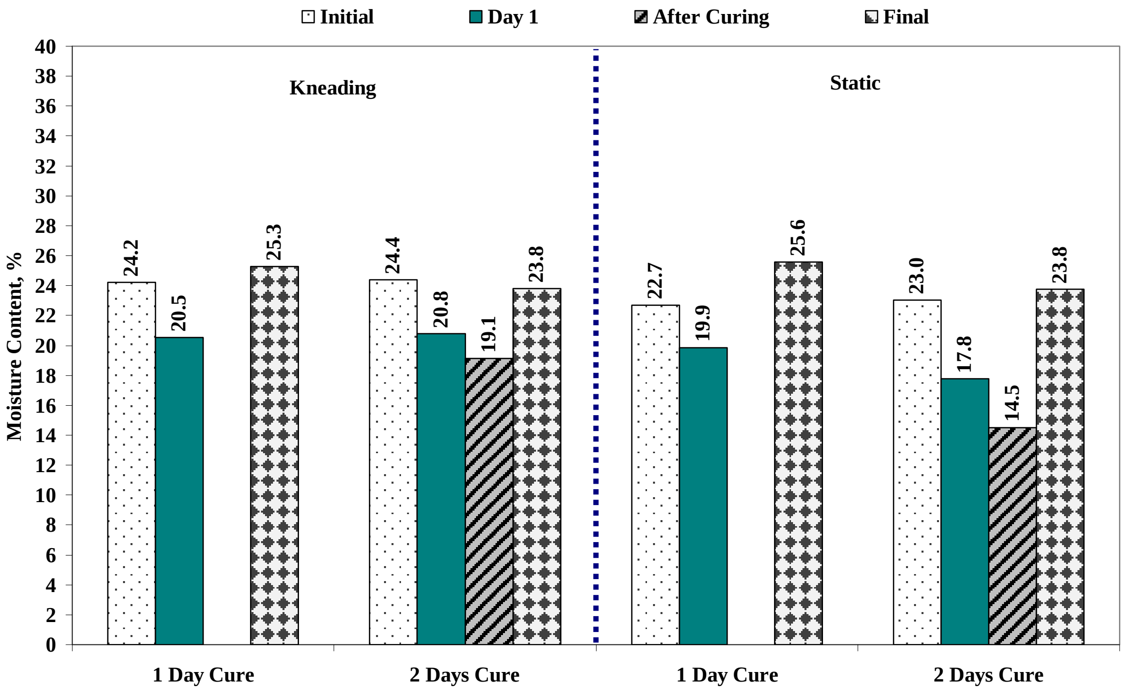


Figure 4.18 – Moisture Contents for 1 Day and 2 Day Cured Specimens

Moduli from specimens subjected to 1-day or 2-day curing for the two types of compaction methods are shown in Figure 4.19. After the curing period, the seismic moduli for all specimens ranged from 180 to 240 ksi. After the moisture conditioning process, moduli were fairly similar.

Impact of Density on Moisture Conditioning

Since during construction a density of 95% is allowed, specimens were prepared with the static compaction method at 100% and 95% of the maximum dry densities. Static compaction was used since the densities can be better controlled. El Paso material was used for this purpose. Figure 4.20 shows the impact of density on the strength. Strengths were consistently smaller for the case of 95% compactive effort. The ratio between the strengths obtained with 95% and 100% densities is close to 0.5, except for the specimens that were moisture conditioned by submergence for 24 hours. As shown in Figure 4.21, the same pattern was observed with the modulus.

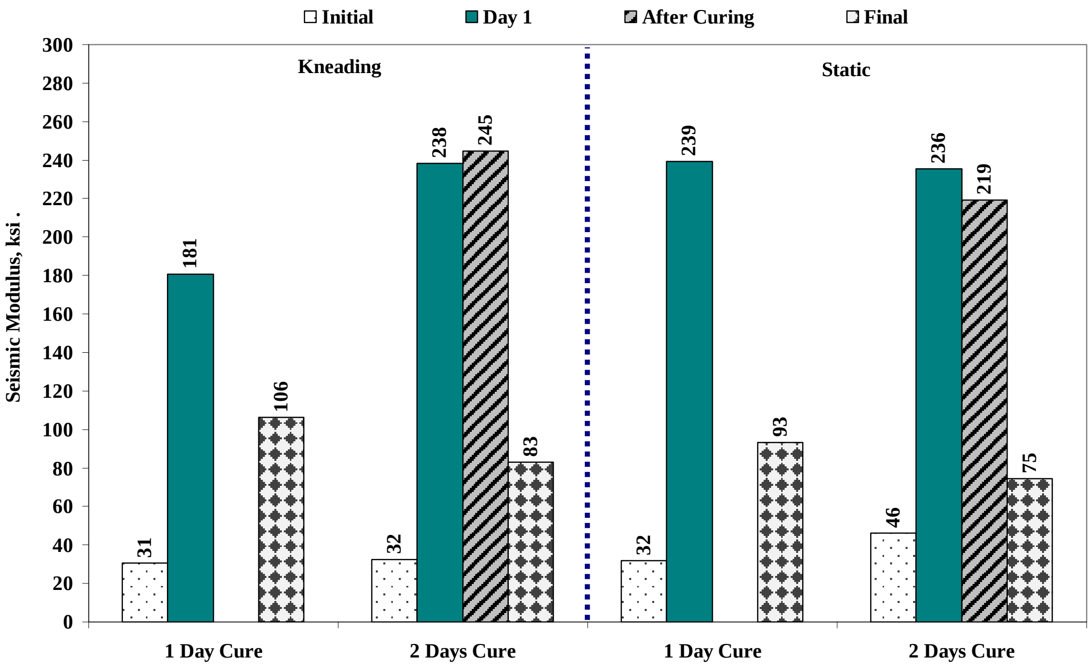


Figure 4.19 – Seismic Moduli for 1 Day and 2 Day Cured Specimens

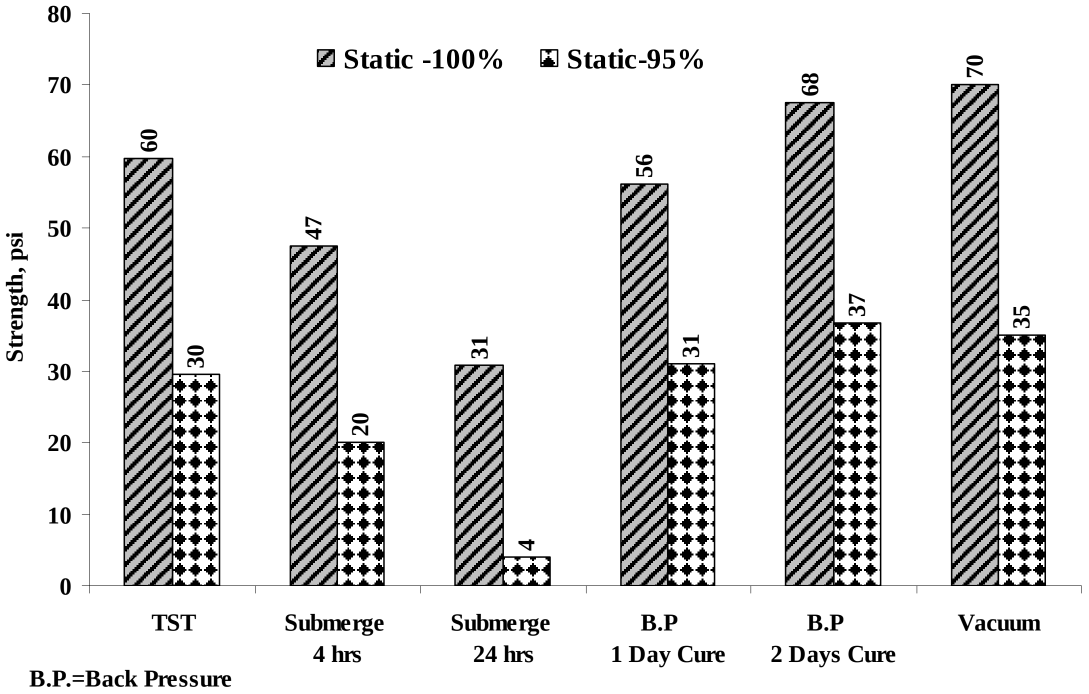
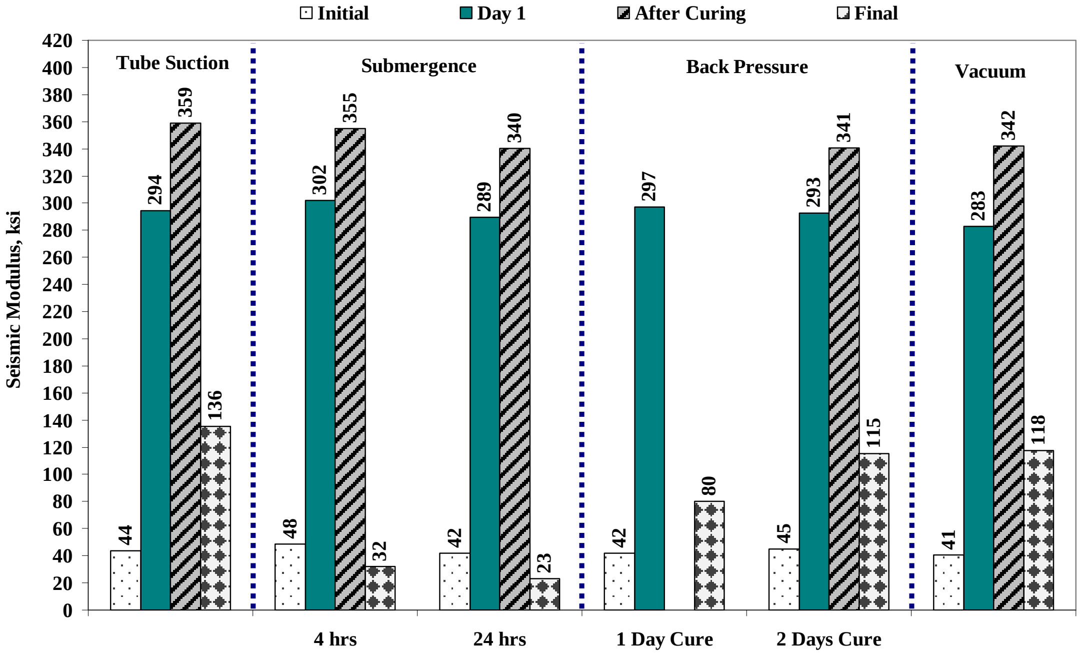
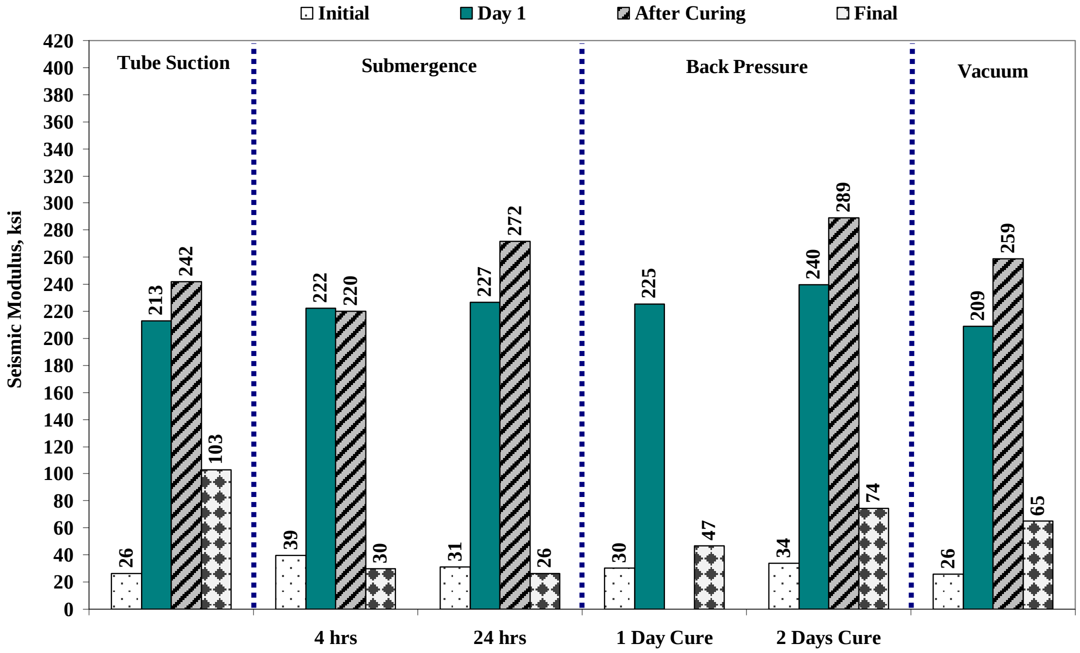


Figure 4.20 – Strength Results for Different Compactive Efforts (El Paso Material)



a) 100% of Maximum Dry Density



b) 95% of Maximum Dry Density

Figure 4.21 – Modulus Variation for Different Levels of Compaction Efforts and for Static Compaction

As shown in Table 4.6, the moisture conditioning for the 95% density specimens were more rapid. Moisture conditioning with back pressure and vacuum methods took between 4 and 6 hours for 100% compaction and 2 to 3 hours for 95% compaction. For vacuum tests these values were 3 and 2 hours respectively. The Tube Suction protocol required more than 48 hours to achieve saturation in both cases.

Table 4.6 – Comparison of Time to Saturation for Different Compaction Efforts

Method	Time to Saturation, hr	
	Static 100%	Static 95%
Back Pressure 1 Day Curing	6	2
Back Pressure 2 Day Curing	4	3
Vacuum	3	2
Tube Suction	>48	>48

Impact of Curing Temperature on Moisture Conditioning

To study the impact of curing temperature, specimens were prepared with the static compaction method at two different temperatures of 104°F and 140°F (40°C and 60°C). The Fort Worth Clay is shown here as an example.

The strengths for specimens cured at 140°F were consistently less than those cured at 104°F as illustrated in Figure 4.22. The higher temperatures seem to negatively impact the integrity of several clays studied here. Some of the submerged specimens were too damaged after moisture conditioning and no strength values are reported.

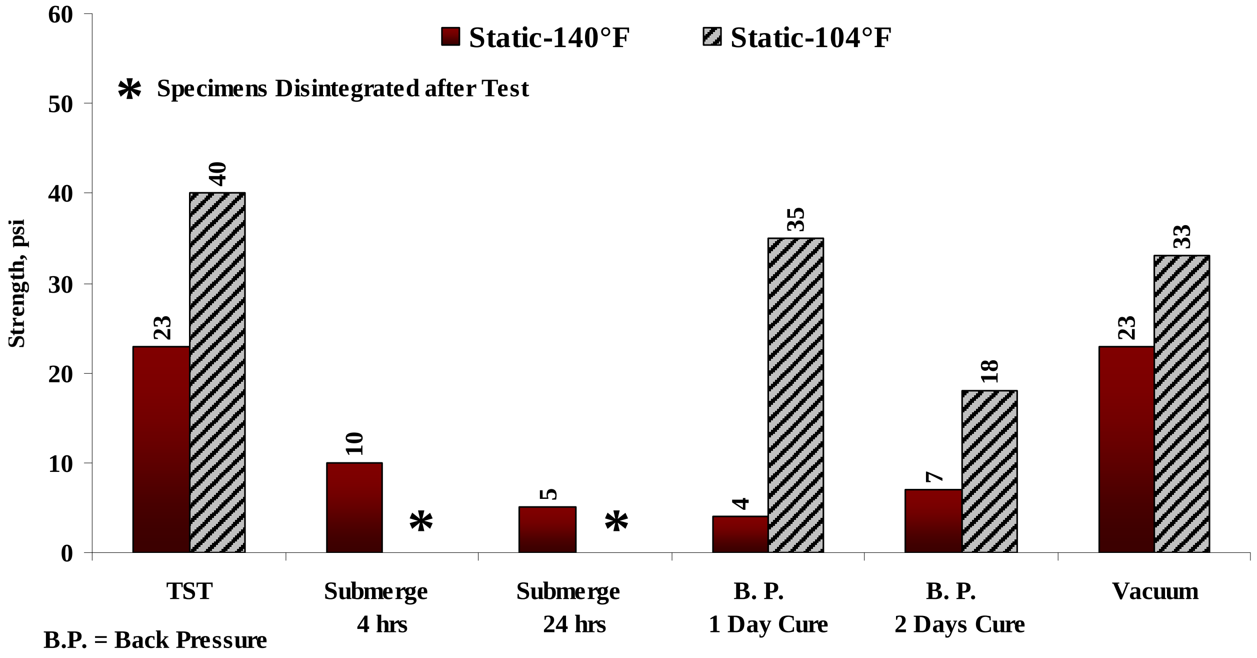
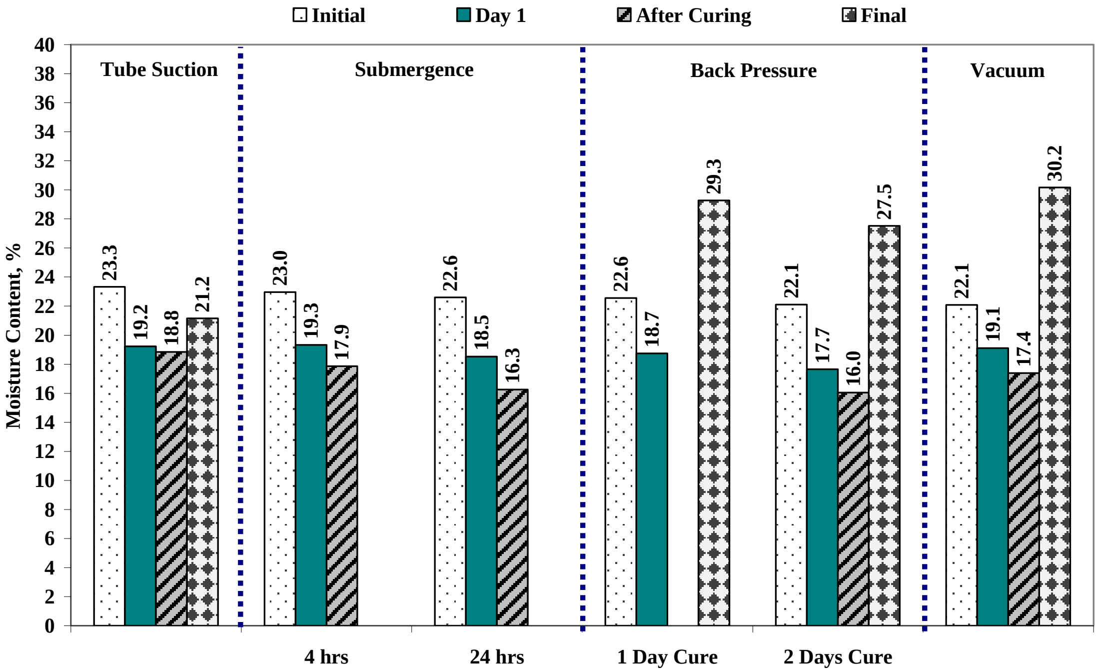
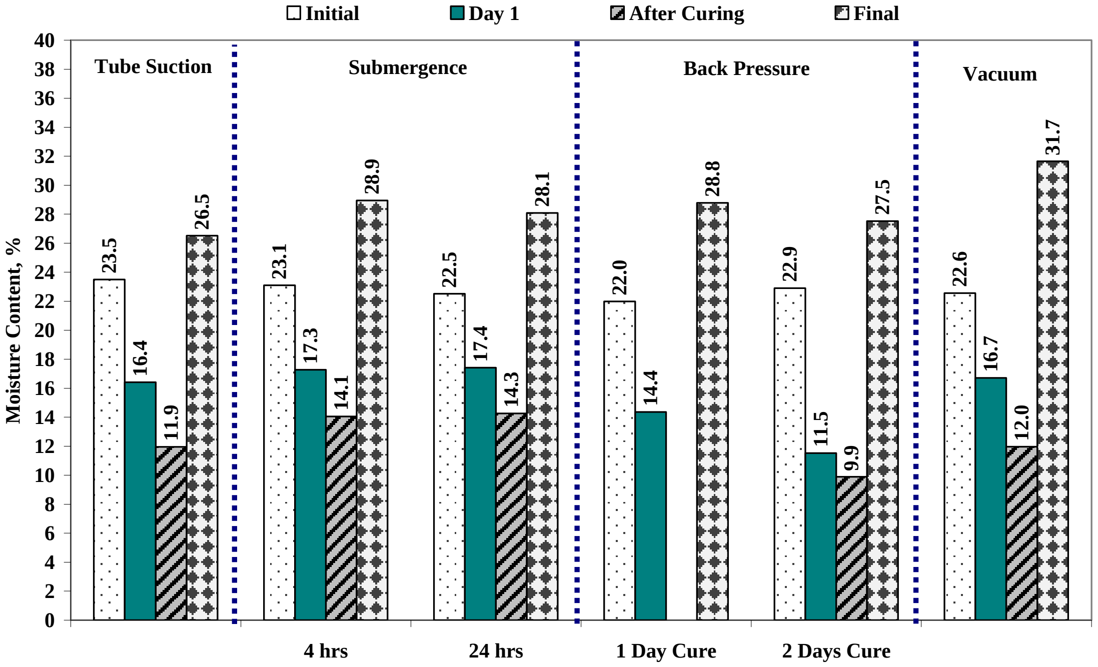


Figure 4.22 – Strength Results on Saturated Specimens for Different Curing Temperatures

The variations in moisture content with time for different curing temperatures are compared in Figure 4.23. It seems that high curing temperatures draw too much moisture out of the specimen too fast which may interrupt the curing of the lime-clay mix.



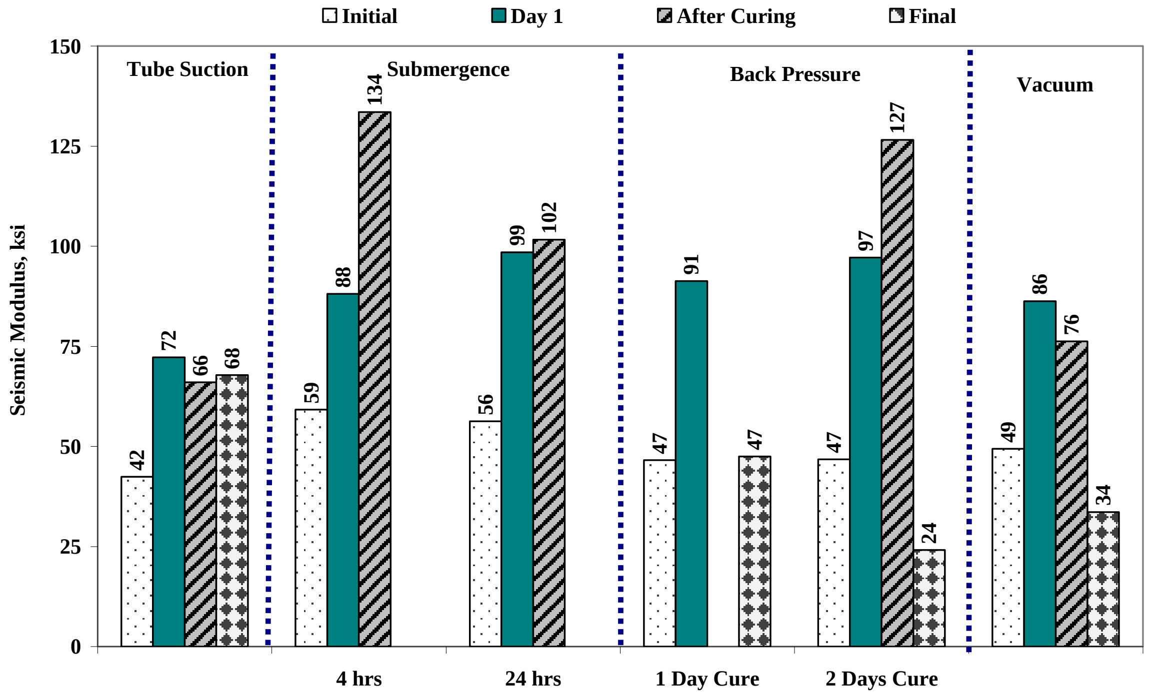
a) Specimens Cured at 104°F



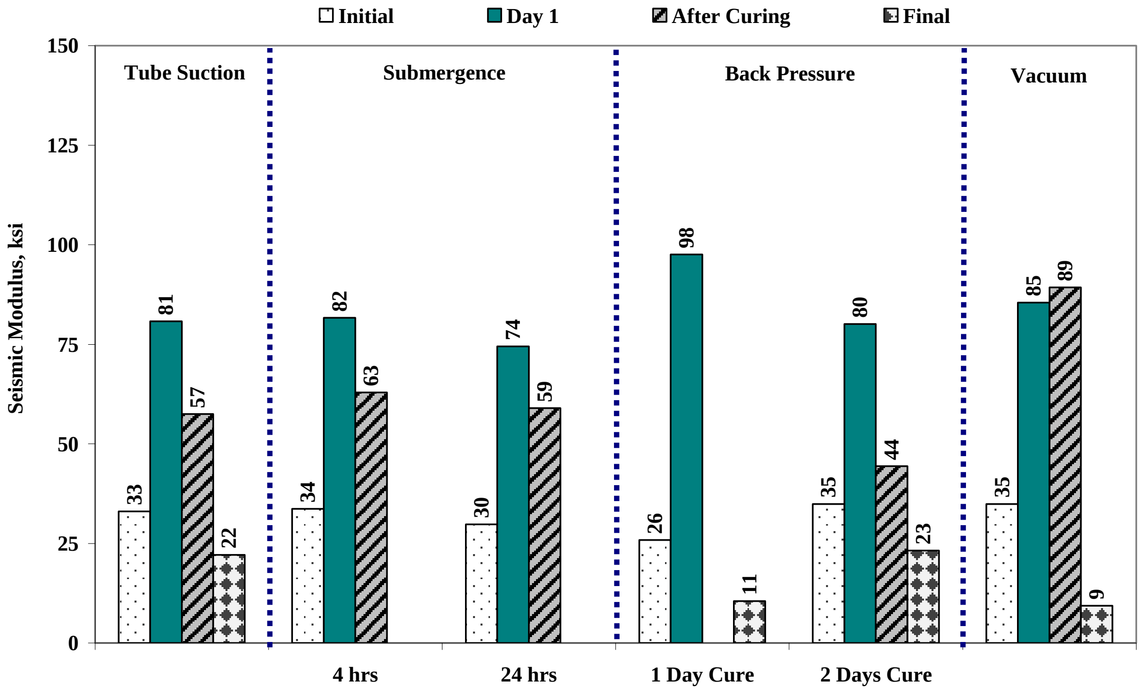
b) Specimens Cured at 140°F

Figure 4.23 – Moisture Content Variation for Different Levels of Curing Temperature and for Static Compaction

The variations in modulus for different curing temperatures are compared in Figure 4.24. In many cases, the gain in modulus is interrupted by the high temperatures mostly due to cracking.



a) Specimens Cured at 104°F



b) Specimens Cured at 140°F

Figure 4.24 – Modulus Variation for Different Levels of Curing Temperature and for Static Compaction

MOISTURE CONDITIONING RESULTS OF CLAYEY MATERIALS

In this section the moisture conditioning protocols described in the previous section are evaluated for the clays studied. For all materials, strength, final moisture content, seismic modulus and dielectric results are compared. Current TxDOT specifications (Tex-121-E) were used as reference and all values are compared to those obtained with kneading compacted specimens under Tex-121-E. To delineate the impact of curing from moisture conditioning, the results were also compared with the TST protocol.

Strength Results

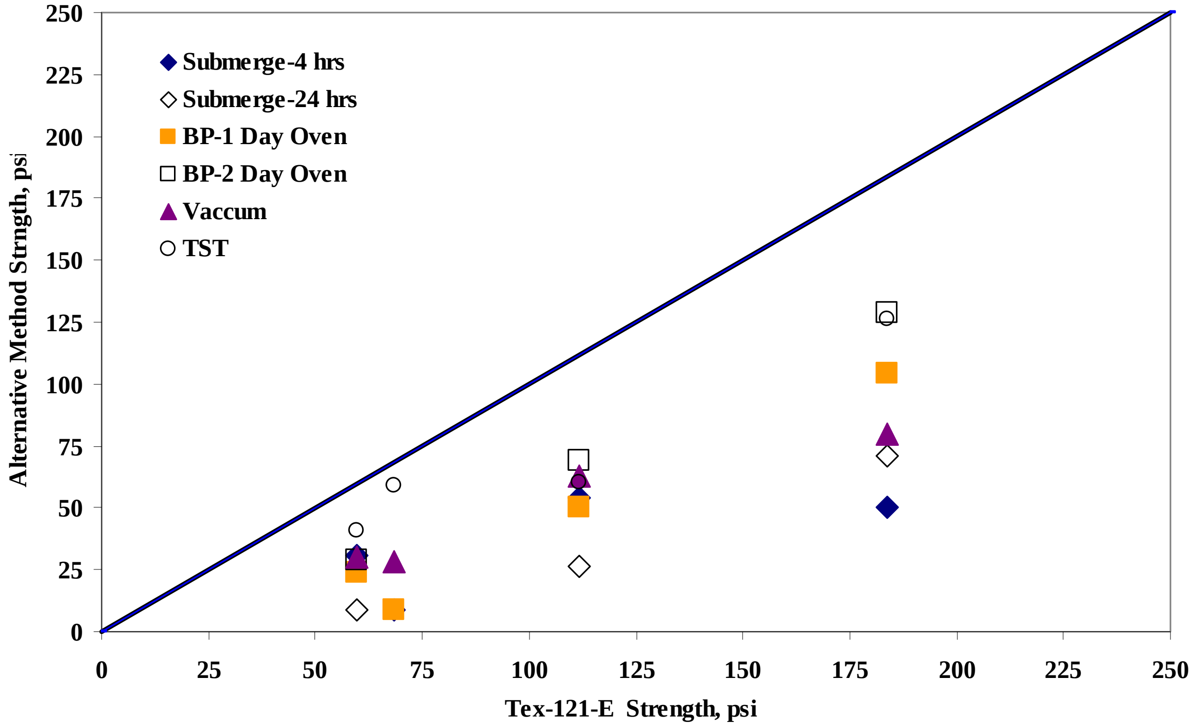
Strengths from alternative moisture conditioning protocols are compared to those obtained with Tex-121-E in Figure 4.25 for the kneading and static compacted specimens. The strengths from Tex-121-E are always greater than the other alternative methods. This occurs because the curing method for Tex-121-E lasts six days whereas for all other methods the specimens were cured for up to 48 hours.

The variations in modulus with time of curing for the four clays are shown in Figure 4.26. The moduli after 7 days as well as the rate of gain in modulus vary depending upon the material used. The moduli obtained from the two days of oven drying used in all moisture conditioning protocols are superimposed on the trend lines from Figure 4.26 in Figure 4.27. The patterns are similar except for one case when the specimens tend to crack at higher temperatures. This indicates that curing at elevated temperatures may not be either necessary or desirable. The second conclusion is that the specimens tend to gain stiffness up to six days of curing. Based on several field observations, the modulus obtained after six days of curing may be too high as compared to the field moduli. One of the focuses of this study is to quantify this difference so that the number of curing days in the laboratory can be reduced.

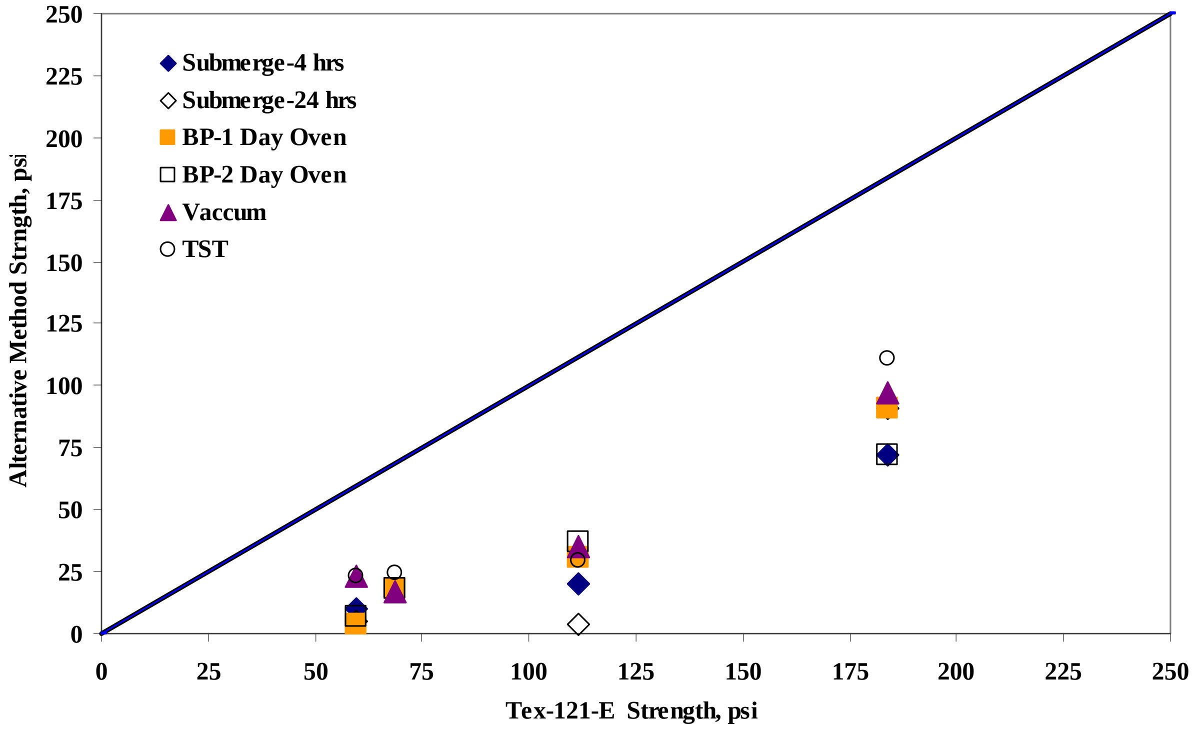
To delineate the impact of curing from moisture conditioning, the moisture conditioned strengths are compared with those from TST in Figure 4.28 since the TST moisture conditioning is identical to Tex-121-E. The results from the backpressure method after two days of curing are similar to those with the TST for the specimens that saturated to the top and lower when the TST specimen would saturate to the top of the specimens. Similar trends were observed for the static compacted specimens. The vacuum-conditioned specimens yielded much lower strengths as compared to the TST specimens for the kneading compacted specimens, but yielded comparable results to the TST specimens for the static-compacted specimens. The submerged specimens either disintegrated or exhibited much lower strength than the TST specimens.

Final Moisture Content Results

The final moisture contents (after moisture conditioning) for all materials and both compaction methods are compared with those from Tex-121-E in Figure 4.29. The final moisture contents were greater in most cases for kneading compacted specimens when compared to Tex-121-E. The final moisture contents from the TST protocol were the closest to the Tex-121-E simply because the two moisture conditioning protocols are identical. As shown in Figure 4.30, for the kneading compacted specimens, all the alternatives resulted in similar final moisture contents to



a) Kneading Compaction



b) Static Compaction

Figure 4.25 – Comparison of Strengths from Tex-121-E with Other Moisture Conditioning Methods

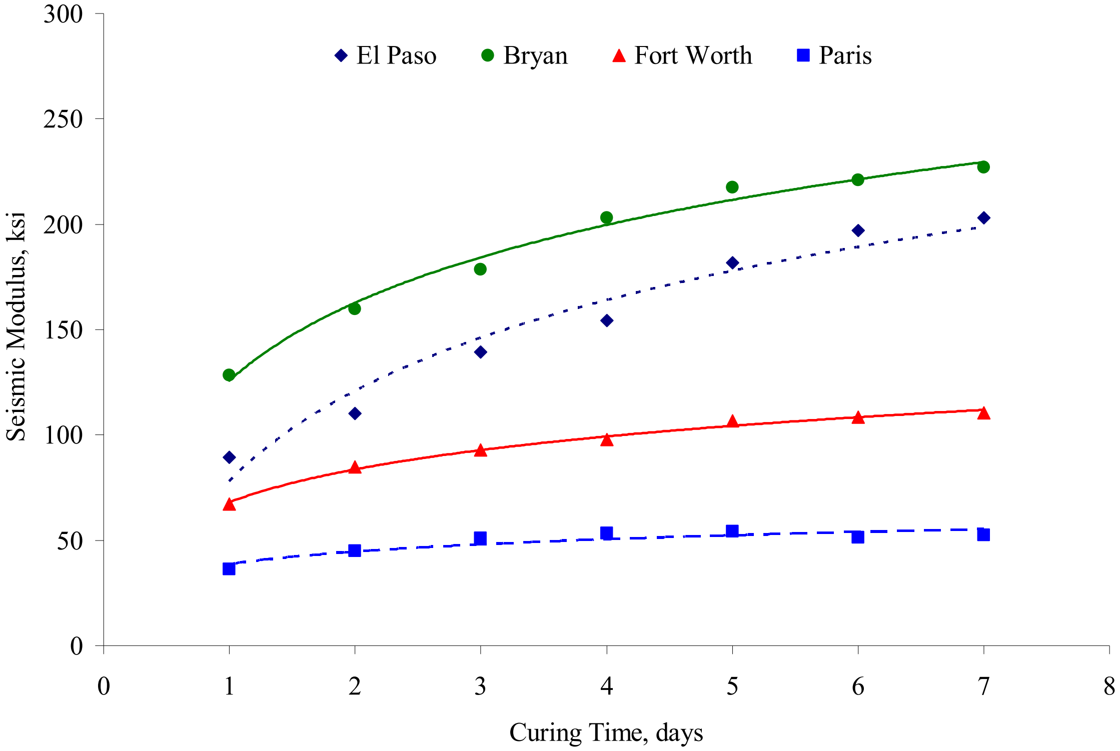


Figure 4.26 – Variation in Moduli with Curing Time for Four Lime-Stabilized Clays

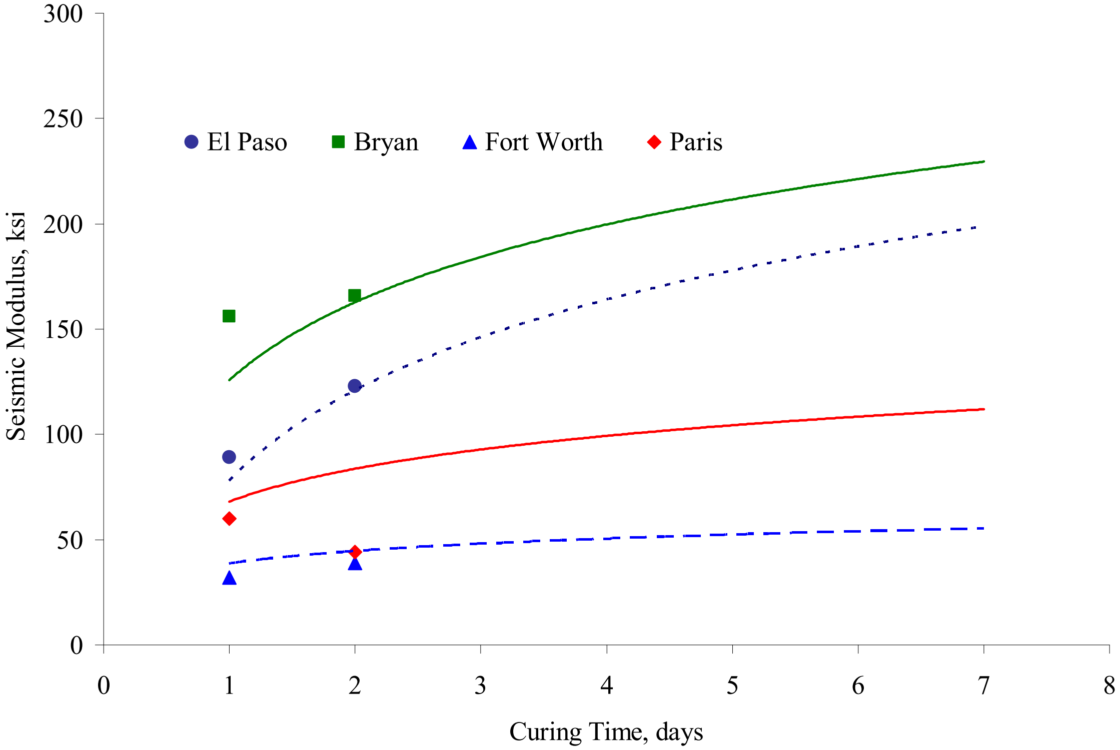
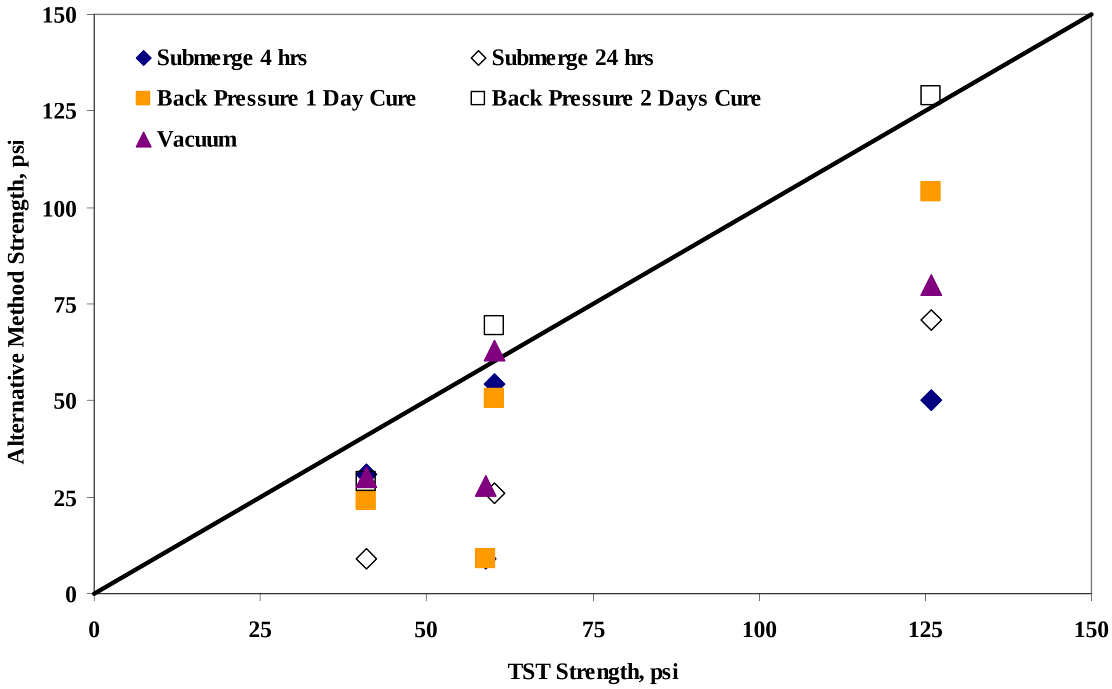
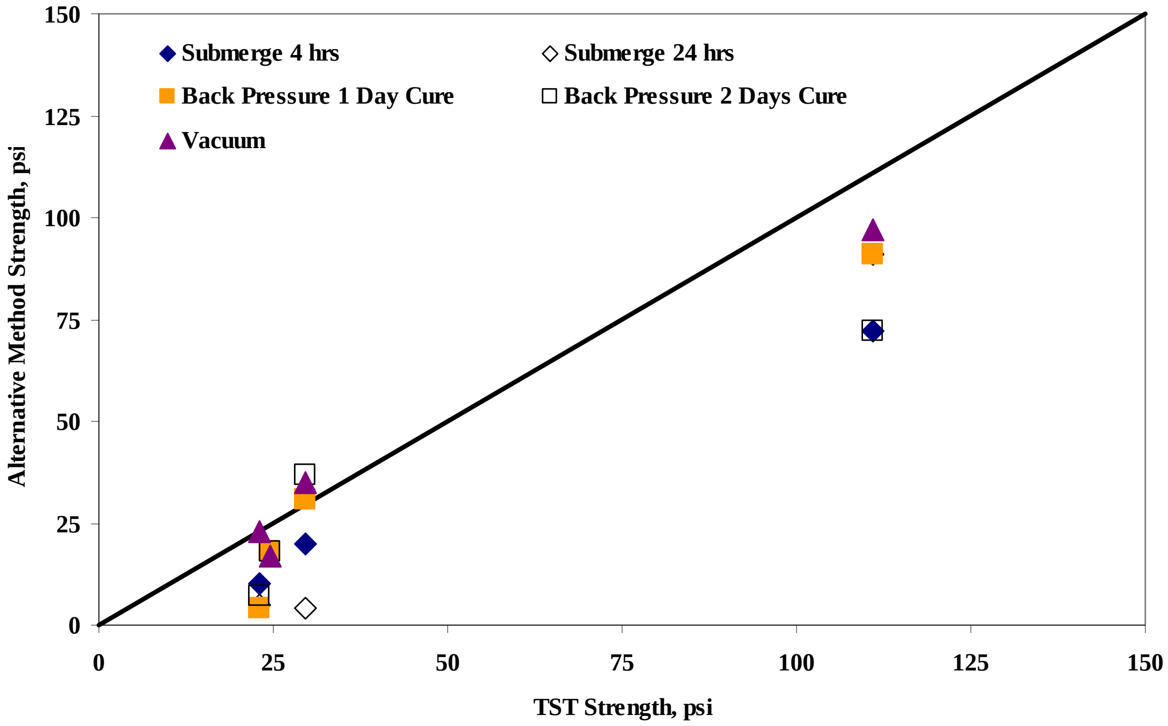


Figure 4.27 – Comparison of Curing Patterns from Tex-121-E with 2 Days at 104ºF Oven

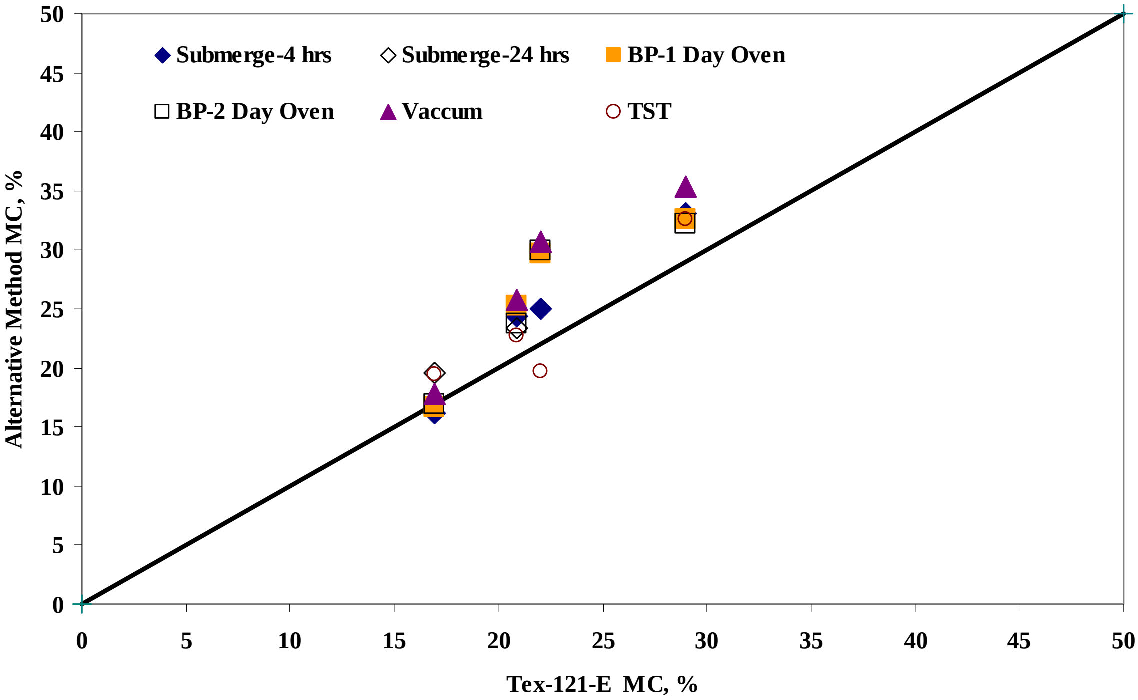


a) Kneading Compaction

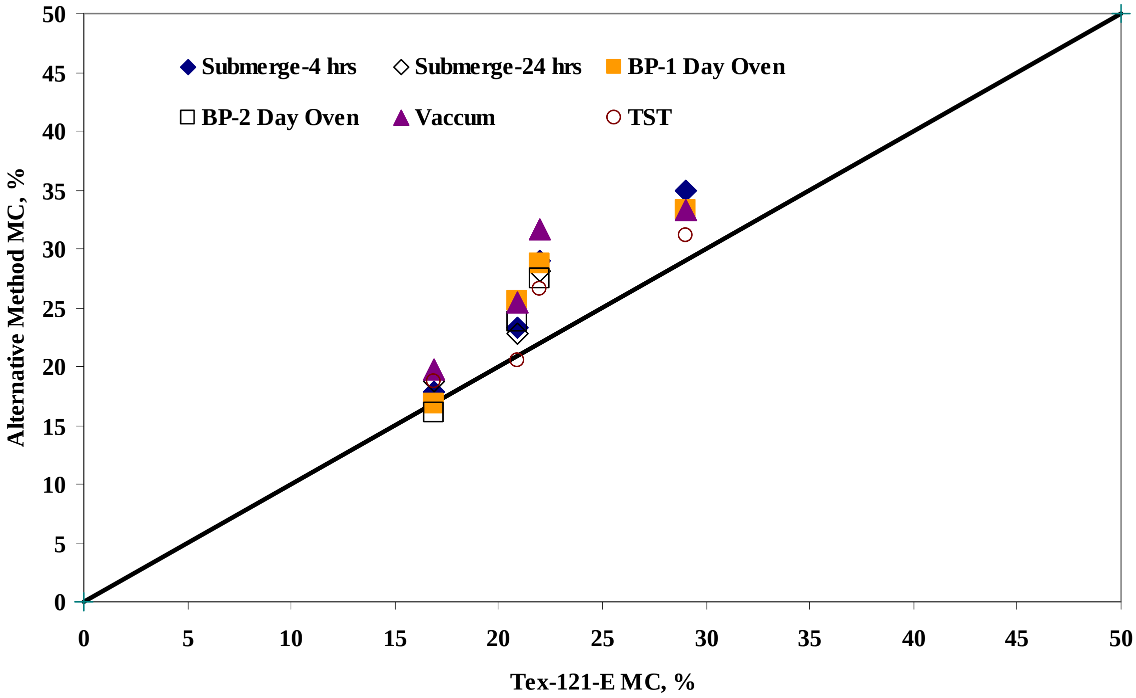


b) Static Compaction

Figure 4.28 – Comparison of Strengths from TST Protocol with Other Moisture Conditioning Methods

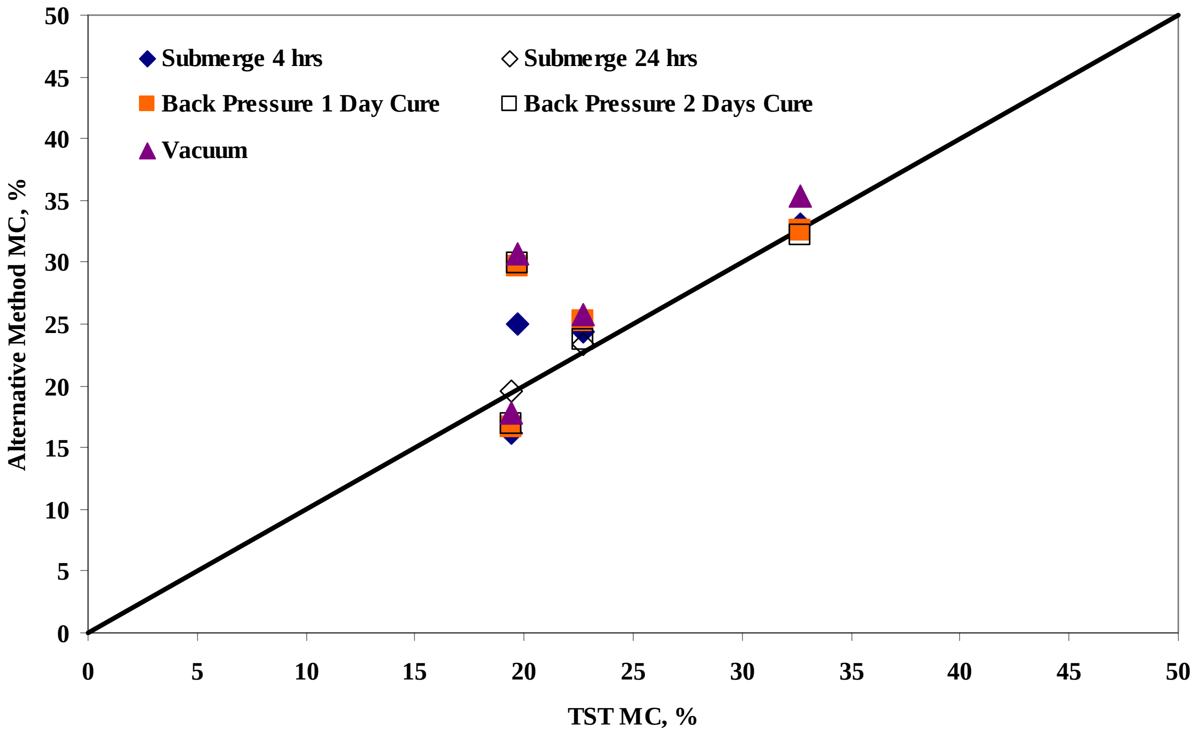


a) Kneading Compaction

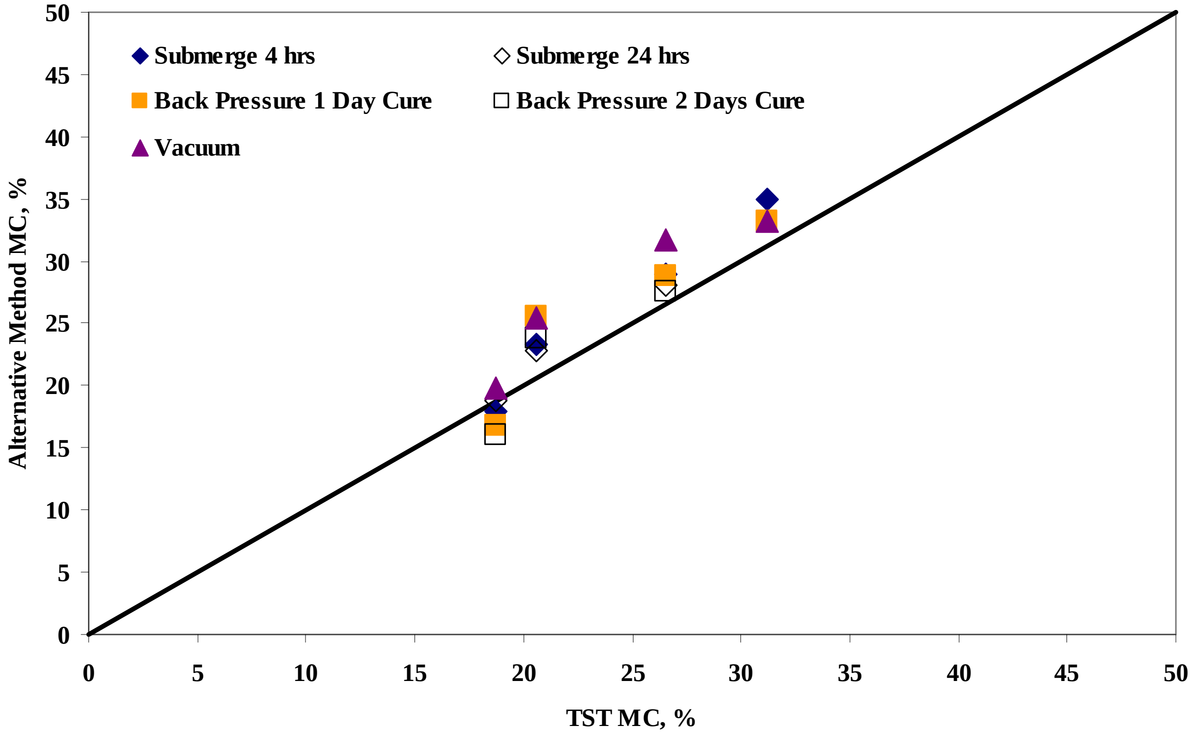


b) Static Compaction

Figure 4.29 – Comparison of Moisture Contents from Tex-121-E Protocol with Other Moisture Conditioning Methods



a) Kneading Compaction



b) Static Compaction

Figure 4.30 – Comparison of Moisture Contents from TST Protocol with Other Moisture Conditioning Methods

the TST protocols when the specimens saturated to the top. For the static compacted methods, the final moisture contents were generally higher than the TST and were quite similar to one another.

Modulus Results

The comparison of the final moduli from the alternative methods with the TST method is presented in Figure 4.31. Final seismic moduli (after moisture conditioning) were always less than those from the Tex-121-E, similar to the strength. The backpressure and vacuum protocols were either close or less than the moduli obtained from the TST specimens, as were the case for the strength.

Dielectric Constant Results

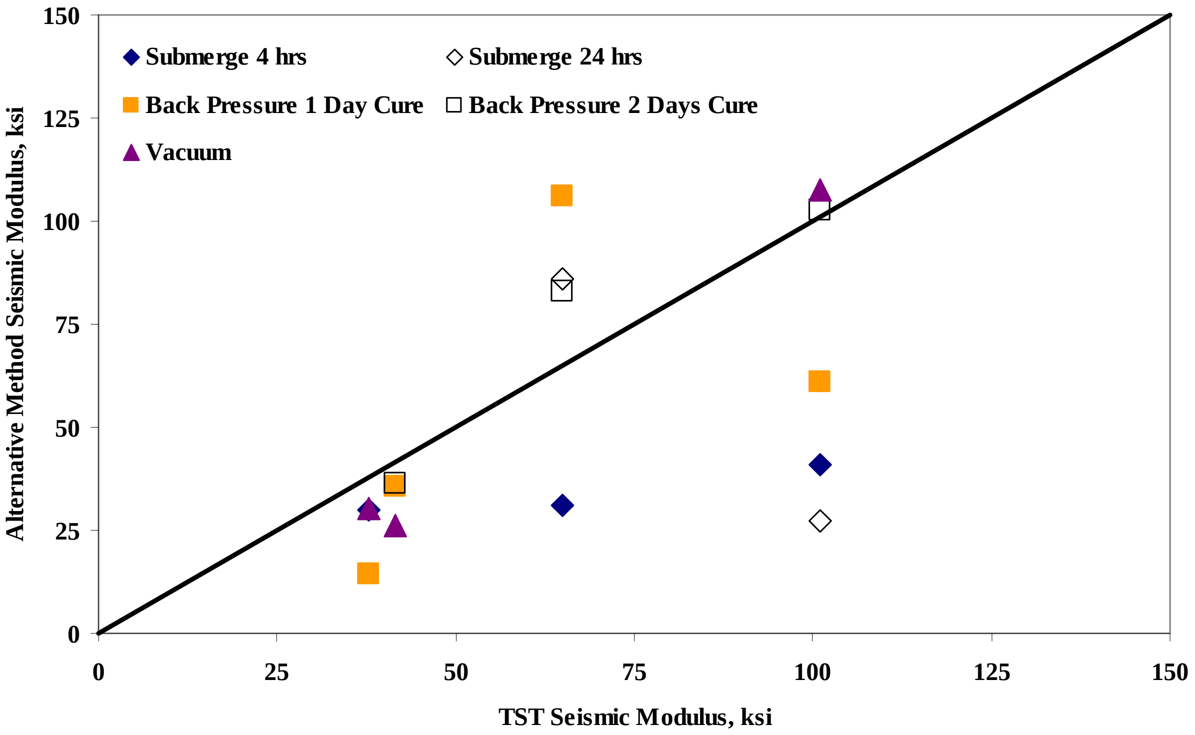
The final dielectric constants are compared with those from TST method in Figure 4.32. For the kneading compacted specimens, the alternative methods yielded greater dielectric constants as compared to the TST, while for the static compacted specimens, the same tendency was observed for all materials except for the case of Fort Worth material.

MOISTURE CONDITIONING RESULTS OF WICHITA FALLS SAND MATERIAL

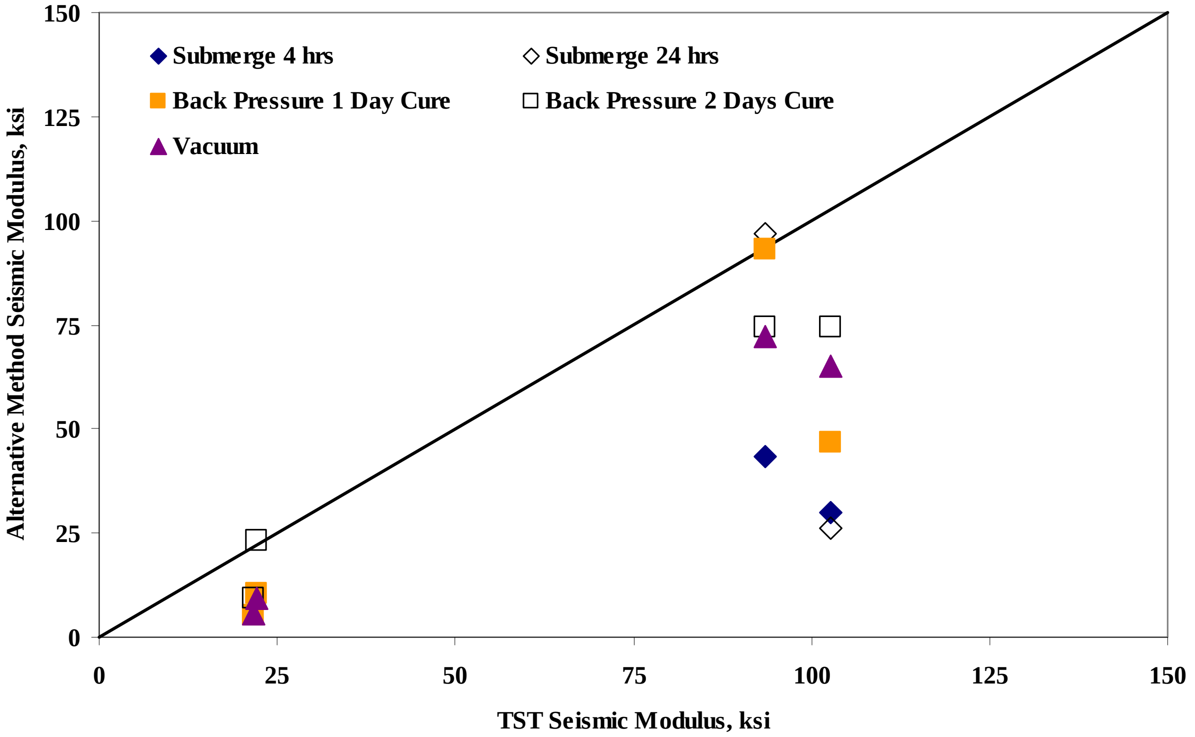
In this part, the moisture conditioning protocols described in the previous section are evaluated for a sandy material from Wichita Falls. For this material, the same parameters studied before (strength, final moisture content, seismic modulus and dielectric constants) are also used for comparison. Specimens prepared with 3% cement (obtained from mix design) are evaluated in this section. TxDOT specifications for soil-cement materials (Tex-120-E) do not contain a moisture conditioning process. Additional specimens were prepared and subjected to moisture conditioning similar to Tex-121-E. This process defined as Moisture Conditioned in Chapter Three is called “Modified Tex-120-E” in this section. All specimens were prepared with kneading compaction since static compaction is not very feasible for sandy materials.

Figure 4.33 shows strengths after moisture conditioning was achieved with all methods attempted. Tex-120-E and TST yielded comparable results. The very high strength obtained with the submerged specimens after 4 hours is unanticipated and may be attributed to experimental error. The specimens submerged for 24 hours disintegrated during testing. Backpressure and vacuum moisture conditioning methods resulted in similar strengths in all cases but smaller than those obtained with Tex-120-E.

The pattern in the strength can perhaps be explained by the variations in moisture contents as shown in Figure 4.34. The initial moisture contents after compaction for all specimens were close to 9%. The final moisture contents varied from 7.3% (for Tex-120-E) to 12.7% (for the vacuum conditioned specimens). Because of the high permeability of the sandy materials, the vacuum and backpressure methods inject more moisture into the specimens, and hence lower final strengths for these moisture conditioning methods.

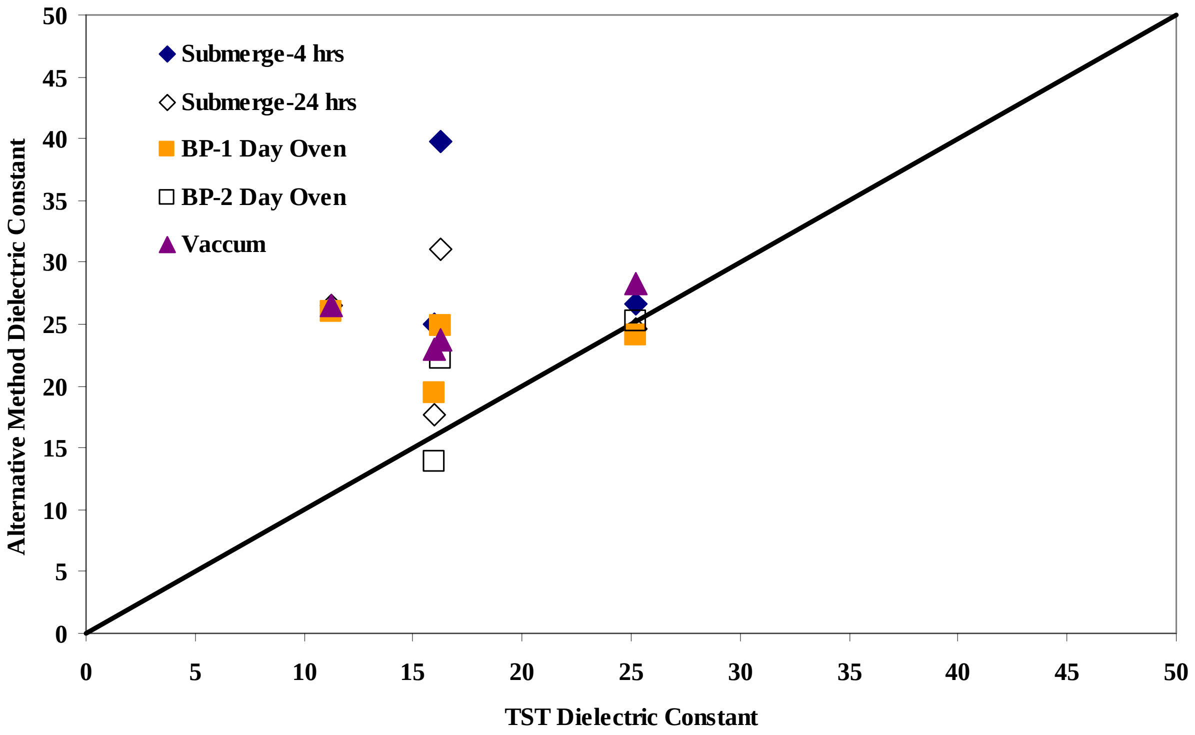


a) Kneading Compaction

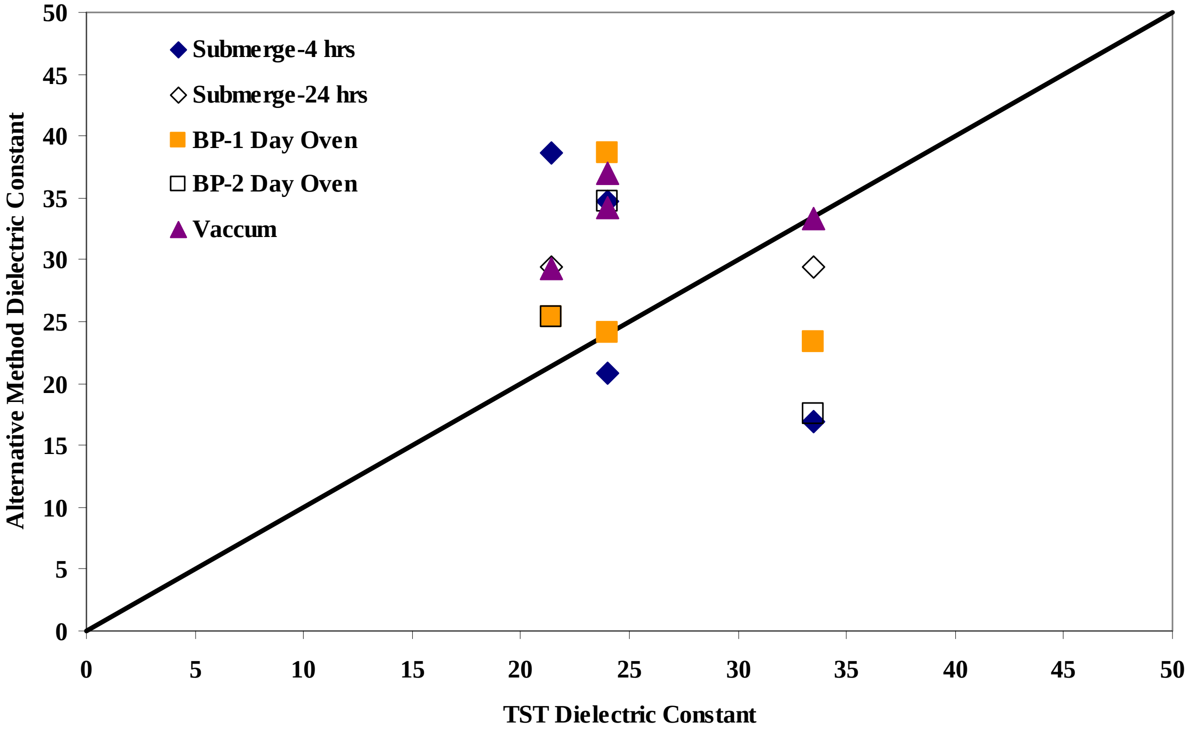


b) Static Compaction

Figure 4.31 – Comparison of Moduli from TST Protocol with Other Moisture Conditioning Methods



a) Kneading Compaction



b) Static Compaction

Figure 4.32 – Comparison of Dielectric Constants from TST Protocol with Other Moisture Conditioning Methods

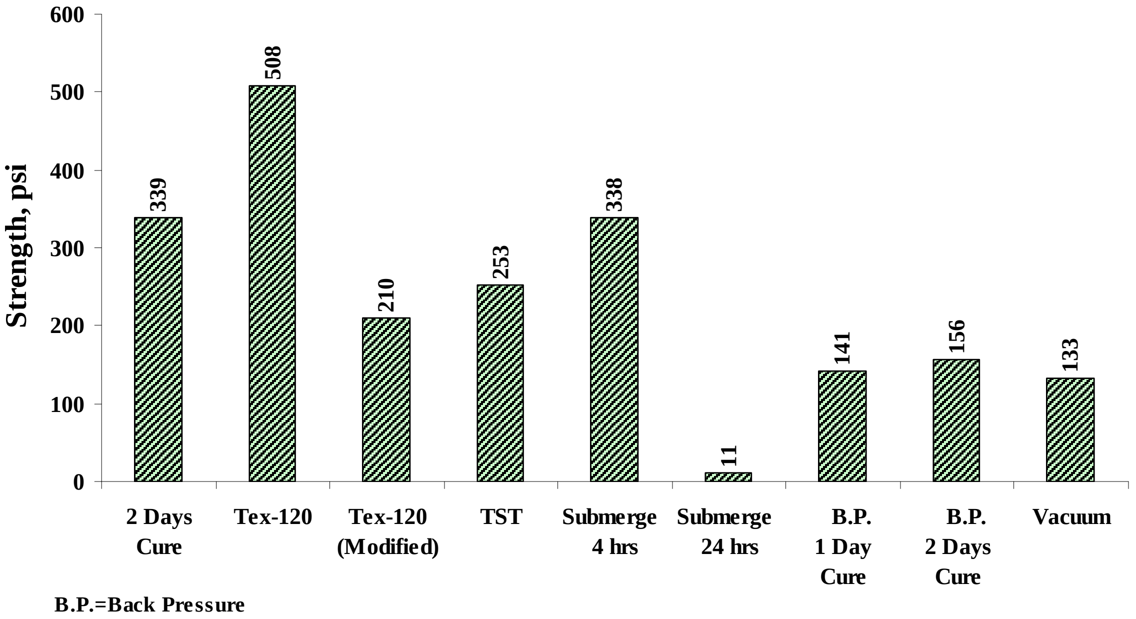


Figure 4.33 – Strength Results of Moisture Conditioning Tests for Wichita Falls Material

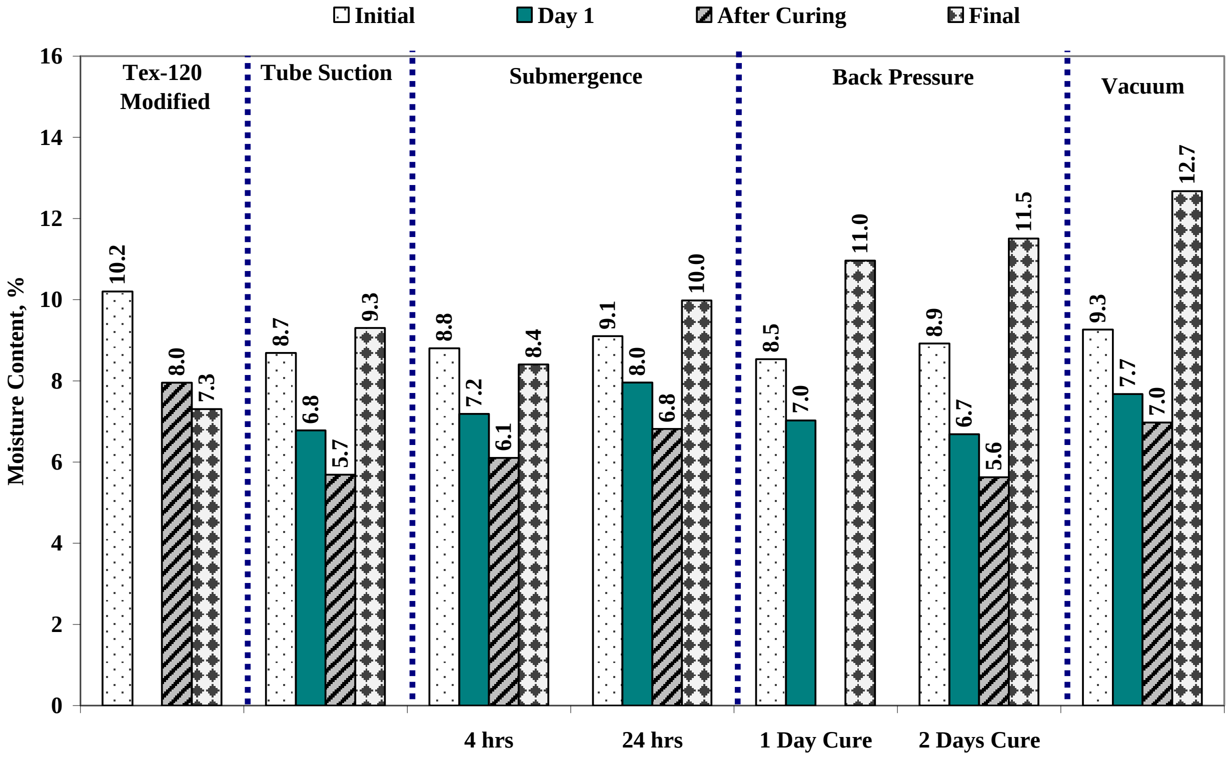


Figure 4.34 – Moisture Content Results of Moisture Conditioning Tests for Wichita Falls Material

The variations in seismic modulus for the soil-cement specimens are shown in Figure 4.35. The initial moduli after compaction were between 51 ksi and 85 ksi. The highest value was obtained for Tex-120-E due to the different curing process performed in that case.

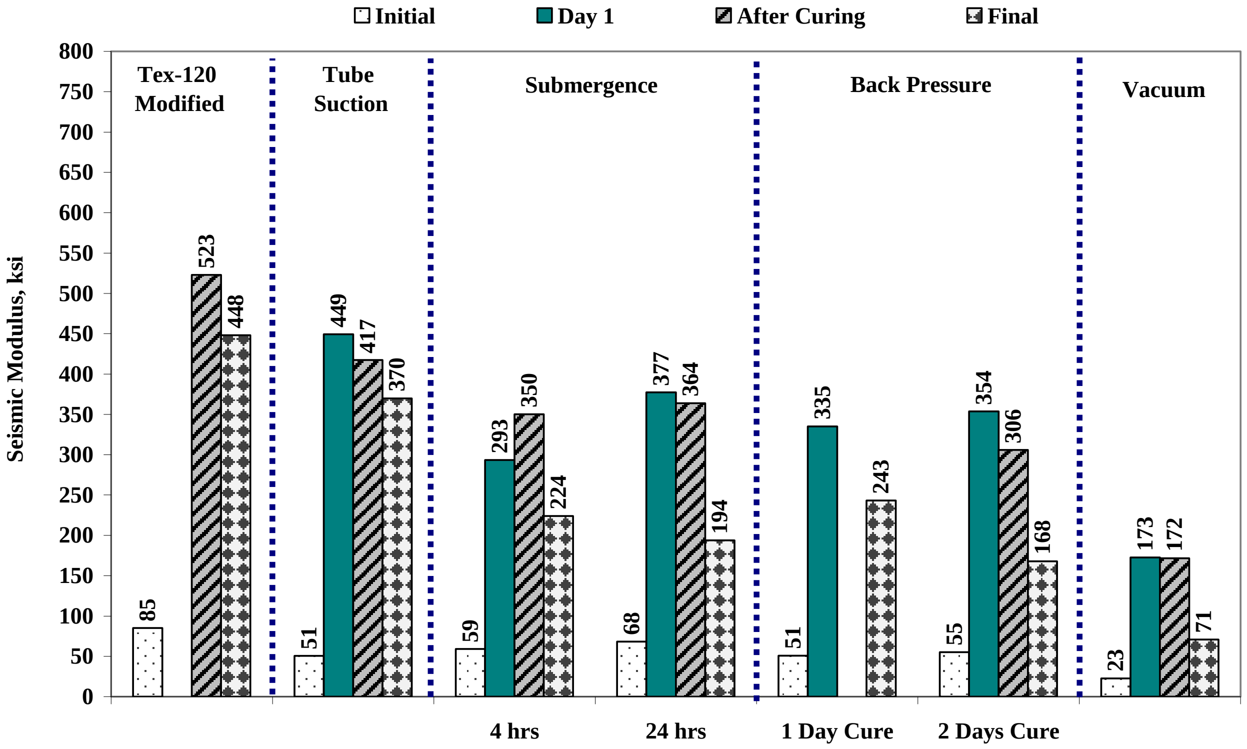


Figure 4.35 – Seismic Modulus Results of Moisture Conditioning Tests for Wichita Falls Material

The retained modulus (defined in Chapter Three) is illustrated in Figure 4.36. The highest retained moduli were obtained from the TST and Tex-120-E specimens, being close to one. For the alternative moisture conditioning methods, retained moduli varied from 0.4 to 0.7. These lower values are more in tune with the results from long-term durability discussed later.

As shown in Table 4.7, the moisture conditioning with backpressure and vacuum methods needed between 2 and 4 hrs to saturate the specimens. The tube suction protocol required more than 72 hrs to achieve a stable value.

Table 4.7– Time to Achieve Saturation for Wichita Falls Material

Method	Time to Saturation, hr
Back Pressure Test-1 Day Curing	2
Back Pressure Test-2 Day Curing	3
Vacuum	4
TST	>72

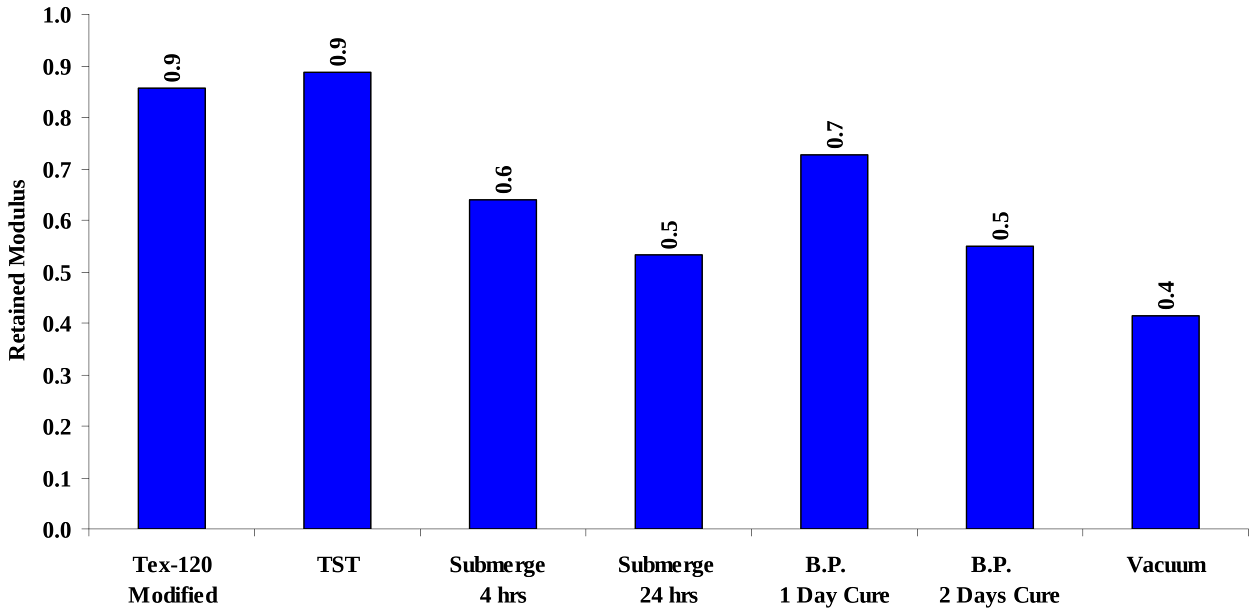


Figure 4.36 – Seismic Moduli Ratios between Specimens after Moisture Conditioning and after Curing Period

MOISTURE CONDITIONING RESULTS OF BRYAN SAND MATERIAL

Figure 4.37 shows strengths after moisture conditioning was completed with all methods. Tex-127-E for 3% fly ash yielded the lowest result with a value of 18 psi. TST and alternative methods resulted in strengths from 30 to 53 psi. For the case of 6% fly-ash, strengths were similar and ranged from 35 to 48. However, strength obtained with the vacuum saturation was 90 psi. Submersion was not attempted for this material, because of the low strengths achieved before moisture conditioning.

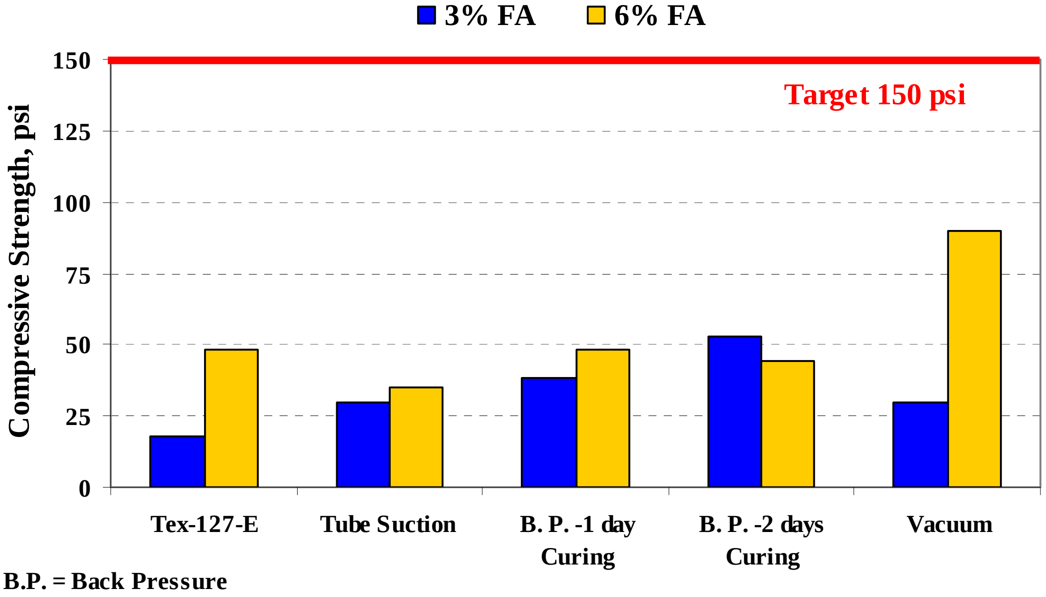


Figure 4.37 – Strength Results of Moisture Conditioning Tests for Bryan Sand Material

The pattern in the strength can be explained by the variations in moisture contents as shown in Figure 4.38. The highest moisture content after saturation corresponded to the Tex-127-E specimens with 3% fly-ash, explaining the low strength observed on these samples.

The variations in seismic modulus for the soil-cement specimens are shown in Figure 4.39. The initial moduli after compaction were between 70 ksi and 113 ksi for 3% fly-ash and from 74ksi and 133 ksi for 6% fly-ash. After the curing process, moduli increased significantly. After moisture conditioning was achieved modulus dropped considerably, because of the moisture increase in the stabilized specimens.

As shown in Figure 4.40, the highest retained moduli were obtained from the vacuum specimens and the lowest for Tex-127-E and TST. For the backpressure methods, retained moduli were about 0.1.

As shown in Table 4.8, Bryan sand specimens required much longer time to achieve saturation than previous materials investigated in this report. Moisture conditioning with backpressure took from 5 to 9 days to achieve complete saturation and 7 days with the vacuum method. Moisture content kept increasing for the tube suction protocol and did not reach a constant value during the capillarity process. For that reason, a time of 8 days is reported as time required for saturation.

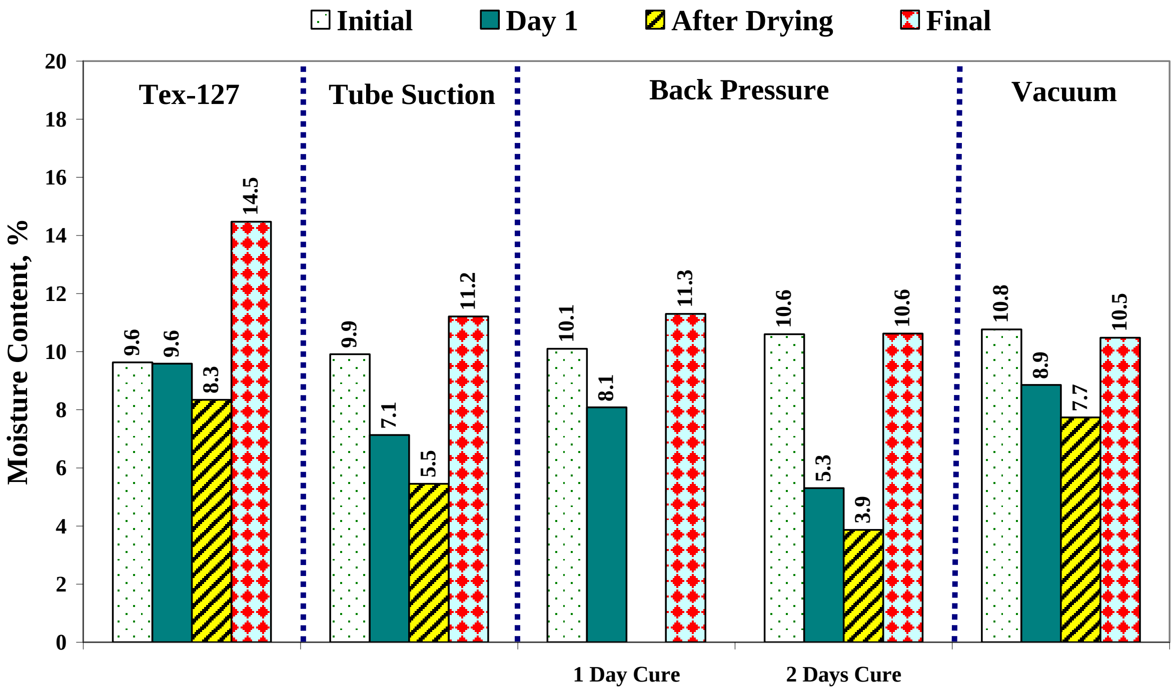
Table 4.8– Time to Achieve Saturation for Bryan Sand Material

Method	Time to Saturation, Days	
	3% Fly Ash	6% Fly Ash
Back Pressure Test-1 Day Curing	5	6
Back Pressure Test-2 Day Curing	9	7
Vacuum	7	7
TST	8	8

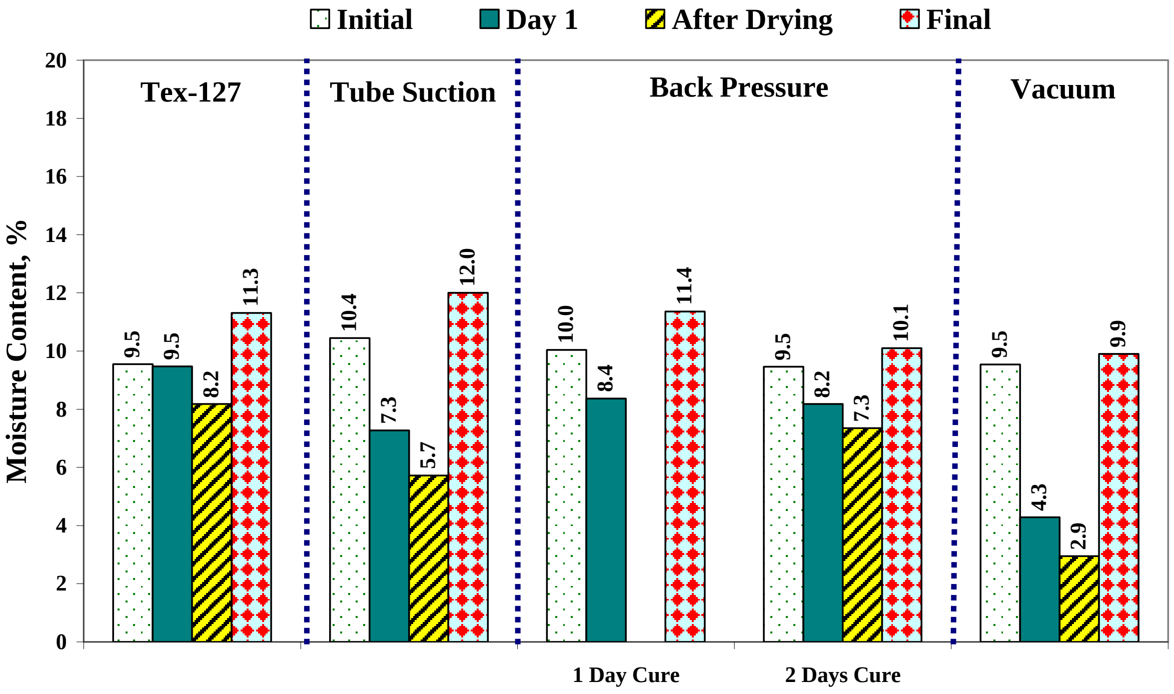
SUMMARY AND RECOMMENDATIONS

The study can be broken down into two aspects: curing and moisture conditioning. The following conclusions can be drawn:

- The TST and current TxDOT moisture conditioning (such as Tex-121-E), did not moisture conditioned the specimens homogenously in most cases. Some of the specimens became super-saturated at the bottom, but remained dry on top.
- The kneading compaction method (such as Tex-114-E) exaggerates the problem with moisture conditioning and curing of specimens. The interface of adjacent lifts act as moisture barriers, and develops a weak plain where the specimens either separate during curing or compression tests.
- The static compaction method in one layer is suggested as an inexpensive alternative. The protocol proposed for this compaction method in Appendix C, provides better control of density and eliminates the problems with the weak planes.

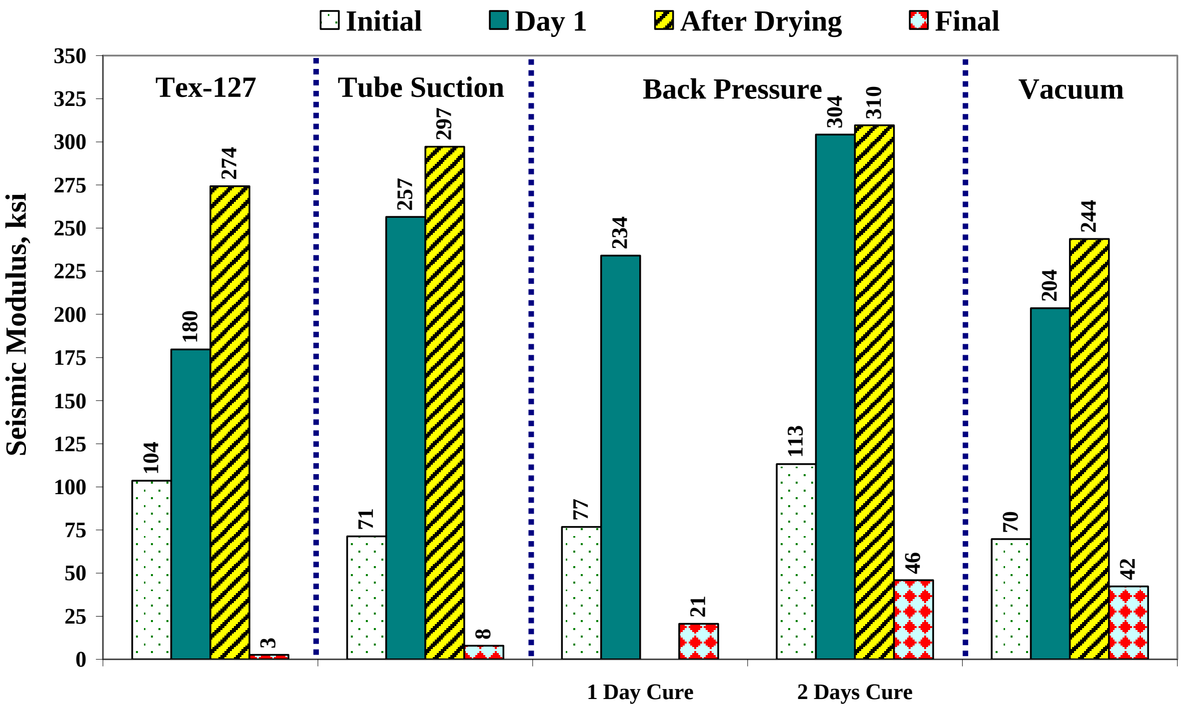


a) 3% Fly Ash

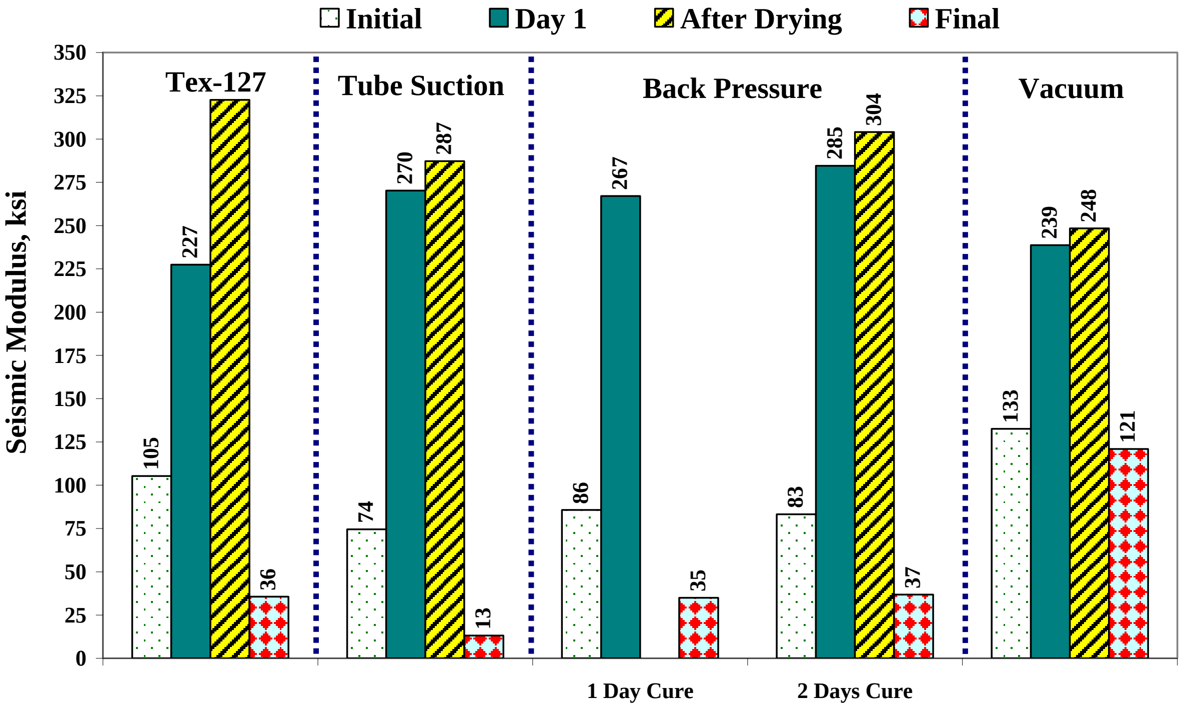


b) 6% Fly Ash

Figure 4.38 – Moisture Content Results of Moisture Conditioning Tests for Bryan Sand Material



a) 3% Fly Ash



b) 6% Fly Ash

Figure 4.39 – Seismic Modulus Results of Moisture Conditioning Tests for Bryan Sand Material

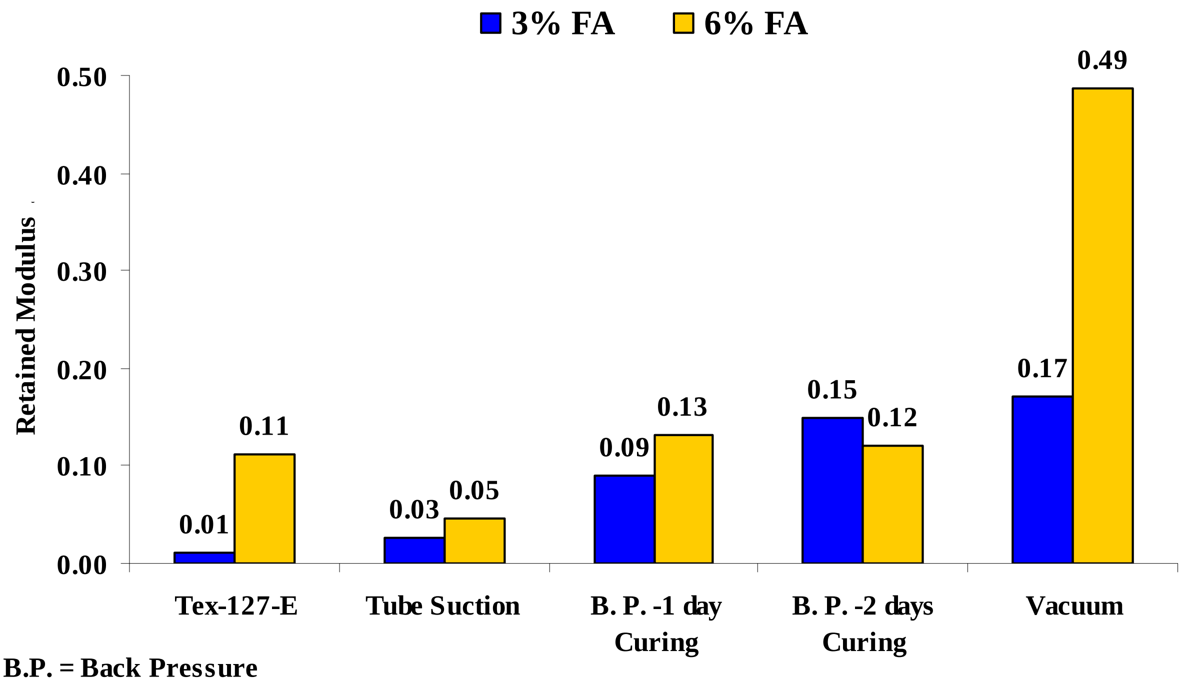


Figure 4.40 – Seismic Moduli Ratios between Specimens after Moisture Conditioning and after Curing Period

- All proposed moisture conditioning methods could moisture condition the specimens in less than 24 hrs for almost all materials except for one material.
- The submergence of the specimens does not seem to be a reliable method. Many specimens disintegrated during moisture conditioning.
- The vacuum conditioning seems to be the fastest method. However, the control of the vacuum suction other than 1 atmosphere is expensive. The vacuum level of 1 atmosphere seems to be too harsh for several specimens.
- The backpressure method is marginally slower than the vacuum method. But the equipment needed is rather simple and it is easier to control the test parameters. In addition, it is easy to obtain additional information such as the permeability of the specimen, and the pH of the water flushed through the specimen.
- The distribution of moisture within the specimen was much more uniform for the vacuum and backpressure methods as compared to the capillary moisture conditioning method currently used.
- The alternative moisture conditioning methods generally resulted in lower strengths than the current specifications Tex-121-E, Tex-120-E and Tex-127-E. This was attributed to the longer curing time imposed on the current specifications.
- When the TST two-day oven curing process was used before the moisture conditioning, the backpressure method provided similar results to the capillary moisture conditioning method when the moisture penetrated throughout the length of the TST specimens.
- The alternative methods resulted in similar or higher final moisture contents than those obtained with the current specifications.

- High temperature curing (140ºF) was found to be detrimental to the strength gain of the lime stabilized specimens. A curing temperature of 104ºF is recommended.
- Tests conducted on specimens prepared at 95% of maximum dry density yielded far less strength, and modulus as compared to specimen compacted to MDD. These specimens were also more permeable and could be moisture conditioned more rapidly than the specimens prepared at MDD.

CHAPTER FIVE – LONG TERM DURABILITY

INTRODUCTION

This chapter mainly focuses on durability and permanence studies on the treated and untreated soils selected in Chapter Three. Two series of tests were conducted to address the permanency and durability of the chemical treatment. The former looks into the leaching problems associated with rainfall infiltration and the latter method addresses the durability of stabilizer by addressing volumetric and strength changes of the soil during wetting and drying cycles. These cycles simulate seasonal changes.

LEACHATE STUDIES

A new test protocol is developed to address the permanency of the chemical stabilization from moisture flows during rainfall events, ground water flows and moisture migration from suction and head differences. This test utilizes a flexible wall mold housing the compacted stabilized soil specimen. Figure 5.1 illustrates a schematic of the test setup used in this research. This

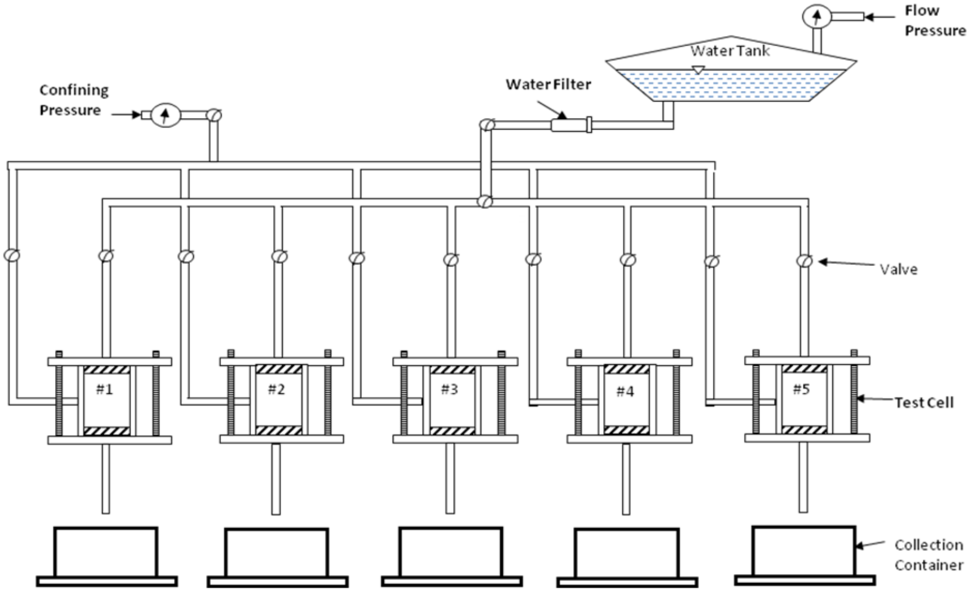


Figure 5.1 – Schematic of the Leachate Process

setup is similar to the one used by McCallister (1990) for leachate studies with the exception of a modification in the size of the soil specimen (6 in. diameter instead of 8 in. diameter). An attempt was made to simulate the flow scenario close to Texas conditions.

The cured soil specimen was subjected to moisture flow from a water tank at a constant pressure. A few preliminary tests were conducted to finalize the pressures to be applied to the water flow. These pressures differed from soil to soil as the ultimate goal is to complete one leaching cycle in one day. One leaching cycle is defined as the amount of leachate volume collected that is equal to one soil specimen's void volume. Specimen void volume can be defined as the total voids/pores (air voids + water voids) present in a compacted specimen. The formulas involved in the calculation of specimen void volume are given in Figure 5.2. In this figure typical calculations for Paris soil sample are presented.

Dry unit weight		$\gamma_d := 90.8\ \text{pcf}$
Unit weight of water		$\gamma_w := 62.4\ \text{pcf}$
Specific Gravity		$G_s := 2.7$
Sample Diameter		$d := 4\text{in}$
Sample height		$h := 6.5\ \text{in}$
Total Volume	$V := \frac{\pi \cdot d^2}{4} \cdot h$	$V = 1.339 \times 10^3\ \text{mL}$
Void Ratio	$e := \frac{\gamma_w \cdot G_s}{\gamma_d} - 1$	$e = 0.856$
Volume of Solids	$V_s := \frac{1}{1+e} \cdot V$	$V_s = 721.376\ \text{mL}$
Pore Volume	$V_v := V - V_s$	$V_v = 617.1\ \text{mL}$

Figure 5.2 – Pore Volume Calculations

The cured samples were kept inside the sample cell and the top plate is secured in place using the fasteners as shown in Figure 5.3. A confining pressure higher than the flow pressure is applied through the confining pressure inlet as shown in Figure 5.4. Then water is allowed to go through the top under a constant flow pressure through the flow pressure inlet and the leachate is collected in the 20 liter carboys shown in the photograph.

Leachate tests were conducted on several identically prepared and cured soil specimens. Leachate samples were collected after 3, 5, 7, 11, and 14 cycles of leaching, while the UCS tests were conducted on soil samples after 3, 7 and 14 cycles of leaching. Leachate samples collected were tested for 'pH' changes and 'amount of calcium' present after the corresponding leachate cycles. Results were statistically analyzed to address the loss of stabilizer due to leaching. An attempt was made to correlate leaching cycles with field moisture movements from rainfall events. Results are presented next.

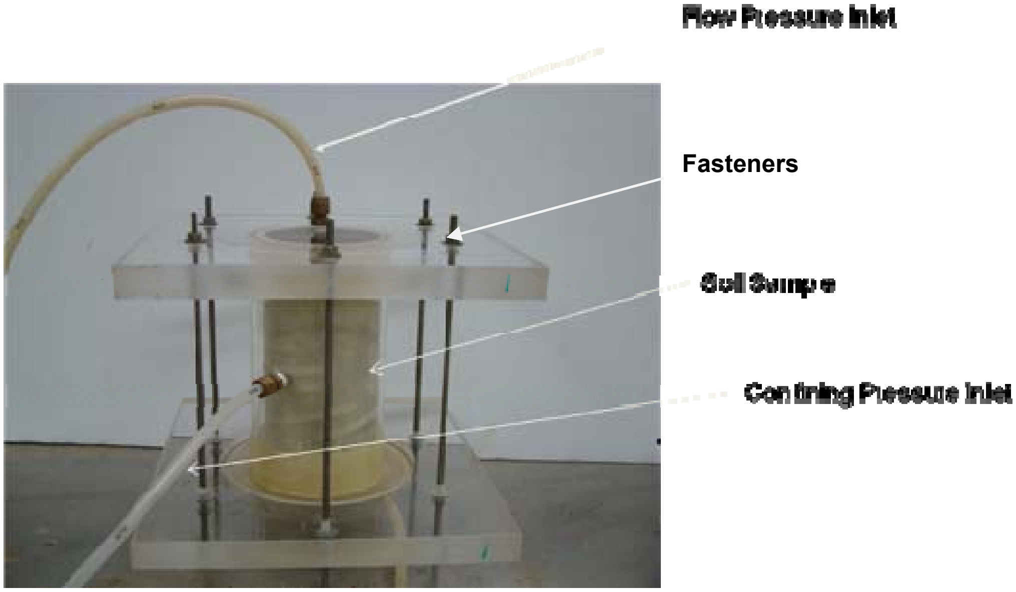


Figure 5.3 – Leachate Cell

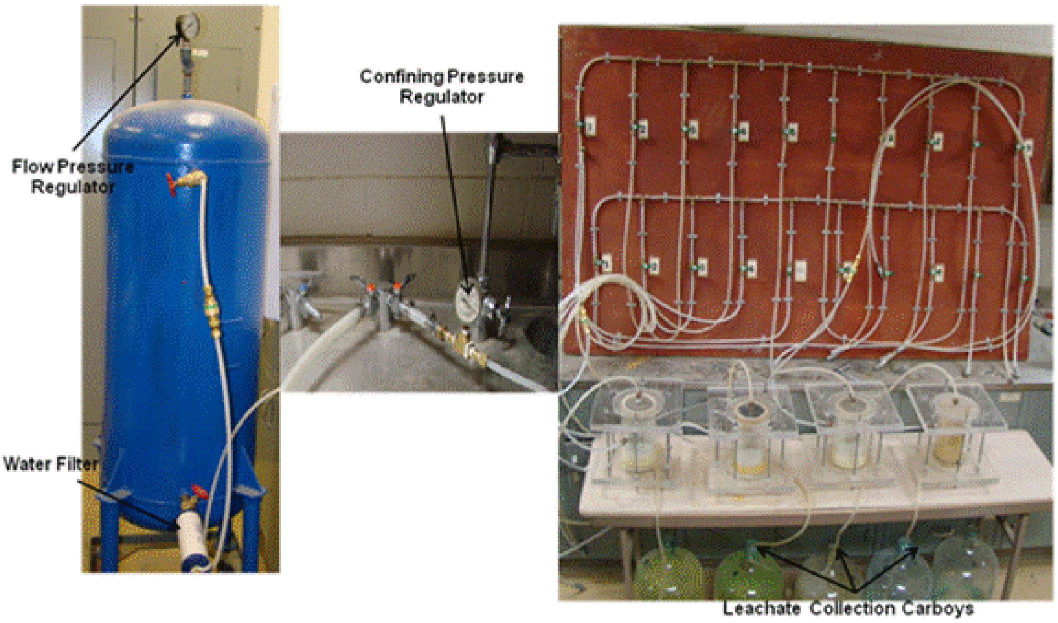


Figure 5.4 – Apparatus Used to Conduct Leachate Studies

The variations in the calcium ion concentration and the number of leaching cycles for different soils are shown in Figure 5.5. There is a decrease in the calcium ion concentration after 14 cycles of leaching in all soil samples, but the decline is not considerable. The amount of calcium ions leached out after 14 cycles, converted into percentage of lime are given in Table 5.1. The lime leached out of the soil is small (less than 0.5%), and therefore the strength drop is not considerable. The percentage of lime and cement are obtained from the calcium ion concentration with the help of the calibration curves presented in Figures 5.6 and 5.7, respectively. These curves were obtained by mixing a known concentration (ppm) of lime or cement in distilled water and then measuring the amount of calcium ions in the solution.

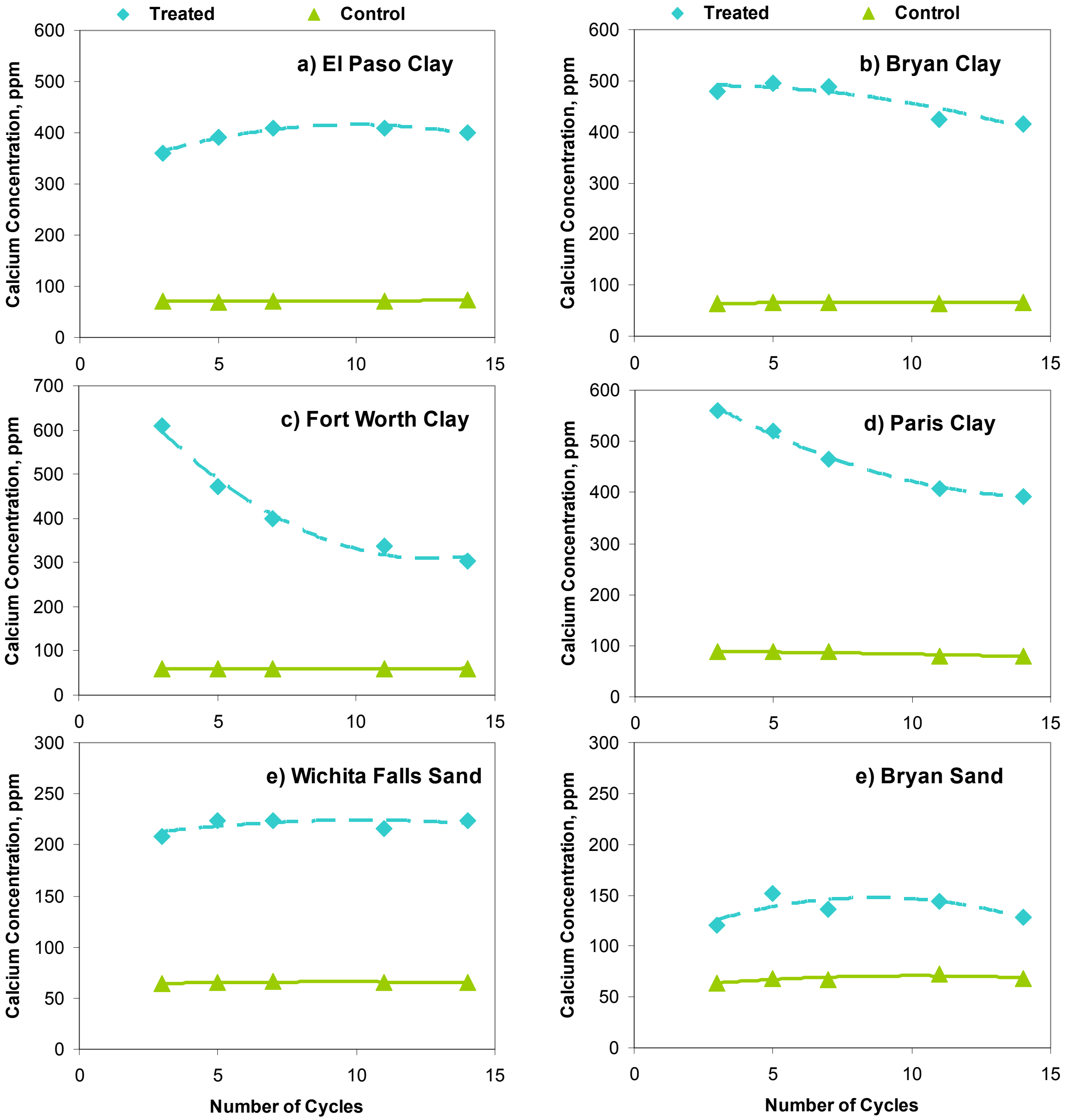


Figure 5.5 – Variation of Calcium Concentration of Different Soil Samples

Table 5.1 – Additive Leached and Strength Loss after 14 Cycles of Leaching

Soil Type	% Additive Leached	% Strength Loss
Bryan	0.39	6
El Paso	0.24	3
Fort Worth	0.51	5
Paris	0.26	2
Wichita Falls	0.24	6
Bryan Sand	0.35	9

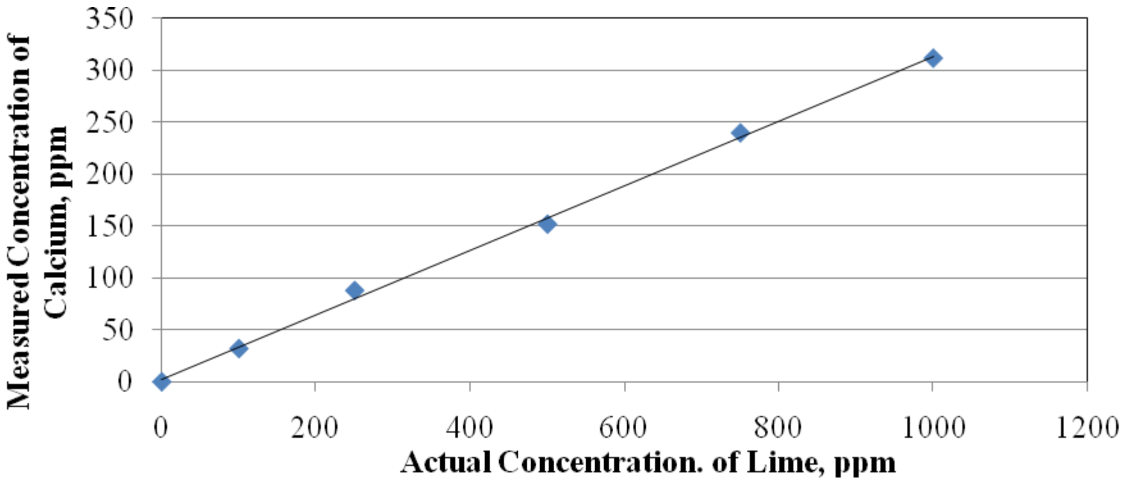


Figure 5.6 – Calibration Curve for Obtaining the Concentration of Lime from Concentration of Ca

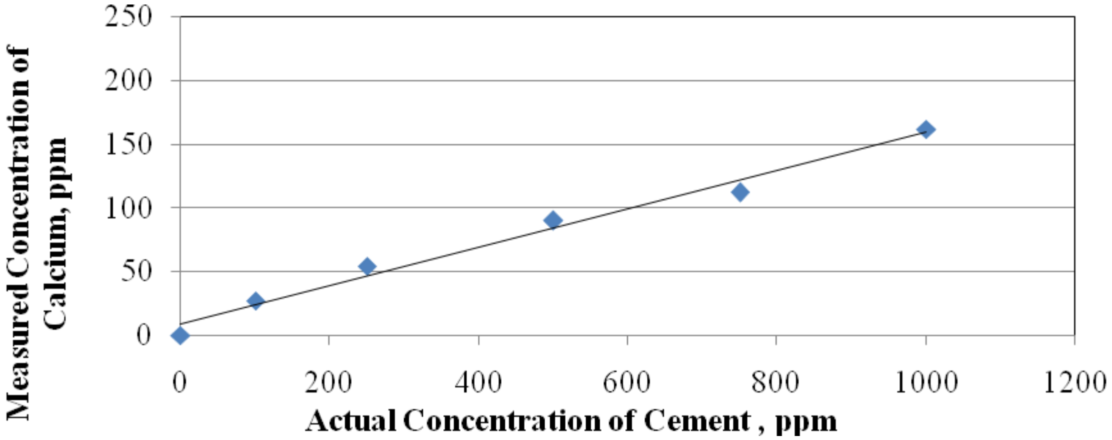


Figure 5.7 – Calibration Curve for Obtaining the Concentration of Cement from Concentration of Ca

Figure 5.8 shows the variations in the pH with the leaching cycles. The changes in the pH with respect to leaching of the stabilizer are rather small. This is expected since the pH of a treated soil can still be the same at both the low and high dosages. As a result, the pH variations with leaching cycles cannot be correlated to the loss of stabilizers.

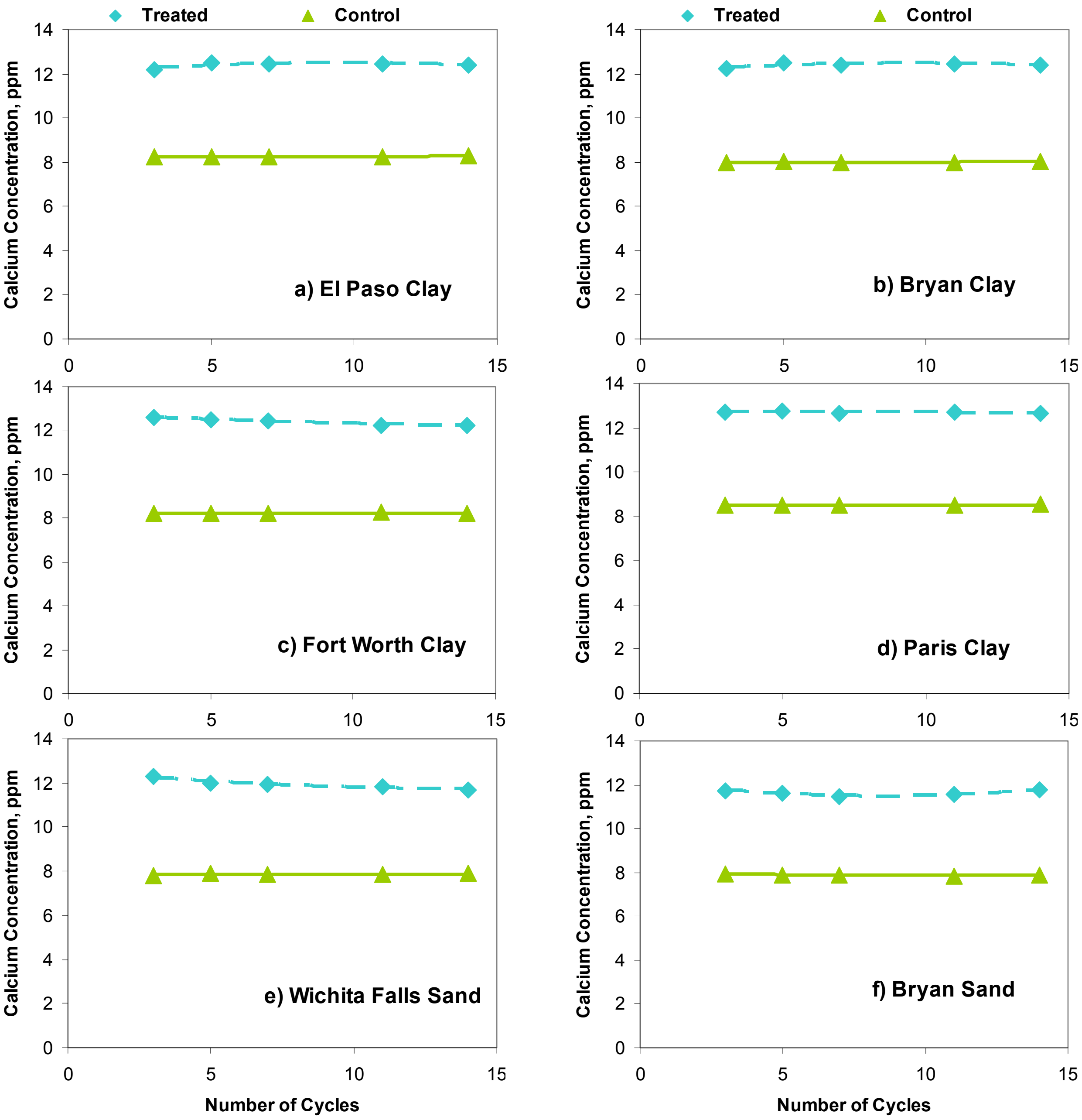


Figure 5.8 – Variation of pH with the Leachate Cycles of Different Soil Samples

The UCS test results after 3, 7 and 14 cycles for all soils are given in Figure 5.9. Overall, the strength loss is rather small after 14 cycles of leaching. The small decrease in strength can be explained by relating the number of cycles of laboratory tests to the field moisture infiltration amount. The infiltration capacity of the soil depends on its permeability and soil moisture content (previous rainfall or dry season). The initial capacity (of a dry soil) is high but, as the storm continues, it decreases until it reaches a steady value termed as final infiltration rate. In the current analysis average infiltration rates are assumed in lieu of a detailed analysis. The field infiltration is always a very small part of the total rainfall. The main sources of infiltration are surface infiltration, groundwater seepage and ice lenses melt water (Huang, 2004).

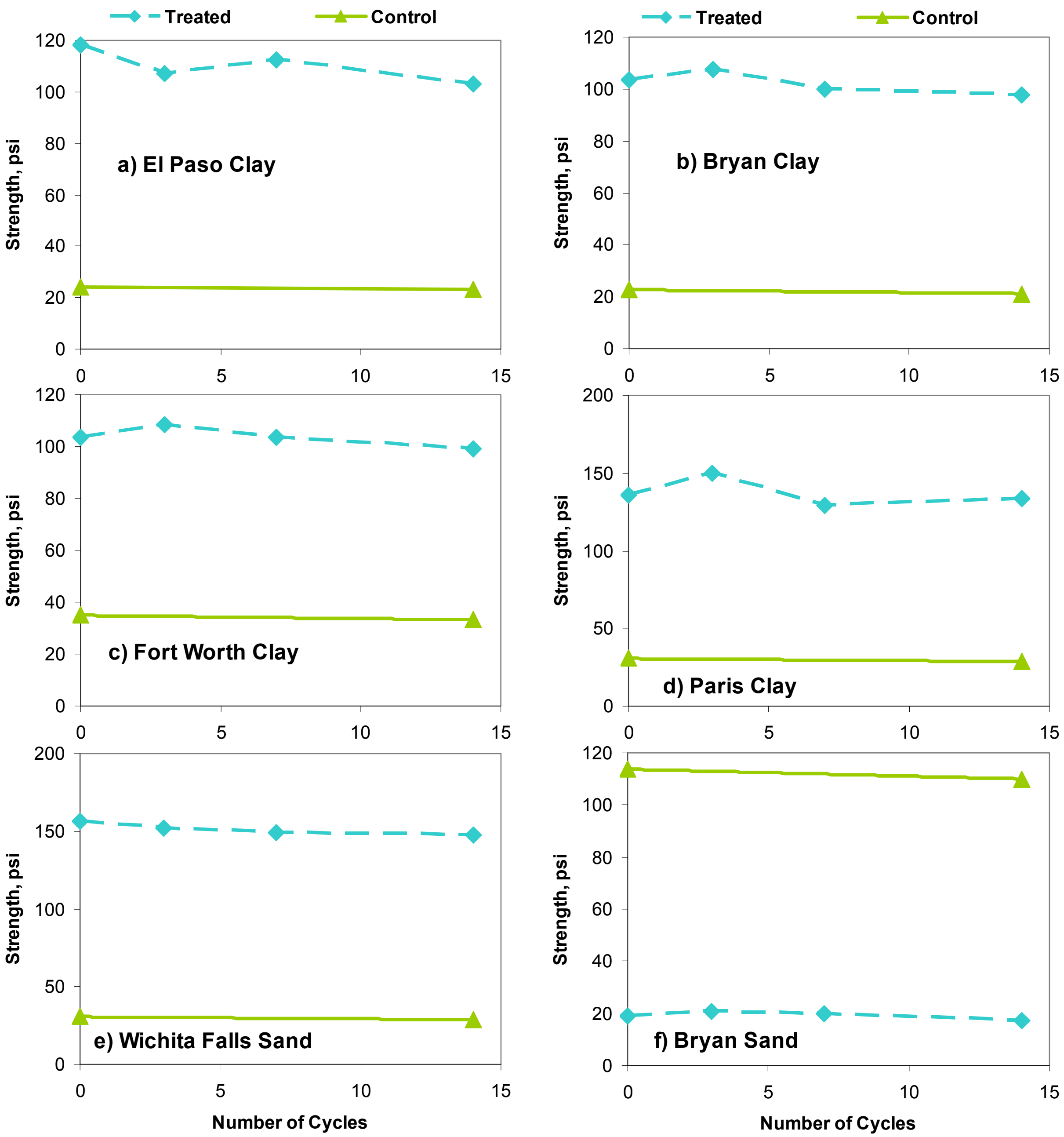


Figure 5.9 – Variation of UCS Strength with the Number of Leaching Cycles for Different Soil Samples

For this purpose the following analysis was conducted by assuming a percentage of annual average rainfall as the infiltration amount in a year. According to the rainfall data given by Texas precipitation records the average annual rainfall for the state of Texas is 30 in./year. Out of this total rainfall only 35% is assumed to infiltrate into subsoils. This number is very conservative and is considered to make a safe estimate of the number of years replicated in the infiltration studies in the laboratory conditions. The detailed procedure is given in Figure 5.10.

Field Infiltration

Average AnuaRainfall in Texas , $AAR := 30\text{in}$ (Texas Precipitation Records)

Approximate % of infiltration, $\%I := 35\%$

Rain water infeltered per year, $I_{field} := AAR \cdot \%I$ $I_{field} = 10.5\text{in}$

Laboratory Infiltration

Diameter of Laboratory section $d := 4\text{in}$

Cross Sectional Area $A := \frac{\pi \cdot d^2}{4}$ $A = 81.1\text{cm}^2$

Volume of water Infiltration for 1 day (Depends on pore volume) $I_l := 591.7\text{ml}$

Volume of water Infiltration for 14 days $I_{14} := I_l \cdot 14$ $I_{14} = 8283.8\text{cm}^3$

Converting Volume to Head of water $I_{lab} := \frac{I_{14}}{A}$ $I_{lab} = 40.2\text{in}$

\# of years replicated in the lab $n := \frac{I_{lab}}{I_{field}}$ $n = 3.8$ yrs

Figure 5.10 – Calculations to Relate the Laboratory Leachate Studies to Equivalent Number of Storm Events

Following the method shown in Figure 5.10, the number of years of field infiltration replicated in the laboratory is presented in Table 5.2. The number of field years replicated in all soils is less than 4 years, which is a short duration compared to the life of the pavement. Considering that there is about 0.5% leaching of stabilizer in the first four years, it is likely that with more number of leachate cycles, the percent leachate of stabilizer could be higher and may exceed 2% in ten to fifteen years. The percent leaching of lime in the present leachate studies is small, but can be considerable if the original stabilizer dosage used is less than 4% or more leachate cycles (greater than 14) are planned. Overall, this study showed that leaching may not be a highly problematic in the initial years if the original treatment dosages are high (6% or high).

Table 5.2 – Number of Years in the Field Replicated in the Laboratory

Soil No.	Soil Type	% Lime Leached	% Strength Retained	Number of years in the field replicated
1	Bryan	0.40	89	3.6
2	El Paso	0.34	88	2.9
3	Fort Worth	0.31	97	4.0
4	Paris	0.54	85	3.9
5	Wichita Falls	0.34	94	2.7
6	Bryan Sand	0.35	91	2.2

DURABILITY STUDIES (WETTING-DRYING CYCLES)

Another important test to address the durability of chemical treated soils is to subject them to various cycles of wetting and drying. During these processes, both the volume change and soil strength and stiffness can be determined. These properties will provide insights into the effects of seasonal moisture fluctuations on the soil property variations. ASTM D 559 is the standard method often used for this purpose.

The procedure outlined by ASTM D 559 method is followed to reflect both wet and dry cycle conditions close to Texas conditions in a reasonably short time period. According to the ASTM D 559 the soil samples should be prepared and cured, submerged in water for 5 hrs, and then oven dried at 160ºF (70°C) for 48 hrs. After removal from the oven, the specimen is subjected to volume change and moisture content measurements. The test is then continued until 21 wet-dry cycles are completed or until the sample fails.

An example of the test setup is shown in Figure 5.11. During wetting and drying periods sample size changes are measured in two dimensions. The vertical movement is measured with a dial gauge and the radial movements are measured using a “pi tape”. The wetting/drying was continued for 21 cycles. After 3, 7, 14 and 21 cycles the samples were subjected to UCS tests. This test procedure may take more than a month to determine if the additive is durable.

a) Wetting **b) Drying**

Figure 5.11 – Setup Used for the Wet/dry Studies

Figures 5.12 and 5.13 are the pictorial representation of the behaviors of untreated and treated soil samples (Paris soil) with different wetting and drying cycles. The untreated Paris soil sample failed after only one W/D cycle, while the treated soil failed after 7 W/D cycles. The treated Paris soil also showed tremendous decrease in strength even after treatment with lime. Similar behavior was observed for the Fort Worth clay soils which survived for 11 W/D cycles. The clay samples from Bryan and El Paso endured 21 cycles with small strength decrease.

After Curing After 1 Wetting Cycle After 1 Drying Cycle After 2 Wetting Cycles

Figure 5.12 – Changes in Paris Soil Sample (Untreated) with Wetting/Drying Cycles

After Curing After 3 Cycles After 5 Cycles After 7 Cycles

Figure 5.13 – Changes in Paris Soil Sample (Treated) with Wetting/Drying Cycles

The volumetric changes for all soils studied during these tests are given in Figure 5.14. The volumetric swelling in the untreated Paris soil is considerable (> 10 %) after one cycle of W/D. The swelling in the treated soil is almost equal to that of untreated soil after 7 cycles, implying that the swelling behavior of a treated soil after seven W/D cycles will be similar to that of an untreated soil. Similar trends were observed for the other materials. In Figure 5.14e, the volumetric changes in the untreated and treated (treatment with cement) sand from Wichita Falls are also depicted. Little swelling in the untreated or treated soil is observed.

Figure 5.15 shows the loss of strength in the untreated and treated soils. The Paris soil sample (Figure 5.15d) did not survive the 21 cycles. The strength loss is drastic from the 3rd to the 7th cycles, and after that the sample lost its integrity and collapsed. Though the Wichita Falls sand lasted for the 21 cycles, significant strength loss is observed between the cycles. The clays from El Paso and Bryan did not show considerable swelling, and the strength drop was not significant, while the clay from Fort Worth lasted for 11 cycles after which it had a similar behavior as Paris clay.

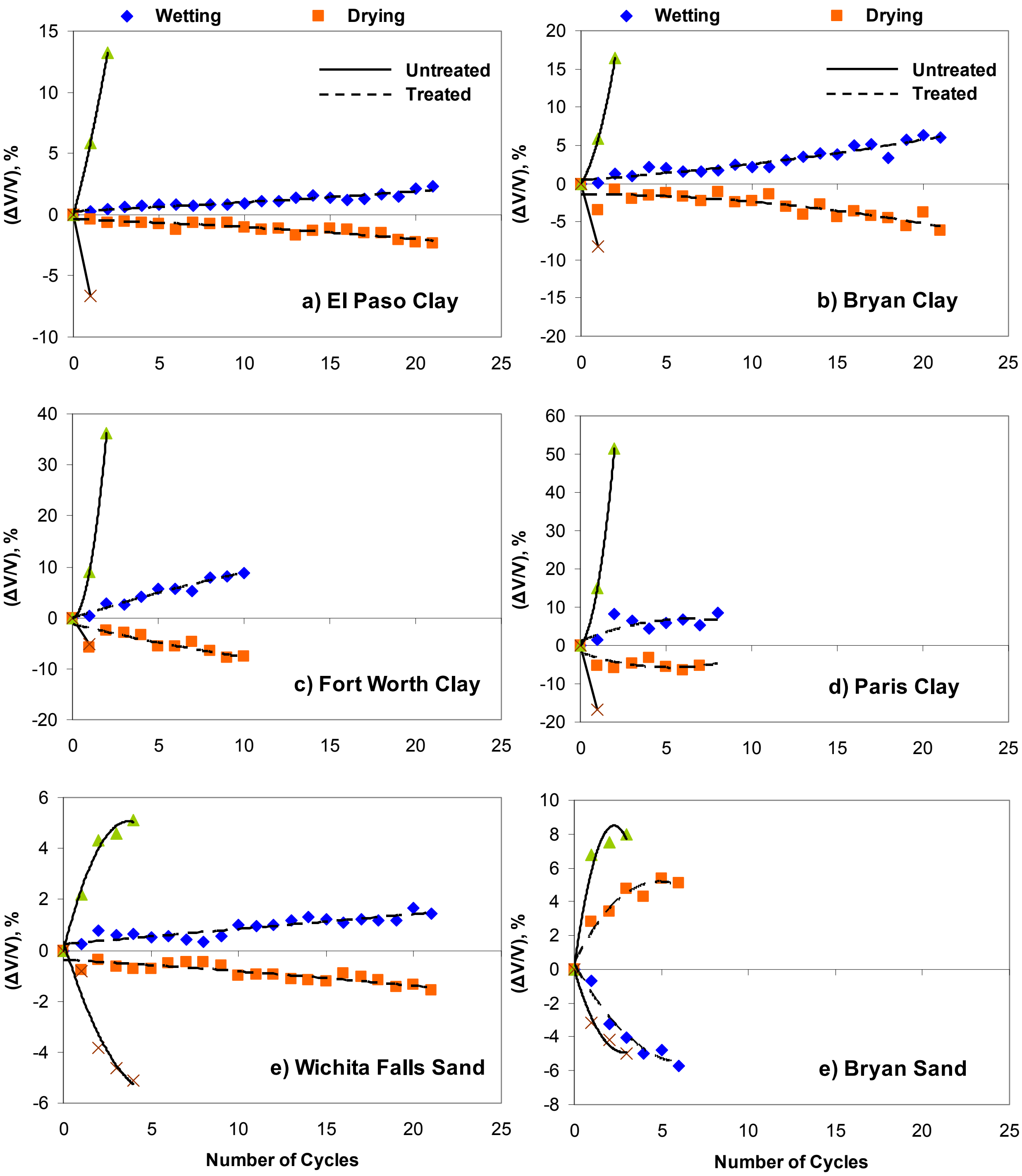


Figure 5.14 – Volumetric Changes in Different Soil Samples with Wetting/Drying Cycles

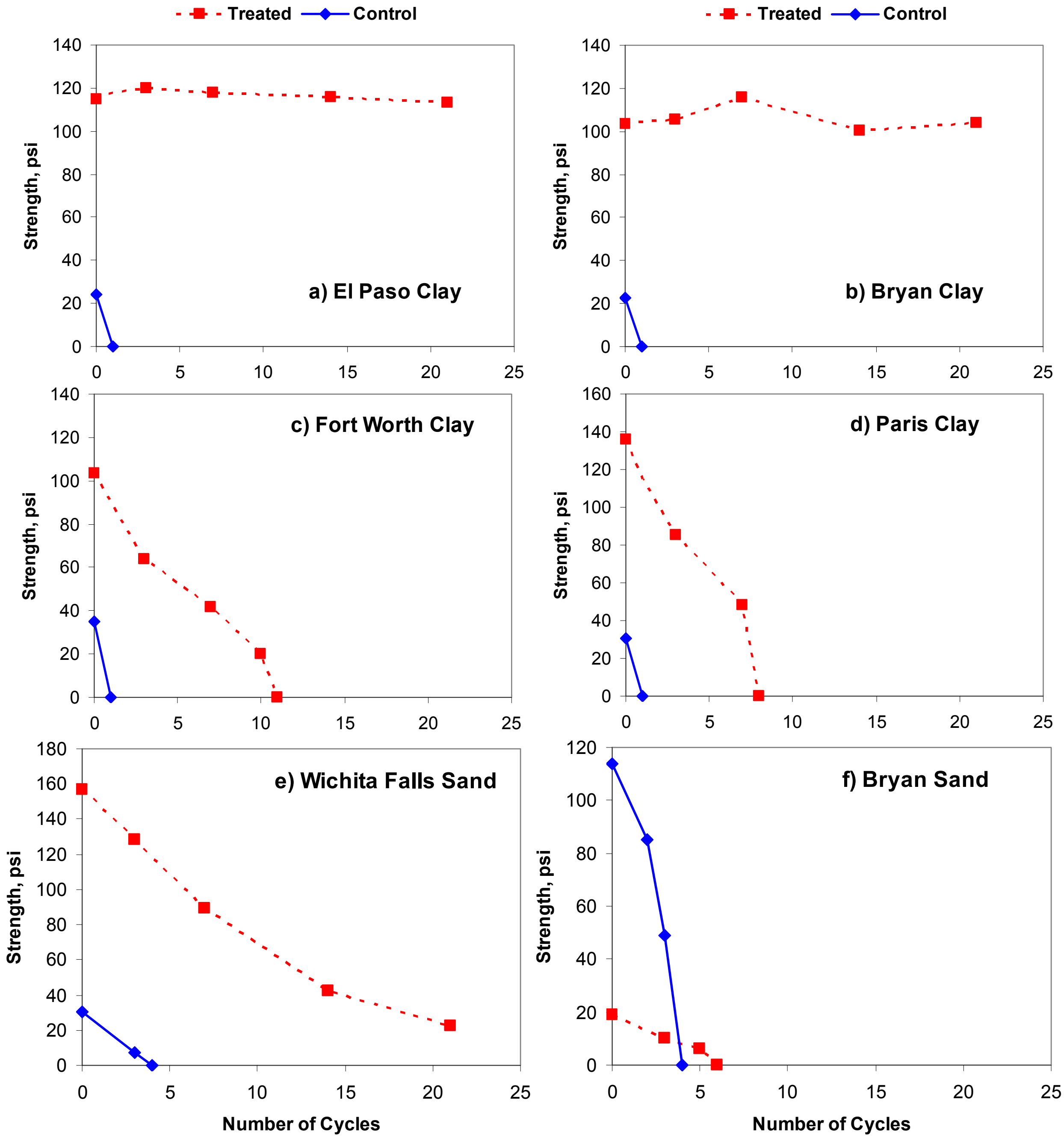


Figure 5.15 – UCS Strengths after Different Cycles of Wetting/drying for Different Soil Samples

Table 5.3 summarizes the percent strength loss with the number of wetting/drying cycles for different soils studied. The reason for the diverse behaviors among soils can be attributed to the dominance of the mineral montmorillonite in the clay fraction of the soils. Soils from Paris and Fort Worth are predominant with montmorillonite clay mineral, and hence they did not survive all the 21 W/D cycles. Soils from El Paso, Wichita Falls and Bryan are not dominated by montmorillonite mineral, and hence they survived all the 21 cycles. It can be concluded that soils containing montmorillonite as a dominant mineral are more susceptible to premature failures after chemical stabilization when they are exposed to swell and shrink related volume changes.

Table 5.3 – Dominant Clay Material, Cycles Survived and Drop in Strength of Different Soil Samples

Soil Name	Dominating Clay Mineral	% Montmorillonite	Additive type	No. of Cycles Sample Survived	% Loss in Strength
Bryan Clay	Kaolinite	28	Lime	21	29
El Paso	Illite	8	Lime	21	12
Fort Worth	Montmorillonite	50	Lime	10	100
Paris	Montmorillonite	70	Lime	7	100
Wichita Falls	Quartz, and Illite	18	Cement	21	86
Bryan Sand	Kaolinite	39	Fly-ash	4	100

The variations in the percent strength loss and percent montmorillonite mineral after different numbers of wetting/drying cycles are shown in Figure 5.16. As the percent montmorillonite increases, the loss of strength increases. This is an important finding as it shows the influence of clay mineralogy on the durability of stabilizer. The current approach of PI based chemical stabilizer does not provide any insights into the chemical stabilizer and its durability.

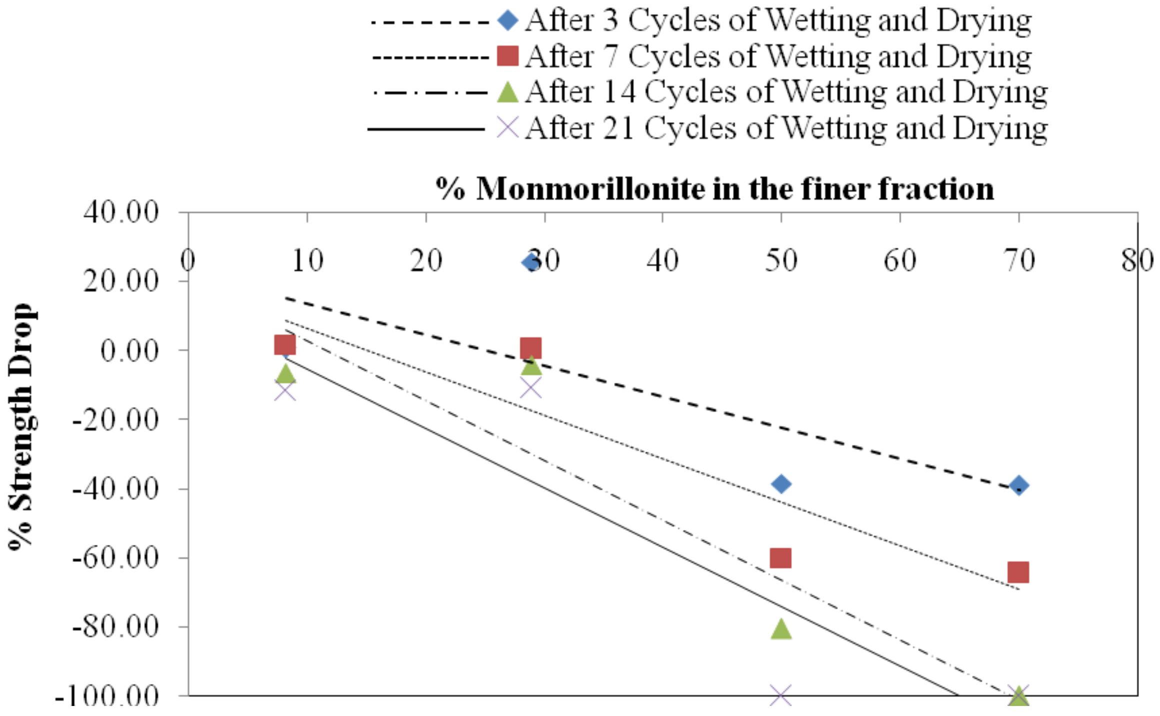


Figure 5.16 – Percentage Drop in Strength Variation with Percentage of Montmorillonite in Different Soils for Wetting Drying Cycles

SUMMARY

Two series of moisture conditioning tests were conducted to address the permanency and leachability of the chemical treatments. The first method looks into the leaching problems associated with rainfall infiltration and the second method addresses the volumetric and strength changes of the soil during wetting and drying cycles from seasonal changes.

The leachate samples collected were tested for pH and the presence of calcium ions. There is a decrease in the calcium ion concentration after 14 cycles of leaching in all soil samples, but the decrease is usually small. The changes in pH were rather small as well. The strength loss after 14 cycles of leaching was small as well. The leachate studies indicate that leaching in clay occurs, however the leach rate is low and may not be problematic for initial time periods following chemical stabilization.

Wetting/Drying studies were also conducted. Results from the Paris clay soil samples show that untreated soils lasted for only 1 W/D cycle while treated soils lasted for 7 cycles. Similar behavior was observed for Fort Worth clay soils which survived for 11 W/D cycles. The volumetric swelling in the untreated Paris soil was considerable (> 10 %) after 1 complete cycle of W/D. The strength drop of the untreated and treated Paris and Fort Worth soils showed different strength decrease though the PI values of these soils are almost similar. The main reason for this diverse behavior can be attributed to the dominance of the clay mineral (montmorillonite) in these soils. Overall, it can be concluded that stabilized soils with montmorillonite as a dominant mineral are more susceptible to durability problems in particular when these soils are exposed to volume changes caused by swell and shrink related volume changes from durability type studies.

The current approach of PI based chemical stabilizer has not shown any insights into the chemical stabilizer and its durability. Hence, the proposed research where clay mineralogy aspects are included, stabilizer design will be more realistic and provide better answers to the permanency of the stabilizer.

CHAPTER SIX – FIELD MONITORING TO VALIDATE PROPOSED GUIDELINES

INTRODUCTION

This chapter presents the results obtained from the field monitoring to validate the preliminary guidelines. The chapter contains the laboratory and field results of a total of six soils obtained from different construction projects within Texas, as detailed in Figure 6.1. The construction sites were located in the following locations:

- Austin Clay subgrade from SH 45
- Keller Clay subgrade from US 377
- 2 Pharr Clays (one high and one medium PI) from FM 511
- Amarillo Clay from US 60
- Wichita Falls Sand from US 277

To implement the field validation process, the mix design provided by TxDOT personnel was evaluated first for each material. Same laboratory tests to characterize the baseline materials (see Chapter Three) were conducted initially only for the amount of stabilizer used in the field. When minimum strength requirements were not achieved, additional tests were conducted with higher concentrations of stabilizer. The comparison between the results from the laboratory tests based on current TxDOT specifications and those proposed as new moisture conditioning methods were also conducted.

Comprehensive information related to laboratory results for sandy material from Wichita Falls was provided in Chapter Three and moisture conditioning results were included in Chapter Four. Laboratory results from other materials are discussed here.

To validate the results, a series of field tests were carried out to characterize the newly constructed subgrades. PSPA testing for the determination of the in-situ subgrade moduli was conducted at a number of stations. Also, nuclear density gauge (NDG) was used to estimate the in-situ dry densities and moisture contents for some of sites mentioned above.

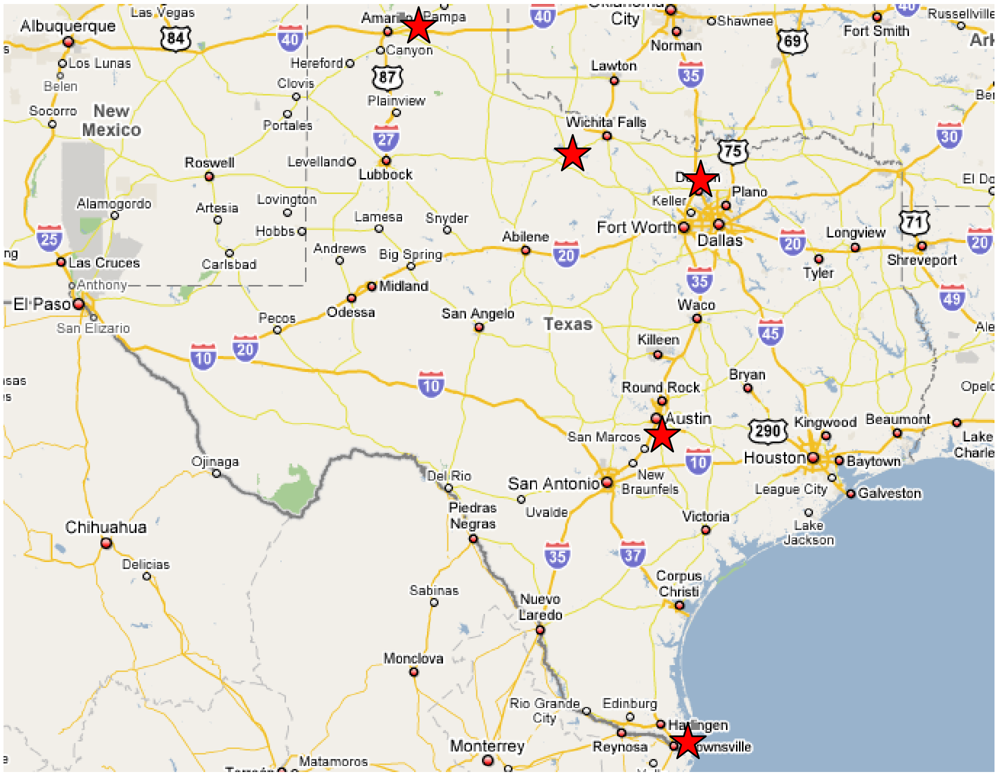


Figure 6.1 – Overall View of Sites Location

Description of Sites

Austin Site

This site was located on the State Highway 45, south of Austin, in Travis County. The section investigated for this site extended from Stations 1043 to Station 1051 of the westbound inside lane. The soil investigated from this project consisted of the top 10-inch in-place lime-treated subgrade.

Keller Site

This site was located close to Keller on the US Highway 377, north of Fort Worth, in Tarrant County. The section investigated stretched from Stations 36 to Station 56 of the southbound outside lane. The lime-stabilized layer was nominally 8 in. thick.

Wichita Falls Site

This site was located on the US Highway 277, southwest of Wichita Falls, in Baylor County. The section investigated was located in the northbound inside lane from Station 860 to 871. The cement-treated subgrade was 8 inch thick.

Pharr Site

This site was located on the FM 511, southeast of Olmito, in Cameron County. Only areas with the medium PI material were investigated, since the sections with high PI material were already covered by a base layer. Three different sections were investigated at different times of curing: 1, 5 and 10 days after compaction. The sections that were cured for 24 hours and 10 days were located on the inside lane of the westbound, between stations C-0-60 and C-1-20, and C-0-30 and C-0-100, respectively. The section that cured for 5 days was located on the outside lane of the eastbound between stations C 2+00 and C 0+60. The lime-treated subgrade was 10 in. thick.

Amarillo Site

This site was located on the US 60, northeast of Amarillo, in Carson County. The stabilized subgrade comprised of the approach to the bridge overpass on the FM 2373. Raw material was retrieved from the site to validate the TxDOT mix design. However, no field testing was conducted on this section, because the fly ash stabilization was canceled for this project.

ASSESING OF MIX DESIGNS BASED ON CURRENT TXDOT SPECIFICATIONS

The type and content of additives used on each site are shown in Table 6.1. The minimum amount of lime necessary as per pH tests for each soil is also included in Table 6.1.

Table 6.2 summarizes the Atterberg limits, gradation results and soil classification for the soils selected. Austin, Keller, Pharr and Amarillo soils are classified as clayey materials and Wichita Falls as a sandy material. Chemical characteristics of these soils are presented in Table 6.3, and the percentage of relevant minerals in Table 6.4. The Austin and High-PI Pharr clays are dominated by Montmorillonite.

The optimum moisture contents (OMC) and maximum dry density (MDD) for different mixes are shown in Table 6.5. Typically the MDD decreases with the increase in the additive concentrations.

Table 6.1 – TxDOT Mix Design for Soils from Validation Sites

Site	Additive Content (%)	Type of Additive
Austin	6 (4)*	Lime
Keller	6 (6)	Lime
Wichita Falls	3	Cement
Pharr (Medium PI)	3 (4)	Lime
Pharr (High PI)	4 (6)	Lime
Amarillo	12	Fly-ash

*** The numbers in parentheses are percentage lime needed based on our pH tests**

Table 6.2 – Soil Classification and Plasticity Index for Soils under Study

Soil	Gradation, %				Classification		Atterberg Limits		
	Gravel	Sand	Silt	Clay	USCS	AASHTO	LL	PL	PI
Austin	0	5	38	57	CL	A-7-6	41	17	34
Keller	0	18	45	37	ML	A-6	25	14	11
Wichita Falls	0	93	6	1	SM	A-4	--	--	N/P
Pharr (Medium PI)	0	3	55	42	CH	A-7-6	56	19	37
Pharr (High PI)	0	2	39	59	CH	A-7-6	67	22	45
Amarillo	0	14	59	27	CL	A-6	31	15	16

Table 6.3 – Chemical Analysis Results for Soils under Study

Soil Region	Sulfate Content*, ppm.	CEC meq/100g	SSA m2/g	Total Potassium, %
Austin	987	101	159	1.74
Keller	675	71	271	1.10
Wichita Falls*	169	58	158	0.072
Pharr (Medium PI)	1249	76	132	1.65
Pharr (High PI)	999	104	306	1.55
Amarillo	189	66	175	0.97

* Measurements based on fine fractions in the soil

Table 6.4 – Percent Clay Mineral Observed for Soils under Study

Soil Region	% Illite	% Kaolinite	% Montmorillonite
Austin	18	39	42
Keller	29	49	22
Wichita Falls	59	23	18
Pharr (Medium PI)	28	45	27
Pharr (High PI)	26	29	45
Amarillo	16	68	16

Table 6.5 – Moisture - Density Characteristics

Soil Type	Type of Additive	Additive Content, %	Optimum Moisture Content, %	Max. Dry Unit Weight (lb/ft^3)
Austin	Lime	0	20.4	106.2
		6	19.5	95.4
		8	20.5	92.4
Keller	Lime	0	13.4	118.5
		6	15.9	100.7
		8	16.9	99.3
Wichita Falls	Cement	0	11.3	115.6
		3	9.0	123.6
Pharr (Medium PI)	Lime	0	24.5	94.3
		3	27.4	89.0
Pharr (High PI)	Lime	0	31.0	83.3
		4	31.0	77.8
Amarillo	Fly-ash	0	17.1	107.1
		12	15.3	106.9

Unconfined compressive strengths obtained under several curing and moisture conditioning for Austin, Keller and Pharr materials are summarized in Figure 6.2. Neither Austin nor Keller material achieved the minimum of 50 psi UCS strength for the amounts of lime used during the construction. As a result, specimens with 2% more lime (8%) were prepared and subjected to the same series of tests. Again, the anticipated 50 psi UCS strength was not achieved. The medium and high PI Pharr clays provided in both cases the minimum requirement of 50 psi for the 3%

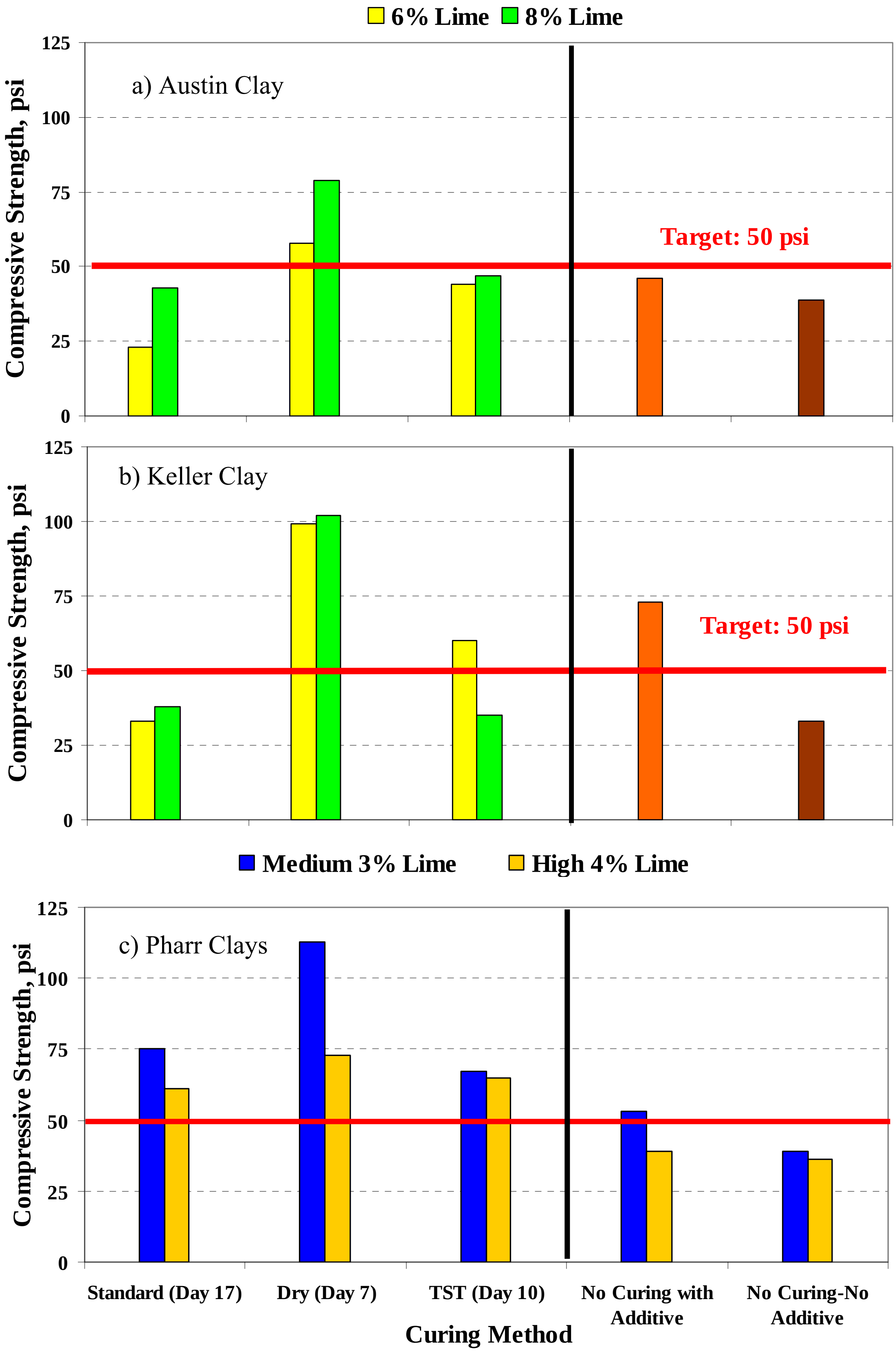


Figure 6.2 – Unconfined Compressive Strength Results for Soil-lime Specimens

and 4% lime content, respectively. As shown in Figure 6.3, the Amarillo material stabilized with 12% fly-ash did not achieve the 150 psi UCS strength established in Tex-127-E.

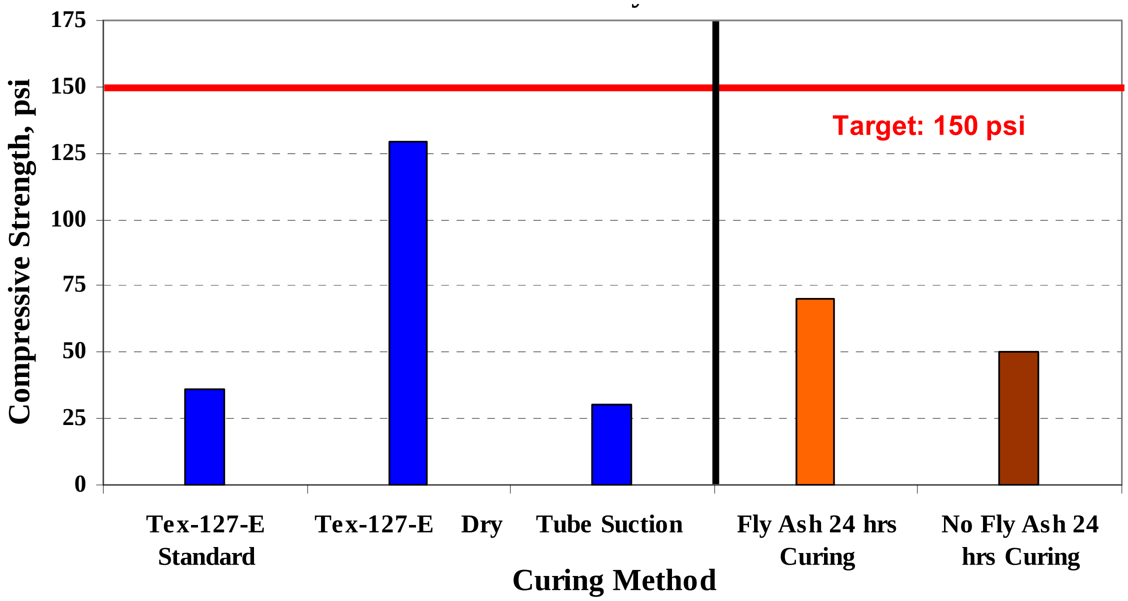


Figure 6.3 – Strength Results for Amarillo Specimens

The retained strengths were below 0.8 except for the Pharr high-PI clay, as shown in Table 6.6. The changes in moisture content were rather high (more than 6%) for all materials except the Pharr High PI clay. This occurs because the limitations of Tex-121-E in preparing homogenous specimens. In general, only the high-PI clay from Pharr fulfils the requirements of the current TxDOT specifications.

Table 6.6 – Retained Strengths and Moduli

Soil Type	Additive Content, %	Dry Curing Strength, psi*	Standard Curing Strength, psi**	Retained Strength	Dry Curing Modulus, ksi*	Standard Curing Modulus, ksi**	Retained Modulus	Change in Moisture Content, % ***
Austin	6	58	23	0.40	66	87	1.32	12.3
	8	79	43	0.54	113	128	1.13	13.3
Keller	6	99	33	0.33	162	95	0.57	12.4
	8	102	38	0.37	160	119	0.74	16.8
Wichita Falls	3	508	210	0.41	964	560	0.58	6.1
Pharr Medium	3	113	75	0.66	117	129	1.10	11.0
Pharr High	4	73	61	0.84	57	84	1.48	1.3
Amarillo	12	129	36	0.28	124	50	0.40	10.7

* *Tex-120-E curing for soil-cement specimens*

** *Tex-120-Modified (moisture-conditioned for soil-cement specimens)*

The variations in FFRC moduli for different curing methods are summarized in Table 6.7. For the Standard curing method, the modulus generally increases with time during the first seven days, after which usually slightly decreases after the 6 hrs of oven curing. The modulus further decreases with the first day of moisture conditioning and then gradually increases with time. For the TST method, the modulus increased during the first two days of curing, decreases after the first day, and then remains constant or gradually increases with time.

Table 6.7 – Variation in FFRC Modulus for Validation Soils

Soil Type	**Time (Days). Standard Curing**					**Time (Days). TST Curing**			
	0	**7***	**7****	**8**	**17**	**0**	**2**	**3**	**10**
Austin 6% Lime	30	97	65	46	87	25	72	28	79
Austin 8% Lime	38	125	114	101	128	37	123	50	93
Keller 6%	55	175	173	96	95	86	176	65	89
Keller 8%	59	171	166	98	119	67	121	30	44
Pharr (Medium)	31	120	126	122	129	31	154	184	117
Pharr (High)	27	86	87	76	84	31	76	72	72
Amarillo	36	135	121	41	50	60	122	20	29

** Before specimen was placed in the oven*
*** After specimen was removed from the oven*

MOISTURE CONDITIONING RESULTS

Strengths from standard specifications, TST and alternative moisture conditioning protocols are presented in Figure 6.4. The Wichita Falls sand results are not included since they were already introduced in Chapter Four. The highest strengths for static compactions are in most cases associated with the TST or Tex-121-E methods.

The overall strengths for all materials using the alternative methods are compared with those from Tex-121-E (Tex-127-E for fly ash) and TST protocols in Figure 6.5. In general, the alternative moisture conditioning processes yield strengths that are less than those from the Tex-121/127 process as found in Chapter 4.

The comparisons of the moisture contents after moisture conditioning are included in Figure 6.6. Alternative methods resulted in greater absorption of moisture as compared to the Tex-121/127 or TST methods. Some of the TST specimens that did not saturate to the top absorbed less moisture when compared to Tex-121/127-E.

The comparisons of moduli of the prepared specimens after moisture conditioning are presented in Figure 6.7. Similar to the strength results, the general trend is that specimens subjected to the alternative moisture conditioning methods yielded lower moduli as compared to the current TxDOT specifications, because alternative methods usually saturated the specimens more effectively.

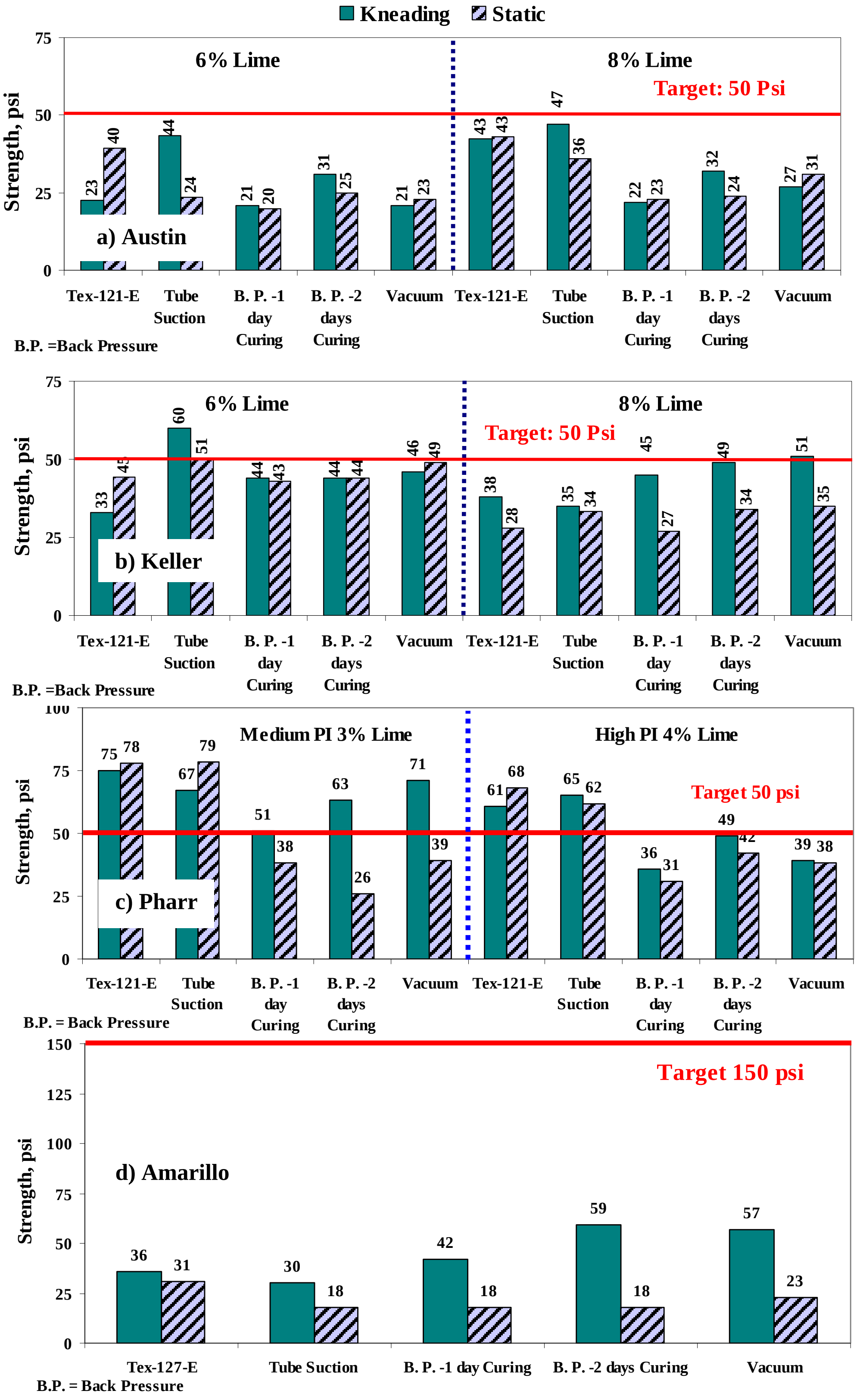
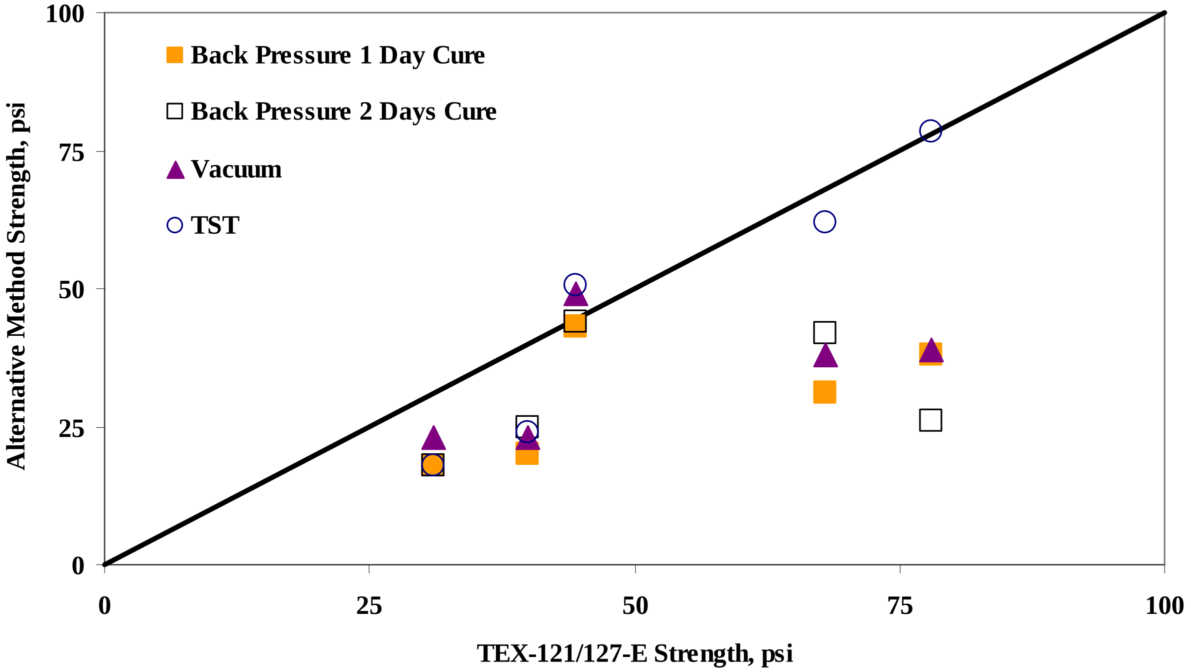


Figure 6.4 – Variations in UCS for Different Moisture Conditioning Processes



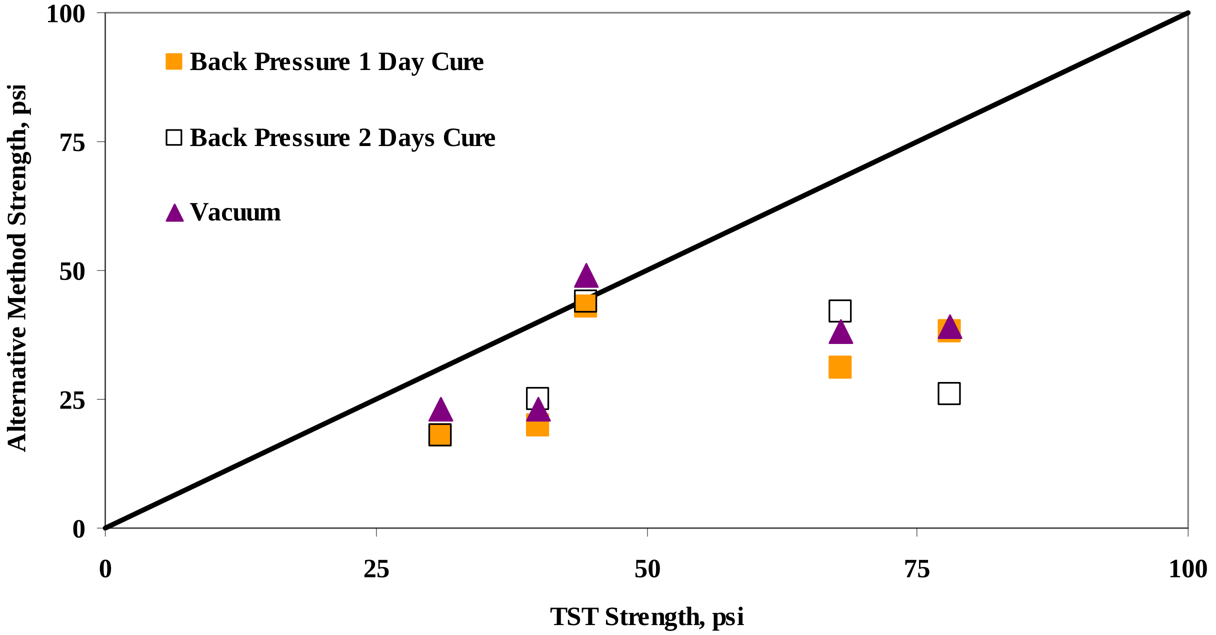
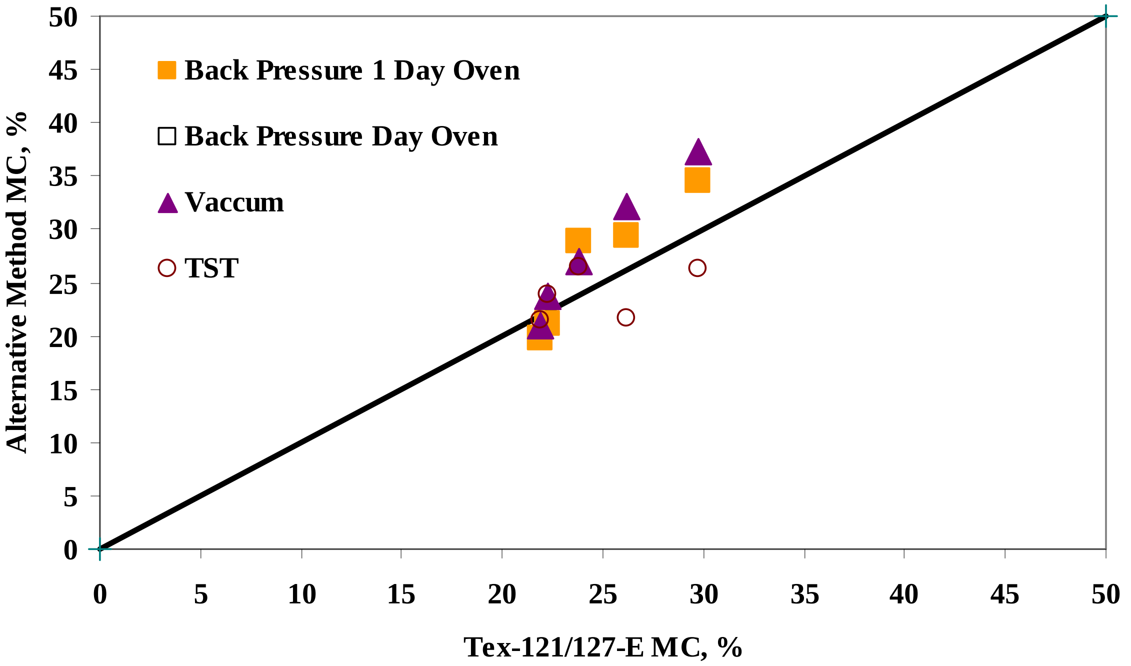


Figure 6.5 – Comparison of Strengths from Different Moisture Conditioning Methods



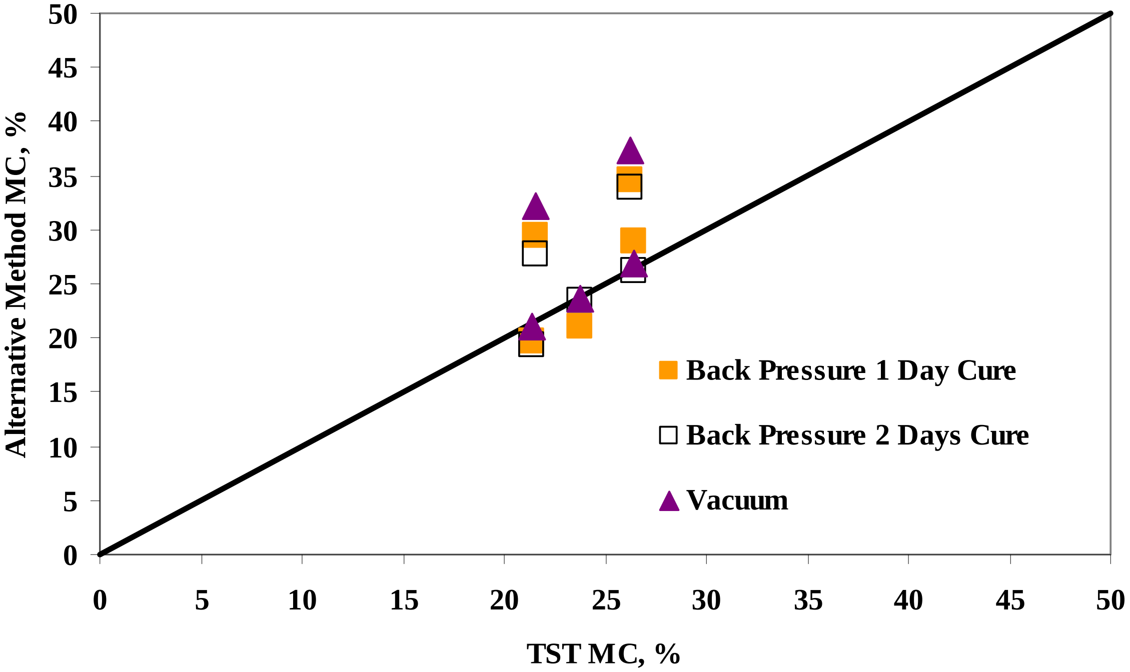


Figure 6.6 – Comparison of Moisture Contents from Different Moisture Conditioning Methods

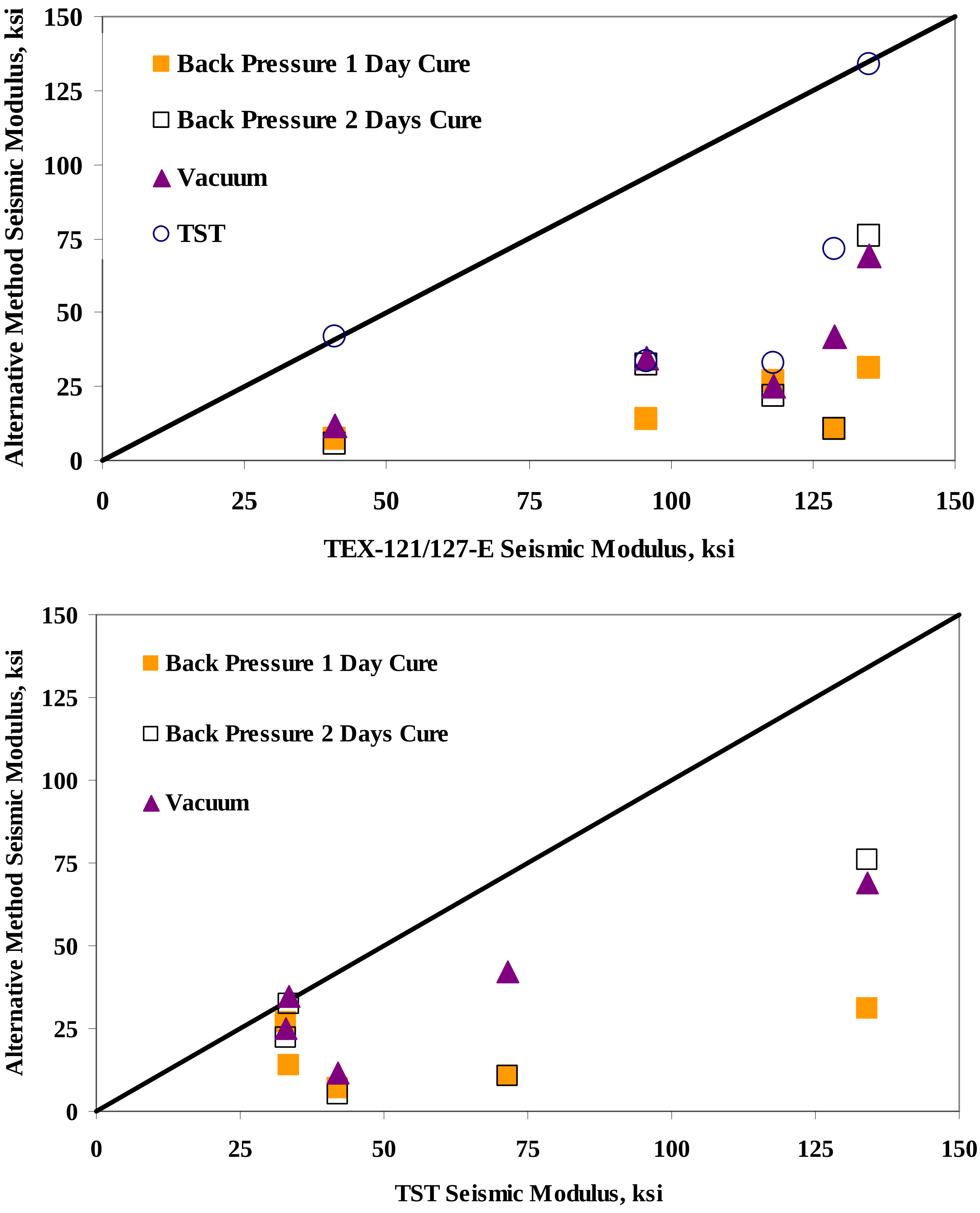


Figure 6.7 – Comparison of Moduli from Different Moisture Conditioning Methods

The average leachate results after 3, 5, 7, 11 and 14 cycles are shown in Figure 6.8 and summarized in Table 6.8. A small (less than 0.6%) decrease in the calcium ion concentration after 14 cycles of leaching was observed. The pH tests on the leachate samples collected increased significantly for rather small amount leached stabilizer, proving that pH variations with leaching cycles do not yield reliable conclusions on the leachability of stabilizers.

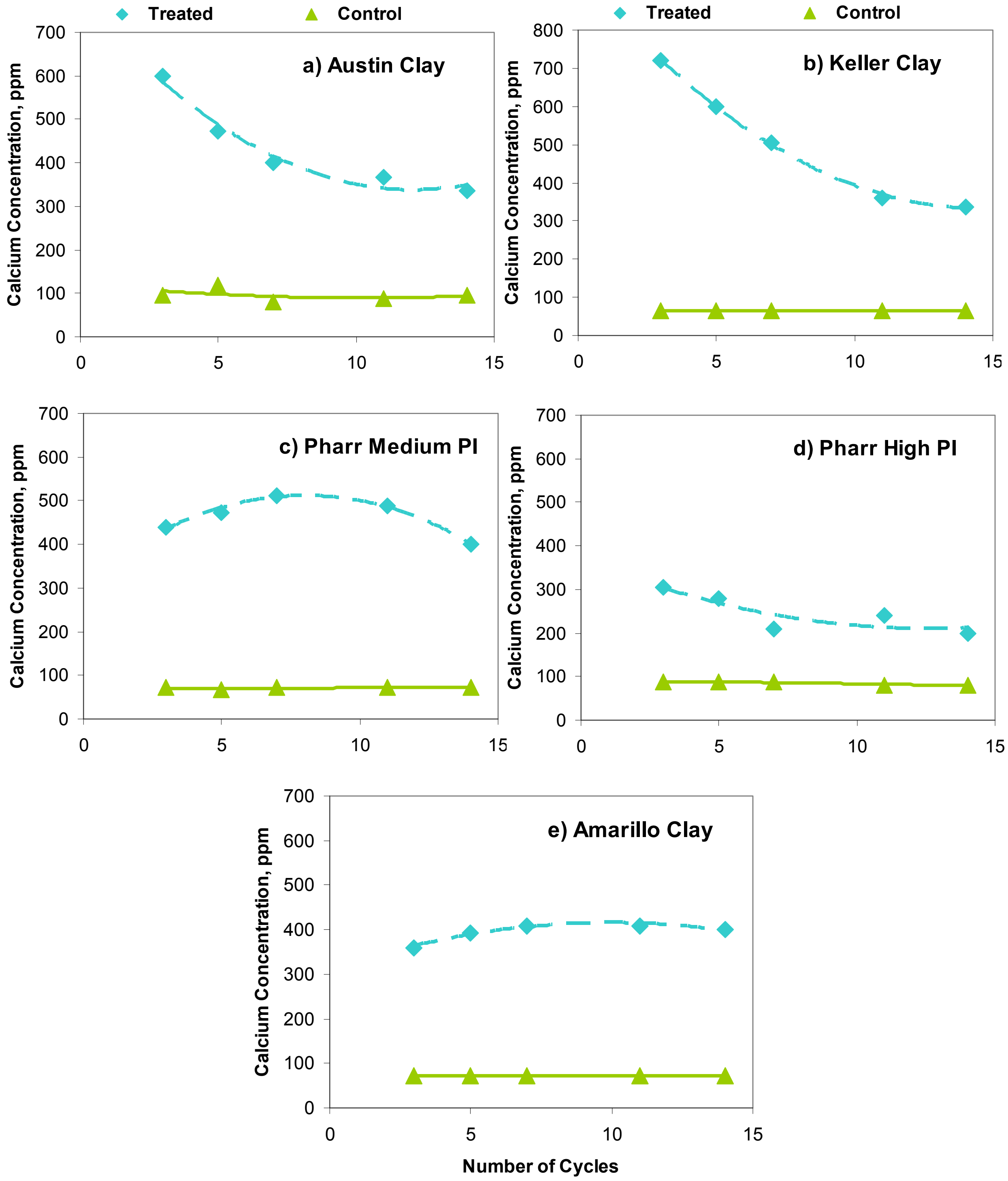


Figure 6.8 – Variation of Calcium Concentration of Validation Soil Samples

Table 6.8 – Additive Leached and Strength Loss after 14 Cycles of Leaching

Soil Type	Dominant Mineral	% Additive Leached	% Strength Retained	Number of years in field replicated
Austin	Montmorillonite	0.4	99	3.2
Keller	Kaolinite	0.4	99	2.6
Pharr Medium PI	Kaolinite	0.6	55	3.8
Pharr High PI	Montmorillonite	0.5	73	4.4
Amarillo	Kaolinite	1.6	78	3.2

UCS strength tests conducted only after 3, 7 and 14 cycles are shown in Figure 6.9. All soils survived the 14 leachate cycles with overall small strength loss, due to the small strength loss from UCS tests. However, this percentage can be considered considerable if the initial additive dosage is low. The Pharr medium PI soil stabilized with 3% lime retained only 55% of its initial strength after 14 leachate cycles (see Table 6.8). Amarillo clay lost about 1.5% of the fly-ash after 14 cycles, but retained 78% strength. The number of field years replicated in all soils is less than 5 years, which is short compared to the design life of the pavement. It is likely that with more number of leachate cycles, the percent loss of stabilizer could be higher and may exceed 2% in ten to fifteen years.

The volumetric changes due to wetting/drying cycles as explained in Chapter Five are depicted in Figure 6.10, and corresponding strength losses in Figure 6.11. The relevant information is summarized in Table 6.9. Almost all clayey materials without treatment only survived 1 W/D cycle. Small amounts of swelling and shrinking were observed for the treated Keller material, and the specimens survived the 21 W/D cycles. Pharr clays exhibited a variation in swelling of around 10%, and only survived 4 and 8 cycles for the medium and high PI materials, respectively. In general, soils with high concentration of Montmorillonite (Austin and high PI Pharr) performed the least satisfactorily as also observed in Chapter Five.

The Amarillo soil stabilized with fly-ash experienced considerable swelling after 3 W/D cycles, and the soil specimens survived only 6 W/D cycles. The treated Austin soil sample failed after twelve W/D cycles. The premature failures in the case of Amarillo clay could be attributed to the incompatibility of the additive (fly-ash) with the resident soil, or the small dosage used (12%).

Table 6.9 – Dominant Clay Material, Cycles Survived and Drop in Strength of Validation Soil Samples

Soil Name	Dominating Clay Mineral	Type and Amount of Additive	No. of Cycles Sample Survived	% Retained Strength
Austin	Montmorillonite	Lime, 6%	6	0
Keller	Kaolinite	Lime, 6%	21	80
Pharr Medium PI	Kaolinite	Lime, 3%	8	0
Pharr High PI	Montmorillonite	Lime, 4%	4	0
Amarillo	Kaolinite	Fly ash, 12%	6	0

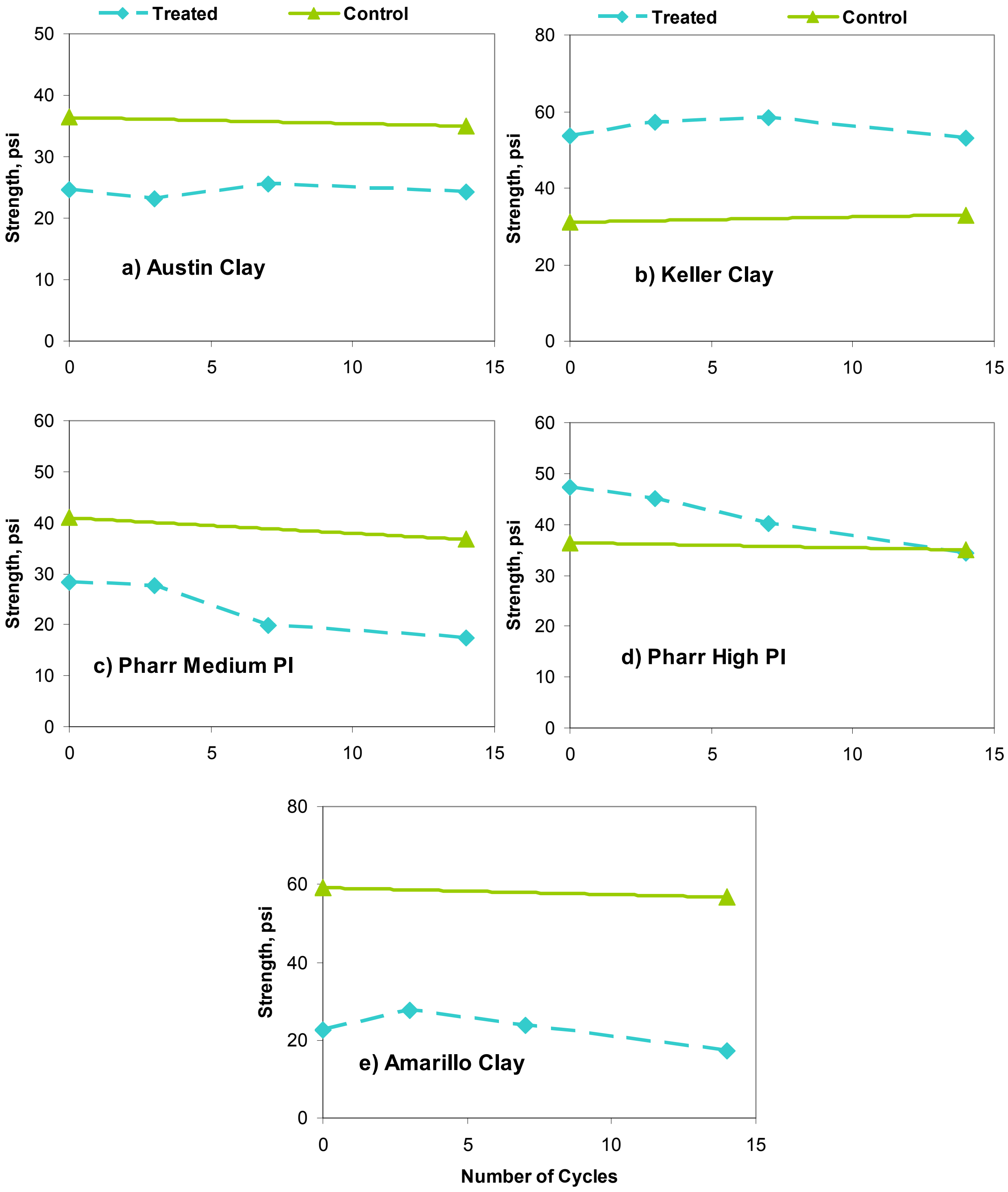


Figure 6.9 – Variation of UCS Strength with the Number of Leaching Cycles for Validation Soil Samples

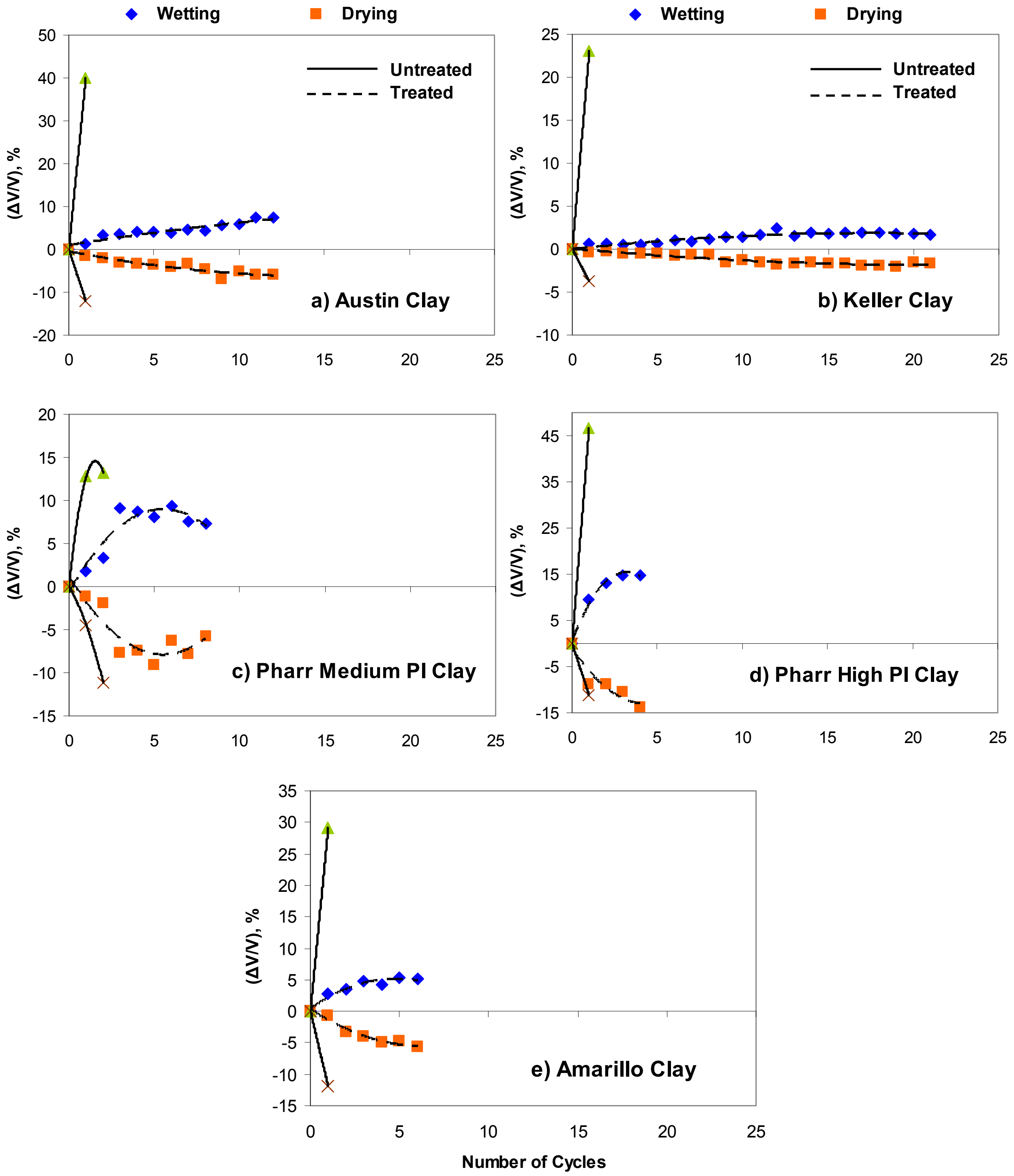


Figure 6.10 – Variation of Volumetric Strains with the Number of W/D Cycles for Validation Soil Samples

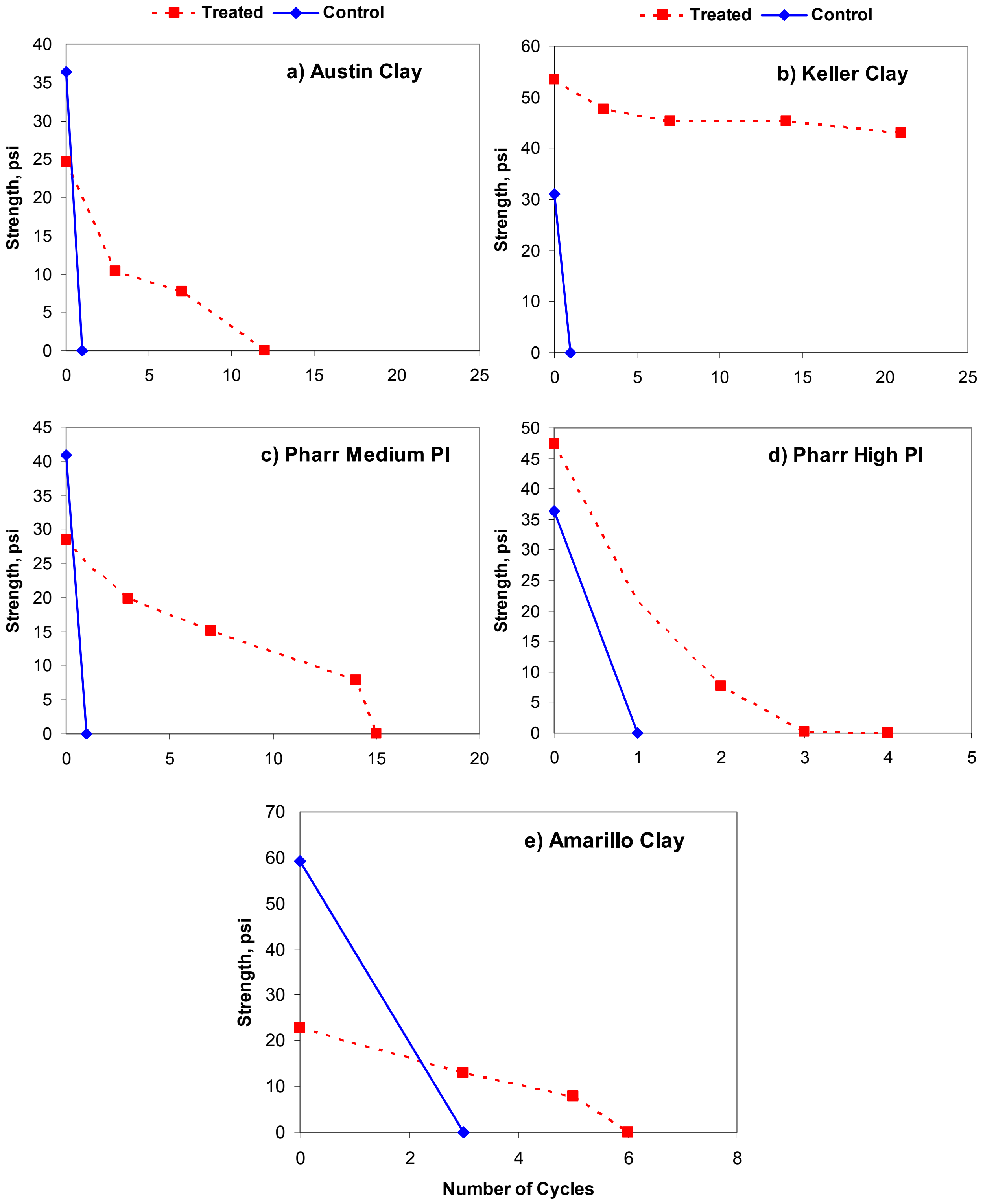


Figure 6.11 – Variation of UCS Strength with the Number of W/D cycles for Validation Soil Samples

COMPARISON OF LONG-TERM, ACCELERATED CURING AND MOISTURE CONDITIONING RESULTS WITH TXDOT SPECIFICATIONS

As reflected in this Chapters 5 and 6, the results from the new procedures proposed and those from TxDOT specifications are different. To establish new design criteria for selection of adequate stabilizer dosage, these differences should be quantified and reconciled. Only the materials that were classified as clay (total of 9) were used for comparison. In addition, considering the length of time required to perform the wetting/drying cycles (up to one month) to evaluate the long-term performance of the soil mixes, these results are correlated to the retained strengths to establish criterion for when to perform the wet/dry cycle tests.

TxDOT Specifications vs. Accelerated Results

To investigate the impacts of sample preparation (static compaction in this study vs. Tex-114-E) and curing methods (104ºF in oven for 2 days in this study vs. 7 days of curing and 6 hours of oven-curing as per Tex-121/127), the so-called dry strengths from the two approaches are compared in Figure 6.12a. In general, the newly-proposed method provides results that are either comparable or more conservative than the current specifications. A threshold of 65 psi is depicted in the figure as acceptance threshold since TxDOT currently does not advocate or specify threshold value for strength before moisture conditioning. This value was obtained by dividing the 50 psi acceptance threshold for moisture conditioned specimens by 0.8. The value of 0.8 is simply equal to a retained strength of 80% advocated in several of TxDOT current specifications. Based on this threshold, six of the materials that would have passed the current specifications would pass the newly-proposed method. Paris and Fort Worth materials which do not meet the newly proposed criterion but they would have met the existing criterion are proven to be problematic in term of performance. On the other hand, the Austin Clay, another problematic soil, marginally passes based on the proposed criterion, but would have failed based on the current methods. Based on this observation, a “dry” strength of 65 psi is considered in the development of the new protocol in Chapter 7.

The comparison of the strengths after curing and moisture conditioning (with backpressure saturation) is compared with those from TxDOT specifications (capillarity suction method) in Figure 6.12b. Once again, the strengths from the proposed methods are generally smaller than those from the existing method. Six of the materials tested achieved the required strength of 50 psi based on the current TxDOT as opposed to two (Bryan and El Paso) based on the proposed method. This was previously attributed to the higher moisture content and higher saturation level obtained with backpressure method. If this criterion is relaxed to 35 psi for the proposed method, two other materials that performed reasonably well in long-term durability (Keller and Pharr low PI) will also pass the new requirements. As such, a new threshold of 35 psi is preliminarily considered for the new protocol.

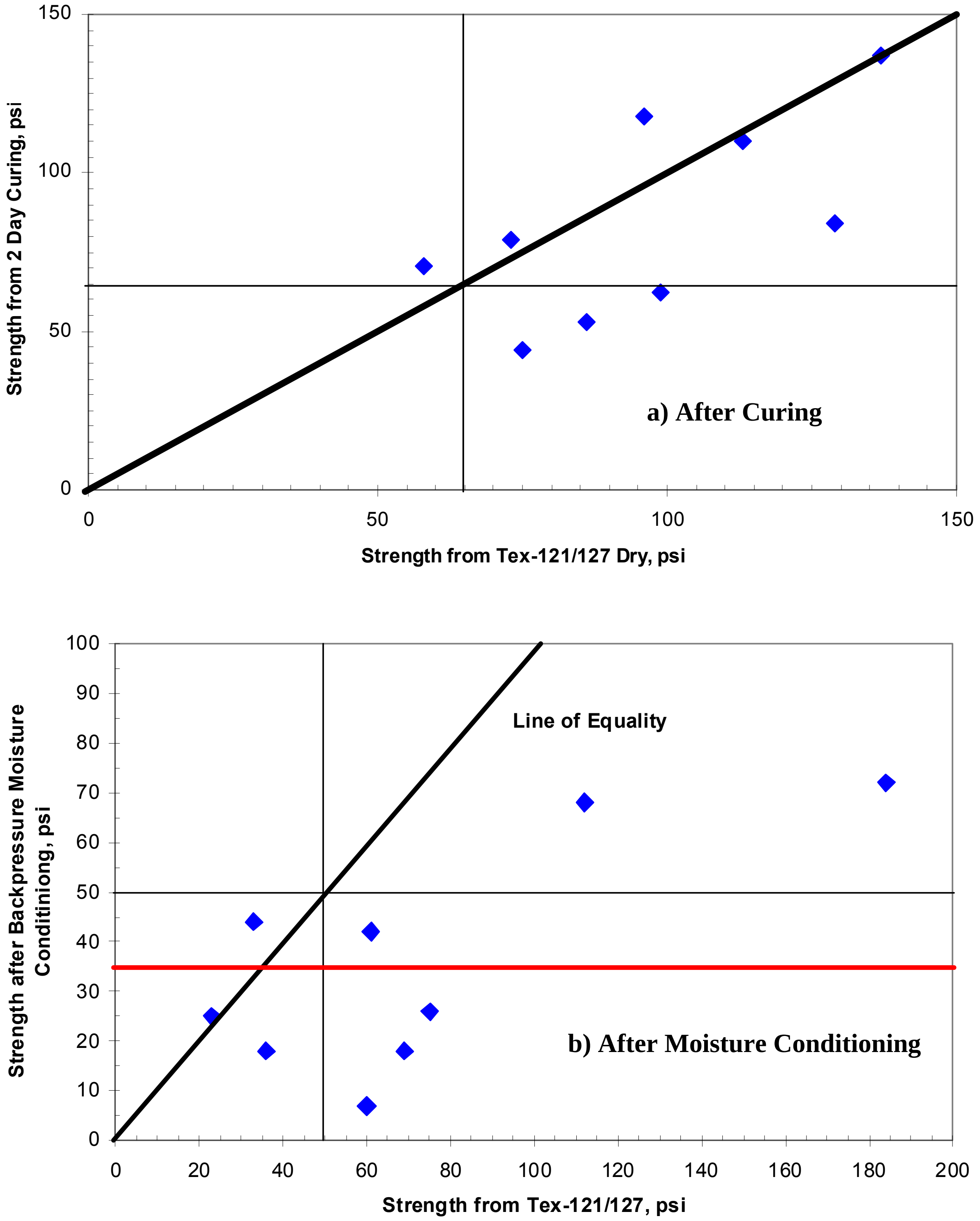


Figure 6.12 – Comparison of TxDOT Specifications Strength with Accelerated Methods

Retained strengths after moisture conditioning with current TxDOT protocols and accelerated methods are compared in Figure 6.13. Results obtained with the current protocols are generally less than those from the conventional methods. For the retained strength of 80%, only four materials (Paris, Pharr High PI, Bryan and El Paso) would pass using the current methods, while none will be considered as acceptable under the proposed procedure. If the threshold for the new procedure is relaxed to 50%, the Bryan, El Paso, Pharr mix designs will be again acceptable in addition to the Keller material. Since all these four materials performed reasonably well during the wetting/drying cycle tests, a retained strength of 50% is preliminarily considered in the new protocol.

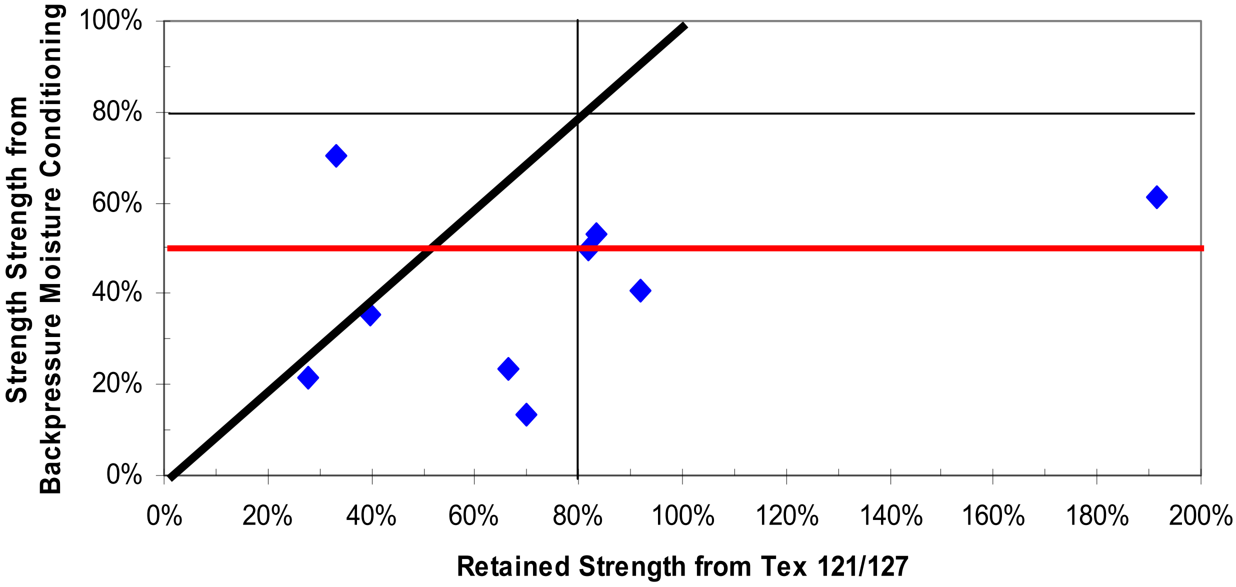
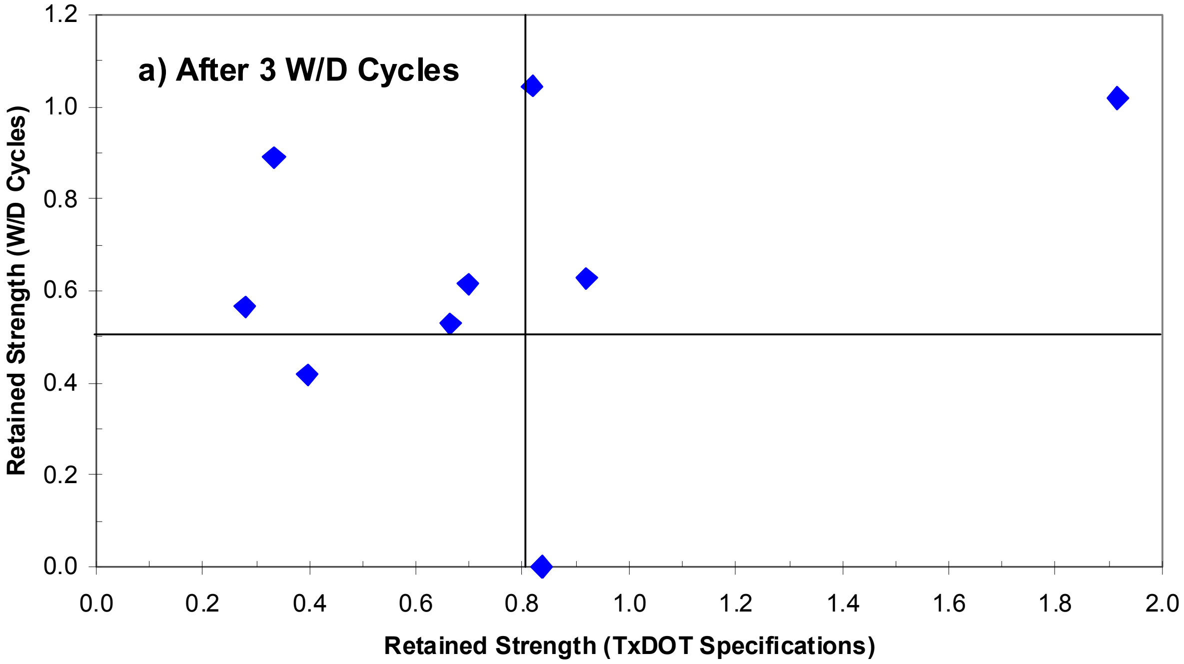


Figure 6.13 – Comparison of Retained Strengths from Current Specifications and Accelerated Methods

Wetting/Drying Cycles vs. Accelerated Moisture Conditioning

Since the theme of this project is to accelerate the mix design, and since wet/dry cycle is rather time consuming, an attempt was made to correlate the results from the wet/dry cycle durability tests with the retained strengths. In that manner, the use of the wet/dry cycle durability tests can be minimized.

The retained strengths after 3 and 21 cycles of wet/dry durability tests are compared with the retained strengths as per current TxDOT specifications in Figure 6.14. Two of the four soils (El Paso and Bryan) that met the 0.8 retained strengths also passes the wet/dry durability tests for 21 cycles, while the other two do not survive the 21 cycles. In Figure 6.15 the same results are shown but for the retained strengths from the proposed method. For a threshold retain strength of 0.50, three of the materials (Bryan, El Paso and Keller) that passed the retained strength criterion also pass the 21 cycles of the dry/wet durability tests. Once again, the Pharr High-PI material passed the retained strength criterion but badly failed the durability tests. Given the amount of data available in this study, the retained strength threshold is increased to 0.6 from 0.5 for conservatism.



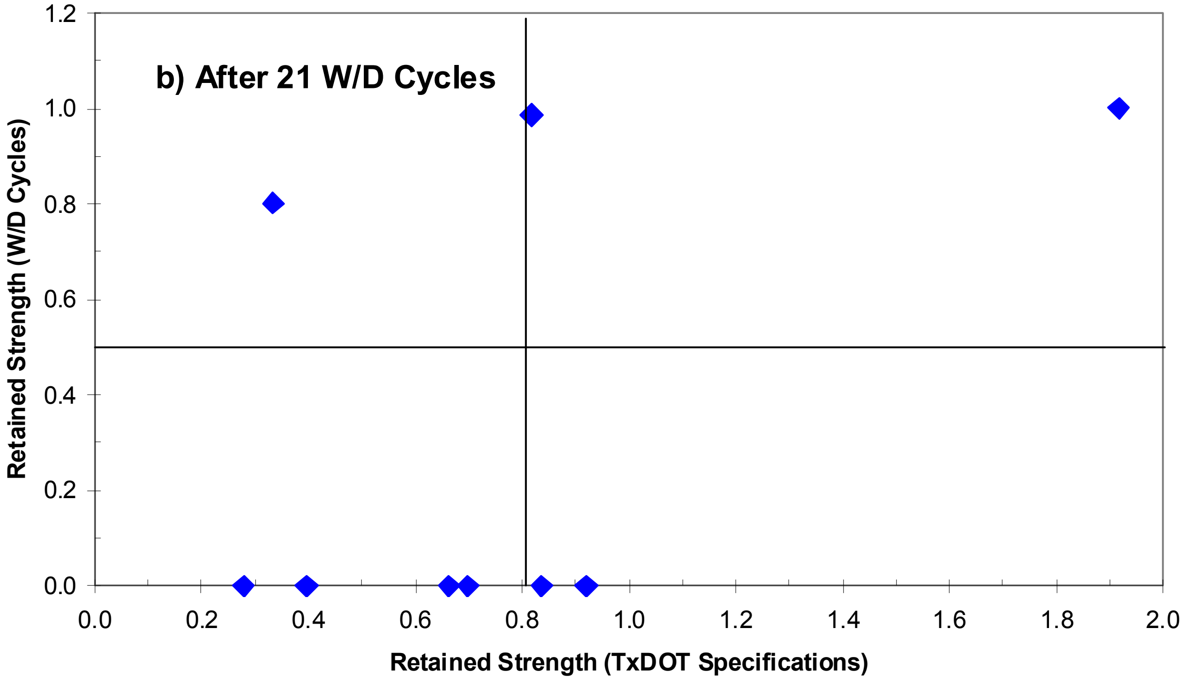


Figure 6.14 – Comparison of Retained Strengths of TxDOT Specifications with Wetting/Drying Cycles

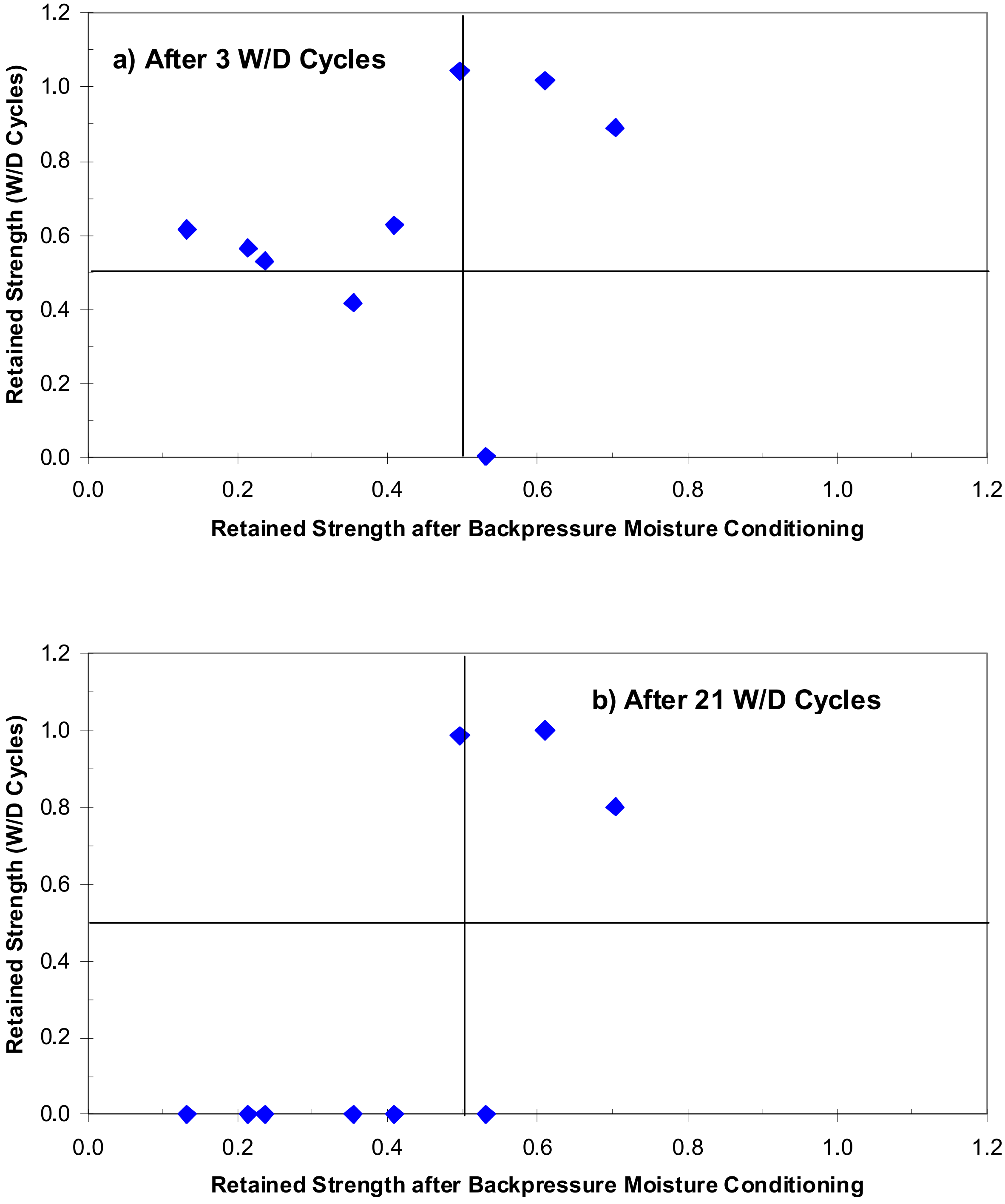


Figure 6.15 – Comparison of Retained Strengths of Accelerated Moisture Conditioning with Wetting/Drying Cycles

FIELD TESTING RESULTS

As indicated before, the Portable Seismic Property Analyzer (PSPA) was utilized to determine the moduli of the stabilized subgrades after construction at several sites. For stabilized materials, the lab moduli depend on the curing and moisture-conditioning. Generally, it is difficult to provide similar curing and moisture conditioning in the lab and in the field. For comparison purposes, the average moduli obtained from the PSPA were compared to the moduli measured with the FFRC device in the lab for different moisture conditioning methods. Complete PSPA results are included in Appendix D.

Austin Site

PSPA tests were carried out on the subgrade 12 days after compaction along the right wheelpath, center and left wheelpath at 17 stations spaced 50 ft. The variations in moisture content and density were measured with a nuclear density gauge (NDG) at 6 different stations (see Appendix D for details). Moisture contents ranged from 15% to 25%, and the densities varied from 90 to 99 pcf. The distribution of modulus ranged from 50 to 130 ksi, with an average value of about 85 ksi and a COV of 16%. The lab seismic moduli on the statically compacted specimens are compared with the average PSPA modulus in Figure 6.16. The average of 85 ksi from field testing is close to the maximum modulus (modulus after drying) obtained in prepared specimens with the same lime content.

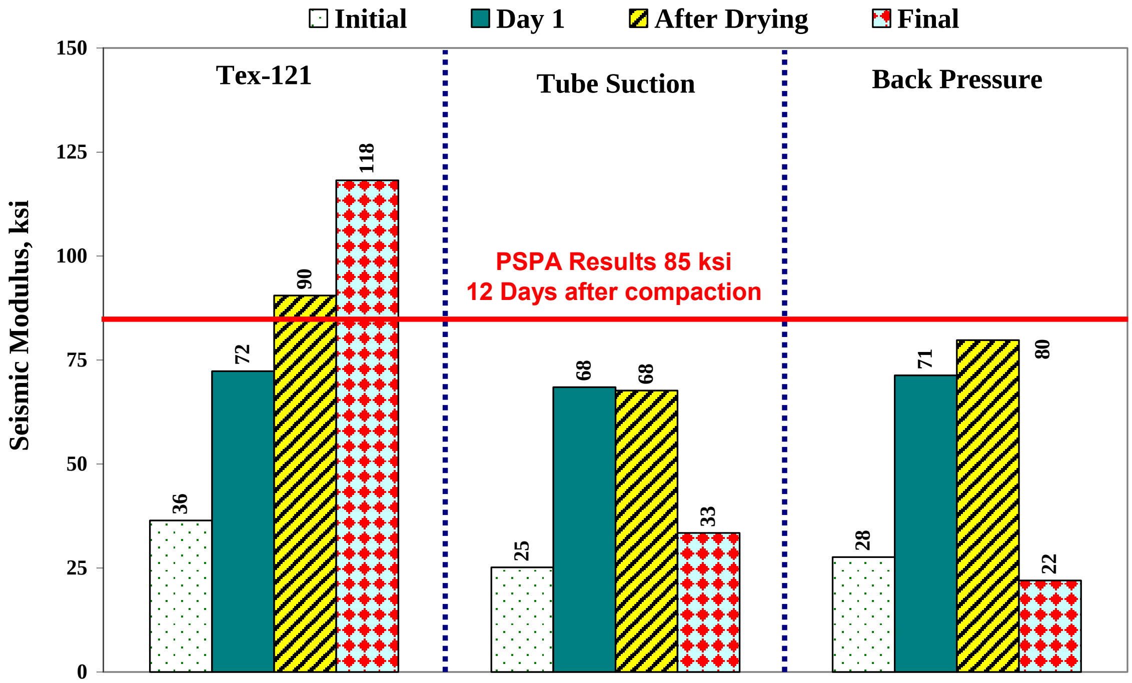


Figure 6.16 – Seismic Modulus Variation of Laboratory and Field Tests for Austin Site

Keller Site

PSPA tests were carried out on a section of the subgrade that was cured for 24 hours after compaction along the right wheelpath, center and left wheelpath at 11 stations spaced 200 feet apart. The modulus ranged from 31 to 113 ksi, with an average value of about 60 ksi and a COV of 34%. NDG tests conducted alongside PSPA tests indicated that the moisture contents and densities varied from 18% to 27% and from 92 pcf to 113 pcf, respectively. The lab and field moduli are compared in Figure 6.17. The average of 60 ksi from field testing is close to the modulus obtained in prepared specimens after compaction and 24 hr of curing (noted as "Initial" in the figure).

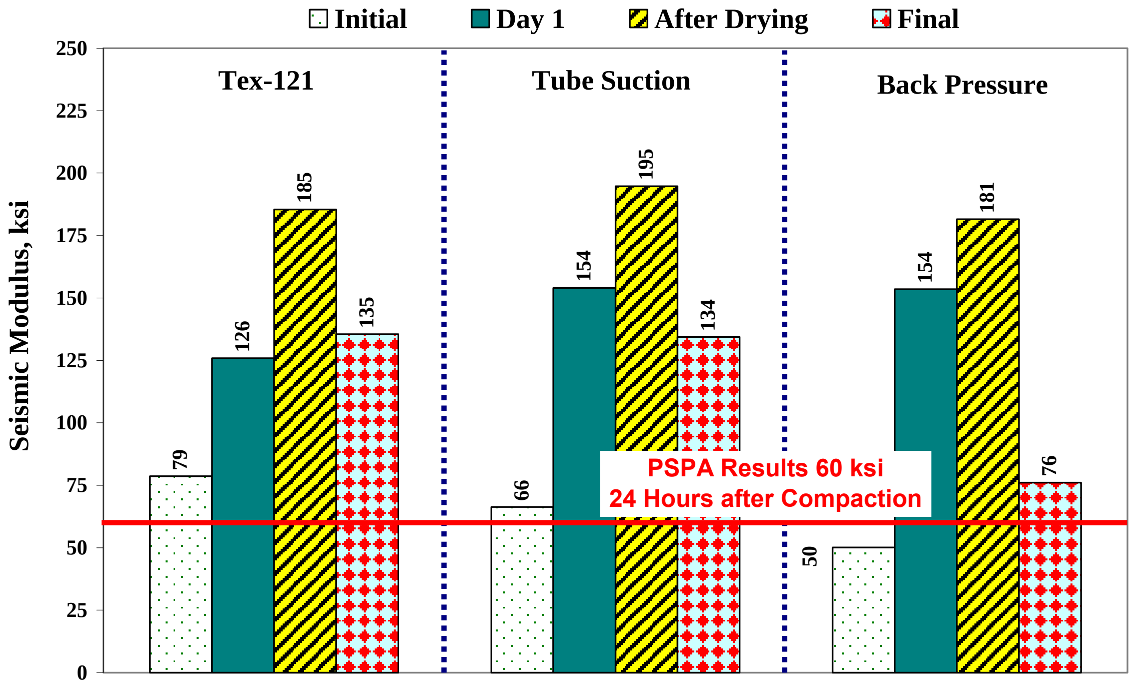


Figure 6.17 – Seismic Modulus Variation of Laboratory and Field Tests for Keller Site

Wichita Falls Site

PSPA tests were carried out on a section of the subgrade that was cured for 24 hours after compaction along the right wheelpath, center and left wheelpath at 50-ft spacing between each station. The distribution of modulus ranges from 70 to 420 ksi, with an average value of about 238 ksi and a COV of 34%. The same stations were evaluated with the PSPA after 3 days of curing. The modulus ranged from 63 to 347 ksi, with an average value of about 166 ksi and a COV of 31%. The drop in modulus between 1 and 3 days of curing was attributed to the extensive surface cracking that appeared on the section after 3 days, as shown in Figure 6.18. The lab tests also show in decrease in modulus after one and two days to curing (see Figure 3.12). The results from backpressure saturation are the closest to the field results. As also seen in Project 0-5223 (Geiger et al., 2006), the lab moduli typically over-estimate the field moduli for cement stabilized materials.

Figure 6.18 – Surface Cracking after 3 Days of Curing

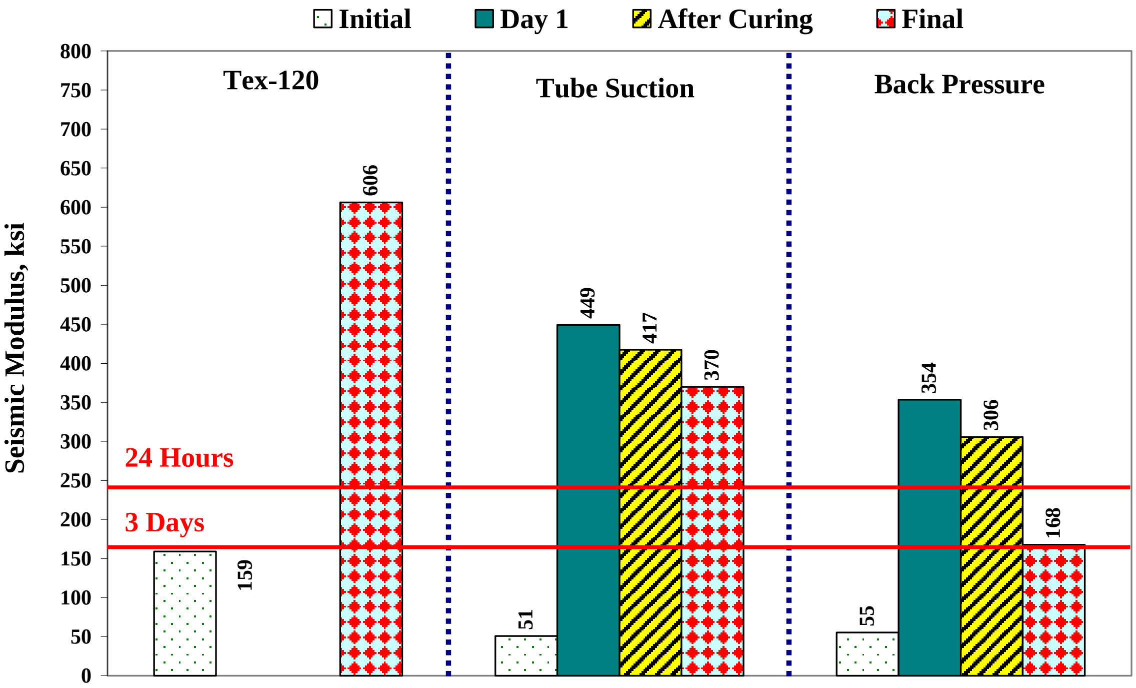


Figure 6.19 – Seismic Modulus Variation of Laboratory and Field Tests for Wichita Falls Site

Pharr Site

PSPA tests were carried out on the stabilized subgrade along the right wheelpath, center and left wheelpath at 50-ft spacing between each station. The distribution of modulus ranges from 24 to 50 ksi, with an average value of about 37 ksi and a COV of 22%. Results for the 5 and 10-day cured sections were also obtained. The modulus ranged from 45 to 95 ksi, with a COV of 31% for the 5-day old section and from 36 to 110, with a COV of 29% for the 10-day old section. The average modulus in both cases was 70 ksi. NDG measurements were obtained on the three sections at several stations on the center of the corresponding lane. Moisture contents and densities varied from 26% to 31% and from 84 pcf to 87 pcf for the section that was cured for 24 hours. For the 5-day and 10-day old sections moisture contents ranged from 16% to 21%, and densities from 86 pcf to 104 pcf.

Lab seismic moduli are compared with the average PSPA modulus in Figure 6.20. The average of 37 ksi obtained with PSPA after 24 hours of curing is very close to those obtained for all specimens compacted in the laboratory. The average of 70 ksi after 5 or 10 days of field curing falls in between the initial modulus and the modulus after curing obtained with all the curing methods. Once again, the initial moduli from lab tests are close to the field moduli and the final modulus from backpressure saturation is close to the field moduli after 5 and 10 days of exposure.

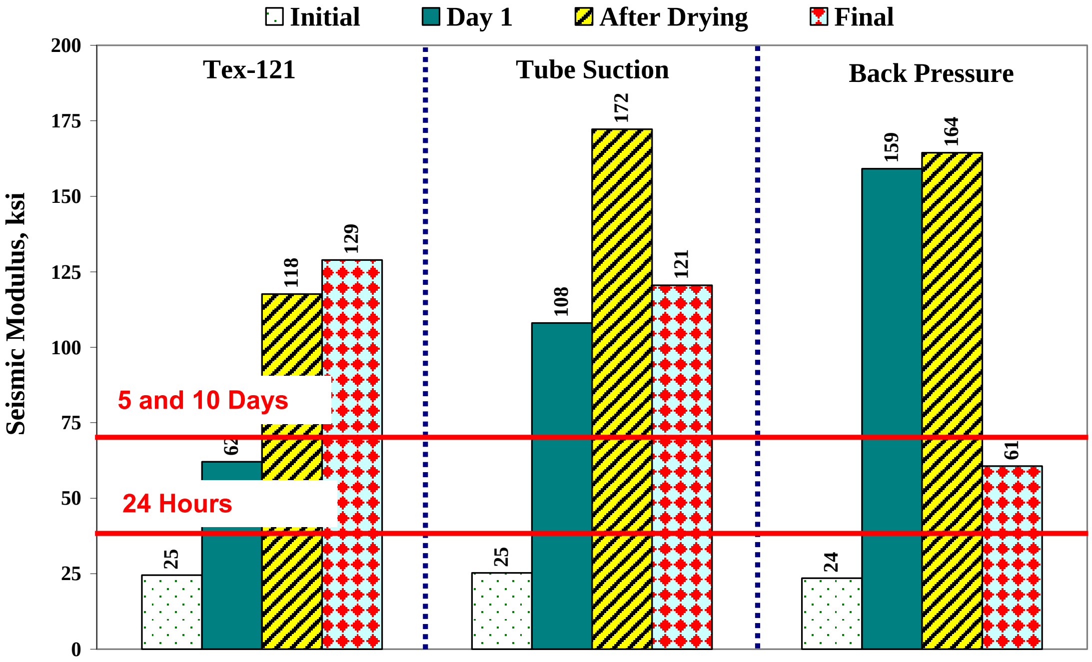


Figure 6.20 – Seismic Modulus Variation of Laboratory and Field Tests for Pharr Site

As judged from the moisture and density measurements in the field, the final field products are quite variable. This variability is reflected in the moduli measured with the PSPA. For lime stabilized materials, the field moduli after 24 hours are reasonably close to the lab moduli after 24 hours of curing. After several days of construction, the field moduli are somewhere between the lab moduli after curing and the final moduli after moisture conditioning depending on the temperature and humidity of the site. For the cement stabilized sandy materials, the curing is very important and typically about half the modulus from lab is anticipated in the field.

CHAPTER SEVEN – TECHNICAL DESIGN MANUAL

Figure 7.1 illustrates the flowchart for the proposed accelerated design procedure. In this study, it was found that the selection of type and concentration of additives based on gradation and plasticity may not be adequate. For example, two subgrades with the same plasticity indices (PIs) react vastly differently to the same stabilizer depending on the dominant clay type in the fines. A lack of a more refined soil classification that considers the clay mineralogy creates a dilemma whether a certain chemical treatment method could be used for all types of clayey subgrades. Hence, better and more reliable strategies are needed for screening clay mineralogy for successful use of a given stabilizer.

Also, the length of time required for the mix design may encourage the users to use their past experience to specify the stabilizer types and concentration with potential short and long term negative implication on the performance of the completed project. As such, accelerated protocols have been developed so that the mix design can be completed in a reasonable amount of time. A step-by-step procedure is proposed in this chapter.

Soil Exploration and Material Sampling

Soil exploration is vital, as it provides material for testing and also reveals conditions in underlying strata that can affect the performance of the pavement structure and treated layers, such as soil mineralogy, water table proximity, and soil strata variation. Material sampling and testing are critical and are required to characterize the physical properties that can affect the performance of the pavement structure. It is important to obtain bulk samples large enough to perform multiple mix designs and soil classifications, if necessary.

Additive Selection Criteria

The selection of appropriate additive(s) is affected by many factors, including:

- Soil mineralogy and content (sulfates, organics, etc…)
- Soil classification (gradation and plasticity)
- Goals of treatment
- Mechanisms of additives
- Desired engineering and material properties (strength, modulus, etc…) Design life
- Environmental conditions (drainage, water table, etc…)
- Engineering economics (cost vs. benefit)

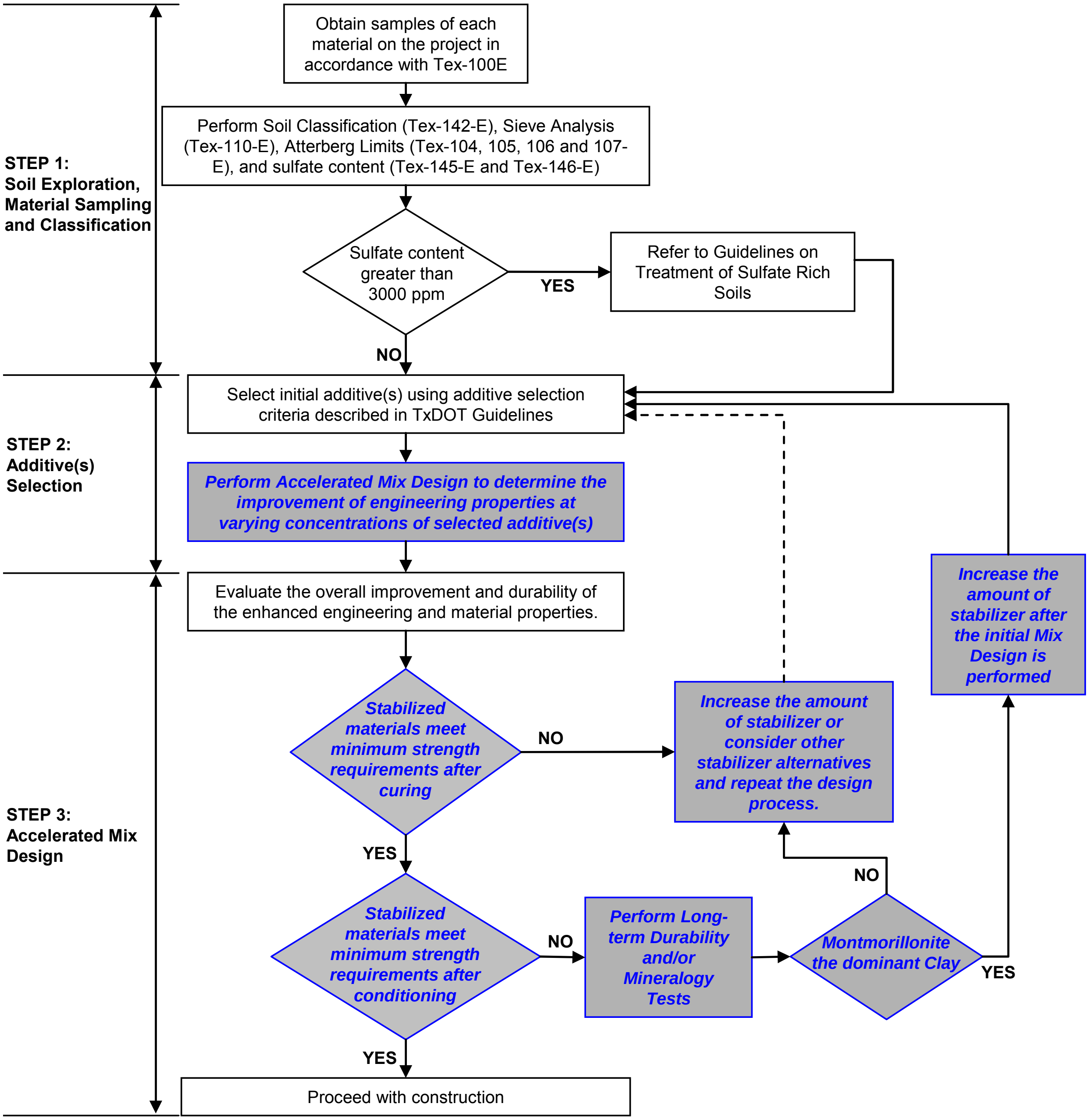


Figure 7.1 – Proposed Flowchart for Subgrade Soil Treatment

Current TxDOT stabilization guideline (previously shown in Figure 2.3) provides a reasonably good reference for the preliminary selection of additives. The Plasticity Index (PI) and gradation are necessary but not sufficient criteria. Further validation testing must be performed to verify whether the selected additive accomplishes the goals and requirements of the treated soil.

Gradation
The gradation should be carried out as per Tex-110-E.

Plasticity Index
Plasticity index is commonly used as an indication of soil shrink/swell properties and constructability. If a soil has the ability to attract and drive off large amounts of moisture, this results in large volumetric change and material instability. The PI can be obtained in accordance with Tex-105-E.

Sulfate and Organic Testing
It is of utmost importance to estimate the sulfate and organic contents of the subgrade. If the sulfate or organic contents are higher than recommended in the current TxDOT guidelines, special care should be taken in selecting the type of stabilizer.

The sulfate and organics contents should be measured to ensure that they are within acceptable levels. The sulfate concentration is estimated using Tex-145-E. If the sulfate levels are above 3000 ppm, the 'Guidelines on Stabilization of Sulfate Rich Soils' should be consulted for further recommendations and guidelines.

Organic soil is a soil that would be classified as a clay or silt except that its liquid limit after oven drying (dry sample preparation) is less than 75% of its liquid limit before oven drying (wet sample preparation). The organic content should be estimated in accordance with ASTM D-2974. If the organics content exceeds 1%, additional additive will need to be added to counter the cationic exchange capacity of the organic material.

Determination of Clay Mineralogy
Aside from the gradation and PI, the clay mineralogy impacts the concentration of the additives the most. It is desirable to include these tests in the day-to-day operation of TxDOT.

The minerals of interest in Texas soils are Kaolinite, Illite and Montmorillonite. If the dominant clay mineral is Montmorillonite, additional steps need to be carried out to ensure a durable stabilized subgrade.

Direct measurements of the clay mineralogy utilizing X-Ray Diffraction (XRD) and Scanning Electron Microscope (SEM) are preferred. These tests are not practical for day-to-day use since they are expensive to perform and require advanced instrumentation. A set of simple indirect methods, such as cationic exchange capacity (CEC), specific surface area (SSA) and total potassium (TP) are proposed to estimate the dominating clay minerals in the subgrades. The test protocols for thee methods are provided in Appendix C. For day-to-day use, the following process is proposed:

Step 1: Determine percent Illite (*%I*) by measuring the total potassium (TP) using:

$$\%I = \left[\frac{TP}{6}\right] \times 100 \tag{7.1}$$

If Illite is the dominant clay mineral, the mineralogy of the clay is not of a concern, and further mineralogical tests are not necessary (i.e. ignore Step 2).

Step 2: Determine percent Montmorillonite (*%M*) and percent Kaolinite (*%K*) by measuring the cationic exchange capacity (CEC) and specific surface area (SSA) using the following equations:

$$\%M = -2.87 + 0.08 \times SSA + 0.26 \times CEC \tag{7.2}$$

$$\%K = 100 - \%I - \%M \tag{7.3}$$

Mix Design

Mix design is essential to optimize the material properties, calculate the optimum percent of additive, measure effectiveness and engineering properties and provide density and moisture control parameters for construction. The steps to achieve the mix design are discussed below. The proposed accelerated design process includes the following steps:

- Estimating the preliminary additive content
- Developing moisture density curve (M/D) with and without additives to obtain the optimum moisture content (OMC) and the maximum dry density (MDD).
- Preparing specimens
- Curing of specimens
- Accelerated moisture conditioning
- Strength testing before and after moisture conditioning, and
- Determining the lowest modifier content to satisfy strength requirements.

Estimating Preliminary Additive Content

For treatments requiring lime, the pH of the soil-lime environment is critical because high pH (basic) mixtures increase the ability of the lime to react with soil minerals, like silica and alumina, which also require high pH levels to dissolve. The preliminary amount of additive content should be determined following the Eades-Grimm method in accordance to Tex-121-E, Part III.

The additive concentration obtained in that manner is preliminary. Actual concentration should be determined based on more rigorous tests described below.

Establishing Moisture Density (MD) Curve

The determination of the MD curve for preparing specimens and for establishing field density control is carried out in accordance with Tex-114-E. It is not uncommon that the OMC and MDD of the materials change with the concentration of the additives. The MD curve should be obtained by adding the preliminary concentration of the additive estimated from pH tests.

It is prudent that the specimens prepared for establishing the MD curve are wrapped in cellophane paper or covered with a membrane for 24 hours and then subjected to the unconfined compressive tests (UCS). In the course of this and other similar studies, it has been found that:

- If the maximum UCS obtained in that manner is substantially (preliminary 50%) less than the target strength (say 50 psi for lime), the effectiveness of the additive in stabilizing the soil is doubtful.
- The variation in UCS with moisture is an indication of the robustness of the mix to construction. If the UCS vary significantly with moisture (especially at moisture contents wet of optimum), additional inspection efforts are needed to ensure a quality final product.

Compacting of Clayey Materials
The specimens should be prepared at a moisture content equal to the OMC and to a density equal to the MDD. The use of a static compactor is suggested in Appendix C for the preparation of the clayey specimens (see Chapter Four for justification). Briefly, the appropriate amount of soil, additive and moisture is combined and mellowed in accordance with the current Tex-114-E. The material is then placed in one lift into the static compaction mold and pressed to the desired density using a number of plungers.

Curing of Specimens
The most representative curing method found in this study consists of placing the specimens in a 104^{O}F oven for 48 hours.

Moisture Conditioning of Specimens
Backpressure saturation as described in Appendix C is proposed to accelerate the moisture conditioning of prepared specimens. This method can typically moisture condition the specimens to saturation in less than 24 hours with the distribution of moisture within the specimen being more uniform as compared to the capillary moisture conditioning method currently used. Another alternative is to use vacuum saturation as also explained in Appendix C.

Establishing Final Stabilizer Content based on Performance Indicators
The primary performance indicator proposed is the unconfined compressive strength (UCS) <u>before and after</u> moisture conditioning. Another parameter that is considered is the retained strength defined as:

$$\text{Re}\, tained\ strength = \frac{Strength\ after\ MoistureConditioning}{Strength\ after\ Curing} \tag{7.4}$$

The secondary parameters are the modulus and the long-term durability of the mix. The optimum stabilizer content is defined as the lowest stabilizer content necessary to satisfy the project requirements as summarized in Table 7.1. Specimens with stabilizer contents ranging from the stabilizer content equal to that obtained from the pH tests to up to 4% above the content obtained from the pH tests should be considered.

Table 7.1 – Strength Requirements for Accelerated Moisture Conditioning Methods

UCS, psi		Retained Strength	Remarks
After Curing	After Moisture Conditioning		
>65	>50	>0.8	Mix is Appropriate
>65	>35	>0.6	Mix is Reasonable, Perform Clay Mineralogy Tests (CEC, SSA and TP)
>50	>25	>0.5	Perform Long-term Durability and Clay Mineralogy Tests (CEC, SSA and TP)

Durability Studies (Wetting-Drying Cycles)

To address the durability of chemical treated soils, a series of specimens prepared using the static compactor should be subjected to various cycles of wetting and drying processes as specified in Appendix C. During wetting and drying periods changes in the sample dimensions are measured vertically and radially. The wetting/drying is continued for 21 cycles. After 3, 7, 14 and 21 cycles the samples are subjected to the UCS tests. A good preliminary relationship between the retained strength and the durability as per this test has been observed. Since this test procedure may take more than a month to determine if the additive is durable, this test can be performed when the retained strength as described in Table 7.1 is less than 0.6.

Establishing Modulus

The resilient modulus of the material as per AASHTO T-307 is needed for pavement design. Currently, TxDOT is not fully equipped to perform these tests routinely. The free-free resonant column (proposed Tex-149) tests can be used in the absence of the resilient modulus test set up to estimate the modulus of the mix. The same specimens prepared to determine the final stabilizer content based on UCS can be used for this purpose. Preliminary, half the value obtained from the FFRC tests can be used as an estimate of the modulus of the layer.

CHAPTER EIGHT – SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

To ensure the performance of stabilized subgrades overtime, it is necessary to conduct some laboratory tests on the material to provide a proper mix design. Currently in TxDOT Tex-120-E, Tex-121-E and Tex-127-E are the test procedures used to stabilize soils with cement, lime and fly-ash, respectively. Since most of these procedures are very time-consuming (up to 27 days), in many cases the percentage of additives selected has been based on previous experience. The main objective of this research project was to accelerate the time required for determination of the proper mix design for subgrade stabilization.

Selection of type and concentration of additive has been based on physical characteristics of soils, such as gradation and plasticity index. However, a number of other parameters such as the interaction between the mineralogy of the materials and additives in presence or absence of moisture, construction methods and curing processes significantly impact the performance of stabilized subgrades.

A comprehensive database for nine types of soils from different regions of Texas with different characteristics and different stabilizers was developed. Results based on these tests and those obtained from specimens subjected to new methods of curing and moisture conditioning were compared. Several parameters including strength, moisture content, dielectric values and variation of modulus with time were considered to achieve the goal of developing new and accelerated methods that are reliable substitutes for current specifications. Based on this evaluation, the following recommendations are made:

- The static compacted method is recommended since it provides specimens with uniform density. In addition, the moisture conditioning for the static compacted specimens is accomplished more rapidly and uniformly.
- A new accelerated protocol is proposed that could minimize the time required for curing, conditioning and testing to complete the design process. The new protocol addresses some of the shortcomings that exist in the current practice to establish whether the stabilizer or stabilization method is deemed to be effective in the field construction projects.
- Several test methods to determine various chemical characteristics of the soils according to their mineralogy are also recommended. Cation Exchange Capacity, Specific Surface

Area, Total Potassium, Exchangeable Potassium and Reactive Alumina can be used to substitute more costly and time consuming methods such as X-ray Diffraction and Scanning Electron Microscope.

Based on the outcome of the research, the following recommendations are presented for the curing, specimen preparation, and moisture conditioning methods of stabilized subgrades:

- The simple chemical test methods presented combined with simple models developed can be used to obtain clay mineralogy of the fine fractions of the soil easily. If Montmorillonite is the dominating mineral, higher stabilizer contents and long-term durability tests should be considered.
- pH tests are good indicators to obtain initial amount of stabilizer when lime is used. However, more tests are needed to complete a reliable and durable mix design.
- In most cases of kneading compacted specimens as per current TxDOT specifications, the moisture did not reach the top of the specimen in capillary suction. In many clay specimens, a weak plain developed at the interface of two lifts where the specimen either broke or did not become completely saturated during moisture conditioning. The static compaction method appeared to overcome these problems.
- Tests showed the significance of density on strength. Specimens compacted at 95% of maximum dry density experienced a drop in strength of about 50% compared to 100% density.
- Temperature was found to be another key factor on the curing process. Strengths of specimens cured at 140°F were consistently less than those cured at 104°F. Also, high curing temperatures provoked cracking in specimens. A curing temperature of 104°F is recommended to prevent damage to specimens during oven-curing.
- Back pressure and vacuum saturation tests are recommended to be employed as a substitution for current TxDOT specifications to complete the moisture conditioning of stabilized specimens. In almost all cases saturation was achieved within 1 day.
- Vacuum saturation is faster and easy to apply. However, back pressure saturation is recommended, since it is more controllable than vacuum saturation. Confining and back pressure levels can be easily controlled, whereas it is a difficult and expensive task for vacuum systems. Besides, back pressure is also capable of providing information related to the permeability of the soil.
- The alternative moisture conditioning methods generally resulted in lower strengths than current specifications. This was attributed to the higher final moisture contents obtained with e new method.
- When the TST two-day oven curing process was used before the moisture conditioning, the backpressure method provided similar results to the capillary moisture conditioning method when the moisture penetrated throughout the length of the TST specimens.
- Two series of tests were carried out to address the permanency and durability of the chemical treatment.
- Leachate tests could be carried out to establish leaching problems associated with rainfall infiltration. It was found that the leach rate was low and may not be problematic initially after stabilization and for high dosages of additive (6% or higher). This could be an issue if the original stabilizer dosage used is less than 4%.

- To evaluate the durability of the chemical treatment wetting and drying cycles are suggested. Stabilized soils with Montmorillonite as a dominant mineral are more susceptible to durability problems in particular when these soils are exposed to volume changes caused by swell and shrink.
- The current approach of PI based chemical stabilizer does not provide any insight into the chemical stabilizer and its durability. Hence, to perform a more realistic design, it is recommended to include clay mineralogy aspects.
- Based on the overall results, accelerated design methods could be used to substitute traditional TxDOT specifications. Mix design could be completed in 3 days in the laboratory, including 2 days in oven-curing at 104 ºF (40 ºC) and 1 day for moisture conditioning using back pressure saturation.
- New specifications that include strengths requirements after curing and after moisture conditioning and criteria based on retained strengths are presented to establish new acceptance criteria of the mix design for stabilized soils.
- Continuation of the study could consist of validation of the testing protocols based on the obtained new sites throughout the State so a bigger database could be completed. This will help to gain confidence on the alternative mix design presented in this study.

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APPENDIX A

SITE SELECTION

SURVEY RESPONSES

We have conducted a survey to identify the most relevant aspects about subgrade stabilization that are currently observed throughout the State.

Survey responses were received from the following 15 districts: Amarillo, Atlanta, Austin, Beaumont, Bryan, Dallas, Fort Worth, Lubbock, Lufkin, Paris, San Antonio, Tyler, Waco, Wichita Falls, and Yoakum. The responses of each district to the survey questions are summarized in this section. Each of the questions presented in the text below is followed in most cases by a figure and a detailed discussion of the answers provided.

Part I

Q.1. What type and percentage of subgrade do you have in your district?

As reflected in Figure A.1, by far the clay is the most prevalent subgrade type, followed by sand. Even though there is only 10% of the subgrade is classified as silt, these are the most troublesome to stabilize.

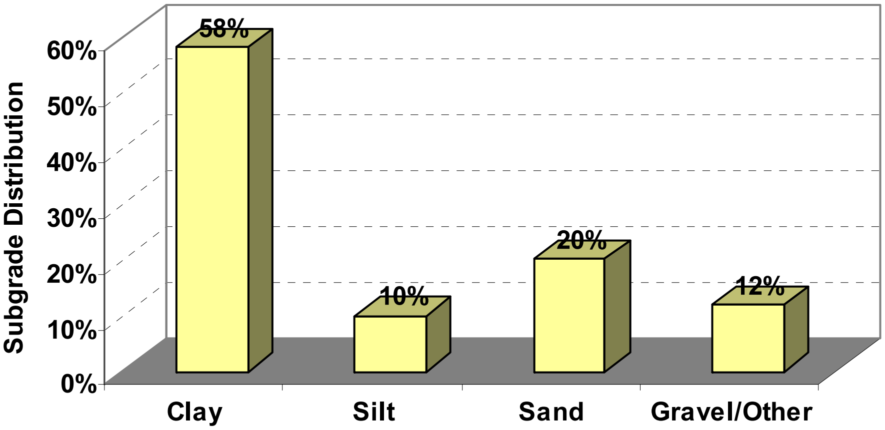

Figure A.1 – Distribution of Subgrade Types in State

Q.2. What is the typical PI range, if applicable?

The average PIs as reported by the districts are summarized in Figure A.2. For clays the average PI is about 38, with a range varying from a maximum of 70 to a minimum of 10. The silts demonstrate an average PI of 13. Surprisingly, not all sands reported to have a PI of close to zero.

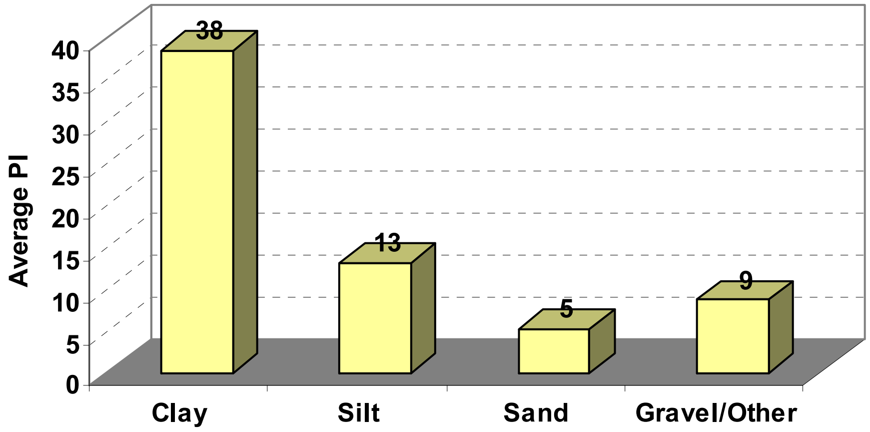

Figure A.2 – Distribution of Plasticity Indices for Subgrade Types in State

Q.3. Does the soil contain reactive sulfate?

As reflected in Figure A.3, a majority of the clays (57%) contain reactive sulfate. As such, the impact of sulfates on the stabilization should be considered. About 20% of the silts and sands also reported to contain sulfates.

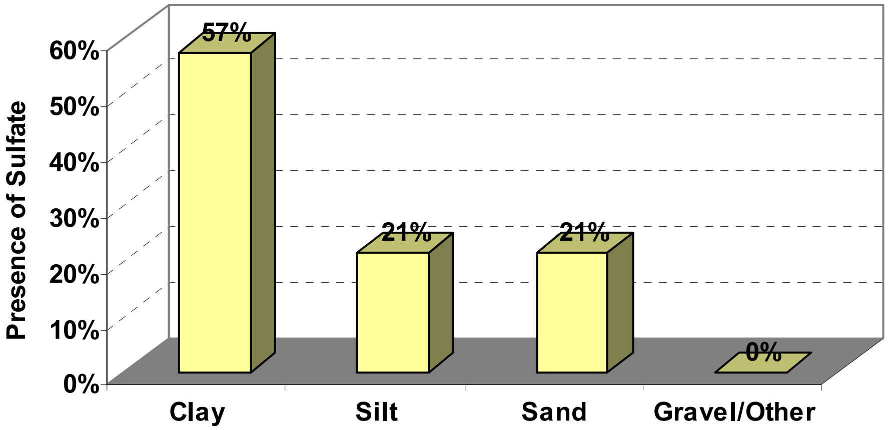

Figure A.3 – Distribution of Reactive Sulfates for Subgrade Types in State

Q.4. Do you use stabilization for your subgrade?

As reflected in Figure A.4, all clays and 80% of sands are stabilized. While only half of the silts are stabilized.

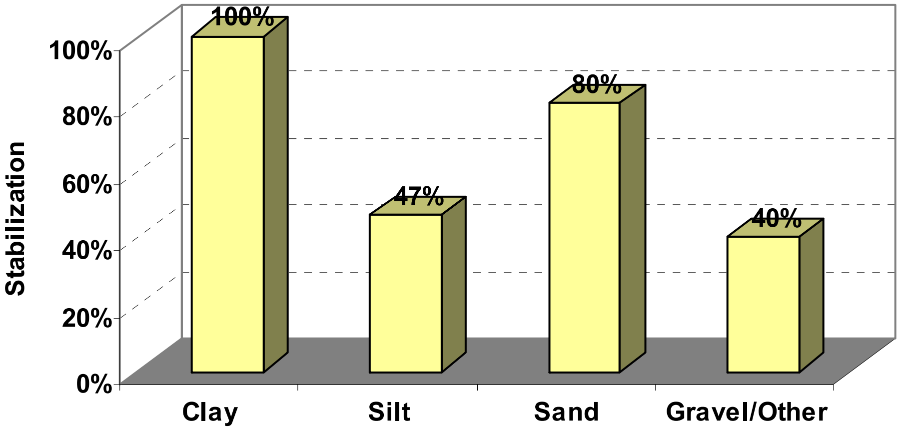

Figure A.4 – Distribution of Stabilization for Subgrade Types in State

Q.5. If Yes, what type of stabilizer do you use?

Almost all clays (93%) are stabilized with lime, followed by 33% of the districts using cement primarily to change the PI of the clays. Vast majority of the silts are stabilized with either lime or cement. This trend is understandable since the silts are the least understood materials in the State. As expected, sands are primarily stabilized with cement. Gravels are treated with either cement or lime (see Figure A.5).

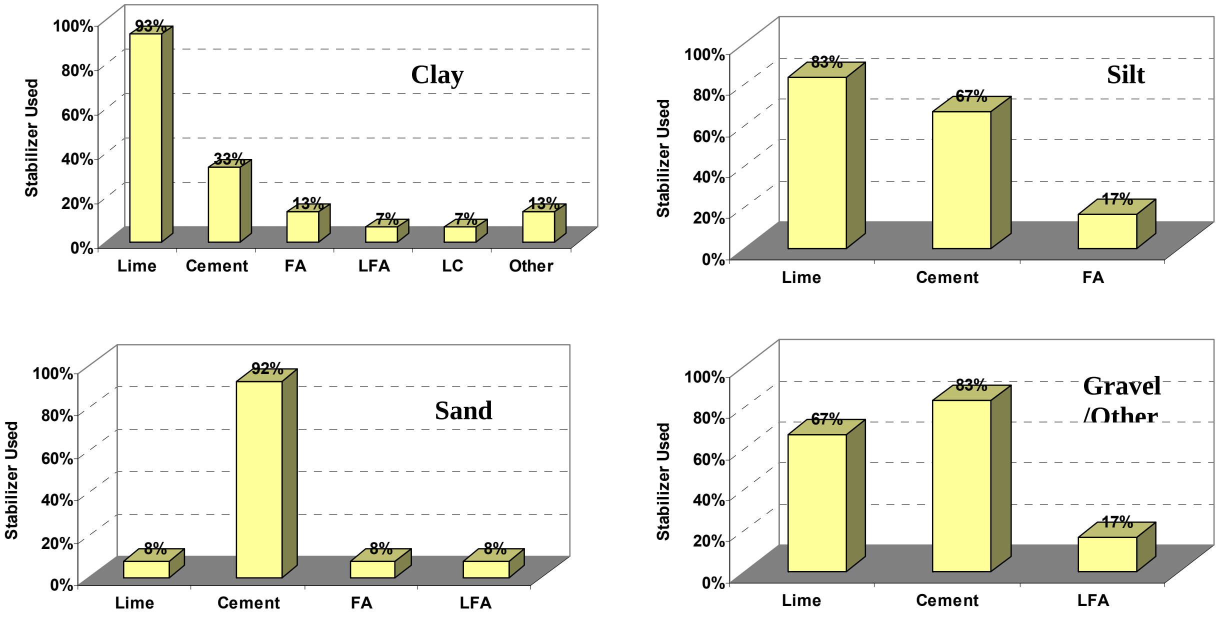

Figure A.5 – Distribution of Stabilization Types for Subgrade Types in State

Part II

Q.1. What factors are considered to select this type of subgrade stabilization for these projects?

All districts that responded consider the type of material at hand for stabilization (see Figure A.6). Cost of project is also significant for more than 70% of the districts. The volume of traffic and the environmental conditions seem to be of less of importance for the districts.

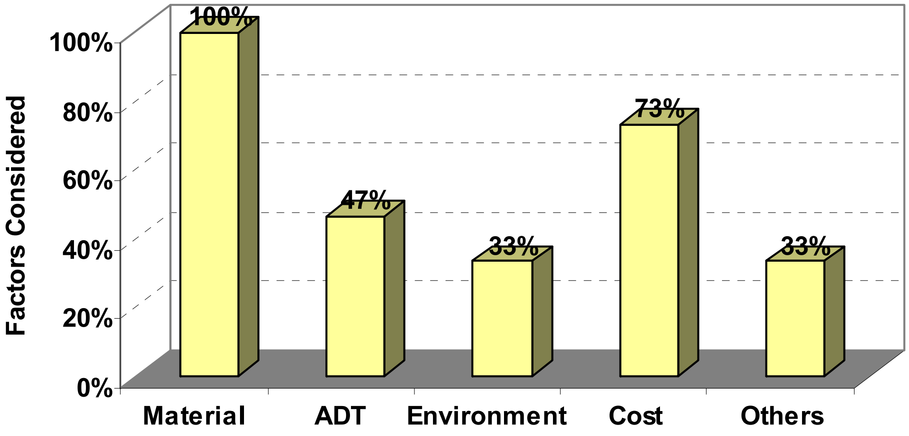

Figure A.6 – Distribution of Parameters Considered for Selection of Additives

Q.2. Do you follow any specifications or lab test procedures to select type and amount of stabilizing agent?

As reflected in Figure A.7, 20% of districts simply specify the percentage of additives based on their experience. The other 80% utilize some type of lab tests to select the concentration of additives. The level of effort for these districts vary from very simple, a ph test, to comprehensive (considering the strength, moisture susceptibility, sulfate content etc.).

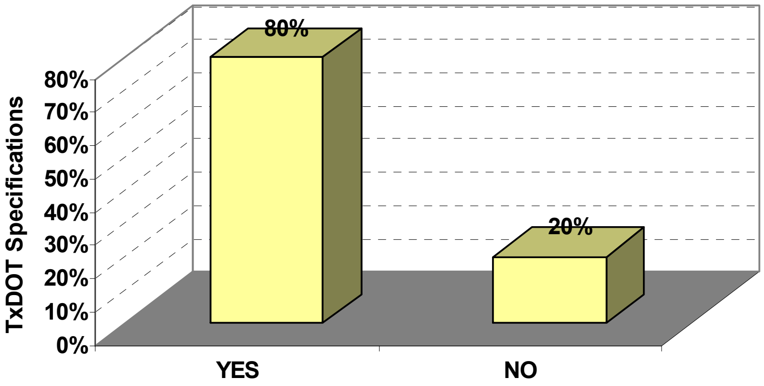

Figure A.7 – Distribution of Districts that Consider Lab Tests for Selection of Additives

Q.3. Do you also conduct any test to check for long-term performance of stabilized subgrade?

As reflected in Figure A.8, more than 70% of the districts do not check the long-term performance of their materials. FWD tests are the most prevalent test for districts that do monitor the long-term performance.

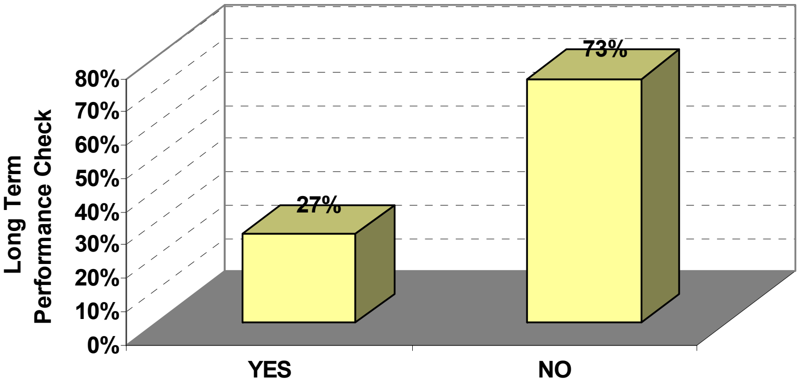

Figure A.8 – Distribution of Districts that Monitor Long-Term Stability of Their Projects

Part III

Q.1. If you used stabilized subgrade in the past, how would you rate the performance of these subgrades?

As reflected in Figure A.9, all but one district is either satisfied or very satisfied with the performance of their stabilized pavements.

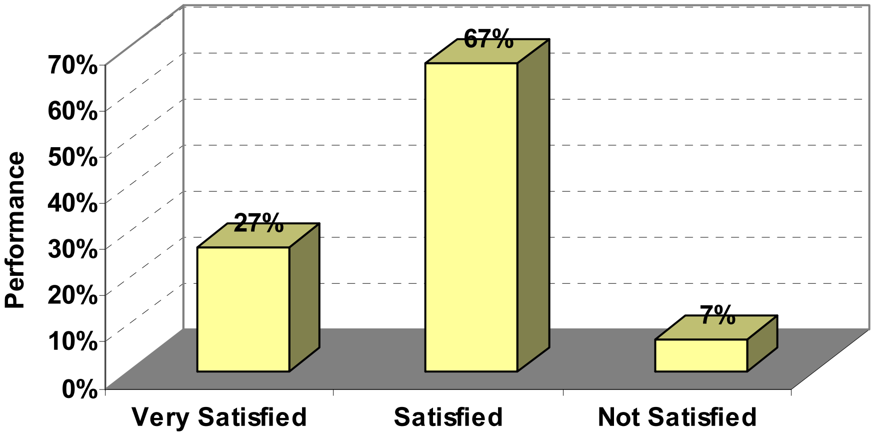

Figure A.9 – Distribution of Overall Satisfaction of Districts with Performance of Their Projects

Q.2. What types of performance problems, if any you encountered that can be attributed to the subgrade?

All but two districts have historically experienced cracking in their projects primarily because of two different reasons. Most districts perceived the lack of permanency of the stabilization as the cause of cracking, while several others attributed the problem to too much additives causing transverse cracking. Rutting and ride quality were also perceived as problems.

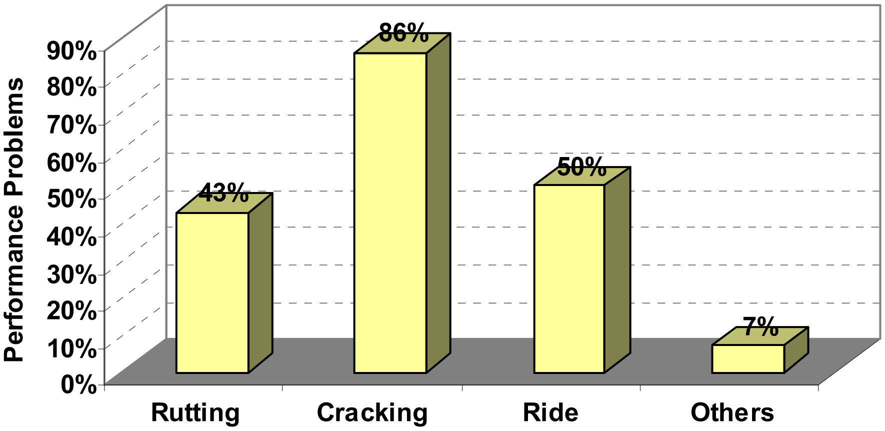

Figure A.10 – Distribution of Distress Types Reported by Districts

SELECTION OF MATERIALS FOR THIS PROJECT

Based on the responses received from the districts and based on the literature survey reported in our first technical memorandum, a recommendation for the selection of soils to be considered in this study is provided below.

In summary, we are recommending the following five materials to initiate the project:

1. Clay from Fort Worth
2. Silt from Amarillo
3. Sand from Wichita Falls
4. Two more clay based on mineralogical analysis from either Bryan, Paris or San Antonio.

The number of materials will be expanded in the second year of project based on the results from this phase. The rational for selecting these materials are provided below.

Since the goal of this study is to accelerate the process of stabilization design and to incorporate the long-term performance of the mixes, the parameters deemed important are:

1. Distribution of materials within the State. Since majority of the subgrades in the State are clays, much attention should be placed on this material. We recommend that two or three of the soils considered being clay, along with one silt and one sand.

2. Mineralogy of materials. As reflected in Figures A.2 and Figures A.3, the PI and reactive sulfate contents of the clays vary significantly. Also, the mineralogical aspects of the clays are quite different. As such we recommend that mineralogical studies to be carried out on a number of clays throughout the State so that the clays would be different. To that end, we are recommending clays from Bryan, Fort Worth, Paris, and San Antonio. These districts are considered because they cover the major clay formations in the State and because instrumented sites in these formations already exist. To complete the tasks of this project in a timely manner, we will recommend that the Fort Worth clay be considered as one material. Fort Worth clay has been known as problematic clay for a long time. In addition, their selection of stabilizer is based on experience.
3. Permeability of materials. It is well-documented that the permeability of the raw material has an impact on accelerating the mix design. Also, the permeability of the raw material may be a factor in the permanency and leaching of the stabilizer. To that end, we propose that a sandy subgrade be considered in the work plan. Based on the returned surveys, we would like to recommend sand from Wichita Falls. They have a significant percentage of sand as subgrade in their district.
4. Historical Performance. Based on the number of forensic studies and problems reported, silts are the least understood materials in terms of stabilization. We recommend that a silt be considered for this reason. We recommend a silt from Amarillo for this purpose. Amarillo reported 50% silt in their district.

APPENDIX B

XRD TEST RESULTS

Table B.1 – Basal d-spacing Matches for Bryan Soil before Treatment

d-spacing	%Intensity	Illite	Kaolinite	Mont.	Quartz
3.33	100.00	★			★
3.36	35.80				★
1.37	20.10	★			
1.37	19.20	★			
4.23	15.70		★		
1.82	11.30				
1.38	10.60				★
14.52	10.40			★	
14.10	10.40			★	
1.54	8.50	★			
2.28	7.20		★		
2.45	6.30	★			
4.49	6.00	★			
1.67	5.90				
1.50	5.30	★			
2.13	5.10		★		
13.59	5.00			★	
4.48	5.00	★			

Note: ★---- Mineral Present

Table B.2 – Basal d-spacing Matches for Bryan Soil after Treatment

d-spacing	%Intensity	Scolecite (CASH)	Prehnite (CASH)	Tobermorite (CSH)	Illite	Kaolinite	Mont.	Quartz
3.35	100.00				★			★
4.26	16.60			★				
1.38	15.70							★
1.82	14.90	★						
3.04	14.30		★					
1.37	12.10			★				
2.46	9.70				★			
4.38	9.30			★		★		
1.54	8.60				★			
4.53	7.70		★	★				
4.49	7.70				★			
2.28	7.60					★		
14.54	7.20		★				★	
4.44	6.50	★			★			
14.99	5.90			★			★	
2.57	5.30					★	★	
2.13	5.20		★			★		
1.67	5.00				★			

Note: ★---- Mineral Present

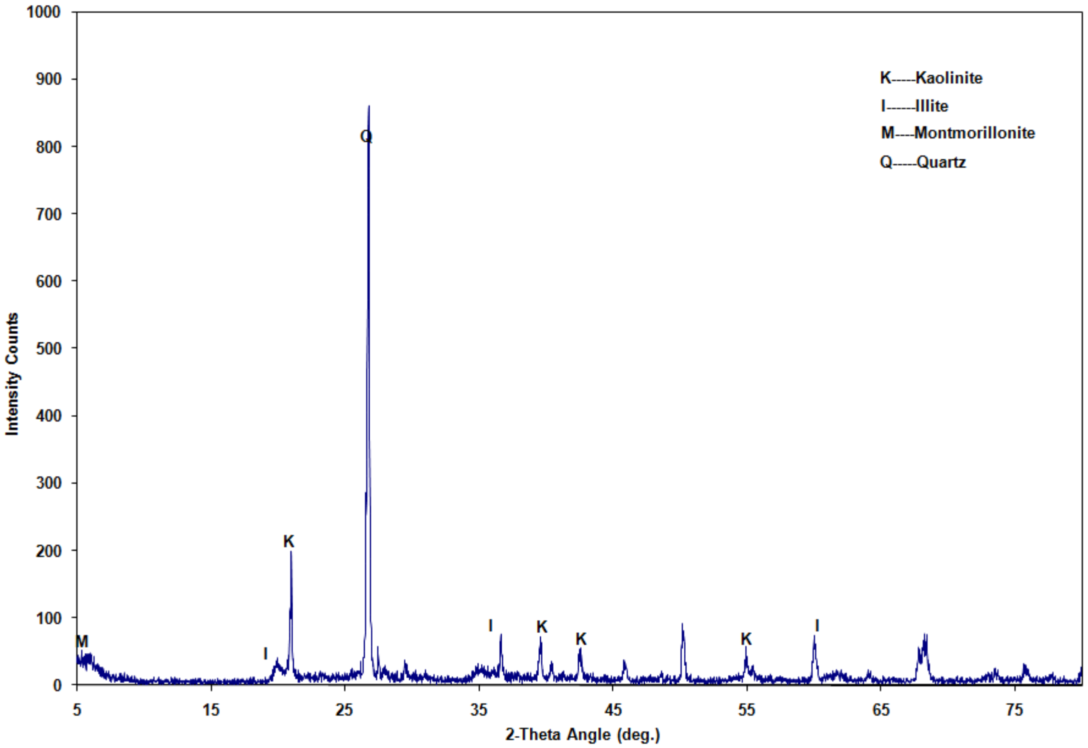

Figure B.1 – XRD Results of Bryan Soil before Treatment

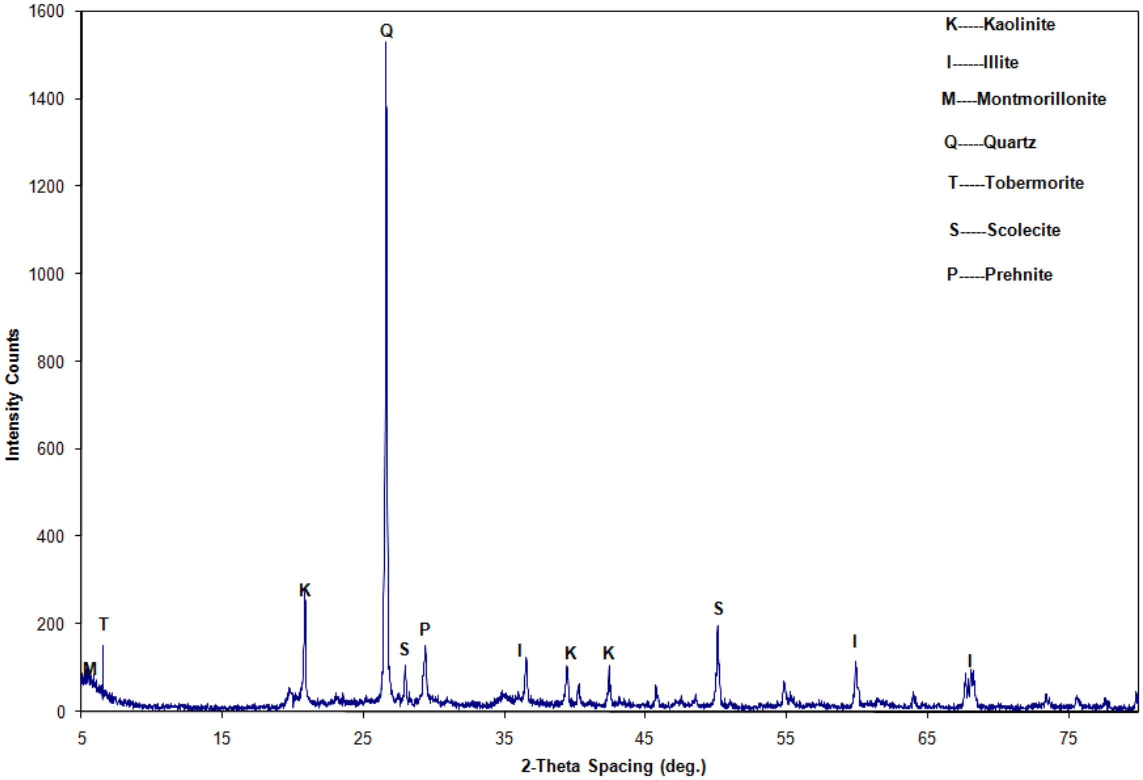

Figure B.2 – XRD Results of Bryan Soil after Treatment

Table B.3 – Basal d-spacing matches for El Paso Soil before Treatment

d-spacing	%Intensity	Illite	Kaolinite	Mont.	Quartz
3.36	100.00	★			★
3.22	24.20	★			
1.82	20.60	★			
3.04	20.10	★			
1.38	19.70	★			
3.20	17.10	★			
1.54	14.40	★			
2.59	10.00			★	
2.58	10.00			★	
1.38	9.90				★
10.32	9.80	★			
4.48	9.40	★			
2.57	9.30			★	
2.29	9.00		★		
2.46	8.00	★			
10.07	6.90	★			
1.37	6.80	★			
2.98	5.80			★	

Note: ★---- Mineral Present

Table B.4 – Basal d-spacing matches for El Paso Soil after Treatment

d-spacing	%Intensity	Scolecite (CASH)	Prehnite (CASH)	Tobermorite (CSH)	Illite	Kaolinite	Mont.	Quartz
3.35	100.00				★			★
3.04	56.60			★			★	
3.22	33.00				★			
1.91	29.30				★	★		
1.38	24.30				★			
4.56	23.10		★		★		★	
4.27	21.90			★			★	
3.78	18.20			★				
1.54	16.00		★					
4.38	15.90				★			
1.37	15.60							★
3.20	14.40	★			★			
1.82	14.40			★				
4.49	14.30				★	★		
2.13	13.00		★			★		
2.29	12.90				★			
2.46	8.20				★			
1.88	8.00	★						★
1.67	7.10		★		★			
3.29	7.00		★					
14.25	6.60			★				

Note: ★---- Mineral Present

Table B.5 – Basal d-spacing Matches for Fort Worth Soil before Treatment

d-spacing	%Intensity	Illite	Kaolinite	Mont.	Quartz
3.37	100.00				★
3.05	48.80				
4.27	34.90	★	★		
16.11	32.20			★	
4.50	28.40		★		
4.53	26.20		★		
2.59	23.40			★	
14.39	20.10			★	
2.56	19.20	★			
2.57	18.90			★	
14.96	18.30			★	
1.37	18.00	★			
1.38	17.80				★
2.29	16.00		★		
1.82	14.70	★			
1.92	13.50		★		
4.36	11.80	★			
1.54	9.90	★			
1.88	9.30		★		
2.46	9.00	★			

Note: ★---- Mineral Present

Table B.6 – Basal d-spacing Matches for Fort Worth Soil after Treatment

d-spacing	%Intensity	Scolecite (CASH)	Prehnite (CASH)	Tobermorite (CSH)	Illite	Kaolinite	Mont.	Quartz
3.34	100.00				★			★
3.03	52.90			★				
4.25	24.00					★		
5.50	18.00			★				
2.57	17.60				★		★	
2.28	16.20							
3.30	15.00	★	★					
4.42	14.20				★			
1.83	13.30	★		★				
1.84	11.00							
1.91	9.70					★		
2.45	9.50				★			
2.99	8.80	★					★	
2.55	8.50							
2.59	8.10	★						
2.49	7.90					★	★	
2.09	7.90							
1.60	7.60							★
1.67	7.00		★					
1.54	6.50		★					
2.13	6.20		★			★		

Note: ★---- Mineral Present

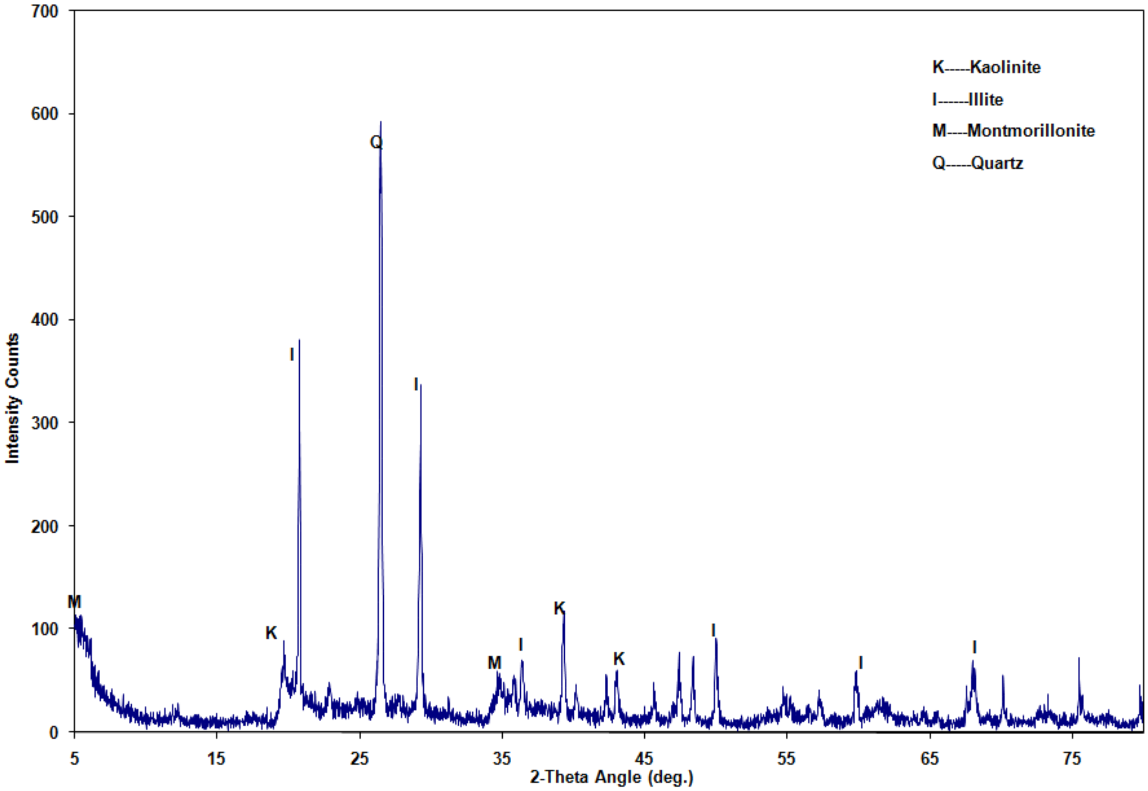

Figure B.3 – XRD Results of Fort Worth Soil before Treatment

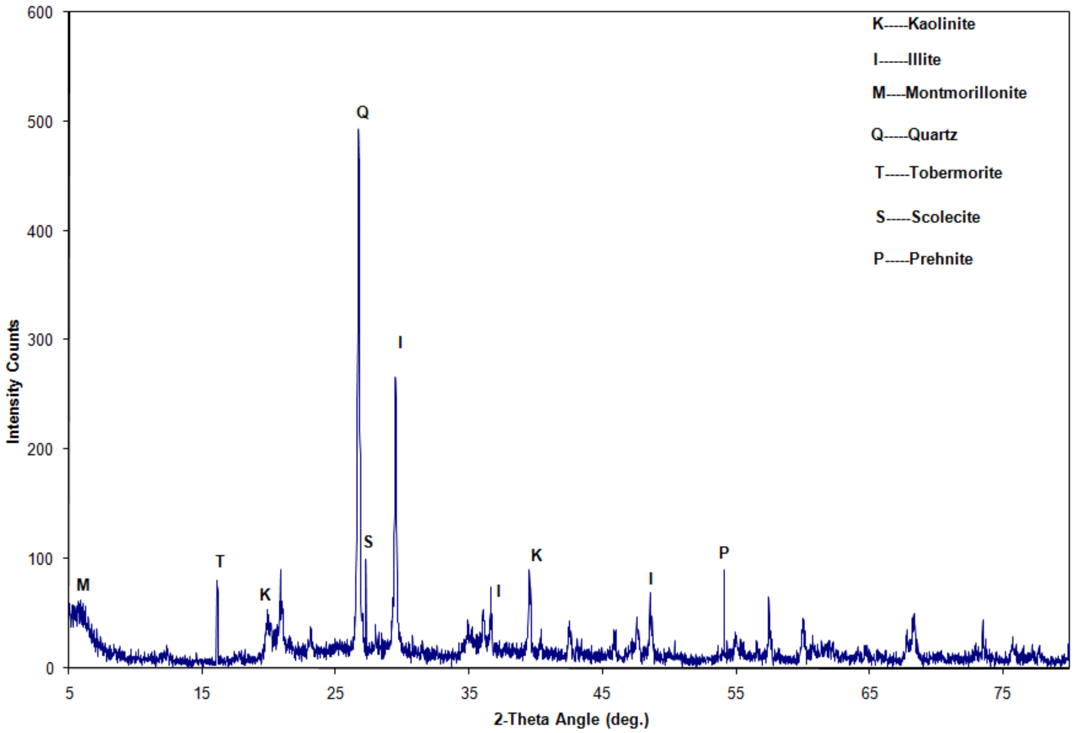

Figure B.4 – XRD Results of Fort Worth Soil after Treatment

Table B.7 – Basal d-spacing Matches for Paris Soil before Treatment

d-spacing	%Intensity	Illite	Kaolinite	Mont.	Quartz
3.35	100.00	★			★
15.07	70.40			★	
15.45	64.00			★	
14.82	38.40			★	
16.12	30.60			★	
16.92	24.80			★	
17.31	22.70			★	
4.27	22.70				★
4.52	20.70			★	
4.49	19.00			★	
4.40	15.60	★			
1.82	14.70	★			
2.57	11.20	★			
13.63	10.50			★	
2.29	10.20		★		
1.54	9.70	★			
1.38	8.40	★			
12.73	6.30			★	
1.67	6.10	★			
2.46	5.80	★			
1.49	5.70	★			
2.13	5.40		★		

Note: ★---- Mineral Present

Table B.8 – Basal d-spacing Matches for Paris Soil after Treatment

d-spacing	%Intensity	Scolecite (CASH)	Prehnite (CASH)	Tobermorite (CSH)	Illite	Kaolinite	Mont.	Quartz
3.35	100.00				★			★
4.29	27.30							
3.04	25.40			★				
1.38	17.90				★			★
1.82	16.30				★			
4.40	15.60	★						
4.48	12.70				★		★	
2.46	12.50				★			
4.37	12.40					★		
14.26	11.30			★				
1.54	9.90				★			
2.29	9.80					★		
4.62	9.60		★					
2.59	8.40	★					★	
2.57	7.20		★				★	
1.66	6.30		★		★			
1.68	6.20		★					
2.79	5.70			★				
1.99	5.20				★			★

Note: ★---- Mineral Present

Table B.9 – Basal d-spacing matches for Wichita Falls Soil before Treatment

d-spacing	%Intensity	Illite	Kaolinite	Mont.	Quartz
3.36	100.00				★
1.55	15.80	★			
1.38	12.80		★		
1.38	12.70	★			
4.33	11.70	★			
4.30	11.50	★			
1.54	8.30	★			
1.82	8.10	★			
2.14	7.40		★		
1.39	7.20		★		
2.29	6.80		★		
2.98	6.57			★	
1.67	5.33				★
1.82	5.30	★			
2.47	5.20		★		
1.29	5.20	★			
2.33	4.04				★

Note: ★---- Mineral Present

Table B.10 – Basal d-spacing matches for Wichita Falls Soil after Treatment

d-spacing	%Intensity	Scolecite (CASH)	Prehnite (CASH)	Tobermorite (CSH)	Illite	Kaolinite	Mont.	Quartz
5.00	6.00				★			
4.49	6.10						★	
4.44	5.40				★			
4.36	21.70					★		
4.22	5.90	★						
3.36	100.00							★
3.23	15.90	★						
3.01	6.40			★				
2.47	8.10				★			
2.29	7.80	★						
2.14	5.40		★			★		
1.82	20.70		★					
1.68	7.60				★			
1.55	11.70		★					
1.39	10.30				★			
1.38	16.10				★			

Note: ★---- Mineral Present

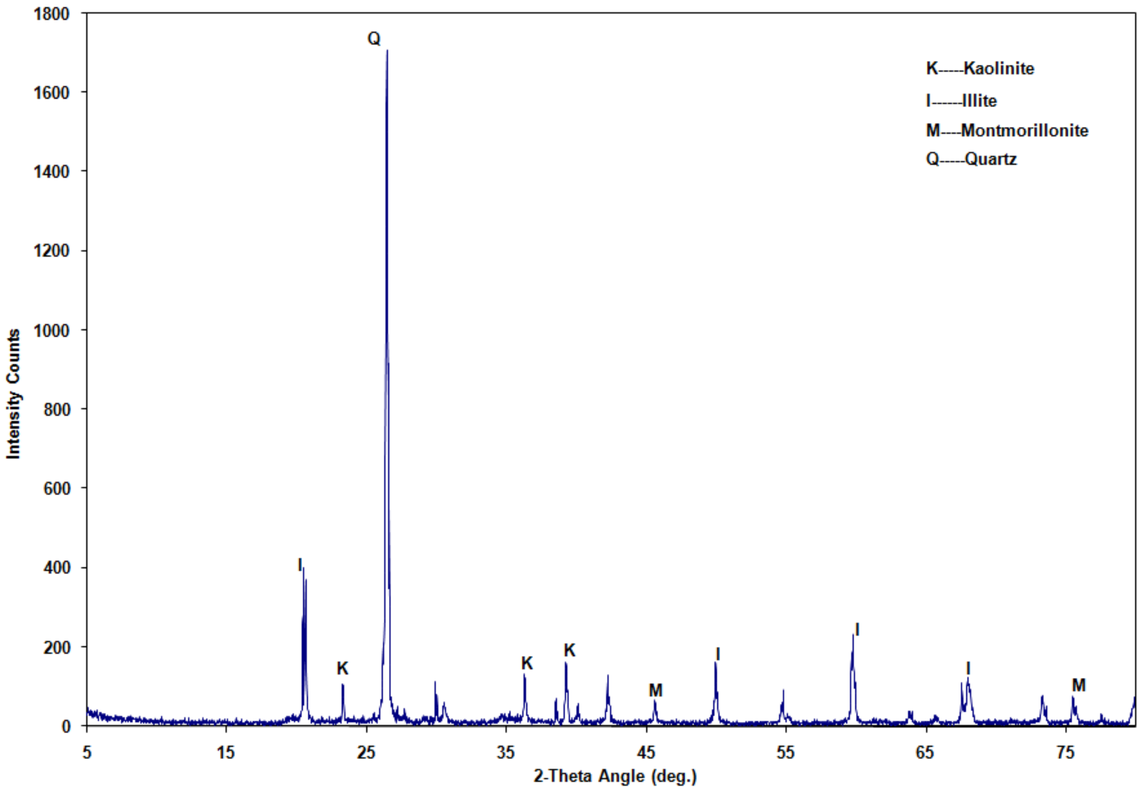

Figure B.5 – XRD Results of Wichita Falls Soil before Treatment

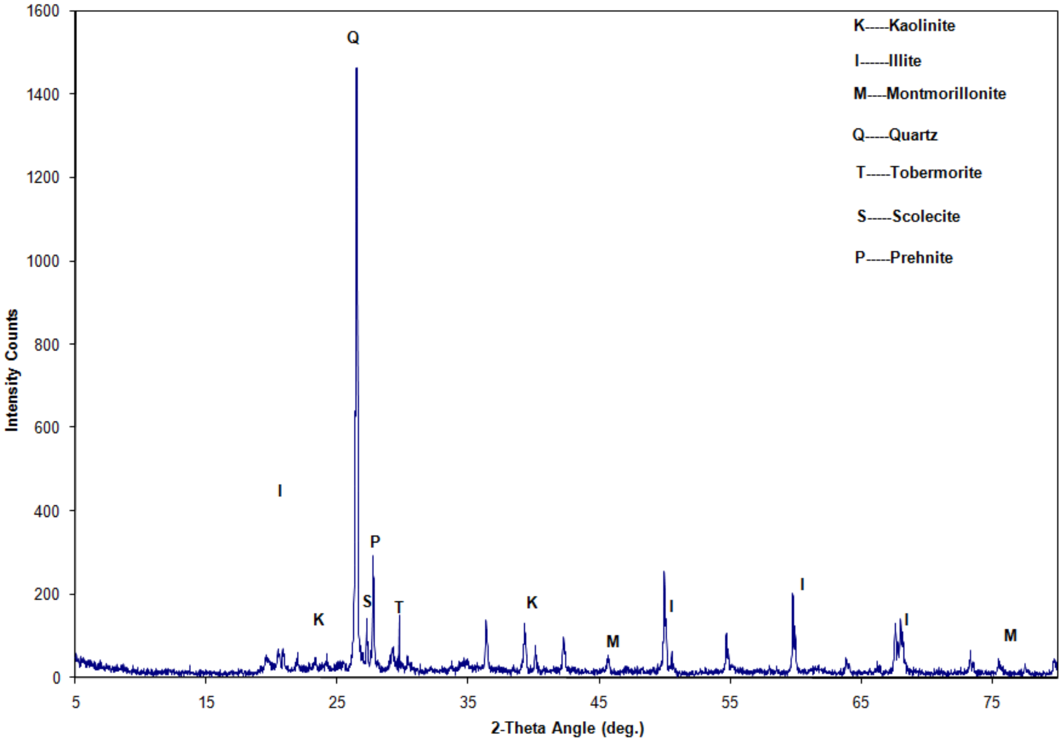

Figure B.6 – XRD results of Wichita Falls Soil after Treatment

APPENDIX C

PRELIMINARY PROTOCOLS

Determination of Reactive Alumina and Silica

Section 1
Overview

Use this method to obtain the amount of reactive alumina and silica within a clay material.

Units of Measurement

The values given in parentheses (if provided) are not standard and may not be exact mathematical conversions. Use each system of units separately. Combining values from the two systems may result in nonconformance with the standard.

Section 2
Apparatus

The following apparatus are required:

1. Spectrophotometer
2. Membrane filters 0.1μm pore size (VVLP type)

Section 3
Material

- Distilled or de-ionized water
- 0.5N Sodium Hydroxide (NaOH)

Reagents:

NaOH 0.5N: Dissolve 20 g of Sodium Hydroxide (NaOH) in 1 liter of distilled deionized water (DDI).

Test Procedure

Follow the steps below to obtain the CEC of the soil:

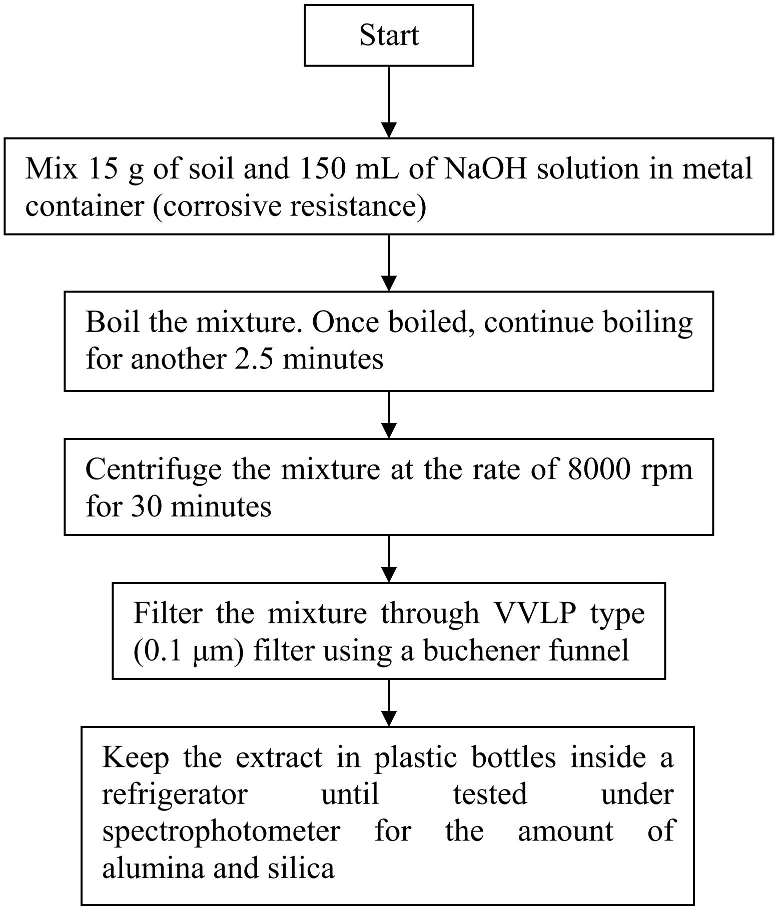

Figure C.1 – Flow Chart Showing the Test Procedure Followed for the Determination of Reactive Alumina and Silica

Cation Exchange Capacity[1]

Section 1

Overview

Use this method to obtain the cation exchange capacity of treated and untreated soil specimens in the laboratory.

Units of Measurement

The values given in parentheses (if provided) are not standard and may not be exact mathematical conversions. Use each system of units separately. Combining values from the two systems may result in nonconformance with the standard.

Section 2

Definitions

ppm — Parts per million; measure of concentration.

meq — Milliequivalent.

Filtrate — (Soil/water) material that has passed through a filter.

nm — Nanometer.

Section 3

Apparatus

The following apparatus are required:

- Mortar and pestle.
- Crusher.
- Volumetric flask (glass or plastic), 250 ml.
- Erlenmeyer side arm glass flasks, 500 ml.
- Graduated cylinder (glass or plastic), 25 ml.
- Oven, capable of maintaining a temperature of 60 ± 5°C (140 ± 9°F).
- Wide-mouth round high-density poly ethylene (HDPE) bottles, 250 ml (8 oz.).
- Buchner Funnel (glass or plastic), 5 cm diameter with short wide stems.
- Filter paper (fine porosity), 5 cm diameter.
- 25 ml glass pipettes.

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[1] **This protocol does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the user to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.**

- 1 ml glass pipettes.
- Wide-mouth round high-density poly ethylene (HDPE) bottles, 250 ml (8 oz.).
- Wash bottle, 500 ml (16 oz.), for distilled or de-ionized water.
- Latex gloves.
- Tweezers.
- Pans, brush, and spatula.
- Spectrophotometer.

Section 4

Material

- Distilled or de-ionized water
- One molar Ammonium Acetate (NH_4OAc) saturating solution: Dilute, in a chemical hood, fifty seven milliliters glacial acetic acid (99.5%) with ~800 ml of distilled water in a one liter volumetric flask. Add sixty eight milliliters of concentrated Ammonium Hydroxide (NH_4OH) mix and cool to room temperature. Adjust pH of the sample to 7.0 using NH_4OH and dilute to one liter using distilled water. (Shelf life: 5 years if stored in cool dark place at room temperature)
- One molar Potassium Chloride (KCl) replacing solution: Completely dissolve 74.5 g KCl (reagent grade) in distilled water and dilute to a final volume of one liter. (Shelf life: 5 years if stored in cool dark place at room temperature)
- Isopropyl alcohol or 2-propanal, 95%. (Shelf life: Poor under normal conditions).

Preparation of Material

The following procedure describes preparing soil samples for determining cation exchange capacity in the laboratory:

Sample Preparation	
Step	**Action**
1	Dry the soil sample (1000 g) in a 60 ± 5°C (140 ± 9°F) oven and allow it to cool to 25 ± 3°C (77 ± 5°F) in a desiccator.
2	After drying, crush, grind, and split to obtain approximately a 500 g (1 lb) representative sample that passes the 4.75 mm (No. 4) sieve.
3	Pulverize 500 g soil sample to pass the 425 µm (No. 40) sieve.
4	Split the sample to obtain three 50 g representative sample and weigh to the nearest 0.1 g.

Test Procedure

Follow the steps below to obtain the CEC of the soil:

Determination of Cation Exchange Capacity	
Step	**Action**

1	Weigh 25 g to the nearest 0.1 g of oven-dried soil and place the soil in a 250 ml (16 oz) HDPE bottle.
2	Add 125 ml of the 1 M ammonium acetate (NH_4OAc) and shake thoroughly and allow it to stand for a minimum time period of 12 hours.
3	Take a 500 ml side arm glass flask.
4	Place the Buchner funnel on top of the 500 ml flask and tighten with the help of a rubber stopper which has a hole in the center for the stem of the Buchner funnel to go through.
5	Place a retentive filter paper (# 40) in the funnel and wet the paper and apply light suction (<0.5 psi) to the stem of the Erlenmeyer flask using a rubber tubing.
6	Filter the soil solution from step 2 through the Buchner funnel into the 500 ml flask.
7	Leach the soil in the Buchner funnel four times by adding 25 ml of ammonium acetate, NH_4OAc using a pipette, allowing each addition to filter through but not allowing the soil to crack or dry.
8	Leach the soil in the Buchner funnel eight times by adding 10 ml of 95% 2-propanol using a pipette allowing each addition to filter through but not allowing the soil to crack or dry. (Discard the leachate and clean the receiving flask).
9	Leach the soil in the Buchner funnel eight times by adding 25 ml of potassium chloride (KCl) using a pipette allowing each addition to filter through but not allowing the soil to crack or dry.
10	Discard the soil and transfer the leachate to a 250 ml volumetric flask.
11	Fill the flask up to the mark with additional KCl.
12	Determine the amount of Ammonia (NH_4) in the solution by using the spectrophotometer.

Calculations:

Use the following formula to calculate the CEC of the soil sample:

If the concentration ammonia is expressed as ppm of Ammonia Nitrogen (NH_3-N), then CEC can be calculated as follows:

$$CEC(meq/100g) = \frac{\mathrm{NH_3\text{ - }N\ (ppm)}}{14} \times 4 \qquad \text{(C.1)}$$

If the concentration ammonia is measured (also expressed as Ammonium (NH_4)) in ppm, then CEC can be determined with the following expression:

$$CEC(meq/100g) = \frac{\mathrm{NH_4\ (ppm)}}{18} \times 4 \qquad \text{(C.2)}$$

Specific Surface Area[2]

Section 1

Overview

Use this method to obtain the specific surface area of treated and untreated soil specimens in the laboratory.

Units of Measurement

The values given in parentheses (if provided) are not standard and may not be exact mathematical conversions. Use each system of units separately. Combining values from the two systems may result in nonconformance with the standard.

Section 2

Apparatus

The following apparatus are required:

- Mortar and pestle.
- Crusher.
- Oven, capable of maintaining a temperature of 60 ± 5°C (140 ± 9°F).
- Oven, capable of maintaining a temperature of 110 ± 5°C (230 ± 9°F).
- Oven, capable of maintaining a temperature of 210 ± 5°C (410 ± 9°F).
- Latex gloves.
- Vacuum desiccator (25 cm or larger in diameter)
- Vacuum pump
- Aluminum container (7.5 cm in diameter and 2.5 cm in height)
- Culture chambers (Glass dishes with cover, 20 cm diameter And 7.5 cm height)
- Hardware cloth
- Tweezers.
- Pans, brush, and spatula.
- Dessication chamber.

Section 3

Material

- Distilled or de-ionized water
- Ethylene Glycol Monoethyl Ether (EGME) reagent grade
- Calcium Chloride ($CaCl_2$), Passing 40 mesh (0.425 mm opening) anhydrous reagent grade.

---

[2] **This protocol does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the user to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.**

Preparation of Material

The following procedure describes preparation of $CaCl_2$-EGME Solvate for determining the specific surface area in the laboratory:

Preparation of $CaCl_2$-EGME Solvate	
Step	**Action**
1	Weigh 120 g of calcium chloride, $CaCl_2$ into a beaker and oven dry at 210°C for 1hr and then weigh 100 g after oven drying.
2	Weigh 20 g of EGME into a 400 ml beaker and add the calcium chloride, $CaCl_2$ immediately after taking out from oven and mix with a spatula to make a solvate.
3	After the solvate is cooled to room temperature, transfer it to a culture chamber (glass dish with a cover) and spread it uniformly over the bottom of the chamber and store it in a desiccator.

The following procedure describes preparing soil samples for determining specific surface area in the laboratory:

Sample Preparation	
Step	**Action**
1	Dry the soil sample (500 g) in a 60 ± 5°C (140 ± 9°F) oven and allow it to cool to 25 ± 3°C (77 ± 5°F) in a desiccators.
2	After drying, crush, grind, and split to obtain approximately a 250 g (0.5 lb.) representative sample that passes the 4.75 mm (No. 4) sieve.
3	Pulverize the 250 g sample to pass the 425 µm (No. 40) sieve.
4	Split the sample to obtain three 50 g representative sample and weigh to the nearest 0.1 g.

Test Procedure

Follow the steps below to obtain the SSA of the soil:

Determination of Specific Surface Area	
Step	**Action**
1	Weigh 1.1 g to the nearest 0.001 g of oven-dried soil and place the soil in an aluminum container.
2	Place the container in an oven at 110°C (230° F) for 24 hours until constant weight is reached.
3	Weigh the dried sample taking care not to adsorb atmospheric water (W_S).
4	Add 3 ml of EGME to the sample and mix it thoroughly.
5	Place the container with slurry (EGME/Soil) into the culture chamber containing $CaCl_2$-EGME solvate onto the hardware cloth in the culture chamber.

6	Close the lid of the chamber and place it in a dessiccator containing $CaCl_2$.
7	Wait for 30 minutes and evacuate it for 45 minutes and close the stopcock.
8	After 6 to 7 hours, release the vacuum.
9	Weigh the soil sample in the aluminum can along with the lid.
10	Repeat weight measurements for every 2 to 4 hours until there is no further decrease in the weight (W_a). The weight will typically stabilize within 12 hrs.

Calculations:

Use the following formula to calculate the specific surface area, A, of the soil sample:

$$A = \frac{W_a}{W_s * 0.000286} m^2 / g \qquad \text{(C.3)}$$

where,

W_a = Final weight of the soil sample after stabilizing with ethylene glycol monoethyl ether, EGME

Ws = Dry weight of the soil sample before mixing ethylene glycol monoethyl ether, EGME

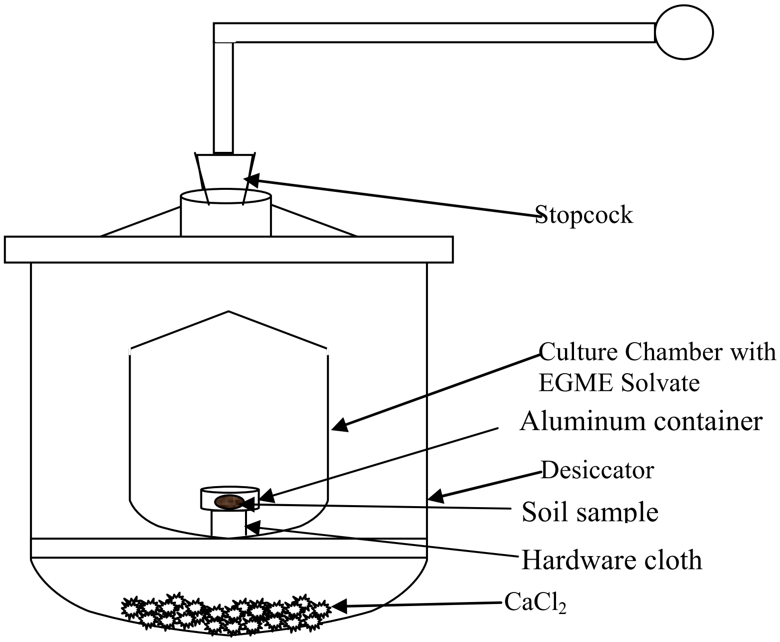

Figure C.2 – Schematic of the Specific Surface Area Apparatus

Table 1 – Work Sheet for SSA Measurements

SNo.	SOIL NAME	EMPTY WT. OF CONTAINER, W1	WT OF CONTAINER WITH DRIED SOIL SAMPLE, W2	Wa = W1-W2	INNITIAL WT. AFTER ADDING EGME	STABILIZED WEIGHT				WS=FINAL STABILIZED WT – W1	SSA
						After 6 hrs	After 8 hrs	After 10 hrs	After 12 hrs		
1											
2											
3											
4											

Total Potassium[3]

Section 1
Overview

Use this method to obtain the total potassium of treated and untreated soil specimens in the laboratory.

Units of Measurement

The values given in parentheses (if provided) are not standard and may not be exact mathematical conversions. Use each system of units separately. Combining values from the two systems may result in nonconformance with the standard.

Section 2
Definitions

ppm — Parts per million; measure of concentration.

Filtrate — (Soil/water) material that has passed through a filter.

nm — Nanometer.

Section 3
Apparatus

The following apparatus are required:

- Mortar and pestle.
- Crusher.
- Volumetric flask (glass or plastic), 50 ml.
- Graduated cylinder (glass or plastic), 25 ml.
- Oven, capable of maintaining a temperature of 60 ± 5°C (140 ± 9°F).
- 25 ml glass pipettes.
- 1 ml glass pipettes
- Wash bottle, 500 ml (16 oz.), for distilled or de-ionized water.
- Latex gloves.

---

[3] **This protocol does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the user to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.**

- Tweezers.
- Pans, brush, and spatula.
- Teflon digestion vessel, 60ml
- Hotplate, capable to rise temperatures to 200°C
- Chemical hood
- Spectrophotometer.

Section 4

Material

- Distilled or de-ionized water
- 65% Hydrofluoric acid
- 70% Perchloric acid
- 6 N Hydrochloric acid

Preparation of Material

The following procedure describes preparing soil samples for determining total potassium in the laboratory:

Sample Preparation

Step	Action
1	Dry 500 g of soil sample in a 60 ± 5°C (140 ± 9°F) oven and allow it to cool to 25 ± 3°C (77 ± 5°F) in a desiccator.
2	After drying, crush, grind, and split to obtain approximately a 250 g (0.5 lb) representative sample to pass the 4.75 mm (No. 4) sieve.
3	Pulverize the 250 g sample to pass the 75 µm (No. 200) sieve.
4	Split the sample to obtain three 5 g representative sample and weigh to the nearest 0.1 g.

Test Procedure

Follow the steps below to obtain the total potassium of the soil:

Determination of Total Potassium

Step	Action
1	Weigh 0.1 g to the nearest 0.001 g of the 5 g sample and place the soil in a Teflon digestion vessel.
2	Wet the soil with few drops of water and then add 5 ml of hydrofluoric acid* (HF) and 0.5 ml of perchloric acid ($HClO_4$) to the vessel.
3	Place the soil-acid mixture in a chemical hood and heat on hot plate until fumes of perchloric acid, $HClO_4$ appear or heat until the temperature is more than 200°C (392°F).
4	Cool the vessel to room temperature and then add 5 ml of hydrofluoric acid, HF.

5	Place the vessel on a hot plate and cover nine tenths of the vessel top using the ported closure.
6	Heat the vessel to 200-225° C (392-437° F) and evaporate the solution to dryness.
7	Again cool the vessel to room temperature and add 2 ml of water and a few drops of perchloric acid, $HClO_4$.
8	Replace the vessel on the hot plate and evaporate until dry.
9	Remove the vessel from the hot plate and bring the contents to room temperature and add 5 ml of 6N hydrochloric acid, HCl, and 5 ml of water to the vessel.
10	Place the vessel back on the hot plate and boil it slightly. If the sample doesn't dissolve repeat the steps 2 through 10.
11	Once the residue completely dissolves in HCl transfer the sample to 50 ml volumetric flask, and dilute the contents to 50 ml volume.
12	Determine the amount of potassium (K^+) in the solution by using the spectrophotometer.

- \- Caution is needed while handling this acid; This acid is required to dissolve the silicate minerals in the soils, which in turn will release potassium ions that are expected within the silicate minerals.

Calculations

Use the following formula to calculate the TP of the soil sample:

$$TP,\% = Conc.ofK^{+} * 0.05 \tag{C.4}$$

where,

TP = % of total potassium
Conc. of K^+ = Concentration of potassium ion from Step 12 above.

Determination of Clay Mineralogy

Section 1

Overview

Use this method to determine the concentration of Kaolinite, Illite and Montmorillonite minerals within a clay material. Direct measurements of the clay mineralogy utilizing X-Ray Diffraction (XRD) and Scanning Electron Microscope (SEM) are preferred. However, these tests are not practical for day-to-day use since they are expensive to perform and require advanced instrumentation. A set of simple indirect methods, such as cationic exchange capacity (CEC), specific surface area (SSA) and total potassium (TP) are proposed[4]. to estimate the dominating clay minerals in the subgrades for day-to-day use.

Units of Measurement

The values given in parentheses (if provided) are not standard and may not be exact mathematical conversions. Use each system of units separately. Combining values from the two systems may result in nonconformance with the standard.

Calculations

Step 1: Determine percent Illite (*%I*) by measuring the total potassium (TP) using:

$$\%I = \left[\frac{TP}{6}\right] \times 100 \tag{C.5}$$

If Illite is the dominant clay mineral, the mineralogy of the clay is not of a concern, and further mineralogical tests are not necessary (i.e. ignore Step 2).

Step 2: Determine percent Montmorillonite (*%M*) and percent Kaolinite (*%K*) by measuring the cationic exchange capacity (CEC) and specific surface area (SSA) using the following equations:

$$\%M = -2.87 + 0.08 \times SSA + 0.26 \times CEC \tag{C.6}$$

$$\%K = 100 - \%I - \%M \tag{C.7}$$

---

[4] Attached as companion specifications

Accelerated Moisture Conditioning of Specimens with Backpressure Method

Section 1

Overview

Use this method to moisture condition soil specimens in laboratory applying backpressure. The test can also be used to measure permeability of a material after the specimen under test is saturated.

To perform test, a soil specimen is prepared in laboratory using Tex-114-E or Static Compaction[5]. The specimen measures 6 in. (150 mm) or 8 in. (200 mm) in height and 4 in. (100 mm) in diameter.

Units of Measurement

The values given in parentheses (if provided) are not standard and may not be exact mathematical conversions. Use each system of units separately. Combining values from the two systems may result in nonconformance with the standard.

Section 2

Apparatus

The following apparatus are required:

- Permeameter, with the following requirements[6]:
  - A calibrated cylinder of 1.25 inches ± 0.02 inches (31.75 mm ± 0.5 mm) inner diameter graduated in millimeters and capable of dispensing 500 ml of water.
  - A confining chamber, which is a hollow plastic mold 12.0 in. (300 mm) in height and 4.25 in. (106 mm) inner diameter with a hose barb fitting in the middle of the mold. The confining chamber should be retrofitted with a flexible latex membrane 0.025 inches (0.635 mm) thick and 14 inches long.
  - An upper clamp assembly for supporting the graduated cylinder and expanding an o-ring against the confining chamber. The opening in the upper cap shall have the same diameter as the inner diameter of the calibrated cylinder mentioned previously.
  - A lower pedestal plate for supporting the soil specimen and expanding o-ring against the confining chamber. The lower pedestal should have a drainage port to allow water flushed through the specimen to be collected.
  - An outlet pipe with an open/close drainage valve connected to the bottom of the lower pedestal to collect water flushed through the specimen.

---

[5] Attached as a companion specification

[6] We have identified a manufacturer that can provide this set up for less than $1,200.

  - O-ring of sufficient diameter and thickness for maintaining a seal against the confining chamber.
- Sealing Agent (vacuum grease).
- A pipette graduated in millimeters and capable of collecting 500 ml of water.
- pH meter with temperature gauge (optional).
- Vacuum and pressure pump, including manometer.
- A pressure cap fitting the top of the graduated cylinder that is also connected to an external pressure pump. This pump provides the backpressure to the system.
- Stopwatch.
- Balance, with a minimum capacity of 15 lbs (7 kg), accurate and readable to 0.001 lb (0.5 g) or 0.1% of the test mass, whichever is greater.
- Two porous stones of 4 in. (100 mm) in diameter and 0.25 in. (6 mm) in thickness.
- Filter paper.

Section 3

Preparation of Soil Specimen

Preparation of Material

Prepare the material according to 'Part II, Preparing Samples for Compaction and Triaxial Tests' of Test Method "Tex-101-E, Preparing Soil and Flexible Base Materials for Testing."

Preparation of Specimen

Prepare specimen according to Test Methods “Tex-114-E, Laboratory Compaction Characteristics and Moisture-Density Relationship of Subgrade, Embankment Soils, and Backfill Material” or “Tex-xxx-E[7], Laboratory Compaction of Clay Specimens by Static Compaction Method.”

Section 4

Moisture Conditioning of Specimen

The following table lists the steps necessary to achieve specimen moisture conditioning. The complete test is divided into Specimen Preparation and Test Procedure. The setup is also capable of measuring water permeability of the prepared specimen.

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[7] Attached as a companion specification

Specimen Preparation for Testing	
Step	**Action**
1	Evacuate the air from the confining chamber/membrane cavity. Use a vacuum pump for this purpose. Complete evacuation can be achieved by slightly pinching and pulling away the membrane from the hose barb fitting while the pump is being actuated.
2	Place one of the porous stones on top of the lower plate and center it. Place a filter paper on top of the stone. Weigh the specimen and place it on top of the porous stone and center it. In the same way, place a filter paper on top of the specimen and the other porous stone above it.
3	Place the confining chamber over the specimen and the lower pedestal. Make sure that the chamber is oriented so that the hose barb fitting is located between the o-rings on the upper cap and the lower pedestal.
4	Insert the top cap assembly into the confining chamber and let it rest on top of the porous stone. The process is aided if the graduated cylinder is already inserted into the upper cap assembly, so it can be used as a handle.
5	Install the two clamp assemblies onto the permeameter frame and evenly tighten each of the locking screws, applying a moderate pressure to the upper cap assembly. This action seals the o-ring against the membrane and confining chamber.
6	Inflate the membrane using the hand pump up to a pressure of 10 ± 0.5 psi (70 ±3.5 kPa). Maintain the pressure throughout the tests using the pump if the system loses pressure. The specimen is ready to start the test.

Moisture Conditioning Test Procedure	
Step	**Action**
1	Fill the graduated cylinder to a level of 650 mm and start the timing device.
2	Secure the top of the graduated cylinder with the pressure cap and apply desired backpressure on top of the graduated cylinder. This pressure should be smaller than the confining pressure to prevent water from migrating through the side of the specimen and to prevent internal damage to the specimen. A backpressure of no more than 5 psi (35 kPa) is recommended.
3	Record the water height in the top graduated cylinder every 2 to 4 hours to evaluate the moisture conditioning progress. The difference in the water height before and after the initiation of the test indicates the amount of water absorbed by the specimen (see Equation 1 in Calculations section).
4	Continue the test until the water is discharged out of the outlet pipe. As soon as the amount of water drop in the graduated cylinder is roughly equal to the water discharged from outlet, the specimen is considered moisture conditioned.
5	Continue the test, if the permeability of the material needs to be calculated
6	After the test is completed, release the pressure cap from the top of the graduated cylinder and remove all remaining water from the cylinder. Release pressure from the confining chamber. Remove the cap assemblies, upper cap and specimen and measure the final weight of the specimen to measure total absorbed water.

Permeability Test and pH Measurement Procedures (optional)	
Step	**Action**
1	Follow Steps 1 through 5 in Moisture Conditioning procedure.
2	Measure the height of water in the top graduated cylinder (h_1) and record the time (t_1)
3	Measure the height of the water in the top graduated cylinder (h_2) after a certain elapsed time (t_2) after the first measurement.
4	Calculate the permeability of the specimen using Equation 2 in the Calculation Section
5	To estimate the discharge of the additive, measure the pH of the discharged water at regular intervals using test procedure Tex-128-E.
6	After the test is completed, release the pressure cap from the top of the graduated cylinder and remove all remaining water from the cylinder. Release pressure from the confining chamber Remove the cap assemblies, upper cap and specimen and measure the final weight of the specimen to measure total absorbed water.

Calculations

The amount of water absorbed by the specimen at any time is given by:

$$W_a = \frac{a*(h_i - h_t)\gamma_w}{1728} \tag{C.8}$$

where:

W_a = Water absorbed by the specimen at any time, lbs
a = Inside cross-sectional area of the graduated cylinder, in^2
h_i = Initial height of water in the graduated cylinder (65 cm or 25.6 in. recommended), in
h_t = Height of water in the graduated cylinder at time t, in.
γ_w = Unit weight of water, 62.4 pcf

The water absorbed can be transformed to moisture content in the specimen with the equation:

$$MC_t = MC_i + \left(\frac{W_a + W_{Ti}}{\left(\frac{W_{Ti}}{1 + MC_i} \right)} - 1 \right) \tag{C.9}$$

where:

MC_t = Moisture content of specimen at time t, %
MC_i = Initial moisture content of specimen, %
W_a = Water absorbed by the specimen, lbs
W_{Ti} = Initial total weight of specimen, lbs

In addition, to calculate the theoretical moisture content of the specimen when saturation is reached (MC_{sat}), the following equation needs to be applied:

$$MC_{sat} = \frac{\gamma_w (1 + MC_i)}{\gamma_{Ti}} - \frac{1}{G_S} \tag{C.10}$$

where:

MC_{sat} = Theoretical moisture content of specimen at saturation, %
γ_w = Unit weight of water, 62.4 pcf
G_S = Specific Gravity of soil
γ_{Ti} = Initial wet (total) unit weight of soil, pcf

The coefficient of permeability, k, is determined by the following equation:

$$k = \frac{aL}{A\Delta t} \ln\left(\frac{h_1 + \Delta h}{h_2 + \Delta h}\right) C * 2.54 \tag{C.11}$$

where:

k = Coefficient of permeability, cm/s
a = Inside cross-sectional area of the graduated cylinder, in.^{2}
L = Average height of the test specimen, in.
A = Average cross-sectional area of the specimen, in.^{2}
h_1 = Height of water in the graduated cylinder measured from bottom of specimen at time t_1, in.
h_2 = Height of water in the graduated cylinder measured from bottom of specimen at time t_2, in.
Δh = Equivalent height due to backpressure, in.
t_1 = Time when h_1 was measured
t_2 = Time when h_2 was measured
Δt = $t_2 - t_1$, s
C = Temperature correction for viscosity of water; see Table 1. A temperature of 68ºF (20ºC) is used as the standard
ln = Natural Logarithm

In addition, Δh (in inches) can be obtained from:

$$\Delta h = \frac{BP}{\gamma_w} * 1728 \tag{C.12}$$

where:

BP = Backpressure applied to the system (in psi)
γ_w = Unit weight of water, 62.4 pcf

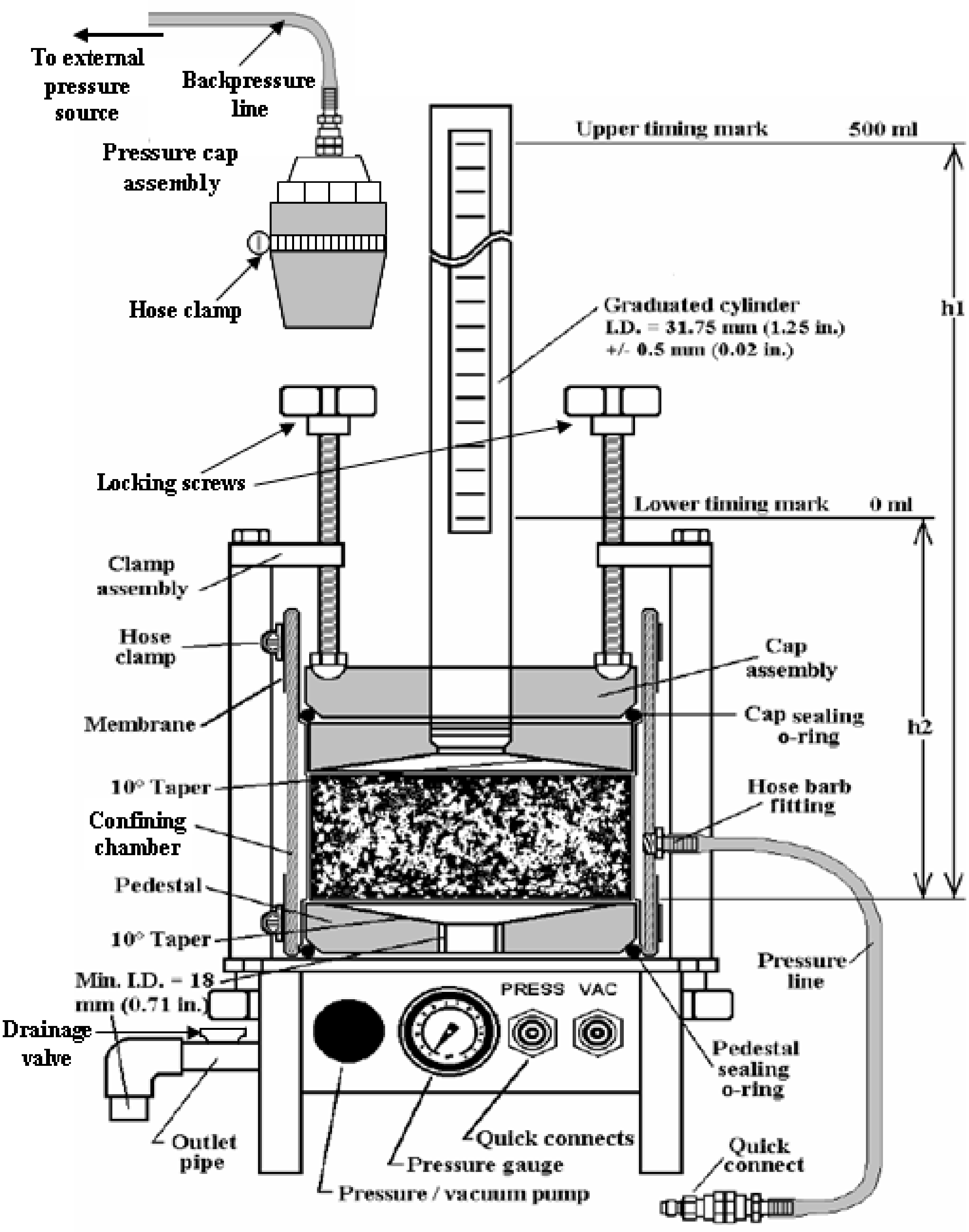

Figure C.3 – Permeability Testing Apparatus

Table C.1 – Temperature Correction for Viscosity of Water, ºF

ºF	0.0	0.2	0.4	0.6	0.8
50	1.30	1.30	1.29	1.29	1.28
51	1.28	1.28	1.27	1.27	1.26
52	1.26	1.26	1.25	1.25	1.24
53	1.24	1.24	1.23	1.23	1.23
54	1.22	1.22	1.21	1.21	1.21
55	1.20	1.20	1.20	1.19	1.19
56	1.19	1.18	1.18	1.17	1.17
57	1.17	1.16	1.16	1.16	1.15
58	1.15	1.15	1.14	1.14	1.14
59	1.13	1.13	1.13	1.12	1.12
60	1.12	1.11	1.11	1.11	1.10
61	1.10	1.10	1.10	1.09	1.09
62	1.09	1.08	1.08	1.08	1.07
63	1.07	1.07	1.06	1.06	1.06
64	1.06	1.05	1.05	1.05	1.04
65	1.04	1.04	1.04	1.03	1.03
66	1.03	1.02	1.02	1.02	1.02
67	1.01	1.01	1.01	1.01	1.00
68	1.00	1.00	0.99	0.99	0.99
69	0.99	0.98	0.98	0.98	0.98
70	0.97	0.97	0.97	0.97	0.96
71	0.96	0.96	0.96	0.95	0.95
72	0.95	0.95	0.94	0.94	0.94
73	0.94	0.93	0.93	0.93	0.93
74	0.92	0.92	0.92	0.92	0.92
75	0.91	0.91	0.91	0.91	0.90
76	0.90	0.90	0.90	0.89	0.89
77	0.89	0.89	0.89	0.88	0.88
78	0.88	0.88	0.88	0.87	0.87
79	0.87	0.87	0.86	0.86	0.86
80	0.86	0.86	0.85	0.85	0.85
81	0.85	0.85	0.84	0.84	0.84
82	0.84	0.84	0.83	0.83	0.83
83	0.83	0.83	0.82	0.82	0.82
84	0.82	0.82	0.81	0.81	0.81
85	0.81	0.81	0.81	0.80	0.80
86	0.80	0.80	0.80	0.79	0.79
87	0.79	0.79	0.79	0.79	0.78
88	0.78	0.78	0.78	0.78	0.77
89	0.77	0.77	0.77	0.77	0.77
90	0.76	0.76	0.76	0.76	0.76
91	0.76	0.75	0.75	0.75	0.75
92	0.75	0.75	0.74	0.74	0.74
93	0.74	0.74	0.74	0.73	0.73
94	0.73	0.73	0.73	0.73	0.72
95	0.72	0.72	0.72	0.72	0.72

Accelerated Moisture Conditioning of Specimens with Vacuum Method

Section 1

Overview

Use this method to moisture condition soil specimens in laboratory by applying vacuum.

To perform test, a soil specimen is prepared in laboratory using Tex-114-E or Static Compaction as per Tex xxx-E[8]. The specimen measures 6 in. (150 mm) or 8 in. (200 mm) in height and 4 in. (100 mm) in diameter.

Units of Measurement

The values given in parentheses (if provided) are not standard and may not be exact mathematical conversions. Use each system of units separately. Combining values from the two systems may result in nonconformance with the standard.

Section 2

Apparatus

The following apparatus are required:

- Permeameter, with the following requirements[9]:
  - A calibrated cylinder of 1.25 inches ± 0.02 inches (31.75 mm ± 0.5 mm) inner diameter graduated in millimeters and capable of dispensing 500 ml of water.
  - A confining chamber, which is a hollow plastic mold 12.0 in. (300 mm) in height and 4.25 in. (106 mm) inner diameter with a hose barb fitting in the middle of the mold. The confining chamber should be retrofitted with a flexible latex membrane 0.025 inches (0.635 mm) thick and 14 inches high.
  - An upper clamp assembly for supporting the graduated cylinder and expanding an o-ring against the confining chamber. The opening in the upper cap shall have the same diameter as the inner diameter of the calibrated cylinder mentioned previously.
  - A lower pedestal plate for supporting the soil specimen and expanding o-ring against the confining chamber. The lower pedestal should have a drainage port to allow water flushed through the specimen to be collected.
  - An outlet pipe with an open/close drainage valve connected to the bottom of the lower pedestal. The pipe is also connected on the other side to a water container through a flexible plastic tube.

---

[8] Attached as a companion specification

[9] We have identified a manufacturer that can provide this set up for less than $1,200.

  - O-ring of sufficient diameter and thickness for maintaining a seal against the confining chamber.
- Sealing Agent (vacuum grease).
- A water container capable of storing at least 500 ml of water.
- pH meter with temperature gauge (optional).
- Vacuum and pressure pump, including manometer.
- A vacuum pump or supply.
- A pressure cap fitting the top of the graduated cylinder that is also connected to an external vacuum pump. This pump provides the vacuum suction to the system.
- Stopwatch.
- Balance, with a minimum capacity of 15 lbs (7 kg), accurate and readable to 0.001 lb (0.5 g) or 0.1% of the test mass, whichever is greater.
- Two porous stones of 4 in. (100 mm) in diameter and 0.25 in. (6 mm) in thickness.
- Filter paper.

Section 3

Preparation of Soil Specimen

Preparation of Material

Prepare the material according to 'Part II, Preparing Samples for Compaction and Triaxial Tests' of Test Method "Tex-101-E, Preparing Soil and Flexible Base Materials for Testing."

Preparation of Specimen

Prepare specimen according to Test Methods “Tex-114-E, Laboratory Compaction Characteristics and Moisture-Density Relationship of Subgrade, Embankment Soils, and Backfill Material” or “Tex-xxx-E[10], Laboratory Compaction of Clay Specimens by Static Compaction Method.”

Section 4

Moisture Conditioning of Specimen

The following table lists the steps necessary to achieve specimen moisture conditioning. The complete test is divided into Specimen Preparation and Test Procedure.

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[10] Attached as a companion specification

Specimen Preparation for Testing	
Step	**Action**
1	Evacuate the air from the confining chamber/membrane cavity. Use a vacuum pump for this purpose. Complete evacuation can be achieved by slightly pinching and pulling away the membrane from the hose barb fitting while the pump is being actuated.
2	Place one of the porous stones on top of the lower plate and center it. Place a filter paper on top of the stone. Weigh the specimen and place it on top of the porous stone and center it. In the same way, place a filter paper on top of the specimen and the other porous stone above it.
3	Place the confining chamber over the specimen and the lower pedestal. Make sure that the chamber is oriented so that the hose barb fitting is located between the o-rings on the upper cap and the lower pedestal.
4	Insert the top cap assembly into the confining chamber and let it rest on top of the porous stone. The process is aided if the graduated cylinder is already inserted into the upper cap assembly, so it can be used as a handle.
5	Install the two clamp assemblies onto the permeameter frame and evenly tighten each of the locking screws, applying a moderate pressure to the upper cap assembly. This action seals the o-ring against the membrane and confining chamber.
6	Inflate the membrane using the hand pump up to a pressure of 10 ± 0.5 psi (70 ± 3.5 kPa). Maintain the pressure throughout the tests using the pump if the system loses pressure. The specimen is ready to start the test.

Moisture Conditioning Test Procedure	
Step	**Action**
1	Before the test is started, the top water level in the water container should be at the same height as the bottom of the specimen tested. This avoids adding extra pressure to the moisture conditioning. In addition, the drainage valve of the outlet pipe needs to be closed and the plastic tube, outlet pipe and opening in the pedestal plate should be filled with water before the test is started.
2	Secure the top of the graduated cylinder with the pressure cap and apply vacuum suction on top of the graduated cylinder. The vacuum is applied by the external pump.
3	Start the test opening the drainage valve of the outlet pipe.
4	Continue the test until the water reaches the top of the specimen. At this point water should become visible at the bottom of the graduated cylinder and the specimen is considered moisture conditioned.
5	After the test is completed, stop the vacuum pump, release the pressure cap from the top of the graduated cylinder, close the drainage valve of the outlet pipe and remove any remaining water from the cylinder. Release pressure from the container. Remove the cap assemblies, upper cap and specimen and measure final weight of the specimen to measure total absorbed water.

pH Measurement Procedures (optional)	
Step	**Action**
1	Follow Steps 1 through 4 in Moisture Conditioning procedure.
2	Continue the test until a considerable amount of water is collected in the graduated cylinder. 150 ml of water are considered enough and this amount corresponds to a height in the graduated cylinder close to the lower timing mark or 0 mm mark.
3	After the test is completed, stop the vacuum pump, release the pressure cap from the top of the graduated cylinder, close the drainage valve of the outlet pipe and collect all water from the cylinder in a separate container to measure pH.
4	To estimate the discharge of the additive, measure the pH of the discharged water using test procedure Tex-128-E.
5	Release pressure from the container. Remove the cap assemblies, upper cap and specimen and measure final weight of the specimen to measure total absorbed water.

Calculations

The water absorbed by the specimen after the test is completed can be transformed to moisture content with the equation:

$$MC_f = MC_i + \left(\frac{W_{Tf}}{\left(\dfrac{W_{Ti}}{1 + MC_i} \right)} - 1 \right) \tag{C.13}$$

where:

MC_f = Final moisture content of specimen, %
W_{Tf} = Final total weight of specimen, lbs
MC_i = Initial moisture content of specimen, %
W_{Ti} = Initial total weight of specimen, lbs

In addition, to calculate the theoretical moisture content of the specimen when saturation is reached (MC_{sat}), the following equation needs to be applied:

$$MC_{sat} = \frac{\gamma_w (1 + MC_i)}{\gamma_{Ti}} - \frac{1}{G_S} \tag{C.14}$$

where:

MC_{sat} = Theoretical moisture content of specimen at saturation, %
γ_w = Unit weight of water, 62.4 pcf
G_S = Specific Gravity of soil
γ_{Ti} = Initial wet (total) unit weight of soil, pcf

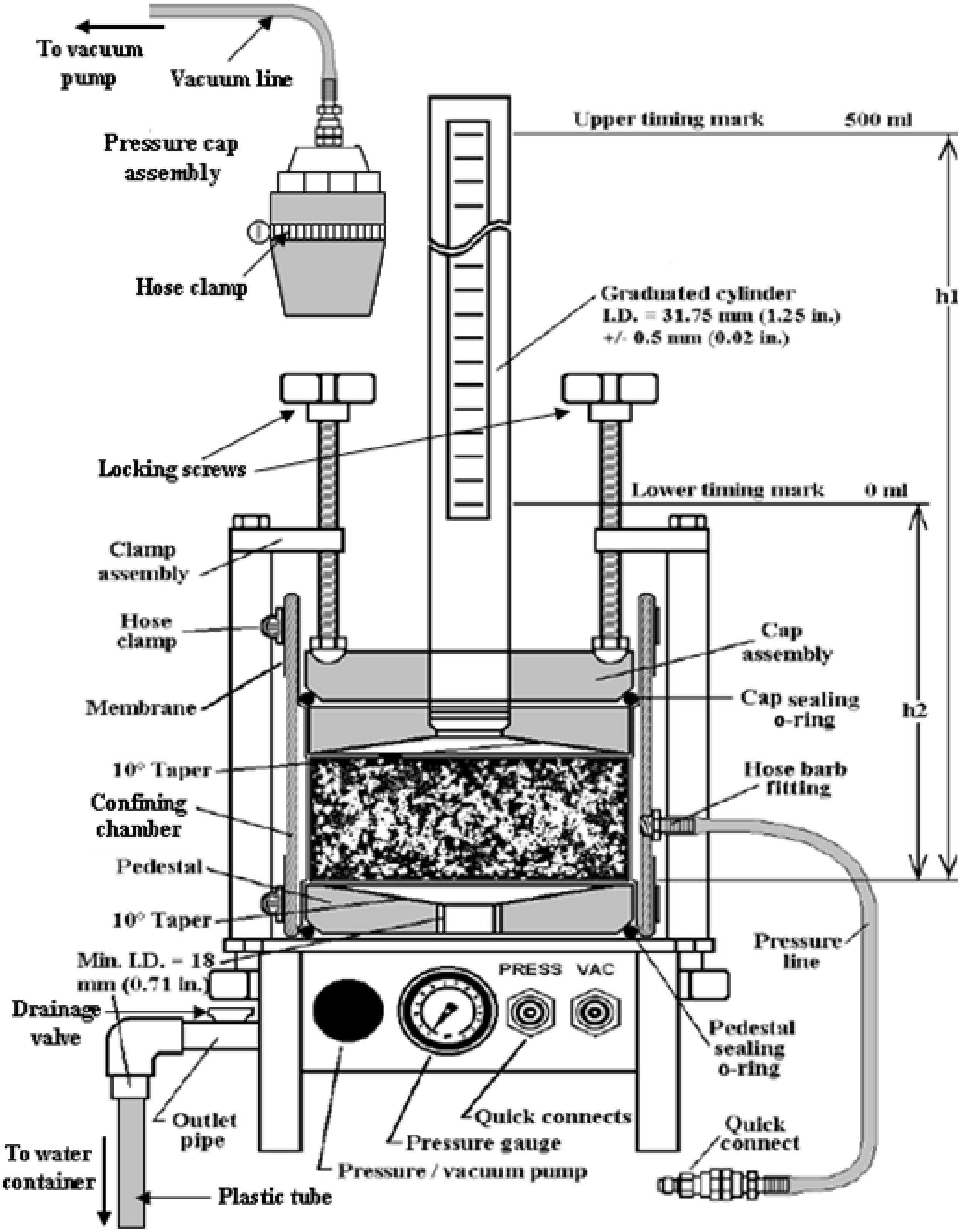

Figure C.4 – Vacuum Testing Apparatus

Laboratory Compaction of Clay Specimens by Static Method

Section 1

Overview

Use this method to compact clay specimens with and without calcium-based additives in the laboratory.

To perform test, a clay specimen is prepared in the laboratory using a static compactor in one single lift. The specimen measures 6 in. (150 mm) in height and 4 in. (100 mm) in diameter.

Units of Measurement

The values given in parentheses (if provided) are not standard and may not be exact mathematical conversions. Use each system of units separately. Combining values from the two systems may result in nonconformance with the standard.

Section 2

Apparatus

The following apparatus are required:

- A hollow metallic mold 13.0 in. (330 mm) in height and 4.0 in. (100 mm) inner diameter
- Three solid metallic blocks 3.9 in. (97 mm) in diameter and 4.05 in. (103 mm), 2.45 in. (62 mm) and 0.50 in. (13 mm) high, respectively
- Static compactor (a loading frame for Unconfined compressive strength tests can be used)
- Balance, with a minimum capacity of 15 lbs (7 kg), accurate and readable to 0.001 lb (0.5 g) or 0.1% of the test mass, whichever is greater
- Hydraulic press and a metal ring to extrude molded specimens
- Drying oven, maintained at 230 ± 9°F (110 ± 5°C)
- Metal pans, wide and shallow for mixing and drying materials
- No. 4 (4.75 mm) sieve
- Lubricant Agent (WD40 or similar)

Section 3

Laboratory Compaction of Clay Specimen

This part uses a static compactor to prepare a 4 in. (100 mm) by 6 in. (150 mm) clay specimen. The clay passing through No. 4 sieve is used to prepare clay specimens.

Preparation of Material

Follow the steps below to prepare material for compaction:

Material Preparation for Compaction

Step	Action
1	Crush clay material fine enough so that it passes through No. 4 sieve. Crush enough material to prepare several specimens (Approximately 10 lbs, 4.5 kg).
2	Sieve crushed material through No. 4 sieve. Use the material passing through No. 4 sieve and discard the remaining material.
3	Dry sieved material in an oven at 230 ± 9^{o}F (110 ± 5^{o}C) for no less than 24 hours.
4	Cool the material and measure approximately 6 lbs (3 kg) of material per specimen in a container.
5	Calculate the mass of the water to be added based on the air-dry mass of the material. (e.g. if you wish to prepare specimen at 20% moisture content then add 6*0.20 = 1.20 lbs of water).
6	Weigh out this amount of water into a tared sprinkling jar.
7	Sprinkle water onto the soil during mixing, in increments.
8	Thoroughly mix each specimen to ensure even distribution of water throughout specimen.
9	Cover the mixed sample and allow sample to stand and cure for at least 12 hr before compacting. When the PI is less than 12, the curing time may be reduced to not less than 3 hr.

Preparation of Specimen

Follow the steps below to prepare specimen for testing:

Specimen Preparation for Testing

Step	Action
1	Measure the amount of material required based on its dry density, degree of compaction and the moisture content at which the specimen is to be prepared using Equation1 in Calculations. (e.g. if dry density = 112 pcf, degree of compaction = 95% and the moisture content = 20%, then the amount of material required to prepare one specimen = $112 * 0.95 * \frac{\pi}{4} * 4^2 * \frac{6}{12^3} * (1+0.20) = 5.571 lb$
2	Clean the inside of the mold with the lubricant agent to ease extraction of the specimen after compaction is completed.
3	Place the 2.45 in. (62 mm) thick metal block at the bottom of the mold and pour the weighed material inside the mold.
4	Place the 4.05 in. (103 mm) thick solid block on top of the poured material and place this assembly in static compactor.

5	Compact the specimen until the top block becomes flush with the mold (the compaction speed should be approximately 2 in./min).
6	Maintain the pressure for one minute and then release the pressure from the mold.
7	Flip the mold so that 2.45 in. (62 mm) thick block is facing up.
8	Place the 0.5 in. (13 mm) thick solid block on top of 2.45 in. (62 mm) solid block.
9	Start the static compactor again and apply pressure until the 0.5 in. (13 mm) thick solid block becomes flush with the mold.
10	Maintain the pressure for one minute and then release the pressure from the mold.
11	Take out the mold and remove the metal blocks from it.
12	Center the mold on top of a hydraulic jack to extract the specimen from the mold.
13	Cover the specimen with a rubber membrane.

Note:

- Just before preparing the specimen the mix should be weighed. The mix should weigh the same as it was initially prepared, if not add additional water in it to make up for the moisture loss.
- Average the moisture contents just before and after preparing the specimen to make sure the exact moisture content of the specimen prepared.

Calculations

Use the following formula to calculate the weight of the wet material required for preparing one specimen:

$$W = DD \times C \times V \times (1 + MC) \tag{C.15}$$

where,

W = Weight of the wet material required for preparing one specimen in lbs
DD = Dry density of the material in pcf
C = Degree of compaction
V = Volume of the specimen in ft^3
MC = Required moisture content for the specimen

APPENDIX D

DETAILED RESULTS OF DSPA MEASUREMENTS

Table D.1 – Field Seismic Modulus obtained with DSPA for Austin Site

Station	Location and Point Average (ksi). Inside Lane			Station Average (ksi)
	LWP	Center	RWP	
1043+00	74	85	69	76
1043+50	80	80	69	76
1044+00	72	87	66	75
1044+50	103	93	90	95
1045+00	81	93	91	88
1045+50	100	96	90	95
1046+00	81	97	69	82
1046+50	89	110	120	106
1047+00	125	87	93	102
1047+50	88	98	73	86
1048+00	76	103	74	84
1048+50	77	81	59	72
1049+00	91	81	62	78
1049+50	90	95	79	88
1050+00	84	85	88	86
1050+50	80	73	85	79
1051+00	57	82	76	71
Average (ksi)	**85**	**89**	**79**	**85**
C.V. (%)	**17.6**	**10.6**	**18.9**	**16.2**

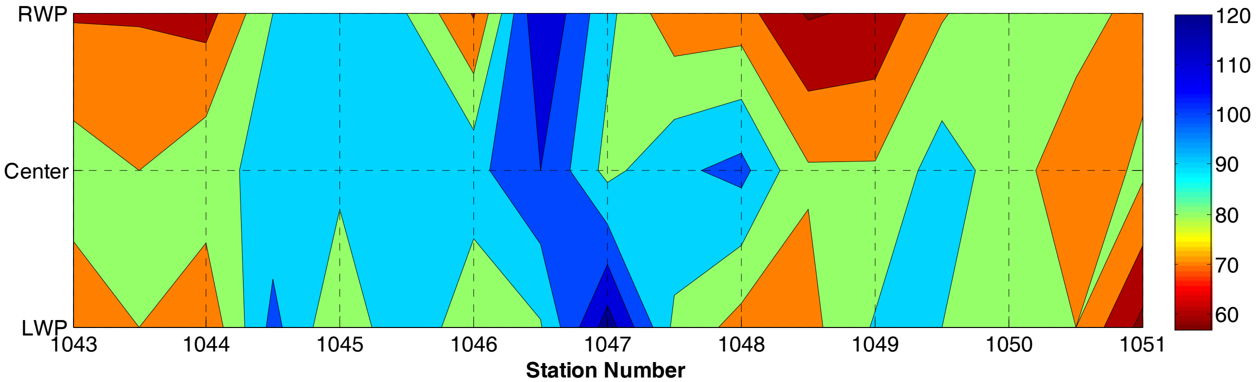

Figure D.1 – Contour Map of Field Seismic Modulus of Austin Site

Table D.2 – Results of NDG Tests on the Stabilized Subgrade of the Austin Site

Date	Station	Moisture Content (%)	Dry Density (pcf)	Wet Density (pcf)
1/14/08	1041+00	14.7	97.8	112.2
	1046+00	20.6	90.3	108.9
	1049+59	22.6	91.4	112.1
	1053+00	15.0	96.2	110.6
Average		**18.2**	**93.9**	**111.0**
C. V. (%)		**21.9**	**3.9**	**1.4**
1/16/08	1049+59	25.2	98.5	123.3
	1050+83	25.9	94.1	118.5
Average		**25.6**	**96.3**	**120.9**
C. V. (%)		**1.9**	**3.2**	**2.8**

Table D.3 – Field Seismic Modulus obtained with DSPA for Keller Site

Station	Location and Point Average (ksi). Inside Lane			Station Average (ksi)
	LWP	Center	RWP	
56	80	100	113	98
54	50	54	48	51
52	43	42	33	39
50	31	62	32	41
48	44	55	75	58
46	62	89	52	68
44	50	43	59	51
42	52	73	36	53
40	74	67	65	68
38	80	33	43	52
36	72	83	75	77
Average (ksi)	**58**	**64**	**57**	**60**
C.V. (%)	**28.7**	**32.9**	**41.9**	**34.0**

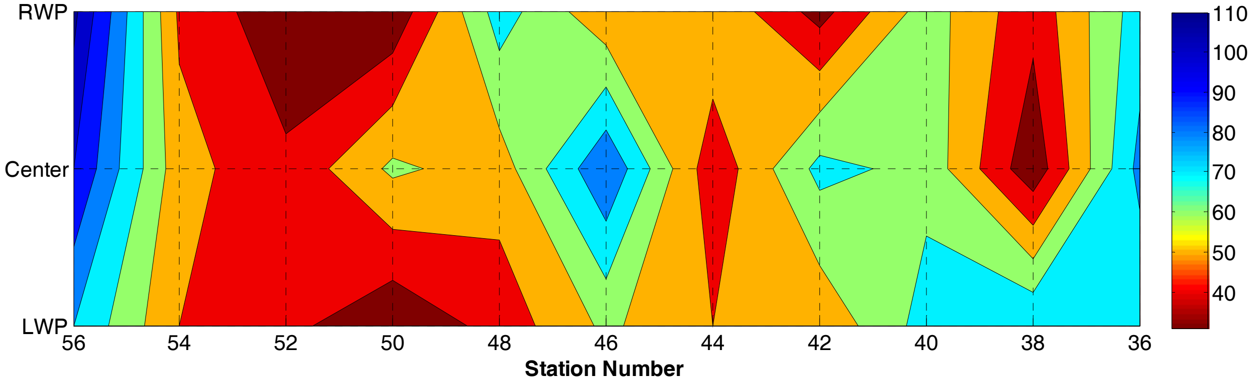

Figure D.2 – Contour Map of Field Seismic Modulus of Keller

Table D.4 – Results of NDG Tests on the Stabilized Subgrade of the Keller Site

Station	Moisture Content (%)	Dry Density (pcf)	Wet Density (pcf)
56+00	17.6	112.5	132.3
54+00	17.9	107.0	126.2
48+00	25.1	91.7	114.7
44+00	26.0	92.5	116.6
40+00	22.6	96.6	118.4
37+00	27.2	92.0	117.0
Average	**22.7**	**98.7**	**120.9**
C. V. (%)	**18.2%**	**9.0%**	**5.7%**

Table D.5 – Field Seismic Modulus obtained with DSPA for Wichita Falls Site after 24 Hours of Compaction

Station	Location and Point Average (ksi). Inside Lane			Station Average (ksi)
	LWP	Center	RWP	
871+00	338	137	190	222
870+50	400	198	235	278
870+00	407	193	160	253
869+50	315	245	118	226
869+00	170	350	145	222
868+50	273	223	278	258
868+00	245	263	420	309
867+50	203	230	70	168
867+00	200	290	115	202
866+50	295	268	240	268
866+00	235	350	290	292
865+50	290	283	420	331
865+00	173	310	310	264
864+50	178	333	178	230
864+00	143	173	238	184
863+50	138	295	215	216
863+00	112	365	260	246
862+50	138	228	225	197
862+00	172	212	228	204
861+50	300	265	230	265
861+00	195	300	160	218
860+50	200	273	212	228
860+00	340	124	109	191
Average (ksi)	**237**	**257**	**219**	**238**
C.V. (%)	**35.9**	**25.6**	**40.2**	**33.9**

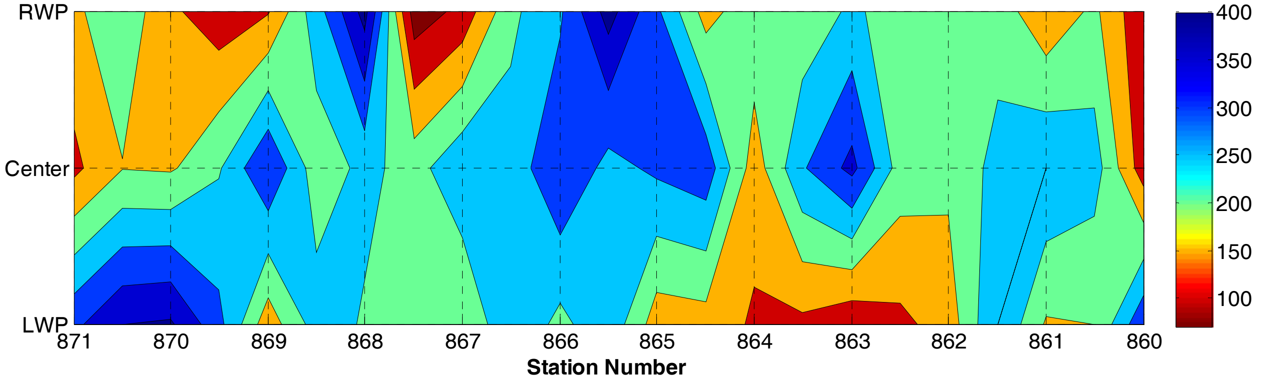

Figure D.3 – Contour Maps of Field Seismic Modulus of Wichita Falls after 24 Hours of Compaction

Table D.6 – Field Seismic Modulus obtained with DSPA for Wichita Falls Site after 3 Days of Compaction

Station	Location and Point Average (ksi). Inside Lane			Station Average (ksi)
	LWP	Center	RWP	
871+00	172	163	110	148
870+50	224	167	173	188
870+00	347	179	195	240
869+50	260	158	236	218
869+00	108	130	103	114
868+50	140	126	170	146
868+00	168	191	260	206
867+50	145	147	63	118
867+00	118	212	196	175
866+50	162	206	195	188
866+00	207	170	266	214
865+50	215	207	232	218
865+00	103	136	228	156
864+50	111	160	109	127
864+00	148	140	163	151
863+50	103	180	145	143
863+00	106	119	267	164
862+50	116	170	120	135
862+00	123	115	120	119
861+50	172	222	148	180
861+00	155	183	103	147
860+50	227	153	180	187
860+00	160	137	96	131
Average (ksi)	**165**	**164**	**169**	**166**
C.V. (%)	**36.3**	**18.6**	**35.7**	**31.0**

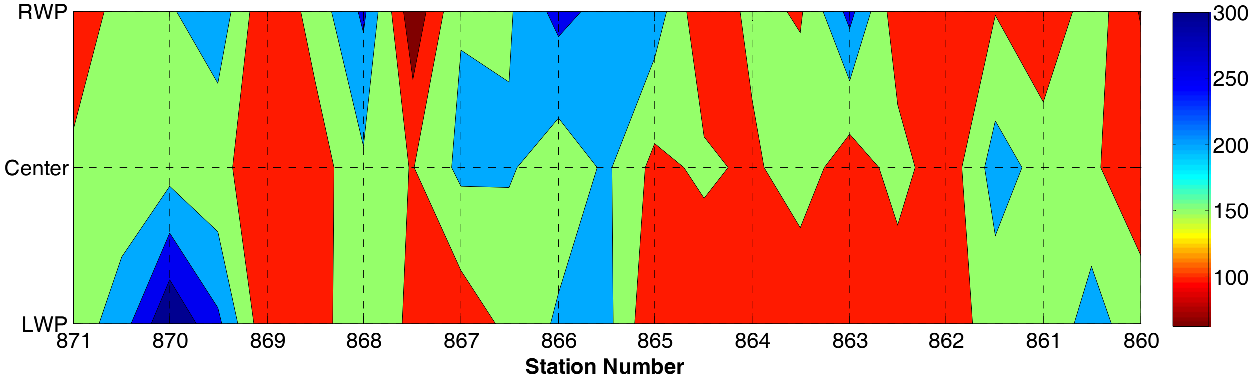

Figure D.4 – Contour Maps of Field Seismic Modulus of Wichita Falls after 3 Days of Compaction

Table D.7 – Field Seismic Modulus for Pharr Site after 24 Hours of Compaction

Station	Location and Point Average (ksi). Inside Lane			Station Average (ksi)
	LWP	Center	RWP	
C-0-60	35	30	41	35
	24	29	50	34
C-0-65	44	40	44	42
	37	30	50	39
C-0-75	33	41	41	38
	36	36	49	40
C-1-20	33	24	28	28
Average (ksi)	**34**	**33**	**43**	**37**
C.V. (%)	**17.6**	**19.2**	**17.9**	**12.9**

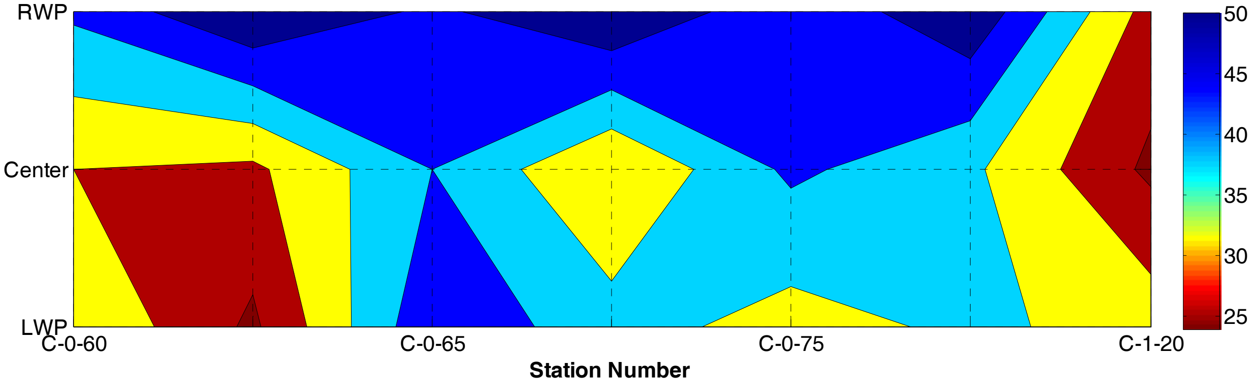

Figure D.5 – Contour Maps of Field Seismic Modulus of Medium PI Pharr Sites after 24 Hours of Compaction

Table D.8 – Field Seismic Modulus for Pharr Site after 5 Days of Compaction

Station	Location and Point Average (ksi). Inside Lane			Station Average (ksi)
	LWP	Center	RWP	
C 2+00	95	65	45	68
C 1+50	83	54	79	72
C 1+00	73	70	84	75
C 0+95	54	73	63	63
C 0+90	83	76	66	75
C 0+85	45	66	60	57
C 0+80	51	83	76	70
C 0+75	83	68	81	77
C 0+70	63	80	66	70
C 0+65	71	66	55	64
C 0+60	87	91	66	81
Average (ksi)	**71**	**72**	**67**	**70**
C.V. (%)	**22.8**	**14.0**	**17.5**	**9.9**

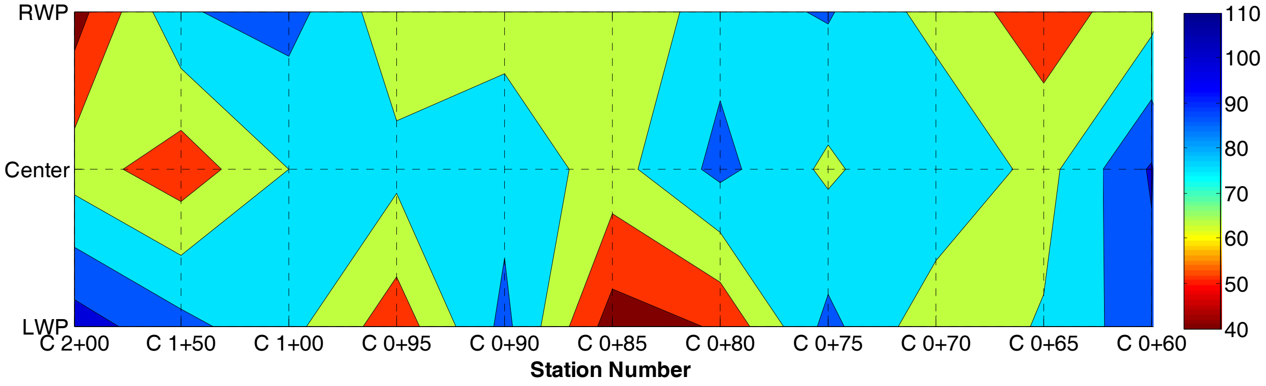

Figure D.6 – Contour Maps of Field Seismic Modulus of Medium PI Pharr Sites after 5 Days of Compaction

Table D.9 – Field Seismic Modulus obtained with DSPA for Pharr Site after 10 Days of Compaction

Station	Location and Point Average (ksi). Inside Lane			Station Average (ksi)
	LWP	Center	RWP	
C-0-30	88	52	73	71
	83	110	94	96
C-0-40	103	60	74	79
	108	65	71	81
C-0-50	110	110	83	101
	91	86	78	85
C-0-60	52	64	45	54
	48	55	98	67
C-0-70	43	80	46	56
	62	64	87	71
C-0-80	63	64	73	66
	73	60	49	61
C-0-90	57	43	46	48
	36	54	53	47
C-0-100	88	61	70	73
Average (ksi)	**73**	**68**	**69**	**70**
C.V. (%)	**33.1**	**28.9**	**25.7**	**22.8**

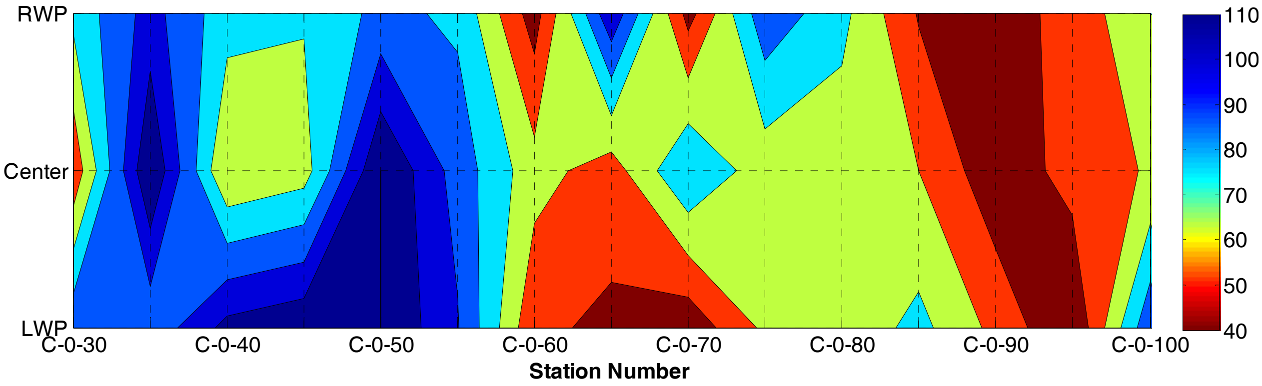

Figure D.7 – Contour Maps of Field Seismic Modulus of Medium PI Pharr Sites after 10 Days of Compaction

Table D.10 – NDG Tests on the Stabilized Subgrade of the Medium PI Pharr Sites

Section	Station	Moisture Content (%)	Dry Density (pcf)	Wet Density (pcf)
1 Day Old	C-0-60	31.3	83.5	109.6
	C-0-65	27.3	85.2	108.5
	C-0-75	30.8	84.9	111.0
	C-1-120	25.5	86.7	108.8
	Average	**28.7**	**85.1**	**109.5**
	C. V. (%)	**9.7%**	**1.5%**	**1.1%**
5 Days Old	C-0-60	18.5	86.4	102.4
	C-0-70	17.4	99.7	117.0
	C-0-80	20.6	96.7	116.6
	C-0-90	18.2	101.6	120.1
	C-1-50	17.1	104.4	122.3
	C-2-00	17.6	103.2	121.4
	Average	**18.2**	**98.7**	**116.6**
	C. V. (%)	**7.5%**	**7.0%**	**6.7%**
10 Days Old	C-0-30	16.2	94.4	109.7
	C-0-40	18.6	94.4	112.0
	C-0-50	17.6	99.3	116.8
	C-0-60	21.3	95.5	115.8
	C-0-70	19.6	97.3	116.4
	C-0-80	17.4	99.3	116.6
	C-0-90	18.6	95.0	112.7
	Average	**18.5**	**96.5**	**114.3**
	C. V. (%)	**8.9%**	**2.3%**	**2.5%**