CLAY MINERALOGY EFFECTS ON LONG-TERM PERFORMANCE OF CHEMICALLY TREATED EXPANSIVE CLAYS

by

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ABSTRACT

CLAY MINERALOGY EFFECTS ON LONG-TERM PERFORMANCE OF CHEMICALLY TREATED EXPANSIVE CLAYS

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Stabilization of expansive soils using chemical additives such as cement and lime has been practiced for several decades and these treatments provide stable treated subgrade foundation for supporting pavements, thereby offering riding comforts to travelers. However, many state Department of Transportation (DOT) agencies in the United States have had subgrade failures even after stabilization with chemical additives due to a loss of stabilizer over a time period, or a stabilizer being ineffective in some soils while other soils with the same index properties respond well to that stabilizer. These problems are attributed to the limitations of the current stabilizer design procedures. One such limitation is the lack of understanding of the complex interactions between the clay mineralogy of the soil and the additives used for soil stabilization. The current plasticity index based design is misleading as soils with different clay mineralogy may still exhibit same plasticity properties. Also, the design procedure is time consuming and hence the specifications are bypassed and the design is conducted based on the local experience. Hence, in this research study, an attempt is made to address some of these limitations in the soil stabilization area. Incorporation of the clay mineralogy aspects of the soils into the stabilization design process was first addressed, followed by durability studies to

address the long-term effectiveness of the stabilization and leachate studies. All these are the main focus of this dissertation investigation.

The first task was to develop a simple procedure to identify the dominating clay mineral in a given soil as the current procedures of mineral quantification are expensive and highly skill oriented. Hence, properties such as Cation Exchange Capacity, Specific Surface Area and Total Potassium were used and statistical regression equations were developed to predict the dominating clay mineral in a given soil. A total of twenty natural soils from different regions of the state of Texas and six (6) artificially mixed soils with known mineral percentages were obtained. Prediction models were developed using the test results' database. Tools such as regression analysis and artificial neural networks (ANN) were utilized to develop different prediction models. These clay mineralogy prediction models were validated using the artificial soil data. Predictions by both methods were compared and it was observed that the regression based prediction model showed better prediction capabilities than ANN based model. However, the differences between predictions are small and practically negligible. Hence any of these models could provide realistic prediction of clay mineralogy in a given soil.

The second task was to assess the effects of clay minerals on the long-term durability of stabilized expansive clays by conducting wetting/drying (W/D) studies replicating moisture fluctuations expected during summer and winter seasons in the field. A total of eight soils were selected for studying the long-term performance of stabilized expansive soils by conducting wetting/drying studies. Stabilizer design was carried out as per the TxDOT methods Tex 120-E (Lime as additive) and Tex 121-E (Cement as additive) and the results are presented. An accelerated curing method was developed and followed in this study for curing and moisture conditioning of the treated soil specimens. The effect of curing methods is studied on four select soils and it is observed that both the curing methods including old and longer curing (Tex 121-E) for 17 days and the present accelerated curing methods of 3 days yielded similar UCS and volume change test results. It has been interpreted from these results that there was no

considerable effect of curing on the long-term performance of these treated soils except for an initial strength since soil was partially saturated.

Soils containing Montmorillonite as a dominant mineral are more susceptible to premature failures after chemical stabilization when they are exposed to volume changes caused by swell and shrink related volume changes. Also, it is understood that low amount of additive dosage can cause premature failures in the pavement structure.

The third task was to assess the performance of the stabilization under severe rainfall conditions where heavy amounts of rain water infiltrates into the soil and causes leaching of the stabilizer and thereby reducing the life time of the stabilization. To understand this behavior leachate studies were conducted on all the eight soils selected to replicate moisture ingress and digress in the field during rainfalls and to study the effect of these moisture infiltrations on the long-term performance of stabilized soils. Leachate samples were collected after 3, 5, 7, and 14 cycles of leaching to address the chemical changes occurring due to leaching of the additive from the soil specimen. Also, unconfined compressive strength tests were conducted on soil specimens after 3, 7 and 14 cycles of leaching to address the strength changes from leaching.

Finally, an attempt was made to highlight the effect of the loss of strength in the treated soils due to the above mentioned climatic changes on the performance of a flexible pavement. Four different flexible pavement sections with varying asphalt concrete layer and base course layer thicknesses were altogether analyzed. The effect of treated base modulus deterioration on the pavement performance was assessed by obtaining the compressive strains on the subgrade top. These strains were used with the Asphalt Concrete Institute (2006) formulation to predict the ESALs required to cause rutting failure.

It is expected that the present research findings will be helpful in the future modifications of current stabilizer design practices by implementing clay mineral identification methods in the initial screening of soils and also selecting the dosages based on both durability and clay mineral information.

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CHAPTER 1

INTRODUCTION

1.1 Introduction

Soil stabilization has been a topic of interest and discussion for many years now, due to potential reduction in the construction and maintenance costs when the pavement infrastructure is built on problem grounds. Moreover, soil treatment enhances riding comforts to travellers. Extensive research was documented with regard to the engineering properties, reliability and durability of various types of stabilized materials (Tayabji, 1982; Haussman, 1989; Moseley and Kirsch, 2004; Puppala et al., 2006). However, many state Departments of Transportation (DOTs) in the United States have had problems with subgrade failure due to a loss of stabilizer over time, or a stabilizer being ineffective in some soils while other soils with the same index properties respond well to that stabilizer (Little et al., 2000). In some cases, the amount of stabilizer was not sufficient to produce a good subgrade foundation system for supporting the pavement structure. All these problems are attributed to the limitations of the current procedures (refer to Figure 1.1) for selecting the optimal additive content to stabilize pavement subsoil layers.

One such limitation is the lack of understanding of the complex interactions between the clay mineralogy of the soil and the additives used for soil stabilization. Also, the design procedure is time consuming and hence, the specifications are bypassed and design is conducted based on local experience. However, several premature failures of pavements on stabilized sub-soils and poor long-term performance of these stabilized soils have prompted a research to incorporate both clay mineralogy aspects into the pavement design practices and then address durability issues in the original design.

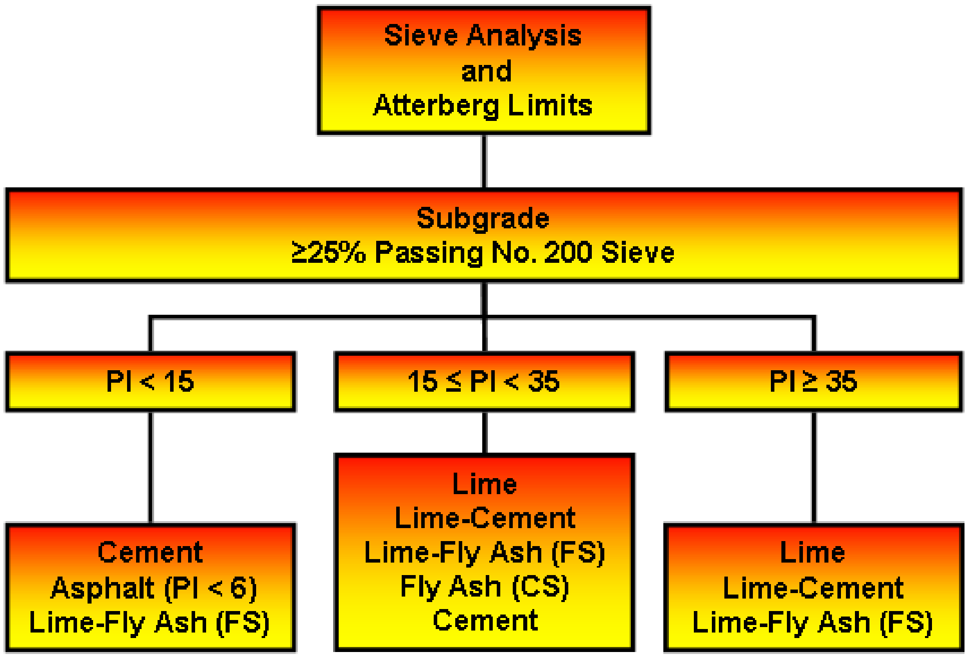


Figure 1.1 Additive selection criteria for subgrade material using soil classification

Different testing methods, design, construction, and quality assurance/quality control (QA/QC) methodologies have been developed for these stabilized materials (ETL-1110, 1999; Little, 1995; Porbaha and Puppala, 2003). Many pavement projects constructed with stabilized materials have achieved satisfactory results. However, challenges still remain in the optimal use of these stabilized materials. Other challenges in stabilization include developing better understanding of the long-term performance of the stabilized materials, better construction methods and using proper quality assurance/quality control (QA/QC) procedures that are effective predictors of the long-term performance of pavement infrastructure with minimal distress problems (Little et al., 2000).

One measure of this long-term efficacy of a stabilized material is the performance of the treated material in the presence of moisture conditioning. Currently many stabilization design guides do not account for the durability of the stabilization and also, potential leaching of chemicals due to constant moisture ingress into the treated soils.

Hence, in this research study an attempt is made to address some of the limitations in soil stabilization area. Incorporation of mineralogy aspects of soils into stabilization design process and conducting durability studies such as wetting/drying cycles and leachate studies to address the long-term effectiveness of stabilization are the main focus of this dissertation investigation. The stabilization under consideration is chemical in nature and the stabilizers considered in this study are lime and cement.

1.2 Research objectives

The main objective of this research study is to develop a rational soil stabilization design method by incorporating clay mineralogy of the soil, incorporate the long-term performance of the stabilizer (as per wetting and drying cycles) and potential leaching of chemical additives from stabilized soil. Specific objectives of this dissertation study are listed below:

1. Conduct chemical tests on a variety of soils to obtain their properties like Cation Exchange Capacity (CEC), Specific Surface Area (SSA) and Total Potassium (TP).
2. Develop a model to determine the clay mineralogy of soil using the above mentioned chemical test properties of soils.
3. Assess the efficacy of chemical stabilization by conducting durability tests including Wetting/Drying studies and Leachate Analyses.
4. Analyze and address the effects of dominating clay mineralogy of the soil on the long-term performance of the stabilization.
5. Develop rational guidelines for designing stabilization methods by incorporating the clay mineral information of the soils.

Figure 1.2 presents a schematic of the experimental program that was performed to accomplish the above specific objectives.

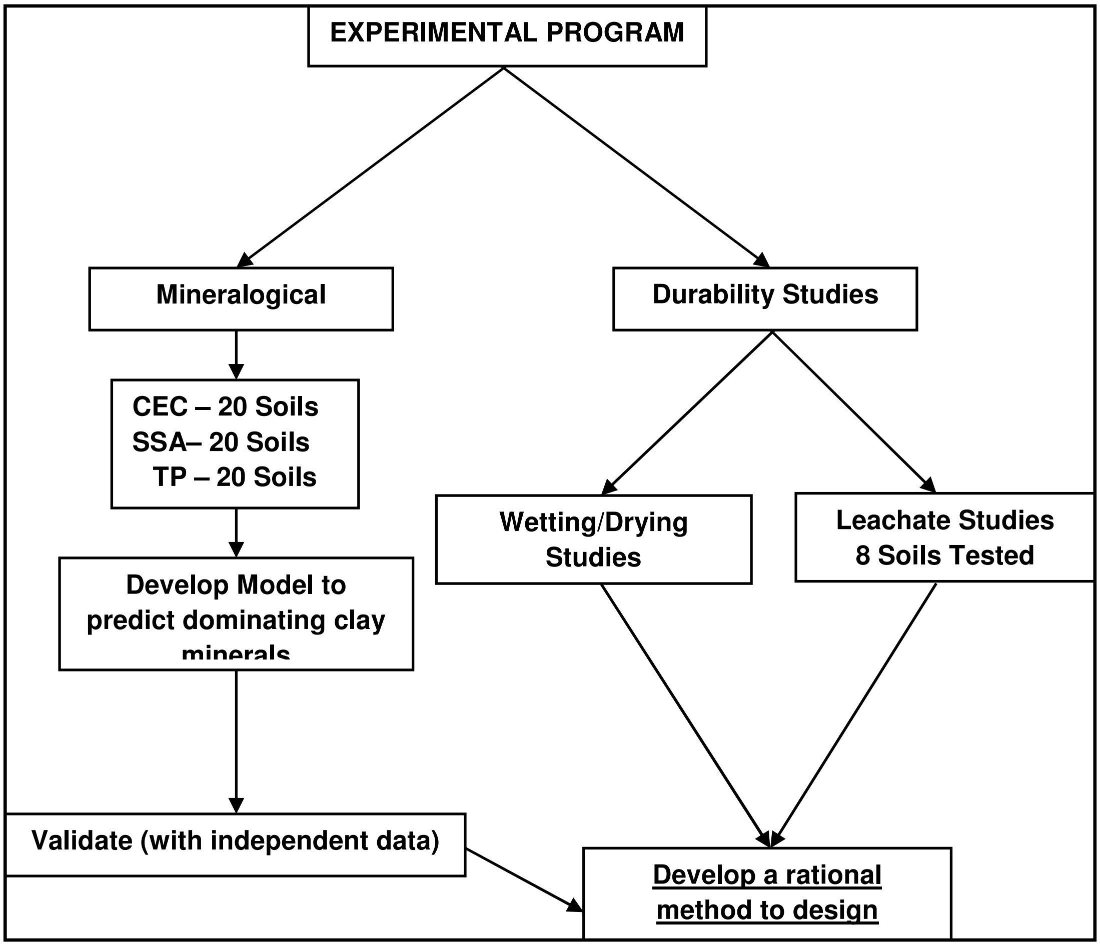


Figure 1.2 Schematic showing the experimental program followed in this research

1.3 Organization and summary

This section provides a brief summary of the contents of various chapters in this dissertation.

Chapter 2 reviews the available literature on expansive soils and their behavioral studies conducted by various researchers. A brief overview of common clay minerals available in expansive soils was presented along with various clay mineral identification methods that are available in the literature. The various methods available in literature for clay mineral quantification are also explained in detail. The current stabilization procedures followed by several state and federal agencies are explained along with their limitations. Previous research studies conducted to address the issues relating to durability and leachability of the stabilization were reviewed in detail.

Chapter 3 explains the details of various test procedures involved in this study. Procedures for the chemical tests on soil to obtain properties like CEC, SSA and TP are given. Sample preparation techniques for treated and untreated soil specimens to be used in wetting/drying and leachate studies were included. The procedures followed for wetting/drying and leaching of soil specimens were also explained in detail.

Chapter 4 describes the method used to determine the clay mineral information from the chemical test data. The database of the clay minerals information developed for the soils in Texas using this method was also included. This chapter also presents the details of two prediction models developed with the help of tools like statistical regression and artificial neural networks to identify the dominant clay mineral in a given soil.

Chapter 5 summarizes the results obtained from the wetting/drying studies conducted on treated and control soil specimens in this study. An accelerated curing method was developed and followed in this study for curing and moisture conditioning of the treated soil specimens. Tests were conducted on both accelerated cured and standard cured specimens. A total of 21 cycles of wetting/drying was performed on each of the soil specimens and strength tests were conducted after 3, 7, 14 and 21 cycles of wetting/drying. Also, volumetric strain changes in the soil specimens were monitored with the wetting/drying cycles. The effects of various factors such as curing time, dominant mineral, additive type and dosage on the performance of chemical stabilization (with lime and cement) under wetting/drying conditions were also studied and summarized in this chapter.

Chapter 6 summarizes the leachate studies conducted on all the soils selected to replicate moisture ingress and digress in the field during rainfalls and to study the effect of these moisture infiltrations on the long-term performance of stabilized soils. A total of 14 cycles of leaching were performed and leachate samples were collected after 3, 5, 7, and 14 cycles of leaching and these samples were subjected to pH and calcium concentration tests. Also, unconfined compressive strength tests were conducted after 3, 7 and 14 cycles of leaching to address the strength changes due to leaching in the soil specimens. The effects of various factors such as curing time, dominant mineral, additive type and dosage on the performance of stabilization under leaching conditions were also studied and summarized in this chapter.

Chapter 7 presents the finite element modeling studies conducted on hypothesized pavement sections to study the effects of deteriorating base modulus due to weakened stabilized base material on the performance of a flexible pavement. The details of the model and the results obtained are explained in this chapter.

Finally, summary and conclusions from this research study, significance of the findings from laboratory studies, and future research needs are addressed in chapter 8.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Stabilization of soils with chemical additives has been practiced for decades to improve the performance of base and subgrade materials, (Tayabji et al., 1982). Soil stabilization has been a topic of interest and discussion for all these years due to potential reduction in the construction and maintenance costs if the pavement infrastructure is built on problem grounds. Moreover, this treatment enhances riding comforts to travels. Extensive research was documented with regard to the engineering properties, reliability and durability of various types of stabilized materials.

Different testing methods, design, construction, and quality assurance/quality control (QA/QC) methodologies have been developed for these stabilized materials. Many pavement projects constructed with stabilized materials have achieved satisfactory results. However, challenges remain in the optimal use of these stabilized materials. These challenges include developing better understanding of the long-term performance of the stabilized materials, better construction methods and using proper quality assurance/quality control (QA/QC) procedures that are effective predictors of the long-term performance of pavement infrastructure with minimal distress problems (Little et al., 2000). This chapter reviews the available literature on soil stabilization and identifies the potential problems.

The first part of this chapter focuses on the expansive soils, problems caused by these soils and the mechanisms for these problems. The second part focuses on the techniques used to counter these expansive soils and their limitations. The third part focuses on the various methods that are available to identify the clay minerals in soils and their limitations. The fourth part reviews the properties that can be used to easily identify the dominating clay minerals in soils. The fifth part presents the literature on the long-term durability of the stabilized expansive soils.

2.2 Expansive soils

Expansive soils, also known as swell-shrink soils have been a problem to the civil infrastructures including roads and foundations from ancient times (Nelson and Miller, 1992). Expansive soils swell and shrink with changes in moisture content. This volume change behavior is the reason for the cracking of the structures such as buildings or pavements. The reason for this behavior is the presence of certain type of heaving 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). Examples of expansive clays include high-plasticity index (high-PI) clays, over-consolidated clays rich with Montmorillonite mineral, and shales.

The damage caused by the expansive soils to the structures built on them is immense. One of the earlier National Science Foundation (NSF) sponsored studies reported that the damage to structures caused by expansive soils - particularly to light buildings and pavements - is more than any other natural disaster, including earthquakes and floods (Jones and Holtz, 1973). As per the detailed review of expansive soils by Gromko (1974), it can be estimated that the annual cost of damage from these soils in the United States alone is $2.3 billion. Petry and Armstrong (1989) noted that it was more economical to perform initial stabilizations than performing remedial treatments later on with existing structures around.

According to Wiseman et al. (1985), the following factors can be used to classify a soil as a problematic or non-problematic type:

1) Soil type that exhibits considerable volume changes associated 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 following index tests (see Table 2.1) and the magnitudes of their test results:

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

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.

2.3 Clay mineralogy

The term clay is used as both a particle size and also to represent a family of minerals (Velde, 1995). When representing particle size, it indicates the soil particles that have their size less than 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 and Soga, 2005). Murray (1999) discusses the importance of clay minerals in various industries. According to him, clay minerals like kaolin, smectite and palygorskite-sepiolite are among the world's most important and useful industrial minerals. They have their importance in various geological applications such as stratigraphic correlations, indicators of environments of deposition and temperature for generation of hydrocarbons (Murray, 1999).

More than one-half of the volume of soils is minerals. 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). Klien and Hurlbut (1993) define a mineral as follows: “A mineral is a naturally occurring homogeneous solid with a definite (but generally fixed) chemical composition and a highly ordered atomic arrangement (crystalline structure)”. Both crystal structure and chemical composition are necessary to define a mineral (Schulze, 2002).

Soils in general contain various amounts of crystalline clay and non-clay minerals, non-crystalline matter and precipitated salts (Mitchell and Soga, 2005). Usually majority of the soil is comprised of crystalline minerals which are primarily non-clay. Hence, the percentage of crystalline clay minerals in a given soil is relatively low. However, the influence of these clay minerals on the properties of the soil is far more than their abundance. Mineralogy of soil controls its size, shape, physical and chemical properties. Based on the mineralogy, the particle size of soil varies from very large cobbles and gravel to very fine silts and clays (Mitchell and Soga, 2005).

Clay minerals in soils belong to a family known as *phyllosilicates* or layered silicates. According to Brindley and Pedro (1972) "Clay minerals contain continuous two-dimensional tetrahedral sheets of composition Si_2O_5, Al_2O_5, Be_2O_5 etc. with tetrahedra linked by sharing three corners of each, and with the fourth corner pointing in any direction. The tetrahedral sheets are linked in the unit structure to octahedral sheets, or to groups of coordinated cations, or individual cations" (Bailey, 1980). A brief overview of the common clay minerals is given in the following section.

2.3.1 Common clay minerals

Clay minerals occur in small particle sizes and their unit cells ordinarily have a residual negative charge. The different clay mineral groups are characterized by the stacking arrangements of sheets (Mitchell and Soga, 2005). Figure 2.1 presents a systematic manner in which synthesis of different clay minerals takes place. The common clay minerals usually found in soil are Kaolinite, Illite and Montmorillonite. Kaolinite is a common phyllosilicate mineral in subgrades; it is most abundant in soils of warm moist climates. 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. A brief discussion of the common clay minerals is as follows:

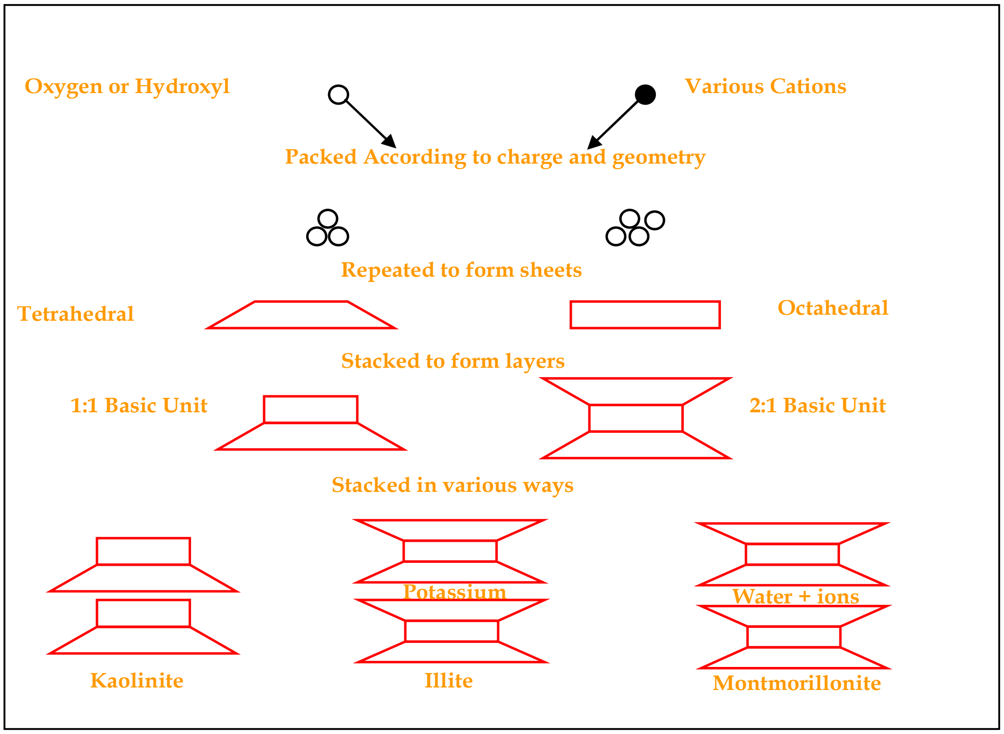


Figure 2.1 Schematic showing the clay mineral formation

Illite is essentially a group name for non-expanding, clay-sized, dioctahedral, micaceous minerals. It is structurally similar to muscovite in that its basic unit is a layer composed of two inward-pointing silica tetragonal sheets with a central octahedral sheet. Figure 2.2 and Figure 2.3 show the mineral structure and scanning electron micrograph or SEM photograph of the mineral Illite. The weaker interlayer forces caused by fewer interlayer cations in Illite also allow for more variability in the manner of stacking (Grim, 1962).

Illites are the dominant clay minerals in argillaceous rocks and are formed by the weathering of silicates (primarily feldspar), through the alteration of other clay minerals, and during the degradation of muscovite. Formation of Illite is generally favored by alkaline conditions and by high concentrations of Al and K. The number of inter particle contacts is less in micas and hence, the cohesive forces between the crystallites are weak (Thompson & Ukrainczyk, 2002). The degree to which Illite crystals contact adjacent grains is a function of soil water content as well as particle size, shape and flexibility.

Kaolinite is a common phyllosilicate mineral in subgrades; it is most abundant in soils of warm moist climates. Kaolinite's structure is composed of alternate silicate sheets (Si_2O_5) and aluminum oxide/hydroxide sheets ($Al_2(OH)_4$) called gibbsite sheets (See Figure 2.4). The silicate and gibbsite layers are tightly bonded together with only weak bonding existing between these silicate/gibbsite paired layers (called s-g layers). The weak bonds between these s-g layers cause the cleavage and softness of this mineral. The structure is very similar to the Serpentine Group and at times the two groups are combined into a Kaolinite-serpentine Group (Figure 2.4). These minerals are also called as 1:1 minerals. Kaolinite shares the same chemistry as the minerals halloysite, dickite and nacrite. The four minerals are polymorphs; meaning they have the same chemistry, but different structures. All four minerals form from the alteration (mostly weathering) of aluminum rich silicate minerals such as feldspars. Kaolinite is by far the most common and most clay deposits contain at least some Kaolinite.

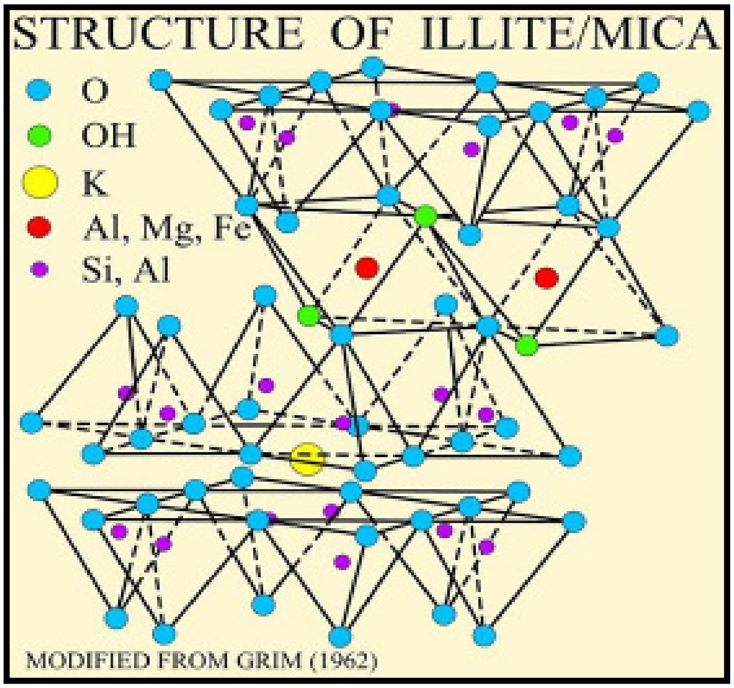


Figure 2.2 Illite crystal structure
(Source: http://pubs.usgs.gov/of/2001/of01-041/htmldocs/images/illstruc.jpg)

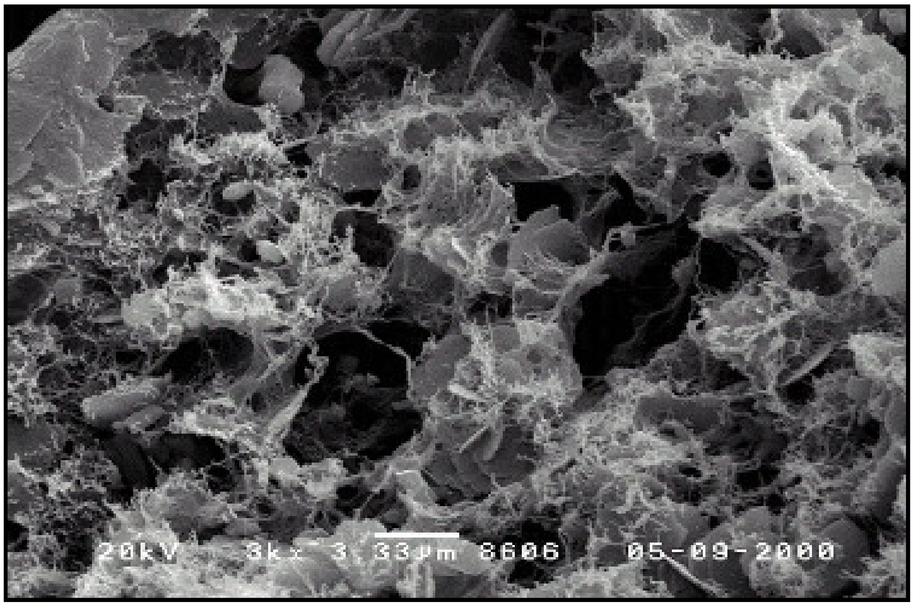


Figure 2.3 SEM photograph of Illite
(Source: http://www.petrotech-assoc.com/images/smectite_Illite_2.jpg)

Kaolinite has low value for the cationic exchange capacity (CEC), its value ranges from 1 to 5 meq/gm (White and Dixon, 2002). Dixon (1989) reported values of specific surface of Kaolinite as low as 5 m^2/g and as high as 39 m^2/g. Presence of Kaolinite in soil is desirable as it reduces the influence of more reactive minerals like smectite by lowering the water holding capacity and plastic properties. According to Di Maio and Fenelli (1994) Kaolinite is unaffected by exchangeable cations and so the Atterberg limits are less sensitive to CEC (Anson and Hawkins, 1998). Sridharan et al., (1988) showed that in soils dominated by Kaolinite, particle arrangement is regulated by liquid limit values. Kaolinitic soils with higher liquid limit have an increased shrinkage limit and sedimentation volume in water (Sridharan et al., 1988).

Figure 2.5 presents the SEM photograph of the mineral Kaolinite.

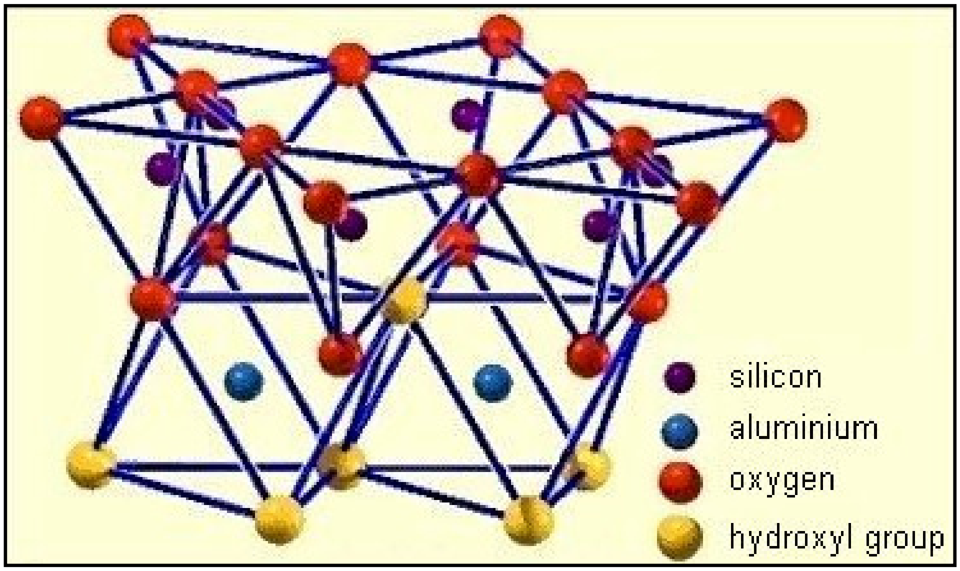


Figure 2.4 Mineral structure of Kaolinite
(Source: http://media-2.web.britannica.com/eb-media/76/2676-004-3893834B.gif)

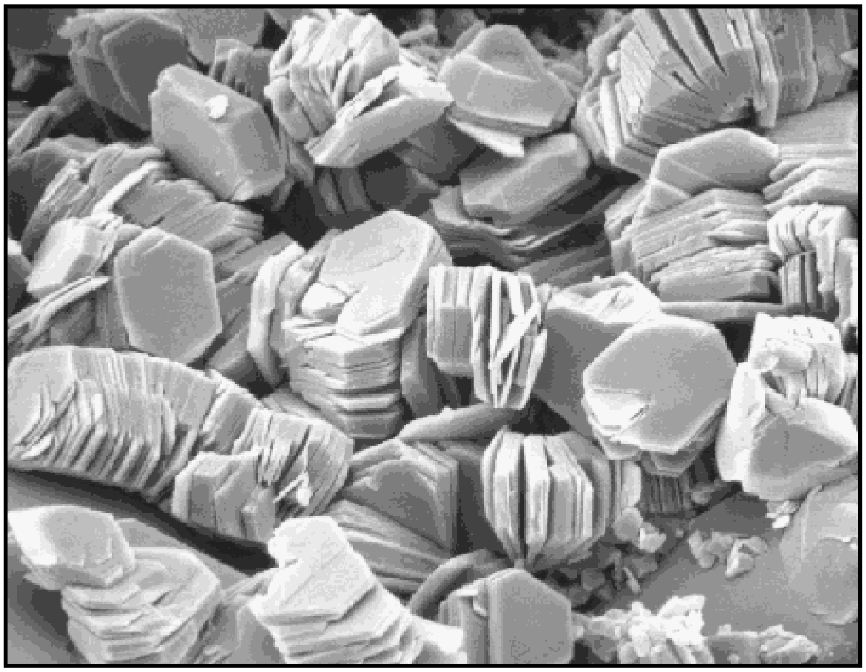


Figure 2.5 SEM photograph of the mineral Kaolinite
(Source: http://www.uni-kiel.de/anorg/lagaly/group/jose/Kaolinite.gif)

Montmorillonite is a member of the smectite group which includes other dioctahedral minerals beidellite, and nontronite, and the trioctahedral minerals hectorite (Li-rich), saponite (Mg-rich), and sauconite (Zn-rich). These are expansive 2:1 layer silicate minerals. The basic structural unit is a layer consisting of two inward-pointing tetrahedral sheets with a central alumina octahedral sheet (refer Figure 2.6). The bonds between layers are weak and have excellent cleavage, allowing water and other molecules to enter between the layers causing expansion (Grim, 1953). 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. Because of this small particle size and interlayer expansion Montmorillonites have very high specific surface area values ranging from 600 m^2/g to 800 m^2/g. The range of CEC values for smectites is given by Borchardt (1989) as 47 to 162 meq/g. Soil mineralogy containing Montmorillonites are more prone to landslides as these soils retain more water and drain more slowly than soils with Kaolinite or soils with little clay.

Figure 2.7 shows a scanning electron microscope (SEM) photograph of the mineral Montmorillonite.

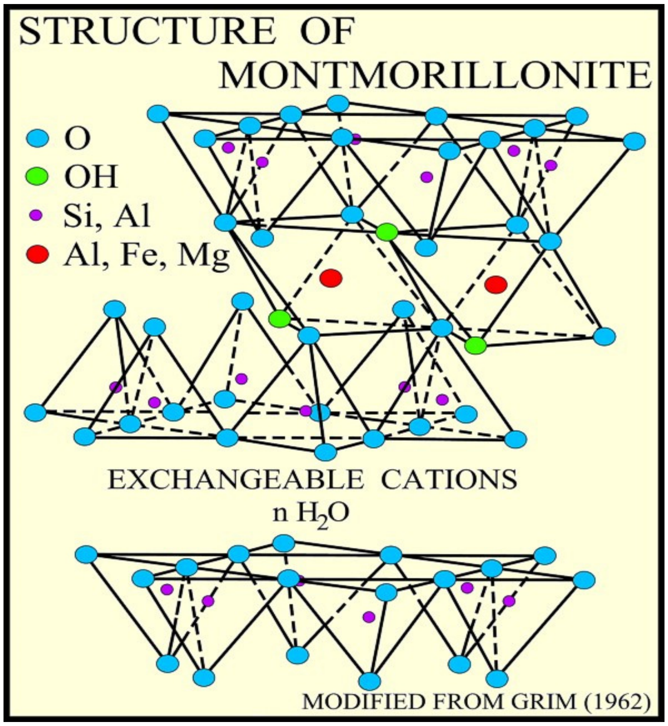


Figure 2.6 Mineral structure of Montmorillonite
(Source: http://pubs.usgs.gov/of/2001/of01-041/htmldocs/images/monstru.jpg)

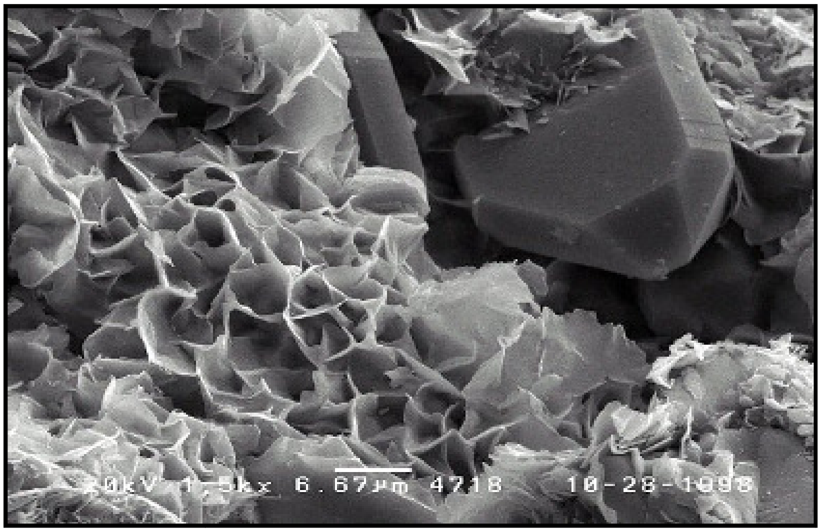


Figure 2.7 SEM photograph of Montmorillonite
(Source: http://www.webmineral.com/specimens/Smectite.jpg)

2.3.2 Clay mineral identification

This section introduces the various methods currently in practice for the process of identification of minerals. A brief explanation on the underlying philosophy of each method is mentioned. The main intent of this section is to provide an overview of philosophy, methodology and current capabilities and limitations of each technique covered in the literature.

There are two contradictory philosophies in practice to identify soil minerals. One is holism which states that the applicability of characterization data decreases when a soil sample is fractionated into specific isolates that do not preserve the original spatial relationships of the various soil components. On the other hand reductionism says that an accurate determination of some mineral phases is not possible without isolation and elimination of interferences caused by the organic matter (Amonette, 2002). However, a combination of both fractionation and holistic approaches is necessary to identify minerals appropriately. 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).

The following sections explain the various methods to identify clay minerals in soils.

2.3.2.1 X-Ray Diffraction (XRD)

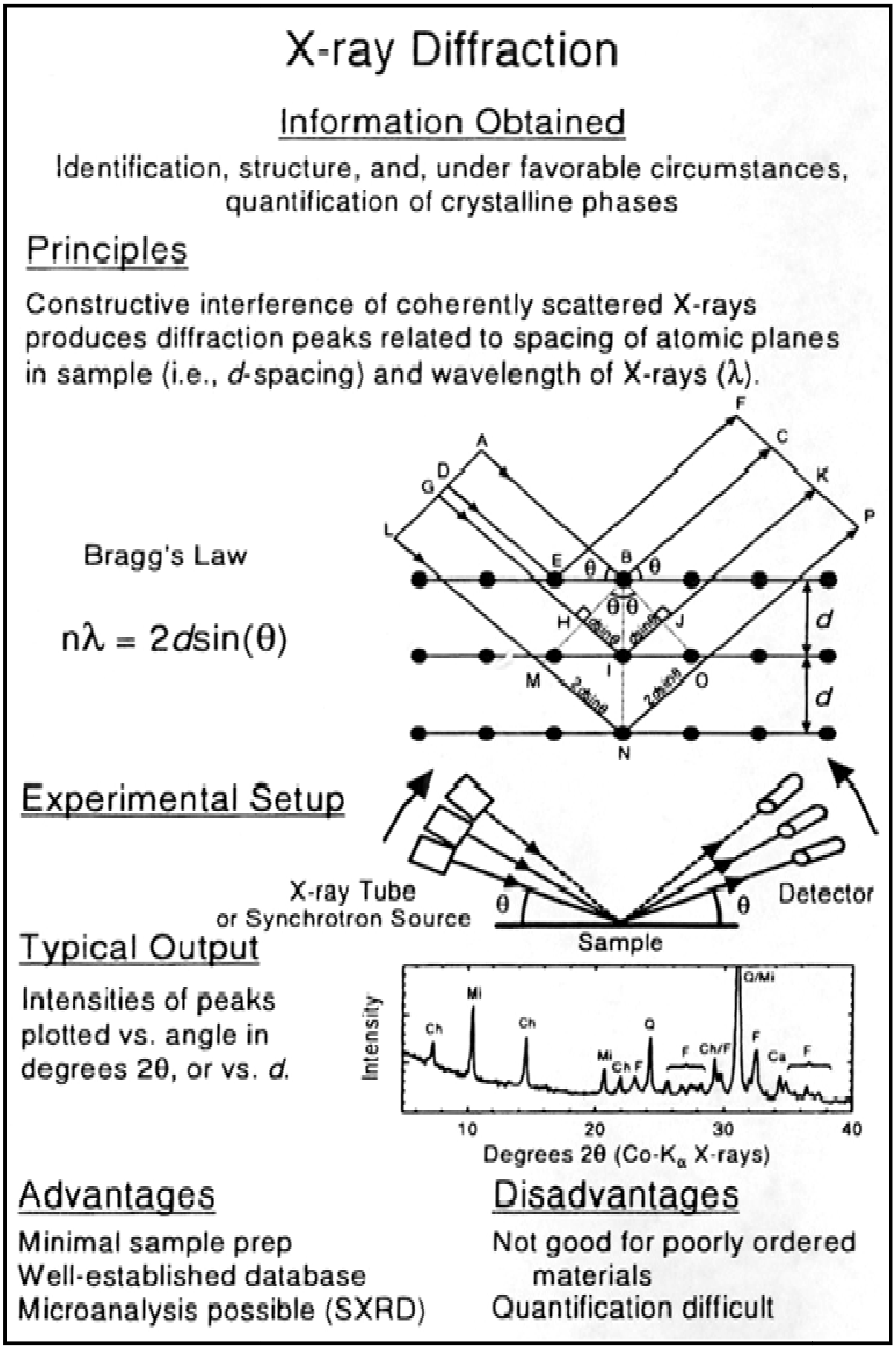
Clay minerals are characterized to be crystalline in nature, i.e. they have long range structure that repeats itself for hundreds of nanometers to millimeters. These kinds of minerals can be identified by X-ray diffraction studies because each crystal contains planes of atoms separated by constant distance which is due to the periodic nature of the standard spacing of the atoms (Whittig & Allardice, 1986). But there are minerals that have a short range structure which extends only to a few nanometers; they are referred to as amorphous or non-crystalline. Identifying these kinds of minerals is a challenge. But, procedures like vibrational spectroscopy or X-ray absorption have been developed to identify these kinds of minerals (Whittig & Allardice, 1986).

When an X-ray beam falls on equally spaced atoms of a crystalline mineral they are transmitted, absorbed or scattered (Cullity, 1978). When scattering, they can be scattered coherently (without loss of energy) or incoherently (with loss of energy). The coherently scattered light will form an interference pattern when the scattering centers are arranged in a regular array and the distance between scattering centers is comparable to the wavelength of the light and this phenomenon is called *diffraction* (Amonette, 2002). When the incident beams of X-rays are diffracted a detector captures the beam and converts the analog signal into digital data which can be plotted.

This data is used to measure the distances between the planes of the atoms using Bragg's law (refer to Figure 2.8). The basis for the identification of crystals using X-ray diffractions was that, no two minerals have same inter atomic distances similar in three dimensions and so have different intensities. This intensity pattern is compared with standard patterns for known materials. Extensive X-ray diffraction data for clay minerals and other soil minerals are given by Grim (1953), Whittig and Allardice (1986), and Moore and Reynolds (1989). A detailed analysis of X-ray patterns is given by Mitchell and Soga (2005).

As the size of the clay particles is small it is difficult to study single crystals, hence p*owder diffraction* method is generally used for soils. In this method small sample of particles, containing all possible orientations is kept under the X-ray beam. This method works because of the large number of particles that insure that some will be oriented in the right way to produce the desired reflection (Whittig & Allardice, 1986).

Narasimha Rao (1993) used both XRD and SEM techniques to successfully identify the formation of the compounds such as Calcium Aluminate Hydrate and Calcium Silicate Hydrate when the soil is treated with lime. In a study conducted by Yan and Jin (2004), clay mineral composition and smectite contents were measured using XRD and dye absorption methods. They have concluded that the laboratory spectral measurement and analysis techniques to the dried and homogenous swelling soils can be productively used for quickly, economically, and conveniently identifying swelling soils, estimating the contents of smectite, colloid, and clay in fields.



X-ray Diffraction

Information Obtained

Identification, structure, and, under favorable circumstances, quantification of crystalline phases

Principles

Constructive interference of coherently scattered X-rays produces diffraction peaks related to spacing of atomic planes in sample (i.e., *d*-spacing) and wavelength of X-rays (λ).

Bragg's Law

$$n\lambda = 2d\sin(\theta)$$

Experimental Setup

Typical Output

Intensities of peaks plotted vs. angle in degrees 2θ, or vs. *d*.

Advantages

Minimal sample prep
Well-established database
Microanalysis possible (SXRD)

Disadvantages

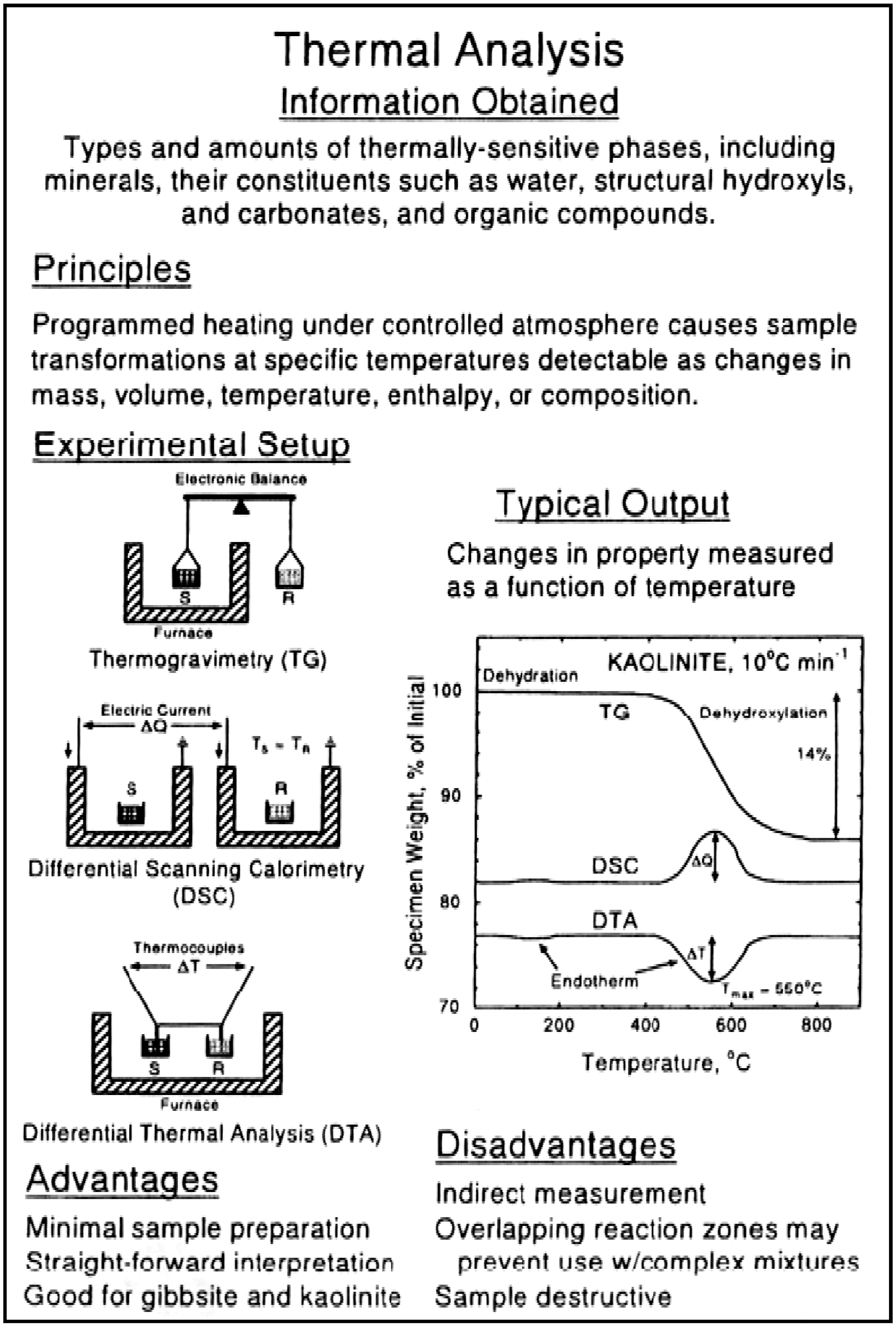
Not good for poorly ordered materials
Quantification difficult

Figure 2.8 Principles, setup and typical output for X-ray diffraction pattern (from Amonette, 2002)

2.3.2.2Thermal Analysis (TA)

In this technique mineral identification is achieved by evaluating the weight loss and enthalpy changes in the soil sample with temperature. Thermal analysis has been used by many researchers (Benham, 1990; Karathanasis and Harris, 1994; Wunderlich, 1990) to identify clay minerals in soils. The most common methods used in TA are thermal gravimetric analysis (TGA) differential scanning calorimetry (DSC) (Beck, 2004). A detailed explanation of these methods is given by Karathanasis and Harris (1994).

In TGA, the weight of the soil sample is monitored by changing the temperature at constant rate. Many researchers (Barshad, 1965; Dish and Duffy, 1990) had used the TGA method to identify and quantify soil minerals. In DSC method the energy required to maintain the soil sample and the reference material at same temperature during heating is measured and evaluated. A detailed explanation of this technique is given by Karathanasis and Harris (1994) and Beck (2004). Figure 2.9 describes in detail the process of thermal analysis which can be used for the identification of minerals.



Thermal Analysis

Information Obtained

Types and amounts of thermally-sensitive phases, including minerals, their constituents such as water, structural hydroxyls, and carbonates, and organic compounds.

Principles

Programmed heating under controlled atmosphere causes sample transformations at specific temperatures detectable as changes in mass, volume, temperature, enthalpy, or composition.

Experimental Setup

Thermogravimetry (TG)

Differential Scanning Calorimetry (DSC)

Differential Thermal Analysis (DTA)

Typical Output

Changes in property measured as a function of temperature

Advantages

Minimal sample preparation
Straight-forward interpretation
Good for gibbsite and kaolinite

Disadvantages

Indirect measurement
Overlapping reaction zones may prevent use w/complex mixtures
Sample destructive

Figure 2.9 Principles, setup, and output for common thermal analysis (from Amonette, 2002)

2.3.2.3 Atomic Absorption Spectrometry (AAS)

Atomic Absorption Spectrometry (AAS) has been used for the identification of elements from quite long time. Atomic absorption as the name implies, depends on the measurement of atomic species. The whole technique depends on the ability to consistently atomize the element of interest in a reproducible manner in both samples and standards. Baker and Suhr (1982) give the principles involved and the different literatures available.

When an atom is excited by thermal energy or other energy sources they emit radiation by dropping down to less energetic states or to the ground states as atoms always try to be in ground state at all times. This can be given by the following equation 2.1 (Baker and Suhr, 1982):

$$M + hv \rightarrow M^{*} \rightarrow M + hv \quad \text{.......} (2.1)$$

Where,

M = Neutral atom,
h = Planks constant, and
v = Frequency of the irradiating light.

AAS relies on the absorption of a photon having the exact energy needed to convert an atom or ion in the ground state to an excited state. For each element to be to be determined, a lamp producing light of appropriate energy irradiates the atomized sample, and the loss of intensity in this light is used to determine the amount of element present (Amonette, 2002). The only difference between AAS and Atomic Emission Spectrometry (AES) is that in former absorbed light intensity i.e. the loss in the intensity is measured and in latter the intensity of the light that is emitted is measured. Figure 2.10 gives a pictorial representation of different methods for elemental analysis in which the sample is completely destructed.

Majority of non-destructive soil/clay testing depend on the excitation of atoms in the specimen to produce X-ray characteristics of each element present based on the energies and the intensities of the X-ray produce elements are identified and quantified. X-rays are produced in two stages. In the first stage a vacancy is created in the inner shell of an electron and then in the second stage that vacancy is filled by a higher energy electron coincident with the release of an X-ray photon. The vacancy is created when the atom absorbs the energy from an energy source in the form of a photon or a charged particle. X-rays produced by absorbing the energy from photons are called X-ray fluorescence (XRF) and that by charged particles are called X-ray emission (XRE). The energy source for XRF can be X-rays from an X-ray tube or gamma rays from nuclear decay. For the XRE a variety of high energy charged particles can be used, although electrons and protons are the most common (Amonette, 2002).

2.3.2.4 Vibrational Spectroscopy (VS)

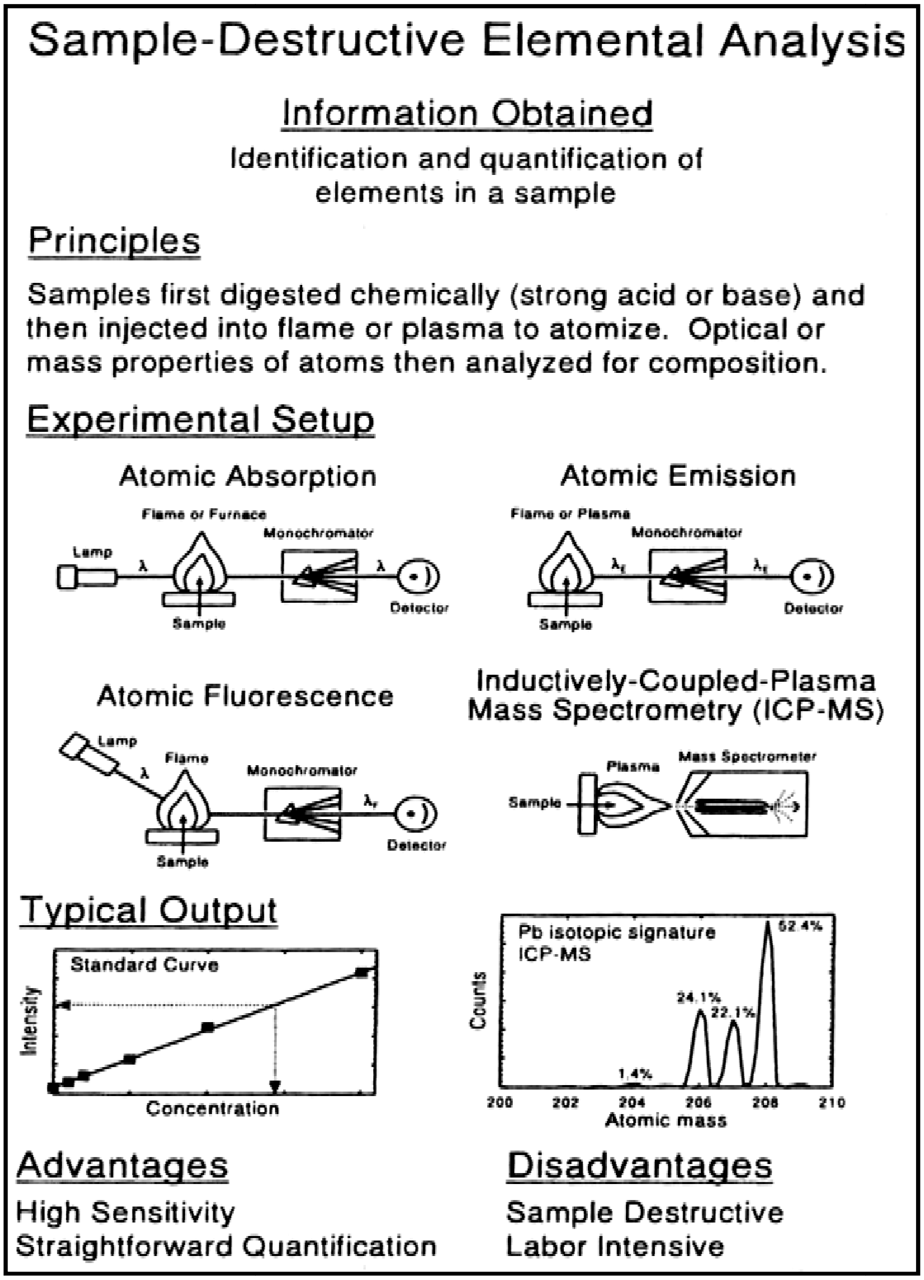
Vibrational spectroscopy provides the most definitive means of identifying the surface species generated upon molecular absorption and the species generated by surface reactions. In principle, any technique that can be used to obtain vibrational data from solid state or gas phase samples including infrared spectroscopy and Raman spectroscopy methods can be applied to study soil surfaces.

Figure 2.11 shows a pictorial representation of the experimental setup of vibrational spectroscopy along with its advantages and disadvantages.

There are, however, only two techniques that are routinely used for vibrational studies of molecules on surfaces. These are INFRARED Spectroscopy and RAMAN Spectroscopy.

Infrared spectroscopy involves the direct measurement of infrared light absorbed or emitted by a specimen. Raman spectroscopy on the other hand, is an incoherent-scattering technique in which the loss or gain in energy by the interaction of light with the atoms in a bond is measured (Amonette, 2002). A detailed review of the vibrational spectroscopy is given in VS. Vibrational spectroscopy is particularly well suited for characterization of minerals containing hydroxyl, carbonate or sulfate groups and for studies of organic molecules associated with minerals surfaces.

Linker et al. (2005) studied that the use of mid-infrared attenuated total reflectance (ATR) spectroscopy enables direct measurement of nitrate concentration in soil pastes. But their accuracy depends on the soil type which again depends on the varying contents of carbonate, whose absorbance band overlaps the nitrate band.



Sample-Destructive Elemental Analysis

Information Obtained

Identification and quantification of elements in a sample

Principles

Samples first digested chemically (strong acid or base) and then injected into flame or plasma to atomize. Optical or mass properties of atoms then analyzed for composition.

Experimental Setup

Advantages

High Sensitivity
Straightforward Quantification

Disadvantages

Sample Destructive
Labor Intensive

Figure 2.10 Different methods for elemental analysis in which the sample is completely destructed (from Amonette, 2002)

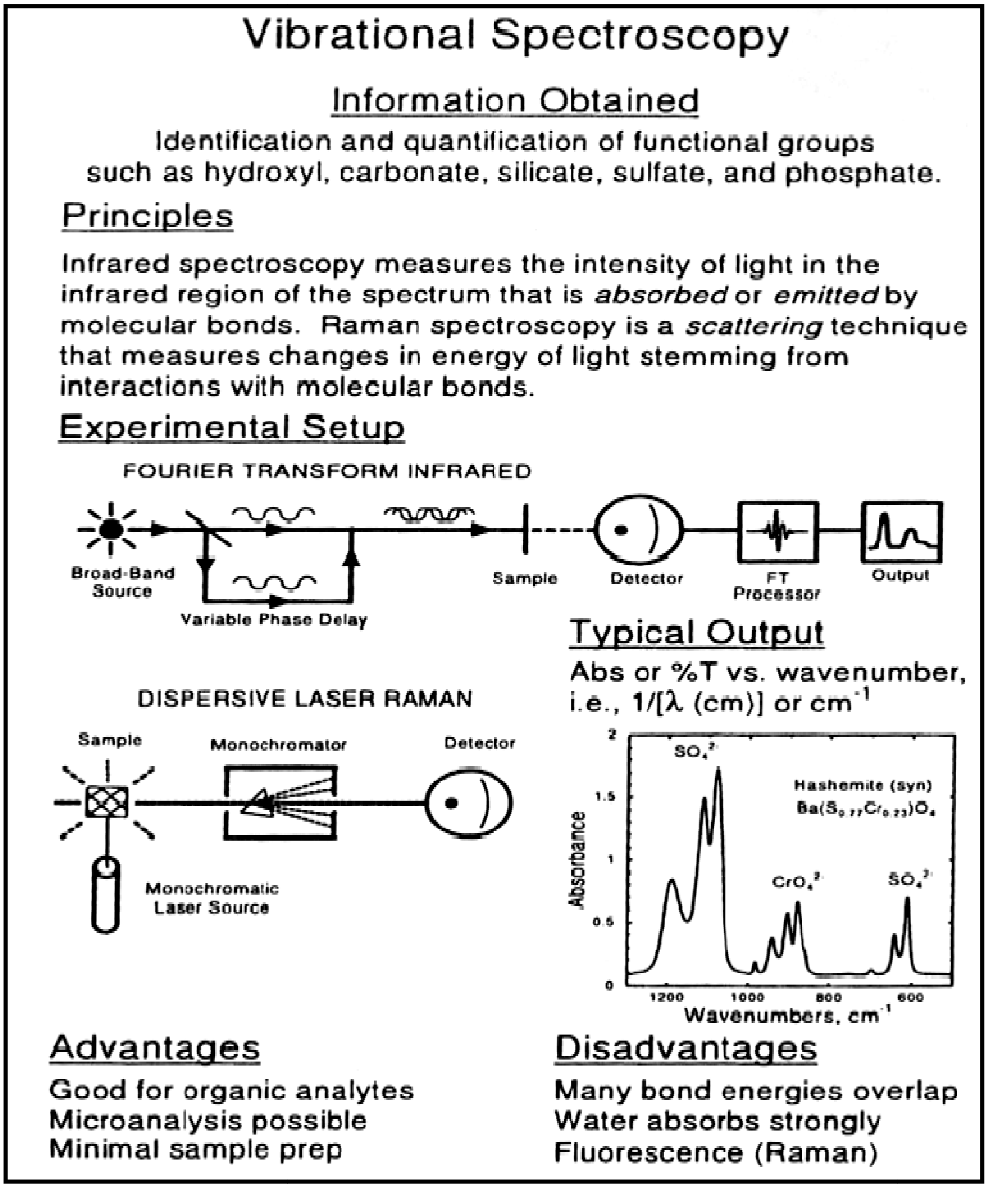
Kariuki et al. (2003) has investigated the effectiveness of spectroscopy in identifying the swelling indicator in soils. They used coefficient of linear extensibility (COLE) and the linear extensibility (LErod), as the controlling methods to classify the soils into swelling potential groups and to assign them to dominant clay mineral types. The study indicated that spectroscopy can be used in classifying soils however; high organic matter and the presence of moisture were found to affect area and depth intensities and would require consideration in such applications.

According to Blake and Gassmann (2001) bench-top Fourier transform infrared spectrometer and specular reflection accessory can be used to detect soil surface contaminants. They found that the volume scattering features showed a significant depolarization of the light, the degree of polarization after reflection is less than or equal 20%, and the surface scattering features retained a much higher degree of polarization upon reflection, >75%.

2.3.2.5 X-ray Absorbance Spectroscopy (XAS)

X-ray absorption spectroscopy is an element-specific probe of the local structure (short range) of elements in a sample (XAS). Interpretation of XAS spectra commonly uses standards with known structures, but can also be accomplished using theory to derive the structure of a material. In either case, the species of the material is determined based on its unique local structure. An important advantage of this technique is its utility for heterogeneous sample, a wide variety of solid and liquids, including whole soils and liquids, can all be examined directly and nondestructively. Additionally, since the local structure does not depend on long-range crystalline order, the structure of amorphous phases (and that of dissolved species) is easily achieved (XAS).

X-ray absorption spectroscopy is commonly divided into two spectral regions; the first is the X-ray absorption near edge structure (XANES) spectral region and the second is termed the extended X-ray absorption fine structure (EXAFS) region (Amonette, 2002). XANES spectra are unique to the oxidation state and speciation of the element of interest, and consequently are often used as a method to determine the oxidation state and coordination environment of materials.



Vibrational Spectroscopy

Information Obtained

Identification and quantification of functional groups such as hydroxyl, carbonate, silicate, sulfate, and phosphate.

Principles

Infrared spectroscopy measures the intensity of light in the infrared region of the spectrum that is *absorbed* or *emitted* by molecular bonds. Raman spectroscopy is a *scattering* technique that measures changes in energy of light stemming from interactions with molecular bonds.

Experimental Setup

Typical Output

Abs or %T vs. wavenumber, i.e., 1/[λ (cm)] or cm^{-1}

Advantages

Good for organic analytes
Microanalysis possible
Minimal sample prep

Disadvantages

Many bond energies overlap
Water absorbs strongly
Fluorescence (Raman)

Figure 2.11 Experimental setup of vibrational spectroscopy and its advantages and disadvantages (from Amonette, 2002)

EXAFS spectra are best described as a series of periodic sine waves that decay in intensity as the incident energy increases from the absorption edge (Amonette, 2002). These sine waves result from the interaction of the elected photoelectron with the surrounding atomic environment. As such, their amplitude and phase depend on the local structure of excited atom. Since this interaction is well understood, theory is sufficiently advanced that the local structure of the excited atom can be determined by matching a theoretical spectrum to the experimental spectrum. This fitting yields many types of information, including the identity of neighboring atoms, their distance from the excited atom, the number of atoms in the shell, and the degree of disorder in the particular atomic shell (as expressed by the Debye-Waller factor) (XAS). These distances and coordination numbers are diagnostic of a specific mineral or adsorbate-mineral interaction; consequently, the data are useful to identify and quantify major mineral phases, adsorption complexes, and crystallinity.

2.3.3 Clay mineral quantification

Several clay mineral quantification techniques using the XRD method have been developed by many researches (Hughes et al., 1994; Salyn & Drits, 1972; Smith, 1989; Jones, 1989). Traditionally the samples used for the clay mineral quantification were oriented specimens of the clay fraction but the modern methods use random powdered samples which may not be totally representative of a large soil sample. Different methods involving XRD data analysis for clay mineral quantification are explained in brief in the following sections.

Theoretically, the diffraction peaks are related to the diffraction planes present in the sample, hence using the relative intensities of the peaks the concentration of the mineral species present in the test specimen can be estimated (Whittig & Allardice, 1986). However, there are other factors such as crystal perfection, chemical composition, variation in sample packing, crystal orientation and presence of amorphous substance that influence the diffraction peaks (Jackson, 1969). A detailed explanation of the influence of these factors on the diffraction maxima can be found in Whittig and Allardice (1986). The influence of these factors in the process of mineral quantification can be overcome by the use of standard mineral. There are two different methods to quantify minerals using standards a) Internal standards b) External standards. In the internal standards, method known amounts of a mineral not already present in the sample is added and the sample is analyzed under the X-ray machine and the diffraction peaks are recorded. These peaks are compared with the standard sample (without the addition of internal standard) to obtain the percentage of the minerals present. A more detailed explanation can be obtained in Whittig and Allardice (1986).

Due to availability of complete diffraction data and the draw backs of above methods, innovative methods involving full pattern modeling are needed to be developed. One such method developed is the Rietveld method, which yields a calculated pattern that can be described as the sum of all patterns for each phase in a given sample (Kahle et al., 2002). In this method, differences between the calculated and observed patterns are minimized by a refinement procedure that uses a least square algorithm. The final quantifications are obtained from the final values of the refined scale factor of each phase. There is another full pattern modeling method known as the Arquant model developed by Blanc et al. (2006) which has been successfully applied for clay mineral quantification in soils and rocks.

Other methods such as Absorption-diffraction method and the methods based on mineral intensity factors have been also developed and a brief explanation of these methods can be found in Kahle et at. (2002). Figure 2.12 gives a flowchart of the protocols for the above referred methods.

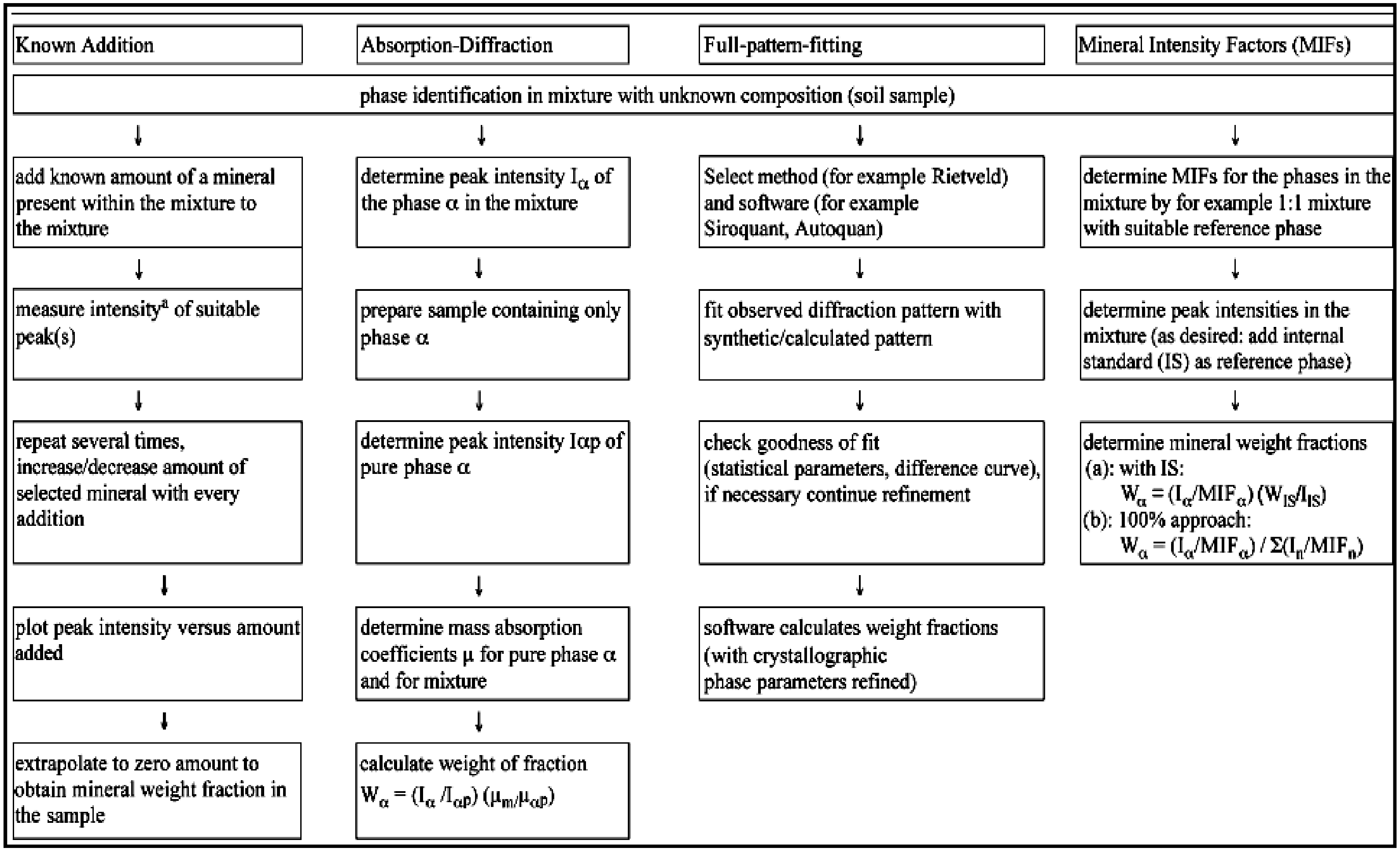


Figure 2.12 Protocols for XRD-based quantification of clay minerals in soil clay samples
(From Kahle et al., 2002)

Smith et al., (1986) have developed a different scheme using XRD data which is based on the analysis of the whole diffraction pattern from 2°- 50° 2θ through comparisons with a reference database of diffraction traces obtained from a set of carefully purified reference clay samples run under the same conditions.

Quantitative determinations of the amount of clay minerals based on simple comparison of the diffraction peak heights or areas are uncertain due to many factors like differences in absorption coefficients, particle orientations, crystallinity and other factors (Mitchell & Soga, 2005). As a result, other techniques have been developed that accounted for the above mentioned differences and are currently used for the clay mineral quantification (Alexaides & Jackson, 1966; Hodgson & Dudeney, 1984; Johnson et al., 1985; Randall et al., 1994).

Hodgson and Dudney (1984) have developed an analysis procedure which uses both XRD data and chemical mass balance concepts to estimate the percentage of each mineral in a soil. Johnson et al. (1985) developed a program of simultaneous linear equations to develop component proportions of minerals in soils and sediments. This method also uses both XRD data and the chemical data to quantify the clay minerals in the soil. Both these methods require a detailed chemical analysis data.

Chemical mass balance alone can be used to quantify clay minerals in soils. Many researchers (Alexaides and Jackson, 1966; Hodgson and Dudeney, 1984; Johnson et al., 1985) have already used elemental mass balance techniques to asses each mineral percentage in the soil samples. In these methods amount of each element is measured with the help of laboratory chemical analysis of the soil sample and this information is used to formulate simultaneous equations which can be solved to obtain the percentage of the minerals in soils.

Randall et al. (1994) compared four such methods of clay mineral quantification using elemental mass balance methods and highlighted their corresponding strengths and weaknesses. A brief description of those methods is given here. These methods primarily are solving a system of simultaneous linear equations which are formulated using the elemental information of the soil specimen and the minerals for which they are being analyzed. A detailed explanation of how these methods work can be found in Randall et al. (1994).

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. Currently clay mineral quantification using elemental information has only been explored. However, there are other properties of soils that can be used to approximate the clay mineral information.

In this research, a new model is developed using few such properties. Those chemical properties of the soils which can be used to assess the dominating clay mineral are described in the following sections.

2.3.4 Cation Exchange Capacity (CEC)

The cation exchange capacity (CEC) of a soil is 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 (see Figure 2.13). The positively charged ions or cations are attracted by negative charges, hence the name ‘cation exchange capacity’. Soil CEC is normally expressed in units of charge per weight of soil. Two different, but numerically equivalent sets of units are used: meq/100 g (milliequivalents of charge per 100 g of dry soil) or $cmol_c/kg$ (centimoles of charge per kilogram of dry soil). CEC is a good indicator of soil reactivity with the chemical species.

The negative charges in the soil are obtained from the following sources and reactions (Rhoades, 1982):

a) Isomorphous substitution within the structures of layer silicate minerals

b) Broken bonds at mineral edges and external surfaces

c) Dissociation of acidic functional groups in organic compounds

d) The preferential adsorption of certain ions on the particle surfaces.

The first of these charges is permanent and is independent of pH and the rest are dependent on pH. CEC is not independent of the conditions under which it is measured hence, it is necessary to measure the soil's capacity to adsorb cations from an aqueous solution of the same pH, ionic strength, dielectric constant and composition as that encountered in field.

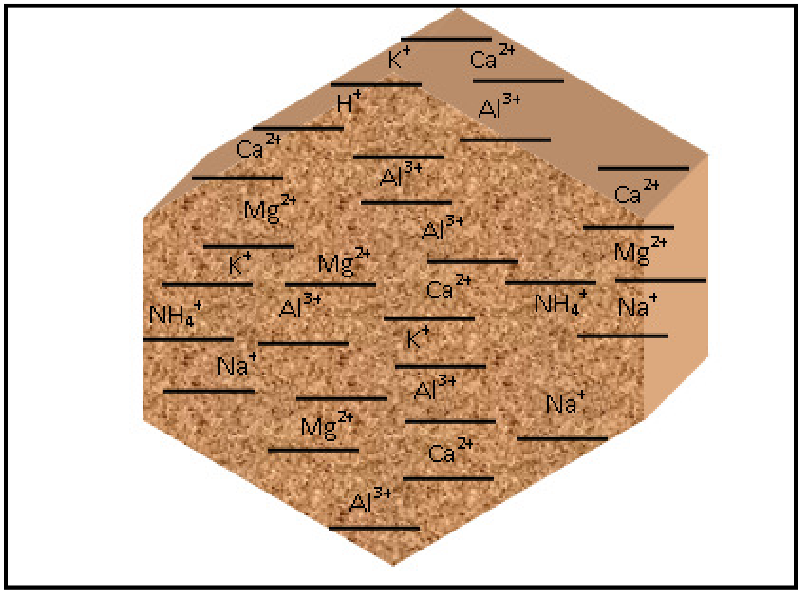


Figure 2.13 Schematic of a clay particle with negative charges on the surface attracting various cations

There are numerous methods for determining CEC and many will give quite different results. As given by Rhoades (1982), the following four methods can be used for CEC determination. Summation method: The exchangeable cations are displaced with a saturating salt solution and the CEC is taken as an equivalent sum of exchangeable cations present in the reacted "leachate". Direct Displacement method: In this method the soil is saturated with an index cation and the adsorbed cation and the small amount of solution entrained by soil after centrifuging are displaced directly with another salt solution without further treat of the soil. The saturating cation and anion are then determined in the resulting extract, and their difference is taken as equal to the CEC of the soil.

Displacement after washing method: After the exchange sites have been saturated with an index cation in the above type of process, the soil can be washed free of excess saturating salt. The amount of index cation adsorbed by the soil can then be displaced and determined.

Radioactive tracer method: In this method, after saturating the soil CEC with a known index cation, the saturating solution can be diluted and labeled with a radioactive isotope of the saturating cation. The concentration of the index cation in the solution is then determined, and the distribution of the isotope (and hence of the total cation) between the two phases is given by measuring the radiation in the solution and the soil plus solution. There is a significant variation in the results obtained by the above four 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 beget higher CEC. Different types of clays have different CECs. Stewart and Hossner (2001) reported unusually high cation exchange capacity (CEC) values relative to clay content for lignite overburden and mine soils. On an average, the CEC values are found to be greater than 100meq/gm. A comparison of methods for particle-size distribution suggests that the major reason lignite overburden samples have CEC to percent clay ratios greater than one is due to incomplete dispersion of aggregates of clay minerals or shale fragments. Another important factor influencing the CEC to percent clay ratio was the presence of organic materials in the samples. Lignite may make a significant contribution to CEC in overburden materials.

2.3.5 Specific Surface Area (SSA)

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. This phenomenon is explained by Campbell (2005) with a simple thought experiment in which a 1 cm^3 cube with a density 1 gm/cm^3 is considered. This cube has a specific surface area of 6 cm^2/g. Now, if this cube is divided into smaller cubes of 1 mm on the side, the resulting 1000 cubes would have the same mass of material, but its specific surface area will be 60 cm^2/g, similarly if the cube were to be divided into 10^{12} cubes of 1 um on a side, the surface area would be 6×10^4 cm^2/g. Hence, it could be understood that within the same mass, presence of smaller particles will result in higher specific surface area. It should be noted here that a soil with high specific surface area has high water holding capacity and greater swell potential.

Various approaches have been used to measure specific surface area, including adsorption of nitrogen and other gases on the soil (Yukselen and Kaya, 2006). The most commonly used method uses the adsorption of ethylene glycol monoethyl ether (EGME) (Carter et al. 1986). This involves saturating prepared soil samples, equilibrating them in a vacuum over a $CaCl_2$-EGME solvate, and weighing to find the point when equilibrium is reached. The specific surface is then determined from the mass of retained EGME in comparison to the amount retained by pure Montmorillonite clay, which is assumed to have a surface area of 810 m^2/gm (Carter et al. 1986). The measurement typically takes around two days to complete. Soil is typically in a hydrated state, and surface area measurements should apply to that state. It would therefore be ideal if water could be used as the probe to determine the specific surface area.

Quirk (1955) reviewed such measurements and concluded that water clusters around cation sites and can therefore lead to errors in the measurements. Recent work which uses more modern methods for measuring the energy state of the water in the soil, have shown promise as simple methods for determining specific surface of soil samples. A comprehensive evaluation of the EGME method for geotechnical usage was done by Cerato and Lutenegger (2002). They concluded that the method is applicable to a wide range of mineralogies and is capable of determining specific surface area ranging from 15 to 800 m^2/g. They also indicated that the test procedure is repeatable and provides reliable results.

2.3.6 Total Potassium (TP)

Potassium is an element which can be used to detect the presence of the mineral Illite. 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. Potassium is the inter layer cation in the clay mineral Illite and Illite is the only clay mineral to have potassium in its structure (Mitchell & Soga, 2003). Hence, measuring the amount of potassium ion in the soil gives a direct indication of the presence of the mineral Illite.

The test procedure formulated by Knudsen et al. (1984) was followed to obtain the amount of total potassium present in the soil. The method involves a double acid digestion technique developed by Jackson (1958) which uses two acids (Hydrofluoric acid and Perchloric acid) to break the mineral structure of the soil and extract the potassium ions from the structure. Once potassium is extracted, its concentration in the solution can be obtained with the help of a spectrophotometer or any other suitable device. It should be noted here that the potassium measurement directly provides the percent Illite clay mineral in a given soil since potassium is solely contributed by the Illite mineral.

2.4 Soil stabilization and problems

Soil stabilization is the process of improving engineering behavior of a soil by changing one or more properties of the soil. In essence, it is the alteration or preservation of one or more soil properties to improve the engineering characteristics and performance of a soil. The main properties that may be required to be altered by stabilization are:

- Shear strength – ability to resist shear stresses developed as a result of traffic loading

- Modulus (stiffness) – ability to respond elastically and minimize permanent deformation when subjected to traffic loading
- Resistance to moisture – the ability to resist the absorption of water, thus maintaining shear strength and modulus, and decreasing volumetric swell
- Stability – the ability to maintain its physical volume and mass when subjected to load or moisture
- Durability – the ability to maintain material and engineering properties when exposed to environmental conditions such as moisture and temperature changes.

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 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.

2.4.1 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, pozzalons, 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.

2.4.2 Chemical stabilization

Chemical stabilization can be brought about in three ways:

- Bonding the soil particles together: In this soils are stabilized by cementing the particles together so that the effect of water on the structure is lessened.
- By waterproofing: In this the soil moisture content is maintained at low level at which it has adequate strength for the intended purpose.
- By a combination of both waterproofing and bonding

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. Coating particles, binding particles together, and formation of new compounds are the main mechanisms that can occur when using an additive. 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 or base treatment and the additive mechanism, composition, and reaction time must all be considered when selecting the best additive for a specific application. A brief description of the three most common additives used in stabilization is presented next:

2.4.2.1 Lime

Lime is formed by the decomposition of limestone at elevated temperatures. When lime is combined with water and the soluble silica and alumina present in clay, a chemical reaction occurs, resulting in the formation of new compounds. When combined with water, its primary function is alteration of particle structure and increased resistance to shrink-swell and moisture susceptibility. A secondary result is binding of particles (when combined with clay) and strength gain. Since alteration of particle structure occurs slowly, depending upon the type of clay present, a mellowing period from 1 to 4 days is allowed to obtain a homogeneous, friable mixture.

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 civil structures (Sherwood, 1995).

Many researchers have used lime as a stabilizer with appreciable amount of success. 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).

When lime and water are added to a clay soil, the following reactions begin to occur almost immediately:

Drying: If quicklime is used, it immediately hydrates (i.e., chemically combines with water) and releases heat. Soils are dried, because water present in the soil participates in this reaction, and because the heat generated can evaporate additional moisture. The hydrated lime produced by these initial reactions will subsequently react with clay particles (discussed below). These subsequent reactions will slowly produce additional drying because they reduce the soil's moisture holding capacity. If hydrated lime or hydrated lime slurry is used instead of quicklime, drying occurs only through the chemical changes in the soil that reduce its capacity to hold water and increase its stability.

Modification: After initial mixing, the calcium ions (Ca^{++}) from hydrated lime migrate to the surface of the clay particles and displace water and other ions. The soil becomes friable and granular, making it easier to work and compact. At this stage the Plasticity Index of the soil decreases dramatically, as does its tendency to swell and shrink. The process, which is called “flocculation and agglomeration," generally occurs in a matter of hours.

Stabilization: When adequate quantities of lime and water are added, the pH of the soil quickly increases to above 10.5, which enables the clay particles to break down. Determining the amount of lime necessary is part of the design process and is approximated by tests such as the Eades and Grim test (ASTM D6276). Silica and alumina are released and react with calcium from the lime to form calcium-silicate-hydrates (CSH) and calcium-aluminate-hydrates (CAH). CSH and CAH are cementitious products similar to those formed in Portland cement. They form the matrix that contributes to the strength of lime-stabilized soil layers. As this matrix forms, the soil is transformed from a sandy, granular material to a hard, relatively impermeable layer with significant load bearing capacity. The process begins within hours and can continue for years in a properly designed system. The matrix formed is permanent, durable, and significantly impermeable, producing a structural layer that is both strong and flexible.

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 pozzalonic reaction. This pozzalonic reaction is the formation of calcium silicate hydrates and calcium aluminate 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.

Little (1999) developed a protocol for lime mixture design based on the following steps:

- Select a soil or aggregate that is mineralogically reactive with lime,
- Establish optimum lime content based on pH testing and compressive strength development (accounting for the effects of moisture-density relationships),
- Evaluate resistance to moisture-induced damage through a capillary suction test in which the surface dielectric value of the cured, lime-treated sample is measured.

2.4.2.2 Cement

Hydraulic cement is a product manufactured to meet a variety of performance criteria by controlling the relative proportions of calcium, silica, alumina, and iron compounds. When combined with water, hydration occurs, resulting in the formation of new compounds, most of which have strength-producing properties. When mixed with soil or base, particles become bound together and the mixture increases in strength and moisture resistance. Depending on the composition of the cement and the soil mineralogy, a chemical reaction can occur between calcium hydroxide and the soluble silica and alumina present in clay, resulting in alteration of particle structure and increased resistance to shrink-swell. Approximately two hours after the soil-cement mixture is exposed to moisture, the soil particles are bound together and compaction must be complete. Additional handling of the treated material will break the bonds that have been established. Strength gain can also continue for several days.

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. These materials are used in pavement base, subbase, and subgrade construction (Little et al., 2000). It is generally more effective and economical to use it with granular soils due to the ease of pulverization and mixing and the 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).

Puppala et al. (2004) studied the effectiveness of sulfate resistant cement stabilizers Types I/II and V in providing better treatment of sulfate rich soils. Experiments were designed and conducted on both control and cement treated sulfate soils to investigate compaction relationships, Atterberg limits, linear shrinkage and free swell strain potentials, unconfined compressive strength (UCS), and low strain shear modulus properties. Sulfate resistant cement stabilizers of Types I/II and V were used and following tests were performed after curing: UCS (ASTM-D 2166), resonant column tests (ASTM-D 4015-92), free swell tests (ASTM-D 4546) with a little modification of using low seating pressures, and linear shrinkage bar tests (TEX-107-E).

Test results indicated significant improvement of soil properties by both cement Types I/II and V and all sulfate rich soils showed similar stabilization trends. Treated soil samples compacted at wet of optimum moisture content yielded higher strength and less swell properties than those compacted at optimum moisture content. This was attributed to more moisture presence in the compacted soils at wet of optimum condition, which facilitates stronger chemical reactions, particularly hydration related reactions between cement stabilizers and soils. An increase in cement content and curing period enhanced soil properties. Both free swell and Atterberg limits reach to zero magnitudes at 5% dosage and 14-day curing. Both low and high sulfate-resistant cement types provided statistically similar and significant improvements to soil properties.

Rollings et al. (1999) examined a project in Georgia that involved a cement-stabilized, sand-based course material that was mixed off-site at the sand borrow pit. Sulfate-induced heave was noted within six months after construction. A preliminary investigation provided no definitive answers as to why the base course heaved. Sulfur was not present in the cement used or in the sand. Closer inspection showed that the mixing water used at the off-site mixing plant contained over 10% sulfur, and that the water was a major contributor of calcium. When the cement was added, the pH increased to about 12 and the alumina and silica in the soil became soluble, leading to the formation of ettringite.

2.4.2.3 Fly Ash

Fly ash is a by-product of coal combustion and its components vary depending upon the specific coal combustion process. Class F is a pozzalon that often requires an activator such as lime or cement. Class C is a combination of a pozzalon and self-setting material. When combined with water, a cementitious reaction occurs, which results in binding of particles together. Depending on the chemical composition, alteration of particle structure and increased resistance to shrink-swell and moisture susceptibility can occur. The reactions prompted by fly ash occur more slowly than cement but more rapidly than lime.

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, and in the case of 20% fly ash addition, the CBR can be increased up to 75%. Tests results show that fly ash increases the compacted dry density and reduces the optimum moisture content. 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). Çokça (2001) found that plasticity index and swell potential decrease with increasing fly ash contents. The fly ash addition rates greater than 20% are comparable to lime addition rates of 8% for reducing plasticity and ultimately swell potential in the example soil.

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). Table 2.2 compares the process, effects and applicable soil type of previously mentioned stabilizing agents.

2.4.2.4 Secondary stabilizers

These are the stabilizers which are not effective when used solely, but are effective when used with another primary stabilizing agent such as lime or cement. Examples of these stabilizers include blast furnace slag, cement kiln dust, lime kiln dust and others.

Poh et al. (2006) has used basic oxygen steel (BOS) slag fines to stabilize English China clay and Mercia mudstone. They found that by using BOS slag fines the strength of the soils can be improved considerably. The results showed that at least 15 to 20 % should be added to see any considerable increase in the strength properties of soils. They have also found that longer curing times are required to attain that increase in strength. MacKay and Emery (1994) gave different case histories documenting the use of various combinations of cements, fly ash, and byproduct kiln dusts, since the mid-1970s to stabilize and solidify a wide range of contaminated materials. Such materials include PCB-contaminated granular road base, steel industry sludge, contaminated lake-bottom sediments, rotary kiln slag from a secondary lead smelter (acid battery reclaimer), and a very wet former fly ash fill site.

Stabilization processes developed have enabled treated materials to satisfy environmental and engineering requirements. The effect of using pozzalonic material as a new stabilizing agent on shear strength and swelling properties of unsaturated clayey soil was studied by Attom et.al (2000). They have used different percentages of pozzalonic material by dry weight of the soils. It was found that the increase in pozzalonic material would decrease the plasticity index and increase the unconfined compressive strength of the soils used. Furthermore, the addition of 10% of pozzalonic material by dry weight of the soil could eliminate both the swelling pressure and the swell potential of the used soils.

Puppala et al. (2003) evaluated the following four types of stabilizers to enhance the strength and reduce free swell and shrinkage strain potentials of soft, expansive and sulfate-rich soils: sulfate-resistant cement, lime mixed with fibers, ground granulated blast furnace slag (GGBFS) and Class F fly ash. Sulfate-resistant cement provided the most effective treatment. Possible mechanisms for these enhancements in soil properties were ion exchange, flocculation, cementation and pozzalonic reactions. The combined lime and fibers stabilization method provided the next best effective treatment. They enhanced UCS and reduced PI, swell and shrinkage strains. The GGBFS stabilizer provided the third best performance. It reduced the swell, shrinkage and plasticity characteristics while increasing the UCS values. The GGBFS- treated soils exhibited less improvement in strength, and swell and shrinkage behaviors compared to the cement and lime plus fiber treatment methods. The Class F fly ash treatment provided low-to-moderate strength improvements that could be attributed to the low amounts of calcium present in this type of fly ash. On the other hand, the fly ash stabilization method was more cost-effective than the other methods.

Table 2.1 – 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.

Al-Rawas et al. (2005) evaluated the effect of lime, cement, and combinations of lime and cement treatment on the swelling potential of expansive soils. The liquid limit of all treated samples except for samples treated with 5% lime plus cement showed an initial increase at the addition of 3% stabilizer, followed by gradual decrease. On the other hand, the samples treated with combinations of lime, cement exhibited an initial reduction at 3% lime + 3% cement, and 5% lime + 3% cement followed by a general increase with further additions. All stabilizers caused a reduction in both swell pressure and swell percent. With the addition of 6% lime, both the swell percent and swell pressure were reduced to zero.

Although chemical stabilization has proven successful in increasing the strength of the natural expansive soils by twenty to fifty times, and is widely used throughout Texas, situations arise where above mentioned approaches cannot be used. For example, chemical stabilization cannot be used when the temperature is below 40^{o}F and in cases there are not enough time for curing before traffic is routed back (Hopkins et al. ,2005).

Presently there are many stabilizers in the market, but not all the stabilizers give the same effect on all soils. A particular stabilizer is good for a particular kind of soil. Now, selecting the stabilizer is a deciding factor as, the effect of the stabilization is a long-term issue and it is not possible to check the correctness of the selection until there is some problem in the future which might be after a year or longer, and it is not always economical or sometimes possible to re-stabilize using a different stabilizer.

Burroughs (2006) has investigated the influence of stabilizers and soil properties on the strength of stabilized, compacted earth. 219 tests were made on 104 different soils varying the soil properties and the stabilizers. The results of the study stress the importance of selecting a soil whose characteristics are favorably predisposed to stabilization in order to attain satisfactory strengths of compacted earth.

2.4.3 Review of current stabilization procedures

A review of the literature has been carried out in order to outline current state of practice and stabilization guidelines followed by several agencies like Texas Department of Transportation (TxDOT), 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 following sections. 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 also summarized.

The goal for any treatment of bases and subgrades is oriented to modify some properties of the unbound layers. Among the reasons for treatment the most common include:

- Reduce shrink/swell of expansive soils.
- Increase strength to provide long-term support for the pavement structure.
- Reduce pavement thickness.
- Reduce moisture susceptibility and migration.
- Utilize local materials.
- Bind salvaged materials used on pavement rehabilitation projects.
- Provide a working platform for construction of subsequent layers by drying out wet areas and/or temporarily increasing strength properties.

Many variables are considered to obtain effective soil stabilization, especially when the objective is to provide a long-term effect on the treated materials. They can be divided into the following three steps: (1) Soil Exploration, Material Sampling, Soil Classification and Acceptance Testing, (2) Additive Selection, and (3) Mix Design. The flowchart shown in Figure 2.14 provides a simplified illustration of the steps required for successful subgrade treatment and Figure 2.15 shows the same flowchart, but for base materials.

2.4.3.1 Soil exploration

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.

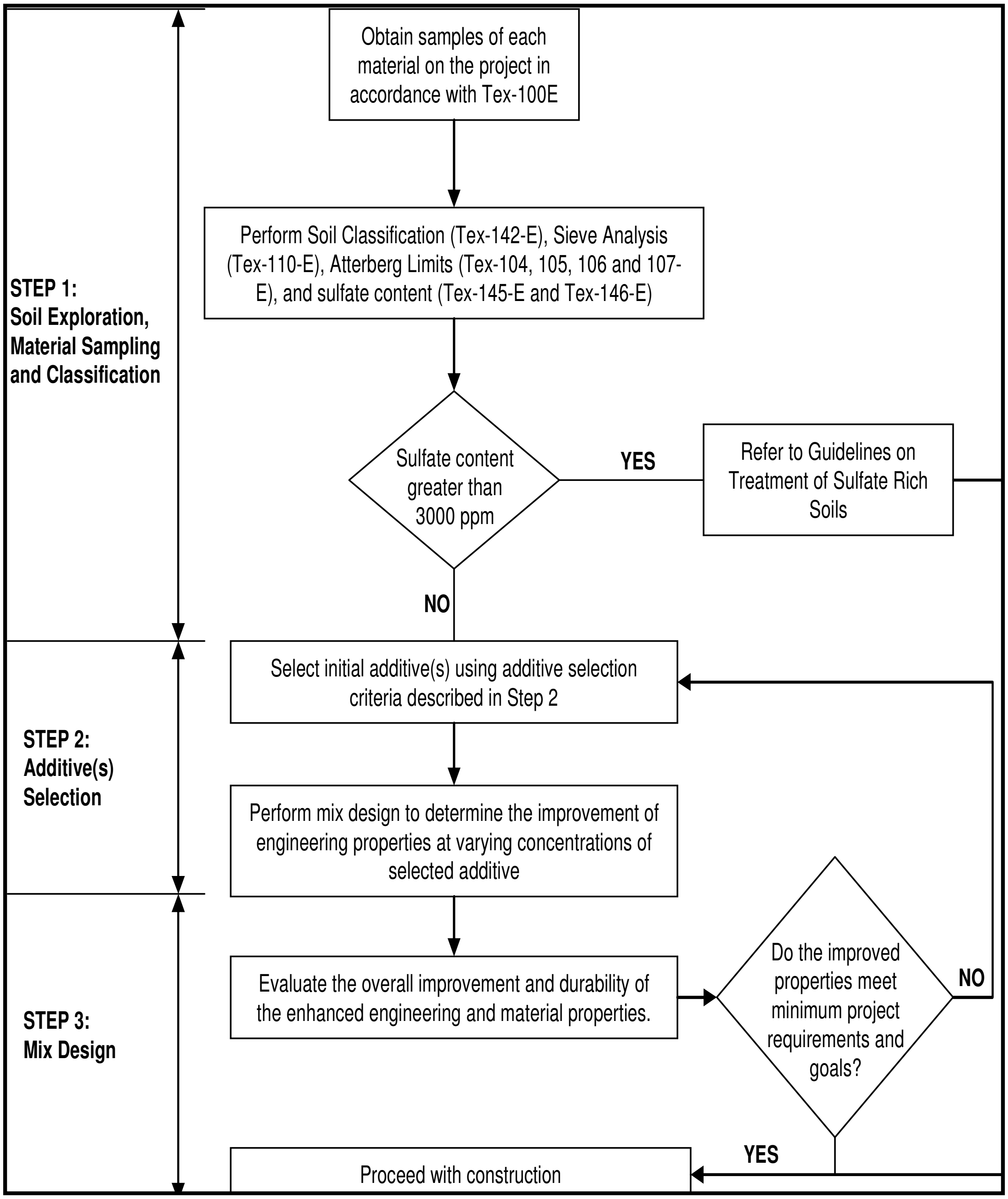


Figure 2.14 Flowchart for subgrade soil treatment (from TxDOT Guidelines)

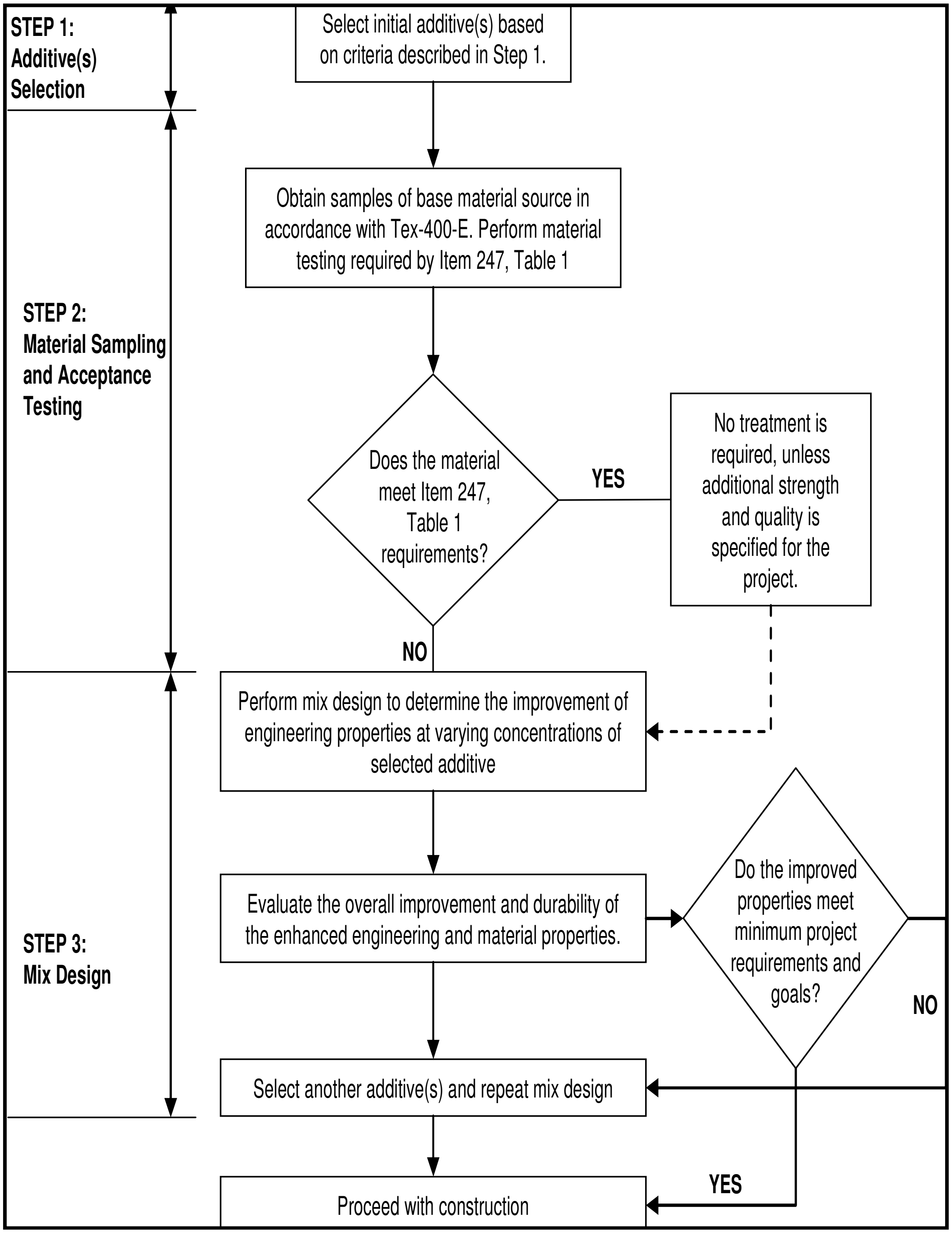


Figure 2.15 Flowchart for base soil treatment (from TxDOT Guidelines)

2.4.3.2 Additive selection

The selection of an 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
- ✓ Environnemental conditions (drainage, water table, etc.)
- ✓ Engineering economics (cost savings vs. benefit)

TxDOT stabilization guidelines provide a quick reference to the selection of additives. The selection of the proper additives is summarized in Figure 2.16 for sub-grades and Figure 2.17 for bases. 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.

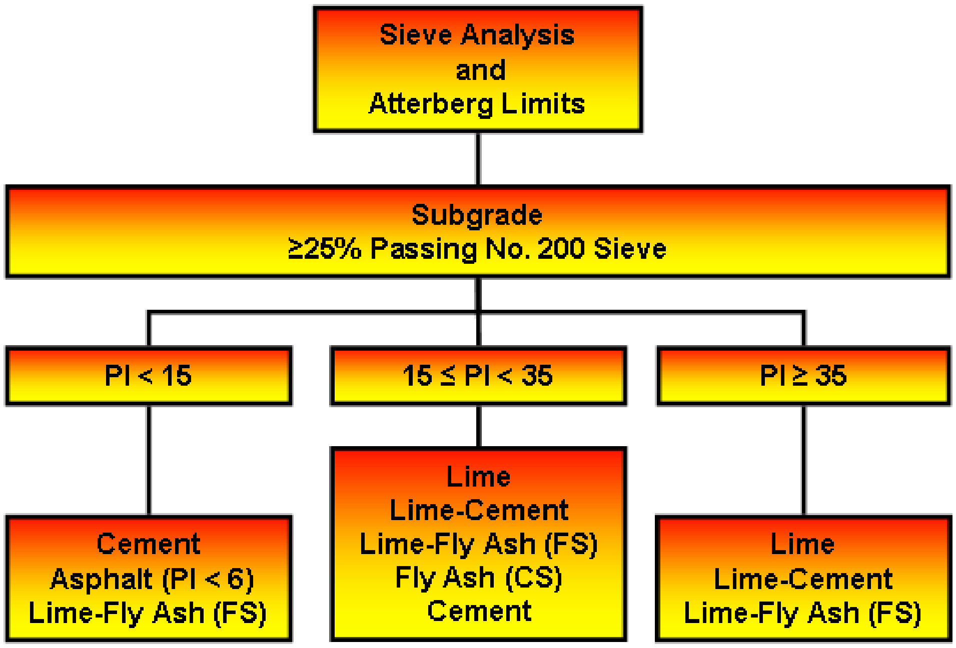


Figure 2.16 Additive selection criteria for subgrade material using soil classification

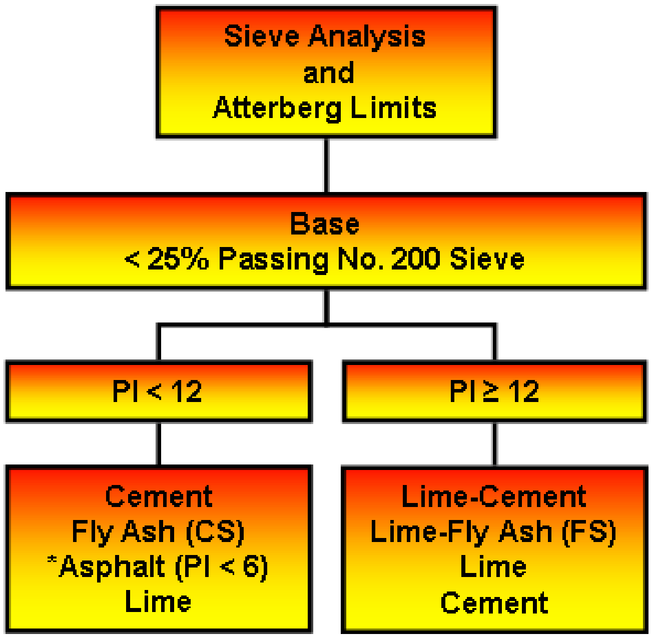


Figure 2.17 Additive selection criteria for base material using soil classification

According to Army and Air force (AAF) stabilization guides the factors that must be considered for the selection of a stabilizer 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 2.18 and Table 2.2. The soil gradation triangle in

Figure 2.18 is based upon the soil grain size and Atterberg limits characteristics, and the triangle is divided into areas of soils with similar grain size. The selection process continues with Table 2.2 which indicates for each area shown in Figure 2.18 candidate stabilizers and restrictions based on grain size and/or PI.

2.4.3.3 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.

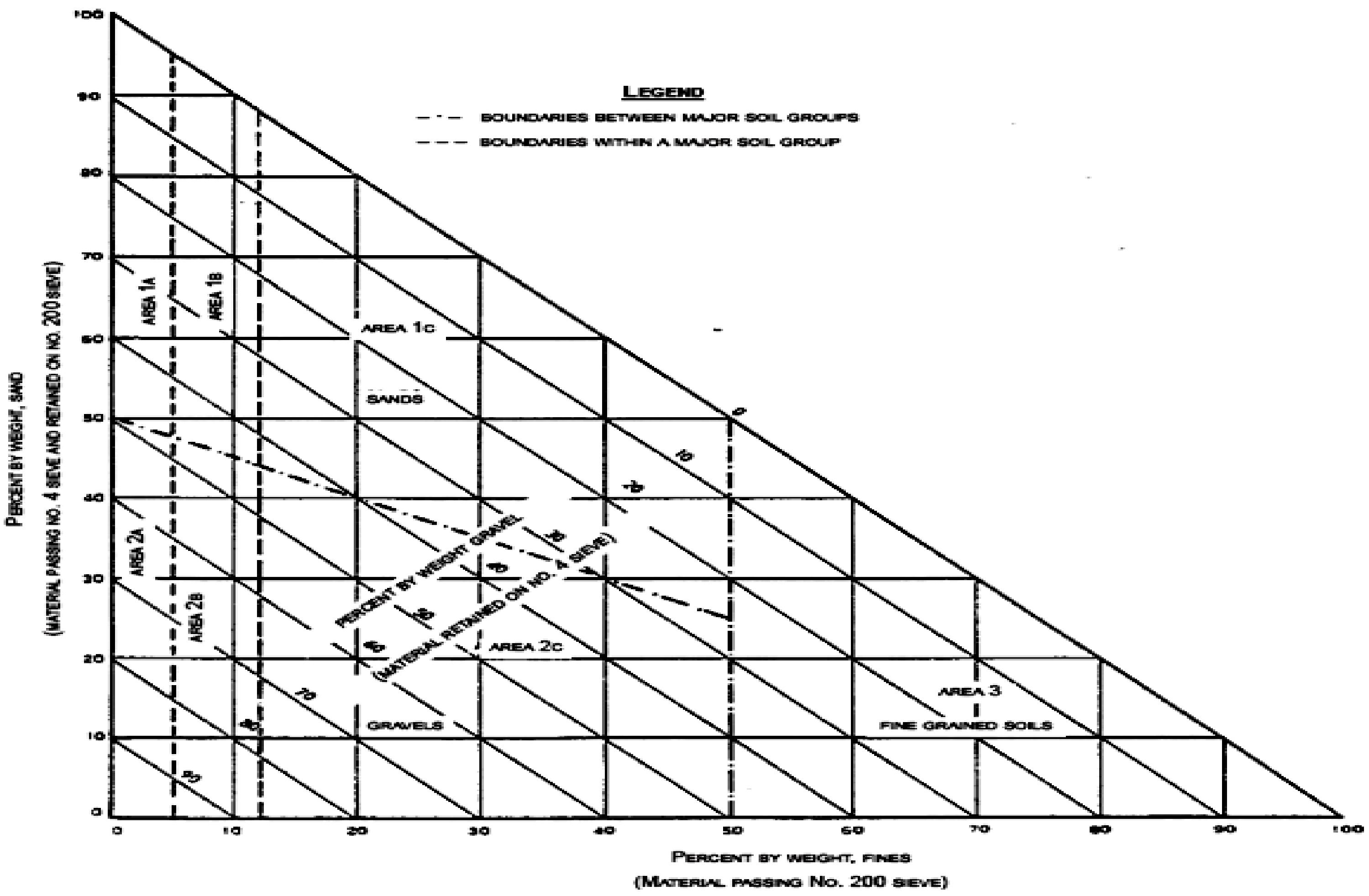


Figure 2.18 Gradation triangle to select a commercial stabilizing agent
(Army TM 5-822-14, Air Force AFJMAN 32-1019)

Table 2.2 Army and Air Force stabilization guidelines 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 ≤ 20 + [(50-percent passing No. 200 sieve) / 4]

2.4.3.4 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 extreme environmental cracking.

According to AAF guidelines the cement content is initially estimated based on the soil classification (see Table 2.4). Using this cement content, maximum dry density and optimum water content of the soil-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.

Unconfined compressive strength (UCS) is the most widely referenced property of soil cement. UCS serves as a criterion for determining the minimum cement content requirements. Typical ranges of UCS after 7 and 28 days of curing for soaked soil-cement mixtures are presented in Table 2.5, classified by several soil groups (American Concrete Institute, ACI 230.1R-90).

Table 2.3 Soil-cement strength requirements as per TxDOT Item 246 specifications

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

Table 2.4 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

Table 2.5 Ranges of UCS for soil-cement (ACI 230.1R-90)

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

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), is fully described in ASTM D 6276 procedures 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 pozzalonic 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 base and 50 psi for soils.

In the AAF guidelines the preferred method for determining initial design lime content is the pH test or “Eades-Grim” test, same as used in current TxDOT specifications. The lowest lime content at which a pH of about 12.4 (the pH of free lime) is obtained is the initial design lime content. An alternate method of determining initial design lime content is by the PI wet method (AASHTO T-220), as shown in Figure 2.19. 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, a higher lime content may be selected and the mix design is evaluated again.

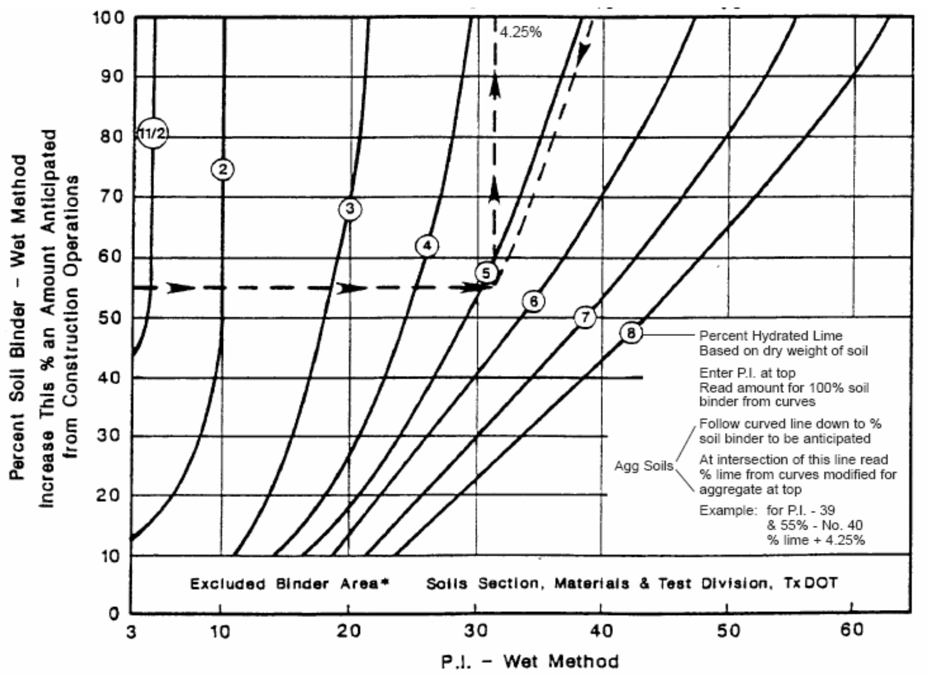


Figure 2.19 PI wet method to calculate amount of lime for stabilization (Tex-121 E)

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 2.6.

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 above.

Table 2.6 Summary of soil-lime mixture design procedures

Mixture Design Procedure	Summary of Methods	Mixture Design Criteria
Eades and Grim (Figure 2.10)	Based on pH	Design lime content is lime required to insure a pH of 12.4
Thompson (Figure 2.11)	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

According to AAF guidelines design with LFA 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. The optimum fines content is referred as the quantity of matrix required for maximum dry density of the total mixture. For LF mixtures it is recommended that the quantity of this matrix should be approximately 2 percent above the optimum fines content. 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 from Table 2.3 is the design LF content.

2.4.4 Durability related issues in soil stabilization

An important objective of any stabilization design procedure is to address the permanency of chemical stabilizer, i.e. the additive to hold the soil particles intact 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 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 60^{o}C, 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.

Rao et al. (2001) conducted a study to assess the long-term performance of stabilized black cotton soils. These black cotton soils are stabilized with waste materials such as wood ash and organic matter (known as ash modified soils) and also with lime. This study is conducted primarily to understand the relative effectiveness of ash modified soils and lime treated black cotton soils. They observed that the ash modified soil became more porous due to cyclic wetting and drying and consequently collapsed significantly. They also found that due to cyclic wetting and drying the beneficial effects of lime treatment were partially lost.

Hoyos et al. (2005) at UTA performed a series of wet and dry cyclic tests (see Figure 2.20) on different types of chemically-treated sulfate soils to evaluate the strength, stiffness and volume change property variations with respect to these cycles. Khattab et al. (2007) conducted both wetting drying studies and leaching studies on FoCa bentonite (FoCa represents the first two letters of the two towns between which this type of soil is excavated: Fourgues and Cahaignes) stabilized with 4% lime. They observed that during wetting/drying processes, the samples that started with initial wetting stage had lower swell potentials than the samples that started with an initial drying stage. They also observed that leaching did not have much influence on the efficiency of the lime treatment.

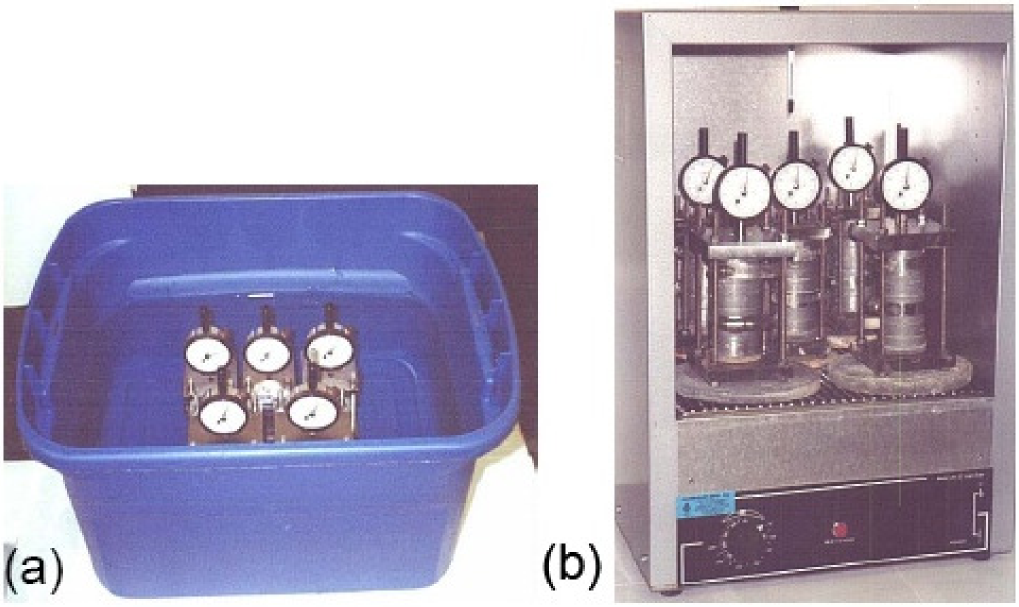


Figure 2.20 Wet (a) and Dry (b) cycles setup used by Hoyos et al. (2005)

The long-term parameters used to measure the effectiveness of base and subgrade stabilization are summarized next. The objective is also to assess the relationship and compare lab and field results. Different methods and tests have been applied to solve this issue.

Some studies have determined that to have reasonable resistance to fatigue cracking damage and prevent strength loss in lime stabilized layers, unconfined compressive strengths should be at least eight times the flexural tensile stress induced by traffic load (Little, 2000). Another conclusion reached from that study indicates that stabilized layers should have reasonable resistance to permanent deformation if the compressive stresses induced by loads are lower than one-half of the unconfined compressive strength. The Tube Suction Test is also presented as an alternative to evaluate moisture sensitivity on stabilized materials. However, more field data is needed to correlate changes in the dielectric value to changes in moisture damage to establish a mixture design and testing protocol.

Other studies have been focused on performing field measurements of vertical compressive stresses at the top of the subgrade and rut depths at the pavement surface on treated soils. Romanoschi et al. (2004) compared the performance of stabilized clayey soils when Portland cement, fly ash and lime were used as stabilizing agents on full scale pavement structures. Initially, UCS tests were performed on the untreated soils and after that on the stabilized material at optimum stabilizer and moisture contents. Tests were performed at several ages ranging from 7 to 150 days. UCS values of the untreated soil were around 207 kPa, while for the stabilized materials all UCS results were above 689.5 kPa; softer materials corresponded to stabilization with fly ash and stronger when cement was employed as additive.

Marshall and Frank (2007) have tried to improve the durability of cement stabilized expansive subgrade by pretreatment with lime. They found that lime treatment, increased resistance to degradation of strength upon saturation and cyclical wet/dry strength testing and maintaining plasticity reduction after a year of exposure to in-place conditions.

In colder regions where the temperature goes below zero the soil stabilization has to be tested for freezing and thawing. 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 560 methods.

Thompson & Dempsey (1976) had developed a suggested procedure to evaluate the freeze-thaw durability of the stabilized materials.

2.4.5 Leachability related issues in soil stabilization

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).

Thompson (1966) observed that soil leaching has a direct influence on the properties such as soil pH, percentage base saturation and calcium/magnesium ratios and is directly related to the permeability of the soil. He stated that soil-lime reactivity decreases in areas of high permeability. In soils with very low permeability i.e. fine grained soils the leaching effects are minimized and hence maintaining the calcium/magnesium ratios and higher soil pH.

From the study conducted by Frenkel et al. (1983) it was observed that the mineral leaching that occurred was dependent on the soil to water ratio, exchangeable cations and the type of dominant mineral when leached with calcium, magnesium and sodium chloride. Suarez and Frenkell (1981) observed that the amount of calcium and magnesium leached due to moisture infiltration is depending on the amount of the exchangeable sodium. They concluded that the higher the exchangeable sodium in the soil the higher the calcium and magnesium loss from the system.

Yong et al. (1985) showed that the leaching has an effect on the strength of the soil specimen. They conducted studies on soils specimens that are leached with distilled water to study this effect. They found that the strength of the soil samples decreased when leached with distilled water.

Few studies have been conducted on the leach test of chemically-treated soils to understand the leaching of chemicals from moisture flows (Barenberg, 1970; McCallister, 1990). 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 2.21) 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 2.21 Cyclic wetting-drying setup

2.5 Summary

This chapter reviewed the available literature on expansive soils and their behavioral studies conducted by various researchers. Different methods that have been developed and used to stabilize these problematic soils are also reviewed in detail. Brief reviews of common clay minerals present in the expansive soils are presented along with a summary of clay mineral identification methods that are available in the literature. Some of the chemical properties that can be used to obtain the dominating clay mineral in soils are explained. Then the current stabilization procedures followed by various agencies were summarized along with their limitations. Finally, the issues with respect to the loss of stabilizer and the durability of stabilization methods were covered.

CHAPTER 3

EXPERIMENTAL METHODOLOGY

3.2 Introduction

This research study was conducted in three phases. The first phase consisted of developing a model for the determination of dominating clay mineral in the soils. A total of 20 soils were collected from various Texas Department of Transportation (TxDOT) districts, which represent different parts of Texas. All these soils were expected to have various types of clay mineralogies expected from the state of Texas. Fifteen of the twenty soils collected were used for developing the clay mineralogy prediction model and the remaining five soils were used for validating the developed model. Three chemical properties of soil namely Cation Exchange Capacity (CEC), Specific Surface Area (SSA) and Total Potassium (TP) were used for the determination of the dominating clay mineral. The test procedures followed to obtain these properties are given in this chapter. Reasons for selecting these methods were already explained in Chapter 2.

For the second and third phases, eight soils with distinct mineralogies were selected. Stabilizer mix design (determining the type and amount of the chemical stabilizer) was performed on these soils. The second phase also consisted of testing the stabilized soils for long-term durability investigations, by subjecting the compacted soil specimens to moisture fluctuations simulating both summer and winter seasons in the field conditions. Then in the third phase the stabilized soil specimens were tested for potential leaching of the stabilizer due to moisture infiltration simulating the field infiltration followed by the heavy rain falls.

In this chapter detailed procedures for determining the chemical properties of the soil were explained along with the methodology to use these properties and evaluate the dominant clay mineral in the soils. The equipment used and the test procedure followed to replicate the moisture fluctuations in the field during summer and winter seasons were explained. Finally the equipment used and the test procedure followed to replicate the moisture infiltration in the field during rainfall is also given in this chapter.

Figure 3.1 presents a schematic of the experimental program that was performed to accomplish the above specific objectives.

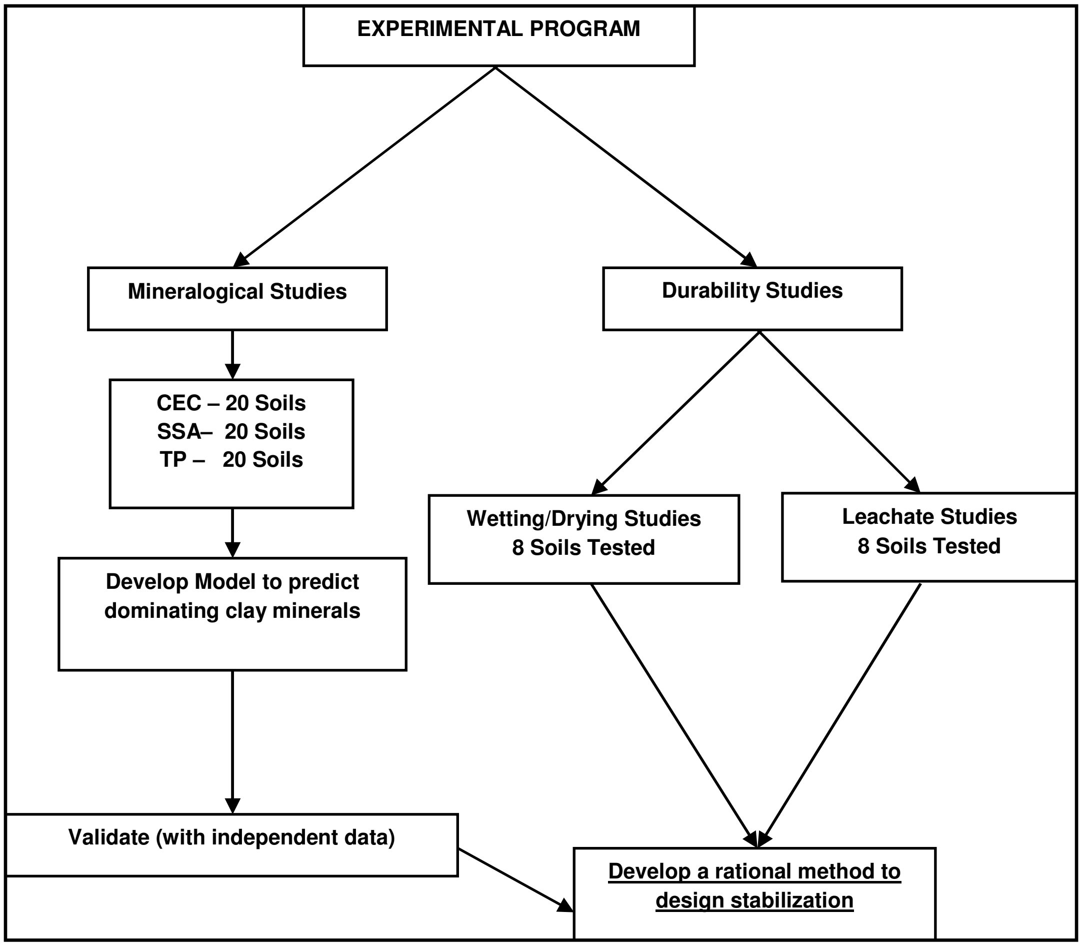


Figure 3.1 Schematic showing the experimental program followed in this research

3.3 Mineralogical studies

An initial X-ray diffraction screening study was conducted on all the soils collected to qualitatively identify the available clay and non-clay minerals in the soils. The test procedure involves, subjecting a powdered sample of soil to an intense X-ray beam and detecting the diffracted beam with the help of a detector. The detector then converts the analog signal into digital data which can be plotted. Using Bragg's law the distances between the planes of the atoms is measured this distance is called d-spacing. These d-spacing are compared with the standard powder diffraction files (PDF) of different minerals. The presence of certain mineral is confirmed if at least 5 to 6 matches are found.

Prior to the X-ray diffraction test, soil specimens were air dried and hand crushed such that most of the material passes through No.200 sieve. Oven drying and pulverizing are not preferred as they may modify the mineralogical structure of the soil sample and some of the peaks may not be observed (Chew et al., 2004). The powdered sample was placed in a sample holder as shown in Figure 3.3 and X-ray diffraction studies were carried out using a CuKα D-500 X-ray Diffractometer (Figure 3.4) with an input voltage of 40 kV and current of 30 mA.

A step scan mode with a step size of 0.03^0 of 2-theta angle and a dwell time of 2s were selected. A typical plot between the 2-theta angle and the intensity counts can be seen in Figure 3.5. Powder diffraction data given by Brown (1961) were used for mineralogical analysis of X-ray diffraction data. Peaks for a particular scan angle and intensity counts, when matched with those of Kaolinite, Illite or Montmorillonite minerals, indicate their presence. The results indicated the presence of the mineral Illite, Montmorillonite and Kaolinite along with quartz mineral. Hence, in the analysis it is assumed that only these minerals were included in the finer fractions of the soils.

Figure 3.2 presents a map of Texas highlighting the regions from which the soils were obtained.

Figure 3.2 Map showing the locations of the different soils that were obtained for mineralogical studies
(Source: http://www.texasira.com/files/15692/texas-map.jpg)

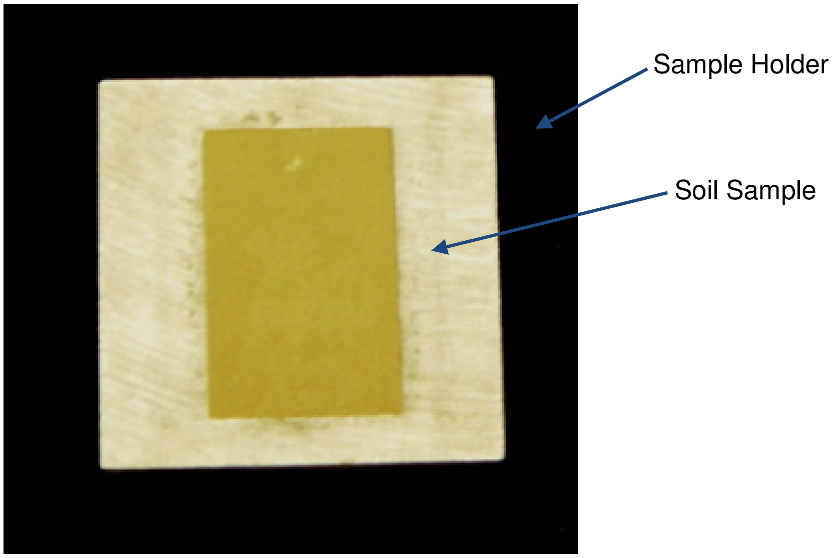


Figure 3.3 Sample holder and the soil sample

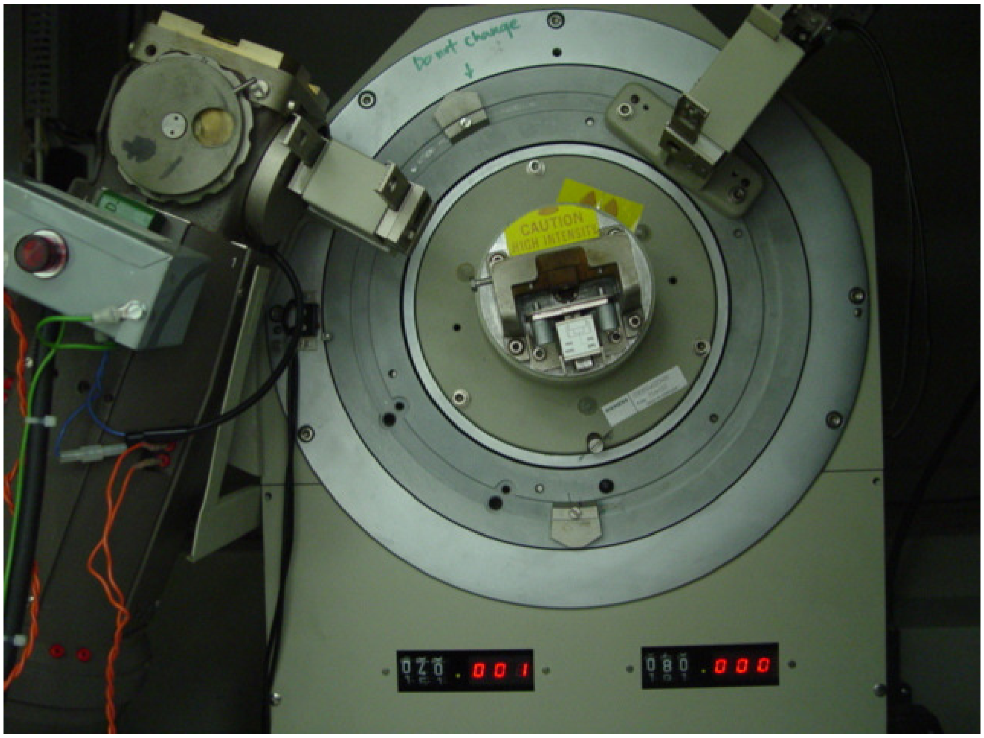
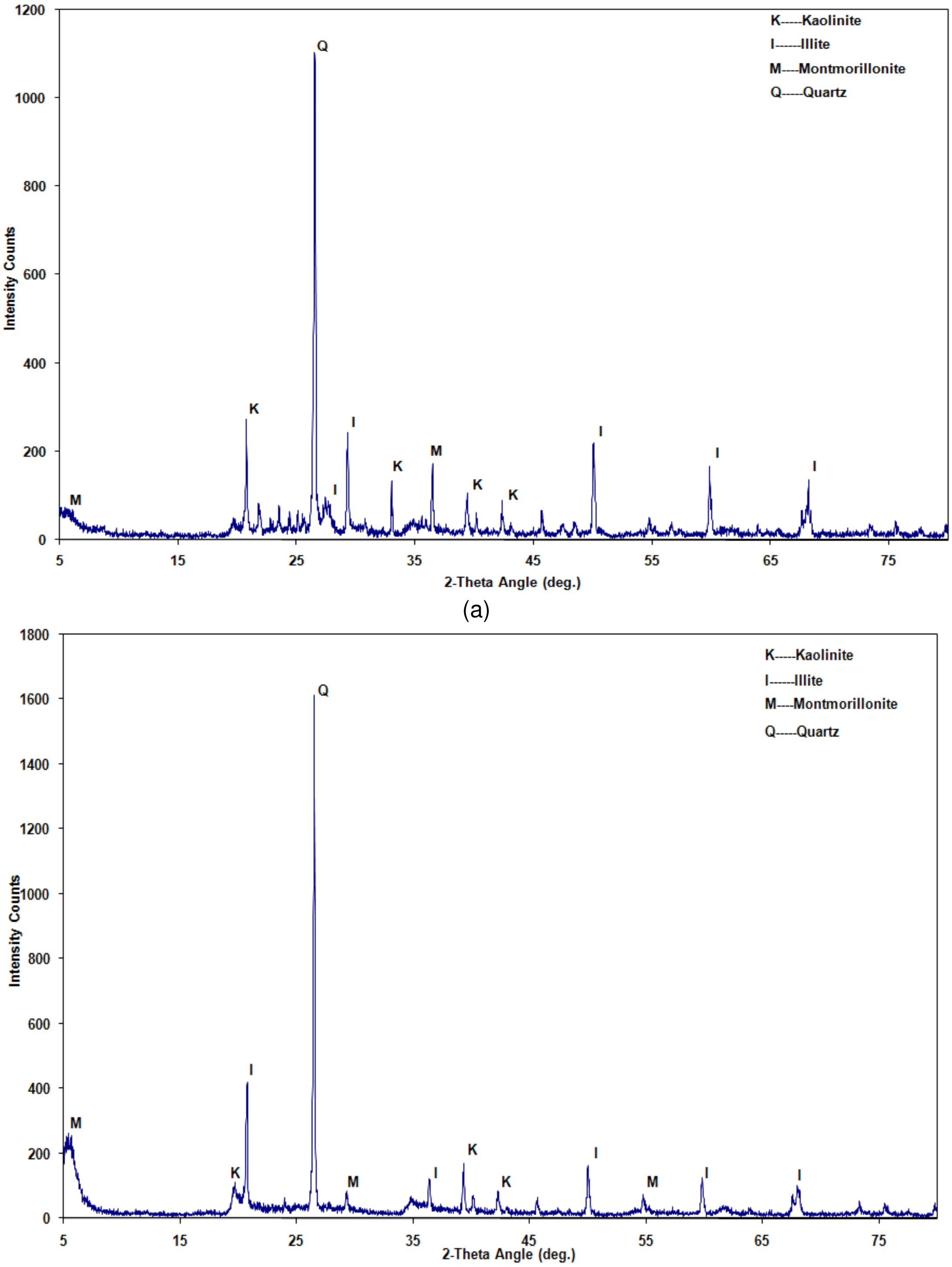


Figure 3.4 D-500 X-ray diffractometer



(a)

(b)

Figure 3.5 Typical XRD peaks of a soil sample (a) El Paso Clay (b) Paris Clay

The following three chemical properties of the soil were used for the determination of the dominating clay mineral:

- Cation exchange capacity (CEC)
- Specific surface area (SSA)
- Total potassium (TP)

3.3.1 Cation exchange capacity

Cation exchange capacity or CEC can be used to determine the mineral composition of a given soil specimen. A soil specimen with a high CEC value indicates a high amount of expansiveness due to the presence of the clay mineral Montmorillonite where as a low CEC indicates the presence of non-expansive clay minerals such as Kaolinite and Illite. CEC of a soil can be defined as the capacity or the ability of the soil to exchange free cations that are available in the exchange locations. One of the earliest methods proposed by Chapman (1965) is the most commonly used method in the field and this method had been selected for the current research. The method involves addition of a saturating solution and then removal of the adsorbed cations using an extracting solution. Procedural steps of this method are presented in Figure 3.6.

The saturating solution used here is ammonium acetate (NH_4OAc) at pH 7. This solution is added to prepared soil specimen (preparation involves treating for organics with 30% hydrogen peroxide (H_2O_2) and set aside for 16 hours after shaking for half hour, to ensure that all the exchange locations are occupied by the ammonium ion (NH_4^+). Then the solution is filtered through a Buchner funnel and washed with 4 different 25 mL additions of NH_4OAc. This step is to bring out all the cations from the soil sample solution that has been replaced by ammonium ion. Excess NH_4OAc was removed by the addition of 8 different 10 mL additions of 2-propanol. Now, all the cation places are replaced by the ammonium ion and excess ammonium was also removed. The CEC of the soil sample can be obtained if we can measure the amount of ammonium ions by taking them out. This was done by washing the sample with 8 different 25 mL additions of 1M potassium chloride (KCl) solution. Though potassium ion (K^+) has similar electro negativity it has higher molecular weight and has the ability to substitute the NH_4^+ ion. The concentration of NH_4^+ in the KCl extract gives the CEC of the soil. The detailed step by step procedure of how the test is conducted is given in the flowchart shown in Figure 3.6 and Figure 3.7.

3.3.2 Specific surface area

Specific surface area or SSA of a soil sample is the total surface area contained in a unit mass of soil. This property of the soil is primarily dependent on the particle size of the soil. Soils with smaller particle size have higher specific surface areas. It should be noted here that a soil with high specific surface area has high water holding capacity and greater swell potential. The most commonly used method in the field of agronomy is the adsorption of ethylene glycol monoethyl ether (EGME) (Carter et al., 1986). This involves saturating prepared soil specimens, equilibrating them in vacuum over a calcium chloride – EGME ($CaCl_2$-EGME) solvate, and weighing to find the point when equilibrium is reached.

Specific surface is then determined from the mass of retained EGME in comparison to the amount retained by pure Montmorillonite clay, which is assumed to have a surface area of 810 m^2/g (Carter et al., 1986). Test procedure typically takes two days to complete. This method was fully evaluated for geotechnical usage by Cerato and Lutenegger (2002) and concluded that the method is applicable to a wide range of mineralogies and is capable of determining specific surface area ranging from 15 to 800 m^2/gm. They also indicated that the procedure is repeatable and gives reliable results. A detailed procedure for the determination of SSA by EGME method that has been followed in the current research is given in Figure 3.8 and Figure 3.9.

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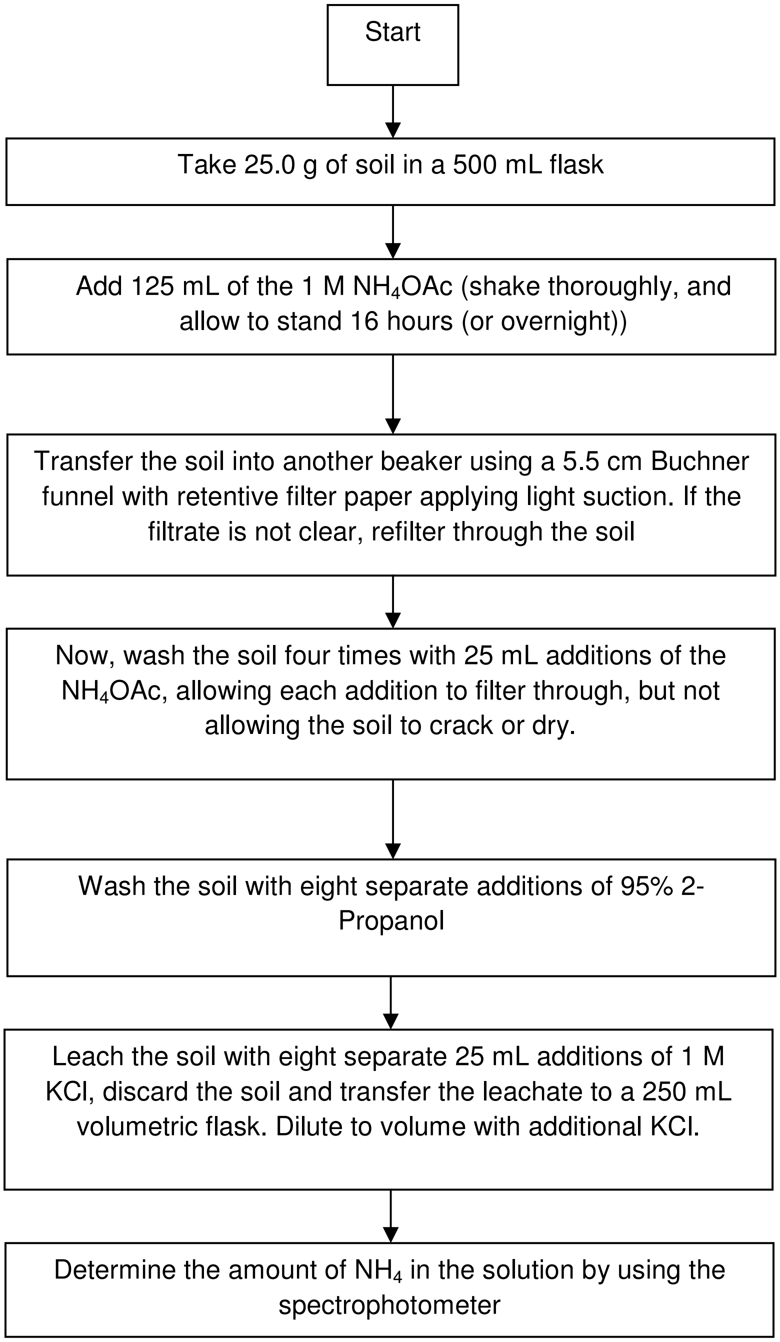


Figure 3.6 Flowchart showing the procedure followed for the determination of Cation Exchange Capacity

Soil sample saturated with NH_4OAc

25 mL 1 M NH_4OAc additions

25 mL Ethanol additions

25 mL 1 M KCl additions

Spectrophotometer

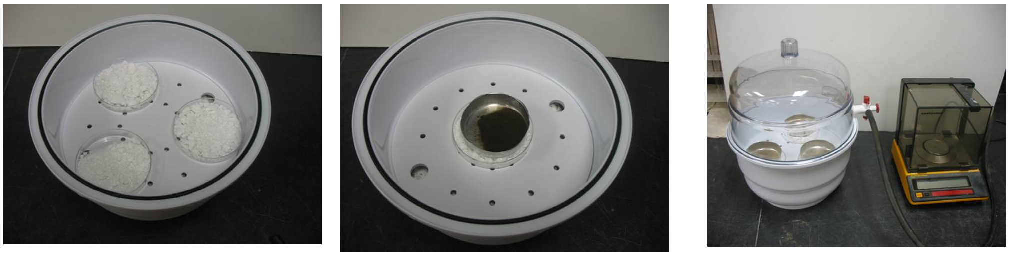
Colorimetric determination of ammonia concentration

Figure 3.7 Photographs of the various steps involved in the determination of CEC

Dried soil sample

Addition of 3 mL of EGME

Desiccator containing $CaCl_2$



Desiccator containing $CaCl_2$-EGME Solvate over $CaCl_2$

EGME mixed soil sample placed in the desiccator placed

Apply vacuum to the desiccator

Figure 3.8 Photographs of the various steps involved in the determination of SSA

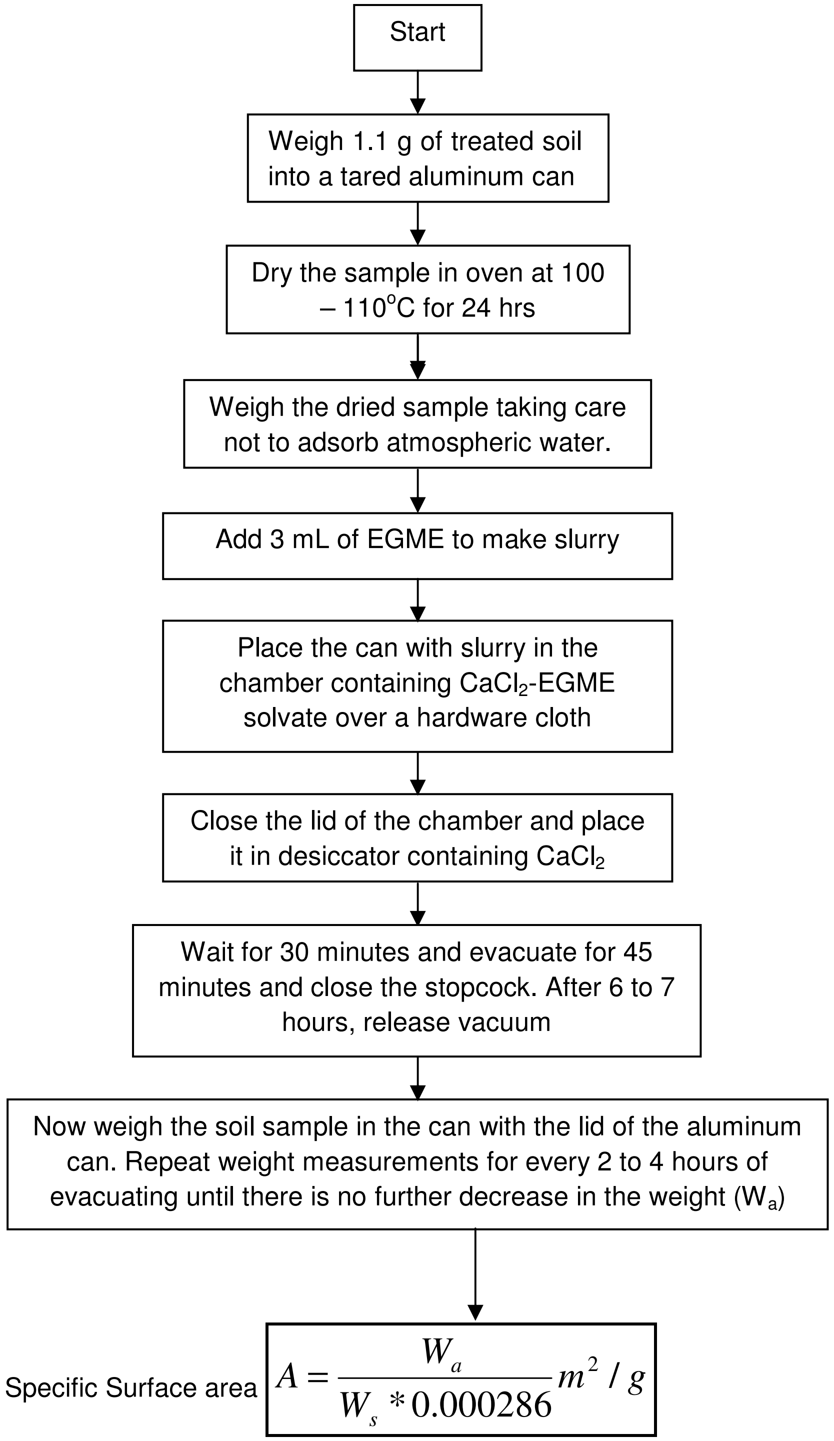


Figure 3.9 Flowchart showing the detailed procedure involved in obtaining the SSA of soil

3.3.3 Total potassium

Potassium is the inter layer cation in the clay mineral Illite and hence Illite is the only clay mineral to have potassium in its structure (Mitchell & Soga, 2003). Hence measuring the amount of potassium ion in the soil gives a direct indication of the presence of the mineral Illite. The test procedure formulated by Knudsen et al. (1984) was followed to obtain the amount of total potassium present in the soil. The method involves a double acid digestion technique developed by Jackson (1958) which uses two acids (Hydrofluoric acid and Perchloric acid) to break the mineral structure of the soil and then extract the potassium ions from the structure. Once the potassium is extracted, its concentration in the solution can be obtained with the help of a spectrophotometer or any other suitable device.

The test starts by taking 0.1 gm of soil in a Teflon digestion vessel. The original method recommends the use of platinum vessels as the hydrofluoric acid used has the ability to dissolve silica and glass is 90% silica. However, the usage of platinum vessel was not possible due to cost constraints hence other possible alternatives were explored and Teflon vessel was found to have resistance to the acids that are being used in the current test procedure (Hydrofluoric acid, Perchloric acid and Hydrochloric acid) and also had high temperature tolerance (200°C). Hence, Teflon vessel was selected.

The test starts with the addition of 5 mL of hydrofluoric acid and 0.5 ml of perchloric acid to 0.1 gm of the soil sample. Hydrofluoric acid dissolves the silicate mineral structure and releases the interlayer cations; perchloric acid is used as an oxidizing agent to oxidize the organic matter in the soil sample. Then the vessel is placed on hot plate and heated to 200°C and then cooled and another addition of HF and $HClO_4$ was made and reheated on the hot plate. Now, the sample is heated until it is dry. The whole process of mixing the acids and reheating was repeated to make sure all the interlayer cations are released from the structure. Finally, 5 ml of 6N HCl and 5 ml of distilled water are added and the solution was transferred into 50 ml volumetric flask. The amount of potassium in this solution is obtained by using a spectrophotometer. Detailed procedural steps followed for the determination of total potassium in this research were outlined in Figure 3.10 and Figure 3.11. The results obtained from all the above explained tests are given in Chapter 4.

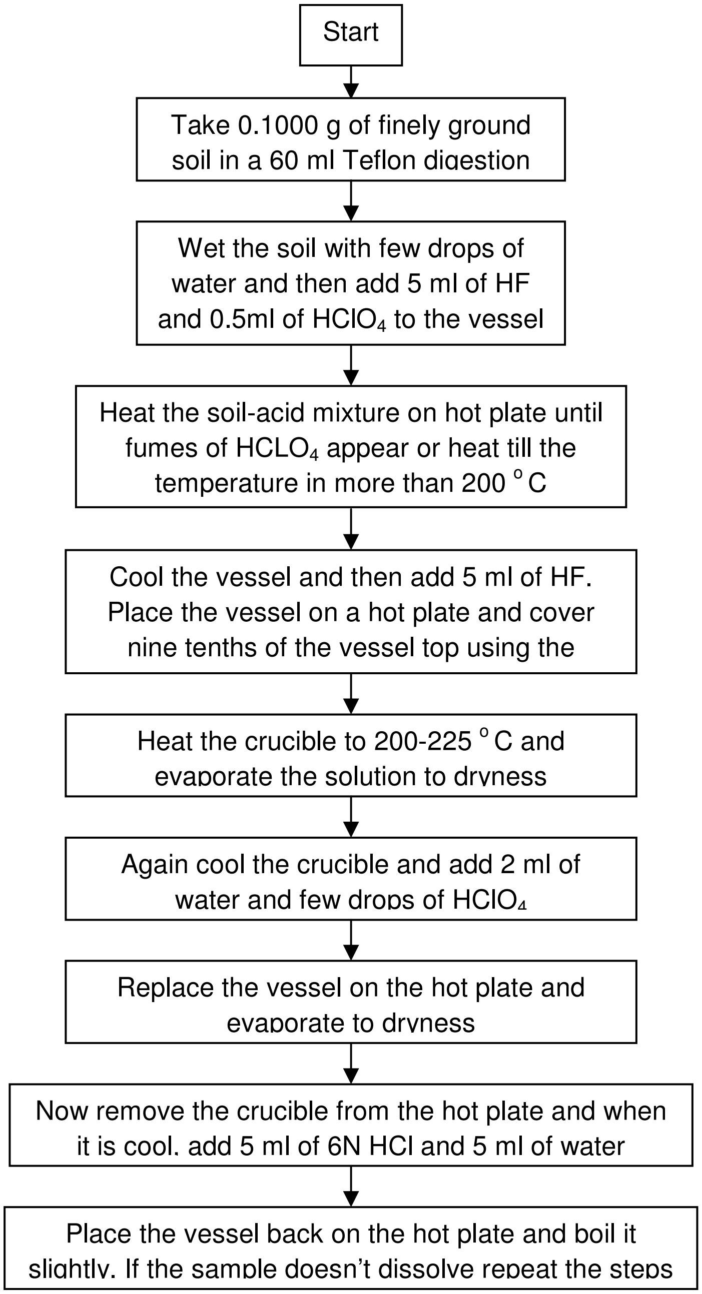
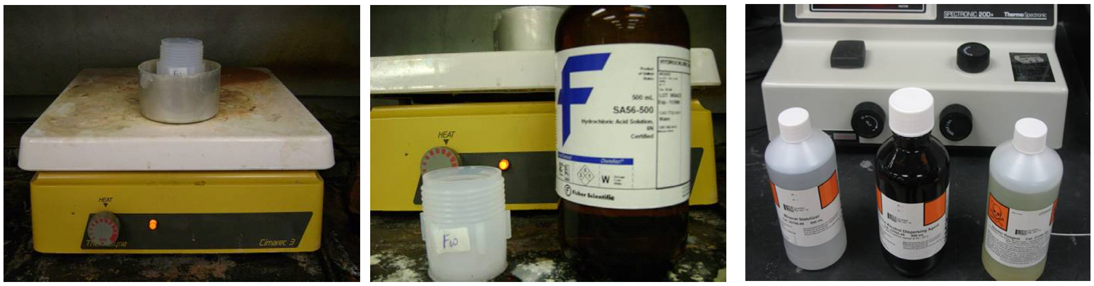


Figure 3.10 Procedure for the determination of total potassium

Weigh 0.1000 gm of the soil sample

Add 5 ml of Hydrofluoric acid

Add Hydrofluoric acid and perchloric acid



Heating the sample on a hot plate

Add 6N HCl

Colorimetric determination of potassium concentration

Figure 3.11 Photographs of the various steps involved in the determination of TP

3.4 Soil selection and preliminary tests

Based on the clay mineralogies of the soils, eight soil types from different locations in the state of Texas were selected to understand the effect of mineralogy on the long-term performance of the chemically treated soils. Preliminary tests were conducted on all these soils to determine their index properties. Test methods such as particle size distribution, Atterberg limits and pH were performed on all soil types and a few test procedure details are given in the following sections. Also test procedure to obtain the maximum dry density and optimum moisture content using standard Proctor compaction test is also explained in detail.

3.4.1 Sample preparation

In order to eliminate the effect of oven drying on the properties of untreated soil, wet preparation method (Tex-101-E) developed by TxDOT was followed to prepare soil samples for determination of Atterberg limits, grain size distribution and pH. The procedure includes soaking of soil sample in tap water for a period of 24 hours and then washing the sample through 2 mm (No. 10) sieve. The portion of the sample passing No. 10 sieve was again washed through 0.075 mm (No. 200) sieve. The soil specimens, thus, obtained were transferred into a plaster of Paris bowl and allowed to dry until the water content was below the liquid limit value.

3.4.2 Gradation details

The sieve analysis test was conducted to obtain the grain-size distribution of soils from all the eight regions. The test was conducted as per TxDOT's Tex-110-E method. A soil sample representative of the region from which it has been collected is prepared for each site as explained in section 3.4.1. The distribution of particle size of the sample portion retained on No. 200 sieve is determined by sieve analysis, while the sizes of the sample portion passed through No. 200 sieve was determined by the hydrometer analysis. Sieve analysis establishes the percentage of the coarse fraction of the soil (Gravel and Sand) while hydrometer analysis establishes the percentage of fine fraction in the soil specimens (Silt and Clay). The detailed procedures for conducting both sieve and hydrometer analyses can be found in Tex- 110-E manual.

3.4.3 Standard compaction tests

In order to determine the compaction moisture content and dry unit weight relationships of the soils in the present research program, it is necessary to conduct standard Proctor compaction tests on soils to establish compaction relationships. The optimum moisture content of the soil is the water content at which the soils were compacted to a maximum dry unit weight condition. Specimens exhibiting a high compaction unit weight are best in supporting civil infrastructure since the void spaces are minimal and settlement will be less. Compaction tests were conducted on all types of soils to determine moisture content and dry unit weight relationships. Standard Proctor test method using Tex-114-E procedure was followed to determine moisture content versus dry density relationships.

3.4.4 Atterberg limit tests

Atterberg limit tests reveal properties related to consistency of the soil. These properties include liquid limit (LL), plastic limit (PL) and shrinkage limit (SL) are essential to correlate the shrink-swell potential of the soils to their respective plasticity indices. Upon addition of water, the state of soil proceeds from dry, semisolid, plastic and finally to liquid states. The water content at the boundaries of these states are known as shrinkage (SL), plastic (PL) and liquid (LL) limits, respectively (Lambe and Whitman, 2000). Therefore, LL is calculated as the water content at which the soil flows and PL is determined as the water content at which the soil starts crumbling when rolled into a 1/8-inch diameter thread. These tests are somewhat operator sensitive and take time to perform. The numerical difference between LL and PL values is known as plasticity index (PI) and characterizes the plasticity nature of the soil.

Representative soil specimens from different locations were prepared following the above mentioned procedure and were subjected to Atterberg limit tests to determine LL and PL following Tex-104-E and Tex-105-E, respectively. The water content of the specimens during tests were measured based on the repeatable data as reported by Hagerty et al. (1990) and Tex-103-E method. Table 3.1 summarizes the index properties of the control soils from eight site locations.

Table 3.1 Soil classification and plasticity index for soils under study

S No.	Soil	Gradation, %				Classification	Atterberg Limits		
		Gravel	Sand	Silt	Clay	USCS	LL	PL	PI
1	Austin	0	5	38	57	CH	41	17	34
2	Bryan	0	13	40	47	CL	45	14	31
3	El Paso	0	37	42	21	CL	30	14	16
4	Fort Worth	0	11	37	52	CH	61	32	29
5	Keller	0	18	45	37	CL	25	14	11
6	Paris	0	9	45	46	CH	60	24	36
7	Pharr-A	0	2	39	59	CH	67	22	45
8	Pharr-B	0	3	55	42	CH	56	19	37

3.5 Stabilizer mix design

The objective of the stabilizer mix design was to determine the type and concentration of additive or stabilizer and evaluate the improvement of engineering properties with varying concentrations of the selected additive. Performing mix design is essential in order to:

- Ensure the optimum percentage of additive(s) used
- Optimize the engineering properties
- Measure the effectiveness of these engineering properties using moisture conditioning
- Observe the effectiveness of the additive(s) with a specific soil and its inherent mineralogy
- Provide density and moisture control parameters for construction
- Mitigate cracking and other distresses associated with material behavior.

A detailed step by step procedure followed to obtain the optimum additive content for the selected soils is shown below.

3.5.1 Selection of stabilizing agent

The Plasticity Index (PI) of the soil has a direct impact on the selection of the additives as described on Figure 3.12. Stabilizer selection was done according to this method of TxDOT and the additives selected were lime and cement. The determination of the optimum moisture content (OMC) and maximum dry unit weight (MDUW) is also important when the material is mixed with the stabilizer.

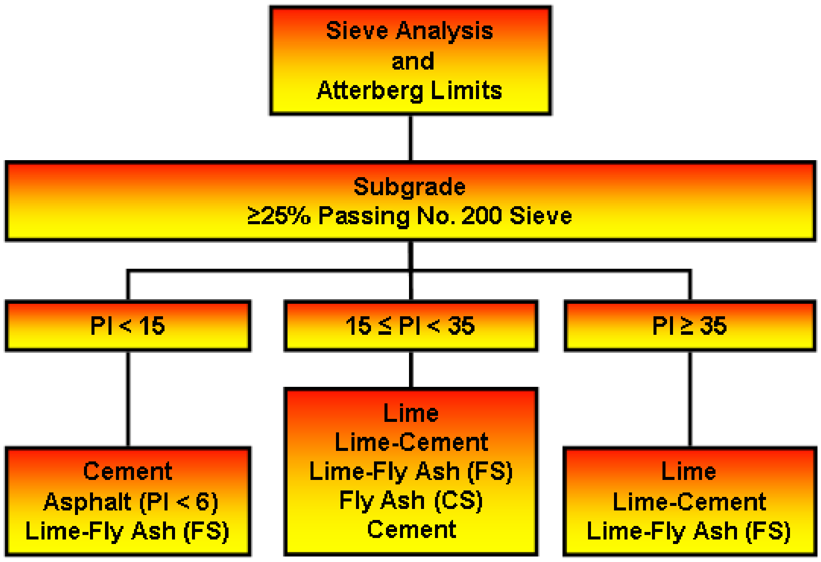


Figure 3.12 TxDOT additive selection criteria

3.5.2 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 3.13. Based on these results, the minimum amount of lime necessary to stabilize the El Paso material is 6%. The appropriateness of the soil-mix is established by strength requirements. Tex-121-E suggests an unconfined compressive strength of 50 psi as adequate for soils treated with lime. The curing time prior to perform unconfined compressive strength tests is 17 days, because specimens were stored at room temperature for 7 days for curing, then air dried in the oven for a period of 6 hours, and finally subjected to capillarity wetting for ten days. The minimum number of specimens to complete this test was three per lime content.

To study the influence of lime content on the strength, different lime contents were considered. The optimum moisture contents (OMC) and maximum dry unit weights (MDUW) for all the soil-lime combinations that are considered were obtained according to Tex-114-E and the result of one soil (El Paso clay) is shown in Table 3.2. As the lime content increases, the OMC decreases, while the MDUW fluctuates by about 3%.

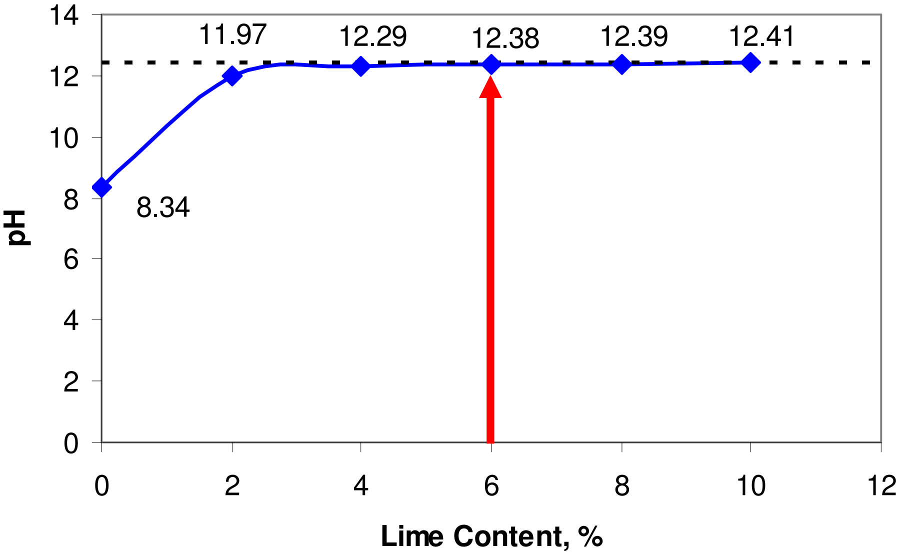


Figure 3.13 Variation in pH with Lime Content for El Paso clay

Table 3.2 Moisture density characteristics of El Paso material for different lime percentages

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

The following series of tests were carried out to determine the appropriateness of the stabilizer and its content based on the strength requirements:

- Unconfined Compressive Strength Standard Curing (UCS STD)

- Unconfined Compressive Strength Dry Curing (UCS Dry)

The time needed and specifications for curing to complete each of these tests are detailed in Table 3.3.

Table 3.3 Curing methods for strength tests on clay

Curing Type	**7-Day**	**Accelerated**
Moist Curing	7 days in wet room	N/A
Drying	N/A	2 days in oven at a temperature not to exceed 60℃
Moisture Conditioning	N/A	1 day back pressure saturation
Total # of days	7 days	3 days

3.5.3 Strength tests

Unconfined compressive strength (UCS) tests were carried out on standard cured, dry and accelerated cured specimens for the three soil-lime combinations according to ASTM D-2166.

3.6 Durability studies

An important test to address the durability of chemically treated soils in arid environments is by exposing the treated soil specimens to various cycles of wetting and drying processes. 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 method is the standard method often used for these wet-dry cycles investigations. The following section details the test procedure followed and the modifications made to ASTM D 559 method.

3.6.1 Sample preparation

Sample preparation was done by using static compaction method instead of the dynamic compaction method. The dynamic method was avoided as this method requires the soil to be compacted in layers and these layers hinder the process of capillary saturation during the wetting process. The static compaction method of sample preparation was done according to the procedure outlined in Wanyan et al. (2008).

A static compactor, suggested in the AASHTO T-307 for preparing fine-grained soil specimens, was used. With this method, specimens with targeted moisture and density levels can be prepared in a short time. One of the concerns with this method is the variation of density along the length of the soil specimen. A comprehensive study was carried out by Wanyan et al., 2008 to develop a process for static compaction of clays in one layer with small variation in density. 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 height. In order to prepare the specimens, three solid blocks with different thicknesses were used. Compaction process that was recommended in that study was used here for the preparation of soil specimens. Figure 3.14 illustrates the step-by-step procedure for static compaction. The steps involved are as follows:

Step 1: The exact amount of required material is weighed to prepare a specimen. This amount is calculated based on desired dry density, degree of compaction and the moisture content

Step 2: A metal mold is taken and the inside surface is lubricated to minimize friction while extruding the sample

Step 3: This metal mold is filled with the desired material

Step 4: Two cylindrical blocks were placed on the top and bottom to compact the specimen

Step 5: The specimen is compacted when the top block becomes flush within the mold

Step 6: After waiting 1 minute in this condition the mold is rotated and the top block is replaced with a thicker plate

Step 7: To get the desired height the load is applied again on top of the plate

Step 8: At the end the specimen is extruded from the mold by using a hydraulic jack

3.6.2 Wetting and Drying

The procedure outlined by ASTM D 559 method was closely followed, which simulates both wet and dry cycle conditions close to local conditions in a reasonably short time period. The soil specimens were allowed to swell and shrink in both lateral and vertical directions. Prior studies by Punthutaecha et al., 2006 noted that the volumetric swell/shrink strains obtained by allowing lateral movement along with vertical movements are in close agreement with the field measured volume changes than those obtained by restraining the lateral movement.

According to the ASTM D 559 method, the soil specimens should be prepared and cured then submerged in water for 5 hrs for wetting cycle and then oven dried at 160ºF for 48 hours for drying cycle. Each wet-dry cycle consists of submerging the soil sample in water for 5 hours and then placing them in a 70℃ oven for 42 hours. 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 were completed or until the sample failed.

The test setup used in this research can be seen in Figure 3.15. During wetting and drying periods, the changes in soil sample size were measured in all the three dimensions. The Vertical movement was measured with the help of a dial gauge and the radial movements were measured using a "pi tape". The wetting/drying was continued for 21 cycles. After 3, 7, 14 and 21 cycles, the specimens were subjected to UCS tests.

3.7 Leachate studies

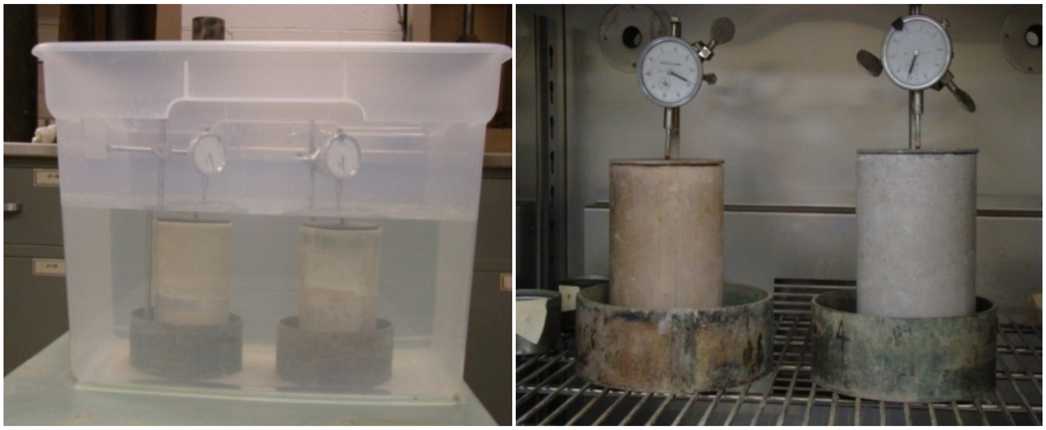
A new test protocol was developed by McCallister (1990) at University of Texas at Arlington to address the permanency of the chemical stabilization from moisture flows during rainfall events, ground water flows and moisture migration from suction and head differences was used in the current research. This test utilizes a flexible wall mold housing the compacted stabilized soil specimen. Figure 3.16 illustrates a schematic of the test setup used in this research. This setup is similar to the one used by McCallister (1990) for leachate studies conducted at UTA 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.

Soil mixed with lime after mellowing **Pouring the soil into the mould** **Static Compaction**

Sample Extraction **Extracted Sample**

Figure 3.14 Sample preparation method for the specimens that are to be subjected to wetting and drying studies



a) b)

Figure 3.15 Apparatus used for the wet/dry studies: a) Wetting b) Drying

Specimens of each soil/additive combinations were prepared at the optimum moisture content and maximum dry density according to the method explained in section 3.6.1. The lime-treated soil specimens were prepared after mellowing period of 24 hours. Soil specimens were then cured for seven days in a moisture room prior to subjecting them to leachate process. 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 here 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 3.17. In this figure typical calculations for Paris soil sample are given.

The cured specimens were kept inside the sample cell (refer to Figure 3.18) and the top plate was fastened in place using the fasteners shown in Figure 3.18. A confining pressure higher than the flow pressure was applied through the confining pressure inlet as shown in Figure 3.19. Then water was allowed to go through the top under a constant flow pressure through the flow pressure inlet and leachate sample was collected in 20 liter carboys shown in the photograph.

Leachate tests were conducted on several identically prepared and cured soil specimens. Leachate specimens were collected after 3, 5, 7, 11, and 14 cycles of leaching, while the UCS tests were conducted on soil specimens after 3, 7 and 14 cycles of leaching. Leachate specimens 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 is made to correlate leaching cycles with field moisture movements from rainfall events. All these results are presented in Chapter 6.

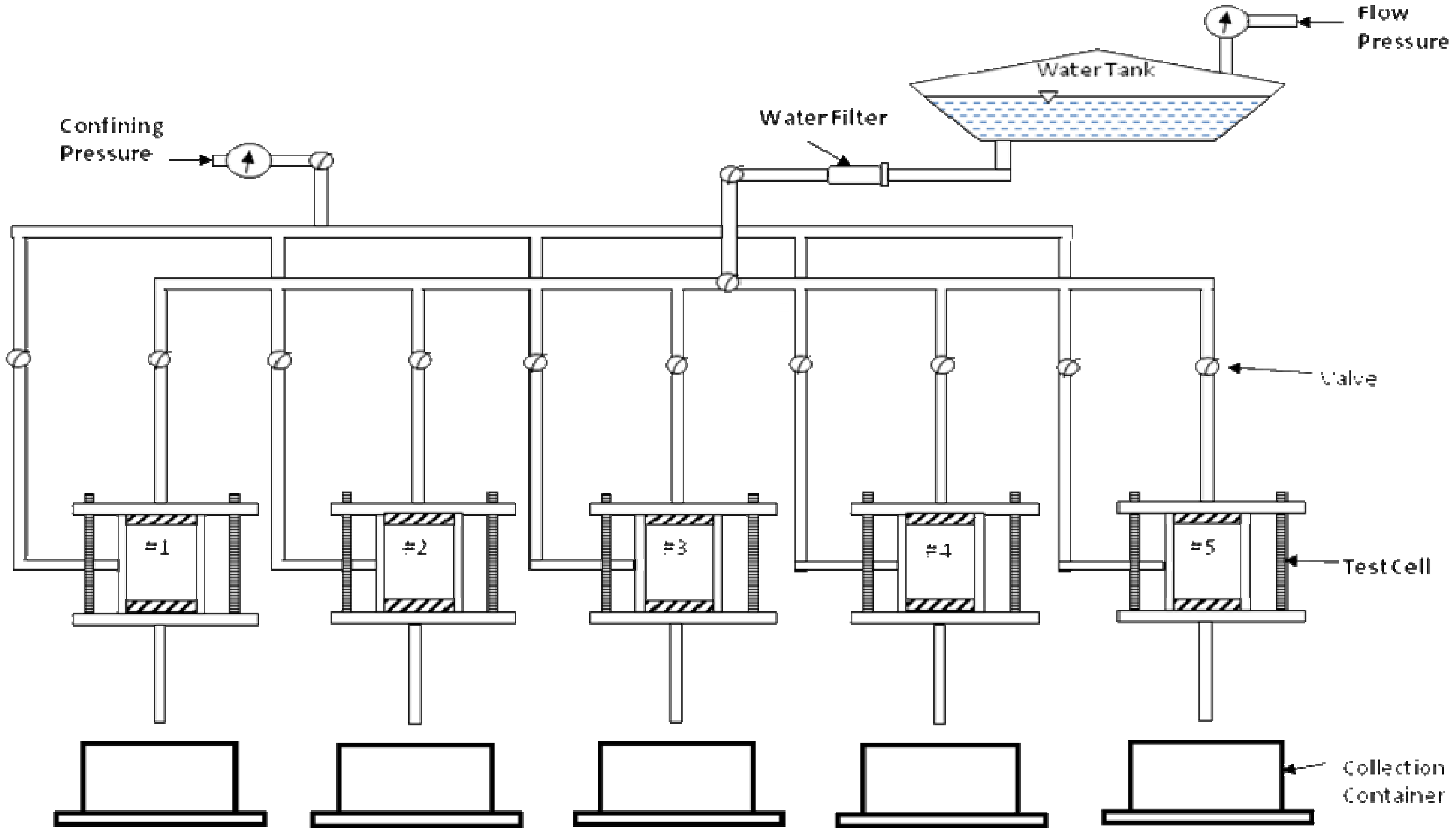
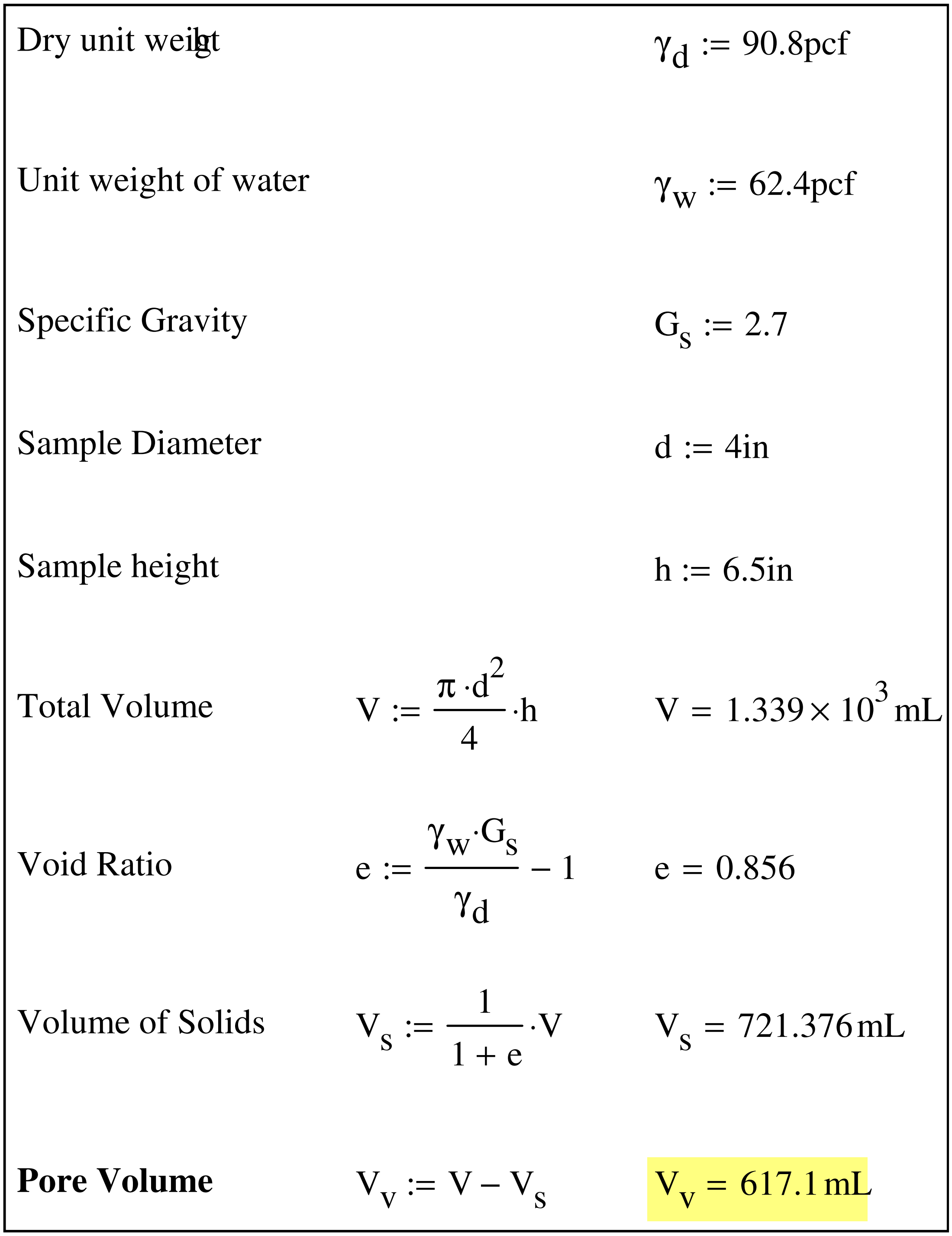


Figure 3.16 Schematic of the leachate process



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 3.17 Pore Volume calculations for paris soil

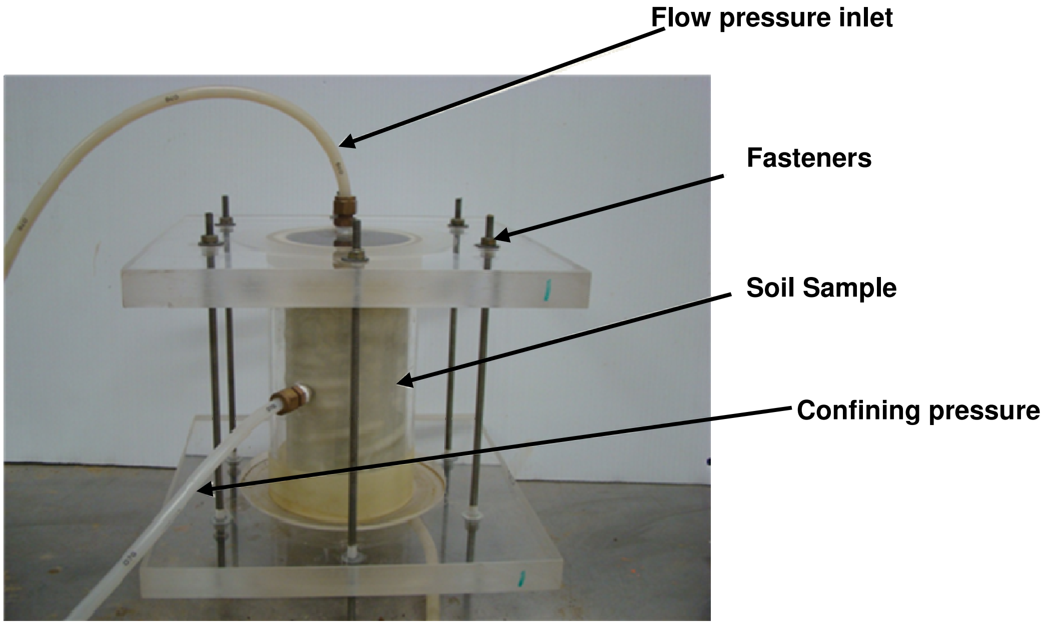


Figure 3.18 Leachate cell

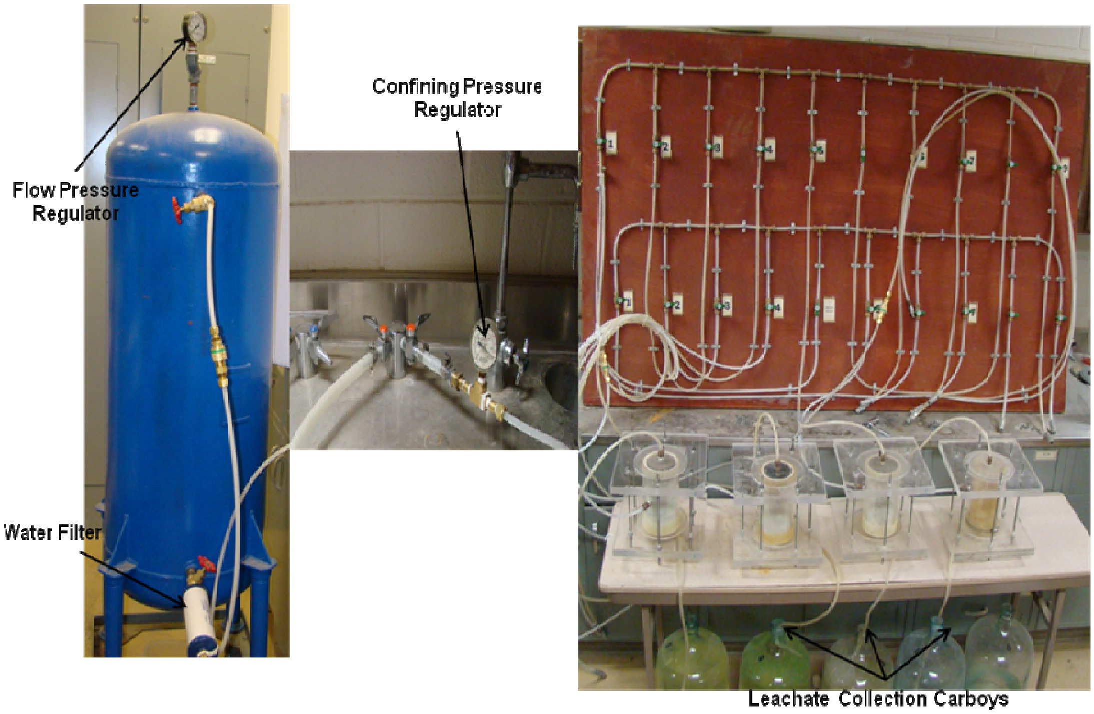


Figure 3.19 Apparatus used to conduct leachate studies

3.8 Summary

In this chapter various test procedures followed in the present research to determine the engineering properties of both control and treated soils were described. The chemical and mineralogical tests conducted for clay mineral quantification were explained in detail. Durability type test procedures followed here to replicate moisture content fluctuations in the field during summer and rainy seasons was described and leachate studies following the simulation of moisture infiltration from rain fall events are presented. The next chapter presents the results obtained from all the above mentioned tests that were conducted on all the soils selected for this study.

CHAPTER 4

MINERALOGICAL STUDIES

4.1 Introduction

This chapter presents a comprehensive analysis of the mineralogical studies that were conducted on selected soils to quantify the clay minerals present in them. The methodology followed to obtain the dominating clay mineral in a given soil is explained. A statistical model and an artificial neural network model to predict the dominating clay mineral in the soils selected are also presented. A total of 20 soils from different regions in the state of Texas were selected for this research study.

4.2 Repeatability of the test methods

To check the repeatability of the test methods, each test procedure was conducted thrice on three types of soils and these results are shown in

Table 4.1. It can be observed in the table that the standard deviation is low, indicating a good repeatability of these test results.

Table 4.2 gives the results of the chemical tests such as *Cation exchange capacity (CEC), Specific surface area (SSA) and Total potassium (TP)* conducted on all twenty (20) soils. Once the three properties were obtained for each soil, the clay minerals present in the soil can be quantified by assuming that each clay mineral in a soil specimen contributes linearly to its content towards each of the property under review. A primary assumption here based on the XRD analysis was that the fine fraction of the soil specimens comprises only of the following common clay minerals: *Kaolinite*, *Illite* and *Montmorillonite*. To quantify the clay minerals based on this assumption it was necessary to obtain the above mentioned chemical properties for these pure clay minerals. This information was collected from the literature and the ranges of these values considered in this research are given in Table 4.3.

Table 4.1 Table showing the repeatability of the test results

Soil Type : El Paso					
	Trial 1	Trial 2	Trial 3	Mean	SD
CEC, meq/ 100 gm	55.2	57.7	53.3	55.40	2.21
SSA, m^2/gm	158	164	161	161.00	3.00
TP, %	3.6	3.6	3.8	3.67	0.12
Soil Type : Paris					
	Trial 1	Trial 2	Trial 3	Mean	SD
CEC, meq/ 100 gm	130.1	133.9	135.4	133.13	2.73
SSA, m^2/gm	431	424	440	431.67	8.02
TP, %	0.77	0.79	0.78	0.78	0.01
Soil Type : Bryan					
	Trial 1	Trial 2	Trial 3	Mean	SD
CEC, meq/ 100 gm	77.4	79.1	75.2	77.23	1.96
SSA, m^2/gm	207	202	204.9	204.63	2.51
TP, %	1.37	1.4	1.32	1.36	0.04

Table 4.2 Values of SSA, CEC and TP for all the soil samples

S 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 4.3 Range values of SSA, CEC and TP for pure minerals

Mineral Type	CEC, meq/100 gm	SSA, m^2/gm	% Potassium
Illite	15 to 60	80 to125	6
Kaolinite	1 to 6	5 to 45	0
Montmorillonite	80 to160	600 to 850	0

4.3 Quantification procedure

The mineralogical compositions of different soils were estimated analyzing the following three simultaneous equations:

$$\%M \times CEC_M + \%K \times CEC_K + \%I \times CEC_I = CEC_{soil} \quad (4.1)$$

$$\%M \times SSA_M + \%K \times SSA_K + \%I \times SSA_I = SSA_{soil} \quad (4.2)$$

$$\%M \times TP_M + \%K \times TP_K + \%I \times TP_I = TP_{soil} \quad (4.3)$$

Where,

%M, %K, %I are the percentages of the minerals Montmorillonite, Kaolinite, and Illite present in the soil sample;

CEC_M, CEC_K, CEC_I are the CEC values of the pure minerals Montmorillonite, Kaolinite, and Illite present in the soil sample, respectively;

SSA_M, SSA_K, SSA_I are the SSA values of the pure minerals Montmorillonite, Kaolinite, and Illite present in the soil sample, respectively;

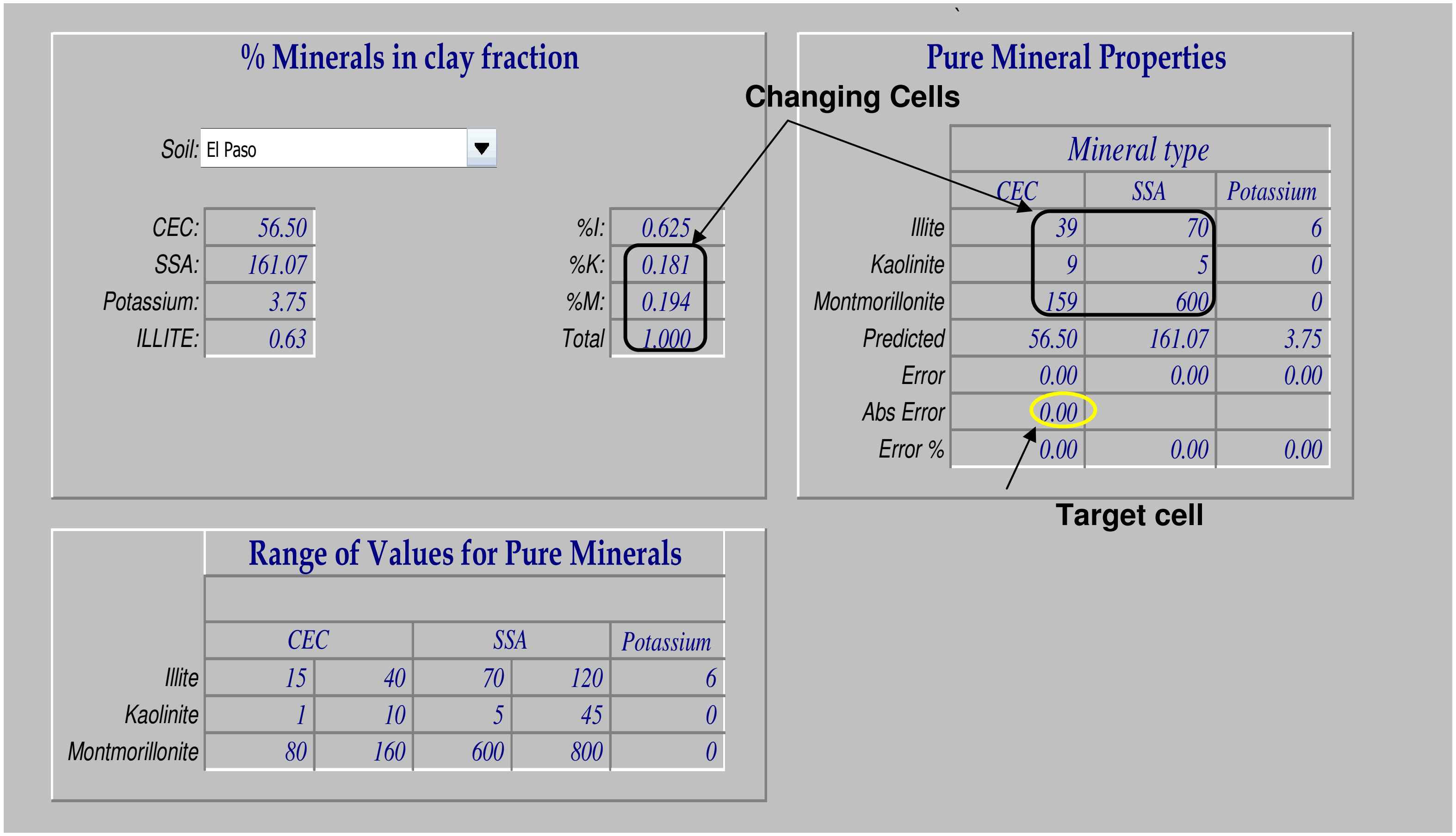
TP_M, TP_K, TP_I are the TP values of the pure minerals Montmorillonite, Kaolinite, and Illite present in the soil sample, respectively;

CEC_{soil}, SSA_{soil}, TP_{soil} are the values of the properties CEC, SSA and TP of the soil sample, respectively.

Solving the simultaneous equations was not an easy task as the CEC and SSA values for the pure minerals are not constant, but fall within a range of values as per the literature review. To solve this problem, a model was developed using the *Solver* function of Microsoft Excel® program. According to the help manual (MS Excel, 2007) provided in Microsoft's Excel® program, *solver* is part of a suite of commands sometimes called what-if analysis tools. Using this function one can obtain an optimal value for a formula in one cell called the target cell in a worksheet by adjusting the values in the cells called 'changing cells' that can be directed to produce the result that is specified in the target cell formula. Constraints can be applied to the changing cells to restrict the values that *Solver* can use in the model to obtain the specified value for the target cell. A more detailed explanation on how the solver function works can be obtained in the help manual of MS Excel (2007).

In this study, the target cell is the absolute error defined as the sum of the difference between the measured CEC and SSA and the calculated CEC and SSA. Calculated CEC and SSA are defined as the CEC and SSA values that are obtained after substituting the acquired mineral percentages back in equations (4.1) and (4.2) respectively. It was imperative that the absolute error should be zero to accept the acquired mineral percentages. Hence, the absolute error value is constrained to have a value equal to zero. The changing cells were CEC and SSA values of the pure minerals along with the percentages of the minerals Kaolinite (%K) and Montmorillonite (%M). The CEC and SSA of the pure minerals are constrained between the range of values provided in Table 4.3 above and %K and %M are constrained to take a value between 0 and 1.

The %K and %M were obtained by adjusting the above mentioned changing cells (CEC, SSA, %K and %M) to obtain a value of zero for the target cell (absolute error) while the percentage of Illite mineral was obtained directly by dividing TP value of the given soil by six as TP is contributed only from the inter layer potassium present in the mineral Illite (Mitchell and Soga, 2005). Figure 4.1 shows a snapshot of this program. Table 4.4 gives the mineral percentages of all the 20 soils obtained by solving the above mentioned equations by the procedure explained. Figure 4.2 and Figure 4.3 shows the variation of CEC and SSA between the measured and calculated numbers respectively. It can be observed from Figure 4.2 that the measured CEC numbers are slightly lower than the calculated CEC, this difference is due to the difficulty in obtaining a unique solution to maintain a zero error for both equations 4.1 and 4.2. However, in the case of SSA, the measured and the calculated values were the same (refer to Figure 4.3) which indicate a zero error in case of equation 4.2. Hence, the percentage values that gave an absolute error value close to zero were selected as the final % minerals.



% Minerals in clay fraction

Soil: El Paso

CEC:	56.50	%I:	0.625
SSA:	161.07	%K:	0.181
Potassium:	3.75	%M:	0.194
ILLITE:	0.63	Total	1.000

Pure Mineral Properties

Changing Cells

	Mineral type		
	CEC	SSA	Potassium
Illite	39	70	6
Kaolinite	9	5	0
Montmorillonite	159	600	0
Predicted	56.50	161.07	3.75
Error	0.00	0.00	0.00
Abs Error	0.00		
Error %	0.00	0.00	0.00

Target cell

Range of Values for Pure Minerals

	CEC		SSA		Potassium
Illite	15	40	70	120	6
Kaolinite	1	10	5	45	0
Montmorillonite	80	160	600	800	0

Figure 4.1 Snap shot of the Microsoft Excel® program used for the quantification of clay minerals

Table 4.4 Percentage mineral information of all the soils tested determined by the proposed method

S 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

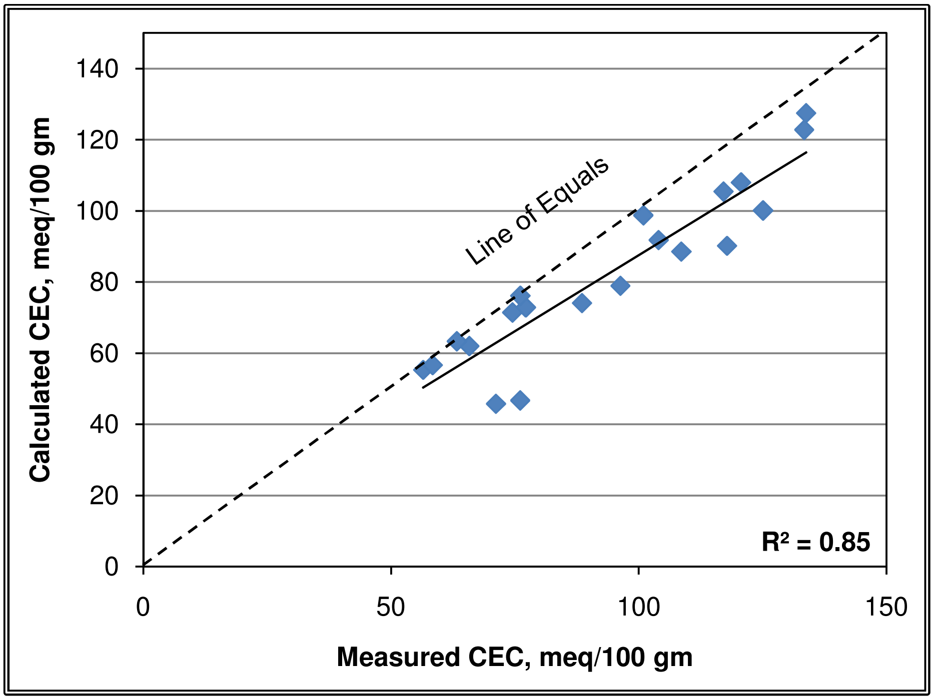


Figure 4.2 Comparisons between the measured and the calculated CEC numbers

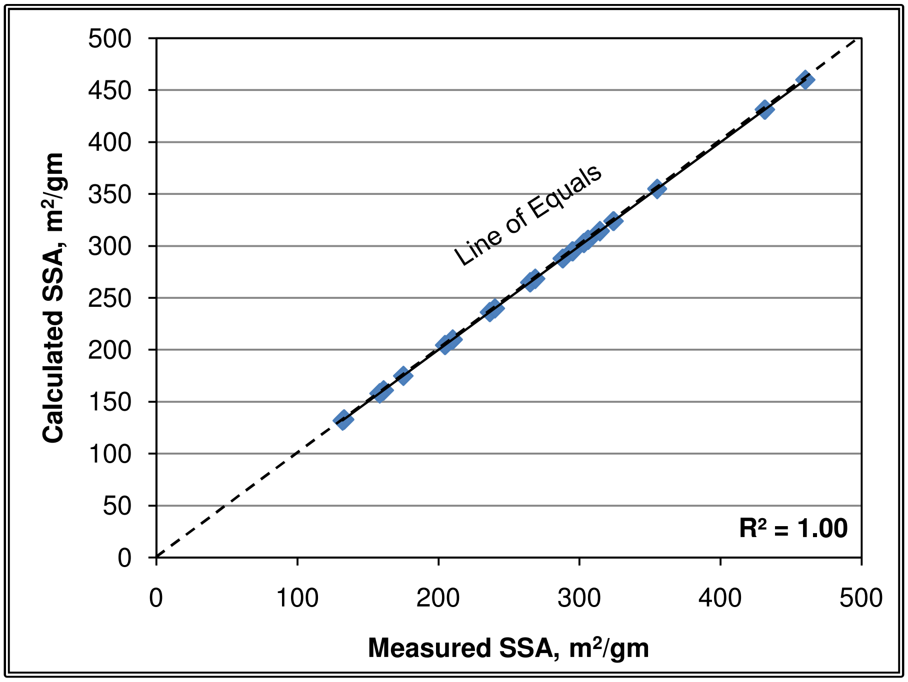


Figure 4.3 Comparisons between the measured and the calculated SSA numbers

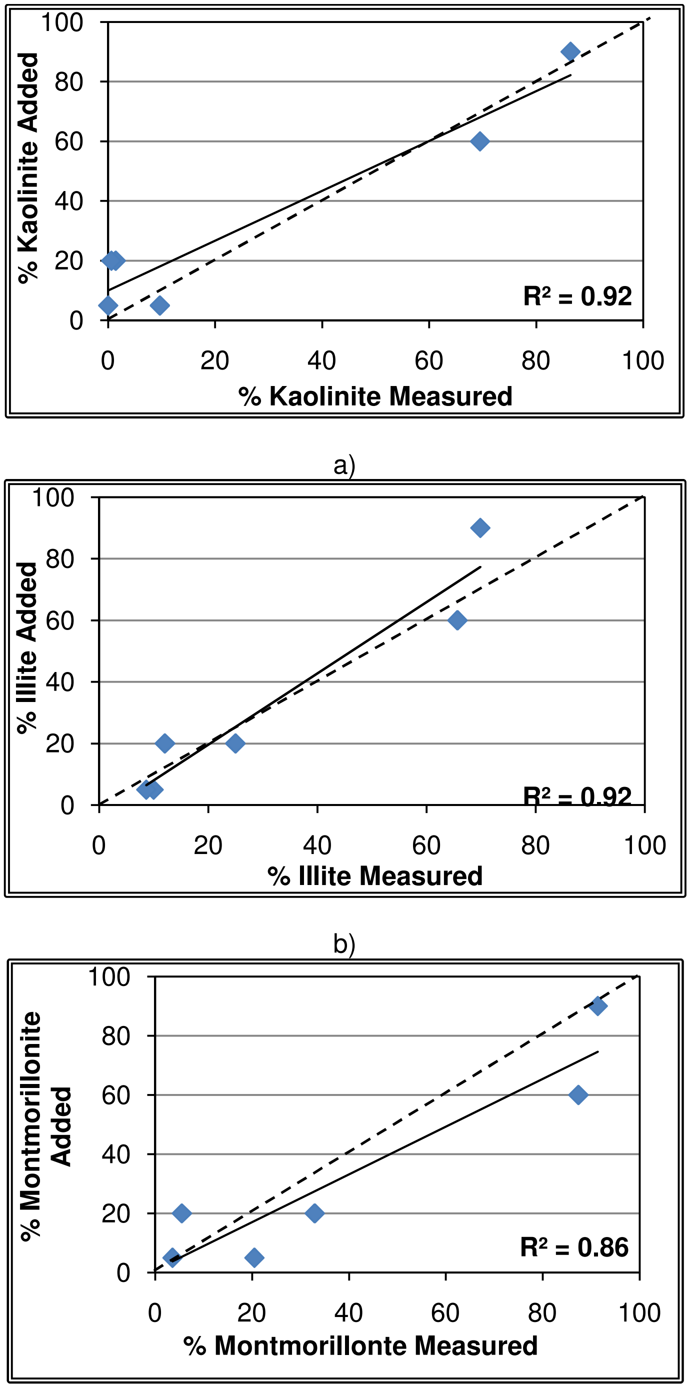
4.3.1 Validation

It was necessary to check the accuracy of the proposed method of calculating mineral percentages in the clay fraction of soil. One way to check the accuracy was to quantify these soils with the established procedures such as X-ray diffraction methods or other chemical mass balance methods. Quantifying all twenty (20) soils using these methods was time consuming and expensive and hence was not undertaken in this research. Another method to check the precision of the method was to artificially prepare a soil sample with known clay mineral percentages and back-calculate the mineral percentages using the method explained above. Hence, to validate this method, six artificial soil specimens with known mineral percentages were prepared. These artificially prepared soils were divided in to three groups I, K, M. Soils in group I were dominant with the mineral Illite, soils in group K were dominant with the mineral Kaolinite and those in group M were dominant with mineral Montmorillonite.

Same chemical tests, CEC, SSA and TP were conducted on all six soil specimens. Table 4.5 presents a summary of the chemical test results of the grouped artificial samples. Figure 4.4 shows the comparisons made between the prepared and the measured percentages of the minerals. From the figure it can be observed that the proposed method predicts the mineral percentages in the soil with high accuracy ($R^2 = 0.92$). However, the method is limited to predicting soils in which these three clay minerals (Illite, Kaolinite, and Montmorillonite) considered as dominant.

Table 4.5 CEC, SSA and TP values of the artificially prepared samples

S No.	Soil Type	CEC, meq/100 gm	SSA, m^2/gm	TP, %
1	I1	27	172	4.19
2	I2	45	244	3.94
3	K1	6	40	0.60
4	K2	18	65	1.50
5	M1	125	650	0.52
6	M2	120	625	0.72



a)

b)

c)

Figure 4.4 Comparisons between the measured and added % of the minerals
a) Kaolinite b) Illite c) Montmorillonite

4.4 Prediction models

Due to the difficulty in maintaining an absolute error value equal to zero while solving the above mentioned simultaneous linear equations, models were developed to predict the percentage minerals using the chemical tests data (refer to Table 4.2 and Table 4.4). The data has been divided into two sets of data; model data and validation data as presented in Table 4.6. The testing data is selected based on the %Montmorillonite to represent low, medium and high percentages of this mineral.

Two different techniques were considered for these predictions and these were statistical regression analysis and artificial neural networks or ANN based models. Details of the techniques and their results are given in the following sections.

4.4.1 Regression analysis

Regression analysis is a statistical tool to establish a relationship between two or more variables. In the current research, the regression analysis was conducted to predict the amount of Montmorillonite using the test data presented in Table 4.6. Correlation coefficients between the chemical properties and the mineral percentages of all the soils were calculated to check the interdependency of the chemical properties and the mineral percentages. Table 4.7 presents the correlation coefficient matrices of the values of soil chemical properties obtained from the test soil samples. From the table it can be observed that cation exchange capacity and specific surface area have good correlations among them and also with %Montmorillonite in the clay fraction. Also, there is a perfect fit between %Illite and total potassium, which was expected as potassium was derived from Illite fraction only.

Table 4.6 Categorized data for model development and validation

S No.	Soil Type	CEC	SSA	TP	% Illite	% Kaolinite	% Montmorillonite
MODEL DEVELOPMENT DATA SET							
1	Amarillo	66	175	0.97	16.17	51.61	32.22
2	Arlington	121	324	0.77	12.83	24.41	62.76
3	Austin	101	288	1.74	28.96	17.67	53.37
4	Bryan	77	205	1.36	22.72	39.93	37.35
5	Bryan Silt	89	210	1.12	18.67	42.37	38.96
6	DC	63	295	1.38	22.92	21.51	55.58
7	El Paso	57	161	3.75	62.50	14.18	23.32
8	Fort Worth	117	314	0.98	16.25	23.38	60.37
9	Gate	118	265	0.80	13.33	35.89	50.77
10	JC # 1	125	355	0.83	13.75	28.93	57.32
11	Keller	71	133	1.10	18.33	62.16	19.51
12	Paris	133	431	0.79	13.13	16.65	70.22
13	Pharr A	104	306	1.55	25.83	26.40	47.77
14	San Antonio	96	269	1.10	18.33	39.38	42.28
15	Seymour	58	158	3.53	58.75	21.92	19.33
16	VC	109	303	1.50	25.00	27.65	47.35
17	I1	27	172	4.19	69.85	9.65	20.49
18	I2	45	244	3.94	65.67	1.40	32.93
19	K1	6	40	0.60	10.00	86.42	3.58
20	M1	125	650	0.52	8.67	0.00	91.33
MODEL VALIDATION DATA SET							
1	T1	134	460	1.22	20.33	5.42	74.25
2	T2	76	236	1.76	29.38	27.74	42.88
3	T3	75	240	0.95	15.83	46.40	37.77
4	T4	76	132	1.65	27.50	54.16	18.34
5	T5	18	65	1.50	25.00	69.50	5.50
6	T6	120	625	0.72	12.08	0.58	87.35

Table 4.7 Table showing the correlation coefficients between chemical properties and mineral percentages

	CEC, meq/100g	**SSA, m^2/g**	**TP, %**	**%I**	**%K**	**%M**
CEC, meq/100g	1.00	0.76	-0.56	-0.56	-0.32	0.86
SSA, m^2/g	0.76	1.00	-0.36	-0.36	-0.61	0.95
TP, %	-0.56	-0.36	1.00	1.00	-0.47	-0.49
%I	-0.56	-0.36	1.00	1.00	-0.47	-0.49
%K	-0.32	-0.61	-0.47	-0.47	1.00	-0.54
%M	0.86	0.95	-0.49	-0.49	-0.54	1.00

Regression equations are formulated to predict the three percent clay minerals using each of the soil or chemical properties. Table 4.8 gives the regression equations developed along with the coefficient of determination (R^2) values. It can be observed from the table that the regression equation developed for predicting %Montmorillonite using the properties CEC and SSA alone have high coefficient of determination than any other regression equation developed using single soil parameters. However, the prediction of %Illite has an R^2 of 1.0 which is a perfect fit because the %Illite value is obtained by dividing the TP value obtained from the chemical test by the TP value of pure Illite which is six (6). Prediction equations were also developed using two soil parameters and three soil parameters. The equations obtained are given in Table 4.9 and Table 4.10.

Table 4.8 Predictions of the % minerals using single parameter along with their coefficients of determination

% Kaolinite Predictions		
Variable	**Equation**	**R^2**
%K = f (CEC)	%K = 45.5 - (0.18 * CEC)	0.10
%K = f (SSA)	%K = 55.7 - (0.10 * SSA)	0.37
%K = f (TP)	%K = 42.8 - (8.15 * TP)	0.22
% Illite Predictions		
Variable	**Equation**	**R^2**

Table 4.8 Continued

%I = f (CEC)	%I = 53.8 - (0.31 * CEC)	0.31
%I = f (SSA)	%I = 41.9 - (0.05 * SSA)	0.13
%I = f (TP)	%I = 16.6 * TP	1.00
% Montmorillonite Predictions		
Variable	**Equation**	**R^2**
%M = f (CEC)	%M = 0.7 + (0.49 * CEC)	0.73
%M = f (SSA)	%M = 2.3 + (0.15 * SSA)	0.90
%M = f (TP)	%M = 57.18 - (8.52 * TP)	0.24

Table 4.9 Predictions of the % minerals using two parameters along with their coefficients of determination

% Kaolinite Predictions		
Variables	**Equation**	**R^2**
%K = f (CEC, SSA)	%K = 49.8 + (0.20 * CEC) - (0.14 * SSA)	0.42
%K = f (SSA, TP)	%K= 90.4 - (0.14 * SSA) - (13.77 * TP)	0.93
%K = f (CEC, TP)	%K = 98.5 - (0.49 * CEC) - (16.42 * TP)	0.73
% Illite Predictions		
Variables	**Equation**	**R^2**
%I = f (CEC, SSA)	%I = 53.0 - (0.38 * CEC) + (0.02 * SSA)	0.32
%I = f (SSA, TP)	%I = 1.42E-14 – (8.32E-16 * SSA) + (16.67 * TP)	1.00
%I = f (CEC, TP)	%I = 4.9E-14 – (4.9E-16 * CEC) + 16.7 * TP	1.00
% Montmorillonite Predictions		
Variables	**Equation**	**R^2**
%M = f (CEC, SSA)	%M = -2.9 + (0.18 * CEC) + (0.12 * SSA)	0.95
%M = f (SSA, TP)	%M = 9.6 + (0.14 * CEC) - (2.90 * TP)	0.93
%M = f (CEC, TP)	%M = 1.480 + (0.495 * CEC) - (0.245 * TP)	0.73

Table 4.10 Predictions of the % minerals using three parameters along with their coefficients of determination

Variables	Equation	R^2
%K = f (CEC, SSA, TP)	%K = 98.2 - (0.14 * CEC) - (0.12 * SSA) - (15.19 * TP)	0.95
%I = f (CEC, SSA, TP)	% I = 5.6E-14 - (7.7E-16 * CEC) + (6.9E-17 * SSA) + (16.6 * TP)	1.0
%M = f (CEC, SSA, TP)	%M= 1.8 + (0.15 * CEC) + (0.12 * SSA) - (1.48 * TP)	0.95

4.4.1.1 Reliability of equations

Reliability analysis was performed for the equations that had R^2 values, more than 0.9. The following approach is used for the reliability analysis of the equations. The problem is divided into the predicted (predicted using the regression equation) and the measured values (measured by solving simultaneous equations). The ratio between the measured and the predicted value is defined as the *Reliability Ratio*. This ratio is expected to be close to 1 in order to be recommended for potential usage. However, this is not always possible in geotechnical engineering hence the probability of failure is calculated for the reliability ratio to be in the range 0.7 to 1.3. The equations that have an R^2 value more than 0.9 which are being analyzed for reliability are listed as follows:

$$\%K = 90.4 - (0.14 * SSA) - (13.77 * TP) \tag{4.4}$$

$$\%K = 98.2 - (0.14 * CEC) - (0.12 * SSA) - (15.19 * TP) \tag{4.5}$$

$$\%M = 2.3 + (0.15 * SSA) \tag{4.6}$$

$$\%M = -2.9 + (0.18 * CEC) + (0.12 * SSA) \tag{4.7}$$

$$\%M = 9.6 + (0.14 * CEC) - (2.90 * TP) \tag{4.8}$$

$$\%M = 1.8 + (0.15 * CEC) + (0.12 * SSA) - (1.48 * TP) \tag{4.9}$$

The measured data and the predicted data are represented with the random variables M and P respectively and are assumed to follow a log-normal distribution. This assumption was validated from chi-square goodness of fit test. This test showed that the distributions were log-normal with 95% degree of confidence. The random variables M and P have means μ_M and μ_P and standard deviations σ_M and σ_P respectively. The reliability analysis is conducted for the reliability ratio ($RR = \frac{M}{P}$) to be in the range 0.7 to 1.3. Note that the random variable RR was also assumed to have lognormal distribution. This assumption is based on the fact that a random variable lf X will have a normal distribution if X follows a lognormal distribution. The log-normal parameters λ_x and ζ_x for a random variable X with mean μ_x and standard deviation σ_x were obtained as follows:

$$\lambda_X = \ln \mu_X - \frac{1}{2}\zeta_X^2 \tag{4.10}$$

$$\zeta_X^2 = ln\left[1 + \left(\frac{\sigma_X}{\mu_X}\right)^2\right] \tag{4.11}$$

After obtaining the lognormal distribution parameters for the random variables M and P the lognormal distribution parameters of the random variable RR are obtained as follows:

$$\lambda_{RR} = \lambda_M - \lambda_P \tag{4.12}$$

$$\zeta_{RR}^2 = \zeta_M^2 + \zeta_P^2 \tag{4.13}$$

Now the probability of success is calculated for RR to be between 0.7 and 1.3.

$$p_s = P(0.7 < RR < 1.3) \tag{4.14}$$

$$P(0.7 < RR < 1.3) = \Phi\left(\frac{ln1.3 - \lambda_{RR}}{\zeta_{RR}}\right) - \Phi\left(\frac{ln0.7 - \lambda_{RR}}{\zeta_{RR}}\right) \tag{4.15}$$

And the reliability of the equation is calculated as p_s. The data obtained is presented in Table 4.11 and Table 4.12. From Table 4.13 it can be observed that the equation 4.7 has a relatively higher reliability 92% hence this equation is recommended for further usage.

Table 4.11 Table showing normal and lognormal distribution parameters for the measured % minerals

Measured Statistics			
Normal distribution Parameters			
Statistic	**% Illite**	**% Kaolinite**	**% Montmorillonite**
Mean, μ	21.69	33.96	44.35
Standard Deviation, σ	6.82	27.55	31.54
Lognormal distribution Parameters			
λ_M	2.53	2.70	3.04
ζ_M^2	1.10	1.66	1.51

Table 4.12 Table showing normal and lognormal distribution parameters for the predicted % minerals using eq. 4 through 9 along with the their reliabilities

Predicted Statistics						
Normal distribution Parameters						
Statistic	**Eq 4**	**Eq 5**	**Eq 6**	**Eq 7**	**Eq 8**	**Eq 9**
Mean, μ	31.46	31.63	46.25	47.23	46.85	47.51
Standard Deviation, σ	25.68	26.45	31.60	31.92	30.37	31.21
Lognormal distribution Parameters						
λ_P	2.62	2.60	3.10	3.13	3.14	3.15
ζ_P^2	1.67	1.70	1.47	1.46	1.42	1.43
λ_{RR}	0.08	0.09	-0.06	-0.09	-0.10	-0.11
ζ_{RR}^2	3.32	3.36	2.97	2.96	2.93	2.94
Reliability	0.926	0.927	0.917	0.940	0.916	0.916

4.4.1.2 Proposed methodology using regression equations

This section explains the method that has to be followed to obtain the final mineral percentages. To arrive at the final percentages of the three minerals under study in a given soil sample the following three equations should be used.

$$\%I = \left[\frac{TP}{6} \right] \times 100 \tag{4.16}$$

$$\%M = -2.87 + 0.08 \times SSA + 0.26 \times CEC \tag{4.17}$$

$$\%K = 100 - \%I - \%M \tag{4.18}$$

Where,

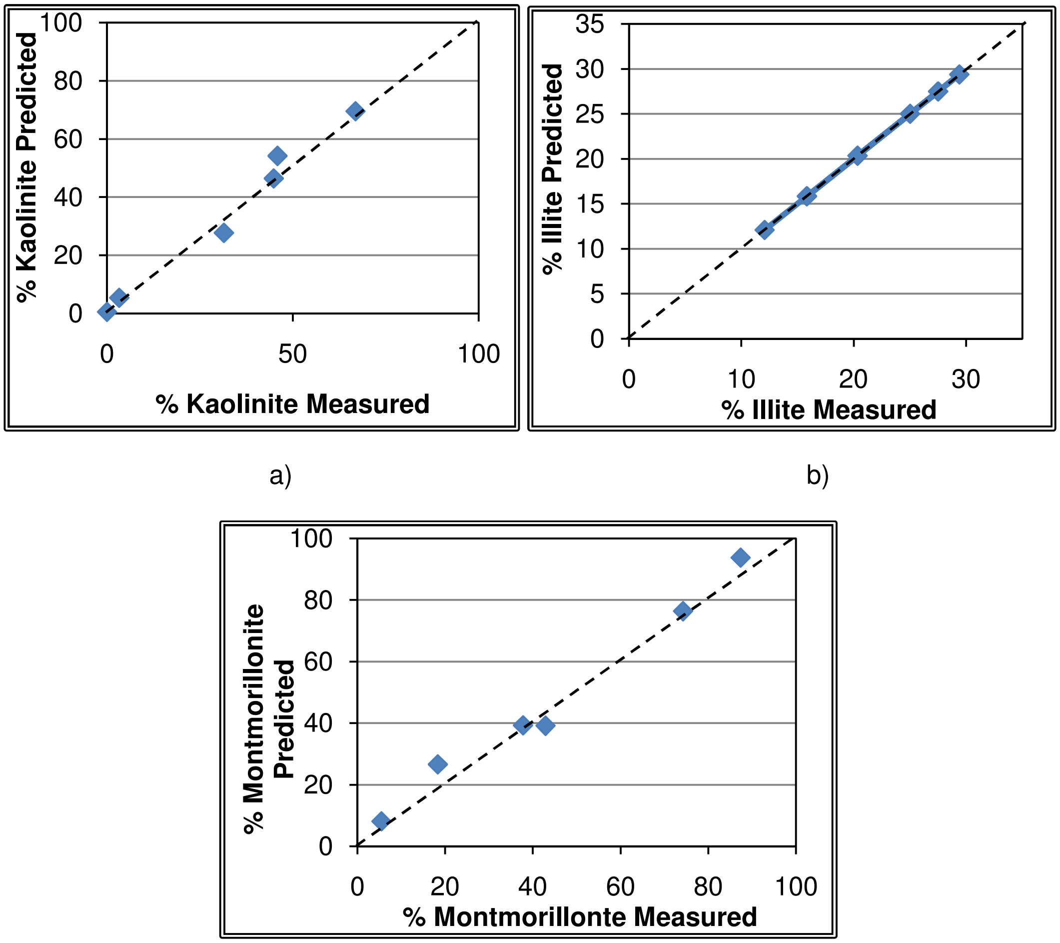
%I = Percentage of the mineral Illite
%M = Percentage of the mineral Montmorillonite
%K = Percentage of the mineral Kaolinite
TP = Total potassium
CEC = Cation exchange capacity
SSA = Specific surface area

Equation (4.16) gives the percentage Illite present in the soil based on the TP of the soil. Equation (4.17) is the regression model developed to obtain the percentage Montmorillonite in the soil and finally equation (4.18) gives the percentage Kaolinite present in the soil sample. The predictive performance of the multiple regression models is measured through the coefficient of determination (R^2). The coefficient of determination (R^2) for equation (4.17) is found to be 0.95, which indicates that the model can predict very well. Figure 4.5 presents a comparison of the mineral predictions from the developed model with the mixed percentages for the artificial samples prepared for the validation of the model. It can be observed from the R^2 values provided in the figures that the above equations can predict with a reasonable amount of accuracy.

4.4.2 Neural network model

Artificial Neural Networks (ANNs), in general, simulates the biological structure of human brain by means of their architecture. ANN technique is an emerging technique that has been applied to many geotechnical engineering applications successfully by many researchers

(Shahin et al., 2001). The Transportation Research Board (TRB) has summarized the state-of-the-art ANN applications in geotechnical engineering (E-C012, 1999). The objective of the model is to predict the relationship between the model input(s) and corresponding output(s). This is obtained by repeatedly feeding the known examples of input/output relationships to the model and then minimizing the error function defined between the measured and predicted outputs by the model.



a) b)

c)

Figure 4.5 Comparisons between the added and the predicted % of the minerals using regression analysis a) Kaolinite mineral b) Illite mineral c) Montmorillonite

4.4.2.1 Methodology

The development of neural network model consists of selecting model input/out parameters, data pre-processing, designing appropriate model architecture, model training and model validation. The data used to train and validate the neural network model were obtained from a series of experimental tests carried out on various soil samples obtained from different regions in the state of Texas as explained before. A total of 20 data sets were obtained from these model test results out of which, 60% of the data was used to train the selected model and 20% of the data was used for the testing and the remaining 20% was used for model validation.

The model development database is divided into three subsets, a training set and a testing set, to develop the neural network model and then a validation set to check the accuracy of the predictions. Both the training set and the validation set contain all the patterns to represent the entire data set. Table 4.13 gives the input variables and the statistics of the test data used. After data preprocessing the next hurdle is arriving at versatile model architecture.

In this step the number of hidden layers and their corresponding nodes in each hidden layer were determined. According to Hornik et al. (1989), a network with a single hidden layer provided with enough connection weights can be used to approximate any continuous function. Accordingly, a network with single hidden layer was trained with different number of nodes (varying from 1 through 15). Finally, the network with a single hidden layer having eight nodes is accepted as the best network architecture for this study. The input layer has 3 nodes, the hidden layer has 8 nodes and the output layer has 3 nodes in this particular network as shown in Figure 4.6.

Training process is the step in which the connection weights were optimized. There are a variety of algorithms available to train the neural network models, but in geotechnical engineering (Goh, 1994) it is a common practice to use the back-propagation algorithm. However, when the number of weights in a network is less than 300, Levenberg-Marquardt algorithm is proven to be effective. This method often performs considerably faster than other algorithms and finds better optima than other algorithms (Zurada, 2004). Hence, this algorithm has been adopted to train the above described network in this study.

After the training stage was completed, the functioning of the trained model was ascertained using the validation test data which has not been used during both training and testing stages. The idea behind validating the process is to guarantee the models predictionability for a generalized data (in the place of the training data) instead of memorizing the input/output relationships that are contained in the training data.

Figure 4.7 presents a correlation between the predicted and measured % minerals. In this research study, the mean absolute error (MAE) and the coefficient of determination (R^2) values were used to evaluate the performance of the developed model. The R^2 value for the selected ANN model is found to be 0.95, which indicates the capability of the model.

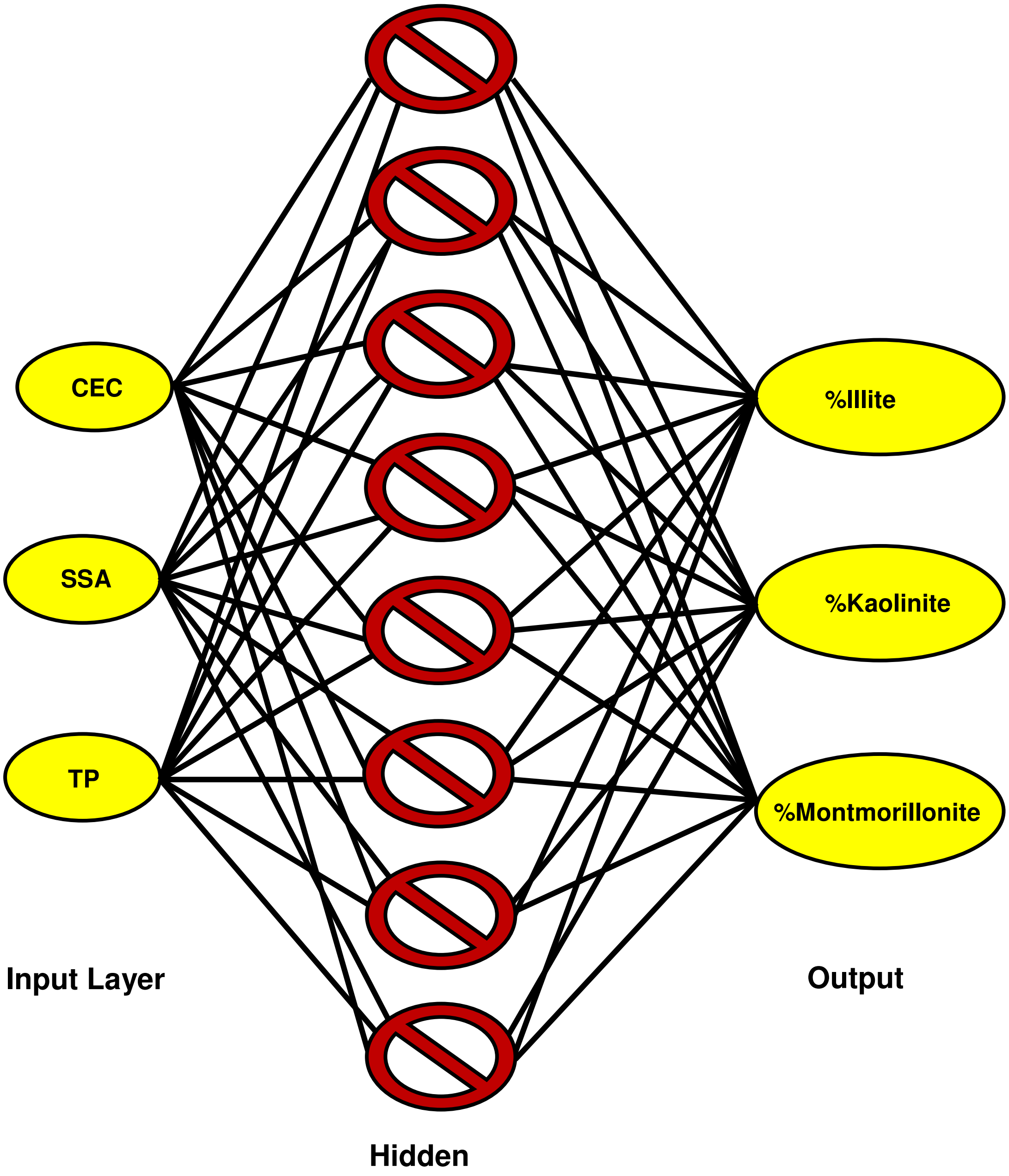
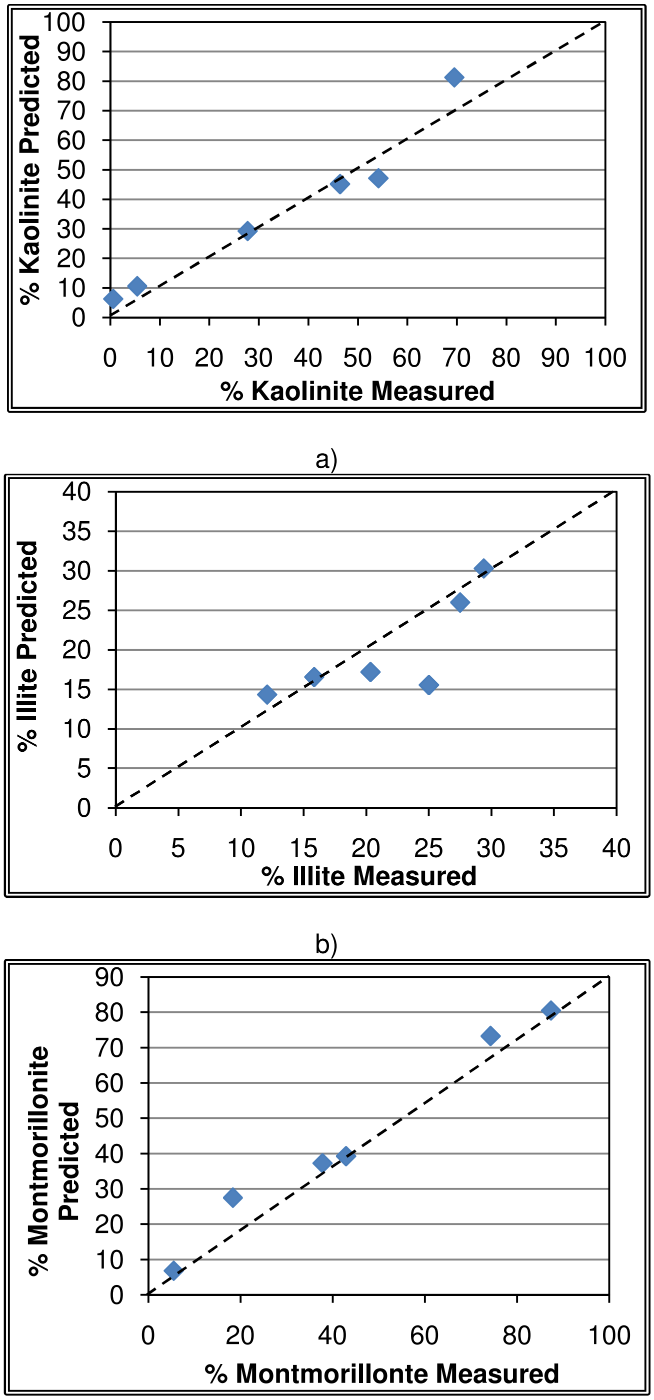


Figure 4.6 Optimized network architecture

Table 4.13 ANN's Input/Output Variable Statistics

Model Input Variables	**Mean**	**Standard deviation**
Cation exchange Capacity (CEC), meq/100 g		
Training set	84	41
Testing set	95	32
Validation set	105	22
Specific Surface Area (SSA), m^2/g		
Training set	225	106
Testing set	301	117
Validation set	283	99
Total Potassium (TP), %		
Training set	1.99	1.50
Testing set	1.94	1.37
Validation set	1.29	1.28
Model Output Variables	**Mean**	**Standard deviation**
%Kaolinite (%K)		
Training set	31	24
Testing set	27	26
Validation set	30	28
%Illite (%I)		
Training set	33	25
Testing set	16	16
Validation set	21	15
%Montmorillonite (%)		
Training set	35	19
Testing set	57	30
Validation set	49	19



a)

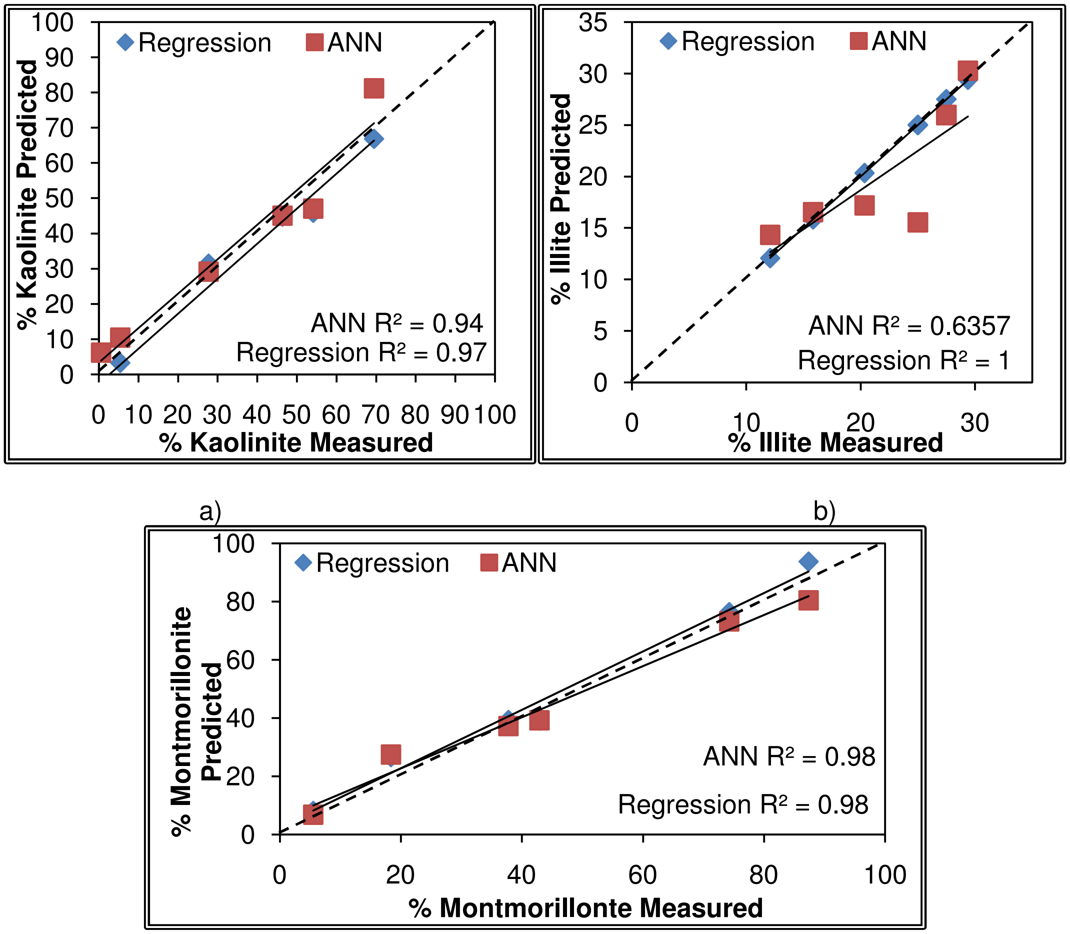
b)

c)

Figure 4.7 Variation of ANN predicted % minerals with measured % minerals
a) Kaolinite mineral b) Illite mineral c) Montmorillonite mineral

4.4.3 Comparisons of the models

The prediction models developed using both regression analysis and artificial neural networks are compared to assess the efficiency of the models in predicting clay minerals. Figure 4.8 shows the plots showing the relative comparison between the predicted and the measured percent clay minerals using both the methods for all the three minerals. For these plots it can be observed that the statistical model developed using regression equations provided better prediction capabilities than that of the artificial neural network model.



a) b)

c)

Figure 4.8 Comparisions between the Regression and ANN predicted % minerals with measured % minerals a) Kaolinite mineral b) Illite mineral c) Montmorillonite mineral

4.5 Summary

In this chapter, models to quantify clay minerals in the fine fraction of the soil sample are presented and analyzed. First, the repeatability of chemical test methods is demonstrated. The proposed method to determine the percentage of clay minerals in a given soil by solving simultaneous linear equations with the help of Microsoft Excel® solver function is explained in detail. The advantages and limitations of this method are highlighted. Then the mineralogy database of different soils tested here is presented. This method is validated by preparing few artificial soil specimens with known percentage of minerals and obtaining the mineral percentages by using the developed method.

Prediction models were also developed using the data obtained from the proposed quantification procedure of clay minerals in a given soil. This data was segregated into two parts model development data and model validation data such that both the data contain all the patterns to represent the entire data set. Tools such as regression analysis and artificial neural networks were used to develop the prediction models. Brief introduction of each of the methods was given and then the approach followed in this research is explained. The methods are evaluated using the validation data and the results were analyzed by plotting both comparisons between measurements and predictions. Overall, it is observed that regression equations are slightly better in the prediction capabilities than ANN models used here.

CHAPTER 5

WETTING/DRYING (W/D) DURABILITY STUDIES

5.1 Introduction

This chapter presents the test results and comprehensive analysis of these test results conducted on eight selected soils to evaluate the effects of clay mineralogy on the long-term performance of the stabilized soils. Wetting/drying studies were conducted primarily to simulate the seasonal moisture fluctuations that might have transpired during summer and winter seasons. Several past studies have also been performed on the wet-dry cycle related tests to address the durability issues and performance of stabilizers in arid conditions where such moisture fluctuations occur.

Hoyos et al. (2005) performed a series of wet and dry cyclic tests on different types of chemically-treated sulfate soils to evaluate strength, stiffness and volume change property variations with respect to seasonal cycles. An attempt was made to review these studies and follow a test protocol that could best simulate field environmental conditions in arid environments such as north Texas. This test protocol was used to study the efficacy of the chemical treatment to provide durable stabilization under seasonal moisture fluctuations. Details of the test procedure followed in this research study were discussed in Chapter 3.

5.2 Selected soils and stabilizer design

Eight different soils with various types and amounts of clay mineralogy were selected from different regions of the state of Texas and these soils were studied in this research to understand the effects of clay mineralogy on the long-term durability of the stabilized expansive clays. Preliminary tests were conducted on these soils to obtain their gradation curves, maximum dry densities, optimum moisture contents and plasticity indices and these results were presented in Chapter 3. Clay mineral quantification studies were conducted on these soils according to the procedure explained in section 3.6 and these results are given in Chapter 4.

Also, stabilizer mix design was performed on these soils as per the current TxDOT stabilizer design procedures (Figure 2.14), which are explained in Chapter 2. Stabilizer types and amounts used are presented in the following section. It should be mentioned here that the final stabilizer design and dosages of a select present soils (Pharr A and Pharr B) were finalized based on the local experience of the district from which these soil samples were collected.

Table 5.1 presents the soil test results, type and amounts of the stabilizer selected along with the dominating clay mineral determined for all eight soils considered in this research. Traditionally the lime was used for stabilizing clayey soils. However, there were instances where cement has been used to stabilize expansive clays. Hence, cement stabilization was also considered on two test soils to study and compare the effect of stabilizer type to provide durable treatments.

Table 5.1 Selected soils and their stabilizer design details along with the dominating clay mineral

S No.	Soil	Classification USCS	Plasticity Index, %	Dominating clay mineral	Type of additive	Amount of additive, (% by weight)
1	Austin	CH	34	Montmorillonite	Lime	6%
2	Bryan	CH	31	Kaolinite	Lime	8%
3	El Paso	CL	16	Illite	Lime	8%
4	Fort Worth	CH	29	Montmorillonite	Lime	6%
5	Keller	CL	11	Kaolinite	Lime	6%
6	Paris	CH	36	Montmorillonite	Lime	8%
7	Pharr-A	CH	45	Montmorillonite	Lime	4%
8	Pharr-B	CH	37	Kaolinite	Lime	3%
9	Pharr-A	CH	45	Montmorillonite	Cement	4%
10	Pharr-B	CH	37	Kaolinite	Cement	3%

5.3 Durability studies

Wetting/Drying studies were conducted on all soil specimens following the procedure explained in Chapter 3. Duplicate specimens were used for studying each variable. A total time of 48 hours was needed to complete one cycle of wetting/drying and hence forty two (42) days were needed to complete a total number of 21 cycles on each soil type. Also, after 3, 7, 14 and 21 cycles, the select soil specimens were then tested for UCS testing and this soil could not be used for any further studies once they were already subjected to chemical treatment. Hence, a total of eight identical specimens were tested for each durability assessment study. Test results from these durability studies are presented in the following sections.

5.3.1 Repeatability of the tests

To check the repeatability of the test methods in providing similar or repeatable results, triplicate soil specimens were tested for two soil types (Austin Clay and Bryan Clay). The mean and standard deviations of the volumetric strain changes and unconfined compressive strengths of these specimens after 3 and 7 W/D cycles were determined and presented in Table 5.2. It can be observed from this table that the standard deviations of test results were low and the coefficients of variation of most of the tests were also below 20%, indicating that both volume change and strength tests were repeatable.

Table 5.2 Repeatability of the wetting/drying test method

Soil Type : Austin						
	Trial 1	**Trial 2**	**Trial 3**	**Mean**	**SD**	**cov, %**
Volumetric strain after 3 W/D cycles, %	3.77	4.50	3.26	3.84	0.62	16.22
Volumetric strain after 7 W/D cycles, %	5.12	4.60	6.15	5.29	0.79	14.91
UC strength after 3 W/D cycles, psi	10.30	14.60	12.00	12.30	2.17	17.61
UC strength after 7 W/D cycles, psi	10.00	6.50	5.60	7.37	2.32	31.55
Soil Type : Bryan						
	Trial 1	**Trial 2**	**Trial 3**	**Mean**	**SD**	**cov, %**
Volumetric strain after 3 W/D cycles, %	1.1	0.85	0.94	0.96	0.13	13.14

Table 5.2 Continued

Volumetric strain after 7 W/D cycles, %	1.4	1.63	1.9	1.64	0.25	15.23
UC strength after 3 W/D cycles, psi	240	180	220	213.33	30.55	14.32
UC strength after 7 W/D cycles, psi	150	200	170	173.33	25.17	14.52

cov - Coefficient of variation
SD - Standard deviation
* - Measured after wetting cycle

5.3.2 Effect of curing methods

The current TxDOT procedure for curing and moisture conditioning (Tex 121-E) of a stabilized soil specimen will take approximately seventeen days to complete the whole process. The method comprises of seven days of curing over the counter in room temperature conditions, followed by 6 hours of oven drying at 104° F and then saturating the same specimen by capillary suction for ten days. In-order to shorten this time frame for curing and moisture conditioning, a new methodology was developed. This method reduced the number of days of curing by increasing the curing temperature and by reducing the time required for moisture conditioning by saturating the specimens via back pressure saturation in the place of capillary suction. Two different curing temperatures of 104° F and 140° F were considered and investigated for this study.

Soil specimens were prepared at OMC using the static compaction method as explained in the section 3.6.1 of chapter 3. These specimens were then cured in an oven set at 104º F and 140º F and then moisture conditioned using a back pressure saturation method for one day and then the specimens were subjected to unconfined compression (UC) strength tests. The soil specimens were tested immediately after moisture conditioning i.e. the moisture level in specimens was close to saturation during the UC test.

Figure 5.1 and Figure 5.2 presents the comparison of the stress - strain plots obtained by conducting unconfined compression tests on soil specimens cured at room temperature (Tex121-E method), 104° F and 140° F for Bryan and Fort Worth soils, respectively.

Figure 5.3 summarizes these results for all the four soils studied which included El Paso and Paris soils. It can be observed from this figure that the samples cured at 140º F had lower strength than those cured at 104º F. The higher temperatures seem to negatively impact the integrity of several clays and their specimens studied here. High curing temperatures has drawn considerable amount of moisture out of the soil specimen rapidly, which might have compromised the curing process of the lime-clay mixture. As a result, these treated soils exhibited lower strengths. Hence it is recommended that the soil samples be cured at 104º F for two days inside an oven.

Figure 5.4 and Figure 5.5 presents the comparison of the stress vs strain plots obtained by conducting unconfined compression tests on soil specimens cured by Tex121-E, AC-1 and AC-2 methods for two typical soils Bryan and Fort Worth respectively.

Figure 5.6 summarizes these results for all the four soils studied in this research. In AC-1 and AC-2 methods, the samples were cured in the oven at 104º F temperature for 1 and 2 days, respectively and then saturated under back pressure conditions for 1 day. From this figure it can be noted that in both AC-1 and AC-2 methods resulted in the failure of the soil specimens at lower strengths.

The high strength measured in the case of a 17-day moisture conditioning and curing method was attributed to lack of moisture migration to the top of the soil specimen through capillary saturation and thereby the soil specimens were not saturated. Hence the specimens were dryer and as a result, exhibited higher strengths. In the case of AC-1 and AC-2 methods, the samples were completely saturated by back pressure saturation hence the specimen was wet and soft. It can be observed from the Figure 5.6 that the soil specimens cured by AC-2 method had slightly lower strengths than those cured by AC-1 method. Hence, among AC-1 and AC-2 methods, the AC-1 method which recommends two days of oven curing is selected for further studies as the AC-2 method needs only one day of oven curing and this time period is considered short for the pozzalonic reactions to occur between the additive and soil. Hence, the curing methods, Tex-121-E and AC-1 were used on all four selected soils to assess the effects of curing methods on the long-term performance of stabilized expansive clays. The four soils selected for this study were Bryan, El Paso, Paris and Fort Worth. The Bryan soil was dominant with mineral Kaolinite whereas El Paso soil was dominant with mineral Illite. Other two soils Paris and Fort Worth contain high amounts of Montmorillonite mineral. Test results were obtained by performing wetting/drying studies on soil specimens under both curing methods and these test results are presented in the following.

Figure 5.7 through Figure 5.10 presents typical plots of vertical strain versus time for Bryan and Fort Worth soil specimens cured by both standard (Tex 121-E) and accelerated (AC-1) methods. Figure 5.11 through Figure 5.14 present the comparison of volumetric strains versus the number of cycles for both standard cured and accelerated cured soil specimens of all the four soil types. Figure 5.15 through Figure 5.18 present comparisons of unconfined compression strengths versus the number of cycles for the same specimens under curing conditions.

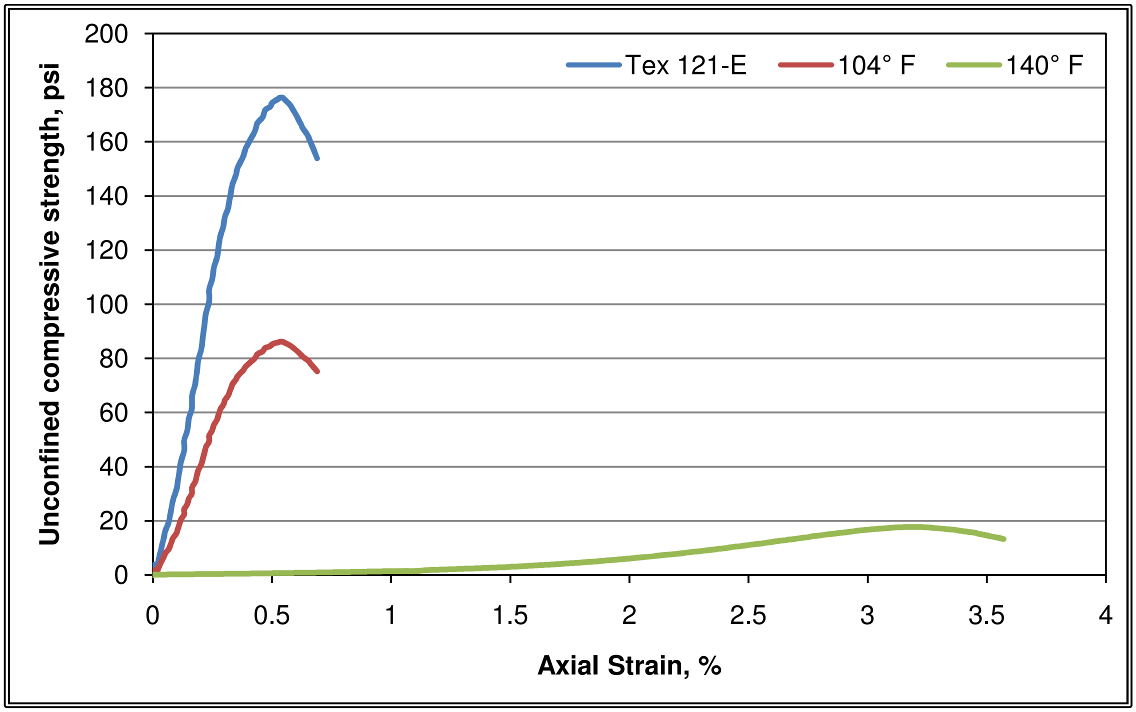


Figure 5.1 Stress-strain plots for Bryan soil specimens cured at different temperatures

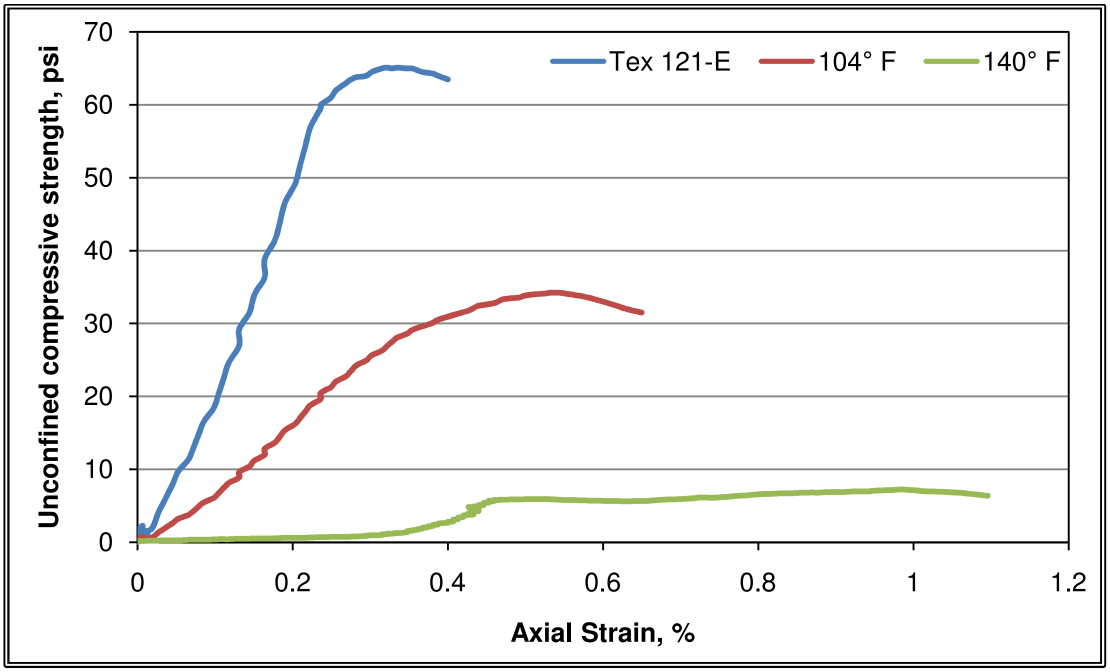


Figure 5.2 Stress-strain plots for Fort Worth soil specimens cured at different temperatures

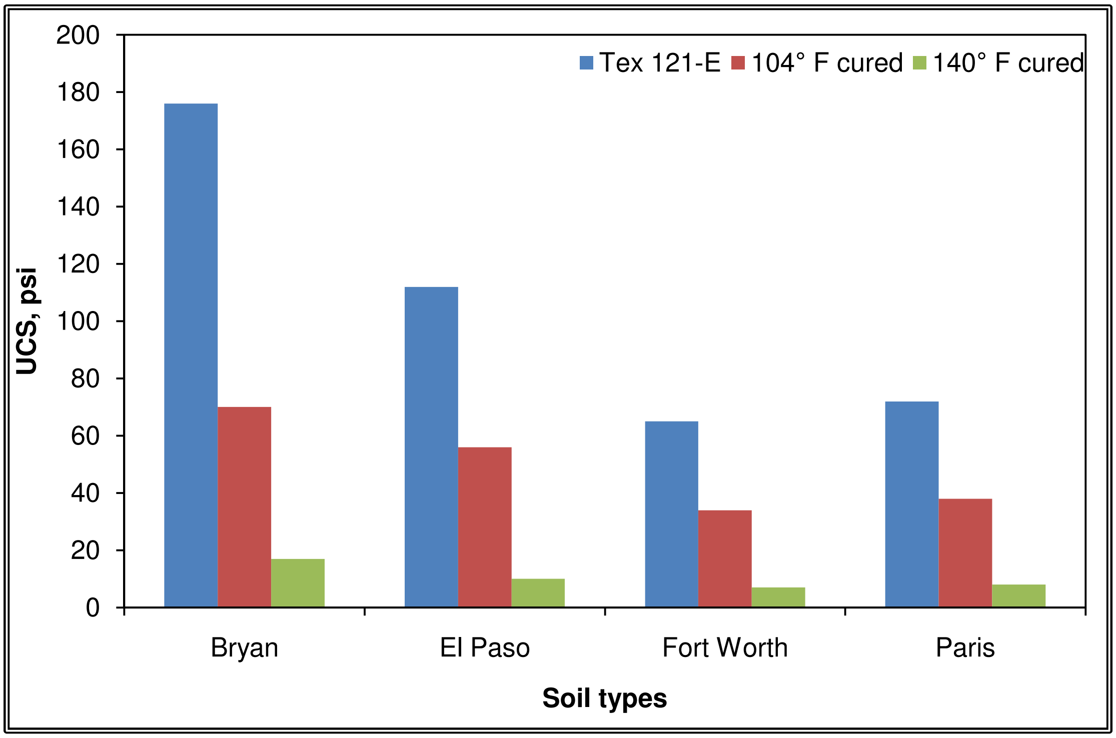


Figure 5.3 Effect of curing temperature on the UC strength of all the four soils

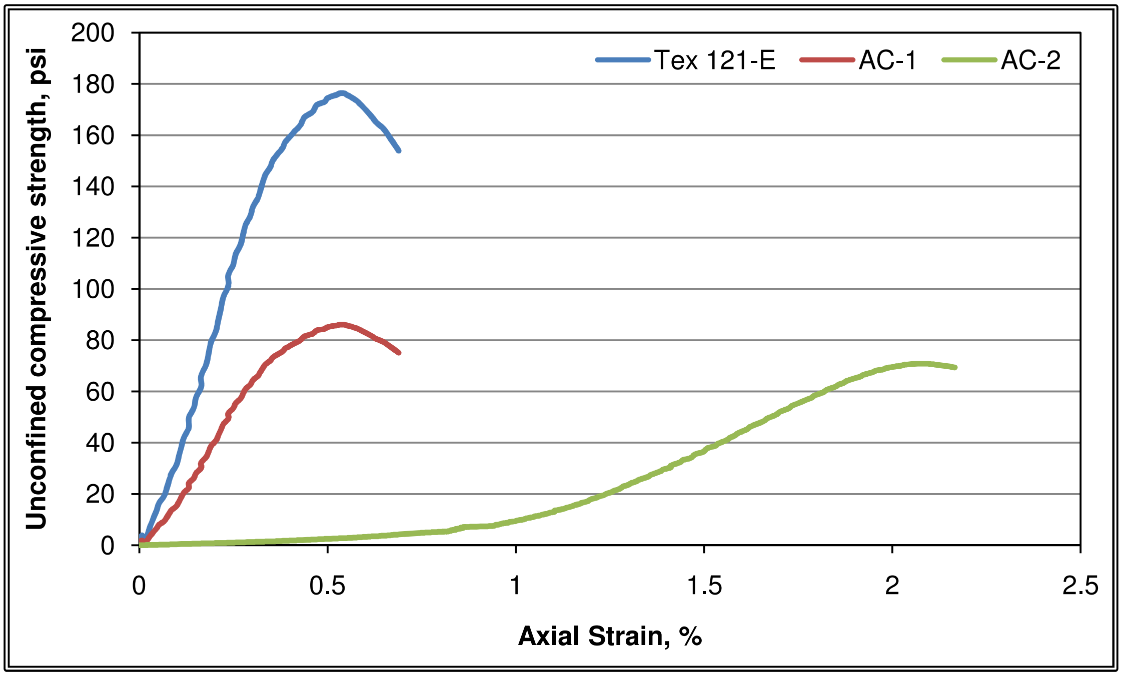


Figure 5.4 Stress-strain plots for Bryan soil specimens cured for different time periods

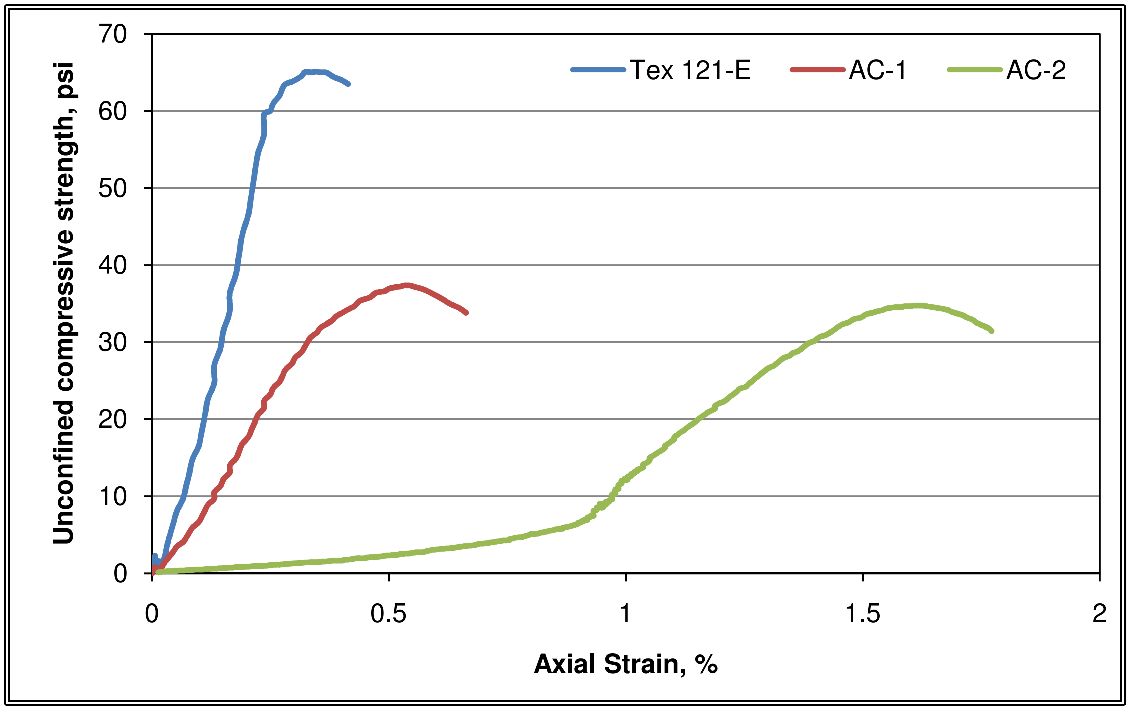


Figure 5.5 Stress-strain plots for Fort Worth soil specimens cured for different time periods

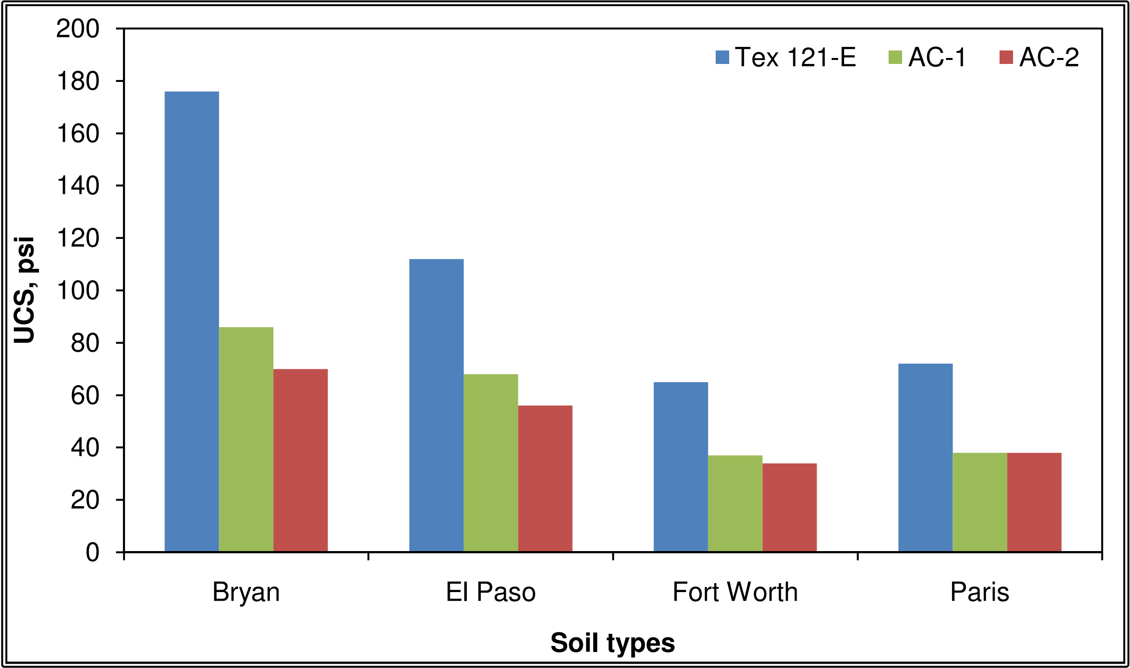


Figure 5.6 UC strengths for different curing and moisture conditioning procedures for four soils selected

From these figures, it can be noted that the volumetric strain is slightly higher in the accelerated cured specimens in all soils. However, this difference was not appreciable (<2% difference) and it can be noted here that there is no considerable difference in volume changes and strengths of the soils under both curing methods. A few soils lasted all twenty one (21) cycles under both curing methods while others showed premature failure after undergoing similar number of W/D cycles. In the case of UCS results, Tex-121 curing always resulted in high strength values as this method always lead to lesser saturations, which are difficult to estimate as soils achieve partial saturations due to capillary rise of water during saturation phase. As a result, partial saturation and mixed strength values obtained in these soils. With the accelerated curing, full saturation was achieved in all soils, which resulted in lesser strengths in these soils.

Because of the consistent high saturation and less time for curing, the accelerated curing method was used in the rest of the experimental studies. Wetting/drying studies were conducted on all eight soils under the accelerated curing and their results are presented in this section.

Figure 5.19 and Figure 5.20 present the pictorial representation of the changes in the soil sample with respect to wetting/drying cycles for untreated and treated Austin soil samples respectively.

Figure 5.21 and Figure 5.22 present the pictorial representation of the changes in the soil sample with respect to wetting/drying cycles for untreated and treated Paris soil samples respectively. The Austin and Paris soils showed a considerable distress and then collapse after their treatment with the lime stabilizer. Similar behavior was observed in Fort Worth, Pharr-A and Pharr-B soils. Soils from Bryan, El Paso and Keller lasted all twenty one cycles and the typical pictorial representations of untreated and treated Keller soil specimens are presented in Figure 5.23 and Figure 5.24, respectively.

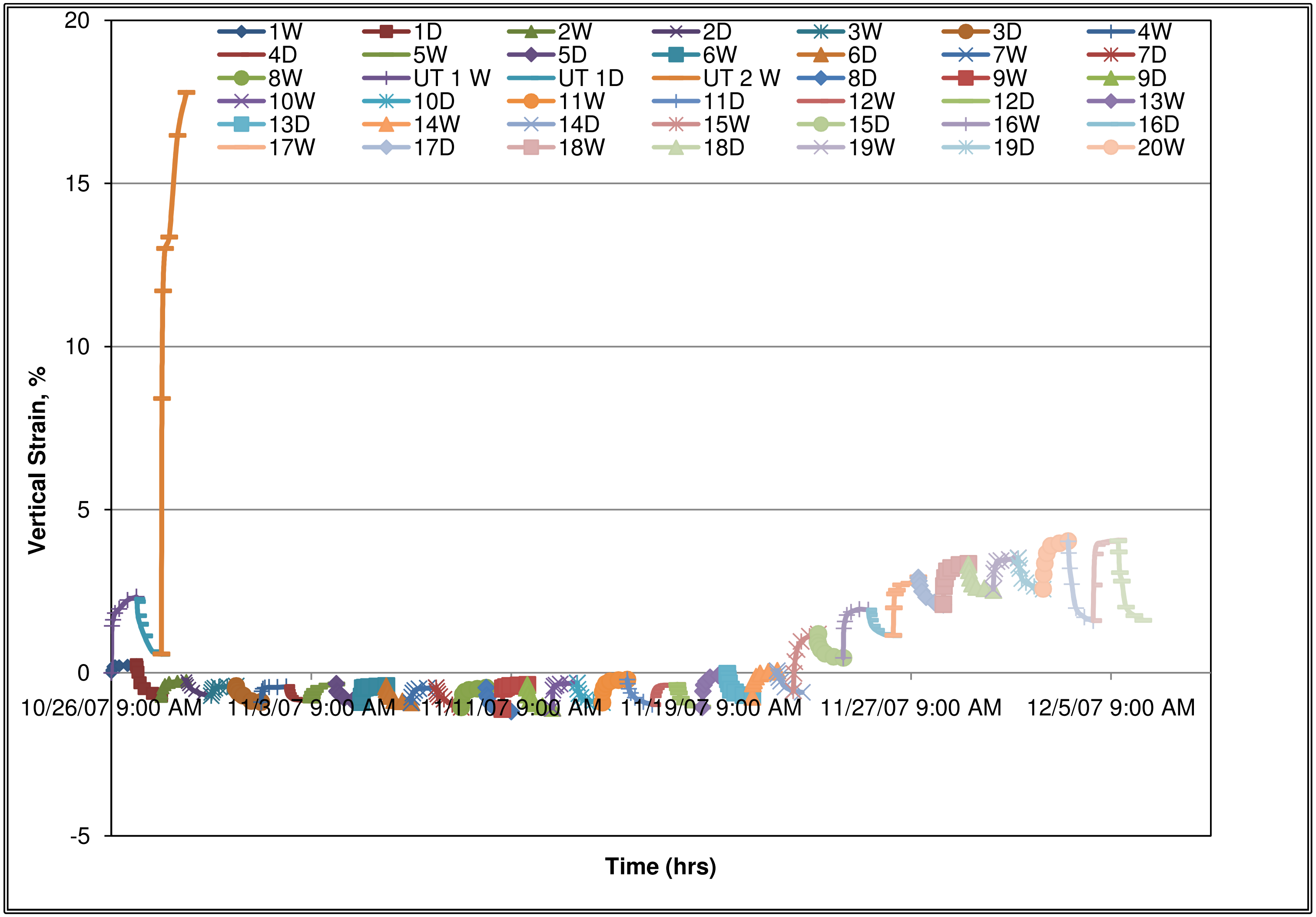


Figure 5.7 Vertical strain vs time plot for control and treated Bryan soil specimens cured by Tex 121-E method

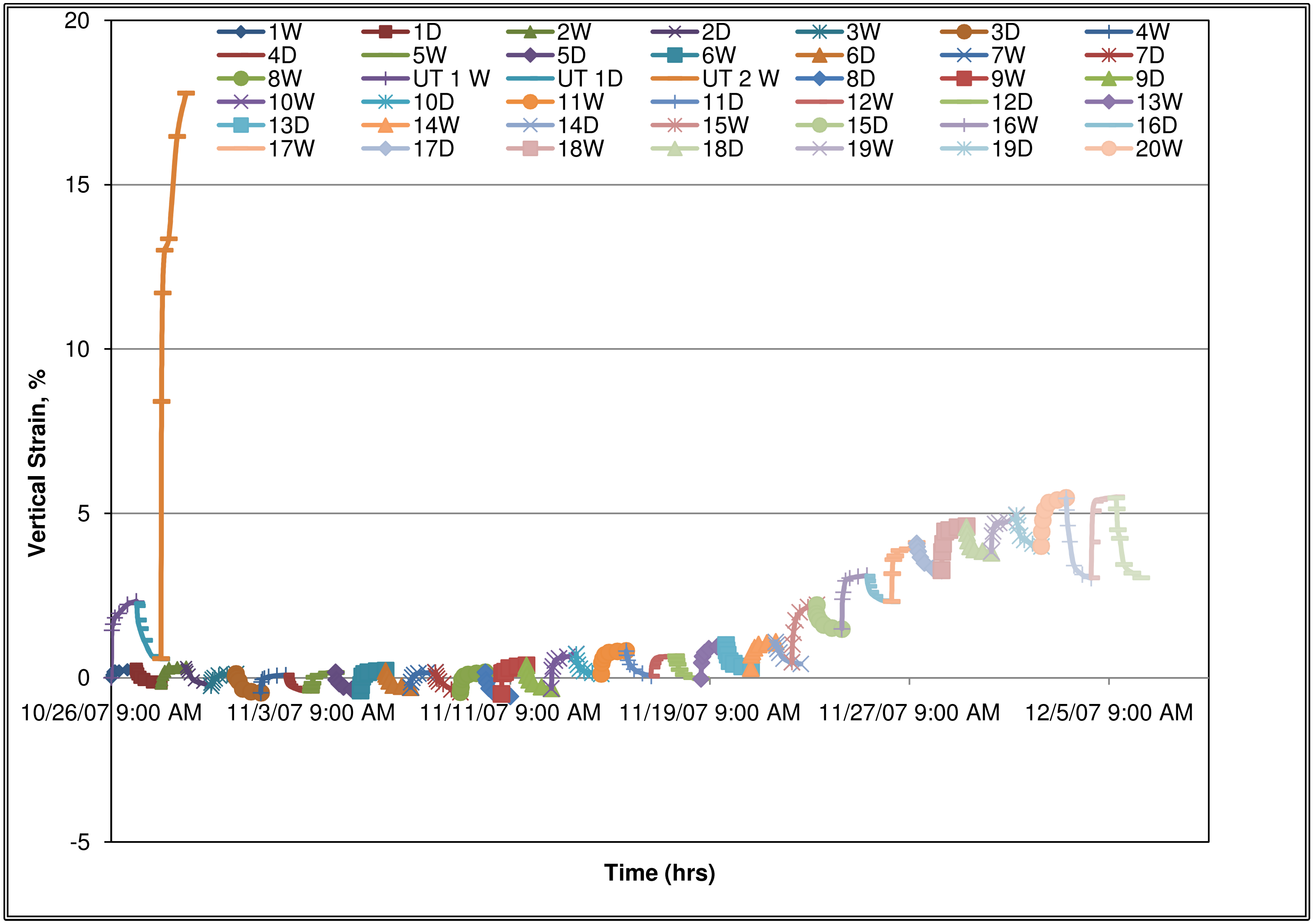


Figure 5.8 Vertical strain vs time plot for control and treated Bryan soil specimens cured by AC-1 method

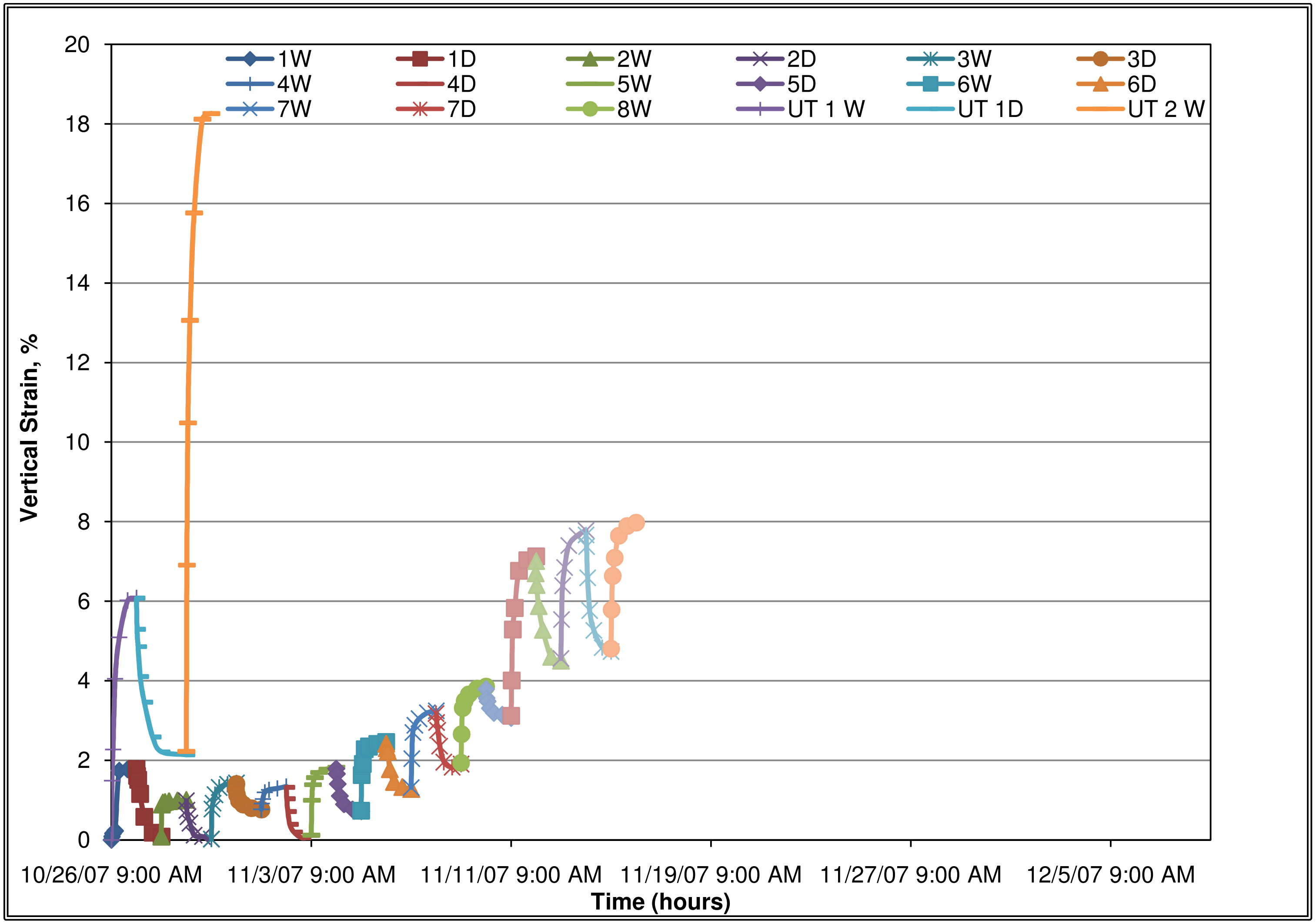


Figure 5.9 Vertical strain vs time plot for control and treated Fort Worth soil specimens cured by Tex 121-E method

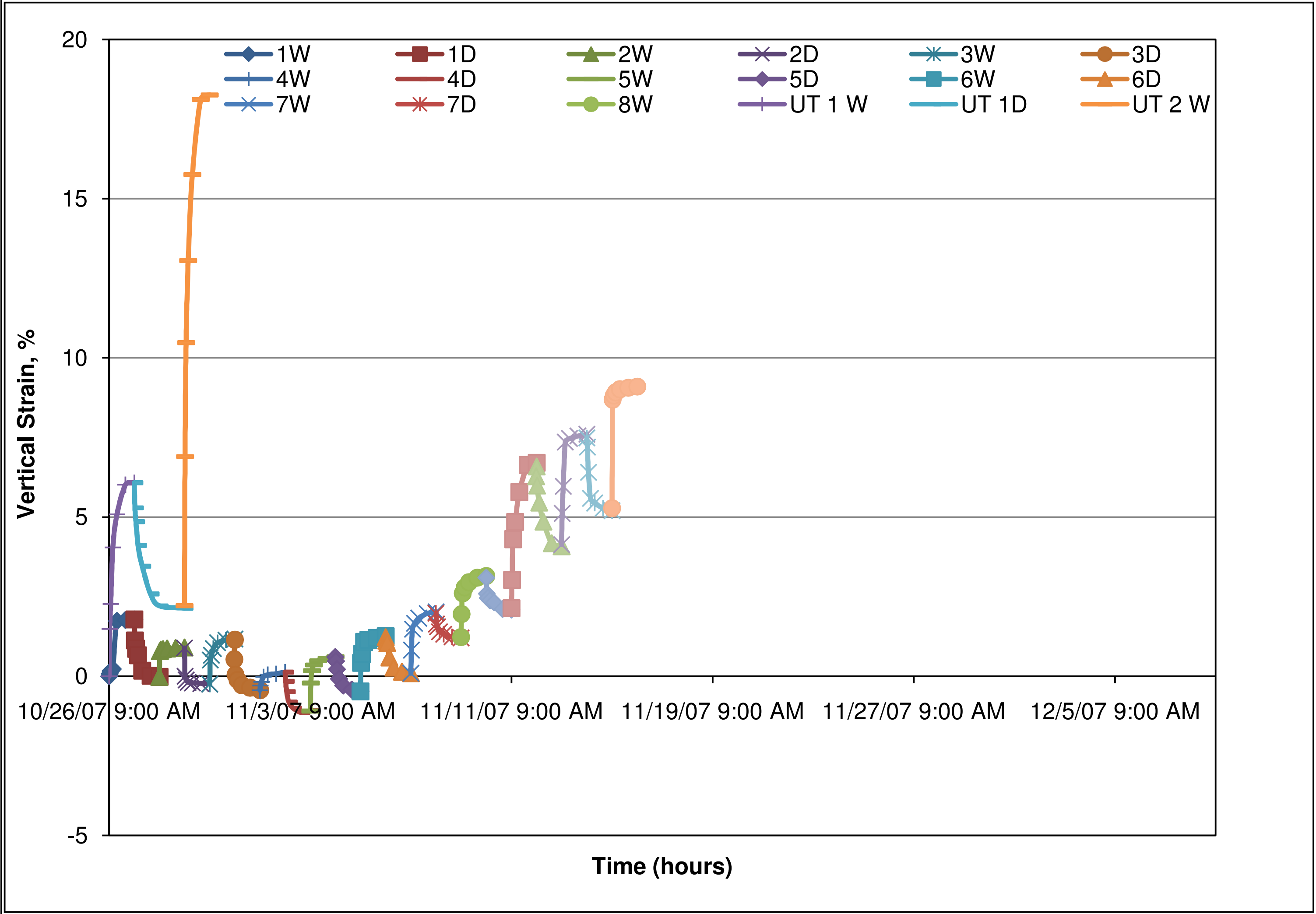


Figure 5.10 Vertical strain vs time plot for control and treated Fort Worth soil specimens cured by AC-1 method

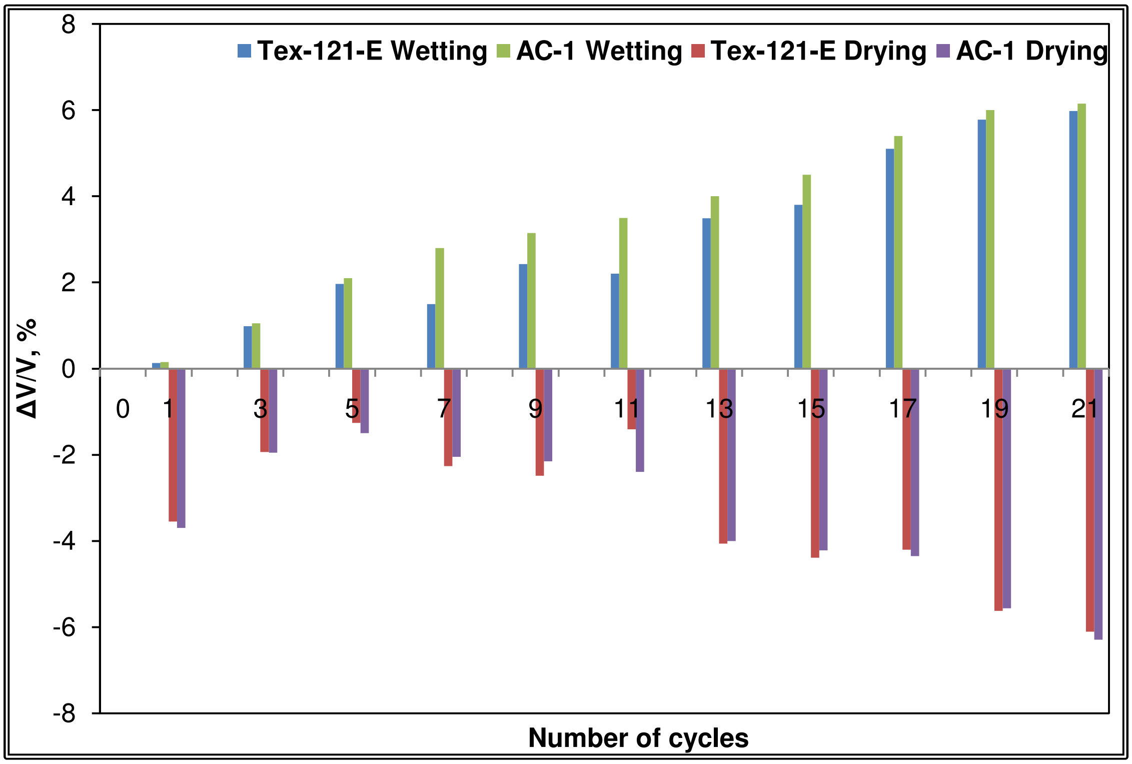


Figure 5.11 Comparison of volumetric changes between the curing methods for Bryan soil

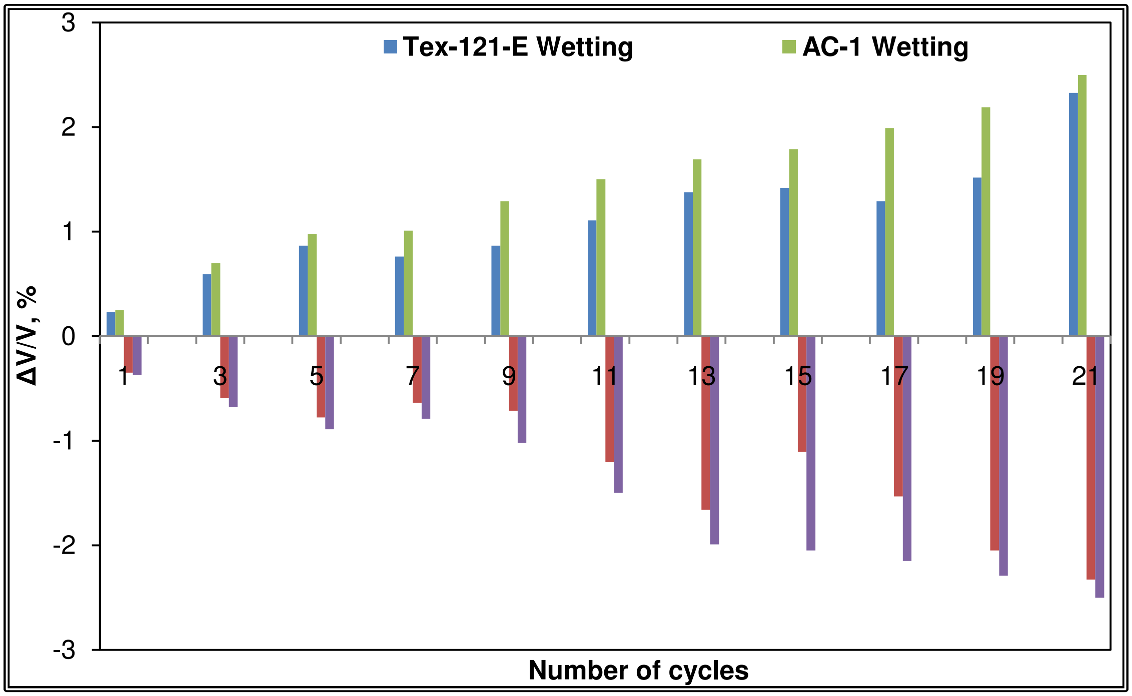


Figure 5.12 Comparison volumetric changes between the curing methods for El Paso soil

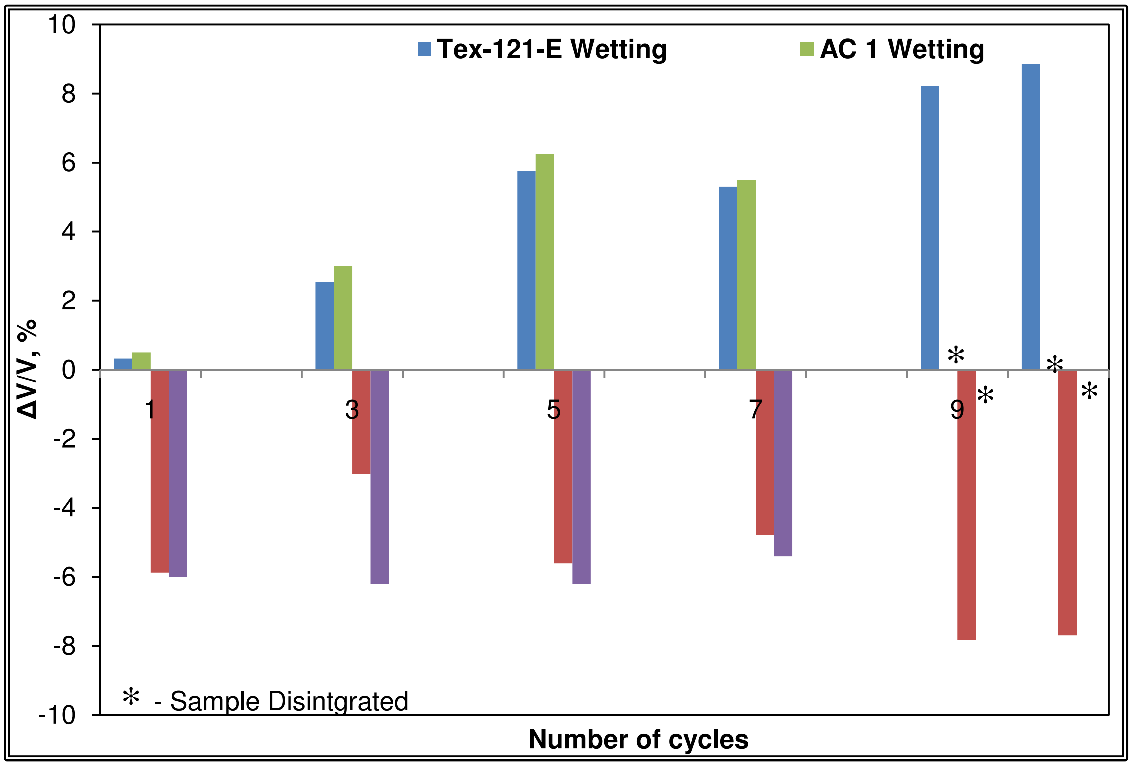


Figure 5.13 Comparison of volumetric changes between the curing methods for Fort Worth soil

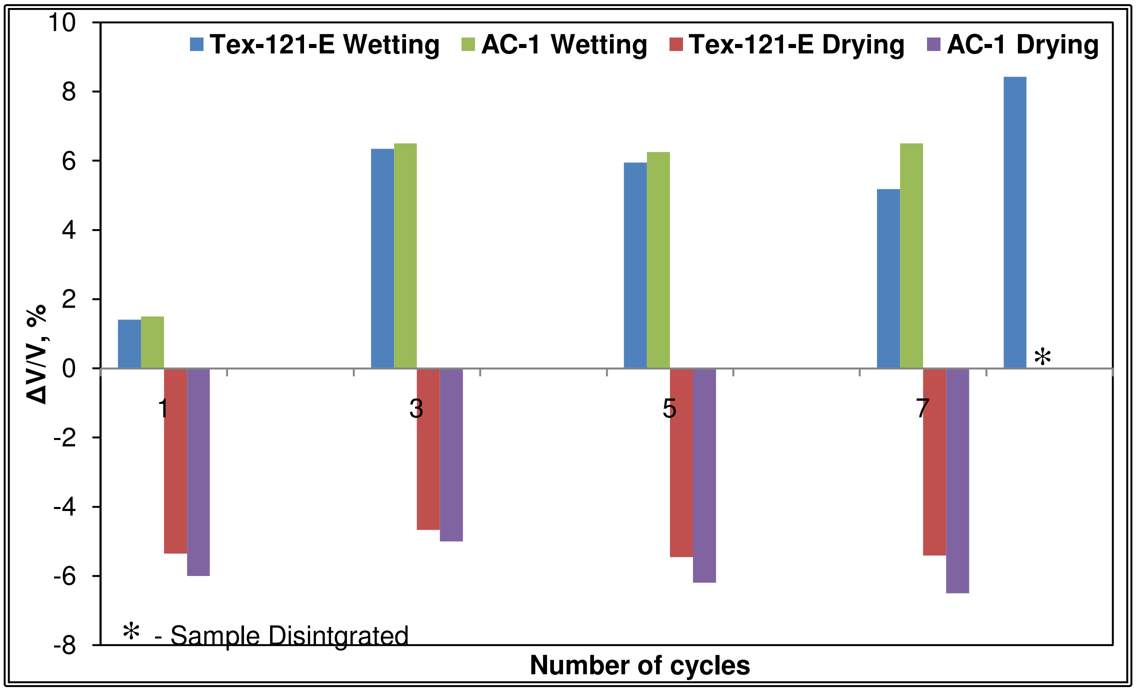


Figure 5.14 Comparison of volumetric changes between the curing methods for Paris soil

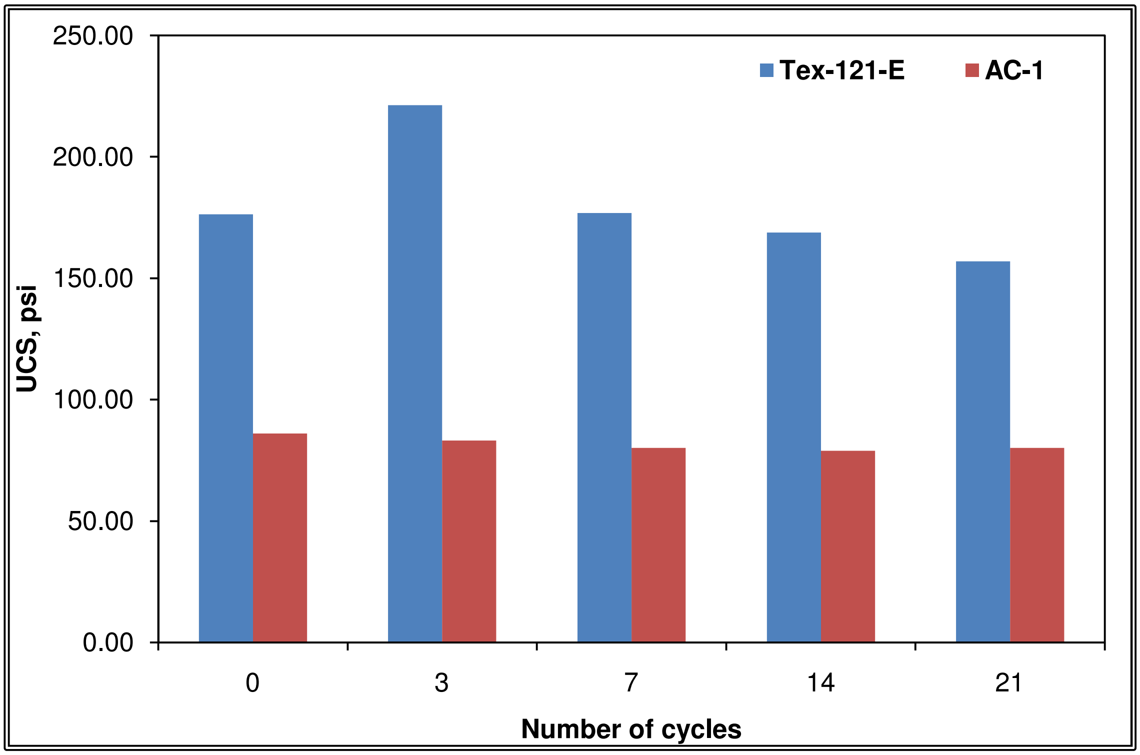


Figure 5.15 Comparison of UC strengths between the curing methods for Bryan soil

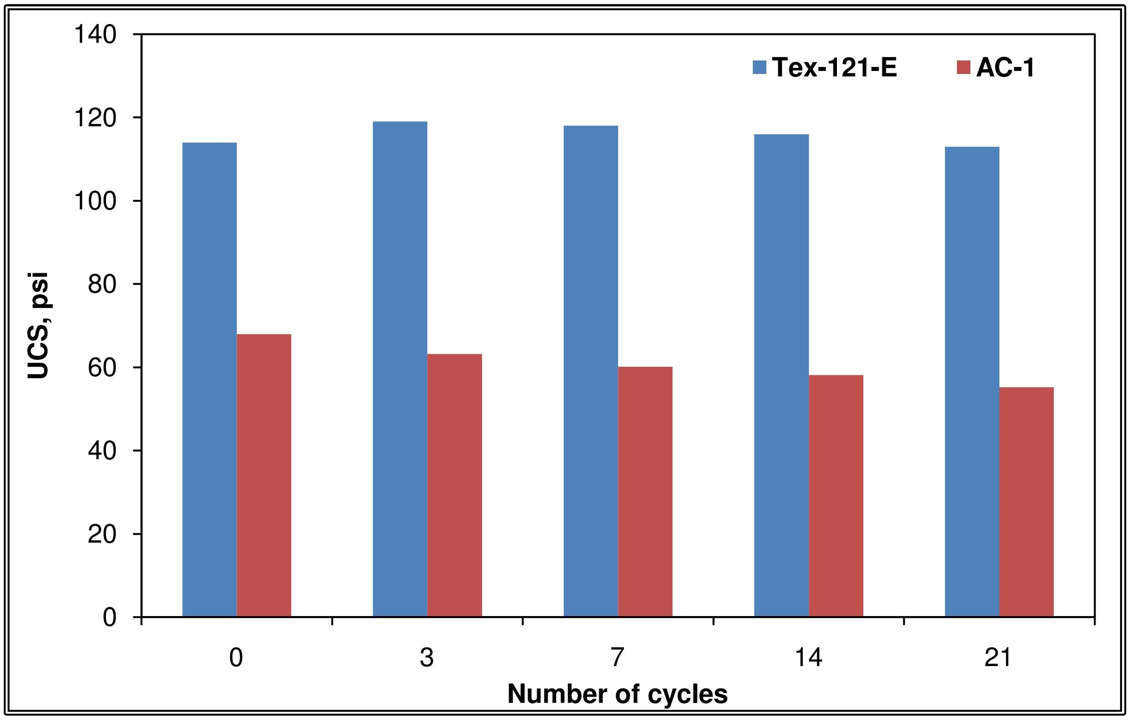


Figure 5.16 Comparison of UC strengths between the curing methods for El Paso soil

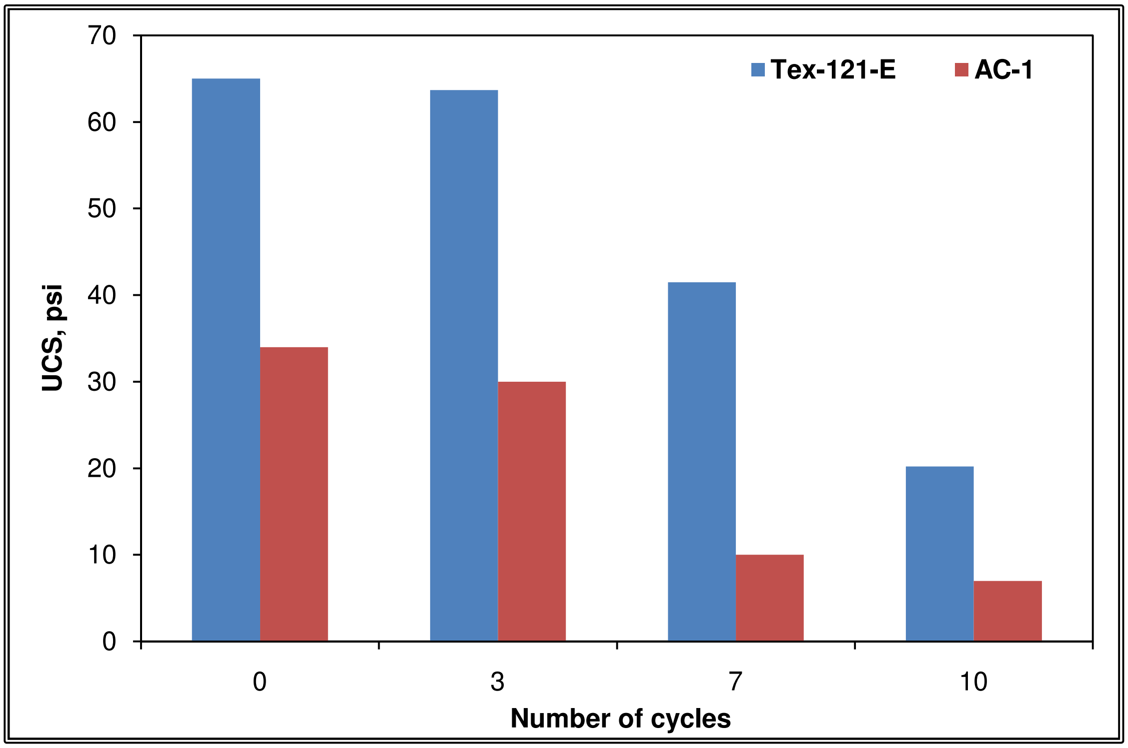
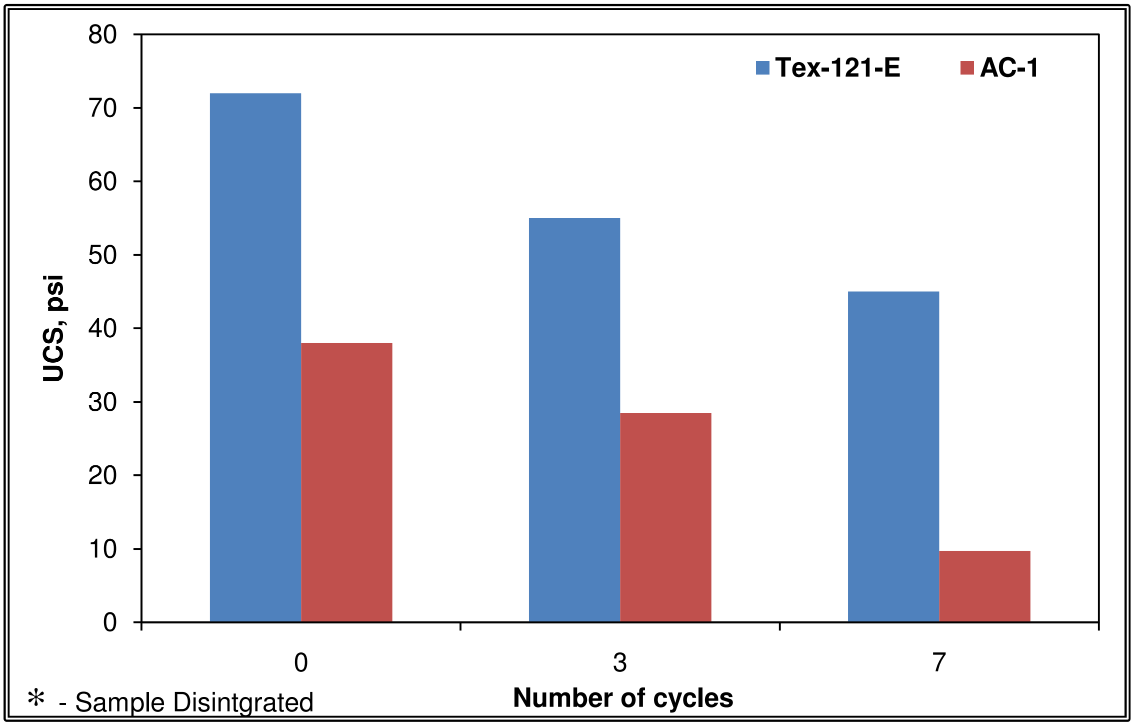
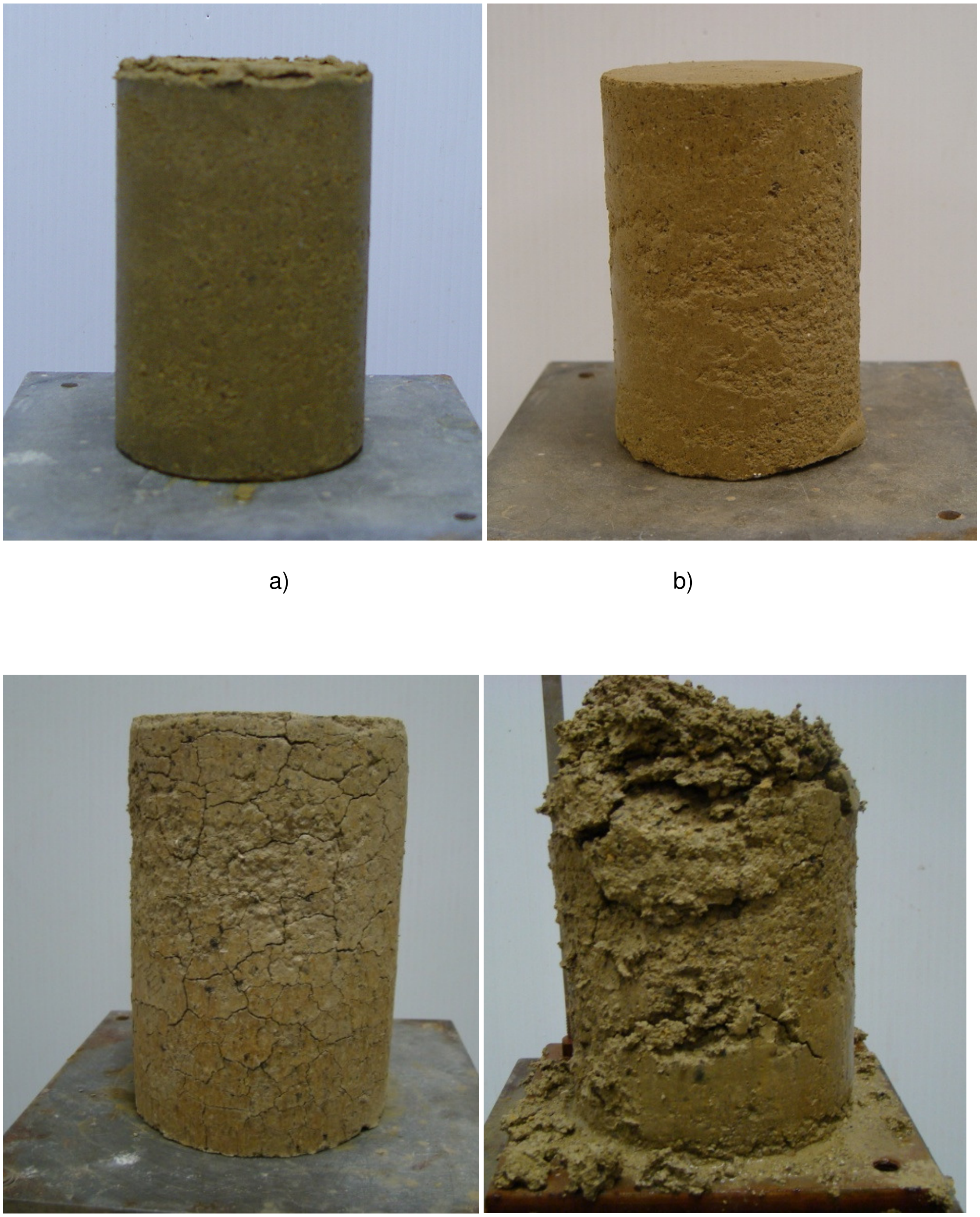


Figure 5.17 Comparison of UC strengths between the curing methods for Fort Worth soil



* - Sample Disintgrated

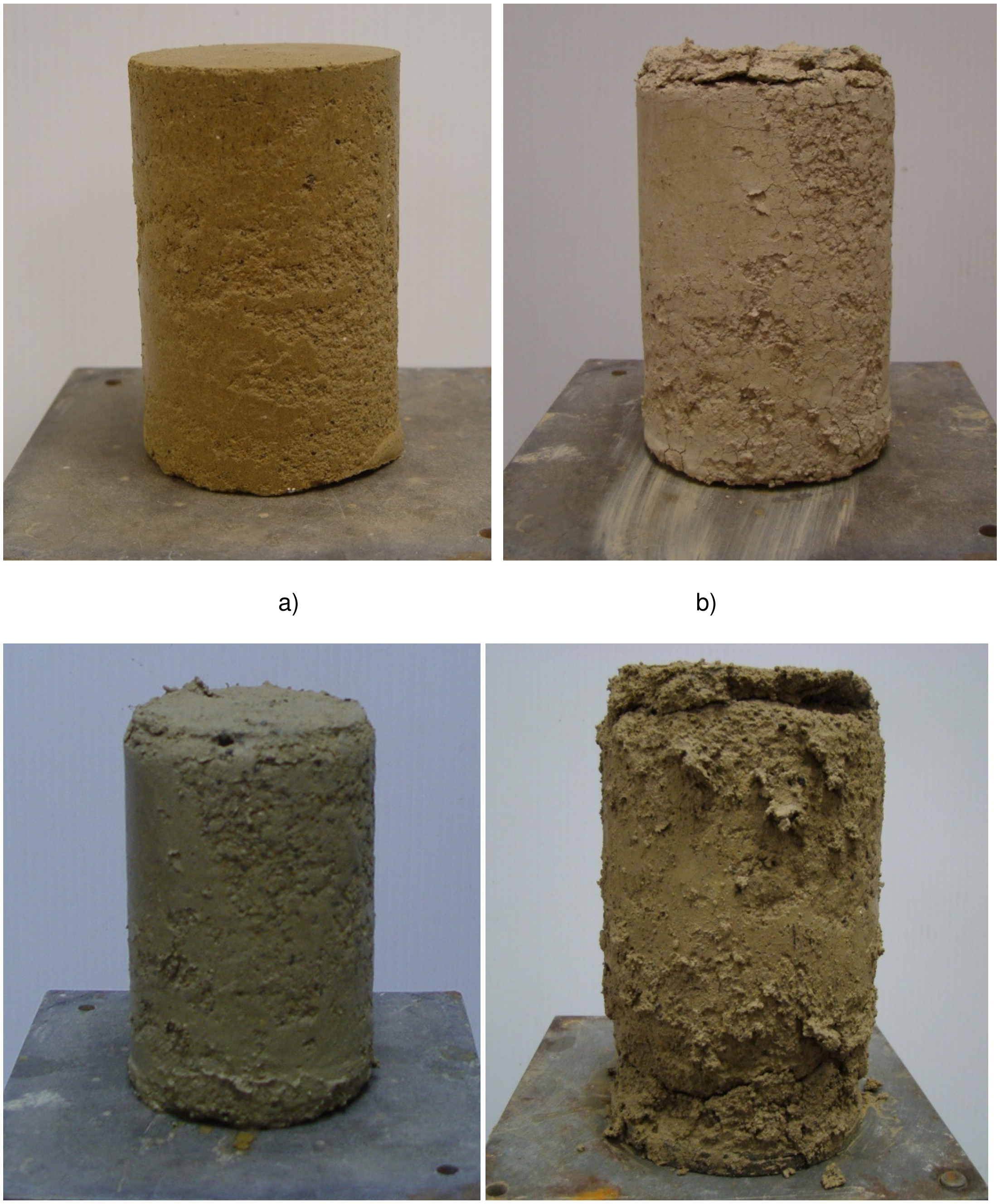
Figure 5.18 Comparison of UC strengths between the curing methods for Paris soil



a) b)

c) d)

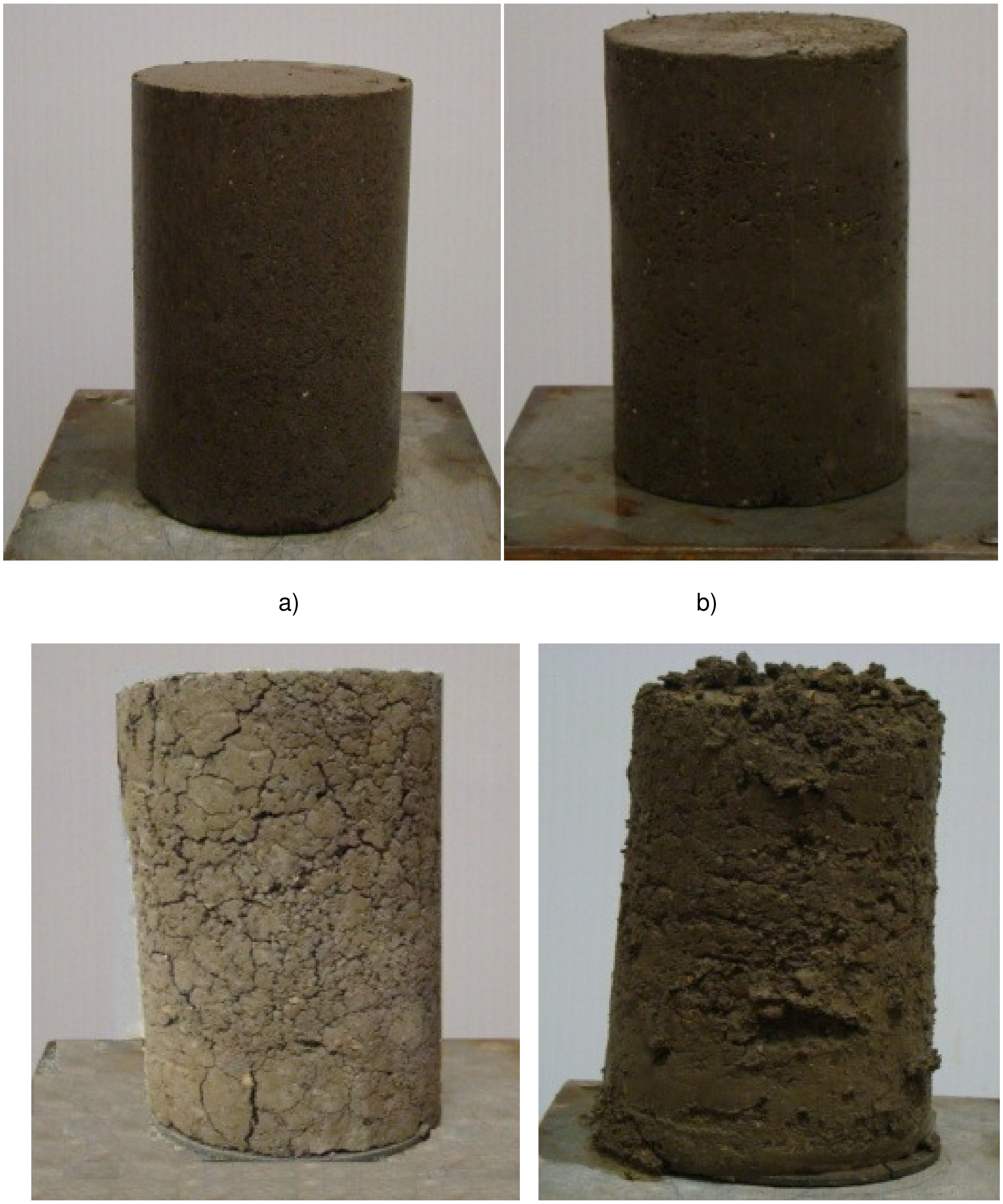
Figure 5.19 Changes in untreated Austin soil specimens with Wetting/Drying cycles
a) At start b) After 1wetting cycle c) After 1 drying cycle d) After 2 wetting cycles



a) b)

c) d)

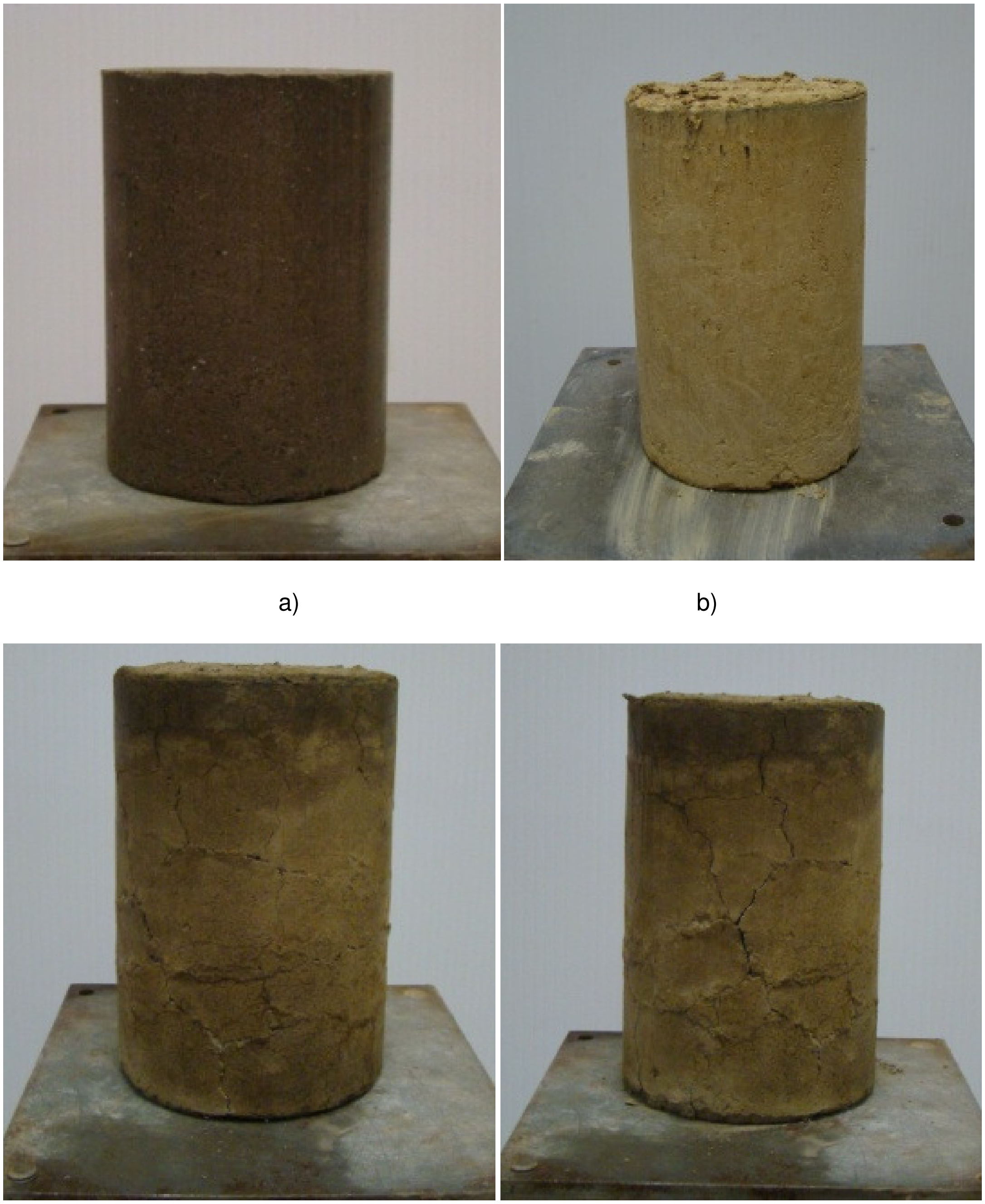
Figure 5.20 Changes in treated Austin soil specimens with Wetting/Drying cycles
a) At Start b) After 3 Cycles of W/D c) After 5 Cycles of W/D d) After 7 Cycles of W/D



a) b)

c) d)

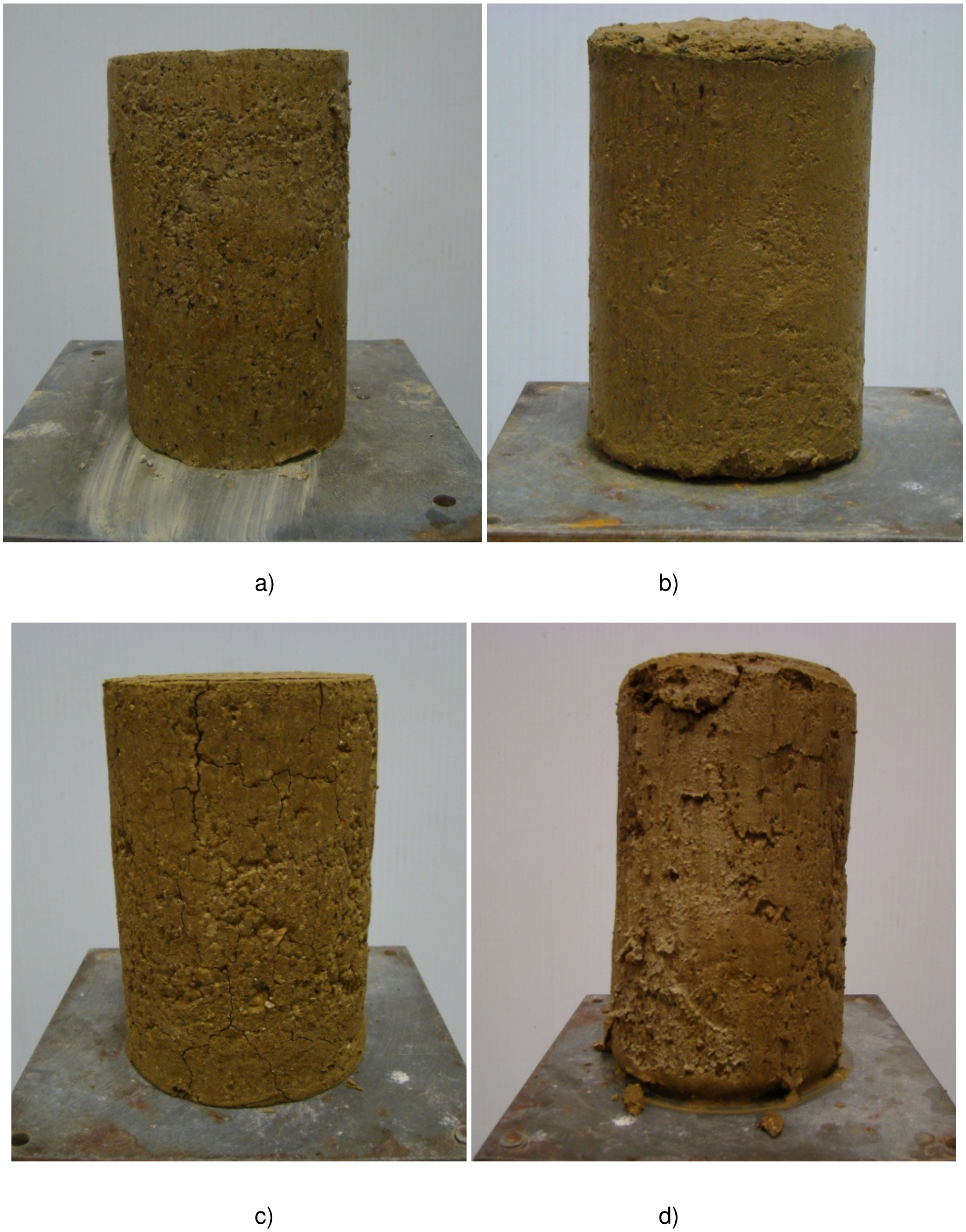
Figure 5.21 Changes in untreated Paris soil specimen with Wetting/Drying cycles
a) At start b) After 1wetting cycle c) After 1 drying cycle d) After 2 wetting cycles



a) b)

c) d)

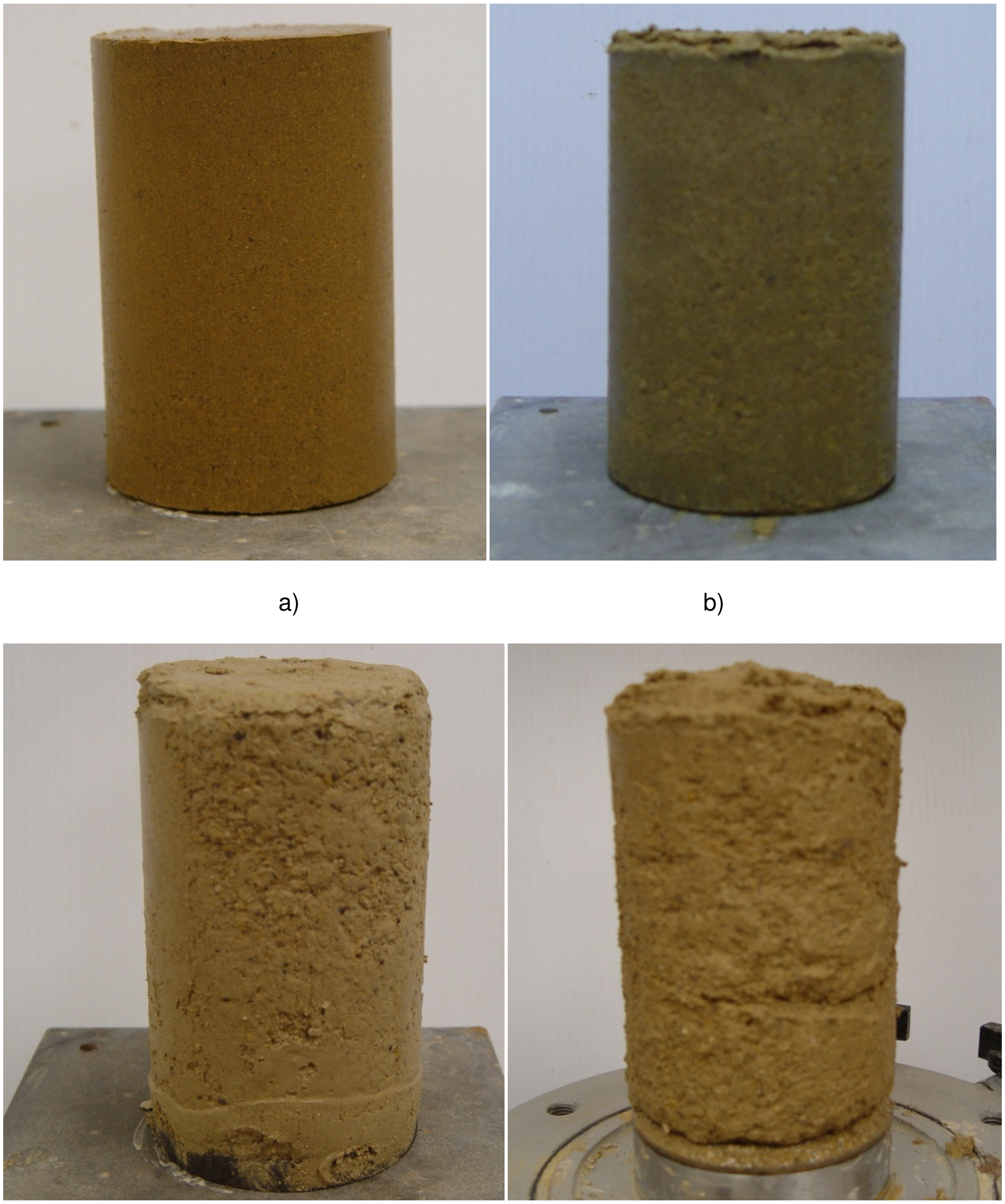
Figure 5.22 Changes in treated Paris soil specimens with Wetting/Drying cycles
a) At Start b) After 3 Cycles of W/D c) After 5 Cycles of W/D d) After 7 Cycles of W/D



a) b)

c) d)

Figure 5.23 Changes in untreated Keller soil specimens with Wetting/Drying cycles
a) At start b) After 1wetting cycle c) After 1 drying cycle d) After 2 wetting cycles



a) b)

c) d)

Figure 5.24 Changes in treated Keller soil specimens with Wetting/Drying cycles
a) At Start b) After 7 Cycles of W/D c) After 14 Cycles of W/D d) After 21 Cycles of W/D

5.3.3 Summary of W/D test results

The results of the wetting/drying studies conducted on all the eight selected soils are presented in the following sections. The soils were grouped into three groups based on their mineral dominance in their clay fraction. Group A consisted of four soils from Austin, Fort Worth, Paris and Pharr A (high PI) whose clay fraction was dominated by the presence of the Montmorillonite. Group B consisted of three soils from Bryan, Keller and Pharr B (medium PI) whose clay fraction was dominated by the presence of the Kaolinite. Group C only consisted of one soil from El Paso which was dominant with the Illite in its clay fraction. Volumetric strain test results measured during the wetting/drying process and the unconfined strength tests conducted on specimens after 3, 7, 14 and 21 cycles of wetting/drying are presented for each of these soils in the following subsections.

5.3.3.1 Group A results

Austin soil consisted of 53.4% of Montmorillonite, 17.7% of the Kaolinite and 28.9% of the Illite in its clay fraction. This soil had a plasticity index (PI) of 41 and was classified as a high compressible fat clay (CH). The soil was treated with 6% lime which was selected based on the current PI based design procedure (Tex-121-E).

Figure 5.25 presents the volumetric strain changes with the number of wetting - drying (W/D) cycles. Test results of both control and lime treated soils are presented in this figure. It can be observed here that the soil samples did not last all 21 cycles of wetting and drying and several samples failed immediately after twelve (12) cycles of wetting and drying. It can be observed from the figure that the maximum volumetric strain in the control soil was 55% after the completion of first W/D cycle and the treated soil had a maximum volumetric strain of 15% after 12 W/D cycles. Though the volumetric strains of treated soil is substantially lower than those of control soil, the total swell and shrink volume magnitudes of 15% is still considered as problematic since this value exceeded a volumetric strain of 5%, a threshold value reported as a problematic value in the literature.

Figure 5.26 presents the variation of the unconfined compressive strength properties with the wetting/drying cycles. It can be noted from the figure that the treated soil had only 25 psi of UC strength and the untreated soil had 32 psi of strength. The higher strength of the untreated soil was attributed to the conditions in which the specimens were tested. The control soil specimen is tested at optimum moisture content while the treated soil specimen was tested after moisture conditioning when the specimen was close to saturation. It can be noted here that the control soil lost all of its strength after the completion of the first W/D cycle while the treated soil lasted for 12 cycles. The soil specimens were holding their strength until 3 W/D cycles where they had retained 90% of the original strength but the strength drop was tremendous after that. After 7 cycles of W/D the soil specimens retained only 31 % of the original strength and they lost their integrity after the 11th cycle of W/D.

Fort Worth soil consisted of 60.4% of Montmorillonite, 23.4% of Kaolinite and 16.2% of Illite in its clay fraction. This soil had a plasticity index (PI) of 29 and was classified as a high compressible clay (CH). The soil was treated with 6% lime which was selected based on the current PI based design procedure (Tex-121-E).

Figure 5.27 presents the volumetric strain changes versus the number of wetting - drying cycles. Test results of both control and lime treated soils are presented in this figure. It can be observed here that the soil samples did not last all the 21 cycles of wetting and drying and several samples failed after ten (10) cycles of wetting and drying. It can be observed from the figure that the maximum volumetric strain in the control soil was 40% after the completion of first W/D cycle and the treated soil had a maximum volumetric strain of 15% after 10 W/D cycles. Though the volumetric strains of treated soil is substantially lower than those of control soil, the total volumetric shrink and swell magnitudes of 15% is still considered as problematic since this value exceeded a volumetric strain of 5%, a threshold value reported as a problematic value.

Figure 5.28 presents the variation of the unconfined compressive strength at various wetting/drying cycles. Test results of both control and lime treated soils are presented in this figure. It can be noted from the figure that the treated soil had exhibited 34.5 psi of UC strength while the control soil had exhibited a UCS value of 35 psi. The higher strength of the untreated soil is attributed to the conditions at which the control and treated specimens were tested. The control soil specimen was tested at optimum moisture content while the treated soil specimen was tested after moisture conditioning when the specimen was close to saturation.

It can be noted here that the control soil lost all of its strength after the completion of the first W/D cycle while the treated soil specimens lasted 12 cycles. The soil specimens were holding their strength until 3 W/D cycles where they had retained 90% of the original strength but the strength loss was considerable beyond 3 cycles. After 7 W/D cycles the soil specimens retained only 30% of the original strength and they lost their integrity after the 9th W/D cycle.

Paris soil consisted of 70.2% of Montmorillonite, 16.6% of the Kaolinite and 13.2% of Illite in its clay fraction. This soil had a plasticity index (PI) of 36 and was classified as high compressible clay (CH). The soil was treated with 8% lime which was selected based on the current PI based design procedure (Tex-121-E).

Figure 5.29 presents the volumetric strain changes versus the number of wetting - drying cycles. It can be observed here that the soil samples did not last all the 21 cycles of wetting and drying and several samples failed after seven (7) cycles of wetting and drying. It can be observed from the figure that the maximum volumetric strain in the control soil was 60% after the completion of first W/D cycle and the treated soil had a maximum volumetric strain of 15% after 7 W/D cycles. Though the volumetric strains of treated soil is substantially lower than those of control soil, the volumetric shrink and swell magnitudes of 15% is still considered as problematic since this value exceeded a volumetric strain of 5%, a threshold value reported as a problematic value.

Figure 5.30 presents the variation of the unconfined compressive strength with the wetting/drying cycles. Test results of both control and lime treated soils are presented in this figure. It can be noted from the figure that the treated soil exhibited only 37 psi of UC strength while the control soil had exhibited a UCS value of 26 psi. It can be noted here that the control soil lost all of its strength after the completion of the first W/D cycle while the treated soil specimens lasted for 7 cycles. The soil specimens were holding their strength until 3 W/D cycles where they had retained 75% of the original strength but the strength drop was tremendous after that. After 7 cycles of W/D the soil specimens retained only 25% of the original strength and they lost their integrity after the 8th cycle of W/D.

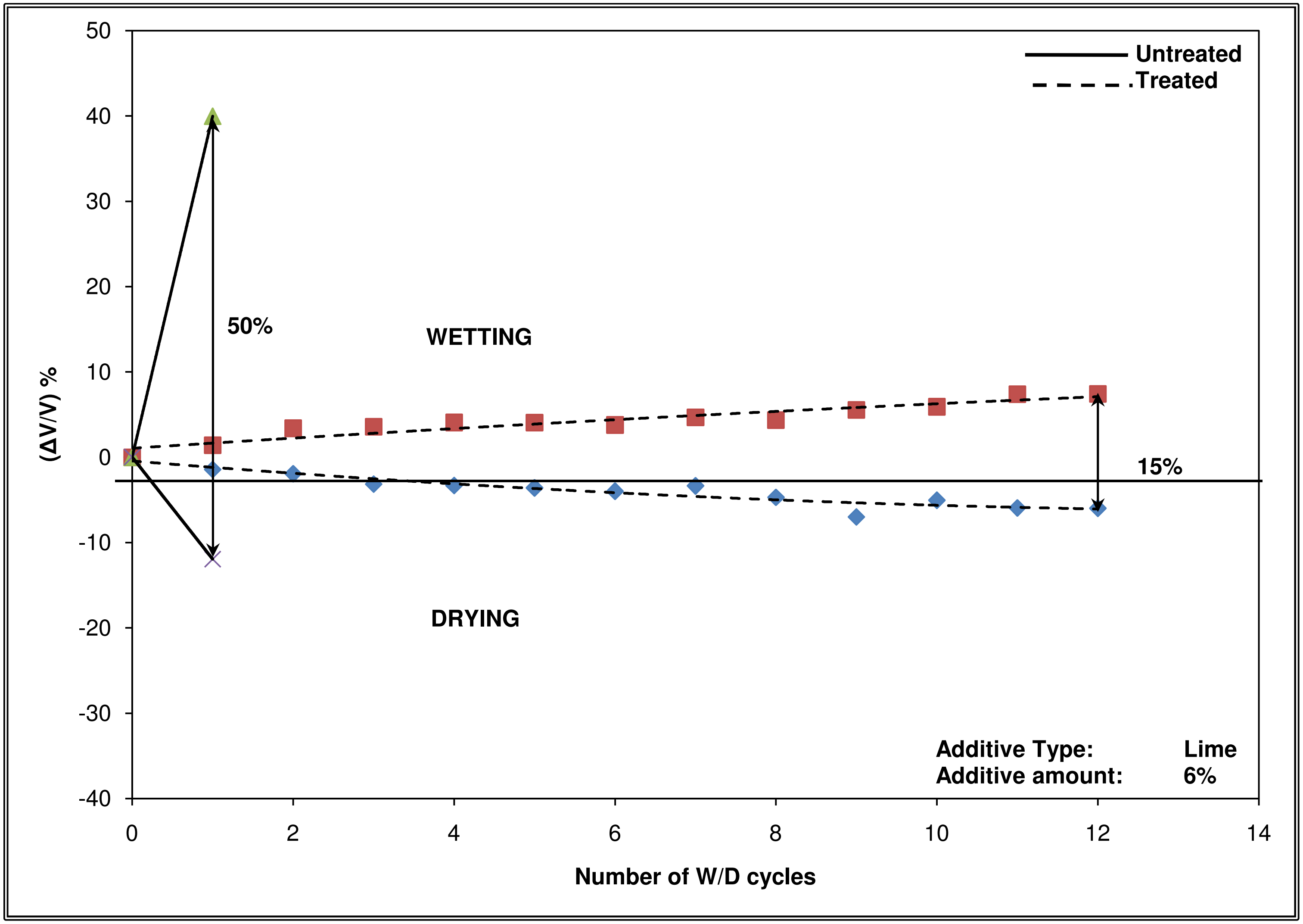


Figure 5.25 Volumetric changes with W/D cycles for treated and untreated Austin soil specimens

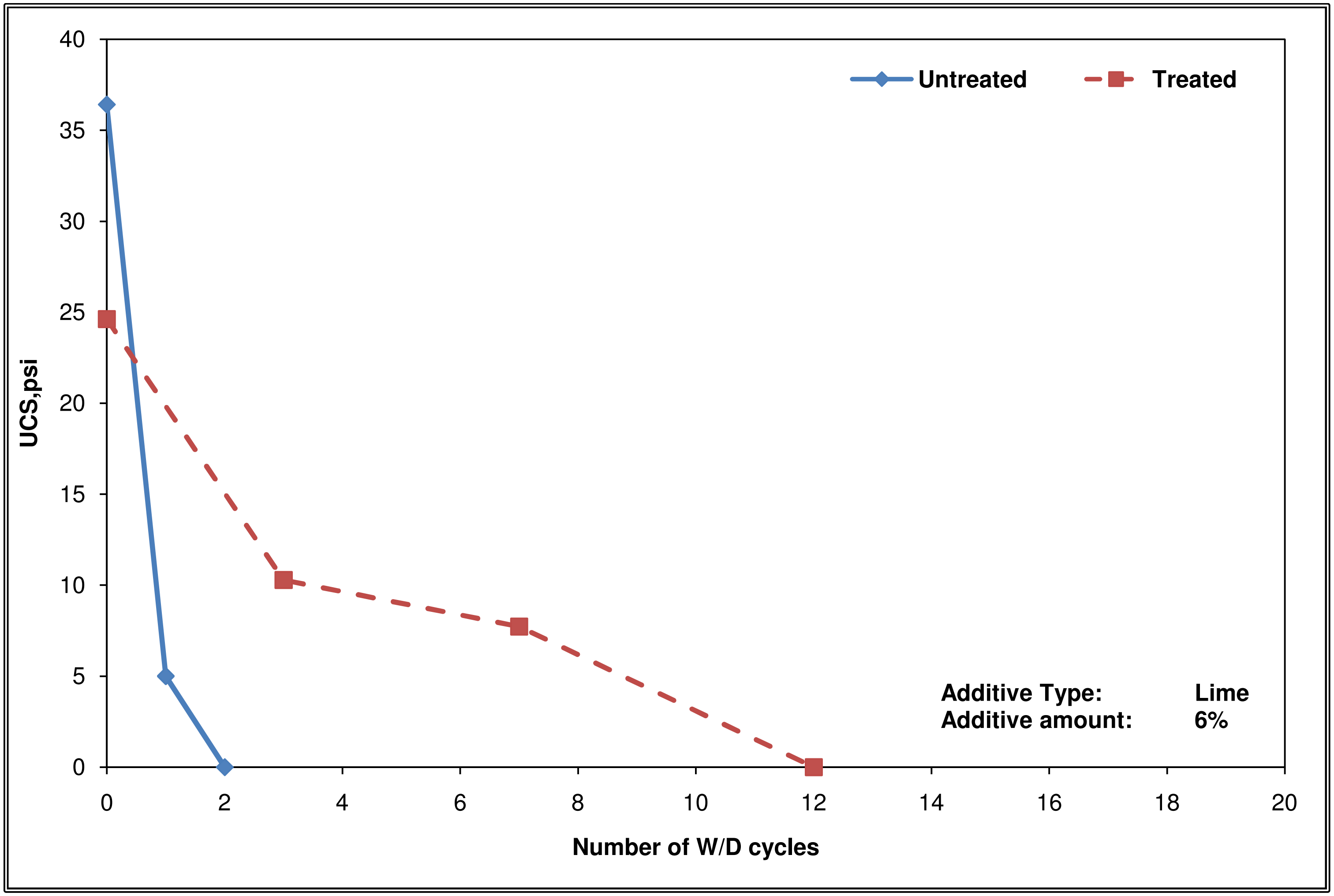


Figure 5.26 Changes in UCS with W/D cycles for treated and untreated Austin soil specimens

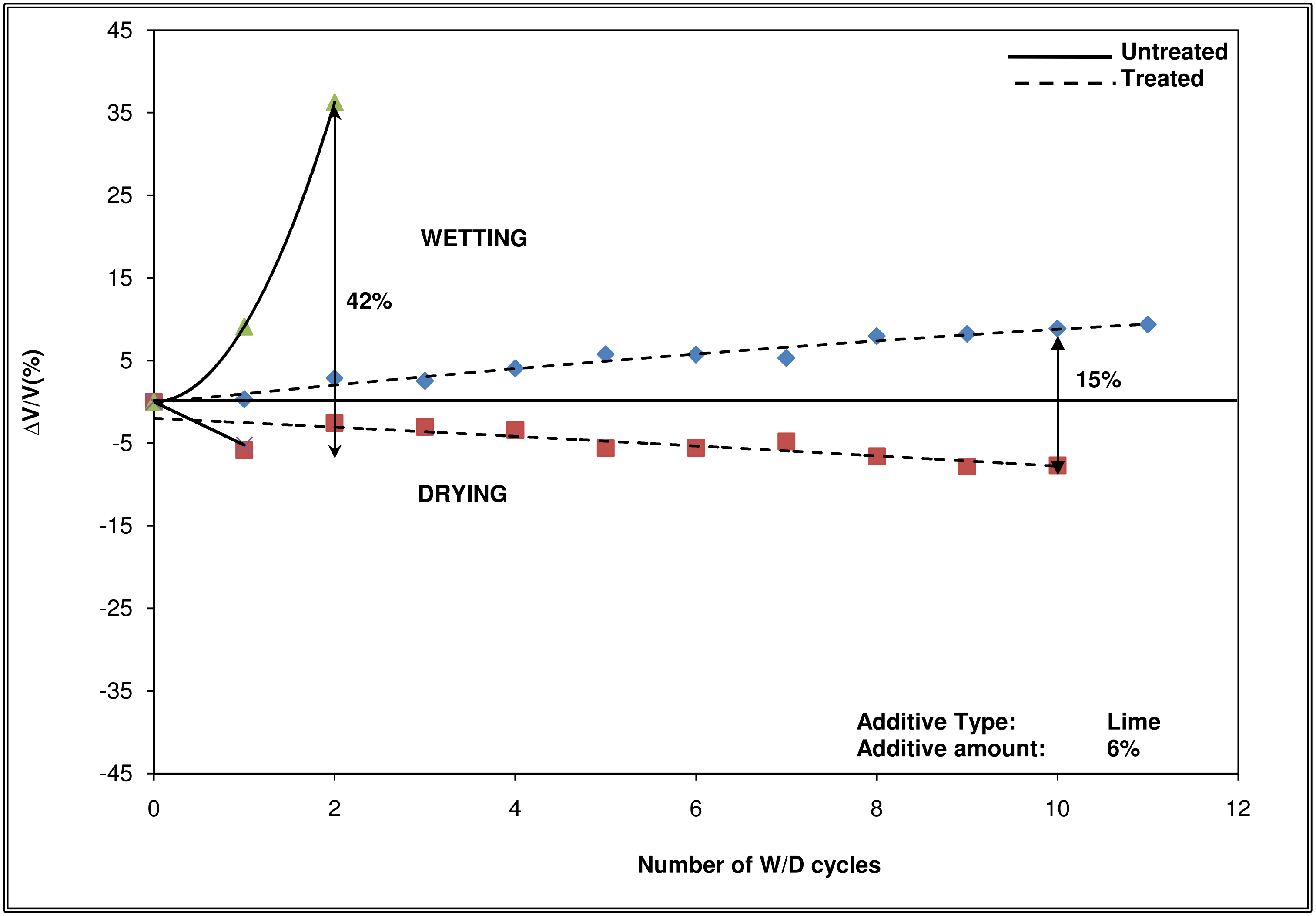


Figure 5.27 Volumetric changes with W/D cycles for treated and untreated Fort Worth soil specimens

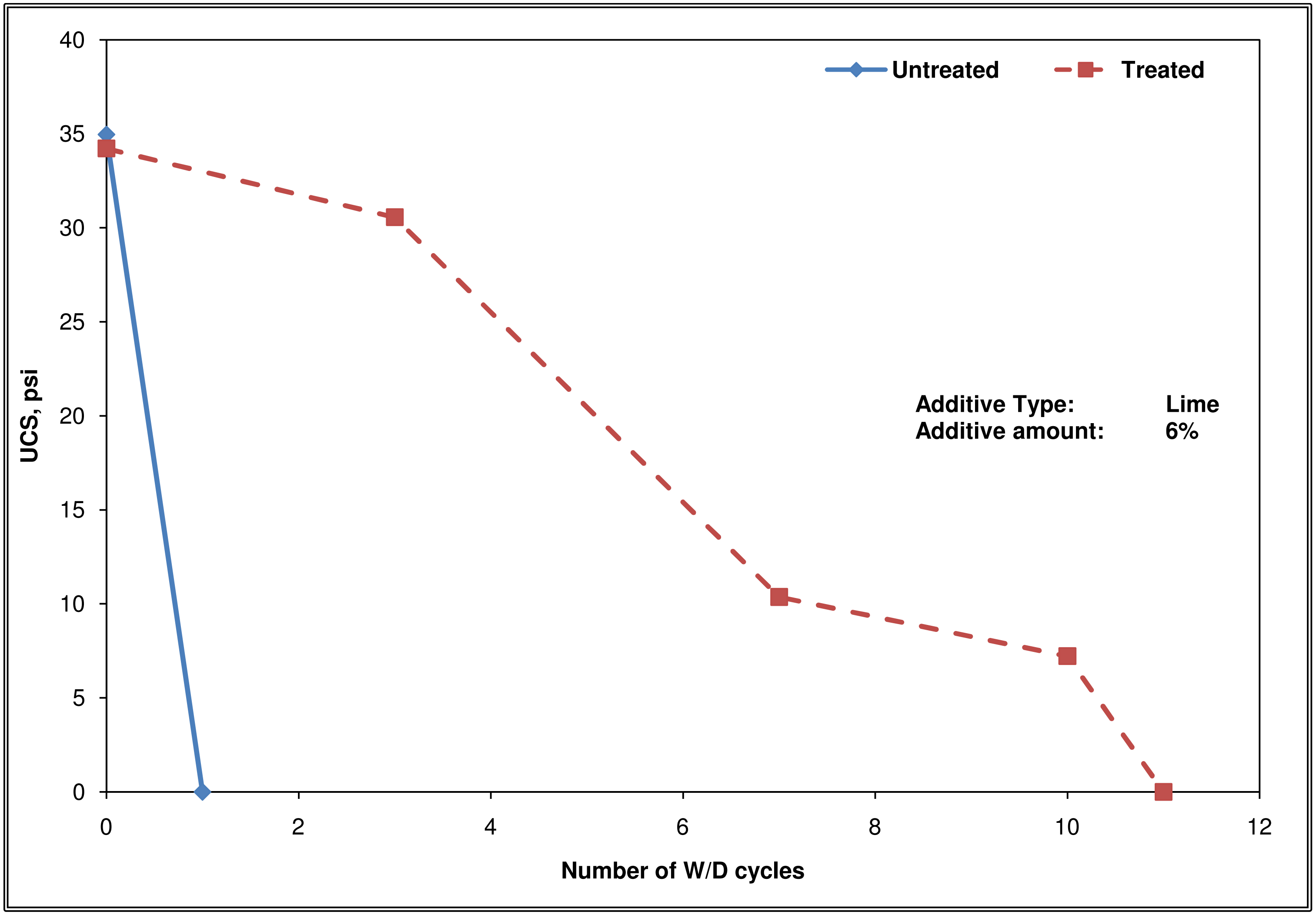


Figure 5.28 Changes in UCS with W/D cycles for treated and untreated Fort Worth soil specimens

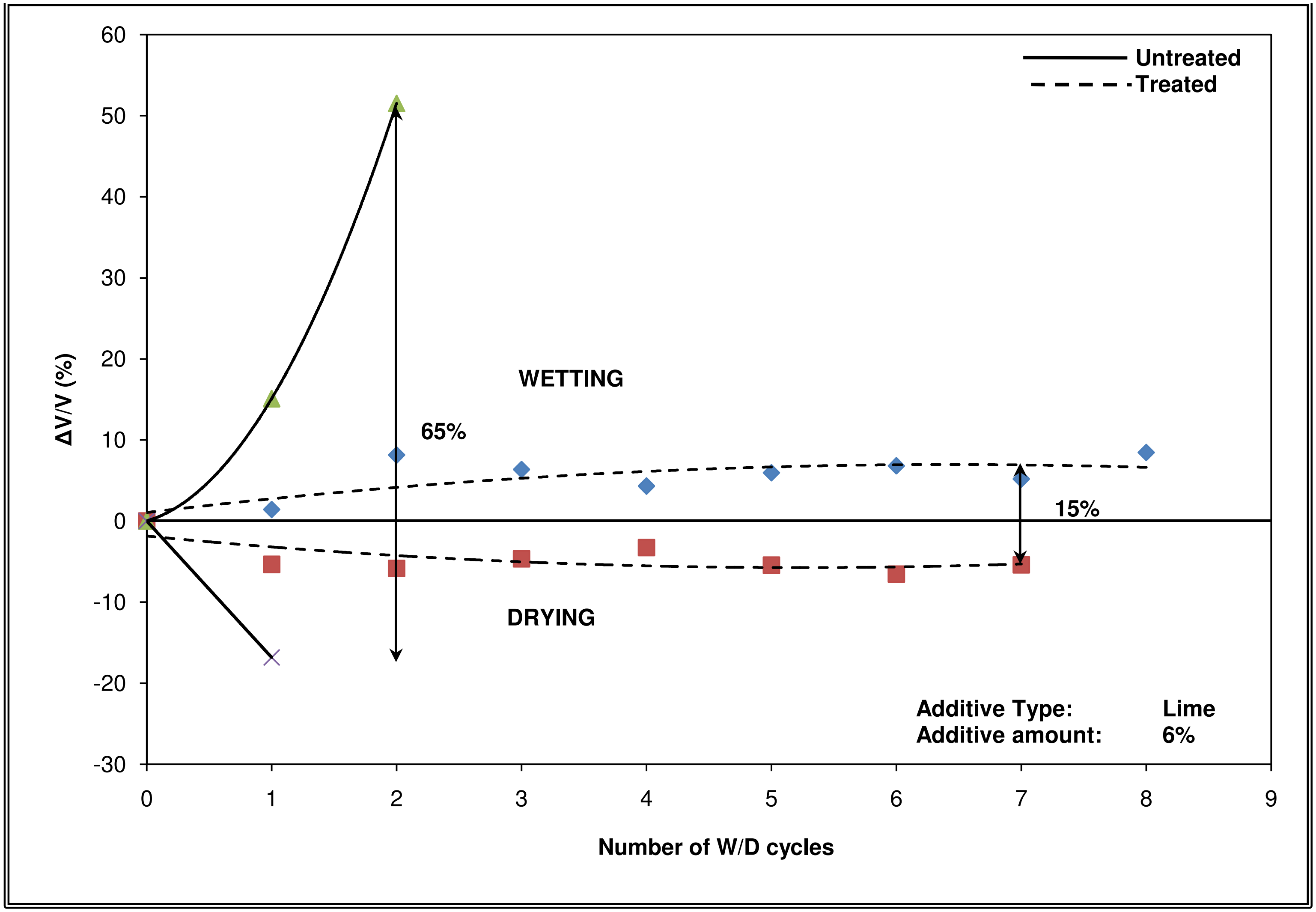


Figure 5.29 Volumetric changes with W/D cycles for treated and untreated Paris soil specimens

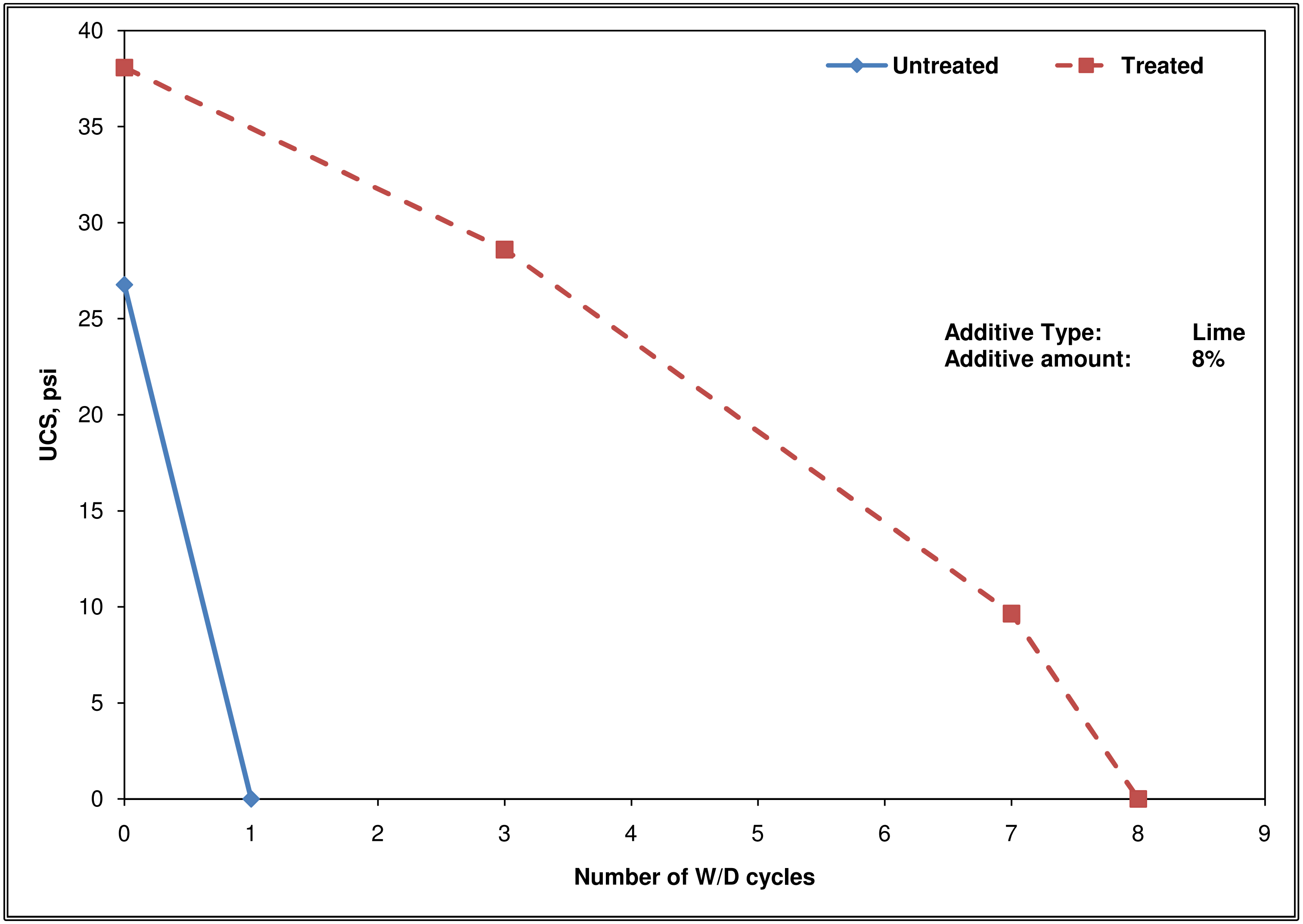


Figure 5.30 Changes in UCS with W/D cycles for treated and untreated Paris soil specimens

Pharr-A soil consisted of 47.8 % of Montmorillonite, 26.4% of Kaolinite and 25.8% of Illite in its clay fraction. This soil had a plasticity index (PI) of 45 and was classified as a high compressible fat clay (CH). The soil was treated with both lime and cement to check the effect of type of additive on the long-term performance of the stabilized soils. 4% lime and 4% cement were selected based on the current PI based design procedures (Tex-120-E for cement and Tex-121-E for lime).

Figure 5.31 presents the volumetric strain changes versus the number of wetting - drying cycles of this soil with and without 4% lime treatment. Test results of both control and lime treated soils are presented in this figure. It can be observed here that the soil samples did not last all the 21 cycles of wetting and drying and several samples failed after four (4) cycles of wetting and drying. It can be observed from the figure that the maximum volumetric strain in the control soil was 60% after the completion of first W/D cycle and the treated soil had a maximum volumetric strain of 30% after only 4 W/D cycles. Though the volumetric strains of treated soil is substantially lower than those of control soil, the total volumetric shrink and swell magnitudes of 30% is still considered as problematic since this value exceeded a volumetric strain of 5%, a threshold value reported as a problematic value.

Figure 5.32 presents the volumetric strain changes with the number of wetting - drying cycles of this soil treated with and without 4% cement. Test results of both control and cement treated soils are presented in this figure. It can be observed here that the soil samples did not last all the 21 cycles of wetting and drying and several samples failed after four (4) cycles of wetting and drying. It can be observed from the figure that the maximum volumetric strain in the control soil was 60% after the completion of first W/D cycle and the treated soil had a maximum volumetric strain of 31% after only 4 W/D cycles. Though the volumetric strains of treated soil is substantially lower than those of control soil, the total volumetric shrink and swell magnitudes of 31% is still considered as problematic since this value exceeded a volumetric strain value of

5%, a threshold value reported as a problematic value. This behavior is similar to the soil treated with lime.

Figure 5.33 presents the variation of the unconfined compressive strength with the wetting/drying cycles of this soil treated with and without 4% lime additive. Test results of both control and lime treated soils are presented in this figure. It can be noted from the figure that the treated soil had only 47 psi of UC strength while the control soil had 36 psi of strength. It can be noted here that the control soil lost 80% of its strength after the completion of the first W/D cycle while the treated soil specimens lasted for 4 cycles. In case of the treated soil specimens there was around 50% of the strength loss after the first W/D cycle and strength went on falling with the W/D cycles after that and finally the soil specimens collapsed after 4 cycles of W/D.

Figure 5.34 presents the variation of the unconfined compressive strength with the wetting/drying cycles of this soil treated with and without 4% cement. Test results of both control and cement treated soils are presented in this figure. The behavior of these soil specimens was similar to the ones treated with lime. It can be noted from the figure that the treated soil exhibited only 40 psi of UC strength while the control soil had exhibited a UCS value of 36 psi. It can be noted here that the control soil lost 75% of its strength after the completion of the first W/D cycle while the treated soil specimens lasted only for four (4) cycles. In case of the treated soil specimens, there was approximately 50% of the strength loss after the first W/D cycle and this strength decrease was evident with each passing W/D cycle and the soil specimen collapsed after 4 W/D cycles.

It should be noted here that the above soils were dominant in Montmorillonite mineral and exhibited premature failures and were not able to sustain the effect of the treatment for longer number of W/D cycles. Both the treatments (lime and cement) could not last for all the 21 cycles. This indicates that these soils when treated with lime or cement are going to show premature cracking on the pavement after few years of service due to the loss of the effect of chemical treatment in the treated subgrade layers.

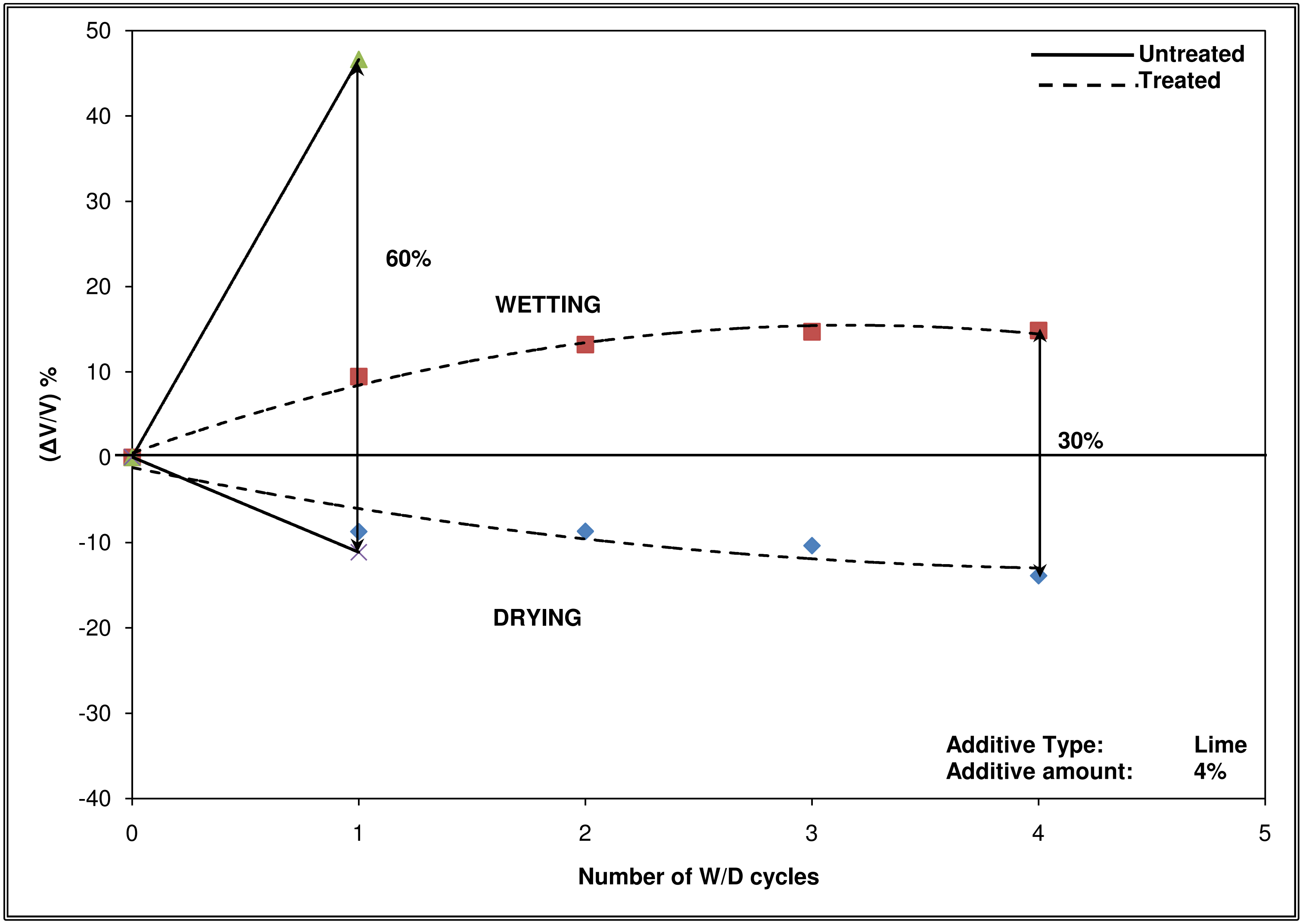


Figure 5.31 Volumetric changes with W/D cycles for treated (with lime) and untreated Pharr-A soil specimens

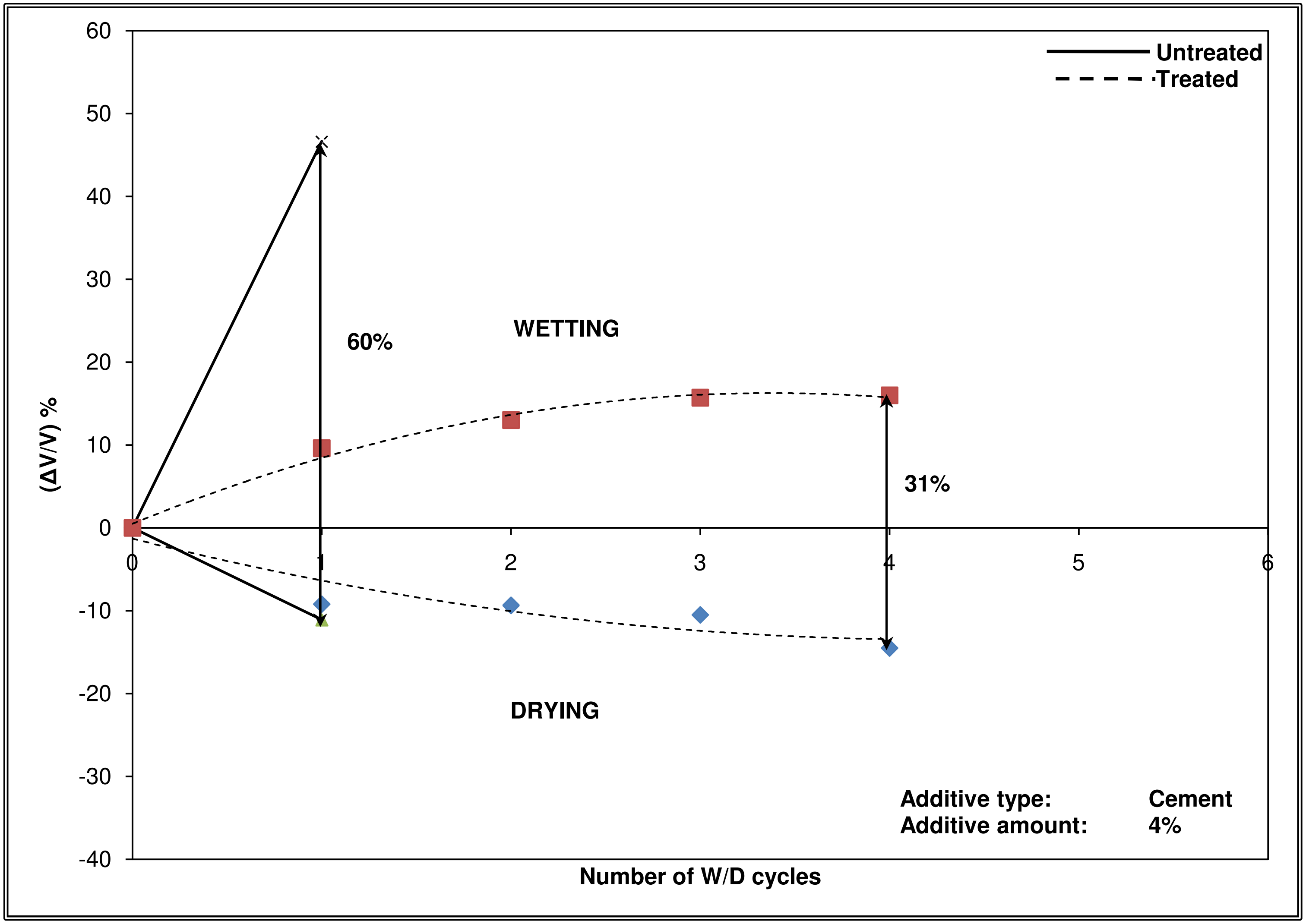


Figure 5.32 Volumetric changes with W/D cycles for treated (With cement) and untreated Pharr-A soil specimens

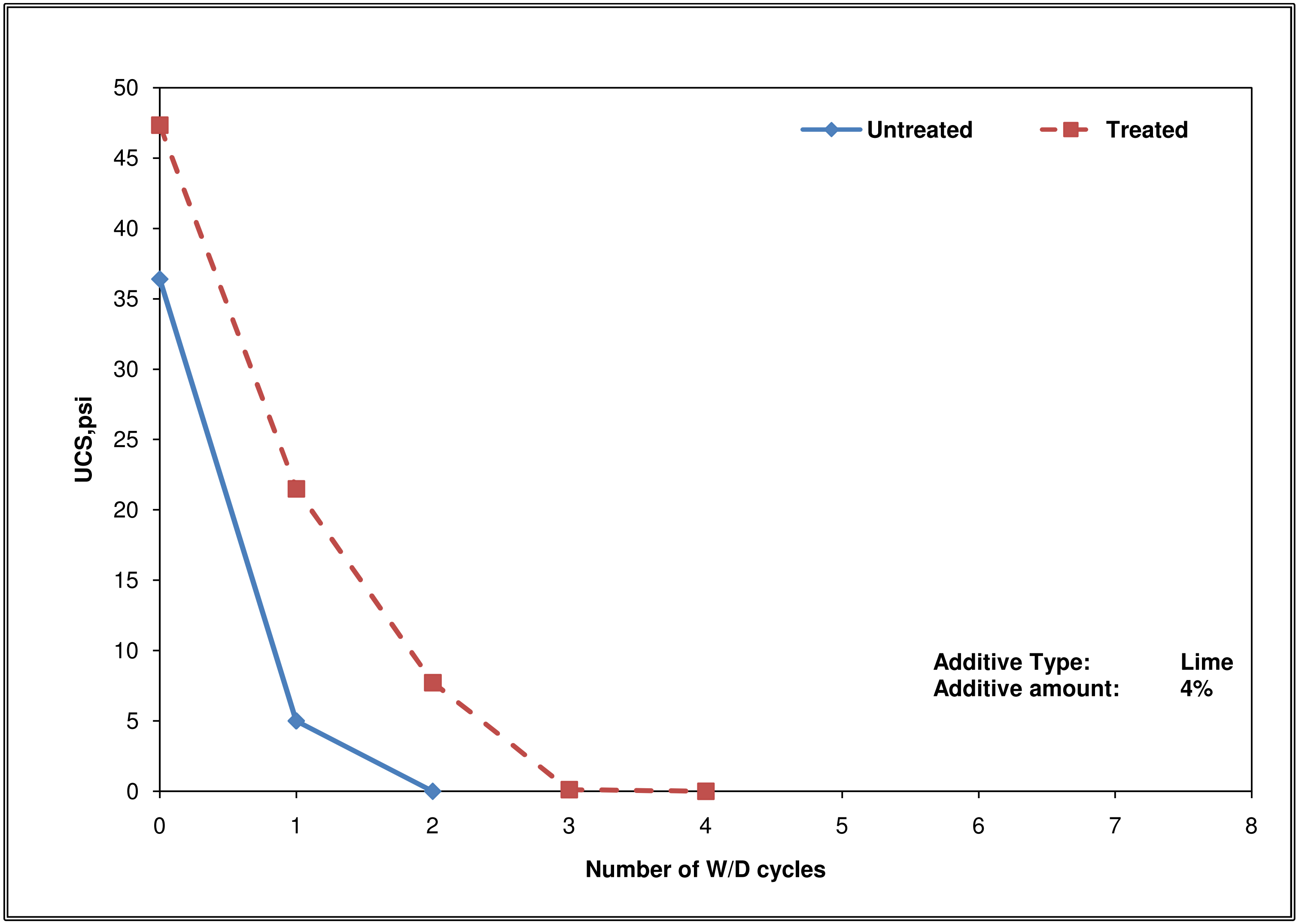


Figure 5.33 Changes in UCS with W/D cycles for treated (With lime) and untreated Pharr-A soil specimens

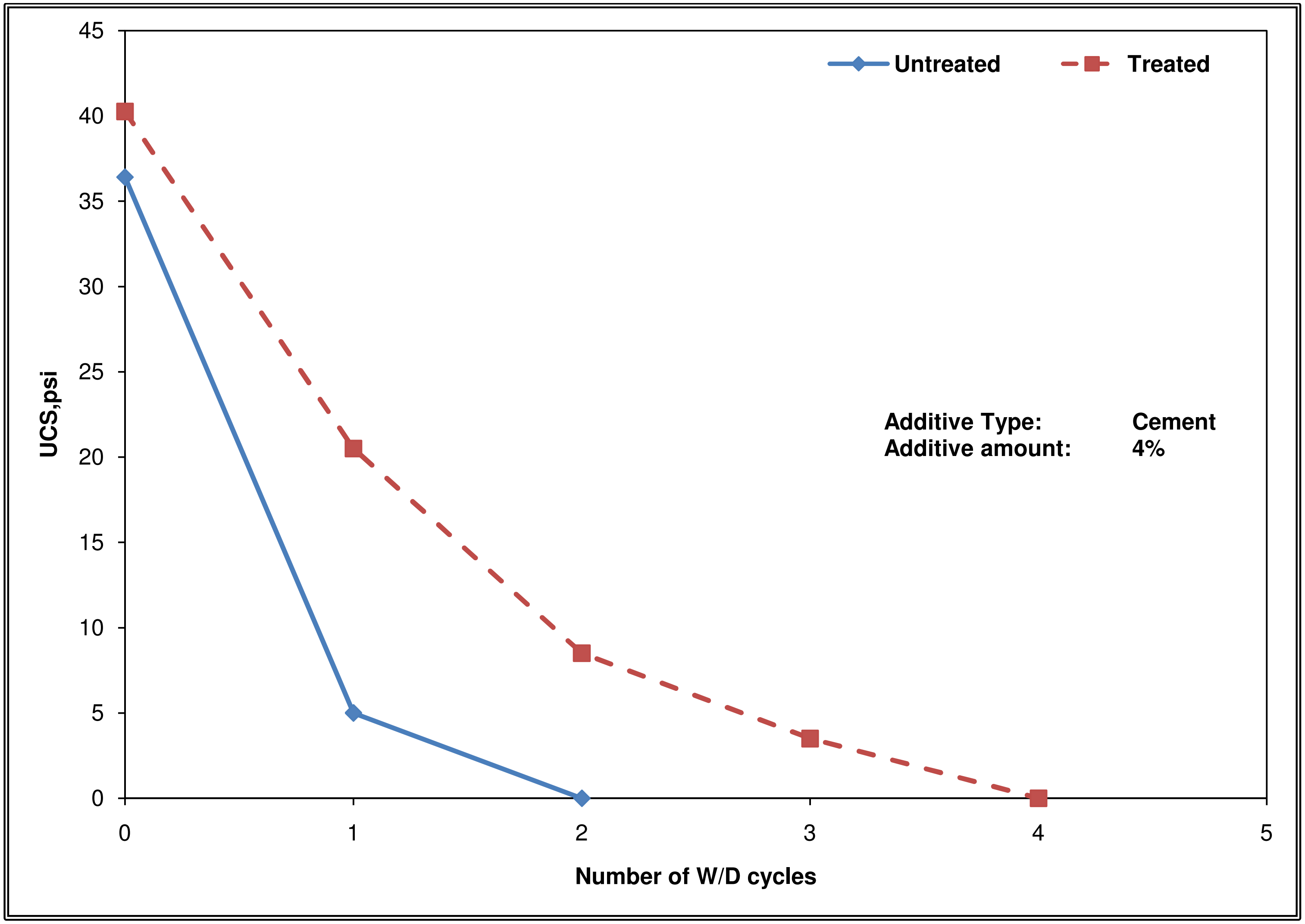


Figure 5.34 Changes in UCS with W/D cycles for treated (With cement) and untreated Pharr-A soil specimens

5.3.3.2 Group B results

Bryan soil consisted of 37% of Montmorillonite, 40% of Kaolinite and 23% of Illite in its clay fraction. This soil had a plasticity index (PI) of 31 and was classified as a low compressible clay (CL). The soil was treated with 8% lime which was selected based on the current PI based design procedure (Tex-121-E).

Figure 5.35 presents the volumetric strain changes with the number of wetting - drying cycles of this soil. Test results of both control and lime treated soils are presented in this figure. It can be observed here that the soil specimens lasted all the 21 cycles of wetting and drying. It can be observed from the figure that the maximum volumetric strain in the control soil was 20% after the completion of first W/D cycle and the treated soil had a maximum volumetric swell and shrink strains of 6% after 21 W/D cycles. Though the volumetric strains of 6% exceeded a volumetric strain value of 5%, a threshold value reported as a problematic value it is still not considered as problematic as the strength loss was not significant.

Figure 5.36 presents the variation of the unconfined compressive strength with the wetting/drying cycles. Test results of both control and lime treated soils are presented in this figure. It can be noted from the figure that the treated soil exhibited 87 psi of UC strength and the untreated soil had exhibited a UCS value of 22 psi . It can be noted here that the control soil lost all of its strength after the completion of the first W/D cycle while the treated soil lasted all the 21 cycles. The soil specimens were holding their strength through all the W/D cycles. Though there was some strength drop towards the end of the W/D cycles the reduction in strength was not considerable. After the completion of all the 21 W/D the retained strength of the soil specimens was 90%.

Keller soil consisted of 19.5% of Montmorillonite, 62.2% of Kaolinite and 18.3% of Illite in its clay fraction. This soil had a plasticity index (PI) of 11 and was classified as a high compressible clay (CL). The soil was treated with 6% lime which was selected based on the current PI based design procedure (Tex-121-E).

Figure 5.37 presents the volumetric strain changes with the number of wetting - drying cycles of this soil. Test results of both control and lime treated soils are presented in this figure. It can be observed here that the soil specimens lasted all the 21 cycles of wetting and drying. It can be observed from the figure that the maximum volumetric swell and shrink strains in the control soil was 27% after the completion of first W/D cycle and the treated soil had a maximum volumetric strain of 5% after 21 W/D cycles. The total volumetric shrink and swell strains of the specimens after 21 W/D cycles was 5% which is equal to the threshold value reported as a problematic value hence this soil is sustaining the effect of treatment over a longer period.

Figure 5.38 presents the variation of the unconfined compressive strength with the wetting/drying cycles. Test results of both control and lime treated soils are presented in this figure. It can be noted from the figure that the treated soil exhibited 52 psi of UC strength and the untreated soil had exhibited a UCS value of 31 psi. It can be noted here that the control soil lost all of its strength after the completion of the first W/D cycle while the treated soil lasted all the 21 cycles. The soil specimens were holding their strength through all the W/D cycles. Though there was some strength drop towards the end of the W/D cycles the reduction in strength was not considerable. After the completion of all the 21 W/D the retained strength of the soil specimens was 82.6%.

Pharr-B soil consisted of 18.3% of Montmorillonite, 54.2% of Kaolinite and 27.5% of Illite in its clay fraction. This soil had a plasticity index (PI) of 37 and was classified as a high compressible clay (CH). The soil was treated with both lime and cement to check the effect of type of additive on the long-term performance of the stabilized soils. 3% lime and 3% cement were selected based on the current PI based design procedures (Tex-120-E for cement and Tex-121-E for lime).

Figure 5.39 presents the volumetric strain changes with the number of wetting - drying cycles of this soil treated with and without 3% lime. Test results of both control and lime treated soils are presented in this figure. It can be observed here that the soil samples did not last all the 21 cycles of wetting and drying and several samples failed after eight (8) cycles of wetting and drying. It can be observed from the figure that the maximum volumetric swell and shrink strains in the control soil was 25% after the completion of first W/D cycle and the treated soil had a maximum volumetric swell and shrink strains of 18% after only 8 W/D cycles. The 18% of volumetric swell and shrink strains in the treated soil is considered as problematic since this value exceeded a volumetric stain value of 5%, a threshold value reported as a problematic value.

Figure 5.40 presents the volumetric strain changes with the number of wetting - drying cycles of this soil treated with and without 3% cement. Test results of both control and cement treated soils are presented in this figure. It can be observed here that the soil samples did not last all the 21 cycles of wetting and drying and several samples failed after fourteen (14) cycles of wetting and drying. It can be observed from the figure that the maximum volumetric swell and shrink strains in the control soil was 25% after the completion of first W/D cycle and the treated soil had a maximum volumetric swell and shrink strains of 13% after 14 W/D cycles. Though the volumetric strains of treated soil is substantially lower than those of control soil, the magnitude of 13% is still considered as problematic since this value exceeded a volumetric strain value of 5%, a threshold value reported as a problematic value. This behavior is similar to the soil treated with lime.

Figure 5.41 presents the variation of the unconfined compressive strength with the wetting/drying cycles of this soil treated with and without 3% lime. Test results of both control and lime treated soils are presented in this figure. It can be noted from the figure that the treated soil exhibited only 28 psi of UC strength while the control soil had exhibited a UCS value of 41 psi. The higher strength of the untreated soil is attributed to the conditions in which the specimens were tested. The control soil specimen is tested at optimum moisture content while the treated soil specimen was tested after moisture conditioning when the specimen is close to saturation. The control soil specimen is tested at optimum moisture content while the treated soil specimen was tested after moisture conditioning when the specimen is close to saturation. It can be noted here that the control soil lost 80% of its strength after the completion of the first W/D cycle while the treated soil specimens lasted for 8 cycles. In case of the treated soil specimens there was around 50% of the strength loss after three W/D cycle and strength went on falling with the W/D cycles after that and finally the soil specimens collapsed after 7 cycles of W/D.

Figure 5.42 presents the variation of the unconfined compressive strength with the wetting/drying cycles of this soil treated with and without 3% cement. Test results of both control and cement treated soils are presented in this figure. It can be noted from the figure that the treated soil exhibited only 56 psi of UC strength while the control soil had exhibited a UCS value of 41 psi of strength. It can be noted here that the control soil lost 80% of its strength after the completion of the first W/D cycle while the treated soil specimens lasted for 14 cycles. In case of the treated soil specimens there was around 10% of the strength loss after three W/D cycles and 35% of the strength loss after three W/D cycle and strength went on falling with the W/D cycles after that and finally the soil specimens collapsed after 13 cycles of W/D.

It should be noted here that the soils the above soils were dominant in Montmorillonite mineral and exhibited premature failures and were not able to sustain the effect of the treatment for longer number of W/D cycles. Both the treatments (lime and cement) could not last for all the 21 cycles. This indicates that these soils when treated with lime or cement are going to show premature cracking on the pavement after few years of service due to the loss of the effect of treatment in the treated subgrade layers.

5.3.3.3 Group C results

El Paso soil consisted of 23% of Montmorillonite, 14.2% of Kaolinite and 62.5% of Illite in its clay fraction. This soil had a plasticity index (PI) of 16 and was classified as a low compressible clay (CL). The soil was treated with 8% lime which was selected based on the current PI based design procedure (Tex-121-E). Figure 5.43 presents the volumetric strain changes with the number of wetting - drying cycles of this soil. It can be observed here that the soil specimens lasted all the 21 cycles of wetting and drying. It can be observed from the figure that the maximum volumetric shrink and swell strains in the control soil was 25% after the completion of first W/D cycle and the treated soil had a maximum volumetric strain of 12% after 21 W/D cycles. Though the volumetric strains of 12% exceeded a volumetric strain value of 5%, a threshold value reported as a problematic value it is still not considered as problematic as the strength loss was not considerable.

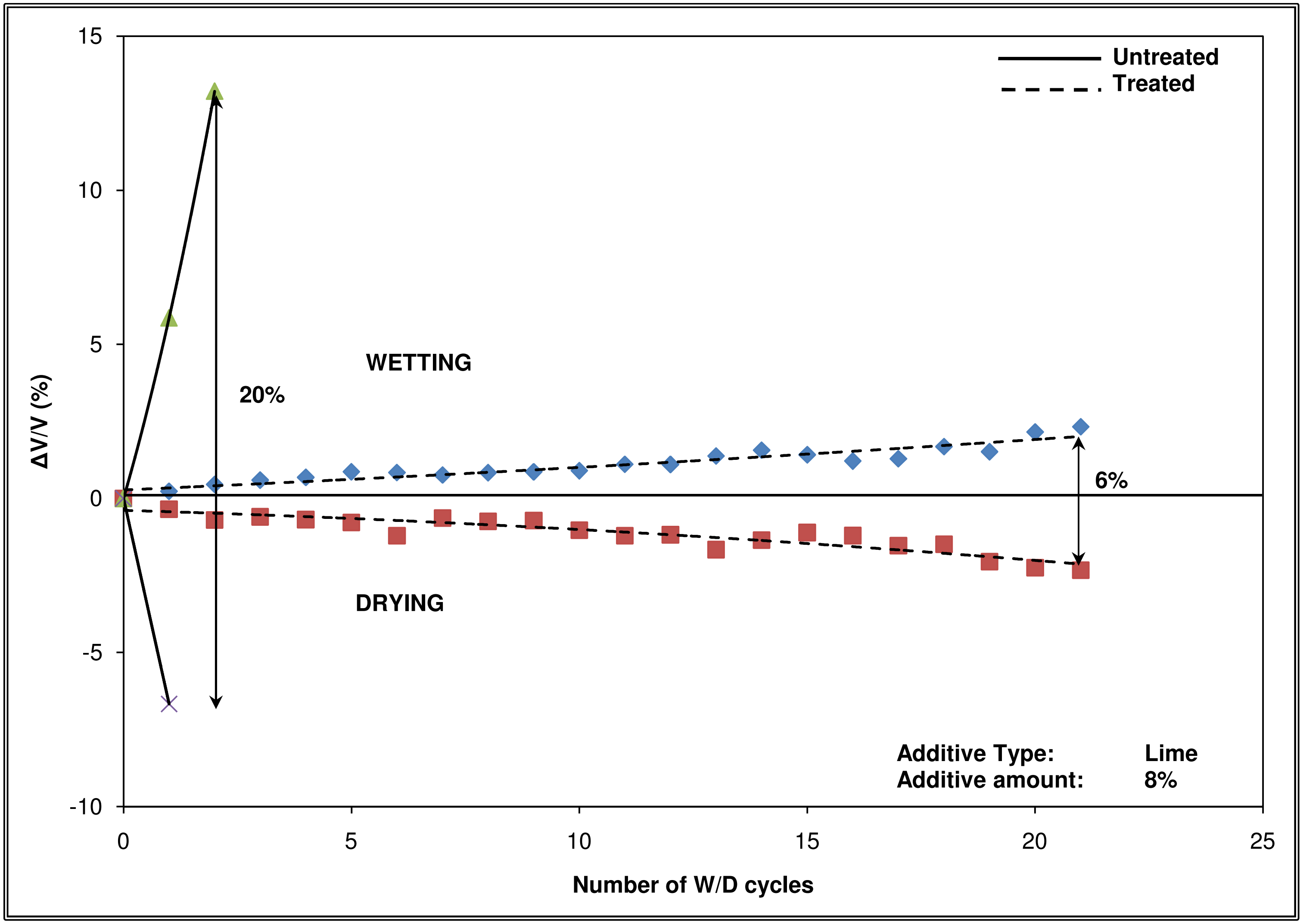


Figure 5.35 Volumetric changes with W/D cycles for treated and untreated Bryan soil specimens

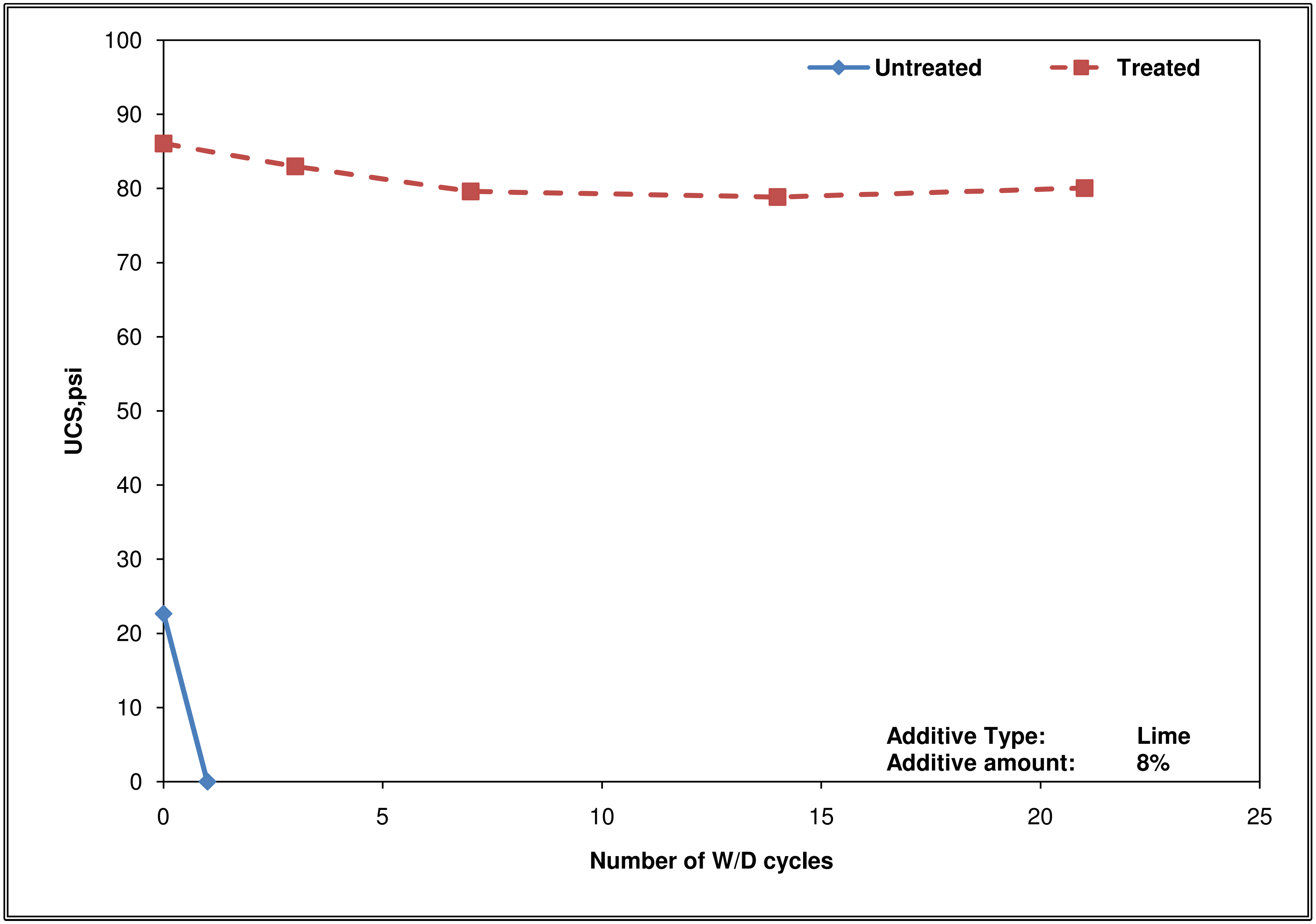


Figure 5.36 Changes in UCS with W/D cycles for treated and untreated Bryan soil specimens

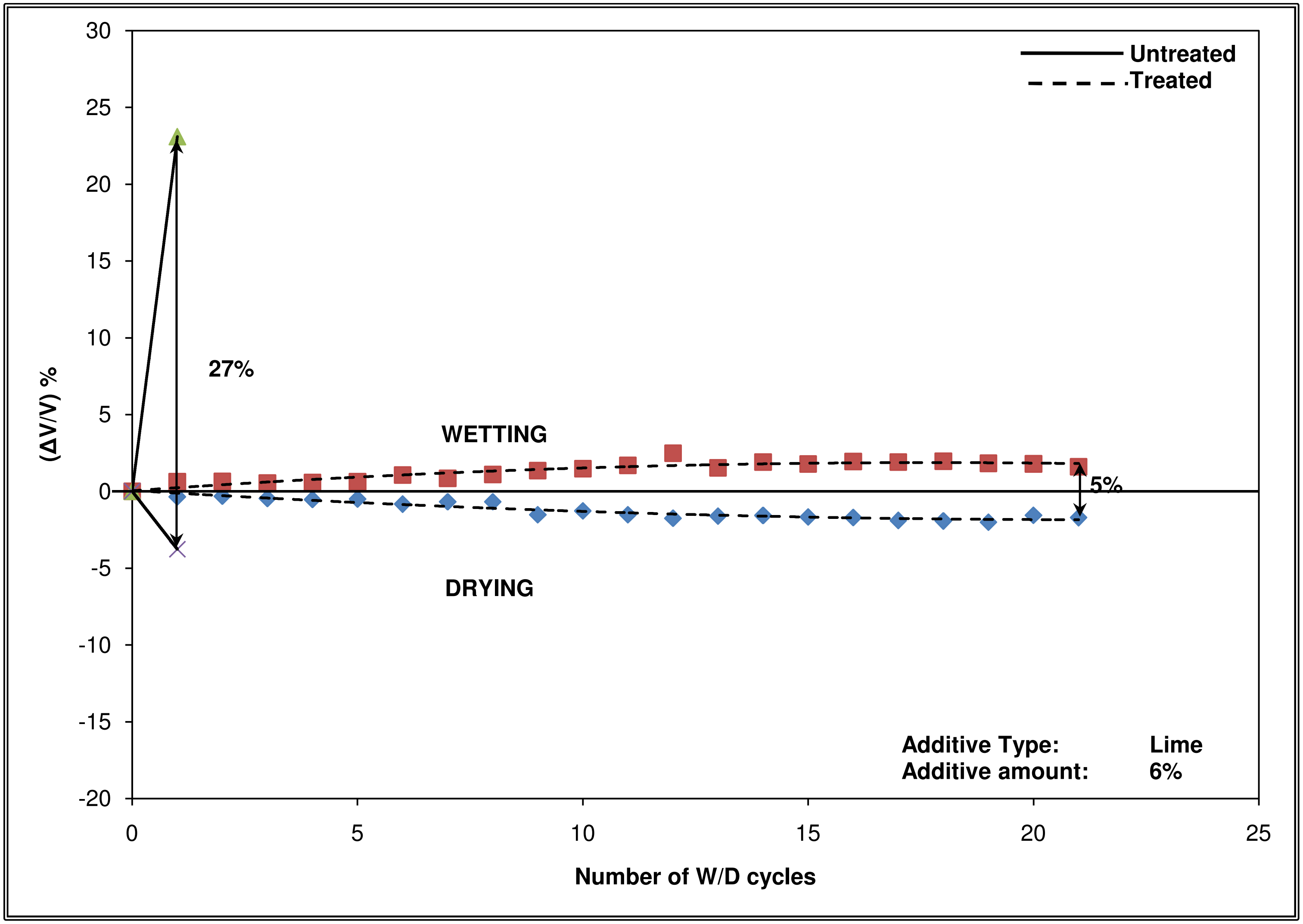


Figure 5.37 Volumetric changes with W/D cycles for treated and untreated Keller soil specimens

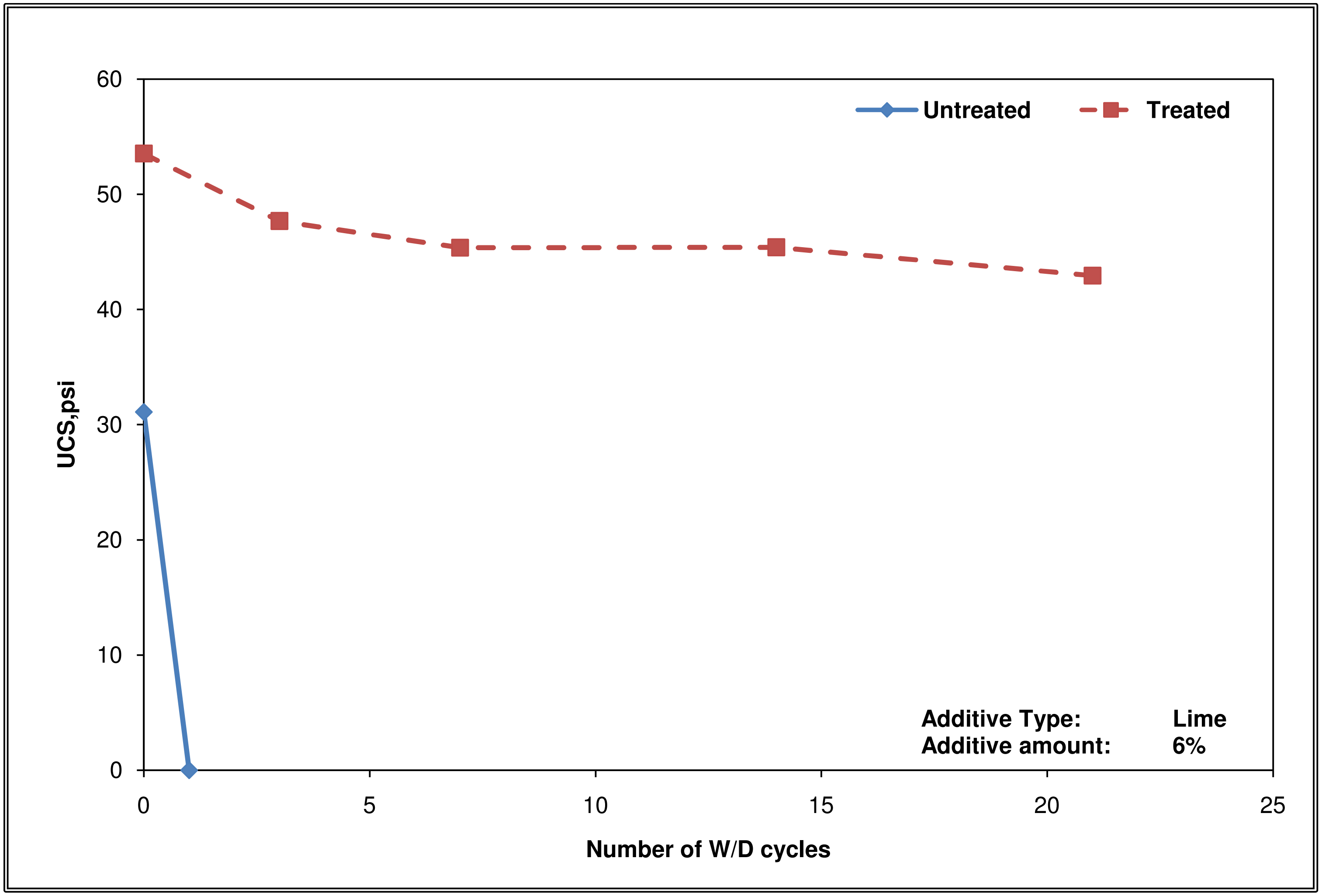


Figure 5.38 Changes in UCS with W/D cycles for treated and untreated Keller soil specimens

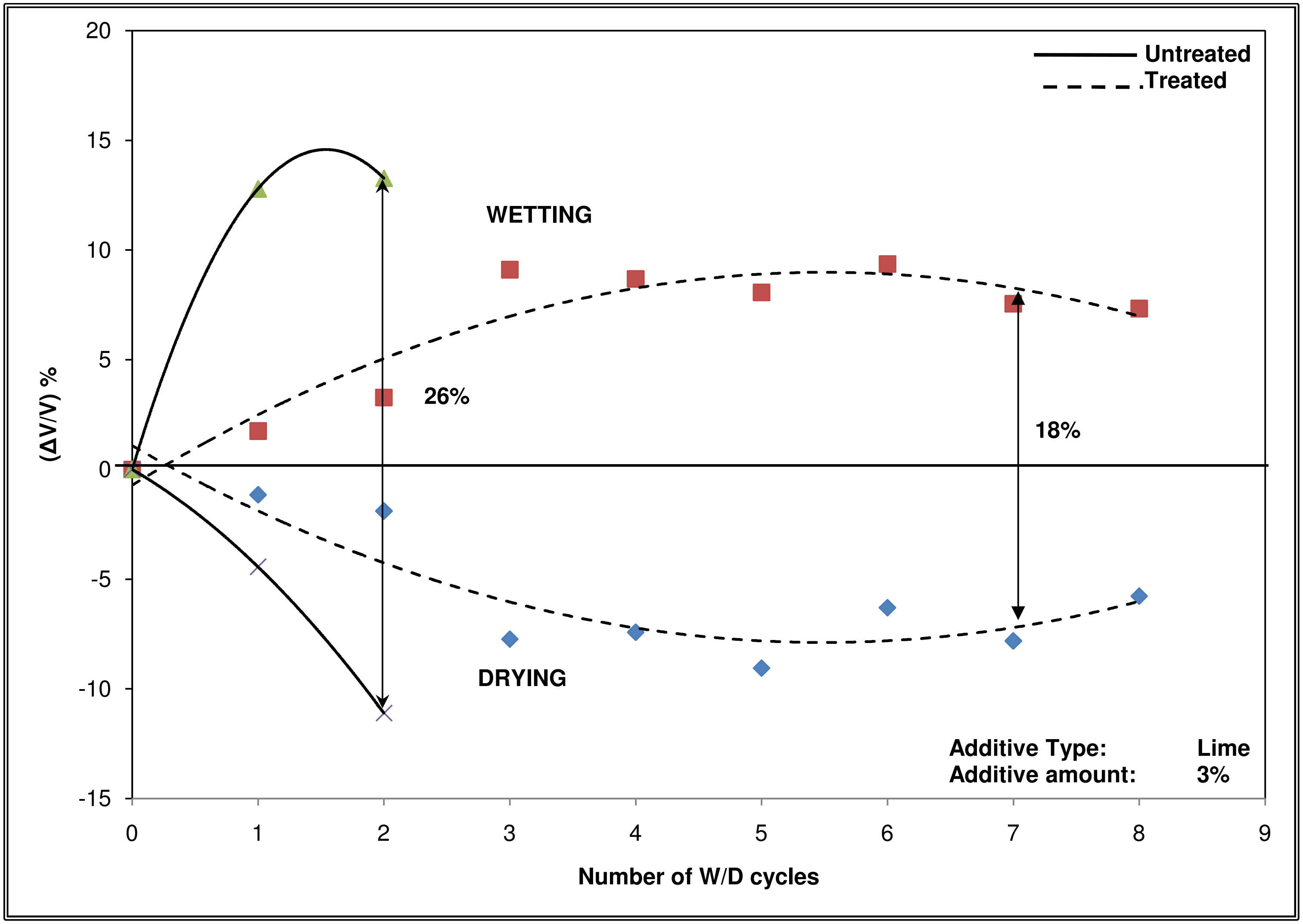


Figure 5.39 Volumetric changes with W/D cycles for treated (with lime) and untreated Pharr-B soil specimens

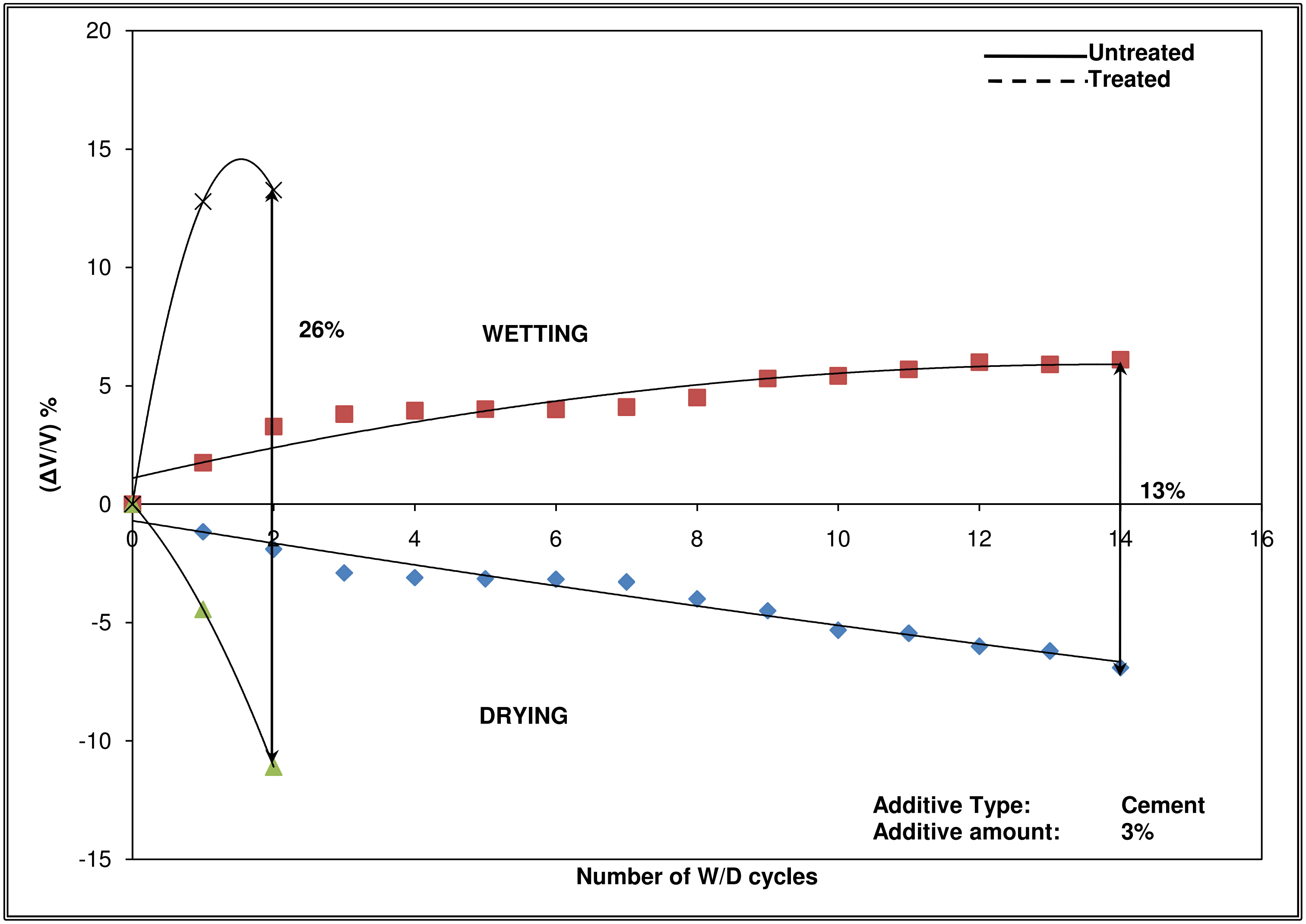


Figure 5.40 Volumetric changes with W/D cycles for treated (with cement) and untreated Pharr-B soil specimens

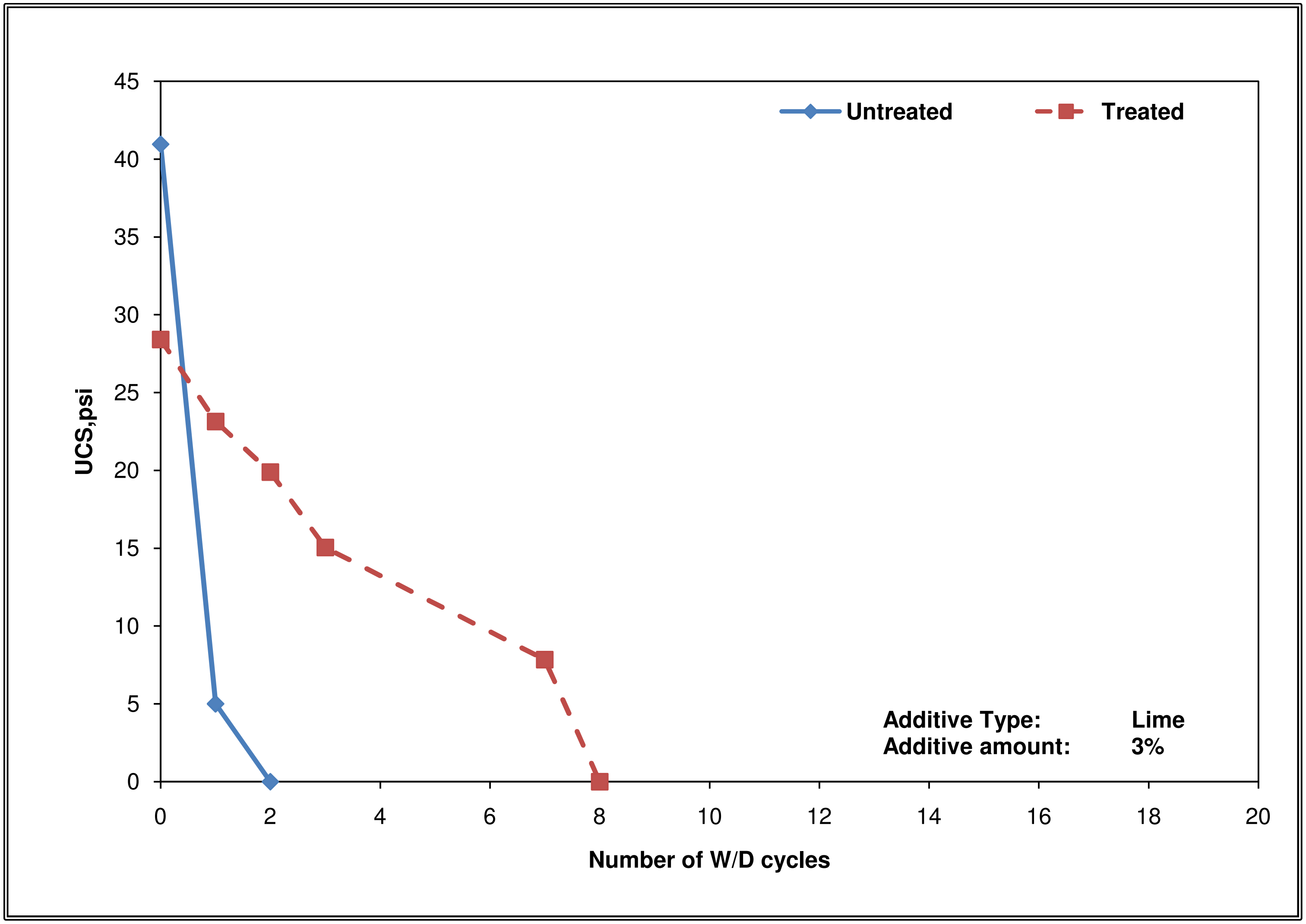


Figure 5.41 Changes in UCS with W/D cycles for treated (with lime) and untreated Pharr-B soil specimens

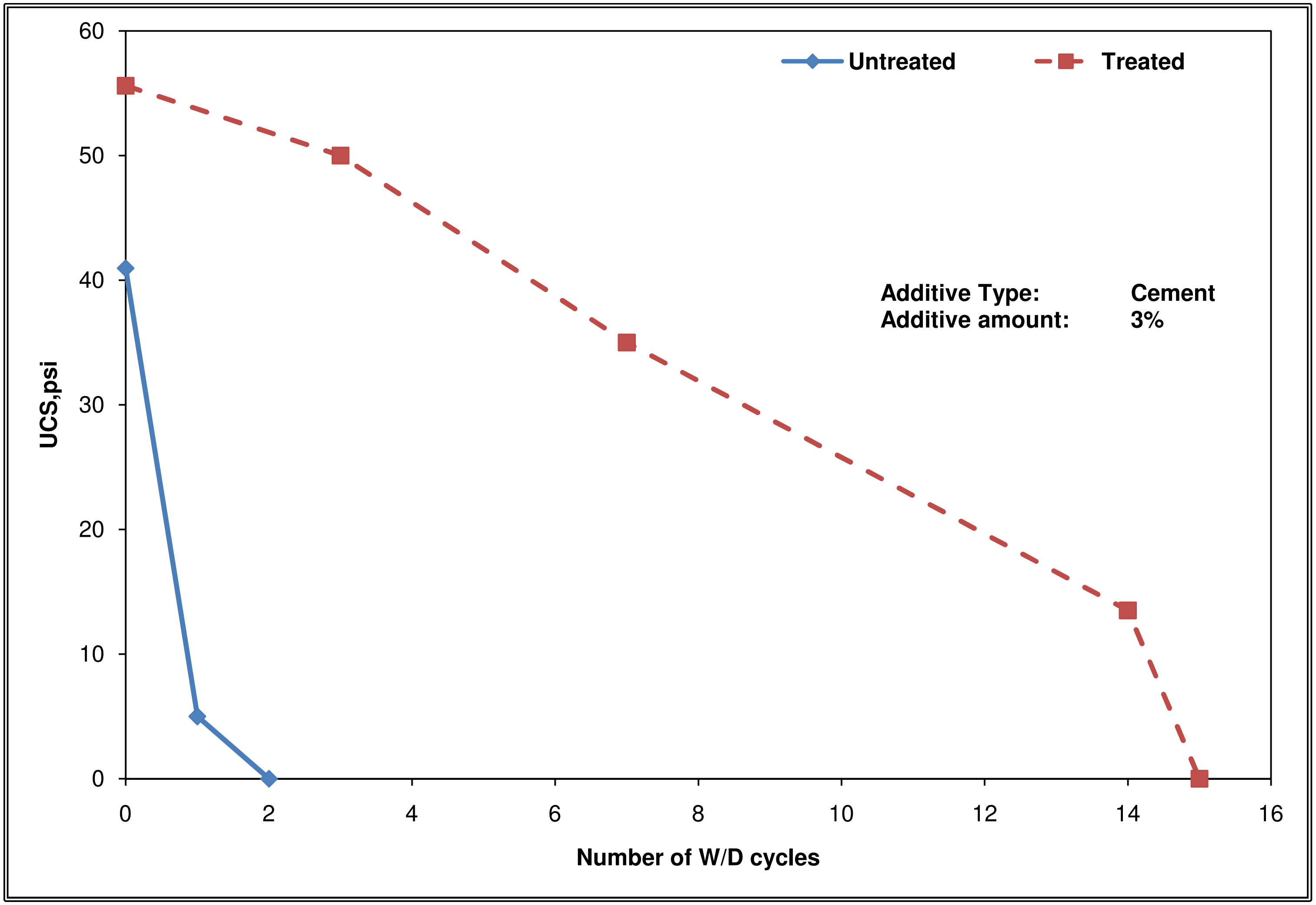


Figure 5.42 Changes in UCS with W/D cycles for treated (with cement) and untreated Pharr-B soil specimens

Figure 5.44 presents the variation of the unconfined compressive strength with the wetting/drying cycles. Test results of both control and lime treated soils are presented in this figure. It can be noted from the figure that the treated soil exhibited 70 psi of UC strength and the untreated soil had exhibited a UCS value of 24 psi. It can be noted here that the control soil lost all of its strength after the completion of the first W/D cycle while the treated soil lasted all the 21 cycles. The soil specimens were holding their strength through all the W/D cycles. Though there was some strength drop towards the end of the W/D cycles the reduction in strength was not considerable. After the completion of all the 21 W/D the retained strength of the soil specimens was 83%.

From the results presented in the sections above it can be noted that the volumetric swelling strains in all the control soils has been considerable (> 5 %) after the first W/D cycle. The volumetric strains in treated Austin, Fort Worth, Paris and Pharr-A clays were considerable after 7, 10, 7 and 4 W/D cycles, respectively and had zero retained strength after these number of cycles. These soils were dominant with Montmorillonite mineral. The volumetric strains in treated Bryan, El Paso, Keller clays were not considerable after 21 cycles of wetting and drying cycles and had around 80% of retained strength for all the soils. These soils were not dominant with Montmorillonite clay mineral. The Pharr-B clay though not dominant with Montmorillonite exhibited considerable volumetric strains even after 8 and 14 cycles when treated with lime and cement additives respectively. Detailed analysis of these test results is carried out in the following sections.

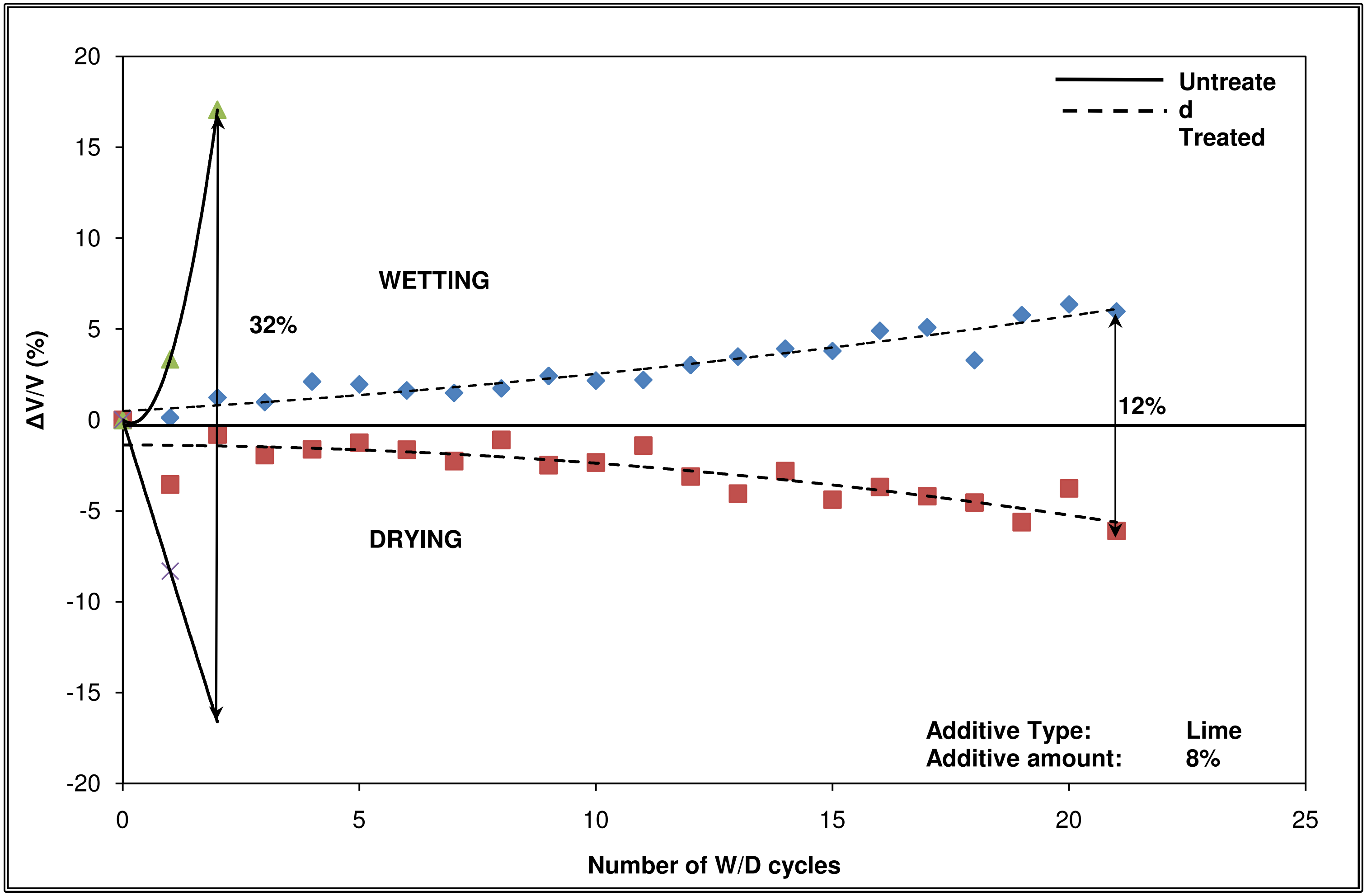


Figure 5.43 Volumetric changes with W/D cycles for treated and untreated El Paso soil specimens

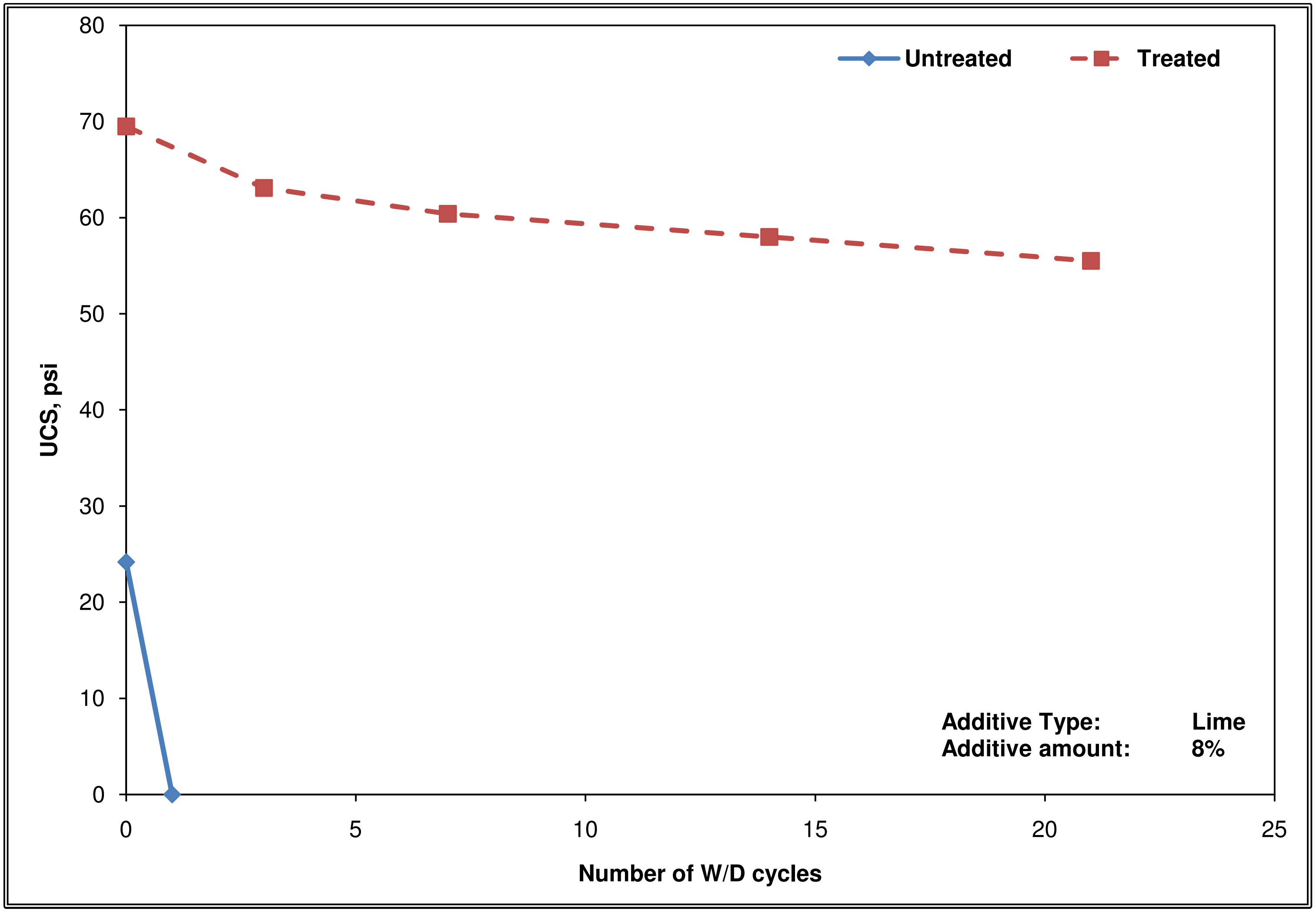


Figure 5.44 Changes in UCS with W/D cycles for treated and untreated El Paso soil specimens

5.3.4 Analysis of Test Results

Analysis of the above test results is carried out in this section. The effects of curing time period, clay mineralogy and additive type and their dosages on the long-term performance of stabilized expansive clays are addressed in the following sections:

5.3.4.1 Effects of curing methods

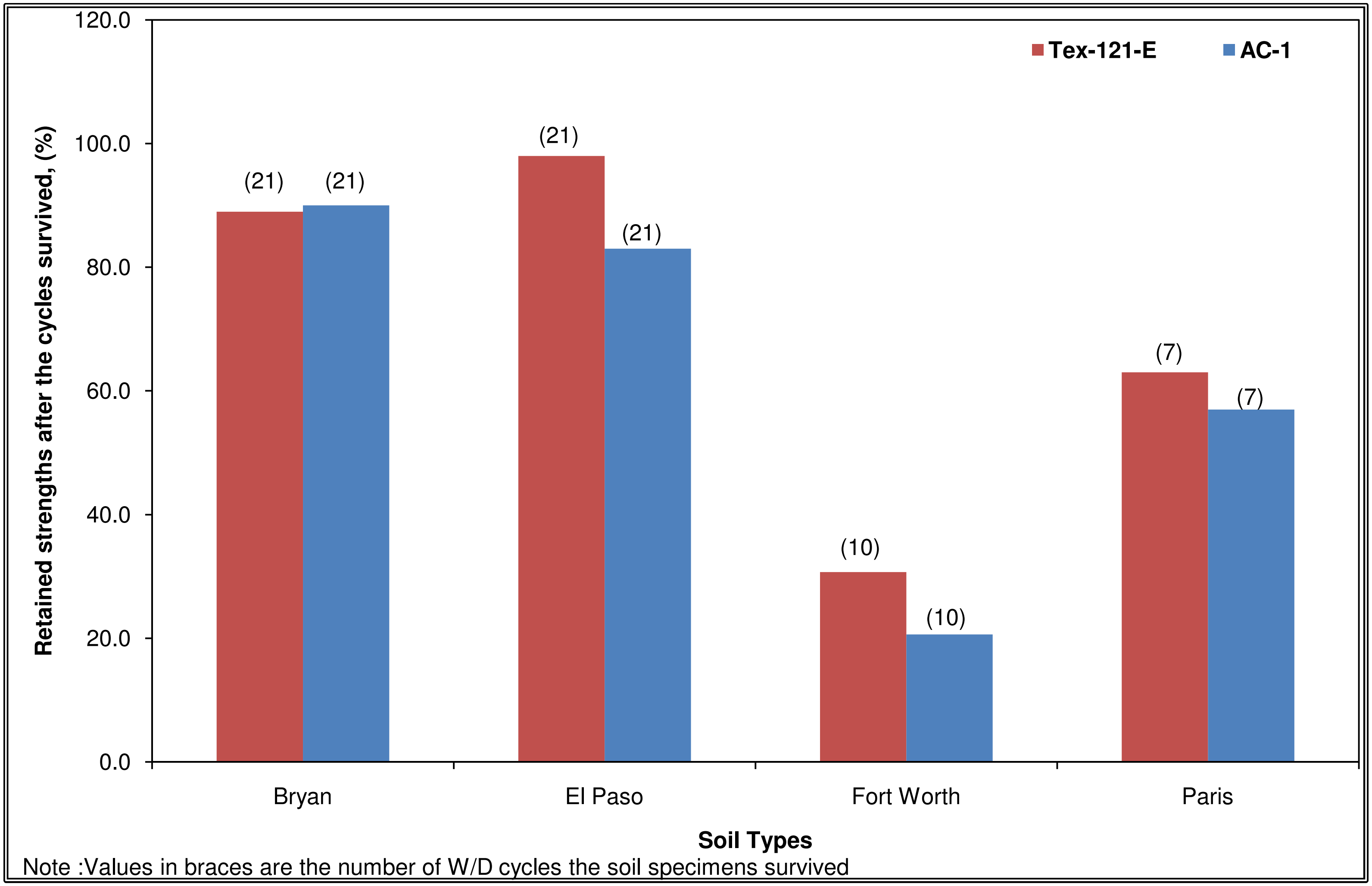
The effect of curing on the long-term performance of chemically treated expansive clays has been examined here. Section 5.3.2 presents these test results. From these results, it has been noted that treated soils under both 'long-term curing and conditioning method (as adapted in Tex-121-E)' and 'a short 3 day curing method, AC-1' provided similar responses under loading.

Figure 5.45 summarizes the test results obtained in this research. In this figure, it can be observed that the two of these soils, Bryan, El Paso soils survived all 21 W/D cycles while the other two soils from Fort Worth and Paris survived only 10 and 7 cycles, respectively. This observation was true in both the cases of curing methods. Also the retained strength measured at the end of the cycles survived was also similar in soils under both the curing conditions. Hence, it can be mentioned that the length of curing time period did not show any considerable effect on the long-term performance of treated soils.

Initial strength was slightly different since soils tested under initial cycles were at different saturation conditions. Overall, accelerated curing method was considered for the rest of the experimental program because of quick time to complete the stabilization design and better control of saturation in the soils at the time of testing.

5.3.4.2 Effect of clay mineralogy

Table 5.4 summarizes the percent retained strength with the number of wetting/drying cycles in different soils studied in this research. The reason for this diverse behavior can be attributed to the dominance of the mineral Montmorillonite present in the clay fraction of the soil.



Note :Values in braces are the number of W/D cycles the soil specimens survived

Figure 5.45 Effects of the curing methods on the long-term performance of stabilization

From Table 5.4, it can be observed that Austin, Fort Worth, Paris and Pharr-A clays have high amounts of Montmorillonite mineral in their clay fractions and all these clays did not last all 21 W/D cycles attempted as a part of the durability studies. Clays from El Paso, Bryan and Keller have other clay minerals as dominant minerals and these clays have only survived 21 cycles. This indicates that soils containing Montmorillonite as a dominant mineral are more susceptible to premature strength failures after chemical stabilization when they were exposed to volume changes caused by swell and shrink related soil movements. Only the Pharr-B clay dominant with the Kaolinite mineral did not survive all 21 cycles. This soil when treated with 3% lime survived only 8 cycles while the same material treated with 3% cement survived only 14 cycles. In both cases, lower dosages of chemical additive were attributed to the failure as the chemical dosage was not selected based on pH or PI considerations. Stabilizer dosage was arbitrarily selected based on the local experience.

Figure 5.46 presents a graph showing the percent retained strength against percent Montmorillonite mineral. Percent retained strength after a certain number of cycles is defined as the ratio between the UC strength of the soil specimens obtained after the corresponding number of cycles and the UC strength at the beginning of the wetting drying process i.e. immediately after the curing and moisture conditioning of the soil specimens..

The results in this figure show that as the percent Montmorillonite amount increases in the clay fraction, the loss of strength in a stabilized soil after W/D cycles increases. This is an important finding as it shows that the influence of clay mineralogy on the durability of chemical stabilizer in providing sustained strength over a long time period. This also indicates that the current approach of PI based chemical stabilizer design has not provided any insights into the chemical stabilizer and its durability.

Table 5.3 Total volumetric change and the number of cycles survived along with the dominant clay mineral

S No.	Soil Name	Dominating clay mineral	Type of additive	Amount of additive, (% by weight)	# of cycles sample survived	Total volumetric change, %
1	Austin	Montmorillonite	Lime	6%	7	15
2	Bryan	Kaolinite	Lime	8%	21	6
3	El Paso	Illite	Lime	8%	21	12
4	Fort Worth	Montmorillonite	Lime	6%	10	15
5	Keller	Kaolinite	Lime	6%	21	5
6	Paris	Montmorillonite	Lime	8%	7	15
7	Pharr-A	Montmorillonite	Lime	4%	4	30
8	Pharr-B	Kaolinite	Lime	3%	8	18
9	Pharr-A	Montmorillonite	Cement	4%	4	13
10	Pharr-B	Kaolinite	Cement	3%	14	31

Table 5.4 Percentage drop in strength of different soil samples and the cycles survived along with the dominant clay mineral

S No.	Soil Name	Dominating clay mineral	Type of additive	Amount of additive, (% by weight)	# of cycles sample survived	% Retained strength
1	Austin	Montmorillonite	Lime	6%	7	0
2	Bryan	Kaolinite	Lime	8%	21	93
3	El Paso	Illite	Lime	8%	21	80
4	Fort Worth	Montmorillonite	Lime	6%	10	0
5	Keller	Kaolinite	Lime	6%	21	80
6	Paris	Montmorillonite	Lime	8%	7	0
7	Pharr-A	Montmorillonite	Lime	4%	4	0
8	Pharr-B	Kaolinite	Lime	3%	8	0

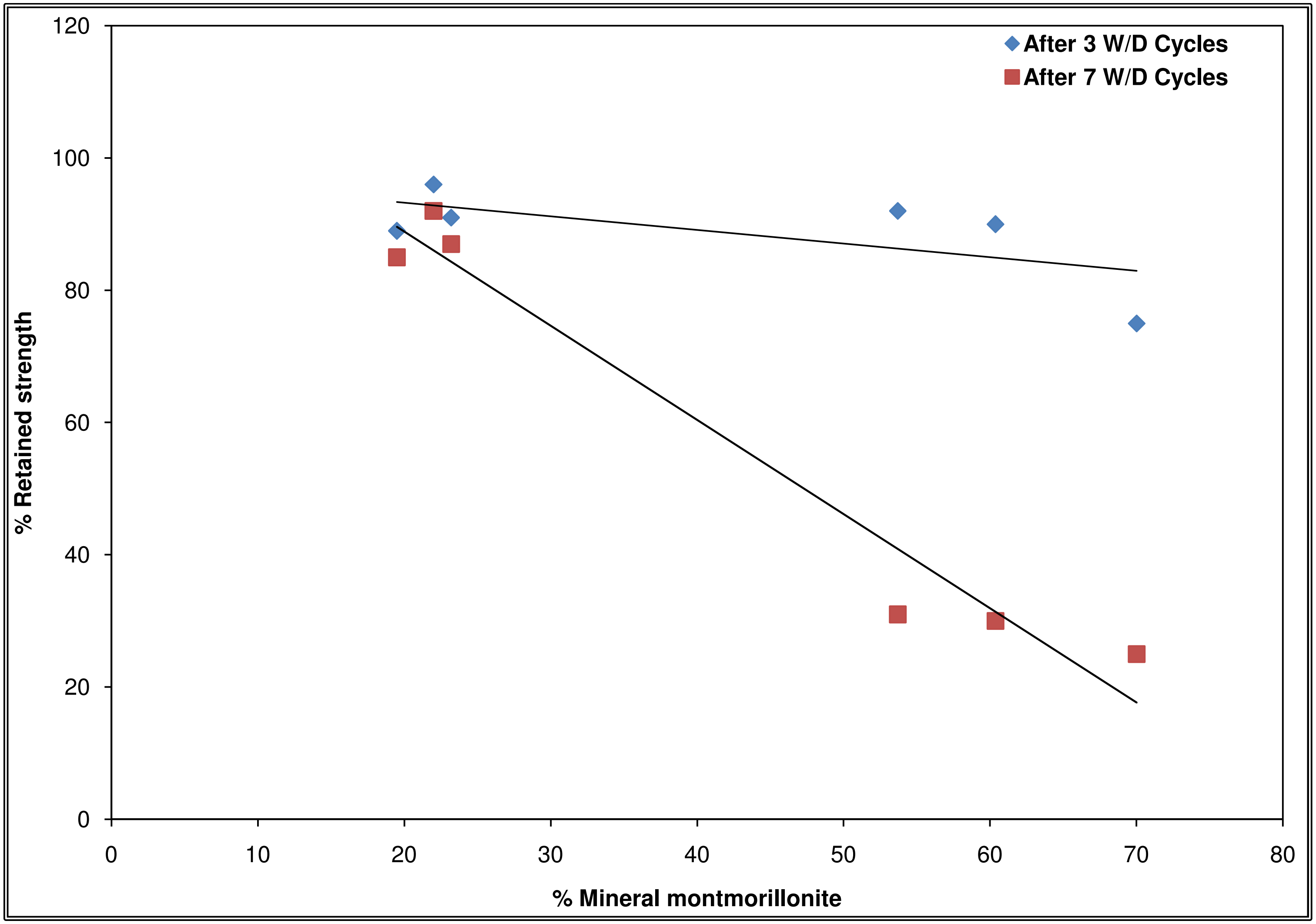


Figure 5.46 Percentage drop in strength variation with percentage of Montmorillonite in different soils for wetting drying cycles

5.3.4.3 Effect of additive type and dosage

To evaluate the effect of type of additive (cement and lime treatments) on the long-term performance of stabilized expansive soils, two soils were tested with same amounts of lime and cement additives. Pharr-A soil which had a high plasticity index (PI) and Pharr-B soil which had a low PI were selected for this study. Table 5.5 summarizes the effect of type of additive used on the durability of the stabilization. It can be observed from the table that cement treatment lasted more number of cycles than lime treatment in medium PI soil. In the case of medium PI soil (Pharr-B), both the treatments (lime and cement) did not last all 21 cycles. For high PI clays, both treatments failed much earlier. The variations in these numbers of cycles of survival were attributed to the presence of the amounts of Montmorillonite in the clay fraction. At high amounts of Montmorillonite, the additive type did not show any appreciable variation in their volume change behavior. At other clay fractions, cement treatment appears to be slightly better than lime treatment. Overall, however, more soils with different PI values are still needed to be tested for further understanding of these additives and their effects under the W/D cycles.

Table 5.5 Comparison of results between the additive types

Soil type	**Additive type**	**Amount of additive, %**	**# of cycles survived**	**% Retained strength after 3 w/d cycles**	**Total Volumetric change after 3 w/d cycles, %**
Pharr-A (High PI)	Lime	4	4	0.5	21.7
Pharr-B (Med PI)	Lime	3	8	83	16.8
Pharr-A (High PI)	Cement	4	4	8.7	26
Pharr-B (Med PI)	Cement	3	14	90	6.8

Table 5.6 summarizes the effect of the amount of chemical additive dosage on the long-term performance of a stabilized subgrade under seasonal moisture variations and volumetric movements due to these moisture fluctuations. Here three different soils Bryan, Keller and Pharr-B which were dominant in the mineral Kaolinite were selected. These soils were treated

with 8%, 6% and 3% lime respectively. It can be observed from the table that the both Bryan and Keller soils survived all the 21 W/D cycles while Pharr-B soil survived only 8 W/D cycles. This premature failure in the case of Pharr-B clay is attributed to low additive dosages used for stabilization. This clearly explains the reasons behind Pharr-B clay being not able to survive all the 21 W/D cycles even though it does not possess Montmorillonite mineral as a dominant fraction in its clay fraction.

Table 5.6 Comparison of results between the additive dosages

Soil type	**Mineral Dominance**	**Additive type**	**Amount of additive, %**	**# of cycles survived**	**% Retained strength after 7 w/d cycles**
Bryan	Kaolinite	Lime	8	21	92
Keller	Kaolinite	Lime	6	21	85
Pharr-B	Kaolinite	Lime	3	8	27.8

5.4 Limitations of the current PI based design methodologies

It has been observed so far in this research study that most of the soils that are stabilized using the current stabilization design procedures did not survive the seasonal moisture fluctuations studied by conducting wetting/drying durability tests on them. The current stabilizer selection criterion followed by various state and federal agencies such as TxDOT, FHWA, Army corps of engineers are based on the plasticity index (PI) of the soil. These organizations have experienced problems with subgrade failure due to a loss of stabilizer over time, or a stabilizer being ineffective in some soils while other soils with the same index properties respond well to that stabilizer. In other cases, the amount of stabilizer was not sufficient to produce a good subgrade foundation system for the pavement structure. All the problems can be attributed to the lack of understanding of the complex interactions between the chemical additive and the clay minerals present in the soils.

In this research study, it has been observed that two different soils from Bryan and Fort Worth with similar PI's of 31 and 29, respectively, behaved differently when treated with lime as an additive. Bryan soil which was dominant with Kaolinite could survive all 21 cycles of wetting/drying whereas Fort Worth soil with Montmorillonite as a dominant clay fraction survived only 10 cycles of wetting and drying. Early failure was attributed to wetting and drying caused by Montmorillonite fraction, which induced early failure of the specimen during durability cycles. Hence, it can be mentioned that the selection of type and amount of stabilizer based on PI of the clay alone is not recommended for the stabilizer design. It is highly important to include the clay mineralogy information in the stabilizer design process to have durable and sustainable subgrade stabilization.

Other limitations are that the practitioners are using lower dosages of chemical additives for stabilization works than they require. In many cases they rely on local experience, which may have come from soils with stronger minerals such as Kaolinite or even Illite minerals. That experience should not be extended to all other soils with similar PI values as this study demonstrated that clays with different clay mineralogy yielded similar PI values, but their engineering behavior could be drastically different. This reconfirms the

5.5 Summary and findings

A total of eight soils were selected for studying the long-term performance of stabilized expansive soils by conducting wetting/drying studies. Stabilizer design was carried out as per the TxDOT methods Tex 120-E (Lime as additive) and Tex 121-E (Cement as additive) and the results were presented. An accelerated curing method was developed and followed in this study for curing and moisture conditioning of the treated soil specimens. The effect of curing methods is studied on four select soils and it is observed that both the curing methods including old and longer curing (Tex 121-E) and present accelerated curing methods yielded similar UCS and volume change results. It has been interpreted from these results that there was no considerable effect of curing on the long-term performance of these treated soils except for an initial strength since soil was partially saturated.

The effect of mineralogy on the long-term performance of stabilized expansive soils was noted in this study. Both volumetric strain and unconfined compressive strength were measured at various numbers of wetting/drying cycles. The results of all the eight soils were presented in detail.

It has been concluded from this study that soils containing Montmorillonite as a dominant mineral are more susceptible to premature failures after chemical stabilization when they are exposed to volume changes caused by swell and shrink related volume changes. However, Pharr B clay though dominant in the mineral Kaolinite as it did not survive all the 21 cycles. The one treated with lime survived only 8 cycles while the one treated with 3% cement survived 14 cycles. The reason for this premature failure could be due to low amount of the additive dosage. The current approach of PI based chemical stabilizer does not provide considerable insights into the chemical stabilizer and its durability. Hence, the proposed research recommends the use of stabilizer design by including clay mineralogy aspects. Such stabilizer design could lead to a more realistic and practical designs and also provide better answers to the permanency of the stabilizer.

Also the effect of the additive type on the long-term performance of stabilized expansive soils was studied in this research. It has been observed that cement treatment lasted more number of cycles in the case of medium PI soil such as Pharr-B soil. Both the treatments (lime and cement) did not last all the 21 cycles in case of Pharr-B, though this soil is not dominant in the mineral Montmorillonite, this contradictory behavior is attributed to low dosages of cement and lime. Further studies are required before making any conclusions about this behavior.

The next chapter describes another aspect of the long-term performance studies of the chemical stabilization, i.e. permanency of stabilizer by addressing additive leachate studies. Moisture infiltration during heavy rain falls and runoff conditions was simulated in these tests. The test results on all eight soils and their analysis are presented in detail in this chapter.

CHAPTER 6

LEACHATE STUDIES

6.1 Introduction

This chapter presents the results and comprehensive analysis of the tests conducted on all eight selected soils to study the effects of clay mineralogy on permanency of the stabilization. The permanency of the additive is affected by the leaching of the chemical stabilizer through moisture movements which may 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 the leaching process emanating from moisture ingress and digress due to seasonal changes and runoff flow conditions. Previous studies report that the leaching through moisture flows in soils result in variations of pH and Calcium ions, which indicate that the permanency of the chemical modifiers are compromised in the treated soils (McCallister, 1990).

A slightly modified test protocol was developed based on the earlier research work performed by McCallister (1990) to address the permanency of the chemical stabilization from moisture flows during rainfall events and ground water flows. This test utilizes a flexible wall mold housing the compacted stabilized soil specimen. Figure 3.16 illustrates a schematic of the test setup used in this research. This setup is similar to the one used by McCallister (1990) for leachate studies conducted at UTA with the exception of a modification in the size of the soil specimen (6 in. diameter in place of 8 in. diameter). The water flow direction was kept in the same direction in the entire test. An attempt is made to simulate the flow scenario close to Texas conditions. The details of the test procedure followed in this research study were already provided in Chapter 3. Test results and their relations with respect to potential moisture flow in field conditions are explained here.

6.2 Leachate studies

Leachate studies were conducted by infiltrating water through the soil specimens and collecting the leachate samples after a certain number of cycles. One full cycle is defined as the time required for flushing out one pore volume of leachate through the soil specimen. A pore volume is defined as the total voids/pores (air voids + water voids) present in a compacted specimen. The steps involved in the calculation of specimen void volume are given in Figure 3.17 of Chapter 3. The time required for each cycle was kept constant as 24 hrs. Hence, the pressure was to be adjusted to collect one void volume of leachate per day. Preliminary tests were conducted to determine these pressures for different soils. A total of 14 leachate cycles were conducted on each of the soil specimen and these results are presented in the following sections.

Leachate studies were conducted on soil specimens following the procedure explained in Chapter 3. Each test was repeated thrice. Leachate samples were collected after 3, 5, 7, 11 and 14 leachate cycles of moisture flow and thereafter unconfined compressive strength tests were conducted on specimens that were subjected to 3, 7 and 14 leachate cycles.

The leachate samples collected were subjected to pH and calcium concentration determination tests. Also, the soil specimens were tested for UCS testing and hence could not be used for any further studies. The results obtained from all tests are presented in the following sections.

6.2.1 Repeatability of the test results

To ensure the repeatability of the test methods in providing similar or repeatable results, triplicate soil specimens were tested. Two typical soil results are presented here (Austin Clay and Bryan Clay). The mean and standard deviations of the calcium concentrations and unconfined compressive strengths of these specimens after 3 and 7 leachate cycles were determined and presented in Table 6.1. It can be observed from this table that the standard deviation of these measurements is low and the coefficient of variation of most of the tests is less than 20% and hence it can be interpreted that both calcium concentrations and strength tests on leachate tested specimens are repeatable.

Table 6.1 Repeatability of leachate tests

Soil Type : Austin						
	Trial 1	**Trial 2**	**Trial 3**	**Mean**	**SD**	**cov, %**
Calcium concentration after 3 leachate cycles, %	750.00	599.00	600.00	649.67	86.89	13.37
Calcium concentration after 7 leachate cycles, %	500.00	390.00	350.00	413.33	77.67	18.79
UC strength after 3 leachate cycles, psi	21.25	30.12	19.50	23.62	5.69	24.10
UC strength after 7 leachate cycles, psi	30.25	25.37	21.50	25.71	4.38	17.06
Soil Type : Bryan						
	Trial 1	**Trial 2**	**Trial 3**	**Mean**	**SD**	**cov, %**
Calcium concentration after 3 leachate cycles, %	400	450	590	480.00	98.49	20.52
Calcium concentration after 7 leachate cycles, %	520	400	490	470.00	62.45	13.29
UC strength after 3 leachate cycles, psi	120.5	90.56	110.4	107.14	15.23	14.21
UC strength after 7 leachate cycles, psi	120.3	100.5	80.25	100.32	20.00	19.94

cov	- Coefficient of variation
SD	- Standard deviation
*	- Measured after wetting cycle

6.2.2 Effect of curing methods

As explained in the previous chapter, two different curing methods were also reviewed here. The current TxDOT procedure for curing and moisture conditioning (Tex 121-E) of a stabilized soil specimen which takes approximately seventeen days and a new accelerated method (AC-1) developed in this study which implements 3 days for curing and moisture conditioning of the soil specimens were compared. The details of the methods were explained in detail in the previous chapter.

The four soils selected for this study are Bryan, El Paso, Paris and Fort Worth. The Bryan soil is dominant in mineral Kaolinite whereas El Paso soil is dominant in mineral Illite. Other two soils Paris and Fort Worth are dominant with Montmorillonite. The test results were obtained by performing leachate studies following both curing methods and these results are presented in the following figures.

Figure 6.1 through Figure 6.4 present the comparison of calcium ion concentration related changes with the number of cycles for both standard cured and accelerated cured specimens of all the four soil types. Figure 6.5 through Figure 6.8 present the comparison of unconfined compression strength changes with respect to the number of cycles for both standard cured and accelerated cured specimens of all four soil types.

From these figures it can be noted that the calcium concentration was slightly high in the case of accelerated cured specimens in all soil samples. However, this value was not appreciable (<50 ppm difference) hence, it can be noted that there is no considerable difference between the curing methods with respect to leachability of the treated soils. All the soils lasted all 21 leachate cycles in both curing methods. The loss of strength in the case of the accelerated cured specimens is attributed to a considerable moisture conditioning obtained in the case of accelerated curing method. Overall, there was no considerable difference among the curing methods. Accelerated curing method was later adapted for the rest of the tests as this method was relatively quick and conservative. The test results obtained after conducting the leachate studies following the accelerated curing method are presented in the following sections.

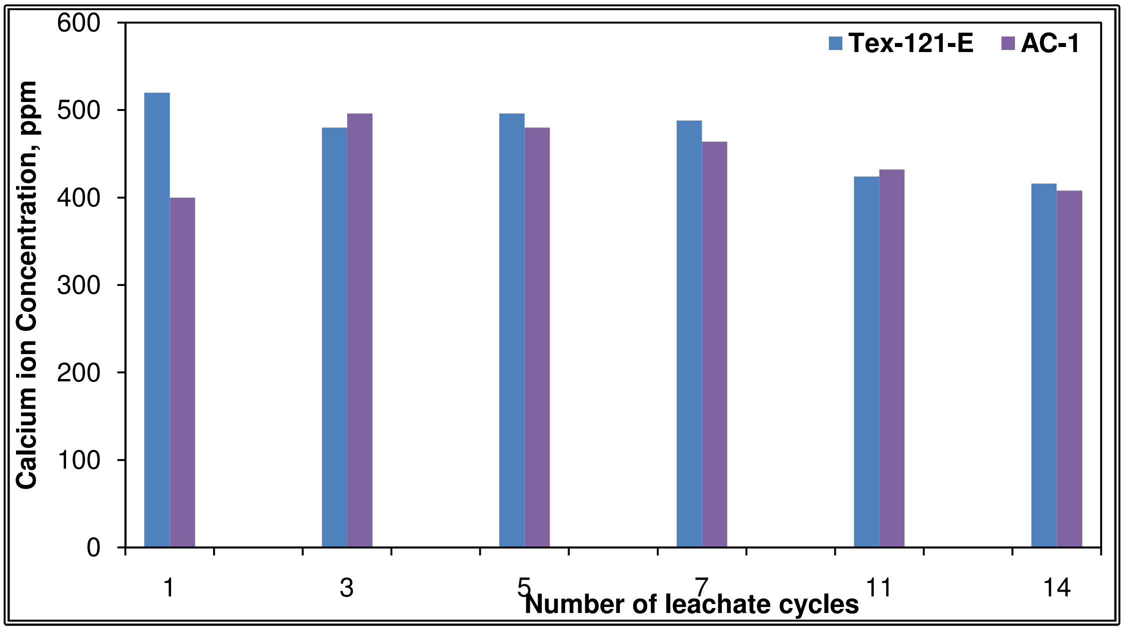


Figure 6.1 Comparison of calcium concentration between the curing methods for Bryan soil specimens

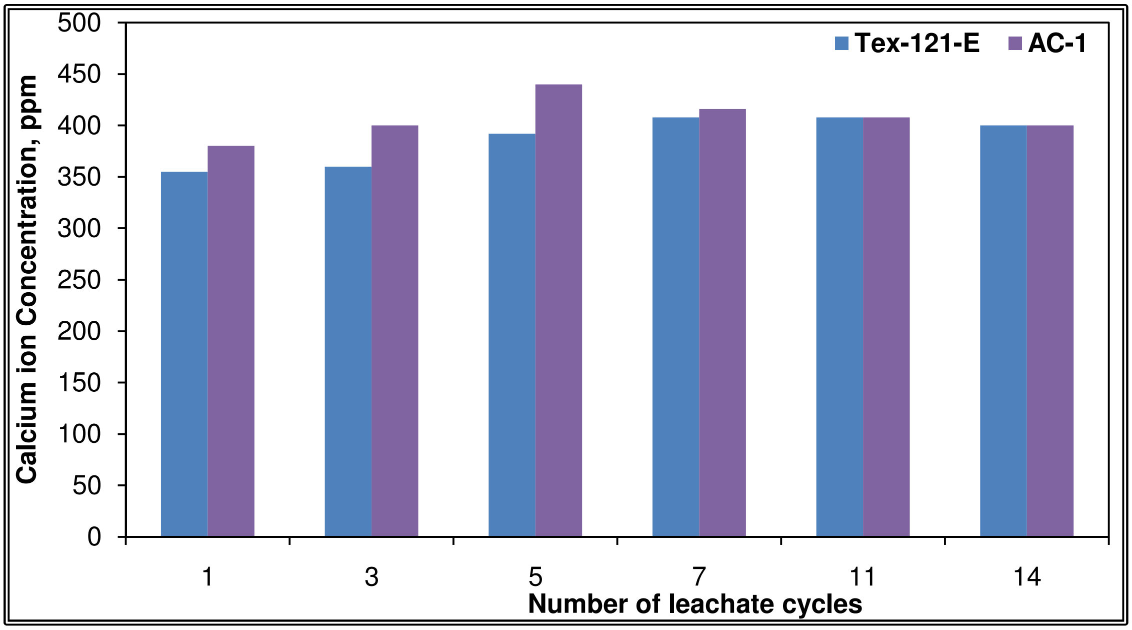


Figure 6.2 Comparison of calcium concentration between the curing methods for El Paso soil specimens

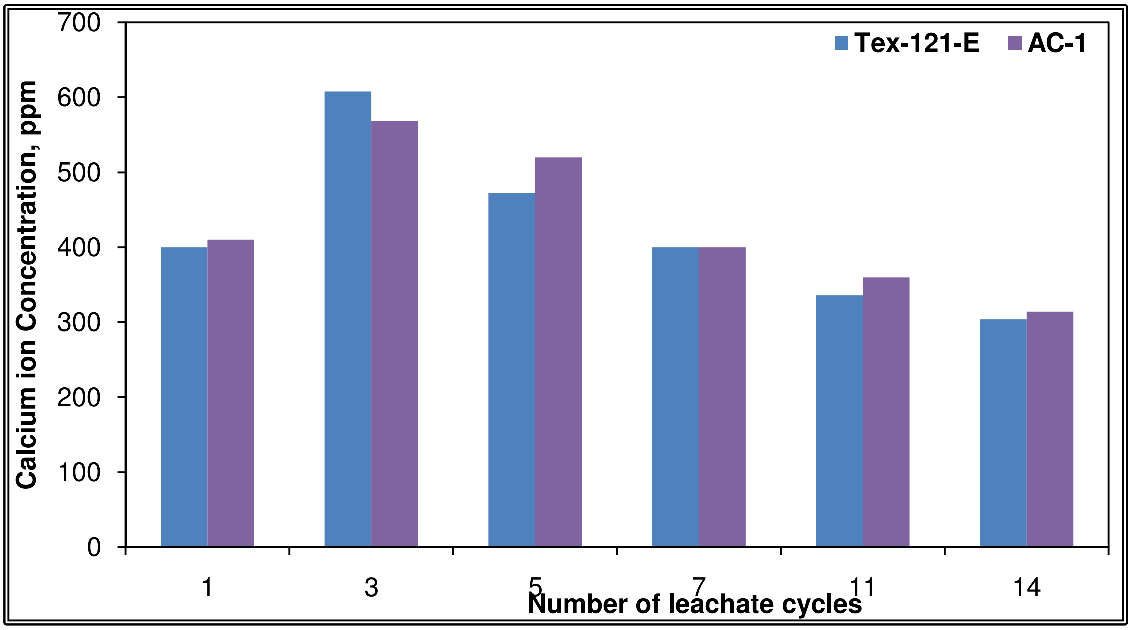


Figure 6.3 Comparison of calcium concentration among the curing methods for Fort Worth soil specimens

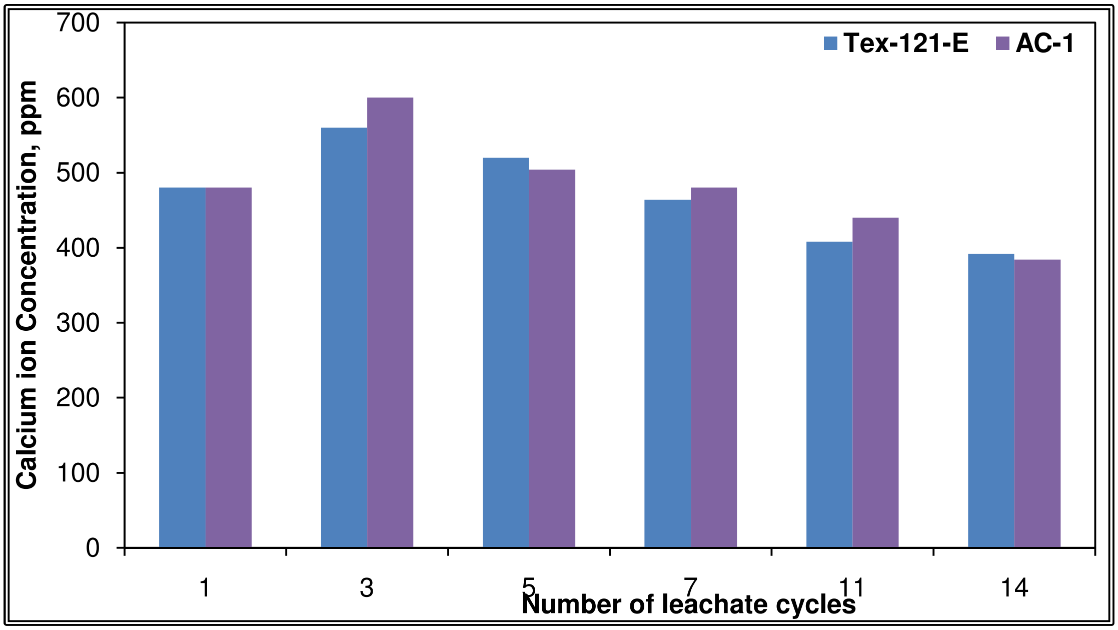


Figure 6.4 Comparison of calcium concentration between the curing methods for Paris soil specimens

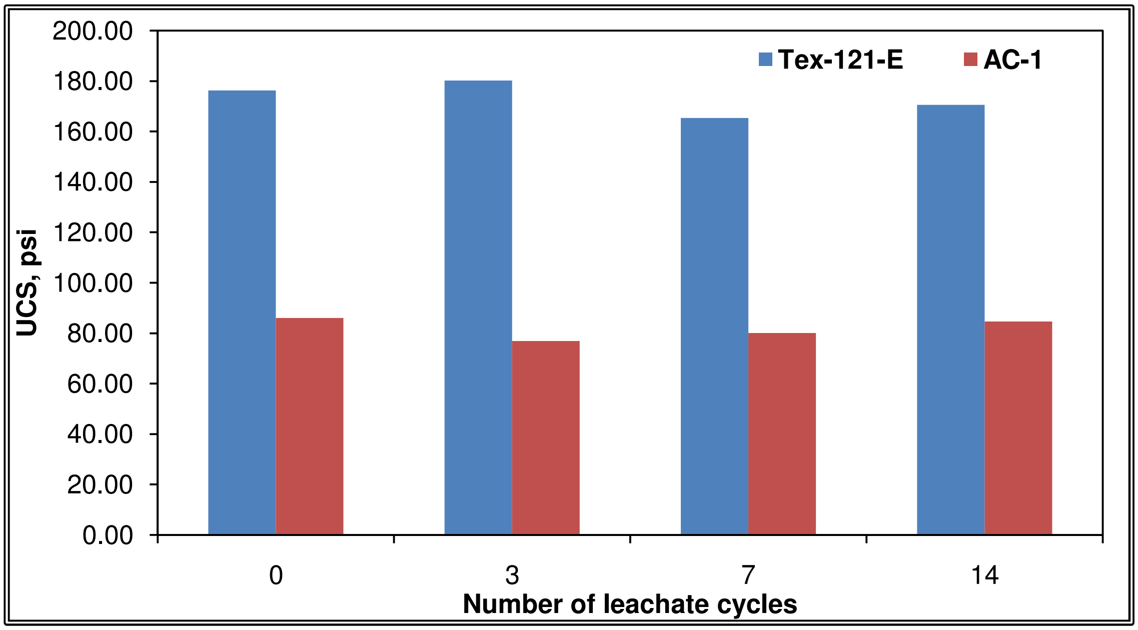


Figure 6.5 Comparison of UC strengths between the curing methods for Bryan soil specimens

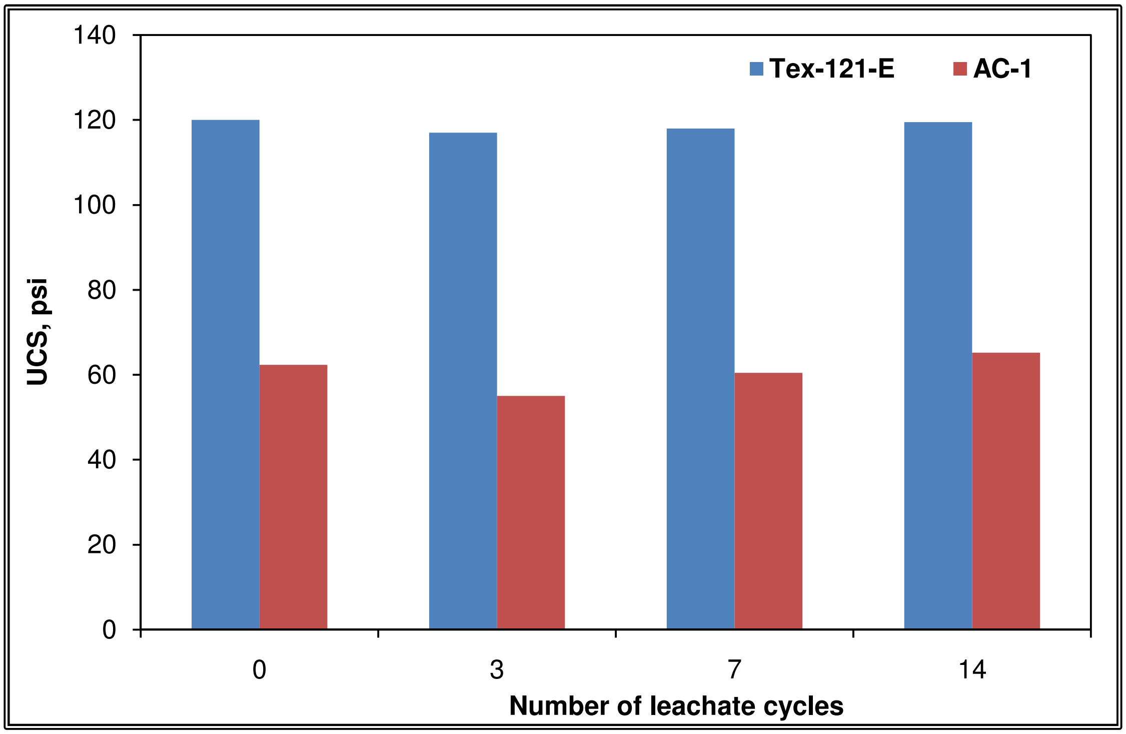


Figure 6.6 Comparison of UC strengths between the curing methods for El Paso soil specimens

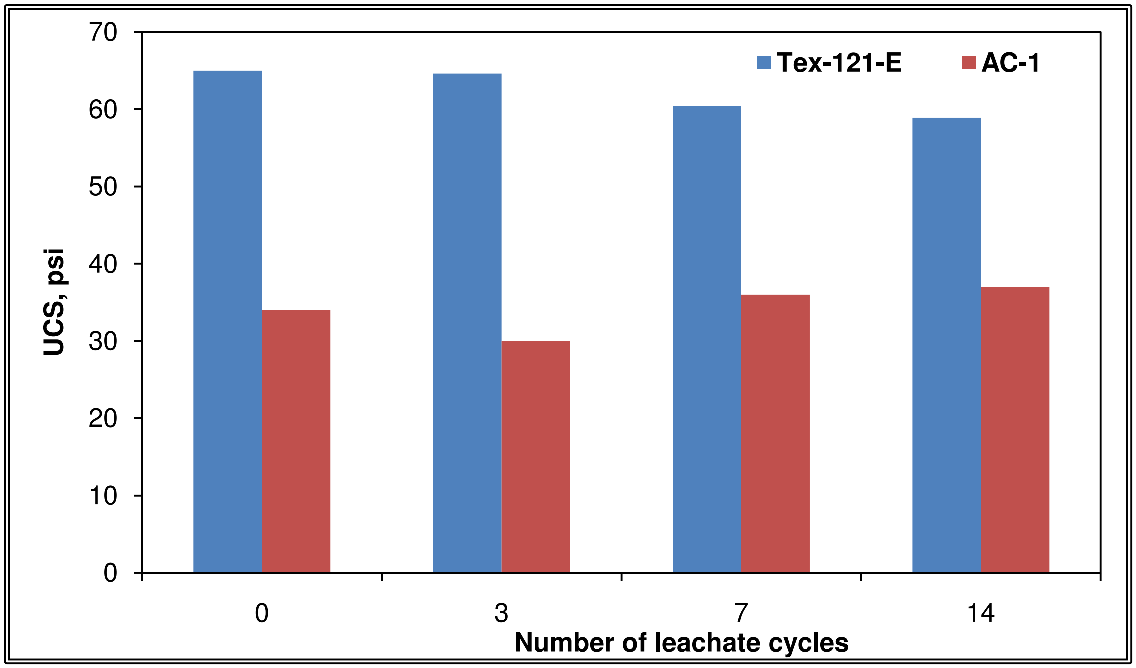


Figure 6.7 Comparison of UC strengths between the curing methods for Fort Worth soil specimens

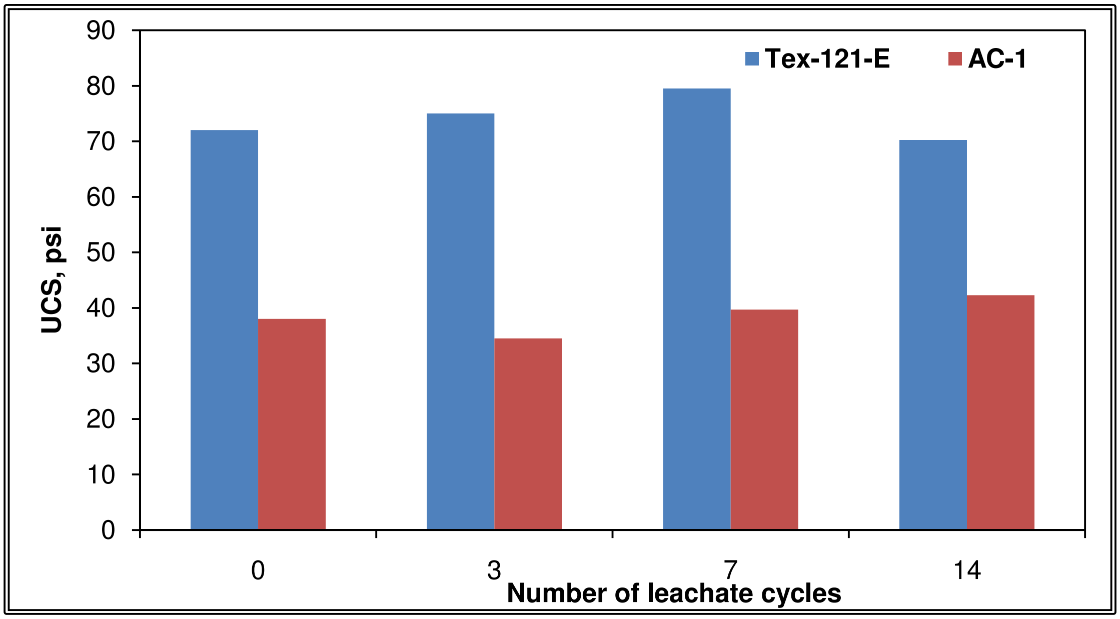


Figure 6.8 Comparison of UC strengths between the curing methods for Paris soil specimens

6.2.3 Summary of Test Results

Results of the leachate tests that were conducted on several identically prepared and cured soil specimens are presented here. Leachate samples collected were tested for ‘pH’ changes and ‘amount of calcium’ present in them after the corresponding leachate cycles. Results were statistically analyzed to address loss of stabilizer due to leaching. An attempt is made to correlate leaching cycles with field moisture movements from rainfall events.

The soils were grouped into three groups based to the mineral dominance in their clay fraction. Group A consisted of four soils from Austin, Fort Worth, Paris and Pharr A (high PI) whose clay fraction is dominated by the presence of the mineral, Montmorillonite. Group B consisted of three soils from Bryan, Keller and Pharr B (medium PI) whose clay fraction is dominated by the presence of the mineral Kaolinite. Finally, group C consisted of soil from El Paso which was dominant with the mineral, Illite in its clay fraction. Results of calcium ion concentration test and pH test conducted on the leachate samples collected after 3, 5, 7, 11 and 14 test results measured during the leachate process and the unconfined strength tests conducted on specimens after 3, 7, and 14 cycles of leachate are presented for each of these soils in the following subsections.

6.2.3.1 Group A results

Austin soil consisted of 53.4% of Montmorillonite, 17.7% of Kaolinite and 28.9% of Illite in its clay fraction. This soil had a plasticity index (PI) of 41 and was classified as a high compressible clay (CH). The soil was treated with 6% lime which was selected based on the current PI based design procedure (Tex-121 E). Figure 6.9 presents the calcium ion concentration changes versus the number of leachate cycles of this soil. It can be observed here that the soil samples lasted all the 14 leachate cycles. Also, the concentration of calcium ion was reducing with the number of leachate cycles in treated soil specimens while that of the control soil specimens remained almost constant. The calcium ion concentration was around 100 ppm in the case of control soil and this value remained same at higher number of leachate cycles. On the other hand, the calcium ion leached out from this soil ranged around 720 ppm at initial leachate cycles and then reduced to 336 ppm at high number of cycles for the treated soil specimens. There is a reduction in the calcium ion concentration in treated soil specimens, indicating that the leaching was taking place. Figure 6.10 presents the pH changes in Austin soil against the number of leachate cycles. The pH of untreated soil ranged from 8.0 to 8.2 over a span of 14 cycles and the same for treated soil ranged from 12.0 to 12.3 over the same span of 14 cycles. It can be observed here that there was very little variation in the pH values in both the control and treated soil specimens.

Figure 6.11 presents the variation of the unconfined compressive strength with leachate cycles. It can be noted from the figure that the treated soil exhibited only 25 psi of UC strength while untreated soil exhibited a UC strength of 32 psi initially. The higher strength of the untreated soil is attributed to the conditions in which the specimens were tested. The control soil specimen was tested at optimum moisture content while the treated soil specimen was tested after moisture conditioning when the specimen was close to saturation. It can be noted here that the control soil retained almost 95% of the initial strength after 14 leachate cycles. The treated soil specimens also retained 96% of the initial strength after 14 cycles of leachate. Hence, it can be observed here that there were no considerable strength changes with the leachate cycles.

The Fort Worth soil consisted of 60.4% of Montmorillonite, 23.4% of Kaolinite and 16.2% of Illite in its clay fraction. This soil had a plasticity index (PI) of 29 and was classified as a high compressible clay (CH). The soil was treated with 6% lime which was selected based on the current PI based design procedure (Tex-121-E).

Figure 6.12 presents the calcium ion concentration changes with the number of leachate cycles of this soil. It can be observed here that the soil samples lasted all the 14 leachate cycles. Also, the concentration of the calcium ion was reducing with the number of leachate cycles in treated soil specimens while that of the control soil specimens remained almost constant. The calcium ion concentration was around 90 ppm in the case of control soil and this value remained the same at higher number of leachate cycles. On the other hand, the calcium ion leached out ranged around 560 ppm at initial leachate cycles and then reduced to 390 ppm at higher number of cycles for the treated soil specimens. There was a reduction in the calcium ion concentration for treated soil specimens indicating the chances of leaching taking place.

Figure 6.13 presents the pH changes with the number of leachate cycles of the Fort Worth soil. The pH of untreated soil ranged from 7.9 to 8.2 over the span of 14 cycles and the treated soil ranged from 12.0 to 12.3 over the span of 14 cycles. It can be observed here that there was no variation in the pH values in both the control and treated soil specimens.

Figure 6.14 presents the variation of the unconfined compressive strength with leachate cycles. It can be noted from the figure that the treated soil exhibited only 34 psi of UC strength while the untreated soil had exhibited 35 psi of strength initially. The higher strength of the untreated soil is attributed to the conditions in which the specimens were tested. The control soil specimen was tested at optimum moisture content while the treated soil specimen was tested after moisture conditioning when the specimen is close to saturation. It can be noted here that the control soil retained almost 99% of the initial strength after 14 leachate cycles. The treated soil specimens retained 99% of the initial strength after 14 cycles of leachate. It can be observed here that there were no considerable strength changes with the leachate cycles.

Paris soil consisted of 70.2% of Montmorillonite, 16.6% of Kaolinite and 13.2% of Illite in its clay fraction. This soil had a plasticity index (PI) of 36 and was classified as a high compressible clay (CH). The soil was treated with 8% lime which was selected based on the current PI based design procedure (Tex-121-E).

Figure 6.15 presents the calcium ion concentration changes with the number of leachate cycles of this soil. It can be observed here that the soil samples lasted all the 14 leachate cycles. Also, the concentration of the calcium ion was reducing with the number of leachate cycles in treated soil specimens while that of the control soil specimens remained almost constant. The calcium ion concentration was around 70 ppm in case of control soil and this value remained same at higher number of leachate cycles. On the other hand, the calcium ion leached out was around 610 at initial cycles of leaching and then reduced to around 310 ppm at higher number of cycles as in the case of the treated soil specimens. There was a reduction in the calcium ion concentration in treated soil specimens indicating the chances of leaching taking place.

Figure 6.16 presents the pH changes with the number of leachate cycles of the Fort Worth soil. The pH of untreated soil ranged from 7.5 to 7.8 over the span of 14 cycles and the treated soil ranged from 12.2 to 11.9 over the span of 14 cycles. It can be observed here that there was not much variation in the pH values in both the control and treated soil specimens.

Figure 6.17 presents the variation of the unconfined compressive strength with leachate cycles. It can be noted from the figure that the treated soil exhibited only 38 psi of UC strength while the untreated soil had exhibited 31 psi of strength initially. This low strength improvement after treatment (from 31 psi to 38 psi) was attributed to the conditions at which the control and the treated soil specimens were tested. The control soil specimen was tested at optimum moisture content while the treated soil specimen was tested after moisture conditioning when the specimen was close to saturation. It can be noted here that the control soil retained almost 90% of the initial strength after 14 leachate cycles. The treated soil specimens retained 85% of the initial strength after 14 cycles of leachate. It can be observed here that there were no considerable strength changes with the leachate cycles.

Pharr-A soil consisted of 47.8 % of Montmorillonite, 26.4% of Kaolinite and 25.8% of Illite in its clay fraction. This soil had a plasticity index (PI) of 45 and was classified as a high compressible clay (CH). The soil was treated with both lime and cement to check the effect of type of additive on the long-term performance of the stabilized soils. 4% lime and 4% cement were selected based on the current PI based design procedures (Tex-120-E for cement and Tex-121-E for lime).

Figure 6.18 presents the calcium ion concentration changes with the number of leachate cycles of this soil treated with 4% lime. It can be observed here that the soil samples lasted all the 14 leachate cycles. Also, the concentration of the calcium ion was reducing with the number of leachate cycles in treated soil specimens while that of the control soil specimens remained almost constant. The calcium ion concentration was around 70 ppm in case of control soil and this value remained same at higher number of leachate cycles. On the other hand, the calcium ion leached out was around 310 at initial leachate cycles and then reduced to 200 ppm at higher number of cycles in case of the treated soil specimens. There was a reduction in the calcium ion concentration in case of treated soil specimens indicating the chances of leaching taking place.

Figure 6.19 presents the calcium ion concentration changes with the number of leachate cycles of this soil treated with 4% cement. It can be observed here that the soil samples lasted all the 14 leachate cycles. Also, the concentration of the calcium ion was reducing with the number of leachate cycles in treated soil specimens while that of the control soil specimens remained almost constant. The calcium ion concentration was around 70 ppm in case of control soil and this value remained same at higher number of leachate cycles. On the other hand, the calcium ions leached out was around 220 ppm at initial leachate cycles and then reduced to 160 ppm at higher number of cycles in case of the treated soil specimens. There was a reduction in the calcium ion concentration in the case of treated soil specimens indicating the chances of leaching taking place.

Figure 6.20 presents the pH changes with the number of leachate cycles of the Pharr-A soil when treated with 4% lime. The pH of untreated soil ranged from 8.1 to 8.2 over a span of 14 cycles and the treated soil ranged from 12.2 to 12.3 over the same span of 14 cycles. It can be observed here that there was little variation in the pH values in both the control and treated soil specimens.

Figure 6.21 presents the pH changes with the number of leachate cycles of the Pharr-A soil when treated with 4% cement additive. The pH of untreated soil ranged from 8.1 to 8.2 over a span of 14 cycles and the treated soil ranged from 12.2 to 12.3 over the span of 14 cycles. It can be observed here that there was no much variation in the pH values in both the control and treated soil specimens.

Figure 6.22 presents the variation of the unconfined compressive strength with leachate cycles for this soil treated with 4% lime. It can be noted from the figure that the treated soil exhibited only 47 psi of UC strength and the untreated soil displayed 37 psi of strength initially. This slight increase in strength (10 psi) after treatment was attributed to the conditions in which the specimens were tested. The control soil specimens were tested at optimum moisture content while the treated soil specimens were tested after moisture conditioning when the specimens were close to saturation. It can be noted here that the control soil retained almost 95% of the initial strength after 14 leachate cycles. The treated soil specimens retained 75% of the initial strength after 14 cycles of leachate. It can be observed here that there were no considerable strength changes with the leachate cycles.

Figure 6.23 presents the variation of the unconfined compressive strength with leachate cycles for this soil treated with 4% cement. It can be noted from the figure that the treated soil displayed only 40 psi of UC strength and the untreated soil exhibited 37 psi of strength initially. This small increase in strength (3 psi) after treatment was attributed to the conditions in which the specimens were tested. The control soil specimens were tested at optimum moisture content while the treated soil specimens were tested after moisture conditioning when the specimens were close to saturation. It can be noted here that the control soil retained almost 95% of the initial strength after 14 leachate cycles. The treated soil specimens retained 85% of the initial strength after 14 cycles of leachate. It can be observed here that there were no considerable strength changes with the leachate cycles.

It should be noted here that all the above soils though dominant with the Montmorillonite mineral and did not show any premature failures from the leachate studies. This indicates that leaching does not have any effect on the long-term performance of stabilized expansive subgrades. However, further analysis is required to make any such conclusions. A detailed analysis of this data is carried out in the following sections.

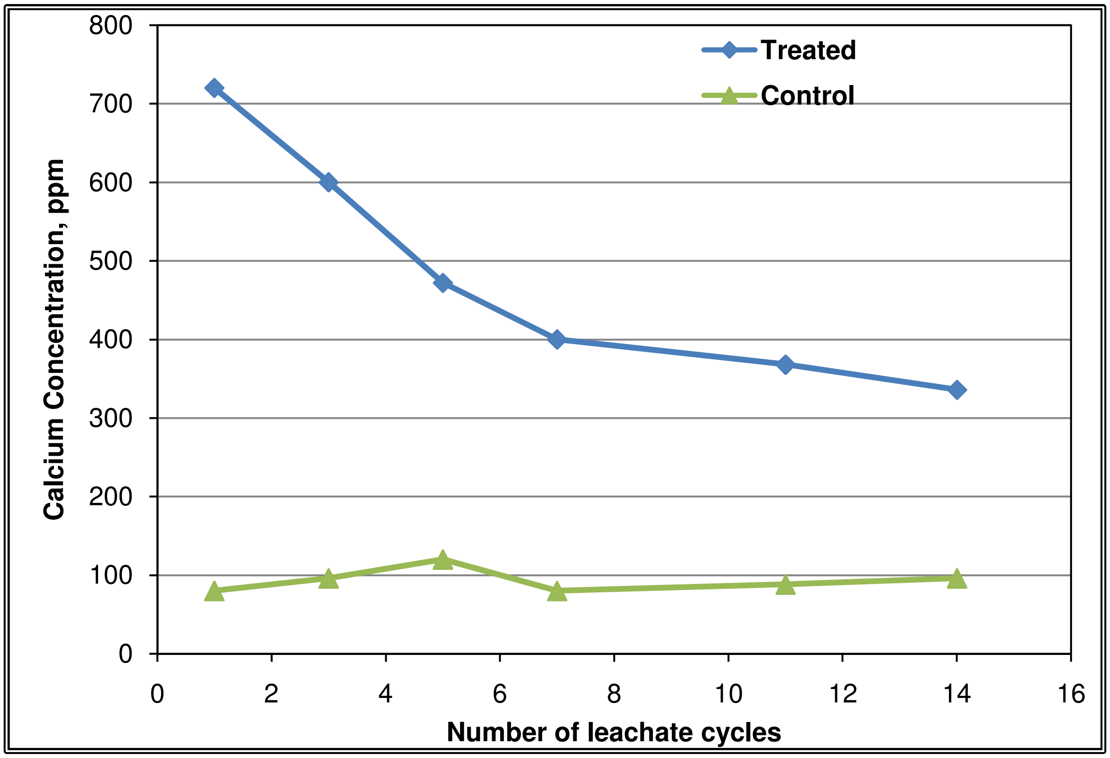


Figure 6.9 Changes in the concentration of calcium ion with leachate cycles for Austin soil specimens

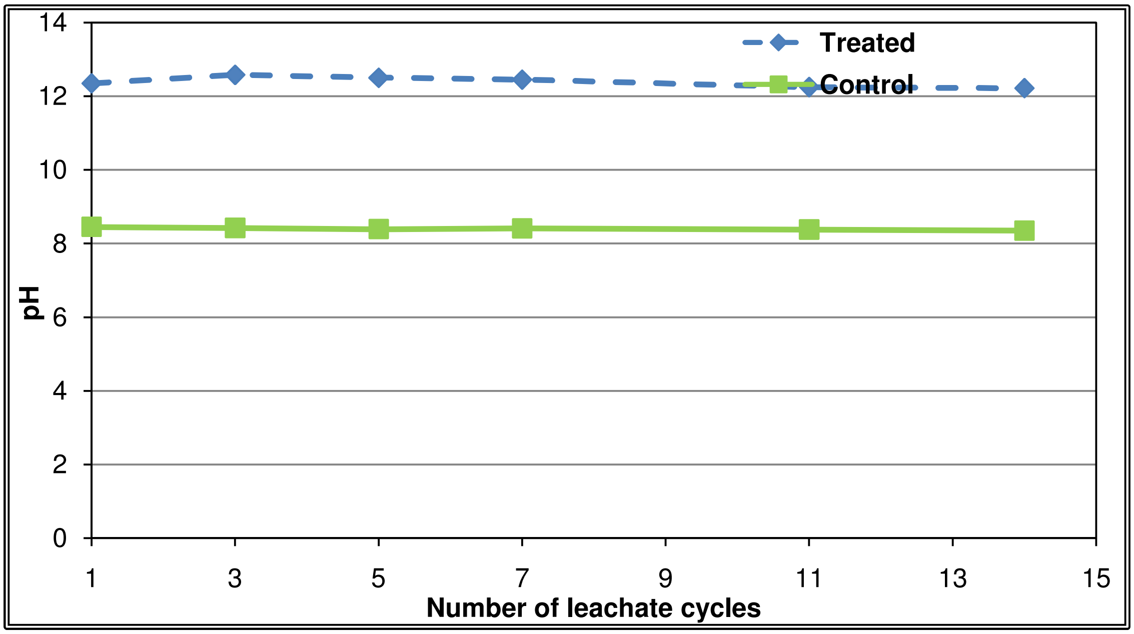


Figure 6.10 Variation of pH with leachate cycles for Austin soil specimens

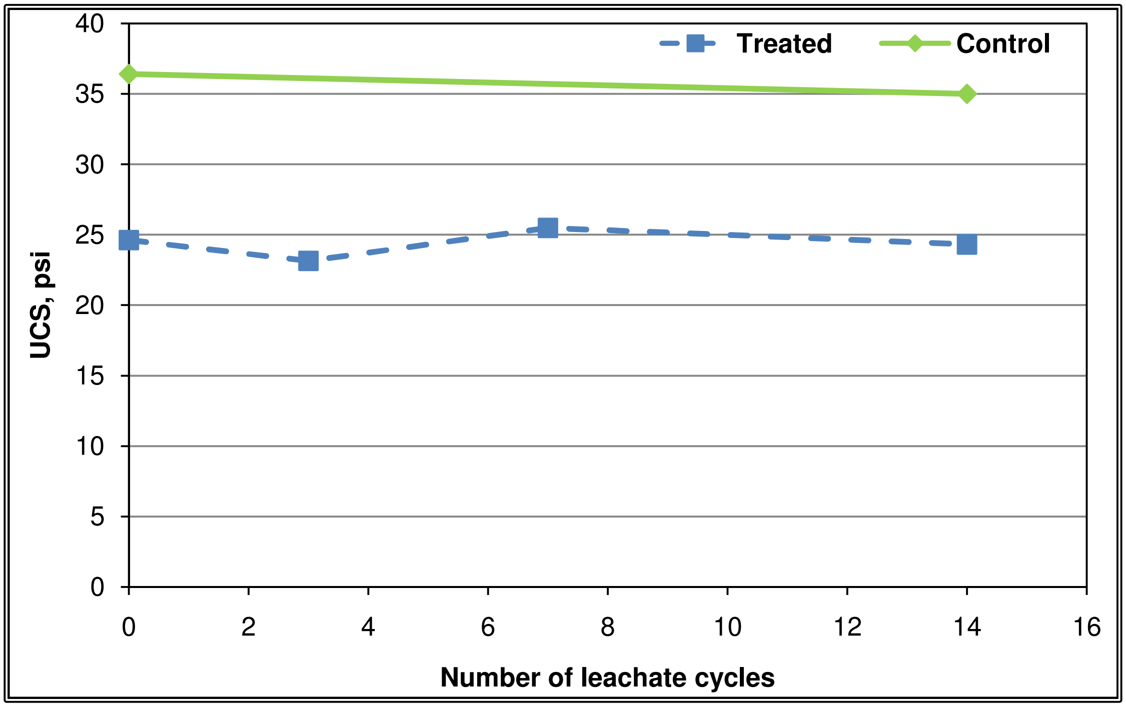


Figure 6.11 Changes in UCS with leachate cycles for Austin soil specimens

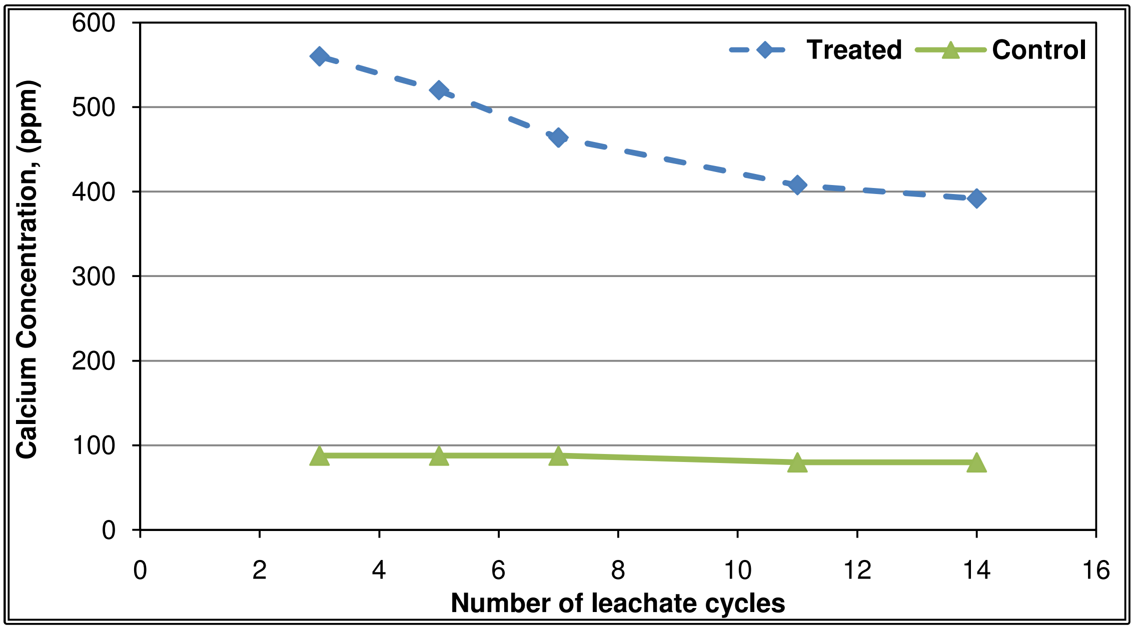


Figure 6.12 Changes in the concentration of calcium ion with leachate cycles for Fort Worth soil specimens

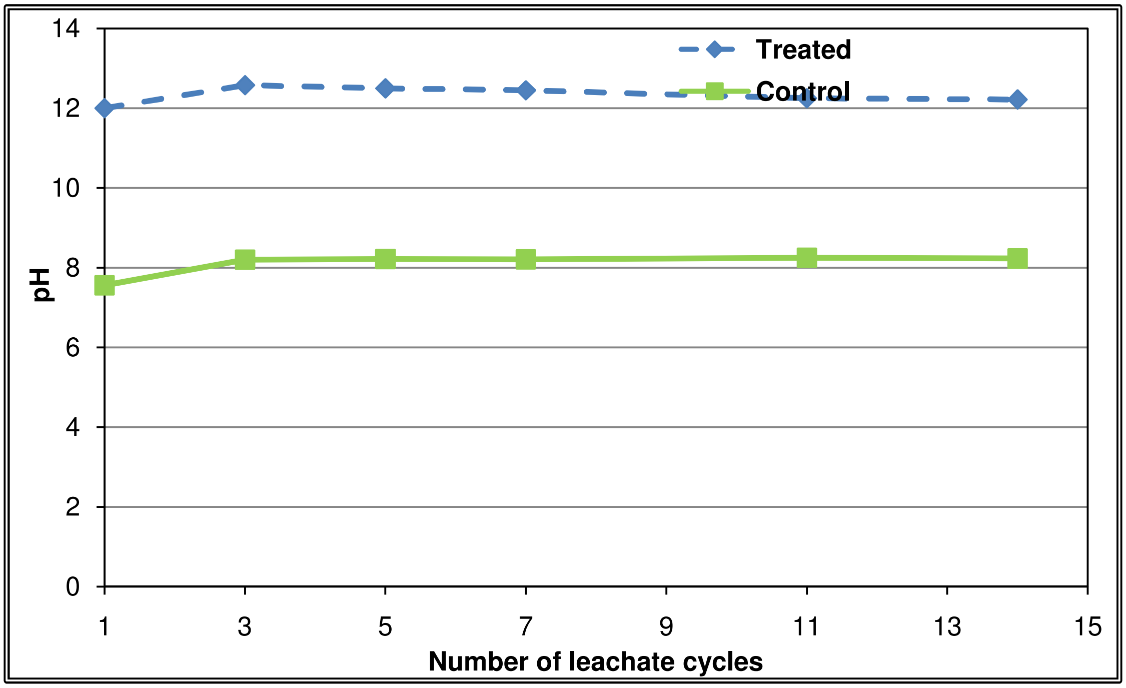


Figure 6.13 Variation in pH with leachate cycles for Fort Worth soil specimens

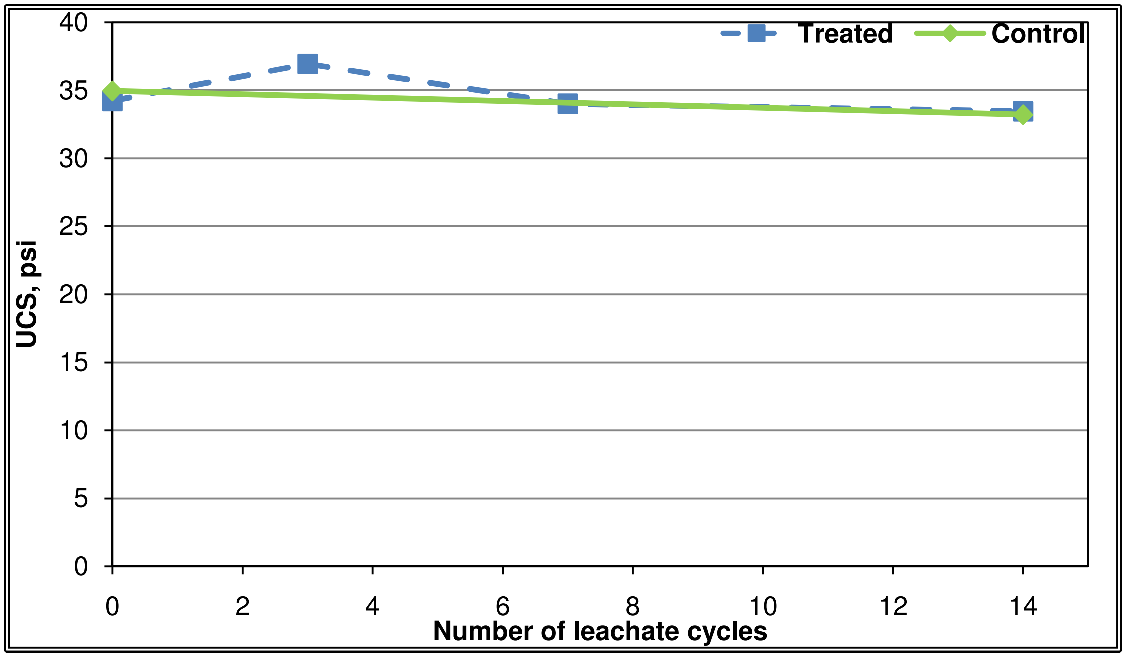


Figure 6.14 Changes in UCS with leachate cycles for Fort Worth soil specimens

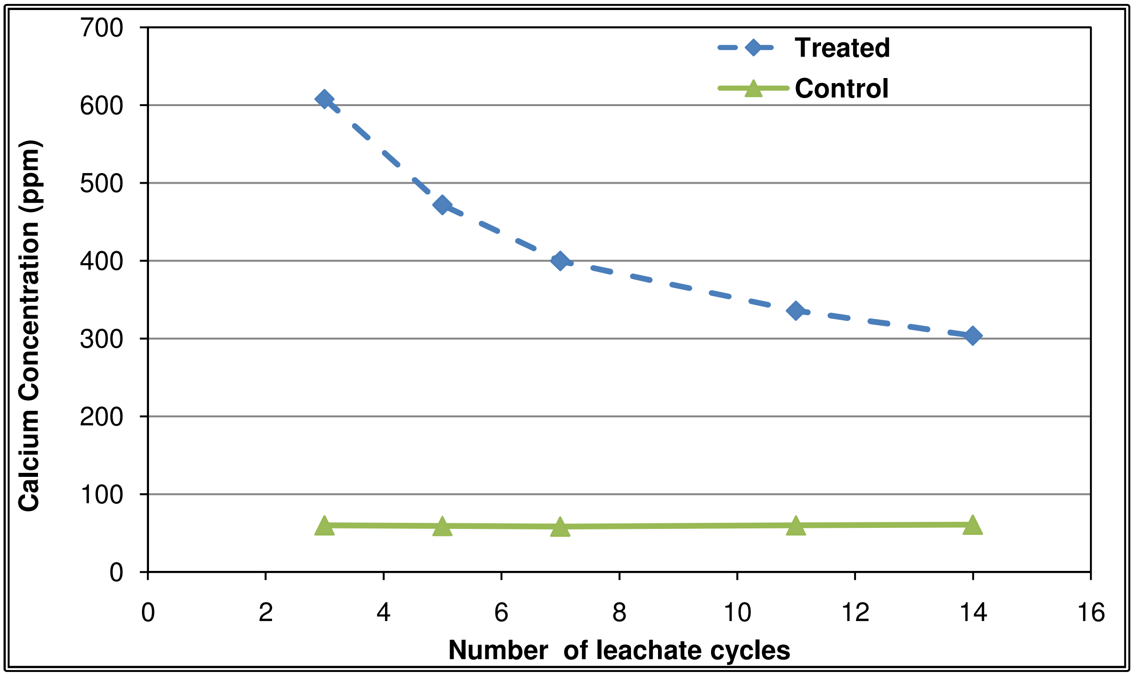


Figure 6.15 Changes in the concentration of calcium ion with leachate cycles for Paris soil specimens

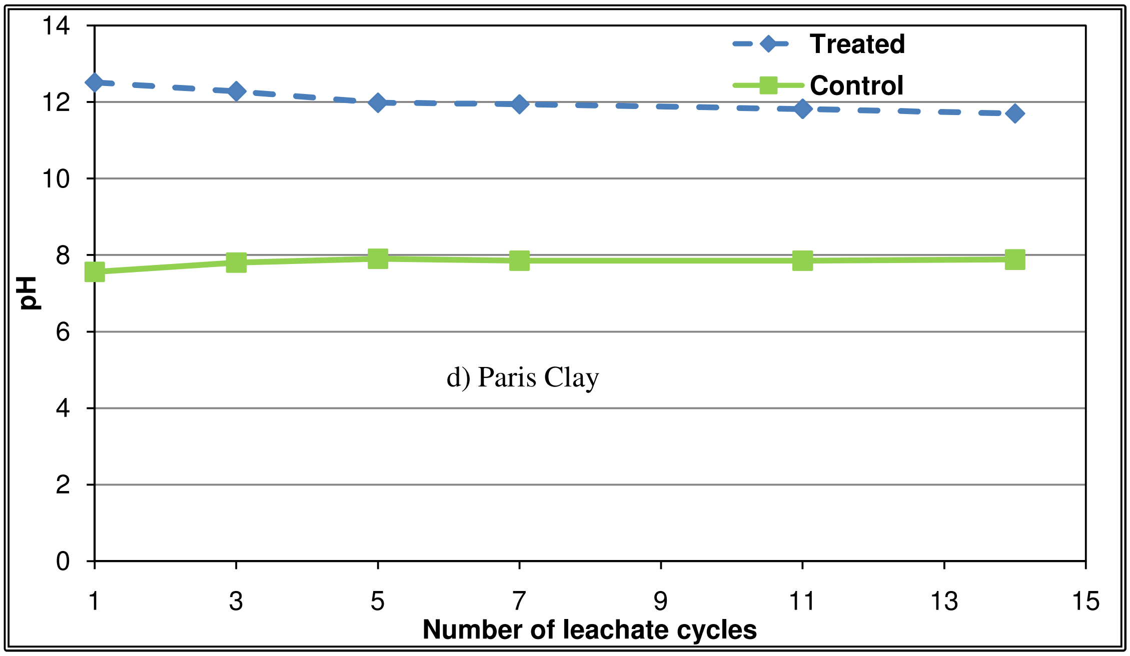


Figure 6.16 Variation in pH with leachate cycles for Paris soil specimens

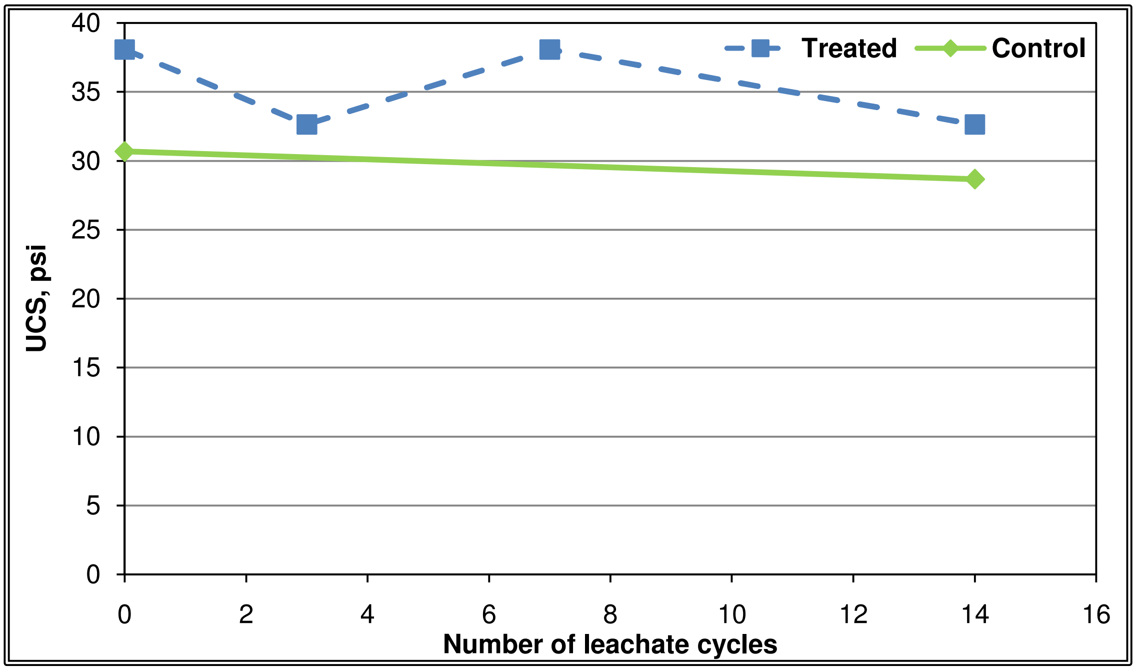


Figure 6.17 Changes in UCS with leachate cycles for Paris soil specimens

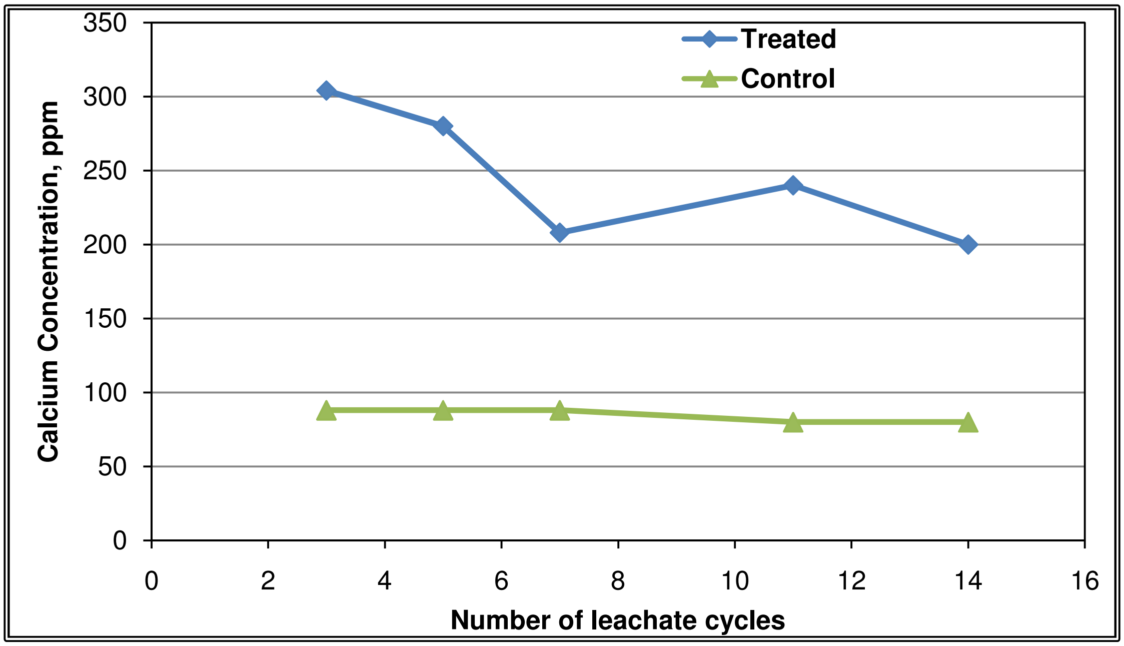


Figure 6.18 Changes in the concentration of calcium ion with leachate cycles for Pharr-A soil specimens

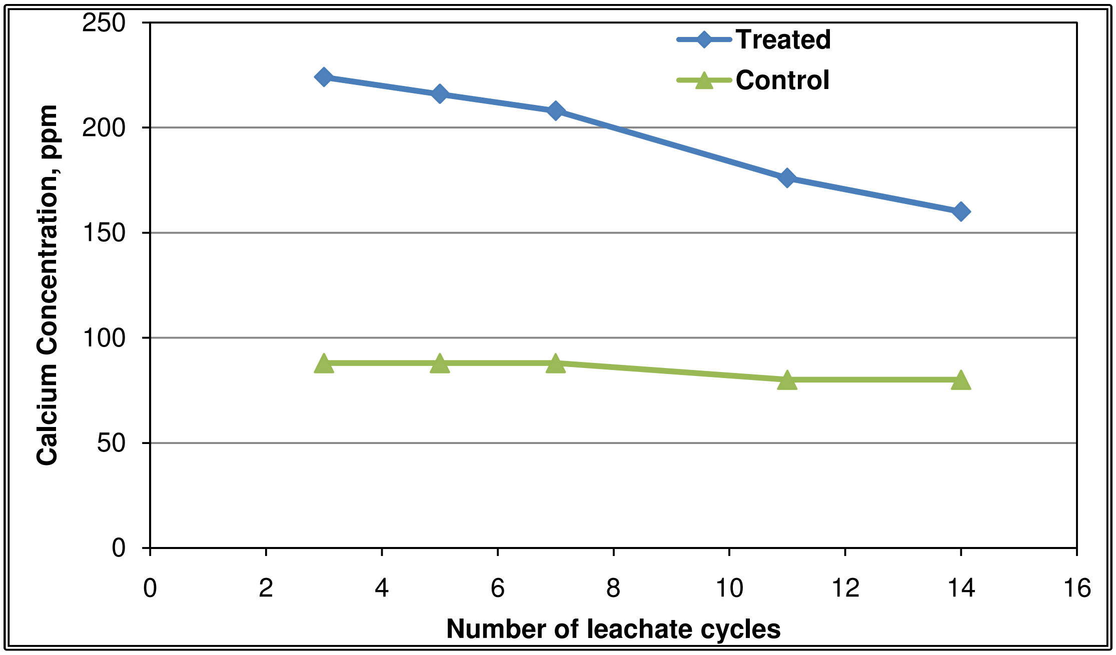


Figure 6.19 Changes in the concentration of calcium ion with leachate cycles for Pharr-A soils

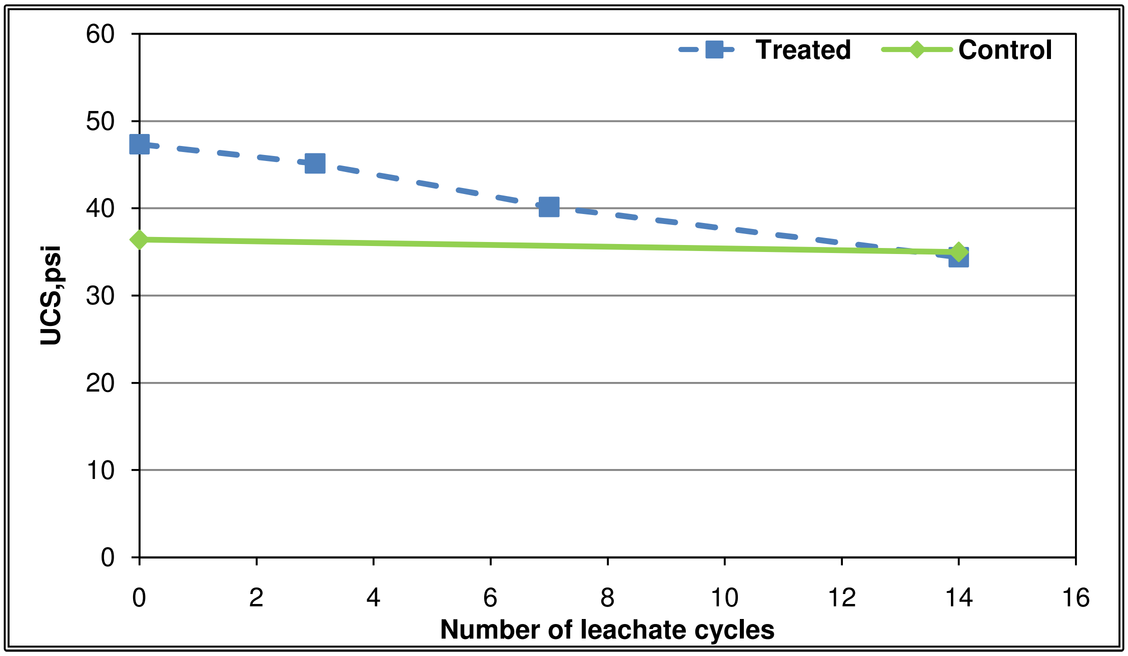


Figure 6.20 Variation in pH with leachate cycles for lime treated Pharr-A soil specimens

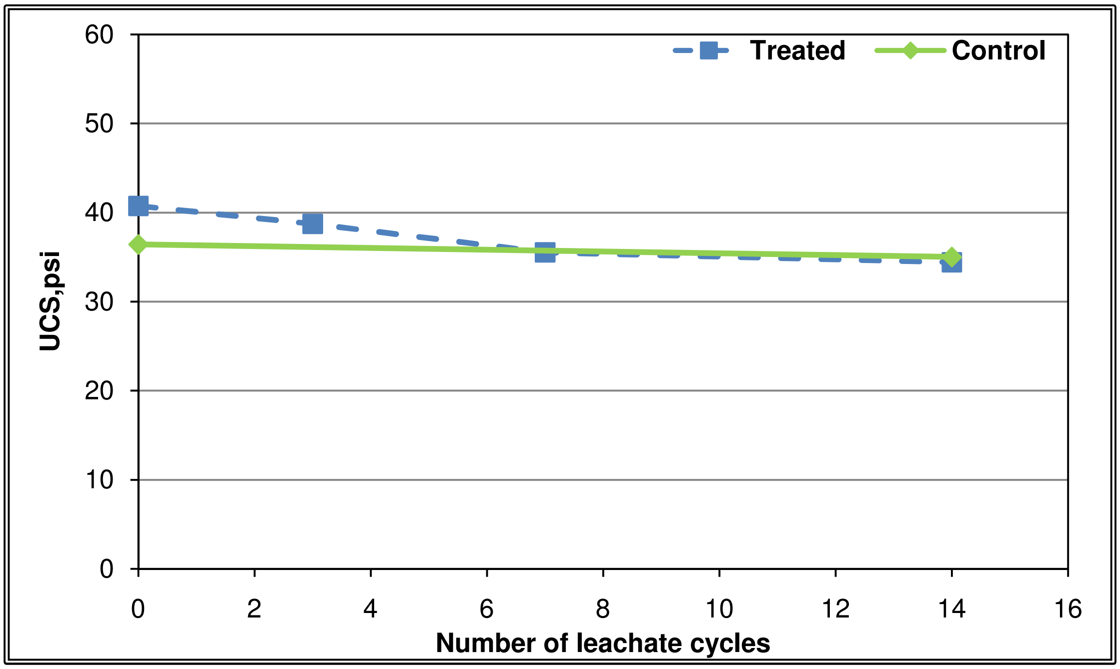


Figure 6.21 Variation in pH with leachate cycles for cement treated Pharr-A soil specimens

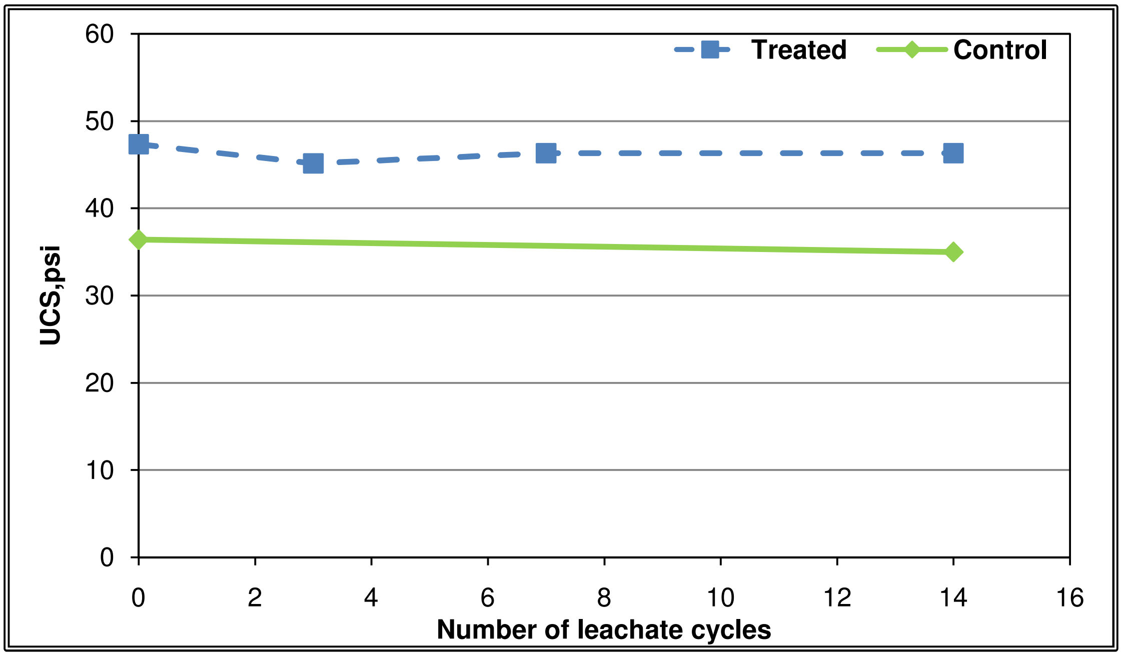


Figure 6.22 Changes in UCS with leachate cycles for lime treated Pharr-A soil specimens

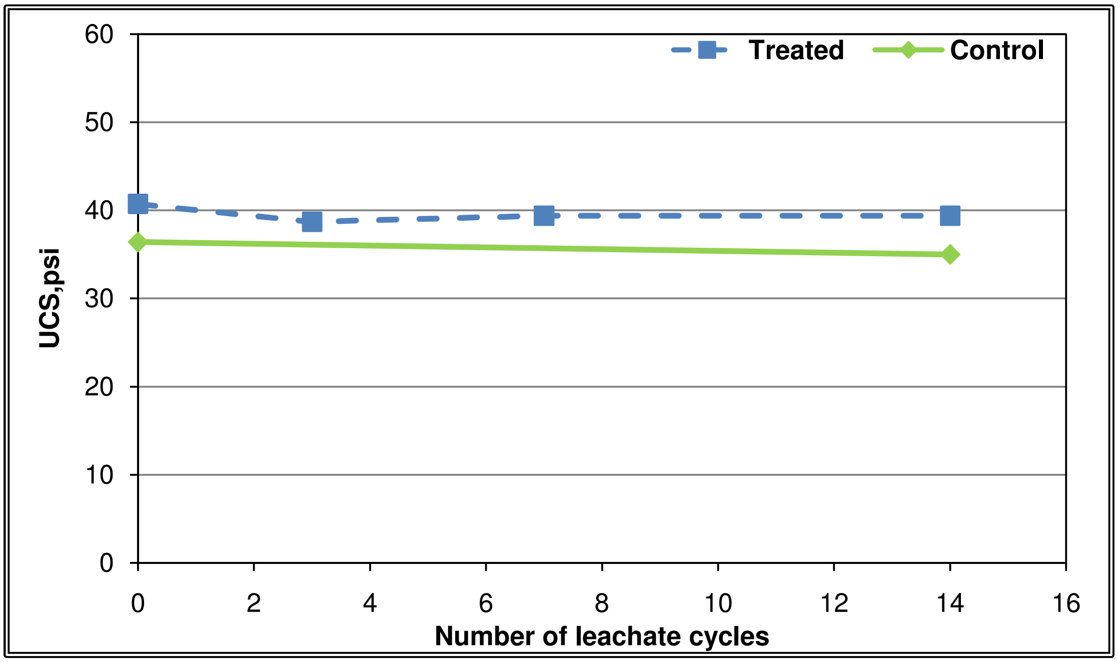


Figure 6.23 Changes in UCS with leachate cycles for cement treated Pharr-A soil specimens

6.2.3.2 Group B results

Bryan soil consisted of 37% of Montmorillonite, 40% of Kaolinite and 23% of Illite in its clay fraction. This soil had a plasticity index (PI) of 31 and was classified as a low compressible clay (CL). The soil was treated with 8% lime which was selected based on the current PI based design procedure (Tex-121-E).

Figure 6.24 presents the calcium ion concentration changes with the number of leachate cycles of this soil. It can be observed here that the soil samples lasted all the 14 leachate cycles. Also, the concentration of the calcium ion was reducing with the number of leachate cycles in treated soil specimens while that of the control soil specimens remained almost constant. The calcium ion concentration was around 80 ppm in case of control soil and this value remained same at higher number of leachate cycles. On the other hand the calcium ion leached out was around 500 at initial leachate cycles and then reduced to 400 ppm at higher number of cycles in case of the treated soil specimens. There was a reduction in the calcium ion concentration in case of treated soil specimens indicating the chances of leaching taking place.

Figure 6.25 presents the pH changes with the number of leachate cycles of the Bryan soil. The pH of untreated soil ranged from 7.9 to 8.0 over the span of 14 cycles and the treated soil ranged from 12.0 to 12.1 over the span of 14 cycles. It can be observed here that there was no much variation in the pH values in both the control and treated soil specimens.

Figure 6.26 presents the variation of the unconfined compressive strength with leachate cycles. It can be noted from the figure that the treated soil exhibited 92 psi of UC strength while the untreated soil displayed only 22 psi of strength initially. Though the conditions at which the soil specimens were tested were different, the treated soil specimens which were tested after moisture conditioning when the specimens were close to saturation exhibited higher strengths than the control soil specimens which were at optimum moisture content at the time of testing showed lower strengths in this case. The control soil retained 95% of the strength while the treated soil specimens retained 89% of the initial strength after 14 cycles of leachate. It can be observed here that there were no considerable strength changes with the leachate cycles.

Keller soil consisted of 19.5% of Montmorillonite, 62.2% of Kaolinite and 18.3% of Illite in its clay fraction. This soil had a plasticity index (PI) of 11 and was classified as a lean clay (CL). The soil was treated with 6% lime which was selected based on the current PI based design procedure (Tex-121-E).

Figure 6.27 presents the calcium ion concentration changes with the number of leachate cycles of this soil. It can be observed here that the soil samples lasted all the 14 leachate cycles. Also, the concentration of the calcium ion was reducing with the number of leachate cycles in the treated specimens while that of the control soil specimens remained almost constant. The calcium ion concentration was around 50 ppm in case of control soil and this value remained same at higher number of leachate cycles. On the other hand the calcium ion leached out was around 720 at initial leachate cycles and then reduced to 330 ppm at higher number of cycles in case of the treated soil specimens. There was a reduction in the calcium ion concentration in case of treated soil specimens indicating the chances of leaching taking place.

Figure 6.28 presents the pH changes with the number of leachate cycles of the Keller soil. The pH of untreated soil ranged from 7.8 to 7.9 over the span of 14 cycles and the treated soil ranged from 12.1 to 12.0 over the span of 14 cycles. It can be observed here that there was no much variation in the pH values in both the control and treated soil specimens.

Figure 6.29 presents the variation of the unconfined compressive strength with leachate cycles. It can be noted from the figure that the treated soil exhibited 53 psi of UC strength while the untreated soil displayed only 31 psi of strength initially. Though the conditions at which the soil specimens were tested were different, the treated soil specimens which were tested after moisture conditioning when the specimens were close to saturation exhibited higher strengths than the control soil specimens which were at optimum moisture content at the time of testing showed lower strengths in this case. The control soil retained 98% of the strength while the treated soil specimens retained 97% of the initial strength after 14 cycles of leachate. It can be observed here that there were no considerable strength changes with the leachate cycles.

Pharr-B soil consisted of 18.3% of Montmorillonite, 54.2% of Kaolinite and 27.5% of Illite in its clay fraction. This soil had a plasticity index (PI) of 37 and was classified as a high compressible clay (CH). The soil was treated with both lime and cement to check the effect of type of additive on the long-term performance of the stabilized soils. 3% lime and 3% cement were selected based on the current PI based design procedures (Tex-120-E for cement and Tex-121-E for lime).

Figure 6.30 presents the calcium ion concentration changes with the number of leachate cycles of this soil treated with 3% lime. It can be observed here that the soil samples lasted all the 14 leachate cycles. Also, the concentration of the calcium ion was reducing with the number of leachate cycles in the treated specimens while that of the control soil specimens remained almost constant. The calcium ion concentration was around 65 ppm in case of control soil and this value remained same at higher number of leachate cycles.

On the other hand the calcium ion leached out was around 510 at initial leachate cycles and then reduced to 400 ppm at higher number of cycles in case of the treated soil specimens. There was a reduction in the calcium ion concentration in case of treated soil specimens indicating the chances of leaching taking place.

Figure 6.31 presents the calcium ion concentration changes with the number of leachate cycles of this soil treated with 3% cement. It can be observed here that the soil samples lasted all the 14 leachate cycles. Also, the concentration of the calcium ion was reducing with the number of leachate cycles in the treated specimens while that of the control soil specimens remained almost constant. The calcium ion concentration was around 65 ppm in case of control soil and this value remained same at higher number of leachate cycles. On the other hand the calcium ion leached out was around 250 at initial leachate cycles and then reduced to 200 ppm at higher number of cycles in case of the treated soil specimens. There was a reduction in the calcium ion concentration in case of treated soil specimens indicating the chances of leaching taking place.

Figure 6.32 presents the pH changes with the number of leachate cycles of the Pharr-B soil when treated with 3% lime. The pH of untreated soil ranged from 8.0 to 7.9 over the span of 14 cycles and the treated soil ranged from 12.2 to 12.3 over the span of 14 cycles. It can be observed here that there was no much variation in the pH values in both the control and treated soil specimens.

Figure 6.33 presents the pH changes with the number of leachate cycles of the Pharr-B soil when treated with 3% cement. The pH of untreated soil ranged from 8.0 to 7.9 over the span of 14 cycles and the treated soil ranged from 12.2 to 12.3 over the span of 14 cycles. It can be observed here that there was no much variation in the pH values in both the control and treated soil specimens.

Figure 6.34 presents the variation of the unconfined compressive strength with leachate cycles for this soil treated with 3% lime. It can be noted from the figure that the treated soil exhibited only 28 psi of UC strength while the untreated soil displayed 41 psi of UC strength initially. The lower strength of the treated specimens was attributed to the low dosage of the additive and conditions in which the specimens were tested. The control soil specimens were tested at optimum moisture content while the treated soil specimens were tested after moisture conditioning when the specimens were close to saturation. It can be noted here that the control soil retained almost 88% of the initial strength after 14 leachate cycles. The treated soil specimens retained 55% of the initial strength after 14 cycles of leaching.

Figure 6.35 presents the variation of the unconfined compressive strength with leachate cycles for this soil treated with 3% cement. It can be noted from the figure that the treated soil exhibited 57 psi of UC strength and the untreated soil displayed 41 psi of strength initially. This small increase of 16 psi after treatment was attributed to the conditions in which the specimens were tested. The control soil specimens were tested at optimum moisture content while the treated soil specimens were tested after moisture conditioning when the specimens were close to saturation. It can be noted here that the control soil retained almost 88% of the initial strength after 14 leachate cycles. The treated soil specimens retained 81% of the initial strength after 14 cycles of leachate. It can be observed here that there were no considerable strength changes with the leachate cycles.

It should be noted here that all the above soils were dominant in the Kaolinite mineral and did not show any premature failures similar to the behavior under wetting/drying process. This indicates that leaching does not have any effect on the long-term performance of stabilized expansive sub-grades. However further analysis is required to make any such conclusions. A detailed analysis of this data is carried out in the following section.

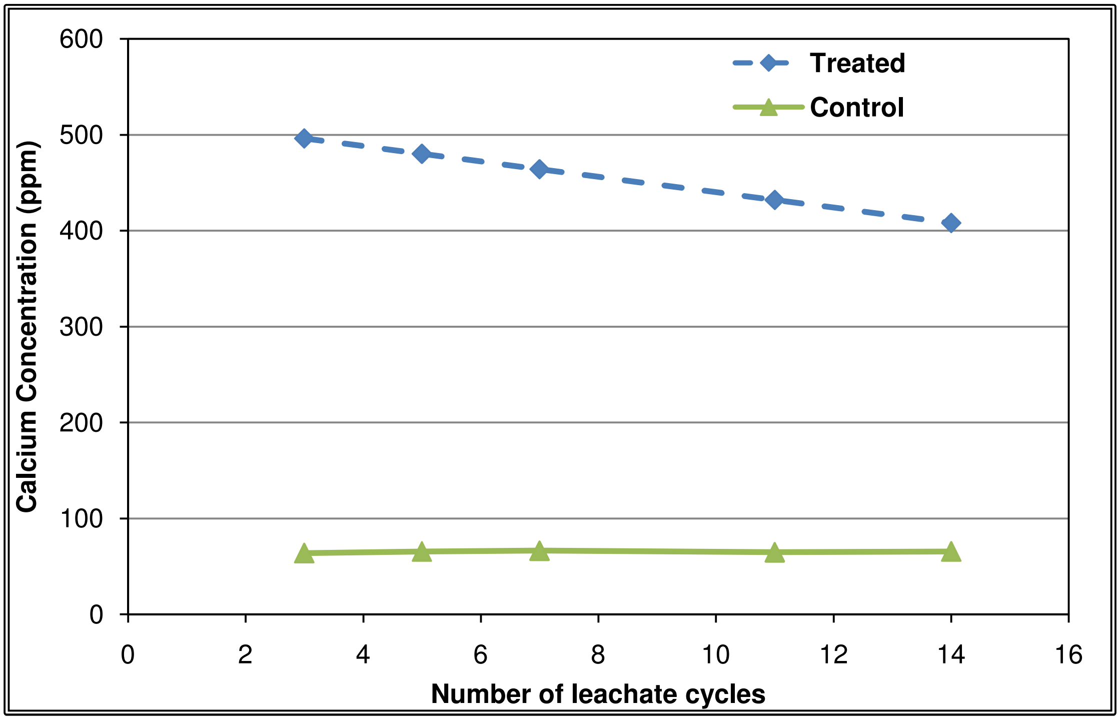


Figure 6.24 Changes in the concentration of calcium ion with leachate cycles for Bryan soil specimens

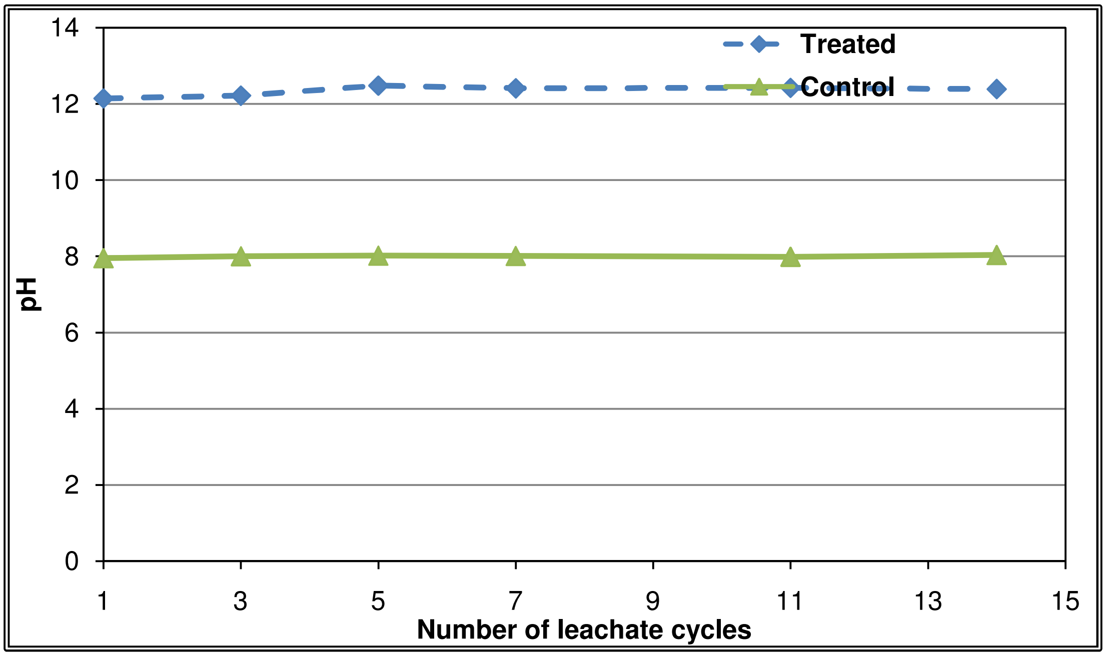


Figure 6.25 Variation in pH with leachate cycles for Bryan soil specimens

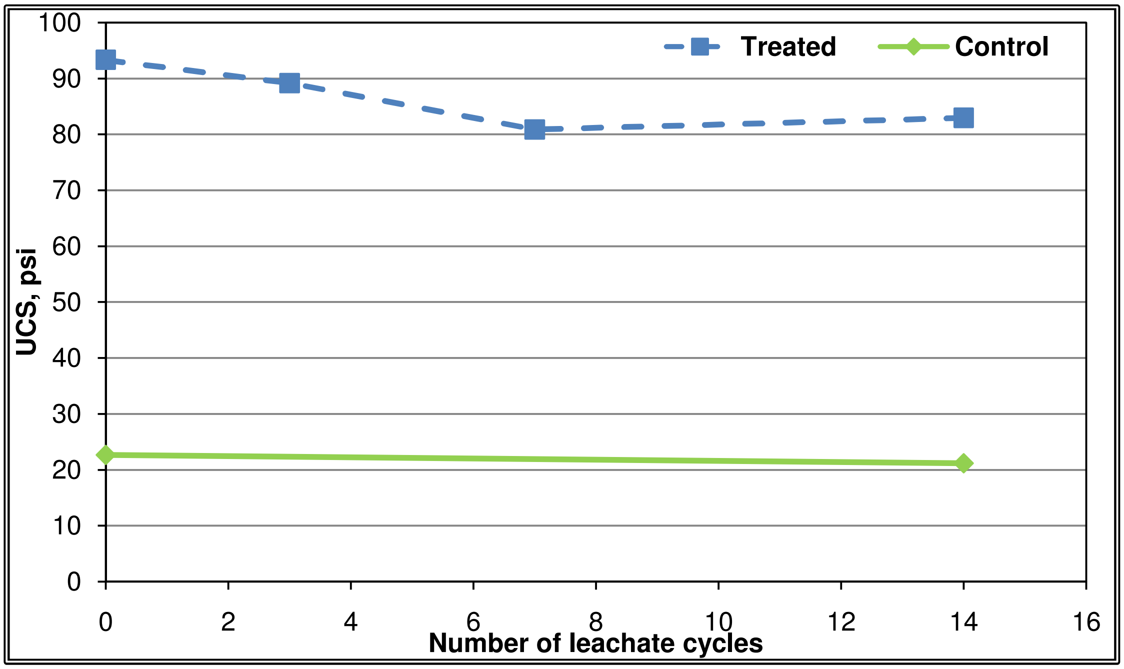


Figure 6.26 Changes in UCS with leachate cycles for Bryan soil specimens

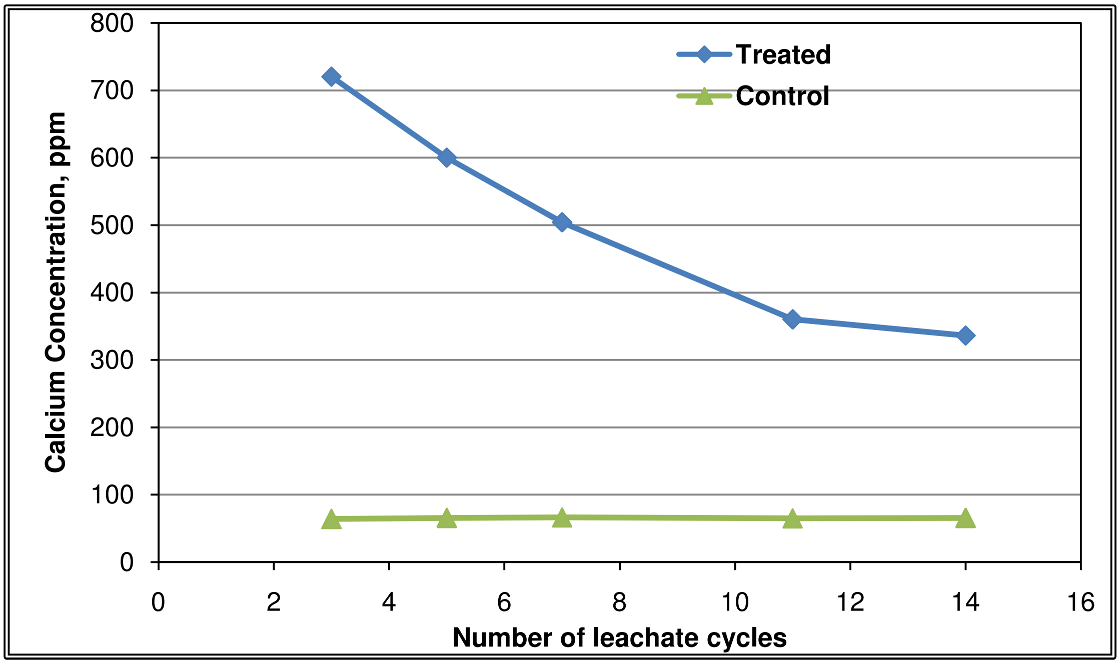


Figure 6.27 Changes in the concentration of calcium ion with leachate cycles for Keller soil specimens

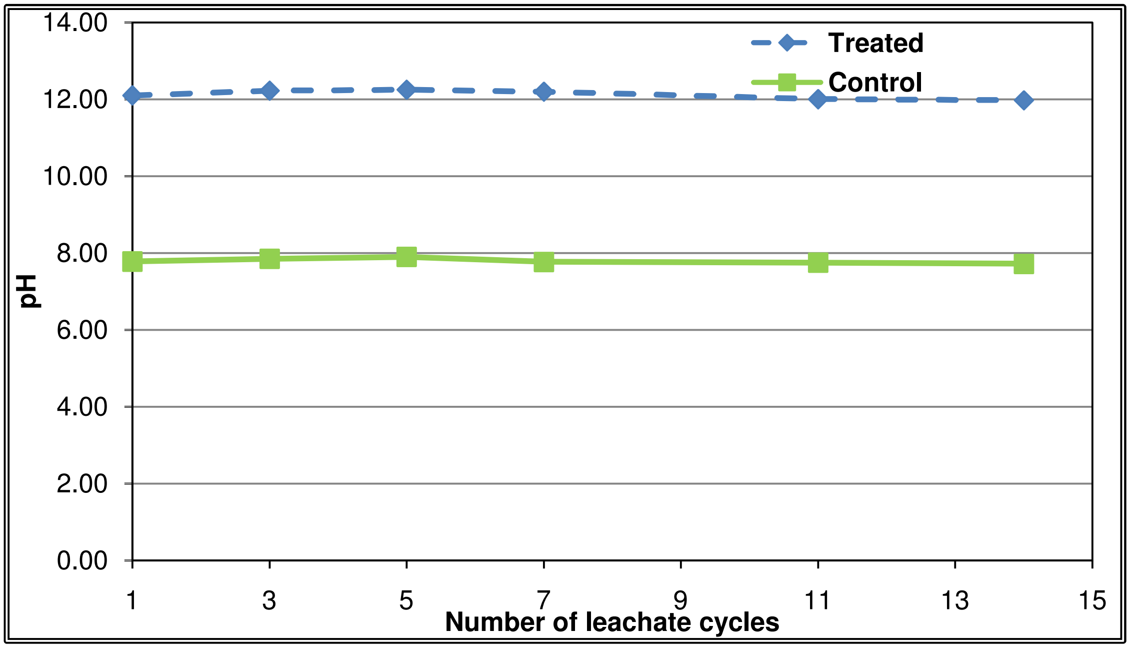


Figure 6.28 Variation in pH with leachate cycles for Keller soil specimens

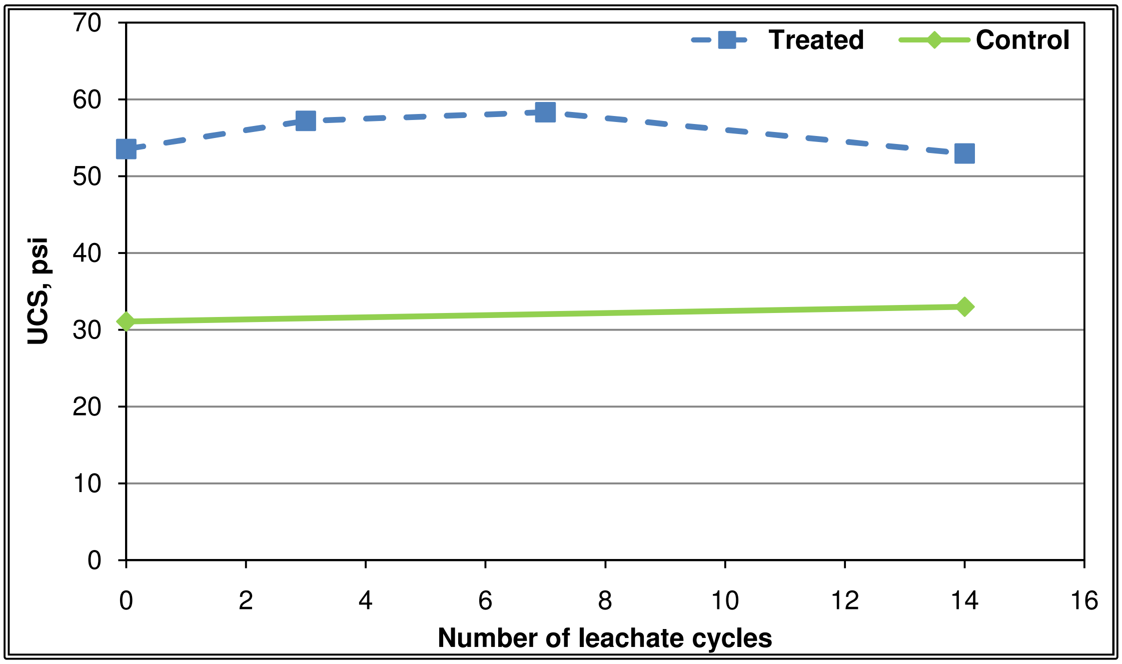


Figure 6.29 Changes in UCS with leachate cycles for Keller soil specimens

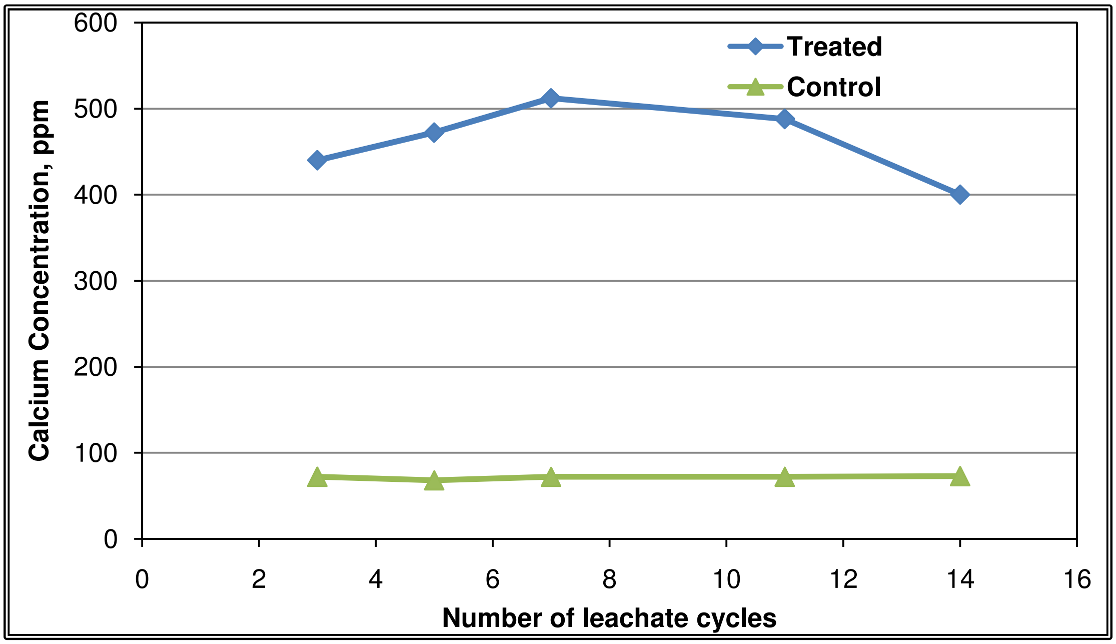


Figure 6.30 Changes in the concentration of calcium ion with leachate cycles for Pharr-B soil specimens

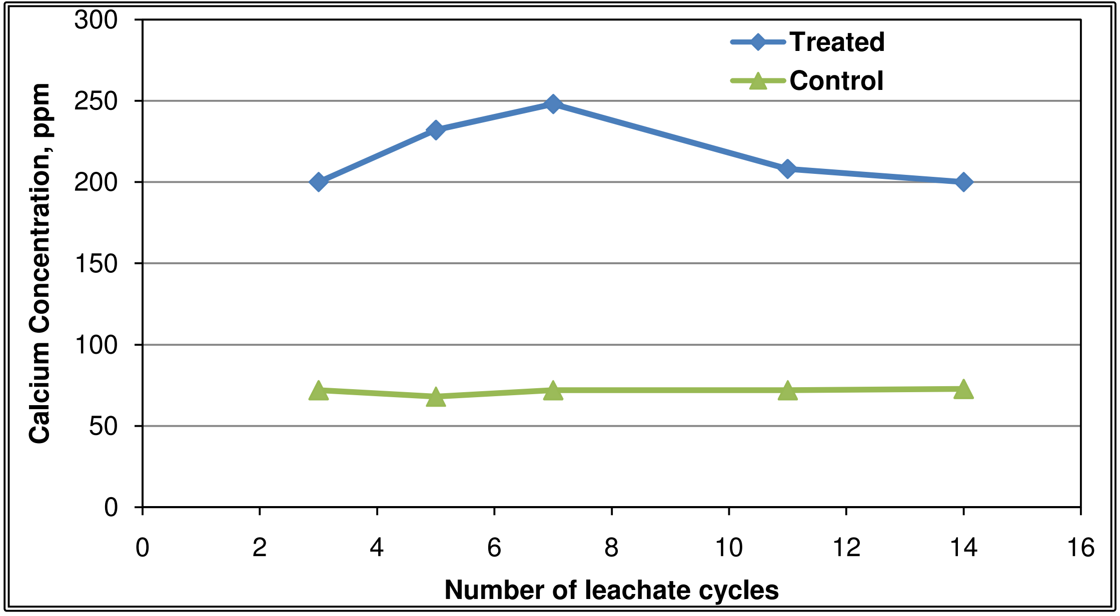


Figure 6.31 Changes in the concentration of calcium ion with leachate cycles for Pharr-B soil specimens

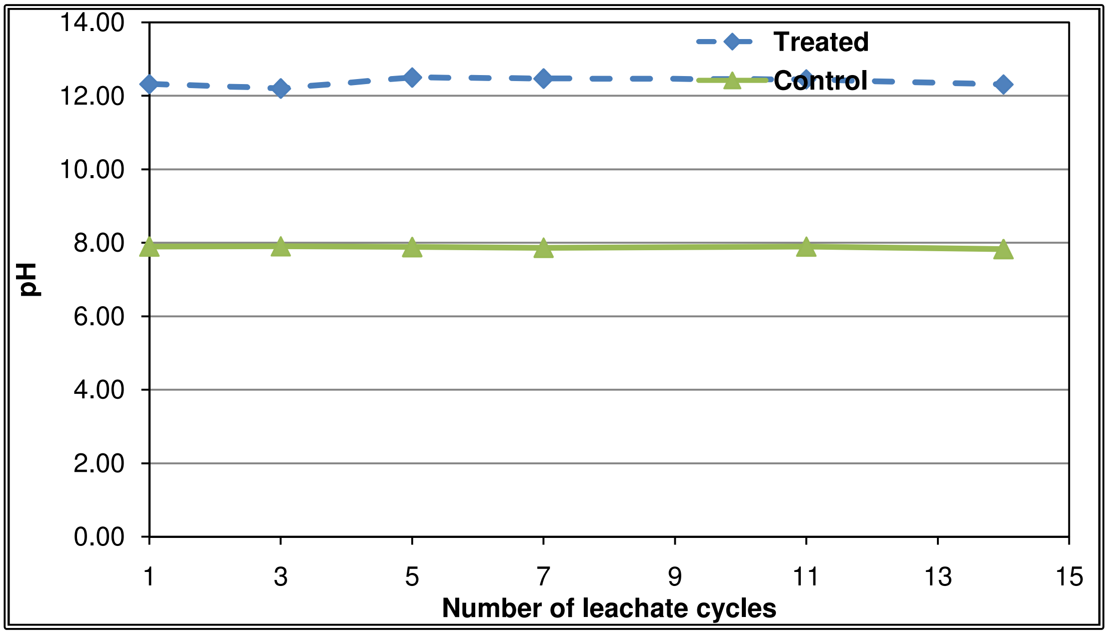


Figure 6.32 Variation in pH with leachate cycles for lime treated Pharr-B soil specimens

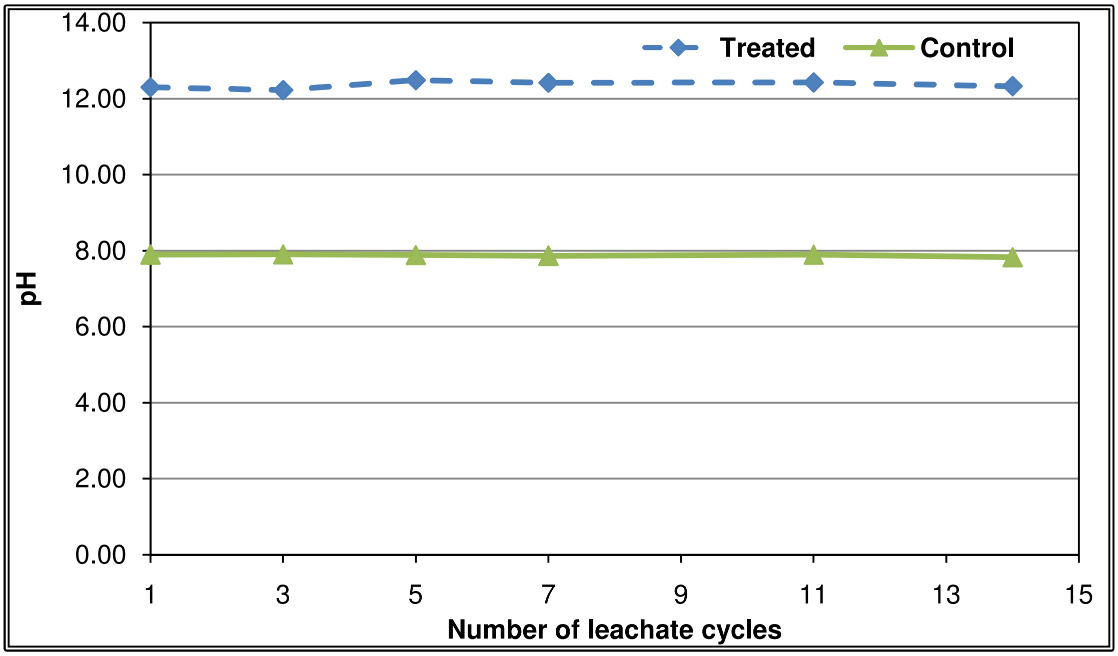


Figure 6.33 Variation in pH with leachate cycles for lime treated Pharr-B soil specimens

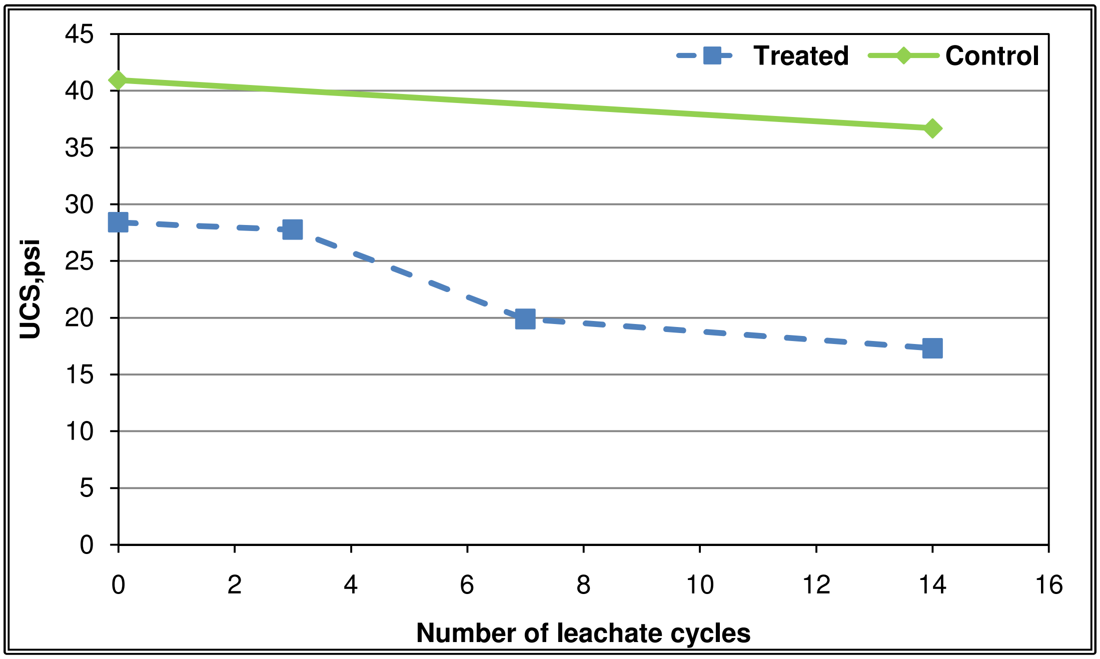


Figure 6.34 Changes in UCS with leachate cycles for lime treated Pharr-B soil specimens

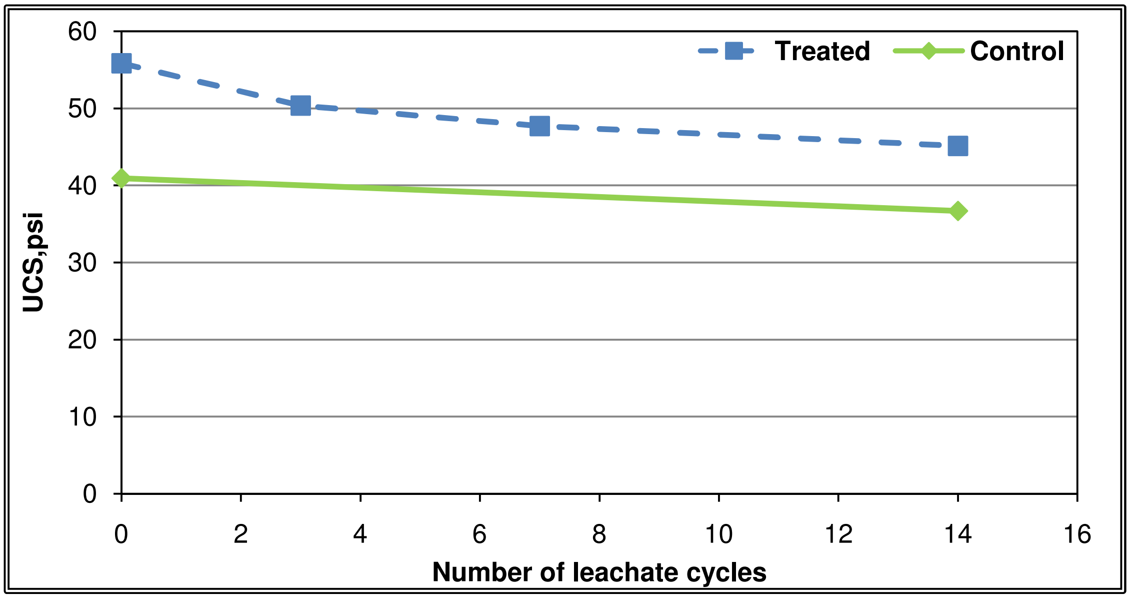


Figure 6.35 Changes in UCS with leachate cycles for cement treated Pharr-B soil specimens

6.2.3.3 Group C results

El Paso soil consisted of 23% of Montmorillonite, 14.2% of Kaolinite and 62.5% of Illite in its clay fraction. This soil had a plasticity index (PI) of 16 and was classified as a low compressible clay (CL). The soil was treated with 8% lime which was selected based on the current PI based design procedure (Tex-121-E).

Figure 6.36 presents the calcium ion concentration changes with the number of leachate cycles of this soil. It can be observed here that the soil samples lasted all the 14 leachate cycles. Also, the concentration of the calcium ion was reducing with the number of leachate cycles in the treated specimens while that of the control soil specimens remained almost constant. The calcium ion concentration was around 70 ppm in case of control soil and this value remained same at higher number of leachate cycles. On the other hand the calcium ion leached out was around 400 at initial leachate cycles and then reduced to 320 ppm at higher number of cycles in case of the treated soil specimens. There was a reduction in the calcium ion concentration in case of treated soil specimens indicating the chances of leaching taking place.

Figure 6.37 presents the pH changes with the number of leachate cycles of the El Paso soil. The pH of untreated soil ranged from 8.1 to 8.1 over the span of 14 cycles and the treated soil ranged from 12.1 to 12.2 over the span of 14 cycles. It can be observed here that there was no much variation in the pH values in both the control and treated soil specimens.

Figure 6.38 presents the variation of the unconfined compressive strength with leachate cycles. It can be noted from the figure that the treated soil exhibited 75 psi of UC strength while the untreated soil displayed only 23 psi of strength initially. Though the conditions at which the soil specimens were tested were different, the treated soil specimens which were tested after moisture conditioning when the specimens were close to saturation exhibited higher strengths than the control soil specimens which were at optimum moisture content at the time of testing showed lower strengths in this case. The control soil retained 92% of the strength while the treated soil specimens retained 98% of the initial strength after 14 cycles of leachate. It can be observed here that there were no considerable strength changes with the leachate cycles.

From the results presented in the sections above it can be noted that the calcium ion concentration in the leachate samples of the treated soil specimens was always higher than that of the control soil specimens for all the eight soils tested. All soil samples survived all the 14 leachate cycles. To better understand the amount of calcium concentration further analysis is carried out to convert the calcium concentration in to the corresponding concentrations of lime and cement. Also it is important to represent the 14 cycles of leaching in terms of the field infiltration amount; this analysis was also carried out and is presented in the following sections.

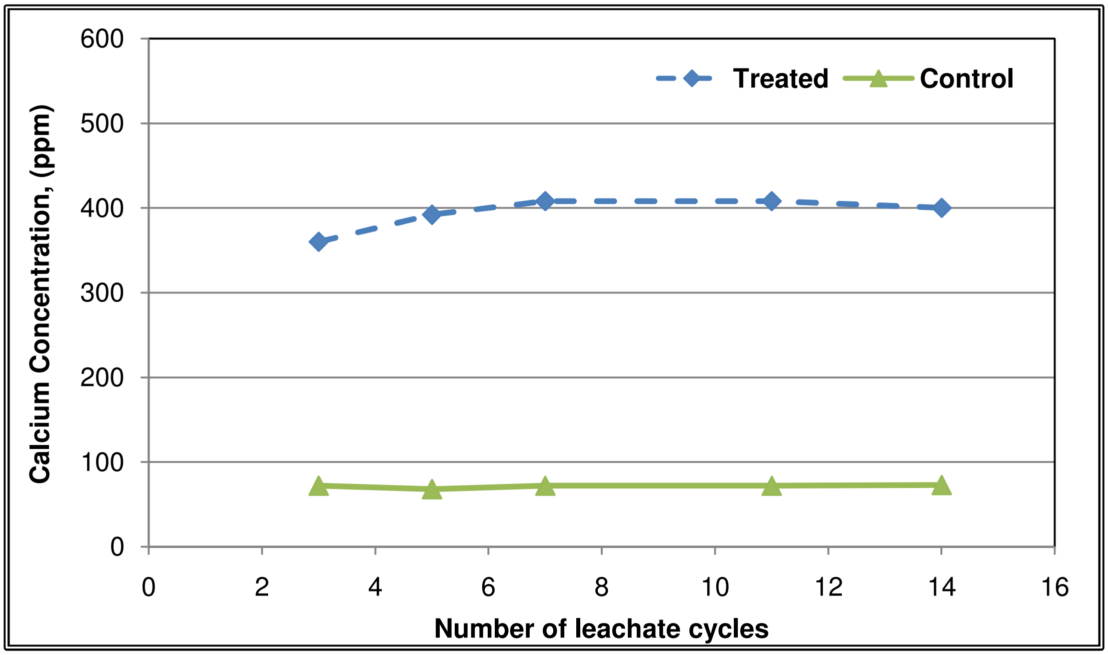


Figure 6.36 Changes in the concentration of calcium ion with leachate cycles for El Paso soil specimens

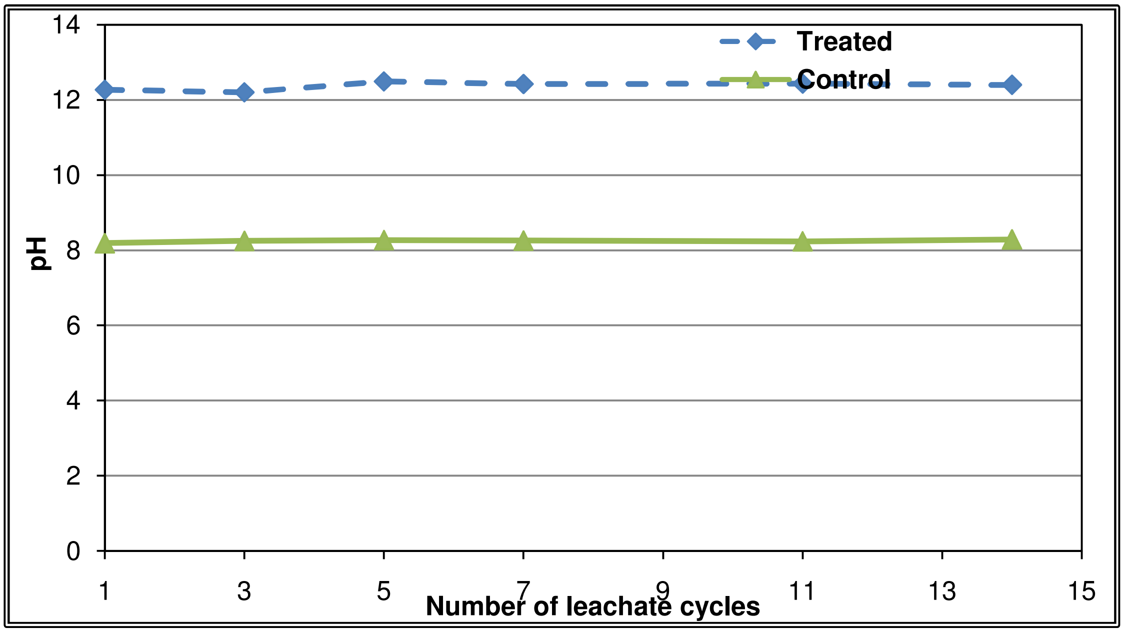


Figure 6.37 Variation in pH with leachate cycles for El Paso soil specimens

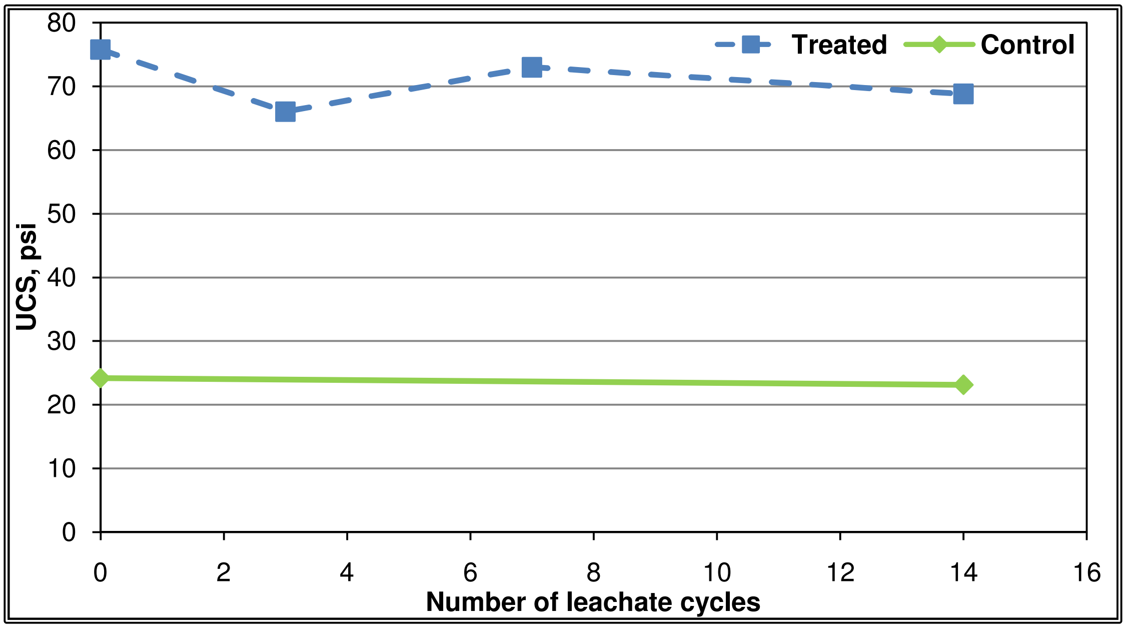


Figure 6.38 Changes in UCS with leachate cycles for El Paso soil specimens

6.2.4 Analysis of Test results

Analysis of the above test results is carried out in this section. The effects of curing time period, clay mineralogy and additive type and dosage on the leachability of stabilized expansive clays are addressed in the following sections:

The leachate samples collected were tested for pH and the presence of calcium ion. The plots between the calcium ion concentration and the number of leaching cycles for all the eight soils tested here indicate that there is a decrease in the calcium ion concentration after 14 cycles of leaching in all the soil samples, but the decline was not considerable. To better understand the effect of the calcium ion concentration on the performance of the treated specimens under leachability this concentration of the calcium ions is converted in to a percentage of lime or cement leached out depending on the type of additive used.

Figure 6.39 summarizes the pH test results collected from lime treated soil specimens of all the eight soils tested in this study. It can be observed from the figure that in all the eight soils response of the pH with leaching cycles was uniform. It can also be identified from the figure that there was not any considerable change in the pH with respect to leaching of the stabilizer. This is expected since the pH of a treated soil can still be same at both low and high dosages. As a result, pH variations with leaching cycles do not yield any valid conclusions.

To interpret the concentration of the calcium ions leached out in terms of percentage lime leached out it is important to obtain the total concentration of the calcium ions that were leached out after 14 cycles of leaching. This is done by averaging the calcium ion concentration obtained from the leachate samples collected after 3, 5, 7,11 and 14 cycles. This averaged number is assumed to represent the average loss of calcium ions after each cycle of leaching. The total concentration of the calcium ions leached out after 14 cycles of leaching is calculated by multiplying this averaged number by fourteen. This number is converted in to a percentage of lime or cement based on the additive used, with the help of calibration curves shown in Figure 6.40and Figure 6.41 respectively. These curves were obtained by mixing known concentration (ppm) of lime or cement in distilled water and then finding out the concentration (ppm) of calcium ions in the solution experimentally.

Table 6.2 presents the percentage of lime approximately leached out after 14 cycles of leaching for all the eight soils tested in this research study. It can be observed from the table that the percentage of the lime leached out of the soil is small (less than 0.6%) and therefore the strength decrease was not considerable. It can be mentioned here that leaching does not have any effect on the strength of the soil specimens as there was only limited amount of lime leached out in the treated soils. However, the duration for which the leachate analyses were conducted was only 14 cycles; this duration must be related to the number of years of moisture infiltration in the field to make any conclusions about the effect of leaching on the performance of the treatment. This analysis is conducted in the following section.

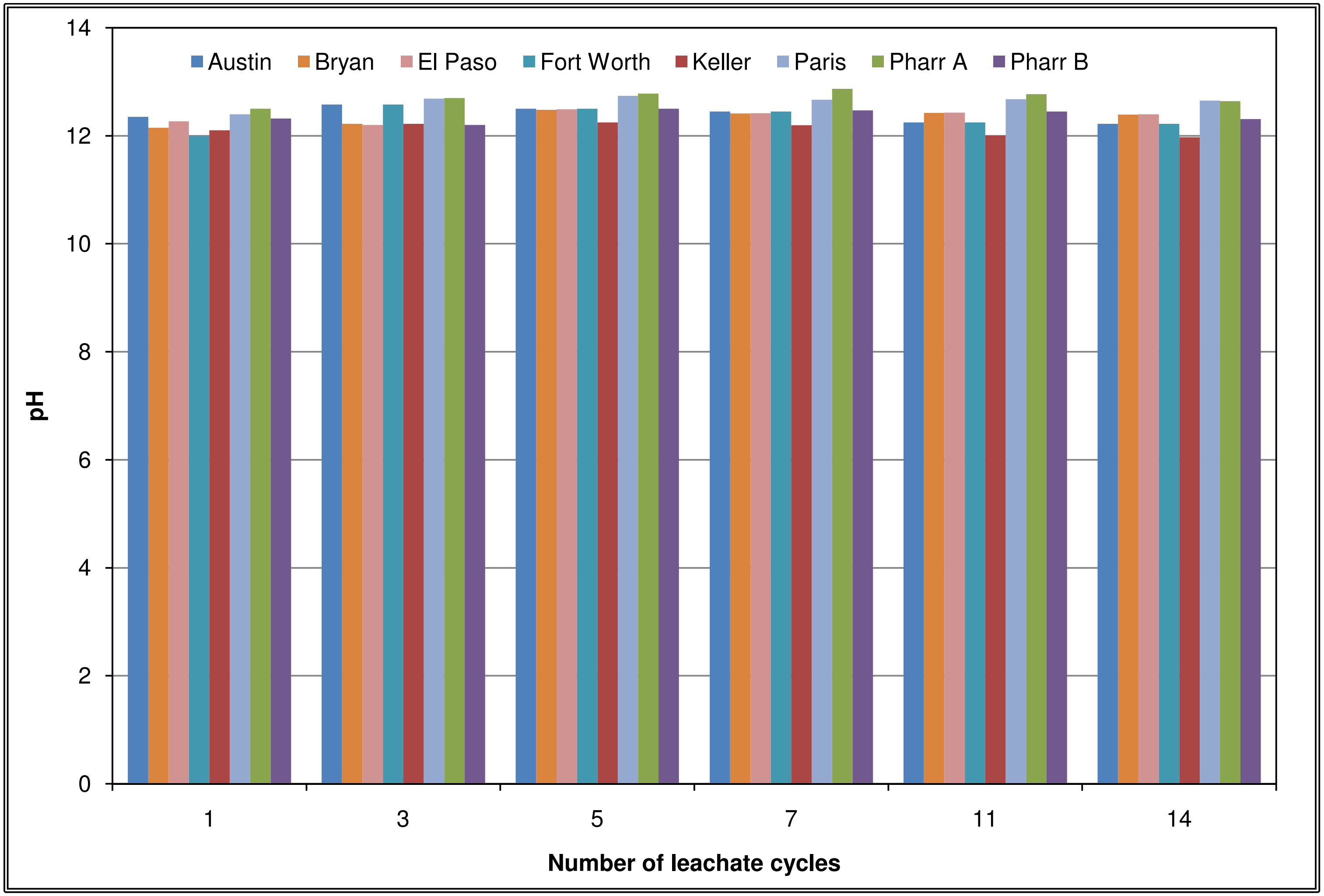


Figure 6.39 Summary of pH results for all the eight lime treated soil specimens

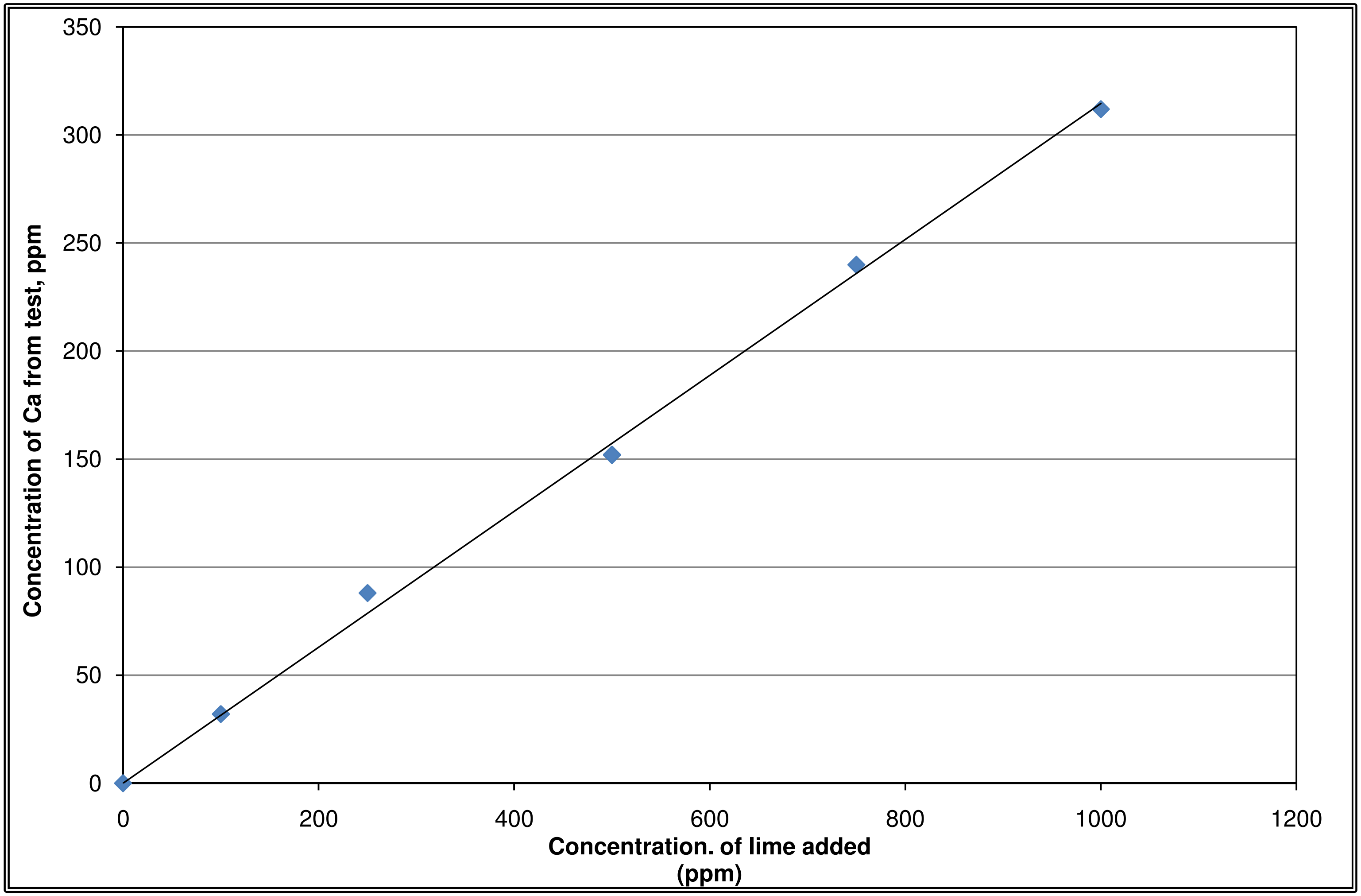


Figure 6.40 Calibration Curve for obtaining the concentration of lime from concentration of Ca

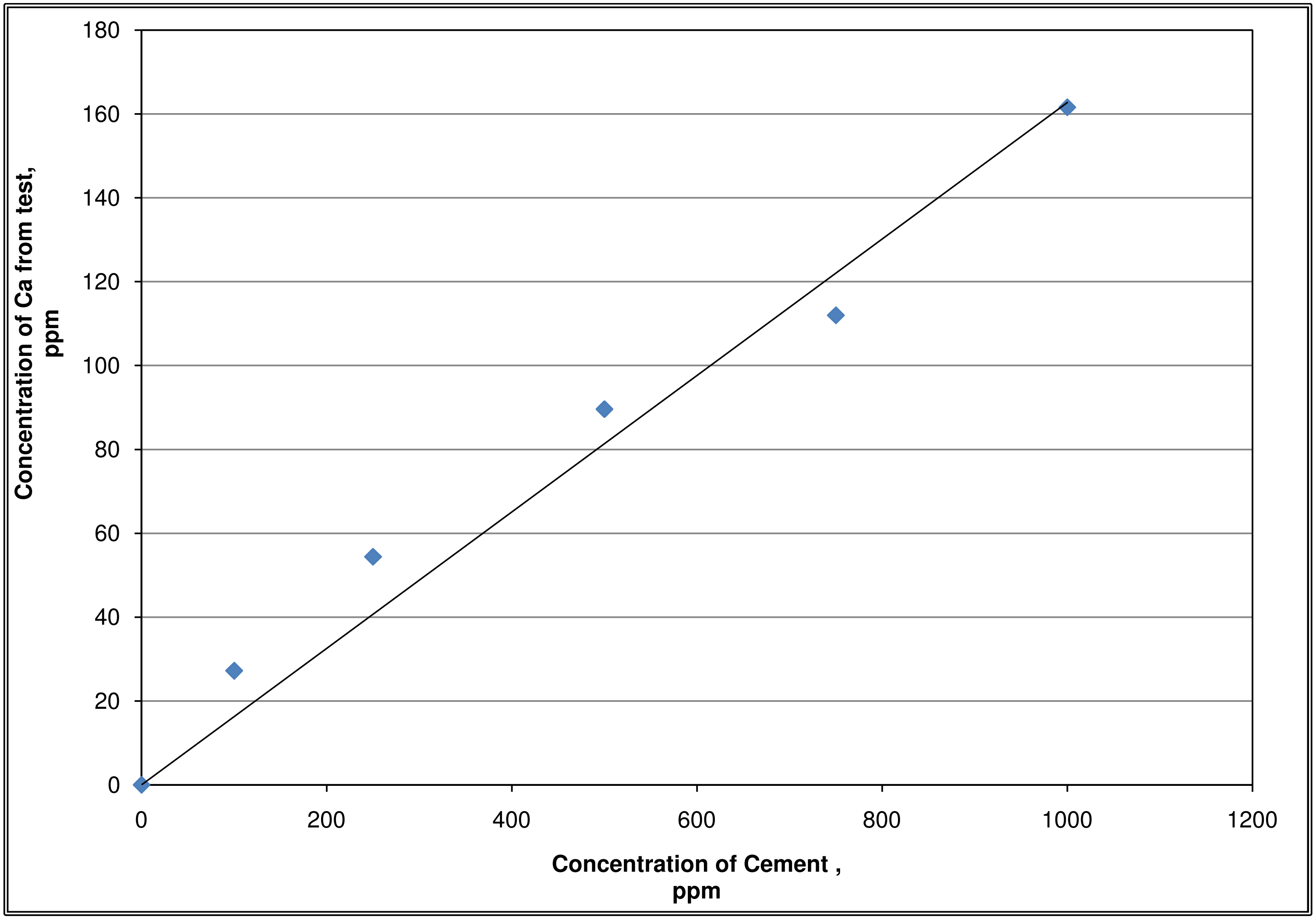


Figure 6.41 Calibration Curve for obtaining the concentration of cement from concentration of Ca

Table 6.2 Concentration of calcium expressed as a percentage of the lime/cement leached along with additive type and amount (Include Dominant Clay mineral column)

Soil Type	Dominating clay mineral	Additive Type	Additive Amount, %	Average Concentration of Calcium ion for each cycle, ppm	% of Lime/Cement leached out after 14 leachate cycles
Austin	Montmorillonite	Lime	6%	435	0.40
Bryan	Kaolinite	Lime	8%	456	0.40
El Paso	Illite	Lime	8%	413	0.34
Fort Worth	Montmorillonite	Lime	6%	430.4	0.51
Keller	Kaolinite	Lime	6%	504	0.39
Paris	Montmorillonite	Lime	8%	481	0.54
Pharr-A	Montmorillonite	Lime	4%	246	0.51
Pharr-B	Kaolinite	Lime	3%	196	0.59
Pharr-A	Montmorillonite	Cement	4%	462	0.42
Pharr-B	Kaolinite	Cement	3%	217.6	0.43

6.2.4.1 Effect of curing methods

The effect of curing on the long-term performance of treated expansive clays has been examined. Section 6.2.2 presents these test results. In this section it has been noted that both long curing and conditioning method (as adapted in Tex-121-E) and a short 3-day curing and conditioning (as developed in this research in AC-1) provided similar responses to soil specimens when subjected to leachate studies.

Figure 6.45 presents the retained strengths after 14 cycles of leaching for all the 4 soil specimens cured by both methods. In this figure it can be observed that all the soil samples survived all 14 leachate cycles. The difference in the unconfined compressive strengths was attributed to the difference in the degrees of saturation among the soil specimens tested for strengths cured by the two methods under review. The soil specimens in the case of the AC-1 curing method were close to saturation due to proper moisture conditioning obtained by back pressure saturation while the soil specimens cured by Tex-121-E method were not at saturation due to improper moisture conditioning obtained by capillary suction. Though the strengths were different the retained strengths measured at the end of all the 14 cycles of leaching were also similar in both the curing conditions. Hence, it can be mentioned that the length of curing time period did not show any considerable effect on the long-term performance of treated soils.

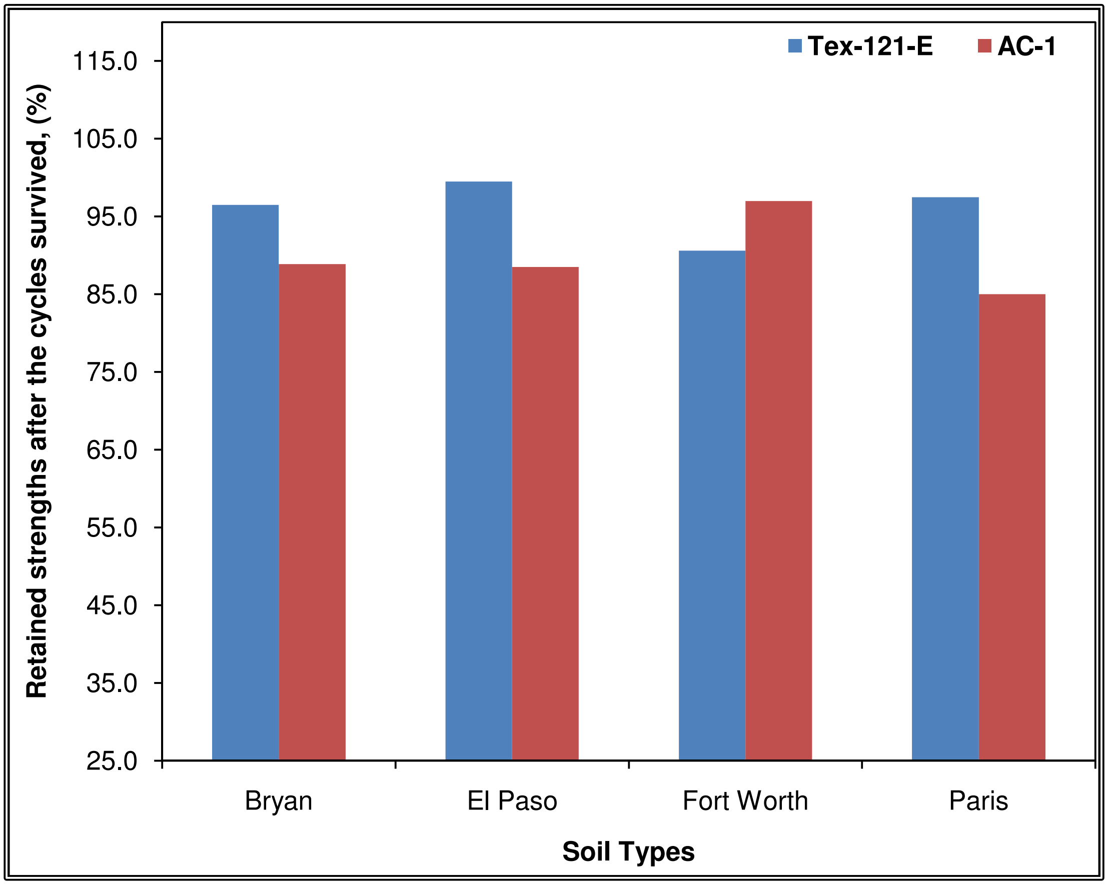


Figure 6.42 Retained strengths after 14 cycles of leaching of soil specimens cured by both methods all the for four clays

The calcium ion concentrations obtained after 14 cycles of leaching for all four clays cured by both the curing methods are summarized in Figure 6.43. From this figure it can be observed that there was not much change among the curing methods for all the four clays. Hence it could be concluded that the duration of curing has no effect on the leachability of treated expansive clays.

6.2.4.2 Effect of clay mineralogy

Table 6.3 summarizes the percent lime leached out after the 14 leaching cycles in all the eight soils studied in this research. This table also gives the dominant mineral along with the type and amount of the chemical additive. A graph is plotted between the percentage lime leached out after 14 cycles of leaching and % Montmorillonite to address if the leaching is dependent on the percentage of mineral Montmorillonite in the clay fraction of the soil. This plot is presented in Figure 6.44.

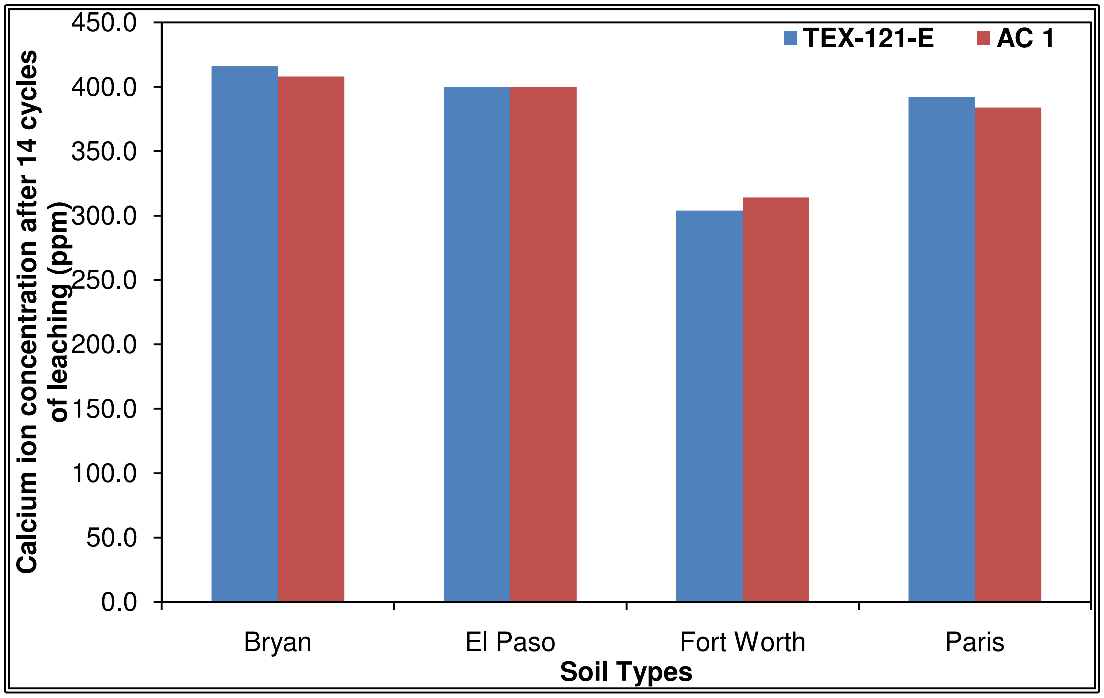


Figure 6.43 Calcium ion concentration after 14 cycles of leaching of soil specimens cured by both methods for all the four clays

It can be observed in the figure that there is no variation of the percentage of lime leached out from the treated soil specimens after 14 cycles of leaching. Hence it could be understood that the percentage lime that leaches out of the soil specimens is independent of the mineral Montmorillonite. On the other hand, the percent leaching of lime were slightly lower for clays with Kaolinite and Illite as dominant minerals. This can be noted from the results shown in Table 1.2.

Table 6.4 summarizes the retained strength after the 14 leaching cycles for all the eight soils studied in this research. This table also gives the mineral dominance along with the type and amount of the additive. The graph plotted between the percent Montmorillonite and the percent retained strength after 3, 7 and 14 cycles of leaching is presented in Figure 6.45. From the figure it can be understood that there is no relationship between the percent retained strength after 14 cycles of leaching and the percent Montmorillonite in the clay fraction of the soil.

Table 6.3 Average concentration of calcium and the number of cycles survived along with the dominant clay mineral and the additive type and amount

S No.	Soil Name	Dominating clay mineral	Type of additive	Amount of additive, (% by weight)	# of cycles sample survived	% of lime leached out
1	Austin	Montmorillonite	Lime	6%	14	0.40
2	Bryan	Kaolinite	Lime	8%	14	0.40
3	El Paso	Illite	Lime	8%	14	0.34
4	Fort Worth	Montmorillonite	Lime	6%	14	0.31
5	Keller	Kaolinite	Lime	6%	14	0.39
6	Paris	Montmorillonite	Lime	8%	14	0.54
7	Pharr-A	Montmorillonite	Lime	4%	14	0.31
8	Pharr-B	Kaolinite	Lime	3%	14	0.59

Table 6.4 Percentage retained strength of different soil samples and the cycles survived along with the dominant clay mineral

S No.	Soil Name	Dominating clay mineral	Type of additive	Amount of additive, (% by weight)	# of cycles sample survived	% Retained strength
1	Austin	Montmorillonite	Lime	6%	14	98.8
2	Bryan	Kaolinite	Lime	8%	14	88.9
3	El Paso	Illite	Lime	8%	14	88.5
4	Fort Worth	Montmorillonite	Lime	6%	14	97
5	Keller	Kaolinite	Lime	6%	14	98.8
6	Paris	Montmorillonite	Lime	8%	14	85
7	Pharr-A	Montmorillonite	Lime	4%	14	95
8	Pharr-B	Kaolinite	Lime	3%	14	55

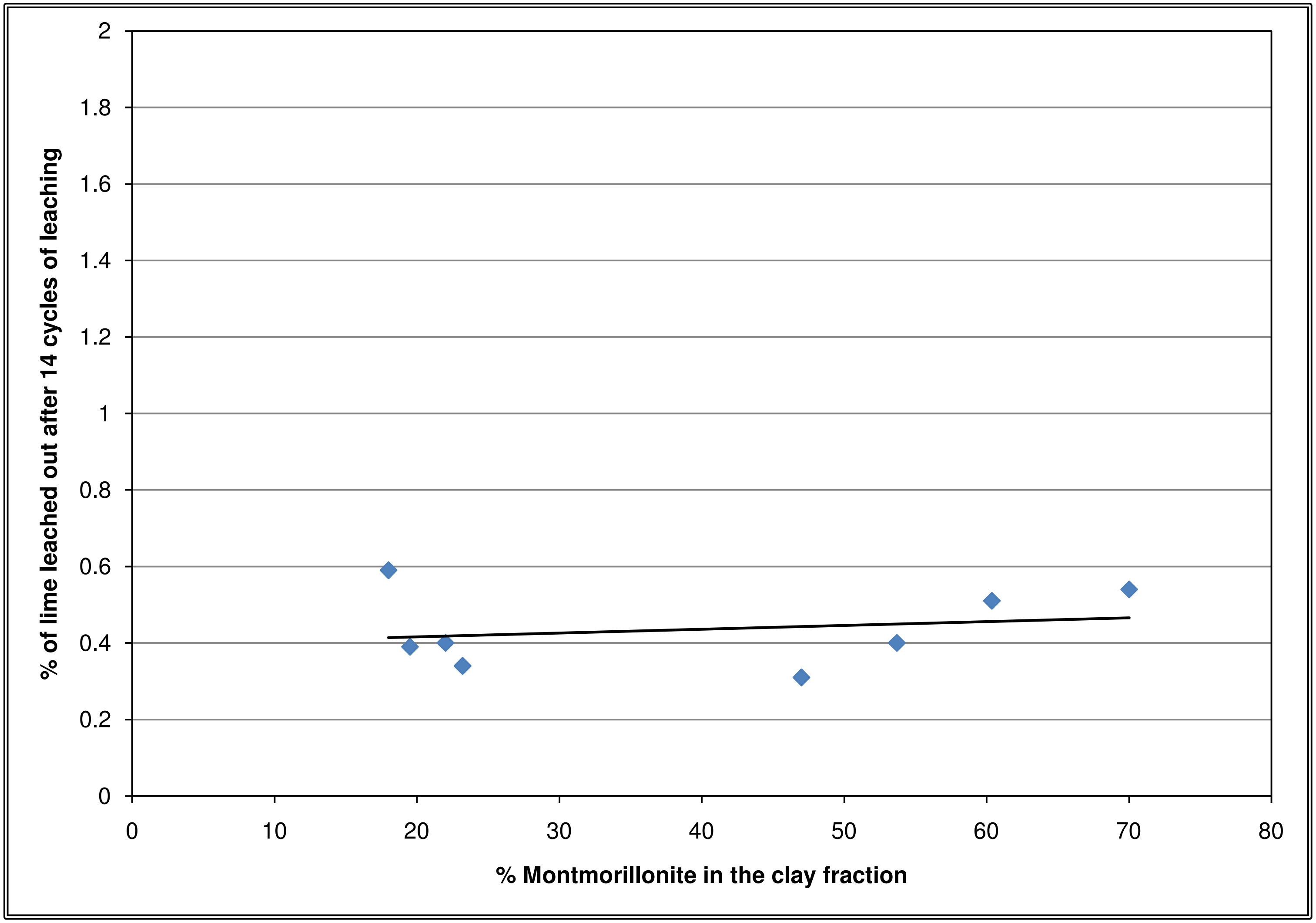


Figure 6.44 Effect of mineralogy on the percentage lime leached out

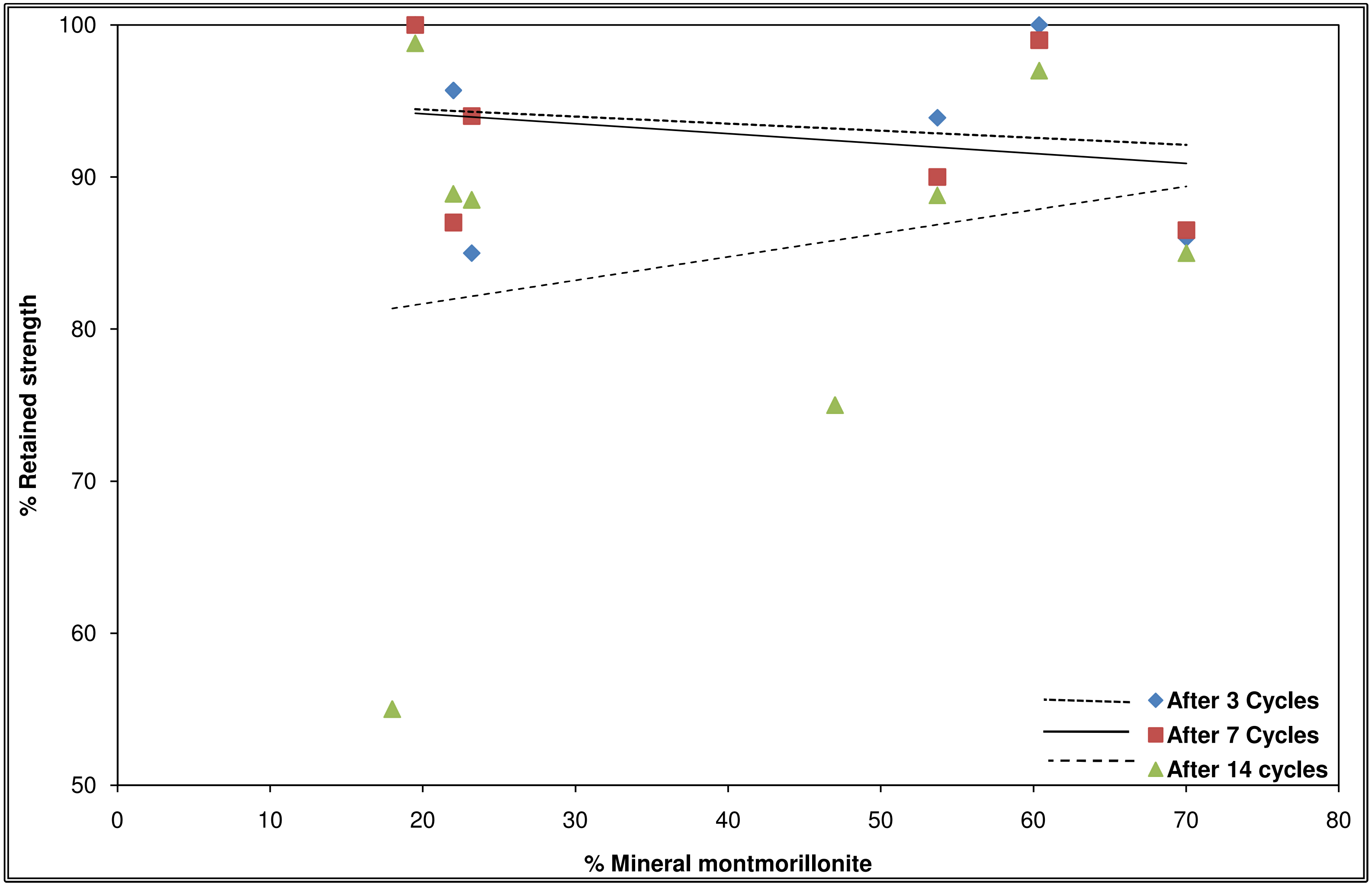


Figure 6.45 Effect of mineralogy on the percentage of retained strength after different number of cycles

6.2.4.3 Effect of chemical additive type and amount

To evaluate the effect of type of chemical additive (cement and lime treatment) on the leachability of stabilized expansive clays, two soils were tested with the same amounts of lime and cement stabilizers. Pharr-A soil which had a high plasticity index and Pharr-B soil which had a low PI were selected as candidate soils for addressing this variable.

Table 6.5 summarizes the effects of type of additives used on the durability of the stabilization. It can be observed from the table that cement treatment retained more strength than the lime treatment in both PI type soils. In the case of medium PI soil (Pharr-B), lime treatment could retain only 55% of the original strength after 14 cycles of leaching. It can also be observed from the table that the percentage of lime additive that has been leached out of the soil sample after 14 cycles of leaching is 0.59%. This percent loss of lime is considerable when compared to the initial amount of lime added which was only 3%. Hence the loss of strength of the lime treated Pharr-B or low PI soil is attributed to the low dosage amount.

. In the case of the high PI clay (Pharr-A soil), lime treatment had resulted in the better retention of the initial strength than cement treatment. Similar results were obtained in the study conducted by Kennedy et al. (1987). They noted that the lime treatment is more effective than cement treatment for high plasticity clays. The percentage of additive leached out after 14 cycles of leaching was also high in case of cement treatment than in lime treatment.

Table 6.5 Comparison of results between the additive types

Soil type	Additive type	Amount of additive, %	% Retained strength after 14 cycles of leaching	% Lime/Cement leached out after 14 cycles of leaching
Pharr-A (High PI)	Lime	4	95	0.31
Pharr-B (Med PI)	Lime	3	55	0.59
Pharr-A (High PI)	Cement	4	85	0.42
Pharr-B (Med PI)	Cement	3	81	0.43

Table 6.6 summarizes the effect of the amount of dosages on the long-term performance of a stabilized subgrade. Three soils from Bryan, Keller and Pharr-B which are dominant with Kaolinite were selected and used. These soils were treated with 8%, 6% and 3% lime, respectively.

It can be observed from the table that both Bryan and Keller soils have still retained more than 85% of the initial strength even after 14 cycles of leaching while Pharr-B soil retained only 55% of the initial strength after 14 cycles of leaching. This indicates that higher dosages of lime resulted in the better stabilization of soils that performed well under leaching studies. These results are in agreement with the conclusions made by McCalister (1990). In their study they found that soils that are treated with lower percentages of lime (lower than the optimum lime required for stabilization from pH test) the soil specimens exhibited loss of strength due to leaching.

Table 6.6 Comparison of results between the additive dosages

Soil type	Mineral Dominance	Additive type	Additive amount, %	% of Lime leached out after 14 leaching cycles	% Retained strength after 14 leaching cycles
Bryan	Kaolinite	Lime	8	0.40	88.9
Keller	Kaolinite	Lime	6	0.39	98.8
Pharr-B	Kaolinite	Lime	3	0.59	55

6.2.4.4 Field infiltration time

In all the above results it is noted that there was high retained strength (>80%) after 14 cycles of leaching except for the Pharr-B clay specimens which had 55% retained strength after 14 cycles. Also the percentage of lime leached was also low (<0.5%) after 14 cycles of leaching in all soil specimens. This small strength decrease or high retained strength and low percentage lime/cement leaching out after 14 cycles of the laboratory tests can be better understood if the laboratory leachate cycles could be correlated to field moisture infiltration amounts. Hence, it is necessary to develop a correlation between the accelerated laboratory leaching data and field precipitation and moisture infiltration information.

The infiltration capacity of the soil depends on its permeability and soil moisture content at the time of infiltration. The initial capacity of a dry soil is high but, as the storm continues, the infiltration decreases until it reaches a steady state value termed as final infiltration rate. In the current analysis average infiltration rates were assumed as a detailed analysis is not warranted due to varying field conditions (1). The field infiltration is always a small and integral part of the total rainfall. The main sources of infiltration are *surface infiltration*, *groundwater seepage* and *ice lenses melt water* (Huang, 2004).

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. per 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 the Figure 6.46. Following the method shown in Figure 6.46 the number of years of filed infiltration replicated in the laboratory are presented in Table 6.7.

It can be observed from the table that the number of field years replicated in all eight soils is less than 5 years which is a short duration when compared to the life of the pavement. Considering that there is 0.5 to 0.6% leaching of stabilizer in the first five years, it is likely that with more number of leachate cycles, the percent leachate of stabilizer could be higher and exceed 2% in ten to fifteen years. Hence, it can be mentioned here that the percent leaching of lime in short term leachate studies is small, but can be considerable if the original stabilizer dosage used is less than 4. Overall, this study clearly showed that leaching may not be highly problematic in the initial years if the treatment dosages are high (6% or high).

Field Infiltration

Average AnualRainfall 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 6.46 Method to correlate laboratory leachate studies to field infiltration years

Table 6.7 Number of years in the field replicated in the laboratory along with percent retained strength and % lime leached

S No.	Soil Type	% Lime Leached	% Strength Retained	Number of years in the field replicated
1	Austin	0.40	98.8	3.2
2	Bryan	0.40	88.9	3.6
3	El Paso	0.34	88.5	2.9
4	Fort Worth	0.31	97.0	4.0
5	Keller	0.39	98.8	2.6
6	Paris	0.54	85.0	3.9
7	Pharr-A	0.31	95.0	4.4
8	Pharr-B	0.59	55.0	3.8

6.3 Summary and findings

A total of eight soils were selected for studying the long-term performance of stabilized expansive soils by conducting leachability studies. Stabilizer design was carried out according to the TxDOT methods, Tex 120-E (Lime as additive) and Tex 121-E (Cement as additive) and these results are presented here. Leachate studies were conducted on several control and treated soil specimens as per the specimen and curing procedures explained in Chapter 3. Leachate samples were collected after 3, 5, 7, and 14 cycles of leaching to address the chemical changes occurring due to leaching of the additive from the soil specimen. Also, unconfined compressive strength tests were conducted after 3, 7 and 14 cycles of leaching to address the strength changes due to leaching in the soil specimens.

An accelerated curing method was developed and followed in this research study along with a long-curing test procedure for performing curing and moisture conditioning the treated specimens. The effects of both curing methods on the leachability of treated expansive clays were studied on four soils and it was observed that both the curing methods yielded similar results. It has been interpreted from the results that there was no considerable influence of the curing period on the long-term performance of these treated expansive clays except for an initial strength drop which was attributed to saturation of the specimens at which the specimens were tested.

The effects of mineralogy on the long-term performance of stabilized expansive soils were observed in this study. The concentration of the calcium ion and the pH of the leachate and unconfined compressive strength were measured with the number of wetting/drying cycles. The results of all the eight soils that were considered in this study are presented in detail. No considerable changes were observed in calcium ion concentration, pH of the leachate among different test soils. To better understand the calcium ion concentration loss with the leachate cycles, the leachate concentration was converted into a percentage of lime/cement additive leached out depending on the additive type used. The percentage lime and cement leached out of the soil specimens after 14 cycles of leaching were low (<0.6% for lime and <0.4 for cement) for all the soils tested.

Also, the percentage retained strength was also similar for all the soils. Hence, it has been concluded from this study that the percentage of Montmorillonite mineral does not have any bearing on the leachability of the treated expansive clays. However, Pharr-B clay did show some strength drop after 14 cycles of leaching which was attributed to the low additive dosage. Overall, the amount of stabilizer was slightly small in soils which contain high amounts of Kaolinite and Illite in clay fraction.

The effect of the additive type on the long-term performance of stabilized expansive soils was also studied in this research. Two soils were selected for this study one with a high PI (Pharr-A clay) and one with medium PI (Pharr-B clay). It has been observed here that both the treatments lasted all the 14 cycles of leaching on both soils. However, in the case of Pharr-B clay the retained strengths after 14 cycles of leaching were lower when treated with lime than with cement. This proves the pre-established fact that cement treatment gives better improvement of the subgrade in case of low PI clays than high PI clays.

Also, the effect of the amount of dosage was also studied in this research. The results indicated that higher dosages of lime resulted in the better stabilization of soils that performed well under leaching studies. These results are in agreement with the conclusions made by McCalister (1990). In their study they found that soil specimens that were treated with lower percentages of lime (lower than the optimum lime required for stabilization from pH test) exhibited loss of strength due to leaching.

Finally, an attempt is made to correlate the laboratory leachate cycles to the field precipitation and moisture infiltration information to understand the high retained strength and low percentage lime leached out after 14 cycles of leaching. It was observed that the number of field years replicated in all the eight soils is less than 5 years which is short duration compared to the life of the pavement. Hence, it was mentioned that the percent leaching of lime in short term leachate studies is small, but can be considerable if the original stabilizer dosage used is less than 4. Overall, this study clearly showed that leaching may not be highly problematic in the initial years if the treatment dosages are high (6% or high).

The next chapter presents results of a numerical study in which the long-term performance of a pavement overlying on a treated base was addressed.

CHAPTER 7

FINITE ELEMENT MODELING OF DETERIORATION IN TREATED BASE

7.1 Introduction

Numerical modeling of complex geotechnical problems provides better responses of structures under different types of loads. Among the numerical methods, finite element method (FEM) is proven to be a powerful technique for understanding and solving many engineering problems dealing with mechanical behavior of solids. With recent advancements in computer technology, the finite element method has become a popular alternative to traditional analysis methods in geotechnical engineering (Wanstreet, 2007). With the introduction of three-dimensional (3D) FEM models, and results reported in the literature, several applications including in the areas of pavement engineering have been successfully modeled by studying various layers and their responses to loading (Hammons 1998; Kim and Hjelmstad 2000). Due to the required running-time and complexity of the FEM, three dimensional (3D) finite element analysis is not always easily implementable.

One of the objectives of this research was to understand the impacts of stabilized base layers and how it impacts the loading behavior of pavement systems. Since the main focus of this research was to understand the weakening of the stabilized expansive soils due to factors such as moisture fluctuations from winter and summer seasons or due to leaching of stabilizer from moisture ingress and digress during heavy rainfall seasons. In previous chapters, it had been shown experimentally that there are problems in stabilized subsoils from these moisture changes or fluctuations. The same is being studied in this chapter by numerically simulating with finite element analysis of a pavement section with hypothesized stabilized subsoil properties at different seasonal conditions as established from the present experimental program.

As there are no models developed so far to replicate the wetting and drying processes or the leaching of stabilized subsoils supporting pavement infrastructure, an indirect method is followed here to investigate the effects of treated subgrade deterioration on the performance of the pavement system.

Typically, flexible pavements are designed by addressing the distress coming from fatigue cracking, and rutting or excessive permanent deformation. Treated subsoils provide a sound foundation system and they also transmit the traffic wheel loads to underlying soils. These loads can induce permanent deformation in each layer, which in turn will contribute to rutting on the pavement surface.

Hence, rutting or permanent deformation or plastic deformation is defined as a permanent deformation of a pavement surface under the wheel path due to repeated loading of traffic (Figure 7.1). High rutting could occur due to several factors such as low asphalt strength either due to insufficient compaction or improper mix design, loss of strength in the base layer due to seasonal moisture changes and weak subgrade or deterioration of subgrade due to continuous loading. The vehicular load applied on to the surface of the pavement causes lateral movements in the base layer as the soil particles try to move down and away from the load. These lateral movements generate vertical strains which lead to a permanent or rut formation in the pavement layer (Hadi and Bodhinayike, 2003).

A commercially available finite element program ABAQUS® 6.8.1 was used to numerically determine the permanent deformation. This deformation can be obtained either by performing dynamic loading analysis, which could take a long period to complete. This was shown by Nazzal et al. (2006) who noted that it will take 2400 hours to complete 1500 cycles of dynamic loading on a pavement system. Alternatively, Asphalt Concrete Institute (ACI) recommends obtaining of a static and maximum compressive strain in a subgrade under an equivalent single axle wheel load applied at the pavement surface. This strain will be indirectly used to assess the number of equivalent single axle wheel loads (ESALs) that can be applied on a pavement before rutting failure occurs. If the interpreted loading cycles are well beyond the design ESALs, then rutting will be considered acceptable and is not a problem in the pavement design. This approach has been primarily used by the previous studies (Nazzal et al., 2006).

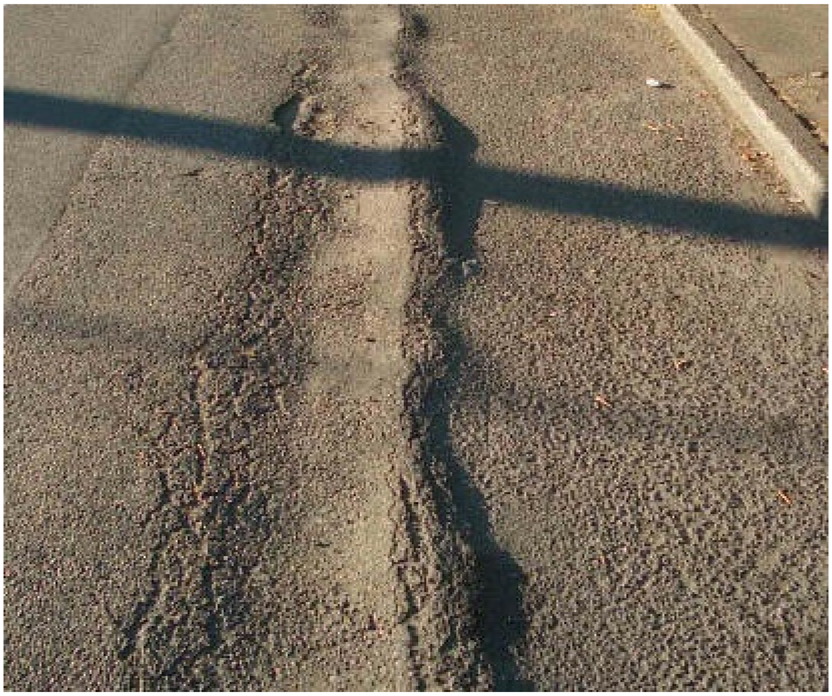


Figure 7.1 Picture showing rutting in a flexible pavement
(Source: http://www.asphaltwa.com/wapa_web/modules/11_guidance/images/thumbnails/rutting2.JPG)

7.2 Analysis Procedural Steps

One of the dissertation objectives of this research study was to assess the effects of deterioration of treated subgrade as a base course under a flexible pavement structure. As a part of this evaluation, the influence of different pavement parameters or variables on the performance of the pavement structure is also studied. The pavement parameters that were studied in this research are thickness of the asphalt concrete layer, thickness of the base course layer (treated subgrade), strength and resilient moduli of the subgrade soil and strength or moduli of the base course layer (chemically treated layer).

To accomplish this analysis, a two-dimensional axi-symmetric finite element model was used to analyze different pavement test sections consisting of an asphalt concrete layer, a base layer and a subgrade layer. This section is typical for low volume roads which experience less than 2000 vehicles per day (AASHTO, 2006). Two different asphalt concrete layer sizes (2 in. and 4 in.), two different base course layer sizes (12 in. and 18 in.), three different subgrades (weak, moderate and stiff) and three different base layer moduli related to initial, midway, and final deterioration state from the durability studies were considered and analyzed. Table 7.1 summarizes the variables that were studied in this research study. Table 7.2 presents the notations used for the four pavement sections that were analyzed in this study.

The finite element model was used to analyze the pavement system and then determine compressive strain on the subgrade under a standard axle 18 kip load. This compressive strain was used to predict the number of equivalent single axle loads (ESALs) that can be applied to the pavement section before rutting failure occurs. The formula given by asphalt concrete institute was used in this analysis. Comparisons were made between different pavement sections and subgrade strengths as well as base layer properties to evaluate the effects of base course degradation and other pavement characteristics on the long-term performance of the pavement structure.

Table 7.1 Summary of the variables studied

Variable	Values used			
Thickness of AC layer	2’’ in.	4’’ in.	-	-
Thickness of base layer	12’’ in.	18’’ in.	-	-
Subgrade strengths	Weak	Medium	Stiff	
Base Modulus	Initial	Intermediate 1	Intermediate 2	Final

7.3 Model details

To understand the effect of base course deterioration due to seasonal moisture changes on different pavement sections, four different flexible pavement test sections were considered and analyzed numerically. Two of the four models had 2 in. thick asphalt concrete layer and the remaining two models had 4 in. thick layer of asphalt concrete. Two different base layer thicknesses (12 in. and 18 in.) were also considered. The parameters that were varied are the subgrade modulus and the deteriorating base modulus at different seasons in service.

Though most geotechnical problems that involve pavement structures model them as three dimensional (3-D) structures, such analyses consume considerable amounts of computer resources and are also time consuming. Hence, their undertaking is usually non-practical, except for very limited and simple cases (Potts and Zdravkovic 1999). Two dimensional idealization of a 3-D problem is considered reasonable as this significantly reduces the amount of computer resources, time taken to run the analyses and the number of finite elements that are necessary for appropriate modeling (Zevgolis, 2007). Also, Hua (2000) showed that rutting in a flexible pavement can be modeled using two-dimensional finite element models rather than three-dimensional models without losing significant accuracy.

In the current study, a two-dimensional axi-symmetric finite element model available from the finite element program ABAQUS 6.8.1 was used to analyze the present hypothetical flexible pavement section. The model consisted of a mesh section that is 11 ft in radius and 10 ft in depth. The depth of the test section was selected such that the maximum induced vertical stress in the subgrade has become insignificantly small (< 1% of the applied tire pressure). The radius was also selected based on a similar criterion in which the vertical and horizontal strains induced due to loading became insignificantly small in all the layers (Nazzal et al., 2006).

Table 7.2 Notations of the pavement sections modeled

S No.	Asphalt layer thickness, in	Base layer thickness, in	Notation
1	2	12	2AC_12B
2	2	18	2AC_18B
3	4	12	4AC_12B
4	4	18	4AC_18B

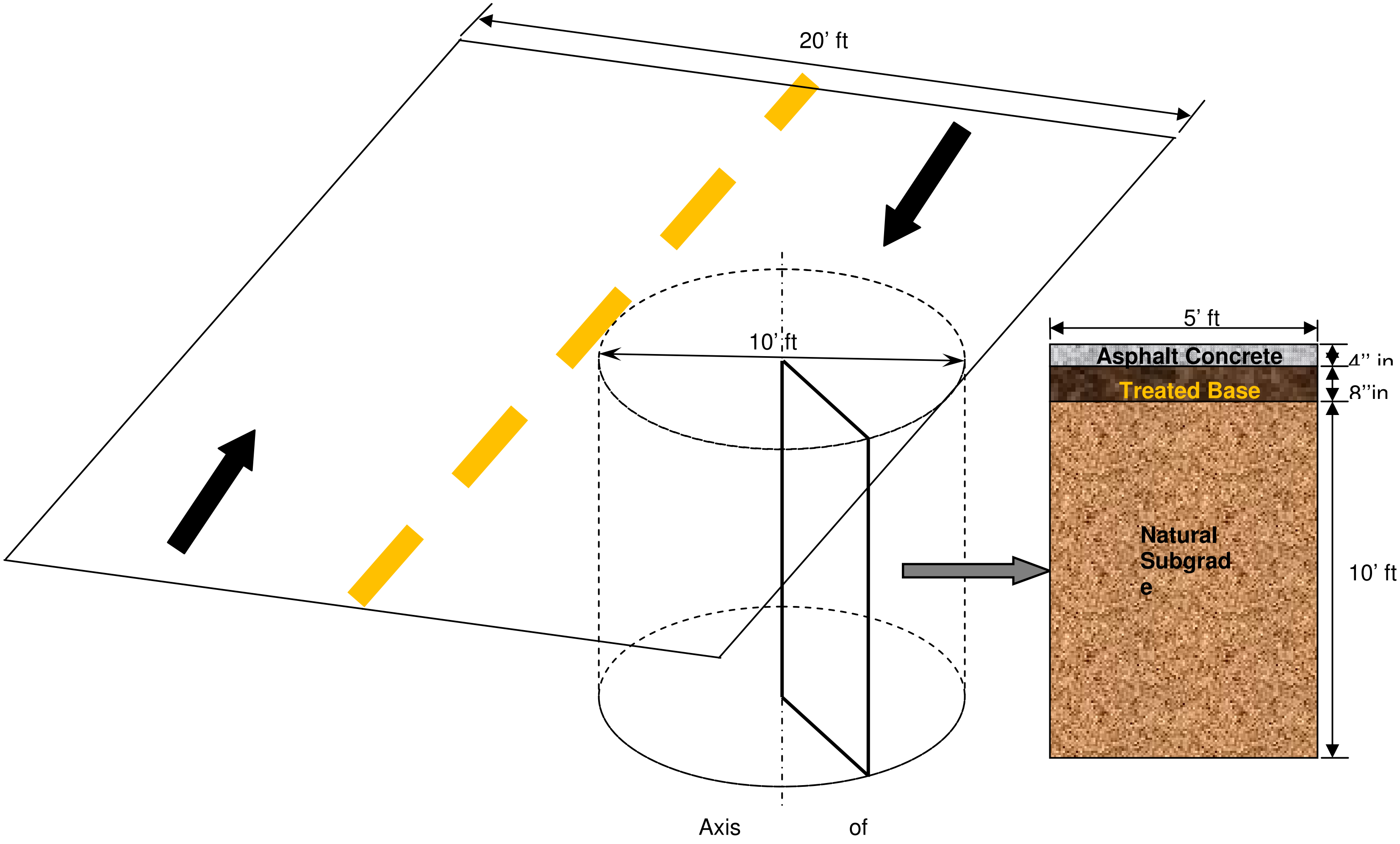


Figure 7.2 Figure showing a typical section of the pavement that is being modeled

7.3.1 Element type and Mesh size

To determine the suitable mesh size for the 2D axi-symmetric model of this study, a series of finite element analyses were performed with decreasing element sizes i.e. increasing the number of elements. Five different element sizes varying from 6 in. to 0.5 in. were considered and analyzed for each of the four models to arrive at the optimum element size. Here eight noded biquadratic quadrilateral axisymmetric elements (CAX8) were chosen, as quadratic elements yield better solutions than linear interpolation elements (Kuo and Huang, 2006). Compressive strains were obtained from finite element analysis of these different element types and were converted into ESALs using the statistical equation given by ACI (as per the following Equation 7.1).

$$N_r = 1.365 \times 10^{-9} \times \varepsilon_v^{-4.477} \quad \text{...(7.1)}$$

Where,

N_r = Number of ESALs

ε_v = Compressive strain on the subgrade top

Figure 7.3 illustrates the plot between the element size and the ESAL's obtained. It can be observed from the figure that as the element size decreases, the ESAL number is increased showing a improvement in the results with the increasing element size. However, among the element sizes of 1 in. and 0.5 in. the improvement was not significant though the computational time required in the case of 0.5 in. element size (2900 sec) was higher than that of 1 in. element size (165 sec). Hence, the element size of 1 in. was decided to use as the optimum size in the subsequent analyses. Figure 7.4 presents the mesh that has been selected and used based on the above analysis.

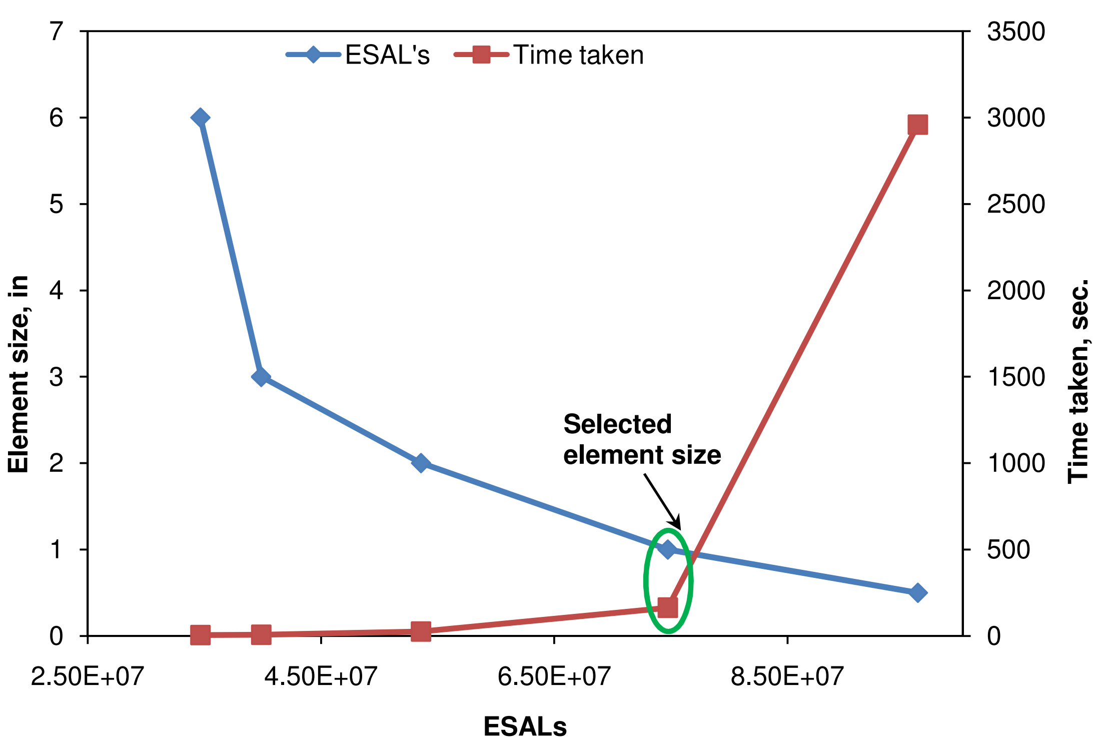


Figure 7.3 Plot showing h-convergence results

However, the mesh needs to be finer only in the loading zone where most of the stresses are generated. Hence, nodes arranged in such a way so that the mesh is finer near the loading zone and gets coarser as it goes away from the loading zone. Based on this consideration Table 7.3 presents a summary of the element types and the number of elements used for each of the materials used in this study along with the thicknesses varied in each case.

Table 7.3 Element details of different materials

Element Layer	Material Type	Element Type	Number of elements	Thickness (in.)
Top Layer	Asphalt	CAX8	360, 420	2.0, 4.0
Middle Layer	Base Course	CAX8	1200, 1890	12.0, 18.0
Bottom Layer	Subgrade Course	CAX8	4900	120.0

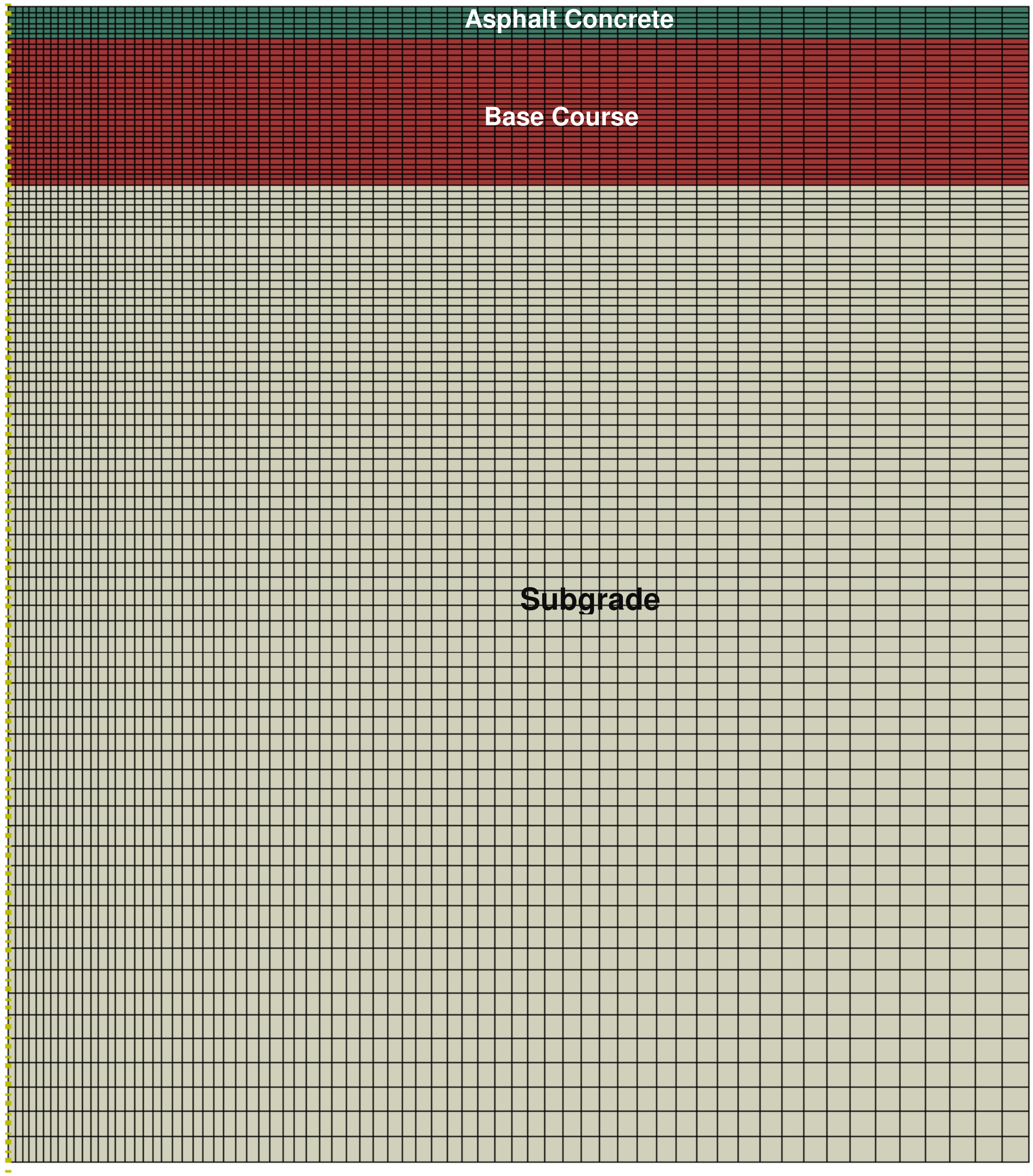


Figure 7.4 Selected mesh size for a typical section analyzed in this study

7.3.2 Boundary Conditions and Loading

Since axi-symmetric elements were considered in the finite element modeling, rotation is not allowed at all nodes. Therefore, only two degrees of freedom have to be considered in defining the boundary conditions. Hence, the boundary conditions applied to the model were a horizontal (x-direction) restraint along the left and the right boundaries and a vertical (y-direction) restraint was applied to the bottom boundary.

Loading consisted of an idealized vehicle load of 18-kips single axle wheel with a total load on each wheel of 9 kips. The wheel contact pressure was taken to be 80 psi and is applied over a circular area of 6 in. in radius, which is similar to the contact pressure that was adopted in the finite element analyses for the development of a new mechanistic-empirical pavement design (NCHRP 1-37 A, 2004) and also used by Nazzal et al. (2006).

Figure 7.5 presents the developed model with the boundary conditions highlighted along with the loading applied.

7.3.3 Material models

In the mechanistic empirical methods such as Asphalt Institute design method and NCHRP 1-37A design method which are commonly used for the analysis of flexible pavement systems under traffic load, the pavement layers are considered as homogenous, linear elastic and isotropic and the loading is considered as static (Liang and Zhu, 1995).These mechanistic methods work reasonably well, if the pavement subgrade system behaves as a linear elastic system. However, in the real situation, these heterogeneous pavement layers behave far from these ideal conditions (Hadi and Bodhinayike, 2003).

Several finite element studies were conducted to evaluate the type of material model (linear or non-linear) that gives a better pavement response. One such study undertaken by Tutumler and Minkwan (2007) studied the usage of linear and non-linear material models for subgrade and base materials on both 3-D and axi-symmetric models. In this study modulus models which were proven (Uzan, 1985) to adequately describe the non-linearity in the pavement geomaterials were programmed in a user material subroutine (UMAT) to perform axisymmetric and three-dimensional (3-D) analyses using ABAQUS. The predicted pavement responses of 2-D axi-symmetric analysis from this study are presented in Table 7.4

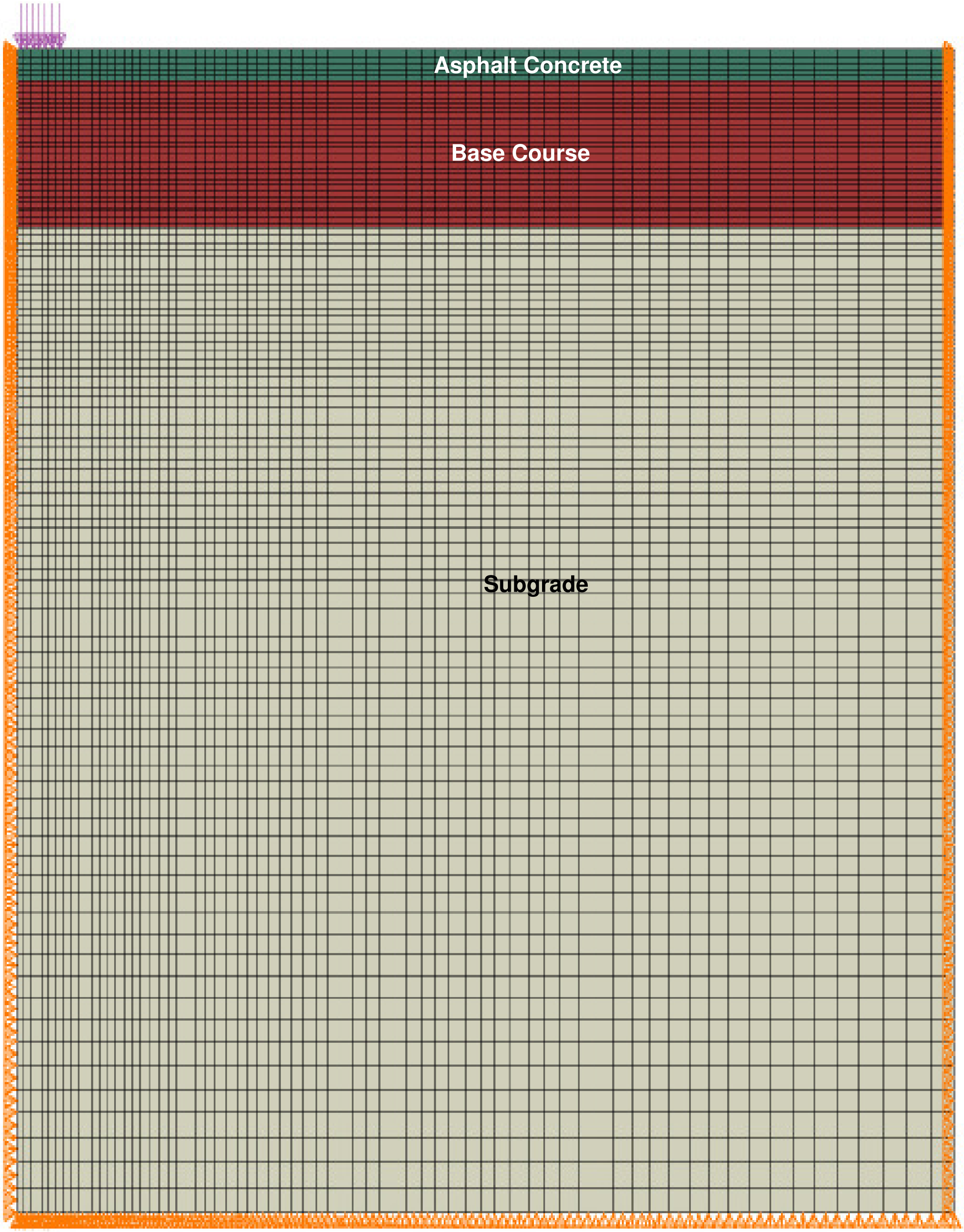


Figure 7.5 Mesh along with the boundary conditions and loading from axisymmetric analysis conducted in this study

In this table it could be observed that the pavement responses differ when the subgrade and base materials are assumed differently. The authors recommend that non linear material models should be used in the analysis to have a better understanding of the pavement response under static loading. However, it can be observed from the table that the values are not significantly different when both the base and the subgrade are assumed to behave as linear elastic or non-linear elastic.

Table 7.4 Pavement response from a 2-D axi-symmetric modeling of a pavement section (From Tutumler and Winkwan, 2007)

Pavement response	**Linear base and linear subgrade**	**Nonlinear base and linear subgrade**	**Linear base and nonlinear subgrade**	**Nonlinear base and nonlinear subgrade**
Displacement on surface, $\delta_{surface}$ (mm)	-0.93	-1.24	-0.76	-0.97
Radial Strain at the bottom of AC, ε_r	227	267	227	257
Vertical strain at the top of subgrade, ε_r	-932	-1,203	-772	-937

The compressive strains on top of the subgrade were determined by applying a single axle wheel load of 18 kips over an area of radius 6 in. This compressive strain is used in eq (7.1) to obtain the number of ESALs required to cause rutting failure. As, this wheel load is not sufficient enough to produce any plastic deformation in the pavement materials,, all three materials were assumed to behave linear elastically. Thus the built-in material model ‘ELASTIC’ in general purpose software ABAQUS is used for the FEM analysis. This kind of analysis is common in the cases where the materials were not loaded beyond their elastic zones.

Table 7.5 presents the elastic moduli of the three subgrade sections, three base course sections and the pavement sections used in the model. Typical subgrade and the asphalt concrete values were taken as suggested by Nazzal et al. (2006). For the properties of the base course material, two soils were selected from the eight soils presented in chapter 5. One was Fort Worth soil which has experienced complete loss of initial strength after 10 cycles of wetting/drying and the other soil was Bryan which retained 85% of the initial strength after 21 wetting/drying cycles. Fort Worth soil was treated with 6% lime and Bryan soil was treated with 8% lime and tested for durability of the treatment by conducting wetting/drying studies. The procedure followed for conducting wetting/drying studies is explained in Chapter 3. As resilient modulus were not measured with the wetting drying cycles, the initial tangent modulus values obtained from the UCS tests conducted after 3, 7 14 and 21 W/D cycles is directly related to the deterioration in the resilient modulus of the base course at different stages after initial compaction.

Table 7.5 Elastic and plastic properties for the different subgrade materials used

Soil Types	**Resilient Modulus, E (psi)**	**Poisson's Ratio, µ**
Subgrade sections		
Weak Subgrade	1000	0.3
Medium Subgrade	2500	0.3
Stiff Subgrade	5000	0.3
Pavement section		
Asphalt concrete	500,000	0.3
Base course section Fort Worth		
Initial	30000	0.35
Intermediate1	15000	0.35
Intemediate2	5000	0.35
Final	500	0.35
Base course section Bryan		
Initial	45000	0.35
Intermediate1	30000	0.35
Intemediate2	28000	0.35
Final	25000	0.35

7.4 Analysis of Test Results

A 2-D axi-symmetric finite element model was developed to evaluate the effect of base course degradation on the performance of a flexible pavement system. The scope of the study and details of the model were explained in the above sections. This section attempts to study the effect of various factors such as AC layer thickness, base course layer thickness and subgrade strength on the long-term performance of a flexible pavement section.

7.4.1 Effect of subgrade strength

Three different subgrade strengths were considered here; soft, medium and stiff. Resilient modulus values of these subgrades are given in Table 7.5 above.

Figure 7.6 and Figure 7.7 present the effect of subgrade modulus on the performance of flexible pavements for four different pavement sections with base layers as treated Bryan soil and treated Fort Worth soil respectively. It can be observed here that as the subgrade strength reduces, the number of ESALs required to cause rutting failure (as obtained from equation 7.1 above) also reduces. This is evident as subgrades with lower strengths with different base layers still undergo high deformation under repeated loading. This is clearly shown in this plot as the ESALs required to cause rutting failure reduce with decreasing subgrade modulus property (weak subgrades).

7.4.2 Effect of base deterioration

Two different base material properties were studied one from lime treated Fort Worth soil and the other from lime treated Bryan soil.

Figure 7.9 and Figure 7.10 show the displacement contours for the modeled pavement section with base materials from treated Bryan and Fort Worth soils. The sections presented here are having a pavement layer thickness of 4 in. and base layer thickness of 18 in. The red zones indicate a displacement magnitude of more than 0.25 in. in the vertical direction. The reason behind selecting 0.25 in as a threshold value is to make sure that all the contours have the same color code. The deterioration in the case of treated Fort Worth soil as a base can be clearly seen as the whole pavement section is experiencing a displacement of 0.25 in. In the case of treated Bryan soil as a base, the red zone is limited to the loading area indicating that the base course is serving its purpose of containing the displacement.

Figure 7.11 and Figure 7.12 illustrate the plots between the ESALs and the base modulus deterioration utilizing Bryan and Fort Worth treated bas properties. It can be observed here that the pavement section with lime treated Bryan soil as base had almost same ESALs for all four cases of deterioration.

In Figure 7.12, ESALs decreased with the time in the pavement section built with treated Forth Worth soil as a base layer. This is because of the deterioration in base modulus with time due to moisture fluctuations, which in turn increased plastic deformation. Hence, this pavement above could not support the traffic loading for the intended design period.

Figure 7.8 presents comparison of the percentage reduction in ESALs with deteriorating base modulus for the three different subgrades studied here. Percentage ESAL degradation was computed based on the original ESALs computed with the original stabilized base material properties. It can be observed here that the reduction in ESALs with degrading base layer strengths was increasing with the strength of the subgrade layer below. This means that a stiffer subgrade reduces the effects of the base layer strength reduction on the pavement performance. However, failure was seen even in stiffer subgrades when the base modulus is less than the subgrade modulus.

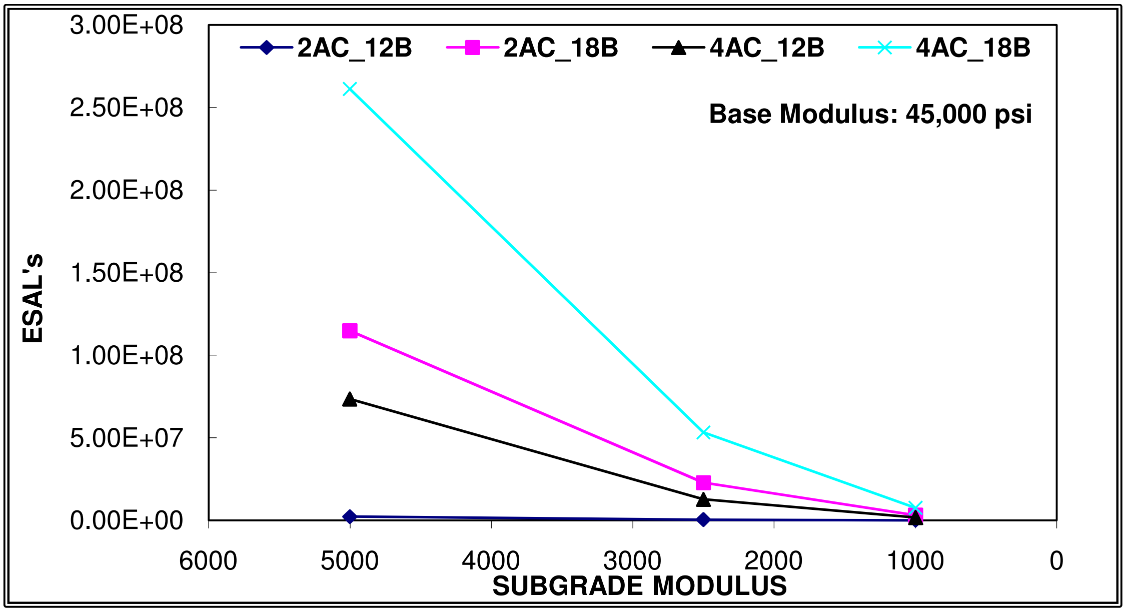


Figure 7.6 Variation of ESALs with the subgrade modulus for different pavement sections with base layer as treated Bryan soil

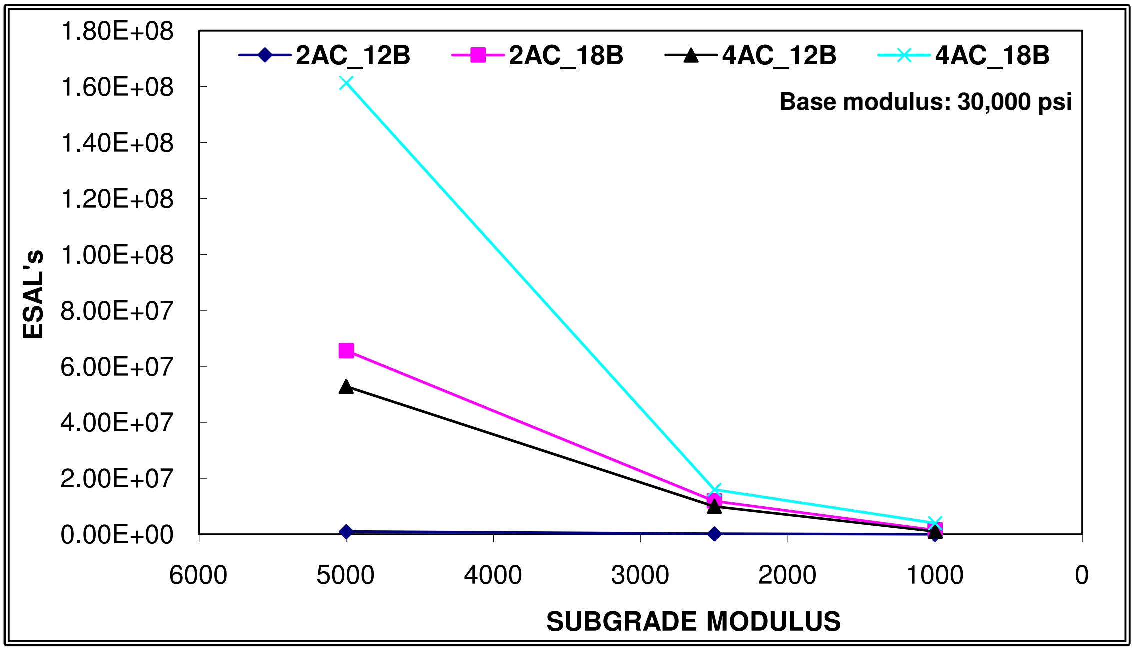


Figure 7.7 Variation of ESALs with the subgrade modulus for different pavement sections with base layer as treated Fort Worth soil

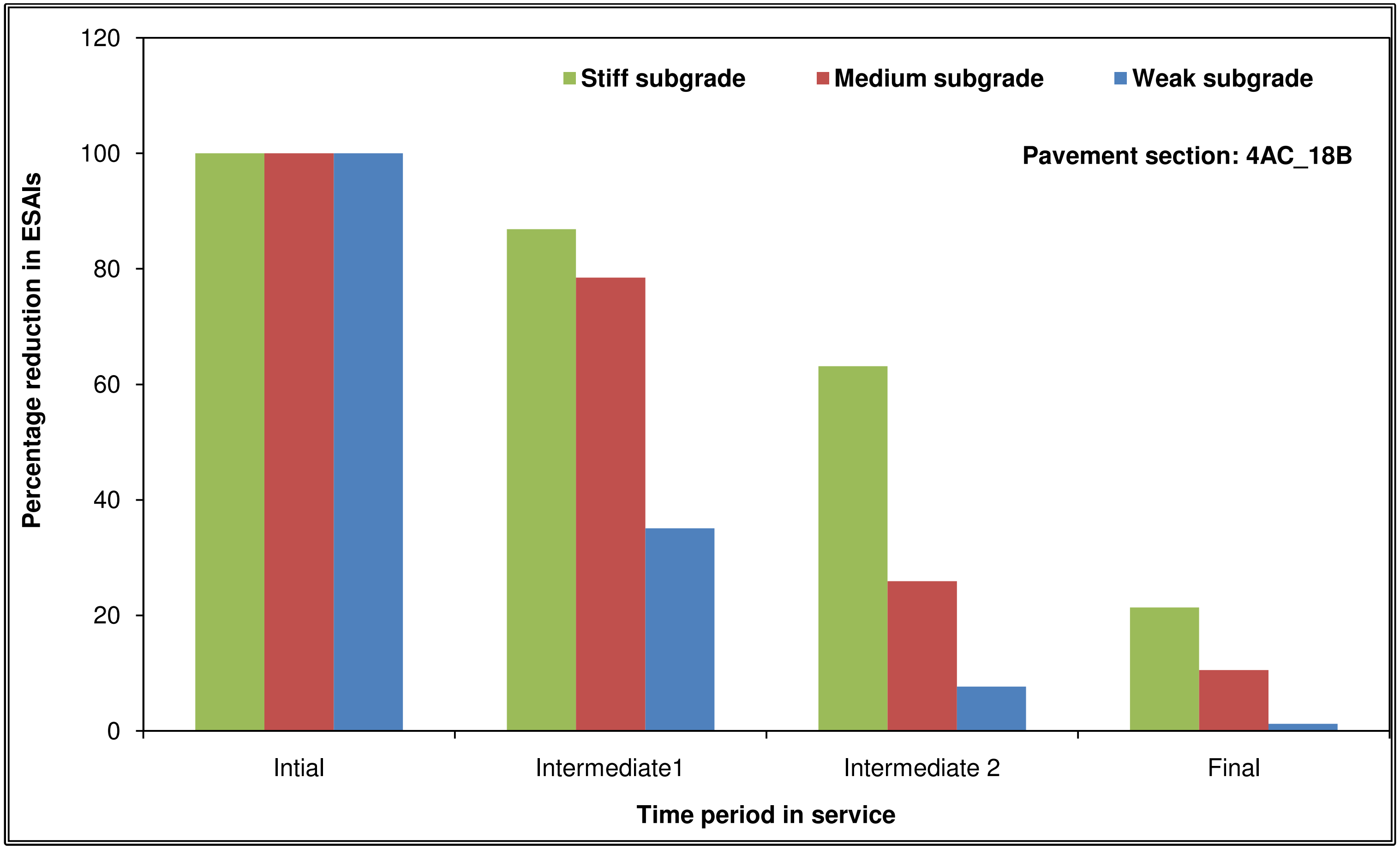
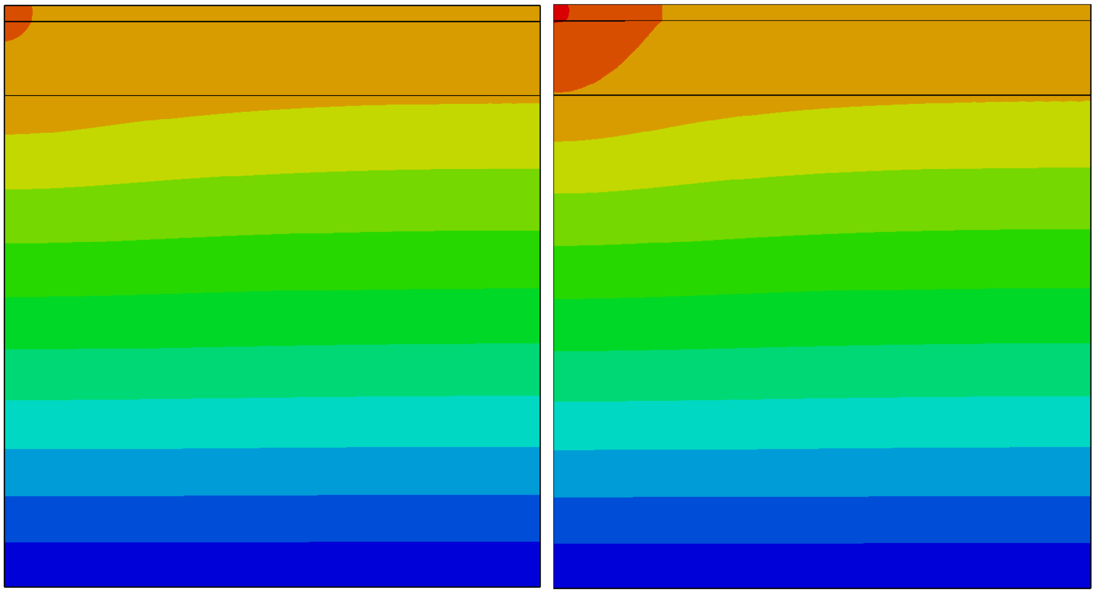
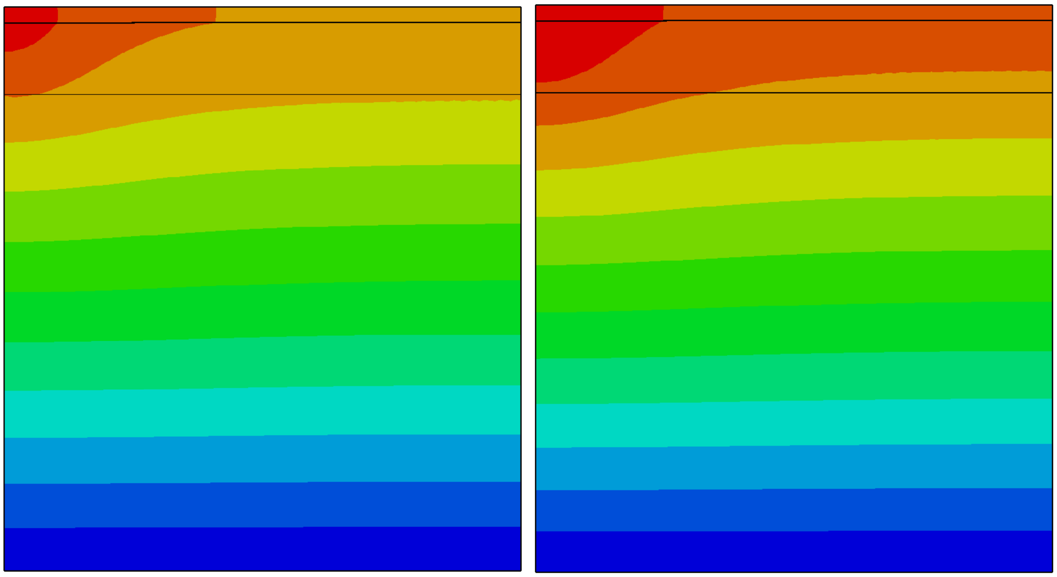


Figure 7.8 Comparison of percentage reduction in ESALs with deteriorating base modulus for different subgrades



Initial

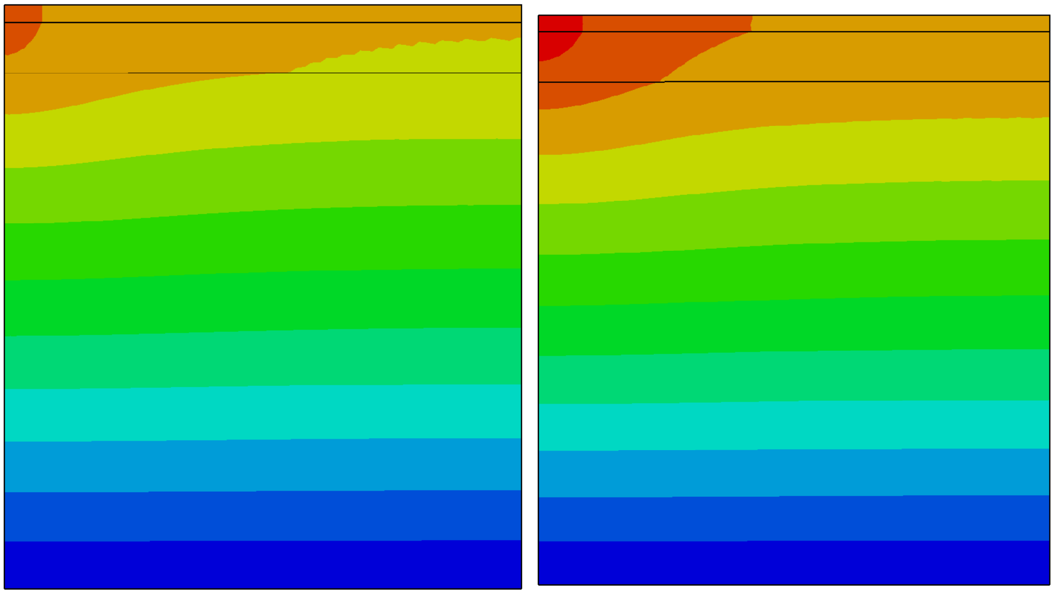
Intermediate1



Intermediate 2

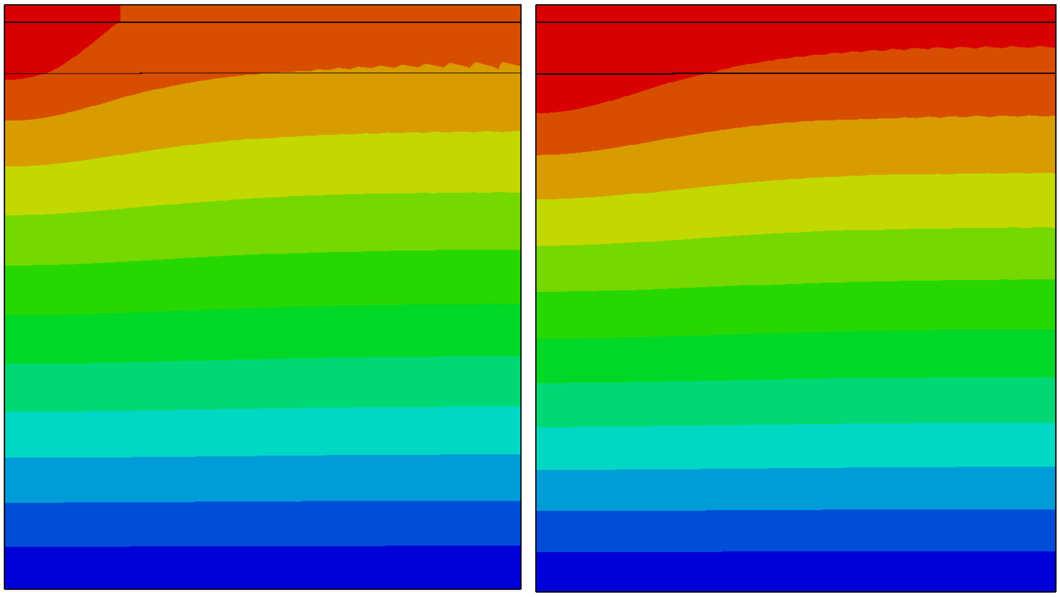
Final

Figure 7.9 Displacement contours showing the pavement response for the section with treated Bryan soil as base



Initial

Intermediate 1



Intermediate 2

Final

Figure 7.10 Displacement contours showing the pavement response for the section with treated Bryan soil as base

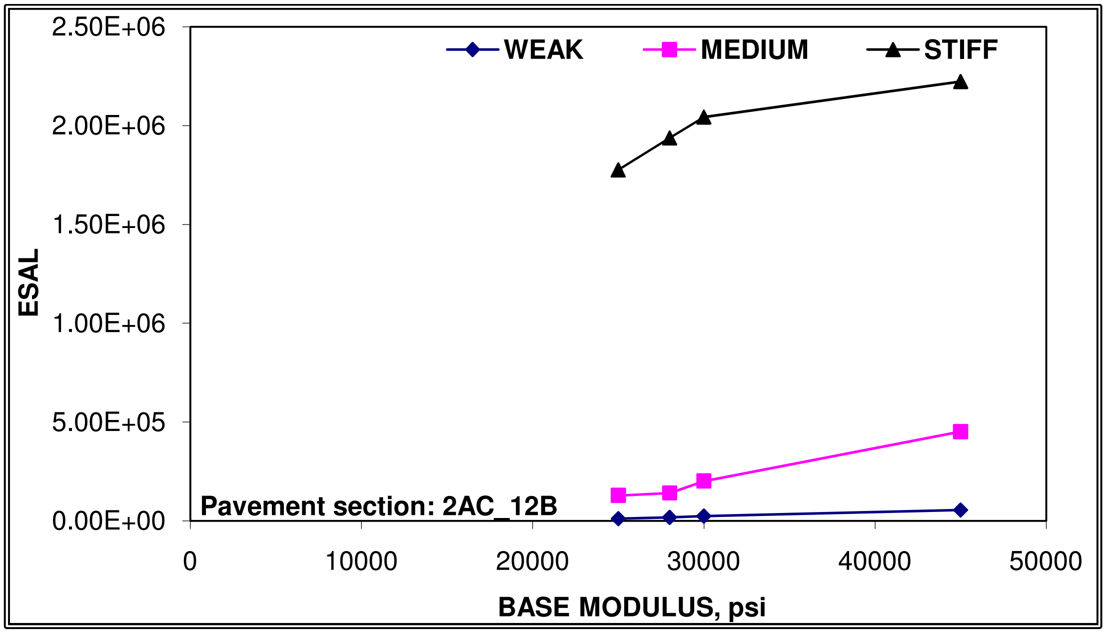


Figure 7.11 Variation of ESALs with deteriorating base modulus of Bryan soil for different sub-grades

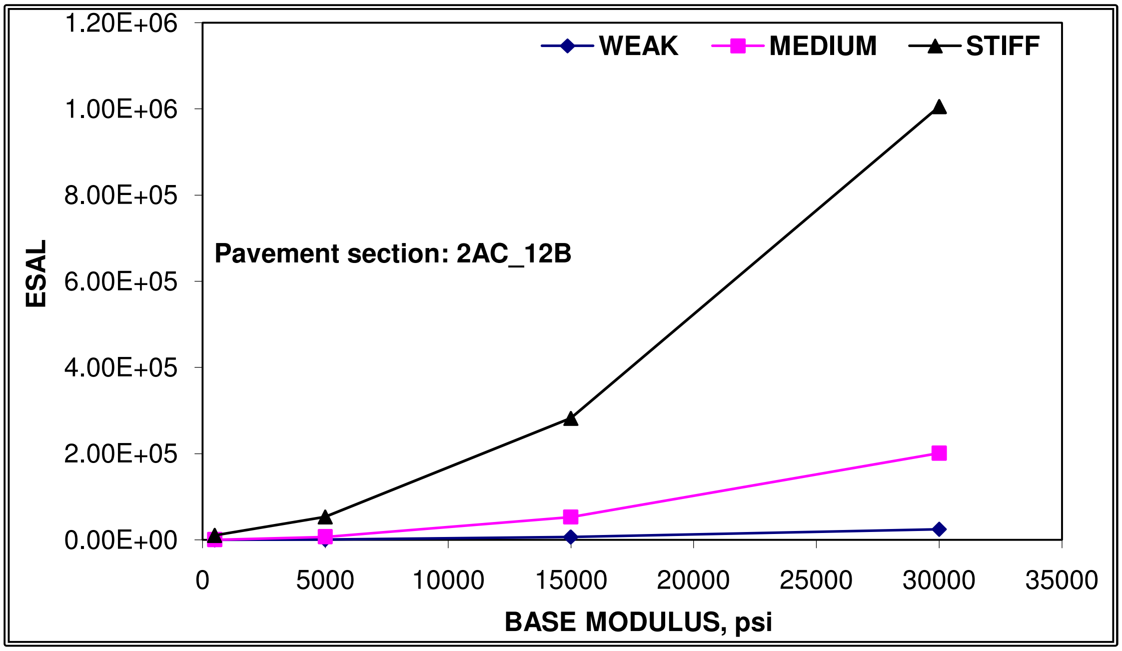


Figure 7.12 Variation of ESALs with deteriorating base modulus of Fort Worth soil for different sub-grades

Figure 7.13 presents a comparison of the percentage reduction in the ESALs as the base modulus is deteriorating with time for the two different base materials used here. It can be observed here that as the modulus is deteriorating, the percentage reduction in the ESALs decreases drastically for the section with treated Fort Worth soil as base while that of the section with treated Bryan soil as base has been almost constant. This shows there is in fact an effect on the pavement response as the base modulus degrades and the pavement does not serve the intended number of ESALs or the design life period.

7.4.3 Effect of base layer thickness

To study the effects of the base course thickness on the performance of the pavement under deteriorating base course strength two different base layer thicknesses with same asphalt layer thickness were studied here. Figure 7.14 presents a typical plot for comparing the percentage reduction in ESALs with degrading base modulus for two different base layer thicknesses for the section with Bryan soil as treated base. Here it can be observed that the percentage reduction in the ESALs was higher for 12 in base layer than 18 in.

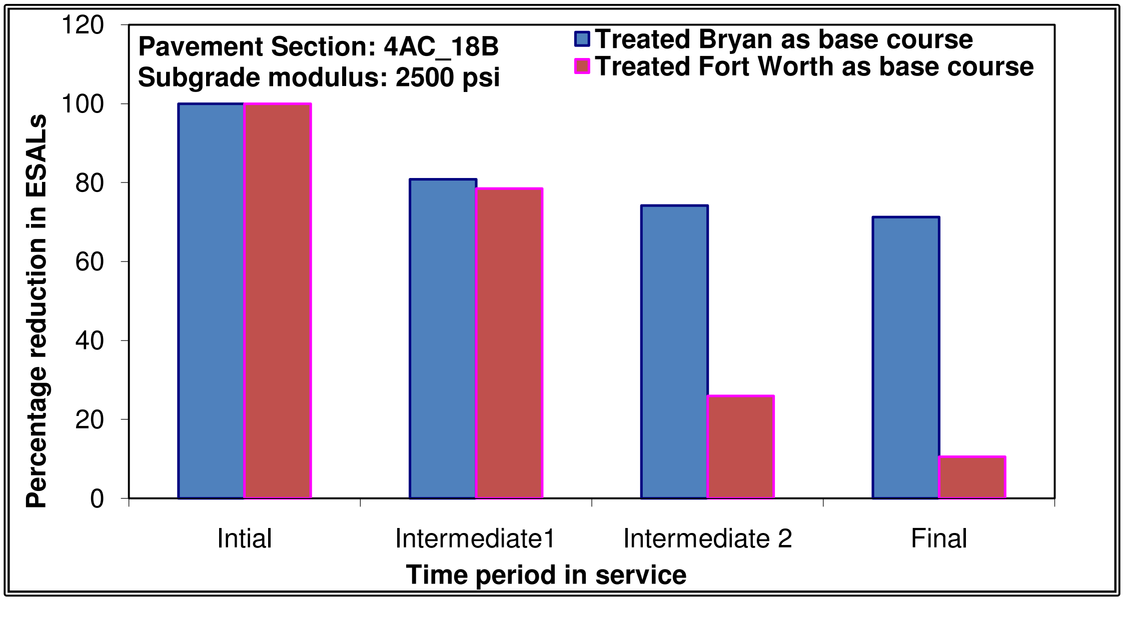


Figure 7.13 Comparison of percentage reduction in ESALs with deteriorating modulus for both the base materials used

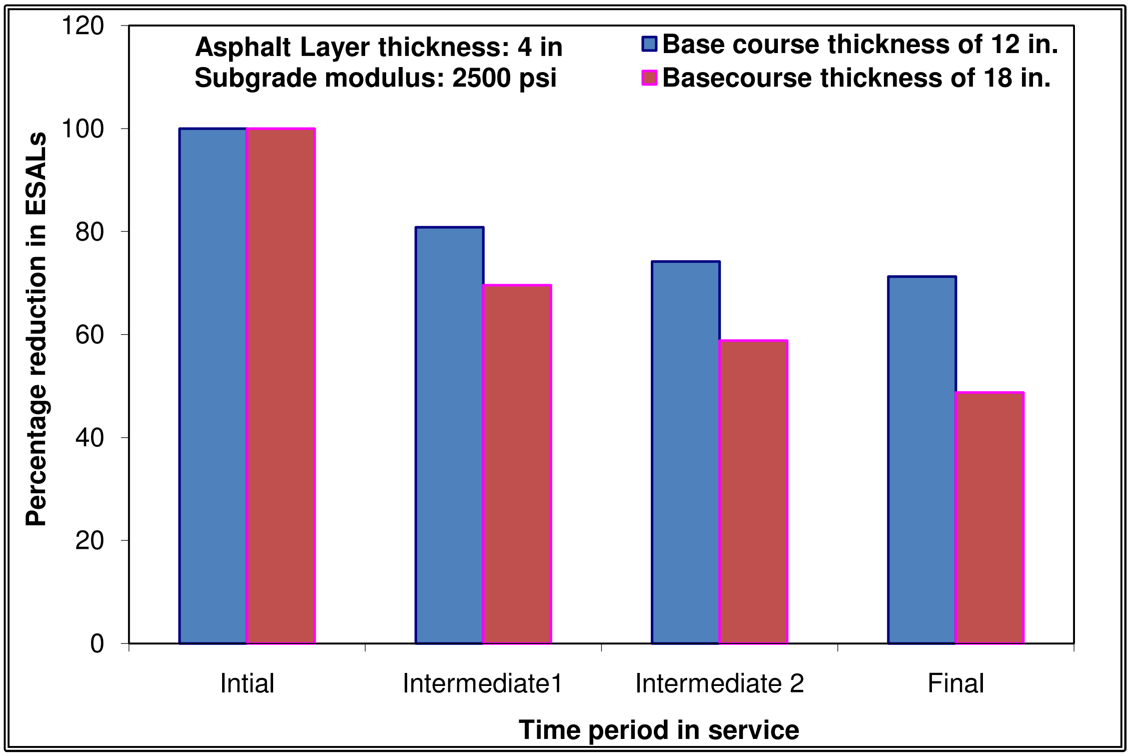


Figure 7.14 Comparison of percentage reduction in ESALs with deteriorating modulus for two different base course thicknesses

7.5 Summary

Finite element analysis using commercially available software ABAQUS 6.8.1 is conducted on a pavement section to study the effect of deteriorating base modulus on the performance of a flexible pavement. Four different flexible pavement sections were analyzed varying asphalt concrete layer and base course layer thicknesses. Three different subgrade moduli (weak, medium and stiff) were considered in this study. Base moduli were obtained at four different time frames during the service of the pavement from the wetting/ drying studies conducted previously. Two sets of this data were obtained from two different materials out of the eight soils studied in chapter 5. The soils selected were Bryan and Fort Worth.

Two-dimensional axi-symmetric finite element analysis was conducted. The element type selected was CAX8 and the element size selected was 1 in. All the materials were assumed to behave linearly elastic. Two asphalt layer thicknesses (2 in and 4 in) and two base layer thicknesses (12 in and 14 in) were selected for this study.

The effect of base modulus deterioration on the pavement performance is assessed by obtaining the compressive strain on the subgrade top by conducting finite element analysis and then using this compressive strain in equation 7.1 above to obtain the ESALs required to cause rutting failure.

It has been shown from the results that as the base modulus deteriorates the number of ESALs required to cause rutting failure also reduces. Hence, it could be understood that the deterioration in the base modulus is going to reduce the life of the pavement. Also, the effect of subgrade stiffness on the pavement performance under the deteriorating base modulus condition was studied. It is observed that as the stiffness of the soil increase the effect of deterioration of the base modulus is reduced initially. However, failure was observed even in the stiffer subgrade when the base modulus was lowered than the subgrade modulus.

The following chapter summarizes all the different studies that were conducted in this dissertation and conclusions were made based on these studies. Also, recommendations for future studies were provided.

CHAPTER 8

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

8.1 Summary and Conclusions

Stabilization of expansive clays with chemical additives has been practiced for decades to improve the performance of base and subgrade materials. Soil stabilization has been a topic of interest and discussion for all these years due to potential reduction in the construction and maintenance costs if the pavement and other civil infrastructure are built on these problematic expansive grounds. Moreover, in the case of pavements, this treatment could enhance riding comforts to travellers.

Extensive research was documented with regard to the engineering properties, reliability and durability of various types of stabilized materials. Different testing methods, design, construction, and quality assurance/quality control (QA/QC) methodologies have been developed for these stabilized materials. Many pavement projects constructed with stabilized materials have achieved satisfactory results. However, state Departments of Transportation (DOTs) in the United States have had problems with premature subgrade failure due to a loss of stabilizer over time, or a stabilizer being ineffective in some soils while other soils with the same index properties respond well to that stabilizer.

In this research an attempt is made to identify the causes of these premature failures and based on these causes, new/improved guide lines are outlined for the better design of stabilizers in the field. One of the causes of the problem was lack of understanding of the chemical interactions between the stabilizer and the soil minerals. Also, durability and permanency related stabilization studies were needed to understand the effect of the clay mineralogy on the long-term performance of stabilized expansive soils. All these will be instrumental in the development of better stabilization guidelines. This dissertation research was an attempt to address these problems.

The first task in this study was identified as developing a procedure to indentify and quantify the clay minerals in the soil. For this purpose chemical and mineralogical tests were conducted on the clay portion of the soil to measure properties such as cation exchange capacity (CEC), specific surface area (SSA) and total potassium (TP). These properties were used to measure the percentages of Montmorillonite, kaolinte and Illite minerals in soil. It was assumed that the clay fraction contains only these three clay minerals. Though other minerals are present in the clay fraction of the soil their percentages are negligible hence were assumed to be zero for simplicity. Inclusion of other non-clay minerals such as quartz and feldspar could be a topic for future research.

A total of 20 natural soils from different parts of Texas region were collected and subjected to above mentioned chemical and mineralogical tests. All these three test procedures were first assessed for repeatability and then these tests were conducted on all these soils. A database of these results was developed. In order to check the accuracy of the a model in predicting the dominating clay mineral in the soils, six soil samples were artificially prepared with known percentages of the minerals. These known percentages of minerals were compared with the percentages predicted by the developed models.

Prediction models were developed using the database. Tools such as regression analysis and artificial neural networks were utilized to develop the prediction models. These clay mineralogy prediction models were validated using the validation artificial soil data. Predictions by both methods were compared and it was observed that ANN based prediction model showed better prediction capabilities than regression model based equations. However, the differences between predictions were small and practically negligible. Hence any of these models could provide realistic prediction of clay mineralogy in a given soil.

The second task was to assess the effects of clay minerals on the long-term durability of stabilized expansive clays by conducting wetting/drying (W/D) studies replicating moisture fluctuations expected during summer and winter seasons in the field. A total of eight soils were selected for studying the long-term performance of stabilized expansive soils by conducting wetting/drying studies. Stabilizer design was carried out as per the TxDOT methods Tex 120-E (Lime as additive) and Tex 121-E (Cement as additive) and the results are presented. An accelerated curing method was developed and followed in this study for curing and moisture conditioning of the treated soil specimens. The effect of curing methods is studied on four select soils and it is observed that both the curing methods including old and longer curing (Tex 121-E) for 17 days and the present accelerated curing methods of 3 days yielded similar UCS and volume change test results. It has been interpreted from these results that there was no considerable effect of curing on the long-term performance of these treated soils except for an initial strength since soil was partially saturated.

The effect of clay mineralogy on the long-term performance of stabilized expansive soils was noted in this study. Both volumetric strain and unconfined compressive strength were measured at various numbers of wetting/drying cycles. The results of all the eight soils showed that if the dominating clay mineral was Montmorillonite, then durability of stabilizers will be a concern.

Leachate studies were conducted on all the eight soils selected to replicate moisture ingress and digress in the field during rainfalls and to study the effect of these moisture infiltrations on the long-term performance of stabilized soils. Leachate samples were collected after 3, 5, 7, and 14 cycles of leaching to address the chemical changes occurring due to leaching of the additive from the soil specimen. Also, unconfined compressive strength tests were conducted after 3, 7 and 14 cycles of leaching to address the strength changes due to leaching in the soil specimens.

The effects of both curing methods on the leachability of treated expansive clays were studied on four soils and it was observed that both the curing methods yielded similar results. It has been interpreted from the results that there was no considerable influence of the curing period on the long-term performance of these treated expansive clays except for an initial strength loss which was attributed to saturation of the specimens at which the specimens were tested.

The effects of clay mineralogy on the long-term performance of stabilized expansive soils were observed. The concentration of the calcium ion and the pH of the leachate and unconfined compressive strength were measured versus the number of leaching cycles. The results of all the eight soils that were considered here are presented in detail. No considerable changes were observed in calcium ion concentration, pH of the leachate among different test soils. To better understand the calcium ion concentration loss with the leachate cycles, the leachate concentration was converted into an equivalent percentage of lime/cement additive that was leached out depending on the additive type used. The percentage lime and cement leached out of the soil specimens after fourteen (14) cycles of leaching were low (<0.6% for lime and <0.4 for cement) for all the soils tested.

An attempt was made to correlate the laboratory leachate cycles to the field precipitation and moisture infiltration information to understand the high retained strength and low percentage lime leached out after 14 cycles of leaching. It was observed that the number of field years replicated in all the eight soils is less than 5 years which is short duration compared to the life of the pavement.

Finally, finite element analysis using a commercially available software ABAQUS 6.8.1 was conducted on hypothesized pavement sections to study the effects of deteriorating base modulus due to weakened stabilized base material on the performance of a flexible pavement. Four different flexible pavement sections with varying asphalt concrete layer and base course layer thicknesses were altogether analyzed. Three different subgrade moduli (weak, medium and stiff) were also simulated in the analysis. Base moduli obtained at four different time frames during durability studies were used as model parameters at different service periods of the pavement. Two-dimensional axi-symmetric finite element analysis using CAX8 elements of 1 in. size were used. All the materials were assumed to behave as a linearly elastic material. Two asphalt layer thicknesses (2 in. and 4 in.) and two base layer thicknesses (12 in. and 14 in.) were selected and studied. The effect of base modulus deterioration on the pavement performance was assessed by obtaining the compressive strains on the subgrade top. These strains were used with the Asphalt Concrete Institute (2006) formulation to predict the ESALs required to cause rutting failure. Based on the all the above mentioned results, the following major conclusions are obtained:

1) A new method was developed for the quantification of clay minerals in a given soil using statistical principles of regression modeling and the effectiveness of the model was demonstrated.
2) Soils containing Montmorillonite as a dominant mineral were more susceptible to premature failures after chemical stabilization when they were exposed to volume changes caused by swell and shrink related volume changes.
3) Also, it is understood that low amount of additive dosage can cause premature failures in the pavement structure.
4) The current approach of PI based chemical stabilizer does not provide considerable insights into the chemical stabilizer and its durability. Hence, the proposed research recommends the use of stabilizer design by including clay mineralogy aspects in to the stabilizer design that included both additive selection and its dosage. Such stabilizer designs could lead to a more realistic and practical designs and also provide better answers to the permanency of the stabilizer.
5) The effects of the additive type on the long-term performance of stabilized expansive soils under wetting/drying conditions were studied in this research. It has been observed that

cement treatment lasted more number of cycles in the case of medium PI soil, Pharr-B soil.

6) Also, in the case of the medium PI soil (Pharr-B soil) which was not dominant in the mineral kaolinte, when treated with both lime and cement premature failures were observed during wetting and drying process. This contradictory behavior is attributed to low dosages of cement and lime.
7) Leachate studies showed that the percentage retained strength after leaching for 14 cycles was similar for all the soils. This demonstrates that percentage of Montmorillonite mineral in a soil does not have any bearing or correlation on the leachability of treated expansive clays.
8) However, low additive dosages in soils could cause damage to the stabilization and the overall structure. This was shown in the case of Pharr-B clay which was treated with 3% lime and retained only 55% of the initial strength after 14 cycles of leaching.
9) Also, the retained strengths after 14 cycles of leaching were lower when treated with lime than with cement. This proves the pre-established fact that cement treatment gives better improvement of the subgrade, particularly in the case of low PI clays than high PI clays.
10) Also, the study on the effect of the amount of dosage indicated that higher dosages of lime resulted in the better stabilization of soils and the soils performed well under leaching studies. These results are in agreement with the conclusions made by McCalister (1990).
11) Hence, it was concluded that the percent leaching of lime in short term leachate studies is small, but can be considerable if the original stabilizer dosage used is less than 4. Overall, this study clearly showed that leaching may not be highly problematic in the initial years if the treatment dosages are high (6% or high).
12) From the finite element analyses conducted, it has been shown that the base modulus deterioration has resulted in the decrease of the number of ESALs required to cause rutting failure.

All these findings have provided valuable data that is used in the next section on new modifications of stabilizer design.

8.2 Proposed modifications

Overall, 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. Hence, the following recommendations are made:

Aside from the gradation and PI properties, the clay mineralogy impacts the concentration of the additives the most. It is recommended to include this information in the stabilization design process. Direct measurements of the clay mineralogy utilizing X-Ray Diffraction (XRD) and Scanning Electron Microscope (SEM) are not practical for day-to-day use since they are expensive to perform and require advanced instrumentation. Hence a set of simple chemical tests, 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 three methods are provided in Appendices A through C. The following process is proposed to obtain the percentage of the three common clay minerals:

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

$$\%I = \left[\frac{TP}{6}\right] \times 100 \qquad (8.1)$$

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 Equations 2 and 3:

$$\%M = -2.87 + 0.08 \times SSA + 0.26 \times CEC \qquad (8.2)$$

$$\%K = 100 - \%I - \%M \qquad (8.3)$$

Figure 8.1 presents the flowchart that is being used in the field as per the current TxDOT stabilization guide lines for reference and Figure 8.2 illustrates the same flowchart with the insertion of some additional chemical analyses and an accelerated mix design steps based on the present research observations.

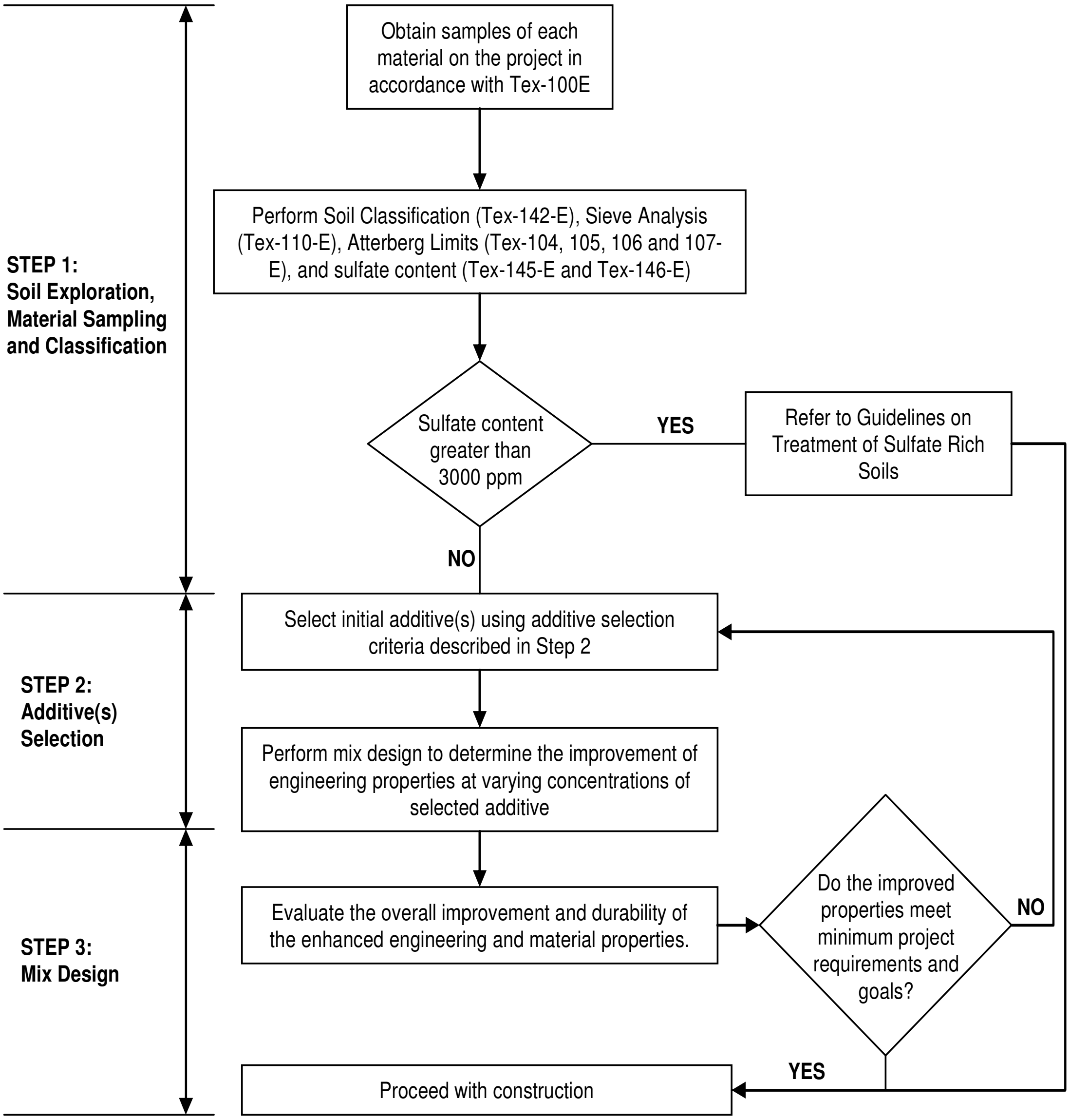


Figure 8.1 Current TxDOT guidelines for subgrade soil treatment

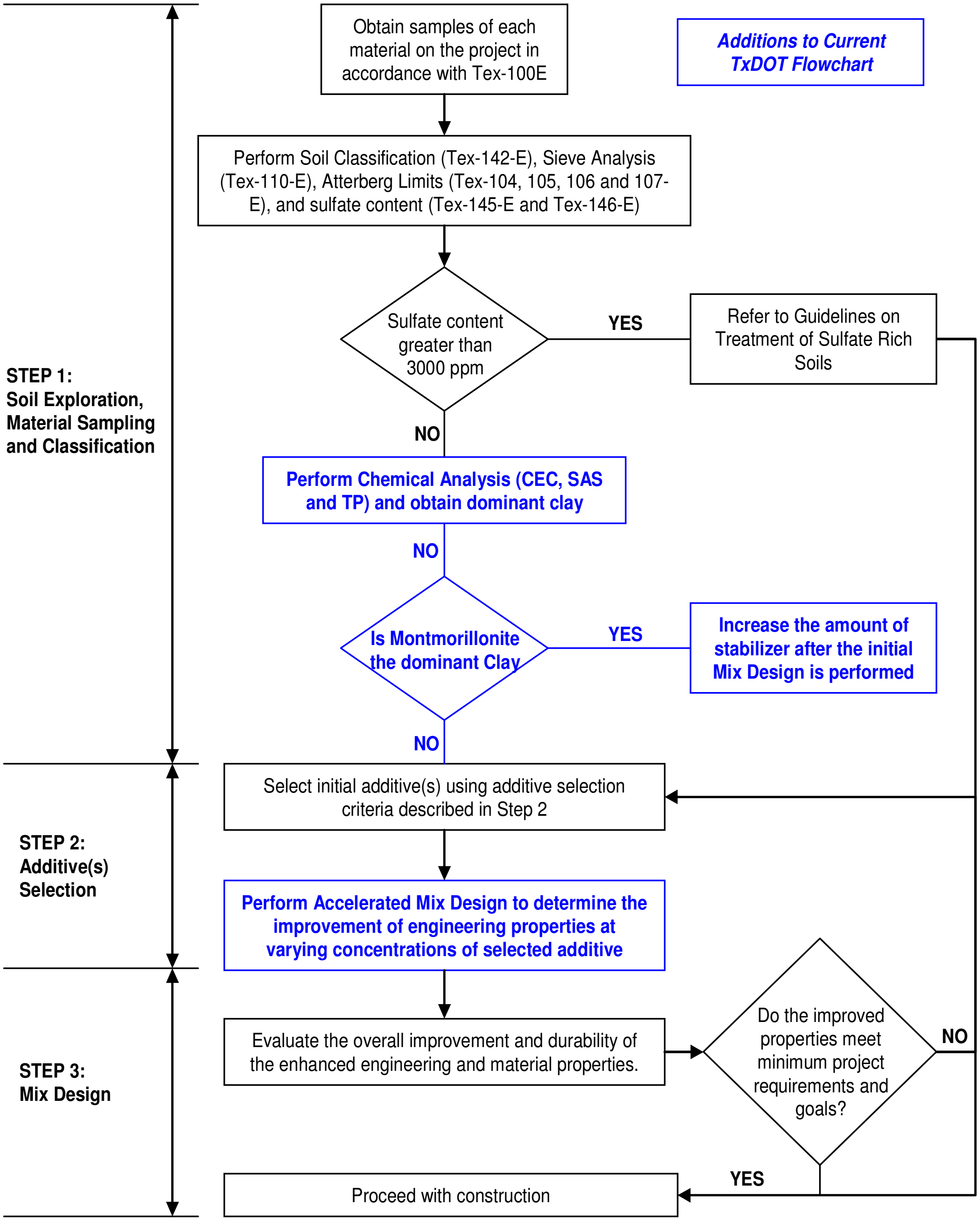


Figure 8.2 Proposed guidelines for subgrade soil treatment

Such designs will help in developing better stabilization guidelines for chemical stabilization of subsoils.

8.3 Recommendations for future research

- The method developed to identify the dominating clay mineral assumes that the clay fraction of the soil contains only the three minerals Kaolinite, Illite and Montmorillonite. However, in reality other non-clay minerals such as quartz, feldspar is also present. Hence, studies should be focused to include one or two more parameters and account for the non-clay minerals.
- Chemical tests need to be conducted during wetting/drying tests to check if there is any additive loss during this process.
- It is recommended to conduct nondestructive tests like resonant column tests on the soil specimens during both wetting/drying and leaching cycles to avoid sample wastage and maintain singularity.
- Monitoring cation exchange capacity with wetting/drying and leaching could give us insights into the mineral modifications that are taking place due to these processes.

APPENDIX A

CATION EXCHANGE CAPACITY

Cation Exchange Capacity

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 glass flasks, 500 ml with vacuum hose.
- 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.
- 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.

Section 4

Material

- Distilled or de-ionized water
- 1 M Ammonium Acetate (NH4OAc) saturating solution: Dilute, in a chemical hood, 57 ml glacial acetic acid (99.5%) with ~800 ml of distilled water in a 1 l volumetric flask. Add 68 ml of concentrated Ammonium Hydroxide (NH4OH) mix and cool. Adjust pH of the sample to 7.0 using NH4OH and dilute to 1 l.
- 1 M Potassium Chloride (KCl) replacing solution: Completely dissolve 74.5 g KCl (reagent grade) in distilled water and dilute to a final volume of 1 l.
- 2-propanal, 95%.

Preparation of Material

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

Table A.1 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 the 500 g sample to pass the 75 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:

Table A.2 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 16 hours or overnight.
3	Take a 500 ml Erlenmeyer glass flask which has a provision for applying vacuum.
4	Place the Buchner funnel on top of the 500 ml Erlenmeyer flask and tighten with the help of a rubber stopper which has a hole in the middle for the stem of the Buchner funnel to go through.

Table A.2 Continued	
5	Place a retentive filter paper in the funnel and wet the paper and apply light suction
6	Filter the soil solution from step 2 through the Buchner funnel into the 500 ml flask
7	Leach the soil four times with 25 ml additions of the Ammonium Acetate, NH_4OAc, allowing each addition to filter through but not allowing the soil to crack or dry.
8	Leach the soil with eight separate 10 ml additions of 95% 2-propanol 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 with eight separate 25 ml additions of potassium chloride (KCl) 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 or ion chromatograph

Calculations:

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

$$CEC(meq/100g) = \frac{\mathrm{NH_3\text{ - }N\ (ppm)}}{14} \tag{A.1}$$

Note:

- NH_3-N to be used in the above formula is NH_3-N in extract - NH_4-N in blank.
- If NH_3-N is expressed as mg/L of NH_4, instead of NH_3-N, divide the result by 18 instead of 14 to obtain CEC.

APPENDIX B

SPECIFIC SURFACE AREA

Specific Surface Area

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 cans
- Culture chambers (Glass dishes with cover, 20 cm diameter And 7.5 cm height)
- Hardware cloth

- Tweezers.
- Pans, brush, and spatula.

Section 4

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.

Preparation of Material

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

Table B.1 Preparation of CaCl2-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, transfer it to a culture chamber and spread it uniformly over the bottom and store it in a desiccator.

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

Table B.2 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 desiccator
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 75 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:

Table B.3 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 a aluminum tare
2	Place the can in an oven at 110^{o}C (230^{o} F) for 24 hours until constant weight
3	Weigh the dried sample taking care not to adsorb atmospheric water (W_s)
4	Add 3 ml of EGME to make slurry
5	Place the can with slurry in the chamber containing $CaCl_2$-EGME solvate over a hardware cloth
6	Close the lid of the chamber and place it in a desiccator containing $CaCl_2$
7	Wait for 30 minutes and evacuate for 45 minutes and close the stopcock.
8	After 6 to 7 hours, release the vacuum
9	Now weigh the soil sample in the can with the lid of the aluminum can.
10	Repeat weight measurements for every 2 to 4 hours of evacuating until there is no further decrease in the weight (W_a).

Calculations

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

$$A = \frac{W_a}{W_s * 0.000286} m^2 / g \tag{B.1}$$

Where,

W_a = Final weight of the soil sample after stabilizing with EGME

Ws = Dry weight of the soil sample before mixing EGME

APPENDIX C

TOTAL POTASSIUM

Total Potassium

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

Table C.1 Sample preparation

Step	Action
1	Dry 500 g of the 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:

Table C.2 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	Heat the soil-acid mixture on hot plate until fumes of perchloric acid, $HClO_4$ appear or heat till the temperature in more than 200^{o}C (392^{o}F)
4	Cool the vessel 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 crucible to 200-225° C (392-437^o F)and evaporate the solution to dryness
7	Again cool the crucible and add 2 ml of water and few drops of perchloric acid, $HClO_4$
8	Replace the vessel on the hot plate and evaporate to dryness
9	Now remove the crucible from the hot plate and when it is cool, add 5 ml of 6N hydrochloric acid, HCl and 5 ml of water
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 volume. This sample will be used to find out the total potassium in the soil using a spectrophotometer.
12	Determine the amount of potassium (K^+) in the solution by using the spectrophotometer or ion chromatograph.

Calculations

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

$$TP,\% = Conc.of K^{+} * 0.05 \qquad (C.3)$$

Where,

TP = % of total potassium

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

REFERENCES

[1] Alexiades, C. A., and Jackson, C. A. (1966). “Quantitative clay mineralogical analysis of soils and sediments.” *Clays and clay minerals*, 14, 35-42.

[2] Al-Rawas, A. A., Hago, A.W., Al-Sarmi, H. (2005). “Effect of lime, cement and Sarooj (artificial pozzolan) on the swelling potential of an expansive soil from Oman.” *Building and Environment*, 40 (5), 681-687.

[3] Amonette, J. E., (2002). “Methods for determination of mineralogy and environmental availability.” Edited by J. B. Dixon and D. G. Schulze, *Soil mineralogy with environmental applications*, SSSA Madison, WI.

[4] Anson, R. W. W., and Hawkins, A. B. (1998). “Effects of calcium ions in pore water on the residual shear strength of Kaolinite.” *Geotechnique,* 48, 787-800.

[5] Attom, M. F., Taqieddin, S. A., Mubeideen, T. (2000). “Shear strength and swelling stabilization of unsaturated clayey soil using pozzolanic material.” *Geotechnical Special Publication*, 99, 275-288.

[6] Bailey. S. W. (1980). “Summary of recommendations of AIPEA nomenclature committee on clay minerals.” *American Mineralogist*, 65, 1-7.

[7] Baker, D.E., and Suhr, N. H. (1982). “Atomic absorption and flame emission spectrometry.” Edited A.L. Page et al., *Methods of soil analysis*. Part 2. (9) SSSA and ASA, Madison, WI, 13–27.

[8] Barenberg, E.J., (1970). “Evaluation of Remolded Field Samples of Lime-Cement-Fly Ash-Aggregate Mixture.” *Highway Research Record No. 315*, Highway Research Board, Washington, DC, 112-121.

[9] Blake, T. A., and Gassman, P. L. (2001). “Detection of soil surface contaminants by infrared reflection spectroscopy.” *Proceedings of SPIE - The International Society for Optical Engineering*, 4577, 239-261.

[10] Borchardt, G. (1989), “Smectites”, In J.B. Dixon and S.B. Weed, eds., *Minerals in Soil Environments*, 2nd ed., Madison, WI: Soil Science Society of America.

[11] Burroughs, S. (2006). “Strength of compacted earth: Linking soil properties to stabilizers”, *Building Research and Information*, 34(1), 55-65.

[12] Camberato, J.J. (2001). Cation exchange capacity – everything you want to know and so much more.http://virtual.clemson.edu/groups/turfornamental/tmi/fertlime/Cation%20exchange %20capacity. Pdf.

[13] Campbell, G. S. (2005). “Measuring Specific Surface of Soil with the WP4”, Application Note, Source: http://www.decagon.com/appnotes/SpecificSurface.pdf.

[14] Carter, D. L., Mortland, M. M. and Kemper, W. D., (1986). “Specific Surface” In A. Klute(ed), *Methods of Soil Analysis*, Part 1, 2nd. ed., Agronomy 9:413-423.

[15] Cerato, Amy B. and Lutenegger, Alan J. (2002). “Determination of surface area of fine-grained soils by the ethylene glycol monoethyl ether (EGME) method”, *Geotechnical Testing Journal*, 25(9), 315-321.

[16] Chapman, H. D., (1965). “Cation-exchange capacity” In: C. A. Black ed., *Methods of soil analysis - Chemical and microbiological properties*, Agronomy 9, 891-901.

[17] Chew, J.Y.M., Paterson, W.R., Wilson, D.I., (2004). “Fluid dynamic gauging for measuring the strength of soft deposits.” *Journal of Food Engineering*, 65(2), 175-187.

[18] Çokça, E. (2001). “Use of Class C fly ashes for the stabilization of an expansive soil.” *Journal of Geotechnical and Geoenvironmental Engineering*, 127(7), 568-273.

[19] Croft, J.B. (1967). “The Influence of Soil Mineralogical Composition on Cement Stabilization.” *Geotechnique,* 17,119–135.

[20] Cullity, B.D. and Stock, S.R. (2001). “Elements of X-Ray Diffraction”, 3rd ed., Addison-Wesley.

[21] Di Maio, C. and Fenelli, G. B. (1994). “Residual strength of Kaolin and Bentonite: The influence of their constituent pore fluid.”, *Geotechnique*, 44, 217-226.

[22] Eades, J. L. and Grim, R. E. (1966). “A Quick Test to Determine Lime Requirements for Soil Stabilization.” *Highway Research Record 139*, HRB, National Research Council, Washington D.C., 61-72.

[23] Ferguson, G. (1993). “Use of self-cementing fly ashes as a soil stabilization agent.” ASCE *Geotechnical Special Publication No. 36*, ASCE, New York.

[24] Frenkel, H., Amrhein, C., and Jurinak, J.J., (1983). “Effects of Exchangeable Cations on Soil Mineral Weathering.” *Journal of Soil Science Society of America*, 49, 649-653.

[25] Goh, A. T. C. (1994). ‘‘Seismic liquefaction potential assessed by neural networks.’’ *Journal of Geotechnical and Geoenvironmental Engineering, ASCE,* 120(9), 1467–1480.

[26] Grim, R. E., (1953). “Clay Mineralogy.” Mc Graw Hill Book Company, New York.

[27] Grim, R.E., (1962). “Applied Clay Mineralogy”, McGraw Hill, New York.

[28] Gromko, G. J. (1974). “Review of Expansive soils.” *Journal of Geotechnical Engineering division*, 100(6), 667-687.

[29] Hagerty, D. J., Ullrich, C. R., and Denton M. (1990). “Microwave drying of soils.” *Geotechnical Testing Journal*, 13(2), 138-141.

[30] Hammons, M. I., (1998). “Validation of three-dimensional finite element modeling technique for joints in concrete airport pavements.” *Transportation Research Record*, 1629, 67-75.

[31] Hicks, R.G. (2002). “Alaska Soil Stabilization Design Guide.” *FHWA*-AK-RD-01-6B.

[32] Hopkins, T.C., Sun, L. and Slepak, M., (2005). “Bearing Capacity Analysis and Design of Highway Base Materials Reinforced with Geofabrics.” University of Kentucky

Transportation Center, College of Engineering, *Research Report KTC-05-21/SPR* 238-02-1F.

[33] Hornik, K., Stinchcombe, M., and White, H. (1989). ‘‘Multilayer feed forward networks are universal approximators.’’ *Neural Networks*, 2, 359–366.

[34] Hoyos, L.R., Laikram, A., and Puppala, A.J (2005). “Assessment of Seasonal Effects on Engineering Behavior of Chemically Treated Sulfate Rich Expansive Clay.” *Proc., GEOPROB2005: International Conference on Problematic Soils*, 25-27.

[35] Hua, J. (2000). “Finite Element Modeling and Analysis of Accelerated Pavement Testing Devices and Phenomenon.” thesis, presented to Purdue University, at West Lafayette, IN, in partial fulfillment of the requirements for the degree of Doctor of Philosophy.

[36] Huang, H.Y. (2004). “Pavement analysis and design.” 2nd ed., Prentice Hall.

[37] Hughes, R. E., Moore, D. M., and Glass, H. D. (1994). “Qualitative and quantitative analysis of clay minerals in soils.” In J.E. Ammonette and L.W. (ed.,) Quantitative methods in soil mineralogy, SSSA Miscellaneous publication, Soil Sci. Soc. Am., Madison, WI. Soil Science society of America, 330-359.

[38] Hussein E.A. (2001). “Viscoplastic Finite Element Model for Expansive Soils.” EJGE paper 1-22.

[39] Jackson, M. L. (1958). “Soil chemical analysis.” Prentice Hall Inc., Englewood Cliffs, N.J.

[40] Johnson, L, J., Chu, C. H. and Hussey, G, A., (1985). “Quantitative clay mineral analysis using simultaneous linear equations.” *Clays and Clay minerals*, 33(2), 107-117.

[41] Johnson, L.D. and Stroman, W.R. (1976). “Analysis of Behavior of Expansive Soil Foundations.” U.*S. Army Engineer Waterways Experiment Station Technical Report* S-76-8.

[42] Jones, D. E. and Holtz, W. G. (1973). “Expansive Soils - the Hidden Disaster.” Civil Engineering (ASCE), 43(8), 49.

[43] Karuiki, P. C., Woldai, T. and Van Der Meer, F. (2006). “Effectiveness of spectroscopy in identification of swelling indicator clay minerals.” *Int. J. Remote Sensing*, 25(2), 455–469.

[44] Khattab, S. A., Al-Mukhtar, M. and Fleureau, J.M. (2007). “Long-Term stability characteristics of a lime-treated plastic soil.” *Journal of Materials in Civil Engineering*, 19(4), 358-366.

[45] Kim, J. and Hjelmstad, K. D. (2003). “Three-Dimensional Finite Element Analysis of Doweled Joints for Airport Pavements.” *Transportation Research Record*, 1853, 100-109.

[46] Klien, C. and Hurlbut, C.S. (1993). Manual of Mineralogy (21st Ed.,) *John Wiley & Sons*, New York.

[47] Knudsen D., Peterson G.A. and Pratt P.F. (1982). “Lithium, sodium and potassium.” In: Page A.L. (ed.,) *Methods of Soil Analysis*, Part 2. American Society of Agronomy, Madison, WI, 229–231.

[48] Kuo, C., and Huang, C. (2006). “Three-dimensional pavement analysis with nonlinear subgrade materials.” *Journal of Materials in Civil Engineering*, 18(4), 537-54.

[49] Lambe, T. W. and Whitman. R. V. (2000). “Soil mechanics.” John Wiley & Sons, New York.

[50] Brown. G. (1961). “The X-ray identification and crystal structures of clay minerals." *Clay Minerals Group*, Mineralogical Society, London.

[51] Linker, R., Shmulevich, I., Kenny, A., and Shaviv, A. (2005). “Soil identification and chemometrics for direct determination of nitrate in soils using FTIR-ATR mid-infrared spectroscopy.” *Chemosphere*, 61 (5), 652-658.

[52] Little, D. N. (1995). “Stabilization of pavement sub-grades and base courses with lime.” Technical report made for Lime association of Texas.

[53] Little, D. N., Males, E. H., Prusinski, J.R. and Stewart, B. (2000). “Cementitious Stabilization.” *79th Millennium Rep. Series*, Transportation Research Board.

[54] Little, D. N. (1999). “Evaluation of Structural Properties of Lime Stabilized Soils and Aggregates, Vol. I. Summary of Findings.” National Lime Association Publication.

[55] Marshall, A. B. and Frank, P. A. (2007). “Extending durability of lime modified clay subgrades with cement stabilization.” *Geotechnical Special Publication*, Proceedings of Sessions of Geo-Denver Soil Improvement, 172.

[56] MacKay, M. and Emery, J. (1994). “Stabilization and solidification of contaminated soils and sludges using cementitious systems selected case histories.” *Transportation Research Record*, 1458 (12), 67-72.

[57] McCallister, L.D. (1990). “The Effects of Leaching on Lime-Treated Expansive Clay.” Thesis submitted to The University of Texas at Arlington, Arlington, Texas, in partial fulfillment for the doctorate degree.

[58] Mitchell, J. K. and Soga, K. (2005). “Fundamentals of soil behavior.” 3rd Ed., Wiley, New York.

[59] Moore, D. M. and Reynolds, R.C. Jr. (1989). “X-Ray Diffraction and the Identification and Analysis of Clay Minerals.”, Oxford University Press, New York, 332-339.

[60] Mosley, M. P. (2004). “Ground Improvement.” 2nd ed, Spon Press, New York.

[61] Murray, H. H. (1999). “Applied clay mineralogy today and tomorrow.”, *Clay Minerals* 34, 39-49.

[62] Narasimha Rao, S., Subba Rao, K. V. and Rajasekaran, G. (1993). “Microstructure Studies of Lime Treated Marine Clays.” Proc. 12th International Conference on *Offshore Mechanics and Arctic Engineering*, 1, 425-431.

[63] Nazzal, M. D., Abu-Farsakh, M. Y., and Mohammad, L. N. (2006). "Numerical Analyses of Geogrid Reinforcement of Flexible Pavements." *Proceedings of GeoCongress* , Georgia, USA.

[64] NCHRP 1-37A Project. (2004). “Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures.” National Cooperative Highway Research Program Final Report for NCHRP 1-37A Project. http://www.NCHRP 1-37A Designdesignguide.com.

[65] Nelson, J.D. and Miller, J.D. (1992). “Expansive Soils: Problems And Practice In Foundation and Pavement Engineering.” John Wiley, New York.

[66] Petry, T. M. and Armstrong, J. C. (1989). “Stabilization of Expansive Soils.” *Transportation Research Record*, 1219, 103-112.

[67] Poh, H. Y., Ghataora, G. S., and Ghazireh, N. (2006). “Soil Stabilization Using Basic Oxygen Steel Slag Fines.” *Journal of Materials in Civil Engineering*, 18 (2), 229-240.

[68] Porbaha, A. and Puppala, A. J. (2003). “In Situ Techniques for Quality Assurance of Deep Mixed Columns.” Proceedings of the Third International Conference (ASCE).

[69] Potts, D. M. and Zdravkovic, L. (1999). “Finite element analysis in geotechnical Engineering – Application.” Thomas Telford Ltd.

[70] Puppala, A. J., Punthutaecha, K. and Vanapalli, S. K. (2006). "Soil-water characteristic curves of stabilized expansive soils." *Journal of Geotechnical and Geoenvironmental Engineering*, 132 (6), 736-751.

[71] Puppala, A.J., Griffin, J.A., Hoyos, L.R. and Chomtid, S. (2004). “Studies on Sulfate-Resistant Cement Stabilization Methods to Address Sulfate-Induced Soil Heave.” *Journal of Geotechnical and Geoenvironmental Engineering.*

[72] Puppala, A.J., Wattanasanticharoen, E. and Punthutaecha, K. (2003). “Experimental Evaluations of Stabilization Methods for Sulphate-rich Expansive Soils.” *Ground Improvement*, 7 (1), 25-35.

[73] Puppala, A. J., Viyanant, C., Kruzic, A. P. and Perrin, L. (2002). “Evaluation of a modified soluble sulfate determination method for fine-grained cohesive soils.” *Geotechnical Testing Journal*, 25 (1), 85-94.

[74] Quirk, J. P. (1955). “Significance of surface areas calculated from water vapor sorption isotherms by use of the B. E. T. equation.” *Soil Science Journal*, 80, 423-430.

[75] Rao, S.M., Reddy, B.V.V. and Muttharam, M. (2001). “The impact of cyclic wetting and drying on the swelling behavior of stabilized expansive soils.” *Engineering Geology*, 60 (4), 223-233.

[76] Rhoades, J.D. (1982). "Cation exchange capacity." Edited by A. L. Page, *Methods of soil analysis* 2, Chemical and microbiological properties. SSSA, Madison, Wisconsin, USA, 149-157.

[77] Rogers, L. E. and Wright, S. G. (1986). “The effects of wetting and drying on the long-term shear strength parameters for compacted Beaumont clay.” Center for Transportation Research Bureau of Eng. Research, The University of Texas at Austin.

[78] Rollings Jr., R.S., Burkes, J.P. and Rollings, M.P. (1999). “Sulfate attack on cement-stabilized sand.” *Journal of Geotechnical and Geoenvironmental Engineering*, 125 (5), 364-372.

[79] Romanoschi, S., Hossain, M. and Gisi, M. (2004). “Accelerated Pavement Testing Evaluation of Stabilized Foundation Layer Materials for Midwest Flexible Pavements.” *Transportation Research Record,* 1896, 199-207. Transportation Research Board, Washington, DC.

[80] Schulze, D. G. (2002). “An introduction to soil clay mineralogy.” Edited by J. B. Dixon and D. G. Schulze., In *Soil Mineralogy with Environmental Applications*. SSSA, Madison, WI, 1-35.

[81] Shahin, M. A., Maier, H. R. and Jaksa, M. B. (2002). "Predicting Settlement of Shallow Foundations using Neural Networks." *Journal of Geotechnical and Geoenvironmental Engineering*, 128 (9), 785-793.

[82] Sherwood, P. T. (1995). "Soil Stabilization with Cement and lime: State of the Art-review." HMSO, London.

[83] Smith, D. K., Johnson, G. G. Jr. and Raud, C. O. (1986). "Clay mineral analysis by automated powder diffraction analysis using the whole diffraction pattern." *Advances in X-ray Analysis*, 29, 217-224

[84] Sridharan, A., Rao, S. M. and Murthy, N. S. (1988). "Liquid limit of Kaolinite soils." *Geotechnique* 38, 191-198.

[85] Stewart, W.M. and Hossner, L.R. (2001). "Factors affecting the ratio of cation exchange capacity to clay content in lignite overburden." *Journal of Environmental Quality*, 30 (4), 1143-1149.

[86] Suarez, D.L. and Frenkel. H. (1981). "Cation Release from Sodium and Calcium-Saturated Clay-Size Fractions." *Journal of Soil Science Society of America*, 45, 716-721.

[87] Tayabji, S. D., Nussbaum, P. J. and Ciolko, A. T. (1982). "Evaluation of Heavily Loaded Cement-Stabilized Bases." *Transportation Research Record*, TRB, National Research Council, Washington, D.C., 839, 6-11.

[88] Texas Precipitation Records: http://web2.airmail.net/danb1/annualrainfall.htm.

[89] Thompson, M. L. and Ukrainczyk, L. (2002). "Micas" Edited by J.B. Dixon and D.G. Schulze, *Soil mineralogy with environmental applications*, SSSA, Madison, WI, 431-466.

[90] Thompson, M. R. and Dempsey, B. J. (1976). "Evaluation of freeze-thaw durability of stabilized materials." *Transportation Research Record*, 612, 62-70.

[91] Tutumluer, E. and Kim, M. (2007). "Considerations for nonlinear analyses of pavement foundation geomaterials in the finite element modeling of flexible pavements." *Geotechnical Special Publication*, Analysis of Asphalt Pavement Materials and Systems: Emerging Methods, 176, 133-140.

[92] Ufzan, J. (1985). "Characterization of granular material." *Transportation Research Record*, TRB, National Research Council, Washington D.C., 1022, 52-59.

[93] Velde, B. (1995). "Composition and mineralogy of clay minerals." Edited by Velde, B., *Origin and mineralogy of clays,* New York, 8–42.

[94] Wanyan, Y., Abdallah, I., Sabnis, A. and Nazarian, S. (2008). "An Expert System for Design of Low-Volume Roads on High-Plasticity Clay Subgrade." *TxDOT Research Report 0-5430-1*, The University of Texas at El Paso, El Paso, TX.

[95] Wattanasanticharoen, E. (2004). "Experimental studies to address volume change behaviors of chemically treated sulfate bearing soils." Submitted to University of Texas, Arlington, TX, in partial fulfillment for the degree of Doctor of Philosophy.

[96] White, D.J., Harrington, D. and Thomas, Z. (2005). "Fly Ash Soil Stabilization for Non-Uniform Subgrade Soils, Volume I: Engineering Properties and Construction Guidelines." *Iowa Highway Research Board Report: IHRB Project TR-461*; FHWA Project 4.

[97] Whittig, L. D. and Allardice, W. R. (1986). "X-Ray diffraction techniques.", In A.Klute, (ed.,) Methods of Soil Analysis, Part 1—Physical and Mineralogical Methods, 2nd ed., *American Society of Agronomy*, Madison, Wisconsin, 331-362.

[98] Wiseman, G., Komornik, A. and Greenstein, J. (1985). "Experience with Roads and Buildings on Expansive Clays." *Transportation Research Record* 1032, 60-67.

[99] Wunderlich, (1990). http://www.osti.gov/energycitations/product.biblio.jsp?osti_id=6805908

[100] Yan, S. and Jin, P. (2004). “A study on the correlation relationships between smectite contents and spectral absorption indices of swelling soils.” *International Geoscience and Remote Sensing Symposium (IGARSS)*, 7, 4674 -4677.

[101] Yong, R N., Elmonayera, D. S. and Chong, T. S. (1985). “The Effects of Leaching on the Integrity of Natural Clay.” *Engineering Geology*, 21, 279-299.

[102] Yukselen, Y. and Kaya, A. (2006). “Prediction of cation exchange capacity from soil index properties.” *Clay Minerals*, 41(4), 827-837.

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