ENGINEERING BEHAVIOR OF LIME TREATED HIGH SULFATE SOILS

By

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ABSTRACT

ENGINEERING BEHAVIOR OF LIME TREATED

HIGH SULFATE SOILS

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Regardless of increased knowledge and awareness of sulfate heave, the Texas Department of Transportation (TxDOT) continues to experience pavement failures during and immediately after construction on transportation infrastructures intended to last at least 20 years. Failures are predominantly observed in sites where high sulfate soils of 8,000 ppm or higher levels. Many of these failures are attributed to sulfate-induced heave where an expansive mineral called Ettringite is formed from calcium-based stabilizers (lime or cement) reacting with water, clay, and sulfates. The reactions responsible for Ettringite formation are extremely complex which make generalizations concerning the rate and magnitude of heaving in different treated soils impossible.

The main objective of this research is to advance the state of knowledge of the engineering behaviors of lime treated high sulfate soils. In this research several tests have been identified to understand the change in volume behavior (swell and shrinkage strains) of lime treated high sulfate soils under two different moisture conditions and different mellowing periods to see the effects on swell strains reduction.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1

INTRODUCTION

1.1 Introduction

Expansive soils undergo large volume changes when subjected to moisture hydration and experience shrinkage strains when subjected to drying. Both swell and shrink volume changes depend on several factors including type and amount of clay minerals, moisture content, dry density, soil structure, confining pressure and climate (Chen, 1988; Nelson and Miller, 1992). These volume changes eventually cause severe damage to structures including pavement infrastructure (Chen, 1988; Nelson and Miller, 1992). Chemical stabilization methods are widely used in the field to control soil heaving (Nelson and Miller, 1992; Puppala et al. 2003). Among them, calcium-based stabilizers including lime and cement stabilizer treatments are widely used for expansive subsoil treatments due to their effectiveness to improve expansive soil properties and controlling volume changes (Chen, 1988; Hausmann, 1990).

Calcium-based stabilizers such as lime and cement have been used in the past to increase strength and decrease plasticity index (PI), swell and shrinkage strain potentials of expansive soils and thereby extend the life of structures built on those soils (Hausmann, 1990). Several recent studies have shown that the calcium-based stabilizer treatments of natural expansive soils

rich in sulfates would lead to a new heave distress problem instead of mitigating the problem (Mitchell, 1986; Hunter, 1988; Mitchell and Dermatas, 1992; Petry, 1994; Kota et al. 1996; Rollings et al. 1999; Puppala et al. 1999). This phenomenon has been termed “sulfate-induced heave” in the literature (Mitchell, 1986; Dermatas, 1988; Hawkins, 1988; Dermatas, 1995).

Sulfate-rich soils are typically found in arid and semiarid regions whereas natural and non-sulfate expansive soils are located in the southeastern and eastern United States. Sulfates are present in natural soils in Texas in various forms such as gypsum or calcium sulfate, sodium sulfate, and magnesium sulfate (Puppala, 2003).

When these soils are stabilized with calcium-based stabilizers such as lime and cement, sulfates in these soils react with calcium component of the stabilizer and free reactive alumina of soils to form highly expansive crystalline minerals, namely Ettringite and thaumasite (Sherwood, 1962; Mehta and Wong, 1982; Mitchell, 1986; Hunter, 1988). Thaumasite forms after Ettringite undergoes certain crystalline changes. These sulfate minerals expand considerably when subjected to hydration process. The mineral also expand due to continuous crystal growth. Both hydration reactions and crystal growth will result in a significant amount of heaving in the sulfate-rich soils.

Infrastructures including buildings, embankments, runways and highways built over lime and cement treated sulfate-bearing soils have been affected by this heave distress. Sulfate induced heave distresses are known to cause

serious problems throughout the state of Texas (Hunter, 1988). Figure 1.1 presents the sulfate-bearing soils in Texas. The soils in Dallas/Fort Worth area are part of Eagle Ford Formation which contains large quantities of gypsum and are expansive clays. The soils in these locations were found out to contain sulfate concentrations in the range of 4,000 to 27,800 ppm (Chen, 2005). The sulfate concentrations were reported as high as 35,000 ppm in Childress district, Texas (Si, 2008).

Though methods for stabilizing soils with low sulfate levels less than 6000 to 8000 ppm are better understood, there are still issues when stabilizing sulfate soils with sulfates exceeding 8000 ppm. The presence of high sulfate soils (sulfates more than 8000 ppm) throughout the state of Texas, including the Fort Worth and Dallas Cities, has resulted in pavement failures due to sulfate induced heave after the subsoils are treated with calcium based stabilizers.

The current mitigation practices for high sulfate soils include removing and replacing high sulfate soils, are prohibitively expensive and are not readily available. Hence Texas Department of Transportation (TxDOT) is exploring research means to explore other methods for treating these high sulfate soils. As a part of the research, the chemical reactions responsible for this heave problem at high sulfate levels need to be better understood in order to develop a better treatment method that results in a more stable pavement foundation system.

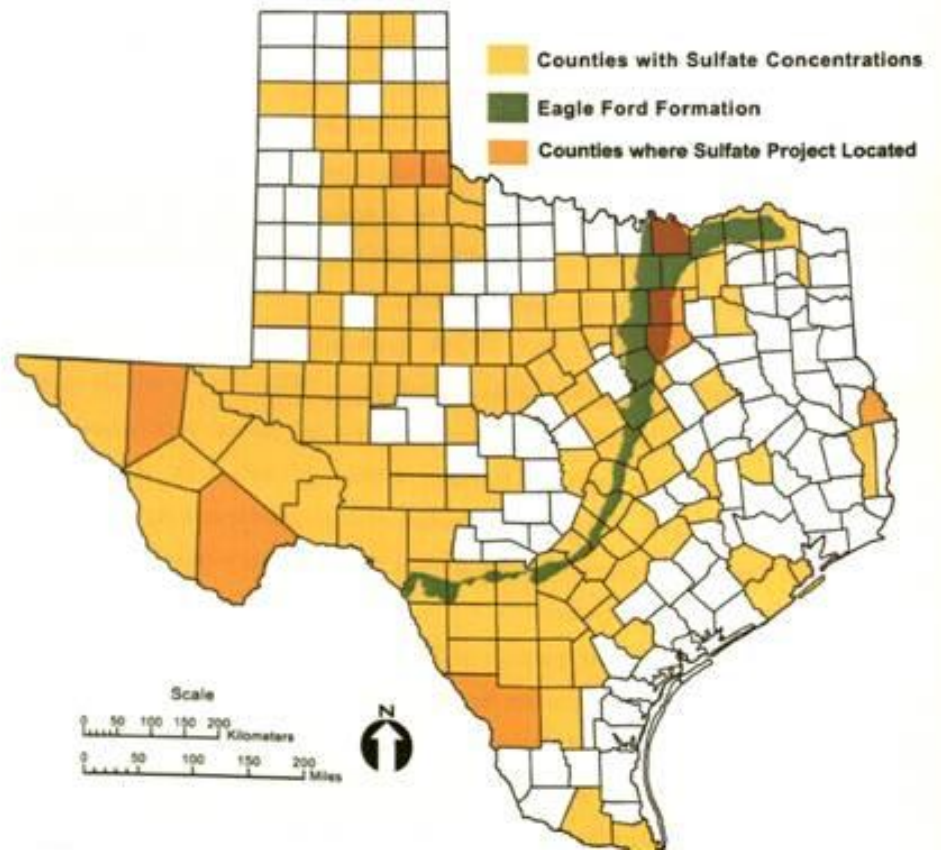


Figure 1.1 Location Map Showing Sulfate-bearing Soils in Texas
(Harris et al., 2004)

1.2 Research Objectives

Based on years of empirical data and multiple tests involving soils with varying sulfate contents, TxDOT guidelines indicate that sulfate levels below 3,000 ppm were not deleterious, sulfates between 3,000 and 8,000 ppm can be treated with techniques such as extended mellowing and provision of additional moisture and lime (Berger, et al. 2001; Harris et al. 2004). But above 8,000 ppm the soil needs to be removed and replaced with a select fill. Recent technical memorandums by TxDOT indicate that sulfate levels in excess of 20,000 ppm were successfully stabilized which further illustrates the need for understanding

the engineering behavior of lime treated high sulfate soils. Also, in the literature, there are studies that have shown effectiveness of stabilizing high sulfate soils with sulfates around 20,000 ppm using other types of stabilizers. These studies need to be reviewed along with other treatment methods as well as changing mellowing time periods to develop better treatments for high sulfate soils.

Overall, the main objectives of the present research study on high sulfate soils (sulfates more than 8000 ppm) are to:

1. Develop a fundamental understanding of variables such as soil type, sulfate levels, moisture levels and amount of lime dosage that induce heaving mechanism in soils.
2. Understand the change in volume behavior (swell and shrinkage) of lime treated high sulfate soils under two different moisture conditions.
3. See the effect of different mellowing periods on swell reduction.
4. Monitor the effect of increasing moisture content during mellowing on the volume change behaviors.
5. Establish general guidance for mitigation of high sulfate soils.

1.3 Research Variables

This research study was conducted at the University of Texas at Arlington (UTA), which was supported by TxDOT. The major variables of the investigation are soil types, moisture contents, density levels and soluble sulfate

levels. Two distinct moisture conditions simulating optimum and wet of optimum moisture conditions are considered.

1.4 Selection of Test Soils

The criterion for selection of test soils are as follows: clayey soils with soluble sulfates above 8,000 ppm and PI values ranging from 35 to 70. Six natural soils are collected from various TxDOT districts including Austin, Childress, Dallas and Paris. These soils are subjected to soluble sulfate measurement studies using TxDOT method (Tex-145-E Determining Sulfate Content in Soils - Colorimetric Method). Based on the test results large quantities of soils are sampled. Figure 1.2 presents Test Soil Locations.

The sulfate test results are presented in Table 1.1. In the current research soils with sulfate contents above 8,000 ppm are termed as high sulfate soils. Based on the sulfate contents the soils are classified into three groups: soils with sulfate contents less than 8,000 ppm, soils with sulfates amount between 15,000 ppm and 20,000 ppm, and soils with sulfate contents above 20,000 ppm. Among the six soils, three soils showed sulfate contents above 20,000 ppm (FM1417, Austin and Childress). Two of the six soils have sulfate levels below 8,000 ppm, (US-82 and Dallas). One of the six soils is a control soil with negligible sulfate contents (Riverside). For soils with sulfate contents below 8,000 ppm, additional amount of sulfates in the form of Gypsum was added to make them high sulfate soils.

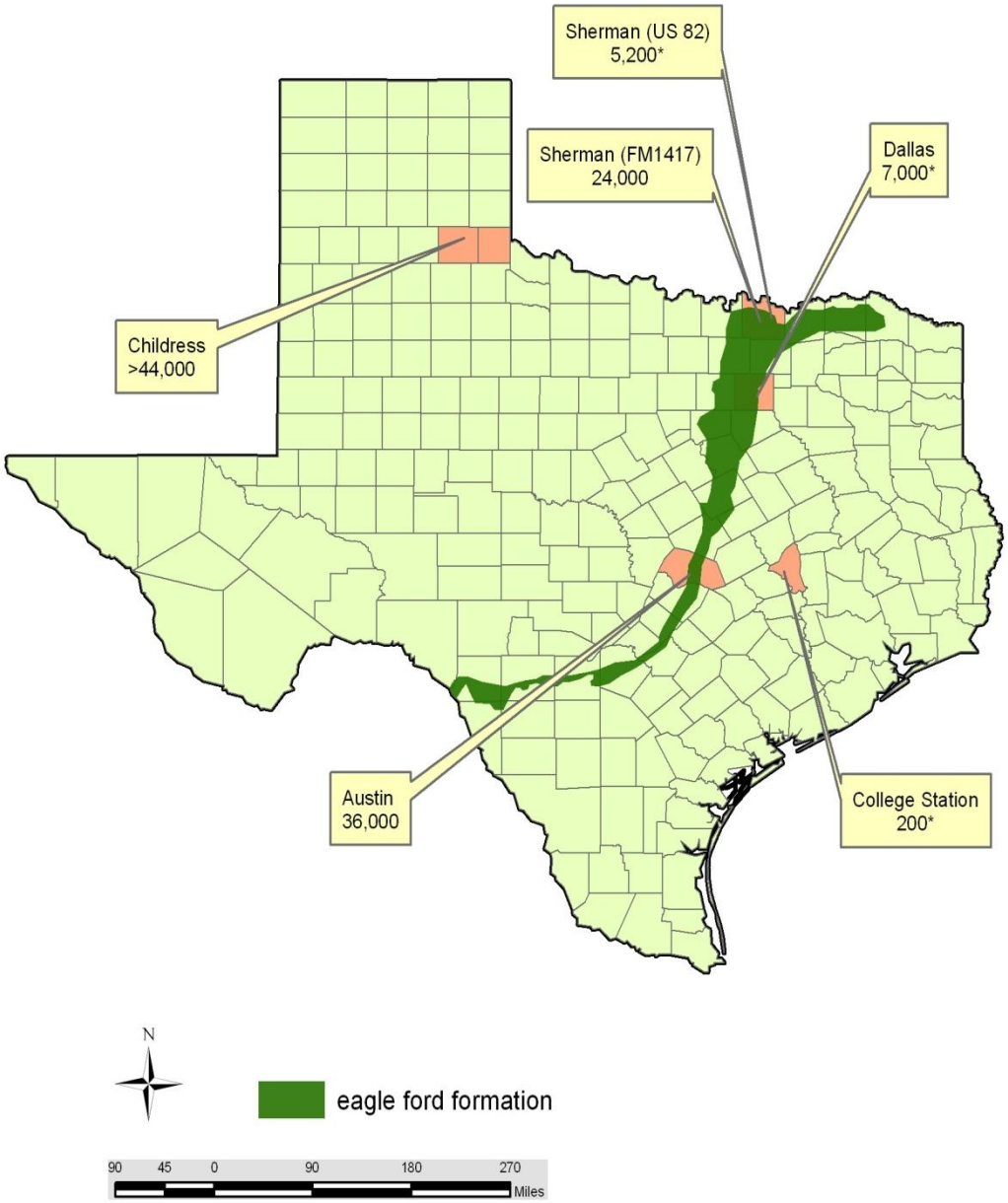


Figure 1.2 Texas Map Showing Test Soil Locations and sulfate amount (ppm)

Table 1.1 Test Soil Locations and Soluble Sulfate Contents of the Selected Soils

	Soil Location	Soluble Sulfates, ppm
Sulfate soils	US-82 (Sherman)	5,200*
	Dallas	7,000*
	FM1417 (Sherman)	24,000
	Austin	36,000
	Childress	>44,000
Control soil	Riverside (College Station)	200*

* Sulfates < 8,000 ppm, additional sulfates were added in the form of Gypsum

Second criteria in the selection of test soils are based on the PI values. Determination of Liquid limit, Plastic limit and Plasticity index of the soils is carried as per TxDOT procedures Tex-104-E, Tex-105-E, Tex-106-E methods, correspondingly. The Atterberg limits of all six soils are shown in Table 1.2. The soils are grouped into two categories based on the PI values. Soils with high PI values (PI> 35) and soils with low PI value (PI< 35) such as the control soil Riverside.

Table 1.2 Atterberg Limits of the Selected Soils

Soil Region	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)
Riverside	35	11	24
Childress	71	35	36
FM1417	72	30	42
Dallas	80	36	44
US-82	75	25	50
Austin	76	25	51

1.5 Thesis Organization

This thesis consists of five chapters: introduction (chapter 1), literature review (chapter 2), equipments used and testing procedures (chapter 3), test results and analysis (chapter 4) and conclusions, future research needs and recommendations (chapter 5).

Chapter 1 provides the introduction, research variables, selection of test soils, objective and thesis organization.

Chapter 2 provides a summary of recent papers, background of sulfate rich soils, sulfate material and heaving mechanisms, case studies on sulfate induced heave, lime treated soils mechanisms and the interaction between soil-lime-sulfate. The literature information on sulfate heave problems was acquired and included in this chapter.

Chapter 3 presents laboratory instrumentation, sample preparation, test methods, procedures, data analysis methods used.

Chapter 4 provides test results of physical, engineering and chemical tests on control and treated soils and potential means for the property variations. Physical tests include soil classification tests such as Atterberg limits and Hydrometer analysis. Engineering tests include Unconfined Compressive Strength (UCS), 3 Dimensional Swell and Volumetric Shrinkage tests. Sulfate measurement and lime dosage determination are the chemical tests covered in this thesis. The basic physical properties of natural soils were presented and

determined as per American Society for Testing Material (ASTM) standards and TxDOT methods and these results are presented in this chapter.

Chapter 5 provides a collective analysis, summary of test results and major conclusions of the present research study. Future research recommendations and directions were also included.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

The literature review presented in this chapter was collected from conventional library resources such as compendex, electronic search engines and various technical reports. The organization of the literature review is as follows; first an introduction to sulfate bearing soils is explained, followed by different sources of sulfates in soils. The heaving mechanisms involved in sulfate laden soils were discussed. Also, various case histories involving sulfate heaving and the mitigation methods to control these heaves are detailed, followed by threshold of problematic sulfate levels in soils.

2.2 Sulfate bearing soils

Sulfate bearing soils are found all across the United States. Gypsum is found to be the most occurring sulfate mineral in the western part of United States (Kota et al., 1996). Figure 2.1 shows the location of soils containing Gypsum and gypsum mines in the US. Calcium based stabilizers are typically used to treat expansive soils around the world. The most common calcium based stabilizers are cement and lime. Addition of cement and lime to soils improves strength and durability characteristics of the soils by the formation of

pozzolonic compounds. Soil-lime-cement reactions decrease the plasticity characteristics of the soils and make them non plastic. When soils containing sulfates are treated with calcium based stabilizers such as cement and lime,

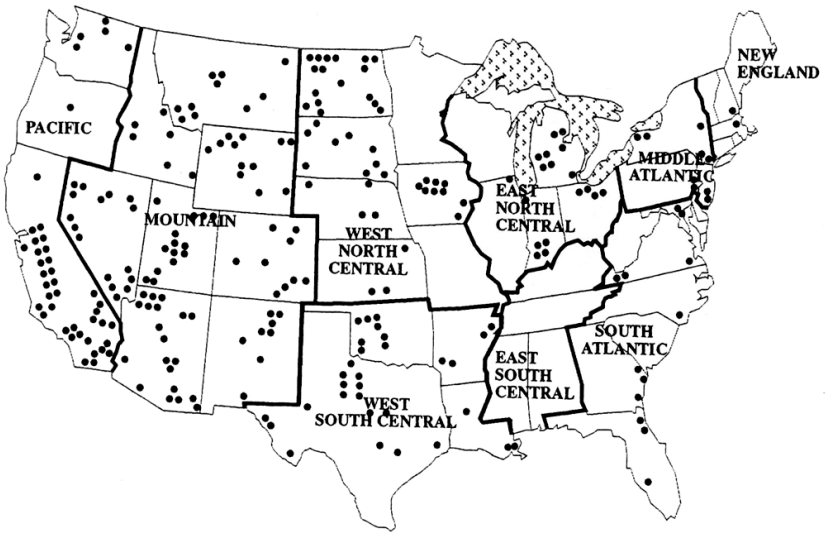


Figure 2.1 Location of soils containing Gypsum in United State
(Kota et al., 1996)

sulfate minerals in these soils react with calcium from stabilizers and alumina from soils to form expansive mineral called Ettringite (Sherwood, 1962; Mitchell, 1986; Hunter, 1988). Ettringite is highly expansive in nature and can swell more than 100% of its own volume. Under favorable conditions Ettringite converts into thaumasite, which is as expansive as Ettringite.

In 1962, Sherwood (Sherwood, 1962) reported problems with sulfates in lime and cement stabilization of soils. However, reports of sulfate-induced heave in subgrade soils received little attention until the mid 1980's. Formation of Ettringite was the cause of heaving in a case study from the southern United

States (Schlorholtz and Demiril, 1984). The term “Sulfate-induced distress” came in to light after Mitchell’s Terzaghi lecture in 1986. Various case studies of sulfate induced heave failures were reported across the nation (Mitchell, 1986; Hunter, 1988; Kota et al., 1996). Several infrastructure facilities, roads, highways and runways built over lime and cement stabilized soils experienced severe damage due to sulfate induced heave. In some cases, the cost of repairs exceeded the cost of stabilization (Hunter, 1988). Experimental studies conducted under controlled environments revealed that sulfate induced heave is more of a concern in clays than sands (Puppala et al., 2005). Also, it was concluded that lime-sulfate reactions are similar in both land and marine clays (Rajasekaran, 2005). Many sulfate induced heave cases were reported in the state of Texas (Perrin, 1992; Puppala et al., 2010). Burkart et al. identified certain geologic formations that possess high sulfates and that gypsum was the most common sulfate in Dallas area soils (Burkart et al., 1999). The most severe heaves in the Dallas/Fort Worth area are associated with the Eagle Ford formation. These soils were found to contain sulfate concentrations ranging from 4,000 ppm to 27,800 ppm (Chen et al., 2005). High sulfate concentrations of 35,540 ppm were reported in Childress district (Zhiming, 2008).

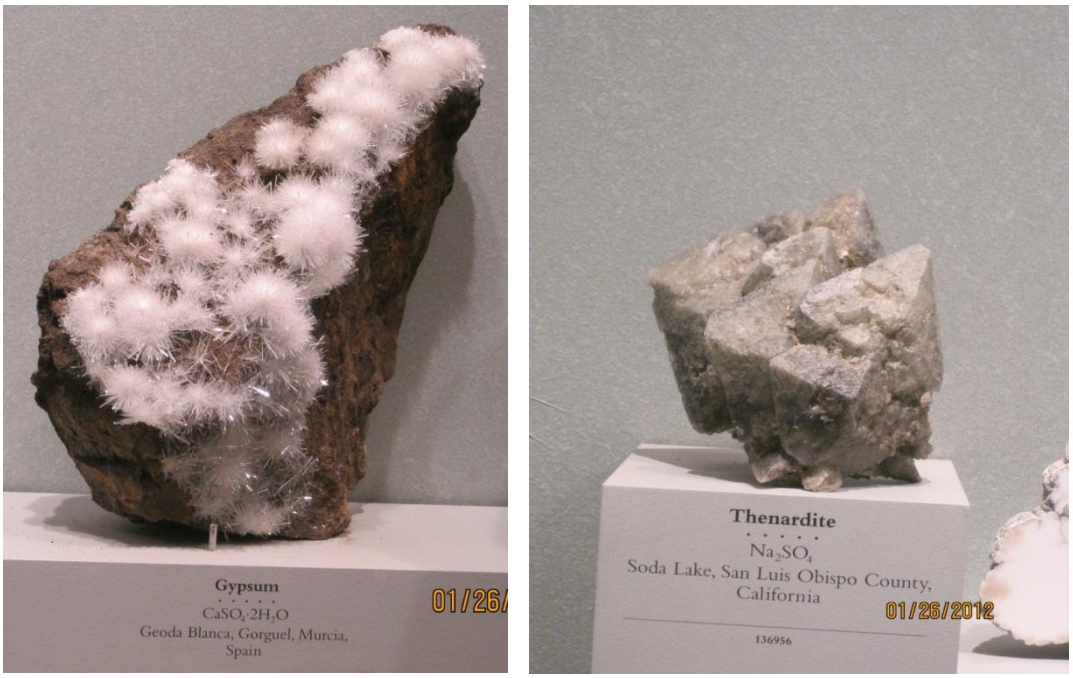
2.3 Sources of Sulfates in Soils

There are several sources of sulfates in soils, produced from a primary or secondary source. Primary sources can be defined as the direct sources of

sulfates in their natural form as sulfate bearing minerals such as Gypsum while the secondary sources are those that are not a direct source of sulfate but give out sulfates as a result of oxidation or other forms of chemical interactions. This following section describes these sources.

2.3.1 Primary Sources

The primary source of sulfates in Texas soils is Gypsum (CaSO4.2H2O). Sulfates are present in natural soils as calcium sulfate (Gypsum), sodium sulfate (Thenardite) and magnesium sulfate (Epsomite) (Puppala et al., 2003). Gypsum and Thenardite in their natural form are presented in Figure 2.2 (a) and (b).



(a) (b)

Figure 2.2 Various sources of sulfates (a) Calcium Sulfate (Gypsum), (b) Sodium Sulfate (Thenardite), (Photo credit to Nagasreenivasu, 2012 taken at National Museum of Natural History)

Natural sulfate minerals have different solubilities based on their dissolution into solution. Due to Gypsum's low solubility (2.58 gm/L) compared to both sodium sulfate (408 gm/L) and magnesium sulfate (260 gm/L) it acts as a steady source of sulfates in soils. The maximum solubility of gypsum occurs at 35°C - 50°C. Sulfates are present in higher concentration in clayey soils than in granular soils. This is fact can be attributed to the low permeability of clayey soils compared to granular soils which do not allow the dissolved sulfates to pass through them; these dissolved sulfates are easily leached out in the case of granular soils.

2.3.2 Secondary sources

Other sources of sulfates in soils are pyrites (sulfides), which on oxidation reaction are converted into soluble sulfates. During rainy season, sulfates in the top layers of the soil dissolve and move into the stabilized layers whereas during dry spell dissolved sulfates move upward due to evaporation into the top layers increasing their concentration (Dermatas, 1995). As previously discussed, the sulfates can be from secondary source. A forensic investigation study was conducted by Rollings et al. (1999) on the failure of Bush Road, Georgia. Extensive bumps and longitudinal cracking were observed after five months of construction on a pavement built on cement stabilized sand. The failure is attributed to the formation of Ettringite in the cement stabilized sand. While no sources of sulfate were found in the native soils, the investigation found that the water from a nearby well used for mixing cement

and compacting is the source of sulfate in the soil. Another example of secondary sulfate contamination is the sulfates transported by ground water.

It can be inferred from the above discussion that there are various sources of sulfates in soils, now it is important to understand how these sulfates present in the soils induce heaving. The following sections details the process of sulfate induced heaving along with the chemical reactions involved in the process.

2.4 Heaving Mechanisms

The formation of Ettringite and its subsequent growth was explained by Cohen (1983). According to him, there are two different swell mechanism associated with Ettringite formation and subsequent growth. They are crystal growth theory and swelling theory. The expansion theories presented in Cohen's paper are based on Ettringite formation and growth in cement concrete. Dermatas (1995) summarized the heaving mechanism in to two categories. The first is a topochemical mechanism where expansion is related to formation and/or oriented crystal growth of Ettringite (Ogawa and Roy, 1982). As per the crystal growth theory, aluminum, calcium and sulfates can concentrate around Ettringite nucleation sites and combine to form additional Ettringite. Ettringite crystal growth, pressure is exerted on the restraining media, when this pressure exceeds the confinement or overburden, swelling occurs.

The second is a thorough solution mechanism where expansion is related to swelling due to hydration.

2.4.1 Heaving due to Crystal Growth

Ogawa and Roy (1982) proposed a heave mechanism based on crystal growth. This results in a large nucleation rate which generates many small Ettringite crystals surrounding the aluminum-bearing particles. According to their theory, Ettringite forms around calcium aluminum sulfate particles in the early stages of cement hydration. This creates a reaction zone around every particle. In later stages with introduction of water into the system, the crystals assume needle shape around particles from which they are formed. When the adjacent reaction zones intersect they exert mutual pressure on each other leading to the swelling of the whole system. This mechanism is favored in high pH and sufficient availability of reactants. In case of soil-lime systems the soil matrix is less rigid and contains larger void spaces compared to the cement concrete system. This allows the accommodation of initial Ettringite in soil voids. With introduction of additional water in to the system in later stages, heaving occurs since the soil matrix can no longer accommodate the continuous Ettringite growth.

2.4.2 Heaving due to Hydration

Swelling due to hydration is proposed by Mehta (1973). He suggested that formation of Ettringite follows a thorough solution mechanism. Also, in the

presence of saturated $Ca(OH)_2$, the rate of hydration of aluminum decreases significantly. This causes the Ettringite to form gel-like and colloidal crystals. These colloidal crystals adsorb large quantities of water molecules due to their high surface area and net negative charge. Expansion of colloidal Ettringite is aided by external water supply. In the absence of lime long rod-like crystals are formed. These crystals have low surface area. They absorb lesser water molecules and hence no expansion is observed in this case. Colloidal or gel-like crystals are formed in high hydroxyl ion concentration whereas rod-like crystals are formed in low hydroxyl ion concentrations. Mehta and Wang (1982) conducted series of experiments and found that coarser Ettringite expands less than finer Ettringite. Scanning electron microscope studies revealed that in presence of lime Ettringite crystals are 1 micron in length and ¼ micron in width. In the absence of lime, longer (6 to 8 inches) and wider (1/2 to 1 micron) crystals are observed. He observed that the amount of expansion is associated with the amount of water adsorbed. The restraint offered by soil system is less compared to concrete system and hence even small swell pressures can cause expansions in soils.

The investigation of soils in Las Vegas, Nevada by Hunter (1989) identified 24% swell due to formation of Ettringite/thaumasite and he attributed the rest to an increase in voids from the soil initially being in such a heavily compacted state. Many investigators (Mitchell and Dermatas, 1992; Wild et al., 1999; Kota et al., 1996) have noted that decreased density may reduce swell

due to more void space to allow for the expansive minerals to form into. Harris et al., 2004 concluded that reduced swell is due to a combination of more voids and a faster reaction rate removing more of the sulfate from the system before compaction.

2.5 Case Studies

Case histories of sulfate heave were reported in the literature by various researchers (Chen et al., 2005; Zhiming, 2008; Puppala et al., 1999; and 2010). Sulfate induced failures of infrastructural facilities were observed in several locations throughout the state of Texas. Sulfate induced heave occurred in both lime and cement treated soils. Some of these studies are discussed in the subsequent sections. For more case studies and their details can be found in Puppala et al. (1999 and 2010).

2.5.1 US 82, Texas

The Texas Department of Transportation (TxDOT) observed heaving on a new construction project on U.S.82 (Chen et al., 2005). The pavement design included a 50 mm asphalt concrete over 300 mm flexible base followed by 200 mm lime treated subgrade. The heaving was observed in east side of the project while no heaving was observed in west side. Figure 2.3 depicts the observed heaving on east side. The soils in both sides have tan colored soils but east side soils contained sparkly bits of gypsum. The subgrade soil from this

project belongs to Eagle Ford formation. Soils in Dallas/ Fort Worth are rich in clay and gypsum.

A forensic investigation was initiated to determine the causes of heaving and possible solutions for future projects. Core samples of lime treated subgrade and raw subgrade were collected and sent for testing by Texas Transportation Institute (TTI). SEM analysis of the core samples confirmed the presence of long fibrous crystals. These are confirmed as Ettringite. These crystals were absent in raw subgrades. Further, chemical analysis was performed to find the unreacted sulfate which could cause heaving in future. In-situ conductivity and pH measurement tests were performed on soils from east side and west side. The pH values are higher in both these cases since the soils are lime treated. Conductivity of samples from east side is found to be higher than west side. Also it was concluded that a six fold increase in conductivity is observed with increase in depth. It was concluded that soluble gypsum seeps from the surface to depths increasing the conductivity. The soils in east side have sulfates in the range of 4,000-27,800 ppm. The soils in the west side contained low amounts of sulfates. The west side of the project is treated with 3 % of lime followed by 72 hr mellowing period and 3 % of lime application. No heaving was observed on west side. It was concluded from the study that conductivity measurement is a good indicator of sulfates in soils.

Figure 2.3 Heaved area on east bound side (Chen et al., 2005)

2.5.2 US 287, Childress, Texas

Extensive longitudinal cracking and severe ride roughness were observed on US 287 near Childress, Texas (Zhiming, 2008). The pavement section consisted of 76.2 mm Type D asphalt hot mix asphalt followed by a 254 mm fly ash stabilized crushed stone base underlain by a 178 mm flexible base and 228 mm hydrated lime treated base. The maximum distress was observed in the North Bound (NB). Two raw subgrade soil samples from North and South right of way (ROW) were sent to laboratory for testing. The soils in the NB ROW area were found to contain sulfates above 35,000 ppm. These soils are termed as high sulfate soils (HS). Soils in the South Bound ROW are called as low sulfate soils (LS). The soils in NB ROW are classified as fine grained soils whereas soils in SB ROW are classified as coarse grained soils. Higher clay

content in the NB soils is the cause of the swelling. The heave observed in the NB area is sulfate induced.

A series of laboratory testing was done to characterize the unmodified soils. Atterberg limits tests, bar linear shrinkage tests, conductivity and calorimetric tests, moisture susceptibility tests, free-free resonant column tests, and unconfined compressive strength tests were performed after 10 days of soaking. Various alternative stabilizer treatments such as hydrated lime, Portland cement, lime and fly ash, lime and slag are conducted. Property enhancements with addition of stabilizers are studied. It was found that a combination of lime and slag seemed to be the better choice for soils with moderate sulfate contents. The combination of lime and fly ash is most suitable for high sulfate soils in terms of swelling and retained unconfined compressive strength. Present study recommended replacing the lime-treated subgrade with select fill or reworking with the stabilizers.

2.5.3 Sulfate Attack on a Tunnel Shotcrete Liner, Dallas, Texas

Cracking and water leakage problems were observed in a tunnel shotcrete liner in Dallas, Texas (Puppala et al., 2010). The tunnel was found on lime stone on top of Eagle Ford geological formation. On careful inspection, a white powder like material and gel like substance are found on the shotcrete liner. Samples of powder material and rock core samples from distress locations are collected and sent to University of Texas at Arlington (UTA)

geotechnical laboratories. These were collected from low distress regions (LDR), medium distress regions (MDR) and high distress regions (HDR). These samples are subjected to extensive mineralogical testing by XRPD and EDAX studies. Rock core samples were subjected to unconfined compressive strength (UCS), indirect tensile strength (ITS) and unconsolidated undrained (UU) tests. A photograph of the distress region is shown in Figure 2.4.

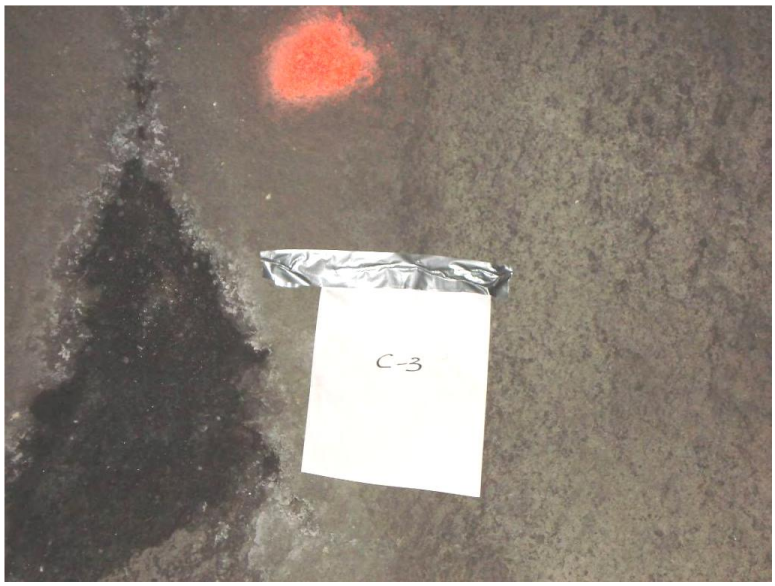


Figure 2.4 Distress region on tunnel lining (C-3)

Mineralogical studies of all the samples indicated the presence of Ettringite and Thaumasite. The powder samples collected from the field and core samples indicated presence of calcium, alumina, silica and sulfates. High moisture contents in order of 15 % are measured in high distress regions and 4 % are measured in low distress regions. Sulfate contents less than 1500 ppm

and greater than 2000 ppm are observed in LDR and HDR areas. UCS testing of the rock samples indicated the decrease in UCS value with increasing distress magnitude. Similar trends are observed with UU tests and ITS tests. Presence of sulfates influenced the rock properties adversely. It was observed that presence of sulfates and continuous moisture leakage from the joints can compromise the tunnel safety. It was concluded that continuous monitoring of the tunnel heave needs to be done. Also use of sulfate resistant Type V cement as shotcrete material is suggested. The necessity for incorporating loss of strength of rock cores with sulfates needs to be included in geotechnical design.

2.6 Threshold Sulfate Levels

One important question that is often sought by practitioners is the threshold of problematic sulfate levels in soils at which sulfate heave problems will be a concern. In the literature, these levels are reported to range between 1,500 ppm to 5,000 ppm and in some cases close to 10,000 ppm for different types of chemical treatments (Puppala et al. 2005). Establishment of such levels based on the database of various case studies is not appropriate as the soil composition and environmental conditions are different in each case study and findings or observations from one particular study may not be valid at other locations. However, studies are ongoing to determine sulfate threshold levels based on mineralogy and geological depositional environments (Adams et al. 2008). A recently completed National Science Foundation (NSF) sponsored

study conducted by the research team at UT Arlington showed that the problematic levels vary for cement and lime treatments and their dosage levels (Puppala et al., 2005).

2.7 Heave Mitigation Methods

Various stabilization methods have been developed to treat expansive soils (Hausmann, 1990). Among these methods chemical stabilization and mechanical stabilization are the most widely used techniques. Chemical stabilization of soils by calcium based stabilizers (lime and cement) gained widespread acceptance by the construction industry. It was observed that, when soils containing dissolved sulfates in natural form are treated with calcium based stabilizers sulfate induced heave occurred in those soils (Mitchell, 1986, Hunter, 1988, Puppala et al., 1999). It was suggested that lime or cement treatment of sulfate laden soils should be done carefully or to be avoided completely (Mitchell, 1986). Alternative stabilization methods for sulfate bearing soils have been proposed by various researchers (Puppala et al., 2004). They are double application of lime, treatment with low-calcium based stabilizers, use of sulfate resistant cements, use of geosynthetics, and several other methods. These are summarized in the following sections.

2.7.1 Double Application of Lime

Double application of lime to treat sulfate soils is based on the assumption that first application of lime allows formation and expansion of

Ettringite, whereas second application of lime accelerates the formation of pozzolonic compounds. These pozzolonic compounds bind the soil particles and improve soil strength. The time gap between the first and second application of lime is an important factor. If the time duration between successive applications of lime is less, the soil sulfates will not get dissolved completely leading to the formation of low-sulfate form of calcium-aluminum-sulfate-hydrate (CASH) compounds. These compounds in later stages get converted in to Ettringite with release of sulfates by rain or oxidation of sulfides. A minimum of three days of curing is required between successive applications of lime. Adding more and more lime does not ensure heave arrest, since additional supply of sulfates by leaching and oxidation of pyrites is possible. In such cases heaving problem is still aggravated. It was observed by Pat Harris et al. (2004) that double application of lime resulted in more heaving than single application. It was concluded that double application of lime is ineffective in soils with high sulfate contents and soils containing sulfides (pyrites). Soils with soluble sulfates up to 7,000 ppm can be effectively stabilized with double application of lime (Kota et al., 1996). Double application technique can be used in low sulfate soils provided soils do not contain sulfides.

In addition to double application of lime, forcing the formation of deleterious compounds before compaction is another option (Berger et al., 2001; Harris et al., 2004). This is achieved by providing adequate mellowing period. Mellowing period as little as 24 hrs and as much as 7 days depending

on the soluble sulfates showed good results. If these compounds form during mellowing period before placement and compaction, the risk of heave is minimized. Adding as much water as possible allows the dissolution of soil sulfates and formation of Ettringite in the initial stages. This prevents the formation of hot spots or concentrated Ettringite growth nuclei. In later stages, due to unavailability of sulfates, Ettringite formation and growth are inhibited. This condition is valid as long as there is no ingress of sulfates from outside source. Adding 3 to 5 percentage of water above optimum moisture content found to give satisfactory results. This technique has been applied with some success in north and south Texas where sulfates in excess of 20,000 ppm were successfully stabilized (Hilbrich, personal comm.).

2.7.2 Low-Calcium Based Stabilizers

When soils containing sulfates are treated with calcium based stabilizers, sulfate induced heave is observed. The most common calcium based stabilizers are cement and lime. Higher the calcium, higher the heave associated with it. The availability of calcium can be limited by using low-calcium stabilizers. Fly ash is one of the calcium-based stabilizers. Fly ash is classified as class C fly ash and class F as per ASTM. The main difference between class C fly ash and class F fly ash is the availability free calcium. Percentage of free calcium is higher in class C fly ash compared to class F fly ash. This is the reason why lime need to be added to class F fly ash for soil stabilization. The factors which affect the reaction rate of fly ash in soils are soil type, specific surface areas of

soils, chemical composition of fly ash admixtures and dosage of fly ash. It was concluded that use of fly ash decrease the swell and shrinkage strains of the soils by decreasing plasticity index of soil (Puppala et al., 2000)

Treatment of soils with low-calcium based stabilizers such as class C fly ash, class F fly ash is attempted by various researchers (Wang et al., 2004; Puppala et al., 2006; Solanki et al., 2009, McCarthy et al., 2009). Treatment of soils with fly ash initiates short-term and long-term reactions. The short-term reactions include flocculation and agglomeration with increased ionic exchange. The long-term reactions include strength enhancements.

In a research study conducted at UTA, three different types of fly ashes are used to stabilize sulfate rich expansive soils from DFW (Dallas Fort Worth) area and Arlington area. According to Unified soil classification system (USCS), these soils are classified as low compressibility clays (CL) with medium plasticity index values. Stabilizer dosage levels used in the study are 0, 10, 15, and 20 % by dry weight of the soil. Three-dimensional free swell, shrinkage strain and pressure swell tests were conducted on treated soils. Based on three-dimensional swell test results, class F fly ash treatment is considered a better treatment method compared to bottom ash treatment. It was concluded that Class F fly ash provided maximum enhancements to the soil properties. Heave severity is reduced from high level to medium or low severity level by

class F fly ash treatment. Maximum property enhancements are observed in combined stabilizer treatment with class F fly ash and nylon fibers.

A similar study conducted at University of Oklahoma, using class C fly ash (CFA) reported the same observation (Solanki et. al., 2009). The soils studied are sulfate rich lean clays from Oklahoma. The soils are treated with 5, 10, & 15 % dosage levels of class C fly ash, lime and cement kiln dust (CKD). The evaluation of stabilizer treatment was done on the long-term and short-term basis. Several engineering tests including resilient modulus (M_R), modulus of elasticity (ME), unconfined compressive strength (UCS), tube suction (TST) and three-dimensional swell tests were performed. Mineralogical tests were conducted to identify the micro-structural developments. Addition of class C fly ash improved MR, UCS, ME after 28 days of curing. The moisture susceptibility is considerably decreased by application of CFA. Overall CFA showed better short-term and long-term performance compared to other stabilizers used in the present study. A recent research study conducted by McCarthy et al. (2009) in UK on fly ash treatment of high sulfate soils yielded satisfactory results. He concluded that swelling reduced by increasing fly ash contents and decreasing lime content. Also use of coarser fly ash is found to be more effective than finer fly ash. Further, he recommended the use of 3 % lime, 10-15 % mass fly ash, allowance of one day mellowing period before application of fly ash, and coarser fly ash.

2.7.3 Use of Sulfate Resistant Cements

Cement has been used as a stabilizing material for expansive soils since many years. Soils stabilized with cement are called as “Cement-stabilized” soils (Hausmann, 1990). Different types of cements are available in the market to meet physical and chemical requirements for various applications. Type I cement is used for general RCC structures, Type II for soils with low sulfate contents, Type III for high strength in early stages, Type IV for dam structures and Type V for high sulfate soils. Cement treatment of soils provide strength enhancements and plasticity reductions through flocculation, cementation and pozzolonic reactions. In sulfate resistant cement stabilization, the series of reactions involved are similar to ordinary cement stabilization. In general tricalcium aluminate (C3A) formed in ordinary Portland cement concrete material provides alumina required for Ettringite formation (Rollings et al., 1999). Sulfate resistant cements inhibit formation of Ettringite by limiting the availability of reactive alumina. The tricalcium aluminate (C3A) concentrations in sulfate resistant cements are low; hence they reduce the amount of alumina available in concrete material to react with soil sulfates to form Ettringite.

Sulfate resistant cements fail in cases where alumina is introduced in to the soil matrix by contamination. It was observed in Halloman Air force Base, NM case study that although sulfate resistant cements are used in recycled

concrete, they became ineffective through soils contamination during mixing and stock piling resulting Ettringite formation and subsequent heaving.

Studies on sulfate resistant cements in stabilization of sulfate soils are carried by various researchers (Griffin, 2001; Puppala et al., 2004). In an experimental study conducted by Puppala et al. (2004), Type I/II and Type V stabilizers are used to treat soils with sulfate levels varying from 1,000- 5,000 ppm and above. The stabilizer dosage levels are established as 5 and 10 % by dry weight of the soil. Two moisture content levels, wet of optimum and optimum moisture content. Strength, stiffness, swell and shrinkage strain tests were carried out on the treated soils. In both treatments, plasticity indices reduced to an insignificant value. Treated soil samples compacted at wet of optimum yielded higher strength and lesser swell properties. Higher moisture contents at wet of optimum facilitated stronger hydration reactions between cement and soil particles. The strength enhancements due to cement treatment are attributed to the formation of pozzolonic compounds (Tobermorite, Jusite and Prehnite) and flocculation. XRD tests results confirmed that both treatments are effective in sulfate induced heave mitigation. Overall both treatments yielded similar results since the soils involved in the study are low to medium sulfate. A similar study is conducted by Deepti et al. (2007) at University of Texas, Arlington. In this study, stabilization potentials of class F fly ash + Type V cement, GGBFS, Type V sulfate resistant cement, lime + fibers and GGBFS are studied on soils with medium sulfate levels. It was found that all the

stabilizers improved the soil properties by reducing liquid limits, plasticity indices, swell and shrinkage strains, and increased UCS and MR values. Based on the ranking analysis, it was concluded that Type V cement showed the best performance among the other stabilizers.

Sarkar and Little (1998) successfully stabilized a crushed concrete base parking lot that was contaminated with sulfates using a Type V cement and Class C fly ash. However, 19% stabilizer was required to stabilize the base due to very high moisture content in the degraded base.

2.7.4 Use of Geosynthetics

A combination of geotextiles and geogrid can be used as an alternative for high plasticity soils with soluble sulfates. Geotextile acts as a separator between pavement and natural subgrade. The purpose of geotextile is to separate layers of pavement from the natural subgrade and to prevent migration of clay into upper layers due to traffic loads. Geogrids were successfully used on stabilization of high sulfate soils. Mechanical stabilization by geogrid instead of chemical stabilization can be an alternative for high sulfate soils (Zhiming, 2008).

2.7.5 Several other methods

The other methods include stabilizing the top portion of select fill, compacting to lower densities and use of polymeric fibers with soil. Soils with no soluble sulfates can be brought from different locations to use as select fill

material over the subgrade. Stabilizing the selected fill material is equivalent to stabilizing the natural subgrade. Proper care should be taken to avoid migration of sulfates in to the select fill material, failure of which again leads to sulfate induced heave. Compaction of stabilizing layer to lower densities is another option available. Compacting at lower densities allows more void spaces in soil matrix. This allows more room for the growth of Ettringite and its overall expansion.

2.8 Summary

Based on the previous literature review, it can be determined that sulfate induced heave is inevitable when soils containing considerable amounts of sulfates are treated with calcium based stabilizers. Cement and lime treatments proved to be effective in treating soils with low sulfate levels, whereas both the treatments failed in treating high sulfate soils. It was also understood that exact measurement of soil sulfates in in-situ conditions is not possible given there is a chance of sulfate migration from external sources.

This research focuses on the impact of lime treatment on strength, swell and shrinkage behaviors of high sulfate soils (> 8,000 ppm). Roles and impacts of mellowing periods and increased moisture contents are also addressed.

CHAPTER 3

EQUIPMENTS USED AND TESTING PROCEDURES

3.1 Introduction

Chapter 3 presents laboratory instrumentation, sample preparation, test methods, procedures, data analysis methods used. The laboratory testing program was designed to determine the properties relating to volume change behavior of high sulfate soil samples taken from six different sites, which are located in Austin, Childress, College Station, Dallas and two sites in Sherman all in Texas. The investigational program includes chemical tests, basic soil properties tests and engineering tests on the soils from previously stated locations. A summary of the laboratory procedures and equipments used are presented in this chapter.

3.2 Basic Soil Properties Tests

3.1.1 Atterberg Limits

Determination of Liquid limit (LL), Plastic limit (PL) and Plasticity index (PI) of the soils is carried as per TxDOT procedures Tex-104-E, Tex-105-E, Tex-106-E correspondingly. These tests were conducted in order to determine the plasticity properties of the soils. Upon addition of water the state of soil proceeds from dry, semisolid, plastic and finally to liquid states. The water

contents 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. Figure 3.1 shows the apparatus used for Atterberg limits.

Figure 3.1 Atterberg limits apparatus

These Atterberg limits are very important to show a relationship between the shrink-swell potential of the soils to their relevant plasticity indices. The numerical difference between LL and PL values is known as plasticity index (PI) and this property is generally used to characterize the plasticity nature of the soil and its expansive potential. The water content of the samples during tests are measured using microwave drying method based on the repeatable data as reported by Hagerty et al. (1990).

3.1.2 Specific Gravity

Soil consists of an accumulation of particles, which may be of a single mineral type or more often, a mixture of a number of mineral types (Head, 1980). This means that different soil types have different specific gravity values. In this research, the standard test as per ASTM D 854 was conducted to determine the specific gravity of the selected soils. Specific gravity results are presented in the next chapters.

3.1.3 Hydrometer analysis

Hydrometer Analysis was carried out as per ASTM D422. The procedure involved taking 50 gram of the oven dried portion that passed No. 200 sieve and mixed with a solution containing a 4% deflocculating agent (Sodium Hexametaphosphate) and soaking for about 8 to 12 hours. The prepared soil was thoroughly mixed in a mixer cup and all the soil solids inside the mixing cup were transferred to a 1000 cc graduated cylinder and filled to mark using distilled water.

The hydrometer readings were recorded at cumulative time of 0.25 min., 0.5 min., 2 min. 4 min., 8 min., 15 min., 20 min., 2 hr., 4 hr., 8 hr., 12 hr., 24 hr., 48 hr., and 72 hr. After taking the readings initially for the first 2 minutes, the hydrometer was taken out and kept in another cylinder filled with distilled water. Necessary temperature corrections, zero corrections and meniscus corrections

were made to the hydrometer readings as per procedure. Figure 3.2 shows a typical gradation curve for the hydrometer analysis test.

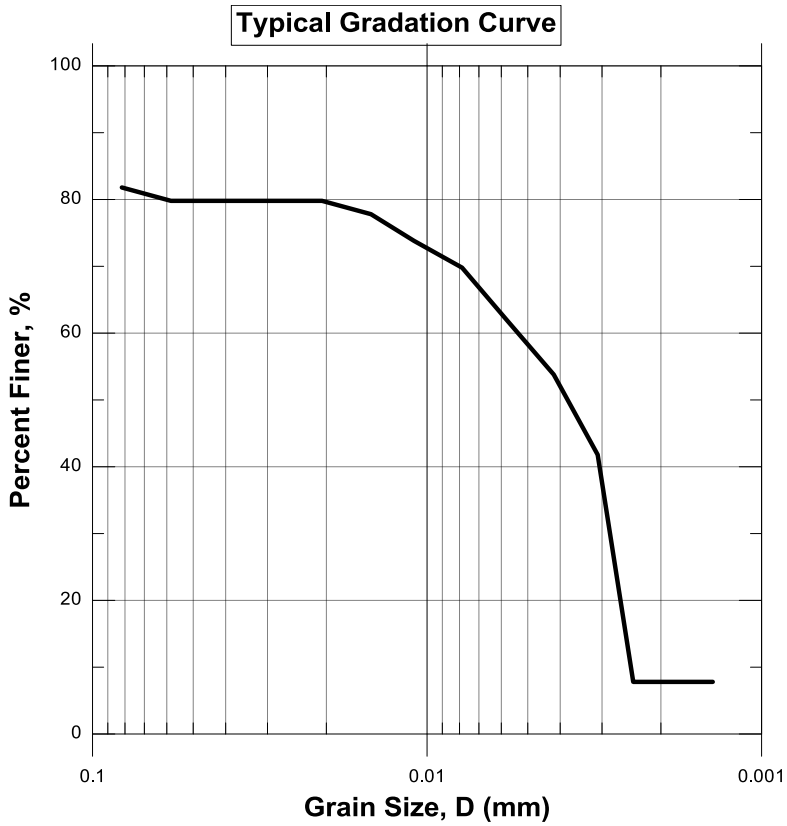


Figure 3.2 Typical gradation curve for the hydrometer analysis test

3.1.4 Standard Proctor Compaction Tests

In order to determine the compaction moisture content and dry unit weight relationships of the soils in the present research program, it was necessary to conduct standard Proctor compaction tests on the soils to establish compaction relationships. The optimum moisture content (OMC) of the soil is the water content at which the soils are 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 soil to determine moisture content and dry unit weight relationships. Standard Proctor test method using Tex-114-E procedure was followed to determine the maximum dry density (MDD) and corresponding optimum moisture content. This test requires use of a hand held rammer, which has a 50 mm diameter face with a weight of 2.5 kg, and a cylindrical mold, which has internal dimensions of 105 mm in diameter and 115.5 mm in height. This gives a volume of 1000 cm^3 (or 1/30 ft^3). Figure 3.3 shows the proctor compaction apparatus.

Figure 3.3 Standard proctor compaction test apparatus

Soil mix was compacted in four layers by applying 25 blows per layer of rammer dropping from the controlled height of 300 mm. After compacting, the compacted soil was weighed and the moisture content was measured. The procedure was repeated after each increment of water was added to the soil

mix. Test results were commonly presented in the form of a compaction curve, which depicts a relationship between dry unit weight and water content. Water contents at 95% of the maximum dry density conditions representing wet of OMC were selected for testing. A typical Proctor's standard compaction curve of soils is presented in Figure 3.4.

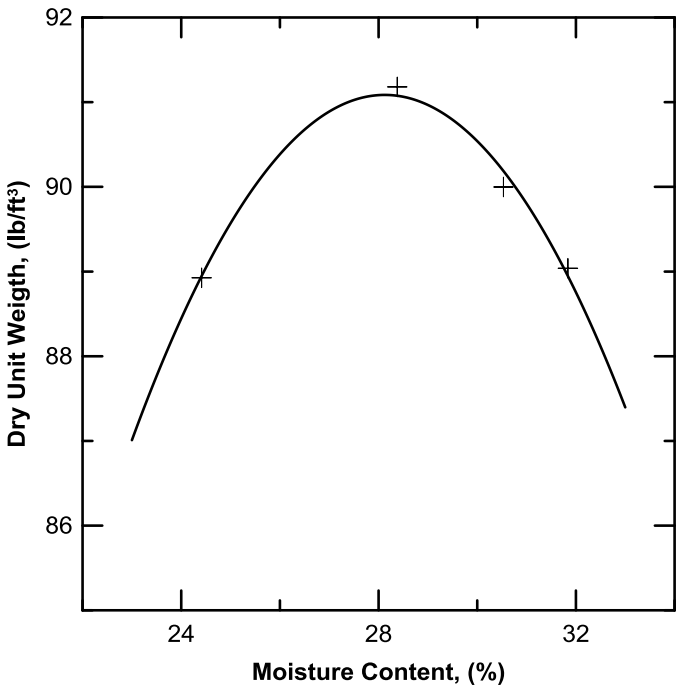


Figure 3.4 Typical Proctor's standard compaction curve

Since stabilized soils exhibit different moisture content and dry unit weight levels, all of the soil samples were mixed with previously determined dosage of lime using TxDOT standard procedure Tex-121-E Soil-Lime Testing, explained in the next chemical tests section, and then subjected to standard proctor tests. The standard proctor compaction results were used to establish moisture content and dry unit weight levels for treated soil sample preparation.

3.3 Engineering Tests

3.3.1 Unconfined Compressive Strength Test

The Unconfined Compressive Strength (UCS) tests were conducted as per ASTM D 2166. This test was conducted on the soil samples under unconfined conditions. The test was conducted on compacted soil specimens of 2.8 inches in diameter and 5.6 inches in height. The soil specimen was first placed on a platform and then raised at a constant strain rate using the controls of the UCS set up until it came in contact with top plate. Figure 3.5 shows the unconfined compressive strength test set up and the computer system used for data acquisition. Once the specimen was intact, it was loaded at a constant strain rate and as the load approached the ultimate load, failure cracks began to appear on the surface of the specimen. Figure 3.6 shows a soil sample after failure. Both deformation and corresponding axial loads on the specimen were

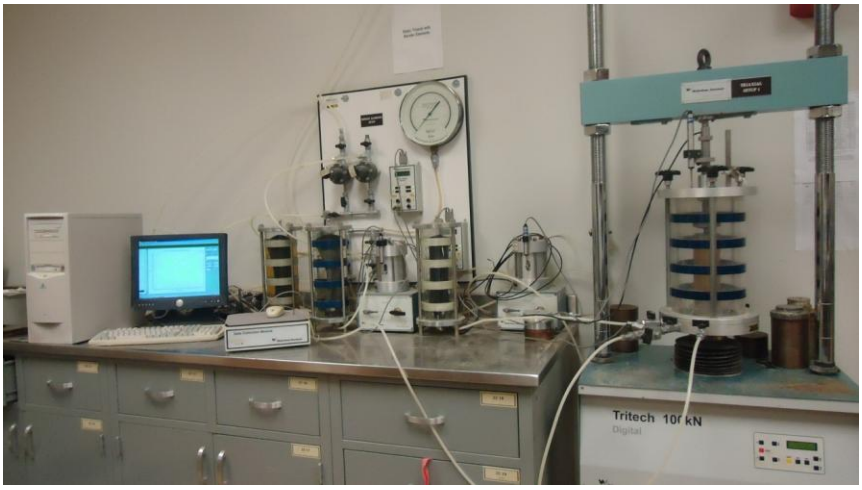


Figure 3.5 Unconfined Compressive Strength Test Setup and the Computer System in the Laboratory.

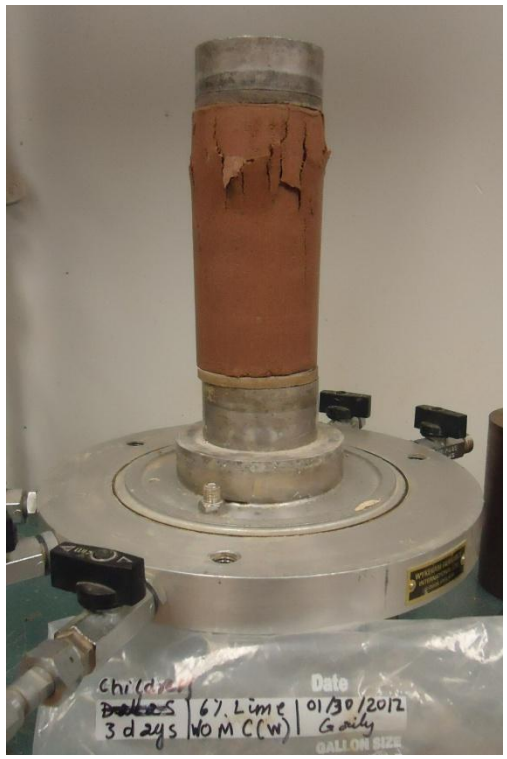


Figure 3.6 Soil sample after failure under UCS

recorded using a Data Acquisition System (DAS). The data retrieved contained load (Q) and deformation (δ) data and the same were analyzed to determine the maximum unconfined compressive strength (q_u) in psi or kPa. The following expressions show the computation of stress (σ) and strain (ε) corresponding to the load-deformation data.

$$\varepsilon = \frac{\delta}{L}\ ;\ \sigma = \frac{Q}{A_c}\ ;\ A_c = \frac{A}{1-\varepsilon}\ \text{ and } q_u = \sigma_{max}$$

Where, δ = change in length, L = total length of specimen, A_c = corrected area of cross section of the specimen and A = initial area of cross section. A typical Stress-Strain curve obtained from UCS test is presented in Figure 3.7.

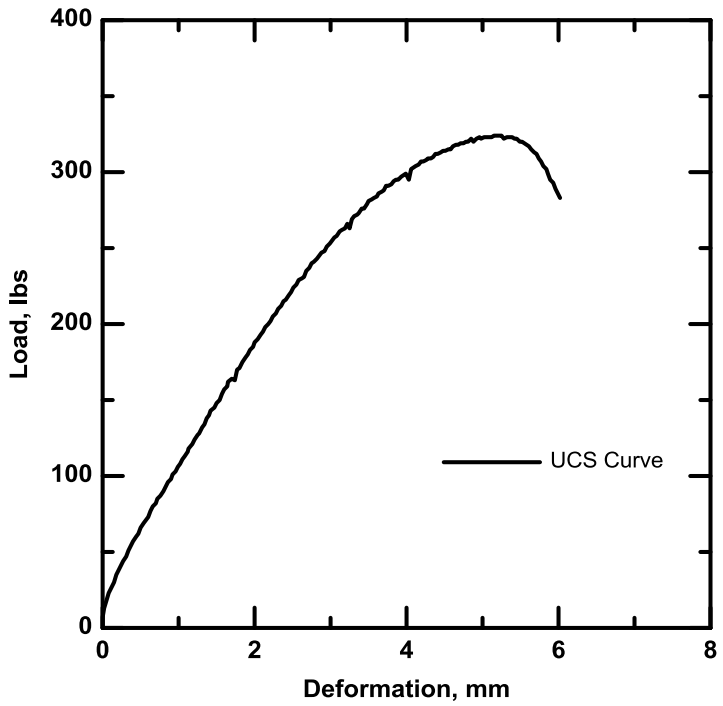


Figure 3.7 Typical Stress-Strain curve obtained from UCS test

3.3.2 Three-Dimensional Free Swell Tests (Volumetric Free Swell Tests)

Free swell test commonly refers to the conventional one-dimensional free swell test, which was conducted on soil specimens that were 2.5 in. diameter with 1 in. thickness. This test was only used to measure the maximum amount of heave in the vertical direction of a confined soil specimen. To examine the maximum vertical, radial, and volumetric swell potential, three-dimensional free swell test was conducted in this research. The following paragraphs describe soil sample preparation method and testing procedures used.

Oven-dried soils were pulverized and mixed at targeted moisture content levels. Both control and treated soil specimens were mixed and then compacted

by using Gyratory Compactor Machine at two moisture content levels, Figure 3.8 shows a Gyratory Compacter Machine and soil sample after extraction.

(a) (b)

Figure 3.8 (a) Gyratory Compacter Machine, (b) Soil sample after extraction

Soil samples were compacted at the optimum (OMC) and at 95% wet-of-optimum moisture content (W/OMC) of standard proctor test curve. Previously determined lime dosage level of 6% by weight of dry soil, were used as chemical stabilizer. Soil was mixed with lime with hand and an attempt is made to ensure proper distribution of lime in the soil mix.

Three-dimensional free swell test allows testing of both treated and untreated soil specimens under unconfined conditions. Figure 3.9 shows the three-dimensional free swell test setup used in this study.

As shown in Figure 3.9, samples, which are 4 in. (101.6 mm) in diameter and 4.6 in (116.8 mm) in height, were covered by a rubber membrane. Porous stones were placed at the top and bottom of the specimens, which facilitated the movement of water to the soil specimen. The specimen was fully soaked with water. The amount of heave in both vertical and diametral direction was monitored until there was no significant swell for 24 hours. Measurements were taken at the top, middle, and bottom circumferences of the samples and averaged at a frequency similar to the Consolidation Test. The percent values are calculated based on the original dimensions of the soil specimen.

Figure 3.9 Three-dimensional free swell test setup

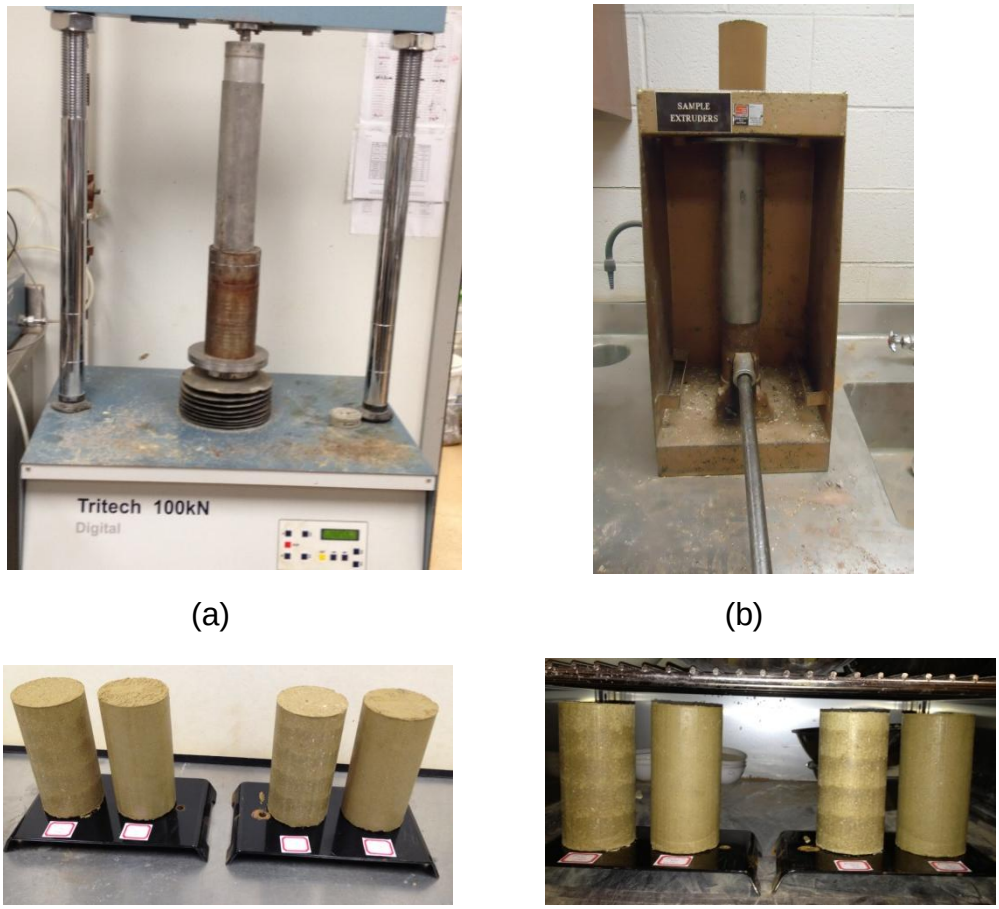
3.3.3 Three-Dimensional Shrinkage Tests (Volumetric Shrinkage Tests)

Due to limitations in the linear shrinkage bar test, researchers at University of Texas at Arlington (UTA) proposed a new test method using

cylindrical compacted soil specimens and subjecting them to drying process and then measuring the volumetric, axial and radial shrinkage strains. This test offers several advantages over conventional linear shrinkage bar test such as reduced interference of boundary conditions on shrinkage, larger amount of soil being tested, and simulates compaction states of moisture content - dry density conditions. This method was published in ASTM geotechnical testing journal (Puppala et al., 2004), which signifies the importance of this method being accepted by the researchers and practitioners.

Volumetric shrinkage tests were conducted to measure the decrease in the total volume of soil specimens due to loss of moisture content from predetermined initial moisture content to a completely dry state. Two different initial moisture contents (optimum, wet of optimum) were used as initial compaction conditions and tests were conducted as per the procedure outline in Puppala et al. (2004). Specimen preparations were performed by mixing the dry clay with appropriate amount of water to achieve the designed water contents, then compacting the soil specimens in 2.26 in. (57 mm) diameter and 5 in. (127 mm) height mold, and measuring the initial height and diameter of the specimen. The specimens were then extracted and left at room temperature for 12 hours and then transferred to an oven set at a temperature of 220º F (104º C) for 24 hours. Upon removal from the oven, the average height and radial dimensions of the dried soil specimens were manually measured. The volumetric shrinkage strain was calculated from these measurements. Figure

3.10 shows a typical example of the specimen's preparation, sample extraction and drying process over bench top followed by oven drying.



(a) (b)

(c) (d)

Figure 3.10 Three-dimensional free shrinkage (a) specimen's preparation (b) specimen's extraction (c) over bench top drying and (d) oven drying

3.4 Chemical Tests

3.4.1 Cation Exchange Capacity

Cation exchange capacity or CEC can be used to determine the mineral composition of a given soil. For example, a soil with a high CEC value of 100

meq/100 gm to 120 meq/100 gm 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. 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 is selected for the current research. The method involves addition of a saturating solution and then removal of the adsorbed cations using an extracting solution. The saturating solution used here is ammonium acetate (NH4OAc) at pH 7. This solution was added to a prepared soil specimen (preparation involves treating for organics with 30% hydrogen peroxide (H2O2)) and set aside for 16 hours after shaking for half hour, to ensure that all the exchange locations are occupied by the ammonium ion (NH4+). Then the solution was filtered through a Buchner funnel and washed with 4 different 25 mL additions of NH4OAc. This step is to bring out all the cations from the soil sample solution that has been replaced by ammonium ions. Excess NH4OAc 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 is also removed. The CEC of the soil sample can be obtained by measuring the amount of ammonium ions that replaced all the exchange locations. This was done by washing the sample with 8 different 25 mL additions of 1M potassium chloride (KCl) solution. The

concentration of NH4+ ion in the KCl extract gives the CEC of the soil. Photographic representation of the different steps involved is presented in Figure 3.11.

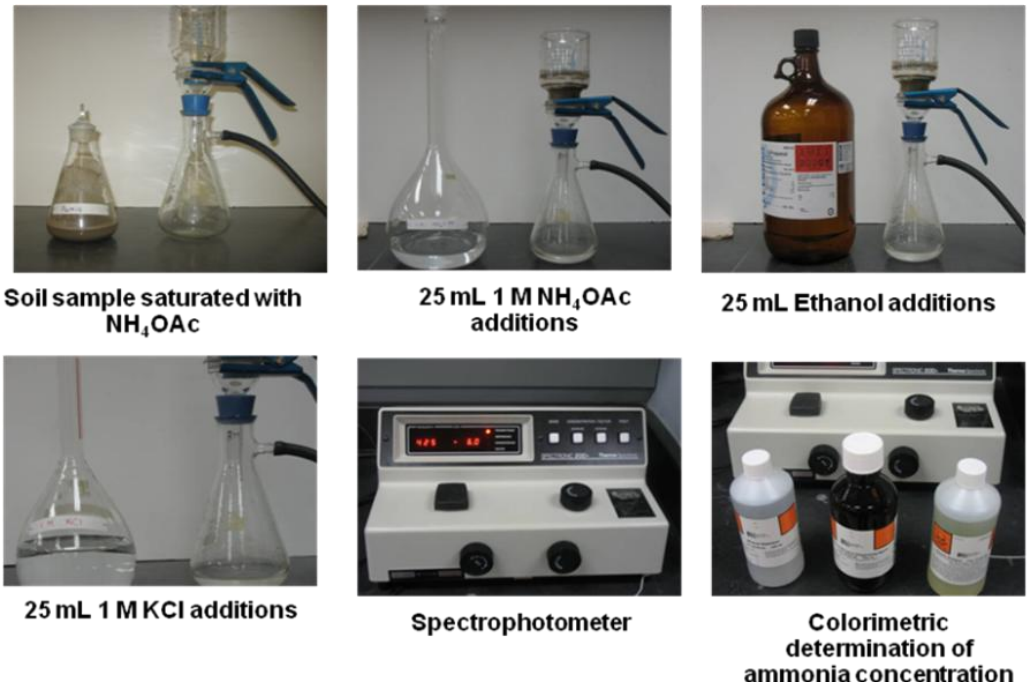


Figure 3.11 Photographs of the various steps involved in the determination of CEC

3.4.2 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 adsorption by Ethylene Glycol Monoethyl Ether (EGME) (Carter et al., 1986) and is

implemented in this research. This involves saturating prepared soil specimens, equilibrating them in vacuum over a calcium chloride – EGME (CaCl2-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 procedures typically take two days to complete. They also indicated that the procedure is repeatable and gives reliable results. Photographic representation of the different steps involved is presented in Figure 3.12.

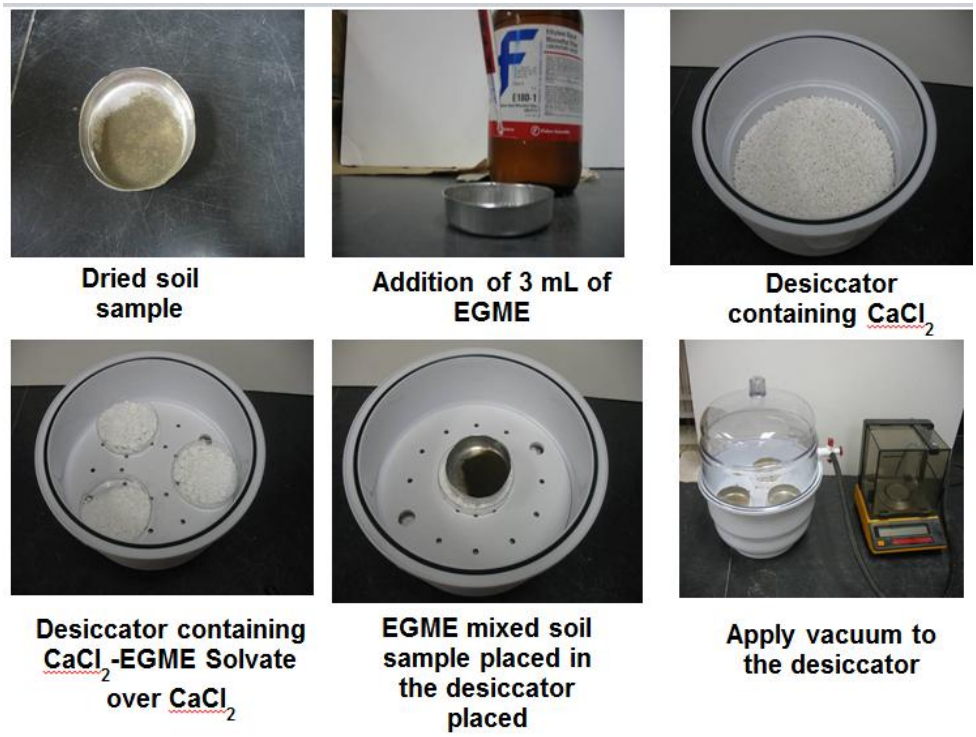


Figure 3.12 Photographs of the various steps involved in the determination of SSA

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 m^2/g to 800 m^2/g.

3.4.3 Total Potassium

Potassium is the inter layer cation in the clay mineral Illite (Mitchell and 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 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 started by taking 0.1 gm of soil in a Teflon digestion vessel. The original method recommended 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 looked at and a 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 high temperature tolerance (200ºC). Hence, Teflon vessel was finally selected.

An amount of 5 ml of Hydrofluoric acid and 0.5 ml of Perchloric acid were added to 0.1 gm of the soil sample. Hydrofluoric acid dissolves the silicate mineral structure and releases the interlayer cations; Perchloric acid was used as an oxidizing agent to oxidize the organic matter in the soil sample. Then the vessel was placed on a hot plate and heated to 200ºC and then cooled, and another addition of HF and HClO4 was made and reheated on the hot plate. The sample was then heated until it was dry. The process was repeated to make sure all the interlayer cations were released and then finally 6N HCl was added and the amount of potassium in this solution was obtained by using a spectrophotometer. Photographic representation of the different steps involved is presented in Figure 3.13.

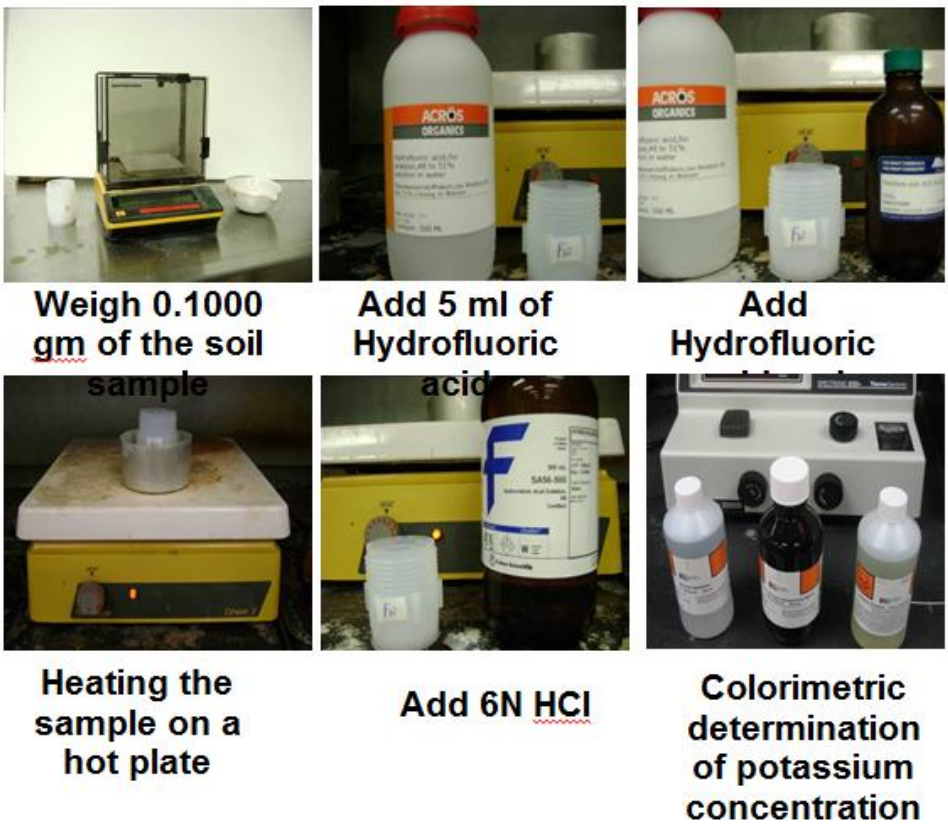


Figure 3.13 Photographs of the various steps involved in the determination of TP

3.4.4 Soluble Sulfate Measurement

The soluble sulfate measurement test in this research is conducted as per TxDOT procedure Tex-145-E (Test Procedure for Determining Sulfate Content In Soils — Colorimetric Method). This method is turbidity based method which involves measuring the cloudiness of a liquid and then translates it into concentration. In this procedure, a 300 g of oven dried soil sample passing through the Sieve No.40 is obtained. The sample was then mixed thoroughly for uniformity. A 20 g of representative soil sample was collected in a High-density

polyethylene (HDPE) bottle and mixed with 400 ml of distilled water to which provided an initial dilution ration of 1:20.

The sample is then tested for conductivity by a conductivity meter. Conductivity is an excellent measure of soluble salts in a solution. If the conductivity is more than 240 μS, further testing is carried out. If the conductivity is below 240 μS, soil can be considered as a low-sulfate soil. The sample was shaked manually for 1 minute and kept for 12 hours. After minimum of 12 hours has elapsed, the sample then filtered through a Whatman No.42 filter paper and the filtrate is collected in a beaker. Using a clean uncontaminated pipette, 10 ml of the clear filtrate is extracted and placed in a sample vial. The sample vial is then cleaned using Kimwipe to remove any finger prints or dirt marks which interfere with the sulfate measurement. The colorimeter is switched on and the mode key is pressed until “SUL” method is displayed. The sample vial is then placed in the colorimeter and zeroed for initial calibration. The sample vial is taken out and a sulfate tablet is added to the solution and tamped with a white plastic rod. A reagent (sulfate test tablet) is used in this method which causes the turbidity in the sample. The sample vial is then placed in the colorimeter and the test key is pressed. The colorimeter displays a reading and an average of three readings is obtained. When the sulfate level of the soil is above the equipment limit an error message is displayed on calorimeter screen. In such cases, higher dilution ratios are used. The results are multiplied with the dilution ratio to obtain the concentration in

ppm. A photograph of colorimeter and conductivity meter is shown in Figure 3.14.

Figure 3.14 Photograph of the colorimeter and conductivity meter

3.4.5 Lime Content Determination (Eades-Grim Test)

The amount of lime to be added to the soils is determined for this research as per TxDOT guidelines (Tex-121-E). The procedure is described below. Weigh to the nearest 0.01 gm a series of 30 gm soil samples and place them in separate containers. A series of lime equivalent to 0, 2, 4, 6, 8 and 10% of dry weight of soil then prepared. Add one of the lime percentages to each of the containers followed by addition of 150 ml of water. The mixture then stirred every 15 minutes for about an hour. At the end of hour, temperature of the mixture was record and adjustment of the pH meter to that temperature was carefully made. Using distilled water the pH meter was normalized to 7.0. After

cleaning the electrodes with distilled water, the pH of the soil-lime solutions is checked, recorded and then plotted on y-axis and lime dosage on x-axis. From the curve, pH value of 12.4 was found and corresponding lime dosage is obtained. A graphical representation of the lime dosage versus pH is given in Figure 3.15.

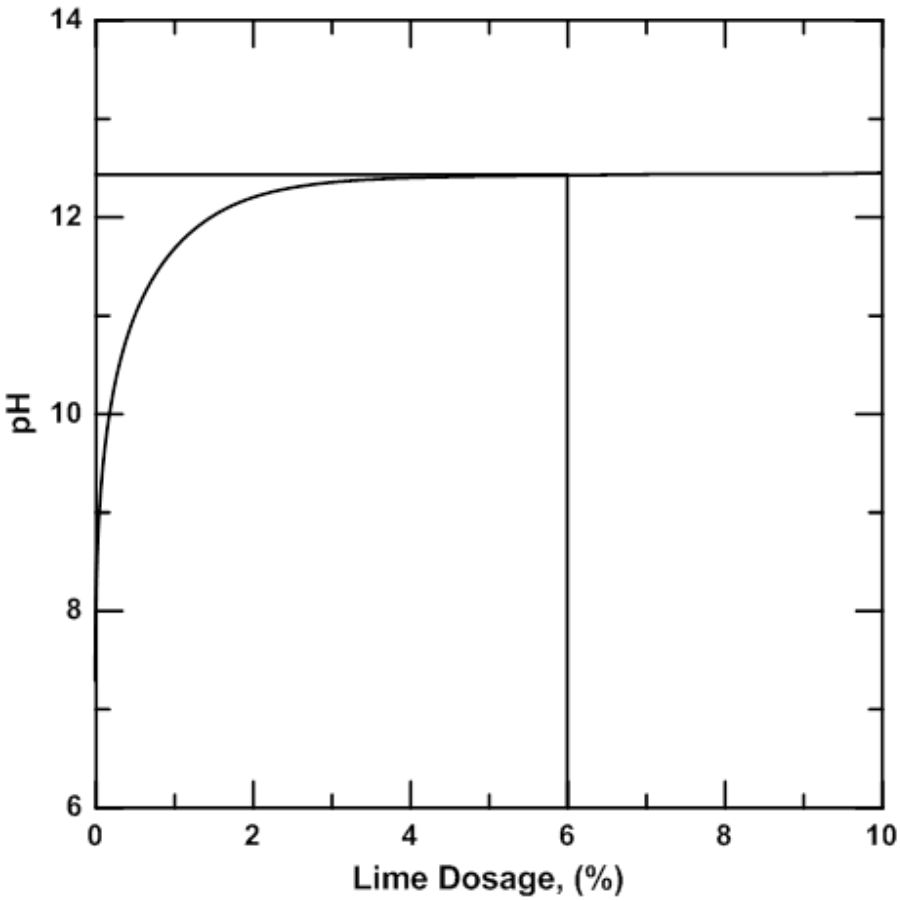


Figure 3.15 Typical graphical representation of the lime dosage versus pH

3.5 Summary

In this chapter various test procedures followed in this research to determine the engineering properties of both control and treated soils are described. Basic and engineering geotechnical testing carried out are explained and typical results are provided. The chemical tests conducted for clay mineral quantification are explained also in this chapter. The next chapter provides the results attained from all the above mentioned tests that were performed on all the soils selected for this research study along with the explanation and discussions.

CHAPTER 4

TEST RESULTS, ANALYSIS AND EXPLANATION

4.1 Introduction

In this chapter, all physical, engineering and chemical test results obtained from the entire laboratory testing program are provided. The effects of lime stabilizers, soluble sulfate contents and compaction moisture contents on the test results were explained and discussed in the following sections. The discussions of findings are based on the majority of the trends noticed in test results of both treated and untreated (control) soils.

4.2 Basic Soil Properties Test Results

4.1.1 Summary of Basic Test Results

This test results are obtained from the Atterberg Limits Test that outlined in Section 3.2.1. The PI values of all six natural soils ranged from 24 to 51. The variation is attributed to different types of soil compositions. From the results, the Riverside soil exhibited low PI value (24) when compared to other soils meanwhile both Austin and US-82 soils exhibited high PI values of (51 and 50) respectively. Table 4.1 shows the Atterberg Limits along with Specific Gravity and Unified Soil Classification System (USCS) classification for each soil type.

Table 4.1 Basic Soil Properties

Soil Region	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Specific Gravity	USCS Classification
Austin	76	25	51	2.61	CH
Childress	71	35	36	2.51	MH
Dallas	80	35	45	2.71	CH
FM1417	72	30	42	2.66	CH
Riverside	35	11	24	2.56	CL
US-82	75	25	50	2.65	CH

4.1.2 Lime Content Determination (Eades-Grim Test)

Based on the procedure that previously detailed in Section 3.4.5, the dosage of lime to be added to the soils for treatment is 6% (by dry weight of soil). This value was obtained from the graph that plotted for the pH vs. Lime dosage in percent which corresponded to pH value of 12.4.

4.1.3 Standard Proctor Compaction Test Results

As explained in Section 3.2.4, the Standard Proctor Compaction test, was conducted on the six soils that are selected for this study. The number of specimens compacted were required to provide at least two (2) “points” on the dry side and minimum of one (1) “point” on the wet side of the estimated optimum moisture content (OMC). This allows a “best-fit” curve to be drawn to

establish the actual percent moisture content and dry density for each soil tested. The percent moisture content and the dry density determined for each "point" are plotted for each soil and are shown in Figures 4.1 through 4.6.

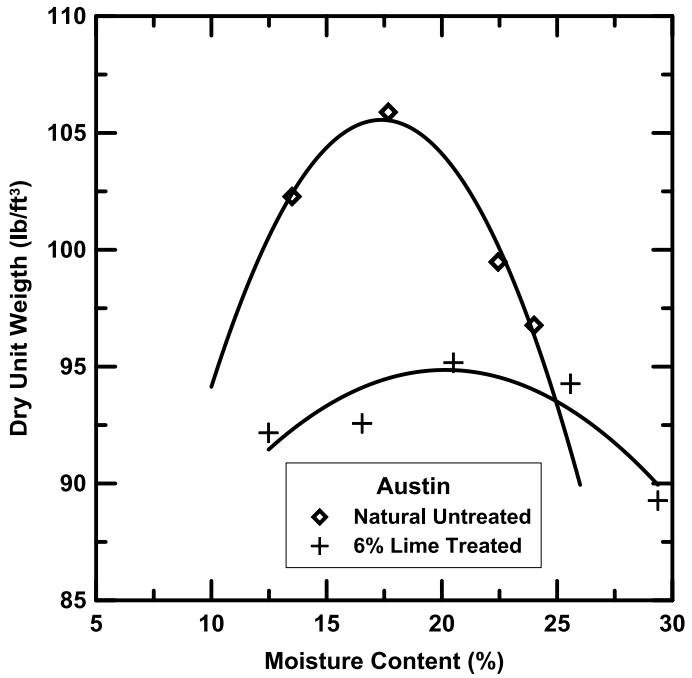


Figure 4.1 Standard Proctor Compaction Curves for Austin soil

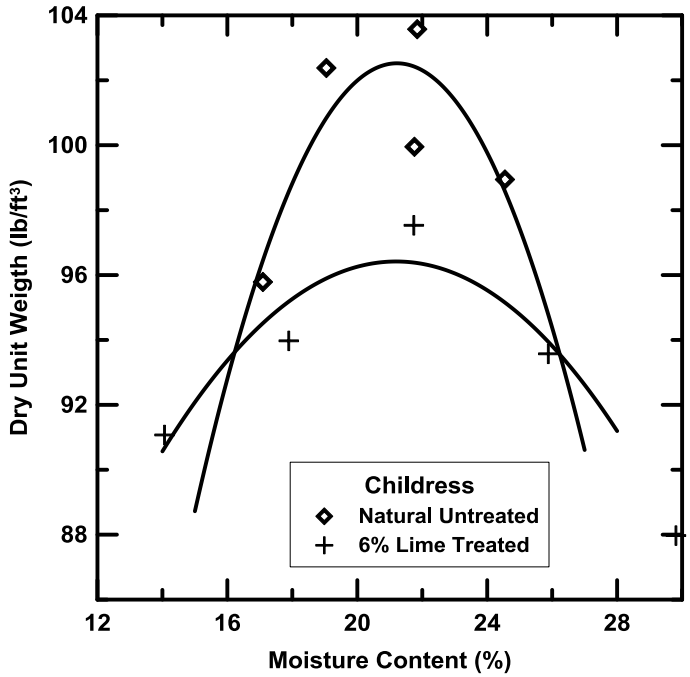


Figure 4.2 Standard Proctor Compaction Curves for Childress soil

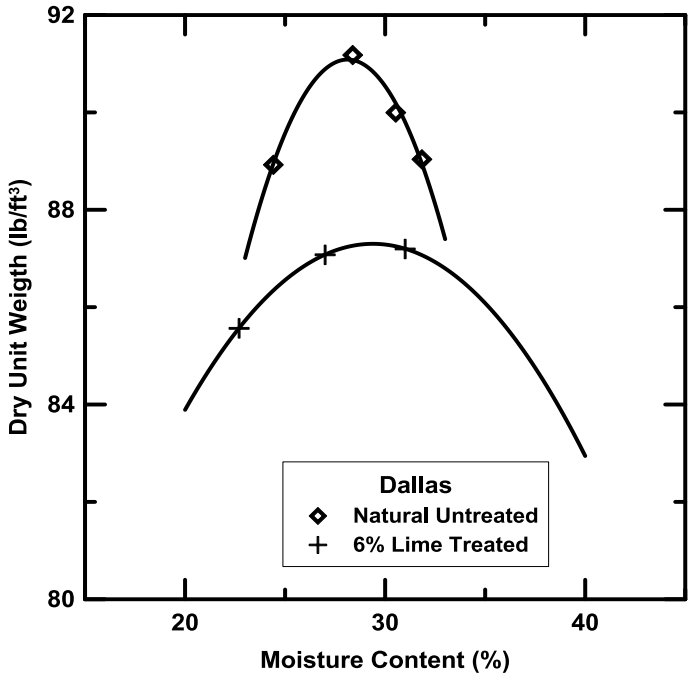


Figure 4.3 Standard Proctor Compaction Curves for Dallas soil

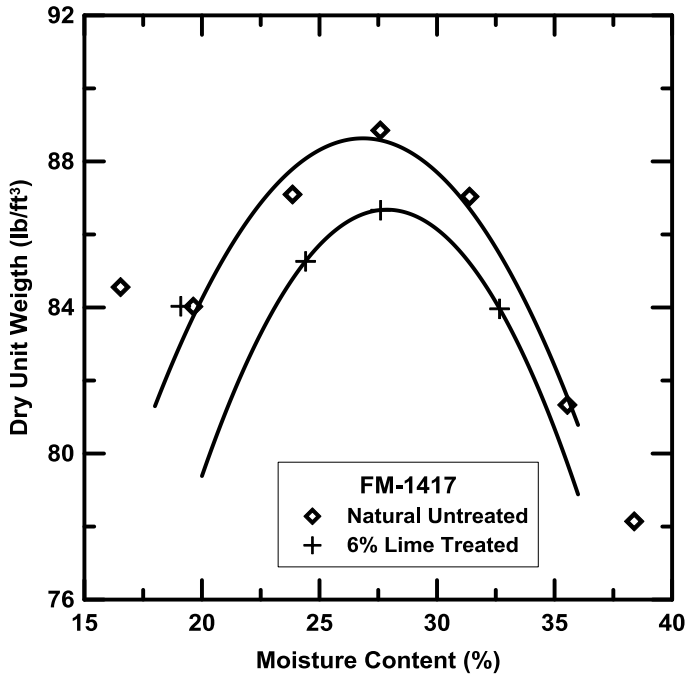


Figure 4.4 Standard Proctor Compaction Curves for FM-1417 soil

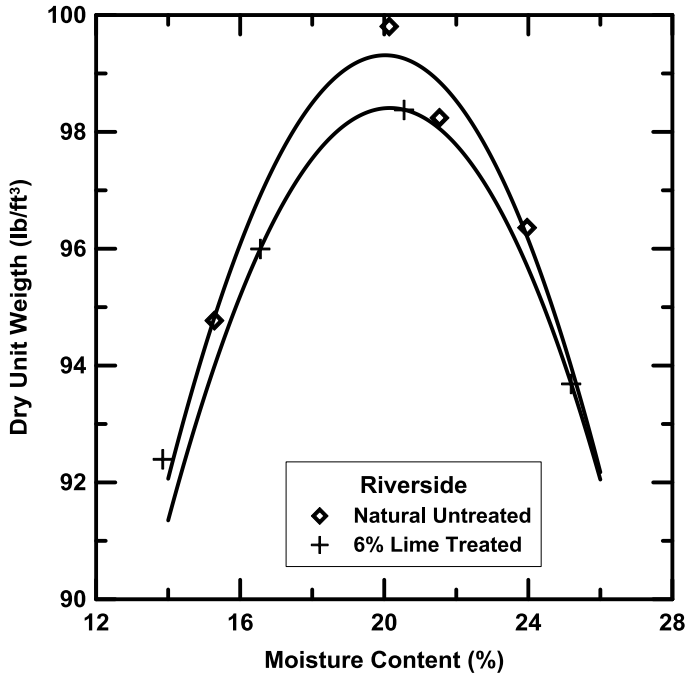


Figure 4.5 Standard Proctor Compaction Curves for Riverside soil

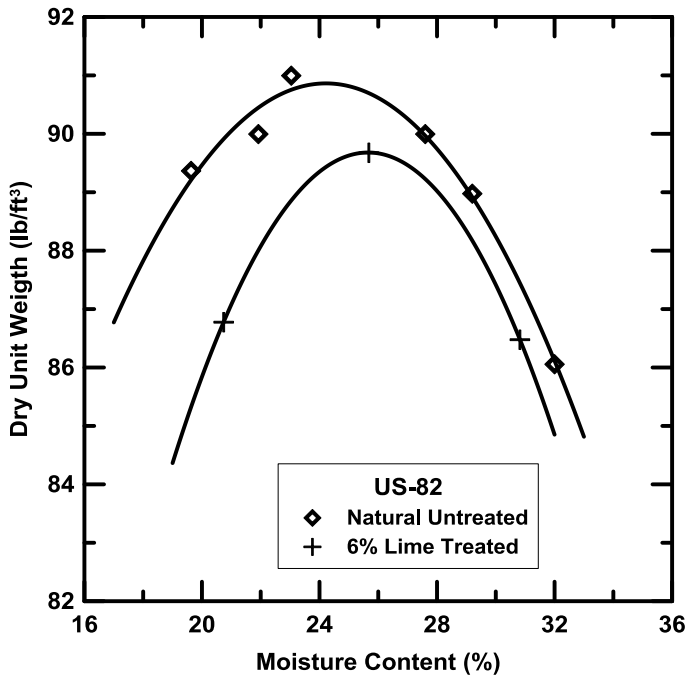


Figure 4.6 Standard Proctor Compaction Curves for US-82 soil

Table 4.2 summarizes the results of the tests run at OMC, and wet of OMC for both natural untreated and 6 % lime treated soils. It is noted that the two compaction moisture content conditions are used as reference moisture contents for other engineering tests performed in this research.

Soils in the field experience moisture content fluctuations during the seasonal changes and consequently it is imperative to establish the properties of soils over a wide range of moisture contents projected in the field. These properties were used for preparing samples and using it for engineering tests such as 3-D swell, 3-D shrinkage and UCS tests.

Table 4.2 Standard Proctor Compaction Test Results of Natural (Untreated) and 6% Lime Treated Soils

Soil Type	Natural (Untreated) Soil				6% Lime Treated Soil			
	Moisture Content (%)		Dry Density (lb/ft³)		Moisture Content (%)		Dry Density (lb/ft³)	
	OMC	WOMC	OMC	WOMC	OMC	WOMC	OMC	WOMC
Austin	18	23	106	100	21	29	95	90
Childress	21	25	103	98	22	28	96	91
Dallas	28	33	92	87	29	39	87	83
FM-1417	27	34	89	84	28	34	87	83
Riverside	20	25	99	94	21	25	98	93
US-82	24	32	91	86	26	32	89	85

4.3 Engineering Test Results

4.2.1 Unconfined Compressive Strength (UCS) Test Results

Table 4.3 and Figures 4.7 to 4.18 presents the UCS test results that were obtained from the procedure detailed in Section 3.3.1. The average unconfined compressive strength properties of natural untreated and 6 % lime treated soils compacted at optimum moisture condition and wet of optimum ranged from 10 to 108 psi.

Table 4.3 UCS Test Results for Natural (Untreated) and 6% Lime Treated Soils

Soil	UCS Strength (psi)					
	Natural (Untreated)		6 % Lime Treated			
			0 day mellowing		3 days mellowing	
	OMC	WOMC	OMC	WOMC	OMC	WOMC
Austin	28	21	88	53	55	33
Childress	23	16	108	78	45	36
Dallas	16	10	92	56	33	30
FM-1417	33	19	81	60	70	46
Riverside	30	18	63	52	47	37
US-82	31	18	73	45	52	38

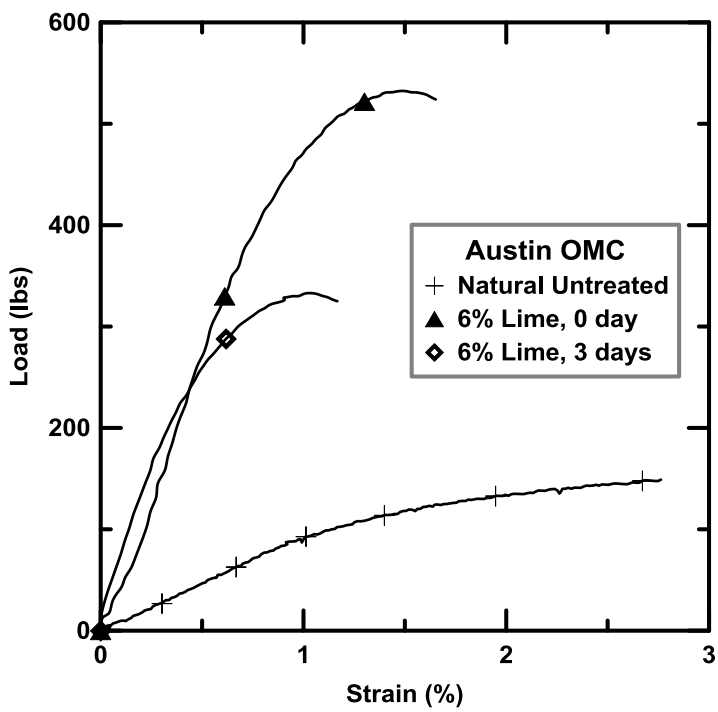


Figure 4.7 UCS Test Results for Austin soil (OMC)

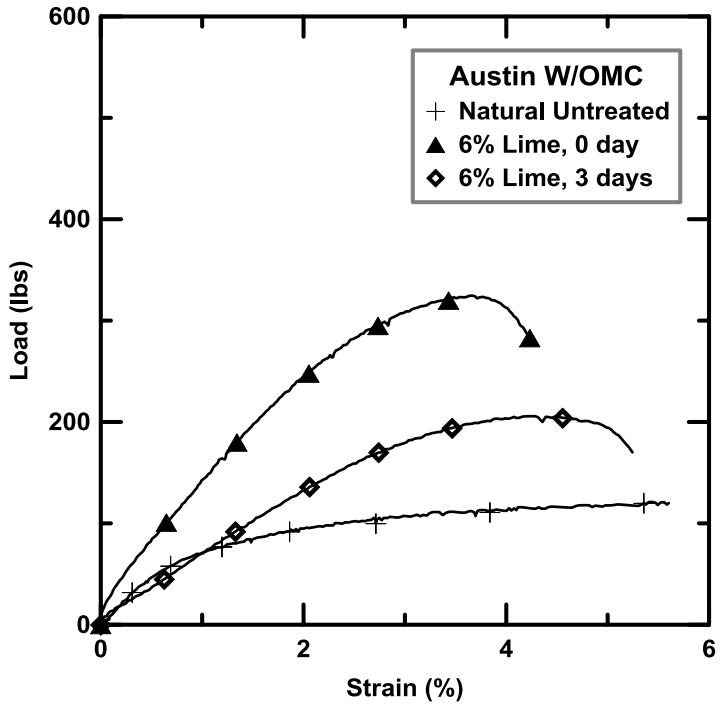


Figure 4.8 UCS Test Results for Austin soil (WOMC)

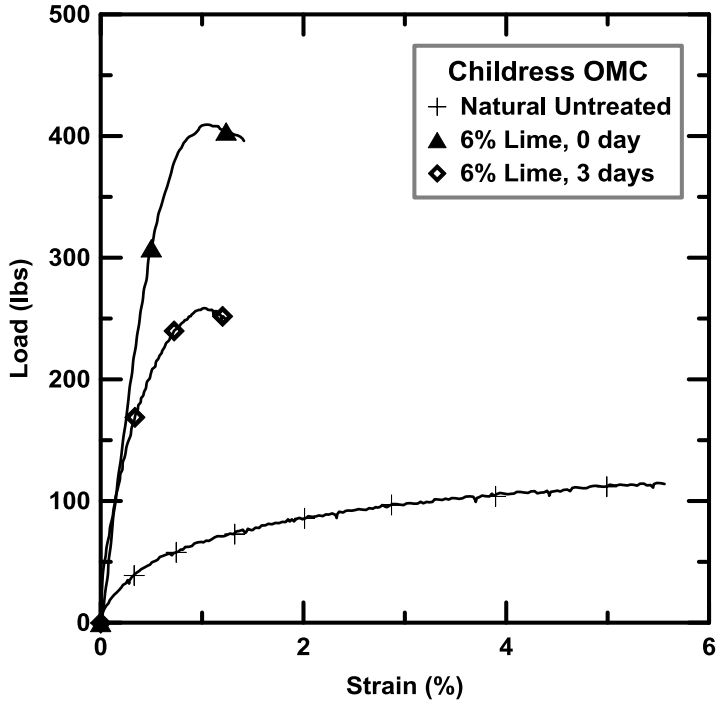


Figure 4.9 UCS Test Results for Childress soil (OMC)

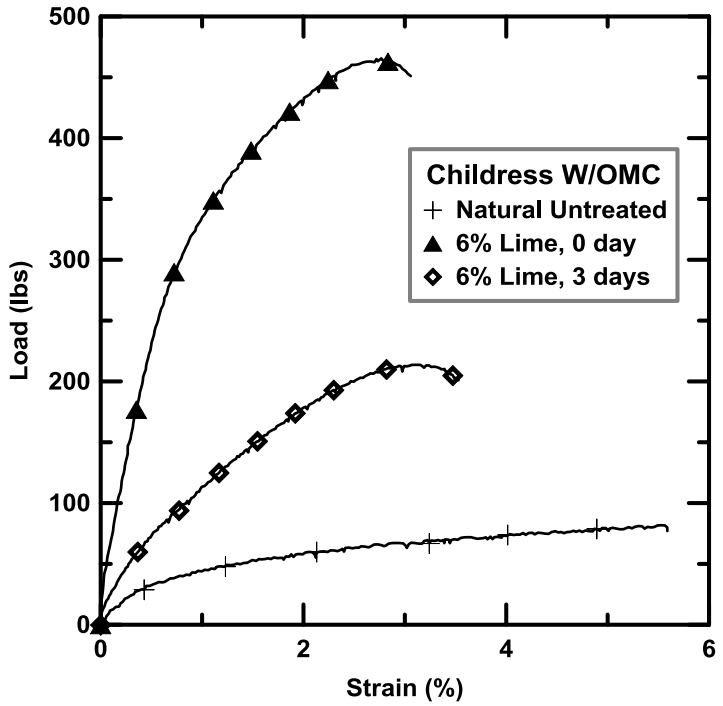


Figure 4.10 UCS Test Results for Childress soil (WOMC)

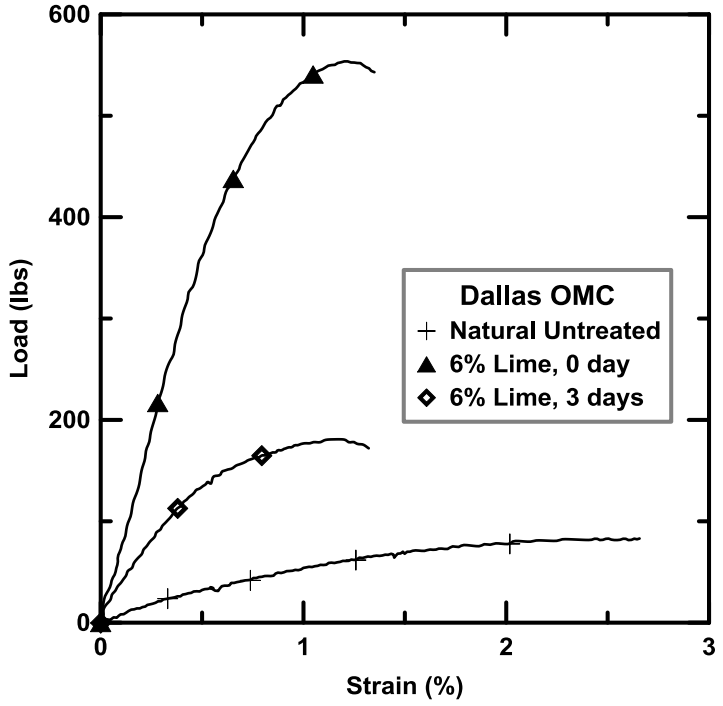


Figure 4.11 UCS Test Results for Dallas soil (OMC)

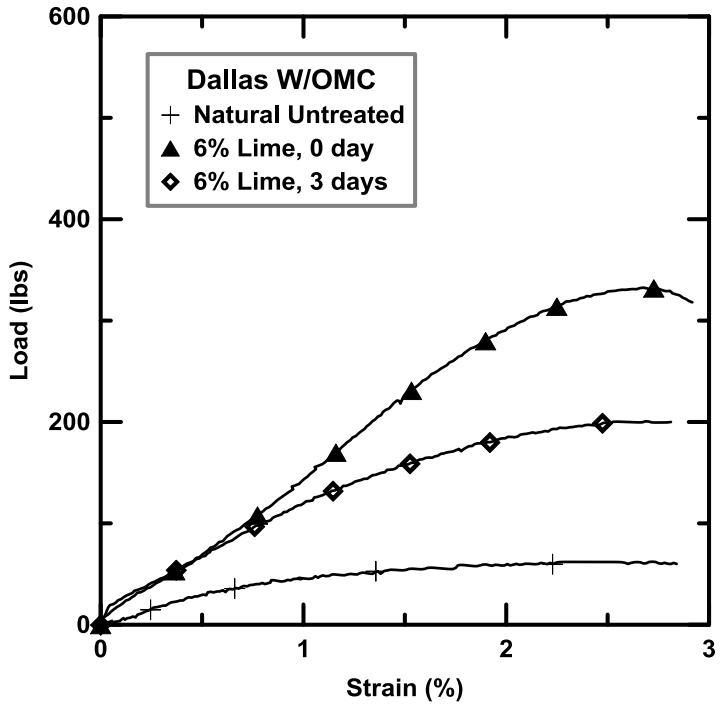


Figure 4.12 UCS Test Results for Dallas soil (WOMC)

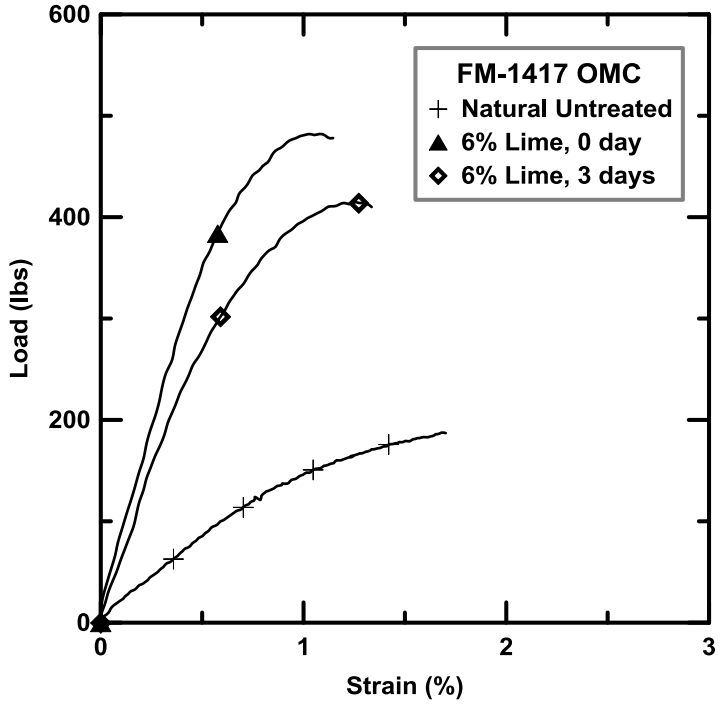


Figure 4.13 UCS Test Results for FM-1417 soil (OMC)

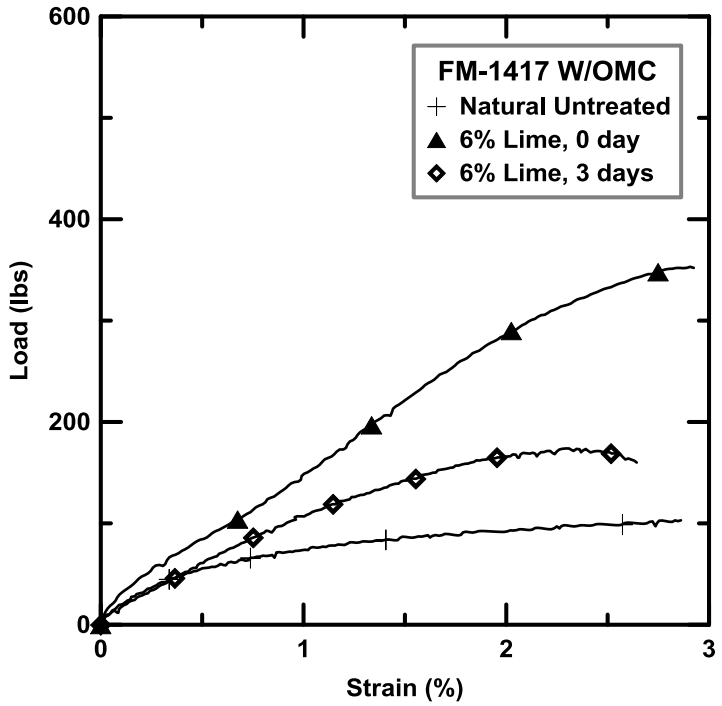


Figure 4.14 UCS Test Results for FM-1417 soil (WOMC)

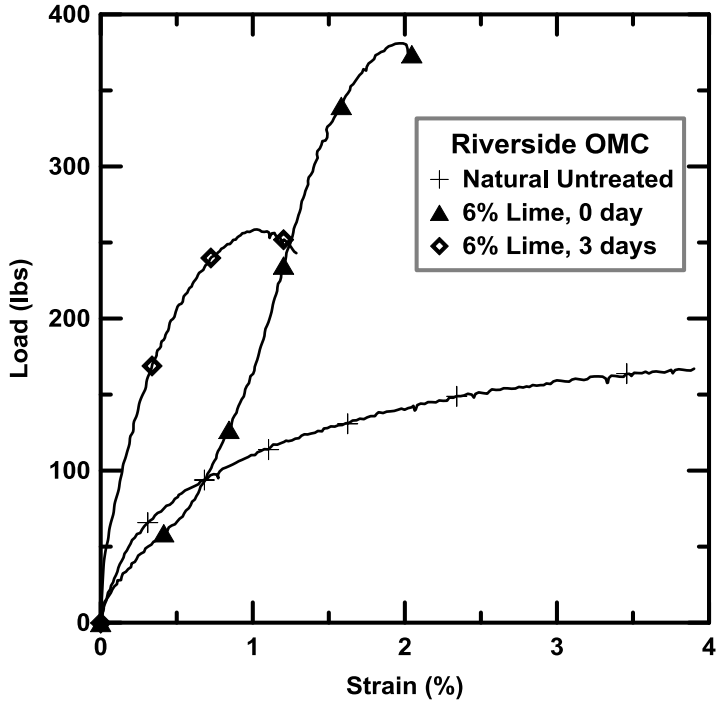


Figure 4.15 UCS Test Results for Riverside soil (OMC)

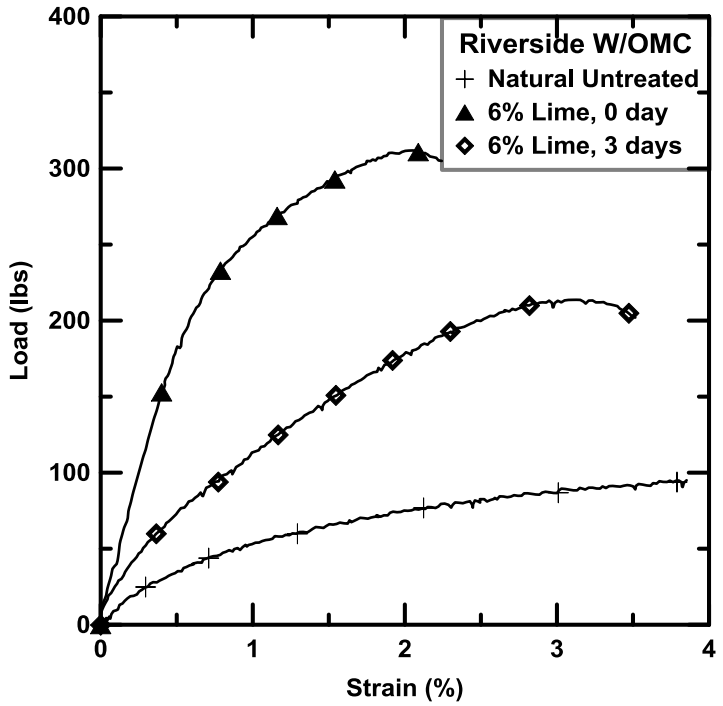


Figure 4.16 UCS Test Results for Riverside soil (WOMC)

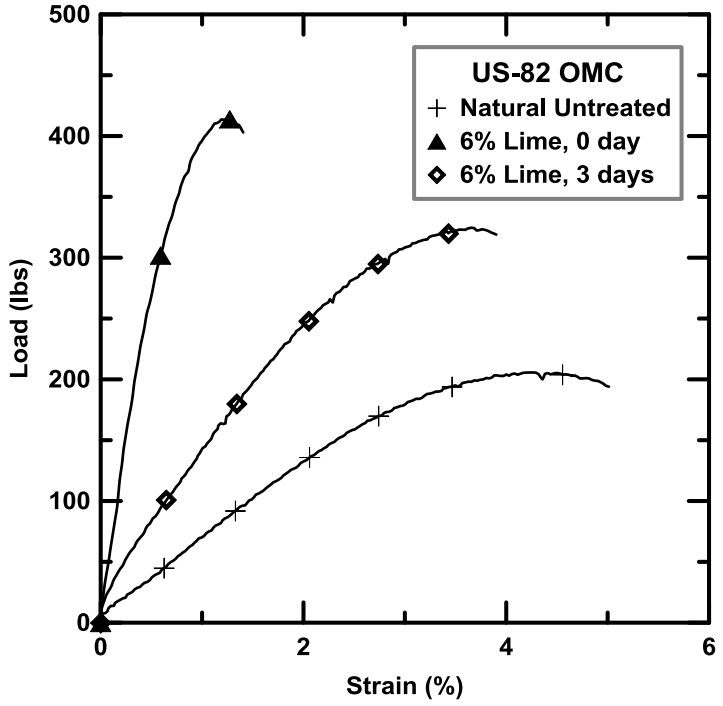


Figure 4.17 UCS Test Results for US-82 soil (OMC)

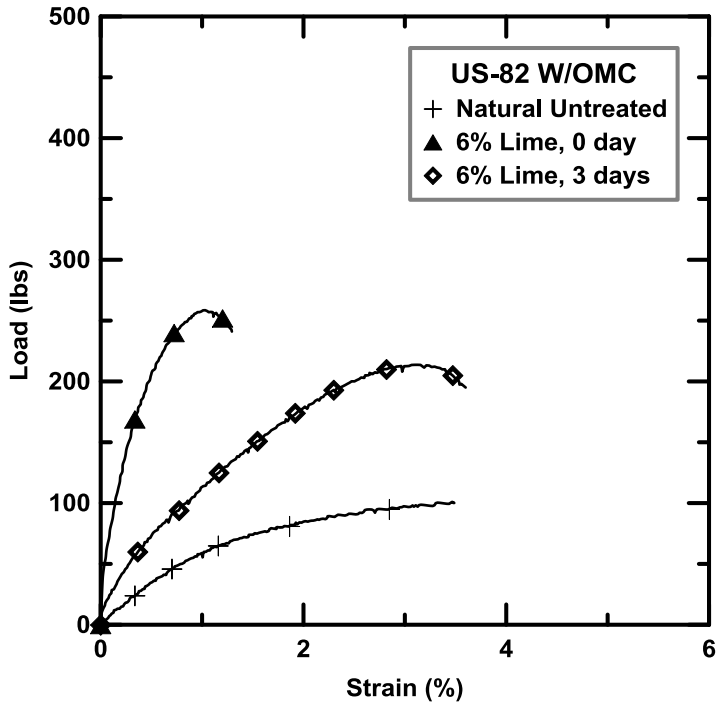


Figure 4.18 UCS Test Results for US-82 soil (WOMC)

4.2.2 Three-Dimensional Free Swell Test Results

The 3-D free swell test measures the potential of the clay to swell in three (3) directions when soaked under water. The three (3) values measured were vertical, radial, and volumetric strains as described in Section 3.3.2. Figures 4.19 through 4.24 depict the vertical swell strains of the six soil types under investigation per this research when conditioned to the OMC and wet of OMC moisture contents. Table 4.4 presents the numeric values of swell strains at different compaction and natural moisture contents and for all three swell components.

Table 4.4 Three-Dimensional Free Swell Test Results (Untreated)

Soil Type	Vertical Strain (%)		Radial Strain (%)		Volumetric Strain (%)	
	OMC	WOMC	OMC	WOMC	OMC	WOMC
Austin	7.6	3.6	4.5	2.5	16.6	8.7
Childress	3.9	0.9	1.8	1.4	7.5	3.7
Dallas	3.8	2.6	3.7	2.6	11.	7.8
FM-1417	6.0	6.1	5.1	1.3	16.2	8.9
Riverside	4.2	4	2.9	3.1	10.0	10.2
US-82	7.2	4.9	5.5	1.5	18.1	7.9

Based on these results and those shown in Table 4.1 Basic Soil Properties, it can be concluded that Austin, Dallas, FM-1417 and US-82 all have high swelling potential. This four soils has a volumetric swell strain (for the OMC condition) value greater than 10% and this value indicates a high degree of expansion potential as per the problematic volumetric swell characterizations mentioned by Chen (1988). In addition, these four soils exhibited a PI value greater than 40.

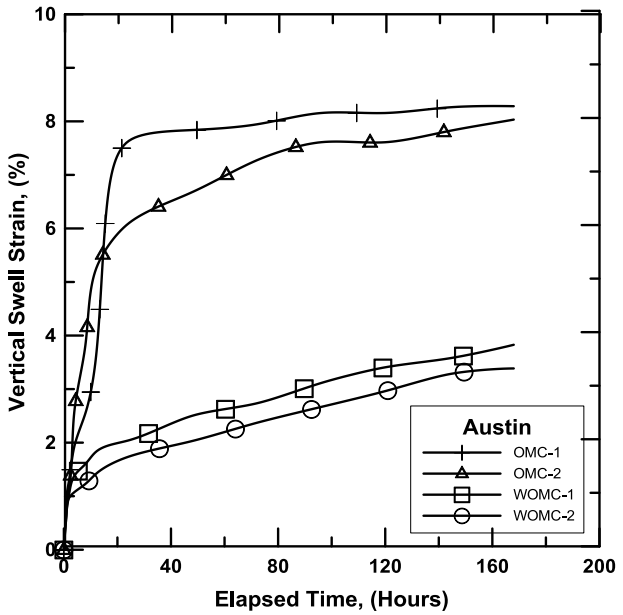


Figure 4.19 Vertical Swell Strain Results for Austin Soil

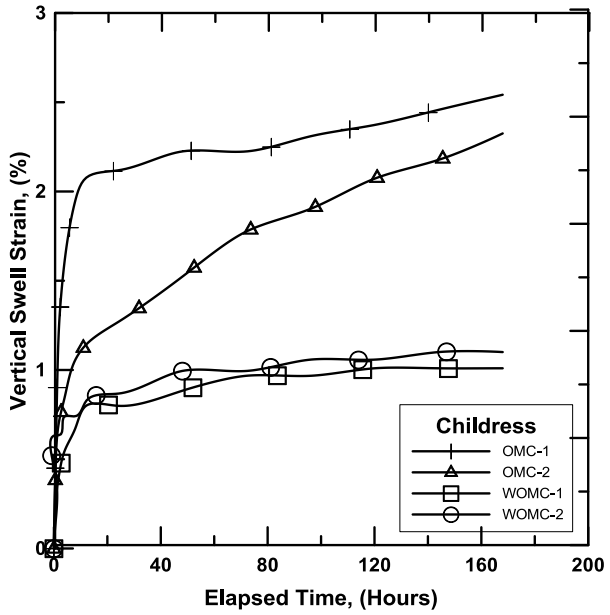


Figure 4.20 Vertical Swell Strain Results for Childress Soil

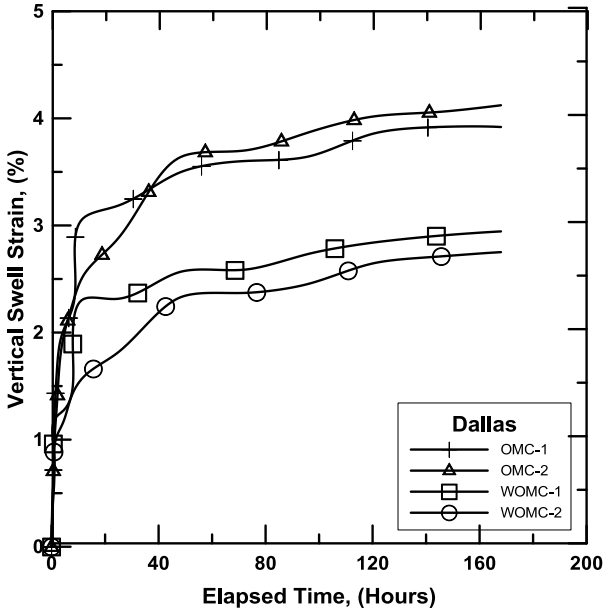


Figure 4.21 Vertical Swell Strain Results for Dallas Soil

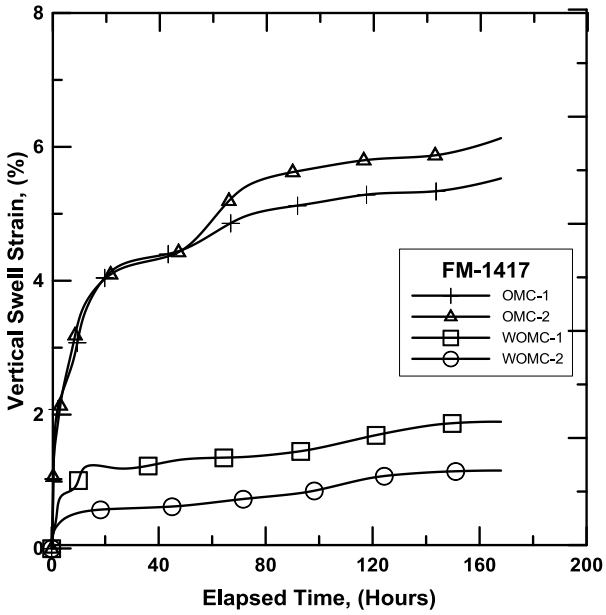


Figure 4.22 Vertical Swell Strain Results for FM-1417 Soil

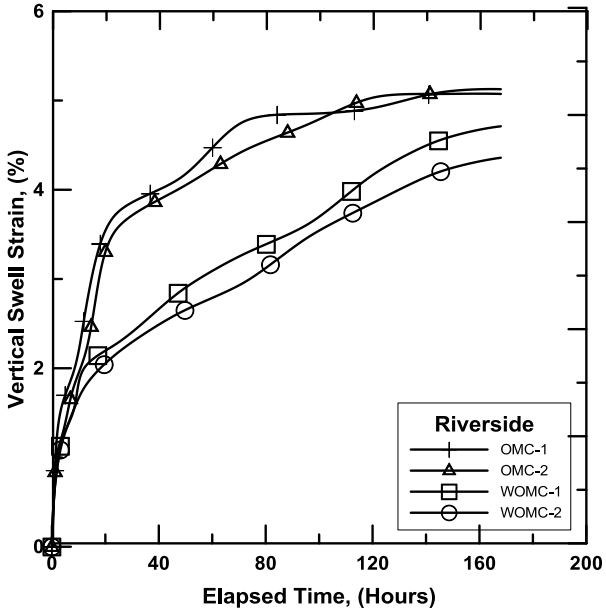


Figure 4.23 Vertical Swell Strain Results for Riverside Soil

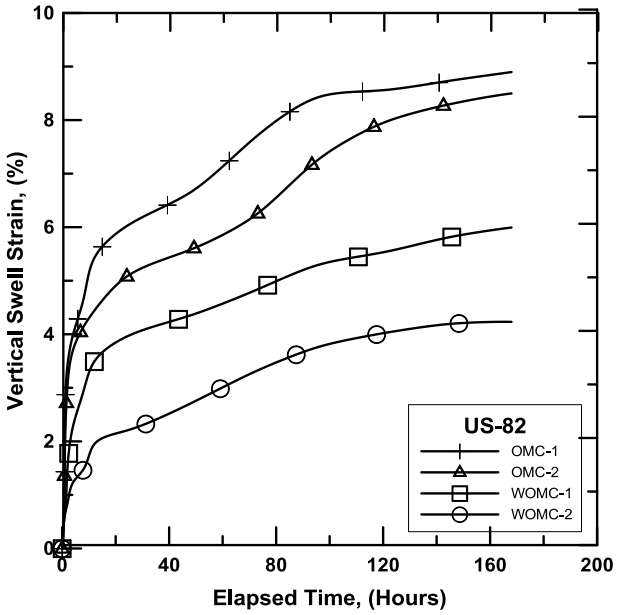


Figure 4.24 Vertical Swell Strain Results for US-82 Soil

To study the effect of mellowing period on treated soils, the 3-D free swell tests were again conducted for all the soils at two different moisture contents of OMC and WOMC with 0 and 3 days mellowing periods. Tables 4.5 and 4.6 presents the numeric values of swell strains at different compaction and natural moisture contents and for all three swell components as well as for 0 and 3 days mellowing periods respectively.

Table 4.7 shows a summary of all the volumetric swell strains for natural, 0 and 3 days lime treated soils along with the sulfate level in each soil.

Table 4.5 Three-Dimensional Free Swell Test Results for 6% Lime Treated (0 day mellowing)

Soil Type	Vertical Strain (%)		Radial Strain (%)		Volumetric Strain (%)	
	OMC	WOMC	OMC	WOMC	OMC	WOMC
Austin	7.4	N/A	0.7	N/A	8.8	N/A
Childress	5.3	5.0	4.6	4.5	14.6	14.1
Dallas	10.3	8.9	7.1	7.4	24.4	23.7
FM-1417	8.5	5.7	6.8	5.2	22.0	16.1
Riverside	5.6	4.6	5.2	4.0	16.0	12.7
US-82	8.9	9.2	7.8	8.5	24.4	26.1

Note: Due to the limited availability of Austin type soil, fewer tests were performed.

Table 4.6 Three-Dimensional Free Swell Test Results for 6% Treated (3 days mellowing)

Soil Type	Vertical Strain (%)		Radial Strain (%)		Volumetric Strain (%)	
	OMC	WOMC	OMC	WOMC	OMC	WOMC
Austin	5.3	N/A	3.2	N/A	11.65	N/A
Childress	6.5	5.2	1.0	2.5	8.5	10.2
Dallas	5.0	4.4	2.7	3.5	10.4	11.3
FM-1417	4.7	2.9	2.8	2.0	10.2	7.0
Riverside	3.6	3.7	2.4	2.7	8.4	9.0
US-82	3.9	3.7	3.7	3.3	11.3	10.3

Note: Due to the limited availability of Austin type soil, fewer tests were performed.

Table 4.7 Three-Dimensional Free Volumetric Swell Test Results for Natural (Untreated) and 6% Lime Treated Soils

Soil	Sulfate Level, ppm	Natural Soil		6% Lime, 0 day mellowing		6% Lime, 3 days mellowing	
		(ΔV/V)%		(ΔV/V)%		(ΔV/V)%	
		OMC	WOMC	OMC	WOMC	OMC	WOMC
Austin	36,000	16.6	8.7	8.8	*N/A	11.7	*N/A
Childress	44,000	7.5	3.7	14.6	14.1	8.5	10.2
Dallas	12,000	11.3	7.8	24.4	23.7	10.4	11.3
FM1-417	24,000	16.2	8.8	22.0	16.1	10.2	7.0
Riverside	20,000	10.0	10.2	16.0	12.7	8.4	9.0
US-82	12,000	18.1	7.9	24.4	26.1	11.3	10.3

* Note: Due to the limited availability of Austin type soil, fewer tests were performed.

Figures 4.25 through 4.30 shows the results for lime treated soils at OMC and Figures 4.31 through 4.35 shows the results for lime treated soils at WOMC.

For the natural soils, the 3-D volumetric swell test results ranged from 7.49% to 18.06% for OMC and the WOMC ranged from 3.66% to 10.24%. Similarly, for the lime treated soils at mellowing period of 0 day, the values of volumetric swell strains ranged from 8.78% to 24.44% for OMC and ranged from 12.67% to 26.10% for WOMC, whereas for the lime treated soils at mellowing period of 3 days, the values of volumetric swell strains ranged from 8.42% to 11.65% for OMC and ranged from 7.00% to 11.30% for WOMC.

Figures 4.36 through 4.40 depicts the volumetric swell strains of the soil types after lime treatment and conditioned to the OMC and wet of OMC moisture contents at different mellowing periods.

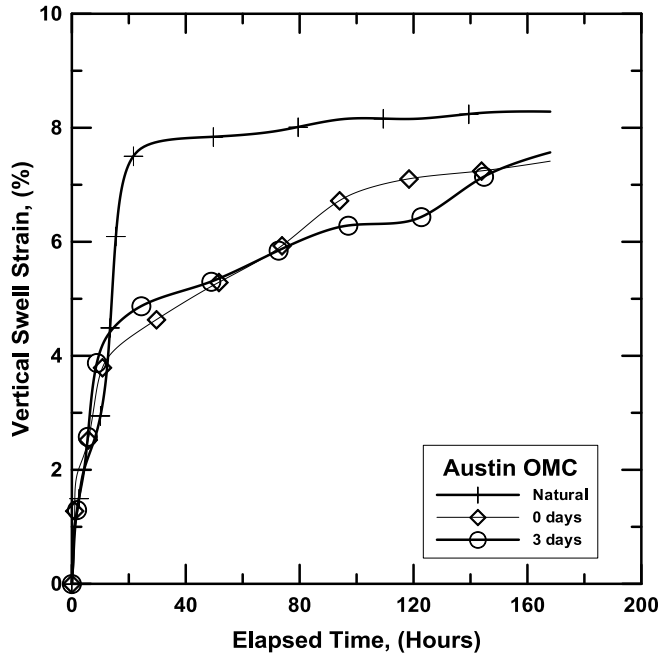


Figure 4.25 Vertical Swell Strain Results for 6% Lime Treated Austin Soil at OMC

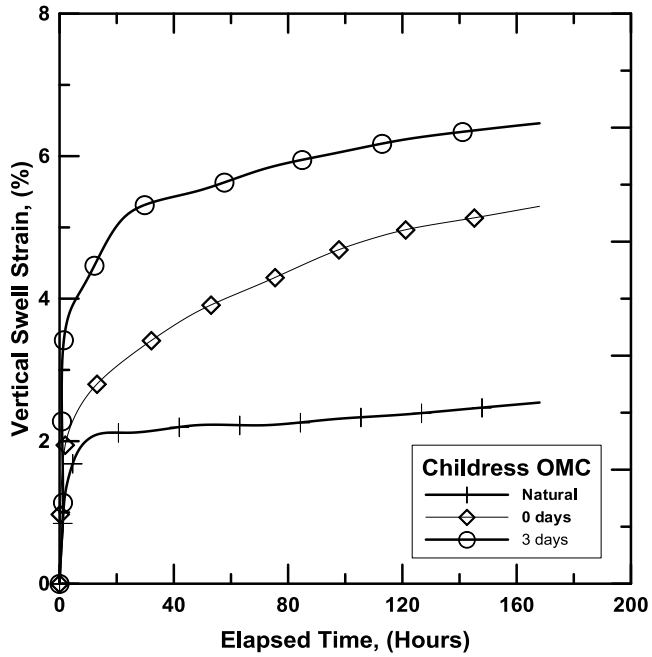


Figure 4.26 Vertical Swell Strain Results for 6% Lime Treated Childress Soil at OMC

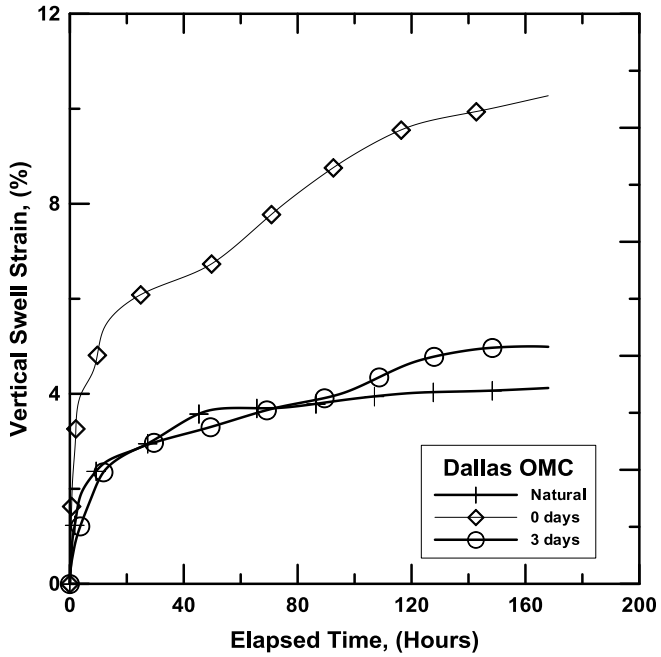


Figure 4.27 Vertical Swell Strain Results for 6% Lime Treated Dallas Soil at OMC

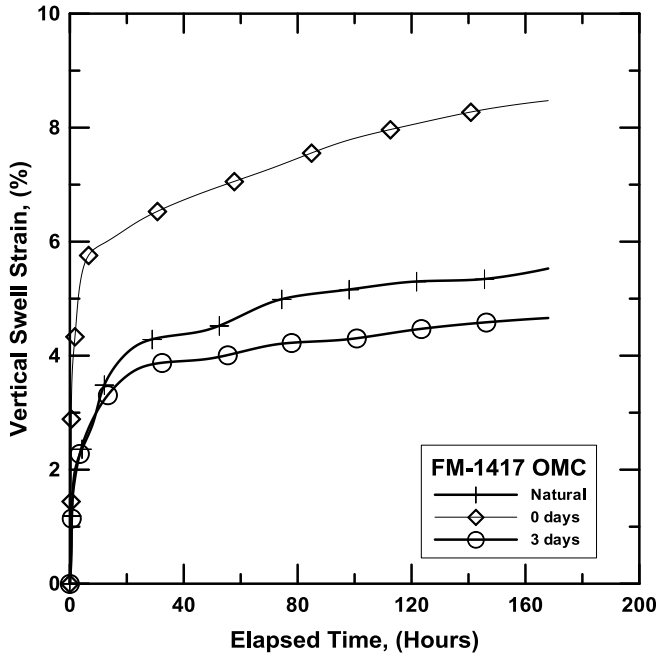


Figure 4.28 Vertical Swell Strain Results for 6% Lime Treated FM-1417 Soil at OMC

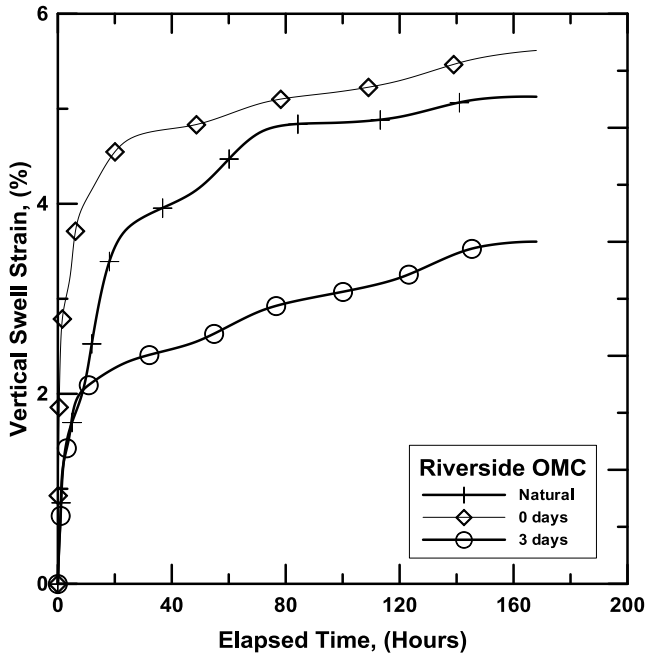


Figure 4.29 Vertical Swell Strain Results for 6% Lime Treated Riverside Soil at OMC

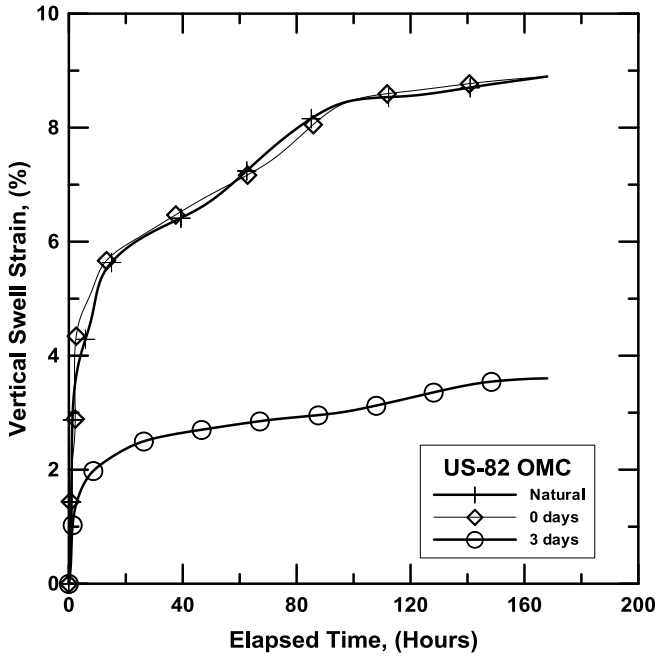


Figure 4.30 Vertical Swell Strain Results for 6% Lime Treated US-82 Soil at OMC

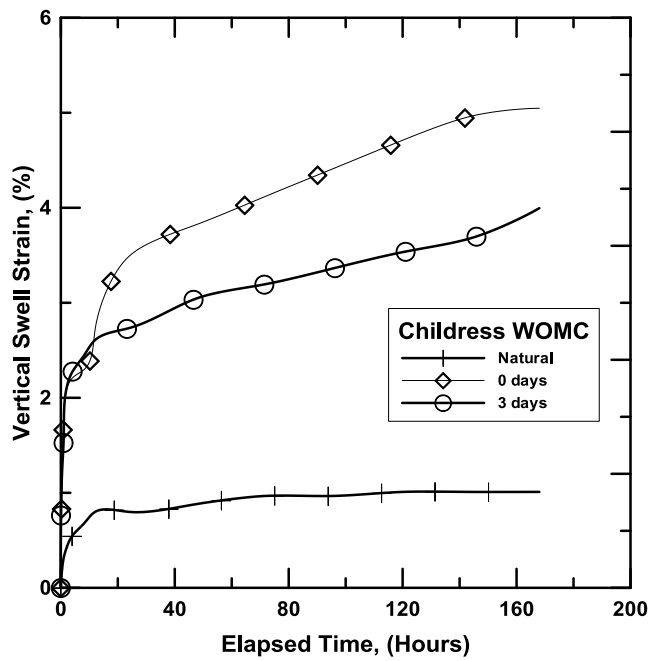


Figure 4.31 Vertical Swell Strain Results for 6% Lime Treated Childress Soil at WOMC

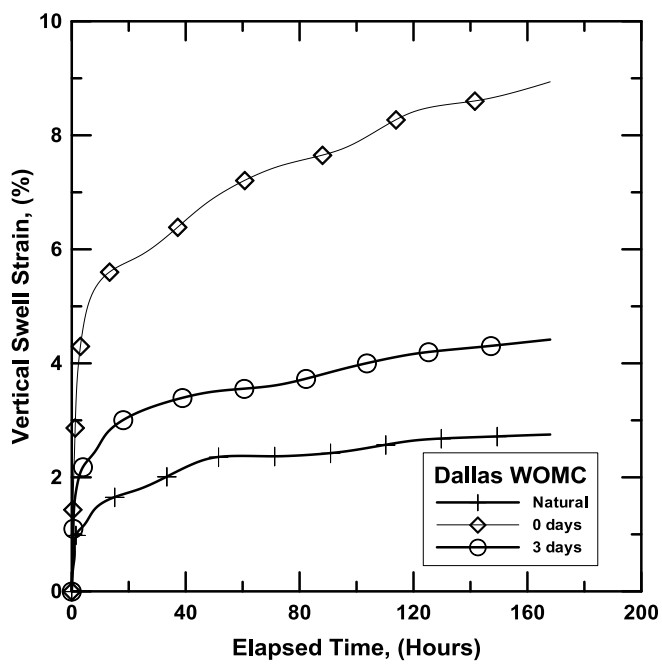


Figure 4.32 Vertical Swell Strain Results for 6% Lime Treated Dallas Soil at WOMC

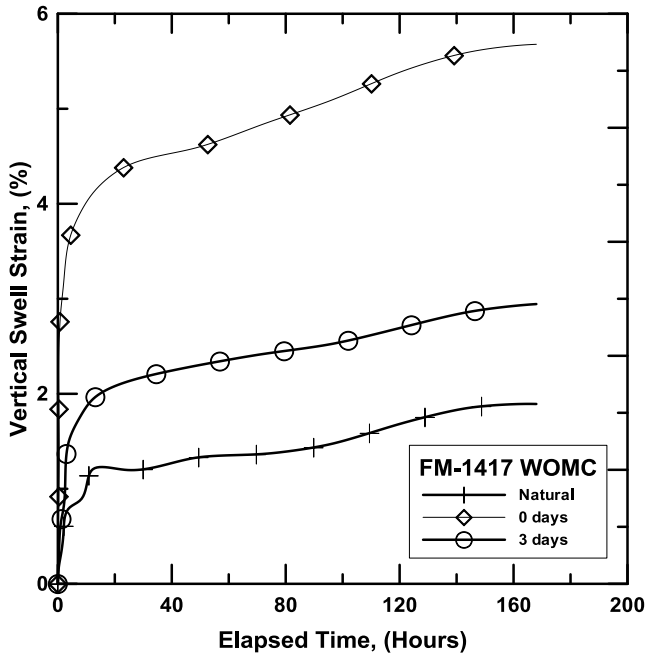


Figure 4.33 Vertical Swell Strain Results for 6%Lime Treated FM-1417 Soil at WOMC

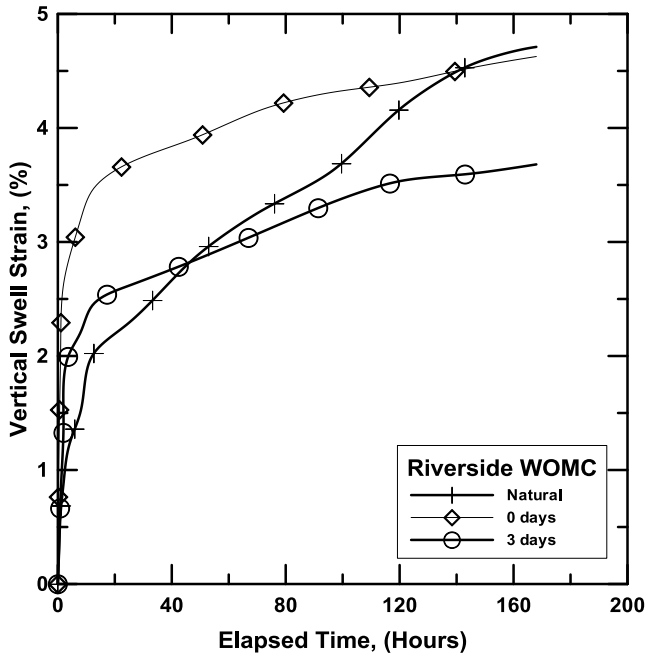


Figure 4.34 Vertical Swell Strain Results for 6% Lime Treated Riverside Soil at WOMC

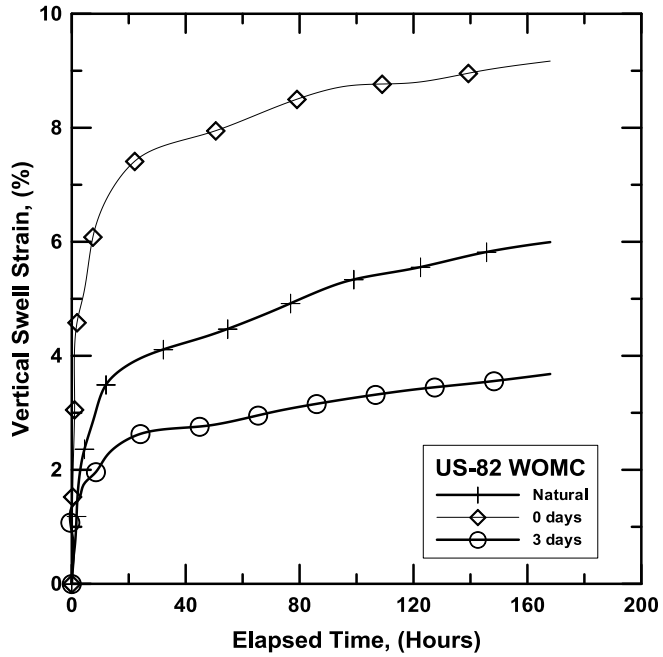


Figure 4.35 Vertical Swell Strain Results for 6% Lime Treated US-82 Soil at WOMC

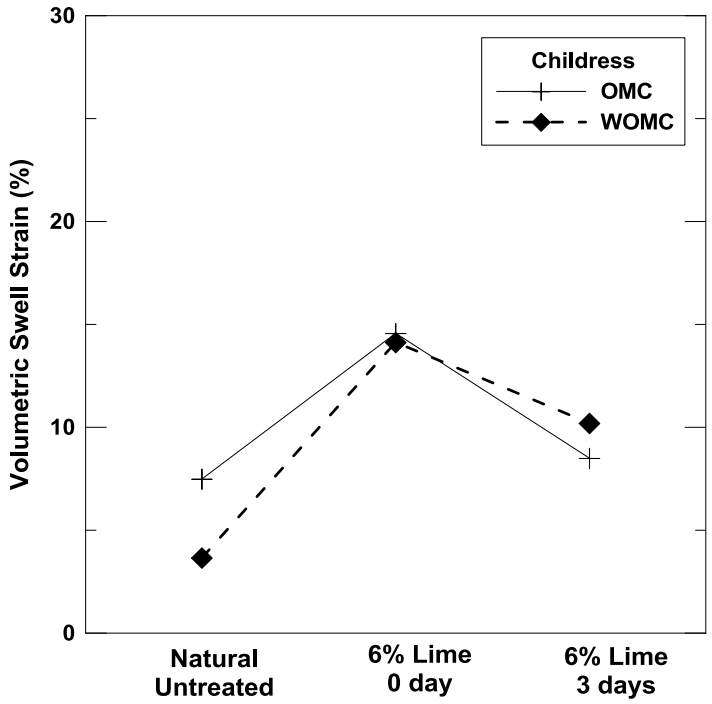


Figure 4.36 Volumetric Swell Strain Results for Childress Soil

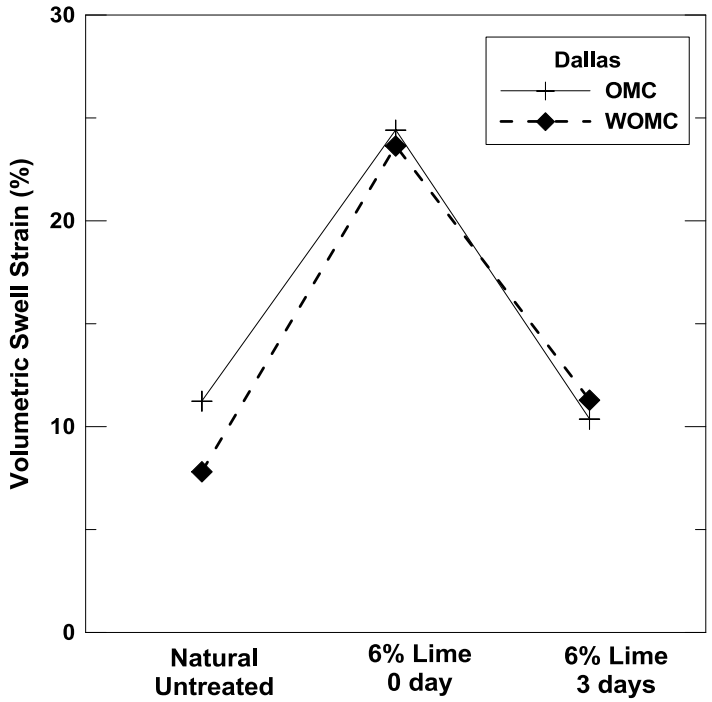


Figure 4.37 Volumetric Swell Strain Results for Natural Dallas Soil

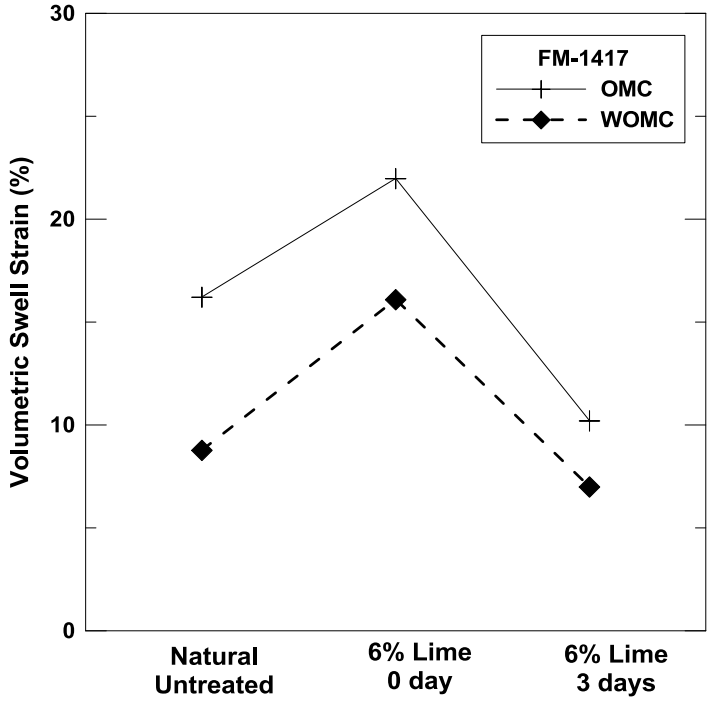


Figure 4.38 Volumetric Swell Strain Results for FM-1417 Soil

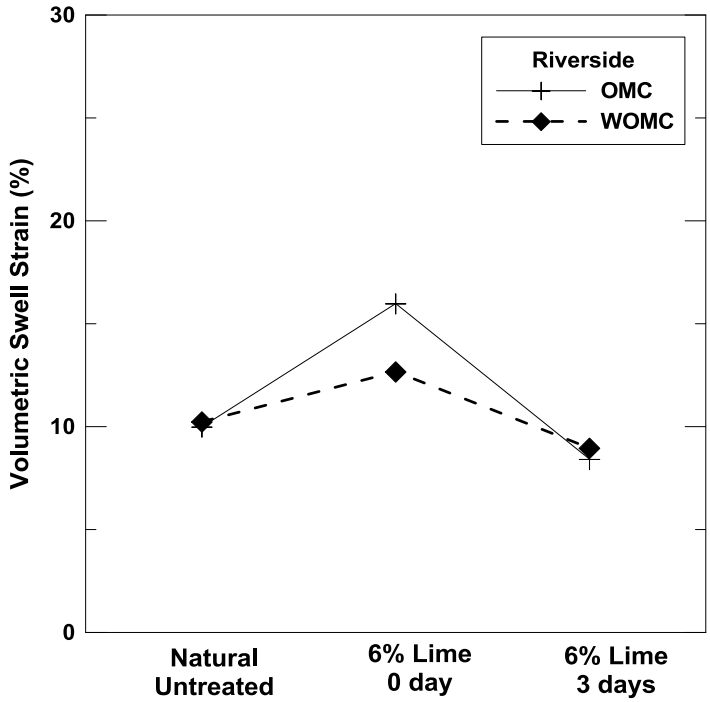


Figure 4.39 Volumetric Swell Strain Results for Riverside Soil

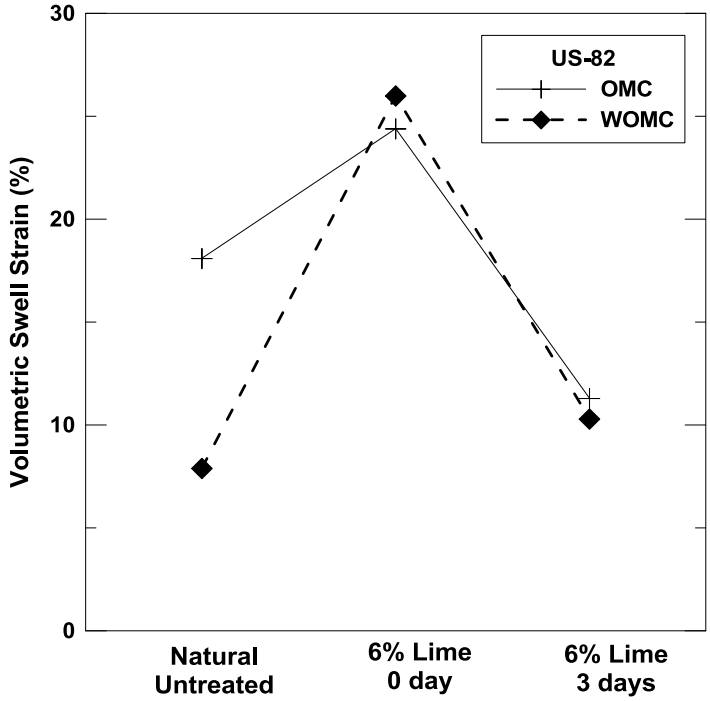


Figure 4.40 Volumetric Swell Strain Results for US-82 Soil

4.2.3 Three-Dimensional (3-D) Volumetric Shrinkage Test Results

The 3-D shrinkage test measures the potential of the clay to shrink in the radial and vertical directions when subjected to drying. This test was carried out according to the procedures described in Section 3.3.3. Table 4.8 presents the numeric values of the volumetric shrinkage strains at two (2) different moisture contents OMC and wet OMC. Figures 4.41 through 4.46 depict the volumetric shrinkage strains of the six soil types under investigation per this research when conditioned to the OMC and wet of OMC moisture contents.

Table 4.8 Volumetric Shrinkage Strain Test Results

Soil	Natural Untreated		6 % Lime Treated			
			0 day		3 days	
	OMC	WOMC	OMC	WOMC	OMC	WOMC
Austin	-9.0	-12.1	-4.8	-8.6	-5.4	-9.7
Childress	-14.3	-15.4	-2.4	-2.6	-3.1	-4.0
Dallas	-20.6	-25.1	-8.8	-11.0	-8.6	-11.0
FM-1417	-15.8	-20.0	-7.6	-8.9	-6.3	-9.4
Riverside	-13.2	-14.5	-3.1	-3.0	-4.6	-3.8
US-82	-13.613	-20.596	-7.080	-8.497	-9.788	-10.382

Note: Negative sign indicate shrinkage.

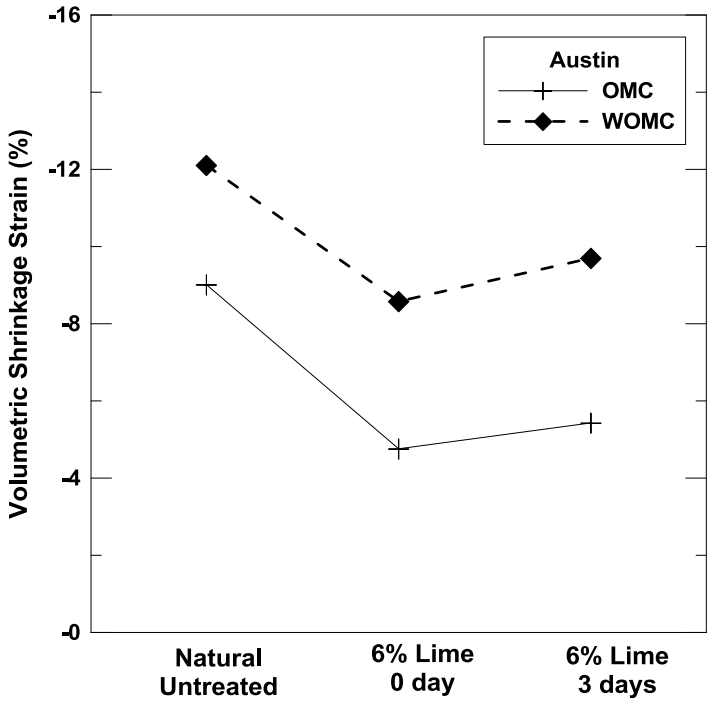


Figure 4.41 Volumetric Shrinkage Strain Results for Austin Soil

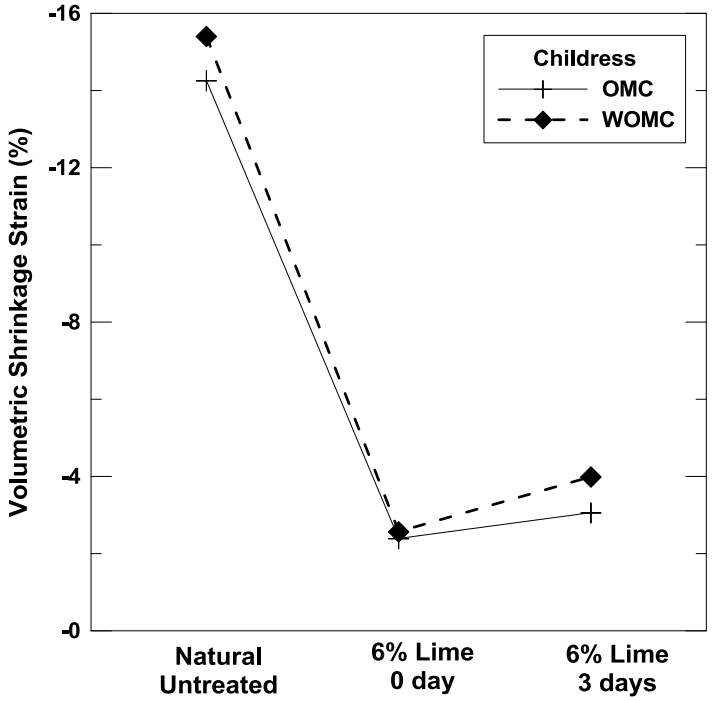


Figure 4.42 Volumetric Shrinkage Strain Results for Childress Soil

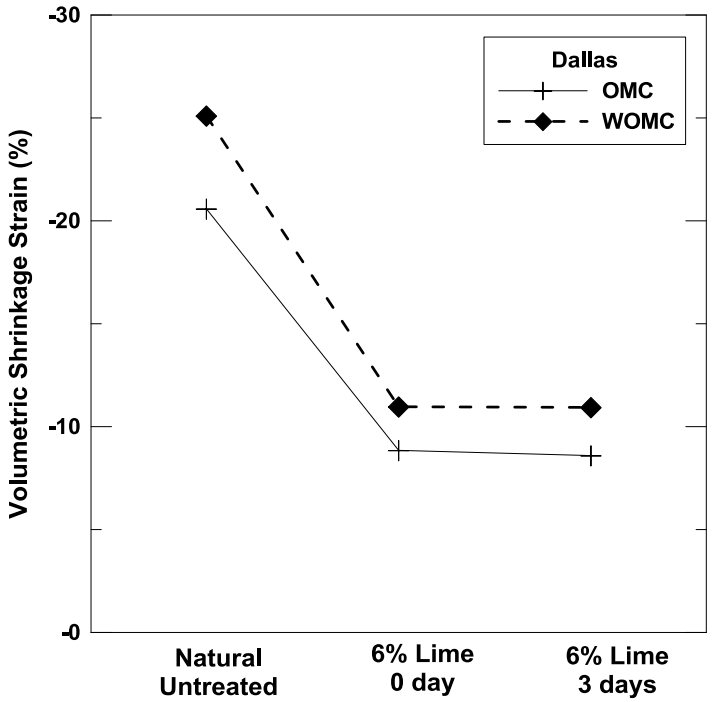


Figure 4.43 Volumetric Shrinkage Strain Results for Dallas Soil

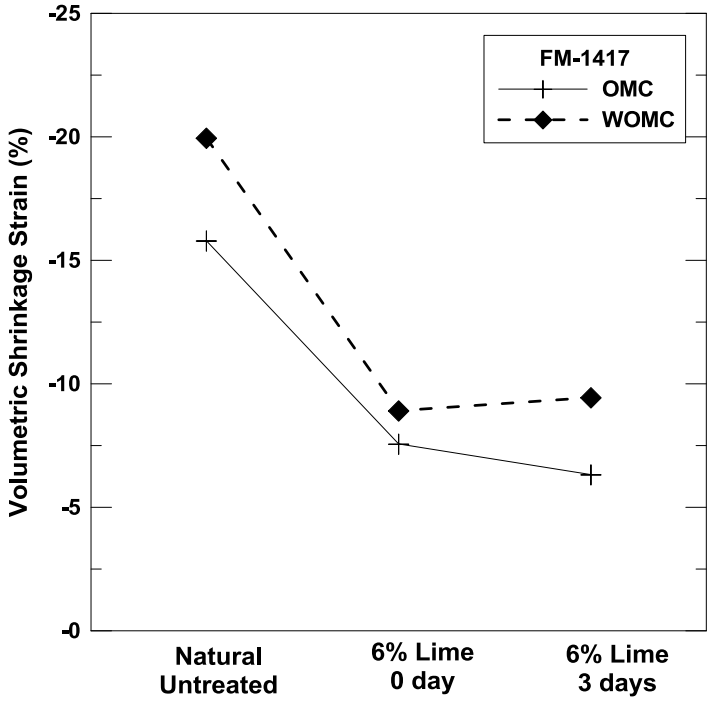


Figure 4.44 Volumetric Shrinkage Strain Results for FM-1417 Soil

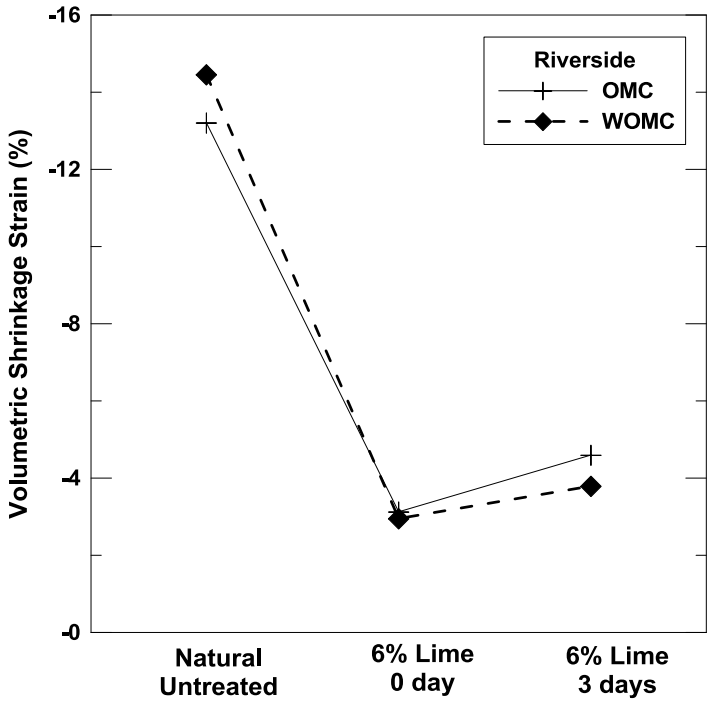


Figure 4.45 Volumetric Shrinkage Strain Results for Riverside Soil

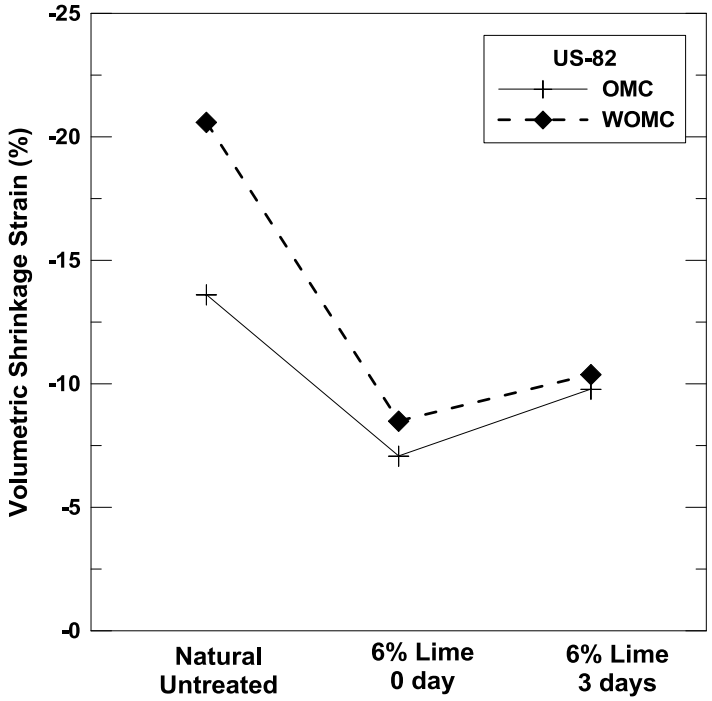


Figure 4.46 Volumetric Shrinkage Strain Results for US-82 Soil

These test results indicate that the present soils experience large volumetric shrinkage strains when subjected to drying. Such strains are expected to induce problematic soil conditions in the field during drought type situations. The volumetric shrinkage for the lime treated soils with three (3) days mellowing period is higher than one (1) days of mellowing, but less than the natural untreated soils. Volumetric shrinkage behavior is following the same trend of UCS behavior since the three (3) days strength is less than one (1) days strength. Provision of addition of 3% moisture could be reason for higher percentage of shrinkage in case of three (3) days mellowing period.

4.4 Chemical Test Results

Chemical tests have been conducted on all the six soils by performing the procedures explained thoroughly in sections 3.4.1 through 3.4.5. These tests include Specific Surface Area (SSA), Cationic Exchange Capacity (CEC) and Total Potassium (TP) measurements. Results of the chemical tests are summarized in Table 4.9.

Based on chemical testing, it can be concluded that except Childress and Riverside all the soils are capable of swelling. Riverside soil is low plasticity clay (CL) whereas Childress soil is a high plasticity silt (MH).

Table 4.9 Chemical Test Results and Mineral Percentages

Soil	Cationic Exchange Capacity (meq/100g)	Specific Surface Area (m^2/g)	Total Potassium (%)
Austin	78.5	242	0.87
Childress	28.7	140	1.10
Dallas	54.7	236	0.91
FM1417	65.2	280	0.79
Riverside	35.2	173	1.27
US82	66.5	284	0.82

4.3.1 Soluble Sulfate Determination Test Results

The soluble sulfate determination test using TxDOT method Tex-145-E was conducted on the soil specimens, both before and after subjecting them to 3-D swell tests. The initial soluble sulfate determination tests (instantly after addition of the calcium sulfate to the soil specimens) were conducted to measure the amounts of soluble sulfate contents of the soil specimens. These measurements are used to confirm that the soil specimens meet with the targeted soluble sulfate levels. Table 4.10 shows the basic soil properties and the sulfate level of each soil type.

Table 4.10 Basic Soil Properties and Sulfate Level

Soil Region	Plasticity Index (PI)	USCS Classification	Sulfate Level ppm
Austin	51	CH	36,000
Childress	36	MH	44,000
Dallas	45	CH	12,000
FM1417	42	CH	24,000
Riverside	24	CL	20,000
US-82	50	CH	12,000

4.5 Effects of Sulfate Loss on Swell Behavior of Treated Soils

After soil specimens were subjected to the 3-D swell tests, the soluble sulfate determination tests were repeated on the same soil specimens in order to determine the percent loss in soluble sulfate contents. The amount of soluble sulfate content loss (in %) was calculated based on the following equation.

$$\%Sulfate\ Loss = \left(\frac{Initial\ Sulfate - Final\ Sulfate}{Initial\ Sulfate}\right) * 100 \qquad (4.1)$$

Where %Sulfate Loss = Percent Soluble Sulfate Loss after Swell Test.

It is well known that in treated soils the soluble sulfate reacts with calcium, clay alumina and water to form Ettringite which is evidenced by the increase in volumetric swell compared to natural soils. In treated soils with different mellowing periods, the sulfate loss varied from 32% to 90%. Highest

sulfate loss (90%) in observed in Riverside soil whereas lowest sulfate loss is observed in Childress soil (33%). Percent sulfate loss for treated soils with varying mellowing periods are presented in Table 4.11

Table 4.11 Percentage Sulfate Loss in 6 % Lime Treated High Sulfate Soils

Soil	Initial Sulfate (ppm)	0 day mellowing				3 days mellowing			
		Final sulfate levels				Final sulfate levels			
		OMC	% loss	WOMC	% loss	OMC	% loss	WOMC	% loss
Austin	36,000	24,600	32	*N/A	*N/A	18,800	48	*N/A	*N/A
Childress	44,000	23,000	48	26,600	40	29,600	33	12,400	72
Dallas	12,000	1,600	87	1,800	85	2,200	82	1,600	87
FM1417	24,000	2,600	89	2,800	88	3,600	85	4,000	83
Riverside	20,000	2,000	90	2,200	89	3,200	84	3,600	82
US82	12,000	1,800	85	1,600	87	4,400	63	4,000	67

* Note: Due to the limited availability of Austin type soil, fewer tests were performed.

The percent sulfate loss values are in good agreement with the 3-D swell values observed. In all of the six soils except Austin soil, with 0 day mellowing lime treatment resulted in more swell than the natural soil. The percent sulfate loss in these soils is as high as 90%. With 0 day mellowing, most of the soluble sulfate participated in the reaction leading to Ettringite formation and increased swell. Whereas in case of 3 days mellowing the percent sulfate loss values are slightly lower indicating lower reaction rates as well as lower swell. In Austin soil, the increase in mellowing period resulted in higher swell values. This fact can be explained by the higher sulfate loss (48%) in 3 days mellowing compared to 0 day mellowing (33%). To see the effect of extended mellowing, 3-D swell tests were carried out on US82 and FM1417 soils with a 7 days mellowing period. The sulfate loss in 7 days mellowing is varying from 52% to 73%. Also, the percent sulfate loss with 7 days mellowing is lower than the 3 days and 0 day mellowing. The observed volumetric swell values in this case are the lowest.

Interestingly in two cases, (Dallas, 0 day mellowing and US82, 3 days mellowing) the observed volumetric swell at wet of optimum is more than the swell at optimum moisture content. In this case the sulfate loss is calculated and it is found to be higher at wet of optimum moisture content. Higher water content in wet optimum condition could have resulted in higher sulfate dilution leading to higher volumetric swell.

4.6 Ettringite Induced Heaving at 0 and 3 Days Mellowing Period

To see the effect of mellowing on lime stabilization in this study, two mellowing periods (0 and 3 days) are considered. The soil samples are treated with 6% lime and allowed to mellow for required period followed by sample moulding. These samples are tested for 3-D volumetric swell.

It has been observed that 0 day mellowing resulted in more swell than the natural soils which confirm the heave caused by Ettringite. In all the soils except Austin, the observed heave is higher than natural. 3-D volumetric swell tests were carried out on all soils with 3 days mellowing period. Followed by 3 days mellowing, five of the six soils exhibited swell less than the natural swell. In case of Austin soil, 3days mellowing resulted in more swelling than 0 day mellowing but lower than natural swell. Figures 4.47 (a) through (f) gives the observed volumetric swell at OMC and WOMC for each soil.

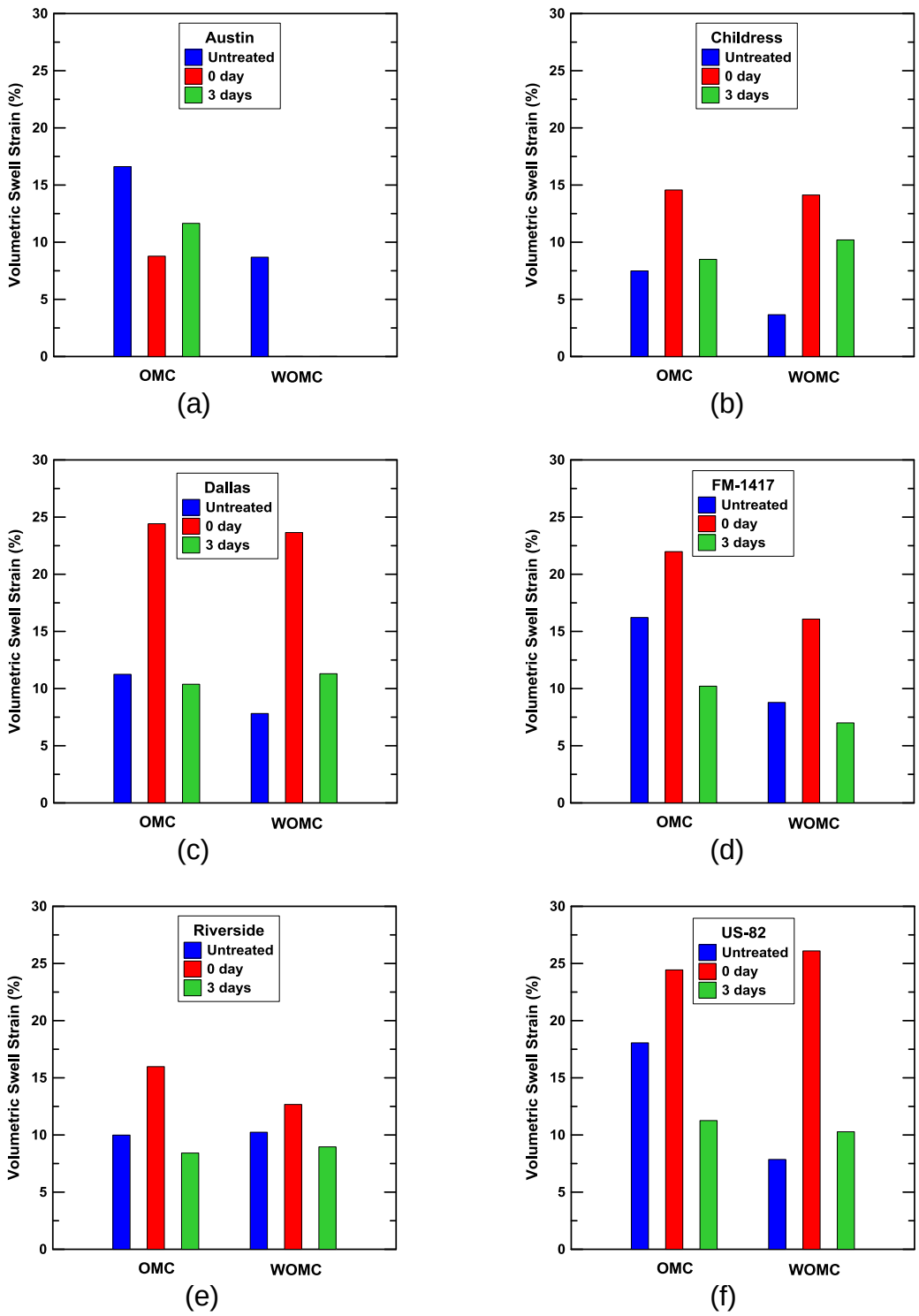


Figure 4.47 (a), (b), (c), (d), (e) and (f) Volumetric Swell Strain Bar Charts

4.7 Effect of Soil Type and Sulfate Level on Ettringite Induced Heave

To see the effect of soil type on Ettringite induced heave, the soils have been divided into three categories. They are low plasticity clay, high plasticity clay and silty soil. To see the effect of sulfate level the soils have been divides into two categories, sulfate level below 30,000 ppm and sulfate level above 30,000 ppm.

From Table 4.7 it can be observed that with 0 day mellowing, lime treatment resulted in higher swell in all of the 5 of the test soils. This behavior is an indication of Ettringite formation and subsequent growth. The other soil which responded well to lime treatment is Austin, in which 0 day mellowing effectively reduced the swell below natural level.

As mentioned earlier, a different mellowing period of 3 days is used in the current study. From Figures 4.47 (a) through (f) it can be seen that the 3 days mellowing period reduced the swell below natural swell level for four of the six test soils which are of clay type. The four soils that responded well to the 3 days mellowing consist of one low plasticity and three high plasticity clays. One more interesting observation is that all the four soils are high sulfate soils with sulfate levels below 30,000 ppm.

The two soils that did not respond well to the 3 days mellowing are Austin and Childress with sulfate levels above 30,000 ppm. Austin soil is high plasticity clay with sulfate level of 36,000 ppm. In this case the extended

mellowing of 3 days did not work well either. The other soil is Childress which is of silty origin with highest sulfate level of 44,000 ppm.

4.6.1 Effects of 7 Days Mellowing Period on Swelling

Since the 3 days mellowing time worked fine for four soils from clayey origin it is decided to try an extended mellowing period of 7 days on two soils. The two soils that were tested are US-82 and FM-1417 both from Sherman county of Paris district. The observed swell values with 7 days mellowing are well below the natural swell. So, it can be concluded that the extended mellowing is a good technique for suppression of sulfate induced heave in clay soils with sulfate level below 30,000 ppm. Figures 4.48 and 4.49 below show the volumetric swell bar charts for the noted two soils.

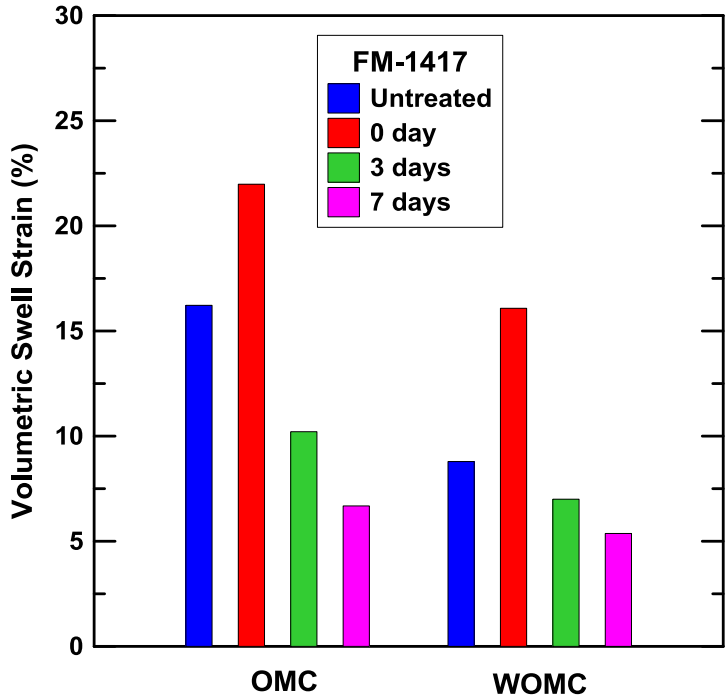


Figure 4.48 FM-1417 Soil Volumetric Swell Strain Results

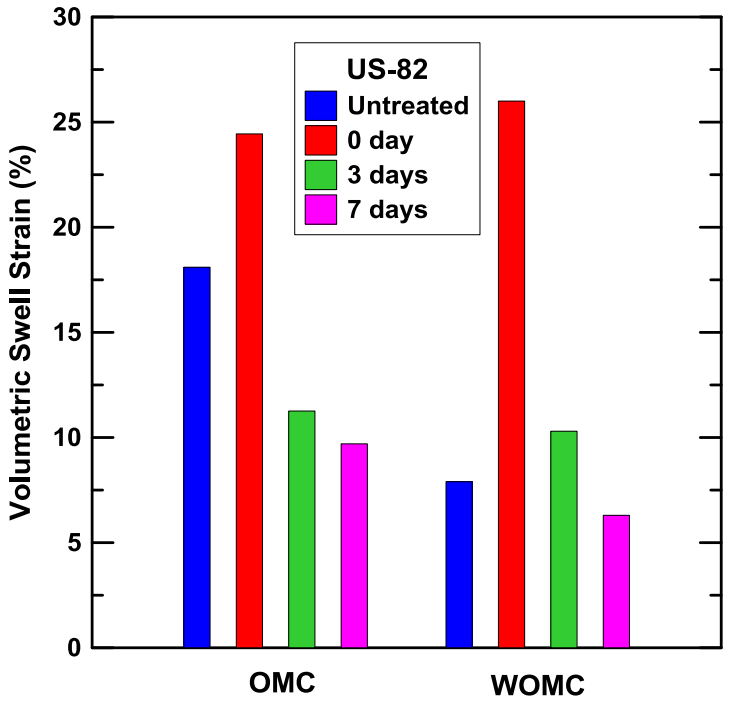


Figure 4.49 US-82 Soil Volumetric Swell Strain Results

4.8 Summary

In this chapter various test results from the procedures performed in the current research to determine the engineering properties of both control and treated soils were presented. The chemical test results were also provided and explained in detail. An explanation of the effect of soil type and sulfate level on Ettringite induced heave were also discussed as a result of the engineering and the chemical tests. Also, for different mellowing periods, the Ettringite induced heaving was explained.

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

CHAPTER 5

SUMMARY AND CONCLUSIONS

1.6 Introduction

A total of 6 natural soils from different parts of Texas region were collected, out of these soils five are sulfate-rich and one is control soil. All soils are then subjected to basic geotechnical, engineering and chemical tests. This chapter presents the summary of the results and the conclusions drawn from the analysis shown in Chapter 4. This chapter also describes further recommendations that can be incorporated into future research.

1.7 Summary and Conclusions

Based on the experimental results from Chapter 4 the following conclusions are obtained:

- The unconfined compressive strength of 6% lime treated soils is much higher than the same of natural soils.
- Mellowing does not have a significant effect on strength improvements because the UCS strength of three days mellowed lime treated soils is found to be lower than the same treated soil with 0 day mellowing. This variation is attributed to the additional moisture content provided in the preparation of lime treated soil at three days mellowing.

- At optimum moisture content, with the exception of Childress soil, the lime treated three days mellowing is effective in reducing the volumetric swell strains for all the soils when compared to natural and lime treated with no mellowing. This can be attributed to the fact that Childress soil is silty in nature and has very large amounts of sulfates.
- Provision of additional moisture content (3% extra) in the case of three days mellowing proved to be effective in reducing the swell of lime treated high plasticity soils below natural levels. At wet of optimum moisture content, the effect of mellowing is not significant in swell reduction.
- For three days mellowing period the volumetric swell reduced below natural swell level for four of the six test soils which are predominantly clays. Another interesting feature is that all four soils that showed improvements have sulfate levels below 30,000 ppm. Provision of seven days mellowing in two of the test soils resulted in the reductions of the volumetric swell strains below a reference value of 10%.
- A significant loss in sulfates is observed in high plasticity clays in which the sulfate heave was detected. The observed sulfate loss in 6% lime treated soils followed the same trend as volumetric swell in high plasticity clays.
- The volumetric shrinkage strains are less for all the lime treated soils when compared to natural soils. The magnitude of volumetric shrinkage

strains of all untreated and treated soils is higher at wet of optimum when compared to the same soils compacted at optimum moisture content. The effect of mellowing period is not a major factor in shrinkage strain reduction since the three days shrinkage strain is more than 0 day shrinkage in almost all the cases and this is due to higher amounts of moistures added to the soils compacted at three days mellowing period.

- Overall, for soils with sulfates less than 30,000 ppm, lime treatment with three days mellowing has resulted in lesser swell strains than natural soils; however the improvements are not significantly. For soils with sulfates more than 30,000 ppm, the lime treatment with three days mellowing period has resulted in no improvements.

1.8 Recommendations for Future Research

1) More chemical tests and analysis needs to be done on soils with high sulfate levels to determine the chemistry and structure of minerals that contribute to heaving mechanisms in the present test soils.
2) For soils with sulfates more than 30,000 ppm, other chemical treatments need to be considered and evaluated for their effectiveness of stabilizing them.
3) The effect of curing period and temperature on treated soils with different mellowing periods needs to be studied.
4) Develop a method to determine the rate of Ettringite formation in sulfate-rich soils.

5) Further analysis by using a combination of X-ray diffraction (XRD), spectroscopic, thermal analysis and selective dissolution techniques for identification and quantification of the minerals in sulfate-bearing soils is recommended.

REFERENCES

1. Adams, A.G., Dukes, O.M., Cerato, A.B., and Miller, A.G. (2008). “Sulfate Induced Heave in Oklahoma Soils due to Lime Stabilization”. Geotechnical Special Publication, n 179, p 444-451, (2008), Proceedings of Sessions of GeoCongress 2008 - GeoCongress 2008: Characterization, Monitoring, and Modeling of GeoSystems, GSP 179
2. Berger, E., Little, D.N., and Graves, R. (2001) “Technical Memorandum: Guidelines for Stabilization of Soils Containing Sulfates”. www.lime.org/publications.html.
3. Chakkrit, S., Puppala, A.J., Vivek, C., Sireesh, S., and Hoyos, L.R. (2008). “Combined Lime and Cement Treatment of Expansive soils with Low to Medium Soluble Sulfate Levels”. Geotechnical Special Publication, n 178, p 646-653, 2008, Proceedings of session of GeoCongress 2008 - GeoCongress 2008: Geosustainability and Geohazard Mitigation, GSP 178.
4. Chen, D.H., Harris, P., Scullion, T., and Bilyeu, J. (2005). “Forensic Investigation of a Sulfate-Heaved Project in Texas”. Journal of Performance of Constructed Facilities, v 19, n 4, p 324-330, November 2005.

5. Chen, F. H. (1988). “Foundations on Expansive Soils,” Developments in geotechnical engineering, Vol.12, pp.12.
6. Dermatas, D.(1995). “Ettringite-induced Swelling in Soils: State-of-the-art.” Applied Mechanics Rev, Vol. 48, pp. 659-673.
7. Harris, J. P., Sebesta, S., and Scullion, T. (2004). “Hydrated lime stabilization of sulfate-bearing vertisols in Texas.” Transportation Research Record. 1868, Transportation Research Board, Washington, D.C., 31–39.
8. Harris, P., Holdt, J.V., Sebesta, S., and Scullion, T.(2006) “Recommendations for stabilization of high sulfate soils in Texas”. Texas Department of Transportation, FHWA/TX-06/0-4240.
9. Harris, P., Holdt, J.V., Sebesta, S., and Scullion, T.(2006). “Database of sulfate stabilization projects in Texas”. Texas Department of Transportation, FHWA/TX-06/0-4240-4.
10. Hausmann, M. R. (1990). “Engineering principles of ground modification,” McGraw-Hill, New York. 632 p.
11. Hawkins, A. B. (1998). “Engineering Significance of Ground Sulfates.” Geotechnical Site Characterization, pp. 685-692.
12. 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

13. Hoyos, L.R., Lakiram, A., and Puppala, A.J., (2006). “Behavior of chemically stabilized sulfate-rich expansive clay under quick-aging environment”. Geotechnical Special Publication, n 152, p 89-96, 2006, Ground Modification and Seismic Mitigation - Proceedings of the GeoShanghai Conference.
14. Hoyos, L.R., Puppala, A.J., and Phonalawut, C. (2004). “Dynamic Properties of Chemically Stabilized Sulfate Rich Clay”. Journal of Geotechnical and Geoenvironmental Engineering, v 130, n 2, p 153-162, February 2004.
15. Hunter, D. (1988). “Lime-induced heave in sulfate-bearing clay soils,” Journal of Geotechnical Engineering, ASCE, 114(2), pp.150-167.
16. Intharasmbat(2003). “Ettringite formation in lime-treated sulfate soils: Verification by mineralogical and swell testing”. The University of Texas at Arlington, 2003, 117 pages; AAT 1414753.
17. Little, D.N., Nair, S., and Herbert, B. (2010). “Addressing Sulfate-Induced Heave in Lime Treated Soils”. Journal of Geotechnical and Geoenvironmental Engineering, v 136, n 1, p 110-118.
18. Mitchell, J. K. (1986). “Practical problems from surprising soil behavior,” ASCE Journal of Geotechnical Engineering, Vol. 112, No.3, p.259-289.
19. Mitchell, J. K. and Dermatas, D. (1992). “Clay soil heave caused by lime-sulfate reactions,” ASTM STP 1135: Innovations and Uses for Lime, Philadelphia, Pennsylvania.

20. Nelson, D. J. and Miller, J. D. (1992). “Expansive soils: problems and practice in foundation and pavement engineering,” John Wiley & Sons. 259 p.
21. Perrin, L. (1992). “Expansion of Lime-treated Clays Containing Sulfates.” ASCE Expansive Soils Research Council, New York, Vol. 1, p. 409-414.
22. Petry, T. (1994). “Studies of Factors Causing and Influencing Localized Heave of Lime-Treated Clay Soils (Sulfate-Induced Heave),” Technical Report for U.S. Army Corps of Engineers, Waterways Experiment Station, Vicksburg, MS.
23. Puppala, A.J., and Cerato, A. (2009). “Heave Distress Problems in Chemically- Treated Sulfate Laden Materials”. Geo-Strata, March/April , 2009
24. Puppala, A.J., and Musenda, C. (2000). “Effects of Fiber Reinforcement on Strength and Volume Change Behavior of Expansive Soils.” Transportation research board 79thAnnual Meeting, paper No. 00-0716, Washington, D.C.
25. Puppala, A.J., Griffin, J.A., Hoyos, L.R., Chomtid, S. (2004). “Studies on sulfate resistant cement stabilization methods to address sulfate induced heave problems.” Journal of Geotechnical and Geoenvironmental Engineering, v 130, n 4, p 391-402, April 2004.

26. Puppala, A.J., Hanchanloet, S., Jadeja, M., Burkart, B. (1999). “Sulfate Induced Heave Distress: A Case Study. Proceedings,” Transportation Research Board Annual Meeting, Washington D.C, USA.
27. Puppala, A.J., Intharasombat, N., and Vempati,R.K.(2005). “Experimental Studies on Ettringite - Induced Heaving in Soils”. Journal of Geotechnical and Geoenvironmental Engineering, v 131, n 3, p 325-337, March 2005.
28. Puppala, A.J., Rupesh, K., Raja, S.M., and Hoyos, L.R. (2006). “Small-Strain Shear Moduli of Chemically Stabilized Sulfate-Bearing Cohesive Soils”. Journal of Geotechnical and Geoenvironmental Engineering, v 132, n 3, p 322-336, March 2006.
29. Puppala, A.J., Wattanasanticharoen, E., Venkata, S.D., Hoyos, L.R. (2007). “Ettringite Induced Heaving and Shrinking in Kaolinite Clay”. Geotechnical Special Publication, n 162, p 7, 2007, Proceedings of Sessions of Geo-Denver 2007 Congress: Problematic Soils and Rocks and In Situ Characterization.
30. Puppala, A.J., Wattanasantichatoen E., Intharasombat L., Hoyos L.R. (2003). “Studies to Understand Soil Compositional and Environmental Variables Effects on Sulfate Heave Problems,” 12th Panamerican Conference on Soil Mechanics and Geotechnical Engineering.

31. Rajasekaran, G. (2005). “Sulphate attack and Ettringite formation in the lime and cement stabilized marine clays”. Ocean Engineering, v 32, n 8-9, p 1133-1159, June 2005.
32. Rollings,R.S., Rollings,M.P., Poole, Toy., Wong, G.S., and Gutierrez, Gene (2006). “Investigation of Heaving at Holloman Air Force Base, New Mexico”. Journal of Performance of Constructed Facilities, v 20, n 1, p 54-63, February 2006
33. Sherwood, P.T. (1962). “Effect of Sulfates on Cement and Lime Treated Soils.” Highway Research Board, Bulletin 353, pp. 98-107.
34. Si, Z. (2008). “Forensic Investigation of Pavement Premature Failure due to Soil Sulfate-Induced Heave”. Journal of Geotechnical and Geoenvironmental Engineering, v 134, n 8, p 1201-1204, August 2008.
35. Wang, L. (2002). “Cementitious Stabilization of Soils in the Presence of Sulfate,” Ph.D. Thesis, Louisiana State University.

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