EVALUATING THE PERFORMANCE OF CONTROLLED LOW STRENGTH MATERIALS (CLSMs) PREPARED USING SULFATE SPIKED HIGH PLASTICITY CLAY

by

SADIKSHYA POUDEL

Presented to the Faculty of the Graduate School of

The University of Texas at Arlington in Partial Fulfillment

of the Requirements

for the Degree of

MASTERS OF SCIENCE IN CIVIL ENGINEERING

THE UNIVERSITY OF TEXAS AT ARLINGTON

December 2013

ACKNOWLEDGEMENTS

Firstly, I would like to thank the Tarrant Regional Water District (TRWD) and the Integrated Pipeline (IPL) team for granting this opportunity to be a part of the research project via the University of Texas at Arlington's geotechnical engineering group under Dr. Anand Puppala. I also thank the University of Texas at Arlington for appointing me to the Graduate Research Assistant position, allowing me to be involved in research program and providing with the resources needed to procure the desired objectives while pursuing my graduate degree in Civil Engineering. I extend my sincere thanks to my advisor, Dr. Anand Puppala and Dr. Bhaskar Chittoori for their continuous encouragement and guidance throughout this research. Without their assistance, the research and the outcomes would not have been possible. In addition, I am indebted towards the university and Dr. Puppala for the financial assistance provided throughout by graduate studies. I am also thankful to Dr. Laureano R. Hoyos and Dr. Xinbao Yu for being a part of my thesis defense committee along with my advisor, Dr. Puppala. I am also thankful to Dr. Aravind Pedarla for his encouragement and help during thesis writing.

My heartfelt thanks to all the members of Dr. Puppala and Dr. Yus' research team namely Spoorthi Reballi, Rathna P. Mothkuri, Tejovikas Bheemasetti, Pinit (Tom) Ruttanaporamakul, Justin D. Thomey, Asheesh Pradhan, Raju Acharya, Chatuphat (Book) Savigamin, Durga Praveen Reddy Vanga, Ujwal K. Patil, Ahmed Gaily, Omar Pena, Kelsey Fort, Jorge I. Almendares, Humberto Johnson, Nagasreenivasu Talluri and Victor (Zhang Nan).

Lastly, I would like to thank my parents for their continuous support and encouragement towards higher education. Also, I would like to extend my sincere thanks to my friend and sister, Sebina Shrestha and Anuja Ghimire, for their love and support all throughout life. Finally I would like to thank my fiancé, Benjamin Steven Stach, for his tireless support, continuous encouragement and love during my graduate studies and in life.

November 18, 2013

ABSTRACT

EVALUATING THE PERFORMANCE OF

CONTROLLED LOW STRENGTH MATERIALS (CLSMs)

PREPARED USING SULFATE SPIKED

HIGH PLASTICITY CLAYS

Sadikshya Poudel, M.S.

The University of Texas at Arlington, 2013

Supervising Professor: Anand J. Puppala

When large scale civil engineering projects such as Integrated PipeLine (IPL) are to be constructed, several factors play a significant role in the successful completion of the project on time and on budget. With pipeline construction, one of the important tasks is excavation, with excavation comes the hauling of the excess excavated trench material to the disposal site. The hauling of unwanted excavated dirt from the site and bringing in select backfill material that meets the project requirements not only adds to the overall cost of the project but also raises concerns about sustainability. Transport of unwanted excavated materials require landfill space, contribute to air pollution by carbon emission from transport vehicles and also cause damage to pavements by the heavy loads being hauled.

In order to account for cost as well as sustainability, Tarrant Regional Water District (TRWD) initiated a research study involving the reusability of in-situ native soil as bedding, haunch and backfill material for the IPL project. One part of the study involved the use of native soil treated with a stabilizer, cement, to prepare a flowable fill or Controlled Low Strength

Material (CLSM). CLSM mix design using native soil not only cuts down cost of the hauling and bringing in select fill, it is also very effective when used as a utility bedding material.

The proposed alignment of the IPL project underlines certain areas that contain expansive soil with elevated levels of sulfate concentration such as the Eagle Ford geological formation. Expansive soils with high levels of sulfates have been reported to be problematic all around Dallas/Fort Worth (DFW) metropolis. Sulfate induced heave has been a growing concern in the civil engineering projects employing calcium based stabilizers for soil treatment.

This study focuses on the effects of using high sulfate expansive soil treated with cement in CLSM sample preparation. In order to achieve this goal, short term strength and long term durability studies were conducted on the samples which comprised of strength, volumetric and weight change and leachate studies. For the study, soil from Eagle Ford geological formation was selected and treated with Portland Cement (Type I/II). The five sulfate concentrations studied were 100 or less ppm (control soil), 2500 ppm, 5000 ppm, 10000 ppm and 20000 ppm.

From the study, several significant conclusions were drawn. The analysis showed that soluble sulfates present in soil used for CLSM preparation do not have adverse effect on the 28-day cured unconfined compressive strength of the CLSM sample. This is in agreement with another study reported by the Japanese researchers who utilized recycled gypsum recycled to stabilize the soft clay soil and achieved acceptable Unconfined Compressive Strength (UCS) values (Kamei et al., 2011). The CLSM samples with elevated levels of soluble sulfate in the form of gypsum under short term strength exhibited higher UCS values. However, for the durability studies, the strength decreased significantly with increase in durability wetting-drying cycles.

In addition, the increase in swell-shrink behavior of expansive soil with elevated levels of soluble sulfates was also distinctly reflected from the study when the same samples are subjected to durability cycles. Higher the concentration of soluble sulfates, higher the swell-shrink behavior exhibited by the CLSM which was evident by the samples failing before reaching 14 complete durability cycles. This confirms the effects of sulfates on the volume change and strength loss behavior of CLSM mixes. Furthermore, the study also showed increase in calcium ion leached out with the increase in sulfate concentration in the CLSM sample. This increase in cement loss could be the reason for the loss of strength of CLSM samples during durability studies.

TABLE OF CONTENTS

LIST OF ILLUSTRATIONS

LIST OF TABLES

LIST OF EQUATIONS

CHAPTER 1

INTRODUCTION

1.1 General

Per the population clock developed by the United States Census Bureau (USCB) based on the 2010 Census data, United States (US) has a net gain of one person every twelve seconds (United States Census Bureau, 2010). Texas being one of the largest states in the US, it is not immune to this trend in population growth. In fact, the population of Dallas/ Fort Worth metropolis is likely to surpass 13 million by the year 2060 (IPL Project). With gradual but steady population growth, the demand for increased water supply is inevitable. In order to prevent the shortage of water supply to the growing DFW population, Tarrant Regional Water District (TRWD) and Dallas Water Utilities (DWU) ventured together to design and build a pipeline, approximately 150 miles long and about 9 feet in diameter which is going to bring in additional water from Lake Palestine, Cedar Creek Reservoir and Richland-Chambers Reservoir to avoid future water scarcity in the metropolis. This pipeline construction project is known as the Integrated Pipeline (IPL) Project.

When large scale civil engineering projects such as IPL are to be constructed, cost among other factors plays an important role in the successful completion of the project on time and on budget. Pipeline construction involves excavation and, with excavation comes the hauling of the excess excavated trench material to the disposal site. The transport of unwanted excavated dirt from the site and bringing in select backfill material that meets the project requirements adds to the overall cost of the project. Moreover, the transport of unwanted excavated materials require landfill space, contribute to air pollution by carbon emission from transport vehicles and also cause damage to pavements by the heavy loads being hauled. These issues raise serious concerns about sustainability.

In order to account for cost as well as sustainability, TRWD initiated a research study involving the reusability of in-situ native soil as bedding, haunch and backfill material for the IPL project. One part of the study involved the use of native soil treated with a stabilizer, cement, to prepare a flowable fill or Controlled Low Strength Material (CLSM).

CLSM mix design using native soil not only cuts down cost of the hauling and bringing in select fill, it is also very effective when used as a utility bedding material. Some of the advantages of CLSMs outlined by Trejo et al. (2004) include solid, uniform pipe support, reduced labor costs, reduced trench preparation time, and reduction of water entrance to the bedding-pipe interface. Karduri (2011) performed a cost analysis study that showed that cost increase to $6,003.18 when imported fill material was used instead of using native chemically treated soil for construction purposes. A cost savings of more than 100% was observed when a combination of cement and fly ash or lime was used in lieu of the imported select fill material. Appendix B shows the CLSM being used as bedding material in the IPL project.

Previous research at the University of Texas at Arlington (UTA) has not only established the CLSM mix design using native soils with selective dosages of cement as a stabilizer but also have conducted long term durability studies using soil from four different geological formations along the IPL alignment.

Furthermore, the research performed by Thomey (2013) on sulfate mapping for the pipeline alignment showed the presence of elevated levels of soluble sulfates in the Eagle Ford (EF) formation ranging from 40 parts per million (ppm) to approximately 20,000 ppm. In construction practices, calcium based stabilizers are typically added to the CLSM mix design to increase their shear strength, reduce compressibility and volume changes (Cerato and Miller, 2009). However, when soils containing soluble sulfates are stabilized using calcium based stabilizers such as lime or cement, the sulfates and alumina/silica present in the soil react with calcium resulting in highly expansive minerals such as Ettringite ($Ca_6.[Al(OH)_6]_2.(SO_4)_3.26H_2O$) and Thaumasite ($Ca_6.[Si(OH)_6]_2.(SO_4).(CO_3)_2.24H_2O$). This mechanism is termed as sulfate heaving and was first observed by Sherwood in 1962 but only gained serious attention when it was reported by Mitchell in the mid-1980s. Ettringite contains 26 molecules of water enabling it to swell more than 137% of its volume (Little et al., 2010). Many cases of sulfate-induced heave were reported in Texas in and around DFW metropolis (Perrin, 1992; Puppala et al., 2010).

Due to the concerns relating to sulfate-heaving, the need for research on long term durability performance of CLSM mix designs prepared using native high plasticity clay with various sulfate concentrations is crucial. This thesis summarizes the results of performing durability test on the above mentioned scenario. This will conclude whether CLSM mix design with expansive clay rich in soluble sulfates stabilized with cement is a feasible, durable, economical and sustainable alternative.

1.2 Research Objectives

The main objective of this research is to evaluate the performance of CLSMs prepared using native soil containing various concentrations of soluble sulfates by performing durability studies. For this purpose, the following tasks were executed:

1. Select native high plasticity clay underlining any section of the IPL pipeline alignment.
2. Determine the sulfate concentration in the selected soil.
3. Treat the selected soil with four different sulfate concentration levels to cover the range (0 to 20,000 ppm) of sulfates identified in the EF formation.
4. Conduct mix-design and verify 28-day strength requirement as per the specifications.
5. Conduct durability studies per standards specified in the laboratory testing section.

   The durability studies will include the evaluation of long term performance of the CLSM samples considering several parameters such as volume change, calcium loss and strength.

   Figure 1.1 below provides a visual representation of the research tasks.

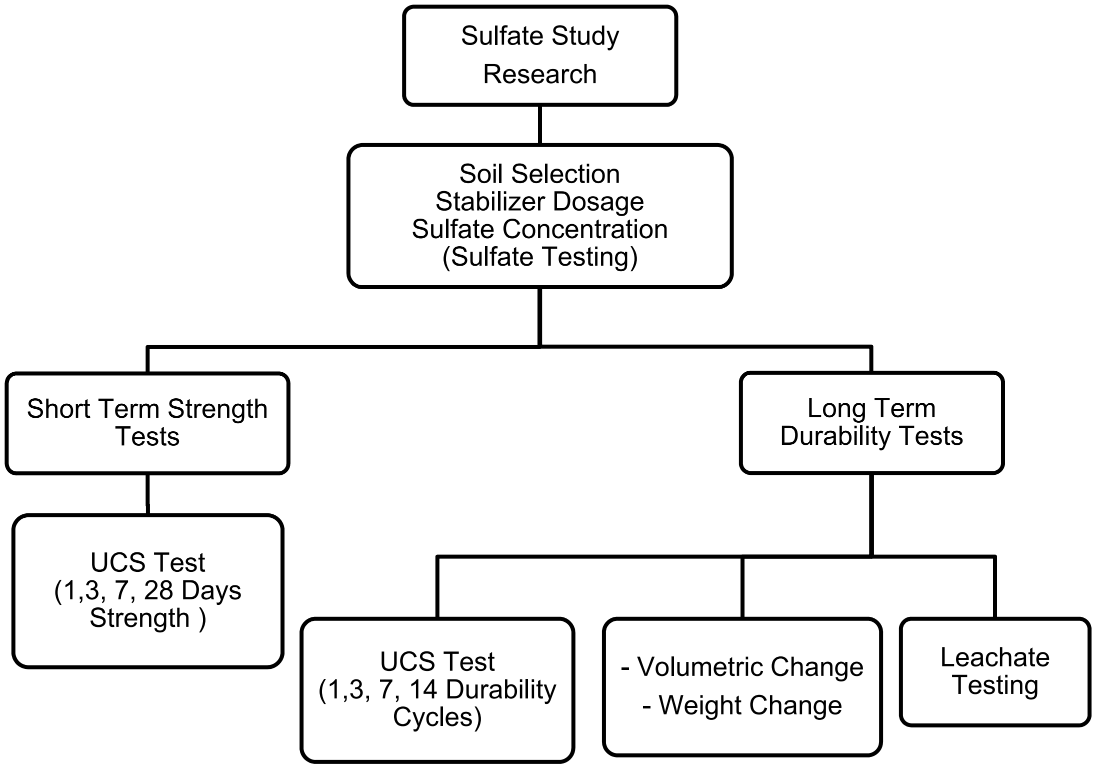


Figure 1.1 Flowchart Representing the Sulfate Study Research Tasks

1.3 Thesis Organization

In order to accomplish the above mentioned objectives, the following tasks will be performed on a chapter basis. There are five chapters outlined in this thesis namely:

Chapter 1 provides a brief introduction on the research task outlined in this paper. This involves background on the IPL project along with the sustainability and practicality of using native-soil CLSM mix design as a bedding material for pipeline construction projects. Furthermore, the concerns and issues related to construction on expansive soil with high concentration of sulfates is also a crucial part of this section. Among other things, a list of research objectives and scope of work including thesis organization are also roofed in here.

Chapter 2 starts with the literature review of historical background on CLSMs, the evolution of CLSM sample preparation techniques, the various ingredients, applications of

CLSM mix design in engineering practices along with the effects of durability (heating/wetting and freezing/thawing) studies on CLSMs and sulfate induced heaving in expansive clay. Several case studies involving CLSM mix design as pipeline bedding material and sulfate heaving mechanism on expansive clay are also presented in this section. Chapter 2 provides an overview for the need of a research task involving CLSM mix design as a pipeline bedding material prepared by using native soil (expansive clay) with high concentration of sulfates.

Chapter 3 presents the various procedures and standards followed to perform laboratory tests to achieve the above outlined objectives. The procedures discussed include soil selection and sampling, various sulfate concentration selection, sulfate testing, CLSM specimen preparation, durability testing (wetting and drying cycles), leachate testing and strength testing. In brief, chapter 3 is a summary of the laboratory procedures and the equipment used for testing.

Chapter 4 gives an overview of the results obtained from the testing. The results will include initial and final sulfate concentration of native soil used in CLSM mix design, unconfined compressive strength (UCS), volumetric and weight changes and the loss of calcium concentration after each durability cycle.

Chapter 5 provides an analysis of the test results obtained. The analysis will be followed by recommendations on using CLSM mix design as a pipeline bedding material. Future research proposals for in depth studies linking expansive soils and CLSM mix designs are also presented in chapter 5.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

This chapter presents the theoretical background of CLSM's history, its application tied in with an overview on expansive soil, sulfate heaving mechanism and case studies on both CLSMs and high sulfate soil stabilizations. The literature review presented here was collected from various sources such as the University of Texas at Arlington library, journal resources, electronic search engines and several technical reports.

The chapter starts with a historical background on CLSMs, its benefits, drawbacks and applications, followed by the effects of durability on CLSMs and a few case histories involving durability studies on CLSMs .A brief discussion on construction in expansive soils and the problems caused by the presence of soluble sulfates in these expansive soils is presented. Also, a brief review of practices in stabilization of expansive soils with high sulfate contents along with case studies showing CLSM used with high sulfate soils is discussed.

The main objective of the literature research on CLSMs and sulfate rich expansive soils is to access whether these soils are problematic or not. In case these types of soil which are abundant in DFW metropolis are problematic, further research needs to be done to find the general practices used in soil stabilization. Another important goal of this chapter is to bring into attention any studies or projects that have been undertaken with CLSM as a bedding material for pipeline construction on expansive soils rich in sulfate. The ultimate goal is to check if CLSM prepared using native soil and treated with a chemical stabilizer can be recommended as a pipeline bedding, haunch or backfill material.

2.2 CLSM Introduction

2.2.1 CLSM Background

American Concrete Institute (ACI) Committee 229 reports CLSM as a self-compacted, cementitious material commonly used as a backfill material to replace conventional compacted fill. CLSM is not only a self-compacting material but also a self-leveling substance with several generic names such as flowable fill, unshrinkable fill, controlled density fill, flowable mortar, plastic soil-cement, soil-cement slurry and K-Krete (ACI, 2005). Earlier definition of CLSM included materials with a 28-day compressive strength of less than 1200 psi regardless of the materials used in the mix-design (Trejo et al. 2004). Other version of ACI report, ACI 116R defines CLSM as a material showing a compressive strength of 300 psi or less. However, the most practical application would be to limit the compressive strength of CLSM material to 300 psi or less in order to ease future excavation if needed (ACI, 2005).

2.2.2 CLSM History

Initially termed as the Controlled Density Fill (CDF), the very first use of materials similar to CLSMs that used soil-cement slurry was reported around 1960's. Trejo et al. (2004) presented a thorough discussion on the birth and development of CLSM. As per that study, the birth of CLSM can be credited to the engineers from the Detroit Edison Company (Detroit, Michigan) and the Kuhlman Corporation (Toledo, Ohio) involved in the Enrico Fermi II Nuclear Station project. The idea was to utilize the by-product of the nuclear plant, fly ash, with concrete to produce ready-mix concrete. Initial laboratory tests and research confirmed that low-strength materials could be used as backfill materials and were called K-Krete®. Later, CLSM was used over CDF to include wide range of fill materials with various applications. Furthermore in 1998, a book, "The Design and Application of Controlled Low-Strength Materials flowable fill," was published by American Society for Testing and Materials (ASTM) which presented the state of art and practice of CLSM in the field as well as in laboratory. At present, there are five ASTM standards for CLSM (Trejo et al., 2004).

2.3 Advantages and Disadvantages of using CLSMs

Using CLSM mix design in construction has its own advantages as well as disadvantages. Most of the benefits of CLSM have been discussed in more detail in ACI 229R-99 report. Table 2.1 below lists some of the benefits and drawbacks of using CLSMs.

Table 2.1 Advantages and Disadvantages of CLSMs

S.No.	Items	Advantages (ACI, 2005)	Disadvantages
1.	Easy Excavation	CLSMs have compressive strengths of 0.3 to 0.7 MPa (50 to 100 psi). Hence, they can be easily excavated using conventional digging equipment.	CLSMs have low strength compared to concrete and cannot be used in areas where high strengths are desirable. (ACI, 2005)
2.	Easy to place	Depending on type and location of void to be filled, CLSM can be placed by chute, conveyor, pump or bucket, because CLSM is self-leveling, and flowable it needs little or no spreading or compacting. This enhances the construction rate and reduces the labor requirements.	Need to anchor lightweight pipes: One of the concerns when using CLSM in construction is its flowable nature. While it is an advantage that CLSM can easily flow around pipes or tight areas, it may cause light-weight pipes to float. (Najafi et al.,2004)
3.	Fast set up time	CLSM sets fast allowing for quick return of traffic in case of road constructions	Eventhough CLSM has a short setting time when compared to concrete; it requires confinement before setting due to its flowing nature. This can sometimes add to the cost of the project. (Najafi et al.,2004)
4.	Versatile	The ingredients used in CLSM are project specific. Some of the common wastes used in CLSM are fly ash, quarry waste products, scrap tires, incinerated sewage sludge ash, crushed stone powder (Horiguchi et al., 2011), ground granulated blast furnace slag (GGBFS), crushed stone (Trejo et al., 2004) etc. The application of recycling by-products from construction and manufacturing makes CLSM mix design a sustainable, environmental friendly design.	Since CLSM has a flowable nature, it exerts lateral pressure when in nearly liquid form. Hence the structure/ pipes where CLSM is being need to withstand the lateral pressure. (Schmitz et al., 2004)

2.4 Applications of CLSM and Case Studies

Since the dawn of CLSM mix design in construction industry in the 1960s, its application has been evolving and expanding gradually ever since. Traditionally, CLSM was used as a backfill material to replace compacted soil backfill (ACI, 2005). However, additional research studies on CLSM have opened additional applications of CLSM mix design. Both traditional and modern applications of CLSM with their case studies have been discussed below.

2.4.1. Traditional CLSM Applications

Some of the traditional applications of CLSM mix design are backfilling, void filling, bridge approaches and utility bedding.

2.4.1.1 Backfilling

One of the most common applications of CLSM mix design is in backfilling in place of conventional backfill materials that need compaction. The application of CLSM as a backfill material generally involves backfilling in trenches or behind retaining walls. The use of CLSM as a backfill material has its own advantages one of them being the reduction of width and size of excavation contributed by its self-leveling and flowable property. Per ACI (2005), conventional granular or site excavated backfill, even when compacted properly in the required layer thickness, may not achieve the uniformity of CLSM.

The case study reported in WI-16-99 prepared by Wilson (1999) from Wisconsin Department of Transportation (WDOT) titled “Flowable Fill as Backfill for Bridge Abutments,” dates back to 1996 where CLSM was used as backfill for bridge abutments. Figure 2.1 shows CLSM placement on west abutment of bridge in Wisconsin.

Figure 2.1 CLSM being Used as a Backfill for Bridge Abutment in Wisconsin
(Wilson, 1999)

2.4.1.2 Void filling

Another common application of CLSM is to fill void spaces. Filling old tunnels or sewer lines requires extreme caution. CLSM is perfect in filling voids in these scenarios since it is flowable and needs no compaction. CLSM not only flows greater distances, it also sets fast speeding up the construction. Per ACI (2005), CLSM was used to fill an abandoned tunnel passing under the Menomonee River in Milwaukee, Wisconsin.

2.4.1.3 Bridge Approaches

One of the major challenges faced by DOT's was when using conventional compacted fill is settlement of the fill used over time. Due to this settlement, "bump at the end of the bridge" is observed on many bridge approaches. This bump is caused by the consolidation of soil at the interface of the bridge and the approach slab (NCHRP, 2008). CLSM is desirable in construction or repair of bridge approaches as subbase for the bridge approach slab or to backfill against the wingwall. When used during initial construction stages, CLSM helps to prevent long term settlement thus reducing chances of bumps at the end of the road significantly.

2.4.1.4 Utility/Pipeline Bedding

Utility/pipeline bedding is another area where CLSM application has been growing rapidly. CLSM not only provides excellent bedding for utilities such as water/sewer pipeline, electrical, telephone and other types of utilities but also fills voids beneath the conduit providing uniform support. Use of CLSM bedding provides solid, uniform pipe support, reduced labor costs, reduced trench preparation time, and reduction of water entry to the bedding-pipe interface (Trejo et al., 2004). Furthermore, the low strength of CLSM (when compared to concrete) makes future excavation easier in the event the underground utilities require repair. Figure 2.2 shows the hardened CLSM used in pipeline bedding. Appendix A shows the typical pipe cross-section with CLSM as bedding material. Appendix B shows the CLSM being used as bedding material in the IPL project.

Figure 2.2 Endview of Pipeline with CLSM (Howard and Bowles, 2008)

2.4.2. Emerging Applications of CLSM

In addition to the four major applications previously discussed, CLSM has been utilized in various applications. New applications are expected to surface as the construction community gets more familiar with this material. Current applications include bridge replacement (Iowa DOT), structural fill, insulation and isolation fill, erosion control, and others (Folliard et al. 2008).

2.5 Constituents and Properties of CLSM

2.5.1. Constituents of CLSM

The most common constituent materials used in CLSM are Portland cement, fly ash, aggregates such as foundry sand, chemical admixtures, and other by-product materials. A significant benefit of CLSM is the ability to use a wide range of local materials, including by-product materials (Folliard, 2008). Recent studies show the use of scrap tire (Pierce and Blackwell, 1999) and recycled gypsum (Kamei et al., 2011) have also been used as constituents in preparing CLSM mix.

2.5.2. Properties of CLSM

This section provides information on the properties of CLSM that most affect its performance in key applications. The properties of CLSM can be divided into two categories namely: Plastic properties and in-service properties. Plastic properties of CLSM per ACI (2005), include Flowability, segregation, subsidence, hardening time and pumping and the in-service properties include strength (unconfined compressive strength), density, settlement, thermal insulation/conductivity, permeability, shrinkage, excavatability, shear modulus and potential for corrosion. Table 2.2 shows the CLSM properties typically specified and measured by state DOTs (Department of Transportation). In case of pipeline application properties such as flowability and unconfined compressive strength are more important than the other properties. Hence, this literature search mainly focuses on these two properties.

Table 2.2 CLSM properties typically specified and measured by state DOTs (Folliard, 1999)

Property	Number of States Testing	Common Test Method
Flow	18	ASTM D 6103 (or similar) and ASTM C143
Compressive Strength	17	AASHTO T22 and ASTM D 4832
Unit Weight	14	AASHTO T 121
Air Content	10	AASHTO T 152
Set Time	7.2	ASTM C 403
Durability	2	pH and resistivity
Shrinkage	1	Visual
Temperature	1	Modified ASTM C 1064
Chlorides/sulfates	1	Determination of ion contents
Permeability	0	None

2.5.2.1. Flowability

One of the most important properties of CLSM that distinguishes it from other fill material is its ability to flow easily into confined areas, without the need for conventional mechanical placing and compacting device. This self-leveling property of CLSM significantly reduces labor and increases construction speed (Folliard, 2008).

Depending on the project requirements, flowability of CLSM can be varied from stiff to fluid. There are various standard flowability test criteria namely the use of a 75 x 150 mm (3x6 in.) open-ended cylinder modified flow test (ASTM D 6103), the standard concrete slump cone (ASTM C 143), and flow cone (ASTM C 939). When using the ASTM D 6103 method, a good flowability is achieved in the absence of noticeable segregation with the CLSM spread diameter of at least 200 mm (8 in.) (ACI, 2005). Per the slump cone test as mentioned in ACI 229R-99 report, flowability ranges can be expressed as follows:

Low flowability: Less than 150 mm (6 in.);

Normal flowability: 150 to 200 mm (6 to 8 in.);

High Flowability: greater than 200 mm (8 in.)

2.5.2.2 Unconfined Compressive Strength

The unconfined compressive strength of CLSM is the most common hardened property measured, and the one most commonly found in state DOT specifications. CLSM compressive strength values are generally used as an index for excavatability or digibility when future excavation is necessary. Materials and mixture proportions must be selected to ensure that these strength values are not exceeded in the long term (Folliard, 2008). Some CLSM mixes initially within the acceptable strength range continue to gain strength over time, making future excavation difficult. However, if the strength of the CLSM mix is lower than the project specified values, changes of infrastructure failure becomes high and can have serious consequences. A CLSM compressive strength of 0.3 to 0.7 MPa (50 to 100 psi) resembles an allowable bearing capacity of a well-compacted soil (ACI, 2005).Mechanical equipment such as backhoes are used to excavate CLSM with compressive strength of 0.7 to 1.4 MPa (100 to 200 psi) whereas CLSM with UCS less than 0.3 MPa (50 psi) can be excavated manually (ACI, 2005). Figure 2.3 shows the CLSM mix being excavated.

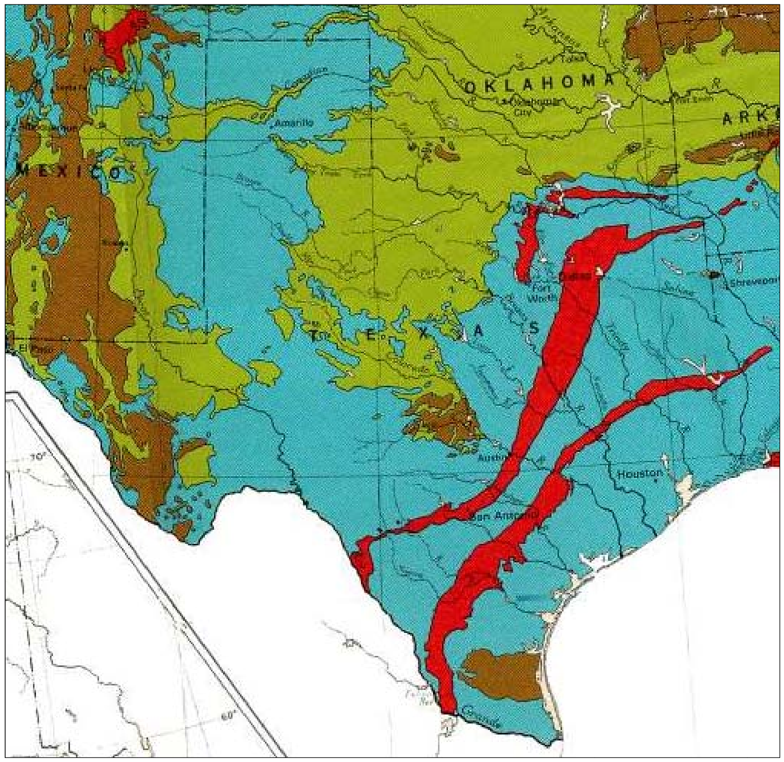
Figure 2.3 Excavating CLSM with a Backhoe (Source: ACI 229R-99, 2005)

2.6 Construction on Expansive Soil

Expansive soils are abundant in the United States as well as all over the world. Expansive unsaturated soils, found in every state, cover one-fourth of the United States (Puppala and Cerato, 2009), about one third of the earth's surface (Chen, 1999) and are found in arid, semi-arid and underdeveloped areas. Many parts of south-western United States, United States, South America, Canada, Africa, Australia, Europe, India, China and the Middle East have reported great distresses while constructing on expansive soils (Chen, 1975). During alternate wetting and drying seasons, these soils exhibit swell-shrink behavior. Due to this periodic swelling and shrinkage, structures built above these soils undergo massive distress in the form of cracking or bulking. Numerous cases of civil infrastructure failures due to expansive soils have also been reported over the years. According to Nelson and Miller (1992), expansive soils are a worldwide problem, and they undergo considerable amounts of volume changes due to moisture content fluctuations. A recent study showed the cost of damage to homes due to expansive soil was approximately $13 billion per year (Puppala and Cerato, 2009). Furthermore, they also call expansive soil a natural hazard whose damage exceeds the average annual damage caused by floods, hurricanes, earthquakes, and tornadoes combined with the exception of Hurricane Katrina. Figure 2.4 shows the location of Eagle Ford geological formation which is known for being problematic high plasticity clay (fat clay) in Texas.

2.6.1 Introduction on Expansive Soil

Expansive soils are soils that inhibit substantial swelling in presence of moisture and significant shrinkage when dry. This swelling and shrinkage of expansive soils result in cracking and buckling of infrastructure built upon them (Puppala and Cerato, 2009). Expansive soils can typically be identified in the lab by their plastic properties. Inorganic clays of high plasticity, generally those with liquid limits exceeding 50 percent and plasticity index over 30, usually have high inherent swelling capacity and are termed as expansive clay or fat clay. Expansion of soils can also be measured in the lab directly, by immersing a remolded soil sample and measuring its volume change (Rogers, Olshansky and R. B. Rogers, 1993). Figure 2.4 shows the map of various sulfate distribution in Texas.



Unit contains abundant clay having high swelling potential (Eagle Ford Formation)

Part of unit (generally less than 50%) consists of clay having high swelling potential

Unit contains abundant clay having slight to moderate swelling potential

Part of unit (generally less than 50%) consists of clay having slight to moderate swelling potential

Unit contains little or no swelling clay

* These maps are sourced from the U.S. Geological Survey publication "Swelling Clays Map Of The Conterminous United States" by W.W. Olive, A.F. Chleborad, C.W. Frahme, Julius Schlocker, R.R. Schneider, and R.L Shuster; 1989.

Figure 2.4 Map Showing the Distribution of Expansive Soil in Texas

2.6.1.1 Swell-Shrink Behavior of Expansive Soils

The swelling and shrinking behavior exhibited by expansive soils is due to the presence of large portion of highly active clay mineral of the smectite group such as montmorillonite in the soil (Sridharan and Prakash, 2000). The minerals in this group have an expanding lattice. Hence, greater the amount of the montmorillonite clay mineral in the soil, greater its swell potential (volumetric change) and the more water it can absorb (Jones and Jefferson, 2012). Clayey soils absorb large amount of water during and after rainfall and become heavy and sticky. Contrariwise expansive soils harden up when dry causing cracking and shrinkage of the ground. This hardening and softening is known as 'shrink-swell' behavior. When supporting structures, the effects of significant changes in water content on soils with a high shrink–swell potential can be severe (Jones and Jefferson, 2012). For most expansive clays, an expansion of 10% of this original volume is not uncommon (Chen 1988; Nelson and Miller, 1992).

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). Examples of expansive clays are high plasticity index (high-PI) clays, over-consolidated clays rich with Montmorillonite mineral, and shales (Chittoori, 2008).

In many areas in Texas, due to dry weather conditions, initial shrinkage occurs in expansive soils that result in large tension cracks. Swelling and shrinkage are not fully reversible processes (Holtz and Kovacs, 1981). The cracks formed due to shrinkage in dry weather conditions upon re-wetting do not close-up completely. This allows the soil to bulk out slightly providing access to rainfall infiltration during wetting period. Swelling pressures can cause heaving, or lifting, of structures whilst shrinkage can cause differential settlement (Jones and Jefferson, 2012). As a result severe cracks and even structural failures are observed on civil infrastructure constructed over poorly treated (with chemical stabilizers) or untreated expansive soils. Figure 2.5 shows the vertical movement of existing pavement measuring upto 3.5 inches, caused by expansive soil in Frisco, Texas.

Figure 2.5 Differential vertical movement caused by expansive soil measures 3.5 inches at a pavement joint failure in the Meadow Creek subdivision in Frisco, Texas
(Source: Richard J. Hammerberg, P.E, www.cenews.com)

2.6.2 Chemical Stabilization of Expansive Soil

Since, expansive soils exhibit large amount of swell-shrink behavior, it is important to stabilize these types of soils before construction over these soils. One of the most common practices of stabilizing expansive soils is by using chemical stabilizers. Chemical stabilization methods are widely used in the field to control soil heaving (Nelson and Miller, 1992; Puppala et al. 2003). 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 extending the life of structures built on those soils (Hausmann, 1990).

2.7 Background on Sulfate Heaving

Often expansive soils are stabilized with chemical stabilizers in order to reduce volumetric expansion due to moisture and salt influx, increase durability, or to achieve proper strength specifications for the particular design (Mitchell 1992; Dermatas 1995; Kota 1996; Azam 2003). The most commonly used chemical stabilizers are cement and lime. Literature shows that if the soil being stabilized is rich in soluble sulfate, it can be problematic. According to Puppala and Cerato (2009), when calcium-based stabilizers such as lime, cement or fly ash are added to sulfate-bearing soils, reactions cause the volume change potential to increase, creating a soil mixture that is more expansive than the soil alone. Also, 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; Puppala and Cerato, 2009). This phenomenon has been termed "sulfate-induced heave" in the literature (Mitchell, 1986; Dermatas, 1988; Hawkins, 1988; Dermatas, 1995).

Texas soils are primarily made of clay or fine grain sedimentary deposits; therefore the most common primary sulfate source of sulfate in Texas soil is in the form of gypsum (Thomey, 2013). Sulfates are present in natural soils as calcium sulfate ($CaSO_4$), Thenordite or Sodium Sulfate (Na_2SO_4), and Epsomite or Magnesium Sulfate ($MgSO_4$) (Puppala et al., 2003). Chemical stabilizers such as cement and lime contain significant amount of calcium. Clayey soils commonly consist of three minerals namely Kaolinite, Illite and Montmorillonite which contain alumina (Aluminum Oxide, Al_2O_3) naturally. When sulfate laden clayey soil is treated with calcium based stabilizers such as lime or cement, the pH of the system is elevated where naturally occurring minerals, alumina and silica, are released into the system. These minerals combine with the calcium from the stabilizers upon the availability of water to form highly expansive minerals such as calcium – alumina – sulfate hydrate compound known as Ettringite

($Ca_6Al_2(SO_4)_3(OH)_{12} \cdot 26H_2O$) or a calcium – silica – hydroxide – sulfate –hydrate compound known as Thaumasite ($Ca_3Si(OH)_6(CO_3)(SO_4) \cdot 12H_2O$) (Sherwood, 1962; Mehta and Klein, 1966; Mehta and Wang, 1982; Mitchell, 1986; Hunter 1988; Perrin, 1992; Petry, 1994; Burkhart et al., 1999). Ettringite contains 26 molecules of water enabling it to swell more than 137% of its volume (Little et al., 2010). Under desirable moisture, humidity and temperature conditions, these Ettringite and Thaumasite grow, causing further swell (Talluri 2013). Upon swelling or shrinking the soil exhibits buckling or cracking as opposed to stabilizing the expansive soil. This clearly shows how chemical stabilization can be a nuisance instead of a boon in stabilizing expansive soils rich in sulfates.

Many cases of sulfate-induced heave were reported in Texas in and around DFW metroplex (Perrin, 1992; Puppala et al., 2010). Researchers called lime treatment of expansive soils containing sulfate “man-made expansive soil” (Puppala et. al., 2012). Expansive soils stabilized with lime treatment undergo a chemical reaction which forms an interlocking get in between the clay particles. This phenomenon increases the strength, reduces plasticity, increases workability, and reduces in swell behavior of the treated soil (Dempsey and Thompson, 1968; Bell, 1989; Thomey, 2013). Similarly, stabilization with cement creates a pozzolanic reaction that lowers the plasticity of the soil and also produces gels that increase the strength of the soil and reduce swelling potential (Bugge and Bartlesmeyer, 1966; Nelson and Miller, 1992; Thomey, 2013). However, Mitchell (1986, 1992) has documented lime induced heave in expansive soils. Furthermore, cement based stabilizers also induce sulfate heave Kota, 1996; Ksaibati, 1996). Thomey (2013), Talluri (2013) have presented several case studies on sulfate induced heave distress in existing infrastructure around Texas as well as in various states in the United States. Figure 2.6 shows the severe pavement failure on Joe Pool Lake area, Texas caused by sulfate heaving.

However, TxDOT’s guidelines for treatment of sulfate-rich soils and bases in pavement structures report that failures have been documented in Texas due to sulfate heave in low plastic soils treated with calcium-based modifiers. This implies that sulfate heaving does not always occur when stabilizing expansive soils with calcium based stabilizers.

Figure 2.6 Severe Pavement Distress on Pavements close to Joe Pool Lake Dam, Texas (Talluri, 2013)

2.7.1 Sulfate Heaving Mechanism

The distress resulting in the presence of Sulfate and Portland cement concrete was established in the early 19th century (ACI 1982; DePuy 1994). During that time it was proven that calcium rich cement mixed with sulfate and free of alumina could create Ettringite and Thaumasite minerals (ACI 1982). Later, Cohen (1983) explained the formation of Ettringite and its subsequent growth in concrete. Cohen established two different growth mechanisms for Ettringite and Thaumasite, crystal growth and hydration. Similarly, (Dermatas 1995) established the same two expansion mechanisms. The first mechanism causes expansion due to the formation and/or oriented crystal growth of Ettringite (Ogawa and Roy, 1982). The second is a thorough solution mechanism where expansion is related to swelling due to hydration Ettringite (Mehta 1973; Mehta and Wang 1982). Basically the sulfate heaving mechanism can be divided into two basic categories namely heaving due to crystal growth and heaving due to hydration. Thomey (2013) and Talluri (2013) provide a detailed explanation of both the theories.

2.7.2 Methods to Measure Sulfate Concentrations in a Soil

Since sulfate induced heaving can cause damage amounting to millions of dollars to infrastructure, it is important to determine the amount of sulfate present in the soil before stabilization. Sulfate detection prior to specifying and constructing calcium treated soils is the only means of prevention of sulfate-induced heave (TxDOT, 2005). Various DOTs (Department of Transportations) are mandating the sulfate concentration measurement in soils prior to chemical stabilization. There are various methods to determine the sulfate concentrations in soil. Sulfate concentration is soil is commonly expressed in parts per millions (ppm). Two basic categories to measure sulfate concentration is soils are gravimetric-based or turbidity-based methods. However, it was reported that these methods often fail to provide consistent and repeatable values (Viyanant, 2000). Furthermore, studies have shown that even under similar testing conditions for known sulfate concentrations, it is difficult to obtain accurate sulfate concentration measurements in the laboratory with only handful inexpensive pieces of equipment (Harris et al. 2003 and Puppala et al. 2002).

Gravimetric procedures determine the sulfate content based on the amount of sulfate precipitated upon the addition of Barium Chloride to soil-water solution. Turbidity-based procedures convert the turbidity caused by the presence of sulfates to sulfate concentration. Some of the techniques used to measure sulfate concentration in soils are:

- Turbidity Based Method
  - TxDOT Method (Tex-145-E)
  - ASTM Method (C1580)
- Gravimetric Method
  - Modified UTA Method
  - AASHTO Method (T 290-95)

Talluri et al. 2012 conducted an experiment to verify the accuracy of the UTA method, the gravimetric AASHTO T 290 – 95, and TXDOT Tex – 145 – E. It was determined that the Modified UTA method resulted in the most accurate readings of sulfate concentrations. Puppala et al. (2002) and Thomey (2013) provide a detailed testing procedure for measuring the sulfate concentration of soil using Modified UTA Method.

2.7.3 Threshold Sulfate Levels

With the increasing number of infrastructure failures due to sulfate heaving mechanism, one of the most common answer researches sought is the sulfate concentration level safe for stabilization. This safe sulfate concentration level below which sulfate heaving is not problematic defines the sulfate threshold level.

Several studies have been conducted over the years to predict the threshold level of sulfates in soil but there has not been consensus on sulfate threshold levels. According to Puppala and Cerato (2009), it is not appropriate to establish a sulfate concentration threshold based on database of several case studies conducted on different soil compositions and environmental conditions and at the same time the findings and observations from the studies may not be true or applicable at other locations.

The threshold levels in some cases have been reported to falling between 1,500 ppm to 5,000 ppm and in other cases the threshold levels were set as high as 10,000 ppm (Harris et al 2004; Puppala et al. 2005; Adams et al. 2008). According to TxDOT practices, sulfate concentrations of 3000 ppm or less pose minimal possibility for sulfate heave. Unfortunately, soil conditions such as plasticity, density, and void ratio coupled with stabilization techniques and environmental factors largely affect these threshold levels (Puppala 2005). Therefore, setting threshold levels “across the board” is nearly impossible (Adams et al. 2008). There are studies hoping to determine sulfate threshold levels based on mineralogy and geological depositional environments, but this will only address some of the issues associated with sulfate heave thresholds (Adams et al. 2008).

Overall it seems true that with varying site conditions, void ratios, site drainage and various soil compositions, it is impossible to draw a definitive sulfate level as a threshold level universally. Hence, a more practical approach would be to limit sulfate threshold to being case specific.

2.7.4 Sulfate Heave Mitigation Methods

Researches indicate numerous methods and practices implemented to alleviate sulfate heaving mechanism. Several factors affect the sulfate heave mitigation methods such as sulfate concentrations in the soil, type of soil (expansive or no-expansive), and site drainage among others. However, the literature search in this section was limited to mitigation methods that indicate the sulfate levels the method worked for. This section mostly focuses on stabilization using cement and some lime since the research's main focus is utilizing cement as a stabilizer. Some of the methods are discussed below:

2.7.4.1 Pre-Compaction Mellowing

As the name suggests, pre-compaction mellowing involves the stabilization of soils with lime, allowed to mellow before being compacted. Mellowing is the process of allowing the lime treated soil to remain in an uncompacted state for a period of time in order for the lime to react with the clay particles and sulfates (TxDOT, 2005).

Soils with sulfate levels around 7,000 ppm with a 3-day mellowing period resulted in acceptable swells in soil (Harris et al., 2004). However when the sulfate concentration in the same soil was spiked to 10,000 ppm, the results were not positive towards sulfate heave mitigation. On the basis of these findings, Texas Department of Transportation limits the use of lime treatment above 8,000 ppm sulfate levels.

In another study conducted by Berger et al., (2001), soils from South Orange County, California, containing 0, 5000 and 8,000 ppm sulfate were treated with 4% lime and 4% lime+8% fly ash and allowed to mellow for periods of one, three and five days. After the mellowing period, samples were cast into cylinders and strength and swell tests were conducted to see the effectiveness of stabilization. All the test soils passed the strength and allowable swell criteria, showing the dominance of pozzolanic reactions.

The, a second set of samples were prepared using 6% lime and spiked with 14,000ppm (±1000ppm) sulfates to see the effect of a higher concentration of sulfates. All the samples were mixed with 6% lime and 6% lime, 12% fly ash and allowed to mellow for periods of one, three and five days. After the mellowing period, the specimens were cast and moist cured for 60 days and tested for swelling in sulfate solution. All the soils exhibited positive effects of stabilization with negligible vertical swell. The pH of the test soils was observed to be higher than 10 in all cases, indicating the occurrence of both pozzolanic and sulfate reactions.

This successful stabilization of all the soils specimen adopting mellowing technique was attributed to the fact that all the expansion occurred during the mellowing period. Hence the sulfates were consumed during the mellowing period leaving less or no sulfates available after compaction. Maximum effects of stabilization were achieved in this case.

2.7.4.2 Sulfate Resistant Cement

Portland cement has been used in numerous aspects of civil engineering projects. There are five different types of cement used in construction practices. The first, Type I, cement is the most commonly used cement in reinforced concrete applications. Type II is used when low to moderate sulfate ions are assumed to present before, during or after construction, Type III for projects that require high strength in early stages, Type IV for concrete applications that are in constant contact with water, such as dam structures and Type V cement for high sulfate soils. Cement treatment of soils provides strength enhancements and plasticity reductions through flocculation, cementation and pozzolanic reactions (Talluri, 2013).

In a study conducted by Puppala et al. (2004), the effects of sulfate resistant cement on UCS, plasticity, free swell and linear shrinkage were studied. Four soil samples were treated with 5% and 10% Type I/II cement and the same four samples were treated with 5% and 10% Type V cement. The research concluded that Type V cement generated a larger UCS strength than Type I/II cement. Free swell of the treated samples was reduced to nearly no expansion in both cement treated samples, and linear shrinkage was reduced by the addition of cement. Overall the sulfate resistant cement treatment showed good results in increasing strength, reducing plasticity, and reducing swell and shrinkage.

However, many practitioners are still skeptical of this method due to the large amounts of alumina present in clay soils. This natural alumina content can counteract the low alumina content of sulfate resistant cements; therefore, further studies must be conducted on the viability of sulfate resistant cements in soil stabilization.

2.7.4.3 Combination of Lime and Cement

It is well known that lime and cement treatment improves the workability and reduces volumetric changes of soils. Researchers from The University of Texas at Arlington used a combination of lime and cement to treat expansive soils from Arlington, Texas containing low to medium sulfates (Chakkrit et. al., 2008). As part of the laboratory studies, two high plasticity clay soils from Arlington were chosen and treated with 12% lime and a combination of 6%lime and 6% cement. Two curing periods, 2 days and 7 days, were considered in this study. Laboratory results indicated that the combination of lime and cement successfully provided strength enhancements and reduced swell and shrinkage characteristics. Also, the combination of lime and cement treatment proved to be more effective than the lime treatment alone. Findings from the laboratory study were implemented successfully in the field, and no issues of heaving were observed.

2.7.4.4 Recycled Gypsum in Wet Environment

Although most of the studies covered in this Chapter shows that sulfate induces heaving phenomenon on expansive soil, researchers in Japan have utilized recycled gypsum ($CaSO_4 \cdot 2H_2O$, soluble sulfate), successfully to stabilize soft clay in a wet environment (Kamei et al., 2011).

This study investigates the influence of wet environment on the compressive strength, dry unit weight and durability performance of soft clay soil stabilized with recycled gypsum and Furnace cement under the wetting and drying cycles are referred to as wet environment in this study. Stabilized soils were prepared and stored in cylindrical tubes. The samples were cured for 28 days in a controlled room with constant temperature and humidity. After the curing period, the specimen were subjected to various wetting-drying (w-d) cycles, and then tested for unconfined compressive strength (UCS), moisture content and volume change.

Results from the tests indicated that the UCS increased with the increase of recycled gypsum content during the w-d durability studies. The increase in recycled gypsum content is associated with the increase of dry unit weight, as well as decreases in moisture content of the stabilized specimens. The UCS of specimen stabilized with recycled gypsum and Furnace cement gradually decreased with an increase in the number of wetting and drying cycles, while the early cycles have the greatest effect on the durability compared to the effect of later cycles. Overall, the influence of durability studies (w-d cycles) were not significant when the UCS and volume change parameters were considered for soils stabilized with recycled gypsum and furnace cement. Hence, the study concluded that the use of recycled gypsum to stabilize soft clayey soil achieved acceptable durability. Furthermore, the study indicated that the effective use of recycled gypsum, which is derived from gypsum waste plasterboard, not only contributes towards a sustainable society but also acts as an economical alternative of waste disposal.

2.7.4.5 Several Other Techniques

The other methods of alleviating sulfate induced heave include stabilizing the top portion of select fill, compacting to lower densities and use of polymeric fibers with soil. Soils with no soluble 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 the stabilizing layer to lower densities is another option available. Compacting at lower densities allows more void spaces in the soil matrix. This allows more room for the growth of Ettringite and its overall expansion. Overall, it seems like for reasons with high sulfate concentrations, mitigation of sulfate heaving can be achieved by using Type V Cement.

2.8 CLSM with High Sulfate Native Soil as Fine Aggregate

Previous research studies at the University of Texas at Arlington (UTA) by Raavi (2012) and Vanga (2013) has not only established the CLSM mix design using native soils with selective dosages of cement as a stabilizer but also have conducted long term durability studies using soil from four different geological formations along the IPL alignment.

Furthermore, the research performed by Thomey (2013) on sulfate mapping for the pipeline alignment showed the presence of elevated levels of soluble sulfates in the Eagle Ford formation ranging from 40 parts per million (ppm) to approximately 20,000 ppm. The soil from Eagle Ford is classified as high plasticity clay (Raavi, 2012). The study conducted by Vanga (2013) show that certain soil performed well even under long term durability testing and retained 50 or close to 50% of their initial strengths. However, Eagle Ford soil that used the highest dosage of stabilizer (18%) than soils from other geological formation along IPL, exhibited the least strength retention (less than 20%) (Vanga, 2013) as shown in Figure 2.7. It can be seen from the figure that the initial UCS strength of the Eagle Ford CLSM at 0-cycle durability study was 101.80 psi (701.6 kPa) which dropped significantly to 17.60 psi (121.30 kPa) at the end of 14-cycle, a reduction in strength by 82.80%, with a strength retention of only 17.30% of its initial strength at 0-cycle. The study does not indicate the sulfate concentration of the Eagle Ford soil used for the preparation of CLSM mix.

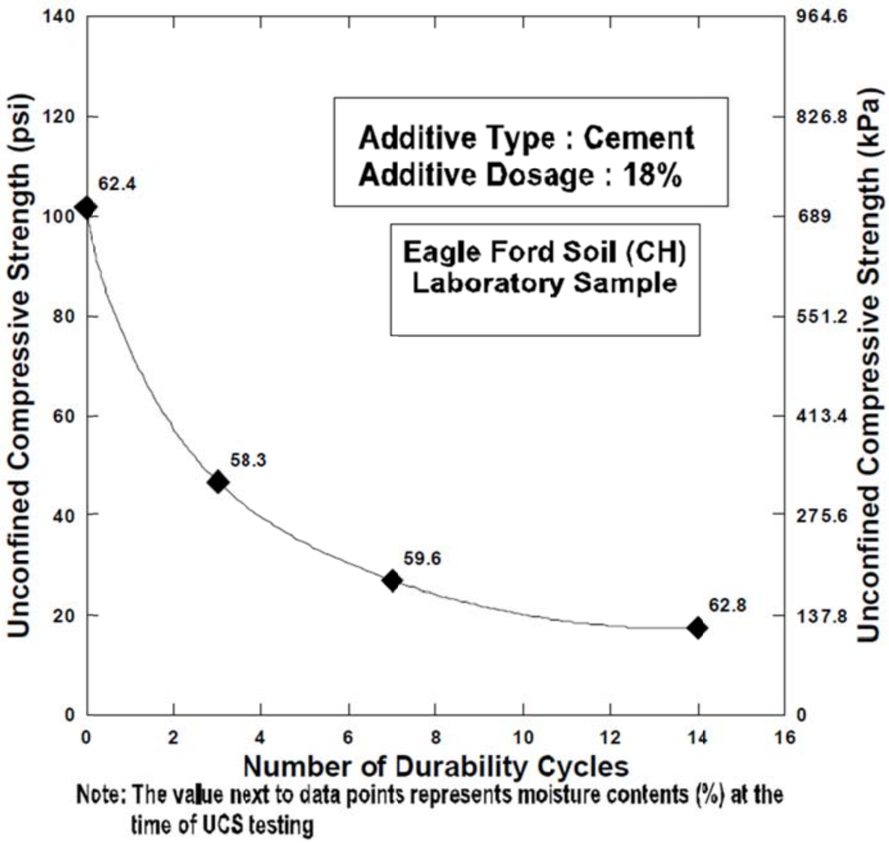


Figure 2.7 Variation of UCS strength for Laboratory Prepared Eagle Ford Soil CLSMs (Vanga, 2013)

2.9. Durability Studies on CLSM

The fluctuation in weather condition changes the temperature and humidity of the region subjecting the soil in the field to severe wetting/drying or freezing/thawing conditions which in turn changes the behavior of the soil. Durability studies refer to a close replication of field climatic conditions of wetting/drying (in hot regions) or freezing/thawing (in cold regions) of any soil specimen in the laboratory and evaluating their performance based on long term strength, volume change, leachability etc. Similar studies were conducted in this present research to address the durability of CLSM mixtures.

Furthermore, in most CLSM mix design, chemical stabilizer acts as an essential additive which holds the soil particles together. However, when CLSM is subjected to the durability cycles, there is loss in additive due to leaching over time. Hence, durability studies not only comprise of the durability cycles but also leachate studies along with corrosion related issues. The sections below discuss the durability related studies on CLSM comprising of freezing and thawing studies, wetting and drying studies, and leachate studies that affect the performance of CLSM directly.

2.9.1. Freezing and Thawing Studies

During cold seasons, in extremely cold regions where the temperature drop below zero degrees Celsius, cyclic freezing and thawing of water in soil causes a physical weathering process. During the freezing process, especially during winter season when the temperatures drop significantly, the water in soil (mostly expansive soils)changes to ice (swelling behavior) causing the upward movement of ground surface. This process is known as frost heave. Similarly, when the temperatures drop during the spring season, ice melts (shrinkage behavior) leaving behind broken pavement, potholes and carbuncle-like humps (Manz, 2011). Frost heave and thawing process severely impair performance of infrastructure in cold regions. Figure 2.8 shows a part of street being lifted due to frost heave in North Dakota.

Figure 2.8 Part of Bismarck Street in North Dakota being lifted as a result of frost heave (Manz, 2011)

A report published by NCHRP (National Cooperative Highway Research Program), Report 597, points out that assessing the freezing and thawing resistance properties of CLSM is challenging. Previously, AASHTO T 161 method that is commonly used to test concrete was used for testing CLSM for freeze/thaw resistance. However, Nantung (1993) found that AASHTO T 161 was far too severe for testing CLSM. Hence, in order to replicate the site conditions in the laboratory to study the effects of freezing and thawing on compacted soil or CLSM, most used ASTM D 560, “Freezing and Thawing of Compacted Soil-Cement Mixtures” procedure (Folliard, 2008). This method provides procedures for determining the soil-cement loss, moisture and volume changes (swell and shrinkage) produced by repeated freezing and thawing of the specimen being tested.

In cold regions, CLSM is susceptible to seasonal freeze-thaw deteriorations (Nantung and Scholer, 1994). Though there have not been many researches relating to the durability studies involving freezing and thawing resistance on CLSMs. Some of the studies mentioned in NCHRP report 597 are the ones conducted by Bernard and Tansley, 1981; Krell, 1989; Burns, 1990; Nantung, 1993; and Gress, 1996. Among these, the study conducted by Gress in 1996 found that CLSM can survive freezing and thawing damage but proposed that the top 50 to 150 mm of CLSM trenches be removed after set and backfilled with a frost heave compatible base material to ensure uniform heaving of pavement and trench (Folliard et al., 2008).In the same NCHRP Report 597, prepared by Folliard et al. (2008) titled “Development of a Recommended Practice for Use of Controlled Low-Strength Material in Highway Construction,” two studies conducted using various CLSM mix design with high air content and high compressive strength exhibited good freeze-thaw resistance.

Another study on durability of CLSM with used foundry sand, bottom ash, and fly ash in cold regions by Horiguchi et al. (2001) observed long-term strength developments of various types of mixtures, along with the frost heaving rate of less than 3%, a relatively smaller value as compared to soil. The study recommended the need for further research to evaluate the long term durability CLSM against frost heaving actions.

2.9.2. Wetting and Drying Studies

Similar to the durability studies based on freezing and thawing in cold regions, hot regions such as Texas call for durability studies based on wetting and drying cycles for chemically treated soil or even CLSM. During dry seasons, the water present in the soil dries out leaving cracks on the ground, a phenomenon of shrinkage. When it rains, water seeps into the ground through the cracks causing the soil to swell, a common phenomenon observed in expansive soils. This seasonal moisture fluctuation causing the soil to shrink and swell impacts the long term performance of the soil.

At present, ASTM D 559, "Standard Test Methods for Wetting and Drying Compacted Soil-Cement Mixtures," provides a testing procedure to simulate the field conditions of severe wetting and drying in the laboratory in a short period of time. The ASTM standard also provides methods to determine the soil-cement losses, water content changes and volume changes (swell and shrinkage) due to repeated wetting and drying process. The procedure compacted soil samples treated with stabilizer (cement in this case) after 7 days curing are submerged in potable water at room temperature for 5 hours. At the end of the wetting period, after recording the volume and weight changes, the sample is placed in an oven at 160°F (71°C) for42 hours. Similarly at the end of the drying period, volume and weight changes are recorded to allow the evaluation of the performance of cement stabilized soil under repeated w-d cycles.

A study conducted by Rogers and Wright (1986) on Beaumont clay which had been used to construct road side embankments showed significant drop in shear strength of soil due to cyclic wetting and drying. As a part of the research, the clay was subjected to thirty w-d cycles, each wetting and drying period lasting to 24 hours taking 48 hours to complete one full w-d cycle. The results show that repeated wetting-drying not only produced significant reduction in shear strength parameters but also showed decline in the factor of safety of the failed embankment. Eventhough the reduction in factor of safety was observed due to repeated w-d process, the value obtained was still greater than unity. They concluded that the uncertainty in the results was due to the small amounts of scatter and recommendations were made for further laboratory testing to understand the effects of wetting-drying on natural high-PI clays.

Hoyos et al. (2005) at the University of Texas at Arlington (UTA) performed a series of wet and dry cyclic tests on chemically stabilized specimens of sulfate-rich expansive clay to access the effects of w-d cycles on long term strength, stiffness and volume changes. The study concluded that among various additives used as a stabilizer, a 10%-by-weight dosage of Type V cement yielded the best overall performance under cyclic w-d. Figure 2.9. (a) and (b) show the wetting and drying setup respectively as used by Hoyos et al. (2005) in the durability study.

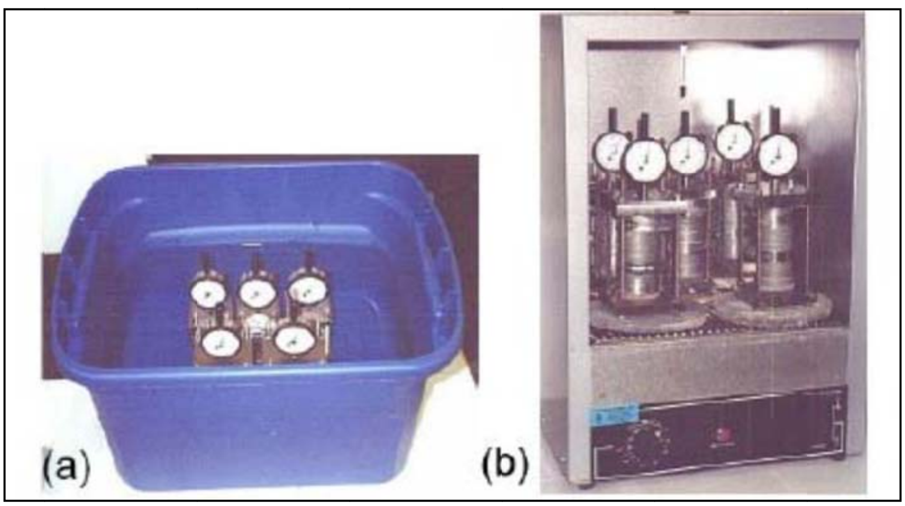


Figure 2.9 (a) Wet and (b) Dry cycle setup used by Hoyos et al. (2005)

Chittoori (2008) Pedarla et al. (2011) and Lad (2012) have also performed durability studies to study to effects of cyclic w-d on chemically stabilized soils. Lad (2012) developed a single device for both wetting and leachate studies. Figure 2.10 shows the new device developed by Lad (2012) for wetting cycles and leachate studies.

Figure 2.10 Wetting and Leachate study device developed by Lad (2012)

Several studies have been conducted to evaluate the long term performance of chemically treated soil under severe w-d environments as opposed to only a handful studies on CLSMs under similar conditions. Vanga (2013) conducted durability studies on CLSM using native soil as fine aggregates. Native soils from four different geological formations around Texas namely Woodbine, Eagle Ford, Austin Chalk and Queen City were used to prepare CLSM mix along with varying dosages of cement and water. Long term performance of these CLSMs was studied by evaluating the influence of cyclic w-d process on unconfined compressive strength (UCS), volumetric and weight changes and additive loss of the specimen. The study concluded that CLSM prepared in the laboratory using Eagle Ford soil, high plasticity clay (CH), experienced the most volumetric and weight change and retained only about 17% of initial strength.

2.9.3. Leachate Studies

One of essential aspects of soil stabilization is to address the permanency of the chemical stabilizer used i.e. the duration the additive holds the soil particles together (Chittoori, 2008). Soil stabilized using chemical stabilizers such as cement upon exposure to water tends to lose its strength over time. One of the factors causing the strength loss of soil is loss of stabilizer through leaching.

Only a handful researches have been conducted on the leachate studies of chemically-treated soils to comprehend the leaching of chemicals from moisture flows (Barenberg, 1970; McCallister, 1990; and Thompson 1966). It was observed that soil leaching has a direct influence on the properties such as soil pH, percentage base saturation and calcium/magnesium ratios and is directly related to the permeability of the soil. He stated that soil-lime reactivity decreases in areas of high permeability. In soils with very low permeability i.e. fine grained soils the leaching effects are minimized thus maintaining the calcium/magnesium ratios and higher soil pH (Chittoori, 2008). Another study by McCallister (1990) reports that leaching through moisture flow cause variation in pH and calcium ions in chemically stabilized soils.

Chittoori (2008) used a modified version of McCallister (1990) test to study leaching behavior of chemically stabilized soil. Two series of moisture conditioning tests for leachate studies were conducted on highly expansive soils from various locations in Texas. The first test addressed issues correlating with rainfall infiltration whereas the second test observed the volumetric and strength changes of soil to evaluate the swell/shrink behavior. The study showed that changes in pH were minor and calcium ion concentrations were found to decrease over the course of 14 cycles of leaching.

Later on, Lad (2012) designed a wetting apparatus for durability studies which constituted both wetting process as well as leachate testing. The apparatus designed by Lad (2012) tries to simulate the field condition of rainfall infiltration in soil during a heavy rainfall in the laboratory. The study showed that it would take as little as 4 to 5 hours to reach the pore volume saturation of the sample during wetting using this new device instead of the traditional 24 hours wetting period for leachate studies. This apparatus not only cut down the testing time significantly but also eliminated the need for having two separate samples for wetting and leachate studies. Figure 2.11 shows the schematic of the device developed adopted by Lad (2012) for wetting cycle and leachate studies. Standard EDTA procedure was used to determine the loss of calcium due to leaching. The study concluded that leaching may not be highly problematic in the initial years if the treatment dosages are high (6% or higher).

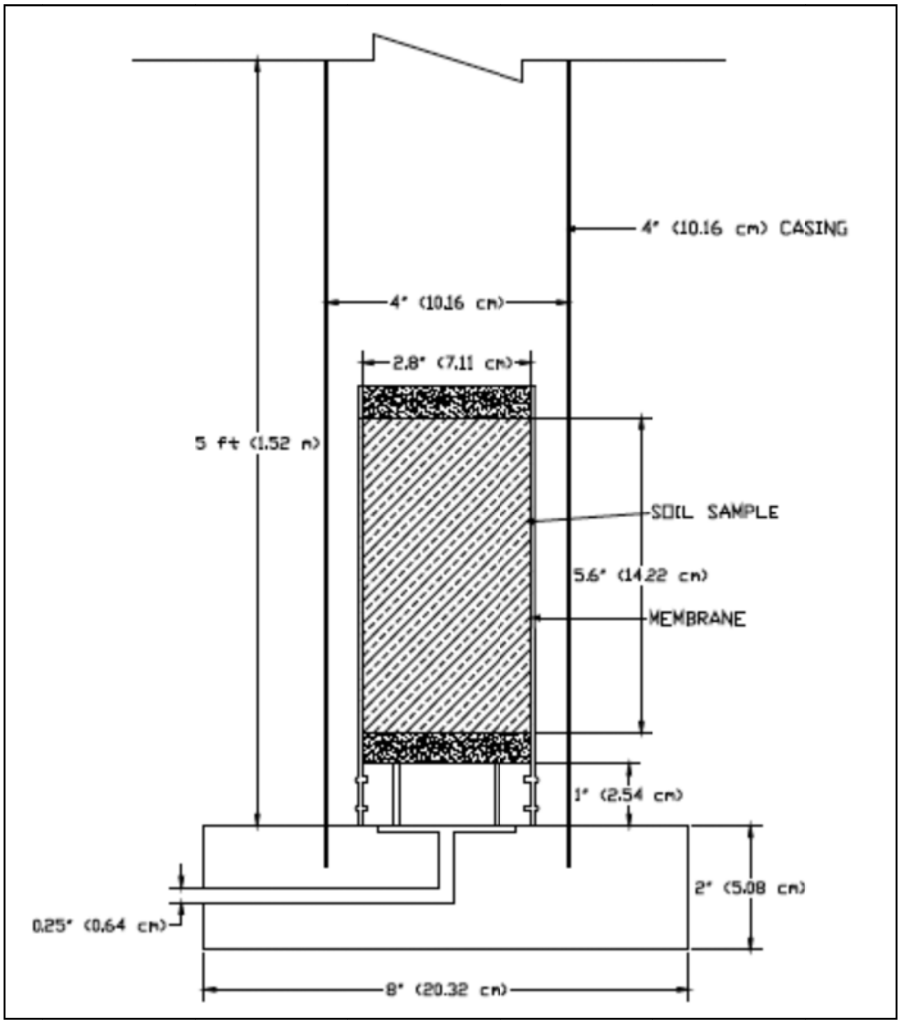


Figure 2.11 The schematic of the device developed by Lad (2012) for wetting cycle and leachate studies

2.10 Summary

Based on the literature, we can understand 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. So far no study has been conducted to evaluate the effects of cyclic w-d on CLSM prepared with high plasticity clay with elevated sulfate concentrations.

Due to the concerns relating to sulfate-heaving, the need for research on long term durability performance of CLSM mix designs prepared using native high plasticity clay with various sulfate concentrations is crucial. This chapter summarizes the benefits of using CLSM mixtures and also addresses the performance under durability studies. The next chapter describes the methodology used in the current research to address the long term performance of sulfate bearing CLSM mixtures.

CHAPTER 3

EXPERIMENTAL PROGRAM

3.1 Introduction

The present research focuses on long-term performance of native soil CLSMs using high sulfate expansive soils. For this purpose, high plasticity clay (CH) from Eagle Ford formation, with elevated concentrations of sulfates was selected. This chapter presents the experimental program followed in this research including, sample preparation and the various laboratory tests and the testing procedures to meet the desired research objectives. Some of the procedures include sulfate testing, CLSM specimen preparation method, durability testing (wetting and drying cycles), leachate and strength testing.

The following sections describe background on soil selection, testing materials, types of laboratory tests performed and test equipment used for the durability and strength studies on CLSM using high plastic soil with elevated sulfate concentrations.

3.2 IPL Project Background

As mentioned previously, this study is a part of the Integrated Pipeline (IPL) project, which is a joint effort between the Tarrant Regional Water District (TRWD) and Dallas Water Utilities (DWU). The project intends to bring additional water supplies to the Dallas/Fort Worth metropolis. As a part of the project, several studies were conducted by the University of Texas at Arlington to evaluate the reuse potential of the excavated materials along the IPL pipeline alignment. One of these studies involved using the excavated material as a constituent in Controlled Low Strength Material often known as CLSM or flowable fill. CLSM can be used as a bedding and backfill material during pipeline construction. The key objectives of this research study are assessing the long-term performance of various CLSMs using Eagle Ford soils with elevated levels of soluble sulfates, as fine aggregate, by conducting the durability studies. The sections below describe the procedures adopted to perform the sulfate studies.

3.3 Soil Selection

For this study, soil from Eagle Ford geological formation was selected, as the soluble sulfate concentrations of this formation ranged from 40 ppm (at 5 ft. depth) to approximately 20,000 ppm (at 20 ft. depth) along the IPL pipeline alignment (Thomey, 2013). Hence, soil from the lowest concentration along the IPL alignment was collected and spiked with different sulfate concentrations to include the range of sulfates observed by Thomey (2013) in this formation. Table 3.2 shows the various concentrations of soluble sulfates used for testing namely, the control soil (no sulfate, 0-100ppm), 2500, 5000, 10000, 20000 ppm based on the range of soluble sulfate concentrations observed on Eagle Ford formation. It should be noted that the CLSM samples prepared using control soil should have soluble sulfate concentrations less than or equal to 100 ppm symbolizing ‘no-sulfate’ scenario for analysis purposes.

Also, the classification details of the Eagle Ford soil being tested in this study are presented in Table 3.1. Sieve and hydrometer analysis were conducted per ASTM D 422 and Atterberg’s Limit Tests (Liquid Limit and Plastic Limit) were conducted per ASTM D 4318 to find the plasticity index of the soil. Soil classification was based on Unified Soil Classification System (USCS). Figure 3.1 shows pulverized Eagle Ford soil used for testing. Figure 3.2 shows the location of Eagle Ford formation along the proposed IPL project pipeline alignment.

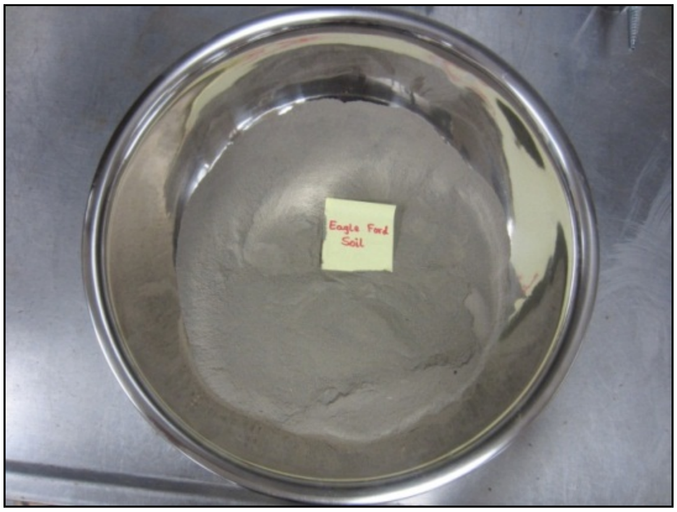


Figure 3.1 Pulverized Eagle Ford (EF) Soil

Table 3.1 Summary of Laboratory Testing on EF soil

Soil Formation	Standard Gradation				Plasticity Index (%)	Soil Type based on USCS
	Sieve Analysis		Hydrometer			
	Gravel (%)	Sand (%)	Silt (%)	Clay (%)		
Eagle Ford	0.5	6.5	43	50	32	CH

Table 3.2 Test Soil Locations and Soluble Sulfate Contents of the Selected Soils

Soil Type	Soluble Sulfates, ppm *
Sulfate Soils	2,500
	5,000
	10,000
	20,000
Control Soil	< 100

* For EF soil with sulfates < desired sulfate concentration,
Additional sulfate was added in the form of Gypsum (Calcium Sulfate Dihydrate, $CaSO_4 \cdot 2H_2O$)

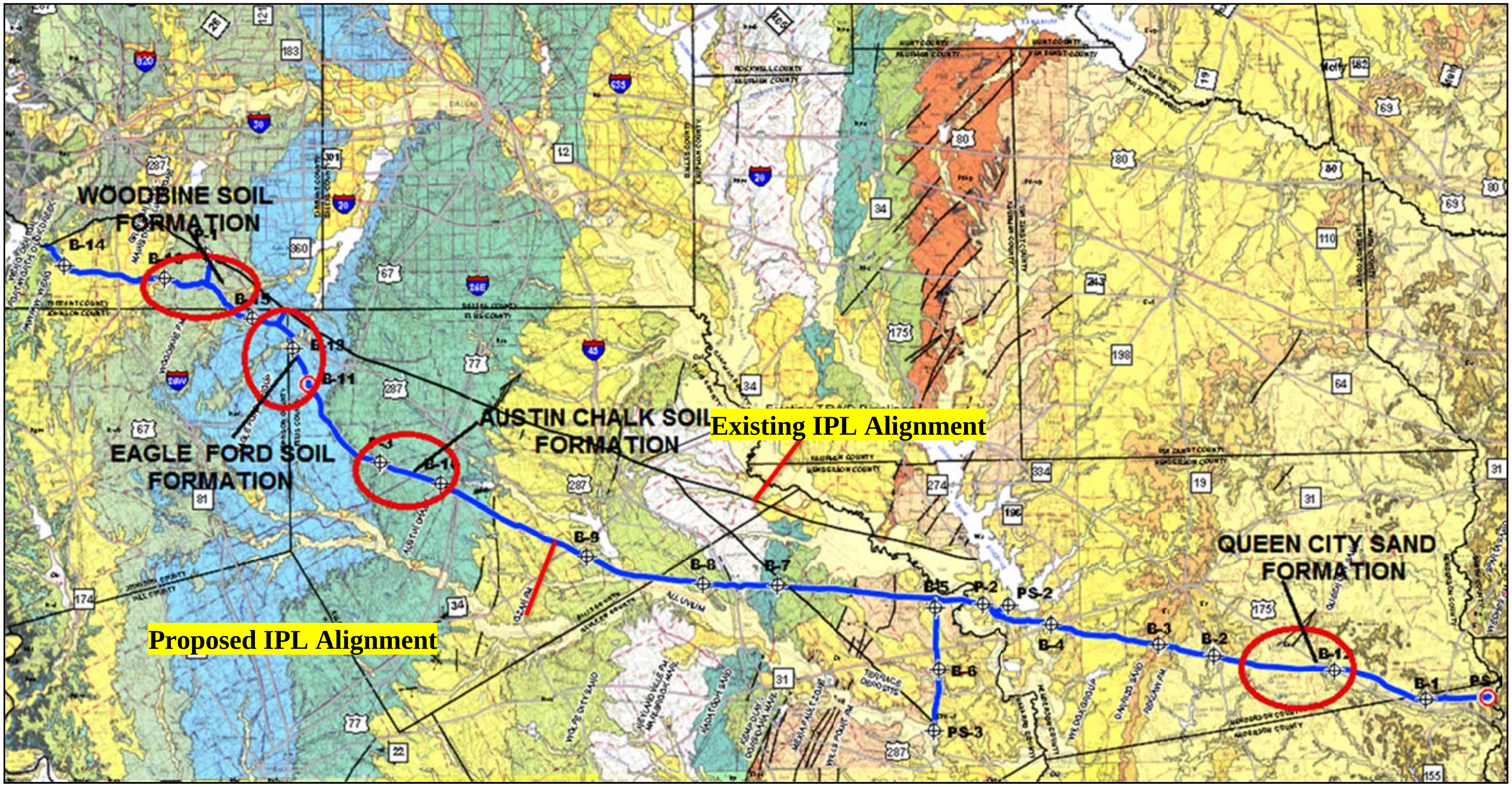


Figure 3.2 Location of Eagle Ford Formation along the Proposed IPL Alignment (Photo Courtesy: IPL Project)

3.4 Methodology

The main purpose of this part of the research is to evaluate the long term performance of CLSM samples using EF (high plasticity clay) soil with elevated sulfate concentrations. In other words, per TRWD's requirements, the primary focus of sulfate research study involves short term strength and long term durability studies. The short term strength studies include 1, 3, 7 and 28 days Unconfined Compressive Strength (UCS). The short term strength tests were performed to evaluate any significant change in strength over a 28 days curing period. It was also one of the requirements of TRWD. The long term durability studies involves 1, 3, 7 and 14 cycles durability test followed by UCS test and leachate testing. A stepwise testing procedures followed to conduct sulfate research study is shown in Figure 3.3. The long term durability test were performed to access the long term behavior of CLSM samples in terms of strength, weight loss, volume change and additive loss over time.

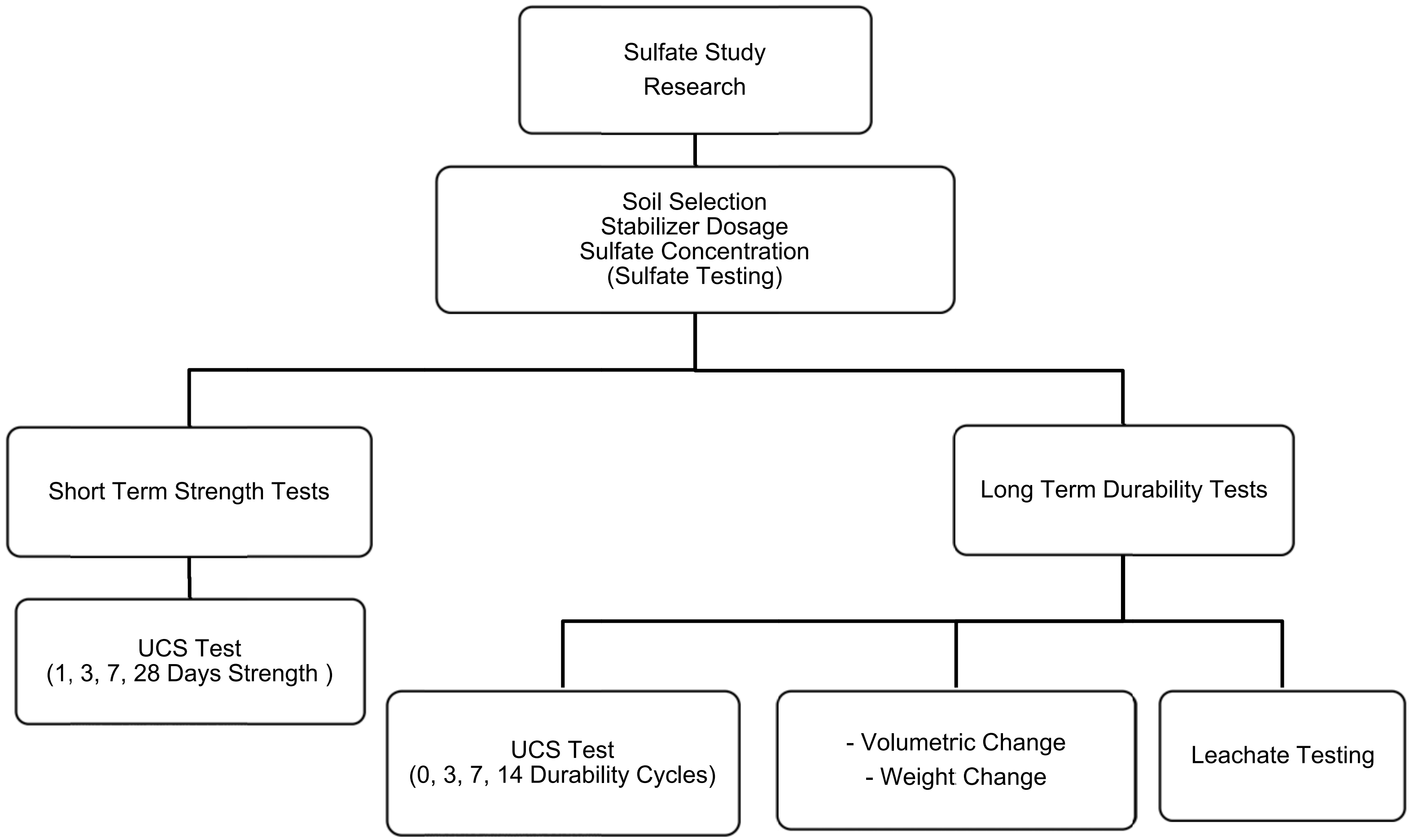


Figure 3.3 Flowchart Showing the Experimental Program Followed in this Research Study

3.5 Testing Parameters

The primary testing parameters or variables included soil, sulfate contents, stabilizer (binder) type, stabilizer dosage, curing conditions and duration. Among these sulfate concentrations, curing time and durability cycles were the testing variables. As mentioned before the soil selected for the sulfate research project was from Eagle Ford formation that was provided by TRWD. The soil was subjected to five different sulfate concentrations with one dosage of Portland cement (TY I/II) as a stabilizer. In most of the cases where sulfate concentrations in the soil fell below 100 ppm, sulfate was added in the form of gypsum (calcium sulfate dehydrate, $CaSO_4 \cdot 2H_2O$) to meet the desired sulfate concentrations. Stabilizer dosage of 18% (by weight) was recommended by Fugro Consultants Inc. and was also used by Vanga (2013) for the durability studies on Eagle Ford soil with less than 100 ppm sulfate concentration. Table 3.3 lists the testing parameters applicable for the current research project.

Table 3.3 Testing Parameters

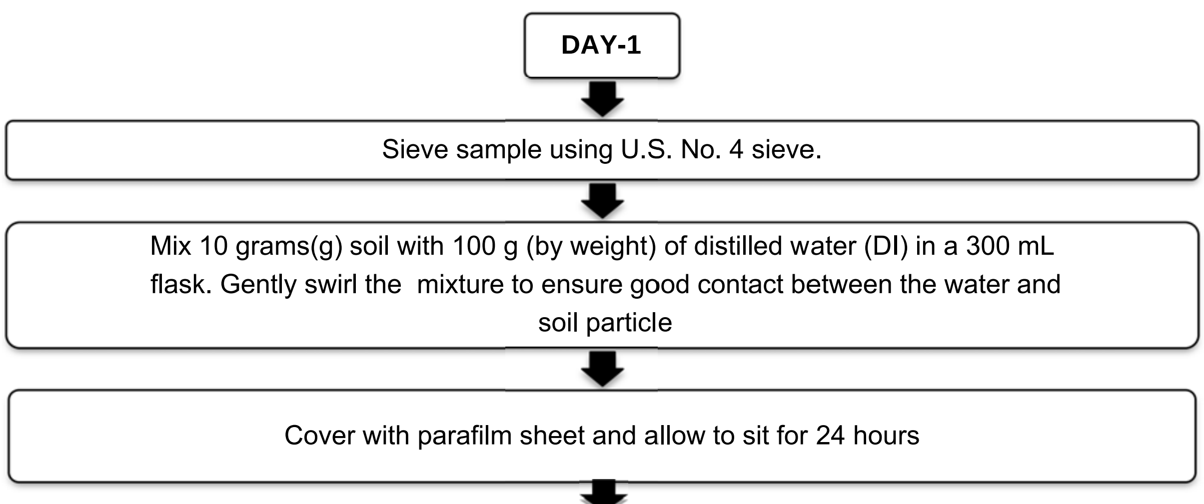
Description	**Variable**	
	Quantity	**Name**
Soil	1	EF Soil
Sulfate Content	5	- Less than 100 ppm (Control Soil) - 2500 ppm - 5000 ppm - 10000 ppm - 20000 ppm
Stabilizer	1	Portland Cement (TY I/II)
Stabilizer Dosage	1	18% (by weight)
Distilled Water	1	(95)% (by weight)to reach flowability (8-12 inches)
Curing Time	2	7 and 28 days
Curing Conditions (for durability tests)	1	7 days counter top, 28 days: 100% relative humidity, 20±3 ^oC
Durability Cycles	4	0, 3, 7 and 14 Cycles (Alternate drying and wetting cycles)

3.6 Soluble Sulfate Testing Procedure

Several sulfate testing procedures are available to determine the concentration of soluble sulfates in soil namely AASHTO Method, TXDOT Method, and Modified UTA Method. However, Thomey's (2013) research work on sulfates showed that Modified UTA Method was fast and was the most accurate among the other two methods.

3.6.1 Modified UTA Method for Soluble Sulfate Determination

This method of soluble sulfate concentration determination was developed by Puppala el al. (2002) which is a modification to the existing UTA method by Petry (1994). The basis for both the methods is the standard gravimetric procedure discussed by Clesceri (1989). The modified procedure helps in finding the sulfate concentration in a soil by precipitating the soluble sulfate using barium chloride solution. The divalent cation of barium (Ba^{2+}) being insoluble in water combines with the divalent sulfate anion (SO_4^{2-}) to form white precipitate of barium sulfate ($BaSO_4$). Using stoichiometry techniques, the weight of sulfate from barium sulfate powder is determined which is then converted to ppm, a typical unit to express soluble sulfate concentration. Figure 3.4 (a), 3.5 (b) and 3.5 (c) show the test procedure for the Modified UTA. Figure 3.7 shows the illustration of the Modified UTA sulfate testing procedure.



(a)

Figure 3.4 Flowchart showing the Stepwise Procedure of Modified UTA (a) Day 1 (Puppala et al., 2002; Thomey, 2013)

DAY-2

Using an Eiberbach shaker, shake in low for 30 minutes

Transfer all the shaken sample to centrifuge tube using DI water. Make sure all the water and soil particles are transferred. Maintain a constant water level in all the centrfuge tubes.

Set the centrifuge device to Program 1(10,200 RPM for 30 mins) and place the centrifuge tubes in the device.

Remove the tubes from the centrigfuge
Filter the supernatant accross 0.1μm filter paper.
Wash the walls of the filter funnel with DI water to ensure all soluble sulfates were transferred.
Use a .45μm filter paper first if you suspect the sample will take a long time to filter.

Transfer the filtrate to a flask and cover with glass stopper (use DI to transfer all liquids)
Dilute the filtrate to 200 mL using DI water and bring to light boil

Add 40 mL of 10% Barium Chloride solution while maintaining the temperature

Remove from heat and allow the solution to cool for 15 minutes. Cover the mouth of the flask with a glass lid and place in oven at 140 °F for 12 hours. Do not exceed 80 °C (140 °F)

Wet 0.1 μm filter paper with DI water in an aluminum weighing tin and place it in oven to dry overnight

(b)

Figure 3.5 Flowchart showing the Stepwise Procedure of Modified UTA (b) Day 2 (Puppala et al., 2002; Thomey, 2013)

DAY-3

Remove the now dry filter paper and tin from the oven, allow it to cool.
Weigh the filter paper with the aluminum tin using the analyitical balance.
Record the value.

Remove the precipitate solution from the oven and filter it with the now dry filter paper from the previous step.
Use DI and a glass stir rod with rubber policeman to ensure all of the precipitate is removed from the flask.

Place the filtered precipitate back in the aluminum tin then place in the 140°F oven to dry overnight. DO NOT EXCEED 80° C.

DAY-4

Remove the tin and precipitate from the oven, allow to cool.
Weigh on the analytical balance.

Compute soluble sulfates in ppm by using spreadsheet. The difference in weights of the (can and paper) and, (can,paper and sulfate) is expressed in grams. This weight of Barium Sulfate is multiplied by a factor of 41,156 to express it in terms of ppm.

(c)

Figure 3.6 Flowchart showing the Stepwise Procedure of Modified UTA (c) Day 3 and 4 (Puppala et al., 2002; Thomey, 2013)

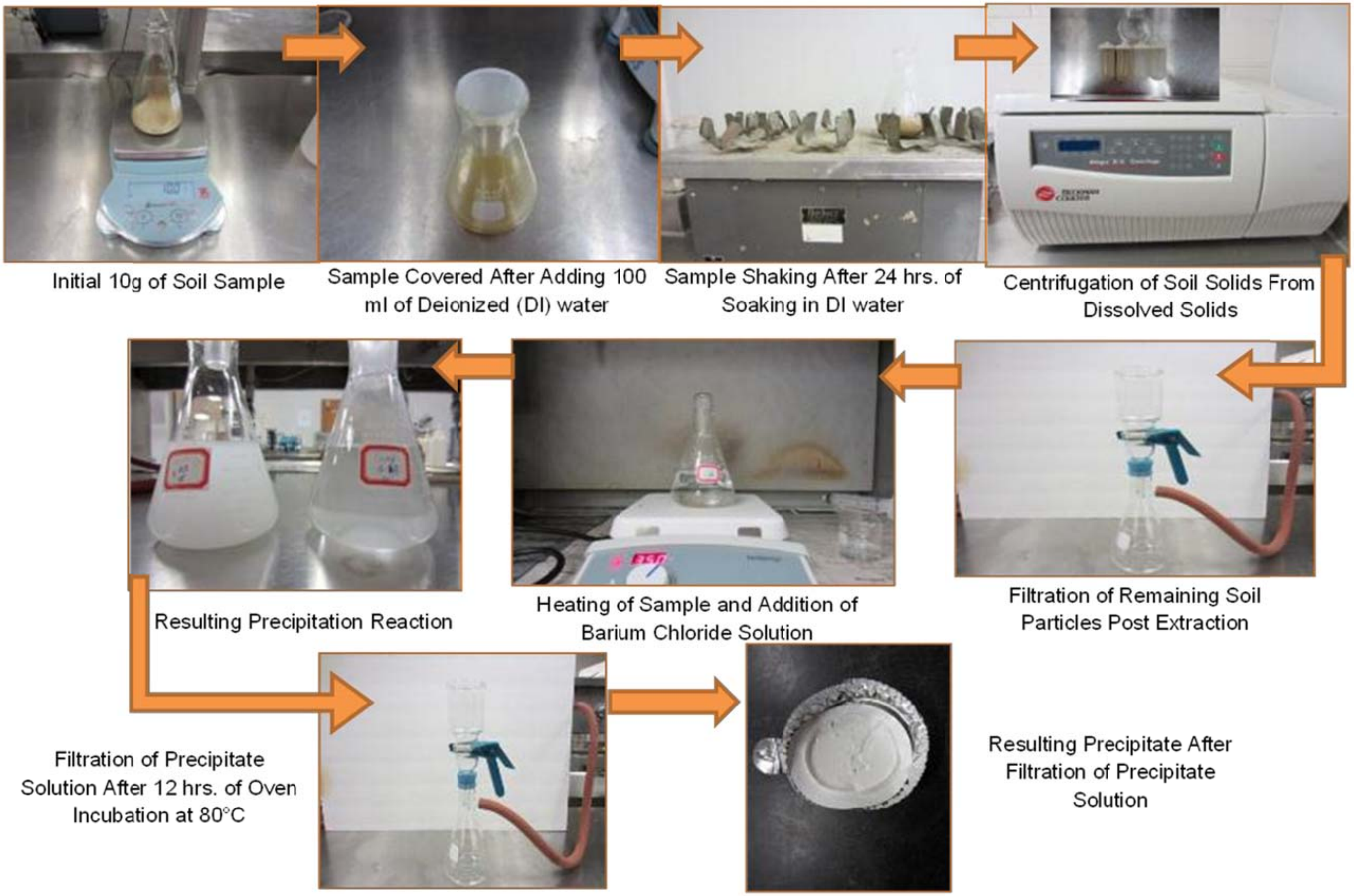


Figure 3.7 Illustrated is the UTA Method for Soluble Sulfates Determination in Soil
(Photo Courtesy: Thomey, 2013)

3.7 CLSM Sample Preparation and Testing

The CLSM sample prepared using native excavated soil can be termed as native-CLSM. Since every CLSM mix design is project specific and varies in its constituents depending on the application, there is no standardized sample preparation procedure available for native-soil CLSM mix design in the literature. However, sample preparation technique discussed by Folliard et al. (2008) was adopted (with some modifications) for this study. Folliard's procedure is most applicable for medium stiff to stiff clay.

3.7.1 Specimen Preparation

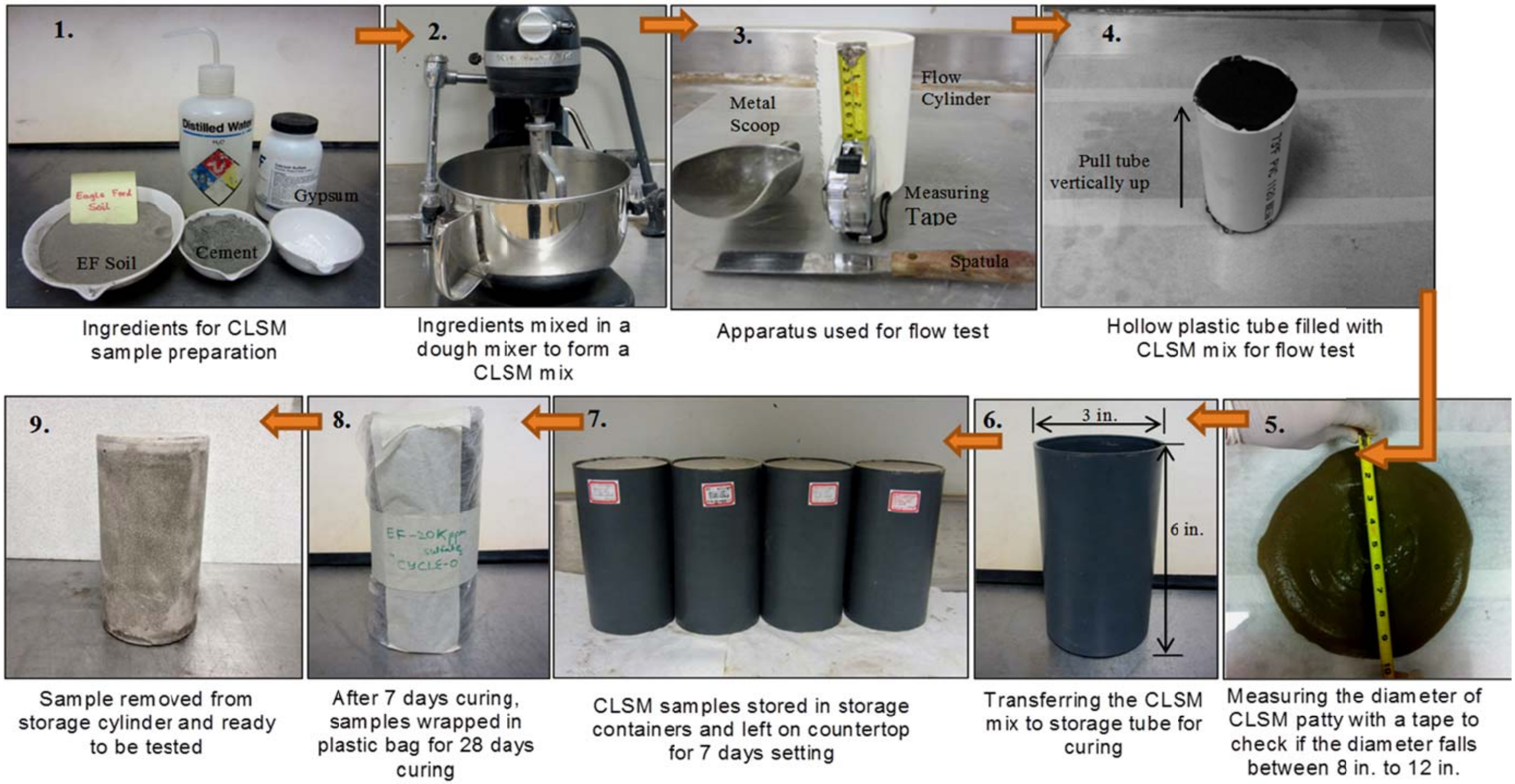
The very first step in sample preparation was to oven dry the Eagle Ford soil provided at 60 °C. The oven died soil was then pulverized to ensure it passed through U.S. Sieve 40 (0.425 mm). Desired amount of crushed soil was then mixed with the specified amount of Portland cement (TY I/II), 18% by weight, along with sulfate (in the form of gypsum). Once the soil is mixed with proposed amount of dry sulfate and cement, the mix is placed in a conventional dough mixer. The mixing rate of the outer and inner spindle was 60 rpm and 752 rpm respectively. These rates were set by Raavi (2012) using trial and error method to allow sufficient mixing time without soil-binder lump formation. Water content in the CLSM mix design is determined by the flow test. Once the water content is set, the specified amount water of was slowly added to the mix. With the aid of a spatula, the water was mixed evenly preventing any soil from sticking at the bottom of the mixer. From previous practices of CLSM sample preparation, about 8 to 10 minutes were allocated for mixing the sample in the mixer after the addition of water. Then the mixer was turned off and the CLSM mix was poured to plastic storage cylinders with dimensions of 6 in. high and 3 in. diameter for curing.

3.7.1.1 Short Term Strength Test Sample Preparation and Testing

A short term strength study only involves the UCS test. For the strength tests, samples for all 1, 3, 7 and 28 days strength test were prepared for all sulfate concentrations. The 1, 3 and 7 Day samples were left on countertop before UCS, however the 28-Day samples were left on countertop for the first 7 days and then transferred to the moist room for the remaining 21 days before UCS test, making the total curing period to 28 days. Leaving the sample outside for 7 days was a typical method adopted by Folliard (2008) which allows surface water to evaporate maintaining the actual moisture content of the sample. This moist room curing allows for the moisture to be conserved in the sample instead of being dried out. After sample preparation, samples were directly subjected to UCS test upon completion of their curing/setting period. For instance, for a 3-day sample, UCS test is conducted on them at the end of 3 full days of setting from the time the samples are casted. A total of 40 samples for all the sulfate concentrations including the control soil were prepared for testing. The samples were left on countertop till their respective test date approached. The samples were then tested for strength using UCS test on 1, 3, 7 and 28 days of casting. Figure 3.8 shows the stepwise procedure for the laboratory preparation of CLSM samples using native Eagle Ford soil in which steps 1 through 6 are followed for short term strength tests.

3.7.1.2 Long Term Durability Test Sample Preparation

For long term durability studies, a total of 20 samples were prepared for 0, 3, 7 and 14 cycles for all sulfate concentrations. After the samples were filled in the cylindrical plastic storage tubes, they were left on countertop for 7 days and then transferred to the moisture room with 100% relative humidity for 21 days, with a total of 28 days curing. The samples for long term durability test were not directly subjected to humidity controlled curing but instead the samples were left on countertop at room temperature for 7 days setting period. The main reason for this curing method is that the CLSM samples are still in wet condition (flowable condition) after casting. The samples were transferred in the moisture room without taking them out of the plastic storage tubes. It is important that the samples be wrapped air-tight with a plastic wrapper before being stored in the moisture room as shown in step 8 of Figure 3.8. This prevents free water present in the moisture room from entering the samples altering the moisture content of the sample. This curing process of 7 and 21 days is similar to the one presented in NCHRP Report 597 by Folliard et al. (2008). Once the samples are cured for 28 days, the samples are subjected to durability tests of wetting and drying cycles. The Figure 3.8 shows the stepwise procedure for the laboratory preparation of CLSM samples using native Eagle Ford soil in which steps 1 through 9 are followed for short term strength tests.



* For short term strength tests, procedure 1 through 6 and for long term durability tests steps 1 through 9 are followed to prepare samples.

Figure 3.8 Step by step procedure for Laboratory Preparation of CLSM Samples

3.7.2 Flow Test

Step two includes the flow test which is performed to determine the amount of water required for the CLSM mix design to meet the ASTM flowability standard. Flow test was conducted per ASTM D 6103 to determine the workability of the CLSM mix design and its ability to flow in confined areas (Raavi, 2012). The criterion as outlined in the ASTM standard is to obtain a target CLSM circular spread diameter of 203 mm (8 in.) to 305 mm (12 in.) by preparing a flowable mix.

The apparatus used for the flow test have been listed below:

- Flow Cylinder - 150 mm (6 in.) high
  - 76 mm (3 in.) inside diameter
- Square acrylic plate - 2 ft. X 2 ft. (non-porous)
- Spatula (as straight edge)

Measuring tape Firstly, to conduct a flow test, an acrylic plate was placed flat on a leveled surface followed by placing a slightly dampened flow cylinder at the center of the plate. Both ends of the flow cylinder were open to allow the passage of CLSM mix through it. The inside wall of the cylinder is smooth to minimize frictional resistance. As a second step, the CLSM mix prepared in the dough mixer was scooped and slowly poured into the flow cylinder avoiding as much air voids as possible. Once the cylinder was filled upto the top, spatula (as a straight edge) was used to remove the excess CLSM and maintain a level top surface. Then the flow cylinder was quickly raised in a vertical direction within 5 seconds allowing the CLSM to spread forming a circular patty. Using a measuring tape, two diameters of the patty, perpendicular to each other were measured. Once the patty diameters were between 8 in. to 12 in., the total water added was recorded and was used as the standard water content to be used for the CLSM mix design. Figure 3.8 (steps 3 to 5) shows the visual representation of the flow test in the laboratory along with the apparatus used for the test.

3.7.3 Long Term Durability Study

For this research task, durability study constituted alternate drying and wetting cycles. The standard test method used for wetting and drying compacted soil-cement mixtures is given by ASTM D 559. This standard helps to stimulate field conditions of seasonal drying and wetting cycles and rainfall infiltration in the laboratory, in a relatively short amount of time. However, for this research purpose, a slightly modified test procedure using a new device devised by Lad (2012) was used. The study conducted by Lad (2012), shows that the new device constitutes both wetting and leachate studies into a single test instead of a traditional approach involving two separate tests. The study also showed that it would take as little as 4 to 5 hours to reach the pore volume saturation of the sample during wetting (saturation) as opposed to the traditional 24 hours wetting period for leachate studies.

Adopting Lad's (2012) wetting and leachate study apparatus, the wetting and drying cycles for CLSM were carried out with the help of a conventional oven maintained at 140 °F. At the end of 28 days curing period, the durability samples were exposed to wetting and drying cycles. For this research task, 0, 3, 7 and 14 durability cycles were studied.

3.7.3.1 Wetting/Drying Procedure for Long Term Durability Studies

The wetting process constituted of wetting the native CLSM samples in potable water for 5 hours at room temperature using the apparatus shown in Figure 3.9 (a). Water head was maintained on the test device at 5 ft. since that was the standard used for previous studies using the device. However, it is not necessary to use a 5 ft. water head for wetting process. It was strictly used for consistency in testing. The CLSM sample was encased with a latex membrane on the outside using vacuum suction and a hollow cylindrical tube. The latex membrane acted as a barrier between the water bath and the sample and held the sample intact during the wetting process. Once inside the membrane, the sample was then transferred to the wetting apparatus and fastened to the base of the apparatus using O-rings which kept the soil specimen intact with the latex membrane. A plastic cap with two holes in it was placed on top of the sample with O-rings around it. This facilitates vertical movement of water through the holes of the plastic cap while still preventing radial water movement. Ultimately, the tall plastic casing was grooved around the base of the apparatus with the help of vacuum grease to prevent water leakage. Then the apparatus was filled with water through the upper outlet using a plastic tube. Upon reaching the 5 ft. tall water height marked on the tall plastic casing, the water tap was stopped and a plastic tube was used to connect the upper and lower outlets allowing vertical circulation of water through the sample. The sample was subjected to 5 hours of wetting process. Figure 3.10 shows a detailed illustration of the wetting process.

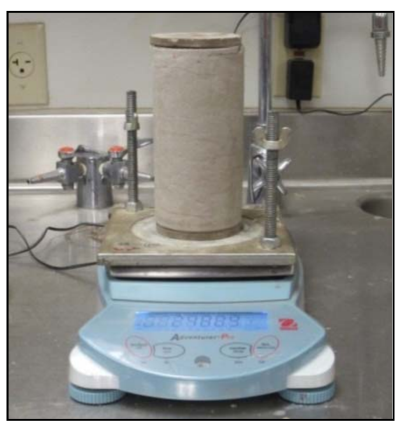
As for the drying process, ASTM D 559 calls for a drying period of 42 hours and a wetting period of 5 hours to constitute one complete wetting/drying cycle. However for this research task, a modified approach involving a drying period of only 24 hours in a conventional oven maintained at 140 °F was considered as shown in Figure 3.9 (b). Because the ASTM's 42 hours drying were found to be too severe, the modified approach of 24 hours drying was found to be a more suitable approach. After the completion of each drying and wetting process, the volume and weight changes were recorded along with a photograph of the sample as shown in Figure 3.9 (c), (d) and (e).

As for the samples in the moist room, at the end of the curing period, the samples for Cycle-0 were submerged in water for 5 hours at room temperature. Followed by the wetting process, UCS tests were conducted on all the Cycle-0 samples. On the other hand, samples for Cycle-3, Cycle-7 and Cycle-14 after 28 days curing, were taken out of the moist room. Height, diameter and weight of the samples were measured and recorded, followed by a 24 hour drying process. Vernier caliper and weighing balance were used for measuring the volume and weight changes of the specimen. Durability cycles were carried out till all the cycles were completed or until the sample collapsed.

Furthermore, at the end of each cycle, for instance, at the end of cycle-0, cycle-3, cycle 7 and cycle-14, UCS test was conducted to determine the strength at the end of that cycle. Since each cycle starts with 24 hours drying process followed by a 5 hour wetting, with the exception to cycle-0, one complete wetting/drying (w-d) cycle ended at a wetting cycle. Hence, for durability studies, all the UCS tests were performed at the end of a wetting cycle and not a drying cycle. Wetting cycle was considered as the cycle closure for several reasons. Creating a worst case scenario for testing and possibility of linking leachability studies to strength variation are some of the reasons for using wetting cycle as a cycle terminator. It should be noted that leachate is collected in a cylindrical tube at the end of cycle before conducting UCS test.

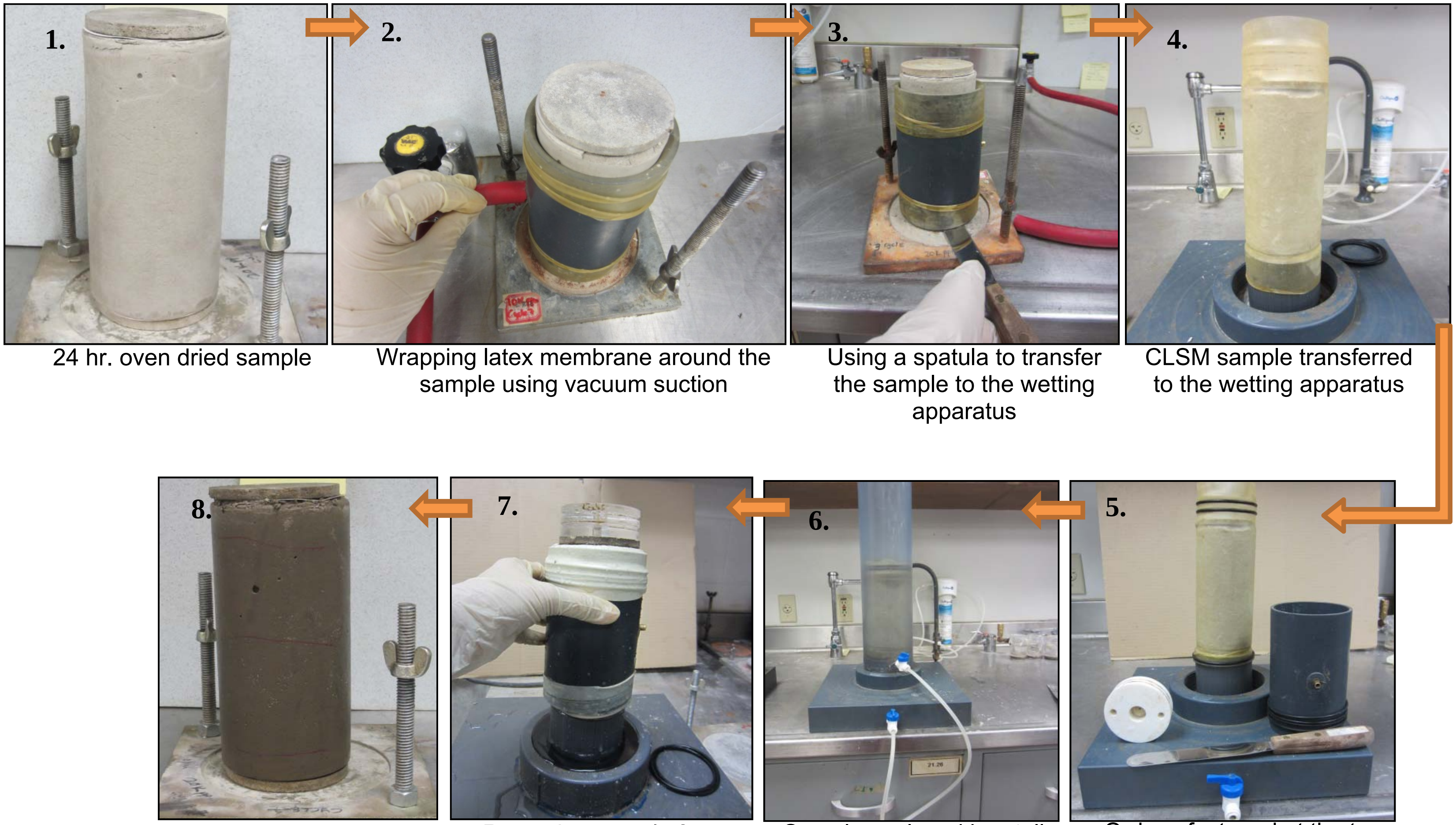
(a) (b)

(c) (d)



(e)

Figure 3.9 Long Term Durability Tests on CLSM Samples showing (a) Wetting Process, (b) Drying Process, (c) (d) Measurement of Volumetric Change using vernier caliper, (e) Measurement of Weight Changes using weighing Balance



1. 24 hr. oven dried sample
2. Wrapping latex membrane around the sample using vacuum suction
3. Using a spatula to transfer the sample to the wetting apparatus
4. CLSM sample transferred to the wetting apparatus
5. O-rings fastened at the top and bottom of the apparatus
6. Sample enclosed in a tall plastic casing filled with
7. Removing sample from wetting apparatus using
8. Sample completed wetting process and ready for 24 hr. oven drying

Figure 3.10 Detailed Illustrations of Steps for Wetting Process

3.7.4 Leachate Studies

One of the most important aspects of soil stabilization is to address the permanency of the chemical stabilizer used i.e. the duration the additive holds the soil particles together (Chittoori, 2008). Soil stabilized using chemical stabilizers such as cement upon exposure to water tends to lose its strength over time. One of the factors causing the strength loss of soil is loss of stabilizer through leaching. McAllister (1990) conducted leachate tests and Chittoori (2008) used a modified test to study leaching behavior of chemically stabilized soil. The wetting and leachate collection device designed by Lad (2012) was used for leachate tests.

3.7.4.1 Leachate Test Procedure

The apparatus designed by Lad (2012) tries to simulate the field condition of rainfall infiltration in soil during a heavy rainfall in laboratory. At the completion of each cycle such as cycle 0, cycle 3 or cycle 7, which always ends at a wetting cycle, before draining all the water out of the tall plastic casing, approximately 50 mL of leachate was collected from the bottom outlet as shown in Figure 3.11. McAllister (1990) indicates that previous studies on leaching report that leaching through moisture flow cause variation in pH and calcium ions in the chemically stabilized soils. In order to access the amount of calcium ions lost due to leaching, standard EDTA method was used.

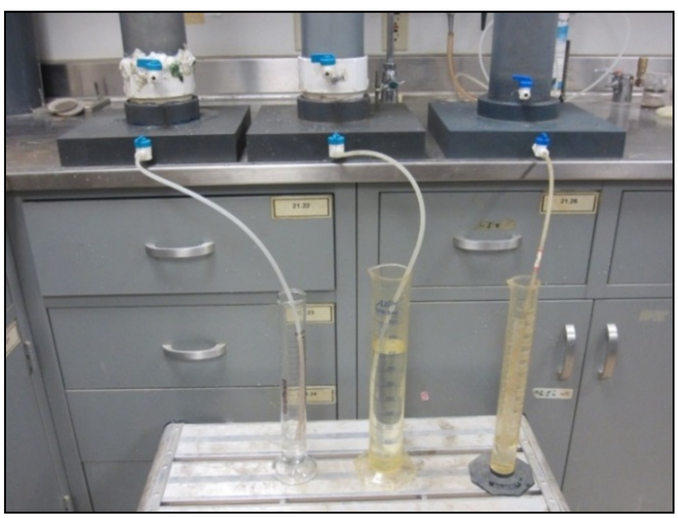


Figure 3.11 Leachate Collecting Apparatus

3.7.4.2 Calcium Determination by Standard EDTA Method

The leachate collected after completion of each durability cycle was subjected to Standard EDTA test to determine the amount of calcium (present in cement) lost due to leaching. Figure 3.12 shows the flowchart presenting a stepwise procedure of Standard EDTA test.

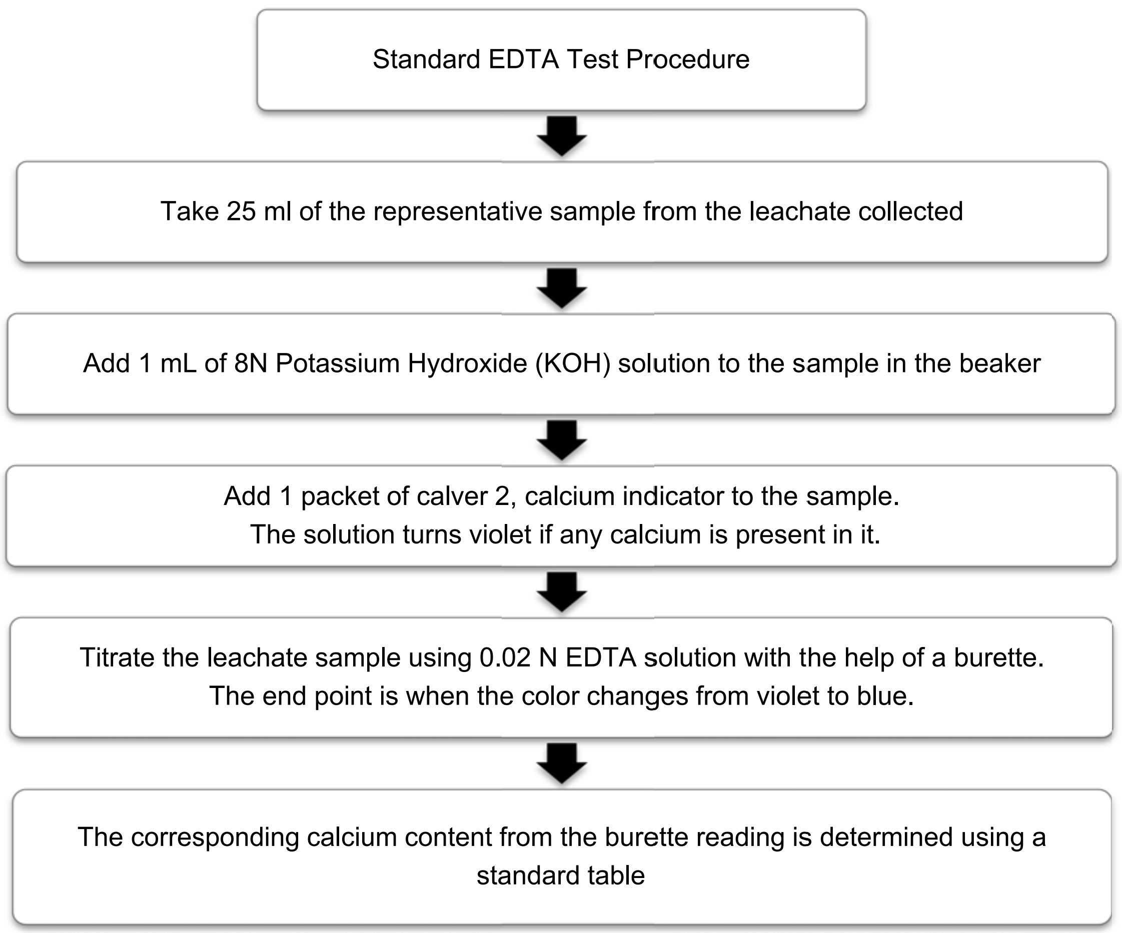
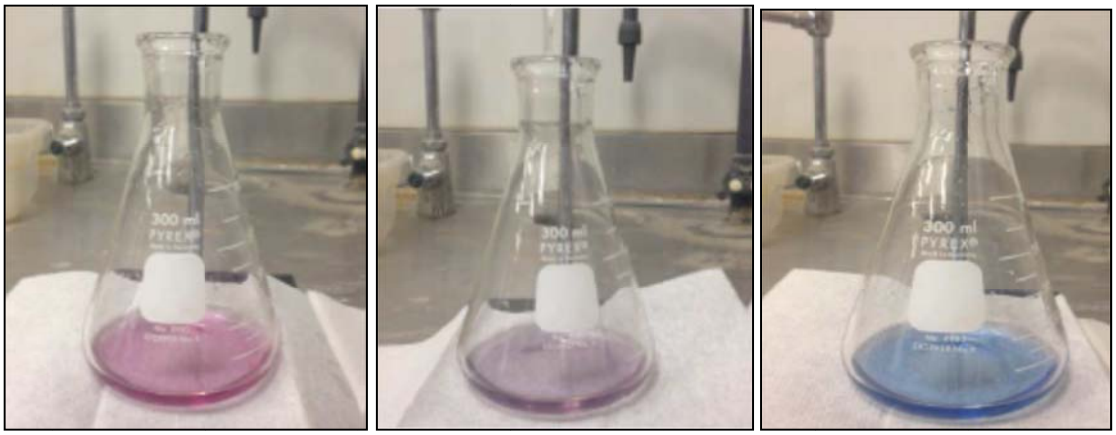


Figure 3.12 Flowchart showing Stepwise Procedure to Determine Calcium Concentration Using Standard EDTA Method (Pedarla, 2008)

Figure 3.13 shows the various stages of the titration process. Initially, after the addition of Potassium Hydroxide (KOH) and calver2 to the leachate sample, the solution turned pinkish purple as shown in Figure 3.13 (a). The solution was gradually titrated with 0.02N EDTA solution. After some time of titration, the color of the solution turned light purple/ violet and ultimately to blue as shown in Figure 3.13 (b) and (c).



(a) (b) (c)

Figure 3.13 Titration stages EDTA Procedure showing (a) Starting Point, (b) Color Change Before End Point, and (c) End Point

Furthermore, to determine the approximate amount of additive (cement) loss from CLSM specimen a Calibration Chart is developed. Determination of % cement leached out helps in better understanding the factors responsible for strength loss for strength loss. CLSMs. Calcium calibration chart helps to assess the total percentage of additives (cement) loss at the end of 14 durability cycles. The chart was developed by plotting known cement contents values as X-Coordinates and their corresponding calcium concentrations as Y-Coordinates. Details about the development of calcium calibration chart are discussed in the following paragraph.

Initially 0.25, 0.5, 0.75, 1, 1.25 and 1.5 grams of oven dried Portland cement (Type-I/II) corresponding to 1%, 2%, 3%, 4%, 5%, and 6% of cement were mixed with 25ml of deionized water to form a cement solution. These solutions were then subjected to EDTA titrations to determine the amount of calcium concentration in terms of calcium carbonate. Upon determining the calcium concentrations (ppm) for each of six known cement solutions, a chart as shown in Figure 3.14 is developed with the cement (%) as Abscissa and Calcium Carbonate (ppm) as ordinate. Using the calibration chart, amount of additive loss can be assessed. Upon knowing the total calcium loss at the end of 14 cycles, the amount of cement% leach out from each specimen can be obtained, correspondingly from calibration graph.

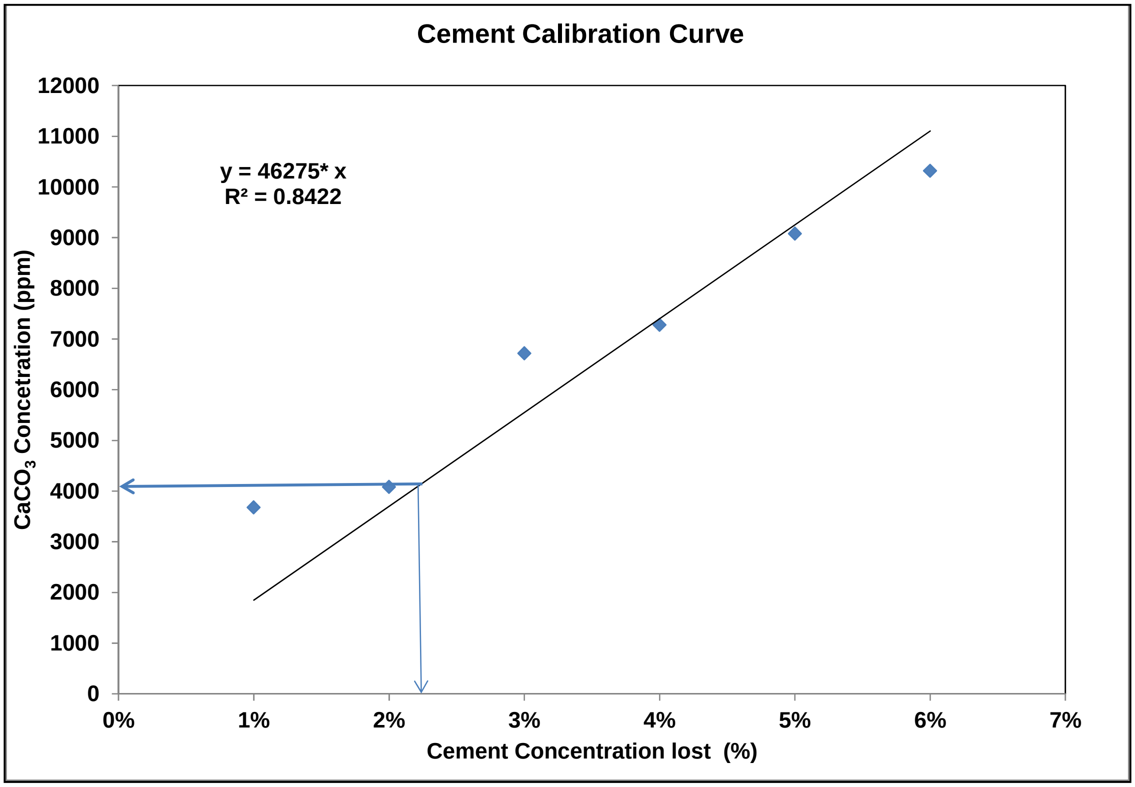


Figure 3.14 Calibration Chart Developed for Determination of Cement Loss (%)

3.7.5 Unconfined Compressive Strength (UCS) Test

All the strength tests discussed in this research, both for short term strength determination and long term durability studies, were conducted using the Unconfined Compressive Strength (UCS) test per ASTM D 2166. As the name indicates the test was performed under unconfined conditions. The primary purpose of UCS test is to quickly obtain the approximate compressive strength of soils that have adequate cohesion to allow testing in unconfined conditions (ASTM D 2166) unlike direct shear or tri-axial test which are extremely time consuming. The UCS testing procedure is described below.

The main components of a UCS test set up include compression device (hydraulic loading device), a load cell, a LVDT reader (to record displacement) and a data acquisition computer system. The CLSM test sample was placed on the loading flat platform of the UCS test set up and raised at a constant strain rate till it came in contact with the top plate. Both the load and deformation indicator should be zero before testing. Then the test sample was loaded at a constant strain rate. When the load reached the maximum value, cracks began to appear along the CLSM sample. Ultimately the sample failed and the values for all the deformations (δ) and load applied (Q) were recorded using the Data Acquisition System (DAS). Using the relationships shown in equation 1, the maximum unconfined compressive strength (q_u) was determined. Figure 3.15 shows the UCS test setup. Figure 3.16 shows an example of a UCS test graph.

$$\varepsilon = \frac{\Delta H}{H}\ ;\ \sigma = \frac{F}{A_c}\ ;\ A_c = \frac{A}{1-\varepsilon}\ \text{and}\ q_u = \sigma_{max} \qquad \text{Eq. (1)}$$

where, ε = Axial Strain

ΔH = change in length,

H = total length of specimen,

A_c = corrected area of cross section of the specimen,

A = initial area of cross section, ($A = \pi r^2$)

σ = Axial Stress , F= Force, q_u = Unconfined compressive strength

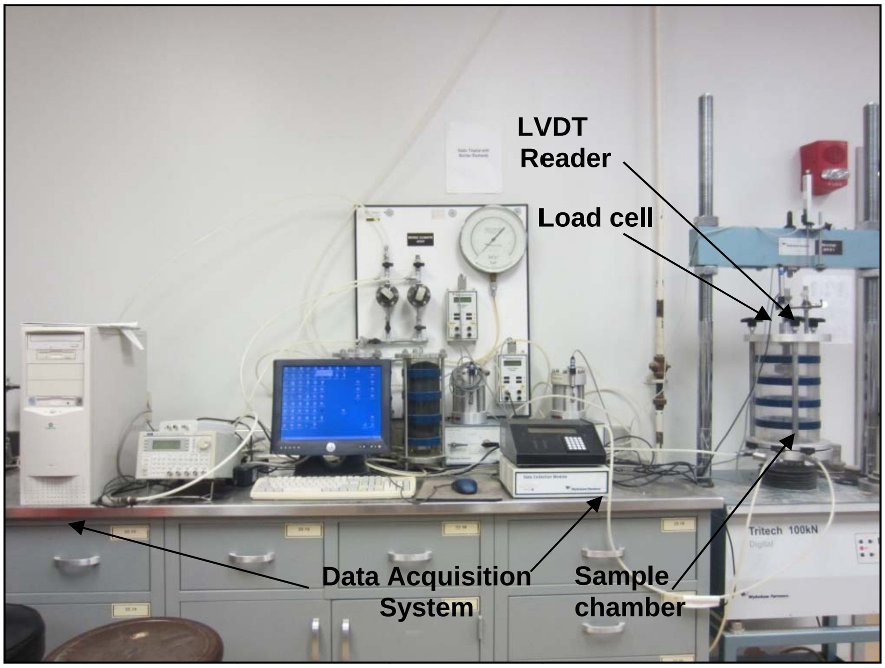


Figure 3.15 UCS Test Setup

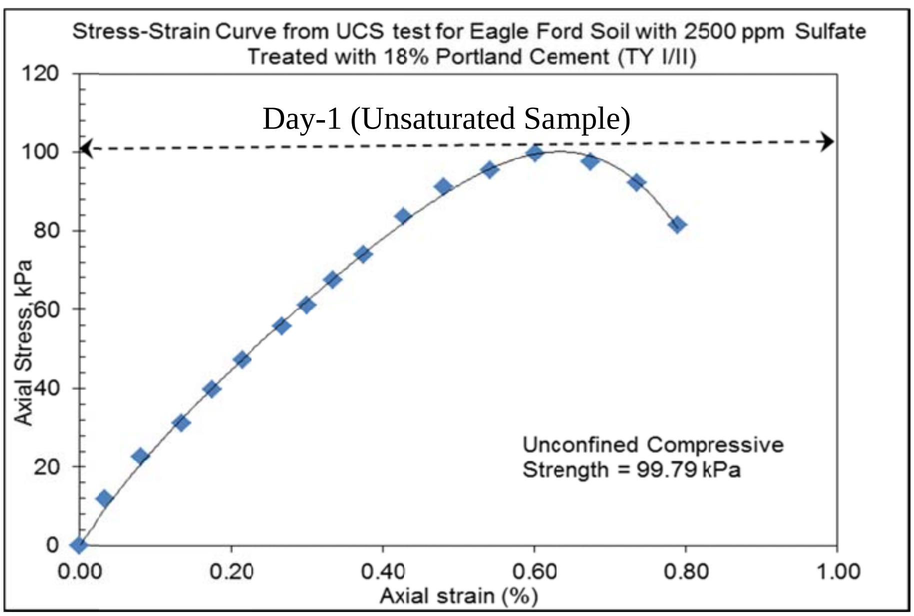


Figure 3.16 Example of a UCS Test Graph

3.8 Summary

Chapter 3 summarized the various laboratory tests conducted to achieve the proposed research objectives. The sample preparation techniques for short term strength studies and long term durability studies are discussed above. Furthermore, some of the test procedures covered in this chapter includes sulfate testing, CLSM specimen preparation method, durability testing (w-d cycles), leachate testing and strength testing. The test apparatus described in this chapter included the ones used for CLSM sample preparation, flow test and UCS test. Test results obtained by following the above mentioned procedures along with a thorough analysis and discussion on these results is presented in the next chapter.

CHAPTER 4

LABORATORY TEST RESULTS AND ANALYSIS

4.1 Introduction

This chapter contains laboratory test results and analysis for the CLSM research study conducted on sulfate spiked expansive soil from Eagle Ford formation. The chapter has been divided into two sections: results and analysis and starts with the sample notation section. Thereafter, in the order the tests were conducted, the results section includes the results from the sulfate test and flow test followed by the UCS tests for short term strength analysis. The short term strength test results are followed by the long term durability test results obtained from the UCS test, volume change, weight change and leachate studies. A summary of results is presented at the end of each section. Subsequently, a thorough analysis based on the results obtained from all the laboratory tests ends the chapter.

4.2 CLSM Sample Notation and Other Naming Conventions Used

Following the methodology and testing parameters mentioned in Chapter 3, CLSM samples were prepared for short term strength and long term durability tests. Table 3.3 in Chapter 3 provides the testing parameters for CLSM samples. Each CLSM sample contains oven dried Eagle Ford soil passing through U.S. sieve # 40, 18% Portland Cement (Type I/II), varying soluble sulfates added in the form of gypsum and distilled water. The amount of water added to the CLSM mix was determined from the flow test results presented in the upcoming section of this chapter.

For easy identification of different CLSM mixes, each mix was assigned a particular notation as shown in Table 4.1. The notation not only provided a shorter name to the sample but also delivered information on the variable constituent present in the sample. As mentioned in Chapter 3, sulfate concentration is the only variable constituent present in this study. The other constituents remain fixed throughout the tests and have been excluded from sample symbolization.

Sample notations adopted are based on the CLSMs sulfate concentration. For instance, EF-2500S refers to Eagle Ford CLSM with 2,500 ppm sulfate concentrations. ‘EF’ stands for the Eagle Ford soil and ‘2500S’ stands for the sulfate concentration present in the sample. Table 4.1 below provides the notations used for various samples.

Additionally, there are certain terminologies used throughout this chapter for convenience. All the ‘cement’ referred in this chapter represent Portland Cement (Type I/II) unless otherwise mentioned. When ‘four sulfate concentrations’ is mentioned, it refers to 2,500; 5,000; 10,000 and 20,000 ppm sulfate concentrations throughout this chapter. Furthermore, one durability cycle constitutes 24 hours of drying and 5 hours of wetting which is also referred to as ‘complete durability cycle’. Durability cycles refer to alternate wetting and drying process. Some samples are denoted as ‘0-CYCLE’ referring to samples that undergo only 5 hours of wetting after being cured for 28 days and then are subjected to UCS test. Similarly, ‘3-Cycle’ sample refers to samples that undergo 3 complete durability cycles before the UCS test. ‘w-d’ refers to cyclic wetting and drying process.

Table 4.1 CLSM Sample Notations

Designation	Description
EF	Test Soil from Eagle Ford formation
EF-NAT	Eagle Ford Soil with natural Soluble Sulfate Concentration (100 ppm or less)
EF-2500S	Eagle Ford Soil with 2500 ppm Soluble Sulfate Concentration
EF-5000S	Eagle Ford Soil with 5000 ppm Soluble Sulfate Concentration
EF-10000S	Eagle Ford Soil with 10000 ppm Soluble Sulfate Concentration
EF-20000S	Eagle Ford Soil with 20000 ppm Soluble Sulfate Concentration

4.3 Sulfate Test Results using Modified UTA Method

The methodology described in Chapter 3 for sulfate testing by Modified UTA Method was followed before each batch of oven dried pulverized soil was used for CLSM sample preparation. During the preparation and testing of both the short term strength as well as long durability samples, a total of 4 batches of sulfate testing were conducted. In each batch, about 10 kilograms of pulverized oven dried Eagle Ford soil provided by TRWD was considered. From the soil bulk, 3 test samples of soil were taken in a flask namely EF-1, EF-2 and EF-3. After conducting the sulfate test, the weights obtained from the tests were entered in the spreadsheet to obtain the concentration of sulfate present in each soil sample in ppm. The results from the three tests were averaged to get the sulfate concentration of that particular soil bulk. The results from the 4 different batches of sulfate tests conducted are presented in Table 4.2. It also shows the CLSM test samples prepared from each soil batch. Additionally, sulfate was added in the form of gypsum to the EF-2500S, EF-5000S, EF-10000S and EF-20000S samples to obtain the desired sulfate concentrations in them before CLSM preparation.

Table 4.2 Eagle Ford Sulfate Test Results using UTA Modified Method

Batch	Sulfate Concentration (ppm)	CLSM Samples Prepared
1	91	- Control Soil (4) - 2,500 ppm sulfate (4) - 5,000 ppm sulfate (4)
2	86	- 10,000 ppm sulfate (4) - 20,000 ppm sulfate (4)
3	156	- Control Soil (0,3, 7 & 14 Cycles) - 2,500 ppm sulfate (0,3, 7 & 14 Cycles) - 5,000 ppm sulfate (0,3, 7 & 14 Cycles)
4	219	- 10,000 ppm sulfate (0,3, 7 & 14 Cycles) - 20,000 ppm sulfate (0,3, 7 & 14 Cycles)

4.4 Flow Test Results

Flow tests were conducted in accordance to ASTM D 6103-97 testing procedure as mentioned in chapter 3. The test was conducted to determine the amount of water desired for the CLSM mix to meet the above mentioned standard. The standard requires the patty diameter to fall between 8 in. to 12 in. One flow test was conducted before EF-NAT, EF-2500S, EF-5000S, EF-10000S and EF-20000S samples were cast. After several trials, the water content to achieve a patty diameter between 8 in. to 12 in. with Eagle Ford native soil mixed with 18% cement was found between 93% to 97% of the total weight of soil and cement added. In other words, for 1000 grams of EF-NAT, with 180 grams cement (18% by weight); 970 mL to 930 mL of water was required to meet the ASTM standard. The average value of 95% was set as the water content for all the sample preparation. In all the flow tests, 95% water was adequate to achieve the desired diameter. Table 4.3 shows the results of the flow test with the patty diameter observed for different CLSM samples.

Table 4.3 Flow test Results for Eagle Ford CLSM

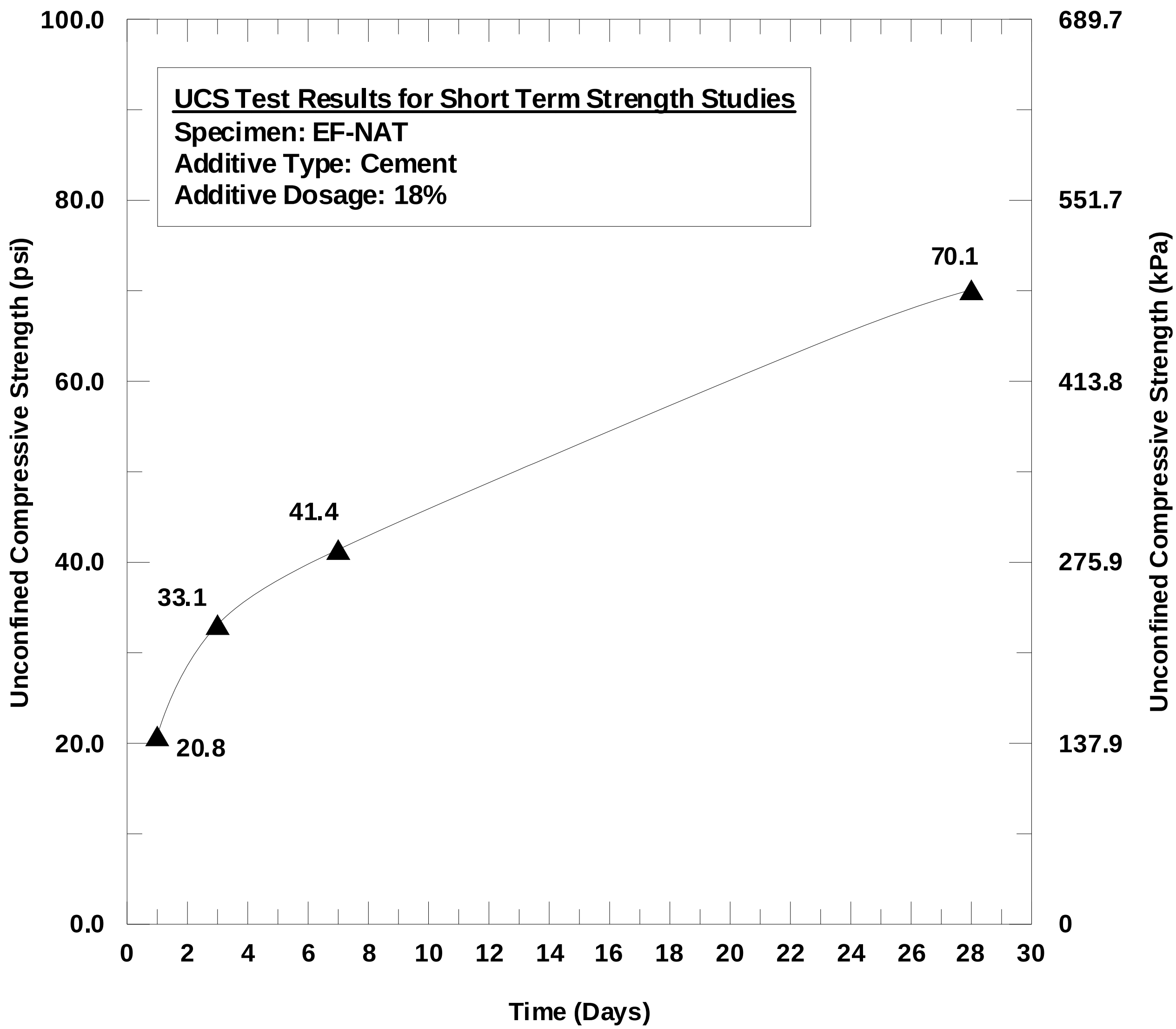
Sample	Water Content (%)	Trial 1 (in.)	Trial 2 (in.)	Average Diameter (in.)
EF-NAT	95	9.00	10.00	9.50
EF-2500S	95	9.50	9.00	9.25
EF-5000S	95	10.00	9.00	9.50
EF-10000S	95	9.00	10.50	9.75
EF-20000S	95	11.00	9.00	10.00

4.5 Short Term Strength Tests

As mentioned in chapter 3, the short term strength test was conducted to evaluate the variation of strength over a period of 28 days. For the short term strength tests, UCS test was the only test conducted. Details of sample preparation and testing procedures are presented in Chapter 3. One of the primary benefits of a short term strength test is that it reflects upon any significant increase or decrease in the strength of the CLSM sample at different curing period. These samples are not subjected to any durability testing. The results from the UCS test on short term strength samples have been presented in the upcoming sub-sections.

4.5.1 Sample EF-NAT

The 1, 3, 7, 28 days UCS tests were conducted on the CLSM samples prepared with the control soil, EF-NAT. The 28-day strength for the sample was noted as 483.4 kPa (70.1 psi) followed by 285.3 kPa (41.4 psi) for 7-day, 228.0 kPa (33.1 psi) for 3-day and 143.0 kPa (20.8 psi) for 1-day. As expected, the strength increased with time; the longer the curing period the higher the strength. Figure 4.1 shows the UCS plot with time of the control soil based CLSM.

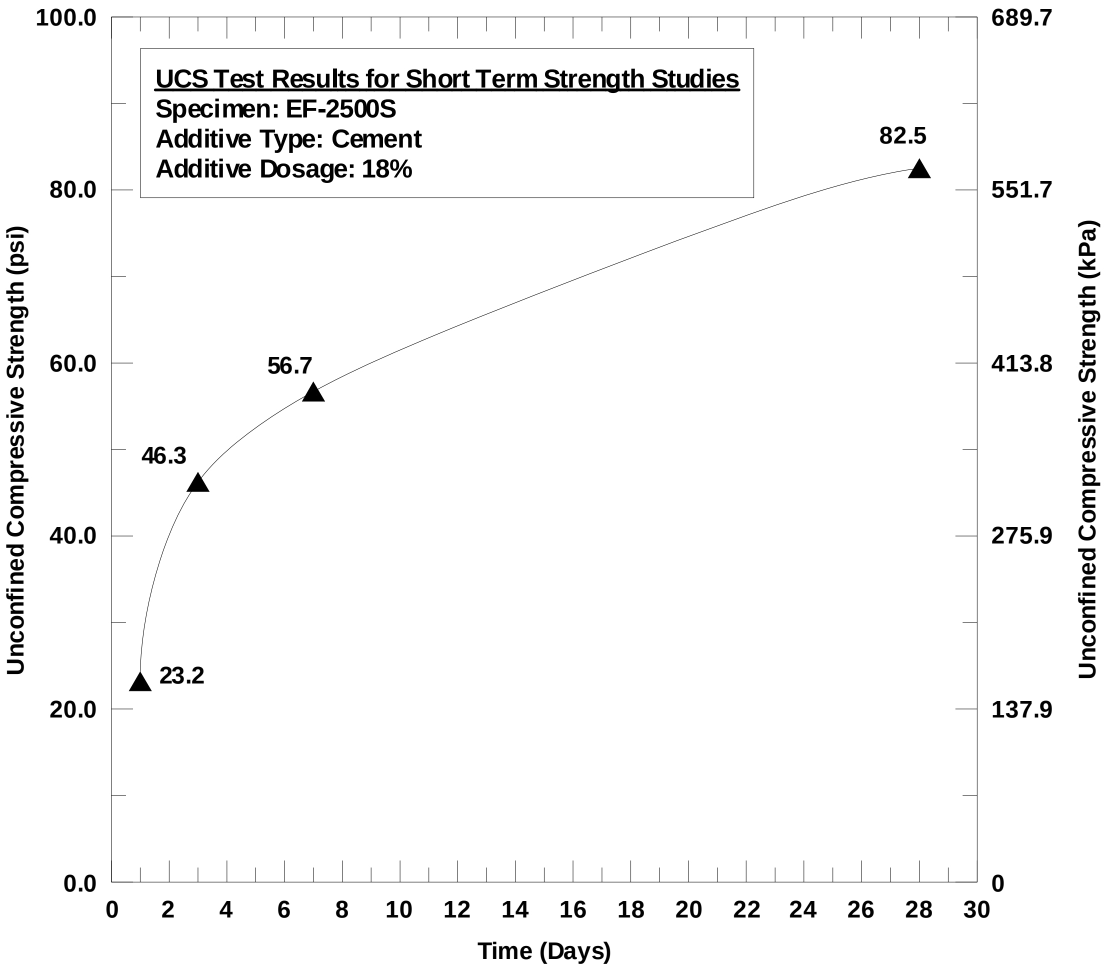


* Values next to the bullets represent Unconfined Compressive Strength (UCS) of the specimen in lb/in^2 (psi).

Figure 4.1 Variation of UCS Value with Time for Control Soil (EF-NAT)

4.5.2 Sample EF-2500S

The 1, 3, 7, 28 days UCS tests were conducted on EF-2500S samples. The 28-day UCS strength for the sample was noted as 569.0 kPa (82.5 psi) followed by 391.0 kPa (56.7 psi) for 7-day, 319.0 kPa (46.3 psi) for 3-day and 160.0 kPa (23.2 psi) for 1-day. As expected, the strength increased with time; the longer the curing period the higher the strength. Compared to EF-NAT, UCS value for the sample at 28-day strength was higher. Figure 4.2 shows the UCS plot with time for EF-2500S sample.

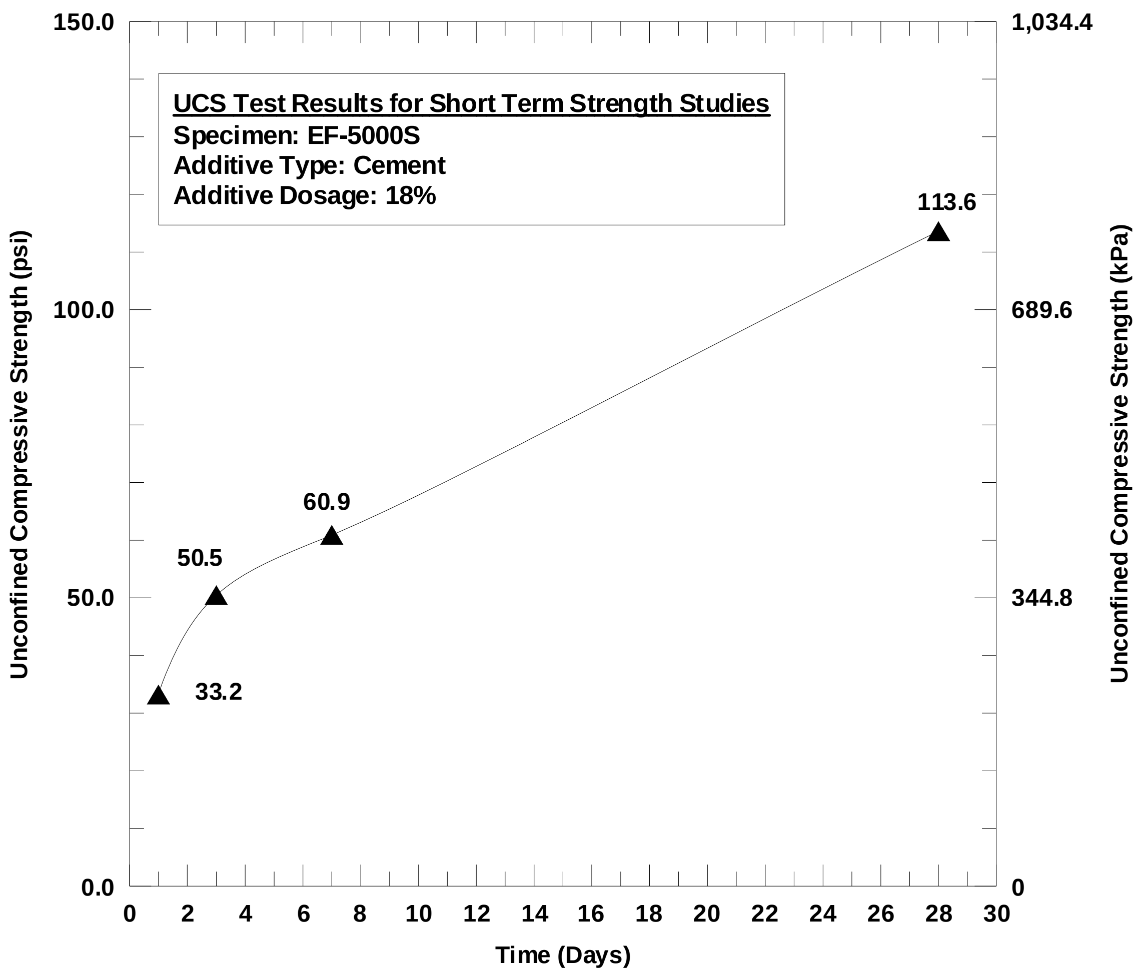


* Values next to the bullets represent Unconfined Compressive Strength (UCS) of the specimen in lb/in^2 (psi).

Figure 4.2 Variation of UCS Value with Time for EF-2500S Sample

4.5.3 Sample EF-5000S

The 1, 3, 7, 28 days UCS tests were conducted on EF-5000S samples. The 28-day UCS strength for the sample was noted as 783.5 kPa (113.6 psi) followed by 420.1 kPa (60.9 psi) for 7-day, 348.1 kPa (50.5 psi) for 3-day and 229.1 kPa (33.2 psi) for 1-day. As expected, the strength increased with time; longer the curing period, higher the UCS strength. Compared to EF-NAT and 2500S samples, UCS value for the sample at 28-day strength was higher. Figure 4.3 shows the variation of UCS value with time for EF-5000S sample.

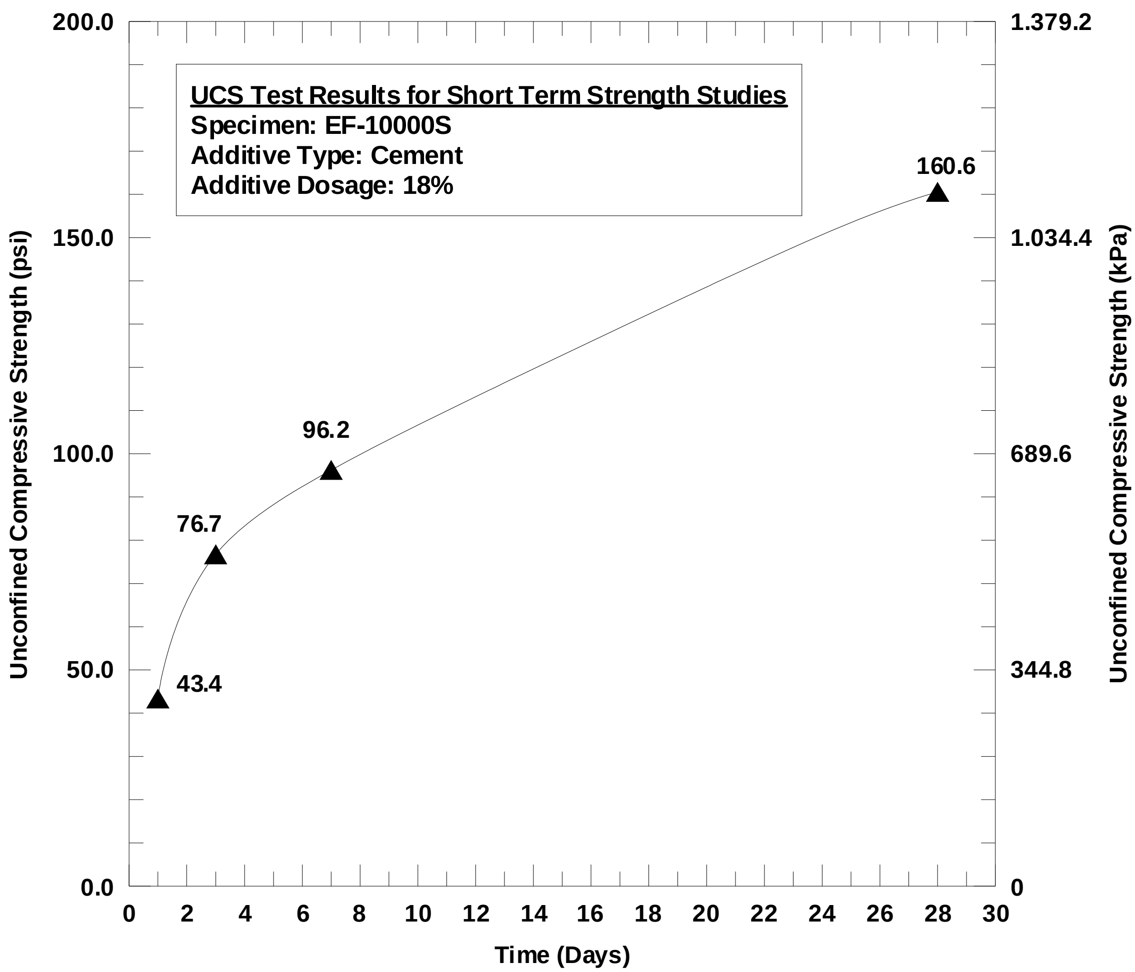


* Values next to the bullets represent Unconfined Compressive Strength (UCS) of the specimen in lb/in^2 (psi).

Figure 4.3 Variation of UCS Value with Time for EF-5000S Sample

4.5.4 Sample EF-10000S

The 1, 3, 7, 28 days UCS tests were conducted on EF-10000S samples. The 28-day strength for the sample was noted as 1107.2 kPa (160.6 psi) followed by 663.7 kPa (96.2 psi) for 7-day, 529.1 kPa (76.7 psi) for 3-day and 299.1 kPa (43.4 psi) for 1-day. As expected, the strength increased with time; longer the curing period, higher the UCS strength. Compared to EF-NAT, 2500S and 5000S samples, UCS value for the sample at 28-day strength was the highest. Figure 4.4 shows the variation of UCS value with time for EF-10000S sample.

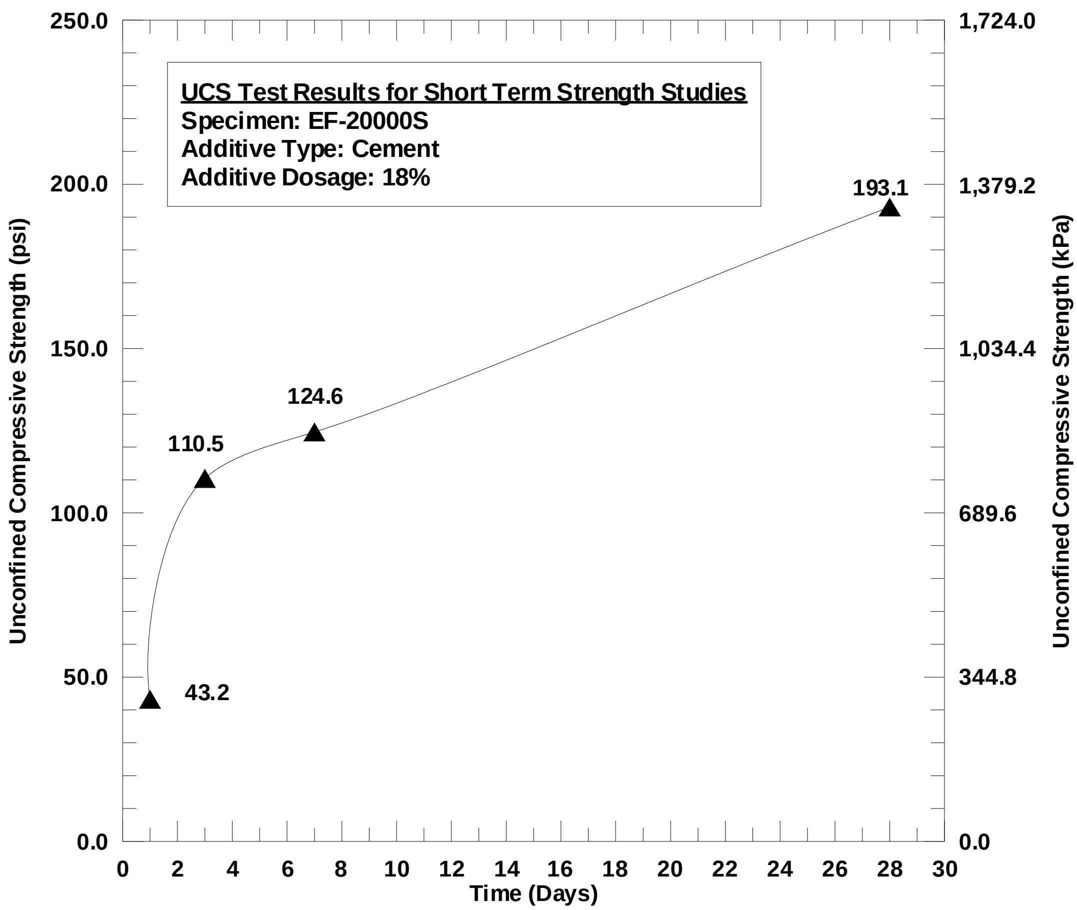


* Values next to the bullets represent Unconfined Compressive Strength (UCS) of the specimen in lb/in^2 (psi).

Figure 4.4 Variation of UCS Value with Time for EF-10000S Sample

4.5.5 Sample EF-20000S

The 1, 3, 7, 28 days UCS tests were conducted on EF-20000S samples. The 28-day strength for the sample was noted as 1331.4 kPa (193.1 psi) followed by 859.3 kPa (124.6 psi) for 7-day, 761.7 kPa (110.5 psi) for 3-day and 298.0 kPa (43.2 psi) for 1-day. As expected, the strength of the sample increased with time; longer the curing period, higher the UCS strength. Compared to all the samples; EF-NAT, 2500S, 5000S and 10000S, UCS value for the sample at 28-day strength was the highest. Figure 4.5 shows the variation of UCS value with time for EF-20000S sample.



* Values next to the bullets represent Unconfined Compressive Strength (UCS) of the specimen in lb/in^2 (psi).

Figure 4.5 Variation of UCS Value with Time for sample EF-20000S

4.5.6 Summary

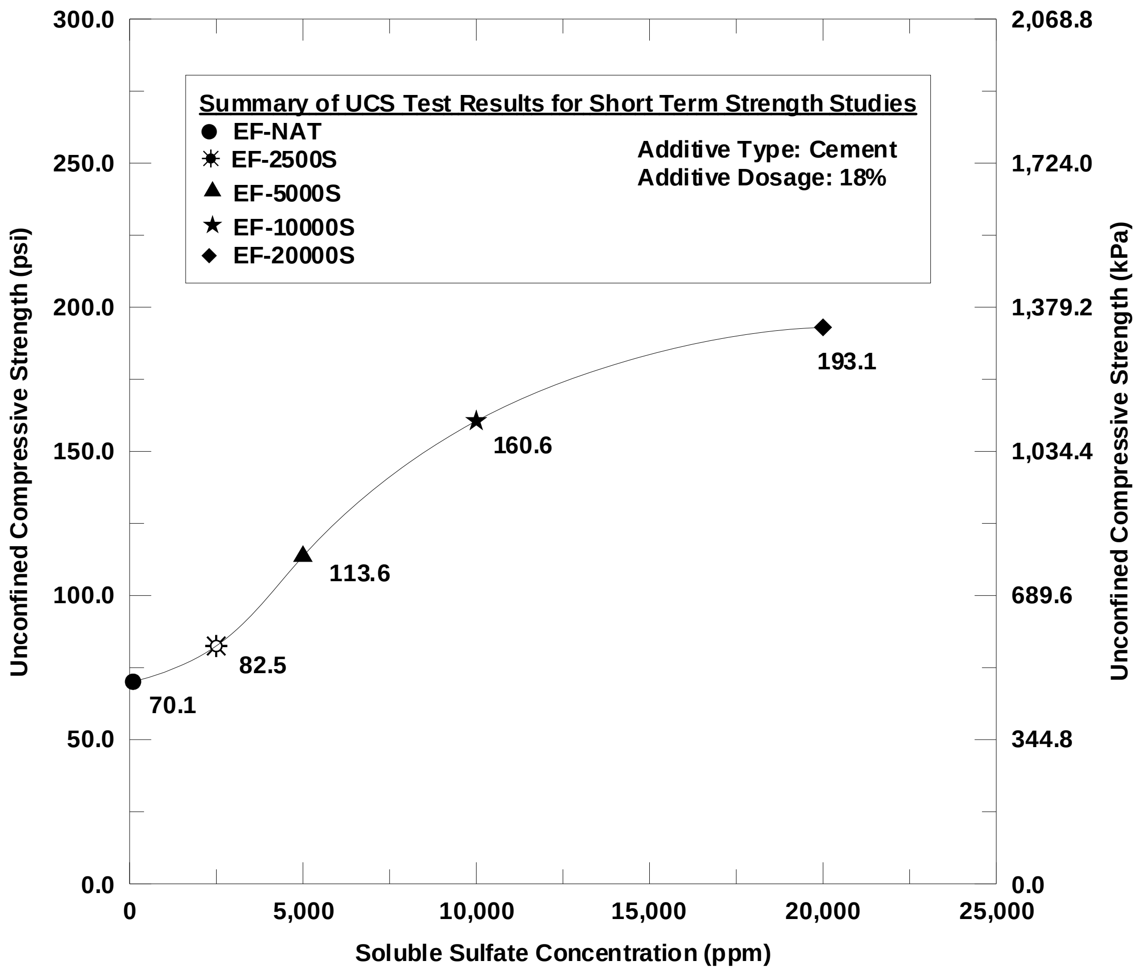
Table 4.4 below summarizes the results of the short term strength tests. From the table it is observed that with the increase in setting time, the UCS strength also increased. Another remarkable observation is that with the increase in sulfate concentration, the UCS value also increased implying that EF-20000S sample exhibited the highest strength value compared to the rest of the samples. The highest UCS value observed was 193.1 psi and the lowest was 20.8 psi. There was no UCS strength losses noticed during the short term strength tests. As expected, a gradual increase in strength was observed from 0-Day sample to 28-Day test samples.

One reason for no influence of sulfate effects is that the CLSM mixture is highly cementitious and as a result, cementing reactions at full hydration conditions are much larger than the disruptive Ettringite reactions which may have resulted in strength enhancements.

Additionally, Figure 4.6 shows the variation of the 28-Day UCS value with soluble sulfate concentration. From the graph, it is clear that the UCS value is the highest for the EF-20000S sample and the lowest for the control sample.

Table 4.4 Summary of Short Term Test on all the Eagle Ford CLSM Samples

Summary of Short Term Strength Tests					
Sample Name	**Sulfate Concentration (ppm)**	**UCS Strength (psi)**			
		1-Day	**3-Day**	**7-Day**	**28-Day**
EF-NAT	100 or less	20.8	33.1	41.4	70.1
EF-2500S	2,500	23.2	46.3	56.7	82.5
EF-5000S	5,000	33.2	50.5	60.9	113.6
EF-10000S	10,000	43.4	76.7	96.2	160.6
EF-20000S	20,000	43.2	110.5	124.6	193.1



* Values next to the bullets represent Unconfined Compressive Strength (UCS) of the specimen in lb/in^2 (psi).

Figure 4.6 Variation of 28-Day UCS Test Result with Sulfate Concentration

4.6 Long Term Durability Test

This section presents the results obtained from the durability studies conducted on all the four different samples with sulfate concentrations of 2500, 5000, 10000 and 20000 ppm including the control soil in which each sample was subjected to 0, 3, 7 and 14 cycles of w-d process.

As mentioned in Chapter 3, one complete cycle constitutes of 24 hours of drying at 140°F followed by 5 hours of wetting with an exception to 0-cycle samples. 0-cycle samples undergo 5 hours of wetting after the curing period and are subjected to UCS test. The long term durability studies included UCS testing, volume and weight change measurements and leachate studies. The tests were conducted on the CLSM samples for all the sulfate concentrations during or after completing the durability cycles. Samples were subjected to 0, 3, 7 and 14 wetting and drying durability cycles. Hence, for each sulfate concentration, 4 samples were prepared for testing at each cyclic condition.

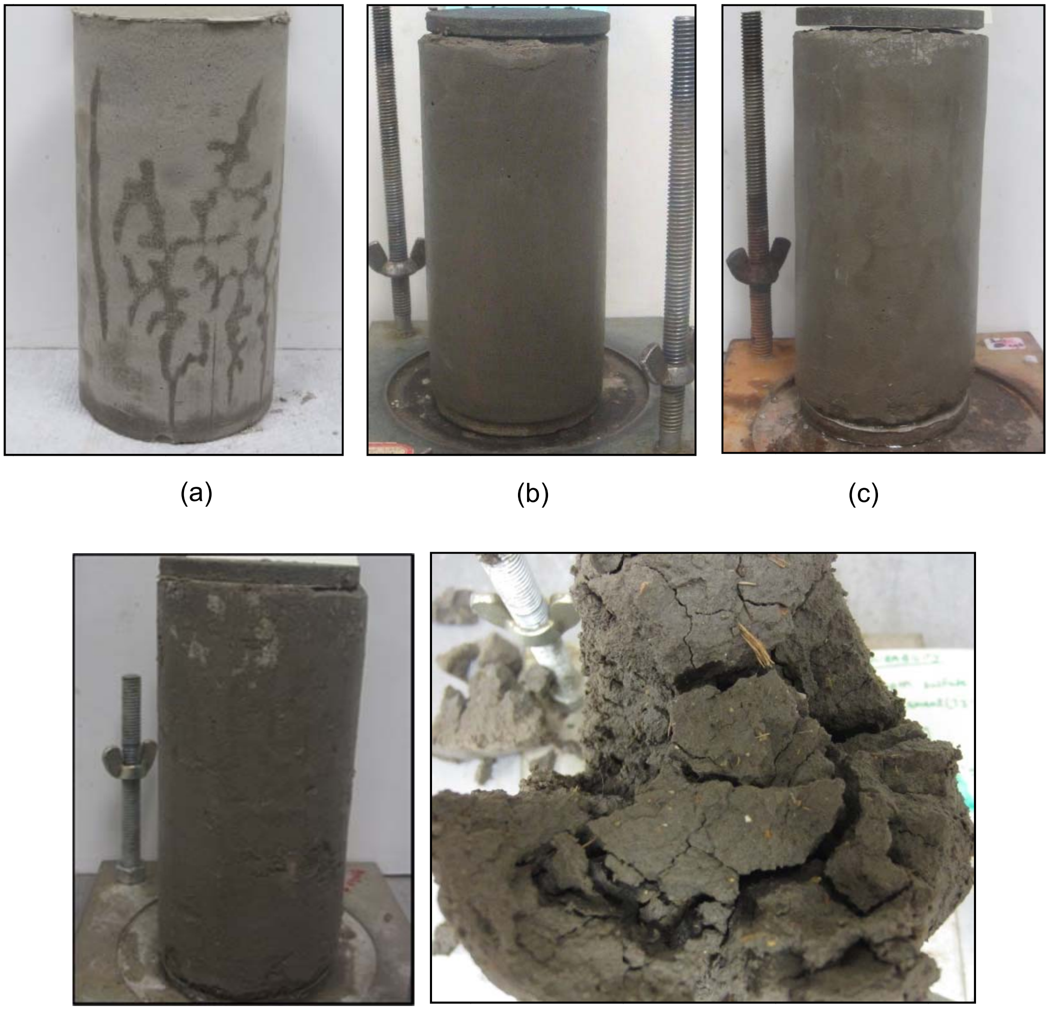
Volumetric strain changes were based on both the changes in diameter and height of the original compacted soil specimens. The maximum volumetric strain is a combination of the percent change for wetting and drying of one cycle of durability. The percent volumetric change for each of wetting and drying cycle is determined with respect to the initial compacted volume of the CLSM sample before durability studies. Then, the percent change in volume for the wetting and drying cycle for that particular cycle is added to determine the maximum volumetric change exhibited by the sample for that complete durability cycle due to moisture hydration. For volumetric strain plots, only the 14-Cycle sample for each sulfate concentration was considered.

Besides recording the height and diameter of the CLSM test samples, weight was also noted after each wetting and drying periods for each cycle. Weight change for each drying and wetting cycle was calculated with respect to the initial compact weight of the CLSM sample before durability testing. Then, the percent change in drying and wetting for that particular cycle were added to obtain the maximum weight change exhibited by the sample during one complete wetting-drying durability cycle due to moisture hydration.

The main purpose of leachate collection is to perform calcium test using EDTA method which shows the amount of calcium present in the leachate sample tested. Ultimately the amount of cement leached out can also be determined. The procedure used for determining the calcium concentration of a prepared soil specimen by EDTA is provided in Chapter 3. The test results of the samples with all the sulfate concentrations (control soil, 2500, 5000, 10000 and 20000 ppm) that were subjected to durability studies are presented in the following sections. A detailed testing procedure has been explained in Chapter 3.

4.6.1 Sample EF-NAT

The control soil CLSM sample lasted all the 14 cycles of durability (w-d) process**.** Figure 4.7 shows the pictures after 28 days curing, 0, 3, 7 and 13 wetting cycles for the 14-Cycle EF-NAT sample. In this section, the results from the UCS test, volume and weight change characteristics along with leachate studies are presented. For all the volume change, weight change and leachate studies, only the 14-Cycle sample was considered. However for the UCS test separate identical samples were prepared in representation of each cycle. In this section, the results from the UCS test, volume and weight change characteristics along with leachate studies are presented.



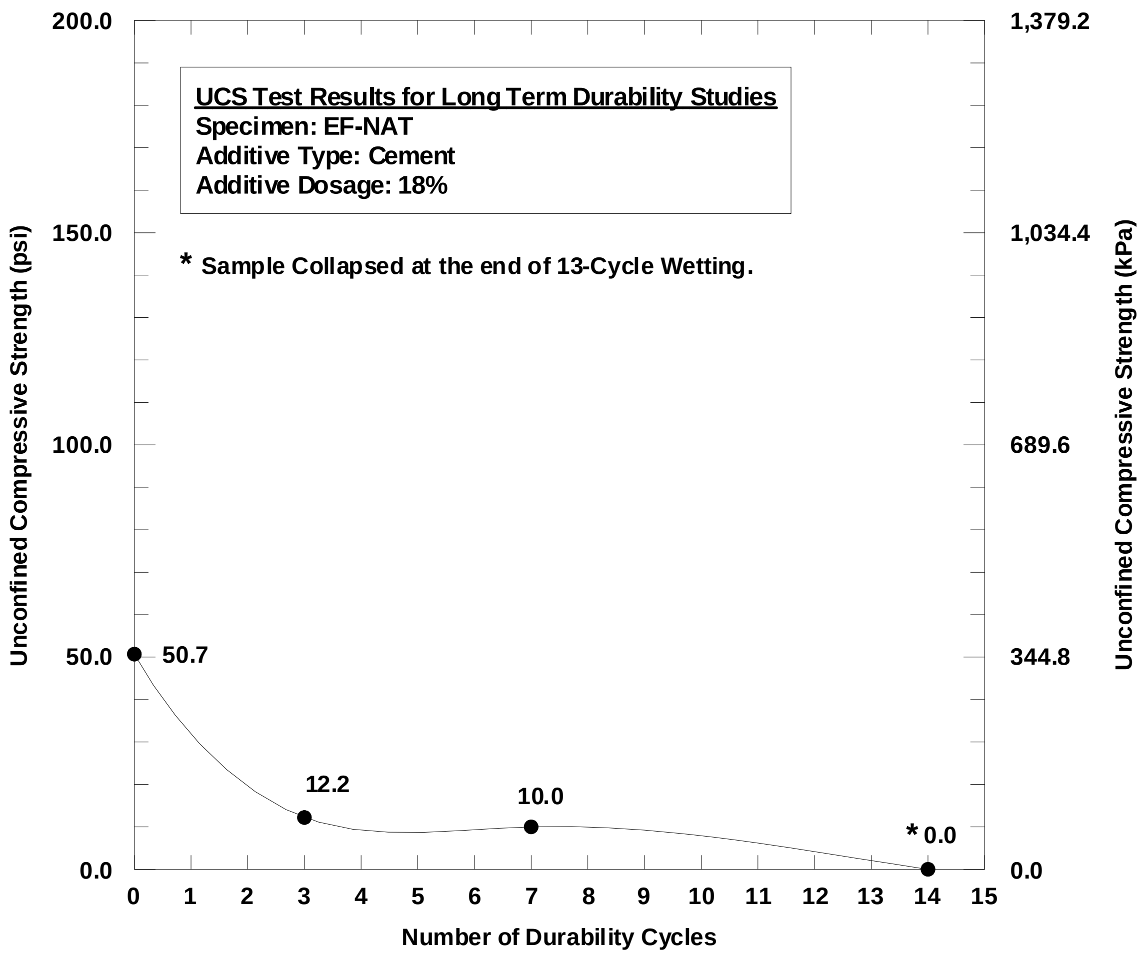
(a) (b) (c)

(d) (e)

Figure 4.7 Pictures of EF-NAT 14-Cycle Sample showing Sample after (a) 28 days Curing, (b) 0-Cycle Wetting, (c) 3-Cycle Wetting, (d) 7-Cycle Wetting and (e) 13-Cycle Wetting

4.6.1.1 UCS Test

The UCS test was conducted on 0, 3, and 7 Cycle EF-NAT samples in accordance to ASTM 2166 procedure. The sample lasted 13 complete cycles and collapsed at the end of 13-Cycle wetting. Hence, no UCS test was conducted on the sample. Figure 4.8 shows the 0, 3, 7, 14 cycles UCS test result for control soil CLSM. The peak UCS value was 349.4 kPa (50.7 psi) for 0-Cycle sample. The strength decreased as the cycles increased. 3-Cycle and 7-Cycle samples exhibited a UCS value of 84.1 kPa (12.2 psi) and 73.2 kPa (10.6 psi) respectively.



Values next to the bullets represent Unconfined Compressive Strength (UCS) of the specimen in lb/in^2 (psi).

Figure 4.8 Variation of Unconfined Compressive Strength with Durability Cycles for Control Soil (EF-NAT)

4.6.1.2 Volume Change Characteristics

Height and diameter of all the samples were measured after completion of each cycle. This information was used to determine the volumetric strain. The maximum volumetric change of 10.1% (represented with red ink in the figure) was observed on Cycle-6 of the total 14 durability cycles of wetting and drying as shown in Figure 4.9. The minimum value was 5.5% and an average volumetric strain of 9.0%. It can be seen that the volume change experienced by the control soil was low. It should be noted that the maximum value is based on the volumetric change per cycle during moisture hydration.

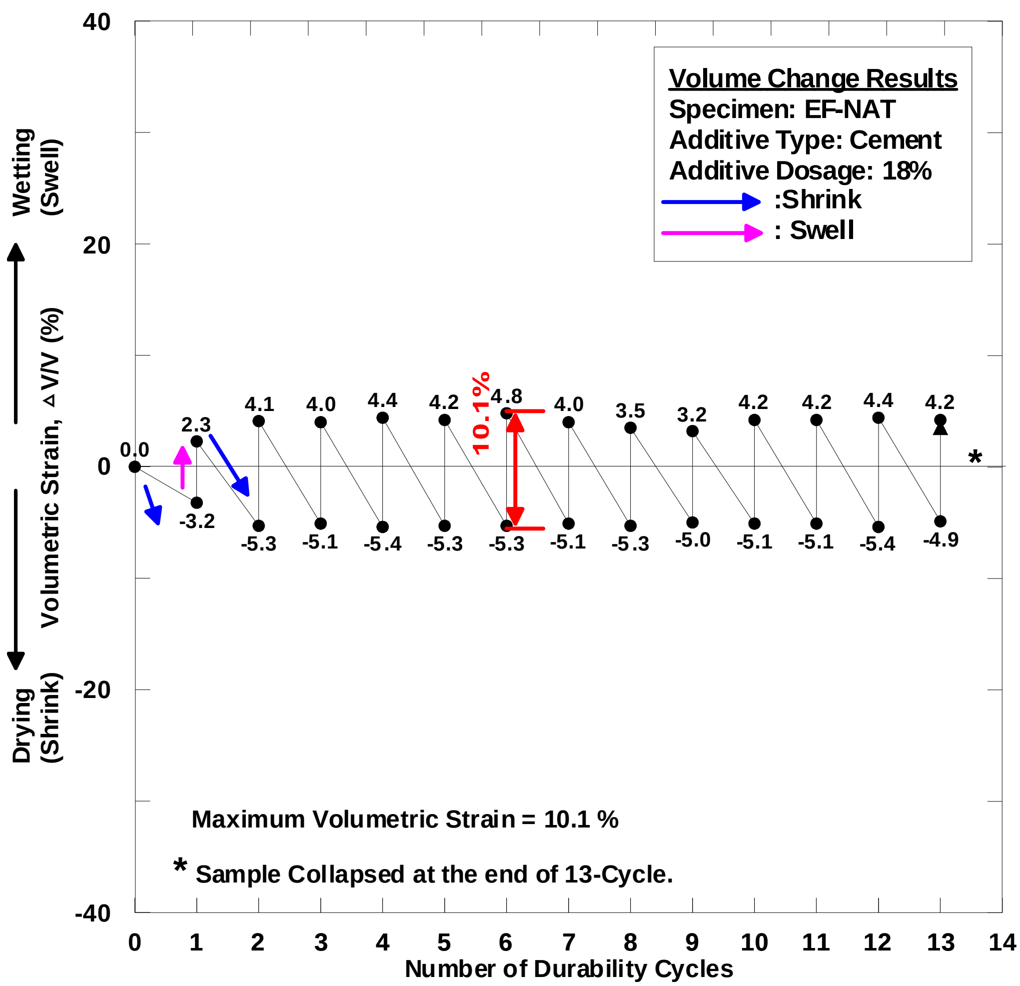


Figure 4.9 Variation of Volumetric Strain with Durability Cycles for Control Soil (EF-NAT)

4.6.1.3 Weight Change Characteristics

After each cycle the weight of the samples was recorded. Figure 4.10 shows the weight change in percent of the control soil for the 14-Cycle sample. The maximum weight change observed was 35.0% (represented with red ink in the figure) and the minimum was 30.4%. The average weight change for EF-NAT sample was reported to be 32.1%. It should be noted that the maximum value is based on the weight change per cycle during moisture hydration

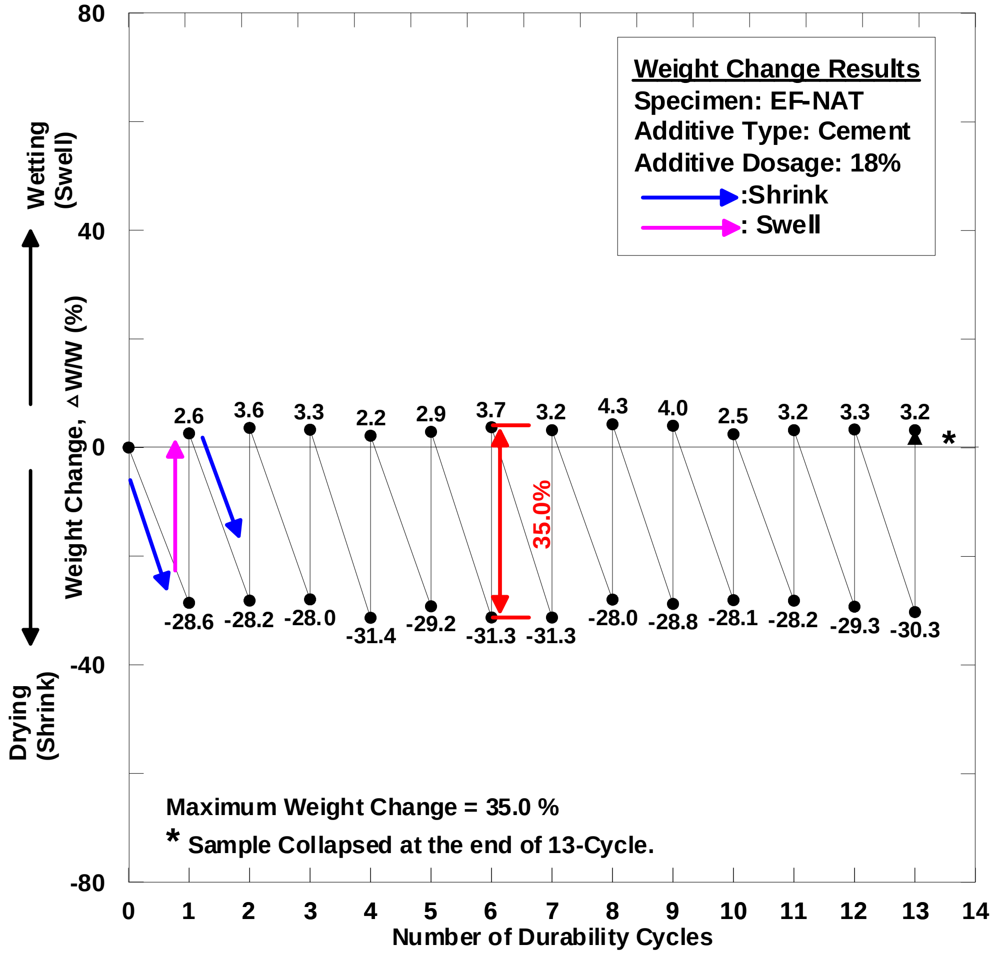


Figure 4.10 Variation of Weight Change with Durability Cycles for Control Soil (EF-NAT)

4.6.1.4 Leachate Studies

Leachate samples were collected at the end of wetting of 1, 3 and 7 cycles from the 14-Cycle sample. Table 4.5 shows the spreadsheet for the determination of calcium loss for the control soil. Figure 4.11 shows the variation of calcium loss with the durability cycles. The maximum and minimum calcium ion concentrations leached out were approximately 640 ppm and 380 ppm that were observed in 7-Cycle and 1-Cycle respectively. The average calcium loss for this sample was 547 ppm. Figure 4.10 presents the 7-Cycle average and totals.

Table 4.5 Calculation of Calcium Ion Concentration Loss for EF-NAT Sample

Cycle #	**Calcium Concentration (ppm)**
1	380
3	640
7	620
Average Calcium Loss for only 7 cycles (ppm) = 547	
Total Calcium loss after 7 cycles (ppm) = 3,827	

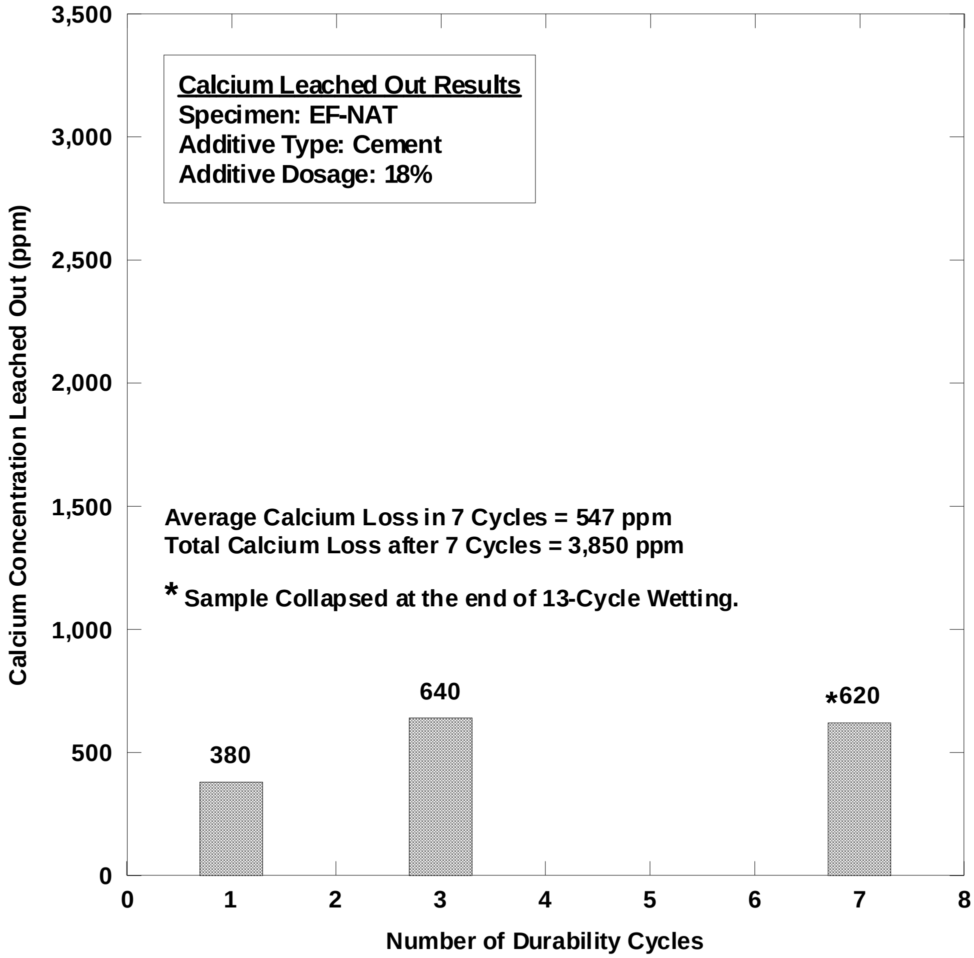


Figure 4.11 Variation of Calcium Concentration with Durability Cycles for Control Soil (EF-NAT)

4.6.2 Sample EF-2500S

In this section, the results from the UCS test, volume and weight change characteristics along with leachate studies are presented. For all the volume change, weight change and leachate studies only the 14-Cycle sample was considered. As mentioned previously, the EF-2500S sample lasted only upto the 13th cycle wetting period without collapsing. Hence, no UCS test was conducted on the 14-Cycle sample. Figure 4.12 shows the pictures after 0, 3, and 7 wetting cycles for the 14-Cycle sample.

(a)

(b)

(c)

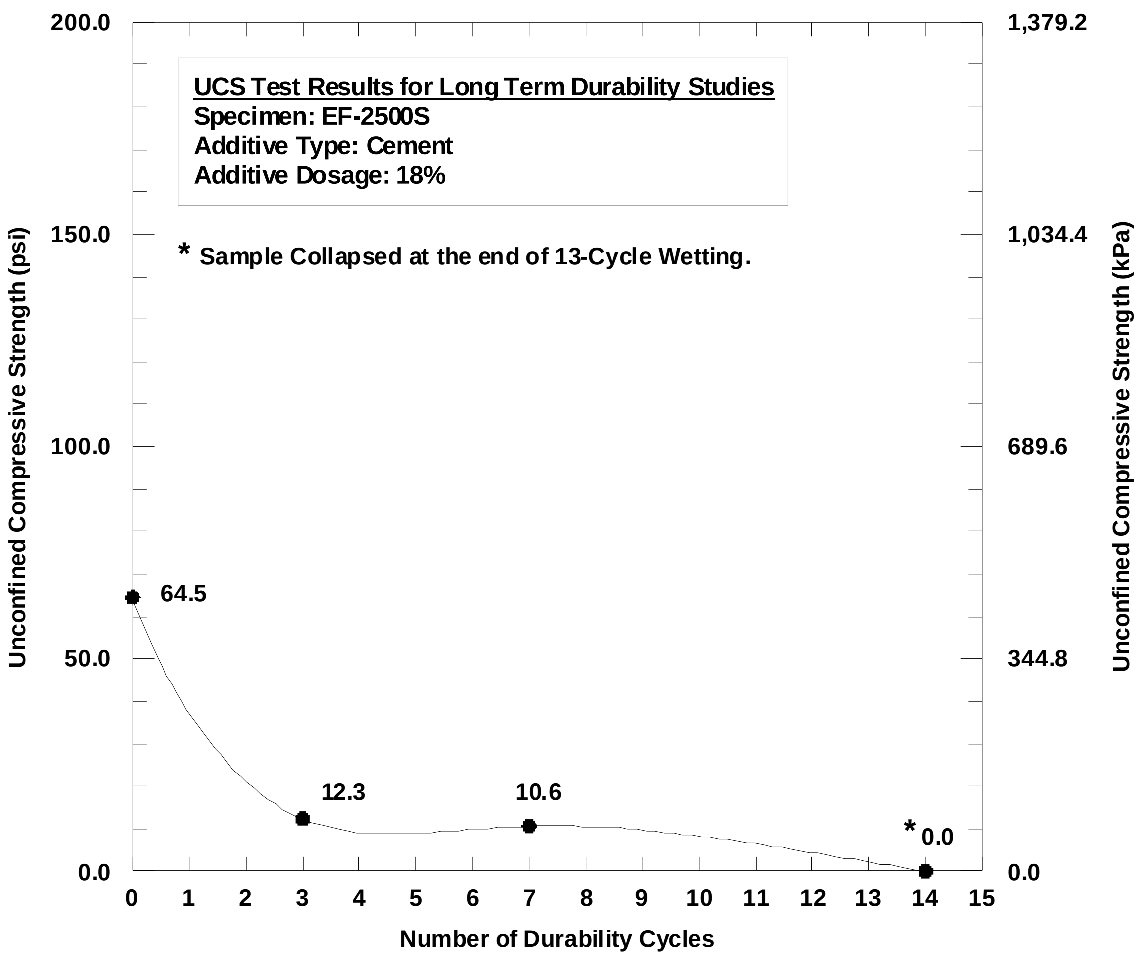
(d)

(e)

Figure 4.12 Pictures of EF-2500S 14-Cycle Sample showing Sample after (a) 28-Day Curing, (b) 0-Cycle Wetting, (c) 3-Cycle Wetting, (d) 7-Cycle Wetting and (e) 13-Cycle Wetting

4.6.2.1 Unconfined Compressive Strength Test Results

Figure 4.13 below shows the 0, 3, 7 cycles UCS test result for EF-2500S samples. The peak UCS value was 444.8 kPa (64.5 psi) for 0-Cycle sample. The strength decreased as the cycles increased. 3-Cycle and 7-Cycle samples exhibited a UCS value of 84.7 kPa (12.3 psi) and 73.2 kPa (10.6 psi) respectively.



Values next to the bullets represent Unconfined Compressive Strength (UCS) of the specimen in lb/in^2 (psi).

Figure 4.13 Variation of Unconfined Compressive Strength with Durability Cycles for EF-2500S Samples

4.6.2.2 Volume Change Characteristics

Figure 4.14 shows the volumetric strain in percent of 14-Cycle EF-2500S sample at various wetting and drying cycles. The maximum volumetric change of 20.1% (represented with red ink in the figure) was observed on 1-Cycle of the total 14 cycles of the durability (wetting and drying test). The minimum value was 5.4% and an average volumetric strain for the sample was reported to be 14.1%. It should be noted that the maximum value is based on the volumetric change per cycle during moisture hydration. The durability cycle starts at drying and ends at wetting to complete one cycle.

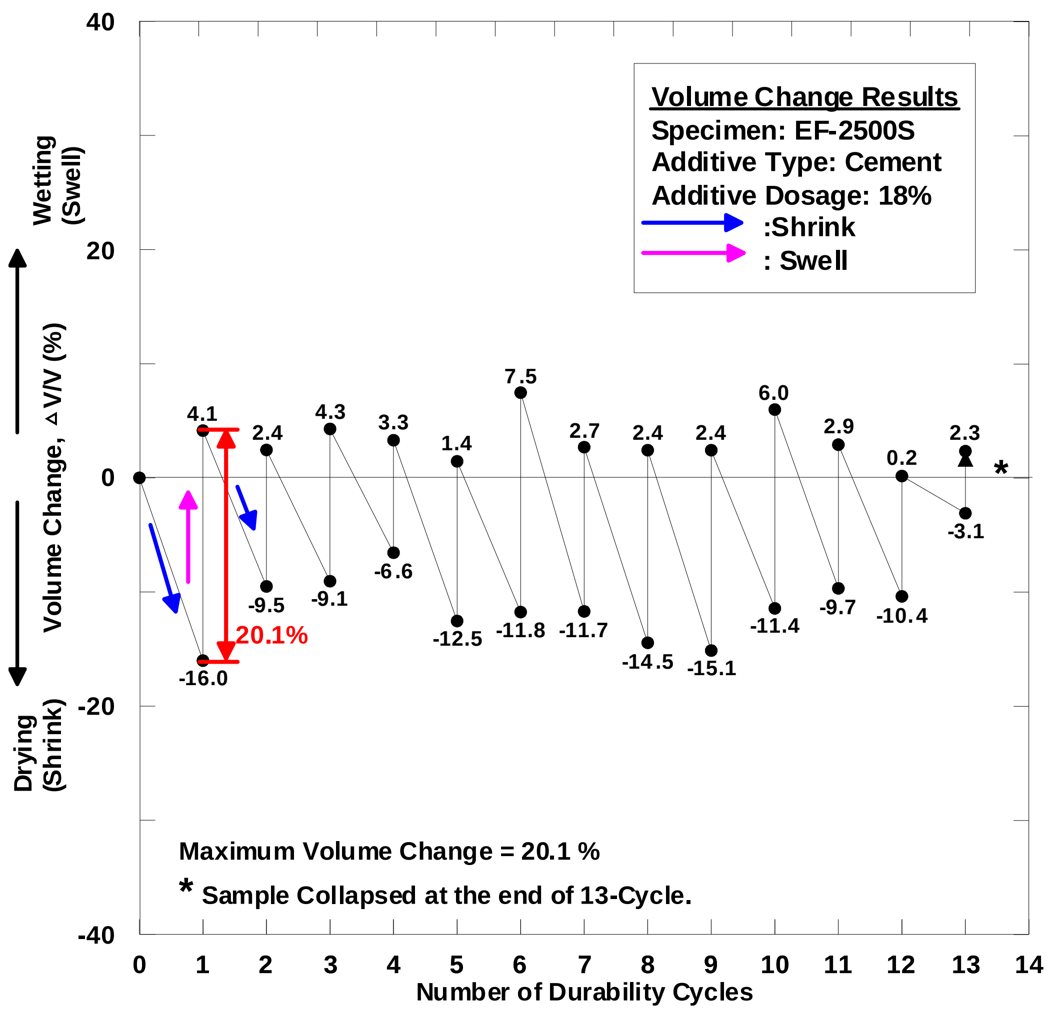


Figure 4.14 Variation of Volumetric Strain with Durability Cycles for EF-2500S Sample

4.6.2.3 Weight Change Characteristics

The EF-2500S sample lasted a complete 12 cycles and collapsed at the end of the 13-Cycle wetting period. Although the UCS test was not conducted on the sample, weight of the sample was recorded at the end of 13-Cycle wetting period. Figure 4.15 shows the weight in percent of 14-Cycle EF-2500S sample at various wetting and drying cycles. The maximum weight change observed was 41.7% (represented with red ink in the figure) and the minimum was 33.7%. The average weight change for EF-2500S sample was reported to be 37.0%. It should be noted that the maximum value is based on the weight change per cycle during moisture hydration.

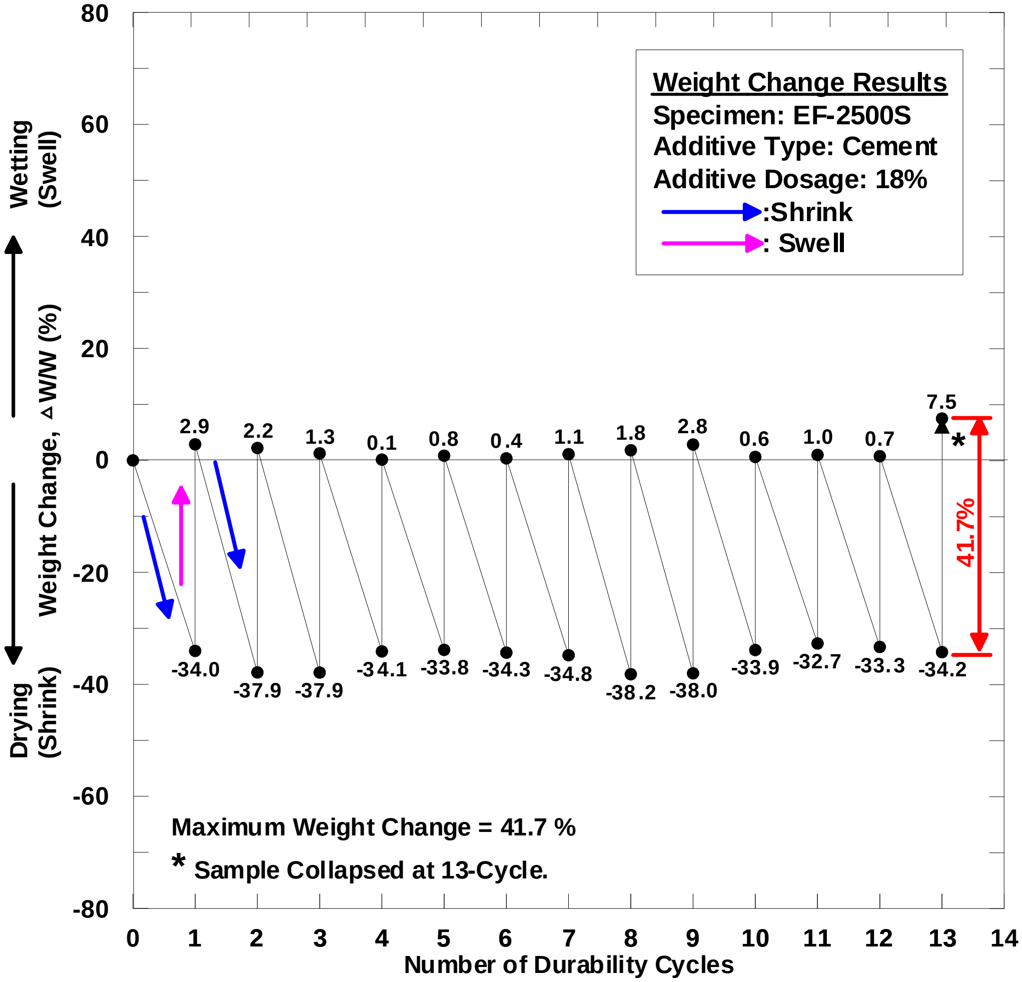


Figure 4.15 Variation of Weight Change with Durability Cycles for EF-2500S Sample

4.6.2.4 Leachate Studies

Since the EF-2500S-18C sample did not last a complete 14 cycles, leachate was collected from 1, 3 and 7 cycles of the 14-Cycle sample only. Table 4.6 shows the spreadsheet for the determination of calcium loss in ppm for EF-2500S sample. Furthermore, Figure 4.16 shows the variation of calcium loss with the durability cycles. The maximum and minimum calcium ion concentrations leached out were approximately 790 ppm and 420 ppm that were observed in the 7-Cycle and 3-Cycle respectively. The average calcium loss for this sample was 1,448 ppm over 14 cycles of durability testing. The average total calcium ion concentration loss was observed to be approximately 580 ppm and the average total calcium loss for this sample was 4,060 ppm.

Table 4.6 Calculation of Calcium Ion Concentration Loss for EF-2500S Sample

Cycle #	**Calcium Concentration (ppm)**
1	530
3	420
7	790
Average Calcium Loss in 7 cycles (ppm) = 580	
Total Calcium loss after 7 cycles (ppm) = 4,060	

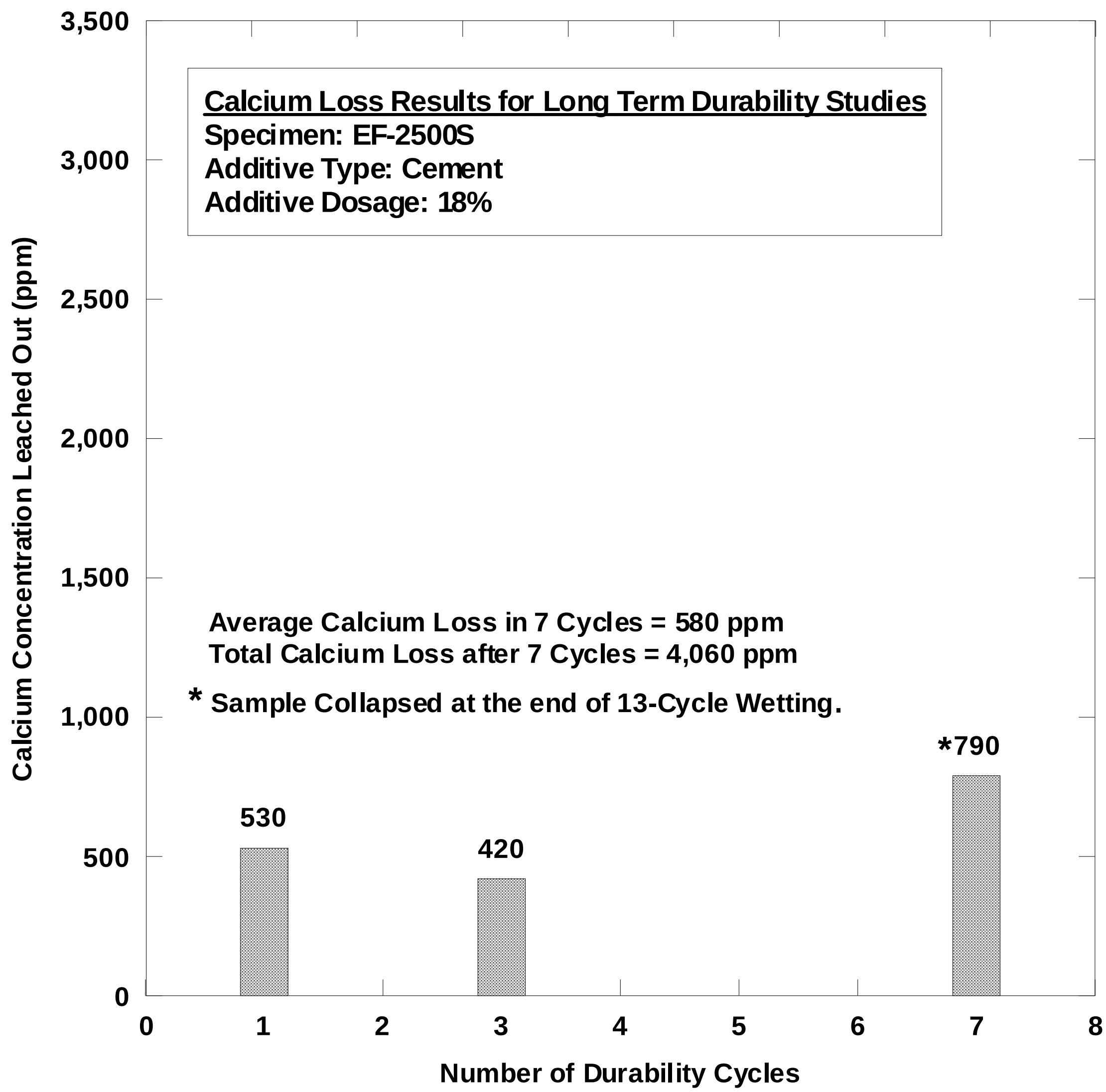


Figure 4.16 Variation of Calcium Concentration with Durability Cycles for EF-2500S Sample

4.6.3 Sample EF-5000S

The sample lasted a complete 12 cycles of wetting-drying process and collapsed at the end of 13-Cycle wetting. Hence, no UCS test was conducted on the 14-Cycle sample. Figure 4.17 shows the pictures after 28 days curing, 0, 3, 7, 12 and 13 wetting cycles for the 14-Cycle sample. In this section, the results from the UCS test, volume and weight change characteristics along with leachate studies are presented. For all the volume change, weight change and leachate studies only the 14-Cycle sample was considered.

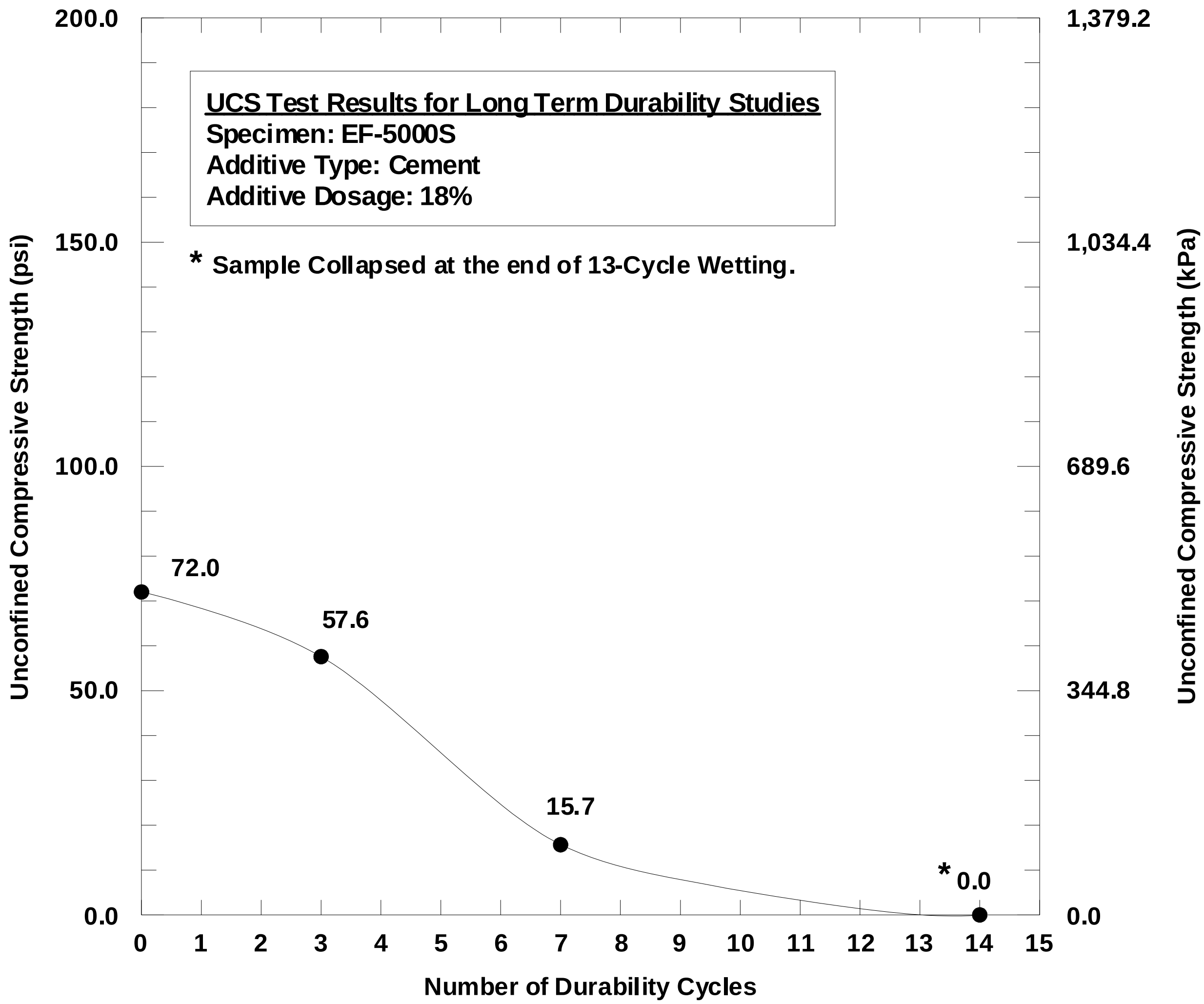
(a) (b) (c)

(d) (e) (f)

Figure 4.17 Pictures of EF-5000S 14-Cycle Sample showing Sample after (a) 28 days Curing, (b) 0-Cycle Wetting, (c) 3-Cycle Wetting, (d) 7-Cycle Wetting, (e) 12-Cycle Wetting and (f) 13-Cycle Wetting

4.6.3.1 Unconfined Compressive Strength Test Results

Figure 4.18 below shows the 0, 3, 7 cycles UCS test result for 5,000 ppm sulfate samples. The peak UCS value was 72.0 kPa (496.6 psi) for 0-Cycle sample. The strength decreased as the cycles increased. 3-Cycle and 7-Cycle samples exhibited a UCS value of 397.2 kPa (57.6 psi) and 107.9 kPa (15.7 psi) respectively.



Values next to the bullets represent Unconfined Compressive Strength (UCS) of the specimen in lb/in^2 (psi).

Figure 4.18 Variation of Unconfined Compressive Strength with Durability Cycles for EF-5000S Samples

4.6.3.2 Volume Change Characteristics

Figure 4.19 shows the volumetric strain in percent of 14-Cycle EF-5000S sample at various wetting and drying cycles. The maximum volumetric change of 22.7% (represented with red ink in the figure) was observed on 7-Cycle of the total 14 cycles of the durability (wetting and drying test). The minimum value was 11.3% and an average volumetric strain for the sample was reported to be 18.9%. It should be noted that the maximum value is based on the volumetric change per cycle during moisture hydration. The durability cycle starts at drying and ends at wetting to complete one cycle.

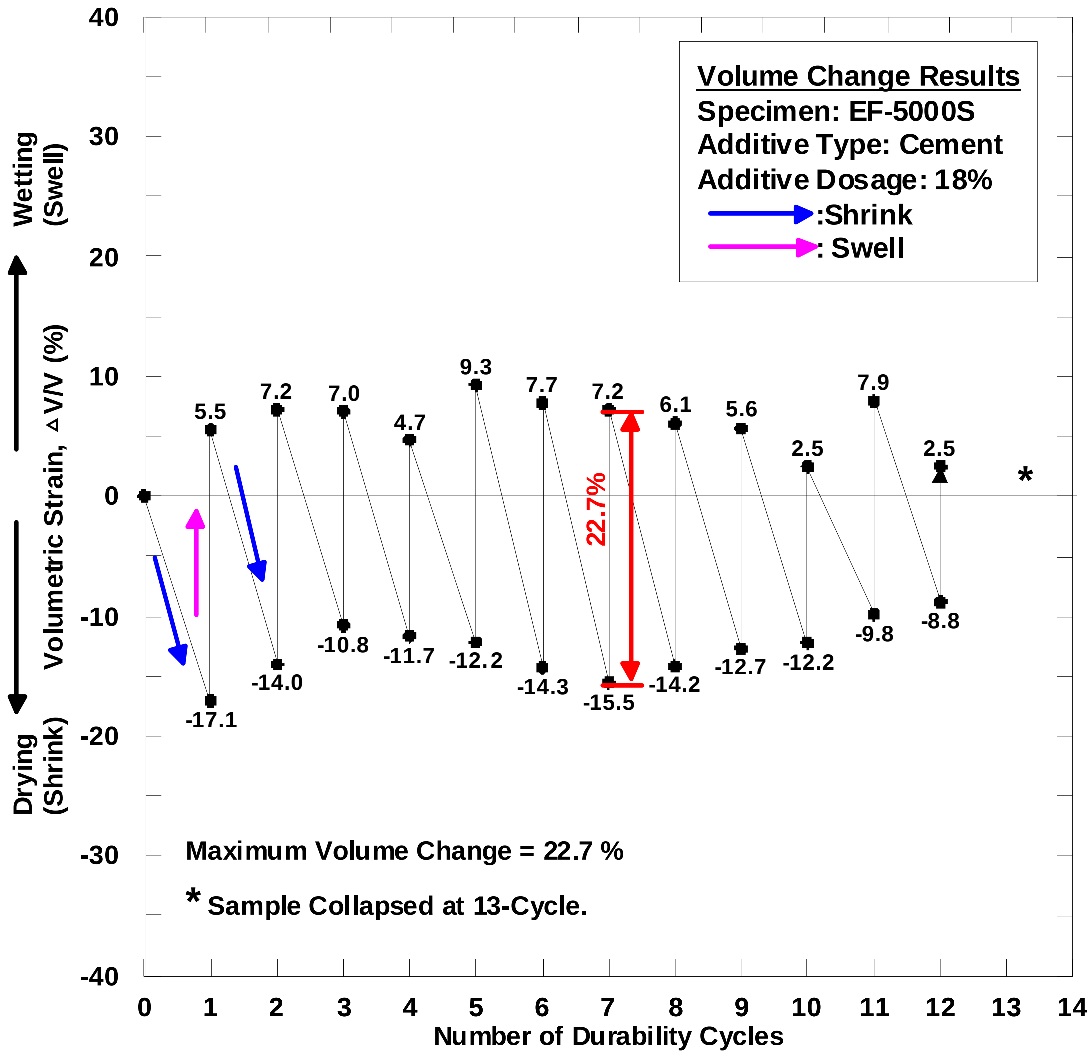


Figure 4.19 Variation of Volumetric Strain with Durability Cycles for EF-5000S Sample

4.6.3.3 Weight Change Characteristics

The EF-5000S sample lasted a complete 12 cycles of wetting-drying process and collapsed at the end of 13-Cycle wetting. Hence no weight change data was available for the 13-Cycle and 14-Cycle durability tests. Figure 4.20 shows the weight in percent of 14-Cycle EF-5000S sample at various wetting and drying cycles. The maximum weight change observed was 41.9% (represented with red ink in the figure) and the minimum was 39.0%. The average weight change for the sample was reported to be 40.2%. It should be noted that the maximum value is based on the weight change per cycle during moisture hydration.

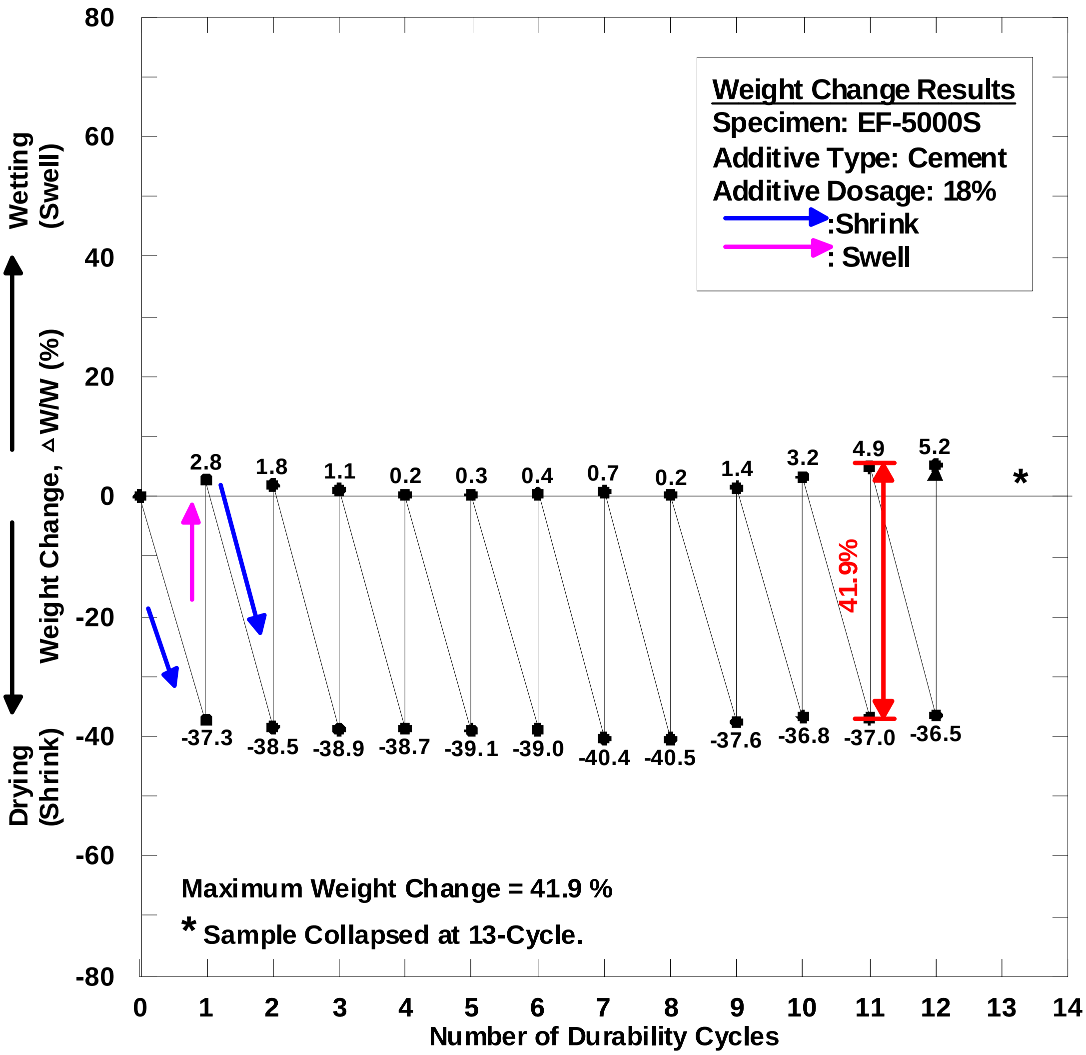


Figure 4.20 Variation of Weight Change with Durability Cycles for EF-5000S Sample

4.6.3.4 Leachate Studies

Since the EF-5000S sample did not last a complete 14 cycles, leachate was collected from 1, 3 and 7-Cycle samples only. Table 4.7 shows the spreadsheet for the determination of calcium loss in ppm for EF-5000S sample. Furthermore, Figure 4.21 shows the variation of calcium loss with the durability cycles. The maximum and minimum calcium ion concentrations leached out were approximately 1,040 ppm and 406 ppm that were observed in the 7-Cycle and 1-Cycle respectively. The calcium loss seems to increase with the increase in durability cycles. The average calcium loss for this sample was 760 ppm. The average total calcium ion concentration loss over 7 cycles of durability testing was observed to be approximately 5,320 ppm.

Table 4.7 Calculation of Calcium Ion Concentration Loss for EF-5000S Sample

Cycle #	**Calcium Concentration (ppm)**
1	406
3	834
7	1,040
Average Calcium Loss in 7 cycles (ppm) = 760	
Total Calcium loss after 7 cycles (ppm) = 5,320	

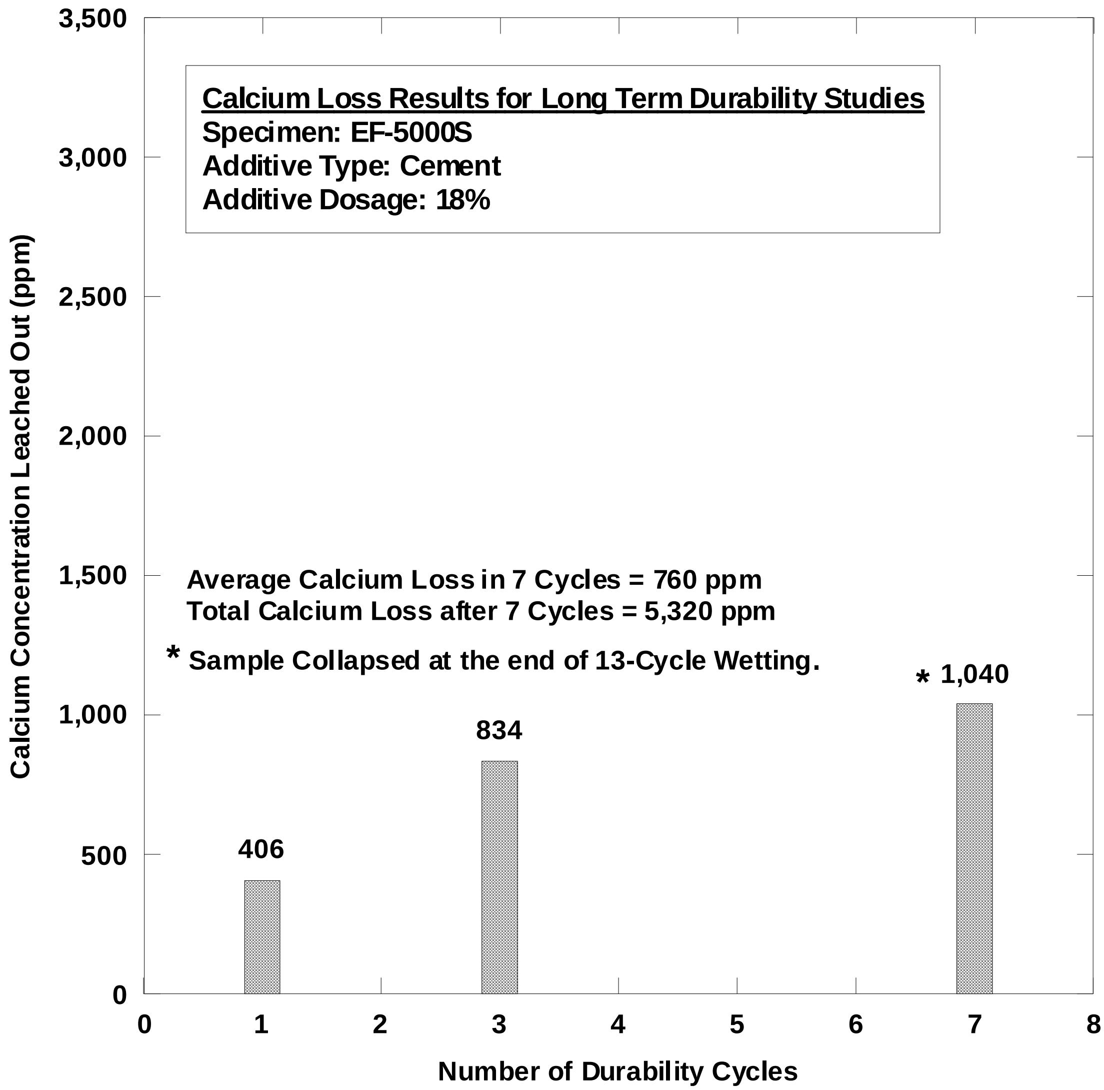
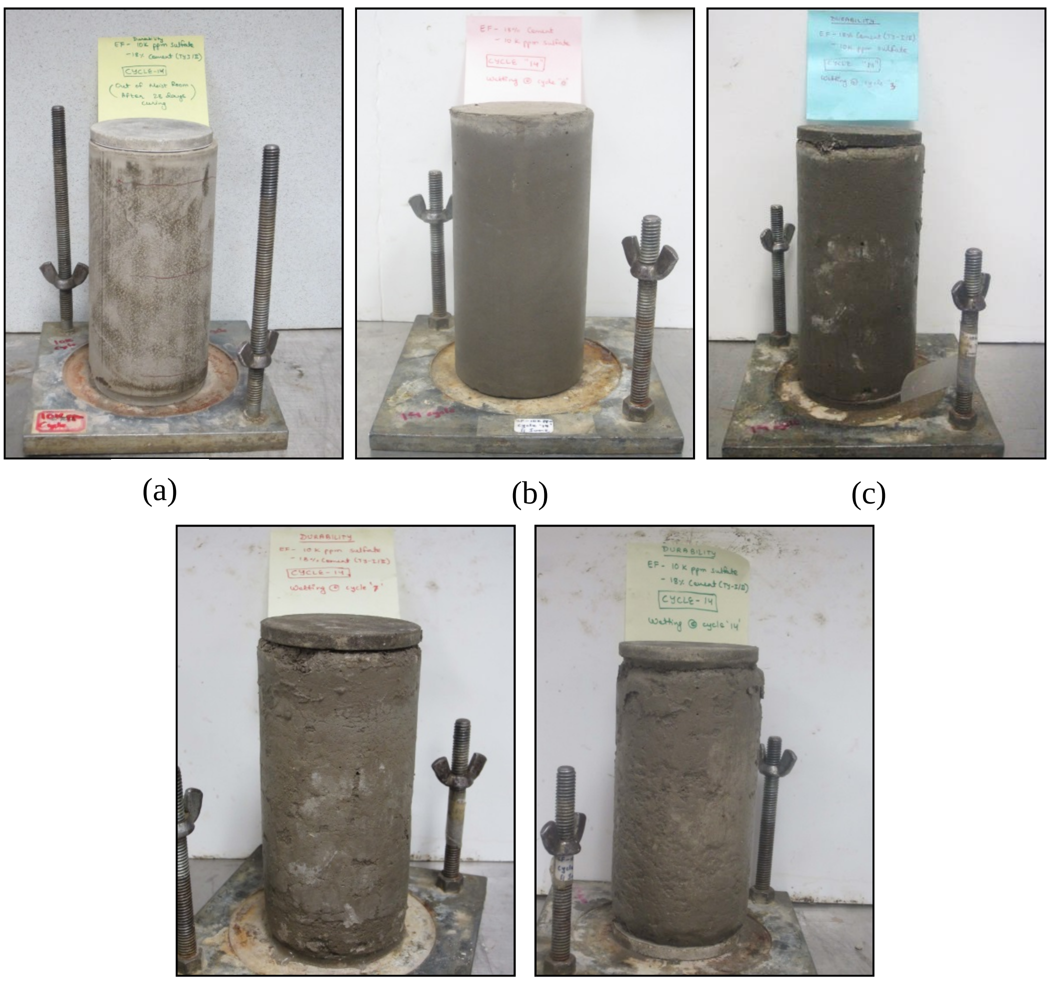


Figure 4.21 Variation of Calcium Concentration with Durability Cycles for EF-5000S Sample

4.6.4 Sample EF-10000S

On the contrary, the EF-10000S sample lasted through all the 14 cycles of wetting and drying process. Figure 4.22 shows the pictures after 28 days curing, 0, 3, 7 and 14 wetting cycles for the 14-Cycle sample. In this section, the results from the UCS test, volume and weight change characteristics along with leachate studies are presented. For all the volume change, weight change and leachate studies, only the 14-Cycle sample was considered.



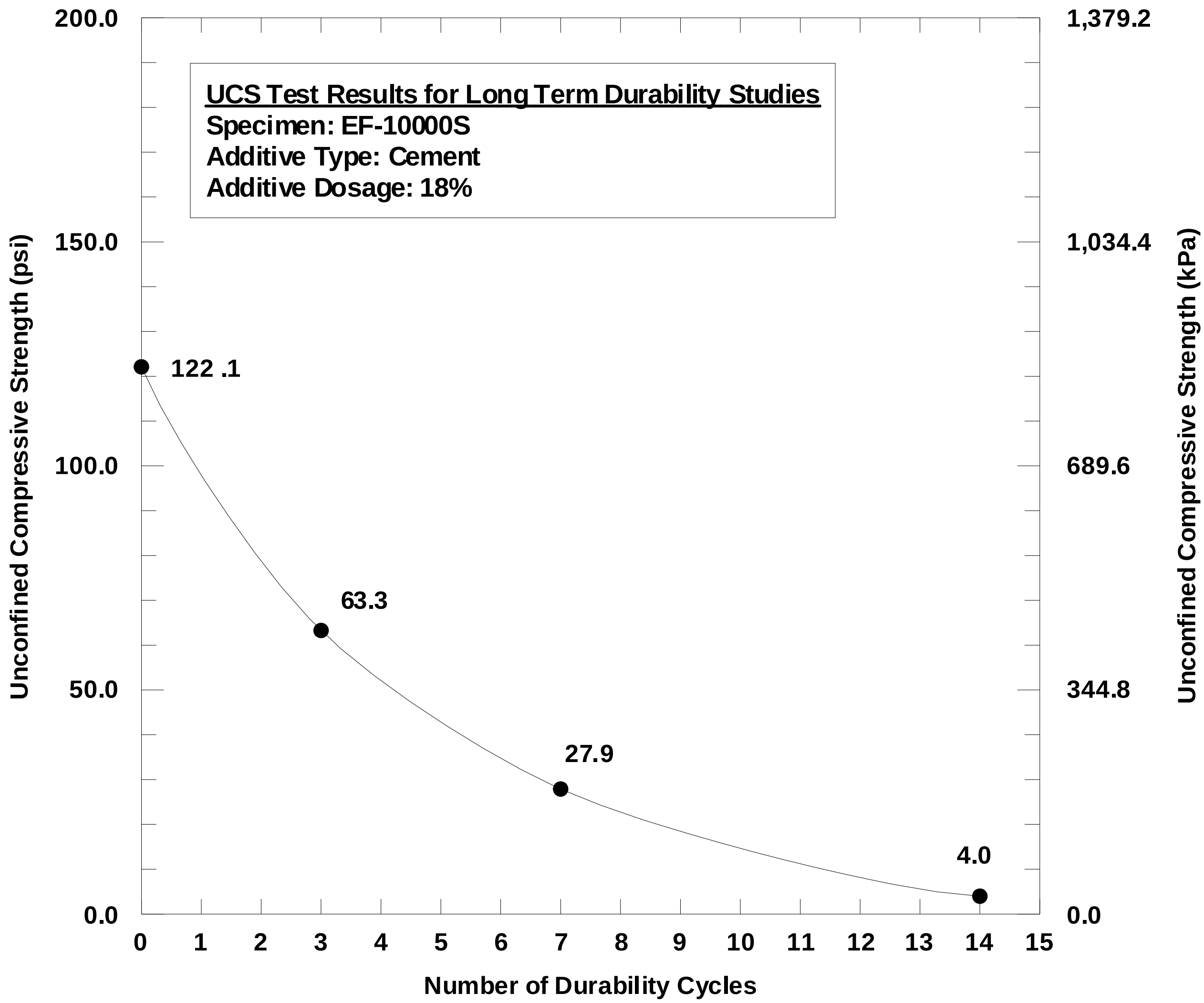
(a) (b) (c)

(d) (e)

Figure 4.22 Pictures of EF-10000S 14-Cycle Sample showing Sample after (a) 28 days Curing, (b) 0-Cycle Wetting, (c) 3-Cycle Wetting, (d) 7-Cycle Wetting and (e) 14-Cycle Wetting

4.6.4.1 Unconfined Compressive Strength Test Results

Figure 4.23 below shows the 0, 3, 7, 14 cycles UCS test result for EF-10000S sample. The peak UCS value was 842.1 kPa (122.1 psi) for 0-Cycle sample. The strength decreased as the cycles increased. 3-Cycle and 7-Cycle samples exhibited a UCS value of 436.3 kPa (63.3 psi) and 192.3 kPa (27.9 psi) respectively. The 14-Cycle sample had a UCS value of 27.4 kPa (4.0 psi).



Values next to the bullets represent Unconfined Compressive Strength (UCS) of the specimen in lb/in^2 (psi).

Figure 4.23 Variation of Unconfined Compressive Strength with Durability Cycles for EF-10000S Samples

4.6.4.2 Volume Change Characteristics

Figure 4.24 shows the volumetric strain in percent of 14-Cycle EF-10000S sample at various wetting and drying cycles. The maximum volumetric change of 35.5% (represented with red ink in the figure) was observed on 2-Cycle of the total 14 cycles of wetting and drying. The minimum value was 11.6% and an average value was 20.6%. The sample exhibited higher volumetric strain than the control soil, EF-2500S and EF-5000S CLSM samples. It should be noted that the maximum value is based on the volumetric change per cycle during moisture hydration. The durability cycle starts at drying and ends at wetting to complete one cycle.

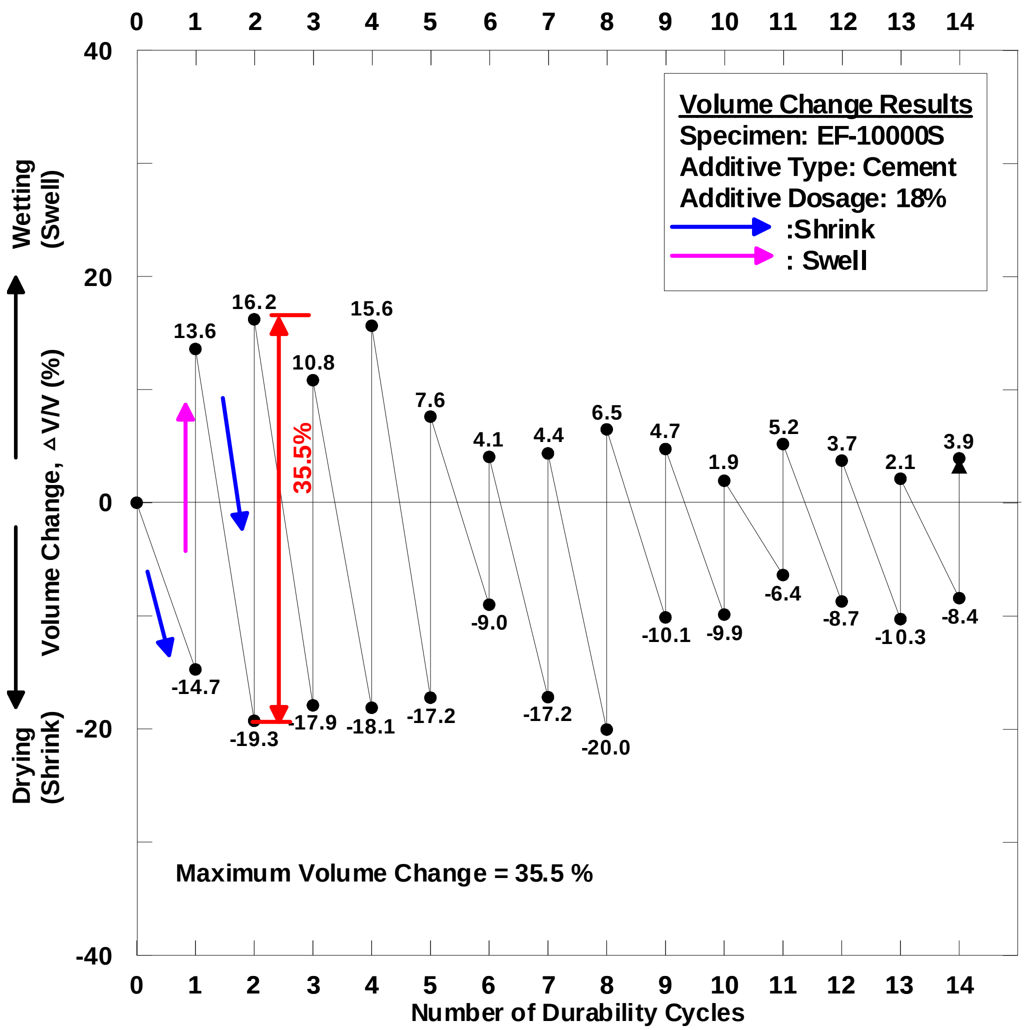


Figure 4.24 Variation of Volumetric Strain with Durability Cycles for EF-10000S Sample

4.6.4.3 Weight Change Characteristics

The EF-10000S sample lasted a complete 14 cycles of wetting-drying process. Figure 4.25 shows the weight change in percent of the sample. The maximum weight change observed was 42.1% (represented with red ink in the figure) and the minimum was 32.3%. The average weight change for EF-10000S sample was reported to be 37.7%. The weight change was higher than the previous samples. It should be noted that the maximum value is based on the weight change per cycle during moisture hydration.

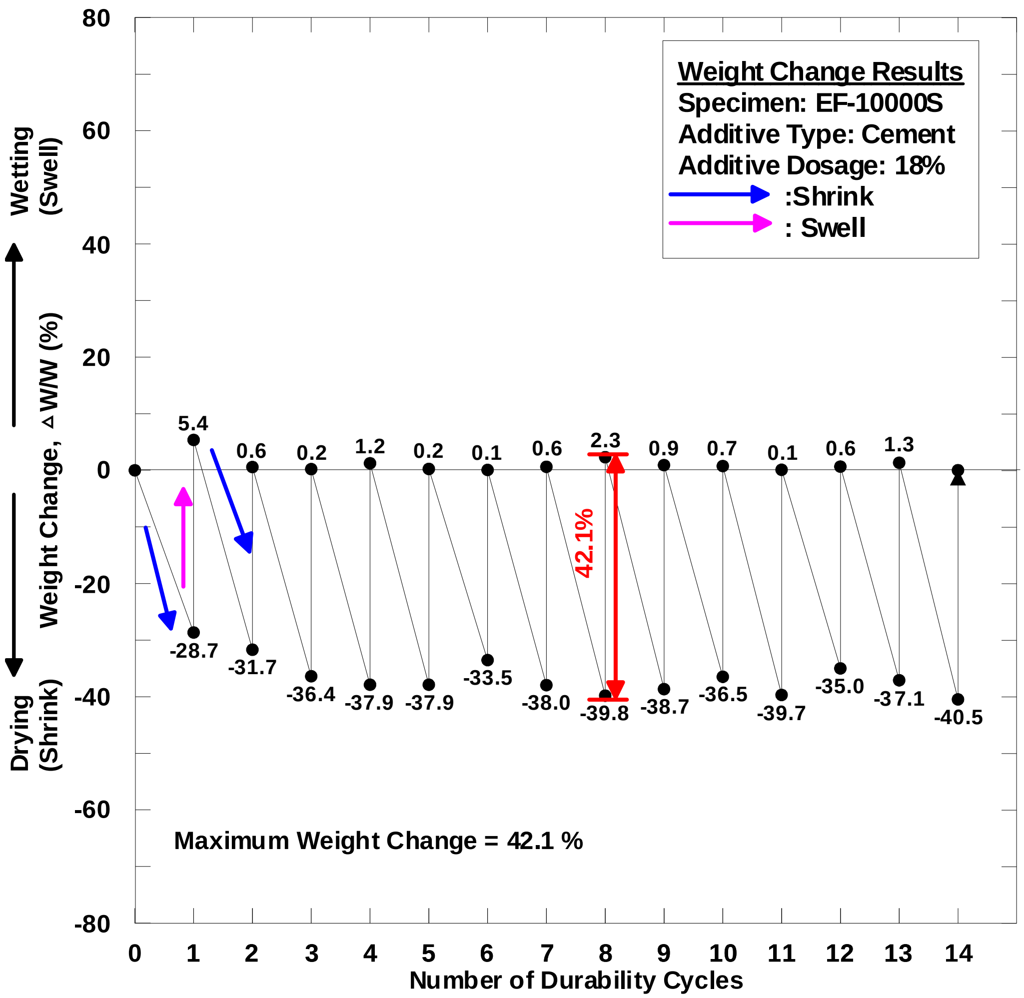


Figure 4.25 Variation of Weight Change with Durability Cycles for EF-10000S Sample

4.6.4.4 Leachate Studies

Leachate samples were collected at the end of wetting of 1, 3, 7 and 14 cycles. Table 4.8 shows the spreadsheet for the determination of calcium loss for EF-10000S sample. Furthermore, Figure 4.26 shows the variation of calcium loss with the durability cycles. The maximum and minimum calcium ion concentrations leached out were approximately 2,386 ppm and 660 ppm that were observed in 14-Cycle and 1-Cycle respectively. The average calcium loss for this sample was 1,448 ppm. The average total calcium ion concentration loss over 14 cycles of durability testing was observed to be approximately 20,272 ppm. Considering 1, 3 and 7 cycles only, the average calcium loss was 1,135 ppm. Figure 4.26 presents both the 7-Cycle and 14-Cycle calcium ion leached average and totals. However, the analysis which is presented in the following sections only considers the 1, 3 and 7-cycle leachate collection due to the fact that none of the other durability samples such as 2500S, 5000S and 20000S survived all the 14 w-d cycles.

Table 4.8 Calculation of Calcium Ion Concentration Loss for EF-10000S Sample

Cycle #	**Calcium Concentration (ppm)**
1	660
3	1,066
7	1,680
14	2,386
Average Calcium Loss for all 14 cycles (ppm) = 1,448	
Average Calcium Loss for only 7 cycles (ppm) = 1,135	
Total Calcium loss after 14 cycles (ppm) = 20,272	
Total Calcium loss after 7 cycles (ppm) = 7,947	

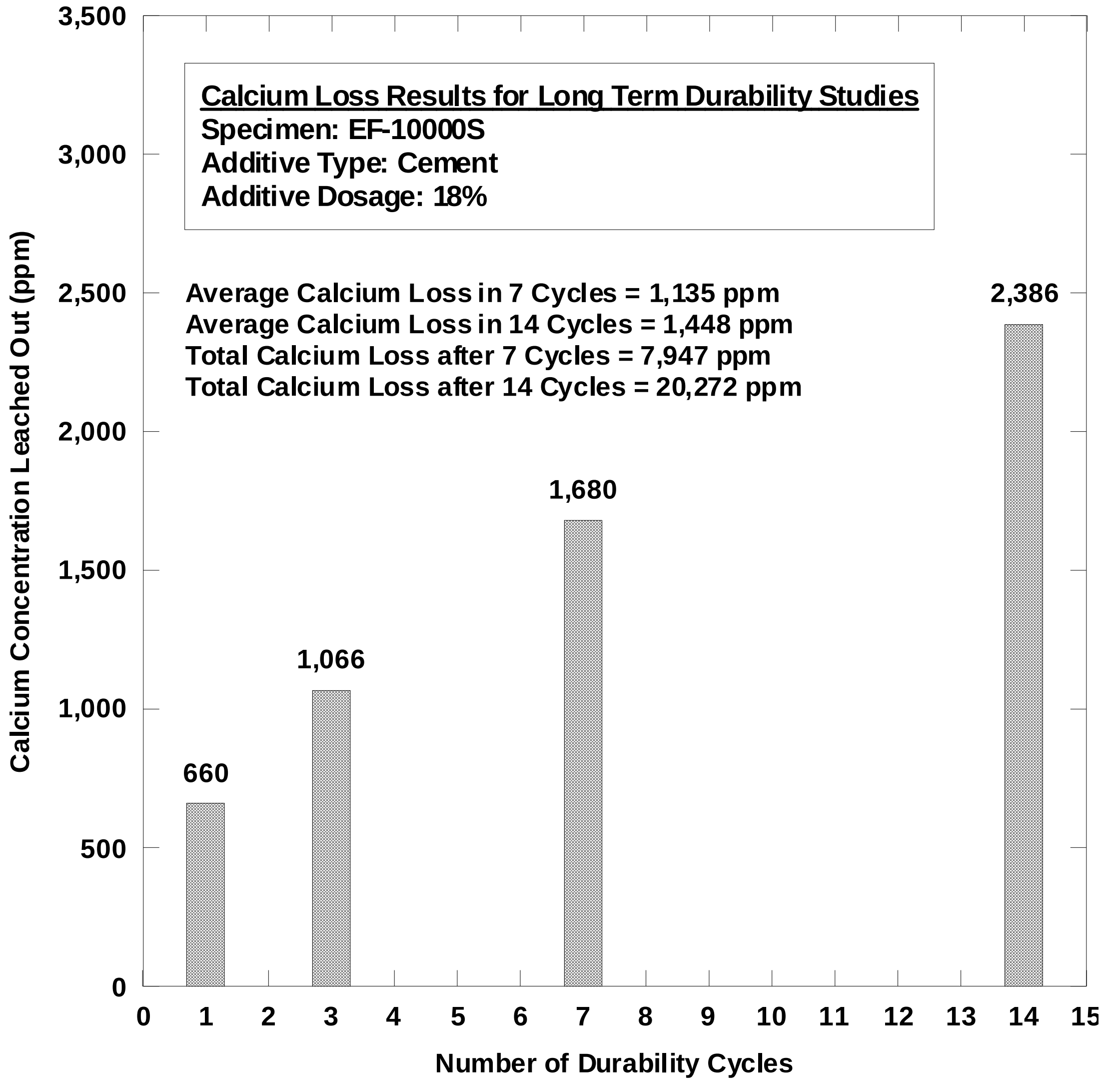


Figure 4.26 Variation of Calcium Concentration with Durability Cycles for EF-10000S Sample

4.6.5 Sample EF-20000S

The EF-20000S sample only lasted 8 cycles of wetting and drying cycles. The sample collapsed at the end of 8-Cycle wetting period. Hence, no UCS test was conducted on the 14-Cycle sample. Figure 4.27 shows the pictures after 28 days curing, 0, 3, 7 and 8 wetting cycles for the 14-Cycle EF-20000S sample. In this section, the results from the UCS test, volume and weight change characteristics along with leachate studies are presented. For all the volume change, weight change and leachate studies only the 14-Cycle sample was considered.

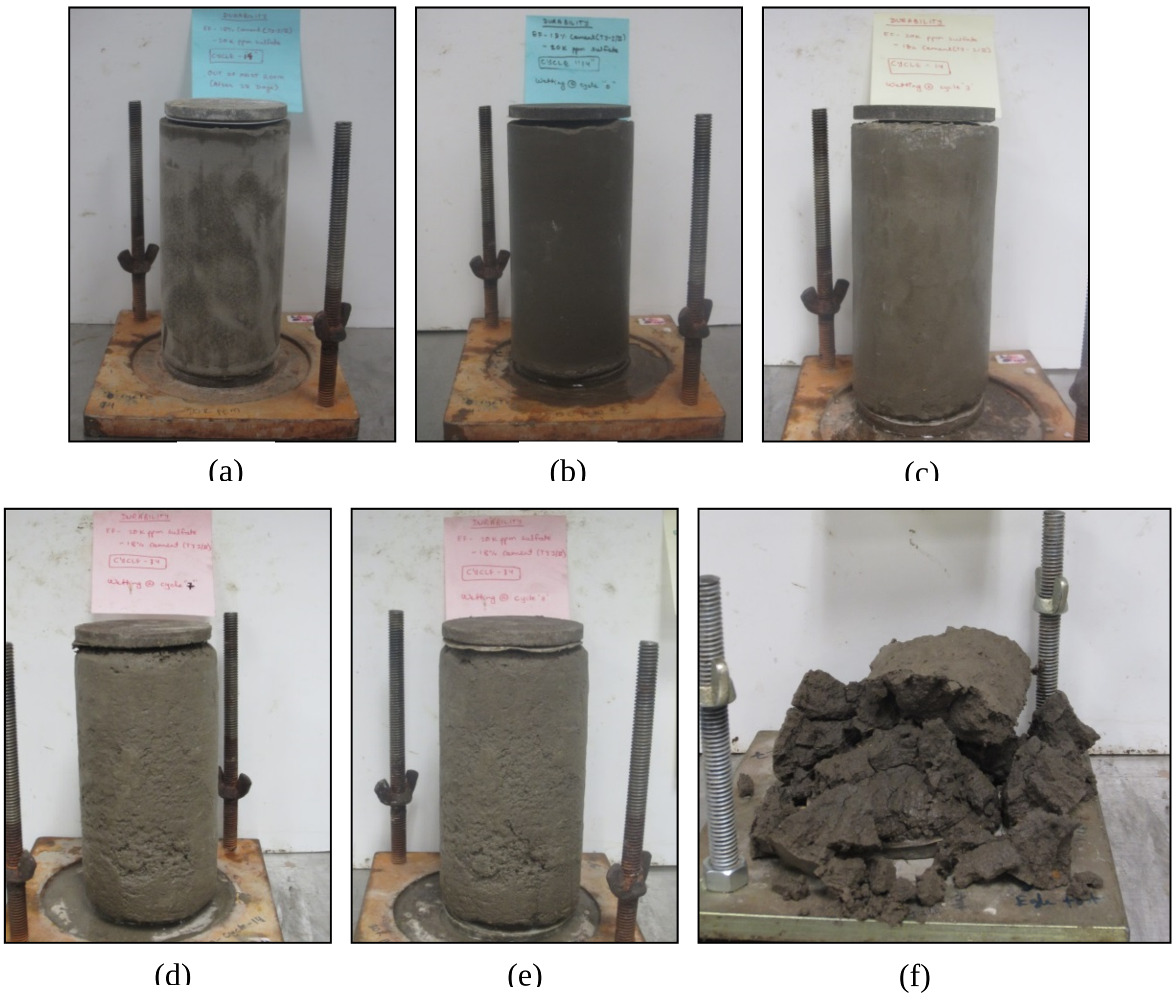
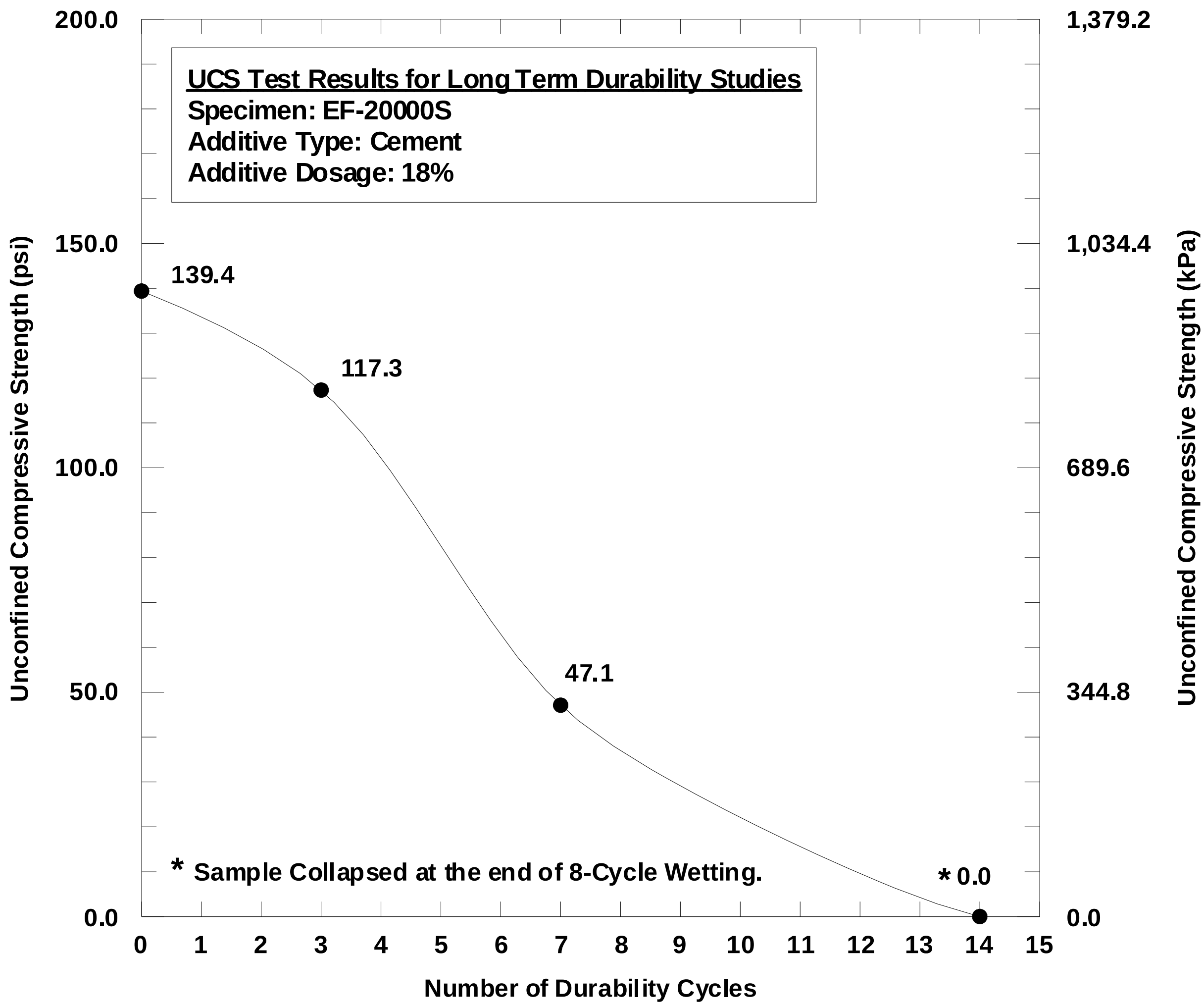


Figure 4.27 Pictures of EF-20000S 14-Cycle Sample showing Sample after (a) 28 days Curing, (b) 0-Cycle Wetting, (c) 3-Cycle Wetting, (d) 7-Cycle Wetting, (e) 8-Cycle Wetting and (f) 8-Cycle Wetting (Sample Collapse)

4.6.5.1 Unconfined Compressive Strength Test Results

Figure 4.28 below shows the 0, 3 and 7 cycles UCS test result for EF-20000S samples. The peak UCS value was 961.4 kPa (139.4 psi) for 0-Cycle sample. The strength decreased as the cycles increased. 3-Cycle and 7-Cycle samples exhibited a UCS value of 809.2 kPa (117.3 psi) and 324.5 kPa (47.1 psi) respectively.



Values next to the bullets represent Unconfined Compressive Strength (UCS) of the specimen in lb/in^2 (psi).

Figure 4.28 Variation of Unconfined Compressive Strength with Durability Cycles for EF-20000S Samples

4.6.5.2 Volume Change Characteristics

Compared to all the different sulfate concentration samples, the EF-20000 samples lasted the least number of durability cycles, 8 cycles. The 14-Cycle sample collapsed at the end of the eighth cycle. Figure 4.29 shows the volumetric strain in percent of 14-Cycle EF-20000S sample at various w-d cycles. The maximum volumetric change of 36.0% (represented with red ink in the figure) was observed in 4-Cycle of the total 14 cycles of wetting and drying. The minimum value was 11.3% and an average value was 21.7%. It should be noted that the maximum value is based on the volumetric change per cycle during moisture hydration. The durability cycle starts at drying and ends at wetting to complete one cycle.

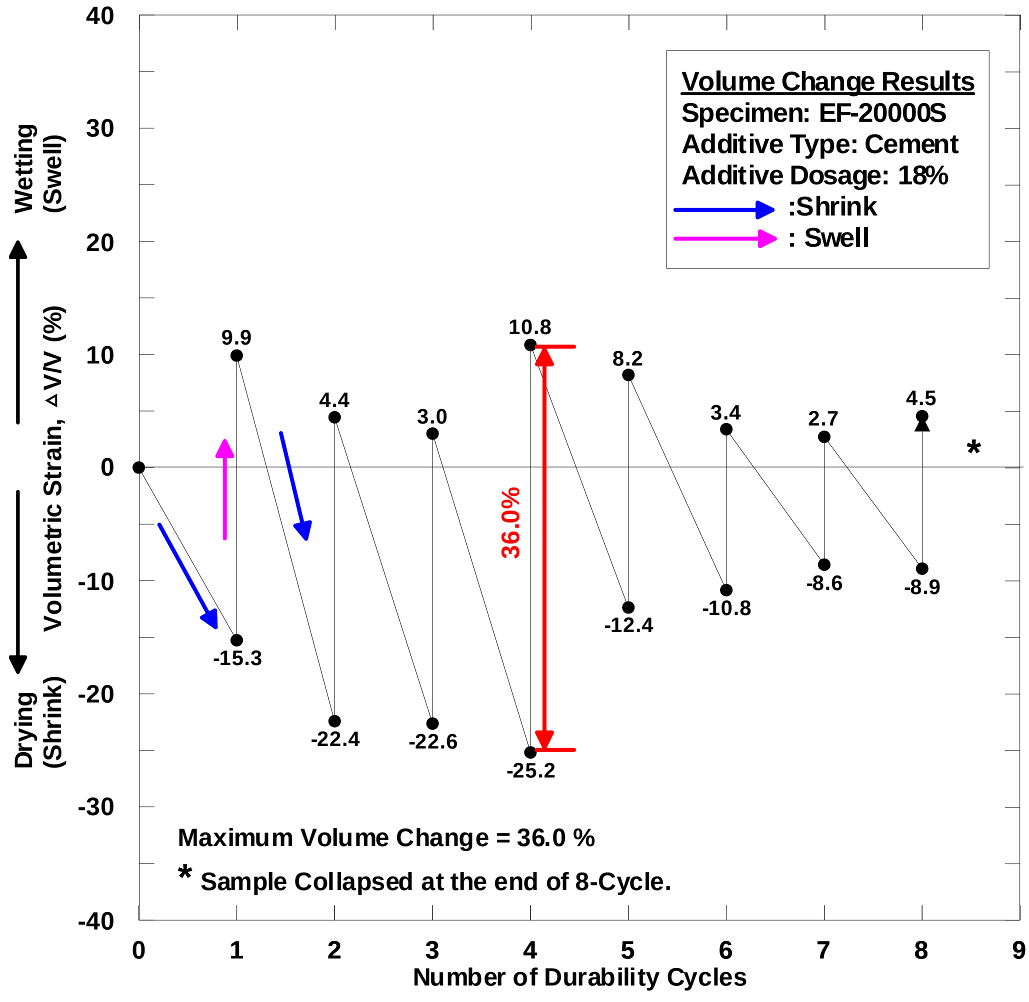


Figure 4.29 Variation of Volumetric Strain with Durability Cycles for EF-20000S Sample

4.6.5.3 Weight Change Characteristics

The EF-20000S sample lasted a complete 7 cycles of wetting-drying process and collapsed at the end of the 8-Cycle wetting period. Although, the UCS test was not conducted on the sample, weight of the sample was recorded at the end of 8-Cycle wetting period with a total of 8 cycles of weight change data recorded. Figure 4.30 shows the weight change in percent of the sample. The maximum weight change observed was 53.0% (represented with red ink in the figure) and the minimum was 35.9%. The average weight change for the sample was reported to be 41.2%. It should be noted that the maximum value is based on the weight change per cycle during moisture hydration.

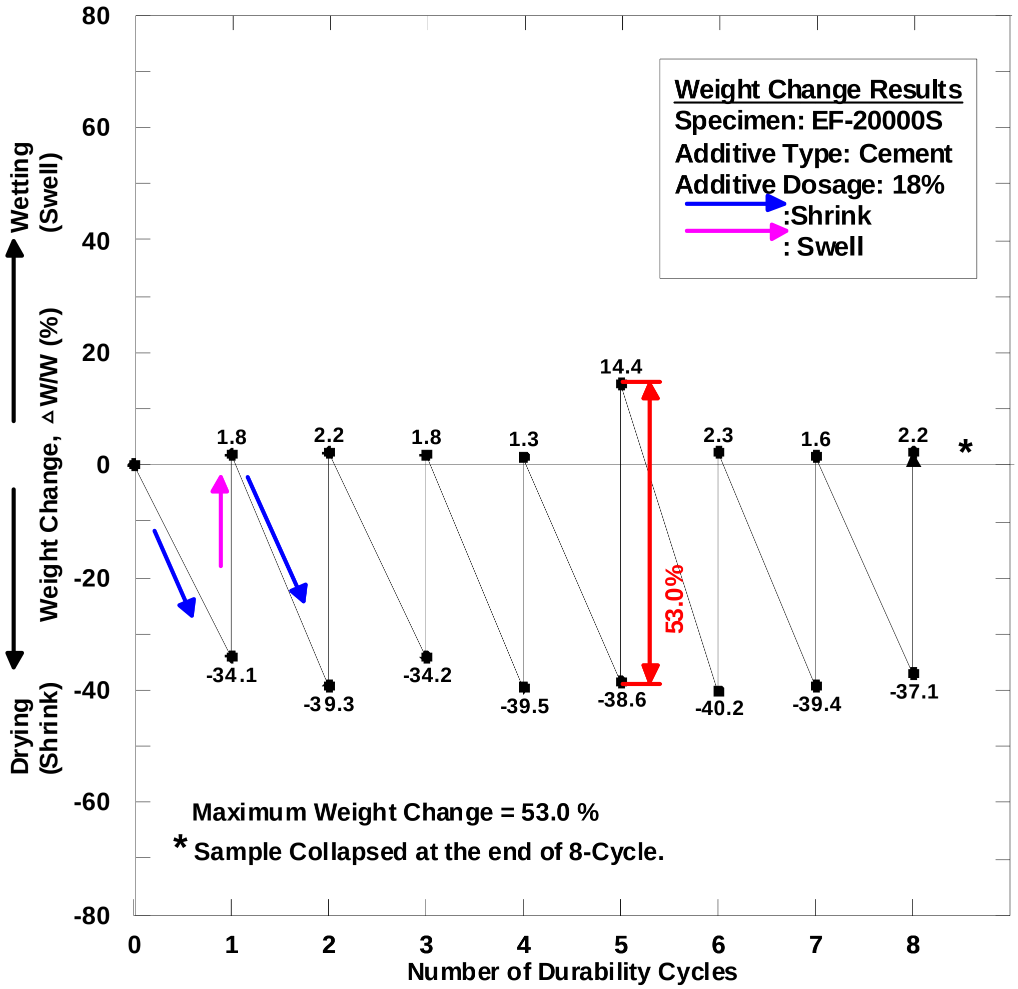


Figure 4.30 Variation of Weight Change with Durability Cycles for EF-20000S Sample

4.6.5.4 Leachate Studies

Since the EF-20000S sample did not last a complete 14 cycles, leachate was collected from 1, 3 and 7-Cycle samples only. Table 4.9 shows the spreadsheet for the determination of calcium loss in ppm for EF-20000S sample. Furthermore, Figure 4.31 shows the variation of calcium loss with the durability cycles. The maximum and minimum calcium ion concentrations leached out were approximately 2,596 ppm and 620 ppm that were observed in the 7^{-}Cycle and 1-Cycle respectively. The calcium loss seems to increase with the increase in durability cycles. The average calcium loss for this sample was 1,621 ppm. The average total calcium ion concentration loss over 7 cycles of durability testing was observed to be approximately 11,345 ppm.

Table 4.9 Calculation of Calcium Ion Concentration Loss for EF-20000S Sample

Cycle #	**Calcium Concentration (ppm)**
1	620
3	1,646
7	2,596
Average Calcium Loss in 7 cycles (ppm) = 1,621	
Total Calcium loss after 7 cycles (ppm) = 11,345	

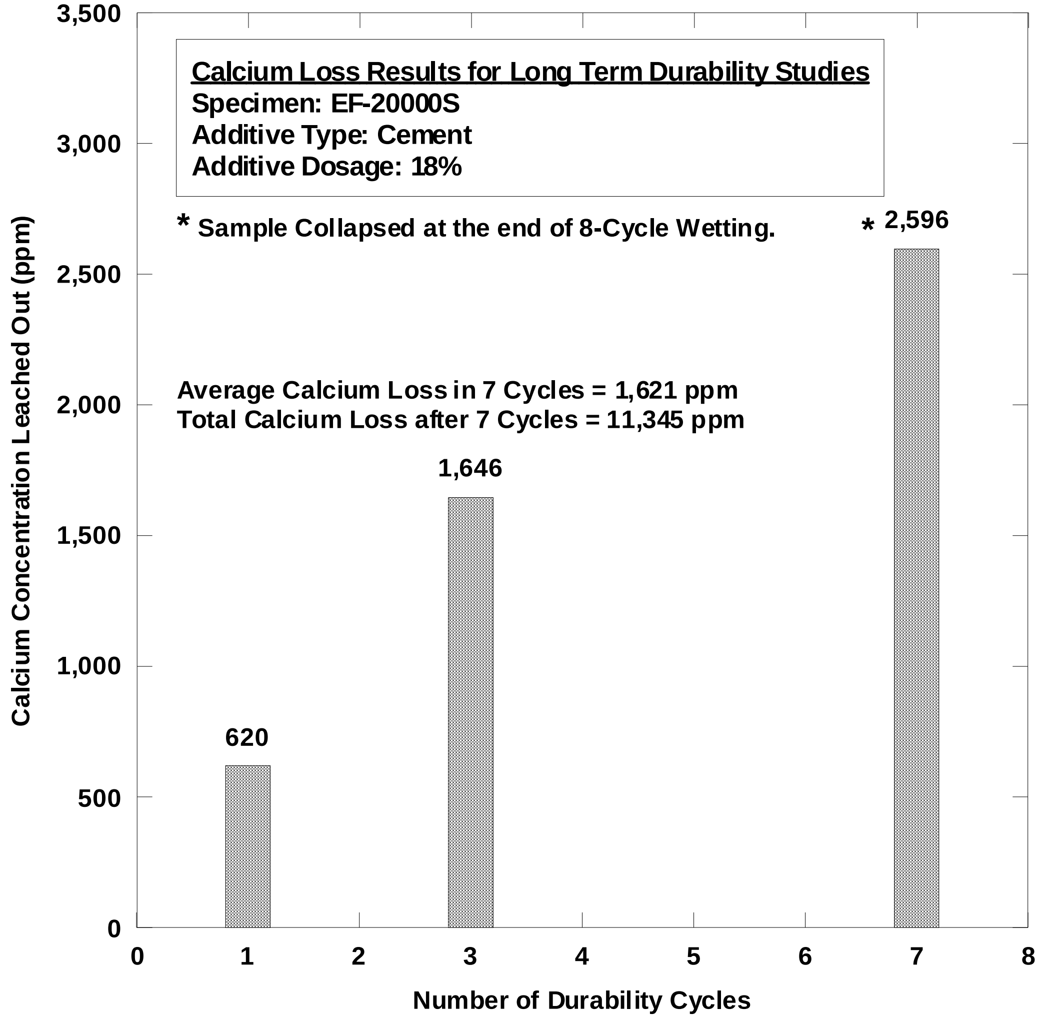


Figure 4.31 Variation of Calcium Concentration with Durability Cycles for EF-20000S Sample

4.6.6 Summary of Long Term Durability Studies Results

The long term durability studies tests included UCS tests, volumetric changes, weight changes and leachate studies on all the EF-2500S, EF-5000S, EF-10000S and EF-20000S samples for 0, 3, 7 and 14 durability cycles. As previously mentioned, for each sample category, 4 identical samples namely 0-Cycle, 3-Cycle, 7-Cycle and 14-Cycle samples were prepared to monitor the changes in 0, 3, 7 and 14 cycles of wetting and drying. For instance, a 2500S sample had 0-Cycle, 3-Cycle, 7-Cycle and 14-Cycle samples. The summary of all the results obtained from the long term durability study are presented in the sections below.

4.6.6.1 Unconfined Strength Test Summary

Table 4.10 provides a summary of the UCS test on all the durability samples. From the table, it is observed that the wetting and drying of durability tests seem to reduce the strength of the samples. The strength of the 0-Cycle samples also indicates that higher the sulfate concentration in the sample, the higher is the unconfined compressive strength. EF-20000S, 0-Cycle exhibited the peak UCS value of 139.4 psi. As the number of durability cycles increased, the samples' strength declined tremendously. EF-NAT, EF-2500S, EF-5000S and EF-20000S 14-Cycle collapsed before completing all the 14 durability cycles. A thorough analysis on the results is presented in the upcoming sections.

Table 4.10 Summary of UCS Test Results for Eagle Ford CLSM Samples under Long Term Durability Studies

Sample Name	**Sulfate Concentration (ppm)**	**Unconfined Compressive Strength (psi)**			
		0-Cycle	**3-Cycle**	**7-Cycle**	**14-Cycle**
EF-NAT	100 or less	50.7	12.2	10.0	0.0 (Sample Collapsed)
EF-2500S	2,500	64.5	12.3	10.6	0.0 (Sample Collapsed)
EF-5000S	5,000	72.0	57.6	15.7	0.0 (Sample Collapsed)
EF-10000S	10,000	122.1	63.3	27.9	4.0
EF-20000S	20,000	139.4	117.3	47.1	0.0 (Sample Collapsed)

4.6.6.2 Volume and Weight Change Summary

Table 4.11 provides the volume change summary of durability studies conducted on Eagle Ford CLSMs. Table 4.12 provides the weight change summary observed during the durability cycles. For both the changes, a maximum, minimum and an average reported values are presented. The 20000S sample experienced the highest volumetric strain of 36.0%. The maximum weight change observed was 53.0% which was exhibited by the EF-20000S sample due to hydrating conditions. Overall, Eagle Ford CLSM exhibited high values of volumetric and weight changes. These high changes must be the characteristic of fat clay (CH Soil).

Table 4.11 Summary of Volumetric Change for Eagle Ford CLSM Samples

Summary of Volume Change under Long Term Durability Test				
Sample Name	**Sulfate Concentration (ppm)**	**Volumetric Change (%)**		
		Maximum	**Minimum**	**Average**
EF-NAT	100 or less	10.4	5.5	9.1
EF-2500S	2,500	20.2	5.4	14.1
EF-5000S	5,000	22.7	11.3	18.9
EF-10000S	10,000	35.5	11.6	20.6
EF-20000S	20,000	36.0	11.3	21.7

Table 4.12 Summary of Weight Change for Eagle Ford CLSM Samples

Summary of Weight Change under Long Term Durability Test				
Sample Name	**Sulfate Concentration (ppm)**	**Weight Change (%)**		
		Maximum	**Minimum**	**Average**
EF-NAT	100 or less	35.0	30.4	32.1
EF-2500S	2,500	41.7	33.7	37.0
EF-5000S	5,000	41.9	39.0	40.2
EF-10000S	10,000	42.1	32.3	37.7
EF-20000S	20,000	53.0	35.9	41.2

4.6.6.3 Leachate Studies Summary

The leachate study results, in terms of the amount of calcium concentration loss (ppm) at the completion of 14 durability cycles are shown in Table 4.13. The initial dosage of additive added to all CLSM samples was 18%. The table provides the total amount of calcium leached out during 7 cycles of alternate wetting and drying period. The 14-Cycle leachate data is presented above and are excluded in the summary section since not all the samples lasted all the 14 complete durability cycles which makes the comparison more reasonable. Also the total cement leached out value was obtained from the calibration chart as described in Chapter 3. It shows that the EF-20000S samples lose the most cement. This could be the reason for the strength loss on the samples over the 14 cycles of alternate drying and wetting.

Table 4.13 Summary of Leachate Studies for Eagle Ford CLSM Samples

Sample Name	Sulfate Concentration (ppm)	Total Calcium Leached Out (ppm)	Initial Cement Dosage (%)	Total Cement Leached Out (%)
EF-NAT	100 or less	3,850	18	2.10
EF-2500S	2,500	4,060	18	2.30
EF-5000S	5,000	5,320	18	2.90
EF-10000S	10,000	7,947	18	4.30
EF-20000S	20,000	11,345	18	6.20

4.7 Analysis of Results

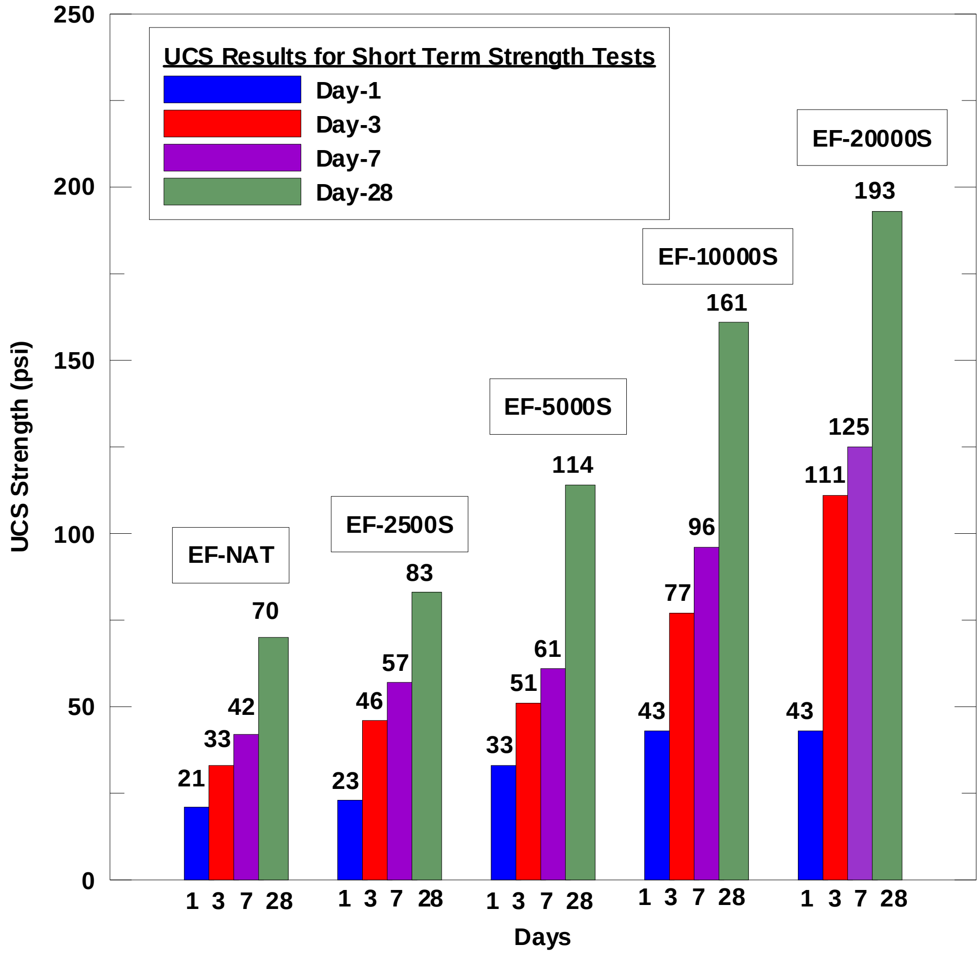
The section presents a thorough analysis of the results obtained in both the short term strength studies and long term durability studies. The analysis part has been divided into two sections namely: The following aspects of this research should be addressed to analyze the test results obtained:

- The effects of sulfate concentration on Short Term Strength Studies
- The effects of sulfate concentration on Long Term Durability Studies

The following sections explain the above two points in more detail.

4.7.1 The effects of sulfate concentration on Short Term Strength Studies

The unconfined compressive strength test was performed for both short term strength samples and long term durability samples. Figure 4.32 shows the summary of UCS comparison between all the different sulfate concentrations tested under short term strength tests. The peak strength of approximately 193 psi was exhibited by EF-20000S sample. The UCS strength for all the four sulfate concentration samples is higher than that of the control soil. The graph also indicates the increase in UCS value with increase in sulfate concentration.



* Values above the bar represent Unconfined Compressive Strength (UCS) of the specimen in lb/in^2 (psi).

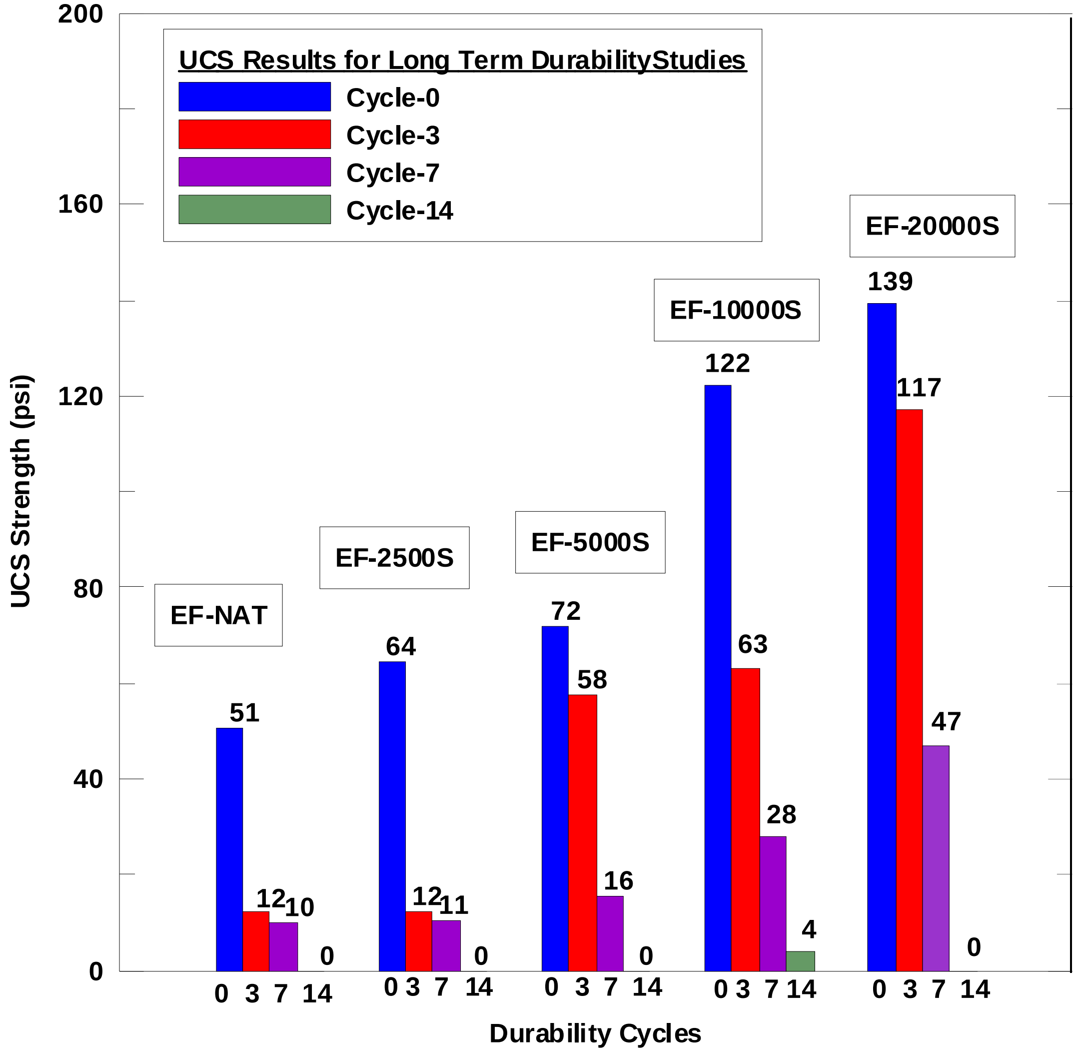
Figure 4.32 Summaries of UCS Test on Eagle Ford CLSMs for Short Term Strength Studies

4.7.2 The effects of sulfate concentration on Long Term Durability Studies

Eagle Ford is classified as CH soil meaning it is high plasticity clay, also known as fat clay. When fat clay is rich with soluble sulfate, according to literature, it could be problematic. Hence, durability studies were conducted to analyze the long term performance of CLSM samples. The long term performance of CLSMs is best assessed by studying the effects of soil type on four engineering parameters: Unconfined Compressive Strength, Volumetric Strain, Weight Change and Calcium ion loss due to leaching. The sections below provide a detailed analysis of each of these parameters based on the results obtained on them.

4.7.2.1 Unconfined Compressive Strength Analysis

For the long term durability samples, the highest UCS value was exhibited by EF-20000S sample. In this case, the strength values decreased significantly with the increase in the durability cycles. Figure 4.33 shows the summary of the UCS test performed on long term durability samples. For all the sulfate concentrations, the 0-Cycle samples exhibited the highest UCS values as expected. When only the 0-Cycle samples are compared, the UCS value increases with the increase in sulfate concentration. The EF-NAT exhibited the least UCS value compared to all the other samples and the EF-20000S exhibited the highest UCS value of approximately 139.4 psi.



* Values above the bar represent Unconfined Compressive Strength (UCS) of the specimen in lb/in^2 (psi).

Figure 4.33 Summary of UCS Test on Eagle Ford CLSMs for Long Term Durability Studies

4.7.2.3 Volume Change Analysis

Another important characteristic to analyze is the volume change data obtained from the results. Since, Eagle Ford is a CH soil, it has a high swelling potential when exposed to continuous supply of moisture. The swell and shrinkage of fat clay during wetting and drying process of durability cycles contributes in strength loss of the samples due to cracking. In addition, the presence of sulfate contributes to the swelling behavior of the soil due to the formation of Ettringite and Thaumasite as discussed in Chapter 2. Figure 4.34 shows the maximum volumetric strain percent exhibited by different CLSM samples. It is clear from the graph that the sample with the highest sulfate concentration exhibited the highest volumetric strain of 36% which was expected of CLSM with high sulfate concentrations. The study also shows that the volumetric strain percent increase with the increase in sulfate concentration in the CLSM samples.

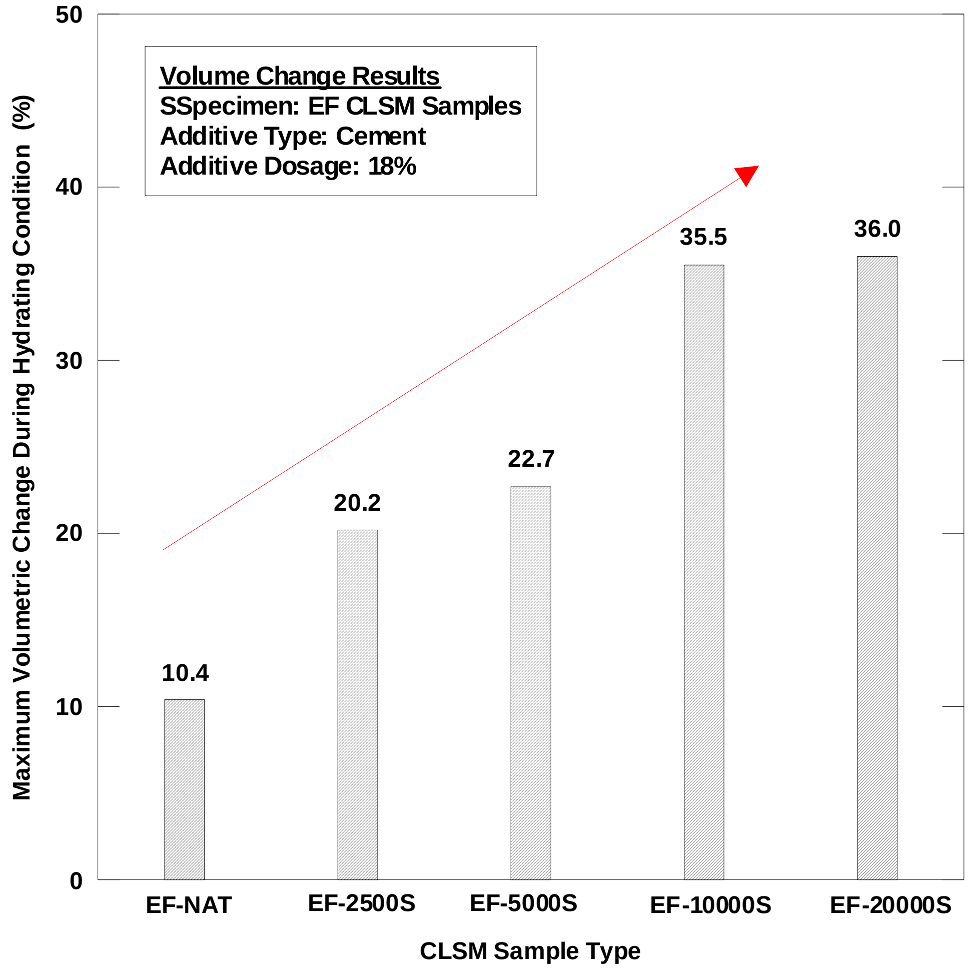


Figure 4.34 Variation of Maximum Volumetric Strain with Different CLSM Samples

4.7.2.4 Weight Change Analysis

Weight change is another parameter that needs to be analyzed during durability studies. Since, Eagle Ford is a CH soil, it has a high swelling potential when exposed to continuous supply of moisture. The swell and shrinkage of fat clay during cyclic w-d process of durability studies contributes in strength loss of the samples. In addition, the presence of sulfate contributes to the swelling behavior of the soil due to the formation of Ettringite and Thaumasite as discussed in Chapter 2.

Figure 4.35 shows the percent weight loss experienced by the samples during durability cycles. The weight loss parameter for this analysis considers the difference in weight between drying of 1-Cycle (initial value) and the drying of 7-Cycle (final value) for all the samples. The main reason for using the 7-Cycle drying as the 'final value' is due to the lack of samples that completed a full 14 cycles of cyclic w-d process. However, all the CLSM samples completed 7 cycles of w-d process.

Furthermore, an oven dry sample can be considered a moisture free sample which provides a firm basis for weight loss studies than a saturated sample. The weight loss percent is obtained by deducting the lowest CLSM sample weight observed from 1-Cycle till 7-Cycle drying (final weight) from the weight of the sample at 1-Cycle drying (initial weight). The difference is divided by the weight of 1-Cycle drying (initial weight) and then multiplied by 100 to represent the result in percentage.

From the figure, it is clear that the samples experienced more weight loss as the concentration of soluble sulfates in them, increased. It can also be inferred that weight loss is an indication of additive loss by the sample. Over time, with the loss in additive, the strength of CLSM also declines.

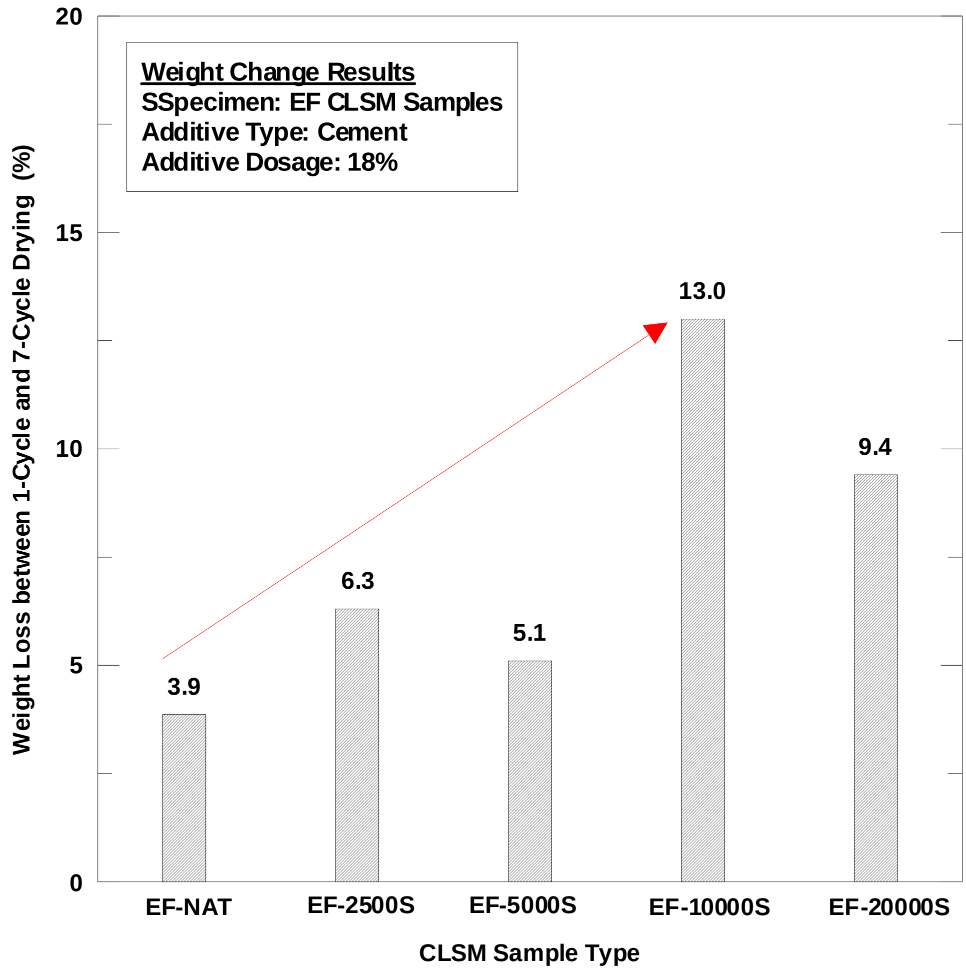


Figure 4.35 Variation of Weight Loss of Different CLSM Samples from 1-Cycle Drying to 7-Cycle Drying

Furthermore, Table 4.12 in the earlier sections of this chapter presents the summary of the results from the weight change studies. Maximum weight change is the summation of the percent changes in weight of wetting and drying process in one cyclic w-d cycle from the initial compacted CLSM weight before durability testing. For this analysis the maximum weight change values are considered instead of the average values. It is observed that the increase in sulfate concentration also triggers the increase in weight change for the cement treated CLSM samples. Higher the soluble sulfate concentration in a soil, the higher is the moisture absorption capacity of the soil. This causes the soil to swell more. Figure 4.37 shows the pictures at the end of the last drying cycle of each sample before they completed the cyclic w-d period or before they collapsed. During laboratory testing, significant amount of weight loss and sample cracking were observed during the drying cycles as opposed to the wetting cycles as evident in Figure 4.36 below. However, significant amount of additive was lost due to leaching during wetting cycles which is discussed in the following sections.

(a) (b) (c)

(d) (e)

Figure 4.36 Pictures of (a) EF-NAT after 13-Cycle Drying, (b) EF-2500S after 13-Cycle Drying, (c) EF-5000S after 13-Cycle Drying, (d) EF-10000S after 14-Cycle Drying, and (e) EF-20000S after 8-Cycle Drying

4.7.2.5 Effects of Calcium Concentration Loss on the Strength of CLSMs

Leachability of a CLSM is the parameter used to measure the permanency of the chemical additive. In actual site conditions, this permanency decreases with time due to environmental effects like surface runoff. Also rainfall infiltration can sometimes leach the additive and reduce the soil strength. In the laboratory, replication of rainfall infiltration can be achieved by conducting the leachate studies.

Figure 4.37 shows the variation of total calcium loss due to leaching from the various samples. It is clear that the calcium ion concentration loss is prominent for the EF-20000S sample. The control soil lost the least amount of calcium during wetting and drying cycles. It is evident from Figure 4.37 that higher the concentration of sulfate higher the amount of calcium ion concentration leached out during wetting process. It is also observed that with higher calcium leached out, the strength of the sample decreases compared to the previous cycle but considering each cycle, the strength increases with the increase in sulfate concentration i.e. EF-20000S exhibits higher strength values upto 7-Cycle of cyclic w-d compared to the rest of the samples. It can be argued that the reason behind this strength increase in CLSMs with elevated sulfate concentrations during earlier stages of w-d process is due to the higher values of calcium loss by leaching during the wetting process. Since cement is being leached out from the sample, not adequate calcium from cement is present in the sample to escalate the swelling behavior of the high sulfate samples. This in turn might have decelerated the formation of high swelling substance such as Ettringite which is attributed to expansive soil's swelling behavior. However, all the samples failed to retain 50% or more of their initial UCS value after 14 w-d process.

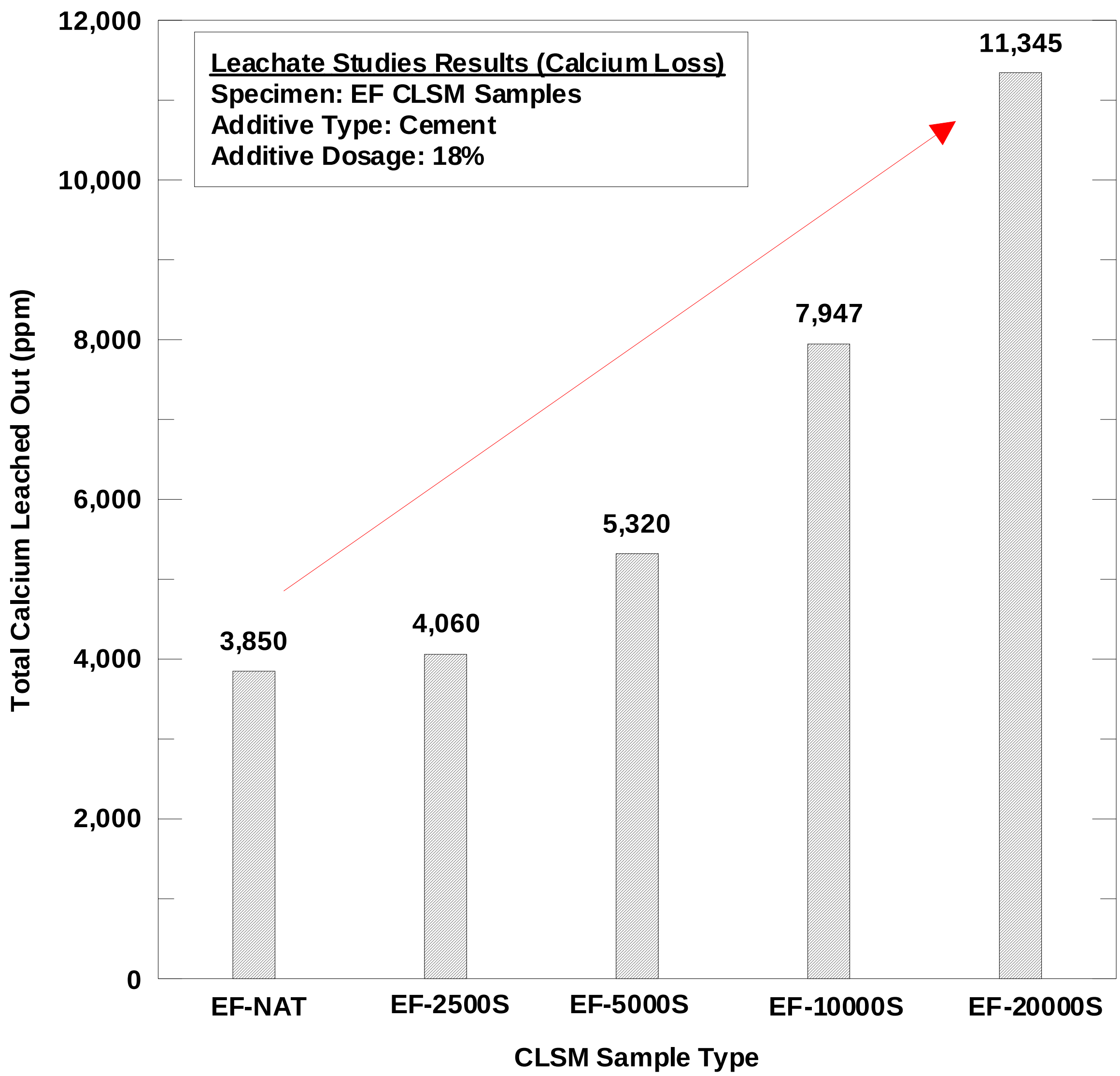


Figure 4.37 Variation of Total Calcium Loss with Different CLSM Samples

Additionally, the amount of additive (cement) loss from the CLSM samples can be determined using a calibration chart as discussed in Chapter 3. The knowledge of the quantity of cement leached out during durability cycles helps in understanding the strength loss characteristics of CLSM samples under wetting and drying process. Table 4.14 shows the values for the total cement leached out, retained and percent cement loss. It is observed that high soluble sulfate concentration in CLSM mixes lead to binder loss up to 35% as exhibited by the EF-20000S sample. The total cement loss is obtained by dividing the total cement leached out by the CLSM sample divided by the initial additive content in the sample (i.e. 18% in this case) and multiplied by 100 to express the value in percent. Figure 4.38 shows the variation of total cement loss for various CLSM samples. EF-20000S experienced the most cement loss and the EF-NAT the least during the durability wetting-drying cycles. The result shows that the cement loss increases with the increase in sulfate concentration.

Table 4.14 Results of Leachate Study showing Values for Additive (Cement)

Sample Name	**Sulfate Concentration (ppm)**	**Total Calcium Leached Out (ppm)**	**Initial Cement Dosage (%)**	**Total Cement Leached Out (%)**	**Total Cement Retained (%)**	**Total Cement Loss (%)**
EF-NAT	100 or less	3,850	18	2.10	15.90	11.7
EF-2500S	2,500	4,060	18	2.30	15.70	12.8
EF-5000S	5,000	5,320	18	2.90	15.10	16.1
EF-10000S	10,000	7,947	18	4.30	13.70	23.9
EF-20000S	20,000	11,345	18	6.20	11.80	34.4

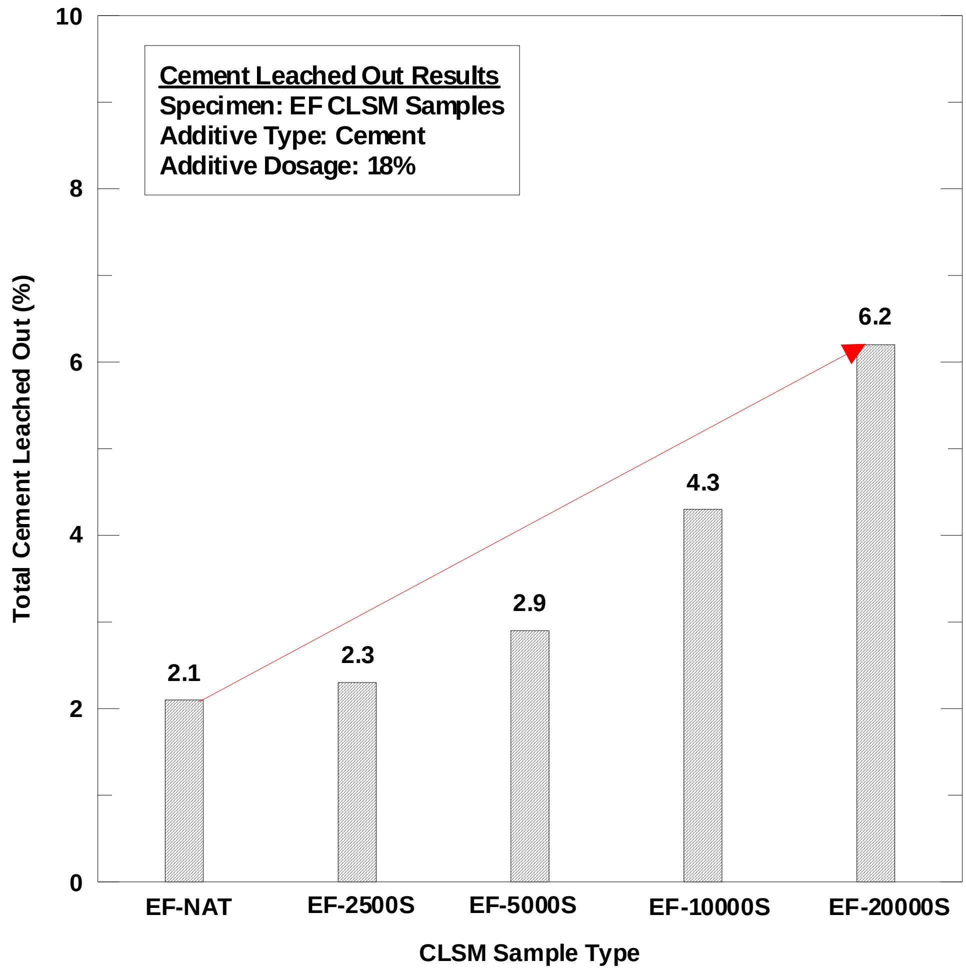


Figure 4.38 Variation of Total Cement Leached Out with Different CLSM Samples

4.8 Summary

During the short term strength studies, it was observed that the increase in sulfate concentration did not seem to affect the UCS values negatively when initial testing cycles are considered. Instead the studies show that the increase in soluble sulfate concentration in a soil (in the form of gypsum) increases the short term strength of the soil. Even in the case of long term durability studies, initially, an increase in UCS value with the increase in sulfate concentration is observed. However, as the durability cycle progresses, the strength reduction is high and can be attributed to durability w-d cycles as well as presence of high concentration of soluble sulfates in the CLSM mix. The volume change, calcium loss and cement loss parameters were the highest for the EF-20000S sample. Therefore, it can be concluded that the presence of higher concentrations of soluble sulfate in soil causes strength reduction of CLSM during durability studies of the CLSM samples. The strength loss can also be attributed to the durability of the sample (i.e of wetting and drying cycles). Even though there is increase in strength of sample with increase in sulfate concentration for high sulfate CLSMs initially, the strength retention value still fell below 50% of their original strength before durability studies. Hence, additional studies need to be conducted to use Portland Cement (TY I/II) as an additive for CLSM prepared using Eagle Ford native soil in order to be used for pipeline bedding material.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 Summary and Findings

This study focuses on the effects of utilizing high sulfate expansive soil treated with cement in CLSM sample preparation. In order to achieve this goal, short term strength and long term durability studies were conducted on the samples which comprised of strength, volumetric and weight change and leachate studies analysis. For the study, soil from Eagle Ford geological formation was selected and treated with Portland Cement (Type I/II). The five sulfate concentrations studied were 100 or less ppm (control soil), 2500 ppm, 5000 ppm, 10000 ppm and 20000 ppm.

From the study, several significant conclusions were drawn and are presented in the following sections. The analysis showed that soluble sulfates present in soil used for CLSM preparation do not have adverse effect on the short term strength of the sample. Also, the CLSM samples with elevated levels of soluble sulfate in the form of gypsum exhibited higher Unconfined Compressive Strength (UCS) values. The increase in swell-shrink behavior of expansive soil with elevated levels of soluble sulfates was also distinctly reflected from the study, where CLSMs with high sulfate concentration exhibited higher shrink-swell behavior. Higher the concentration of soluble sulfates, higher the swell-shrink behavior exhibited by the CLSM samples. However the loss of strength with durability cycles is higher with increase in sulfate concentrations.

Although low strength values were observed during durability w-d cycles, it should be noted that these tests conducted in laboratory were under much harsh conditions than that experienced by CLSMs in the field. Additionally, there are no specific design guidelines for preparation and testing of the CLSMs. Handling of samples during wetting and drying process also contributes to the strength loss during durability studies.

From the test results and the analysis conducted, the conclusions are presented in the following sections. Below is the list of findings from the research:

1. Unconfined Compressive Strength (UCS):

   For short term strength analysis, the UCS value increased with the increase in sulfate concentration.

   For long term durability studies, initially the UCS value increased with the increase in sulfate concentration when 0-Cycle samples were considered. The CLSM with highest sulfate concentration exhibited the highest UCS value. However, with increase in the durability cycles, the UCS value decreased significantly. At the end of 14 cycles of w-d, the strength retained was close to zero for all the samples.

   The UCS loss per cycle increased with the increase in sulfate concentration for both short term strength and long term durability studies as indicated by Figure 4.34.

2. Volume Change:

   For long term durability studies, the volume change per w-d cycle increased with the increase in sulfate concentration meaning, the sample with the highest sulfate concentration exhibited highest volumetric change and the control soil exhibited the least.

3. Weight Change:

   Similar to volume change analysis, for long term durability studies, the weight change per cycle increased with the increase in sulfate concentration meaning, the sample with the highest sulfate concentration exhibited highest weight change and the control soil exhibited the least. For weight change analysis, the weight loss increased with the increase in the sulfate concentration.

4. Leachate studies:

In leachate studies, calcium loss and cement loss were evaluated. The total calcium ion leached out increase with the increase in sulfate concentration. Similarly, the total cement loss also increased with the increase in sulfate concentration in the CLSM sample. The analysis of these results showed that the increase in sulfate concentration lead to binder loss upto 35% as exhibited by the EF-20000S sample.

5.2 Conclusions of Testing

Based on the results obtained from the laboratory testing conducted on CLSM samples prepared using Eagle Ford soil with various sulfate concentrations and 18% cement as an additive, the following conclusions can be made:

1. The strength of CLSM increases with the increase in sulfate concentration instead of decreasing for both short term strength tests and long term durability tests. Most studies indicated that presence of elevated levels of soluble sulfates in expansive soil is problematic and causes strength loss of treated soil over time. Literature shows that the use of recycled gypsum has helped in increasing the unconfined compressive strength of samples during durability studies (Kamei et al., 2011).

2. During long term durability studies with alternate wetting-drying cycles, the UCS value decreased with the increase in durability cycles for all the 5 samples with different sulfate concentrations. However, the UCS value was the highest for the CLSM with the peak sulfate concentration (i.e. 20,000 ppm) for all the 0, 3 and 7 durability cycles represented in Table 4.10 in Chapter 4. This concludes that high concentration of soluble sulfates in CLSM mix does not cause short term strength reduction. However with seasonal wetting-drying cycles the deterioration in strength of the CLSM samples is very high. For most of the samples, by the end of 14 durability cycles, UCS value was close to zero as most of them collapsed before UCS test. Also it should be noted that the tests are conducted under harsh conditions in the laboratory when compared to the

field conditions. Hence it can be concluded that these results predict the worst case scenarios.

3. When high plasticity, expansive soil such as the one from the Eagle Ford geological formation is used to prepare CLSM mix design, the test results showed that the increase in sulfate concentration in expansive soils also increases the volumetric change characteristics of the sample.

4. When 18% Portland cement (Type I/II) was used as a chemical stabilizer in the expansive soil prepared CLSM mix design, the samples could not retain 50% or more of their original UCS after 14 w-d cycles. This proves that using 18% by weight Portland Cement (TY I/II) as a binder material for CLSM prepared using Eagle Ford native soil is ineffective under harsh testing conditions as used in the laboratory which is not necessarily the field conditions.

5. The leachate studies indicated that the increase in sulfate concentration in CLSM samples increased the calcium and cement loss during durability wetting-drying cycles. This increase in cement loss with the increase in sulfate concentration in CLSM samples could be the reason for the initial strength increase in the samples due to the lack of Ettringite formation in the sample. However, with the increase in durability cycles, it was observed that high sulfate concentration in CLSM mixes lead to binder loss upto 35% as exhibited by the EF sample with 20,000 ppm soluble sulfate. This clearly supports the loss of strength of these samples with durability w-d cycles.

5.3 Recommendations

In order to enhance the knowledge and understanding of CLSM mixes with sulfate spiked expansive soils, and to develop a sustainable and long term durable CLSM mix design for civil engineering projects such as pipeline construction or utility bedding, the following recommendations are made:

1. Studies with higher quantity of Portland cement (Type I/II) as a stabilizer for the Eagle Ford CLSM has to be conducted to establish an acceptable UCS value in order to use eagle Ford native soil CLSM for IPL project bedding or backfill purposes. An acceptable UCS value would be strength retention of 50% or more than the initial strength of the CLSM mix after durability cycles.
2. It is recommended to conduct studies on Eagle Ford CLSM with other chemical stabilizing alternative besides Portland cement (Type I/II). Studies have shown that use of lime mellowing (Talluri, 2013) and Type V Cement (Puppala et al.,2004) have been used in stabilizing expansive soils with high soluble sulfate concentrations.
3. It is recommended that further research studies be conducted on CL soil or lean clay such as that from Ozan formation in preparation of CLSM mixes. Ozan formation is one of the geological formations that falls under the IPL alignment and has elevated levels of soluble sulfate concentrations (Thomey, 2013). The study would help in understanding the effects of elevated levels of sulfates on CLSM from both fat and lean clay.
4. It is recommended to perform chemical studies on tested CLSM samples to understand the formation of Ettringite in the CLSM samples or to understand the “increase in strength with increase in sulfate content” phenomenon exhibited by native expansive soil CLSMs.

APPENDIX A

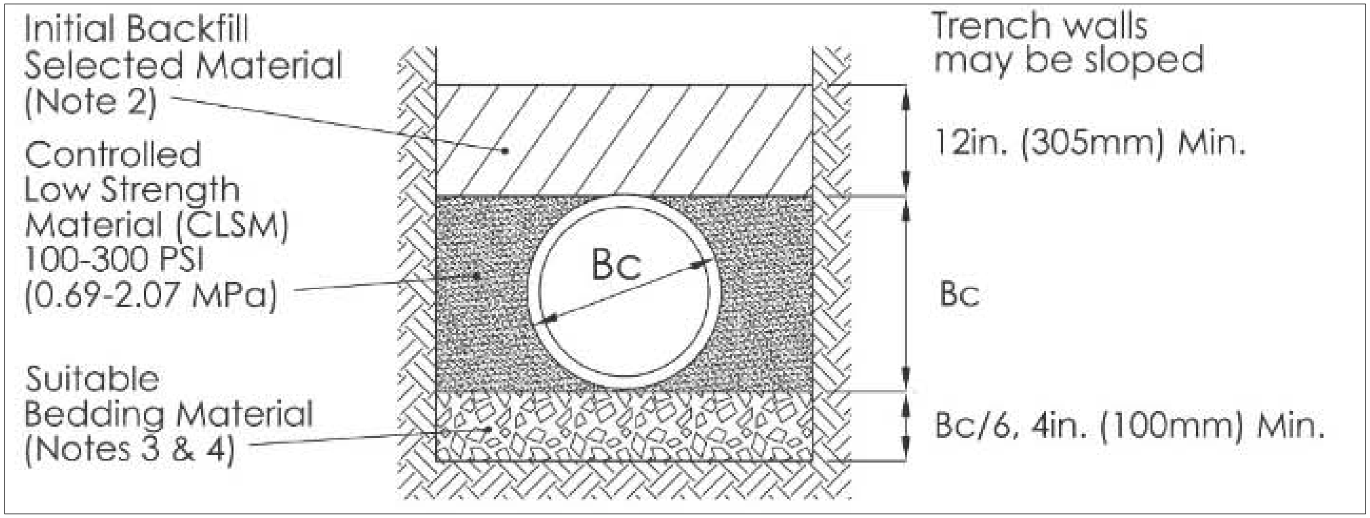


Figure A Pipe Cross-Section using CLSM as bedding material (Boschert and Butler, 2013)

APPENDIX B

Figure B CLSM Mix Being Poured as Pipeline Bedding Material for IPL Project

REFERENCES

1. ACI (2005). “Controlled Low-Strength Material.” ACI Publication 229R-99, Reapproved 2005. *American Concrete Institute*, Farmington Hills, Mich., 1999, 15 pp.
2. Adams, A.G., Dukes, O.M., Cerato, A.B., and Miller, A.G. (2008). “Sulfate Induced Heave in Oklahoma Soils due to Lime Stabilization”. *Proceedings of Sessions of GeoCongress,* Geotechnical Special Publication, (179): 444-451.
3. American Society for Testing and Materials (ASTM) D559-03. “Standard Test Methods for Wetting and Drying Compacted Soil-Cement Mixtures.”
4. Azam, S. (2003). “Influence of Mineralogy on Swelling and Consolidation of Soils in eastern Saudi Arabia.” *Canadian Geotechnical Journal*. 40 (5), 964-975.
5. Barenberg, E.J., (1970). “Evaluation of Remolded Field Samples of Lime-Cement-Fly Ash-Aggregate Mixture.” *Highway Research Record No. 315*, *Highway Research Board*, Washington, DC,: 112-121.
6. Bell, F.G. (1989). “Lime Stabilization of Clay Soils.” Bulletin of the International Association of Engineering Geology, 39 (1),: 67-74.
7. Berger, E., Little, D.N. and Graves, R. (2001). “Technical Memorandum: Guidelines for Stabilization of Soils Containing Sulfates”. <www.lime.org/publications.html>.
8. Bernard, R. D. and R. S. Tansley (1981). “Laboratory Testing Program for development of a Lean Mix Backfill Specification.”, *Department of Housing and Urban Development*, Washington, DC,: 31.
9. Boschert, J. and Butler, J. (2013). “CLSM as a Pipe Bedding: Computing Predicted Load using the Modified Marston Equation.” *ASCE Pipelines Conference,* 2013: pp. 1201-1212.
10. Bugge, W. A. and Bartelsmeyer, R. R. (1966) ‘‘Soil Stabilization with Portland Cement.’’ *Highway Research Board, Bulletin* 292, National Research Council, Washington, D.C., : 1–15.
11. Burkart, B., Goss, G. C. and Kern, J. P. (1999). “The Role of Gypsum in Production of Sulfate-Induced Deformation of Lime-Stabilized Soils”. *Environmental and Engineering Geoscience*, V (2) : 173-187.
12. Cerato A.B., M. G. A., Madden M.E., Snethen D.R. (2009). "Calcium-Based Stabilizer Induced Heave in Oklahoma Sulfate-Bearing Soils." *Oklahoma Department of Transportation*, Oklahoma City, Oklahoma: 7.

13. Chakkrit, S., Puppala, A. J., Chikyala, V., Saride, S., and Hoyos, L.R. (2008). “Combined Lime and Cement Treatment of Expansive Soils with Low to Medium Soluble Sulfate Levels”. *Geocongress 2008.*
14. Chen, F. H. (1988). “Foundations on Expansive Soils”, *Developments in geotechnical engineering,.*12: p12.
15. Chen, F. H. (1999). “Soil Engineering: Testing, Design, and Remediation.” Edited by M.D. Morris, (1999). CRC Press, 74-75.
16. Chittoori, B. C. S. (2008). “Clay mineralogy effects on long-term performance of chemically treated expansive clays.” Ph.D. dissertation, University of Texas at Arlington, TX, p 326; Accession Order No. AAT 3339038.
17. Cohen, M.D. (1983). “Theories of Expansion in Sulfoaluminate – Type Expansive Cements: Schools of Thought.” Cement and Concrete Research, 13,: 809-818.
18. Dempsey, B. J., and Thompson, M. R. (1968). ‘‘Durability Properties of Lime-Soil Mixtures.’’ *Highway Research Record*, Washington, D.C. 235: 61–75.
19. DePuy, G. W. (1994). “Chemical Resistance of Concrete: Significance of Tests and Properties of Concrete and Concrete Making Materials.” *ASTM 169C*,: 263-281.
20. Dermatas, D. (1995). “Ettringite-induced Swelling in Soils: State-of-the-art.” *Applied Mechanics Rev*., 48: 659-673.
21. Folliard, K. J., Trejo, David, and S. A. Sabol (1999). “Controlled Low-Strength Material for Backfill, Utility Bedding, Void Fill, and Bridge Approaches”, *NCHRP 24-12 Phase I Interim Report*, Delaware Transportation Institute, University of Delaware, Newark.
22. Folliard, K. J., Trejo, David, Sabol, S. A., and Leshchinsky, D., (2008). “Development of a Recommended Practice for Use of Controlled Low-Strength Material in Highway Construction”, *NCHRP 597 Report*: 5-39.
23. Gress, D. (1996). “The Effect of Freeze-Thaw and Frost Heaving on Flowable Fill.” UNH Civil Engineering #1096-1 for New Hampshire Department of Transportation, University of New Hampshire,: 56.
24. Harris, P., T. Scullion, S. Sebesta, and G. Carlos, (2003), “Measuring Sulfate in Subgrade Soil - Difficulties and Triumphs.” *Transportation Research Board:* Record, No. 1837, National Research Council, Washington, D.C., pp. 3-11.
25. Harris, J. P., Sebesta, S., and Scullion, T. (2004). “Hydrated lime stabilization of sulfatebearing vertisols in Texas.” *Transportation Research Board*. Washington, D.C.,: 31–39.
26. Hausmann, M. R. (1990). “Engineering principles of ground modification,” *McGraw-Hill*, New York.:632.

27. Hawkins, A. B. (1998). “Engineering Significance of Ground Sulfates.” *Geotechnical Site Characterization*,: 685-692
28. Holtz, R.D. and Kovacs, W.D. (1981) “An introduction to geotechnical engineering.” *Prentice-Hall*, New Jersey.
29. Horiguchi, T., Okumura, H., and Saeki, N. (2001). “Optimization of CLSM mixture proportions with combination of clinker ash and fly ash”; ACI Special Publication SP-199, American Concrete Institute, Farmington Hills, MI, 2001, Editor: V. M. Malhotra, pp. 307-325.
30. Horiguchi, T., Okumura, H., and Saeki, N. (2001). “Durability of CLSM with used foundry sand, bottom ash, and fly ash in cold regions”; ACI Special Publication SP-200, American Concrete Institute, Farmington Hills, MI, 2001, Editor: V. M. Malhotra, pp. 325-347.
31. Howard, A. and Bowles, L. (2008). " Flowable Fill and the Ridges Basin Inlet Conduit" *ASCE Pipelines 2008*: 1-11.
32. 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.
33. Hunter, D. (1988). “Lime-Induced Heave in Sulfate-Bearing Clay Soils,” *Journal of Geotechnical Engineering*, ASCE, Vol. 114, p150-167.
34. Integrated Pipeline (IPL). *Tarrant Regional Water District.* <http://www.iplproject.com> (Jun. 15, 2013).
35. Jones, L. D. and Jefferson, I. (2012). “Expansive soils”. *ICE Publishing* : 413-441
36. Kamei, T., Ahmed, A., and Ugai, K. (2012). “The performance of soft clay soil stabilized with recycled gypsum in wet Environment.” *In the Proceedings of 2011 Pan-Am CGS Int. Geotechnical Conference*. GeoCongress 2012: pp. 1026-1035.
37. Karduri, R. K. R. (2011). “Sustainable Reutilization of Excavated Trench Material.” M.S. Thesis. Thesis, Dept. of Civil Engineering, University of Texas at Arlington, Arlington, TX, AAT No. 1506300: p116.
38. Kota, P. B. V. S., Hazlett, D., and Perrin, L. (1996). ‘‘Sulfate-Bearing Soils: Problems with calcium based stabilizers.’’ Transportation Research Record 1546, Transportation Research Board, Washington, D.C.
39. Krell, W. C. (1989). “Flowable Fly Ash.” Transportation Research Record 1234, TRB, *National Research Council*, Washington, DC,: 8–12.
40. Ksaibati, K., and Huntington, George S. (1996). ‘‘Evaluation of Sulfate Expansion in Soil-Cements.’’ *Geotechnical Testing Journal*, 19 (3): 269–276.

41. Lad, P. P. (2012). “Development of a New Device to Evaluate Stabilization Durability of Expansive Soils by Addressing Wetting/Drying and Leachate Issues.” M.S. Thesis. University of Texas at Arlington, TX,: 326.
42. Little, D. N., Nair, S., and Herbert, B. (2010). “Addressing Sulfate-Induced Heave in Lime Treated Soils”. *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE: 110-118.
43. Manz, Lorraine. (2011). “Frost Heave.” *Geo News*, 32 (2): 18-24.
44. McCallister, L.D. (1990). “The Effects of Leaching on Lime-Treated Expansive Clay.” Ph.D. dissertation, University of Texas at Arlington, TX,: 452
45. Mehta, P. K., and Klein, A. (1966). “Investigation of hydration products in the system 4CaO.3Al2O3.SO3.CaSO4.CaO.H2O”. *Highway Research Board*, Special report no.90,: 328-352.
46. Mehta, P. K. (1973). “Mechanism of Expansion Associated with Ettringite Formation.” *Cement and Concrete Research*, 3(1),:1–6.
47. Mehta, P. K., and Wang, S. (1982). Cement Concrete Research, 12, :121-122.
48. Mitchell, J. K. (1986). “Practical problems from surprising soil behavior,” *ASCE Journal*
49. 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.
50. Nantung, T. E. (1993). Design Criteria for Controlled Low Strength Materials. Ph.D. Dissertation, Purdue University, 345 p.
51. Nantung, T. E. and Scholer, C.F. (1994). “Freezing and Thawing Durability and Early Set and Strength Development of CLSM.” Special Publication ACI . Vol. 150, p 53-86.
52. Najafi, F.T., Tia, M., Lovencin, W., Javed, A., Chaudry, H., and Abbas, A. (2004). “Use of Accelerated Flowable Fill in Pavement Sections.” Florida Department of Transportation, State Project no. 872. Contract No. BC354-53, Research Management Center, Tallahassee, FL.
53. Nelson, J.D. and Miller, D.J. (1992). “Expansive soils: problems and practice in foundation and pavement engineering.” *John Wiley and Sons, inc*.,New York.
of Geotechnical Engineering, 112 (3),: 259-289.
54. Ogawa, K., and Roy, D. M. (1982). “C4A3S hydration ettringite formation and its expansion mechanism. II: Microstructural observation of expansion.” Cement and Concrete Research, 12 , 101–109.
55. Olive, W.W., Chleborad, A.F., Frahme, C.W., et.al.(1989). “Swelling Clays Map Of The Conterminous United States." United States Geological Survey (USGS).

56. Pedarla, A. (2009). “Durability Studies on Stabilization Effectiveness of Soils Containing Different Fractions of Montmorillonite.” M.S.Thesis. Retrieved from ProQuest Dissertations and Theses.
57. Pedarla, A., Chittoori, S., and Puppala, A. (2011). “Influence of Mineralogy and elasticity Index on Stabilization Effectiveness of Expansive Clays.” *Journal of Transportation Research Board*, 2212, : 91-99.
58. Perrin, L. (1992). "Expansion of Lime-treated Clays Containing Sulfates." *SCE Expansive Soils Research Council*, 1: 409-414.
59. 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.
60. Pierce, C.E. and Blackwell, M.C. (1999). “ Potential of Scrap Tire Rubber as Lightweight Aggregate in Flowable Fill.” University of South Carolina, Department of Civil and Environmental Engineering, Columbia, SC. p 30.
61. 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.
62. Puppala, A. J., Viyanant, C., Kruzic, A. P., and Perrin, L., (2002). “Evaluation of a Modified Soluble Sulfate Determination Method for Fine-Grained Cohesive Soils”, *Geotechnical Testing Journal*, 25(1),:85–94.
63. Puppala, A. J., Wattanasanticharoen, E. and Punthutaecha, K. (2003). “Experimental Evaluations of Stabilization Methods for Sulphate-rich Expansive Soils.” *Ground Improvement*, 7 (1): 25-35.
64. 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*,130 (4),: 391-402
65. Puppala, A.J., Intharasombat, N., and Vempati,R.K.(2005). “Experimental Studies on Ettringite - Induced Heaving in Soils”. *Journal of Geotechnical and Geoenvironmental Engineering*, 131(3),: 325-337.
66. Puppala, A.J., et al., (2010). “Forensic Investigations to Evaluate Sulfate – Induced Heave Attack on a Shotcrete Tunnel Liner,” *Journal of Materials in Civil Engineering*, ASCE, 22: 914-922.

67. Puppala, A.J., Naga S. Talluri., Bhaskar S. Chittoori and Ahmed Gaily. (2012). "Lessons Learned from Sulfate Induced Heaving Studies in Chemically Treated Soils." *Proceedings of the International Conference on Ground Improvement and Ground Control*. Research Publishing, 1,: 85-98.

68. Raavi, Anil Kumar. (2012). "Design of Controlled Low Strength Material for bedding and backfilling using high plastic clay" M.S. Thesis, Dept. of Civil Engineering, University of Texas at Arlington, Arlington, TX, :244

69. Richard J. Hammerberg. "Design and construction protocols help mitigate pavement distress caused by high traffic loads and expansive soils*." Paving the way to Better Roads.*<http://www.cenews.com/print-magazinearticle-paving_the_way_to_better_ roads- 4790. html>. (Mar. 5, 2013).

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

71. Rogers, J. David, Robert Olshansky, and Robert B. Rogers. (1993). "Damage to foundations from expansive soils." *Claims People* 3 (4): 1-4.

72. Rollings R.S., Burkes, J. P., Rollings M.P. (1999). "Sulfate Attack on Cement Stabilized Sand Journal of Geotechnical and Geoenvironmental Engineering, 125 (5),: 364-372.

73. Schmitz, M.E., Parsons, R.L., Ramirez, G., and Zhao, Y. (2004). "Use of Controlled Low-Strength Material as Abutment Backfill." *Kansas Department of Transportation.* Report No. K-TRAN:KU-02-06, p76.

74. Sridharan, A. and Prakash, K. (2000). "Classification procedures for expansive soils." *Proceedings of the ICE - Geotechnical Engineering*, 143 (4): 235 –240.

75. Talluri, N., Gaily, A., Puppala, A. J, and Chittoori, B. (2012), "A Comparative Study of Soluble Sulfate Measurement Techniques", *GeoCongress*, Oakland, GSP 225,: 3372-3381.

76. Talluri, N., et al. (2013). "Stabilization of High Sulfate Soils by Extended Mellowing." *Transportation Research Board Annual Meeting*, Washington, D.C., U.S.A.

77. Texas Department of Transportation (TxDOT). (2005). "Guidelines for Treatment of Sulfate-Rich Soils and Bases in Pavement Structures." *Construction Division, Materials and Pavements Section, Geotechnical, Soils and Aggregates Branch.* p. 30.

78. Thomey, J. D. (2013). "Quantification and Geostastical Mapping of Soluble Sulfates in Soils along a Pipeline Alignment." M.S. Thesis, University of Texas at Arlington, Arlington, TX: 145p.

79. Thompson, M. R. and Dempsey, B. J. (1976). "Evaluation of freeze-thaw durability of stabilized materials." *Transportation Research Record*, 612, 62-70.
80. Trejo D., F. K. J., Du L. (2004). "Sustainable development using controlled low-strength material. K. Wang (Ed.)." *Proceedings of international workshop on sustainable development and concrete technology, Beijing, China*, Iowa State University: 231-250.
81. United Stated Census Bureau (USCB). (2010). <http://www.census.gov/popclock/> (July. 10, 2013).
82. Vanga, D.P.R. (2013). “Durability Studies of Controlled Low Strength Material Using Native Soils as Fine Aggregates.” M.S. Thesis, Dept. of Civil Engineering, University of Texas at Arlington, Arlington, TX, p 129.
83. Viyanant, C. (2000). “Laboratory Evaluation of Sulfate Heaving Mechanisms in Artificial Kaolinite Soil”. M.S. Thesis, University of Texas at Arlington, Arlington, Texas.
84. Wilson, J. (1999). “Flowable Fill as Backfill for Bridge Abutments.” Wisconsin Department of Transportation, Report No. WI-16-99, DTID/BHC/Pavements Section/Technology Advancement Unit, Madison, WI. p 30.

BIOGRAPHICAL INFORMATION

Sadikshya Poudel was born in Kathmandu, Nepal and spent most of her early educational career in Nepal. After completing high school she attended Wesleyan College in Macon, GA to pursue a bachelor's degree. Since the college offered only a pre-engineering degree, she transferred to the University of Texas at Arlington in Fall of 2005 to complete her bachelor's degree in Civil Engineering. She graduated from the University of Texas at Arlington and received her Bachelor's of Science in Civil Engineering in December, 2008. After graduation, she worked in the industry for approximately two years before returning for graduate studies. In the Spring of 2012, she was accepted and enrolled into the University of Texas at Arlington's graduate program. Sadikshya will receive her Master of Science in Civil Engineering (Geotechnical) in December, 2013. While pursuing her Master's degree, Sadikshya was involved in sulfate research studies for the IPL (Integrated Pipeline) project funded by Tarrant Regional Water District (TRWD).