PERFORMANCE STUDIES ON RIGID PAVEMENT SECTIONS BUILT ON STABILIZED SULFATE SOILS

by

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ABSTRACT

PERFORMANCE STUDIES ON RIGID PAVEMENT SECTIONS BUILT ON STABILIZED SULFATE SOILS

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Soils of Southeast Arlington are highly expansive and rich in sulfates. They undergo sulfate induced heaving when traditional calcium based stabilizers are used for soil stabilization (Puppala et al. 1998). Traditional stabilizers do not provide effective solution since they are known to induce heaving, termed in the literature as sulfate induced heaving (Hunter, 1988). Both swelling and softening of these soils rich in sulfates induce considerable damage to overlying pavement infrastructure.

Typically, high sulfate soils treated with calcium based stabilizers form ettringite mineral. Ettringite undergoes heaving when hydrated (Hunter, 1988; Puppala et al. 2001). Since this sulfate-induced heave is caused by soil stabilization with

calcium-based stabilizers, it is regarded as a manmade or post treatment expansive soil problem (Puppala et al. 2005). These problems are further aggravated by seasonal temperature disparity typical to North Texas and may eventually damage the pavement (Chen, 1988; Nelson and Miller, 1992).

Constant maintenance problems on the existing pavement infrastructure resulted in the initiation of a research study to explore and investigate new methods for subgrade stabilization. The study has been conducted in University of Texas at Arlington as a part of research for City of Arlington. The research work conducted aims at selection of an ideal stabilization method or methods for stabilizing sulfate rich soils of Southeast Arlington.

This research study was conducted to evaluate the stabilization potentials of Sulfate Resistant Type V Cement, Class F Fly ash with Type V Cement, Ground Granulated Blast Furnace Slag, Lime with Polypropylene Fibers and Lime. Rigid pavement test sections were constructed on the five sections of stabilized subgrade soils and these sections were instrumented and monitored for twenty six months. Instrumentation data obtained from strain gauges and pressure cells as well as elevation surveys were analyzed to address any heave related movements and load carrying potentials of treated subgrades. DCP tests were also conducted to monitor the strength characteristics of stabilized soils. In addition, chemical tests and mineralogical tests were conducted on the stabilized samples collected from the test site to address the formation of Ettringite mineral.

Overall, based on the long term analysis, Type V Cement-Fly ash treatments proved to be the most effective treatment for stabilizing sulfate bearing soils with no heave distress was followed by Type V Cement and GGBFS treatments.

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CHAPTER 1

INTRODUCTION

1.1 Introduction

Natural expansive soils are found in several countries across the world, and are present in majority of the states in the United States (Chen, 1988). Subgrade soils in North Texas especially in Southeast Arlington and Dallas and Forth Worth locations are recognized to be problematic expansive soils as they demonstrate low strength, high swell and shrinkage characteristics (Kota et al., 1996; Chen, 2005). Expansive soils generally undergo large volumetric changes due to moisture changes from seasonal variations. These volumetric movements' instigate cracking in subgrades which in turn result in the swelling of soil when the soil absorbs water consequently (Nelson and Miller, 1992). Both low strength and volumetric movements weaken subgrade and cause structural distress in the pavements leading to the development of cracks and differential heave movements. Maintenance and repair costs of these distressed pavements are quite high (Nelson and Miller, 1992).

A number of control methods are extensively used in the field to control heave distress in expansive soils which include treatment with calcium-based stabilizers, non-calcium-based stabilizers, asphalt-stabilization, and by geo-synthetic reinforcement (Kota et al., 1996). Soil stabilization is known as an alteration of soil properties to meet particular engineering requirements and among these stabilization methods, calcium based stabilizers like lime; cement and fly ash are most commonly used. Calcium based stabilizers are normally used, as it enhances the soil strength, reduces the plasticity index (PI) and is cost effective. The reductions in plasticity index have been used to extend the design life of structures built over expansive clayey soils (Kota et al., 1996).

1.2 Problem Statement

Soils of Southeast Arlington are known to be highly expansive and rich in sulfates. Calcium-based stabilizers, including lime and cement, have been used to increase strength and to decrease plasticity index and swell and shrinkage strain potentials of expansive soils (Hausmann, 1990). Several studies have shown that calcium-based stabilizer treatments of natural expansive soils rich with sulfates may lead to a new heave distress problem instead of mitigating it (Mitchell, 1986; Hunter, 1988; Mitchell and Dermatas, 1992; Petry, 1994; Kota et al., 1996; Puppala et al., 1999; Rollings et al., 1999). Sulfate-induced heave is primarily attributed to the presence of sulfates in natural expansive soils and usually occurs when lime or cement treatments are used for stabilizing these soils (Hunter, 1988; Mitchell and Dermatas, 1990; Petry and Little, 1992). Reaction of calcium components of stabilizers with free alumina and soluble sulfates in soils at a basic environment (pH between 11 and 13) lead to the formation of an expansive sulfate mineral, to form ettringite mineral (Hunter, 1988). Ettringite, a weak sulfate mineral, will undergo significant heaving when subjected to hydration and this mineral will continue to form as long as there are sufficient amounts of reactants present in the soil (Puppala et al., 2005). These problems are further supplemented by seasonal temperature disparity typical to North Texas. Therefore, traditional calcium-based stabilizers do not provide effective solution since they are known to induce sulfate induced heaving (Hunter, 1988).

The City of Arlington budgets large amounts of funds for the annual maintenance and repair costs of distressed pavements. Therefore, it is necessary to appraise and explore new and alternate stabilization methods with the aim of constructing stronger and stable subgrades with negligible heave distress. Development of such methods will not only reduce the maintenance cost but also improves the riding comforts for the passengers. With this aim, City of Arlington funded a research study in University of Texas at Arlington to evaluate four novel stabilization methods in laboratory and field conditions and select an ideal stabilization method or methods for stabilizing sulfate rich soils of Southeast Arlington. Based on laboratory evaluations and literature reviews, the following four stabilization methods were considered for evaluation:

- Sulfate Resistant Type V Cement
- Class F Fly ash with Type V Cement
- Ground Granulated Blast Furnace Slag
- Lime with Polypropylene Fibers

Several laboratory tests were conducted by (Wattanasanticharoen, 2000; Chavva, 2002 and Ramakrishna, 2002) in order to evaluate the basic and engineering properties of field subgrade soil and the selected stabilizers.

Although these stabilizers provided demonstrated good performance in the laboratory, it was required to assess these stabilizers in real field conditions. Field studies are essential since the soil in natural field conditions undergo true moisture and temperature fluctuations which may affect the stabilization mechanisms. Performance assessment of stabilization by sophisticated field instrumentation studies provides an accurate evaluation of the performance of stabilizers. Moreover, field stabilization studies using sensor-instrumented pavements will provide the efficiency of each stabilizer in controlling pavement distress such as differential heave, rutting and pavement cracking. Mohan (2002) and Pillappa (2005) designed and developed appropriate field instrumentation to evaluate treated subgrade soils. Strain gauges and pressure cells were installed and data was collected in order to measure compressibility strain potentials and load carrying potentials of stabilized subgrades. In addition to pavement instrumentation, elevation surveys and DCP tests were also conducted. Elevation surveys were performed in order to evaluate the heave and other types of soil related movements including erosions of stabilizer treated soils. DCP tests performed to analyze the in-situ strength and modulii properties of treated subgrade soils. Chemical tests and mineralogical tests were also conducted to identify the presence of ettringite mineral which is the sulfate heave source mineral in treated soils.

1.3 Research Objectives

The main objectives of this research project was to evaluate these four novel stabilization methods in field conditions in order to select an ideal stabilization method or methods for stabilizing sulfate rich soils and minimize sulfate induced heave distress in Southeast Arlington. In order to achieve these objectives following responsibilities were carried out during the research study:

- Acquisition of load and strain data from the sensors on a weekly basis.

- Carry out elevation surveys and monitor heave related movements of stabilized pavement sections on a regular basis.
- Visual field inspection for any cracks or any deterioration in the stabilized pavements.
- Conduct dynamic cone penetration (DCP) test to acquire in-situ strength of the treated soils.
- Perform chemical tests namely pH test and soluble sulfate test to determine if the soil conditions are prone to sulfate heave due to ettringite mineral formation.
- Perform mineralogical studies namely, X-ray diffraction (XRD) analysis and scanning electron microscopic (SEM) studies to identify the formation of Ettringite mineral if any.

1.4 Thesis Report Organization

This thesis report is composed of six chapters: introduction (chapter 1), theoretical background and literature review (chapter 2), selection of stabilizers for Harwood road soil (chapter 3), field and laboratory testing program (chapter 4), results and discussion (chapter 5), summary , conclusion and recommendation (chapter 6).

Chapter 1 provides the introduction, problem statement, research objectives and thesis organization. The studies conducted to meet the research objectives are briefly mentioned in this chapter

Chapter 2 provides the background of the soil type, characteristics of stabilizers used in this research and case reviews on pavement distress. It also includes summarization of the importance of instrumentation, different instrumentation methodologies based on the application areas and case reviews involving instrumentation and their findings.

Chapter 3 presents the laboratory tests conducted to determine the basic soil properties of the test site, selection of stabilizers, sample preparation, test methods, basic and engineering properties of stabilized soil of Southeast Arlington.

Chapter 4 provides a complete description of field studies, chemical tests and mineralogical tests conducted to assess the performance of stabilizers. The details of instrumentation design, installation of sensors, data collection procedures, elevation survey and field DCP test details are also discussed.

Chapter 5 includes the results and analyses of field studies, chemical tests and mineralogical test conducted in this research.

Chapter 6 provides the summary, conclusions of the research study results. Some recommendations of stabilizers based on the study results are also included.

CHAPTER 2

THEORETICAL BACKGROUND AND LITERATURE REVIEW

2.1 Introduction

Background information and literature review presented in this chapter was collected from libraries, journal resources, and research reports. An introduction to expansive soils and sulfate-induced heave is first given, followed by a detailed description of problems pertaining to sulfate induced heave with few case studies. Comprehensive descriptions of problems associated with application of calcium based stabilizers for the treatment of sulfate rich soils are described. The natural process of Ettringite formation in soils and possible heave mechanisms, which cause distress to structures are also explained. The later part of the chapter explains the details of pavement instrumentation and their advantages, commonly used sensors in monitoring geotechnical earth structures followed by a few case studies on current sensor

2.2 Problematic Expansive Soils

Problematic soils are those that can cause distress to the structures above them which includes soft soils, expansive soils, collapsible soils and active clays. The origin and distribution of expansive materials in the United States are generally a function of geologic history, sedimentation and present local climatic conditions. Expansive soils are located in many states in United States and are estimated to occupy one-fifth of the whole continental area particularly in the western, central, and southeastern United

States. Volume change resulting from moisture variations in expansive soil sub grades is estimated to cause damage to streets and highways in excess of $ 1.1 billion annually (Jones and Holts, 1973). Expansive soils causes more damage to structures, particularly pavements and light buildings, than any other natural hazards like earthquake or floods (Jones and Holts, 1973). The study conducted by National Science Foundation on "Building Losses and Natural Hazards" in 1978, noted that the expansive soils occupied the second place among the most destructive natural hazards that damage the infrastructure in United States (Wiggins, 1978). The current annual cost estimates to repair buildings, roads, and other structures built on expansive soils are expected to be more than $10 billion (Steinberg, 1998). The state of Texas has the most extensive network of surface-treated pavements in the nation. This network has suffered from detrimental effects of expansive soils in subgrades for decades (Petry and Little, 2002). Much has been learnt about their behavior over the past 60 years, and relatively successful methods have been developed to modify and stabilize them.

Several control methods are widely used in the field to control heave distress in expansive soils. These control methods include stabilization with calcium-based stabilizers, non-calcium-based stabilizers, asphalt-stabilization, geo-synthetic reinforcements and compaction of the subgrade. Soil Stabilization is known as a modification of soil properties to meet specific engineering requirements. Among these stabilization methods, calcium based stabilizers like cement; lime and fly ash are most commonly used. Calcium based stabilizers are commonly used as it increases the soil strength, decreases the plasticity index (PI) and is cost effective. The reductions in plasticity index have been used to extend the design life of structures built over expansive clayey soils (Kota et al., 1996). Lower the PI, lesser is the amount of heaving.

2.3 Mechanism of Sulfate Heave in Soils

Sulfate-bearing expansive soils are found in several states in the U.S. particularly in the southwestern and western states. Many states including Kansas, Oklahoma, Nevada, Arizona, New Jersey, Texas, Colorado, and California reported sulfate-induced heave as one of the major distresses that damaged embankment and pavement structures (Hunter, 1988; Perrin, 1992; Dermatas, 1995; Puppala et al., 1999).

Sulfates are introduced into the soils in many different forms such as acid rain, construction water, underground water flow, or moisture percolation due to evapo-transpiration process (Dermatas, 1992). The sulfates are present in natural soils in various forms such as gypsum or calcium sulfate, sodium sulfate, and magnesium sulfate (Puppala et al. 2003). The most common sulfate mineral present in soils is Gypsum ($CaSO_4.2H_2O$) because of its relatively low solubility (2.6 gm/L) level when compared to both sodium sulfate Na_2SO_4 (408 gm/L) and magnesium sulfate or $MgSO_4$ (260 gm/L) (Puppala et al., 2003).

Mitchell's Terzaghi lecture was the first time sulfate induced heave received national recognition (Mitchell, 1984) He used a parking lot in Las Vegas that experienced heave 2 years after construction as an example to highlight the importance of physicochemical and biological changes in soil mechanics and reported ettringite and thaumasite were the cause of failure. Hunter (1988) explained many of the hysicochemical details concerning sulfate heave.

Hunter's experiments determined that four ingredients are necessary for heaving to occur. The ingredients include: (1) clay minerals (aluminate source), (2) calcium-based stabilizers (lime or cement), (3) sulfate or sulfide minerals or ions, and (4) copious amounts of water are needed to generate sulfate heave at 77ºF (25ºC), with sulfate ions being the key ingredient.

Several studies show that when soils containing calcium sulfates and other sulfate minerals are stabilized with calcium-based stabilizers such as lime or cement to improve the soil properties, the sulfate minerals appeared in these soils react with calcium component of the stabilizer and free reactive alumina of soils to form highly expansive crystalline minerals namely, ettringite and thaumasite (Sherwood, 1962; Mehta and Wong, 1982; Mitchell, 1986; Hunter, 1988). The sulfates in the soils tend to react with the free alumina (possibly in amorphous structure) liberated from the clay particles and calcium component from the stabilizers in order to form a combination series of calcium-aluminum-sulfate hydrate compounds (Mitchell and Dermatas, 1995) which lead to the formation of ettringite minerals. When subject to hydration, ettringite crystals ($Ca_6\ [Al\ (OH)\ _6]_2*(SO_4)_3*26H_2O$) has the potential to expand twice or three times of their original sizes. The chemical structure of ettringite crystals are hexagonal prisms and are often seen in elongated form with different shapes: needle-like, lath-like or rod-like depending on the time and pH conditions during the formation period. Figures 2.1 illustrate the structure of ettringite crystals.

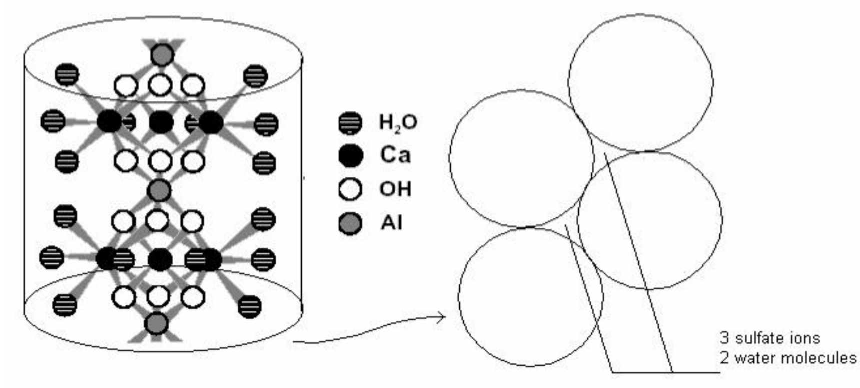


Figure 2.1 Structure of Ettringite Column (Intharasombat, 2003)

When the temperature of this system reaches less than 15°C and presence of soluble carbonate content, ettringite is transformed by a series of intermediate reactions to thaumasite mineral, $[Ca_3Si(OH)_6]_2(SO_4)(CO_3)_2{*}26H_2O$. Thaumasite crystal is very expansive when exposed to water hydration and its expansion potential is much higher than that of ettringite.

2.3.1 Case Reviews

Although reports of sulfate attack on stabilized materials are uncommon, when such attacks do occur they are highly destructive resulting in major repair or replacement costs.

2.3.1.1 Georgia Case

In spring 1992, unexpected bumps began to appear on Bush Road Pavement (Rollings et al., 1999) within six months of its construction. As the initial bumps began forming, additional bumps followed them. The bumps developed along the length of the road and were 3.1 m wide and 63 mm high. With the excavation of the bumps apparent expansion and cracking in the base course material was found. This was the cause for the formation of bumps on the pavement surface. The soil used in the cement stabilized base course was predominantly a sand material with some fines and was described as clayey sand. The base course material was stabilized with 5 to 6% Portland cement. Laboratory tests were conducted with the samples collected from the distressed area. The results revealed the presence of ettringite due to sulfate attack on cement stabilized base. It was concluded that the presence of ettringite was the cause of pavement distress. The sulfate attack of cement-stabilized section was highly destructive and the whole base course material had to be excavated and re-constructed.

2.3.1.2 Stewart Avenue Case

Among the heaving related case studies, the well-known documented case is the Stewart Avenue case in Las Vegas, Nevada. Stewart Avenue is a major east-west roadway through downtown Las Vegas, Nevada. The local soils mainly consist of clay minerals, and evaporate (Hunter, 1988). The road was constructed on thick basin-fill sediments and some parts of the road were placed over bedded gypsum deposits. The road was reconstructed and widened from two to four lanes in order to serve the increased daily traffic (Hunter, 1988). The designed pavement section consisted of 10 cm of asphalt concrete, 13 cm or 20 cm of aggregate base, and a 30 cm of lime-treated local soil. The soils were treated with quicklime at 4.5% by weight and cured for a minimum of 16 hours prior to field compaction. Within a period of six months after the completion of this pavement construction, pavement started exhibiting heaving related cracks. The heaving magnitude measured as high as 30 cm, which was almost the original thickness of lime-treated subbase (30 cm) (Hunter, 1988).

Areas of distress were investigated and the results showed that the cracks or damages were seen at the areas where excessive water was present and could gain access to the treated subbase. (Hunter, 1988) investigated and reported that the distressed site has soluble sulfates ranging from 700 ppm to as high as 43,500 ppm. Further some, due to the presence of soluble sulfates, Hunter in 1988 concluded that this heaving was due to the reactions between soluble sulfates and calcium based component in lime material. Major pavement rehabilitation was required within two years after the construction. Figure 2.2(a) and 2.2(b) illustrates the heave distress in Stewart Avenue. The total rehabilitation cost for this site was estimated close to $2.7 millions.

(a) (b)

Figure 2.2 Illustration of Heave Distress in Stewart Avenue (a) Heave (b) Rut (Hunter, 1988)

2.3.1.3 Dallas - Fort Worth Airport Case

Another example for heave distress due to soluble sulfates is the heave observed on Taxiway sections of the Dallas-Fort Worth International Airport, Texas. In May 1997, a localized heave distress was investigated on taxiway section P. The central taxiway was 18-in. rigid concrete section which was built on 4-in. to 12-in. lime treated base soil. Within a few months after the completion of the taxiway and paved shoulder construction, heave distress was noticed (Puppala et al., 1998). Heave related cracks were observed at several locations on the shoulder sections of the taxiway.

Swell and shrinkage characteristics of the treated soil were determined by conducting several laboratory tests. Vertical swell strain results ranged from 2% to 18%, and swell pressures ranged form 18.6 kPa to 63.4 kPa. Shrinkage strain bar test results varied from 3% to 11%. However, the plasticity index of the soils was around 10. Scanning electron micrograph (SEM) studies were conducted and the resulting images as presented in Figure 2.3 showed the presence of ettringite minerals in the soils.

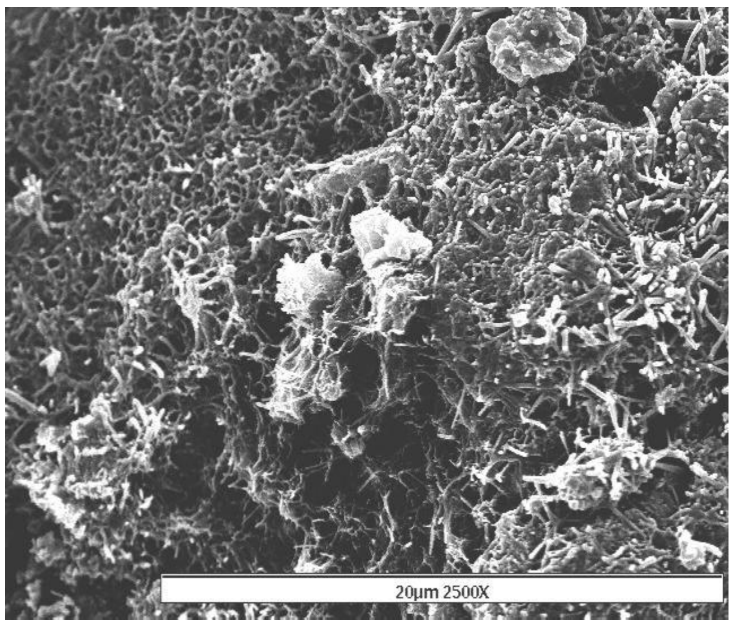


Figure 2.3 Presence of Ettringite Mineral in the treated soils of DFW Airport (Wattanasanasanticharoen, 2004)

Based on these test results, it was concluded that the heave distress was due to the reaction of soluble sulfates in the soils with lime material. It was also observed that there was no major distress was noted on concrete main taxiway sections but the heave mainly affected the asphalt pavement test sections placed on the shoulders.

From the case studies discussed above, we can conclude that that all damages reported were due to the presence of soluble sulfates in natural soils which in turn resulted in sulfate-induced heave. Therefore, it is important to understand the formation of ettringite and possible swell mechanisms in chemically treated sulfate bearing soils which can help in developing new and better methods for stabilizing the sulfate bearing soils.

In order to treat soft and expansive soils which are rich in sulfates using chemical stabilization, a comprehensive literature review was conducted at The University of Texas.

2.3.1.4 Green Oaks Boulevard Case

Green Oaks Boulevard is located in Southeast Arlington, Texas (Kota et al. 1996; Perrin, 1992). The pavement was constructed on 8 in. thick lime-treated subgrade, 2 in. thick HAMC base and 7 in. thick Portland cement concrete surface. Within seven days of lime treatment and compaction, the heaving behavior was observed. The heaving magnitude was about 1 to 10 in. above the initial stratum. The road had to be reconstructed by replacing lime treated section with HMAC base of 8 in. The repair cost was about $70,000.

2.3.1.5 Joe Pool Lake and Lloyd Park Case

Joe Pool Lake is located in southwest Dallas, Texas and Lloyd Park is located on the western arm of the lake. Perrin in 1992 reported the presence of large amount of quartz, gypsum and calcite in the lime treated sections of the pavements. Immediately after the placement of lime stabilized layer, the heaving distress was observed in the form of linear ridges or bumps in both longitudinal and transverse directions of the road. The magnitude of heave was as high as 4 in. with reference to the original pavement surface. The thickness of lime-stabilized layer was measured to be 7 in. to 8 in. after the heave occurred. The heavily damaged locations of the pavement occurred at the locations with poor drainage conditions and water ponding which accelerated the heaving process (Perrin, 1992). Perrin also reported the presence of soluble sulfates at the site which ranged from 2,000 to 9,000 ppm. The repair costs in this pavement were up to $70,000.

2.4 Sulfate Heave in Texas Soils

Sulfate induced heave distress have been observed for years. Recently it has become a more recognized serious problem in soil stabilization using calcium based stabilizers such as lime and cement throughout the state of Texas (Hunter, 1988). Figure 2.4 shows the sulfate concentrations in the state of Texas. Highways in Texas are now constructed much more rapidly than they were 20 years ago.

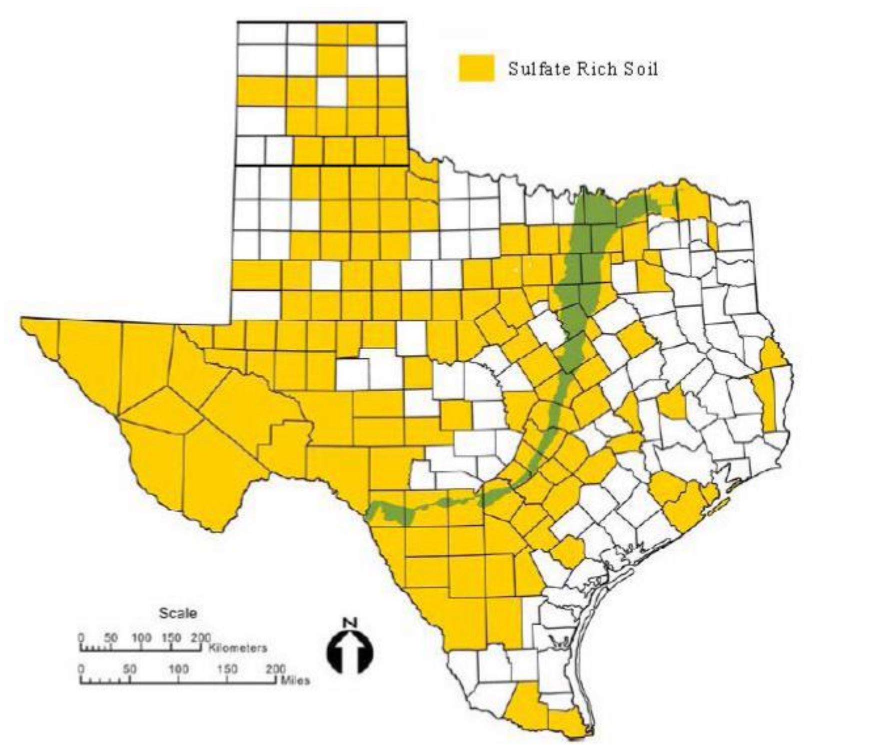


Figure 2.4 Texas Map Showing Sulfate Concentrations (Harris et al., 2004)

Several infrastructures in Texas have suffered severe damage due to expansive minerals formed from the reactions of calcium based materials used to stabilize sulfate bearing soils. This distress termed as sulfate-induced heave distress in the literature (Mitchell, 1986; Mitchell and Dermatas, 1992; Dermatas 1995; Hawkins, 1998) results in the poor performance of infrastructure and considerable reduction in the design life of structures. Remediation costs for projects that suffer sulfate induced heave damage are very high, because often the entire pavement may have to be removed and reconstructed (Kota et al., 1996).

Soils located in North Texas are highly expansive and rich in sulfate that induce sulfate-based distress to pavement when stabilized with lime or cement (Puppala et al., 2000) and (Kota et al., 1996). A research project was conducted at University of Texas at Arlington to address the above mentioned problem. In order to investigate the properties of soils in Southeast Arlington, Texas and select suitable stabilizers for their treatment, an experimental program was designed and conducted. As a part of the research four novel stabilization methods were assessed to come up with effective stabilization of sulfate rich expansive soils.

Harwood Road located in Southeast Arlington, Texas was selected as the test site in the present research. The soils of this test site are highly expansive and rich in sulfates. These soils were treated using four different types of stabilizers namely, Sulfate Resistant Type V Cement, Class F Fly ash with Type V Cement, Ground Granulated Blast Furnace Slag (GGBFS), and Lime mixed with Polypropylene Fibers. These stabilizers contain low amounts of calcium and are known to control sulfate heaving and therefore they can be efficient and suitable chemicals for sulfate soil stabilization (Kota et al., 1996; Vijayant, 2000).

Modified stabilization methods are required to stabilize soils of Southeast Arlington as they have low strength, high swelling and shrinkage problems with high amount of sulfates. The following sections provide description on four selected chemical stabilization methods and also cover the description of the mechanisms of reactions and their effectiveness in enhancing soil properties.

2.4.1 Sulfate Resistant Type V Cement

Cement has been used as a stabilizing material for soils since many years. Soils with cement additives are termed as “cement-stabilized” soils (Hausmann, 1990). Cement stabilization has been practiced over years due to its compatibility with a wide range of soil types, its effectiveness, availability and low cost. In 1935, the first cement-stabilized road was constructed in Jacksonville in South Carolina and it is still in usage condition (Das, 1941). Cement stabilization is a process in which the pulverized soil is mixed with cement and water. This process is followed by compaction to a required density and it is protected against moisture loss. The compacted soil is then cured for a specified time to enhance the soil properties. Mixing and compaction of the soil with cement and water allows interaction between soil particles.

During the process of cement stabilization, the initial cement hydration due to the release in the calcium ions, the plasticity characteristics reduces immediately (Bugge et al., 1961). The immediate results of the cement hydration are the reduction of plasticity of the soil. The ability to attract and hold water is known as plasticity in clay soils. As the hydration of the cement results in the release of calcium ions, and the clay particles have a charge deficiency, they are capable of cation exchange by means of replacement. Replacement consists of an exposed hydroxyl of the clay particle being replaced by another type of cation (Mitchell, 1993). Due to the electrical charge that the clay particles hold, they are attracted to one another. The attraction between these clay particles initiates the structure to flocculate. Flocculation of particles helps increase the overall strength and stability of the cement stabilized clay structure. Hydration is a primary process that supplies the compounds required for the secondary reaction.

The secondary process results in the cementation of contact points of flocculated clay particles. In the primary process, number of complex reactions takes place and material needed for cementation of the structure is formed. The following transformed compounds are the byproducts for the interaction between Portland cement and water (Kezdi, 1979). As shown in equation 2.1, tricalcium silicate with addition of water forms tobermorite gel (silicate hydrates) and calcium hydroxide. At the next stage, as shown in equation 2.2, bicalcium silicate with addition of water will form tobermorite (silicate hydrates) and calcium hydroxide.

$$2(3CaO.\ SiO_2) + 6H_2O – 3CaO.\ 2SiO_2.\ 3H_2O + 3Ca\ (OH)_2 \qquad (2.1)$$

(Tricalcium Silicate with addition of water will form tobermorite gel and calcium hydroxide)

$$2(2CaO.\ SiO_2) + 46H_2O = 3CaO.\ 2SiO_2.\ 3H_2O + Ca\ (OH)_2 \qquad (2.2)$$

(Bicalcium silicate with the addition of water will form tobermorite and calcium hydroxide)

$$4CaO.Al_2O_3.Fe_2O_3+10H_2O+2Ca(OH)_2 = 6CaO.Al_2O_3Fe_2O_3.\ 12H_2O \qquad (2.3)$$

(Tetracalciumaluminoferrite (C_4AF) with the addition of water and calcium hydrocide will form calcium aluminoferrite hydrate)

$$3CaO.Al_2O_3+12H_2O+ Ca(OH)_2 = 3CaO.Al_2O_3.Ca(OH)_2 .12H_2O \quad (2.4)$$

(Tricalcium aluminate with the addition of water and calcium hydroxide will form tetracalcium aluminate hydrate)

$$3CaO.Al_2O_3+10H_2O+CaSO_4. 2H_2O = 3CaO.Al_2O_3. Ca (SO)_4 12H_2O \quad (2.5)$$

(Tricalcium aluminate with the addition of water and gypsum will form calcium monosulfoaluminate).

Compounds formed from equation 2.1 and 2.2 produce tobermorite gel, which contributes in the strength increase. Calcium hydroxide increases the ability of the clay to flocculate by means of cation exchange during cement hydration. The reactions shown in equation 2.3, 2.4 and 2.5 produces the silicates and aluminates required for the cementation of clay particles. Through these reactions, the clay particles are transformed into a stabilized matrix structure which consists of clay sheets attracted by forces created by various transformed compounds. These compounds initiate the process of cementation of clay sheets through flocculation process. The new arrangement of cement particles results in increased strength and reduced volumetric changes in the structure. The overall benefits of cement-stabilized soils are increased strength and stiffness, reduced volumetric changes, and increased durability.

Several types of cements are available in the market. In order to meet different physical and chemical requirements for various applications, eight types of cement are manufactured. These are Type I to Type V and Type IA, IIA and IIIA are Portland cements (Zaniewski, 1999). Type I cement is used for general purpose such as RC structures. Type II is used for structures built on soils with moderate amounts of sulfate content. Type III cement is one which develops high strength at an early stage usually in a week. Type IV is used for massive structures such as dams as it moderates the heat generated by hydration process. Type V cement is often suitable to stabilize high sulfate soils as it resists chemical attack due to sulfates. Table 2.1 presents the chemical compositions of Type V Cement used in the present research.

Table 2.1 Chemical Composition of Type V Cement used in this Research

Chemical Composition	**Percent**
Calcium Oxide (CaO)	53.10 %
Silicon Dioxide (SiO_2)	29.33 %
Aluminium Oxide (Al_2O_3)	NA
Sulfur Trioxide (SO_3)	3.30 %
Magnesium Oxide (MgO)	1.44 %
Loss of Ignition (LI)	0.93%
Total Alkalies (Na_2O_{eq})	0.59 %
Insoluble Residue (IR)	13,72 %
Class F Ash	20.75 %
Sulfate Expansion (C-1012)	NA

2.4.2 Class F Fly ash with Type V Cement

Coal ash is one of the byproducts generated from coal combustion in electrical generating units (Ferguson, 1993). Coal ash is composed of three components namely; fly ash (flue gas stream), boiler slag (coats boiler tubes) and bottom ash (sand size material + boiler slag). Based on the type of coal, burners and boiler, 65 % - 85 % of the organic material is fly ash. The components of coal ash have different percentages of compositions and particle sizes. The finest particle is the fly ash which is collected from the suspension in the exhaust gases of combustion chamber and most of the fly ash is Class F type of fly ash. Bottom ash is relatively a coarser and denser material than fly ash and is collected by gravity of the lower level (Nicholson and Kashyap, 1993).

Fly ash is classified as class F and class C as per ASTM. Class F fly ash is made from the burning of bituminous or anthracite coals and Class C fly ash is produced from the burning of sub-bituminous or lignite materials. Table 2.2 presents the chemical requirements as per ASTM C 618 to classify the fly ash.

Table 2.2 Chemical Requirements for Fly ash Classification as per ASTM C618

Properties	**Fly ash Class**	
	Class F	**Class C**
Silicon dioxide (SiO_2) plus aluminium oxide (Al_2O_3) plus iron oxide (Fe_2O_3), min, %	70.0	50.0
Sulfur trioxide (SO3), max, %	5.0	5.0
Moisture Content, max, %	3.0	3.0

The main difference between Class F fly ash and Class C fly ash is the amount of calcium, silica, alumina and iron content in the ash. Calcium content in Class F fly ash typically ranges from 1 to 12 percent, mostly in the form of calcium hydroxide, calcium sulfate, and glassy components in combination with silica and alumina. Another difference between Class F fly ash and Class C fly ash is the amount of alkalies which is the combination of sodium and potassium. Sulfates are generally higher in Class C fly ash when compared to Class F fly ash. Percentage of free calcium in Class C fly ash is higher when compared to Class F fly ash.

Considerable increase in soil strength and stiffness properties is shown upon the addition of cement to fly ash (McManus and Nataraj, 1993).When fly ash is mixed with soil for stabilization, it commences both short term and long term reactions (Diamond and Kinter, 1965; Usmen and Bowders, 1990; Glenn and Handy, 1963; Davidson et al., 1958). In short term reactions flocculation and agglomeration of clay particles takes place due to ionic exchange at the surface of soil particles. In long term reactions, increase in the strength properties in the treated soil can be observed. Increase in the strength can be seen over a period of time which may take place in a few weeks or years and is dependent on the rate of chemical breakdown and hydration reactions of silicates and aluminates. The hydration reaction improves and binds the soil grains together to form cementitious materials (Nicholson and Kashyap, 1993). Reaction of free lime (CaO) with the pozzolans (AlO_3, SiO_2, Fe_2O_3) in the presence of water is required to form cementitious material in hydration process. The hydrated calcium silicate gel or calcium aluminate gel (cementitious material) can bind inert material together. In order to have cementation reaction, availability of pozzolans is very important as they are the sourse of silica and alumina and allows hydration reactions. These hydration reactions occur by alkali or alkali earth hydroxides to form cementitious products in the presence of moisture at ordinary temperatures (Nicholson and Kashyap, 1993).

In case of Class C fly ash, the calcium oxide (lime) in the fly ash reacts with the siliceous and aluminous materials (pozzolans) present in the fly ash itself. Addition of lime is very much necessary in class F fly ash since the lime content is relatively low, for hydration reaction with the pozzolans of the fly ash. Fly ash may be used as an admixture to eliminate the pozzolanic deficiency of soil with the supply of steady source of pozzolans. Usmen and Bowders in 1990 conducted a comprehensive study on the factors influencing soil strength properties.

The parameters which can affect the reaction rate of the fly ash reactions are soil types, higher surface area of soil particles, temperature, moisture content, chemical composition of fly ash admixture, and amount of stabilizer used in the mixture. In case of stabilization of soils with lime, pozzolanic reactions depend on the siliceous and aluminous materials provided by the soil and the reactions that occur during stabilization are;

$$Ca\,(OH)_2 \Rightarrow Ca^{++} + 2[OH]^- \qquad (2.6)$$

$$Ca^{++} + 2[OH]^- + \underset{\text{(silica)}}{SiO_2} \Rightarrow \underset{\text{(gel)}}{CSH} \qquad (2.7)$$

$$Ca{+}{+} + 2[OH]{-} + \underset{\text{(alumina)}}{Al2O_3} \Rightarrow \underset{\text{(gel)}}{CAH} \qquad (2.8)$$

One of the primary cementitious products provided due to hydration of tricalcium aluminate in the flyash as shown in the equation (2.6). The quick setting of these materials are due to the rapid rate of hydration of the tricalcium aluminate which causes delays in compaction result and lower strengths of the stabilized materials.

The use of cement along with flyash will act as better treatment for the soils of Southeast Arlington (Wattanasanticharoen, 2000). Workability characteristics of the soils can be improved by stabilization with Cement and Fly ash which can provide an immediate reduction in swell, shrinkage, and plasticity properties (Nicholson and Kashyap, 1993). Few research studies conducted at The University of Texas at Arlington have also shown that fly ash stabilization decreases both swell and shrinkage strains of soils by decreasing the plasticity index of the soil (Puppala et al., 2000).

In this research study, Class F fly ash was considered as one of the stabilizers to be evaluated because soils of SE Arlington contain high amounts of sulfates. Use of Class F fly ash gains preference over class C fly ash as treatment with class C fly ash leads to the formation of ettringite based heaving (Mamlouck and Zaniewski, 1999). The chemical composition of the Class F fly ash used in this research is presented in table 2.3.

Table 2.3 Chemical Characteristics of Class F Fly ash (Wattanasanticharoen, 2000)

Chemical Analysis	**Results**
Silicon Dioxide (SiO_2), %	56.7
Aluminum Oxide (Al_2O_3), %	29.5
Iron Oxide (Fe_20_3), %	4.9
Sum of $Si0_2$, Al_20_3, Fe_20_3, %	91-1
Calcium Oxide (CaO), %	1.1
Magnesium Oxide (MgO), %	0.8
Sulfur Trioxide (SO_3), %	0.1
Moisture Content, %	0.2
Loss on Ignition, %	2.2
Amount Retained on No. 325 Sieve, %	29.8
Specific Gravity	2.28

2.4.3 Ground Granulated Blast Furnace Slag (GGBFS)

Blast furnace slag is a by-product material of iron production and the composition if slag is siliceous components of iron ore and limestone flux Coal Ash used for melting iron (Sherwood, 1995). The chemical composition of GGBFS is similar to that of Portland cement. During the manufacture of metals from their ores, various kinds of slags are produced but the only product that is suitable for use as a cementitious material is ground granulated iron blast furnace slag (Ozyildirim et al., 1990).

Molten slag rises up to the surface of the iron in the blast furnace that can be collected and the molten slag can be further used to form a granulated glassy material which possess once it is ground to the fineness of cement. The granules are called ground granulated blast furnace slag (GGBFS) when ground to the fineness of Portland cement. Once this material is mixed with cement, the alkalies are released by hydration of cement which is often adequate for the activation of GGBFS. GGBFS powder has been successfully applied as a raw material of cement block, pavement block, and slag cement.

The addition of cement to GGBFS can increase the strength of the treated soil. It is possible to achieve complete cost efficient soil stabilization by choosing designing appropriate cement and GGBFS proportions. The cost of GGBFS is much lesser than the cost of cement. This has been confirmed in a project in which the replacement of parts of Portland cement in soil stabilization with slag resulted in a significant decrease in the cost of the soil stabilization (Ozyildirim et al., 1990).

The permeability of the soil mixture can be reduced significantly, depending on the amount of the slag used (Ozyildirim et al., 1990). The permeability is reduced by a reduced pore size associated with the production of dense calcium silicate hydrates in hydration process that take place during mixing (Ozyildirim et al., 1990). This decrease in permeability can provide high chemical resistance in aggressive environments increases the sulfate resistance of the soil.

Ground Granulated Blast Furnace Slag (GGBFS) is used as a soil stabilizer in many countries such as US, UK, Germany, Holland and other Asian countries. The slag stabilization not only increased the sulfate resistance in soils, but also increased the shear strength, decreased the plasticity index, swelling potential and shrinkage strains (Ozyildirim et al., 1990).The percentage of slag in the Portland cement clinker, with which it is ground, may vary from very low to as high as 85 percent. The properties of these cements are essentially similar to those of Portland cement (Sherwood, 1995).

Considerable research was in England to study the behavior of the GGBFS stabilization on sulfate rich soils (Wang et al., 1998). The test results from this research showed that there is a significant strength increase, reduction in plasticity index, swelling and shrinkage strains achieved in soils after three days of curing by addition of 20% of GGBFS stabilizer (Wang et al, 1998). Taking into consideration these positive results, GGBFS was considered as one of the four stabilizers in the present research. The chemical composition of the GGBFS used in this research is presented in table 2.4.

Table 2.4 Composition of Blast Furnace Slag (Wattanasanticharoen, 2000)

Chemical Constituents (Oxides)	**Range of Composition, (Percent by mass)**
SiO_2	32-40
Al_2O_3	7-17
CaO	29-42
MgO	8-19
S	0.7-2.2
Fe_2O_3	0.1-1.5
MnO	0.2-1.0

2.4.4 Lime mixed with Polypropylene Fibers

Lime is perhaps the most common and the oldest chemical treatment used to improve soil properties (Gedney and Weber, 1978). Lime has been used in various applications in the civil industry such as highway, railroad, runway construction projects and hydraulic protection structures. Several historical places such as the Appian Way, Rome, Italy and the pyramids of Shersi in Tibet have been constructed using compacted mixtures of clay and lime material (Winterkorn & Pamukcu, 1991). During the past two decades, lime stabilization has significantly increased, especially in the US, Scandinavia and Southeast Asia (Bergado et al., 1991; Broms, 1984; Holm et al., 1983). In 1987 US alone had used 750,000 tons of lime in soil stabilization projects all across the country. Approximately 80% of the lime used was hydrated lime and the remaining 20% of the lime was quicklime (Gillott, 1987).

Basically there are three types of lime based on their compositions are Quick lime, which is chemically calcium oxide (CaO) and hydrated lime which is calcium hydroxide, $Ca(OH)_2$. The third type which is less frequently used in soil stabilization is calcium carbonate ($CaCO_3$), which is a carbonate of lime. The relations between these three types of lime are explained as follows (Sherwood, 1995):

$$CaCO_3 + Heat \Rightarrow CaO + CO_2 \quad (2.9)$$

$$CaO + H_2O \Rightarrow Ca(OH)_2 + heat \quad (2.10)$$

$$Ca(OH)_2 + CO_2 \Rightarrow CaCO_3 + H_2O \quad (2.11)$$

The first reaction which is a reversible reaction occurs and results in the production of quick lime from chalk or limestone at high temperatures in the order of 500 degrees as shown in equation 2.8. The production of the hydrated lime by mixing quick lime with CO_2 is presented in equation 2.9. These two types of lime; quick lime and hydrated lime are used in soil stabilization Different types of lime can be found in following forms.

Hydrated lime is available in the form of a fine, dry powder and quicklime is available either in granular form or as a powder form. Both hydrated lime and quicklime are used in slurry form by mixing with water. The addition of lime to expansive soils helps increase the Plasticity Index value, and further leads to reduction in swell and shrinkage strains, an increase in shear strength and a decrease in the compressibility and permeability properties (Broms and Boman, 1979; Littlle, 1987; Puppala et al., 1998).

A dehydration reaction will take place and slaked lime is created when a certain amount of quicklime is mixed with clayey soil. This dehydration process results in immediate reduction in water content due to the drying effect in the soil which is advantageous in improving the soil plasticity characteristics in moist clays (Schoute, 1999).Calcium hydroxide is a product of the dehydration reaction and when $Ca(OH)_2$ is mixed with water it dissociates and in turn increases the electrolytic concentration and the pH of the soil. The calcium hydroxide dissociation is explained in the equation 2.12 (Schoute, 1999):

$$Ca(OH)_2 \Rightarrow Ca^{2+} + 2(OH)^- \qquad (2.12)$$

The calcium ions released will participate in the cation exchange reactions in soils and the following important processes occur in soils are due to the mentioned reactions (Rogers et al., 1997):

- Reduction in susceptibility to water addition due reduced thickness of electric double layer.
- Flocculation of the clay particles with weak bonds between the particles which is caused by an increase in mutual attraction due to decrease in electric double (Diamond & Kinter, 1965).
- Internal angle of friction between the particles increases due to flocculation.
- Textural change from plastic clay to a granular, friable material.

Due to few limitations associated with the lime stabilization method such as leaching problems, they are not suitable for soils where significant strength enhancements are necessary and when granular deposits are present. It is noted that sulfate induced heave distress problems are experienced when lime is added to sulfate rich soil (Kota et al., 1996). Several project sites in states such as Texas, Oklahoma, Kansas, Nevada and Colorado experience sulfate induced heave distress problems due to the formation of ettringite mineral when treated with lime (Puppala et al., 2000). Therefore, several research studies are conducted to understand the heaving mechanism in chemically stabilized sulfate rich soils and to develop appropriate stabilization methods to control sulfate induced heave (Viyanant, 2000).

In this effort, polypropylene fiber materials are used to reinforce soils since they are cost effective and also reduce the intake of natural raw materials when compared to other materials including chemicals. They can be manufactured with desirable properties from recyclable materials to the specified dimensions and can overcome leaching problems. Currently, the fibers are used to enhance the soil strength properties, to reduce the shrinkage properties and to overcome chemical and biological degradation (Gregory, 1996; Puppala and Musenda, 1998). Fibers are used in concrete and mortar to reduce shrinkage related cracks (Reibeiz et al., 1994). Similar positive effects are expected to enhance the soil properties in the lime stabilization method by controlling the volume change behavior. There would be significant cost savings in future projects if it can be proven experimentally and analytically that combined stabilization method with lime and fibers does enhance the soil properties significantly. Hence the combination of quicklime and polypropylene fibers was used as on of the stabilization methods in this research. The properties of the fibrillated polypropylene fibers used in this research are summarized in table 2.5 (Boral Material Technologies):

Table 2.5 Fiber Properties used in this Research (Wattanasanticharoen, 2000)

Physical Properties	**Magnitudes**
Material	100 % virgin polypropylene
Tensile Strength	97 ksi
Young's Modulus	580 ksi
Melt Point	330 F
Ignition Point	1100 F
Specific Gravity	0.91
Bulk Density	56 lbs/cubic ft
Dosage	1.5 lb/cubic yard
Form	Fibrillated Polypropylene
Fiber Count	8 - 12 million/lb
Chemical Resistance	Excellent
Alkali Resistance	Excellent
Acid and Salt Resistance	High
Fiber Length	¾ "
Absorption	NIL

2.5 Pavement Instrumentation

The critical problems that face pavement researchers are the rapid and continuously growing demands to design and build better performing pavements. This may be achieved through pavement instrumentation. Pavement instrumentation is a process to monitor the behavior of a specific pavement system which comprises identification of critical locations in the pavement, selection of sensors, calibration of the sensors, identification of possible errors, installation, and finally data collection. During the past three decades, attempts were made to enhance pavement analysis and design by measuring the stresses and strain at critical locations inside a pavement system, and compare them to calculated strain levels at critical locations in the pavement system for determining failure strains (Battiato et al., 1977; Ku et al., 1967).

The primary requirement of any pavement instrumentation project is that it should be part of a lucid pavement research program to obtain maximum benefits (Nassar, 2001). A few published reports were found on the variability associated with pavement instrumentation. The process itself is complex, with a lot of variability associated with the installation, sensor-pavement interactions, data acquisition, and interpretations. Studying pavement performance through the use of instrumentation without proper assessment of the sensor performance may lead to unreliable results (Nassar, 2001). The benefits from pavement instrumentation projects are undoubtedly significant and a lot of information can be learned. Once proper planning is accomplished, collected data can be used to serve two main purposes. The prime purpose is to validate existing or novel design approaches which are accomplished by evaluating field-measured parameters like stresses, strains and deflections in the field. This part of the literature review discusses the instrumentation facilities found to date. A review of the different sensors used and the typical responses obtained from these instrumentation projects are presented in the second part.

As technological capability advances, so does the entire supporting infrastructure (Nassar, 2001). Thus today pavement instrumentation is experiencing a technological uprising to withstand the infrastructure demands by understanding the material performance in the field as well as pavement system response to loading and environment. This uprising is directed towards developing sensors to measure pavement response parameters. Parameters that need to be considered in the field include strains, stresses, deflections, moisture, and temperature. Measuring these parameters in the field allows for accurate performance and design better pavements (Nassar, 2001).

Soil properties required in the analysis and design in geotechnical engineering have been conventionally determined based on laboratory and in-situ test results (Jamiolkowsky et al., 1985). The laboratory tests conducted in controlled environments provides the physical strength and compressibility characteristics of the soils. Soils demonstrate large variations in its behavior in real field conditions due to its heterogeneous nature when compared to other civil engineering materials. The variation in its nature can be attributed to geological history of soil formation, location of formation, temperature and environmental characteristics. As a result, laboratory test results can only provide an approximate behavior and demands more accurate and reliable methods to measure physical and engineering properties of soil in the field conditions.

With different accelerated pavement testing projects being constructed today, different sensor manufacturers are finding a new practice of marketing their products. Before implementing any of these products, one has to make sure of their applicability and usefulness in pavement applications.

2.5.1 Instrumentation Devices Used in Geotechnical Engineering

Geotechnical Instrumentation has been incorporated in array of applications which includes foundations, retaining walls, monitoring the slope stability, and excavations. Parameters like volumetric and gravimetric moisture contents, pore water pressure, overburden pressure, displacement and strain are very important to be monitored as they directly influence the behavior of soil and structural response. Accordingly, pavement instrumentation is crucial to understanding material performance in the field, as well as pavement system response to loading and environment. The following sections summarize the various instrumentation devices used in geotechnical engineering.

2.5.1.1 Strain Measurement Devices (Turner and Hill, 1999)

During the past three decades, attempts were made to enhance pavement analysis and design by measuring the strains at critical locations inside a pavement system, and compare them to calculated strain levels at critical locations in the pavement system for determining failure strains (Ku et al.,; 1967 Battiato et al., 1977). Measurement or calculation of traffic-induced pavement strains at specific locations is important to predict the failure mechanisms and understand material performance in the field. Initially mechanical strain measurement devices were used and they were laborious as the strain readings had to be recorded manually. Presently, electrical and electronic strain gauges are widely used due to the ease in installation and data acquisition. The only advantage in using mechanical devices in that, it can be operated without power.

Electrical gauges operate by relating the resistance values to calibrated strain readings. Electrical strain gauges are the most commonly used device and these gauges are available in quarter-bridge, half bridge and full bridge configurations which corresponds to a normal wheat stone bridge. Full bridge configurations are mostly preferred as they are equipped with bridge balancing mechanism, which is very important to produce consistent and repeatable readings under same conditions. Readings from these gauges can be obtained by using readout boxes and data acquisition systems.

2.5.1.2 Displacement Measurement Devices (Dally et al., 1993)

Displacement measuring devices are classified as linear displacing measuring devices and angular displacement measuring devices. Their working principle is same as that of strain measurement devices. All these devices require an anchor support and measures the displacements relative to anchor support. The descriptions of these devices are as follows:

- Potentiometers: These gauges moving frame, which on movement results in potential drop in electric potential and this drop is calibrated to measure the displacement.
- LVDT or Linear Voltage Differential Transformer: These gauges work on the principle of variable inductance. A linear change in inductance of the transformer present in the gauge is caused due to the displacement of rod. Displacements are related to these variations in inductance values.
- Optical Displacement Measurements: In order to analyze the signals and interpret those values, this method includes the utilization of fiber optics,

digital videos, and high-speed photography camera along with specialized computer programs. This method of instrumentation does not require any physical contact with the soil to be monitored. These systems are comparatively expensive when compared to other displacement devices.

- Extensometers: The relative displacements with respect to the anchor embedded can be measured through extensometers. They are widely used devices as they are relatively cheap and can be easily connected to data loggers to digitize and automate the data collection.
- Tilt meters, Inclinometers and Electro Levels: These devices are used to measure the rotational deformation but however due to their high cost and difficulties in installation, they are not widely used.

2.5.1.3 Force and Pressure Measurement Devices (Dally et al., 1993)

Most of these gauges contain an elastic member which uses the force or pressure exerted to produce strain data. These strains are transformed to their corresponding pressure values by strain converting units. The parameters which affect the performance of these gauges are its shape and actual area of the gauge, which is in direct contact with the soil.

- Load Cells: There are types of load cells currently available but the main principle of all these devices are the same in which strain of the elastic member inside is measured and transformed to the force applied.
- Pressure Gauges: The primary use of the pressure cells is to measure the subgrade pressure (Sargand et al., 1997; Selig et al., 1997; Metcalf, 1998). Although measurement of strain is clearly important in determining certain

major failure modes, the relative importance of stress/pressure measurement cannot be overlooked. The primary function of pressure cells is to monitor the change in the stress-state of the overlying layers and to measure the increase in vertical pressure due to dynamic traffic loading. The main distinction between the load cells and pressure gauges are that the load cells measures the total load on the surface where as the pressure cell calculates the average pressure caused to develop the tangential strain throughout the surface area of the gauge.

- Piezometers: Monitoring ground water levels and periodic analysis of pore pressure distributions in soils are of prime importance in geotechnical engineering projects (Hoek and Bray, 1981). In case of retaining walls and slopes assessment of pore pressure distributions are important to ascertain the stability of the structure and also the drained or undrained conditions of these structures Therefore, the main application of piezometers is to monitor ground water levels and for pore pressure measurements.

2.5.1.4 Other Instruments

- Temperature Gauges: These gauges are usually thermocouples and monitor the temperature of the soils. These gauges are generally used if the influence of temperature on soil properties is anticipated and they are also used to include data corrections in the acquired data.

2.5.1.5 Instruments for Data Acquisition

Data acquisition systems are used along with electrical gauges to record continuous or periodic responses or variations from the installed gauges. Based on the format in which data is recorded, these systems are subdivided into analog and digital systems. Readout which is similar to voltmeters box is a commonly used analog device. Readings are obtained based on converting the potential difference readings and manual recording of data from these gauges. The advantage in this module is that these can be installed in the site and the data can be obtained at regular intervals. Digital acquisitions are usually used in which is usually carried out with the help of data loggers that have an internal storage unit and acquisition cards that connect to computers and transfers data immediately to the computer. These are used in research projects that entail discrete data and where the time interval between two readings is considerably high. The advantage of these modules when compared to data loggers are that these are comparatively cheaper in cost and do not require continuous on site power supply.

2.5.2 Case Reviews

As instruments are continuously getting upgraded, it is very important for design and practicing engineers to be aware of these technological advances. Instrumentation is done either to monitor structural disintegration, to assess the quality assurance of the construction or to develop, verify and modify analytical models. This section presents case reviews of the different sensors used and the typical responses obtained from these instrumentation projects.

2.5.2.1 Field Instrumentation from PENNDOT (Stoffels et al., 2006)

The Pennsylvania Department of Transportation (PENNDOT) is sponsoring the Superpave in-situ stress/ strain investigation (SISSI). SISSI is a state-of-art instrumentation project that includes eight Superpave sections across Pennsylvania in four projects are newly constructed pavements and rest four are overlays over existing pavements. The objective of this project was full scale investigation of pavement performance with field instrumentation. It includes monitoring of construction process, materials characterization, detailed load response information, traffic and environmental data. Figure 2.4 shows the layout of gauges at one of the SISSI sites.

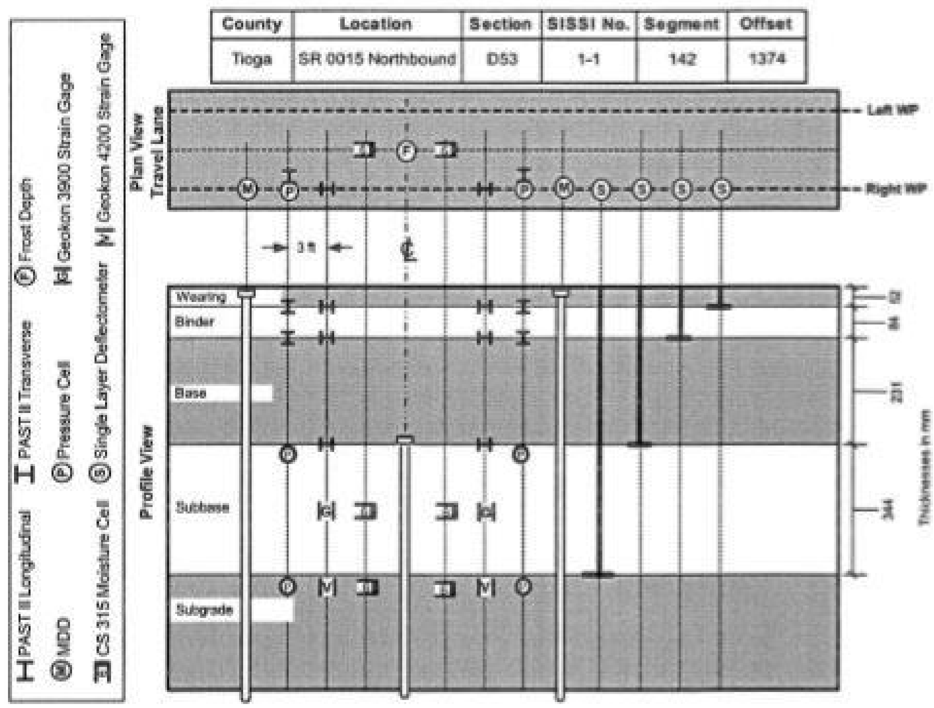


Figure 2.5 Layout of Gauges at SISSI site (Stoffels et al., 2006)

The instrumentation program included Dynatest PAST II Strain Gauge, CTL Multi-depth Deflectometer, Geonor Pressure Cell and Geokon 3900 Strain Gauge. The main aim of this instrumentation program was to capture the dynamic data at various seasons of the year. The magnitude of strains and pressure experienced at different seasons by the pavement under truck loading were recorded. There was significant difference seen in response of pavement at different seasons helping to understand how deflections and pressures vary with loading and seasonal variation in a pavement structure.

2.5.2.2 FHWA Pavement Testing, Virginia (Mitchell et al., 2006)

In summer 2002, 12 full-scale lanes of pavements with various modified asphalts were constructed at FHWA pavement testing facility in Virginia. The objective of this study was to use FHWA's two accelerated loading facility machines and validate and refine changes being proposed in Superpave binder specification. Each machine is capable of applying an average of 35,000 wheel passes per week from half-axle load ranging from 33 to 84 kN (7500 lbs to 19000 lbs). During the construction, 12 lanes were instrumented with strain gauges and survey plates. Multiple-depth deflectometers (MDDs) were installed in selected lanes. Pavement responses for both strain gauges and MDDs were measured after the construction and during loading in pavement rutting and fatigue tests. The thickness of HMA and CAB layers is 26-in and was constructed on silty clay soil. Lanes 1 through 7 were constructed with a 4-in. thick HMA and lanes 8 through 12 were constructed with 6-in. thick HMA layer.

Each pavement lane has four test sites for full-scale testing for two failure modes; rutting (at sites 1 and 2) and fatigue cracking (at sites 3 and 4). All 12 test lanes were instrumented during construction with strain gauges and survey plates. Thermocouples were installed in each site shortly before loading. MDDs were installed in selected lanes. Figure 2.5 presents the instrumentation locations for the test site. The strain gauges were of H-bar type, embedded asphalt strain gauges. A total of 60 strain gauges were installed in 12 pavement lanes. Five strain gauges were embedded at the bottom of HMA layer in each lane. These gauges were placed both longitudinally and transversely. Two sets of MDDs were installed prior to loading in each site 1 of lanes 4 and 11 respectively.

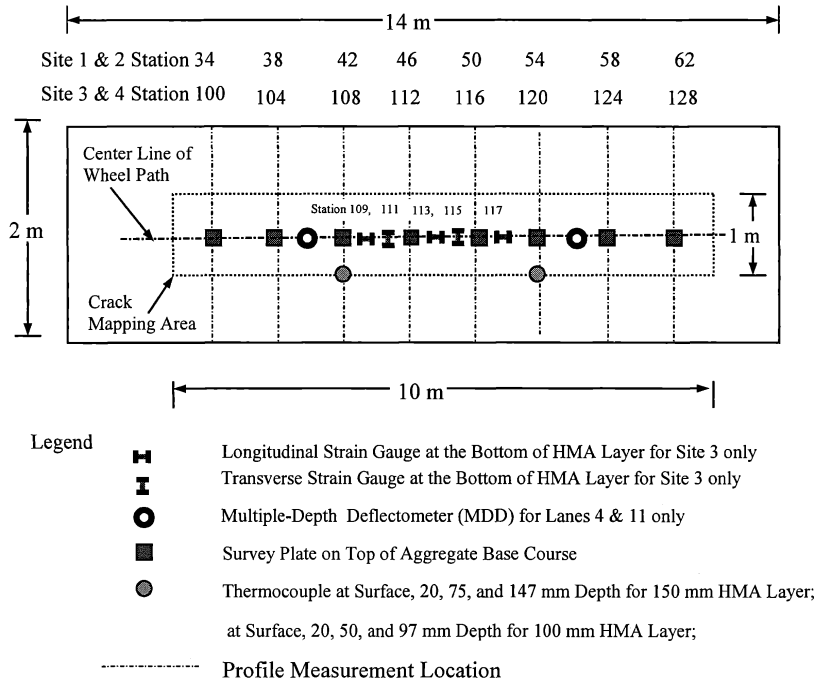


Figure 2.6 Instrumentation Locations for Test Site

Permenant deformations were recorded during rutting tests with MDDs for lanes 4 and 11. Strain responses were measured under loading at various conditions. The results showed that the predicted strains were consistently lower than measured strains for all the lanes all offset distances.

2.5.2.3 Bedford Project Case (Al-Qadi, 1999)

The main focus of this project was to study the effectiveness of the use of geosynthetic in flexible pavements and how it can be factored in the design procedure. Nine instrumented secondary road test sections were constructed as part of the realignment of Routes 757 and 616 located in Bedford County, Virginia. Each test section was 15 m long. Three test sections were constructed using a geogrid, three with a geotextile, and three were non-stabilized. The constructed base course thicknesses were 100, 150, and 200mm. The HMA thickness averaged 8.9mm. The pavement test sections were instrumented with earth pressure cells, soil strain gauges, soil moisture sensors, and thermocouples. The geotextiles and geogrids were also instrumented with strain gauges. The majority of the instruments were placed in the right wheel path of the inside lane of the test sections. All instrumentation, cabling, and data acquisition facilities were located underground. The data acquisition system was triggered by truck traffic passing over piezoelectric sensors, and was operated remotely. Once the system was triggered, the instrumentation was continuously sampled at a frequency of 200 Hz for a period of either 12 or 10 seconds depending on the triggering location (Al-Qadi, 1999). The corresponding data were transferred to Virginia Tech via a modem for processing.

All instruments were placed during construction of each corresponding layer. Instruments located in the subgrade were Kulite earth pressure cells, Carlson earth pressure cells, soil strain gauges, thermocouples, and gypsum blocks. Pressure cells, gypsum blocks, and thermocouples were installed below the compacted surface of the base course and backfill each sensor to avoid instrument damage from large angular aggregate.

2.5.2.4 Pavement Responses in Denver Airport (Rufino and Roesler, 2006)

In 1992, the Federal Aviation Administration (FAA) initiated a major research in an effort to study the in-situ response and performance of Portland cement concrete pavements. FAA, in cooperation with the U.S. Army Corps of Engineers and Waterways Experiment Station (CEWES), instrumented several pavement slabs in the take-off area of Runway 34R at the Denver International Airport (DIA). During the construction of the Denver International Airport, the FAA and the U.S. Army Corps of

Engineers instrumented 16 slabs in the takeoff area of runway 34R-16L (Lee et al., 1997). The instrumented section is located 121.9 m (400 ft) from the runway threshold and is 22.9 m (75 ft) wide and 24.4 m (80 ft) long. There are 460 static and dynamic sensors to monitor pavement responses (Dong et al., 1997). As an aircraft passes over the instrumented section, infrared sensors trigger the dynamic sensors and data acquisition system, which then captures the pavement responses due to combined aircraft loading and environmental conditions. Dynamic responses include strains, vertical displacements, and aircraft information (position, speed, and acceleration). The information related to each aircraft pass is stored in a database as a unique event. Position sensors cast in the concrete slabs during construction are used to identify the aircraft lateral location. The methodology developed to identify aircraft location within the instrumented pavement section is described in (Rufino et al., 2001). FSingle and paired H-bat strain gauges and linear variable differential transtruders (LVDTs were used to collect strains and deflections respectively during each aircraft pass. Figure 2.6 shows eight of the 16 instrumented slabs with the location of H-bar strain gauges and LVDT sensors. The focus of the test was to determine the effect of aircraft loading on pavement design and service life, as well as monitor deterioration of pavement due to environmental loading. Both multidepth deflectometer (MDD) and strain gauges were used to characterize the interface condition and determine the most significant factors affecting this interface condition. Measured slab responses from actual aircraft passes were also used in comparisons with theoretical results for the two extreme interface conditions.

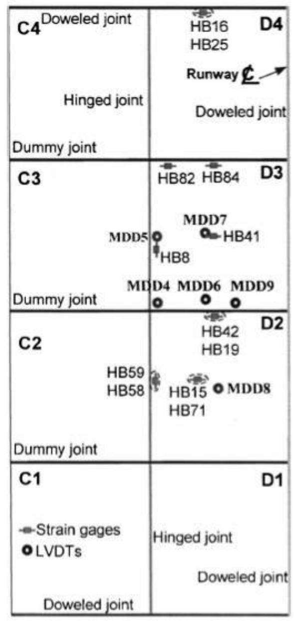


Figure 2.7 Location of H-bar Strain Gauges and LVDT Sensors

2.5.2.5 Instrumentation in Open Car Park (Aboumoussa and Iskander, 2002)

An expansion joint in an open car park of 295.4 ft in length and 235 ft in width was instrumented and monitored for over a period of one year. The joints were instrumented with four vibrating-wire displacement transducers with integrated temperature sensors which were connected to data loggers. Transducer measurements were recorded on an hourly basis. Figure 2.7 shows the placement of vibrating wire displacement transducer.

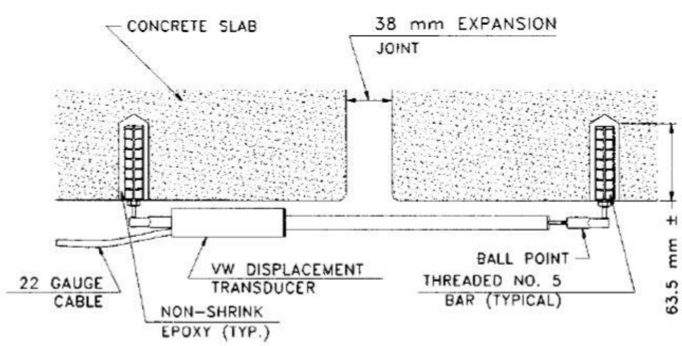


Figure 2.8 Placement of Vibrating Wire Displacement Transducer

From the instrumentation data, it was found that the use of concrete walls with relatively large mass and rigidity, cast monolithically with floor slabs will impose additional restraint on thermal movements of the slabs and result in ineffective presence of thermal joints due to reduced thermal co-efficient.

2.5.2.6 Minnesota Road (MnRoad)

A pavement research facility was constructed in the state of Minnesota: Minnesota Road (MnRoad) which consists of approximately 40-160m of pavement test sections. Twenty-three of these test sections were loaded with freeway traffic, and the remaining sections were loaded with calibrated trucks. Freeway traffic loading began in June 1994. 4572 electronic sensors were embedded in the roadway and 1151 of them were used to measure pavement response to dynamic axle loading. The specific brands and models of each type of sensor were selected based on recommendations by Minnesota Department of Transportation (MnDOT's) which was derived from research contracts for evaluation of pavement sensors, and by consultation with government agencies and worldwide instrumentation experts. The main purpose of this instrumentation was to verify and improve existing pavement design models and learn more about the factors that affect pavement response and performance, which can help developing new pavement models that can allow building and maintaining more economical roadways.

There are several other research studies where instrumentation is extensively used in monitoring the performance of different structures and theory verification. Some of them are listed in table 2.6.

Table 2.6 Research Studies with Instrumentation

Authors	Studies	Instrumentation Type	Purpose
Zhong et al., 2006	Instrumentation and Accelerated Testing on Louisiana Flexible Pavements	Multi-Depth Deflectometer, Pressure Cells	Helped in studying the pavement performance with static and dynamic loading.
Loulizi et al., 2006	Difference between in-situ flexible pavement measured and calculated stresses and strains	H-type Strain Gauge, Pressure Cells	Compare measured vertical compressive stresses and measure transverse horizontal strain under HMA layer
Xu et al., 2007	Dynamic Response of Suspension Bridge to Typhoon and Trains. I: Field Measurement Results	Anemometers, accelerometers, and level sensing systems.	The measurement results clearly demonstrated the dynamic behavior of the bridge with running trains during high winds
Reay et al., 2006	Long-Term Durability of State Street Bridge on Interstate 80	Strain gauges, tiltmeters, thermocouples, and humidity sensors	Evaluate the long-term durability of the carbon fiber reinforced polymer (CFRP) composite and externally CFRP-reinforced concrete of the State Street Bridge.
Rollings (1992)	Field instrumentation ad performance monitoring of rigid pavements.	Instrumentation used to develop rigid pavement design procedures.	Field instrumentation helped in developing new models for pavement incorporating thermal stress and load transfer effects in to the design procedure.

Table 2.6 - *Continued*

Ortigao et al. (1996)	Monitoring during Tunnel construction.	Extensometers and Inclinometers.	Instrumentation helped in successful completion of the tunneling project due to the availability of continuous settlement data.
Whitman (1991)	Field data was used extensively to develop appropriate sequencing and scheduling strategies.	Piezometers, inclinometers and extensometers.	Values obtained from the instrumentation provided methods and proved vital for the completion of the project.
Wong et al. (1997)	Field performance of nailed soil wall in residual soil.	Inclinometers, Strain gauges, Pressure cells.	Instrumentation provided data assured the effectiveness of soil nailing to restraining the lateral movements of the soil.
Sparrevik (1996)	Development of new platform foundation concept through instrumentation.	Strain gauges.	Data obtained from instrumentation helped to verify the proposed concepts and make some inclusions.
Baker Jr. et al. (2001)	Temperature effects on contact earth pressure cells were studied.	Pressure transducers.	Temperature coefficient for temperature effects on transducer in pressure cells was established. Theoretical temperature correction factor was found to be dependent on elastic properties of the soil surrounding the cell.

Table 2.6 - *Continued*

Wong et al. (1997)	Field performance of nailed soil wall in residual soil.	Inclinometers, Strain gauges, Pressure cells.	Instrumentation provided data assured the effectiveness of soil nailing to restraining the lateral movements of the soil.
Huslid (2001)	Full scale monitoring of troll of a concrete platform.	Pressure cells.	The amount of pressure transferred was obtained using the pressure gages.
McGrath, Timothy et al. (1999)	Instrumentation for monitoring buried pipe behavior during backfilling.	Strain gauges, Pressure gauges, Nuclear gauges, Penetrometers.	Pipe- soil Interface properties were studied and led to development of parameters for the same.

2.6 Summary

This chapter has presented a wide-ranging summary on problems due to sulfate rich expansive soils and alternative chemical stabilization methods considered in this research. Several case studies explaining the practical problems faced due to sulfate soils have been covered. The second part of the chapter has summarized the various geotechnical instrumentations for measuring strains, pressures, displacements, moisture, inclinations and temperatures. Several case reviews revealing the importance of instrumentation have also been included.

CHAPTER 3

SELECTION OF STABILIZERS FOR HARWOOD ROAD SOIL

3.1 Introduction

In order to investigate the properties of soils in Southeast Arlington, Texas and select suitable stabilizers for their treatment, an experimental program was designed and conducted. Harwood Road located in Southeast Arlington, Texas was selected as the test site in this research. The soils of this test site are highly expansive and rich in sulfates. These soils were treated using four different types of stabilizers namely, Sulfate Resistant Type V Cement, Class F Fly ash with Type V Cement, Ground Granulated Blast Furnace Slag, and Lime mixed with Polypropylene Fibers.

This chapter presents the physical and chemical properties of treated and control soils, laboratory tests performed to determine their basic and engineering properties.

3.2 Determination of Basic Properties of Harwood Road Soil

The properties of materials are of prime importance as it helps one understand and appreciate the behavior of materials when used to stabilize the pavement infrastructure. Therefore, the following parts of the chapter represent the properties of the materials used and their behavior in laboratory conditions.

3.2.1. Harwood Road Soil

This soil was sampled from Harwood Road located in South Cooper Estate Village in Southeast Arlington, Texas. In order to investigate the performance of stabilized soil, several engineering tests were performed in laboratory and field conditions by Wattanasanticharoen, (2000), Chavva, (2002) and Ramakrishna, (2002). Basic soil property tests such as sieve analysis, Atterberg limits, specific gravity, soluble sulfate and hydrometer tests were first conducted. Atterberg limit tests were conducted as per ASTM D-4318 method to determine the consistency of the soil (Chavva, 2002). Figure 3.1 shows the gradation results of sieve and hydrometer analysis. The gradation curve of this soil represents the presence of different ranges of fine to course grained particles.

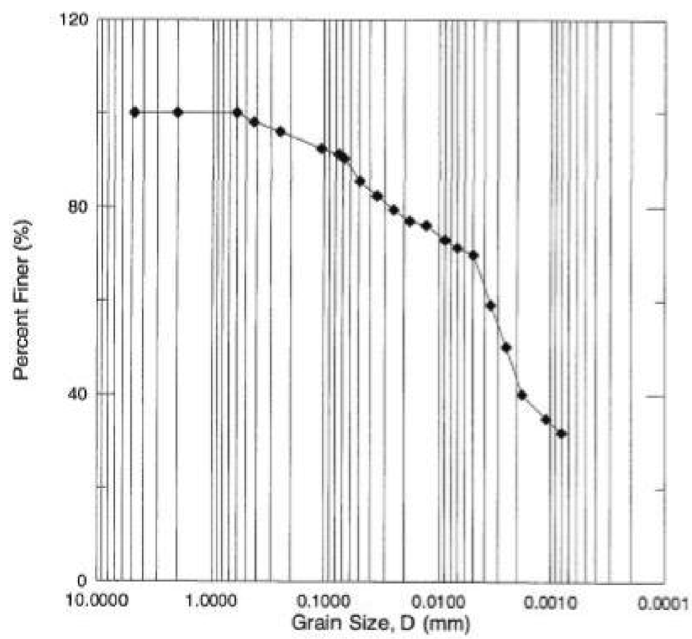


Figure 3.1 Grain-size distribution Curve of Harwood Road Soil (Chavva, 2002)

Based on gradation and Atterberg limits, this soil was classified as A-7-6 as per AASHTO classification method, and as sandy fat clay with gravel (CH) as per USCS classification method (Chavva, 2002). Soil samples obtained for these tests also exhibited the following properties (Chavva, 2002);

Table 3.1 Basic Soil Properties of Harwood Road (Chavva, 2002)

Soil Properties	**Results**
Color	Dark Brown
Passing #200 (%)	91.2
Specific gravity	2.73
Liquid Limit (%)	55.5
Plasticity Index	22.2
Natural Moisture Content (%)	7.0
Soluble Sulfate Content (ppm)	4737
pH	8.13
AASHTO Classification	A-7-6
USCS Classification	CL

3.2.2. Stabilizers Considered for Research

Four types of stabilizers were considered for treating sulfate rich expansive soils in the current research, namely Sulfate Resistant Type V Cement, Class F Fly ash with Type V Cement, GGBFS and Lime mixed with Polypropylene Fibers. The reasons for selecting these stabilizers are discussed in this section.

3.2.2.1 Sulfate Resistant Type V Cement

The soils in Southeast Arlington are classified as soft, expansive soils and rich in sulfates. As discussed in the previous chapter, treatment of sulfate rich expansive soil with calcium based stabilizers like Type I Cement or Lime results in sulfate induced heaving. Therefore, in our present research study, Type V Cement was selected as cement stabilizer.

3.2.2.2 Class F Fly ash

An immediate stabilizing effect was noticed when soil treated with Class F fly ash due to ion exchange at the surfaces of soil particles causing flocculation and agglomeration. Therefore, Class F fly ash was selected as one of the four stabilizers in this research which results in enhancement of workability characteristics in soils and gives an immediate reduction in swell and shrinkage properties of soil (Wattanasanticharoen, 2000).

3.2.2.3 Ground Granulated Blast Furnace Slag (GGBFS)

GGBFS stabilization method is known to increase the sulfate resistance of the soil. High chemical resistance in aggressive environments benefits from the reduction in permeability. Permeability is reduced due to the reduction in pore size associated with the production of dense calcium silicate hydrates in hydration process that takes place during mixing. This reduction is in turn determined by the amount of slag used (Ozyildirim et al., 1990). GGBFS stabilization not only increases sulfate resistance in soils but also increases the shear strength, decreases the plasticity index, swelling potential and shrinkage strains (Ozyildirim et al., 1990). Therefore, GGBFS was selected as another stabilizer.

3.2.2.4 Lime mixed with Fibers

The types of lime that commonly used in the stabilization of soils are Dry hydrated lime, Quicklime and Slurry lime. Quicklime was used in this research as it is more economical when compared to hydrated lime. Quicklime is faster in drying action than hydrated lime on wet soils. Faster reactions can be seen with quick lime in wet soils. Lime stabilization helps in decreasing the plasticity index, swelling, and shrinkage, and increases the soil strength. In order to decrease shrinkage based cracking and increase shear strength of the soil, fibers are recommended to be used with lime. Hence, the combination of lime and fibers was used in this research.

3.2.3. Standard Proctor Compaction Test

Standard proctor compaction tests were conducted in order to determine the optimum moisture content and dry unit weight of both control and treated soil. Samples were prepared as per ASTM D-4218 (Ramakrishna, 2002). Standard proctor test results are presented in Table 3.2 and Figure 3.2 respectively.

Table 3.2 Moisture Content and Dry Unit Weight of Raw and Treated Soil (Ramakrishna, 2002)

Sl. No.	**Soil Type**	**Optimum Moisture Content (%)**	**Dry Unit Weight (pcf)**
1	Control Soil	18.65	105.50
2	Type V Cement (8%)	16.70	106.90
3	Class F Fly ash (15%) and Type V Cement (5%)	18.70	104.20
4	GGBFS (20%)	16.00	107.30
5	Lime (8%) and Polypropylene Fibers (0.15%)	18.00	96.00

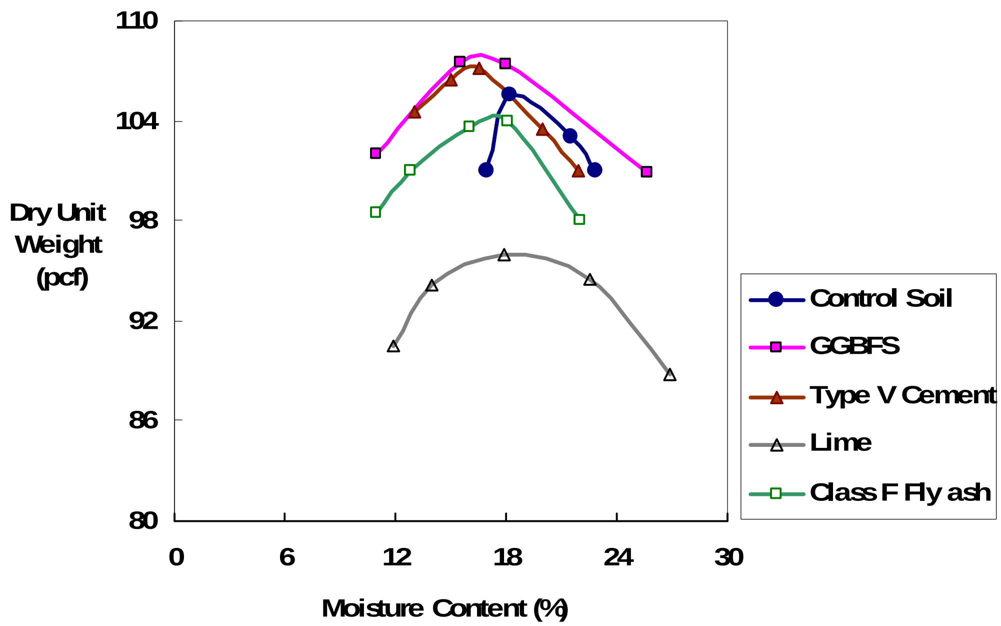


Figure 3.2 Standard Proctor Compaction Test Results (Ramakrishna, 2002)

3.3 Determination of Engineering Properties of Harwood Road Soil

Several engineering tests were performed by (Chavva, 2002) which included unconfined compressive strength tests (UCS), linear shrinkage strain test and free swell tests. The UCS and free swell were conducted at three different moisture contents which were optimum, dry of optimum and wet of optimum moisture contents of control soil. Dry and wet side moisture levels correspond to 95% of optimum dry unit weight. Resilient Modulus tests (M_R) was conducted by (Ramakrishna, 2002) to accurately measure the modulii with varying moisture content.

3.3.1 Sample Preparation

The compositional and environmental conditions such as moisture content, dry unit weight of soils and stabilizer dosages usually influences the plasticity index, swelling, shrinkage, and strength properties of soils. The variable conditions considered by (Chavva, 2002) in sample preparation are listed in Table 3.3. The soil was first oven dried and pulverized. Then the soil was mixed with selected chemical stabilizers at optimum moisture content and dry unit weight levels. The specimens were compacted in Standard Proctor molds and then carefully extracted in case of UCS test and Resilient Modulus test. Then the samples were wrapped and stored in humidity rooms for curing for forty eight hours before compacting. Specimen preparation in case of soil mixed with lime and fiber stabilizers were in accordance to ASTM D3551-90. The samples for other stabilizers i.e. cement, fly ash, and GGBFS were prepared immediately after mixing (Chavva, 2002).Resilient modulus tests was conducted at different moisture contents to measure the moduli by (Ramakrishna, 2002). Table 3.4 presents the dry, optimum and wet moisture contents for different stabilizers.

Table 3.3 List of Variable Conditions in Sample Preparation (Chavva, 2002)

Description	**Variables**
Soil Type	Sulfate Rich Expansive Soil from Harwood Road
Stabilizers	Sulfate Resistant Type V Cement, Class F Fly ash with Type V Cement, GGBFS and Lime with Polypropylene Fibers
Stabilizer Dosage	One
Moisture Contents	Optimum, Dry of Optimum and Wet of Optimum
Temperature Conditions	Room Temperature
Curing Period	Seven Days

Table 3.4 Moisture Contents for Different Stabilizers (Chavva, 2002)

Soil Type	**Optimum Moisture Content (%)**	**Dry of Optimum (95%)**	**Wet of Optimum (95%)**	**Dry Unit Weight (Pcf)**
Control Soil	18.65	16.5	21.2	105.50
Type V Cement (8%)	16.70	14.5	19.7	106.90
Class F Fly ash (15%) and Type V Cement (5%)	18.70	14.3	22.8	104.20
GGBFS (20%)	16.00	15.4	22.1	107.30
Lime (8%) and Polypropylene Fibers (0.15%)	18.00	15.5	22.8	96.00

3.3.2 One Dimensional Free Swell Test

Free swell test measures the amount of heave in a confined specimen. Both control and treated specimens measuring 2.5 inches in diameter and 1 inch in thickness were included as per ASTM standards. Porous stones were placed at the top and bottom of the specimen to facilitate water movement. These specimens were then placed in a container and filled with water. The amount of heave was measured by a micrometer dial gauge against elapsed time and actual time. Maximum swell values were observed for over a period of three days (Chavva, 2002). The final displacements and the original heights were used to calculate the free swell values in the vertical directions. Table 3.5 presents the free vertical swell of control and treated soil after three days.

Table 3.5 Free Vertical Swell Strain for Control and Treated Soils (Chavva, 2002)

Soil Type	Free Vertical Swell Strain at Dry of Optimum (%)	Free Vertical Swell Strain at Optimum (%)	Free Vertical Swell Strain at Wet of Optimum (%)
Control Soil	8.3	7.5	5
Type V Cement (8%)	0.22	0.1	0
Class F Fly ash (15%) and Type V Cement (5%)	0.27	0.1	0
GGBFS (20%)	0.43	0.1	0
Lime (8%) and Polypropylene Fibers (0.15%)	0.73	0.64	0

The decrease in the free swell in all the stabilized soils was due to the decrease in plasticity index due to chemical treatments. From the observed results, lime and fiber treatment experienced maximum heaving which can be attributed to the addition of fibers which must have induced open fabric and low unit weight in treated expansive soils.

3.3.3 Linear Shrinkage Strain Test

Linear Shrinkage Strain Test was conducted as per Texas Department of Transportation (TxDOT) method specified by Tex-107-E (Chavva, 2002). Soil paste mixed at moisture content level of liquid limit state is placed in the linear shrinkage mould. The samples are air dried at room temperature for twelve hours and then oven dried for twenty-four hours. The length of dried samples is measured by using vernier calipers and the linear shrinkage was expressed as percentage of its original length. Table 3.6 presents the results for linear shrinkage strain test. It can be noticed from the results that the linear shrinkage values of control section is the highest. And with the treatment, the shrinkage strain values were reduced. All four stabilization methods displayed similar low shrinkage strains which were due to the reduced plasticity index of the soil due to treatment.

Table 3.6 Linear Shrinkage Strain for Control and Treated Soils (Chavva, 2002)

Soil Type	**Linear Shrinkage Strain (%)**
Control Soil	6.2
Type V Cement (8%)	1.4
Class F Fly ash (15%) and Type V Cement (5%)	1.5
GGBFS (20%)	2.3
Lime (8%) and Polypropylene Fibers (0.15%)	1.4

3.3.4 Unconfined Compressive Strength Test

The UCS tests were conducted were as per the ASTM D-2166 standards and samples were prepared as mentioned in section 3.3.1. Once the sample was prepared and cured, they were placed on the compressive test platform. The sample is then loaded at a constant rate which was controlled by a loading device control. Deformation data and axial load was collected from a computer attached to a test setup. The maximum axial compressive load at which the sample failed was used to determine the unconfined compressive strength of the soil sample. Five specimens were tested for each stabilizer at particular moisture content and the average was considered (Chavva, 2002). Table 3.7 shows the UCS strength values.

Table 3.7 UCS Strength Values for Different Stabilizers (Chavva, 2002)

Sl. No.	**Soil Type**	**Dry of Optimum (psi)**	**Optimum (psi)**	**Wet of Optimum (psi)**
1	Control Soil	31.3	36.3	20.9
2	Type V Cement (8%)	200.9	225.8	198.2
3	Class F Fly ash (15%) and Type V Cement (5%)	123.9	154	113.9
4	GGBFS (20%)	99.8	108.8	83.7
5	Lime (8%) and Polypropylene Fibers (0.15%)	54.4	50.9	35.4

We can notice that Type V cement has the highest strength value. In case of lime and fibers, the presence of fibers enhanced the cohesion intercept of the treated specimen but at the expense of puzzolanic cementation effects of lime due to which the UCS value for lime and fibers was higher at dry of optimum (Chavva, 2002). The strength increase in the stabilized soil specimens were ascribed to the cementitious reactions from the stabilization process.

3.3.5 Resilient Modulus Test

The resilient modulus (M_R) is defined as the ratio of the axial deviator stress to the recoverable axial strain (Puppala and Mohammad, 1995). Resilient modulus test was conducted to understand the effect of compaction moisture and confining pressure on M_R properties of control and stabilized soil. It was also conducted to analyze the effects of stabilizers on M_R properties of soil. The tests on materials were conducted with the confining and deviator stress levels following the procedure specified by AASHTO TP46. The method consisted of applying a repeated deviatoric load on the specimen with fixed load duration of 0.1 sec and 0.9 sec period relaxation. The test consisted of a conditioning phase and testing phase, with 1000 cycles of conditioning and 100 cycles for each testing phase. Samples were conditioned by applying one thousand repetitions of a specified deviator stress to eliminate the effects of specimen disturbance caused by sampling, compaction and specimen preparation procedures (Ramakrishna, 2002). Once the sample conditioning was done, test was conducted as specified by AASHTO TP46 to cover the service range of stress that a subgrade material experiences due to traffic loading and over-burden conditions. The resilient modulus results were assessed for repeatability and were analyzed to understand the effects of moisture content, stress influence on the resilient properties of soil and stabilizer reactions (Ramakrishna, 2002). Table 3.8 presents the average resilient modulus results for control and stabilized soils at a confining pressure of 14 kPa.

Table 3.8 Resilient Modulus (M_R) for Control and Treated Soils at Confining Pressure of 14 kPa (Ramakrishna, 2002)

Soil Type	**M_R at Dry of Optimum (MPa) (Increase in M_R in Mpa)**	**M_R at Optimum (MPa) (Increase in M_R in Mpa)**	**M_R at Wet of Optimum (MPa) (Increase in M_R in Mpa)**
Control Soil	133.1	80.3	35.8
Type V Cement (8%)	268.9 (135.8)	762.7 (682.4)	198.2 (266.8)
Class F Fly ash (15%) and Type V Cement (5%)	169 (35.9)	389.8 (309.5)	186.0 (150.2)
GGBFS (20%)	151.2 (18.1)	351.8 (271.5)	195.1 (159.3)
Lime (8%) and Polypropylene Fibers (0.15%)	187.7 (54.6)	80	53.7 (17.9)

The results above indicate that the highest enhancement in the magnitudes of M_R can be seen in case of Sulfate resistant cement and the lowest values of enhancement in case of lime and polypropylene fibers.

Based on the laboratory evaluations and the rankings presented by Chavva, (2002); Ramakrishna, (2002); and Wattanasanticharoen, (2000), following four stabilization methods were recommended to stabilize soils of Harwood road.

1. Sulfate Resistant Type V Cement
2. Class F Fly ash with Type V Cement
3. GGBFS
4. Lime with Polypropylene Fibers
5. Control Soil (Lime treated soil)

3.4 Summary

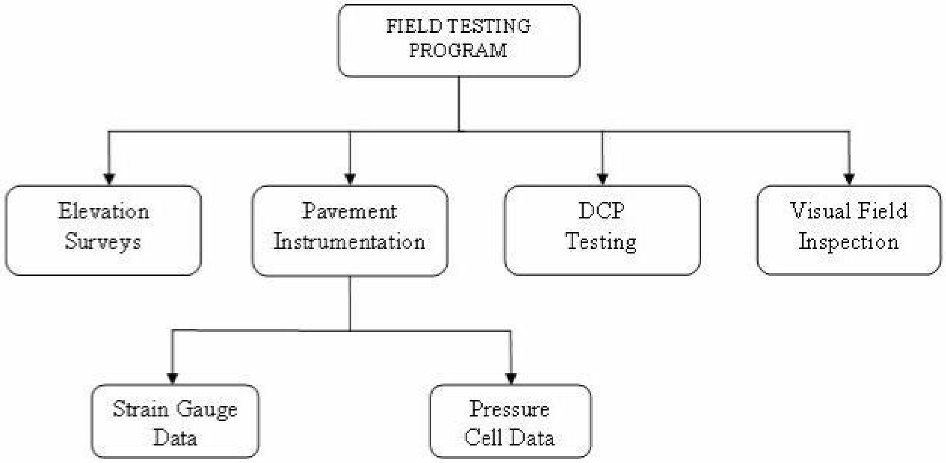
The physical and chemical properties of treated and control soils were summarized in this chapter. Laboratory tests performed to determine their basic and engineering properties of different stabilized soils were also elaborated in this section. Although these stabilizers provided good laboratory results, it was required to assess the performance of all the above mentioned stabilizers in real field conditions over a period of time. The performance of the above selected stabilizers in real field conditions is presented in Chapter 5.

CHAPTER 4

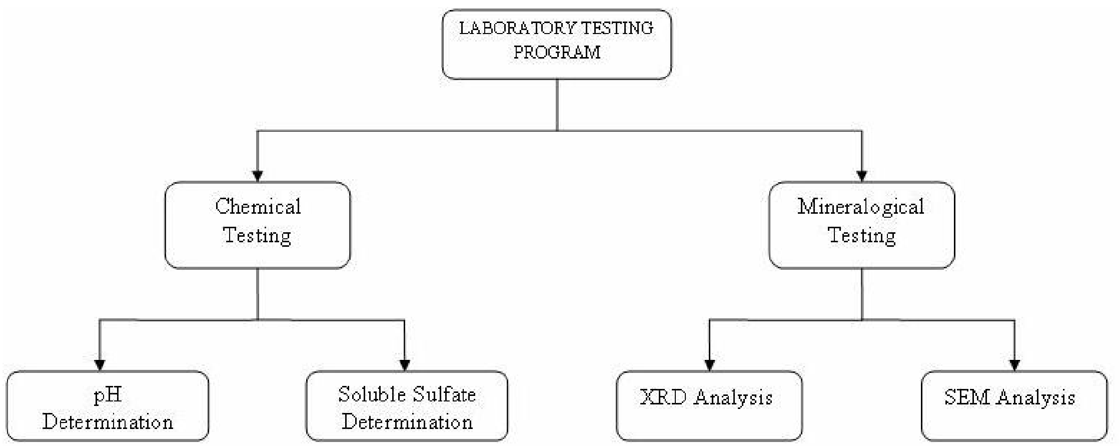
FIELD AND LABORATORY TESTING PROGRAM

4.1 Introduction

The main aim of the present research project was to design and select appropriate stabilization methods for treating expansive sulfate rich subgrade soils in Southeast Arlington, Texas. Field monitoring, assessment of chemical and mineralogical characteristics of stabilized subgrades and documentation of their performance are considered important to validate the effectiveness of stabilizers. Stabilizers showing promising results in the laboratory conditions were selected to be used in the field. The success of achieving the present research objective is dependent on addressing the long-term potentials of soil stabilization in real field conditions where they are exposed to variations in temperature, humidity, rainfall and various external disturbances. Field testing is generally associated with instrumentation studies to monitor the performance of pavements built on stabilized subsoils. Figure 4.1 shows the details of the present research program which includes field and laboratory testing programs.



(a)



(b)

Figure 4.1 Research Program (a) Field Testing Program (b) Laboratory Testing Program

This chapter explains in detail the field and laboratory studies conducted which include pavement instrumentation, elevation surveys, chemical tests and mineralogical tests to efficiently assess the performance of treated subgrade soils. The knowledge and understanding of the site conditions are very crucial in defining the functions of sensors and appropriate data acquisition features in pavement instrumentation and its features. Therefore, details of site conditions are also addressed in this chapter

4.2 Site Conditions

The primary step was to familiarize and understand the site condition. Harwood road, a sublet from Collins Street, located in Southeast Arlington, Texas was selected to perform stabilizer evaluation studies. Soils at this site are expansive in nature and rich in sulfates and prone to sulfate heave when treated with calcium based stabilizers like cement and lime.

4.3 Field Testing Program

The prime goal of this research is assessment of the stabilized sections in the real field conditions. In order to analyze and address any heave related movements and load carrying potentials of treated subgrades of sulfate rich expansive soils of Harwood road, pavement instrumentation, elevation surveys and finally Dynamic Cone Penetration test (DCP) were incorporated in the field testing program.

4.3.1. Pavement Instrumentation

As technological capability advances, so does the entire supporting infrastructure. Within the last decade, pavement instrumentation has been recognized as an important tool for quantitative measurement of pavement performance and response under different environmental and traffic conditions. The environmental influencing factors include temperature, freeze-thaw cycles, and moisture content changes due to seasonal changes. The loading factors include magnitude, type and distribution of traffic loads. The response variables measured in-situ are subgrade and traffic induced stresses, strains and deflections. In-situ measurements of these parameters provide the data needed to analyze the performance of stabilized subgrades. Thus, pavement instrumentation today is experiencing a technological revolution to withstand the demands on the infrastructure by understanding the material performance in the field as well as pavement system response to loading and environment. This revolution is directed towards developing sensors to measure pavement responses parameters. Moreover, field stabilization studies with sensor-instrumentation will provide the efficiency of each stabilizer in controlling pavement distress such as differential heave, rutting and pavement cracking. Additionally, field studies utilizing instrumentation to monitor the performance of pavement can more comprehensively capture the effectiveness of stabilization efforts.

4.3.1.1 Pavement Design

The overall cost of the project depends mainly on number of engineering properties to be evaluated in the field. Therefore, selecting appropriate engineering properties which has to be measured in the field to evaluate the performance of stabilizers plays a vital role in instrumentation design. The parameters that need to be measured in the present research include strains and deflections. Measuring these parameters in the field allows the assessment of stabilizers, mechanisms of heave behavior and load carrying potentials of underlying treated subgrades.

Since soils of southeast Arlington have low strength, high swell and shrinkage properties, they are highly susceptible to pavement distress. In order to evaluate the compressive strain potentials of treated soils in Harwood road under traffic loads, strain-measurement devices were included in the instrumentation program.

Subgrades underneath pavements are exposed to varying intensities of dynamic loads from the traffic. These dynamic loads add to deformations in treated and untreated subsoils. Therefore, it is required to monitor the pressure or load experienced by the soil and also to explain the causes for the sudden changes in the strain values. To study these aspects, pressure cells were installed at this test site. The primary use of the pressure cells is measuring the subgrade pressure under traffic loads (Sargand et al., 1997; Selig et al., 1997; Metcalf, 1998). Pressure cells are used to monitor the pressure levels in the underlying layers, and may also provide the dynamic stress response of the base and subgrade. The main objective of pavement stabilization is to reduce the effects of traffic loading on the subgrade. The data obtained from strain gauges and pressure cells can be used to asses the effectiveness of stabilizers and the load carrying potentials of the underlying soil.

4.3.1.2 Sensors

Various types of strain measurement devices, which included surface, mounted mechanical strain gauges; extensometer, embedment type and vibrating wire were considered in this research project. Of all these gauges, embedment type gauges were considered suitable due to their accurate measurements. Also, as the thickness of the treated subgrade layer was only 8 in., placement of strain gauges would not cause any problems. To address the low survivability criterion of strain gauges as mentioned and documented by (Green et al., 1985), two strain gauges were installed for each pavement section, so that one would act as a back-up in the case of failure of any gauge.

In the current instrumentation program, swell strain changes in the soil were expected to be over large intervals of time. Based on this assumption, portable data acquisition module was found to be suitable. Based on these discussed requirements, IOtech's Wavebook along with the expansion module known as WBK-16 were considered appropriate due to its portability, high precision (can detect even 1 micro strain change) and its affordable costs (Mohan, 2002). In addition to the above mentioned sensors, a laptop computer was also required for reading data from the wave book module. The catalog of the data acquisition module provided the configuration for the computer (Pillappa, 2005). Table 4.1 lists the sensors installed at five test sections of the test site.

Table 4.1 Details of Instrumentation (Mohan, 2002)

Instrument Type	**Name**	**Manufacturer**	**Quantity**
Strain Gage	EGP-5-350	Micro-Measurements	40
Pressure cells	Geokon 3500-2-200	Geokon	10
Data Acquisition	WaveBook + WBK16	IOtech	1

Five pavement sections, each of 300 ft long, were selected and would be built on subgrades treated stabilized with four novel stabilizers and one control stabilization method. Due to similar soil types at the site, same instrumentation method is designed for all sections. All sections have an 8 in. thick stabilized subgrade and 6 in. thick concrete pavement. The pavement is situated in a low volume traffic category. Different stabilization methods considered and their corresponding proportions are presented in Table 4.2. The construction of these sections was initiated on September 20, 2004 and was concluded on November 5, 2004. More details of the construction of the test sections can be found in Pillappa (2005).

Table 4.2 Stabilizer Proportions

Soil Designation	**Percentage by dry weight**
Type V Cement	8
Class F Flyash and Type V Cement	15 and 5
Lime and Polypropylene fibers	8 and 0.15
GGBFS	20

After construction of individually treated sections, the pressure cells, strain gauges and data boxes were immediately instrumented. Figure 4.2 shows the typical plan-view and cross-section details of treated and instrumented sections.

The key objective was to instrument the most critical section to measure maximum strains under traffic loads. The pavement critical sections were underneath the wheel path (Mohan, 2002). Therefore, strain gauge was placed in a vertical manner and pressure cells in horizontal manner. Figure 4.3 shows a schematic diagram of the placement of sensors in the test section. Figure 4.4 and figure 4.5 shows the placement of strain gauges and pressure cells respectively. The wires from the ends of the sensors were cased within a high density polyethylene pipe to ensure that the movement of vehicles did not disrupt their continuity. The wires were then lead into the galvanized steel boxes via the conduit pipes. Then, the ends were soldered to the DB9 pins.

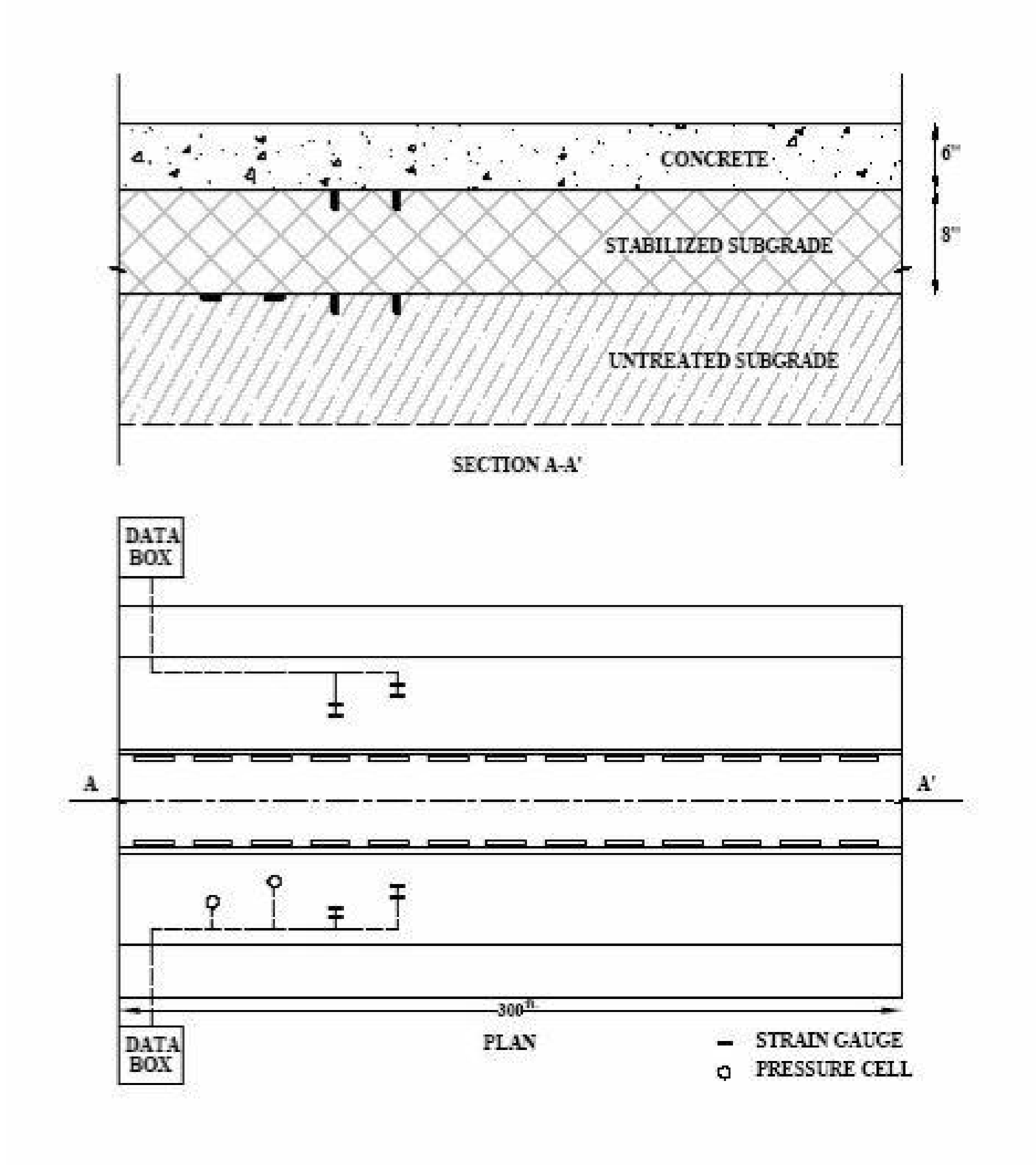


Figure 4.2 Typically Stabilized Pavement Test Section

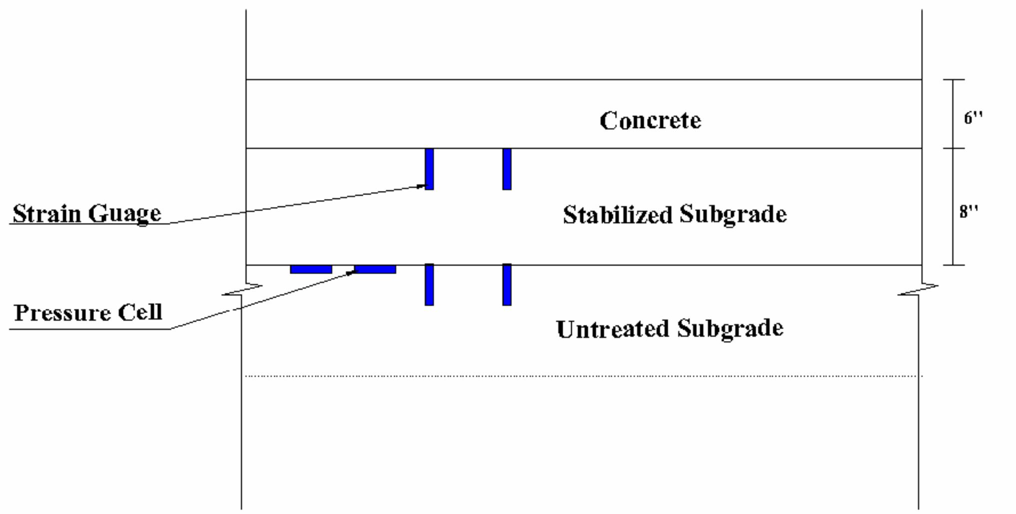


Figure 4.3 Placement of Sensors

Figure 4.4 Placement of Strain Gauges (Pillappa, 2005)

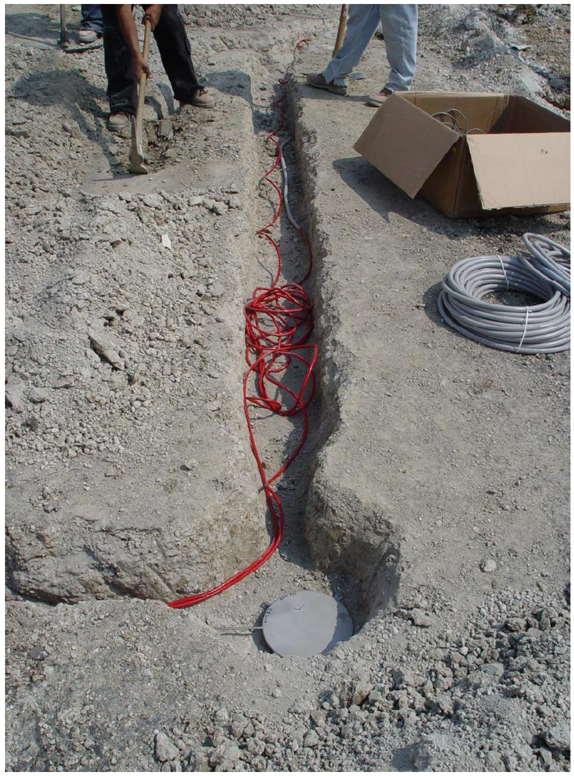


Figure 4.5 Placement of Pressure Cells (Pillappa, 2005)

A mobile data acquisition unit along with an expansion module was considered suitable because of its portability, high precision, and affordable price (Mohan, 2002).The locations of monitoring stations were designed and selected depending on the distance between the gauges, the length of the sensor cables, and the length of each individually treated section. Minimizing the distance between the sensors and the data module will provide noise free readings. Also, grouping of many sensors together decreases the manual effort required to collect the data every time. Based on these considerations the sensors were grouped and positioned as shown in figure 4.6.

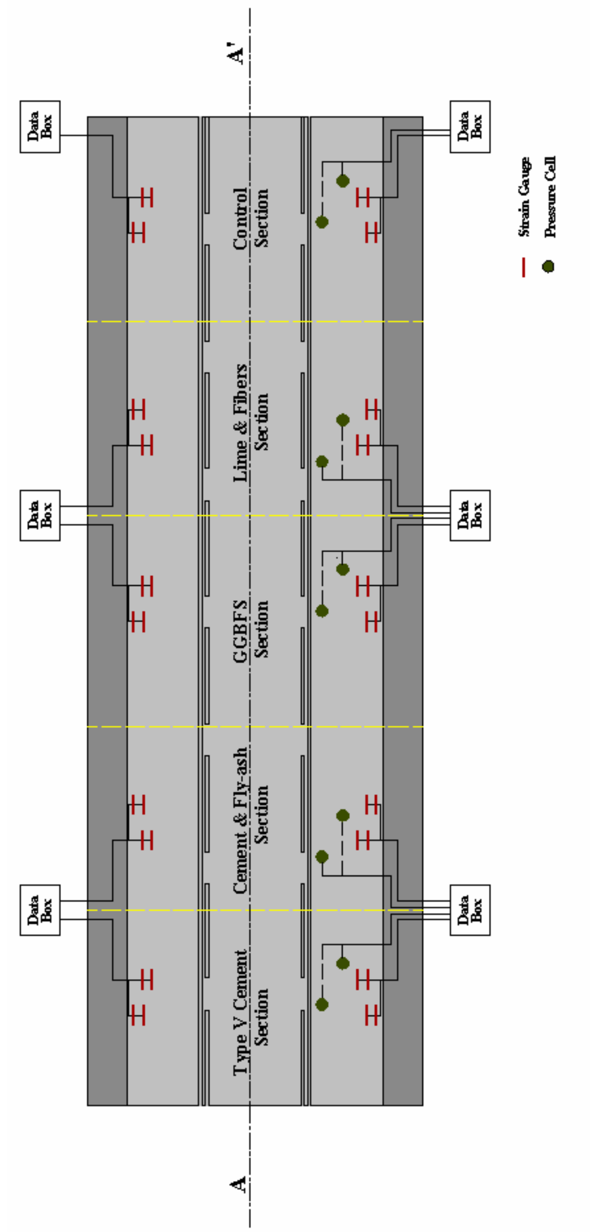


Figure 4.6 Placement of Sensors Plan View

The wavebook and WBK-16 was integrated and was then connected to the laptop through parallel port and interfacing cables. Software to access the acquisition modules were installed on the computer. The selected strain gauge measures strain based on the Wheat stone bridge imbalance principles explained in the previous chapter.

EGP-5-350 (strain gauge) was a quarter bridge type strain gage and hence it was required to use bridge calibration resistors to complete the circuit (Mohan, 2002). The pressure cells were full bridge strain gage type sensors. The WBK-16 module provided CN-115 headers (A small resistor holder) to include the bridge calibration resistors (Mohan, 2002). Shunt calibration method was used in order to compensate the resistance increase of the strain gages (Mohan, 2002).

The pavement section was opened to traffic in early February 2005 (Pillappa, 2005). Data collection was initiated during the month of January 2005 and continued till April 2007. The data was collected on a weekly basis. In case of sudden climatic changes like an occurrence of heavy rainfall, data was collected within twenty-four hours of precipitation. A mid-size passenger car with a gross weight of approximately 3,000 pounds has been used for data collection. The loading on the sensors was performed using different methodologies. In the first method, the data from both strain gauges and pressure cells are collected with no direct dynamic load applied to the sensors. In the second method, the passenger car is paced on the sensors and the sensor data is collected. The third method of loading on the sensors was to drive the car back and forth over the sensors and simultaneously collect the data from strain gauges and pressure cells. These three methods were implemented for all the stabilized sections.

The sensor readings contained a significant amount of noise when noticed from the data acquired. For the sensor data, a running average transformation was considered due to its simplicity and accuracy. The measured test data was first imported into MATLAB®. A five point running average algorithm was then implemented to reduce the noise. A single iteration of running average did not give significant reduction in the noise, due to high sampling rate, and hence a total of sixty iterations were implemented for strain gauges and thirty iterations were used for pressure cells. The peaks corresponding to the vehicular activity on both the travel lanes were taken into consideration. In case of loading condition, combination of data collected from the sensors which were both directly below the wheel path and in between the wheel loads were considered to analyze strains and pressures. Further, the difference of the readings (one with the loading and one without loading) was used in order to evaluate strains and pressures for each treated section.

4.3.2. Elevation Surveys

Elevation Surveys were performed using a 'total station setup' in order to evaluate the heave and other types of soil related movements including erosions of stabilizer treated soils. This monitoring data of the past 26 months is used in the present analysis. Eight points were chosen in each section, four along each lane. Figure 4.7 shows the plan view of the elevation survey points and the reference total station point. The points are evenly spaced at sixty feet intervals. The nearest permanent non-heaving structure was chosen as a benchmark or reference point.

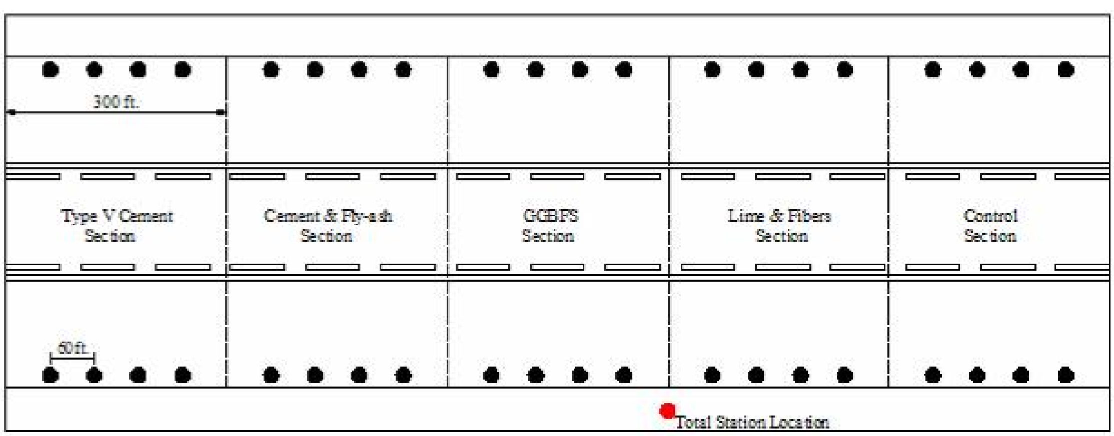
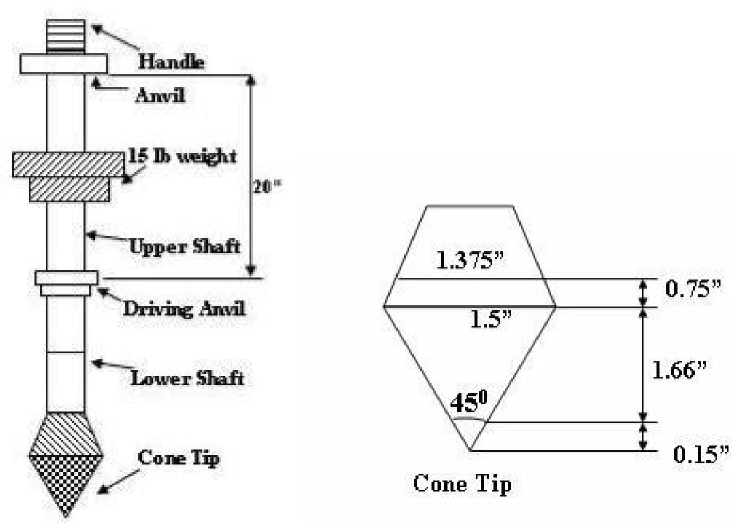


Figure 4.7 Plan View of Elevation Survey Points

After investigating the effectiveness in treating sulfate rich, expansive, soft soils of SE Arlington with pavement instrumentation and elevation surveys, the next stage of study was to perform DCP test. In order to predict the in-situ strength and moduli properties of the treated subsoils, DCP test was carried out on all the five treated sections in the field.

4.3.3 DCP Test

The Dynamic cone penetrometer (DCP) equipment mainly consists of two components, upper and lower shafts, which are attached to each other at the midpoint. Figure 4.8 shows the various components of (DCP) equipment. The handle at the top is held vertical using a plumb rod system during penetration testing. It is very important to hold the DCP shaft upright to avoid the development of friction at the sides of the shaft which can disturb the transfer of energy from shaft to cone.



(a) (b)

Figure 4.8 Components of DCP Equipment: (a) Body of DCP (b) Cone Tip

The steel hammer of the DCP placed on the upper shaft weighs exactly 15 lb. This hammer is raised and dropped repeatedly to penetrate the cone down into the soil. At the top of the lower shaft, the driving anvil is placed which serves as a platform to hold the hammer. The lower shaft is marked in 5mm increments, in order to record the readings. A cone tip with 45 degrees of angle is attached at the bottom of the shaft. If the penetration of the cone is less than 3 mm for 10 consecutive drops, the test has to be stopped else the cone tip will be damaged as noted by (Jones and Rolt, 1991).

The DCP shown in figure 4.8 utilizes a 15-lb steel mass falling from a height of 20-in to strike the anvil in order to drive a 1.5-in diameter hardened cone tip. The kinetic energy from the dropping hammer is transferred through the lower shaft to the cone to drive the tip into the soil. To maintain the consistency in energy imparted to the cone, the pullout anvil is fixed in place to ensure the height drop is always 20-in. Resistance of the soil can be defined as the work done to stop the cone and can be calculated as follows:

$$R_s = \frac{W_s}{P_d} \tag{4.1}$$

Where, R_s is the soil resistance; W_s is the work done to stop the cone; and P_d is the distance traveled by the penetrometer through the soil.

The energy produced due to each hammer drop can be calculated using kinetic energy relation. Soil resistance (R_s) for this hammer is 3.39 kN/cm i.e. each cm of penetration of the cone through the soil will experience a force of 3.39 KN.

The DCP test results are expressed in terms of dynamic penetration index or DPI of soil. The DPI is the amount of cone penetration due to one drop of the hammer and hence the unit used for expressing DPI is cm per blow or inches per blow. Figure 4.9 presents the schematic showing parameters to calculate DPI.

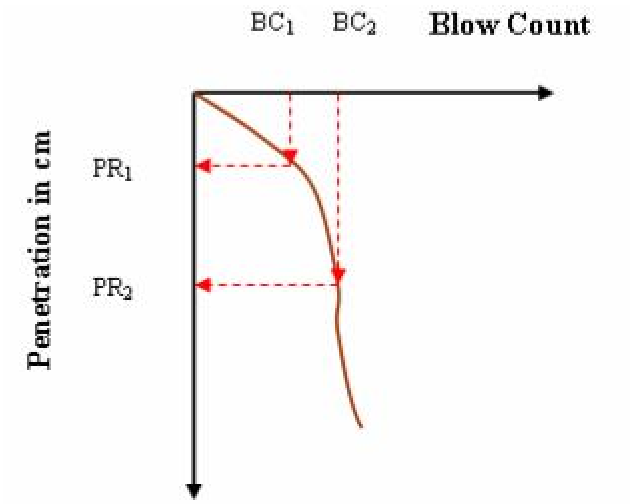


Figure 4.9 Parameters to Calculate DPI

The DPI can be obtained at a certain depth using the following formula:

$$DPI = \frac{PR_2 - PR_1}{BC_2 - BC_1} \tag{4.2}$$

Where, *PR* is the penetration reading in cm and BC is the blow count. (PR_2-PR_1) is the difference between two consecutive penetration readings at different depths and (BC_1-BC_2) is the difference between two consecutive blow counts.

4.3.4. Difficulties Experienced in Field

Few difficulties were experienced in the field during data acquisition and they are summarized below:

- The DB9 pins of the sensors were corroded as shown in figure 4.10 due to the accumulation of moisture in the data box which tampered the performance of sensors. The data box required frequent cleaning and re-soldering the DB9 pins were necessary.

- Due to the construction activities beside the test sections, the data boxes were frequently buried with soil as shown in figure 4.11. As a result, the soil had to be cleared before accessing the data box.
- During the process of data collection the traffic using the other side of the lane may have influenced the stresses applied to the sensors.
- The presence of high tension electric cable towers in the vicinity of instrumented sections led to significant amount of noise in the data. In order to eliminate the noise, MATLAB filter was required to filter the data.

Figure 4.10 DB9 Pins Rusted

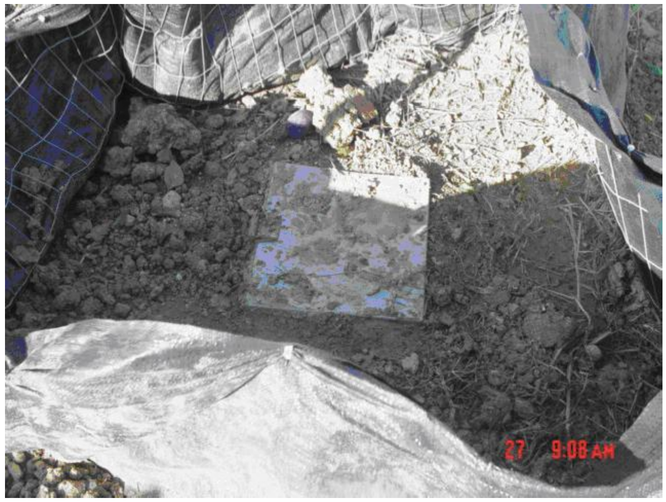


Figure 4.11 Buried Data Box

4.4 Laboratory Testing Program

4.4.1. Formation of Ettringite Mineral

As discussed in Chapter 2, when sulfate-rich soils are treated with calcium based stabilizers, the sulfates in the soils react with the alumina liberated from the clay particles and calcium component from the stabilizers in order to form a combination series of calcium-aluminum-sulfate hydrate compounds (Mitchell and Dermatas, 1995). These compounds will lead to the formation of ettringite crystals ($Ca_6 [Al (OH)_6]_2 * (SO_4)_3 * 26H_2O$) which can expand twice or three times of their original sizes when subjected to hydration. This in turn leads to pavement distress due to sulfate heaving.

Two types of laboratory tests were conducted on all the treated subbase soil samples in this research to monitor the formation of ettringite. These were chemical tests and mineralogical tests. Chemical tests were conducted in order to determine the pH and soluble sulfate content in the treated soils. Mineralogical tests were conducted to determine the formation of Ettringite in stabilized soils which included X-Ray Diffraction (XRD) test and Scanning Electron Microscope (SEM) analysis.

4.4.2. *Chemical Tests*

Two types of chemical tests were conducted which included pH test and soluble sulfate determination test. Determination of pH and sulfate content in the treated soil helps in realizing if the soil conditions are favorable for ettringite formation.

4.4.2.1 pH Determination of Stabilized soils

The pH test was performed to understand the acidic and basic conditions of the stabilized soils. This test was performed by following ASTM D-4972 specification. In order to find the pH value of the soil samples, a 1:1 ratio of dried soil to distilled water was used in this method. The soil samples were first mixed with distilled water and then shaken and mixed again to ensure thorough mixing. Then, the pH was monitored by inserting an electrometric indicator into the soil mix, which provides the pH conditions in the soil. The test conducted on both control and all the treated soil samples.

4.4.2.2 Soluble Sulfate Determination

In order to monitor the amount of soluble sulfates in the soil samples after treatments with stabilizers, soluble sulfate determination test was performed. This test was considered important to assess the sulfate heaving process in chemically treated soils. The method used in this research was a modified procedure from the standard gravimetric method outlined in the seventeenth edition of Standard Methods for the Examination of Water and Wastewater by (Clesceri et al., 1989; Viyanant, 2000). As recommended by (Petry and Little, 1992), the water extraction ratio was 1:10. Therefore, 10 grams of dried soil was diluted with 100 mL of distilled water. The extraction of the solution was obtained by centrifuging with the speed of 14,000 rpm. The pH values of the prepared sample solution were controlled within the range of 5 to 7 by the addition of Hydrochloric acid.

The solution is heated up to the boiling point. Barium Chloride ($BaCl_2$) was then added in the boiling solution to bring out sulfate in the form of Barite ($BaSO_4$). The solution was placed in an 85°C oven for 12 hours. In this process, the digestion process continues in which precipitation takes place to obtain Barite by gravimetric process. To obtain the soluble sulfate contents in the soil samples, the barite precipitated from this process was calculated. In order to segregate the small particles from the solution, (Puppala et al., 2002) used a smaller pore size filter of 0.1 µm and higher speed of centrifuging of 14,000 rpm with longer time. This modified method provided results that match the ion chromatography measurements. Hence, the modified method was adopted in the present research. Figure 4.12 presents the soluble sulfate test procedure used in this research.

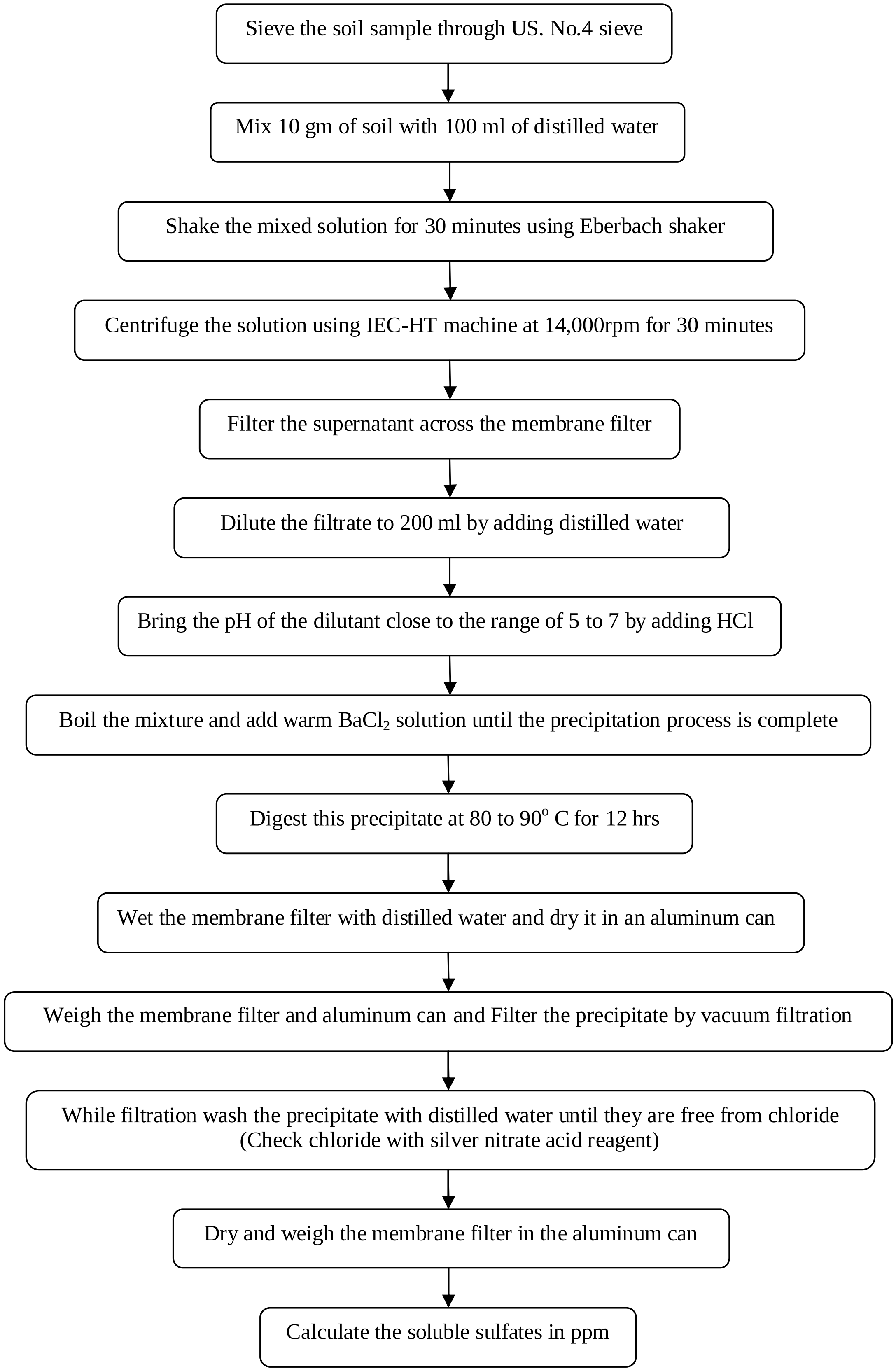


Figure 4.12 Soluble Sulfate Determination (Puppala et al., 2002)

4.4.3. Mineralogical Tests

Mineralogical tests were conducted in this research to evaluate the presence of ettringite in soil samples. X-Ray diffraction (XRD) and Scanning Electron Microscope studies were conducted on all the stabilized soil samples.

4.4.3.1 X-Ray Diffraction Test

Powder X-Ray diffraction method was used to identify the mineral composition of the soil samples. This test can identify the formation of Ettringite minerals in the stabilized soils. Soil samples were first obtained from the field and then oven dried and pulverized. Samples were collected randomly from different locations of all the treated sections. The treated samples were ground into a fine powder and sieved using sieve No. 230. These samples were then subjected to CuKα radiation at a speed of 0.05 degrees per minute with a graphite monochrometer over a 2θ range of 1^o to 80^o in order to read the basal spacing of the minerals present in the soil samples. The data was recorded and analyzed to determine the heaving problems in the present research study. Figure 4.13 illustrates the X-Ray diffraction test setup.

Figure 4.13 Illustration of X-Ray Diffraction Test Setup

4.4.3.2 Scanning Electron Microscope Study

The scanning electron microscope study was conducted to determine the morphology of ettringite in the stabilized soils. Samples were randomly collected from different stabilized sections for SEM analysis. The samples were copper coated and then scanned. Several digital images at different magnifications were recorded. In a scanning electron microscope, a tiny electron beam is scanned across the sample. Simultaneously, the generated signals being recorded, and an image is formed pixel by pixel. The ettringite minerals typically appear in needle shapes at higher magnifications.

EDAX was used to analyze chemical compositions of the specimen. In this technique, electrons are bombarded in the area of desired elemental composition. The elements present will emit characteristics x rays, which were then recorded on a detector.

4.5 Summary

This chapter provides a complete description of field studies, chemical tests and mineralogical tests conducted to assess the performance of stabilizers. The details of instrumentation design, installation of sensors, data collection procedures, elevation survey and DCP test details are also discussed. Field monitoring was performed for twenty six months on a weekly basis. The results analyzed to evaluate the performance of the different stabilizers which includes field and laboratory test results are discussed in Chapter 5.

CHAPTER 5

ANALYSIS OF TEST RESULTS

5.1 Introduction

This chapter presents a comprehensive summary of field and laboratory data acquired and its analysis. The instrumentation and elevation survey data acquired for a period of 26 months has been analyzed in this chapter. The results of other tests which include DCP tests, pH tests, soluble sulfate content determination, X-ray diffraction (XRD) analysis and scanning electron microscopic (SEM) study are also discussed.

The analysis is presented in different sections. The first section covers the instrumentation data collection and analysis of monitored data and presents the strain gauge and pressure cell data individually. The second section presents the results of elevation surveys carried out to monitor the heave related movements of the treated sections. The third section presents the results of DCP test, which was performed to analyze the strength properties of the treated subgrade soils at different time periods after stabilization. Section four covers the results of chemical tests, which includes pH test and soluble sulfate determination. The penultimate section presents the results of the two mineralogical studies conducted on stabilized soil samples collected from the field to detect the presence of ettringite mineral, which includes X-ray diffraction studies and SEM studies. This chapter concludes with the summary of all the tests conducted.

5.2 Analysis of Instrumentation Data

The instrumentation data monitoring included both strain gauge data and pressure cell data. The data collection was initiated in the month of January 2005 and was continued till March 2007. The data from the sensors were collected on a weekly basis and in case of extreme weather conditions such as thunderstorms, snowfall and hurricane conditions, the sensors' responses were collected within twenty-four hours of the event.

The vehicle used for the data collection was a mid-size passenger car with a gross weight of approximately 3,000 pounds. In order to ensure that the contact area with the pavement was equal throughout the data collection period, the tire pressure was kept at a standard pressure of 32 psi. The loading on the sensors was performed using different methodologies as explained in Chapter 4.

The data from each sensor was acquired over a ten-minute period. The raw data was obtained in ASCII format and was then converted to engineering data with Excel. This data was then subjected to filtering using MATLAB (Ver 7.0.4) and the filtered data was segregated section-wise and normalized by Excel. After filtering the data, their averages were calculated using the spreadsheet. The strain and pressure differences were calculated by subtracting the averaged data acquired without car on the sensors and with car placed over the sensors. The filtered and edited data was then compared against each of the sections.

5.2.1 Strain Gauge Data

The filtered strain data was compared against each of the sections in a bar graph and line graph formats. Figure 5.1 presents the bar graphs for the comparison of strains in the month of February 2005, July 2005, February 2006, July 2006 and March 2007. The data collected in February 2005 and July 2005 were the initial readings, collected by (Pillappa, 2005) immediately after opening the road to traffic.

Figure 5.2 presents the comparisons of the increase in vertical strains over the entire period of data collection.

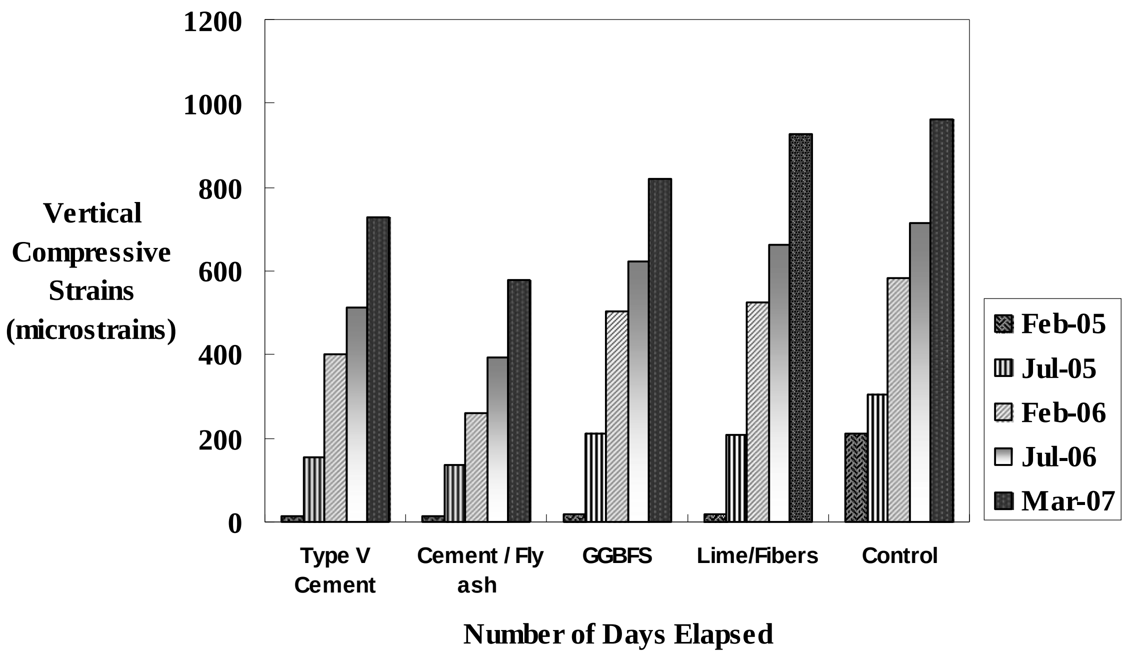


Figure 5.1 Comparisons of Strains at Different Periods of Data Collection

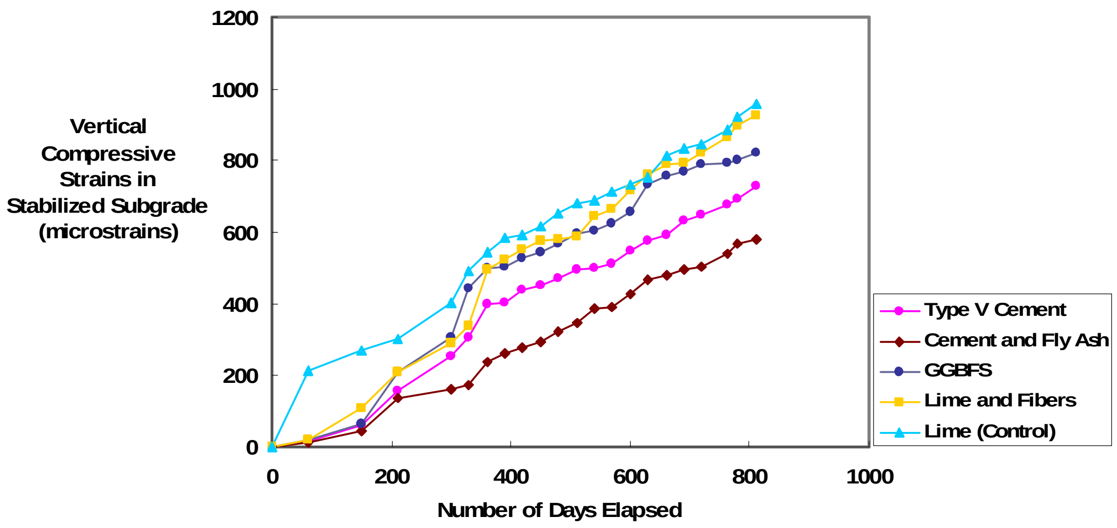


Figure 5.2 Comparison of Strains for the Entire Period of Data Collection

In both the figures illustrated above, we can observe that the lime/ fiber section and lime treated control section experienced large vertical strains under vehicular loads. Appreciable increase in compressions was observed in GGBFS section as well. Type V cement and type v cement with fly ash treated sections demonstrated the lowest strains explaining the stiffer material being formed and susceptible to low rutting with respective chemical treatment.

5.2.2 Pressure Cell Data

The same procedure was followed for data acquisition from pressure cells. However, in the case of pressure cells installed in GGBFS section, the data could not be acquired as the wires were severed and could not be repaired as they were installed underneath the pavement. It should be noted that the pressures measured at the bottom of the treated section during traffic loading is a combination pressures coming from all four wheel loads and a small static over burden pressures. Figure 5.3 shows the pressure cell results obtained in the month of February 2005, July 2005, February 2006, July 2006 and March 2007 respectively. The data in the month of February 2005 and July 2005 were collected by Pillappa (2005).

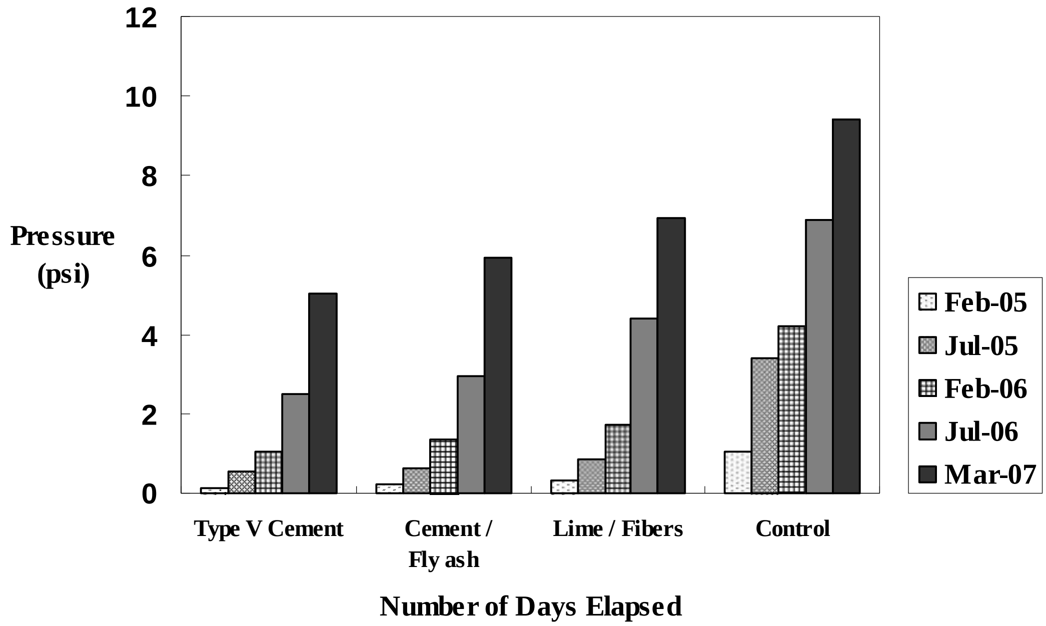


Figure 5.3 Comparisons of Pressures at Different Periods of Data Collection

It is interesting to note that the initial readings taken at the site showed small pressure values. However, with time, these values are increased which could be attributed to micro-cracking in the treated section. Figure 5.4 presents the comparisons of the increase in pressure over the entire period of data collection.

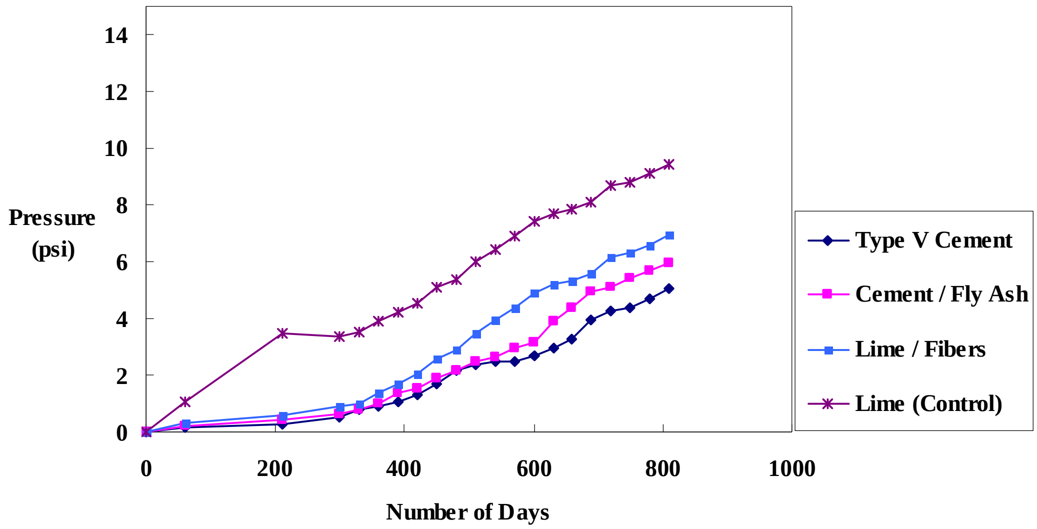


Figure 5.4 Comparisons of Pressures for the Entire Period of Data Collection

Overall, from the results presented, we can observe that the cement treated section has demonstrated good performance with lowest pressure readings, which is followed by, cement and fly ash cement treated section. Lime-fiber section demonstrated moderate performance. Lime treated section has shown the highest pressure readings. Therefore, Type V cement treated section and type v cement with fly ash treated sections are susceptible to low rutting under loading conditions. Changes in pressure cells are attributed to continuous permanent deformations of the subsoils. Nevertheless, the pressures are within expected levels under traffic loads except for lime treated sections which exhibits higher pressures due to lower strengths of this section.

5.3 Analysis of Elevation Survey Data

Elevation surveys were performed for over a period of twenty-six months in order to monitor the heave related movements of the stabilized pavement sections. The plan view of the elevation survey points and reference total station point has been discussed in detail in chapter 4. Figure 5.5 shows the elevation survey results.

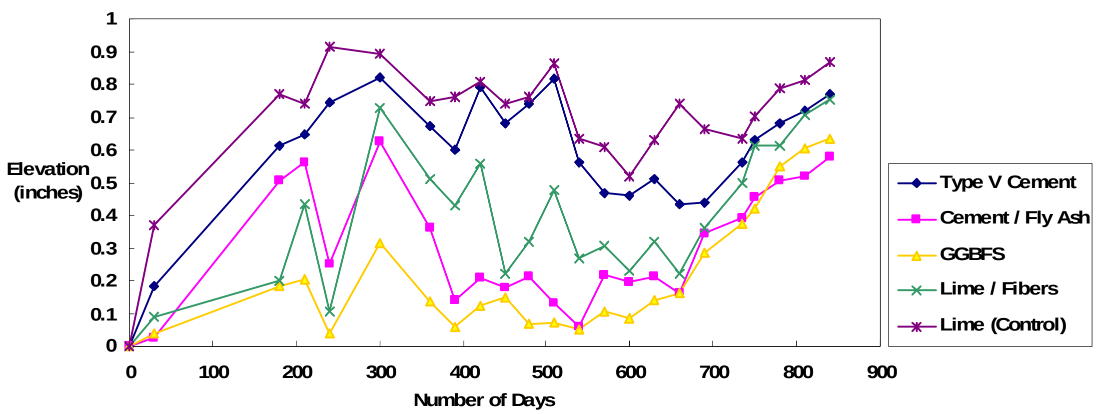


Figure 5.5 Elevation Survey Results

It can be observed from the graph that the lime control section experienced the highest heave related movements. The next highest heaving was observed in cement treated section and lime with fiber section respectively. Overall, GGBFS treated section and type v cement with fly ash section demonstrated good performance with low heave related movements. Fluctuations in the elevation surveys can be attributed to seasonal related soil movements that the test sections are experiencing during the monitoring period. Overall, it should be noted that lime treated section experienced lesser volume change movements after initial high changes when compared to other treatments.

5.4 DCP Test Results for Treated Sections

The DCP tests conducted after 28 days of curing period and the other after 26 months of pavement construction are presented in this section. Figure 5.6 represents the DCP equipment after 28 cm of penetration into the pavement courses.

Figure 5.6 Penetration of DCP Equipment into the Pavement Courses

Figure 5.7 shows the DCP results for the test conducted by (Enayatpour, 2005) after 28 days curing period. Figure 5.8 shows the DCP results after 26 months of pavement construction. The DCP test results are expressed in terms of dynamic penetration index or DPI of soil. DPI is the dynamic cone penetrometer index expressed in cm/blow. The DPI values were taken between 10 cm and 15 cm as the slopes of DCP results between these depths. Hence, in present research, the PR_1 and PR_2 depths are 10 and 15 cm respectively. Table 2.1 presents the DPI values for the DCP tests conducted after 28 days and 26 months of pavement construction.

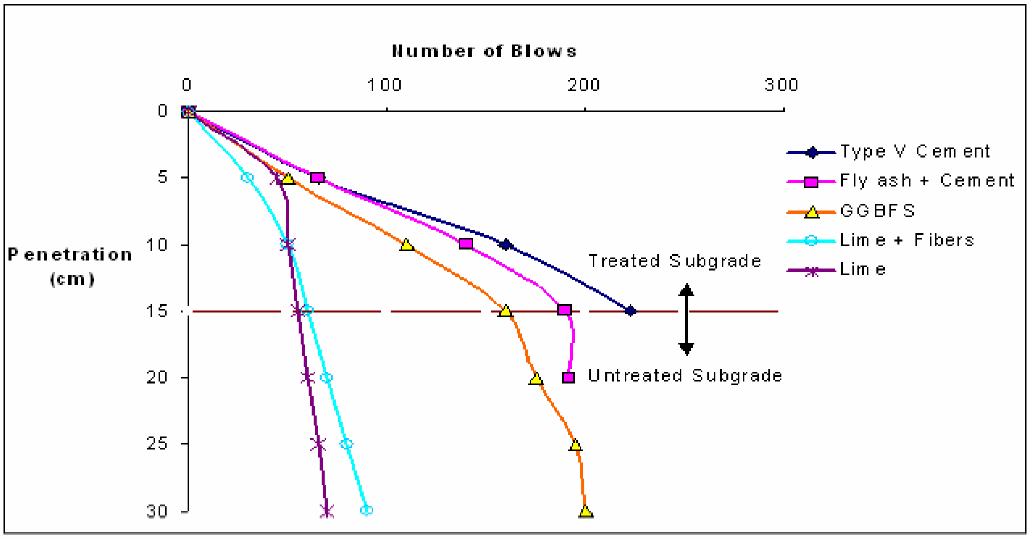


Figure 5.7 DCP Results after 28 days of curing (Enayatpour, 2005)

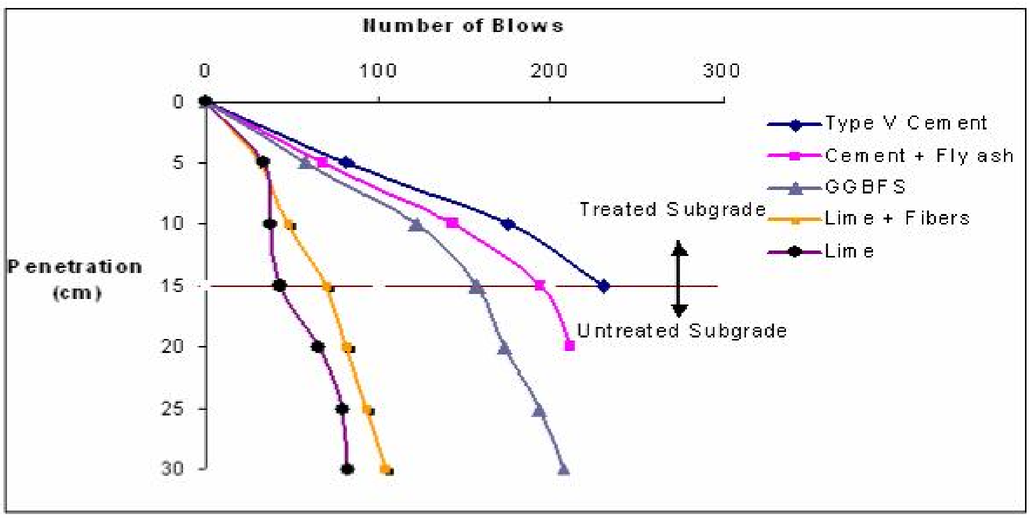


Figure 5.8 DCP Results after 26 months of Pavement Construction

Table 5.1 DPI Values for DCP Tests

Stabilizer	DPI (cm/blow)	
	After 28 days curing	**After 26 months**
Type V Cement	0.063	0.066
Cement and Fly ash	0.080	0.078
GGBFS	0.090	0.101
Lime and Fibers	0.330	0.263
Lime	1.000	1.110

As the two plots exhibit the slope or rate of "change of blows" versus penetration decreases beyond the 15 cm depth, which is an indicative of 15 cm stabilized base course beyond which, the number of blows required to advance the cone penetrometer decreased. One more observation from the DCP results as seen in the plot is that the DCP tests had to be stopped in certain tests due to difficulties in advancing the cone through stiff treated layers.

From figure 5.14 representing results after 28 days of curing, the number of blows for 15 cm of penetration for treated layers with cement, fly ash with cement, GGBFS, lime-fibers and lime (control) sections are 223, 190, 110, 50 and 50 respectively. Figure 5.15 shows the same trend even after 26 months of pavement construction. The results in figure 5.14 and 5.15 have the same trend as the strength enhancement is rapid in the first two weeks of curing and then it becomes slower. The results indicate that the cement treatment, cement with fly ash and GGBFS treatments exhibits better strength gain when compared to lime – fibers and lime (control) sections. One important observation from both the plots and the DPI values is that there is no considerable deterioration seen in strength in any of the treated layers due to climatic changes and varying traffic volume.

5.5 Chemical Tests

The results for pH test and soluble sulfate determination are presented in the following sections in order to predict the ettringite formation and sulfate induced heave.

5.5.1 pH Test Results

pH tests were conducted on all the treated samples at two different time periods to understand acidic and basic conditions during soil stabilization process. The pH reading for all the specimens were conducted at room temperature with the pH meter. The first set of pH tests was conducted by (Ekarin, 2000) before the construction of stabilized pavement.

The second set of pH tests was conducted after 26 months of stabilized pavement construction. These tests were conducted to predict the formation of ettringite, which is influenced by the pH of the treated subgrade soil. Table 5.2 presents the pH test results before and after construction of stabilized pavement. In chemically treated soils, the chemical compounds, which include calcium, alumina, and sulfates, are dissolved into a basic solution at pH greater than 10, leading to the formation of Ettringite (Hunter, 1988).

Table 5.2 pH Test Results for Stabilized Soils

Stabilized Soil	pH	
	Before Pavement Construction	**After 26 months of pavement construction**
Type V Cement	12.4	12.8
Cement and Fly ash	12.0	12.3
GGBFS	11.3	11.7
Lime and Fibers	13.5	13.4
Lime (Control)	13.6	13.6

From the pH results, it can be observed that the pH range for all the stabilized subgrade soils is in between 11.3 – 13.57. As the pH for all the stabilized soils are above 10, it can be mentioned that all stabilizers are still present and no major leaching was recorded. It also leads to a concern that ettringite formation may still occur since alumina disassociation from clay minerals typically happens at high pH conditions. Hence, this may result in heaving in the future.

5.5.2 Soluble Sulfate Test Results

It can be noted that the sulfate content as a percent of dry weight of soil needed to induce heaving varied from 200 ppm to as high as 5,000 ppm. Puppala et al., 2005 showed that even at low sulfate levels, heaving will be possible if the void space is small and ettringite mineral growth is continuous. Soluble sulfate tests were conducted on both untreated and treated soils collected from the field. Table 5.3 presents the soluble sulfate content for the stabilized soils. It is interesting to note that these levels are smaller in the treated soils, which could be attributed to possible conversions of them into insoluble sulfate mineral forms such as ettringite and also to sampling variations of sulfates in the subsoils, which are found in pockets.

Table 5.3 Soluble Sulfate Content for Stabilized Soil Sections

Stabilizers	**Soluble Sulfate Content (ppm)**
Untreated	4737.0
Type V Cement	730.9
Cement and Fly ash	396.1
GGBFS	362.2
Lime and Fibers	1014.0
Lime (Control)	1128.0

5.6 Mineralogical Characteristics

5.6.1 X-ray Diffraction (XRD) Analysis

XRD Analysis was conducted on all the treated soil samples collected from the field in order to study the crystalline mineral, ettringite formation. This test was mainly conducted to identify ettringite mineral formation, which is known to cause sulfate induced heave. This has been the prime concern in the present research. Figure 5.9 through 5.13 presents the XRD analysis charts for the five treated sections. Table 5.4 through 5.8 shows the presence of ettringite at their corresponding d-spacing values.

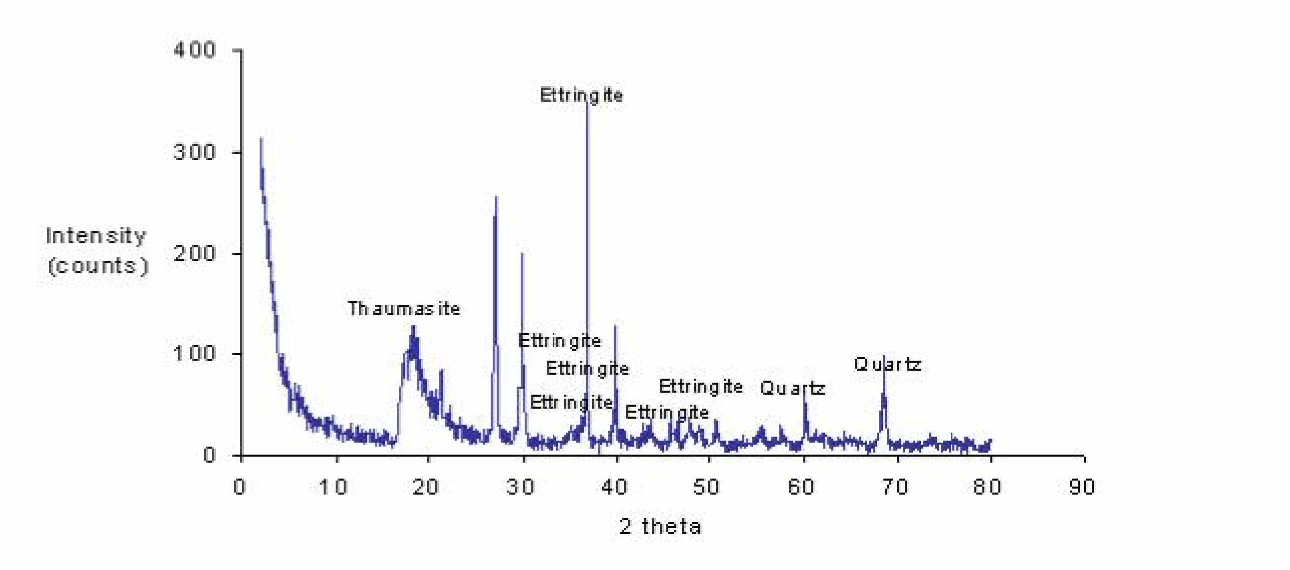


Figure 5.9 XRD Analysis for Type V Cement Treated Section

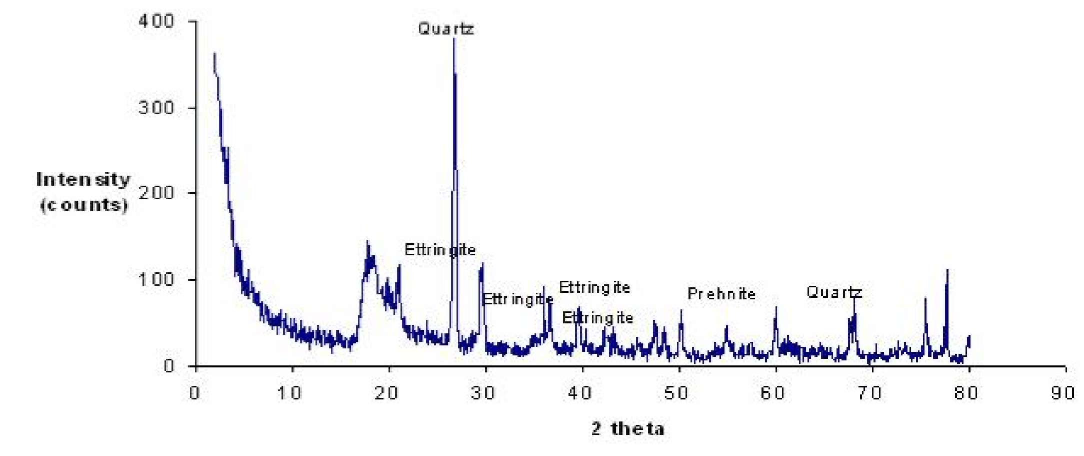


Figure 5.10 XRD Analysis for Cement and Fly ash Treated Section

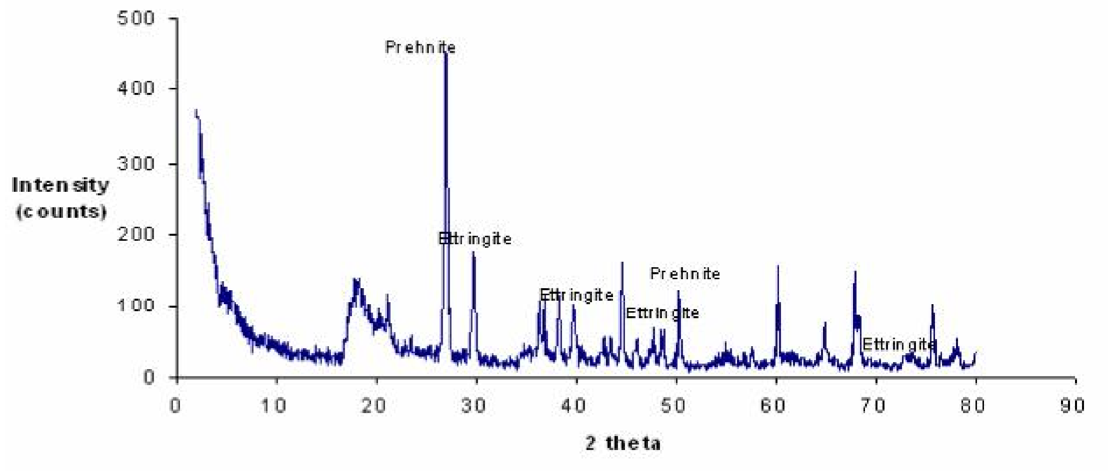


Figure 5.11 XRD Analysis for GGBFS Section

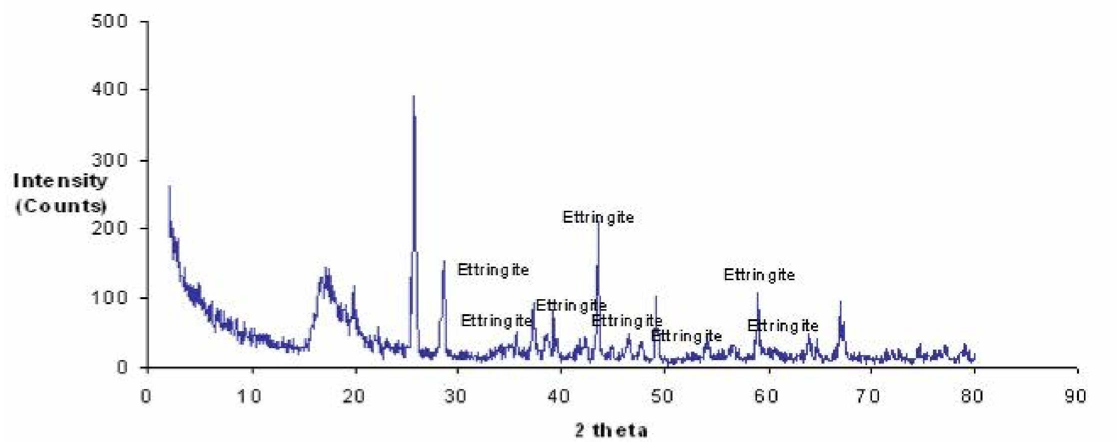


Figure 5.12 XRD Analysis for Lime-Fiber Section

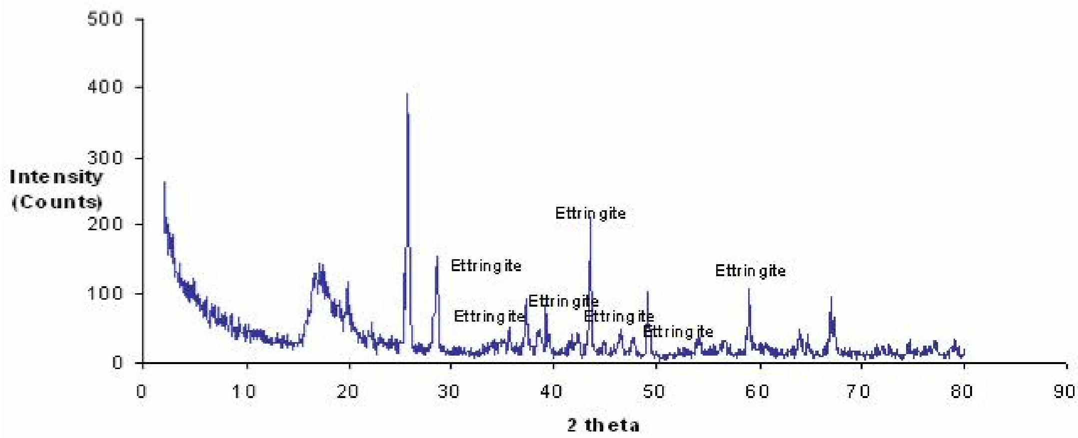


Figure 5.13 XRD Analysis for Lime (Control) Section

Table 5.4 XRD Results for Type V Cement Treated Section

Intensity (%)	d - Spacing (A)	Quartz	Ettringite (1)	Ettringite (2)	Thaumasite	Prehnite
100	2.4326	×	✓	×	×	×
72	3.2995	×	×	×	×	✓
33.4	2.2595	×	×	×	×	×
16	1.5382	✓	×	×	×	✓
14.2	4.8862	×	×	×	✓	×
9.6	1.9521	×	×	✓	✓	×
9.3	1.9792	×	×	✓	×	×
9.3	1.3765	×	×	×	×	✓
7.2	1.8013	✓	×	✓	×	×
6.3	2.074	×	✓	✓	×	×
5.4	1.6562	×	×	×	×	✓

Note: (✓) Indicates the probable presence of the mineral
(x) Indicates the absence of the mineral

Table 5.5 XRD Results for Cement and Fly ash Treated Section

Intensity %	d - Spacing (A)	Quartz	Ettringite (1)	Ettringite (2)	Thaumasite	Prehnite
100	1.6601	×	✓	×	×	✓
17	1.519	×	×	×	×	×
16.4	6.3736	✓	×	×	×	×
15.2	2.8116	×	✓	×	×	✓
14	13.7296	×	×	×	×	×
13.1	2.1254	×	✓	✓	×	✓
12.8	1.9678	×	×	×	×	×
12.5	4.0363	×	✓	×	×	✓
10.4	1.4648	×	×	×	×	✓
8.9	2.5974	×	×	×	×	×
8.6	2.6728	✓	×	×	×	×

Note: (✓) Indicates the probable presence of the mineral
(x) Indicates the absence of the mineral

Table 5.6 XRD Results for GGBFS Treated Section

Intensity %	d - Spacing (A)	Quartz	Ettringite (1)	Ettringite (2)	Thaumasite	Prehnite
100	3.3053	×	×	×	×	×
25.2	1.8136	×	✓	×	×	✓
20.7	2.3474	×	✓	×	×	×
17.1	2.4742	×	×	✓	×	×
17.1	4.0363	×	×	×	×	×
12.6	4.1952	×	×	×	×	✓
7.8	2.114	×	✓	×	×	×

Note: (✓) Indicates the probable presence of the mineral
(x) Indicates the absence of the mineral

Table 5.7 XRD Results for Lime- Polypropylene Fiber Section

Intensity %	d - Spacing (A)	Quartz	Ettringite (1)	Ettringite (2)	Thaumasite	Prehnite
100	1.3737	✓	×	×	×	×
51.8	2.0294	×	✓	×	✓	×
29.4	2.345	×	✓	✓	×	×
14.6	4.8576	×	✓	✓	×	×
10.6	3.2379	×	✓	✓	×	×
9.6	2.9561	✓	×	×	×	×
7.8	2.2436	×	✓	✓	×	×
7.1	5.216	×	×	×	✓	×
6.4	2.2855	✓	×	×	×	×

Note: (✓) Indicates the probable presence of the mineral
(x) Indicates the absence of the mineral

Table 5.8 XRD Results for Lime (Control) Section

Intensity %	d - Spacing (A)	Quartz	Ettringite (1)	Ettringite (2)	Thaumasite	Prehnite
53.4	2.0741	×	✓	×	×	✓
24.8	1.5623	×	✓	✓	×	×
23.4	5.287	×	×	×	×	✓
20.1	2.4063	×	✓	×	×	×
12.9	5.5311	×	×	×	✓	×
9.4	1.9475	×	✓	✓	×	×
9.1	1.4555	×	×	×	×	×
8.3	2.5155	×	✓	✓	×	×
8	2.1349	×	✓	×	×	×
7.2	2.1653	×	✓	✓	✓	×
6.3	1.9051	×	✓	×	×	×
5.8	1.6952	×	✓	×	×	×
5	56.457	×	✓	×	×	×

Note: (✓) Indicates the probable presence of the mineral
(x) Indicates the absence of the mineral

It can be observed from the above charts and tables, that the ettringite mineral is present in all stabilized soil sections. Ettringite (1) contains Calcium Aluminum Sulfate Silicate Carbonate Hydroxide Hydrate and ettringite (2) contains Calcium Aluminate Sulfate Hydroxide Hydrate. The presence of other minerals can also be seen which includes Quartz, Thaumasite and Prehnite minerals in some of the stabilized soils. Thaumasite is a form of ettringite, which forms at cold temperatures, and contains Calcium Carbonate Silicate Sulfate Hydrate. This mineral also contributes to sulfate induced heave distress.

Prehnite consists of Calcium Aluminum Silicate Hydroxide and is a known stabilization compound. Its traces are found in all treated soils suggesting that stabilization reactions involving CSH did occur in the present treated soils.

From the results it is also evident that the presence of ettringite is more evident as more traces match with d-spacing of pure ettringite of both lime-fiber section and lime (control) sections. Type V Cement with Fly ash and GGBFS sections showed the ettringite traces, which was followed by Type V Cement Section. Higher heave related movements could be seen in elevation survey results in the case of lime-fiber and lime (control) sections, and parts of these could be attributed to the formation of ettringite and its heaving. However, current practice does not specify direct evaluations of sulfate heave in the field conditions. Future research should focus on developing such methods, probably utilizing non-destructive field studies.

5.6.2 Scanning Electron Microscope (SEM) Analysis

The stabilized soil samples collected from the field were subjected to SEM analysis. Figure 5.14 through 5.18 presents the SEM images for all the five stabilized soils. Though images show some traces of ettringite and thaumasite minerals, they are not definitive and difficulty to identify. Other possible reasons could be the loss of minerals due to drying of the samples or the dry periods during the sampling.

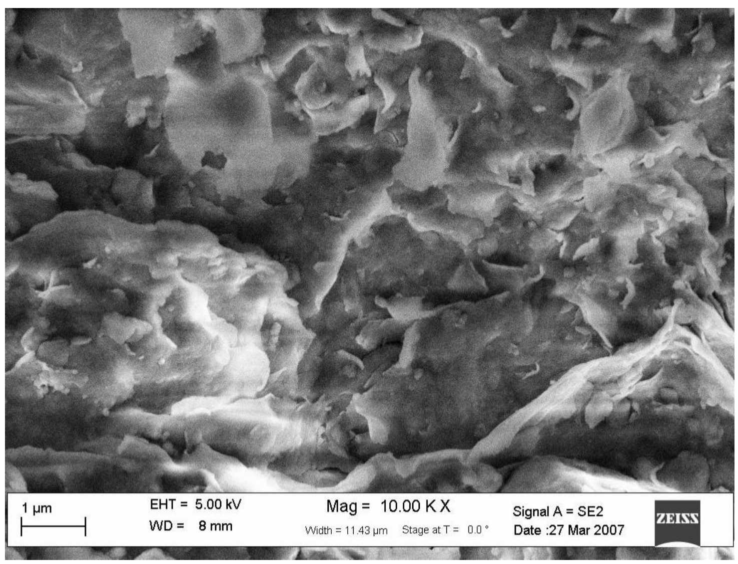


Figure 5.14 SEM Image for Type V Cement Treated Soils

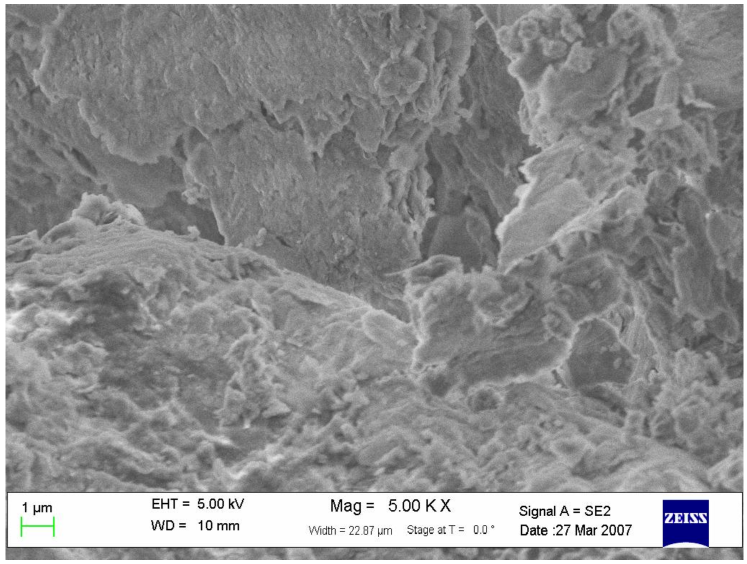


Figure 5.15 SEM Image for Cement with Fly ash Treated Soils

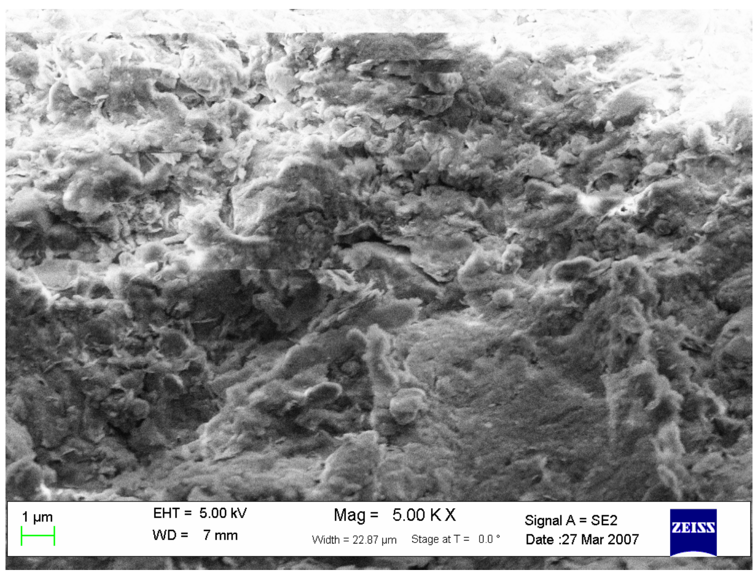


Figure 5.16 SEM Image for GGBFS Treated Soils

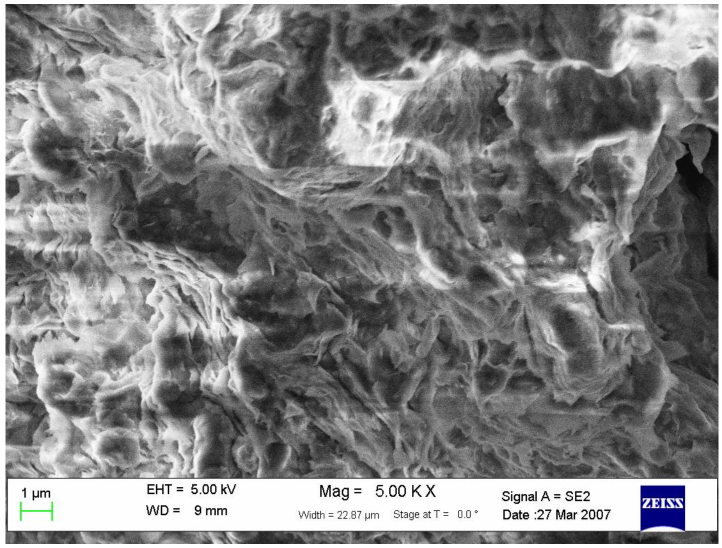


Figure 5.17 SEM Image for Lime with Fibers Treated Soils

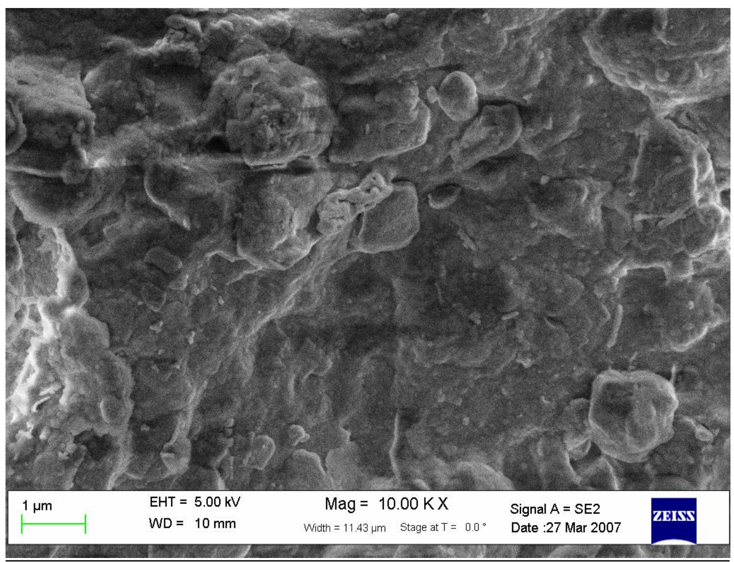


Figure 5.18 SEM Image for Lime (Control) Treated Soils

5.6.3 Energy Dispersive X-Ray Microanalysis (EDAX) Results

In the present research, specimens were tested using Energy Dispersive X-ray Analysis (EDAX) technology to identify the chemical species present in the treated soils collected from the field. EDAX results are presented from figure 5.19 through 5.23 for all the treated soils.

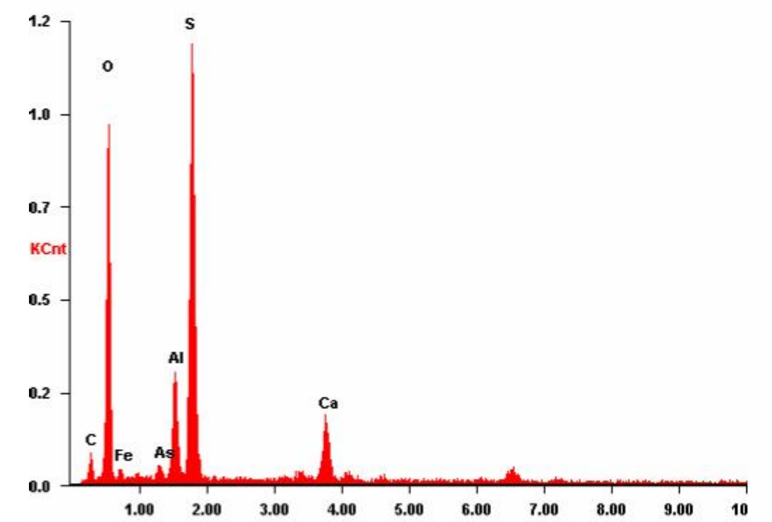


Figure 5.19 EDAX Results for Type V Cement Treated Soils

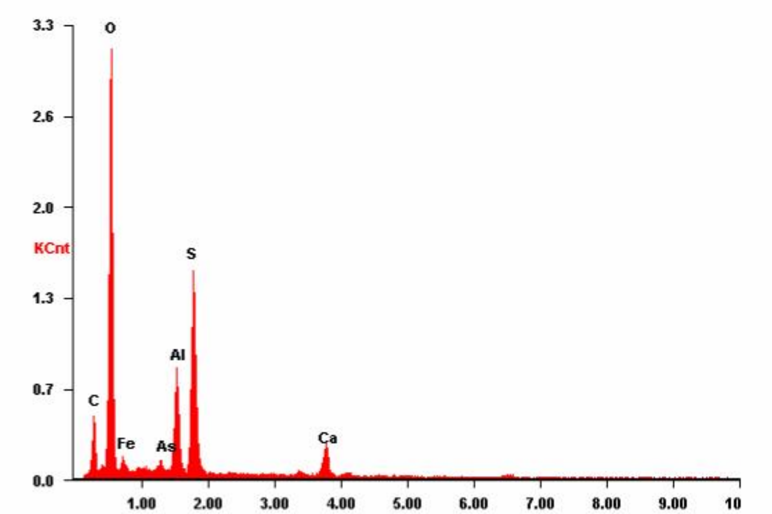


Figure 5.20 EDAX Results for Type V Cement and Fly ash Treated Soils

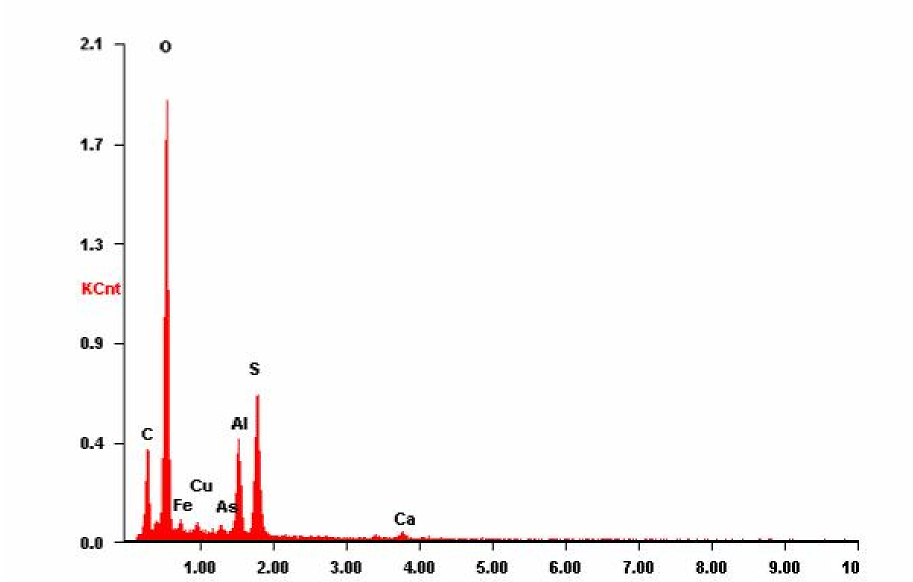


Figure 5.21 EDAX Results for GGBFS Treated Soils

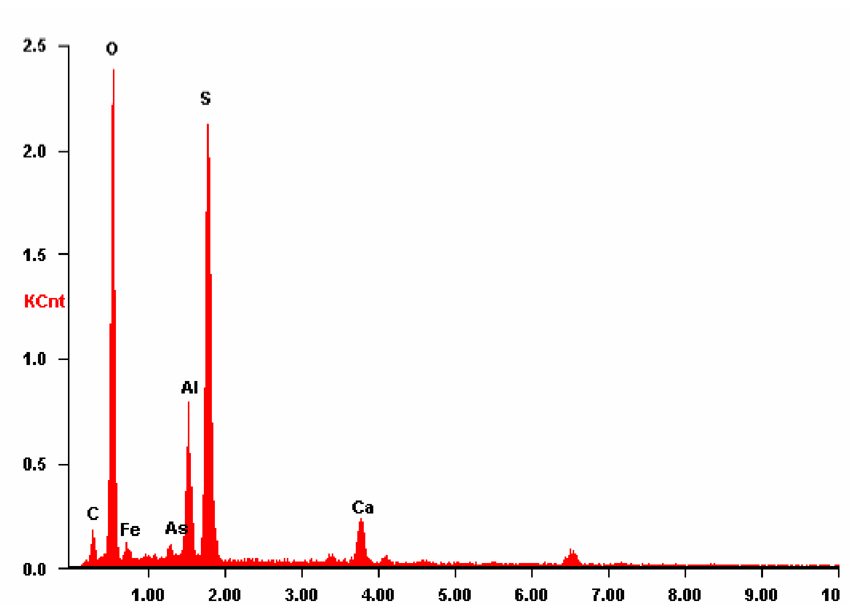


Figure 5.22 EDAX Results for Lime and Fibers Treated Soils

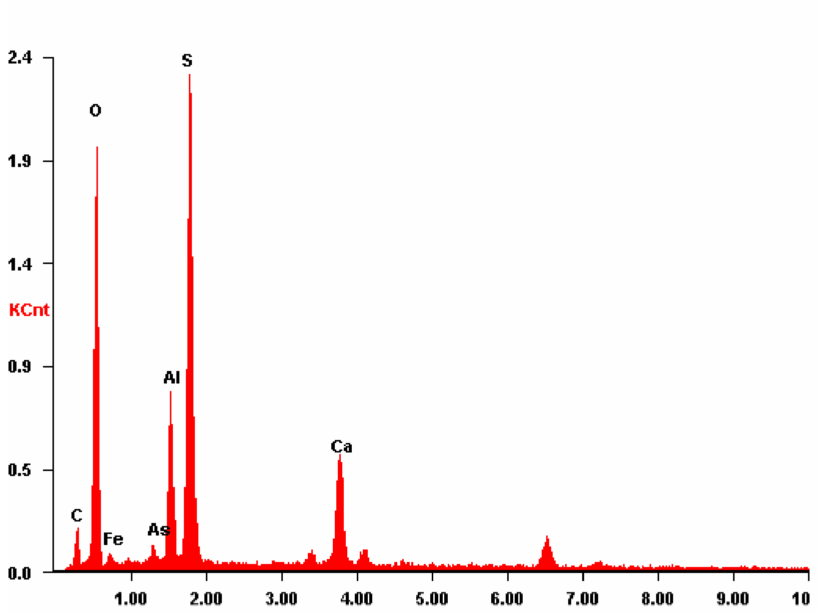


Figure 5.23 EDAX Results for Lime (Control) Treated Sections

EDAX analyses on treated samples as presented in figures above showed all the ingredients mainly, suphur (S), aluminum (Al), and calcium (Ca) needed for ettringite formation. Higher peaks in a spectrum, represents higher concentration of the element in the specimen. Hence, it can be mentioned that ettringite formation is possible in the present treated soils. However, the mere formation may not result in heaving since high amounts of formation and then hydration will only result in sulfate heaving. At present, in type V cement, type v cement with fly ash and GGBFS treated soil, this heaving appears to be small at the current time of monitoring. Hence, it can be mentioned that based on the current monitoring period of 26 months, sulfate heaving was not highly evident in these treated soils. Higher heaving in the lime treated sections from elevation surveys and mineralogical analyses point out that the ettringite induced heaving may have lead to certain amount of overall heaving. However, this heaving is still small and not raised to levels that could cause cracking to pavements.

5.7 Summary of Performance of Stabilizers

Table 5.9 presents the summary of performance of stabilizers based on field and laboratory studies. These rankings are based on the trends noted in the analysis. Overall, based on the summary provided in the following table, Type V Cement, Type v Cement with Fly Ash and GGBFS sections have performed well. The lime-fiber section provided adequate performance whereas the lime (control) section has demonstrated poor performance.

Table 5.9 Summary of Qualitative Performance of Stabilizers

Study Type	**Type V Cement**	**Cement and Fly ash**	**GGBFS**	**Lime and Fibers**	**Lime (Control)**
Instrumentation Studies	H	H	M	M	M
Elevation Studies	M	H	H	M	L
DCP Studies	H	H	M	L	L
Laboratory Studies	H	H	H	M	L
Mineralogical Studies	M	M	M	L	L

Note- H – High Performance; M – Medium Performance; L – Low Performance

5.8 Summary

This chapter provides a comprehensive summary of field and laboratory data analyses, which includes field instrumentation results, elevation surveys and DCP test results. This chapter also summarizes the chemical and mineralogical characteristics of the stabilized soils. From the above analyses, Type V Cement and Type V Cement with Fly ash sections performed the best based on a long term study which was followed by GGBFS section.

CHAPTER 6

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

6.1 Summary

The main objective of this research project was to assess the performance of four novel stabilization methods in the field conditions in order to select an ideal stabilization method or methods for stabilizing sulfate rich soils to minimize sulfate induced heave distress in Southeast Arlington. These four stabilization methods namely, type V cement, type v cement with fly ash, GGBFS, and lime with fibers were compared against a control section treated with lime. The assessment program included both field and laboratory studies in which the stabilized pavement sections were monitored and evaluated based on which ideal stabilization methods for the present soil conditions can be selected.

It should be noted that the laboratory tests were earlier conducted by Chavva (2002) and Ramakrishana (2002), which indicated that all the four stabilizers improved the liquid limits, plasticity index values unconfined compressive strength, swelling and shrinkage potentials of soils of Southeast Arlington. In the field, Type V Cement with Fly ash proved to be the most effective treatment by increased strength and reduced swell and shrinkage potentials. The second most effective treatment was Type v Cement which was followed by GGBFS treatment. Compared to the earlier treatments, Lime-

Fiber and Lime (control) treatment methods exhibited moderate and poor improvements in enhancing the soil properties, respectively.

6.2 Conclusions and Recommendations

This study, which is primarily a field monitoring study, yielded the following major conclusions:

- From the strain gauge readings, it is evident that all sections are undergoing compression related strains with the continuing traffic. Among these total strains experienced so far, lime control section experienced the maximum amounts of axial compression strain (959.6 micro strains), which was followed by Lime-Fibers, GGBFS and Type V Cement sections. Type V Cement with Fly Ash section experienced the lowest amount of the same strain.
- Based on the pressure cell responses, the amount of pressure distributed to underlying soils increased with time. This increase was attributed to permanent strain experienced in the treated subgrades. Lowest pressure transfer was noted for 'type V cement' and 'Type V cement with fly ash' treated sections indicating these sections will undergo low compression due to low magnitudes of pressures transmitted to these layers. Lime-fiber section demonstrated moderate pressures whereas lime treated section have shown the highest pressure readings under traffic loading. This means the Type V Cement and Type V Cement with Fly Ash treated sections have potential to undergo low rutting during service.
- From the elevation survey results, Lime treated control section exhibits high swell and shrink movements with seasonal changes. The next high movements were recorded by Type V Cement, Type V Cement - Fly Ash and Lime - Fiber

stabilized sections. GGBFS treated section performed the best with low soil related movements. Though large heave movements were recorded in two sections, no visible pavement cracking or roughness of pavement sections (as a function of pavement undulations) were present on all these test sections.

- The DCP test results indicate that the Type V Cement treatment, Type V Cement with Fly Ash treated sections exhibits higher strength gain, which is followed by GGBFS treated section. Lime – Fibers and Lime (control) sections demonstrate lower strength gain when compared to other treated sections. From the measured DPI values, it can be observed that there is no considerable deterioration in strength in any of the treated layers due to climatic changes and varying traffic volume in the last two and half years after the stabilization.
- From the chemical tests conducted, it observed that the pH range for all the treated soils is in between 11.3 – 13.6. The pH conditions indicate that no major leaching was recorded in the present test sections and hence it can be mentioned that the stabilization was still intact in this short time period of monitoring. As the pH of all the treated sections is above 10 these pH conditions can support the growth of ettringite mineral. Lime-Fiber, Lime (control) sections have demonstrated higher pH values when compared against other treated sections.
- From, soluble sulfate test results, it can be noted that the there is reduction in sulfate levels in treated soils which could be attributed to possible conversions of sulfates into insoluble sulfate minerals including ettringite and thaumasite forms and also due to variations of sulfate levels due to sampling locations.

- X-ray diffraction analysis showed traces of ettringite in all the treated sections. The presence of ettringite is clearly evident in Lime-Fiber section and lime (control) sections due to larger number of matches with the basal spacings of ettringite. Other sections also showed the traces of ettringite formation, which implies that sulfate heave is still possible in the present test sections. Lime (control), Lime-Fiber and Type V Cement treated sections experienced higher heave related movements and some part of it could be attributed to the formation of ettringite and its hydration.
- From EDAX results, it is clearly evident that all treated soils have all the chemical components necessary, i.e. calcium, alumina, and sulfates to produce sulfate heaving mineral, ettringite. As a result, it can be mentioned that ettringite formation is possible in the present treated soils. Currently, in Type V Cement, Type V Cement with Fly Ash and GGBFS treated soils, this heaving appears to be small. Based on mineralogical results, it can be mentioned that in the long term study period, the higher heaving demonstrated in Lime treated sections may be due to sulfate heaving which is still not exceedingly apparent in these treated soils. Ettringite induced heaving may have resulted to certain amount of overall heaving which is still at lower levels and could not cause cracking to pavements.

From the results presented in chapter 5 and the summary and conclusions listed above, Type V Cement with Class F Fly Ash stabilized section has demonstrated consistent performance with low compressive strains, low pressures at the bottom of the treated subgrade, higher strength properties from DCP tests and lower heave related movements.

Overall, based on the long term field studies and laboratory tests, it can be mentioned that the ‘Type V Cement with Fly Ash’, ‘Type V Cement’, and ‘GGBFS’ treated sections provided good performance with small distress problems. Though enhancements are not as high as the cement and GGBFS sections, both ‘lime with fiber’ and ‘lime (control)’ sections performed adequately in the field with some heave related concerns. If laboratory data is included in the overall assessments, Lime section performed poorly.

Continuous monitoring and periodic assessments with profiler and other non-destructive pavement related devices will further help in the final assessments of stabilization potentials of chemicals considered in this research.

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