# FIELD AND EXPERIMENTAL STUDIES TO ASSESS THE PERFORMANCE OF STABILIZED EXPANSIVE CLAY

by

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ABSTRACT

FIELD AND EXPERIMENTAL STUDIES TO ASSESS THE

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EXPANSIVE CLAY

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A vast majority of the natural expansive soils are montmorillonite-rich clays, over consolidated clays and shales. Certain sulfate rich soils such as those encountered in south Arlington also exhibit heaving after chemically treating them with calcium based stabilizers including cement and lime. Both swelling and softening of these soils induce damage to overlying pavement infrastructure.

The present research was conducted at the University of Texas at Arlington as a part of a research study for the City of Arlington to explore and develop alternate stabilization methods for sulfate rich soils of South Arlington. Based on literature review of sulfate rich expansive soil treatments and comprehensive laboratory studies,

the following four novel stabilizers were recommended for field treatment studies: Sulfate Resistant Cement (Type V), Low Calcium Class F Fly Ash with Type V Cement, Ground Granulated Blast Furnace Slag and Lime mixed with Polypropylene Fibers. The laboratory studies focused on Atterberg limits, unconfined compressive strength, resilient modulus, linear shrinkage, vertical swelling of chemically treated south Arlington soils.

The selected five stabilizers including control treatment with lime were used to modify subsoil near Harwood road in South Arlington. Rigid pavements were then constructed on the stabilizers' sections and these sections were instrumented with strain gages and pressure cells. Field monitoring of pavement instrumentation was then performed to address both pressure transfer mechanisms and stabilized material compression behavior under traffic loads. Data monitoring was conducted for a period of seven months.

Elevation surveys were also conducted to evaluate swell movements of the treated subsoils. X-ray diffraction analyses were also conducted on treated subgrade samples to address the formation of Ettringite mineral. Based on all these evaluations for the short time period, the performance of four stabilization methods were addressed against the control lime section. Overall, cement treatment method followed by the other three alternate methods provided stable and uniform pavement support to the traffic.

TABLE OF CONTENTS

LIST OF ILLUSTRATIONS

LIST OF TABLES

# CHAPTER 1

# INTRODUCTION

## 1.1 Introduction

The swell and shrinkage distinctiveness of expansive soil causes significant damage to structures such as buildings and pavements. This damage can be attributed to moisture fluctuations caused by seasonal variations. Volumetric changes weaken the subgrade by inducing cracking which metes out damage to the overlying structures. A vast majority of the expansive soils are montmorillonite-rich clays, over consolidated clays and shales (Nelson and Miller, 1992).

In the United States, several states have been affected by subgrade-related heaving and shrinkage problems for many years (Nelson and Miller, 1992). Likewise, in the regions of North Texas, and more specifically in the Dallas - Fort Worth Metroplex, problematic soils are encountered as they exhibit low strength, high swell, and shrinkage characteristics. Both low strength and volumetric movements deteriorate the subgrade and, in due course, influence pavement weakness by the development of cracks caused by differential heave settlements. Furthermore, these soils contain high amounts of sulfates which react with the calcium component of stabilizers to form ettringite. Ettringite undergoes heaving in the presence of moisture - a phenomenon termed as sulfate-induced heaving (Hunter, 1988).

Presently, the U.S. alone spends billions of dollars annually on the repair and maintenance of afflicted buildings, roads and other structures built on expansive soils (Nelson and Miller, 1992). The City of Arlington allocates resources for the maintenance and repair of pavements. In order to lower costs of rehabilitation, it is mandatory to investigate new, innovate and alternate stabilization methods to construct stronger and more stable foundation subgrade with the least amount of heave.

The soils of Southeast Arlington contain high amounts of sulfates which can lead to potential ettringite formation when traditional stabilizers are used. Additionally, these soils exhibit high PI values and show evidence of natural expansive behavior. These problems are further compounded by the seasonal temperature variations typical to North Texas. This has prompted the City of Arlington to explore novel and innovative methods of subgrade stabilization.

The present research project was conducted at the University of Texas at Arlington as a part of the exploration project for the City of Arlington. Mainly, field investigations and performance monitoring of pavement with different subgrades were carried out. Based on a comprehensive literature review of expansive soil treatment methods, the following four novel stabilizers were identified and preferred:

- Sulfate Resistant Cement (Type V)
- Low Calcium Class F Fly Ash with Cement
- Ground Granulated Blast Furnace Slag, and
- Lime mixed with Polypropylene Fibers

These stabilizers were considered in this research as they either showed potential as effective stabilizers or were known to reduce heaving problems in sulfate soils (Hawkins et al., 1998, Kota et al., 1996, Puppala et al., 1999, Viyanant, 2000).

While these stabilizers provided good results in the laboratory, it is required to evaluate their performance in real field conditions. Field evaluations are essential since soils in natural conditions undergo moisture and temperature fluctuations which may affect the stabilization process. Furthermore, field stabilization studies utilizing sensor instrumented pavements will provide the competence of each stabilizer in controlling pavement distresses such as differential heaves or bumps, rutting and pavement cracking. This is significant because the initial data obtained from the site is essential for verifying the stabilization effects of the soil.

The preliminary data for this thesis was obtained from various sources. The evaluation of the novel stabilizers for providing effective stabilization of Southeast Arlington soils was conducted at The University of Texas at Arlington (Wattanasanticharoen, 2000). The laboratory characterization, focusing on resilient modulus of the subgrade after stabilization, was in accordance with the 1986 Interim and 1993 American Association of State Highway and Transport Officials (AASHTO) design guide, which recommended a resilient modulus parameter for the design of flexible pavements (Ramakrishna, 2002). The laboratory characterization of field subgrade soil was also performed (Chavva, 2002). Subsequently, research was conducted to acquire the proper instrumentation needed to study the effects of stabilization in the field. These were then calibrated, and a proper method of installation

was arrived at, which in turn could act as precursors to face and address the problems which may arise during field installation. The final outcome was to redesign and develop an instrumentation system that can be used in the field to provide data to address the effectiveness of various stabilization methods in providing strength and volume change-related aspects of soils (Mohan, 2002).

The focus of this thesis is mainly on the procurement and analysis of data obtained from the sensors installed in the field (pressure cells and strain gauges). Raw data was collected by means of a data acquisition system which was specifically selected to ensure that it captured the sensor data from various channels that are linked to the sensors. Elevation surveys were also conducted during the course of this research to calculate the amount of relative elevation differential in each section and are compared.

## 1.2 Research Objectives

The main objective of the present thesis was to analyze and present the data acquired from real field conditions with regard to the performance of the alternate stabilizers used. The secondary objectives were to conduct elevation surveys to assess the heave related elevation changes for the present test sections and also evaluate the Ettringite mineral formation in the chemically treated material.

The following specific tasks were carried out during the course of the proposed project:

1. A comprehensive literature review on selected soil stabilization methods.
2. Suitable scheduling and installation of selected instrumentation in all the different stabilized sections.

3. Data verification from the sensors to ascertain its correctness with respect to the loads being applied.
4. Acquisition of load and strain data from sensors on a regular basis.
5. Carry out elevation surveys on all the treated sections, with the sections being marked at regular intervals for consistency.
6. Tabulate and analyze the control and other section test data and convert the sensor readings to engineering units for effective understanding.
7. Preparation of a final report summarizing findings and drawing conclusions.

## 1.3 Thesis Organization

This thesis is composed of six chapters: introduction (chapter 1), literature review (chapter 2), laboratory data of Harwood Road soil (chapter 3), design of instrumentation and field construction methodology (chapter 4), field data monitoring and analysis (chapter 5) and summary and conclusions (chapter 6).

Chapter 1 provides the introduction, objective and thesis organization. The problem definition and objectives of research and the preliminary investigations are briefly mentioned here.

Chapter 2 gives a summary of the background of various stabilizers used in this research project. It also provides a summary of the different instrumentation methodologies and the reasons for the selection of the chosen sensors with their application areas.

Chapter 3 presents the basic soil physical properties of the control soil. These were carried out according to the American Society for Testing Material (ASTM)

standards and these results are included. It also highlights the laboratory instrumentation, sample preparation, test methods, procedures and data analysis methods used in the research.

Chapter 4 presents the criteria considered for the selection and procurement of the gauges and sensors. Field placements methods and difficulties encountered are also mentioned in detail.

Chapter 5 details the monitoring of sensors, the implementation of elevation surveys and the laboratory X-ray diffraction analysis results.

Chapter 6 provides the summary, findings and conclusions of the research study.

# CHAPTER 2

# LITERATURE REVIEW

## 2.1 Introduction

The literature reviewed in this chapter is a compilation obtained from journals, books, conference proceedings and electronic sources. First, an introduction to expansive soils which induce distress in pavements is presented. Next, the four selected stabilization methods along with their advantages and limitations are described extensively. This is followed by a discussion of the more commonly used sensors in geotechnical engineering. Comments are then presented on current sensor instrumentation practices in expansive soils and in concrete structures.

Advances in technology have made it possible for new methodologies to be incorporated into civil engineering projects. However, in spite of such developments which should lead to realistic analyses, innovative designs, and faster construction techniques, variations continue to exist between laboratory and field results. These variations essentially stem from disparate lab and field conditions. For example, tests are often conducted on disturbed or semi-disturbed soil specimens. Another point of variation is that laboratory results are obtained in almost ideal conditions of a controlled environment which differs from the varying environmental conditions in the field. The gap between the results can be bridged by accurate sampling and testing, especially in-situ testing.

## 2.2 Expansive Soils

Expansive soil is a term used for soils which exhibit moderate to high plasticity, low to moderate strength, and high swell and shrinkage characteristics (Holtz and Gibbs, 1956). They show evidence of large volume changes under varying moisture conditions from seasonal changes (Nelson and Miller, 1992). Such soils are commonly found in many arid and semi-arid areas of the world such as Australia, Canada, China, India, Israel, Italy, South Africa, UK, and USA. The mechanical and hydraulic properties of soils are significantly influenced by the volume change characteristic of expansive soils. According to a study conducted by the National Science Foundation, damage to structures caused by expansive soils - particularly to light buildings and pavements - is more than any other natural disaster, including earthquakes and floods (Jones and Holtz, 1973).

Parts of the USA, particularly the southwestern and western parts, have large expansive soil areas. The state of Texas is known for having expansive soil problems. Most of these problems are centered around the North Texas region near the Dallas-Fort Worth Metroplex, where more than one hundred foundation contractors perform foundation repairs for residential and industrial buildings distressed by the heave movements of underlying soils.

Three factors play an important role in the heave and swell properties of soils: (i) soil properties such as compaction, natural moisture content variation, dry density, and plasticity index (ii) environmental conditions, which include temperature and humidity and (iii) natural overburden pressure and foundation loading conditions. The

degree of saturation in a typical expansive soil increases in stages from 40% to 70% to 100% or near saturation levels when the soil starts to heave due to soaking and wetting conditions. Hence, it can be inferred that swell magnitudes depend on the natural and compacted moisture content. Swelling characteristics are associated with the wetting of soil particle surface area and the void distribution between them. Plasticity index has also been used as a test to estimate the swelling potentials of natural subgrades. Practitioners often design and select chemical stabilizers to reduce the plastic nature of expansive soils from problematic levels (high PI values) to non problematic levels (close to zero PI values) (Chen, 1988).

In the city of Arlington, the infrastructure construction that has been steadily growing in all parts of the city is being affected by the challenging soil movements. This is due to the presence of soft and expansive soils which undergo significant volumetric movements which in turn affect the long term performance and design life of pavements and buildings. Another concern in south Arlington soils is that these soils here contain high amounts of gypsum or low soluble calcium sulfates. For such soils, traditional chemical stabilizers are not suitable as they are calcium rich materials, which lead to sulfate-induced heave distress problems. This sulfate heave may further aggravate the overall heaving of soils. This heave is predominantly caused by ettringite mineral that is formed when the calcium component of stabilizers react with sulfates and reactive free alumina of soils. Because of these problems, there is a strong research need to find new and innovative stabilization methods to address these challenging soil issues.

## 2.3 Chemical Stabilization

There are several stabilization methods to treat soft, sulfate-rich, and expansive soils. These have been implemented to improve soil properties but have failed to provide durable and cost effective solutions (Hausmann, 1990). Over the past three decades, extensive research has been conducted to enhance these treatment methods, which can now be classified into four main categories based on soil processing mechanisms: Mechanical stabilization, chemical stabilization, thermal stabilization and electrical stabilization (Hausmann, 1990). Among these, mechanical and chemical are the most widely used because they provide fast, efficient, repeatable and reliable improvements to raw soil properties (Hausmann, 1990). Mechanical methods deal with modifying the physical characteristics of the soil to achieve desired results.

Chemical stabilization is an effective method to enhance soil properties by combining chemical additives to the soil. This technique often enhances shear strength properties and lowers the swelling and shrinkage characteristics of soils, thus offering a better foundation base to the pavements (Mitchell and Fossberg et al., 1969; Stewart, 1971). Commonly used additives like cement, lime, and fly ash have been successful in enhancing soil properties to requisite levels. However, these stabilizers have their individual limitations and consequently do not provide effectual solutions in all soils. A complete summary of several stabilization methods is provided in Hausmann (1990).

As mentioned earlier, the soils of southeast Arlington have low strength and high swell and shrinkage properties. Additionally, they contain high amounts of sulfates. Thus, when a stabilizer such as cement or flyash is added to such soils,

ettringite-based sulfate heaving is exhibited (Mitchell, 1986). Ettringite is a crystalline mineral which is formed by the reaction between the calcium components of stabilizers with free alumina components of clay minerals and ground sulfates. Ettringite can be highly expansive when it comes in contact with water which in turn may cause damage to the overlying pavement structure. Sulfate heaving movements as high as 45 cm or 1.5 ft were reported in the literature and such movements damage the overlying structures (Mitchell, 1993; Hunter, 1988).

The stabilizers chosen in the laboratory phase of this study contain low calcium and have been known to reduce sulfate heaving, thus proving to be efficient and suitable chemicals for the soil stabilization process, especially in sulfate-rich soils (Kota et al., 1996; Viyanant, 2000; Wattanasanticharoen, 2000). The following sections provide descriptions of the four selected stabilizers including their performance in the laboratory investigations conducted by Wattanasanticharoen (2000). Current literature on the mechanisms of reactions and their effectiveness in enhancing soil properties is also covered.

*2.3.1 Sulfate Resistant Cement Stabilization Method*

Cement stabilization has been in use for more than sixty years and its applications cover areas such as construction of highways, roads, and earth dams. In the United States, the first controlled soil-cement construction was tested in Jacksonville, North Carolina in 1935 (Das, 1941). This method basically consists of mixing pulverized earth, fixed quantity of Portland cement, and water. This mixture is then

compacted to the required density and protected against moisture loss during a specified curing time.

This method immediately reduces the plasticity characteristics of the soil which are caused by calcium ions released during the initial hydration reactions (Bugge et al., 1961. It is noticed that a small addition of cement stabilizer increases the aggregation reactions rapidly which in turn decreases the Plasticity Index value.

The hydration reactions of Portland cements are due to the production of different compounds and gels that increase the soil strength through complex pozzolanic activity (Chen, 1988; Nelson and Miller, 1992). This bonding not only serves to protect aggregates but also prevents them from swelling and softening from absorption of moisture and from detrimental freezing and thawing (Bugge et al., 1961).

The cementing action in granular soils is similar to that of concrete except that cement paste does not fill the voids in the aggregates. Rather, it tends to join at the points of contact between particles, which is dominated by the availability of moisture and presence of finer particles (Bugge et al., 1961).

The reactions that occur during cement chemical process can be listed as follows (Kezdi, 1979; Schoute, 1999):

$$2\,(3CaO.SiO_2) + 6H_2O \rightarrow 3CaO.2SiO_2.3H_20 + 3Ca\,(OH)_2 \quad (2.1)$$

(tricalcium silicate) (water) (tobermorite gel) (calcium hydroxide)

$$2\,(2CaO.SiO_2) + 4H_20 \rightarrow 3CaO.2SiO_2.3H_2O + Ca\,(OH)_2 \quad (2.2)$$

(bicalcium silicate) (water) (tobermorite gel) (calcium hydroxide)

$$3CaO.Al_2O_3 + 2H_2O + Ca\,(OH)_2 \rightarrow 3CaO.Al_2O_3.Ca(OH)_2.12H_2O. \quad (2.3)$$

(tricalcium aluminate) (water) (calcium hydroxide) (tetracalcium aluminate hydrate)

We see from equation 2.1 that tricalcium silicate first dissolves in water, forming fibrous calcium silicate hydrate which is also known as tobermite gel. Tobermite gel is also formed by bicalcium silicate, which is shown in equation 2.2. Equation 2.3 shows the formation of calcium aluminate hydrate. The cementation of the particles at the points of contact is due to the calcium silicate hydrate and tetracalcium aluminate hydrate. This results in high strength in the cement-clay skeleton and clay matrix. The individual units of this matrix contain a core of hydrated cement gel surrounded by a zone of flocculated clay which is glued together by secondary cementation at inter-domain contacts (Schoute, 1999).

Presently several variants of cement are available in the market. Although Portland cement is the same, eight types of cement are manufactured to meet the different physical and chemical requirements for specific applications. They are classified from Type I to Type V Portland cements (Mamlouk and Zaniewski, 1999). Type I is general purpose cement appropriate for use as RC structures. Type II is used for structures in water or soil containing moderate amounts of sulfate, or when heat build-up is a concern. Type III cement provides high strength at an early stage, typically in a week or less. Type IV moderates heat generated by hydration and is specifically used for massive concrete structures such as dams. Type V cement resists chemical attack by soil and water high in sulfates (Wattanasanticharoen, 2000).

The following Table 2.1 presents the chemical composition of the Type V cement used in this research.

Table 2.1 Chemical Composition of Type V Cement

| **Chemical Composition** | **Percent** |
|---|---|
| Calcium Oxide (CaO) | 53.10% |
| Silicon Dioxide ($SiO_2$) | 29.33 % |
| Aluminum Oxide ($Al_2O_3$) | NA |
| Sulfur Trioxide ($So_3$) | 3.30 % |
| Magnesium Oxide (MgO) | 1.44% |
| Loss of Ignition (LO!) | 0.93 % |
| Total Alkalies as ($Na_2Oeq$) | 0.59 % |
| Insoluble Residue (IR) | 13.72% |
| % Class F Ash | 20.75 % |
| Sulfate Expansion (C-1012) | NA |

*2.3.2 Low Calcium Class F Fly Ash with Cement Stabilization Method*

Coal ash is one of the by-products generated when coal is burnt in thermal power plants (Ferguson, 1993). It typically comprises of fly ash (flue gas stream), boiler slag which coats boiler tubes, and bottom ash (sand size material and boiler slag). The amount of fly ash recovered from coal ash depends on the type of coal, the burners and the boiler and is generally from 65% to 85%. Fly ash is the fine grained fraction collected from the suspension of the exhaust gases of the combustion chamber which maybe extracted using an electrostatic precipitator. Boiler ash is relatively coarser and denser material and is collected by gravity at the lower level (Nicholson and Kashyap, 1993).

According to ASTM standards, fly ash is classified either as Class C or Class F, depending on the original source of the coal used in the power generators. Class F fly ash is produced by the burning of bituminous or anthracite coals found in the Midwestern, Eastern, and Southern parts of the United States, while Class C fly ash is obtained from the burning of sub-bituminous or lignite coals. The main difference between these two types is the amount of calcium, silica, alumina and iron content.

Typically in Class F fly ash the calcium content varies from 1 – 12% and mainly in the form of calcium hydroxide and calcium sulfate, whereas Class C flyash may have calcium content as high as 30 - 40%. Also, the amount of alkalies (combined sodium and potassium) and sulfates is generally higher in Class C flyash. All these factors tend to make Class C flyash a better soil stabilizing agent (Nicholson and Kashyap, 1993). The ASTM specifications for the chemical requirements to classify any fly ash (ASTM C 618) are shown below on Table 2.2.

Table 2.2 Chemical Requirements for Fly Ash Classification (ASTM C 618)

| **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 ($SO_3$), max, % | 5.0 | 5.0 |
| Moisture Content, max, % | 3.0 | 3.0 |

The addition of lime or cement to fly ash has shown increased soil strength and stiffness properties (McManus and Nataraj, 1993). When flyash 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). The short-term reactions cause flocculation and agglomeration of the clay particles due to the ion exchange at the surfaces of the soil particles. The final result is to enhance the workability characteristics in soils and provide an immediate reduction of swell, shrinkage, and plasticity properties (Nicholson and Kashyap, 1993).

Long-term reactions which lead to increased strength in the treated soil, take place over a periods of time ranging variably from a few weeks to several years. The amount of time is dependant on the rate of chemical breakdown and on the hydration reactions of the silicates and aluminates. These reactions help to improve and bind the soil grains together to form cementitious materials (Nicholson and Kashyap, 1993).

For lime stabilization of soils, pozzolanic reactions depend on the siliceous and aluminous materials provided by the soil. The following are the pozzolanic reactions that occur during stabilization:

$$Ca(OH)_2 \Rightarrow Ca^{++} + 2[OH]^- \tag{2.4}$$

$$Ca^{++} + 2[OH]^- + \underset{\text{(silica)}}{SiO_2} \Rightarrow \underset{\text{(gel)}}{CSH} \tag{2.5}$$

$$Ca^{++} + 2[OH]^- + \underset{\text{(alumina)}}{Al_2O_3} \Rightarrow \underset{\text{(gel)}}{CAH} \tag{2.6}$$

As can be seen from equation 2.6, hydration of tricalcium aluminate in the fly ash provides one of the primary cementitious products. The rapid rate of hydration of

the tricalcium aluminate results in the quick setting of these materials, and is the reason why delays in compaction result in lower strengths of the stabilized materials. Since the hydration chemistry of fly ash is very complex in nature, the stabilization application must be based on the physical properties of the ash-treated stabilized soil and cannot be predicted based on the chemical composition of the fly ash.

This research study considered Class F fly ash as a stabilizer seeing as the soils in southeast Arlington contain high amounts of sulfates. In sulfate soil environments, Class C fly ash may lead to ettringite formation (Mamlouk and Zaniewski, 1999). Table 2.3 presents the chemical composition of the Class F fly ash used in this research.

Table 2.3 Chemical Characteristics of Class F Fly Ash

| **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 |

*2.3.3 Ground Granulated Blast Furnace Slag (GGBFS) Stabilization Method*

Blast furnace slag is a by-product of iron production and consists of a combination of the siliceous component of the iron ore and the limestone flux used for melting iron (Sherwood, 1995). It has a chemical composition similar to Portland cement. Different kinds of slags are produced during the manufacture of metals from their ores 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 in the furnace floats to the top and is used to form a granulated glassy material that possesses hydraulic properties after it is ground to cement fineness. When the granules are ground to Portland cement (PC) fineness, it is called ground granulated blast furnace slag (GGBFS). When this material is mixed with cement, the alkalies released by hydration of cement are often adequate for the activation of GGBFS. Activation of GGBFS may be achieved by direct mixing with lime water and for the production of super-sulfated cement by adding sulfates (Olorunsogo and Wainwright, 1998).

Slag is formed and collected in different ways. The following methods are based on various cooling methods utilized on the residuals after leaving the blast furnace (Sherwood, 1995):

- The residuals are exposed to open air in order to cool slowly. This results in a crystallized slag formation which is later used for crushing and as an aggregate base. This slag is known as “air-cooled slag”.

- The residuals are subjected to immediate cooling performed by either water or air which produces a vitrified slag. In the first case, the slag produced is known as “granulated slag” and in the second case, it is known as “pelletised slag”. In this research, the granulated-type slag was used for soil stabilization.
- The residuals may be water-cooled under certain conditions where the steam produced gives rise to what is known as “expanded slag”.

The use of slag reduces the permeability of the soil mixture significantly, the reduction depending on the amount of the slag used (Ozyildirim et al. 1990). An advantage of this decrease in permeability is high chemical resistance in aggressive environments. The permeability is lessened by a reduction in the pore size associated with the production of dense calcium silicate hydrates in the hydration process that takes place during mixing (Ozyildirim et al., 1990). Slag also increases the sulfate resistance of the soil. The slag stabilization not only increased the sulfate resistance in soils, but also increased the shear strength and decreased the plasticity index, swelling potential, and shrinkage strains (Ozyildirim et al., 1990).

In England, considerable research conducted to study the behavior of the GGBFS stabilization on sulfate-rich soils (Wang et al., 1998). Test results from this research showed that by incorporating 20% of GGBFS stabilizer, a significant strength increase was achieved in soils after three days of curing. Also, the GGBFS treatment decreased the plasticity index and swelling and shrinkage strains (Wang et al., 1998). Considering these positive results, the present research considered GGBFS as one of the four stabilizers.

Table 2.4 presents the chemical composition of the GGBFS used in this research.

Table 2.4 Range of Composition of Blast Furnace Slag

| **Chemical Constituents (Oxides)** | **Range of Composition, Percent by mass** |
|---|---|
| Silicon Dioxide ($SiO_2$) | 32-40 |
| Aluminum Oxide ($Al_2O_3$) | 7-17 |
| Calcium Oxide (CaO) | 29-42 |
| Magnesium Oxide (MgO) | 8-19 |
| Sulfur (S) | 0.7-2.2 |
| Iron Oxide ($Fe_2O_3$) | 0.1-1.5 |
| Magnesium Oxide (MnO) | 0.2-1.0 |

Note: Range of Chemical Composition of Blast Furnace Slag
In the United States and Canada

Wattanasanticharoen (2000) study showed that this GGBFS stabilizer alone provided considerable enhancements to sulfate rich soils. Also, swell tests conducted on chemical treated soils did not result in Ettringite formation and related heaving in laboratory conditions. Hence, this stabilizer was selected for the present field investigations.

*2.3.4 Lime mixed with Polypropylene Fibers Stabilization Method: Background*

Lime stabilization is one of the oldest stabilization methods to improve soil properties. Lime is perhaps the most common chemical treatment, with cement grout and potassium also being used in high numbers (Gedney and Weber, 1978). The application of lime as a stabilizer can be traced back to more than 5,000 years ago

(Schoute, 1999). Lime material was used in the construction of several historical places such as the Appian Way, Rome, Italy and the pyramids of Shersi in Tibet, (Winterkorn & Pamukcu, 1991). These structures were built by using compacted mixtures of clay and lime (Winterkorn & Pamukcu, 1991).

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

$$CaCO_3 + \text{Heat} \Rightarrow CaO + CO_2 \tag{2.7}$$

$$CaO + H_2O \Rightarrow Ca(OH)_2 + \text{heat} \tag{2.8}$$

$$Ca(OH)_2 + CO_2 \Rightarrow CaCO_3 + H_2O \tag{2.9}$$

At high temperatures in the order of 500 degrees, the first reaction is a reversible reaction that results in the production of quick lime from chalk or limestone (equation 2.8). Equation 2.9 represents the production of hydrated lime by mixing quick lime with $CO_2$. Due to these immediate reactions, these two types of lime are used with soil for stabilization. Calcium carbonate is a useful chemical in agriculture as a soil additive to adjust pH conditions. However, it is not used for soil stabilization in civil engineering (Sherwood, 1995).

The types of lime can be found in different forms. Hydrated lime is available in the form of a fine, dry powder. Quicklime is available either in granular form or as a

powder. Both hydrated lime and quicklime are also used in slurry form by mixing with water.

The addition of lime to clay or soil results in an increase in the plastic limit and a decrease in the Plasticity Index value. This leads to further reduction in swell and shrinkage strains, an increase in shear strength, and a decrease in its compressibility and permeability properties (Broms and Boman, 1979; Littlle, 1987; Puppala et al., 1998). When a certain amount of quicklime is mixed with clayey soil, a dehydration reaction takes place and slaked lime is created. This dehydration process induces a drying effect in the soil causing an immediate reduction in water content. This drying action has the advantage of improving soil plasticity characteristics especially in moist clays (Schoute, 1999).

Calcium hydroxide is a product of the dehydration reaction and a stabilizer itself. When $Ca(OH)_2$ is mixed with water it dissociates. This increases the electrolytic concentration and the pH of the soil. The calcium hydroxide dissociation is explained in the following reaction (Schoute, 1999):

$$Ca(OH)_2 \Rightarrow Ca^{2+} + 2(OH)^- \qquad (2.10)$$

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

- Reduction in the thickness of the electric double layer - this causes a reduction in susceptibility to water addition.

• Flocculation of the clay particles with weak bonds between the particles which is caused by an increase in mutual attraction due to the decrease in electric double layer (Diamond & Kinter, 1965).

• Increase in the internal angle of friction between the particles due to flocculation.

• Textural change from plastic clay to a granular, friable material.

Figure (2.1) shows the effect of lime stabilization on Atterberg limit values.

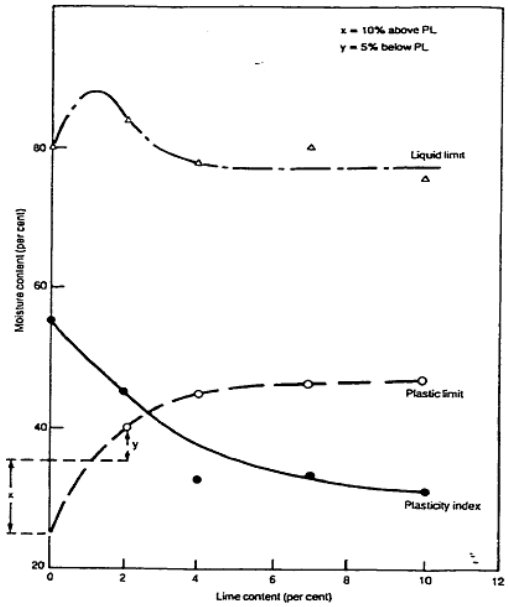


Figure 2.1 Decrease in PI due to Lime addition

The lime reaction rate depends on the type of clay mineral present in the soil. During mixing, all clay minerals react with the lime stabilizer. Three-layer minerals are more reactive than two-layer minerals; hence, the reaction of lime with montmorillonite clay is quicker than the reaction with kaolinitic clay (Bell, 1988). The temperature of the soil and surroundings also plays a vital role in the effectiveness of the stabilization

technique (Ahnberg et al., 1989). A higher curing temperature results in a faster increase in the shear strength of the soil. The decrease in the swelling and shrinkage is attributed to the decrease in the plasticity index values. These reactions also account for the decrease in volume change properties (Nelson and Miller, 1992; Hausmann, 1990).

Polypropylene fiber materials are used to reinforce soils because they are more cost effective than other materials including chemicals. They can be manufactured to the specified dimensions and with the desirable properties and also reduce the intake of natural raw materials. They do not create leaching problems and can be manufactured from recyclable materials. They are also unaffected by chemical and biological degradation (Gregory, 1996; Puppala and Musenda, 1998). This implies that the fiber stabilization method can potentially reduce the landfill costs of plastic wastes by utilizing them in a cost effective manner (Musenda, 1999).

Table 2.5 shows the properties of the fibrillated polypropylene fibers used in this research (Boral Material Technologies):

Table 2.5 Fiber Properties used in this Research

| 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 |

Wattanasanticharoen (2000) study showed that lime-polypropylene stabilizer provided third best enhancements to sulfate rich soils among four methods studied in the research. Field studies planned in this research will further evaluate this stabilization in real field conditions.

## 2.4 Instrumentation in Civil Engineering

With advances in technology, various methodologies using in-situ tests, field instrumentation or both have been used in civil infrastructures for quality construction and assessments. These methodologies have led to innovative design, realistic analysis, and easier and faster construction techniques. Even with these advancements, the discrepancy between the laboratory and field is yet to be bridged. For instance, the soils sampled for testing from the field can never be obtained without any disturbance.

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 provide data on the physical, strength, and compressibility characteristics of the soils. Because soil is a highly heterogeneous media, it shows larger variations of behavior in real field conditions when compared to other civil engineering materials. This is because soil properties depend on the geological history of soil formation, location of formation, temperature and environmental characteristics, and other site factors that differentiate soils from one place to another.

As a result, laboratory test results can only provide an approximate behavior. These results need further assessments in real field conditions for accurate civil engineering analysis and design. These difficulties show the importance and need for more accurate and reliable methods to measure physical and engineering properties of soil in the field condition.

As mentioned earlier, advances in electrical and electronics engineering have revolutionized the instrumentation methodology to study civil engineering problems. The decreasing sizes of electrical gauges in the field of microelectronics have led to the development of many new devices that utilize micro mechanics principles. The increased reliability and capabilities of these gauges supplemented with their lower cost can solve the performance-monitoring needs of civil engineers (Durham, 1999). In performance monitoring aspects, civil engineering, and particularly geotechnical engineering, is lagging behind other engineering fields in incorporating and utilizing sensor devices (Glazer, 2001). One reason for this is that the variation in material properties at the soil and superstructure interfaces especially is high. Also, monitoring them in earth structures requires rugged instruments able to withstand the external disturbances and moisture infiltration of the field (Negussey et al. 2001). For this reason, rugged sensors with capabilities to provide interface properties are necessary for geotechnical engineering practices (Mohan, 2002).

Technological advancements in manufacturing gauges have helped to overcome the above-mentioned disadvantages causing a recent widespread use of such sensors in research and other construction. This helps researchers develop accurate analytical models for better simulation of field conditions and performance. For example, use of instrumented geosynthetic reinforced retaining walls for research studies leads to better internal and external stability analyses (www.ltrc.lsu.edu) (Mohan, 2002). Sensor data is also helpful in providing valuable forewarnings in excavation, tunneling and earthquake related projects.

Due to the anisotropic nature of soils and wide variation of soil properties from various locations at the same site, it is difficult to generalize soil theories and practices that can be universally applied to all soils. For this reason, geotechnical structures need to be continuously monitored for the verification and modification of theories and practices. An example is lime stabilization which is used effectively to modify various soil types, but has not provided effective treatment for sulfate-rich soils (Puppala et al. 2001). Field and laboratory monitoring on sulfate soils have resulted in better understanding of this problem. Thus, we see the need for field instrumentation and continuous performance monitoring in civil engineering projects.

## 2.5 Current Instrumentation

Previously, instrumentation in the field of geotechnical engineering had been used for a variety of applications including monitoring of slopes, foundations, retaining walls and natural and man-made excavations. Generally, the measurement of parameters such as overburden pressure, pore water pressure, volumetric and gravimetric moisture contents, displacement, and strain have been monitored as they are most likely to influence soil behavior and structural response.

The following section provides an insight into the various instruments being used in the field. They have been categorized based on the engineering parameters they measure.

### *2.5.1 Strain Measurement Devices (Turner and Hill, 1999)*

Initially, strain measurement devices were mechanical in nature and used to correlate displacement of the arm of the gauge to the strain. This method was laborious

since the strain recorded had to be measured manually. The only advantage of these mechanical gauges over electrical or electronic gauges was that they did not need electricity for their operation. However, considering the ease of installation and relatively simple methods of data acquisition, electrical gauges are preferred. Electrical gauges operate by relating the change in the resistance values to calibrated strain readings. The most commonly used strain-measuring device is the electrical strain gauge which is available in quarter bridge, half bridge and full bridge configurations. (The bridge corresponds to a normal Wheatstone bridge). Data is obtained by using data acquisition systems or readout boxes.

*2.5.2 Displacement Measurement Devices (Dally et al., 1993)*

These can be classified further as linear displacement measuring devices and angular displacement measuring devices even though they have the same working principle as strain measuring devices. These gauges need anchor support and the measurements obtained are relative to the anchor support. Some of these devices are listed below:

- Potentiometers: They have a moving frame on which a movement results in a drop in electrical potential. By measuring this drop, the strain can be calculated.
- Linear Voltage Differential Transducer (LVDT): These gauges work on the principle of linear inductance. The displacement of the rod causes a linear change in inductance of the transformers and this value is correlated to the displacements.

- Optical displacement measurements: This method utilizes digital videos, fiber optics, and a high-speed photography camera along with specialized computer programs to analyze the signals and interpret those values to provide displacement readings. Although these systems are comparatively more expensive than other displacement measurement sensors, this method of instrumentation does not require physical contact with the soil of the monitoring area.
- Extensometers: Here, the device gives the relative displacement with respect to the anchor embedded. They are relatively cheap, widely used, and can be easily connected with data loggers to automate and digitize the data collection process (Ding et el., 2000).
- Tilt meters, Inclinometers and Electro levels: These are used to measure the rotational deformation. They are not widely used on account of their high cost and installation difficulties.

*2.5.3 Force and Pressure Measurement Sensors (Dally, 1993)*

These devices generally consist of an elastic member, which measures the force or pressure exerted to give the strain readings. Strain converting units transform these strains to the corresponding pressure values. A critical parameter that affects the effective working of these gages is in the shape of the actual area of the gage, which is in direct contact with soil.

- Load Cells: Different types of load cells are available in the market, but they all have the same underlying principle of measuring the strain in the elastic member and transforming it to the force applied.
- Pressure gauges: the main difference between a pressure gauge and load cell is that the latter measures the total load on the surface whereas the former calculates the average pressure developed to cause a tangential strain all through the surface area of the gauge.
- Piezometers: Many research and project works in geotechnical engineering require monitoring ground water levels and provide continuous or periodic pore pressure distributions in soils (Hoek and Bray, 1981). This is important to be able to ascertain the stability of slopes and retaining walls and to assess the drained or undrained conditions that prevail.

*2.5.4 Instruments for data collection*

These systems are used in conjunction with gauges to record continuous, periodic, or discrete responses from the gauges. They can be classified as analog or digital based systems depending on the format used to collect and save data. A commonly used analog device is the readout box, which is similar to voltmeters. Readings are obtained by converting the potential difference reading obtained from these gages. This also involves manual recording of data.

Digital acquisition is usually carried out with the help of data loggers that have an internal storage unit and acquisition cards that connect to computers and transfer data immediately to the computer. The advantage in the former module is that it can be

installed on site and the data can be obtained at regular intervals. The latter is used in research projects that require discrete data and where the time interval between two readings is considerably high. The advantage of latter modules over the data loggers is that these are comparatively cheaper in cost and do not require continuous on site power supply.

Advancements in instrumentation have led to better performing and less expensive gauges and sensors. In many projects, appropriate instrumentation can be designed and installed by learning from previous case studies. Mohan (2002) has described various case studies in which instrumentation was used in the field.

## 2.6 Case Histories

This section lists out various case histories wherein different gauges have been used. Based on the main intent of instrumentation, the following section is divided into three groups (Mohan, 2002):

- Instrumentation to monitor structural disintegration,
- Instrumentation to assess quality assurance of construction, and
- Instrumentation to develop, verify and modify analytical models.

### *2.6.1 Instrumentation to monitor structural disintegration*

Development of cracks in concrete which eventually lead to failures in super structures need to be monitored before failure happens. Generally, plastic deformation continuously increases and reaches a level considered as a failure level. One such failure available in literature is discussed in this section (Iskander et al., 2001). The layout of instrumentation is presented in figure 2.2. failure available in literature is

discussed in this section (Iskander et al., 2001). The layout of instrumentation is presented in figure 2.2.

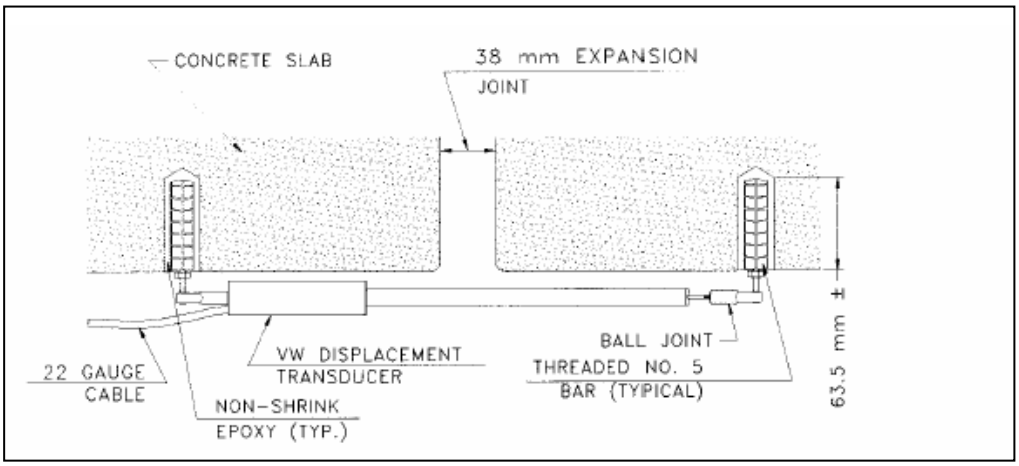


Figure 2.2 Layout of Parking Garage.

In the case study reported by Iskander et al. (2001), an underground parking garage in a multi-storied building was monitored with several deflection measurement gauges to trace the disintegration process. Data was acquired for a period of one year to find out the safety of the building against excessive displacements. Crack meters were installed in the expansion joints that form the critical section of failure along with tilt meters to establish the tilt of the building due to external pressure. These, along with temperature measurement units, were connected to data loggers to collect the deflection data. It was concluded that the movements were predominantly due to temperature variations. It is to be noted that temperature, which was the crucial factor leading to the failure, could not have been identified by other test procedures and laboratory investigations.

Instrumentation design has become a vital part of dam construction projects. To explain this further, Mohan (2002) cites USBR-8314-1 report in which an upper still

water dam in northern Utah was built (USBR-8314-1). Due to adequate instrumentation in this dam, the sliding of the foundation was detected at early stages and remedial measures were taken. The strain measurement readings from extensometers were documented and analyzed. These analyses were able to predict the mode of movement based on the rate of displacement data. Without this instrumentation, the failure of this dam would have been catastrophic and would have required millions of dollars to repair and rehabilitate.

The above two examples illustrate the need for instrumentation to predict, monitor and repair structures undergoing disintegration. In Table 2.6 (Mohan, 2002) describes various other case studies addressing structural disintegration problems.

*2.6.2 Instrumentation to assess quality assurance of construction*

At construction sites, continuous monitoring is required to avoid Quality Control and Quality Assessments (QC/QA) mistakes such as poor and inadequate use of material use, poor compaction of backfill, etc. Such errors often take place during construction and need to be addressed immediately. Monitoring of such errors can preempt improper design and initiate remedial measures to the structures.

To state an example, the use of instrumentation to measure the displacement response of structures during tunneling operations in Boston's “Big Dig” project proved to be very useful. Another instrumentation related quality assessment is to record the weight measurements of volumes of cement mixing constituents to understand the QA aspects of deep cement mixing jobs in soil stabilization projects. Table 2.7 (Mohan,

2002) describes various other case studies that used instrumentation for quality assessments and parameter identification.

*2.6.3 Instrumentation to develop, verify and modify analytical models*

Instrumentation can provide behavioral responses of earth structures that can be simulated with either analytical or numerical models (Mohan, 2002). For example, in order to assess the suitability of geosynthetic materials, reinforced soil structures built with geosynthetics were monitored with sophisticated instrumentation including strain and displacement sensors.

The following example describes one such research study conducted by Appea and Al-Qadi (2000). The research was supplemented by regularly surveying the data to verify the working of strain gauges. The five year research concluded that the efficiency of the geotextile as a separator was good, and the geo-grid was partially effective. In this research, the instrumentation provided data for performance assessments and provided data for modeling. Table 2.8 (Mohan, 2002) lists other research publications that used instrumentation for model and theory verification.

Table 2.6 Structural Disintegration

| Source | Description | Instrumentations Used | Conclusions |
|---|---|---|---|
| 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. |
| Gasmo et al. (1999) | Study of infiltration characteristics of an unsaturated slope. | Tensiometer, Inclinometers, Piezometers and Tipping bucket rain gauge. | Helped in studying the variation of pore pressure with seasonal changes and thereby providing cautions during structural instability. |
| Lim et al. (1996) | Monitoring of pore pressure to predict the occurrence of a rainfall induced land slide. | Piezometer, Tipping bucket rain gauge and Tensiometer. | Seasonal variation of matrix suction was studied and its influence on the soil slope was studied. |
| Kulesza (1996) | Displacement monitoring in tunnels. | Strain gauge. | International Journal of rock mechanics and mining sciences & geomechanics Oct 1996. |
| Charles (1996) | Long term measurements of ground movements. | Extensometers and Piezometers. | Proceedings of ICE: Geotechnical Engineering 1996. |
| 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 which proved to be vital for the completion of the project. |

Table 2.7 Instrumentation for Quality Assessments

| **Source** | **Description** | **Instrumentations Used** | **Conclusions** |
|---|---|---|---|
| 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. |
| McGrath, Timothy et al. (1999) | Instrumentation for monitoring buried pipe behavior during backfilling. | Profilemeter, Strain gauges, Pressure gauges, Nuclear gauges, Penetrometers. | Pipe-soil interface properties were studied and led to development of parameters for the same. |
| 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. |
| Barlett et.al. | Instrumentation was used to verify the assumptions made during the initial design of foundation and other earthwork. | Settlement plates, Piezometers, Total earth pressure cells, strain gauge, Extensometers, Inclinometers. | Data from instrumentation helped to modify the original design and also helped in evaluating the causes for the variation in the original design to the values obtained. |
| Sabatier, (1996) | Probe microphone instrumentation for determining soil physical properties testing in model porous materials. | Acoustic technique for evaluating soil physical properties. | Physical properties of soil were evaluated based on the response from acoustic sounding techniques. |

Table 2.8 Theory Development and Model Verification

| Source | Description | Instrumentations Used | Conclusions |
|---|---|---|---|
| Huslid (2001) | Full scale monitoring of troll of a concrete platform. | Pressure cells. | The amount of pressure transferred was obtained using the pressure gages. |
| 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. |
| Davis, et al. (2001) | Study of response of wildlife during and after earthquake. Also investigations for the development of alternate hypothesis for the development of liquefaction. | Piezometer. | Instrumentation provided valuable data for investigating and validating an alternative hypothesis for liquefaction development. |
| Ledesma (1996) | Estimation of parameters in Geotechnical back analysis –I maximum likely hood approach. | Sliding micrometer, Inclinometer and Deflectometer. | Algorithms that help in estimating the parameters in back analysis and verification through instrumentation was described in detail for further research and as a schema for similar research studies. |

Table 2.8—*Continued.*

| | | | |
|---|---|---|---|
| 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. |
| Russell et al. (1999) | Evaluation of preloading to increase settlement of the soil. | Settlement plate, Liquid settlement gauge, Piezometers. | Instrumentation data helped in deciding the time of preloading and also the amounts and effectiveness of preloading. |
| Montanelli et al. (1999) | The model technique developed for evaluating field stress and deformation of steep reinforced slope using special interface elements. | Strain gauge, Pressure cells. | The use of field instrumentation provided assurance about working of the modeling technique developed and the use of FEM to obtain field stress and deformation properties. |
| Feng et al. (2004) | Application of neural network based system. | Strain gauges, frequency meters. | Establish a finite element model of an instrumented highway based on measuring traffic induced vibrations. |
| Donahue et al. (2004) | 3-D modeling of wharves and piers for seismic analysis. | Seismic analyzers, ground motion detectors. | Continuous monitoring of ground action to determine action of wave and ground on performance of wharves and piers. |

## 2.7 Summary

This chapter provided a comprehensive summary on expansive soils and the different chemical stabilization methods that were used in this research. These methods are cement stabilization, fly ash with cement stabilization, GGBFS stabilizations, and lime with fibers stabilization. The individual properties of the stabilizers and their influences on soils including sulfate soils were detailed. The selection methodology was also explained. This was followed by a comprehensive summary of the current instrumentation techniques adopted in civil engineering. Several case studies which reveal the importance of using such sensors in the field were also noted. In conclusion, the research needs for the present thesis are also outlined.

# CHAPTER 3

# LABORATORY DATA OF HARWOOD ROAD SOIL

## 3.1 Introduction

An experimental program was designed and implemented to test expansive soil samples obtained from the research site at Harwood Road located in southeast Arlington, Texas. The tests were carried out at The University of Texas at Arlington to determine physical and chemical properties of the soil (Wattanasanticharoen, 2000; Ramakrishna, 2002; Chavva, 2002). The soil was treated with four different types of stabilizers, namely Sulfate Resistant Type V Cement, Class F Flyash with Type V Cement, GGBFS, and lime mixed with polypropylene fibers. The following sections describe the types of laboratory tests performed, the test procedures followed, and the actual test results.

## 3.2 Subgrade Soil Properties

The soil was sampled from Harwood Road located in southeast Arlington, Texas. The objective was to evaluate these four novel stabilization methodologies in real field conditions. Soil samples obtained were slightly dark in color and exhibited a moisture content of 7% at the time of sampling (Ramakrishna, 2002). Both basic and engineering properties are discussed in the following section.

*3.2.1 Basic Soil Properties*

The basic soil tests conducted for this research project included Atterberg limit tests, specific gravity tests, sieve analysis, and hydrometer analysis. Table 3.1 summarizes the physical properties that were evaluated as a part of this research. The grain size distribution, which is shown in Table 3.1, was measured by sieve and hydrometer analysis. Depending on the gradation and Atterberg limit values, the soil was classified as A-7-6 as per the AASHTO classification method and as CL as per the USCS classification method (Chavva, 2002).

Table 3.1 Basic Soil Properties (Ramakrishna, 2002)

| **Soil Properties** | **Results** |
|---|---|
| Passing #200(%) | 91.2 |
| Specific Gravity | 2.73 |
| Liquid Limit | 44.50% |
| Plasticity Index (%) | 22.20% |
| Natural Moisture Content (%) | 7 |
| AASHTO Classification | A-7-6 |
| Soluble Sulfate Content (ppm) | 4737 |
| USCS Classification | CL |

Figure 3.1 shows the grain size distribution results of Harwood road soil.

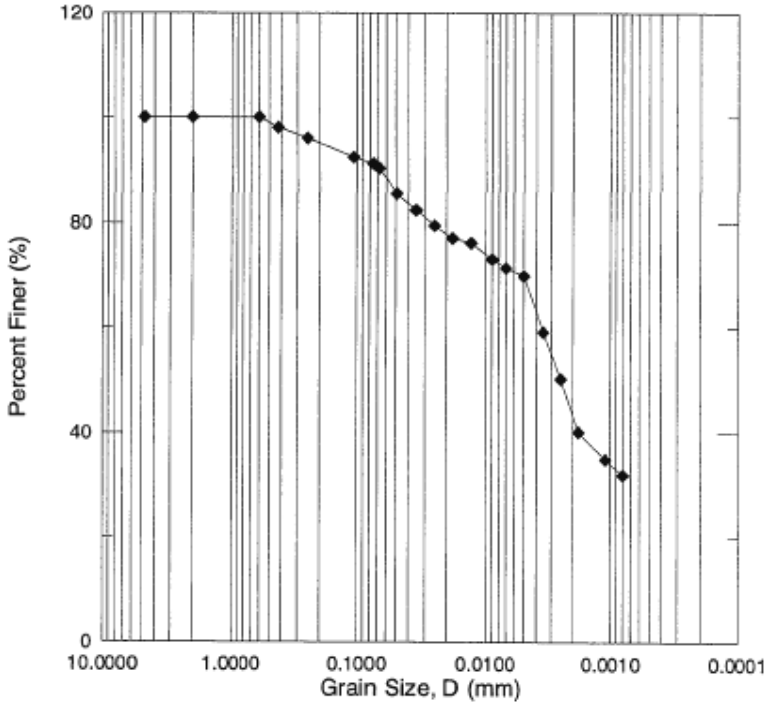

Figure 3.1 Grain Size Distribution Curve of Harwood Road soil (Ramakrishna, 2002)

Standard proctor compaction tests were then conducted on both control soil and treated soil samples. These are presented in Table 3.2 below and in Figure 3.2 in graphical format.

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 | Lime (8%) and Polypropylene Fibers (0.15%) | 18.00 | 96.00 |
| 3 | Type V Cement (8%) | 16.70 | 106.90 |
| 4 | Class F Flyash (15%) and Type V Cement (5%) | 18.70 | 104.20 |
| 5 | GGBFS (20%) | 16.00 | 107.30 |

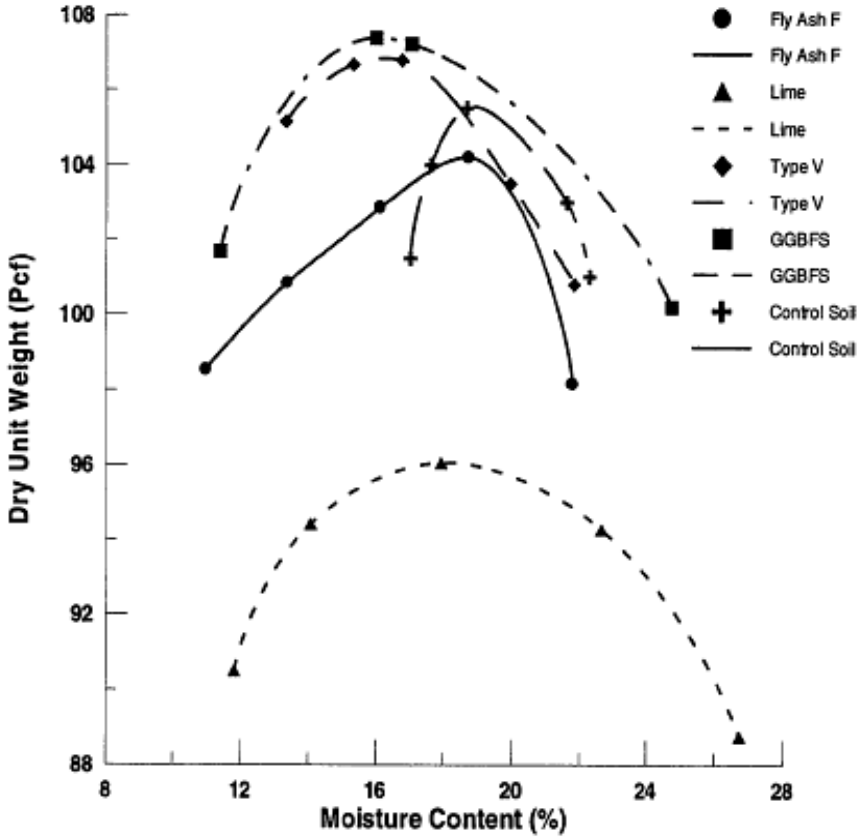


Figure 3.2 Standard Proctor Compaction test results of control and treated soils (Ramakrishna, 2002)

*3.2.2 Basic and Engineering Properties*

Chavva (2002) carried out laboratory tests which included unconfined compressive strength tests (UCS), free swell and linear shrinkage tests. The UCS and free swell tests were performed at the three moisture content levels of optimum, dry of optimum, and wet of optimum contents of control soil. The dry and wet side moisture levels corresponded to those at 95% of optimum dry unit weight of standard Proctor soil curves.

Ramakrishna (2002) conducted resilient modulus tests to measure the moduli at different moisture contents. One of the tests was close to the optimum while the other

two were corresponding to 95% of dry unit weight, with one being on dry and the other on the wet side of the Proctor curve. To check the consistency and repeatability of the results, tests were conducted on five identical samples.

## 3.3 Specimen Preparation

The plasticity index, swelling, shrinkage, and strength potential of soils are usually affected by both compositional and environmental conditions such as moisture content, the dry unit weight of soils, and stabilizer dosages (Chavva, 2002). Table 3.3 below lists the variable conditions that were considered by Chavva (2002).

Table 3.3 List of Variable Conditions (Chavva, 2002)

| **Description** | **Variables** |
|---|---|
| Soil Types | Harwood Road Soil |
| Stabilizers | 4 types: Class F Flyash combined with Type V Cement, Sulfate Resistant Type V Cement, GGBFS, Lime with Fibers |
| Moisture Content | 3 moisture contents; Optimum, Dry of Optimum, Wet of Optimum |
| Stabilizer Dosage Level | One |
| Temperature Conditions | Room Temperature |
| Curing Period | Seven days |

The soil was first oven dried and pulverized. Next, it was mixed with required chemical stabilizers at optimum moisture content and dry unit weight levels. Specimens were then compacted in Standard Proctor molds (for Resilient Modulus and UCS tests) and then carefully extracted. Afterwards, the samples were wrapped in geotextiles and

transferred to humidity controlled rooms for curing. Here, the storage bags used were of plastic type in order to prevent any loss of moisture through the evaporation of water from the soil samples. The sample preparation for soil-lime-fiber mixing was done in accordance with standard ASTM D3551-90. The treated samples were cured for forty-eight hours before being compacted. This initial curing period is called the "mellowing period". The specimens for the other stabilizers, cement, flyash, and GGBFS, were prepared immediately after mixing (Chavva, 2002). Table 3.4 below shows the optimum, dry of optimum, and wet of optimum values for the different stabilizers.

Table 3.4 Dry, Optimum and Wet of Optimum Moisture Contents (Chavva, 2002)

| **Sl No.** | **Soil Type** | **Optimum Moisture Content (%)** | **$\gamma_d$ (lb/ft$^3$)** | **Dry of Optimum (95%)** | **Wet of Optimum (95%)** |
|---|---|---|---|---|---|
| 1 | Control Soil | 18.6 | 105.5 | 16.5 | 21.2 |
| 2 | Lime (8%) and Polypropylene fibers (0.15%) | 18.0 | 96.0 | 15.5 | 22.8 |
| 3 | Type V Cement (8%) | 16.7 | 106.9 | 14.5 | 19.7 |
| 4 | Class F Flyash (15%) and Type V Cement (5%) | 18.7 | 104.2 | 14.3 | 22.8 |
| 5 | GGBFS (20%) | 16.0 | 107.3 | 15.4 | 22.1 |

## 3.4 Test Procedure

As mentioned, laboratory tests were conducted on the soil samples to measure changes in both physical and engineering properties with respect to chemical stabilization. To check for accuracy and consistency, testing was conducted with

different dosages and on a number of specimens. The following sub-sections outline the various tests performed.

*3.4.1 Atterberg Limits and Proctor Compaction Tests*

Atterberg limit tests which calculate the plasticity index were carried out to determine the consistency of the soil. Raw and cured samples were used as per the ASTM D-4318 method (Chavva, 2002). Standard Proctor results have already been mentioned in the earlier sections.

*3.4.2 Resilient Modulus Test*

The tests on materials were performed at the confining and deviator stress levels conforming to the latest version of AASHTO TP46. Samples were first conditioned by applying one thousand repetitions of a specified deviatoric stress (Ramakrishna, 2002). This was done to eliminate the effects of specimen disturbance caused by sampling, compaction, and specimen preparation procedures and it also assisted in minimizing the effects of imperfect contacts between the end platens and the specimen. Following the sample conditioning, a test procedure specified in AASHTO TP46 was conducted to cover the service range of stress that a pavement or subgrade material experiences due to traffic loading and over-burden conditions.

The resilient modulus results were assessed for repeatability. They were then analyzed to understand the effects of stabilizer reactions, moisture content effects, and stress influence on the magnitudes of resilient properties (Ramakrishna, 2002).

*3.4.3 One-Dimensional Free Swell Test*

This test measures the amount of heave (in the vertical direction) of a confined specimen. In accordance to ASTM standards, both control and treated specimens measuring 2.5 inches in diameter and 1 inch in thickness were included. Porous stones were placed at the top and bottom to facilitate water movement. These were then shifted and placed in a container and then filled with water. The amount of heave was measured by a micrometer dial gauge against elapsed time and actual time. Swell values were observed over a period of three days until there was no significant change in the dial gauge readings. The period of three days was recommended since the investigation was conducted on new stabilization materials (Chavva, 2002). The final displacements and the original heights were used to calculate free swell values in the vertical direction.

*3.4.4 Linear Shrinkage Bar Test*

The procedure followed for performing this test was in accordance with the Texas Department of Transportation (TxDOT) method specified by Tex-107-E. Here, the horizontal shrinkage of a bar of soil paste in the mould mixed at its moisture content level of liquid limit state is measured. The samples are air dried at room temperature for twelve hours and then oven dried for twenty-four hours. The length of the dried samples is measured by using vernier calipers and the linear shrinkage was expressed as a percentage of its original length (Chavva, 2002).

*3.4.5 Unconfined Compressive Strength*

The UCS tests conducted were in accordance with ASTM D-2166 standards. Samples were prepared as explained in earlier sections. Cured samples were then placed

on the compressive test platform and loaded at a constant rate which was controlled by a loading device control. Axial load and deformation data was simultaneously collected by a computer attached to the test setup. The maximum axial compressive load at which the sample failed was used to determine the unconfined compressive strength of the samples. This strength usually depends on cohesion and particle interlocking of the soil particles. The UCS test is generally recommended for cohesive or mixed soils and samples.

## 3.5 Summary of Laboratory Results

This section summarizes the results and findings of the laboratory test procedures conducted at The University of Texas at Arlington (Chavva, 2002; Ramakrishna, 2002). The following sub-sections present the conclusions observed from each test.

### *3.5.1 Atterberg Limits*

In the tests which were performed to calculate the Plasticity Index of the different soil samples, it was observed that the control soil has a PI of 22.2%. It was also noticed that the PI of the soils decreased to a maximum extent and almost becomes non-plastic after curing for seven days (Chavva, 2002). This is attributed to chemical reactions such as cationic exchanges and flocculation reactions which resulted in a decrease of the diffused double layer thickness around clay particles. The thinning of the layer causes flocculation and hence decreases the PI values. The decrease of PI values was more evident in lime and cement treated sections than other stabilized

sections (Chavva, 2002). The GGBFS stabilization method also exhibited characteristics similar to cementitious materials.

*3.5.2 Resilient Modulus*

The resilient modulus test was conducted to understand the following aspects:

- Effect of compaction moisture and confining pressure on $M_R$ properties of control and stabilized soil.
- Effects of stabilizers on $M_R$ properties of soil.

The results provided resilient modulus and the coefficient of variation (CV). The CV values varied between 0.2% and 25.8% with an average of 15%. This implies that the present repeated load triaxial test equipment is verified as it provides repeatable results for identical soil samples. Table 3.5 below displays the average resilient modulus values of control and treated soils at a confining pressure of 14 kPa.

Table 3.5 Average resilient modulus values of control and treated soils at a confining pressure of 14 kPa (Ramakrishna, 2002)

| **Designation** | **$M_R$ at Dry 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%) Treated Soil | 268.9 (135.8) | 762.7 (682.4) | 302.6 (266.8) |
| Class F Flyash (15%) and Type V Cement (5%) Treated Soil | 169 (35.9) | 389.8 (309.5) | 186.0 (150.2) |
| GGBFS (20%) Treated Soil | 151.2 (18.1) | 351.8 (271.5) | 195.1 (159.3) |
| Lime (8%) and Polypropylene fibers (0.15%) Treated Soil | 187.7 (54.6) | 80 | 53.7 (17.9) |

For the ranking procedure, a composite soil k-value above 600 ksi was considered as an excellent treatment characteristics. A value between 500 and 600 ksi was considered as good while a value between 300 and 500 ksi was considered as fair treatment characteristics.

The stabilizers were ranked in accordance to the k values determined from the resilient modulus results since subgrade moduli of soil reaction was the main variable for rigid pavement design (Wattanasanticharoen, 2000). The ranking was based on the effectiveness to increase the subgrade modulus of reaction by using the k values reported above. This is presented in Table 3.6. It should be noted that all stabilizers showed considerable improvements to enhance the subgrade modulus reaction properties of control soils.

Table 3.6 Ranking of Stabilizers (Wattanasanticharoen, 2000)

| **Variable** | **Untreated Control Soil** | **Control Soil &Type V Cement** | **Control Soil & Class F Flyash with Type V Cement** | **Control Soil & GGBFS** | **Control Soil & Lime with Fibers** |
|---|---|---|---|---|---|
| Composite k-value (pci) | 310 | 770 | 680 | 610 | 500 |
| Treatment Characteristics | Fair | Excellent | Excellent | Excellent | Good |
| Relative Ranking | 5 | 1 | 2 | 3 | 4 |

As can be seen from the above table, the order of effective treatment is as listed below starting with the best stabilizer.

1. Type V Cement
2. Class F Flyash with Type V Cement
3. GGBFS
4. Lime with Polypropylene Fibers
5. Control Soil (Lime only)

*3.5.3 One Dimensional Free Swell Test*

In this test, the maximum amount of swell was recorded after a 3-day or 72-hour soaking period (Chavva, 2002). Table 3.7 presents the free vertical swell of control and treated soils after three days.

Table 3.7 Free vertical swell strain of control and treated soils (Chavva, 2002)

| **Soil Designation** | **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 Flyash (15%) and Type V Cement (5%) | 0.27 | 0.1 | 0 |
| Lime (8%) and Polypropylene fibers (0.15%) | 0.73 | 0.64 | 0 |
| GGBFS (20%) | 0.43 | 0.1 | 0 |

The decrease in the free swell of all the stabilizers was attributed to the PI decrease due to chemical treatments. Among treated soils, lime and fiber treatment experienced more heaving which is attributed to the presence of fibers. Fibers must

have induced open fabric and low unit weight in treated expansive soils and hence resulted in more heaving than other treated soils.

*3.5.4 Linear Shrinkage Strain Test*

The results for the linear shrinkage swell test are presented in Table 3.8. It can be seen that the linear shrinkage values of the control soil ranged between 1 to 2.3%. All four stabilization methods displayed similar low shrinkage strain results, which were attributed to the decrease in PI values.

Table 3.8 Linear shrinkage strain values of control and treated soil (Chavva, 2002)

| **Soil Designation** | **Linear Shrinkage Strain (%)** |
|---|---|
| Control Soil | 6.2 |
| Type V Cement (8%) | 1.4 |
| Class F Flyash (15%) and Type V Cement (5%) | 1.5 |
| Lime (8%) and Polypropylene fibers (0.15%) | 2.3 |
| GGBFS (20%) | 1.4 |

*3.5.5 Unconfined Compressive Strength*

The UCS test is a special type of test (Unconfined-Undrained) that is commonly used to measure the shear strength of soils under unconfined conditions (Das, 1998). Here, five specimens were tested for each stabilizer at a particular moisture content and the average was calculated (Chavva, 2002). The coefficient of variation was less than 6%, which verifies that the UCS results are repeatable and thus reliable. Table 3.9

below demonstrates that Type V cement yielded the highest value, followed by the Class F Flyash with Type V Cement. The UCS value of lime and fibers was higher at dry of optimum (Chavva, 2002). This was due to the presence of fibers, which enhanced the cohesion intercept of the treated soil specimen, but at the expense of the pozzolanic cementation effects of lime (Chavva, 2002). The same trend was observed for the resilient modulus tests conducted by Ramakrishna (2002). Overall, strength increase in all the treated soil specimens were attributed to the cementitious reactions from the stabilization treatment process.

Table 3.9 UCS strength values (Chavva, 2002)

| **Soil Designation** | **Dry of Optimum (psi) (CV)** | **Optimum (psi) (CV)** | **Wet of Optimum (psi) (CV)** |
|---|---|---|---|
| Control Soil | 31.3 (3.0) | 36.3 (2.5) | 20.9 (3.0) |
| Lime (8%) and Polypropylene fibers (0.15%) | 54.4 (2.4) | 50.9 (4.6) | 35.4 (3.3) |
| Type V Cement (8%) | 200.9(3.3) | 225.8 (3.4) | 198.2 (2.5) |
| Class F Flyash (15%) and Type V Cement (5%) | 123.9 (4.7) | 154.0 (3.6) | 113.9 (4.7) |
| GGBFS (20%) | 99.8 (5.2) | 108.8 (3.2) | 83.7 (5.4) |

Note: CV is the Coefficient of Variation

## 3.6 Summary

The influences of all four stabilization methods on the laboratory tests performed to evaluate the physical soil properties and the engineering properties were addressed in this chapter. In all, Type V cement provided the maximum enhancements

of soil properties followed by Class F flyash with Type V cement, GGBFS, and lastly lime and fibers (Chavva, 2002; Ramakrishna, 2002; Wattanasanticharoen, 2000).

# CHAPTER 4

# DESIGN OF INSTRUMENTATION AND FIELD CONSTRUCTION METHODOLOGY

## 4.1 Introduction

This thesis research aims to design and select the proper stabilization methods to effectively treat sulfate rich expansive soils. For the successful completion of this research, an accurate understanding of each of the present stabilizers is required. Focusing on this objective, the present research was attempted to monitor all pavement sections built on five field treated sections to address the stabilization potential of soils in real field conditions. To study the treated soil behavior, field instrumentation is designed and installed.

This chapter elaborates the steps involved in selecting appropriate instrumentation used to assess the performance of treated subgrades. This requires a knowledge and understanding of site conditions, defining the roles of various sensors, and selection of appropriate data acquisition system and its features. In this chapter, construction procedures followed for each of the five different stabilized sections including the control section are addressed. This chapter covers the method of sensor placement of the sensors in the treated subgrades.

## 4.2 Engineering Properties Evaluated

The selection of engineering properties to be measured in actual field conditions is of vital importance to instrumentation design since the general expenditure of the project instrumentation depends on the number of engineering properties that need to be evaluated. Since the research objective for this project was to evaluate the swell and compressive strain potentials of the treated soil at Harwood Road, strain measurement devices were utilized here to address compressive strain behavior of treated soils. Elevation surveys were also conducted to address swell movements of treated soils.

Due to existing traffic conditions, subgrade soils beneath the pavements are subjected to varying intensities of dynamic loading, thus possibly leading to elastic and permanent deformations. Hence, it is necessary to measure the pressure or load intensities experienced by the treated soils and to explain any corresponding changes in vertical compression strains. To study this pressure property, load cells were used in this research. The data obtained from strain gauges and pressure cells was used to assess the effectiveness of stabilizers and the load carrying potentials of the underlying soil.

## 4.3 Data Acquisition Method

Appropriate data acquisition modules were selected to monitor various sensors that were installed in the field. The parameters to be measured and the rate of data retrieval were the key factors that influenced this decision. If the frequency of data acquisition is less, readings could be taken by using portable modules or read out boxes. Otherwise, an automated data logger would have to be installed which would monitor the readings at regular intervals. In this research, strain changes were expected to be

gradual and would require large intervals of time. Hence, a data acquisition module was deemed suitable and was selected for strain monitoring purposes. Although it would have been preferred that the monitoring for vertical loads be continuous, factors such as costs and shielding stations led to the usage of portable modules.

Based on these requirements, IOTech's Wave Book along with its expansion module (WBK-16) was considered appropriate because of its portability, high precision, and affordable price (Mohan, 2002). Another advantage of this module was the supplementary data acquisition software provided to acquire and interpret the readings obtained from the sensors.

## 4.4 Placement of Sensors

Strain gauges are classified depending on their type of installation as extensometer, surface mounted gauges, vibrating wire, and embedment. For this research, embedment type sensors were considered appropriate as they are more accurate. Also, since the thickness of the treated subgrade layer was only seven inches, placement would not pose any problems. Alternately, the placement location of the sensors along the length and breath of the pavement was given considerable thought.

The main intent was to instrument the most critical section to measure maximum strain. The pavement critical sections were below the wheel path and the track in between both wheel loads (Mohan, 2002). Due to low survivability rates of about 60% of the strain gauges (Green et al., 1985), it was decided to instrument two strain gauges for each pavement section below and in the center of the wheel paths. This would also address the issue of repeatability of the data obtained and would act as a

back-up in case of failure of any gauge. While a higher number of gauges would increase reliability, their number was restricted because of financial constraints.

The location 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 would reduce noise in the readings. Moreover, grouping the sensors decreases the labor effort required to collect the data each time. With these considerations, the sensors were placed as shown in Figure (4.1). The length of the cables were calculated and specified as part of the gauge specification (Mohan, 2002). Figure (4.2) shows the measurement details of two adjacent sections and the wiring detail.

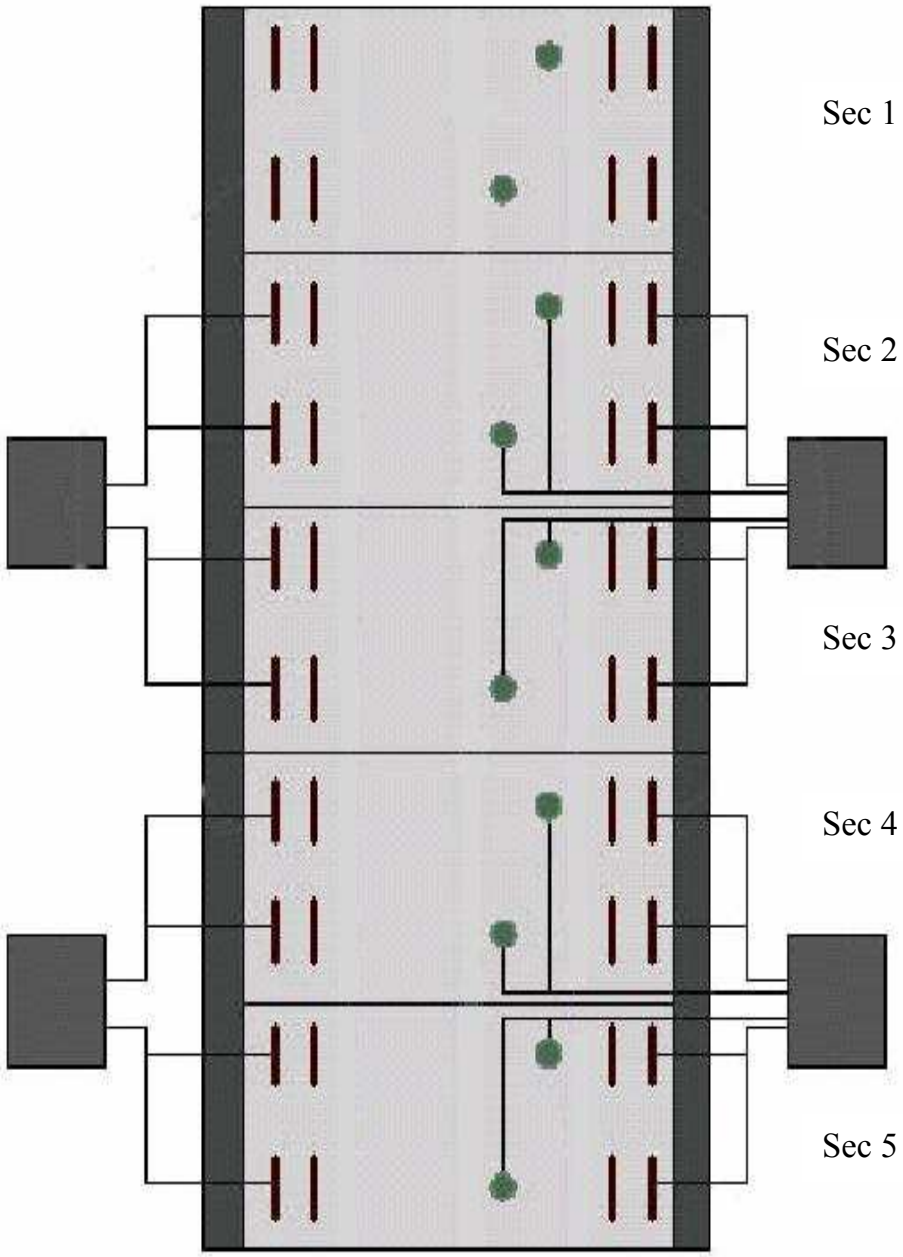


Figure 4.1 Proposed Positioning of Sensors

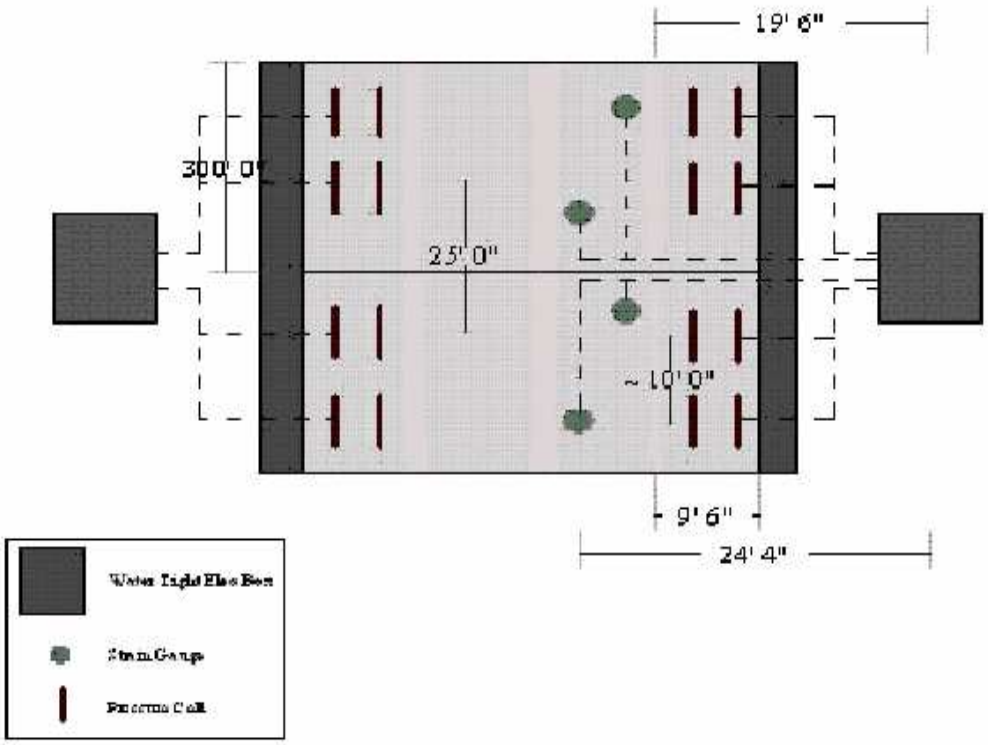


Figure 4.2 Proposed Positioning of Sensors (Details of two sections)

## 4.5 Procurement of Sensors

Based on the factors mentioned earlier in this chapter, many sensors were considered. Information on different sensors including their costs and specifications were obtained from various vendors, manufacturers, and distributors. Table 4.1 below summarizes the sensors that were selected for this research. The lengths of cables were calculated and then communicated to the manufacturer for customization.

The durability, response to climatic changes, performance history, simplicity, overall costs, and compatibility of the sensors were some of the principal aspects taken into consideration before they were procured. Although cost was a major factor in the selection of gauges, priority was also given to obtain high quality instruments as suggested by Sherard (1982). These apart, a laptop computer was also purchased to collect and store the information obtained from the data module. The specifications for the laptop were in accordance with or higher than the requirements stated in the data module manual. A higher memory system was preferred as there is a direct correlation between the scans per second and the amount of memory available (Mohan, 2002).

Table 4.1 Details of Procured Instrumentation

| **Sensor** | **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 |

## 4.6 Integration of Instrumentation, Data Module and Computer

The integration of the wave book and WBK-16 module was done and preliminary tests were conducted as per the manuals of the same (Mohan, 2002). These were then connected to the laptop computer via the parallel port and corresponding interface cables. The data collection software and the drivers for the data module provided by the manufacturer were successfully installed on the laptop computer.

The chosen strain gauges measured strain based on Wheatstone bridge imbalance principles and the EGP-5-350 strain gauge worked on the quarter bridge principle. Hence, a 350 ohm resistor was used to complete the circuit (Mohan, 2002). The pressure cells procured were of full bridge strain gauge type. The WBK-16 had provisions for such upgrades had CN-115 headers to induct the bridge calibration resistors.

To ensure the proper functioning of the sensors, the whole setup was arranged externally on a bread-board. To compensate the resistance increases due to the cable extensions, Mohan (2002) used shunt calibration methods. This was appropriate as the expected strains were of very small magnitude.

## 4.7 Field Construction Methodology

The following sub-sections cover the construction steps of the five treated sections which included a control section where lime was the only stabilizer used.

### *4.7.1 Site Details*

The Harwood Road pavement section in southeast Arlington was selected as the test site for the proposed field studies. Figure 4.3 shows the topographical site map explaining the site location. Five pavement sections, each 300 feet long, were constructed on subgrades stabilized with four select methods and one control stabilization method. All sections have an eight inch thick stabilized subgrade and a six inch thick concrete pavement.

The construction of the sections began on September 20, 2004 and concluded on November 5, 2004. Prior to construction, soil from the test plots was collected and

evaluated in the laboratory. Both the physical and engineering properties of the four treatment methods and the lime method were first determined in the laboratory and are presented in the earlier chapters. These properties were analyzed by a ranking scale system developed from existing literature. Based on the engineering properties evaluated, various proportions of stabilizer were established for field treatments. Table 4.2 presents the different stabilizers considered in this research and their corresponding proportions.

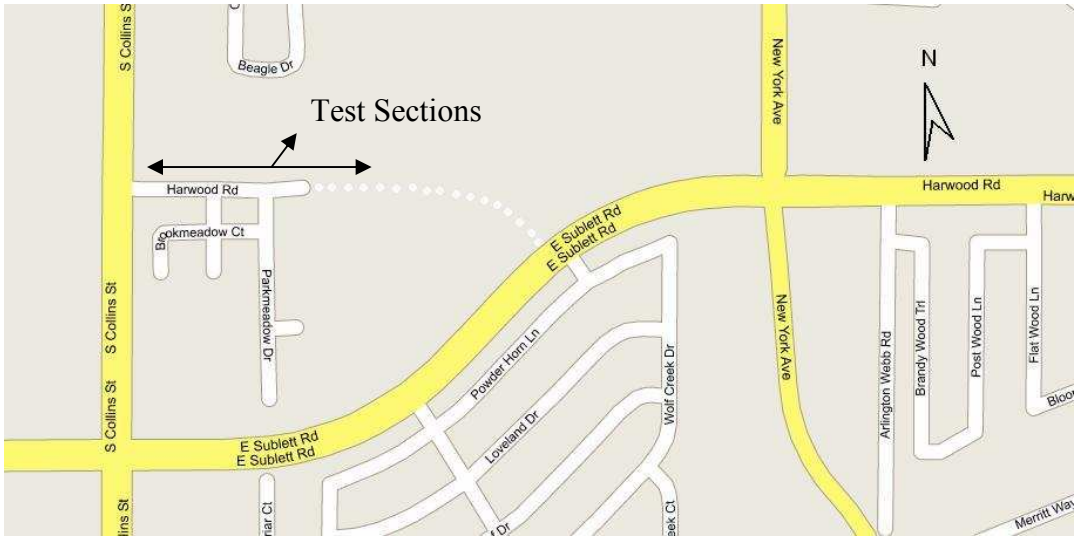

Figure 4.3 Topographic View of Harwood Road

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 |

Figure 4.4 shows the typical Plan-view and cross-section details of a treated and instrumented section.

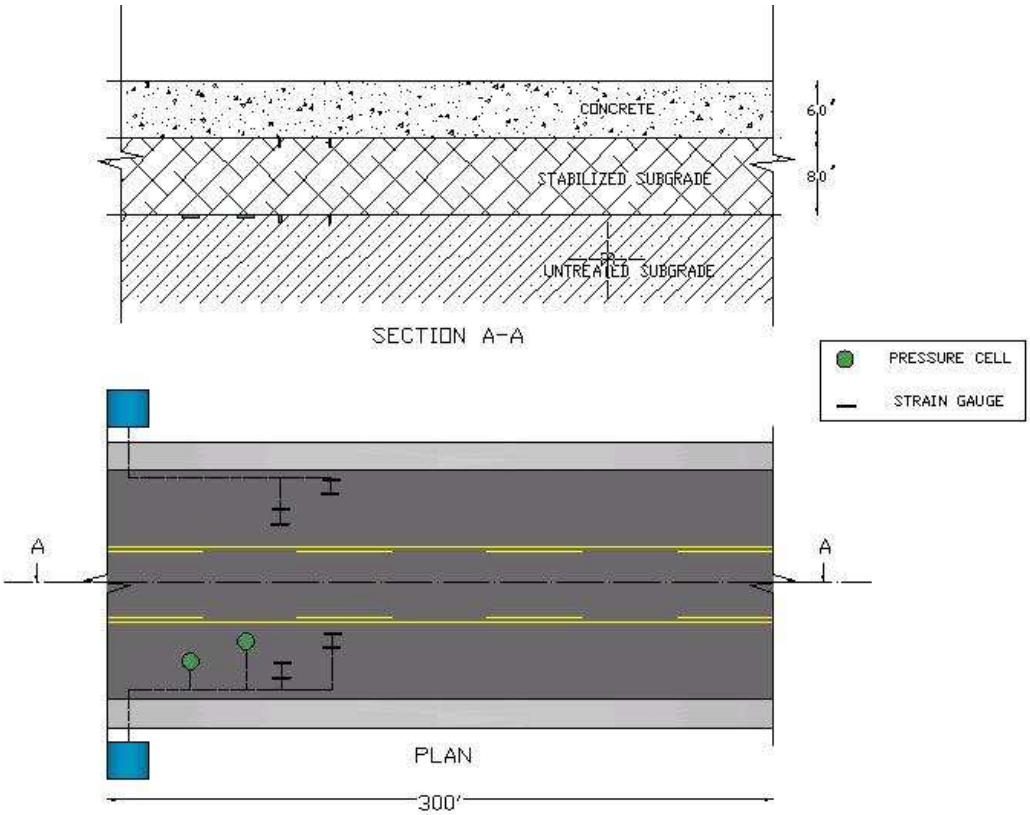


Figure 4.4 Plan and sectional view of a typical section

*4.7.2 Construction of Test Plots*

The section construction was started by first loosening or scarifying the top six to eight inches of soil with a tiller. The construction was done one section at a time, so the scarifying was done for 300 feet of the pavement at a stretch. The process was repeated until there were no lumps or blocks of soil present in the tilled soil in order to ensure an even and homogeneous mixing of the stabilizer with the soil. Figure 4.5 shows the tilling action being carried out in the field.

The stabilizers were then spread evenly over the tilled soil with the help of a truck with a dispensing mechanism. Figure 4.6 shows the application of GGBFS

stabilizer over a tilled section. The stabilizers were mixed with the top soil using the same tiller. A minimum of six passes were made by the tiller for initial soil mixing. This was followed by spraying water uniformly over the entire mixed soil. The section was again mixed for a minimum of seven times. Figure 4.7 shows the application of water to the test section.


Figure 4.5 Scarifying the top soil with tiller

Figure 4.6 Application of the GGBFS stabilizer over scarified soil


Figure 4.7 Application of water over treated section

Figure 4.8 shows the treated section after it has been thoroughly mixed.


Figure 4.8 Thoroughly mixed treated section

The whole section was then compacted by using a sheep-foot roller compacter. Numerous passes were made by the compactor to ensure even compaction and until the required dry density of the treated subgrade was attained. Both the moisture content and the dry unit weights of the compacted material were measured in the field using a nuclear gauge at different locations by an agency appointed by the City of Arlington. Afterwards, the treated section was compacted further with a pneumatic tire roller. These compactors are shown in Figure 4.9 and 4.10 respectively.

Figure 4.9 Sheep-foot roller used for soil compaction

After compaction was performed to the required density, grading was carried out on the section to provide proper drainage both in the longitudinal and transverse directions. The angle of inclination for drainage was in accordance with the City of Arlington standards. Numerous passes were made on the treated section until the requisite grade was achieved. Figure 4.11 shows the grade maintainer that was used as a part of the research.

Figure 4.10 Pneumatic tire roller used for soil compaction

After the grading was completed for all the sections, the excess soil was removed from the pavement and transported to dump sites for disposal.

Figure 4.11 Grade maintainer used for grading the section

*4.7.3 Installation of Sensors*

After construction of the individually treated sections, the pressure cells, strain gauges, and data boxes were installed. The criteria of selection for the above mentioned sensors are discussed in earlier chapters. Both pressure cells and strain gauges were selected as they can provide real time data regarding stresses and strains induced in the soil. Also, elevation surveys and X-Ray Diffraction analysis were conducted to study the swelling characteristics and the formation of ettringite respectively. Elevation surveys were carried out with Total station since these elevation changes provide heave or volume change information of underlying soils.

The sensors were placed after the construction of each stabilized section. This ensured that the weight of the heavy equipment would not damage the sensitivity of the instrumentation.

Initially, concrete pedestals were constructed to place the data-boxes along the side of the pavement. This was to fix the galvanized steel boxes firmly in place to avoid any damage to the DB9 pin heads which were housed in these boxes. The set concrete pedestal is shown in Figure 4.12.

Figure 4.12 Finished concrete pedestal

After the concrete pedestal was set, a set of holes were drilled to fasten the galvanized steel box firmly. This is shown in Figure 4.13 below.

Figure 4.13 Fastening steel box to pedestal

Trenches were then dug carefully from the base of the pedestal into the treated section. Pressure cells were installed under the wheel path and in between the wheel loads. This was done at the interface of the untreated and treated subgrade. Strain gauges were also installed directly under the path of wheel load and in between them, but they were at both the interfaces, i.e. at the juncture of treated and untreated subgrade and at the interface of the treated subgrade and the concrete pavement.

Figure 4.14 shows the trench cutting operation being carried out. Prior to placing the sensors, a sand bed was prepared to ensure full contact of the face of the pressure plate with the backfill. The strain gauges were installed in a vertical manner. Small vertical dig-outs were made to ensure that the strain gauges were properly aligned. The placement of the sensors is shown in Figures 4.15 and 4.16. The wires

from the ends of the sensors were cased within a high density plastic 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, as shown in Figure 4.17.

Figure 4.14 Trench cutting operation

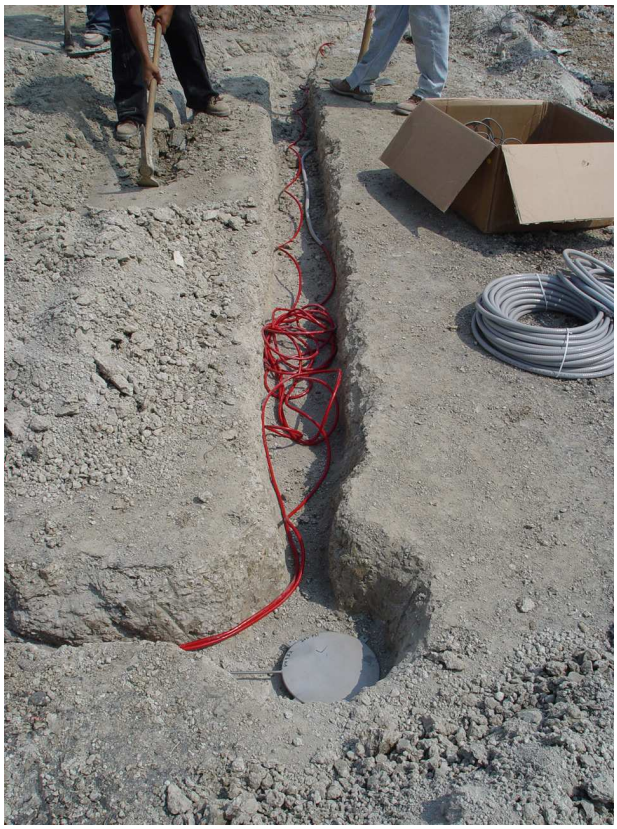

Figure 4.15 Placement of Pressure Cell

Figure 4.16 Placement of Strain Gauge

Figure 4.17 DB9 pins soldered and placed in steel box

Next, the excavated soil was placed back into the trenches and was again compacted using pneumatic tire rollers to ensure that the compaction was similar to adjoining soils. After all the sections were instrumented, initial testing was carried out to check the working of the sensors.

Site visits to collect data were carried out on a regular basis. In case of sudden climatic changes like an occurrence of rainfall, data was collected within twenty-four hours of precipitation. Topographic surveys were conducted with the help of a total station to evaluate the grading changes in both longitudinal and transverse directions. X-ray diffraction analysis was also conducted on samples of treated soil obtained from all sections including the control lime section.

## 4.8 Summary

This chapter provides a comprehensive review and description of the various steps involved in the construction of the four treated sections and one control lime treated section. Sensor details and other assessment methodologies such as elevation surveys and X-ray diffraction studies are also mentioned. Field monitoring was performed for seven months time frame and these results were analyzed to evaluate the performance of the different stabilizers.

# CHAPTER 5

# FIELD DATA MONITORING AND ANALYSIS

## 5.1 Introduction

This chapter provides a comprehensive summary of the data collection methodology, the type of data acquired and its analysis. Other tests included X-ray diffraction (XRD) analysis and elevation surveys, whose results are discussed as well.

The analysis is presented in different sections. The first section covers the data collection methodology including frequency of data collection. The next section includes the waveform filtering which had to be implemented to the raw or unfiltered data to eliminate the noise which was recorded. Section three of this chapter covers the difficulties faced during installation of sensors and during data collection. The fourth section of this chapter deals with the analysis of the monitored data and presents the strain gauge and the pressure cell results individually. Section five details both implementation and results from elevation surveys carried out on all the five treated sections. The penultimate section describes the procedure and results of the X-ray diffraction studies carried out on the stabilized soil samples obtained from the field. This chapter concludes with the summary of all the above-mentioned sections.

## 5.2 Data Collection Methodology

The vehicle used during data collection was a mid-size passenger car with a gross weight of approximately 3,000 pounds. The tire pressure of the vehicle was kept

at a standard of 32 psi to ensure that the contact area with the pavement was equal throughout the data collection period.

The data from the sensors was collected on a weekly basis. However, during extreme weather conditions such as thunderstorms, hurricane conditions and snow-fall events, sensor responses were collected within twenty-four hours of the event. Figure 5.1 below shows the setup of the data box, the data collection module and the laptop, which were used in this research.

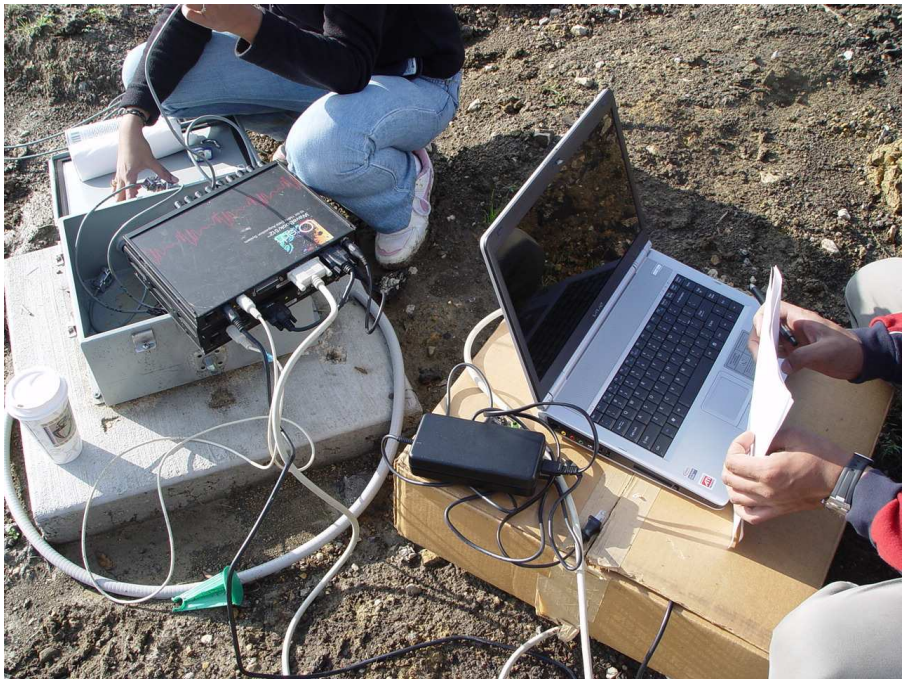

Figure 5.1 Setup for data acquisition

The loading on the sensors was performed using different methodologies. In the first method, no direct dynamic load was applied on the sensors. The only weight acting was the dead weight of the pavement and overburden weight of the treated section. The data from both strain gauges and pressure cells were collected. Consequently, the passenger car was then placed directly above these sensors and the readings were

recorded. The third method of load application was to drive the car back and forth over the sensors while data was being simultaneously collected. This was performed to check if there was any effect of residual strains or pressures that may still be acting on the sensors. These three methods were executed for the sensors installed in all the stabilized sections.

The Harwood Road pavement section built on different stabilized sections was opened to traffic in early February 2005. Data collection was initiated during the month of January 2005 and continued till July 2005.

## 5.3 Waveform Filtering

It was noticed from the data acquired that the values contained a significant amount of noise. This is attributed to the acquisition system as it operates on alternating current (AC power) and the interference from the cables carrying alternating currents.

The interference of the A/C signals created fluctuations in the test results, creating a band of values in the final output results instead of a thin line of output. Thought this could be used for analysis, it was considered appropriate to filter the noise and present the data as final results.

There are three types of filtering which are commonly used to eliminate noise. They are by using filters in the electrical circuitry, transformations and isolation (like transformer isolation during acquisition time). For the sensor data, a running average transformation was considered due to its simplicity and accuracy. For this, MATLAB® (Ver. 7.0.4) was used as it has the ability to process large data files and also contains several built-in filtering functions. Its user friendly approach was an added advantage.

The measured test data was first imported into MATLAB®. A five point running average algorithm was then implemented to reduce the noise. Due to high sampling rate, a single iteration of running average did not give significant reduction in the noise and hence a total of sixty iterations were implemented for strain gauges and thirty iterations were used for pressure cells. The code used in the MATLAB® program is presented in Appendix I.

Figure 5.2 shows the raw data of a strain gauge reading and Figure 5.3 shows the same after it has been filtered. Also, Figure 5.4 presents the raw data from a pressure cell while figure 5.5 shows the same after it has been filtered.

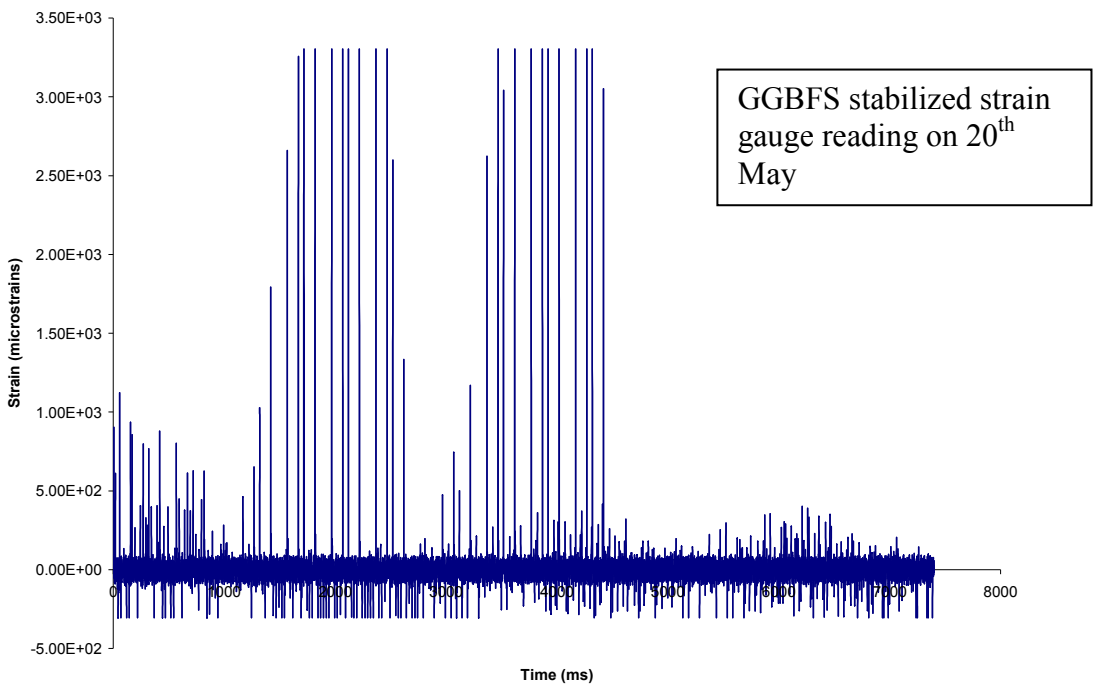


Figure 5.2 Unfiltered Strain Gauge Reading

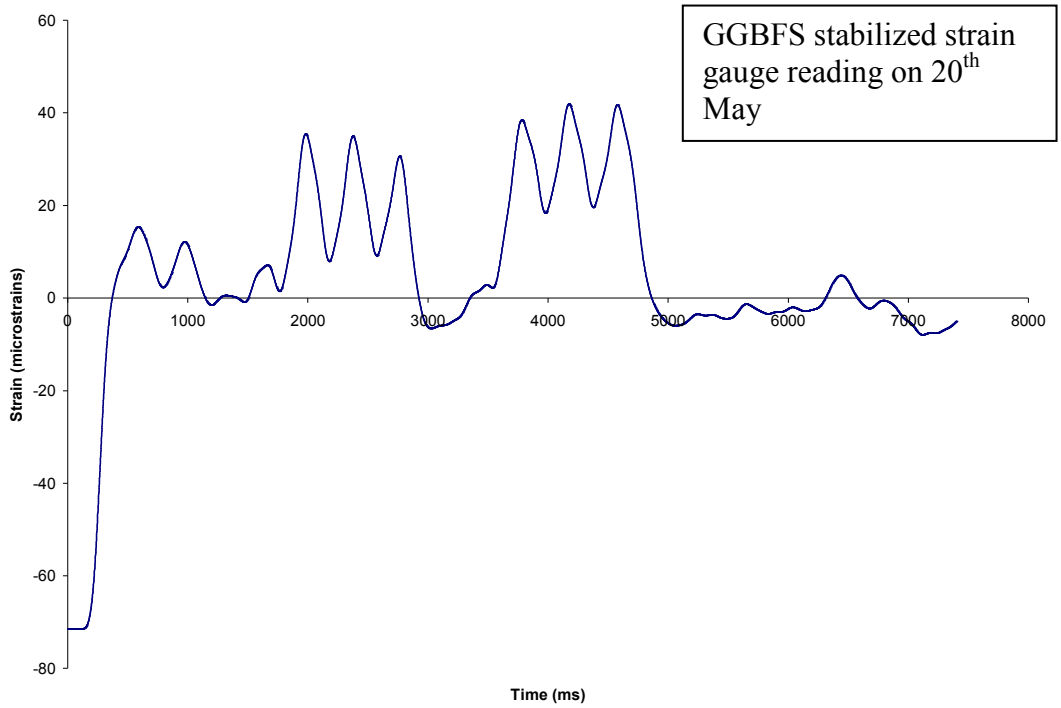


Figure 5.3 Filtered Strain Gauge Reading

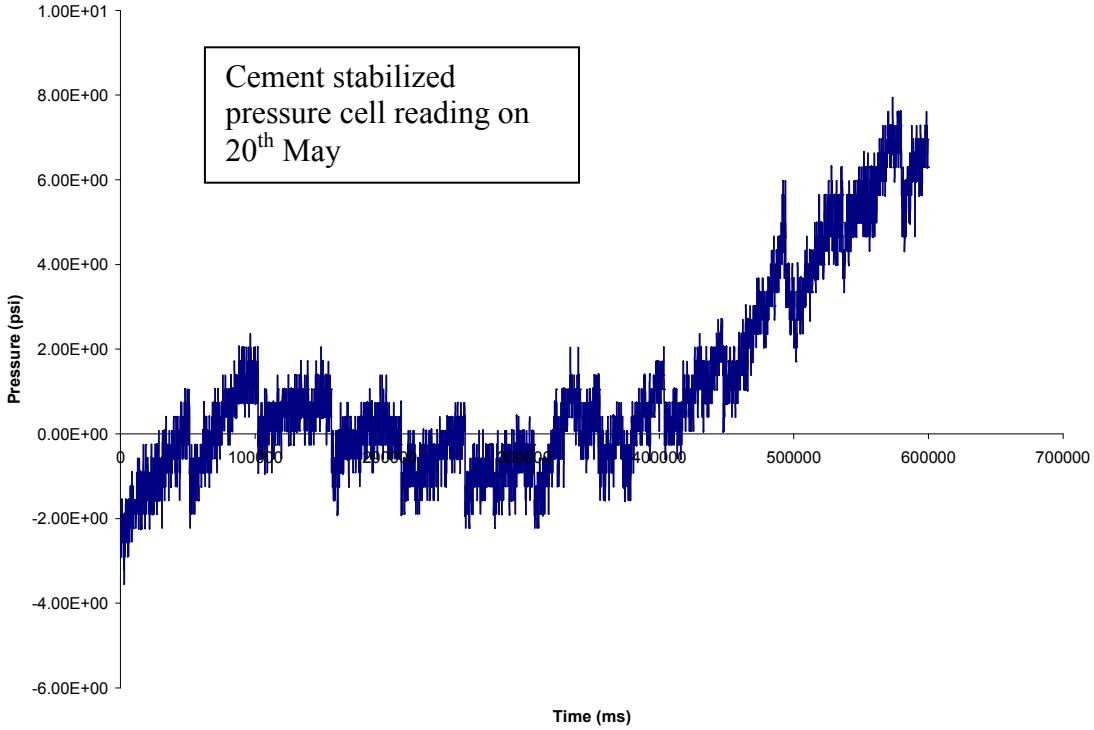


Figure 5.4 Unfiltered Pressure Cell Reading

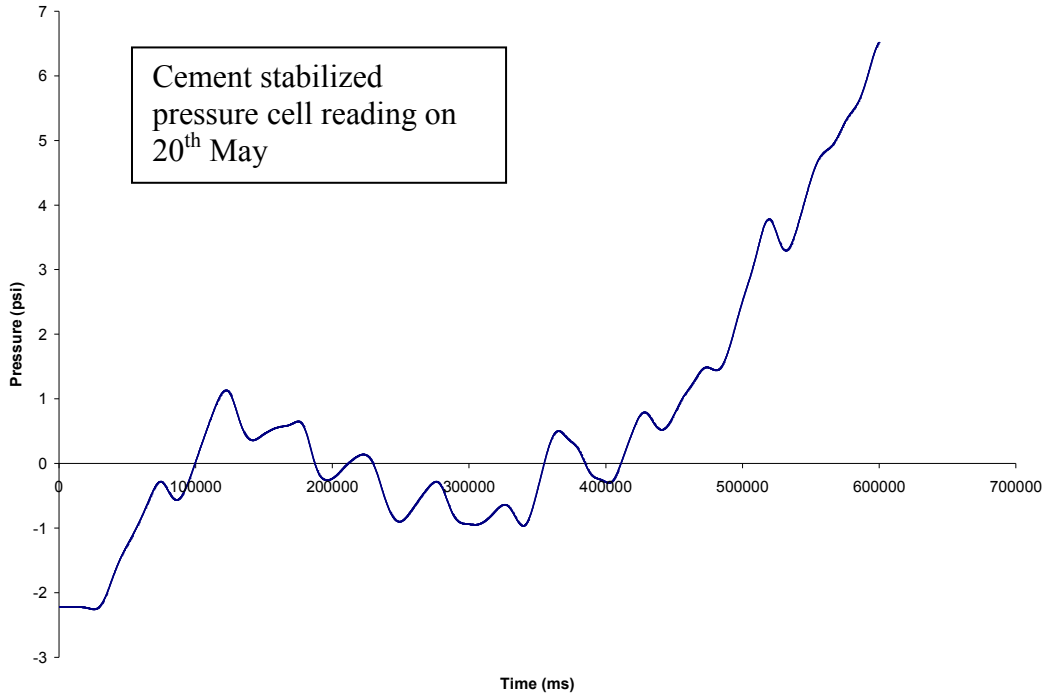


Figure 5.5 Filtered Pressure Cell Reading

In the figures above, we see that the strain gauge and pressure cell readings have been reduced from banded portions to simple lines. Hence, we can infer that the five point running average MATLAB® code provided a considerable reduction in the noise present in the raw data. This procedure was followed in the filtering of the raw data collected from the sensors.

In the figure 5.6 below, strain data is presented when adjacent vehicular movement was allowed. The peaks formed in the graph correspond to vehicular activity on both travel lanes. The difference of these readings (one with loading and one without loading) was used in the evaluation of strains and pressures for each of the treated sections. Also, data was collected from the sensors which were both directly below the

wheel path and in between the wheel loads. Both these data were normalized and the combination was taken to analyze the strain and pressure

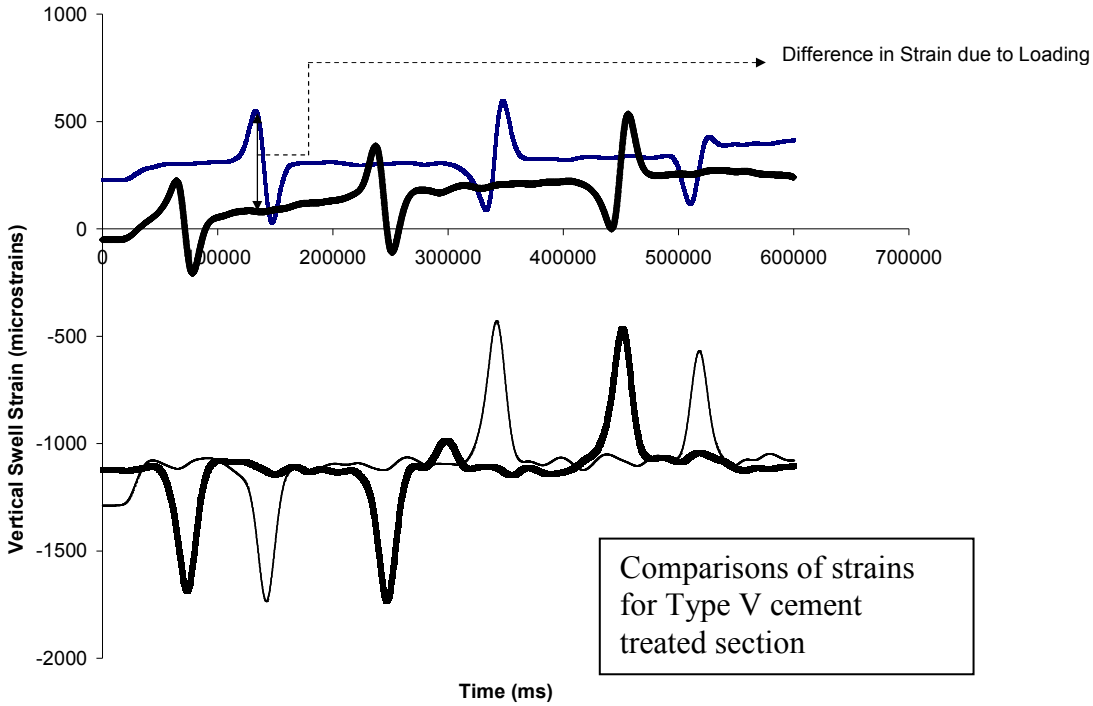


Figure 5.6 Comparison of filtered strain data with adjacent vehicular movement

## 5.4 Difficulties Faced

A few difficulties were faced during the installation of the sensors and during data collection. These are summarized as below:

- During the application of the stabilizers, especially fly ash and GGBFS, there was considerable amount of dust rising due to winds. Also, since the particle size was finer, it resulted in the suspension of the stabilizer in the air. This may be a matter of concern for the neighborhoods and the construction crew involved in the operations. This is demonstrated in figure 5.7 below. One way to reduce

this problem is to use the stabilizer in hydrated form, which will not result in the raising of the stabilizer in dust form.

- The wires from the sensors in Section 3 (GGBFS) were severed by accident. Considerable time was spent trying to fix these wires and to get them to function but proved futile.
- The data acquisition boxes which were water tight did not perform well due to poor drainage in the field. During rains, moisture and soil used to pond near the boxes, which were at a lower level. This is shown in figure 5.8 in the following page.
- The moisture created problems like rusting of DB9 pins of the sensors hampered their performance. These needed periodic cleaning, which was time intensive. This can be seen in figure 5.9.
- The presence of high tension electric cable towers in the vicinity of the sensors has led to noise in the data.
- During collection of data, vehicles using the other side of the road may have influenced the stresses applied to the sensors.
- As mentioned earlier, there was a significant amount of noise in the final output readings. MATLAB filter had to be applied to these to eliminate the noise.

Figure 5.7 Dust due to application of GGBFS stabilizer

Figure 5.8 Presence of water in data box

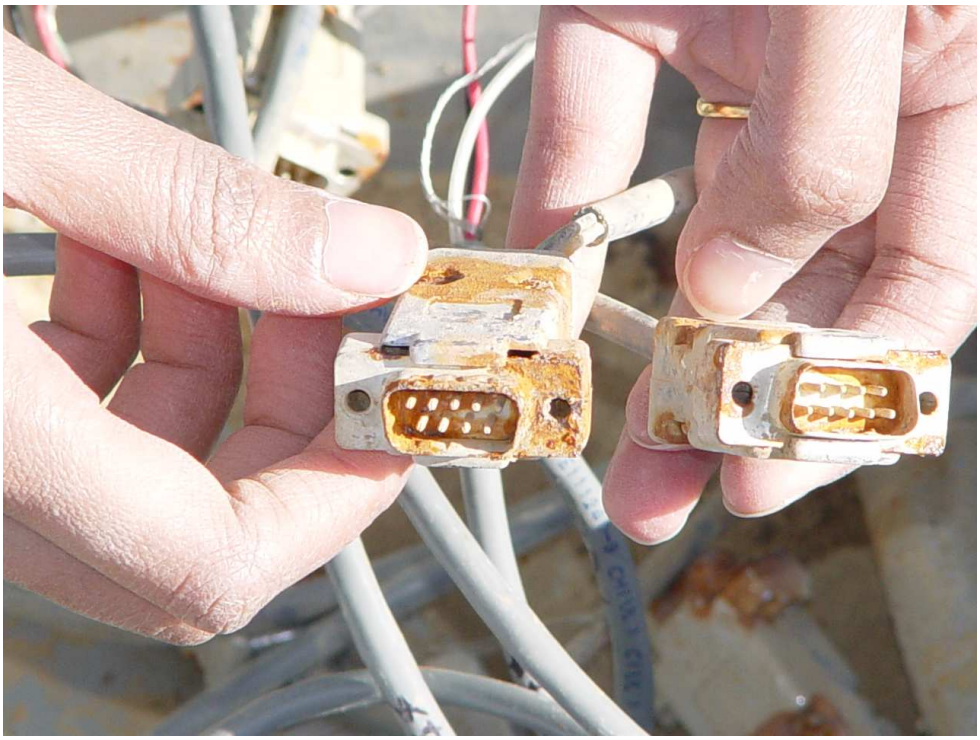

Figure 5.9 Rusted DB9 Pins

## 5.5 Analysis of Monitored Data

The data from each sensor was acquired over a ten minute period. The raw data was obtained in ASCII format which was then converted to engineering data by using Excel. This data was then subjected to five point average filtering using MATLAB® (Ver 7.0.4). The filtered data was then segregated section-wise and normalized by Excel. As mentioned earlier, the strain differences were calculated by subtracting the values which were obtained without the car on the sensors and with the car placed over the sensors.

The data from the different files were then normalized and their averages were calculated using spreadsheets. This data was again segregated based on the date of acquisition. This edited, filtered and segregated data was then compared against each of

the sections in bar graph format, which are presented below. Figure 5.10 below shows the bar graph for the comparison of strains which were obtained in the month of February 2005. These can be classified as initial readings as these were taken immediately after opening the roads to traffic. Figure 5.11 shows the intermediate readings which were obtained in the month of May 2005. In conclusion, figure 5.12 shows the final comparison strains which were obtained in the month of July 2005.

In all three figures, the lime treated control section experienced large vertical strains under vehicular loads. Other treated sections did not undergo any appreciable compressions, explaining the stiffer material being formed with the respective chemical treatment. To further understand the enhancement of stabilization, variations in vertical strains were determined and plotted against the elapsed time in the form of months (Figure 5.13).

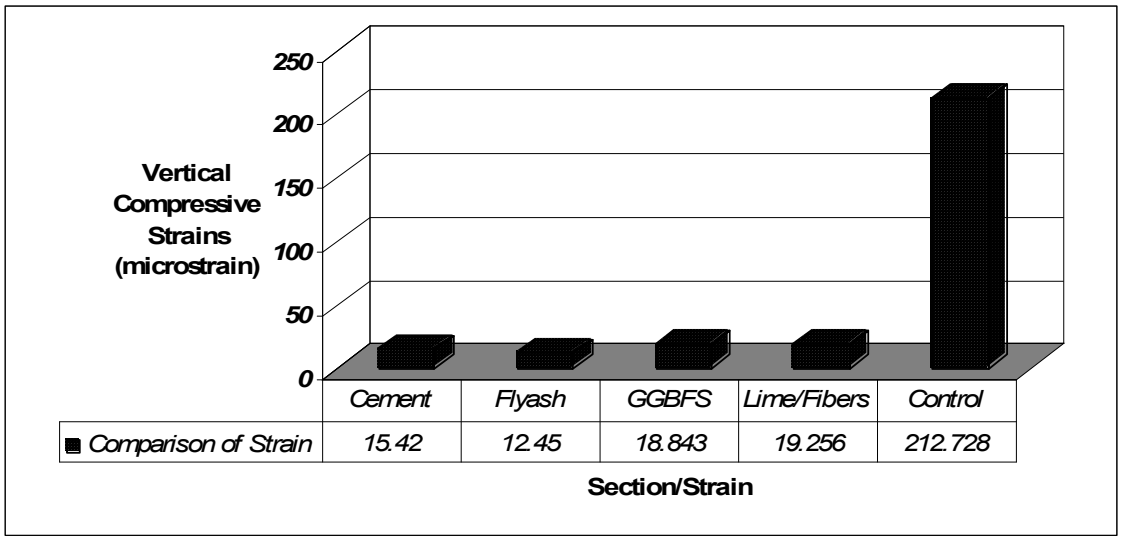


Figure 5.10 Comparison of strains in February, 2005

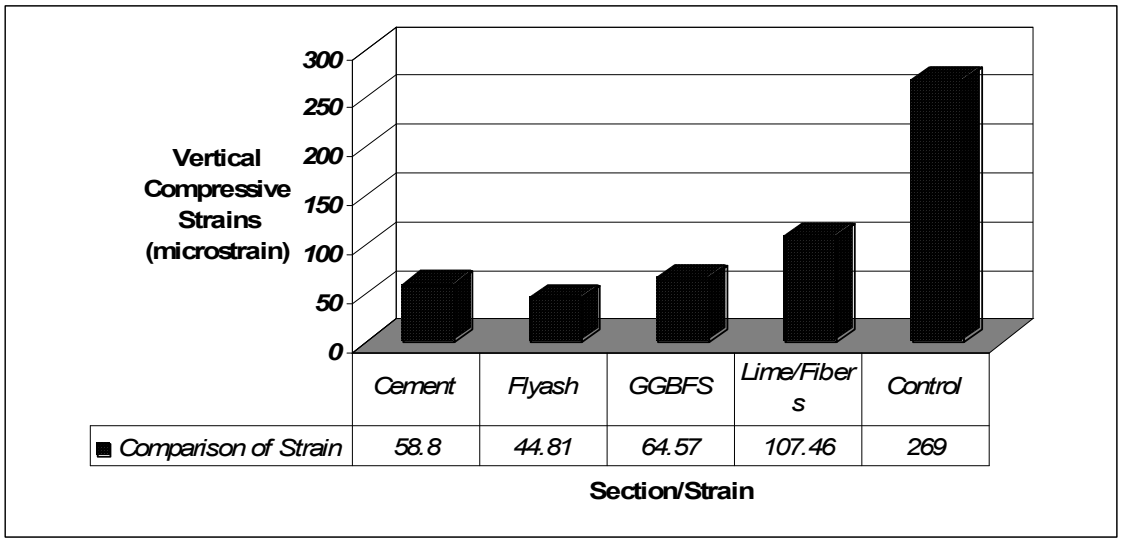


Figure 5.11 Comparison of strains in May, 2005

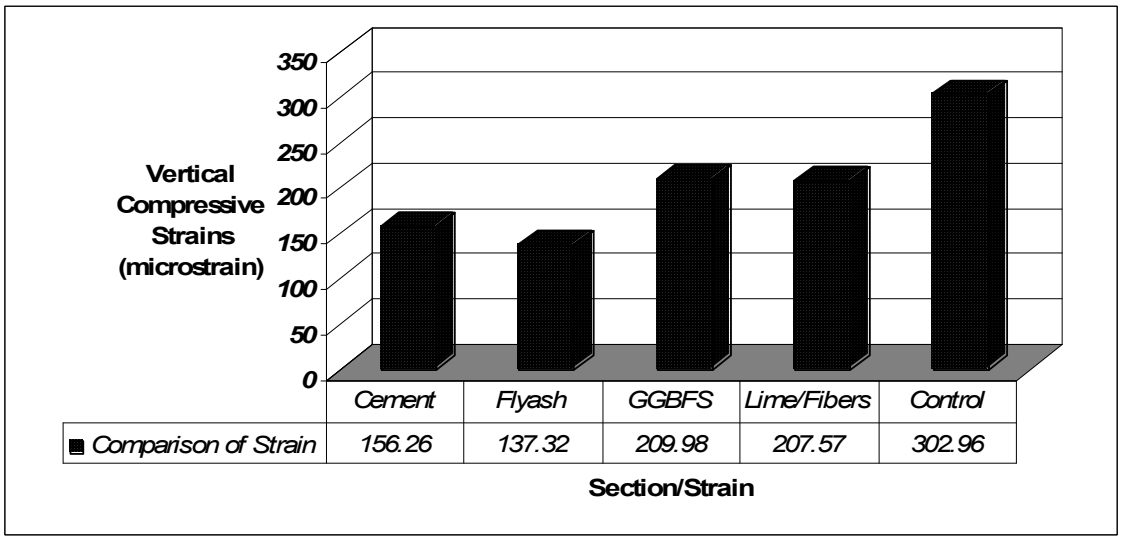


Figure 5.12 Comparison of strains in July, 2005

As can be seen from the above graphs, the strain value is highest in the case of the control lime section. The performances of the other sections are better than the control section. From figure 5.12, it is noticed that fly ash and cement stabilization has the least amount of strain value.

Figure 5.13 presents the comparisons of the increase in vertical strains over the entire period of data collection. It is noticed that the initial value of control section is quite high when compared to the other sections. However, due to stiffening with curing, the consequent data values from this section gave consistently same strain readings. This explains that the lime treatment provided enhancements that are time dependent. Other sections showed stiffening with curing, except that they were slightly less time dependent beyond 30 days of curing. However, it should be mentioned here that all other chemical stabilizers were more effective in enhancing sub-soils during the first month following the field treatment. .

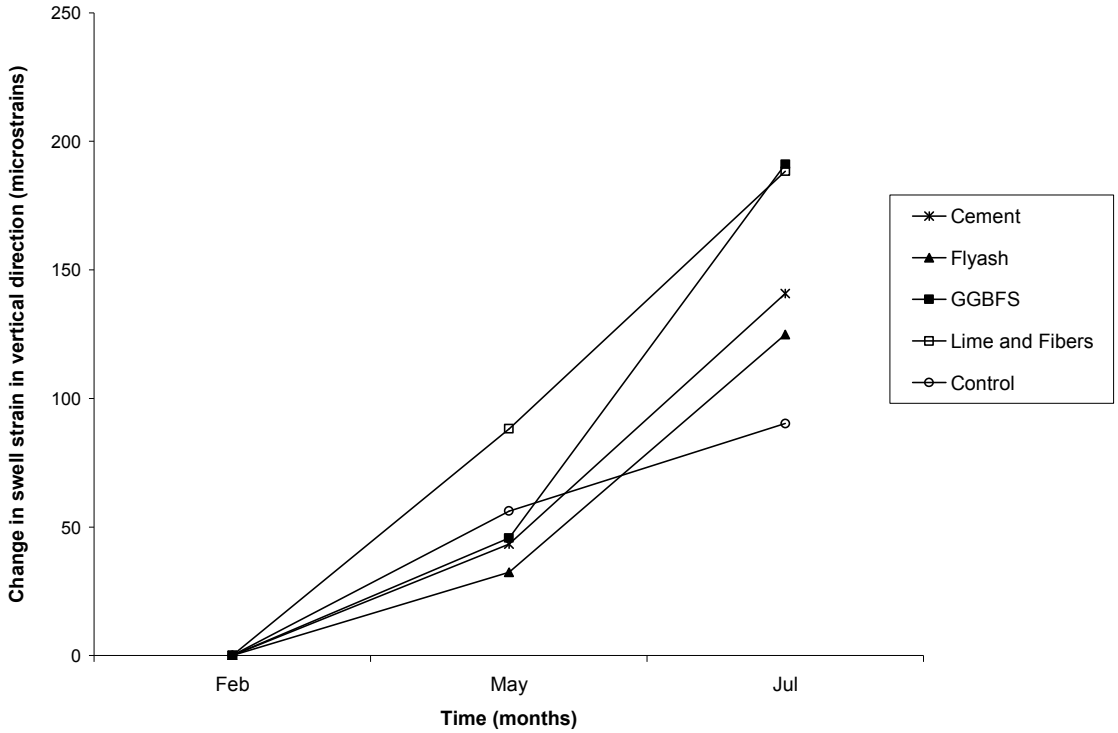


Figure 5.13 Comparison of swell strain of all treated sections

The same procedure was followed for the data obtained from the pressure cells. The bar graphs are presented below. However, in the case of pressure cells installed in

GGBFS stabilized base, the readings could not be obtained as the wires were severed and could not be repaired since they are installed underneath the pavements. Figure 5.14 shows the pressure cell readings in bar graph format for the data obtained in the month of February 2005. Figure 5.15 shows the bar graphs of the pressure cell data obtained in the month of July, 2005. It should be noted that these pressure readings are due to overburden weights of pavements and treated soils above the pressure cells.

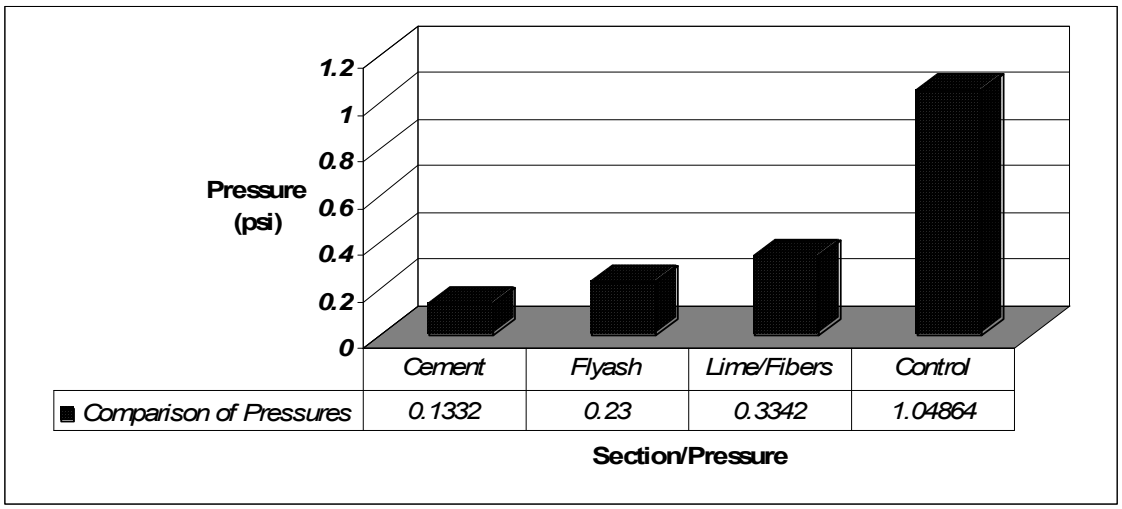


Figure 5.14 Comparison of pressures in February, 2005

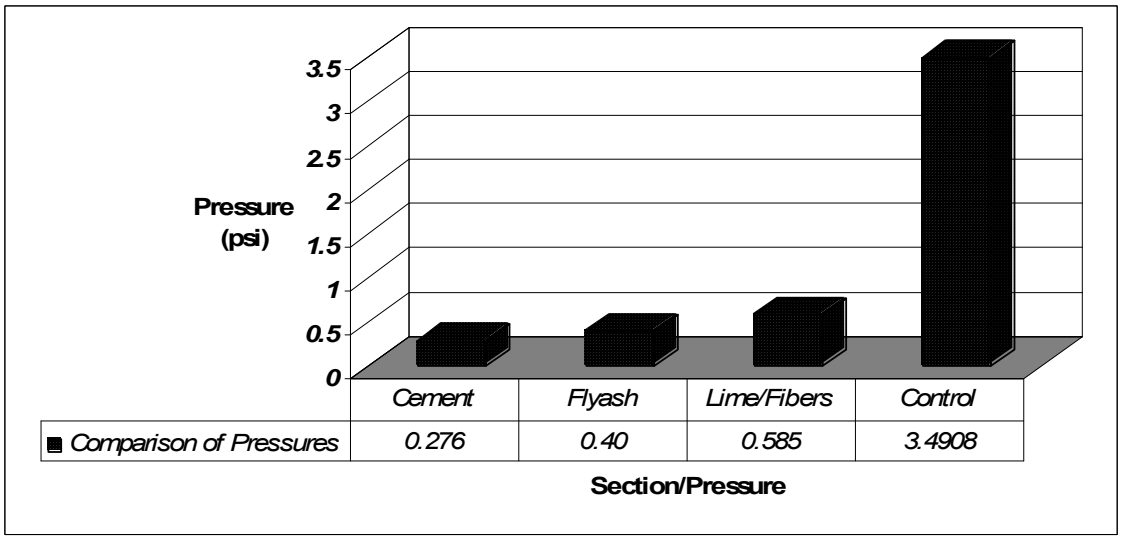


Figure 5.15 Comparison of pressures in July, 2005

From the above pressure and strain comparisons, it is seen that the cement treated section performed the best under the loading conditions. This was followed by the flyash and cement treated section. Third and fourth were GGBFS and the lime and fibers section respectively. The performance of the control section (lime treated) was moderate since the pressure readings are still low.

## 5.6 Elevation Survey Results - Comparison

In order to assess the heave related movements of the stabilizers in the field, elevation surveys were performed over a period of eight months. Figure 5.16 shows the total station equipment used to conduct this survey.

Figure 5.16 Total Station used for Elevation Survey

Eight points were chosen in each section, four along each lane, to obtain elevation results. The plan-view of the elevation survey points and reference total station point is shown in figure 5.17. These were evenly spaced at sixty feet intervals. The benchmark or reference point chosen was independent of the heaving of the pavement and a nearest permanent non-heaving structure was chosen. The height of the instrument and other details were calibrated every time the readings were taken. The results for the elevation surveys are shown below in figure 5.18. It is seen here that the lime treated section suffered the maximum amount of heave than any other section. Cement and fly ash-cement mixtures exhibited the next highest swell movements. Note that the final heave movements are resulting from natural or man-made sulfate heaving as well as compression or settlements under dynamic traffic loading. Overall, the final decrease in elevations is attributed to the summer season started in late May, 2005.

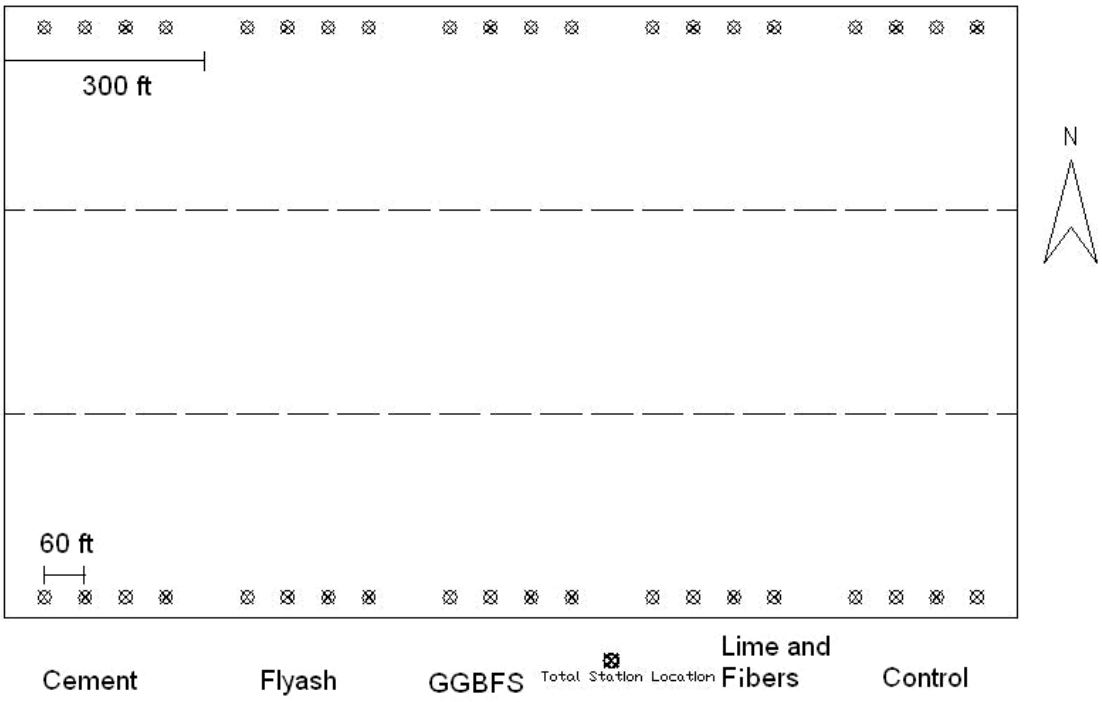


Figure 5.17 Plan-view of elevation survey points

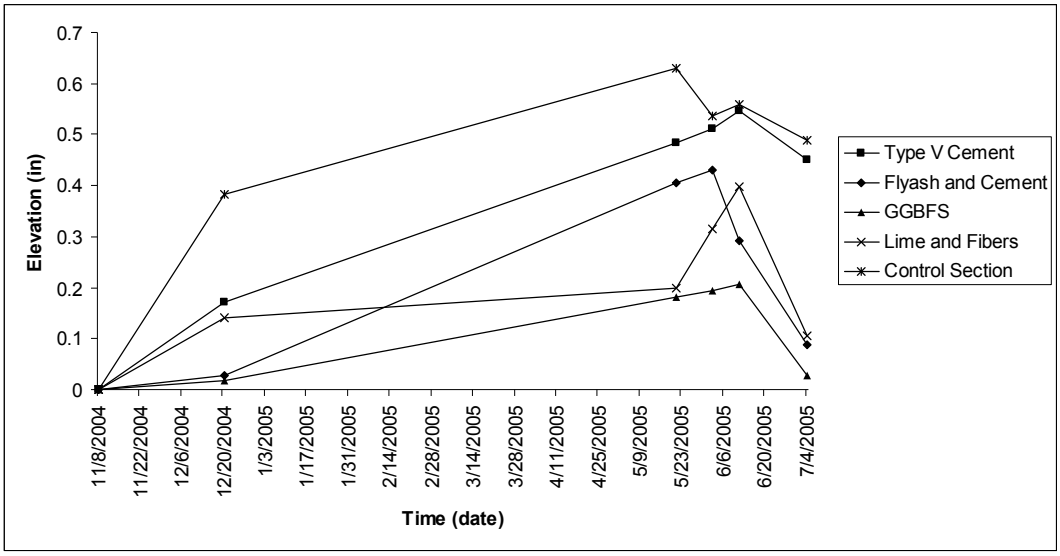


Figure 5.18 Elevation survey results

## 5.7 Mineralogical Studies: X-ray Diffraction Analysis

The lime treated control section displayed the maximum amount of heave from the elevation survey results. Since the main intent of this research is to address sulfate heave, an attempt was made to study crystalline mineral formation in the present treated soils via mineralogical studies. Hence, samples were collected randomly from different plan locations of the treated sections.

X-ray diffraction analysis was then conducted on these samples to study the presence of ettringite formation. The soil samples were first obtained from the field and then oven dried and pulverized. These were then subjected to the X-ray diffraction machine. The analysis for all the five sections is shown below in figures 5.19 through 5.28. Two samples were analyzed from each treated section to check for consistency and repeatability.

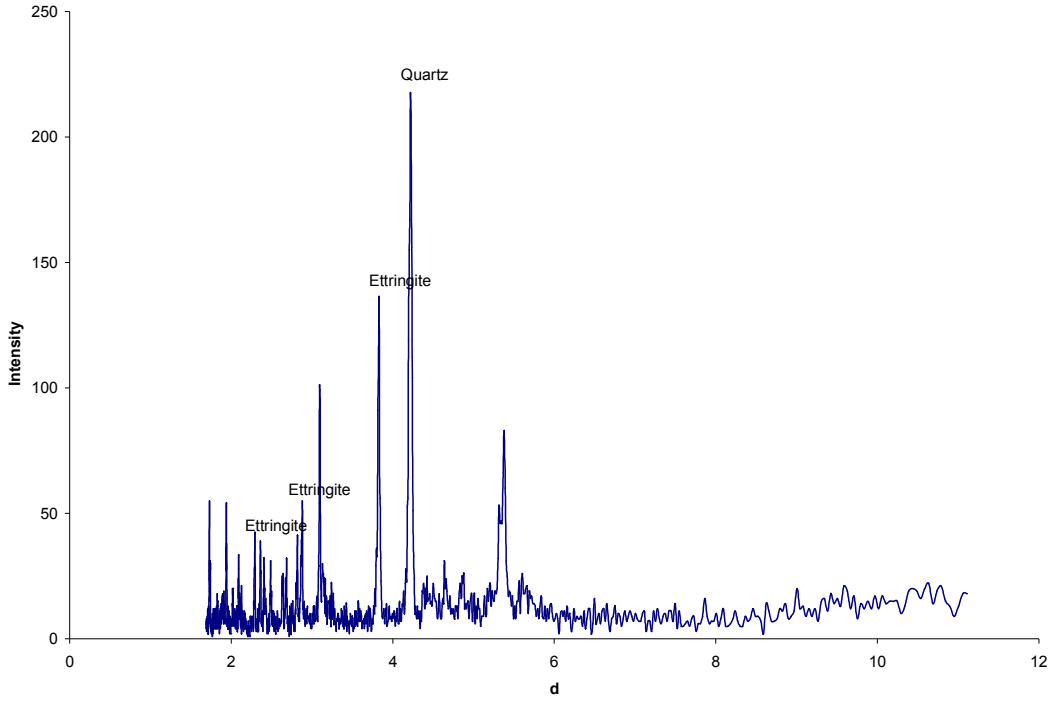


Figure 5.19 XRD analysis for Type V cement stabilization (Sample 1)

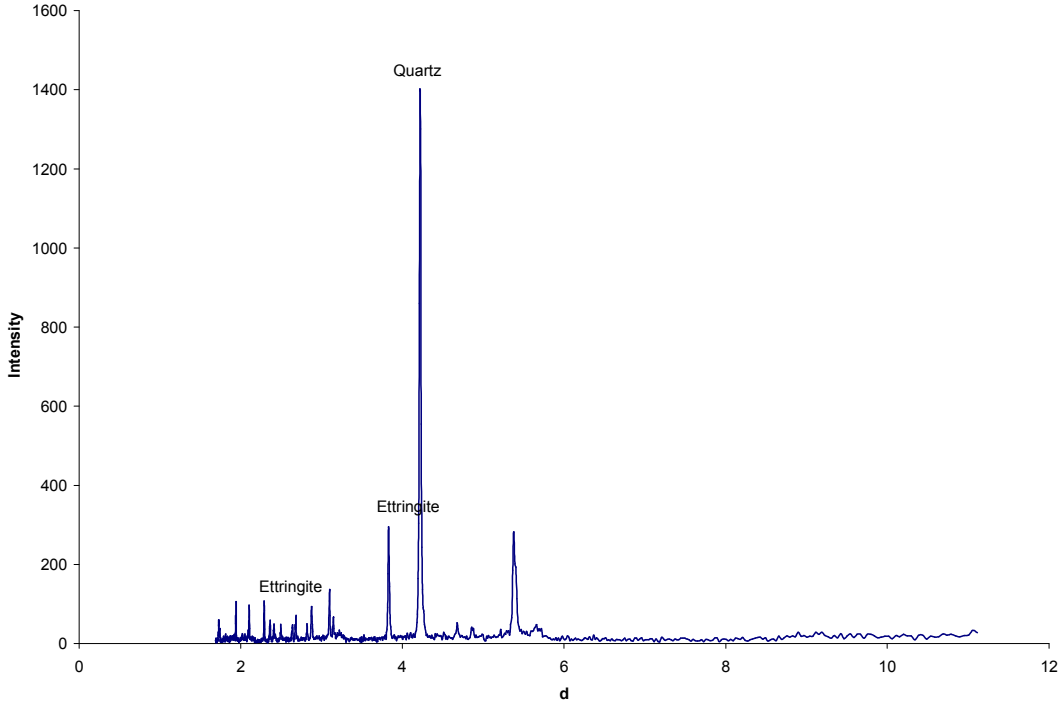


Figure 5.20 XRD analysis for Type V cement stabilization (Sample 2)

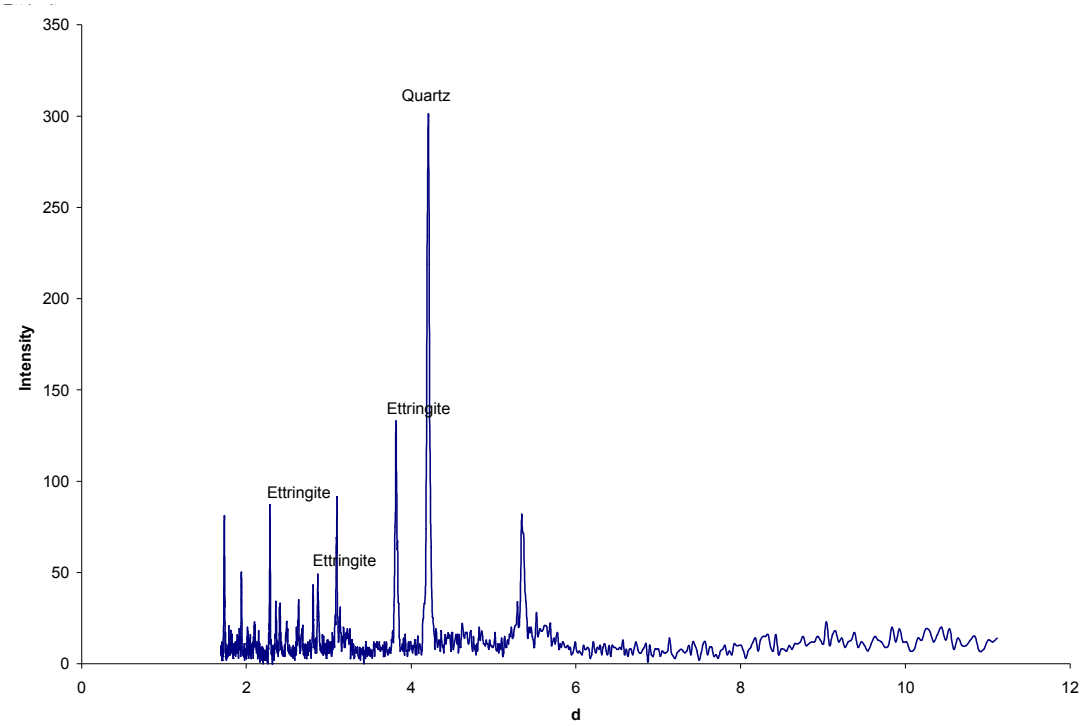


Figure 5.21 XRD analysis for Flyash and Cement stabilization (Sample 1)

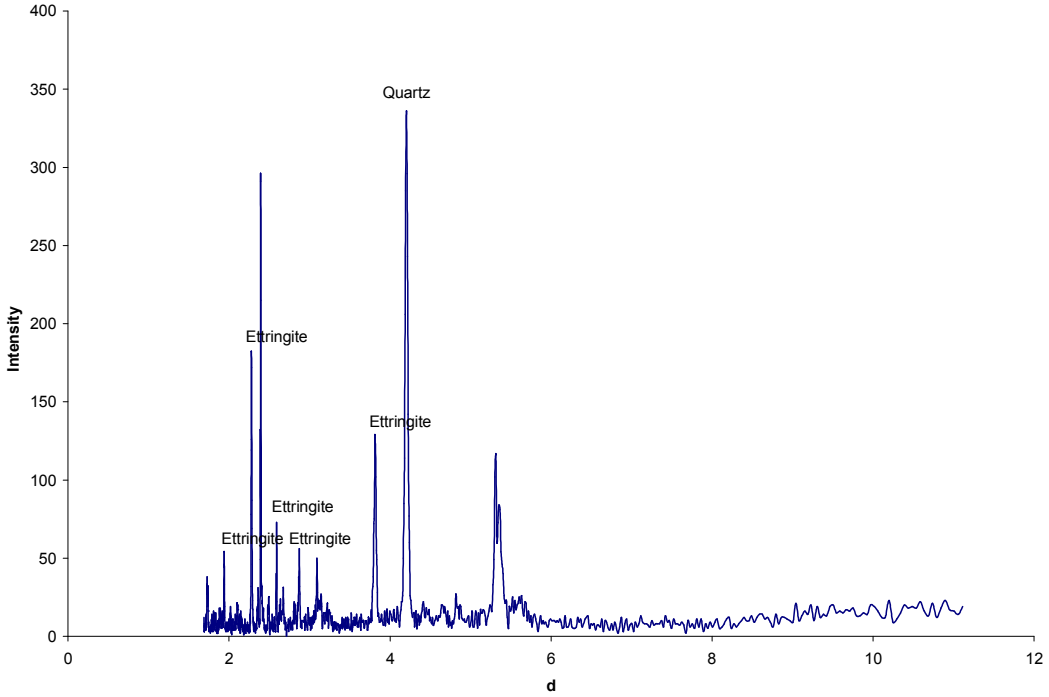


Figure 5.22 XRD analysis for Flyash and Cement stabilization (Sample 2)

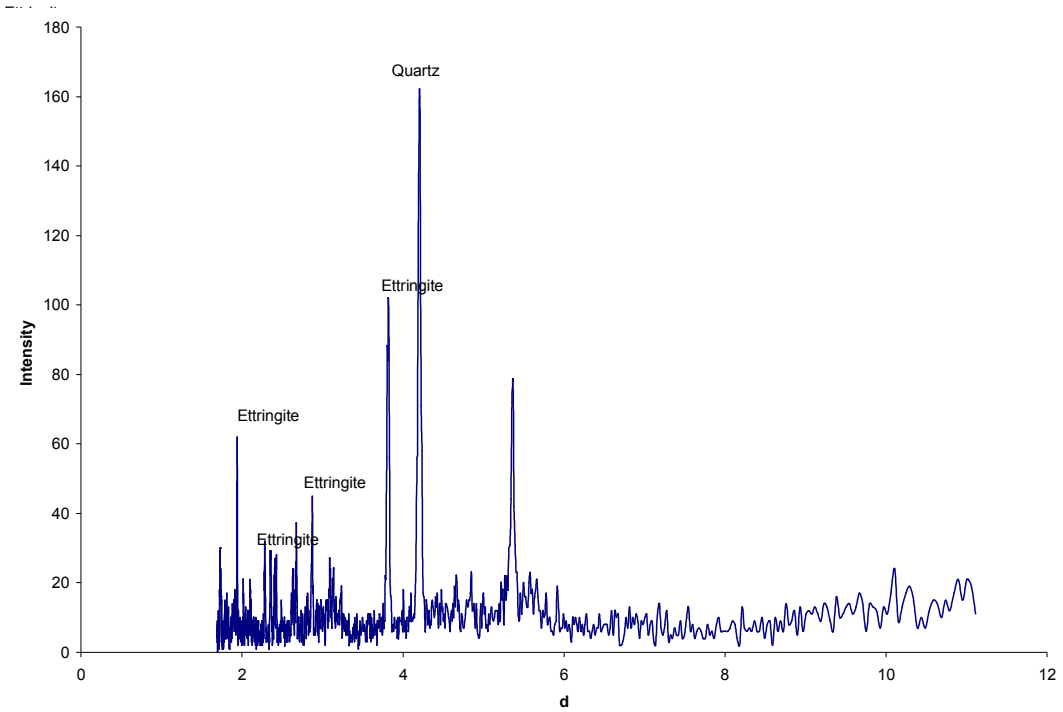


Figure 5.23 XRD analysis for GGBFS stabilization (Sample 1)

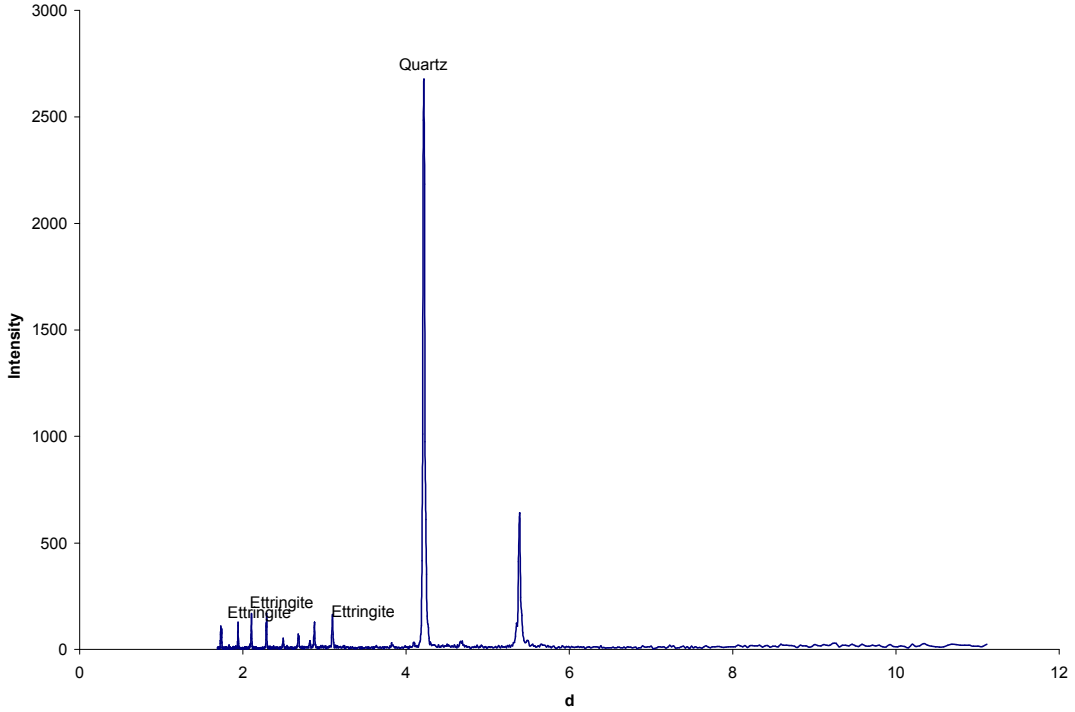


Figure 5.24 XRD analysis for GGBFS stabilization (Sample 2)

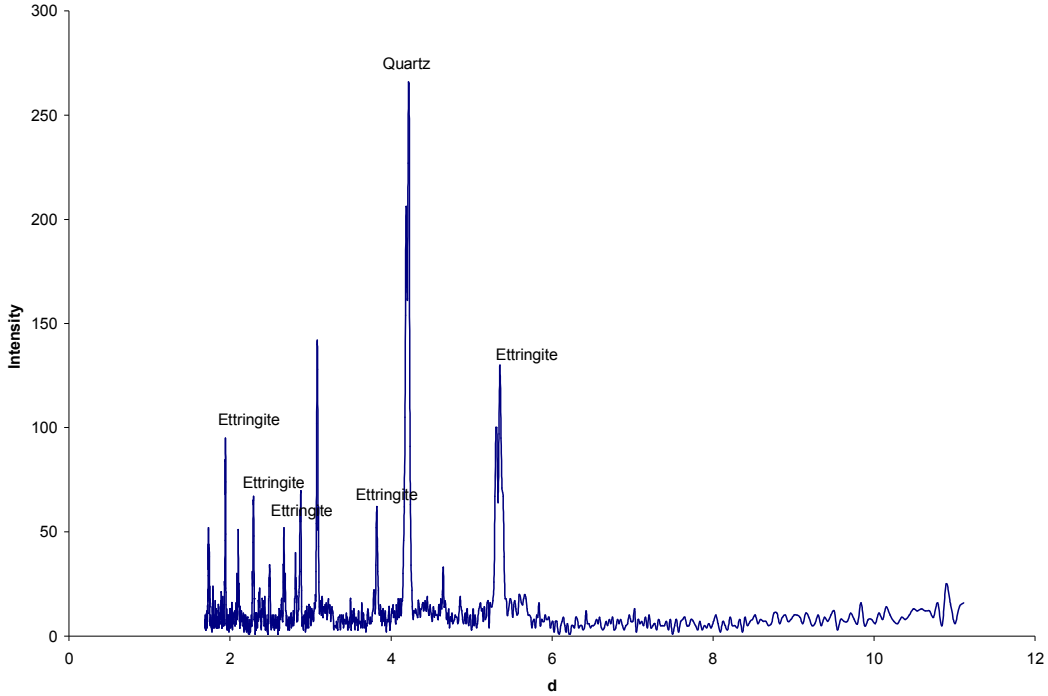


Figure 5.25 XRD analysis for Lime and fibers stabilization (Sample 1)

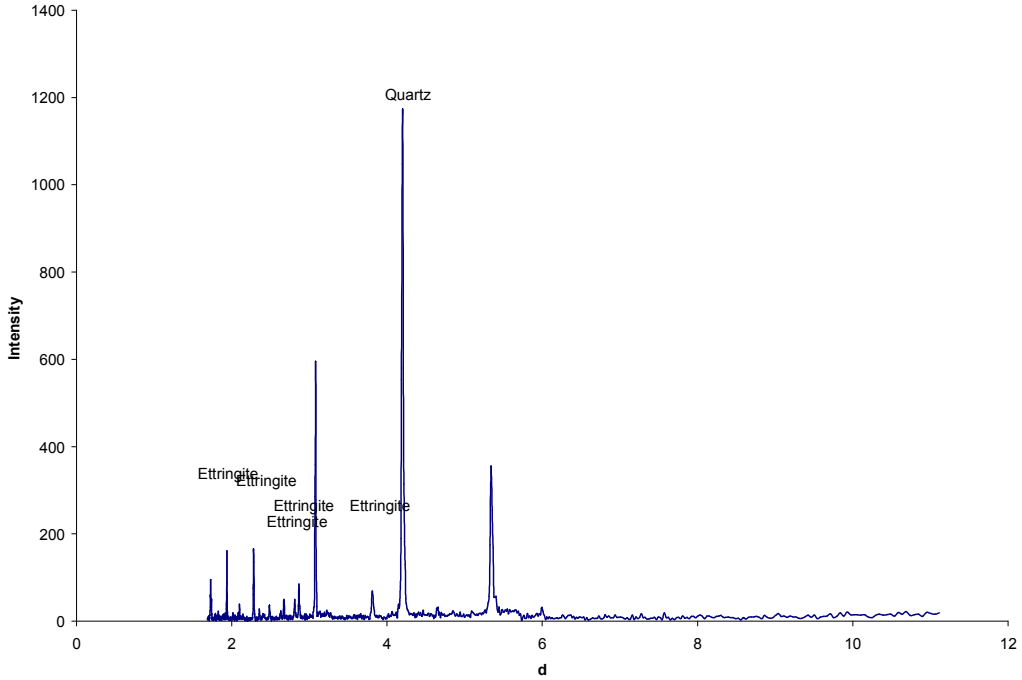


Figure 5.26 XRD analysis for Lime and fibers stabilization (Sample 2)

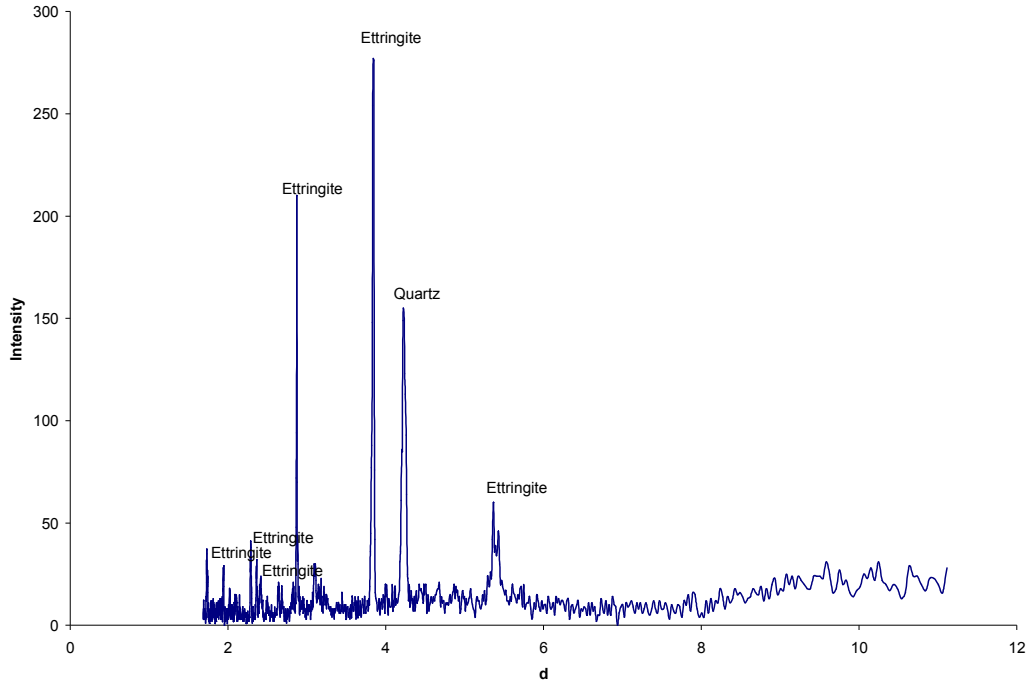


Figure 5.27 XRD analysis for Lime stabilization (Sample 1)

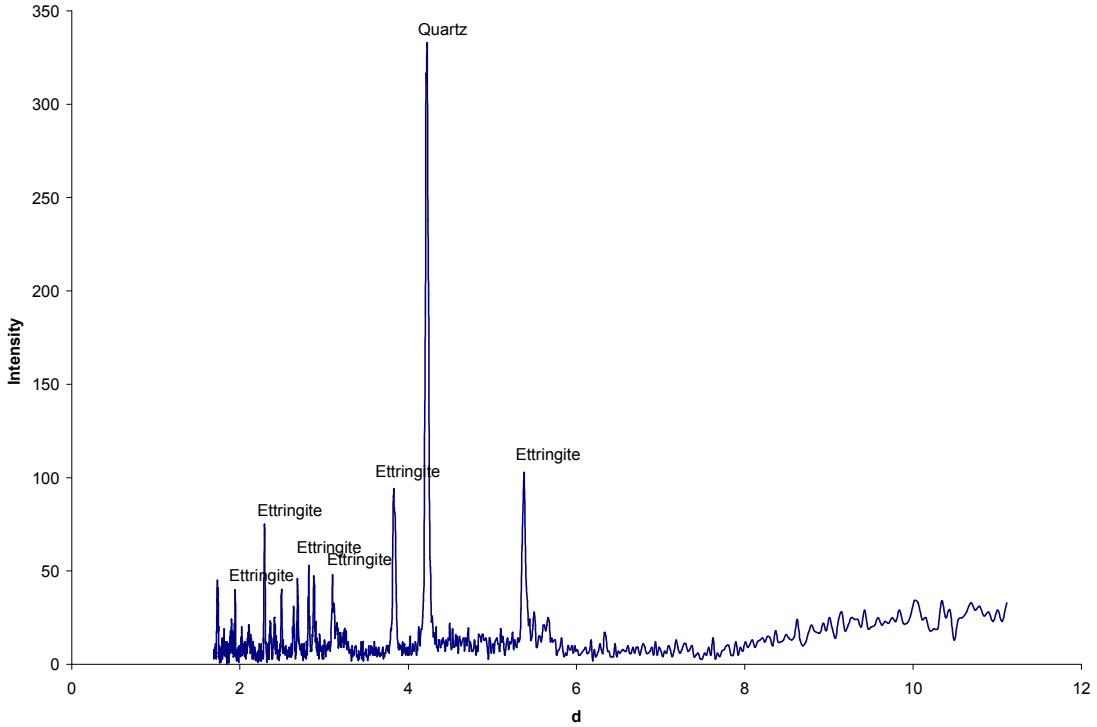


Figure 5.28 XRD analysis for Lime stabilization (Sample 2)

Table 5.1 X-ray diffraction of Type V Cement Section

| Element | d-Spacing | Type V Cement | |
|---|---|---|---|
| | | Sample 1 | Sample 2 |
| Ettringite | 9.67 | | |
| Ettringite | 5.6 | | |
| Ettringite | 4.98 | | |
| Ettringite | 4.68 | | |
| Ettringite | 3.88 | x | x |
| Ettringite | 3.48 | | |
| Ettringite | 3.24 | | |
| Ettringite | 2.76 | x | |
| Ettringite | 2.56 | | |
| Ettringite | 2.204 | x | x |
| Ettringite | 1.946 | x | x |
| Ettringite | 1.663 | | |

Table 5.2 X-ray diffraction of Flyash and Cement Section

| Element | d-Spacing | Flyash and Cement | |
|---|---|---|---|
| | | Sample 1 | Sample 2 |
| Ettringite | 9.67 | | |
| Ettringite | 5.6 | | |
| Ettringite | 4.98 | | |
| Ettringite | 4.68 | | |
| Ettringite | 3.88 | x | x |
| Ettringite | 3.48 | | |
| Ettringite | 3.24 | | |
| Ettringite | 2.76 | x | x |
| Ettringite | 2.56 | | x |
| Ettringite | 2.204 | x | x |
| Ettringite | 1.946 | | x |
| Ettringite | 1.663 | | |

Table 5.3 X-ray diffraction of GGBFS Section

| Element | d-Spacing | GGBFS | |
|---|---|---|---|
| | | Sample 1 | Sample 2 |
| Ettringite | 9.67 | | |
| Ettringite | 5.6 | | |
| Ettringite | 4.98 | | |
| Ettringite | 4.68 | | |
| Ettringite | 3.88 | x | |
| Ettringite | 3.48 | | |
| Ettringite | 3.24 | | |
| Ettringite | 2.76 | x | x |
| Ettringite | 2.56 | | |
| Ettringite | 2.204 | x | x |
| Ettringite | 1.946 | x | x |
| Ettringite | 1.663 | | |

Table 5.4 X-ray diffraction of Lime and Fibers Section

| Element | d-Spacing | Lime and Fibers | |
|---|---|---|---|
| | | Sample 1 | Sample 2 |
| Ettringite | 9.67 | | |
| Ettringite | 5.6 | x | x |
| Ettringite | 4.98 | | |
| Ettringite | 4.68 | | |
| Ettringite | 3.88 | x | x |
| Ettringite | 3.48 | | |
| Ettringite | 3.24 | | |
| Ettringite | 2.76 | x | x |
| Ettringite | 2.56 | | |
| Ettringite | 2.204 | x | x |
| Ettringite | 1.946 | x | x |
| Ettringite | 1.663 | | |

Table 5.5 X-ray diffraction of Lime Section

| Element | d-Spacing | Control Section | |
|---|---|---|---|
| | | Sample 1 | Sample 2 |
| Ettringite | 9.67 | | |
| Ettringite | 5.6 | x | x |
| Ettringite | 4.98 | | |
| Ettringite | 4.68 | | |
| Ettringite | 3.88 | x | x |
| Ettringite | 3.48 | | |
| Ettringite | 3.24 | | x |
| Ettringite | 2.76 | x | x |
| Ettringite | 2.56 | | |
| Ettringite | 2.204 | X | x |
| Ettringite | 1.946 | X | x |
| Ettringite | 1.663 | | |

The above tables 5.1 through 5.5 show the presence of ettringite at the corresponding standard basal or d-spacing values of Ettringite mineral. It is observed that though all treated section exhibit traces of ettringite formation, the maximum presence is noticed in the lime and fibers and lime treated section. The compound ettringite is formed by the reactions among sulfates, reactive alumina and calcium from the stabilizer and this mineral reacts and hydrates in the presence of water. It is known that this mineral can expand twice the original volume during hydration and such swelling cause severe distress to pavements overlying on these treated soils. Also, from the above graphs, we can state that Type V cement stabilization, flyash and cement stabilization and GGBFS stabilization showed lower traces of Ettringite and can be stated that they performed better than the lime and fibers stabilization and lime stabilization over the present shorter duration of the time frame used in this assessment.

However, long term monitoring and mineralogical studies are still needed for a better understand of the effects of sulfate heaving phenomenon in the present treated sections.

## 5.8 Summary

This chapter provided the discussion of present data collection methodology, the filters that were applied to the data acquired to eliminate noise, the difficulties faced during installation of sensors and during data collection. This was followed by the analysis of the acquired data for both strain gauges and pressure cells. Elevation survey details and their results are used to explain the heave movements Also, X-ray diffraction analysis results are presented and discussed to address sulfate heave generation.

From the above analyses, it is seen that the Type V cement stabilization method performed best under all conditions for the present short time of monitoring. This was followed by flyash and cement stabilization and GGBFS stabilization. Lime and fibers stabilization was ranked fourth overall. Though the control section did not under-perform, as in there were no visible cracks on the concrete pavement surface, its performance was over-shadowed by the performance of other stabilization methods used in this research.

# CHAPTER 6

# SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

## 6.1 Summary and Conclusions

The objective of this research study was to examine the performance of four alternate stabilization methods which were used to enhance the properties of sulfate rich subgrade at Harwood Road in southeast Arlington. These treatment methods were compared against a control lime treated section. These treatments were evaluated by designing an experimental program and performing tests at the geotechnical laboratory facilities of the University of Texas at Arlington. Field instrumentation studies were also conducted at the site to address the consistency of both laboratory and field treatment results. The monitoring response results were analyzed to address the performance of stabilization section to provide stable and uniform support to pavements.

Four types of novel stabilization techniques and a control lime treatment method were implemented in the course of this present experimental phase. Ranking was attempted individually based on both laboratory tests and actual field data collected from sensors installed in the field. The data acquisition from the sensors installed in the field was carried out over a period of seven months.

The following provides the summary and major conclusions obtained from the laboratory testing phase and the field monitoring data analyses:

1. From the laboratory tests performed to evaluate the physical and engineering soil properties, it was noticed that all four stabilization techniques improved the physical properties, liquid limit, plasticity index values, unconfined compressive strength and resilient modulus of Hardwood Road soil while decreasing the swell and shrinkage potentials of the same soil.
2. Type V Sulfate Resistant Cement proved to be the most effective treatment for the soil by enhancing the shear strength properties and reducing swell, shrinkage and plasticity characteristics. This was followed by Class F Flyash and Cement, GGBFS and lime and fibers treated samples respectively. Though the lime treated control samples displayed an increase in the strength characteristics, it was low when compared to the other stabilization methodologies.
3. Field instrumentation was initially carried out over a period of two weeks and the zero day data was recorded. It was noticed that 80% of the sensors performed satisfactorily after installation in the treated subgrade. MATLAB® filters had to be applied to the raw data obtained by the data collection module to eliminate the noise signals.
4. The analysis of the normalized and filtered sensor data showed that Type V Cement treated section was the most efficient in real field conditions. This was followed by Class F Flyash and cement treated section while GGBFS treated section was ranked third. Lime and fibers stabilization was ranked fourth and was followed by the control section.

5. From the elevation surveys conducted, it was noticed that the lime treated control section experienced heaving more than other treated sections. This was followed by the cement, flyash-cement and lime-fiber treated sections. GGBFS treated section performed the best.
6. X-ray diffraction analyses were also performed on the treated soil samples obtained from the site of research. Though all the stabilization methodologies showed traces of Ettringite formation, lime and lime-fibers sections showed a higher percentage of Ettringite traces with slightly higher intensities.

By comparing the laboratory and field results, we can comment that there was a similarity in the results obtained in both phases. Overall, based on short time frame of monitoring, Type V cement was ranked first and was followed by fly ash-cement stabilization technique. GGBFS stabilization was ranked third and lime-fibers treatment was ranked fourth best treatment. Though the control lime treatment method increased the subgrade properties, it was ranked last among all the stabilization techniques conducted in this research.

## 6.2 Recommendations

From the results obtained in the course of this research, the following recommendations should be considered for future works:

1. Longer duration of monitoring is needed to address these stabilization effects.

2. The data acquisition from the sensors should be carried out over a whole year to study the effects of various seasons on the performance of the pavement and the treated subgrades.
3. Ettringite formation in the subgrade should be monitored periodically.

# APPENDIX A

# MATLAB® CODE

```
% Reading Excel Sheet into Matlab
x= xlsread('Unfiltered Data in MS Excel format');
% Five Point Running Average Loop; where j represents number of iterations
for j=1:30
  for i=1:length(x)
     if(i<5)
        x(i)=mean(x(1:i));

     else
        x(i)=mean(x(i-4:i));
     end
  end
x
end
```

# APPENDIX B

## Comparison of Strain Data

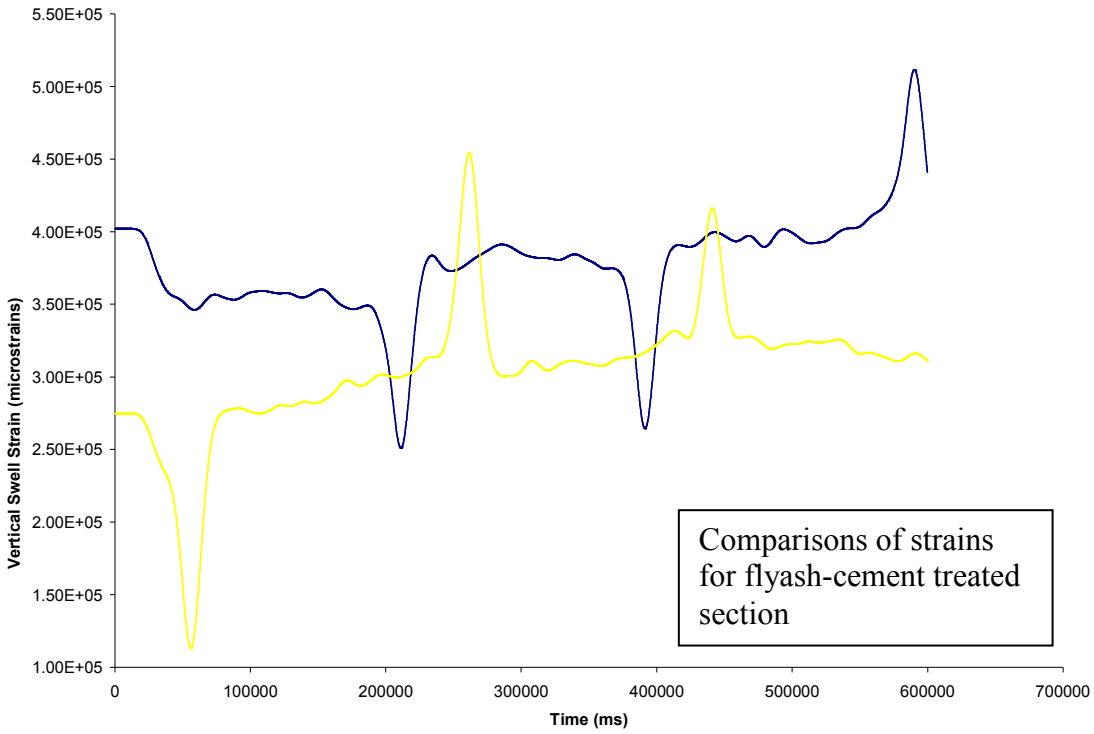


Figure B.1 Comparison of filtered data with adjacent vehicular movement for flyash-cement treated section

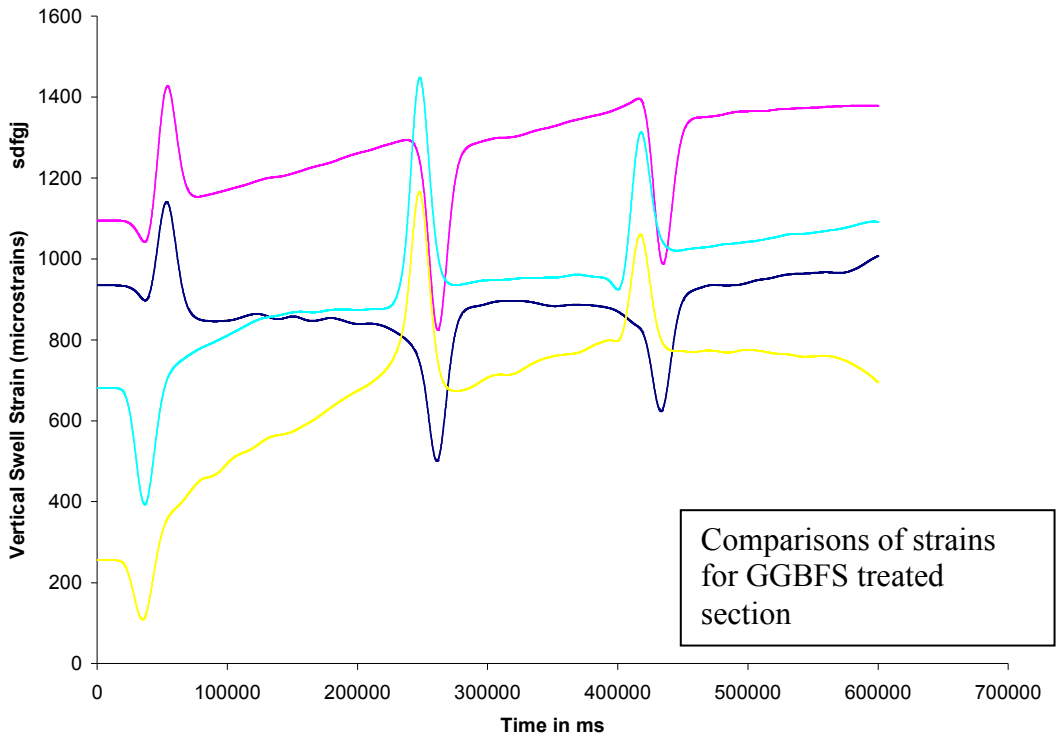


Figure B.2 Comparison of filtered data with adjacent vehicular movement for GGBFS treated section

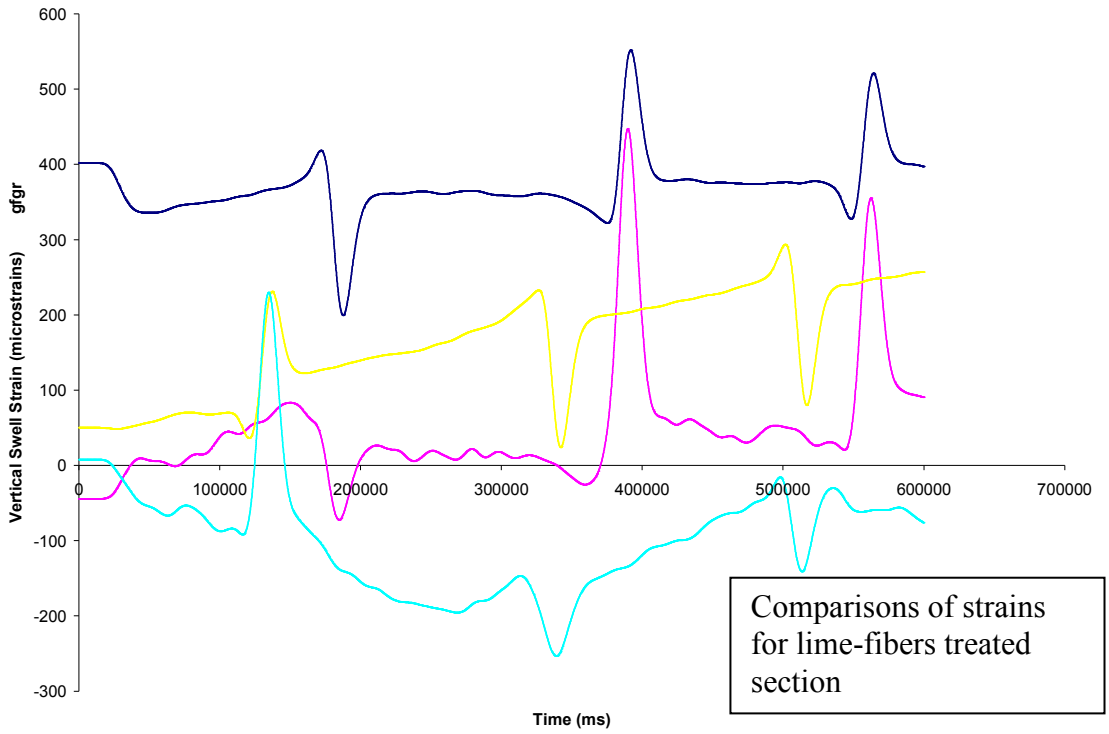


Figure B.3 Comparison of filtered data with adjacent vehicular movement for lime-fibers treated section

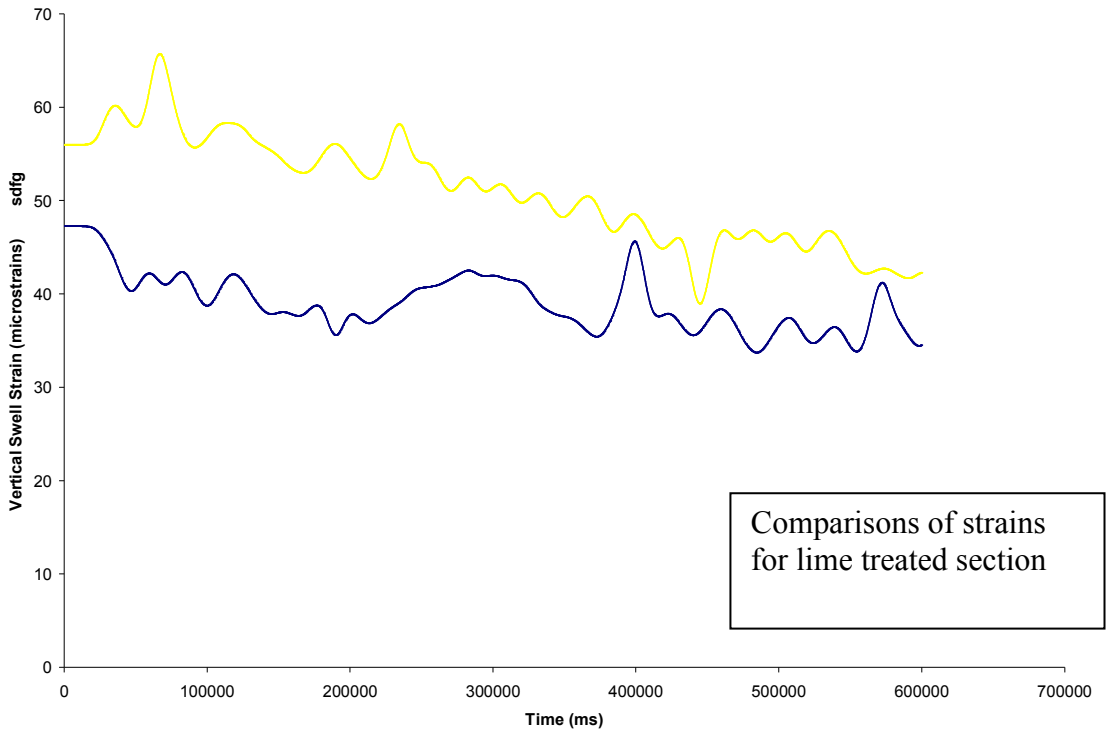


Figure B.4 Comparison of filtered data with adjacent vehicular movement for lime treated control section

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