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An Innovative Hybrid Sensor for Rapid Assessment of Sulfate-Induced Heaving in Stabilized Soils

Final Report for
Highway IDEA Project 154

Prepared by:
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An Innovative Hybrid Sensor for Rapid Assessment of Sulfate-Induced Heaving in Stabilized Soils

Final Research Report

Submitted By

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Executive Summary

Heaving has been observed in sulfate soils when they are treated with lime or cement additives. This heaving is attributed to the formation of an expansive mineral known as Ettringite. Ettringite is known to form from reactions of calcium ions from the chemical additives, sulfates in soils and free reactive alumina released from treated clayey soils and stabilizers. Since chemically-treated bases have been used to support the pavement infrastructure, this type of heave has distressed the pavements and as a result, it became necessary to develop alternate stabilization techniques to treat sulfate soils. Evaluation of the sulfate heaving requires long laboratory-based mix designs, since it is important to perform the long term swell tests on treated soils. Hence, it is important to develop a faster and reliable device and test method to assess and evaluate sulfate heaving in chemically-treated sulfate soils in a short time frame. The intent of the present research was to devlop an innovative hybrid sensor, BM sensor comprised of Bender Element (BE) and moisture based Time Domain Reflectometry (TDR) technologies to assess the sulfate heave in treated soils in a quick time frame. This hybrid sensor was successfully used in the laboratory for quick assessments of soil stiffness and moisture content variations in cement and lime-treated sulfate soils. After succesful and quick assessments of the heaving, the sensor was used in the field test section to monitore stiffness and moisture content changes. Both laboratory and field studies showed that this sensor can be used in the field to assess sulfate heaving. More field studies will further enhance and promote the use of this sensor for quick evaluation of sulfate heaving.

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1 INTRODUCTION

Chemical stabilization of expansive soils using lime, cement and other additives has been preferred by practitioners over the years as this stabilization improves the plasticity characteristics, moisture stability and strength characteristics (Hausmann, 1990). Lime and cement are grouped as calcium-based stabilizers since calcium constitutes a major portion of these chemicals. Though-calcium based stabilizers improve the volume changes and strength charectersitics of the expansive soils, limitations still exist in calcium-based chemical stabilization of soils. These limitations include treatment of soils containing organics and soluble sulfates. It was reported that the presence of organic carbon in excess of 1% can interfere with the pozzolanic reactions, leading to low strength gains. The presence of sulfates is also a major concern bacause lime or cement treatement in these soils will lead to excessive heaving due to formation of heaving mineral and this heaving damages pavement (Mitchell 1986, Hunter 1988, Mitchell et al., 1992, Puppala et al. 1999, 2003, 2012).

The pavement distress are attributed to the formation of the expansive mineral, ettringite, which is caused from a reaction that occurs among soil sulfates, clay alumina, and calcium from stabilzer in the presence of moisture. This phenomenon is termed as "Sulfate-Induced Heave" in the literature. Sulfate-induced heave was first reported by Sherwood in 1962. However, the sulfate-induced heave phenomenon received little attention until the mid1980's when Terzaghi's lecture by Mitchell (1986) mentioned the potential severity of the heave. Repair and reconstruction of the distressed pavement infrastructure are costing taxpayers millions of dollars (Kota, 1996). Under favorable moisture content, humidity and temperature conditons, the expansive minerals can further grow, causing more swelling. Researchers called lime treatment of expansive soils containing sulfate as a "man made expansive soil problem" (Puppala et. al., 2012). Figure 1 below shows the heave-induced failures caused by the formation of the ettringite mineral in various case studies.

Soils containing natural sulfate are found all across the United States. Gypsum is the most frequently occurring sulfate mineral in the western part of the United States (Kota et al., 1996). Figure 2 shows the locations of soils containing gypsum, as well as gypsum mines in the US. Sulfate-induced heave failures have been reported in several parts of the United states, as more than 15 state transportation agencies have identified heave-induced failures. The failures are predominantly in the western, midwestern and southwestern United States; however a few states in the southeastern and eastern US have also started recognizing this problem (Puppala and Cerato, 2009).

Figure 1 Photographs Showing Sulfate-Heave Distress Problems from Nevada (left), Texas (top, right) and Dallas Fort Worth (bottom, right) (Hunter, 1988)

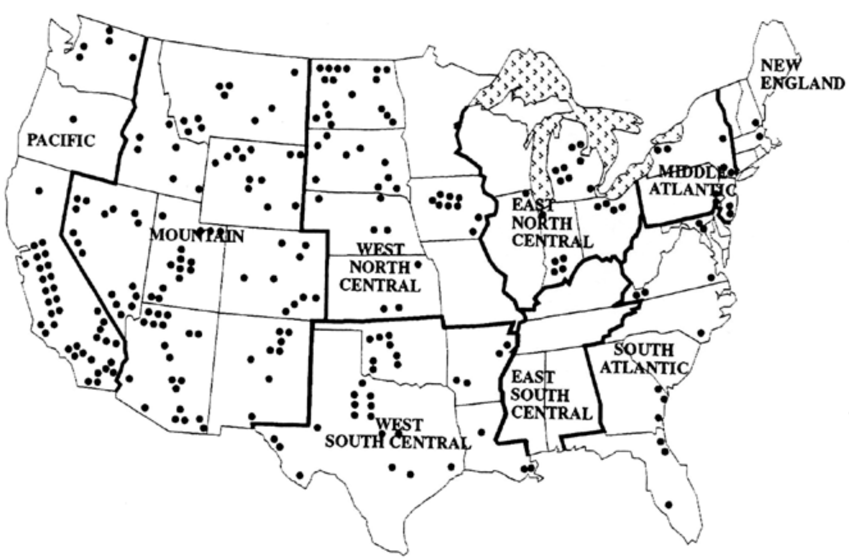


Figure 2 Locations of Sulfate-bearing Soils in the US (Kota et al. 1996)

It was reported in the literature that sulfate-induced heave can occur within a few weeks to several months after chemical stabilization, depending upon the environmental conditions and soil clay mineralogy at a given site. Existing studies on the evaluation of sulfate-induced heave in the laboratory are based on volumetric swell measurements on treated soils and these studies will take several weeks to months to complete and this often result in not fully evaluating the stabilization methods for effectively treating sulfate rich soils. Hence, it is necessary to develop a new sensor or a device and test procedure that can identify the heaving mechanisms faster than conventional laboratory tests and such approach should be applicable in both laboratory and field conditions.

2 RESEARCH OBJECTIVE

Hence, the main objective of this proposed research was to develop a sensor capable of detecting sulfate heaving in chemically treated soils by assessing various constitutive soil parameters. The integrated sensor developed in the current research is totally new, and a radical departure from the current test procedures for heave assessments. Swell assessments were made indicrectly by measuring water content, and stiffness property vairations in a treated soil with a hybrid sensor when it was subjected to moisture content fluctuations. The integrated hybrid sensor using time domain refractometry (TDR) and bender element based wave propoagation to measure both mositure content and soil stiffness at the same time. Also, the sensor was developed in such a way that it can be used in both the laboratory and in the field.

As a part of the research, a thorough literature search was first performed to understand the heaving mechanisms in chemically-treated sulfate soils. This understanding was helpful in the development of the hybrid sensor. After initial trials and then fabrication of the sensor, tests were conducted on treated sulfate soils with this sensor in both laboratory and filed conditions. After validation studies, a test procedure was developed for future implementations in the field. The following sections summarizes results from various task performed to accomplish the research objective.

3 LITERATURE REVIEW

Expansive soils are known to be one of the problematic soils found in the world. They undergo swell and shrinkage upon moisture wetting and drying from seasonal changes. Both swell and shrinkage behaviors of expansive soils can cause severe damage to civil engineering structures, in particular pavement structures. In the United States alone, it has been reported that the annual losses due to expansive soils range between $6 to $11 billion in total damages caused to residential houses, roads, runways and others. Expansive soils exhibit large volumetric changes and these depend on several factors, including type and amount of clay minerals, moisture content, dry density, soil structure, confining pressure and climatic conditions (Nelson et al., 1992).

Chemical stabilization is the most widely adopted technique for stabilizing the expansive soils in order to improve the soil properties and meet the specific engineering requirements. Calcium-based stabilizers, especially lime and cement, are commonly used stabilizers due to their cost effectiveness and ability to improve expansive soil properties. However, this stabilization technique has shown to cause problems in the presence of soil sulfates. When these soils are stabilized with calcium-based stabilizers such as lime or cement for foundation improvements, the sulfate minerals in these soils react with the calcium component of the stabilizer and free reactive alumina of soils to form highly expansive crystalline minerals: ettringite and thaumasite (Mitchell, 1986; Hunter, 1988).

In order to form sulfate minerals, the free alumina content from the original clay mineral interstices should be released during the early period of the hydration process, which usually occurs at the pH conditions greater than 10.5, as in the case of lime and cement treatments. In the case of cement treatment, the alumina is liberated from pozzolanic compounds formed in the cement treatment. At this stage, the presence of soluble sulfates and calcium ions from chemical stabilizers should be present to form the ettringite mineral. The last, but the most important factor, is the presence of water, which facilitates the chemical reactions needed for final formation of this mineral. Overall, the resulting amount of heaving is primarily a function of the quantity of ettringite formed, the crystal morphology and size, restraint of the system, and ion accessibility. All of these depend on different environmental conditions, including pH conditions, presence of soluble sulfates and carbonates and water.

Ettringite the most often formed chemical mineral in the treated sulfate soils. Thaumasite forms only after ettringite undergoes certain crystalline changes at low temperature conditions. These sulfate minerals expand considerably when subjected to hydration process which results in heaving in soils. Also, The expansive minerals formed can continue to grow and such growth known as crystal growth also damages structures. The heave distress, due to the presence of sulfate, is termed in the literature as sulfate-induced heaving (Puppala et al. 2012).

Mehta and Klein (1966) reported that the formation of the monosulfate hydrate is favored in a relatively high alumina environment or dry conditions. The formation of trisulfate hydrate [ettringite, $Ca_6[Al(OH)_6]_2*(SO_4)_3*26H_2O$] leads to substantial increase in volume changes upon wetting. Once the ettringite crystal is formed, it continues to grow in an almost pure form. When the temperature of the system reaches less than 15°C and with presence of carbonates in the system, ettringite is transformed by a series of intermediate reactions to the thaumasite mineral [$Ca_3Si(OH)_6]_2(SO_4)(CO_3)_2*26H_2O$]. This transformation in mineral structure occurs by isostructural substitution of silica for alumina and carbonate for sulfate (Mehta and Klein, 1966).

The chemical structure of ettringite crystals are hexagonal prisms, often in an elongated form. They can have different shapes depending on the time and pH conditions during the formation period, and these shapes are needle-like, lath-like or rod-like. Rod-like crystals form at the early stage when the solution phase in the soil has a high amount of hydroxyl ion concentrations. This implies that the solution is in high pH condition (Intharasombat, 2003). A pictorial representation of the mineral ettringite is given in Figure 3. Lath-like crystals form as concretions of smaller crystals align in the same direction, and needle-like crystals form at later stages when the pH decreases.

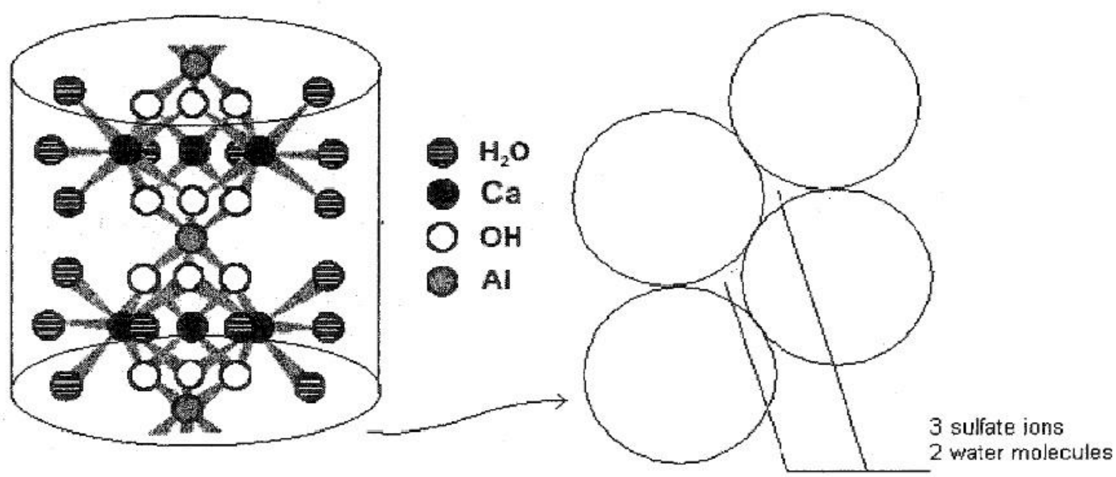


Figure 3 Pictorial Representation of the Mineral Ettringite (Intharasombat, 2003)

Infrastructure facilities, including building structures, embankments, runways and highway pavements built on lime and cement-treated sulfate-bearing soils, have been affected by this heave distress. This distress is termed as sulfate-induced heave distress in the literature (Mitchell and Dermatas, 1992; Dermatas 1995; Hawkins, 1997) and typically results in the poor performance of infrastructure and considerable reduction in the design life of structures. Also, the increase in the use of industrial wastes and Phosphogypsum (used frequently in the southeastern US) for soil stabilization and solidification further signifies the importance of understanding the heave mechanisms of chemically-treated sulfate soils (Dermatas, 1995). Waste materials, such as Phosphogypsum and other sulfate wastes, are used as base and subbase materials to support pavements. These wastes leach sulfate ions, which can increase the sulfate levels in soils. Also, sulfates can occur in soils from the construction water used in the projects. Such sulfate levels could potentially lead to heaving when calcium stabilizers are used to stabilize them.

Many states, including Kansas, Oklahoma, Nevada, New Mexico, Louisiana, Arizona, New Jersey, Virginia, Texas, Colorado, California and others have reported sulfate-induced heave as one of the major distresses that damages embankment and pavement structures (Perrin, 1992; Dermatas, 1995; Puppala et al., 2006, 2012). Repair and maintenance costs of heave-distressed pavements and runways are reported to be millions of dollars annually (Petry, 1994; Kota et at. 1996). The city of Las Vegas, Nevada spent close to 2.7 million dollars toward repair and maintenance of the pavements damaged by the sulfate-induced heave distress (Hunter, 1988). The United States Army Corps of Engineers rebuilt an auxiliary runway of Laughlin Air Force Base near Spofford, Texas at a cost of more than 1.5 million dollars. These costs depict the severity of the problem.

As noted above, the literature suggests that more states are recognizing sulfate-induced heave as a widespread problem across the USA. Several states, including Oklahoma and Texas, have already begun implementing sulfate characterization methods as routine subgrade screening methods in preliminary geotechnical studies. Also, sulfate heaving in the field is dependent on subgrade type, field compaction and environmental conditions, including field temperature conditions (Puppala et al. 2006; Puppala and Cerato, 2009).

One important question that is often asked by practitioners is the threshold of problematic sulfate levels in soils at which sulfate heave problems will be a concern. Based on the previous studies, researchers across the USA have reported different threshold levels of sulfates at which heave distress was recorded (Hunter, 1988; Mitchell and Dermatas, 1992; Petry and Little, 1992;

Kota et al. 1996; Puppala et al. 1999; Viyanant, 2000; Little et. al., 2005). No conclusive threshold levels of sulfate can be established, and this is primarilyattributed to variability in soil types studied under varying site conditions.

Petry et al., 1992 stated that if the level of soluble sulfate is below about 2,000 ppm or 0.2 percent, the development of expansive minerals will not be an issue in stabilized soils. Berger et al. (2001) indicated that soluble sulfates below 0.3 percent (3000 ppm of sulfates) are of no problem. Soluble sulfates between 0.3 and 0.5 percent represent moderate risk of harmful reaction. Sulfates between 0.5 to 0.8 percent indicate moderate to high risk. Soils with soluble sulfates levels greater than 0.8 percent pose serious threat to civil infrastructure facilities. Studies conducted at the University of Texas at Arlington (Puppala et. al., 2003) confirmed that at low sulfate levels (around 1000 ppm), lime stabilization plays an important role of reducing swelling of natural soils. At sulfate levels ranging from 1000 to 2500 ppm, both the lime stabilization reactions and sulfate heave reactions occur simultaneously, but the magnitude and extent of heave depends on the lime concentration. At higher lime dosages, swell magnitudes are suppressed, indicating the dominance of stabilizing reactions. Also, when the sulfate concentrations exceed 2500 ppm, the increase in lime dosage results in increased heaving due to increased amounts of ettringite formed. Puppala et. al., 2003 reported that void ratio and compaction conditions play important roles in the sulfate-induced heaving phenomenon. If the void ratios are small, the soil matrix is dense and cannot accommodate any heave associated with ettringite formation and growth leading to the pavement heave.

Research studies conducted by Harris et al. (2004) indicated that at or below 3000 ppm sulfate concentrations, sulfate heaving is of no concern and lime stabilization can be effectively implemented. Also, between 3000 and 7000 ppm sulfate concentrations, lime stabilization can be performed in soils with some caution. In most of the cases, the sulfate levels to induce heaving range from 320 ppm to as high as 43,500 ppm (Puppala et al. 1999; 2003). The time for sulfate heave appearance after chemical stabilization ranges from a few days to 18 months. Also, soils that experienced this sulfate heave included sands to silts and clays, with all these soils containing significant clay fractions.

Overall, it can be seen that there are no conclusive threshold sulfate levels above which sulfate heaving occurs. This is due to the fact that soil properties such as void ratios, environmental and site drainage conditions are different from site to site. One recently completed

National Science Foundation sponsored study conducted by the principal investigator from the University of Texas at Arlington showed that the problematic sulfate levels vary for cement and lime treatments and their dosage levels (Puppala et al., 2005).

Figure 4 shows various treated soil specimens that were cured prior to strength and stiffness testing. Each specimen, with different sulfate levels, was stabilized with ordinary Portland cement. It can be observed that the specimen on the right side experienced severe heaving during curing, and bulging in the specimen can be observed. Based on swell strain magnitudes measured in the laboratory environment, the problematic sulfate levels varied between 1000 ppm for soil-cement treatments to 2000 ppm for lime treatments. These levels, however, may vary, as the field environmental condition can often result in higher heaving as some of the sulfate source minerals may form in larger quantities in a high temperature environment. This raises an important and practical concern: whether this heave could be best characterized as a problem by studying it in the laboratory simulating field conditions.

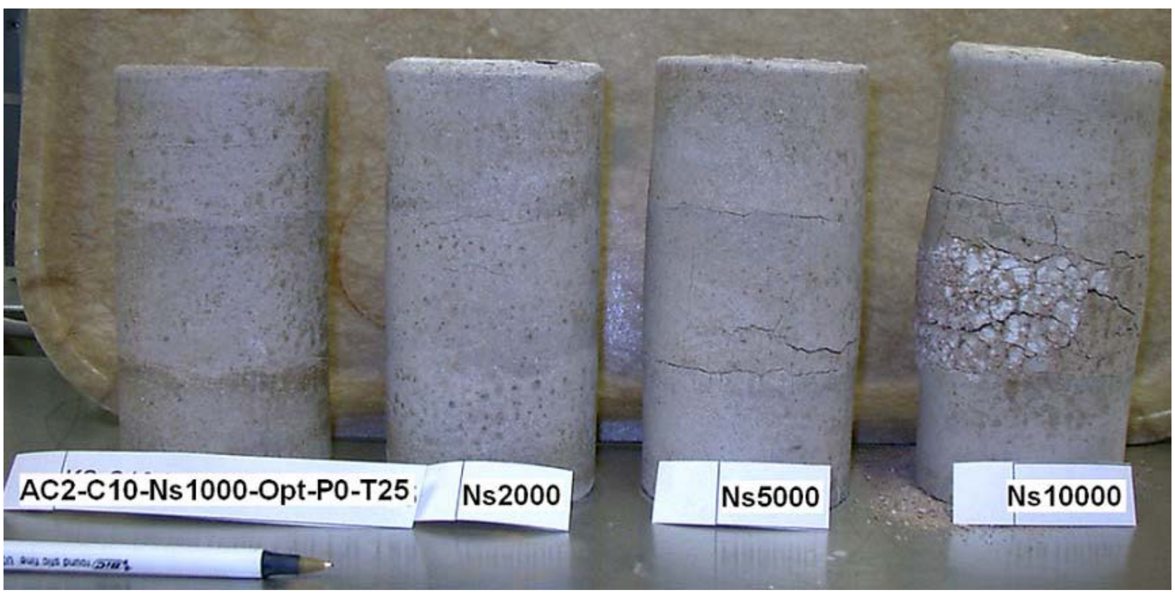


Figure 4 Treated Soil Samples with Varying Sulfate Contents

Based on the threshold sulfate levels research, Texas Department of Transportation has developed guidelines for stabilizing soils containing sulfates. According to these guidelines, sulfate concentrations upto 3000 ppm can be stabilized by traditional lime with one day of mellowing. Soils with sulfate concentrations upto 8000 ppm can be stabilized by providing additional moisture, along with other chemical treatments including combined lime and fly ash treatments. When sulfate concentrations exceed 8000 ppm, alternative treatment such as remove—and-replace or blending in non-plastic soils is recommended. Though guidelines have been devloped, application of guidelines needs thorough laboratory evaluation before field implementation.

For field implementation of stabilizer design, laboratory mix design needs to be performed prior to final selection of stabilization additive for field application. It may take weeks, if not months, to completely understand the macro swelling in the treated soils. Thus there is an important research need to develop a sensor-based test procedure to identify the heave problems in both laboratory and field conditions in a relatively short turnaround time. This research attempted a novel attempt to develop a rapid approach utilizing a hybrid sensor-based technology for predicting heaving potential caused by sulfate, soil and chemical stabilizer reactions.

Past research studies conducted by Puppala et al. (2006) showed that shear modulus is an excellent parameter to represent sulfate-induced material degradation in lime/cement treated soils. Tests conducted on lime/cement-treated soils have shown that small strain shear modulus increased, with time, in treated soils with sulfate contents of 1,000 ppm; whereas, soils with 10,000 ppm showed a decrease in shear modulus with time **(**Kadam, 2003).

With this in mind, the present sensor was developed using a bender element technology to measure and monitor small strain shear modulus (G_{max}). In addition, moisture content data was measured by using Time Domain Reflectometry or TDR principles. Both technologies are fitted on a small sensor which can be embedded in a treated soil in either in the laboratory or in the field. This sensor is non-destructive and the measurements are termes as non-destructive type measurements. This sensor hence ensures that the relevant soil parameters can be measured by embedding it in the same soil. The non-destructive nature of the test procedure ensures assessment of time rate moisture content and soil stiffness changes in the treated soil matrix, which is typical of the microstructure development in a chemically stabilized soil.

Time Domain Reflectometry (TDR) is the current field standard method for the measurement of soil water content. The PI has used TDR probes in a large number of instrumentation projects in geotechnical and pavement areas and was involved in field instrumentation technology to address various professional practice needs. An innovative flat TDR probe was designed and integrated with bender elements to fabricate a BM sensor, and this sensor can be placed at strategic locations in a treated soil specimen to monitor moisture content, dry density and stiffness properties during the curing process. The flat cable ensures good coupling with adjacent soils, and the rugged nature of the sensor will make it function even under heavy construction loads.

An earlier investigation was conducted by Zhang et al. (2007) to study the use of the TDR probes for distributive moisture sensing. The sensor cable is fabricated with an inexpensive stainless steel strips, with the total materials costing less than $100 for a sensing length of 15 ft. The evaluation shows that the TDR sensor signals are very sensitive to moisture content variations. The thickness of the sensing cable, which is approximately 1 mm, makes it more flexible than a rigid design, thus it is easier to operate under field conditions. The flexibility also simplifies the procedure of installation and survival of field construction loads. In this study, the bender elements were included in the TDR unit by integrating it along the sensing cables. This device design was conducted with careful collaboration and discussion between PI and consultant.

4 LABORATORY STUDIES

The testing program of the current project has been divided into four different tasks, as mentioned in the original proposed plan. In the first task, an integrated sensor was developed which is capable of measuring shear modulus and moisture content with respect to time. In the second task, the developed sensor was used in the laboratory testing program to test chemically-treated soils to analyze sulfate heaving mechanism, and an algorithm was developed based upon the laboratory results and their assessment. In the third task, the sensors were installed in the field for validation with the laboratory data. In the final task, an implementation procedure is developed for the present sensor, which is capable of predicting the heaving potential in sulfate rich soils.

4.1 Hybrid BM Sensor

In this research, progress was first made to develop a BM hybrid sensor (using Bender Element & Moisture Content based TDR), which is capable of measuring both shear moduli and moisture contents with respect to elapsed time periods. The measurements of shear moduli and moisture content can provide a quick assessment of sulfate heaving in chemically treated soils. The new BM sensor was developed by the integration of the electromagnetic wave technology and seismic wave technology into a single unit produced by Time Domain Reflectometry (TDR) and bender element. Proposed Bender Element (BE) and Time Domain Reflectometry (TDR) probes are depicted in Figure 5.

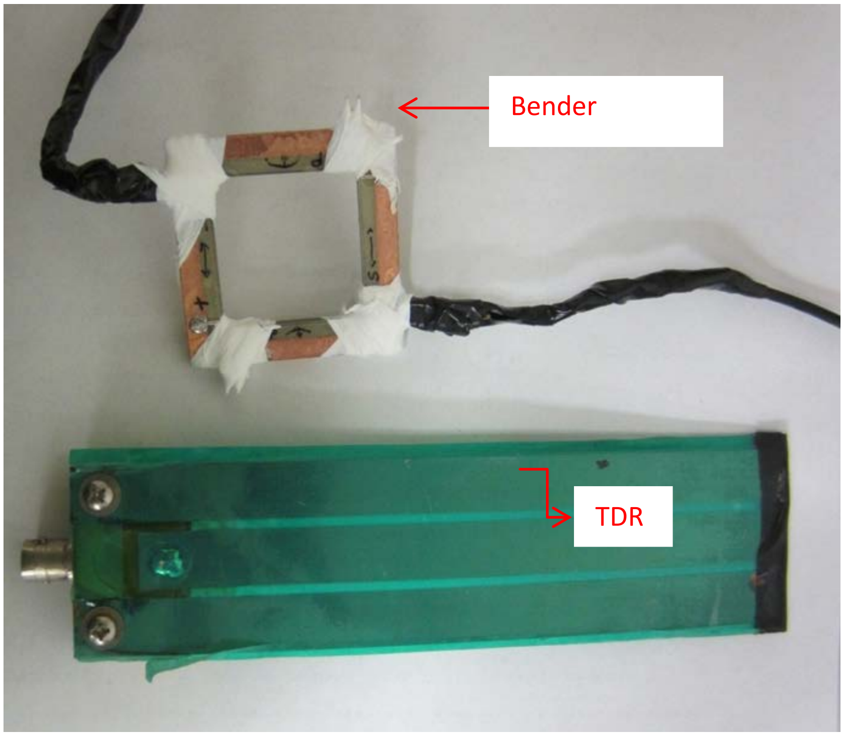


Figure 5 Photograph Showing Bender Element & TDR Probe

A Bender element of 2 in. x 2 in. was fabricated using piezoelectric ceramics. An E-glass frame was used, which is sufficiently strong but has no corrosion problem for long term burial under soil. The thickness of the e-glass frame was optimized for sensing element protection. A protective cover was provided to prevent soil from becoming trapped inside the element. The geometry of beam is such that it is slender and sufficiently stiff. The connector was refined to prevent damages and to allow for installation. The BM sensor was coated with wax to prevent moisture from entering the joints between the sensing elements and connecting wires.

Time Domain Reflectometry (TDR) has been widely used in various areas of civil engineering such as bridge scour monitoring, compaction control, slope movement and monitoring of concrete crack development. In this research TDR has been used to monitor the variation of the moisture content with respect to time. TDR is a flat strip constructed with three

12.7-mm stainless steel metal strips. The strips are accurately aligned in parallel with a distance of 3mm. The air gaps between the strips are filled with silicon tape. Protective coating is applied to both sides of the aligned strips. A connection head was made to transmit signals from the pulse generator to the TDR cable. The configuration of the TDR device includes a pulse generator, a sampler, a connection cable and a measurement probe. A pictorial representation of the TDR probe is given in Figure 6.

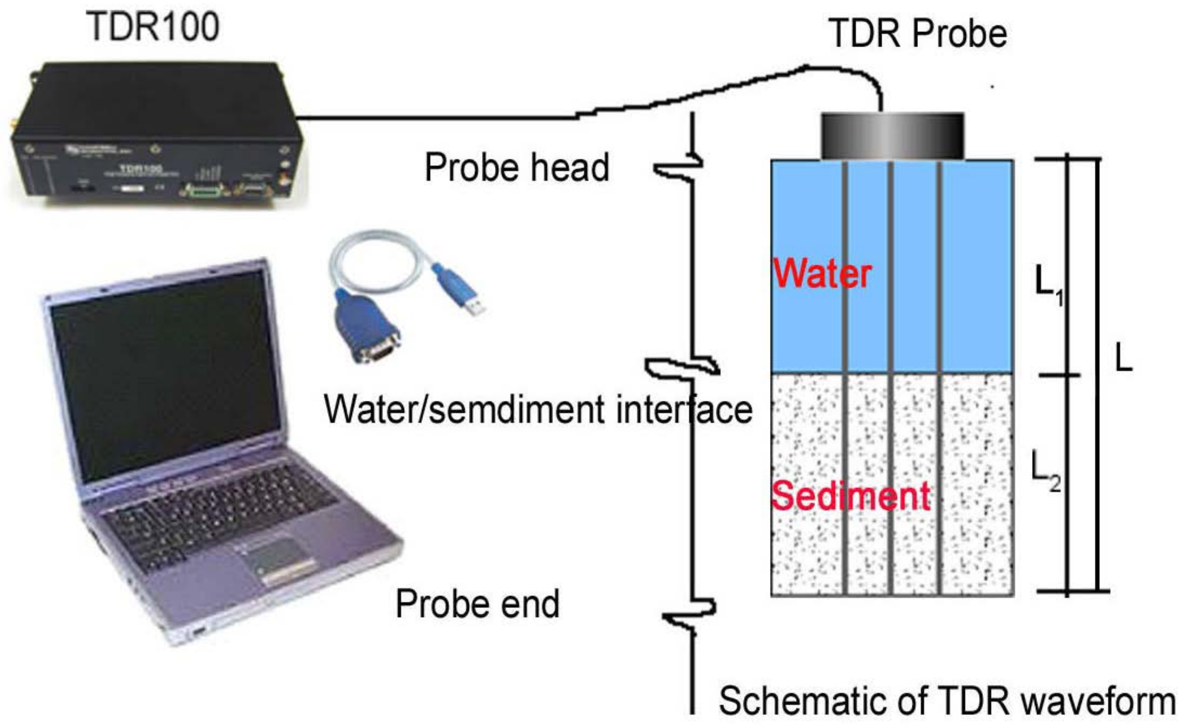


Figure 6 Configuration Setup of the TDR

The operating principle of TDR involves sending a fast rising step impulse or impulse to the TDR cable and measuring the reflections due to the change of system geometry or material dielectric permittivity. Typical waveforms obtained are between relative voltage (V) and scaled distance (m) as shown in Figure 7.

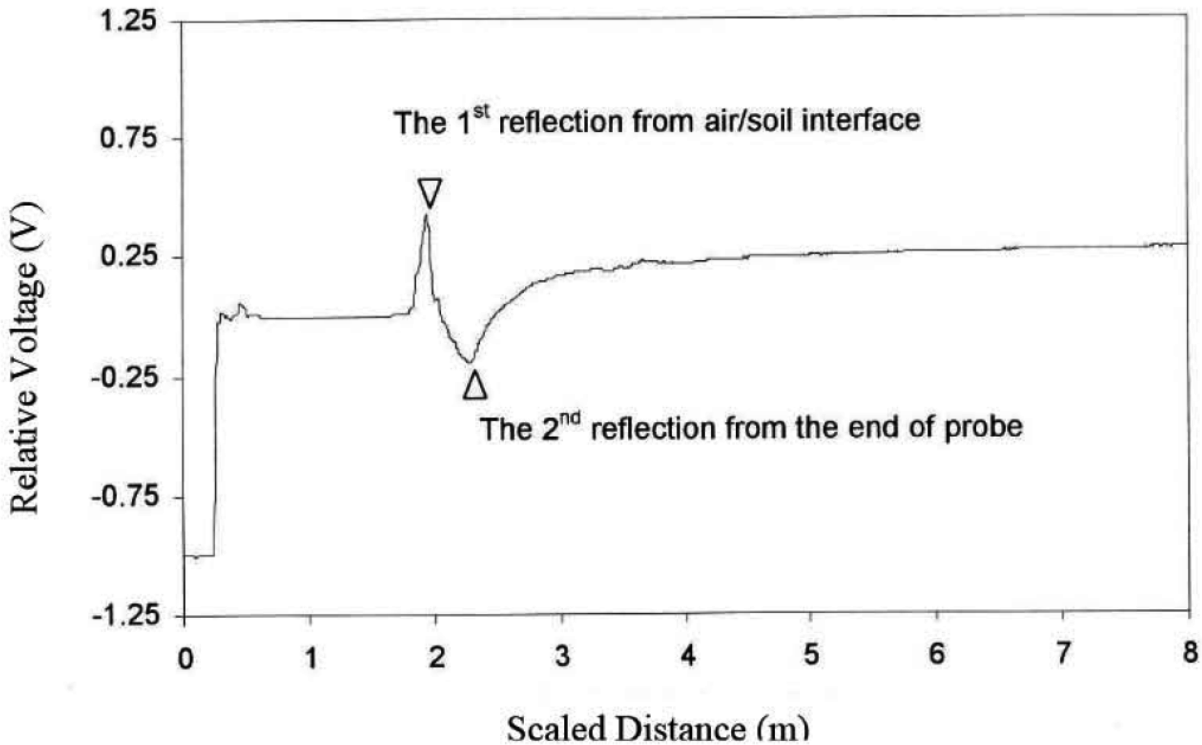


Figure 7 Typical TDR Wave Form (Yu and Drnevich. 2004)

The reflections observed in the waveform are analyzed to determine the dielectric constant of the soil. The first reflection observed is due to change in material property at the air and soil interface, and the second reflection occurs at the end of the TDR probe (Yu and Drnevich, 2004). Thus, by analyzing the two reflection points obtained from the TDR wave form, a dielectric constant can be determined using the relationship given by Baker and Allmaras (1990).

$$K_a = \left(\frac{L_p}{L_a}\right)^2 \tag{3-1}$$

The above relationship can be used to determine the dielectric constant from the measured TDR reflection, where L_a is the apparent length, which is scaled horizontal distance between the two reflections, and L_p is the length of the soil probe (Yu and Drnevich, 2004). After obtaining the dielectric constant of the soil, using the expression 3-2 developed by Siddiqui and Drnevich (1995), the water content of the soil can be determined.

$$\sqrt{K_a}\frac{\rho_w}{\rho_d} = a + bw \tag{3-2}$$

Where “a” and “b” are soil specific calibration constants, ρ_w is the density of water, ρ_d is the dry density of soil, and K_a is the dielectric constant of the soil and w is the gravimetric water content.

4.2 Integration of Bender Element and TDR

Integration of both the Bender element and TDR was done in order to measure the shear modulus (G) and moisture content (w) with respect to elapsed time period in chemically treated soil samples. In the prepared soil specimen, the bender element was placed at half the depth of the soil specimen, and the TDR was placed beside the bender element, aligning vertically along the soil sample. The figure 8 below shows the Bender element in the soil sample and the TDR cable.

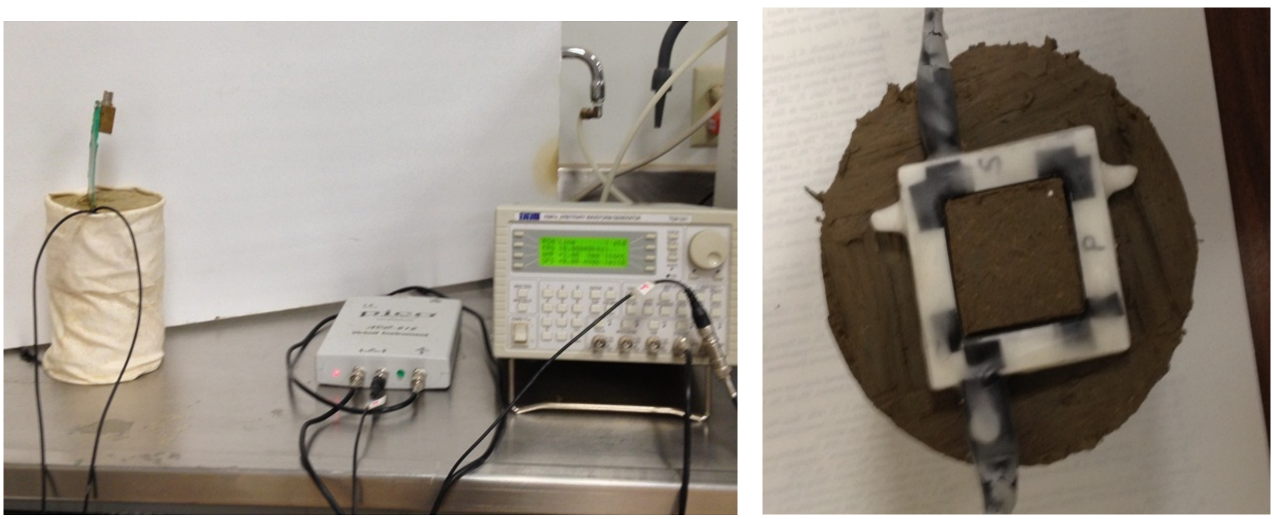


Figure 8 Integration of Bender Element and TDR

4.3 Test Soils and Basic Soil Classification

As a part of the laboratory research studies, three different soils with distinct sulfate levels were selected from Texas and Oklahoma. These soils were used for evaluating the performance of the BM sensor in the laboratory studies. The three soils were termed as Oklahoma, Riverside and Burleson soils. Sulfate contents of the test soils were determined using the modified UTA method described by Puppala et al. (2002). The results of this method are consistent with minimum standard deviations. Soils with low sulfate contents (Burleson and Riverside soils) were spiked with highly soluble ‘Sodium Sulfate’ (Na_2SO_4) to bring the sulfate content to 12,000 ppm and 20,000 ppm. Based on the sulfate contents, soils in this study were classified as low, medium and high sulfate soils. Soluble sulfate contents and elevated sulfate contents of the test soils are presented in Table 1. Atterberg limit tests were conducted on all the three soils as per ASTM D-4318 method to determine the Liquid limit, Plastic limit and Plasticity Index of the soils. The results are summarized in Table 2. Soils with low sulfate contents (Riverside and Burleson soils) were spiked with 'Sodium Sulfate' (Na_2SO_4).

Table 1 Sulfate Contents of the Test Soils

Soil Location	Initial Sulfate Contents, ppm	Elevated Sulfate Contents, ppm
Burleson	1,900	12,000
Oklahoma	15,000	15,000
Riverside	500	20,000

Table 2 Atterberg Limits and Soil Classification

Soil	Atterberg Limits			USCS Classification
	Liquid Limit	Plastic Limit	Plasticity Index	
Burleson	55	18	37	CH
Oklahoma	35	11	24	CL
Riverside	42	21	21	CL

4.4 Testing Variables

Testing variables include soil types, sulfate contents, compaction moisture contents, and types of stabilizers and their dosages. Three different soils from Texas and Oklahoma were chosen as test soils and these are classified as high plasticity clay (CH) and low plasticity clay (CL), with different geological origins. Elevated sulfate contents in these soils varied from 12,000 ppm to 20,000 ppm. Two compaction moisture contents, namely optimum moisture content (corresponding to maximum dry density, MDD) and wet of optimum moisture content (corresponding to 95% maximum dry density, WOMC) were chosen as test moisture contents for treated soils. Lime and cement were two stabilizers studied in this research, and the dosages considered were 4% and 8% for lime and 3% and 6% for cement additive, respectively. Dosages are based on the dry weight of the test soil. Testing variables used in this laboratory study are summarized in Table 3.

Table 3 Testing Variables

Description	Variables
Soils	Three (Burleson, Oklahoma and Riverside)
Sulfate Contents	Three (12,000 ppm, 15,000 ppm and 20,000 ppm)
Moisture Contents	Two (Optimum and Wet of Optimum Moisture Contents)
Stabilizer	Cement and Lime
Stabilizer Dosages	3% and 6% (Cement) & 4% and 8% (Lime)

4.5 Laboratory Testing Program

4.5.1 Standard Proctor Compaction Tests

In order to determine the compaction moisture content and dry unit weight relationships of the soils in the present research program, it was necessary to conduct standard Proctor compaction tests on all three soils to establish compaction relationships. ASTM-D 698 procedure was followed to determine the compaction curves and then esablish maximum dry density (MDD) and corresponding optimum moisture content. The tests soils were treated with lime and cement at respective dosages, and Proctor curves were established for treated soils. Compaction test results on natural and treated soils are summarized in Tables 4, 5, 6 and 7, respectively.

Table 4 Summary of Standard Proctor Test Results on Untreated Soils

Soil	Sulfate Content, Ppm	Moisture Content (%)		Maximum Dry Density (lb/ft^3)	
		OMC	WOMC	WOMC	WOMC
Burleson	12,000	20	24.8	99.2	94.24
Oklahoma	15,000	18	22.4	104	98.8
Riverside	20,000	16	20.2	106.4	101.08

Table 5 Summary of Standard Proctor Test Results on 3% Cement-Treated Soils

Soil	Sulfate Content, Ppm	Moisture Content (%)		Maximum Dry Density (lb/ft^3)	
		OMC	WOMC	OMC	WOMC
Burleson	12,000	19.6	21.8	104.8	99.6
Oklahoma	15,000	17.8	19.8	106.8	101.6
Riverside	20,000	15.6	17.8	99.8	94.8

Table 6 Summary of Standard Proctor Test Results on 6% Cement-Treated Soils

Soil	Sulfate Content, Ppm	Moisture Content (%)		Maximum Dry Density (lb/ft^3)	
		OMC	WOMC	OMC	WOMC
Burleson	12,000	19.4	21.5	105.2	99.9
Oklahoma	15,000	17.6	19.6	108	102.6
Riverside	20,000	15.3	17.5	100.4	95.4

Table 7 Summary of Standard Proctor Test Results on 4% Lime-Treated Soils

Soil	Sulfate Content, Ppm	Moisture Content (%)		Maximum Dry Density (lb/ft^3)	
		OMC	WOMC	OMC	WOMC
Burleson	12,000	20.8	25.7	103.2	98.0
Oklahoma	15,000	20.4	24	99.6	94.6
Riverside	20,000	19	23.6	98.8	93.8

Table 8 Summary of Standard Proctor Test Results on 8% Lime-Treated Soils

Soil	Sulfate Content, Ppm	Moisture Content (%)		Maximum Dry Density (lb/ft^3)	
		OMC	WOMC	OMC	WOMC
Burleson	12,000	22	26.2	102.5	97.4
Oklahoma	15,000	21	24.6	98	93.1
Riverside	20,000	20	25.1	98.4	93.5

4.5.2 Three Dimensional Volumetric Swell Tests (3-D Swell)

In the current research program, treated soil samples were embedded with the developed BM sensor to measure small strain shear modulus (G_{max}) of treated soils. Before conducting the shear modulus measurements, maximum possible volumetric swell strain need to be measured. Though the laboratory swell tests do not give vertical and horizontal swell strains in field situation, they provide the maximum amount of swelling that is possible in an ideal conditions. These swell strains are used to correlate the strength or stiffness property changes due to the sulfate reactions. Hence, any stiffness or small strain shear modulus property changes indirectly acounts for the heave phenomenon transpired in treated sulfate soils.

To determine the maximum volumetric swell potential, a three-dimensional free swell test was conducted in the research. Three dimensional volumetric swell strain tests were conducted using the “double inundation technique” to determine the maximum possible radial and vertical swell strain of a large soil specimen. It should be noted the double inundation technique for measuring the volumetric swell was used successfully in the past by various researchers across the United States and the UK. Double inundation represents the worst possible scenario in a field where 100% saturation of the soil can be achieved after a continuous rainfall event. Maximum expansive heave is possible in a short testing period in the laboratory environment. Oven-dried soils were pulverized and mixed with stabilizers at targeted moisture content levels. Both control and treated soil specimens were mixed and then compacted using a Gyratory Compactor Machine at two preestablished compaction moisture content levels. Figure 9 shows the gyratory compacting machine used and the soil sample taken after extraction.

Figure 9 (a) Gyratory Compactor Machine, (b) Soil Sample after Extraction

Figure 10 shows the setup used for 3-D swell tests. Samples were 4 in. (101.6 mm) in diameter and 4.6 in (116.8 mm) in height and were covered by a rubber membrane. Porous stones were placed on both top and bottom of the soil specimens, which facilitated the movement of water to the soil specimen. The specimen was fully soaked under water in a large container. As noted earlier, swell tests were performed on chemically-treated sulfate soils under moisture inundation from both ends of the soil specimen. Sulfate soils need the presence of moisture content that will facilitate reactions for ettringite crystal formation and its hydration. Hence, swell tests were performed under full soaking conditions. The amount of soil heave in both vertical and diametrical directions was continuously monitored until there was no significant swell for 24 hours.

At the end of the monitoring period, radial measurements were taken at the top, middle, and bottom circumferences of the soil samples and averaged at a frequency similar to the Consolidation Test. The percent vertical and radial strain values are calculated based on the original dimensions of the soil specimen and these strains are used in the estimation of total volumetric strains. Tables 9 and 10 shows the volumetric swell of cement and lime-treated soils. Tables 11 thru 15 show the vertical, radial and volumetric swell strains of natural and treated soils (3%C, 6%C, 4%L and 8%L). Vertical swell strain vs. elapsed time was plotted for all the natural and treated soils at optimum moisture content and is presented in Figures 11 thru 13.

As seen from Tables 9 and 10, the swell strain values of both lime and cement treated soils are significantly higher than the same of the Control soils, and this increase was attributed to deleterious sulfate reactions that occurred in the treated soils. This shows that the present soils used in the research are excellent test soils to understand sulfate induced heave problems. Same soils were further monitored for stiffness and moisture variations with the embedded sensors.

Figure 10 Three Dimensional Swell Test Setup

Table 9 Volumetric Swell Strains of Natural and Cement-Treated Soils

Soil	Sulfate Content, ppm	Natural		3 % Cement		6% Cement	
		OMC	WOMC	OMC	WOMC	OMC	WOMC
Burleson	12,000	10.9	5.2	12.8	10.3	16.1	11.3
Oklahoma	15,000	8.4	5	11.2	9.8	14.3	10.5

Riverside	20,000	10.2	10	13.8	10.4	15.2	11

Table 10 Volumetric Swell Strains of Natural and Lime-Treated Soils

Soil	Sulfate Content, ppm	Natural		4% Lime		8% Lime	
		OMC	WOMC	OMC	WOMC	OMC	WOMC
Burleson	12,000	10.9	5.2	17.2	13.1	15.6	10.8
Oklahoma	15,000	8.4	5	10.8	8.4	14.6	11
Riverside	20,000	10.2	10	14.8	11.6	16	12.7

Table 11 Vertical, Radial and Volumetric Swell Strrains (Natural)

Soil Type	Natural Soil					
	Vertical Strain (%)		Radial Strain (%)		Volumetric Strain (%)	
	OMC	WOMC	OMC	WOMC	OMC	WOMC
Burleson	5.2	3.2	2.75	1.0	10.9	5.2
Oklahoma	4.2	2.6	2.1	1.2	8.4	5
Riverside	4.8	4.0	2.7	3.0	10.2	10

Table 12 Vertical, Radial and Volumetric Swell Strains (3% Cement)

Soil Type	3% Cement					
	Vertical Strain (%)		Radial Strain (%)		Volumetric Strain (%)	
	OMC	WOMC	OMC	WOMC	OMC	WOMC
Burleson	6.6	5.0	3.1	2.65	12.8	10.3
Oklahoma	6.2	5.2	2.5	2.3	11.2	9.8
Riverside	7.3	5.2	3.25	2.6	13.8	10.4

Table 13 Vertical, Radial and Volumetric Swell Strains (6% Cement)

Soil Type	6% Cement					
	Vertical Strain (%)		Radial Strain (%)		Volumetric Strain (%)	
	OMC	WOMC	OMC	WOMC	OMC	WOMC
Burleson	7.6	6.0	4.25	2.65	16.1	11.3
Oklahoma	7.4	5.8	3.45	2.35	14.3	10.5
Riverside	8.2	6.0	3.5	2.5	15.2	11

Table 14 Vertical, Radial and Volumetric Swell Strains (4% Lime)

Soil Type	4% Lime					
	Vertical Strain (%)		Radial Strain (%)		Volumetric Strain (%)	
	OMC	WOMC	OMC	WOMC	OMC	WOMC
Burleson	8.2	6.1	4.5	3.5	17.2	13.1
Oklahoma	6.2	4.4	2.3	2.0	10.8	8.4
Riverside	5.8	4.2	4.5	3.8	14.8	11.6

Table 15 Vertical, Radial and Volumetric Swell Strains (8% Lime)

Soil Type	8% Lime					
	Vertical Strain (%)		Radial Strain (%)		Volumetric Strain (%)	
	OMC	WOMC	OMC	WOMC	OMC	WOMC
Burleson	8.0	5.6	3.8	2.6	15.6	10.8
Oklahoma	6.5	5.2	4.05	2.9	14.6	11
Riverside	7.2	4.9	4.4	3.9	16	12.7

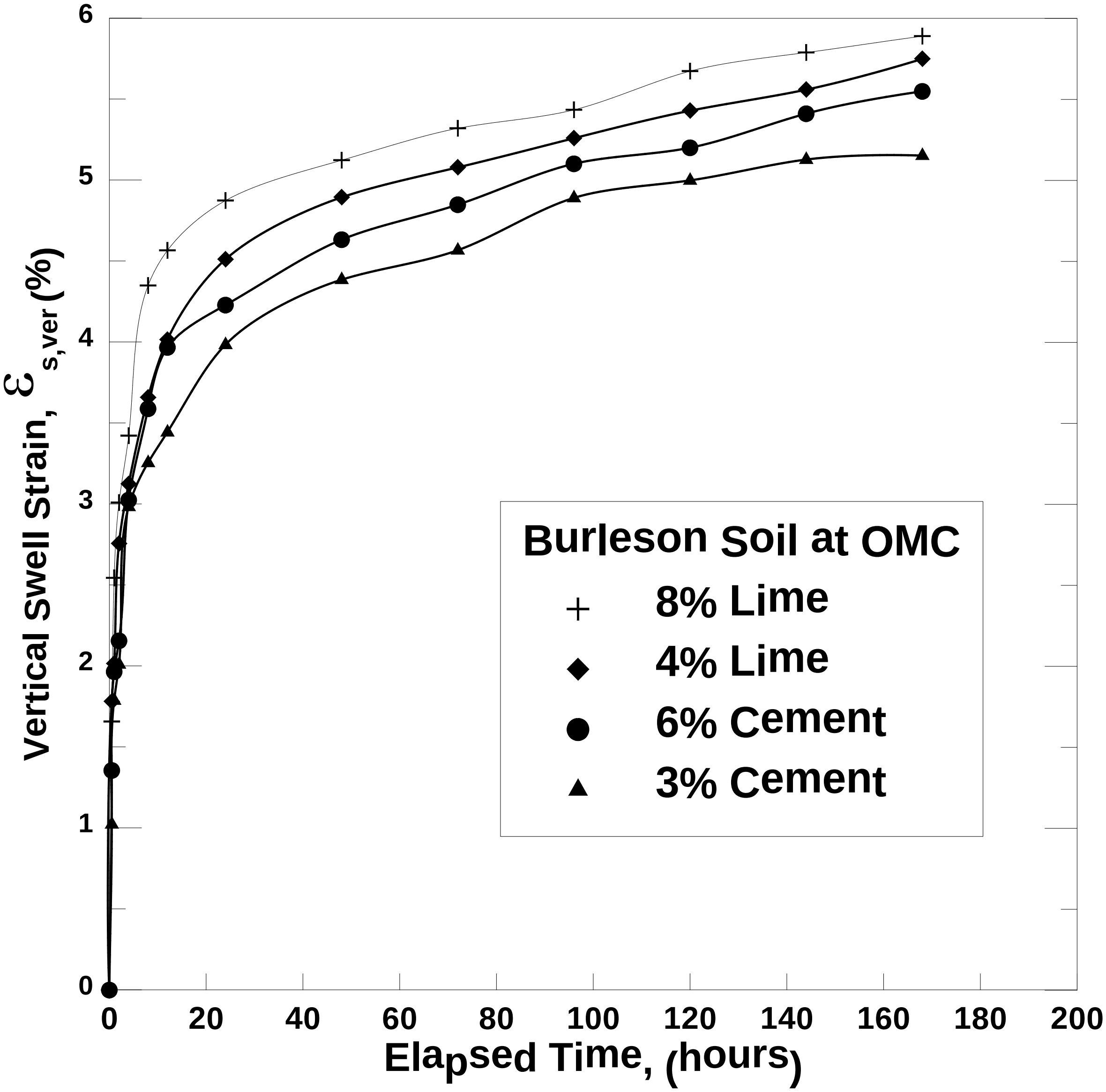


Figure 11 Vertical Swell vs. Elapsed Time (Burleson Soil; 12,000 ppm)

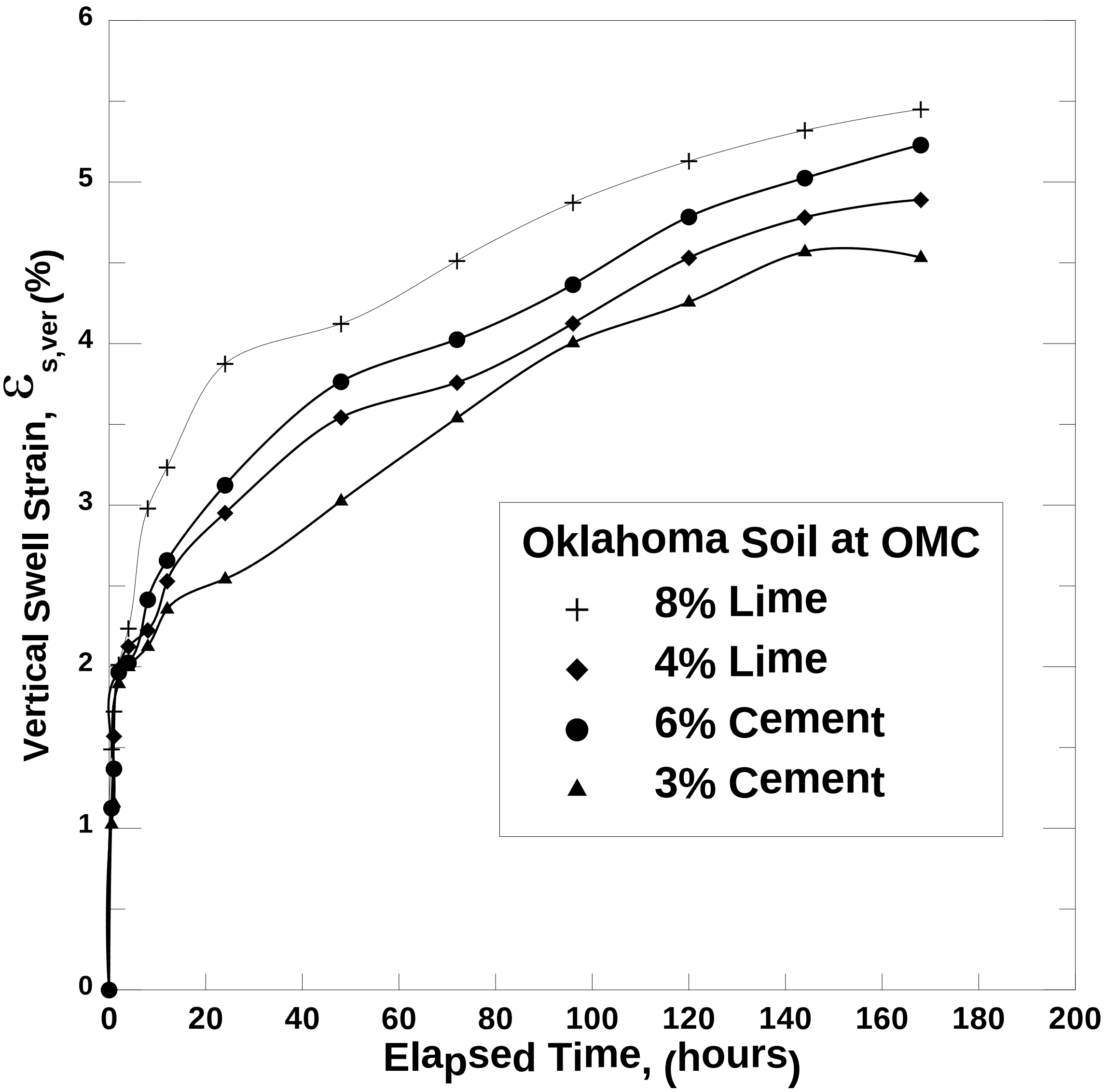


Figure 12 Vertical Swell vs. Elapsed Time (Oklahoma Soil; 15,000 ppm)

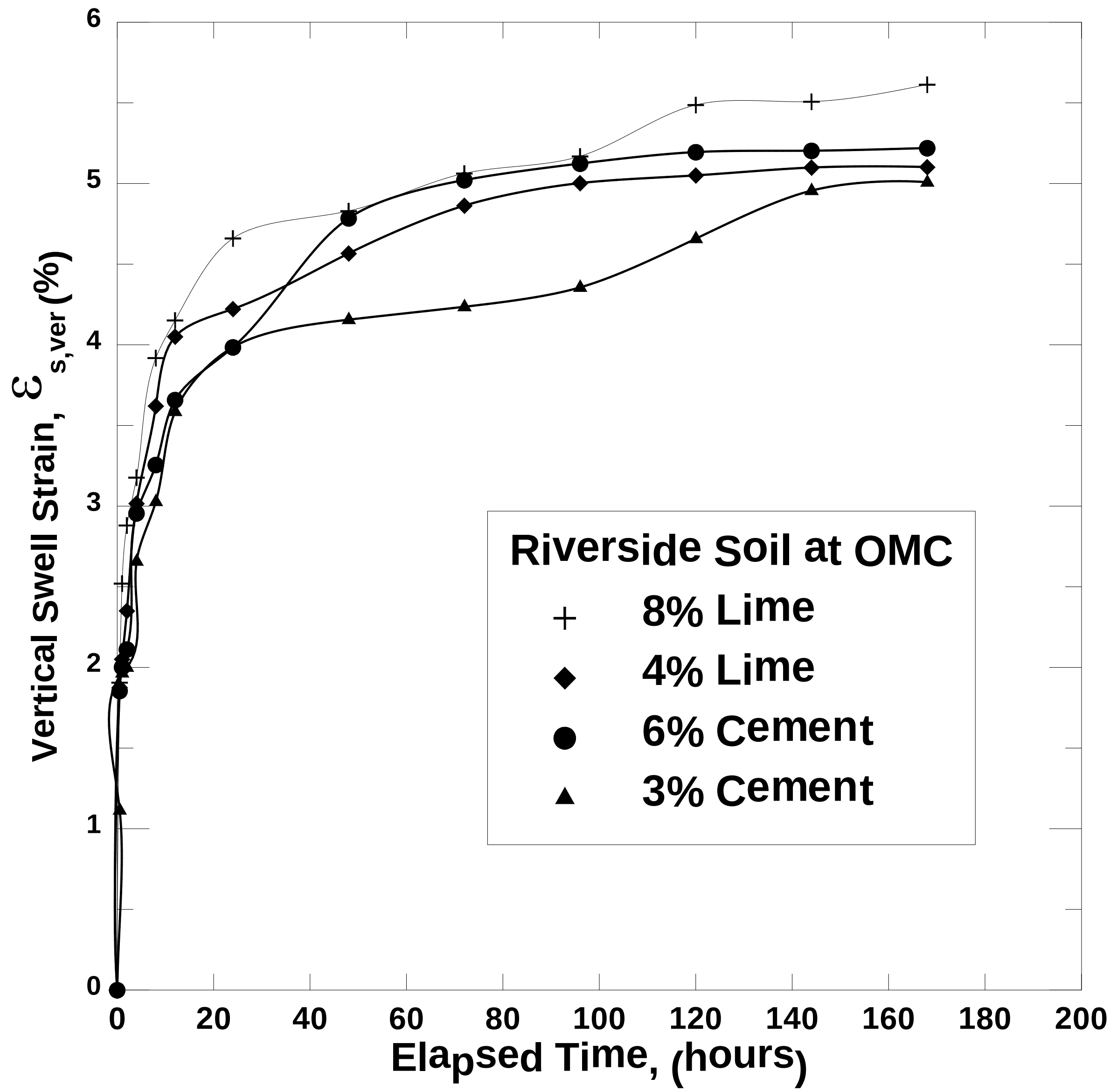


Figure 13 Vertical Swell vs. Elapsed Time (Riverside Soil; 20,000 ppm)

4.5.3 Calculation of Small Strain Shear Modulus (G_{max}) from the BM sensor

In this section, the small strain shear modulus (G_{max}) determined from the monitored shear wave velocity is presented. One end of the BM sensor was connected to a signal generator (transmitter), and the other end was connected to a receiver to receive the output signal. A Sinusoidal pulse was sent from the transmitting end, and the receiving signal was collected and analyzed to determine the time of travel of the shear wave through the material, i.e. treated high sulfate soil specimen. The transmitted and received signals were collected and displayed on the computer screen by a digital oscilloscope connected parallel to the computer. Small strain shear modulus was calculated from the velocity of shear wave thru the soil sample. A sample output of the signal is presented in Figure 14.

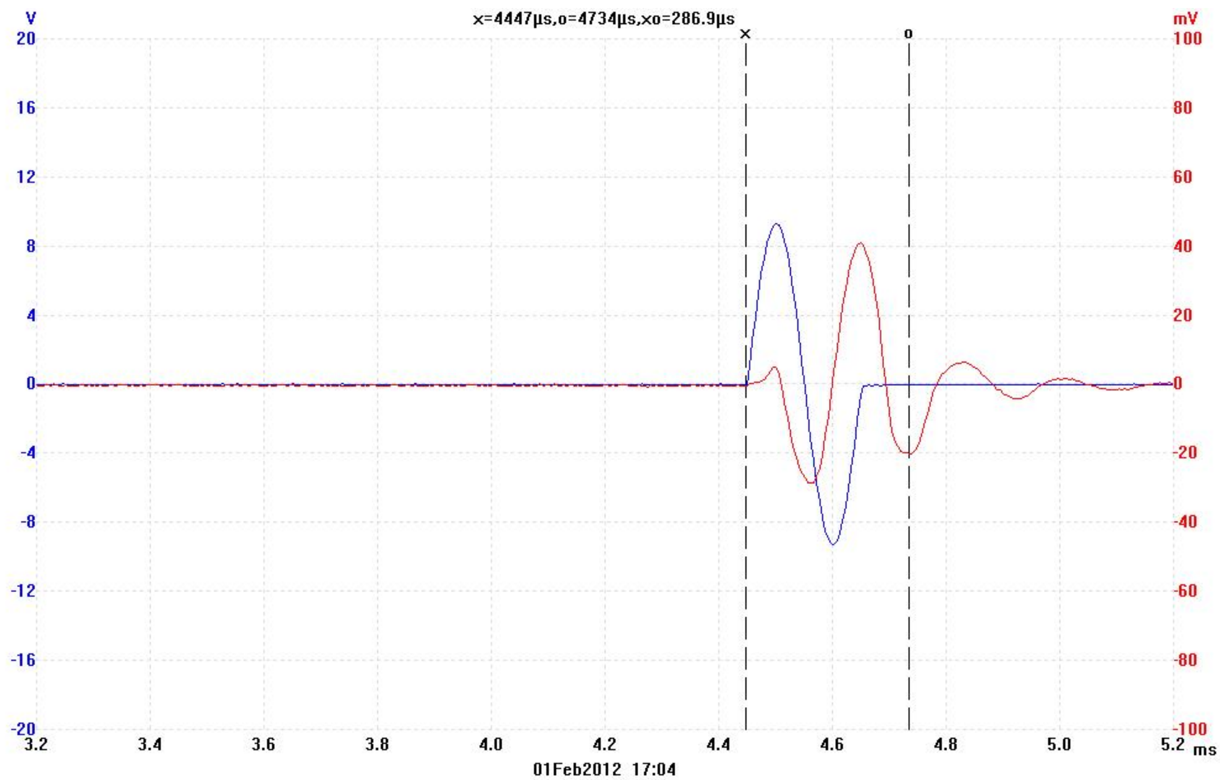


Figure 14 Signal Output from the Oscilloscope

In general, the first significant inversion of the output signal is considered for calculation of the travel time. It was observed in the current study that first inversion gives an error since the wave passes through the soil specimen and the BM sensor at the same time. In order to reduce the error, a second significant inversion of the output signal is considered for calculation of shear modulus in the current study. A sample calculation is presented below for calculation of shear modulus.

From Figure 14, $t = 286.9\ \mu s = 286.9 \times 10^{-6}\ sec$

$$Shear\ Wave\ Velocity, V_s = \frac{L}{t}\ (m/\text{sec})$$

$where\ L = Length\ of\ Travel = 0.033\ m; t = Time\ of\ Travel$

$$V_s = \left(\frac{0.033}{286.9\ X\ 10^{-6}}\right) = 120\ m/s$$

$$Small\ Strain\ Shear\ Modulus\ (G_{max}\ ) = \rho\ .V_s^2$$

$$where\ \ \rho = Mass\ Density\ of\ the\ Material = 1700\ {}^{kg}/_{m^3}$$

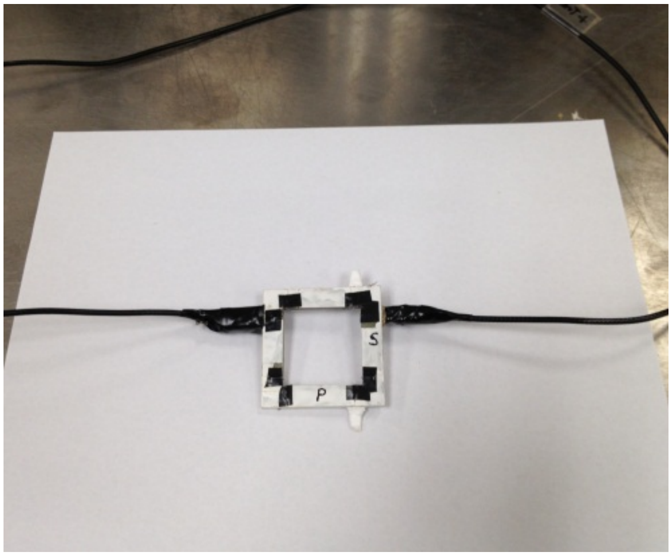
$$G_{max}\ \ = (1700)x(120)^2 = 24.5\ MPa$$

$$G_{max}\ \ = 24.5\ MPa$$

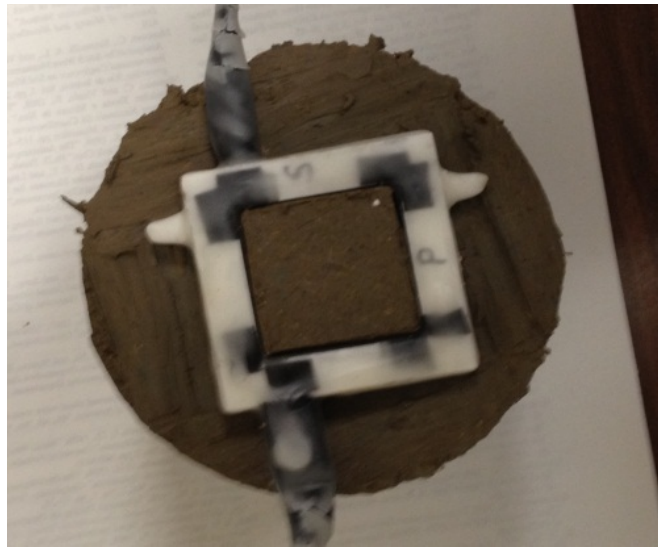
4.5.4 Small Strain Shear Modulus (G_{max}) Measurements

The main intent of this task was to develop a test database of moisture content, dry density and moduli properties of chemically-treated sulfate soils that are near or away from problematic heaving conditions. This data is analyzed in developing criteria for algorithms to be used with the BM sensors for field and laboratory testing conditions. The test BM sensors were used to monitor moisture content, dry unit weight and stiffness property variations. All three soils were used here.

Natural and treated soil samples were prepared at two moisture conditions: OMC and WOMC (95% of maximum dry density). Both natural and treated specimens were embedded with the new BM sensor developed in Task 1. A photograph of the BM sensor embedded in soil sample is given in Figure 15. Treated specimens were cured at room temperature and soaked for swell testing. The samples were soaked for a period until the degree of saturation reached unity. Sample saturation time varied from one day for untreated soils to three days for cement and lime-treated samples. Once the sample was fully saturated, soil moisture, stiffness and density changes were monitored continuously. In addition to this, simultaneous volume changes were measured, which are presented in the earlier section.



(a)



(b)

(c)

Figure 15 Developed BM sensors, Embedment in Soil Specimen and Stiffness Measurement

As mentioned before, natural and treated soil samples were embedded with the new BM sensor, and stiffness measurements were made on them. It is well known that cement and lime treatment improves the moduli properties of soils. In the case of sulfate-bearing soils, Puppala et al., (2006) reported that at high sulfate contents, strength enhancements due to lime and cement treatment were minimal. In some cases, strength reductions were observed. Also, less strength enhancements were seen in samples cured by soaking compared to those cured in a humidity room. In the current study, samples were cured by submerging them in water.

For comparison purposes, Riverside soil, in its natural condition, was also treated with both the stabilizers (lime & cement), and small strain shear modulus measurements on the treated soils were taken. The reason for choosing the Riverside soil for this investigation is that it has a natural low sulfate content of 500 ppm, which is not considered problematic for lime and cement treatments (Puppala et al., 2003). In Riverside soil, shear modulus enhancements up to 50% were observed with lime and cement treatment, indicating no ettringite reactions taking place in the low sulfate soils.

For the high sulfate soils considered in the current study, it was observed that small strain shear modulus (G_{max}) decreased with an elapsed time period in both lime and cement treated soils. This could be attributed to the formation of ettringite and subsequent expansion leading to the softening of the treated soil. Higher shear moduli values were observed in cement-treated soils when compared to the lime-treated soils. This could be attributed to the early pozzolonic reactions occurring with the cement treatment. Though the shear moduli of cement treated soils were higher, the percent loss of moduli was more in the case of cement-treated soils compared to lime-treated soils.

Soaked soil samples with the BM sensor were weighed before stiffness measurements. It was reported in the literature that ettringite formation and their crystal growth enhances the moisture retention of the soil causing further softening of the material. Observed moisture contents varied from 20% to 25% for lime treated soils and 18% to 23% for cement-treated soils. Higher moisture contents recorded in lime-treated soils are consistent with the low shear modulus values.

Initial and final small strain shear modulus values are calculated to assess the reductions in shear modulus with time in chemically treated sulfate bearing soils. Initial and final shear modulus values for nonsulfate soil (Riverside) is presented in Figure 16. Initial and final shear moduli values for natural, lime and cement-treated high sulfate soils are presented in Figures 17 to 19. Shear modulus variation with time is presented in Figures 20 to 27 for the treated non-sulfate soil (Riverside), lime and cement-treated high sulfate soils.

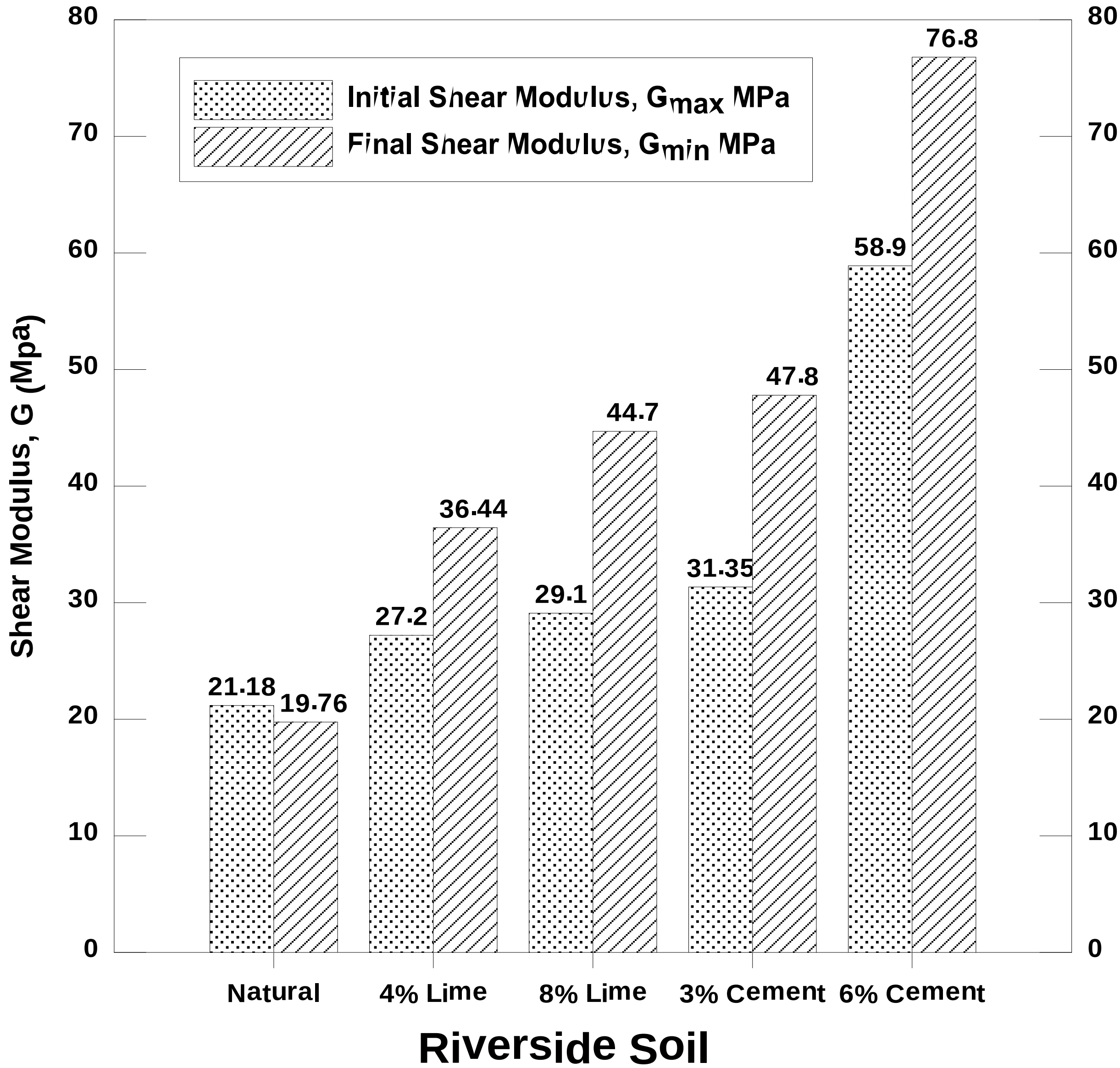


Figure 16 Initial and Final Shear Modulus: Riverside Soil

(Natural Condition; Sulfate Content:500 ppm)

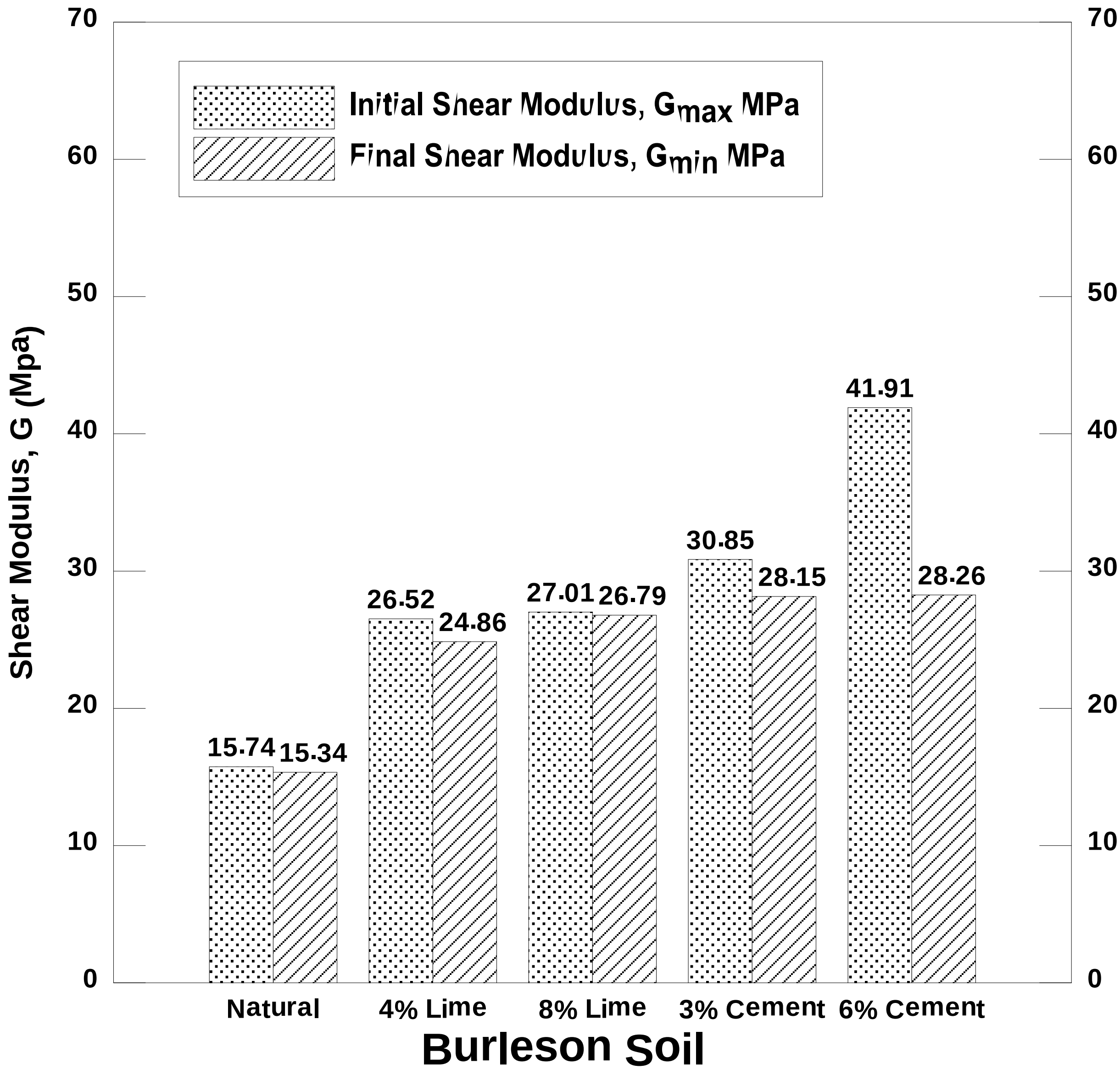


Figure 17 Initial and Final Shear Modulus: Burleson Soil (12,000 ppm)

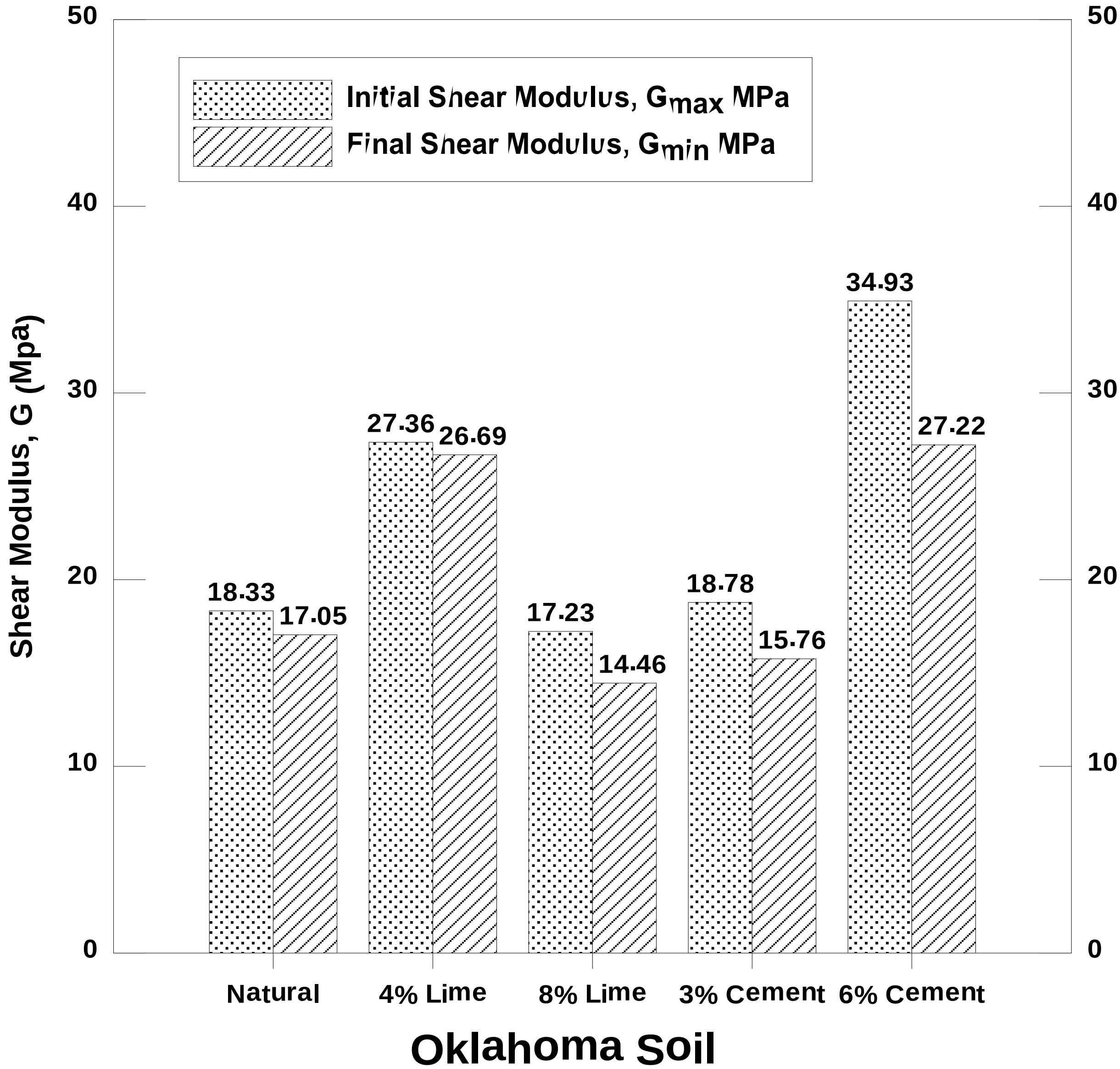


Figure 18 Initial and Final Shear Modulus: Oklahoma Soil (15,000 ppm)

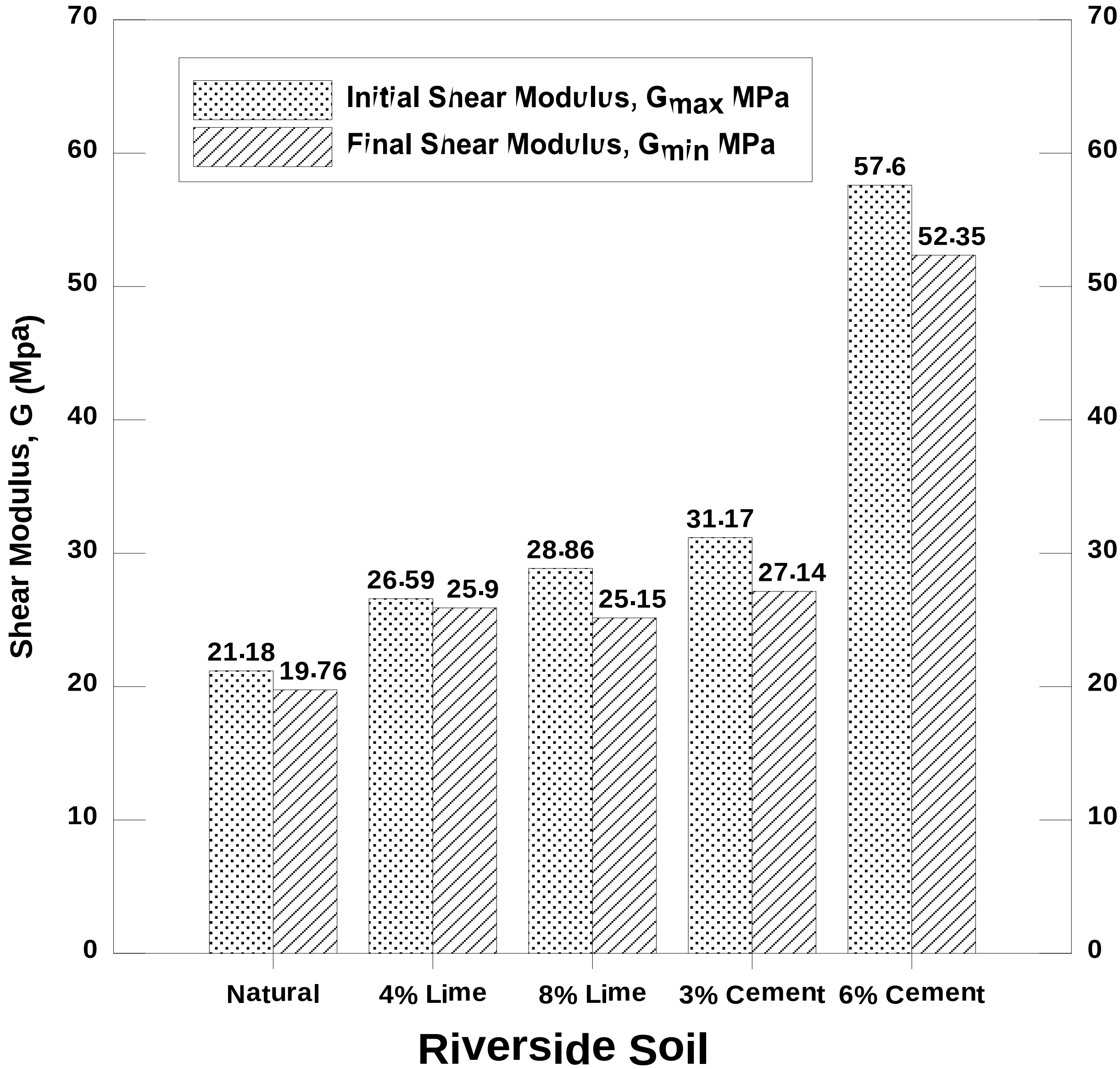


Figure 19 Initial and Final Shear Modulus: Riverside Soil (20,000 ppm)

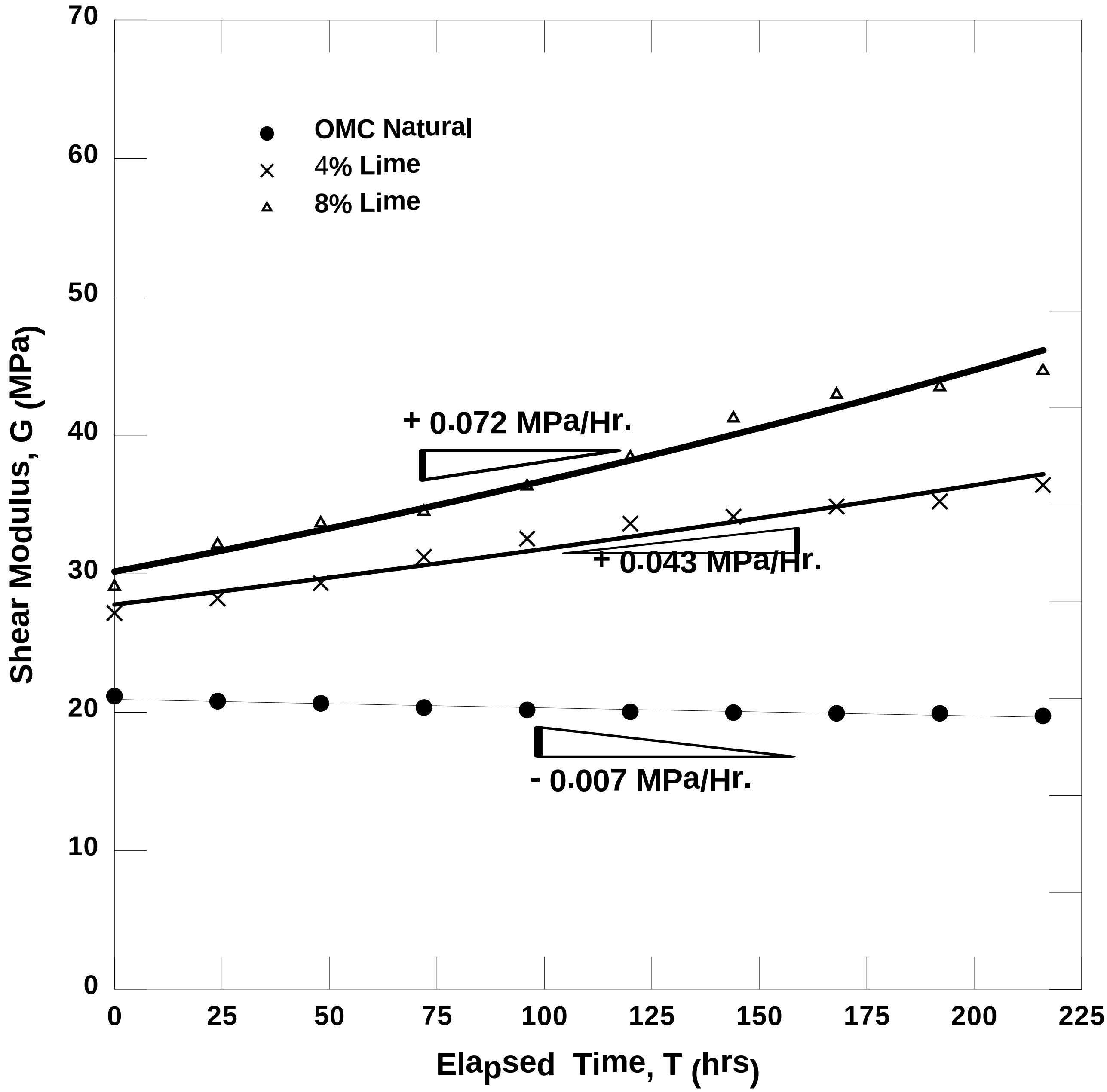


Figure 20 Shear Moduli Variation with Soaking Time Period: Riverside Soil (Natural Condition, 500 ppm sulfates; Lime Treatment)

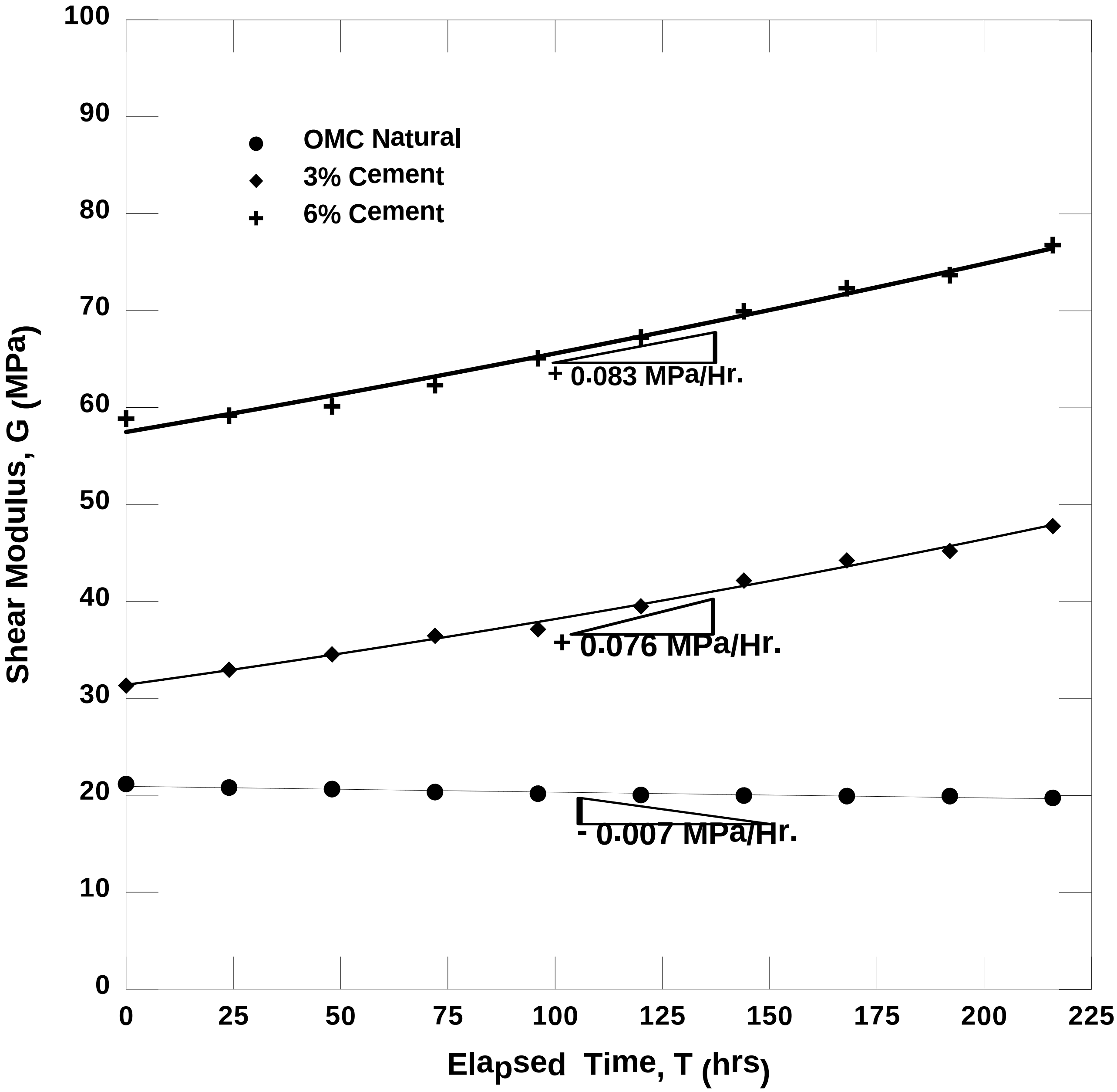


Figure 21 Shear Moduli Variation with Soaking Time Period: Riverside Soil (Natural Condition, 500 ppm sulfates; Cement Treatment)

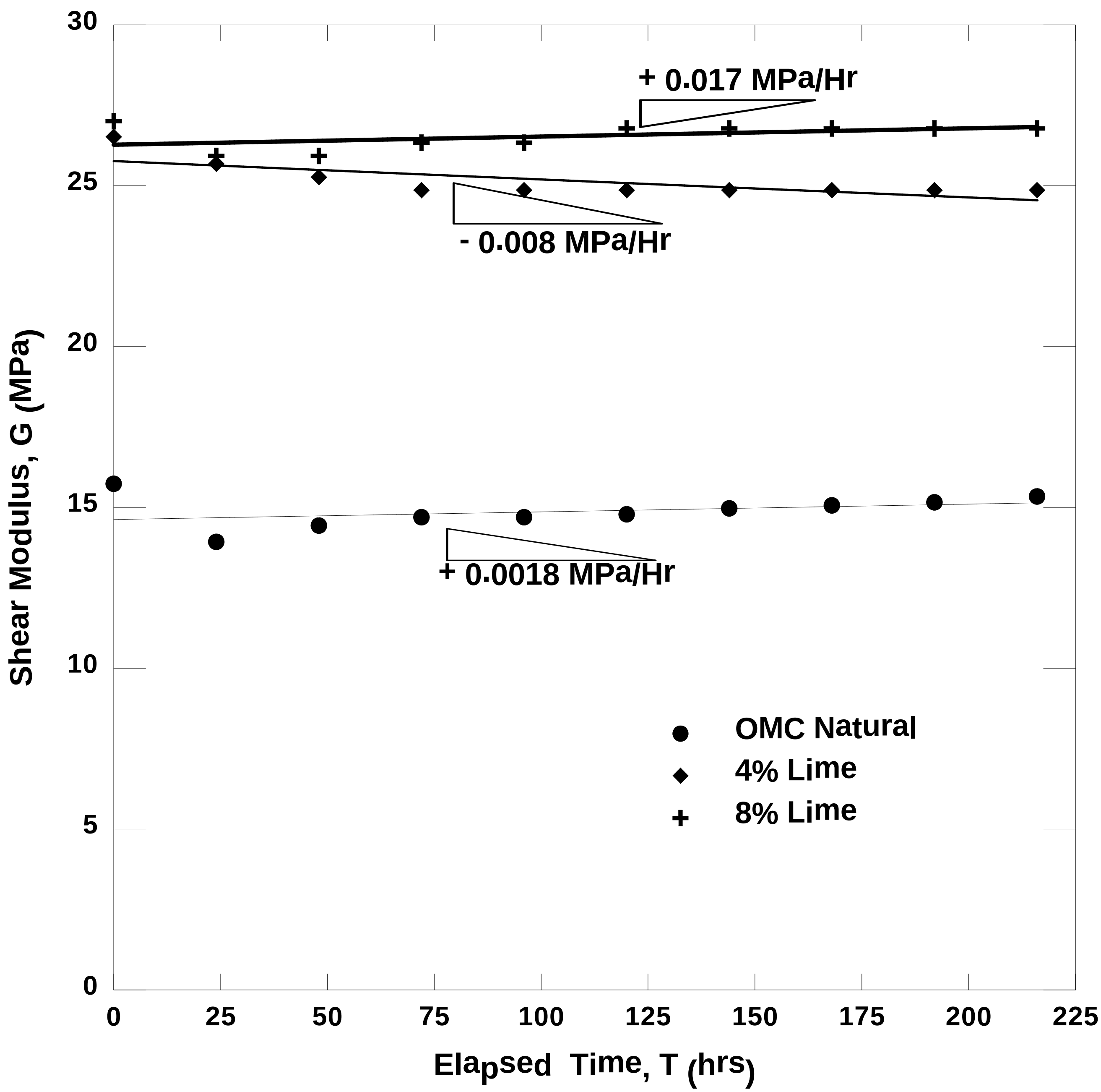


Figure 22 Shear Moduli Variation with Soaking Time Period: Burleson Soil

(15,000 ppm sulfates; Lime Treatment)

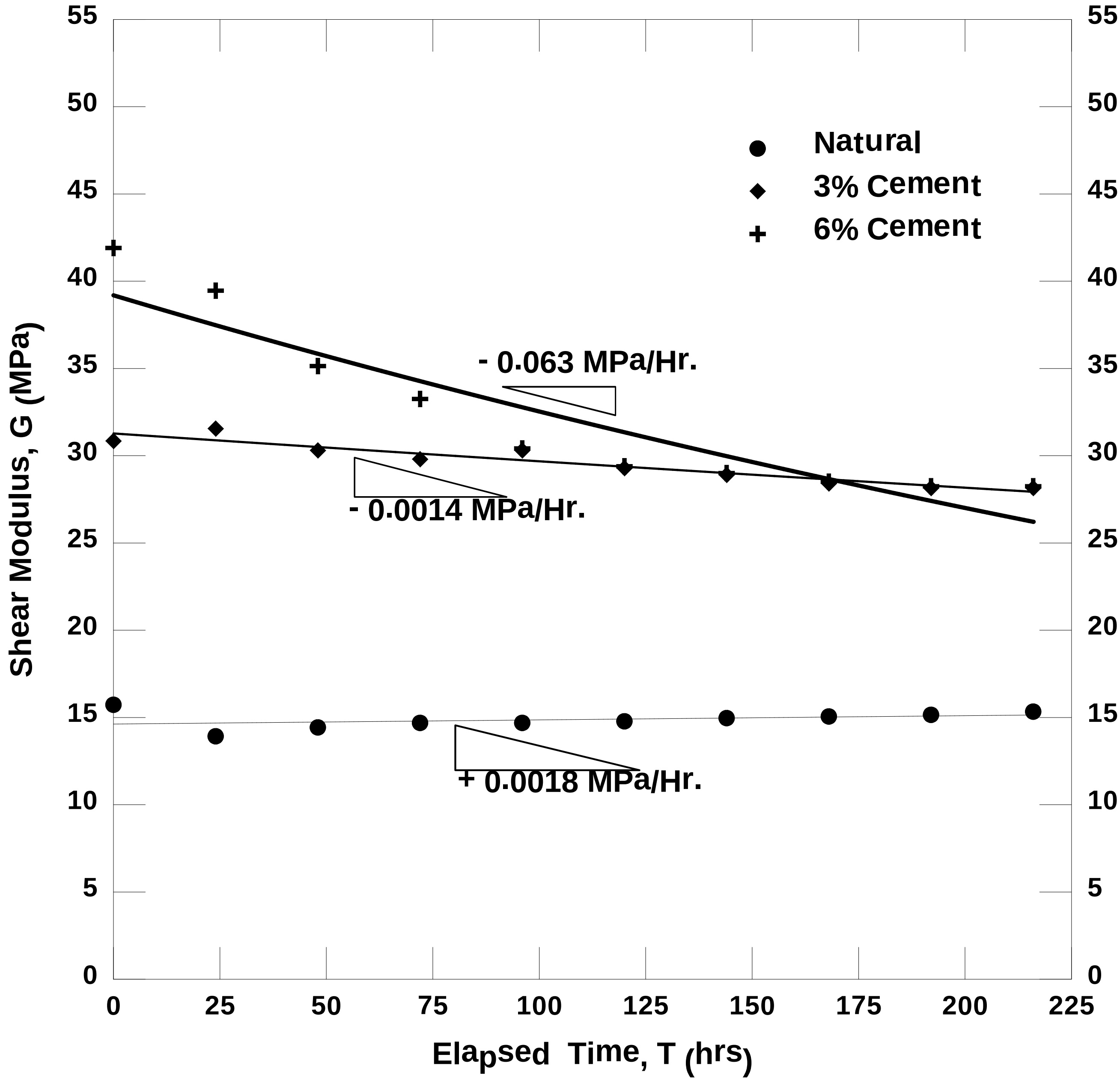


Figure 23 Shear Moduli Variation with Soaking Time Period: Burleson Soil (15,000 ppm sulfates; Cement Treatment)

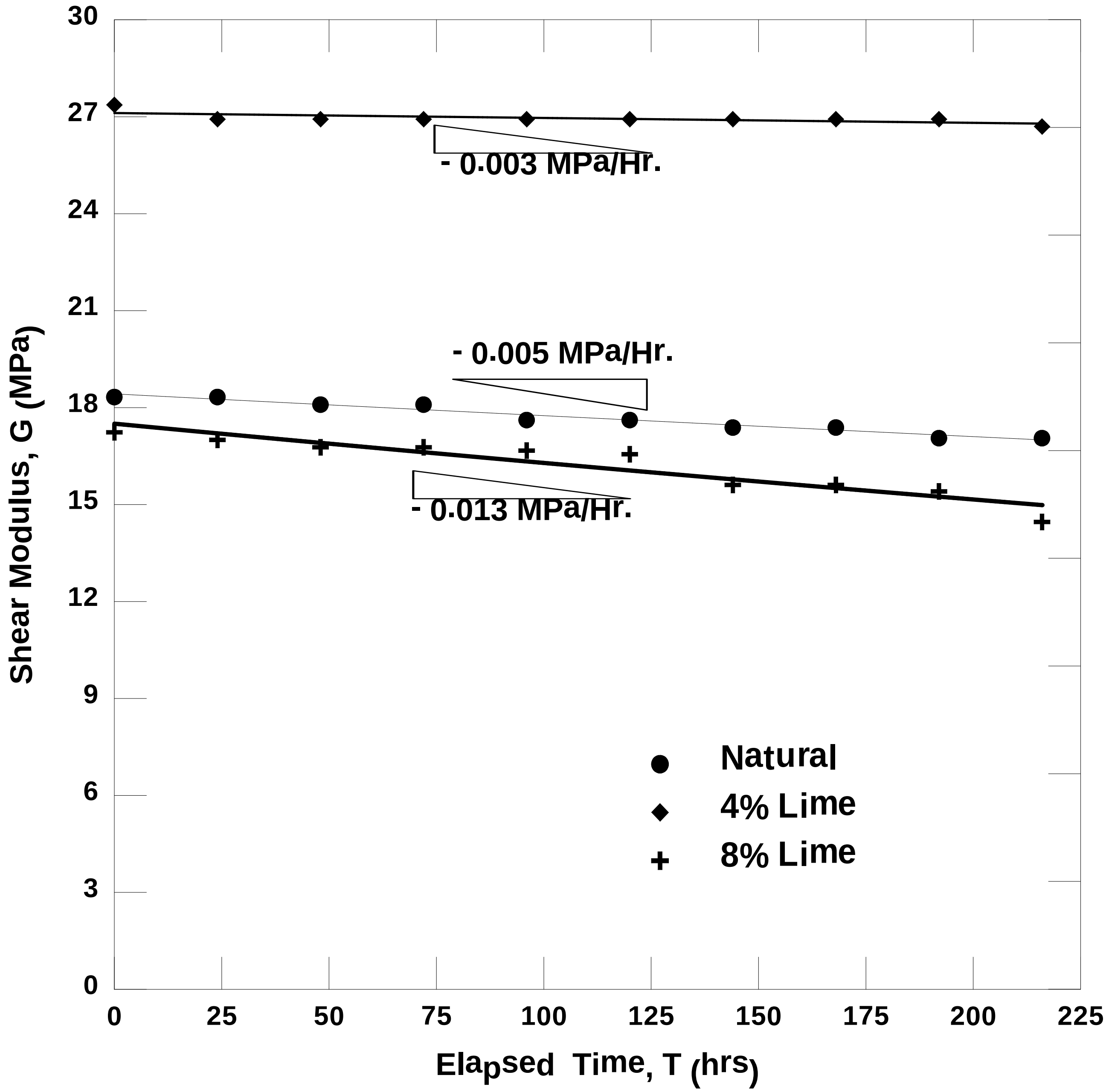


Figure 24 Shear Moduli Variation with Soaking Time Period: Oklahoma Soil (15,000 ppm sulfates; Lime Treatment)

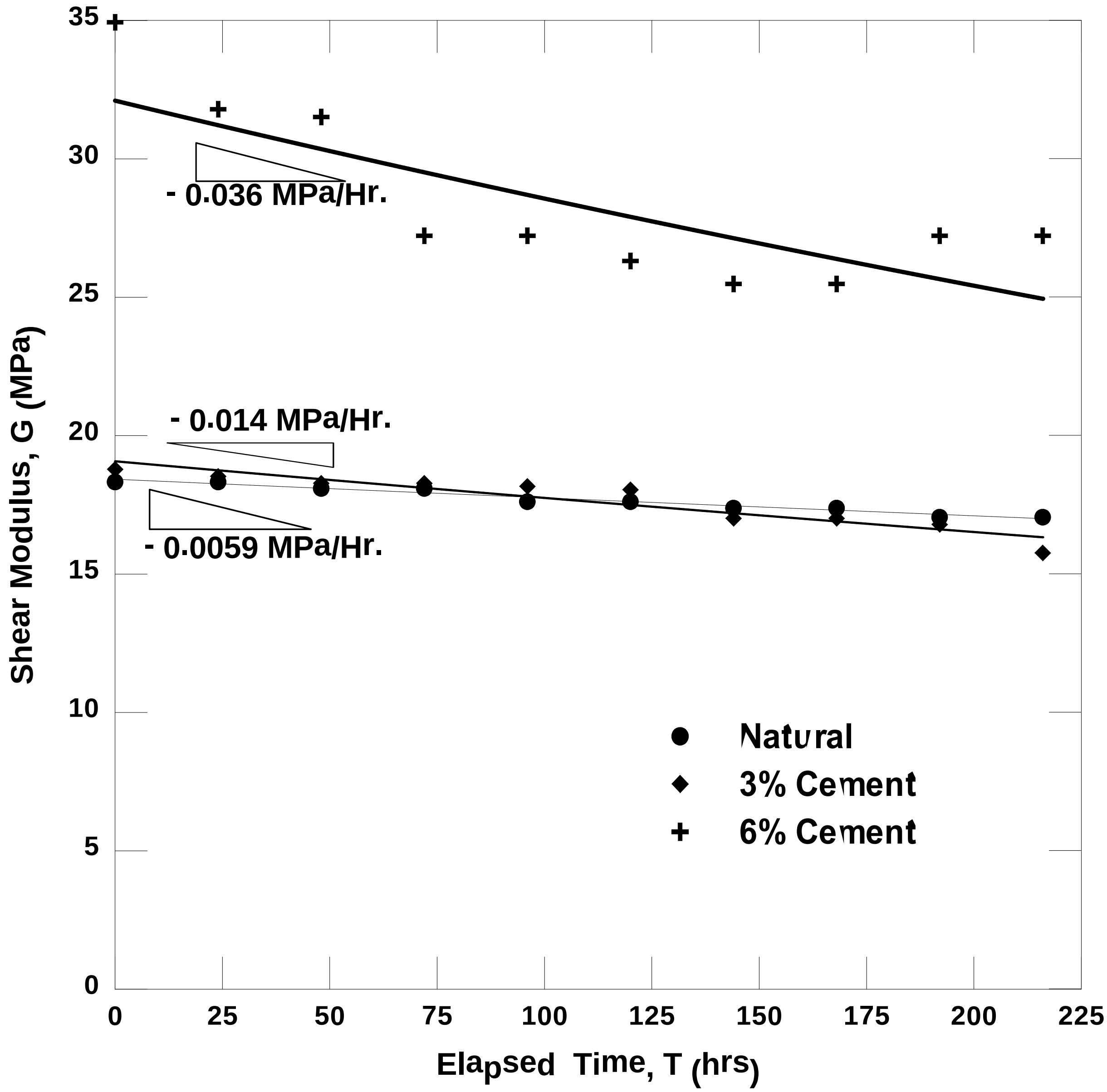


Figure 25 Shear Moduli Variation with Soaking Time Period: Oklahoma Soil

(15,000 ppm sulfates; Cement Treatment)

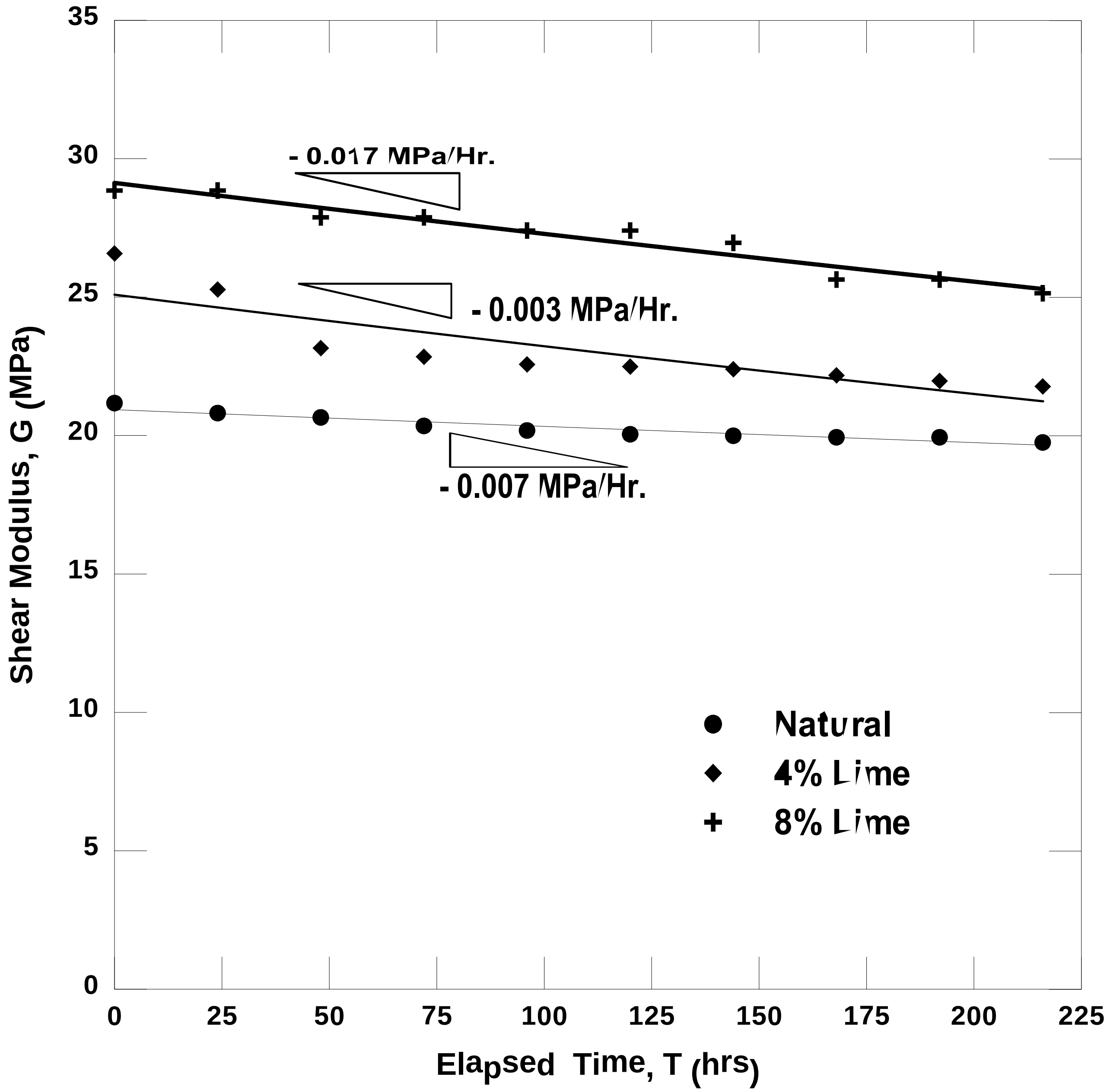


Figure 26 Shear Moduli Variation with Soaking Time Period: Riverside Soil

(20,000 ppm sulfates; Lime Treatment)

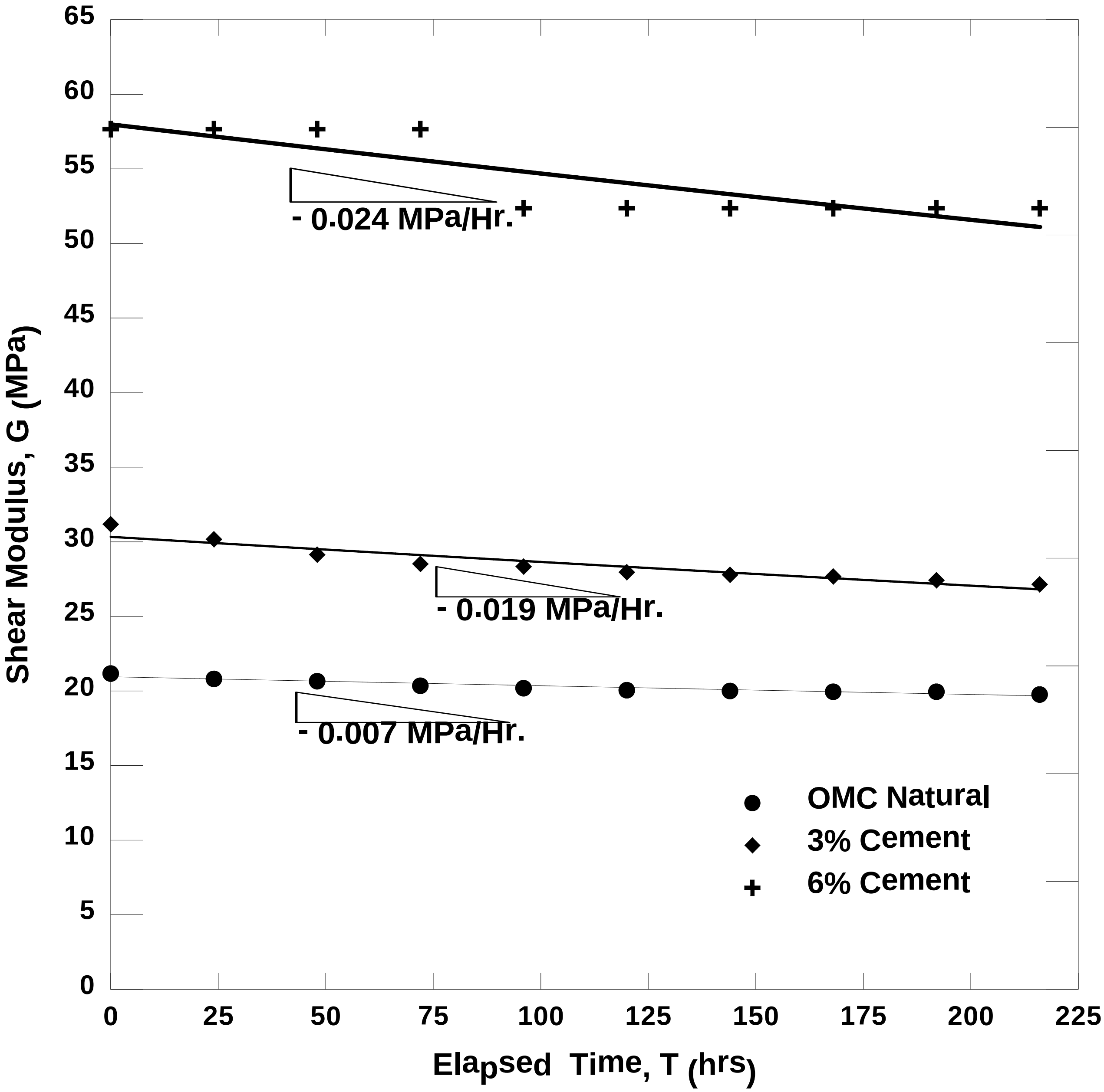


Figure 27 Shear Moduli Variation with Soaking Time Period: Riverside Soil (20,000 ppm sulfates; Cement Treatment)

4.5.5 Analysis of Test Results

In this section, results from the laboratory testing are analyzed and presented. Small strain shear moduli measurements were conducted using the BM sensor embedded in the treated soil samples. Typical small strain shear modulus values for treated soils are presented in Table 16.

Shear modulus enhancements were observed for both cement and lime treated riverside soils. The lowest initial shear modulus was recorded for 4% lime-treated soil (27.2 MPa) and the highest shear modulus value was recorded at 6% cement dosage (58.9 MPa). Final shear modulus values were 50% higher than the initial shear modulus values in both cement and lime-treated soil. Maximum enhancements to shear modulus were obtained in the Riverside soil, as the original sulfate level of this soil is too low (500 ppm) to cause deleterious ettringite formation reactions.

For this soil, both lime and cement-treatments showed reduced shear moduli at elapsed time periods. For 4% lime treated soils, the initial shear modulus varied from 26.5 MPa to 27.3 MPa. For the 8% lime-treated soils, the initial shear modulus varied from 17.2 MPa to 28.8 MPa. It was observed that in lime-treated soils, the increase in stabilizer dosage had minimal impact on the shear moduli values.

In Oklahoma soil, the shear modulus at 8% lime dosage was 17.2 MPa; whereas the same at 4% lime dosage was 27.4 MPa. For 3% cement treated soils, the shear moduli values varied from 18.78 MPa to 31.17 MPa and for 6% cement-treated soils, the intital shear moduli varied from 34.93 MPa to 57.6 MPa. For cement-treated soils, shear modulus increased with an increase in dosage levels in general. Overall, it can be seen that shear moduli values were higher for cement-treated soils than for lime-treated soils. Though higher shear modulus values were observed with increased stabilizer dosages, loss in shear modulus was the highest for cement treatment when compared to lime treatment of the same materials.

Table 16 Typical Small Strain Shear Modulus Values for Soils

Soil Type	**G_{max} (MPa)**	
	Min.	**Max.**
Soft Clays	3	14
Firm Clays	7	35
Silty Sands	28	138
Dense Sands and Gravel	69	346

Also, the shear modulus values for different treatments were plotted against the elapsed time periods, and the slopes of the time rate related moduli changes are determined and they are expressed as MPa/Hr. For the Riverside soil, in its natural condition, the slope of the line is negative since strength gain was recorded in this case. Slope of the line in MPa/Hr for other treatements is presented in Figure 17 for Riverside soil. For other test soils, the threshold values of stiffness losses were calculated from the slopes of the line drawn at different stabilizer dosages. Inital and final shear modulus and threshold stiffness loss (MPa/Hr.) for different treatments considered for all test soils are determined and presented in Tables 18 thru 21.

From Tables 18 thru 21, it can be observed that in 4% lime treated soils, the threshold stiffness loss is calculated as 0.005 MPa/Hr; whereas the same for 8% lime treatment soils, this value is around 0.010 MPa/Hr. From threshold stiffness loss values, one can note that at higher lime dosages, the sulfate reactions occur at a faster pace, leading to material softening and consequently reduction in small strain shear moduli. In cement-treated soils, the threshold stiffness loss varied from 0.015 MPa/Hr. to 0.040 MPa/Hr at 3% and 6% cement treatments, respectively. The increase in threshold stiffness loss at higher cement dosages is indicative of the destabilizing reactions in cement-treated sulfate-bearing soils. The observed threshold stiffness losses are in line with the volumetric swell values observed in the soils under study.

Among the three soils considered in the current study, the Burleson soil is a fat CH clay type and the soils from Oklahoma and Riverside are CL lean clays. The observed threshold stiffness loss overall is higher in thefat clayey soil when compared with the lean clayey soils. The high pasticity nature of the Burleson soil and sulfate contents in excess of 10,000 ppm could be the reasons for larger moduli reduction rates than in low plasticity soils.

Table 17 Rate of Change of Stiffness in MPa/Hr. for Lime & Cement Treated Riverside Soil

Soil	Riverside Soil (Sulfate Content : 500 ppm)			
	Initial	**Final**	**Gain**	**Stiffness Rate (MPa/Hr.)**
4% Lime	27.2	36.44	9.2	0.043*
8% Lime	29.1	44.7	15.6	0.072
3% Cement	31.35	47.8	16.45	0.076
6% Cement	58.9	76.8	17.9	0.083

***Positive Sign Indicates Strength Improvements**

Table 18 Rate of Change of Stiffness in MPa/Hr. for 4% Lime Treatment

Soil	4% Lime				
	Initial	Final	Loss	Stiffness Rate (MPa/Hr.)	Volumetric Swell (%)
Burleson	26.52	24.86	1.66	-0.008*	17.2
Oklahoma	27.36	26.69	0.67	-0.003	10.8
Riverside	26.59	25.9	0.69	-0.003	14.8

***Negative Sign Indicates Strength Losses**

Table 19 Rate of Change of Stiffness in MPa/Hr. for 8% Lime Treatment

Soil	8% Lime				
	Initial	Final	Loss	Stiffness Rate (MPa/Hr.)	Volumetric Swell (%)
Burleson	27.01	26.79	0.22	-0.001*	15.6
Oklahoma	17.23	14.46	2.77	-0.013	14.6
Riverside	28.86	25.15	3.71	-0.017	16

***Negative Sign Indicates Strength Losses**

Table 20 Rate of Change of Stiffness in MPa/Hr. for for 3% Cement Treatment

Soil	3% Cement				
	Initial	Final	Loss	Stiffness Rate (MPa/Hr.)	Volumetric Swell (%)
Burleson	30.85	28.15	2.7	-0.013*	12.8
Oklahoma	18.78	15.76	3.02	-0.014	11.2
Riverside	31.17	27.14	4.03	-0.019	13.8

***Negative Sign Indicates Strength Losses**

Table 21 Rate of Change of Stiffness in MPa/Hr. for 6% Cement Treated Soils

Soil	6% Cement				
	Initial	**Final**	**Loss**	**Stiffness Rate (MPa/Hr.)**	**Volumetric Swell (%)**
Burleson	41.91	28.26	13.65	-0.063*	16.1
Oklahoma	34.93	27.22	7.71	-0.036	14.3
Riverside	57.6	52.35	5.25	-0.024	15.2

***Negative Sign Indicates Strength Losses**

Overall, the threshold loss of stiffness values are higher in cement-treated soils when compared to the lime-treated soils. The threshold stiffness loss in cement-treated soils is 3 to 4 times higher than in the lime-treated soils.

4.5.6 Laboratory testing program for Calibration of TDR for field applications

Calibration of TDR is necessary for every specific soil in order to evaluate the soil specific constants “a” & “b”. Once the soil specific constants are obtained, it can be used for field application to evaluate the moisture content. This testing includes calibration, which is explained below.

In order to determine the moisture content of the soil, the TDR has to be calibrated with the field soil. Calibration of TDR can be summarized using following steps:

1) Determine the volume of the mold and mass of the empty mold.
2) Obtain soil samples from the representative testing site.
3) Air-dry the required amount of soil sample that will be used for calibration testing, using the oven.
4) Use the air dried sample to prepare three soil specimens for different moisture contents (20%, 25% and 30%). (The moisture contents are selected such a way that it simulates the expected range of moisture contents observed in the field.)
5) Place the soil in the mold to a certain height and compact it, using a aluminium rod. Place the TDR probe on top of the soil and fill the rest of the mold with the soil specimen. (Proper care should be taken while compacting the soil along the TDR probe so that no damge is done to the probe.)

6) Weigh the mold along with the wet soil. (Since the weight of the empty mold and the volume of the mold is known, the density of the soil can be calculated.)
7) Test the prepared soil specimens in the mold to obtain TDR waves using the pulse generator named TDR100, as shown in Figure 28. Campbell Scientific software is used to monitor the the generated wave form.
8) Once the TDR waveforms are generated, collect the soil sample from each specimen to measure the gravimetric water content of the soil in accordance with ASTM D 2216.

After performing the required tests, the gravimetric water content of the soil and dielectric constant values are calculated, along with the density of the soil, which are tabulated in Table 22. Specific soil constants are calculated by performing series of linear regression plots. Soil constants "a" & "b" are found by plotting $\sqrt{K_a}\frac{\rho_w}{\rho_d}$ vs ω(gravimeteric) , where ω is the gravimetric water content, ρ_w is the density of water, ρ_d is the dry density of soil, and K_a is the dielectric constant of the soil. A best-fit line is obtained from the data where "a" is the zero intercept of the line which is 0.0198, and "b" is the slope of the line 10.733 as shown in Figure 29.

Figure 28 Laboratory Soil Specimen Used for Calibration of TDR

Table 22 Data Measured for Individual Soil Specimens

Soil Specimen	Target Moisture Content (%)	Dry Density of the Soil, ρ_d (kg/m^3)	Dielectric Constant,Ka	TDR Paramter, $\sqrt{K_a}\frac{\rho_w}{\rho_d}$	Gravimetric Water Content, ω
1	20	1438.79	6.70	1.80	0.17
2	25	1428.23	11.98	2.42	0.23
3	30	1483.97	19.67	2.99	0.28

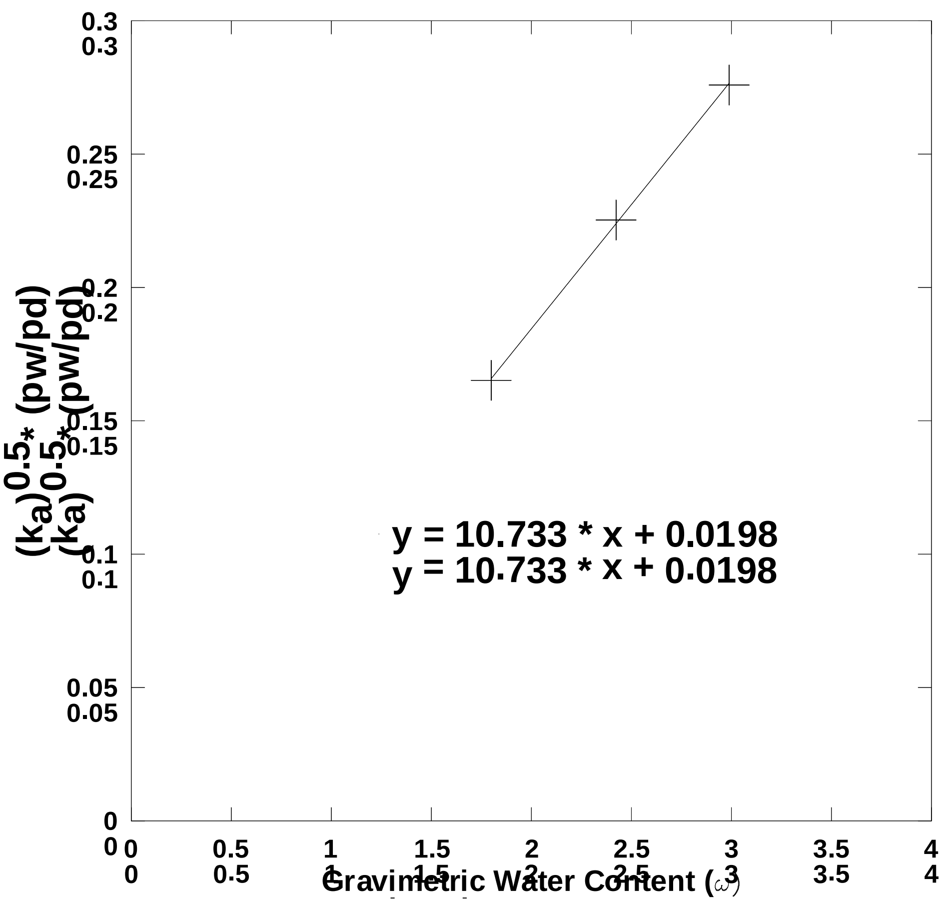


Figure 29 Gravimetric Water Content vs TDR Parameter, $\sqrt{K_a}\frac{\rho_w}{\rho_d}$

5 FIELD STUDIES

5.1 Field Validation Studies for the BM sensor

After completing the laboratory studies, the BM sensor still need to be tested and validated in the real field conditions for measuring the stiffness property changes in treated high sulfate soils. This section is dedicated to the field validation of the developed BM sensor for detecting the sulfate-induced heave in the test section.

For field implementation of the IDEA research results, a lime-treated test section was constructed in a high sulfate soil environment. The BM sensor was then embedded in the newly built area, and shear modulus measurements were conducted over a period of time. North Gate Constructors from Dallas has agreed to assist research team with the construction of test section on high sulfate soils. The test section is part of the DFW Inter Connector project. High sulfate soils are present at this site and several sulfate heave issues were reported on pavements around this site. The test section was located in the median area between Highway 114 and International parkway, close to the north entrance of the Dallas/Fort Worth Airport. Sulfate tests were conducted on the natural soils, and the results showed sulfate content in excess of 30,000 ppm. A picture depicting the shiny gypsum crystals in the natural soil formation is presented in Figure 30. The soils in this area are high plasticity clays, with plasticity index values are greater than 50. Soil properties from the test location are determined in the laboratory and these results are presented in Table 23. Figure 31 shows the aerial image of location of the test section.

Figure 30 Gypsum Crystals in the Natural Soil Formation at the Test Site

Table 23 Summary of Field Soil Prpoperties

Soil	Sulfate Content (ppm)	Atterberg Limits				Compaction Properties: 6% Lime Treated Soil	
		LL	PL	PI	USCS Classification	OMC (%)	MDD (lb/ft^3)
Burleson	32,000	76	24	52	CH	23	96

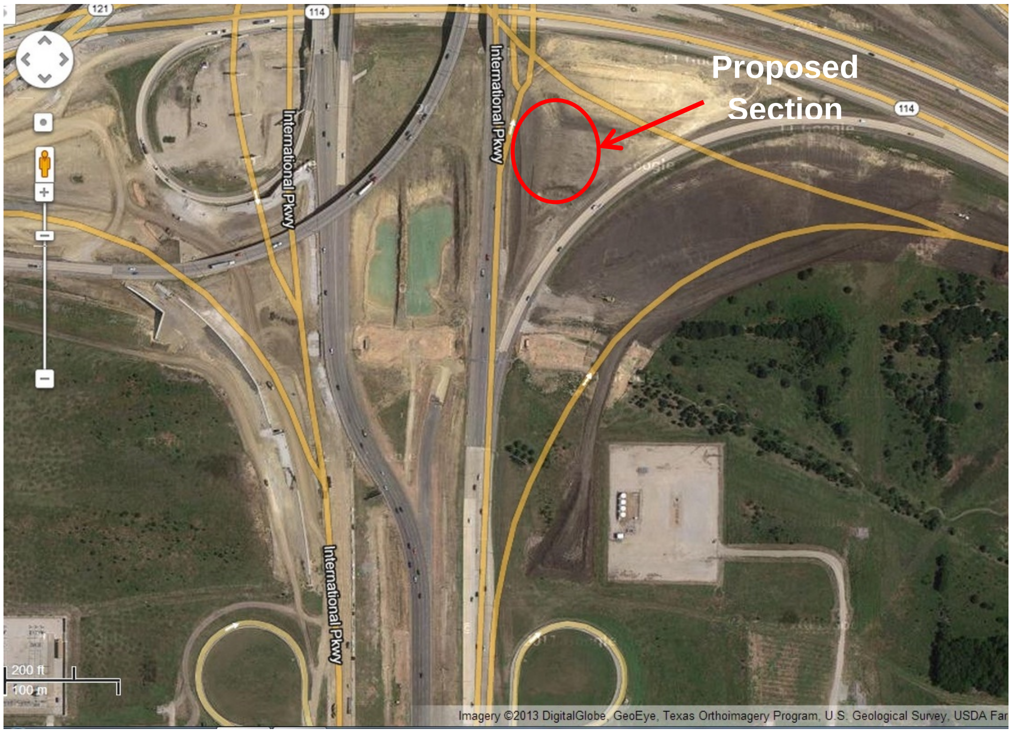


Figure 31 Location of the Test Section

As mentioned before, soils in this area have sulfate content in excess of 30,000 ppm. From this area, samples were collected and sent to the University of Texas at Arlington geotechnical laboratory. Lime dosage was determined and this value was 6% based on the pH tests conducted. Lime-treated proctor curves were developed, and the optimum moisture content was determined as 23% and maximum dry density as 96 lb. /ft^3. The proposed area was scarified and treated with 6% lime slurry. Based on the laboratory proctor test results, soils were compacted to the targeted density in a 25ft. X 60ft. section. Construction sequence of the test section is presented in Figure 32.

Figure 32 Construction Sequence (a) Initial Subgrade Preparation (b) Lime Treatment and Water Application (c) Final Compaction (d) Finished Section

The BM sensor, along with Time Domain Reflectometry (TDR) sensor was embedded at a depth of 8 in. in the treated section and re-compacted. Continuous monitoring of shear modulus and moisture content values were monitored for a period of one month at the test section. The treated section was watered three times a day to keep a continuous supply of moisture to induce sulfate reactions in the treated soil. Figure 33 illustrates the embedment of the hybrid BM sensor and the data collection module from the embedded BM sensor.

Figure 33 (a) Integrated BM sensor (BE & TDR) Embedment in Treated Subgrade Soil (b) Data Collection Module

5.1.1 Analysis of Field Test Results

Treated soil in the field was allowed to cure for 72 hours to gain initial strength. The field testing procedure required only a few minutes of setup and about 2 to 3 minutes to perform the test. In a week, sulfate heave assessment could be completed. This reduction in time is a significant advantage when compared to the laboratory procedures that takes several weeks to complete the sulfate heave assessment. The generated wave for first day measurement of moisture content using TDR probe is shown in Figure 34. A summary of the moisture content values obtained from the TDR probe are presented in Table 2. The procedure to determine the moisture content of the soil using the waveform can be analyzed as follows:

1. Connect the TDR probe to the single pulse generator named TDR100 to obtain the waveform of the The TDR.
2. Obtain and plot the first derivative of all the points in the waveform,, as shown in Figure 35.
3. Using method of tangents, analyze the waveform and 1st derivative of all the points to obtain the apparent length (L_a) as shown in Figure 36.
4. Once the apparent length is obtained, calculate the dielectric constant of the soil, using equation 3-1.
5. Now, with the calculated dielectric constant of the soil and specific soil constants, determine the moisture content of the soil by using equation 3-2.
6. Repeat the steps 1 through 5 for different waveforms to obtain moisture content of the soil at different time periods.

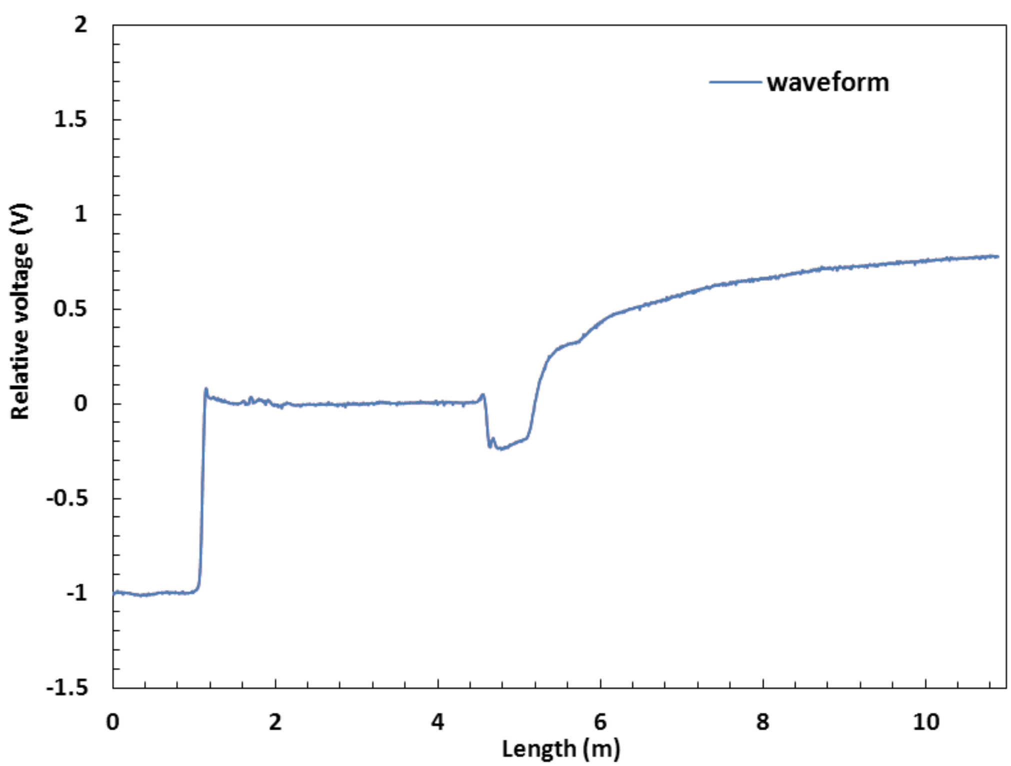


Figure 34 TDR Waveform for 1st day

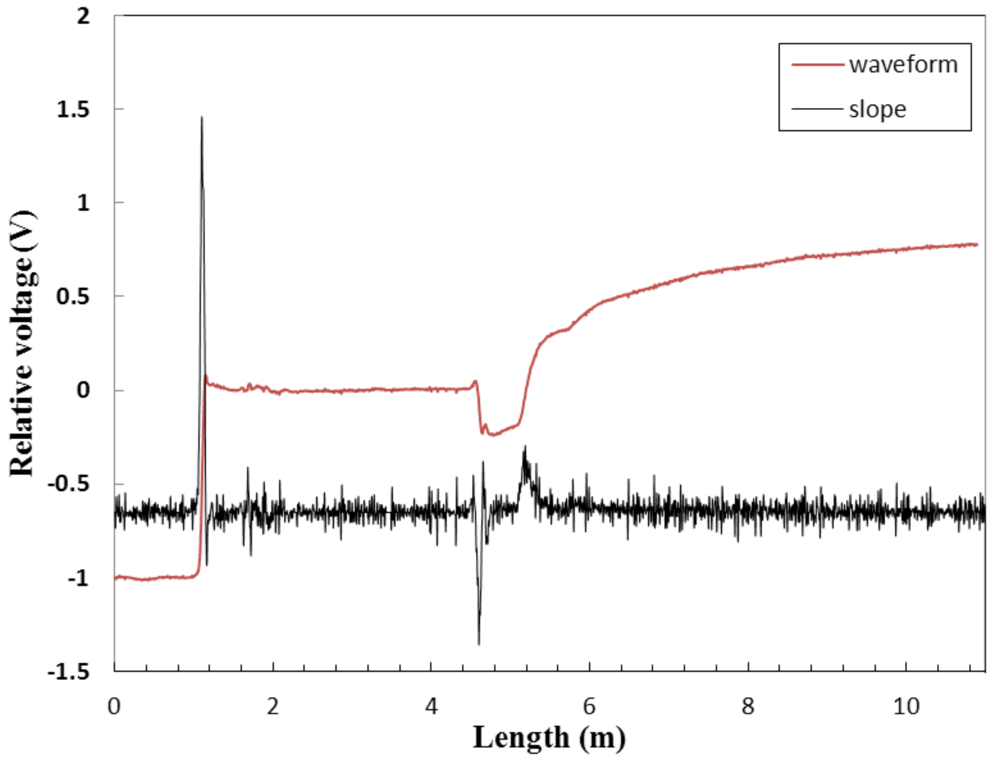


Figure 35 1st Derivative of the TDR Waveform

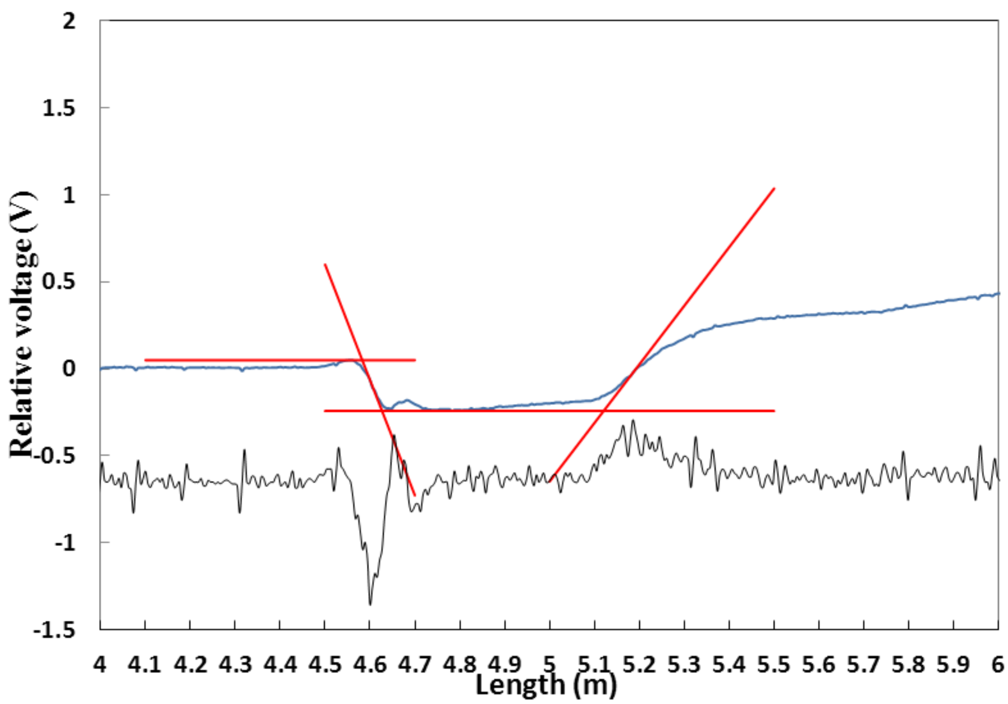


Figure 36 Analysing TDR Waveform using Method of Tangents

Table 24 Summary of the Results Obtained from TDR Waveform

Day	Dielectric Constant, K_a	Moisture Content, ω (%)
1	12.84	22.8

On the first day, the moisture content of the soil of 22.8% (moisture that was added was 23%) was obtained. During the second day, a proper signal could not be detected from the TDR probe, which is depicted in Figure 37. Due to loss of connections, the moisture readings of TDR beyond first day was not able to complete. The movement of a heavy water truck (needed to provide continuous moisture access to soils) over the test site was attributed to loss of contacts. Despite the loss of moisture content measurements, the stiffness measurements and their data collection was continued for the next 30 day monitoring period.

Rate of shear moduli changes over 30 days are presented in Figure 38. After a month of monitoring, the bender element sensor connection was damaged due to the same truck and a photograph is shown in Figure 39. Nevertheless, the data collected for thirty days was more than sufficient to evaluate the performance of the BE sensor in the field evaluations.

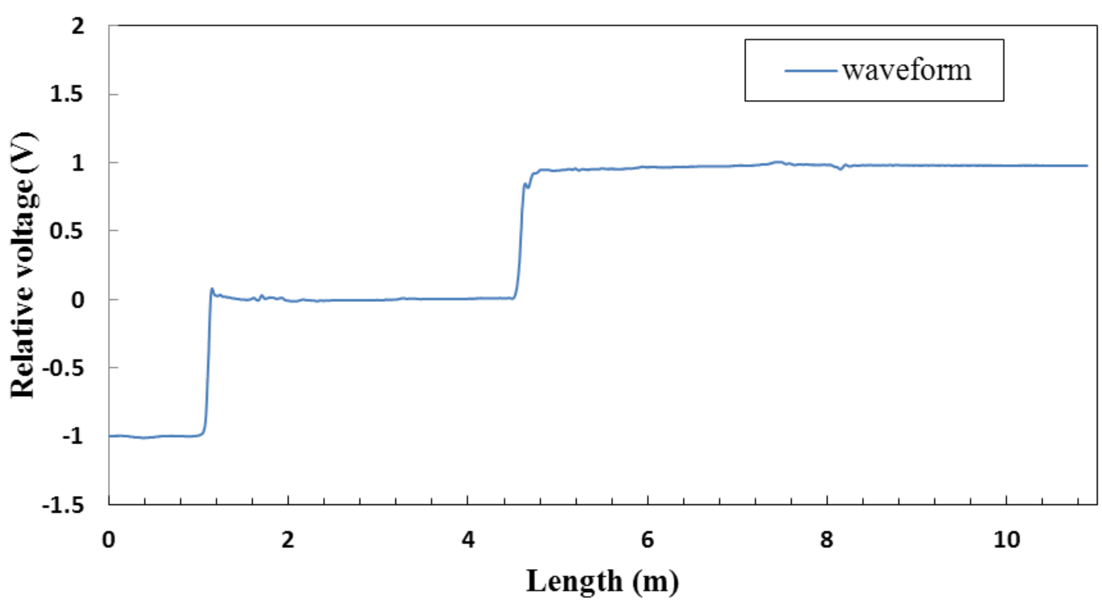


Figure 37 TDR Waveform for 2nd Day

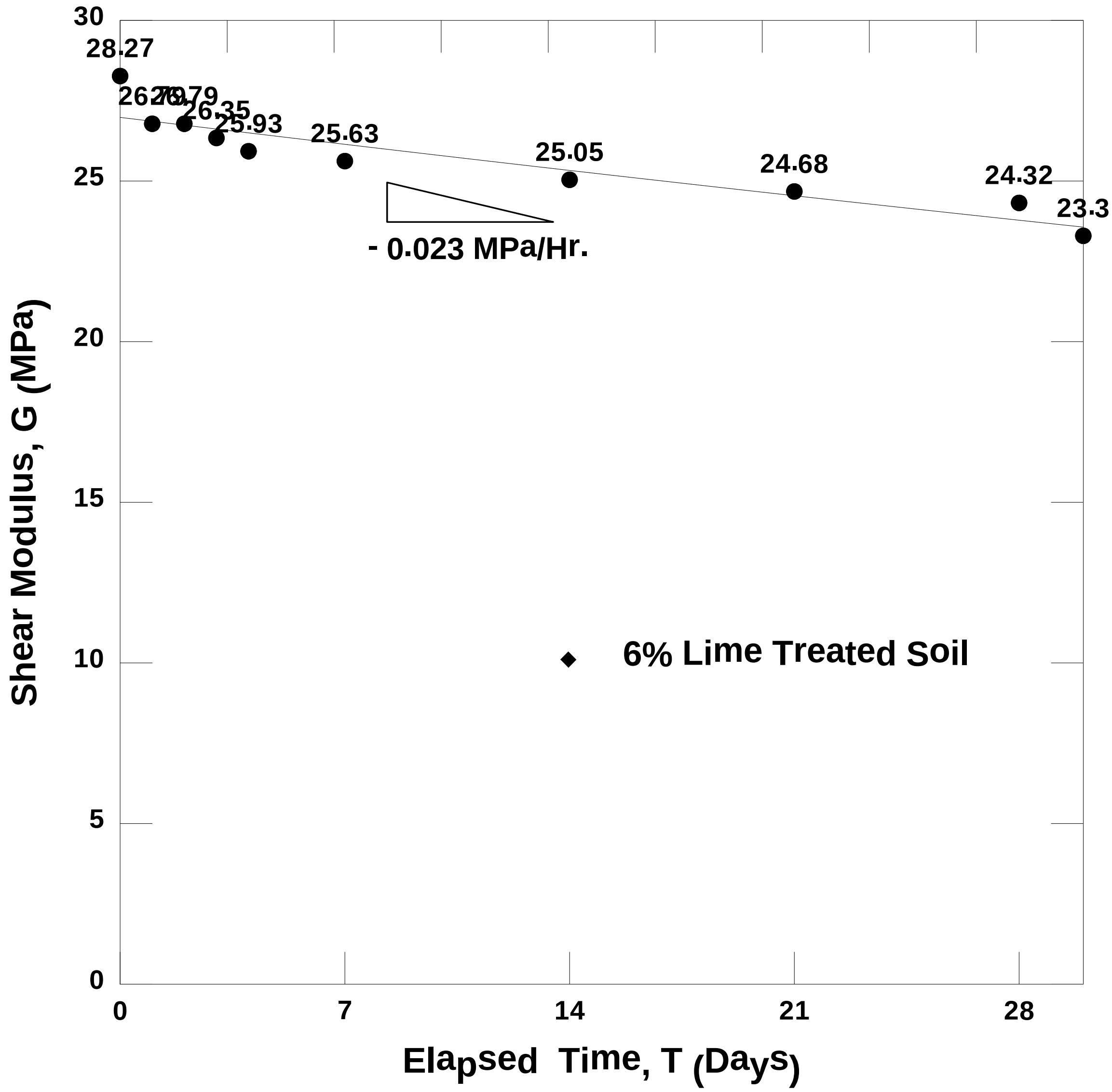


Figure 38 Shear Modulus Variation with Time (Field Section)

Figure 39 Damaged BM Sensor Cables

From Figure 38, it can be seen that the shear modulus of 6% lime- treated soil reduced with time. The highest shear modulus recorded was 28.3 MPa at 0 days, and the same shear modulus was reduced to 23.3 MPa after 30 days elapsed timer period. The threshold stiffness loss in this case was calculated as 0.023 MPa/Hr, which is close to the laboratory evaluated lime treated samples. Additional data and more field studies would have given more insights into the performance of the sensor. The shear modulus reduction could have been higher if there had been a rainfall event, but no rainfall event was recorded during the monitoring period. Additional moisture provided by the rainfall would have entered into the weak subgrade section and worsened the situation. Overall, the BE sensor development showed that the sensor can be succesfully utilized for quick sulfate heave assessments in the field or in laboratory conditions.

6 COMMERCIALIZATION

Both the researcher and the consultant are currently exploring various opportunities to further modify the sensor and then they will try to commercialize it by presenting its abilities to various sensor companies. Currently, the PI is in contact with the UTA Research Commercialization Enterprise on the marketability of the present sensor. These discussions are in the preliminary stage; however, it is anticipated that a patent application may be filed in the coming months if the market analysis by the UTA research commercialization show promising results.

7 SUMMARY AND CONCLUSIONS

In this research study, sulfate-bearing soils from the states of Texas and Oklahoma were treated with cement and lime additives and embedded with a hybrid BM sensor with TDR probes. Continuous monitoring of stiffness and moisture content information was collected and analyzed to assess the rate of changes of small strain shear modulus degradation. Following the laboratory testing on three different soils, field validation study at one site was conducted by embedding the hybrid BM sensor in a treated subgrade section. The following conclusions are drawn from these laboratory and field studies:

1. Measurement of shear modulus in treated sulfate-bearing soils is an important indicator of on-going sulfate heave reactions and subsequent material degradation.
2. At low sulfate contents, Riverside soil showed shear modulus enhancements upon lime and cement treatment.
3. Cement-treated soils showed higher initial shear modulus values compared to the lime-treated ones. Higher loss of stiffness was observed in cement-treated soils compared to the lime-treated ones.
4. Threshold stiffness losses for low lime dosages was 0.005 MPa/Hr, and 0.010 MPa/Hr. for high lime dosages. For cement-treated soils, threshold stiffness loss was 0.015 and 0.04 MPa/Hr. for low and high dosages. Threshold loss of shear modulus was higher for cement-treated soils compared to the lime-treated soils.
5. The BM sensor worked successfully in measuring the stiffness changes, which in turn showed that the indicate soil swelling with the sample as any swell in soils could result in loss of stiffness values. The hybrid BM sensor needs to be protected from heavy moving loads for accurate determination of soil properties.

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