STIFFNESS RESPONSE OF CHEMICALLY STABILIZED

SULFATE RICH SOIL VIA RESONANT COLUMN TESTING

by

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Abstract

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Stabilization of expansive soils using lime and cement additives have been used by practitioners over the last few years. However, recent heaving and premature pavement failures in lime and cement-treated subgrades containing sulfates led to questioning the validity of calcium based stabilization. When expansive soils containing sulfates are treated with calcium-based stabilizers, the calcium from the stabilizer reacts with soil sulfates and alumina to form the expansive mineral Ettringite.

A series of resonant column (RC) tests (ASTM D 4015-92) was conducted on several chemically stabilized specimens of high-plasticity, sulfate-rich expansive clay from Sherman, Texas. Test results were used to assess the influence of mellowing period on the stiffness properties of stabilized soil. Specimens were tested for different stabilizer, mellowing periods, curing times, and confining pressures, and elapsed times under constant confinement.

Two stabilizers, 6% lime and 4% lime + 8% fly ash, were used. Soil stiffness parameters investigated include small-strain shear modulus (G) and small-strain material damping ratio (D). Tests were also conducted at mid- to high-strain levels to study stiffness degradation effects of torsional shearing. 4% lime + 8% fly ash treatment method shows the best results.

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Chapter 1

Introduction

1.1 Background and Importance

The definition of soil varies depending on the person considering it. To a miner, it is just some worthless material that is in the way and must be removed, but to a civil engineer planning a construction site, soil is whatever unconsolidated material happens to be found at the surface. There are three major types soils, which are sand, clay and silt, each one has unique characteristics like color, texture, structure, and mineral content, and each soil comes with different problems.

In engineering construction, the problems with soil always occur even during construction or after construction. This happen as the soil cannot reach the required specification such as the bearing capacity of soil is too weak to support superstructure above it. The existing soil at a construction site are not always be totally suitable for supporting structures such as buildings, bridges, highways, and dams. Hence, if a structure constructed on poor soil, many problems will occur after the construction completed.

One of the major problems is settlement. Settlement usually occurs by the movement of the soil which caused by the surcharge or change of ground water table. Preventing settlement problems begins with the recognition of the soil of a foundation rested, recognize the differences among soil types, determine the solution for the soils that respond to building loads and identify potential problems.

In the past, once the bearing capacity of the soil was poor, there were only three options, change the design to suit site condition, remove and replace the in situ soil, or abandon the site. Abandoned sites due to undesirable soil bearing capacities dramatically increased, and the outcome of this was the scarcity of land and increased demand for natural resources. Affected areas include those which were susceptible to liquefaction and those covered with soft clay and organic soils. Other areas were those in a landslide and contaminated land. However, in most geotechnical projects, it is not possible to obtain a construction site that will meet the design requirements without ground modification. The current practice is to modify the engineering properties of the native problematic soils to meet the design specifications. Nowadays, soils such as soft clays and organic soils can be improved to the civil engineering requirements by soil stabilization.

Soil stabilization aims at improving soil strength and increasing resistance to softening by water through bonding the soil particles together, water proofing the particles or combination of the two. Usually, the technology provides an alternative provision structural solution to a practical problem. The simplest stabilization processes are compaction and drainage (if water drains out of wet soil it becomes stronger). The other process is by improving gradation of particle size and further improvement can be achieved by adding binders to the weak soils. Soil stabilization can be accomplished by several methods. All these methods fall into two broad categories namely; mechanical stabilization, chemical stabilization.

However, during the past few decades a number of cases have been reported where sulfate rich soils stabilized by cement or lime underwent a significant amount of heave leading to pavement failure. Therefore, the main objective of this work is to study the dynamic properties of chemically stabilized soils.

1.2 Objectives

The main objective of this work is to study the stiffness response of chemically stabilized, sulfate rich expansive clay from Sherman, TX using two selected stabilizers, lime and fly ash. In order to achieve this goal, a series of a free-fixed type of resonant column tests (Isenhower 1979; Huoo-Ni 1987; Stokoe et al. 1991; ASTM D 4015-92 1993) was conducted at small shear strain amplitude levels (< 0.0001%) for different stabilizer type, different mellowing period, different curing time, and different confining pressure. Stiffness properties investigated include linear shear modulus G_{max} and material damping ratio D_{min}. The half-power bandwidth method was used to determine material damping D_{min} (Richart et al. 1970). Tests were also conducted at small- to mid-shear strain amplitude levels (0.0001-0.01%) to assess the threshold strain limit γ_{th} for each treatment method and study the effects of torsional shearing on the rate of degradation of normalized modulus G/G_{max} of treated soil.

1.3 Organization

The thesis is organized into six chapters. Chapter 1 is an introduction that has given an overview of the thesis and states the main objectives of this work.

In Chapter 2, fundamental concepts of soil stabilization are presented by reviewing previous studies regarding the subject topic.

In Chapter 3, fundamentals of resonant column (RC) testing technique are presented. This chapter also gives an overview of the test devices used to accomplish the experimental program.

In Chapter 4, a brief description of the basic properties of the test soils and the test methods used to accomplish this thesis are presented. This chapter also summarizes the experimental variables and specimen preparation methods.

In Chapter 5, the experimental program and analysis of test results are presented. The chapter includes all test results and data plots, providing a summary and thorough analysis of all resonant column test results.

Chapter 6 summarizes the main conclusions from this thesis work and provides some recommendations for future studies.

Chapter 2

Literature Review

2.1 Introduction

The engineering behavior of expansive soils has long been investigated by a considerable number of researchers using a variety of laboratory and field testing techniques. The literature survey presented herein includes typical behavior and engineering properties of sulfate-rich expansive clays commonly encountered in northeast Texas, along with some case studies related to soil heaving in the region. This review also focuses on the various chemical stabilization methods currently used and under investigation.

2.2 Fundamentals of Soil Stabilization

Soils can be stabilized by the addition of a small percentage of cement or lime. Such stabilization processes enhance many of the engineering properties of the treated soils and produce an improved construction material. Its overall benefits include an increase in soil strength, stiffness and durability, and a reduction in soil plasticity and swelling/shrinkage potential (Hausmann 1990; Sherwood 1995; Prusniski and Bhattacharya 1999). The concept of stabilization can be dated back to 5000 years ago. McDowell (1959) mentioned that stabilized earth roads were used in ancient Mesopotamia and Egypt, and that the Greeks and Romans once used soil-lime mixtures. The first tests involving soil stabilization were carried out in the United States in 1904 (Clare and Cruchley 1957). Cement was first used as a stabilizing agent of soil when a street in Sarasota, FL, was constructed in 1915 (ACI 1997), and lime was first used in modern construction practice in 1924 on short stretches of highway (McCaustland 1925) with the expansion of roads to cater for the growth of vehicle traffic in the 1930s.

However, during the past few decades, a number of cases have been reported where pavement bases stabilized by cement or lime underwent a significant amount of heave leading to pavement failure (Mitchell 1986; Hunter 1988; Little et al. 1989, Perrin 1992; Kota et al. 1996; Ksaibati et al. 1999; Rollings et al. 1999). The literature also indicated that heave is occurs due to the formation of a highly expansive crystalline mineral, namely, Ettringite ($Ca_6.[Al(OH)_6]_2.(SO4)_3.26H_2O$), a weak and unstable sulfate mineral, that undergoes significant expansion when exposed to hydration, which results in differential heaving and distress-induced cracking of pavements and spread footings. This is the product of sulfate attack on stabilized soils. Sulfate attack of conventional Portland cement concrete is a widely recognized phenomenon, and appropriate methods of protection against sulfate attack have been established as a function of the sulfate exposure level (ACI 1982; DePuy 1994), but the actual chemical reactions and products involved have not been completely understood.

Similarly, the potential for sulfate attack on both cement and lime stabilized soils was established in the 1960s (Sherwood 1962). However, only limited research work has been carried out on this subject and little constructive guidance on how to deal with the problem is available. Similar to cement concrete, the PH value, moisture availability, temperature, sulfate levels, and clay mineralogy may all affect sulfate attack of cement-stabilized soils. These factors should therefore be determined when stabilized soils are susceptible to sulfate attack.

Several roads, airfield pavement, and parking lots in Texas have also suffered severe pavement damage due to expansive minerals formed from the reactions of calcium-based materials used to stabilize sulfate-bearing soils. Perrin (1992) summarized the findings of investigations on three projects, where these reactions caused considerable damages. On these projects, heaves caused linear ridges or bumps as much as 300mm high in both the transverse and longitudinal directions on the roads or parking lots. Generally, the damage appeared to be most severe in areas of poor drainage. Shear planes observed at the damaged areas confirmed that the heaving was caused by horizontal expansion resulting in buckling and not concentrated vertical heaves. Soluble sulfate was traced from mixing water or gypsum seams near the surface.

2.3 Sulfate Heave Case Histories

2.3.1 Joe Pool Dam, Texas

During 1988 and 1989, several park roads in Joe Pool Lake experienced severe heaving problems (Perrin, 1992). The pavement sections contained 150mm thick subgrade layers stabilized with 5-6% lime. Soils in this area were lean clays and clayey sands, with less than three percent clay swelling clay mineral. The extent of heaving is depicted in Figure 2-1. These soils belonged to the Eagle Ford Shale formation. Though the soils contained barely detectable sulfate contents, the lime-treated base materials contained 2,000-9,000 ppm sulfate. Though the reason for the increase in sulfate content was not known, it was hypothesized that sulfates have migrated from the surrounding soil through continuous supply of fresh water. Upon mineralogical investigation, it was found that Ettringite and Thaumasite were the cause of the heaving. The roadway was re-compacted, but the issue continued. Finally, the entire lime treated layer was replaced with gravel base and non- expansive fill, and no issues of heaving were observed.

Figure 2-1 heaving in lime treated subgrade (Reproduced from Talluri, 2013)

2.3.2 Sulfate heave issues at DFW airport, Texas

One of the taxiway sections built at the Dallas/Fort Worth international Airport (DFW) showed signs of sulfate induced heave. Localized areas of heave distress were observed along both shoulder of the taxiway section. The shoulders were constructed of flexible asphalt pavement, which rest open a lime treated base soil. The main taxiway section did not show any signs of heaving since it was built with the rigid reinforced concrete pavement overlaying a four to twelve inch lime-treated base course. The natural subgrade soil was shale clay with sandy seams and occasional Gypsum deposits. The base course consists of the native subgrade soil stabilized with lime.

The shoulder exhibited pavement cracking associated with heave distress. At several locations, the amount of heave ranged from 5cm to a high as 30cm. this heave pattern was irregular and sometimes affected small localized areas of one to two feet in diameter. Other heave related cracks in the asphalt pavement appeared near the junction between rigid concrete and asphalt concrete sections. Significant lateral movement of pavement edge had also occurred at certain locations. On the other hand, the rigid pavement was in good condition, with few minor shrinkage cracks at very few locations.

Reasons for the observed distress were location of drainage ditches near the shoulders and topography of the site which might have contributed to the increase in the heaving of lime treated soils. The heavy rain fall occurred in the last six months of 1996 and early 1997 may have raised the water levels under the pavement sections. This moisture may have contributed to the hydration necessary for the formation of Ettringite and Thaumasite compounds. These compounds, went further hydrated, may have resulted in the heaving of the flexible pavement section. The west shoulder exhibit more damage than the east shoulder. Part of this may be attributed to the water pooling near the base of the embankment which is located next to the west shoulder. Another factor for the less distress on the east shoulder could be attributed to the better draining of the east shoulder section due to particular topographical features.

Figure 2-2 heave distress pattern on west shoulder of taxiway

(Reproduced from Talluri, 2013)

2.4 Stabilization of High Sulfate Soils

If the total level of soluble sulfates is below 0.3%, or 3,000 parts per million (ppm), by weight of soil, then lime stabilization should not be significant concern. The potential for a harmful reaction is low. However, good mix design and construction practices should be followed as usual. If soluble sulfates are detectable at all, lime slurry should be used, if possible, instead of dry lime and adequate water (optimum for compaction plus at least 3%) should be used for mixing (Talluri, 2013). Total soluble sulfate levels of between 0.3% (3,000 ppm) and 0.5% (5,000 ppm) are of moderate concern. Generally, these sulfate levels do not result in harmful disruption, but on occasions have caused localized distress. Localized distress is often due to seams of higher sulfate concentration not detected in testing. The potential for some localized distress is a “fact of life” with sulfate levels in this range. When encountering sulfate levels in the range of 0.3% to 0.5%, it is important to follow good mix design and good construction techniques explicitly. Special attention must be given to the usage excess water during mixing, mellowing and curing. Mixing water should be at least 3% to 5% above optimum for compaction. Lime slurry should be used instead of dry quicklime or hydrated lime. The mellowing period should typically be at least 72-hours, but may need to be longer depending upon experience.

Total soluble sulfate levels between 0.5% (5,000 ppm) and 0.8% (8,000 ppm) represent moderate to high risk. These soils can be successfully treated but require very close attention to construction technique. Generally, the same mix design and construction guidelines as described for soils containing sulfate levels between 0.3% and 0.5% should be followed. However, before treating these soils with lime, laboratory testing to determine swell potential is recommended. This testing will not only establish the approximate amount of swell but also will help establish the required period of mellowing between mixing and compaction. Total soluble sulfate levels of greater than 0.8% (8,000ppm) are generally high risk to stabilize with lime. In certain situation, such soils have been successfully treated. However, the risk is generally too high for routine works.

Treatment of such high sulfate soils require lime slurry, mixing, mellowing, curing, water content of 3% to 5% above optimum for compaction and may also require an extended mellowing period of longer than 72-hours. The required mellowing period may be as long as 7-days during which monitoring of density is recommended. Double application technique may be effective in successfully treating high sulfate soil.

Sulfate with total soluble sulfate content greater than 1.0% (10,000ppm) generally are not suitable for lime stabilization. However, such concentration often exists as seams on a project as opposed to being evenly districted throughout a site. A Limited amount of work (Ferris 1991, Wild et al. 1996) has been done to investigate ways to control sulfate attack on stabilized soil. Most often these methods are costly. Moreover, if they are not properly evaluated before use in the field, the damage can occur and may be severe. In general, stabilization is cost effective on the basis of initial construction costs. However, if the pavement survives only for a short period, its cost can greatly exceed non stabilized methods.

The concept of double application of lime (Ferris 1991) is based on the assumption that the first application of lime allows formation and subsequent expansion of Ettringite, whereas the second applications helps to complete the pozzolanic reaction and the formation of cementing agents and to bind the soil particles and often increases the strength. The delay period between the first and second treatments is vital to this technique. If sulfates in the soil are only partially soluble during the double application of lime, a low-sulfate form of calcium-sulfate-aluminate-hydrate may form. However, upon release of a high level of sulfates, such as from a subsequent rain, the low sulfate form may transform sulfate form of calcium sulfate aluminates hydrate, which can produce substantial later expansion (Little et al. 1992). If sulfate are present in the soil, they may increase sulfate concentrations due to oxidation. A double application of lime may be effective if the natural soil has a low level of soluble sulfates and the soil does not have sulfate minerals. However, a double application of lime will be effective only if it is used with sufficiently long delay periods between the two applications, with more than an optimum amount of water in the first application, and when low percentages of lime are involved.

2.5 Summary

Currently used chemical stabilization methods (i.e., lime, Portland “Type I/II” cement, and moderate-to-high calcium “Class C” fly ash) are not recommended for sulfate-rich environments. The use of these conventional methods on high plasticity, sulfate-rich expansive clays, as those predominant in the Dallas/Fort Worth Metroplex area, may lead to the formation of Ettringite mineral and, therefore, sulfate-induced heaving.

Recently, novel stabilization methods, such as sulfate resistant “Type V" cement, low calcium “Class F" fly ash, and lime mixed with polypropylene fibers, have been explored. These novel methods increase strength and decreases welling/shrinkage potential of soils treated at optimum moisture content conditions, generally after curing period. In the present research study, the influence of mellowing periods on stiffness properties of treated soils was investigated by conducting a series of resonant column (RC) tests at different compaction moisture contents.

Chapter 3 describes the fundamentals of the resonant column (RC) device and testing technique.

Chapter 3

Fundamentals of Resonant Column (RC) Testing Technique

3.1 Introduction

The resonant column (RC) testing technique was first used to study dynamic properties of rock materials in the early 1930s, and has been continuously evolving since then for the dynamic characterization of a wide variety of geologic materials (Huoo-Ni, 1987). During the late 1970s, Professor Stokoe and his co-workers at the University of Texas at Austin developed a new version of the fixed-free torsional shear/resonant column (TS/RC) device suitable for testing solid or hollow specimens with shearing strain amplitudes up to 0.4% (Stokoe et al., 1978).

The device originally developed at UT-Austin is known as the Stokoe torsional shear/resonant column device (TS/RC), and has been continuously refined in the last two decades. The Stokoe TS/RC testing method has been standardized by the American Society for Testing and Materials (ASTM D 4015-92), and is one of the most reliable, efficient, and pragmatic laboratory test methods used nowadays for testing shear modulus (G) and material damping (D) of soils.

In the present work, only the resonant column (RC) capability of the Stokoe TS/RC device has been utilized, since the purpose was to study dynamic response of chemically stabilized sulfate-rich clay at low-to-mid shearing strain amplitudes. The following sections describe the fundamentals of the RC testing technique, main components of the RC device, step-by-step assembling process, and the typical soil stiffness parameters obtained from a RC test.

3.2 Apparatus Components and Assembly

After compaction and curing the specimen was placed on the Base Pedestal. The top surface of the pedestal is extremely roughed to avoid slippage between the soil specimen and the pedestal during torsional vibration. The specimen was covered with Latex membrane and tied with O-ring to avoid linkage. Figure 3-1 shows placement of specimen on the base pedestal of the apparatus.

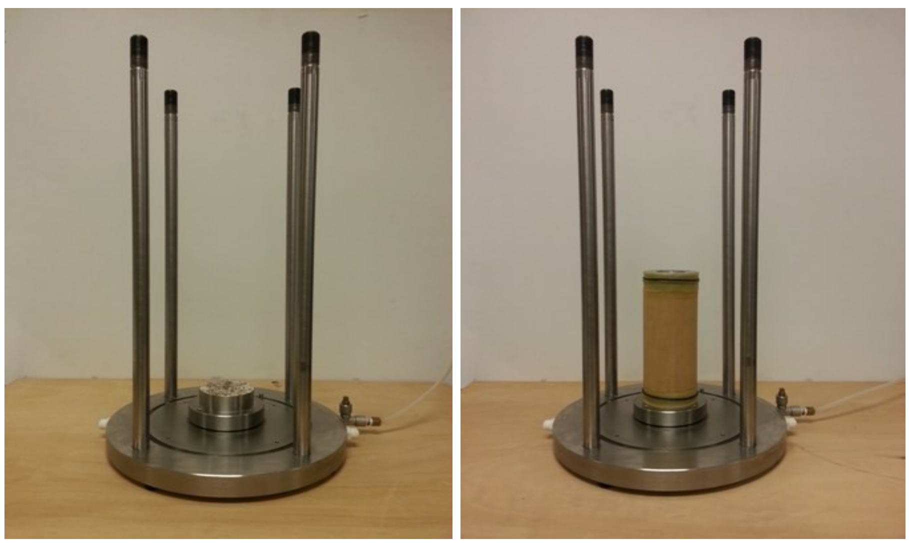


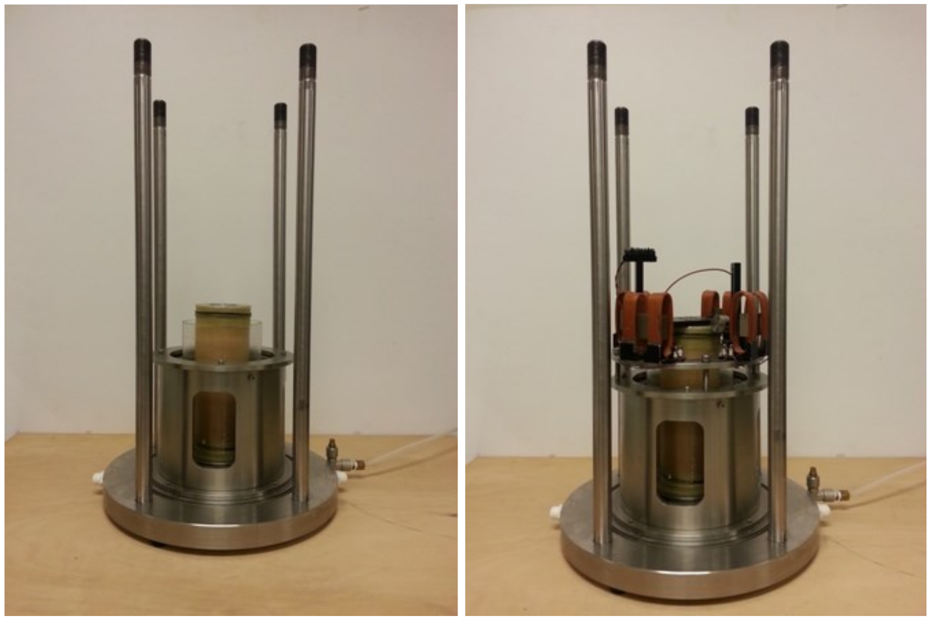
Figure 3-1 (a) base pedestal (b) specimen placed on base pedestal

After the specimen rested on the base pedestal the fluid bath was applied. An inner water-bath acrylic cylinder is placed over the soil specimen and securely fitted into the slip O-ring of the base pedestal until it makes full contact with the base plate. The water bath was applied between acrylic cylinder and the specimen. The fluid bath is helps to distribute the confining pressure around the specimen. Figure 3-2 shows installation of acrylic cylinder and application of water bath.

Figure 3-2 application of water bath between acrylic cylinder and soil specimen

Over the acrylic cylinder, stainless steel cylindrical cage placed which is firmly attached to the base plate. On the top of the cage the torsional drive mechanism was fitted using screws. The placement of the driver coils needs caution, so that each magnet is encircled by a pair of coils without contact. Figure 3-3 shows placement of stainless steel cage and driver.

The driver is the main part of the apparatus, and it is used to apply torque to the top of the specimen. The top cap has a rough surface on the side making contact with the specimen to insure that no slippage occurs between the specimen and the driver during torsional excitation. The driver is a four armed plate with the magnets attached to the end of each arm, and the eight drive coils encircle the ends of the four magnets as shown in Figure 3-4 each coil is elliptically shaped so that the magnets can descend inside the coils.



(a) (b)

Figure 3-3 (a) stainless steel cylindrical cage attached to the base pedestal

(b) torsional driver mechanism placed on cylindrical cage

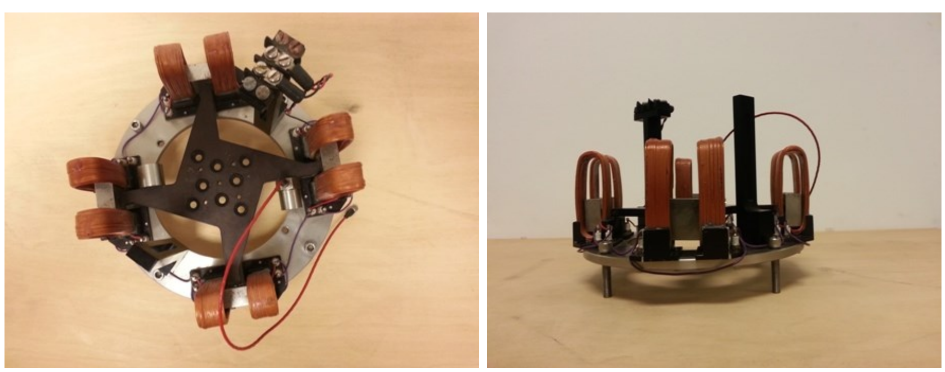


Figure 3-4 torsional drive mechanism (Driver)

After the RC apparatus fully assembled and connected to torsional monitoring system, and the electrical wiring of the SR785 dynamic signal analyzer and the 4102M charge amplifier box is then connected to the corresponding microdot connectors on the outer side of the thin wall cylinder, that is, the input signal coaxial wire and the accelerometer input wire. The analyzer is then configured at the desired test settings, including amplitude of sinusoidal signal, range of frequency scale, swept-sine testing mode, and number of data points to be recorded.

Figure 3-5 fully assembled confining chamber

The torsional motion monitoring system is used to capture the frequency response of the soil column during RC testing, and it includes an accelerometer rigidly attached to one of the arms of the spider, and an associated counterweight installed on the opposite side of the four-armed spider. The voltage response of the accelerometer is sent to a charge amplifier and then recorded by a dynamic signal analyzer.

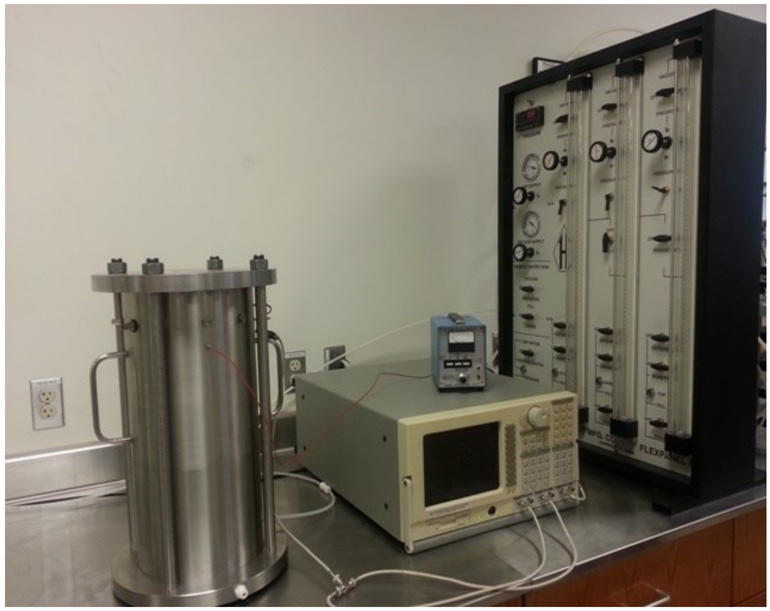
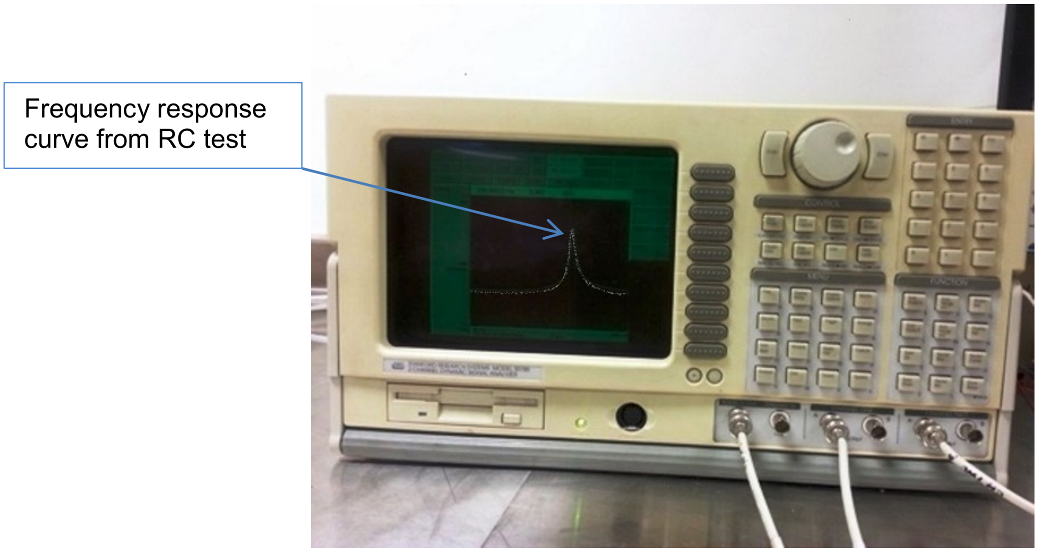


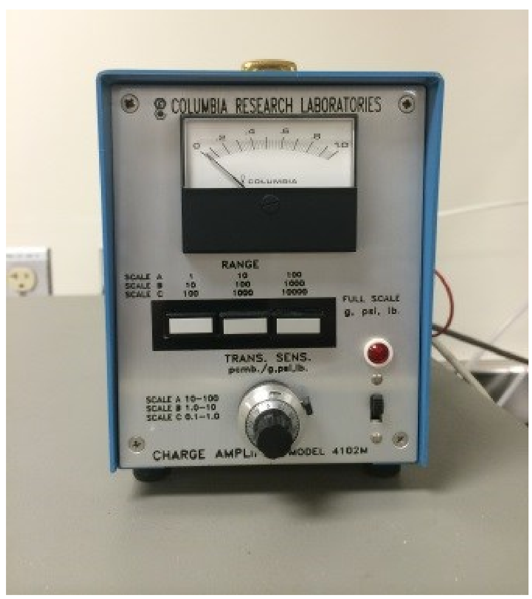
Figure 3-6 fully assembled resonant column test setup

The frequency response measurement system used in this work includes a dynamic signal analyzer, a charge amplifier box. The analyzer is a dual-channel SR785 model dynamic signal analyzer acquired from Stanford Research Systems, Inc. The amplifier is a 4102M-model charge amplifier box acquired from Columbia Research Laboratories, Inc. Photographs of analyzer and charge amplifier box (resting on top of the analyzer) are shown in Figure 3-9.

From the dynamic signal analyzer, a constant-amplitude sinusoidal current is sent to the driver fixed on top of the soil column (figure 3.7). The sinusoidal current travels along a coaxial cable that transmits the signal, via microdot connectors on the thin wall of the confining chamber, to the driver's input current connection. The signal is distributed among the drive coils of the driver system inducing a sinusoidal torsional excitation on the specimen via the reacting magnets of the spider.



(a)



(b)

Figure 3-7 (a) Dynamic Signal Analyzer (DSA) displaying frequency response curve

(b) a charge amplifier box

The amplitude of vibration is captured by the accelerometer rigidly attached to one of the arms of the spider, and sent to the charge amplifier box in the form of output voltage response. The amplified signal from the charge amplifier is sent back to the dynamic signal analyzer. A frequency response curve is then obtained by sweeping the entire preset frequency scale in the analyzer, and it can be displayed on the screen of the SR785 analyzer (Figure 3-7 (a)).

The SR785 analyzer allows for storage and graphic display of the captured data in a PC-based computer terminal. A photograph of the dynamic analyzer and charge amplifier interacting with the RC device is shown in Figure 3-7.

After all RC components were assembled. An initial isotropic confining pressure σ' = 2.5psi was applied, and a series of RC tests were performed at 0 and 24 h elapsed time from moment when the initial pressure was applied. The pressure was kept constant during these first 24h. All RC tests were performed by sending a low-amplitude (250mV peak-to-peak) sinusoidal signal from a dynamic signal analyzer (DSA) to the torsional driver fixed on top of the specimen. The frequency of the signal was incrementally changed by sweeping the frequency scale in the DSA until the frequency response curve was obtained (Figure 3-8), which allows for the determination of linear shear modulus G_{max} and damping ratio $D_{min.}$

3.3 Calculation of Shear Modulus (G)

Typical frequency response curve obtained in this research work is shown in Figure 3.8. The resonant frequency (f_r), corresponding to the peak of the curve, is then evaluated. Typical values of resonant frequency for soil specimens range from 6 to 150 Hz (Stokoe and Huoo Ni, 1985). Stiffness soil properties such as G_{max} and D_{min} are then determined from f_r and the frequency response curve.

After the resonant column test the result converted to text file, and imported to excel data to plot the required graphs. First the dynamic frequency response curve was plotted, and the maximum resonant frequency obtained. The shear modulus of the soil calculated using the following formula.

$$G = \rho\,(2\pi L)^2\,[\frac{fr}{Fr}]^2$$

Where: G = shear modulus

L = length of specimen

f_r = maximum resonant frequency

F_r =0.393 (constant from driver mechanism)

ρ = the total mass density of the soil

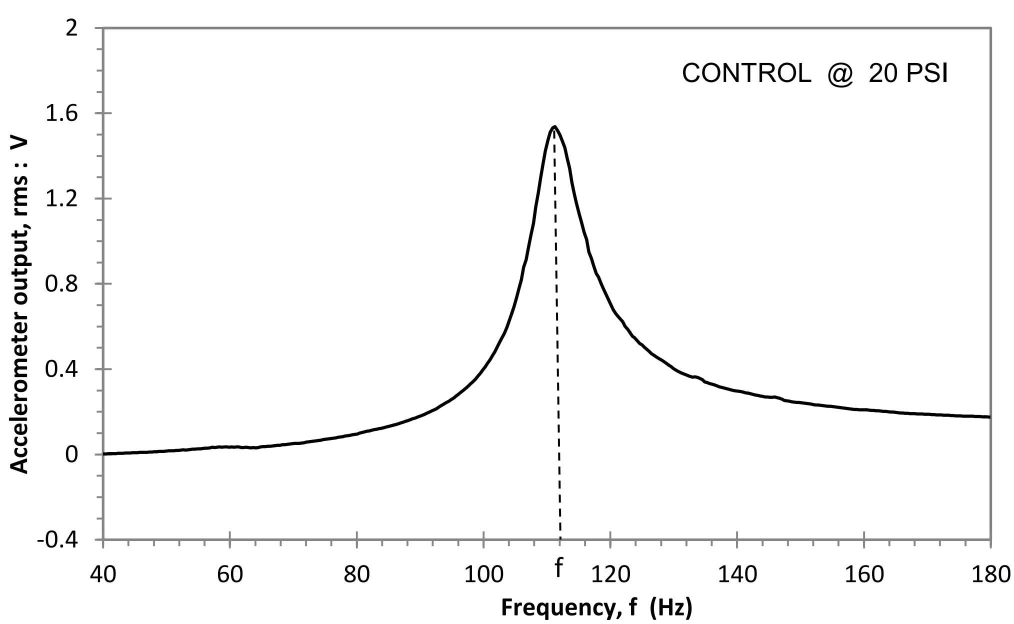


Figure 3-8 typical frequency response curve

3.4 Calculation of Shear Strain (γ)

The shearing strain (γ) at any given point within the soil column depends on the distance between this point and the center of the soil column, and it is the function of accelerometer output (the maximum output voltage), the length, and diameter of the specimen. In this work the shear strain was calculated by the following equation.

$$\gamma = \frac{X(0.707\ D)/(2\ in)}{L}$$

$$X = \frac{2\sqrt{2}}{5}\,\frac{980.44}{4\pi^2 fr^2}\,V_{rms}$$

Where:

X = accelerometer location (2 in. from center)

0.707 D = location of average shear displacement

L = length of specimen

D = diameter of specimen

f_r = resonant frequency

V_{rms} = accelerometer output

3.5 Calculation of Damping Ratio (D)

The damping ratio of the specimen was calculated using Bandwidth Method (Richard et al., 1970). To get the damping ratio, the half power point (0.707Arms) was calculated and located on the graph, and the corresponding frequency values (f1 and f2) were read from frequency response curve then the damping ratio calculated using the following equation.

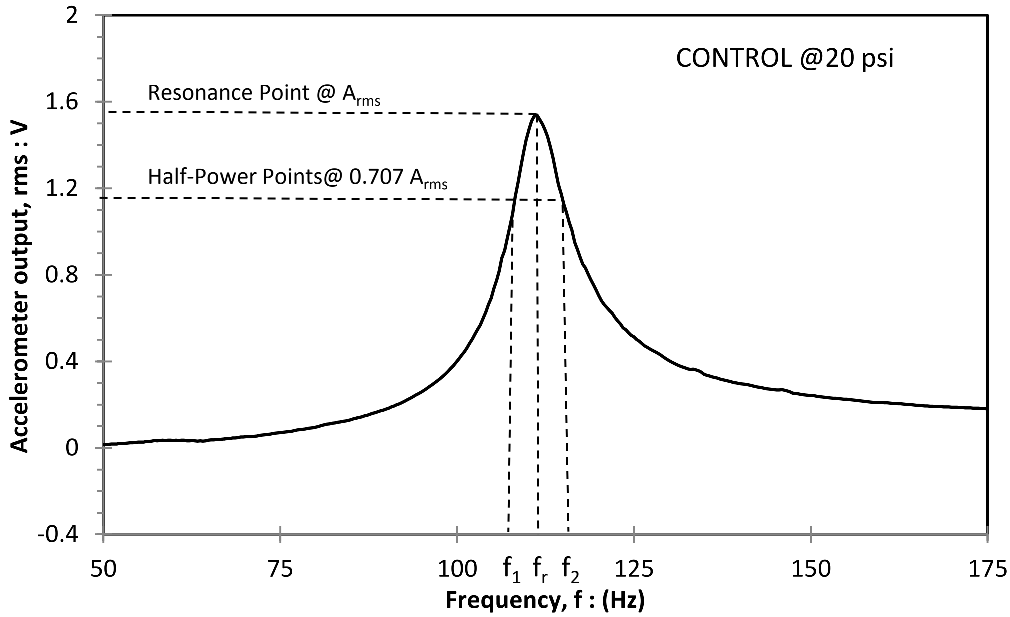


Figure 3-9 bandwidth method determination of damping ratio (D)

$$D = \frac{1}{2}\left(\frac{\mathrm{f2} - \mathrm{f1}}{\mathrm{fr}}\right)$$

Where:

f_r = the maximum resonant frequency (Hz)

f_1 and f_2 = Half-power frequencies (Hz)

3.6 Summary

The basic components of the RC device are: confining chamber, torsional drive mechanism, and torsional motion monitoring system. A dynamic signal analyzer and a charge amplifier box form the frequency response measurement system. The Frequency response curve can be read from the dynamic signal analyzer, and the data can be converted to excel text and the required stiffness parameters can be calculated. Shear modulus (G) can be obtained from the resonant frequency (f_r) and the geometrical constants of the soil-cap-driver system. Material damping ratio (D) can be obtained via the half-power bandwidth method.

Chapter 4

Testing Soil and Experimental Variables

4.1 Introduction

The experimental program followed in this work was designed to study the influence of mellowing period and curing time on the stiffness properties of chemically stabilized sulfate-rich clays. Nine identically prepared specimens and one control soil of highly expansive, sulfate-rich clay from Sherman, Texas were treated with the two selected stabilizers described in Chapter 2, at mellowing period, and then tested in the resonant column (RC) testing device described in Chapter 3. The following sections provide the basic engineering properties of the testing soil and the physical and chemical characteristics of the chemical stabilizers soil used in this study, along with a detailed description of all experimental variables and the soil specimen preparation method for RC testing.

4.2 Testing Soil

The soil used in this investigation was sampled from FM-1417, Sherman, Texas. This soil is a high-plasticity, sulfate-rich clay, dark yellow in color, with natural moisture content (w) of 7.14%, standard Proctor optimum moisture content (w_{opt}) of 27%, specific gravity (G_S) of 2.85, liquid limit (LL) of 72 %, plasticity index (PI) of 30%, and it has soluble sulfate content of 24,000 ppm.

The soil is classified as A-7-6 and CH according to the AASHTO and USCS, respectively, and it was selected for this work because of its soluble sulfate content (24,000ppm) and high plasticity index of (30%). Soluble sulfate content and plasticity index are the most critical factors that yield to sulfate-induced heaving. The selection criteria are presented in the next section.

4.3 Soil Selection Criteria

Two criteria were considered in the selection of the test soil. The first criterion for selection of the test soil is the amount of sulfate content. Soluble sulfates were determined using the modified UTA method (Puppala et al. 2002). First, 10 grams of soil is mixed with 100 mL of distilled water and left overnight, then shaken for 30 minutes. Then soluble sulfates are extracted by centrifuging, the solution filtered through 0.1μm filter paper. The filtrate is diluted, and the pH of the dilutant is adjusted between 5 and 7. The solution is boiled until the bubbles appear (Figure 4-1(a)), then warm $BaCl_2$ (10%) solution is added. Then the precipitate is digested and filtered (Figure 4-1(b)). Finally, the difference between the weights of dry filter paper and filter paper with barium sulfate precipitate gave the sulfate content in the soil.

The results were consistent, with minimum differences. Based on the sulfate content, the soil was categorized as high sulfate content soil with sulfate content greater than 8,000 ppm.

(a) (b)

Figure 4-1 modified UTA method determination of soluble sulfate content

The second criterion in the selection of the test soil was the PI value. Liquid limit (LL), Plastic limit (PL) and Plasticity index (PI) of the soils was determined per ASTM procedure D4318-10. These tests were conducted in order to determine the plasticity properties of the soil. Upon addition of water, the state of soil proceeds from dry, semisolid, plastic and finally to liquid states. The water contents at the boundaries of these states are known as shrinkage SL, plastic PL and liquid LL limits, respectively (Lambe and Whitman, 2000). Therefore, LL is calculated as the water content at which the soil flows, and PL is determined as the water content at which the soil starts crumbling when rolled into a 1/8-inch diameter thread.

These Atterberg limits are very important to show a relationship between the shrink-swell potential of the soils and their relevant plasticity indices. The numerical difference between LL and PL values is known as plasticity index (PI), and this property is generally used to characterize the plasticity nature of the soil and its expansive potential.

The water content of the specimen during tests is measured using the microwave drying method, Figure 4-2, based on the repeatable data as reported by Hagerty et al. (1990). The Atterberg limits of the soil are summarized in Table 4-1.

Table 4-1 test soil location and properties

Soil Location	Soluble sulfates, ppm	Atterberg Limits			USCS Classification
		LL	PL	PI	
FM-1417 (Sherman,Texas)	24,000	72	30	42	CH

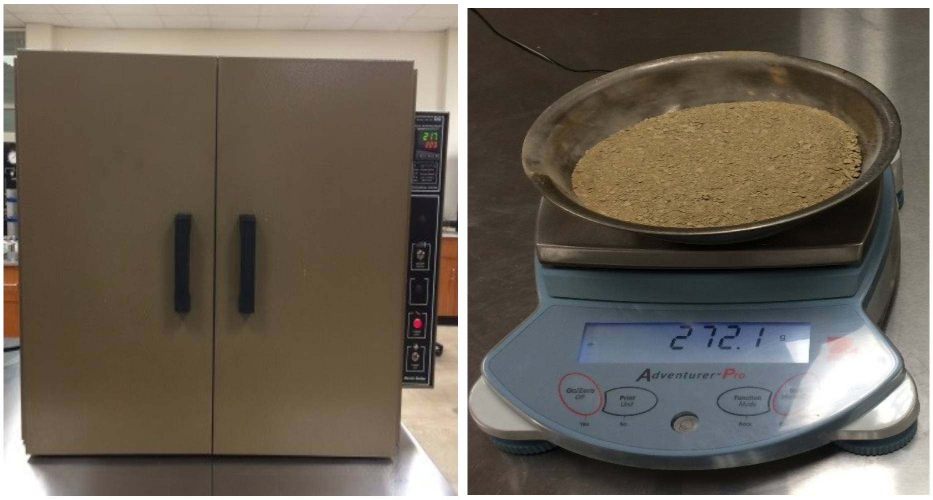


Figure 4-2 microwave drying method of measuring water content (a) soil drying in the side microwave (b) measuring of weight of dry soil

4.4 Standard Proctor Compaction Tests

In order to determine the compaction moisture content and dry unit weight relationships of the soils, it was necessary to conduct standard Proctor compaction tests. The optimum moisture content (OMC) of the soil is the water content at which the soils are compacted to a maximum dry unit weight condition. Specimens exhibiting a high compaction unit weight are best at supporting civil infrastructure since the void spaces are minimal and settlement will be less. Figure 4-3 shows the compaction curve of the control soil. Table 4-2 shows the optimum moisture content and the maximum dry density values of control and treated soil.

Table 4-2 summary of proctor test on control and treated soil from Sherman, TX

	Moisture content (%)	Dry Density (lb/ft^3)
Natural soil	27	89
6% Lime treated soil	28	87
4% Lime+ 8% fly ash treated soil	21	90

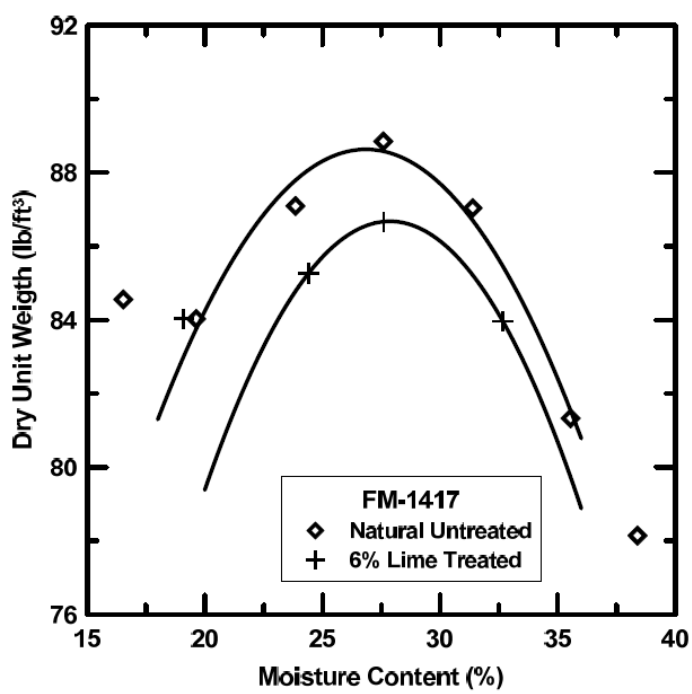


Figure 4-3 standard proctor compaction curve for control and treated soil

4.5 Chemical Stabilizers

Two stabilization methods, lime and lime + fly ash, were used in the present study. The “Pre-compaction mellowing” technique was used in stabilizing the high sulfate soils. Three and two different mellowing periods were considered in this study for lime treated soil and for lime + fly ash treated soil, respectively: 0 days, 3 days and 7 days for the lime treated soil; and 0 days and 3 days for lime + fly ash treated soil. Test soils were treated with lime and allowed to mellow in a moisture-controlled environment. Following the mellowing period, the soil were mixed thoroughly and statically compacted. Resonant column tests were then conducted on the compacted soil specimens to study the effect of mellowing period on stiffness response of chemically stabilized soil.

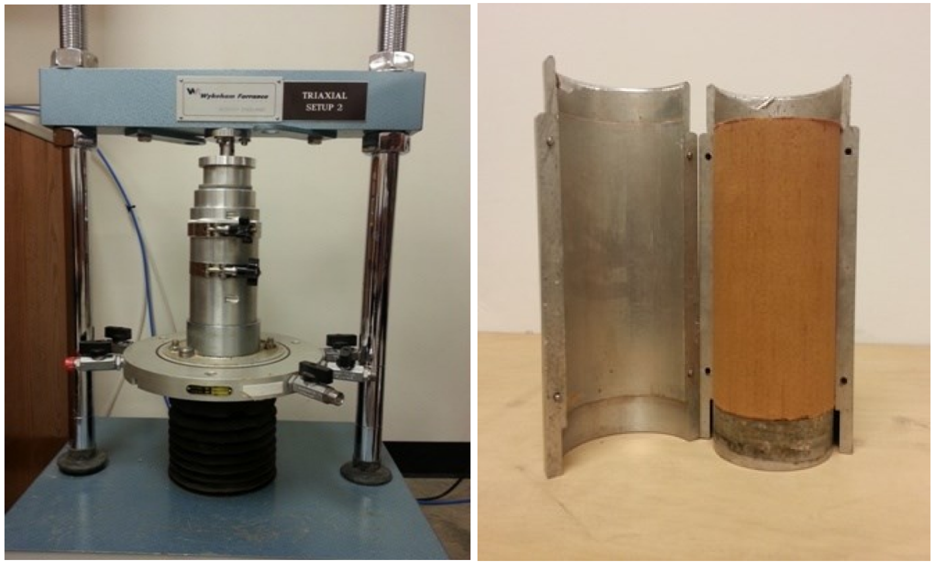
4.6 Experimental Variables

Testing variables include stabilizers, mellowing period, and curing time. The soil was treated with 6% Lime, and 4% Lime + 8% Fly ash. For the 6% Lime treated soil, three mellowing periods which are 0, 3, and 7 days were considered, and two additional 7 day mellowed soil prepared and cured for 7 and 28 days. For the 4% Lime + 8% Fly ash stabilized soil, only 0 and 3 days of mellowing were considered, and two additional 3 day mellowed specimens prepared and cured for 7 and 28 days. Including the natural soil, a total of ten test specimens were prepared.

4.7 Specimen Preparation

The amounts of water and stabilizer, by dry weight of soil, were calculated from the desired compaction moisture content (Table 4-2) prior to RC testing. Dry soil was thoroughly mixed with the required amounts of water and stabilizer until ensuring homogeneity. After the mixing process is completed, the soil is covered with a plastic bag and kept in a humidity controlled room for a desired mellowing period (Figure 4-4). After the mellowing period completed, all specimen for the testing program were statically compacted to 72 mm in diameter and 140 mm in height using a conventional triaxial loading frame (Figure 4-5 (b)). The specimen is compacted in seven lifts into a 20mm height. After compaction, the specimen was covered with a latex membrane, rested on a layer of porous stones, and allowed to soak water in plastic container for 7-day and 28-days of curing time. The curing method is chosen to prevent the water from dissolving the specimen. The specimen gets moisture from the water through the porous stones. Figure 4-6 shows a compacted specimen inside a plastic container for curing.

Figure 4-4 mixed soil covered with plastic bag for mellowing.



(a) (b)

Figure 4-5 (a) static compaction (b) compacted specimen

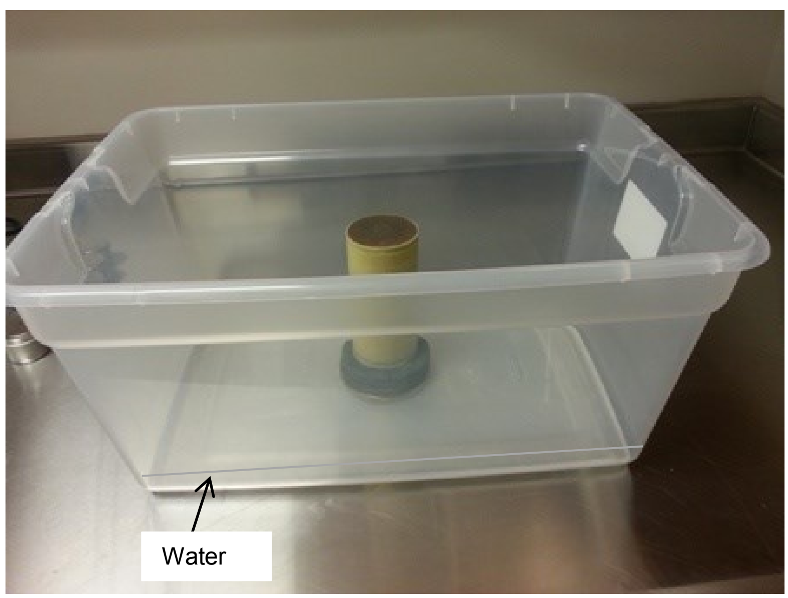


Figure 4-6 curing of compacted specimen

4.8 Summary

A high-plasticity, sulfate-rich clay from Sherman, Texas was selected for this investigation because of its high plasticity index and sulfate content, which are the critical factors that yield to sulfate-induced heaving. Two stabilization methods were selected lime and lime + fly ash. The experimental variables contemplated in this work include different stabilizers, mellowing period and curing. The amounts of water and stabilizer, by dry weight of soil, were calculated from this desired compaction moisture content. Chapter 5 describes the experimental program followed in this work and presents a comprehensive analysis of all test results.

Chapter 5

Experimental Program and Analysis of Test Results

5.1 Introduction

In this research work, a total of 150 resonant column tests were performed on 10 specimens of sulfate-rich clay combining all the experimental variables described in Chapter 4 . This chapter presents the experimental program followed in this work and a comprehensive analysis of all RC test results, including effects of most relevant test variables on soil's shear modulus (G_{max}), material damping ratio (D_{min}), and shear strain (γ).

5.2 Specimen Notation

A simple notation for specimen identification purposes was adopted in order to facilitate the reading of all variables considered in the preparation of a specific RC test specimen, particularly those variables referred to stabilizer type, mellowing period, and curing time. Table 5.1 shows all the notation symbols used in this work for identification of RC test specimens. For instance, a specimen identified as 4L 8FA 3DM 28DC implies that this is a specimen made of natural soil mixed with 4% lime + 8% fly ash (by weight), the soil is kept in a humidity controlled room in a plastic bag for 3 days of mellowing, and it was then allowed to soak water in a plastic container for 28 days for curing (Figure 4-6).

Table 5-1 testing variables and adopted specimen notation

Specimen Notation	Description
CNTROL	Control untreated soil
6L-0DM-0DC	6% Lime, 0 Days of Mellowing, and 0 Days of Curing
6L-3DM-0DC	6% Lime, 3 Days of Mellowing, and 0 Days of Curing
6L-7DM-0DC	6% Lime, 7 Days of Mellowing, and 0 Days of Curing
6L-7DM-7DC	6% Lime, 7 Days of Mellowing, and 7 Days of Curing
6L-7DM-28DC	6% Lime, 7 Days of Mellowing, and 28 Days of Curing
4L-8FA-0DM-0DC	4% Lime + 8% Fly Ash, 0 Days of Mellowing, and 0 Days of Curing
4L-8FA-3DM-0DC	4% Lime + 8% Fly Ash, 3 Days of Mellowing, and 0 Days of Curing
4L-8FA-3DM-7DC	4% Lime + 8% Fly Ash, 3 Days of Mellowing, and 7 Days of Curing
4L-8FA-3DM-28DC	4% Lime + 8% Fly Ash, 3 Days of Mellowing, and 28 Days of Curing

5.3 Experimental Program and Procedure

All 10 RC test specimens of control and stabilized soil listed in Table 5-1 were tested in the resonant column (RC) device following the procedure summarized in the following section.

Once the specimen has been fully compacted with the optimum moisture content and maximum dry density, using the compaction procedure described in Chapter 4, it was immediately covered with a latex membrane, kept in a plastic container for curing, and then assembled into the base pedestal of the RC apparatus, following the step-by-step assembling procedure described in Chapter 3; finally, all the remaining components of the RC device were set in to place. An initial isotropic confining pressure 2.5 psi was then applied to the specimen via the pressure control panel, and two RC tests were performed at 0 and 24 hours elapsed times. The pressure was kept constant during these first 24 hours.

All RC tests were performed by sending a 250 mV peak to peak sinusoidal signal from the Dynamic Signal Analyzer (DSA) to the torsional driver fixed on top of the specimen (Chapter 3). The frequency of the signal was incrementally changed by sweeping the frequency scale in the DSA until the resonant frequency (f_r) of the soil-driver system was obtained and the complete frequency response curve generated. This low amplitude signal induces a linear response in the specimen, and it allows for the determination of the low amplitude values of G_{max} and D_{min}.

Right after the last RC test was completed, the isotropic confining pressure was increased from 2.5 to 5 psi, and the same series of two RC tests were performed at 0 and 24h elapsed times. The same test procedure was repeated for isotropic confining pressures of 10 psi and 20 psi on the same specimen.

As described in Chapter 4, each specimen listed in Table 5-1 was tested at different isotropic confining pressures (σ_0) in order to assess the effect of different isotropic stress states on the stiffness properties of treated soil. The range of isotropic confining pressures considered in this work, that is, 2.5, 5, 10, and 20 psi, is aimed at closely reproducing the in situ stress states at different locations within a pavement or shallow foundation structure. Figure 5-1 shows typical frequency response curves obtained for specimen Control and 4L-8FA-3DM-28DC (Table 5-2) at different isotropic confining pressures (σ_0). Each curve was generated after 24 hours under the corresponding constant pressure. It can be noted that resonant frequency (f_r), and therefore the G_{max}, increases with isotropic confinement, a phenomenon that can be explained by the fact that soil stiffness is directly related to the shear wave velocity (V_s), which increases significantly with the confinement of the packed media (Huoo-Ni, 1987).

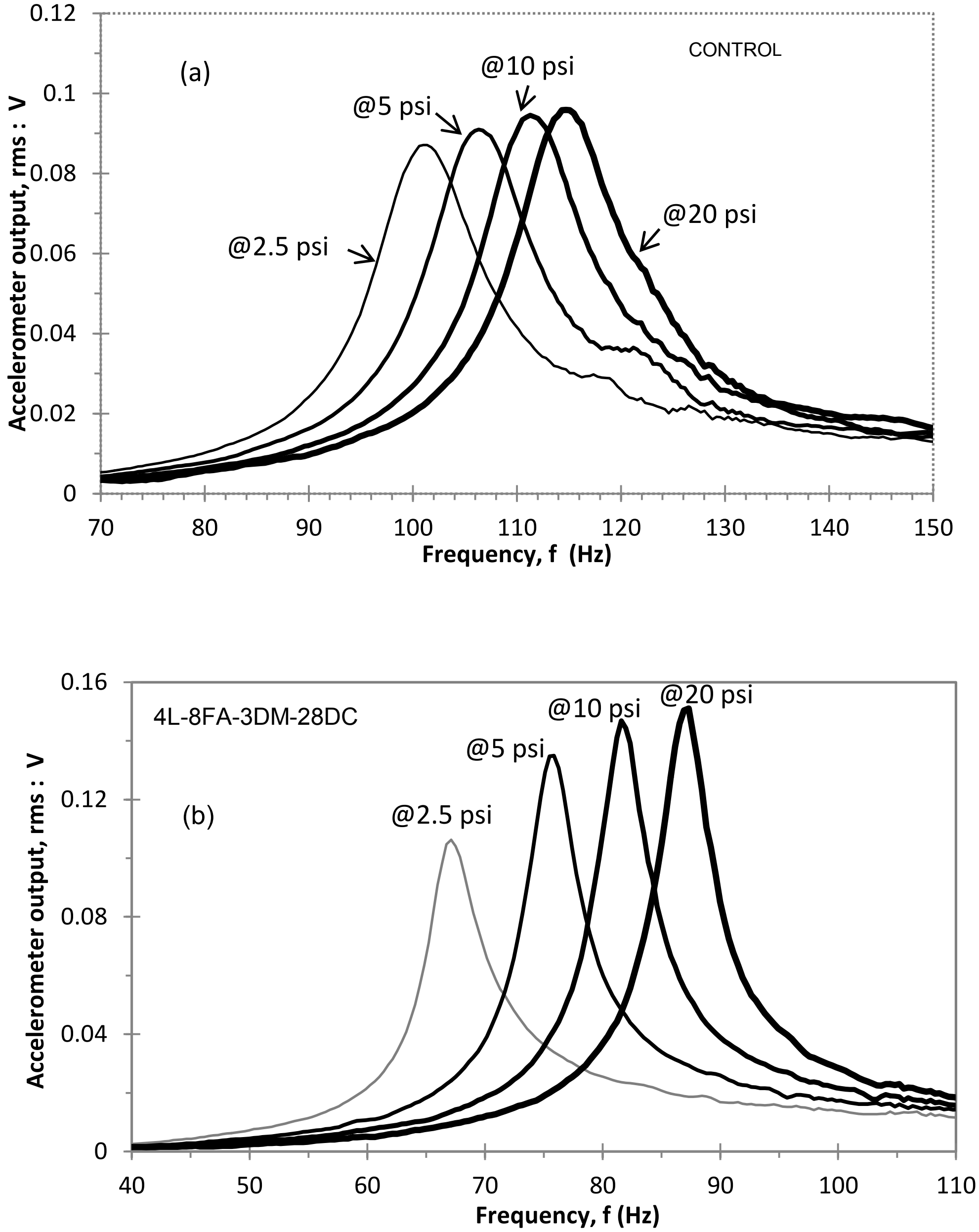
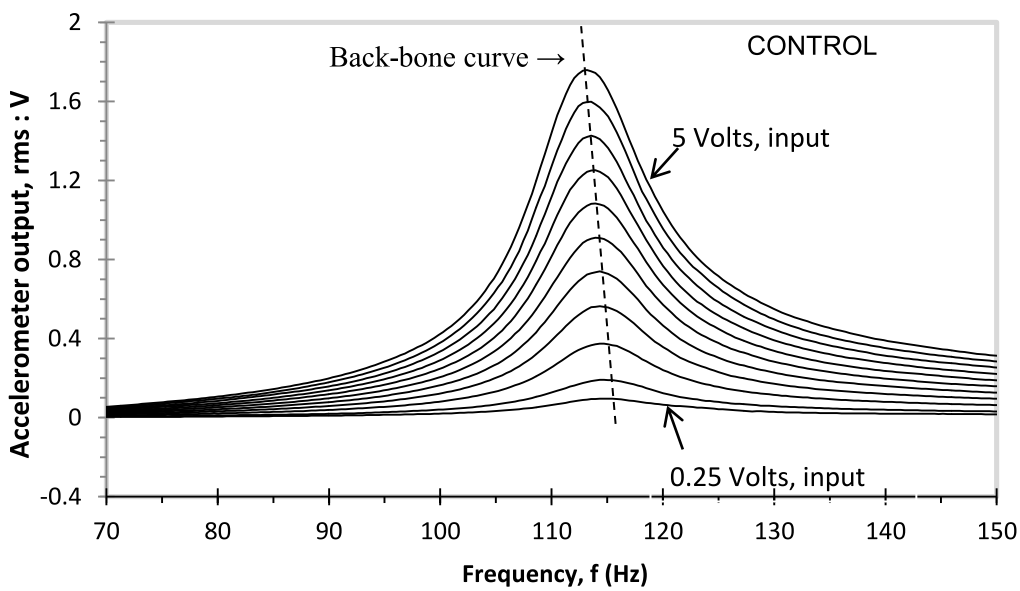


Figure 5-1 typical frequency response curves under different confinement pressure

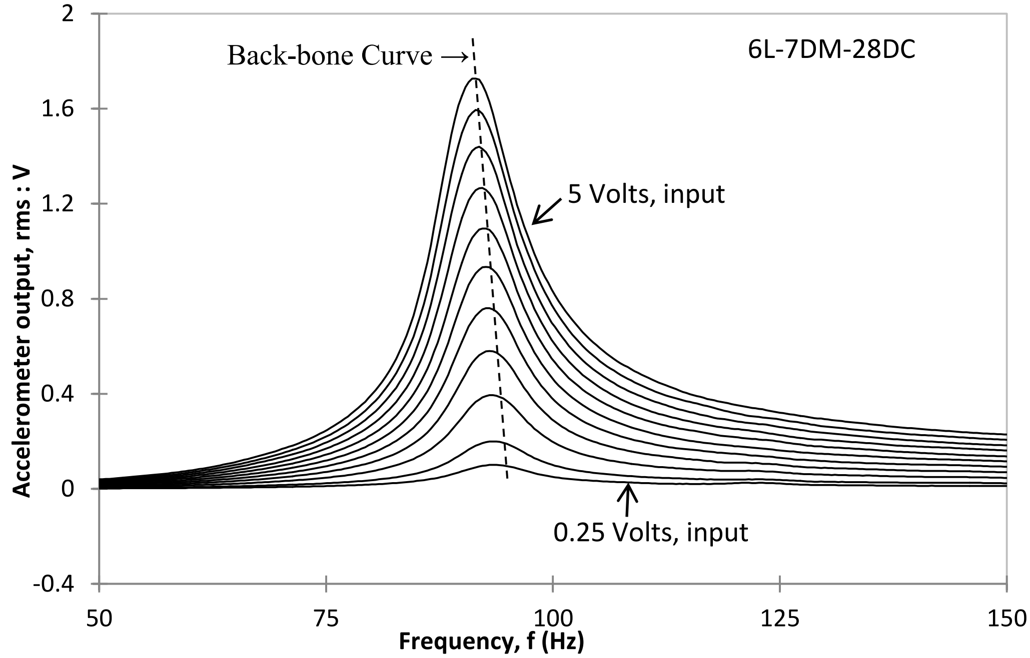
(a) control soil (b) 4L-8FA-3DM-28DC treated soil

After the last RC test under a 20-psi confining pressure was completed, this pressure was kept constant and an additional series of ten (10) RC tests were conducted, though now for different amplitudes of the input signal sent to the driver coils: 0.5, 1, 1.5, 2, 2.5, 3, 3.5, 4, 4.5, and 5 Volts (Figure 5-2). The main purpose with these additional tests was to assess the potential degradation effects of mid to high shear strain amplitude levels on control and treated soil's stiffness properties at relatively high confining levels.

All the procedures described above were identically followed for all the 9 RC test specimens of stabilized soil listed in Table 5-1. A single untreated specimen of natural soil, used as control soil, was also tested in the RC device. This control specimen was prepared at optimum moisture content and then tested at the same confining pressures and elapsed times used for stabilized soils. The following sections present a comprehensive analysis of all RC test results.



(a)



(b)

Figure 5-2 typical back-bone curve (a) control soil (b) 6L-7DM-28DC treated soil

5.4 Linear Soil Response at Low-Amplitude Shear Strains

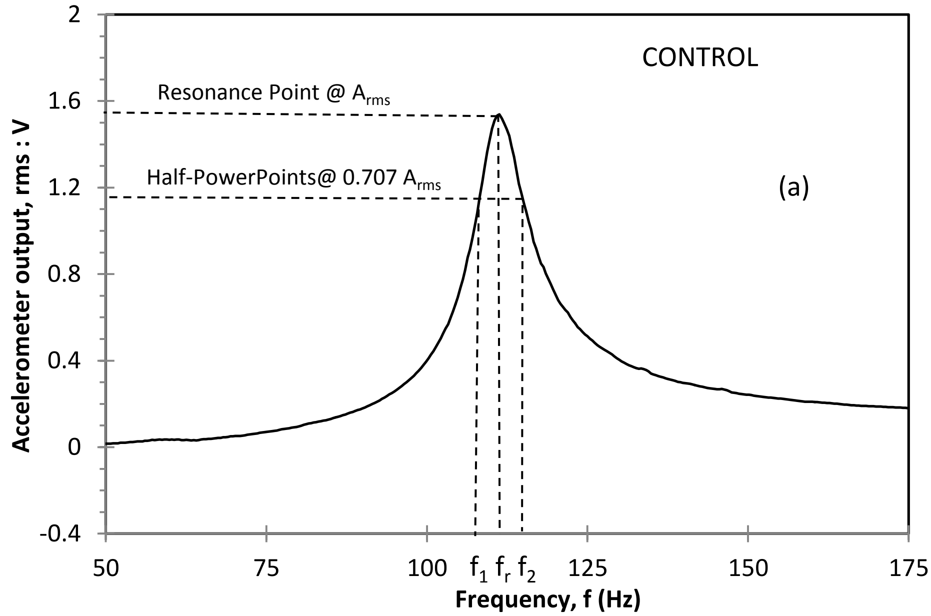
5.4.1 Threshold strain limit, γ_{th}

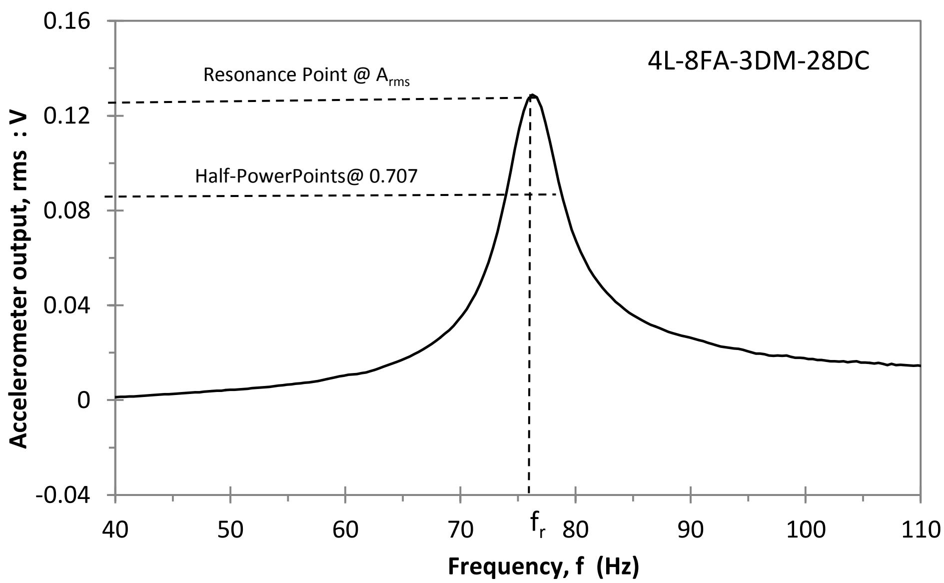
The shear strain amplitude below which stiffness properties of soils are independent of shearing strain, that is, the material exhibits linear elastic behavior, is known as threshold strain (γ_{th}). If soils are cycled at shear strain levels greater than this threshold limit, soil stiffness properties will exhibit nonlinear behavior and the soil undergo shear strain induced softening or degradation (Isenhower, 1979; Huoo-Ni, 1987). Shear strains (γ) below this threshold limit are called low-amplitude shear strains, and the key soil stiffness properties measured at these low amplitude shear strain levels are the low-amplitude (linear or maximum) shear modulus (G_{max}), and the low-amplitude (linear or minimum) damping ratio (D_{min}).

5.4.2 Typical frequency response

Figure 5-3 shows a typical frequency response curve obtained for specimen 4L-8FA-3DM-28DC under 5 psi isotropic confinement and low-amplitude excitation. The resonant frequency (f_r), corresponding to the peak of the frequency response curve, and the half-power points (f_1 and f_2) are used to determine low-amplitude or linear dynamic soil properties (G_{max}) and (D_{min}) for these particular compaction and confinement conditions, as described in Chapter 3.

In this research work, each specimen listed in Table 5.1 was tested at different elapsed times under constant confinement in order to assess the effects of compaction moisture content on stiffness properties of treated soil, as mentioned in Chapter 4. For each confining pressure (σ_0), specimens were tested after 0 and 24 hours elapsed under the constant isotropic confinement. The resonant frequency (f_r), and therefore the shear modulus (G_{max}), increases with elapsed time, a phenomenon known as cementation or mechanical aging of soils, which improves soil properties with time (Schmertmann, 1992).





(b)

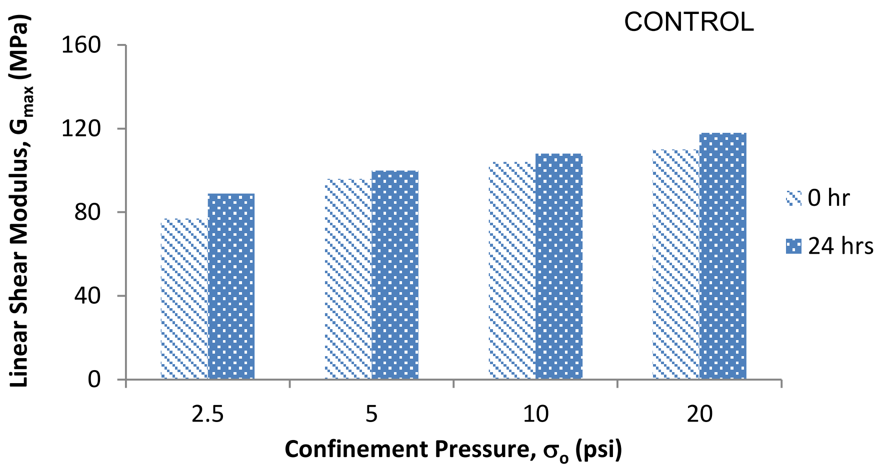
Figure 5-3 typical frequency response curve of specimen (a) control soil

(b) 4L-8FA-3DM-28DC treated soil

5.4.3 Natural (control) sulfate-rich soil

The natural, (control) sulfate-rich soil used in this investigation was compacted at optimum moisture content (w_{opt}) conditions in order to compare its linear dynamic response with that of stabilized soil. The soil specimen was tested for the range of isotropic confining pressures (σ_o) and elapsed times selected in this study. Figure 5-4 shows variation of low-amplitude shear modulus (G_{max}) and low amplitude damping ratio (D_{min}) with elapsed time for different isotropic confinement pressures. It can be readily noted that the shear modulus (G_{max}) increases, and damping ratio (D_{min}) decreases, with elapsed time under constant confinement.

It is also observed that G_{max} increases and D_{min} decreases with isotropic confinement σ_0 (Figure 5-5). As previously mentioned, this is a phenomenon that can be directly attributed to the fact that the soil stiffness is directly related to shear wave velocity (V_s), which is largely sensitive to confinement of the packed media (Huoo-Ni, 1987).



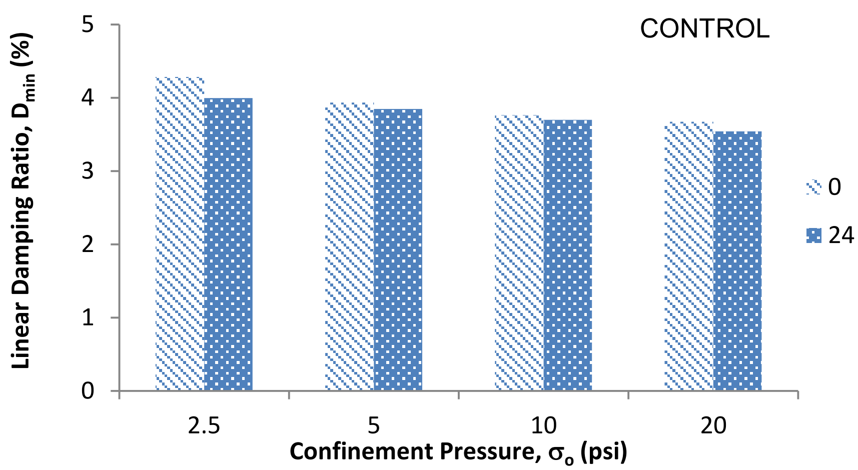


Figure 5-4 variation in G_{max} and D_{min} with time under constant confinement pressure of control soil

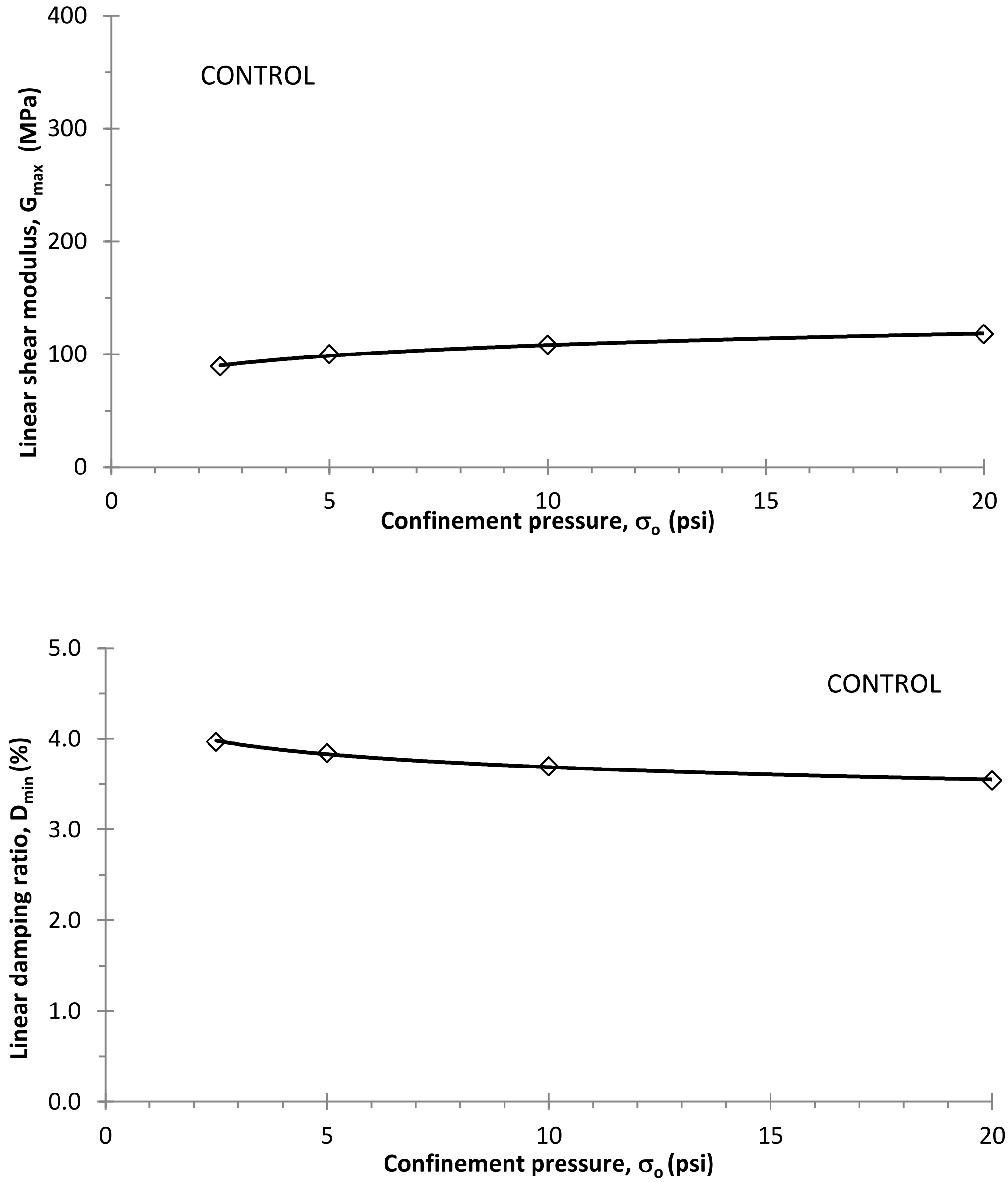


Figure 5-5 variations in G_{max} and D_{min} with confinement pressure for control soil

5.4.4 6% lime treated soil

A series of resonant column (RC) tests were conducted on several specimens, natural soil and 6% lime treated soil with different mellowing period, to determine relationships between low-amplitude shear modulus (G_{max}) and low-amplitude damping ratio (D_{min}) with elapsed time (t) under constant confining pressure (σ_0). Figures 5-6 and 5-7 present the variations in low-amplitude shear modulus (G_{max}) and low-amplitude damping ratio (D_{min}) with the elapsed time, respectively under constant confinement as the soil column was undergoing an input signal of 2.5 Volts (rms).

Figures 5-8 through 5-12 show the effect of confining pressure (σ'_0) on the G_{max} and D_{min} values of 6% lime stabilized soil after a 24-hr confinement. It can be seen that in general G_{max} increases and D_{min} decreases with confinement σ_0. Again, this can be explained by the fact that the higher the confinement level, the more the specimen consolidates, and hence the stiffer it becomes. As it can be observed from these figures, the specimen treated with 6% lime and 3 days of mellowing yields the highest values of G_{max}, which also corresponds to the lowest values of D_{min} as compared to any other lime treated specimens at any elapsed time or confining pressure. Hence, it can be concluded that allowing for proper mellowing period is critical in gaining best performance of stiffness properties of lime treated sulfate-rich soil.

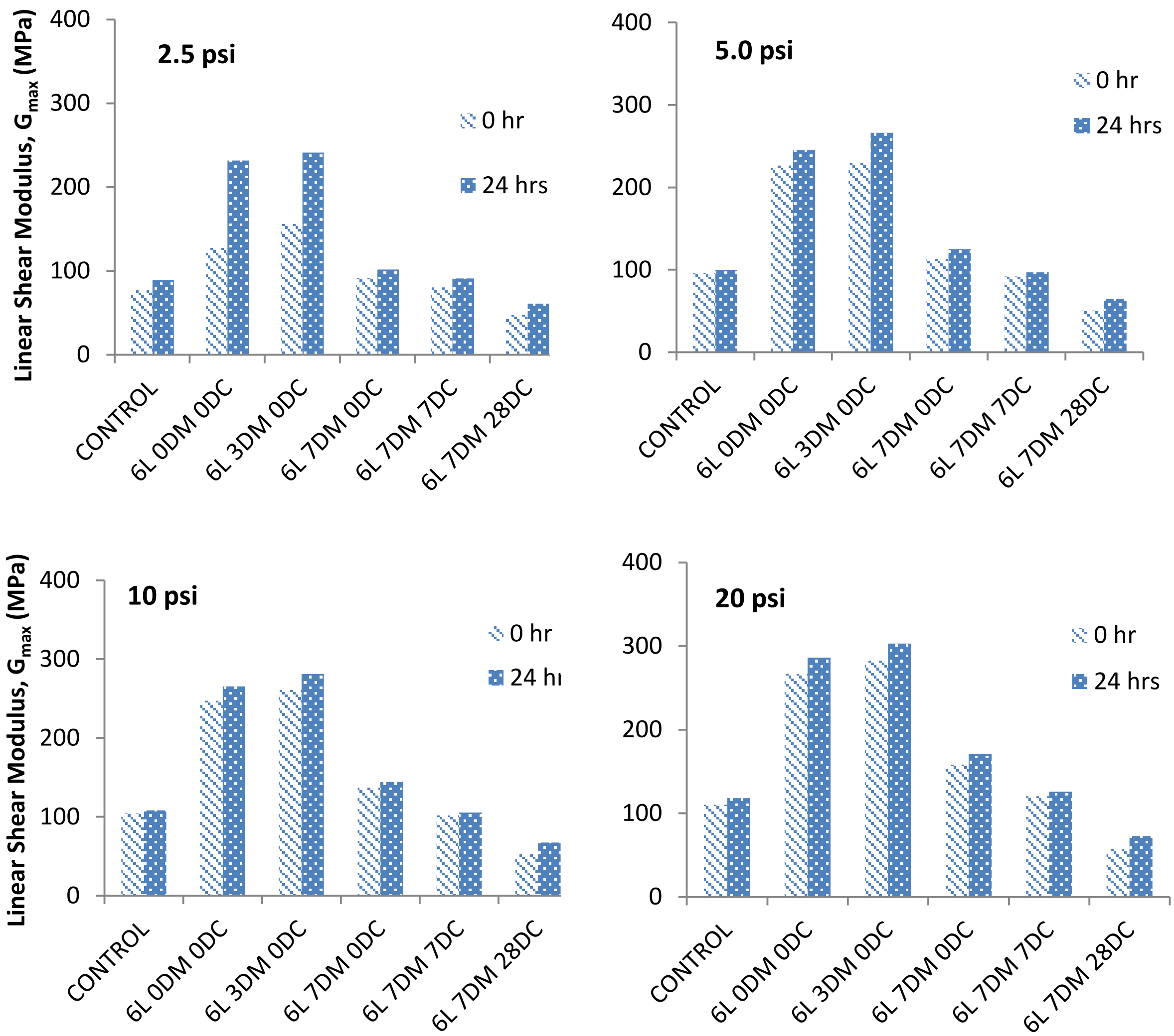


Figure 5-6 variation of G_{max} with elapsed time under constant confining pressure for control and 6% lime treated soil with different mellowing period

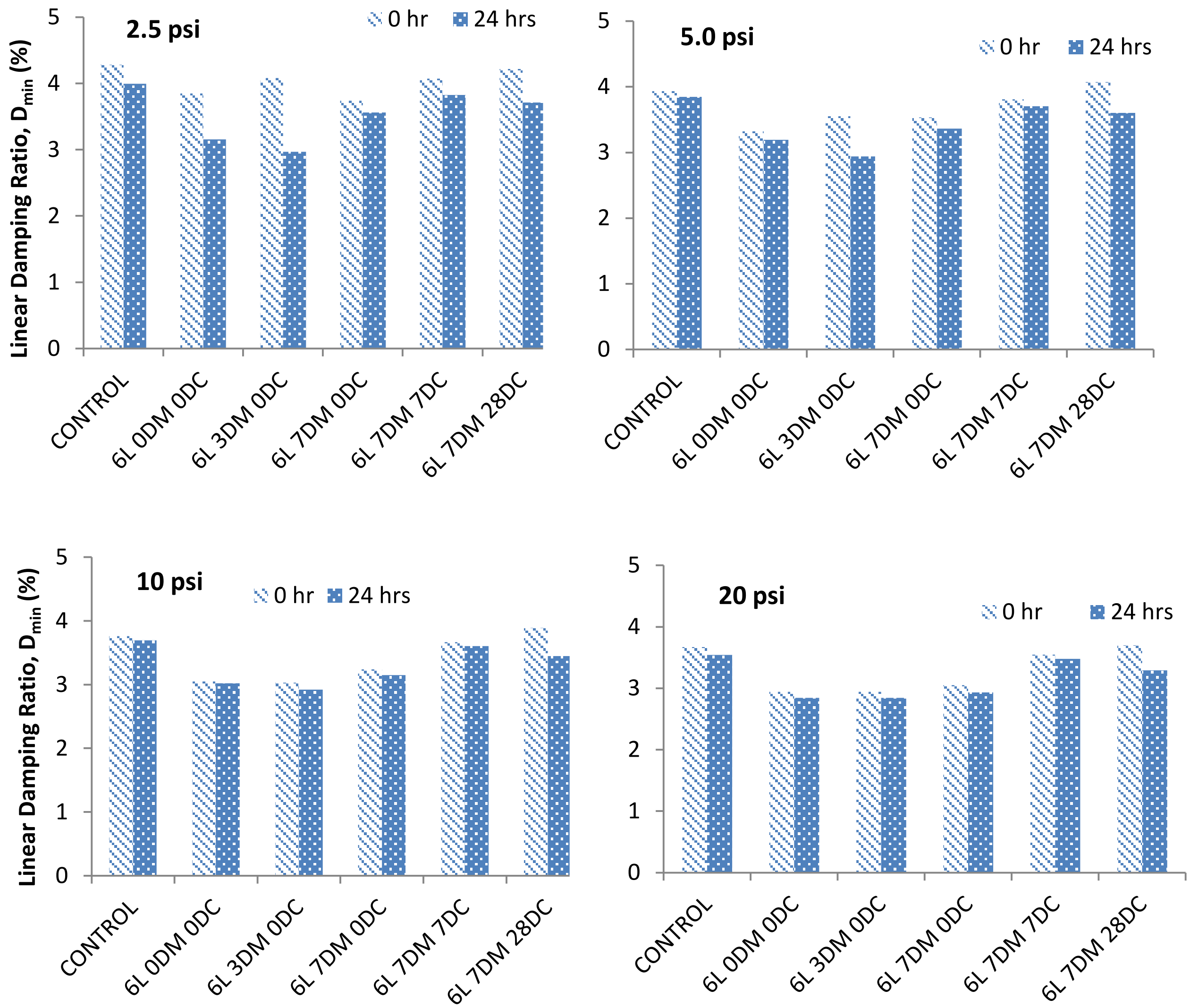


Figure 5-7 variation of D_{min} with elapsed time under constant confining pressure for control and 6% lime treated soil with different mellowing period

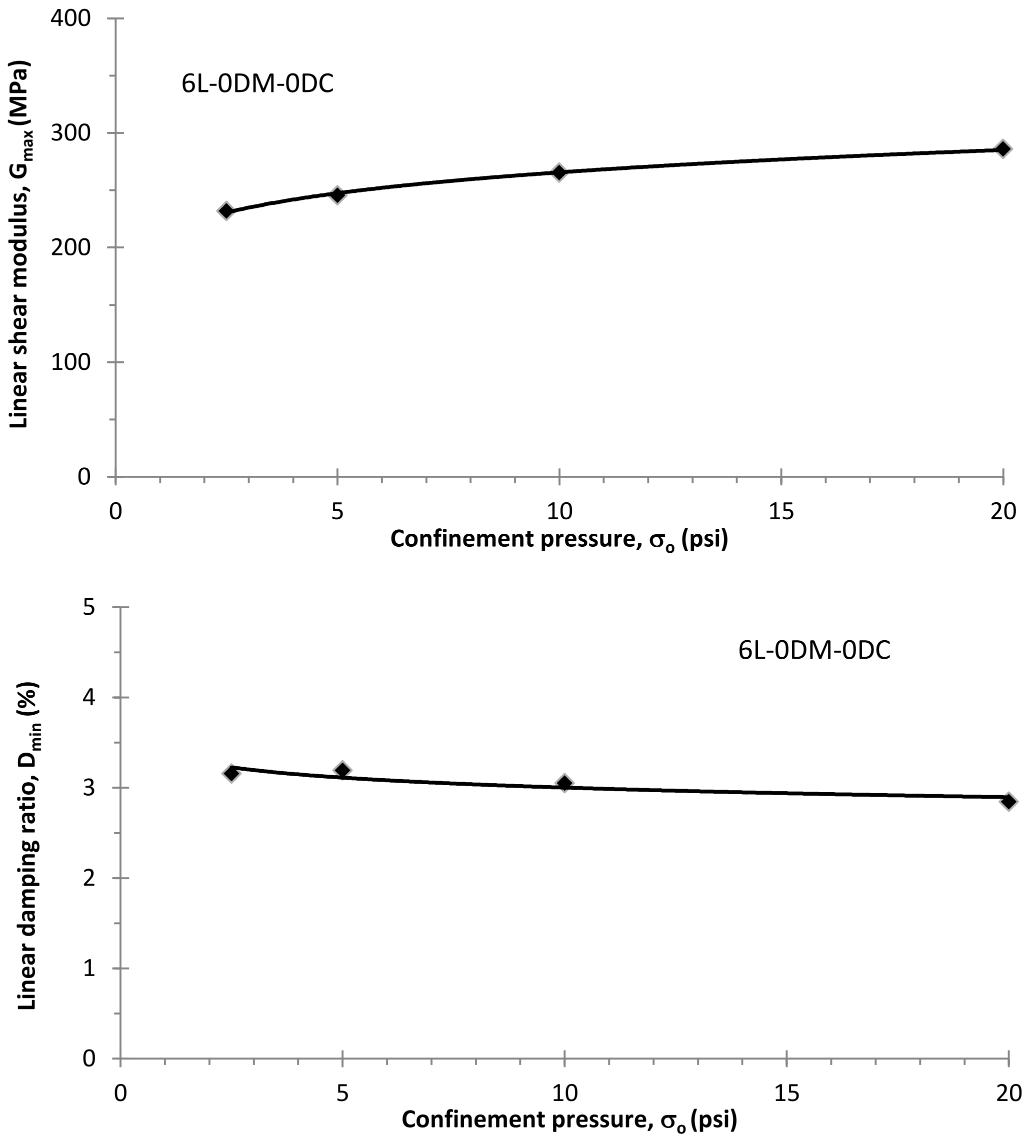
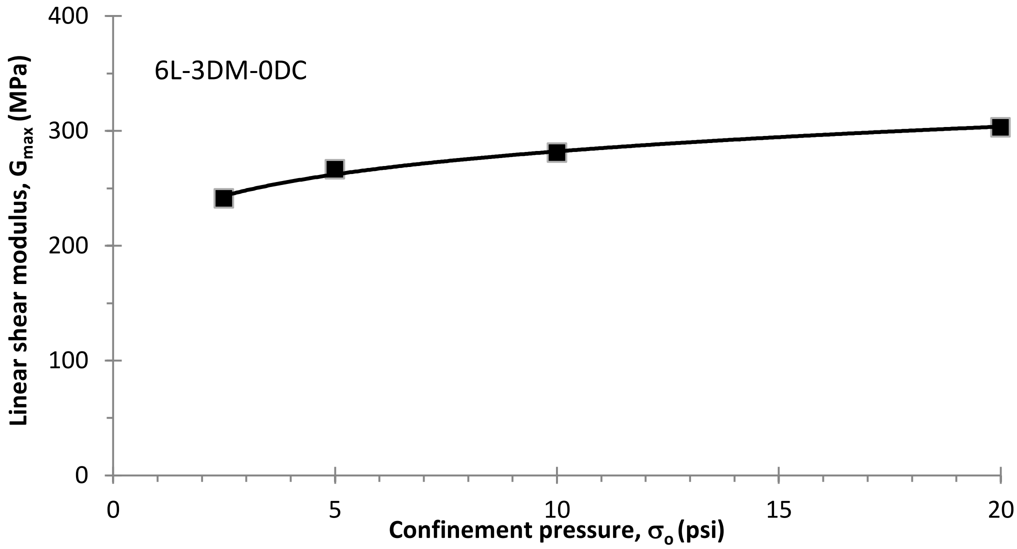


Figure 5-8 variation of G_{max} and D_{min} with σ_0 for 6L-0DM-0DC treated soil



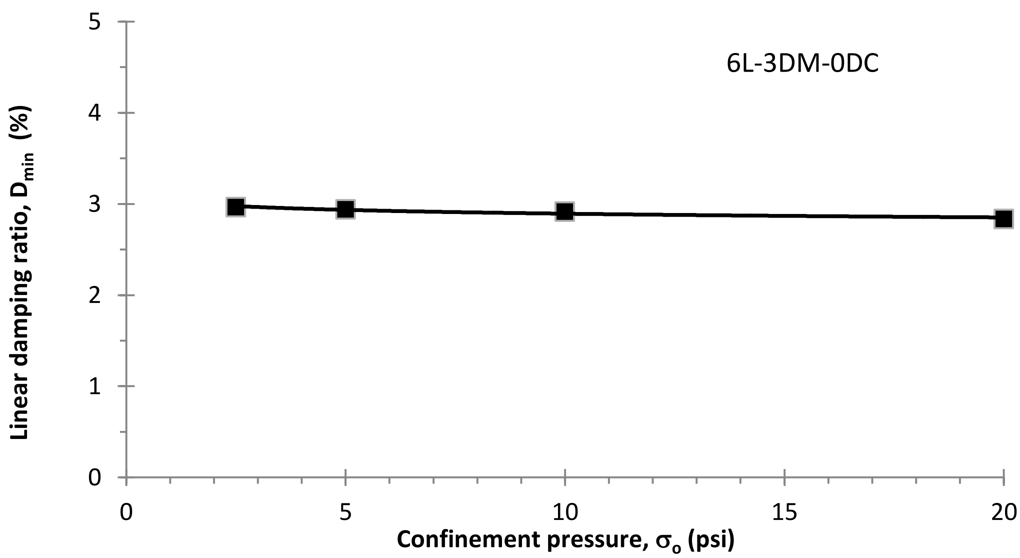


Figure 5-9 variation of G_{max} and D_{min} with σ_0 for 6L-3DM-0DC treated soil

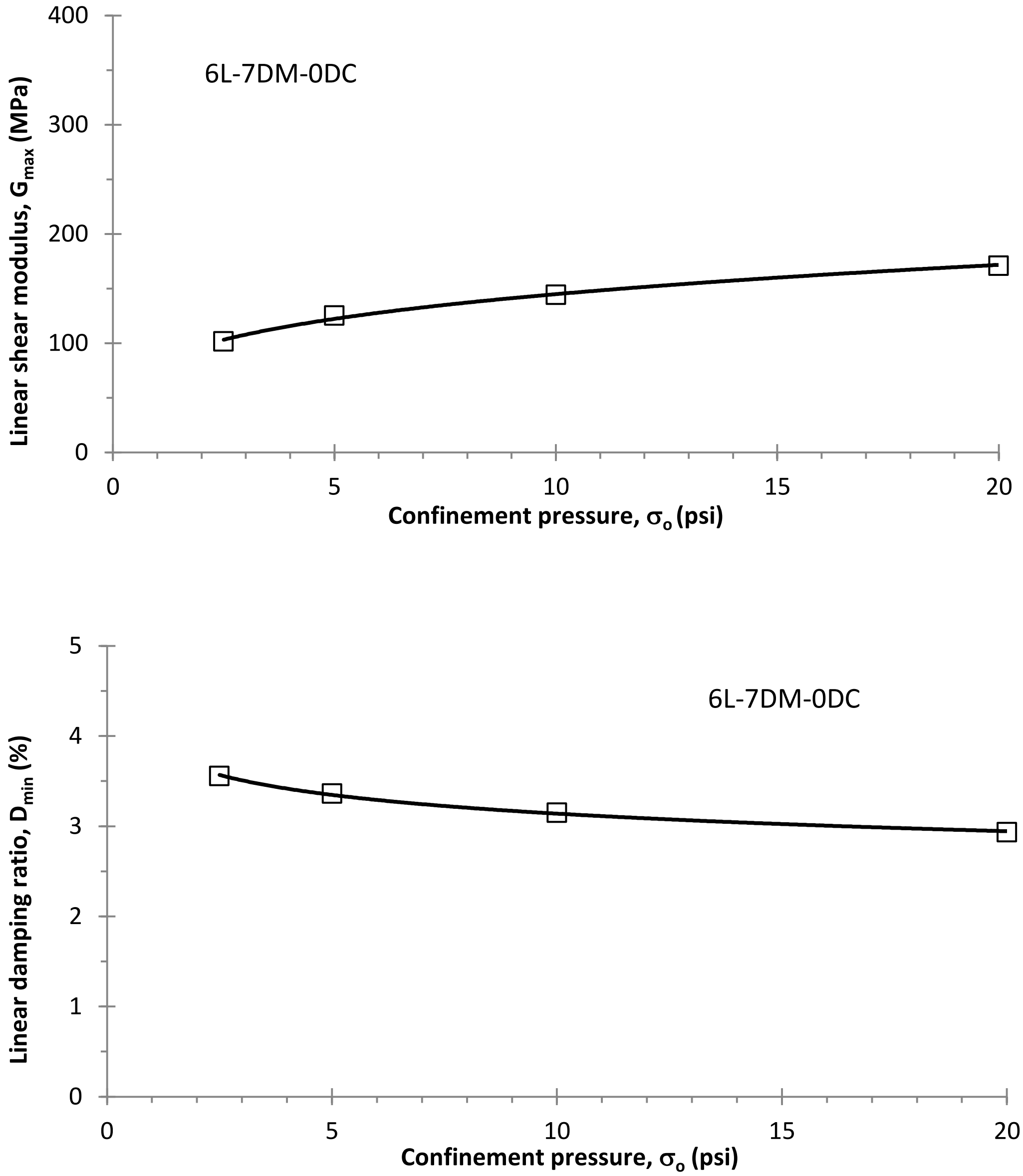


Figure 5-10 variation of G_{max} and D_{min} with σ_0 for 6L-7DM-0DC treated soil

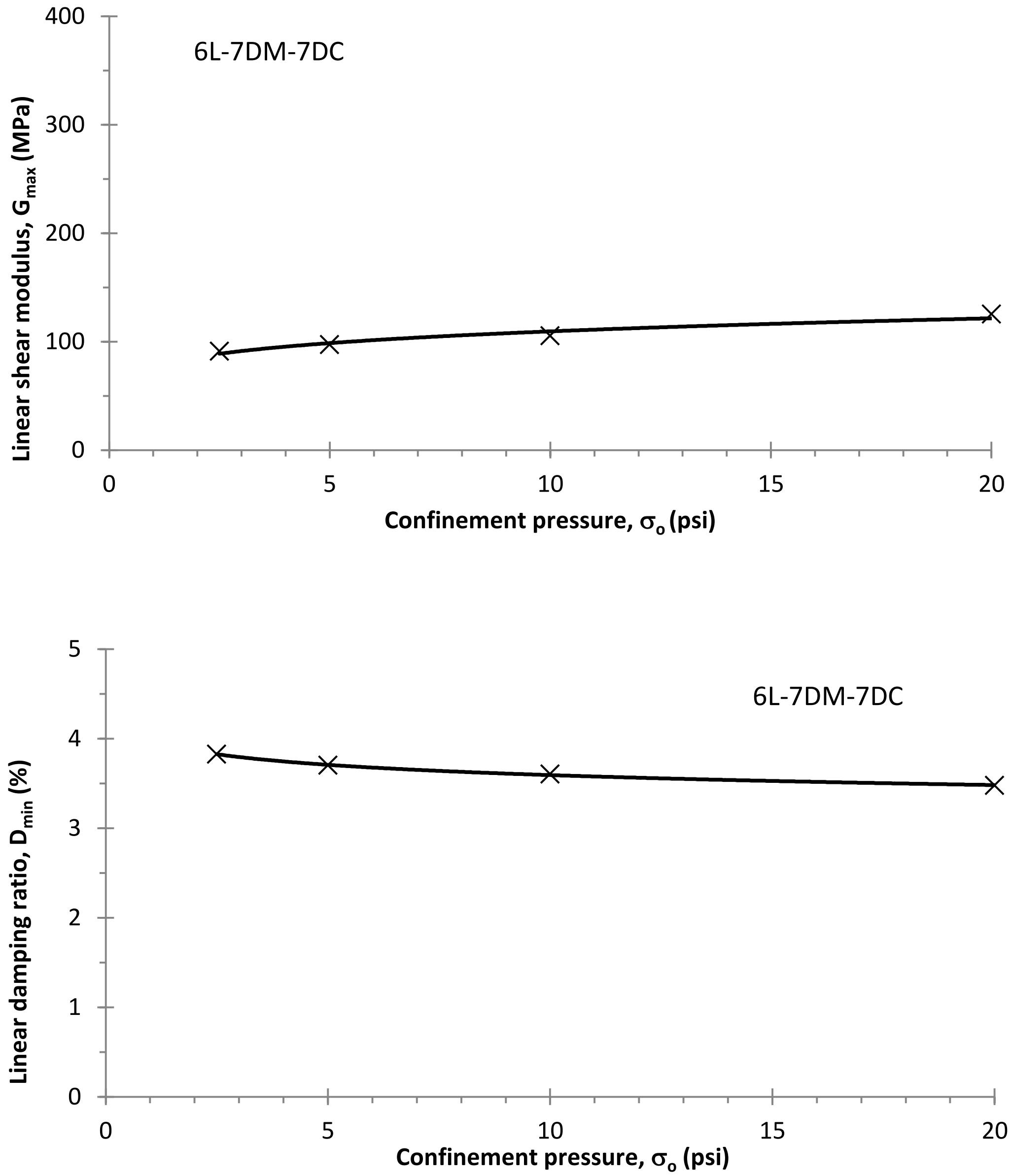
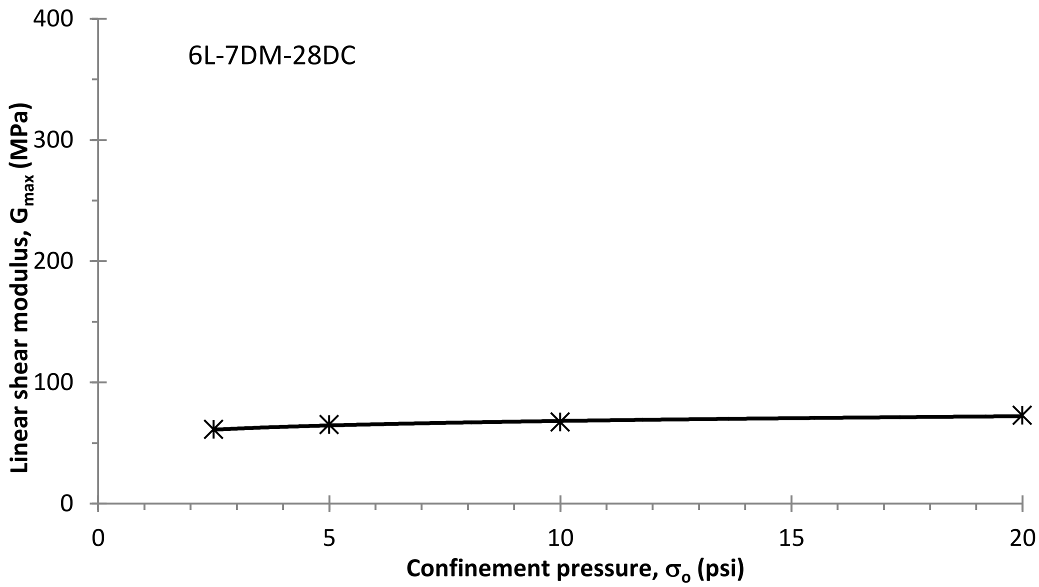


Figure 5-11 variation of G_{max} and D_{min} with σ_0 for 6L-7DM-7DC treated soil



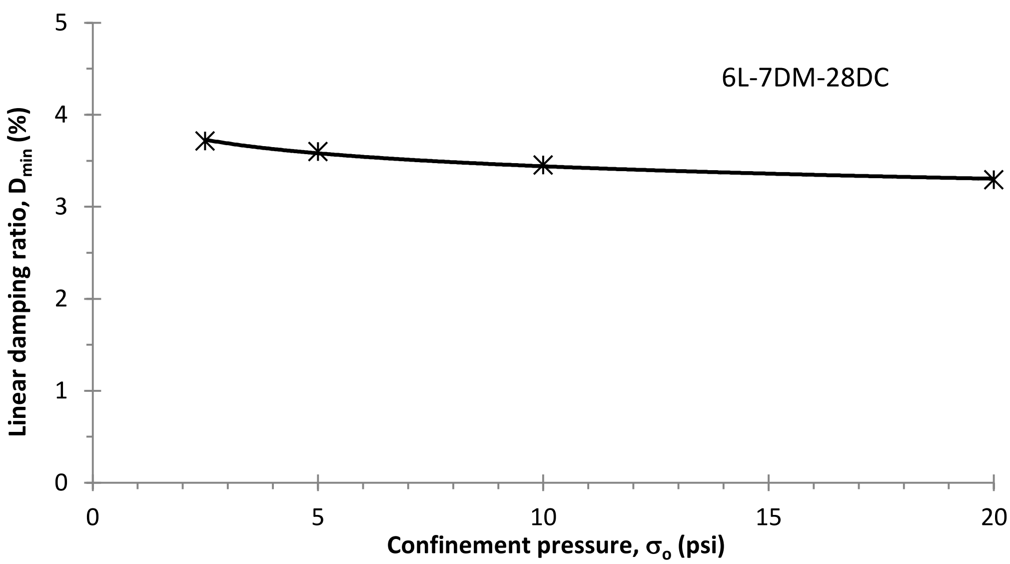


Figure 5-12 variation of G_{max} and D_{min} with σ_0 for 6L-7DM-28DC treated soil

5.4.5 4% lime + 8% fly ash treated soil

Figures 5-13 and 5-14 present the variation in low-amplitude shear modulus G_{max} and low-amplitude damping ratio D_{min}. With elapsed time for specimens of natural soil mixed with 4% lime + 8% fly ash. Each figure corresponds to a different confining pressure. These figures show that the low-amplitude shear modulus G_{max} increases, and low-amplitude D_{min} decreases with elapsed time. As with 6% lime treated soil, it may be concluded that the effect of elapsed time under constant confinement on soil stiffness properties is quite small.

Figures 5-15 through 5-18 show the variation in low-amplitude shear modulus G_{max} and low-amplitude damping ratio D_{min} with isotropic confining pressure $\sigma_{0,}$ after a 24hr confinement. It is observed that G_{max} increase, and D_{min} decreases, with confinement σ_o. It is observed that all four specimens show the increase in G_{max} with increase σ_o .The low-amplitude damping ratio D_{min} slightly decreases with confinement $\sigma_{0,}$ that is, the rigidity of the material or the packed media increases with the confining stress, but not in such a well-defined fashion as the low-amplitude shear modulus G_{max} does. Figure 5-16(a) shows that soil specimens prepared with 4%lime + 8% fly ash, and 3 days mellowed specimen gives the highest values of low-amplitude G_{max}, as compared to the stabilizer treated soils. As with the lime, it can be stated that the selection of a proper mellowing period is critical for the best performance of dynamic and stiffness properties of lime + fly ash treated soil.

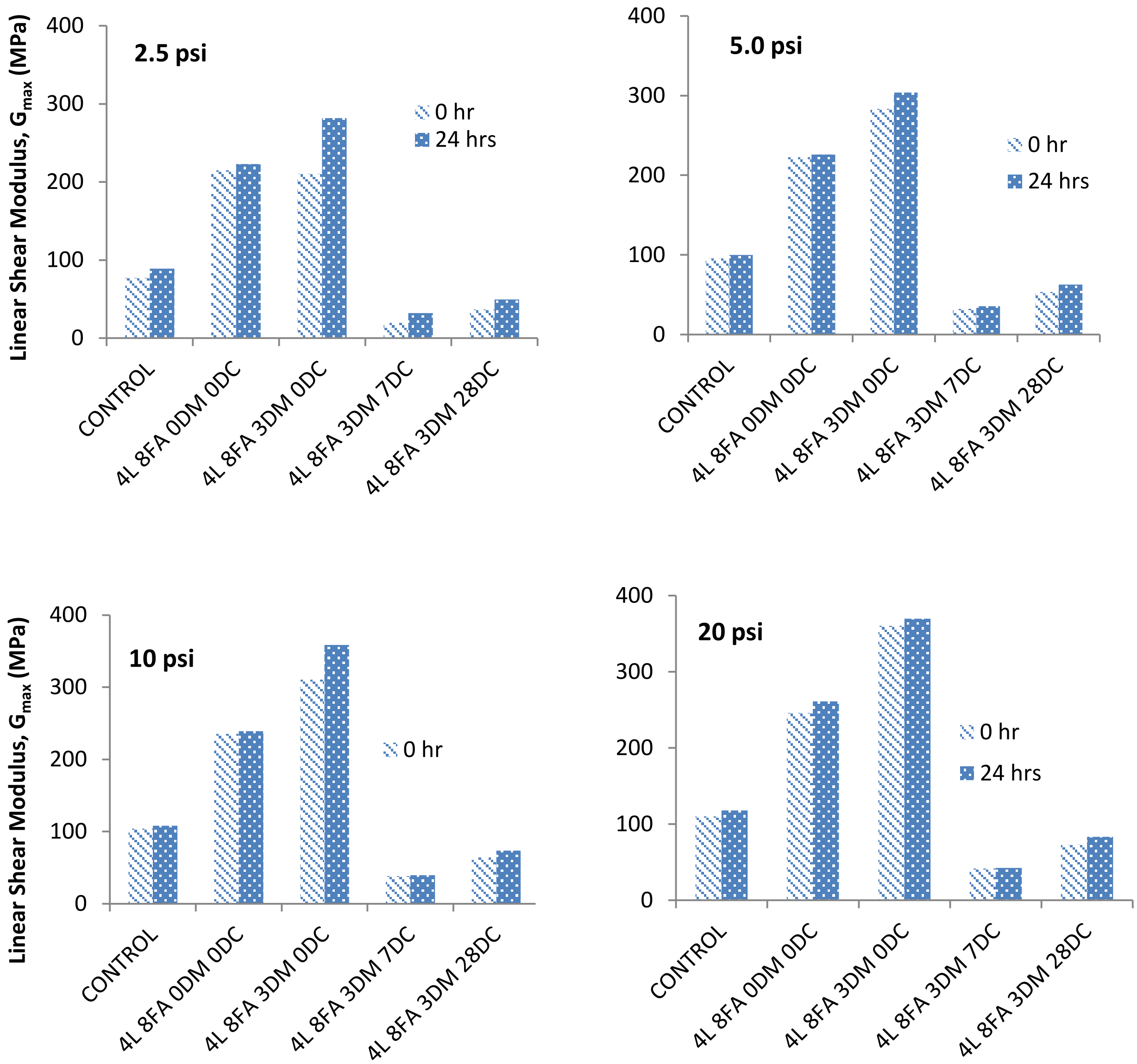


Figure 5-13 variation of G_{max} with elapsed time under constant confining pressure for control and 4% lime + 8% fly ash treated soil with different mellowing period

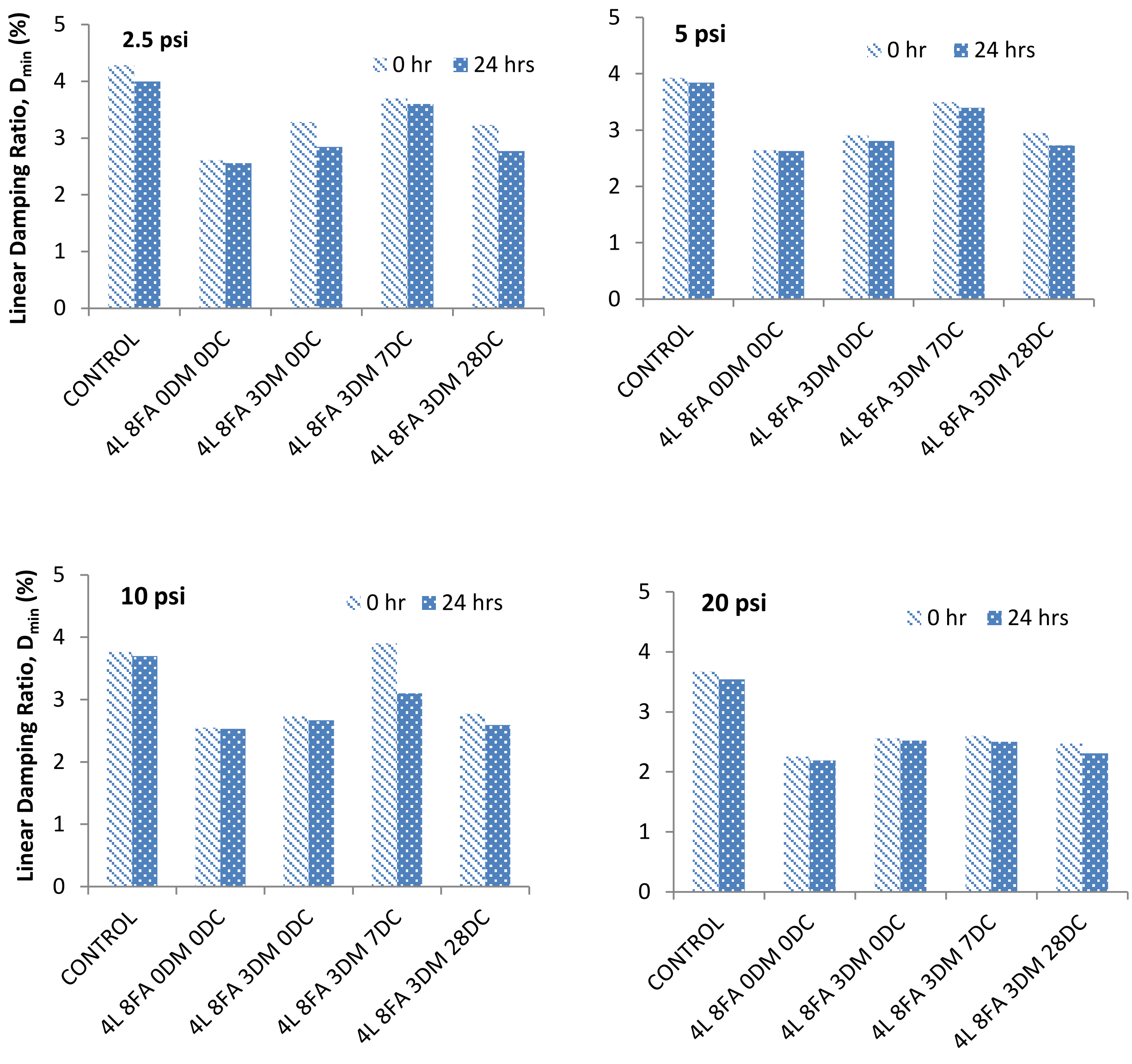


Figure 5-14 variation of D_{min} with elapsed time under constant confining pressure for control and 4% lime + 8% fly ash treated soil with diffrent mellowing period

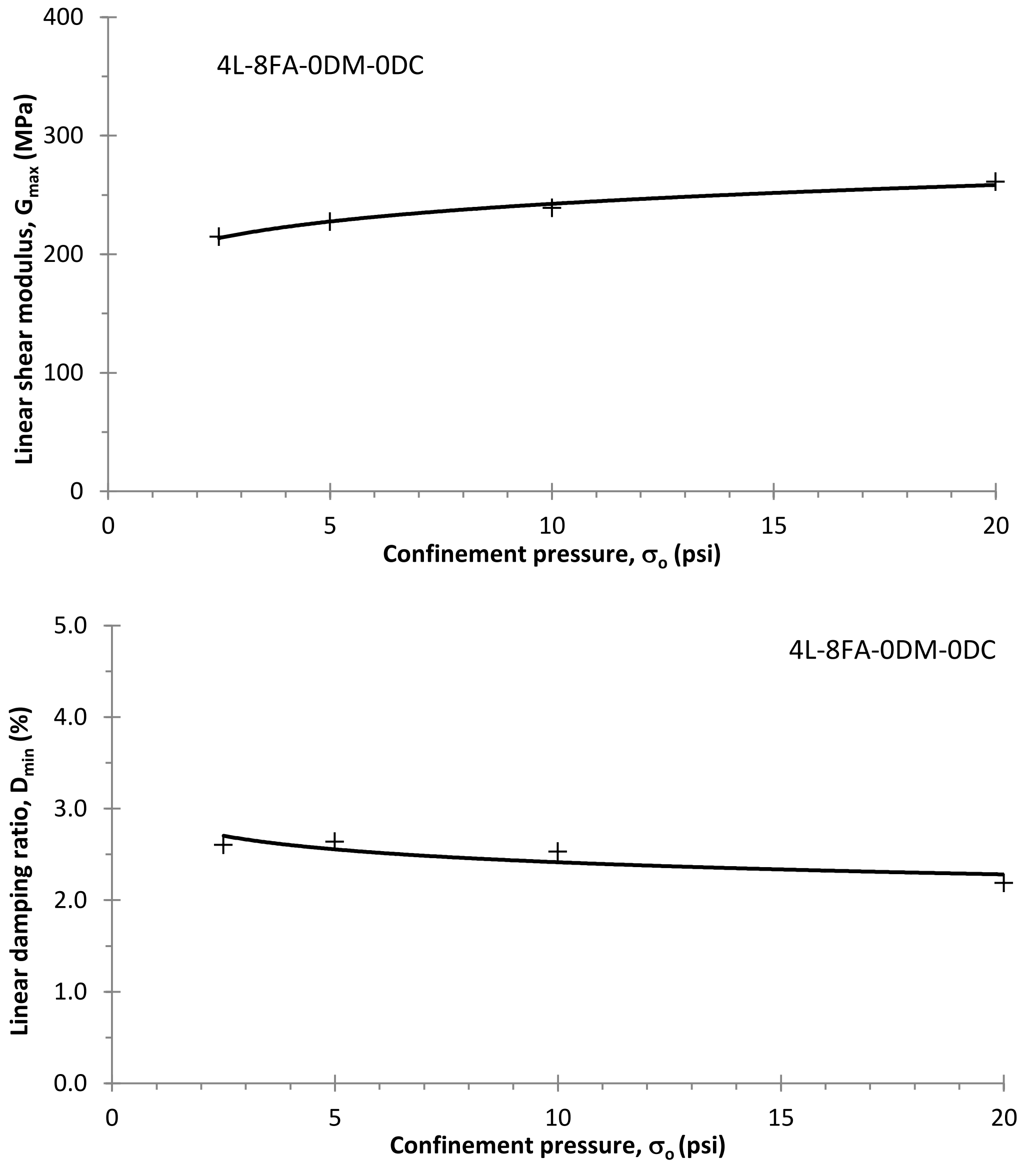
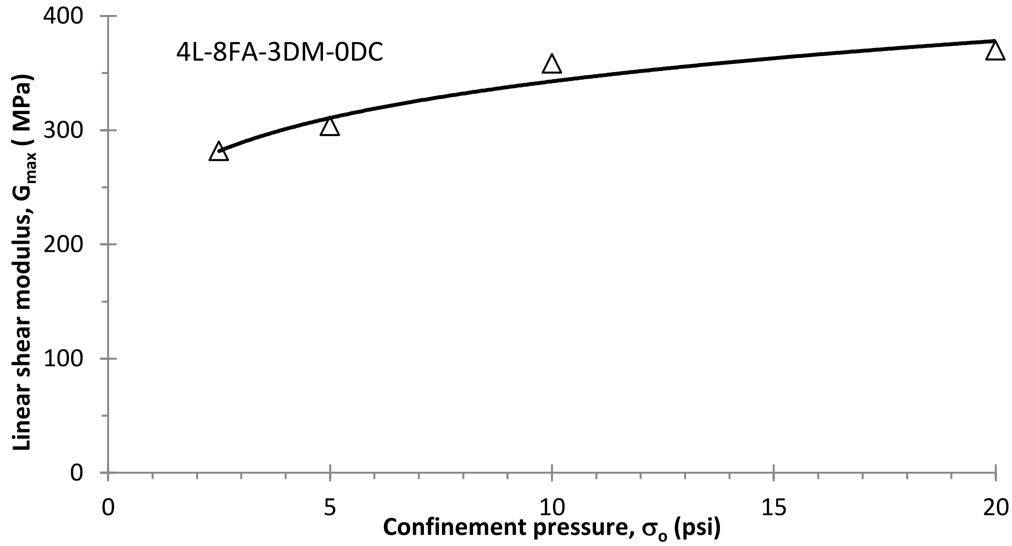


Figure 5-15 variation of G_{max} and D_{min} with σ_0 for 4L-8FA-0DM-0DC treated soil



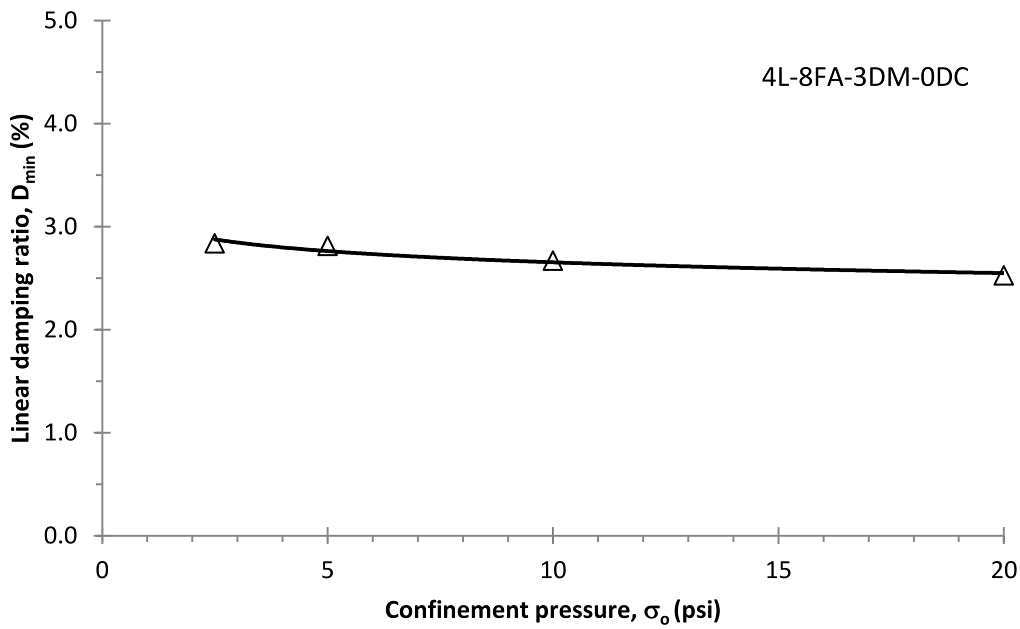
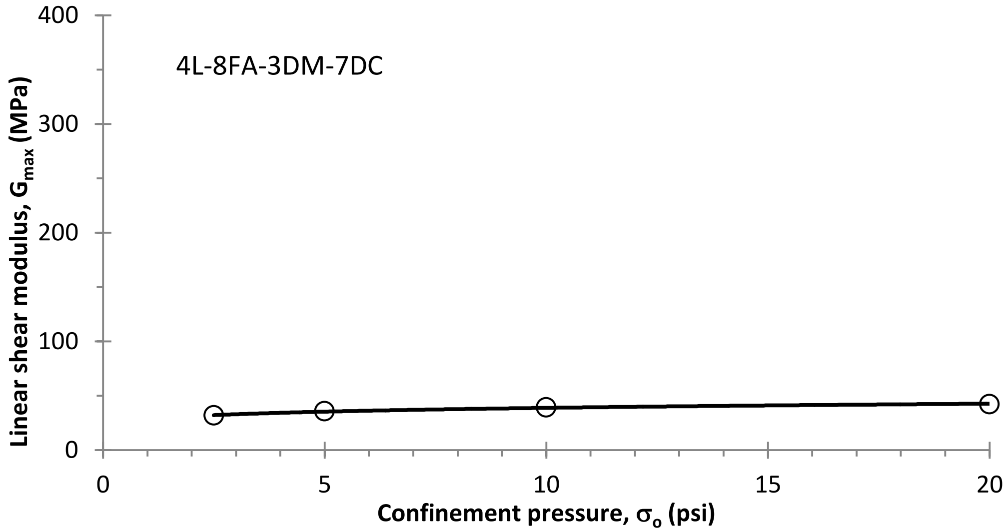


Figure 5-16 variation of G_{max} and D_{min} with σ_0 for 4 L-8FA-3DM-0DC treated soil



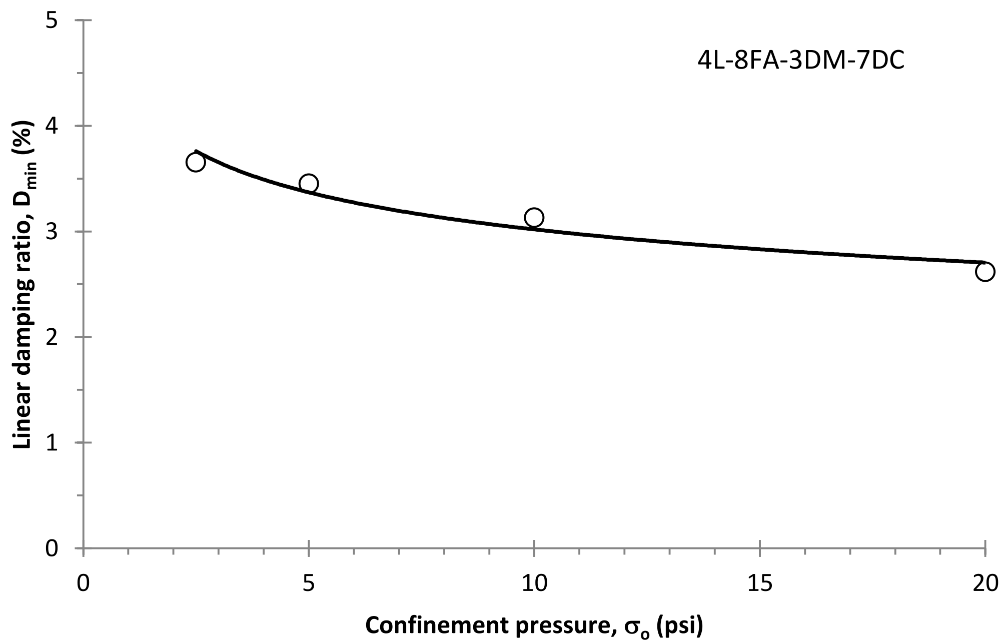


Figure 5-17 variation of G_{max} and D_{min} with σ_0 for 4 L-8FA-3DM-7DC treated soil

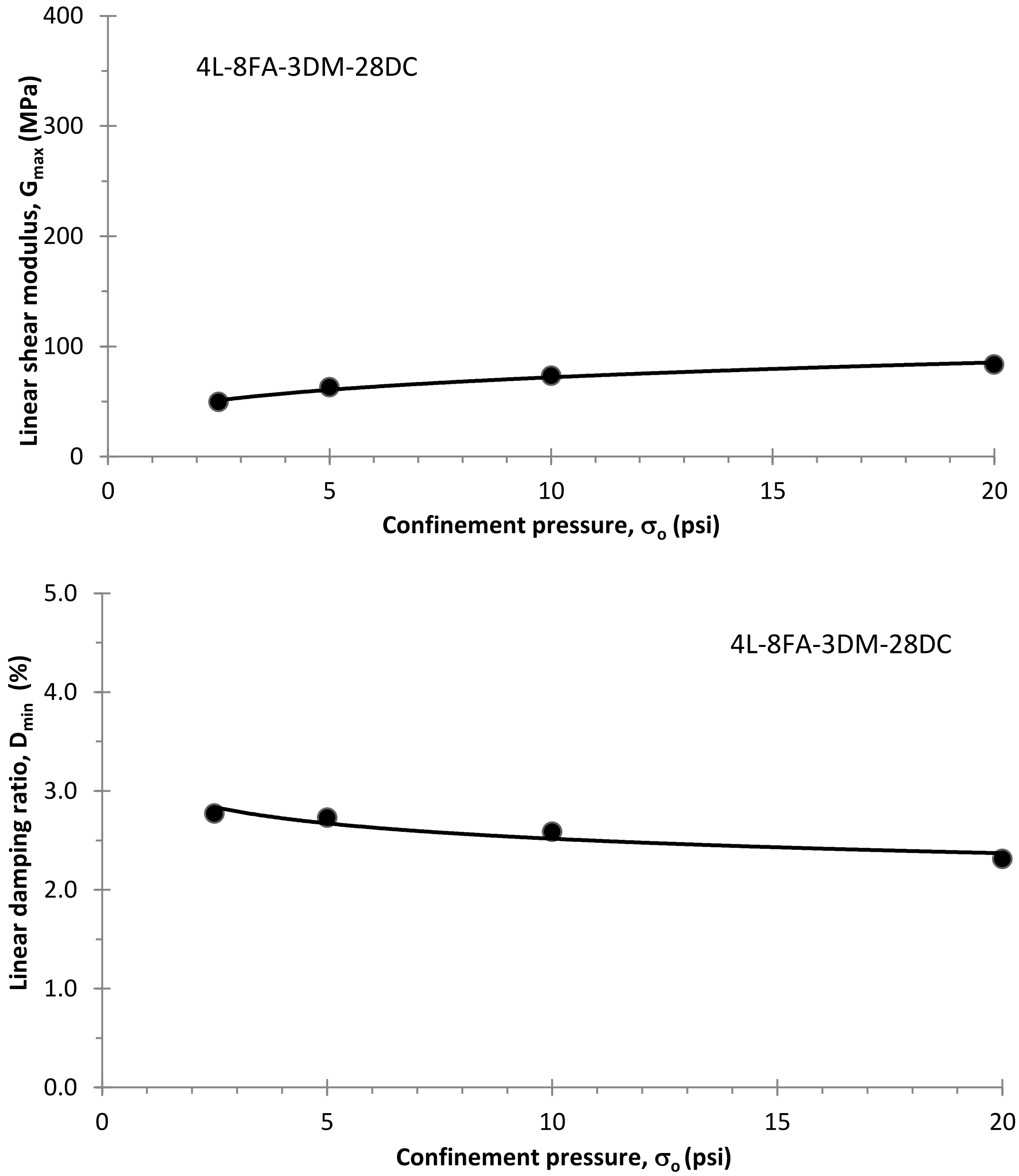


Figure 5-18 variation of G_{max} and D_{min} with σ_0 for 4L-8FA-3DM-28DC treated soil

5.4.6 Influence of mellowing period on stiffness property of treated soil

Figure 5-19 (a) (b) shows the variations in low-amplitude shear modulus G_{max} and low-amplitude damping ratio D_{min} with effective isotropic confining pressure σ_0 for natural (control) and lime treated soil with different mellowing period. It can be noticed that 6% lime treated soil with 3 days of mellowing gives the highest value of low-amplitude shear modulus G_{max} and the lowest low-amplitude damping ratio $D_{min,}$ and it is followed by 6% lime treated soil with 0 days mellowing, 6% lime treated with 7 days mellowing and finally control soil.

Figure 5-20 (a) and (b) also shows the comparison between 4% lime + 8% fly ash treated soil with different mellowing period. From this figure it can observed that 4% lime + 8% fly ash treated soil with 3 days mellowing gives the highest low-amplitude shear modulus G_{max} and the lowest low-amplitude damping ratio D_{min} followed by 4% lime + 8% fly ash treated soil the 0 day mellowing, and the control soil. From these two figures it can be concluded that. Stabilization of soil increases the stiffness of the treated soil and 3 days mellowing is the perfect mellowing period to increase stiffness of treated soil for both lime and lime + fly ash treated soils.

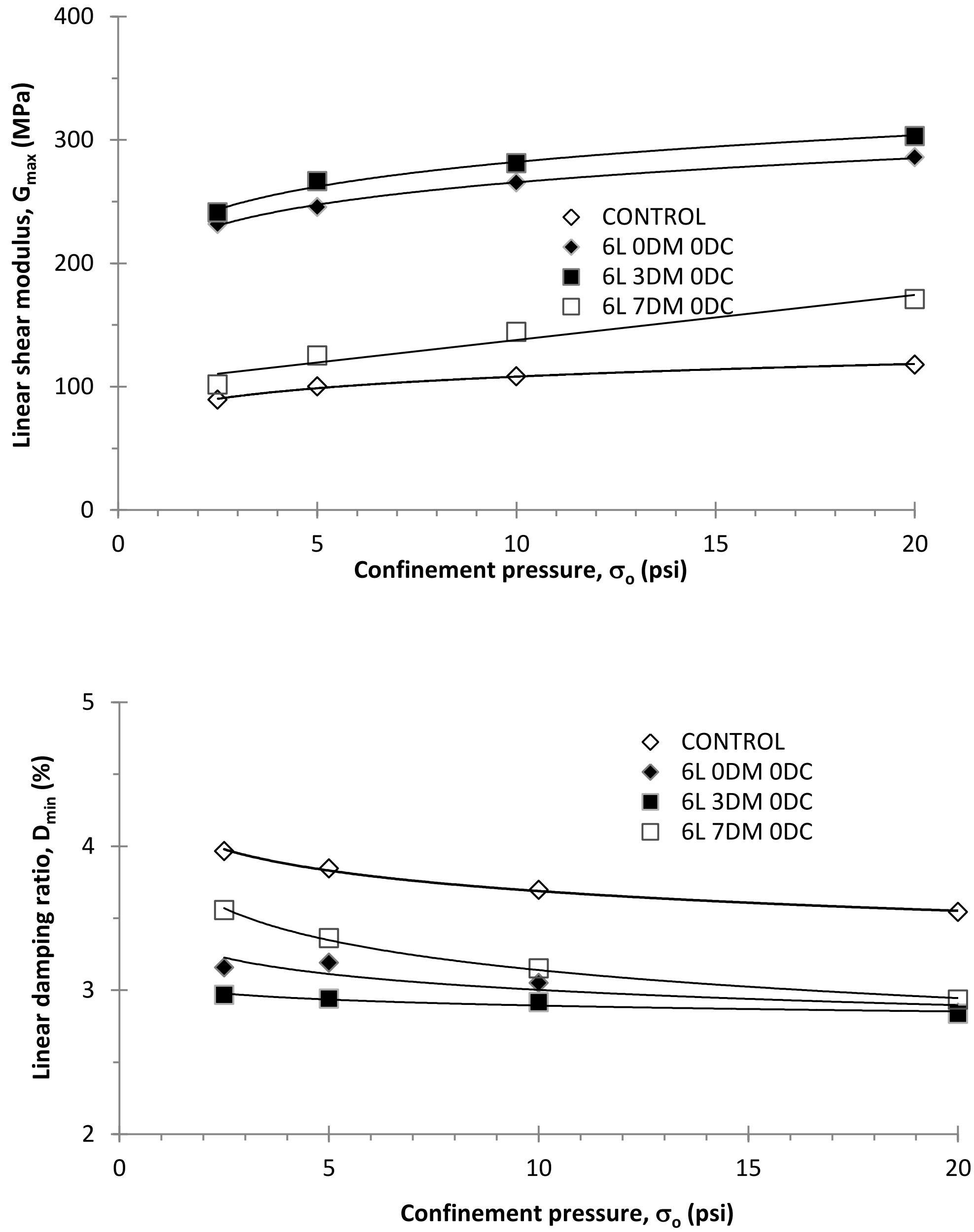
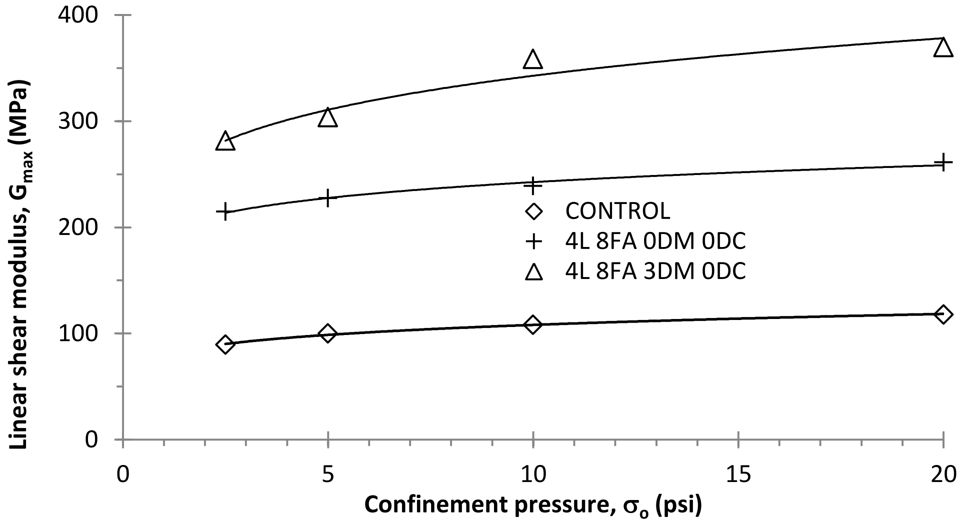


Figure 5-19 effect of mellowing period on stiffness properies of control and 6% lime treated soil



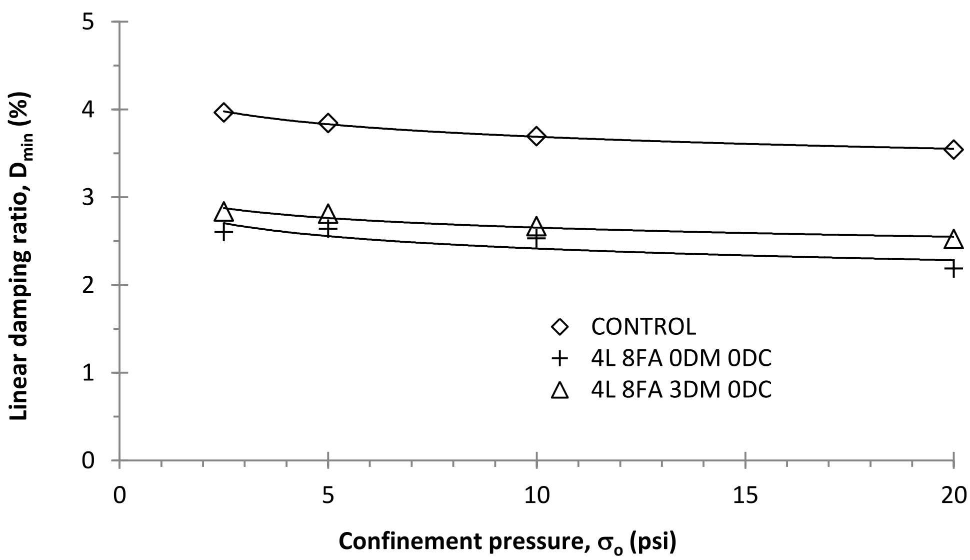


Figure 5-20 effect of mellowing period on stifness properties of control and 4% lime + 8% fly ash treated soil

5.4.7 Influence of curing time on stiffness property of treated soil

Figure 5-21 (a) and (b) show the variations in low-amplitude shear modulus G_{max} and low-amplitude damping ratio D_{min} with effective isotropic confining pressure σ_0 for natural (control) and lime treated soil with different curing time, and it can be noticed that, 6% lime treated soil with 7 mellowing days and 0 days of curing gives the highest value of low-amplitude shear modulus G_{max} and the lowest low-amplitude damping ratio $D_{min.}$ The 7 days cured soil shows low-amplitude shear modulus G_{max} of almost equal to the control soil, and the 28 days cured specimen shows low-amplitude shear modulus G_{max} even lower than the control soil.

Figure 5-22 (a) and (b) also show the comparison between 4% lime + 8% fly ash treated soils with different curing time. From this figure it can observed that 4% lime + 8% fly ash treated soil with 3 days mellowing and 0 day of curing gives the highest low-amplitude shear modulus $G_{max,}$ but the 7 day cured and 28 days soil show a lower shear modulus G_{max} than the control soil. From these figures it can be concluded that curing decreases the shear modulus of the treated soil and stiffness of the soil as well. Additionally excess curing time even lowers the stiffness of the soil lower that the control soil.

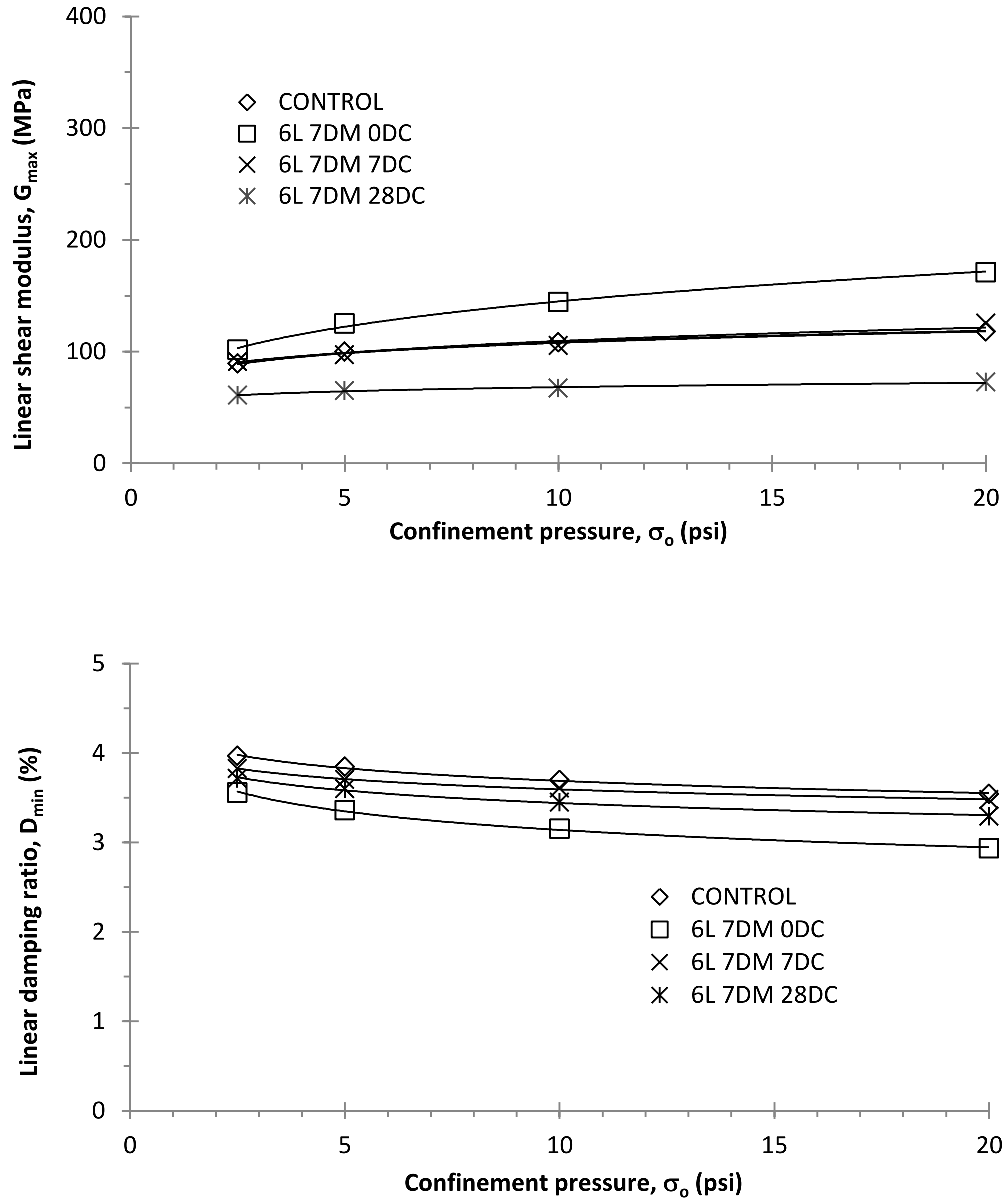


Figure 5-21 effect of curing time on stiffness properties of control and 6% lime treated soil

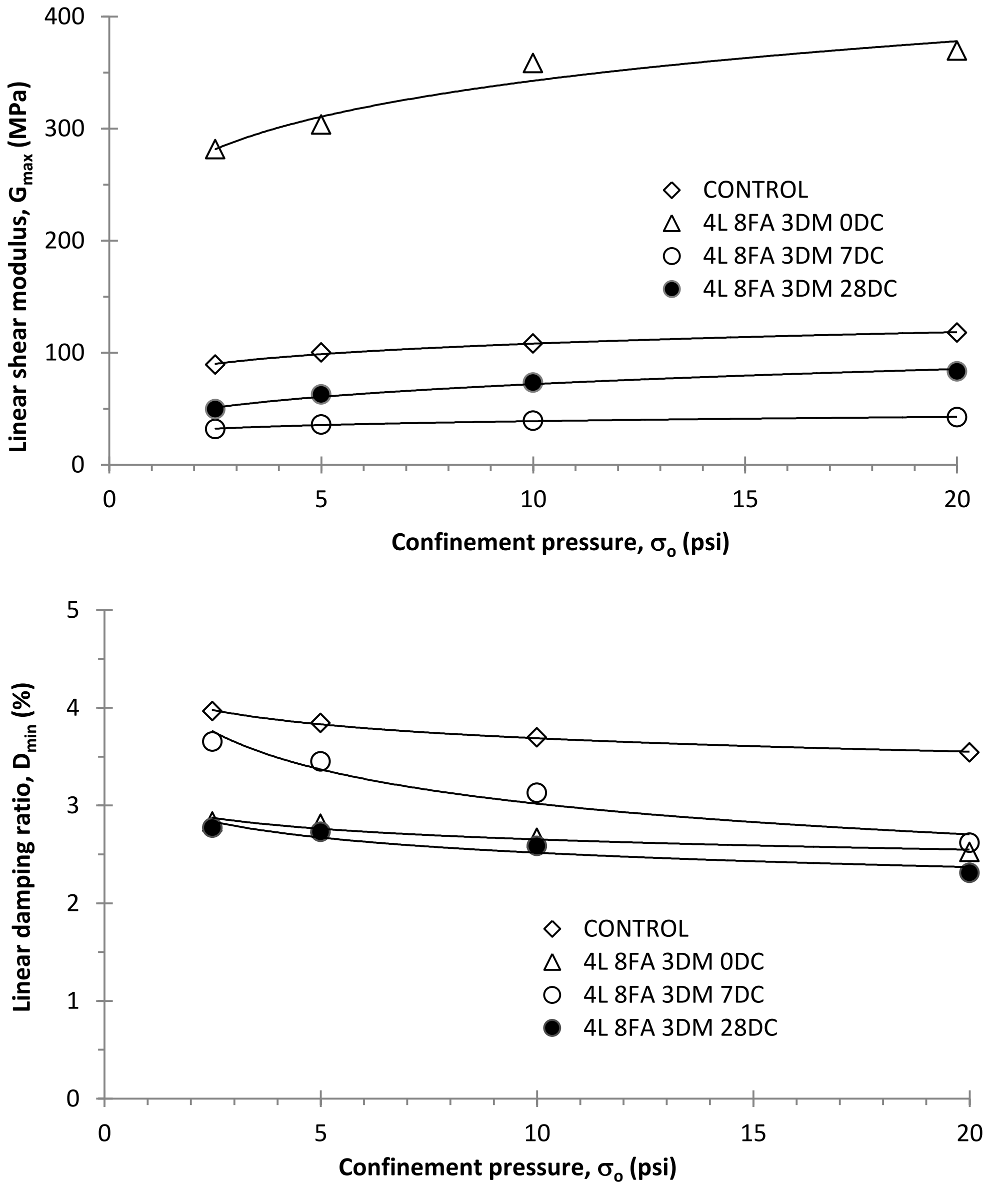


Figure 5-22 effect of curing time on stiffness properties of control and 4% lime + 8% fly ash treated soil

5.4.8 Best-performing treatment method

Figure 5-23 and 5-24 show the variations in low-amplitude shear modulus G_{max} and low-amplitude damping ratio D_{min} with effective isotropic confining pressure σ_0 for natural (control) and chemically treated soil at best-performance conditions; that is, at best-performing mellowing periods, 0 days of mellowing and 3 days of mellowing, and after 24 hours of elapsed time under constant confinement. From figure 5.19(a) and (b), it can be observed that 6% lime treated soil with 0 mellowing days gives the highest values of low-amplitude shear modulus G_{max}, and correspondingly, the lowest values of low-amplitude damping ratio D_{min}, which is followed by 4% lime + 8% fly ash treated soil with 0 days of mellowing.

As a result, it can also be concluded that, stabilization of soil increases the stiffness of the treated soil, and lime treatment plays a better roll in increasing the stiffness of treated soil than lime + fly ash in the case of 0 day of mellowing. On the other hand, in the case of 3 days mellowing, Figure 5-20 (a) (b), 4% lime + 8% fly ash treated soil shows a higher shear modulus than 6 % lime treated soil. So it can be concluded that mellowing has a better effect on lime + fly ash treated soils than lime treated specimens. Additionally, it can be clearly inferred that the influence of mellowing period varies from one stabilizer to another.

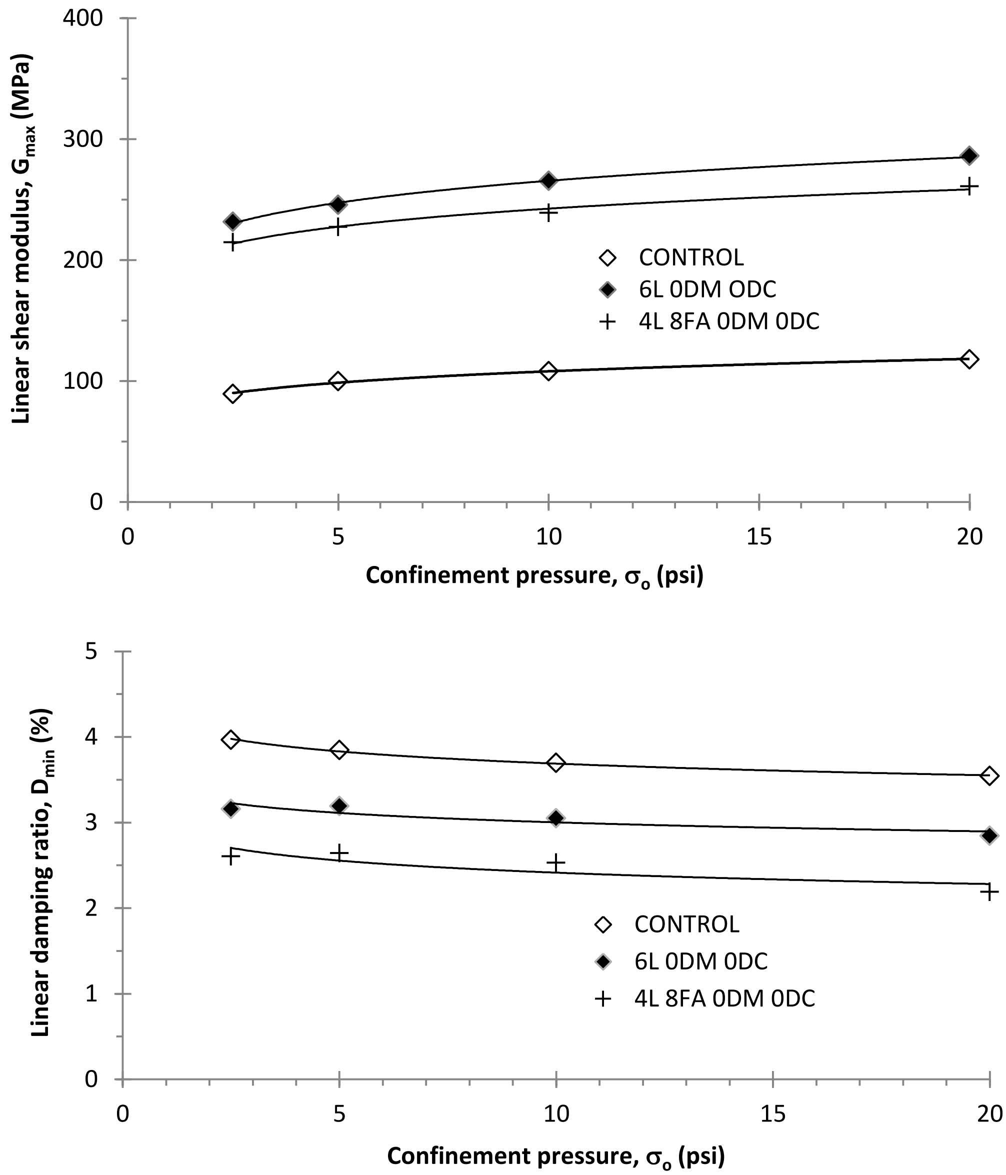
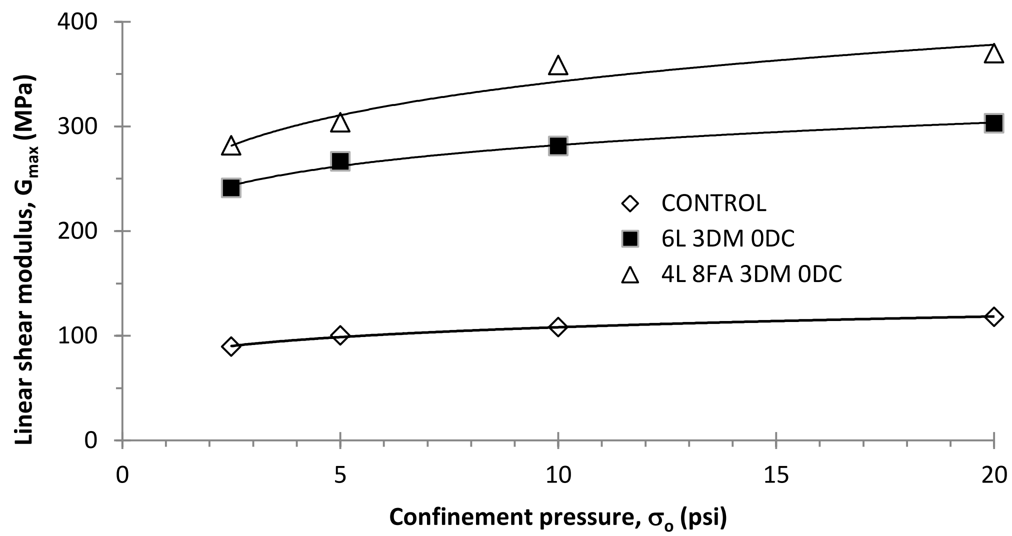


Figure 5-23 effect of treatment method on stiffness properties of sulfate-rich soil with 0 days of mellowing



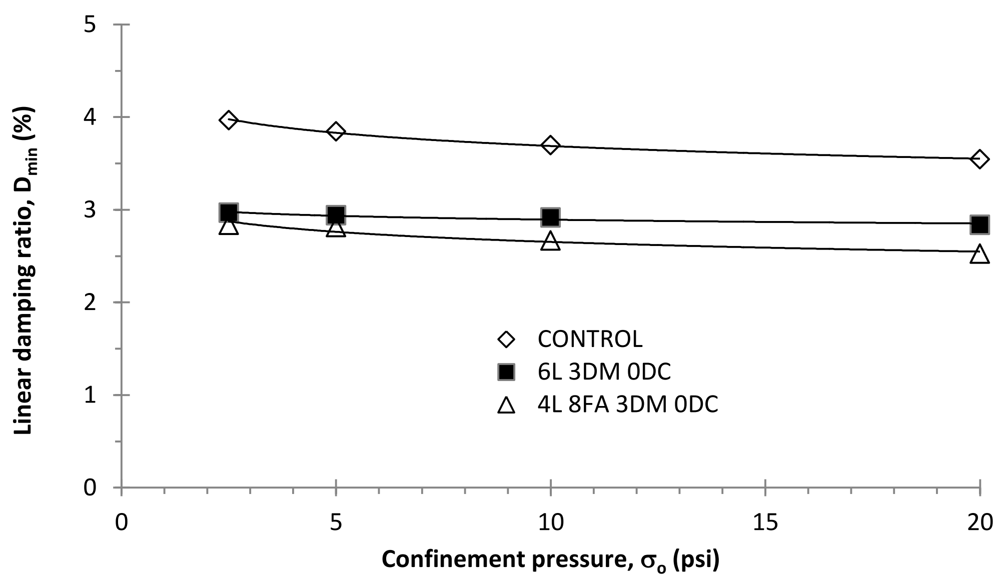


Figure 5-24 effect of treatment method on stiffness properties of sulfate-rich soil with 3 mellowing period

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5.5 Non-Linear Soil Response at Mid-to-High-Amplitude Shear Strains

5.5.1 Typical frequency response

During high-amplitude testing, cyclic shearing strains exceeding the threshold strain are applied to the soil specimen, and it goes significantly into the nonlinear range where stiffness soil properties become strongly strain dependent. In this study, the test specimen were subjected to different input-voltage amplitudes generated by the dynamic analyzer (0.25, 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, and 5.0 V) at the constant confining pressure of 20 psi, hence generating a range of frequency response curves with different resonant frequencies and peak strain amplitudes. Figure 5-26, 5-31, and 5-35 shows a typical best-fit back-bone curve obtained in this work.

It has been observed that the resonant frequency of a given soil specimen increases as the applied stress increases. On the other hand, for a constant state of stress, resonant frequency is expected to decrease with increasing strain amplitude due to the softening and degradation (plastification) effects experienced by the soil material under mid- to high-amplitude torsional vibration. Figure 5-26, 5-30, and 5-34 show a typical resonant frequency versus strain amplitude response obtained in this work.

5.5.2 Natural (control) sulfate-rich soil

In order to establish an average threshold limit, it is necessary to normalize the shear modulus G_{max} and the damping ratio D_{min} with respect to shearing strain (γ). Figure 5-25, 5-29, and 5-33 show the variation in normalized shear moduli G/G_{max} with shear strain (γ) for control soil, 6L-7DM-28DC, and 4L-8FA-3DM-28DC treated soil respectively.

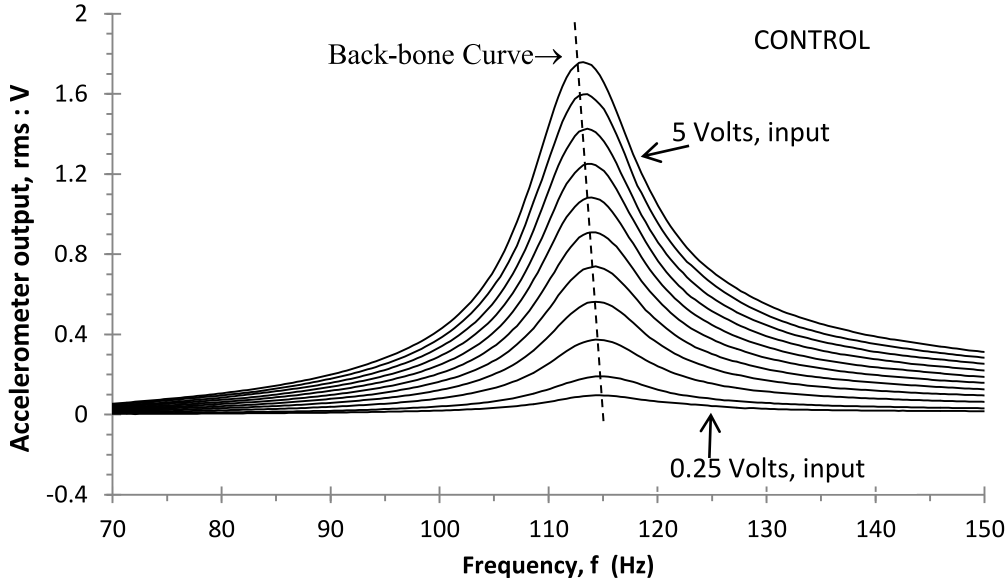

Figure 5-25 typical back-bone curves for control soil

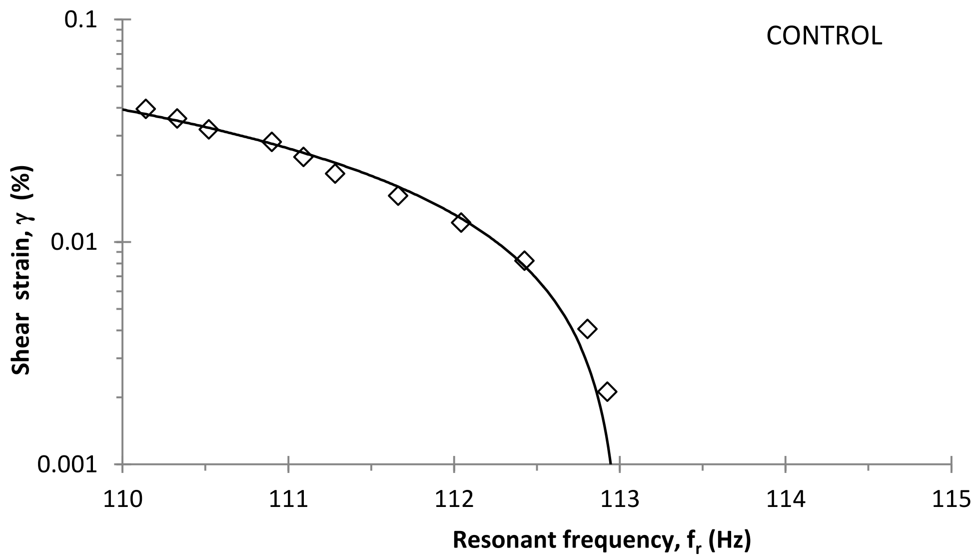

Figure 5-26 variation of shear strain over resonant frequency for control soil

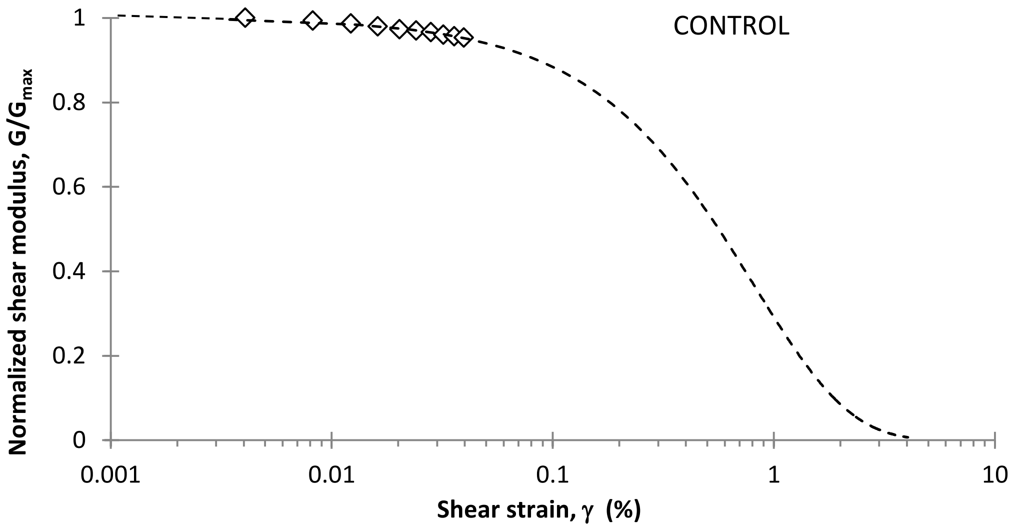

Figure 5-27 variation of normalized shear modulus over shear strain for control soil

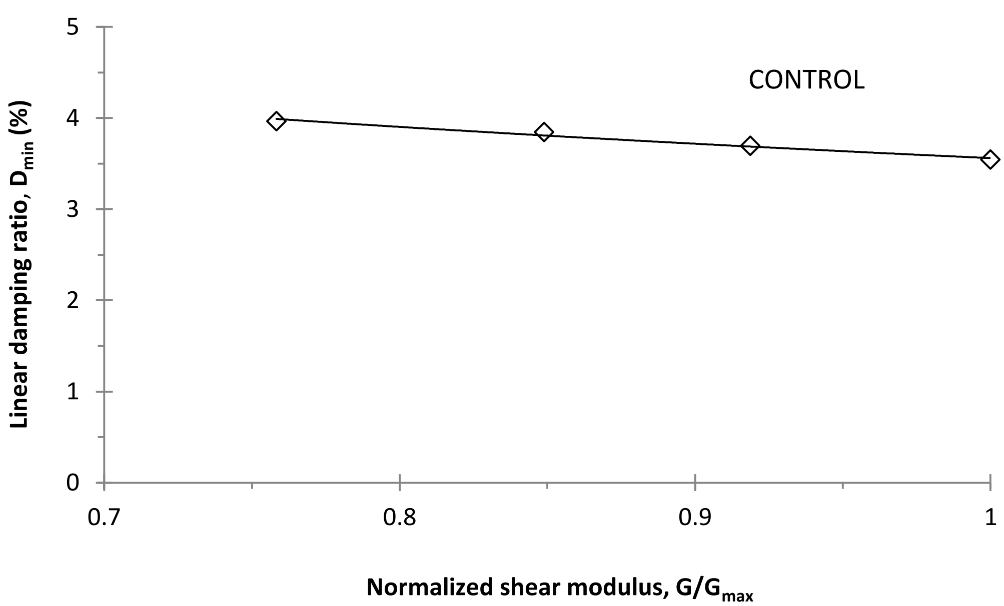

Figure 5-28 variation linear damping ratio over normalized shear modulus for control soil

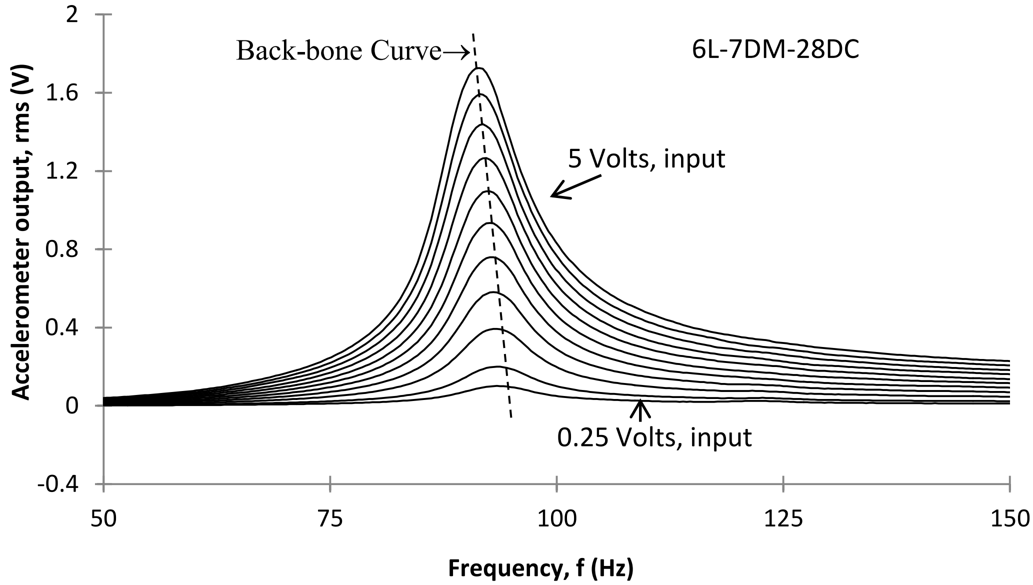

Figure 5-29 typical back-bone curves for 6L-7DM-28DC treated soil

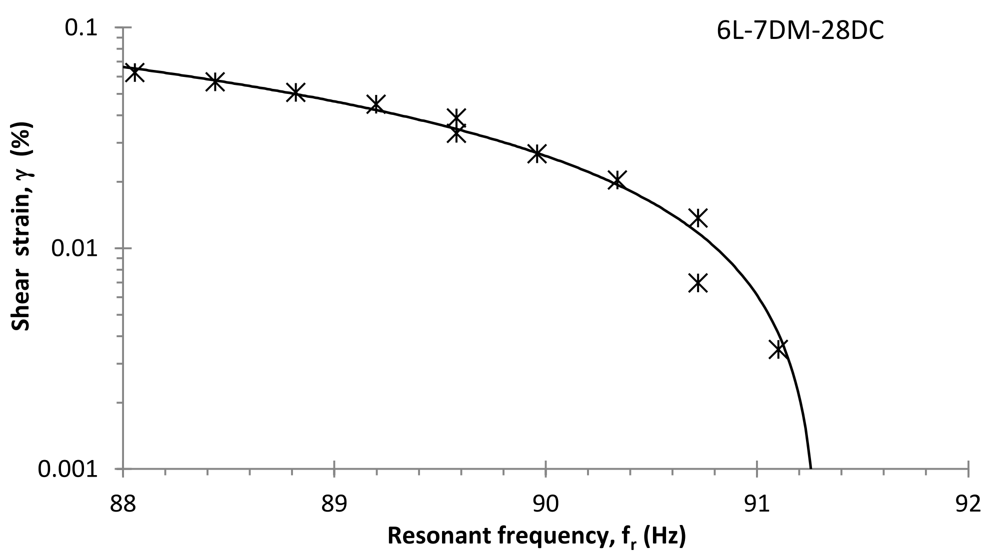

Figure 5-30 variation of shear strain over resonant frequency for

6L-7DM-28DC treated soil

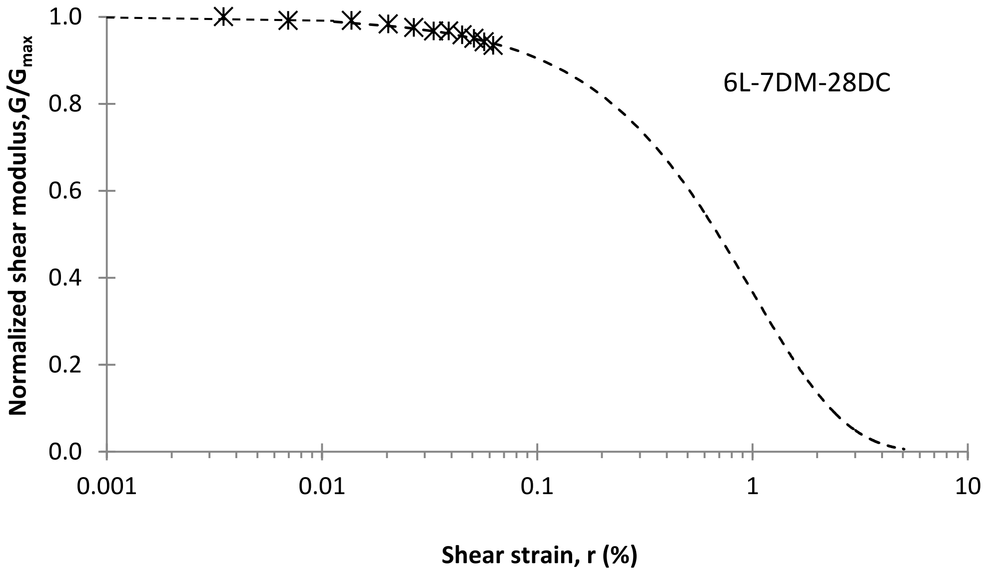

Figure 5-31 variation of normalized shear modulus over shear strain for 6L-7DM-28DC treated soil

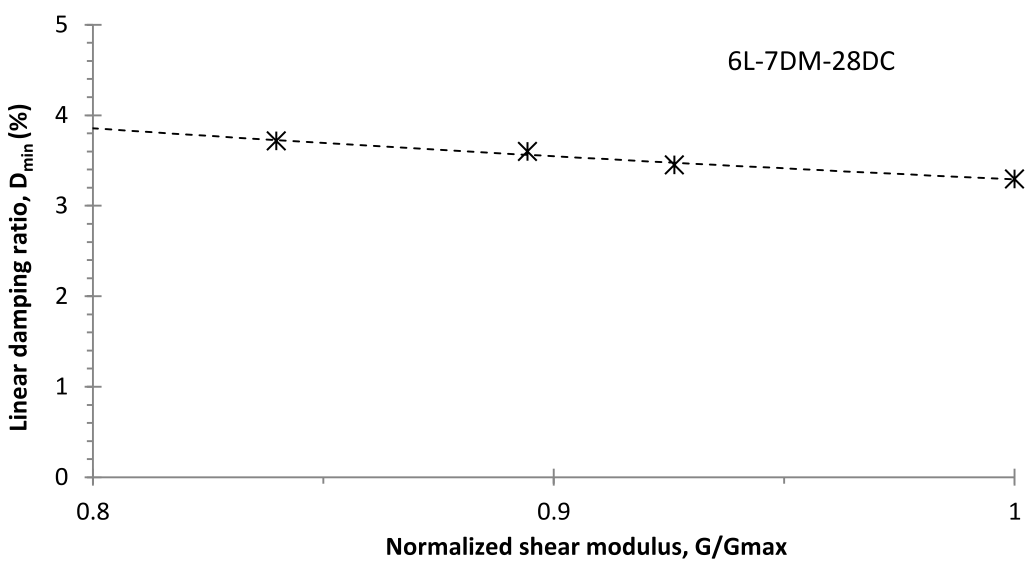

Figure 5-32 variation linear damping ratio over normalized shear modulus for 6L-7DM-28DC treated soil

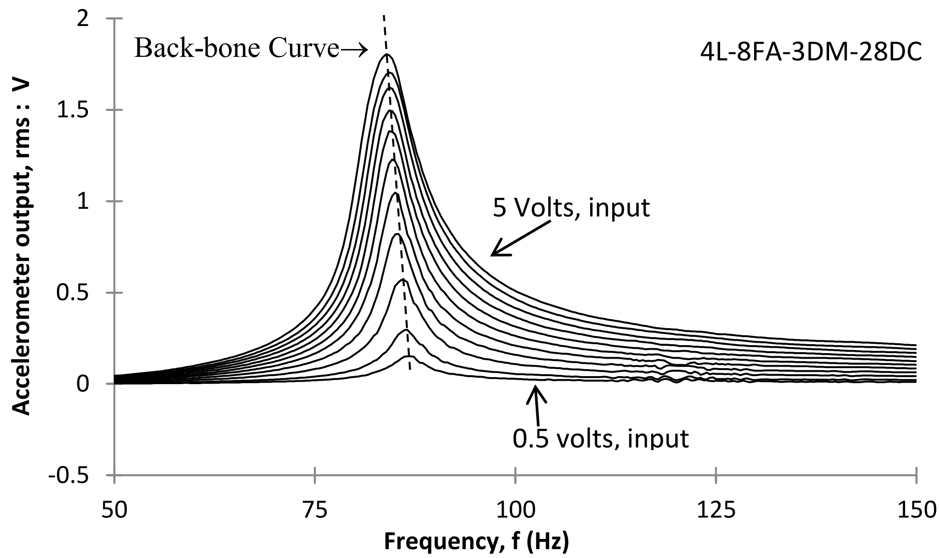

Figure 5-33 typical back-bone curve for 4L-8FA-3DM-28DC treated soil

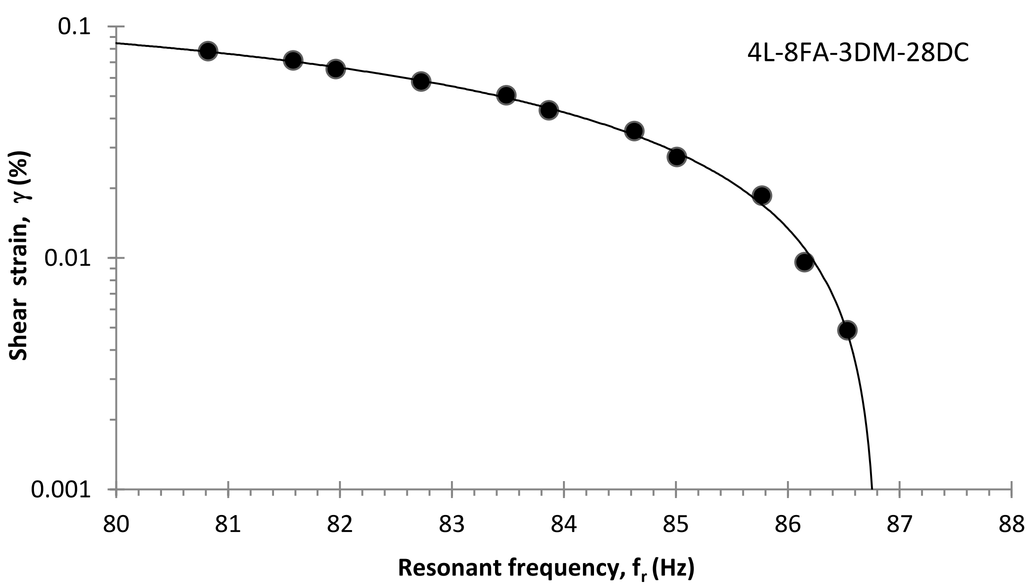

Figure 5-34 variation of shear strain over resonant frequency for

4L-8FA-3DM-28DC treated soil

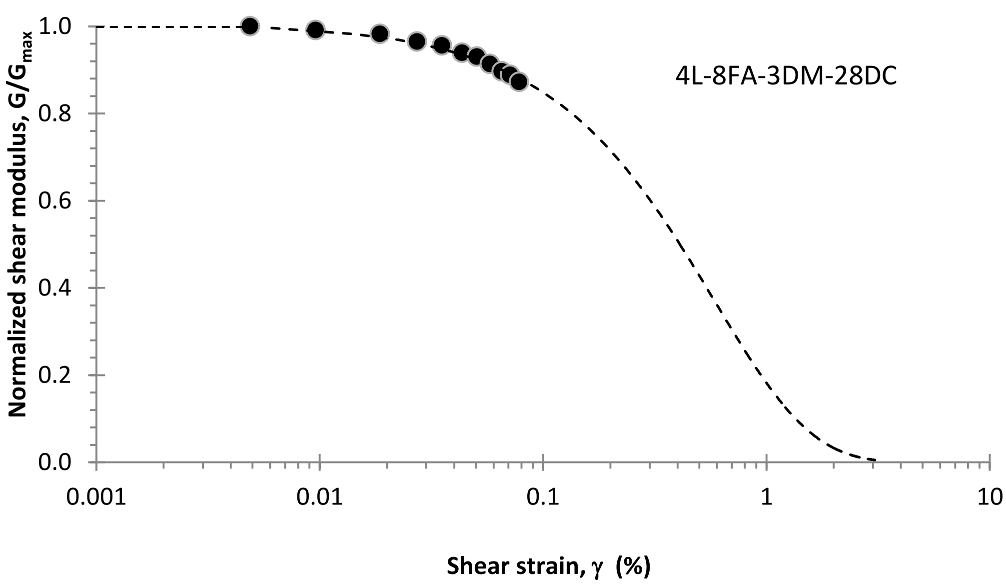

Figure 5-35 variation normalized shear modulus over shear strain for

4L-8FA-3DM-28DC treated soil

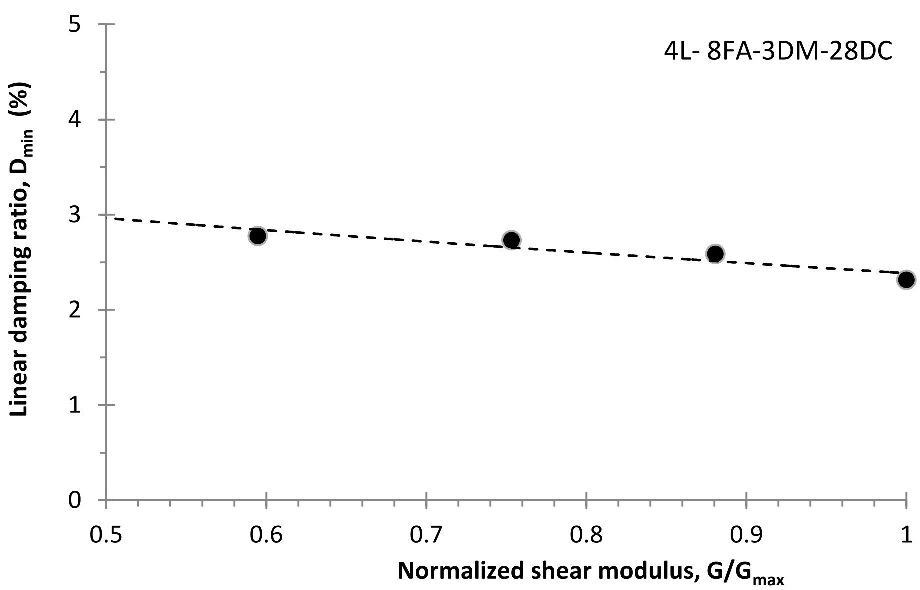

Figure 5-36 variations of linear damping ratio over normalized shear modulus for

4L-8FA-0DM-28DC treated soil

5.5.3 Influence of mellowing period on modulus degradation with shear strain

Figure 5-37 and 5-38 show the variations of G/G_{max} with shear strain (γ) and Variation of Linear damping ration (D) over normalized shear modulus G/G_{max}, respectively, for 6% lime treated soil specimens at different mellowing period. As expected, normalized values of shear moduli decrease with shear strain increases. An average threshold strain γ_{th} of 0.001 may be established. It can be observed that 6% lime treated soil with 7 days of mellowing shows higher values of shear strain followed by control, 6% lime 3 days of mellowing, and 6% lime 0 day mellowing. That is, a significant increase in mellowing period exerts a remarkable influence on the material softening.

Also, figures 5-39 and 5-40 show the variations of G/G_{max} over shear strain (γ) and Linear damping ration (D) over normalized shear modulus G/G_{max} for 4% lime + 8% fly ash treated soil at different mellowing period. Again, it can be observed that 4% lime + 8% fly ash treated soil treated soil with 3 days of mellowing shows lesser value of values shear strain. Therefore, in this treatment method, 4% lime + 8% flay ash, 3 days mellowed specimen has a better soil stiffness property than the other specimens.

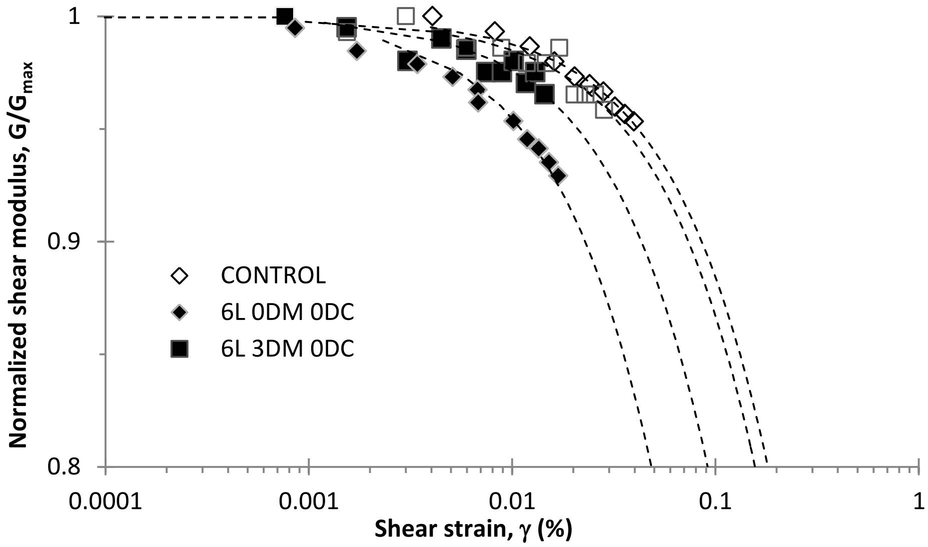

Figure 5-37 effect of mellowing period on modulus degradation for control and 6% lime treated soil

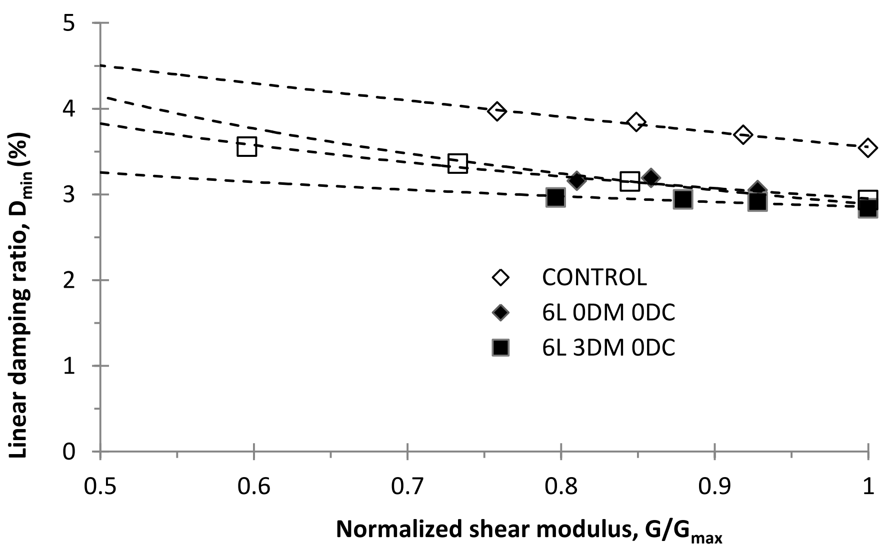

Figure 5-38 variation of linear damping ratio over normalized shear modulus for control and 6% lime treated soil

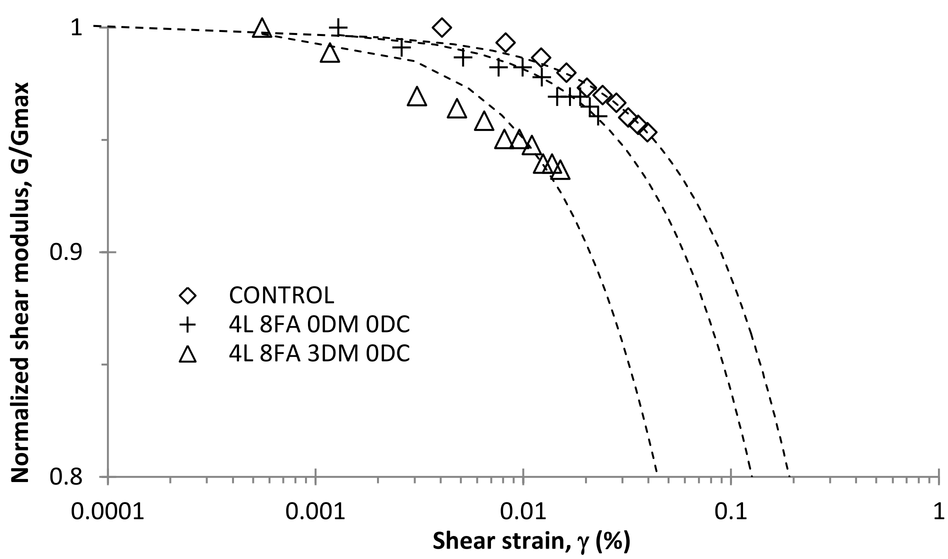

Figure 5-39 effect of mellowing period on modulus degradation for control and 4% lime + 8% fly ash treated soil

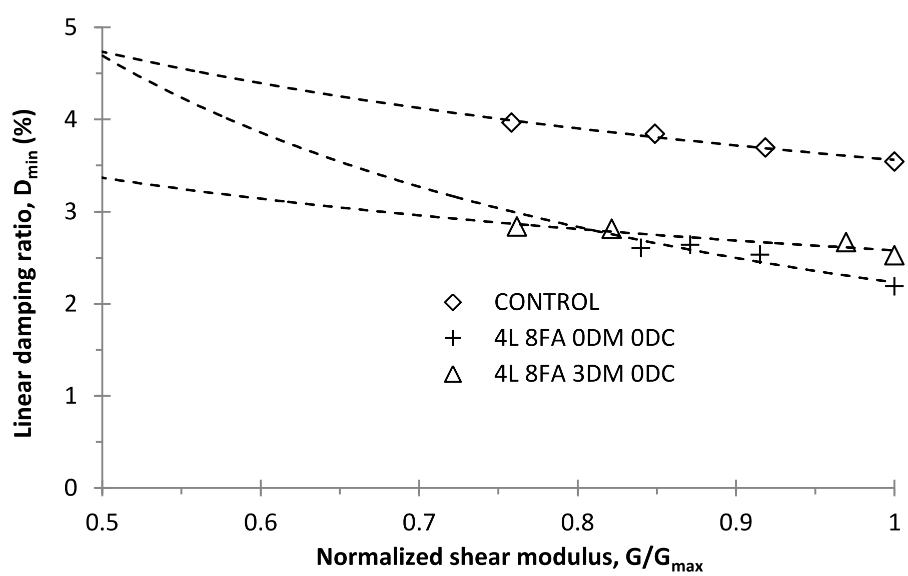

Figure 5-40 variation of linear damping ratio over normalized shear modulus for control and 4% lime + 8% fly ash treated soil

5.5.4 Comparison of treatment methods on the basis of modulus degradation

Figures 5-41 and 5-42 show the variations of G/G_{max} with shear strain (γ) and variation of linear damping ration (D) over normalized shear modulus G/G_{max}, respectively, for different treatment method. From these figures it can be observed that , the 6% lime treated soil shows a lesser value of strain, followed by 4% lime +8% flay ash treated soil, and the control soil. Therefore, it can be concluded that the 6% lime treated specimen is stiffer than the other two specimens.

Similarly, figures 5-43 and 5-44 show the variations of G/G_{max} over shear strain (γ) and Linear damping ration (D) over Normalized shear modulus G/G_{max} for different treatment methods with tree days of mellowing period. In this case, the 4% lime +8% fly ash treated soil shows a lesser value of shear strain followed by, the 6% lime treated 3 days mellowed specimen, and the control soil. Therefore, it can be concluded that the 4% lime + 8% fly ash treated soil become becomes more stiffer than the 6% lime treated specimen in the case of 3 days mellowing. This is an indication of cementation property of fly ash.

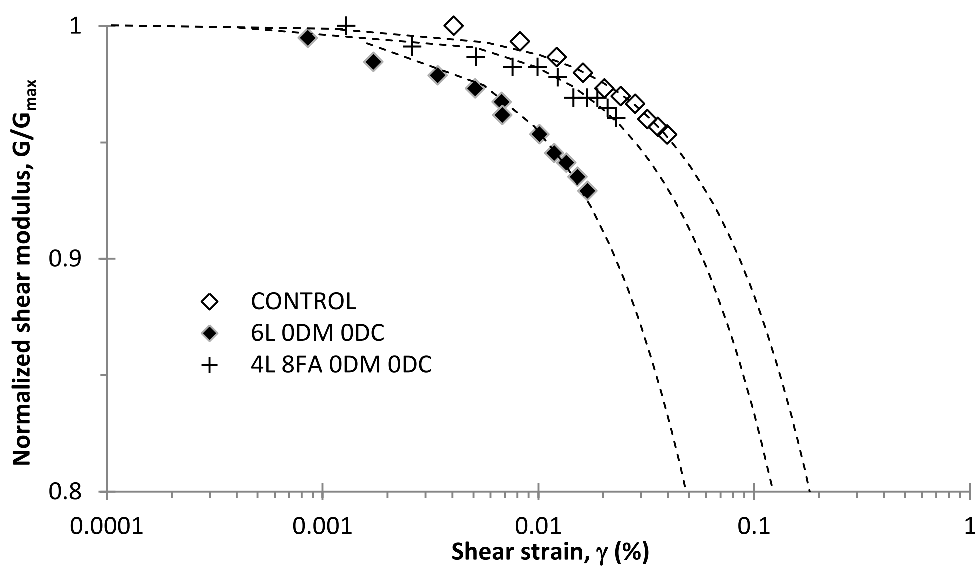

Figure 5-41 effect of treatment methods on modulus degradation for sulfate-rich soil with 0 day mellowing

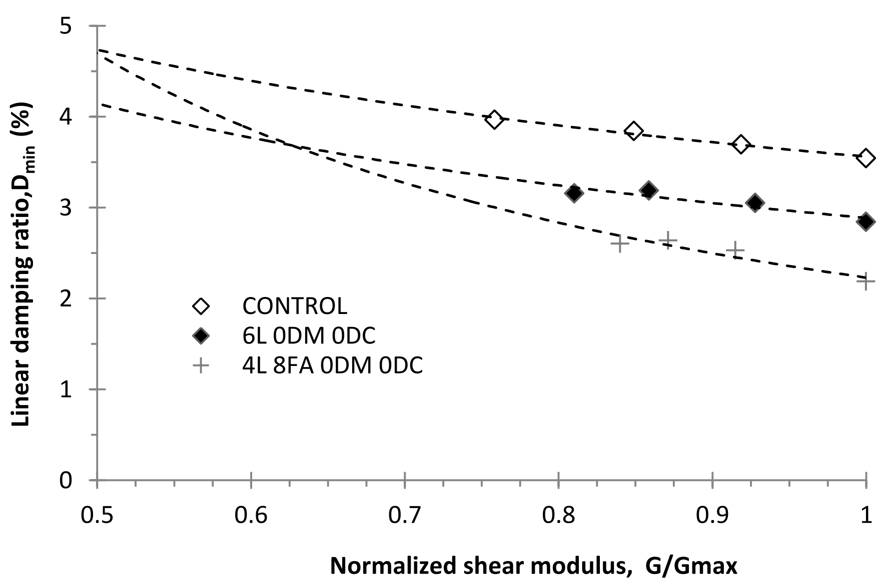

Figure 5-42 variation of linear damping ratio over normalized shear modulus for different stabilization methods with 0 mellowing day.

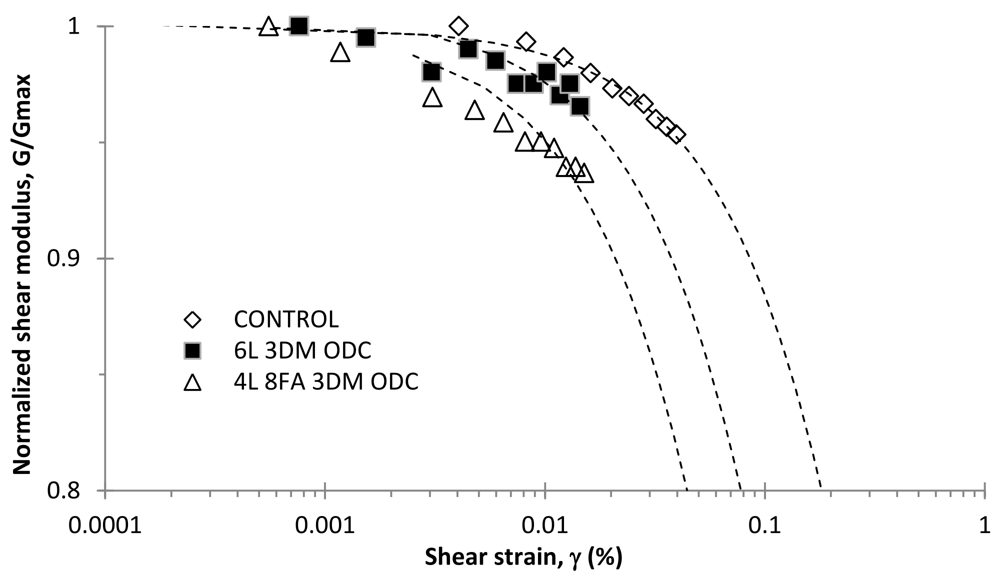

Figure 5-43 effect of treatment method on modulus degradation for sulfate-rich soil with 3 mellowing day

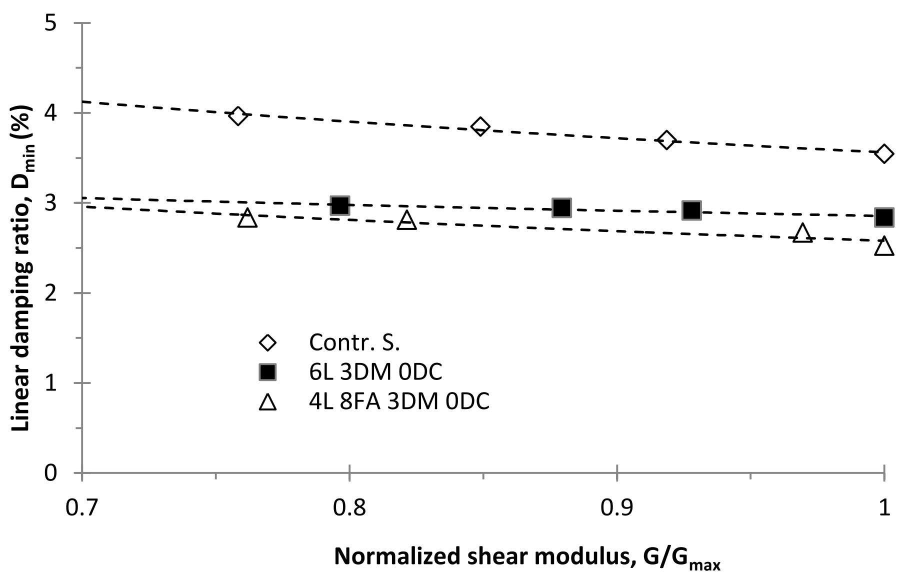

Figure 5-44 variation of linear damping ratio over normalized shear modulus for different stabilization methods with 3 days of mellowing period.

Chapter 6

Summary, Conclusions, and Recommendations

6.1 Summary

A comprehensive series of resonant column (RC) tests (ASTM D 4015-92) was conducted on several chemically stabilized specimens of high-plasticity, sulfate-rich expansive clay from Sherman, Texas. Test results were used to assess the influence of mellowing period and curing time on the stiffness properties of stabilized soil. Specimens were tested for different stabilizer, mellowing period, curing time, confining pressure, and elapsed times under constant confinement.

Two chemical treatment methods were used: 6% lime, and 4% lime + 8% fly ash. Soil stiffness parameters investigated include low-strain shear modulus (G) and low-strain damping ratio (D). Tests were also conducted at mid- to high-strain levels to study stiffness degradation effects of torsional shearing.

6.2 Main Conclusions

The main conclusions from the analysis of all RC experimental data can be summarized as follows:

1. Isotropic confining pressure (σ_o) exerts no significant influence on the low-strain (linear) dynamic and stiffness response of treated soil, regardless of the treatment method.
2. Mellowing period plays a paramount role in the low-strain (linear) stiffness response of chemically stabilized soils.
3. The 4% lime + 8% fly ash treated soil, with 3 days mellowing, yields best-performing stiffness response among all chemically stabilized soils.
4. The 6% lime treated soil, with 3 days mellowing, yields the second best-performing stiffness response of chemically stabilized soils.

5. In all cases, normalized shear moduli G/G_{max} tend to decrease with shearing strain (γ), an indication of the plastification (modulus degradation) induced on the treated soil when sheared at mid-to-high torsional amplitudes of vibration.
6. An average value for threshold strain, $\gamma_{th} = 0.0001$, can be established for all chemically treated soils.
7. The 6% lime treated soil with 7 days of mellowing period shows higher values of shear strain. As a result, it can be concluded that increase in mellowing period exerts a remarkable influence on the material softening.

6.3 Recommendations

Continuing research efforts are recommended to advance the understanding on the influence of mellowing period on the long-term stiffness properties of chemically-stabilized soil, including:

1. The use of longer curing periods, so that the cementation effects on the strength and stiffness properties can be used to predict the long-term behavior of the treated soils.
2. Further RC testing for regression-based analysis of all experimental data, including analytical relationships between soil stiffness properties, mellowing period, and confining pressure.
3. Laboratory assessment of aging effects induced by seasonal changes on long-term strength and stiffness properties of chemically stabilized high-plasticity, sulfate-rich expansive clays.
4. Field studies on subgrade materials to advance the understanding of the optimum conditions at which appropriate chemical stabilizer should work.

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