EFFECTS OF WATER-CEMENT RATIO ON DEEP MIXING TREATED EXPANSIVE CLAY CHARACTERISTICS

by

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To my Advisor and my Beloved Parents

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ABSTRACT

EFFECTS OF WATER-CEMENT RATIO ON DEEP MIXING TREATED EXPANSIVE CLAY CHARACTERISTICS

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The structures built on the unstabilized expansive soils are subjected to distress due to swell shrink behavior due to seasonal fluctuations. Medium stiff expansive clays with moderate and high PI were collected from two sites located at IH 820 N bound in Fort Worth, Texas. Deep soil mixing technique was proposed as a potential solution to counter the shrink swell movements of the expansive soil. These soils were stabilized using lime and cement as a whole and in combinations at different proportions in laboratory conditions simulating field deep soil mixing. Similar studies performed on soft soils revealed several factors mainly including binder dosage, binder proportion, curing periods and w/c ratio. The present study focuses on the effects of these factors on swell, shrink and stress strain behaviors of treated medium stiff expansive soils.

The binder dosage and proportion (Lime: Cement) has varied from 100 to 200 kg/m^3 and 100:0 to 0:100 respectively. The proportions of 100:0 and 0:100 represent 100% lime and cement respectively. The affects of above binder dosage rates and proportions on strength enhancements were studied at w/c ratios 0.8 and 1.3. All the treated samples were subjected to curing in 100% humidity room and were tested for UCS, bender elements, swell, shrink and suction after 7 and 14 days.

Results show that the unconfined compressive strength values decreased with increase in w/c ratio. Maximum strength enhancements were noted at increasing binder dosages and cement proportions in the lime: cement ratio. No significant swell-shrink movements were observed for treated specimens at both the w/c ratios. Shrinkage strain magnitudes increased with increase in w/c ratio. Shear moduli of soils treated at 0.8 w/c ratio were greater than the same at 1.3.

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CHAPTER 1

INTRODUCTION

1.1 General

Light to medium structures like pavements, single to double storey buildings, and airport runways constructed on expansive soils were subjected to cracking due to differential swell-shrink movements from seasonal moisture fluctuations. Expansive soils contain clay minerals such as montmorillonite, vermiculite, illite expand when they are hydrated and shrink when they are exposed to drying. Montmorillonite is considered to be the predominant clay mineral associated with expansiveness and can be found in most of the expansive soils (Snethen et al. 1975).

The factors that influence variations in moisture content are climatic environment, vegetation, drainage at a given site, amount and type of clay minerals, clay activity, thickness of expansive soil layer and active zone changes at the site due to human activity and ground water table location (Skempton 1953, Simmons 1984 and Francis 1996).

It is estimated that in United States, approximately 20 % of area is underlain by expansive clayey soils and the costs associated with damage caused by these soils amounts to about $7 billion per year, which is more than twice the damage from flood, hurricanes, earthquakes etc. (Jones and Holtz 1973 and Krohn and Slosson 1980). These soils are regarded as problematic soils and can be found mostly in Texas and in areas along the Gulf coast, Appalachian states and Great Plains (Krohn and Slosson 1980). Figure 1.1 depicts the regions in United States with expansive soils. Jayatilaka (1999) reported several research works that attempted for better understanding of expansive soils behavior and methods such as alteration of expansive material by mechanical, chemical or physical means and control of moisture fluctuations by placing horizontal (slabs) and vertical (impermeable fabric membranes) barriers. But most of them or all of these methods were proved to be ineffective in long term, as the improvement is only up to shallow depths.

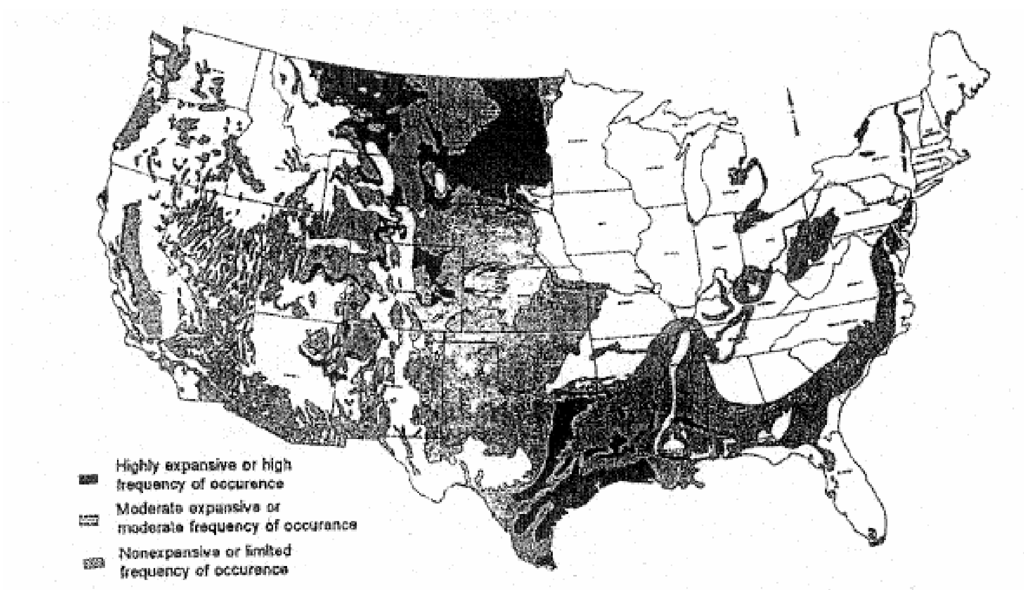


Figure 1.1 Distributions of Expansive Soils in the United States (U.S. Army Engineer Waterways Experiment Station, 1977)

1.2 Problem Statement

In Fort Worth, Texas, the freeway IH 820 N was underlain by expansive soil. The active zone, according to PVR method, was estimated to be approximately 12 ft. As a result of swell-shrink behaviors of underlying active expansive soil, the pavements constructed on these soils will be subjected to distress resulting in surface unevenness extending to considerable lengths (Figure 1.2). This in turn, causes riding discomfort and increases maintenance costs of the free way.

Because of these reasons, the University of Texas at Arlington (UTA) is currently conducting a research study of improving or stabilizing the expansive soils at IH 820 N, Fort Worth, Texas through in situ deep soil mixing (Deep Mixing Technology). Till recently, this technique has been applied for stabilization of soft soils and is a popular ground improvement technique in both Scadinavian countries and Japan. As a part of this research, the present thesis work conducted an experimental program simulating the deep soil mixing process in laboratory environment and results and evaluated the stress-strain behaviors of lime-cement treated expansive soils with respect to soil type, binder dosage, binder proportion, water/cement ratio and curing period. The locations of proposed sites and sampling points are depicted in Figure 1.3.

Figure 1.2 Longitudinal and Transverse Cracks Resulting from Pavement Distress at IH 820 N, Fort Worth, Texas

1.3 Deep Mixing Technology

Deep soil mixing techniques, which were developed in the 1960s, were first reported in literature during early 1970s (Broms and Boman, 1979; Holm et al. 1981; Rathmayer, 1996; Okumara, 1996; Kamon, 1996; Porbaha, 1998). Deep mixing (DM) technology involves the auger mixing of soils extending to large depths with cement, lime, or other types of stabilizers.

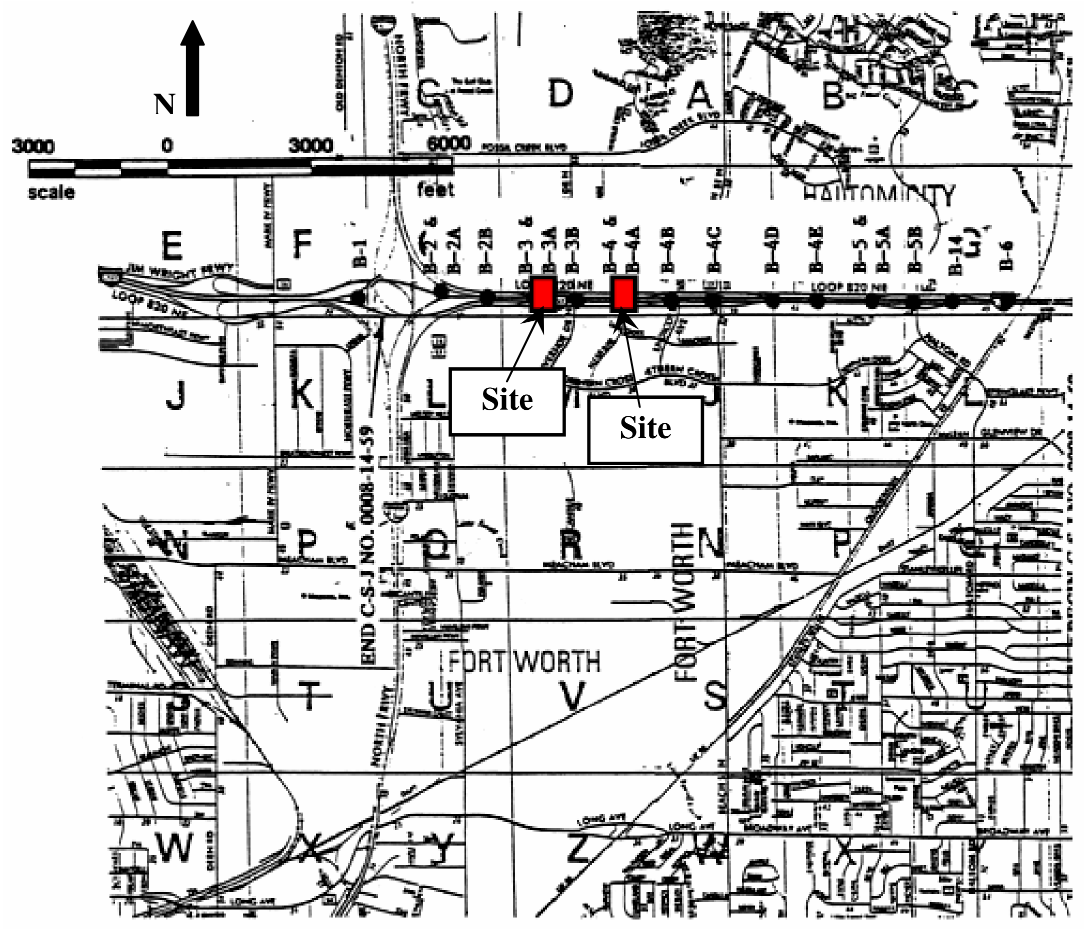


Figure 1.3 Sampling points and location of test site on north bound IH 820 N, Fort Worth, Texas

DM method is a ground modification technique that improves the quality of ground by in situ stabilization of problematic soils (Porbaha, 1998). The stabilizing process typically takes place by mechanical dry mixing or by wet mixing (Rathmayer, 1996; Porbaha, 1998; Holm, 1999). Wet mixing is recommended for dry and arid environments or sites with deep water table and dry mixing for sites with high water table. In projects where soil compressibility properties need to be enhanced to reduce undesirable settlements, either lime or combinations of lime with cement or other additives are typically used in the DM treatments. In the case of arid and semi arid regions with deeper active depth, shallow stabilization of subgrade soils is not considered since it is neither practical nor economical to stabilize the subsoil to such great depths. Several other stabilization strategies including stone columns and soil replacement method were explored to stabilize expansive soils. Deep soil mixing is considered one such method, which needs to be researched as a prospective stabilization technique for expansive subgrades with considerable active depths to support pavements or roadways and embankments.

Deep mixing columns can be formed in different configurations such as isolated columns, compound columns, panels, and grids. All these configurations are used in different site conditions based on site soil characteristics, project requirements, load transfer mechanisms and settlement characteristics (Bruce and Bruce, 2003; Puppala, 2003). For example, isolated columns are used in areas where the design area ratio (ratio of areas of treated soil to an untreated soil) is less than 40 to 50%. Compound columns are used when the design area ratio at the site is higher than 50%. Panels and grids are also used in high area ratio environments and when superstructures are large in size such as embankments, dams and retaining walls. In highway applications, typically single or multiple columns are used for stabilization.

1.4 Thesis Organization

Chapter 1 introduces the general nature and the problems associated with structures on expansive soils. The later part deals with the problem statement followed by introduction of deep soil mixing as a prospective alternative for deep stabilization of expansive subgrades in lieu of the conventional shallow stabilization techniques in practice. Much detail description of the deep mixing method is discussed in chapter 2.

Chapter 2 presents an overview of literature review on the nature of clay behavior and various methods developed to predict the vertical movement in expansive soils. This is followed by various developments adopted for the stabilization of expansive clays for the past few decades. Later part of the chapter deals with deep soil mixing in detail in the field and its application areas followed by some laboratory studies such as the effects with varying parameters were discussed.

Chapter 3 explains the work plan in flow chart, experimental program, research variables studied, sample preparation, laboratory instrumentation and the development of test procedures. The calculations involved in arriving at per batch weights (batch of 4 samples) of dry soil, natural water content, binder dosage and two water-cement ratios are explained in detail along with a sample calculation.

Chapter 4 presents the summary of test results. These results were analyzed and compared with the previous research work done by Bhadriraju (2005). The performance of each stabilizer is assessed as a function of dosage rate, lime to cement or L:C ratio and curing period. Based on the results, the best combination of water-cement ratio, dosage rate and proportion is given the highest rank and is proposed for latter part of the research involving design and field implementation of deep mixed columns to stabilize expansive soils.

Chapter 5 presents the conclusions and recommendations from the current study.

CHAPTER 2

LITERATURE REVIEW

2.1 General

Expansive soils include clays and very fine silts shrink as their moisture content decreases and swell as their moisture content increases. Shrink swell movements in expansive soils have historically caused wide spread problems with respect the serviceability performance of light weight structures supported on shallow and relatively flexible footing systems. Shrinking and swelling of unsaturated expansive clays in response to water content change may be considered as a world wide phenomenon. It is one of the most common geotechnical causes of damage to residential buildings and other civil infrastructure including transportation infrastructure (Jones and Holtz 1973; Krohn and Slossen 1980; Freeman et al. 1991).

2.2 Behavior of Expansive Soils

Significant attention on clay behavior from the past few decades has been discussed in this section. Casagrande (1932) wrote one of the earliest treatises, which discussed clay structure and its impact on foundation engineering. Simpson (1934) published a paper describing experiences in foundations on clay in Texas. Further, observations explained the moisture variations beneath Texas pavements and the effects associated. Skempton (1953) described the effect of clay activity ratio on swell potential of expansive clays. Palit (1953) described a method to determine swell pressure of black cotton soils. Also, Jennings and Knight (1957) described how heave can be predicted using oedometer test data, and Felt (1953) described the influence of vegetation on the moisture content of clays. Altmeyer (1955) and Holtz and Gibbs (1956) discussed engineering properties of expansive clays. McDowell (1959) published a paper on the relationships between laboratory testing and design of pavements and other structures on expansive soils.

Later in 1960's, Lambe (1960) developed a method to identify swelling clays and to assess their swelling potential via the potential vertical rise (PVR) meter. Seed et al. (1962a) discussed swelling characteristics of compacted clays. Seed et al. (1962b) published information on how to predict swell in compacted clays. Chen (1973), Lytton (1994, 1995, 1997), and Wray (1978, 1984, 1987) established new approaches for slabs on grade built on expansive clays. Johnson (1973, 1977), Johnson and Snethen (1978), McKeen (1980, 1981), Snethen (1979, 1984), and Snethen et al. (1975) developed the concept of soil suction and methods to measure it. Vijayvergiya and Ghazzaly (1973) started the process of creating predictive models. Tucker and Poor (1978) studied the behavior of existing slabs and assessed the causes of damage in them. Mitchell (1976) provided the profession with a landmark and comprehensive text on soil behavior.

The Post-Tensioning Institute (PTI) published its first manual on the design of post-tensioned slabs on expansive clays (PTI 1980). Chen (1988) presented his views on foundations in expansive soils. Dempsey et al. (1986) used soil suction to estimate moisture content in the subgrade soils in a moisture equilibrium model. Lytton (1994) described how simple laboratory tests can be used to determine important properties of expansive soils. McKeen and Johnson (1990) described simple, rational methods to calculate the active zone depth and slab-edge penetration distance in clay soils. McKeen (1992) was also able to classify the heave potential of expansive soils on the basis of the ratio of suction change to moisture content change and the soil compression index. Snethen and Huang (1992) described soil suction-heave prediction methods. Houston et al. (1994) provided further information on the use of filter paper to measure suction.

2.3 Characterization of Expansive Soils

In order to select effective treatment alternatives for a foundation in expansive soils, the two most important factors are identifying the expansive soils and estimating the potential volume change. The available methodologies of identifying expansive soils include mineralogical (x-ray diffraction, differential thermal analysis, infrared analysis. dye adsorption, cation exchange capacity), physical properties (Atterberg Limits, colloid content), and soil classification systems (Jayatilaka 1999). As an expedient methodology for identifying potentially expansive soils, Snethen(1979c) recommends the U.S Army Corps of Engineers Waterways Experiment Station (WES) Classification system which is given in Table 2.1.

In order to quantitatively characterize the expansive soils, numerous methods have been developed in the past. As the main cause of damage to the structures built on expansive soil is due to the volume change behavior of such soils, in all the methods available for quantitative characterization involve the estimation of swell pressure or percent swell. The techniques fall into three categories: a) oedometer tests, b) empirical methodologies, and c) soil suction tests.

Table 2.1 WES Classification of Potential Swell

Liquid Limit%	Plasticity Index%	Natural Soil Suction, kPa	Potential Swell
>60	>35	>383.0	High
50-60	25-35	143.6-383.0	Marginal
<50	<25	<143.6	Low

2.3.1 Oedometer Tests

Oedometers can be used to estimate either the swell pressure or the amount of free vertical swell depending on the structure being, then the swell pressure is measured. If the applied load is large and the structure is regid, then the swell pressure is measured. If the applied load is light and the structure is relatively flexible, then the amount of swell is measured. A large number of oedometer testing procedures have been proposed by many researchers. Two basic types of oedometer swell tests are the consolidation swell test, and constant volume or swell pressure test (Snethen 1979c; Nelson and Miller 1992). In the consolidation-swell test, the sample is allowed to swell freely in vertical direction under a seating load and then the sample is allowed to swell under that load when water is added. After swelling, the sample is further loaded until the initial void ratio is reached. Then the specimen is rebounded in decrements and the final void ratio is measured. The swell pressure is defined as the pressure required to recompress the fully swollen sample to its original volume. The amount of swell is calculated from the following relationship;

$$\frac{\Delta H}{H} = \frac{e_f - e_o}{1 + e_0}$$

where

e_o = initial void ratio,

e_f = final void ratio,

ΔH = heave, and

H = layer thickness.

In the constant volume test, the sample is inundated while preventing the sample from swelling. Loads are added at regular intervals to keep the sample from swelling. Here, swell pressure can be defined as the maximum applied stress required to maintain constant volume. The swell measurements obtained from oedometer results have been compared with the actual measured heave in the field in Sudan and Saudi Arabia (Osman and Sharief 1987). Both the studies predict that the oedometer methods overestimate the insitu heave.

2.3.2 Empirical Methods

These methods are based on the correlation between laboratory or field measurements and soil indices such as liquid limit, plasticity index and clay content. Though, there are large numbers of these equations presented in the literature, the use of such equations on a global basis is questionable (Puppala et al. 2003). Snethen (1984) estimated the percent swell of 20 expansive soil samples using 17 published equations and compared them to the values obtained from laboratory tests. The conclusion was that only four equations showed a balance with respect to their accuracy and conservatism. Rao and Smart (1980) evaluated four such equations using 10 different soils and showed that none of the equations considered were able to predict the swell accurately. They concluded that a strict test of similarity (geological, mineralogical and textural) was in needed in developing and using such equations. Zein (1987) applied five empirical equations to predict both swell percent and swelling pressure of nine Sudanese black compacted residual black cotton soils and compared the laboratory results. He concluded that with the exception of one swell percent equation, none of the considered equations yielded acceptable predictions. In Texas the Potential Vertical Rise (PVR) method may be most widely accepted empirical procedure used in the estimation of volume change behaviour of expansive soils. The procedure was developed by correlating volumetric swell with basic soil properties (McDowel 1956).

2.3.3 Soil Suction Tests

Methods available for measurement of suction in soil include the (a) filter paper method, (b) thermocouple psychrometer, (c) thermal moisture sensor, (d) tensiometer, (e) vacuum desiccator, and (f) pressure plate apparatus (Ridley and Wray 1995). Hamberg (1985) presented the following model to estimate the vertical movement:

$$\sum_{i=1}^{N}\left[\frac{H_i}{(1+e_0)_i}\right]*[C_h * \Delta\log(h)]_i$$

where

ΔH = vertical movement,

N = number of layers to depth of active zone,

H_i = thickness of layer I,

e_0 = initial void ratio of layer i,

C_h = suction index with respect to void ratio (slope of void ratio verses soil suction in logarithmic scale), and

h = soil suction (total and matric).

In terms of water content, the above model takes the following form (Hamberg 1985):

$$\Delta H = \sum_{i=1}^{N}\left[\frac{H_i}{(1+e_0)_i}\right]*[(C_w * \Delta w)_i]$$

where

C_w = modulus ratio (slope of void ratio versus water content), and

Δw = change in water content

Various other models developed by using soil suction as the parameter to estimate the vertical heave are reported in the literature by Snethen (1979), Miller et al. (1995), and Lytton (1977 and 1995).

2.4 Stabilization of Expansive Clays

Barshad (1950) a mineralogist, discussed the effects of interlayer cation types on expansion of clays. Allison, Kefaumer, and Roller (1953) discussed ammonium fixation in clay soils. Dubose (1955) described how to control heavy clays using compaction. Jones (1958) discussed stabilization of expansive clay using hydrated lime and portland cement. Taylor (1959) explained the process of ion exchange in clays. McDowell (1959) described how Texas soils were being stabilized with lime and lime-fly ash combinations. Hilt and Davidson (1960) discussed lime fixation in clays heralding a decade of advancing technology in lime and portland cement stabilization of clay soils. Mitchell and Hooper (1961) described the effect of time of mellowing on the properties of lime-modified clay soils and noted that this mellowing period affected the workability and compaction response.

Eades and Grim (1963) developed their quick test to determine required lime content. Eades and Grim (1963) identified lime-soil reaction products. Mateos (1964) described the stabilization of soils with lime and portland cement. Davidson et al. (1965) described the effects of pulverization and lime migration in treated soils, while Townsend and Klym (1966) discussed the durability of lime-stabilized soils. Diamond and Kinter (1965) published their findings of the mechanism of soil stabilization with lime. Diamond and Kinter's (1965) classic paper clarifies the need for the pozzolanic surface reaction to achieve strength gain as well as plasticity reduction in plastic clay soils, especially those that are calcium-dominated prior to stabilization. Lundy and Greenfield (1968) were the first to describe the process of stabilization by high pressure lime-slurry injection. Higgins (1965) described the process of deep in situ soil stabilization.

In early 1970's Arman and Munfakh (1970) described how lime can treat even highly organic soils. Marks and Haliburton (1972) described the acceleration of lime reactions with the addition of salt. Thompson (1972) described deep-plow lime stabilization for pavement construction. Carroll and Starkey (1971) published their findings on reactivity of clay minerals with acids and alkalines. Thus, the use of chemical agents other than lime to improve clay soils has been introduced. O'Bannon et al. (1976) described the use of electroosmosis and potassium to improve a clay highway subgrade. Later in 1970's Lee and Kocherhans (1973) described the use of moisture barriers to stabilize clays.

Kennedy (1988) and Kennedy and Tahmovessi (1987) provided summaries of the effective use of lime in clay soils. Little (1987) described the fundamentals of lime-soil reactions for the National Lime Association, and the Transportation Research Board (1987) published its *State of the Art Report 5* on lime stabilization of soils. Petry and Lee (1989) discussed the benefits of using quicklime slurries over hydrated lime slurries in the treatment of clays, and Petry and Wohlgemuth (1989) described the effect of degree of pulverization during mixing on the products of lime and portland cement treatment of heavy clays. Mitchell (1986), in a Terzaghi lecture, reintroduced the profession to calamities associated with sulfate-induced heave in lime stabilized clay soils.

2.5 Ground Improvement- Deep Mixing Technology

2.5.1 Stabilizers used for Ground Improvement

The purpose of ground improvement is to stabilize the soil which lacks the strength to carry the design loads of the structure or to increase the required safety factors of the ground and its stability (Kamon, 1996). The stabilizers in practice used for stabilization of clay subgrades and subbases were divided as follows.

i) Traditional stabilizers such as hydrated lime, Portland cement, and fly ash.

ii) Byproduct stabilizers such as cement kiln dust, lime kiln dust, and other forms of byproduct lime.

iii) Nontraditional stabilizers such as sulfonated oils, potassium compounds, ammonium chloride, enzymes, polymers and so on.

Various uncertainties were encountered with the byproduct stabilizers and nontraditional stabilizers. For example, for stabilizing subgrades they are confined either to stabilize volume change characteristics or modify plasticity or workability. On the other hand, traditional stabilizers such as lime and Portland cement were considered to be apt in this case which adopts the subgrade layer not only be volumetrically stable, but also supports traffic or building loads. Thus, lime and cement may be considered as the best suited and top ranked stabilizers for most of these processes. A brief overview of these treatments can better explain their importance.

2.5.1.1 Lime Treatment

The forms of lime most frequently used for stabilizing many types of soils are hydrated high calcium lime $Ca(OH)_2$, monohydrated dolomitic lime $Ca(OH)_2.MgO$, calcite quicklime CaO, and dolomitic quicklime $CaO.MgO$ (Carlos, 2000). A simplified qualitative representation of typical soil-lime reactions is shown below (Mitchell and Dermatas 1992):

$$Ca(OH)_2 \longrightarrow Ca^{2+} + 2OH^- \quad (1)$$

$$Ca^{2+} + 2OH^- + SiO_2 \text{ (Clay Silica)} \longrightarrow CSH \quad (2)$$

$$Ca^{2+} + 2OH^- + Al_2O_3 \text{ (Clay Alumina)} \longrightarrow CAH \quad (3)$$

where $C = CaO$, $S = SiO_2$, $A = Al_2O_3$, and $H = H_2O$

The improvements in the engineering properties of the treated soil include increase in strength, reduction in plasticity index (PI), reduced swell/shrink potential etc. The swell/shrink potential is attributed to a decreased affinity for water of the calcium-saturated clay and the formation of a cementitious matrix that resists volumetric expansion (Lime Stabilization 1987).

2.5.1.2 Cement Treatment

Portland cement is most commonly used for the ground treatments. Portland cement is mainly composed of C_3S, C_2S, C_3A crystals, and a solid solution described as C_4AF (Herzog and Mitchell 1963). Carlos (2000) observed two stages of reaction in soil-cement mixtures. The first is independent of soil type and occurs immediately upon contact of cement with water. At the second stage of reactions, the high pH ($pH > 12.4$) may cause alumino-silicates in clays to dissolve and then be available to combine with calcium to form secondary cementitious products.

In order to minimize the effects of the problematic soils, improve workability, it is derived from previous literature that the combination of lime and cement is preferred as the best stabilization mixture. Up to late 1970's, unslaked lime was only considered as a suitable binder for soft clay materials. Mixtures of lime and cement were best proven from early 1990's and replaced the technology of feeding purely unslaked lime. The change in the admixtures can be seen in Figure 2.1 for various deep mixing applications in Sweden can best explain the performance of lime-cement binder mixture.

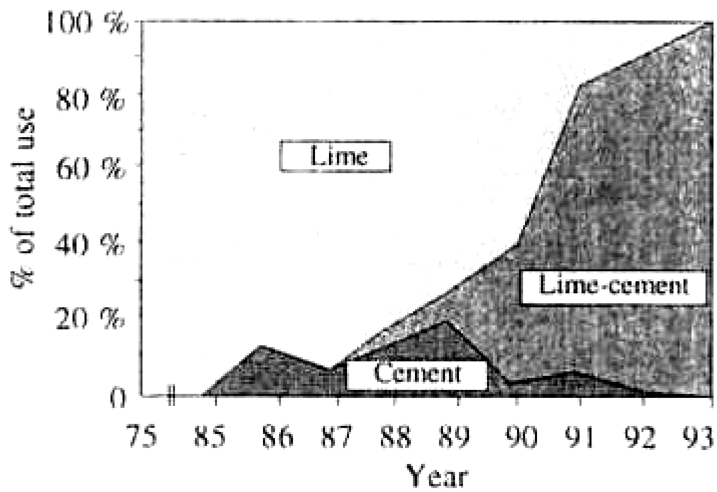


Figure 2.1. Change in admixture types used in Sweden (Rathmayer 1997)

2.5.2 Deep Soil Mixing

A number of ground improvement methods have been developed and were in practice for various civil engineering works. Solidification may be considered as one of the ground improvement techniques, which is being practiced more in civil engineering projects. Soils are stabilized with various types of add mixtures in order to enhance the engineering properties like strength, volume change potential and permeability characteristics. As discussed in the earlier section, the additives, which are most commonly in use, are lime, cement, fly ash, and combinations of them. The solidification technique is having wide range of applications in both shallow and deep ground improvements. The principle function of this solidification process is to embed cementation materials into the soil pores by injecting admixtures. Thus, freezing the pore spaces and form tight inter particle bonding.

Deep mixing and grouting were noted as most common and effective solidification methods. Jet grouting, where the injected area can be controlled may be considered as advancement to normal grouting. Deep soil mixing techniques, which were developed in the 1960s, were first reported in literature during early 1970s (Broms and Boman, 1979; Holm et al. 1981; Rathmayer, 1996; Okumara, 1996; Kamon, 1996; Porbaha, 1998).

Deep soil mixing is an *in-situ* mixing methodology that mixes existing soil with cementitious, chemical or even biological materials in the form of slurry or powder. The binders to be considered in this technique depend on the type of the project. Lime, cement and combination of both are the most frequently adopted binders in the deep mixing process (Puppala 2003). Cement stabilization provides substantial strength increase in a short time frame, due to cement hydration and pozzalonic reactions, cementation and agglomeration, as well as ionic exchange and flocculation mechanisms (Sherwood, 1995; Hosoye et al. 1996). This stabilization is quite effective on soft clays, peats, mixed soils, and loose sandy soils (Hausmann, 1990; Rathmayer, 1996; Porbaha, 1998; Bruce, 2001).

2.5.3 Description of Method in the Field

An overview of the DSM method can be obtained in the CIRIA publication (Harris et al., 1995). The technique involves mixing inplace soils with cement grout or other reagents in slurry form using special augers, equipped with mixing paddles that mix up the soil as they rotate. The main differences between DSM and jet grouting are that in DSM there is no high pressure used and that DSM columns are made to specific dimensions (Munfakh, 1997). Arranging the soil-cement columns in different configurations gives the technique a wide range of applications.

The equipment used for soil mixing varies from single shafted augers to complex configurations of multi-shafted augers, depending on the purpose of mixing, (Yang, 1997). In Figue 2.2, an example of a detached single-shafted auger is shown, while Figure 2.3 shows a multi-shafted auger set-up. Single shafted large diameter augers are normally used for shallow soil mixing, which is usually limited to around 12m due to soil consistency and torque requirements (Burden 2000). Depending on the type of the project, deep mixing also uses multi shafted auger mixing system such as

Figure 2.2 Example of single-shafted site auger (http://www.dot.state.fl.us/)

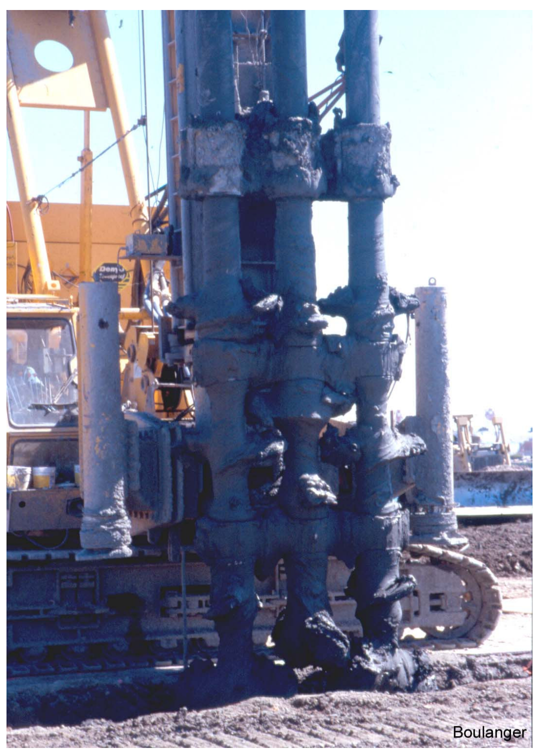


Figure 2.3 Full site set up of a multi- shafted auger (http://cgpr.ce.vt.edu/)

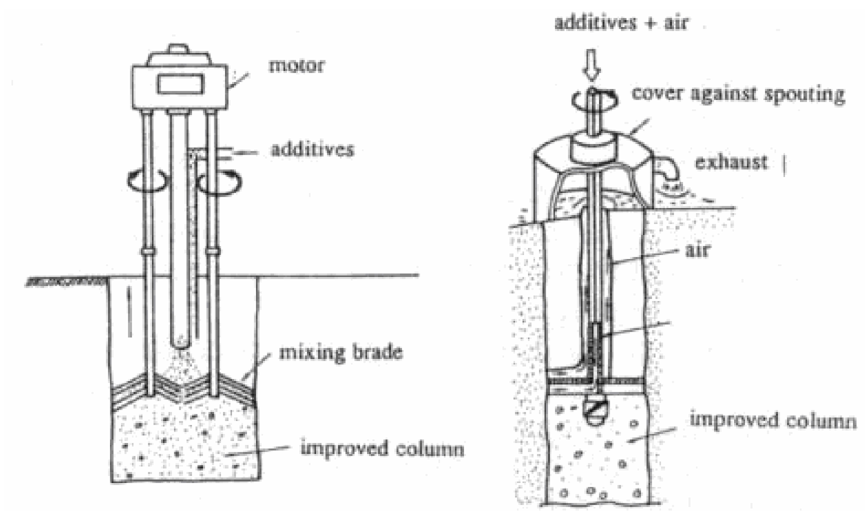
the one shown in Figure 2.3. With both shallow and deep mixing, it is possible to inject slurried agents and also dry powder reagents. This process of reagents transferring pneumatically is known as dry mix method. On the other hand as discussed earlier, in wet mix method additives and reagents are pumped with pressure. Mostly cement and lime are used in these operations. Sometimes, other stabilizing agents like fly ash can also be used. Cement may be used in conjunction with silicates, thermoplastics and polymers for the treatment of organic contaminants (Porbaha, 1998). Other materials that can be considered for this purpose are gypsum and blast furnace slag.

Classification of soil mixing is further broken down into rotary or rotary plus jet, end and shaft mixing (Holm, 1999).

- Rotary or rotary plus jet: If the binder is mixed with the soil via rotary energy, then the process is rotary, while mixing which is enhanced by both rotary energy and high pressure jet is referred to as rotary plus jet mixing.
- End mixing: This process of mixing action takes place near to the drilling tool.
- Shaft mixing: This process of mixing action is continued along the shaft for a considerable distance above the drilling tool. This is relatively new environmental remediation technique (Al-Tabbaa et al., 1998). The method is favourable where generation of spoils on the surface is not desirable and handling of excavated soil could lead to possible hazards, by placing all the treatment materials in the ground with minimal disruption of surface activities.

Unlike some installation methods DSM is less noisy and does not involve vibration. It is one of the most suitable ways of remediation in that by simply mixing the native soil with reactive media, resources are effectively used (Porbaha, 1998). In DSM the success of treatment is heavily dependent on the effectiveness of the grout, its injection and subsequently mixing with the soil. The grout should be flowable for effective mixing, and the mixing should be thorough so that homogeneous well-mixed soil-additive monolithic columns are obtainable (Al-Tabbaa et al., 1997, Al-Tabbaa et al., 1999).

Deep mixing method is the *in-situ* mixing of stabilizers with soft soils to form columns, walls, grids or blocks. The mechanical mixing methodology where slurry state or dry powder state stabilizer used is shown in Figure 2.4 (Kamon 1997). DM columns and their surrounding soils are commonly considered as a composite ground (Shen et al. 2003).



(a) (b)

Figure 2.4 Mechanically mixing methods (Kamon 1997) (a) slurry state supplying; and (b) dry powder state supplying

Most studies considered the increased bearing capacity of the composite ground resulted solely from the increased strength of the columns. The phenomenon of strength increase is identical to the property changes in surrounding soils due to installation of driven piles.

2.6 Effects of Deep Soil Mixing

2.6.1 Property Changes in Surrounding Clays Due to DMM

Rajasekaran and Rao (1996) performed laboratory scale tests by installing lime columns and observed that the strength in the surrounding clay with a radial distance of eight times the column radius increased by four to eight times the original values. The compressibility of the surrounding ground was reduced to about $1/3^{rd}$ to $1/4^{th}$ the original values of the untreated soil. Shen et al (2003) identified six major factors affecting the surrounding soils during and after installation of deep mixing columns in the laboratory.

2.6.1.1 Soil Thixotropy

Thixotropy is one of the controlling factors for the strength reduction and recovery during and after soil mixing. The degree of thixotropy depends on the type of clay mineral, clay structure, water content, and type and concentration of the ions dissolved in the pore water. Since a deep mixing column is constructed through mixing in-situ soft clay with chemical admixtures using rotating blades, there exist two types of forces acting on the surrounding clay around the columns (Shen et al. 2003b). The first force is created by an expanding action. This expanding action is caused by the injection pressure of chemical admixtures. The second force is created by a shearing action, and it results from blade rotation with a shear force. These dual effects can generate excess pore water pressures and disturb the surrounding clay so that a plastic zone is formed around the column. This disturbance can result in a decrease of strength in the surrounding clay. The degree of disturbance depends on the sensitivity of the clay, injection pressure, and configuration of the blades. This mechanism was investigated in this study by remolding a soil sample and then setting it to rest for designed time periods.

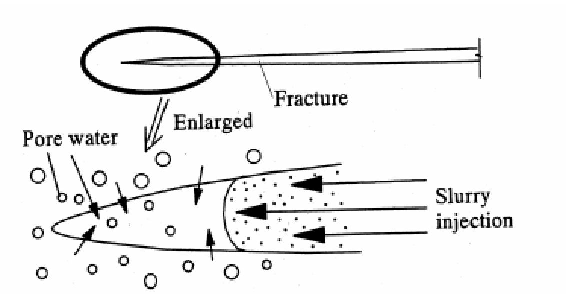


Figure 2.5 Infiltration of slurry and pore water into fractured cracks (Shen et al. 2003)

2.6.1.2 Soil Fracturing

Shen and Miura (1999) and Shen et al. (2003b) demonstrated that soil fracturing in the surrounding clay occurred during installation of DM columns. The soil fracturing is caused by shearing action of rotating blades and expansion action of injecting admixtures. The combination of these two actions makes soil fracturing develop easily even under a relatively low admixture injection pressure. Shen and Miura (1999) and Shen et al. (2003b) determined that this fracturing occurred within a region around a column of two to four times its column diameter. This mechanism was further investigated in this study using a vane shear device.

2.6.1.3 Cement Penetration and Diffusion

As explained in Figure 2.5, fractures are filled with two fluids one from injected slurry to open the fracture, and the other from pore water that infiltrates the fracture. Due to the difference in ion concentration and pore water pressure between the column and the surrounding clay, ions in the cement slurry would diffuse into the surrounding clay (Mitchell 1993; Rajasekaran and Rao 1997). Since there exists fractures in the vicinity of the columns after mixing, cement slurry can conveniently fill in the fractures under the expanding force during mixing. The ions in cement slurry can diffuse into the soil mass not only from columns but also from fractures filled with slurry. Mathew and Rao (1997) reported that the diffusion of lime from lime columns can change the inherent property of clay minerals, such as Ion migration and further reactions due to the diffusion of the binder slurry and the existing pore water into the fissures generated during the mixing action of the augers can change the inherent properties of the clay minerals such as the cation exchange capacity (CEC), pH and pore water characteristics (Mitchell 1993; Rajasekaran and Rao 1997).

2.6.1.4 Cementation

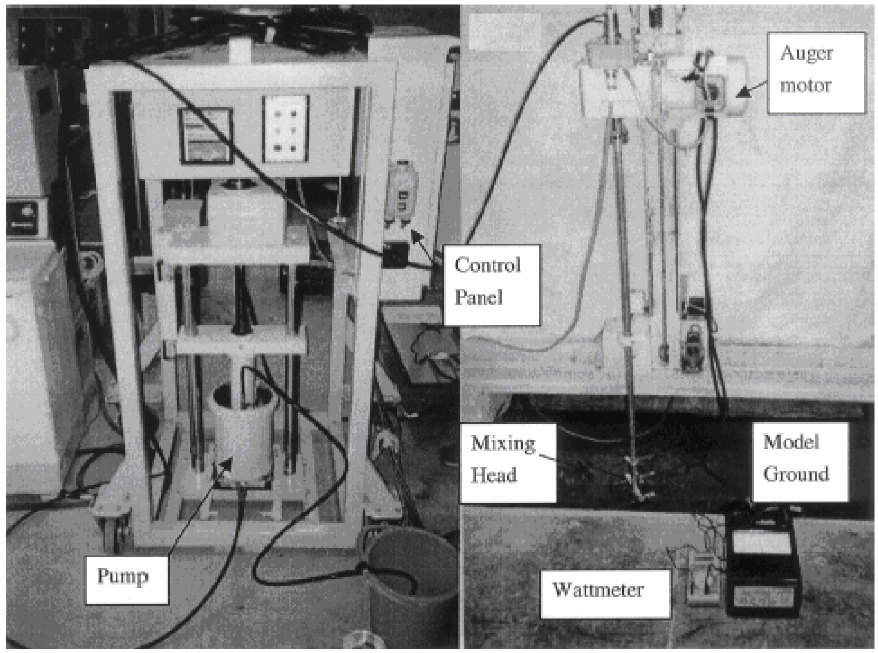
The increase in electrolyte concentration due to the injection of binder slurry causes an imbalance in the clay water electrolyte system. The equalization of concentrations would be achieved various chemical reactions including, hydrogen bonding, dipole orientation and London dispersive forces (Mitchell 1993). This process shortens the duration for strength regain.

2.6.1.5 Consolidation

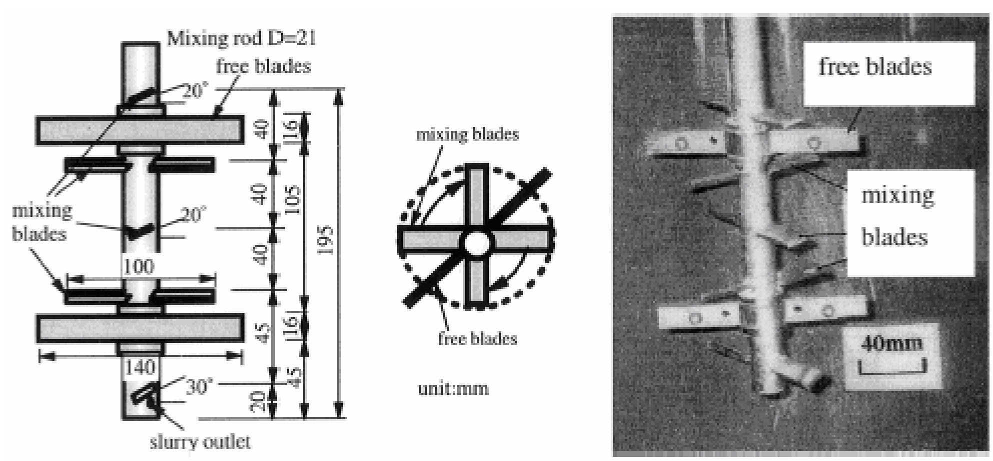
The dissipation of excess pore water during the mixing action of the blades and installation of columns causes consolidation of clay surrounding the columns and induces an apparent strength gain. (Miura et al. 1998b; Sakai and Tanaka 1986; Shen and Miura 1999; Shen et al. 2003b).

2.6.1.6 Hydration

This phenomenon is predominant in the case of dry mixing using cement and lime as binders. The heat of hydration elevates the temperature resulting in the reduction of water content aiding to faster consolidation of clay (Tsuchida et al. 1991). Shen et al (2003) tested Ariake clay to investigate the installation effects on the property changes of surrounding clay. The model column preparing device along with the mixing blade used in this study is shown in the Figure 2.6.



(a) (b)

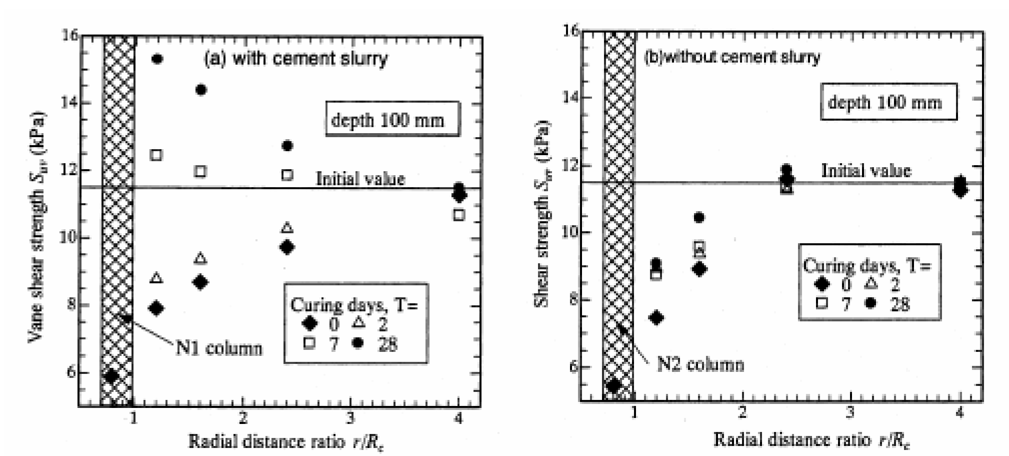


(c)

Figure 2.6 Model column preparing device and mixing head apparatus (Shen et al. 2003) (a) control panel; (b) auger motors and mixing blades; and (c) structure of mixing head in the model device.

As shown in Figure 2.7, the strength reduction and recovery after mixing were different when one was mixed with cement slurry injection (Column N1) and the other was mixed without cement slurry injection (Column N2). In comparing these two cases, the case with cement injection had a wider range of the clay with reduced strength than that without cement injection. This is because the cement slurry injection in Column N1 would extend the fractures developed in the clay. Since there was no chemical agent added in Column N2, the strength recovery in this case resulted mainly from the thixotropic effect. However, strength recovery in the case with cement injection resulted from a combination of the thixotropic effect and cementation.

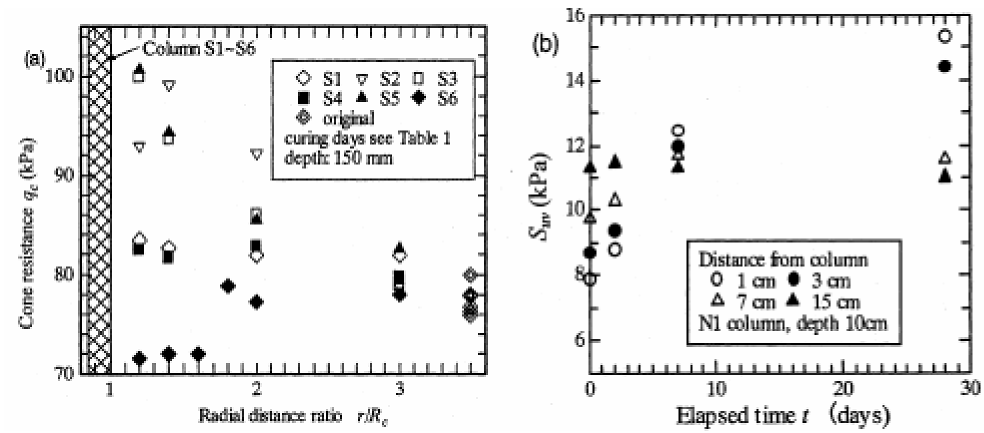
It is shown that significant strength increase occurred near the column as compared to that distant from the column in the case with cement injection. For the case with cement injection, the reduced strength was completely regained after a 7-day curing period. The clay had about 1.5 times the original shear strength in the region close to the column. For the case without cement injection, the soil only regained 70% of the original shear strength even after being cured for 28 days.



(a) (b)

Figure 2.7 Comparison of vane shear strength reduction and recovery after mixing: (a) Column N1 (with cement slurry injection); (b) Column N2 (without cement slurry injection) (Shen et al. 2003)

Figure 2.8 shows the variation of cone penetration resistance along the radial distance and with curing period.



(a) (b)

Figure 2.8 Changes of cone penetration resistance (a) along radial distance; (b) effect of curing period (Shen et al. 2003)

Further investigations in the field on the same soil by Shen et al (2005) indicated that the method of deep mixing is an additional parameter that can be studied to affect the degree of disturbance of the surrounding clay in the vicinity of the treated column. Three commonly adopted methods including the high pressure jet mixing (HJM), powder jet mixing (PJM) and slurry mixing method (SLM) were investigated. The results indicate that the HJM method has the highest degree of disturbance followed by the PJM and SLM. Figure 2.9 shows the increase in unconfined compression strength after treatment, q_u (t) indicated as a relative effect to the untreated soil strength, q_u (0).

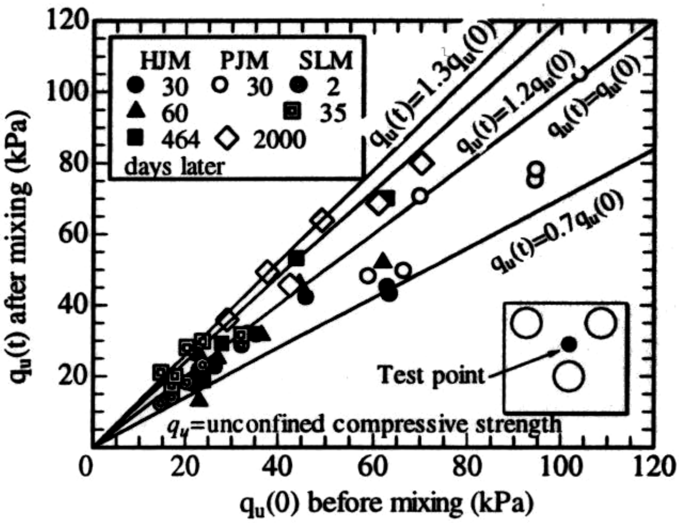
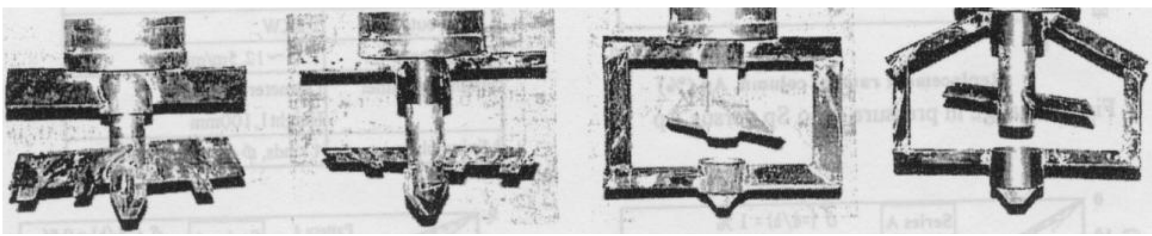


Figure 2.9 Effect of deep mixing method on the unconfined compressive strength in field (Shen et al. 2005)

2.6.2 Laboratory Investigations on Mixing Conditions and Sample Preparation Techniques

The final successful result of any deep mixing project is mostly dependant on an extensive laboratory testing program. Despite a considerable advance in the laboratory simulating procedures in different countries across the world practicing deep mixing, there is no fixed laboratory testing methodology for mixing conditions and sample preparation techniques. Thus, in this section an attempt was made to summarize the various sample preparation techniques as observed in the literature from various parts of the world.

Dong et al. (1996) performed a set of laboratory tests to clarify the effects of several factors including the shape of mixing blade, revolution speed, and velocity of penetration and retrieval of auger. The specifications and various shapes of blades used in the investigation can be found in the above reference. Two kinds of mixing blades each subdivided into two types according to their thickness were considered as shown in Figure 2.10. As shown in the Figure 2.11, it can be found that the strength increased with increasing rotary speed regardless of different shapes of mixing blade. The UCS properties were found to be improved with thinner mixing blades.



(a) (b) (c) (d)

Figure 2.10 Types of mixing blades (a) Type A-1; (b) Type A-2; (c) Type B-1; and (d) Type B-2 (Dong et al. (1996)

Al-Tabbaa (1998, 1999) performed a set of tests to understand the effects of sample preparation and storage. All the samples survived the cyclic wet-dry cycles but failed to resist the freeze-thaw cycles. Further laboratory auger mixing on uniform, fine and medium dense sand at near saturated conditions have shown increased unconfined compressive strength at higher number of blade revolutions. Pousette et al. (1999) studied the effect of sample diameter, mixing duration and the curing conditions on stabilized peat material. Jacobson et al. (2002) described the effects of sample preparation techniques to facilitate proper mixing in the laboratory. Table 2.2 presents a brief review of existing laboratory standards for laboratory sample preparation and testing.

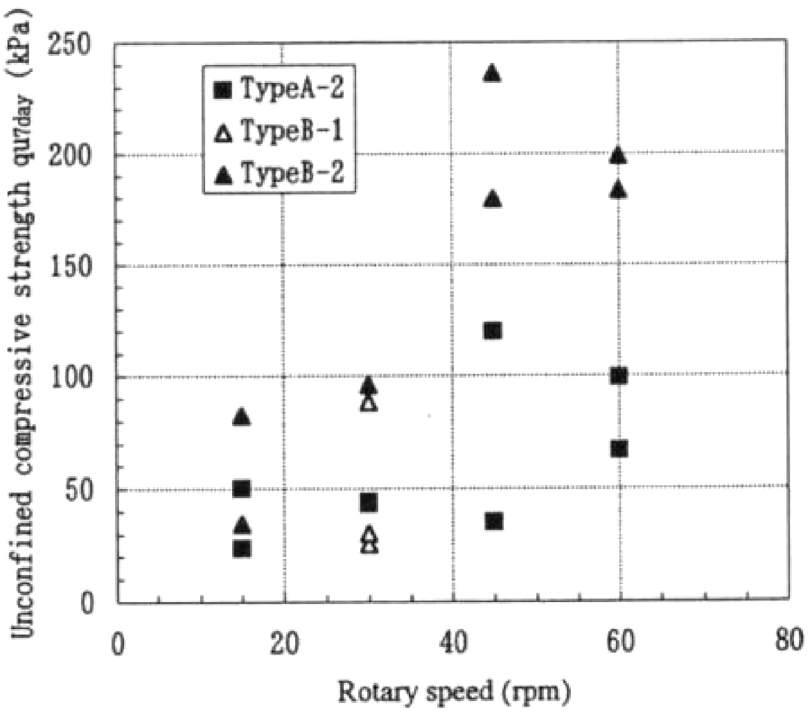


Figure 2.11 Relationship between rotary speed and improved strength
(Dong et al. 1996)

Table 2.2 Review of existing laboratory standards for laboratory sample preparation and testing (Bhadriraju, 2005)

Preparation standards	**Field sampling and storage**	**Sample preparation molds**	**Type of soil mixer**	**Sample preparation procedure**	**Curing conditions**
Japanese Geotechnical Society, JGS 0821-2000, Section 7.2	Thin walled sampling, store the specimens at original water content	The standard size of the mold is defined to create a specimen with 5-cm diameter and 10-cm height.	Domestic dough mixer with 5,000 to 30,000 cm^3 mixing bowl and hook type paddle, capable of 120 to 300 rpm planetary motion (Figure 2.15)	Mixing duration: 10 minutes with occasional hand mixing, compacted in 3 lifts with poking using 5 mm metal rod and light tamping to exclude air voids	The sample ends are properly sealed with specified sealants and stored at $20\pm3^{\circ}C$ for specified time at 95% relative humidity
EuroSoilStab, CT97-0351. (Project No. BE 96-3177)	Tube, piston or Delft samplers, stored at *in situ* conditions	Plastic tubes or plastic coated cardboard, 5 cm diameter and 10 cm height coated with oil or wax in the inner side	dough mixer or kitchen mixer with sufficient capacity and rpm for all soil types	Mixing duration: 5 minutes and is a variable depending on the soil type. Circular steel stamp 10 mm thick and 45 mm diameter, attached to a 50 mm long rod. Static load of 100 kPa may be used for 2 seconds on each layer	No mention of humidity, store samples at a constant temperature of 18-22 $^{\circ}C$ in properly sealed conditions

Table 2.2 *Continued*

Al-Tabba et al. (1999) and Shen et al. (2003)	N/A	50, 100 and 150 mm diameter soil mixed columns are prepared in test pits with same principle as the DM column installing machine in field	Sensor controlled speed and rpm of the augers. The equipment mainly consists of slurry injection part, a mixing device and controlling panel pressure control	Control panel operated and is dependant on soil type. Injection pressure can be adjusted from several kPa to several hundred kPa. Consolidation pressure can be simulated through air pressure	Cured at room temperature for a specific curing period
Jacobson et al (2002), Virginia Tech and VDOT, United States	Bulk samples with minimized exposure to air and stored at 100% RH at 20°C	50 mm diameter and 100 mm tall one time use plastic molds which can be easily during sample extraction	Kitchen aid dough mixer with dough hook. Outer spindle rotating at 155 rpm and inner spindle at 68 rpm to mix sufficient sample to form a batch of eight	Mixing duration of 5 minutes with intermittent hand mixing. 25 mm (1 inch) thick lifts in molds, poking with 5 mm brass rods evenly 25 times. 100 kPa pressure for 5-10 seconds using a 48 mm aluminum piston.	Cured at 100% relative humidity and 20±3 °C for 7, 14, 28 and 56 days

2.6.3 Effect of Dosage Rates

Dosage rate may be termed as the amount of stabilizer added to the soil. These can be defined in two different ways: kg of stabilizer per m^3 of treatable soil (which bases the amount of the stabilizer on the volume of the soil that is to be treated) and kg of stabilizer per m^3 of treated soil (which bases the amount of stabilizer on the volume of the soil after treatment). Typically, in practice, dosage rates vary from 80 to 150 kg/m^3.

Jacobson et al. (2002) performed a series of experiments on three soils to investigate the influence of dosage rate, UCS on other parameters. It was observed that as dose rate increases, strength increases for all soils (Figure 2.12). The addition of cement and lime provided a continuing increase in strength for all ranges of dosage rates, lime appearing to be less effective than cement. Taki and Yang (1990) identified that the soil type as the most sensitive factor influencing the strength of treated ground. The same degree and type of treatment used in different soils produces results with a wide ratio given the variation in the rate of adsorption and pozzalonic reaction in various soils. It was also found that the strength increase in cohesive soils was slower in comparison with those in sand and gravelly soils. The strength of treated soils was higher than that of cohesive soils by a factor of 2.5 to 5, all other variables remaining constant.

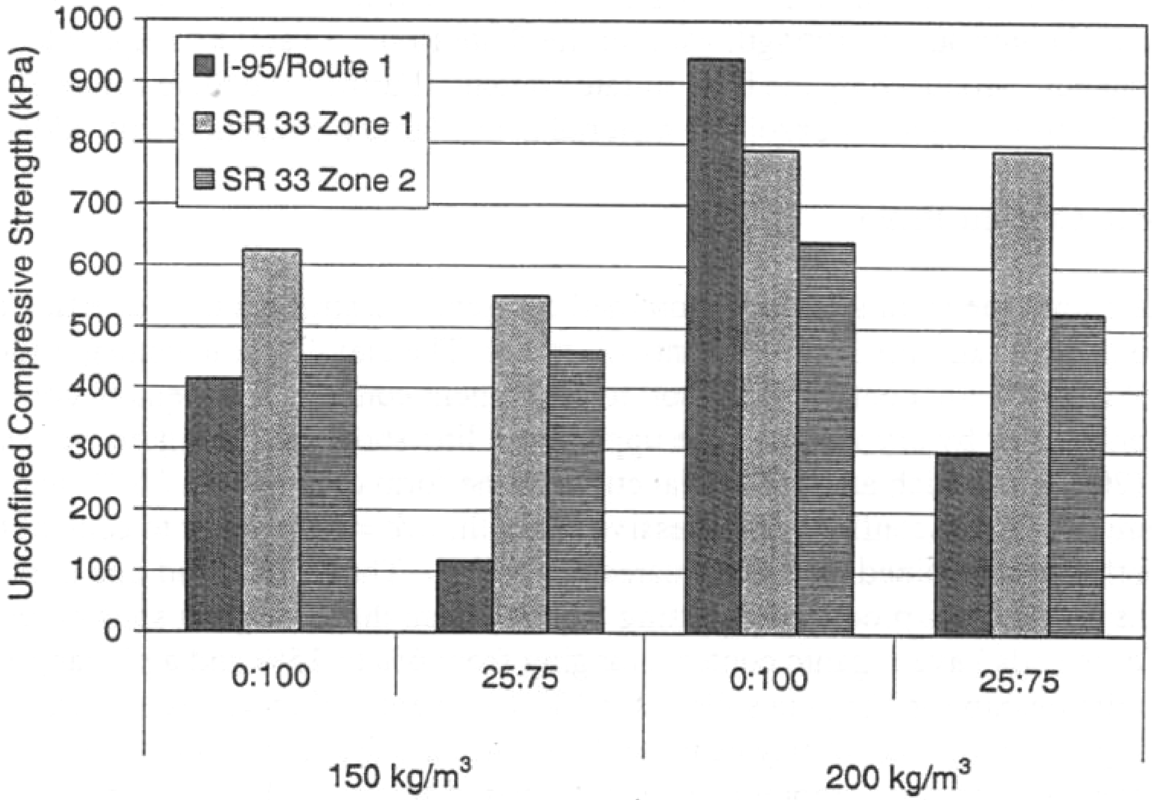


Figure 2.12 Strength comparisons with binder dosage (Jacobson et al. 2002)

2.6.4 Effect of Water-Cement Ratio

Wet process of deep soil mixing involves the ejection of slurry in the form of a jet. In this process, the binder used to stabilize is mixed with water. Thus, this proportion of cement and water is very important and careful consideration to the given for the amount of water added to the in situ soil along with the binders. In practice, typical range of the water cement ratio is 0.8 to 1.5.

Studies on soil improvement for an excavation support system by Matsuo et al. (1996) reported that the mean improved compressive strength q_u decreases with the increase of water cement ratio and the quality of the consolidated mass is poor at these mixing conditions. Figure 2.13 explains the improved compressive strength for various water-cement ratios. A typical w/c ratio of 1.0 is recommended to obtain the highest compressive strength and it is always a function of the soil type and mixing procedures adopted in the field.

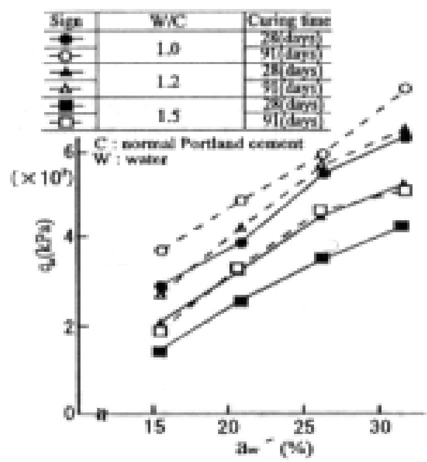


Figure 2.13 Increase in strength with varying w/c ratio (Matsuo et al. 1996)

Jacobson et al. (2002) reported the results of unconfined compressive strength tests as a function of clay-water to cement ratio for 100% cement-soil mixes for three soils by the dry method. These results were compared with the results of the experiments performed on Hong Kong clay mixed by wet mixing method by Miura et al. (2002) as shown in Figure 2.14.

The exponential curve suggested by Miura et al. (2002) is in the form given in form of equation 1 was fitted to the data.

$$q_{u\,(28\ day)} = a/b^{(w/c)} \tag{4}$$

where

q_u = the unconfined compressive strength,

w/c = clay-water to cement ratio

a, b = coefficients obtained by least-squares regression

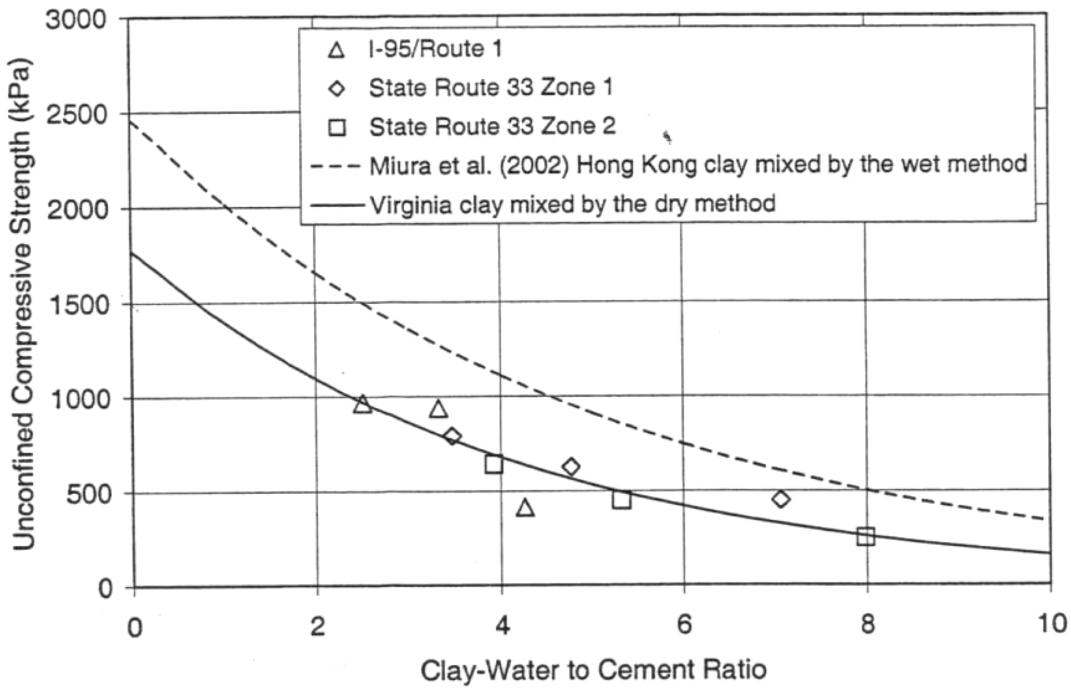


Figure 2.14 Relationship between unconfined compressive strength and clay-water to cement ratio (Jacobson et al. 2002)

Horpibulsuk et al. (2003) reported that based on the target strength properties in the field, the clay water ratio (ratio of initial water content of the clay (%) and the cement content, A_w) can be calculated from the following relation. However, once the clay-water cement ratio is fixed in the field, if the in situ water content changes, the equations presented below can be used to estimate the corresponding adjustments to be made to the water added to form the cement slurry so as to obtain the design strength parameters determined in the laboratory.

$$\left(\frac{q_{(wc/C),D}}{q_{(wc/C),28}}\right)=1.24^{\{(wc/C),28-(wc/C),D\}}\left(0.038+0.281\ln D\right) \quad (5)$$

where:

$q_{(wc/C),\ D}$	=	target strength required
$q_{(wc/C),\ 28}$	=	strength developed after 28 days of curing
$(w_c/C)_D$	=	strength required at a curing period of D days

2.6.5 Effect of Curing Conditions

The major curing conditions that influence strength properties of treated soils are curing time, curing temperature, curing humidity and confining pressure. Esrig (1999) stated that most strength gain occurs within the first 28 days after mixing and strength continues to increase at a slower rate thereafter. Most of the literature indicates that at same curing time, the higher the curing temperature, the more rate of pozzolanic reactions and thus, the greater the soil strength.

Ahnberg et al. (1989) states that lime-cement column curing conditions can produce temperatures ranging from 10^0C to 100^0C or more, depending on ambient temperature, soil thermal conductivity, configuration and density of columns, stabilizer type and amount etc. Attempts have been made to express the relation between the three factors of curing temperature, curing time and strength. In research on concrete, the relation between curing temperature, curing time and strength is expressed as maturity (curing temperature plus 10^0C, multiplied by curing time) (Babasaki et al. 1996). The variation of UCS with respect to maturity for 6 different soils is shown in the Figure 2.15.

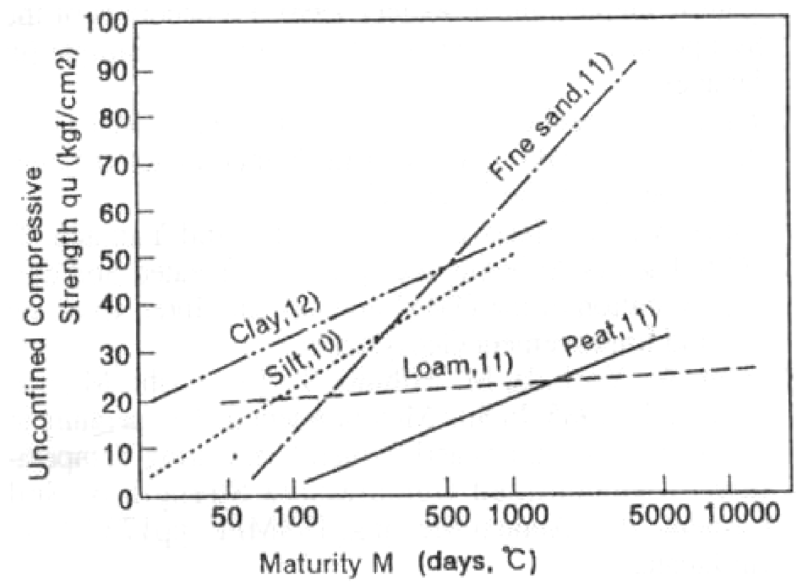


Figure 2.15 Relation between unconfined compressive strength q_u and maturity M (Babasaki et al. 1997)

Curing humidity is also an important factor influencing strength property, though its control is not stratified. Den Haan (2000) recommended several methods like curing samples in sealed, airtight tubes etc. for controlling humidity in curing rooms. Hampton and Edil (1998) found that providing the samples access to water while applying a confining pressure during curing, which may imitate field conditions more accurately, reduces strength.

Confining pressure is sometimes applied to samples during curing inorder to mimic overburden stresses (Jacobson et al. 2002). The effect of confining pressure is more dramatic in peat samples (Hebib and Farrell 2002). Pousette et al. (1999) found that for peat samples, increasing the load during curing time from 10 to 40 kPa increased strength by 85%.

2.7 Summary

The chapter summarizes an extensive literature review on expansive clay behavior and its stabilization. Various methods for the characterization of expansive soils as a measure of volume change are discussed. An introduction to ground improvement methods and stabilizers used predominantly for them specifically for deep mixing method is presented. Various types of equipment and methodologies employed for performing deep soil mixing in the field were discussed. The later part of the chapter discusses elaborately the various factors affecting the behavior of the deep mixing treated soils in the laboratory environment.

CHAPTER 3

EXPERIMENTAL STUDIES

3.1 Introduction

A step by step procedure for treated sample preparation of medium stiff expansive clays, simulating the deep soil mixing, curing and then storage, was proposed by Bhadriraju (2005) after careful review of various standards in literature. The same procedure was followed in the experimental studies conducted here and the details are presented in latter sections. It should be noted that these standards were derived from various countries practicing DM techniques to stabilize different types of soft soils as no universal testing procedures developed on the specimen preparation were found in the literature.

This chapter discusses research variables considered, sample preparation and various tests performed on treated soil specimens to understand the stress-strain behavior of treated soils and the extent of improvement compared to control soils, reported in Bhadriraju (2005), due to laboratory deep soil mixing. Bhadriraju (2005) limited his study to a water cement (w/c) ratio of 1, the current experimental program investigates the effects of w/c ratio < 1 and > 1 on strength properties of lime cement treated soils. The ranges of other parameters, binder dosage ratio, binder proportions, and curing period were considered to be same as those listed in Bhadriraju (2005). A detailed work plan of the present research study is depicted in Figure 1.1.

In field, it is expected that the treated ground will be in partially saturated or unsaturated state. Therefore, matric suction developed as a result of soil-lime-cement columns prepared in the field would to contribute to the volume change behavior of treated soil columns. To determine the range of matric suction that can be developed in field for different binder dosage rates (100, 150 and 200 kg/m^3) and binder proportion (25:75), this experimental program investigated total suctions of lime-cement treated soils following the filter paper method developed by Bulut et al. (2001).

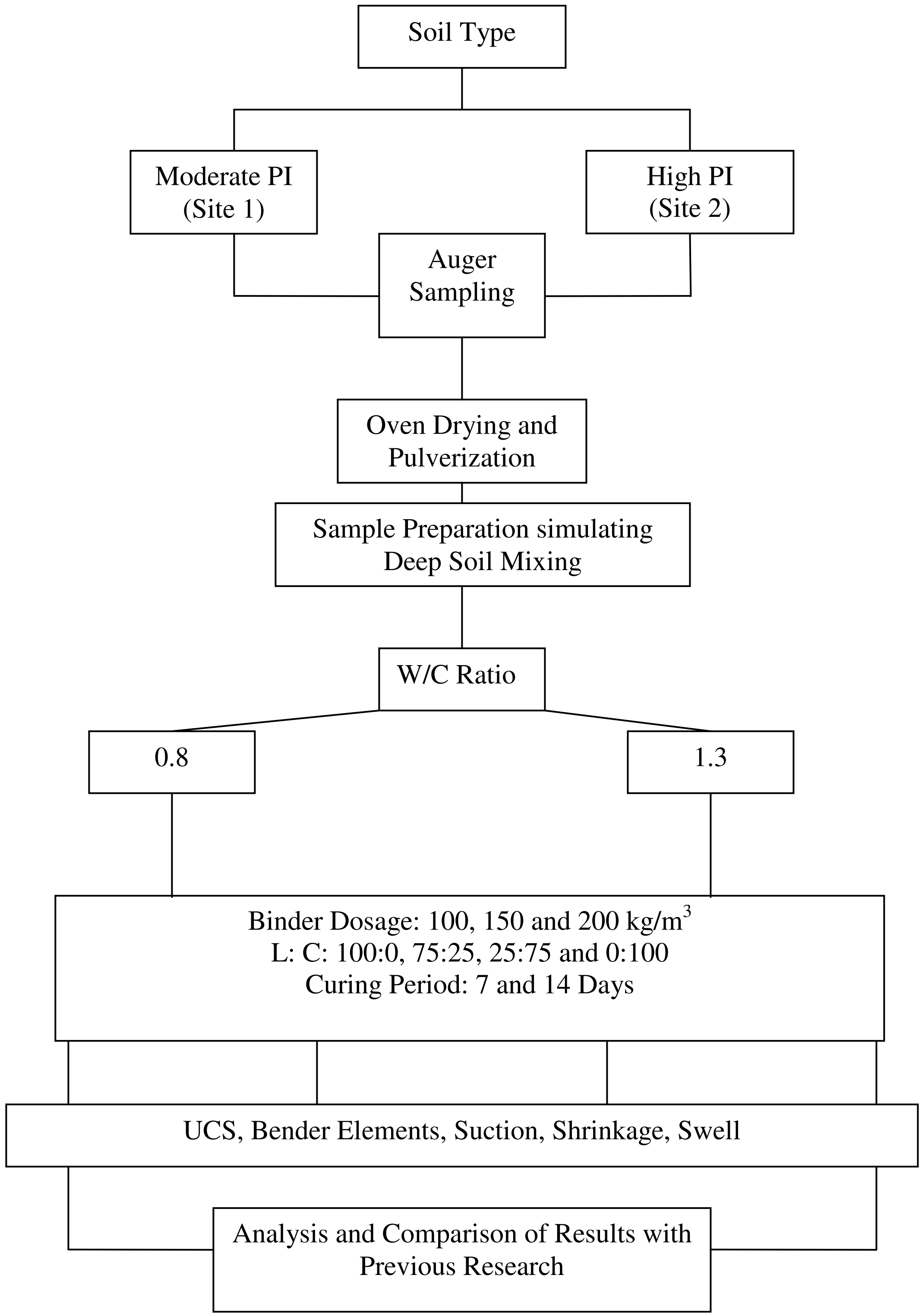


Figure 3.1 Schematic Outline of the Experimental Study

3.2 Field Sampling

The control soils for the experimental program were obtained from the two test sites, characterized as moderate and high PI sites by Bhadriraju (2005), located along the median of IH 820 North in Fort Worth, Texas. Bulk sampling was performed using triple auger to obtain remolded samples. The sampling was conducted to depths to which improvement was recommended after careful evaluation of active depth based on PVR method by TxDOT. Both sites are approximately 0.7 miles apart. During sampling, samples were collected into moisture cans to determine the differences in moisture content, if any, with previous research conducted by Bhadriraju (2005). It is noted the there is negligible variation in field moisture levels as the sampling was performed in span of 5 months span from Fall 2004 to Spring 2005 (in October 2004 and March 2005). The remolded samples were then transported to laboratory and stored in 100 % relative humidity room maintained at room temperature ($20 \pm 3^{o}C$).

This research work is an extension from the recommendations given by Bhadriraju (2005). As such, the soils used in present experimental program were same as those used in previous research work. Therefore, the basic engineering tests including Atterberg limits, determination of organic content, sulfate levels, free swell, shear modulus using Bender Elements and unconfined compression tests on control soils were not repeated. The results on control soils reported by Bhadriraju (2005) were used here to study the improvements in treated soils.

3.3 Tests Performed on Control Soils

Experimental studies conducted on treated medium stiff expansive soils were only reported in the present study. The engineering tests including soil classification, Atterberg limits, linear shrinkage, unconfined compression, determination of sulfate level, organic content, pH and shear modulus on control soils were performed and reported by Bhadriraju (2005) and hence these tests not repeated here again.

3.4 Research Variables

The strength and deformation behavior of deep mixing treated soils show a strong dependency on various factors under laboratory testing conditions. Based on the literature review performed, factors such as soil type, binder type, binder contents, binder proportions, curing period, curing conditions and water cement ratio are the primary factors affecting the stress strain response of the treated soil. Therefore it is necessary to study the influence of the above variables on the stress-strain, swell-shrink behaviors and stiffness properties of the treated expansive soils. It has been observed that a water cement ratio of 1 was most commonly adopted in the field installation of DM columns (Babasaki et al. 1996). The previous research conducted by Bhadriraju (2005) studied the effects of above variables on the behavior of treated soils by limiting the water cement ratio to 1. The current study was extended from previous research to study the effects of water cement ratio, along with the above mentioned variables. Table 3.1 presents variables studied in the present investigation. A final mix design was arrived at by optimizing the performance of the treated soil in the laboratory testing sequence to be considered best for field implementation.

3.5 Specimen Notation

For identification of different soil types stabilized with different combinations of binder dosage levels, proportions, water cement ratio and curing period, a simple notation system was followed throughout the study. Every specimen was assigned a

Table 3.1 Research variables considered for the present research

Description	**Variables**
Soil types	2 [medium and high PI]
Binder content (BC)	3 [100 (6%), 150 (9%) and 200 (12%) kg/m^3)]
Stabilizer proportions (Lime:Cement)	4 [100:0, 25:75, 75:25, 0:100]
Curing time (days)	2 [7 and 14]
Water cement ratios	0.8 and 1.3
Curing conditions	1 [100% relative humidity, 20±3 ºC]

notation, for example, in the form of S1-0.8-100-LC-0:100-7-1. The first letter (S1) of the notation indicates the site from where the soil was obtained. Symbols S1 and S2 were used for site 1 and site 2 respectively. The second (0.8) and third (100) symbols / numericals indicate the water cement ratio and binder content in kg/m^3, used for particular combination of variables, respectively. The fourth (LC) and fifth (100:0) symbols represent in the order and proportion of the stabilizers respectively. The letters

L and C stand for lime and cement respectively, while, in this case, 100 and 0 stand for 100 % lime and 0 % cement. The following numerical 7 or 14 represent the curing period in days. Duplicate samples of each combination were tested to ensure repeatability of test results. The numerical following the curing period in the sample notation represents the sample number. The following table 3.2 presents the detailed description of the specimen notation used.

Table 3.2 Summary of sample notation

Symbol/Numerical	Description
S1	Site 1
S2	Site 2
0.8 and 1.3	Water cement ratios
100, 150 or 200	Binder content in kg/m^3
L:C	Proportions of stabilizers in the order lime and cement
100:0	100 % lime and 0 % Cement
75:25	75 % lime and 25 % cement
25:75	25 % lime and 75 % cement
0:100	0 % lime and 100 % cement
7 or 14	Curing period in days

3.6 Calculations for the Current Study

3.6.1 Calculations for Dry Weight of Soil

Bulk unit weight, γ_b (kg/m^3 or pcf) = W_{core}/V_{core} (6)

Initial clay water content in the field, w_n (%) (7)

Dry unit weight, γ_d (kg/m^3 or pcf) = $\dfrac{\gamma_b}{1+w_n}$ (8)

Total volume per one sample mix, V = Volume of mold for UCS testing + Volume of swell mold + Volume of linear shrinkage mold (9)

Dry weight of soil per sample, W_S = $\gamma_d \times V$ (10)

3.6.2 *Calculations for Binder Quantities*

Binder content, a_w (W_B/W_S) in % = Ratio of weight of binder to weight of soil both reckoned in the dry state (11)

Binder factor, α (kg/m^3 or pcf) = $$\frac{a_w \times \gamma_b}{100(1+w_n)} \tag{12}$$

Weight of binder per sample, W_{LC} = $\alpha \times V$ (13)

Lime: Cement Proportions, LC = 0:100. 25:75, 75:25, 100:0 (14)

Weight of lime, W_L = $$\frac{L}{100} \times W_{LC} \tag{15}$$

Weight of cement, W_C = $$\frac{C}{100} \times W_{LC} \tag{16}$$

Water cement ratio, w:c = $\frac{W_{w,slurry}}{W_{LC}}$, (typically 0.8 to 1.3) (17)

Weight of water for slurry, $W_{w,\,slurry}$ = W_{LC} × water cement ratio (18)

Total water content of the mix, w_T = $w_n + w_{w,\,slurry}$ (19)

Total clay water cement ratio, w_T: c = w_T/W_{LC} (20)

The above mentioned calculations are performed per batch. A batch creates four UCS, one free swell and two linear shrinkage bar samples for the respective tests described in this section.

3.6.3 Typical Calculations per Specimen

The current section explains a set of sample calculations pertaining to one sample mix preparation for the notation: S2-0.8-100-LC-100:0-7. The unit weight and water content considered in the calculations are derived from the average values of undisturbed cores of control soil at different depths. The subsequent calculations for determining the quantities of dry soil and binders are based on these average values.

Average bulk unit weight, γ_b (from undisturbed cores of 7 cm dia and 14 cm height): 2050 kg/m^3

Average water content, w_n (%): 24.14%

Average dry unit weight, γ_d (from equation 3): 1652 kg/m^3

Total volume for one specimen mix, V:

Volume of UCS mold	5.69×10^{-4} m^3
Volume of free swell mold	9.64×10^{-5} m^3
Volume of linear shrinkage mold	4.62×10^{-5} m^3
Total volume (V)	7.59×10^{-4} m^3

Dry weight of soil, W_S (from equation 5): 1.254 kg

Binder dosage rate, α: 100 kg/m^3

Equivalent binder content, a_w (from equation 7)

$$a_w = \frac{100 \times 100 \times (1 + 0.2414)}{2050} = 6\%$$

Weight of binder per sample, $W_{LC} = \alpha \times V$:	0.076 kg
Lime: Cement proportion:	100:0
Weight of lime (from equation 10):	0.076 kg
Weight of cement (from equation 10):	0.0 kg
Water cement ratio:	0.8
Weight of water for slurry, $W_{w,\ slurry} = 0.076 \times 0.8$:	0.061 kg
Weight water required to bring the soil to field water content, W_n, ($w_n \times W_s$)	0.303 kg
Total water content of the mix ($W_n + W_{w,\ slurry}$)	0.364kg

3.7 Soil-Lime-Cement Mixing, Specimen Preparation and Curing Procedures

3.7.1 Equipment Needed

1. Split type acrylic molds (70 mm and 150 mm in diameter and height, respectively) with three stainless steel hose clips and acrylic base plate
2. 5 mm poking rods and light rammer (1 kg base and 5 cm height of fall)
3. 70 mm diameter, 25 mm height plastic molds for free swell testing
4. Linear shrinkage bar mold (six slots)
5. Kitchen aid domestic mixer with 10 speed, 575 watt electric dough mixer, dough hook and beater.
6. Mixing bowl to facilitate preparation of one batch (4 specimens) in each run.
7. Commercial blender for mixing and preparation of slurry

8. Moisture tins, hand gloves, sand paper and permanent marker.
9. Raymond 914 protective film and plastic zip bags.
10. Commercially available hydrated lime and Portland cement type I/II (technical details provided in Appendix A).
11. Straight edge, scale and vernier calipers

3.7.2 Mixing Procedure and Sample Preparation

A laboratory procedure was developed for soil-lime-cement mixing and specimen preparation by Bhadriraju (2005). The steps followed were close to wet method of soil mixing in the field. This procedure was developed for treating medium stiff expansive clays. The following section elaborates 24-step procedure followed here for soil mixing and sample preparation:

1. Obtain approximately 3500 g of dry soil passing No. 40 sieve for preparing a batch of four specimens (2 per each sample notation and curing period) for UCS, free swell and linear shrinkage tests. As the control soil was obtained in bulk from all the depths it is assumed to be representative of all the soil types encountered through out the depth.
2. Weigh the appropriate amount of lime/cement based on the stipulated proportions and binder factor, α in kg/m^3 using Equations (6) to (10). Also, determine the total water content from equation (14) which includes the remolding or in situ water content and the water intended for the preparation of lime/cement slurry i.e. from water/cement ratio)

3. The proportions for various combinations of lime and cement should be measured and mixed in dry conditions in a separate bowl prior to addition of water for slurry preparation.
4. A commercial blender shown in Figure 3.2 is used to mix the total water content with the binder for approximately 2-3 minutes to ensure uniform binder slurry.
5. The dry soil collected in step 1 is transferred into the mixing bowl and the mixing rate of the outer spindle is preset at 60 rpm (level 2) and inner spindle rotated at about 152 rpm. A dough hook is attached to the spindle for the purpose of uniform mixing.
6. The binder slurry is slowly introduced with the mixer running at the preset speed. Care should be taken to avoid the soil from forming lumps which may be difficult to break after certain period. A flexible spatula or beater can be used to avoid the soil from sticking to the sides and bottom of the mixing bowl. The process is continued approximately two minutes
7. At the end of two minutes, the mixer is stopped and the soil in the bowl is quickly transferred into a large mixing bowl. The mixing is then performed with hand for approximately two minutes to break the lumps and uniformly distribute the binder with the soil. This procedure was found necessary though not to simulate the field mixing but to atleast attain a homogeneous mix without lumps.

8. The soil is transferred back into the mixing bowl and the mixing process at the preset speed is continued for another 2-3 minutes with constant stirring and removal of soil from the sides and bottom of the bowl. Step 6 is repeated after this step. The total mixing time was approximately around 7-8 minutes.

3.7.3 Sample Preparation for UCS Testing

9. A split type acrylic mold, 70 mm in inner diameter and 156 mm length with 10 mm thick acrylic base plate and three intermittent steel hoses was used for UCS sample preparation. Figure 3.3 depicts the pictures of acrylic mold, base plate and hammer.
10. The empty weight of the mold with steel clamp fastened and excluding the base plate is recorded. A very thin layer of grease was applied to the inner surface of the mold and to the surface of the base plate to reduce side friction and for easy removal of soil specimen.
11. The soil is transferred in to the mold using a spoon and slightly compacted in 5 layers each 30 mm thick as explained below. Care should be exercised so that the final height of the specimen should not be less than 14 cm to preserve the aspect ratio of ≥ 2 triaxial specimens.
    a. Pour the loose soil-lime-cement mix into the mold upto a height of 60 mm from the latest lift and compact it to 30 mm
    b. Compaction should be done by poking using a 5 mm rod (Figure 3.6) for approximately 30-40 times spanning the whole surface area of the specimen to remove the entrapped air with in the specimen

c. It was observed that the clay is displaced in the direction opposite to that of the application of force forming hair line cracks along the surface. This was resolved using slight tapping and compaction with a light hammer (1 kg in weight and 50 mm height of fall, seen in Figure 3.6), imparting 25 blows around the surface of the specimen evenly not to allow any extrusion of soil through the edges and bottom of the mold.

d. A grid type grooves were formed at all intermittent layers using a spatula to ensure continuity in the specimen.

e. The final layer should be perfectly leveled to avoid bedding error during UC testing. It is recommend to use a spatula slightly wetted to obtain a flat surface

12. Two protrusions, 2 mm wide, 12 mm long and 8 mm deep (0.07" x 0.47" x 0.31") were made on either surfaces of the sample for facilitating the bender element testing for stiffness measurements. It is an important step as the grooving tool may not penetrate into the treated soil after the curing period.

13. The final weight of the mold without the base plate is recorded and the mold is sealed using a thin protective film (Reynolds 914 Film is commercially available for this purpose). The final setup is enclosed in a plastic air tight zip bag and is appropriately labeled. It should be noted that the air in the bag is excluded prior to sealing the bag.

14. The final assembly is stored in a 100 % relative humidity room with temperature control at 20±3 ºC.

15. Repeat steps 9-13 within 20 minutes from the actual mixing of the soil and binder. Obtain a total of 4 UC specimens two per each curing period (7 and 14) for repeatability of test results.
16. The samples placed in the curing room are removed after 16-18 hours and the molds are stripped out. The samples are then carefully sealed in the same plastic bags and transferred back to the curing room. The molds were used for preparation of samples for further testing.

3.7.4 Preparing Specimens for Free Swell

17. An acrylic mold, 70 mm diameter and 25 mm height was used for the preparation of samples for free swell testing.
18. The empty weight of the mold is recorded and a thin layer of grease or similar material is applied to the inner surface of the mold.
19. The mold was placed on the base plate and the loose soil-lime-cement mix was transferred into the mold in two lifts. Slight compaction of the soil as mentioned in step 11 was followed to prepare compacted specimens. A different swell mold of same dimensions can be used as a collar for the final lift.
20. The weight of the swell mold along with soil and excluding the base plate is recorded. The mold was sealed using the protective film and placed in an air tight zip bag and appropriately labeled after excluding the entrapped air in the bag.

3.7.5 Preparing Specimens for Linear Shrinkage

21. A linear shrinkage bar mold with an assembly of six bars of dimensions 102 mm long × 19 mm wide × 19 mm deep is used for the purpose. The empty weight of the bar was recorded and the inner surfaces are properly greased using a thin layer of grease or similar material.
22. The soil-lime-cement mixture was placed into the first slot of the mold followed by slight tamping and jab using a spatula to remove any entrapped air. The final weight of the mold with one slot filled was recorded.
23. The adjacent slot was filled with the same mix but at its liquid limit (Tex-107-E). For this purpose, a portion of the mixed soil is wetted with sufficient water and placed in the Casagrande cup to determine the closure of the grove in approximately 25 blows. The slot was filled with the mixture with water content close to liquid limit. The final weight was recorded with two slots filled. This was repeated at every filling of the mold to obtain the wet unit weight.
24. The slots in the mold were appropriately labeled under the bottom of the mold and then covered with a wet geotextile and placed in an air tight zip bag for curing in the humidity room. The samples are taken out of the curing room every two days and water is sprinkled above the geotextile to minimize the heat of hydration and subsequent shrinkage of the mixture even before drying.

Figure 3.2 Domestic mixer to simulate mixing procedure and commercial blender for slurry preparation (Bhadriraju 2005)

Figure 3.3 Details of compaction rammer, poking rod, free swell mold, UCS mold, base plate and linear shrinkage mold

3.8 Laboratory Testing of Treated Soils

The test procedures followed for determination of treated soil properties including free swell, linear shrinkage at molding water content and at liquid limit, bender element for shear moduli measurements and UCS were explained in following sections. The tests were conducted at the end of the curing time and the data was recorded accordingly. However, flowing precautions should be taken prior to conducting the bender elements and unconfined compression tests:

1. The protrusions provided for bender element testing were checked to the required dimensions using a damp grooving tool to ensure proper coupling of the bender elements with the specimen. Any excess water is removed using a high absorbent paper
2. The ends of the soil specimens after the curing period are carefully made flat, if required, by rasping them with sand paper to avoid bedding error that might create a large scatter in the strength results (Tatsuoaka and Kohata 1996).

3.8.1 Stiffness Measurements Using Bender Element Test

Bender element testing is a wave propagation based technique that has been successfully used in geotechnical engineering to estimate stiffness measurement and shear moduli of soils at very small shear strains (less than 10^{-3} %). QC/QA studies can be done using G_{max}. The shear modulus, G, estimated at very small strains is considered as maximum and is nearly constant with strain at very low range of strains. Thus, the shear modulus is represented as G_{max} and is related to shear wave velocity as follows:

$$G_{max} = \rho \times V_s^{\,2} \tag{21}$$

where G_{max} = Small strain shear modulus,

V_s = Shear wave velocity at small strains = L_{eff} / t (m/sec)

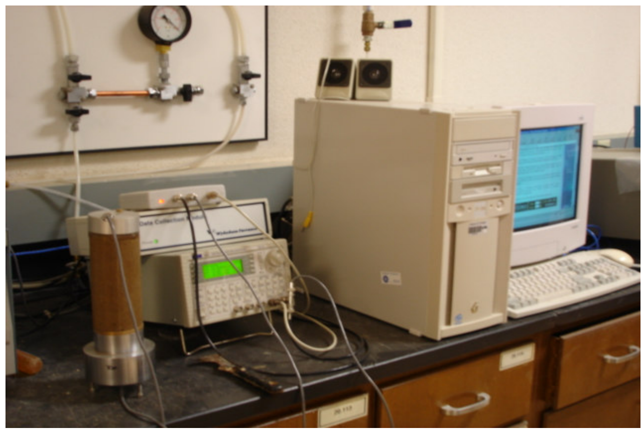
L_{eff} = Effective length (m) = length of specimen (L, m) - 2× (8/1000).

ρ = Mass density of soil specimen (kg/m^3).

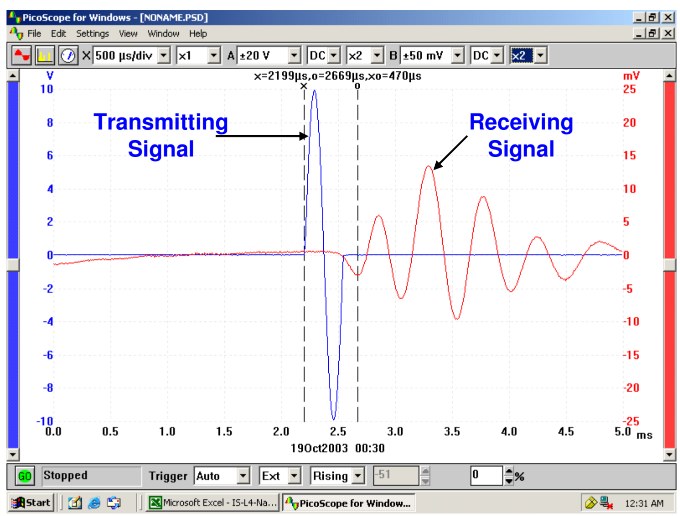
The bender element (BE) set up consists of piezoceramic bender elements (transmitter and receiver), signal generator, oscilloscope and a personal computer for data acquisition and processing reduction tasks (Figure 3.4 a). Both transmitter and receiver bender elements were inserted into the protrusions made at both ends of the treated soil specimen ensuring proper coupling and isolation of the specimen and oscilloscope from the surrounding vibrations which might affect the shear wave velocities. The test was conducted under unconfined conditions by sending a triggered single sinusoidal signal of ±20 V amplitude to the BE transmitter. A frequency between 1.5 to 2.5 kHz was selected until the received signal had an optimal amplitude and shape. Vertical lines marked ‘x’ and ‘o’ on the output screen (Figure 3.4 b) represent the start of transmitter signal and received signal, respectively. The first significant inversion of received signal was considered as the true arrival of shear wave. The difference in time of transmitted and received shear waves represent the time of flight, which is indicated at the top of output screen as ‘xo’.

The experiment was repeated for three times, i.e. the shear wave was sent thrice, and the average time of flight was recorded. Time of flight represents the time taken by the shear wave to travel a effective length, tip to tip distance of BE elements (L_{eff} = L-

2×8 mm), across the treated soil specimen. Prior to testing, the mass of the specimen was measured for determining the bulk unit mass (ρ). The small strain shear modulus was then estimated using Eq. (16). Further information on the BE test can be found in Kadam (2003) and Puppala et al. (2005).



(a)



(b)

Figure 3.4 Bender Element test setup for stiffness measurements (a) test setup and accessories; (b) real time capturing of the shear wave

3.8.2 Unconfined Compressive Strength Test

The unconfined compression strength tests were performed on the same specimens after the BE test as per ASTM D 2166. The water content of the core after shearing was measured and recorded using the microwave drying method.

3.8.3 Determination of Linear Shrinkage Strains and Free Swell

The preparation of linear shrinkage bars and free swell specimens of treated soil was explained in detail in previous sections. Tex-107-E method was followed to determine the linear shrinkage of the treated soil specimens. After the soil specimens were cured for the specified time period, the molds were taken out of curing room and then transferred into an oven set at 110±5 ºC for 24 hours. The change in length is determined accurately using a vernier calipers and the linear shrinkage strain is reported in percentage.

Treated free swell soil samples were taken out of 100% relative humidity room after the specified curing period and were subjected free swell testing as per ASTM D5890-02 procedure. The one dimensional free swell of the sample was monitored using a dial gage on top of the specimen, at no load condition for a period after which no further volume change in vertical direction was observed. The heave of the expansive soil, measured as strain is termed as free swell index (FSI). Figure 3.5 depicts the experimental setup for determination of free swell of treated soil specimen.

Figure 3.5 Free swell setup on treated soil specimen

3.8.4 Determination of Total and Matric Suction

Suction is usually used in soil mechanics to explain the mechanical behavior (strength, deformation and permeability) of unsaturated soils. The more quantative definition of the soil suction states that suction is a negative gage pressure which represents the interaction between soil particles and water. The concept of soil suction should not be confused with pore water pressure since the latter is normally associated with the density of water, distance from ground water surface, and surface tension forces (McKeen, 1981). It can also be termed as measure of the affinity of a soil for water (i.e., the intensity with which it will attract water). Suction is considered to consist of two parts: matric suction (h_m) and osmotic suction (h_o). The sum of both is referred to as total suction (h_τ).

Total suction may be defined as the negative gage pressure, relative to the external gas pressure on the soil water, to which a pool of pure water must be subjected in order to be in equilibrium through a semi-permeable membrane with the soil water. Whereas, matric suction is related to the capillary phenomenon (often illustrated by the rise of water surface in a capillary tube) arising from the tension of water. Pores with small radii in a soil mass act as capillary tubes which, at low degrees of saturation, can hold water at very high negative pressures. Fig 3.5 shows an idealized air water interface in a soil mass similar to the water surface in a capillary tube. By equilibrium of forces acting on the air-water and pressure, the matric suction can be defined by

$$h_m = (u_a - u_w) = \frac{2T_s}{r}$$

where h_m = matric suction

u_a = air pressure

u_w = water Pressure.

T_s = Surface Tension in the air-water interface membrane

r = radius of the idealized sphere representing the bottom of the air channel in the soil mass.

Adsorptive forces exerted on the water molecules by the soil particle surfaces also contribute to matric suction. These forces may generate tensile stresses in the soil water in excess of one atmosphere and also account for the fact that the curvature of the water film along the surface of soil particles is opposite to the curvature of the water film between particles (Nelson and Miller 1992), as shown in Figure 3.6.

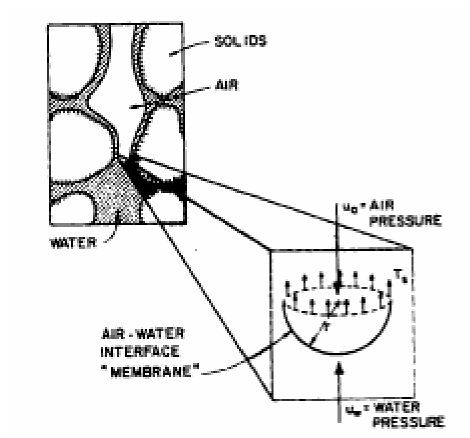


Figure 3.6 Air Water Interface in soils (Nelson and Miller 1992)

3.8.4.1 Procedure for measurement of suction

Soil Suction can be measured by either direct or indirect techniques. In order to measure suction directly, a pressure differential is created between the soil pore water and the atmosphere to reduce or increase the pore water until its suction equals the applied air pressure (Chen 1988).

Filter paper method developed by Bulut et al. (2001) for the measurement of both total and matric suctions was followed in the present study. Schleicher & Schuell No. 589-WH ash free filter papers were placed with the soil specimen inside an air tight container for atleast a week, at a constant temperature (usually room temperature), and then their water contents were determined and correlated to the calibration curve developed for Schleicher & Schuell No. 587-WH filter paper for the suction value. It is assumed that the soil sample and filter paper will reach equilibrium with respect to moisture flow by either vapor or liquid moisture exchange (Fredulund and Rahardjo 1993). After equilibrium, the suction in the filter paper will be at the same value as that in the soil. A step wise procedure as shown in Figure 3.7 was followed in the present for the measurement of both total and matric suctions for treated lime cement soil specimens was as follows:

1. The soil specimens were prepared similar to the procedure employed for the preparation of free swell specimens in section 3.7.4 [Figure 3.7 (a)].
2. The prepared specimens were wrapped in a plastic sheet in order to avoid any moisture loss and were used at the time of the test. A Schleicher & Schuell No. 589-WH 5.5 cm in diameter filter paper is sandwiched between two larger diameter protective filter papers [Figure 3.7 (b)].
3. The sandwiched filter papers are inserted into the soil sample in a very good contact manner [Figure 3.7 (c)]. An intimate contact between the filter paper and the soil is very important.
4. The two halves of the cylindrical samples are brought together and sealed with electrical tape to keep the two specimens together in a good contact manner [Figure 3.7 (d)].
5. Then, the whole sample with embedded filter papers is put into the glass jar container [Figure 3.7 (e)]. The smaller the empty space remaining in the glass jar, the smaller the change in the soil specimen water content as a result of the release of water vapor in to the empty space in the jar. A 2.0 inch height ring type PVC tube support is put on top of the soil to provide a non-contact system between the filter paper and the soil [Figure 3.7 (e)].

6. Two Schleicher & Schuell No. 589-WH filter papers one on top of the other are inserted on the ring using tweezers [Figure 3.7 (f)]. The filter papers should not touch the soil, the inside wall of the jar, and underneath the lid in any way.
7. The glass jar lid is sealed very tightly [Figure 3.7 (g)].
8. The Steps 1 through 7 are repeated for every soil specimen. All the glass jars were labeled and were placed undisturbed at a constant temperature [Figure 3.7 (h)].

3.8.4.2 Calculations

The filter paper water contents for obtaining total suctions are calculated as follows (see Table 3.3).

1. Mass of dry filter paper, $M_f = M_2 - T_h$
2. Mass of water in filter paper, $M_w = M_1 - M_2 - T_c + T_h$
3. Filter paper water content, $W_f = M_w / M_f$
4. Steps 1, 2, and 3 are repeated for every filter paper.

After obtaining all of the filter paper water contents, the equation for the wetting filter paper calibration curve is employed to get total suction values of the soil samples.

$h_1 = -8.247W_f + 5.4246 \qquad (h_1 > 1.5 \log kPa)$

$h_2 = -8.247W_f + 6.4246 \qquad (h_2 > 2.5\ pF)$

Where:

h_1 = total suction (in log kPa)

W_f = filter paper water content (in decimals)

h_2 = total suction (in pF)

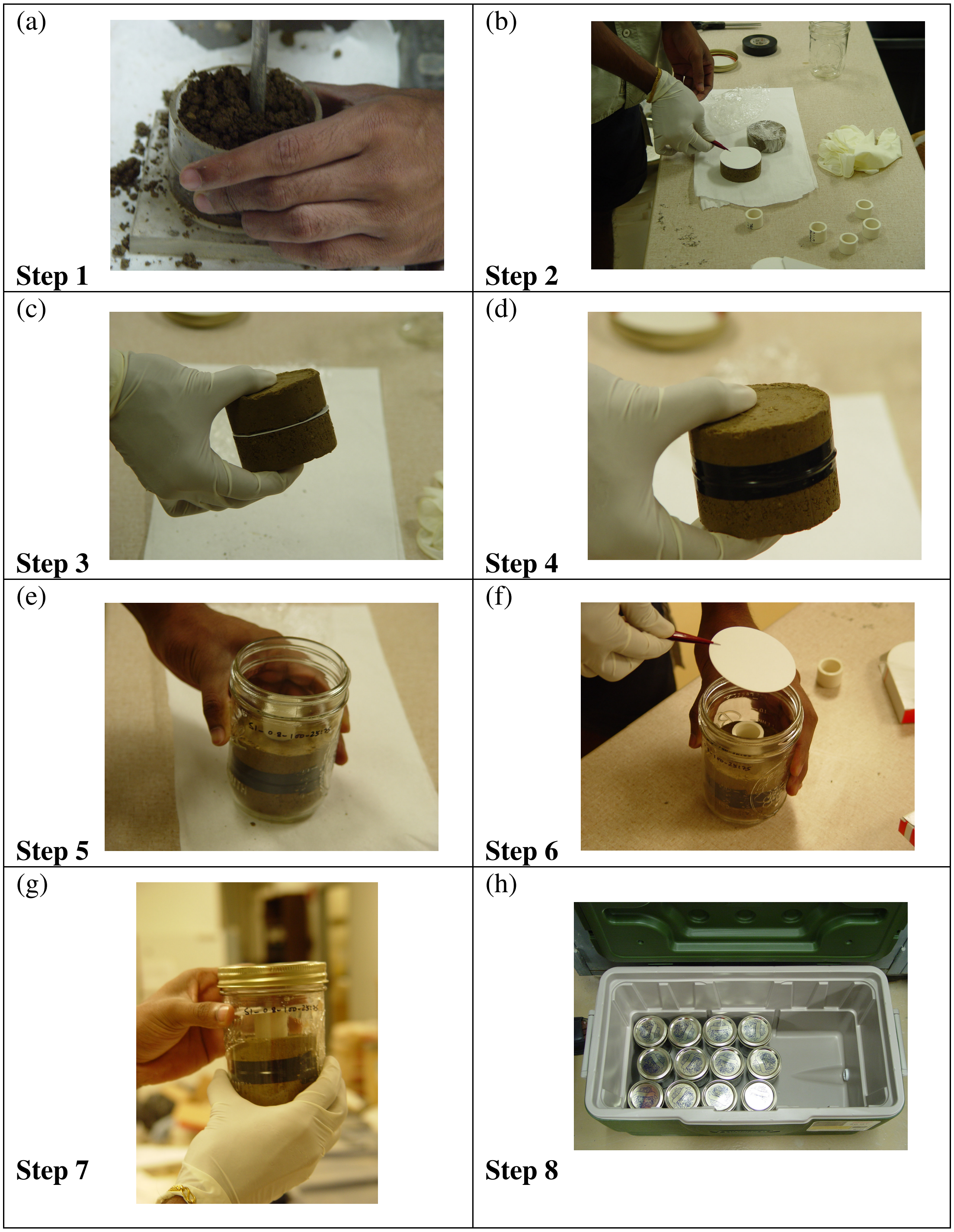


Figure 3.7 Step wise procedures for measurement of total and matric suctions (a) sample preparation (b), (c) and (d) filter paper sandwiched between samples and sealing with electrical tape (e), (f) and (g) placing the whole specimen in a glass jar (h) storing specimens at constant temperature

The filter paper water contents for obtaining matric suctions are calculated as follows (see Table 3.3).

1. Mass of dry filter paper, $M_f = M_2 - T_h$
2. Mass of water in filter paper, $M_w = M_1 - M_2 - T_c + T_h$
3. Filter paper water content, $W_f = M_w / M_f$
4. Steps 1, 2, and 3 are repeated for every filter paper.

After obtaining all of the filter paper water contents, the equation for the wetting filter paper calibration curve is employed to get matric suction values of the soil samples.

$h_1 = -8.247W_f + 5.4246 \quad (h_1 > 1.5 \log kPa)$

$h_2 = -8.247W_f + 6.4246 \quad (h_2 > 2.5 pF)$

Where:

h_1 = matric suction (in log kPa)

W_f = filter paper water content (in decimals)

h_2 = matric suction (in pF)

Table 3.3 The filter paper method suction measurements worksheet (Bulut et al. 2001)

<table>
<tr><td colspan="10">THE FILTER PAPER METHOD SUCTION MEASUREMENTS WORKSHEET</td></tr>
<tr><td colspan="9">Date Sampled:</td><td>Date Tested:</td></tr>
<tr><td colspan="9">Boring No.:</td><td>Tested By:</td></tr>
<tr><td colspan="10">Sample No.:</td></tr>
<tr><td colspan="2">Depth</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td colspan="2">Moisture Tin No.:</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td colspan="2">Total or Matric Suction</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td colspan="2">Top or Bottom Filter Paper</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Cold tare mass, g</td><td>T_c</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Mass of wet filter paper + cold tare mass, g</td><td>M_1</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Mass of dry filter paper + hot tare mass, g</td><td>M_2</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Hot tare mass, g</td><td>T_h</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Mass of dry filter paper, g ($M_2 - T_h$)</td><td>M_f</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Mass of water in filter paper, g ($M_1 - M_2 - T_c + T_h$)</td><td>M_w</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Water content of filter paper, g (M_w / M_f)</td><td>W_f</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Suction, log kPa</td><td>h_1</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Suction, pF</td><td>h_2</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
</table>

3.9 Summary

The present chapter explains various test procedures followed in the current research. The variables considered in the current research are summarized and the notations followed for easier identification are explained. A detailed procedure explaining sample preparation is also included in the chapter. Step by step procedures for strength and stiffness (bender element tests) as well as suction, swell and shrinkage properties are explained in detail starting from the preparation of control soil followed by specimen preparation, curing conditions and storage for treated soils.

CHAPTER 4

ANALYSIS AND DISCUSSION OF RESULTS

4.1 Introduction

This chapter presents a detailed summary of results obtained from laboratory tests conducted on treated soil specimens. In each section, first, the test results were analyzed to study variations in stress-strain behavior and then compare with those reported by Bhadriraju (2005) at a w/c ratio of 1.0. The effects of soil type, dosage rate, curing period and proportions of lime and cement stabilizers on stress-strain behavior of moderate and highly expansive soils at different water-cement ratios were analyzed and discussed.

Specimens were tested at three different binder contents (100 kg/m^3, 150 $kg/m^{3,}$ 200 kg/m^3), four lime cement proportions (100:0, 25:75, 75:25, 0:100), two curing periods (7 day and 14 day) and two water-cement ratios (0.8 and 1.3). Test results, from linear shrinkage, free swell, unconfined compressive strength, bender elements and total and matric suction tests were presented and explained in detail in the sections in the following pages.

4.2 Summary of Test Results on Control Soils

Based on the preliminary data provided by the commercial laboratory from the bore logs, the basic and engineering tests were performed and reported by Bhadriraju

(2005). Figures 4.1 and 4.2 show the borehole logging data obtained from various tests performed at different depths on the control soils. The results of tests conducted on the control soils were reported in Table 4.1.

Table 4.1 Index and engineering properties of control soil from site 1 and 2 (Bhadriraju 2005)

Property	**Test Designation**		**Site 1**	**Site 2**
	ASTM	**TxDOT**		
Specific gravity	ASTM D854	Tex-108-E	2.70	2.72
Gravel (%)	ASTM D422	Tex-110-E	0	0
Sand (%)	ASTM D422	Tex-110-E	3	2
Silt (%)	ASTM D422	Tex-110-E	32	24
Clay (%)	ASTM D422	Tex-111-E	59	50
Organic content (%)	ASTM D2974		5.24	2.96
Soluble sulfates (ppm)	UTA Method	TxDOT	922.6 / 2156	94.66 / 0.00
pH	ASTM D4972	TeX-121-E	7.95	7.88
Bar linear shrinkage %		Tex-107-E	22.42	18.32
USCS classification	ASTM D2487-00	Tex-142-E	CH	CH

From the bore hole log data of site 1, designated as BH4-D1, approximately 10 feet thick fill layer was identified. The fill material was followed by a 5 ft of dark brown to grayish brown clay with calcareous nodules. Weathered lime stone was encountered at a depth of 15 to 16.5 feet. Site 2 designated as BH3-D1 comprised of fill material in the top 5 to 6 ft. The fill material was underlained by dark clay with calcareous nodules. Weathered lime stone was encountered at a depth of approximately 19 feet.

DRILLING LOG

WinCore
Version 3.0

County	Tarrant	Hole	BH4-01	District	Fort Worth
Highway	Loop 820	Structure	Pavement	Date	11/10/2004
CSJ	DA6221	Station		Grnd. Elev.	100.00 ft
		Offset		GW Elev.	N/A

Elev. (ft)	Strata Description
	FILL, CLAY, sand with gravel and limestone pieces, dark brown, grayish brown,light brown, light gray (SC)
90. 10	CLAY, with calcareous nodules, dark brown, grayish brown (CH)
85. 15	WEATHERED LIMESTONE, with clay layers, light brown
83.5	
20	

Triaxial Test: Lateral Press. (psi)	Triaxial Test: Deviator Stress (psi)	MC	LL	PI	Wet Den. (pcf)	Additional Remarks
		30	64	39	114.73	P = 2.0, qu=11.06 psi, FS=4.8
		18.66			140.2	P=1.5, qu=22.72 psi, FS=12.6
		23.27	61.5	39.5	129	P=4.5, qu=58.61, FS=20.4
		24.22			134.6	P=3.0, qu=40.17 psi, FS=22
		13			148.55	P=3.0, qu=76.3 psi, FS=12.1
		24	46	22	132.3	P=3.5, qu=41.67, FS=6.2
		23			126.2	P=4.0, qu=20.83, FS=0.8

Remarks: Ground water was not encountered during or after drilling completion.

The ground water elevation was not determined during the course of this boring.

Driller: David
Logger: MB
Organization: CTL Thompson Texas, LLC

Figure 4.1 Bore log data and engineering properties of test site 1 (Low PI site)

DRILLING LOG

WinCore
Version 3.0

County	Tarrant	Hole	BH3	District	Fort Worth
Highway	Loop 820	Structure	Pavement	Date	11/10/2004
CSJ	DA6221	Station		Grnd. Elev.	100.00 ft
		Offset		GW Elev.	N/A

Elev. (ft)	Texas Cone Penetrometer	Strata Description	Triaxial Test: Lateral Press. (psi)	Triaxial Test: Deviator Stress (psi)	MC	LL	PI	Wet Den. (pcf)	Additional Remarks
		FILL, CLAY, sand with limestone pieces, dark brown, grayish brown, light			30	64	32	15.4	P=0.5, qu=15.4 psi, FS=0.5
					24.2	66	42	125.4	P=4.5+, qu=16.2 psi, FS=7.5
5					27.25			122	P=4.5+, qu=122.20 psi, FS=8.2
94.		CLAY, with calcareous nodules, dark brown, grayish brown (CH)			24.7	79	56	134.6	P=3.0, qu=28 psi, FS=15.6
					25			131.2	P=2.5, qu=24.5 psi, FS=6.3
10					26.5	74	50	127.3	P=2.25, qu=26.3 psi, FS=12.5
					24.6			131.48	P=3.0, qu=43.08 psi, FS=6.0
87.		CLAY, with calcareous nodules and limestone pieces, light brown (CH)							
15					22.5			130	P=3.75, qu=33.42 psi, FS=16.1
80. 20									

Remarks: Ground water was not encountered during or after drilling completion.

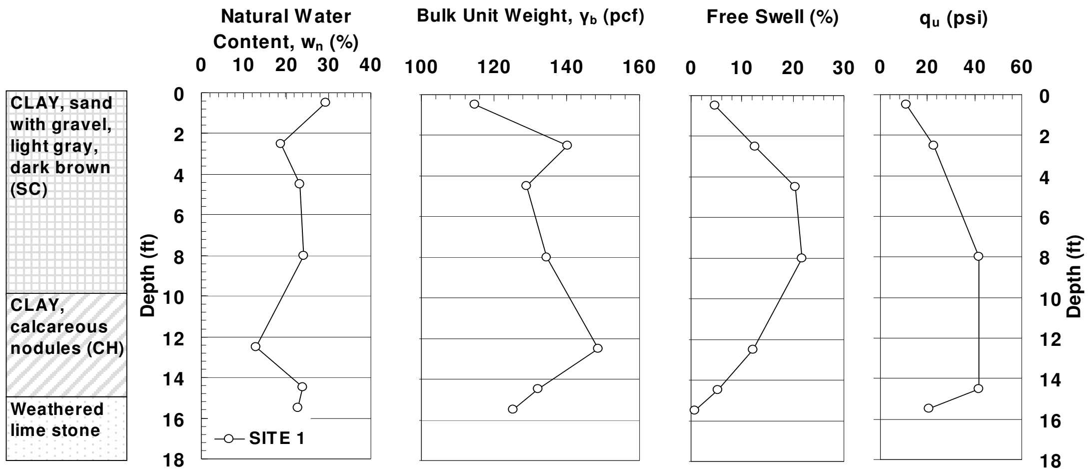
The ground water elevation was not determined during the course of this boring.

Driller: David | Logger: NB | Organization: CTL Thompson Texas, LLC

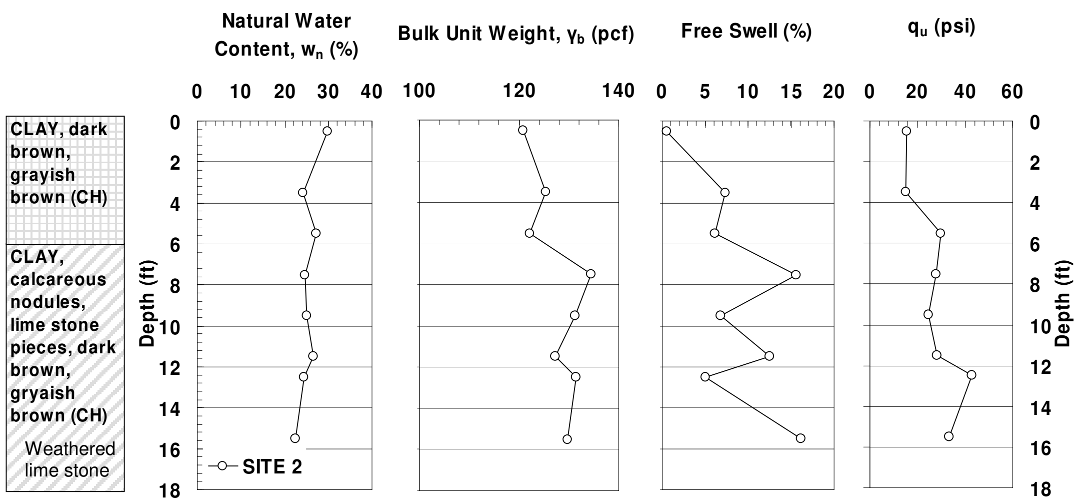
Figure 4.2 Bore log data and engineering properties of test site 2 (High PI)

It can be noticed from Table 4.1 that the soils at site 1 and site 2 are moderate (22 to 39 %) and high (32 to 58 %) PI clays, respectively. The variations of natural water content, bulk unit weight, percentage of free swell and unconfined compression strength of undisturbed control specimens were depicted in Figure 4.3. The UCS values for site 1 ranged from 70-275 kPa (10-40 psi) and for site 2 from 100-300 kPa (15-46 psi) with average values of X and Y kPa, respectively. Based on the average UCS values with depth, the control soil from both the sites can be classified as medium to stiff clays. The improvements in strength of treated soil specimens, reported in latter sections, are with respect to the average strength values of control soils.

Bender element tests were also performed to assess the variation of stiffness properties of the undisturbed cores prior to UCS testing. The small strain shear modulus (G_{max}) was calculated from Eq 3.1. The initial tangent modulus, E_i was estimated from the stress strain response under triaxial compression in unconfined conditions. The results of small strain shear modulus and initial tangent modulus with depth for sites 1 and 2 are presented in Table 4.2. A more detailed discussion on the results from basic engineering, unconfined compression and bender element tests on control soils can be found in Bhadriraju (2005).



(a)



(b)

Figure 4.3 Zone wise classification of physical and index properties of control soil (a) site 1 and (b) site 2

Table 4.2 Shear moduli, G_{max} and initial tangent moduli, E_i of control soil from sites 1 and 2 with depth

Depth (m)	Shear modulus, G_{max} (MPa)		Initial Tangent Modulus, E_i (MPa)	
	Site 1	Site 2	Site 1	Site 2
0-0.3	40.52	26.16	6.6	25.68
0.3-0.6	NT	43.98	NT	39.89
1.5-1.8	60.14	NT	47.4	NT
1.8-2.1	72.68	61.05	30.48	53.5
2.1-2.4	NT	60.23	NT	73.46
2.7-3.0	NT	59.05	NT	51.45
4-4.3	66.28	65.05	21.58	37.5
4.3-4.6	61.00	39.76	9.9	7.5

4.3 Present Test Results on Deep Mixing Treated Soils

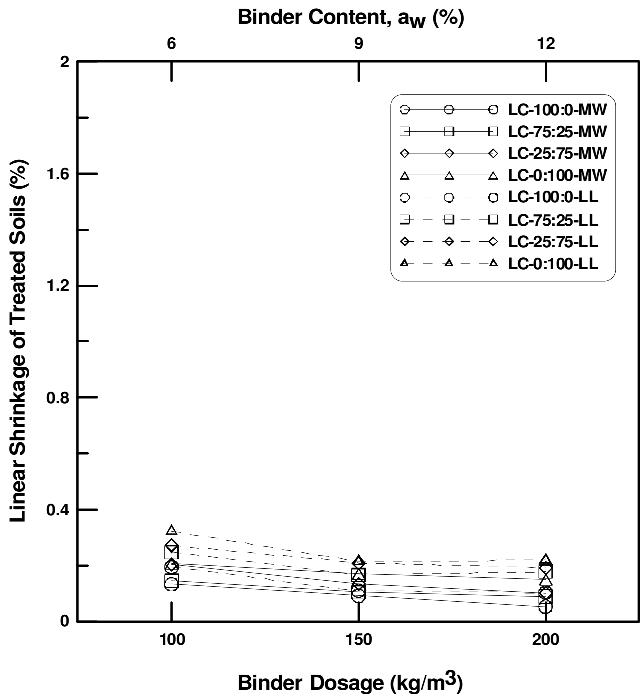
4.3.1 Linear Shrinkage Test Results

The linear shrinkage tests were conducted on treated soils at both molding water content and the liquid limit of the soil-binder mixture at different water-cement ratios (0.8 and 1.3). The shrinkage strains of all treated soil specimens improved considerably relative to the control soil property and yielded values corresponding to those that are characterized as low severity levels. The mechanisms involved in the linear shrinkage strains can broadly be characterized as a result of tensile failure, loss of contact points due to propagation of cracks from the surface of the soil and the effect of thermal conductivity on the magnitude of shrinkage. Though the latter is out of scope of the current discussion, crack formation due to tensile stresses developed within the soil mass plays a vital role in shrinkage strains. Conventionally, shrinkage in expansive soil can be directly related to the change in moisture content in the soil structure, which results in the formation of discontinuities in the soil medium due to crack propagation.

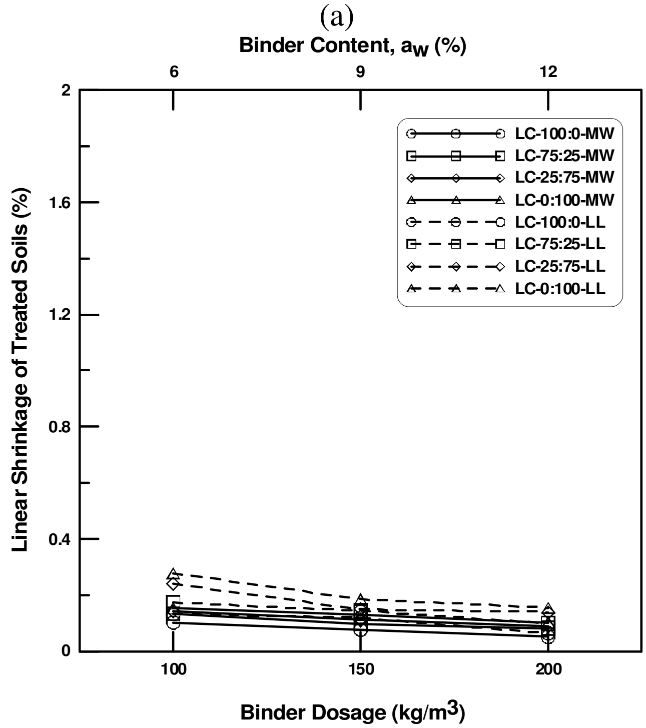
Figures 4.4 and 4.5 depict the effect of binder content, a_w (%) on the linear shrinkage strains of deep mixing treated samples at liquid limit relative to the specimens compacted from sites 1 and 2 materials prepared at 0.8 and 1.3 w/c ratios respectively. Though there is a negligible difference in the magnitude of % shrinkage strains of different chemical treated soils, the linear shrinkage strains were found to be greater for the samples treated at 1.3 w/c ratio than those treated at 0.8 w/c ratio. Moreover it can also be observed that in both water cement ratios, higher values of shrinkage strains were observed at dosages with high cement proportions for L:C ratio. This ratio could be attributed to the greater heat of hydration of the cement during curing. Lower bound values in shrinkage strains were observed for L:C proportions of 100:0. The treatment was effective as there were no patterns of warping or curling of the linear shrinkage bars after drying as was the case in control soil. The formation of hairline surficial cracks gradually developed in the depth direction of the linear shrinkage bar mold under constrained boundary conditions. The localization of shrinkage cracks is strongly dependant on the zones of moisture concentration with in the specimen. As expected, the increase in moisture content to liquid limit (LL) of the soil-binder mixture increased with the percentage of shrinkage strains. This behavior could be attributed to the availability of more moisture in the case of specimens prepared at close to saturation moisture content.

Majority of the voids in the three phase system of the stabilized soil is occupied by water which predominantly governs the interparticle bonding forces. The resultant void spaces created during drying due to hydration or mobilization of excess water along the linear shrinkage bar might have resulted in gradient of moisture concentrations and subsequently the generation of tensile stresses. The disruption or disturbance in the soil structure due to the domination of the tensile stresses at the initial crack surfaces results in the propagation of the cracks along the depth of the treated soil. This results in an open fabric, which yields space for rearrangement of soil particles in the voids. The collapse of the soil structure could both be in transverse and longitudinal directions.

Visual observation of all the treated specimens prepared at both (0.8 and 1.3) w/c ratios confirms this behavior in shrinkage patterns. Also, there is a decrease in unit weight for samples compacted at liquid limit and hence, more water and relatively fewer solid particles exist per unit volume. It should also be noted that the difference in shrinkage strains at liquid limit relative to the specimens compacted at the molding water content is insignificant.

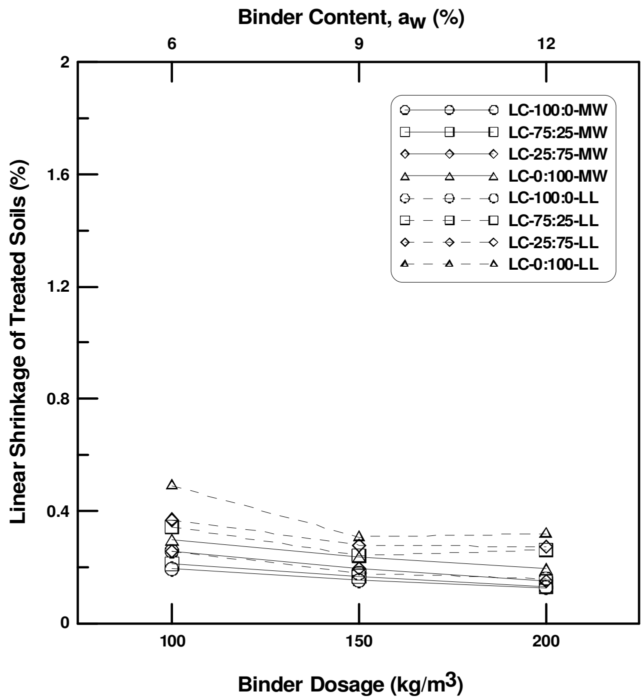


(a)

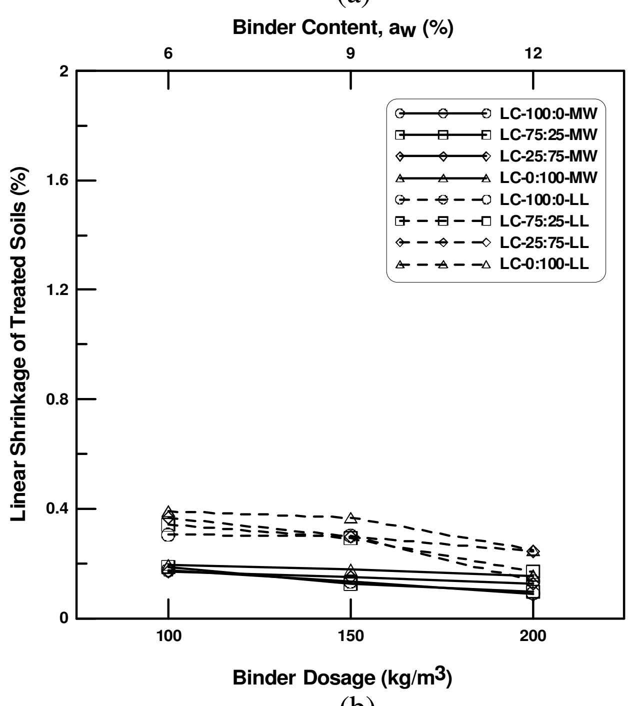


(b)

Figure 4.4 Effect of binder content at 0.8 w/c ratio, a_w (%) on linear shrinkage strains at 7 day curing; (a) treated samples from site1 (b) treated samples from site 2



(a)



(b)

Figure 4.5 Effect of binder content at 1.3 w/c ratio, a_w (%) on linear shrinkage strains at 7 day curing; (a) treated samples from site1 (b) treated samples from site 2

The results obtained from the present study were compared with the previous work by Bhadriraju (2005). Typical calculations are shown in the Table 4.3 for specimens treated at 25:75 (L:C) stabilizer proportion and 200 kg/m^3 binder dosage for both the sites in order to address the amount of quantities used for the preparation of the UCS, free swell and shrinkage samples. Table 4.4 shows the linear shrinkage strains at liquid limit and molding water content for site 1 after 7 day curing period with varying dosage rates for different w/c ratios (0.8, 1.0 and 1.3). Similar variations of linear shrinkage strains for site 2 were shown in Table 4.5.

Typical plots at 25:75 (L:C) proportions, 200 kg/m^3 binder dosage and 7 day curing showing the variation of linear shrinkage strains with liquid limit and molding water content at varying w/c ratios were shown in the Figure 4.6 for sites 1 and 2, respectively. As discussed earlier though the shrinkage strain is negligible for all the treated samples, it was observed that the specimens at 1.3 w/c ratio shrunk more than the ones treated at 1.0 and 0.8. Hair line cracks were observed on the specimens treated at 0.8 w/c ratio and shrinkage strains can almost be neglected. The same trend was observed in all the stabilizer proportions and dosage rates. Figure 4.7 shows typical plots of the variation of linear shrinkage strains with varying stabilizer proportions at liquid limit after 7 day curing period for different w/c ratios at 200 kg/m^3 binder dosage for sites 1 and 2 respectively. It was observed that with increase in w/c ratio for any particular L:C proportion, linear shrinkage strain increased proportionally.

Table 4.3 Typical calculations for 25:75 (L:C) binder proportion, 200kg/m^3 binder dosage material quantities per batch*

	w-c Ratio	**Wt of Dry Soil (gm)**	**(1) Natural Water (gm)**	**(2) Water Added (gm)**	**(1+2) Total Water (gm)**	**(3) Lime (gm)**	**(4) Cement (gm)**	**(3+4) Total Binder (gm)**
Site 1	0.8	3761	908.4	364.35	1272.74	113.86	341.57	455.43
	1	3761	908.4	455.43	1363.82	113.86	341.57	455.43
	1.3	3761	908.4	592.05	1500.45	113.86	341.57	455.43
Site 2	0.8	3761	843.06	364.35	1207.41	113.86	341.57	455.43
	1	3761	843.06	455.43	1298.49	113.86	341.57	455.43
	1.3	3761	843.06	592.05	1435.12	113.86	341.57	455.43

* Batch in this case represents 4 UCS specimens + 1 Free swell specimen + 2 Linear shrinkage specimens

Table 4.4 Linear shrinkage strains in (%) for site 1 after 7 day curing period with varying dosage rates for different w/c ratios at (a) LL and (b) MW

(a)

Binder Dosage (kg/m^3)	w/c	100-0	75-25	25-75	0-100
	0.8	0.132	0.145	0.203	0.207
100	1	0.174	0.1894	0.238	0.272
	1.3	0.193	0.212	0.257	0.298
	0.8	0.091	0.104	0.135	0.17
150	1	0.115	0.128	0.164	0.189
	1.3	0.152	0.167	0.196	0.237
	0.8	0.052	0.088	0.101	0.149
200	1	0.093	0.104	0.138	0.172
	1.3	0.127	0.131	0.149	0.193

(b)

Binder Dosage (kg/m^3)	w/c	100-0	75-25	25-75	0-100
	0.8	0.194	0.247	0.271	0.322
100	1	0.216	0.3	0.328	0.406
	1.3	0.257	0.344	0.369	0.490
	0.8	0.109	0.165	0.209	0.214
150	1	0.154	0.208	0.233	0.278
	1.3	0.178	0.241	0.278	0.307
	0.8	0.102	0.177	0.191	0.219
200	1	0.134	0.222	0.227	0.262
	1.3	0.159	0.261	0.273	0.319

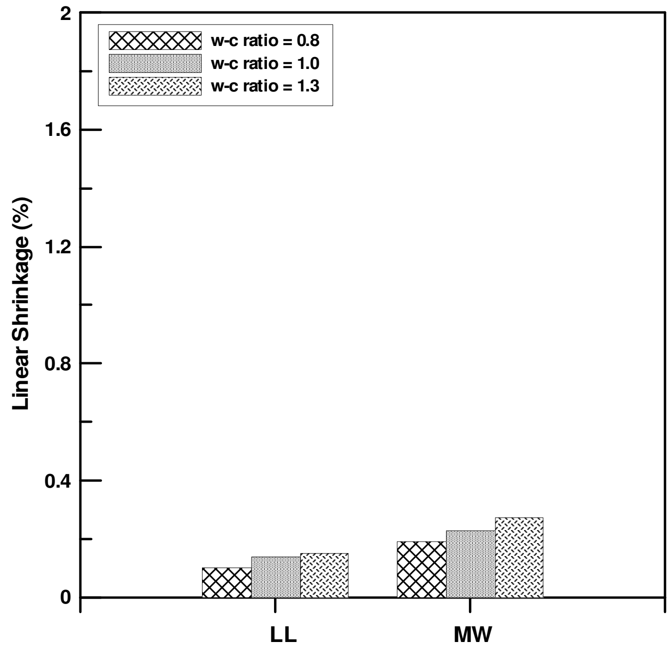
Table 4.5 Linear shrinkage strains in (%) for site 2 after 7 day curing period with varying dosage rates for different w/c ratios at (a) LL and (b) MW

(a)

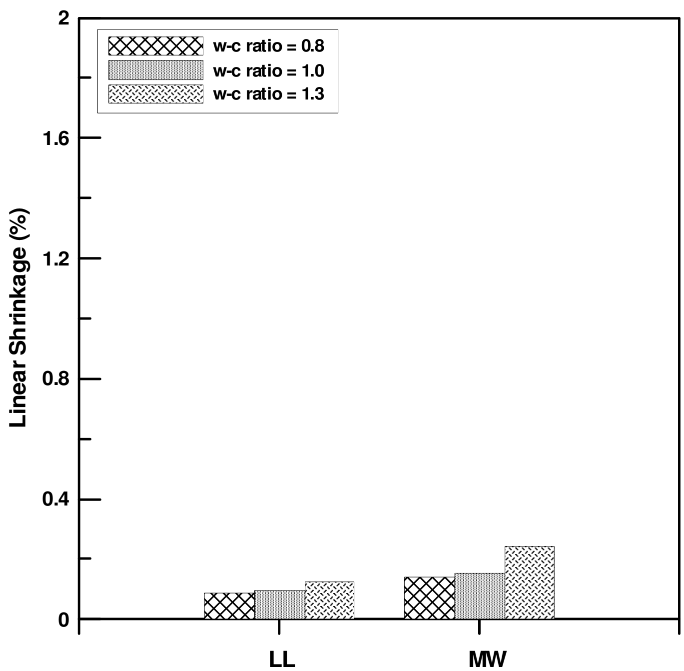
Binder Dosage (kg/m^3)	w/c	100-0	75-25	25-75	0-100
	0.8	0.102	0.134	0.141	0.153
100	**1**	0.138	0.116	0.153	0.172
	1.3	0.175	0.188	0.17	0.194
	0.8	0.076	0.098	0.111	0.129
150	**1**	0.108	0.097	0.125	0.148
	1.3	0.133	0.124	0.149	0.177
	0.8	0.051	0.082	0.088	0.102
200	**1**	0.068	0.068	0.096	0.12
	1.3	0.090	0.096	0.124	0.155

(b)

Binder Dosage (kg/m^3)	w/c	100-0	75-25	25-75	0-100
	0.8	0.133	0.173	0.239	0.277
100	**1**	0.266	0.285	0.328	0.33
	1.3	0.304	0.341	0.366	0.39
	0.8	0.119	0.144	0.149	0.185
150	**1**	0.158	0.181	0.195	0.237
	1.3	0.301	0.292	0.298	0.367
	0.8	0.064	0.099	0.139	0.155
200	**1**	0.104	0.125	0.151	0.173
	1.3	0.135	0.169	0.244	0.249

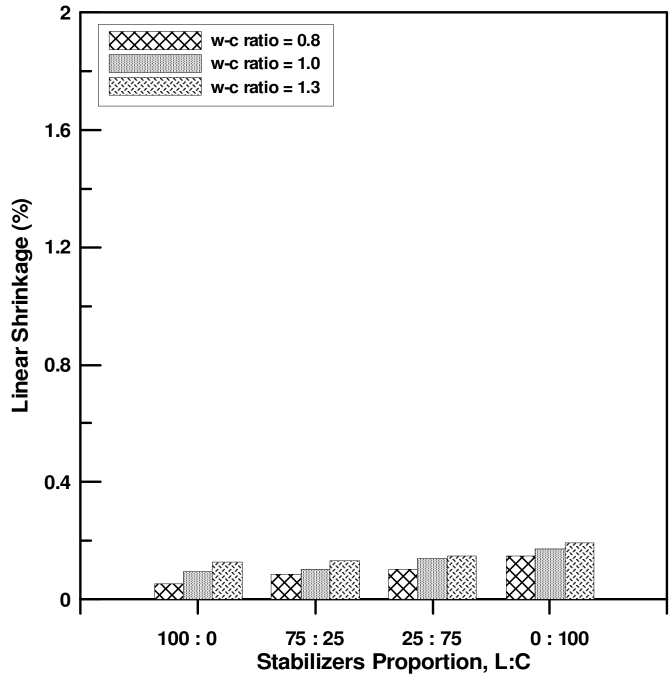


(a)

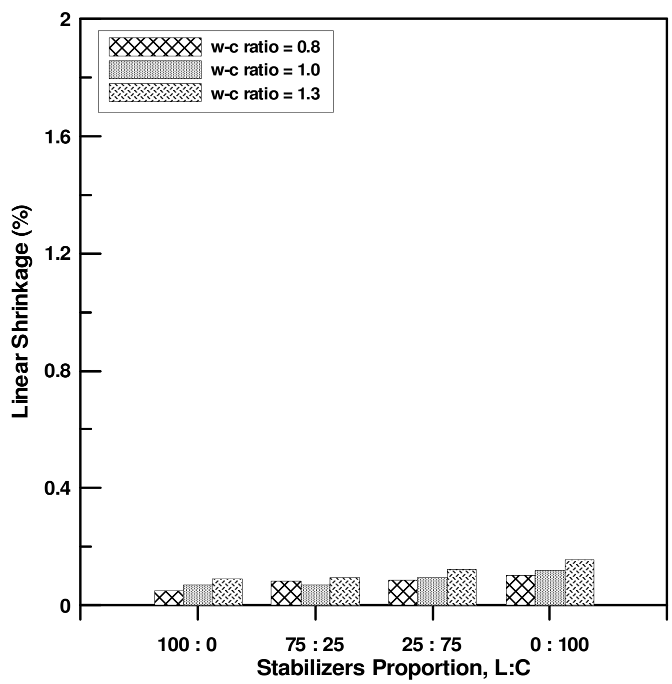


(b)

Figure 4.6 Typical plots for samples treated at 25:75 (L:C) proportion 200 kg/m^3 binder dosage and 7 day curing showing the variation of linear shrinkage strains with LL and MW at varying w/c ratios for (a) site 1 and (b) site 2



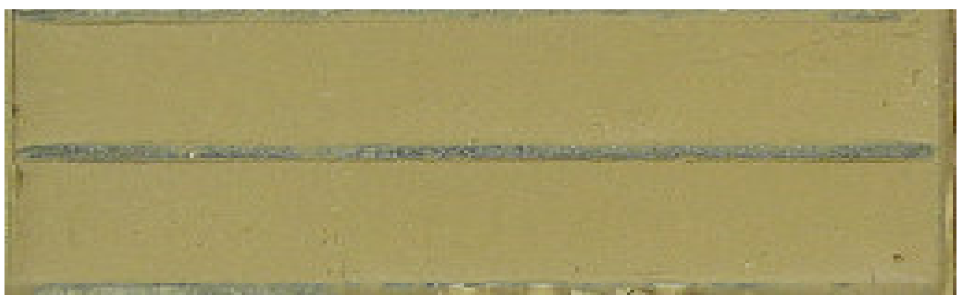
(a)



(b)

Figure 4.7 Typical plots for samples treated at 200 kg/m^3 binder dosage after 7 day curing period showing the variation of linear shrinkage strains with varying stabilizer proportions at LL for different w/c ratios for (a) site 1 and (b) site 2

Figure 4.8 (a) shows the linear shrinkage bar with control soil at molding water content before subjecting to shrinkage phenomenon. Figure 4.8 (b) shows the patterns of shrinkage in control soil along the transverse and longitudinal directions. The untreated samples also were brittle and warped considerably in the vertical direction due to the rigidity of the walls of shrinkage mold. Figure 4.9 shows the linear shrinkage patterns due to the deep mixing treatment using 25:75 (L:C) stabilizer proportion at 200 kg/m^3 and 7 day curing at different w/c ratios. As explained earlier all the treated samples exhibited negligible cracks along the transverse direction and no warping or curling of the specimens was observed.

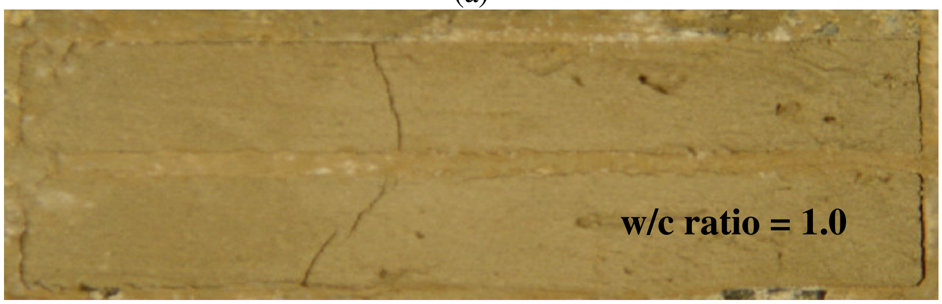


(a)

(b)

Figure 4.8 Linear shrinkage patterns of control soil from site 1 (a) before subjecting to shrinkage and (b) after shrinkage

(a)



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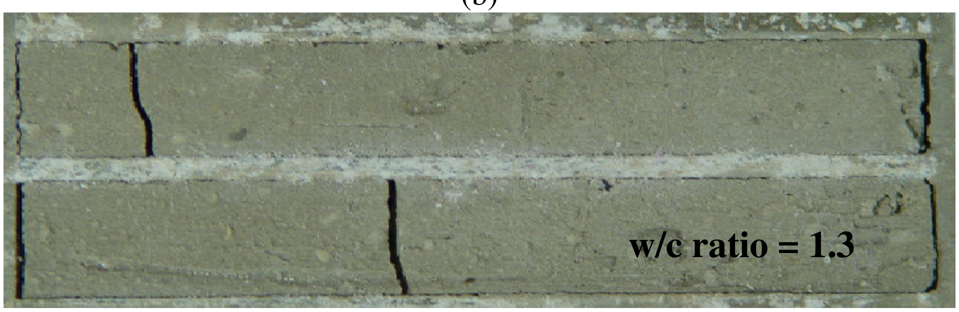


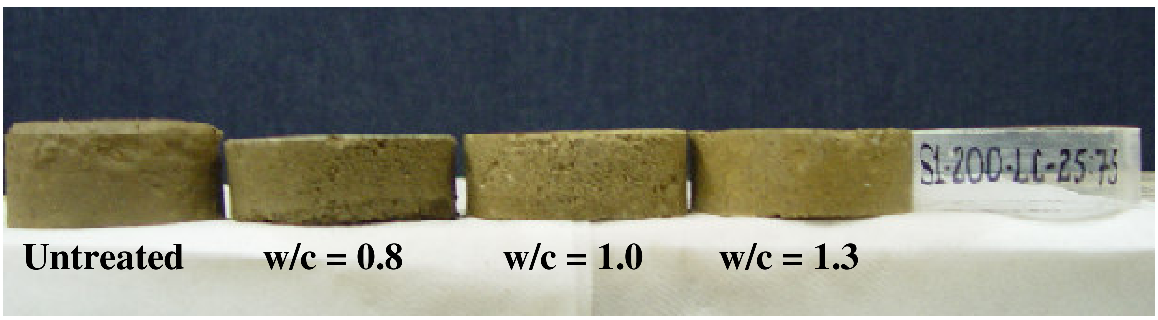
(c)

Figure 4.9 Linear shrinkage patterns due to the deep mixing treatment using 25:75 (L:C) stabilizer proportion at 200 kg/m^3 and 7 day curing at different w/c ratios

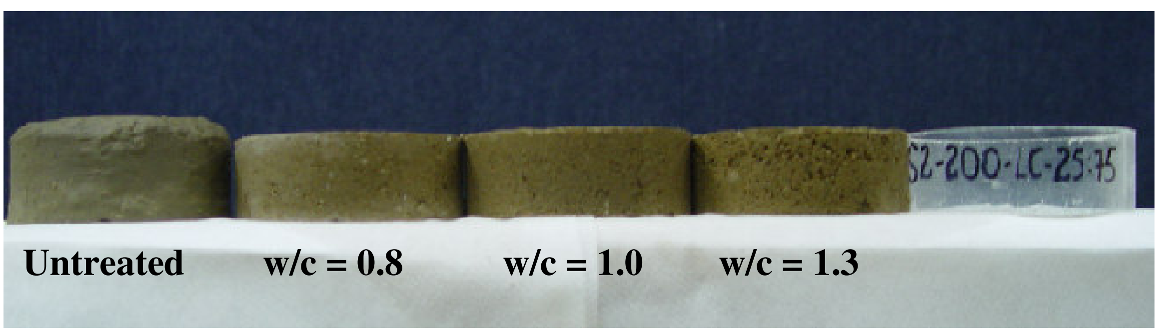
4.3.2 Free Swell Test Results

Free swell tests were conducted on treated samples from both sites at 0.8 and 1.3 w/c ratios, all lime-cement proportions, binder contents and 7 and 14 day curing periods. The free swell of the treated samples was reduced to near zero. No data was reported in the current study as the magnitude of potential free swell was close to zero. This could be due to very stiff bonding between the binders and the soil over period of curing or may be explained as due to high compacted density of the treated free swell specimens resulting in delayed saturation with time and reduced heaving.

Figure 4.10 better explains the comparison of specimens subjected to swelling treated at different w/c ratios and 25:75 (L:C) stabilizer proportion and 200 kg/m^3 binder dosage along with the untreated specimen from both the sites. Thus, it is observed that the untreated specimens from both the sites shows considerable swell whereas, no swell effect observed on the treated ones.



(a)



(b)

Figure 4.10 Comparisons of treated specimens at different w/c ratios along with the untreated specimen for (a) site 1 and (b) site 2

4.3.3 Unconfined Compressive Strength Test Results

The unconfined compressive strength (UCS) tests were performed to study the variation in the stiffness property with respect to the variables mentioned earlier at w/c ratios of 0.8 and 1.3. The treated specimens exhibited a brittle failure and a sudden drop in the post peak strength was noticed in the stress strain response of the treated specimens. A typical plot showing the stress strain response, post peak strength and failure axial strain profiles treated at 25:75 (L:C) proportion, 200 kg/m^3 binder dosage and 7 day curing period for site 1 demonstrates the reduction in post peak strength with variations in w/c ratios (see Figure 4.11). It was observed that with increase in w/c ratio, the peak (failure) unconfined compressive strength decreased. The failure strains were observed to be in between 1-2% for all the specimens and no particular trend was observed in the failure strains with change in w/c ratios. The UCS values were evaluated and compared for all three w/c ratios. The effect of mixing duration and mixing speed was also not considered during the study.

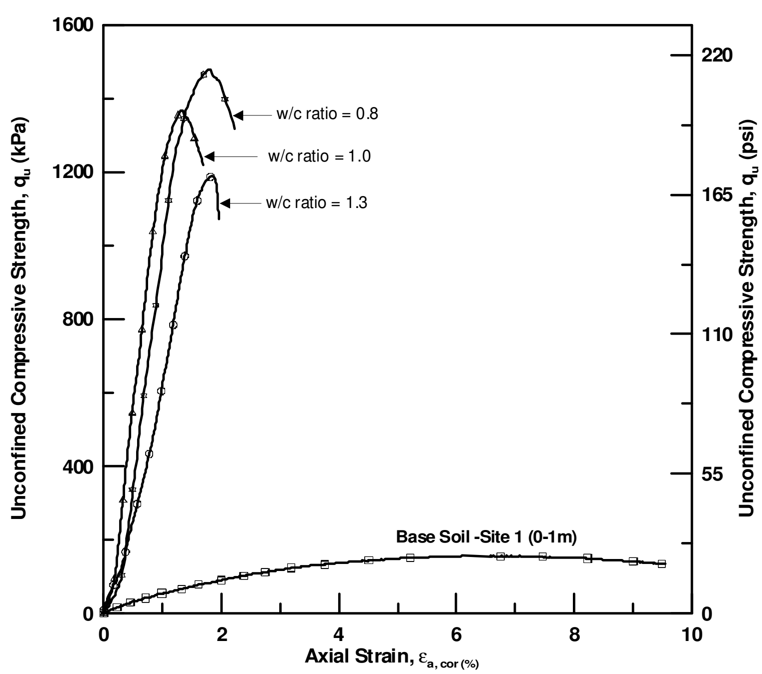
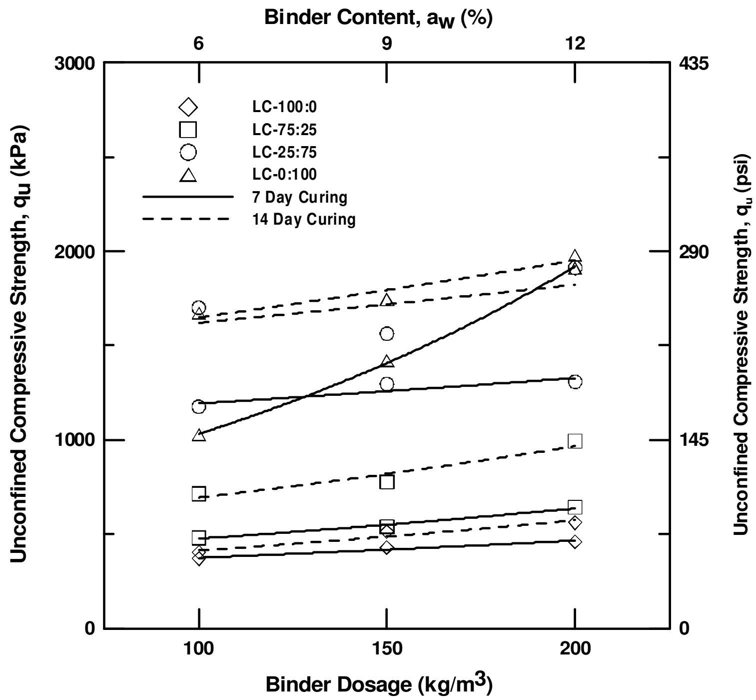


Figure 4.11 Stress strain response, post peak strength and failure axial strain profiles at different w/c ratios - Site 1, 25:75 (L:C) proportion, 200 kg/m^3 and 7 day curing

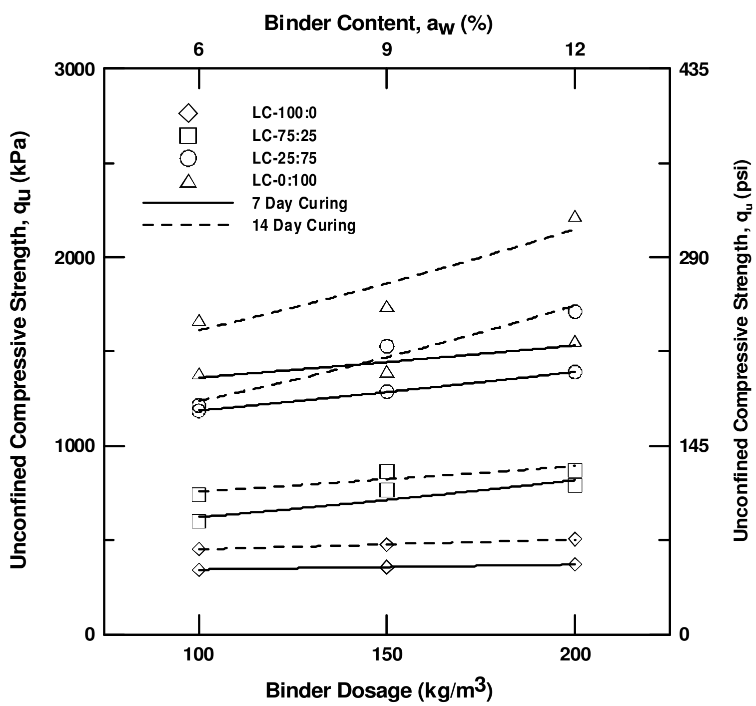
The results from UCS tests were analyzed to study the effects of dosage rate, binder proportion and curing period on stiffness property.

4.3.3.1 Effects of Dosage rates

Figures 4.12 and 4.13 depict the effect of binder content, a_w (%) on the unconfined compressive strengths of treated soils from both sites at 0.8 and 1.3 w/c ratios respectively. For both curing periods, the increase in strength with dosage rate was prominent for a binder proportion of 0:100 (L:C) compared with those of 100%, 75% and 25% lime proportion. Present study did not notice any enhancement in strength for 100% lime treatment. This may be because of short curing periods considered here.



(a)



(b)

Figure 4.12 Effects of binder dosage, binder proportions and curing period on UCS values of treated specimens at 0.8 w/c ratio for (a) site 1 and (b) site 2

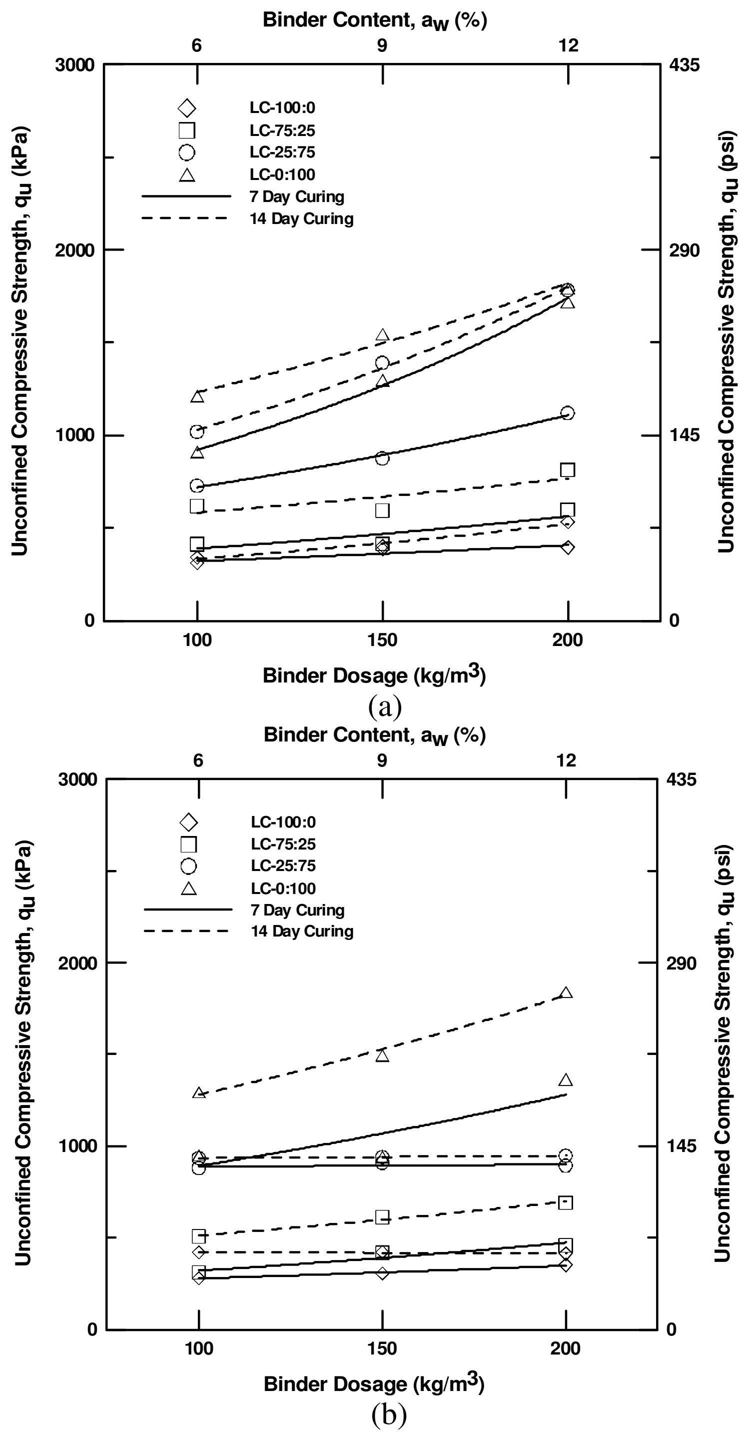


Figure 4.13 Effects of binder dosage, binder proportions and curing period on UCS values of treated specimens at 1.3 w/c ratio for (a) site 1 and (b) site 2

Usually lime reactions are slow in nature compared to those associated with cement treated and therefore the improvements were noticed at least after a month of curing. The trend of strength gain with dosage rate was identical for both w/c ratios and soil types. Increased scatter in strength was observed in most of the cases with increase in the dosage rate in each water-cement ratio. As expected, strength gain in the treated soil samples showed a consistent increasing trend with increasing curing period due to formation and hardening of pozzalonic compounds with time.

The results obtained from the present study were compared with those from previous research by Bhadriraju (2005). Table 4.5 shows the UCS values of the specimens prepared at different dosage rates, binder proportions and w/c ratios 0.8, 1.0 and 1.3 subjected to two curing periods for site 1. Similar variations of UCS values for site 2 were presented in Table 4.6.

Typical plots, at 25:75 (L:C) proportion and 14 day curing showing the variation of unconfined compressive strength with dosage rate at w/c ratios 0.8, 1 and 1.3 were depicted in Figure 4.14 for sites 1 and 2. It can be noticed for all binder dosage rates, the UCS values decreased with increase in w/c ratio. Also, for a given w/c ratio in most cases, the strength increased with increase in dosage rate. Moreover, identical response in strength to an increase in binder dosage rate can be observed for other w/c ratios. The decrease in UCS values with increase in w/c ratio may be due to larger amount of water content making the material softer and less stiff, thus developing weak bonding between the binder and the soil particles.

Table 4.5 UCS values in kPa of the specimens treated at different dosage rates at varying w/c ratios (0.8, 1.0 and 1.3) for site 1 at (a) 7 day curing and (b) 14 day curing

(a)

Binder Dosage (kg/m^3)	w/c	100-0	75-25	25-75	0-100
	0.8	370	480	1175	1025.8
100	**1**	357	443.02	795.8	990.47
	1.3	310	412.7	727	911.23
	0.8	428.5	540	1295.6	1420.6
150	**1**	409	528.66	901.13	1330
	1.3	386.2	413.9	874.8	1297
	0.8	460	642.26	1308	1910.2
200	**1**	422.66	613.45	1256.36	1824
	1.3	394.1	598	1120	1719

(b)

Binder Dosage (kg/m^3)	w/c	100-0	75-25	25-75	0-100
	0.8	403	714	1700	1672
100	**1**	369	649	1218.4	1448.8
	1.3	340	617.9	1019.2	1211.1
	0.8	511.3	775.4	1562	1744.1
150	**1**	480	740	1426.6	1654
	1.3	402.2	594	1390	1547
	0.8	560.2	994	1911.5	1980
200	**1**	540	845	1836	1928.2
	1.3	532	813.3	1782	1790

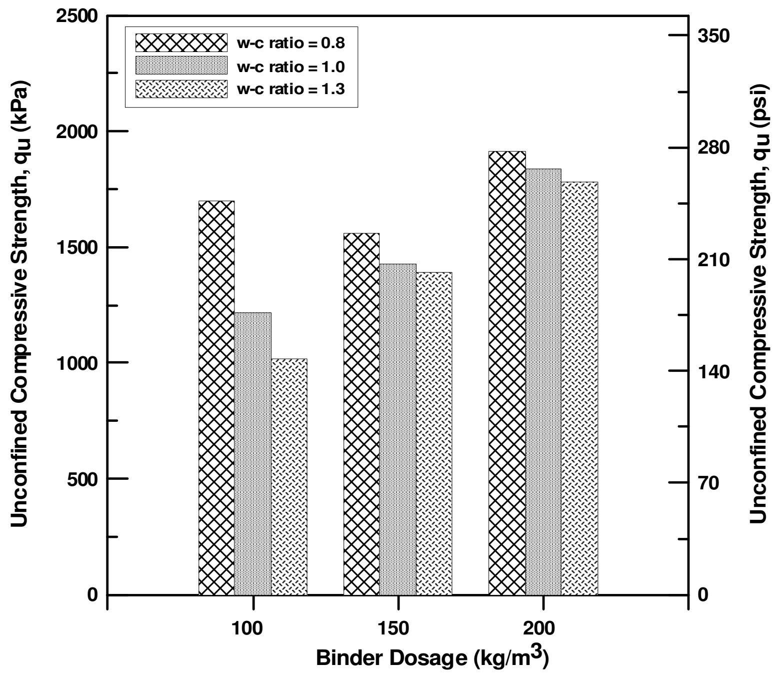
Table 4.6 UCS values in kPa of the specimens treated at different dosage rates at varying w/c ratios (0.8, 1.0 and 1.3) for site 2 at (a) 7 day curing and (b) 14 day curing

(a)

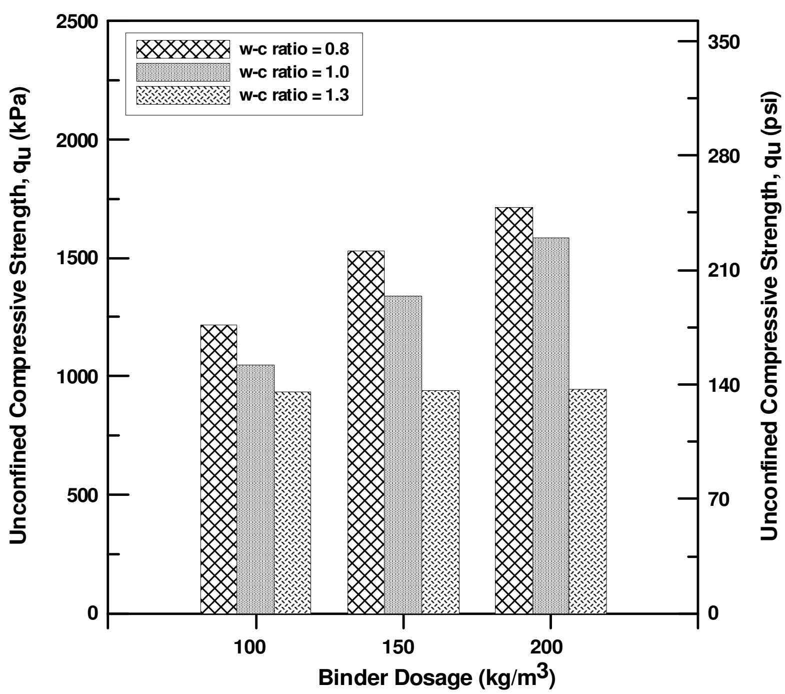
Binder Dosage (kg/m^3)	w/c	100-0	75-25	25-75	0-100
	0.8	344	600	1186	1386
100	**1**	302	390	946	1123.5
	1.3	280	310	880	949.9
	0.8	357	765	1287	1395
150	**1**	344.7	442.5	1190	1216
	1.3	307.1	419	908.3	940.5
	0.8	370.3	790.7	1391.2	1559
200	**1**	353	481	1351	1422
	1.3	351	457.5	893	1363.3

(b)

Binder Dosage (kg/m^3)	w/c	100-0	75-25	25-75	0-100
	0.8	453	740	1214	1667
100	**1**	430	513.8	1047	1595
	1.3	419.7	507	934	1294
	0.8	475	865	1528	1742
150	**1**	450	653.5	1341	1605
	1.3	422.6	611.8	940	1493
	0.8	505	870.6	1711.3	2221
200	**1**	487	716.7	1585	1996
	1.3	413	690	947	1841



(a)



(b)

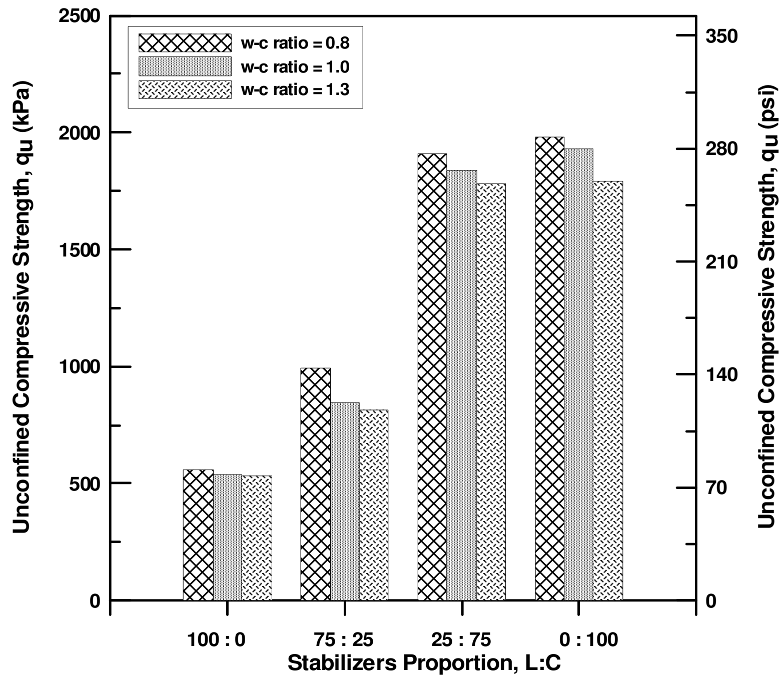
Figure 4.14 Typical plots at 25:75 (L:C) proportion and 14 day curing showing the variation of UCS values with different dosage rates at varying w/c ratios for (a) site 1 and (b) site 2

4.3.3.2 Effects of Binder proportions

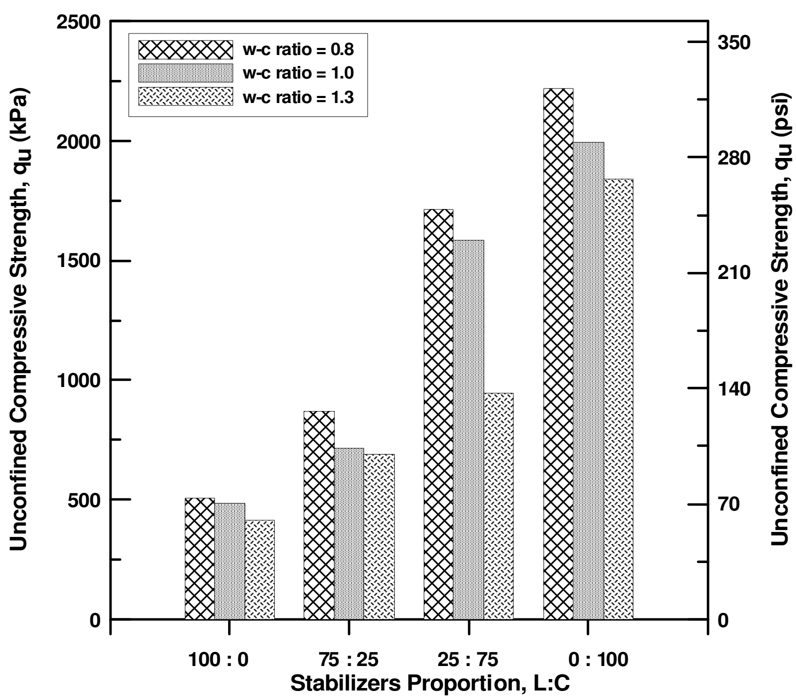
Figures 4.12 and 4.13 also illustrate the strength gain with different lime-cement proportions and dosage rates corresponding to 7 and 14 day curing periods. Based on the observed trends, the following discussion was presented.

The strength gain for specimens treated at 100% cement was observed to be four times more than the strength gain by specimens treated at 100% lime for any particular w/c ratio. This may be due to the faster pozzalonic reactions taking place when cement was treated with soil particles. The highest 14-day strength of treated specimens at 0.8 w/c ratio from site 1 was approximately 1980 kPa (12.6 ksi) and from site 2 was approximately 2220 kPa (14.1 ksi) both at 200 kg/m^3 and 100% cement proportion. Similarly, the highest 14-day strength of treated specimens at 1.3 w/c ratio from site 1 was approximately 1790 kPa (11.4 ksi) and from site 2 was approximately 1840 kPa (11.7 ksi) both at 200 kg/m^3 and 100% cement proportion. For treated soil specimens from both sites, the highest strength was achieved when 100 percent cement was mixed with soil at various dosage rates. The addition of lime however decreased the strength gain in the treated soils. For both the w/c ratios, the UCS of 100 percent lime treated specimens at a dosage rate of 200 kg/m^3 is approximately half the value of unconfined compressive strength observed for 100 percent cement treated soil at a dosage rate of 100 kg/m^3. Hence it is important to understand the effects of adding lime, which primarily is governed by the application of the deep mixing. In cases where strength gain may not be of considerable importance, but the area of influence of treatment is, higher percentages of lime are more preferred due to active fracture of the clay structure and increased radius of influence after deep mixing treatment. In such cases, the center to center spacing if isolated columns are being used can be increased. Cement stabilization is more preferred in soft clays and where mass stabilization or solidification is intended to achieve high strengths.

Figure 4.15 shows typical plots depicting the variation of UCS values after 14 day curing period for different binder proportions at a binder dosage of 200 kg/m^3 for both w/c ratios. It was observed that with increase in w/c ratio for a given L:C proportion, unconfined compressive strengths decreased proportionally.



(a) Site 1

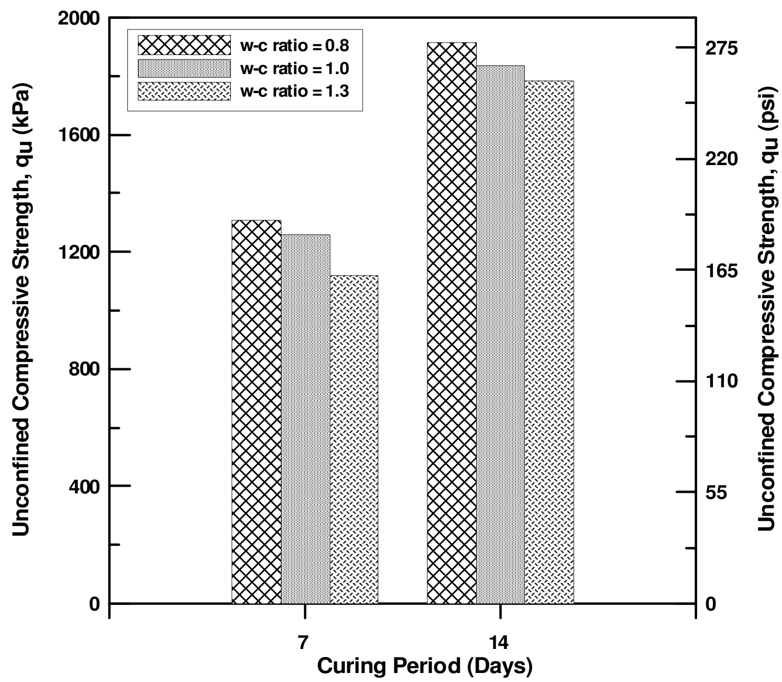


(b) Site 2

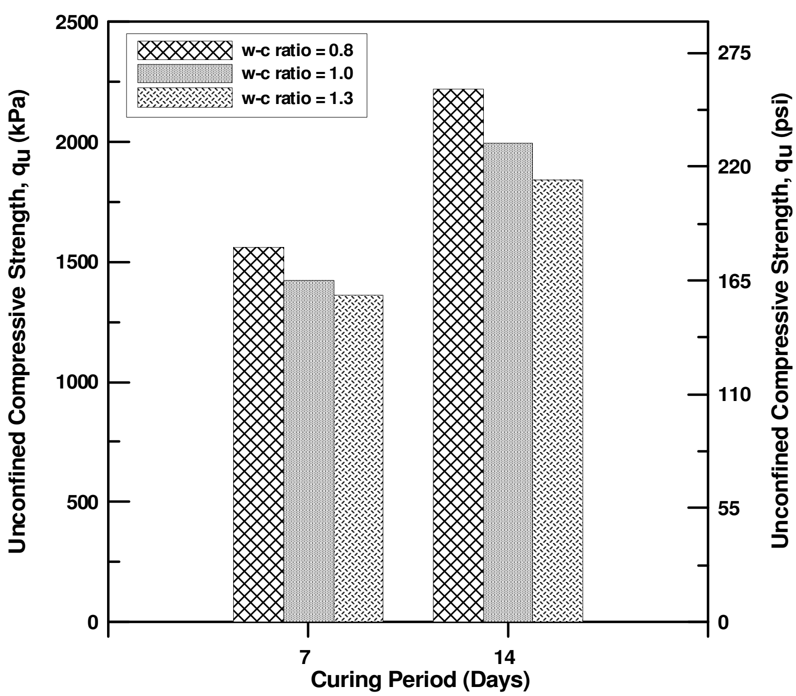
Figures 4.15 Typical plot of the variation of the UCS values after 14 day curing period with varying stabilizer proportions for different w/c ratios at 200 kg/m^3 binder dosage

4.3.3.3 Effect of Curing Period

The effect of curing period on strength gain for both w/c ratios was studied at 7 and 14 days. Typical plots at 25:75 (L:C) proportion, 200 kg/m^3 binder dosage showing the variation of the UCS values with curing periods (7 and 14) for different w/c ratios for sites 1 and 2 were shown in Figures 4.16. Various plots were developed in order to predict the improvement in the unconfined compressive strength with curing period. Typical failures of the UCS specimens treated at 25:75 (L:C) proportion, 200 kg/m^3 binder dosage and 14 day curing were shown in Figure 4.17.



(a)



(b)

Figure 4.16 Typical plots at 25:75 (L:C) proportion, 200 kg/m^3 binder dosage showing the variation of the UCS values with curing periods (7 and 14) for different w/c ratios for (a) site 1 and (b) site 2

(a)

(b)

Figure 4.17 Typical failures of UCS specimens at 25:75 (L:C) binder proportion and 200 kg/m^3 binder dosage after 14 day curing period (a) 0.8 and (b) 1.3 w/c ratio

Improvement ratio may be defined as the ratio of unconfined compressive strength of treated soil after a curing period of t days ($q_{u,\ t}$) to the unconfined compressive strength of untreated control soil ($q_{u,\ o}$). A factor, α defined as improvement ratio is included to better understand the viability of choosing the binder proportion and dosage rate based on the target strength properties and the strength properties of the untreated soil.

$$\alpha = \frac{q_{u,t}}{q_{u,o}}$$

For estimation purposes, $q_{u,\ t}$ corresponds to the ultimate strength of the treated samples, usually at the end of curing period viz. 14, 28 or 56 days. Figure 4.18 present the improvement ratio with various binder dosages corresponding to different L:C proportions for sites 1 and 2 respectively at 0.8 w/c ratio. Similar trends were also observed for 1.3 w/c ratio and depicted in Figure 4.19. Knowing the improvement ratio required for a particular deep mixing project application, the required binder dosage can be calculated from the graphs based on several factors including treatment area ratio, spacing, economics in selection of binder etc. For any particular w/c ratio, the specimens treated with 100% cement followed by those treated at 75% were best proven in improving strengths. The improvement ratios of these were found to be 3 to 4 times more than the specimens treated at 25% or 0% cement proportions.

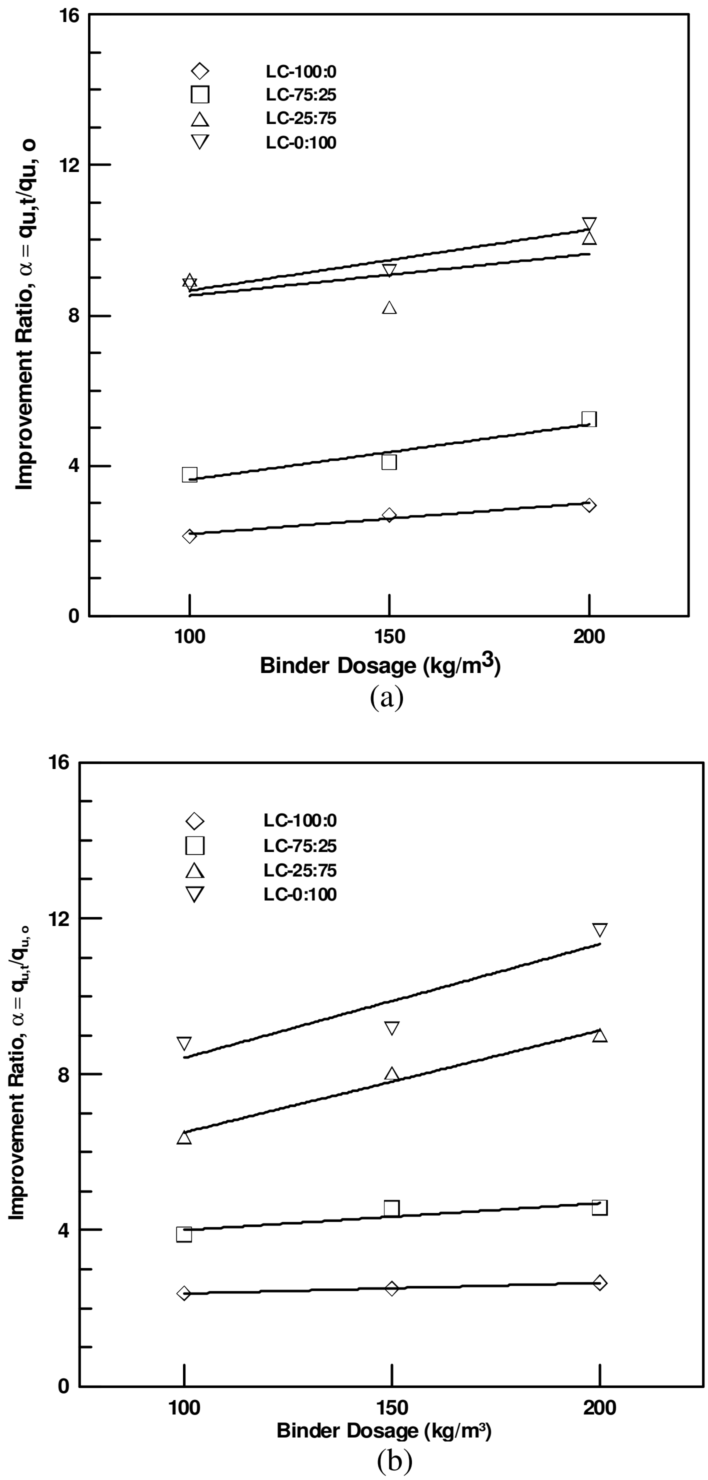
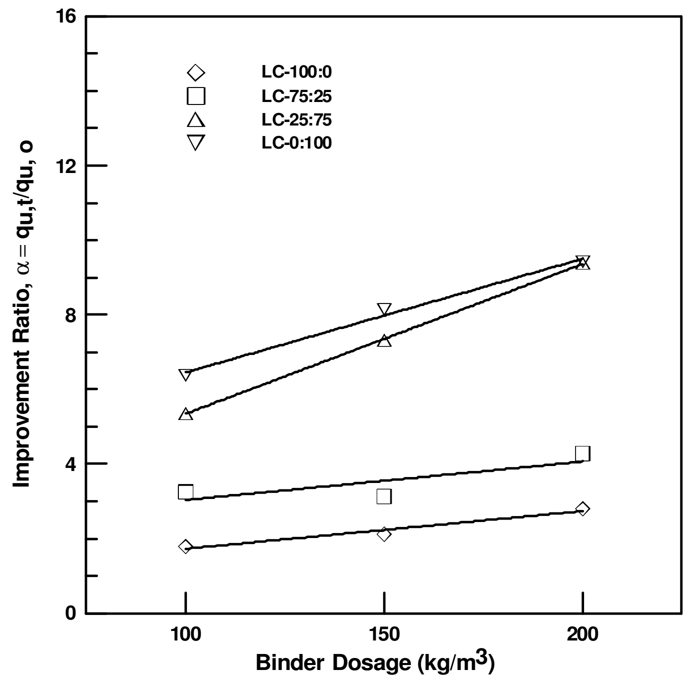
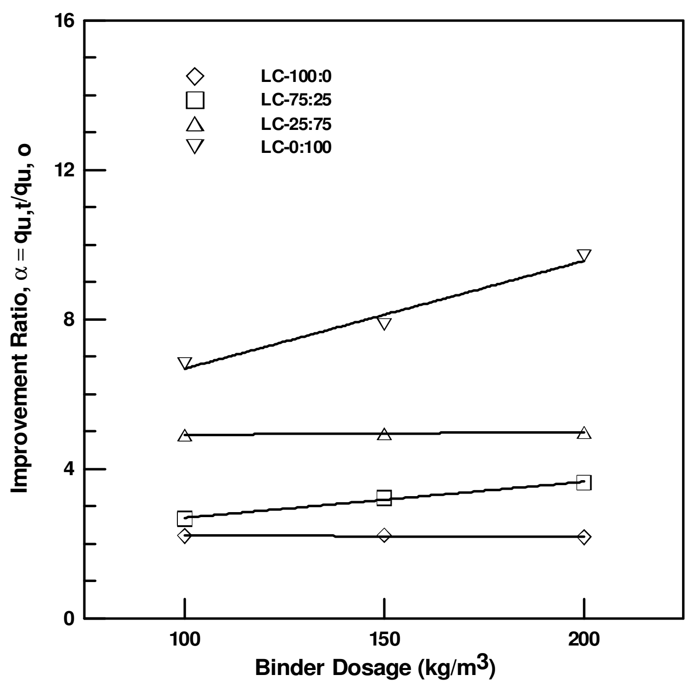


Figure 4.18 Improvement ratio versus binder dosage treated at 0.8 w/c ratio, 14 day curing for (a) site 1 and (b) site 2



(a)



(b)

Figure 4.19 Improvement ratio versus binder dosage treated at 1.3 w/c ratio, 14 day curing for (a) site 1 and (b) site 2

4.3.4 Stiffness measurements from Bender Element Tests

Tables 4.7 and 4.8 present the shear wave velocity data obtained from the bender element tests at varying w/c ratios for sites 1 and 2 respectively. In each w/c ratio, the s-wave velocity increased with increasing binder dosages as a result of closer packing due to pozzalonic reactions and continuity in the soil-structure. Increase in cement proportion also increased the shear wave velocity and hence the shear modulus of the soil. Increase in curing period from 7 to 14 decreased the time of flight and hence increased the shear wave velocity. However, the increase was not very considerable. The bender element data corresponds to low to very low shear strains in the initial linear elastic region and hence provide maximum shear modulus,G_{max}.

Typical plots at 25:75 (L:C) proportions, 14 day curing showing the variation of shear wave velocity values with different dosage rates at varying w/c ratios were shown in the Figure 4.20 for sites 1 and 2 respectively. Similarly, plots at 25:75 (L:C) proportions, 200 kg/m^3 binder dosage showing the variation of the UCS values with curing periods (7 and 14) for different w/c ratios for sites 1 and 2 were shown in Figure 4.21. Though there is no significant difference, shear moduli values of soils treated at 0.8 w/c ratio were greater than the values at 1.0 and 1.3. This effect may be attributed to close packing of soil particles making sample stiffer at 0.8 w/c ratio than sample compacted at other two w/c ratios (1.0 and 1.3).

Table 4.7 Shear wave velocities, V_s (m/s) of the samples from site 1 treated at different proportions and varying w/c ratios at (a) 7day and (b) 14day curing

(a)

Binder Dosage (kg/m^3)	w/c	100-0	75-25	25-75	0-100
	0.8	181.18	194.3	211.9	222.2
100	1	170.5	187.76	199.45	210.05
	1.3	161.24	188.28	187.06	198.14
	0.8	204.73	226.4	254.1	264.81
150	1	208.95	218.63	256.68	256.77
	1.3	180.9	194.11	220.5	240.8
	0.8	229.31	240.4	277.18	279.08
200	1	213.08	226.46	264.09	282.4
	1.3	203.6	209.9	215.6	230.3

(b)

Binder Dosage (kg/m^3)	w/c	100-0	75-25	25-75	0-100
	0.8	186.08	187.6	231.24	249.33
100	1	173.36	189.63	224.02	237.05
	1.3	165.4	180.1	196.07	204.11
	0.8	229.06	249.93	270.07	280.19
150	1	212.56	217.27	257.82	274.3
	1.3	211.14	221.01	238.44	255.5
	0.8	236.13	250.04	289.18	314.41
200	1	230.96	237.48	278.77	301.03
	1.3	214.7	220.6	233.3	276.19

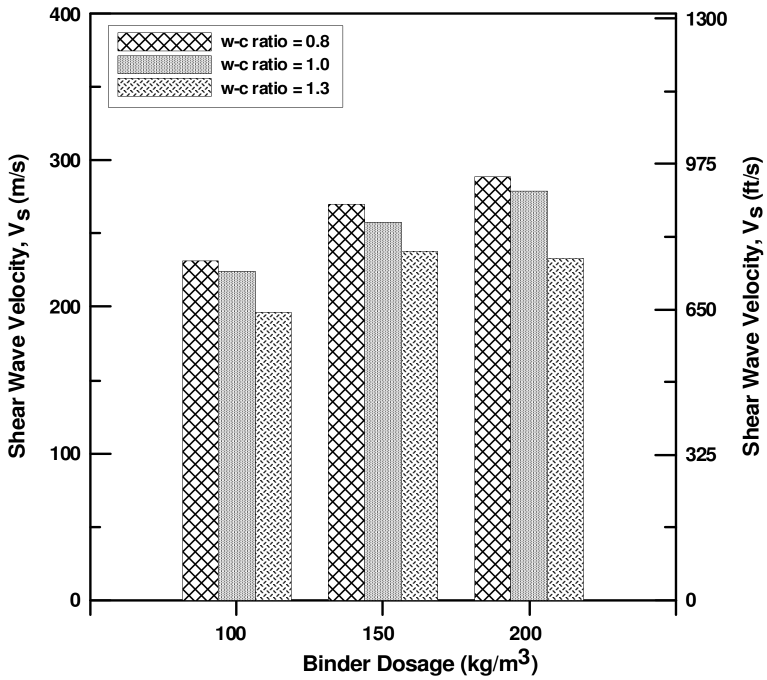
Table 4.8 Shear wave velocities, V_s (m/s) of the samples from site 2 treated at different proportions and varying w/c ratios at (a) 7day and (b) 14day curing

(a)

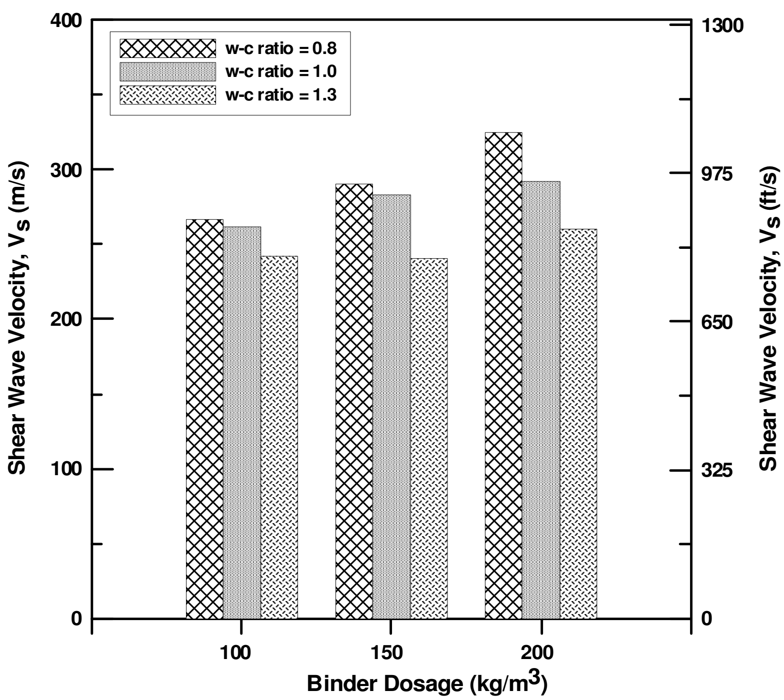
Binder Dosage (kg/m^3)	w/c	100-0	75-25	25-75	0-100
	0.8	170.11	183.21	219.79	232.16
100	1	176.2	180.9	224.28	225.83
	1.3	178.07	161.14	199.96	208.64
	0.8	214.0	229.12	260.06	358.5
150	1	198.62	203.7	262.3	292.45
	1.3	173.37	188.6	214.45	247.16
	0.8	240.13	227.74	279.0	372.47
200	1	213.5	212.41	277.28	316.38
	1.3	200.08	207.23	241.32	294.14

(b)

Binder Dosage (kg/m^3)	w/c	100-0	75-25	25-75	0-100
	0.8	220.6	242.08	266.73	315.5
100	1	220.01	237.5	261.66	284.56
	1.3	188.33	197.0	241.92	257.48
	0.8	227.05	249.5	290.61	370.0
150	1	239.72	251.54	283.05	305.01
	1.3	206.11	227.72	240.87	270.96
	0.8	235.14	256.9	325.11	391.91
200	1	219.10	223.27	292.13	322.55
	1.3	208.14	231.44	260.5	301.5

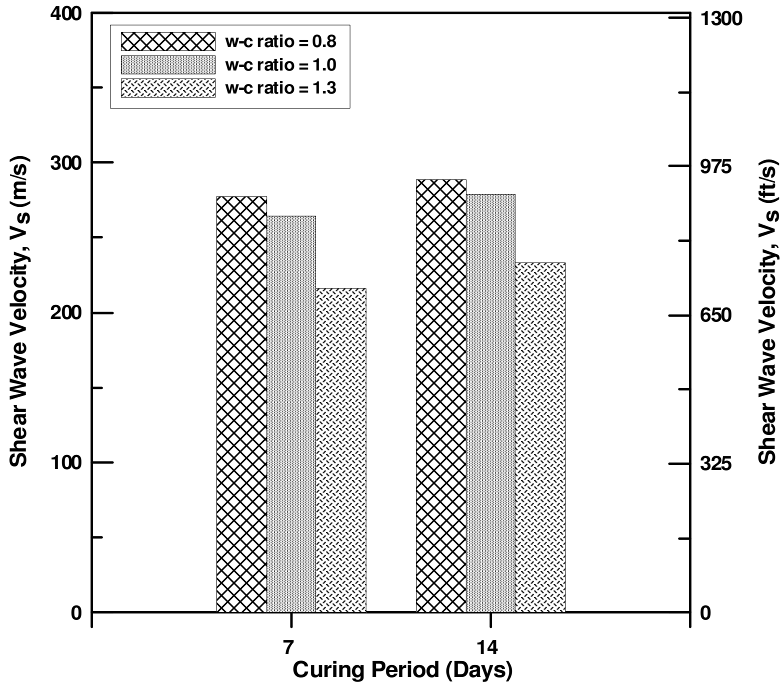


(a)

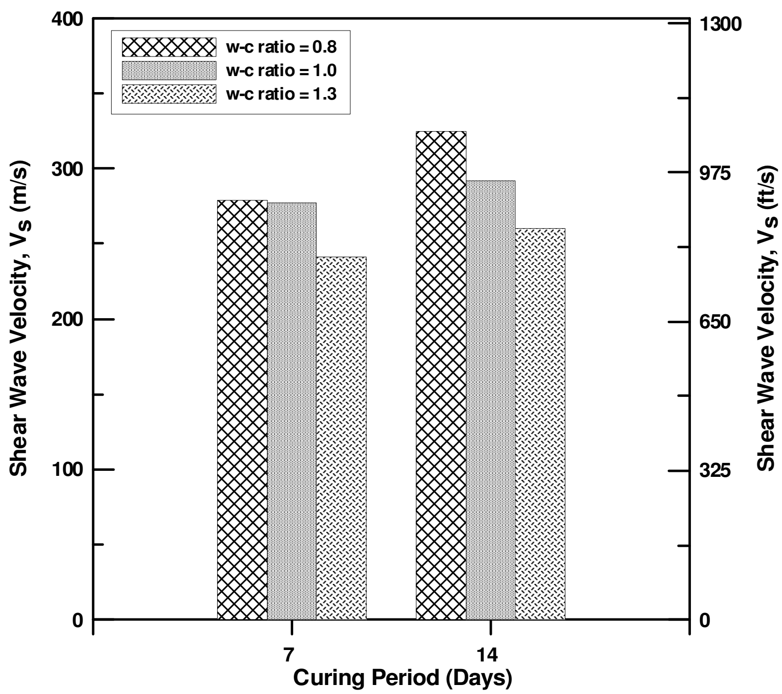


(b)

Figure 4.20 Typical plots at 25:75 (L:C) proportion and 14 day curing showing the variation of shear wave velocity values with different dosage rates at varying w/c ratios for (a) site 1 and (b) site 2



(a)



(b)

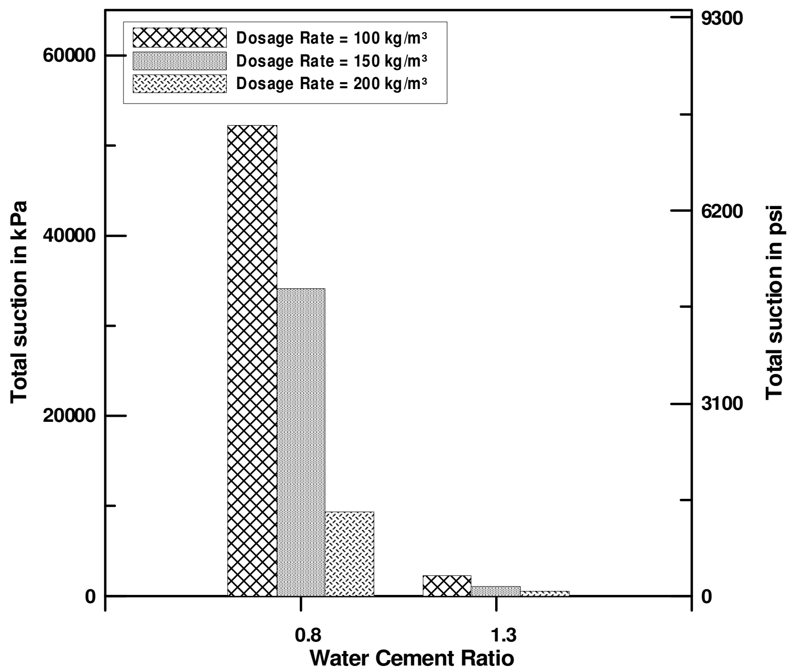
Figure 4.21 Typical plots at 25:75 (L:C) proportions, 200 kg/m^3 binder dosage showing the variation of the UCS values with curing periods (7 and 14) for different w/c ratios for (a) site 1 and (b) site 2

4.3.5 Suction Results from Filter Paper Method

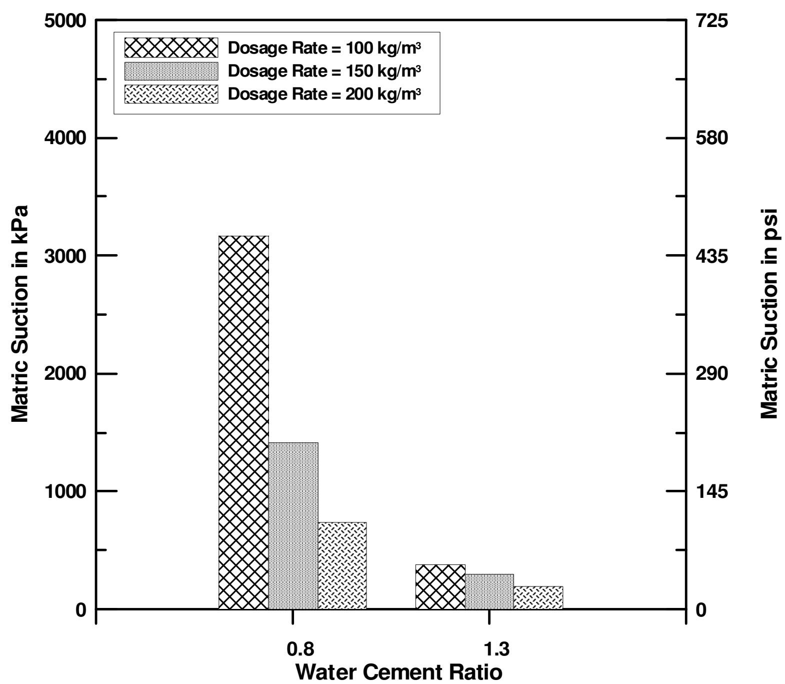
Suction tests were performed on the samples treated at 25:75 (L:C) stabilizer proportion at different binder dosages (100 kg/m^3, 150 kg/m^3 and 200 kg/m^3) and w/c ratios (0.8 and 1.3) for both the sites. The variation of suction values with w/c ratios were plotted below. As per the literature, there is a reasonable trend observed, showing the decrease in suction values with increase in w/c ratio.

It was also observed that for any particular w/c ratio, with increase in binder dosage, the suction values were found to slope down (decreased) along the horizontal axis. This decrease of the suction values may be predicted to be indicative of an increase in void space with increase in binder dosage. This substantial increase in the void space for the binder mixes may be due to the ion exchange and flocculation reactions. This may be a possible cause for the reduction of suction values with increasing the dosage rates. Normally this phenomenon is most likely applicable for 100% lime treatment. This also takes place in 100% cement treatment, but at a lower rate. However, most of the suction tests on treated specimens show a decrease in void space due to the formation of cementitious products, thus, showing increase in suction values with increase in binder dosage.

Typical plots of total and matric suctions treated at 25:75 (L:C) stabilizer proportion at different binder dosages with w/c ratios were shown in the Figures 4.22 and 4.23 for sites 1 and 2 respectively.

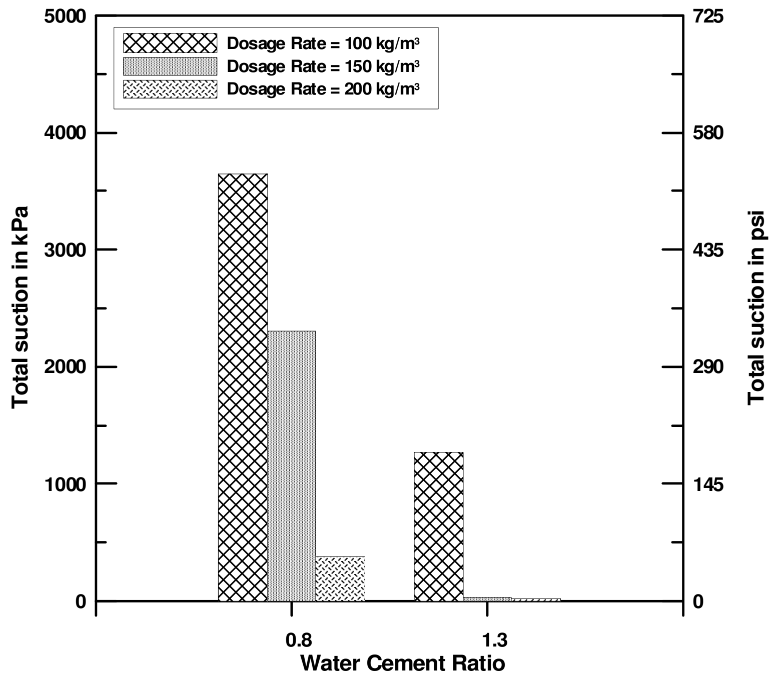


(a)

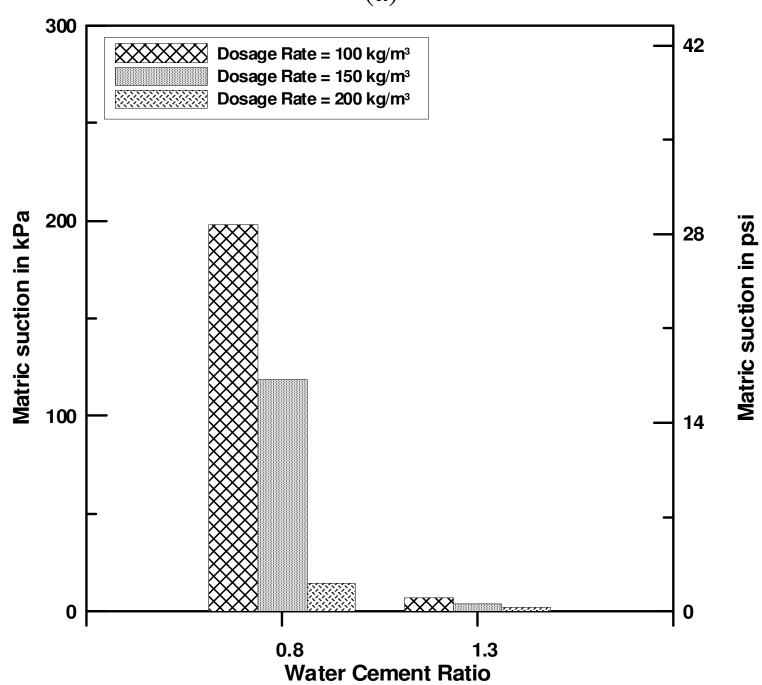


(b)

Figure 4.22 Typical plots of suction values at different dosage rates with w/c ratio treated at 25:75 (L:C) stabilizer proportion for site 1 (a) total and (b) matric suction



(a)



(b)

Figure 4.23 Typical plots of suction values at different dosage rates with w/c ratio treated at 25:75 (L:C) stabilizer proportion for site 2 (a) total and (b) matric suction

4.4 Ranking of Stabilizers Based on the Overall Performance

A simplified ranking analysis was performed to evaluate the best performing stabilizer with respect to their soil property characterization. Equal weightage to free swell, linear shrinkage and UCS properties were assigned in this analysis. Different weightage factors were added to emphasize on the intended project application. The ranking scale is designed in which each soil property varies from problematic to non problematic levels and the severity of the problem. On a scale of 1 to 5, the worst soil conditions is assigned a rank of 1 where as the best performing stabilizer is assigned a value of 5. Table 4.9 shows the ranking allocation with respect to free swell magnitudes computed as a percentage strain of actual sample height.

Table 4.9 Stabilizer performance classification based on vertical free swell strain (Chen et al. 1988, Puppala et al. 2004)

Vertical Free Swell (%)	Description of severity	Rank
0-0.5	Non-Critical	5
0.5-1.5	Marginal	4
1.5-4.0	Critical	3
> 4.0	Highly Critical	2
> 8.0	Severe	1

Table 4.10 presents the ranking based on linear shrinkage strain magnitudes, which are presented in terms of linear expansion of the soils. The ranking is based on the linear shrinkage strains reported by Nelson and Miller (1992).

Table 4.10 Stabilizer performance based on linear shrinkage strains (Nelson and Miller 1992)

Linear Shrinkage Strain (%)	Description of severity	Rank
< 5.0	Non-critical	5
5.0-8.0	Marginal	4
8.0-12.0	Critical	3
12.0-15.0	Highly critical	2
> 15.0	Severe	1

Table 4.11 presents the ranking of the stabilizer performance based on the UCS values. The ranking is allotted based on the distribution of UC strength with sufficient factor of safety, assuming that the strengths achieved in the field will be approximately $1/3^{rd}$ to $1/5^{th}$ of the strengths achieved in laboratory.

Table 4.11 Stabilizer performance classification based on UCS values

Unconfined Compressive Strength (kPa)	Rank
< 2000	5
1200-1600	4
800-1200	3
400-800	2
> 400	1

Table 4.12 presents the comprehensive ranking of the stabilizers based on the values allotted for the respective test results.

Table 4.12 Ranking table and index values for different w/c ratios based on the above classification for site 1

		W/C = 0.8		W/C = 1		W/C = 1.3	
Soil Type		CR^1	CR^2	CR^1	CR^2	CR^1	CR^2
Control Soil		1	1	1	1	1	1
100 kg/m^3	100-0	4	3.5	3.6	3	3.6	3
	75-25	4	3.5	4	3.5	4	3.5
	25-75	5	5	4.3	4	4.3	4
	0-100	5	5	4.6	4.5	4.6	4.5
150 kg/m^3	100-0	4	3.5	4	3.5	4	3.5
	75-25	4	3.5	4	3.5	4	3.5
	25-75	4.6	4.5	4.6	4.5	4.6	4.5
	0-100	5	5	5	5	4.6	4.5
200 kg/m^3	100-0	4	3.5	4	3.5	4	3.5
	75-25	4.3	4	4.3	4	4.3	4
	25-75	5	5	5	5	5	5
	0-100	5	5	5	5	5	5

CR^1: Cumulative Ranking Based on Equal Weight Factor, 0.33

CR^2: 0.25 (FS) + 0.25 (LS) + 0.5 (q_u)

Based on the results reported in the table, it can be mentioned that pure cement at a dosage of 200 has achieved highest cumulative ranking in the present analysis. This was closely followed by combined lime_ cement treatment at the same dosage. The later treatment was considered for field applications due to potential effectiveness of this method for expansive clays and moderate economical advantages of lime over cement treatments.

4.5 Summary

Chapter 4 presents the analysis of the present study on two w/c ratios, 0.8 and 1.3 and comparison of those results with the previous work at 1.0 w/c ratio. Typical plots showing the effects of binder dosage, binder proportion and curing period at different w/c ratios have been developed along with the effects of research variables on small strain shear modulus. Typical plots of total and matric suction were also discussed. A detailed summary of test results with respect to different w/c ratios is explained and ranked towards the end of the chapter.

CHAPTER 5

CONCLUSIONS AND FUTURE RECOMMENDATIONS

5.1 Summary

Two test sections on the north bound IH820 located near northwest Fort Worth were proposed to be constructed over deep mixing columns, which are intended to reduce the swell and shrink properties with the moisture fluctuations and enhance the strength properties of the underlying expansive soils. Thus, in order to achieve a sophisticated design of the columns in all aspects with apt material parameters or quantities, extensive laboratory work has been performed. Lime and cement in different proportions were used as binders in this laboratory experimental work.

The effects of w/c ratios (0.8, 1.0 and 1.3) on swell, shrink and strength characteristics were discussed. Observations and conclusions were discussed separately for each test performed in the following paragraphs.

5.1.1 Linear Shrinkage Tests

(i) For the specimens treated at both the w/c ratios, the results from the linear shrinkage bar test indicated that the shrinkage strains of all treated soil specimens improved considerably when compared with the control soil.

(ii) It was also observed that, though there is an increase in water content due to increase in w/c ratios from 0.8 to 1.3, the shrinkage strains were still small

(iii) and can be characterized as low severity levels. The magnitudes of the shrinkage was almost negligible for the specimens treated with 0.8 w/c ratio and was around 0.5% for the specimens treated with 1.3 w/c ratio.

(iv) The samples prepared at liquid limit consistently exhibited higher shrinkage strains relative to those prepared at the molding water content.

(v) Though the shrinkage strains were found to be very low, a similar trend of increase in the shrinkage strain magnitude with increase in cement-lime dosage rate at a particular curing period was observed for all soil specimens treated with 0.8, 1.0 and 1.3 w/c ratios. This may be considered due to the cement hydration effects as well as higher heat of hydration at higher cement ratios.

(vi) The shrinkage strains were found to increase from 0.8 to 1.3 w/c ratio at all treatments and this variation in shrink magnitude may be due to the moisture fluctuations in the specimens.

5.1.2 Free Swell Tests

Swell was found to be completely eliminated for the soil specimens treated at 0.8, 1.0 and 1.3 w/c ratios relative to the untreated specimen. Swell decrease was attributed to particle bonding and aggregation developed between soil particles, and high compacted unit weights at the w/c ratios. High unconfined compression strengths of test specimens resulted in low swelling nature of the treated soil specimens.

5.1.3 Unconfined Compressive Strength Tests

(i) It was observed that for all the w/c ratios (0.8, 1.0, 1.3), the unconfined compressive strength values increased when compared to control soil property, and this increase is directly related to increase in binder dosages and cement content in the lime cement proportions.

(ii) The strength increase was considerable for both the curing periods. Moreover, the strength increase was found to be almost similar for all the specimens. Typical calculations for the samples treated at 25:75 (L:C) binder proportion and 200 kg/m^3 dosage rate for site 1, (a) show that the strength increase was 46% from 7 day curing to 14 day curing for 0.8 w/c ratio, 46% for 1 w/c ratio and 59% for 1.3 w/c ratio, (b) after 14 day curing show an increase of 4% strength at 0.8 w/c ratio and a decrease of 11% strength at 1.3 w/c ratio with w/c ratio equal to 1.0.

(iii) The UCS values were found to decrease with an increase in w/c ratio from 0.8 to 1.3 at any particular dosage rate and binder proportion. The bonding between soil particles and binders gets weaken and makes the material softer as the moisture content increases.

(iv) The failure strains were observed to be in between 1-2% for all the specimens and no particular trend of increasing or decreasing was observed in the failure strains with change in w/c ratios.

(v) For all the w/c ratios, the percentage increase in strength for (100:0) stabilizer combination has shown the least strength improvement irrespective of the curing period at all dosages than others at 75%, 25% and 0% lime treatments. Also, the increase in binder dosage at 100% lime treatment has produced a relatively flatter curve showing very less improvement.

(vi) The strength gain for specimens treated at 100% cement was observed to be four times more than the strength gain by specimens treated at 100% lime for any particular w/c ratio. This may be due to the faster pozzalonic reactions taking place when cement was treated with soil particles. Moreover, for all the w/c ratios, the unconfined compressive strength of 100 percent lime treated specimens at a dosage rate of 200 kg/m^3 is approximately half the value of unconfined compressive strength observed for 100 percent cement treated soil at a dosage rate of 100 kg/m^3. At any particular w/c ratio, the improvement ratios of the specimens treated with 100% cement followed by those treated at 75% were found to be 3 to 4 times more than the specimens treated at 25% or 0% cement proportions.

5.1.4 Bender Element Tests

(i) It was observed that in each w/c ratio (0.8, 1.0 and 1.3), the s-wave velocity increased with increasing binder dosages as a result of closer packing due to pozzalonic reactions and continuity in the soil-structure. Also, increase in cement proportion increased the shear wave velocity and hence the shear modulus of the soil.

(ii) The shear moduli values of soils treated at 0.8 w/c ratio were greater than the values at 1.0 and 1.3. This effect may be attributed to close packing of soil particles (cementation and aggregation) making sample stiffer at 0.8 w/c ratio than sample compacted at other two w/c ratios (1.0 and 1.3).

5.1.5 Suction Tests

(i) It was observed that with increase in w/c ratio, the suction values decreased. This is because, with increase in w/c ratio (via. increase in moisture content) the degree of saturation is also increased. As, degree of saturation is inversely proportional to suction, the suction values are decreased.

(ii) Total and matric suction values were calculated for the treated specimens and observed that for any particular w/c ratio, with increase in binder dosage, the suction values were found to be decreased. This may be due to the ion exchange, agglomeration, pozzalonic and flocculation reactions taken place between the binder material and soil particles, thus increasing void space.

5.2 Future Research Recommendations

- The strength characteristics for increased curing periods should be studied to understand the trend of strength increase.

- As no significant shrink/swell movements were observed at any of the w/c ratios (0.8, 1.0 and 1.3), more optimum or lesser dosage rates may be adopted for an economical design.
- The testing procedure may be more characterized avoiding bedding error, density control, sample preparation techniques and more data points on the stabilizer effects leading in developing correlations which can directly be implemented by knowing the strength and in situ conditions of the control soil.
- Suction tests at all (L:C) binder proportions should be discussed and compared with the raw data from control soils in order to analyze and develop correlations.

APPENDIX A

SPECIFICATIONS OF BINDERS

UNITED STATES LIME & MINERALS, Inc.

HYDRATED LIME SPECIFICATIONS

FREE-MOISTURE:	< 1.5 %
QUICKLIME	< 1.5 %
CaOH2	> 90 %
BULK DENSITY:	25 TO 29 lbs/cuft
% RETAINED 6 MESH:	0
% RETAINED 30 MESH:	3%
SPECIFIC GRAVITY	2.24

Note: Additional information on USLM, Hydrated lime may be accessed at

http://www.uslm.com/ProductServices/HydratedLime.htm

1341 West Mockingbird Lane · Dallas, Texas 75247 · 972.647.6700 · www.txi.com

MATERIAL SAFETY DATA SHEET

SECTION 1 - IDENTITY

Name
TXI OPERATIONS, LP

Emergency Telephone Number
(972) 647-6700

Person Responsible for Preparation
NANCY GARNETT

Common Name (used on label)
PORTLAND CEMENT

Chemical Name
DOES NOT APPLY

Trade Name & Synonyms
PORTLAND CEMENT, HYDRAULIC CEMENT TYPES I, II, I/II, III, CLASS A, CLASS C AND CLASS H.

Address
1341 MOCKINGBIRD LANE, DALLAS, TEXAS 75247

Date
NOVEMBER 1998

Chemical Family
DOES NOT APPLY

Formula
A HETEROGENEOUS MIXTURE OF BASIC CALCIUM SILICATES, ALUMINATES AND FERRITES TOGETHER WITH SMALL IMPURITIES OF MAGNESIA, CHROMIUM AND COMPLEX ALKALI METAL COMPOUNDS.

SECTION 2 - HAZARDOUS INGREDIENTS

Hazardous Component	CAS #	% Typical	TLV (Units)	PEL (Units)
PORTLAND CEMENT	65997-15-1	90 - 96	10 mg/m³	5 mg/m³*
GYPSUM	13397-24-5	2 - 5	10 mg/m³	5 mg/m³*
ANHYDRITE	14798-04-0	1.5 - 5	10 mg/m³	5 mg/m³*

* Respirable

PEL: Permissible Exposure Limit established by the Occupational Safety and Health Administration (OSHA).
TLV: Threshold Limit Value established by the American Conference of Governmental Industrial Hygienists (ACGIH).

SECTION 3 - PHYSICAL DATA

Boiling Point
DOES NOT APPLY

Percent Volatile by Volume
0%

Percent Soluble in Water
SLIGHT (0.1 - 1.0%)

Appearance and Odor
FINE, GRAY POWDER, NO ODOR

Specific Gravity (H_2O = 1)
3.05 - 3.20

Vapor Density (Air = 1)
DOES NOT APPLY

Reactivity in Water
WILL NOT EVOLVE FLAMMABLE OR TOXIC GASES

Vapor Pressure (mm = Hg)
DOES NOT APPLY

Evaporation Rate (n=Butyl Acetate)
DOES NOT APPLY

Hazardous Material Information System Identifier (HMIS)
HEALTH = 2   FLAMMABILITY = 0   REACTIVITY = 1   PERSONAL PROTECTION = X

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