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Deepa Rajendran and Robert L. Lytton

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The Texas A&M University System
College Station, Texas 77843-3135

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16. Abstract

There are three purposes for stabilizing the natural soil in place beneath pavements: (1) to provide a working table for construction equipment during wet weather; (2) to reduce the roughness due to an expansive clay subgrade; and (3) to provide a permanent layer in the pavement structure that is stiffer than the natural soil. No stabilized layer only 0.2 m (8 in) thick will reduce expansive clay roughness very much. This report shows that there are non-calcium stabilizers that will serve successfully as a working table (Purpose No. 1) but the study did not and could not determine the permanence of the treatment (Purpose No. 3). That determination must await the results of monitored pavement test sections.

Lime has been a commonly used and successfully applied stabilizing agent. However, lime reacts with sulfate-rich clay soils to create expansive minerals, which cause the pavement surface to buckle and to become undesirable rough. This report gives the details of a successful effort to find non-calcium stabilizers, which can replace lime in providing a construction-working table in sulfate-rich soils. It also gives the ranges of electrical conductivity below which lime stabilization will cause no lime-sulfate reaction problems and above which non-calcium stabilizers should be considered. The electrical conductivity test is a simple and inexpensive test that can be run in the field or in the lab to identify those lengths of roadway where non-calcium stabilizers should be considered. The details of the test equipment and procedure are provided in an appendix to the report.

The objective of this research was to find, if possible, non-calcium stabilizers which impart the following properties to the treated soil:

- Increase the strength and stiffness of the soil above that of the natural soil so as to form a working table for construction traffic,
- Reduce the swelling of the treated soil below that of the natural soil and eliminate the lime-sulfate interaction, if possible,
- Decrease the permeability of the treated soil below that of the natural soil so as to reduce the amount of water that will penetrate into the natural subgrade from the surface, and
- Decrease the suction of the treated soil below that of the natural soil so the treated layer will not attract water from below.

The results of the study point towards the use of two commercial stabilizers that can be used in lieu of lime in lime-sulfate heave areas. These stabilizers have been shown to be superior to lime in terms of strength, stiffness, permeability, and swell resistance potential.

17. Key Words

Lime-Sulfate Reaction, Non-Calcium Stabilizers, Swelling Resistance, Strength, Stiffness, Permeability, Expansive Clay, Pavement Roughness, Construction Working Table, Electrical Conductivity

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REDUCTION OF SULFATE SWELL IN EXPANSIVE CLAY SUBGRADES IN THE DALLAS DISTRICT

by

Deepa Rajendran
Research Assistant
Texas Transportation Institute

and

Robert L. Lytton
Research Engineer
Texas Transportation Institute

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TEXAS TRANSPORTATION INSTITUTE
The Texas A & M University System
College Station, Texas 77843-3135

IMPLEMENTATION STATEMENT

The practice of using lime as a stabilizer during construction of pavements is widespread and is considered economical. However, under certain conditions the beneficial effects of lime stabilization are overridden. Proximity of gypsum deposits in the soil formations indicates the presence of soluble sulfates. In such areas, when lime is added for stabilization during construction operations, instead of the cementitious reactions, a heaving phenomenon results. This swell induced by a lime-soluble sulfate reaction is attributed to the formation of expansive minerals and due to the disruption of the pozzolanic reaction. The researchers investigated the material properties of soils in two problem areas, SH 161 and IH 635 when stabilized with commercially available nonstandard stabilizers. From the tests conducted, two of the additives tested, namely, Roadbond EN1 and EMC Squared, were found to show superior qualities regarding strength, stiffness, permeability and other properties when compared to lime. Hence, it is recommended that these two additives be used at these two sites in place of lime, and at all other sites where soils that are high in soluble sulfates are encountered.

A principal reason for stabilizing a subgrade is to provide a working table for construction equipment during wet weather. No stabilizer, not even lime, should be used in the hope of reducing expansive clay roughness in pavements unless a very large depth (over 1.3 m (4 ft)) is used. While this study did not attempt to determine the permanence of stabilization with the non-calcium additives tested, there is still a need to establish the permanence beyond the construction period. The other physical properties that are imparted to the soil by these non-calcium additives assure that they will serve the construction purpose well.

The choice of which of the additives is best for a particular sulfate-rich subgrade soil must be determined by tests that were tried by and recommended in this study. The soils can be identified by electrical conductivity and a dielectric constant test and verified by a soluble sulfate test by an agricultural soils testing laboratory. Triaxial tests of the treated and untreated soil will determine which of the additives is best. Test sections using these additives in sulfate-rich subgrades will provide the ultimate demonstration of the value of these non-calcium stabilizers.

DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official view or policies of the Texas Department of Transportation (TxDOT) or Federal Highway Administration (FHA). This report does not constitute a standard, specification, or regulation, nor is it intended for construction, bidding, or permit purposes. The engineer in charge of this project was Robert L. Lytton.

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SUMMARY

Four sites in North Dallas, namely, SH 161, Section of Belt Line Road between Premier Road and Cabell Drive, SH 114 and IH 635 were experiencing the sulfate related heave phenomenon. Field trips were made to these four locations and tests were carried out to determine the electrical conductivity of the slurries made with those soils and their sulfate contents. The test results demonstrated the usefulness of the hand held conductivity meter and the test procedure to determine electrical conductivity in the field. The test results also suggested a detailed investigation into the problem sites at SH 161 and IH 635 should be conducted.

Based on the recommendations from previous projects, the researchers decided to investigate the feasibility of using alternate stabilizers in the two sites experiencing lime induced sulfate swell. A detailed laboratory test program was developed and three commercial stabilizers along with two other standard chemicals and lime were used for treating the soils from the two sites. The two commercial stabilizers, Roadbond EN1 and EMC Squared, showed promising results towards improving the strength and stiffness, and decreasing the expansion potential and permeability of the treated soils from SH 161 and IH 635, respectively. Other properties such as electrical conductivity, dielectric constant and suction showed marked improvement over those of the natural soil.

The final chapter of this report specifies the tests that can be used to identify the best stabilizer for the soil under study and also suggests the steps to be taken during the field implementation of the selected stabilizer.

CHAPTER I

INTRODUCTION

GENERAL

A pavement is underlaid by a subbase, which supports the surface and base course and provides load transfer to the subgrade. In addition, the subbase acts as a working platform for construction operations. In fine-grained (silty/clayey) highly plastic soils, the subbase requires stabilization among other reasons to provide a working table for construction equipment during the wet weather that may occur during construction. This is the original purpose of stabilizing the natural soil in place and lime has been a commonly used and successfully applied stabilizing agent. However, lime reacts with sulfate-rich clay soils to create expansive minerals which cause the pavement surface to buckle and are a source of undesirable roughness. This report gives the detain of a successful effort to find non-calcium stabilizers which can replace lime in providing a construction working table.

As a stabilizer used for clay soils, lime offers the following advantages:

- easily available and inexpensive,
- reduces the plasticity and shrink-swell characteristics of the soil,
- improves the bearing strength and permeability of the natural soil, and
- reduces the maximum dry density of the soil.

The chemistry of stabilization using lime can be summarized under the following four reactions (1).

1. Cationic Exchange
2. Flocculation/Agglomeration
3. Carbonation Reactions
4. Pozzolanic Reactions

The first two reactions increase the soil workability through changes in the electrical charges of the clay minerals (1). The second two are cementation reactions that produce the increase in bearing strength (1). These reactions occur at a high pH of 12.4 or above. The product of these reactions is a cementing compound called Calcium Silicate Hydrate (CSH) which accounts for the strength of the stabilized layer. Lime raises the pH of the soil to above 10 due to the ionization reaction (Ca++ ions are released) at which the clay minerals become unstable and deteriorate. Dissolution of clay provides the siliceous and alumnus pozzolans for reaction with calcium in the lime.

However, when lime is used in some sulfate bearing soils, the desired effect is not attained. In most of these situations, heaving of pavement surfaces was observed with subsequent failures of pavement sections. The reason for this adverse phenomenon is attributed to the disruption of the cementation reaction. Research in this area points to the reaction between alumna and Ca++ ions, in the presence of sulfates released by sulfate minerals and water, which leads to the formation of ettringite. The presence of excessive sulfates slows down the formation of CSH caused by a drop in pH and promotes the formation of ettringite (1). Ettringite is stable in both wet and dry conditions and can expand to a volume equal to 227% of the total volume of reactant solids (2). Ettringite can be transformed into thaumasite (another expandable mineral) when a sufficient amount of carbonate and dissolved silica are present in the soil system at temperatures between 4.5° C and 15° C (40° F and 59° F). Under favorable conditions these reactions can occur in as short a time as a few hours.

In the United States, the lime-induced sulfate heave related phenomenon has been reported from Southern California, Nevada, Kansas, Utah and Texas. The study conducted by Bredenkamp and Lytton (3) was successful in developing a field method to locate sulfate-bearing soils using a permittivity probe to measure the electrical conductivity of in situ soils. A model that relates the amount of expansion of clay soils on lime stabilization to the electrical conductivity was also proposed. This research (Study No: 3929-7) is a continuation of the above mentioned study (3) and is performed in cooperation with the Texas Department of Transportation under the research study "Highway Planning and Operation of Dallas District Phase III."

SCOPE AND OBJECTIVE

This study can be broadly classified into the following two objectives.

(1) To standardize the electrical conductivity test. This is done by measuring the electrical conductivity of the soils in probable sulfate rich soils. The soils are then tested for their sulfate content. A relatively high value of sulfate content corresponding to a relatively high value of electrical conductivity may be taken as confirmation of the results given by the conductivity test. In the ongoing research study, the above mentioned tests were carried out at four problem locations in the Dallas district. All of the test results confirm that a higher value of electrical conductivity points to a higher sulfate content.

(2) To investigate the availability and use of alternate (possibly non-calcium) stabilizers that can be applied to sulfate rich soils in place of lime, so that the swell problem associated with lime stabilization can be avoided. Using information from the literature, several non-calcium stabilizers were purchased and detailed tests were performed to identify the stabilizer that can best address the heave problem, at the same time imparting exceptional strength and permeability properties, especially during the period of construction.

Two locations in the Dallas District were identified having high sulfates. Soil samples were taken and an extensive test series was conducted to evaluate the capabilities of different stabilizers to satisfy the needs of a working table during construction. The testing yielded favorable results towards stabilizing the sulfate rich soils in the selected locations with non-calcium stabilizers. The work plan for this research is represented in figure 1.

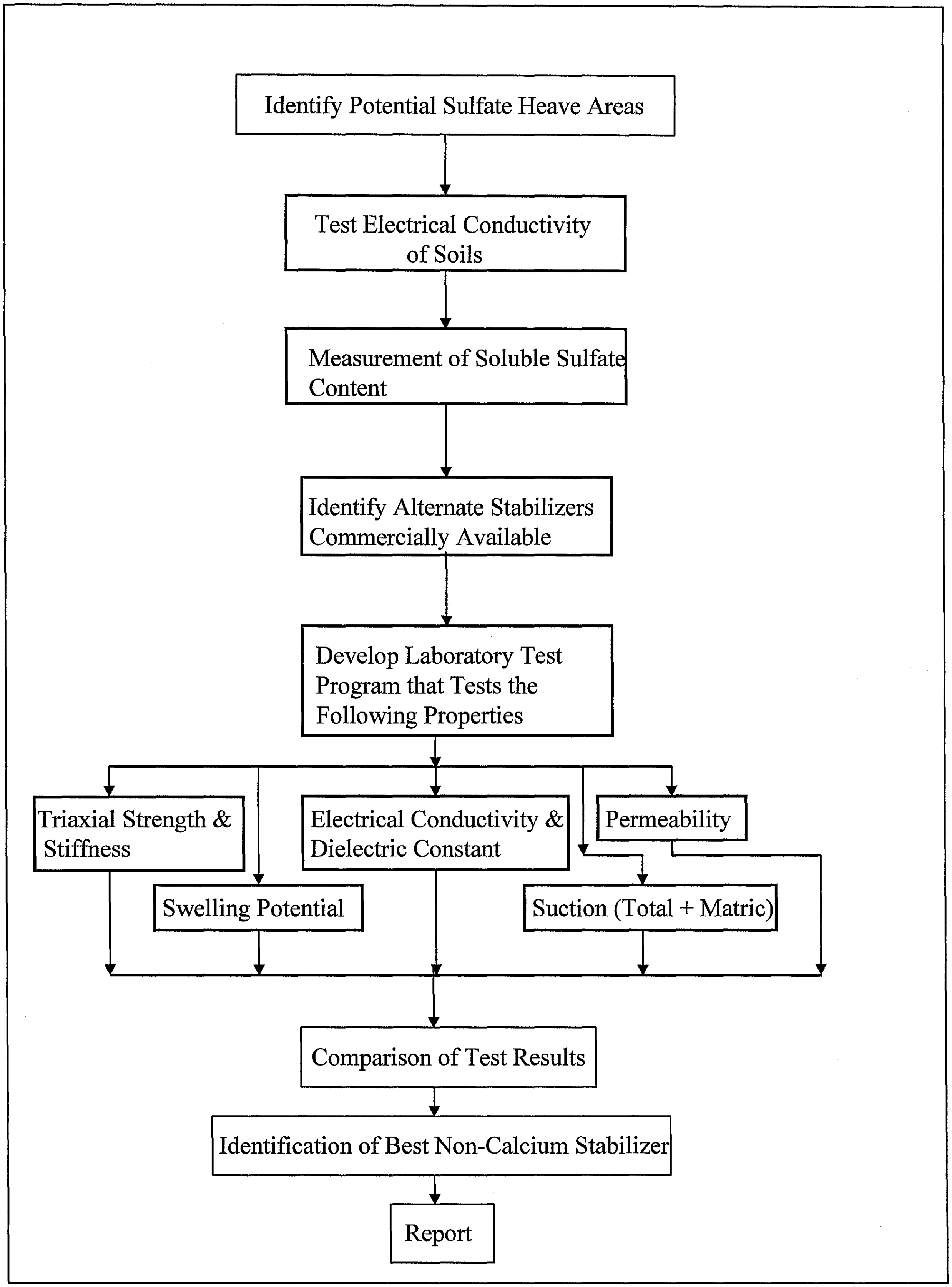


Figure 1. Work Plan for Study No: 3929-7

CHAPTER II

TECHNICAL BACKGROUND OF THE STUDY

INTRODUCTION

The research aims to establish two main objectives.

1. To standardize the conductivity test by further evaluation in probable sulfate bearing soils.
2. To investigate the availability and use of alternate stabilizers that can be used in problem sites.

Electrical conductivity of the soil is related to its sulfate content as suggested by Task 16 of Study No: 7-1994. A field test method for measuring the conductivity of the soil was developed in Study No: 7-1994 (3). A high value of conductivity corresponding to a high value of sulfate content can be taken as a confirmation of the test. The test method was tested in five different soils in Texas in the previous study and currently the test method was used in four soil types in Northern Dallas.

Mitchell concluded that when sulfates are present in the soil it could cause excessive heave in clay soils that have been stabilized with lime, due to the formation of ettringite (5). In Study No: 7-1994, it was shown that soils that contained relatively high sulfate contents expanded more upon lime stabilization than soils that do not contain sulfates.

The first objective, if accomplished, would enable future investigators to identify the sulfate rich soils where lime stabilization should be avoided at all costs. Instead alternate stabilizers that reduce the growth of ettringite and associated heaving can be tried through a comprehensive laboratory testing program. In the current study, a conductivity test was carried out on four different soils and based on the results, two soil types were identified as sulfate rich soils where lime stabilization had been carried out. The second objective is accomplished with a laboratory test program whose main purpose is to identify an alternate stabilizer that imparts the following properties upon mixing with the natural soil:

- increase the strength of the soil above that of the natural soil so as to form a working table for construction traffic,
- reduce the swelling of the treated soil below that of the natural soil and eliminate the lime sulfate interaction, if possible,
- decrease the permeability of the treated soil below that of the natural soil so as to reduce the amount of water that will penetrate into the treated soil and the natural subgrade from the surface during construction, and
- decrease the suction of the treated soil below that of the natural soil so that the treated layer will not attract water from below.

This chapter discusses the field testing methodology adopted for determining electrical conductivity of the soil, the percentage expansion expected after lime stabilization, soil types encountered during field investigation of the problem sites and commercially available alternate stabilizers that could be used in place of lime stabilization in the problem sites.

FIELD TEST FOR ELECTRICAL CONDUCTIVITY

Electrical Conductivity in Soils

Electrical conductivity is defined as the reciprocal of electrical resistivity and is expressed in mhos per centimeter (3) and in Siemens (S) and milliSiemens per centimeter (mS/cm). In Study No.: 7-1994 two major types of tests were performed.

1. Expansion tests were performed to determine expansion properties of soils that contain natural sulfates and the soils that contain added sulfates, upon hydrated lime stabilization and also upon stabilization with low calcium fly ash (3).
2. Electrical conductivity measurements were performed to investigate a possible relationship, between electrical conductivity in soils and sulfate content in soils (3).

The tests were performed on the following two groups of soils:

- Four naturally occurring clay soils found in Texas namely, Houston Black Clay, Beaumont Clay, Eddy Clay and a Kaolinitic Clay (3), and
- Soil Samples from various locations along IH 45 and FM 1382 near Dallas, where heaving has been encountered. These soils vary from sandy loam to heavy clays (3).

It was found that samples with the highest amount of expansion were those with the largest percentage of sulfates (3). Also, for samples that contain the same percentage of added sulfates, the greatest percentage of expansion was shown by samples that were stabilized with lime (3). All of these experiments verify Mitchell's findings on the heave phenomenon in California (4).

The electrical conductivity of the soils under investigation relates well to the sulfate content in the soils and can be used to determine whether or not the soil contains sulfates (3). Regression models that relate electrical conductivity to sulfate content were developed (3).

Field Test

A methodology was developed to determine the electrical conductivity of in situ soils using a test equipment kit that consists of the following (3):

- hand held conductivity meter,
- calibration solution for the conductivity meter,
- digital scale that can measure up to 500 g, and
- distilled water.

The conductivity meter measures the electrical conductivity of a soil-water paste (1:20) in mS/cm (3). The researchers proposed a model that can predict the amount of expansion, if the electrical conductivity and the sulfate content are known (3). The model gives an approximate amount of expansion due to sulfates in the soil as a function of electrical conductivity. Also of importance are equations that give the sulfate content of in itu soil based on % expansion and electrical conductivity. These equations are proposed based on models developed from tests done on five different soils found in Texas.

DISTRESS LOCATIONS

Many of the sulfate rich soil formations contribute to extensive heave related damages (5). Several sites in Texas, Corps of Engineers Projects, various TxDOT projects (5), certain streets in LasVegas, Nevada (4), are examples of locations that suffered extensive heave damages after the lime stabilization.

One such TxDOT project site was SH 161 whose entrance and exit ramps were constructed with asphalt concrete over a lime stabilized subgrade using 7% lime (5). Heaving was observed as early as six months after the construction (5). An investigation into the problem site revealed an increase in soluble sulfates at the site and suggested the possibility of gypsum laden water washing into the pavement (5). A double application of lime was suggested as a possible alternative (5). However six months after reconstruction, heaving was again observed at one location (5). (The current study investigated the problem sites at SH 161.)

Another recent TxDOT highway project site affected by sulfate-induced heave was SH 118 in Brewster County near Alpine, Texas (5). Experimental test sections were stabilized with lime and cement as two alternate stabilizers (5). Several months after construction, heaves in the form of transverse ridges appeared in both lime and cement stabilized sections. This occurred within a few days after rain subsequent to construction (5).

Mitchell reported failure of lime-stabilized pavement bases for a three mile stretch of a major arterial street in Las Vegas, Nevada. The earliest signs of distresses were noticed almost two years after construction. The distresses were in the form of surface heaving and cracking (4). Other areas of lime stabilization near the same locality, such as another street (Owens Street) and a school parking lot, also developed similar extensive damages and hence failed (4). Significant amounts of ettringite and thaumasite were indicated by X-Ray Diffraction in both failed and unfailed zones along the sections of the street (4). Proximity to water sources in these areas can worsen the swell problems as occurred in Stewart Avenue, where water probably gained access through the joints between the pavements (4).

As mentioned, the Corps of Engineers projects that suffered heave-related damages include several park roads near Joe Pool Lake area near Dallas and an auxiliary runway of Laughlin Airforce Base (5). The first one of these projects at Lloyd Park showed linear ridges in both transverse and longitudinal direction (5). The site at Cedar Hill Park showed only transverse heaving whereas the runway at Laughlin Air Force Base showed significant longitudinal heaving (5). In the locations of the park roads, in spite of the low amounts of swelling clay minerals and gypsum, the quantity of soluble sulfates were found to be high (5). It is suspected that fresh/added soluble sulfates might have entered the soil from the surrounding soil or a continued supply of fresh water might have extracted more sulfates (5). In certain cases where increased heaving was consistently observed in cuts of Eagle Ford Shale, it appears that poor drainage alone may not have been the reason (5).

Little and Petry concluded the following from a study of the heave phenomenon of the Joe Pool Lake Park area: “The formation of ettringite is necessary for this phenomenon to take place and control of this formation is a key to stabilization. Heave caused by this phenomenon cannot occur unless sufficient quantities of water are available. The presence of soluble sulfates in sufficient quantities in either the soil or water entering the soil is necessary for ettringite formation (6).”

In this context, it becomes imperative that alternate stabilizers be experimented at these sites, so that a sulfate swell problem does not occur. All of the above mentioned case histories warrant the investigation of the use of alternate/non-calcium stabilizers to quell the swell problem. Also of importance is the provision of proper drainage, to eliminate the entry of water into the stabilized section. The ideal stabilized section should offer minimum water affinity, low permeability and enough strength to serve as a working platform for construction traffic and to provide a permanent extra layer in the pavement structure.

ALTERNATE STABILIZERS

In recent years, a variety of nonstandard stabilizers has been manufactured and has been made available for use in street and highway stabilization. It is felt that these products are yet to be used in an effective way and many of their benefits and properties still remain undiscovered.

A study sponsored by the U.S. Department of Transportation and Federal Highway Administration as part of the Coordinated Federal Lands Highway Technology Implementation Program discusses some of the commercially available nonstandard stabilizers and their chemical properties. The study reviews applications and performances of several nonstandard stabilizers and by product pozzolanas used with subgrade soils, aggregate bases and aggregate surface sources in low volume roads and in highway construction (7). Because standard testing procedures were not available for testing these products at the time of testing, evaluations mentioned in the report of the study are based upon the performance of over 160 miles of test sections (7). Stabilizer types included in this report consist of pozzolans, bioenzymes, sulfonated oils, Ammonium Chloride, mineral pitches and clay fillers (7).

Stabilizers for clayey soils are chemical substances that can enter into the natural reactions taking place in the aggregate and control the moisture getting to the clay particles, thereby converting the clay fraction from a lubricant to a permanent cement that binds the aggregate mass together (7). In order to perform well as a stabilizer, a chemical must provide strong, soluble cations that can exchange with the weaker clay cations to force water from the clay lattice, resulting in a permanent structural change and significantly higher densities in the soil mass (7). In addition, the chemical stabilizer must be non-toxic, easily applied and readily available. The availability implies that it is commonly in use by industry (7). The clay stabilizers mentioned in this study, fall into the categories of enzymes, sulfonated naphthalene, sulfonated D-limonene and ammonium chloride. The sulfonated oils perform chemically as weak organic bases and when combined with the strong sulfuric acid, the reaction proceeds in the direction of forming a weaker species, resulting in a weaker acid which has high chemical stability and ionizing capability (7). This contributes to a significant increase in density and strength. The change is permanent and the clay mass becomes insensitive to variations in moisture (7).

Ammonium Chloride is non-toxic and environmentally safe. The reaction is different from that of sulfonated oils. Because of its size, it cannot easily penetrate the clay lattice, and most of its effect is at the edges of the clay particles where its powerful force removes ionized water and draws the lattice together (7). However the drying effect is permanent (7). The concentration of NH4+ ions present in clay is extremely small (7). With increased concentrations of the NH4+ ions, the soil capillarity is reduced (7). So this calls for considerable effort to achieve adequate mixing during the construction process (7).

An enzyme by definition is an organic catalyst which rapidly carries a chemical reaction to completion, without becoming part of the end product, the reaction being one that would normally take place at a much slower rate (7). Enzymes are non-toxic and environmentally harmless. The reactions that normally take place here are those that occur during the process of decomposition of clay in a natural environment (7). It is noteworthy that this process, which takes millions of years, is rushed through in a matter of hours and days during the process of stabilization of the same clay using an enzyme (7). The enzyme is regenerated by the reaction and hence goes on to perform again (7). Because the ions are very large, the osmotic migration of ions is very nominal and, hence, intimate mixing is required (7).

The commercial stabilizers studied are:

- Sulfonated Naphthalene - Condor SS,
- Sulfonated D-Limonene - Roadbond EN1,
- Enzymes/Bioezymes - BioCat, EMC Squared, Permazyme, PSCS-320, and
- Ammonium Chloride - Consolid 444.

Manufacturers of the above mentioned stabilizers were contacted through their addresses mentioned in the report. Among the manufacturers, only those of Roadbond EN1 and EMC Squared were readily available for contact and comment. Hence, those products were ordered. Manufacturers of Permazyme, PSCS-320 and Consolid 444 could not be located at all. BioCat, sister product of EMC Squared, was reported to be out of production line as per their manufacturer. Besides the two commercial stabilizers mentioned above, a third product by the name ClayPack was also included in the laboratory test program. The following table lists the commercial nonstandard stabilizers used in the test program.

Table 1. Details of Commercial Stabilizers Used

Name of Stabilizer	Category	Manufacturer
Roadbond EN1	Sulfonated D-Limonene	C.S.S. Technology, TX
EMC Squared	Enzyme	Soil Stabilization Products, CO, CA
ClayPack	Enzyme	Soil Bond International, TX

SOIL FORMATIONS IN THE PROBLEM SITES UNDER STUDY NO: 3929-7

On consultation with various TxDOT personnel, a list of four places was drawn up which was suspected of bearing lime-induced heaving and associated distresses. The four locations are SH 161, SH 114, IH 635 and Belt Line Road. A study of the soil survey map of Dallas County revealed information regarding the soil formations occurring in the above mentioned four locations.

The soil formations in SH 161, SH 114 and Belt Line Road fall broadly into the following categories: Heiden Clay, Feris Heiden Complex, Ferris Urban Land Complex, Ovan Clay, Normangee Clay, and Burleson Clay. The first four formations are noted to have gypsum crystal deposits. The soil formations occurring in IH 635 are Batsil Clay, Burleson Clay, Crockett Series, Ferris Heiden Complex, Heiden Clay, Houston Black Clay, Lewisville Urban Land Complex, Ovan Series, Normangee Series, Silawa fine loam, Silstid loamy sand, Trinity Clay and Wilson Clay Loam. Of these, Crockett Series, Ferris Heiden Complex, Heiden Clay, and Ovan Series are mentioned to have gypsum deposits. It is felt that the proximity of gypsum deposits could be one of the reasons of ettringite formations and subsequent heaving upon lime stabilization of the soil.

ELECTRICAL CONDUCTIVITY AND DIELECTRIC CONSTANT

Studies conducted to research electrical methods to classify the strength properties of aggregate base courses have established that both dielectric value and electrical conductivity reflect the strength and deformation properties of the base course aggregates (8). If magnetic permeability is neglected, earth materials can be characterized by their electrical conductivity and dielectric permittivity (ε) that is, in general, a complex and function of frequency (8).

The relative permittivity K (dielectric constant) is defined as

$$K = \varepsilon / \varepsilon_0 \tag{1}$$

where ε_0 is the dielectric permittivity and the dielectric constant can be expressed by

$$K^* = K' - K''' \tag{2}$$

where K' is the real part of the dielectric constant and K''' is the imaginary part or loss part of the dielectric constant (8). Imaginary part K''' can also be separated into high frequency components of loss in the form of

$$K^* = K' + i\,(K'' + \sigma_{dc} / \omega\, \varepsilon_0) \tag{3}$$

where σ_{dc} is the dc conductivity (S/m) and ω is the angular frequency, $2\pi f$. K'' is the frequency dependent loss associated with the relaxation response phenomenon (8). The dielectric value can vary between 1 (air) and 80 (free polar water at 20° C).

The water within an aggregate system exists as either free water or as adsorbed water, the properties of which depend upon the type and amount of clay minerals (9). The outer adsorption layer around a soil particle is loosely bound and is very sensitive to changes in pressure and temperature (9). Soil moisture suction is a measure of the energy state of moisture within the soil that directly relates the environment of the soil to its engineering properties (9).

The dielectric is a measure of the “free” or unbound water within the aggregate sample (9). It is an assessment of the state of bonding of water within the fine aggregates and thereby related to its material strength (9). The dielectric values of most (oven) dry soils vary between 4 and 6 (8). Hence the dielectric constant of soils gives information on both the volumetric water content of soil and also the amount of bound adsorption water (8).

The electrical conductivity in soil is electron and ion movement with free or limited dislocations (8). Most of the ionic or covalent rock-forming minerals are nonconductors and when these come into contact with liquid water, electrolytes are formed and ionic transmission is generated by an external field causing electrical conduction (8).

From studies conducted by Saarenketo, it has been proven that a correlation exists between the CBR value, and the dielectric. Also studies conducted by Scullion and Saarenketo show that the rate of increase in surface dielectric in a capillary-soaked cylindrical sample is an indication of the suction levels that exist within the base. It was also shown in these studies that the final dielectric values are solely related to how the moisture is distributed between adsorbed and unbound water (9).

Studies conducted by Saarenketo show that a CBR value stays stable until the electrical conductivity value reaches over 100 mS/cm, at which point the strength properties drop.

PERMEABILITY

One of the properties measured for in the treated/untreated soil samples is the permeability or hydraulic conductivity. It is a soil property, which describes how water flows through the soils. The flow of water through the pores or voids in a soil mass can, in most cases, be considered laminar (10). For laminar flow, the velocity is proportional to the hydraulic gradient or

$$v = k.i. \tag{4}$$

where k is the constant of proportionality, and is known as permeability.
Eq. (4) was combined with the continuity equation and the definition of hydraulic gradient and given as

$$q = vA = kiA = k\frac{\Delta h}{L}A \tag{5}$$

which is Darcy's law.

Here q is the total rate of flow through the cross-sectional area A and the proportionality constant k is called the coefficient of permeability. Permeability has units of velocity and is expressed as cm/sec or in/month.

Pavement bases/subgrades are classified as unsaturated soils and hence they do not have a constant value of permeability unlike saturated soils (11). Unsaturated soil permeability is very much dependent on the matric suction of the soil which is related to the stress state of the soil (11). However, the flow of water in an unsaturated soil is not fundamentally dependent upon matric suction and is controlled by hydraulic conductivity (11). But unsaturated conductivity is very complex to predict and various empirical equations are used for the same.

SUCTION OF SOILS

As mentioned in the previous section, in unsaturated soil the movement of gravitational water depends on the structural characteristics and porosity of soil but owing to the presence of air in the soil pores, Darcy's law with constant permeability can no longer be applied (12). The water held in the soil is retained in a state of tension or suction, which may be termed the soil moisture suction or soil suction (12). As the moisture content of the soil is reduced and the water interfaces recede into the smaller pores, their radii of curvature decrease, indicating an increase in soil suction (12).

The value of soil suction rises from zero at saturation to many thousands of pounds per square inch in oven dry soil (12). This large variation makes the use of a logarithmic scale necessary if the soil suction/moisture content relationship is being considered as a whole (12). In this connection, the pF scale, introduced by Schofield, is frequently used. If the soil suction is expressed in terms of the length of suspended column of water, the common logarithm of this length expressed in centimeters of water is equivalent to the pF values of the soil moisture (12).

Table 2 shows the relationship between the pF scale and soil suction expressed in lb/sq (12). It is to be noted that, owing to the logarithmic nature of the unit, pF = 0 does not correspond exactly to zero suction (12).

Table 2. Relationship Between pF and Soil Suction (12)

pF	Equivalent Suction	
	cm of water	lb/Sq in
0	1	0.0142
1	10	0.142
2	100	1.42
3	1,000	14.2
4	10,000	142
5	100,000	1,420
6	1,000,000	14,200

Total suction or soil suction is composed of two components, matric suction and osmotic suction. Matric Suction is the difference in pressure across the air - water interface and is related to the surface tension forces of water films (11). The osmotic suction is attributed to the chemical makeup of the soil water, such as the presence of dissolved salts in the soil water (11).

Total Suction is expressed by a thermodynamic equation and Osmotic Suction is expressed by Vant Hoff's Equation. The Matric Suction is the Total Suction less the Osmotic Suction (11). The Matric Suction component corresponds to the pressure head in normal hydraulic usage (13). It plays the same role as hydraulic head in saturated soils regarding movement of moisture (13). Total suction and both of its components, matric and osmotic, can be measured using the following methods.

Table 3. Suction Measuring Devices (11)

Name of Instrument/Setup	Component of Suction	Range (pF)
Tensiometer	Matric Suction	1.0-3.0
Thermocouple Psychrometer	Total Suction	3.0-4.5
Thermistor Psychrometer	Total & Matric	3.0-4.5
Transistor Psychrometer	Total Suction	3.0-4.5
Chilled Mirror Dew Point	Total Suction	3.0-4.5
Filter Paper Method	Total & Matric	2.0-5.5
Electrical Conductivity	Osmotic Suction	3.0-7.0
Pressure Plate	Matric Suction	>850 cm
Heat Dissipation Sensor	Matric Suction	Up to 3.0

CHAPTER III

DESCRIPTION OF WORK PLAN

To achieve the objectives of this project, a work plan was developed. This plan can be broadly grouped into two categories as follows.

- *Towards Accomplishing Objective I* - Identifying the chief problem sites in Dallas district where prior investigations have not yielded any successful results/where no prior investigations have been carried out.
- *Towards Accomplishing Objective II* - Investigate the availability and use of alternate stabilizers and their feasibility with respect to application in the problem sites identified.

The activities that were performed under Category I and II are listed below.

Category I

- Identifying the problem areas in the Dallas district,
- Identifying the soil types in the problem areas,
- Field trips to the problem sites,
- Testing the soil samples for electrical conductivity and soluble sulfate content, and
- Identifying the problem sites relevant to this study based on the test results.

Category II

- Purchase/procurement of alternate stabilizers,
- Setting up a laboratory test program for the selected sites,
- Determination of proportions of stabilizers to be used,

- Discussion of the test procedures to be followed and their implementation,
- Presentation of test results,
- Discussion and evaluation of test results, and
- Recommendations and implementation.

The tasks grouped under Category I are presented in Chapter IV. The first four bullets under Category II are presented in Chapter V. Chapter VI presents the fifth bullet on presentation of results. Chapter VII features the discussion and evaluation of test results. The final chapter Chapter VIII presents the recommendations and implementation programs.

CHAPTER IV

IDENTIFICATION OF MAJOR DISTRESS SITES

INTRODUCTION

The activities that were performed to accomplish Objective I of the study, namely "Identifying the chief problem sites in the Dallas district where prior investigations have not yielded any successful results/where no prior investigations have been carried out" are discussed in this chapter. These activities are in Category I as outlined in the previous chapter.

IDENTIFYING THE PROBLEM AREAS IN THE DALLAS DISTRICT

As a first step towards achieving this goal, a detailed review of relevant literature was made which dealt with problem sites in and around the Dallas district. This gave the researchers an idea about the potential problem areas, which could be included in the current study. With this information in mind, several TxDOT personnel (Dallas & Austin) were contacted over the phone and through the mail, to inquire about the problem sites that have been brought to their attention. A comprehensive list was compiled from these inquiries and it contained the following problem site locations:

1. SH 161 - Western Dallas County,
2. SH 114 - Bounded by Valley View Road and Belt Line Road,
3. Joe Pool Lake Area - Southwest to Dallas along FM 1382,
4. IH 45 - Ellis County,
5. North of Ferris - Ellis County,
6. IH 635 - Dallas County (Belt Line Road Overpass), and
7. Belt Line Road - Dallas County (South of IH 635), bounded by Cabell Drive and Premier Road.

Locations 3, 4 and 5 had been investigated previously. Location 1 had shown a lime-induced swell problem after construction (4). As a remedial measure, a double application of lime was applied to this site. This experiment was not a success because swells appeared two months after the double lime treatment. Location 7, i.e. the section of Belt Line Road bounded by Cabell Drive and Premier Road, experienced a lime-induced swell problem soon after construction. A double application of lime at this site seems to have reduced the swell problem. Hence it was felt that this site warranted further investigation. Location 2 and 6 had not been investigated and since these locations reported substantial swell problems, further investigation would be of benefit to the researchers. It was decided to investigate the four areas: SH 161, SH114, IH 635 and Belt Line Road (bounded by Cabell Drive and Premier Road). Figures 2, 3, 4 and 5 show the location of the problem sites. The boring locations of SH 161, Belt Line Road and bore hole #2 in SH 114 are shown in figure 2. Figure 3 shows locations of boring #3, 4 and 5 of SH 114. Figure 4 and 5 show the locations of borehole #5, and 1, 2 and 3 of IH 635.

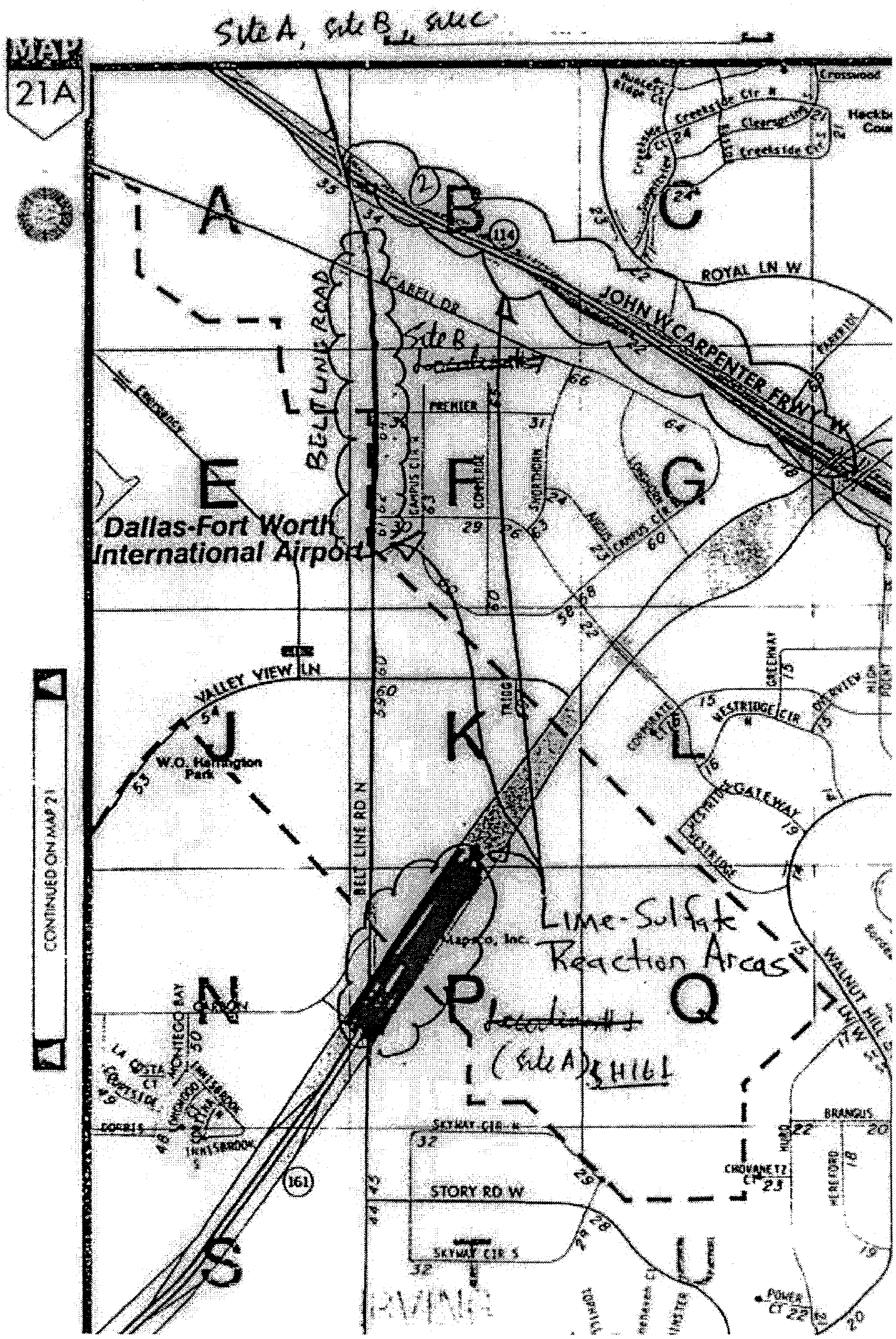


Figure 2. Location of Problem Sites at SH 161, SH 114 (Boring #2) and Belt Line Road

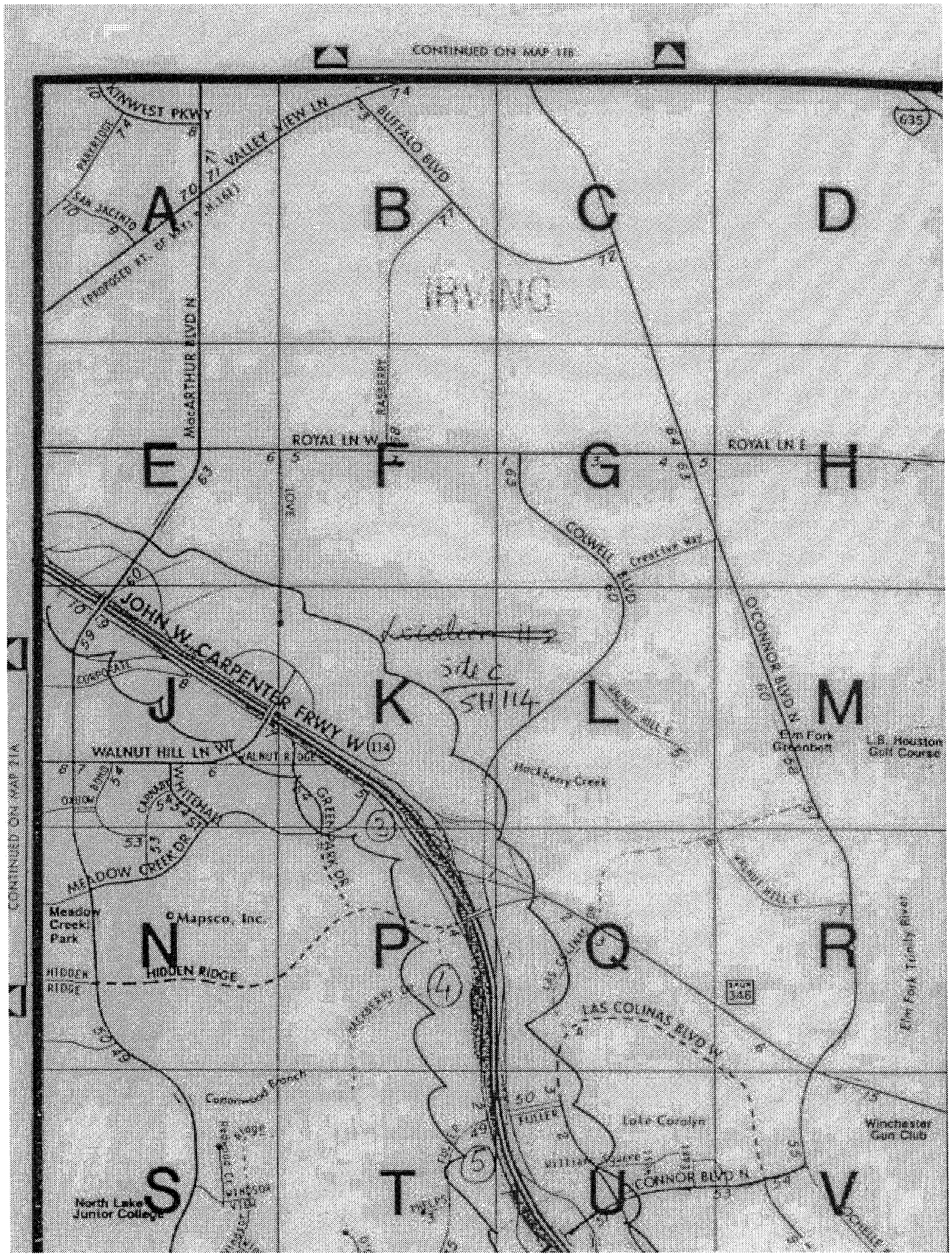


Figure 3. Location of Problem Sites at SH 114 (Boring #3, 4, and 5)

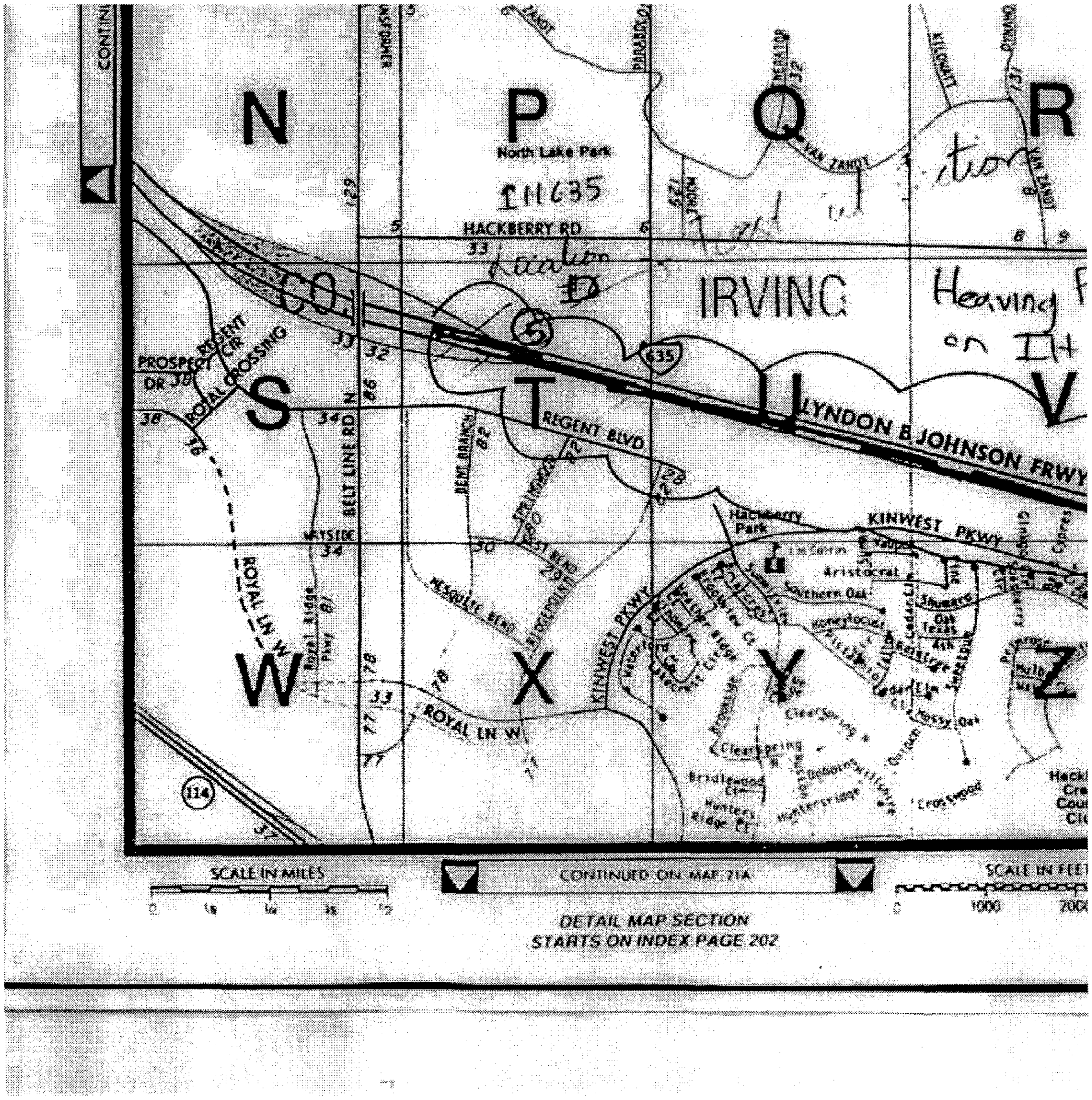


Figure 4. Location of Problem Sites at IH 635 (Boring #5)

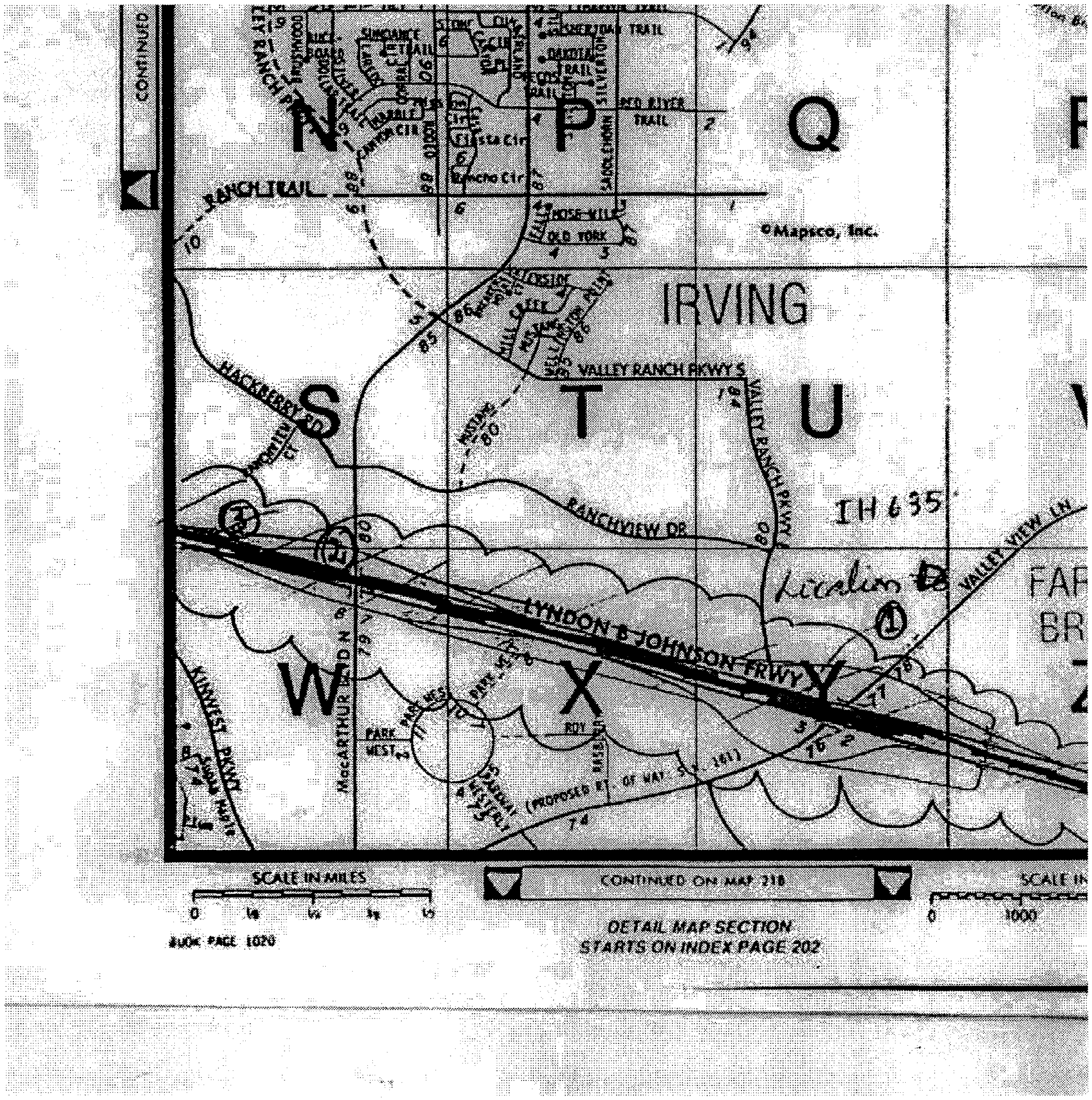


Figure 5. Location of Problem Sites at IH 635 (Boring #1, 2, and 3)

IDENTIFYING THE SOIL TYPES IN THE PROBLEM AREAS

Using soil survey maps of Dallas county, location maps and pavement cross section maps of the problem sites, the soil types occurring in these sites were studied. They fall into the following categories:

- Ferris Heiden
- Houston - Black Heiden
- Trinity Frio
- Silawa Silstid Bastsil

The following information is taken from Soil Survey of Dallas County (14). The Ferris clay consists of deep well-drained sloping clayey soils. The typical pedon of Ferris clay, in an area of the Ferris-Heiden complex, extends from the intersection of Belt Line Road and Pioneer Road and is located about 2.4 km (1.5 miles) south of Mesquite. The Ferris soil makes up about 60% of this complex, the Heiden soil makes up about 20% and minor soils make up the rest. Permeability is very low and the available water capacity and tendency towards erosion for this formation is very high. The soil formation is montmorillonitic and has high shrink-swell potential. The soil is calcareous throughout and some pedons show gypsum crystals.

The Houston Black Series is made up of deep, moderately well-drained clayey soils. The typical pedon of Houston Black clay is on slopes of 1% to 3%; it extends from the intersection of IH 635 and Town East Boulevard in Mesquite. Permeability is very slow and the available water capacity is very high. The soil is montmorillonitic and has high shrink-swell potential.

The Trinity Series is made up of deep poorly drained soils. The soil is moderately alkaline calcareous, montmorillonitic and has high shrink-swell potential. The Silawa-Silstid formation is made up of the Silawa series and Silstid series. Permeability is moderate and the available water capacity is low. The soil is siliceous. The soil formations occurring under SH 161, SH 114 and Belt Line Road are Heiden Clay, Ferris Heiden Complex, Ferris Urban

Land Complex, Ovan Clay, Normangee Clay, and Burleson Clay. The first four are noted to have gypsum deposits. The soil formations noted in IH 635 are Bastil Clay, Burleson Clay, Crockett Series, Ferris Heiden Complex, Heiden Clay, Houston Black Clay, Lewisville Urban Land Complex, Ovan Series, Normangee Series, Silawa fine loam, Silstid loamy sand, Trinity Clay and Wilson Clay Loam. Of these, the Crockett Series, Ferris Heiden Complex, Heiden Clay, and Ovan Series are mentioned to have gypsum deposits.

FIELD TRIPS TO THE PROBLEM SITES

Based on the information collected from the soil survey maps and the pavement cross section maps sent from TxDOT (Dallas district), it was possible to approximately locate the potential swell areas in these four sites. After a series of discussions about the data gathered from the locations, the researchers decided to conduct a field trip to the four sites. The following activities were conducted during the field trip.

- Soil samples were taken using as auger drill for running the conductivity tests and for determining the soluble sulfate determination.
- Photographs of the problem sites were taken with reference to the swells on the pavement.

The soil samples were obtained from the shoulder where no stabilization work was done, at a distance of 3 m (10 ft) off the pavement. The soil sampling was done to a depth of 0.6 m (2 ft) at every 0.3 m (1 ft) depth. Soon after the soil samples were procured, they were collected in plastic bags and sealed to preserve the moisture and chemical content. The bags were then labeled to identify the source and location. In addition, photographs of the problem areas were taken at all four locations for future reference and study. Figures 6-13 show the problem areas in the four locations.

Some observations were made during the field trip regarding the texture of the soil sampled and the size and extent of possible lime-induced swell in the four sites. The observations are listed below.

- The soil in SH 161, SH 114 and IH 635 was observed to be very sticky and wet, notably that of SH 161. The soil in the section of Belt Line road between Cabell Drive and Premier Road was dry and coarse.
- Of the four locations, SH 161 and IH 635 were the most affected.
- The entrance and exit ramps of SH 161 showed significant amounts of heaving.
- In some locations of IH 635, the heaves were so pronounced that the pavement was offering poor rideability. Here some of the heaves were accompanied by cracks.
- SH 114 showed some moderate heaving near the edges of the pavement.

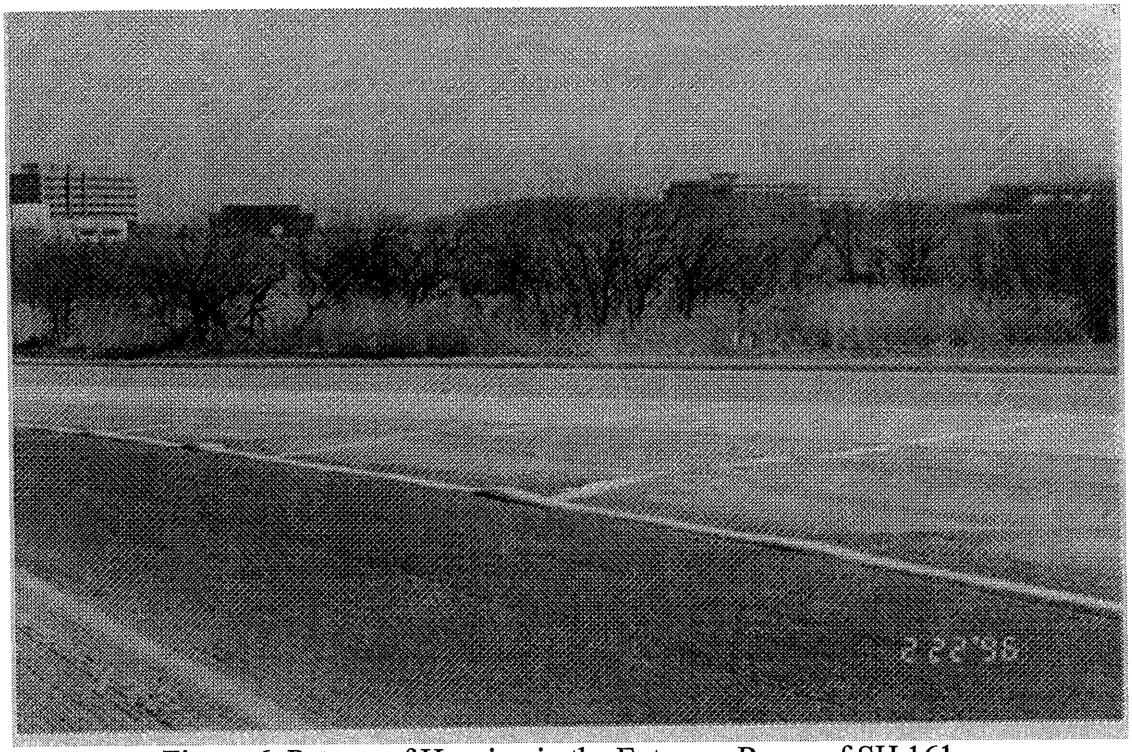


Figure 6. Pattern of Heaving in the Entrance Ramp of SH 161

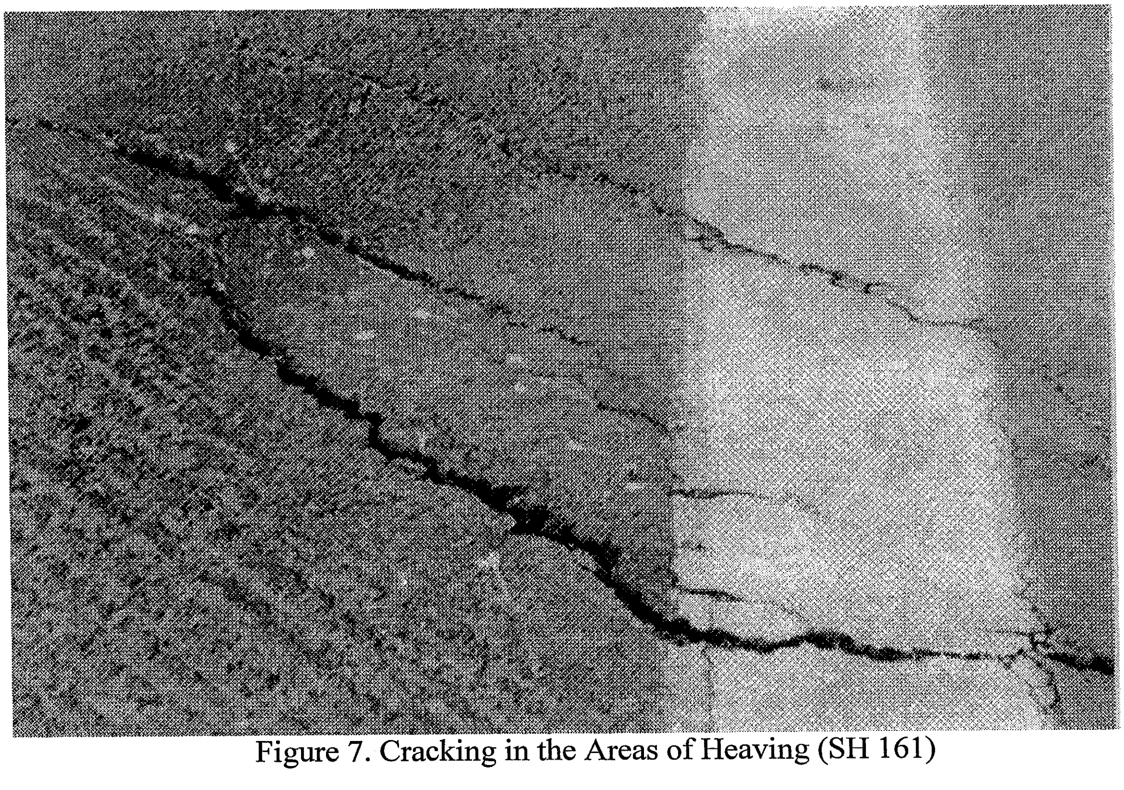


Figure 7. Cracking in the Areas of Heaving (SH 161)

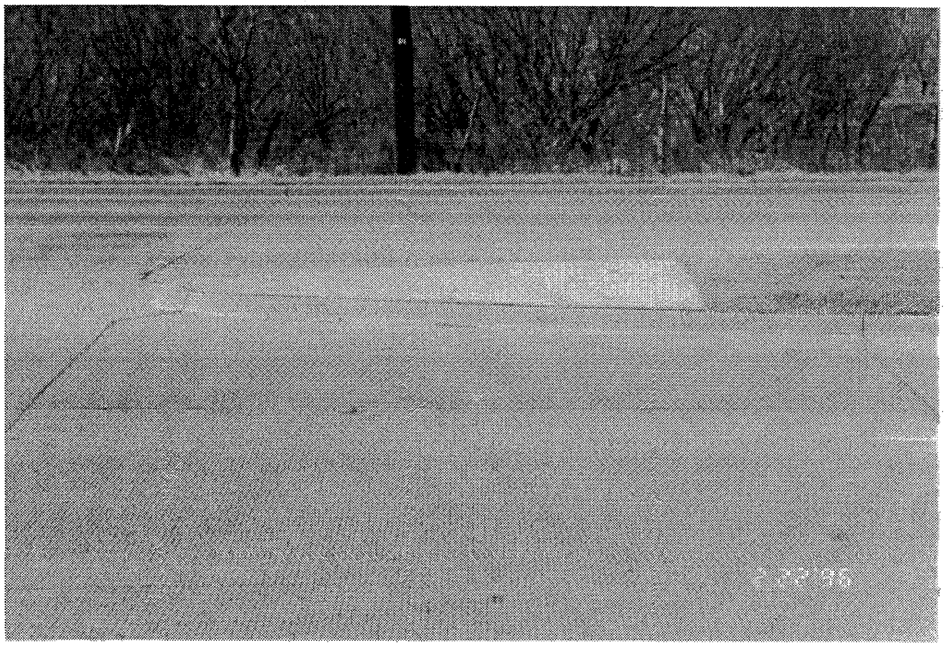


Figure 8. Reconstructed Section of Belt Line Road Between Premier Road and Cabell Drive

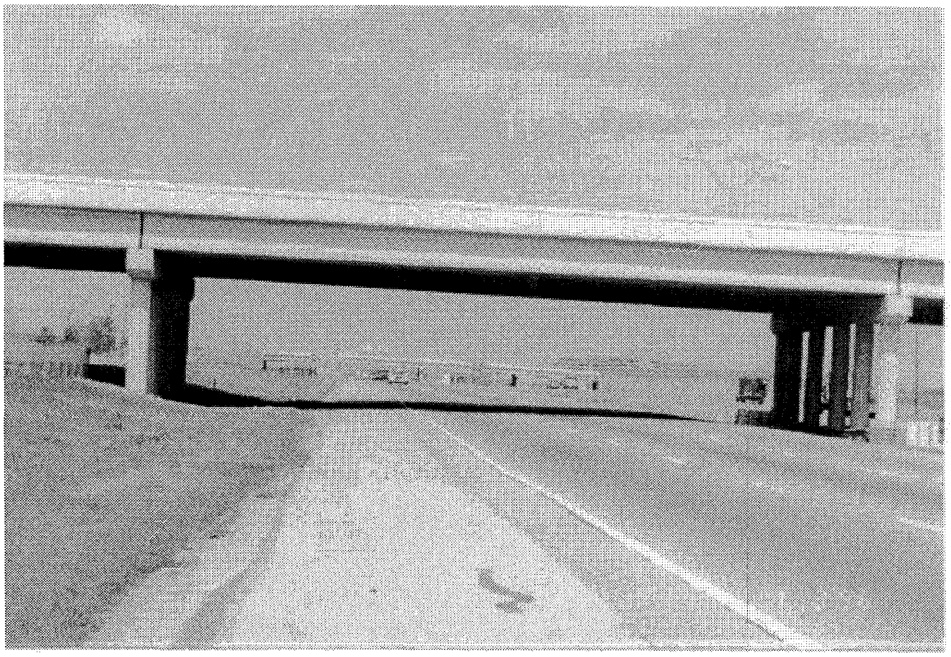


Figure 9. Sections of SH 114 Experiencing Moderate Heaving (Note the undulating pattern of white line near the shoulder)

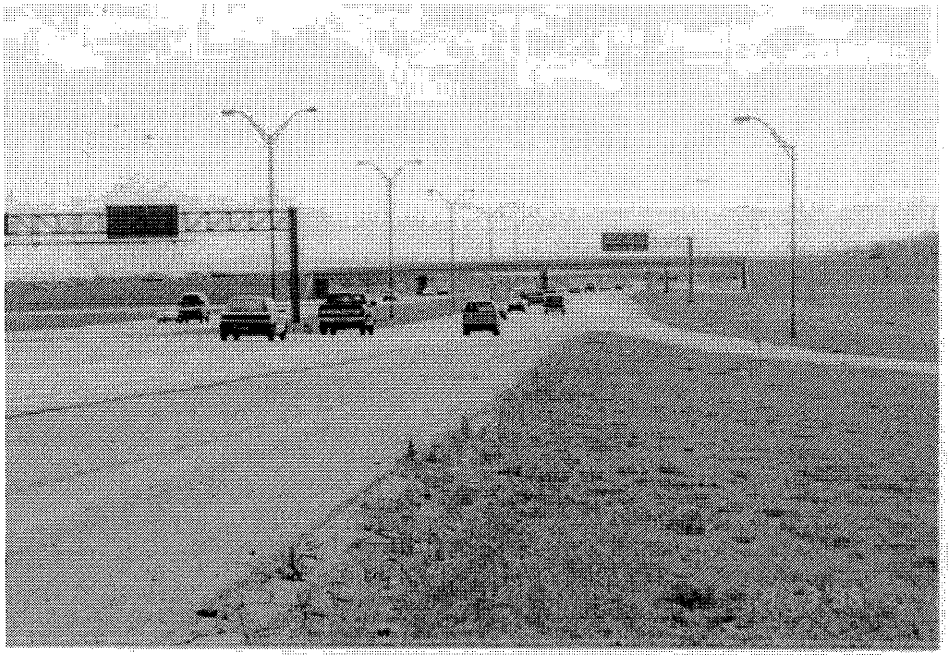


Figure 10. Heaved Up Sections of SH 114

Figure 11. Sections of IH 635 Experiencing Heaving

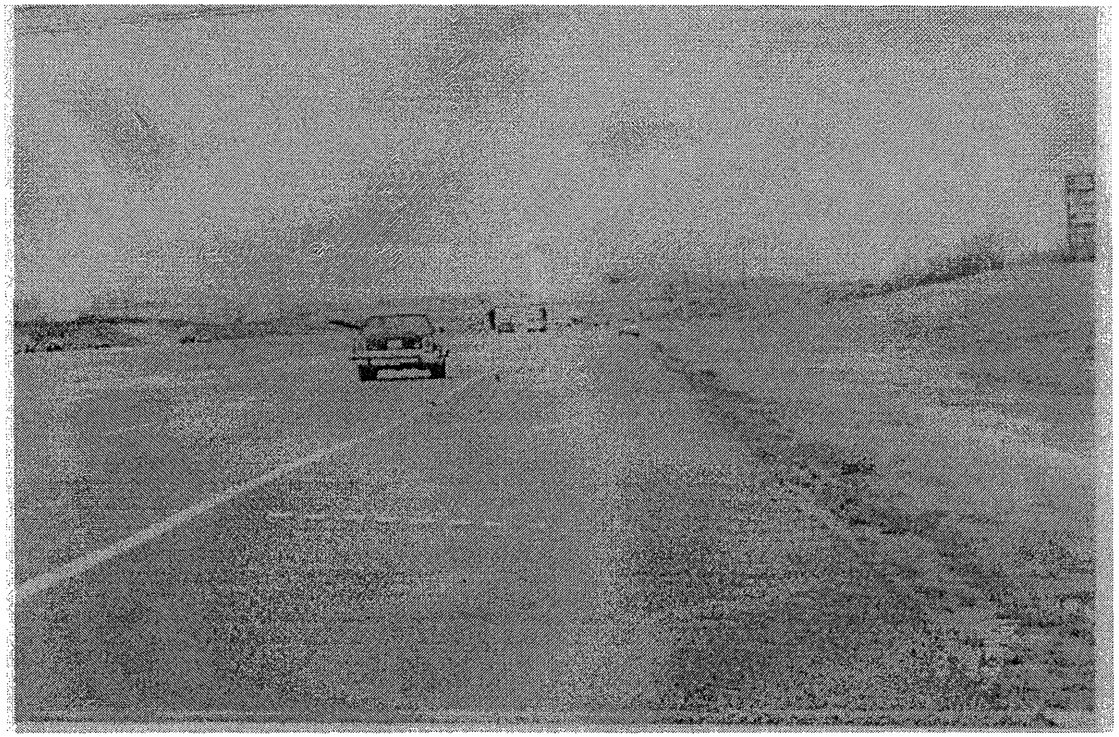


Figure 12. Sections of Moderate to Severe Heaving (IH 635)

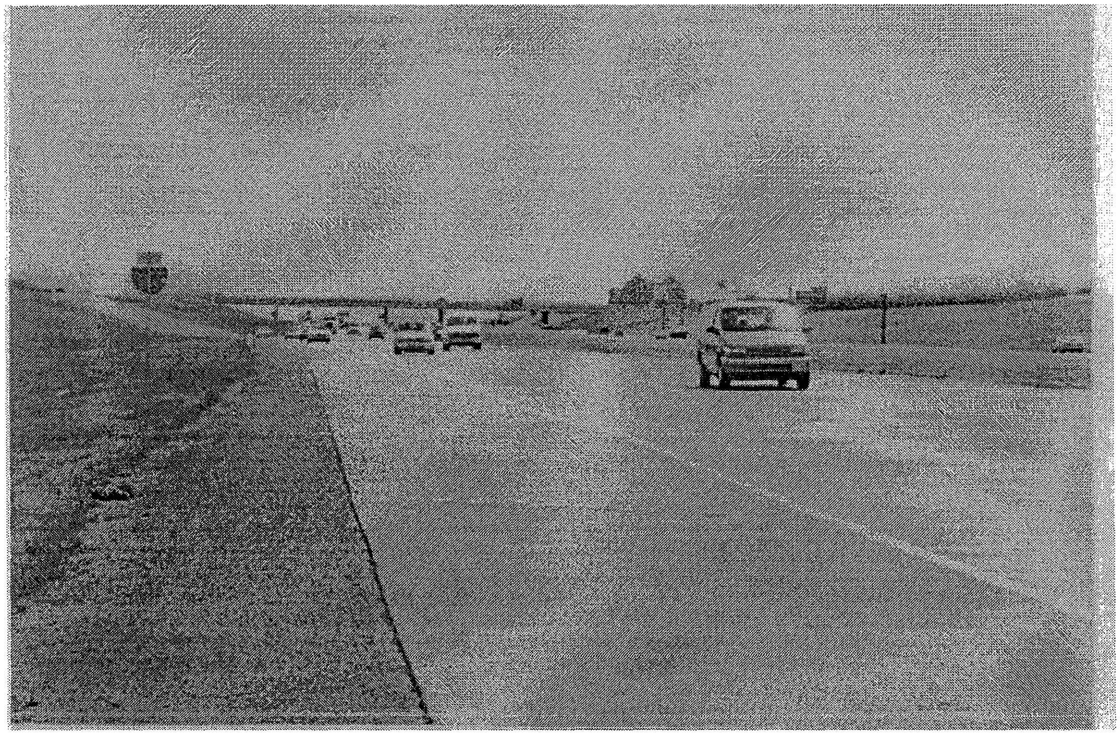


Figure 13. Sections of IH 635 Experiencing Heaving

- Belt Line Road under study showed no signs of lime-induced heave. According to the area engineer, this section showed heaving soon after construction, but upon reconstructing the section with 12% lime, the problem did not occur again.
- While auger drilling, certain white crystal deposits were found in the soils sampled. This was observed in SH 161 and IH 635. (Large layers of gypsum deposits were again observed at these sites when soil was excavated for conducting detailed tests at later stage.)
- The soil at both SH 114 and IH 635 was wet and sticky, though according to the TxDOT personnel there were no rains for the past four to five months. There appears to be a serious water retention problem.

TESTING THE SOIL SAMPLES FOR ELECTRICAL CONDUCTIVITY AND SOLUBLE SULFATE CONTENT

The soil samples were brought to the lab for determining the electrical conductivity. The testing methodology described in the report of Study No: 7-1994 was followed. It can be summarized as follows.

1. Approximately 5 g of soil is taken and airdried for about 24 hrs. The dried soil is then crushed in a crucible using a pestle until it becomes fine.
2. Distilled water in the proportion of 20:1 (20 parts of water to 1 part of soil), i.e., 100 g is taken.
3. The weighed measures of soil and water are taken in a container (that is provided with a lid), closed, and is shaken until the soil particles appear mixed well in the water.

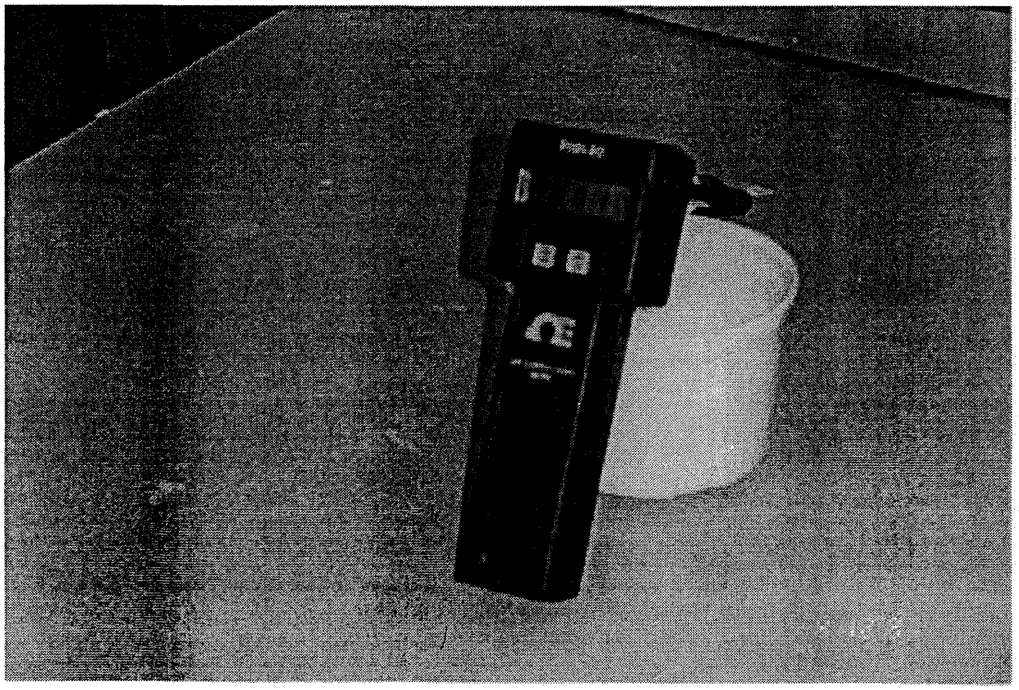


Figure 14. Hand Held Pocket Conductivity Meter

4. The hand held electrical conductivity meter is calibrated with standard solutions whose conductivity approximately matches the expected values of soil under study (figure 14).
5. The conductivity probe is then uncapped and the meter is immersed into the soil/water solution with the liquid level up to half the length of conductivity probe.
6. The meter gives the electrical conductivity value of the soil in μS.

The soil samples from the four locations were tested in this way for electrical conductivity determination. There were 16 samples (8-top 0.3 m (1 ft), 8-bottom 0.3 m (1 ft)) from SH 161, 18 samples (9-top 0.3 m (1 ft), 9-bottom 0.3 m (1 ft)) from Belt Line Road, and 10 (5-top 0.3 m (1 ft), 5-bottom 0.3 m (1 ft)) each from SH 114 and IH 635. The samples are labeled as shown in tables 4 and 5.

Table 4. Samples from SH 161

Samples from SH 161 - Top 0.3 m (1 ft)								
A-1-t	A-2-t	A-3-t	A-4-t	A-5-t	A-6-t	A-7-t	A-8-t	A-9-t
Samples from SH 161 - Bottom 0.3 m (1 ft)								
A-1-b	A-2-b	A-3-b	A-4-b	A-5-b	A-6-b	A-7-b	A-8-b	A-9-b

Table 5. Samples from Belt Line Road

Samples from Belt Line Road - Top 0.3 m (1 ft)								
B-1-t	B-2-t	B-3-t	B-4-t	B-5-t	B-6-t	B-7-t	B-8-t	B-9-t
Samples from Belt Line Road - Bottom 0.3 m (1 ft)								
B-1-b	B-2-b	B-3-b	B-4-b	B-5-b	B-6-b	B-7-b	B-8-b	B-9-b

Table 6. Samples from SH 114

Samples from SH 114 - Top 0.3 m (1 ft)				
C-1-t	C-2-t	C-3-t	C-4-t	C-5-t
Samples from SH 114 - Bottom 0.3 m (1 ft)				
C-1-b	C-2-b	C-3-b	C-4-b	C-5-b

Table 7. Samples from IH 635

Samples from IH 635 - Top 0.3 m (1 ft)				
D-1-t	D-2-t	D-3-t	D-4-t	D-5-t
Samples from IH 635 - Bottom 0.3 m (1 ft)				
D-1-b	D-2-b	D-3-b	D-4-b	D-5-b

In the tables, A -> SH 161, B -> Belt Line Road, C -> SH 114 and D -> IH 635. For example, B-1-t, '1' indicates the borehole #1, and 't' indicates the top 0.3 m (1 ft) of soil. Similarly, 'b' indicates the bottom 0.3 m (1 ft).

Study No: 7-1994 also reports a regression equation, which calculates the percent expansion that can be anticipated 7 days after lime stabilization and moist cure. This equation was used for calculating the percent expansion in this case and is based on the electrical conductivity values. The equation is given as

$$\%\ \text{Expansion (6\% Lime)} = 0.143\ \text{Ln}\ (\text{EC in}\ {}^{mS}/_{cm}) + 0.448 \times (\text{EC in}\ {}^{mS}/_{cm}) - 0.784 \qquad (6)$$

Based on the results of electrical conductivity, certain soil samples that showed relatively high values were selected for soluble sulfate content determination. The selected soil samples were then sent to Soil Analytical Services Inc., for determining soluble sulfate content. The method used for the above process is EPA 375.4 -1. It is summarized below.

Sulfate ion is converted to a barium sulfate suspension under controlled conditions. The resulting turbidity is determined by a nephelometer or spectrophotometer and compared to a curve prepared from standard sulfate solutions (EPA Method 375.4 -1).

Tables 8, 9, 10 and 11 show the electrical conductivity values and corresponding percent expansion for all the soil samples from the four locations. Percent expansion for certain values of electrical conductivity gave negative values when substituted in the equation. They are shown as zero to indicate that there will be no expansion. Table 12 shows the selected samples for soluble sulfate determination and corresponding electrical conductivity in mS.

Table 8. Values of Electrical Conductivity and % Expansion (SH 161)

Samples From SH 161					
Top 0.3 m (1 ft)			Bottom 0.3 m (1 ft)		
Borehole #	EC in $^{mS}/_{cm}$	% Exp	Borehole #	EC in $^{mS}/_{cm}$	% Exp
A-1-t	1.95	0.185	A-1-b	2.52	0.477
A-2-t	1.81	0.112	A-2-b	2.25	0.340
A-3-t	2.21	0.319	A-3-b	2.28	0.355
A-4-t	2.33	0.381	A-4-b	2.28	0.355
A-5-t	2.28	0.355	A-5-b	2.39	0.411
A-6-t	2.43	0.432	A-6-b	2.39	0.411
A-7-t	1.8	0.106	A-7-b	0.74	0
A-8-t	1.4	0	A-8-b		
A-9-t	2.26	0.345	A-9-b	2.27	0.350

Table 9. Values of Electrical Conductivity and % Expansion (Belt Line Road)

Samples from Belt Line Road					
Top 0.3 m (1 ft)			Bottom 0.3 m (1 ft)		
Borehole #	EC in $^{\mathrm{mS}}/_{\mathrm{cm}}$	% Exp	Borehole #	EC in $^{\mathrm{mS}}/_{\mathrm{cm}}$	% Exp
B-1-t	1.54	0	B-1-b	2.26	0.345
B-2-t	0.29	0	B-2-b	1.68	0.043
B-3-t	0.95	0	B-3-b	0.82	0
B-4-t	0.45	0	B-4-b	0.76	0
B-5-t	1.82	0.117	B-5-b	1.88	0.149
B-6-t	1.86	0.138	B-6-b	1.54	0.
B-7-t	0.85	0	B-7-b	1.44	0
B-8-t	0.69	0	B-8-b	1.17	0
B-9-t	0.36	0	B-9-b	0.96	0

Table 10. Values of Electrical Conductivity and % Expansion (SH 114)

Samples from SH 114					
Top 0.3 m (1 ft)			Bottom 0.3 m (1 ft)		
Borehole #	EC in $^{\mathrm{mS}}/_{\mathrm{cm}}$	% Exp	Borehole #	EC in $^{\mathrm{mS}}/_{\mathrm{cm}}$	% Exp
C-1-t	0.210	0	C-1-b	0.220	0
C-2-t	0.210	0	C-2-b	0.200	0
C-3-t	0.270	0	C-3-b	0.230	0
C-4-t	0.230	0	C-4-b	0.430	0
C-5-t	0.240	0	C-5-b	0.240	0

Table 11. Values of Electrical Conductivity and % Expansion (IH 635)

Samples from IH 635					
Top 0.3 m (1 ft)			Bottom 0.3 m (1 ft)		
Borehole #	EC in $^{\mathrm{mS}}/_{\mathrm{cm}}$	% Exp	Borehole #	EC in $^{\mathrm{mS}}/_{\mathrm{cm}}$	% Exp
D-1-t	0.480	0	D-1-b	1.850	0.133
D-2-t	0.900	0	D-2-b	1.740	0.075
D-3-t	1.650	0.027	D-3-b	0.490	0
D-4-t	0.180	0	D-4-b	0.300	0
D-5-t	0.260	0	D-5-b	0.230	0

Table 12. Values of Electrical Conductivity and Sulfate Content

Sample Name	Electrical Conductivity in $^{mS}/_{cm}$	Sulfate Content in ppm
A-1-b	2.52	5337
A-3-b	2.28	3826
B-1-b	2.26	1968
B-5-b	1.88	1663
B-5-t	1.82	1615
C-4-t	0.23	558
C-4-b	0.43	1602
D-2-b	1.74	2641
D-2-t	0.90	2631
D-3-t	1.65	1766

The results as shown in the above tables clearly indicate that higher electrical conductivity values result in higher sulfate contents. This simple lab test of determining the electrical conductivity of in situ soil can be used in future projects to identify problem areas. As seen from the results, the conductivity test gives results that can be confirmed with other proprietary tests. Also test results from soluble sulfate contents can be used to relate between the electrical conductivity values.

IDENTIFYING THE PROBLEM SITES, RELEVANT TO THIS STUDY, BASED ON THE TEST RESULTS

From the test results shown in tables 8-12, it can be seen that the highest values of electrical conductivity are shown by samples from SH 161 and IH 635 and a few at the Belt Line Road site. The values of corresponding soluble sulfate contents are also high. It was decided that a detailed investigation should be carried out at these two locations since it is evident these locations are experiencing lime-sulfate reaction induced heave distresses.

Most of the samples taken at the Belt Line Road and all of the samples from SH 114 show relatively low values of electrical conductivity and soluble sulfate content. The heaving experienced in SH 114 is not due to the lime-sulfates reaction but is probably due to drainage problems which are the other major cause of expansive clay roughness.

Soil was collected from the shoulders of SH 161 and IH 635 for testing with stabilizers. Trenches of 0.6 m (2 ft) depth were dug and soil was excavated and transported to the lab in barrels. It may be noteworthy that during this excavation, several layers of crystal-like minerals were encountered, presumably gypsum, which attained a white color upon losing moisture.

CHAPTER V

DISCUSSION OF LABORATORY TESTING PROGRAM

INTRODUCTION

The activities that were pursued to accomplish Objective II namely, "To investigate the availability and use of alternate stabilizers and their feasibility with respect to application in the problem sites identified" are discussed in this chapter. The first four activities outlined under Category II in Chapter III are presented in this chapter

PURCHASE/PROCUREMENT OF ALTERNATE STABILIZERS

A comprehensive list of commercial stabilizer manufacturers was drawn up after reviewing the relevant literature (4,10). Only four manufacturers were readily available for any comment or contact. Their products are Roadbond EN1, EMC Squared, ClayPack, and Soil Sta I.

Thus, it was decided to pursue a testing program including the four commercial stabilizers. The fourth product, Soil Sta I, was later removed from the testing program, although the product was purchased, due to lack of handling and user guidelines from the manufacturer.

Besides the commercial stabilizers, other standard chemical stabilizers proposed to be used were potassium hydroxide, ammonium chloride and lime. Potassium hydroxide has been reported to be imparting properties similar to those of lime treated soil. The commercial stabilizer Consolid 444 (manufacturer could not be traced with their given address/phone #) has ammonium chloride as its key ingredient along with propylene combined in an inverted emulsion (10). Lime was used to compare the test results obtained from lime stabilized soil samples and remaining soil samples. In addition, the researchers decided to use raw samples for comparison purposes.

A total of six different soil stabilizers were used besides the raw soil. They are listed below.

1. Roadbond EN1
2. EMC Squared
3. ClayPack
4. Potassium Hydroxide
5. Ammonium Chloride
6. Lime

SETTING UP A LABORATORY TEST PROGRAM FOR THE SELECTED SITES

The soil brought from the locations at SH 161 and IH 635 was emptied into trays and placed in ovens so that they dried completely. This is to ensure the addition of the correct amount of moisture during compaction to prepare standard soil samples. The dried soil was then run through a "chipmunk" (crusher) so that the large clay clods would break down into particles small enough to pass the #40 sieve.

Table 13 shows the soil stabilizer matrix that was prepared which lists the tests to be carried out and the stabilizers to be used. This table applies to both SH 161 and IH 635 soil samples.

Table 13. Soil Stabilizer Matrix

Description of Tests	Description of Stabilizers to be Used						
	EMC^2	**EN1**	**ClayPack**	**KOH**	**NH_4Cl**	**Lime**	**Raw**
Triaxial Strength & Stiffness Test							
Swell Test							
Electrical Conductivity & Dielectric Constant							
Permeability Test							
Suction (Total + Matric)							

Of the above mentioned properties, electrical conductivity and dielectric constant are measured using the soil samples (treated/untreated) that are subjected to swell tests before and after the swell tests. Also measured are both total and matric suction of soil samples subjected to permeability tests before and after the permeability tests. Suction tests are carried out on the leacheate from permeability tests.

DETERMINATION OF THE PROPORTIONS OF STABILIZERS TO BE USED

The commercial stabilizers, namely Roadbond EN1, EMC Squared, ClayPack and lime were purchased from the respective manufacturers. Except for lime, all three commercial stabilizers were in a concentrated liquid form. Potassium hydroxide and ammonium chloride were purchased from Fisher Scientific. These two chemicals were in the form of solutions, the strength of Potassium Hydroxide being 50% wt/vol and that of ammonium chloride being 85 g/ℓ.

For Roadbond EN1 and EMC Squared, the manufacturer's recommendation of proportions of stabilizer to water by weight to be used varied over a range of 1:220 - 1:300 - 1:500. Soil samples for triaxial tests and swell tests were prepared at these three proportions for the above two stabilizers.

In the case of potassium hydroxide, the proportion of 50g/3ℓ of water and 150g/3ℓ of water were roughly worked out using the criteria of pH. The corresponding values of pH are 10 and 12 respectively. The proportion of ammonium chloride used, i.e., 25.5g/3ℓ is the maximum strength at which it can be prepared using the commercially available grade (85 g/ℓ is the maximum strength available commercially). For ClayPack, the manufacturer's recommendation of proportion of stabilizer to water by weight of 1:300 was used. For lime treated samples, three proportions, namely 12%, 6% and 3% by weight of soil, were adopted.

DISCUSSION OF THE TEST PROCEDURES TO BE FOLLOWED

Determination of Optimum Moisture Content for Molding of Soil Samples

The samples to be subjected to triaxial tests, swell tests and permeability tests are molded at their optimum moisture water content at which the samples will have the maximum dry density. A watercontent (w/c) vs. dry density curve is used to determine the optimum moisture content and Standard Proctor Compaction is used for molding soil samples. The procedure is described below.

About 4500 g of dried soil is taken. An approximate percentage of water by weight of soil is added to the soil and mixed thoroughly. The wetted soil is then compacted in layers in a standard mold of 4" diameter and 8" height using 25 blows of a 10 lb heavy hammer. A portion of the wetted soil is then kept in the oven for water content determination. The molded soil sample is then weighed along with the mold. The weight of the mold is then determined after extruding the sample. From the weight of the mold, weight of the mold + compacted soil specimen and percentage by weight of water added, the dry density of the specimen can be determined.

The next few points to plot the curve are obtained as follows. Depending on how wet the soil was in the previous case, the percentage of water to be added in subsequent cases can be made either greater or less than the previous one. About four or five points are obtained this way and they are plotted with water content in % along the X-axis and the dry density in lbs/ft^3 along the Y-axis. The water content at which the dry density is maximum gives the optimum moisture content (OMC) for that particular soil sample.

Determination of Optimum Moisture Content for Treated and Raw Soil Samples

Treated Soil -Roadbond EN1 and EMC Squared

The ratio of dilution, by weight of stabilizer to that of water, as recommended by manufacturers varied from 1:220 through 1:300 to 1:500. It was decided to determine the

OMC of the sample treated with the proportion 1:220 and then apply the same Optimum Moisture Content for the other two proportions.

Roadbond EN1

The stabilizer is diluted to a proportion of 1:220, by taking 1 part by weight of stabilizer (in the liquid form) to 220 parts by weight of water. The stabilizer was added to the water in a glass/plastic bottle large enough to hold the contents, corked and then shaken well so that the stabilizer-water is mixed thoroughly.

An approximate percentage of diluted stabilizer solution by weight of soil is added to the weighed dried soil and mixed thoroughly so that the liquid completely wets the soil. As per the manufacturer's recommendation, plastic containers and plastic mixing tools are used. It is also recommended that adequate caution be exercised and protective clothing (thick rubber gloves and goggles) should be worn during the diluting and mixing process as the liquid, even in the diluted form, is warned to be corrosive.

The wetted soil is then wrapped and sealed in polyethylene bags labeled (for identification purposes) and allowed to mature over a period of 36 hours at room temperature (manufacturer's recommendation) after which they are molded into samples.

The OMC is determined by varying the percentage of stabilizer solution added to the soil and thus, getting a w/c vs. dry density curve. The OMC obtained for the proportion 1:220 is used for molding samples at 1:300 and 1:500 proportions of stabilizers for triaxial and swell tests.

EMC Squared

The manufacturers of EMC Squared (Earth Materials Catalyst) recommend that for improved reduction in hydraulic conductivity, the additive EMS (Earth Materials Sealant) be used in conjunction with EMC Squared. In this study, the researchers used such a combination after the recommendation by the manufacturer. The recommended proportion of EMC^2 to EMS is 1:2. As before in the case of Roadbond EN1, it was decided to determine the OMC of the solution at a proportion of 1:220. To make the required stabilizer mix, the percentages by weight of diluted stabilizer solution to be added to the soil to determine the OMC is approximated. From this value and the proportion 1:220, weight of stabilizer to be added to the water is determined. The amount of EMS sealant is twice that of EMC^2. The quantities for EMS and EMC^2 are worked out in this manner and are added individually to the compaction water and are then mixed thoroughly. This stabilizer solution does not require any special mixing tools or protective clothing (except goggles) during the mixing or diluting process, as per the manufacturer.

The diluted stabilizer solution is then added to the soil at a certain starting percentage by weight of the soil, to the weighed dried soil and mixed thoroughly. The wetted soil is then wrapped in polyethylene bags, labeled and allowed to mature for a period of 36 hours at room temperature (as per the manufacturer's recommendation) after which the soil is molded into cylinders. As in Roadbond EN1, the OMC is determined by varying the percentages of diluted stabilizer solution until a w/c vs. dry density curve is obtained. The OMC thus obtained using the proportion of 1:220 is used for molding the soil samples at the proportions of 1:300 and 1:500 of stabilizer.

ClayPack

For ClayPack, the manufacturer's recommendation of 1:300 of proportion of stabilizer to water by weight was followed. One part of stabilizer was mixed with 300 parts of water and mixed well.

A certain percentage of diluted stabilizer by weight of soil is added to the dried soil and mixed thoroughly until the soil is wetted. After the soil is mixed, the wetted soil is then wrapped in polythene bags and allowed to mature for 24 hours at room temperature (manufacturer's recommendation). Molding is done after this period. The OMC is determined as described previously.

Potassium Hydroxide

The concentration of KOH solution purchased was 50% wt/vol. The researchers decided to do the testing at pH levels of 10 and 12. One hundred grams of concentrated KOH solution was added to 2900 ml of water and a part of the solution was used to wet the soil. The wet soil in a solution of distilled water gave a pH of 10. Similarly 300 g of concentrated KOH solution was added to 2700 ml of water and the resulting soil solution in distilled water was tested with a pH meter, which showed a pH of 12. Although the KOH solution gave a pH of 14 at a concentration of 400 g in 3000 ml of water, the resulting soil solution failed to give a pH above 13. Hence, it was decided to test the soil at the above mentioned concentrations of KOH only. The OMC of the soil at a pH of 10 was determined and the same was used for molding the soil samples using the solution of pH 12. The OMC was determined in the following way.

The KOH solution was prepared at a pH of 10 using the predetermined concentrations and a certain percent of the solution by weight of the soil is added to the dried weighed soil and mixed thoroughly. The wetted soil is then wrapped in polythene bags at room temperature and allowed to mature overnight. Molding was done the next day. The percentages of solution added to the soil was varied, until a curve of w/c vs. dry density is obtained, from which the maximum dry density is obtained. The OMC thus obtained was used for molding samples which were wetted with the solution that gave a pH of 12.

Ammonium Chloride

The concentration of ammonium chloride purchased from Fisher Scientific is 85 g/ℓ. The pH of the solution in the undiluted form was 6.7. On adding water, the pH shot up to 8 and remained around 8.7 or so after the further addition of water. A volume of 300 ml of the concentrated solution was added to 2700 ml of water (25.5 g/ℓ) and the pH of the resulting solution was found to be 7.4. Since a substantial variation in pH was not found possible with ammonium chloride, it was decided to test the soils at the above mentioned concentration of 25.5g/3ℓ only.

The procedure followed for ammonium chloride for curing, molding and determination of OMC are the same as that followed for potassium hydroxide.

Lime

It was decided to use three proportions of lime, namely 12%, 6% and 3% by weight of soil. The OMC would be determined for 12% lime + soil combination and this value of OMC would be used for molding the 6% and 3% lime treated soil samples. Dried soil is mixed with 12% lime by weight of soil and kept open in the room for a minimum of four hours, after which a certain percentage of water by weight of soil+lime is added and mixed well. The wetted soil is then molded immediately after the mixing. The subsequent points in the w/c vs. dry density curve is obtained by changing the percentage of water added. The OMC obtained from the 12% lime treated soil curve is used for molding 6% and 3% lime treated soil.

Raw Soil

The procedure followed is the same as that of the general method described earlier. Tables 14 and 15 give an outline of the proportion of stabilizers used and the OMC's at which the samples are molded for triaxial and swell tests for SH 161 and IH 635, respectively. Figures 15 and 16 give the w/c vs. dry density curves of the soil samples of SH 161 and IH 635.

Table 14. Stabilizer Treatment Rates for Soils on SH 161

Treated/Untreated	Proportion	OMC %
Roadbond EN1	1:220,1:300,1:500	17.4%
EMC Squared	1:220,1:300,1:500	14.4%
ClayPack	1:300	17.2%
Potassium Hydroxide	Ex: 50g/3ℓ, 150g/3ℓ	16%
Ammonium Chloride	25.5g/3ℓ	14%
Lime	12%,6% and 3%	20.2%
Raw	Not Applicable	17.6%

Table 15. Stabilizer Treatment Rates for Soils on IH 635

Treated/Untreated	Proportion	OMC %
Roadbond EN1	1:220,1:300,1:500	16.2%
EMC Squared	1:220,1:300,1:500	15.4%
ClayPack	1:300	14.6%
Potassium Hydroxide	Ex: 50g/3ℓ, 150g/3ℓ	14.6%
Ammonium Chloride	25.5g/3ℓ	14.4%
Lime	12%,6% and 3%	17.1%
Raw	Not Applicable	14.4%

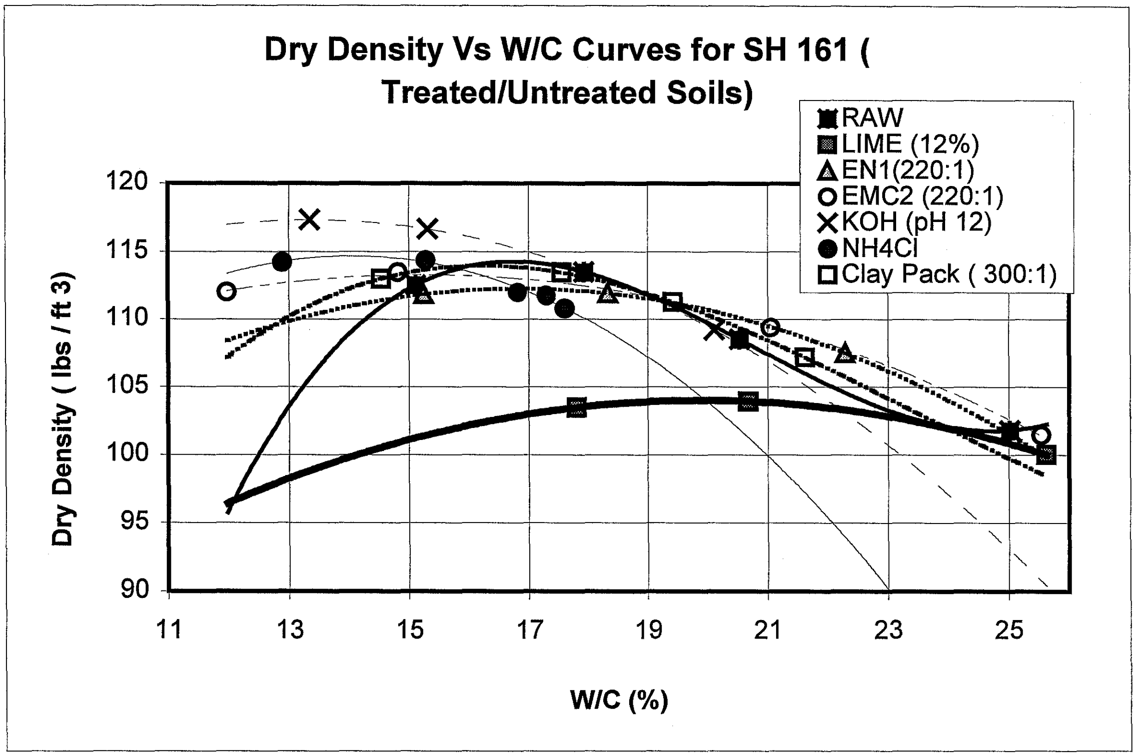


Figure 15. Dry Density Vs W/C Curves for SH 161

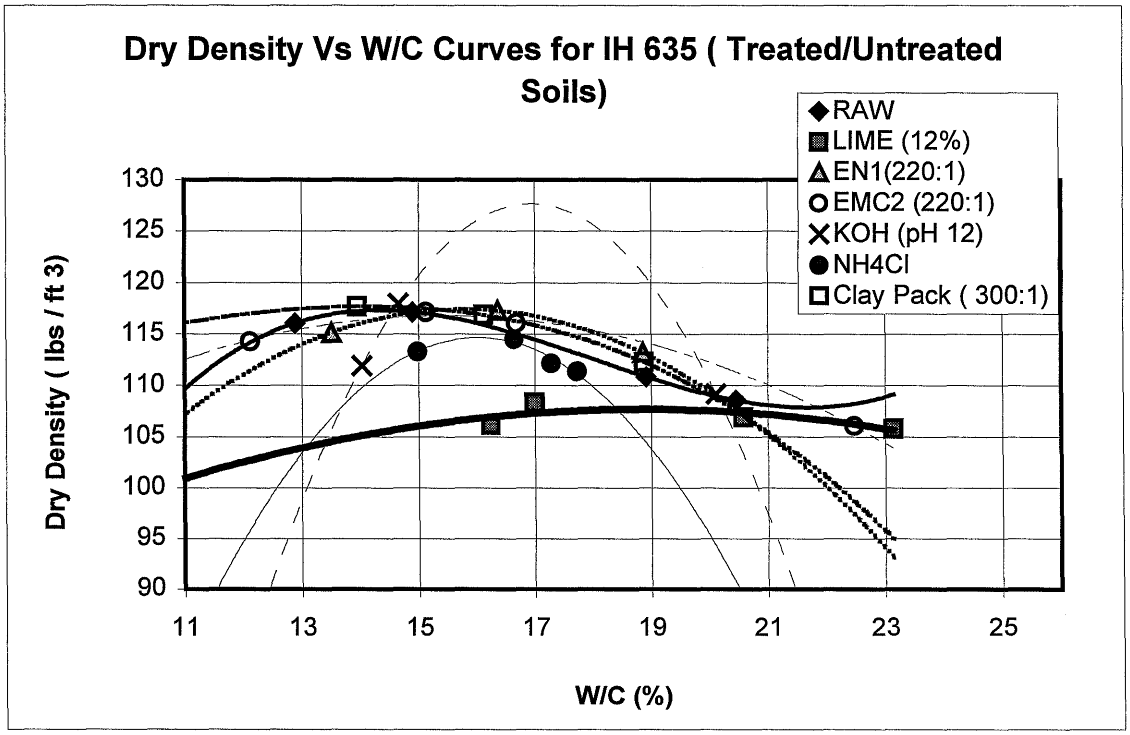


Figure 16. Dry Density Vs W/C Curves for IH 635

DESCRIPTION OF TRIAXIAL TESTING METHODOLOGY

The triaxial testing is performed for each treated/untreated category of soil. For each category of soil, three test samples were molded at the same OMC values. The molded samples were then cured as per the guidelines given in table 16. For commercial stabilizers, the methods of curing followed were as per the instructions of the manufacturers. In the case of standard stabilizers like lime and chemicals like potassium hydroxide and ammonium chloride, the methods of curing adopted are based on experience and general guidelines mentioned in relevant literature.

Table 16. Methods of Curing Adopted

Name of Stabilizer	Method of Curing Adopted
Roadbond EN1	Molded samples are left exposed to cure at room temperature for a period of 36 hours, after which they are tested.
EMC Squared	"
ClayPack	Molded samples are wrapped in cellophane, immediately after molding and a few holes are made at the base of the sample to allow for water flow which is generated during the exothermic reaction taking place in the treated soil. The samples are then cured over a period of 48 hours at room temperature after which they are tested.
Potassium Hydroxide	The samples are wrapped in cellophane after molding and kept in a 40° C room to be cured for seven days. Testing is done on the eighth day.
Ammonium Chloride	"
Lime	"
Raw	"

Automated triaxial testing equipment was used for the test. For each category of soil, testing was done at 0, 4 and 8 psi of confining pressures by increasing the axial load until the specimen failed (e.g., sample #1 of EN1 1:220 was tested at 0 psi cell pressure, sample #2 was tested at 4 psi cell pressure and sample #3 was tested at 8 psi cell pressure).

The corresponding value is of maximum load, deflection, corrected vertical stress at failure and all the other data points were recorded using the automated data acquisition system and a corrected vertical stress vs. percent strain curve was plotted.

Using the data points recorded during the test, a second graph was plotted with percent strain along the X-axis and (percent strain/corrected vertical stress) along the Y-axis. The shape of this graph is a straight line. The reciprocal value of the slope and Y intercept of the straight line portion of this graph gives is the corresponding ultimate strength and stiffness of the tested soil specimens, respectively.

The above mentioned graphs were plotted for all 90 samples (45 each for SH 161 and IH 635) obtaining regression equations of the form y = mx+c, m being the slope and c being the Y-intercept of the graph. These graphs are given in the appendix.

Tables were drawn up for soil from each of the locations listing the category of soil (treated/untreated), values m and c, ultimate strength, strength at failure, strength ratio and stiffness. Using the data in these tables, charts were plotted that compare the strengths at failure of each category of soil (treated/untreated) at three different confining pressures for both SH 161 and IH 635. Also, tables that compare the stiffnesses of the tested specimens at zero confining pressures of both SH 161 and IH 635 were drawn up. These charts are given in chapter VI. Figure 17 shows the picture of automatic triaxial testing equipment.

Figure 17. Trixial Strength Testing Equipment (INSTRON)
(Inside McNew Lab)

DESCRIPTION OF SWELL TEST METHODOLOGY

In the swell test we measure the percentage expansion of the stabilized/raw soil samples over a period of time during which 90% of swell of soil samples has occurred in a controlled environment. A total number of 84 samples, 42 for each highway, were molded at a water content equal to their OMC values. Details of samples are given in table 17.

Table 17. Description of Samples for Each Location

Description of Soil Sample	No. of Samples in each Location	
	SH 161	**IH 635**
Roadbond EN1 @ 1:220	3	3
Roadbond EN1 @ 1:300	3	3
Roadbond EN1 @ 1:500	3	3
EMC^2@1:220	3	3
EMC^2@1:300	3	3
EMC^2@1:500	3	3
ClayPack@1:300	3	3
KOH(50g/3ℓ)	3	3
KOH(150g/3ℓ)	3	3
NH_4Cl(25.5g/3ℓ)	3	3
Lime (12%)	3	3
Lime (6%)	3	3
Lime (3%)	3	3
Raw	3	3
Total # of Samples	42	42

Immediately after molding, porous stones are placed at the bottom of core samples and the whole sample with the porous stone is wrapped in a rubber membrane. The samples were then placed in a water bath, which covered two thirds of the bottom porous stone. The whole setup was placed in an environmental control chamber at 10° C (50° F) and 100% relative humidity. The diameter of each sample was measured at the top, bottom and middle of the sample. Measurements of the diameter and height of the samples were taken on a weekly basis. For each category, three samples were prepared and the dimensions of all three were measured and corresponding volume change calculated. An average of the percent volume changes of the three samples is plotted along the Y-axis against the time, in days, against the X-axis.

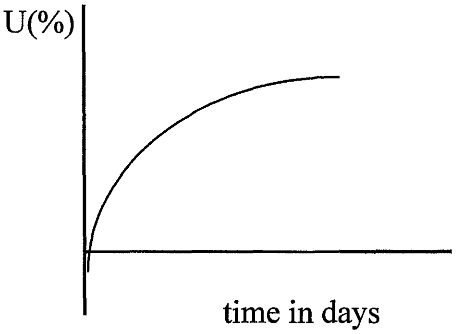
The time, in days, at which 90% swell occurred, was calculated as follows. The trend of average percentage volume change vs. time follows the equation

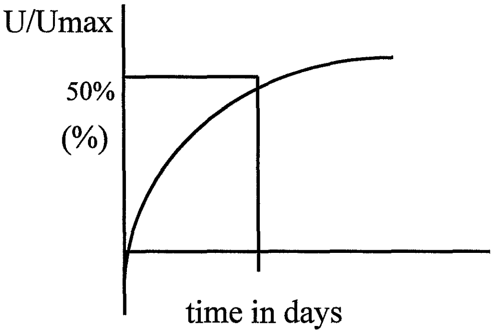
$$U = U_{max}(1-e^{-\alpha t}) \tag{7}$$

where U = average percentage volume change at any time 't'

U_{max} = maximum swell that can occur

α = slope of the $\ln(\delta U/\delta t)$ vs. t curve





Using the Excel chart utility, this equation was force-fitted with the given data points until the value of the correlation coefficient became maximum. The corresponding values of U_{max} and α were taken as the best values for the given data points. Now a curve of U/U_{max} (%) vs time was plotted, from which the time taken for 90% swell was determined. It was found that all the samples have attained 90% of swell within a period of four months since the experiments started.

TESTING METHODOLOGY OF ELECTRICAL CONDUCTIVITY AND DIELECTRIC CONSTANT

Dielectric values indicate the position of the material on the aggregate moisture range and electrical conductivity can give information about whether the material is on drying or wetting cycle (11). The high electrical conductivity values indicate loss of strength of base course (11).

Before the soil samples were placed in the environmental control chamber for the swell tests, their electrical conductivity and dielectric constants were measured using a dielectric constant and electrical conductivity meter. The measurements were made again after the swell tests were completed. The dielectric constant and conductivity meter is manufactured for engineering applications by Adek LTD. of Estonia and it costs approximately $3,000 (8). This meter is equipped with two probes, one of which is the surface measurement probe. The second is a tube measurement probe developed for internal dielectric measurements. The operating frequency for dielectric constant measurements is between 40 MHz and 50 MHz and the surface probe diameter is 50 mm. In this study, the surface probe was used to measure the electrical conductivity and dielectric constants.

Both electrical conductivity and dielectric constants are related to the available unbound moisture content of the soil. Low dielectric values (5.5 - 6.5) in compacted samples indicate the presence of small amounts of adsorption water and optimum strength properties (11). Values < 4.5 - 5 indicate the lack of adsorption water contact and the tensile strength of the material is also lower (11). Higher values indicate that the material is sensitive to moisture and dielectric values over 9 - 10 are considered undesirable (11). At value greater than 16, plastic deformation will occur in the structure. Also the suggested optimum electrical conductivity value for the base material is less than 100 mS/cm (11).

Setup and Operation of Adek Dielectric Constant and Conductivity Meter

1. The appropriate electrode-measuring device (probe/tube) is connected to the meter.

2. Press the off/on switch depending on whether the instrument is operated by battery.
3. To measure the conductivity, the σ / ε_r button should be up and to measure ε_r, the button should be down. Also for both measurements, the button marked I/II should be up. Depending on the material whose properties are to be measured, the normal/fixed rocker switch can be selected, normal for soft material like sand, soil etc. and fixed for hard materials like wood or plastic.
4. After setup, each knob (ε_r and σ) is set to 1 and 0 respectively, by pressing the σ / ε_r button and adjusting each value accordingly, while holding the probe in the air.
5. To read the ε_r with the surface probe, place the probe firmly on top of the sample, keeping as much of the probe in good contact with the sample surface as possible. The ε_r values are read off the scale labeled Er.surf.soI.
6. To read the electrical conductivity with the surface probe, the scale used is labeled as $\sigma._{surf.so}$. The conductivity scale is logarithmic.

Figure 18 shows the Adek dielectric constant and conductivity meter.

METHODOLOGY FOR MEASURING PERMEABILITY

In this study, permeability of molded samples is measured using the rigid wall permeameter technique. As discussed in chapter II, Darcy's equation is followed for determining permeability, assuming that the soil samples are fully saturated, while making the measurements. Rigid wall permeameter is suited for clay samples, where the conductivity is very low and hence the samples need long time monitoring to the order of two to three months. The methodology followed is discussed below.

From the triaxial tests and swell tests, the proportions of stabilizers needed for further testing were determined. The proportion which gives the maximum strength at failure is adopted for stabilizers such as Roadbond EN1, EMC^2, KOH and Lime.

The permeameters consist of the following parts: mold, collar, base with two vertical rods fixed to which the mold and collar can be clamped, a lid with two openings, one for connecting air supply line and the other to remove gas bubbles formed in the soil (this is optional), wing bolts, washers, porous discs, leacheate collector line originating from the base, through which the water flowing through the soil sample can be collected, a beaker with a rubber stopper (holed so that the collector line can be inserted through) to collect the leacheate, and an air supply line to provide pressure, a manifold in case a number of samples need to be tested at the same time, and Teflon gaskets. Figure 19 shows the details of a permeameter.

Based on the strength test results and swell test results, seven samples from each location were tested, each sample was representative of the treatment given.

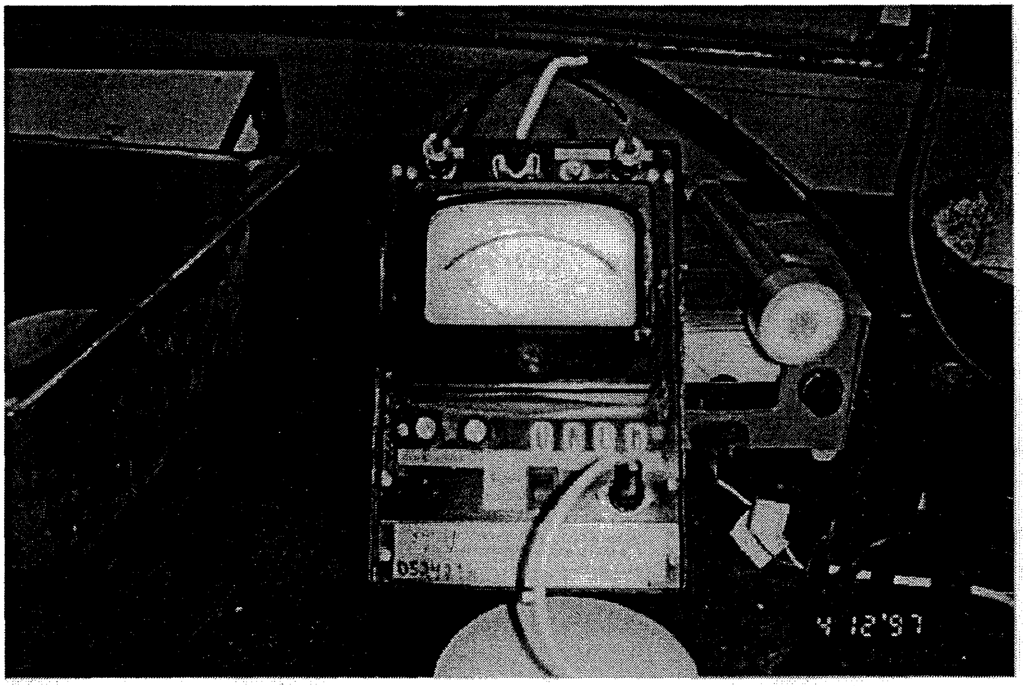


Figure 18. Dielectric Constant and Electrical Conductivity Meter
(Adek Ltd, Estonia)

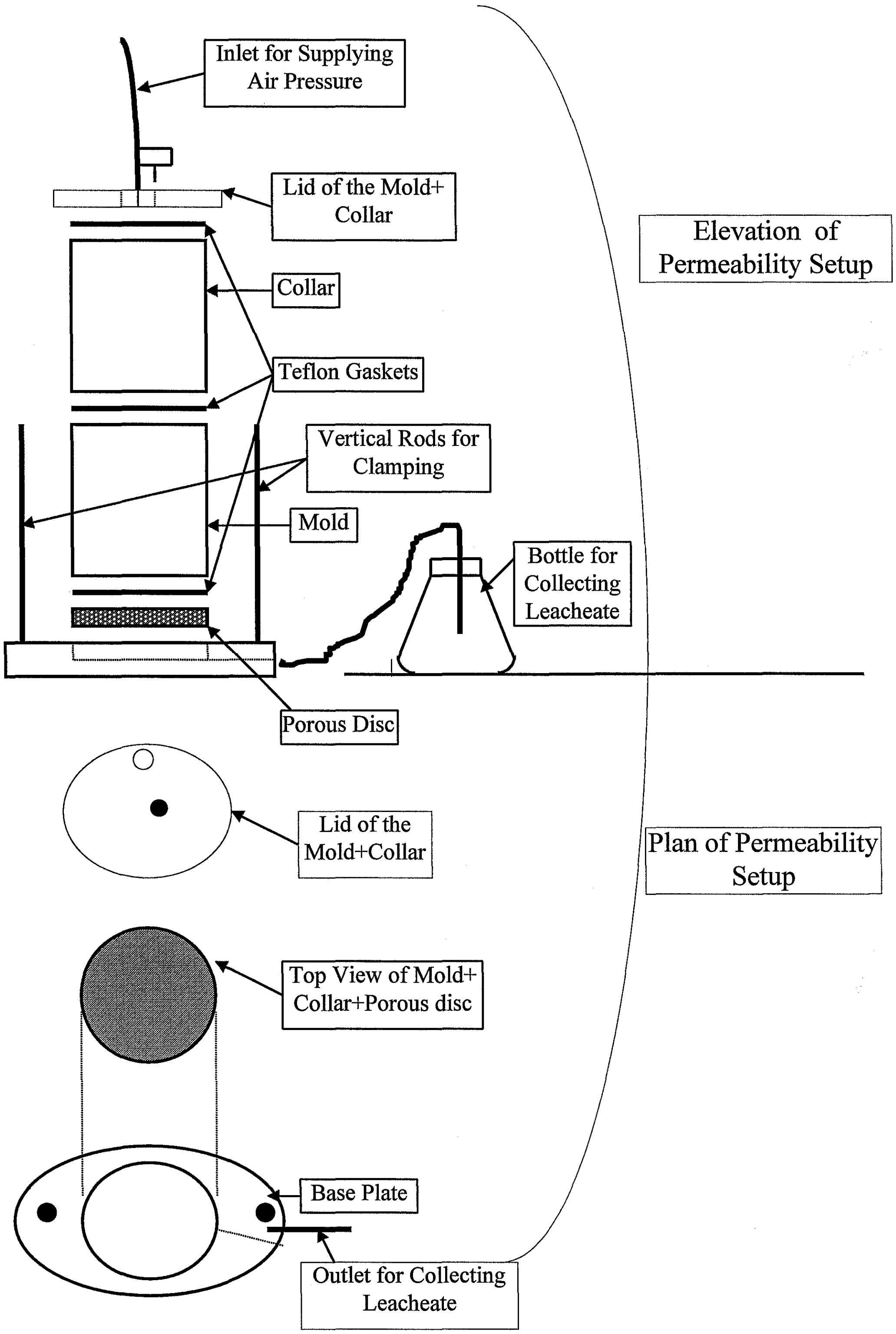


Figure 19. Permeability Test Setup

The researchers decided to start testing all 14 samples simultaneously. A manifold was designed which could supply air pressure to all 14 samples at the same time. The manifold has seven limbs, each limb serving two samples. Figure 20 shows the schematic of the manifold.

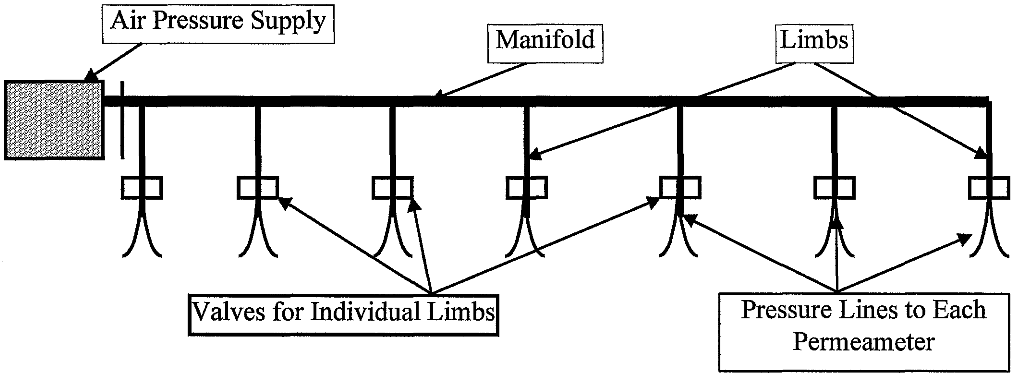


Figure 20. Schematic of Permeameter Setup

The following steps illustrate the method used for setting up the soil samples and making measurements for the permeability test.

1. About 2500 g of soil was taken. The stabilizers/water were added for treated/untreated samples according to the methodology suggested in the previous sections and mixed and cured as per the instructions given.
2. The wetted soil, after proper maturing, was molded at the respective OMC's using the Standard Proctor Compaction Method as follows.

   A square base was put over the pin on the compactor's base, over which the mold was placed. The soil was added in layers and compacted. Then the collar was placed on top of the mold. After the last layer was compacted, the collar was removed, the soil was leveled with the edges of the mold and the collar was replaced.

   The mold with the collar and the trimmed soil samples are placed on an elliptical base over a porous stone. A Teflon gasket was placed between the base and the mold to ensure a good fit. Another Teflon gasket was provided between the mold and the collar and between the collar and the lid.

The lid should slide through the vertical rods in the elliptical base and rest on top of the collar. The whole arrangement is made as tight as possible by bolting down the lid with wing nuts and washers. Also, silicon grease is used around the openings to make the arrangement airtight and leakproof. The air pressure line was connected through the top. The collector line for the leacheates was provided at the base and it was connected through to the porous stone. This line opened out into a flask to collect the liquid.

3. The lid was opened and slid up through the rods. Distilled water was poured into the mold up to the brim of the collar, then the lid was replaced and tightened. The permeant was allowed equilibrate for 24 hours. After 24 hours, it was pressurized.
4. The air pressure line was connected through the top and the pressure on the individual valve was turned on. The whole assembly was checked for leaks especially at either end of supply line, at the permeammeter lid and at all the gasket joints. The date and time at the start of pressurization is noted. In this study, the individual pressure lines supplied 15 psi air pressure. The mold and beaker are to be labeled for purposes of identification.
5. The leacheate is collected and its quantity is measured at least once a day at the same time as the previous day. This step helps in checking the quantity accrued per day. The amount and the time collected are recorded. The air pressure should be maintained as constant as possible. Each setup is checked carefully for leaks daily.
6. Permeability is calculated as follows.

   $Q = A. V = A . k . i$

   Now, $k = Q/A.i$

   'Q ' is the quantity of leacheate collected/second in ml/sec

   If q is the quantity of leacheate collected in mℓ over a time of T days, then

   $Q = q /(T.24x60x60)$

   'A' is the cross sectional area of the mold - 80.1 cm2

   'i' is the hydraulic gradient = $(h+11.5) /11.5$

11.5 cm is the height of the fluid chamber and 'h' is the hydraulic head, and in this case, a constant at a given pressure. At 15 psi air pressure, 'i' is 92.79 cm. Hence the equation used for calculating permeability in this study can be rewritten in the following form.

$k = q / (T \times 6.421 \times 10^8)$

where k is in cm/sec, q is in ml or cc and T is in Days.

METHODOLOGY FOR MEASURING SUCTION (MATRIC AND TOTAL)

In this study, suction is measured of all the 14 samples subjected to permeability tests. Total and matric suction are measured from the soil samples before molding for the permeability tests. Suction is again measured of the core samples after the permeability tests are completed. Total suction is measured of the leacheates from the permeability tests (Appendix D). In the lab total suction is measured using a transistor psychrometer and matric suction is measured using filter paper method.

Transistor Psychrometer

A transistor psychrometer (figure 21) measures the temperature difference between two transistors, one acting as a wet bulb and the other as a dry bulb. The temperature difference between the two transistors is recorded as a voltage output. Testing time for one batch of samples (during measurement or calibration) is only one hour. This instrument can give suction values in the range of pF 3 to pF 5.5 with an accuracy of pF$\pm$0.01. This equipment is strictly for use in a laboratory, because of necessity for temperature control (17).

The instrument consists of a thermally insulated chamber, 12 psychrometer probes and a 12-channel logger. The logger can help keep track of the output, switch between various displays, and redirect the output to a printer etc. The accuracy of the instrument depends on the degree of temperature control during the duration of the test. When the probes are

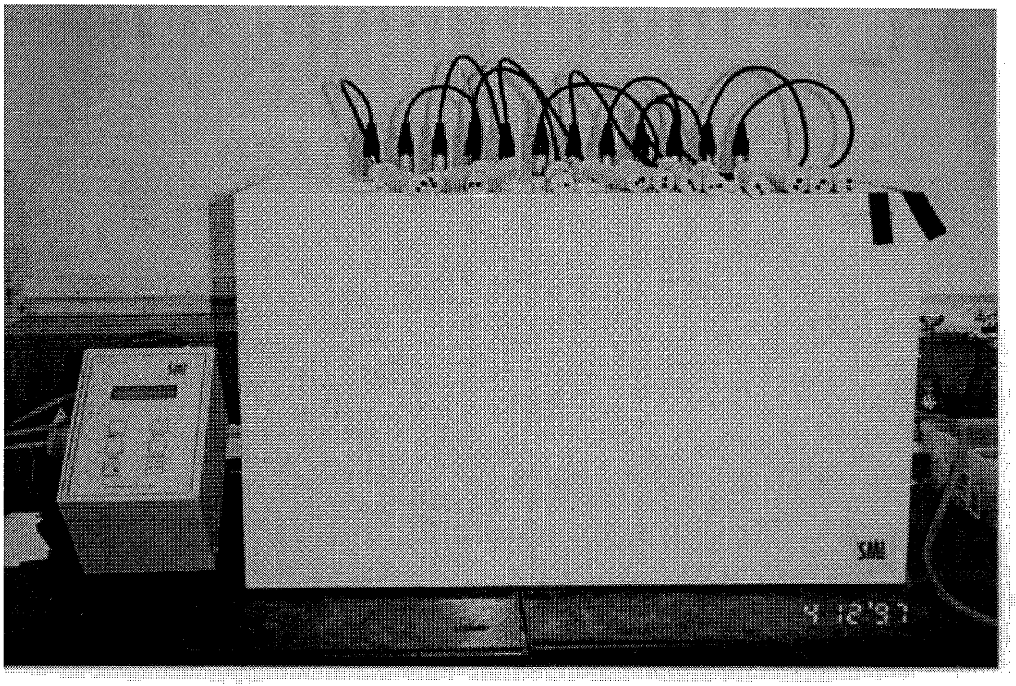


Figure 21. Transistor Psychrometer with Twelve Probes and Channel Logger

inserted into the thermally insulated chamber, the probes and samples are ensured to attain and remain at a constant temperature during the period of any test.

Zeroing/Initialization of Psychrometer

Before the psychrometer is used for making suction measurements or used for calibration purposes, it has to be zeroed or initialized. This is done using a pF 2.0 solution. Filter paper of #2 thickness (Wattman Grade 2, 50 mm diameter) cut to the diameter of calibration caps and are placed in the caps. One drop of pF 2.0 solution is introduced in each cap into the filter paper. A drop of distilled water is introduced into the wet bulb of each probe. The probes are then fitted with the caps and all the probes are inserted into the constant temperature chamber and allowed to remain for 12-14 hours, with the instrument in the switched on mode, to attain equilibrium. At the end of zeroing period, it can be ensured whether zeroing has occurred by checking the output values displayed by the channel logger. Values in the range -10 to -23 milliVolts are considered acceptable. Calibration or suction measurement is made immediately after zeroing.

Prior to placing the samples or the calibration cups, the probes are brought to zero values (by checking the logger) by turning the screws provided on top of the probes, using a screwdriver provided for the same purpose. Immediately after this, the sample caps /calibration caps are placed and sealed to the probe with minimum disturbance to the water drop in the wet bulb. The readings given by the logger after 50 minutes or one hour can be considered as the final output for the given batch of samples. Zeroing is done prior to soil suction measurement or calibration.

Calibration of Transistor Psychrometer

Before the instrument is used for making actual measurements, it has to be calibrated. The readings given by the instrument are in millivolts. Solutions of known pF are prepared which range from pF 3 to 5 using various proportions of sodium chloride in water. The outputs are given by the instrument for each solution are recorded and are plotted in a semilogarthmic chart. The output in millivolts is plotted in a logarithmic scale along the X-axis and the suction values in pF are plotted along the Y-axis in natural scale. For each probe, the output in milliVolts recorded for all the solutions of pF from 3 to 5 are plotted against the corresponding pF values. The resulting scatter of points are fitted best by a straight line. The regression equation to this scatter of points can be used for determining the pF, given an output value in milliVolts. The charts with the respective equations are shown in Appendix C for all probes except probe #7.

A spreadsheet was prepared incorporating the regression equations of all the 12 probes. The spreadsheet calculates the suction in pF corresponding to the output given by psychrometer in milliVolts.

Filter Paper Suction Methods

In this study, filter paper methods are used for measuring matric suction only, though it can be used for measuring both matric and total suction. Before actual measurement, the filter paper used for the measurement is calibrated. This is done as follows. Solutions of known pF are prepared and taken in plastic containers and labeled. Three thicknesses of filter paper are placed in the container in such a way that they do not touch the solution. The containers are sealed and left in a temperature controlled room to attain equilibrium. The time required to attain equilibrium is seven days. After this period, the containers are opened, the sandwiched filter paper is determined for its water content. The measured water content is correlated with the corresponding known suction values in pF. The w/c is plotted along the X-axis on a semilog plot and the suction in pF is plotted on the Y-axis on natural scale. The resulting curve is used for interpreting suction in pF with a given value of measured w/c (Appendix C). After calibrating the filter paper, matric suction is measured as follows. Filter papers of three thicknesses are sandwiched between soil samples in sealed containers and allowed to attain equilibrium for seven days in a temperature controlled room. At the end of seven days, the filter paper in the middle of the three, is determined for its water content. The corresponding value of suction is then read off from the chart prepared earlier.

CHAPTER VI

RESULTS FROM THE LABORATORY TESTING PROGRAM

INTRODUCTION

The results obtained from each test in the test program are presented in this chapter. The results are accompanied by figures and tables to supplement the relevant information.

Results from Triaxial Strength Tests

From the data acquisition system of the automatic triaxial test equipment, the strength at failure for the samples tested is obtained. Also charts that plot % Strain along the X-axis and % strain/corrected vertical stress along the Y-axis are shown in Appendix A. The reciprocal values of slope and Y intercept of the straight line portion of the graphs gives the corresponding ultimate strength and stiffness of the test samples, respectively. From the corrected plots and from the strength at failure, the ultimate strength, strength ratio and stiffness of all the tested samples are determined and tabulated (presented in Appendix B).

Tables 18 and 19 show the ultimate strength and strength at failure for all the samples at 0 psi cell pressure are shown. Tables 20 and 21 compare the stiffnesses of the samples at 0 confining pressures for SH 161 and IH 635. Test samples are subjected to the worst condition of loading at 0 psi cell pressure. Figures 22 and 23 compare the strengths at failure of each category of soil (treated/untreated) at three different confining pressures for both SH 161 and IH 635.

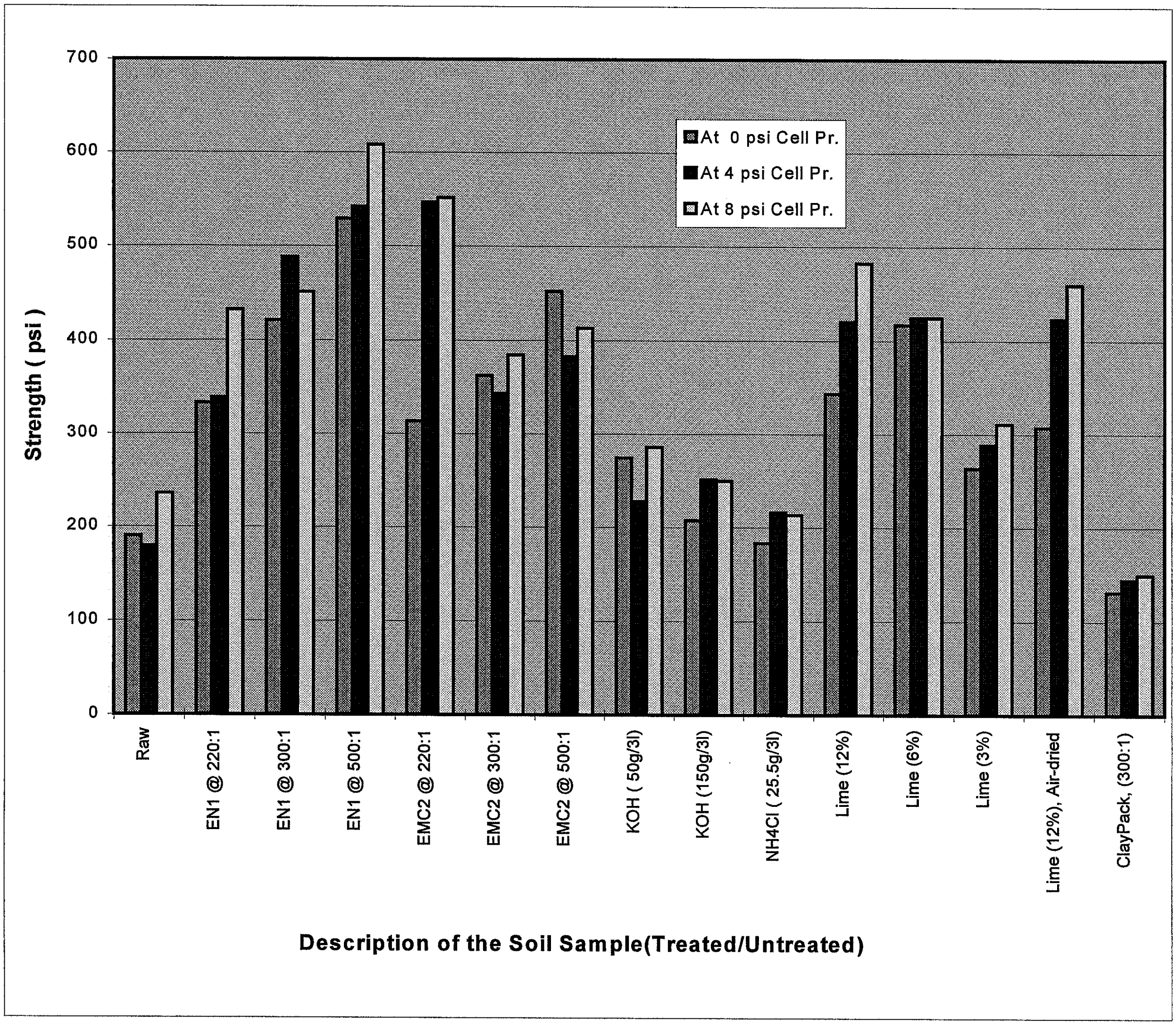


Figure 22. Comparison of Strengths of Soil Samples at Failure S_f, Tested Triaxially Under Three Different Cell Pressures - SH 161

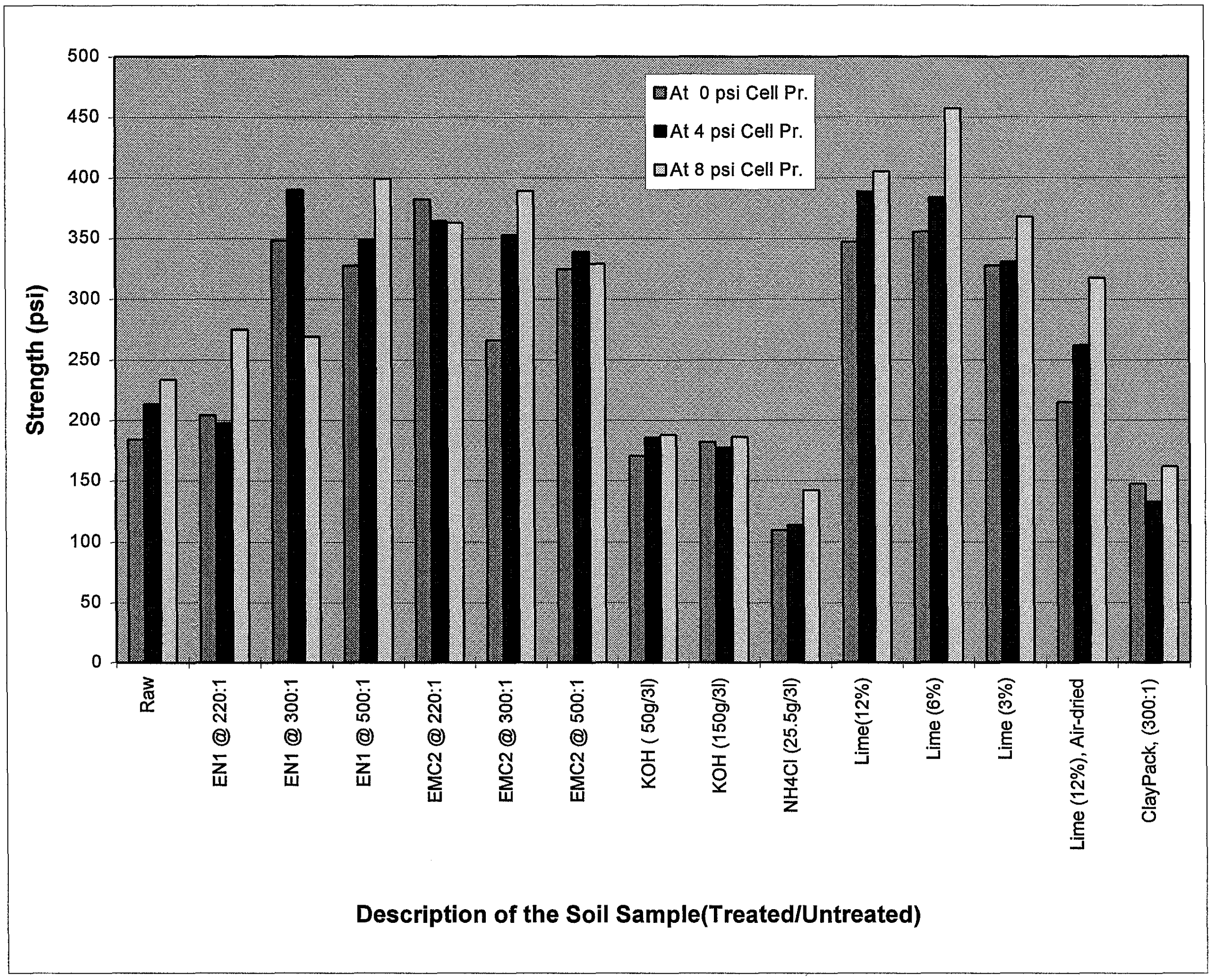


Figure 23. Comparison of Strengths of Soil Samples at Failure S_f, Tested Triaxially Under Three Different Cell Pressures - IH 635

Table 18. Comparison of Strengths Among Samples of SH 161

Treated/Untreated	Ultimate Strength (psi)	Strength at Failure (psi)
EN1 @ 1:300	434.78	421.45
EN1 @ 1:220	400.00	332.76
EN1 @ 1:500	**526.32**	**529.46**
EMC^2 @ 1:220	312.50	313.27
EMC^2 @ 1:300	384.62	362.70
EMC^2 @ 1:500	**500.00**	**451.97**
KOH (50g/3ℓ)	**294.12**	**274.22**
KOH (150g/3ℓ)	243.90	207.46
Lime (12%)	344.83	343.60
Lime (6%)	**416.67**	**417.07**
Lime (3%)	263.16	263.08
ClayPack	163.93	131.63
Raw	232.56	189.95
NH_4Cl	227.27	183.24

Table 19. Comparison of Strengths Among Samples of IH 635

Treated/Untreated	Ultimate Strength (psi)	Strength at Failure (psi)
EN1 @ 1:220	238.10	204.50
EN1 @ 1:300	**434.78**	**347.75**
EN1 @ 1:500	344.83	327.54
EMC^2 @ 1:220	**416.67**	**382.57**
EMC^2 @ 1:300	476.19	266.32
EMC^2 @ 1:500	344.83	324.56
KOH (50g/3ℓ)	204.08	170.55
KOH (150g/3ℓ)	**243.90**	**182.07**
Lime (12%)	357.14	347.19
Lime (6%)	**370.37**	**355.61**
Lime (3%)	333.33	327.77
ClayPack	192.31	147.5
Raw	217.39	184.19
NH_4Cl	196.08	109.72

Table 20. Comparison of Stiffnesses Among Samples of SH 161

Treated / Untreated	Stiffness, psi
EN1 @ 1:220	1111.11
EN1 @ 1:300	5000.00
EN1 @ 1:500	5000.00
EMC^2 @ 1:220	5000.00
EMC^2 @ 1:300	5000.00
EMC^2 @ 1:500	5000.00
KOH (50g/3ℓ)	3333.33
KOH (150g/3ℓ)	1666.67
Lime (12%)	5000.00
Lime (6%)	2000.00
Lime (3%)	2500.00
ClayPack	384.62
Raw	588.24
NH_4Cl	909.09

Table 21. Comparison of Stiffnesses Among Samples of IH 635

Treated / Untreated	Stiffness, psi
EN1 @ 1:220	2000.00
EN1 @ 1:300	1250.00
EN1 @ 1:500	3333.33
EMC^2 @ 1:220	2500.00
EMC^2 @ 1:300	1111.11
EMC^2 @ 1:500	3333.33
KOH (50g/3ℓ)	769.23
KOH (150g/3ℓ)	555.56
Lime (12%)	909.09
Lime (6%)	1111.11
Lime (3%)	1666.67
ClayPack	526.32
Raw	833.33
NH_4Cl	135.14

The highlighted lines in tables 18 and 19 indicate the concentration/proportion of stabilizers at which the tested soil samples showed the greatest strength at failure, when tested triaxially at 0 psi cell pressure. Using the data in tables given in Appendix A, charts (figures 22 and 23) were plotted that compare the strengths at failure of each category of soil (untreated/treated) at three different confining pressures for both SH 161 and IH 635.

From tables 18 and 19 and figures 22 and 23 it can be seen that the maximum strength at failure is offered by EN1 @ 1:500 (529.46 psi) for location SH 161 and by EMC^2 @ 1:220 (382.57psi) for location IH 635. Based on the comparisons of strengths at failure at zero confining pressure for the soil samples, (worst case), and using tables 18 and 19 the researchers decided to use the proportions of stabilizer shown highlighted for the remaining tests, i.e., permeability and suction measurement.

RESULTS FROM SWELL TESTS

A total number of 84 samples, 42 for each highway, were molded at a water content equal to their OMC. For each category, three samples were prepared and the dimensions of all three were measured and the corresponding volume change calculated. Using the measurements made on a weekly basis of all the soil samples, charts were prepared which show the variation in dimensions and the resulting % expansion of the soil samples over a period of four months. For all the samples, 90% of swell occurred within four months of the testing start date. The percentage change in volume is plotted along the Y-axis against time in days along X-axis.

Figures 24 to 31 and 32 to 39 give the plot of percentage change of volume of samples for SH 161 and IH 635, respectively. Figures 40 to 47 and 48 to 53 show the photographs of soil samples after the completion of 90% of swell for SH 161 and IH 635, respectively.

A comparison of all figures from 24 to 31 show that Lime gives the maximum swell for SH 161 samples. Figure 25 indicates that among the three stabilizers, i.e., Lime, EN1 and EMC^2, EN1 (1:500) has the least % expansion for SH 161. Figure 30 shows that the least swell of all treated/untreated soil samples is given by ClayPack. As stated in the previous section, EN1(1:500) shows maximum strength at failure for SH 161. From the swell results, it is seen that its % expansion is relatively low. Figure 33 shows that of the three, i.e., Lime, EN1 and EMC^2, EN1(1:300) shows the least swell for IH 635 samples. Figure 33 also shows that the next least swell is shown by EMC^2 (1:500). Figure 36 indicates that the least swell of all soil samples (treated/untreated) for IH 635 is shown by EN1 (1:300).

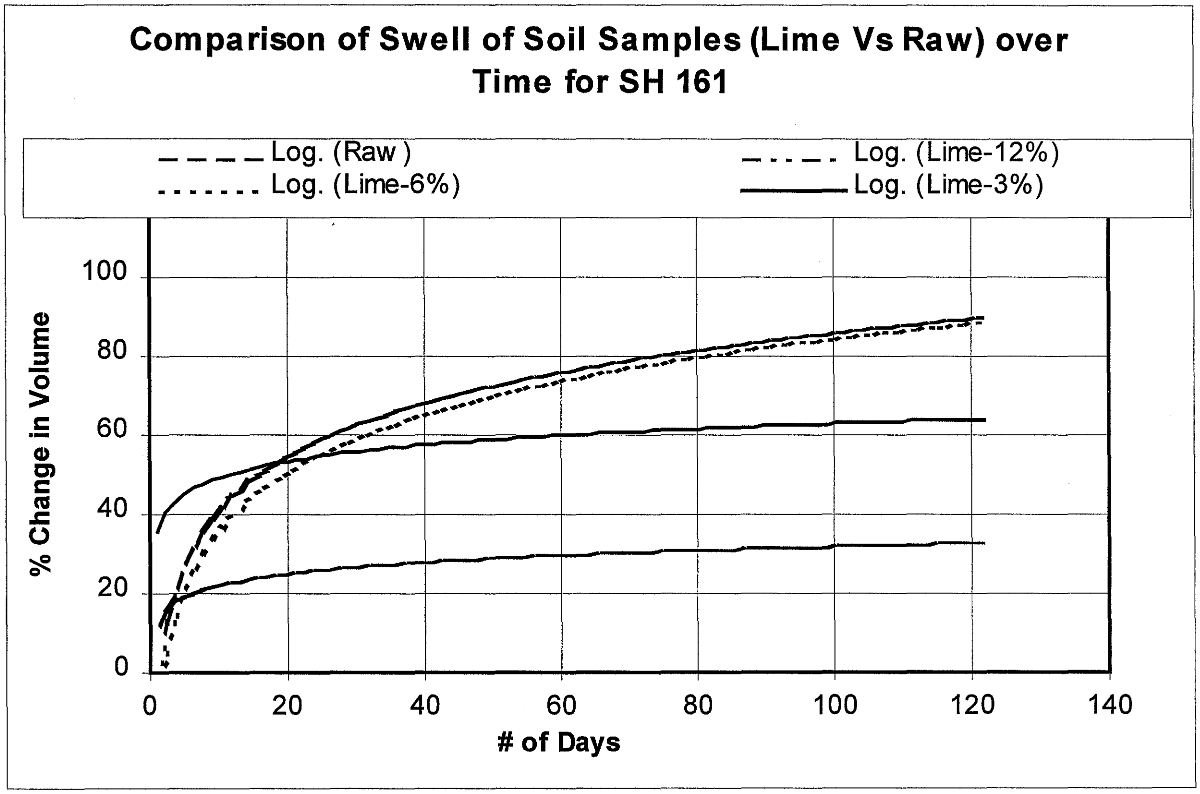


Figure 24. Comparison of Swell of Soil Samples (Lime Vs Raw) for SH 161

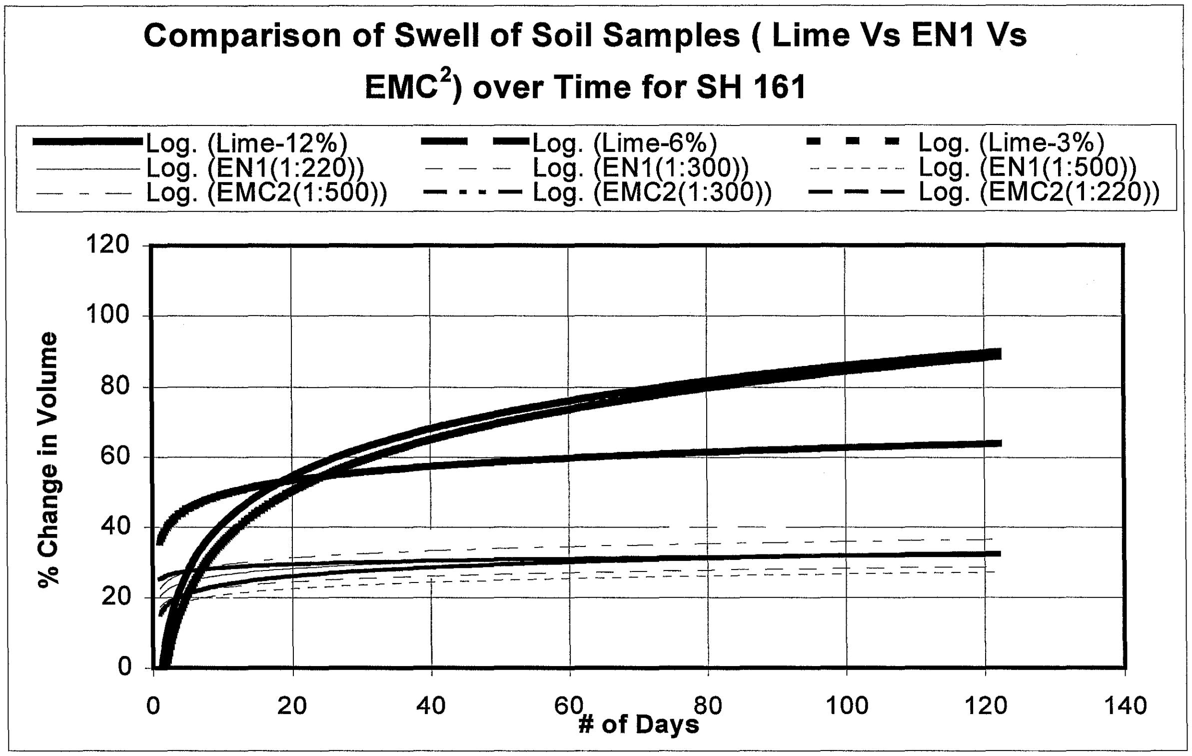


Figure 25. Comparison of Swell of Soil Samples (Lime Vs EN1 Vs EMC^2) for SH 161

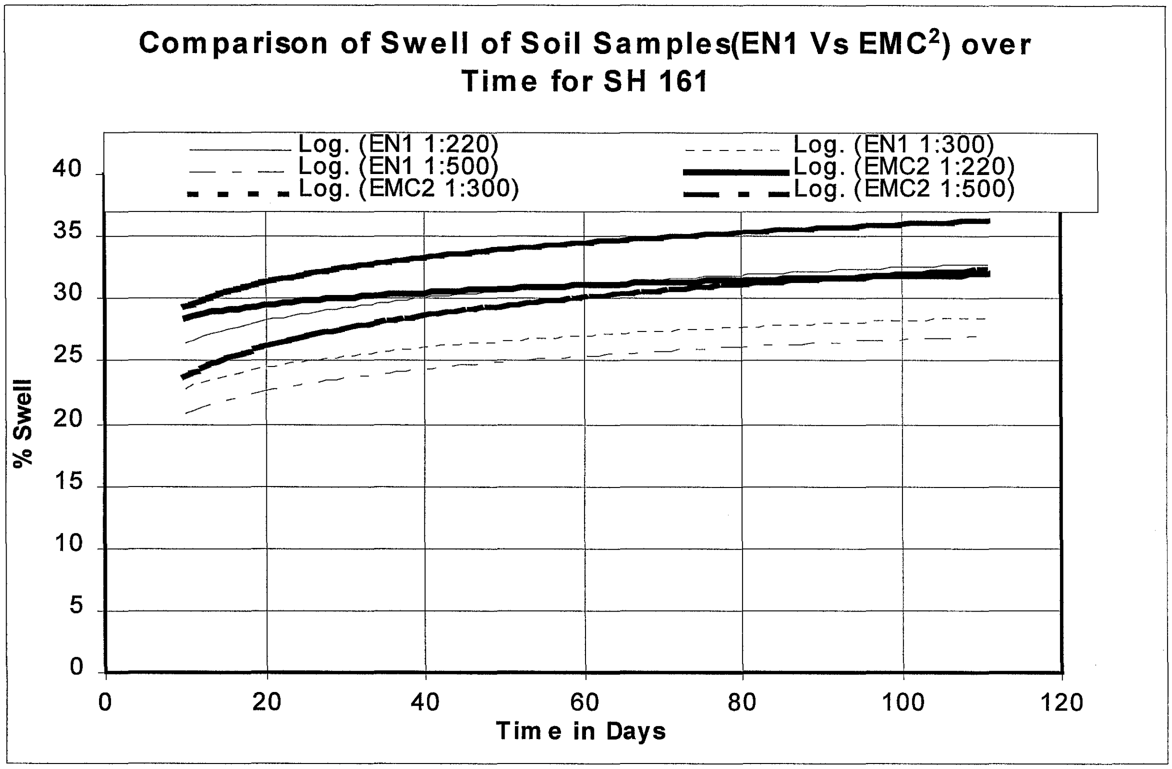


Figure 26. Comparison of Swell of Soil Samples (EN1 Vs EMC^2) for SH 161

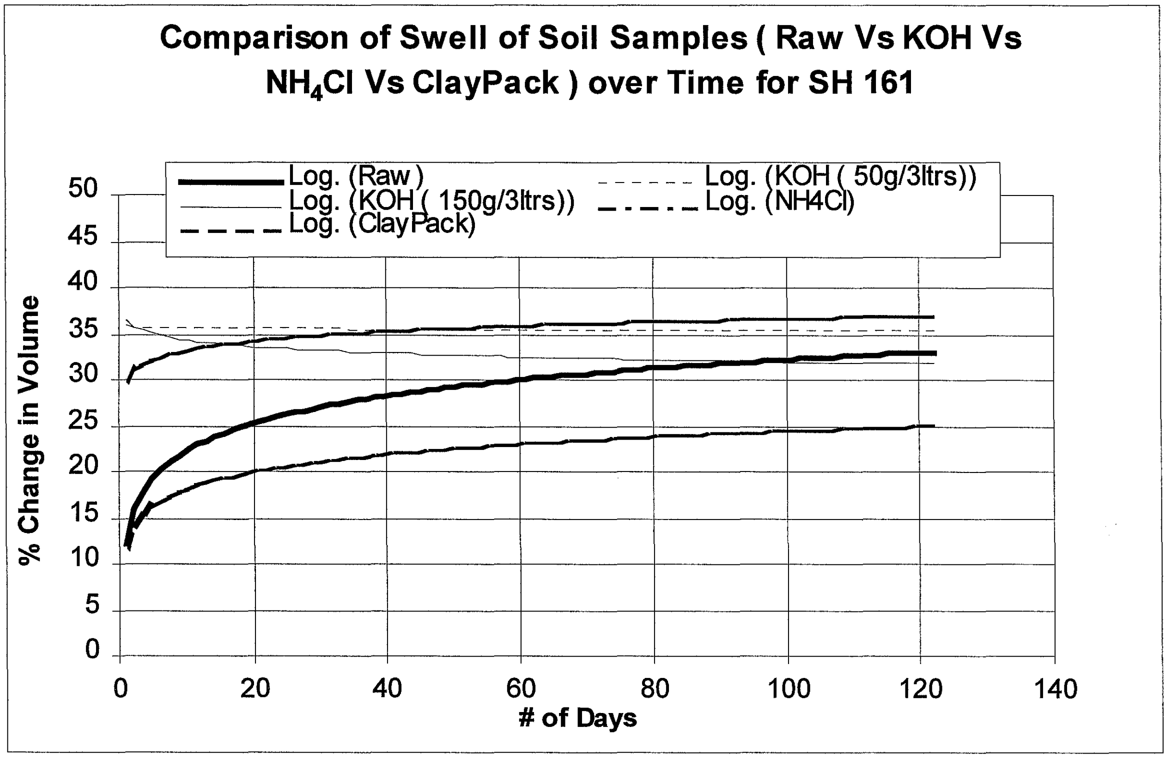


Figure 27. Comparison of Swell of Soil Samples (Raw Vs EN1 Vs EMC^2) for SH 161

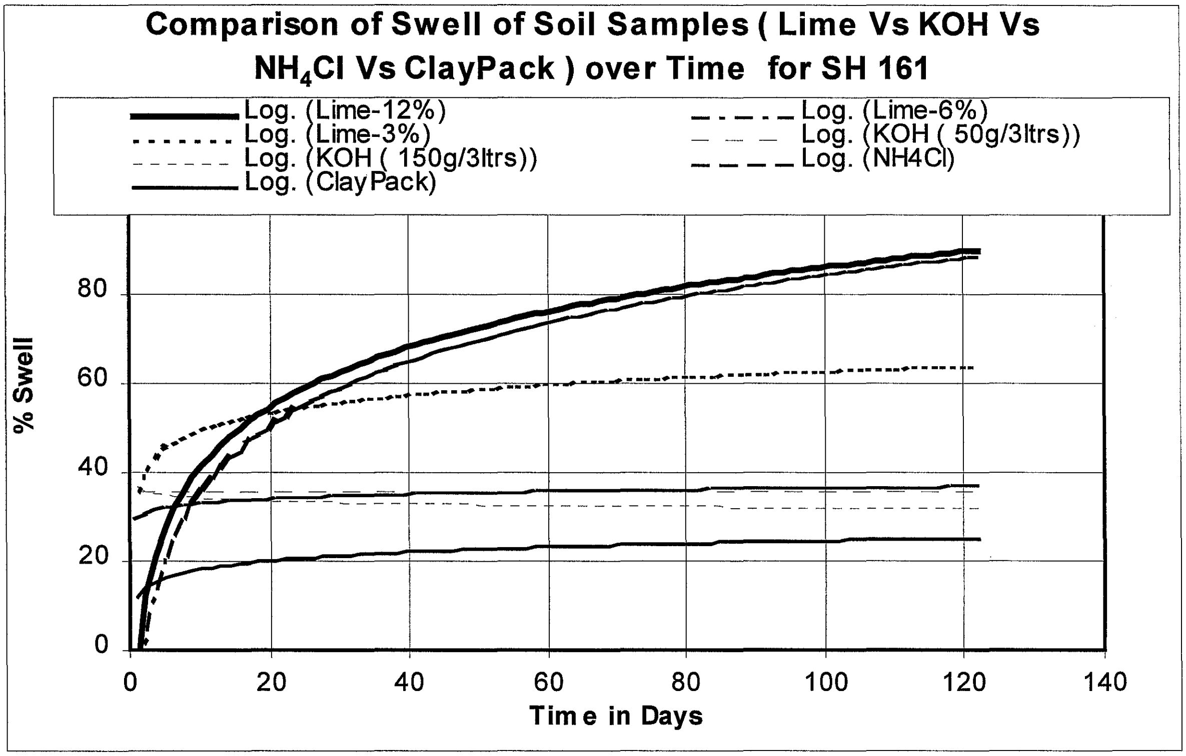


Figure 28. Comparison of Swell of Soil Samples (Lime Vs KOH Vs NH_4Cl Vs ClayPack) for SH 161

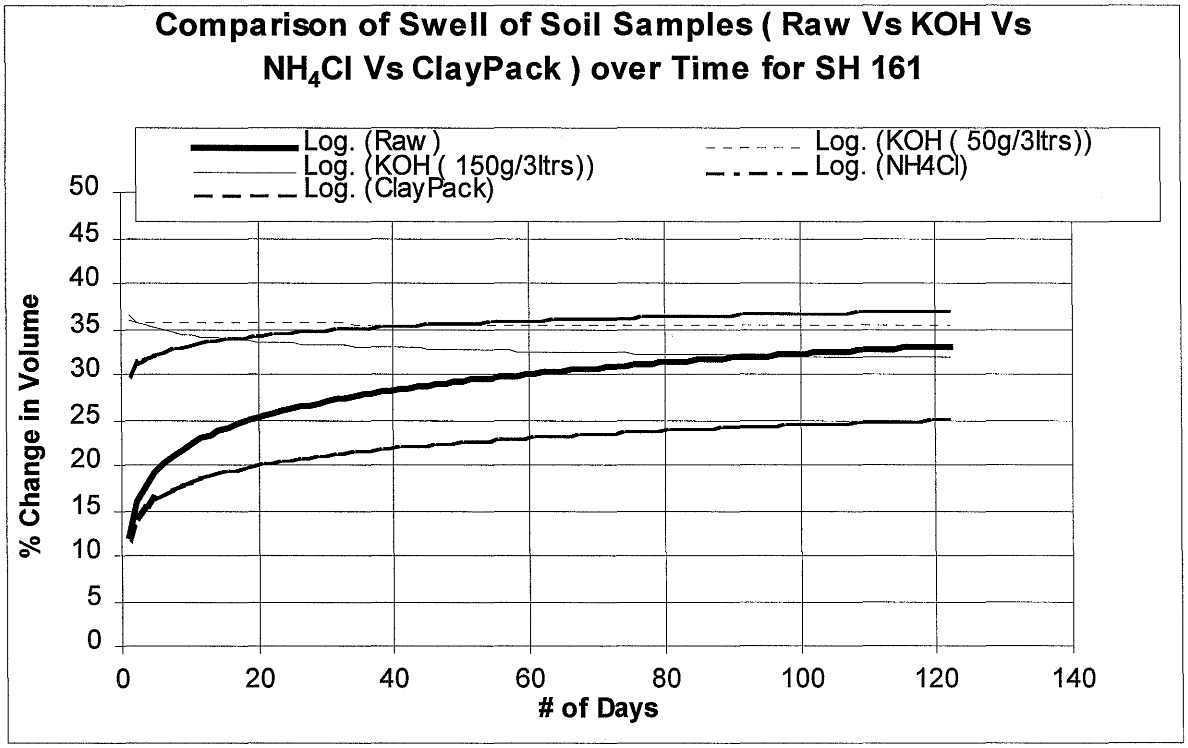


Figure 29. Comparison of Swell of Soil Samples (Raw Vs KOH Vs NH_4Cl Vs ClayPack) for SH 161

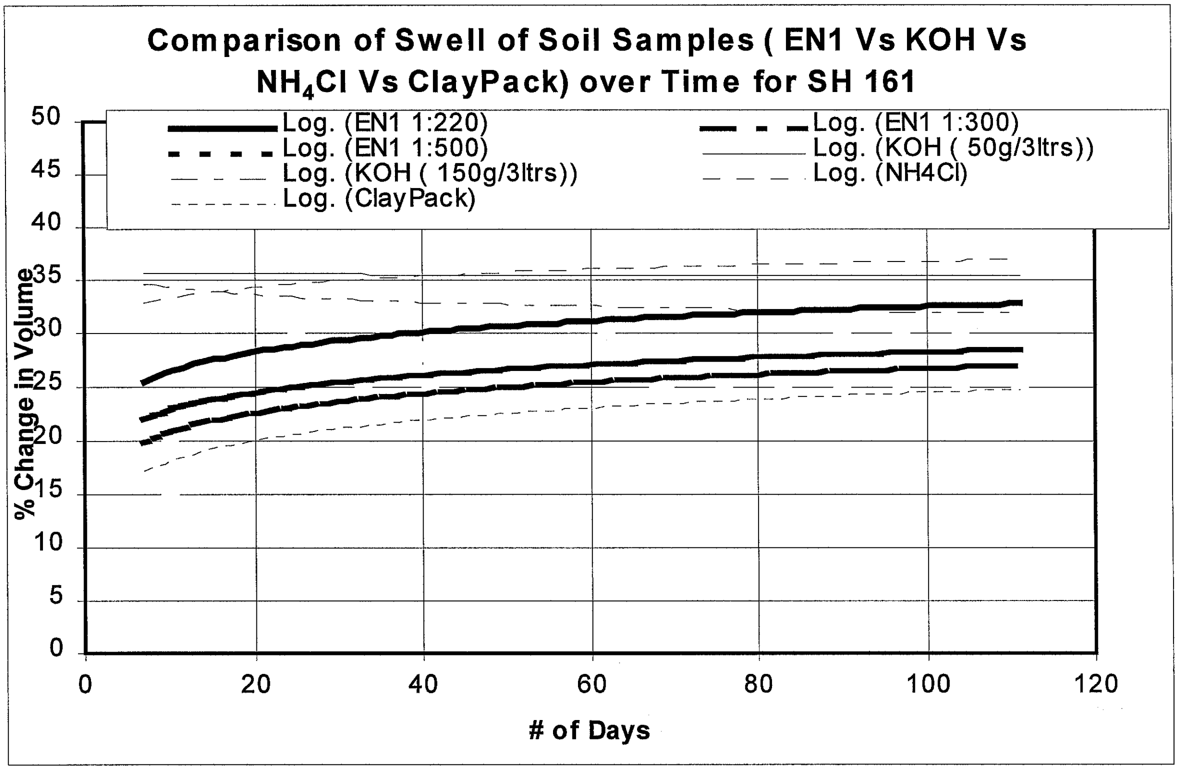


Figure 30. Comparison of Swell of Soil Samples (EN1 Vs KOH Vs NH_4Cl Vs ClayPack) for SH 161

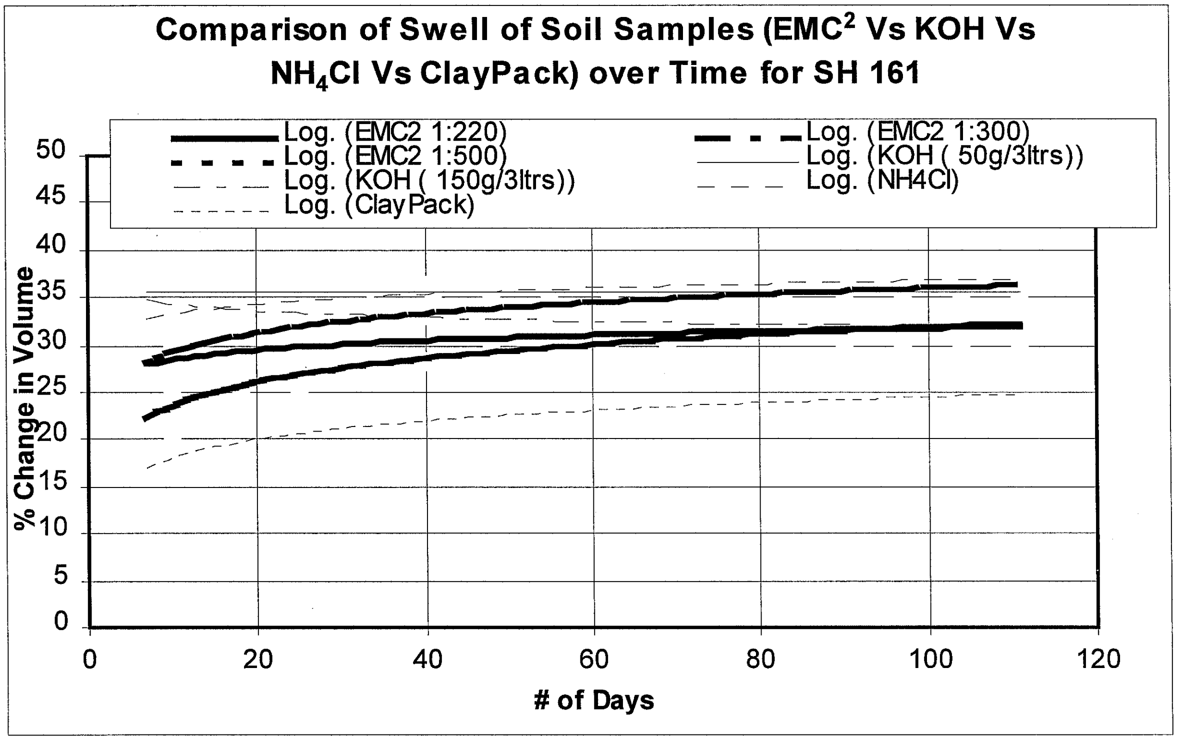


Figure 31. Comparison of Swell of Soil Samples (EMC^2 Vs KOH Vs NH_4Cl Vs ClayPack) for SH 161

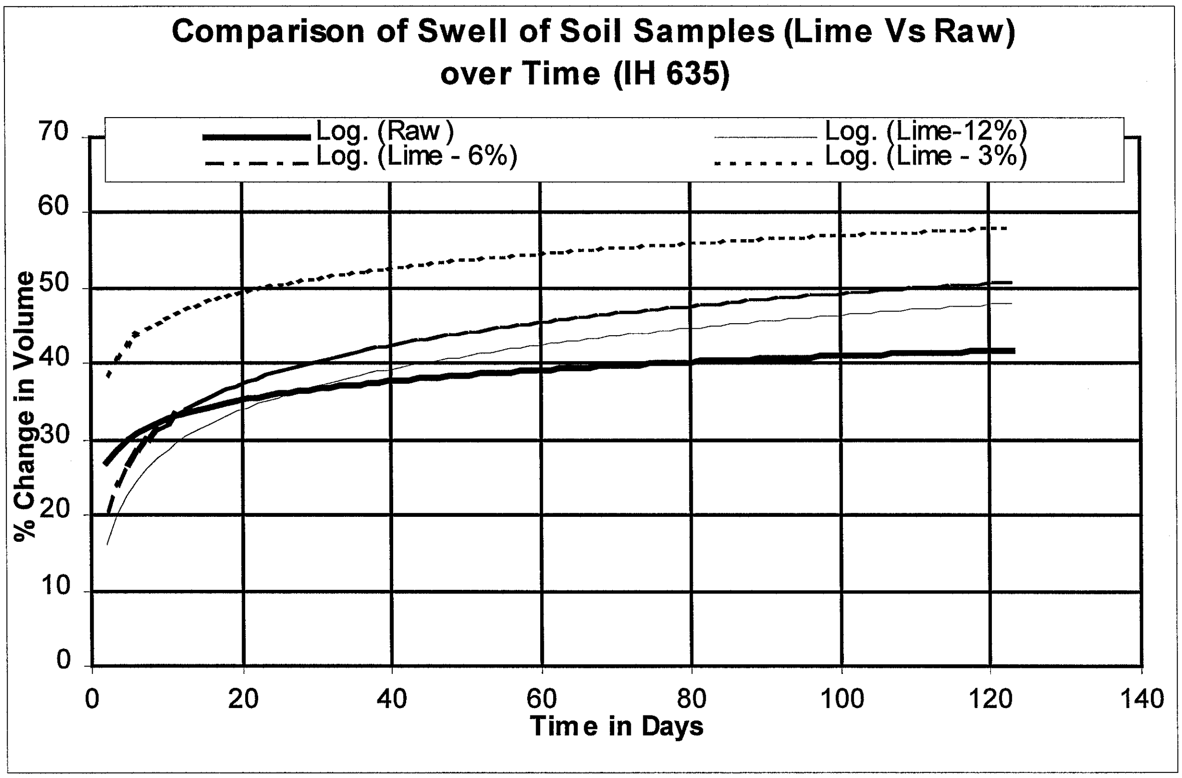


Figure 32. Comparison of Swell of Soil Samples (Lime Vs Raw) for IH 635

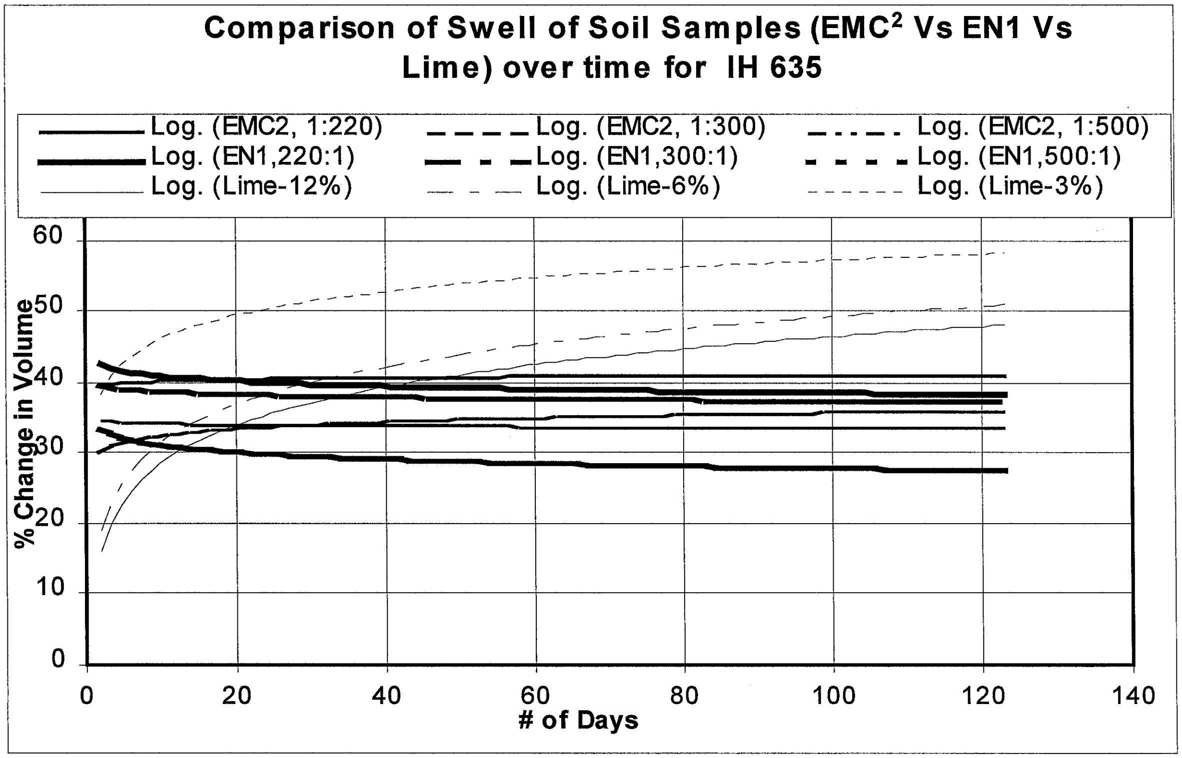


Figure 33. Comparison of Swell of Soil Samples (Lime Vs EN1 Vs EMC^2) for IH 635

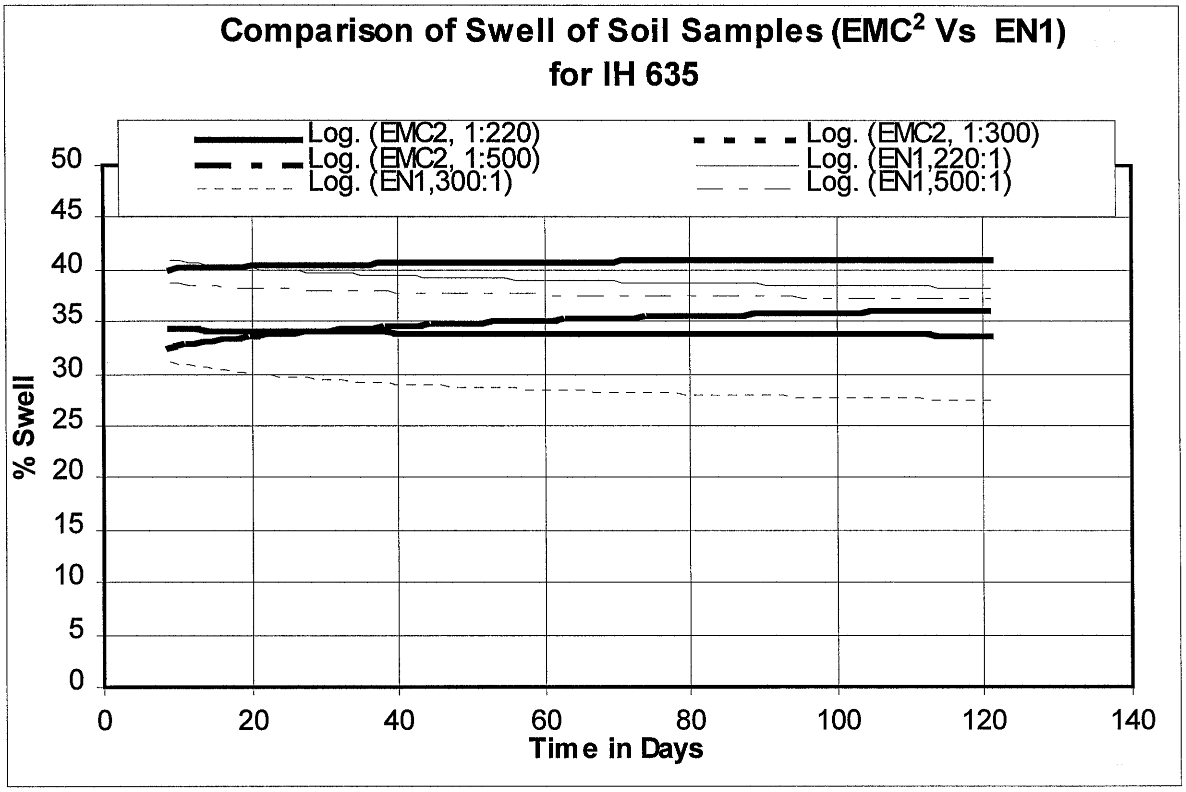


Figure 34. Comparison of Swell of Soil Samples (EN1 Vs EMC^2) for IH 635

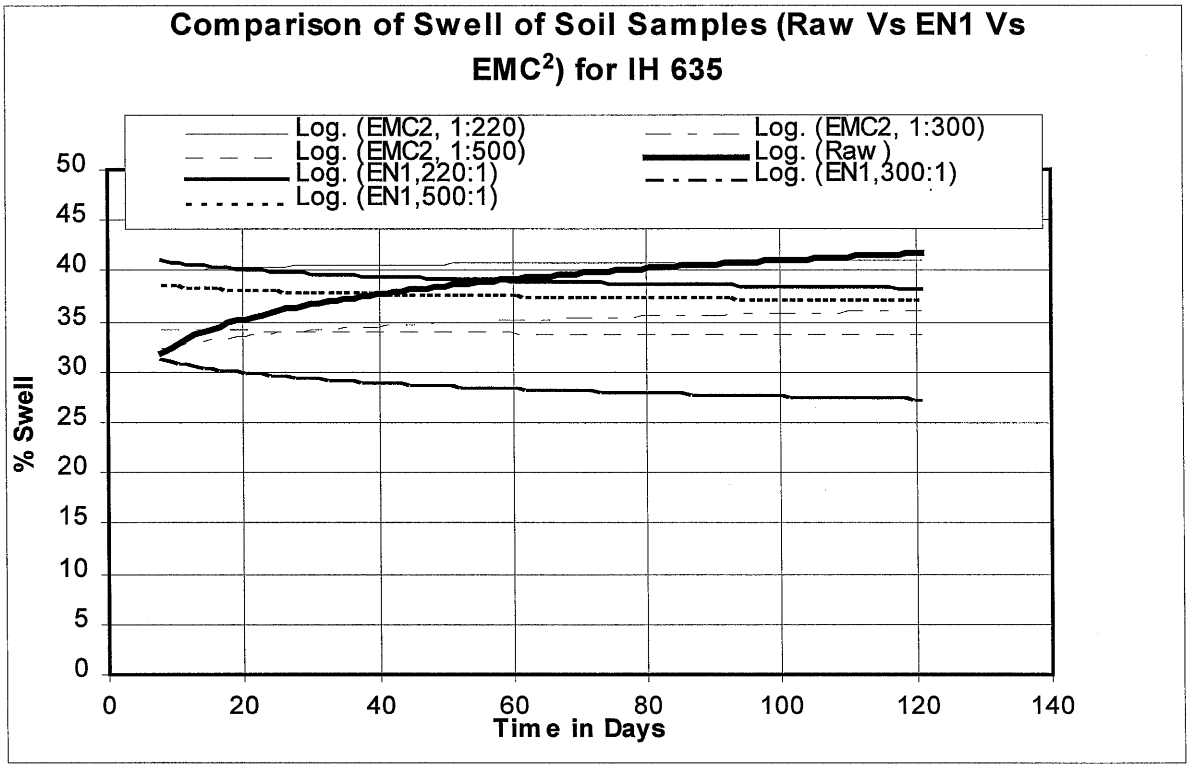


Figure 35. Comparison of Swell of Soil Samples (Raw Vs EN1 Vs EMC^2) for IH 635

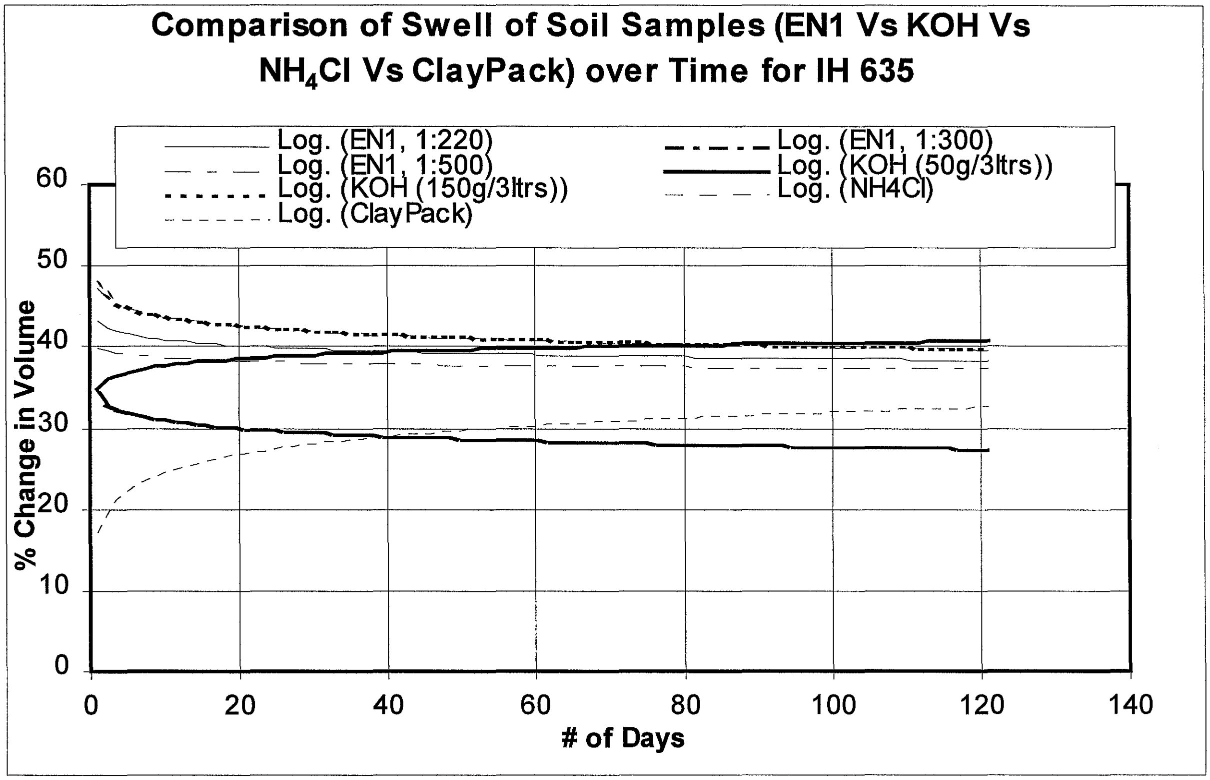


Figure 36. Comparison of Swell of Soil Samples (EN1 Vs KOH Vs NH_4Cl Vs ClayPack) for IH 635

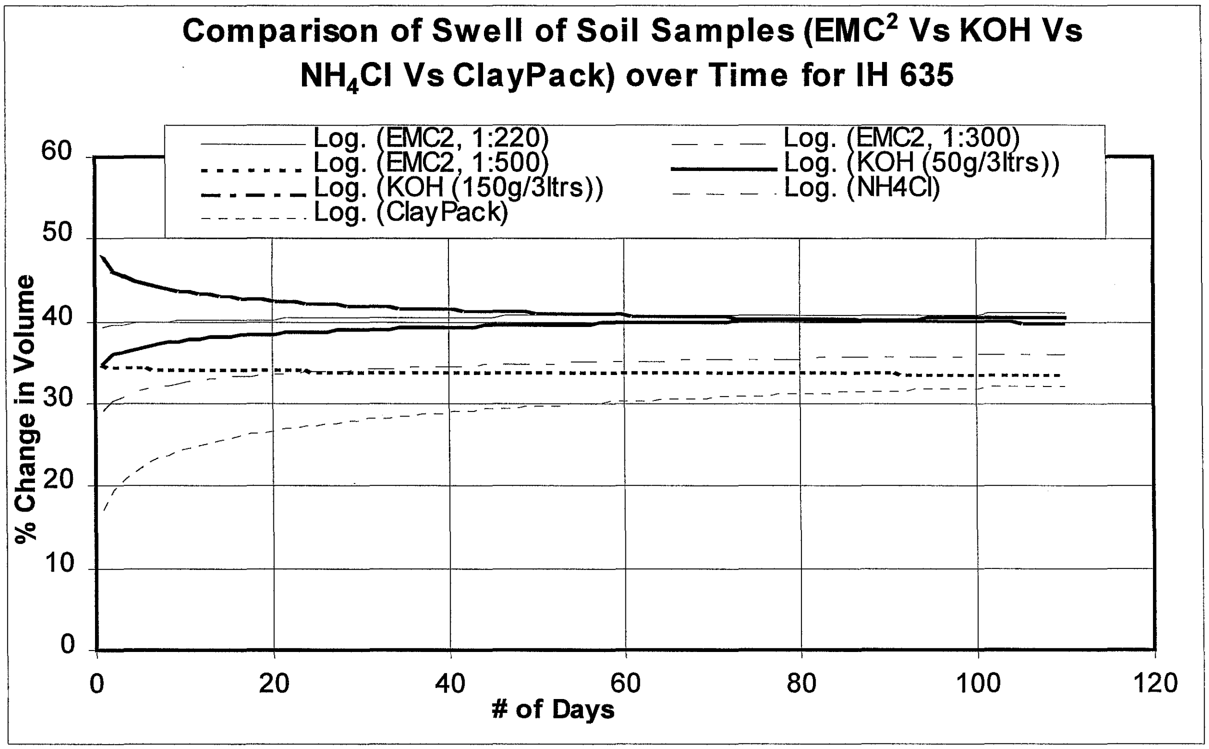


Figure 37. Comparison of Swell of Soil Samples (EMC^2 Vs KOH Vs NH_4Cl Vs ClayPack) for IH 635

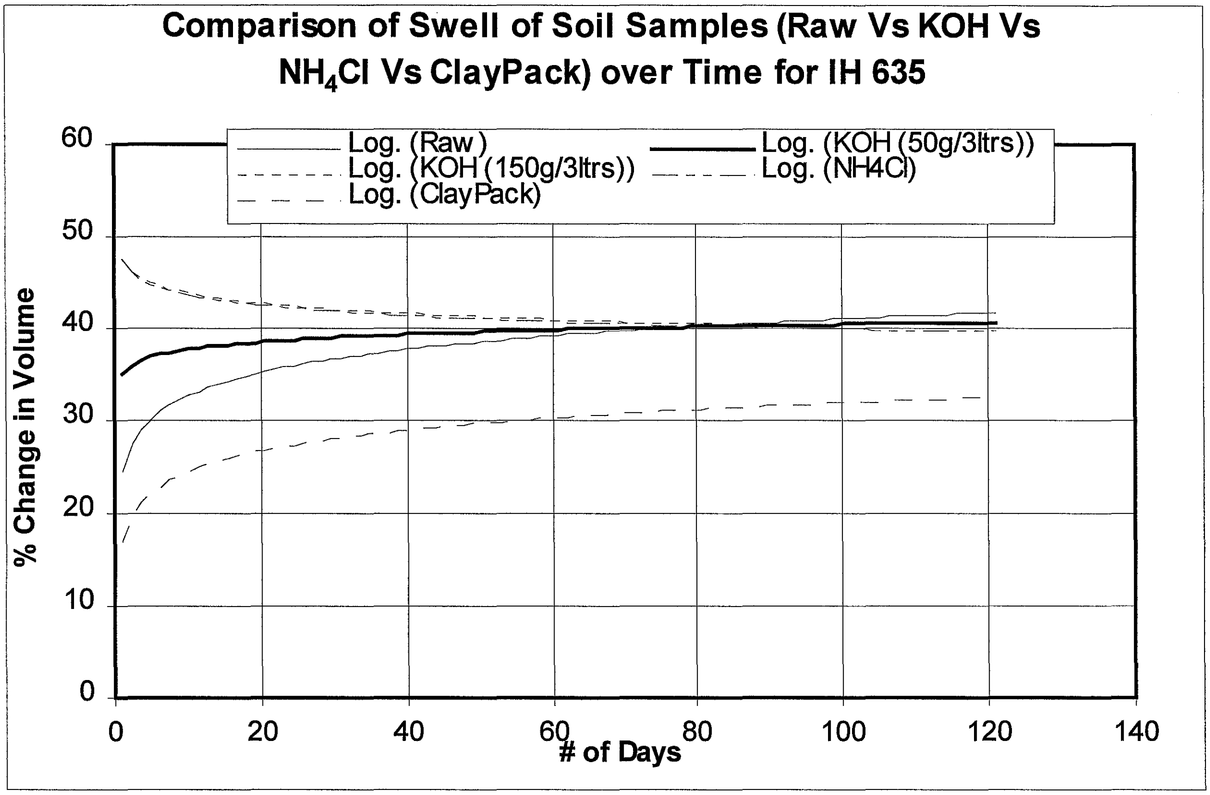


Figure 38. Comparison of Swell of Soil Samples (Raw Vs KOH Vs NH_4Cl Vs ClayPack) for IH 635

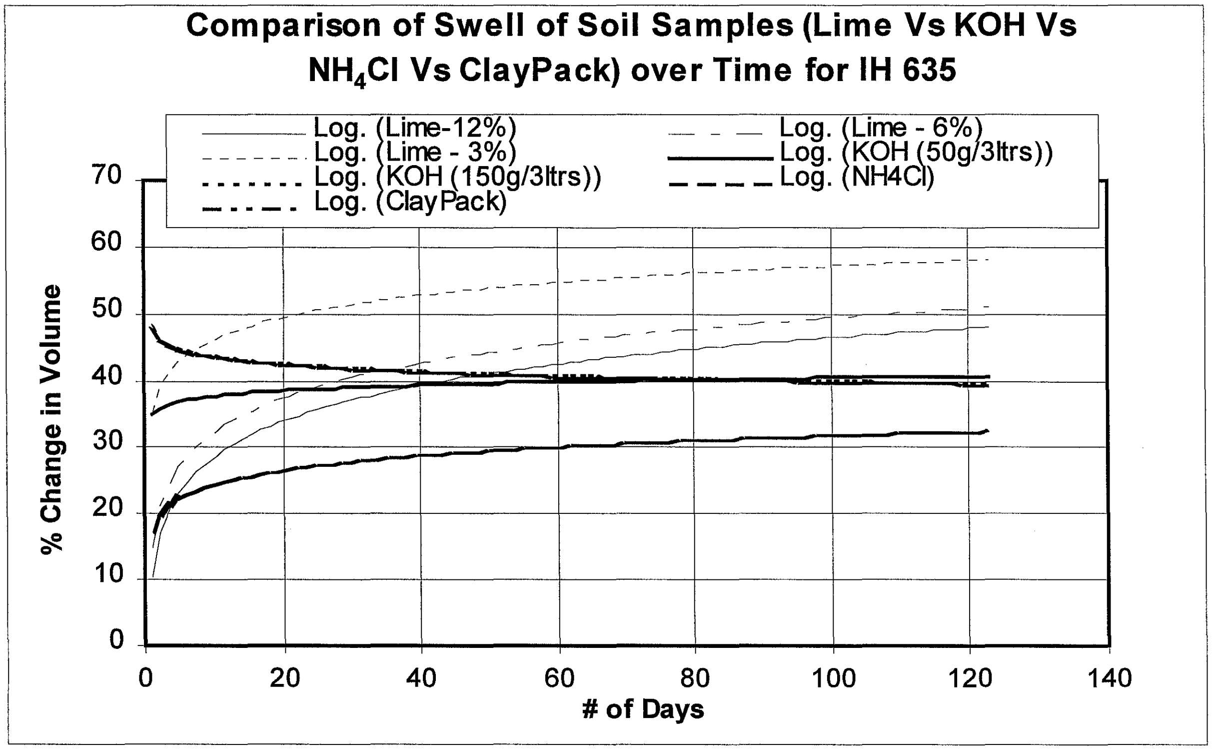


Figure 39. Comparison of Swell of Soil Samples (Lime Vs KOH Vs NH_4Cl Vs ClayPack) for IH 635

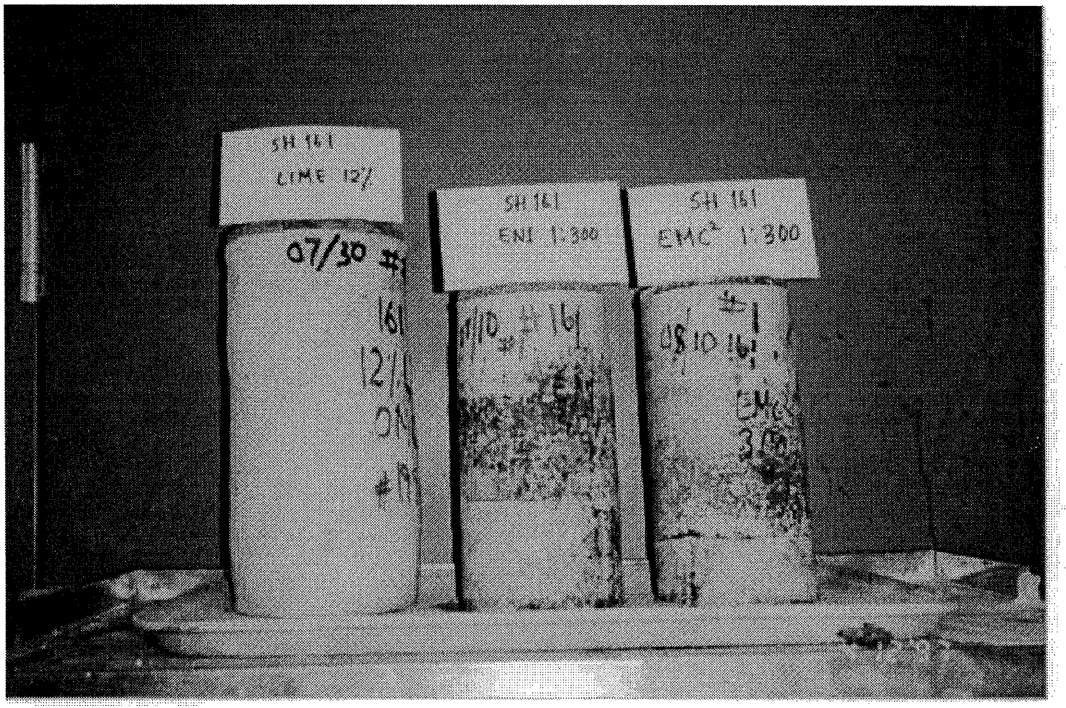


Figure 40. Photographs Showing Comparisons of Swell (Lime -12% Vs EN1-1:300, EMC^2-1:300) - SH 161

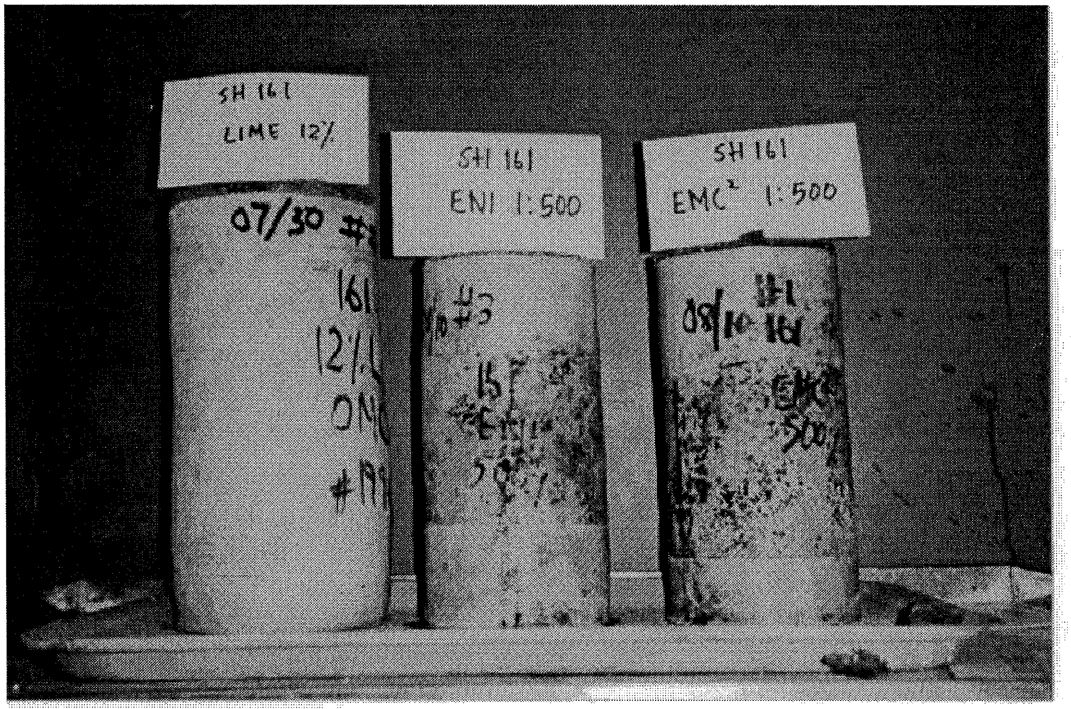


Figure 41. Photographs Showing Comparisons of Swell (Lime -12% Vs EN1-1:500, EMC^2-1:500) - SH 161

Figure 42. Photographs Showing Comparisons of Swell (Lime -6% Vs EN1-1:500, EMC^2-1:500) - SH 161

Figure 43. Photographs Showing Comparisons of Swell (Lime -3% Vs EN1-1:500, EMC^2-1:500) - SH 161

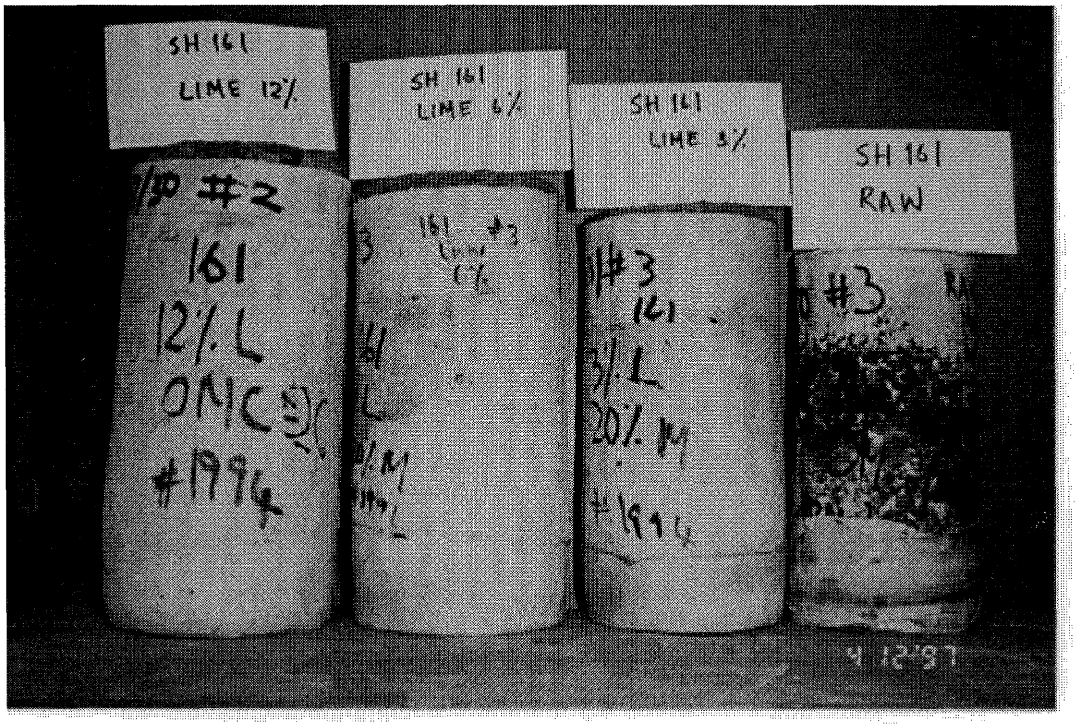


Figure 44. Photographs Showing Comparisons of Swell (Lime with Raw) - SH 161

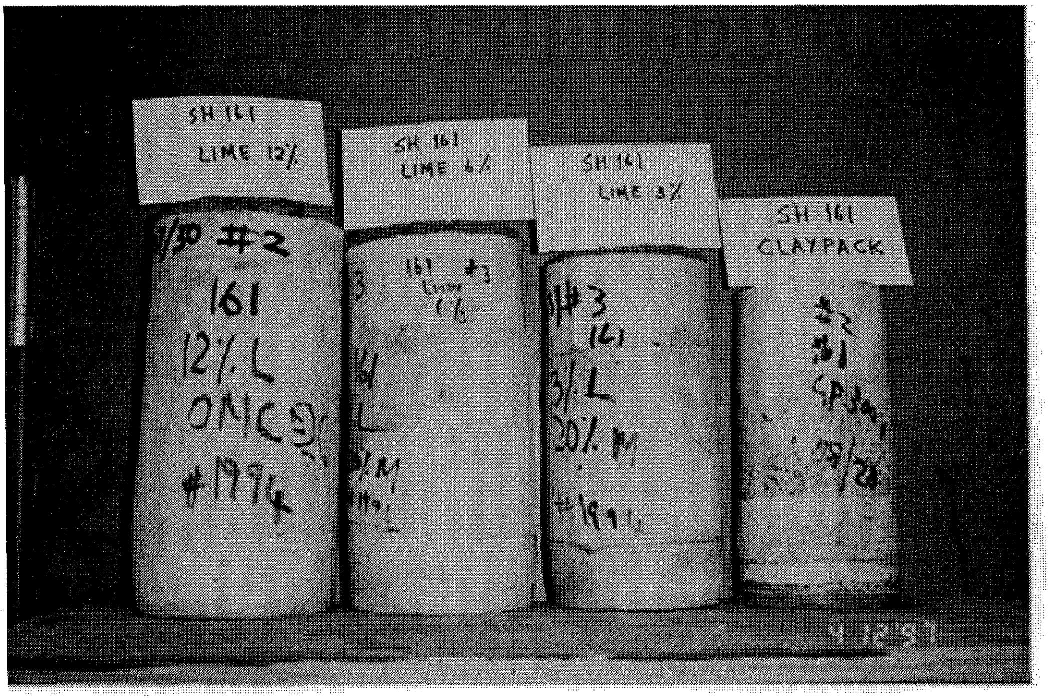


Figure 45. Photographs Showing Comparisons of Swell (Lime with ClayPack) - SH 161

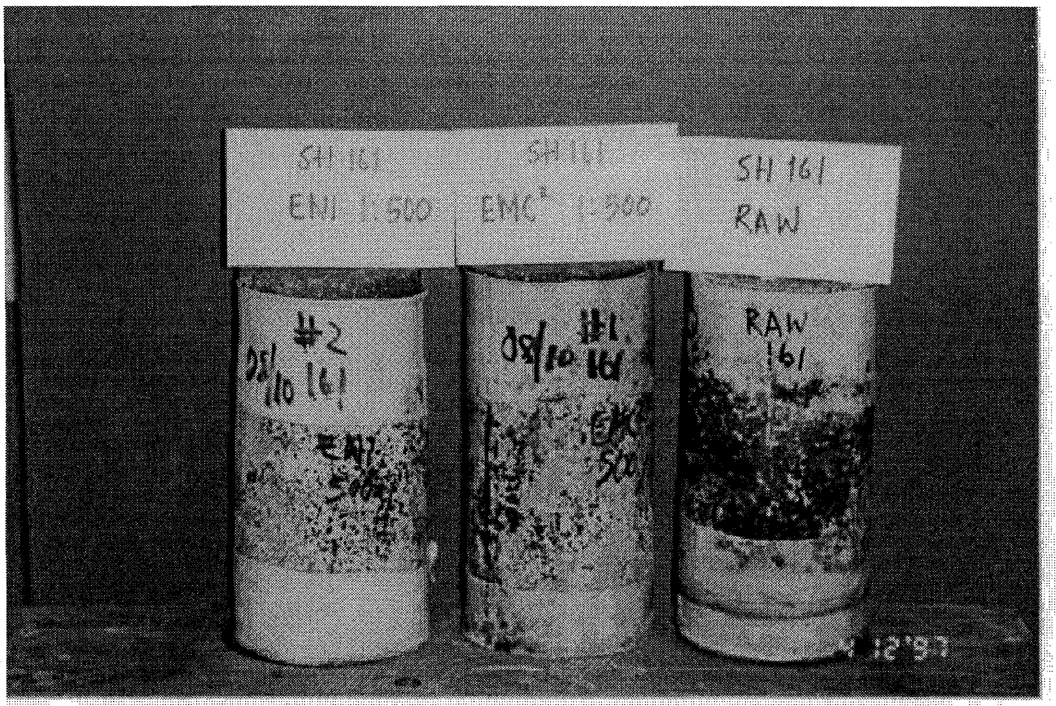


Figure 46. Photographs Showing Comparisons of Swell (EN1, EMC^2 with Raw) - SH 161

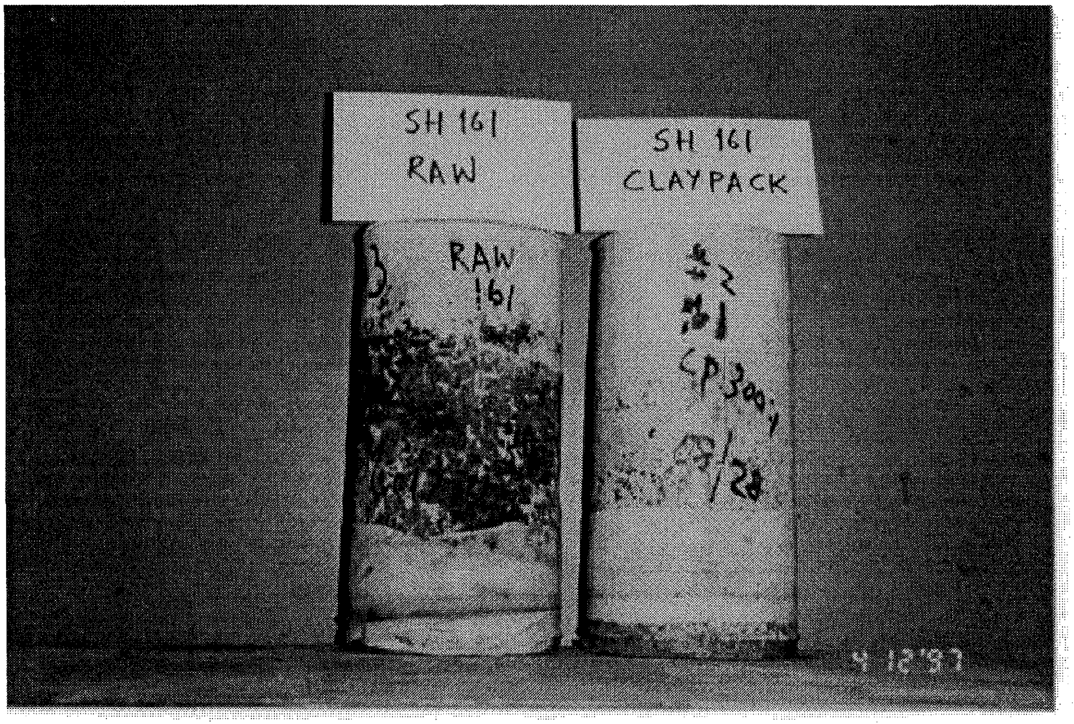


Figure 47. Photographs Showing Comparisons of Swell (Raw with ClayPack) - SH 161

Figure 48. Photographs Showing Comparisons of Swell (Lime -3% Vs EN1-1:300, EMC^2-1:300) - IH 635

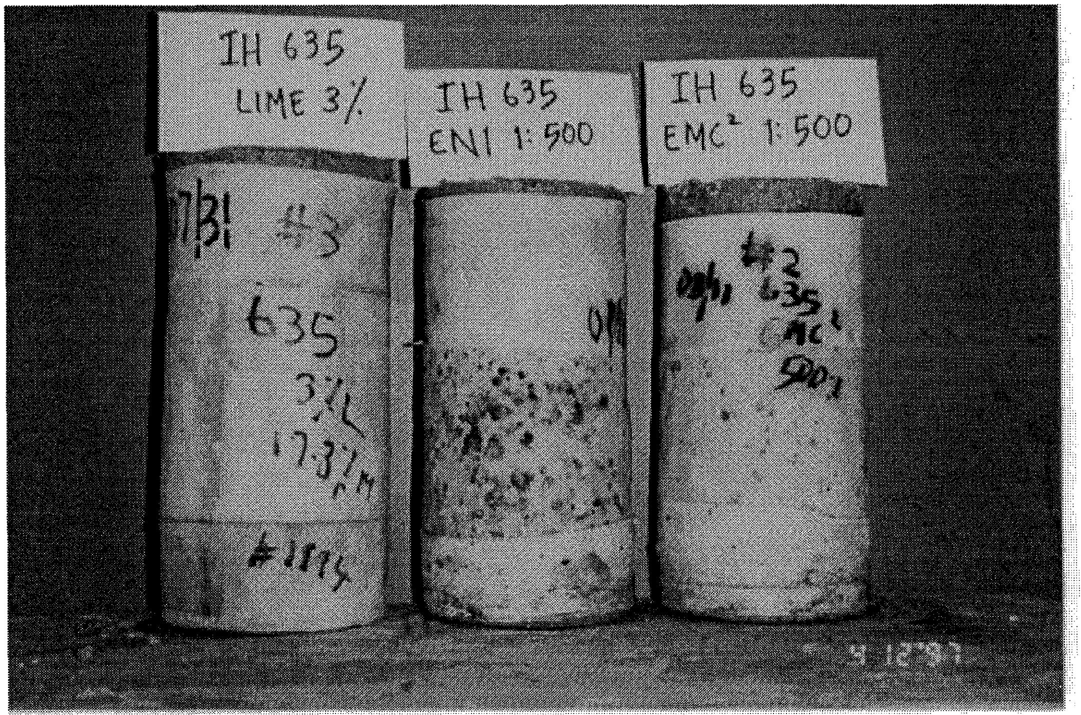


Figure 49. Photographs Showing Comparisons of Swell (Lime -3% Vs EN1-1:500, EMC^2-1:500) - IH 635

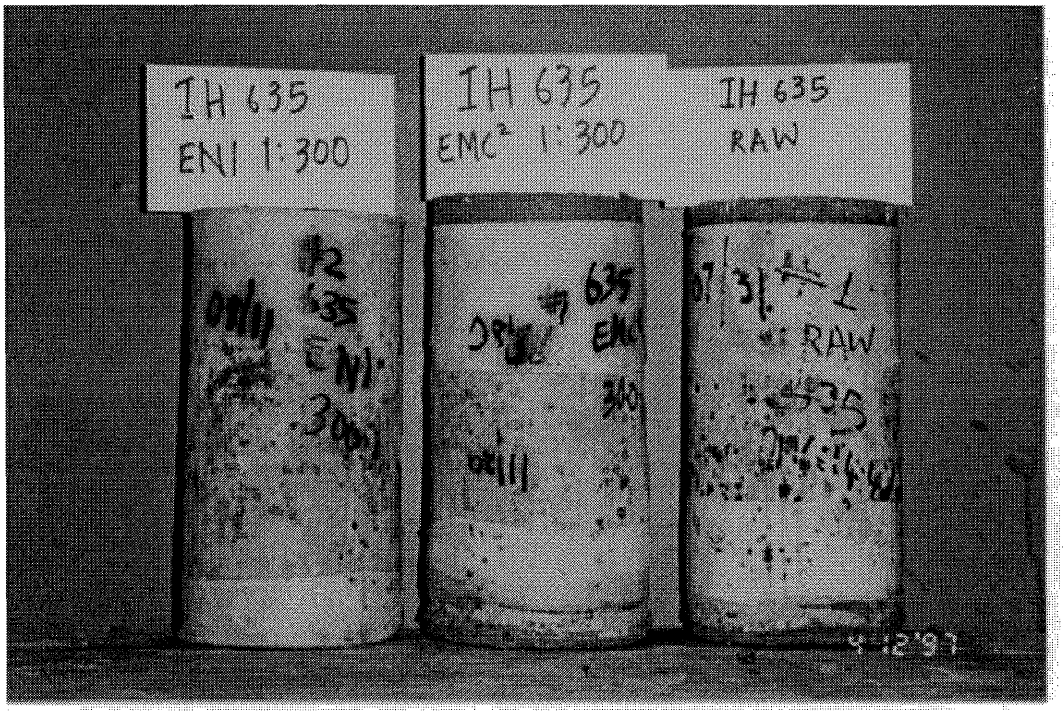


Figure 50. Photographs Showing Comparisons of Swell (Raw Vs EN1-1:300, EMC^2-1:300) - IH 635

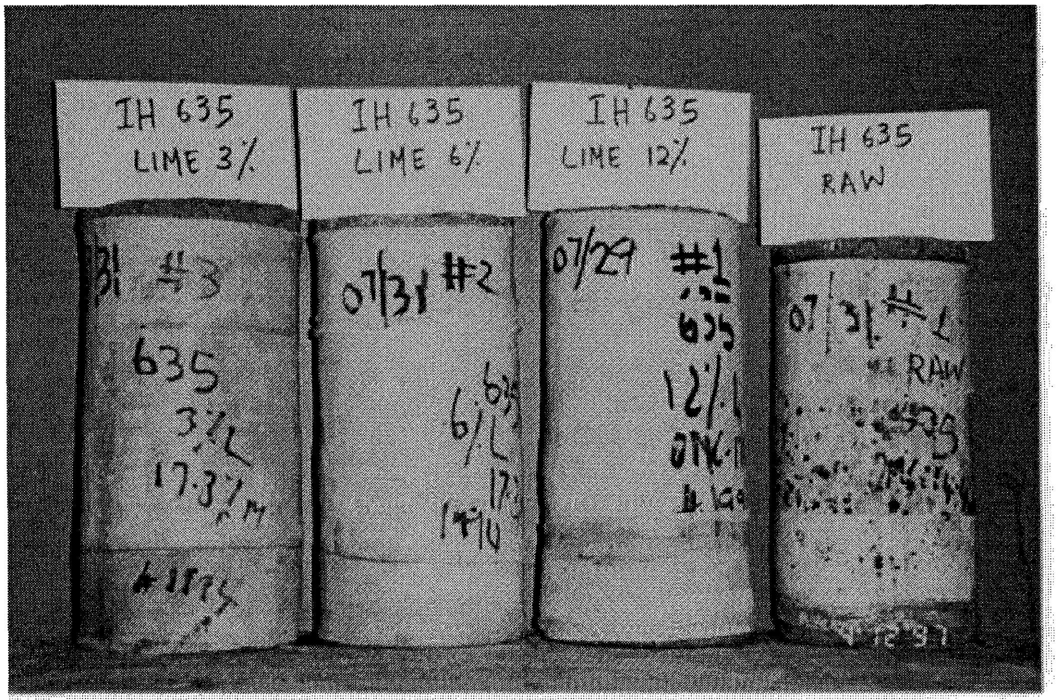


Figure 51. Photographs Showing Comparisons of Swell (Lime Vs Raw) - IH 635

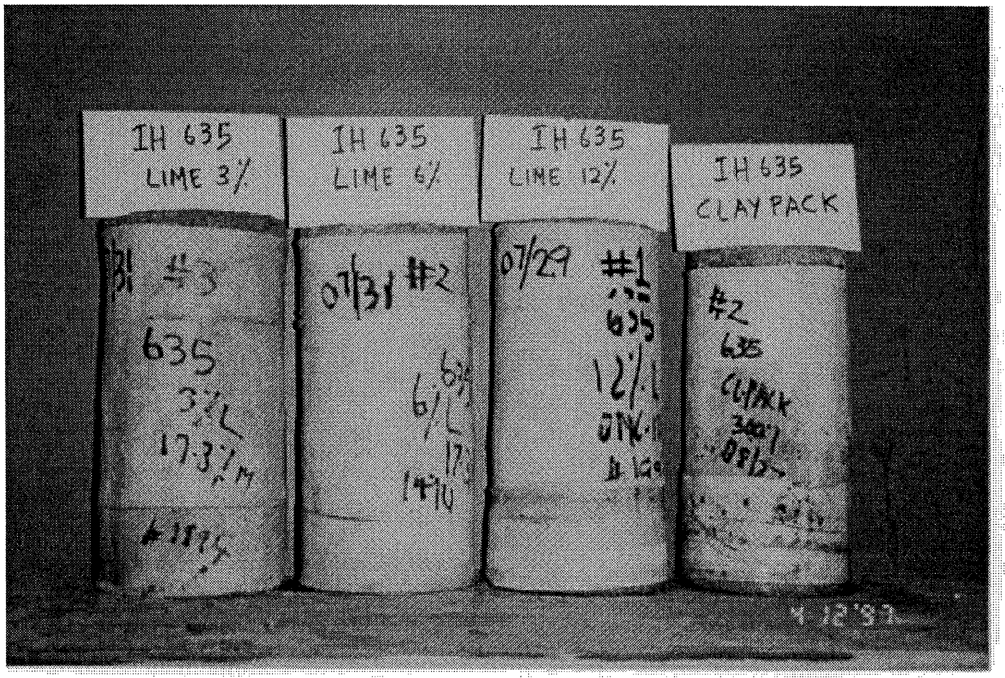


Figure 52. Photographs Showing Comparisons of Swell (Lime Vs ClayPack) - IH 635

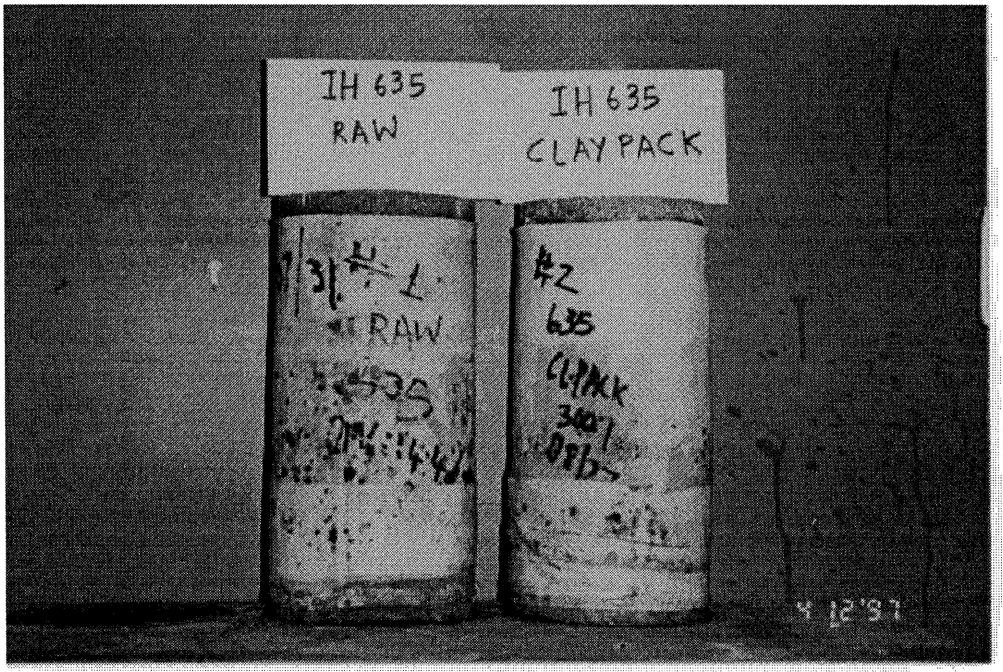


Figure 53. Photographs Showing Comparisons of Swell (Raw Vs ClayPack) - IH 635

RESULTS OF MEASUREMENTS OF ELECTRICAL CONDUCTIVITY AND DIELECTRIC CONSTANTS

Immediately after the samples were molded for the swell tests, they were measured for the electrical conductivity and dielectric constants. These properties are again measured upon the completion of swell tests. Tables 22 and 23 give the properties before and after the swell tests.

Table 22. Electrical Conductivity and Dielectric Constants Before and After the Swell Tests (SH 161)

Treated/Untreated	Before the Swell Tests		After the Swell Tests	
	EC mS/cm	DC	EC mS/cm	DC
Raw	62.50	35.83	26.67	28.00
Lime 12%	62.50	7.33	146.67	56.67
Lime 6%	60.83	37.50	89.17	37.00
Lime 3%	68.75	34.17	150.00	52.50
EN1, 1:220	55.42	4.17	31.50	24.50
EN1, 1:300	54.17	4.33	3.33	19.33
EN1, 1:500	68.75	5.33	4.67	22.67
EMC^2, 1:220	60.42	3.00	20.00	29.67
EMC^2, 1:300	46.67	2.50	16.00	27.33
EMC^2, 1:500	54.17	3.50	67.50	40.00
KOH (50g/3ℓ)	33.33	14.67	110.42	37.50
KOH(150g/3ℓ)	41.67	13.67	150.00	41.67
NH4Cl	58.33	27.00	150.00	48.33
ClayPack	70.83	8.50	4.67	23.00

Table 23. Electrical Conductivity and Dielectric Constants Before and After the Swell Tests (IH 635)

Treated/Untreated	Before the Swell Tests		After the Swell Tests	
	EC mS/cm	DC	EC mS/cm	DC
Raw	62.50	6.00	72.92	33.33
Lime 12%	52.50	6.67	114.17	38.17
Lime 6%	60.83	3.83	25.83	18.67
Lime 3%	54.58	4.33	16.67	13.50
EN1, 1:220	60.83	3.83	25.83	18.67
EN1, 1:300	54.17	15.00	110.00	40.00
EN1, 1:500	45.00	9.00	120.00	43.33
EMC^2, 1:220	52.50	17.58	100.42	36.67
EMC^2, 1:300	45.83	11.83	82.50	36.67
EMC^2, 1:500	64.58	22.67	89.58	33.33
KOH (50g/3ℓ)	38.33	20.75	103.33	33.33
KOH (150g/3ℓ)	40.00	26.92	127.08	36.67
NH_4Cl	57.50	25.33	150.00	38.67
ClayPack	64.58	10.33	29.25	22.67

As seen from table 22, the electrical conductivity and dielectric constant for all samples of SH 161 are less than 100 mS/cm before the swell test, and after the swell test, two of the three lime samples along with KOH samples, show an increase over 100 mS/cm. Also the dielectric constant of the compacted samples before the swell test is the lowest for EN1 and EMC^2 samples. From table 23, it is seen that for IH 635, the electrical conductivity of all samples is below 100 mS/cm before the swell test. But the dielectric constants and the swell properties indicated by the commercial stabilizers do not show a good correlation, though the dielectric constants of EN1 samples seem to be within the limit of "alarm values" than those

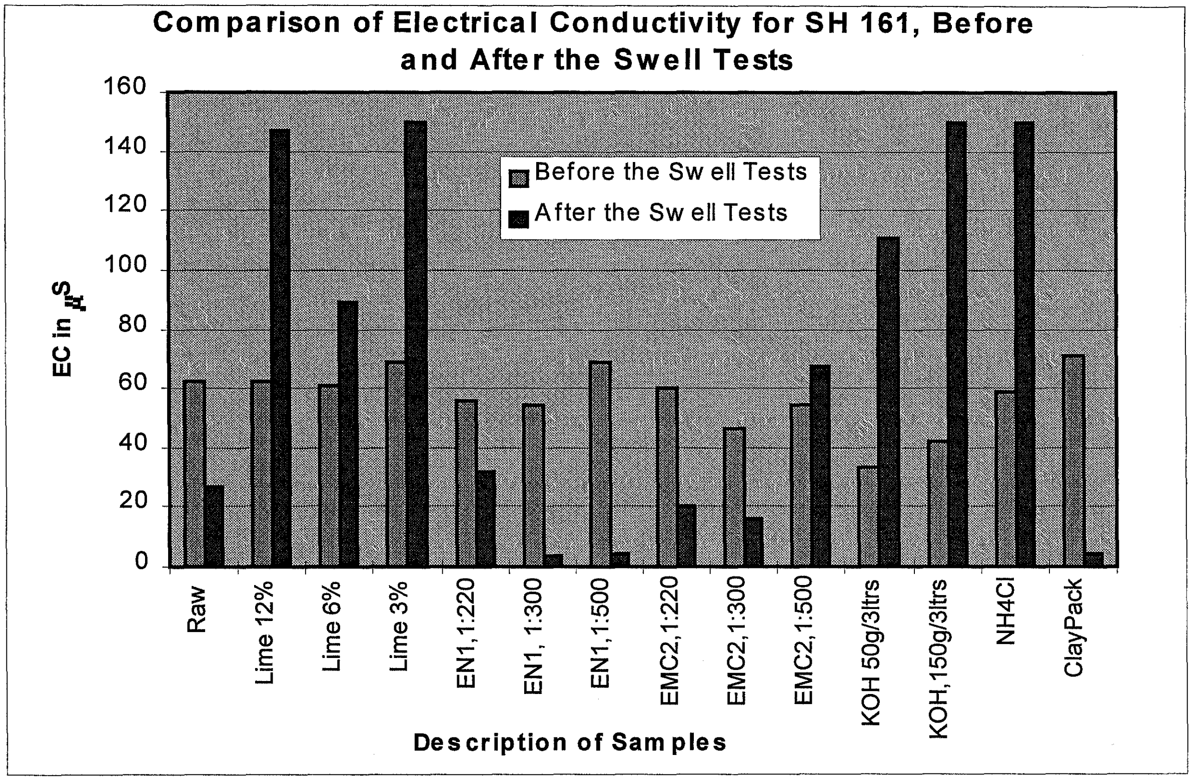


Figure 54. Comparison of Electrical Conductivity Before and After the Swell Tests - SH 161

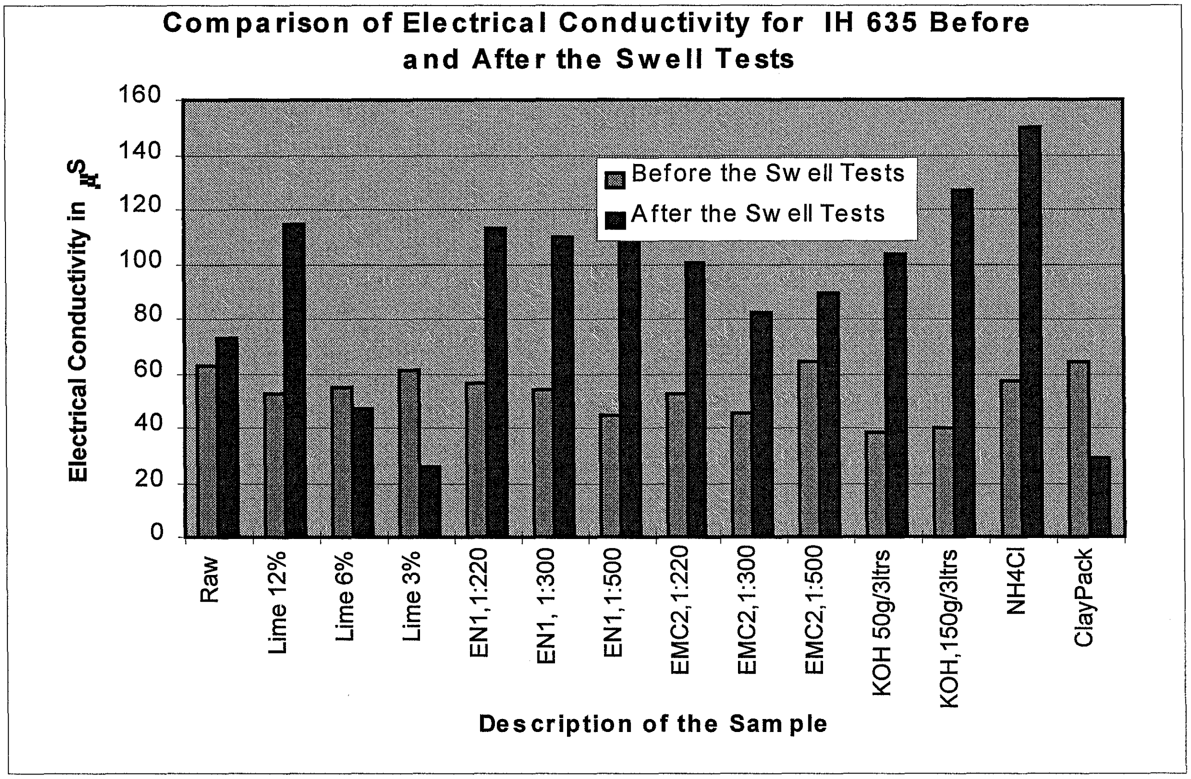


Figure 55. Comparison of Electrical Conductivity Before and After the Swell Tests - IH 635

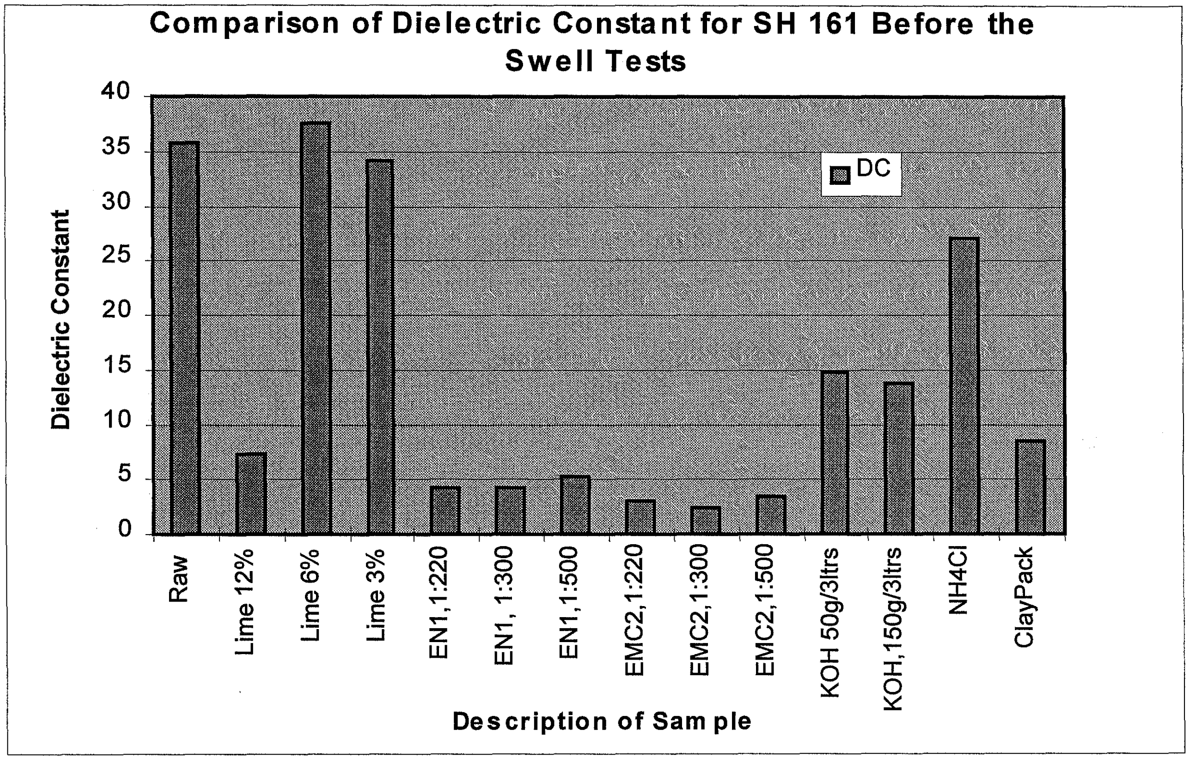


Figure 56. Comparison of Dielectric Constant Before and After the Swell Tests - SH 161

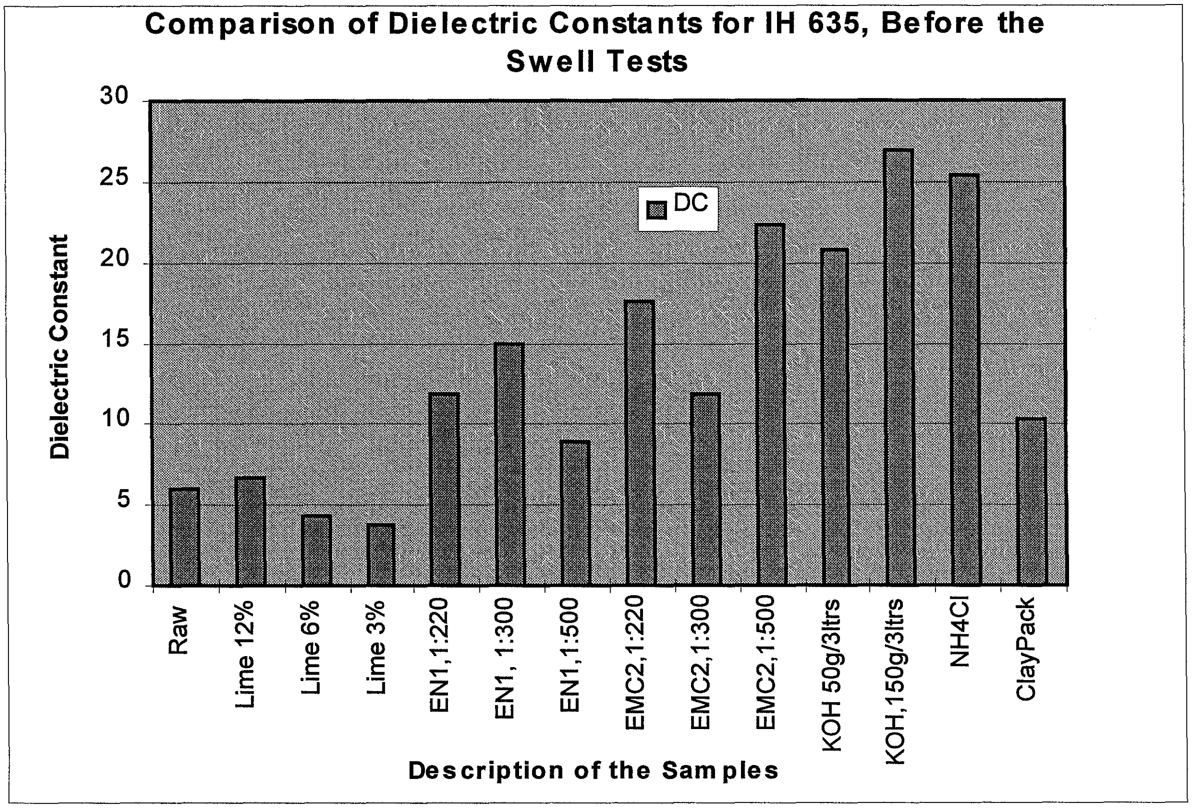


Figure 57. Comparison of Dielectric Constant Before and After the Swell Tests - IH 635

of EMC^2 samples. It is to be noted that the electrical conductivity of all EN1 samples is above 100 mS/cm after the swell tests, whereas the EMC^2 samples (1:300, 1:500) show values less than 100 mS/cm, even though their dielectric constants are higher than those of EN1 samples. Figures 54 and 55 show a plot of electrical conductivity for SH 161 and IH 635 samples. Figures 56 and 57 show a plot of dielectric constants of SH 161 and IH 635 samples.

From the results given here, it is observed that EN1 - 1:300, EMC^2 - 1:300 and EMC^2 - 1:500 are good choices as commercial stabilizers for SH 161. For IH 635, EMC^2 - 1:300 seems to be the only choice between EN1 and EMC^2, as based on the EC and DC test results.

RESULTS FROM PERMEABILITY TESTS

Based on the triaxial strength test results, the following proportions were selected for the commercial stabilizers, standard stabilizers and lime for continuing the testing program.

Table 24. Description of Samples Subjected to Permeability Tests - SH 161

Treated/Untreated	Proportion
Raw	-
NH_4Cl	25.5g/3ℓ
ClayPack	1:300
KOH	50g/3ℓ
Lime	6%
EN1	1:500
EMC^2	1:500

Table 25. Description of Samples Subjected to Permeability Tests - IH 635

Treated/Untreated	Proportion
Raw	-
NH_4Cl	25.5g/3ℓ
ClayPack	1:300
KOH	150g/3ℓ
Lime	6%
EN1	1:300
EMC^2	1:220

The permeability test results are given below in table 26, in the increasing order of permeability.

Table 26. List of Treated/Untreated Soils in Increasing Order of Permeability - SH 161

Treated/Untreated	Permeability in cm/sec	Permeability in in/mo
161 EMC^2	8.90×10^{-10}	0.00090871
161 EN1	9.47×10^{-10}	0.00096634
161 CP	1.06×10^{-9}	0.00108074
161 NH_4Cl	1.56×10^{-9}	0.00158955
161 KOH	1.96×10^{-9}	0.00204419
161 Raw	3.81×10^{-9}	0.00388341
161 Lime	5.66×10^{-9}	0.00577849

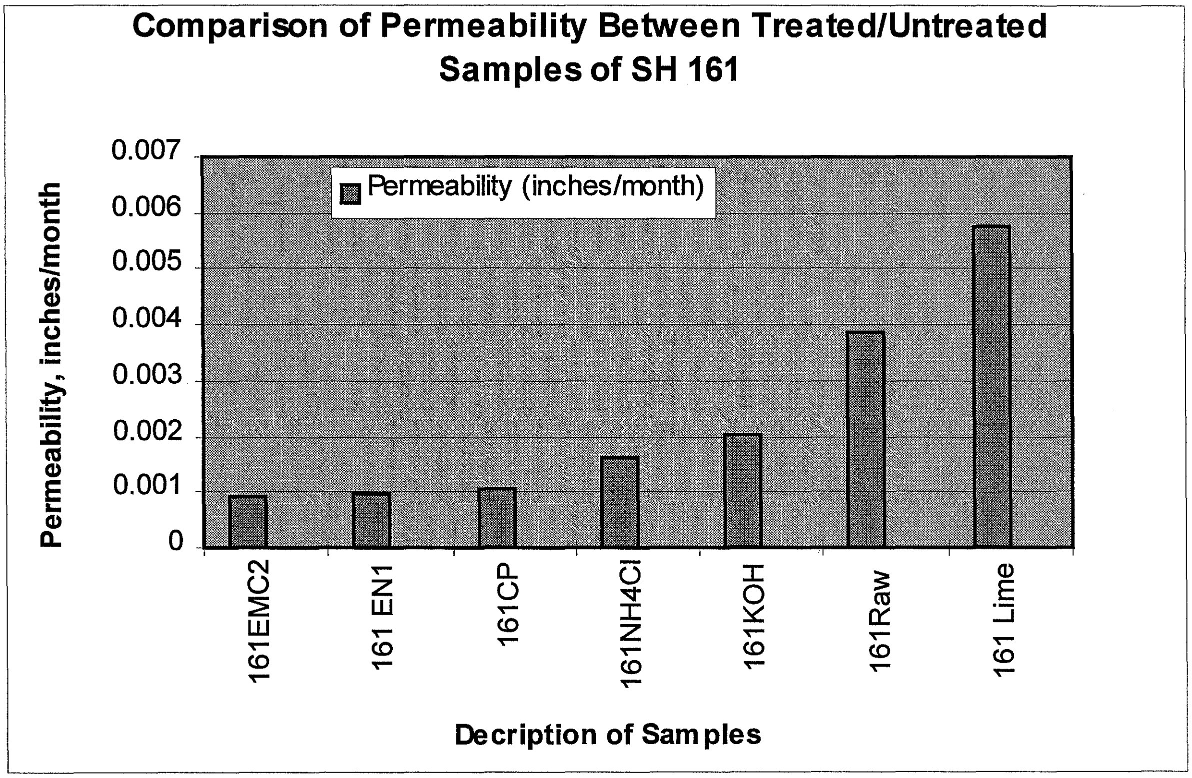


Figure 58. Comparison of Permeability Between Treated/Untreated Samples - SH 161

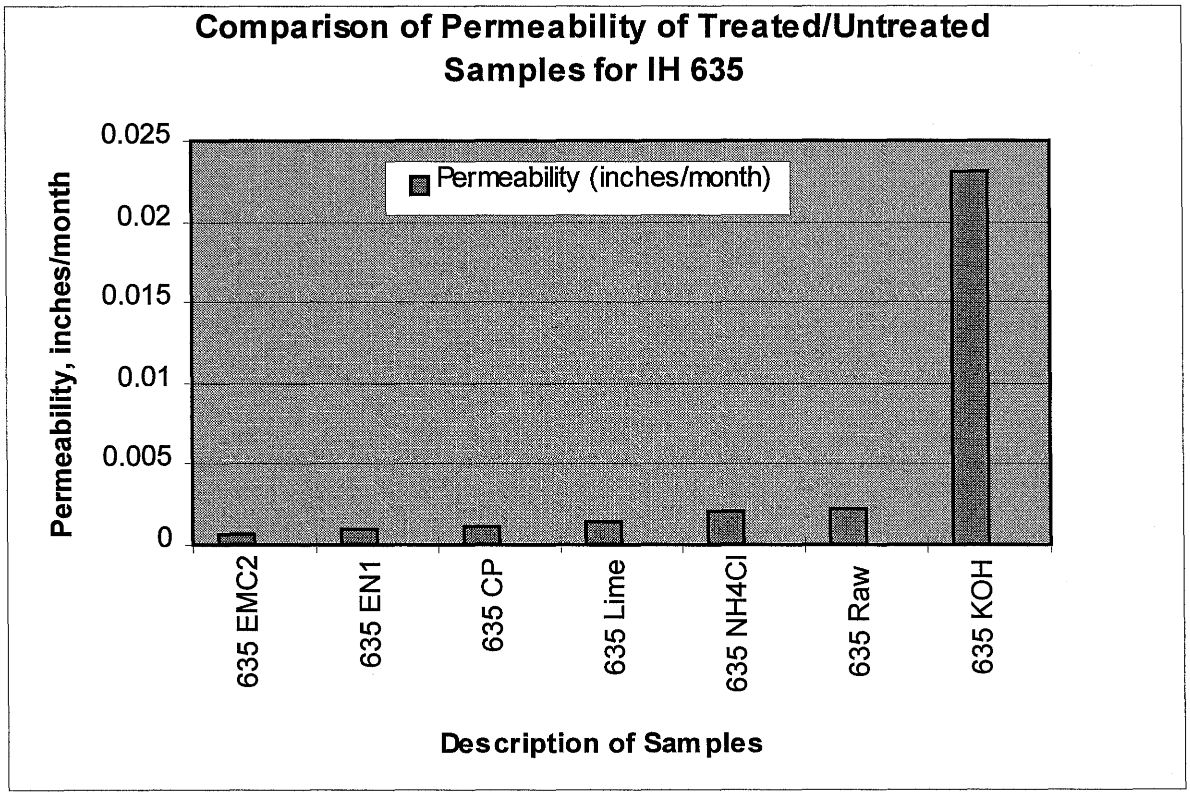


Figure 59. Comparison of Permeability Between Treated/Untreated Samples - IH 635

Table 27. List of Treated/Untreated Soils in Increasing Order of Permeability - IH 635

Treated/Untreated	Permeability in cm/sec	Permeability in in/mo
635 EMC^2	6.67×10^{-10}	6.81×10^{-4}
635 EN1	9.05×10^{-10}	9.23×10^{-4}
635 CP	1.04×10^{-09}	1.06×10^{-3}
635 Lime	1.37×10^{-09}	1.40×10^{-3}
635 NH_4Cl	2.09×10^{-09}	2.13×10^{-3}
635 Raw	2.19×10^{-09}	2.23×10^{-3}
635 KOH	2.27×10^{-08}	2.31×10^{-2}

The above test results are presented in figures 58 and 59. They compare the permeabilities of various samples of SH 161 and IH 635 respectively. From the results, it is seen that EMC^2 and EN1 give the lowest permeability. The values of permeability are very close to each other in the case of samples of 161 than for those of 635.

RESULTS FROM SUCTION MEASUREMENTS

Suction measurements were performed on all the samples subjected to the permeability test. Immediately before the molding of samples for permeability tests, (after the completion of maturing period), a small portion of soil was taken from the wetted mix and subjected to total and matric suction. After the completion of permeability tests, samples were extruded from the molds and matric and total suction tests were conducted on different sections of the samples. Total suction tests were carried out using the transistor psychrometer and matric suction using the filter paper method. The results of the tests are shown in figures 60 and 61.

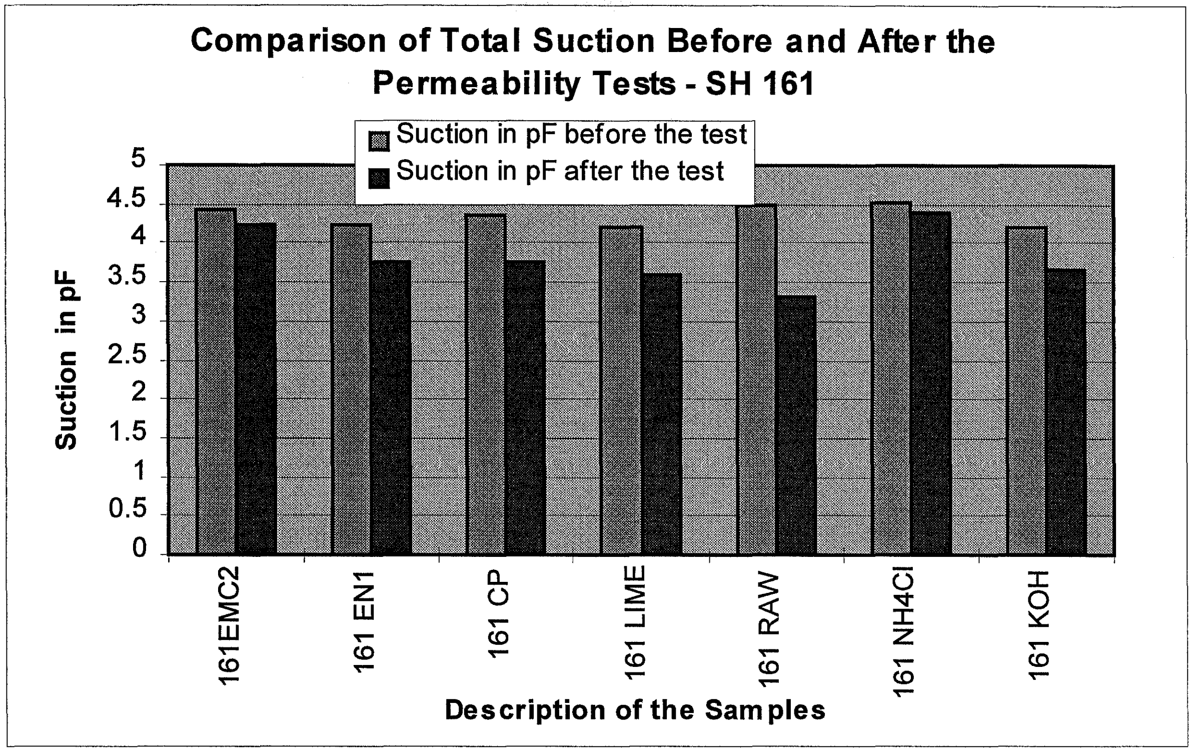


Figure 60. Comparison of Total Suction Before and After the Permeability Tests - SH 161

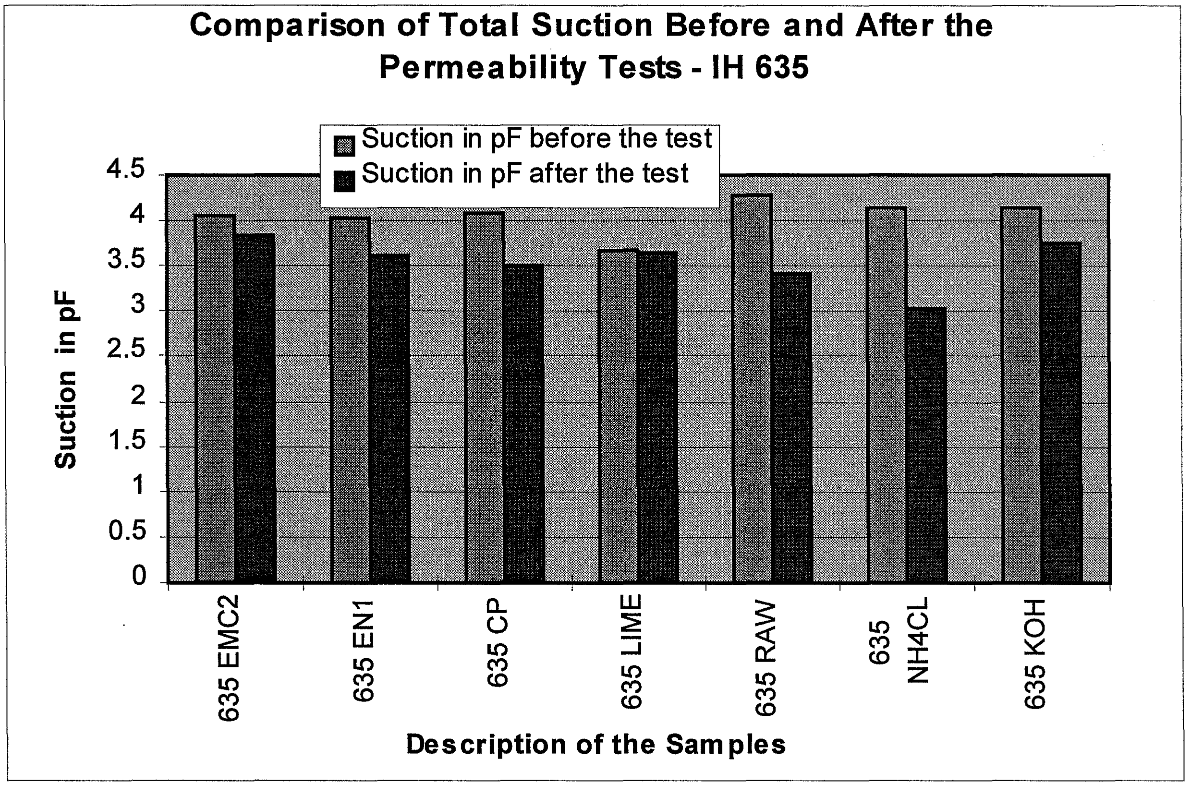


Figure 61. Comparison of Total Suction Before and After the Permeability Tests - IH 635

Table 28. Total Suction Before and After the Permeability Tests - SH 161

Treated/Untreated	Total Suction Before the Permeability Tests (pF)	Total Suction After the Permeability Tests (pF)
161 EMC^2	4.43	4.25
161 EN1	4.23	3.75
161 CP	4.35	3.76
161 Lime	4.19	3.60
161 Raw	4.49	3.32
161 NH_4Cl	4.51	4.38
161 KOH	4.19	3.66

Table 29. Total Suction Before and After the Permeability Tests - IH 635

Treated/Untreated	Total Suction Before the Permeability Tests (pF)	Total Suction After the Permeability Tests (pF)
635 EMC^2	4.05	3.83
635 EN1	4.03	3.62
635 CP	4.08	3.51
635 Lime	3.66	3.64
635 Raw	4.27	3.43
635 NH_4Cl	4.14	3.04
635 KOH	4.14	3.75

The above results are also presented in figures 60 and 61. These figures compare the total suction for SH 161 and IH 635 samples. It is seen that the least total suction is shown by EN1 and lime followed closely by ClayPack and EMC^2 in the case of SH 161. For IH 635, the least value of total suction is shown by lime followed by EN1, EMC^2 and ClayPack with very similar values. The results of matric suction are presented in tables 30 and 31.

Table 30. Matric Suction Before and After the Permeability Tests - SH 161

Treated/Untreated	Matric Suction Before the Permeability Tests (pF)	Matric Suction After the Permeability Tests (pF)
161 EMC^2	4.31	3.98
161 EN	4.13	2.82
161 CP.	4.25	2.81
161 Lime	4.00	3.14
161 Raw	4.35	2.49
161 NH_4Cl	4.41	2.94
161 KOH	3.95	2.72

Table 31. Matric Suction Before and After the Permeability Tests - IH 635

Treated/Untreated	Matric Suction Before the Permeability Tests (pF)	Matric Suction After the Permeability Tests (pF)
635 EMC^2	3.90	2.81
635 EN1	3.91	2.90
635 CP	3.95	2.71
635 Lime	3.53	3.15
635 Raw	4.05	2.91
635 NH_4Cl	4.00	2.90
635 KOH	3.91	3.05

The results are presented in figures 62 and 63. From the test results, it can be seen that the matric suction is minimum for lime in the case of SH 161 samples. The matric suction is slightly higher for EN1 samples. The matric suction after the permeability test is lower in value for EN1 than it is for lime. For SH 635, the matric suction is the lowest for lime, followed by EMC^2 and EN1.

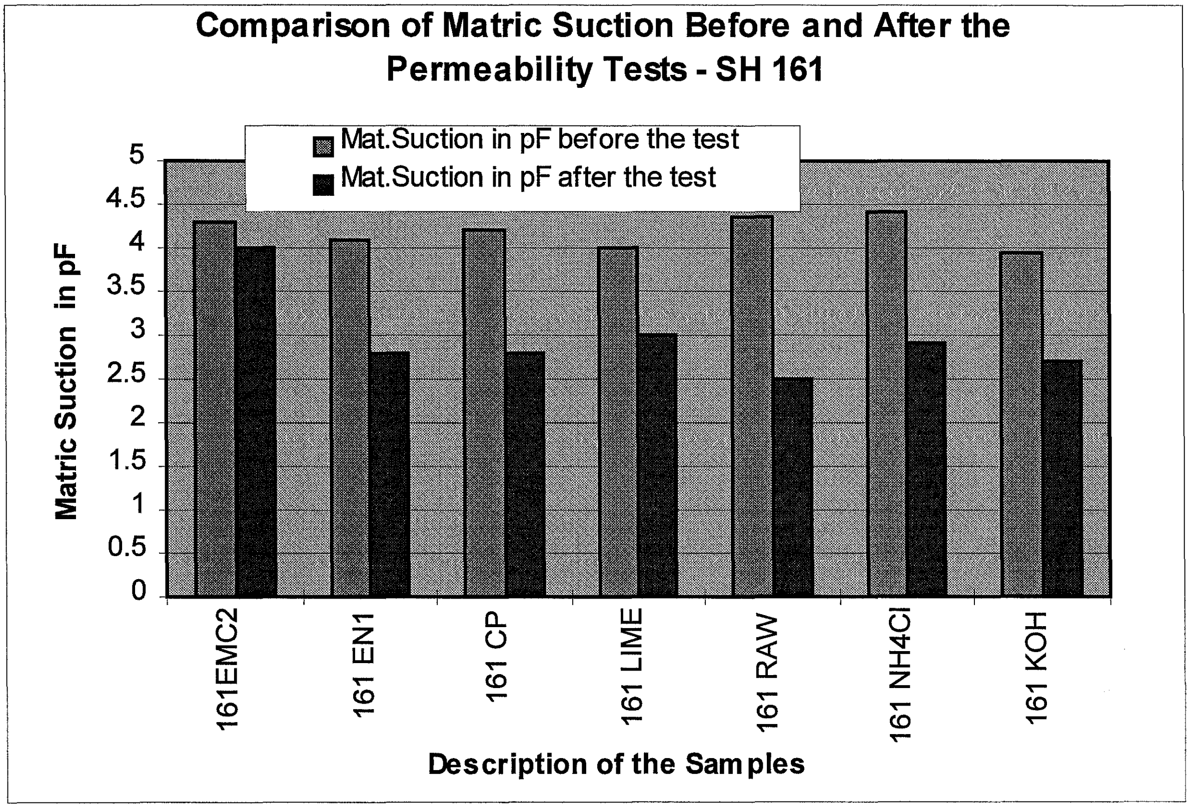


Figure 62. Comparison of Matric Suction Before and After the Permeability Tests - SH 161

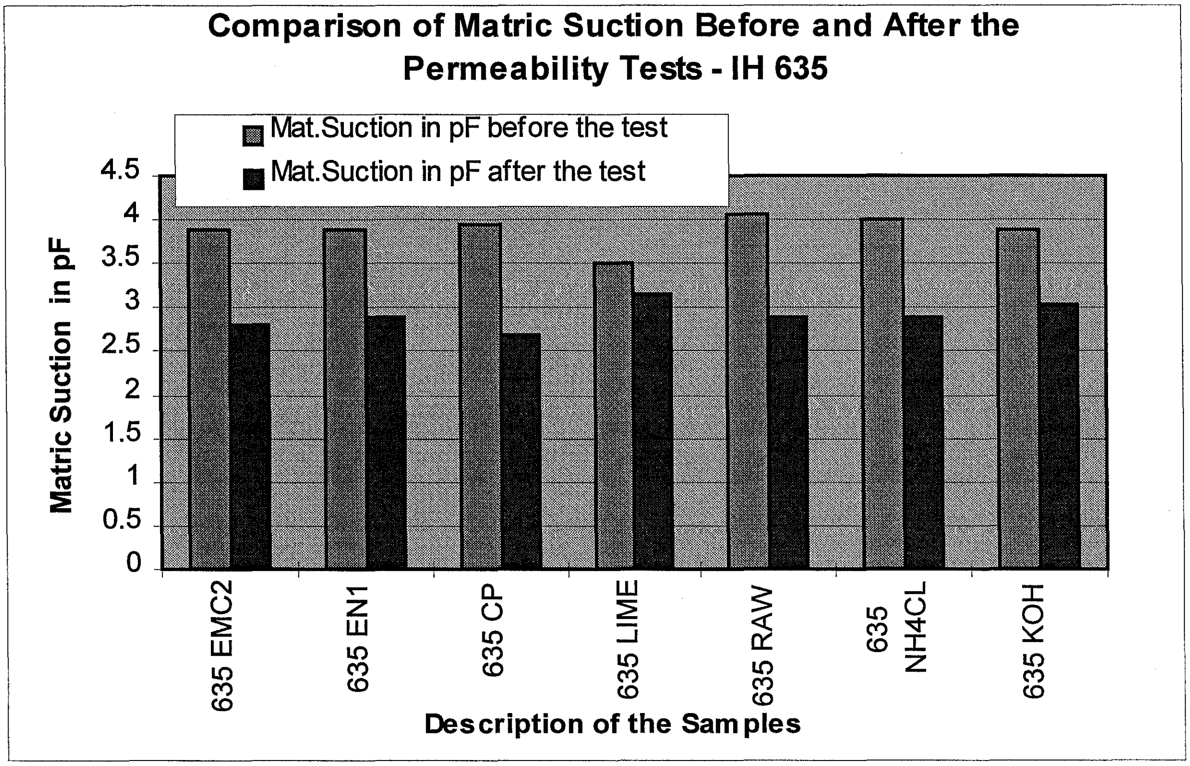


Figure 63. Comparison of Matric Suction Before and After the Permeability Tests - IH 635

In all four cases of total and matric suction, the commercial stabilizers and lime showed suction values lower than that of the compacted raw soil.

CHAPTER VII

DISCUSSION AND EVALUATION OF RESULTS

INTRODUCTION

In this chapter a general summary of the results from all the tests conducted on SH 161 and IH 635 samples is listed. Following these tables, a decision on the stabilizer to be chosen for each highway, based on the results, is reached. The concluding part of this chapter discusses the anticipated benefits and future course of action.

SUMMARY OF RESULTS

The results from the tests are presented in a tabular form for each highway. The results given by the top five performers for each test are ranked in a decreasing order of preference from 1 to 5, those ranked 1 are superior to those rated 2, 3 or 4. The following legend can be associated with the two tables:

Rank 1 - Excellent (Strength at Failure), Lowest (Swell Potential, EC & DC, Permeability & Suction),
Rank 2 - Good (Strength at Failure), Lower (Swell Potential, EC&DC, Permeability & Suction),
Rank 3 - Fair (Strength at Failure), Low (all other properties), and
Rank 4 - Average (all properties).

The ranking criteria is based entirely on the values of strength at failure, and those stabilizers (or proportions) that give the top four values are selected and their results on other tests are compared among themselves except for permeability and suction tests and ranking is given accordingly.

Table 32. Test Results from SH 161

	Ranking			
Tests Conducted	1	2	3	4
Strength at Failure	EN1-1:500	EMC^2-1:500	EN1-1:300	Lime - 6%
Stiffness	EN1-1:500	EMC^2-1:500	EN1-1:300	Lime - 6%
Swell Test	EN1-1:500	EN1-1:300	EMC^2-1:500	Lime - 6%
Electrical Conductivity	EN1 - 1:300	EMC^2-1:500	Lime - 6%	EN1 - 1:500
Dielectric Constant	EN1 - 1:500	EN1-1:300	EMC^2-1:500	Lime - 6%
Permeability	EMC^2-1:500	EN1-1:500	ClayPack	NH_4Cl
Total Suction	Lime - 6%	EN1-1:500	ClayPack	EMC^2 - 1:500
Matric Suction	Lime -6%	KOH-50g/3ltrs	EN1-1:500	ClayPack

Table 33. Test Results from IH 635

	Ranking			
Tests Conducted	1	2	3	4
Strength at Failure	EMC^2- 1:220	Lime - 6%	EN1 -1:300	Lime - 12%
Stiffness	EMC^2- 1:220	EN1 -1:300	Lime - 6%	Lime - 12%
Swell Test	EN1 1:300	EMC^2 1:220	Lime - 12%	Lime - 6%
Electrical Conductivity	Lime - 6%	EMC^2 1:220	Lime - 12%	EN1 1:300
Dielectric Constant	Lime - 12%	Lime - 6%	EN1 1:300	EMC^2 1:220
Permeability	EMC^2 1:220	EN1 1:300	ClayPack	Lime - 6%
Total Suction	Lime - 6%	EMC^2 1:220	EN1 1:300	ClayPack
Matric Suction	Lime - 6%	EMC^2 1:220	EN1-1:300	KOH-50g/3ℓ

From table 32, based on the ranking given, it is found that EN1@1:500 has the maximum triaxial strength at failure. Following EN1@1:500, EMC^2@1:500, EN1@1:300 and lime @6% have the next higher values of strength at failure. From a comparison of the remaining properties, it can be seen that EN1@1:500 has the least swell potential, though its values of permeability and total suction are ranked second to that of EMC^2 and lime, respectively.

From a comparison of rankings given in table 33, it is found that EMC2@1:220 shows the maximum stiffness and strength at failure. Its value of permeability is the lowest, though it is ranked second to EN1 in terms of swell resistance potential and to lime regarding electrical conductivity and suction values.

DISCUSSION OF RESULTS

A summary table was created which represents the tested properties and their quality as shown by each of the three stabilizers.

Table 34. Summary Table for SH 161

	EN1 - 1:500	EMC^2 - 1:500	Lime - 6%
Strength at Failure	1	2	4
Stiffness	1	1	2
Swell Potential	1	3	4
Electrical Conductivity	4	2	3
Dielectric Constant	1	3	4
Permeability	2	1	>4
Total Suction	2	4	1
Matric Suction	3	>4	1

Table 35. Summary Table for IH 635

	EMC^2 - 1:220	EN1 - 1:300	Lime - 6%
Strength at Failure	1	3	2
Stiffness	1	2	3
Swell Potential	2	1	4
Electrical Conductivity	2	4	1
Dielectric Constant	4	3	1
Permeability	1	2	4
Total Suction	2	3	1
Matric Suction	2	3	1

What the researchers hope to find from studying these tests is an additive that will do the following:

- Increase the strength of the soil above that of the natural soil so as to form a working table for construction traffic,
- Reduce the swelling of the treated soil below that of the natural soil and eliminate the lime-sulfate interaction, if possible,
- Decrease the permeability of the treated soil below that of the natural soil so as to reduce the amount of water that will penetrate into the natural subgrade from the surface,
- Decrease the suction of the treated soil below that of the natural soil so the treated layer will not attract water from below,
- Limit the electrical conductivity of the treated soil to no more than 100 mS/cm, and
- Limit the dielectric constant to values between 5 and 12.

From the two summary tables (34 and 35), it can be seen that :

- For SH 161, of the three stabilizers considered, based on the values of strength at failure, EN1 @1:500 shows superior qualities than EMC^2 or lime, and
- For IH 635, the most superior qualities are shown by EMC^2@1:220 as compared to EN1@1:300 or Lime (6%).

CHAPTER VIII

RECOMMENDATIONS AND IMPLEMENTATION

To fully appreciate the observations made and test results received from this study, a review of the objectives of the study will be highly helpful. The objectives are:

- To standardize the electrical conductivity test, and
- To investigate the availability and use of alternate (possibly non-calcium) stabilizers that can be applied to sulfate rich soils in place of lime, so that the swell problem associated with lime stabilization can be obviated.

The activities performed to accomplish these objectives were discussed in the previous chapter and they can be summarized as follows.

TO ACHIEVE OBJECTIVE 1

A list of places where heaving was observed and suspected to be lime-induced was compiled and four of the sites were selected to be investigated in this study. Field Trips were made to these locations and soil samples were collected from the unstabilized shoulders of the pavements. Electrical conductivity tests were performed on the soil solutions using a hand held conductivity meter and based on the test results, samples from certain borings were selected and sent to a commercial agricultural soils testing lab for soluble sulfate determination. The soluble sulfate contents and the electrical conductivity values correlated fairly well. Samples with relatively high values of electrical conductivities showed fairly high values of soluble sulfate contents. Hence, the electrical conductivity measurement using the hand held conductivity meter can be used as a diagnostic test towards identifying the presence and extent of soluble sulfates in potential lime-induced sulfate heave areas.

TO ACHIEVE OBJECTIVE 2

Based on the results from electrical conductivity tests and soluble sulfate contents, two sites were selected which showed relatively high values. A laboratory test program was then designed which aimed to test the soils from locations SH 161 and IH 635 with commercially available stabilizers. Also, the soils were tested with lime and other standard stabilizers. Tests were also performed on the untreated (raw) soils. Discussion of the test results from the laboratory test program is given in chapters 6 and 7. The results are summarized as below. Of the properties tested, triaxial strength at failure was taken as the most important criteria.

SH 161

- All the treated samples showed failure strengths @ zero confining pressures greater than those of untreated samples except for samples treated with Claypack and ammonium chloride.
- The maximum strength at failure was given by EN1@1:500.
- The stiffness values of treated samples especially those of EN1 and EMC^2 are higher than those of lime treated samples except for two samples (EN1@1:220 and Lime-12%).
- The Claypack treated samples give the least values of stiffness.
- Of all the samples, lime treated samples gave the maximum swell.
- EN1 and EMC^2 treated samples showed superior swell resisting properties, EN1@1:500 giving the least swell next to Claypack.
- The electrical conductivity values of all samples before and after the swell tests were well below 100 mS/cm.
- The electrical conductivity values shown by lime and EN1 and EMC^2 treated samples were almost in the same range.
- The dielectric constant values of lime treated samples were greater than the "alarm" values of 11 and this suggests that the lime treated samples are susceptible to plastic deformation.

- The dielectric constant values of EN1 and EMC^2 treated samples were well below 10, the most desirable value being given by EN1 @ 1:500, a value of 5.33.
- The highest value of permeability was given by lime treated samples, which makes it the least water tight of all samples.
- The least value of permeability is shown by EMC^2 and EN1 treated samples, the latter being slightly higher.
- The suction values given by lime and EN1 are comparable and the least of all the samples.

IH 635

- All samples treated with commercial stabilizers and lime give a higher value of strength at failure than that of raw samples except for samples treated with Claypack.
- At zero confining pressure, EMC^2 @1:220 gave the maximum strength at failure.
- EMC^2 @ 1:220 has a higher stiffness at zero confining pressure than lime.
- The highest value is shown by EMC^2@1:500 and EN1@1:500.
- Of all the samples, the maximum swell is given by lime treated samples.
- The least swell is given by EN1@1:300.
- Both EMC^2 and EN1 treated samples show superior swell resistant properties as compared to lime.
- The electrical conductivity values of all samples, both treated and untreated are less than 100 mS/cm.
- The electrical conductivity values shown by the lime treated samples and EN1 and EMC^2 treated samples are in the same range, with those of EN1 and EMC^2 slightly lower.
- Lime showed superior dielectric properties as compared to EN1 and EMC^2 treated samples.
- The lowest value of permeability is given by EMC^2 sample.
- The value of permeability shown by lime is higher than the values given by any of the commercial stabilizers.

- The lowest value of suction is shown by lime treated sample, followed by those treated with the three commercial stabilizers, which are well below those of untreated samples.

CONCLUSIONS

From the above mentioned observations, it is clear that, the maximum strength at failure, superior values of stiffness, swell resistance and permeability are given by EN1@1:500 for the SH 161 soil and EMC^2 @ 1:220 for the IH 635 soil. These stabilizers also impart reasonably good dielectric and electrical conductivity properties. The stabilizers also reduced the suction of the treated soil below that of the natural soil.

The question may arise concerning the permanence of the treatments with these successful, non-calcium stabilizers. Lime treatment is permanent but carries with it the high expansion and roughness due to a lime-sulfate reaction. No determination of the permanence of any of the treatments was attempted in this study partly because of the limited time of the study and partly because it was considered unnecessary. As noted in the Introduction (Chapter I), the main reason for stabilizing the upper 150-200 mm (6-12 in) of subgrade soil is to provide a working table for construction equipment during wet weather. It is not and should not be used in the hope of reducing expansive clay roughness. The need for a wet weather-working table is temporary, limited to the time of construction, and so there is no need for a subgrade to be permanently stabilized unless it is expected to contribute a part of the pavement structure.

This study has established the superiority of the commercial stabilizers EMC^2 and EN1 to lime in strength, stiffness, swell resistance and permeability. The stabilized subgrade has a lower permeability and a lower suction than the untreated soil below it. This means that it will shed rain water and not soak up water from the soil below it, and any water that enters the stabilized layer will move much more slowly than it would in the untreated soil. This assures that the stabilized soil will serve well as a working table during construction and will not carry with it the hazard of subsequent expansion due to a lime-sulfate reaction.

Another study will be required to determine the structural permanence of soils stabilized with these non-calcium treatments.

From the experiments carried out in this study, the following tests can give valuable information regarding the properties of the treated soil.

1. Electrical Conductivity and Dielectric Constant

   These properties are measured of the samples molded at their OMC's. Electrical Conductivity values below 100 mS/cm and dielectric constants between 5 and 11 indicate that the treated soil can adequately resist plastic deformation and can maintain sufficient water tightness. The test is quick. This test, however, does not identify the best stabilizer for the treated soil.

2. Triaxial Strength Test

   The properties measured are strength at failure and stiffness. The molded samples are tested after the specified curing period so that the samples gain adequate strength. Testing can be done only after curing. The curing period for the stabilizers studied here varied from two days (EN1, EMC^2) to seven days (Lime). The duration of the test is about 15 to 20 minutes. This test conclusively establishes the stabilizer best suited to the soil under investigation.

3. Swell Test and Permeability Test.

   These tests also give conclusive results, based on the % swell and the permeability values. The test setup is not as expensive as Triaxial Test Equipments. However, these tests require a longer duration and long-term monitoring of about three to four months and should not be attempted unless there is not a severe time crunch.

RECOMMENDATIONS

The results of the study point towards the use of two commercial stabilizers that can be used in lieu of lime in lime-sulfate heave areas. These stabilizers have shown to be superior to lime in terms of strength, stiffness, permeability and swell resistance potential.

Also recommended is the use of triaxial testing as a conclusive method to determine the stabilizer best suited to the soil under study. An automated triaxial test serves the purpose best. The electrical conductivity and dielectric constant test is a rapid indicator of whether a soil contains soluble sulfates and is a candidate for the use of the non-calcium stabilizers identified in this study.

IMPLEMENTATION

Two of the proprietary additives tested appear to stabilize the soil, making it more impervious, increasing its strength and stiffness and reducing its swell without triggering the expansive lime-sulfate reaction. Also, the treated soil renders fairly low electrical conductivity, dielectric constant and suction thus showing a consistent trend towards achieving the objectives specified in chapter I.

Implementation will probably require some test sections in the Dallas district (or any others that are interested) in which actual construction with these two additives has been tried and monitored. The manufacturers of the commercial stabilizers should be contacted for information on to how the mixing and curing processes are carried out on the field. The manufacturers should be involved in planning the test sections from the beginning. Future testing and monitoring can be carried out in the same lines as specified in this study. The soil from the location under investigation should be tested for its electrical conductivity and sulfate content. Either a detailed test program or a single conclusive test (i.e., triaxial test) can be set up depending on the availability of time and resources.

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APPENDIX A

Corrected Charts of Percentage Strain Vs (Percent Strain /Corrected Vertical Stress) for both SH 161 and IH 635

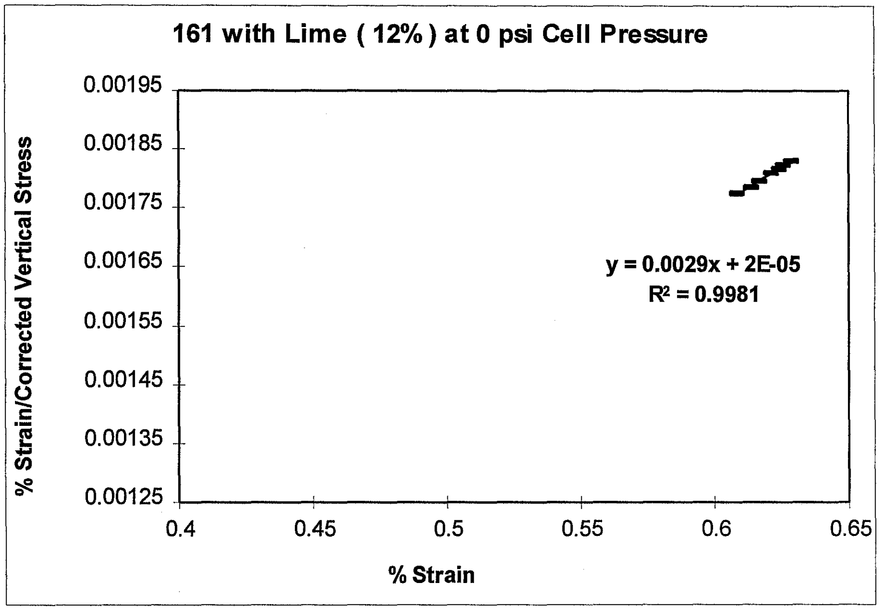

Figure 1. Modified Plot for Lime (12%) at 0 psi Cell Pressure (SH 161)

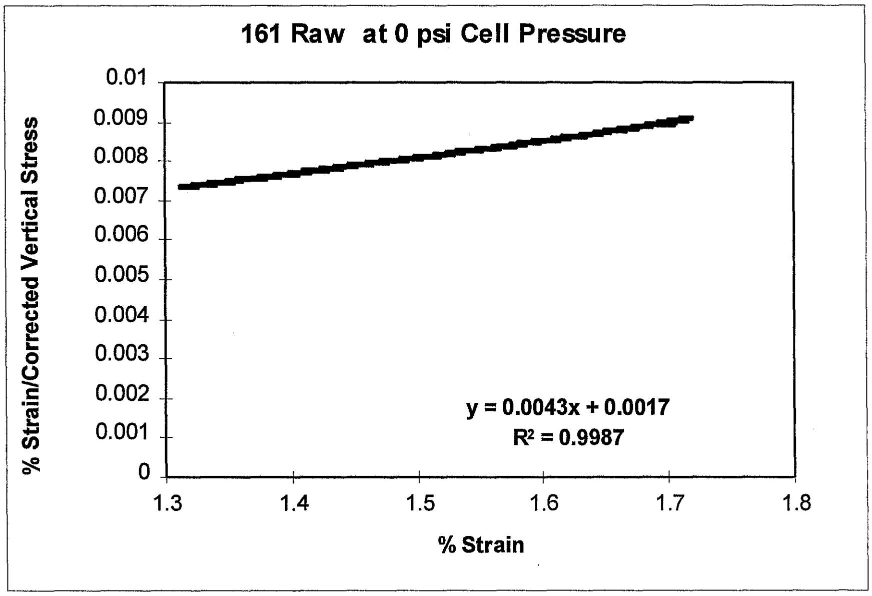

Figure 2. Modified Plot for Raw at 0 psi Cell Pressure (SH 161)

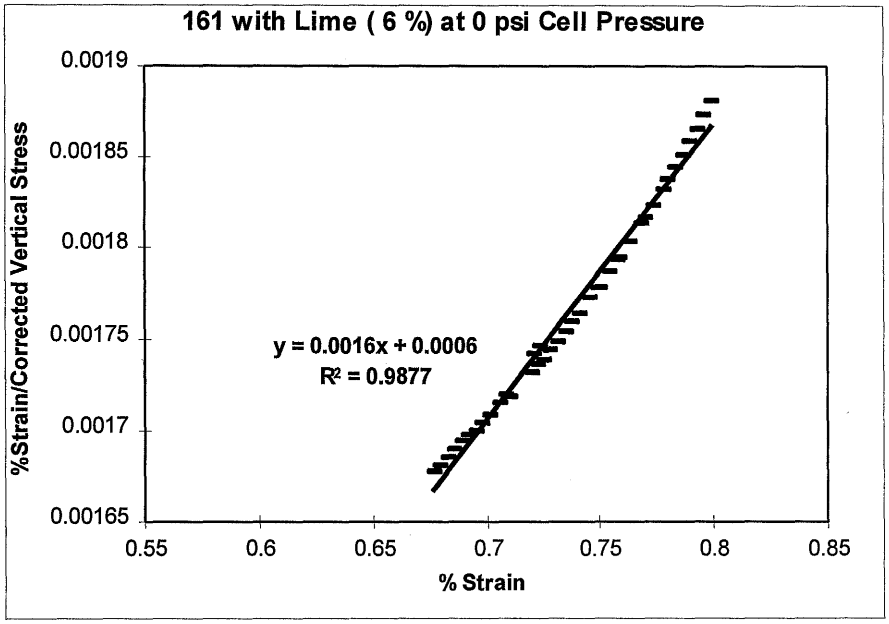

Figure 3. Modified Plot for Lime (6%) at 0 psi Cell Pressure (SH 161)

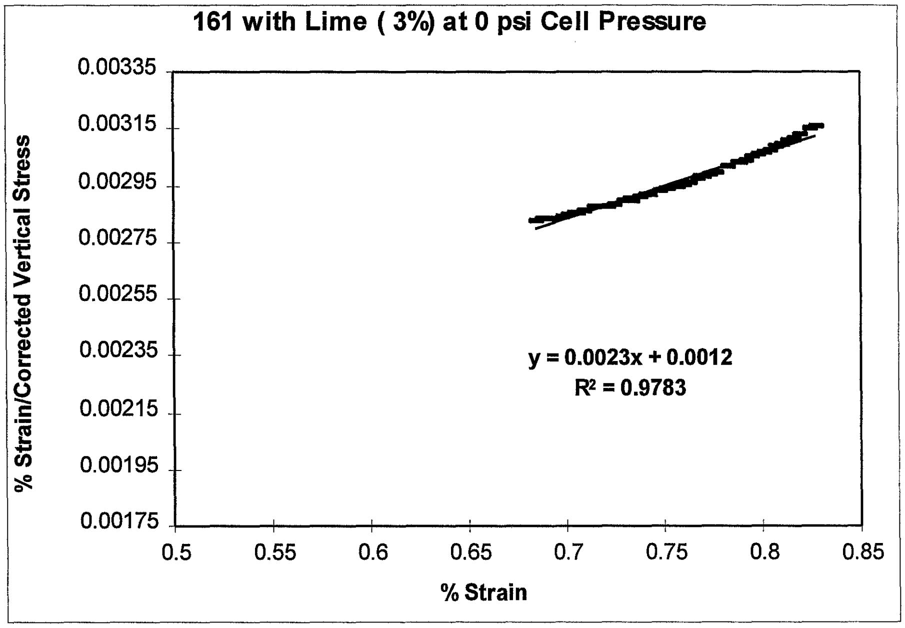

Figure 4. Modified Plot for Lime (3%) at 0 psi Cell Pressure (SH 161)

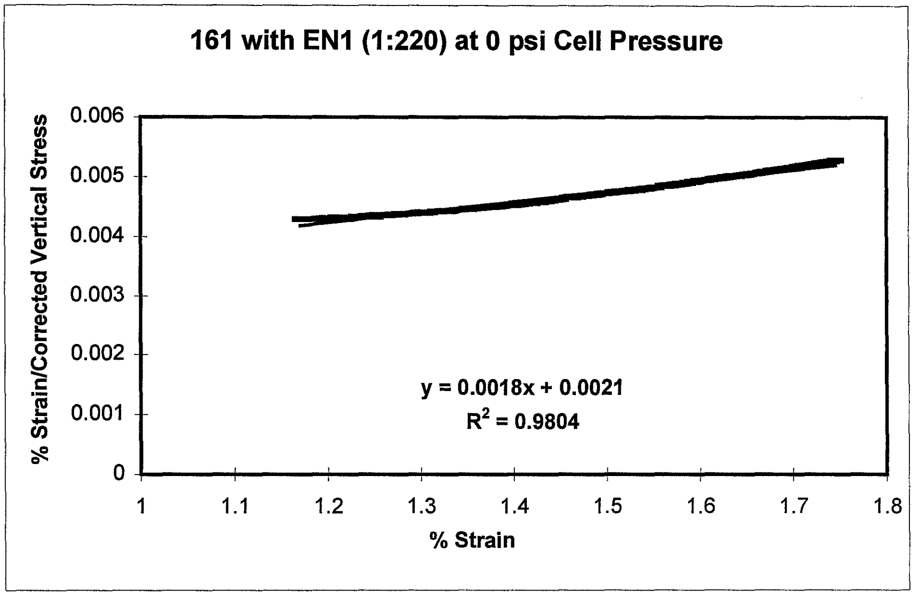

Figure 5. Modified Plot for EN1 (1:220) at 0 psi Cell Pressure (SH 161)

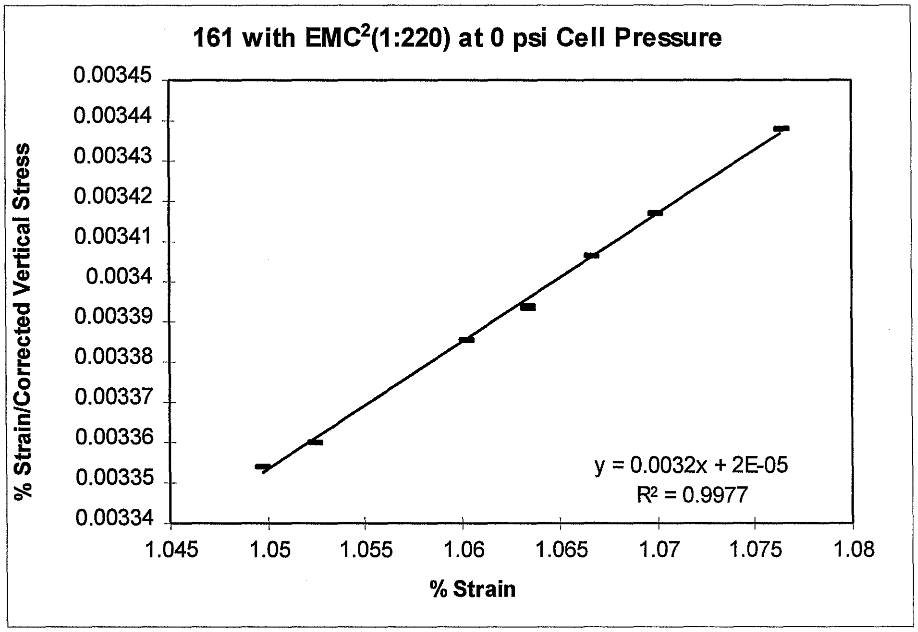

Figure 6. Modified Plot for EMC^2(1:220) at 0 psi Cell Pressure (SH 161)

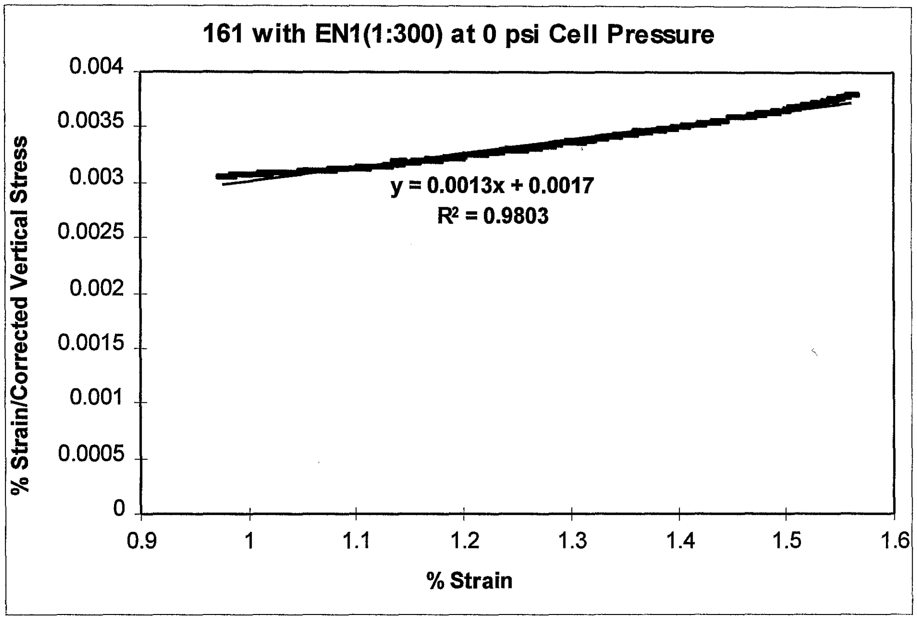

Figure 7. Modified Plot for EN1 (1:300) at 0 psi Cell Pressure (SH 161)

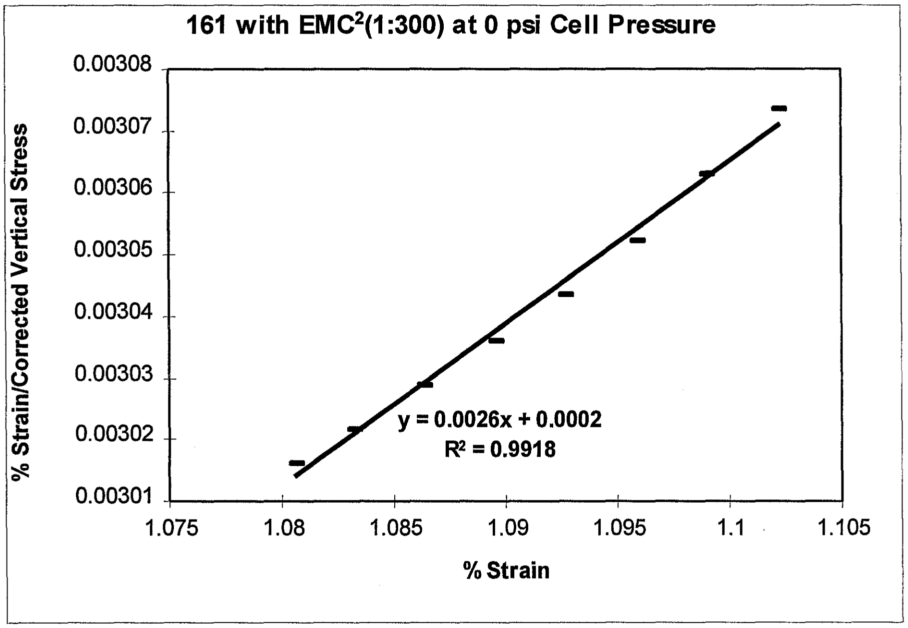

Figure 8. Modified Plot for EMC^2 (1:300) at 0 psi Cell Pressure (SH 161)

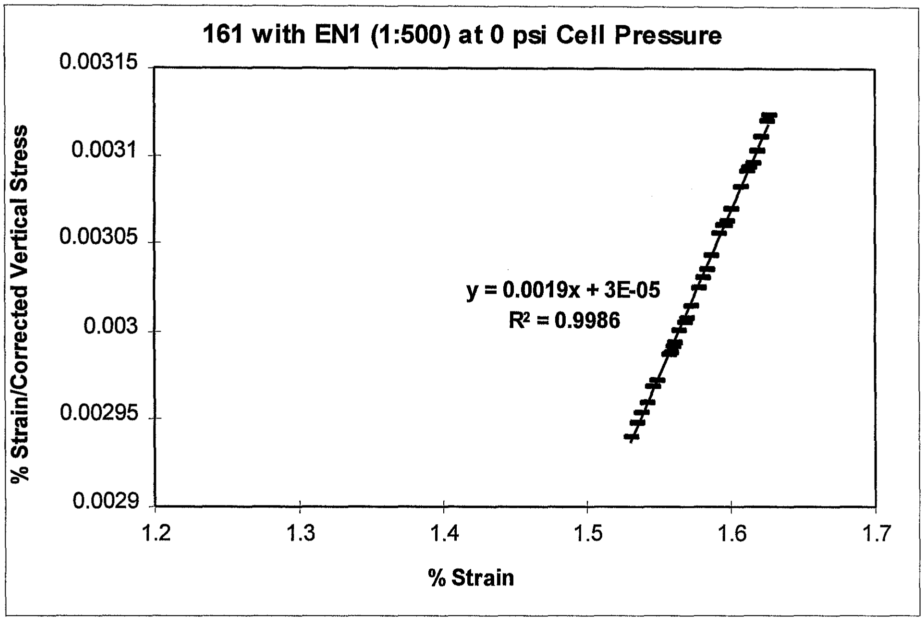

Figure 9. Modified Plot for EN1 (1:500) at 0 psi Cell Pressure (SH 161)

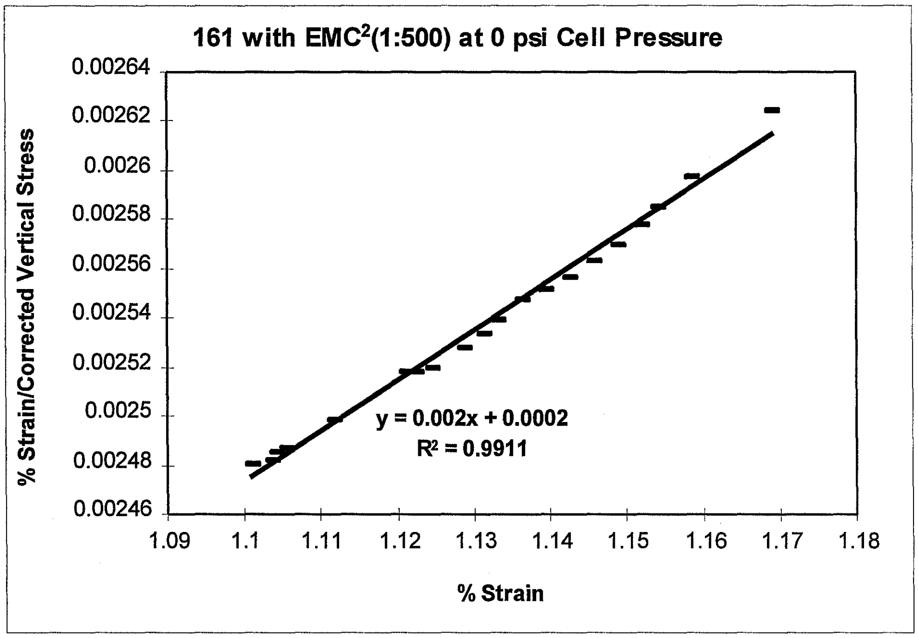

Figure 10. Modified Plot for EMC^2 (1:500) at 0 psi Cell Pressure (SH 161)

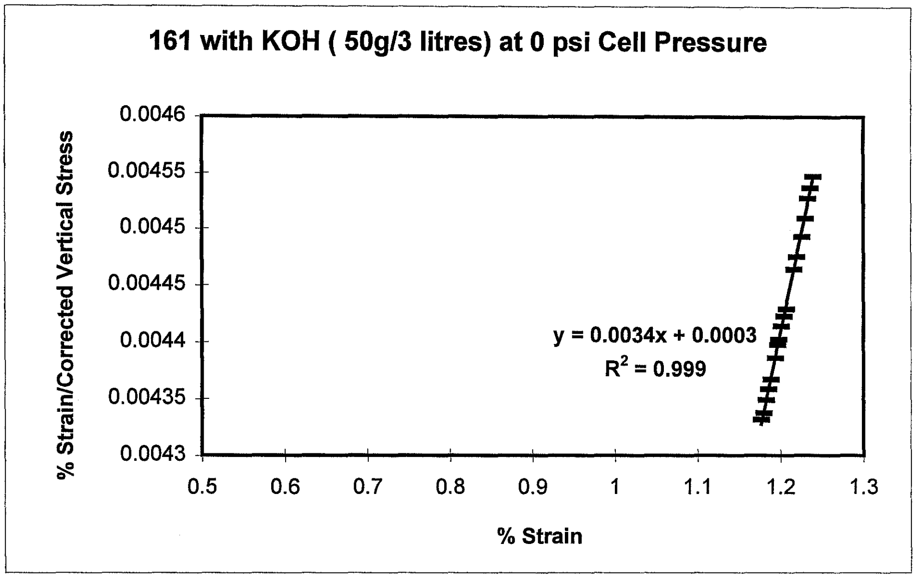

Figure 11. Modified Plot for KOH (50g/3ℓ) at 0 psi Cell Pressure (SH 161)

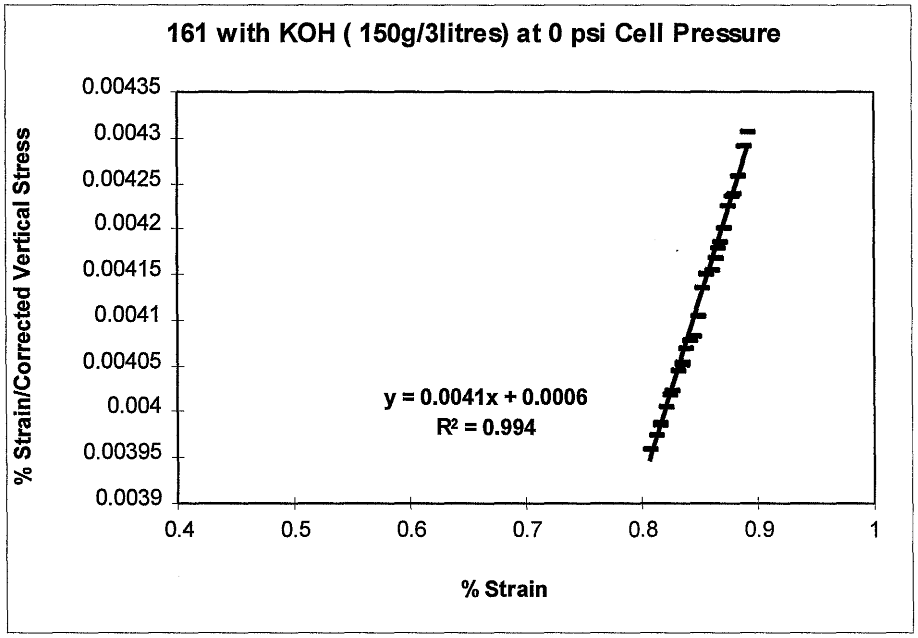

Figure 12. Modified Plot for KOH (150g/3ℓ) at 0 psi Cell Pressure (SH 161)

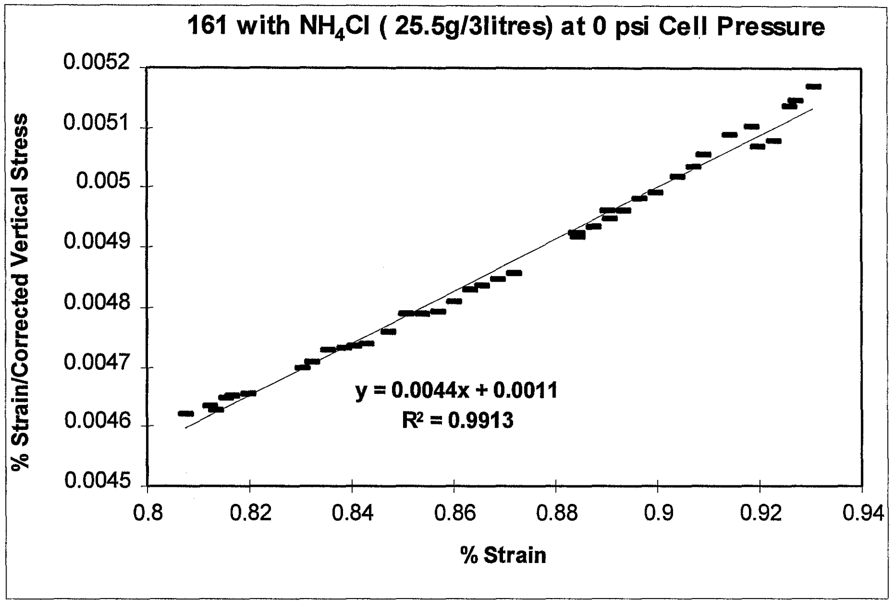

Figure 13. Modified Plot for NH_4Cl at 0 psi Cell Pressure (SH 161)

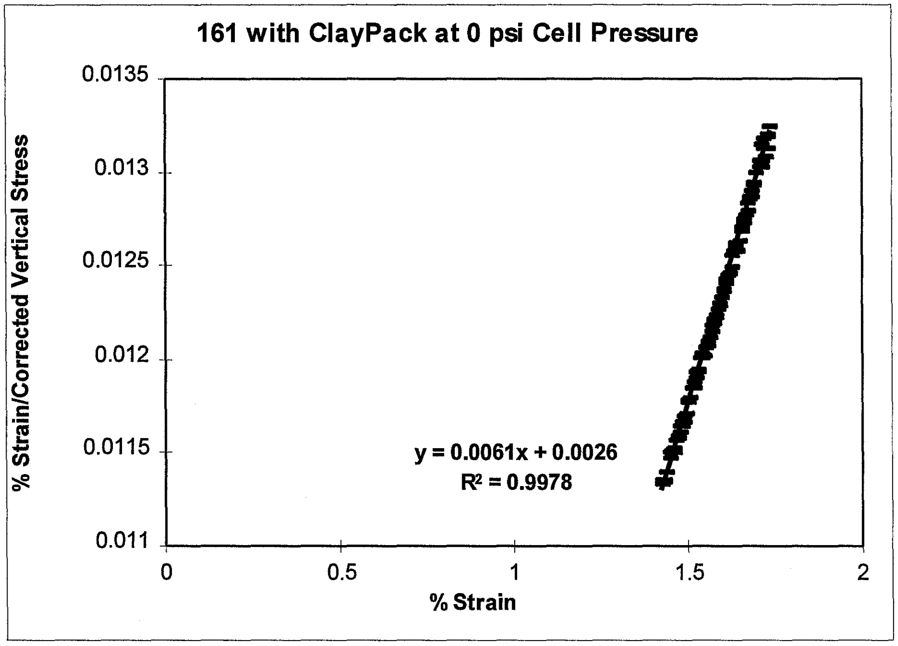

Figure 14. Modified Plot for ClayPack at 0 psi Cell Pressure (SH 161)

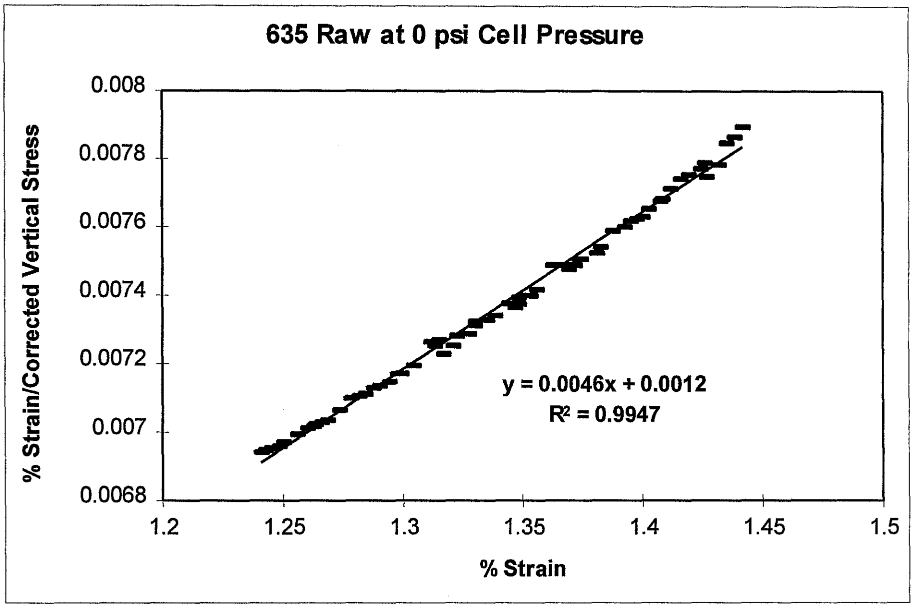

Figure 15. Modified Plot for Raw at 0 psi Cell Pressure (IH 635)

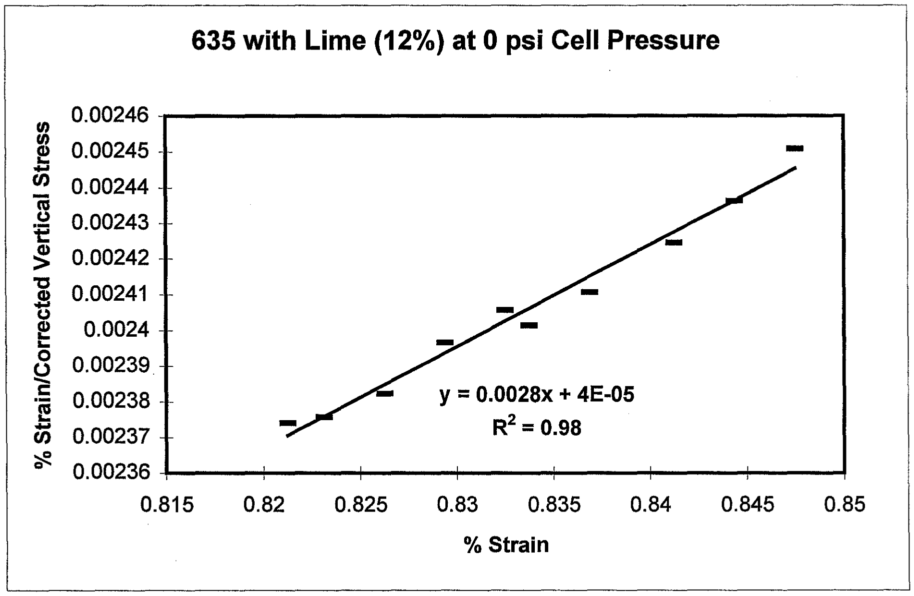

Figure 16. Modified Plot for Lime (12%) at 0 psi Cell Pressure (IH 635)

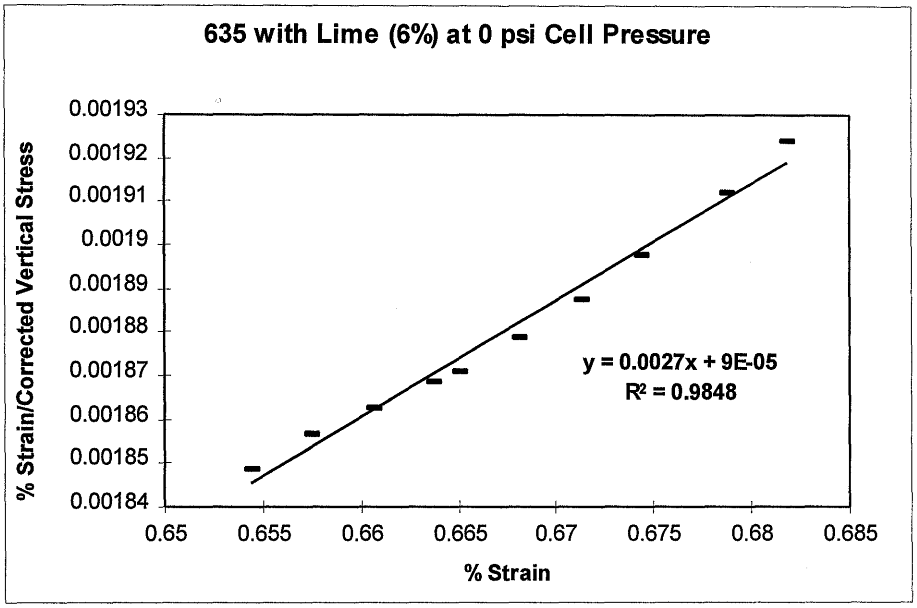

Figure 17. Modified Plot for Lime (6%) at 0 psi Cell Pressure (IH 635)

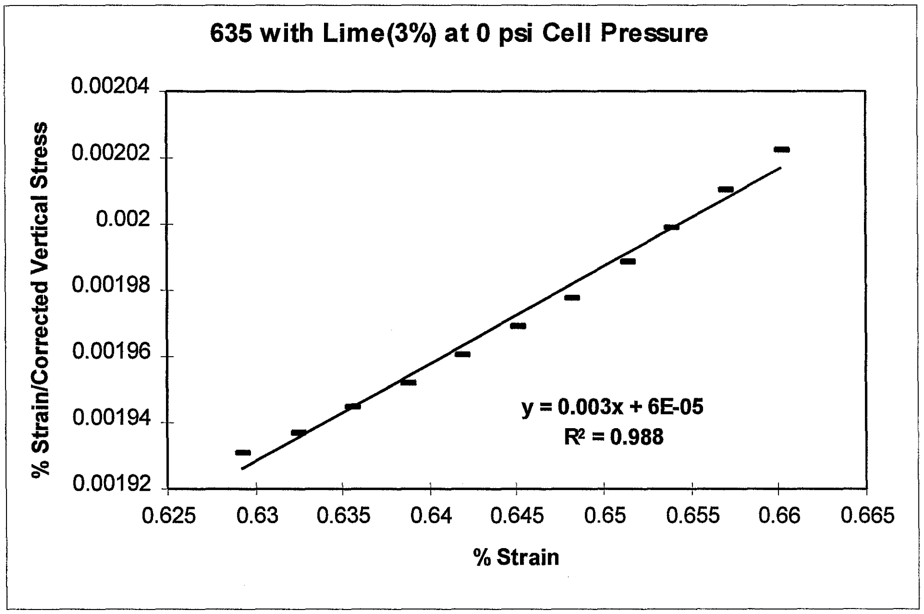

Figure 18. Modified Plot for Lime (3%) at 0 psi Cell Pressure (IH 635)

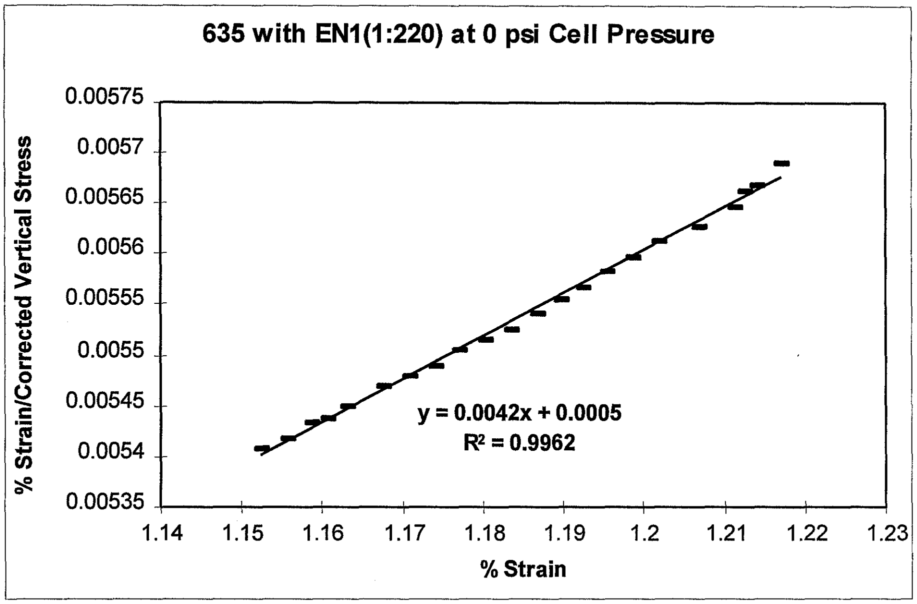

Figure 19. Modified Plot for EN1 (1:220) at 0 psi Cell Pressure (SH 161)

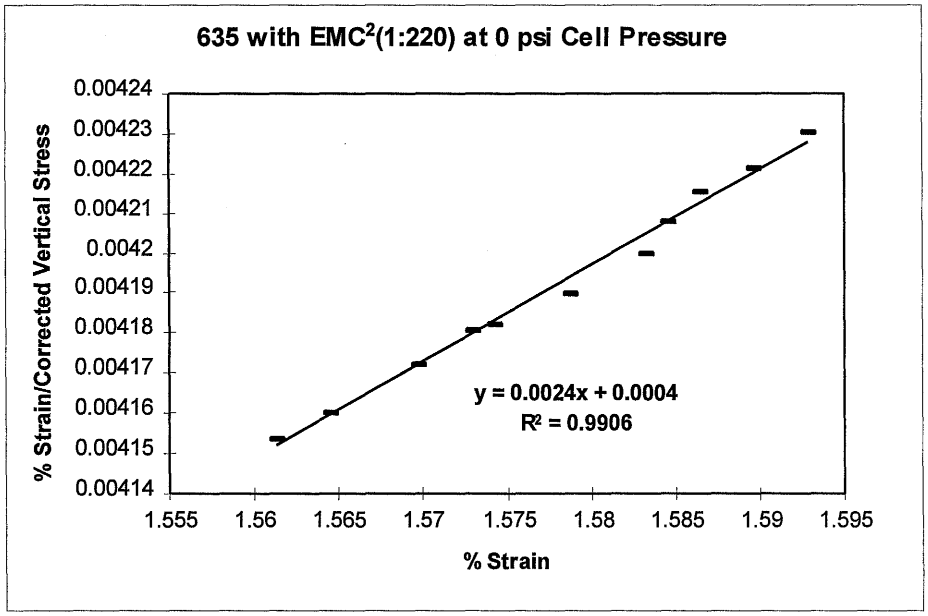

Figure 20. Modified Plot for EMC^2 (1:220) at 0 psi Cell Pressure (IH 635)

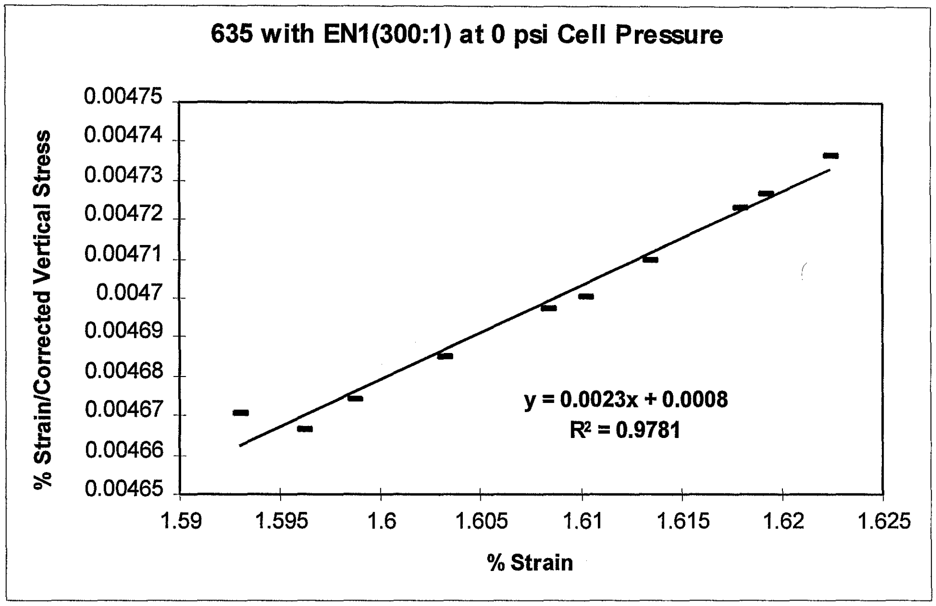

Figure 21. Modified Plot for EN1 (1:300) at 0 psi Cell Pressure (IH 635)

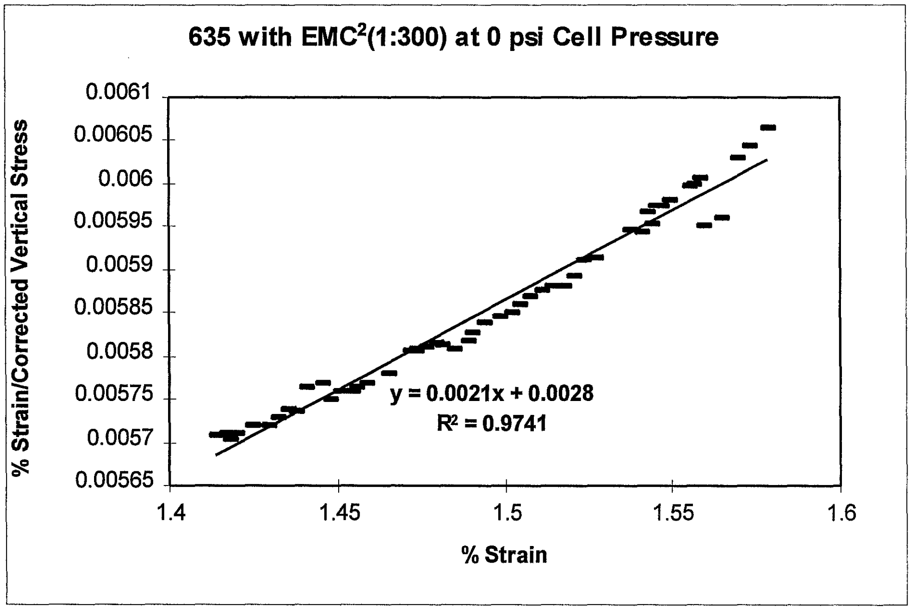

Figure 22. Modified Plot for EMC^2 (1:300) at 0 psi Cell Pressure (IH 635)

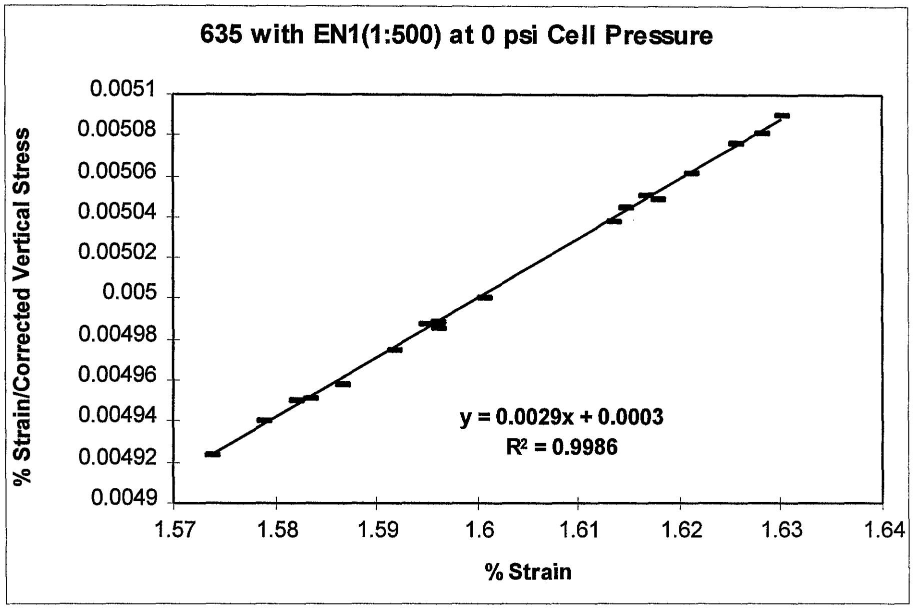

Figure 23. Modified Plot for EN1 (1:500) at 0 psi Cell Pressure (IH 635)

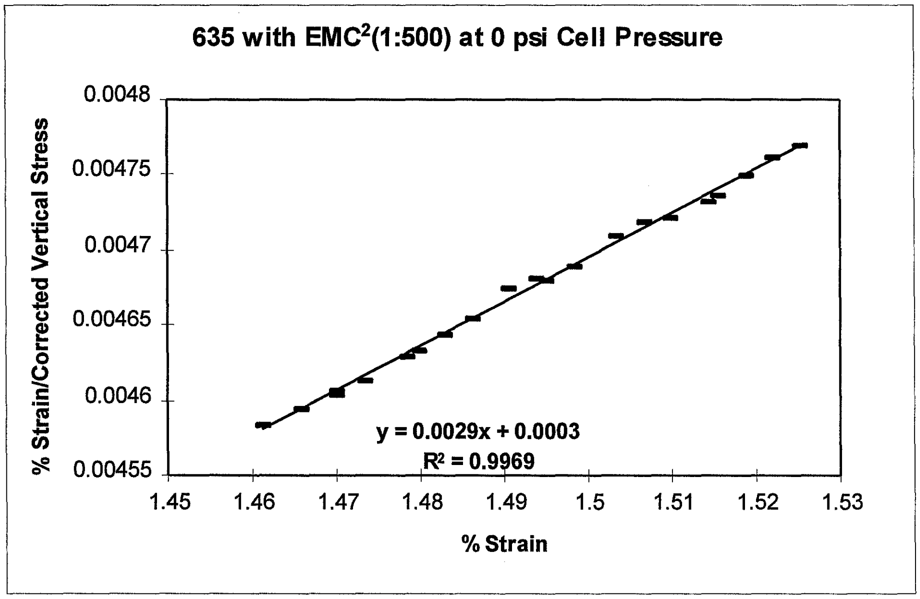

Figure 24. Modified Plot for EMC^2 (1:500) at 0 psi Cell Pressure (IH 635)

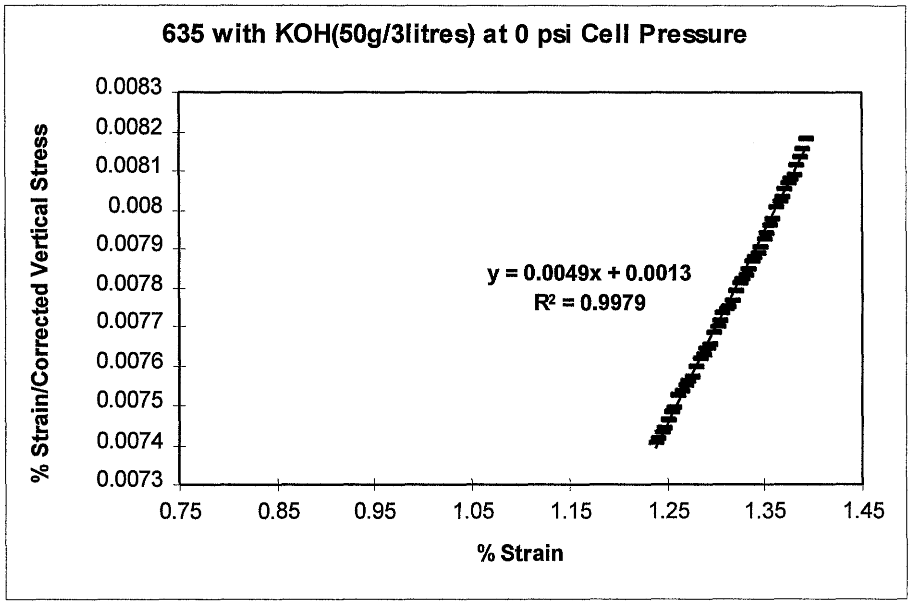

Figure 25. Modified Plot for KOH (50g/3ℓ) at 0 psi Cell Pressure (IH 635)

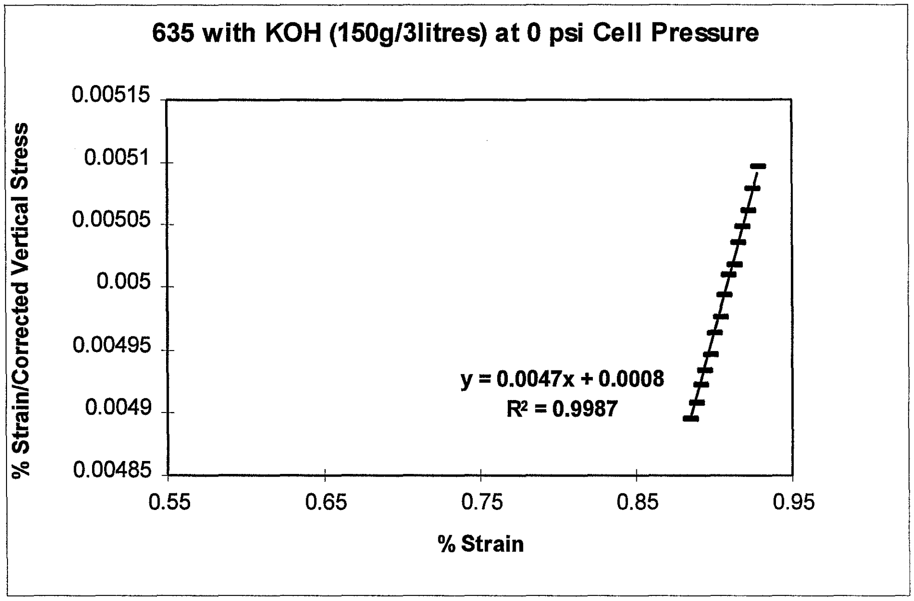

Figure 26. Modified Plot for KOH (150g/3ℓ) at 0 psi Cell Pressure (IH 635)

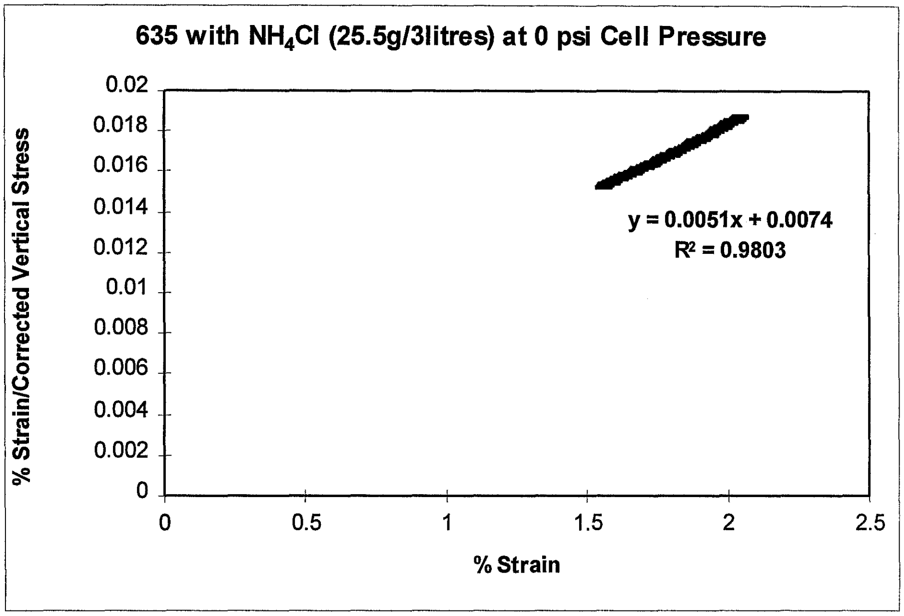

Figure 27. Modified Plot for NH_4Cl at 0 psi Cell Pressure (IH 635)

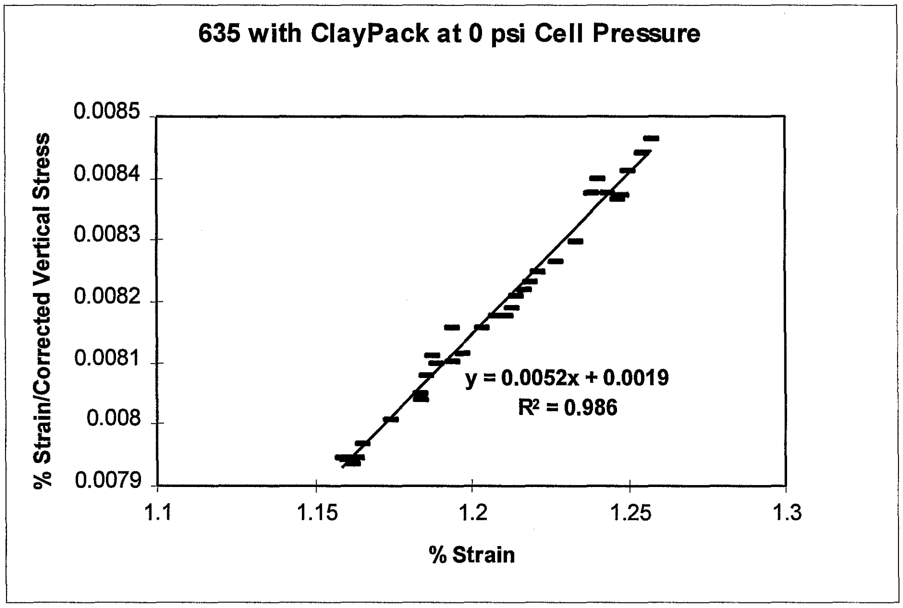

Figure 28. Modified Plot for ClayPack at 0 psi Cell Pressure (IH 635)

APPENDIX B

Tables Showing the Values of 'm' and 'c' from the Corrected Charts in Appendix A, Strength at Failure, Ultimate Strength and Stiffness

Table 1. Results from the Triaxial Test Curves - SH 161

Description of the Test Sample	Values of m and c from the charts		Ultimate Strength Su 1/m (psi)	Strength at Failure S_f	Ratio R $R=S_f / S_u$	Stiffness 1/c
	m	c				
Treated/Untreated - Raw						
At 0 psi Cell Pressure	0.0043	0.0017	232.56	189.95	0.82	588.24
At 4 psi Cell Pressure	0.0041	0.0001	243.90	179.60	0.74	10000.00
At 8 psi Cell Pressure	0.0041	0.0003	243.90	235.68	0.97	3333.33
Treated/Untreated- EN1 @ 1:220						
At 0 psi Cell Pressure	0.0025	0.0009	400.00	332.76	0.83	1111.11
At 4 psi Cell Pressure	0.0024	0.0012	416.67	338.49	0.81	833.33
At 8 psi Cell Pressure	0.0024	0.0012	416.67	432.56	1.04	833.33
Treated/Untreated- EN1 @ 1:300						
At 0 psi Cell Pressure	0.0023	0.0002	434.78	421.45	0.97	5000.00
At 4 psi Cell Pressure	0.0018	0.0004	555.56	487.99	0.88	2500.00
At 8 psi Cell Pressure	0.0022	0.0002	454.55	450.9	0.99	5000.00
Treated/Untreated- EN1 @ 1:500						
At 0 psi Cell Pressure	0.0019	0.0002	526.32	529.46	1.01	5000.00
At 4 psi Cell Pressure	0.0017	0.0004	588.24	542.27	0.92	2500.00
At 8 psi Cell Pressure	0.0014	0.0004	714.29	607.99	0.85	2500.00
Treated/Untreated - EMC2 @ 1:220						
At 0 psi Cell Pressure	0.0032	2.00E-05	312.50	313.27	1.00	50000.00
At 4 psi Cell Pressure	0.0018	0.0001	555.56	546.69	0.98	10000.00
At 8 psi Cell Pressure	0.002	0.0003	500.00	551.76	1.10	3333.33
Treated/Untreated - EMC2 @ 1:300						
At 0 psi Cell Pressure	0.0026	0.0002	384.62	362.7	0.94	5000.00
At 4 psi Cell Pressure	0.0028	0.0001	357.14	342.83	0.96	10000.00
At 8 psi Cell Pressure	0.0025	9.00E-05	400.00	384.61	0.96	11111.11
Treated/Untreated - EMC2 @ 1:500						
At 0 psi Cell Pressure	0.002	0.0002	500.00	451.97	0.90	5000.00
At 4 psi Cell Pressure	0.0023	0.0003	434.78	382.93	0.88	3333.33
At 8 psi Cell Pressure	0.0022	0.0003	454.55	413.3	0.91	3333.33
Treated/Untreated - KOH (50g/3R)						
At 0 psi Cell Pressure	0.0034	0.0003	294.12	274.22	0.93	3333.33
At 4 psi Cell Pressure	0.0039	0.0007	256.41	227.7	0.89	1428.57
At 8 psi Cell Pressure	0.0034	0.0002	294.12	285.57	0.97	5000.00

Table 1 Continued

Description of the Test Sample	Values of m and c from the charts		Ultimate Strength Su 1/m (psi)	Strength at Failure S_f	Ratio R $R=S_f / S_u$	Stiffness 1/c
	m	c				
Treated/Untreated - KOH (150g/3R)						
At 0 psi Cell Pressure	0.0041	0.0006	243.90	207.46	0.85	1666.67
At 4 psi Cell Pressure	0.0023	0.0014	434.78	251.21	0.58	714.29
At 8 psi Cell Pressure	0.0037	0.0004	270.27	249.66	0.92	2500.00
Treated/Untreated - NH4Cl (25.5g/R)						
At 0 psi Cell Pressure	0.0044	0.0011	227.27	183.24	0.81	909.09
At 4 psi Cell Pressure	0.0037	0.0009	270.27	215.72	0.80	1111.11
At 8 psi Cell Pressure	0.0044	0.0003	227.27	213.21	0.94	3333.33
Treated/Untreated - Lime (12%)						
At 0 psi Cell Pressure	0.0029	2.00E-05	344.83	343.6	1.00	50000.00
At 4 psi Cell Pressure	0.0019	0.0003	526.32	420.36	0.80	3333.33
At 8 psi Cell Pressure	0.002	8.00E-05	500.00	482.4	0.96	12500.00
Treated/Untreated - Lime (6%)						
At 0 psi Cell Pressure	0.0024	5.00E-06	416.67	417.07	1.00	200000.00
At 4 psi Cell Pressure	0.0022	0.0001	454.55	424.69	0.93	10000.00
At 8 psi Cell Pressure	0.0023	5.00E-05	434.78	424.65	0.98	20000.00
Treated/Untreated - Lime (3%)						
At 0 psi Cell Pressure	0.0038	4.00E-05	263.16	263.08	1.00	25000.00
At 4 psi Cell Pressure	0.0032	0.0002	312.50	288.1	0.92	5000.00
At 8 psi Cell Pressure	0.0026	6.00E-04	384.62	310.64	0.81	1666.67
Treated/Untreated - ClayPack, (1:300)						
At 0 psi Cell Pressure	0.0061	0.0026	163.93	131.63	0.80	384.62
At 4 psi Cell Pressure	0.0057	0.0025	175.44	144.83	0.83	400.00
At 8 psi Cell Pressure	0.0062	0.0009	161.29	149.32	0.93	1111.11

Table 2. Results from the Triaxial Test Curves - IH 635

Description of the Test Sample	Values of m and c from the charts		Ultimate Strength S_u 1/m (psi)	Strength at Failure S_f	Ratio R $R=S_f/S_u$	Stiffness 1/c
	m	c				
Treated/Untreated - Raw						
At 0 psi Cell Pressure	0.0046	0.0012	217.39	184.19	0.85	833.33
At 4 psi Cell Pressure	0.0042	0.0006	238.10	213.79	0.90	1666.67
At 8 psi Cell Pressure	0.0039	0.0005	256.41	233.94	0.91	2000.00
Treated/Untreated- EN1 @ 1:220						
At 0 psi Cell Pressure	0.0042	0.0005	238.10	204.5	0.86	2000.00
At 4 psi Cell Pressure	0.0026	0.0033	384.62	197.72	0.51	303.03
At 8 psi Cell Pressure	0.0029	0.0013	344.83	275.13	0.80	769.23
Treated/Untreated- EN1 @ 1:300						
At 0 psi Cell Pressure	0.0023	0.0008	434.78	347.75	0.80	1250.00
At 4 psi Cell Pressure	0.0023	0.0005	434.78	390.17	0.90	2000.00
At 8 psi Cell Pressure	0.0031	0.0021	322.58	269.3	0.83	476.19
Treated/Untreated- EN1 @ 1:500						
At 0 psi Cell Pressure	0.0029	0.0003	344.83	327.54	0.95	3333.33
At 4 psi Cell Pressure	0.0027	0.0004	370.37	348.95	0.94	2500.00
At 8 psi Cell Pressure	0.0025	2.00E-05	400.00	399.38	1.00	50000.00
Treated/Untreated - EMC^2 @ 1:220						
At 0 psi Cell Pressure	0.0024	0.0004	416.67	382.57	0.92	2500.00
At 4 psi Cell Pressure	0.0026	0.0002	384.62	364.6	0.95	5000.00
At 8 psi Cell Pressure	0.0025	0.0006	400.00	363.02	0.91	1666.67
Treated/Untreated - EMC^2 @ 1:300						
At 0 psi Cell Pressure	0.0021	0.0009	476.19	266.32	0.56	1111.11
At 4 psi Cell Pressure	0.0025	0.0007	400.00	352.82	0.88	1428.57
At 8 psi Cell Pressure	0.0026	3.00E-05	384.62	389.27	1.01	33333.33
Treated/Untreated - EMC^2 @ 1:500						
At 0 psi Cell Pressure	0.0029	0.0003	344.83	324.56	0.94	3333.33
At 4 psi Cell Pressure	0.0024	0.001	416.67	338.91	0.81	1000.00
At 8 psi Cell Pressure	0.0024	0.0011	416.67	329.22	0.79	909.09
Treated/Untreated - KOH (50g/3ℓ)						
At 0 psi Cell Pressure	0.0049	0.0013	204.08	170.55	0.84	769.23
At 4 psi Cell Pressure	0.0045	0.0012	222.22	185.74	0.84	833.33
At 8 psi Cell Pressure	0.0049	0.0006	204.08	188.11	0.92	1666.67

Table 2. Continued

Description of the Test Sample	Values of m and c from the charts		Ultimate Strength S_u 1/m (psi)	Strength at Failure S_f	Ratio R $R=S_f / S_u$	Stiffness 1/c
	m	c				
Treated/Untreated - KOH (150g/3ℓ)						
At 0 psi Cell Pressure	0.0041	0.0018	243.90	182.07	0.75	555.56
At 4 psi Cell Pressure	0.0049	0.0007	204.08	177.34	0.87	1428.57
At 8 psi Cell Pressure	0.0045	0.0008	222.22	186.24	0.84	1250.00
Treated/Untreated - NH_4Cl (25.5g/3ℓ)						
At 0 psi Cell Pressure	0.0051	0.0074	196.08	109.72	0.56	135.14
At 4 psi Cell Pressure	0.0077	0.0023	129.87	113.74	0.88	434.78
At 8 psi Cell Pressure	0.0052	0.033	192.31	142.44	0.74	30.30
Treated/Untreated - Lime (12%)						
At 0 psi Cell Pressure	0.0028	0.0011	357.14	347.19	0.97	909.09
At 4 psi Cell Pressure	0.0026	7.00E-04	384.62	388.6	1.01	1428.57
At 8 psi Cell Pressure	-0.008	0.00053	-125.00	405.47	-3.24	1886.79
Treated/Untreated - Lime (6%)						
At 0 psi Cell Pressure	0.0027	9.00E-04	370.37	355.61	0.96	1111.11
At 4 psi Cell Pressure	0.0029	0.0002	344.83	384.18	1.11	5000.00
At 8 psi Cell Pressure	0.0025	0.0002	400.00	457.16	1.14	5000.00
Treated/Untreated - Lime (3%)						
At 0 psi Cell Pressure	0.003	6.00E-04	333.33	327.77	0.98	1666.67
At 4 psi Cell Pressure	0.003	5.00E-05	333.33	330.77	0.99	20000.00
At 8 psi Cell Pressure	0.0025	0.0001	400.00	367.86	0.92	10000.00
Treated/Untreated - ClayPack, (1:300)						
At 0 psi Cell Pressure	0.0052	0.0019	192.31	147.5	0.77	526.32
At 4 psi Cell Pressure	0.0057	0.002	175.44	133.05	0.76	500.00
At 8 psi Cell Pressure	0.0054	0.001	185.19	161.6	0.87	1000.00

APPENDIX C

Calibration Charts for the 12 Probes of Psychrometer Except Probe #7

Calibration Chart for Wattman Filter Paper Grade 2, 55mm Diameter

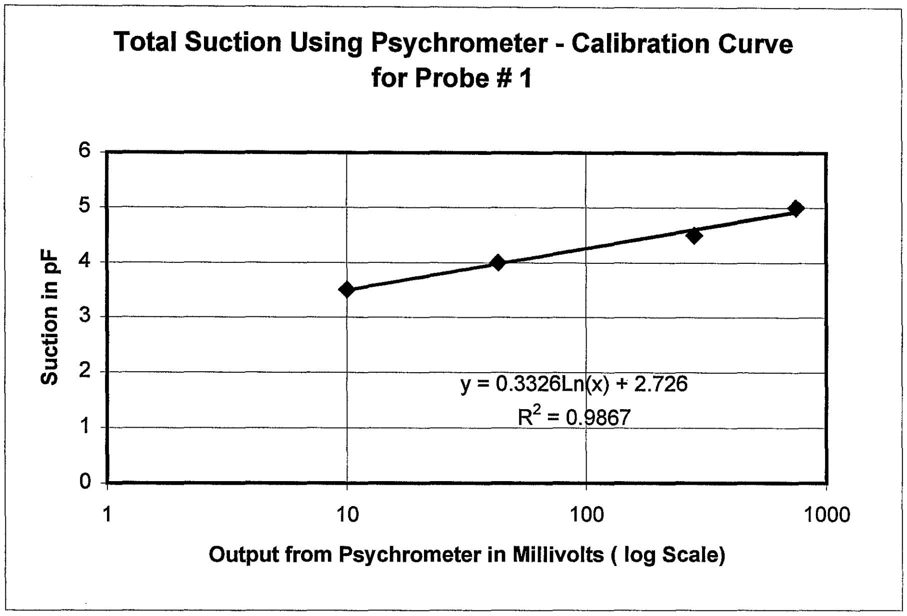

Figure 29. Calibration Chart for Probe #1

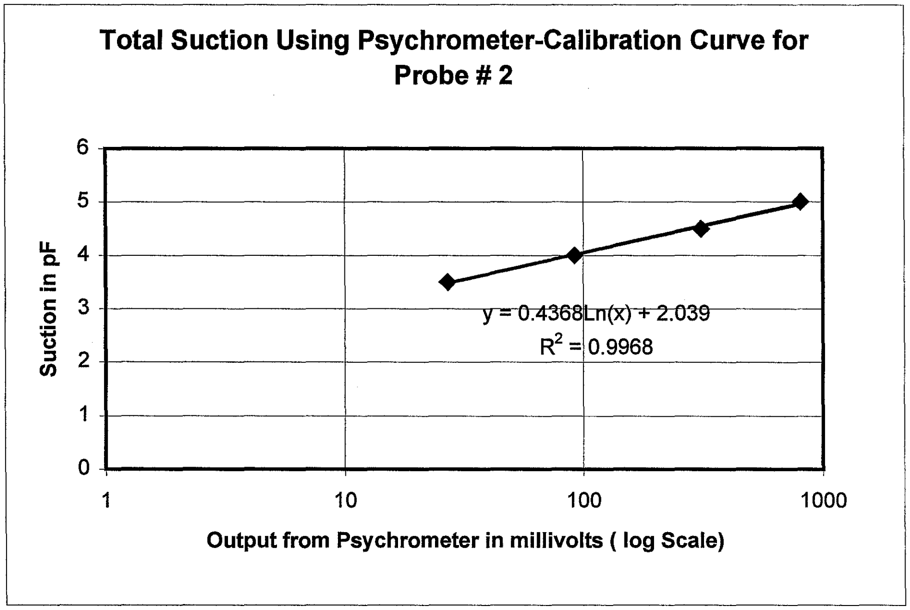

Figure 30. Calibration Chart for Probe #2

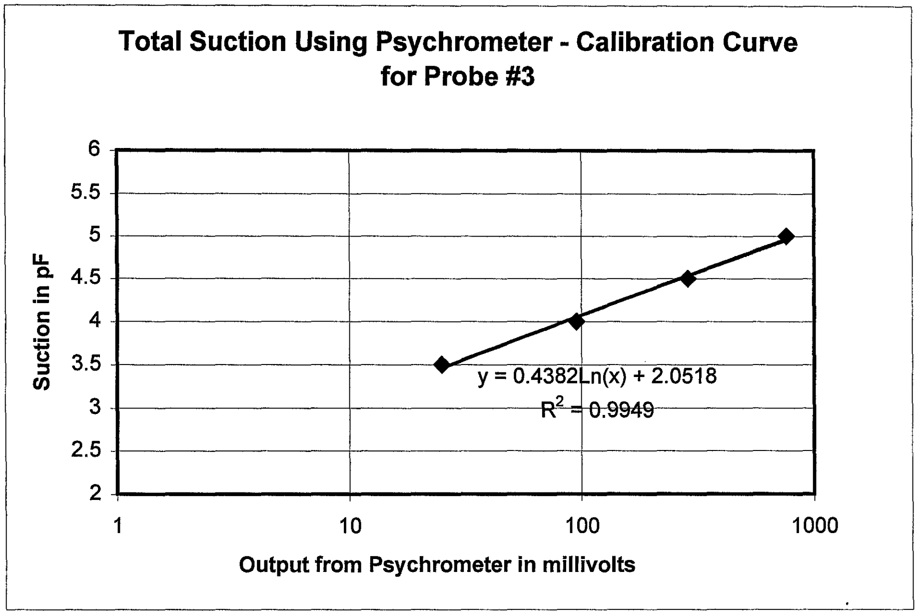

Figure 31. Calibration Chart for Probe #3

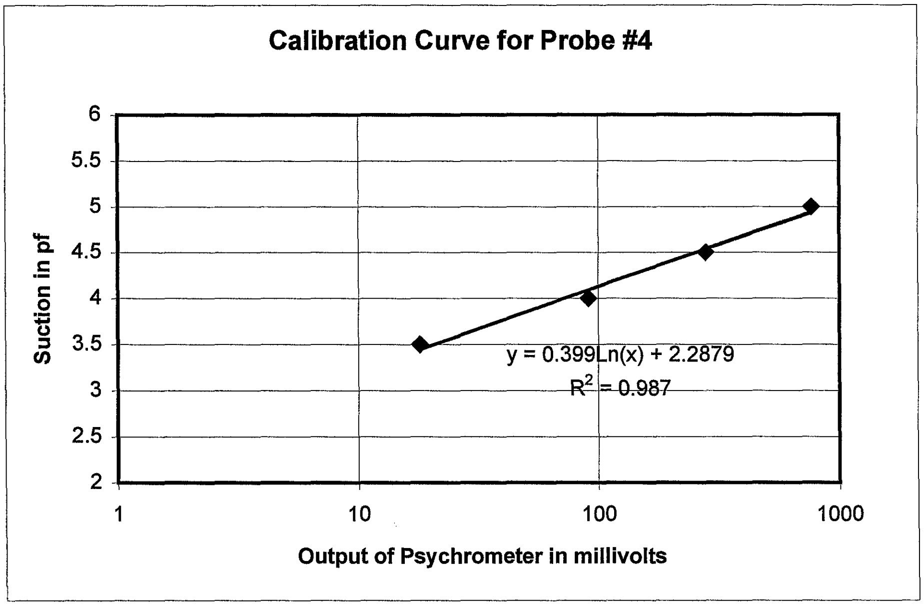

Figure 32. Calibration Chart for Probe #4

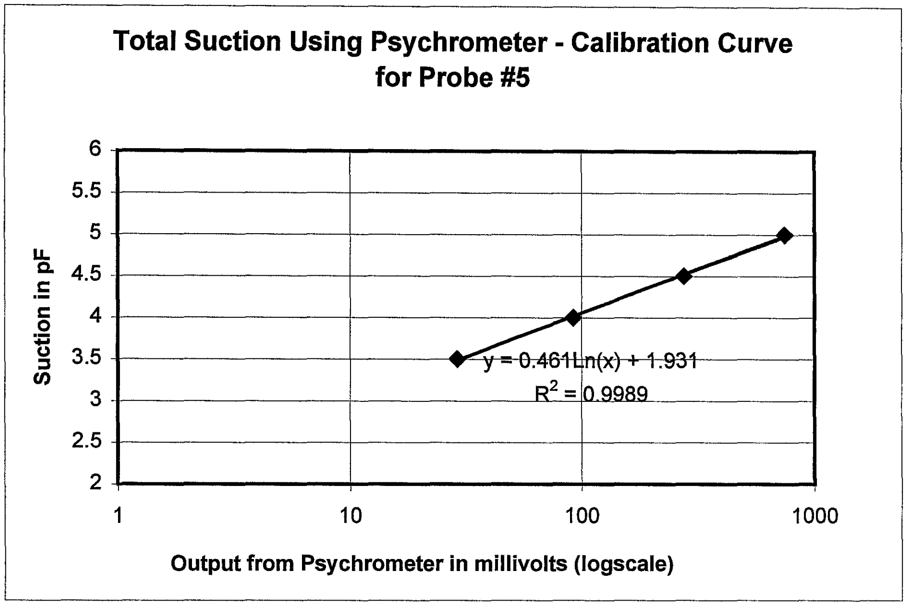

Figure 33. Calibration Chart for Probe #5

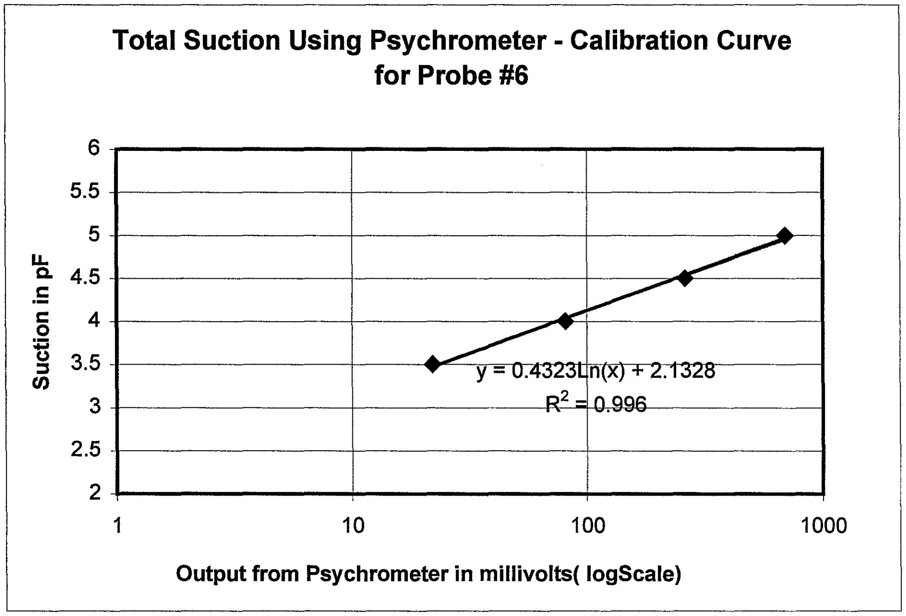

Figure 34. Calibration Chart for Probe #6

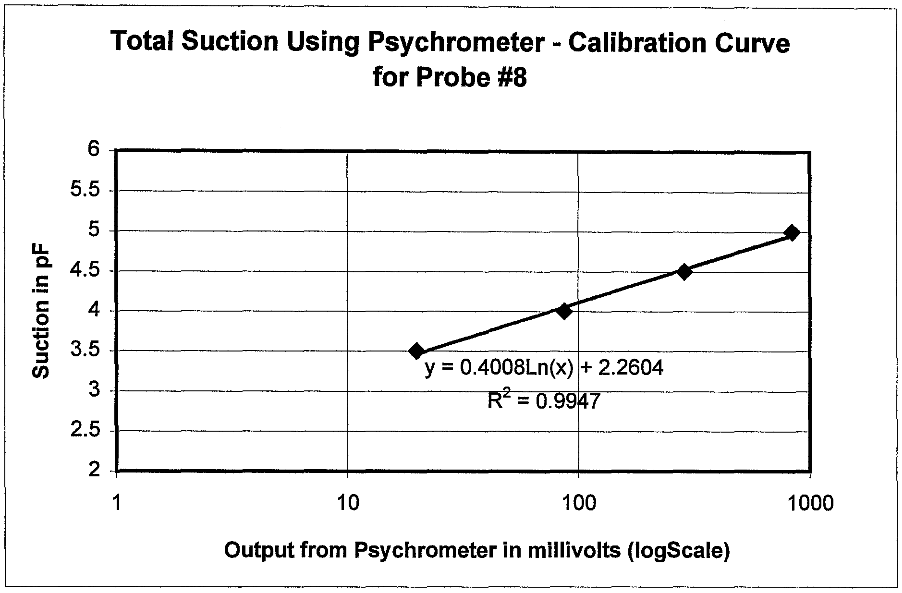

Figure 35. Calibration Chart for Probe #8

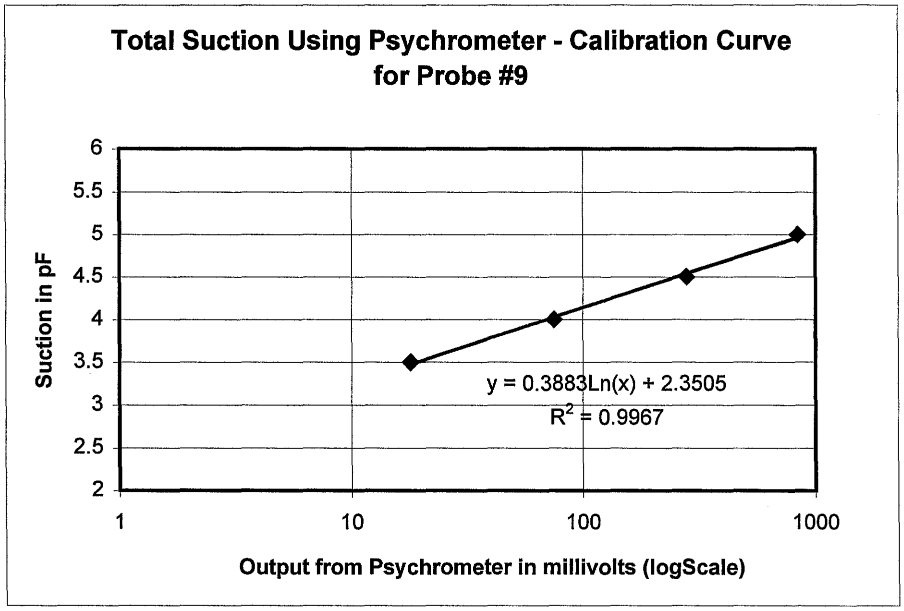

Figure 36. Calibration Chart for Probe #9

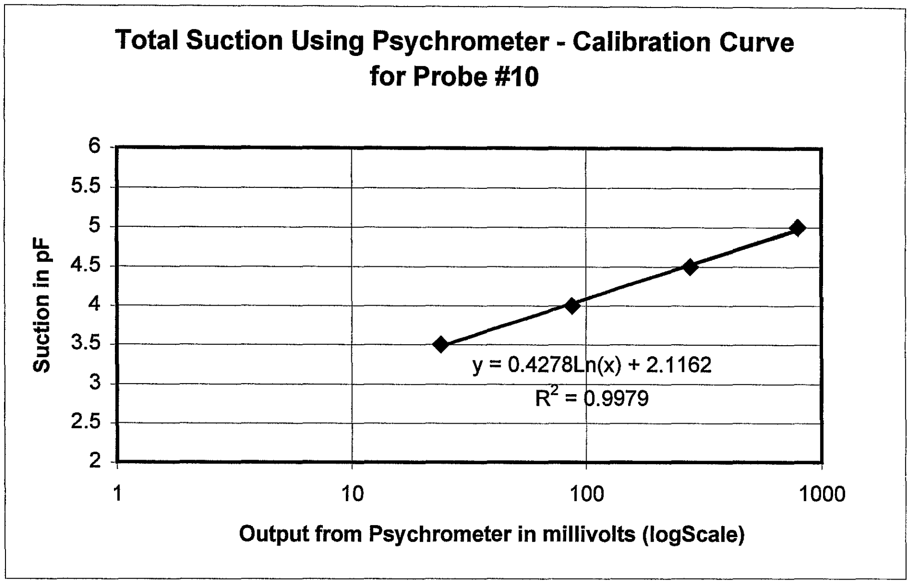

Figure 37. Calibration Chart for Probe #10

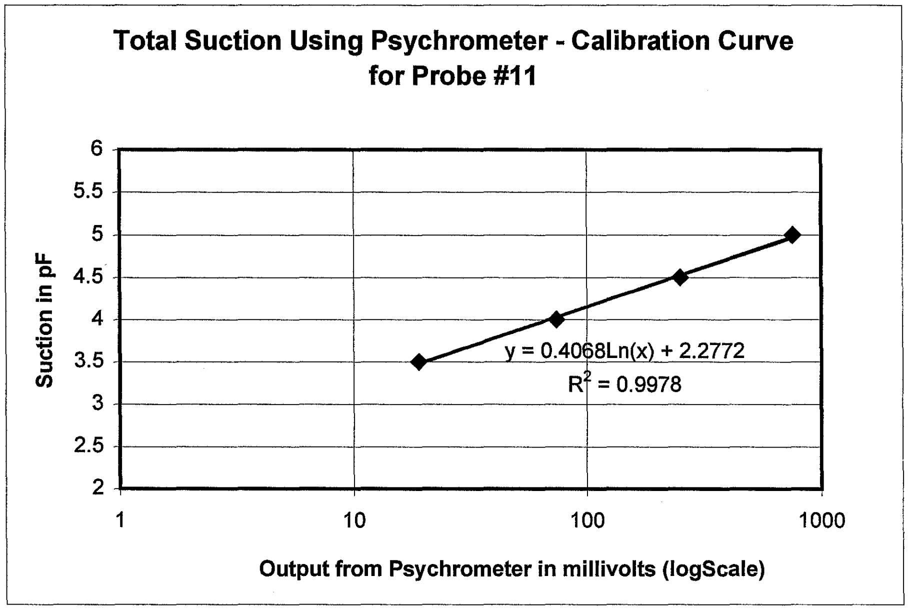

Figure 38. Calibration Chart for Probe #11

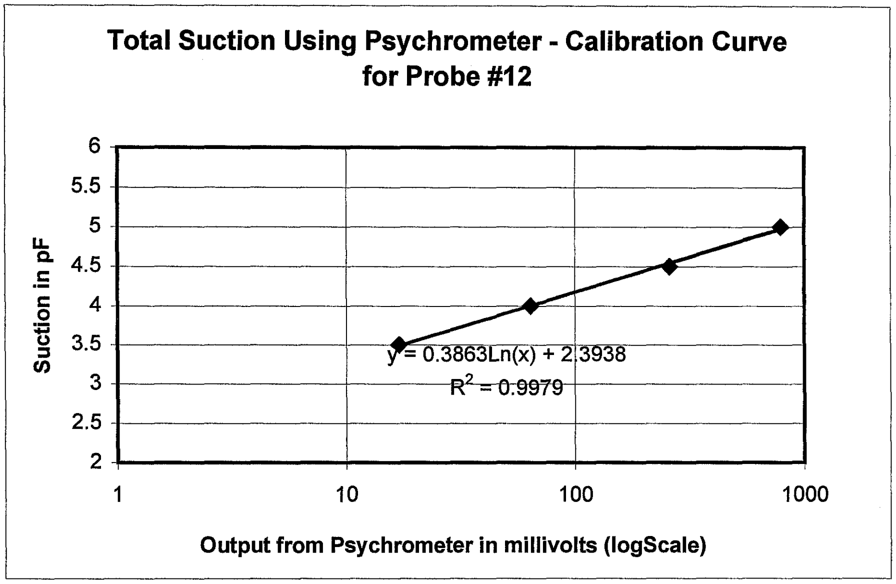

Figure 39. Calibration Chart for Probe #12

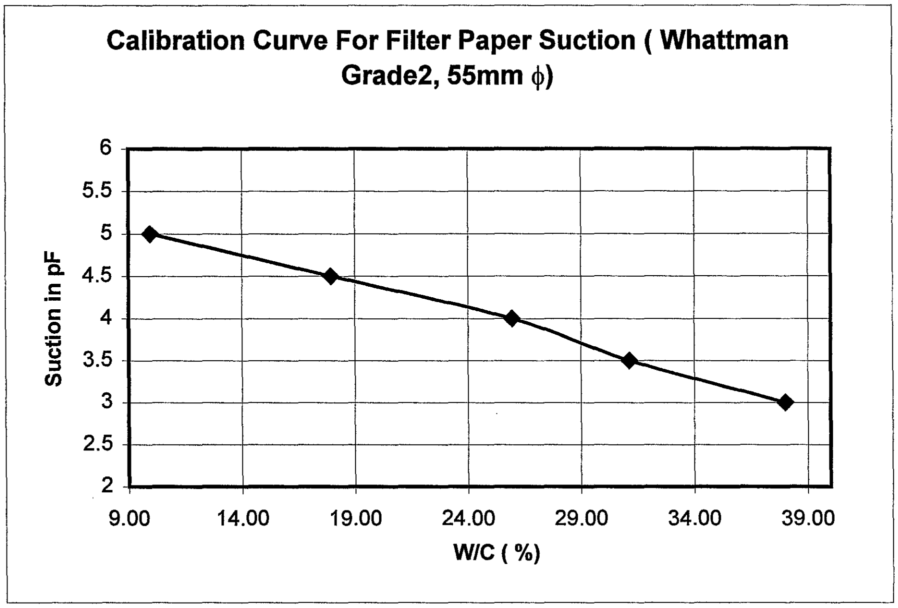

Figure 40. Calibration Chart for Matric Suction When Using Whattman Grade 2 Filter Paper of Diameter 55 mm

APPENDIX D

Total Suction of Leacheates from Permeability Test

Table 1. Total Suction Values of Leacheate (Treated / Untreated - SH 161

Treated/Untreated	Total Suction of the Leacheates (pF)
161 EMC^2	3.35
161 EN1	3.77
161 CP	3.42
161 Lime	3.26
161 Raw	3.37
161 NH_4Cl	3.49
161 KOH	3.24

Table 2 - Total Suction Values of Leacheate (Treated / Untreated) - IH 635

Treated/Untreated	Total Suction of the Leacheates (pF)
635 EMC^2	3.67
635 EN1	3.34
635 CP	3.24
635 Lime	3.38
635 Raw	3.00
635 NH_4Cl	3.32
635 KOH	3.25

APPENDIX E

ELECTRICAL CONDUCTIVITY TEST STANDARD

ELECTRICAL CONDUCTIVITY TEST STANDARD FOR LIME-SULFATE EXPANSION POTENTIAL

This is a description of the equipment needed and procedure to be followed in measuring the electrical conductivity of soils. The results of the test have been found to predict accurately the amount of swelling that will occur due to the lime-sulfate reaction, under standard conditions as described below. The test can be conducted in the field or in the lab. In the field, it can be used to provide an indication of whether the soil being tested will undergo a lime-sulfate reaction if it is stabilized with lime.

The equipment needed to perform a field evaluation of the sulfate content in soils includes the following:

a) Wide mouth plastic containers with water-proof lids,
b) Distilled water,
c) Battery driven digital scale that can measure up to 500 g (figure E-1),
d) Hand held conductivity meter (figure E-2), and
e) Calibration solutions for the conductivity meter.

The entire package costs less than $600.

The procedure is as follows:

Step 1: Find the location where the sulfate test is to be performed and use an auger to obtain two small soil samples at approximately 10 and 20 cm below the soil surface. Only 5 g of soil are needed to perform the test.

Step 2: Weigh approximately 5 g of each soil sample into two separate plastic containers. If the soil is wet, break lumps apart and leave the soil to air-dry for 1 to 2 hours. Record the exact dry weight of the samples.

Step 3: Now add distilled water with a mass of exactly 20 times the dry weight of the soil sample to the dry sample. Tightly close the lid of the plastic container and shake vigorously until the soil dissolves and forms a homogenous solution.

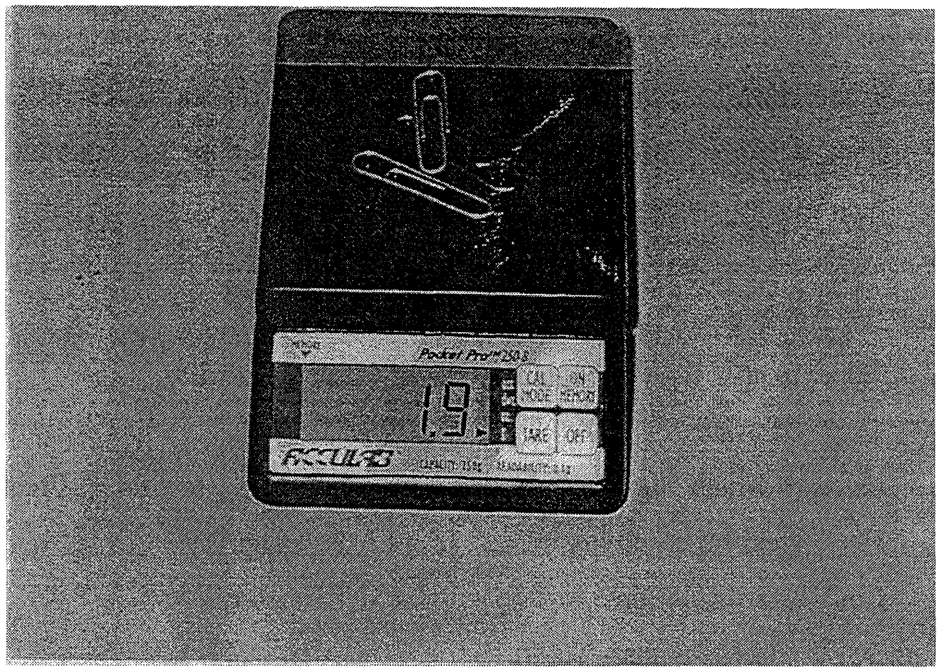

Figure E.1. Battery Driven Digital Scale

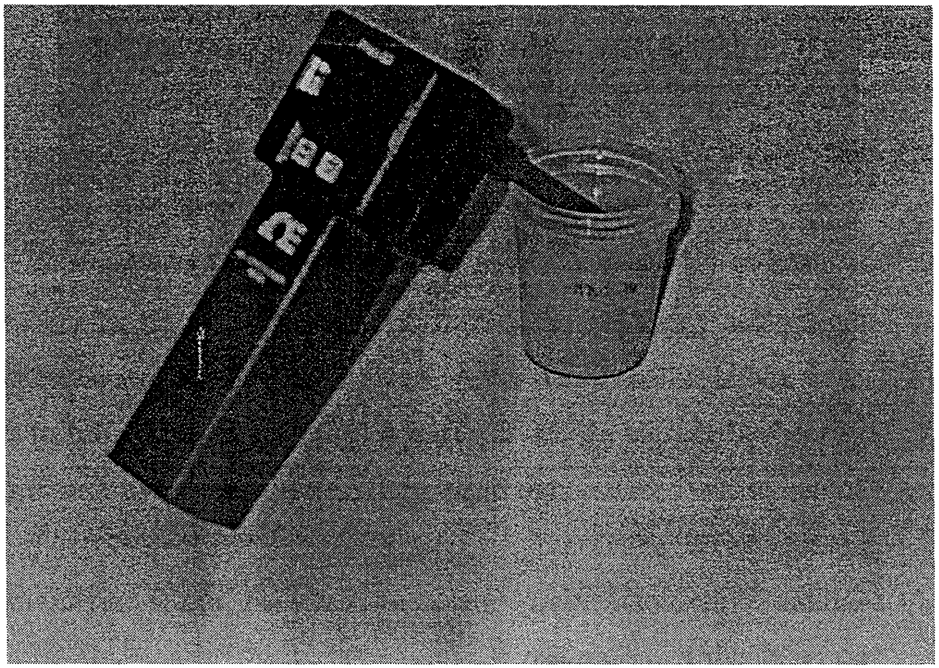

Figure E.2. Hand Held Electrical Conductivity Meter

Step 4: Calibrate the conductivity meter as described in the instruction manual accompanying the device.

Step 5: Take conductivity measurements on each soil:water mixture and record the data in milliSiemens per centimeter (mS/cm). Mixtures with a conductivity of more than 7 mS/cm have a potential to cause severe expansion problems.

Step 6: Use the following equations to determine an estimated amount of volumetric expansion that will occur in a sample of soil after curing in a moist environment for 7 days:

$$\%\ \text{Expansion (No Lime)} = 1.328 - 0.429\ \ln(\text{EC}) + 0.307\ (\text{EC})$$

$$\%\ \text{Expansion (6\% Lime)} = -0.784 + 0.143\ \ln(\text{EC}) + 0.448\ (\text{EC})$$

where EC = Electrical Conductivity measurement in mS/cm as measured by the conductivity probe.

A graph of the two functions is shown in figure E.3. The two curves cross at an electrical conductivity value of 7.05 mS/cm, and above this point, corresponding to a percent expansion of 2.65 percent, lime stabilization should not be used. Instead, one of the commercially available non-calcium stabilizers identified in this report should be used to provide a wet weather construction working table.

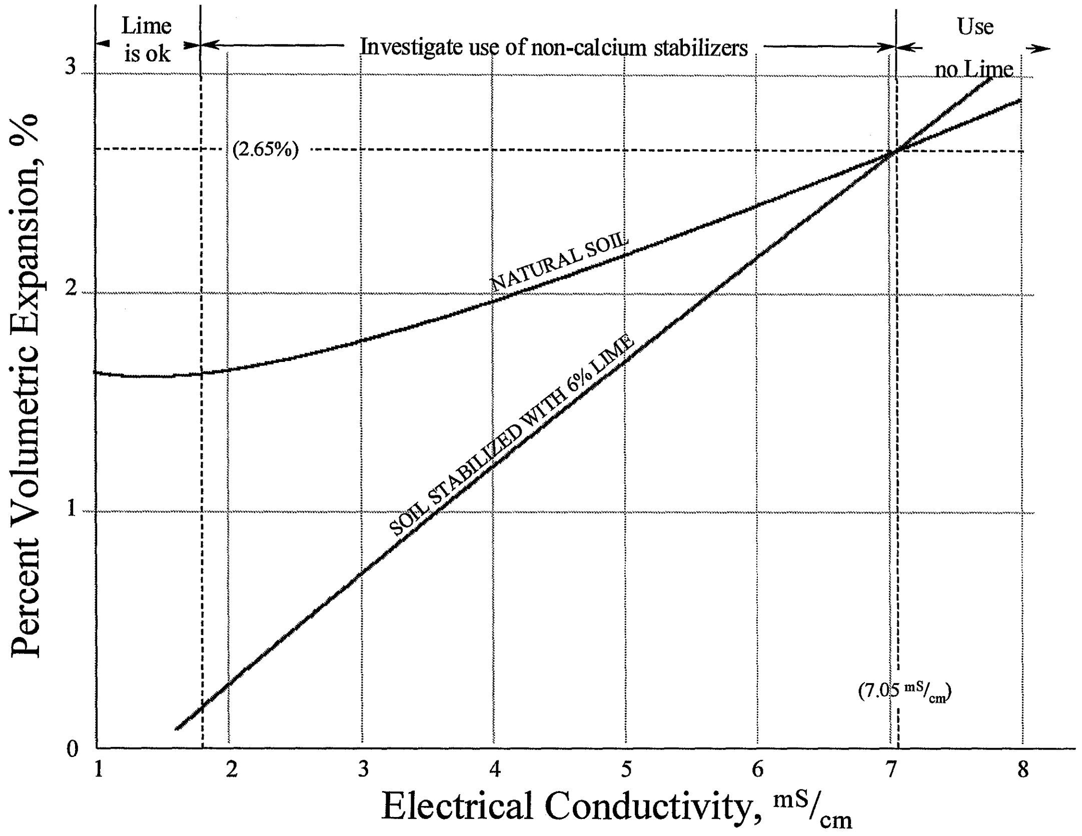

Figure E.3. Percent Volumetric Expansion of Natural and Lime Stabilized Soil As Predicted by Electrical Conductivity of the Soil

For soils with electrical conductivities in the range between 1.8 and 7.05 mS/cm, further investigation of engineering properties of stiffness and strength should be conducted using the triaxial test. Within this range, non-calcium stabilizers may prove to give less expansion and provide properties for a wet weather construction working table that are superior to those provided by lime stabiliazation. The test results in Research Report 3929-1F demonstrate this fact.

Below the electrical conductivity level of 1.8 mS/cm, corresponding to a soluble sulfate content of 0.16%, lime stabilization may usually provide lower volumetric expansion than the natural soil and will produce engineering properties for a wet weather construction working table that are competitive with those produced by the non-calcium stabilizers.

The electrical conductivity test is intended to be used as an index test and not as the only or final determination of whether to use lime or a non-calcium stabilizing agent to provide a wet weather construction working table.

The range of values of electrical conductivity and volumetric expansion from which these equations were derived is as follows.

Range of Measured		
Measurements	No Lime Added	6% Lime Added
Electrical Conductivity in Natural Soil Mixed with a Distilled Water in a 1:20 Ratio, (mS/cm)	1.6 - 27.7	1.6 - 27.7
Percent Volume Expansion, % (7 days of moist cure)	0.31 - 8.62	0.31 - 13.31

Inferences outside the range of these data cannot be supported by the data.

The curves shown in figure E.3 are not definitive of the long-term behavior of either natural soil or soil stabilized with 6% lime. The expansion test was run for a limited period of time only (7 days) and reflects the fact, which is established in this report, that compacted natural soil is more pervious than lime stabilized soil. It is also established in this report that the same soil stabilized by a non-calcium stabilizer will also have a lower permeability that the compacted natural soil. The expansion of both lime and non-calcium stabilized materials will be greater with a longer period of exposure to a moist environment longer than 7 days. The soil beneath a pavement is always moist. It is normally at a relative humidity of 98% or more. This explains why it is important, in using this standard test, to make other tests comparing the engineering properties (and swelling) of the soil stabilized with both lime and the non-calcium stabilizers when the electrical conductivity readings are between 1.8 and 7.05 mS per cm.