EFFECT OF IONIC STABILIZATION ON VERTICAL MOVEMENT IN EXPANSIVE SUBGRADE SOILS IN TEXAS

A Thesis

by

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ABSTRACT

An important consideration for the successful design of flexible pavement systems in Texas is the prediction and control of the vertical change in height of the subgrade soils due to swelling upon wetting and shrinkage upon drying. The purpose of this study was two-fold. The first objective was to measure the volume change characteristics of clayey subgrade soils from the SH130 corridor in Texas through a suction based approach using the pressure plate. The second goal was to test the effects of treating the soils with the EcSS-3000 chemical stabilizer and lime on controlling the vertical movement and moisture susceptibility.

Recent research studies have indicated that the suction compression index, γ_h, is the parameter that has the most significant direct influence on the amount of vertical movement taking place in expansive soils. The results indicate a 40-50 % reduction in the average γ_h values and a similar magnitude of reduction in combined swell and shrink potential.

Further, the resilient modulus (M_r) of representative samples was compared prior to and after treatment separately with 6% hydrated lime and EcSS-3000. The purpose of measuring the M_r of the soils was to analyze the moisture susceptibility of the soils and to study the effects of subgrade stabilization on performance of typical pavement systems against the common distresses using the ME-PDG software tool. Also, the contribution of the expansive soils to pavement roughness was measured in terms of loss of serviceability (ΔPSI) using the measured M_r and vertical movement values. The

analysis indicate a significant reduction in drop of M_r values of the lime and EcSS-3000 treated soils and a marked improvement in cracking and subgrade rutting characteristics of the pavements. An average reduction in ΔPSI of the pavements by 0.2 to 0.3 points was observed on the stabilized soils.

This study on expansive subgrades and the associated effects of ionic stabilization have yielded the information necessary as guidance for dealing with relevant engineering problems due to expansive soils.

DEDICATION

To my father, N.Hariharan,

and to my mother, L.Jayanthi,

for their kind support, love and patience.

ACKNOWLEDGEMENTS

I take this opportunity to express my sincere thanks to my academic advisor and mentor, Dr. Dallas Little for his guidance, encouragement and kind co-operation amidst his busy schedule. I consider myself blessed and fortunate to have had the opportunity to work with Dr. Little and will cherish these memories for the rest of my life.

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1. INTRODUCTION

1.1 BACKGROUND

Countries all over the world have reported numerous engineering problems due to the presence of expansive clay soils. These problems include occurrence of various distresses and damages to pavement structures resulting in huge monetary loss. There is an abundance of expansive soils throughout Texas primarily due to the deposition of clay minerals (smectite) along the Central sea bed as a consequence of reaction between the volcanic emissions from the Pacific with the sea water several thousand years back. Expansive soils are characterized by their property to expand and swell upon wetting and shrink upon drying. This susceptibility to variations in moisture results in frequent volume changes. As the soils dry out naturally after a period of wetting, the difference in moisture energy results in formation of cracks which further aid in penetration of water, and thereby continuing the volume change cycle. This swell-shrink process leads to change in the vertical height of the subgrade soils causing structural damages to pavements. Hence, an important component of the successful design of pavement structures in Texas is the assessment and control of the total vertical movement (swelling and shrinkage) and accounting for their contribution to the accumulation of pavement roughness.

1.2 PROBLEM STATEMENT

As previously stated, an important component of the successful design of flexible pavement systems in Texas are the assessment and control of the vertical change in height of the subgrade soils due to swelling upon wetting and shrinkage upon drying.

Texas Department of Transportation's Potential Vertical Rise (PVR) method (Tex-124-E 1999) is the standard and most commonly used approach used to measure vertical movement (Naiser 1997). However, the PVR approach only predicts rise of a "column" of soil and ignores the shrinking component, considering only the movement resulting due to swelling. Also, the PVR is an empirical approach and the accuracy of the estimates obtained has been critiqued as being overly conservative, by Lytton et. al (2004). Hence there is need for using a more comprehensive suction based approach for predicting the movement in soils. For this purpose, the pressure plate test is performed in this study to track the volume change in soils upon subjection to wetting-drying cycles. Recent research studies have indicated that the suction compression index, γ_h, is the parameter that most significantly influences the vertical movement in soils. The ASTM (2001) D2325 standard, upon which the pressure plate test protocol is based on, recommends measuring the volume change in soils at the end of the 0.5 bar suction cycle and after the 15 bar cycle. However, there is need for more accurate laboratory measurement of γ_h values corresponding to different levels of suction ranging between 0 to 15 bar. This modification has been implemented in the experiments performed for this study, thereby resulting in more precise predictions of vertical movement.

The most common practice to address the problem of swelling and shrinking soils in recent years has been to stabilize the subgrade layer using lime or cement. However, there are issues with using lime and cement due to reflection cracks developing in the upper layers as a result of water drawn from the upper layers during the hydration process. Hence, a cheaper and eco-friendly ionic stabilizer 'EcSS-3000' has been tested in this study to monitor the performance of the expansive subgrades with respect to cracking, rutting, moisture susceptibility and strength in terms of resilient modulus apart from analyzing its effect on controlling the swell-shrink potential of the soils and the associated contribution to pavement roughness.

1.3 RESEARCH TASKS

This section outlines and summarizes the research tasks performed during this study. These tasks were as follows:

1. Use of a refined pressure plate protocol to determine the soil-water characteristic curves (SWCCs) for Natural and EcSS-3000 treated soils and from these SWCCs determine incremental γ_h values to be used in the calculation of swelling and shrinkage of the clay soils within the active zone as predicted by the Mitchell equation (1979).
2. Establish a database of γ_h values representative of the SH 130 corridor and analyze the impact of EcSS-3000 treatment on controlling $\gamma_{h.}$
3. Use the repeated load triaxial test to measure and compare the M_r of the natural, lime treated and EcSS treated soil samples and test the sensitivity of M_r to moisture variations before and after stabilization.

4. Analyze the performance of typical flexible pavement systems for common distresses using ME-PDG and establish the improvements as a result of stabilizing the subgrade soils.
5. Devise a spreadsheet for the calculation of swelling and shrinkage of the soils within an active zone defined in terms of depth and fluctuation in suction values according to the Mitchell equation and suggest material and calibration parameters for the spreadsheet.
6. Using the results from item 2 and 3, predict the accumulation of roughness in the pavement structure as the result of the magnitude of swelling and shrinkage and subsequently evaluate the impact of changes in any and all parameters related to the swell, shrinkage and roughness through a comprehensive sensitivity analysis using the spreadsheet.
7. Analyze and reflect on the overall impact of ionic stabilization using EcSS-3000 treatment on controlling vertical movement and roughness in expansive soils and improving the overall response of flexible pavement systems to distresses. .

1.4 THESIS OUTLINE

This thesis is organized in five sections as subsequently described. Section 1 presents an introduction that includes background information, problem statement and research methodology. Section 2 presents a literature review of relevant topics including a review of the existing methods to predicting vertical movement and pavement roughness, the role of suction in the study and some review of the stabilization mechanisms. Section 3 outlines the test procedure, sampling protocol and significant findings from the various experimental tests conducted. The section mainly focuses on providing information about the pressure plate test and the resilient modulus test. Section 4 presents details of analysis performed using the experimental results. This section describes the two major analysis tools used in this study, namely the spreadsheet and ME-PDG and the results of the parametric studies carried out using these tools. Section 5 presents the conclusions and recommendations for future work.

2. LITERATURE REVIEW

This section includes a summary of the necessary background information required to meet the objectives and understand the limitations of the study proposed. The topics covered in this section include a review of the assumptions and limitations in the standard approach to measuring vertical movement, the salient characteristics of the suction based approach proposed by Lytton et. Al (2004), the relationship between suction profile and crack growth and also a review of the stabilization mechanisms for controlling vertical movement, the significance of the subgrade resilient modulus and the methods to predict pavement roughness in terms of loss of serviceability.

2.1 METHODS TO PREDICT VERTICAL MOVEMENT IN SOILS

Methods of predicting movement in expansive soils can be divided into three categories: oedometer, empirical, and suction methods. According to Hong et al. (2006a), the oedometer method uses consolidation theory in reverse and studies show that this theory is conservative and over predicts the movement in soils in most cases except in places having a high water table. Other empirical methods are based on correlating the soil characteristics based on laboratory experiments and field measurements that may not be valid for non-localized data sets or when extrapolated beyond the data base. The PVR method developed in the 1950s by McDowell (1956) is one of these empirical methods. The Texas A&M University (TAMU) suction-based method has the benefit of using the moisture energy (suction) for predicting the swell and shrinkage of soils (Hong et al., 2006a). This approach hence evaluates the vertical movement in expansive soils by considering the effects of both moisture and the mechanical stresses that surround the soil.

Texas Department of Transportation's Potential Vertical Rise (PVR) method (Tex-124-E) is the standard and most commonly used approach to assess pavement roughness. However, the PVR approach does not directly assess roughness but rather predicts rise of a "column" of soil.

In Texas Department of Transportation Report 0-4518, Lytton et al., 2004, the authors identify in substantial detail, the limitations in calculation of swelling or PVR based on the assumptions associated with the PVR method, TEX-124-E, and these limitations as described in 0-4518 are:

1. Access to capillary water to the soil at all depths.
2. Empirical conversion of volumetric strain to vertical strain by a factor of one-third.
3. Neglecting the use of a shift factor to approximate the laboratory compacted soils to in situ soil condition.
4. Direct estimation of ride quality based on the potential vertical ride value.
5. Use of Atterberg limits to estimate volume changes in soil.

Lytton et al. (2004) challenge assumption 1 based on the restraint of swell due to overburden and the non-uniform rate of water flow at greater depths. They ultimately refer to the utility of the calculation of depth of the moisture active zone and suggest a suction based approach taking into account the variation in moisture energy as a function of both time and depth of the active zone. The equation proposed by Mitchell (1979) forms the basis of this approach.

Lytton et al. (2004) challenge the second assumption as unreasonable at greater depths since the effect of the high confining pressure overpowers the effects of swelling. This leads to under prediction of vertical strain based on the PVR method.

Lytton et al. (2004) challenge assumption 3 based on observations that in situ, undisturbed soils are distinctly different from remolded soils and that their swell properties are impacted by soil fabric, including presence of roots, wormholes, cracks, etc. However, Lytton et al. (2004) ultimately state that volume change predictions can be made based on some basic soil properties like Atterberg limits, percent of soil particles smaller than 75 μm, and the percent of soils smaller than 2 μm.

Concerning assumption 4, Lytton et al. (2004) refer to a study over 3 to 15 years that indicates that the sum of shrinkage and swell calculated movements at a certain depth is affected more by the rate of increase in roughness rather than the pavement roughness itself. They found that the difference between the "bump height" associated with a typical beginning serviceability index (4.0) and a typical terminal serviceability index (2.5) is only between 1.0 inches and 0.5 inches.

In summary, Lytton et al. (2004) identify assumptions 1 through 5 of the TEX-124-E as unrealistic or inadequately validated with actual measurements.

Lytton et al. (2004) recommend a suction based method, which is a result of several decades of work at the Texas Transportation Institute (TTI), to replace the PVR method.

This new method contains key features that include:

1. Determination of moisture-active zone depth limited by the moisture diffusivity properties of the periodic moisture-suction variations driving shrink-swell processes controlled by local climate, vegetation, and conditions of drainage.
2. Recognition of the impact of mechanical stress and the existence of cracks in the soils mass on the relationship between soil volume change and vertical deformations.
3. Ability to predict both shrinkage and swell of subgrade soils.
4. Consideration that both vertical deformations together with design traffic loads influence the time history of pavement performance, and this predicted history of roughness and serviceability should be used as a design basis.

2.2 REVIEW OF ESTIMATION OF LOSS OF SERVICEABILITY: AASHTO 1993 APPROACH

Appendix G of the American Association of State Highway and Transportation Officials (AASHTO) Guide for Design of Pavement Structures, 1993, outlines a simplified method to assess damage in the form of loss of serviceability due to swelling subgrade soils. The form of the relationship that defines loss of serviceability $\Delta PSI_{swelling}$ is presented in equation (1) as:

$$\Delta PSI_{swelling} = 0.00335 \cdot V_R \cdot P_s \cdot \left(1 - e^{-\theta t}\right) \tag{1}$$

where, V_R is potential vertical rise, P_s is swell probability, θ is swell rate constant, and t is time in years.

2.2.1 Potential Vertical Rise, V_R

In Appendix G of the AASHTO Guide, Figure G.3 provides guidance in estimating the potential vertical rise based on plasticity index (PI), moisture condition, and thickness of the layer in question. This figure is reprinted below as Figure 1.

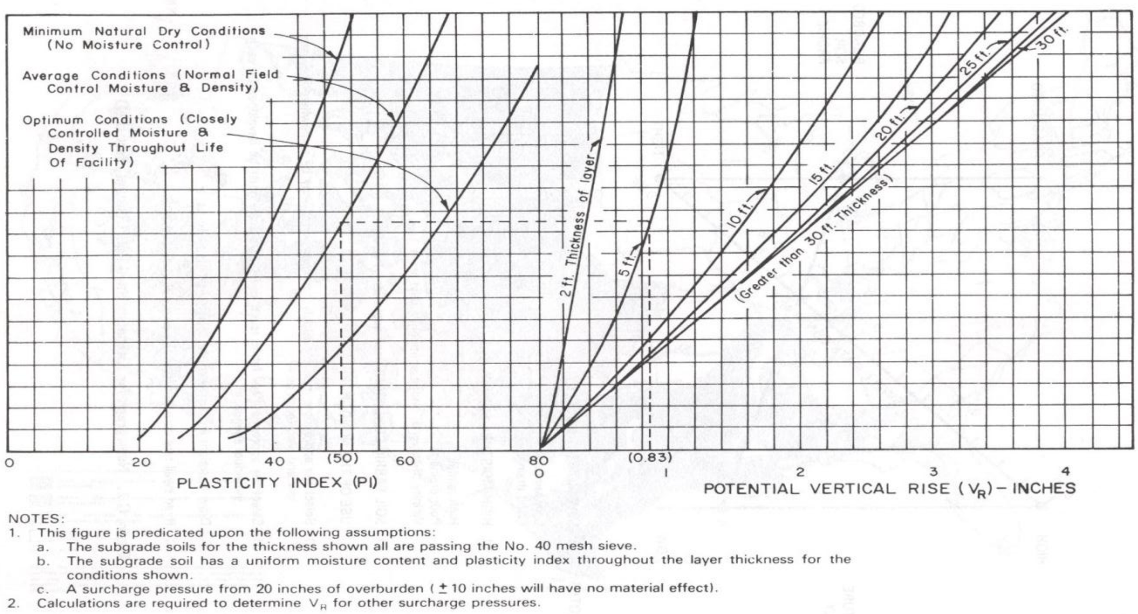

Figure 1: Chart for Estimating the Potential Vertical Rise of Natural Soils. (Produced from AASHTO Guide for Design of Pavement Structures, 1993.)

2.2.2 Swell Rate Constant, θ

The swell rate constant usually ranges between 0.04 and 0.20 and gives a measure of the rate at which swelling occurs. A higher swell rate constant indicates more potential for the soil to swell and is generally typical of soil exposed to rainfall or other sources of moisture. Appendix G.2 of the AASHTO Guide (1993) provides the relationship to estimate the swell rate constant. However, this chart is very subjective. A less subjective method is to assume that a water supply is available at the site. This is reasonable as at some point in the year, for pavements along the SH 130 corridor or in the Dallas – Fort Worth area, a water source is likely to be available to the soils. Given an available water source, the next step is to consider the fabric of the roadbed soil, which may vary from highly fractures and desiccated to wet and tight. The empirical relationship used in TEX-124-E to estimate the moisture state of clay soils can be used to take out some of the subjectivity of the estimate of the nature of the soil fabric. If the in situ moisture content is less than the dry condition in TEX-124-E, i.e., moisture content < 0.2 (LL + 9), then the swell rate constant should be greater than 0.16 but less than 0.20 depending on how much less the water content is than 0.2 (LL + 9). If, on the other hand, the in situ moisture content is equal to or above the wet condition, i.e., moisture content > 0.47 (LL + 2), the swell rate constant should be less than 0.08, but not less than 0.04. Values of moisture content between the dry and wet values should be used to judge swell rate constant values between 0.08 and 0.16.

2.3 ROLE OF SUCTION IN IMPACTING VERTICAL MOVEMENT

Figure 2 is reproduced from Hong et al. 2006b. It illustrates the impact of considering features such as vertical cracks and how they impact the suction profiles. For example consider the Fort Worth suction profile, which when constructed from the Mithchell equation is symmetric about the equilibrium suction value, U_e. However, in the case of Atlanta, Texas, U.S. 271, a high suction is maintained to a depth of over 10 feet due to a vertical crack. One can clearly see that the impact of wetting from the 4.5 pF suction profile (the drying suction due to the deep vertical crack) over a depth of over

10 feet, will clearly lead to more volume change than transitioning from the equilibrium dry profile to the equilibrium wet profile over the same depth. Figure 2 illustrates the impact of equilibrium suction or suction profile at a given point in time on the potential for vertical swell or shrinkage. Other factors also obviously impact the vertical changes and will be discussed in detail in the forthcoming sections of the thesis. These include the diffusion coefficient, α; the number of annual cycles of wetting and drying, n; and, of course, the suction compression index, γ_h and the mean principal stress compression index, γ_σ.

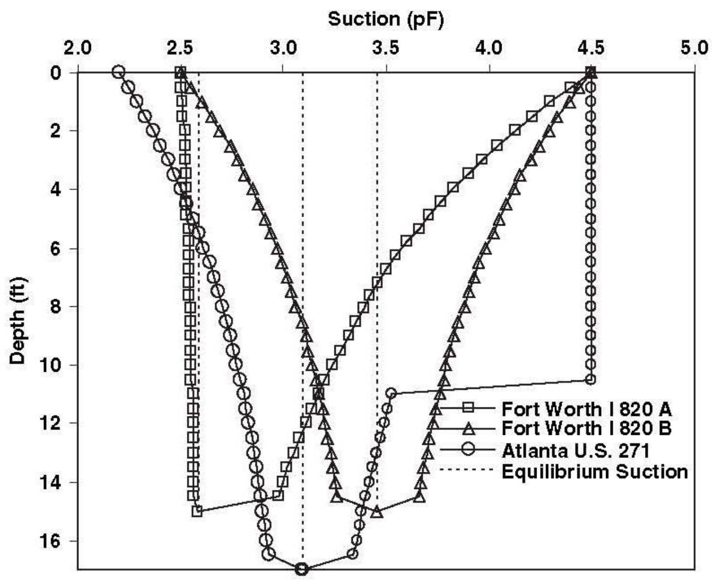

Figure 2: Typical Suction Profiles. (After Hong et al., 2006b)

2.4 CONTROLLING VERTICAL MOVEMENT IN SUBGRADE SOILS

The subgrade is the lowermost layer in a pavement system. The subgrade is usually composed of native soil which often lacks the required strength to support heavy traffic loading. Information about the characteristics of the subgrade soil under the local climatic and moisture conditions is essential prior to the construction of a new pavement. The long-term performance of pavements is governed by the strength and stiffness of the materials used and hence it is critical to optimize the performance of the subgrade by making efficient use of the existing subgrade materials. In the event of encountering soft and wet subgrades, proper stabilization techniques need to be employed to improve the engineering properties of the weak subgrade soil (Dhakal 2012). The choice of stabilizer depends on the type of soil at the location in consideration. The SH130 corridor has vast deposits of highly plastic clay soils. Lime and EcSS-3000 are two popular soil stabilizers for treating expansive clays and hence are investigated in this study for potential use on the SH130 subgrade soils.

2.4.1 Stabilization of Subgrade Soils

Soil stabilization is a common technique employed to improve the strength and moisture stability of subgrade soils.

2.4.1.1 Lime Stabilization

Lime has been proved to be one of the most effective stabilizers for a wide range of soils. Studies have shown that the strength of most fine-grained soils improves significantly with lime stabilization due to the cation exchange reactions that occur in the lime-soil system followed by flocculation-agglomeration (Little 1987). Further, lime stabilization is extremely effective on highly plastic soils having a high swell-shrink potential (Little 2000). Hence, for several decades, expansive clay soils have been treated with lime to improve their modulus and other strength gain properties.

2.4.1.2 EcSS-3000 Stabilization

EcSS-3000 is a relatively newer product and is marketed as an environment friendly chemical stabilizer primarily for stabilizing expansive clay soil beneath new and existing pavement structures. EcSS-3000 is an ion exchange solution that is composed of a mixture of sulfuric acid and naphthalene. The manufacturers of the product recommend diluting one part of EcSS-3000 with three hundred parts of water (1:300) before injecting it into the soil system. EcSS-3000 reduces the absorbed water attached to the clay particles by leaching out the aluminum atoms responsible for the overall negative charge of clay. This mechanism reduces the shrink-swell potential of the soil and results in an increase in strength due to soil densification. (ESSL, LLC).

2.5 ESTIMATION OF SUBGRADE RESILIENT MODULUS

Resilient modulus of the subgrade soil is an important property that characterizes the performance of flexible pavements. Pavement design guides, including the 1993 AASHTO Pavement Design Guide and the Mechanistic-Empirical Pavement Design Guide (ME-PDG 2000) describe design methodologies that use resilient modulus as the main input criteria for the subgrade. The advantage with using resilient modulus as the property to characterize subgrades is the fact that it can be used for analysis of distresses and other mechanical properties (Masada et. al 2006). There exist numerous methods, both in the field and laboratory, to measure the resilient modulus of subgrade soils in flexible pavements. The laboratory methods using the repeated loading triaxial (RLT) test, measure resilient moduli of tested materials whereas the field non-destructive methods use back calculation approaches to arrive at an estimate of the resilient modulus

(Nazzal et. al. 2010 and Puppala 2008). While the Falling Weight Deflectometer (FWD) test is used frequently to estimate the resilient modulus of subgrades under existing pavement systems primarily because it is non-destructive in nature, the highest level input (Level 1) in the ME-PDG requires laboratory measurement of resilient modulus of candidate subgrade soils recovered from and representative of the actual project site in question.

3. EXPERIMENTAL PROCEDURE

This section provides details about the experimental tests conducted during the study and a discussion of the results obtained. The first part of this section describes the pressure plate test and its associated results and significant findings. The second part of this section provides details about the resilient modulus test conducted on the natural and stabilized soils.

3.1 PRESSURE PLATE TEST

The Pressure Plate test was used to establish the Soil Water Characteristic Curve (SWCC) and subsequently estimate the γ_h values corresponding to swelling and shrinking of the soil sample. SWCCs describe the relationship between the suction levels and moisture content of the soil.

3.1.1 Theory

The pressure plate test controls the matric suction applied on the soils. The matric suction is defined as the difference between the air pressure and the water pressure. In this technique, the air pressure is increased from 0 to 1500 kPa while the water pressure is kept equal to the standard atmospheric pressure. With increasing pressure levels, the pressure plate test simulates a cycles of shrinking of the soils in the field. A high air entry porous ceramic plate is used to draw the moisture lost from the samples with increasing pressure levels. The porous ceramic plate scores over the conventional porous stone due to its ability to generate high levels of matric suction and maintain pressure conditions without cavitation of water. Typically the ASTM (2001)

D2325 standard recommends five levels of pressure between 0-1500 kPa. The time to reach equilibrium at the respective pressure cycles varies between 24 to 96 hours and increases at higher pressures. Up to six samples can be placed in the pressure plate test cell simultaneously. The weights of the samples are recorded at the end of each of the pressure cycles to aid in calculating the water content and hence establish the soil water retention curves of the samples. The pressure plate test apparatus with the soil samples is shown in Figure 3.

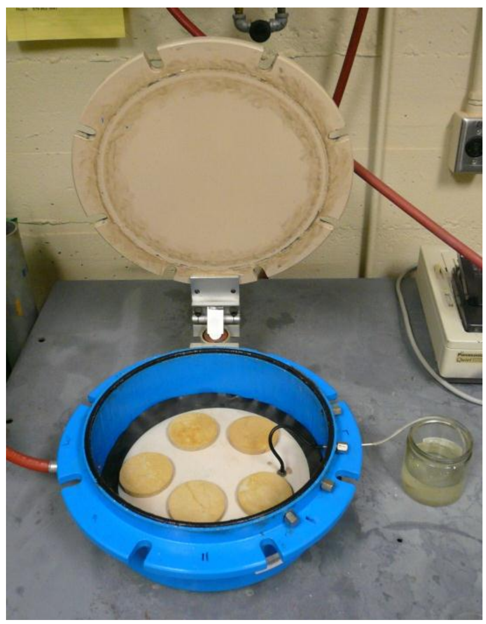

Figure 3: Pressure Plate Test Apparatus

3.1.2 Sampling and Testing Protocol

The candidate soils were extruded from standard Shelby tubes. The dimension of the samples prepared for the test was 3”$\times$ 0.4" (diameter $\times$ thickness). All the ten candidate samples were tested separately in their natural states and after treatment with a 1:300 EcSS-3000 solution. The trimmed samples were then placed on the pressure plate test chamber after placing them in a 100% RH setting for 96 hours prior to the test, to ensure maximum saturation. The pressure plate tests were performed in accordance to ASTM D2325 under five matric suction levels between 50 and 1500 kPa. The samples were weighed after completion of each of the pressure cycles to get the corresponding water content and hence establish the SWCC of the soils. A sample of the data sheet used for recording the Pressure Plate test results is provided in Appendix A. The volume of the samples was also recorded after every cycle using the *New Volume Measurement Method* described in the Appendix section of this thesis. Figure 4 and Figure 5 respectively shows the SWCCs of the samples before and after treatment with EcSS-3000.

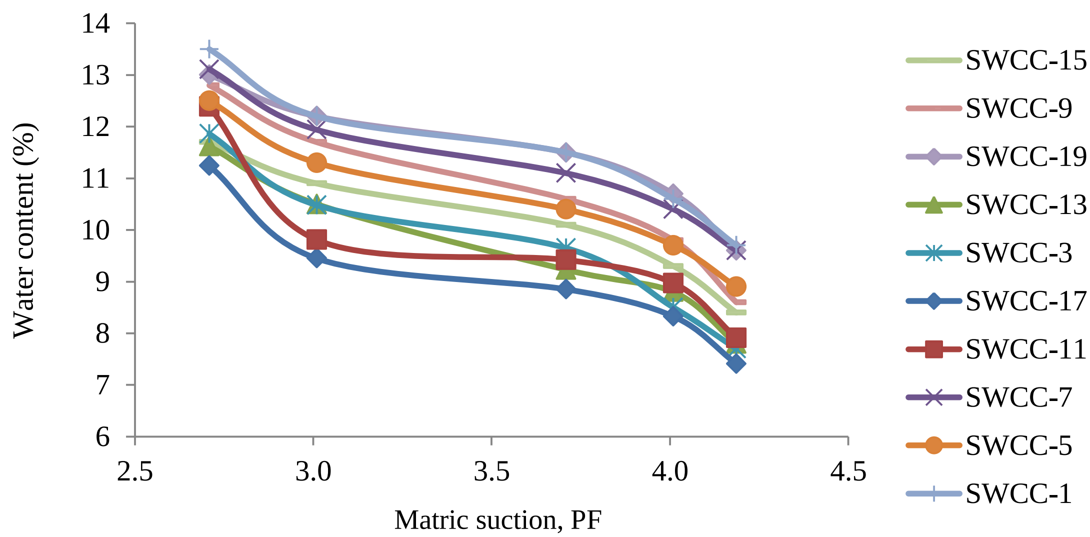

Figure 4: SWCCs of Untreated Soil Samples

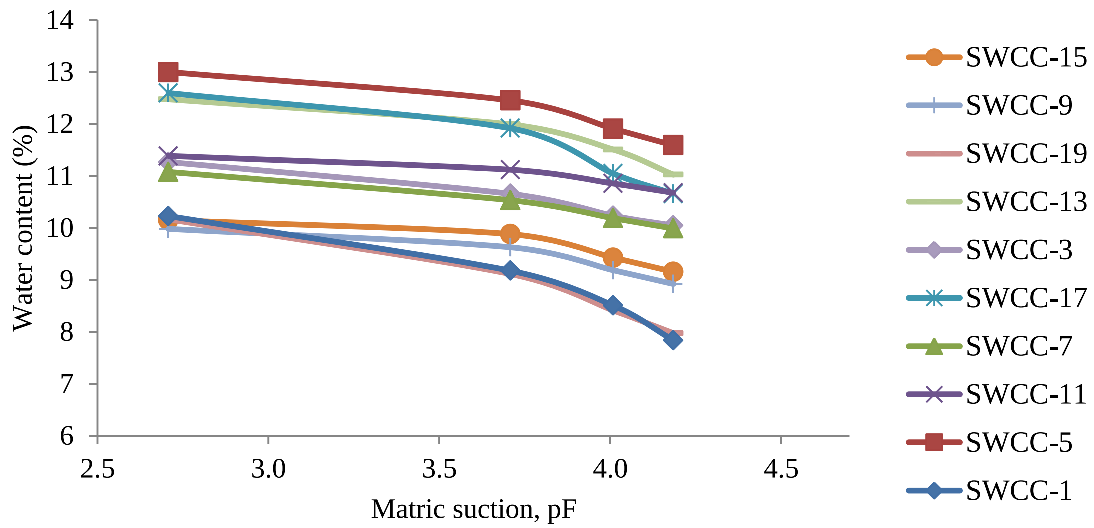

Figure 5: SWCCs of Soil Samples after treatment with EcSS-3000

The water content v. suction curves in Figure 4 and Figure 5 illustrate the relationship between the suction applied to the samples in the pressure plate test and the amount of water the sample is able to retain under that pressure before and after treatment with EcSS-3000. Since the level of saturation of each sample at the beginning of the test varies among the samples, the review of the data in the SWCC's must be accomplished considering this fact (Houston et.al 2006). For example, most of the higher PI soils are positioned higher on the plot and the lower PI soils are positioned lower on the plot. This is because the higher fines and high PI soils normally hold more water. However, based on the unique conditions that the soil sample may have be subjected to before and during extraction and test preparation, this initial moisture content will be expected to vary. However, one would expect that soils with a higher PI and higher fines content would be able to retain more water over the range of suction pressures applied during testing. Therefore, not only the position of the curve on the plot but also, and more importantly, the slope of the water content v. suction curve, γ_h defines the nature of the soil.

3.1.3 Protocol to Determine γ_h

Once the SWCC of the soil sample is established and the volume of the samples are collected at the desired suction levels usually ranging from about pF 2.5 to 4.2, the suction change $\Delta pF = (pF_{final} - pF_{initial})$ and the volumetric strain ($\Delta V/V$) for the corresponding suction change can be determined. Incremental γ_h values can then be determined separately for swelling and shrinkage based on equation (2):

$$\gamma_h = \frac{\frac{\Delta V}{V}}{\Delta pF} \tag{2}$$

where, V is the volume at pF_{final} for swelling calculations and the volume at $pF_{initial}$ for shrinkage calculations. A sample calculation for γ_h is provided in Appendix B.

3.1.4 Results and Discussion

Table 1 summarizes the results of pressure plate testing conducted following the protocol described in the previous section. All the ten samples tested were obtained from borings along Segments 5 and 6 on the SH 130 corridor. The samples were reported as being taken from two sources; Cut and Embankment and had a plasticity index ranging between 45 and 64. The ten samples were identified with alternate numbers from 1 through 20.

It is to be noted that the γ_h values for swell and shrink are nearly the same and no major change is observed. However, a comparison of γ_h values of the soil samples before and after treatment with EcSS-3000 makes interesting reading. It can be said from Table 1 that the mean γ_h of the natural soils for the three suction intervals between 0 and 15 bar are respectively 0.035, 0.063 and 0.051 while these values are reduced to 0.019, 0.038 and 0.031 after treatment with EcSS-3000. This amounts to a significant average reduction in the γ_h of roughly 40% across suction increments. A good degree of consistency was observed in the results of the ten tested samples. It has been previously stated that γ_h the single most influential parameter in the calculation of soil vertical movement and the significance of 40% drop in γ_h after treating the soils with EcSS can be appreciated in the sensitivity analysis study that follows later in this report. Table 1 hence established the database of γ_h values representative of the soils in the SH 130 corridor.

Table 1: Summary of Atterberg Limits of Shelby Tube Samples and γ_h Results

Source	Sample I.D.	PI (%)	Pressure (bar)	Natural Soil		EcSS Treated Soil	
				γ_h (swell)	γ_h (shrink)	γ_h (swell)	γ_h (shrink)
Cut	15	62	0-5.0	0.038	0.037	0.011	0.011
			5.01-10.0	0.052	0.052	0.035	0.034
			10-15.0	0.045	0.045	0.027	0.027
	9	50	0-5.0	0.033	0.032	0.009	0.008
			5.01-10.0	0.078	0.076	0.023	0.023
			10-15.0	0.064	0.064	0.021	0.020
	19	56	0-5.0	0.045	0.043	0.022	0.021
			5.01-10.0	0.068	0.066	0.037	0.036
			10-15.0	0.094	0.092	0.044	0.043
	13	54	0-5.0	0.023	0.023	0.015	0.015
			5.01-10.0	0.076	0.074	0.035	0.035
			10-15.0	0.056	0.056	0.045	0.045
	3	60	0-5.0	0.034	0.033	0.009	0.009
			5.01-10.0	0.045	0.045	0.019	0.019
			10-15.0	0.050	0.050	0.018	0.018
Embankment	17	48	0-5.0	0.042	0.043	0.008	0.007
			5.01-10.0	0.064	0.063	0.016	0.016
			10-15.0	0.045	0.045	0.032	0.032
	7	45	0-5.0	0.033	0.032	0.007	0.007
			5.01-10.0	0.078	0.076	0.022	0.022
			10-15.0	0.064	0.064	0.020	0.020
	11	64	0-5.0	0.045	0.043	0.004	0.004
			5.01-10.0	0.068	0.066	0.011	0.011
			10-15.0	0.094	0.092	0.016	0.016
	5	58	0-5.0	0.023	0.023	0.011	0.011
			5.01-10.0	0.076	0.074	0.019	0.018
			10-15.0	0.056	0.056	0.028	0.028
	1	52	0-5.0	0.034	0.033	0.010	0.010
			5.01-10.0	0.045	0.045	0.030	0.030
			10-15.0	0.050	0.050	0.033	0.033

3.2 RESILIENT MODULUS TEST

Resilient modulus in the laboratory is most commonly estimated using the triaxial test method by applying repeated loading for several cycles. This test is easily repeatable and has been known to produce reasonably accurate results (George 2004). The test is performed in the laboratory in accordance with the AASHTO (1999) T307 standard which outlines the procedure to determine the resilient modulus of subgrade soils.

The axial recoverable strain (resilient strain, ε_r) of the specimen tested is measured after which the resilient modulus is calculated using equation (3) as:

$$M_r = \frac{\sigma_d}{\varepsilon_r} \tag{3}$$

where, σ_d is the deviatoric stress.

3.2.1 Laboratory Testing Program

The candidate soil samples obtained from the SH130 corridor were processed and subsequently compacted to obtain specimens of desired dimensions of 150 mm height and 150 mm diameter. The IPC 100 mm Rapid Triaxial Tester (RaTT) cell attachment was fit into a UTM25 hydraulic testing frame to record the measurements. The axial strain values of the specimens were recorded by locating two internal LVDTs at the top and bottom of the test cell. Internal LVDTs were preferred to help reduce the possible errors in measurement due to external deformations. The specimens were subjected to a constant confining pressure by applying vacuum. Figure 7 shows the test equipment and setup used. In accordance with the AASHTO (1999) T307 standard, the samples were first pre-conditioned by applying 1000 loading cycles, primarily to avoid surface irregularities. This step would also ensure suppression of the initial unrecoverable deformation (Soliman et. al 2010). Following the pre-conditioning of the samples, 100 cycles of load are applied in the form of a haversine shaped load pulse at a constant confining stress (V_c) of 41.4 kPa. The resilient modulus was measured at a single deviatoric stress level of 41.4 kPa. The load pulse used in this study had a 1.5 sec load duration and 1.5 sec rest period. The deformation response observed during the final ten cycles of the loading sequence for the HC1 soil is shown in Figure 6.

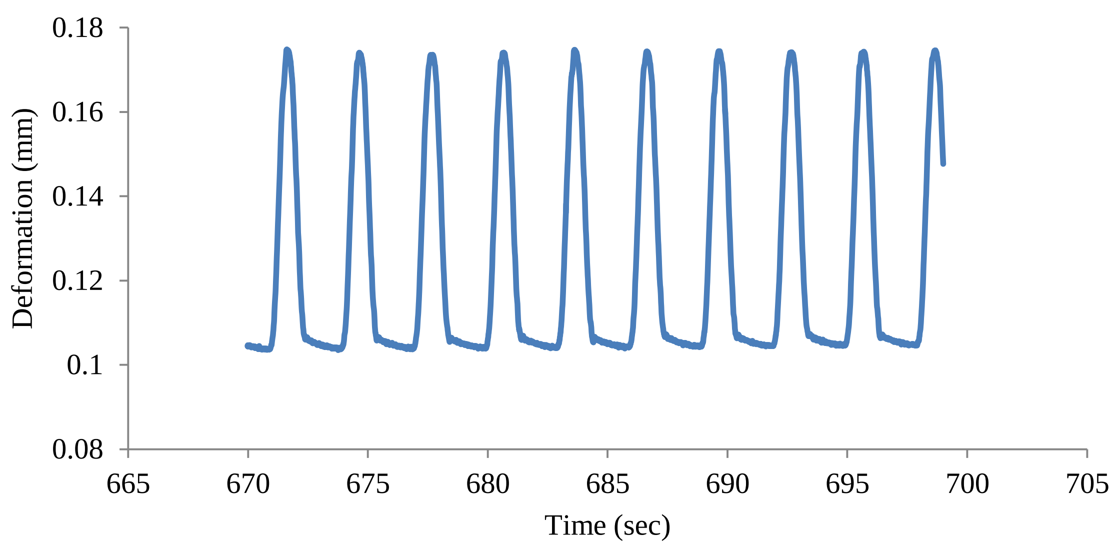

Figure 6: Deformation Response during Load Cycles 91 to 100

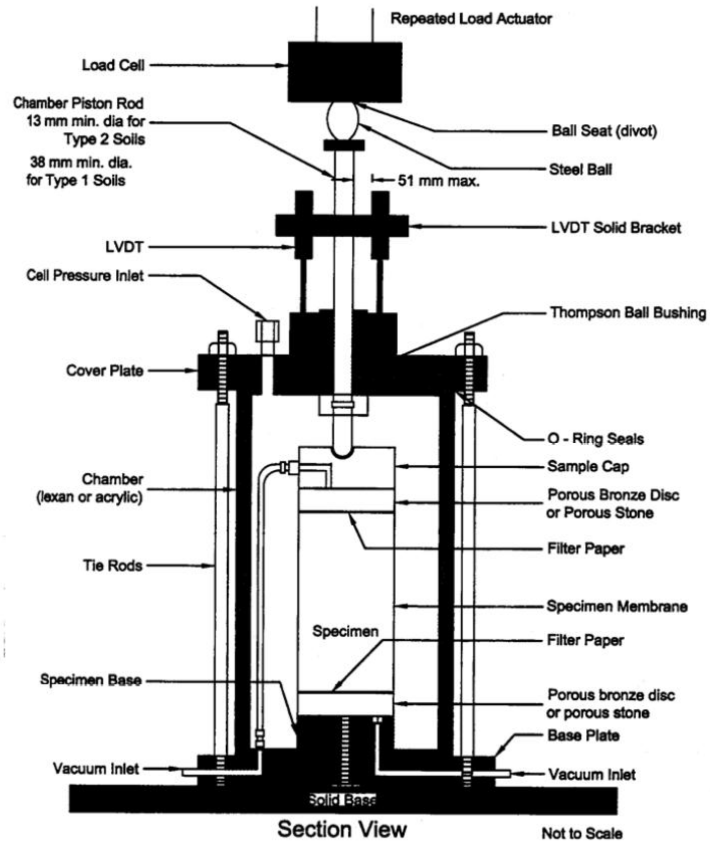

Figure 7: Schematic of the Triaxial Chamber (Produced from Elias et. al., 2006)

3.2.2 Sampling Protocol

The M_r tests were conducted on specimens obtained from Shelby tube samples collected in the field. The repeated load triaxial test was performed on three candidate soil samples, each of them investigated in their natural condition and separately after treatment with 6% hydrated lime and a 1:300 diluted EcSS-3000 solution. The soils were treated with the stabilizers at the mentioned concentrations after crushing and drying the as-received Shelby tube samples. They were then compacted using a 150 mm base at the required moisture contents in three lifts each of 50 mm according to the modified

Proctor test (ASTM 2009 D1557). Finally, the 150 mm x 150 mm soil samples were each tested at three moisture levels; at optimum moisture content (OMC), 2% below OMC and 2% above OMC. The plasticity index and the respective moisture contents used for testing the three samples, re-identified as heavy clays (HC1, HC2 and HC3) and corresponding to samples #15, #13 and #17 respectively, are specified in Table 2. Each of the separate tests was replicated thrice to ensure consistency in results.

Table 2: Soil Sample Classification and Moisture Contents for M_r Tests

Sample ID	Plasticity Index(PI)	OMC (%)	Moisture Content for M_r Tests (%)
HC1	62	26.4	24
			26
			28
HC2	54	22	20
			22
			24
HC3	48	19.4	17
			19
			21

3.2.3 Results and Analysis

A summary of the repeated load triaxial test results conducted on the three soils investigated as received and after stabilization are shown in Table 3. The test was conducted on the specimens compacted at γ_d max and at the moisture contents specified in Table 2.

Table 3 shows the mean M_r values with the respective variances and standard deviations. It is to be noted that each of the soils were tested for three replicates in accordance to the AASHTO (1999) T 307 procedure. The modulus values were computed based on the strains recorded in cycles 91-100 of the respective test sequences. The coefficient of variations on the sample measured ranged between 0.15 and 1.07 % as shown in Table 3. In general, the samples showed a fair deal of consistency in measurements.

Figure 8 presents a representation of the results in Table 3. Inspection of Figure 8 reveals that all the three highly plastic clay soil samples, HC1, HC2 and HC3 show significant sensitivity to moisture content variation on either side of their respective optimum moisture states. The resilient modulus of all the soil samples expectedly decreased on moving from a dry to wet state.

The resilient modulus of the HC1 soil decreased from 85.49 MPa to 36.54 MPa, a drop of 57%, upon increasing the moisture content from 24.0% to 28.0%, while the resilient modulus of the HC2 and HC3 soils dropped by 52% and 50% respectively on moving 2% wet off their optimum moisture content.

The magnitude of increase in resilient modulus upon testing at 2% below the optimum moisture content was 31%, 35% and 21% respectively for the three soils HC1, HC2 and HC3.

Table 3: Summary of Resilient Modulus Test Results at V_c =6 psi and σ_d=6 psi

Sample I.D.	Moisture State	M_r (MPa)- Natural			M_r (MPa)-Lime Stabilized			M_r (MPa)-EcSS-3000 Stabilized		
		Mean	**SD**	**CV**	**Mean**	**SD**	**CV**	**Mean**	**SD**	**CV**
HC1	Dry	85.49	0.26	0.30	182.69	0.53	0.29	133.05	0.55	0.41
	OMC	65.49	0.40	0.61	123.40	0.62	0.50	77.21	0.72	0.93
	Wet	36.54	0.36	0.99	113.06	0.65	0.57	67.56	0.38	0.56
HC2	Dry	112.37	0.23	0.20	190.96	0.48	0.25	138.57	0.24	0.17
	OMC	83.42	0.12	0.14	147.53	0.67	0.45	98.58	0.76	0.77
	Wet	53.77	0.32	0.60	108.93	0.33	0.30	78.59	0.39	0.50
HC3	Dry	104.10	0.43	0.41	174.42	1.03	0.59	136.50	0.87	0.64
	OMC	86.18	0.31	0.36	133.05	1.42	1.07	104.79	0.68	0.65
	Wet	51.71	0.28	0.54	108.24	0.78	0.72	75.14	0.49	0.65

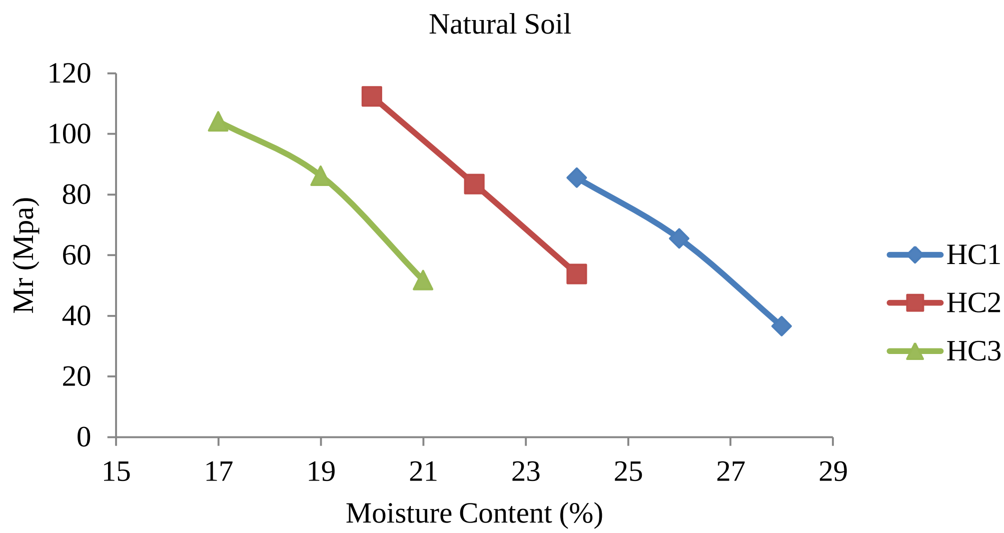

Figure 8: Sensitivity of M_r of the Untreated Soils to Moisture Variation

Figure 9 shows a comparative picture of the sensitivity of the HC1 soil to moisture variations upon stabilizing with lime and EcSS-3000 with respect to the natural condition. It can be inferred from Figure 9 that both lime and EcSS-3000 stabilizers improve the resilient modulus of the soils significantly and make it less susceptible to moisture intrusion.

A similar trend was observed in the behavior of the soils HC2 and HC3 upon treatment with the two stabilizers.

At their respective optimum moisture contents, the resilient modulus of the three samples increased by 88%, 77% and 54% compared to the untreated state upon stabilizing with 6% hydrated lime. The increase however, was far less significant with the EcSS-3000 stabilizer compared to lime but still resulted in an average increase in M_r of 20%. The more glaring fact is the impact of the stabilizers on the reduction in modulus drop upon wetting and the increase in modulus hike upon drying. Analysis show that the M_r of the stabilized soils upon wetting is higher compared to the M_r of the undisturbed soils compacted at OMC. Also, treatment with lime results in a mean increase in M_r of 90 MPa while EcSS-3000 results in a 40 MPa increase in M_r when tested 2% below OMC (dry condition).

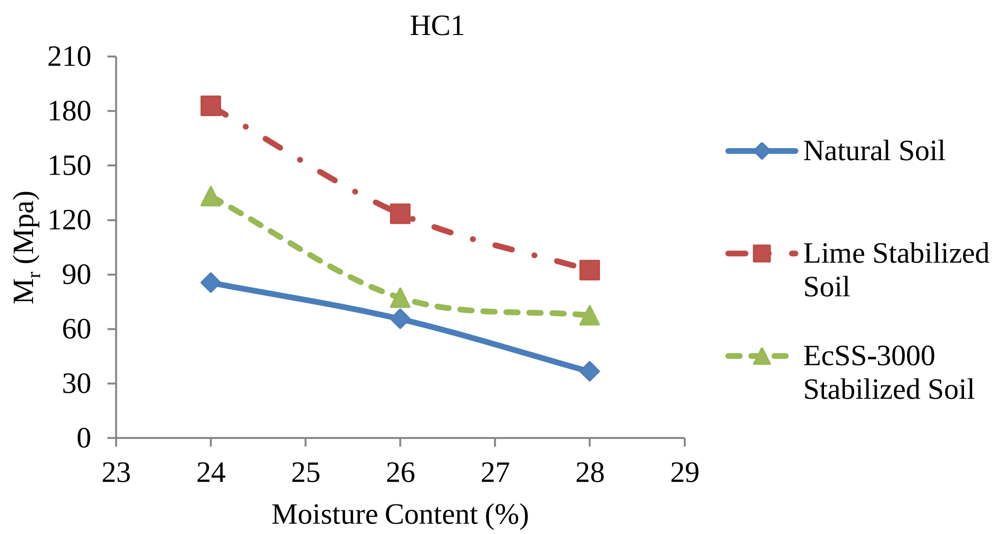

Figure 9: Comparison of M_r Sensitivity to Moisture upon Stabilization

The results indicate that lime stabilization has a higher influence on the resilient modulus and subgrade performance of the investigated soil samples compared to the EcSS-3000 stabilizer. However, the M_r results observed with EcSS-3000 treatment are considerably better in comparison with the untreated soils. We observe a good reduction in moisture sensitivity of the highly plastic subgrade soils upon treatment with EcSS-

3000, which is critical in minimizing vertical movement (swelling and shrinking) of the expansive soils.

To further elucidate the above discussion, an extrapolated relationship between *log* (M_r/M_{Ropt}) vs. $(S- S_{opt})$ is given in equation (4).

$$\log\left(\frac{M_R}{M_{R_{opt}}}\right)=\left[a+\frac{b-a}{1+\exp\left(\beta+k_s\left(S-S_{opt}\right)\right)}\right] \quad (4)$$

where, a and *b* are respectively the minimum and maximum values of $\log(Mr/M_{Ropt})$ corresponding to wet and dry moisture conditions i.e. 2% above and below OMC as considered in this study. β is the location parameter and is defined as -ln (b/a) while k_s is the regression parameter assumed to be equal to 6.1324.

$(S-S_{opt})$ defines the difference in saturation levels of the soil with respect to an optimum value.

Figure 10 is plotted on the HC1 soil for different $(S-S_{opt})$ values ranging from -75% to 25% in 5% increments by predicting the corresponding values of M_r from equation (4) following the above description.

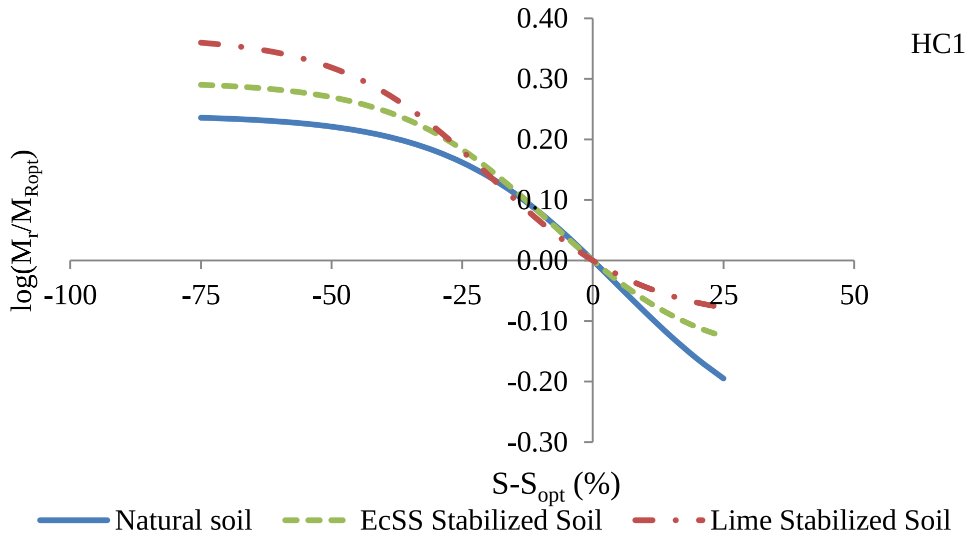

Figure 10: Comparison of Modulus Ratios after Stabilization

Figure 10 confirms and validates the discussion about the impact of subgrade stabilization with lime and EcSS-3000 on improving the resilient modulus and minimizing its sensitivity to moisture variations. It is worthy to note that the drop in modulus ratios with wetting is significantly lower in the stabilized subgrade soils, with lime being slightly more effective than EcSS-3000.

4. DATA ANALYSIS METHODOLOGY

This section provides the theoretical background and approach used to calculate the swell and shrink in the subgrade soils and the pavement roughness in terms of serviceability loss over its design life.

4.1 CALCULATION OF VERTICAL MOVEMENT AND PAVEMENT ROUGHNESS

The first part describes the suction based approach to estimate combined vertical movement in soils due to swelling after wetting and shrinkage after drying. The second part presents the serviceability loss calculation taking into account the effects of traffic and vertical movement, using the sigmoidal equation approach proposed by Lytton et. al (2004).

4.1.1 Basis for Vertical Movement Calculations

The suction based approach to estimate vertical movement of soils begins with the calculation of a suction profile as a function of depth below the surface according to equation (5) developed by Mitchell (1979).

$$U(z) = U_e \pm U_0 \exp\left[-\left(\frac{n\pi}{\alpha}\right)^{0.5} z\right] \tag{5}$$

where U_e is equilibrium suction in pF, which is defined for a specific climatic region; U_0 is the amplitude of variation in suction values, also in units of pF; ‘n’, in units of cycles per year is the number of wetting-drying excursions occurring during the year in question; however, the value of n must be used in units of cycles per second in solving equation (5), cycles per year x 3.17 x 10^{-8}; α is the diffusion coefficient in cm^2/second; and z is the depth of the moisture active zone below the surface of the subgrade layer in cm. The subgrade is divided into incremental depths (30 cm) starting at a depth of 15 and continued to the active zone depth, z. It is to be noted that 'z' and 'U_e' can be estimated based on the Thornthwaite Moisture Index (1948) distribution of the United States. Equation (6) and equation (7) from Abdelmalak et.al (2006) presented below are respectively used in the process of estimating 'z' and 'U_e':

$$z = 1.387 + 0.939exp\left(\frac{-I_m}{24.843}\right) \tag{6}$$

$$U_e = 3.563exp(-0.0051I_m) \tag{7}$$

Once the suction profile is established as a function of depth, U_z, either the equilibrium suction, U_e, or an in situ measured profile of suction v. depth can then be compared to the two extreme conditions calculated from equation (5): the drying profile in which the maximum value of $U_{z(dry)}$ is calculated – higher value of suction and the smallest value of $U_{z(wet)}$ is calculated for the wetting profile. The difference between either the measured or equilibrium suction value at a specific depth and $U_{z(wet)}$ is the energy potential that promotes swell while the difference between the measured or equilibrium suction and $U_{z(dry)}$ at a specific depth is the energy potential that promotes shrinkage. Once suction differences as a function of depth are determined, the volumetric swell is determined form equation (8):

$$\left(\frac{\Delta V}{V}\right) = -\gamma_h \log\left(\frac{U_f}{U_i}\right) - \gamma_\sigma \left(\frac{\sigma_f}{\sigma_i}\right) \tag{8}$$

In equation (8) the γ_h term is the suction compression index and γ_σ is the mean principal stress compression index. . These values may be thought of as compression moduli, and their values are highly variable from one literature source or reference to the other. Values taken from Lytton et al. (2004) for Texas soils are wide ranging and typically vary from about 0.0065 to about 0.070 with typical nominal values of between about 0.010 and about 0.020 for soils with plasticity indices greater than about 40 and clay content greater than about 40 percent.

Lytton et al. (2004) also describe methods for estimating γ_σ based on soil properties such as the compression index, C_c and void ratio. However, calculated values based on these indices resulted in values much larger than the values of $\gamma_{h.}$ Hence, we adopt the approach used by Wray (1978) that $\gamma_\sigma = \gamma_h$.

The values σ_i and σ_f represent an initial value of overburden stress and a final value of overburden stress. The initial value is a constant or reference value, in our case determined as the value of overburden stress at a depth of 80 cm. The value of σ_f is determined as the value at a depth of z_i. The value of σ_f, for example is calculated according to equation (9):

$$\sigma_f = \left[\frac{(1+2K_0)\sigma_z}{3}\right] \tag{9}$$

In which σ_z is the vertical overburden stress (the product of the total unit weight of the soil above depth z_i ($\gamma_{soil,\ zi}$) and the depth, z_i; K_0 is the earth pressure at rest.

The vertical strain $\frac{\Delta H}{H}$ is then computed according to equation (10):

$$\frac{\Delta H}{H} = f\left(\frac{\Delta V}{V}\right) \tag{10}$$

where, f is 0.5 for drying and 0.8 for wetting. Finally, the summation of vertical movement is calculated from equation (11):

$$\Delta H_{total} = \sum_{i=1}^{n} f_i \left(\frac{\Delta V}{V}\right) \Delta z_i H \tag{11}$$

4.1.2 Basis for Pavement Roughness Calculations

The change loss of serviceability in terms of PSI as a function of time is defined by a sigmoidal decay relationship based on Lytton et.al (2004) as:

$$\Delta PSI_t = (PSI_0 - 1.5) exp\left[-\left(\frac{\rho_s}{t}\right)\right]^{0.66} \tag{12}$$

where,

$$\rho_s = A_s - B_s \Delta H_{total} \tag{13}$$

and,

$$B_s = 17.96 + 4.195 Z_R \tag{13a}$$

$$A_s = t\left[\ln\left(10^{-\lambda}\right)^{1.52}\right] \tag{13b}$$

Z_R in equation (13a) is the standard normal deviate.

λ in equation (13b) is defined by equation (14) as:

$$\lambda = \left[0.4 + \frac{1{,}094}{(SN+1)^{5.19}}\right] x \left[\log_{10} W_{18} - 9.36 \log_{10}(SN+1) + 8.27 - 2.32 \log_{10} M_r + Z_R\, s_0\right] \tag{14}$$

The approach here would be to define the vertical strain due to shrinkage and swelling so that ΔH_{total} is the sum of the two. Next, the design variables for the pavement in question are fed in equation (14) to solve for λ . Using λ, A_s is calculated as a function of t = 480 months (time to fully mobilize clay volume change effects according to Lytton et al., 2004) using which ρ_s can be obtained and, in turn, the value of ΔPSI_t can be calculated from equation (12) as a function of t. This should be able to be done with two values of ΔH: 0 and the value calculated as the sum of ΔH_{swell} and $\Delta H_{shrink.}$ The difference should allow one to decouple the impact of shrink/swell on roughness from roughness as a function of traffic over time. The general form of the spreadsheet is to calculate the loss of serviceability, ΔPSI_t, as a function of time, t, in months. This can be calculated for a specific value of total vertical movement, given by equation (15) as:

$$\Delta H_{total} = \left|\Delta H_{shrinkage}\right| + \left|\Delta H_{swell}\right| \tag{15}$$

4.1.3 Overview of the Spreadsheet

The Excel spreadsheet is designed to be a handy tool for calculating the vertical change in height of the subgrade due to fluctuations in suction i.e. between an initial suction, U_i, and a final value of suction, U_f. U_i is the equilibrium suction value, sometimes labeled U_e, characteristic of the climatic profile of the location in question and is determined from the I_m distribution as discussed previously. The final suction value, U_f, could be the result of drying or wetting and would dependently result in shrinkage or swelling, respectively. The sum of the shrinkage and swell defines the overall vertical change in the subgrade, ΔH_{total} that would be expected to impact pavement roughness. The spreadsheet also has the capacity to calculate the change in pavement serviceability index (ΔPSI drop) over its design life using ΔH_{total} and other design parameters. The guidelines for selecting the various input parameters are provided in the User's Manual of the workbook.

The Excel workbook devised contains eight pages: (1) User's Manual describing how the workbook can be most effectively used; (2) Spreadsheet for development of the soil water characteristic curve, SWCC, from pressure plate test data; (3) User's interface for entering data for calculating vertical change in height due to expansive soils within the active zone; (4) User's interface for entering data for calculating change in serviceability due to expansive soils within the active zone: (5) Spreadsheet for swell calculations; (6) Spreadsheet for shrink calculations; (7) Tabulations for total change in height; (8) Roughness or loss of serviceability calculations.

The first page of the workbook is a User's Manual for the following components: (1) Background statement of what the workbook provides and the fundamental basis of the workbook; (2) Description of how values of γ_h are determined directly from pressure plate measurements or from direct input into the spreadsheets based on approximations; (3) Description of how the total change in height is calculated and including tabulated guidelines for selection of key input data such as diffusivity coefficient (α), number of seasonal fluctuations (n), amplitude of suction (U_0), equilibrium suction (U_e), soil coefficient at rest (K_0), and drying/wetting coefficient (f); and (4) Calculation of pavement roughness.

4.1.4 Sensitivity Analysis on Vertical Change and Roughness Calculations

Table 4 summarizes a sensitivity analysis performed in the referenced workbook for calculating ΔH_{swell} and ΔH_{shrink}. The baseline variable input for the calculations were: $2U_0$ = pF of 3.0; γ_h = 0.084; I_m= -10; K_0 for wetting = 0.67 and 0.33 for drying; f = 0.8 for wetting and 0.5 for drying. It is apparent that these baseline input data yield very large values of total vertical change, especially for shrinkage, ΔH_{swell} = 3.41 inches and ΔH_{shrink} = 2.64 inches. Reducing the value of γ_h by fifty percent reduces the change in height for both shrink and swells by approximately 50 percent. Changing n from 0.75 cycles per year to 1.5 cycles per year results in 2.54 inches of swell and 2.09 inches of shrinkage (25.5 percent and 20.8 percent change, respectively). The cumulative effect of changing the values of n and γ_h results in 1.40 inches of swell and 1.13 inches of shrinkage, reductions of 58.9 percent and 57.9 percent, respectively. It can be observed from Table 4 that a change in I_m from -10 to -16 does not significantly impact the swelling and shrinkage results. While there is no change observed in the swell depth, shrinkage increases roughly by about 10 percent upon changing the I_m from -10 to -16.

Also, sensitivity analysis was performed for the serviceability drop (ΔPSI) in the pavement during its expected life of 40 years. The design parameters of the flexible pavement were assumed to be constant in order to appreciate the impact of ΔH_{total} on pavement serviceability. The values of design inputs for the calculations were: Structural Number (SN) = 5; Cumulative Traffic Volume = 15 million ESALs, Resilient Modulus (M_r) of the Natural Subgrade Soil = 8500 psi; Initial Serviceability (PSI_0) = 4.2; Reliability =90%. A ΔH_{total}=6.05 in., corresponding to the baseline input data, results in a ΔPSI of 2.05 and a terminal serviceability (PSI_t) of 2.15 after 40 years of pavement service life. Upon changing n from 0.75 to 1.50 and reducing γ_h to 50 percent of the baseline input value, the resulting ΔH_{total}=2.53 in. reduces the serviceability drop over 40 years from 2.05 to 1.86, thereby increasing PSI_t from 2.15 to 2.34. It is obvious that the above discussed variables that significantly impact of ΔH_{total} in turn contribute to an accelerated drop in pavement serviceability and hence impact the roughness characteristics of the pavement.

Table 4: Summary of Parametric Analysis Results.

I_m	U_e (pF)	H_s (cm)	n	γ_h	α (cm^2/sec)	ΔH_{swell} (in.)	ΔH_{shrink} (in.)	ΔH_{total} (in.)	ΔPSI (after 40 years)
-10 (Austin)	3.75	280	0.75	0.084	0.0021	3.41	2.64	6.05	2.05
				0.042	0.0026	1.86	1.39	3.25	1.89
-10 (Austin)	3.75	280	1.5	0.084	0.0021	2.54	2.09	4.63	1.97
				0.042	0.0026	1.40	1.13	2.53	1.86
-16(San Antonio)	3.86	318	0.75	0.084	0.0021	3.41	2.98	6.39	2.07
				0.042	0.0026	1.86	1.58	3.44	1.9
-16(San Antonio)	3.86	318	1.5	0.084	0.0021	2.54	2.20	4.74	1.97
				0.042	0.0026	1.40	1.29	2.69	1.87

4.2 ME-PDG ANALYSIS OF FLEXIBLE PAVEMENT SYSTEMS

The resilient modulus results obtained from laboratory testing of the EcSS stabilized and natural soils were fed into the ME-PDG analysis of flexible pavement sections to investigate their influence on the performance of the SH130 corridor pavement.

4.2.1 Design Inputs

The Mechanistic-Empirical Pavement Design Guide (ME-PDG 2004) software (version 1.1), considered to be the state-of-art guide for analysis of pavement systems was used for this task. ME-PDG analysis requires and considers three major input categories, namely the traffic distribution, climate and the structural configuration of the pavement with the mechanical properties of all the layers. The ME-PDG output provides a thorough description of the impact of the increased resilient modulus of the subgrade as a result of stabilization in terms of the pavement response to various types of distresses like rutting, surface-down cracking and bottom-up cracking.

4.2.1.1 Traffic Inputs

A design traffic volume of 30 million equivalent single axle loads was considered over a design period of 20 years. The initial two-way annual average daily truck traffic (AADT), which is one of the required traffic inputs in ME-PDG, was taken to be 6375 trucks per lane in correspondence with the design traffic volume considered. The default values provided in ME-PDG was used for all the other traffic related design parameters such as axle load distribution factors and axle configuration.

4.2.1.2 Climate Inputs

A realistic estimate of the climatic conditions in SH130 containing the temperature and precipitation data is a necessary input in the ME-PDG analysis. The climate data file of Austin, Texas, located within a few miles from SH130, was imported for use from the built-in climate database present in the ME-PDG software.

4.2.1.3 Pavement Structure

The pavement structure typical of the one in use in SH130 was used as a baseline for comparison with the two proposed pavement structures with stabilized subgrades. A summary of the thickness and properties of the three structures is presented in Table 5.

All the three pavement systems consist of four main layers; asphalt concrete top layer, an unbound aggregate base over a natural/stabilized subgrade on top of the local vertisol soil. Figure 11 shows the various layers of the pavement structure adopted. A PG 76-22 grade asphalt binder was used in the analysis for all the three pavement systems.

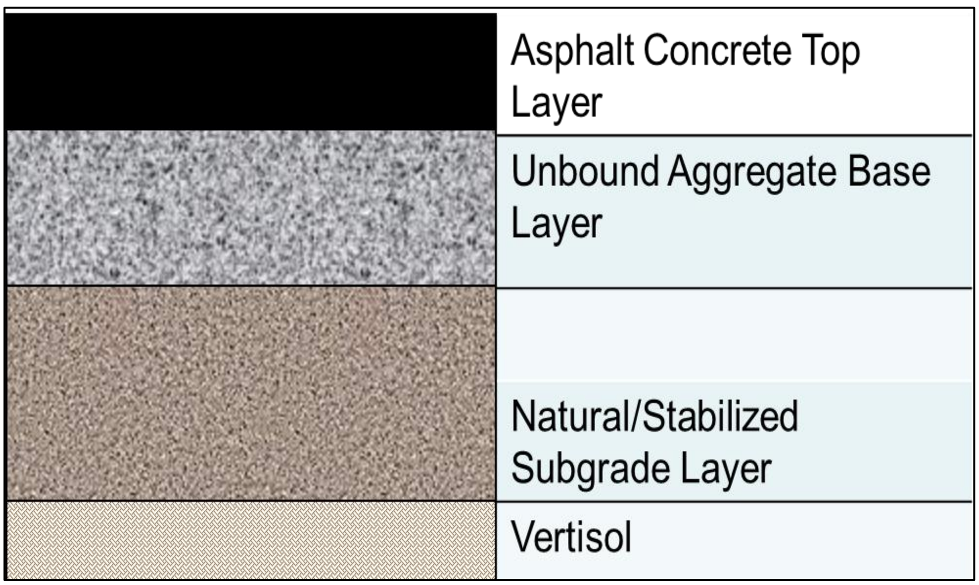

Figure 11: General Structural Configuration of the Pavement Systems

Table 5: Pavement Types and Structural Configuration

Layer	Pavement Structure 1		Pavement Structure 2		Pavement Structure 3	
	Type	Thickness (in.)	Type	Thickness (in.)	Type	Thickness (in.)
AC	PG 76-22	10	PG 76-22	10	PG 76-22	10
UAB	Crushed Gravel	10	Crushed Gravel	10	Crushed Gravel	10
Subgrade	Natural-Heavy Clay	Last Layer	Lime Stabilized	24	LSS	24
					EcSS Treated	60
Vertisol	-	-	Natural-Heavy Clay	Semi-Infinite	Natural-Heavy Clay	Semi-Infinite

4.2.1.4 Resilient Modulus Inputs

The resilient modulus values for the corresponding subgrade layers present in the proposed pavement structures was input based on the laboratory measurement results presented in Table 3. The mean resilient modulus values of the three soils HC1, HC2 and HC3 at optimum moisture levels was selected. Hence, a resilient modulus of 78 MPa, 94 MPa and 135 MPa was chosen respectively for the natural, EcSS-3000 stabilized and lime stabilized subgrade layers.

4.2.2 Results and Discussion

Figure 12, Figure 13, and Figure 14 respectively show a comparison of the surface-down cracking, bottom-up cracking and rutting patterns of the three pavement structures over their design life of twenty years. It can be observed that addition of a 24in. LSS layer (increase of modulus from 78 MPa to 135 MPa) reduces the top-down and bottom-up cracking in the pavement. Figure 12 shows that stabilizing the pavement subgrade does not have a significant influence on the total rutting of the pavement structure. However, increasing the modulus of the subgrade layer through stabilization is expected to change the critical location of rutting and reduce rutting in the subgrade. This finding is important since subgrade rutting requires full-depth reclamation while rutting in the top layers can be overcome by various surface treatment measures.

Longitudinal (top-down) cracking is almost entirely eliminated in pavement structure 3, with a 24 in. LSS layer on top of a 60 in. EcSS-3000 stabilized subgrade layer, although there is a marginal increase in bottom-up cracking. It is worthy to note that longitudinal cracking is a critical pavement distress especially in locations such as SH130 where the natural vertisol consists of highly expansive clays with a high moisture susceptibility and swell-shrink potential.

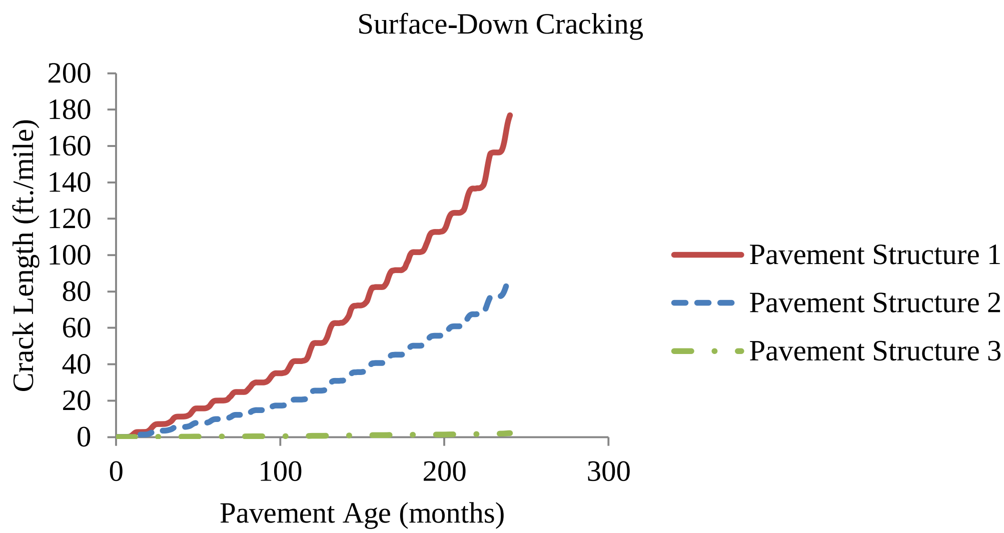

Figure 12: Comparison of Pavement Structures for Surface-Down Cracking

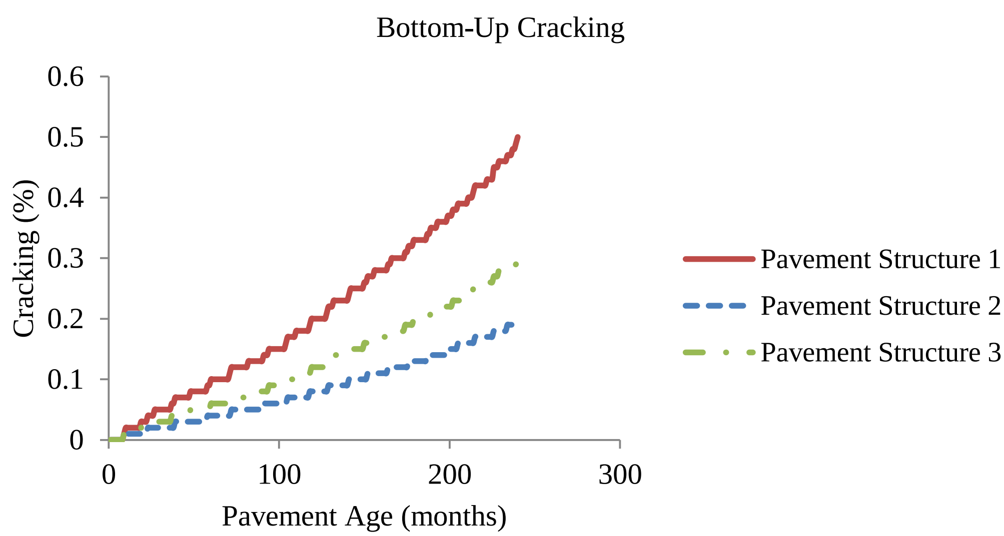

Figure 13: Comparison of Pavement Structures for Bottom-Up Cracking

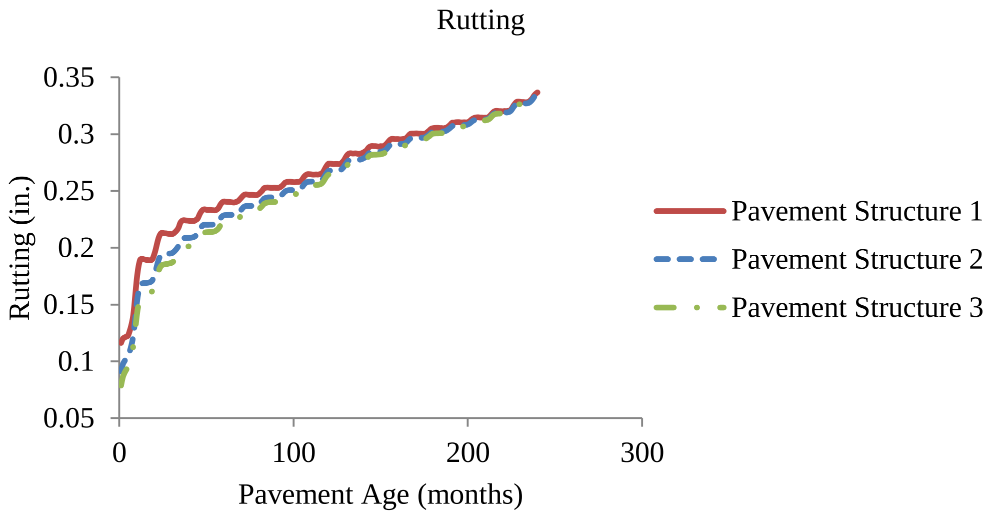

Figure 14: Comparison of Pavement Structures for Rutting

4.3 COMPARATIVE CASE STUDY BETWEEN NATURAL AND EcSS-3000 TREATED SOILS

In the case study below, a comparative analysis has been presented to better appreciate the effect of EcSS-3000 on minimizing vertical movement (swelling and shrinkage) of expansive clays and also the consequent reduction in serviceability drop of the pavement over its expected life. The soil sample in question for this comparative study, I.D. 15, was both tested in its natural untreated state and after treatment with a recommended 1:300 diluted EcSS-3000 solution. The sample, in both its forms, was first subjected to pressure plate testing within a suction range of 0-15 bar accompanied by simultaneous volume measurements at four intermediate suction levels namely 0.5, 5, 10 and 15 bars. With the help of the test results, the Soil Water Characteristic Curves

(SWCCs) of the sample was established and subsequently the suction compression index **(γ_h)** was also estimated over the corresponding suction increments considered.

Table 6 shows a summary of the test results. It is to be noted that, in order to facilitate easy comparison between the two cases, the water contents and volumes of the two samples have been proportionally scaled to negate the minor initial variations in moisture conditions and sample volume. These results however show the exact picture of the effects of EcSS-3000 on the concerned parameters. Also, as previously discussed, since there was no significant difference observed in γ_h for swell and shrink, in this comparative study, a single characteristic γ_h is used for analysis.

Table 6: Summary of Pressure Plate Results of Sample #15

Suction		Water content (%)		Volume (cm^3)		γ_h	
(Bar)	pF	Natural soil	EcSS-3000 Treated	Natural soil	EcSS-3000 Treated	Natural soil	EcSS-3000 treated
0	0	13.9	13.90	63.2	63.2	0.03821	0.0112
0.5	2.7	13.1	12.85	62.7	63	0.03821	0.0112
5	3.71	12.4	12.05	60.92	61.9	0.05234	0.035
10	4.01	12.2	11.78	60.2	61.7	0.0456	0.0278
15	4.19	12.05	11.54	59.9	61.6	0.0456	0.0278

4.3.1 Volume Change Comparison

Figure 15 shows the volume change the sample undergoes before and after stabilization with EcSS-3000. It can be observed from Figure 15 that the EcSS treated Soil undergoes approximately 52% lesser volume reduction in comparison to Natural soil over the matric suction range of 0-4.2 pF.

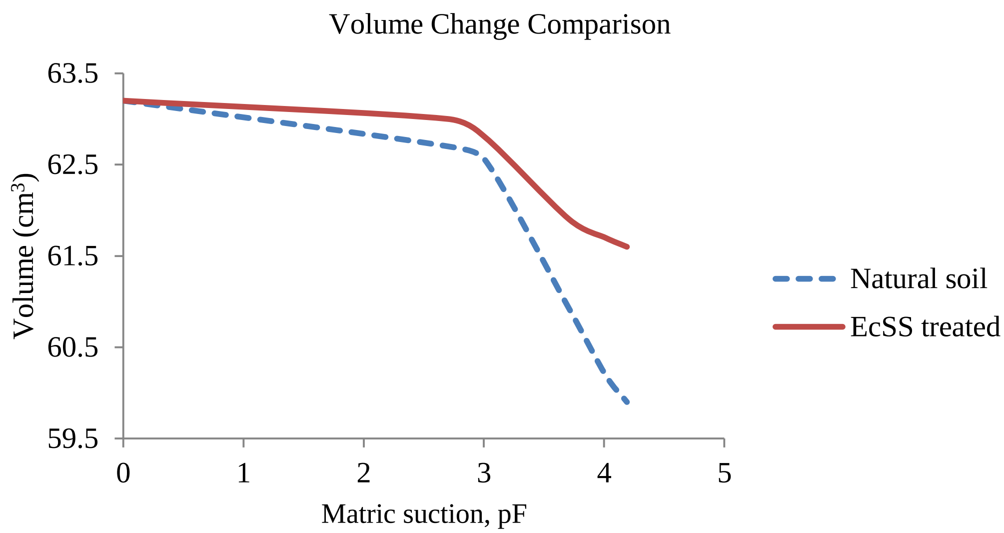

Figure 15: Volume Change comparison of Sample #15 after EcSS-3000 Treatment

4.3.2 γ_h Comparison

Figure 16 shows the spread of γ_h over the suction range of 0-10 bar. It is to be noted that the γ_h value plotted at 10 bar is the γ_h representative between suction interval of 10.01 to 15 bar. It is obvious from Figure 16 that treating Sample #15 with a 1:300 solution of EcSS-3000 results in a significant average drop of γ_h by about 45% in comparison to natural soil.

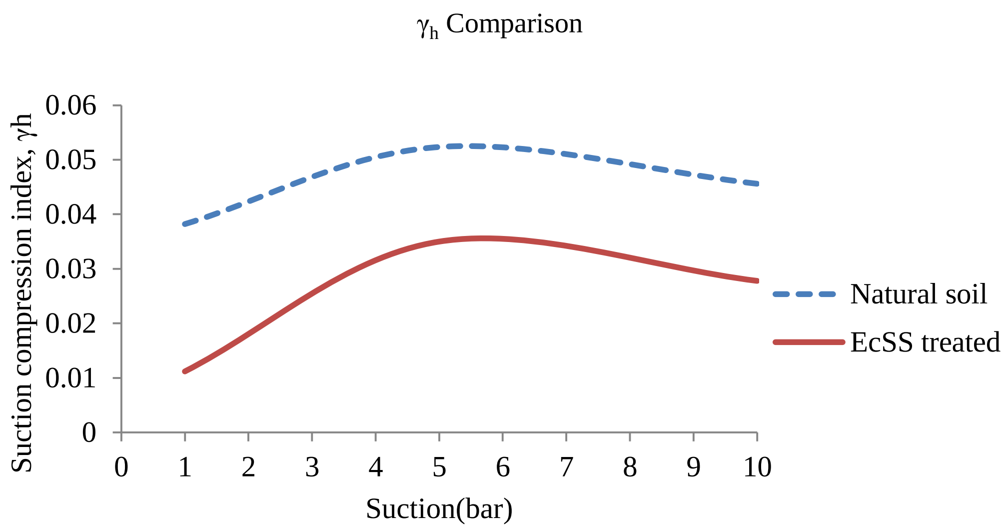

Figure 16: γ_h comparison of Sample #15 after EcSS-3000 Treatment

4.3.3 Vertical Change and Roughness Comparison

Table 7 presents a comparison of swelling and shrinkage results and the serviceability drop after 40 years for a typical flexible pavement before and after treatment with EcSS-3000. It is to be noted that the resilient modulus for this study was assumed to be the mean M_r value at OMC from Table 3 for the respective cases and γ_h values of Sample #15 at the measured suction intervals was taken from Table 1. All calculations were made using the spreadsheet.

The baseline values used for this exercise are: I_m=-10; U_e=3.8, H_s=280 cm; n=0.75 cycles/year and flexible pavement design inputs of SN=5, W_{18}= 4 million ESALs; PSI_i=4.2 and a reliability of 90%.

Table 7: Comparison of Parameters before and after treatment with EcSS-3000

Sample I.D.	I_m (Austin)	n (cycles /year)	Suction Interval (bar)	γ_h	α (cm^2/sec)	ΔH_{swell} (in.)	ΔH_{shrink} (in.)	M_r (psi)	ΔPSI
Natural Soil	-10	0.75	0-5.0	0.038	0.0030	2.33	1.52	8500	0.95
			5-10	0.052	0.0026				
			10-15	0.045	0.0025				
EcSS Treated Soil	-10	0.75	0-5.0	0.011	0.0032	1.60	0.94	11800	0.64
			5-10	0.035	0.0029				
			10-15	0.027	0.0026				

It can be observed from Table 7 that the impact of 45% drop in γ_h after treatment with EcSS-3000, results in reducing the total vertical change from 3.85 in. to 2.54 in. which amounts to a significant near 30% reduction in swelling and shrinkage. Also the combined effect of this reduction and the 25% increase in M_r from 8.5 ksi to 11.8 ksi, results in a ΔPSI change of 0.64 as compared to a 0.95 drop prior to EcSS treatment. Hence, the terminal PSI of the pavement after 40 years of service increases from an undesirable 3.25 to an acceptable 3.56. Figure 17 shows the pavement serviceability profile with time for the two cases.

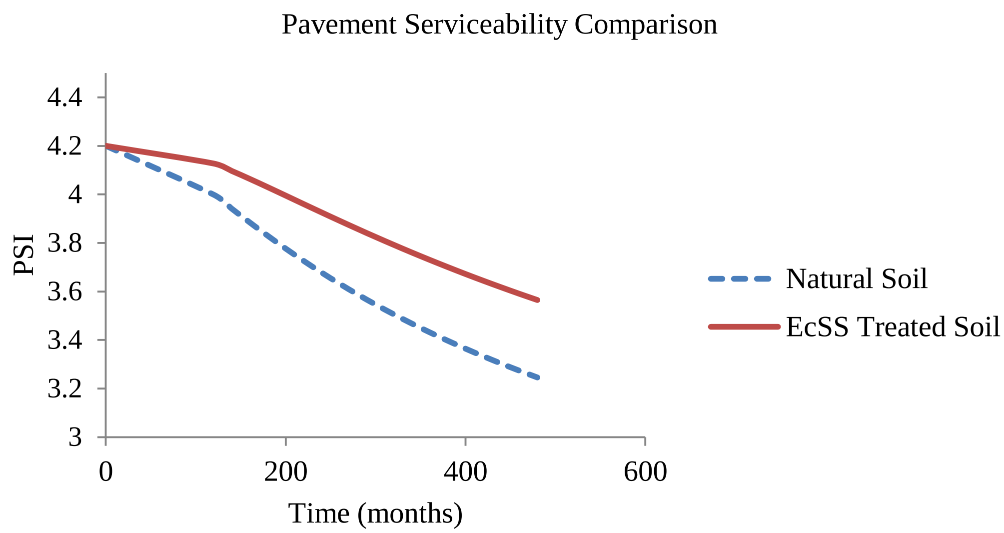

Figure 17: Pavement Serviceability Comparison after EcSS-3000 Treatment

5. SUMMARY OF FINDINGS AND FUTURE WORK

This section summarizes the main findings from all the experiments and analysis that were described in the previous sections of this thesis and offers recommendations for the continued investigation in this field of study.

5.1 DETAILED SUMMARY OF FINDINGS

Based on the analysis carried out using the spreadsheet and ME-PDG software, the results obtained from the pressure plate test and the resilient modulus test have yielded two are readily apparent observations: ionic stabilization of the clayey subgrades with EcSS-3000 is effective in controlling the swell-shrink potential of the soils the associated contribution to pavement roughness in terms of serviceability loss, and the EcSS-3000 treated soils show improved resilient modulus are significantly less sensitive to variations in moisture compared to the soils in their natural condition.

5.1.1 Pressure Plate Test

The pressure plate protocol offers a reliable way to compare volume change sensitivity due to drying and wetting cycles and offers a platform to control γ_h. Furthermore, the associated spreadsheet offers a valuable tool by which to determine vertical change in soils and roughness characteristics of pavements and to evaluate the impact of change in the related parameters.

The database of γ_h values of the soils along the SH 130 corridor measured is high compared to values recorded in the literature. This is to some degree expected for vertisols of this type and with such high plasticity index. The large values of vertical change observed from calculations using the spreadsheet are directly associated with the high γ_h values recorded. Upon treating the soils with EcSS-3000, the γ_h values reduced by about 40-50 %, thereby impacting the movement predictions by the same order of magnitude. Also the net drop in serviceability index of flexible pavement systems is lowered by 0.2-0.3 points after EcSS-3000 treatment.

5.1.2 Resilient Modulus Test and ME-PDG Analysis

The effect of stabilization on impacting the moisture susceptibility of the resilient modulus was monitored using both lime and EcSS-3000. Resilient modulus values of the samples were measured at three moisture levels in the laboratory using the repeated load triaxial test. A performance analysis was carried out on two proposed subgrade stabilized pavement systems using the Mechanistic-Empirical Pavement Design Guide (ME-PDG) software, keeping the existing SH130 pavement structure as the basis for comparison. ‘Pavement structure 1’ consisted of an untreated subgrade while ‘Pavement structure 2’ consisted of a 24 in. LSS and ‘Pavement structure 3’ consisted of 24 in. LSS on top of a 60in. EcSS-3000 stabilized layer were compared with the existing SH130 pavement structure.

The following conclusions can be drawn from the results of this analysis:

- Both lime and EcSS-3000 stabilizers increased the resilient modulus of the SH130 subgrade soils.
- Lime was more effective in increasing the modulus of the soils and controlling the modulus drop compared to EcSS-3000.

- The resilient modulus of the soil samples showed greater sensitivity to moisture variations prior to stabilization with either of lime or EcSS-3000. The modulus ratios predicted from the log (M_r/M_{Ropt}) vs. ($S-S_{opt}$) curve showed that the drop in modulus with increasing levels of saturation was significantly lesser for the stabilized soils compared to the natural soil.
- ME-PDG analysis validated the resilient modulus measurements by showing that the pavement with a stabilized subgrade performed better than the pavement with a natural subgrade under all the three major distresses; bottom-up cracking, surface-down cracking and rutting.
- Pavement structure 3 responded better to surface-down cracking and rutting while the pavement structure 2, without the additional EcSS-3000 stabilized layer, responded slightly better against bottom-up cracking with stabilization having an insignificant impact on reducing the total rutting in the pavement.

5.2 FUTURE WORK

Although some of the limitations of previous methods relevant to the work done in this study were addressed, there is a lot of scope for further research in the following associated topics: investigation of the interactions in the region between the movement active zone and the moisture active zone of soils, performance analysis using concentrations of EcSS-3000 higher than 1:300 and development of more sophisticated customized analysis tools similar to WinPRES.

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APPENDIX

1. PRESSURE PLATE TEST DATA SHEET

Table 8 shows a sample of the Pressure Plate data sheet used to establish the SWCC of the soil samples. The Pressure Plate test was performed in the laboratory in accordance to the procedure outlined in ASTM (2001) D2325.

Table 8: Pressure Plate Test Data Sheet

Operator		*Narain Hariharan*		Test date	9/10/2012
Sample #		15		EcSS-3000 Treated	
Sample Description		Pale yellow appearance		Notes:	1 bar= 100 kPa
Sample Location		Segment 5.2, SH 130			pF=1.01+log kPa
W_{dry} (g)		83.6			
W_{sat} (g)		93.96			
Initial Water content (%)		12.39			
Pressure		Suction		Weight	Water content
Bar	kPa	Log kPa	pF	g	%
0.5	50	1.70	2.71	93.6	11.9617
1	100	2.00	3.01	91.5	9.4498
5	500	2.70	3.71	91	8.8517
10	1000	3.00	4.01	90.56	8.3254
15	1500	3.18	4.19	89.57	7.4100

2. VOLUME MEASUREMENT METHOD

This new and improved volume measurement method is based on measuring the mass of displaced Ottawa sand. As previously stated, the volume of the samples is measured at the end of each of the 0.5,5,10 and 15 bar pressure cycles. The reason for measuring the volume drop is to calculate the suction compression index, γ_h for swelling and shrinkage.

2.1 Apparatus

(a) **(b)** **(c)**

Figure 18: Step-wise Illustration of Volume Measurement using Ottawa Sand

2.2 Calibration

A PVC sample block sample with a smooth surface, having the same dimension as that of the samples was used to calibrate the volume measurement equipment and obtain a relation between the change in mass and volume of the samples. The density of the PVC block used is 1.38 g/cm3.

The Calibration procedure is the same as the test procedure described in the next section. The only difference is that the three PVC blocks with a known density are used instead of the soil samples.

The calibration procedure resulted in the following relationship for Volume estimation, presented in equation (16):

$$V = 38.065 + (0.5981 \times \Delta Mass) \tag{16}$$

where V is the Volume of the sample in cm^3 and ΔMass is defined in '*Procedure'* section.

2.3 Procedure

1. The soil samples are taken out from the pressure plate and weighed and stored in a vacuum desiccator until completion of the set of volume measurements.

2. At least three empty runs are performed. This means the jar is filled with Ottawa sand until it becomes full, the surface is trimmed off, the weight recorded and then the jar is emptied.

3. This average empty run weight is recorded as the 'Empty Run Weight'.

4. The samples, are taken one at a time from the desiccator, and placed into the jar carefully in a vertical position as illustrated in Figure 18 (a). In order to surround the sample completely with the Ottawa sand, the sample must stay in the jar in a vertical position.

5. The jar is then filled with the Ottawa sand carefully until a small hill is formed on top of the jar as illustrated in Figure 18 (b). The Ottawa sand needs to fall into the jar from

one height at a constant speed. Care is taken not dump the sand on the sample as this may result in the Ottawa sand sticking to the sample.

6. Next, a ruler is used to trim out the surface of the jar as shown in Figure 18 (c). This needs to be done only once. Also while trimming the Ottawa sand, the ruler must be held in a perpendicular position to the surface of the jar.

7. The weight of the full jar is recorded for a minimum of five repetitions of the Steps 1 through 7. The average weight is recorded as the ‘Trimmed Average Weight (TAW)’.

8. After the last repetition, the sample is cleaned gently with a soft brush and placed back in the desiccator/pressure plate extractor.

$$\Delta Mass = (Empty\ Run\ Weight) - (TAW - Weight\ of\ the\ sample) \qquad (17)$$

The Volume, V of the sample is then calculated using the equation (16).

9. Once the volume of the sample is measured after each of the suction levels, γ_h for swelling and shrinkage is estimated for each suction change interval.

2.4 Results

Table 9 presents an example of the volume measurement and subsequent γ_h calculation data sheets.

Table 9: Volume Measurement Data Sheet

Sample #	15 (EcSS Treated)			
Pressure (Bar)	0.50	5.00	10.00	15.00
Matric suction, pF	2.71	3.71	4.01	4.19
Trimmed Average weight (g)	1620	1620.40	1621.60	1622.00
Δ Mass (g)	78.33	77.24	75.74	75.08
Volume, V (cm^3)	84.91	84.26	83.37	82.97
Swelling, γ_h	0.01107			
		0.03575		
			0.0270182	
Shrinkage, γ_h	0.01098			
		0.03537		
			0.02689	

3. USER'S MANUAL FOR VERTICAL MOVEMENT AND ROUGHNESS CALCULATIONS

3.1 γ_h Calculation using Suction Based Approach

The Suction Compression index, γ_h, for swell and shrinkage plays an influential role in the final calculations for vertical movement and hence an accurate estimation of γ_h is a critical first step in the process. In this approach, γ_h is estimated from the pressure plate test over four suction intervals between a 0-15 bars. The γ_h values are then correlated with the suction v. depth profile to come up with reasonable estimates of γ_h with depth, H_s.

Note: 'H_s' is the depth below the surface of the subgrade layer in cm and the typical depth beyond which no significant suction change occurs is expected to be around 600 cm (20 ft.)

The diffusion coefficient, α, for swell and shrink, in units of cm^2/sec, is also computed using the pressure plate data and the resulting soil water characteristic curve (SWCC) using an empirical relationship involving S and γ_h at the corresponding suction interval. Equation (18) from Lytton et.al (2007) is used:

$$\alpha(swell\ or\ shrink) = 0.0029 - (0.000162 * S) - 0.0122\gamma_h(swell\ or\ shrink) \quad (18)$$

where, S is the slope of the SWCC of the sample in question.

To aid the user in the computation of γ_h and α, the pressure plate test and volume measurement data sheets are incorporated into the ΔH calculation. The user is given the option of entering the raw data from the laboratory suction test or simply entering the γ_h values directly to influence calculations.

Option 1: Suction Measurement Data Sheet

There are essentially two parts to this datasheet. The first part consists of the raw data from the pressure plate test, while the second part logs the volume measurement data and the subsequent γ_h computation for swelling and shrinkage between four suction intervals: 0-1 bar, 1-5 bar, 5-10 bar and 10-15 bar.

Step-Wise Instructions for Data Entry in the 'SWCC' Worksheet

Note: All weight measurements are in grams and volume measurements in cm^3.

1. Record the sample I.D. and description in the respective cells.
2. Enter the dry and saturated weights of the soil sample.

3. Enter the weights of the sample at the end of each of the 0.5, 1, 5, 10 and 15 bar pressure cycles.
4. Enter the trimmed average weight of a minimum of five readings of the volume measurement jar filled with Ottawa sand and the soil sample at the end of the 1, 5, 10 and 15 bar pressure cycles.

Steps 2 and 3 will give the variation of water content (%) within the 0-15 bar pressure range. This is represented in the SWCC of the soil sample.

Step 4 yields the γ_h values within the respective suction intervals for swelling and shrinkage.

Note: The water content and γ_h values are automatically updated in the 'Input data' worksheet for ΔH calculation.

Option 2: Direct Input of γ_h and Water Content Values

In the event γ_h and moisture content are readily available or are estimated in lieu of recording the values from the pressure plate testing, the user must skip the suction data sheet and directly enter the values of these parameters in the 'Input data' worksheet.

Note: Refer to the next section for step-wise instructions.

3.2 Calculation of Vertical Movement (ΔH_{total})

As previously discussed, the vertical change in the height of the subgrade is a result of two components: swell and shrinkage. The contributions of swelling and shrinkage are each separately evaluated and the absolute values of the respective vertical change in heights are summed up to calculate the total vertical movement (ΔH_{total}).

Note: The worksheets named 'Swell data', 'Shrink data' and 'ΔH_{total}' give more information about the respective calculations.

<u>Instructions for Navigating Through 'User Interface-1' Worksheet</u>

General Notes

1. Cells in green are required inputs and have a direct impact on calculation.
2. Lists are attached with every input parameter except Step 7, to aid the user in data entry and also as a means of data validation.
3. Refer to the guideline tables attached during data entry.
4. Cells in brown indicate results.

Step-Wise Data Entry Procedure

1. Enter the location under study.
2. Enter the Thornthwaite Moisture Index (1948), TMI or I_m corresponding to the location, from the reference map provided in Figure 19. The equilibrium suction, U_e, in pF and the active zone depth, H_s, in cm are computed from the TMI.
3. Choose the amplitude of suction variation, U_o, in pF, for the location in question, from the list provided.
4. Enter the number of wet/dry excursions, n, cycles/year (or) n is calculated automatically based on Table 11.
5. Pick a value for the crack fabric factor, f, for swelling and shrinkage from Table 12.
6. Pick a value for the earth pressure at rest, K_o, for wetting and drying cycles separately from Table 10.

7. Only if the user chooses Option 2 in Section 1 - Input the γ_h and water content (w.c) values under 'suction measurement inputs'.

   Special Note: Make sure to identify water content and γ_h for all the required suction intervals. If only single values of γ_h and w.c are available, use the same value for all the intervals. Do not leave them blank.

Vertical Movement Results

1. The vertical change in height of the subgrade resulting due to swell, ΔH swell, in inches, can be viewed.
2. The vertical change in height of the subgrade resulting due to shrinkage, ΔH shrinkage, in inches, can be viewed.
3. The total vertical change in height of the subgrade, ΔH total, in inches, can be viewed.

Table 10: Guidelines for K_o Determination (Modified from Lytton et.al. 2004)

Soil Condition	K_0
Cracked	0
Drying (active)	0.33
Equilibrium (at rest)	0.5
Wetting (within active zone)	0.67
Wetting (below active zone)	1
Swelling near surface (passive pressure)	3

Table 11: Guidelines for Selecting 'n' (Modified from Mckeen and Johnson, 1990)

Frequency, n cycles/year	**Potential Active Zone Depth**
0.5	Deep (>6m)
0.75	Moderate (2-6 m)
1	Shallow (<2m)
1.25	Unstable Climate

Table 12: Guidelines for Selecting 'f' (Modified from Mckeen and Johnson, 1990)

Condition	**f**
Drying	0.5
Wetting	0.8

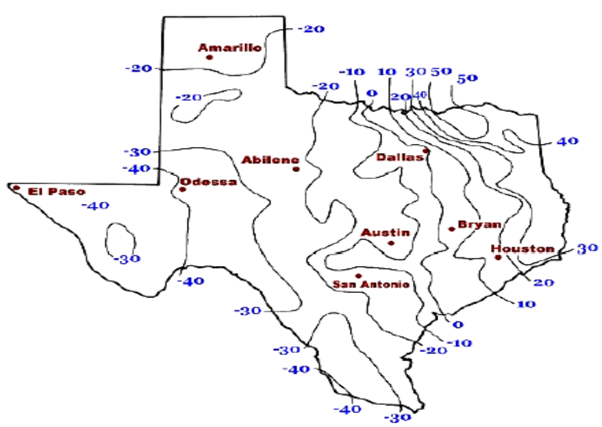

Figure 19: Distribution of Thornthwaite Moisture Index in Texas (After Thornthwaite, 1948)

Table 13: Reference Guidelines for Input Parameters for Diffusivity and Suction

Parameter	Range
α, in cm2/sec	0.00025 – 0.0040 (nominal value of 0.00175)
	0.003 – 0.004 (Austin and Fort Worth Districts, Texas)
$2U_o$, in pF	5.0 extreme climates
	4.0 Normal design
	3.0 Moderate design
	2.0 Mild climates
	1.0 Stable climates
U_e, in pF	4.1 Amarillo
	3.5 Dallas
	3.0 Houston
	3.8 San Antonio
	4.5 Gallup, NM
	3.85 Seguin, TX
	4.5 El Paso

3.3 Calculation of Pavement Roughness

The impact of the vertical change in height of the subgrade due to swelling and shrinkage, computed in the previous section, on the roughness of a pavement section is presented in the worksheet entitled 'User interface-2'. The consequent reduction in pavement serviceability (ΔPSI) over the assumed analysis period of 40 years is estimated.

Instructions for Navigating Through 'User Interface-2' Worksheet

General Notes

1. Cells in green are required inputs and have a direct impact on this calculation.
2. It is highly recommended to refer to the guideline tables attached during data entry.
3. Cells in brown indicate results.

Step-Wise Data Entry Procedure

1. Enter the Structural number (SN) representing the flexible pavement in question.
2. Enter the total traffic the pavement is expected to accommodate during its life, cumulative 18-kip ESALs.
3. Enter the resilient modulus of the subgrade, in psi.
4. Enter the initial serviceability rating, scale of 0-5.

Note: Guideline for selecting values to be used in step 1 to 4 is provided in Table 14.

Roughness Results, as a Measure of ΔPSI

1. A plot of the PSI drop over the 40-year analysis period is generated.

2. ΔPSI at the end of 40-years and the final pavement serviceability rating can be viewed.

Table 14: Guidelines for Selection of Pavement Properties (Modified from AASHTO 93)

Parameter	**Range**
SN	1-3 Poor
	4-6 Moderate
	>6 Good
M_r (psi)	<5000 Poor
	5000-8000 Moderate
	>8000 Good
Cumulative 18-kip ESALs	< 5 million: Low traffic density
	5-15 million: Moderate traffic density
	>15 million: High traffic density

3.3 Spreadsheet Snapshots

3.3.1 User-Interface 1

Table 15: Input Data for Vertical Movement Calculations

Location	Amarillo, TX
Thornthwaite Moisture Index (TMI or I_m)	-20.0000
Equilibrium suction, U_e (pF)	3.9459
Depth of Active Zone, H_s or z(cm)	349
Amplitude of suction variation, U_o (pF)	1.5000
Wet/dry excursions n (cycles per year)	0.7500
Crack Fabric Factor, f (swell/wet)	0.8000
Crack Fabric Factor, f (shrink/dry)	0.5000
K_o (wet)	0.6700
K_o (dry)	0.3300

Table 16: Suction Measurement Inputs from Pressure Plate Test Results

Suction range (pF)	Water content (%)	S=dh/dw	γ_h(swell)	γ_h (shrinkage)	α (swell)	α (shrink)
0.0-3.0	14.95	-1.21	0.0071	0.0071	0.0030	0.0030
3.01-3.70	12.49	-2.54	0.0071	0.0071	0.0032	0.0032
3.71-4.00	12.22	-1.07	0.0357	0.0353	0.0026	0.0026
4.01-4.19	11.95	-.997	0.0245	0.0244	0.0028	0.0028
>4.19	11.77				0.0029	0.0029

Table 17: Vertical Movement Results

ΔH swell(in.)	1.9448
ΔH shrinkage (in.)	-0.5643
ΔH total (in.)	2.5092
ΔH total (mm)	63.7333

3.3.1 User-Interface 2

Table 18: Pavement Design Input Parameters

Structural number (SN)	5
Cumulative ESALs (W_{18})	5.00E+06
Resilient modulus (M_r) in psi	7500
Design Reliability (%)	90
Z_R (95% reliability)	-1.282
s_0	0.44
ΔH Total (mm)	63.7333
ΔH Total (mm)	0.0000
Initial Serviceability, PSI_i	4.2

Table 19: Serviceability Loss Results

Initial Serviceability, PSI_i	4.2
ΔPSI	0.72
Terminal Serviceability, PSI_f	3.48