RESILIENT MODULI PROPERTIES OF COMPACTED UNSATURATED SUBGRADE MATERIALS

BY

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To My Family

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ABSTRACT

RESILIENT MODULI PROPERTIES OF COMPACTED UNSATURATED SUBGRADE MATERIALS

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According to the new Mechanistic Empirical Pavement Design Guide (MEPDG) and 1993 AASHTO flexible pavement design guide, Resilient Modulus (M_R) has been used extensively as an important material property in structure design of pavement. The modulus is used as the primary input parameter to determine the stiffness parameters and constitutive behavior of pavement components. The system of pavement basically consists of the layers of surface, base, subbase (optional), and subgrade. The compacted subgrade soils supporting pavement structure are typically unsaturated with degrees of saturation varying from 75% to 90%. The effect of unsaturated soil behavior on

the mechanical properties of compacted pavement materials become an important variable and need to be considered.

The main purpose of this study is to study the resilient moduli properties of compacted and unsaturated subgrade materials and to determine the effect of compaction moisture content, which is related to matric suction of the soils, on the resilient moduli properties. The second objective is to study the use of MEPDG models to calibrate resilient moduli properties either as a function of moisture content or soil suction variables. To accomplish these objectives, soil specimens were prepared at five different moisture content and dry density conditions and tested using conventional resilient modulus testing as per AASHTO T-307 procedure.

The basic soil tests such as grain size distribution, Atterberg's limits, and standard proctor compaction were initially performed. Then, the advanced soil tests consisting of soil water characteristic curve (SWCCs), unconfined compressive strength (UCS) test, and conventional resilient modulus test were conducted. The soil suction conditions of the prepared specimens were determined based on the SWCCs information and the compaction moisture content. The test results indicate that compaction moisture content affected the values of resilient modulus of the subgrade soils. The specimens compacted at dry side of optimum moisture content (OMC) showed higher values of resilient modulus compared with the specimens compacted at OMC and wet side of OMC. The testing data were also analyzed with the models provided in MEPDG program. The level 2 input for predicting SWCCs provided in MEPDG gave the predicted SWCCs in similar trend to the measured SWCCs. However, the curves were not quite well matched. Lastly, the modified universal model and the model proposed by Cary and Zapata (2010) were studied and analyzed in detail. The results showed that the universal model is well suited for predicting the resilient modulus of the subgrade soils. However, the resilient modulus values predicted by the model of Cary and Zapata, sometime, showed the higher values than measured results especially, the specimens compacted at 0.8OMC.

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CHAPTER 1

INTRODUCTION

1.1 Overview

A pavement is defined as a hard surface constructed over the natural soils. The purpose of pavement is providing a stable, safe and smooth transportation medium for vehicles. The system of a pavement consists of the layers of surface, base, subbase, and subgrade materials. The performance of a pavement depends on many factors such as the structural adequacy, the properties of the materials used, traffic loading, climatic conditions and construction method. The subgrade soil is a significant portion of the pavement construction. The traffic loading is finally distributed to subgrade layer. Greater subgrade structural capacity can improve pavement strength and performance and can results in thinner and more economical pavement structures.

The new Mechanistic Empirical Pavement Design Guide and 1993 AASHTO flexible pavement design guide use Resilient Modulus (M_R) as the primary input parameter when characterizing subgrade and unbound bases. Resilient modulus of soils is typically determined either by using different types of laboratory tests or using different methods of in situ nondestructive tests. These tests measures the stiffness of cylindrical specimen that is subjected a cyclic or repeated axial load; it creates a relationship between deformation and stresses in different materials and soil conditions, such as moisture and density.

Pavements are constructed on compacted subsoils that are typically unsaturated with degrees of saturation varying from 75 to 90%. The effect of unsaturated soil behavior on the mechanical properties of compacted pavement materials needs to be considered. The soil suction, which is the negative pore water pressure, occurring due to the presence of water in soil particles has a significant effect on the pavement foundation stiffness and strength properties (MnDOT 2007 guidelines).

Different types of models accounting for soil suction and water content properties have been proposed by many researchers for modeling the resilient modulus of soil and aggregates for several years. In this study, two models proposed by Cary and Zapata (2010) and Modified universal model are used to model the resilient response of the subgrades. Both models are analyzed with respect to providing realistic resilient properties of the soils.

1.2 Research Objectives

The main objective of the research is to study the resilient moduli properties of compacted and unsaturated subgrade materials and to determine the effect of compaction moisture content, which is related to matric suction of the soils, on the resilient moduli properties. The second objective of this research is to study the use of MEPDG models to calibrate resilient moduli properties either as a function of moisture content or soil suction variables. To accomplish these objectives, the resilient moduli properties of subgrade materials were tested using a repeated load triaxial test at different compaction conditions. The following tasks are performed to address the above research objectives.

- Review the available literature on subgrade materials, resilient modulus testing, and fundamental concepts of unsaturated soil mechanics.
- Determine the soil water characteristic curve at three different compaction related moisture content - dry density conditions using a Tempe cell (Fredlund SWCC device) and filter paper technique.
- Perform the resilient modulus testing using the repeated load triaxial test equipment on three subgrade materials compacted at five different moisture content levels that are related to different matric suction levels.
- Analyze the effects of moisture content and soil suction on resilient properties.

All necessary index and moisture-density tests were carried out as per standard test methods. Resilient modulus testing was carried out as per AASHTO T-307 procedure.

1.3 Thesis Organization and Summary

A brief description of the content of each chapter included in the thesis is presented in the following paragraphs.

In Chapter 2, an overview of literature review to cover the basic concepts of resilient modulus testing on unbound materials is presented. In addition, the review also presents different ways to estimate resilient modulus and the factors affecting resilient modulus.

Chapter 3 summarizes the basic properties of the base material. The experimental program, sample preparation, laboratory test equipment including unconfined compressive strength, repeated load triaxial test and, data acquisition procedure. Fundamentals of Tempe cell, filter paper technique and the test procedures used to determine the Soil Water Characteristic Curve (SWCC) is also given in detail.

Chapter 4 summarizes the results obtained from the advanced tests conducted on the base materials. Tests results from UCS, resilient modulus testing and SWCC are presented. The second part of this chapter provides the analysis performed on the test results obtained in this study. Measured and predicted values of resilient modulus and SWCC are compared.

Finally, Chapter 5 provides summary and conclusions derived from the test results, as well as some recommendations for future research are presented.

List of references are included towards the end of the report supporting the current research.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

This chapter presents the current literature review on resilient modulus testing on unsaturated subsoils. The main objective of this chapter is to present a brief review of resilient modulus (M_R) concept, followed by the different approaches used in the literature to either determine or estimate the M_R of soils. The fundamental parameters that impact the resilient modulus of compacted subsoils are also presented.

2.2 Resilient Modulus, M_R

The definition of resilient modulus has been defined by many researchers. According to Puppala (2008), the resilient modulus is similar to the elastic modulus used in elastic theories. It is defined as a ratio of deviatoric stress to resilient or elastic strain experienced by the materials under repeated loading conditions that simulate traffic loading. A schematic of the resilient modulus parameter (M_R) and the equation representing the definition of resilient modulus are presented in Figure 2.1.

During resilient modulus testing, the specimen is subjected to a dynamic cyclic deviatoric stress and a static confining stress provided by means of a triaxial pressure chamber. The static confining pressure simulates the lateral stress caused by the overburden pressure and dynamic cyclic stress simulates the traffic wheel loading induced stress in soil.

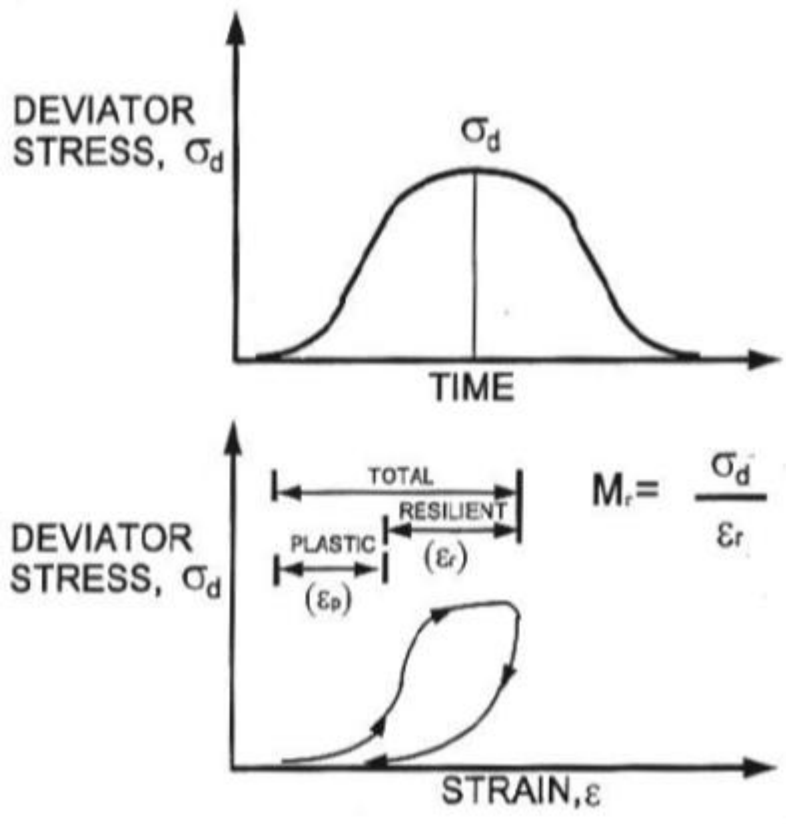

Figure 2.1 Definition of resilient modulus (Puppala 2008)

Loads applied on the soil specimens in the resilient modulus laboratory test are small when compared with ultimate loads of the same soils at failure conditions and hence resilient modulus tests are often referred to as non-destructive load testing. The deformations measured during the test cycles are considered as recoverable or elastic deformations and are used to estimate the resilient modulus parameter.

Figure 2.2 shows the other forms of moduli properties. In general, the modulus is defined as a ratio of the stress applied and measured strain. As reflected in the Figure 2.2, depending on the magnitude of the stress applied, the modulus can be an initial tangent modulus, E_{max}, or a secant modulus (E_1 through E_3); while, the resilient modulus is measured from repeated applied stress. The importance of using resilient modulus as the parameter for subgrades and bases is that it represents a basic material property and can be used in the mechanistic analyses for predicting different distresses such as rutting and pavement roughness.

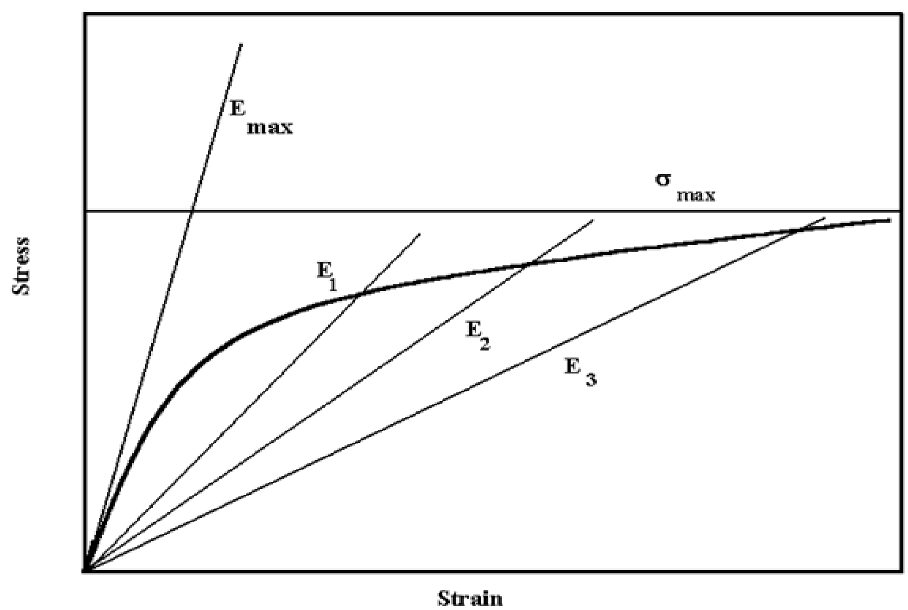

Figure 2.2 Definition of modulus (Nazarian et al. 1996)

As stated previously, in laboratory testing, the soil specimens are subjected to a series of load pulses with distinct rest period, measuring the recoverable axial deformation and the applied load. Thus, from the definition, the resilient modulus is calculated as per the following expression:

$$M_R = \frac{\sigma_d}{\varepsilon_r} \tag{2-1}$$

Where σ_d is the axial deviatoric stress and is calculated from

$$\sigma_d = \frac{P}{A_i} \tag{2-2}$$

Where P is the applied load and A_i is the original cross-sectional area of the specimen. Parameterε_r, the resilient strain, is calculated from

$$\varepsilon_r = \frac{\Delta L}{L_i} \tag{2-3}$$

Where ΔL is the recoverable axial deformation along a gauge length, L_i.

2.3 Determination of Resilient Modulus

Basically, the resilient modulus can be determined three commonly used methods, laboratory testing methods, field testing methods, and empirical correlations or soil properties based calibrations. A brief description of each of these methods (termed as Levels in MEPDG) is provided in the following sections.

2.3.1 Level 1-Laboratory or Field Testing

2.3.1.1 Laboratory Testing

Laboratory tests are essential to study the parameters that affect the properties of materials. By conducting laboratory tests including resilient modulus test, the behavior of a material in terms of variation in modulus with stress level, strain amplitude, and the strain rate is best established. However, resilient moduli from laboratory tests are moderately or significantly different than the in-situ test results. These differences can be due to sampling disturbance, differences in the state-of-stress between the specimen and in-place pavement material, non-representative specimens, long-term time effects, and inherent errors in the field and laboratory test procedures (Anderson and Woods 1975). A brief discussion of the laboratory tests to determine M_R properties is summarized below.

2.3.1.1.1 Repeated Load Triaxial Test

The Resilient Modulus test using Repeated Load Triaxial (RLT) test equipment is designed to simulate traffic wheel loading on in situ subsoils by applying a sequence of repeated or cyclic loads on compacted soil specimens. A compacted soil specimen is, initially, prepared by using impact compaction or other methods and then, is transferred into triaxial chamber. After that, confining pressure is applied to the specimen. Then, testing is performed by applying various levels of deviatoric stresses as per the test sequence (Puppala 2008).

The test process requires both conditioning followed by actual testing under a multitude of confining pressure and deviatoric stresses. At each confining pressure and deviatoric stress, the resilient modulus value is determined by averaging the resilient deformation of the last five deviatoric loading cycles. Hence, from a single test on a compacted soil specimen, several resilient moduli values at different combinations of confining and deviatoric stresses are determined. The RLT test is most prominent method to estimate resilient modulus because this test is standardized by AASHTO and its features better simulation of traffic loading.

2.3.1.1.2 Other Laboratory Tests

A hollow cylinder test simulates stress conditions close to the field traffic loading, including the principle stress rotations taking place in the subgrade caused by wheel load movements (Barksdale et al. 1997). In this test, a hollow cylindrical soil specimen is enclosed by a membrane both inside and outside the sample. Stresses are applied in axial or vertical, torsional, and radial directions. Repeated loads can be simulated in this setup and related moduli can be determined. Because of the possible application of various types of stresses, different stress path loadings simulating field loading conditions can be applied. Also, this setup can be used to perform permanent deformation tests.

Resonant column test can also be used to determine resilient modulus of subgrade soils. The small-strain shear modulus G is first calculated from the frequency response curve generated during the test and displayed on the analyzer main screen for post-test data processing. The small-strain shear modulus is then converted to resilient modulus values.

The principal advantage in using laboratory procedures to determine the resilient modulus is basically the capability of performing a controlled test. When performing a laboratory test; it is possible to control the confining pressure level as well as the shear stress level or both. However, laboratory procedures are time consuming and have a high cost which makes the procedure economically unsuitable for routine pavement design.

2.3.1.2 Field Tests

Several in situ methods have been used to predict or interpret the resilient moduli or stiffness of unbound materials and subgrades (pavement layers). Various test procedures and their methods for measuring resilient modulus properties are described in the NCHRP synthesis by Puppala (2008). These methods are grouped into two categories: nondestructive methods and intrusive methods. In this section, few field methods are presented briefly.

2.3.1.2.1 Nondestructive Methods

Nondestructive methods for determining the stiffness (*E*) are based on several principles, including geophysical principles. Some of the methods involve the measurement of deflections of pavement sections subjected to impulse loads and then employ back-calculation routines to estimate the stiffness properties of pavement layers such that the predicted deflections match with the measured deflections.

2.3.1.2.1.1 Dynaflect

Dynaflect is a light-weight two-wheel trailer equipped with an automated data acquisition and control system. The pavement surface is loaded using two counter-rotating eccentric steel weights, which rotate at a constant frequency of eight cycles per second (8 Hz). This movement generates dynamic loads of approximately ±500 lb (227 kg) in magnitude (Choubane and McNamara 2000).

The total load applied to a pavement system is a combination of the static weight of the trailer and the dynamic loads generated by the rotating weights. The deflections of the pavement system are measured by five geophones suspended from the trailer and placed at 1 ft intervals. Deflection data monitored during the loading is then analyzed using both theoretical and empirical formulations to determine the modulus of subgrade and base layers.

2.3.1.2.1.2 Falling Weight Deflectometer (FWD)

FWD applies an impulse load on the pavement surface by dropping a weight mass from a specified height and then measures the corresponding deflections from a series of geophones placed over the pavement surface. Deflection profiles under different impulse loads will be measured and analyzed with different theoretical models of distinct constitutive behaviors to determine the modulus of various layers in the pavement system.

The analysis uses back-calculation routines that assume a different modulus for each layer of the pavement and then use a specific algorithm to predict the deflections of the pavement surface. If the predicted deflection pattern and magnitudes match with the measured deflections, then the assumed moduli are reported as the moduli of the pavement layers.

2.3.1.2.1.3 Light Falling Weight or Portable Deflectometers

Among nondestructive assessment of pavement layers, portable deflectometer–type devices have been receiving considerable interest by several DOT agencies. Similar to the full-scale FWD-type tools, these devices utilize both dynamic force and velocity measurements by means of different modes such as transducers and accelerometers. These measurements are then converted to elastic stiffness of the base or subgrade system, which is equivalent to homogeneous Young’s modulus of the granular base and subgrade layers, using equations that assume underlying layers as homogeneous elastic half-space. Factors that influence the stiffness estimation of field devices also influence these methods, and hence some variations in moduli values are expected with the same group of devices that operate on different principles. A few of these Light Falling Weight or Portable Deflectometers used in the field are LFWD, LFD and PFWD.

2.3.1.2.2 Intrusive Methods

Intrusive or in situ penetration methods have been used for years to determine moduli properties of various pavement layers. Intrusive methods can be used for new pavement construction projects and also in pavement rehabilitation projects wherein the structural support of the pavement systems can be measured (Newcomb and Birgisson 1999). Few intrusive methods are briefly reviewed in following.

2.3.1.2.2.1 Dynamic Cone Penetrometer (DCP) Test

The DCP is a simple testing device including of a slender shaft and a sliding hammer weight. The DCP test is a widely used in situ method for determining the compaction density, strength, or stiffness of in situ soils. The slender shaft is driven into the compacted subgrades and bases using the sliding hammer weight and the rate of penetration are measured. Penetration is carried out as the hammer drops to reach the desired depth. The rod is then extracted using a specially adapted jack. Data from the DCP test are then processed to produce a penetration index, which is simply the distance the cone penetrates at each drop of the sliding hammer. Typically, in this test, the measured soil parameter from the test is the number of blows for a given depth of penetration. Several parameters from DCP tests are typically determined and these are termed as dynamic cone resistance (q_d) or DCP index (DCPI) in millimeters per blow or inches per blow or blows per 300 mm penetration. These parameters are used to evaluate the compaction density, strength, or stiffness including resilient moduli of in situ soils.

2.3.1.2.2.2 Plate Load Test

Plate load tests (PLTs) were used for resilient moduli interpretations and a few states, including Florida and Louisiana, have attempted to use them for correlating with the resilient modulus of subgrades (Abu-Farskh et al. 2004). The PLT operations involve loading a circular plate that is in contact with the layer to be tested and measuring the deflections under load increments. Circular plates usually 30 cm (12 in.) in diameter are generally used and the loading is transmitted to the plates by a hydraulic jack.

During the test, a load- deformation curve will be recorded and these data will be used to estimate the moduli of the load-deformation or stress-strain plot, which is referred to as E_{PLT}. If the field test is performed in cyclic mode, then the slope of the stress–strain curve provides the moduli. The moduli measured from this test are regarded as composite moduli as the depth of influence is considered to extend more than one layer (Abu-Farsakh et al. 2004). Nelson et al. (2004) also reported the use of PLTs to estimate the moduli of compacted retaining wall backfill material. Though the PLT method is primarily used for rigid pavements, several researchers have attempted to correlate the moduli with the elastic moduli of the subgrades. More research is still needed to better understand the applicability of this method in evaluating the resilient properties of subgrades and bases. Dilatometer, Pressuremeter, Static Cone Penetrometer are few other intrusive methods used in the field to determine the stiffness properties of the soils.

2.3.2 Level 2-Correlations with Other Material Properties

The resilient modulus testing, sometimes, is considered as a complicated and time-consuming method for some agencies to receive the resilient moduli values. Because of this,a simple method, which is the use of correlations with other soil properties such as the CBR, has been proposed for estimating the M_R of the geomaterials (Hossain 2008). Therefore, in the case that no resilient modulus test data is available, the modulus value can be calculated using one of the empirical relationships presented in Table 2.1.

Table 2.1 Summary of Correlations to Estimate Material Properties

Strength/Index Property	**Model**	**Comments**	**Test Standard**
CBR	$M_r = 2555(CBR)^{0.64}$	CBR=California Bearing Ratio, percent	AASHTO T 193-The California Bearing Ratio
R-value	$M_r = 1155 + 555R$	R=R-value	AASHTO T190-Resistance R-Value and Expansion Pressure of Compacted Soils
AASHTO layer coefficient	$M_r = 30000(\frac{a_i}{0.14})$	a_i=AASHTO layer coefficient	AASHTO Guide for the Design of Pavement Structures (1993)
PI and Gradation	$CBR = \frac{75}{1 + 0.728(w.PI)}$	w.PI = P200*PI P200=percent passing No.200 sieve size PI=plasticity index, percent	AASHTO T27-Sieve Analysis of Coarse and Fine Aggregates AASHTO T90-Determining the plastic and Plasticity Index of Soils
DCP	$CBR = \frac{292}{DCP^{1.12}}$	DCP=DCP index, in/blow	ASTM D6951-Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications

2.3.3 Level 3-Typical Values (Based on Calibration)

In this level, the little or no testing information is provided; therefore, the default values of resilient moduli need to be defined. The resilient moduli are selected based on the material classification. Table 2.2 presents the determination of resilient moduli based on soil classification. These relationships, which are typically based on the index proprieties of geomaterials, can be used in design stages as the first approximation. However, due to the general nature of these relationships and inherent variability in the geomaterials, the level of uncertainty in the estimated values is rather high.

Table 2.2 Resilient moduli Recommended by the MEPDG Based on the Soil Classification

AASHTO Symbol	**Typical CBR Range**	**M_R Range (ksi)**	**M_R Default (ksi)**
A-7-6	1-5	2.5-7	4
A-7-5	2-8	4-9.5	6
A-6	5-15	7-14	9
A-5	8-16	9-15	11
A-4	10-20	12-18	14
A-3	15-35	14-25	18
A-2-7	10-20	12-17	14
A-2-6	10-25	12-20	15
A-2-5	15-30	14-22	17
A-2-4	20-40	17-28	21
A-1-b	35-60	25-35	29
A-1-a	60-80	30-42	38

Puppala (2008) summarized that the correlations proposed by various researchers show the accurate prediction of the resilient moduli for different types of geomaterials in their own inference spaces. However, the model correlations provide poor prediction when they are tested on different soils which are not used to develop the relationships (Von Quintus and Killingsworth 1998; Yau and Von Quintus 2002; Wolfe and Butalia 2004; Malla and Joshi 2006). Such problems should be expected because correlations are developed from data that may have shown large variations for similar types, similar compaction, and stress conditions. Table 2.3 provides the assessment to estimate the modulus of geomaterials either through laboratory or field testing or through empirical relationships.

Table 2.3 Assessment of Approaches in Estimating Moduli of Compacted Geomaterials (Puppala, 2008)

Correlation Type	**Reliability**	**Needs Additional Laboratory Studies for Verification?**	**Stress Estimation in Bases and Subgrades**
Laboratory Determined Parameters	Moderately Reliable	Yes	Not Needed
Field Determined Parameters	Moderately Reliable	Yes	Not Needed
Indirect Parameters	Low to Moderately Reliable	Yes	Needed

2.4 Factors Impacting Resilient Modulus of Compacted Geomaterials

The factors impacting the resilient modulus property of a compacted geomaterial have been studied by many researchers (Uzan 1985; Thom and Brown 1988; Mohammad et al. 1994a and b; Drumm et al. 1997; Lee et al. 1997; and Puppala 2008). A consensus on the major factors affecting the resilient modulus of geomaterials is proposed. The factors generally include the stress state, moisture content (including degree of saturation or suction), stress history, density (including void ratio), gradation (including the percentages of fines) and Atterberg limits. In the following section, those factors affecting the resilient modulus property are explained.

2.4.1 State of Stress

The state of stress is considered as the most significant factor that affects the resilient modulus of soils. The impact of the state of stress on the modulus is well-reflected in Equation 2-4. This nonlinear constitutive model has been recently developed by most agencies that are considering mechanistic-empirical design guide. According to Seim (1989), the confining pressure has greater effect on the resilient response of granular materials than deviatoric stress; while, for fine grain soils, the resilient behavior is more dependent on the deviatoric stress.

$$E = k_1 \sigma_c^{k_2} \sigma_d^{k_3} \tag{2-4}$$

From the stress conditions in the pavement materials presented in Figure 2.3, during the passing of traffic loads, the soil element is subjected by confining pressure (σ_c or σ_3) and deviatoric pressure (σ_d); therefore, it is reasonable to use Equation 2-4to explain the resilient behavior of the soils.

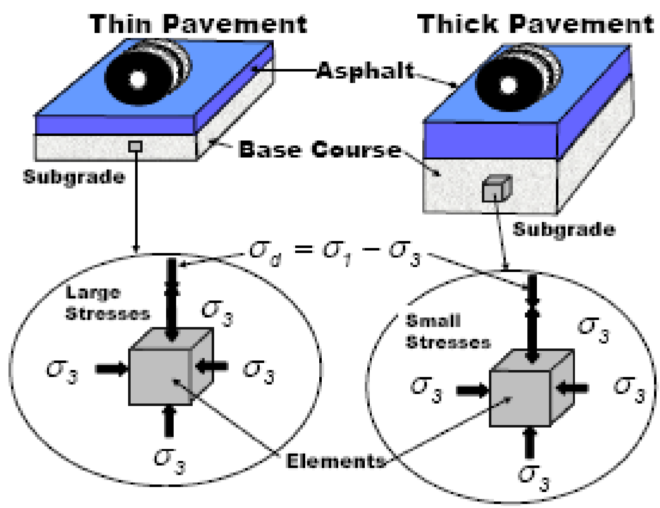

Figure 2.3 Stress level in a pavement (Hopkins et al. 2007)

The two-parameter models provided by the 1993 AASHTO design guide can be derived from Equation 2-4 by assigning a value of zero to k_2 (for fine-grained materials) or k_3 (for coarse-grained materials). As such, considering one specific model does not impact the generality of the conclusions drawn.

The term $k_1\sigma_c^{k_2}$ in Equation 2-4 corresponds to the initial tangent modulus at a given confining pressure. Since parameter k_2 is positive, the initial tangent modulus increases as the confining pressure increases. Parameter k_3 suggests that the modulus decreases with an increase in the deviatoric stress (or strain). The maximum feasible modulus from Equation (2-4) is equal to $k_1\sigma_c^{k_2}$, i.e. the initial tangent modulus.

The state of stress is bound between two extremes, when no external loads are applied and under external loads imparted by a truck. When no external load is applied the initial confining pressure, σ_{c_init}, is:

$$\sigma_{c\_init} = \frac{1+2k_0}{3}\sigma_v \tag{2-5}$$

where σ_V is the vertical geostatic stress and k_o is the coefficient of lateral earth pressure at rest. The initial deviatoric stress, σ_{d_init} can be written as:

$$\sigma_{d\_init} = \frac{2-2k_0}{3}\sigma_v \tag{2-6}$$

When the external loads are present, additional stresses, σ_x, σ_y and σ_z, are induced in two horizontal and one vertical directions under the application of an external load. A multi-layer algorithm can conveniently compute these additional stresses. The ultimate confining pressure, σ_{c_ult} is:

$$\sigma_{c\_ult} = \frac{1+2k_0}{3}\sigma_v + \frac{\sigma_x+\sigma_y+\sigma_z}{3} \tag{2-7}$$

and the ultimate deviatoric stress, σ_{d_ult}, is equal to

$$\sigma_{d\_ult} = \frac{2-2k_0}{3}\sigma_v + \frac{2\sigma_z-\sigma_x-\sigma_y}{3} \tag{2-8}$$

Under truck loads, the modulus can become nonlinear depending on the amplitude of confining pressure σ_{c_ult} and deviatoric stress of σ_{d_ult}. In that case

$$E = k_1\sigma_{c\_ult}^{k_2}\sigma_{d\_ult}^{k_3} \tag{2-9}$$

In the new mechanistic-empirical design guide (MEPDG), the resilient modulus constitutive model provided in Equation 2-10 is utilized. The model is generally referred as universal model. The main advantage of the model is being the consideration of the stress stage of the material during testing and generally provides a good fit to measured data.

$$M_R = k_1 p_a \left(\frac{\theta}{p_a}\right)^{k_2} \left(\frac{\tau_{oct}}{p_a} + 1\right)^{k_3} \tag{2-10}$$

Where, k_1, k_2, k_3 = material specific regression coefficients, θ = bulk stress,

p_a = atmospheric pressure (i.e., 14.7 psi), and τ_{oct} = octahedral shear stress.

According to Nazarian et al. (2011), the dependency of the modulus on the state of stress brings about several practical complications in the context of this study. These complications can be summarized into the following items:

- The modulus of a given geomaterial placed in a pavement section is not a unique value and depends on the underlying and/or overlying layers.
- The state of the stress of a given geomaterial placed in a pavement section can only be estimated if the moduli of all layers are known. As such, the estimation of the target modulus based on the design modulus has to be carried out iteratively using an analytical layered structural model (based on linear-elastic layered theory or nonlinear finite element).
- The sophistication of the selected analytical structural model impacts the design and target modulus.

2.4.2 Moisture Content

The impact of the moisture content (or alternatively degree of saturation or suction) is well studied in the literature. Excellent overviews of the impact of moisture content can be found in Richter (2006), Zaman and Khoury (2007) and Cary and Zapata (2010).

Wolfe and Butalia (2004), Hopkins et al. (2004) and Ooi et al. (2006) described the effects of degree of saturation on the resilient moduli of various compacted geomaterials. Kung et al. (2006) also evaluated the variations of resilient modulus and plastic strain with the post-construction moisture content and soil suction for cohesive subgrade soils. The testing results from the study indicated that the resilient deformation decreases when matric suction is increased, which result in an increase of the resilient modulus. Figure 2.4 presents a typical resilient modulus measurement of a subgrade material at different moisture content and related saturation conditions (Wolfe and Butalia 2004). The graph shows that the resilient moduli decrease when degree of saturation of the soil increase and a decrease of close to 70 MPa was observed in the moduli value when the soils was subjected to full saturation.

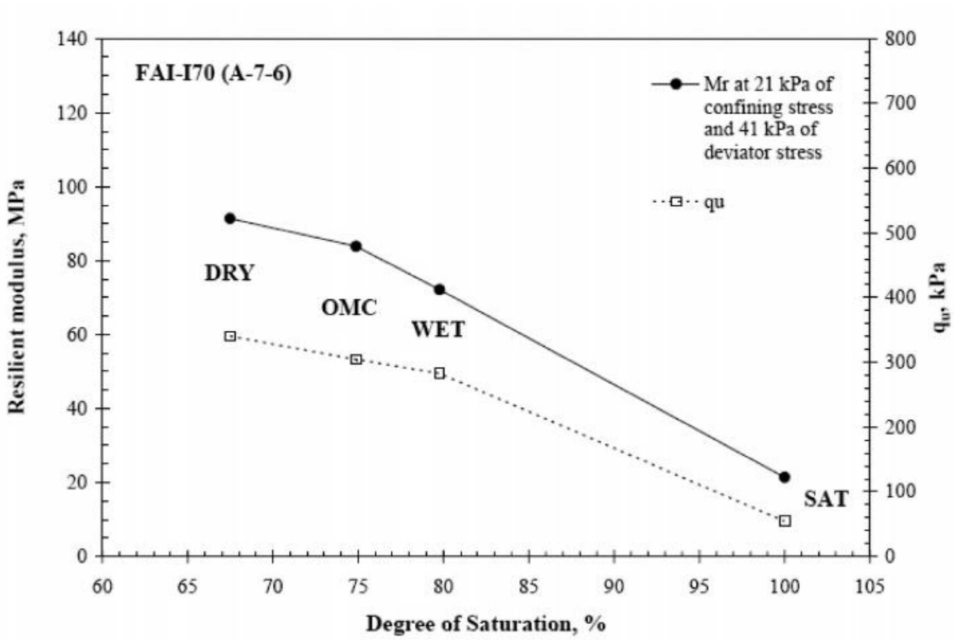

Figure 2.4 Resilient moduli at different saturation conditions
(Wolfe and Butalia 2004)

Drumm et al. (1997) investigated the variation of resilient modulus with an increase in post compaction moisture content and proposed a method for correcting the resilient modulus for increased degree of saturation. The resilient modulus at higher saturations is estimated using the gradient of resilient modulus with respect to degree of saturation. Based on that research, the following model was developed for resilient modulus:

$$M_R = k_3\ (\sigma_d + X\ \Psi_m)^{k4} \tag{2-11}$$

where k_3 and k_4 are regression parameters; M_R = resilient modulus; σ_d = deviatoric stress; Ψ_m = matric suction; and X = function of degree of saturation.

Zaman and Khoury (2007) also focused on evaluating the effect of post-compaction moisture content on the resilient modulus of selected soils in Oklahoma. To test specimens at different suction levels, the authors used wetting and drying process on samples compacted at predetermined water contents. For example, samples compacted at optimum moisture contents (OMC) were dried to OMC-4% and then wetted to OMC+4%. After the completion of resilient modulus testing, the filter paper tests were performed. Suction tests at various moisture levels were used to establish soil-water characteristic curve (SWCC) profiles. Authors observed that the resilient modulus-moisture content relationships of all the selected soils exhibit a hysteric behavior due to wetting and drying processes. The resilient modulus showed an increasing trend with soil suction.

Nazarian and Yuan (2008) studied the relationship between resilient modulus and moisture content of soils. They determined the layer moduli under different moisture contents using seismic based nondestructive testing. Moreover, laboratory tests were carried out to quantify the moisture susceptibility of the materials. Resilient modulus tests were conducted on soil samples compacted at OMC and then left for drying and wetting process for a span of 15 days. Test results were analyzed to monitor the changes in modulus with moisture content. Nazarian and Yuan observed that:

- Under constant compaction effort, the maximum modulus (M_R) was obtained at a moisture content lower than OMC.
- The difference between the optimum moisture content and the moisture content at which the maximum modulus was determined was dependent on the fine content of the mixture.

According to Cary and Zapata (2010), the effects of the environmental factors on the M_R can be evaluated and expressed as the following function

$$M_R = F_{env} \times M_{Ropt} \quad (2\text{-}12)$$

Where F_{env} is the composite environmental adjustment factor and $M_{R\text{-}opt}$ is the resilient modulus at optimum conditions and at any state of stress. The model internally used by the MEPDG program to estimate the effect of moisture change on moduli is given by:

$$Log\left(\frac{M_R}{M_{R-opt}}\right) = a + \frac{b-a}{1+EXP\left(\beta+k_m*(S-S_{opt})\right)} \quad (2\text{-}13)$$

Where M_R = Modulus at any degree of saturation, S = current degree of saturation (decimal), $M_{R\text{-}opt}$ = Modulus at OMC and MDD, S_{opt} = Degree of saturation at OMC (decimal), a = Minimum of log(M_R/$M_{R\text{-}opt}$), b = Maximum of

$\log(M_R/M_{R\text{-}opt})$, β = Location parameter as a function of a and b = $\ln(-b/a)$, and K_m = Regression parameter.

Hossain (2008) conducted a study for the Virginia DOT to evaluate the use of resilient modulus values in the MEPDG design and analysis. Quick direct shear test was performed at confining pressure of 5 psi at the end of resilient modulus testing to develop correlations between resilient moduli and shear strength properties. To verify the saturation based MEPDG resilient modulus model, a set of samples were compacted and tested at OMC and at WOMC (20% more moisture than the OMC).

Sawangsuriya et al. (2008) studied matric suction, small strain shear modulus and compaction properties of various soils to present various empirical relations. Various compaction moisture content regimes including dry to wet of optimum with Proctor and reduced Proctor energies were studied. A generalized relationship among modulus-suction-compaction conditions was developed.

Edil et al. (2006) provided the relationship between resilient modulus and matric suction of subgrade soils as shown in Figure 2.5. The values of resilient modulus, M_R, increase with an increase in matric suction. This is because an increase in suction is always associated with dry condition of the soil specimen (low water content) and according to the previous studies, the resilient modulus increases when soil is in dry condition.

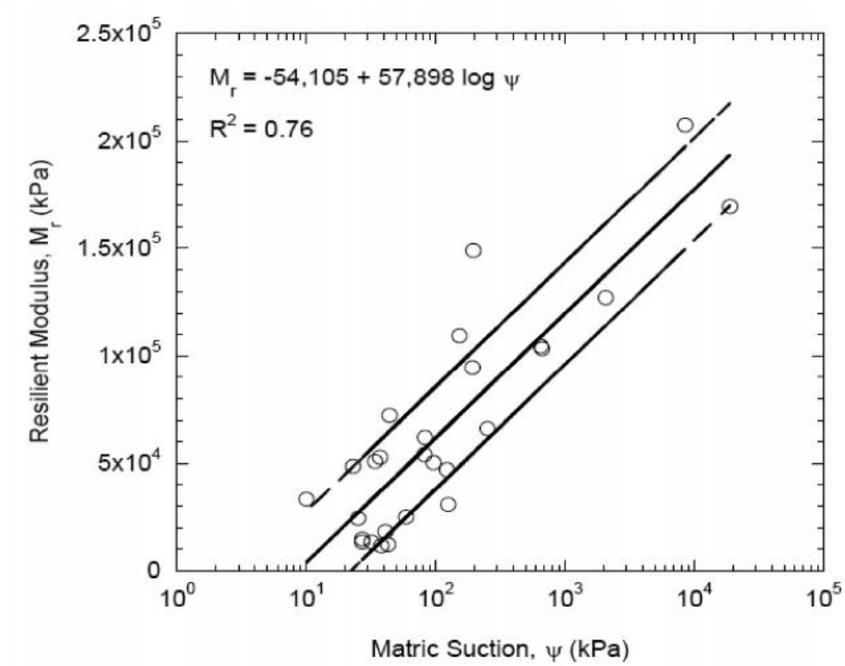

Figure 2.5 Effect of matric suction on resilient modulus (Edil et al. 2006)

Siekmeier (2011) based on the compilation of a number of studies proposed the following equation for estimating modulus as a function of moisture content:

$$M_r = k_1 \times p_a \times \left(\frac{\sigma_{eb} + f_s \theta_w \Psi}{p_a}\right)^{k_2} \times \left(\frac{\tau_{oct}}{p_a} + 1\right)^{k_3} \quad (2\text{-}14)$$

where p_a= atmospheric pressure, σ_{eb}= external bulk stress, τ_{oct}= octahedral shear stress, θ_w= volumetric moisture content, θ_{sat}= volumetric moisture content at saturation, and Ψ= matric suction. Siekmeier proposed relationships for estimating parameters k_1 through k_3 and f_s andΨ for fine-grained soils.

2.4.3 Dry Density

The impact of dry density on modulus has not been studied as extensively as the impact of moisture content since field acceptance of compacted geomaterials in most specifications is based on achieving a certain density. Increase in density should intuitively correlate to increase in modulus. In many field studies a strong correlation between modulus and density could not be found (e.g., Mooney et al. 2009; and Von Quintus et al. 2010).

Pacheco and Nazarian (2011) attributed the lack of strong correlation to the differences in the compaction efforts between the field and lab tests and to the complex interaction between the moisture content, dry density and degree of saturation of a given material. In laboratory tests, the maximum dry density is obtained by preparing a series of specimens with different moisture contents and then, compacting them with a constant energy. Irrespective of the moisture content, the same material is compacted in the field with the minimum number of passes to achieve a desired density. As such, there may be a significant difference in the field and lab compactive energy that may impact the modulus of the materials.

To test this concept, Pacheco and Nazarian prepared a number of specimens to the same maximum dry density but with different moisture contents following the Proctor method. The amount of energy to achieve a target density at each moisture content, which was determined by trial and error, varied from 32 hammer blows (for wet specimens) to more than 90 blows (for dry specimens). The modulus decreased in all cases as the compaction moisture content increased, even though the densities of the specimens were more or less the same. The ratios of the moduli at the dry and wet compaction states varied by as low as 2 for clays to as high as 17 for a high-fines content unbound aggregate base material.

2.4.4 Gradation and Plasticity

The impact of gradation and plasticity on modulus have been extensively qualified (see Richter 2006; Puppala 2008) and to lesser extent quantified. Table 2.4 contains several relationships developed to quantify the impact of these variables. In general, as the plasticity of the material and the percent fines increases, the modulus decreases.

Table 2.4 Parameters Relating Modulus to Index Properties of Geomaterials

Model Type	**Developed by**	**Gradation and Plasticity Parameters Included**
$M_r = k_1 \ P_a \left(\frac{\theta}{P_a}\right)^{K_2} \left(\frac{\tau_{OCT}}{P_a}\right)^{K_3}$	Malla and Joshi (2008)	Percent passing 3", 1", $1\frac{1}{2}$", #40, #20, #200 sieves Percent fine sand C_U=Uniformity coefficient C_C= Coefficient of curvature
$M_r = k_1 P_a \left(\frac{\theta}{P_a}\right)^{k_2} \left(\frac{\sigma_d}{P_a}\right)^{k_3}$	Glover and Fernando (1995)	Liquid limit and plastic limit Specific gravity of soil binder Percent passing sieve #40 Dialectic constant
$\frac{M_r}{\sigma_{atm}} = K_1 \left(\frac{\sigma_{oct}}{\sigma_{atm}}\right)^{K_2} \left(\frac{\tau_{oct}}{\sigma_{atm}}\right)^{K_3}$	Mohammad (1999)	Liquid limit and plastic limit
$M_r = k_1 P_a \left(\frac{\theta}{P_a}\right)^{k_2} \left(\frac{\tau_{OCT}}{P_a} + 1\right)^{k_3}$	Amber (2002)	Percentage passing sieve #$\frac{3}{8}$, #4, #40 Liquid limit and plastic limit

2.4.5 Long-term and Short-term Behaviors of Geomaterials

In a proper field compaction, the geomaterial is placed near the optimum moisture content and the moisture change is due to either evaporation or the introduction of moisture. The moduli obtained from this process could be different than the moduli measured in the lab under a constant compaction effort (Khouryand Zaman 2004; Sabnis et al. 2009 and Pacheco and Nazarian 2011). This may be the reasons that the past experiences in correlating the laboratory and field moduli have yielded mixed success (Hossain and Apeagyei 2010).

Significant work has been done to predict the long-term changes in the moisture content/suction and modulus of the compacted geomaterials under the in service pavement. However, the amount of work related to short term behavior of exposed geomaterials (as related to the quality management) has been limited.

The Enhanced Integrated Climatic Model (EICM) is an integral part of the Mechanistic Empirical Pavement Design Guide (MEPDG), and perhaps the most common algorithm used for predicting the long term change in modulus of compacted soils. EICM involves analysis of moisture and heat flow through different pavement layers under different boundary conditions. One of the main functions of the EICM in the MEPDG design guide is to evaluate the relationships between the change in water content and mechanical properties of unbound pavement layers.

The EICM estimates the change in water content in the pavement layers, the drainage and conductivity characteristics of the layers, and the water storage capacity of each layer, based on the boundary conditions on the ground surface, the depth of moisture change zone, and equilibrium moisture content (or suction), and initial conditions. The current EICM uses empirical relationships between the modulus of compacted soils and the degree of saturation.

The EICM consists of four major parts: The Precipitation (PRECIP) model, the Infiltration and Drainage (ID) model, the Climatic-Material-Structural Model (CMS) model, and the U.S. Army Cold Regions Research and Engineering Laboratory (CRREL) model for Frost Heave-Thaw Settlement. These models are integrated to some extent through the use of some common boundary conditions with typical inputs and outputs (Zapata 2009). Since each of these models was originally developed as a separate program for a specific use, there is significant amount of overlap between the functions, capabilities, and limitations of these models (McCartney et al. 2010).

Zapata and Houston (2008) conducted a comprehensive study to incorporate new empirical relationships into the EICM for the saturated hydraulic conductivity and the SWCC for further improvement of the EICM model. The study involved collection of soils from 30 sites throughout the USA, collection of weather data from online databases for these sites, and prediction of the water content from the sites using the EICM. The study also involved in comparing the predicted water content from the EICM with the field measurements. Although this extended database improved the capabilities of the EICM model, the current MEPDG empirical equation that correlates the resilient modulus and degree of saturation with the regression fitting parameters, do not consider the effects of mechanical stress on the resilient modulus in detail.

More recent studies have focused more on the combined effects of the two stress state variables as adopted in unsaturated soil mechanics (i.e., suction and mechanical stress). Gupta et al. (2007) observed a more consistent trend between resilient modulus and suction at constant confining stresses. Suction and degree of saturation relationship for each soil is unique and is established through the SWCC. As such, the degree of saturation (or water content) may not be the primary variable affecting the resilient modulus. In other words, the degree of saturation for different soils at the same suction may be different. Nevertheless, it may be more practical to consider moisture content or degree of saturation in the day-to-day protocols to be implemented by highway agencies.

2.5 Others Literatures Cited on Resilient Modulus Properties

In this section, others research reports and papers pertaining to resilient modulus of subgrades materials are thoroughly reviewed and collected. In the following, few literatures related to resilient modulus topic are briefly described.

Thompson and Robnett (1976) studied resilient modulus properties of several Illinois subgrade soils at the University of Illinois, Urbana-Champaign. This study reported the test results and the development of correlations between resilient modulus and subgrade soils properties. They proposed an arithmetic model to describe the resilient properties of fine-grained soils.

Shook et al. (1982) later discussed an Asphalt Institute method of designing flexible pavements in which a bulk stress model was used to model the resilient behavior of cohesionless soils. Then, in 1985, Uzan explained the limitations of the bulk stress model with two model constants and introduced a two-parameter model consisting of both bulk and deviatoric stresses with three model constants to simulate the resilient behavior of subsoils.

Elliott et al. (1988) reported resilient moduli test results obtained on different cohesive subgrades in Arkansas. The main intent of this research was to explain the effects of field moisture content on the resilient moduli of compacted subgrades. This study also addressed the effects of compaction procedures and moisture content variation on the resilient moduli properties. The test results showed that M_R values of the subgrade soils decrease when moisture content increases. Figure 2.6 presents the variation of resilient modulus of fat clay (CH) from Jackport, with respect to percent changes in optimum moisture content. A decrease in the M_R value close to 1 ksi was reported for percent increase in the optimum moisture content. Similar findings were reported on other subgrade soils in this study. Moreover, this study was supported by similar testing results and conclusions provided in the study of resilient properties on 14 different Nebraska soils by Woolstrum (1990).

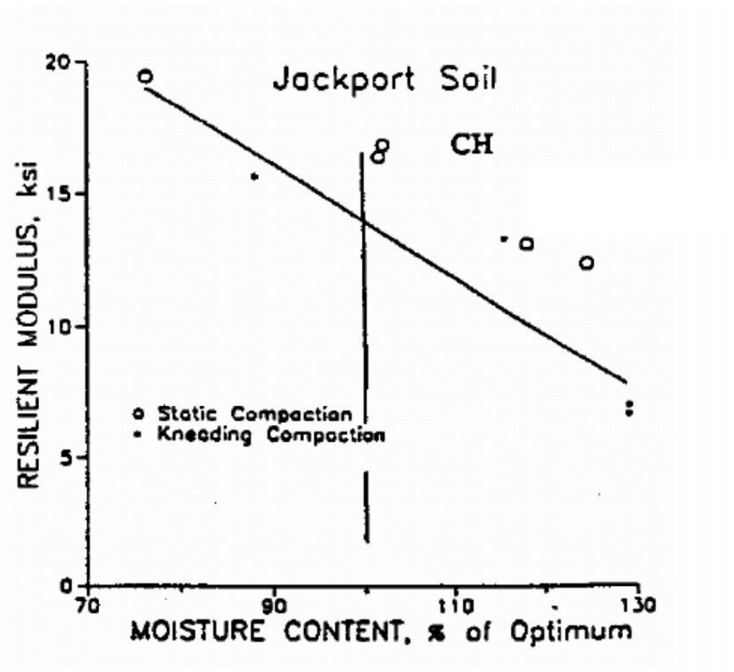

Figure 2.6 Resilient modulus of CH soil from Arkansas versus percent optimum moisture content (Elliott et al. 1988)

Santha (1994) studied the resilient properties of subgrade soils from 35 test sites in Georgia, using T-274 procedure. These results show wide variations in M_R with respect to soil type and compaction procedures used in the testing. The measured data were used and analyzed with two-parameter and three-parameter models. The authors found that the three-parameter model captured the measured resilient properties better than two-parameter model.

Burczyk et al. (1995) investigated resilient properties of Wyoming subgrades. The RLT test setup and AASHTO T-294 procedure were used in this study. Various fundamental soil properties that influence resilient properties of the subgrade soils were also discussed and various back calculation methods used to interpret the resilient properties were also evaluated by the researchers.

Mohammad et al. (1994a, b, 1995) studied on the resilient modulus properties of Louisiana subgrades. They performed a complete evaluation of RLT setup and AASHTO test procedures T-292 and T-294, as well as measurement systems including linear variable differential transformers (LVDTs) placed for the middle third of the specimen (referred to as middle) and at the end of the soil specimen (referred to as end), both being internal deformation systems in yielding reliable resilient properties. Figure 2.7 presents resilient moduli determined from both internal measurement systems of a sandy specimen tested in the research.

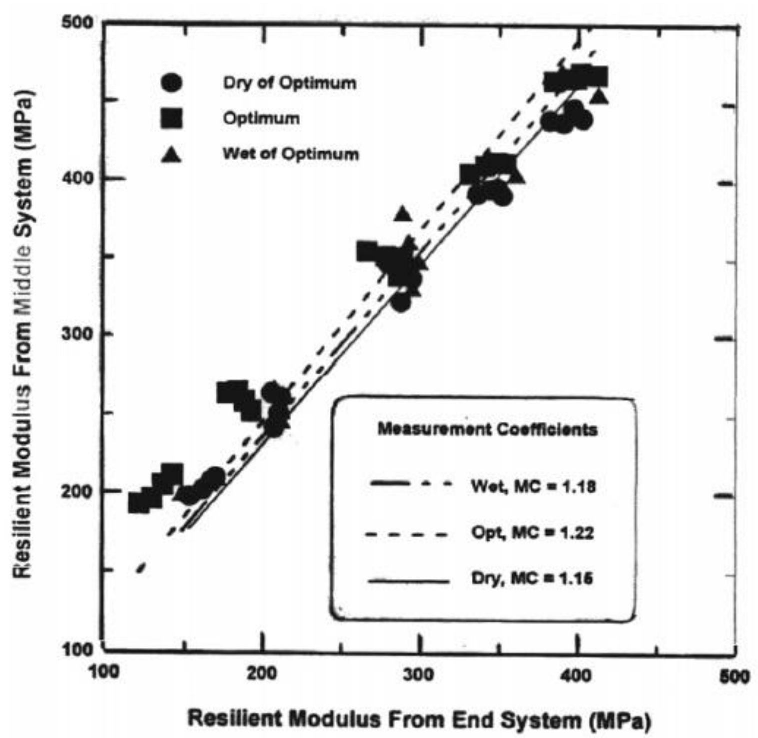

Figure 2.7 Resilient modulus results measured from both end and middle deformation measurement systems (Mohammad et al. 1994a, b)

Higher moduli values were measured from the middle LVDT system than the end LVDT system. The researchers found that comparing with the resilient moduli measured from the end LVDT system, a 15% to 20% increase in moduli values obtained with the middle measurement system.

Liang et al. (2008) studied on a new predictive model on the resilient modulus of cohesive soils, using the soil suction concept. The model used for predicting the effect of moisture variation on resilient modulus of unsaturated cohesive soils takes the following form

$$M_r = k_1 \times p_a \times \left(\frac{\theta + \chi_w \Psi_m}{p_a}\right)^{k_2} \times \left(\frac{\tau_{oct}}{p_a} + 1\right)^{k_3} \qquad (2\text{-}15)$$

where p_a = atmospheric pressure, θ= bulk stress, τ_{oct} = octahedral shear stress, χ_w = Bishop's parameter, Ψ_m = matric suction, and k_1, $k_2$and k_3 = regression constants.

The soil water characteristic curve (SWCC) for the soil samples were firstly measured using filter paper technique and then, the value of Bishop's parameter and matric suction could be evaluated from the SWCC. The proposed model provides good prediction of the variation of resilient modulus due to change of moisture. Moreover, the model also compares well with the empirical equation in the new mechanistic empirical pavement design guide (MEPDG) in predicting the effect of moisture content variation on the resilient modulus. Figure 2.8 shows the comparison between predicted M_R and measured M_R of the A-4 soil samples. It can be concluded that the proposed model (with suction) provides better prediction of M_R than the model without suction.

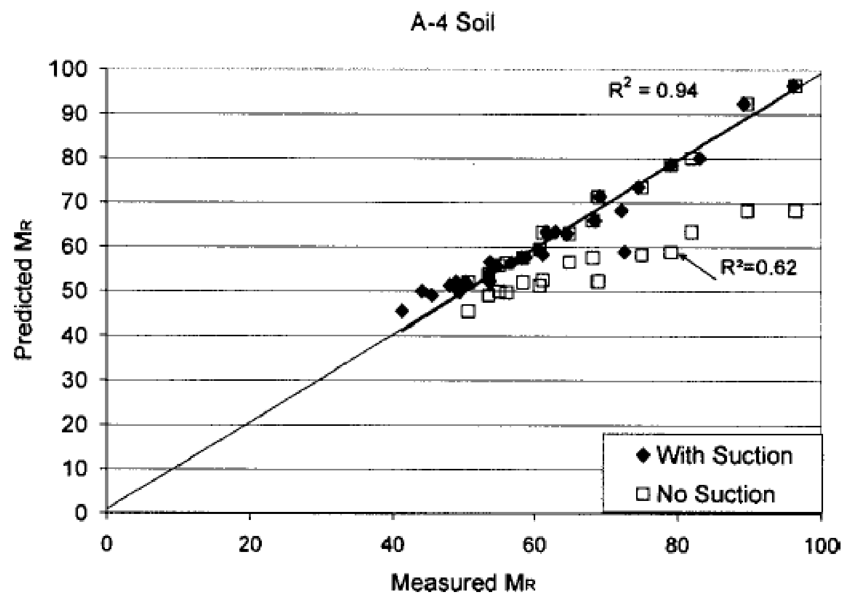

Figure 2.8 Comparison of predicted and measured resilient modulus for A-4 soils (Liang et al. 2008)

Yang et al. (2008) developed a suction-controlled testing system integrating the resilient modulus test and axis-translation technique to examine the effects of matric suction on the resilient modulus of two compacted subgrade soils. The testing system was also created to validate the suitability of the axis-translation technique on the resilient modulus test. The experimental data indicate that matric suctions measured in the specimen after consolidation and resilient modulus tests are consistent with the matric suction deduced from the SWCC corresponding to the same moisture content. The resilient modulus obtained by the suction-controlled resilient modulus test appears to be reasonable and the trends of the resilient modulus are consistent with those obtained by the conventional resilient modulus test.

2.6 Summary

This chapter covered the concept of resilient modulus and the approaches to estimate the modulus (laboratory tests, field tests, and correlations). The factors affecting the modulus were also described. Previous literatures cited on resilient modulus theory and the research conducted on the resilient modulus parameters of unsaturated soils were reviewed and were also presented in this chapter. In the next chapter detailed test procedures followed in the current research are presented.

CHAPTER 3

EXPERIMENTAL PROGRAM

3.1 Introduction

This chapter presents the details of experimental program performed to determine the resilient properties of the subgrade soils collected from three different sites. The experimental program consists of basic soil tests and advanced soil tests, which were conducted to find the physical properties and engineering properties of the soils respectively. The following sections describe the physical properties and testing materials used in this research, types of laboratory tests performed, test equipment, and the test procedures adopted.

3.2 Basic Soil Tests

The subgrade materials used in this study are collected from three different sites, which are Minnesota, Mississippi, and Louisiana. In order to determine physical properties of the soils, the basic soil tests including grain size distribution test, specific gravity, and proctor compaction test were accomplished. Those basic soil tests were done in accordance with the current TxDOT and AASHTO standard testing procedures.

3.2.1 Grain Size Distribution

The test procedure for particle size analysis of soils (sieve analysis test), Tex-110-E, provided in TxDOT Designation was followed to obtain the grain-size distribution of the soils. Sieve analysis test provides the percent amount of various size fractions of the soils (%gravel, %sand, and %fines). Moreover, the distribution of particle size of the soils retained on No. 200 sieve is also provided by sieve analysis test.

3.2.2 Atterberg's Limits

Upon addition of water, the states of soil changes from dry, semi-solid, plastic and finally to liquid limit states. The water content at the boundaries of these states are known as shrinkage (SL), plastic (PL) and liquid (LL) limits, respectively (Lambe and Whitman 2000). Also known as Atterberg's limits, the above mentioned soil properties are essential to correlate the shrink-swell potential of the soils to their respective plasticity indices. LL is known as the water content at which the soil flows and PL is determined as the water content at which the soil starts crumbling when rolled into a 1/8-inch diameter thread. The numerical difference between LL and PL is known as plasticity index (PI) and characterizes the plasticity nature of the soil. Representative soil specimens from different locations as mentioned before were subjected to Atterberg limit tests to determine LL and PL following Tex-104-E and Tex-105-E, respectively.

The results of the sieve analysis of Minnesota, Mississippi, and Louisiana subgrade soils are shown in Table 3.1 and the summary of physical properties that were evaluated as a part of this study are provided in Table 3.2. From the gradation results, the soils can be classified by using the USCS classification method. The Minnesota soil was classified as CH; while, the Mississippi and Louisiana soil were classified as ML and SC respectively. The grain size distributions of the soils plotted with sieve analysis results are illustrated in Figure 3.1. According to the MEPDG design guide (NCHRP 2004), the data received from grain size analysis and Atterberg's limits test results can be used in the predictions of maximum dry density ($\gamma_{d,max}$), optimum moisture content (w_{opt}), degree of saturation at optimum moisture content (S_{opt}), and specific gravity (G_s) including the soil water characteristic curves (SWCCs) of the soils in case the laboratory test data of these values are not available.

Table 3.1Sieve Analysis

Sieve	Opening (mm)	Percent Passing (%)		
		Minnesota	**Mississippi**	**Louisiana**
1"	25.4	100	100	100
7/8"	22.4	100	100	100
3/8"	9.52	100	100	100
#4	4.75	100	100	100
#40	0.425	99	78	100
#100	0.15	98	60	47
#200	0.075	97	59	45

Table 3.2 Basic Soil Properties

Soil Properties	**Minnesota**	**Mississippi**	**Louisiana**
Gravel (%)	0.0	0.0	0.0
Sand (%)	2.8	41.5	55.0
Fine (%)	97.2	58.5	45.0
Liquid Limit, LL	86	0	23
Plasticity Index, PI	53	0	12
Specific Gravity, G_s	2.78	2.65	2.72
USCS Classification	CH	ML	SC

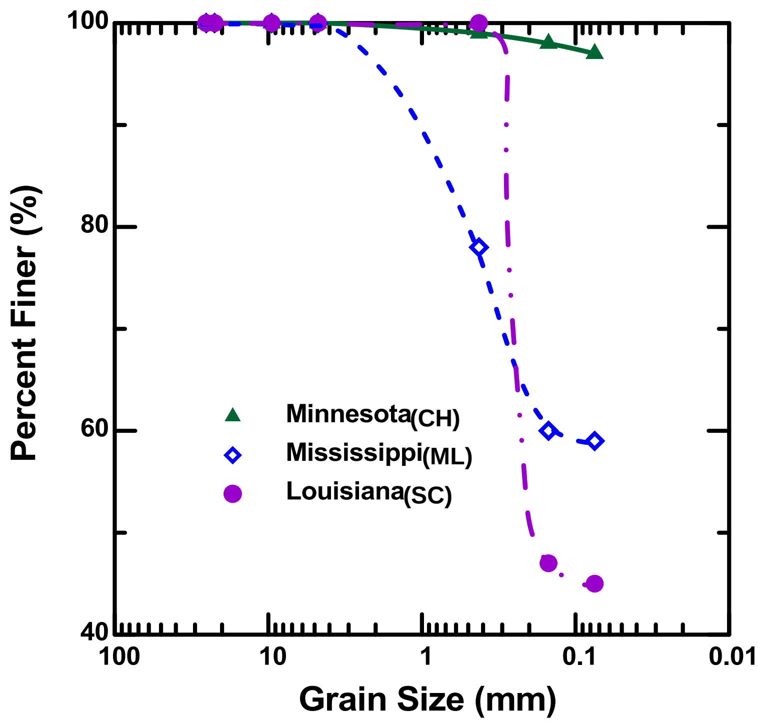

Figure 3.1 Grain size distributions

3.2.3 Standard Proctor Compaction Tests

The purpose of standard proctor compaction test is to find the optimum moisture content of soils at which the maximum dry unit weight is attained. Specimens exhibiting a high compaction unit weight are best in supporting civil infrastructure due to low volume of voids (Pedarla 2009). The standard proctor compaction tests were conducted as per the TxDOT procedure (Tex-114-E), on Minnesota, Mississippi, and Louisiana soil samples to establish the laboratory compaction characteristics and the moisture-density relationships. Then, the optimum moisture content and maximum dry unit weight of the soils were determined.

The Tex-114-E procedure requires a compactive effort of 7.30 ft.-lb./in^3 (604 kN-m/m^3). Based on this requirement, a subgrade specimen size of 4 in. (101.6 mm) in diameter and 6 in. (152.4 mm) in high is molded in four layers by using a 5.5 lb. (2.5-kg) hammer dropped 25 times per layer from a height of 12 in. (304.8 mm). Table 3.3 presents the summary of the compaction parameters used in the standard compaction test for the subgrade materials. Figure 3.2 below shows the compaction dry unit weight and moisture content relationship of Minnesota, Mississippi, and Louisiana subgrade soils.

Table 3.3 Compaction Parameters

Required Compactive Effort (ft-lb/in^3)	7.30
Weight of Hammer (lb)	5.5
Height of Drop (in)	12
Diameter of Sample (in)	4
Height of sample (in)	6
Volume of Molded Specimen (in^3)	75.4
No. of Layers	4
Drops per Layer	25
Applied Compactive Effort (ft-lb/in^3)	7.30

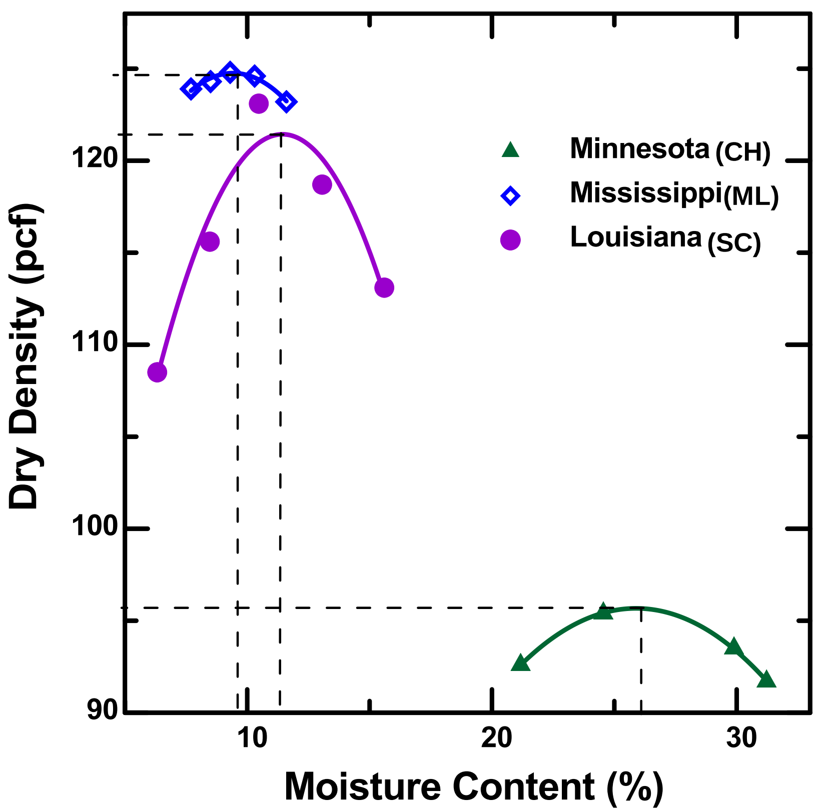

Figure 3.2 Compaction curves

From the moisture content and dry density relationship curves provided in Figure 3.2, the values of maximum dry density (MDD) and optimum moisture content (OMC) of the soils were determined and summarized in Table 3.4.

Table 3.4 Maximum Dry Density and Optimum Moisture Content Properties of Three Subgrade Soils

Subgrade Soils	**MDD (pcf)**	**OMC (%)**
Minnesota (CH)	96	26
Mississippi (ML)	124.6	9.4
Louisiana (SC)	121.4	11.4

3.3 Advanced Soil Tests

In this research, advanced tests conducted on the subgrade soil specimens consist of unconfined compressive strength test and conventional resilient modulus test. Soil water characteristic curves (SWCC) of the soil samples were first determined using Tempe cell (Fredlund device) and filter paper technique. Brief descriptions on the processes of the advanced soil tests conducted here are provided in the following sections.

3.3.1 Specimen Preparation Procedure

In order to determine the SWCCs of the soil samples, soil specimens were prepared at three different moisture content-dry density conditions, which are OMC, dry-side of OMC (0.8XOMC), and wet-side of OMC (1.2XOMC) conditions. The points at which the soil specimens were prepared are presented in Figure

3.3. From the moisture content and the corresponding dry density conditions, the water and dry soil weights required for each specimen were calculated. Then, soil specimens of 2.5 in. in diameter and 1 in. thick were compacted with a constant strain rate using static compaction equipment. This specimen prepared was used for testing with both Tempe cell device and filter paper techniques.

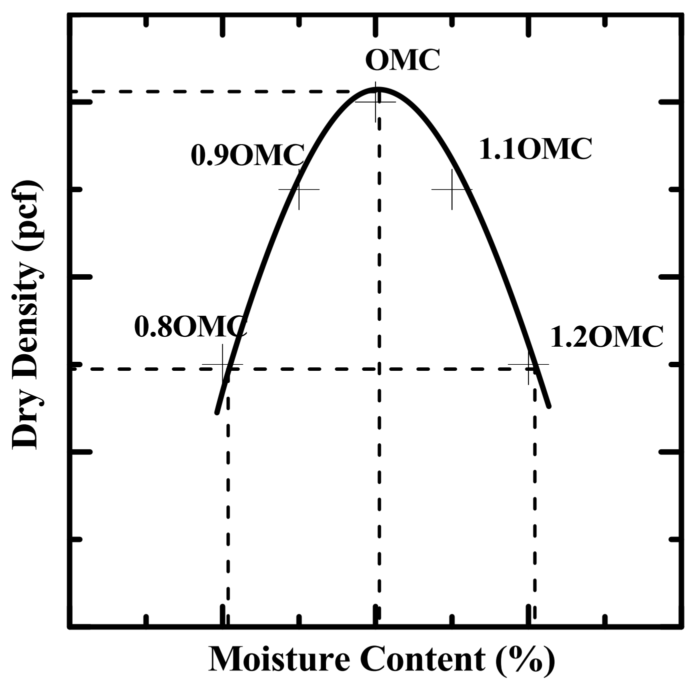

Figure 3.3 Specimen preparation compaction points for SWCC and M_R tests

The specimens for Unconfined Compressive Strength (UCS) test and Resilient Modulus test were compacted with a constant strain rate using static compaction equipment. The specimens were prepared at five different moisture contents (0.8OMC, 0.9OMC, OMC, 1.1OMC, and 1.2OMC) with their respective dry density as presented in Table 3.5.

Table 3.5 Sample Preparation Points for UCS and M_R Testing

Specimen No.	Minnesota (CH)		Mississippi (ML)		Louisiana (SC)	
	Dry Density (pcf)	Moisture Content (%)	Dry Density (pcf)	Moisture Content (%)	Dry Density (pcf)	Moisture Content (%)
1	92.5	21.0	123.4	7.4	118.7	9.1
2	95.0	23.5	124.3	8.4	120.7	10.3
3	96.0	26.0	124.6	9.4	121.4	11.4
4	95.0	28.5	124.3	10.4	120.7	12.5
5	92.0	31.0	123.4	11.4	118.7	13.7

The dimension of specimen prepared for unconfined compressive strength test and resilient modulus test was 2.8 in. (71 mm) in diameter and 5.6 in. (142 mm) in height. After compaction, the specimens have been extruded and were kept in the humidity room for at least one day for uniformity distribution of moisture in the specimens and then tested.

3.3.1.1 Saturation Process

In the first process to determine SWCCs of the soils, the prepared specimens needed to be saturated. The specimens were placed in a stainless mold and were clamped with two iron plates on the top and bottom in order to restrict the volume change. The process of specimen saturation is shown in Figure 3.4. The saturation process is carried out in the following steps.

i. De-ionized water was filled till the half the height of the sample and left for 10 to 12 hours.

ii. After initial saturation, the sample is completely submerged in the water for a period of 12 hours.

The purpose of this process is to eliminate any entrapped air in the sample, which ensures chances of fully saturation in the sample. After saturation, the specimen with stainless mold is immediately placed on the ceramic disk and installed into the Tempe cell device.

Figure 3.4 Saturation of a soil sample

3.3.2 Soil Water Characteristic Curve

Soil water characteristic curve (SWCC) describes a unique relationship between matric suction and moisture content of soils. The water content defines the amount of water contained in the soil pores, which can be expressed as gravimetric water content (w), volumetric water content (θ), or degree of saturation (S). The volumetric water content is most commonly used in soil science (Leong and Rahardjo 1996); however, for geotechnical practice, gravimetric water content is most commonly used (Thudi 2006).

Matric suction is the capillary component of free energy and is the major contributor to the total suction as osmotic suction arising from salt solutions in a soil is typically small. In general, matric suction is the difference between pore air pressure and pore water pressure. Matric suction is generally related to the surrounding environment and it may vary from time to time. The recent advances in the design of pavements including mechanistic pavement design guide (MEPDG) has emphasized the importance of unsaturated soil properties and the role of matric suction on the subgrade stiffness property, resilient modulus, and its use in the pavement design (Puppala et al. 2012).

There are many types of methods, using filter paper, Tempe cell, and pressure plate, to measure SWCCs of soils. In this study, both Tempe cell and filter paper methods were considered to be used for generating the SWCCs of the subgrade soil samples.

3.3.2.1 Tempe Cell Method

Tempe cell, which is also called the Fredlund SWCC device, is a simple unsaturated soil testing apparatus with great flexibility for applying matric suction. The device can be used to obtain the complete SWCC for a soil. The Tempe cell works on the principle of axis translation technique which involves the soil matric suction in different steps and measuring the resulting water content after equilibrium is reached at each air pressure applied.

The main components of Tempe cell device consist of a pressure panel with dial gauges and regulators as well as volume indicator tubes, a pressure cell, and a bottom plate with embedded ceramic disk. The schematic of the Fredlund SWCC device is illustrated in Figure 3.5. In the test, a saturated soil specimen was placed on top of a saturated ceramic disk. The ceramic disk acts like a semi-permeable medium and allows water, but not air, to pass through the disk up to a rated air pressure value (i.e., air entry value of the ceramic). The ceramic disk with a capacity of 5 bars (500 kPa) air entry pressure was used in this study. After setting up the specimen with ceramic disk completely, the pressure cell was installed and connected to the pressure panel. Figure 3.6 (a) presents the GCTS setup used in this research and Figure 3.6 (b) shows the saturation of a mounted ceramic disk.

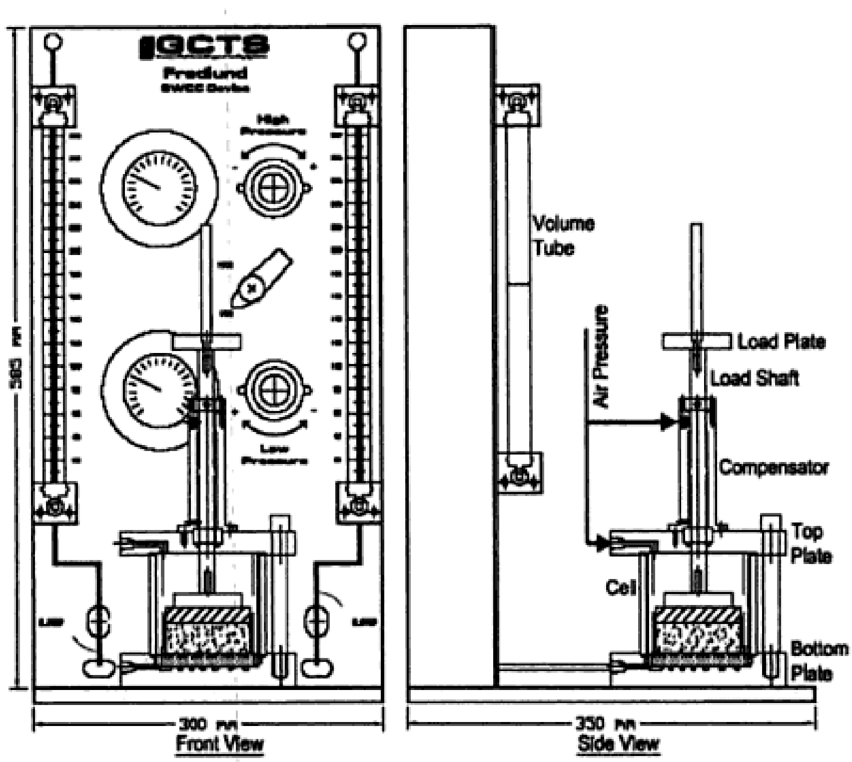

Figure 3.5 Schematic of the Fredlund SWCC device (Padilla et al. 2005)

(a) (b)

Figure 3.6 (a) Tempe cell setup used in this research
(b) Saturation process of HAE disk

Pore-air pressure (u_a) was applied to the soil specimen using the pressure regulators on the pressure panel. The air pressures used in this research were 10 kPa, 50 kPa, 100 kPa, 200 kPa, 300 kPa, and 450 kPa. The bottom plate of the Tempe cell has two external ports connected to the volume indicator tubes to measure the water extracted from the soil specimen. This device allows the use of single soil specimen to obtain the entire SWCC with any number of data points.

Before applying the air pressure on the installed specimen, the Tempe cell system needed to be flushed in order to remove the diffused air bubbles below the ceramic disk. The flushing process consists of pushing de-ionized water through the spiral compartment below the ceramic disk. The water was flushed back and forth until no air bubbles were observed.

3.3.2.2 Filter Paper Method

The filter paper method, introduced by Gardner (1937), has been used for soil suction measurement for over seventy years. This method is based on the water-absorption characteristics of a filter paper. When a filter paper is placed into the environment of a soil, it will absorb the moisture until equilibrium condition is reached. The suction in the soil can be estimated from the water content absorbed by the filter paper with a calibration of suction versus water content relationship of the filter paper. Either matric suction or total suction (a summation of matric suction and osmotic suction) can be estimated by the filter paper method, depending on the condition of contact between the filter paper and the soil (Guan 1996).

This study focus on the estimation of matric suction which can be determined by placing the filter papers directly in contact with soil specimen so that the equilibrium can be achieved by exchanging of moisture between the soil and filter papers via capillary flow of water. The soil specimen and filter paper are allowed to equilibrate for a period of at least seven days at a constant temperature of $25 \pm 1°C$ (Puppala et al. 2011).

Based on the ASTM D 5298-03 method, the most commonly used filter papers are Whatman No. 42 and Schleicher & Schuell No. 589 filter papers. For this research, the Whatman No. 42 filter paper was selected. As stated previously, the suction in the soil can be evaluated with the filter paper calibration curves. Figure 3.7 presents the calibration curves of suction-water content for the both types of filter papers (Whatman No. 42 and Schleicher & Schuell No. 589), adopted by the ASTM D 5298-03. Equations 3-1 and 3-2 show the relationships between the matric suction and filter paper water content for the Whatman No. 42 filter paper, and the same is presented in the Figure 3.7.

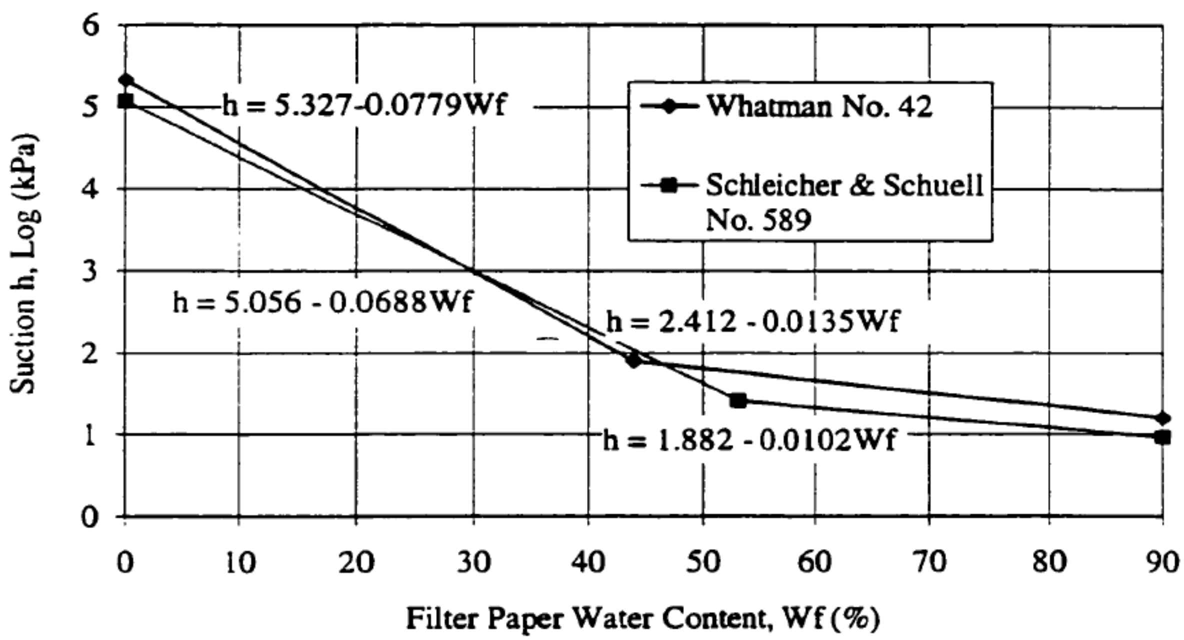

Figure 3.7 Calibration of suction-water content curves for filter papers (from ASTM D 5298-03, Guan 1996)

The equations of the matric suction calibration curves for the Whatman No. 42 filter paper are:

$$\log h = 5.327 - 0.0779\, w_f, \qquad \text{for } w_f < 45.3\ \% \qquad (3\text{-}1)$$

$$\log h = 2.412 - 0.0135\, w_f, \qquad \text{for } w_f \geq 45.3\ \% \qquad (3\text{-}2)$$

In this research, the specimens tested from the Tempe cell method were also used to determine the matric suction using the filter paper method. First, the weight and dimensions of specimen were measured in order to calculate the moisture content of the specimen. Then, double layers of the Whatman No. 42 filter paper with caps were placed on both top and bottom of the soil specimen. After that, the specimen was completely wrapped with the plastic wrap and then, was packed in a container. The specimen was allowed to equilibrate for 7 to 10 days. After equilibrium, the moisture contents of the filter papers were determined and the soil suction was estimated from the calibration curves. Then, the soil specimen was dried and the filter paper technique was then repeated to determine the corresponding matric suction at the dry condition. The same steps were repeated for other drying condition.

Finally, the data obtained from both Tempe cell method and filter paper technique were employed in the development of a complete SWCC profile of the soils. The steps of specimen preparation in the filter paper technique are presented in Figure 3.8.

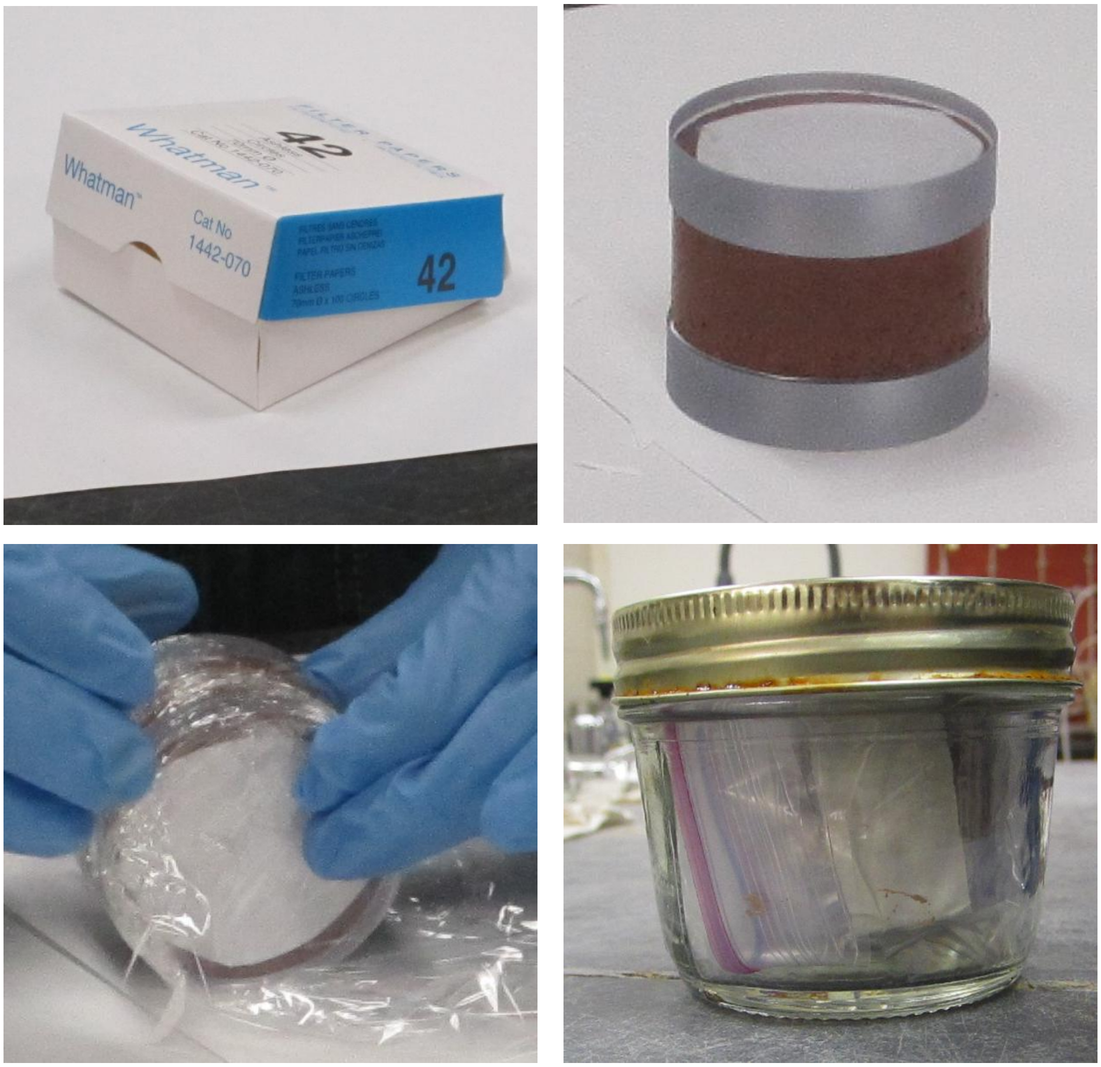

Figure 3.8 Specimen preparations in filter paper testing

3.3.3 Unconfined Compressive Strength (UCS) Test

In an unconfined compression test, a cylindrical core sample is loaded axially to failure, with no confining pressure. The peak value of the axial stress is taken as the unconfined compressive strength of the sample. In addition to axial stress, axial strain is also monitored during the test and the graph plotted between the stress and strain values is provided. In this study, the unconfined compressive strength (UCS) tests were conducted using triaxial test equipment. The testing setup consists of a loading frame including a circular base with a central pedestal; a triaxial cell fitted to the top of the base plate with the help of 3 wing nuts; and external LVDT transducers. In the testing process, a cylindrical specimen of 2.8 in. (71 mm) in diameter and 5.6 in. (142 mm) in height was sheared at a constant strain rate (1.27 mm/min) with the help of a loading ram. The triaxial setup used in this study is presented in Figure 3.9 below.

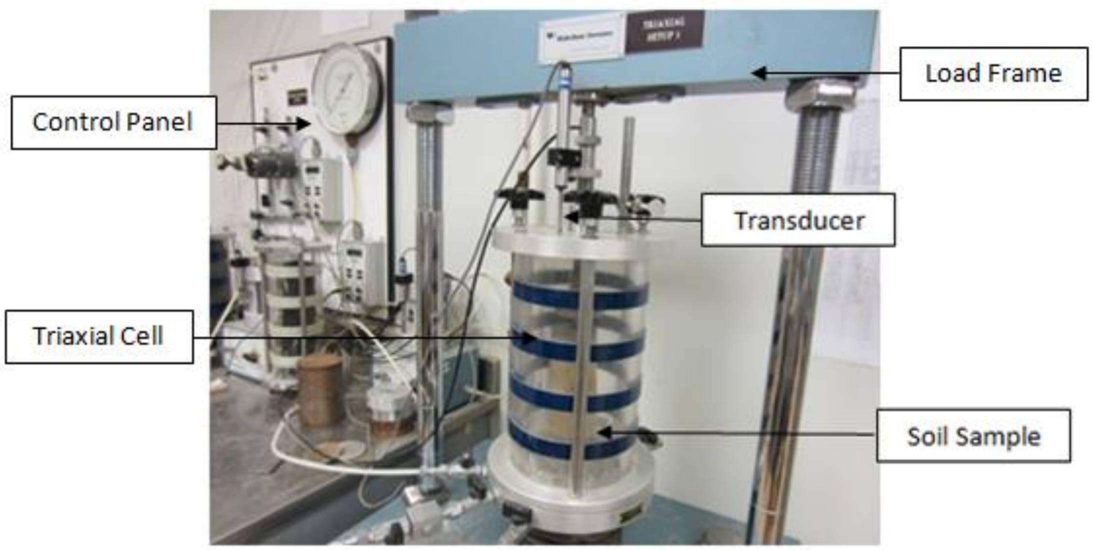

Figure 3.9 Triaxial equipment

3.3.4 Conventional Resilient Modulus Test

In this study, the resilient modulus tests were conducted using the cyclic triaxial test equipment. The equipment is designed to simulate the traffic wheel loading on the in situ soils by applying a sequence of repeated or cyclic loading on the soil specimens. The test was performed in accordance with AASHTO

Designation T 307-99, the standard method of test for determining the resilient modulus of soils and aggregate materials. The stress levels used for testing specimens for resilient modulus are based upon the location of the specimen within the pavement structure as standardized by AASHTO. The samples located within the subgrade are subjected to lower stress level as compared to the specimens that are from the base and subbase. Water was used as the medium to apply the desired confining pressure to the specimen.

As per the resilient modulus testing standard, the testing sequence for subgrade soils shown in Table 3.6 is employed in the procedure. The confining pressure typically represents overburden pressure of the specimen location in subgrade. The axial deviatoric stress is composed of two components; cyclic stress, which is the applied deviatoric stress, and a contact stress, typically represents a seating load on the soil specimen. The contact stress is typically equivalent to 10% of overall maximum axial stress.

In this study, the load pulse selected to be used is in accordance with the AASHTO T 307-99 method. The loading pulse for the subgrade soils has a haversine shape form with a loading duration of 0.1 seconds followed by a rest period of 0.9 seconds as shown in Figure 3.10.

Table 3.6 Resilient Modulus Testing Sequence for Subgrade Materials

No.	Confining Pressure		Max. Axial Stress		Cyclic Stress		Contact Stress		No. of Load Cycles
	kPa	*psi*	*kPa*	*psi*	*kPa*	*psi*	*kPa*	*psi*	
0	41.4	6	27.6	4	24.8	3.6	2.8	0.4	500-1000
1	41.4	6	13.8	2	12.4	1.8	1.4	0.2	100
2	41.4	6	27.6	4	24.8	3.6	2.8	0.4	100
3	41.4	6	41.4	6	37.3	5.4	4.1	0.6	100
4	41.4	6	55.2	8	49.7	7.2	5.5	0.8	100
5	41.4	6	68.9	10	62.0	9.0	6.9	1.0	100
6	27.6	4	13.8	2	12.4	1.8	1.4	0.2	100
7	27.6	4	27.6	4	24.8	3.6	2.8	0.4	100
8	27.6	4	41.4	6	37.3	5.4	4.1	0.6	100
9	27.6	4	55.2	8	49.7	7.2	5.5	0.8	100
10	27.6	4	68.9	10	62.0	9.0	6.9	1.0	100
11	13.8	2	13.8	2	12.4	1.8	1.4	0.2	100
12	13.8	2	27.6	4	24.8	3.6	2.8	0.4	100
13	13.8	2	41.4	6	37.3	5.4	4.1	0.6	100
14	13.8	2	55.2	8	49.7	7.2	5.5	0.8	100
15	13.8	2	68.9	10	62.0	9.0	6.9	1.0	100

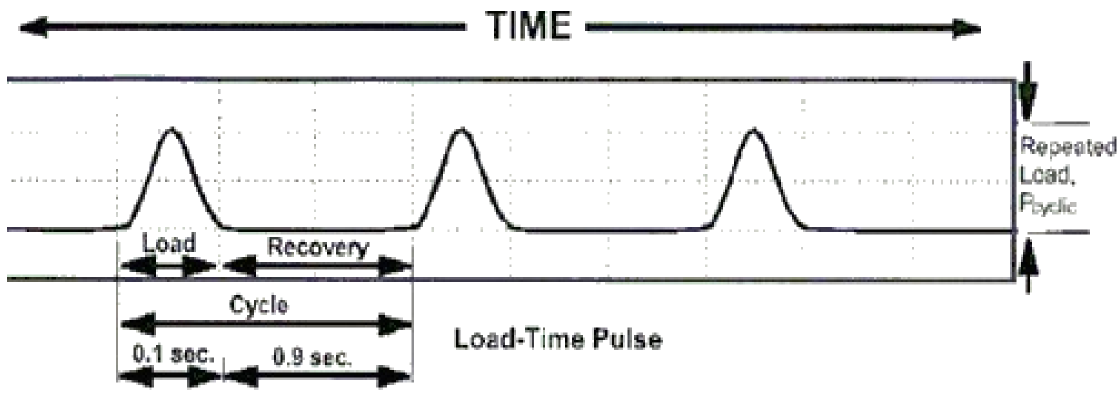

Figure 3.10 The haversine-shaped load pulse applied to the test specimen

As presented in Table 3.6, the test process requires both conditioning followed by actual testing under a magnitude of confining pressure and deviatoric stresses. At each confining pressure and deviatoric stress, the resilient modulus value was determined by averaging the resilient deformation of the last five deviatoric cycles. Hence, from a single test on a compacted soil specimen, several resilient moduli values at different combinations of confining ad deviatoric stresses were determined.

3.4 Equipment Employed for the Resilient Modulus Testing

The UTM-5P dynamic triaxial system used in the resilient modulus testing (RMT) is a closed loop, servo control, materials testing machine. The UTM-5P system is designed to facilitate a wide range of triaxial testing. The major components of the UTM-5P system consist of loading frame, controller and data acquisition system. The following sections provide the descriptions on those major components of the UTM-5P system.

3.4.1 Loading Frame

The loading frame of the resilient modulus testing equipment is shown in Figure 3.11 below. The components of the frame consist of a heavy flat base plate supported on four leveling screws; two height adjustable rods supporting the crosshead beam; and a pneumatic actuator mounted at the center of the crosshead. The frame is of heavy construction to limit deflection and vibrations that could influence the accuracy of measurements during dynamic repeated loading tests. The loading forces are applied through the shaft of the pneumatic actuator. The sensitive, low friction displacement transducers attached to the crosshead enable measurement of the permanent and small resilient deflections of the soil specimen during testing.

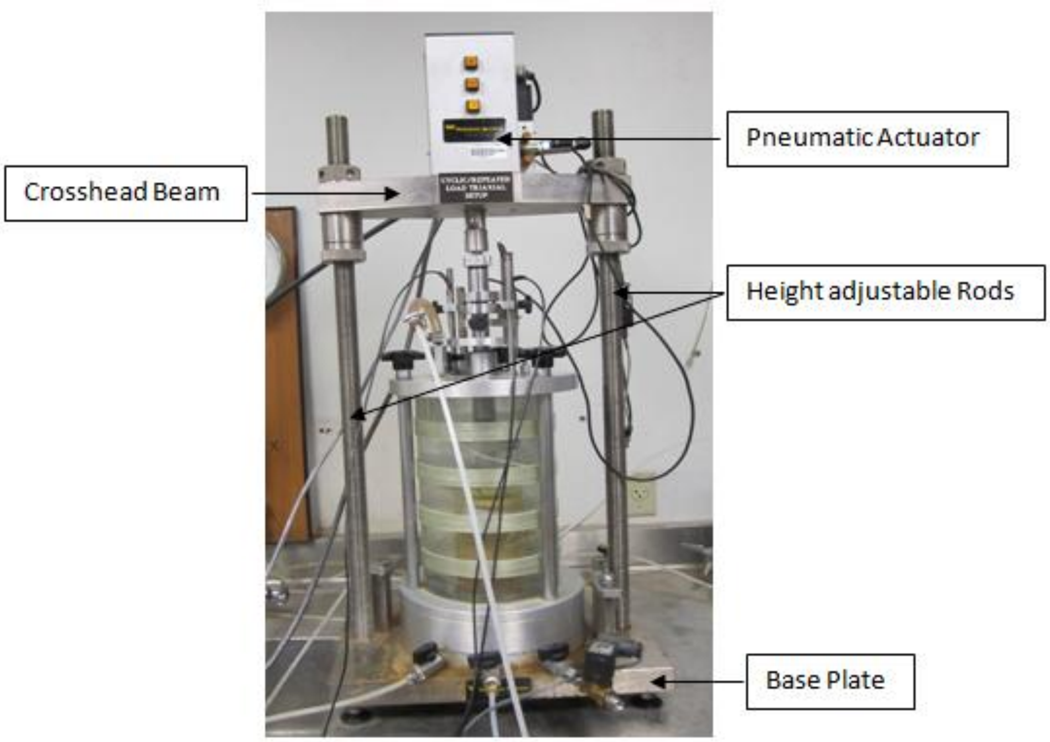

Figure 3.11 The loading frame and triaxial cell

3.4.2 The Pneumatic Loading System

The UTM pneumatic system is an air compressor controller unit used to control both load and pressures applied on soil specimens. For asphalt tests, only the vertical force pneumatics is required, while the unbound tests on soils require both confining and axial deviatoric pressure pneumatics. The system requires a filtered clear air supply at a minimum supply pressure of 800 kPa. Lower supply pressures will prevent the system from achieving the maximum specified stresses or forces, as selected by the operator. Figure 3.12 shows the Pneumatic system at the UTA geotechnical lab facility.

Figure 3.12 The pneumatic system

3.4.3 Triaxial Cell

The triaxial pressure cell used is suitable for testing specimens having dimensions of up to 200 mm height by 100 mm diameter. This unit is rated to a maximum confining pressure of 1700 kPa. To provide maximum visibility, the cell chambers are made of Lucite-type material. The cell is designed to contain pressurized liquid only and so the use of any compressible gas as a confining medium is dangerous.

3.4.4 Control and Data Acquisition System

The UTM Control and Data Acquisition System (CDAS) is a compact, self-contained unit that provides all critical control, timing and data acquisition functions for the testing frame and transducers. The CDAS consists of an Acquisition module (analog input/output) and a Feedback Control module (analog input/output).

Acquisition module has eight normalized transducer input channels that are digitized by high speed 12 bit Analog to Digital (A/D) converters for data analysis and presentation. In addition two 14 bit Digital to Analog (D/A) converters are available to provide computer control of the voltage to pressure converters. The air pressure is controllable over the range 0 – 700 kPa. There are two output channels provided for applying confining pressures. The SOL1 is used as the trigger input to the feedback control module that creates and controls the waveform. The SOL2 output is used for the digital control signal from computer to control the confining pressure solenoid for triaxial tests.

The Feedback Control module has three normalized input channel controls. These channels are dedicated to the actuator position, actuator force and general purpose input (Aux) for on-specimen transducers. This module has a dedicated communication interface of its own that provides for an uninterrupted, simultaneous communication with the PC enabling increased speed of operation and flexibility. The figure 3.13 below shows the control and data acquisition system.

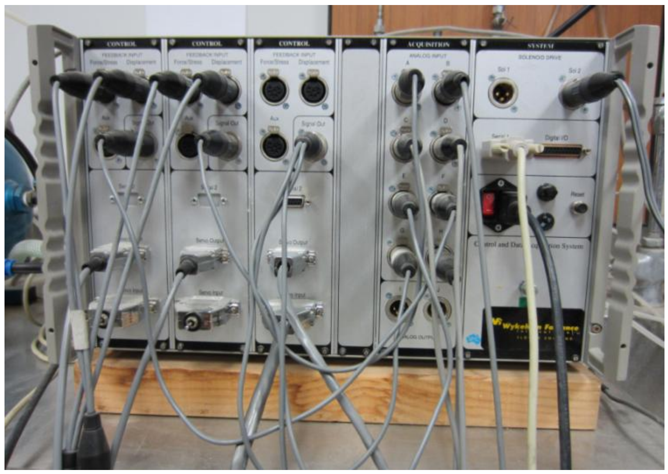

Figure 3.13 The control and data acquisition system

3.4.5 Linear Variable Displacement Transducers (LVDTs)

Based on the AASHTO testing procedure T 307-99, high resolution LVDTs are needed to measure the soil displacements. Two external LVDTs are used to record the vertical displacements. This external displacement transducer is easy to install and provides a simplified procedure to reset the initial zero reading. The LVDTs are placed on the top cover of the cell and fitted to the load shaft. The maximum scale stroke for these two LVDTs is +5 mm, with a resolution of 0.001 mm accuracy. The output from each LVDT is monitored independently and compared to the output of the other LVDTs. Figure 3.14 shows the external transducer assembly employed in this project.

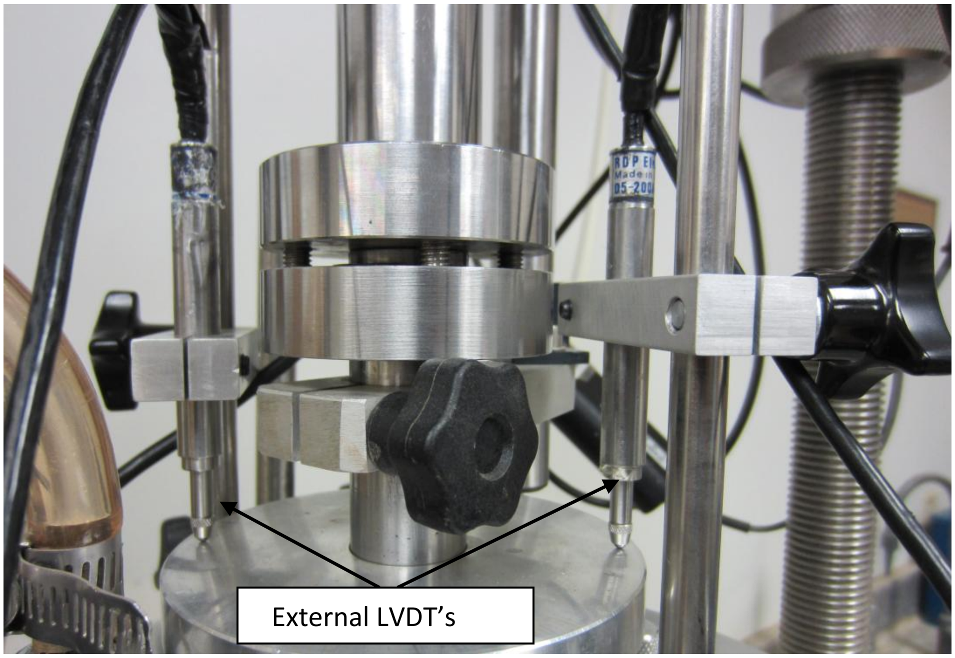

Figure 3.14 External LVDTs assembly

3.4.6 Software

The UTM software is used for equipment control and data acquisition operations. In this software, there are programs available for several test procedures, which include unconfined compressive strength test, resilient modulus test, unconsolidated undrained test, consolidated undrained test, consolidated drained test and a provision for user defined programs. The user program is a program that is provided for operators to create their own testing methods and protocols. In this research, the AASHTO T 307-99 test protocol for the determination of resilient modulus of aggregate base materials was used for performing these tests.

3.5 Summary

In this chapter, brief descriptions of basic soil tests including the summary of basic properties of selected subgrade materials are provided. In addition, the procedures of advanced soil testing used in this research are also described. The advanced soil tests considered in this research consist of the determination of SWCCs of the subgrade soils, unconfined compressive strength test, and conventional resilient modulus test. Moreover, the description on the equipment employed for the resilient modulus testing is also presented. In the next chapter, the results obtained from the tests mentioned previously, that were conducted on the three subgrade materials selected for this study, will be presented and analyzed.

CHAPTER 4

ANALYSIS OF STRENGTH AND RESILIENT MODULUS TEST RESULTS

4.1 Introduction

In this chapter, the behavior of the three subgrade materials under different engineering tests including unconfined compressive strength test and conventional resilient modulus test are discussed. Specimens were compacted and tested at five different moisture content and dry density conditions. Also, soil water characteristic curves were determined by testing same soils at three different moisture content and dry density conditions including one at dry of optimum, one at optimum and one at wet of optimum conditions.

This chapter also provides the regression analysis attempted on the resilient modulus test results. Modified Universal model and Cary and Zapata's model are used to analyze the measured resilient modulus results. Additionally, a step wise procedure used in MEPDG program is used to obtain the SWCC of the present materials. Fredlund and Xing equation was used in this procedure. A comparison of predicted SWCC with the measured results is made to evaluate the capabilities of SWCC predictions by the MEPDG model.

4.2 Unconfined Compressive Strength

Unconfined Compressive Strength (UCS) tests were conducted on the subgrade specimens at five moisture content-dry density points, as clearly explained in Table 3.5 in the previous chapter. As mentioned before, the specimens for UCS test were compacted in a 2.8 x 5.6 in. (diameter x height) mold and were kept in humidity room for at least one day to perform the uniform distribution of moisture in the specimens. After that, the specimens were tested under unconfined conditions. Tables 4.1, 4.2, and 4.3 summarize the unconfined compressive strength test results of the Minnesota, Mississippi, and Louisiana subgrade materials respectively. Also, Figures 4.1, 4.2, and 4.3 provide the curves that clearly illustrate the UCS test results of the subgrade materials at the selected five moisture content-dry density points.

Table 4.1 UCS Test Results of Minnesota Subgrade Soil

Points of Moisture Content	**Minnesota (CH)**			
	Target Moisture Content (%)	**Nominal Moisture Content (%)**	**Dry Density (pcf)**	**UCS (psi)**
0.8 OMC	21.0	21.7	92.5	38
0.9 OMC	23.5	24.5	95.0	36
OMC	26.0	26.1	96.0	34
1.1 OMC	28.5	29.5	95.0	29
1.2 OMC	31.0	31.2	92.0	18

Table 4.2 UCS Test Results of Mississippi Subgrade Soil

Points of Moisture Content	**Mississippi (ML)**			
	Target Moisture Content (%)	**Nominal Moisture Content (%)**	**Dry Density (pcf)**	**UCS (psi)**
0.8 OMC	7.4	7.7	123.4	19
0.9 OMC	8.4	8.5	124.3	17
OMC	9.4	9.3	124.6	18.5
1.1 OMC	10.4	10.3	124.3	17
1.2 OMC	11.4	11.6	123.4	15

Table 4.3 UCS Test Results of Louisiana Subgrade Soil

Points of Moisture Content	**Louisiana (SC)**			
	Target Moisture Content (%)	**Nominal Moisture Content (%)**	**Dry Density (pcf)**	**UCS (psi)**
0.8 OMC	9.1	9.3	118.7	49
0.9 OMC	10.3	10.4	120.7	41
OMC	11.4	11.4	121.4	32
1.1 OMC	12.5	12.3	120.7	23
1.2 OMC	13.7	13.4	118.7	11

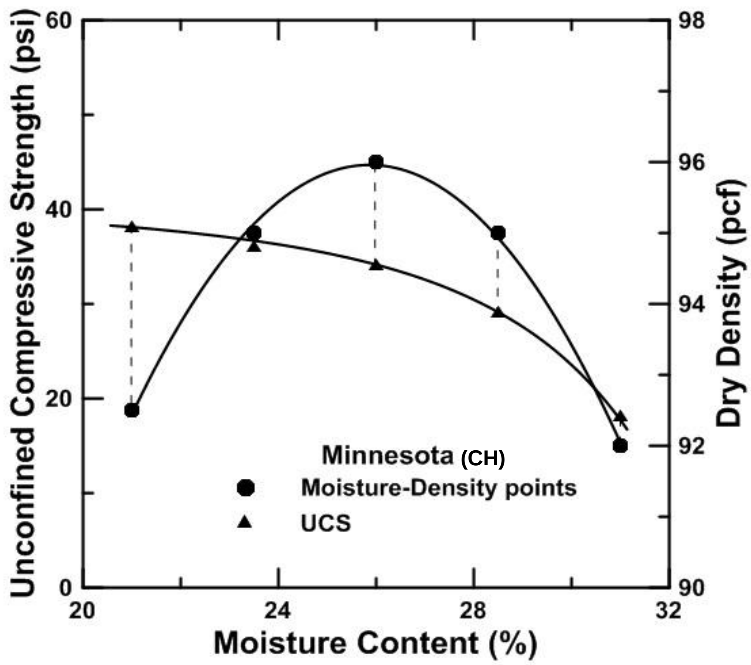

Figure 4.1 Unconfined compressive strength results of Minnesota specimens

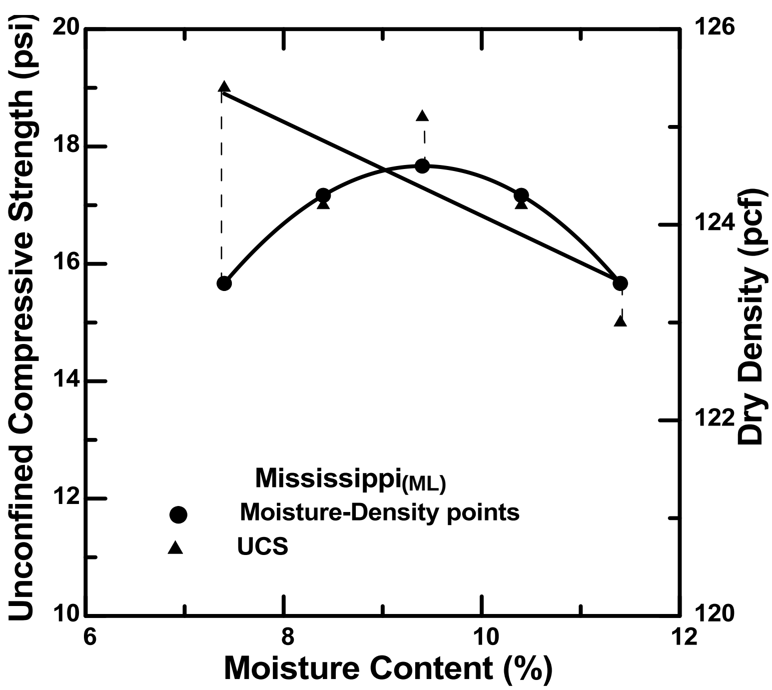

Figure 4.2 Unconfined compressive strength results of Mississippi specimens

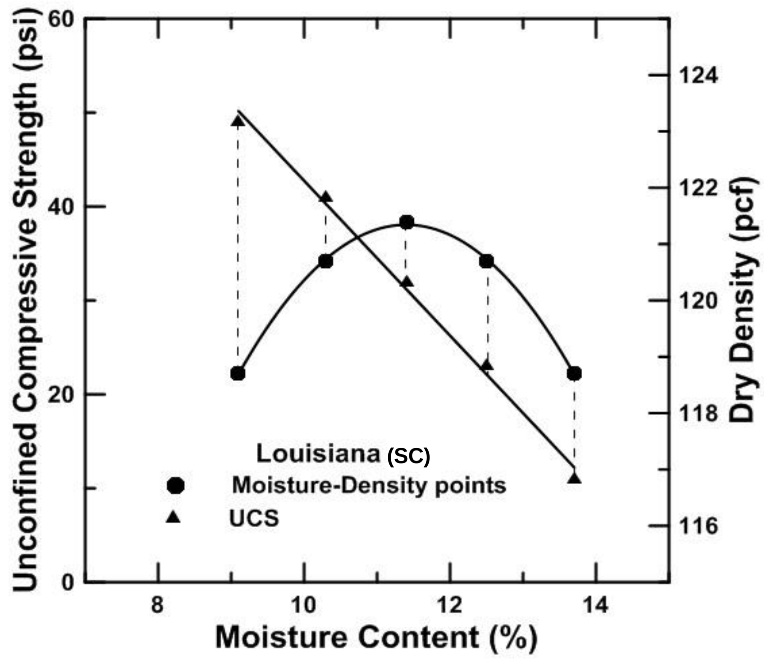

Figure 4.3 Unconfined compressive strength results of Louisiana specimens

From the tables and figures shown above, it can be noticed that the highest unconfined compressive strengths (UCS) of the three subgrade materials are observed at the sample test condition of 0.8 OMC moisture content (dry of optimum). The highest unconfined compressive strengths of Minnesota, Mississippi, and Louisiana subgrade soils observed from the UCS tests are 38 psi, 19 psi, and 49 psi, respectively.

<u>4.3 Soil Water Characteristic Curve (SWCC) Studies</u>

As stated in the previous chapter, in this study, the SWCCs for Minnesota, Mississippi, and Louisiana subgrade soils were measured using both Tempe cell method and filter paper technique. Soil samples of 2.5 in diameter and 1 in thickness were compacted at three different moisture content and dry density conditions, one on the wet side of the optimum (1.2 OMC), one on the optimum moisture content (OMC), and one on the dry side of the optimum moisture content (0.8 OMC) conditions. Before testing, the prepared samples were saturated for 24 hours. In the SWCC test, the ceramic disk which has an air entry value of 500 kPa was used and the air pressures of 10, 50, 100, 200, 300, and 450 kPa were applied for measuring the SWCCs. At equilibrium condition of these pressures, the related moisture contents were recorded and these results are used to create SWCC profiles.

Figures 4.4, 4.5, and 4.6 show the SWCCs of Minnesota, Mississippi, and Louisiana subgrade soils respectively. The SWCCs are plotted with matric suction on the x-axis (in log scale) and gravimetric water content on the y-axis.

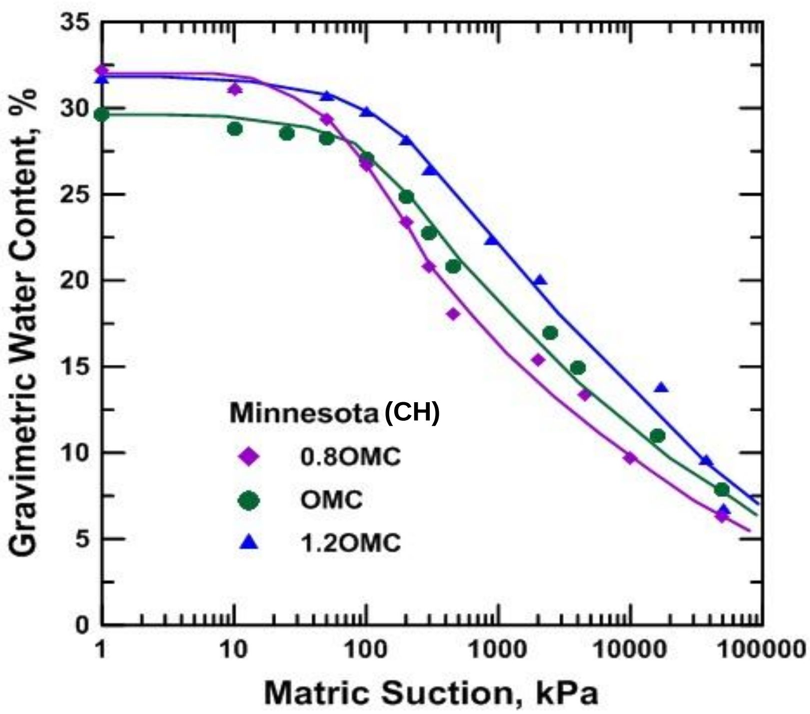

Figure 4.4 SWCCs of Minnesota specimens

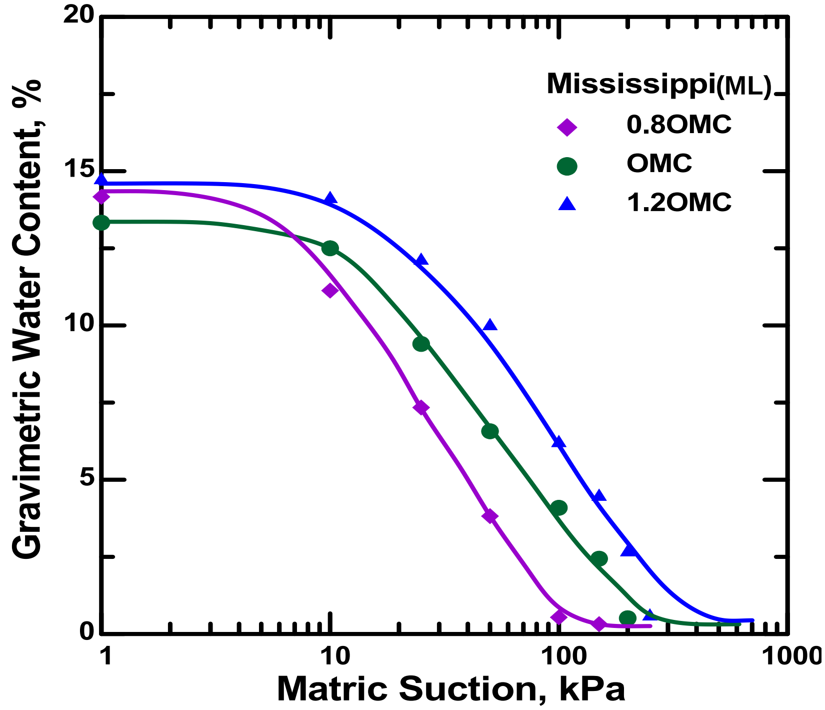

Figure 4.5 SWCCs of Mississippi specimens

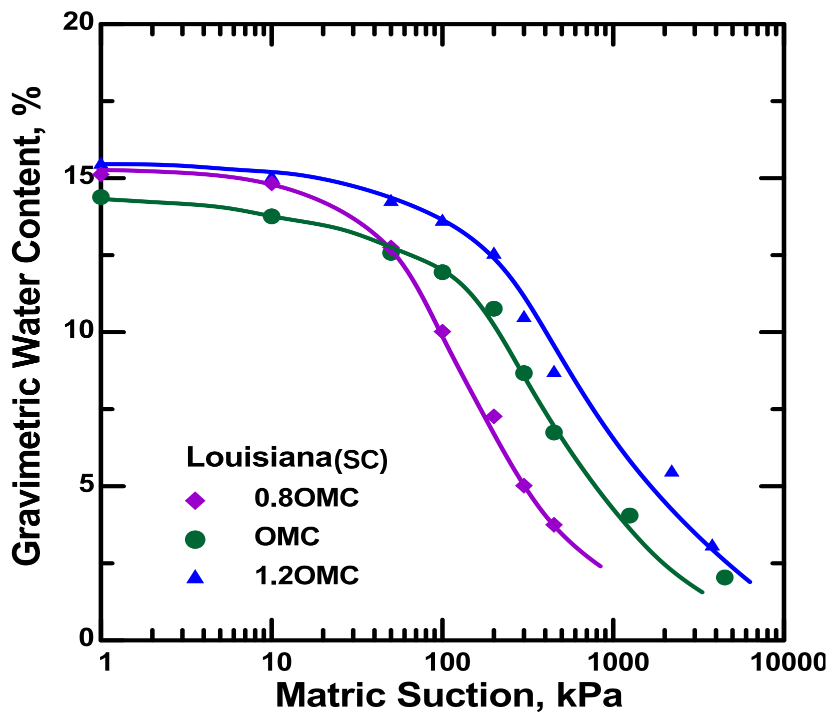

Figure 4.6 SWCCs of Louisiana specimens

From the SWCCs provided above, the air entry value, which is defined as matric suction when air starts to enter the largest pores in the soil sample at certain gravimetric water content-matric suction curve can be determined as shown in Table 4.4 below.

Table 4.4 Air Entry Values obtained from the SWCCs

Soil	**Air entry value (kPa)**		
	0.8 OMC	**OMC**	**1.2 OMC**
Minnesota (CH)	30	70	90
Mississippi (ML)	7	11	18
Louisiana (SC)	31	80	110

From the table, it can be observed that the air entry values of the samples compacted at 1.2 OMC are highest when compared to the samples compacted at OMC and 0.8 OMC conditions. Besides, the Louisiana soil samples show the highest air entry values when compared with the samples of Minnesota and Mississippi subgrade soils.

4.4 Conventional Resilient Modulus Testing Results

As stated previously, all three subgrade soils were tested at different five moisture content and dry density points in accordance with the AASHTO standard test procedure, T-307-99. The combinations of various confining and deviatoric stresses applied in the test sequences have been presented in Table 3.6 presented in Chapter 3. In each test sequence, the specimen was subjected to three different confining pressures (2, 4, and 6 psi) and for each confining pressure, five different deviatoric stresses (2, 4, 6, 8, and 10 psi) were applied to the soil specimen. During the test, average vertical deformation of the specimen was monitored and recorded by using two external variable displacement transducers (LVDTs) placed on the top of the triaxial cell. At each confining pressure and deviatoric stress, the resilient modulus value was determined by averaging the resilient deformation of the last five deviatoric cycles.

In this study, the soil specimens were compacted at five different moisture content-dry density conditions as per Table 3.5 and the soil suction conditions at the compaction stage were determined based on the SWCCs information and the compaction moisture contents. Table 4.5 presents the soil suction conditions of the specimens prepared for this M_R testing.

Table 4.5 TheSoil Suction Conditions of the Prepared Specimens

Soil	**Initial Compaction-induced Soil Suction**									
	0.8 OMC		**0.9 OMC**		**OMC**		**1.1 OMC**		**1.2 OMC**	
	kPa	**psi**	**kPa**	**psi**	**kPa**	**psi**	**kPa**	**psi**	**kPa**	**psi**
Minnesota (CH)	310	45	290	42	180	26	105	15	50	7.3
Mississippi (ML)	26	3.8	26	3.8	25	3.6	24	3.5	23	3.3
Louisiana (SC)	120	17.4	118	17.1	115	16.7	110	16.0	90	13.0

The resilient moduli test results of the soils specimens were determined based on the external displacement measurements and these results are presented in Figures 4.7 through 4.21. In these figures, the measured resilient moduli are plotted as a function of different deviatoric and confining pressures.

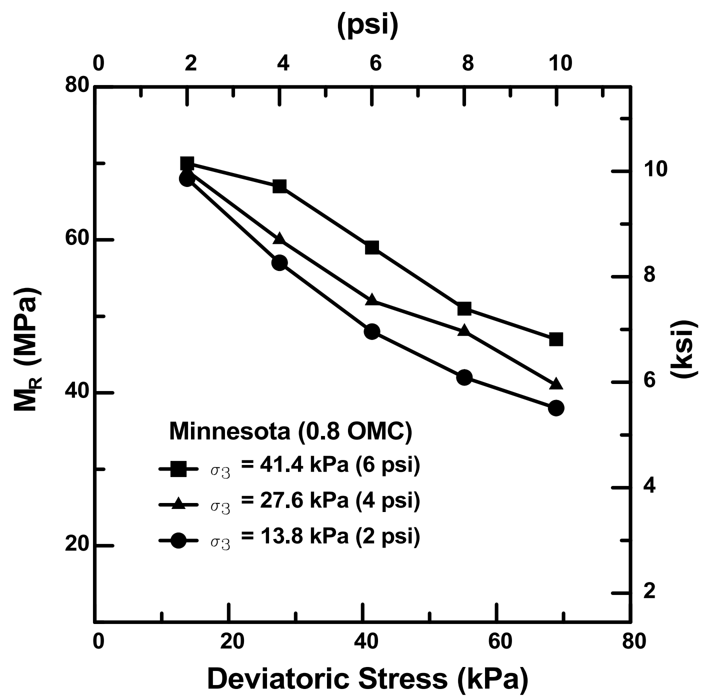

Figure 4.7 Variation of resilient modulus (M_R) of Minnesota specimens (CH) compacted at 0.8OMC

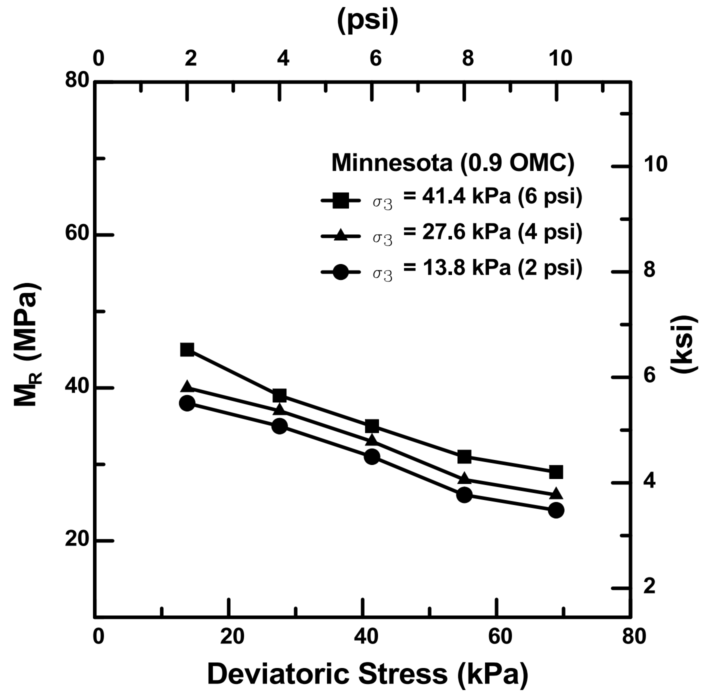

Figure 4.8 Variation of resilient modulus (M_R) of Minnesota specimens (CH) compacted at 0.9OMC

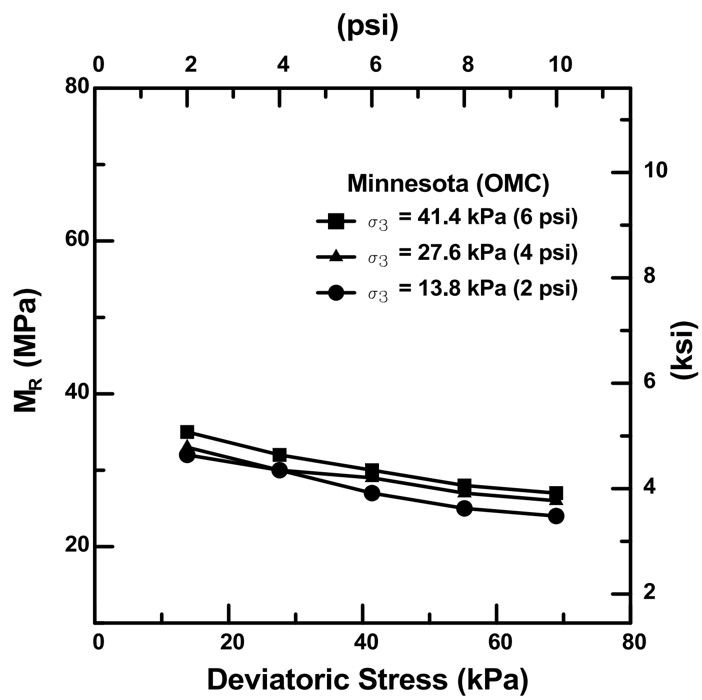

Figure 4.9 Variation of resilient modulus (M_R) of Minnesota specimens (CH) compacted at OMC

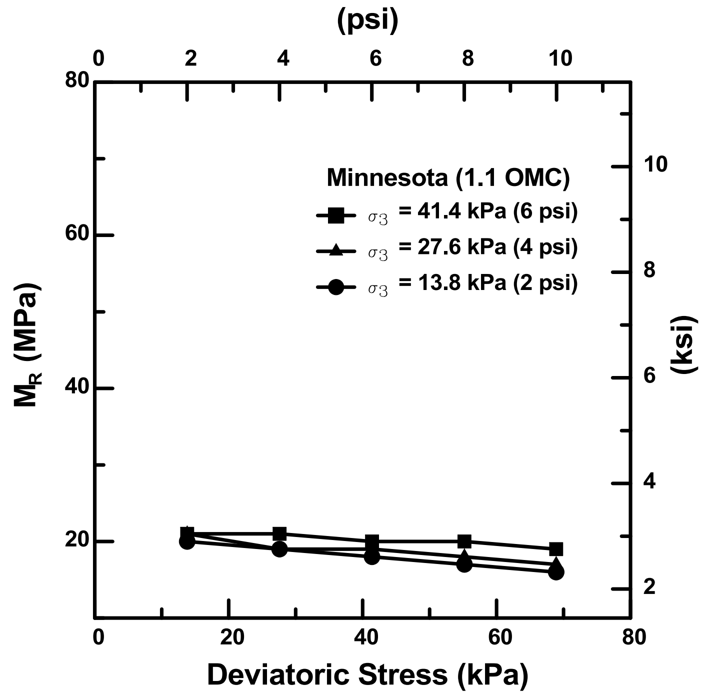

Figure 4.10 Variation of resilient modulus (M_R) of Minnesota specimens (CH) compacted at 1.1OMC

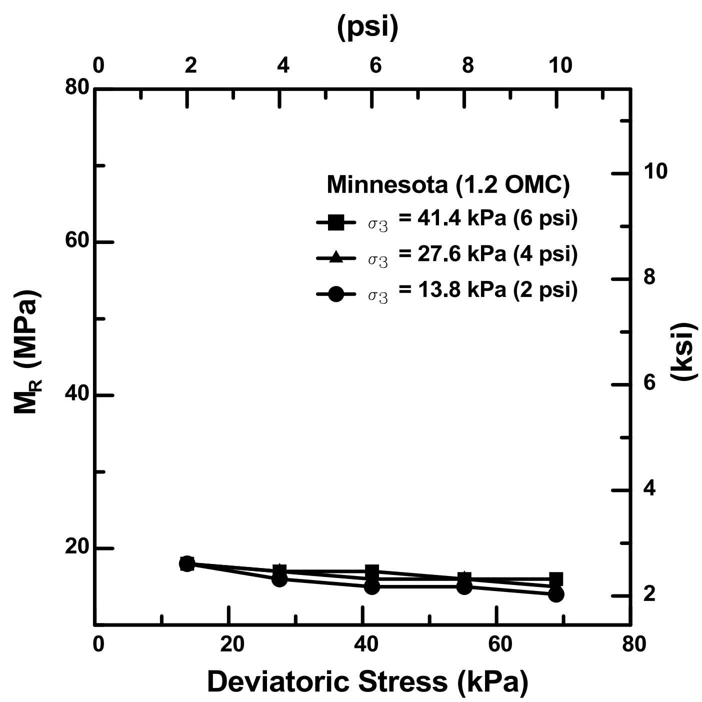

Figure 4.11 Variation of resilient modulus (M_R) of Minnesota specimens (CH) compacted at 1.2OMC

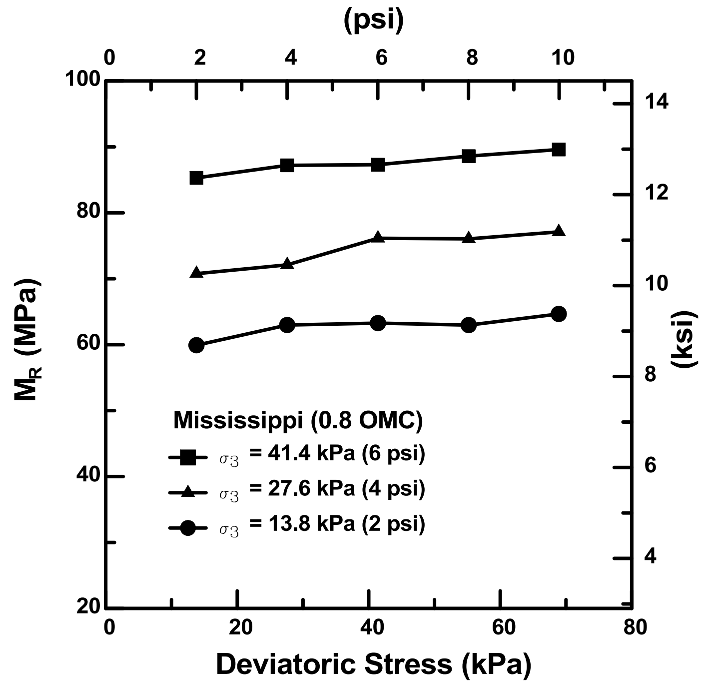

Figure 4.12 Variation of resilient modulus (M_R) of Mississippi specimens (ML) compacted at 0.8OMC

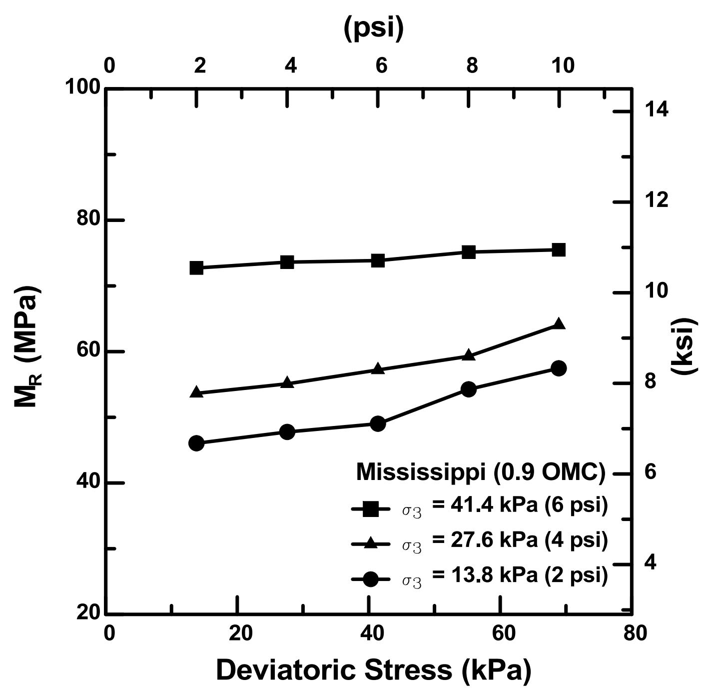

Figure 4.13 Variation of resilient modulus (M_R) of Mississippi specimens (ML) compacted at 0.9OMC

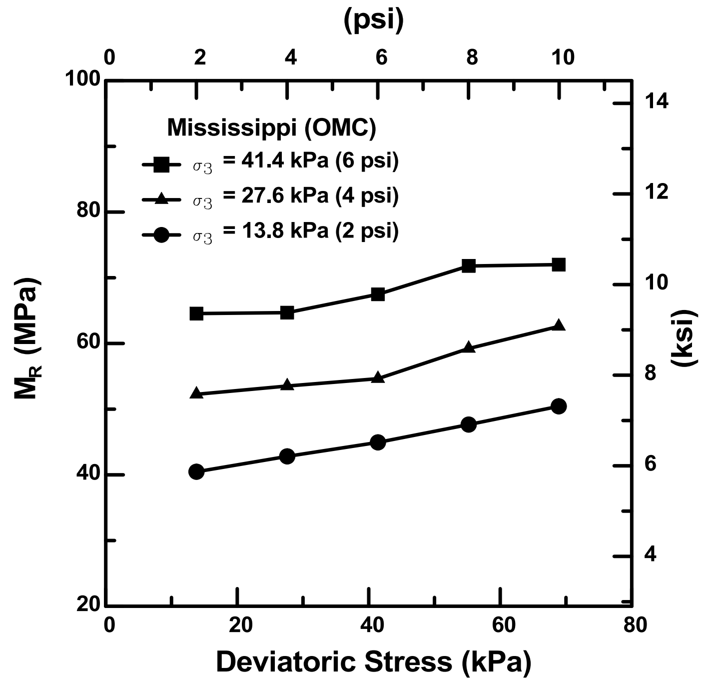

Figure 4.14 Variation of resilient modulus (M_R) of Mississippi specimens (ML) compacted at OMC

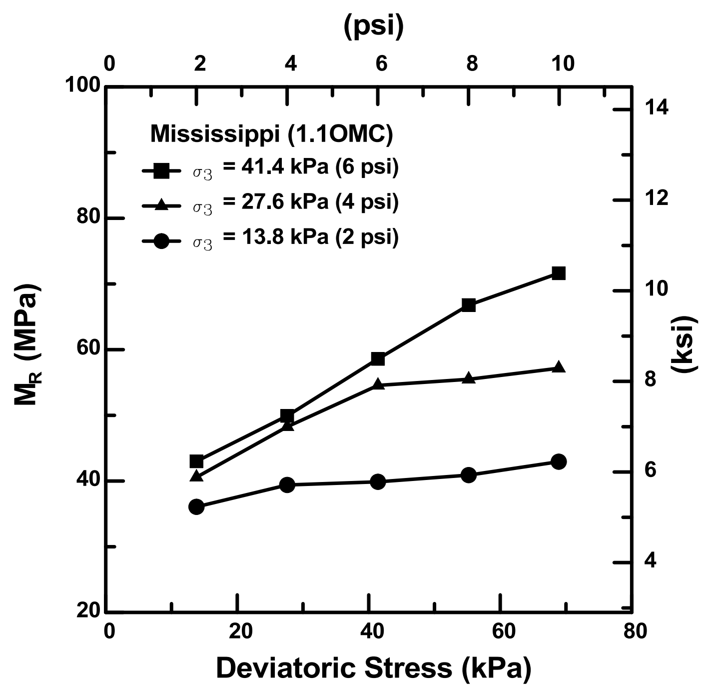

Figure 4.15 Variation of resilient modulus (M_R) of Mississippi specimens (ML) compacted at 1.1OMC

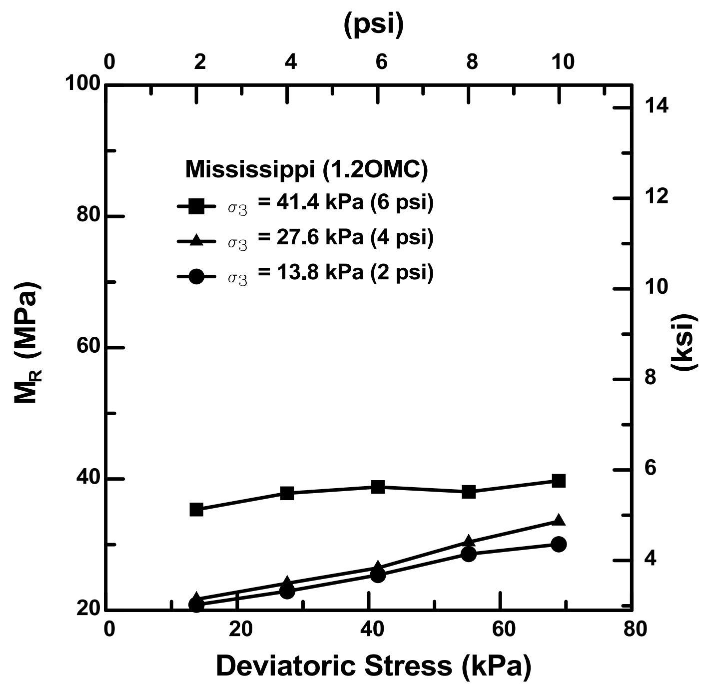

Figure 4.16 Variation of resilient modulus (M_R) of Mississippi specimens (ML) compacted at 1.2OMC

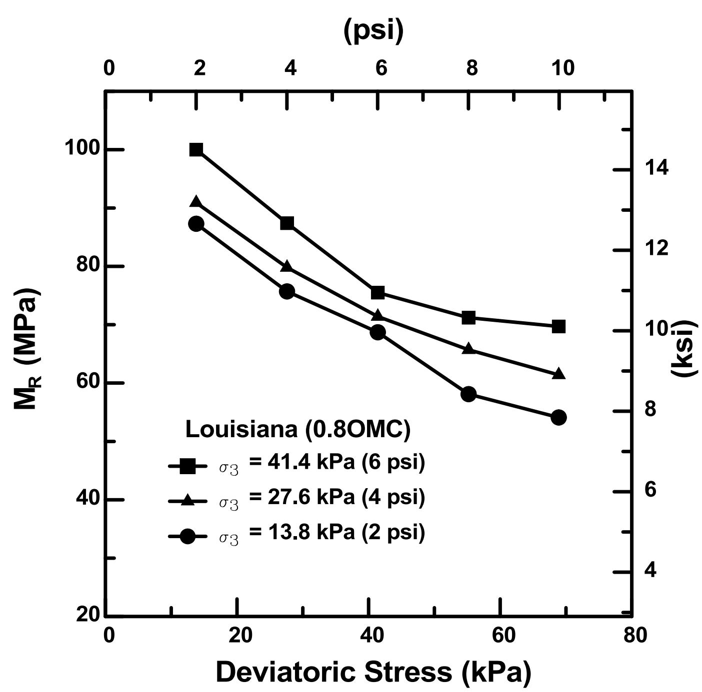

Figure 4.17 Variation of resilient modulus (M_R) of Louisiana specimens (SC) compacted at 0.8OMC

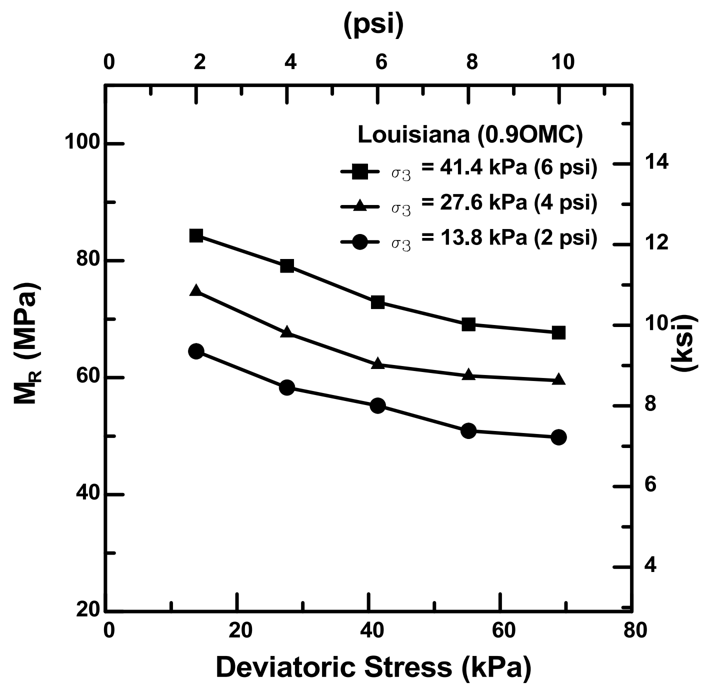

Figure 4.18 Variation of resilient modulus (M_R) of Louisiana specimens (SC) compacted at 0.9OMC

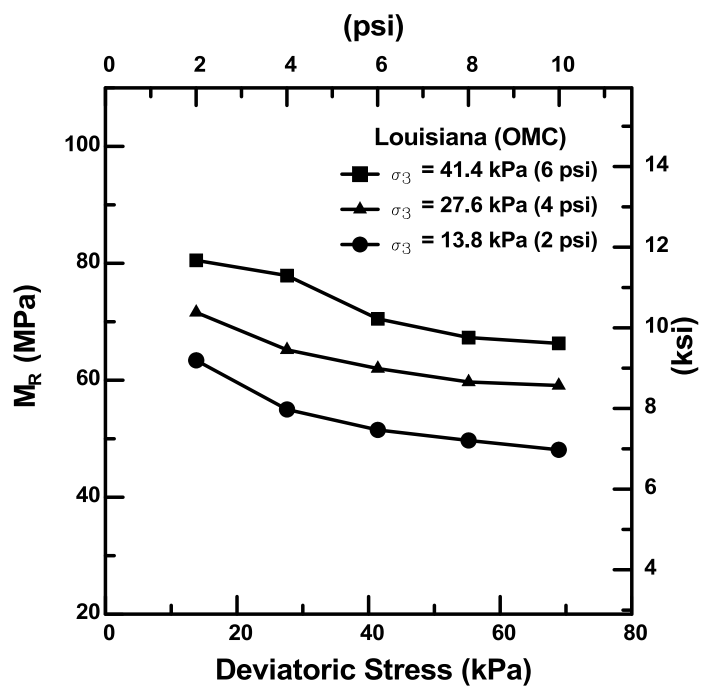

Figure 4.19 Variation of resilient modulus (M_R) of Louisiana specimens (SC) compacted at OMC

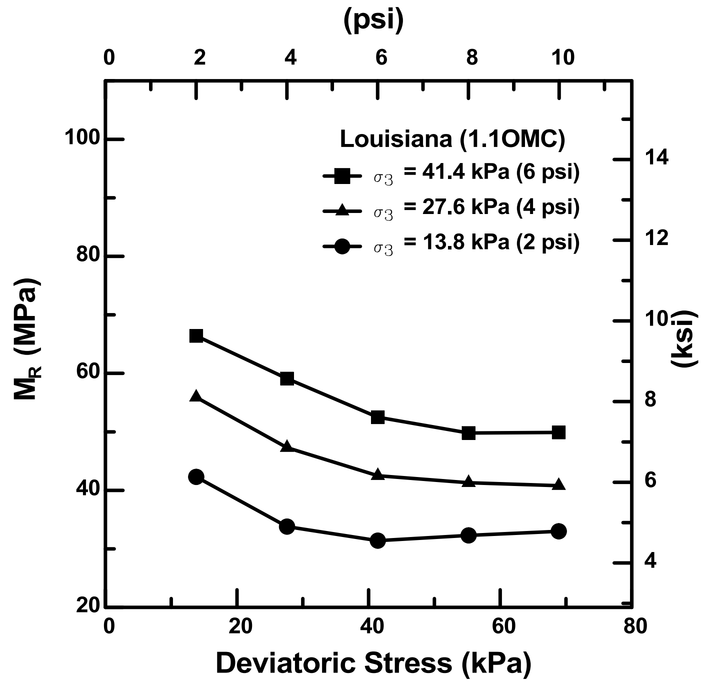

Figure 4.20 Variation of resilient modulus (M_R) of Louisiana specimens (SC) compacted at 1.1OMC

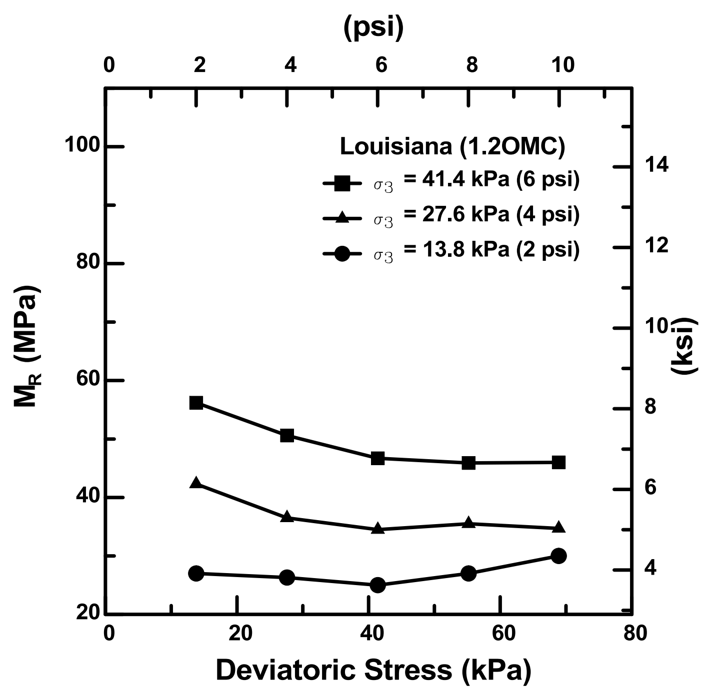

Figure 4.21 Variation of resilient modulus (M_R) of Louisiana specimens (SC) compacted at 1.2OMC

From the figures, it can be observed that both confining and deviatoric stresses have a significant influence on resilient modulus values of the compacted subgrade soils. With the increasing of deviatoric stresses, the resilient modulus of Minnesota and Louisiana subgrade soils decreased due to stress softening of the specimens. However, Mississippi soils showed an increase in the resilient modulus due to stress hardening of the specimens when the deviatoric stresses were increased.

A slight increase in resilient moduli of the subgrade soils was also observed when higher confining pressures were applied to soil specimens. This is expected for the low to high plasticity soils. Also, the effects of moisture contents on the resilient moduli of the subgrade soils are also observed. In all the above cases, samples compacted at the moisture content of 0.8OMC showed higher resilient modulus values than any other samples while the sample compacted on the wet side of OMC, which are 1.1OMC and 1.2OMC, have lower resilient modulus values. This trend is expected as samples at higher moisture content condition tend to exhibit lower strength and consequently lesser moduli values.

4.5 Modeling Analysis

4.5.1 SWCC modeling and comparisons

In this section, the SWCCs predicted from MEPDG model are fully evaluated. The equation used for predicting the SWCCs is provided by Fredlund and Xing (1994). The guide for Mechanistic-Empirical Pavement Design (NCHRP

2004) provides the approach to characterize the SWCC parameters from soil properties. According to MEPDG (NCHRP 2004), three input levels are provided to determine the parameters of SWCC. In this study, the Level 2 and prediction parameters are used to predict the SWCCs of the selected subgrade soils. The predicted SWCCs are compared with the measured SWCC from Tempe cell and filter paper technique.

The procedure to predict SWCC of subgrade soils provided in MEPDG can be summarized in following steps:

The basic input parameter needed from laboratory testing for Level 2 analysis:

1. Optimum gravimetric water content (w_{opt}) and maximum dry unit weight (γ_{dmax})
2. Specific gravity of the solids (G_s)
3. Passing #200 sieve, effective grain size corresponding to 60 percent passing by weight (D_{60}) and Plasticity Index

Using these input variables, the SWCC model parameters such as a_f, b_f, c_f, and h_r are computed. The following steps show the procedure to obtain the SWCC parameters and then these parameters are used with the Fredlund and Xing model to predict the SWCC of the soil.

1. Calculate P200*PI
2. Estimation of S_{opt}, θ_{opt} and θ_{sat}: These parameters are calculated using γ_{dmax}, w_{opt} and G_s using the equations given below:

- $\theta_{opt} = \frac{w_{opt}\,\gamma_{d\,max}}{\gamma_{water}}$ (4-1)

- $S_{opt} = \frac{\theta_{opt}}{1-\frac{\gamma_{d\,max}}{\gamma_{water}\,G_s}}$ (4-2)

- $\theta_{sat} = \frac{\theta_{opt}}{S_{opt}}$ (4-3)

3. Determine the SWCC model parameters a_f, b_f, c_f, and h_r by using correlations with P_{200}, PI and D_{60}

3.1 If (P_{200}) (PI) > 0

$$a_f = \frac{0.00364\ (P_{200}\ PI)^{3.35} + 4\ (P_{200}\ PI) + 11}{6.895}\ , psi \quad (4\text{-}4)$$

$$\frac{b_f}{c_f} = -2.313\ (P_{200}\ PI)^{0.14} + 5 \quad (4\text{-}5)$$

$$c_f = 0.0514\ (P_{200}\ PI)^{0.465} + 0.5 \quad (4\text{-}6)$$

$$\frac{h_r}{a_f} = 32.44\ e^{0.0186\,(P_{200}\ PI)} \quad (4\text{-}7)$$

3.2 If (P_{200}) (PI) = 0

$$a_f = \frac{0.8627\,(D_{60})^{-0.751}}{6.895}\ , psi \quad (4\text{-}8)$$

$$b_f = 7.5 \quad (4\text{-}9)$$

$$c_f = 0.1772\ \ln(D_{60}) + 0.7734 \quad (4\text{-}10)$$

$$\frac{h_r}{a_f} = \frac{1}{D_{60} + 9.7\ e^{-4}} \quad (4\text{-}11)$$

4. The SWCC will then be established using the Fredlund and Xing equation:

$$\theta_w = C(h) \times \left[\frac{\theta_{sat}}{\left[ln\left[EXP\,(1) + \left(\frac{h}{a_f}\right)^{b_f} \right] \right]^{c_f}} \right] \quad (4\text{-}12)$$

$$C(h) = \left[1 - \frac{ln\left[1+\frac{h}{h_r}\right]}{ln\left[1+\frac{1.45 \times 10^5}{h_r}\right]}\right] \tag{4-13}$$

By following the above steps, the SWCCs of Minnesota, Mississippi, and Louisiana subgrade soils are determined. Figures 4.22, 4.23, and 4.24 present the comparisons between the measured SWCCs and the predicted SWCCs of Minnesota, Mississippi, and Louisiana subgrade soils respectively. The comparisons were made at optimum moisture content condition.

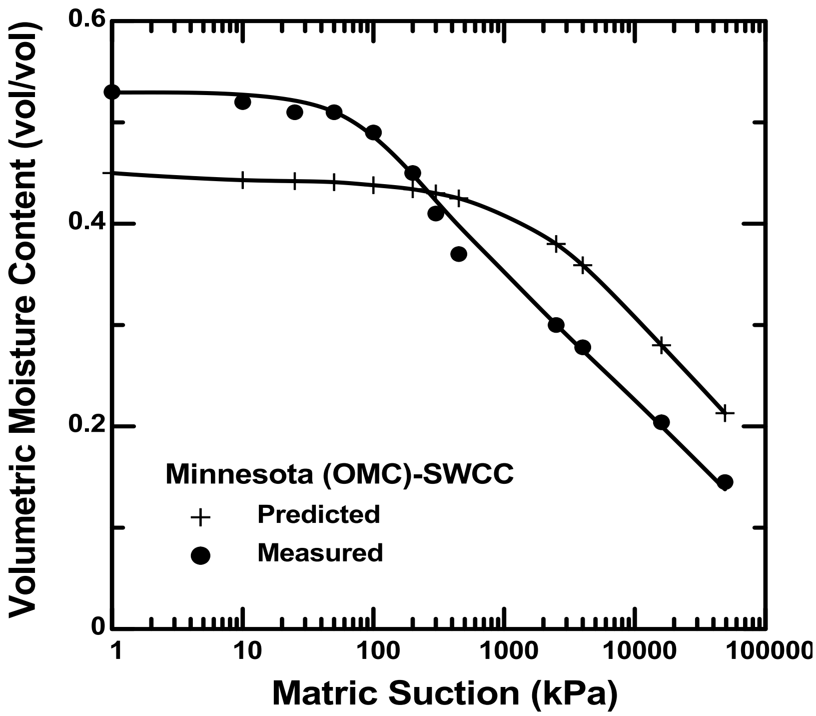

Figure 4.22 Predicted and measured SWCCs of Minnesota subgrade soil (CH)

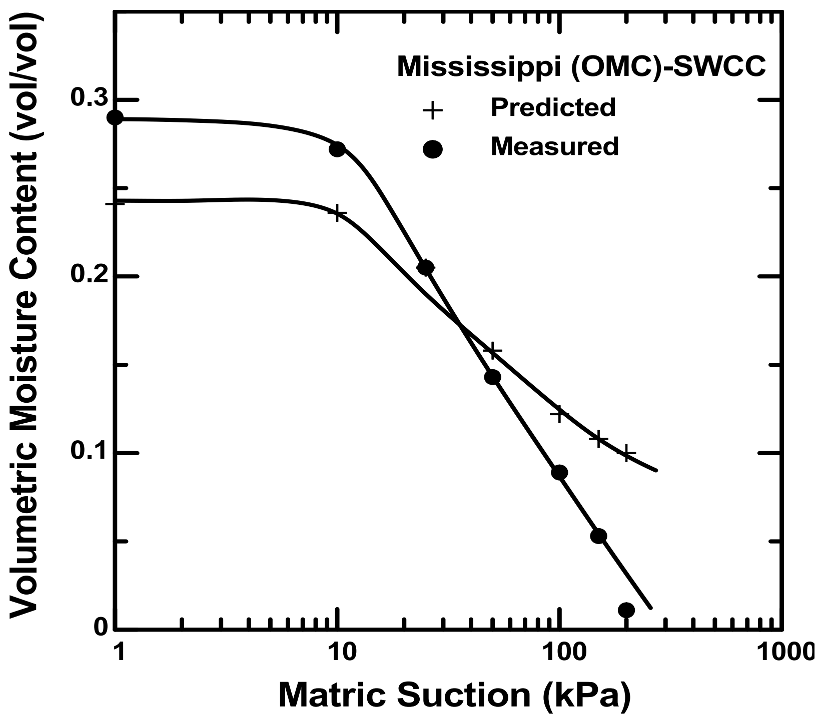

Figure 4.23 Predicted and measured SWCCs of Mississippi subgrade soil (ML)

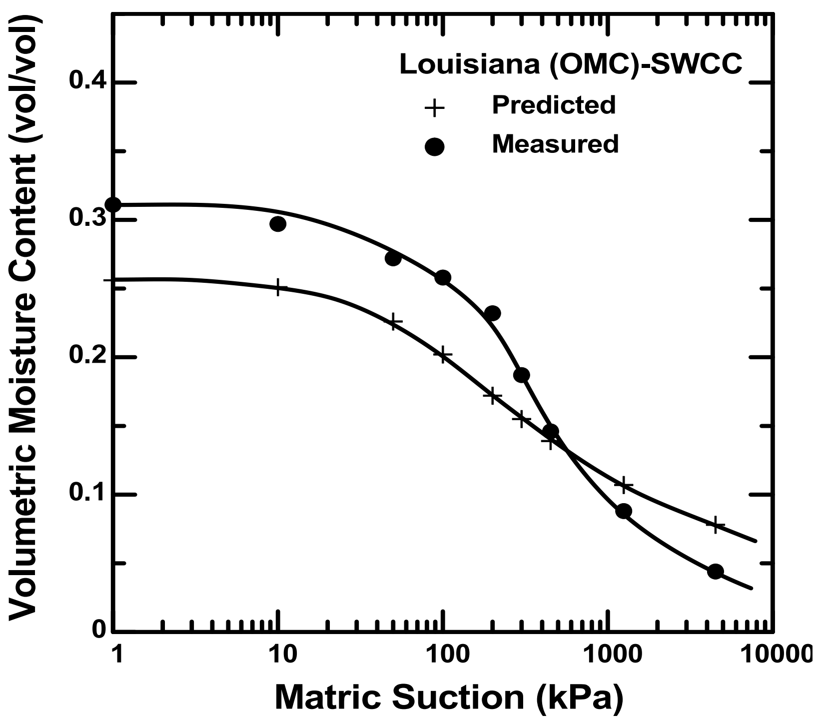

Figure 4.24 Predicted and measured SWCCs of Louisiana subgrade soil (SC)

From the figures, it can be noticed that the measured SWCCs of the subgrade soils show similar trend with the SWCCs predicted from Fredlund and Xing's model (1994). However, the curves are not quite well matched. At low suctions, the measured SWCCs showed higher volumetric moisture content whereas at the high suctions, the predicted SWCCs expressed higher volumetric moisture content. Overall, it is still preferable to use SWCCs from measurement as this curve has a paramount influence on the MEPDG design of pavements.

4.5.2 Resilient Modulus Models

4.5.2.1 Modified Universal Model

In this section, the data received from the M_R testing program were analyzed with the Modified Universal Model expressed in the following Equation (4-14). The regression coefficients k_1, k_2, and k_3 for each soil specimen are determined using a regression analysis on all data points.

$$M_R = k_1 \times p_a \times (\frac{\theta}{p_a} + 1)^{k_2} \times (\frac{\tau_{oct}}{p_a} + 1)^{k_3} \tag{4-14}$$

where, M_R = resilient modulus; p_a = atmospheric pressure; k_1, k_2, k_3 = regression constants; θ = bulk stress; τ_{oct} = octahedral shear stress.

A non-linear regression analysis procedure included in a statistical software package named ProStat was used to determine the regression constants (k_1, k_2, k_3) for each specimen. Tables 4.6, 4.7, and 4.8 present the values of the regression coefficients and the representative resilient modulus of Minnesota, Mississippi, and Louisiana samples, respectively. The representative

M_R was selected from the M_R value at a bulk stress of 12 psi and octahedral shear stress of 2.8 psi (at σ_3 = 13.8 kPa = 2 psi and σ_d = 41.4 kPa = 6 psi).

Table 4.6 Regression Coefficients of Minnesota Soil

Sample ID		w (%)	γ_d (pcf)	k_1	k_2	k_3	R^2	M_R (ksi)	Ψ (kPa)
1	0.8OMC	21	92.5	624	0.41	-2.83	0.96	7.0	310
2	0.9OMC	23.5	95.0	370	0.40	-2.60	0.98	4.5	290
3	OMC	26.0	96.0	304	0.26	-1.53	0.97	3.9	180
4	1.1OMC	28.5	95.0	180	0.30	-1.14	0.88	2.6	105
5	1.2OMC	31.0	92.0	164	0.20	-1.03	0.87	2.2	50

Table 4.7 Regression Coefficients of Mississippi Soil

Sample ID		w (%)	γ_d (pcf)	k_1	k_2	k_3	R^2	M_R (ksi)	Ψ (kPa)
1	0.8OMC	7.4	123.4	415	0.88	-0.72	0.96	9.2	26
2	0.9OMC	8.4	124.3	293	1.01	-0.50	0.94	7.1	26
3	OMC	9.4	124.6	252	1.10	-0.46	0.98	6.5	25
4	1.1OMC	10.4	124.3	213	1.03	0.00	0.83	5.8	24
5	1.2OMC	11.4	123.4	123	1.10	0.00	0.83	3.7	23

Table 4.8 Regression Coefficients of Louisiana Soil

Sample ID		w (%)	γ_d (pcf)	k_1	k_2	k_3	R^2	M_R(ksi)	Ψ (kPa)
1	0.8OMC	9.1	118.7	790	0.43	-2.44	0.96	10.0	120
2	0.9OMC	10.3	120.7	491	0.77	-2.02	0.97	8.0	118
3	OMC	11.4	121.4	461	0.80	-1.97	0.96	7.5	115
4	1.1OMC	12.5	120.7	260	1.27	-2.76	0.94	4.6	110
5	1.2OMC	13.7	118.7	153	1.60	-2.30	0.96	3.6	90

From the tables, it can be noticed that the quality of the data collection was good and the fitted models described the collected data well as judged by the R^2 values which are in the high range of 0.83 to 0.98. As expected, the values of representative resilient modulus increased with the decreasing of water content. The specimens compacted at 0.8xOMC showed the highest resilient modulus compared to the modulus of specimens compacted at other moisture contents. Moreover, the values of matric suctions of the soil specimens are also presented in the above tables. The variation of representative resilient modulus with respect to matric suction of the selected subgrade soils are provided in Figure 4.25.

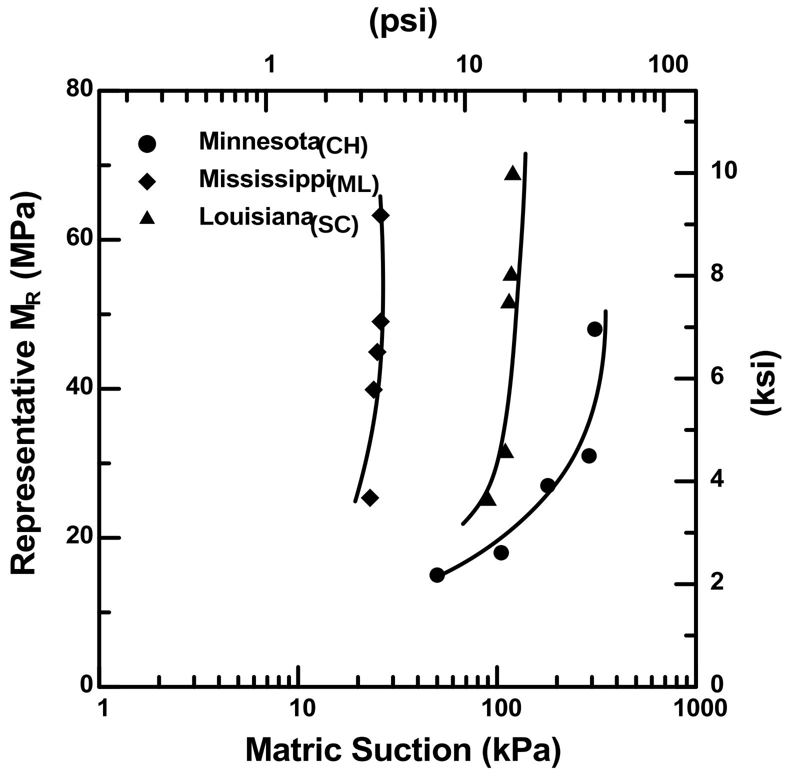

Figure 4.25 M_R - matric suction relationships

From the figure, the relationships between representative resilient modulus and matric suction (log scale) are presented. In general, M_R values increase with the increasing in matric suction. In this study, the samples are compacted at different densities which indicate inconsistency in the matric suction values; with this reason, it can be observed that the suctions, sometime, are not much different although the specimens were compacted at different moisture content (see the curves of Mississippi and Louisiana soils in Figure 4.25).

By substituting the nonlinear parameters k_1, k_2, and k_3 into the Modified Universal model presented in Equation (4-14), the resilient modulus of the subgrade soils can be back calculated. Figures 4.26, 4.27, and 4.28 illustrate the comparisons between predicted M_R and measured M_R of Minnesota, Mississippi, and Louisiana subgrade soils respectively. These results are compared by plotting predicted M_R on Y-axis and measured M_R on X-axis.

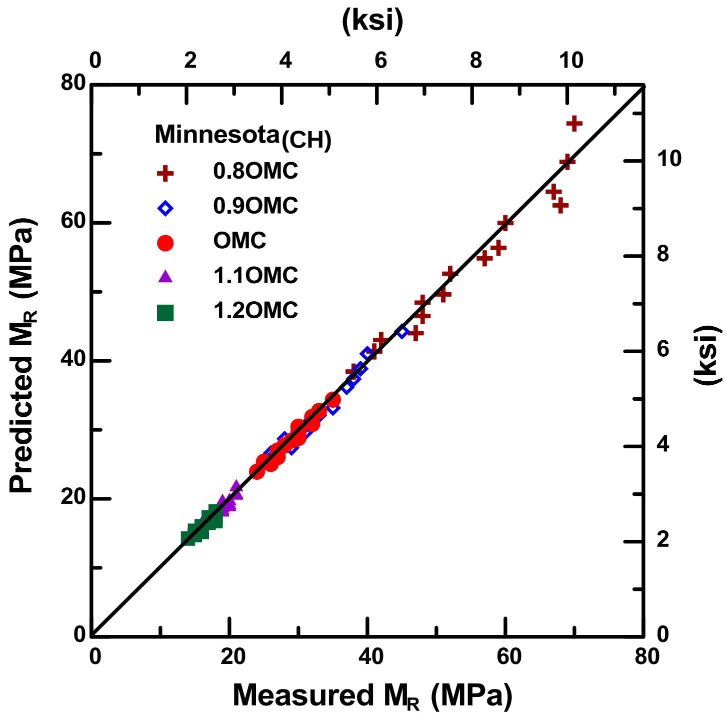

Figure 4.26 Comparison between measured M_R and predicted M_R (Universal model) of Minnesota soil

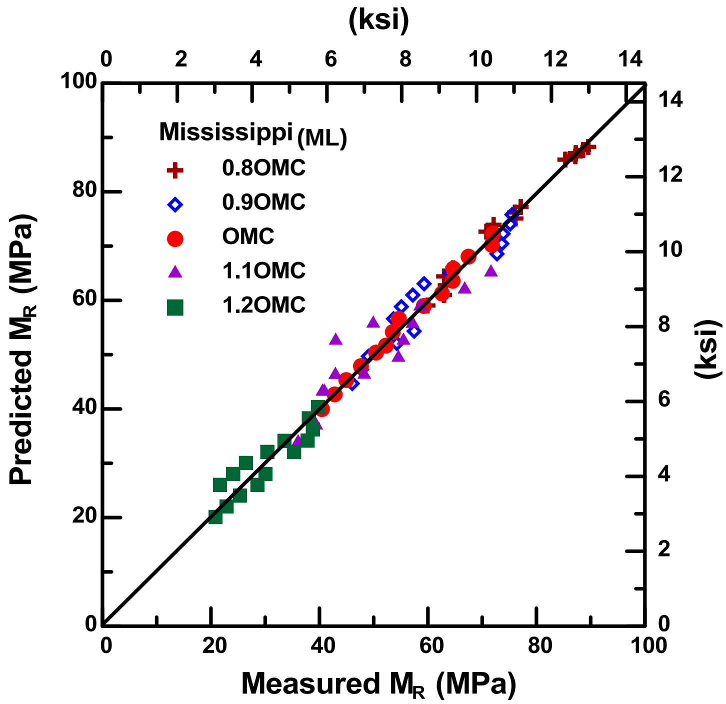

Figure 4.27 Comparison between measured M_R and predicted M_R (Universal model) of Mississippi soil

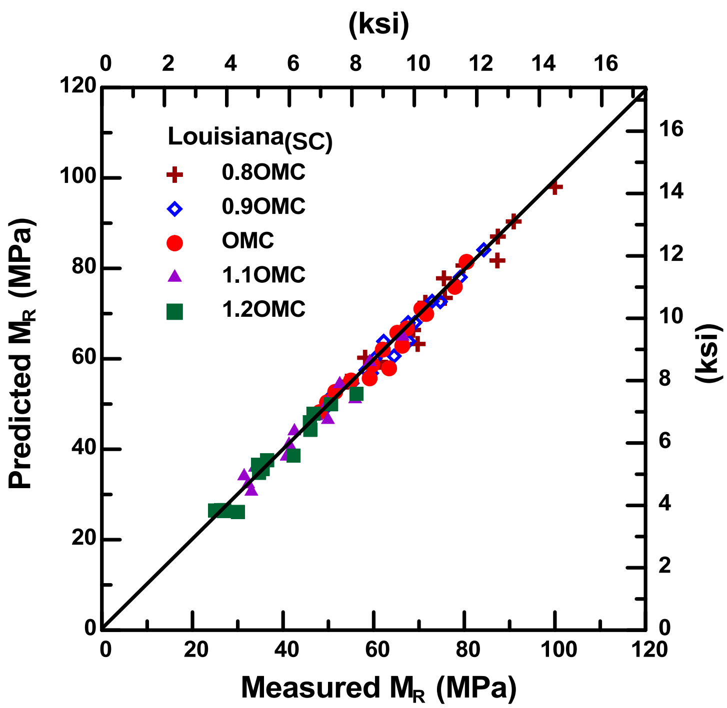

Figure 4.28 Comparison between measured M_R and predicted M_R (Universal model) of Louisiana soil

From the Figures 4.26 to 4.28, it can be observed that the modified universal model provides the predicted M_R values that have a best fit to the measured M_R values of Minnesota, Mississippi, and Louisiana specimens. The predicted resilient moduli are nearly the same as the measured results for all selected subgrade soils specimens.

Another suction based model was also studied in the following analysis. The results are discussed here.

4.5.2.2 Cary and Zapata model

As previously discussed in chapter 2, various research studies have been conducted to predict M_R values. In addition, most of the models are based on regression analysis on specific types of soils. In this study, revised model of Cary and Zapata (2010) was used to predict the M_R values. This model was proposed for both fine- and coarse- grained materials in terms of particle size and plasticity of the materials:

$$log\left(\frac{M_R}{M_{R-opt}}\right) = (\alpha + \beta \times e^{-wPI})^{-1} + \frac{(\delta + \gamma \times wPI^{0.5}) - (\alpha + \beta \times e^{-wPI})^{-1}}{1 + e^{\left(\ln\left(\frac{-(\delta + \gamma \times wPI^{0.5})}{(\alpha + \beta \times e^{-wPI})^{-1}}\right) + (\rho + \omega \times e^{-wPI})^{0.5} \times \left(\frac{S - S_{opt}}{100}\right)\right)}} \quad (4\text{-}15)$$

Where M_R = resilient modulus at a given time and moisture level,

M_{R-opt} = resilient modulus at optimum moisture content,

w = water content,

PI = Plasticity Index,

S = degree of saturation corresponding to M_R,

S_{opt} = optimum degree of saturation corresponding to M_{Ropt}, and

α, β, δ, γ, ρ, ω = model fitting parameters as function of soil type and gradation.

Based on the previous studies and results, Cary and Zapata (2010) concluded the model fitting parameters as

α = -0.6, β= -1.87194, δ= 0.8, γ= 0.08, ρ= 11.96518, and ω= -10.19111

By using the model parameters and the data provided from lab testing (w, PI, S, S_{opt}, and $M_{R\text{-}opt}$), resilient modulus at any different moisture level (0.8OMC, 0.9OMC, 1.1OMC, and 1.2OMC) can be predicted. The measured and predicted resilient modulus results of Minnesota, Mississippi, and Louisiana soil specimens are tabulated in the following Tables 4.9, 4.10, and 4.11. Moreover, the comparisons between measured and predicted M_R of the subgrade soils specimens are also provided in Figures 4.29 to 4.31.

Table 4.9 Measured vs Predicted M_R Results of Minnesota Specimens

σ_3 (kPa)	σ_d (kPa)	M_{R-opt} MPa (ksi)	Resilient Modulus M_R, MPa (ksi)							
			0.8OMC		0.9OMC		1.1OMC		1.2OMC	
			M*	P*	M*	P*	M*	P*	M*	P*
41.4	13.8	34 (4.9)	70 (10)	84 (12)	45 (7)	54 (8)	21 (3)	24 (3)	18 (3)	23 (3)
	27.6	32 (4.6)	67 (10)	78 (11)	39 (6)	50 (7)	21 (3)	23 (3)	17 (3)	21 (3)
	41.4	30 (4.4)	59 (9)	73 (11)	35 (5)	47 (7)	20 (3)	21 (3)	17 (3)	19 (3)
	55.2	28 (4.1)	51 (7)	68 (10)	31 (5)	44 (6)	20 (3)	20 (3)	16 (2)	18 (3)
	68.9	26 (3.8)	47 (7)	64 (9)	29 (4)	41 (6)	19 (3)	18 (3)	16 (2)	17 (3)
24.6	13.8	33 (4.8)	69 (10)	80 (12)	40 (6)	51 (7)	21 (3)	23 (3)	18 (3)	21 (3)
	27.6	30 (4.4)	60 (9)	75 (11)	37 (5)	48 (7)	19 (3)	22 (3)	17 (3)	20 (3)
	41.4	28 (4.1)	52 (8)	70 (10)	33 (5)	45 (7)	19 (3)	20 (3)	16 (2)	19 (3)
	55.2	27 (3.9)	48 (7)	65 (9)	28 (4)	42 (6)	18 (3)	19 (3)	16 (2)	18 (3)
	68.9	25 (3.6)	41 (6)	62 (9)	26 (4)	39 (6)	17 (3)	18 (3)	15 (2)	16 (2)
13.8	13.8	31 (4.5)	68 (10)	76 (11)	38 (6)	48 (7)	20 (3)	22 (3)	18 (3)	20 (3)
	27.6	29 (4.2)	57 (8)	71 (10)	35 (5)	45 (7)	19 (3)	20 (3)	16 (2)	19 (3)
	41.4	27 (3.9)	48 (7)	66 (10)	31 (5)	42 (6)	18 (3)	19 (3)	15 (2)	18 (3)
	55.2	25 (3.6)	42 (6)	62 (9)	26 (4)	40 (6)	17 (3)	18 (3)	15 (2)	17 (3)
	68.9	24 (3.5)	38 (6)	59 (9)	24 (4)	38 (6)	16 (2)	17 (3)	14 (2)	16 (2)

* M-measured & P-predicted

Table 4.10 Measured vs Predicted M_R Results of Mississippi Specimens

σ_3 (kPa)	σ_d (kPa)	M_{R-opt} MPa (ksi)	Resilient Modulus M_R, MPa (ksi)							
			0.8OMC		0.9OMC		1.1OMC		1.2OMC	
			M*	P*	M*	P*	M*	P*	M*	P*
41.4	13.8	64 (9)	85 (12)	75 (11)	73 (11)	69 (10)	43 (6)	60 (9)	35 (5)	57 (8)
	27.6	66 (10)	87 (13)	78 (11)	74 (11)	71 (10)	50 (7)	62 (9)	38 (6)	59 (9)
	41.4	68 (10)	87 (13)	81 (12)	74 (11)	74 (11)	59 (9)	64 (9)	39 (6)	61 (9)
	55.2	70 (10)	89 (13)	83 (12)	75 (11)	76 (11)	67 (10)	66 (10)	38 (6)	63 (9)
	68.9	72 (10)	90 (13)	86 (12)	76 (11)	78 (11)	72 (10)	68 (10)	40 (6)	65 (9)
24.6	13.8	52 (7)	71 (10)	61 (9)	54 (8)	56 (8)	41 (6)	49 (7)	22 (3)	46 (7)
	27.6	54 (8)	72 (11)	64 (9)	55 (8)	58 (8)	48 (7)	51 (7)	24 (4)	49 (7)
	41.4	57 (8)	76 (11)	67 (10)	57 (8)	61 (9)	55 (8)	53 (8)	26 (4)	51 (7)
	55.2	59 (9)	76 (11)	70 (10)	59 (9)	64 (9)	55 (8)	55 (8)	30 (4)	53 (8)
	68.9	61 (9)	77 (11)	73 (11)	64 (9)	66 (10)	57 (8)	58 (8)	34 (5)	55 (8)
13.8	13.8	40 (6)	60 (9)	47 (7)	46 (7)	43 (6)	36 (5)	38 (5)	21 (3)	36 (5)
	27.6	43 (6)	63 (9)	51 (7)	48 (7)	46 (7)	39 (6)	40 (6)	23 (3)	38 (6)
	41.4	45 (7)	63 (9)	54 (8)	49 (7)	49 (7)	40 (6)	43 (6)	25 (4)	41 (6)
	55.2	48 (7)	63 (9)	57 (8)	54 (8)	52 (8)	41 (6)	45 (7)	29 (4)	43 (6)
	68.9	50 (7)	65 (9)	60 (9)	57 (8)	55 (8)	43 (6)	47 (7)	30 (4)	45 (7)

* M-measured & P-predicted

Table 4.11 Measured vs Predicted M_R Results of Louisiana Specimens

σ_3 (kPa)	σ_d (kPa)	$M_{R\text{-}opt}$ MPa (ksi)	Resilient Modulus M_R, MPa (ksi)							
			0.8OMC		0.9OMC		1.1OMC		1.2OMC	
			M*	P*	M*	P*	M*	P*	M*	P*
41.4	13.8	81 (12)	100 (15)	140 (20)	84 (12)	105 (15)	66 (10)	67 (10)	56 (8)	60 (9)
	27.6	76 (11)	87 (12)	131 (19)	79 (12)	99 (14)	59 (9)	63 (9)	51 (7)	56 (8)
	41.4	71 (10)	76 (11)	122 (18)	73 (11)	92 (13)	53 (8)	59 (9)	48 (7)	53 (8)
	55.2	67 (10)	71 (10)	116 (17)	69 (10)	87 (13)	50 (7)	55 (8)	46 (7)	50 (7)
	68.9	63 (9)	70 (10)	109 (16)	68 (10)	82 (12)	50 (7)	52 (8)	46 (7)	47 (7)
24.6	13.8	70 (10)	91 (13)	121 (18)	75 (11)	91 (13)	56 (8)	58 (8)	42 (6)	52 (8)
	27.6	66 (10)	80 (12)	114 (17)	68 (10)	86 (12)	47 (7)	55 (8)	37 (5)	49 (7)
	41.4	62 (9)	71 (10)	107 (16)	62 (9)	80 (12)	43 (6)	51 (7)	35 (5)	46 (7)
	55.2	59 (9)	66 (10)	102 (15)	60 (9)	77 (11)	41 (6)	49 (7)	36 (5)	44 (6)
	68.9	56 (8)	61 (9)	97 (14)	60 (9)	73 (11)	41 (6)	46 (7)	35 (5)	42 (6)
13.8	13.8	58 (8)	87 (13)	100 (15)	65 (9)	75 (11)	42 (6)	48 (7)	27 (4)	43 (6)
	27.6	55 (8)	76 (11)	95 (14)	58 (9)	71 (10)	34 (5)	46 (7)	26 (4)	41 (6)
	41.4	53 (8)	69 (10)	91 (13)	55 (8)	69 (10)	31 (5)	44 (6)	25 (4)	39 (6)
	55.2	50 (7)	58 (8)	86 (13)	51 (7)	65 (9)	32 (5)	41 (6)	27 (4)	37 (5)
	68.9	48 (7)	54 (8)	83 (12)	50 (7)	62 (9)	33 (5)	40 (6)	30 (4)	36 (5)

* M-measured & P-predicted

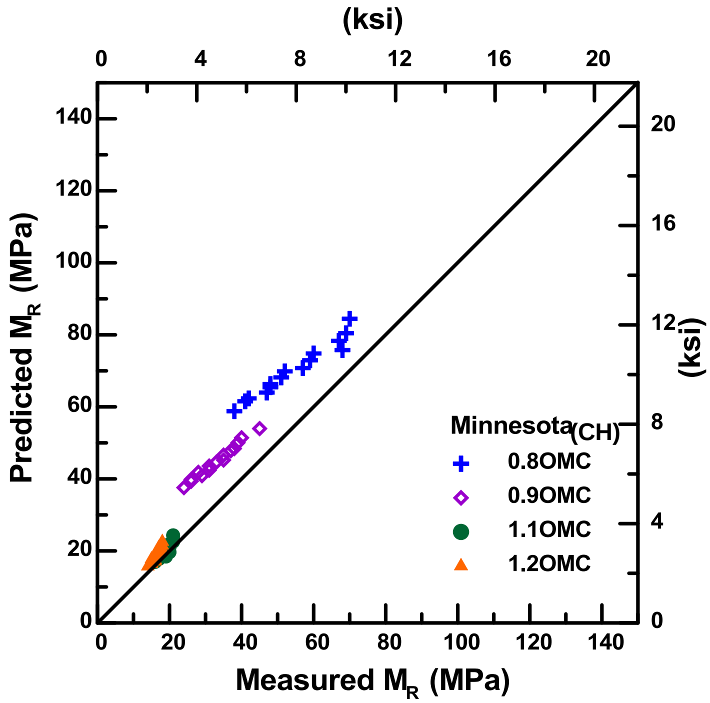

Figure 4.29 Predicted vs. Measured M_R results (Cary and Zapata model) of Minnesota specimens

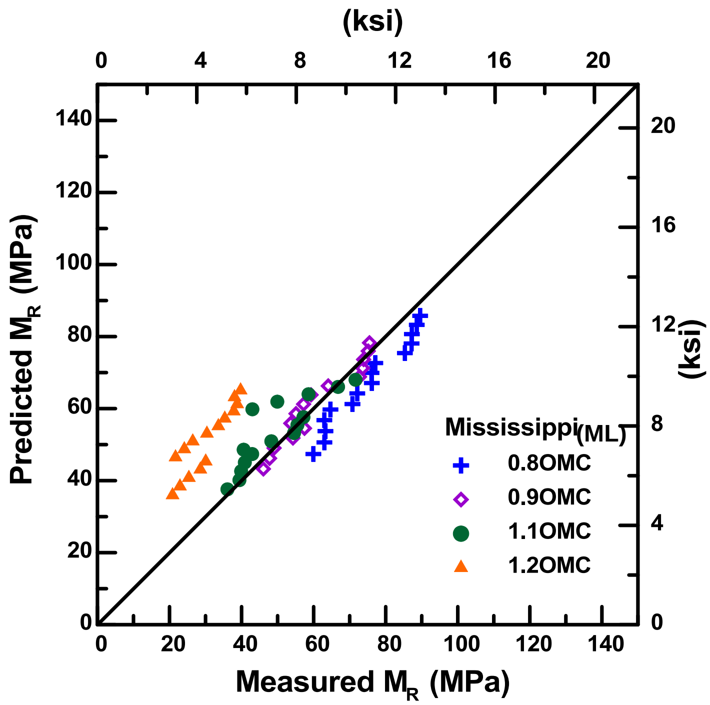

Figure 4.30 Predicted vs. Measured M_R results (Cary and Zapata model) of Mississippi specimens

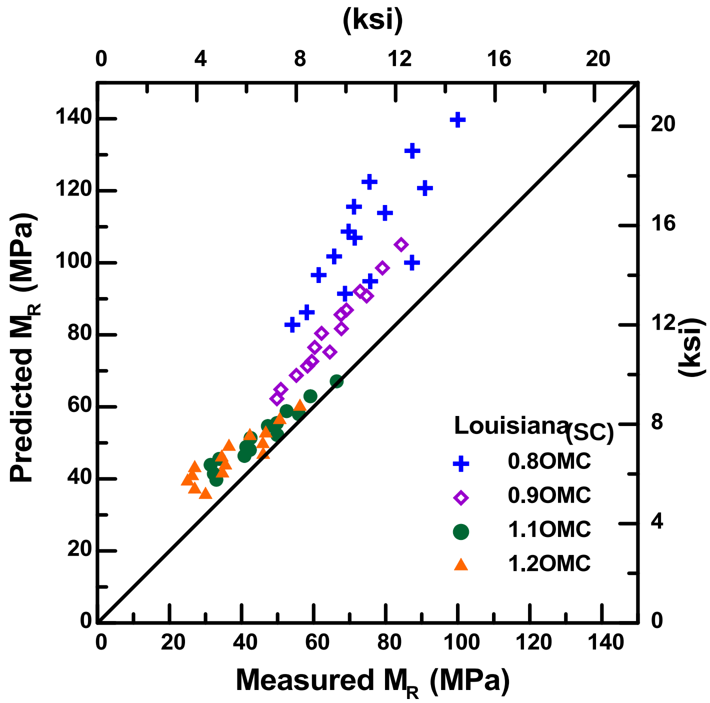

Figure 4.31 Predicted vs. Measured M_R results (Cary and Zapata model) of Louisiana specimens

From the above figures, it can be noticed that for Minnesota and Louisiana specimens, the model proposed by Cary and Zapata provides the predicted M_R higher than the measured M_R especially for the specimens compacted at 0.8OMC. However, the predicted results of the specimens compacted on the wet side (1.1OMC and 1.2OMC) show a well match with the measured results. For Mississippi specimens, the measured resilient modulus results are well matched with the predicted results; however, for the specimen compacted at 1.2OMC, the predicted M_R are slightly higher than the measured M_R.

Comparing between the resilient modulus values predicted by Modified Universal model and Cary and Zapata (2010) model, it can be observed that the

Modified Universal model provided the predicted resilient modulus better fit to the measured results than those predicted by Cary and Zapata (2010) model. This is because the Modified Universal model requires more testing information to calculate the regression coefficients and generate the model for predicting the values of resilient modulus at the different moisture content of the soil specimens. But, for Cary and Zapata (2010) model, only the testing results of the specimens compacted at optimum moisture content are required and from the results, the values of resilient modulus at any moisture content are predicted.

4.6 Summary

This Chapter mainly discusses the advanced engineering soil tests which include unconfined compressive strength, soil water characteristic curve and resilient moduli testsproperties of Minnesota, Mississippi, and Louisiana subgrade materials. Effect of confining pressure, deviatoric stress and matric suction on the resilient properties are explained. The final section covers the regression modeling analysis of the resilient moduli results using three parameter confining pressure and deviatoric stress model (Modified Universal model) and Cary and Zapata model. Also, the soil water characteristic curve predictions using MEPDG method was evaluated for the present base materials.

The next chapter summarizes all the different tests that were conducted in this research and conclusions were made based on these studies. Also, recommendations for future studies were provided.

CHAPTER 5

SUMMARY AND CONCLUSIONS

5.1 Summary

In this thesis research work, three different subsoils were selected and studied for their resilient moduli characterization. Basic soil tests such as sieve analysis, Atterberg limits, and Proctor compaction tests were first conducted. Soil water characteristic curves (SWCCs) of the subgrade materials compacted at different conditions are established by performing tests using Tempe cell method together with filter paper technique. Then, other tests such as unconfined compressive strength and resilient modulus tests were performed with the specimens compacted at five different moisture content-dry density conditions. The results of the tests were collected and analyzed. Both Universal model and Cary and Zapata (2010) model were used to model test results and predict the resilient moduli of the selected subgrade soils. Based on the experimental data and analyses performed, the following conclusions are drawn:

1. The higher unconfined compressive strength was observed on dry side of the optimum condition for all three soils. The highest unconfined compressive strength of 38 psi was observed for Minnesota specimen

compacted at the condition of 0.8OMC moisture content whereas the highest compressive strength of 19 psi and 49 psi were observed at the same condition (0.8OMC) in Mississippi and Louisiana specimens respectively.

2. The shape of the soil water characteristic curves of the compacted subgrade soils depends on compaction conditions. This is because at varied compaction water content and dry density condition, the pore structure inside specimens are changed and hence resulted in the changing in shape of the SWCCs. Higher air entry suctions are obtained when specimens were compacted at wet side of optimum water content condition (1.2OMC).
3. From the resilient modulus test results, it can be concluded that both confining and deviatoric stresses have shown a major influence on the resilient moduli values of the subgrade materials. An increase in deviatoric stress resulted in decreasing of the resilient modulus of Minnesota and Louisiana subgrades whereas for Mississippi soil, the resilient modulus increased with the increasing of the deviatoric stress. Slightly higher resilient modulus was also observed when higher confining pressures were applied. These trends are similar to those expected for cohesive soils.
4. Compaction moisture content and related soil suction value has affected the values of resilient modulus of the subgrade soils. The specimens

compacted at dry side of optimum moisture content showed higher values of resilient modulus than the sample compacted at OMC or wet side of OMC.

5. The level 2 input for predicting SWCCs of subgrade soils provided in MEPDG provided the predictions which showed similar trends as the measured SWCCs of all three soils tested. However, the curves (predicted and measured SWCCs) were not quite well matched with the measured magnitudes. The SWCCs from measurement is preferable and recommended to use in the MEPDG design of pavements.
6. From the results obtained from resilient modulus analysis, it was noted that resilient modulus values of Minnesota, Mississippi, and Louisiana specimens have shown to be modeled well with the modified universal model. For the soil suction based model proposed by Cary and Zapata, for Mississippi specimens, the predicted resilient moduli are well matched with the measured results. However, for Minnesota and Louisiana specimens, the model provided the predicted resilient modulus higher than the measured results especially, the specimens compacted at 0.8OMC.

5.2 Recommendations for Future Research

From the experience received from this study, some important recommendations for future research are proposed.

1. Soils used in testing should be collected from a broader range of soil types in order to increase the amount of testing and validate results for all types of cohesive soils.
2. For the future research, suction controlled resilient modulus testing on the subgrade materials should be considered.

REFERENCES

1. AASHTO Designation T307-99: “Standard Method of Test for Determining the Resilient Modulus of Soils and Aggregate Materials”. American Association of State Highway and Transportation Officials, Washington, D.C.
2. ASTM Standard D5298-03, (2003). “Standard Test Method for Measurement of Soil Potential (Suction) Using Filter Paper”. Annual Book of ASTM Standards, 4(8), ASTM International, West Conshohocken, PA.
3. Abu-Farsakh, M., Alshibli, K., Nazzal, M., and Seyman, E. (2004). “Assessment of In Situ Test Technology for Construction Control of Base Course and Embankments”. Final Report No. FHWA/LA.04/385, Louisiana Transportation Research Center, Baton Rouge.
4. Anderson, D.G. and Woods, R.D. (1975). "Comparison of field and laboratory shear moduli". Proceedings, Conference on In situ Measurement of Soil Properties. Specialty Conference of the Geotechnical Engineering Division A.S.C.E., Raleigh, North Carolina, vol.1, pp.69-92.
5. Barksdale, R.D., Alba, J., Khosla, P.N., Kim, R., Lambe, P.C., and Rahman, M.S. (1997). “Laboratory Determination of Resilient Modulus for Flexible Pavement Design”. NCHRP Web Document 14, Federal Highway Administration, Washington, D.C., 486 pp.

6. Burczyk, J. M., Ksaibati, K., Sprecher, R. A., and Farrar, M. J. (1995). “Factors Influencing Determination of a Subgrade Resilient Modulus Value”. Transportation Research Record 1462, Transportation Research Board, National Research Council, Washington, D.C., pp. 72-78.
7. Cary, C.E. and Zapata, C.E. (2010). “Enhanced Model for Resilient Response of Soils Resulting from Seasonal Changes as Implemented in Mechanistic-Empirical Pavement Design Guide”.
8. Choubane, B. and McNamara, R.L. (2000). “Flexible Pavement Embankment Moduli Using Falling Weight Deflectometer (FWD) Data”. Research Report FL/DOT/SMO/00-442, State Materials Office, Florida Department of Transportation, Tallahassee.
9. Drumm, E. C., Reeves, J. S., Madgett, M. R., and Trolinger, W. D. (1997). “Subgrade Resilient Modulus Correction for Saturation Effects”. Journal of Geotechnical and Geoenvironmental Engineering, Vol. 123, No. 7, pp. 663-670.
10. Edil, T., Benson, C., and Sawangsuriya, A. (2006). “Resilient Behavior of Unsaturated Subgrade Soils”. Interim Report to University of Minnesota, University of Wisconsin–Madison.
11. Elliot, R. P., Thorton, S. I., Foo, K. Y., Siew, K. W., and Woodbridge, R. (1988). “Resilient Properties of Arkansas Subgrades”. Final Report, TRC-94, Arkansas Highway and Transportation Research Center, University of Arkansas, Fayetteville.

12. Fredlund, D. G., and Xing,A.(1994). "Equations for the Soil-Water Characteristic Curve". Canadian Geotechnical Journal, Vol. 31, No. 4, pp. 521– 532.

13. Gardner, R. (1937). "A Method for Measuring the Capillary Tension of Soil Moisture over a Wider Moisture Range". Soil science, Vol. 43, pp. 277-293.

14. Guan, Y. (1996). "The Measurement of Soil Suction". A thesis Submitted to the Faculty of Graduate Studies and Research, Department of Civil Engineering, University of Saskatchewan, Saskatoon, Canada.

15. Gupta, S., Ranaivoson, A., Edil, T., Benson, C., Sawangsuriya, A. (2007). "Pavement Design Using Unsaturated Soil Technology". Report No. MN/RC-2007-11, Final Research Report submitted to Minnesota Department of Transportation, University of Minnesota, Minneapolis.

16. Hopkins, T., Beckham, T. L., Sun, C., and Ni, B. (2004). "Resilient Modulus of Kentucky Soils". Final Research Report for Kentucky Transportation Cabinet, Kentucky Transportation Center, University of Kentucky.

17. Hossain M. S. (2008). "Characterization of Subgrade Resilient Modulus for Virginia Soils and Its Correlation with the Results of Other Soil Tests". Final Report VTRC 09-R4, Virginia Transportation Research Council.

18. Hossain M.S. and Apeagyei A.K. (2010). "Evaluation of the Lightweight Deflectometer for In-Situ Determination of Pavement Layer Moduli". Research Report VTRC 10-R6, Virginia Department of Transportation.

19. Khoury, N. N., and Zaman, M. M. (2004). “Correlation between Resilient Modulus, Moisture Variation, and Soil Suction for Subgrade Soils”. Transportation Research Record: Journal of Transportation Research Board 1874, National Research Council, Washington, D.C., pp. 99-107.

20. Kung, J.H.S., Lin, H.D., Yang, S., and Huang, W. (2006). “Resilient modulus and plastic strain of unsaturated cohesive subgrade soils”. Unsaturated Soils, ASCE 2006, pp. 541-552.

21. Lambe, T. W. and Whitman, R. V. (2000). Soil Mechanics, John Willy and Sons Inc., New York, US.

22. Lee, W., Bohra, N.C., Altschaeffl, A.G., and White, T.D. (1997). “Resilient modulus of cohesive soils”. Journal of Geotechnical and Geoenvironmental Engineering, Vol. 123, No. 2, Feb. 1997, pp. 131-136.

23. Leong, E.C. and Rahardjo,H.(1996). “A Review on Soil-Water Characteristic Curve Equations”. Geotechnical Research Report, NTU/GT/96-5, Nanyang Technological University, NTU-PWD Geotechnical Research Center, Singapore.

24. Liang, R. Y., Rabab’ah, S., and Khasawneh, M. (2008). “Predicting Moisture-Dependent Resilient Modulus of Cohesive Soils Using Soil Suction Concept”. Journal of Transportation Engineering, ASCE, January, pp.34-40.

25. Malla, R. B. and Joshi, S. (2006). “Establishing Subgrade Support Values for Typical Soils in New England”. Report No. NETCR 57, New England Transportation Consortium, Fall River, Mass.

26. McCartney, J.S., Selvam, R.P., King, J., and Khosravi, A. (2010). "Evaluation of the enhanced integrated climatic model for the Arkansas State Highway and Transportation Department". Research Report, University of Arkansas-Fayetteville.

27. Mohammad, L. N., Puppala, A. J., and Alavilli, P. (1994a). "Influence of Testing Procedure and LVDTs Location on Resilient Modulus of Soils". Transportation Research Record 1462, Transportation Research Board, National Research Council, Washington, D.C., pp. 91-101.

28. Mohammad, L. N., Puppala, A. J., and Alavilli, P. (1994b). "Investigation of the Use of Resilient Modulus for Louisiana Soils in Design of Pavements". Final Report No. 283, Louisiana Transportation Research Center, Federal Highway Administration, Louisiana Department of Transportation and Development.

29. Mohammad, L.N., Puppala, A. J., and Alavilli, P. (1995). "Resilient Modulus of Laboratory Compacted Subgrade Soils". Transportation Research Record 1504, Transportation Research Board Record, National Research Council, Washington, D.C., pp. 87–102.

30. Mooney, M.A., Rinehart, R.V., Facas, N.W., Musimbi, O.M., White, D.J., and Vennapusa, P.K.R. (2010). "Intelligent Soil Compaction Systems," NCHRP Report 676, Transportation Research Board, Washington, DC, pp. 178.

31. Nazarian, S. and Pezo, R. (1996). “An Approach to Relate Laboratory and Field Moduli of Base Materials.” Report No. 1336-2F, Texas Department of Transportation in cooperation with Federal Highway Administration, Texas.
32. Nazarian, S. and Yuan, D. (2008). “Variation in Moduli of Base and Subgrade with Moisture." ASCE Geotechnical Special Publication. Volume 178. Page(s) 570-577.
33. NCHRP (2004). “Guide for Mechanistic Empirical Design of New and Rehabilitated Pavement Structure: Part 2 Design Inputs: Chapter 3 Environmental Effects”. Transportation Research Board, National Research Council.
34. Nelson, C.R., Petersen, D. L., Peterson, R. L., Rudd, J. C., and Sellman, E. (2004). “Design and Compaction Control for Foundation Soil Improvements.” T.H. 61 Reconstruction, New Port, Minnesota (CD-ROM), Presented at the 83rd Annual Meeting of the Transportation Research Board, Washington, D.C.
35. Newcomb, D.E. and Birgisson, B. (1999).“NCHRP Synthesis of Highway Practice 278: Measuring In-Situ Mechanical Properties of Pavement Subgrade Soils”. Transportation Research Board, National Research Council, Washington, D.C.

36. Ooi, P. S. K., Sandefur, K. G., and Archilla, A. R. (2006). “Correlation of Resilient Modulus of Fine-Grained Soils with Common Soil Parameters for Use in Design of Flexible Pavements,” Report No. HWY-L-2000-06, Hawaii Department of Transportation, Honolulu.
37. Pacheco, L. G. and Nazarian, S. (2011). “Impact of Moisture Content and Density on Stiffness-Based Acceptance of Geomaterials,” In Journal of Transportation Research Board 90th
38. Padilla, J. M., Perera, Y. Y., Houston, W. N., and Fredlund, D. G. (2005). “A New Soil-Water Characteristic Curve Device”. Proceeding of Advanced Experimental Unsaturated Soil Mechanics, an International Symposium, EXPERUS 2005, Trento, Italy. pp. 15-22.
39. Pedarla, A. (2009). “Durability studies on stabilization effectiveness of soils containing different fractions of montmorillonite”, Thesis submitted to the department of civil engineering, The University of Texas at Arlington, Texas in partial fulfillment of the master degree.
40. Puppala, A.J. (2008). “Estimating Stiffness of Subgrade and Unbound Materials for Pavement Design,” NCHRP Synthesis 382, Transportation Research Board, Washington, DC, 129 p.
41. Puppala, A.J., Thammanoon, M., Nazarian, S., Hoyos, L.R., and Chittoori, B. (2012). “In-situ matric suction and moisture content measurements in expansive clay during seasonal fluctuations”. Geotechnical Testing Journal, Vol.35

42. Richter, C. (2006). “Seasonal Variations in the Moduli of Unbound Pavement Layers,” Publication No. FHWA-HRT-04-079, Turner–Fairbanks Highway Research Center, McLean, Va.
43. Sabnis, A., Abdallah, I., Pacheco, L.,Nazarian, S., and Puppala, A.J. (2009). “Impact of Moisture Variation on Strength and Deformation of Clays,” Research Report No. 0-5430-01, Center for Transportation Infrastructure Systems, The University of Texas at El Paso, El Paso, Texas.
44. Santha, B.L. (1994). “Resilient Modulus of Subgrade Soils: Comparison of Two Constitutive Equations,” Transportation Research Record 1462, Transportation Research Board, National Research Council, Washington, D.C., pp. 79–90.
45. Sawangsuriya, A., Edil, T. B., and Bosscher, P. J. (2008). “Modulus-suction-moisture relationship for compacted soils.” National Research Council, Can.Geotech. J. Vol. 45.
46. Seim, D. K. (1989). “A Comprehensive Study on the Resilient Modulus of Subgrade Soils.” Oregon State University, Corvallis, Oregon.
47. Shook, J. F., Finn, F. N., Witczak, M. W., and Monismith, C. L. (1982). “Thickness Design of Asphalt Pavements – The Asphalt Institute Method”. Proc., 5th International Conference on the Structural Design of Asphalt Pavements, Vol. 1, the Netherlands, pp. 17 – 44.

48. Siekmeier J. A. (2011). "Unsaturated Soil Mechanics Implementation during Pavement Construction Quality Assurance," proceedings of 59th Annual Geotechnical Engineering Conference, St. Paul, MN.
49. TxDOT Designation: TEX-110-E (1999). "Test Procedure for Particle Size Analysis of Soils". Texas Department of Transportation, TX, pp. 1-13.
50. Thom, N. H. and Brown, S. F. (1988). ''The effect of grading and density on the mechanical properties of a crushed dolomitic limestone.'' Proc., 14th ARRB Conf., Part 7, 94–100.
51. Thomson, M. R. and Robnett, Q. L. (1976). "Resilient Properties of Subgrade Soils". Final Report-Data Summary, Transportation Engineering Series 14, Illinois Cooperative Highway Research and Transportation Program Series, University of Illinois, Urbana.
52. Titus-Glover, L. and Fernando, E.B. (1995). "Evaluation of Pavement Subgrade Material Properties and Test Procedures," Report No. FHWA/TX-96/1335-2, Texas Transportation Institute, Texas A&M University, College Station. TX.
53. Thudi, H. R. (2006). "Assessment of soil-water retention properties of lime and cement treated clays", M.S. Thesis, The University of Texas at Arlington.
54. Uzan, J. (1985). "Characterization of Granular Materials," Transportation Research Record 1022, Transportation Research Board, National Research Council, Washington DC., pp. 52-59.

55. Von Quintus, H. L. Rao. and Killingsworth, B. (1998). “Analyses Relating to Pavement Material Characterizations and Their Effects on Pavement Performance,” FHWA-RD-97-085, Federal Highway Administration, McLean, Va.

56. VonQuintus, H.L., Rao, C., Bhattacharya, B., Titi, H., and English, R. (2010). “Evaluation of Intelligent compaction technology for Densification of Roadway Subgrades and Structural Layers,” Submitted to the Wisconsin Highway Research Program (WHRP), Draft Final Report.

57. Wolfe, W. E. and Butalia, T. S. (2004). “Seasonal Instrumentation of SHRP Pavements,” Final Report for Ohio Department of Transportation, Ohio State University, Columbus.

58. Woolstrum, G. (1990). “Dynamic Testing of Nebraska Soils and Aggregates”. Transportation Research Board, National Research Council, Washington, D.C., pp. 27-34.

59. Yang, S., Lin, H., Kung, J. H. S., and Huang, W. (2008). “Suction-Controlled Laboratory Test on Resilient Modulus of Unsaturated Compacted Subgrade Soils”. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, September, pp. 1375-1384.

60. Yau, A. and Von Quintus, H. L. (2002). “Study of LTPP Laboratory Resilient Modulus Test Data and Response Characteristics,” Report No. FHWA-RD-02-051, Federal Highway Administration, McLean, Va.

61. Zaman, M., and Khoury, N. (2007). "Effect of Soil Suction and Moisture on Resilient Modulus of Subgrade Soils in Oklahoma," Final Report for Oklahoma Department of Transportation. Norman. Oklahoma.

62. Zapata, C.E.(2009). "Consideration of Climate in New AASHTO Mechanistic-Empirical Pavement Design Guide," Transportation Research Board Annual Meeting CD, Paper #09-2254.

63. Zapata, C.E. and Houston, W.N. (2008). "Calibration and Validation of the Enhanced Integrated Climatic Model for Pavement Design," NCHRP Report 602, National Cooperative Highway Research Program, Transportation Research Board, of the National Academies, Washington, DC., pp. 62.

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