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Innovation in Transportation

Laboratory Dynamic modulus of asphalt mixes and Resilient Modulus of Soils Throughout New Mexico tor the Implementation of Mechanistic Empirical Pavement Design Guide (MEPDG)

PART 2: Laboratory Resilient Modulus of Soils

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LABORATORY DYNAMIC MODULUS OF ASPHALT MIXES AND RESILIENT MODULUS OF SOILS THROUGHOUT NEW MEXICO FOR THE IMPLEMENTATION OF MECHANISTIC EMPIRICAL PAVEMENT DESIGN GUIDE (MEPDG)

PART 2: LABORATORY RESILIENT MODULUS OF SOILS

Report Submitted to

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SUMMARY PAGE

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Resilient modulus (M_R) of soils is a well-accepted and essential parameter for structural design of flexible pavements using the Mechanistic Empirical Pavement Design Guide (MEPDG). A comprehensive laboratory study is undertaken to determine the resilient modulus (M_R) and unconfined compressive strength (UCS) of untreated and lime treated subgrade soils and a granular aggregate base soil. Two subgrade soils: AASHTO class A-6 and A-7-6 and one granular base material (50%-50% Aggregate-RAP blend) are collected from US 491 and I-40 projects, respectively. Two lime percentages (5% and 7%) are selected on the basis of pH test. Cylindrical soil samples are prepared according to NCHRP 1-28A at different percentages of lime (0%, 5%, 7%) with three molding moisture contents: optimum moisture content (OMC), dry state (OMC-2%) and wet state (OMC+2%). M_R test is conducted on the untreated soils and base materials following the AASHTO T 307 stress sequences. However, M_R tests on lime treated soils are conducted with a modified stress sequence incorporated in the AASHTO T 307 procedure based on past literature and laboratory experience gained in this study. Test results reveal that M_R and UCS values increase due to lime treatment depending on soil type and lime dose. A-6 subgrade soil shows M_R and UCS values increase going from 5% to 7% lime dose. In A-7-6 soil, M_R and UCS values decrease going from 5% to 7% lime dose. Compaction moisture affects M_R and UCS values of lime stabilized soils more than untreated clayey soils. For granular base material, M_R values increases with the increase of cell pressure and deviator stress, which was expected. In addition, regression coefficients (k's) of MEPDG soil constitutive equation are developed for each soil for using in the MEPDG (now DARWin-ME) software.</td></tr>
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PREFACE

The research report herein determines the resilient modulus of soils for the state of New Mexico. The project aims at developing resilient modulus data as a level 2 inputs of MEPDG for New Mexico Department of Transportation.

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Chapter 1 INTRODUCTION

1.1. Background

Traditionally, pavement thickness has been designed empirically (1993 Pavement Design Guide), that is, based on past experience, soil support value, and under static loading (e.g., a plate load or CBR test). Mechanistic-empirical analysis method has been adopted recently by the American Association of State Highway and Transportation Officials (AASHTO) for designing pavements. Resilient modulus (M_R) is an essential input of Mechanistic Empirical Pavement Design Guide (MEPDG). M_R is used to characterize stress-strain behavior of unbound base, subbase, and subgrade material layers supporting asphalt surface layer. Therefore, accurate determination of resilient modulus is required for designing an optimum pavement thickness. For M_R testing in the laboratory, a cylindrical soil specimen is subjected to combinations of deviator and confining stress in a triaxial pressure chamber. Cyclic load and corresponding recoverable deformation are measured to determine resilient modulus of soil.

MEPDG requires M_R value of aggregate base and subgrade materials. Recycled asphalt pavement (RAP) materials are currently used widely in base layers to produce structurally sound green pavement. Similarly, chemical stabilization of clayey subgrade soils has gained popularity recently due to long-term performance and improved engineering properties of subgrade soil. Among different stabilizing agents: cement, lime, fly ash etc. recommended by the MEPDG design guide, lime is the oldest and the most common stabilizer used in New Mexico. Introduction of calcium hydroxide increases the pH, causing the silica and alumina in the clay particles to become soluble and interact with calcium in a pozzolanic reaction. A pozzolanic reaction between silica and alumina in the clay particles and calcium from the lime occurs to create a cemented structure that increases the strength of the stabilized soil. Thus lime treatment results in higher resilient modulus (M_R) and unconfined compressive strength (UCS) of lime treated soils than untreated soils.

M_R value of subgrade soils may be affected by compaction moisture content. Construction specification requires to compact pavement subgrades with more than 90% of maximum dry density and at optimum moisture content (OMC). However, Uzan (1998) documented that clayey material underneath the pavement increases its moisture content to approximately 20 to 30% higher than its plastic limit and reaches equilibrium condition during first 3 to 5 years of service life. Moreover, subgrade moisture level can vary with the rising levels in the water table, infiltration of water and inadequate drainage facility. Extensive literature of past studies (Khoury et al. 2012) concluded that M_R decreases with an increase in moisture content and vice-versa. Though it is believed that strength of lime treated subgrade soil are less sensitive due to seasonal variation of moisture, limited studies addressed moisture sensitivity and soaking potential in the laboratory determination of resilient modulus of lime stabilized soils.

MEPDG recommends two standard protocols for determining resilient modulus in the laboratory. They are AASHTO T 307 and NCHRP 1-28A. As resilient modulus testing includes small deviator stress and small deformation measurement, accurate load and deformation measurements are critical. Moreover, these test protocols have stress sequences for untreated subgrade soils. Using these stress sequences, it is very difficult to measure deformation response of stabilized soils. Therefore, a modified stress sequence is sought for M_R testing of lime-treated soils in this study.

With the movement towards calibrating and implementing MEPDG in New Mexico, laboratory determined resilient modulus values of aggregate base and subgrade soils are required for pavement engineers' of New Mexico Department of Transportation (NMDoT). Therefore, granular base and untreated subgrade soil resilient modulus test are conducted for department engineers' immediate use and MEPDG implementation. For lime stabilized subgrade soil, AASHTO M_R test method is modified to overcome the limitations of available test standards for accurate estimation of M_R for stabilized soils

1.2. Objectives

Primary objective of this study is to determine laboratory resilient modulus (M_R) and unconfined compression strength (UCS) of New Mexico base, untreated and lime-treated subgrade soils. Two common subgrade soils i.e. AASHTO class A-6 (lean clay) and A-7-6 (fat clay) are tested as untreated (0% lime). These soils are treated with 5% and 7% lime and tested for M_R at three different molding moisture content. Selected aggregate base material has 50% RAP and 50% granular aggregate. Specific objectives are to:

- Develop moisture-density relationship for each soil type with or without stabilization
- Conduct repeated load triaxial tests to determine M_R and static UCS values
- Determine coefficients of soil constitutive model used in MEPDG for each subgrade soils.

1.3. Organization of Report

Chapter 1 describes the research needs and objectives of the study. Technical literature review focused on previous research relevant to the report includes in Chapter 2. Development of M_R test protocols, differences between current test standards, factors affecting M_R of soils, deformation measurement methods and past researches on untreated, lime-treated subgrade soils and RAP-Base soil are described in Chapter 2. Chapter 3 includes the summary of materials collection and material classifications. Chapter 4 introduces detail information about sample preparation method and experimental setup. M_R test results are presented in Chapter 5. M_R and UCS test results are listed in tabular and graphical forms with MEPDG predictive models. Chapter 6 summarizes the findings of this study.

Chapter 2 LITERATURE REVIEW

2.1. Definition of Resilient Modulus

Resilient modulus (M_R) is a measure of stiffness corresponding to resilient strains due to cyclic loads at various stress combinations designed to simulate traffic loads a soil element would experience based on its respective location within the pavement structure. Mathematically M_R is defined by Eq. (2.1):

$$M_R = \frac{\sigma_{cyclic}}{\epsilon_r} \tag{2.1}$$

where σ_{cyclic} is the maximum axial cyclic stress, and ϵ_{r} is the resilient strain associated.

Direct measurement of resilient modulus can be made in the laboratory by applying a combination of deviator and confining stresses while measuring the corresponding recoverable axial strain. This chapter focuses mainly on:

- Development of M_R test protocols
- Differences among current test standards
- Test equipment effects and deformation measurement techniques
- Factors affecting M_R of soils
- Past researches on lime-treated subgrade soils and RAP-base soil

This chapter provides the reader with a background understanding of resilient modulus as it applies to subgrade soils, the factors that influence M_R test results.

2.2. Resilient Modulus Test Protocols

The concept of resilient modulus was originally defined by Seed et al (1962) as the ratio of applied dynamic stress (σ_d) to the resilient or recovered (elastic) strain component (ε_r) under transient dynamic pulse load. This definition of M_R value was accepted by the pavement community because the elastic pavement deflection showed better correlation to field performance than to total pavement deflection (Witczak et al. 1995).

A sinusoidal stress pulse of varying magnitude is exerted to pavement structure from traffic. Stress pulse decreases in magnitude, and the pulse duration increases with depth (Lee et al. (1997)). This can be simulated by a repeated axial stress that is separated by a rest period applied to a sample confined with a constant static pressure. In a repeated load test, the dissipated energy in a given loading cycle decreases as the number of loading cycle increases. After a number of loading cycles the modulus becomes nearly constant (i.e., materials become resilient) and the response can be assumed approximately elastic. This steady value of modulus is defined as resilient modulus and is assumed to occur after 200 cycles of loading (Drumm et al. (1990)).

First modern test method for resilient modulus adopted was AASHTO T 274 (1982). In this protocol, vehicle speed and depth beneath the pavement surface were considered in selecting the appropriate axial compressive stress pulse time to use in repeated load testing. The concept of resilient modulus was subsequently incorporated into the 1986 AASHTO Guide for design of pavement structures. Criticism on these methods was on test procedures, length of test duration (5 hours), and insufficient description of displacement measurement devices (Puppala (2008))

In 1988, a thorough review of ASHTOO T 274 was conducted by the LTPP materials Expert Task Group (ETG) and the LTPP (Long term pavement performance) team. This group identified areas within the standard that were ambiguous or that required alternatives. Through this process, LTPP Protocol P46, was developed and issued in 1989 with external deformation measurement methods. Over the years, the protocol was revised and amended and was issued in its final form in 1996. In between of these, AASHTO adopted T 292 (1991) and T 294 (1992) with recommendation of internal deformation measurement techniques and introduction of two parameters regression model. Due to complexity in using internal measurement techniques, P46 was balloted through the AASHTO process and was adopted (with some modification) as AASHTO standard T 307-1999 (Puppala (2008), Groeger et al. (2003)).

Subsequently, National Cooperative Highway Research Program (NCHRP) conducted a project 1-28 (1997) on M_R. The primary objective of this study was to develop enhanced laboratory test procedures for determining resilient moduli of asphalt concrete, aggregate base/subbase materials and subgrade soil using limited multi-lab validation and comparative field measured value. This test protocols more accurately account for varying field conditions, such as temperature of the asphalt surface layer and moisture content of a subbase or subgrade layer with advantages of reduced test time and more reproducible test results. Andrei (1999) went into great depth in detailing and comparing the four recent standards at that time: AASTHO T 292, T 294, P46 and NCHRP 1-28 Draft-97. The objective was to harmonize existing standards into a single protocol. Many key differences were found between these protocols including: deformation and load measurement locations, stress magnitudes and sequences, confining stresses and unconfined resilient modulus testing, material type characterizations and compaction methods. The result of the Andrei's study was the development of the NCHRP 1-28A (2004) protocol. Currently, two protocols are considered as accepted standards in determining resilient modulus by MEPDG: NCHRP 1-28A and AASHTO T 307. However, none of these protocols has any comments how to conduct M_R testing of lime-stabilized subgrade soils.

2.3. Differences Among Current Resilient Modulus Test Protocols

Differences between the AASHTO T307 and NCHRP 1-28A protocols for fine-grained subgrades include: deformation and load cell location, stress sequences (number, load duration, magnitudes), sample size and compaction methods. Summary of these differences are listed in Table 2.1.

2.3.1. Load Cell and Deformation Sensor Location

AASHTO T 307 requires load cell and spring guided Linear Variable Differential Transducer (LVDT) mounted outside the pressure chamber. However, these are placed inside the triaxial cell in NCHRP 1-28A. Standard gauge lengths are ¼ diameter points, i.e. for 2:1 height to diameter samples this is the middle half of the sample for both the methods. The standard includes notes on maximum ranges, and minimum sensitivities. Capacity and range of load cell and LVDTs according to AASHTO T307 are listed in Table 2.2.

2.3.2. Test Sequences

Both AASHTO T 307 and NCHRP 1-28A use a haversine shaped loading pulse. However, the duration of the pulse is 0.2 seconds followed by a 0.8 second rest period for NCHRP 1-28A, rather than a 0.1 second pulse followed by a 0.9 second rest period in AASHTO method. While groupings of test sequences 1-5, 6-10, and 11-15 have confining pressures of 6, 4, and 2 psi respectively for fine grained subgrade soil in AASHTO T 307. There are 16 test sequences for 1-28A compared to 15 for T 307 and the cyclic stresses keep values that can be grouped in sequences, i.e. sequences 1-4, 5-8, 9-12, and 13-16 have cyclic stresses of 4,7,10, and 14 psi respectively. Also note that the contact stress is not 10% of the deviator stress for 1-28A and the minimum and maximum cyclic stresses during the test are higher for NCHRP 1-28A than for AASHTO T 307.

2.3.3. Sample Size and Compaction Method

Another difference between the two standards is the type of compaction method when reconstituting specimens. AASHTO T 307 mentions multiple methods compaction for fabricating resilient modulus specimens namely, vibratory, static, and kneading compaction. Though AASHTO T 307 allows a standard or modified compaction effort (AASHTO T 99 or T 180) to determine the optimum moisture content and maximum dry density of a remolded resilient modulus specimen, it does not mention standard or modified compaction efforts as method for fabricating test samples. NCHRP 1-28A allows for impact compaction, however only samples of 4 inch and 6 inch diameters are considered. AASHTO T307 permits three standard sizes of resilient modulus specimens: 2.8, 4, and 6 inch diameters with 2:1 height to diameter ratio. However, NCHRP 1-28A does not include reconstituted 2.8 inch diameter specimens, only undisturbed specimens of this size are considered.

2.4. Equipment Effects and Deformation Measurement Techniques

The two main pieces of equipment discussed in this section are deformation sensors (LVDTs) and load cells. Resilient modulus is a stress controlled test in which stress and strain values are used to calculate M_R. Therefore, accurate measurements of load and deformation are critical for a successful test.

2.4.1. Load Cell

Most load cells used in resilient modulus testing are strain gauge type load cells, with a design stiffness and linear deflection range up to maximum load output. Primary physical issues concerning the load cell in resilient modulus testing are concerned with the location of the load cell, i.e. inside or outside the triaxial cell.

Groeger et al. (2003) pointed concerns for using both external and internal load cells. During the use of an external load cell, attention must be paid to ensure that friction between the loading piston and confining chamber is minimized. This deformation becomes a concern with T 307 because test deformation measurements are taken outside the triaxial cell, thus load cell deformation contributes directly to strain values and reduces the M_R values. A study performed by Bejarano et al. (2003) supports concern for frictional forces influencing load cell readings when the load cell is placed externally. Results showed that external load readings become 15% higher than internal for high stress measurement. In conclusions to a study on triaxial cell interaction examining drag forces on the loading rods, Boudreau and Wang (2003), recommended that internal load and deformations measurements can "eliminate or reduce the inherent errors associated with equipment variation." They added that such decision introduces a tester to the difficulty and extra time associated with implementing internal instruments.

2.4.2. Deformation Transducer

Historically internal and external deformation techniques for resilient modulus have been shown to produce results, which differ greatly from one another. Boudreau and Wang (*22*) concluded that though internal measurements for stress and strain can reduce the errors associated with the equipment variation, M_R test becomes very difficult and time consuming with internal LVDTs. External LVDTs make the test setup less difficult than mounting the internal LVDTs, however many publications have pointed out the difference in values between the two measurement methods and/or the potential influence of erroneous deformations on resilient modulus values when using external LVDTs (Kim and Drabkin (1994), Burczyk et al. (1994), Greoger et al. (2003), Bejarano et al. (2003), Konrad and Robert (2003), Boudreau and Wang (2003)). Many of these studies compared measurement on sample using spring-type LVDTs located in ring clamps on the specimen versus actuator mounted external LVDTs.

Barksdale et al. (1997) and Andrei (2003) found that in regards to deformation measurement, the best method to determine resilient modulus, though more difficult, is fixing buttons to the specimen. Barksdale et al. (1997) included a comparison between ring clamp and epoxied button LVDTs and concluded that extraneous displacements and potential slip of ring-clamp LVDTs can be eliminated by epoxied button LVDTs. Andrei (2003) compared internal measurement setups using synthetic specimens of varying stiffness values and reported questionable results in non-linearity range. He also pointed out that the existing testing standards do not provide for modulus values measured smaller than the non-linearity range. One synthetic sample obtained modulus value of 210 ksi which author pointed out is achievable by a subgrade soil in a dry state. Though the resolution was increased 10 times (from a 0.2 to 0.02 in range), one-quarter of the data fell below the non-linearity range. In defense of the non-linearity behavior affecting the resilient modulus, the author showed that in one instance the modulus values decreased and eventually stabilized when the deformation was beyond the non-linearity range while in another instance it increased then stabilized. Since the synthetic specimen used as an example should behave elastically and should not show stress-strain dependency in this case, the author stated that the values at the low strain levels should equal the stabilized values outside the non-linear portion of the LVDTs.

2.5. Factors affecting Resilient Modulus of soils

Resilient modulus value of cohesive subgrade soils decreases with deviator stress due to softening effect. For granular base and subbase materials, the resilient modulus increases with increasing deviator stress, which typically indicates strain hardening due to reorientation of the grains into a denser state. Factors affect the resilient modulus of soils are: moisture content, unit weight, loading conditions, loading characteristics, confining stress, compaction method etc. Within the scope of this study, brief review of effect of resilient modulus due to moisture, unit weight and confining pressure are discussed below.

2.5.1. Effects of Moisture Content & Unit Weight

Research studies showed that the moisture content and unit weight (or density) have significant effects on the resilient modulus of subgrade soils. The M_R of subgrade soil decreases with the increase of the moisture content or the degree of saturation (Fredlund et al. (1977), Mohammad et al. (1996), Drumm et al. (1997), Huang (2001), Butalia et al. (2003), Heydinger (2003) and Titi et al. (2006)). Moisture content is the primary variable for predicting seasonal variation of resilient modulus of subgrade soils (Heydinger (2003), Titi et al. (2006)). Mohammad et al. (1996) attributed this reduction in the resilient modulus to an increase in positive pore pressures with an increase in moisture content associated with greater levels of saturation (Butalia et al. (2003)). The effect of unit weight on the resilient modulus of subgrade soils also has been largely investigated by Drumm et al. (1997) and Titi et al. (2006). They concluded that resilient modulus of the soil compacted on the dry side of optimum is larger than that of the soil compacted at the wet of optimum. In regards to material influences, fine grained soils are influenced by moisture changes, with the difference in resilient modulus between wet and dry conditions being 100% or larger (Barksdale et al.1997).

2.5.2. Influence of Confining Pressure

Confining pressure in the upper soil layers under pavements are normally less than 35 kPa (5 psi). Most laboratory studies on unbound materials showed that resilient modulus increases with the increment of confining pressure (Seed et al. (1962), Thompson and Robnett (1976), Butalia et al. (2003) and Titi et al. (2006)). However, studies on cohesive soils showed confining stresses had little effect on resilient modulus values. Thompson and Robnett (1976) found that repeated loading testing with no confining pressure was acceptable for resilient modulus testing of cohesive soils. Fredlund et al. (1977) found that for a soil with a PI of nearly 17 percent, confining stresses from 3 to 6 psi were insignificant. Muhanna et al. (1999) performed resilient modulus tests on 4 in. diameter A-5 and A-6 specimens at varying moisture contents. They showed that there was no significant effect due to the number of load applications, rest period, or load sequence. Authors concluded that confining pressures in the range of 0 to 10 psi had less than a 5% effect on resilient modulus.

2.6. Studies on Lime Stabilized Subgrade Soils

Several researchers have conducted intense studies on subgrade soils stabilized with cement, lime, Class C Fly Ash (CFA), Cement Kiln Dust (CKD). Lime stabilization is only considered within the scope of the study. Prushinki et al (1999) summarized the studies of lime stabilized soils. After that, Geiman (2005), Osinubi and Nwaiwu (2006) discussed only the improvement of unconfined compressive strength (UCS) due to lime stabilization. Other studies (Chang (1995), Qubain et al. (2000), Kim and Siddiki (2006), and Mooney et al. (2010)) included both resilient properties and failure strength. Little (2000) and Solanki et al. (2009) investigated not only stiffness and strength improvement, but also highlighted long term performance parameters (moisture susceptibility and three-dimensional swell).

Chang (1995) conducted resilient properties of a fine grained Lateritic soil stabilized with CFA and lime. AASHTO T 274-82 protocol was used for resilient modulus test with external deformation measurement methods, thus obtained M_R value varied between 125 to 250 MPa (18 to 36 ksi).

Little (2000) performed extensive study on lime stabilized subgrade soil test procedure and attempted to address all the required design inputs for the MEPDG. This test procedure namely Mixture Design Testing Procedure (MDTP) is the recommended method for characterizing lime stabilized soils by the M-E Design Guide. According to the procedure, soil having Plasticity Index (PI) greater than 10 and more than 25% passing to no. 200 sieve is suitable for lime stabilization. He also recommended accelerated curing of 7 days at 105 °F as this curing time is short enough to be feasible for mix design purposes yet long enough to provide reasonable values for long term cure at ambient temperature in the field. AASHTO T 294-92 and ASTM D5102 were used for resilient modulus and unconfined compressive strength test respectively. Samples were tested following a 24 hour capillary soak to simulate critical moisture state in the field. He reported that lime stabilization often induced an increase of 1,000% or more in resilient modulus values than that of the untreated soil or aggregate. M_R test results obtained 35.6-79.2 MPa (5.2-11.5 ksi) and 210-625 MPa (30.5-90.6 ksi) for untreated and lime-treated clayey subgrade soil respectively. But no comment was made for deformation measurement techniques. Values of back calculated (from field FWD testing) resilient moduli typically fall within a range of from 210 MPa (30 ksi) and 3,500 MPa (507 ksi). Thus laboratory obtained M_R largely fall behind the field obtained FWD value. He reported UCS value exceeds 1400 kPa (200 ksi) due to lime stabilization, even as high as 7,000 to 10,000 kPa (1,000 to 1,450 psi). This study also concluded that stabilization not only improved M_R and UCS values but also reduced the sensitivity of strength to the effects of moisture.

Kim et al. (2006) investigated lime and LKD (lime-Kiln-Dust) in subgrade soils as soil modifier, not stabilizer. They did the resilient modulus test just after five hours of mixing. They reported that untreated and treated soils showed similar resilient behavior changes due to changes in confining pressure and treated soils showed negligible effects due to deviator stresses as compared to the untreated soil. However, addition of lime and LKD resulted in increased unconfined compressive strengths.

Solanki et al. (2009) conducted intense experiments on engineering properties of four subgrade soils stabilized by lime, CFA and CKD. AASHTO T 307 and ASTM D1633 were used for resilient modulus and unconfined compressive strength test for all soil combinations. Results showed that all stabilizers improved the strength/stiffness properties from raw soil as formation of crystals with soil matrix. At lower application rates (3% to 6%), the lime-stabilized soil specimens showed the highest improvement in the strength/stiffness. Maximum improvement in strength was obtained for the soil which has lower PI. However, a reduction in stiffness was obtained for a type of soil beyond a certain percentage of lime addition.

Study performed by Mooney et al. (2010) revealed that the relationship between resilient modulus and UCS in MEPDG level-2 was conservative in its prediction of M_R from UCS. They used external deformation measurement techniques for resilient modulus testing, thus obtained small values of resilient modulus. They concluded that there was no universal equivalent accelerated curing duration for all lime-stabilized subgrade soil with respect to unconfined confined strength results, thus 5 day 100 °F accelerated curing protocol was recommended in field application.

Though some of the studies mentioned above are relevant to the present study, independent studies for New Mexico is important as textural and mineralogical characteristics of soils are different in every region.

2.7. Studies on Recycled Asphalt Pavement (RAP) Base Materials

Bejarano et al. (2001) conducted experimental program to evaluate a rehabilitation strategy that involves recycling an existing asphalt surface and a portion of the base material into a new base layer. Resilient modulus testing was performed according to AASHTO TP-46 on two base materials with one source of RAP. The RAP material was stiffer than the two base materials at 95% and 100% densities, and M_R values increased as density increased. They reported resilient modulus value ranged from 300 to 450 MPa (43 to 65 ksi) for the RAP-Base material at 95% compaction.

Effect of RAP on aggregate base mechanical properties was studied by Cooley (2005) with different RAP content. The California bearing ratio (CBR) test was used to measure strength, the resonant column test was used to measure stiffness, and the tube suction test (TST) was used to measure moisture susceptibility. This study reported that CBR values decreased as RAP content increased, with the greatest percentage reduction occurring with the addition of 25 percent RAP. For stiffness testing for each blend at the optimum moisture content, the general trend was a decrease in stiffness from 0 percent RAP to 25 percent RAP, followed by a steady increase in stiffness as the RAP content was increased from 25 to 100 percent. Following a 72-hr drying period at 140 ºF, however, the general trend reversed; an increase in stiffness occurred as the RAP content was increased from 0 to 25 percent, and a steady decrease in stiffness was observed for RAP contents above 25 percent.

Laboratory tests were conducted by Mokwa & Peebles (2006) using four different aggregates blended with milled asphalt over a broad range of mix percentages. Test results indicate that at higher levels of RAP, the characteristics of the asphalt millings increasingly control the behavior of the blend. This results in decreased stiffness and reduced shear strength. The decreasing trends observed in laboratory tests may not be significantly detrimental, and do not necessarily preclude the use of RAP/aggregate blends in highway pavement sections.

Kim et al. (2007) investigated compaction effort and effect of RAP content on stiffness of base aggregate. They compared vibratory hammer compaction (suggested by standard protocols) with gyratory compaction. Test results summarized that hammer compaction did not perform well in the resilient modulus test with a significant permanent deformation and increasing modulus during the tests due to specimen densification in mid sequences. Thus gyratory compactor was recommended for RAP-Base sample compaction as it can better simulate field conditions too. NCHRP 1-28A test sequences were followed for resilient modulus testing. They found that increase in RAP percentages resulted in increase in stiffness. Samples compacted 65% of optimum moisture content were consistently stiffer than samples compacted at optimum moistures at all confining pressures. 50/50 RAP-aggregate was equivalent in stiffness to 100% aggregate at lower confining pressures and stiffer at higher confining pressures but exhibited more deformation during the conditioning sequence which the authors pointed out for further research.

Table 2.1: Differences among Resilient Modulus Test Protocols for Subgrade Soil

	Test Protocol	
Topic	AASHTO T 307	NCHRP 1-28A
Load Cell Location	Outside Cell	Inside Cell
LVDT Location	Outside Triaxial Cell, Mounted on Loading Piston	Inside Triaxial Cell, Attached to Specimen
Option of 2.8 in. Diameter Reconstituted Specimen?	Yes	No
Option of Impact Compaction?	No	Yes
Testing Sequences for Fine-Grained Subgrades	15 ea. 2,4,6,8,10 psi deviator stresses and 2,4,6 psi confining stresses	16 ea. 4,7,10,14 psi deviator stresses and 2,4,6,8 psi confining stresses
Load Pulse Form, Duration	Haversine, 0.1 s	Haversine, 0.2 s

Table 2.2: AASHTO T307 Load Cell and LVDT Capacity and Range Requirement

Specimen Diameter mm (inch)	Load cell capacity		LVDT range mm (inch)
	Maximum load capacity KN (lbs)	Required accuracy N (lb)	
71 (2.8)	2.2 (500)	±4.5 (±1)	±1 (±0.04)
102 (4.0)	8.9 (2000)	±17.8 (±4)	±2.5 (±0.1)
152 (6.0)	22.2 (5000)	±22.2 (±5)	±6 (±0.25)

Chapter 3 MATERIALS SOURCE AND PROPERTIES

3.1. General

This chapter highlights source and classification of materials used in this study. Moisture density relationship was conducted on untreated and lime-treated subgrade soils and base-RAP materials. It is required to characterize soils according to grain-size distribution, plasticity index, and moisture-density relationships prior further testing. These customary tests provide pertinent information prior to resilient modulus testing to fabricate the sample at desired density and moisture contents, as well as determining the size of specimens for testing according to grain-size distribution of the soil.

3.2. Material Source and Classification

Two subgrade soils (AASHTO classification A-6 and A-7-6) and one granular base soil (50%-50% aggregate RAP blend) were used in this study. Subgrade soils were fabricated and tested in untreated and lime stabilized conditions. Detail material source, grain-size distribution and moisture-density relationship are discussed below:

3.2.1. Subgrade Soils

Subgrade soils were sampled in untreated raw condition from the right of way of a highway project on US 491 near Shiprock in Northwestern New Mexico. 26 and 34 bags (20 lbs each) of A-6 and A-7-6 materials were collected from Station 6310+00 and 6120+00 respectively. The soils were excavated in large chunks using a backhoe loader at a depth of 5′ (A-6) and 3.5′ (A-7-6) from roadway. Shoveling was done to facilitate bagging and transportation of soils in loose state to the laboratory. After collection, soils were processed and hand pulverized to pass through the U.S. standard sieve #4 prior to sample preparation for atterberg limit and moisture-density test. Figure 3.1 and Figure 3.2 shows field soil collection and processed soil photos. Grain-size distribution of the soils was obtained based on field test near the project location. Gradation information and atterberg limit results are summarized in Table 3.1.

3.2.2. Granular Base Soil

Base material was collected from I-40 construction site, 15 miles west from I-25 and I-40 junction (Milepost 141, Station 515). The soil consisted of 50-50 base aggregate and RAP material. Gradation of this soil is given in Table 3.2, which was obtained from the closest station field data (Milepost 141, Station 517+50).

3.2.3. Additive Properties

Lime was used as additive to subgrade soils for this study. Type N hydrated lime was collected from Lafarge plant of Albuquerque, NM which was manufactured by Chemical Lime Company of Texas. Manufacturer assured that provided lime contains calcium hydroxide (>90%), magnesium oxide and hydroxide (<5%), calcium carbonate (<3%) and silicon dioxide (<2%). Other physical and chemical characteristics are: boiling and melting point 2850 °C and 580 °C, respectively, with specific gravity of 2.2-2.4 g/cc.

3.3. Selection of Lime Percentage for Stabilization

Prior to moisture-density test, lime requirement of soils were determined using Eades and Grim pH test (ASTM D6276). Subgrade soils were mixed with de-ionized (DI) water and a different amount of lime, periodically shaken and tested using a pH meter after an hour. The pH meter was calibrated using 4, 7 and 10 pH buffers with an R^2 value of 0.98. According to ASTM D6276, minimum lime content of the soil-lime mixture was confirmed when pH of 12.4 was attained. At this point it indicates that sufficient lime is available to sustain reaction for stabilizing the treated soil.

In the laboratory, it was found that pH value increased with the increase of percentage of lime. For A-6 and A-7-6 soil, pH of 12.4 was attained at lime percentage of 4 percent and 5 percent, respectively, thus indicated the minimum lime content for these soils. NMDOT's standard practice of testing soil-lime mixture is 3, 5 and 7 percent. Thus, 5% and 7% was selected as the two lime dosages for this study.

3.4. Moisture-Density Relationships

Moisture density relationship for subgrade and base aggregate was established according to AASHTO T 180 using a modified proctor test. Soils were compacted for five points (in dry and wet side) to determine the maximum dry density and optimum moisture content. Moisture contents for both loose soil and compacted solid sample were measured after oven-dried at 240 °F for 24 hours. The assumed and actual moisture content varied within 1%. The optimum moisture content (OMC) and maximum dry density (MDD) for untreated and lime-treated subgrade soils and aggregate base soil are summarized in Table 3.3. Brief discussion for sample preparation and moisture test results are given below:

3.4.1. Subgrade Soils

Sufficient quantities of soils passing the No. 4 sieve were obtained in order to determine the optimum moisture content (OMC) and maximum dry density (MDD). A manual mechanical mixer was used to mix soil, water and lime (when required) based on dry unit weight of soils in 5 gallon plastic bucket (

Figure 3.3). The mix was then sealed and allowed to mellow prior to compaction.

Moisture-density test results of untreated subgrade soils indicated that A-6 soils had OMC and MDD of 15.1% and 120 pcf (19.7 kN/m^3) respectively. For A-7-6 soil, OMC and MDD were found to be 14.7% and 124.5 pcf (20.4 kN/m^3) respectively.

For lime treated soils, two percentages of lime (5% and 7%) were added with two subgrade soils based on dry weight of soils. Soil and lime was mixed for a minute in dry state and 5 minutes or more with DI water until uniformity in soil mix was achieved. Laboratory results (Table 3.3) showed an increase in OMC and decrease in MDD with the increase in percentage of lime for each soil. Little (1996) reported that OMC increases with the increase of lime content because sufficient water is available for the soil-lime chemical reactions. Solanki et al. (2009) commented that such behavior can be attributed to the increased number of fines in the mix due to addition of lime.

3.4.2. Granular Base Soil

Sufficient quantities of base material (50%-50% aggregate-RAP) passing through 3/4″ sieve were obtained in order to determine the moisture-density relationships by modified proctor method. Optimum moisture content and maximum dry density were obtained 6.7% and 134.5 pcf respectively which was close to field obtained value (6.0% and 133.0 pcf).

Figure 3.1: Material Collection from The Site

Figure 3.2 : Processed A-6 and A-7-6 Soil Passing U.S. #4 Sieve

Figure 3.3 : Soil Mixing in Bucket Using Mechanical Mixer

Table 3.1: Subgrade Soil Gradation and Atterberg Limits

Soil Designation		
AASHTO classification	A-6	A-7-6
USCS Symbol	CL (lean clay)	CH (fat clay)
Gradation		
Sieve Opening	% passing	
1 inch	100	100
3/4 inch	100	99
1/2 inch	100	98
3/8 inch	100	97
No. 4	100	95
No. 10	98	93
No. 40	96	91
No. 50	53.7	90
No. 200	49.6	74.2
Atterberg Limits		
Liquid Limit	29	69
Plastic Limit	16	27
Plasticity Index	13	42

Sieve size	Percent passing	Specification

Table 3.2: Granular Base Soil Classification with Contractor's Specification

1 inch	100	100
3/4 in.	95	80-100
1/2 in.	69	-
3/8 in.	56	-
No.4	38	30-60
No.10	25	20-45
No. 50	9	-
No.200	4.6	3-10

Table 3.3: Summary of OMC-MDD of Untreated, Lime-Treated Subgrade and Granular Base soil

Type of Soil	% of lime additive	OMC (%)	Maximum Dry Density	
			pcf	kN/m^3
A-6	0	15.1	120.0	19.7
	5	17.2	110.6	18.1
	7	18.2	109.2	17.9
A-7-6	0	14.7	124.5	20.4
	5	16.4	115.0	18.9
	7	17.5	114.5	18.8
Base-RAP	0	6.7	134.5	22.0

Chapter 4 EXPERIMENTAL METHODOLOGY

4.1. General

This chapter describes sample fabrication and compaction method adopted for different soils for this study. Brief description of experimental setup and deformation measurement techniques for resilient modulus and unconfined compression strength testing are included. Limitations of available test protocols for lime-stabilized soils are addressed and new test sequences adopted in this study are described.

4.2. Specimen Preparation and Compaction

4.2.1. Sample Size

In general, there are three standard sizes of resilient modulus samples: 2.8, 4, and 6 inch diameters with 2:1 height to diameter ratio. Fine grained cohesive soils meet the AASHTO T 307 criteria for being reconstituted into a 2.8 inch diameter sample size according to the largest particle diameter being smaller or equal to one-fifth the size of the mold diameter. Conversely, NCHRP 1-28A does not allow 2.8 inch diameter samples. Thus, cylindrical 4 inch (101.6 mm) diameter by 8 inch (203.2 mm) height sample size was selected for subgrade soils as it is recommended by both protocols. In the interest of using less material, time and effort, 2.8 inch sample size for subgrade soil gains popularity. Therefore, 2.8 inch lime stabilized specimens were prepared and test results were compared with 4 inch samples.

For granular base material, NCHRP 1-28A recommends 6 inch diameter sample if maximum aggregate size is greater than 3/4″. As available base soil (50%-50% base-RAP) has maximum aggregate size of 1 inch, cylindrical 6 inch (152.4 mm) diameter by 12 inch (304.8 mm) height sample size was chosen for compacting base material of this study.

4.2.2. Sample Compaction

AASHTO T 307 requires a standard or modified compaction effort to determine OMC and MDD of a remolded resilient modulus specimen but it does not mention standard or modified compaction efforts as method for fabricating test specimens. NCHRP 1-28A allow for impact compaction. Table 4.1 summarizes the sample size and compaction methods followed by AASHTO T 307 and NCHRP 1-28A.

Barksdale et al. (1997) and Muhanna et al. (1999) showed the effect of impact versus kneading compaction. They concluded that resilient modulus of cohesive subgrade soils do not significantly affected particularly those compacted at optimum and wet of optimum conditions. In addition, they commented on the difficulty in obtaining target densities and moisture contents when using kneading compaction as well as on the fact that impact compaction is the standard which dictates in-situ values. While moisture-density relationships used for this study are defined using energy delivered by impact compaction, NCHRP 1-28A standard for impact compaction is considered for both subgrade and granular base soils. Impact compaction energy was provided by a modified proctor effort which delivers 56,000 ft-lbf/ft^3. In order to reconstitute different sample diameter using impact compaction for this study, necessary number of blows per layer was calculated using Eq. (3.1) from NCHRP 1-28A.

$$n = \frac{CE*V}{N*W*h} \tag{3.1}$$

For modified proctor effort:
n = number of blows
CE = compactive effort of 56,000 ft-lbf/ft^3
V = Volume of specimen
N = number of layers
W = weight of drop hammer, 10 lbf
h = drop height in feet, 1.5 ft

Based on equivalent energy delivered by the modified proctor hammer, it was found that desired densities of 95-100% of maximum dry density were always achieved. Table 4.2 summarizes number of blows per lift for different sample size.

4.2.3. Sample Preparation

All samples were compacted in split molds lined with membranes and vacuum pressure applied. Figure 4.1 shows different size of split molds used in the study. Vacuum ports at two locations, especially at a point near the highest lift, were needed due to the pressure of the soil on the membrane sealing the lower port and cutting off the vacuum. When this happens the membrane becomes loose and can get caught by the proctor hammer and in between lifts causing deformities in samples. Also multiple reinforcing hose clamps were needed to keep the split mold closed during compaction. Figure 4.2 depicts the photograph of 2.8 inch diameter mold with membrane and vacuum connection. Modified proctor hammer and impact compaction process for soil preparation is also included. Detail procedure of sample fabrication and storage mechanism for different soil types are described below:

4.2.3.1. Subgrade Soils

Subgrade soils were fabricated at three different lime percentages (0%, 5%, 7%) and three different moisture states, i.e., dry (OMC-2%), optimum and wet (OMC+2%). Raw processed subgrade soils (passing through #4 sieves) were mixed with water to achieve 1% greater than target moisture content to overcome moisture loss during the time of reconstitution, storing and test preparation. During compaction, weights of each lift were determined based on desired specimen densities and as mixed moisture contents. 4 inch diameter specimens were compacted with target dry densities between 95-100% of maximum dry density. Target ranges for dry and wet specimens were ±2-3% relative to optimum.

Lime stabilized subgrade soil mixing was performed according to ASTM D3551, same as described for the OMC and MDD determination. After mixing, soils were stored in plastic bucket for at least 8 hours for mellowing purpose. Then, the mixture was compacted in cylindrical molds of 2.8 inch and 4 inch diameter. A curing tank was set up to cure samples in an accelerated manner at 105°F for 7 days (Little (2000)). Samples were fully sealed with combinations of several rounds of plastic wrap and plastic bags. Then they were placed in the cylindrical plastic containers and submerged in water to ensure accelerated reaction of lime through the heating process. Figure 4.3 illustrates storing of treated soil in the curing tank with insulating foam in the top of the tank.

4.2.3.2. Granular Base Soil

As 6 inch diameter of base soil required approximately 30 lbs of soils, they were mixed with water with extreme care having one-third of soils in each case. Cylindrical sample was compacted at 0.3% higher than optimum to achieve MDD using modified proctor impact compaction effort.

4.2.4. End Treatments

Difficulties were encountered in achieving smooth surfaces when using impact compaction and split molds. Samples were capped in this study to ensure smooth top surfaces required for even stress distributions during testing (Mooney (2010)). The material used for capping was gypsum cement with a 0.45 water to cement ratio. Only the top end of the sample that directly exposed to the impact hammer was capped, the bottom had smooth finish. Figure 4.4 shows capping samples with portions of plastic containers, hose clamps, and a level for ensuring the smooth finish. Samples were capped immediately after compaction, sealed with plastic wrap and stored in cylinder container to mitigate moisture loss.

4.3. Experimental Program

Two tests were conducted on each soil combination i.e. resilient modulus test and unconfined compressive strength test. Load and deformation measurements play vital roles in accurate estimation of resilient modulus value.

4.3.1. Load and Deformation Measurement Techniques

For accurate estimation of resilient modulus, LVDTs are to be mounted on the specimen and load cell is be placed within the triaxial cell. These have been ensured to minimize the extraneous deformation from load cell and system compliance. Cabrera (2012) compared external and internal deformation measurements and concluded that the majority (>80%) of the deformations were extraneous when measuring externally.

Figure 4.5 shows two types of LVDTs for deformation measurements in this study. Spring guided glued button LVDTs and hollow core ring clamp LVDTs were used for subgrade and granular base soil respectively. First technique was to put the metal buttons (1/4 inch diameter by 3/8 inch height) in the holes at a distance of gauge length, then glue to the sample. This deformation measurement setup is the same as the one used for dynamic modulus testing of asphalt samples. It considered the best method to measure the deformation on the specimen due to loading, though it requires much effort and time to put the buttons and set the epoxy. Other internal deformation measurement method used in this study (for granular base materials) utilized hollow core LVDTs and a pair of ring clamps that was fitted around the membrane along with 1-72 threaded rod and small O-rings for fixing the clamps. As LVDT requirements provided in AASHTO T 307 are mentioned for external use (Table 2.2), it was sufficient to use one grade lower LVDTs i.e. ± 0.04 inch (± 1.0 mm) and ± 0.1 inch (± 2.5 mm) range LVDTs for subgrade and granular base soil respectively. Figure 4.6 and Figure 4.7 depict spring guided LVDT setup procedure and both LVDT types with corresponding soils.

Strain gauge internal load cells having capacity of 500 lbs and 2000 lbs were used for 2.8 inch and 4 inch diameter subgrade resilient modulus testing. However, 5000 lbs capacity was used for 6 inch diameter base material M_R and all UCS testing.

4.3.2. Resilient Modulus Test

Resilient modulus test for untreated subgrade and granular base soil was conducted according to AASHTO T 307 test sequences for subgrade and base soil respectively (Table 4.3). The test procedure consists of different combinations of deviator and confining stresses applied in 15 stress sequences. Additionally, untreated subgrade soils were tested for unconfined state with same deviator stress to observe the effects of confining pressure on resilient modulus value. Haversine shaped load form of $(1 - cos\theta/2)$ is applied in a triaxial pressure chamber as it is the recommended pulse shape to simulate the induced load in pavement layers (Barksdale (1997)). Each test sequence has 0.1 s of load pulse and 0.9 s of rest period. Resilient modulus for a particular test sequence is determined by averaging from the last five cycles. Stress and strain behavior for the last five cycles of a typical test sequence is illustrated in Figure 4.8. Figure 4.9 shows the GCTS universal testing system (FRM 100/SCON 1500) with subgrade soil sample. Two criteria were maintained for reporting a successful M_R test results:

1. Vertical deformation ratio (R_V) of two internal linear variable deformation transducers (LVDTs) needs to be less than 1.5.

2. Coefficient of variance of last five cycles M_R value keeps within 5.

Testing of treated materials using test sequences of untreated soils (AASHTO T 307) was a concern to Barksdale et al. (1997), because untreated soils' stress sequences are not large enough to produce measureable deformations using internal LVDTs for treated soils. At the same time, it was understood that higher stresses are not representative of those experienced by a soil element in the subgrade level. However, due to limitations of the test protocols in regards to very stiff materials, high stress sequences are needed for the LVDTs to respond outside of the typical non-linearity error range. Moreover, the preconditioning stage and confining pressure has little effect on resilient modulus of subgrade soil (Muhanna et al. (1999)). Therefore, based on the literature and laboratory experience gathered from the present study, it was decided to conduct the M_R test on lime treated soils at unconfined high stress sequences (i.e. deviator tress of 10, 15, 20, 25, 30, 40 and 50 kPa) with 200 preconditioning cycles. Table 4.3 summarizes test sequences used for this study for different soils.

4.3.3. Unconfined Compressive Strength Test

UCS is an important design indicator of mechanical behavior of subgrade soils according to MEPDG. Thus, UCS test was conducted on both untreated and lime-treated samples in which resilient modulus test was conducted before. AASHTO T 208 and ASTM D5102 is the standard test methods of UCS for untreated and lime stabilized subgrade soil respectively. These two methods have difference in maximum strain limit (15% and 5% for untreated and lime stabilized soil respectively), having similar strain rate. A strain controlled test program was created within the GCTS software giving a displacement rate command that equated to strain rate of 0.5% per minute. Test was stopped either at maximum axial strain level or the maximum stress attained, whichever comes earlier. Low precision frame mounted LVDT (± 2inch) was used for strain measurement. Thus LVDTs used for resilient modulus testing is not adequate for UCS test because the deformation is high enough to destroy them at failure point.

4.3.4. Soaking of untreated and lime-treated subgrade soils

To observe the performance of lime-stabilized subgrade soil under fully submerged or soaked condition, a sample compacted at optimum moisture state was fully submerged in water and stored in the bucket for 48 hours prior to UCS testing. All buttons for deformation measurement and plastic wrap were removed to penetrate water through any surface of soil. This was done for untreated soil also. It was observed both untreated raw A-6 and A-7-6 sample collapsed and crumbled completely to mud formation. But all lime-stabilized samples remained intact. The surface of the samples were then dried by using towels and immediately wrapped again with plastic. Weight of the sample was not comparable as some soil and glue were lost from the button space during soaking. Buttons were replaced again and capping was done prior resilient modulus testing. Figure 4.10 shows the photograph of untreated and lime-treated A-6 soil after 48 hours soaking.

4.4.Test Matrix

Total 58 specimens were compacted and tested for this study. Table 4.4 illustrates the detail description of test matrix.

2.8″ and 4″ diameter subgrade soil mold   6″ diameter base soil mold with compacted soil

Figure 4.1: Split Mold Used for Compaction

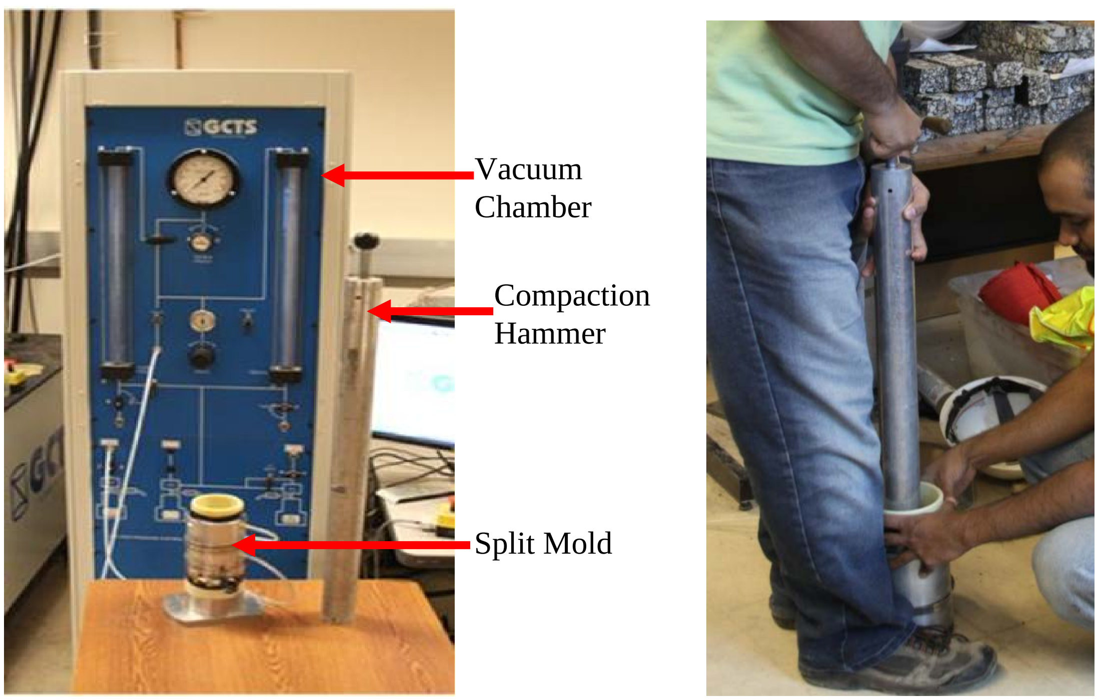


Figure 4.2: Laboratory Compaction Process with Relevant Equipment

Figure 4.3: Stabilized Subgrade Soil in the Curing Tank

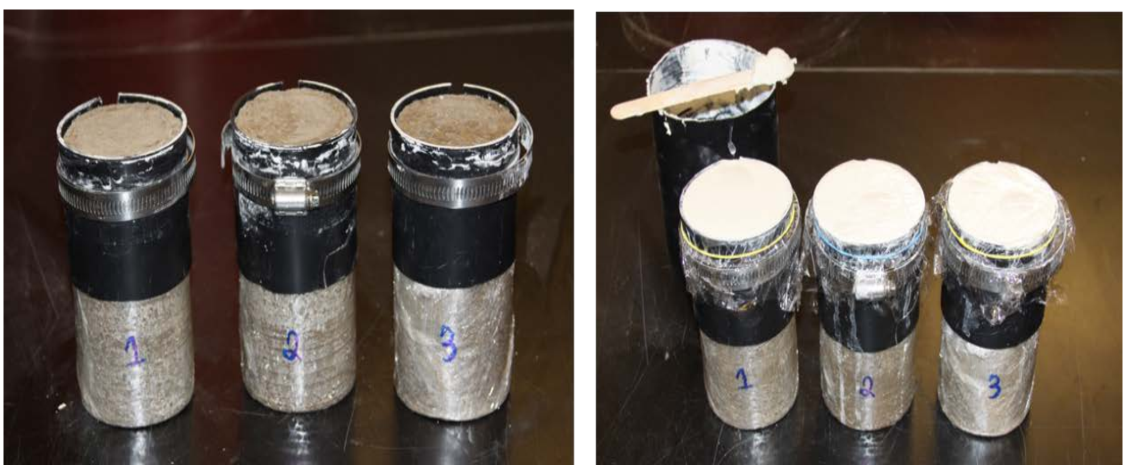
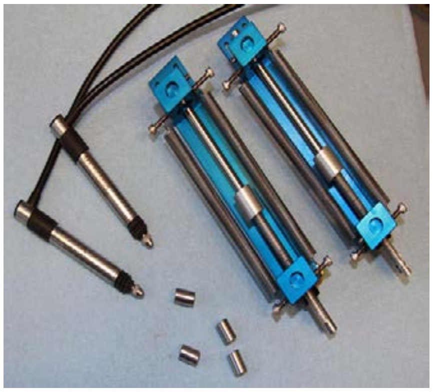


Figure 4.4: Capped Plastic Wrap Subgrade Samples



Spring guided LVDTs

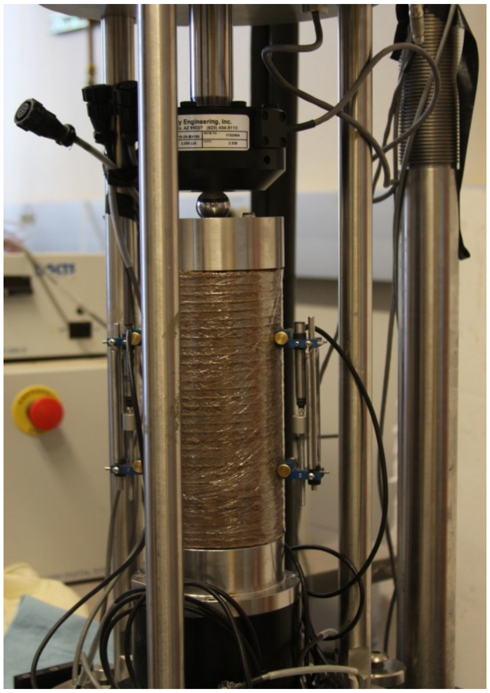
Hollow core LVDTs

Figure 4.5: Internal LVDTs Used for This Study

Placing buttons

Setting LVDT holders

Figure 4.6: Spring Guided LVDT Setup Procedure



Subgrade soil

Base soil

Figure 4.7: Soil Sample with LVDT Setup

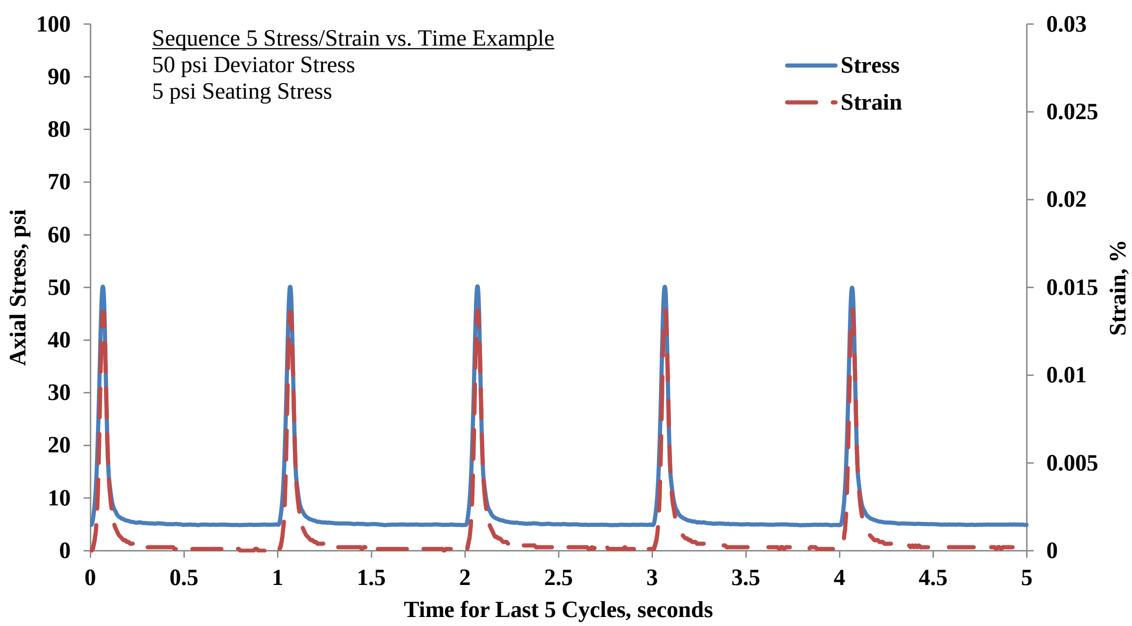


Figure 4.8: Stress-Strain vs. Time for Last Five Sequence of a typical Resilient Modulus Test

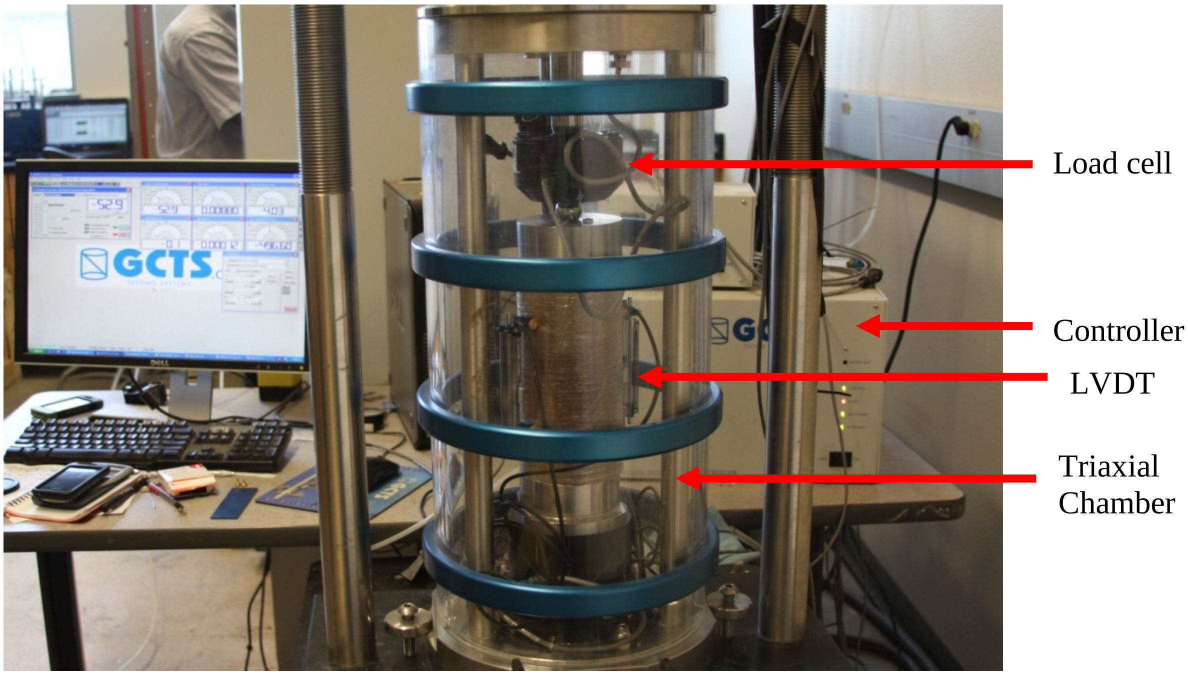


Figure 4.9: Subgrade Resilient Modulus Test Setup in Triaxial Chamber

a) Untreated A-6 soils b) Lime treated A-6 soil

Figure 4.10: Soaked Soil Sample

Table 4.1: Sample Size and Compaction Method According to Resilient Modulus Test Standards

Test Standard	Reconstituted Mr specimen diameter (in)		
	2.8	4.0	6.0
AASHTO T307	Static, Kneading, Vibratory	Static, Kneading, Vibratory	Static, Kneading, Vibratory
NCHRP 1-28A	N.A.	Impact, Kneading, Vibratory	Impact, Kneading, Vibratory

Table 4.2: Equivalent Blows per Lift Using Impact Compaction

Specimen Diameter (in)	2.8	4	6
Compactive Effort, CE (ft-lbf/cft)	56000		
Volume of specimen, V (cft)	0.02	0.058	0.196
No of Lifts, N	3	8	27
Weight of drop hammer, W (lbf)	10		
Drop height, h(ft)	1.5		
No of blows/lift, n	~25	~27	~27

Table 4.3: Test Sequences Adopted for Different Soils of This Study

Sequence Number	Untreated Subgrade soil		Granular Base soil		Lime-treated Subgrade soil	
	Confining pressure (psi)	Deviator Stress (psi)	Confining pressure (psi)	Deviator Stress (psi)	Confining pressure (psi)	Deviator Stress (psi)
Conditioning	6	4	15	15	0	8
1	6	2	3	3	0	10
2	6	4	3	6	0	15
3	6	6	3	9	0	20
4	6	8	5	5	0	25
5	6	10	5	10	0	30
6	4	2	5	15	0	40
7	4	4	10	10	0	50
8	4	6	10	20		
9	4	8	10	30		
10	4	10	15	10		
11	2	2	15	15		
12	2	4	15	30		
13	2	6	20	15		
14	2	8	20	20		
15	2	10	20	40		

Table 4.4: Test Matrix

Type of soil	% of additive used	Moisture State			Total samples
		Dry	Optimum	Wet	
Subgrade A-6	0	3	3	3	9
	5	2	6	3	11
	7	2	6	3	11
Subgrade A-7-6	0	2	2	2	6
	5	2	5	2	9
	7	2	5	2	9
Base	0	0	3	0	3

Chapter 5 RESULTS AND DISCUSSION

5.1. General

This chapter is devoted to results and discussions of the resilient modulus and unconfined compressive strength values of untreated and lime treated subgrade soils and granular base soils. Additionally, statistical regression analysis is performed to fit available soil constitutive equations for MEPDG applications.

5.2. Resilient Modulus Test Results

Average resilient modulus test results of replicate samples for different soil combinations are shown in Table 5.1 to Table 5.5. It is to be mentioned that, generating consistent resilient modulus results is very difficult even among replicate samples because of potential variations introduced when reconstituting soils. However, average results are used to observe the effects of associated parameters on stiffness or strength of particular soils. All replicated sample results are listed in the appendix.

5.2.1. Untreated Subgrade Soils

Resilient modulus test results using AASHTO T 307 load sequences in confined and unconfined state are given in Table 5.1. M_R values for untreated subgrade soils show consistent results for all cell pressures. M_R values decrease with the increase of deviator stress and moisture content. It is clearly seen that A-6 subgrade soil has higher modulus values than A-7-6 soil in all moisture state (Figure 5.1). Untreated A-6 and A-7-6 subgrade soil have M_R values in a range of 634-999 MPa (92-145 ksi) and 517-730 MPa (75-106 ksi), respectively, at OMC. However, typical MEPDG level-3 resilient modulus value at optimum moisture state for A-6 and A-7-6 soils are reported to be 93-165 MPa (13.5-24 ksi) and 34.5-93 MPa (5-13.5 ksi) respectively. It is speculated that as past researches mostly conducted M_R test using external LVDTs, their measured deformation were high and therefore they obtained very small M_R values. Thus high M_R values obtain from this study is reasonable (Barksdale (1997), Andrei (2003)). In addition, M_R values not always increase with the confining pressure for same deviator stress. Detail effects of confining pressure on resilient modulus values are discussed later in this chapter.

5.2.2. Lime-Treated Subgrade Soils

For this study, resilient modulus test was conducted with unconfined high stress sequence. It is established that resilient modulus values increased due to lime stabilization (Little (2000) and Solanki et al. (2009)). Table 5.1 to Table 5.3 support the mentioned statement but the increment varies with soil types and lime doses. A-6 subgrade soil becomes stiffer in all lime percentages for this study, but reduction in resilient modulus is observed for A-7-6 soil at 7% lime dose. This result justifies other studies (Osinubi and Nwaiwu, 2006; Solanki et al. 2009). Possible explanation for that is excess lime behaved as low strength filler and effectively weakened the lime-soil mixture

(Osinubi and Nwaiwu, 2006). In general, resilient modulus values increase with the decrease of molding moisture content. Only dry moisture state of lime stabilized A-6 soil behaved like granular material, i.e. increase of resilient modulus value with deviator stress increment. However, wet and optimum moisture state of this soil and all moisture states of lime stabilized A-7-6 soils show opposite trend same as untreated subgrade soils. Effects of sample size on resilient modulus test results are focused later.

5.2.3. Granular Base Soils

Figure 5.2 represents the resilient modulus value with deviator stress for different confining pressure for granular base material. It is clear from Table 5.5 and Figure 5.2 that resilient modulus values increase with the increment of cell pressure and deviator stress for 50%-50% RAP-granular base materials.

5.3. Unconfined Compression Test Results

Table 5.6 shows unconfined compressive strength test results conducted on untreated and lime-treated subgrade soils using universal test program. Following similar trend of resilient modulus result, UCS value increases due to lime addition except 7% lime dose for A-7-6 soil. Moreover, UCS increases with the decrement of moisture content. Figure 5.3 illustrates the typical failure pattern for untreated and lime treated subgrade soils.

5.4. Discussion

Effects of confining pressure, sample size, lime addition, molding moisture, soaking on test results are addressed in this segment.

5.4.1. Effects of Confining Pressure

Effects of confining pressure on resilient modulus values are reported for untreated raw soil. Unconfined test was done with same deviator stress sequences prior testing with AASHTO T 307 sequences. Resilient modulus values for same deviator stresses are compared to one another as percent difference between 2 to 6 psi, 0 to 4 psi and 0 to 6 psi confining pressure relative to lowest confining pressure M_R values. Comparison is done for both A-6 and A-7-6 soils and demonstrated in Table 5.7. A sample equation to compare the resilient modulus values corresponding to 6 psi with respect to 0 psi confining pressure is shown below:

$$\%\ difference_{0\ to\ 6\ psi} = \frac{Mr\ (0\ psi) - Mr\ (6\ psi)}{Mr\ (0\ psi)} \times 100 \tag{5.1}$$

Table 5.1 shows the differences in resilient modulus values due to change in confining pressure is small. It supports studies that cohesive fine-grained samples are not significantly affected by small confining pressures. Table 5.7 has the evidence that average difference in resilient modulus values relative to low confining pressure shows less than 10% changes in all cases. Thus it can be concluded that confining pressure has small effect on resilient modulus values of subgrade soils.

5.4.2. Effects of Sample Size

To compare the effects of available sample size on resilient modulus results, it was decided to reconstitute lime stabilized optimum moisture state sample having 2.8 inch diameter and observe the difference in resilient modulus results with 4 inch diameter sample. Table 5.4 shows average M_R test results are close for different sample sizes at different soil combinations. However, Table 5.8 demonstrates the difficulties to obtain consistent test results with 2.8 inch diameter sample. Coefficient of variance (COV) is calculated among replicate samples for each deviator stress cases. COV is a normalized measure of dispersion of a probability distribution and calculated as the ratio of standard deviation to the mean. 2.8 inch diameter sample has maximum COV of 13.4 which is always much higher than 4 inch diameter sample. Probable reason for this is side-wall restraint effect of 2.8 inch sample due to insufficient surface area for modified proctor hammering (Cabrera (2012)). 4 inch diameter sample can overcome this limitation.

5.4.3. Effects of Lime Content

It is obvious that resilient modulus and UCS value increases due to lime stabilization, but this increment varies with soil type and lime dose. For comparison purpose, 10 psi deviator stress with zero confining pressure is chosen as only this value is common for two different test sequences of untreated and lime treated soil. Figure 5.4 to Figure 5.6 depict the effect of lime addition on stiffness (resilient modulus) and strength (UCS) of subgrade soils. 5% lime addition increases M_R values approximately 600% and 1000% in A-6 and A-7-6 soils respectively. However, 7% lime dose decreases M_R value to 41% from 5% lime dose in A-7-6 soil. Similarly, 250% and 352% increment of UCS value are seen for 5% and 7% lime dose, respectively, in A-6 soil. Again, 7% lime addition in A-7-6 soil causes 16% decrease in UCS value from that of 5% lime dose. Thus, it can be concluded that 5% lime is the design optimum lime content of A-7-6 soil, whereas for A-6 soil it may be 7% or more, which is not being able to determine exactly from this study.

5.4.4. Effects of Molding Moisture

Pavement subgrade soil faces seasonal variation of moisture. Furthermore, lime stabilized subgrade soil is considered as moisture-insensitive thus variation of resilient modulus and UCS of clayey subgrade soil is significantly minimized due to moisture changes. Therefore, this study examines the change in M_R or UCS value with respect to optimum moisture. Average water content among replicate samples in this study was maintained within ±2-3% of optimum water content (Table 5.1 to Table 5.3). Percent difference in M_R and UCS results due to different moisture state from optimum are calculated using Eq. (5.2) and (5.3).

$$\%\ difference\ of\ UCS\ in\ dry\ state = \frac{Dry\ UCS - Optimum\ UCS}{Optimum\ UCS} \times 100 \tag{5.2}$$

$$\% \, difference \, of \, UCS \, in \, wet \, state = \frac{Wet \, UCS - Optimum \, UCS}{Optimum \, UCS} \times 100 \qquad (5.3)$$

Figure 5.7 and Figure 5.8 demonstrate the percent differences of resilient modulus and UCS between dry and wet state results from optimum for all subgrade soil combination. For resilient modulus, value of 10 psi deviator stress sequence is compared. It is clear that moisture affects the M_R and UCS of lime treated soil, but influence of moisture minimizes in comparison to untreated subgrade soils. About 22% and 42% reduction in M_R value can be seen from Figure 5.7 for A-6 with 7% lime and A-7-6 with 5% lime respectively. Moreover, variation of M_R with moisture decreases drastically due to the addition of lime in A-6 soil than that in A-7-6 soils.

5.4.5. Effects of Soaking

Resilient modulus test was conducted on sample prior and post soaking in water. It is observed that resilient modulus value increases in most cases in lime-stabilized soils after 48 hours soaking. Table 5.9 shows the resilient modulus results before and after soaking. Paired two sample tests of means is done on these results at 95% level ($\alpha = 0.05$). Results show that there is no significant difference in modulus values due to soaking for all cases except A-6 soil with 7% lime. Increment of modulus value indicates that fully submerged condition works as an enhanced curing time for lime-stabilized soil. Thus lime stabilization is an excellent option for low-volume paved road with high ground water table.

5.5. Constitutive Modeling of Resilient Modulus

The constitutive model used in Level 1 inputs for resilient modulus in MEPDG is a generalized model of the normalized log-log proposed by NCHRP 1-28 shows in Eq. (5.4).

$$M_R = k_1 P_a \left(\frac{\theta}{P_a}\right)^{k_2} \left(\frac{\tau_{oct}}{P_a} + 1\right)^{k_3} \qquad (5.4)$$

For laboratory confined resilient modulus test:

M_R = resilient modulus

θ = bulk stress = $\sigma_1 + \sigma_2 + \sigma_3 = \sigma_d + 3\sigma_3$

σ_d = Deviator stress

σ_1 = major principal stress = $\sigma_d + \sigma_3$

σ_2 = intermediate principal stress = σ_3 for M_r test on cylindrical specimen

σ_3 = minor principal stress = confining pressure

τ_{oct} = octahedral shear stress

$= \frac{1}{3}\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_1 - \sigma_3)^2 + (\sigma_2 - \sigma_3)^2} = \frac{\sqrt{2}}{3}\sigma_d$

P_a = atmospheric pressure = 14.7 psi

k_1, k_2, k_3 = regression constants

For laboratory unconfined resilient modulus test: (σ_2, σ_3 = zero)

θ = bulk stress = σ_d

τ_{oct} = octahedral shear stress = $\frac{\sqrt{2}}{3}\ \sigma_d$

All other parameters are same as confined case.

Regression constant signs: k_1 and k_2 should always positive and k_3 negative. The constant k_1 is proportional to the resilient modulus. As this value is always positive, thus k_1 must be positive. Term k_2 should be positive based on the term it is affecting (bulk stress θ). An increase in bulk stress should induce material stiffening i.e. an increased modulus so k_2 is likely to be positive. Increase in the octahedral shear stress softens the material thus reduce the modulus, for this reason k_3 should be negative. This is true for subgrade soils for which resilient modulus decreases with the increase of deviator stress. Unbound granular materials (like RAP-Base soil) show opposite trend, thus regression coefficient signs can be altered.

In the current study, multiple linear regression analysis is performed to find k_1, k_2, k_3 values. Table 5.10 presents the regression constants for untreated and lime-treated subgrade and granular base soils. Results are evaluated based on multiple correlation coefficients (R^2) and percent of error among predicted and measured M_R values. MEPDG design guide recommends a R^2 value 0.9 or greater to ensure good predictability of constitutive models. It is seen that R^2 of 0.9 or greater for all subgrade soil combinations is obtained, thus ensure good predictability of this constitutive model. Regression constant k_2 is negative because bulk stress equals deviator stress for lime stabilized soils and M_R value decreases with an increment of deviator stress. Small percent error of predicted and laboratory measured M_R value is obtained for all soil. Thus stress depended MEPDG model for M_R shows good predictability for subgrade and granular base soil considered in this study.

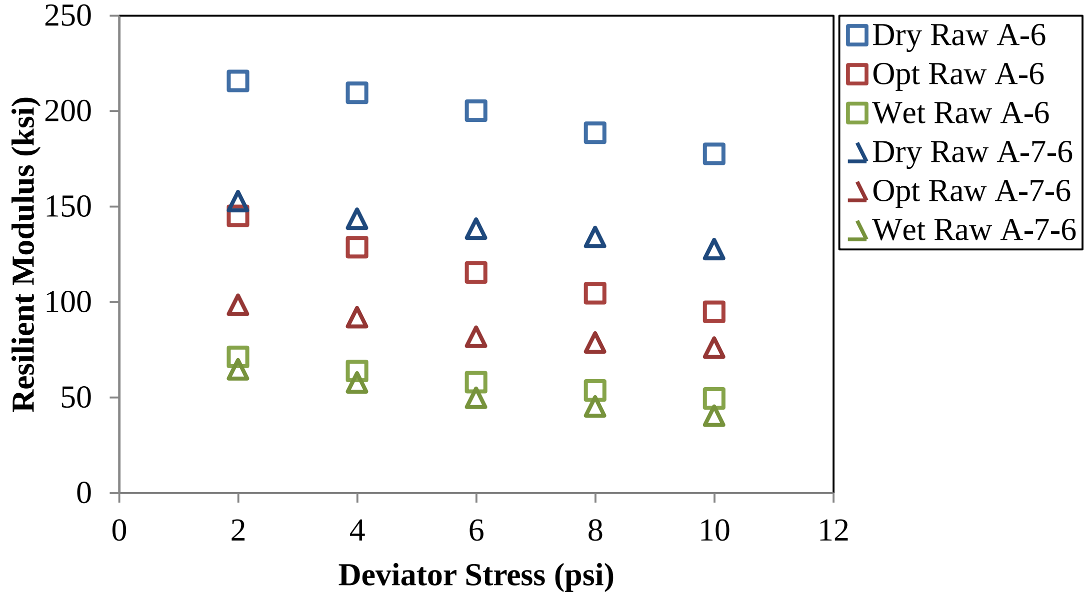


Figure 5.1: Resilient Modulus Plot with Deviator Stress for Untreated Subgrade in All Moisture State (Confining Pressure 4 Psi)

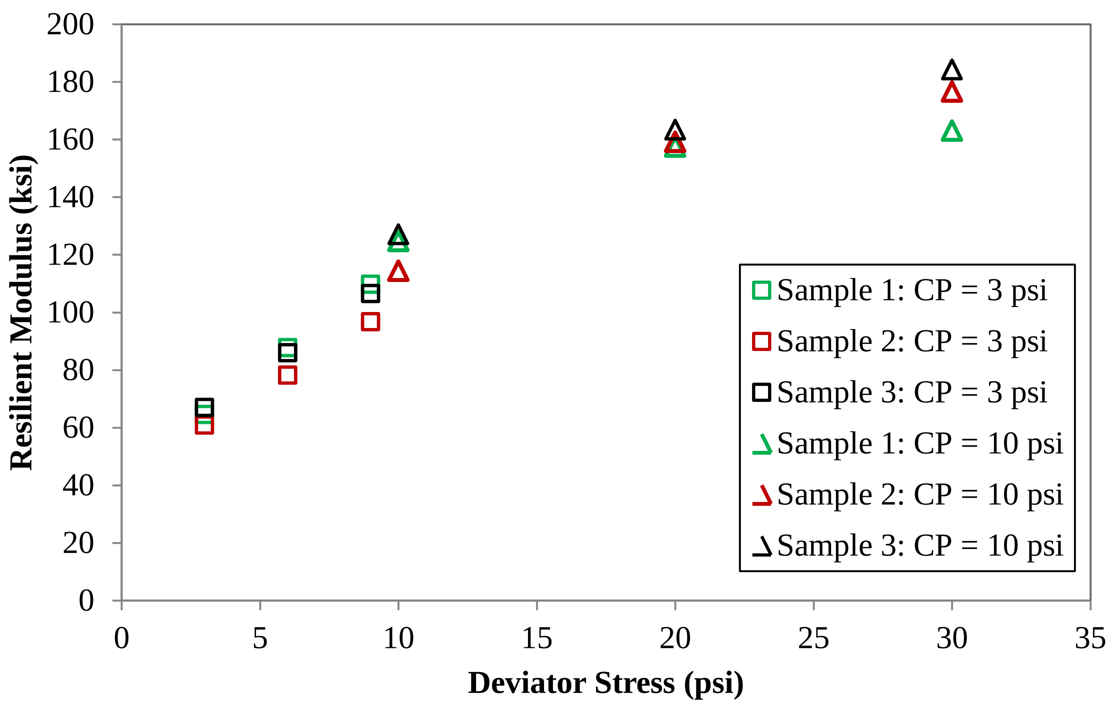


Figure 5.2: Resilient Modulus Values for RAP-Base Material for Different Cell Pressure

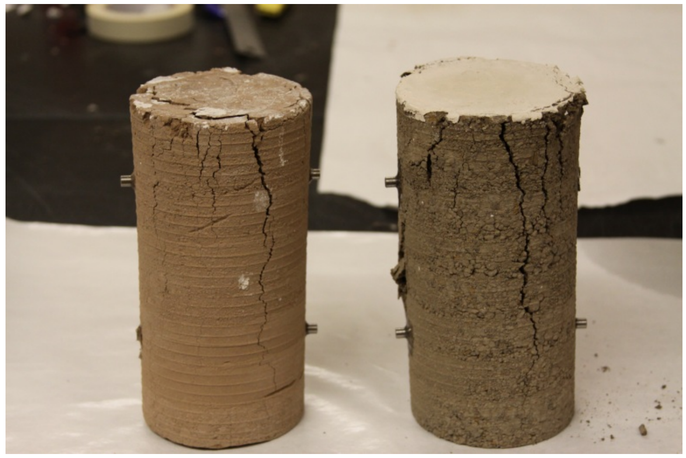


Figure 5.3: Lime Treated A-6 and A-7-6 Soil after Unconfined Compressive Strength Test

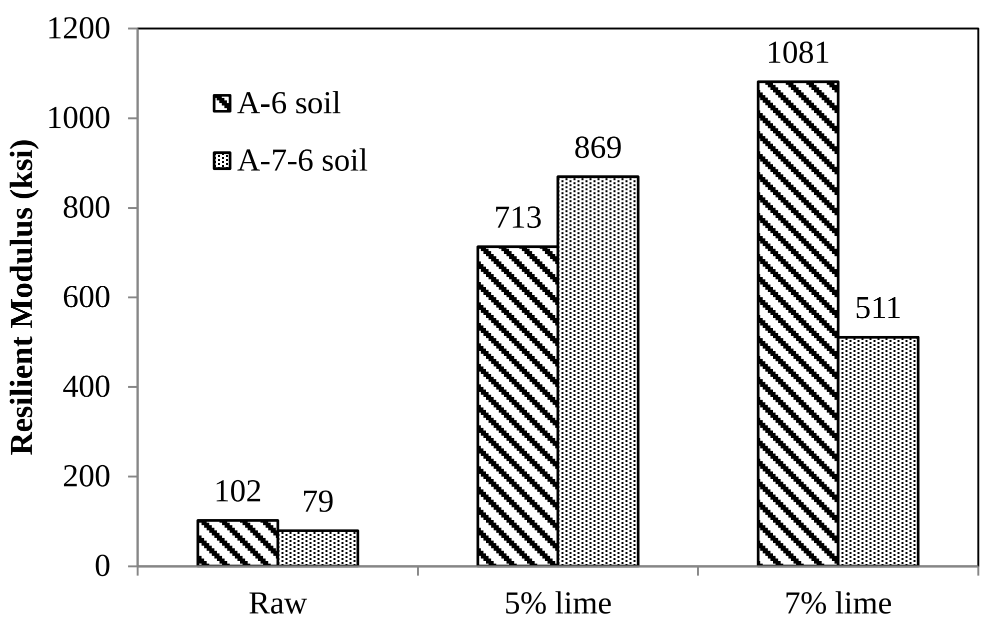


Figure 5.4: Variation of Resilient Modulus with Soil and Lime Dose (SD =10 Psi, CP = 0 Psi)

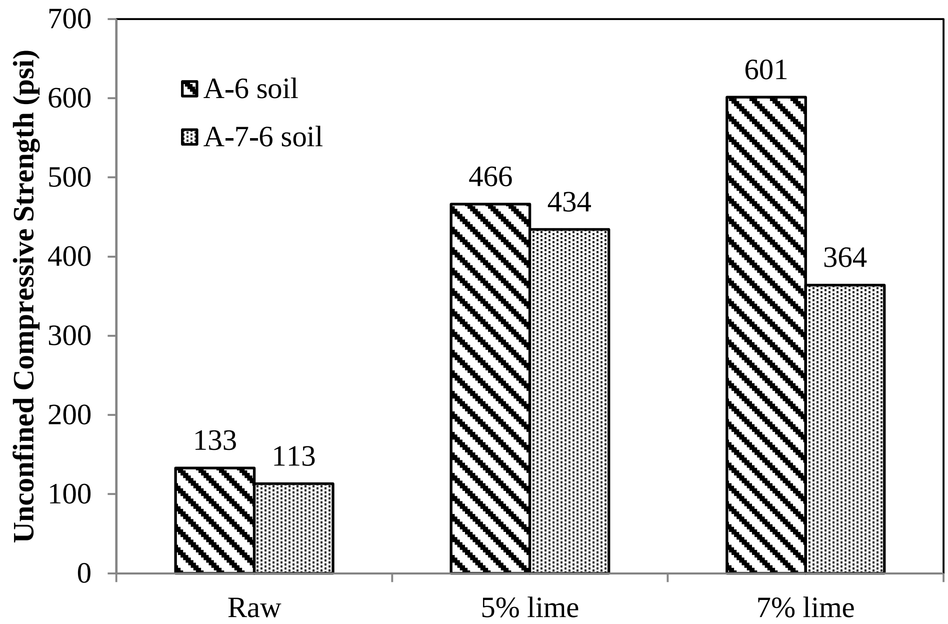


Figure 5.5: Variation of Unconfined Compressive Strength with Soil and Lime Dose

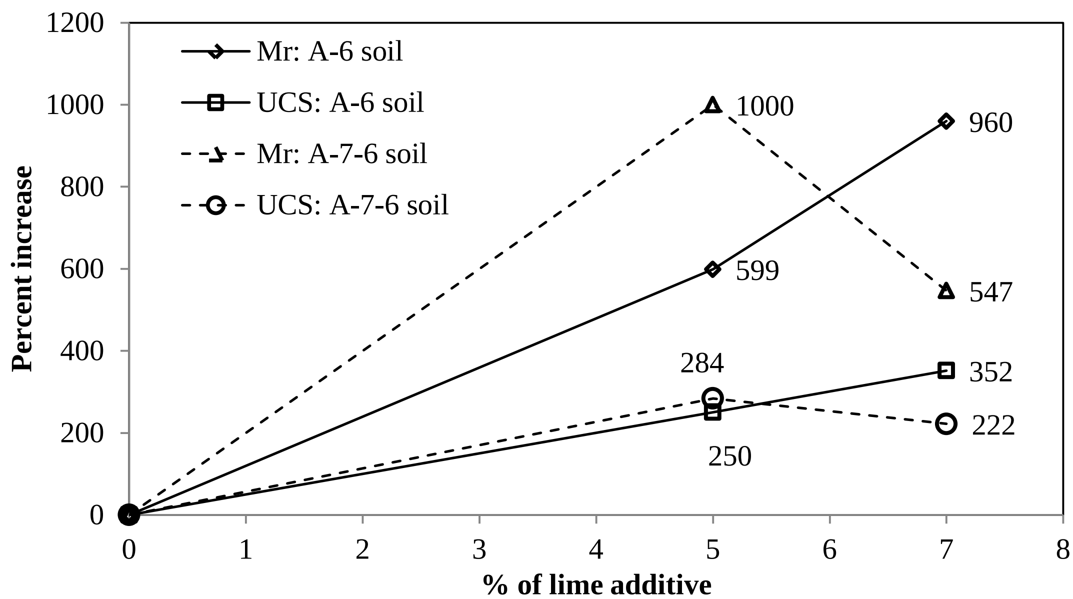


Figure 5.6: Percent Increment of M_R and UCS with Soil and Lime Dose

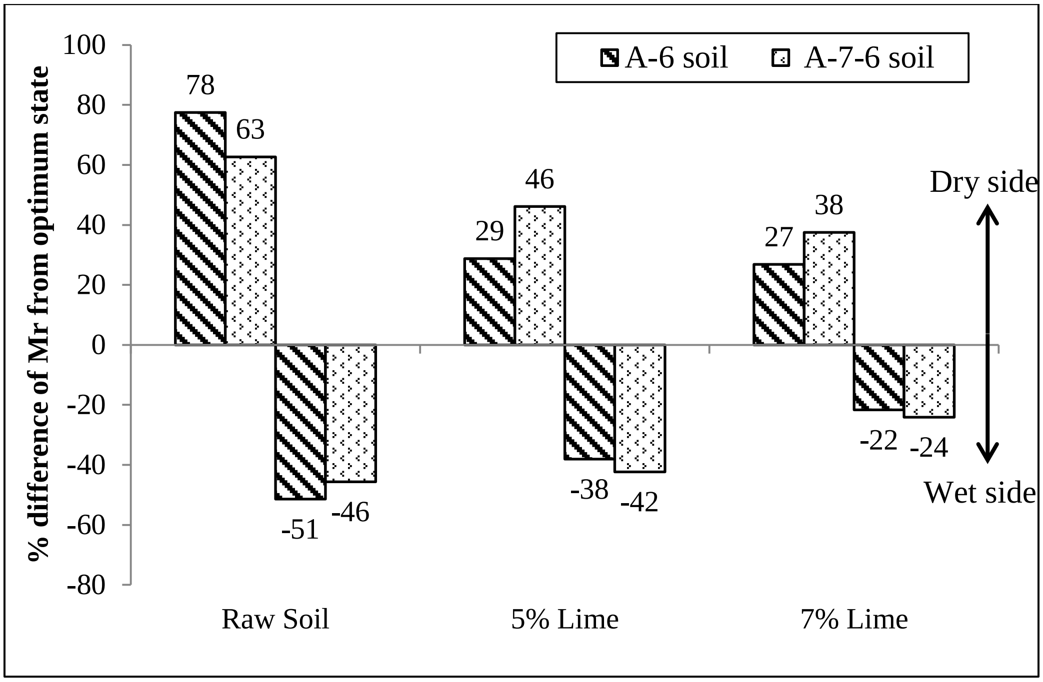


Figure 5.7: Percent Difference from Optimum Moisture in Resilient Modulus Results

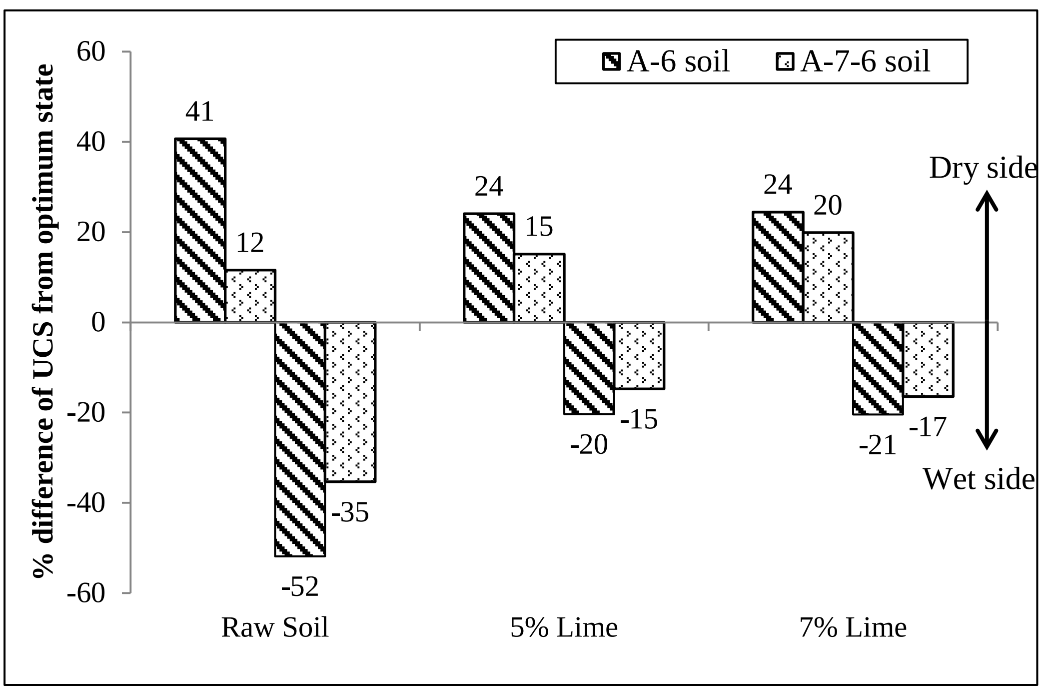


Figure 5.8: Percent Difference from Optimum Moisture in UCS Results

Table 5.1: Average Resilient Modulus Data for Untreated Subgrade Soil at All Moisture State

Cell Pressure (psi)	Deviator stress (psi)	Max Cyclic Stress (psi)	Raw A-6			Raw A-7-6		
			Dry	Opt	Wet	Dry	Opt	Wet
			w=13.5 %	w=15.5 %	w=17.7 %	w=13.0 %	w=15.1 %	w=17.3 %
			Mr (ksi)	Mr (ksi)	Mr (ksi)	Mr (ksi)	Mr (ksi)	Mr (ksi)
6	2	1.8	217	143	71	150	103	64
6	4	3.6	207	128	66	144	99	57
6	6	5.4	197	112	60	140	85	49
6	8	7.2	186	101	56	136	79	45
6	10	9	174	92	52	129	75	40
4	2	1.8	216	145	71	153	99	65
4	4	3.6	209	129	64	144	92	58
4	6	5.4	200	115	58	138	82	50
4	8	7.2	189	105	54	134	79	45
4	10	9	178	95	50	128	76	40
2	2	1.8	215	141	70	153	97	64
2	4	3.6	210	132	63	145	96	56
2	6	5.4	201	116	57	139	85	50
2	8	7.2	189	104	52	133	82	44
2	10	9	178	94	48	128	78	39
0	2	1.8	216	144	72	156	106	66
0	4	3.6	209	133	66	149	101	58
0	6	5.4	201	120	60	142	91	52
0	8	7.2	191	108	55	134	85	47
0	10	9	181	102	50	129	79	43

Table 5.2: Avg. Resilient Modulus for Lime-Treated A-6 Subgrade Soil at All Moisture State

Deviator Stress (psi)	Max Cyclic Stress (psi)	A-6 with 5% lime			A-6 with 7% lime		
		Dry	Opt	Wet	Dry	Opt	Wet
		w=15.6%	w=17.6%	w=19.7%	w=16.2%	w=18.5%	w=20.5%
		Mr (ksi)	Mr (ksi)	Mr (ksi)	Mr (ksi)	Mr (ksi)	Mr (ksi)
10	9	918	713	516	1370	1081	846
15	13.5	974	682	500	1392	1024	785
20	18	1012	638	478	1430	982	743
25	22.5	1023	603	448	1452	930	715
30	27	1077	564	432	1485	892	686
40	36	1102	525	409	1543	855	630
50	45	1189	488	385	1574	816	579

Table 5.3: Avg. Resilient Modulus for Lime-Treated A-7-6 Subgrade Soil at All Moisture State

Deviator Stress (psi)	Max Cyclic Stress (psi)	A-7-6 with 5% lime			A-7-6 with 7% lime		
		Dry	Opt	Wet	Dry	Opt	Wet
		w=14.6%	w=16.4%	w=19.5%	w=16.2%	w=18.5%	w=20.5%
		Mr (ksi)	Mr (ksi)	Mr (ksi)	Mr (ksi)	Mr (ksi)	Mr (ksi)
10	9	1270	869	501	702	511	387
15	13.5	1262	824	468	657	490	375
20	18	1248	785	420	635	477	361
25	22.5	1235	727	373	621	464	350
30	27	1230	675	336	616	447	344
40	36	1205	645	289	596	427	333
50	45	1179	593	261	587	401	321

Table 5.4: Avg. Resilient Modulus for Lime-Treated Subgrade Soil for Two Sample Sizes

Deviator Stress (psi)	A-6 with 5% lime		A-6 with 7% lime		A-7-6 with 5% lime		A-7-6 with 7% lime	
	Optimum state		Optimum state		Optimum state		Optimum state	
	2.8" x 5.6"	4" x 8"	2.8" x 5.6"	4" x 8"	2.8" x 5.6"	4" x 8"	2.8" x 5.6"	4" x 8"
	Mr (ksi)	Mr (ksi)	Mr (ksi)	Mr (ksi)	Mr (ksi)	Mr (ksi)	Mr (ksi)	Mr (ksi)
10	715	713	1122	1092	853	869	475	511
15	685	666	1088	1016	790	824	435	490
20	653	611	1057	966	726	785	392	477
25	629	564	1014	905	700	727	372	464
30	605	514	978	870	675	675	353	447
40	560	479	923	841	622	645	333	427
50	485	448	881	802	565	593	297	401

Table 5.5: RAP-Base Resilient Modulus Test Results According to AASHTO T 307

Cell Pressure (psi)	Deviator stress (psi)	Max Cyclic Stress (psi)	Sample Size: 6" x 12"			
			Sample 1 (w= 6.9%)	Sample 2 (w=6.8%)	Sample 3 (w=6.8%)	Average M_R (ksi)
			M_R (ksi)	M_R (ksi)	M_R (ksi)	
3	3	2.7	64.5	60.9	67.0	64.2
3	6	5.4	87.8	78.2	86.1	84.0
3	9	8.1	109.8	96.8	106.5	104.4
5	5	4.5	88.3	76.2	81.8	82.1
5	10	9	120.5	114.3	119.7	118.1
5	15	13.5	140.4	137.2	145.2	141.0
10	10	9	124.8	114.4	127.1	122.1
10	20	18	157.6	159.2	163.4	160.1
10	30	27	163.1	176.7	184.3	174.7
15	10	9	117.7	107.7	111.5	112.3
15	15	13.5	131.5	124.7	133.2	129.8
15	30	27	164.5	182.8	189.0	178.7
20	15	13.5	130.5	130.6	131.7	130.9
20	20	18	146.0	146.2	150.8	147.7
20	40	36	173.1	187.7	191.4	184.1

Table 5.6: Average Unconfined Compressive Strength Results for Subgrade Soils

Type of soil	% of lime additive	Average UCS (psi)		
		Dry	Opt	Wet
A-6	0	187	133	64
	5	578	466	371
	7	692	601	512
A-7-6	0	126	113	73
	5	540	434	345
	7	436	364	304

Table 5.7: Effects of Confining Pressure on Resilient Modulus Test Results of Subgrade Soil

Deviator stress (psi)	Raw A-6 soil			Raw A-7-6 soil		
	% Difference in Mr			% Difference in Mr		
	2 to 6 psi	0 to 4 psi	0 to 6 psi	2 to 6 psi	0 to 4 psi	0 to 6 psi
2	-1.61	-0.76	0.74	-5.40	7.03	3.30
4	2.97	3.04	3.77	-3.61	8.93	1.98
6	3.20	4.02	6.93	-0.24	9.56	5.80
8	2.70	3.54	6.77	3.02	6.86	6.06
10	1.32	6.96	9.44	3.73	3.80	5.38
Average	1.71	3.36	5.53	-0.50	7.24	4.50

Table 5.8: Sample Size Comparison Using Resilient Modulus Results

Deviator Stress (psi)	A-6 with 5% lime		A-6 with 7% lime		A-7-6 with 5% lime		A-7-6 with 7% lime	
	Optimum state		Optimum state		Optimum state		Optimum state	
	2.8"x5.6"	4" x 8"	2.8"x5.6"	4" x 8"	2.8"x5.6"	4" x 8"	2.8"x5.6"	4" x 8"
	COV (%)	COV (%)	COV (%)	COV (%)	COV (%)	COV (%)	COV (%)	COV (%)
10	10.07	1.36	0.80	1.76	10.18	4.44	17.88	2.93
15	8.19	2.45	2.20	2.25	8.22	4.43	14.38	1.31
20	8.07	4.60	5.08	2.28	6.61	4.54	14.24	1.52
25	7.03	7.04	6.32	5.19	8.81	5.43	12.53	1.15
30	6.35	9.88	7.59	6.59	9.87	4.08	12.31	2.00
40	3.57	10.15	10.55	3.50	8.75	3.32	11.93	2.31
50	8.42	9.61	9.75	2.55	7.27	3.04	10.45	4.91
Average	7.39	6.44	6.04	3.45	8.53	4.18	13.39	2.30

Table 5.9: Effect of Soaking On Resilient Modulus Results Using Statistical Analysis

Deviator stress (psi)	A-7-6, 5% Lime		A-7-6, 7% Lime		A-6, 5% Lime		A-6, 7% Lime	
	Mr (ksi)	Mr after soaking (ksi)	Mr (ksi)	Mr after soaking (ksi)	Mr (ksi)	Mr after soaking (ksi)	Mr (ksi)	Mr after soaking (ksi)
10	858	876	500	521	721	734	1116	1157
15	811	817	494	493	692	673	1066	1090
20	798	796	482	484	645	644	1025	1050
25	701	722	468	480	616	618	980	1017
30	643	647	441	446	587	573	927	1004
40	629	622	421	418	538	543	883	972
50	582	579	387	398	494	490	840	929
p value	0.2416		0.0729		0.5430		0.0027	
Statistical Significance	No		No		No		Yes	

Table 5.10: Statistical Summary of MEPDG Constitutive Model

Type of soil	% of Lime	Model: $M_R = k_1 * P_a * (\theta/P_a)^{k2} * (\tau_{oct}/P_a + 1)^{k3}$			R^2	M_R (ksi) @ σ_d = 10 psi		% error
		k_1	k_2	k_3		Predicted	Measured	
Subgrade A-6	0	10814.0	-0.0253	-1.8131	0.9805	96.9	102	-5.00
	5	52110.3	-0.1358	-0.4005	0.9884	722.1	712.7	1.32
	7	67530.4	-0.2187	0.0556	0.9925	1096.8	1091.6	0.48
Subgrade A-7-6	0	7339.9	-0.0346	-1.2188	0.9299	77.9	79	-1.39
	5	69092.9	-0.0113	-0.5507	0.987	875.2	869.1	0.70
	7	40388.8	0.0548	-0.475	0.996	509.4	511	-0.31
Base soil	0	5006.0	0.1387	0.9816	0.8876	104.1*	118.1*	-11.85

*deviator stress of 10 psi and confining pressure of 5 psi is considered for base soil

Chapter 6 CONCLUSIONS

Resilient modulus is a complex and non-trivial laboratory test. The current study attempts to conduct the resilient modulus testing of only few soils and aggregates in New Mexico to develop appropriate test methods and measurement techniques and gain laboratory experience. From the laboratory testing and analysis of M_R value, the following conclusions can be made:

1. Modified proctor compaction used for preparing resilient modulus specimen is found to be suitable and consistent method for subgrade soils. Study on granular base soils is limited, thus other appropriate method may be checked in a future study.
2. Moisture-density relationship indicates that optimum moisture content increases and maximum dry density decreases with the addition of lime in clayey subgrade soil.
3. For the accurate estimation of strain during M_R testing, deformation transducers (LVDTs) should be attached to the specimen, not outside the specimen chamber.
4. Resilient modulus and unconfined compressive strength values increase due to lime stabilization depending on soil type and lime dose.
5. Though smaller M_R specimen (2.8 inch diameter) requires small amount of material, time and effort; insufficient surface area during modified proctor hammering for 2.8 inch sample triggers the inconsistency in M_R test results. This study concludes the recommended sample size for subgrade soils is 4 inch diameter by 8 inch height for resilient modulus testing.
6. As confining pressure has little effects on resilient modulus values of lime-treated soils, unconfined test mode can be used to have flexibility.
7. The AASHTO T 307 standard protocol of resilient modulus testing has stress sequences for untreated soils only. Higher stress test sequences are suggested in this study as an option to eliminate errors of deformation transducers and obtain accurate estimation of resilient modulus value of lime treated soils.
8. M_R and UCS values increase with the decrease in molding moisture and deviator stress for untreated and lime treated subgrade soils. An opposite trend in the relation between the resilient modulus and deviator stress is observed for granular base soils.
9. According to the pH test results, minimum lime content for A-6 and A-7-6 subgrade soils are 5% and 4%, respectively. However, design lime content for these soils were 7% (or higher) and 5% respectively according to construction data.
10. Lime addition can reduce the moisture sensitivity of resilient modulus and UCS value of untreated clayey subgrade soils.
11. Resilient modulus value of lime stabilized soil is not altered due to submergence in water. Thus lime stabilization is a viable option for pavements on high ground water table locations.

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APPENDIX

A.1. Test Results for Untreated A-6 Subgrade Soils:

Cell Pressure (psi)	Deviator stress (psi)	Dry state				Optimum state				Wet state			
		Sample 1	Sample 2	Sample 3	Average Mr (ksi)	Sample 1	Sample 2	Sample 3	Average Mr (ksi)	Sample 1	Sample 2	Sample 3	Average Mr (ksi)
		Mr (ksi)	Mr (ksi)	Mr (ksi)		Mr (ksi)	Mr (ksi)	Mr (ksi)		Mr (ksi)	Mr (ksi)	Mr (ksi)	
6	2	208	216	227	217	150	134	145	143	84	61	68	71
6	4	190	211	219	207	132	129	122	128	80	56	61	66
6	6	177	200	213	197	113	114	109	112	75	51	55	60
6	8	169	195	194	186	104	101	98	101	71	46	50	56
6	10	161	189	173	174	97	91	89	92	67	44	46	52
4	2	221	203	223	216	152	133	150	145	84	61	69	71
4	4	205	213	210	209	134	124	128	129	79	53	59	64
4	6	186	211	203	200	114	120	112	115	75	48	51	58
4	8	177	200	189	189	106	109	99	105	71	45	46	54
4	10	166	191	175	178	100	94	91	95	67	42	39	50
2	2	218	204	222	215	143	130	149	141	83	59	66	70
2	4	203	213	212	210	140	129	126	132	79	53	58	63
2	6	192	208	203	201	119	119	109	116	75	47	50	57
2	8	180	201	187	189	109	104	99	104	71	44	43	52
2	10	172	192	170	178	102	89	90	94	67	41	37	48
0	2	219	208	219	216	148	133	151	144	86	61	69	72
0	4	214	203	211	209	143	123	132	133	83	57	60	66
0	6	209	195	199	201	130	120	111	120	78	50	53	60
0	8	200	186	186	191	123	105	97	108	73	43	47	55
0	10	193	177	173	181	121	93	92	102	69	39	41	50
UCS (psi)		198	184	178	187	135	144	121	133	69	59	64	64
Post UCS MC (%)		13.7	13.3	13.5	13.5	15.7	15.6	15.2	15.5	17.4	17.8	18	17.7

A.2. Test Results for Untreated A-7-6 Subgrade Soils:

Cell Pressure (psi)	Deviator stress (psi)	Max Cyclic Stress (psi)	Dry state			Optimum state			Wet state		
			Sample 1	Sample 2	Average Mr (ksi)	Sample 1	Sample 2	Average Mr (ksi)	Sample 1	Sample 2	Average Mr (ksi)
			Mr (ksi)	Mr (ksi)		Mr (ksi)	Mr (ksi)		Mr (ksi)	Mr (ksi)	
6	2	1.8	156	143	150	98	107	103	59	70	64
6	4	3.6	152	137	144	95	103	99	53	61	57
6	6	5.4	146	134	140	80	91	85	48	51	49
6	8	7.2	141	130	136	76	83	79	43	46	45
6	10	9	136	122	129	74	76	75	40	40	40
4	2	1.8	159	147	153	93	104	99	59	71	65
4	4	3.6	150	137	144	88	96	92	52	64	58
4	6	5.4	144	133	138	80	84	82	47	53	50
4	8	7.2	139	129	134	77	80	79	43	48	45
4	10	9	136	119	128	75	77	76	40	41	40
2	2	1.8	160	145	153	92	103	97	59	69	64
2	4	3.6	150	139	145	91	100	96	52	61	56
2	6	5.4	144	134	139	81	89	85	47	52	50
2	8	7.2	140	127	133	79	85	82	43	44	44
2	10	9	137	120	128	76	79	78	40	38	39
0	2	1.8	163	150	156	105	107	106	61	70	66
0	4	3.6	156	142	149	98	104	101	55	62	58
0	6	5.4	150	133	142	90	91	91	50	54	52
0	8	7.2	144	124	134	83	86	85	46	47	47
0	10	9	139	118	129	78	80	79	43	43	43
UCS (psi)			124	128	126	109	117	113	69	76	73
Post UCS MC (%)			12.7	13.2	13	15	15.2	15.1	17.5	17.1	17.3

A.3. Test Results for A-6 Soil with 5% Lime:

Test on different sample size at optimum moisture content:

Deviator Stress (psi)	Max Cyclic Stress (psi)	Sample Size: 2.8" x 5.6"				Sample Size: 4" x 8"			
		Sample 1	Sample 2	Sample 3	Average Mr (ksi)	Sample 1	Sample 2	Sample 3	Average Mr (ksi)
		Mr (ksi)	Mr (ksi)	Mr (ksi)		Mr (ksi)	Mr (ksi)	Mr (ksi)	
10	9	714	787	643	715	721	704	721	713
15	13.5	681	743	631	685	659	673	692	666
20	18	640	711	608	653	591	631	645	611
25	22.5	617	678	592	629	538	591	616	564
30	27	584	649	581	605	486	541	587	514
40	36	540	580	561	560	444	513	538	479
50	45	438	512	505	485	414	482	494	448
UCS (psi)		382	372	421	392	446	448	483	459

Test on different moisture state on 4 inch sample:

Deviator Stress (psi)	Dry state			Optimum state			Wet state		
	Sample 1	Sample 2	Average Mr (ksi)	Sample 1	Sample 2	Average Mr (ksi)	Sample 1	Sample 2	Average Mr (ksi)
	Mr (ksi)	Mr (ksi)		Mr (ksi)	Mr (ksi)		Mr (ksi)	Mr (ksi)	
10	905	932	918	704	721	713	534	499	516
15	949	998	974	673	692	682	519	481	500
20	994	1031	1012	631	645	638	494	462	478
25	1031	1014	1023	591	616	603	467	429	448
30	1061	1094	1077	541	587	564	452	411	432
40	1116	1087	1102	513	538	525	437	380	409
50	1159	1220	1189	482	494	488	414	357	385
UCS (psi)	557	598	578	448	483	466	404	338	371
MC (%)	15.7	15.4	15.6	17.5	17.6	17.6	20	19.4	19.7

A.4. Test Results for A-6 Soil with 7% Lime:

Test on different sample size at optimum moisture content:

Deviator Stress (psi)	Max Cyclic Stress (psi)	Sample Size: 2.8" x 5.6"				Sample Size: 4" x 8"			
		Sample 1	Sample 2	Sample 3	Average Mr (ksi)	Sample 1	Sample 2	Sample 3	Average Mr (ksi)
		Mr (ksi)	Mr (ksi)	Mr (ksi)		Mr (ksi)	Mr (ksi)	Mr (ksi)	
10	9	990	1182	1193	1122	1138	1045	1116	1092
15	13.5	985	1133	1146	1088	1051	981	1066	1016
20	18	977	1091	1104	1057	994	939	1025	966
25	22.5	912	1054	1077	1014	929	881	980	905
30	27	867	1025	1042	978	883	857	927	870
40	36	830	960	978	923	855	827	883	841
50	45	813	891	940	881	812	792	840	802
UCS (psi)		571	648	645	621	717	583	618	639

Test on different moisture state on 4 inch sample:

Deviator Stress (psi)	Dry state			Optimum state			Wet state		
	Sample 1	Sample 2	Average Mr (ksi)	Sample 1	Sample 2	Average Mr (ksi)	Sample 1	Sample 2	Average Mr (ksi)
	Mr (ksi)	Mr (ksi)		Mr (ksi)	Mr (ksi)		Mr (ksi)	Mr (ksi)	
10	1329	1410	1370	1045	1116	1081	869	824	846
15	1342	1442	1392	981	1066	1024	798	773	785
20	1375	1485	1430	939	1025	982	754	732	743
25	1401	1504	1452	881	980	930	745	685	715
30	1425	1545	1485	857	927	892	710	662	686
40	1476	1609	1543	827	883	855	649	610	630
50	1507	1642	1574	792	840	816	584	575	579
UCS (psi)	698	686	692	618	583	601	541	482	512
Post UCS MC (%)	16	16.4	16.2	18.5	18.5	18.5	20.6	20.3	20.5

A.5. Test Results for A-7-6 Soil with 5% Lime:

Test on different sample size at optimum moisture content:

Deviator Stress (psi)	Max Cyclic Stress (psi)	Sample Size: 2.8" x 5.6"				Sample Size: 4" x 8"		
		Sample 1	Sample 2	Sample 3	Average Mr (ksi)	Sample 1	Sample 2	Average Mr (ksi)
		Mr (ksi)	Mr (ksi)	Mr (ksi)		Mr (ksi)	Mr (ksi)	
10	9	848	861	851	853	880	858	869
15	13.5	786	775	809	790	837	811	824
20	18	699	711	768	726	773	798	785
25	22.5	667	682	750	700	754	701	727
30	27	642	649	734	675	706	643	675
40	36	589	580	698	622	661	629	645
50	45	561	512	622	565	604	582	593
UCS (psi)		402	361	421	395	446	421	434

Test on different moisture state on 4 inch sample:

Deviator Stress (psi)	Dry state			Optimum state			Wet state		
	Sample 1	Sample 2	Average Mr (ksi)	Sample 1	Sample 2	Average Mr (ksi)	Sample 1	Sample 2	Average Mr (ksi)
	Mr (ksi)	Mr (ksi)		Mr (ksi)	Mr (ksi)		Mr (ksi)	Mr (ksi)	
10	1227	1313	1270	880	858	869	509	494	501
15	1214	1310	1262	837	811	824	467	468	468
20	1196	1299	1248	773	798	785	418	422	420
25	1182	1289	1235	754	701	727	375	371	373
30	1173	1288	1230	706	643	675	341	332	336
40	1148	1263	1205	661	629	645	293	284	289
50	1121	1236	1179	604	582	593	267	255	261
UCS (psi)	531	549	540	446	421	434	351	338	345
Post UCS MC (%)	14.2	14.8	14.6	16.3	16.5	16.4	19.8	19.2	19.5

A.6. Test Results for A-7-6 Soil with 7% Lime:

Test on different sample size at optimum moisture content:

Deviator Stress (psi)	Max Cyclic Stress (psi)	Sample Size: 2.8" x 5.6"				Sample Size: 4" x 8"		
		Sample 1	Sample 2	Sample 3	Average Mr (ksi)	Sample 1	Sample 2	Average Mr (ksi)
		Mr (ksi)	Mr (ksi)	Mr (ksi)		Mr (ksi)	Mr (ksi)	
10	9	457	568	401	475	521	500	511
15	13.5	419	504	382	435	485	494	490
20	18	381	453	343	392	472	482	477
25	22.5	368	421	328	372	460	468	464
30	27	353	397	310	353	453	441	447
40	36	337	370	291	333	434	421	427
50	45	284	332	274	297	415	387	401
UCS (psi)		389	294	367	350	374	353	364

Test on different moisture state on 4 inch sample:

Deviator Stress (psi)	Max Cyclic Stress (psi)	Dry state			Optimum state			Wet state		
		Sample 1	Sample 2	Average Mr (ksi)	Sample 1	Sample 2	Average Mr (ksi)	Sample 1	Sample 2	Average Mr (ksi)
		Mr (ksi)	Mr (ksi)		Mr (ksi)	Mr (ksi)		Mr (ksi)	Mr (ksi)	
10	9	684	720	702	521	500	511	378	397	387
15	13.5	640	675	657	485	494	490	366	383	375
20	18	621	649	635	472	482	477	346	375	361
25	22.5	608	634	621	460	468	464	340	360	350
30	27	603	628	616	453	441	447	333	355	344
40	36	589	602	596	434	421	427	319	347	333
50	45	587	588	587	415	387	401	307	335	321
UCS (psi)		419	452	436	374	353	364	292	315	304
Post UCS MC (%)		15	15.5	15.3	17.7	17.6	17.7	20.1	19.5	19.8