MEASUREMENT AND EVALUATION OF SUBGRADE SOIL PARAMETERS: *PHASE I - SYNTHESIS OF LITERATURE*

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Final Report

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THE STATE OF MONTANA
DEPARTMENT OF TRANSPORTATION

in cooperation with
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FEDERAL HIGHWAY ADMINISTRATION

September 2009

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RESEARCH PROGRAMS

MDT

Measurement and Evaluation of Subgrade Soil Parameters: Phase I – Synthesis of Literature

Final Project Report

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A key material property used in the mechanistic-empirical pavement design guide (MEPDG) is the resilient modulus (M_r), which either can be obtained from experimental testing or can be backcalculated from other measured soil properties. The determination of a representative M_r value for a given subgrade, considering seasonal variations and testing intricacies, is not an easy or straightforward task. Over 30 different correlations equations were reviewed in this study. Selected equations were further examined using data from two MDT soil survey reports. Results from the literature review and preliminary data evaluation indicate there is little to no consistency between equations for predicting M_r from soil index and classification properties. Most of the equations were developed from relatively small sample sets and often for region-specific soil types.

Until a more detailed assessment is conducted, the authors discourage the general use of M_r correlation equations without prior testing and verification of the suitability and reliability of the modulus estimates. The authors suggest that full scale implementation of a repeated load triaxial testing program for the determination of M_r on a routine project basis may not be the most cost effective approach for MDT. Rather, it is recommended that additional evaluation of MDT soil survey data be conducted to identify potentially useful correlation equations and to identify the soil parameters that may be most denotative of soil stiffness. A subsequent phase of focused repeated load triaxial testing could then be conducted in an efficient manner to measure M_r for specific soil types and to verify the suitability and applicability of previously identified correlation equations.</td></tr>
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EXECUTIVE SUMMARY

A critical component of highway pavement design involves a thorough and reliable characterization of the subgrade; i.e., the foundation of the pavement riding surface. Laboratory test methods are available to characterize the strength and stiffness of subgrade soils including the Resistance value (R-value), California Bearing Ratio (CBR), and repeated load triaxial tests. In-situ tests that have been used to evaluate subgrade properties include, among others: falling weight deflectometer, in-situ CBR, plate load, miniature cone penetrometer, and dynamic cone penetrometer tests.

A key material property used in the mechanistic-empirical pavement design guide (MEPDG) is the resilient modulus (M_r), which either can be obtained from laboratory testing or can be backcalculated from other measured soil properties. The determination of a representative M_r value for a given subgrade, considering seasonal variations and testing intricacies, is not an easy or straightforward task. The standard laboratory repeated-load triaxial compression test (AASHTO T307) is complex, time-consuming, and costly, and is likely not warranted for all soil types.

Because of uncertainties in testing methods and the large diversity of subgrade soils across the U.S., numerous correlation equations for estimating M_r are available in the technical literature and new equations continue to appear on a rather frequent basis. These correlations were typically developed for specific groups of soil types or for soils obtained from specific geographic regions. Most M_r correlation equations were developed using regression analyses in which RLT resilient modulus test results were compared to results obtained from less expensive or more routine tests, such as R-value, CBR, unconfined compression, and index property tests. Over 30 different correlation equations were reviewed in this study. Selected equations were further examined using data from two MDT soil survey reports. The evaluation indicated there is little to no consistency between equations for predicting M_r from soil index and classification properties. Most of the equations were developed from relatively small sample sets and often for region-specific soil types. Until a more detailed assessment is conducted, the authors discourage general use of any correlation equation without prior testing and verification of the suitability and reliability of the equation for use in specific applications.

An extensive number of correlation equations have been developed over the past 20 years. We believe it would be prudent to conduct additional analyses of existing data to help narrow the field of equations and focus subsequent testing programs on specific soil types and soil parameters. We suggest that full scale implementation of a repeated load triaxial testing program for the determination of M_r on a routine project basis may not be the most cost effective approach for MDT. Rather, it is recommended that additional evaluation of MDT soil survey data be conducted to identify potentially useful correlation equations and to identify soil parameters that may be most denotative of soil stiffness. A subsequent phase of focused RLT testing could then be conducted in an efficient manner to measure M_r for specific soil types and to verify the suitability and applicability of previously identified correlation equations.

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1 INTRODUCTION

Although a pavement's wearing course is most prominent, the success or failure of a roadway section is often dependent upon the underlying subgrade. The strength, stiffness, compressibility and moisture characteristics of the subgrade can have significant influences on pavement performance and long-term maintenance requirements. The subgrade must be strong enough to resist shear failure and have adequate stiffness to minimize vertical deflection. Stronger and stiffer materials provide a more effective foundation for the riding surface and will be more resistant to stresses from repeated loadings and environmental conditions.

A critical component of the pavement design involves a thorough and reliable characterization of the subgrade; i.e., the foundation of the pavement riding surface. A number of laboratory methods are available to characterize the strength and stiffness of subgrade soils including the Resistance value (R-value), California Bearing Ratio (CBR), and repeated load triaxial laboratory tests. In-situ tests that have been used to evaluate subgrade properties include, among others: falling weight deflectometer (FWD), in-situ CBR, plate load, miniature cone penetrometer, and dynamic cone penetrometer tests. Several state transportation agencies are evaluating the potential of using correlations with index tests such as Atterberg limits and grain size distributions for estimating soil parameters for use in mechanistic pavement design methods.

Many soils in Montana pose significant problems for constructability and long-term pavement performance. The current method (R-value testing) used by the Montana Department of Transportation (MDT) to quantify the suitability of these soils for subgrade strength may yield unsatisfactory or inconsistent results. Other investigatory techniques may yield more consistent and reliable results, which will improve pavement performance and save significant construction and maintenance funds.

A key material property used in the AASHTO pavement design guide and the new mechanistic-empirical pavement design method is the resilient modulus (M_r), which is defined as the ratio of deviatoric stress to elastic (resilient) strain experienced by the material under repeated cycles of loading. The determination of a representative M_r value for a given subgrade, considering seasonal variations and testing intricacies, is not an easy or straightforward task. The standard laboratory repeated-load triaxial compression test (AASHTO T307) is complex, time-consuming, and costly, and is likely not warranted for all soil types.

Because of uncertainties in testing methods and the large diversity of subgrade soils across the U.S., numerous empirical and semi-empirical correlations for estimating M_r are available in the technical literature. These correlations were typically developed for specific groups of soil types or for soils obtained from specific geographic regions. There is no currently recognized or unified general approach for using correlation equations, and the reliability of these equations is often uncertain. Most correlations are site or region specific, and most correlations do not account for important variations in soil type and consistency. Adding to the confusion are the various modifications, adjustments, and simplifications that have been proposed as improvements to the laboratory repeated load triaxial test method for determining M_r. For example, the proceedings from the 2008 meeting of the Transportation Research Board in Washington D.C. contained seven papers on alternative methods for measuring, correlating, or estimating M_r. Even with the recent escalation of work in this area, some agencies still rely on rather dated and quite general correlation charts such as those first published in the 1960s by the Portland Cement Association and the Federal Highway Administration (Terrel et al. 1979).

This report provides a synthesis and overview of laboratory and in-situ test methods that have been used by state agencies and researchers to measure the stiffness of unbound base courses and subgrade soils. The focus of the literature synthesis is on methods for measuring M_r; either indirectly using correlations with other more readily measured soil parameters, or directly using laboratory and in-situ test methods. Data from two MDT soil survey reports were used to conduct a preliminary comparison of selected correlation equations. Results from the synthesis and analyses are summarized in Chapter 5 and specific recommendations are provided for implementation by MDT.

2 TEST METHODS FOR SUBGRADE EVALUATION

In recent decades, characterization of the subgrade for purposes of pavement design has focused on the engineering behavior (stress-strain response) of the soil caused by traffic loads. Resilient modulus is the primary soil parameter for most pavement design methodologies in the U.S. While the repeated load triaxial (RLT) test is widely recommended as a test method for determining resilient modulus, other laboratory and field methods have been used to estimate resilient modulus, including California Bearing Ratio (CBR), Resistance Value (R-value), and Falling Weight Deflectometer (FWD), among others. This chapter provides information about laboratory and field test methods that can be used to characterize the subgrade for pavement design.

2.1 Laboratory Tests for Subgrade Evaluation

Tests can be conducted in the laboratory on remolded soil samples or relatively undisturbed field specimens. Some methods are conducted at specific moisture-density conditions such as: optimum, near optimum, worst-case, or at various moisture-density permutations to illustrate potential variations in stiffness as the soil water content changes throughout the year. A few test methods reportedly provide a "better" resilient modulus value because of more advanced test protocols and control of stress, strain, and pore pressures; whereas, other tests are merely related to resilient modulus based on empirical correlations. The correlation approach is often chosen because of past experience and lack of evidence that more complicated and expensive tests are necessary. This section provides an overview of laboratory and in-situ tests used for subgrade characterization and for estimating a resilient modulus for pavement design.

2.1.1 R-value

The resistance value (R-value) of a soil is determined with remolded soil samples in a stabilometer device after finding the exudation pressure. Soil specimens are prepared in a kneading compactor at different near-saturation water contents and placed in an exudation indicator device and tested in compression at a rate of 2000 pounds per minute. After the exudation pressure is recorded, loading is stopped to allow the soil to rebound. The soil is then placed in a stabilometer and a horizontal pressure of 5 psi induced. A displacement-controlled vertical load is applied at 0.05 inches per minute until the load reaches 2000 pounds. The horizontal pressure is recorded before the vertical load is reduced to 1000 pounds and the horizontal pressure reduced to 5 psi. Finally, the number of turns on a calibrated handle (referred to as turns displacement) required to increase the horizontal pressure from 5 to 100 psi is determined. The R-value is calculated based on the turns displacement and the horizontal pressure corresponding to the 2000-pound vertical load. The calculated R-value can range between 0 and 100, although values less than 5 are usually reported as "minus 5," "-5," or "<5" because the test is less accurate in this range. Silty and clayey soils often have low R-values; generally in the range of 5 to 20. Whereas, sandy gravel and crushed base coarse aggregate generally have R-values in the 60 to 80 range, depending on the gradation and mineralogy of the aggregate source. Additional details of the test apparatus and protocol are provided in ASTM D2844 and AASHTO T190.

R-value is not intrinsically related to resilient modulus; nevertheless, it is still used as a test method by several state DOTs because of its familiarity and historical use in pavement design. Numerous correlations have been developed to relate the R-value of a soil to resilient modulus; a selection of these correlations are presented in Chapter 3.

The R-value test is performed on soil samples prepared at different water contents to generate a range of exudation pressures between 100 and 800 psi. The R-value corresponding to a specific exudation pressure can be interpolated. For pavement design, MDT uses the R-value corresponding to 300 psi exudation pressure.

2.1.2 California Bearing Ratio

The California bearing ratio (CBR) is determined with remolded soil samples in a load frame in which a two-inch-diameter piston is forced 0.5 inches into the soil surface at a constant rate of 0.05 inches per minute. The load associated with 0.1-inch and 0.2-inch displacements of the soil is compared to a "standard" load of 1,000 and 1,500 psi for a crushed aggregate material. Cylindrical 6-inch diameter, 4.58-inch tall soil samples are prepared in a 7-inch tall mold. The additional height provides space for surcharge weights, which represent the overburden pressure caused by the overlying pavement section. Usually, several soil samples are prepared at different water contents that are typically referenced to the optimum Proctor water content. After compaction, the samples are typically soaked for 96 hours, unless instantaneous "end of construction" CBR values are desired. The mold base and a surface plate are slotted to allow water to enter the soil specimen from both ends. The surcharge weights are in place for the soaking period and during the penetration step. Additional details of the test apparatus and protocol are provided in ASTM D1883 and AASHTO T193. Plastic clayey subgrades tend to have CBR values less than about 5; whereas, base course aggregates tend to have CBR values greater than about 40.

Like R-value, CBR is not inherently related to resilient modulus. In any case, the test is still used by several state DOTs in lieu of the more expensive and time consuming repeated load triaxial resilient modulus test. Correlations to facilitate pavement design have been developed to relate CBR to resilient modulus; these correlations are discussed in Chapter 3.

2.1.3 Repeated Load Triaxial Resilient Modulus Test

The repeated load triaxial (RLT) test can be conducted on remolded or undisturbed field samples. The test is conducted on samples in a triaxial chamber and performed at various levels of confining pressure (σ_3) and various levels of repeated deviatoric stress (σ_d). The deviatoric stress (σ_d) is the difference between the total axial stress (σ_1) and the confining pressure (σ_3). The test was designed to better simulate the loads induced by traffic than the R-value and CBR tests.

Considerable research and development has been conducted to further refine details of the test method. The AASHTO standards for this test have been revised and replaced several times since the first standard was adopted in 1982. Sometimes revisions to a protocol were deemed too dissimilar to merely revise the existing standard; in this case a new standard (with a new number) was adopted. Because of this, the following historic AASHTO standards for resilient modulus of unbound materials have existed; in chronological order they are: T274, T292, T294, and TP46. The current AASHTO standard for RLT determination of resilient modulus is T307, which replaced the four previous standards that were subsequently withdrawn. AASHTO T307 is based largely on the Strategic Highway Research Program's (SHRP) Long Term Pavement Performance Program's (LTPP) efforts to modify previous AASHTO standards to provide more consistent and repeatable test results. Subsequent to the adoption of T307, additional standards have been proposed, including NCHRP Project 1-28 in 1997 and Project 1-28A in 2004, which provided a protocol that reportedly harmonized the 1-28 protocol with the AASHTO standards. The NCHRP 1-28A proposed standard is written in AASHTO format, but as of this writing it has not formally been adopted as an AASHTO standard. Appendix A provides a table with details of each test protocol and emphasizes the wide variation in how RLT resilient modulus tests have been conducted. Unfortunately, there is not an acceptable method to cross-relate resilient modulus parameters obtained from different test protocols (Puppala 2008).

Puppala (2008) provides a comprehensive synthesis of literature pertaining to laboratory resilient modulus tests, including RLT test method development and effects of compaction, soil type, confining pressure, deviatoric stress, instrumentation, and data analysis. The following points are noted from Puppala's (2008) NCHRP synthesis:

- Research prior to 1986 primarily addressed development of RLT test protocols and apparatus, models for data analysis, and correlations to soil strength and index properties.
- Research between 1986 and 1996 primarily addressed repeatability and reliability issues of RLT tests; methods for quantifying the effects of soil type, preparation and compaction procedures; and models used for data analysis. This period also witnessed a growing database of resilient modulus test results for soils in various localized regions, although not all used the same RLT test protocol.
- Research after 1996 has been dominated by state DOT-sponsored projects in which laboratory RLT resilient modulus tests were performed on region-specific materials. These studies included topics such as: recommendations to modify or simplify the test protocols, comparisons between laboratory RLT tests and various field tests, and evaluation of analytical methods for calculating the resilient modulus for pavement design.
- RLT testing is the preferred laboratory method to determine resilient modulus for subgrade characterization needed for pavement design.
- State DOTs are hesitant to adopt routine RLT resilient modulus testing because of continual modifications to standardized test procedures.
- In lieu of conducting laboratory RLT tests, use of local correlations is considered preferable to correlations developed for national use.

As part of a RLT resilient modulus testing program on Arkansas subgrade soils, Elliott et al. (1988) used elastic layer theory to estimate the deviator stress (σ_d or σ_{cyclic}) induced by a 9000 lb wheel load for typical pavement cross sections. Results from the study indicate that variations in σ_d can be estimated based on the structural number of the pavement system. The deviator stress is approximately equal to 4 psi for structural numbers greater than 2.5 and approximately 8 psi for structural numbers less than 2.

Using an elastic layer analysis method, George (2004) calculated typical stress states for Mississippi subgrade soils under a 4500 lb wheel load to be σ_d = 7.4 psi and lateral confining stress (σ_3 or σ_c) = 2 psi.

Mohammad et al. (2007) reported that on average typical subgrade stress levels from the literature were σ_d = 6 psi and σ_3 = 2 psi.

Hossain (2008) tabulated a collection of σ_d and σ_3 values that were used in various research projects cited in the literature. Based on a synthesis of the values shown in Table 2.1, Hossain chose to use σ_d = 6 psi and σ_3 = 2 psi.

Table 2.1. Typical Confining and Deviatoric Stress Values, from Hossain 2008

Confining Stress (psi) (σ_c or σ_3)	Deviatoric Stress (psi) (σ_d or σ_{cyclic})	Reference
2	5.4	Rahim (2005)
2	7.4	George (2004)
2	5	Ping et al. (2001)
2	6	Asphalt Institute (as cited by Ping et al. 2001)
2	2	Daleiden et al. (as cited by Ping et al. 2001)
3	6	Lee et al. (1997)
2	6	Jones and Witczak (1977)

Part 2, Chapter 1, Section 2.1.3.4 of the MEPDG design guide (NCHRP 1-37A, 2004) provides a procedural outline for calculating σ_d and σ_3 using elasticity theory, the coefficient of lateral earth pressure at rest (k_o) and the densities of overlying soil and pavement layers.

2.2 Field Tests for Subgrade Evaluation

Field tests can provide measurements of resilient modulus and other soil parameters for the conditions existing at the site at the time the test is conducted. Thus, consideration should be given to the influence of climate and in-situ conditions on the measured parameters relative to design needs. Tests conducted in the field to characterize subgrade soils for pavement design can be categorized in two broad categories: 1) nondestructive or 2) intrusive. Nondestructive methods usually involve the measurement of small deformations induced by an impulse load;

whereas, intrusive methods are often based on penetration of standard pistons or cones (Puppala 2008). This section provides information about field tests used for subgrade characterization and how in-situ tests can be used to determine resilient modulus.

2.2.1 Falling Weight Deflectometer (FWD) and Lightweight FWD

Falling weight deflectometers can be used to determine the moduli of pavement layers by inducing an impulse load on the surface and measuring deflections with geophones. The moduli are determined from the measurements using iterative back-calculation computer programs. Numerous computer programs are available; unfortunately, these programs can yield inconsistent results and different moduli values. According to Puppala (2008), the two most commonly used back-calculation programs are EVERCALC and MODULUS. According to Alavi et al. (2008), DARWin and ELMOD are the most commonly used programs. In general, a modulus determined by FWD will be higher than a modulus determined from RLT resilient modulus tests. Thus, AASHTO recommends a correction of 0.33 to 0.5 be applied to moduli determined by FWD. The study by Ping et al. (2002) confirmed AASHTO's correction factors, while other studies suggest a smaller correction may be more appropriate (e.g., Rahim and George 2003). The FWD test is used by at least 45 state DOTs. Ninety percent of these states use the FWD to estimate pavement layer moduli according to a recent survey conducted by Alavi et al. (2008) for *NCHRP Synthesis 381*. Puppala (2008) provides a comprehensive overview of research sponsored by state DOTs involving the use of FWDs and Lightweight or Portable FWDs.

2.2.2 Dynamic Cone Penetration Test

The dynamic cone penetrometer (DCP) device consists of a cylindrical rod with a cone tip that is driven into the soil by repeatedly dropping a 17.6 or 10.1 pound weight from a height of 22.6 inches. The cone tip has a 60° angle and a 0.8-inch-diameter base. Disposable tips can be used in which the tip remains in the soil when the rod is extracted. The cumulative penetration is measured and recorded with the number of blows. Penetration readings are typically measured for each blow in soft soils and every 5 or 10 blows in stiffer soils. For rehabilitation or reconstruction design, only small cores (diameter as little as one inch) need to be drilled through the pavement surface to expose the underlying unbound materials for DCP investigation. After several decades of research, ASTM D6951 was adopted in 2003 to standardize the apparatus and protocol for shallow pavement applications. The ASTM standard provides a correlation between DCP index and in-situ CBR based on research conducted by the U.S. Army Corps of Engineers. Several other correlations have been proposed to determine elastic and resilient modulus from DCP index, which can be found in Puppala (2008).

2.2.3 Plate Load Test

A plate load test is conducted by applying an axial load to a set of steel bearing plates and observing the deflection of the soil. A set of up to four one-inch-thick plates with diameters ranging from 6 to 30 inches are stacked in a pyramid fashion. The bottom plate is seated firmly on the soil layer using fine sand and/or plaster of Paris as a leveling aid. Dial gauges are used to measure vertical deflection as the plates are loaded with a hydraulic jack connected to heavy mobile equipment or to a structure, which provide a reaction against the loading.

ASTM D1195 describes the process for loading the plates in a repetitive manner in which a load is applied and maintained until the rate of deflection is less than 0.001 inches per minute, for three minutes. The load is then released, the soil rebound measured, and the same load reapplied six times. Load and deflection are continually monitored during the load and unload cycles. The loading cycles are repeated at two consecutively higher axial loads.

ASTM D1196 and AASHTO T222 describe the process for non-repetitive loading, in which load is applied in increments until a predetermined total deflection is obtained or the load capacity of the equipment is reached. Each load increment is held constant until the rate of deflection is less than 0.001 inches per minute for three minutes before the next load increment is applied. The standards do not provide specific methods for interpreting the bearing capacity of the soil; however, instructions are provided to create plots that could be used in analysis or design.

2.2.4 In-Situ CBR

The in-situ CBR test procedure is analogous to the laboratory procedure in which the load required to penetrate a two-inch diameter piston into soil at a rate of 0.05 inches per minute is measured. Loading is usually obtained by a manually operated jack with reaction provided by a stiff beam connected to heavy mobile equipment. The field procedure uses 10-inch diameter surcharge weights; whereas, smaller six-inch outer diameter weights are used in the laboratory test because additional confinement is provided by the mold. Specific details of the test apparatus and procedure are provided in ASTM D4429.

2.2.5 Dilatometer Test

A dilatometer (DMT) test provides an estimate of the lateral in-situ modulus. A drill rig or other field equipment outfitted with a hydraulic press is used to push a flat steel blade into the soil. At selected depths, a thin circular steel membrane located on one side of the blade is expanded with pressurized gas. Gravels and aggregates can damage the sensitive membrane; consequently, the DMT is primarily used to test sands and fine-grained soils. Details of the test procedure are provided in ASTM D6635. Borden et al. (1985) reported the DMT test correlated well with unsoaked CBR values for A-5 and A-6 soils. Borden et al. (1986) later published a relationship between dilatometer modulus and constrained modulus. Additional testing and research is necessary to establish a reliable relationship between these moduli and the resilient modulus.

2.2.6 Cone Penetration Test

Cone penetration testing (CPT) is increasingly common with geotechnical sections of state DOTs according to a recent survey reported in NCHRP Synthesis 368 (Mayne 2007). During a cone penetration test, an instrumented cone is pushed into the soil and measurements of cone tip resistance, sleeve friction, and sometimes pore pressure are electronically recorded. The CPT provides useful geotechnical and geoenvironmental information on soils and groundwater at depth. However, the device is not commonly used to obtain properties at very shallow depths (less than about 2 ft) because of the relatively low confining pressures near the ground surface in relationship to the size of the cone. Mohammad et al. (2007) developed resilient modulus correlations using a nonstandard standard miniature CPT probe (Continuous Intrusion Miniature CPT) for fine and coarse-grained Louisiana subgrade soils. Based on our literature review, it appears these correlations are not used widely outside the state of Louisiana.

2.3 Summary

This chapter provides information regarding laboratory and field tests that can be used for subgrade characterization for purposes of pavement design. Recent pavement design methods such as the 1986 and 1993 AASHTO design guides and the new mechanistic-empirical pavement design procedure characterize the subgrade in terms of resilient modulus. The RLT laboratory test method was designed to determine the resilient modulus of a soil sample, but the test protocol has changed several times since it was first introduced in the 1980s. A detailed comparison of eight versions of the test protocol is provided in Appendix A. All of these methods have been used at one time or another for both research and design purposes. Consequently, databases and research publications that contain RLT test results may not always be comparable, especially for establishing correlations with other soil properties.

Other laboratory test methods that were used for subgrade characterization prior to the use of resilient modulus are the CBR and R-value tests. These tests are still conducted by several state DOTs for pavement design by utilizing a correlation between CBR or R-value and resilient modulus. The Falling Weight Deflectometer is the most widely used field test for subgrade characterization of existing roads. A number of other in-situ methods have been used with limited success at estimating M_r, including: dynamic cone penetration, miniature cone penetration, plate load, in-situ CBR, dilatometer tests.

3 CORRELATION EQUATIONS FOR RESILIENT MODULUS

The resilient modulus approach was first incorporated into pavement analysis and design in the 1980s after several decades of research. Since that time, there has been significant effort to relate the resilient modulus to more readily measured soil parameters using index and strength tests. One noteworthy complication to such an evaluation is the lack of a widely accepted test procedure to measure resilient modulus. Many laboratory and field approaches have been proposed; consequently, it is important to examine the specific details of any study before applying a correlation equation in design or before incorporating a correlation equation into an agency-wide standard. Specific details that could significantly affect the reliability of any correlation equation include resilient modulus test protocols as well as information on the soil type and moisture conditions. This chapter summarizes previous research attempts to establish correlations between M_r and more readily measured soil parameters. Whenever possible, testing details and soil conditions are documented with the applicable equations.

3.1 R-value Correlation Equations

3.1.1 Buu (1980)

The Idaho Transportation Department (ITD) commissioned a study in the late 1970s to develop a correlation between resilient modulus and R-value (Buu 1980). The RLT test was used to measure M_r and then correlated to the R-value test result. The RLT tests were conducted at the University of Idaho using customized triaxial equipment and the R-value tests were conducted at ITD headquarters in Boise, Idaho. The correlation equations are reported in Yeh & Su (1989) as:

$$\mathrm{M_r}(\mathrm{ksi}) = 1.455 + 0.057 \times R \tag{1}$$

$$\mathrm{M_r}(\mathrm{ksi}) = 1.600 + 0.038 \times R \tag{2}$$

The correlations correspond to resilient modulus test conditions of σ_d = 6 psi and σ_3 = 2 psi. Where σ_d is the vertical deviatoric stress and σ_3 is the lateral confining pressure. Eq. (1) was developed from tests on 10 fine-grained soils with R-values between 46 and 68; Eq. (2) was developed from tests on 14 coarse-grained soils with R-values between 9 and 82 (Yeh and Su 1989, Sandefur 2003).

The coefficient of determination (R^2 value) of a regression equation indicates the ability of the equation to predict the outcome of a given set of inputs. An R^2 value close to unity indicates the data fits the correlation equation very well. The R^2 values for Eq.s (1) and (2) are 0.10 and 0.82, respectively (Yeh and Su 1989), indicating Eq. (2) is a better predictor of M_r for the soil samples considered in the analysis.

Tri Buu was contacted by the authors of this study to learn more information about the history of the equations because various attempts to locate the research report were unsuccessful. The phone conversation revealed that recent advances in data acquisition and displacement sensors cast doubt on the validity of the results that were generated in the study. ITD no longer uses these correlation equations. It is our understanding that ITD recently purchased a resilient modulus testing machine to develop a new database of test results to correlate M_r and R-value (Tri Buu, personal communication, February 26, 2009).

3.1.2 Asphalt Institute (1982)

The Asphalt Institute (1981) design method recommends that RLT tests be performed to characterize the subgrade soil for pavement design. However, because many state DOTs do no have the necessary equipment to perform laboratory resilient modulus tests, the Asphalt Institute (1982) also provides two correlation equations. Eq. (3) was developed based on data collected from road tests in San Diego County (California) during the 1960s and 70s.

$$\mathrm{M_r}(\mathrm{ksi}) = 0.772 + 0.369 \times R \tag{3}$$

Additional evaluations by the Asphalt Institute (1982) led to Eq. (4), which was implemented in the thickness design manual (Asphalt Institute 1981), although no details were provided about whether additional soils were tested.

$$\mathrm{M_r}(\mathrm{ksi}) = 1.155 + 0.555 \times R \tag{4}$$

To evaluate the applicability of these equations, the Asphalt Institute extended the original study to include R-value and M_r tests on six additional soils. The results are summarized in Table 3.1. For these six soils, the sum of the squared errors (SSE), between the predicted and actual M_r, indicate Eq. (3) is actually a better predictor overall than Eq. (4) (SSE = 258 and 863, respectively). When considering soils with lower R-values (R < 21), Eq. (4) is more applicable based on the sum of the squared errors (SSE = 94 for Eq. (3) and SSE = 38 for Eq. (4)). However, Asphalt Institute (1982) cautions that when applied to higher R-value soils (i.e., R > 60), the correlations tend to overestimate M_r beyond a level appropriate for pavement design.

Table 3.1. Evaluation of the Asphalt Institute R-value Correlation Equations (adapted from Asphalt Institute 1982)

Soil Type	R-value[a]	M_r (ksi)[b]	M_r (Eq. 3) Prediction (ksi)	M_r (Eq. 3) Error[c] (%)	M_r (Eq. 4) Prediction (ksi)	M_r (Eq. 4) Error[c] (%)
Sand	60	16.9	22.9	36	34.5	104
Silt	59	11.2	22.5	101	33.9	203
Sandy loam	21	11.6	8.5	-27	12.8	10
Silt-clay loam	21	17.6	8.5	-52	12.8	-27
Silty-clay	18	8.2	7.4	-9.6	11.0	34
Heavy clay	<5	1.6	<2.6	64	<3.9	144

[a] R-value at exudation pressure of 240 psi
[b] M_r RLT test at σ_d = 6 psi, σ_3 = 2 psi, at optimum moisture and density
[c] Percent Error = 100 x (predicted M_r – measured M_r)/(measured M_r)

Based on the small quantity of test data and the large range of percent error, it is recommended that only limited confidence be placed in these correlation equations.

3.1.3 Washington Department of Transportation

The Washington Department of Transportation (WSDOT) developed a relationship between R-value and resilient modulus by testing soils ranging from coarse aggregates (A-1) to silty and clayey materials (A-7). R-values were measured according to WSDOT's test method in which the R-value is determined at an exudation pressure of 400 psi. The soils in this study had R-values between 25 and 75. RLT resilient modulus tests were conducted according to AASHTO T274. The reported correlation between R-value and M_r had an R^2 value of 0.67 (Muench et al. 2009).

$$\mathrm{M_r}(\mathrm{ksi}) = 0.72\left(e^{0.0521 \times R} - 1.0\right) \tag{5}$$

3.1.4 Colorado Department Transportation

Colorado Department of Transportation (CDOT) developed a multi-stepped correlation that converts soil support value to an R-value then R-value to an approximation of M_r (Yeh and Su 1989). Soil support value was used in the 1961 and 1972 AASHTO pavement design guides to characterize the subgrade soil until it was replaced with resilient modulus in the 1986 design guide. The soil support value varied between 1 and 10 and was determined indirectly by CDOTs past experience or from R-value or CBR test results (Yoder and Witczak 1975, Huang 1993, George 2004). Eq. (6) shows the correlation from R-value to M_r, but no information was presented to document the soils or M_r test protocols used to develop the equation.

$$\mathrm{M_r}(\mathrm{ksi}) = 0.001 \times 10^{a} \tag{6}$$
$$\text{where } a = \{[(R-5)/11.29] + 21.72\}/6.24$$

3.1.5 Yeh and Su (1989)

Yeh and Su (1989) developed a correlation for CDOT based on lab tests conducted on 19 soil samples. The initial phase of testing was conducted in 1985 on six clays with R-values between 5 and 40. We developed the following linear equation by applying a regression analysis to the Yeh and Su (1989) data.

$$\mathrm{M_r}(\mathrm{ksi}) = 1.859 + 0.219 \times R \tag{7}$$

The R^2 value for this equation is 0.97, which indicates a relatively good fit with the data used to develop the equation.

In a subsequent phase of testing, 13 additional soils were tested, including a wider variety of Colorado cohesionless subgrade soils. The resilient modulus was determined at the Advanced Soils Lab at the University of Colorado at Denver and the R-value was determined at CDOT. The RLT tests used to measure resilient modulus were reportedly performed in substantial accordance to AASHTO T274. The 19 soils were compacted to 95% of standard Proctor density (AASHTO T99); 16 were saturated prior to M_r testing and three were tested at optimum water content. The six clay soils were tested at σ_3 = 0, 3, and 6 psi and σ_d = 2, 5, 7, and 10 psi. The other 13 soils were tested at σ_3 = 3 and 6 psi and σ_d = 1, 2, 4, and 8 psi. The R-value test was conducted according to CDOT's procedure, which is similar to the AASHTO procedure. The reported R-value corresponded to an exudation pressure of 300 psi. After the M_r test, 10 soils were re-tested using the R-value method to quantify any change in R-value. For this unusual re-test conducted on 10 of the samples, the R-value increased for two soils, decreased for seven soils (average decrease of 24), and remained unchanged for 1. The silty soils were most susceptible to a decrease in R-value when re-tested, which may be indicative of the sensitivity and the difficulty in obtaining repeatable results for these types of soils. The average of the pre- and post-M_r R-values were computed for these 10 soils. No correction was applied to the other nine soils in the development of the correlation; instead, the R-values prior to the M_r tests were used. In other words, no distinction was made between soils that were re-tested and those that were tested only once.

The results of this subsequent testing yielded the following equation by Yeh and Su (1989) for a confining pressure of 3 psi and a deviatoric stress of 6 psi.

$$\mathrm{M_r}(\mathrm{ksi}) = 3.5 + 0.125 \times R \tag{8}$$

We plotted the reported data and calculated an R^2 value of approximately 0.5. The average (absolute) percent difference between the actual and calculated M_r for all samples is 22 percent. There is more variation for R-values greater than 60, suggesting additional soils in this range should be tested to further calibrate the method.

3.1.6 1993 AASHTO design

The correlation equation in the 1993 AASHTO pavement design guide is similar to Eq. (2) from the Asphalt Institute except the intercept is slightly different (as reported by Puppala 2008). The following equation is reportedly valid for fine-grained soils with R-values less than or equal to 20.

$$\mathrm{M_r}(\mathrm{ksi}) = 1.0 + 0.555 \times R \tag{9}$$

3.2 CBR Correlation Equations

3.2.1 Heukelom & Klomp (1962)

Heukelom and Klomp (1962) developed a commonly referenced CBR correlation based on dynamic modulus measurements and in-situ CBR tests. The in-situ CBR test results were correlated with moduli measurements obtained using an instrumented vibratory compactor in the field; not from RLT tests in a laboratory. A correlation was developed based on a combination of three sets of data:

1) wave velocity data reported by Jones (1958) for CBR values between 2 and 20,

2) wave velocity measurements conducted by Heukelom and Klomp (1962) for CBR values between 3 and 200, and

3) stiffness measurements conducted by Heukelom and Klomp (1962) for CBR values between 3 and 200.

Heukelom and Klomp (1962) used these three data sets to calculate a dynamic modulus from the wave velocity. Even though Heukelom and Klomp (1962) did not refer to the modulus as a resilient modulus, the study is presented herein because their correlation equation (Eq. 10) is referenced in several sources; sometimes as Heukelom and Klomp (1962) and sometimes as the Shell Laboratory method (Asphalt Institute 1982, Drumm et al. 1990, Witczak et al. 1995, Sukumaran et al. 2002, Puppala 2008).

$$M_r(\text{ksi}) = 1.42 \times \text{CBR} \tag{10}$$

While the regression coefficient of 1.42 provides the best fit for 69 test results with CBR values ranging from 2 to 200, it could easily vary from 0.7 to 2.8 because of the large scatter in the data. Most references to Eq. (10) in the literature simply round the coefficient to 1.5, which is likely a result of this wide range. Lofti (1984) postulated that the lack of a term for deviatoric stress in the model is responsible for the wide scatter in the data. Most references indicate the correlation is only reasonable for soils with CBR values less than 10 or less than 20. Again, there are some inconsistencies in how the original Heukelom and Klomp (1962) work is presented in later publications.

3.2.2 Green & Hall (1975)

The US Army Corps of Engineers developed the M_r and CBR relationship shown in Eq. (11) by comparing vibratory wave propagation measurements to in-situ CBR measurements obtained at several different road test projects (Green and Hall 1975). Similar to Heukelom and Klomp (1962), the correlation provides dynamic modulus, not resilient modulus. Data with CBR values from 2 to 200 were included in the database used to develop Eq. (11).

$$M_r(\text{ksi}) = 5.409 \times \text{CBR}^{0.711} \tag{11}$$

3.2.3 Powell et al. (1984)

In the course of developing a structural design method for asphalt roads in the United Kingdom, Powell et al. (1984) created a correlation equation for subgrade characterization based on in-situ CBR tests and wave propagation techniques. The study's authors did not use laboratory RLT tests because at the time of their research the authors reportedly believed that RLT tests were still primarily a research tool and not yet suitable for routine applications. Powell et al. (1984) incorporated a database of measurements originally published by Jones (1958), which involved 23 data points with CBR values all less than 20. Eq. (12) developed by Powell et al. (1984) includes empirical modifications to account for strain discontinuity between in-situ CBR tests (high strain), wave propagation tests (low strain), and vehicle-induced strains.

$$M_r(\text{ksi}) = 2.554 \times CBR^{0.64} \tag{12}$$

Unfortunately, no information about soil types or the theoretical or empirical corrections used to determine Eq. (12) was published. Powell et al. (1984) indicated the equation is only applicable for soils with CBR values between 2 and 12.

3.2.4 Lofti (1984) and Lofti et al. (1988)

Lofti (1984) and Lofti et al. (1988) developed a relationship between resilient modulus and CBR in which the deviatoric stress is included as a model parameter. Laboratory CBR and RLT resilient modulus tests were performed on a fabricated pulverized kaolinite clay (USCS classification = ML, with LL = 48 and PI = 42) at 13 different moisture-density permutations. The unsoaked CBR values ranged from 2 to 21. The RLT tests were conducted at σ_3 = 3 psi with σ_d = 3, 5, 10, 15, 20, 40, and 80 psi. Lofti (1984) used their measured data along with additional data from Barker (1982) to develop Eq. (13), which had an R^2 = 0.93 for the test data. The units of σ_d are psi in Eq. (13).

$$\log M_r(\text{ksi}) = 1.0016 + 0.043(CBR) - 1.9557\left(\frac{\log\sigma_d}{CBR}\right) - 0.1705\log\sigma_d \tag{13}$$

3.2.5 Ohio DOT (2008)

The Ohio Department of Transportation (ODOT) uses a correlation that relates M_r to CBR; however, instead of measuring CBR directly, CBR is estimated based on the group index (GI) of the soil, which is a function of the liquid limit (LL), plasticity index (PI), and percent passing the No. 200 sieve (p#200). After GI is calculated, the CBR is estimated from Figure 1, and Eq. (14) is used to calculate M_r.

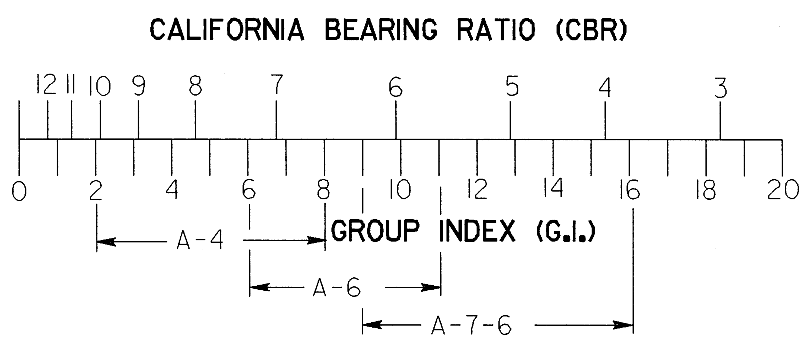

Figure 1. Relationship between CBR, Group Index and AASHTO classification (adapted from ODOT 2008).

$$M_r(\text{ksi}) = 1.2 \times CBR \tag{14}$$

3.2.6 South African Council

The South African Council on Scientific and Industrial Research (CSIR) uses Eq. (15) to estimate M_r from laboratory CBR results (reported by Witczak et al. 1995 and Sukumaran et al. 2002):

$$M_r(\text{ksi}) = 3.0 \times \text{CBR}^{0.65} \tag{15}$$

No additional information about the resilient modulus test method or the soil types used to develop the equation is available. The original source of the equation was not provided in the cross-references and the authors of the current study were not able to locate the original published work despite extensive attempts.

3.3 Soil Property Correlation Equations

3.3.1 Jones & Witczak (1977)

Jones & Witczak (1977) developed two correlation equations for A-7-6 subgrade soils in California. Resilient modulus RLT tests were performed at σ_d = 6, 12, and 18 psi and σ_3 = 2, 4, 6, 8, and 12 psi. The regression equations can be used to calculate M_r at specific stresses (σ_d = 6 psi and σ_3 = 2 psi) by inputting water content (w) and degree of saturation (S):

1) Eq. (16) is based on M_r results of 10 remolded soil samples compacted to modified Proctor density (R^2 = 0.94):

$$\log M_r(\text{ksi}) = -0.1328w + 0.0134S + 2.319 \tag{16}$$

2) Eq. (17) is based on M_r results of 97 undisturbed field samples (R^2 = 0.45):

$$\log M_r(\text{ksi}) = -0.1111w + 0.0217S + 1.179 \tag{17}$$

where, w is the water content and S is the degree of saturation. Jones and Witczak (1977) postulated that one possible reason for the differences between Eq.s (16) and (17) is that the remolded samples compacted wet of optimum may have had a dispersed structure; whereas, the undisturbed field samples most likely had a natural flocculated structure.

3.3.2 Robnett and Thompson (1973) and Thompson and Robnett (1979)

Thompson and Robnett (1979) tested A-4, A-6, and A-7 Illinois subgrade soils from 50 locations around the state. The soils were compacted with a kneading-type action to 95 or 100 percent of standard Proctor density at optimum, optimum plus 1%, or optimum plus 2% water contents. The samples were cured for seven days before undergoing RLT resilient modulus testing. All resilient modulus tests were performed at zero confining pressure. The axial stress path was approximately triangular-shaped and a load duration of 60 milliseconds was used. After applying 1,000 axial load applications for the conditioning step, a deviatoric stress of 3 psi was applied for 10 repetitions and then slowly increased at 3 to 5 psi increments for 10 repetitions until substantial permanent deformation was observed (this usually occurred at a deviatoric stress of about 25 psi). The stress applied for the conditioning step varied between 7 and 22 psi, depending on the soaked CBR value of the subgrade. The conditioning stress was determined for each soil based on the anticipated vertical load of an Illinois Class III flexible pavement with a 9,000 pound dual wheel load at 80 psi tire pressure (Robnett and Thompson 1973).

A plot of resilient modulus versus deviator stress exhibited a bilinear trend in which the resilient modulus decreased as deviator stress increased. The resilient modulus associated with the intersection of the two lines is termed the breakpoint resilient modulus, as shown in Figure 2. The slope was steeper for deviator stresses less than 6 psi and then shallower (about 85 percent less steep) for increasing values of deviator stress above 6 psi (Thompson and Robnett 1979).

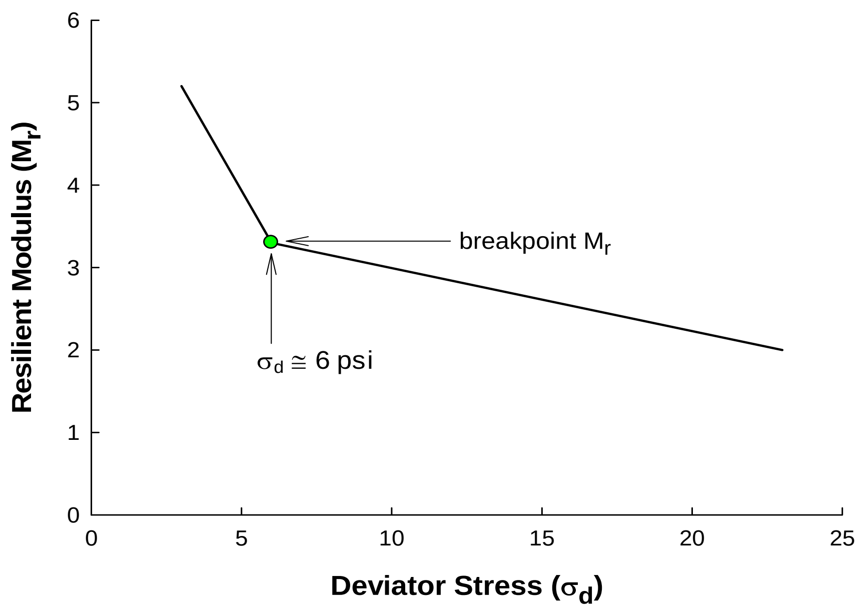

Figure 2. Example plot illustrating the breakpoint modulus concept.

Multiple linear regression analyses of the breakpoint resilient modulus (the resilient modulus corresponding to a deviator stress of approximately 6 psi for the soils tested) indicated the modulus was primarily a function of several different soil parameters. Eq. (18) yielded the strongest relationship (R^2 = 0.64) between M_r and soil index properties, in which M_r is a function of plasticity index (PI), percent silt (%SILT), percent clay (%CLAY), percent organic carbon (%OC), and group index (GI):

$$\text{(a)} \quad \begin{aligned} M_r(\text{ksi}) = 6.37 + 0.450(\text{PI}) - 0.0038(\%\text{SILT}) \\ + 0.034(\%\text{CLAY}) - 1.64(\%\text{OC}) - 0.244(\text{GI}) \end{aligned} \tag{18}$$

Rather than Eq. (18), Illinois DOT uses Eq. (19), which has a lower R^2 value (0.40), but requires fewer parameters than Eq. (18), (Marshall Thompson, personal communication, April 16, 2009).

Additionally, when Thompson and LaGrow (1988) developed a flexible pavement design procedure for Illinois DOT, they chose Eq. (19) to provide the relationship for a representative subgrade resilient modulus value.

$$\text{(b)} \quad \mathrm{M_r}(\mathrm{ksi}) = 4.46 + 0.098(\%\mathrm{CLAY}) + 0.119(\mathrm{PI}) \tag{19}$$

Thompson and Robnett (1979) developed Eq. (20) by correlating the breakpoint resilient modulus to measured unconfined compressive strength (q_u in psi). Eq. (20) had an R^2 value of 0.47 using the measured test data.

$$\text{(c)} \quad \mathrm{M_r}(\mathrm{ksi}) = 0.86 + 0.307 q_u \tag{20}$$

Thompson and Robnett (1979) also examined the effects of degree of saturation (*S*) on resilient modulus and developed the following two correlations. Eq. (21) gives the breakpoint resilient modulus (M_r at σ_d = 6 psi) for soils compacted to 95% standard Proctor dry density, whereas Eq. (22) is for 100% standard Proctor dry density. The R^2 values are 0.41 and 0.50, respectively.

$$\text{(d)} \quad \mathrm{M_r}(\mathrm{ksi}) = 32.9 - 0.334 \times S \tag{21}$$

$$\text{(e)} \quad \mathrm{M_r}(\mathrm{ksi}) = 45.2 - 0.428 \times S \tag{22}$$

where, *S* is the degree of saturation.

3.3.3 Carmichael and Stuart (1985)

Carmichael and Stuart (1985) collected over 3,300 resilient modulus records from tests conducted on 250 soils from across the U.S. to develop a correlation for the U.S. Forest Service. As a result of missing data points and other complications in the regression analyses, two formulations were developed:

1) Eq. (23) for fine-grained soils (CH, MH, ML, and CL) was based on 418 data points (R^2 = 0.76):

$$\begin{aligned} \mathrm{M_r}(\mathrm{ksi}) = {} & 37.43 - 0.457(\mathrm{PI}) - 0.618w - 0.142(\mathrm{p\#200}) \\ & + 0.179\sigma_3 - 0.325\sigma_d + 36.42(\mathrm{CH}) + 17.10(\mathrm{MH}) \end{aligned} \tag{23}$$

w = water content (%)
p#200 = % passing the #200 sieve
PI = plasticity index
CH = 1 for CH soil, 0 otherwise
MH = 1 for MH soil, 0 otherwise

2) Eq. (24) for coarse-grained soils (GW, GP, GM, GC, SW, SP, SM, and SC) was based on 583 data points (R^2 = 0.84):

$$M_r(\text{ksi}) = 0.523 - 0.0225w + 0.544\log\sigma_T + 0.173(\text{SM}) + 0.197(\text{GR}) \tag{24}$$

w = water content (%)
σ_T = total stress ($\sigma_d + 3\sigma_3$)
SM = 1 for SM soil, 0 otherwise
GR = 1 for GM, GW, GC, or GP soils, 0 otherwise

3.3.4 Elliott et al. (1988)

Elliott et al. (1988) investigated the effect of compaction, moisture, freeze-thaw, and combinations of confining and deviator stresses on the resilient modulus of 15 Arkansas subgrade soils. Most of the soils were classified as A-4, A-6, and A-7 with the exception of a single A-2 soil. All resilient modulus tests were performed following the AASHTO T274 RLT test procedure using three confining pressures (σ_3 = 0, 3, and 6 psi) and five deviator stresses (σ_d = 1, 2, 4, 8, and 10 psi). In addition to the RLT tests conducted to correlate resilient modulus to soil properties, the authors employed a simplification to the RLT test procedure using fewer load cycles and fewer deviator stresses.

Two correlation equations were developed based on the "backward elimination" regression technique. Resilient modulus was most sensitive to percent clay (%CLAY), plasticity index (PI), and optimum water content (w_{opt}), although other factors were considered (maximum dry density, liquid limit, percent colloids, and percent organic content). Eq. (25) was recommended for computing a resilient modulus corresponding to σ_d = 4 psi (R^2 = 0.80). Eq. (26) was recommended for computing a resilient modulus corresponding to σ_d = 8 psi (R^2 = 0.77).

$$M_r(\text{ksi}) = 11.21 + 0.17(\%\text{CLAY}) + 0.20(\text{PI}) - 0.73w_{opt} \tag{25}$$

$$M_r(\text{ksi}) = 9.81 + 0.13(\%\text{CLAY}) + 0.16(\text{PI}) - 0.60w_{opt} \tag{26}$$

3.3.5 Drumm et al. (1990)

Drumm et al. (1990) developed correlation equations based on the breakpoint resilient modulus, using an approach similar to the one used by Thompson and Robnett (1979). The resilient modulus test was performed according to the AASHTO T274 RLT test procedure on 11 sources of A-2, A-4, A-6, and A-7 Tennessee subgrade soils at zero confining pressure. Unconfined compressive strength tests were conducted on samples in the triaxial cell immediately after completing each M_r test. Index tests were also performed.

Unconfined compressive strength test data were analyzed to determine the modulus at the initial portion of the stress-strain curve (at a stress well below the yield stress) because the lower strain range was deemed more compatible with the small measured strains of resilient modulus tests. In general, the initial portion of the modulus (E) versus strain (ε) curve was approximately linear. The inverse of the y-intercept of this plot was termed the initial tangent modulus (1/*a*, with units of psi).

Eq. (27) was developed using a backward elimination regression technique. The properties that most significantly correlated to breakpoint resilient modulus at σ_d = 6 psi were initial tangent modulus (1/*a*), unconfined compressive strength (q_u, in psi), plasticity index (PI), dry unit weight (γ_d, in pcf), degree of saturation (*S*), and percent passing No. 200 sieve (p#200). The R^2 value was 0.83 for 22 tests (11 soils each at two different water contents).

$$M_r(\text{ksi}) = 0.001\left[45.8 + 0.00052\left(\frac{1}{a}\right) + 0.188q_u + 0.45(\text{PI}) - 0.216\gamma_d - 0.25S - 0.15(\text{p\#200})\right] \tag{27}$$

Drumm et al. (1990) developed a second more general equation for a range of deviator stresses using the same resilient modulus test results. In addition to the parameters used in the previous equation, Eq. (28) also requires percent clay (%CLAY) and liquid limit (LL).

$$\begin{aligned} M_r(\text{ksi}) &= \frac{a' + b'\sigma_d}{\sigma_d} \\ a' &= 318.2 + 0.337q_u + 0.73(\%\text{CLAY}) + 2.26(\text{PI}) - 0.915\gamma - 2.19S - 0.304(\text{p\#200}) \\ b' &= 2.10 + 0.00039\left(\frac{1}{a}\right) + 0.104q_u + 0.09(\text{LL}) - 0.10(\text{p\#200}) \end{aligned} \tag{28}$$

Eq. (28) was developed with data for σ_d ranging from 2.5 to 25 psi. For the correlation of a', R^2 = 0.81 and for the correlation of b', R^2 = 0.73. The overall R^2 value for the equation based on the M_r results of each value of σ_d tested was 0.80, suggesting a relatively good fit to the test data. However, the expression appears to be less accurate at lower values of σ_d because of the hyperbolic nature of the equation.

3.3.6 Farrar and Turner (1991)

Farrar and Turner (1991) collected nine clayey and four silty subgrade soils from Wyoming and developed two resilient modulus correlation equations, which were based on: 1) R-value test results and 2) soil index properties. Clayey soils (A-6 and A-7-6) and silty soils (A-4) were included in the research. Resilient modulus was determined with RLT test equipment conforming to AASHTO T274 and performed at σ_3 = 0, 3, and 6 psi and σ_d = 4, 8, and 10 psi. Specimens were compacted using a kneading-type compactor to three different target densities: 1) standard Proctor density at minus 1% of optimum water content, 2) standard Proctor density at plus 2% of optimum water content, and 3) the moisture and density corresponding to the conditions of the R-value specimens at 300 psi exudation pressure. The 4 inch diameter, 8-inch long specimens were compacted in four layers.

The clayey soils all had reported R-value results of minus five; consequently, the authors decided a different approach was necessary to develop a reliable correlation. A modified R-value procedure was developed for 4-inch diameter 2.5-inch tall samples. Specimens were compacted using two layers instead of four. The R-value was determined with a Hveem stabilometer according to AASHTO T190 except the exudation and swell phases of the test were not performed. The clayey soils produced modified R-values between 10 and 50. The silty soils produced modified R-values between 9 and 64.

The correlation equation (Eq. 29) between modified R-value (R_m) and natural logarithm of resilient modulus requires the user to input whether the soil is clayey or silty according to the AASHTO classification system. The R^2 value for the data used to develop Eq. (29) was 0.59.

$$\mathrm{M_r}(\mathrm{ksi}) = 0.001e^{\mathrm{a}} \qquad (29)$$

$$\mathrm{a} = 7.16 + 0.0389\mathrm{R_m} - 0.049\sigma_{\mathrm{d}} + 0.040\sigma_3 + 1.01\mathrm{X_c}$$

X_c = 1 for clayey soils, 0 otherwise

A separate correlation was developed based on degree of saturation (*S*), plasticity index (PI), and percent passing the No. 200 sieve (p#200). The R^2 value for the data used to develop Eq. (30) was 0.66.

$$\begin{aligned}\mathrm{M_r}(\mathrm{ksi}) = {} & 30.280 - 0.359S - 0.325\sigma_{\mathrm{d}} + 0.237\sigma_{\mathrm{c}} \\ & + 0.086(\mathrm{PI}) + 0.107(\mathrm{p\#200})\end{aligned} \qquad (30)$$

3.3.7 Rahim and George (2004)

Rahim and George (2004) conducted 180 RLT resilient modulus tests on Mississippi subgrade soils in accordance with AASHTO TP46. Undisturbed Shelby tube samples were collected for laboratory RLT tests at depths ranging from 1 to 3 ft below the subgrade from 60 locations on two Mississippi roads (SR25 and US45). The subgrade soil on SR25 classified as A-6; whereas, US45 had A-2-4, A-3, and A-6 subgrade soils. Correlation equations (31) and (32) were developed for fine-grained and coarse-grained soils, respectively, at σ_d = 5.4 psi and σ_3 = 2 psi.

1) Fine grained soils, 77 samples (R^2 = 0.70):

$$\mathrm{M_r}(\mathrm{ksi}) = 2.429\left[\left(\frac{\mathrm{LL}}{w} \times \frac{\gamma_{\mathrm{d}}}{\gamma_{\mathrm{d-s}}}\right)^{2.06} + \left(\frac{\mathrm{p\#200}}{100}\right)^{-0.59}\right] \qquad (31)$$

2) Coarse-grained soils, 49 samples (R^2 = 0.75):

$$\mathrm{M_r}(\mathrm{ksi}) = 44.58\left(\frac{\gamma_{\mathrm{d}}}{w}\right)^{0.86} \times \left(\frac{\mathrm{p\#200}}{\log C_{\mathrm{u}}}\right)^{-0.46} \qquad (32)$$

where, LL is the liquid limit, *w* is the water content, p#200 is the percent passing the #200 sieve, $\gamma_{d\text{-}s}$ is the maximum standard Proctor dry density, γ_d is the in-situ dry density, and C_u is the uniformity coefficient.

Of the 180 resilient modulus tests performed, 70 percent of the test results (representing 126 samples) were used to develop the two correlation equations. The other 30 percent of the data was used to check the models by comparing the laboratory results to the predicted M_r values using the correlation equations. Measured and predicted M_r values for the 33 additional fine-grained samples compared well, with an R^2 value of 0.75 for Eq. (31). The measured and predicted M_r values for the 21 additional coarse-grained samples showed even better agreement, with an R^2 value of 0.82 for Eq. (32).

In a separate study, George (2004) compared the results of AASHTO TP46 RLT resilient modulus test results from eight Mississippi test sections to values predicted by seven correlation equations, including the two developed by Rahim and George (2004). Two test sections had coarse-grained subgrade soils and the other six had fine-grained subgrade soils; none of the test sections were used in the development of Eq.s (30) and (31). The best agreement between measured and predicted values was obtained when Eq. (30) was used. This supports the premise that correlations developed for localized regions (such as a single state) likely provide better estimates of resilient modulus than equations developed for nationwide or nonspecific soil conditions. Consistent use of the same RLT test protocol for both the George (2004) and the Rahim and George (2004) testing may have also contributed to the relatively good agreement between predicted and measured results.

3.3.8 Resilient Modulus in MEPDG Computation Models

Pavement response computation models incorporated into the mechanistic-empirical design approach require the input of M_r values to represent the stiffness of supporting layers, which include unbound granular base materials and subgrade soils. The MEPDG identifies a 3-level hierarchical approach for inputting resilient moduli based on the significance of the project.

<u>For Input Level 1</u>, the designer conducts specific tests to measure M_r directly. The recommended standard laboratory methods for modulus testing are NCHRP 1-28A and AASHTO T307. The general resilient modulus constitutive equation recommended in the mechanistic-empirical design procedure is as follows:

$$\mathrm{M_r} = k_1 p_a \left(\frac{\theta}{p_a} \right)^{k_2} \left(\frac{\tau_{oct}}{p_a} + 1 \right)^{k_3} \tag{33}$$

where, M_r is the resilient modulus in psi, θ is the bulk stress, τ_{oct} is the octahedral shear stress ($\tau_{oct} = \sigma_1 + \sigma_2 + \sigma_3$), σ_1, σ_2, σ_3 are principal stresses, and p_a is the atmospheric pressure.

Dai and Zollars (2002) refer to this as the "universal model" because the equation is applicable for all unbound materials and it incorporates the effects of both deviatoric and volumetric stresses on M_r. For rehabilitation and reconstruction of existing pavement layers, M_r values for input Level 1 can also be obtained by performing nondestructive testing using a falling weight deflectometer. Input Level 1 provides the most accurate results with the highest reliability and lowest level of uncertainty. However, this level of input requires extensive effort and is the highest cost option of the three input categories.

For Input Level 2, general correlation equations that include soil index and strength properties are used to estimate Mr. These relationships can be direct or indirect such as the numerous correlation equations described in this chapter that are based on R-value, CBR, soil classification, etc. The correlation equations are empirically based and have been developed on local levels as well as regional and national approximations. Level 2 input provides an intermediate echelon of accuracy for pavement design and is the most commonly used approach by state transportation agencies (Puppala 2008).

For Input Level 3, M_r is estimated from experience or historical records. For example, the mechanistic-empirical design guide provides a table of typical representative M_r values covering a wide range of soil types. Approximate M_r values in the MEPDG range from about 40,000 psi for A-1-a granular soils to about 8,000 psi for A-7-6 clayey soils. Figure 3 shows a distribution of M_r for soil types based on the AASHTO soil classification system and Figure 4 shows a typical M_r distribution for soil types based on the Unified Soil Classification System (USCS). The data in these plots was obtained from Table 2.2.51, Chapter 2 of the MEPDG (NCHRP 1-37A, 2004). Level 3 inputs, such as the typical values presented in Figure 3 and Figure 4, provide the lowest level of accuracy. Level 3 input values should only be used for designs in which there are "minimal consequences of early failure" (NCHRP 1-37A, 2004). The plots of typical M_r values may also be useful as a check on laboratory or field measured values of M_r (e.g., Level 1 input) or as a check on the reasonableness of soil property correlations (e.g., Level 2 input).

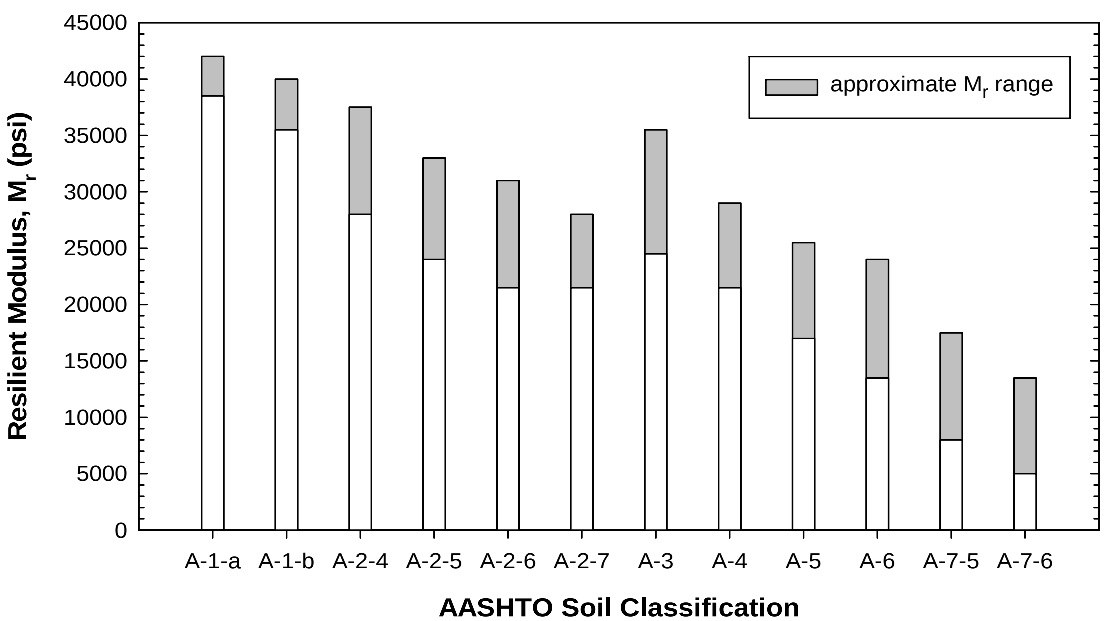

Figure 3. Typical values of M_r for AASHTO classified soils (data from NCHRP 1-37A, 2004).

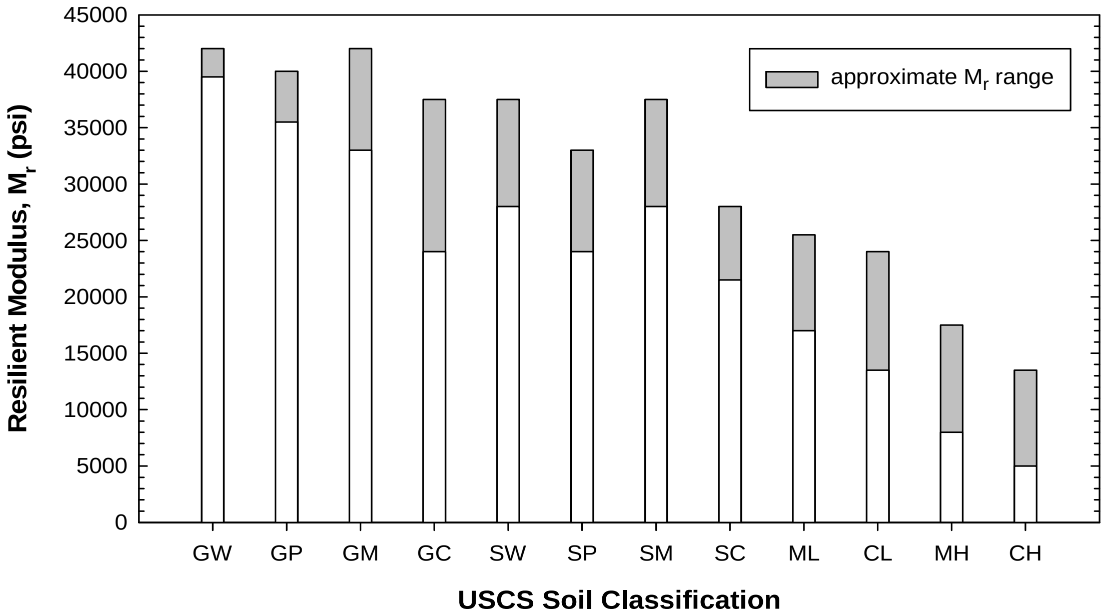

Figure 4. Typical values of M_r for USCS classified soils (data from NCHRP 1-37A, 2004).

3.3.9 Other types of correlations

In general, the correlation equations described in this chapter were developed using regression analyses in which RLT resilient modulus test results were compared to the results of less expensive or more routine tests, such as R-value, CBR, unconfined compression, and index tests. These are classified as direct correlations by Puppala (2008). Indirect correlation equations take the form of $M_r = f(\sigma_d, \sigma_3$, other σ expressions, $k_1, k_2, k_3, \ldots k_i)$ in which the k_1, k_2, k_3, …k_i parameters are correlated to soil properties. The indirect correlations are created by conducting a series of RLT resilient modulus tests on various soils and analyzing the results of each test with any of the two, three, or four parameter models mentioned in Chapter 2. The k_1, k_2, k_3, … k_i parameters are then determined for each test and are referred to as "measured" parameters. By correlating the measured parameters to soil properties (such as LL, PI, %CLAY, *w*, *S*, etc), the resilient modulus of other similar soils can be estimated at a variety of stress states without conducting another RLT test. Puppala (2008) provides a review of various indirect correlation attempts as well as an extensive list of two-, three-, and four-parameter models.

3.4 Summary

A variety of correlation equations are provided in the technical literature for estimating the resilient modulus using soil strength and index properties. More than 30 equations were identified and are summarized in Table 3.2. Eight equations were proposed for nation-wide use (Eq.s 3, 4, 9, 10, 11, 13, 23, and 24), 22 equations were developed using soil from individual

U.S. states (Eq.s 1, 2, 5, 6, 7, 8, 14, 16, 17, 18, 19, 20, 21, 22, 25, 26, 27, 28, 29, 30, 31, and 32 for Idaho, Washington, Colorado, Ohio, California, Illinois, Arkansas, Tennessee, Wyoming, and Mississippi), and 2 equations were developed for nations outside the U.S. (Eq.s 12 and 15 for the United Kingdom and South Africa, respectively).

Most of the correlations were developed with reference to resilient modulus values determined from RLT laboratory tests. The few exceptions involve the use of in-situ wave propagation measurements to obtain a dynamic low strain modulus that is then related to the resilient modulus. To further complicate matters, different test methods or protocols were used when conducting the laboratory RLT tests. For example, several researchers followed specific RLT test protocols (AASHTO T274 was cited for 9 equations, TP46 was cited for 2 equations); whereas, other researchers or agencies either did not report the specific test method that was followed, or the RLT tests were conducted using non-standard procedures or equipment that were not in adherence to any specific test standard. Unfortunately, the relationship between M_r results obtained from different RLT test protocols is not understood and has not been quantified.

Information regarding soil properties and RLT testing procedures was poorly documented for many of the correlation equations (e.g., Eq.s 3, 4, 5, 6, 9, 10, 11, 14, and 15). The large number of correlation equations and the poor agreement between predicted results may partially be attributed to inconsistent test methods and soil types. The lack of adequate documentation may also lead to extrapolation of an equation beyond its original level of calibration, which can further reduce the reliability of any prediction. The next chapter further examines the variability in M_r predictions by applying the correlation using soil index properties obtained from measurements conducted on Montana soils.

Table 3.2. Summary of Correlation Equations for Resilient Modulus

Reference	Soils Tested [No. included]	Input Parameters	M_r Test Conditions	Eqn. No.	Comments
Buu 1980	Fine-grained [10]	R-value	RLT, protocol not specified	1	R^2 = 0.10; Idaho subgrades
"	Coarse-grained [14]	R-value	RLT, protocol not specified	2	R^2 = 0.82; Idaho subgrades
Asphalt Institute 1982	NP	R-value	RLT, protocol not specified	3, 4	Applicable for soils with R<60
Muench et al. 2009	A-1 to A-7, 25<R<75	R-value	AASHTO T274	5	R^2 = 0.67, Washington DOT equation, R-value at exudation pressure of 400 psi
Yeh & Su 1989	NP	R-value	NP	6	Colorado DOT equation
"	Clay [6], 5<R<40	R-value	AASHTO T274, $\gamma_{d\text{-}s}$, OMC-S	7	R^2 = 0.97, provides M_r for σ_3=3 psi, σ_d=6 psi; Colorado subgrades
"	A-1, A-4, A-6, A-7 [19]	R-value	AASHTO T274, $\gamma_{d\text{-}s}$, OMC, OMC-S	8	R^2 = 0.50, Applicable for R < 50, provides M_r for σ_3=3 psi, σ_d=6 psi; Colorado subgrades
Puppala 2008	NP	R-value	NP	9	1993 AASHTO equation , for soils with R≤20
Heukelom & Klomp 1962	2<CBR<200	in-situ CBR	instrumented vibratory compactor (wave propagation)	10	Provides dynamic modulus, but widely referenced as correlation for resilient modulus
Green & Hall 1975	2<CBR<200	in-situ CBR	instrumented vibratory compactor (wave propagation)	11	US Army Corps of Engineers correlation, provides dynamic modulus, but widely referenced as correlation for resilient modulus
Powell et al. 1984	CBR<20	CBR	wave propagation	12	Applied theoretical correction to get resilient modulus instead of dynamic modulus
Lofti 1984	Silt [1]	CBR	RLT; σ_3=3 psi; soils within 5% of OMC for modified, standard, and low compaction energy	13	R^2 = 0.93, 1 soil tested at 13 different compaction states, unsoaked CBR, also included Barker (1982) data
Ohio DOT 2008	NP	CBR or GI	NP	14	calculate GI, look up CBR, then calculate M_r

Reference	Soils Tested [No. included]	Input Parameters	M_r Test Conditions	Eqn. No.	Comments
CSIR (Witczak et al. 1995)	NP	CBR	NP	15	South African Council on Scientific and Industrial Research equation
Jones & Witczak 1977	A-7-6 [10]	*w, S*	RLT, γ_{d-m}	16	R^2 = 0.94, provides M_r for σ_3=2 psi, σ_d=6 psi; California subgrades
"	A-7-6 [97]	*w, S*	RLT, undisturbed field samples	17	R^2 = 0.45, provides M_r for σ_3=2 psi, σ_d=6 psi; California subgrades
Thompson & Robnett 1979	A-4, A-6, A-7 [50]	PI, %CLAY, %SILT, %OC, GI	RLT, σ_3=0 psi, kneading compaction, 95% γ_{d-s}	18	R^2 = 0.64, provides breakpoint resilient modulus (approx. σ_d=6 psi); Illinois subgrades
"	A-4, A-6, A-7 [50]	PI, %CLAY	"	19	R^2 = 0.40, provides breakpoint resilient modulus (approx. σ_d=6 psi) ; Illinois subgrades
"	A-4, A-6, A-7 [50]	q_u	"	20	R^2 = 0.47, provides breakpoint resilient modulus (approx. σ_d=6 psi) ; Illinois subgrades
"	A-4, A-6, A-7 [50]	*S*	"	21	R^2 = 0.41, provides breakpoint resilient modulus (approx. σ_d=6 psi) ; Illinois subgrades
"	A-4, A-6, A-7 [50]	*S*	RLT, σ_3=0 psi, kneading compaction, 100% γ_{d-s}	22	R^2 = 0.50, provides breakpoint resilient modulus (approx. σ_d=6 psi) ; Illinois subgrades
Carmichael & Stuart 1985	Fine-grained [418 data points from literature]	*w*, PI, p#200, USCS class, σ_3, σ_d	NP	23	R^2 = 0.76, for CH, MH, ML, CL soils
"	Coarse-grained [583 data points from literature]	*w*, USCS class, σ_3, σ_d	NP	24	R^2 = 0.50, for GW, GP, GM, GC, SW, SP, SM, SC soils
Elliott et al. 1988	A-4, A-6, A-7 [15]	w_{opt}, PI, %CLAY	AASHTO T274	25	R^2 = 0.80, provides M_r for σ_d=4 psi; Arkansas subgrades

Reference	Soils Tested [No. included]	Input Parameters	M_r Test Conditions	Eqn. No.	Comments
Elliott et al. 1988	A-4, A-6, A-7 [15]	w_{opt}, PI, %CLAY	AASHTO T274	26	R^2 = 0.77, provides M_r for σ_d=8 psi; Arkansas subgrades
Drumm et al. 1990	A-2, A-4, A-6, A-7 [11]	q_u (strength & modulus), PI, γ_d, S, p#200	AASHTO T274, σ_3=0 psi	27	R^2 = 0.83, provides breakpoint resilient modulus (approx. σ_d=6 psi); Tennessee subgrades
"	A-2, A-4, A-6, A-7 [11]	σ_d, q_u (strength & modulus), %CLAY, PI, LL, γ, S, p#200	AASHTO T274, σ_3=0 psi	28	R^2 = 0.77; Tennessee subgrades
Farrar & Turner 1991	A-4, A-6, A-7-6 [13]	R_m, AASHTO class, σ_3, σ_d	AASHTO T274,	29	R^2 = 0.59; Wyoming subgrades
"	A-4, A-6, A-7-6 [13]	S, PI, p#200, σ_3, σ_d	AASHTO T274,	30	R^2 = 0.66; Wyoming subgrades
Rahim & George 2004	A-2-4, A-3, A-6 [110]	LL, w, $\gamma_{d\text{-}s}$, γ_d, p#200	AASHTO TP46, undisturbed field samples	31	R^2 = 0.70, for fine-grained subgrade soils, provides M_r for σ_3=2 psi, σ_d=5.4 psi; Mississippi subgrades
"	A-2-4, A-3, A-6 [70]	γ, w, p#200, C_u	AASHTO TP46, undisturbed field samples	32	R^2 = 0.75, for coarse-grained subgrade soils, provides M_r for σ_3=2 psi, σ_d=5.4 psi; Mississippi subgrades

Table Notes

NP = Not provided

Input parameters: R-value, modified R-value (R_m), California Bearing Ratio (CBR), water content (w), optimum water content (w_{opt}), dry density (γ_d), maximum standard Proctor density ($\gamma_{d\text{-}s}$), degree of saturation (S), plasticity index (PI), liquid limit (LL), percent passing No. 200 sieve (p#200), group index (GI), percent clay (%CLAY), percent silt (%SILT), % organic carbon (%OC), unconfined compressive strength (q_u), uniformity coefficient (C_u), deviatoric stress (σ_d), hydrostatic confining pressure (σ_3), pounds per square inch (psi), regression equation coefficient of determination (R^2)

M_r test conditions and sample preparation: Resilient modulus (M_r), Repeated Load Triaxial (RLT), standard Proctor density ($\gamma_{d\text{-}s}$), modified Proctor density ($\gamma_{d\text{-}m}$), optimum moisture (OMC), optimum moisture then soaked (OMC-S), American Association of State Highway and Transportation Officials (AASHTO)

4 EVALUATION OF CORRELATIONS AND THE RLT TEST

4.1 Introduction

This chapter describes a preliminary evaluation of the correlation equations described in Chapter 3. These equations provide a less expensive and less cumbersome method of estimating M_r for pavement design. The equations generally are used in lieu of conducting laboratory repeated load triaxial tests to measure M_r. In most cases, these equations were developed for a relatively limited quantity of test samples and for specific geographic regions of the country. Consequently, the reliability and accuracy of correlation equations for general use may be less than satisfactory. The following analyses and discussions are presented as a means of examining the consistency and variability of the equations.

4.2 R-value Correlation Equations

Results from an extensive nation-wide survey conducted by Puppala (2008) indicates the majority of state DOTs do not measure M_r directly in the laboratory using repeated load triaxial (RLT) tests. Instead of the relatively complex and expensive RLT test, most agencies use correlations to estimate moduli of both subgrade and unbound base geomaterials using more familiar methods such as R-value, CBR, soil classification, and soil index properties. MDT currently uses the R-value laboratory test to estimate the subgrade stiffness for pavement design, and either the AASHTO (1993) equation or a hand drawn line on a plot in the MDT Surface Design Manual to estimate M_r from the measured R-value. The correlation equation provided in the AASHTO (1993) pavement design guide is repeated here as Eq. (34). This R-value correlation for estimating M_r is referred to hereafter in this report as the AASHTO 1993 equation.

$$\mathrm{M_r}(\mathrm{ksi}) = 1.0 + 0.555 \times R \tag{34}$$

No citation or reference is available for the (M_r) – (R-value) plot in the MDT Surface Design Manual; for reference purposes, a scanned copy of the plot is provided in Figure 5. MDT materials personnel use the curve identified in the figure as "*curve added by surfacing design*" (Dan Hill, personal communication, February 20, 2009). Eq. (35) was developed by the authors of this study as a mathematical representation of the hand-drawn line (the line that is overlaid with "x" symbols in Figure 5).

$$\mathrm{M_r}(\mathrm{ksi}) = -0.00050925 \times R^2 + 0.29691215 \times R + 3.291856 \tag{35}$$

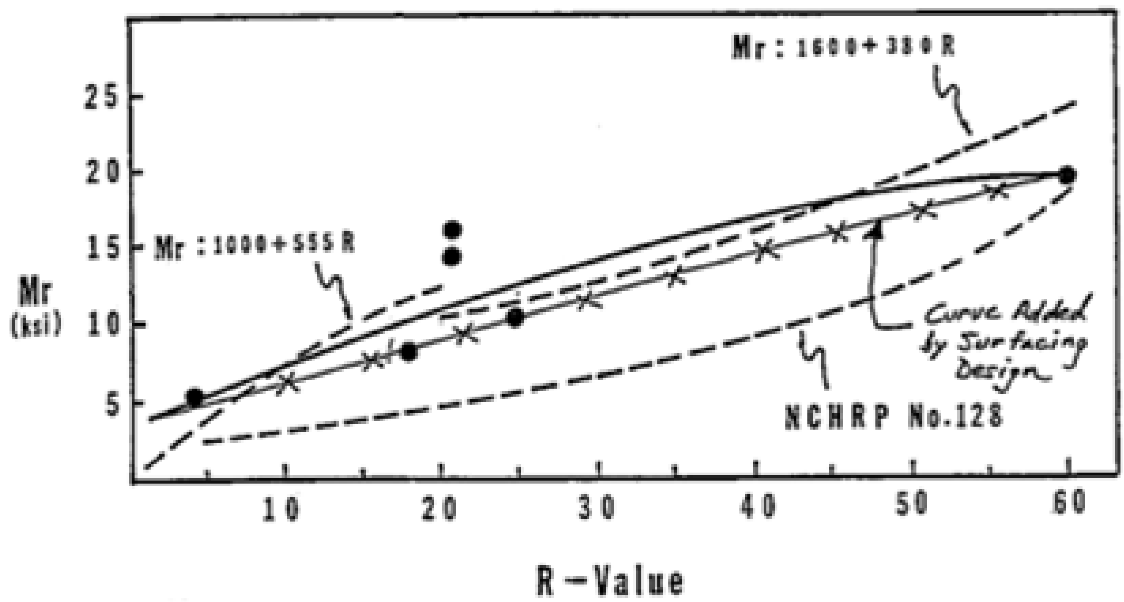

Figure 5. Scanned plot from MDT Surface Design Manual.

As an aside, there appears to be an error in the dashed line labeled as: M_r = 1600 + 380R. We were unable to locate any correlation equation in our literature search that matched this line or equation. Our records indicate the slope of this equation may be incorrect and should be 38 instead of 380. If this change is made, the equation would plot lower on the graph and would be identical to the Buu (1980) correlation, as plotted in Figure 6.

The R-value correlation equations described in Chapter 3 and the MDT surfacing design curve (Eq. 35) are shown as functions of M_r in Figure 6. The AASHTO R-value correlation provides the least conservative estimate of M_r (i.e., highest M_r for a given R-value). The two correlation equations by Buu (1980) provide overly conservative estimates of M_r. The Buu (1980) relationships appear to indicate there is little to no change in M_r even over a wide range of R-values. As discussed in Chapter 3, this inconsistency is one reason Idaho no longer uses these correlation equations. The Yeh and Su (1989) equation (Eq. 8) represents the next most conservative formulation. However, the difference between the Yeh and Su (1989) and the AASHTO R-value correlation is significant. For example, at R-values of 20, 40, and 60 the percent differences are 67.4%, 93.6%, and 102.9%, respectively. The Muench et al. (2009) formulation has an exponential shape and yields proportionally low vales of M_r at smaller R-values and progressively larger values of M_r, in proportion to the other formulations, at higher R-values. The MDT expression (Eq. 35) provides M_r values between the other two formulations. Unfortunately, no background information is available on the MDT formulation, and to the best of our knowledge, no specific testing has been conducted to ascertain the merit of any of these correlations for Montana soil and climatic conditions.

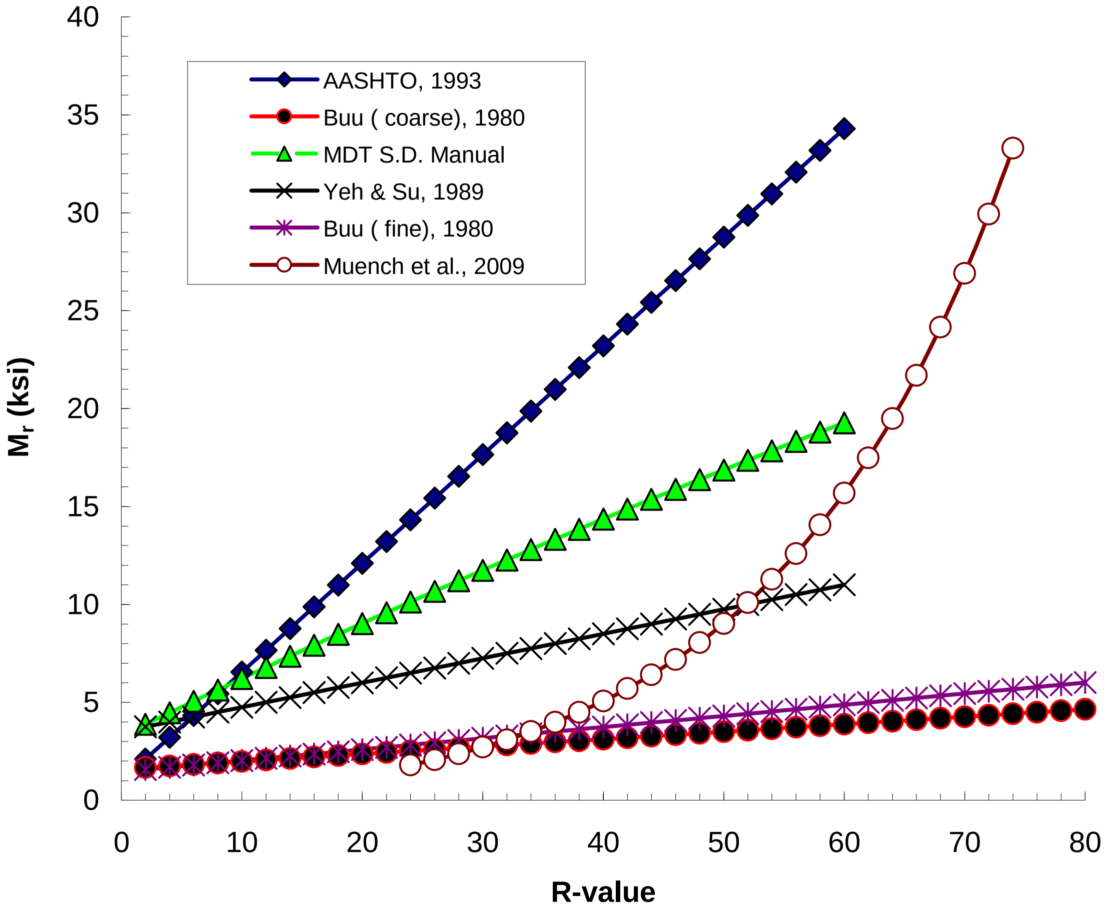

Figure 6. Resilient modulus based on R-value correlations.

4.3 CBR Correlation Equations

A comparison of seven CBR correlation equations obtained from the literature review is shown in Figure 7. Similar to the R-value comparison, there is substantial variation between M_r estimates. The Green and Hall (1975) correlation represents the least conservative estimation of M_r, while the Powell et al. (1984) correlation provides the most conservative estimate of M_r. The differences between these two bracketing formulations are relatively large and comparable to the distribution observed in the R-value correlations. For example, at CBR values of 20, 40, and 60 the percent differences are 89.3%, 93.1%, and 95.3%, respectively. The Heukelom and

Klomp (1962) and the Asphalt Institute (1982) equations yield essentially identical results. They are both shown here because they are cited individually in many references.

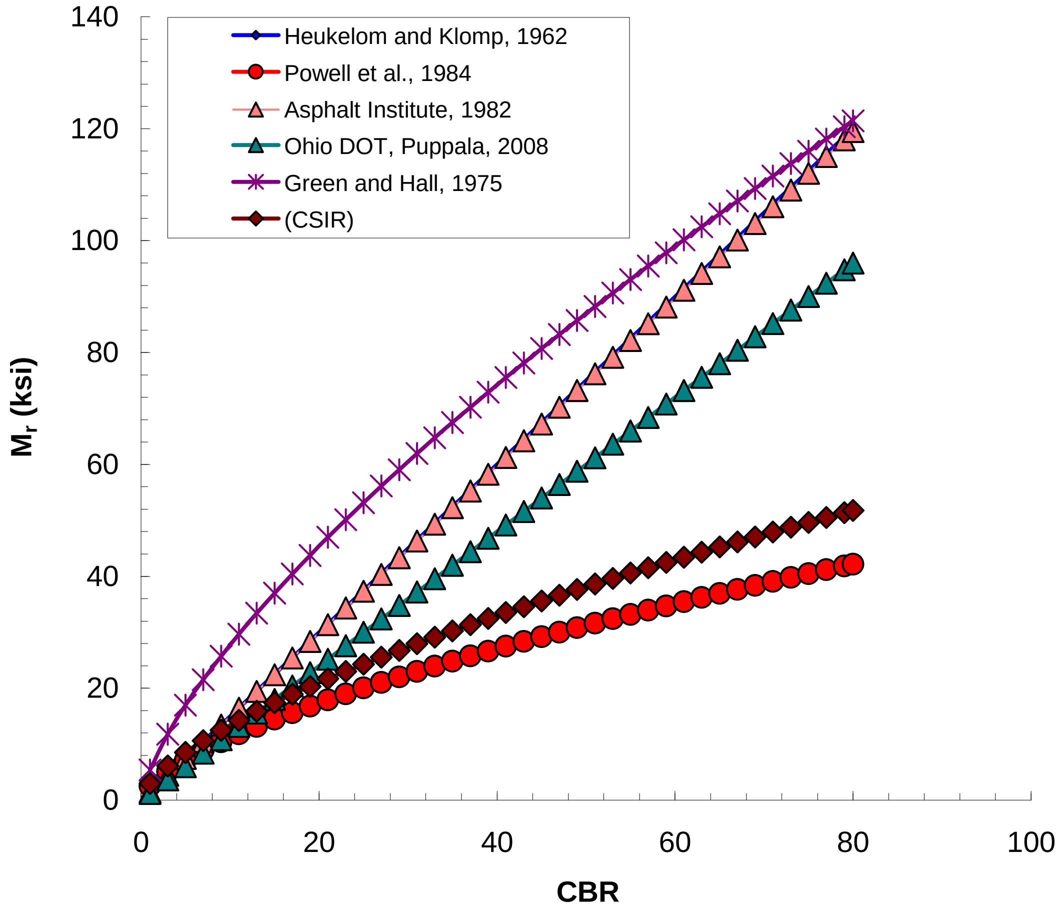

Figure 7. Resilient modulus based on CBR correlations.

4.4 Soil Property Correlation Equations

In addition to R-value and CBR correlations, numerous additional formulations are presented in technical publications that relate M_r to various soil index and classification tests, including: water content *(w),* degree of saturation (*S*), specific gravity (G_s), unit weight (γ), Proctor density, optimum water content, % clay, % silt, and soil classification. We conducted a preliminary comparison of these methods using data from the following two soil survey reports that were provided by MDT for the purpose of this analysis:

1) Two Dot to Harlowton (11/24/1997): 24 applicable samples

2) Augusta Interchange to Hardy Creek (12/9/1997): 8 applicable samples.

The AASHTO 1993 equation (Eq. 34) was used as the standard to evaluate the consistency of the correlation equations. Any of the R-value correlations shown in Figure 6 could have been used in this comparison; Eq. (34) was selected because of its wide use and notoriety.

The soil survey reports contained test data for 27 samples from the Two Dot project and about 180 samples for the Augusta project. For the M_r correlation comparison, a relatively comprehensive suite of test parameters are necessary, including: R-value, Proctor optimum values of γ_d and w, AASHTO soil classification, Atterberg limits, and particle gradation. The Two Dot project contained 24 samples that met these criteria, while the Augusta project contained 8 samples. Seventeen of the 32 samples were either A-1 or A-2 soils (predominately coarse-grained); the remaining 15 samples were classified as A-4 through A-7 soils (predominately fine-grained). Assumptions had to be incorporated in some of the calculations because the soil survey data was not originally collected for the purpose of this study. Table 4.1 summarizes assumptions that were used to supplement the soil survey data. Thompson and Robnett (1979) presented numerous correlation equations in their publications, as described in Section 3.3.2 of this report. Only three of the equations utilized variables that are consistent with soil parameters available from the soil survey reports. To match the numbering system used in Section 3.3.2, the three Thompson and Robnett (1979) equations are labeled with a, b, and d postscripts in Table 4.1 and Table 4.2.

Table 4.1. Assumptions Utilized in the Correlation Equations

Correlation Equation	Assumptions used for the Soil Survey Data
Jones & Witczak (1977)	Assumed G_s = 2.70, when tests results were not available. Used γ_{dmax}, w_n, and G_s to calculate e and S. Original M_r tests used σ_d = 6 psi and σ_3 = 2 psi. Equation was developed using clayey samples.
Thompson & Robnett (a) (1979)	%Silt assumed = p#200 for A-2-4, A-2-5, A-4, and A-5 soils. %Clay assumed = p#200 for A-2-6, A-2-7, A-6, and A-7 soils. Original M_r tests used σ_d = 6 psi and σ_3 = 0. Equation was developed using clayey samples.
Thompson & Robnett (b) (1979)	%Clay assumed = p#200 for A-2-6, A-2-7, A-6, and A-7 soils. %Clay was assumed = 0 for other soil types. Equation is recommended for fine-grained subgrades.
Thompson & Robnett (d) (1979)	Assumed G_s = 2.70, when tests results were not available. Used γ_{dmax}, w_n, and G_s to calculate e and S.
Carmichael & Stuart (1985)	Assumed G_s = 2.70, when tests results were not available. Used γ_{dmax}, w_n, and G_s to calculate e and S. Assumed σ_d = 6 psi and σ_3 = 3 psi. Equation is recommended for fine-grained subgrades.
Elliot et al. (1988), 4 psi	Original M_r tests used σ_d = 4 psi. Equation is recommended for cohesive subgrades.
Elliot et al. (1988), 8 psi	Original M_r tests used σ_d = 8 psi. Equation is recommended for cohesive subgrades.
Farrar & Turner (1991)	Assumed G_s = 2.70, when tests results were not available. Used γ_{dmax}, w_n, and G_s to calculate e and S. Assumed σ_d = 6 psi and σ_3 = 3 psi. Equation is recommended for fine-grained subgrades.

DEFINITIONS OF TERMS

G_s = specific gravity	γ_{dmax} = maximum Proctor dry density
e = void ratio	w_n= natural water content
S = saturation	σ_d = deviator stress = (σ_1 - σ_3)
p#200 = percent passing the #200 sieve	σ_3 = confining pressure = triaxial cell pressure

A compilation of correlated M_r values is shown in Figure 8, which incorporates applicable data from both soil survey reports. The 1:1 line in the plot demarks the "ideal" condition in which the (M_r) – (R-value) correlation agrees precisely with a calculated M_r value obtained from one of the seven soil property correlation formulations listed in the figure legend. As shown in Figure 8, considerable scatter in the data is evident with no discernable trends or agreement between correlation results. A simple statistical analysis was performed to further examine the correlation results by normalizing the estimated M_r values with respect to the AASHTO 1993 correlation (Eq. 34), as follows:

$$x = \frac{M_{r-\text{correlation}}}{M_{r-\text{AASHTO1993}}} \tag{36}$$

where, x is the normalized value, $M_{r\text{-correlation}}$ is determined using one of the soil property correlation equations, and $M_{r\text{-AASHTO1993}}$ is the M_r value determined using the AASHTO 1993 equation.

Calculations of mean values and standard deviations of x for each correlation formulation were performed to examine the relative accuracy and distribution of the predictive correlation equations. These values are summarized in Table 4.2. A mean (μ) of unity indicates the predicted M_r based on soil index properties matches the (M_r) – (R-value) from the AASHTO 1993 equation, on an average basis. A $\mu < 1$ indicates the $M_{r\text{-correlation}}$ is lower than the $M_{r\text{-AASHTO1993}}$ estimate; while, a $\mu > 1$ indicates the $M_{r\text{-correlation}}$ exceeds the $M_{r\text{-AASHTO1993}}$ estimate, on average.

Normal frequency distributions, f(x), for each of the correlation formulations are shown in Figure 9, based on the μ and σ values shown in Table 4.2. As shown in Figure 9, and as quantified by the μ values, three of the correlation equations (Thompson and Robnett (d), 1979; Carmichael and Stuart, 1985; Farrar and Turner, 1991) yielded M_r values greater than M_r predicted by the AASHTO 1993 equation, on average. The other five correlation equations yielded M_r values that were predominately less than M_r calculated from the AASHTO 1993 equation. Standard deviations ranged from 0.41 to 0.99 indicating overall a broad distribution of calculated M_r values with no discernable trends or potential relationships. The Carmichael and Stuart (1985) correlation yielded the lowest coefficient of variation (0.44) indicating the best relative agreement of the group; however, this value is still considered poor from a practical user's perspective. The Jones and Witczak (1977), Elliot et al. (1988), and Farrar and Turner (1991) equations all had coefficients of variation greater than 0.80 indicating very large scatter in computed results.

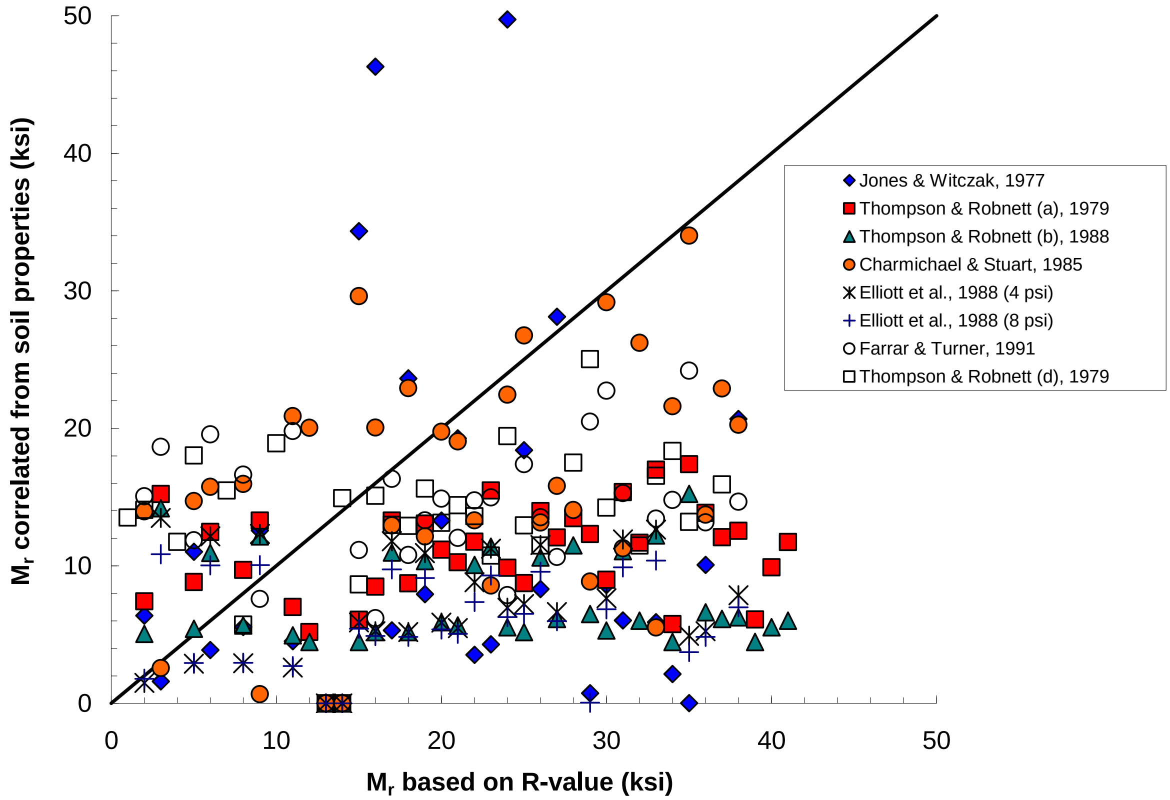

Figure 8. M_r correlation using MDT soil survey data.

Table 4.2. Comparison of Correlation Ratios: $M_{r\text{-correlation}}/M_{r\text{-AASHTO 1993}}$

Correlation Formulation	Mean, μ	Standard Deviation, σ	Coefficient of Variation, COV
Jones & Witczak (1977)	0.95	0.84	0.89
Thompson & Robnett (a) (1979)	0.92	0.71	0.77
Thompson & Robnett (b) (1979)	0.61	0.48	0.79
Thompson & Robnett (d) (1979)	1.18	0.86	0.73
Carmichael & Stuart (1985)	1.24	0.54	0.44
Elliot et al. (1988), 4 psi	0.58	0.52	0.89
Elliot et al. (1988), 8 psi	0.51	0.41	0.81
Farrar & Turner (1991)	1.25	0.99	0.80

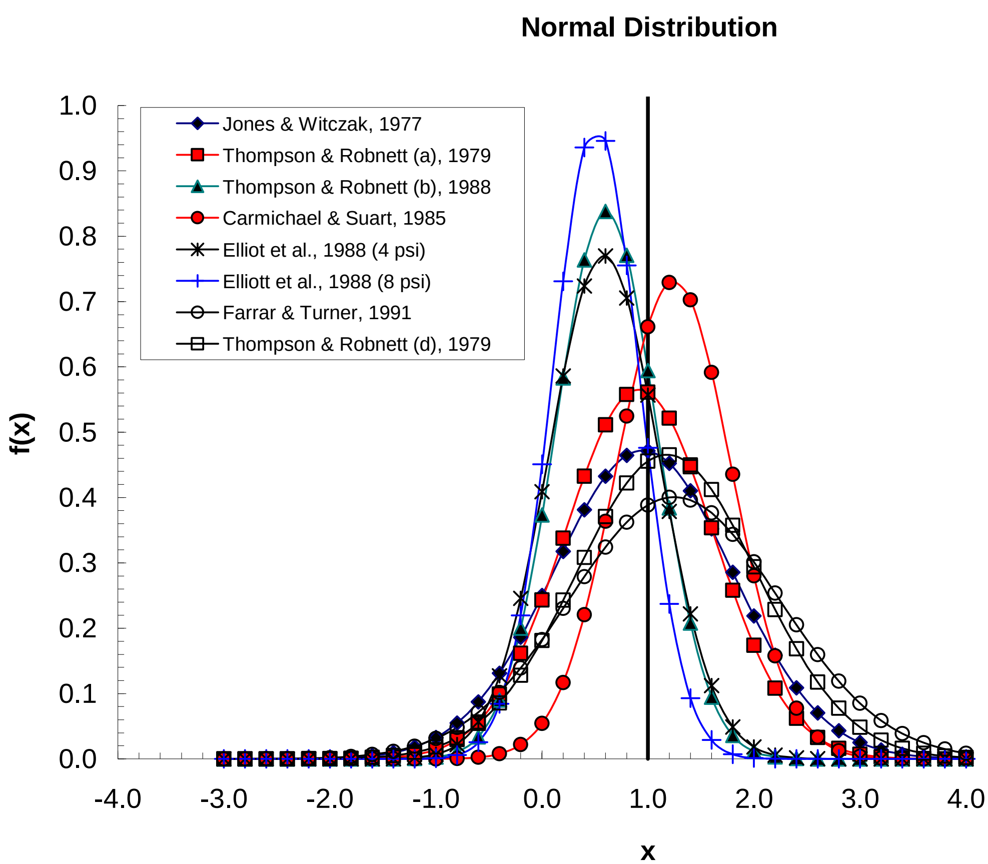

Figure 9. Normal distribution of M_r correlation equations.

4.5 Evaluation of MDT Repeated Load Triaxial M_r Data

Von Quintus and Moulthrop (2007) conducted a study for MDT in which they evaluated performance characteristics of variables used in distress prediction models found in the MEPDG flexible pavement design software. As part of this study, RLT tests were conducted on a variety of soils obtained from different sites in Montana. Modulus measurements from RLT tests and the corresponding soil index properties for 10 soils (obtained from Von Quintus and Moulthrop Tables 1-20 and 1-22) were evaluated by the authors of this current study using equations that correlate soil index properties to M_r. Calculated values of M_r are compared to the corresponding measured values of M_r in Figure 10. The 1:1 line in the plot demarks the "ideal" condition in which the correlated M_r agrees precisely with the measured M_r. Specific gravity (G_s) values were not reported for these soils; consequently, a G_s of 2.7 was assumed for the correlation equation calculations.

Normal frequency distributions, f(x), for each of the correlation formulations are shown in Figure 11. Von Quintus and Moulthrop (2007) conducted a relatively large quantity of laboratory tests; however, only ten soils in their study contained the necessary combination of soil index and M_r tests for this correlation evaluation. These soils were all described as nonplastic poorly graded sand with silt (AASHTO classification A-2-4). As shown in Table 4.3, there is minimal variation in the measured M_r values or soil index properties. For that reason, the distributions of M_r shown in Figure 10 and Figure 11 are limited in their usefulness. The Thompson and Robnett (a) (1979) correlation provided the best agreement to measured values. The Carmichael and Stewart (1985) correlation yielded the highest estimated values of M_r, while the two Elliot et al. (1988) correlations yielded the lowest calculated values of M_r.

Table 4.3. Statistical Summary of Data from Von Quintus and Moulthrop (2007) Tests

	Mean, μ	Standard Deviation, σ	Coefficient of Variation, COV
M_r RLT (measured)	6.41 ksi	0.70 ksi	0.109
Percent passing #200 sieve	24.6 %	6.7 %	0.272
Maximum dry unit weight (AASHTO T90)	106.2 pcf	4.44 pcf	0.042
Optimum water content (AASHTO T90)	14.6 %	1.51 %	0.103

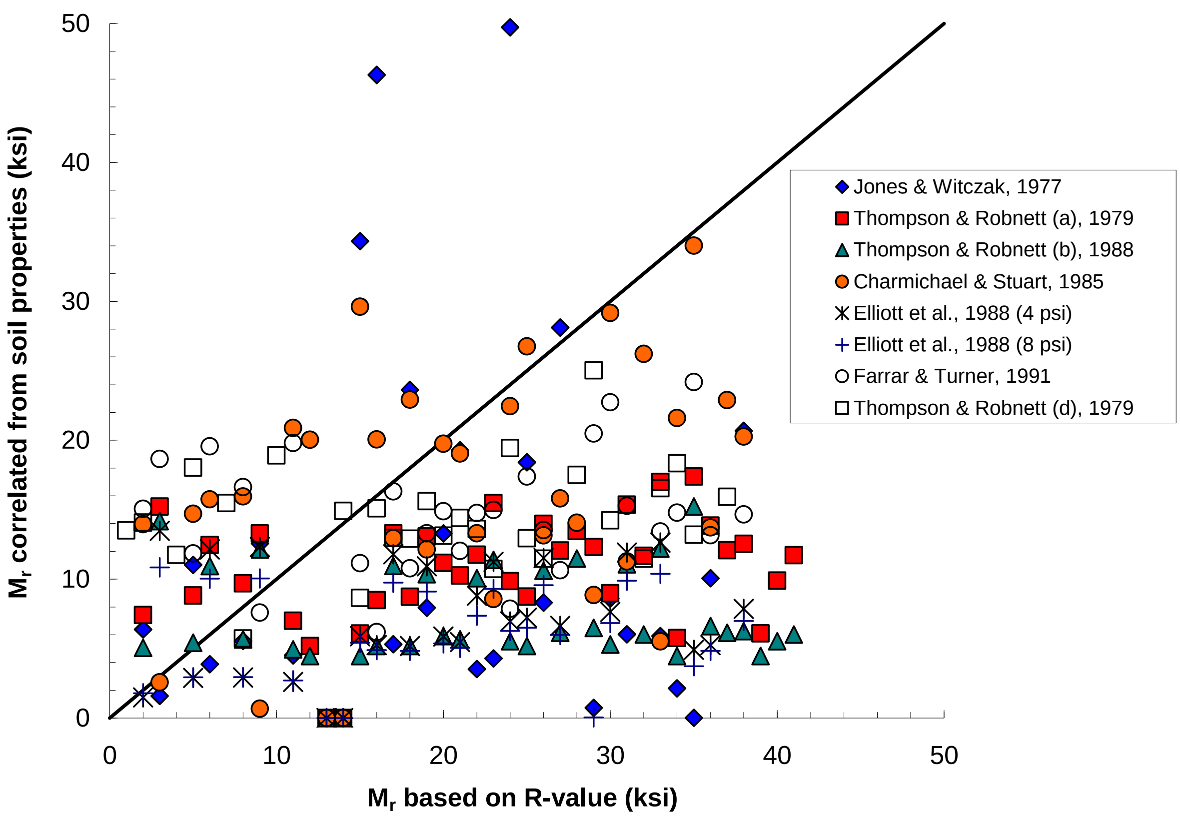

Figure 10. Correlated M_r values compared to RLT measured M_r.

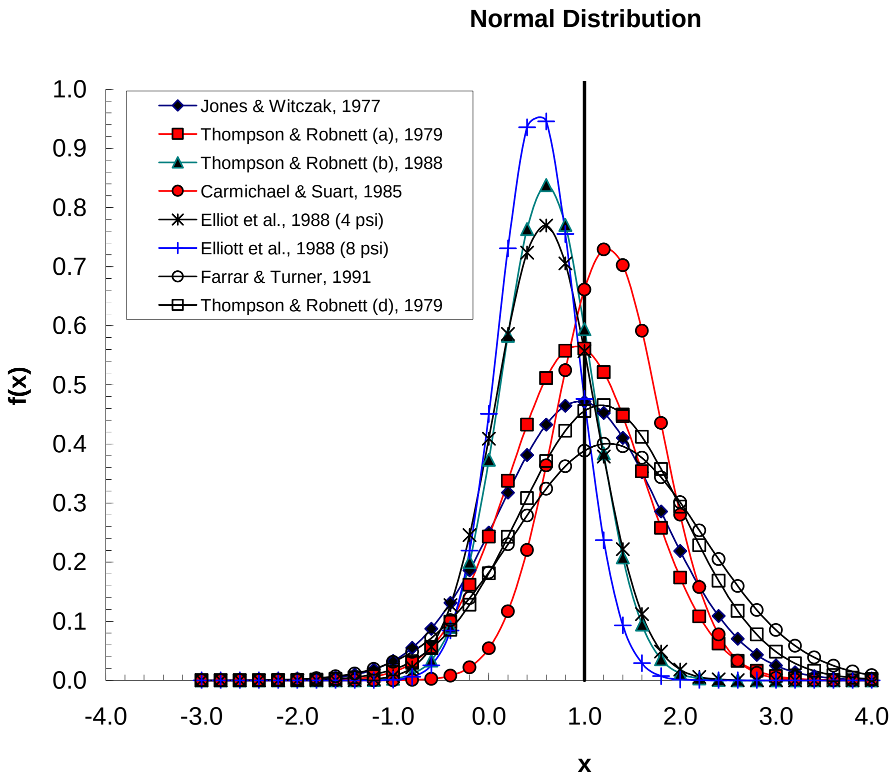

Figure 11. Normal distribution of M_r from correlation equations.

4.6 Sensitivity Analysis

The resilient modulus for the roadbed (subgrade) soils is used to represent the stiffness of the soil layers in the new mechanistic-empirical flexible pavement design method and the pre-existing empirical performance equations in the AASHTO (1993) design guide. Other than some environmental and climatic modifications, the AASHTO approach is relatively unchanged since the original interim design guide was first published in 1961 (Huang 2004). Based on our understanding, MDT designers currently use the AASHTO (1993) guidelines for calculating the structural number (SN) of the pavement section and for determining the thickness of each layer in the pavement section. This is currently the most common approach used by state transportation agencies, based on responses to a nation-wide survey conducted by Puppala (2008). This recent survey indicates that 31 state DOTs (out of 40 responses to this question) use the AASHTO design guide approach. Only one state agency reported using the mechanistic-empirical design guide. The remaining DOTs use design procedures developed specifically for their state (Puppala 2008).

Because of the popularity of the AASHTO design procedure, the sensitivity of M_r to the calculated value of SN was evaluated using the following AASHTO (1993) pavement design equation:

$$\log_{10}(W_{18}) = Z_R S_o + 9.36\log(\mathrm{SN}+1) - 0.20 + \frac{\log[\Delta\mathrm{PSI}/(4.2-1.5)]}{0.4 + 1094/(\mathrm{SN}+1)^{5.19}} + 2.32\log_{10}(M_r) - 8.07 \quad (37)$$

where, W_{18} is the predicted number of 18-kip equivalent single axle load applications; Z_R is the standard normal deviate; S_o is the combined standard error of the traffic prediction and performance prediction; ΔPSI is the difference between the initial design serviceability index, p_o, and the design terminal serviceability index, p_t; and M_r is the resilient modulus in units of psi.

Eq. (37) was used to evaluate the sensitivity of the SN value to changes in M_r for a range of equivalent single axle loads (ESALs). An Excel macro that uses the bisection numerical technique to solve for the root of an equation in a sequential fashion was employed to solve Eq. (37) for SN, given a wide range of M_r for different levels of traffic (W_{18}). The baseline values used in this analysis are summarized in Table 4.4. These are typical values that could be applied to interstate highway design in Montana (Dan Hill, personal communication, February 20, 2009).

Table 4.4. Baseline Conditions for Sensitivity Study

Input Parameter	Design Value
reliability	90%
standard normal deviate, Z_R	-1.282
combined standard error, S_o	0.45
initial serviceability index, p_i	4.2
terminal serviceability index, p_t	2.5
change in serviceability, ΔPSI	1.7

Results of the sensitivity analysis are shown in Figure 12 for ESALS ranging from 200 to 3000. For comparison purposes, Interstate 94 (I-94) and I-15 in Montana generally experience ESALs in the 400 to 500 range (outside the influences of populated areas), whereas I-90 near the Wyoming border has an ESAL of about 1200, and I-15/I-90 through Butte experience ESALs of about 2300. The predicted number of 18-kip equivalent single axle load applications (W_{18}) are calculated by multiplying the ESAL by the design life (20 yr x 365 day/yr). As can be seen in Figure 12, SN is most sensitive to change at lower values of M_r; i.e., values generally less than about 8 ksi. For example, when M_r is changed from 2 to 4 ksi, SN changes by 1.27 (19.3% change); in comparison, a change in M_r from 14 to 16 ksi results in a change of SN of only 0.17 (5.0% change). The two ranges of M_r used in this example are significant to MDT because it is our understanding that MDT designers will often assume M_r = 3.75 ksi for subgrades with R ≤ 5, and assume M_r = 15 ksi for subgrades consisting of A-1-a material (Dan Hill, personal communication, February 20, 2009). Figure 12 also indicates that SN is more sensitive to changes in traffic volumes at relatively low ESAL values (e.g., 200 to 500) than at higher ESALs (e.g., 2300 to 3000), as can be observed in the relative spacing between curved lines plotted in Figure 12.

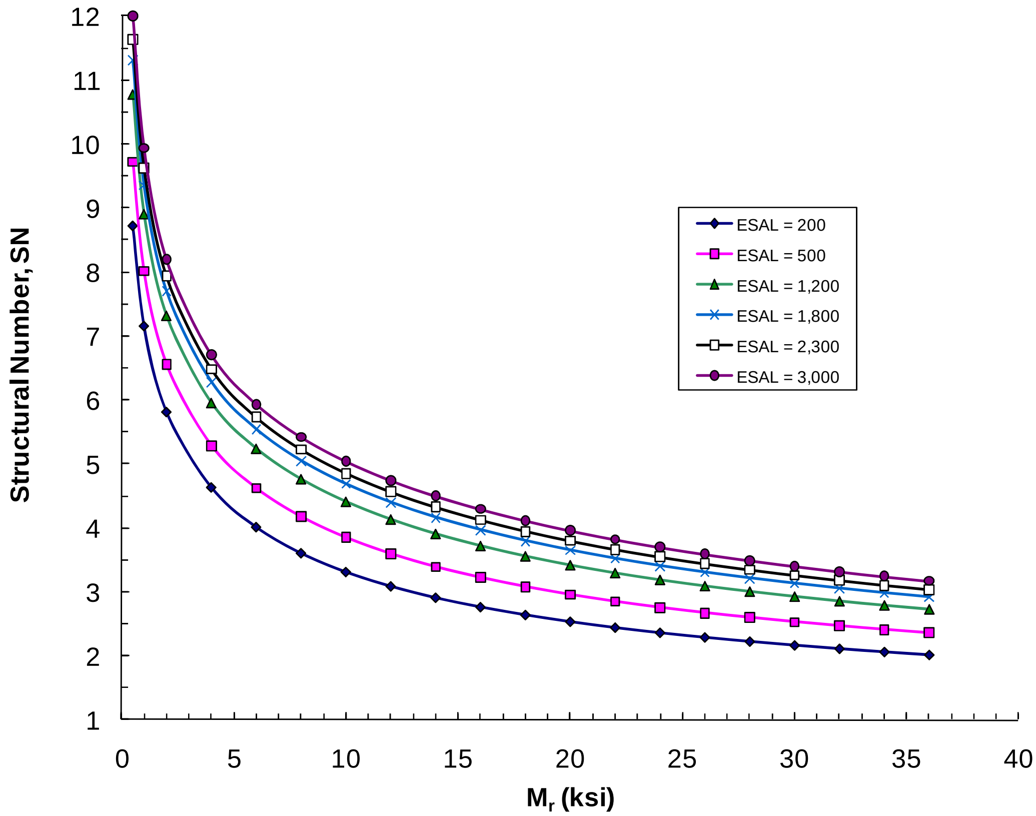

Figure 12. Sensitivity of flexible pavement Structural Number to M_r as a function of daily ESALs.

4.7 Repeated Load Triaxial Test Equipment

Repeated load triaxial (RLT) testing provides a comprehensive array of data for determining M_r over a range of pressures and shear stresses. RLT testing can be used to define the relationship between deviator stress, confining pressure, and modulus, which is an important relationship in the new mechanistic-empirical design procedure. A repeated load triaxial (RLT) testing system consists of the following primary systems and components:

- load frame,
- servo-hydraulic actuator and hydraulic pump,
- triaxial cell,
- electro-pneumatic cell pressure regulator,
- electronic equipment and sensors, and
- computer and data acquisition system.

A standard triaxial load frame is used to support the triaxial chamber and testing apparatus. Air is used as the confining fluid and an electro-pneumatic pressure regulator cell is used to control the air pressure within the triaxial chamber or cell.

The hydraulic actuator provides the cyclic component of axial stress, which can be controlled using a closed-loop servohydraulic system to apply a repeated axial deviator stress of fixed magnitude with a load duration of 0.1 sec and a cycle duration of 1 to 3.1 sec. The system should be capable of applying a haversine-shaped load pulse. Some systems incorporate more advance components to accurately control and adjust the load throughout the various phases of a test. For soft samples, this can be important because the applied load will fluctuate and change as the sample deforms and becomes stiffer during the test. Marr et al. (2003) recommend the use of a proportional-integral-derivative (PID) controller to automatically track and change system loading control parameters as the stiffness of the specimen changes during the test. Pneumatic load systems have been used in the past; however, the frequency of load pulse for these systems cannot be controlled as accurately as servohydraulic systems. AASHTO standards (T-307) were modified to accommodate pneumatic load systems by extending the relaxation period from 1 sec to 3.1 sec.

The triaxial cell is used to contain the specimen and the confining fluid during a test. A cross-section of a triaxial cell with typical components is shown in Figure 13. Vertical displacements of the sample are electronically measured throughout the test using linear variable displacement transducers (LVDTs). The latest AASHTO test standard (T-307) indicates the LVDTs should be positioned outside the triaxial cell to measure deformations (as shown in Figure 13), rather than inside the cell as was done in the past (as described in AASHTO T-274). Hydraulic pumps are used to apply the cell pressure, which is measured and controlled using pressure transducers.

There is no standard off-the-shelf RLT that is widely used or recognized by professionals in the testing industry. It appears that most research agencies and DOTs that use RLT testing have assembled and developed systems using components from a variety of vendors and suppliers. No two systems are identical. The servo-hydraulic loading system is the most expensive component and the two most commonly used systems appear to be manufactured by either the MTS Corporation or the Instron Corporation. Capital costs for building a fully operational RLT testing system can vary over a range as wide as \$100,000 to \$350,000 depending on the flexibility of the system and the level of sophistication of components, instrumentation, and system control mechanisms.

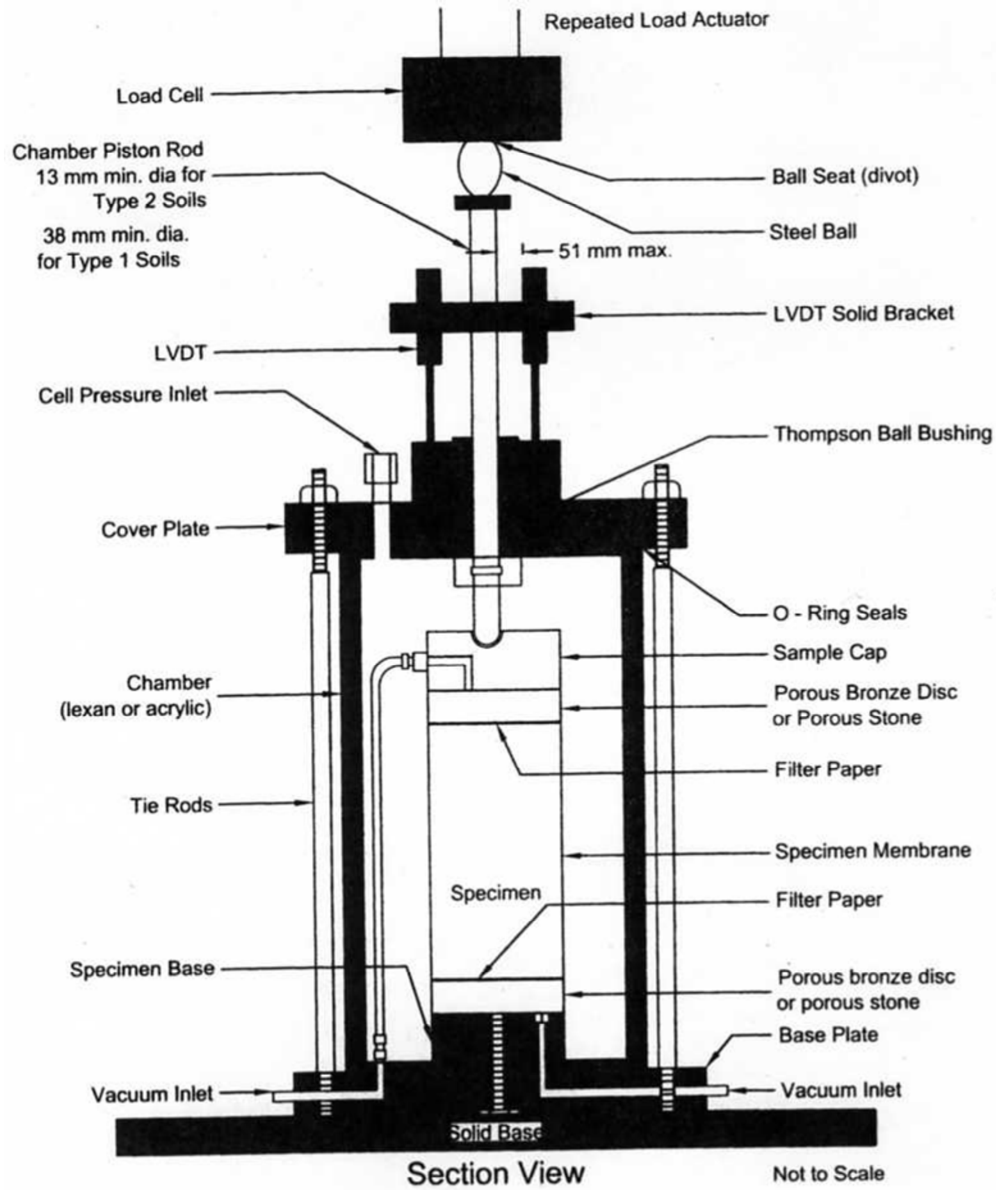

Figure 13. Cross-section of typical triaxial setup (from AASHTO T307).

4.8 Summary

An extensive survey conducted in 2007 indicates that the majority of state DOTs use indirect methods for estimating M_r, rather than direct laboratory or field methods (Puppala 2008). The recent survey by Puppala (2008) indicates that the limited use of RLT tests for obtaining direct measurements of M_r is partly attributed to a relatively poor view and limited satisfaction of the test by state DOTs. Several reasons have been given by state DOTs for this generally negative impression of the RLT test for determining M_r, including: constant modification of test procedures, measurement difficulties, design-related issues, overly complex test, and poor repeatability of results.

Several correlation equations obtained from the literature were evaluated in this chapter using data from two MDT soil survey reports. Preliminary results from this evaluation indicate there is little to no consistency between equations for predicting M_r from soil index and classification properties. Most of these equations were developed from relatively small sample sets and often for region-specific soil types. Until a more detailed assessment is conducted, the authors discourage the general use of any correlation equation without prior testing and verification of the suitability and reliability of the equation for use in pavement design.

Based on our sensitivity analysis of the AASHTO (1993) design guide equation, it was observed that the calculated value of SN is most sensitive to small changes in input values when the subgrade is relatively soft and the traffic volumes are relatively low. From a practical perspective, this presents difficulties to the designer because the soil types most likely to have low R-values fall in the A-4, A-6, and A-7 AASHTO classification categories. These are also the most difficult soils to obtain accurate and repeatable R-values and CBR measurements in the laboratory due to complications in testing soft fine-grained soils at relatively high water contents.

5 CONCLUSIONS AND RECOMMENDATIONS

5.1 Summary and Conclusions

The widely used AASHTO empirical pavement design guide and the new mechanistic-empirical pavement design procedure characterize the stiffness of the subgrade and unbound base layers in terms of the resilient modulus. At this writing, there is no widely accepted in-situ or laboratory test procedure to measure resilient modulus. While the repeated load triaxial test is recommended by AASHTO as a test method for determining resilient modulus, the majority of state transportation agencies do not use the test. This report summarizes the historical development of the RLT test and describes the primary advantages and disadvantages of the test. Other more common test procedures for estimating M_r such as R-value, CBR, and FWD are also described.

The RLT laboratory test method was designed to determine the resilient modulus of a soil sample; however, the test protocol has changed several times since it was first introduced in the 1980s. A detailed comparison of eight versions of the test protocol is provided in Appendix A. All of these methods have been used at one time or another for both research and design purposes. Consequently, databases and research publications that contain RLT test results may not always be comparable, especially for establishing correlations with other soil properties.

Numerous approaches are described in the literature for relating M_r to more readily measured soil parameters using index and strength tests. Most M_r correlation equations were developed using regression analyses in which RLT resilient modulus test results were compared to the results of less expensive or more routine laboratory tests, such as R-value, CBR, unconfined compression, and index property tests. The Falling Weight Deflectometer is the most widely used in-situ test for subgrade characterization of existing roads. A number of other in-situ methods have been used with limited success at estimating M_r, including: dynamic cone penetrometer, miniature cone penetrometer, plate load, in-situ CBR, and dilatometer tests.

Over 30 different correlation equations were reviewed in this study. Selected equations were further examined using data from two MDT soil survey reports. This evaluation indicated there is little to no consistency between equations for predicting M_r using soil index and classification properties. Most of the equations were developed from relatively small sample sets and often for region-specific soil types. Until a more detailed assessment is conducted, the authors discourage the general use of any correlation equation without prior testing and verification of the suitability and reliability of the equation for use in pavement design.

5.2 Recommendations

Based on the review of literature and the preliminary analyses conducted in this study, the authors conclude that full scale implementation of a repeated load triaxial testing program for the determination of M_r on a routine project basis may not be the most cost effective approach for MDT. Rather, the authors recommend that additional evaluation of MDT soil survey data be conducted to identify potentially useful correlation equations and to identify the soil parameters that may be most denotative of soil stiffness. A subsequent phase of focused RLT testing could then be conducted in an efficient manner to measure M_r for specific soil types and to verify the suitability and applicability of previously identified correlation equations.

The evaluation of correlation equations presented in Chapter 4 should be viewed as a preliminary starting point. The evaluation was conducted to illustrate an approach that could be explored further using a larger database of measured soil properties that are regionally specific to MDT projects. We recommend an analysis of this kind be conducted on data from a range of soil types using soil survey reports obtained from projects around the state. The data could be evaluated to discern the potential usefulness of selected existing correlation equations or to possibly develop a new equation or set of equations specifically for Montana soils.

We suggest examining 10 to 20 additional soil survey reports. The optimum number of reports for the follow-on study will depend on the type of tests that were conducted in support of the original investigation and the range of soil types that were tested. Less assumptions are necessary in the calculations when more test data is available for a given soil sample. To minimize assumptions in the calculations, the ideal soil sample would include the following suite of tests: maximum Proctor dry density and optimum water content, percent passing the number 200 sieve, specific gravity, R-value, natural water content, liquid limit, plastic limit, and AASHTO soil classification. A few of the correlation equations require input of the percent clay or percent silt in the sample. These measurements would be useful; however, this would require hydrometer testing, which MDT does not currently conduct as part of the soil survey investigation.

We hypothesize that more than one correlation equation may be necessary for estimating M_r. Additional data analysis could provide insight regarding the primary metric for subdividing correlation calculations into two or more equations, if necessary. The primary metric may involve one of the following divisions:

- predominately coarse-grained and predominately fine-grained,
- cohesive (plastic) fines and noncohesive (nonplastic) fines,
- relatively low R-value and relatively high R-value,
- multiple equations based on AASHTO soil classification, and
- multiple equations based on layer thickness or overburden pressure.

Other divisions could be explored; the items listed above are most commonly cited in the literature.

An extensive number of correlation equations have been developed over the past 20 years and new equations continue to appear on a rather frequent basis. We believe it would be prudent to conduct additional analyses of existing data before pursuing an extensive RLT testing program. Additional analyses will help narrow the field of equations and focus subsequent testing programs on specific soil types and soil parameters.

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APPENDIX A – COMPARISON OF RLT TEST PROTOCOLS

	AASHTO T274-82	*AASHTO T292-91*	*AASHTO T294-92 (SHRP P46)*	*AASHTO T P46-94*	*LTPP Protocol P46 (1996)*	*AASHTO T307-99 (2003)*	*NCHRP 1-28*	*NCHRP 1-28A*
Status	withdrawn	withdrawn	withdrawn	withdrawn		active	proposed AASHTO standard in 1997	proposed AASHTO standard in 2004
Reference	AASHTO T274-82	Witczak 2000	Witczak 2000	Witczak 2000	FHWA 1996	AASHTO T307	Barksdale et al. 1997	NCHRP 2004
Loading Device	Any device capable of providing varying repeated loads in fixed cycles of load and release	Any device capable of providing varying repeated loads in fixed cycles of load and release	Closed loop electro-hydraulic	Top-loading closed loop electro-hydraulic testing machine with a function generator capable of applying repeated cycles of haversine shaped load pulse (0.1 sec loading time, 0.9 second rest time)	Top-loading closed loop electro-hydraulic testing machine with a function generator capable of applying repeated cycles of haversine shaped load pulse (0.1 sec loading time, 0.9 second rest time)	Top-loading closed loop electro-hydraulic testing machine with a function generator capable of applying repeated cycles of haversine shaped load pulse (0.1 sec. loading time, 0.9 sec. rest time) or electro-pneumatic testing machine (load pulse 0.1 sec., 0.9-3.0 sec. rest)	Top-loading closed loop electro-hydraulic testing machine with a function generator capable of applying repeated cycles of haversine shaped load pulse (0.1 sec loading time, 0.9 second rest time)	Same as NCHRP 1-28
Triaxial Chamber	Fluid: air, water, or water/alcohol mixture	Fluid: air	Fluid: air	•Fluid: air •Triaxial chamber made of see-through material	•Fluid: air •Triaxial chamber made of see-through material	•Fluid: air •Triaxial chamber made of see-through material	•Fluid: air •Triaxial chamber made of see-through material •Triaxial chamber used only for cohesionless materials	•Fluid: air •Triaxial chamber made of see-through material
Data Acquisition	•Suitable excitation, conditioning, and recording equipment •Simultaneous recording of axial load and deformation •Record average of deformation	•Strip chart or computer •Simultaneous recording of axial load and deformation •Do not use filters that attenuate signals	•LVDTs monitored separately •Simultaneous recording of axial load and deformation •Filters should have a frequency that cannot attenuate the signal	•Simultaneous recording of axial load and deformation •Filters should have a frequency which cannot attenuate the signal •Minimum 200 data points for each of the two LVDTs per load cycle •The sensors shall be wired so that each transducer is read and the results are reviewed independently	•Simultaneous recording of axial load and deformation •Filters should have a frequency which cannot attenuate the signal •Minimum 500 data points for each of the two LVDTs per load cycle •The sensors shall be wired so that each transducer is read and the results are reviewed independently	•Simultaneous recording of axial load and deformation •Filters should have a frequency which cannot attenuate the signal •Minimum 200 data points for each of the two LVDTs per load cycle •The sensors shall be wired so that each transducer is read and the results are reviewed independently	•Analog to digital data acquisition system required •Automatic data reduction •25 µs A/D/ conversion time •12 bit resolution •Minimum 200 data points for each of the two LVDTs per load cycle •The sensors shall be wired so that each transducer is read and the results are reviewed independently	Same as NCHRP 1-28
Types of Materials	**Granular** •Scalp off material greater than 0.75 in. **Cohesive** •A-2-6, A-2-7, A-6,	**Granular** •A-1, A-2-4, A-2-5, A-3 •For PI<6: A-4, A-5 •Scalp off	**Material Type 1** •All unbound granular bae/subbase material •All untreated	**Material Type 1** •All untreated granular base/subbase materials and all untreated subgrade soils which meet the criteria of less	**Material Type 1** •All untreated granular base/subbase materials and all untreated subgrade soils which meet the criteria of less	**Material Type 1** •All untreated granular base/subbase materials and all untreated subgrade soils which meet the criteria of less than 70% passing the No. 10 sieve	**Material Type 1** •All unbound granular base and subbase material and all untreated subgrade soils which meet the criteria of less than 70% passing the No. 10 sieve	**Material Type 1** •Includes all unbound granular base and subbase materials and all untreated subgrade soils with maximum particle sizes greater than 0.375 in.

	AASHTO T274-82	*AASHTO T292-91*	*AASHTO T294-92 (SHRP P46)*	*AASHTO T P46-94*	*LTPP Protocol P46 (1996)*	*AASHTO T307-99 (2003)*	*NCHRP 1-28*	*NCHRP 1-28A*
	A-7	material greater than 1.5 in. **Cohesive** •A-2-6, A-2-7, A-6, A-7 •For PI>6: A-4, A-5	subgrade material for which less than 70% passes the No. 10 sieve and less than 20% passes the No. 200 sieve •A-1-a •May include: A-1-b, A-2, A-3 •If more than 5% of a sample is retained on the 1.25-in. sieve, the specimen shall not be tested, otherwise remove all material greater than 1.25 in **Material Type 2** •All untreated subgrade material not meeting the above criteria •Thin-walled tube samples of untreated subgrades •A-4, A-5, A-6, A-7 •May include: A-1-b, A-2, A-3	than 70% passing the No. 10 sieve and less than 20% passing the No. 200 sieve and which have PI<10 •Due to edge effects, all material greater than 1.5 in. shall be scalped off prior to testing **Material Type 2** •All untreated granular base/subbase materials and all untreated subgrade soils not meeting the criteria for material Type I •Due to edge effects, all material greater than 0.5 in. shall be scalped off prior to testing	than 70% passing the No. 10 sieve and less than 20% passing the No. 200 sieve and which have PI≤10 •Due to edge effects, all material greater than 1.5 in. shall be scalped off prior to testing **Material Type 2** •All untreated granular base/subbase materials and all untreated subgrade soils not meeting the criteria for material Type I •Due to edge effects, all material greater than 0.5 in. shall be scalped off prior to testing	and less than 20% passing the No. 200 sieve and which have PI<10 •Due to edge effects, all material greater than 1.5 in. shall be scalped off prior to testing **Material Type 2** •All untreated granular base/subbase materials and all untreated subgrade soils not meeting the criteria for material Type I •Due to edge effects, all material greater than 0.5 in. shall be scalped off prior to testing	and less than 20% passing the No. 200 sieve and which have PI<10 •**Type 1a** material shall have 100% passing the 1.5-in. sieve •**Type 1b** material shall have 100% passing the 1.0-in. sieve •If 10% or less of a Type 1a sample is retained on the 1.5-in. sieve, the material greater than 1.5 in. shall be scalped off and replaced by 1.0-1.5 in. material prior to testing **Material Type 2** •All unbound granular base/subbase and untreated subgrade soils not meeting the criteria for material Type 1 •If 10% or less of a Type 2 sample is retained on the 0.5-in. sieve, the material greater than 0.5 in. shall be scalped off and replaced by 0.375-0.5 in. material prior to testing	•All material greater than 1.0 in. shall be scalped off prior to testing. **Material Type 2** •Includes all unbound granular base and subbase materials and all untreated subgrade soils with maximum particle sizes less than 0.375 in. and less than 10% passing No. 200 sieve. **Material Type 3** •Includes all untreated subgrade soils with maximum particle size less than 0.375 in. and more than 10% passing No. 200 sieve **Material Type 4** •Includes thin-walled tube samples of untreated subgrade soils

	AASHTO T274-82	AASHTO T292-91	AASHTO T294-92 (SHRP P46)	AASHTO T P46-94	LTPP Protocol P46 (1996)	AASHTO T307-99 (2003)	NCHRP 1-28	NCHRP 1-28A
Specimen Preparation Types of Materials	•Minimum specimen diameter is 2.8 in or six times maximum particle size •For compaction select the method that best simulates field conditions (gyratory, kneading, static, vibratory)	•Minimum specimen diameter is 2.8 in. •For compaction select the method that best simulates field conditions (impact, kneading, static, vibratory)	•Minimum specimen diameter is 2.8 in. or five times the nominal particle size •2.8-in. diameter undisturbed specimen from thin walled tube samples used for cohesive subgrade soils (Material Type 2) •2.8-in. diameter mold used to reconstitute Type 2 test specimens •4-in. diameter split molds used to reconstitute Type 1 soils when the nominal particle size does not exceed 0.75 in. •6-in. diameter split mold used to prepare Type 1 materials with nominal particle sizes between 0.75 and 1.25 in. without removing any coarse aggregate •Type 1 soils: vibratory compacted •Cohesionless soils: compacted with small hand-held air hammer •Type 2 soils: compacted by static loading	•Soils classified as Type 1 will be molded in 6-in. diameter mold; recompacted using split molds and vibratory compaction •Remolded Type 2 specimens will be compacted in a 2.8-in. diameter mold; static compaction •Cohesionless soils shall be compacted in 6 lifts in a split mold mounted on the base of the triaxial cell; vibratory impact hammer without kneading action	•Soils classified as Type 1 will be recompacted in 6-in. diameter 12-in. tall split mold in 6 lifts using vibratory compaction •Remolded Type 2 specimens will be compacted in a 2.8-in. diameter 6- or 6.5-in. tall mold using static compaction	•Soils classified as Type 1 will be molded in 6-in. diameter mold; recompacted using split molds and vibratory compaction •Remolded Type 2 specimens will be compacted in a 2.8-in. diameter mold with static compaction or a 2.8-in.-plus diameter mold with kneading compaction •Soils should be compacted to in-place wet density if known, otherwise max dry density and optimum moisture (standard or modified Proctor)	•Materials classified as Type 1a should be molded in a 6-in. diameter mold •Materials classified as Type 1b can be molded in either 4- or 6-in. diameter mold •Use 4-in. diameter molds for maximum particle size less than 1.0 in. •Type 1 materials will be recompacted using split molds and vibratory compaction •Cohesionless soils shall be compacted in 6 lifts in a split mold mounted on the base of the triaxial cell; vibratory impact hammer without kneading action •Remolded Type 2 specimens can be compacted in either 2.8- or 4.0-in. diameter molds. The compaction method should simulate the field conditions.	•Type 1 materials use 6- or 4-in. diameter molds; impact or vibratory compaction •Type 2 materials use 4-in. diameter molds; vibratory compaction •Type 3 materials use 4-in. diameter molds; impact or kneading compaction •Type 4 materials are undisturbed 2.8-in. diameter samples

	AASHTO T274-82	*AASHTO T292-91*	*AASHTO T294-92 (SHRP P46)*	*AASHTO T P46-94*	*LTPP Protocol P46 (1996)*	*AASHTO T307-99 (2003)*	*NCHRP 1-28*	*NCHRP 1-28A*
Axial Load	Electronic load cell, preferably located inside the triaxial chamber; provide corrections if externally mounted	Electronic load cell located inside the triaxial cell (on top of specimen)	Electronic load cell located inside the triaxial cell	Electronic load cell located between the actuator and the chamber piston rod (outside triaxial cell)	Electronic load cell located between the actuator and the chamber piston rod (outside triaxial cell)	Electronic load cell located between the actuator and the chamber piston rod (outside triaxial cell)	Electronic load cell located inside the triaxial cell; non-fatigue rated cell recommended	Same as NCHRP 1-28
Axial Deformation	•2 LVDTs - clamped to specimen if M_r>15 ksi - externally mounted if M_r<15 ksi	•Clamps (internally) mounted LVDTs •Clamps mounted in the 1/4 position •Use spring-loaded LVDTs •If the specimen is soft enough to be damaged by clamps or slippage of clamps is suspected, use externally mounted LVDTs (on the loading piston)	•Externally mounted LVDTs	•Externally mounted spring loaded LVDTs •An acceptable displacement ratio is defined as $R=Y_{max}/Y_{min}$ (Y_{max} equals the larger of the two measurement values and Y_{min} the smaller)	•Externally mounted spring loaded LVDTs	•Externally mounted spring loaded LVDTs	•Optical extensometer, •Non-contact sensors, or •Clamps (internally) mounted LVDTs - Clamps mounted in the 1/4 position - Use spring loaded LVDTs - If the specimen is soft enough to be damaged by clamps or slippage of clamps is suspected use top-bottom platen measurements	•Optical extensometer, Non-contact sensors, or Clamps (internally) mounted LVDTs • If choose LVDTs: - Clamps mounted in the 1/4 position - Use spring loaded LVDTs - If the specimen is soft enough to be damaged by clamps or slippage of clamps is suspected use top-bottom platen measurements
Load Pulse	• 0.1 load pulse and cycle duration of 1 to 3 seconds; haversine, rectangular or triangular shapes may be used	•Fixed load duration 0.1, 0.15, 1.0, 0.5 sec.; haversine, rectangular or triangular shapes may be used	•Haversine shaped 0.1 sec. load pulse followed by 0.9 sec. rest period	Same as AASHTO T294-92	Same as AASHTO T294-92	•Haversine shaped 0.1 sec. load pulse followed by 0.9 sec. rest period (or 0.1 sec load pulse followed by 0.9 to 3.0 sec. rest period for pneumatic systems)	•Haversine shaped load pulse - Use 0.1 sec load pulse, 0.9 sec rest period except for non-plastic granular materials may use 0.15 sec. load pulse and/or 0.4 sec. rest period to shorten testing time	•Haversine shaped load pulse - Base/Subbase materials use 0.1 sec load pulse, 0.9 sec rest period - Subgrade materials use 0.2 sec load pulse, 0.8 sec rest period - For non-plastic granular materials may use 0.15 sec. load pulse and/or 0.4 sec. rest period to shorten testing time

	AASHTO T274-82	AASHTO T292-91	AASHTO T294-92 (SHRP P46)	AASHTO T P46-94	LTPP Protocol P46 (1996)	AASHTO T307-99 (2003)	NCHRP 1-28	NCHRP 1-28A
Stress Sequence 1 v/w,x,y,…/z where x=confining stress (psi), y=deviatoric stress (psi), z=number of load repetitions for each deviatoric stress	**Cohesive soils** •Conditioning: 6/1, 2, 4, 8, 10/200 •Testing: 6/1, 2, 4, 8, 10/200 3/1, 2, 4, 8, 10/200 0/1, 2, 4, 8, 10/200	**Cohesive soils** •Conditioning: 3/3/1000 •Testing: 3/3, 5, 7, 10, 15/50 - stress ranges should be selected to cover the expected in-service range - To determine the number of repetitions necessary, compare the recoverable axial deformations at the 20th and 50th repetitions. If the difference is greater than 5% apply an additional 50 repetitions at this stress state.	**Type 1 Soils** •Conditioning: 15/15/1000 •Testing: 3/3, 6, 9/100 5/5, 10, 15/100 10/10, 20, 30/100 15/10, 15, 30/100 20/15, 20, 40.8/100 - If at any time the permanent strain of the sample exceeds 10%, stop the test and report the results on an appropriate worksheet	**Subgrade Soils** •Conditioning: 6/4/500 - if the sample is still decreasing in height at the end of the conditioning process, stress cycling should be continued up to 1000 repetitions prior to testing - if the vertical permanent strain reaches 5%, during conditioning then the conditioning process shall be terminated (inadequate compaction) •Testing: 6/2, 4, 6, 8, 10/100 3/2, 4, 6, 8, 10/100 0/2, 4, 6, 8, 10/100 - if at any time the permanent strain of the sample exceeds 5% stop the test and report the result on an appropriate worksheet - After the completion of the test, if the total vertical permanent strain did not exceed 5%, continue with a quick shear test procedure	**Subgrade Soils** •Conditioning: 6/4/500 - if the sample is still decreasing in height at the end of the conditioning process, stress cycling should be continued up to 1000 repetitions prior to testing - if the vertical permanent strain reaches 5%, during conditioning then the conditioning process shall be terminated (inadequate compaction) •Testing: 6/2, 4, 6, 8, 10/100 4/2, 4, 6, 8, 10/100 2/2, 4, 6, 8, 10/100 - if at any time the permanent strain of the sample exceeds 5% stop the test and report the result on an appropriate worksheet - After the completion of the test, if the total vertical permanent strain did not exceed 5%, continue with a quick shear test procedure	**Subgrade Soils (Type 1 or 2)** •Conditioning: 6/4/500 - if sample height still decreasing at 500 repetitions apply up to 500 more repetitions - if vertical permanent strain exceeds 5%, terminate and discard sample (inadequate compaction) •Testing: 6/2, 4, 6, 8, 10/100 4/2, 4, 6, 8, 10/100 2/2, 4, 6, 8, 10/100 - terminate test if vertical permanent strain exceeds 5% and report results - if permanent strain <5% at end of test, perform quick shear test	**Unconfined Test for Cohesive Subgrade Soils** - includes all undisturbed or laboratory compacted specimens of cohesive subgrade soils (PI>10; hold together during test) - soil should exhibit stress softening characteristics - unconfined test - for stiff to very stiff specimens (undrained shear strength>750psf) axial deformation should preferably be measured either directly on the specimen or else between the solid end platens using grouted specimen ends - for soft specimens do not use clamps; grouting is not needed if the measured resilient modulus is less than 10ksi •Conditioning: 0/4/200 - if the vertical permanent strain reaches 5% during conditioning then the conditioning process shall be terminated (inadequate compaction) •Testing: 0/2, 4, 6, 8, 10/50 - after completion of the test, perform the rapid shear test with no confining pressure	**Cohesive Subgrade Soils (Procedure II)** - includes all undisturbed specimens of cohesive subgrade soils and all lab-compacted subgrade soils with percent passing No. 200 sieve greater than 35% - Haversine shaped load of 0.2 sec load pulse, 0.8 sec rest period •Conditioning: 4/7/1000 - if the vertical permanent strain reaches 5% during conditioning then the conditioning process shall be terminated (inadequate compaction) •Testing: 8/4, 7, 10, 14/100 6/4, 7, 10, 14/100 4/4, 7, 10, 14/100 2/4, 7, 10, 14/100 - if at any time the total permanent strain of the sample exceeds 5% then stop the test and report the results on an appropriate worksheet

<table>
<tr><th></th><th>AASHTO T274-82</th><th>AASHTO T292-91</th><th>AASHTO T294-92 (SHRP P46)</th><th>AASHTO T P46-94</th><th>LTPP Protocol P46 (1996)</th><th>AASHTO T307-99 (2003)</th><th>NCHRP 1-28</th><th>NCHRP 1-28A</th></tr>
<tr><td>Sress Sequence 2</td>
<td>Granular Soils
•Conditioning:
5/5, 10/200
10/10, 15/200
15/15, 20/200
•Testing:
20/1, 2, 5, 10, 15, 20/200
15/1, 2, 5, 10, 15, 20/200
10/1, 2, 5, 10, 15/200
5/1, 2, 5, 10, 15/200
1/1, 2, 5, 7.5, 10/200</td>
<td>Granular Subgrades
•Conditioning:
15/12/1000
•Testing:
15/7, 10, 15/50
10/5, 7, 10, 15/50
5/3, 5, 7, 10/50
2/3, 5, 7/50
- stress ranges should be selected to cover the expected in-service range
- To determine the number of repetitions necessary, compare the recoverable axial deformations at the 20th and 50th repetitions. If the difference is greater than 5% apply an additional 50 repetitions at this stress state.</td>
<td>Type 2 Soils
•Conditioning:
6/4/1000
•Testing:
6/2, 4, 6, 8, 10/100
3/2, 4, 6, 8, 10/100
0/2, 4, 6, 8, 10/100
- If at any time the permanent strain of the sample exceeds 10%, stop the test and report the results on an appropriate worksheet</td>
<td>Base/Subbase Materials
•Conditioning:
15/15/500
- if the sample is still decreasing in height at the end of the conditioning process, stress cycling should be continued up to 1000 repetitions prior to testing
- if the vertical permanent strain reaches 5%, during conditioning then the conditioning process shall be terminated (inadequate compaction)
•Testing:
3/3, 6, 9/100
5/5, 10, 15/100
10/10, 20, 30/100
15/10, 15, 30/100
20/15, 20, 40.8/100
- if at any time the permanent strain of the sample exceeds 5% stop the test and report the result on an appropriate worksheet
- After the completion of the test, if the total vertical permanent strain did not exceed 5%, continue with the quick shear test procedure or permanent deformation test</td>
<td>Base/Subbase Materials
•Conditioning:
15/15/500
- if the sample is still decreasing in height at the end of the conditioning process, stress cycling should be continued up to 1000 repetitions prior to testing
- if the vertical permanent strain reaches 5%, during conditioning then the conditioning process shall be terminated (inadequate compaction)
•Testing:
3/3, 6, 9/100
5/5, 10, 15/100
10/10, 20, 30/100
15/10, 15, 30/100
20/15, 20, 40/100
- if at any time the permanent strain of the sample exceeds 5% stop the test and report the result on an appropriate worksheet
- After the completion of the test, if the total vertical permanent strain did not exceed 5%, continue with the quick shear test procedure</td>
<td>Base/Subbase Soils (Type 1 or 2)
•Conditioning:
15/15/500
- if sample height still decreasing at 500 repititions apply up to 500 more repititions
- if vertical permanent strain exceeds 5%, terminate and discard sample (inadequate compaction)
•Testing:
3/3, 6, 9/100
5/5, 10, 15/100
10/10, 20, 30/100
15/10, 15, 30/100
20/15, 20, 40/100
- terminate test if vertical permanent strain exceeds 5% and report results
- if permanent strain <5% at end of test then perform quick shear test if desired</td>
<td>Granular and Low Cohesion Subgrade Soils
- includes all Type 1 subgrade materials and Type 2 with PI<10
•Conditioning:
6/8/500
- if the sample is still decreasing in height at the end of the conditioning process, stress cycling should be continued up to 1000 repetitions prior to testing
- if the vertical permanent strain reaches 5% during conditioning then the conditioning process shall be terminated (inadequate compaction)
•Testing:
2/2, 3, 4/50
3/3, 4, 6/50
4/4, 6, 8/50
6/4, 6, 8/50
- if at any time the permanent strain of the sample exceeds 5%, stop the test and report the result on an appropriate worksheet
- after completion of the test, if the total vertical permanent strain did not exceed 5%, continue with the quick shear test procedure</td>
<td>Coarse-Grained Subgrade Soils (Procedure Ib)
- includes all lab-compacted subgrade soils with percent passing No. 200 sieve less than 35%
- Haversine shaped load of 0.2 sec load pulse, 0.8 sec rest period

•Conditioning:
4/8/1000
- if the vertical permanent strain reaches 5% during conditioning then the conditioning process shall be terminated (inadequate compaction)
•Testing:
2/1, 2, 4, 6/100
4/2, 4, 8, 12/100
6/3, 6, 12, 18/100
8/4, 8, 16, 24/100
12/6, 12, 24, 36/100
- if at any time the total permanent strain of the sample exceeds 5% then stop the test and report the results on an appropriate worksheet</td></tr>
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	AASHTO T274-82	AASHTO T292-91	AASHTO T294-92 (SHRP P46)	AASHTO T P46-94	LTPP Protocol P46 (1996)	AASHTO T307-99 (2003)	NCHRP 1-28	NCHRP 1-28A
Stress Sequence 3		**Granular Base/Subbase Materials** •Conditioning: 20/15/1000 •Testing: 20/10, 20, 30, 40/50 15/10, 20, 30, 40/50 10/5, 10, 20, 30/50 5/5, 10, 15/50 3/5, 7, 9/50 - stress ranges should be selected to cover the expected in-service range - To determine the number of repetitions necessary, compare the recoverable axial deformations at the 20th and 50th repetitions. If the difference is greater than 5% apply an additional 50 repetitions at this stress state.					**Base/Subbase Materials** - includes all base and subbase materials •Conditioning: 15/15/200 - if the sample is still decreasing in height at the end of the conditioning process, stress cycling should be continued up to 1000 repetitions prior to testing - if the vertical permanent strain reaches 5% during conditioning then the conditioning process shall be terminated (inadequate compaction) •Testing: 3/3, 6, 9/50 4.5/4.5, 9, 13.5/50 6/6, 12, 18/50 9/9, 18, 27/50 14/9, 14, 28/50 - if at any time the permanent strain of the sample exceeds 5%, stop the test and report the result on an appropriate worksheet - after completion of the test, if the total vertical permanent strain did not exceed 5%, continue with the quick shear test procedure or permanent deformation test	**Base/Subbase Materials (Procedure Ia)** - includes all unbound granular base and subbase materials - Haversine shaped load of 0.1 sec load pulse, 0.9 sec rest period •Conditioning: 15/30/1000 - if the vertical permanent strain reaches 5% during conditioning then the conditioning process shall be terminated (inadequate compaction) •Testing: 3/1.5, 3, 6, 9, 15, 21/100 6/3, 6, 9, 12, 18, 30, 42/100 10/5, 10, 20, 30, 50, 70/100 15/7.5, 15, 30, 45, 75, 105/100 20/10, 20, 40, 60, 100, 140/100 - if at any time the total permanent strain of the sample exceeds 5% then stop the test and report the results on an appropriate worksheet

	AASHTO T274-82	***AASHTO T292-91***	***AASHTO T294-92 (SHRP P46)***	***AASHTO T P46-94***	***LTPP Protocol P46 (1996)***	***AASHTO T307-99 (2003)***	***NCHRP 1-28***	***NCHRP 1-28A***
Predictive Equation	Calculate M_r for each stress sequence combination: $M_r = \frac{\sigma_d}{\varepsilon_r}$	•Cohesive: $M_r = k_1(\sigma_d)^{k_2}$ •Granular: $M_r = k_1(\theta)^{k_2}$ $\theta = \sigma_1 + \sigma_2 + \sigma_3$ k_i = regression constants	•Cohesive: $M_r = k_1(\sigma_d)^{k_2}$ •Granular: $M_r = k_1(\theta)^{k_2}$ $\theta = \sigma_1 + \sigma_2 + \sigma_3$ k_i = regression constants	None			$M_r = \frac{\sigma_{cyclic}}{\varepsilon_r}$ $\varepsilon_r = k_1(\sigma_{cyclic})^{k_2}(\sigma_3)^{k_3}$ k_i = regression constants	$M_r = k_1(p_a)\left(\frac{\theta - 3k_6}{p_a}\right)^{k_2}\left(\frac{\tau_{oct}}{p_a} + k_7\right.$ $\theta = \sigma_1 + \sigma_2 + \sigma_3$ $\tau_{oct} = \frac{1}{3}\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_1 - \sigma_3)^2 (\sigma}$ p_a = atmospheric pressure k_i = regression constants

Symbol	**Definition**
σ_d	deviatoric stress
ε_r	resilient strain
$k_1, k_2, k_3, ...k_i$	regression model constants
θ	bulk stress = $\sigma_1 + \sigma_2 + \sigma_3$
σ_1	major principal stress
σ_2	intermediate principal stress
σ_3	minor principal stress
τ_{oct}	octahedral shear stress = $1/3[(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2]^{1/2}$
σ_{cyclic}	cyclic normal stress
p_a	atmospheric pressure
LVDT	Linear Variable Differential Transducer
PI	Plasticity Index

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