

**SELECTION OF SUBGRADE MODULUS FOR
PAVEMENT OVERLAY DESIGN PROCEDURES**

by

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Disclaimer

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Preface

This report describes a study jointly conducted by the University of Wyoming and the Wyoming Department of Transportation to better understand how selecting a M_R value influences the thickness of an asphalt overlay pavement. The objectives of this study were to: 1) investigate the importance of several fundamental soil properties (water content, plasticity index, liquid limit, group index) on selecting a design subgrade resilient modulus value for cohesive soils; 2) define the actual relationship (correction factor) between back calculated and laboratory based M_R values for typical cohesive subgrade soils in Wyoming; 3) compare actual subgrade field deviator stresses to the deviator stress assumed in determining a design M_R value from laboratory testing; and 4) determine the effect of selecting a M_R value on the design overlay thicknesses for typical pavement sections in Wyoming. The data analysis resulted in several important conclusions about factors that influence the determination of the subgrade resilient modulus value and how this value affects the final design overlay thickness for a given pavement section.

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TABLE OF CONTENTS

	<u>Page</u>
CHAPTER 1: INTRODUCTION.....	1
BACKGROUND.....	1
PROBLEM STATEMENT.....	2
OBJECTIVES.....	2
ORGANIZATION OF STUDY.....	3
CHAPTER 2: LITERATURE REVIEW.....	5
INTRODUCTION.....	5
TRADITIONAL SUBGRADE TESTING PROCEDURES.....	6
California Bearing Ratio (CBR).....	6
Resistance Value (R-Value).....	8
DEVELOPMENT OF THE RESILIENT MODULUS TEST.....	10
RESILIENT MODULUS LABORATORY TESTING.....	12
BACK CALCULATION OF RESILIENT MODULUS.....	13
Non-Destructive Testing Equipment.....	15
Back Calculation Computer Programs.....	17
M _R DETERMINATION FROM CORRELATION STUDIES.....	20
SELECTION OF A DESIGN M _R VALUE.....	22
UTILIZATION OF SOIL M _R IN THE AASHTO OVERLAY DESIGN PROCEDURES.....	24

The 1993 AASHTO Overlay Design Procedure	24
Determining the Need for an Overlay.....	26
CHAPTER SUMMARY	29
CHAPTER 3: DESIGN OF EXPERIMENT.....	32
INTRODUCTION.....	32
SITE SELECTION	32
DATA COLLECTION	32
LABORATORY TESTING AND RESILIENT MODULUS DETERMINATION.....	35
Laboratory Testing for Resilient Modulus	36
Back Calculation of M_R	39
DATA BASE PREPARATION AND DATA ANALYSIS	40
EVALUATING THE EFFECT OF M_R SELECTION ON OVERLAY THICKNESSES	40
CHAPTER SUMMARY	41
CHAPTER 4: RESULTS FROM LABORATORY AND FIELD EVALUATIONS.....	43
INTRODUCTION.....	43
SITE CHARACTERISTICS	43
RESULTS FROM SOIL PROPERTY TESTS	44
LABORATORY RESILIENT MODULUS VALUES BASED ON 41.4-kPa (6-psi) DEVIATOR STRESS.....	47
BACK CALCULATED RESILIENT MODULUS VALUES.....	49

RESULTS FROM R-VALUE TESTS	52
STATISTICAL ANALYSIS	52
Relationship Between Resilient Modulus and R-Value	56
The Effect of Sensor Locations on M_R Measurements.....	58
The Effect of Sample Locations on M_R Values.....	59
Relationship Between Back Calculated and Laboratory M_R Values	59
Relationship Between M_R Values and Soil Properties.....	60
CHAPTER SUMMARY	62
CHAPTER 5: EFFECT OF M_R SELECTION ON OVERLAY THICKNESSES	65
INTRODUCTION.....	65
LABORATORY RESILIENT MODULUS VALUES BASED ON ACTUAL FIELD STRESSES	65
Comparison of Laboratory M_R Values.....	67
OVERLAY THICKNESS RESULTS	69
STATISTICAL ANALYSIS	73
CHAPTER SUMMARY	78
CHAPTER 6: CONCLUSIONS AND RECOMMENDATIONS.....	79
SUMMARY.....	79
RECOMMENDATIONS FOR NEEDED FUTURE RESEARCH.....	80
REFERENCES	83
APPENDIX A: Specifications for Laboratory M_R Testing.....	87

APPENDIX B: M_R Calculation Spreadsheet Example	103
APPENDIX C: Summary Sheets for Actuator LVDT M_R Values (Summer of 1992 & Spring of 1993).....	113
APPENDIX D: Overlay Spreadsheet Example	155

LIST OF TABLES

	<u>Page</u>
TABLE 2.1 Testing Sequence for Type II Soils	13
TABLE 3.1 Location of Test Sections	34
TABLE 3.2 AASHTO Specification Summary	35
TABLE 3.3 Summary of Typical Material Properties in Wyoming	39
TABLE 4.1 Thicknesses of Test Sections	44
TABLE 4.2 Fundamental Soil Properties for Samples Collected in the Summer of 1992	45
TABLE 4.3 Fundamental Soil Properties for Samples Collected in the Spring of 1993	46
TABLE 4.4 M_R Values for Samples Collected in the Summer of 1992	50
TABLE 4.5 M_R Values for Samples Collected in the Spring of 1993	51
TABLE 4.6 Back Calculated Resilient Modulus Values (Summer of 1992)	53
TABLE 4.7 Back Calculated Resilient Modulus Values (Spring of 1993)	53
TABLE 4.8 R-Values for Samples Collected in the Summer of 1992	54
TABLE 4.9 R-Values for Samples Collected in the Spring of 1993	55
TABLE 4.10 Summary of Test Sites Included in Each Period	56
TABLE 4.11 Correlations Between LMR and R-Value	57
TABLE 4.12 Relations Between LMRR and LMRA	58
TABLE 4.13 Back Calculation Correlations (N = 13)	61
TABLE 4.14 Back Calculation Relationships (N = 13)	61
TABLE 4.15 Coefficients of Determination for Soil- M_R Relations	63

TABLE 4.16	Parameter Estimates \pm Standard Error for Model with GI	63
TABLE 5.1	M_R Values Based on 41.4-kPa (6-psi) and Actual Field Deviator Stresses for Summer of 1992 Data.....	67
TABLE 5.2	M_R Values Based on 41.4-kPa (6-psi) and Actual Field Deviator Stresses for Spring of 1993 Data	68
TABLE 5.3	Testing Significance of Differences for Granular Base Sites (Summer of 1992 & Spring of 1993).....	69
TABLE 5.4	Testing Significance of Differences for Treated Base Sites (Summer of 1992 & Spring of 1993).....	69
TABLE 5.5	Summary of M_R Values from 3 Methods (Summer of 1992)	71
TABLE 5.6	Summary of M_R Values from 3 Methods (Spring of 1993).....	72
TABLE 5.7	SN_f Values for the Summer of 1992 Data	74
TABLE 5.8	SN_f Values for the Spring of 1993 Data.....	74
TABLE 5.9	Summary of SN_{eff} Values for the Summer of 1992 Data	75
TABLE 5.10	Summary of SN_{eff} Values for the Spring of 1993 Data	75
TABLE 5.11	Summary of D_{01} Values for the Summer of 1992 Data.....	76
TABLE 5.12	Summary of D_{01} Values for the Spring of 1993 Data.....	76

LIST OF FIGURES

	<u>Page</u>
Figure 2.1 Schematic Diagram of Stabilometer.....	8
Figure 2.2 Strains Under Repeated Loads	11
Figure 2.3 LVDT Locations on Testing Equipment.....	14
Figure 2.4 Benkelman Beam.....	16
Figure 2.5 KUAB 2m-FWD.....	18
Figure 2.6 Sensors from KUAB 2m-FWD.....	18
Figure 2.7 Correlation Chart for Common Soil Tests.....	21
Figure 2.8 Structural Capacity Loss Over Time and with Traffic	26
Figure 3.1 Data Collection and Evaluation Strategies.....	32
Figure 3.2 Geographic Locations of Test Sections.....	33
Figure 3.3 Layout of FWD Sensors.....	34
Figure 3.4 Interlaken Resilient Modulus Testing Equipment.....	37
Figure 3.5 Resilient Modulus Testing Chamber.....	37
Figure 4.1 M_R Summary Spreadsheet.....	48
Figure 5.1 Assumptions Made in Calculating Actual Field Stresses	66

CHAPTER 1

INTRODUCTION

by

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BACKGROUND

Material characterization plays a vital role in the design, construction, and maintenance of roadways. Because the loads generated on these facilities are distributed through the pavement structure to the underlying soil layer (the subgrade), engineers need to determine how this soil will perform. Over the years, several testing procedures have been developed to analyze and predict the subgrade's response to highway loading. Some of these tests frequently used by state highway agencies include the Resistance Value (R-value), the California Bearing Ratio (CBR), and the Texas Triaxial Classification. The latest method for evaluating the subgrade's behavior is called Resilient Modulus (M_R). By definition, resilient modulus is a material property that measures the elastic (load-unload) response of a soil under repeated loading (Claros et al., 1990). Numerically, it is the ratio of the deviator stress to the resilient or recoverable strain ($M_R = \sigma_d/\epsilon_r$). This subgrade property is used in the design of new pavement sections as well as in the rehabilitation of existing roadways. Without adequate information on the roadbed soil, a pavement structure will be improperly designed and as a result, it may fail prematurely.

PROBLEM STATEMENT

The American Association of State Highway and Transportation Officials (AASHTO) Guide for Design of Pavement Structures (1993) requires selecting a value for the design subgrade resilient modulus. This value may be based on laboratory testing, back calculation programs using deflection measurements, resilient modulus correlation studies, or the equation presented in the AASHTO overlay design procedures based on deflection measurements. Each of these methods of determining M_R present a variety of decisions and assumptions.

First, laboratory tests may be completed on soil samples obtained from the field under undisturbed or disturbed conditions. After completing testing, one must choose a design M_R value. This design value can be based on an assumed deviator stress of 41.4-kPa (6-psi), suggested in the literature, or based on actual field stresses. Second, M_R can be determined from back calculation programs. This method uses an indirect approach and the resilient modulus values tend to be higher than the actual values. As a result, the back calculated M_R value should be multiplied by a correction factor. This provides a conservative M_R value that is consistent with the assumptions made in the AASHTO pavement design procedures. Third, correlation studies require a reliable relationship between the test used for material characterization and the resilient modulus value.

OBJECTIVES

Because the above procedures for determining the subgrade resilient modulus may give variable results, one would want to know how these variations may influence the resulting overlay thicknesses for a construction project. Therefore, the University of Wyoming and the Wyoming Department of

Transportation (DOT) conducted a joint research project to address this problem. The principal objectives of this study were to:

1. investigate the importance of several fundamental soil properties (water content, plasticity index, liquid limit, group index) on selecting a design subgrade resilient modulus value for cohesive soils,
2. define the actual relationship (correction factor) between back calculated and laboratory based M_R values for typical cohesive subgrade soils in Wyoming,
3. compare actual subgrade field deviator stresses to the deviator stress assumed in determining a design M_R value from laboratory testing, and
4. determine the effect of selecting a M_R value on the design overlay thicknesses for typical pavement sections in Wyoming.

ORGANIZATION OF STUDY

This study examined the characteristics of cohesive subgrade soils at nine sites representing typical primary highways in the State of Wyoming. The roadbed soils included in the experiment had the following AASHTO classifications: A-4, A-6, and A-7-6. Samples for laboratory testing, deflection data, and pavement condition surveys were collected in the summer of 1992 and the spring of 1993. Next, an extensive laboratory testing program, several back calculation analyses, and overlay thickness designs were completed. Finally, the results were summarized in a computerized data base and a comprehensive statistical analysis was performed on the data.

Chapter 2 of this report reviews the traditional methods used to characterize subgrade soils, methods to determine resilient modulus for subgrade soils, and the AASHTO overlay design procedure. Chapter 3 describes the data collection process and overall evaluation strategies followed in this research. Chapter 4 discusses the laboratory testing, back calculation testing, and several important results on the factors that influence the selection of a design subgrade resilient modulus value. Chapter 5 discusses the impacts of selecting a particular method for determining a design resilient modulus value on the resulting overlay thickness. Chapter 6 summarizes the study, presents the conclusions, and makes recommendations for needed future research.

CHAPTER 2

LITERATURE REVIEW

INTRODUCTION

Pavement engineers continuously look for ways to improve pavement service life and performance. Historically, pavement design procedures were empirical. In many cases, relationships were based on factors such as traffic loading and volumes, materials, layer configurations and the environment (Mahoney et al., 1991). During the last decade, however, traditional pavement design procedures have been changed to incorporate elastic and/or viscoelastic theories as well as experience and various empirical tests. These new mechanistic-empirical procedures address two different aspects of pavement design. The mechanistic element allows engineers to examine the stresses, strains, and deflections in the pavement structure. The empirical element, on the other hand, tries to establish a relationship between these mechanistic responses and the performance of the pavement structure.

Most newly developed pavement and overlay design procedures also require the characterization of materials. This requirement resulted in the development of several laboratory tests to simulate actual field conditions in the laboratory. One of these tests is the resilient modulus test for subgrade soils. It is believed that the adoption of this new testing procedure will result in more reliable and cost-effective designs of pavement structures. This chapter presents a background of the tests traditionally used for roadbed soil characterization and the latest test, resilient modulus. This discussion includes three different procedures for determining the resilient modulus value: laboratory testing, back

calculation, and correlation studies. Finally, the chapter describes how the methods used for material characterization fit into the latest AASHTO overlay design procedure.

TRADITIONAL SUBGRADE TESTING PROCEDURES

Over the years, several testing procedures have been developed by state highway agencies to characterize roadbed soils. Two of the most common tests include the California Bearing Ratio (CBR) and Resistance Value (R-value). Both of these tests estimate the "strength" of the subgrade for use in the pavement design procedures.

California Bearing Ratio (CBR)

The CBR test was first developed by the California Division of Highways around 1930 (Asphalt Institute, 1978). During World War II, the U.S. Army Corps of Engineers modified the original procedure in order to incorporate the test into their flexible pavement design method for airport runways. Later, this test was adopted by the American Society of Testing Materials (ASTM) in 1961 and by the American Association of State Highway and Transportation Officials (AASHTO) in 1972. Both organizations, however, adopted procedures with minor modifications to the test used by the Corps (Asphalt Institute, 1978).

The CBR is a shear strength test based on penetration that can be completed on the soil in the field (ASTM D 4429) or on "undisturbed" or disturbed samples in the laboratory (ASTM D 1883, AASHTO T 193). In order to properly design a pavement structure based on the CBR value, the test is completed using samples at or near saturated soil conditions to represent the worst subgrade strength.

Therefore, the field testing procedure is primarily used for evaluating the properties on existing pavement sections while laboratory testing is completed on saturated soil samples.

Laboratory testing for the CBR value, using disturbed samples, involves several steps. First, the subgrade soil is compacted in molds 152-mm (6-in.) in diameter and 152 to 178-mm (6 to 7-in.) in height. In order to simulate field conditions, samples should be prepared using the expected moisture content, density, and method of compaction. After preparing the samples, a dead weight is applied to the sample to simulate the loading of the overlying pavement structure (base and pavement layers). Next, the assembly (soil, mold, and dead weight) is submerged in water for 4 days. This step allows the sample to become saturated and, therefore, allows the test to be completed on the worst subgrade strength. After removing the sample and draining it for 15 minutes, loading is applied to the assembly with a piston having an area of 1,935-mm² (3-in.²). This rod penetrates through the soil at a rate of 1.3-mm (0.05-in.) per minute and the load is recorded at the following penetrations: 2.5, 5.0, 7.5, 10.0, and 12.5-mm (0.1, 0.2, 0.3, 0.4, 0.5-in., respectively). A graph of load versus penetration is then constructed using the above results. The resulting plot is often not linear because of surface irregularities and consolidation during testing and must be corrected by re-zeroing the load-penetration curve. Finally, the following equation is used to determine the CBR value by substituting the corrected value of the unit load at 2.5-mm (0.1-in.) penetration:

$$CBR = \frac{\text{unitload at } 2.5 - \text{mm}(0.1 - \text{in.}) \text{ penetration}}{1000} (100) \quad (2.1)$$

The value in the denominator corresponds to the pressure required to reach the amount of penetration in

a standard crushed rock. For example, it takes 6.9-MPa (1000-psi) to obtain 2.5-mm (0.1-in.) penetration in crushed rock. Each level of penetration has a corresponding pressure. Typically, the CBR value decreases as penetration increases. As a result, the ratio at 2.5-mm (0.1-in.) of penetration is frequently used to determine the CBR value for pavement design (Wright & Paquette, 1987). The CBR values range from 0 to 100, characterizing a roadbed soil as bad to excellent, respectively.

Resistance Value (R-Value)

The R-value is also used to evaluate roadbed soil for highways. This test was originally developed at the California Division of Highways by F. N. Hveem and R. M. Carmany in 1948. It is a closed-system triaxial test that measures the internal friction or "resistance" of the soil in a stabilometer.

Figure 2.1 presents a basic schematic diagram of the stabilometer test. This test is

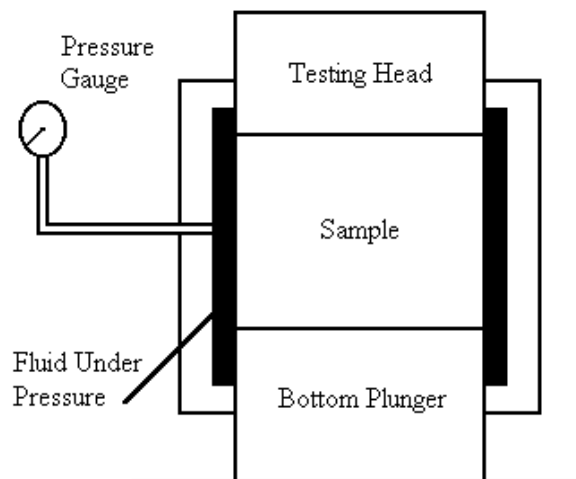


Figure 2.1 Schematic Diagram of Stabilometer

SOURCE: Huang (1993)

usually completed on disturbed samples in the laboratory (ASTM D 2844, AASHTO T 190). First, a sample, 102-mm (4-in.) in diameter and 62 to 65-mm (2.45 to 2.55-in.) in height, is prepared using a mechanical kneading compactor which simulates field compaction techniques. Next, this sample is placed into the stabilometer between a testing head and a bottom plunger. A vertical pressure of 1.1-MPa (160-psi) is then applied to the testing head, creating a horizontal pressure on the fluid within the rubber membrane that surrounds the sample (refer to Figure 2.1). This horizontal pressure is measured and recorded as p_h . Next, the applied vertical pressure is reduced to 0.55-MPa (80-psi) and the horizontal pressure reduced to 35-kPa (5-psi) with the stabilometer pump handle. After zeroing the displacement dial indicator on the stabilometer, the calibrated pump handle is turned to increase the horizontal pressure to 690-kPa (100-psi). The number of revolutions is recorded as D_2 . The following formula is then used to determine the R-value:

$$R = 100 - \frac{100}{(2.5 / D_2)(p_v / p_h - 1) + 1} \quad (2.2)$$

where: R = Resistance Value (R-value)
 p_v = applied vertical pressure of 1.1-MPa (160-psi)
 p_h = transmitted horizontal pressure at p_v of 1.1-MPa (160-psi)
 D_2 = displacement of stabilometer fluid necessary to increase horizontal pressure from 35 to 690-kPa (5 to 100-psi) measured in revolutions of a calibrated pump handle.

Hveem (1949) explained that the applied vertical pressure of 1.1-MPa (160-psi) was chosen arbitrarily and this value is not a critical matter in the R-value test. He supports this statement from laboratory testing that showed no effect on the ratio of p_v/p_h where the applied vertical

pressure varied from 0.7 to 2.8-MPa (100 to 400-psi). Because of this observation, some states use a different vertical pressure in their R-value testing to ensure that the sample is saturated. California uses an exudation pressure of 1.7-MPa (240-psi) while Washington uses 2.1-MPa (300-psi) (Huang, 1993). The R-values also range from 0 to 100, but characterize a roadbed soil as a liquid ($p_h = p_v$) to a rigid sample ($p_h = 0$), respectively.

DEVELOPMENT OF THE RESILIENT MODULUS TEST

Overall, the traditional soil tests listed above do not fully simulate actual loading conditions in the field. Instead, they measure different soil properties related to the strength of the soil. As a result, the resilient modulus test was developed by Seed et al. (1963) to reflect several observations in the field and from research projects.

One important idea came from the American Association of State Highway Officials (AASHO) Road Test which was conducted from October 15, 1958 to November 30, 1960 in Ottawa, Illinois. Researchers concluded that when a load is applied to the pavement surface the resulting deflection is a strong indicator of pavement performance (HRB, 1962). A majority of the surface deflection can be accounted for by the load-induced strain within the subgrade. Approximately 60 to 80 percent of the measured surface deflection was found to develop in the subgrade at the AASHO Road Test (HRB, 1962). Therefore, the resilient modulus test for subgrade soils models an important part of flexible pavement performance.

Another important observation contributing to the development of the M_R test is the stress in the pavement structure resulting from loading. The stress at a given point in the pavement structure is zero when the wheel load is at a considerable distance away. However, when this load is directly above the

point, the stress is at its maximum value. In many cases, it is reasonable to assume the stress pulse to be a haversine or triangular loading even though the duration of the pulse depends on the vehicle speed and the depth of the point below the pavement surface (Huang, 1993). Because the vehicle speed varies a great deal and the depth of the material may not be known during design, the AASHTO specifications recommend a haversine load wave with a duration of 0.1 second and a rest period of 0.9 second (AASHTO, 1992). As a result, the M_R test accounts for the type and duration of loading expected in the field.

A third important observation is the fact that most paving materials experience some permanent deformation after each load application (Huang, 1993). Figure 2.2 shows how the amount of strain under repeated loading in a material changes over time. In the beginning, the material shows a considerable increase in the amount of permanent deformation (accumulated plastic strain). However, as the number of loads increases, the accumulated plastic strain levels off and the material is essentially elastic (recoverable strain). This phenomenon usually occurs

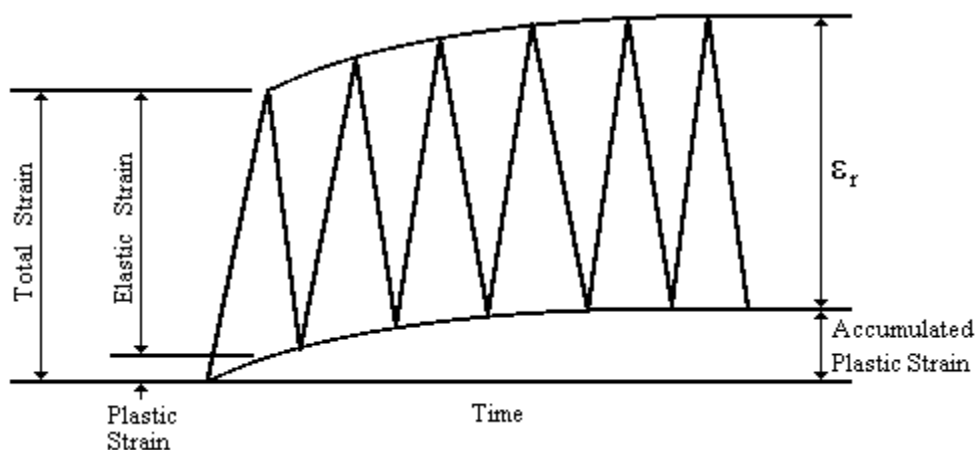


Figure 2.2 Strains Under Repeated Loads

SOURCE: Huang (1993)

after 100 to 200 load applications. Because the applied load is smaller than the material's strength, the M_R test can be completed on the same sample for several different loadings and environmental conditions (Huang, 1993).

In 1986, the resilient modulus test became the basis for the AASHTO Guide for Design of Pavement Structures. According to AASHTO (1993), the M_R value has three important advantages over the soil support value used in the previous editions:

1. It indicates a basic material property which can be used in mechanistic analysis of multi-layered systems for predicting distresses such as roughness, cracking, rutting, and faulting.
2. It has been recognized internationally as a method for characterizing materials for use in pavement design and evaluation.
3. Techniques are available for estimating the M_R properties of various materials in-place from non-destructive tests.

With the above observations and advantages, it is clear that resilient modulus testing can directly measure the strength of the subgrade soil and provide information which reflect field conditions.

RESILIENT MODULUS LABORATORY TESTING

The Interim Method of Test for Resilient Modulus of Unbound Granular Base/Subbase Materials and Subgrade Soils - SHRP Protocol P46 (AASHTO: T 294-92 I) outlines the latest testing procedure (refer to Appendix A). This specification separates subgrade material into two different categories: Type I (granular) and Type II (cohesive). Each type of soil has a different conditioning cycle and fifteen loading sequences, varying in confining and deviator stresses. Overall, Type I soils

undergo higher stresses, both confining and deviator, because of their higher resistance to deformation. The loading sequence for Type II soils is presented in Table 2.1. The amount of deformation in the soil sample is recorded using two linear variable differential transducers (LVDT's) outside of the testing chamber. However, the original AASHTO T-274

TABLE 2.1 Testing Sequence for Type II Soils (AASHTO, 1992)

Sequence No.	Confining Pressure S_3 , psi	Deviator Pressure S_d , psi	Number of Load Repetitions
0*	6	4	1000
1	6	2	100
2	6	4	100
3	6	6	100
4	6	8	100
5	6	10	100
6	3	2	100
7	3	4	100
8	3	6	100
9	3	8	100
10	3	10	100
11	0	2	100
12	0	4	100
13	0	6	100
14	0	8	100
15	0	10	100

* preconditioning

specifications required 2 LVDT's on rings within the test chamber. These LVDT's are normally placed at a specified gage length depending on the size of the sample. Figure 2.3 shows both of these LVDT locations.

BACK CALCULATION OF RESILIENT MODULUS

The laboratory resilient modulus test is relatively complex and it requires obtaining field samples.

As a result, several agencies have looked into non-destructive back calculation

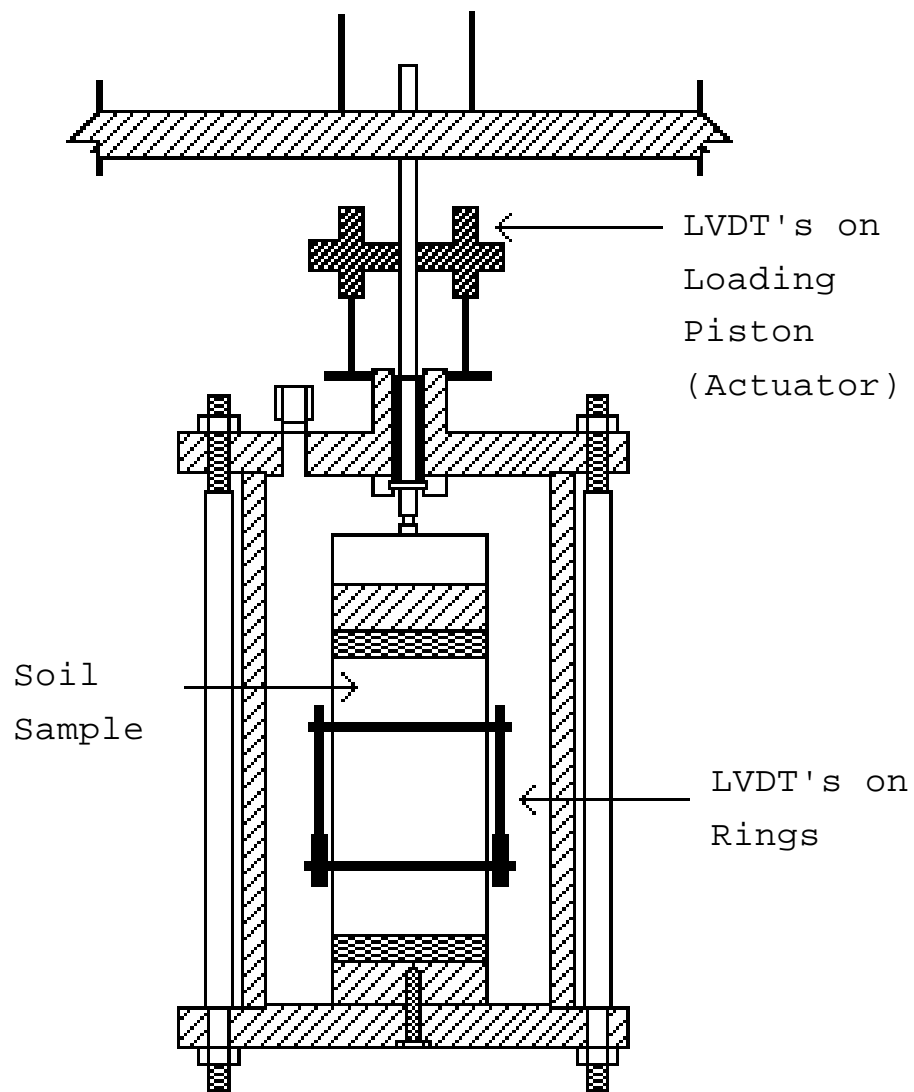


Figure 2.3 LVDT Locations on Testing Equipment

procedures to estimate the strength of the soils in-place. The back calculation procedures involve collecting surface deflection data in the field on existing pavement sections through non-destructive testing and then plugging these values into a computer program to obtain the M_R values. Surface deflection measurements provide pavement engineers with a rapid, relatively inexpensive, and non-destructive method of examining the basic response of the pavement structure to applied loads (Ali & Khosla, 1987). Because this analysis is normally performed on existing highway sections, the back calculation of resilient modulus values is primarily used in designing pavement overlays.

Non-Destructive Testing Equipment

Several different types of testing equipment were developed to examine the in-situ characteristics of a pavement structure. Non-destructive testing (NDT) equipment can be divided into four general categories: static deflection, steady-state deflection, impulse load deflection, and wave propagation. However, only the first three categories provide deflection measurements.

Static deflection devices measure the pavement's response to loads applied with a slow moving vehicle or a stationary loading frame (Stoffels & Lytton, 1987). Three common NDT devices in this category include: Benkelman beam, California traveling deflectometer, and LaCroix deflectometer. Figure 2.4 shows a picture of the Benkelman beam which was widely used by highway agencies. The measurement probe on the beam is placed between the rear dual tires of a 80-kN (18-kip) single-axle load truck. As the truck slowly moves away from the support (reference) beam, the rebound deflection of the probe is measured at specific distances, creating a deflection basin. Overall, this measuring device is easy to use, but it is a slow process and has several other disadvantages. Because the support beam must be an immovable reference

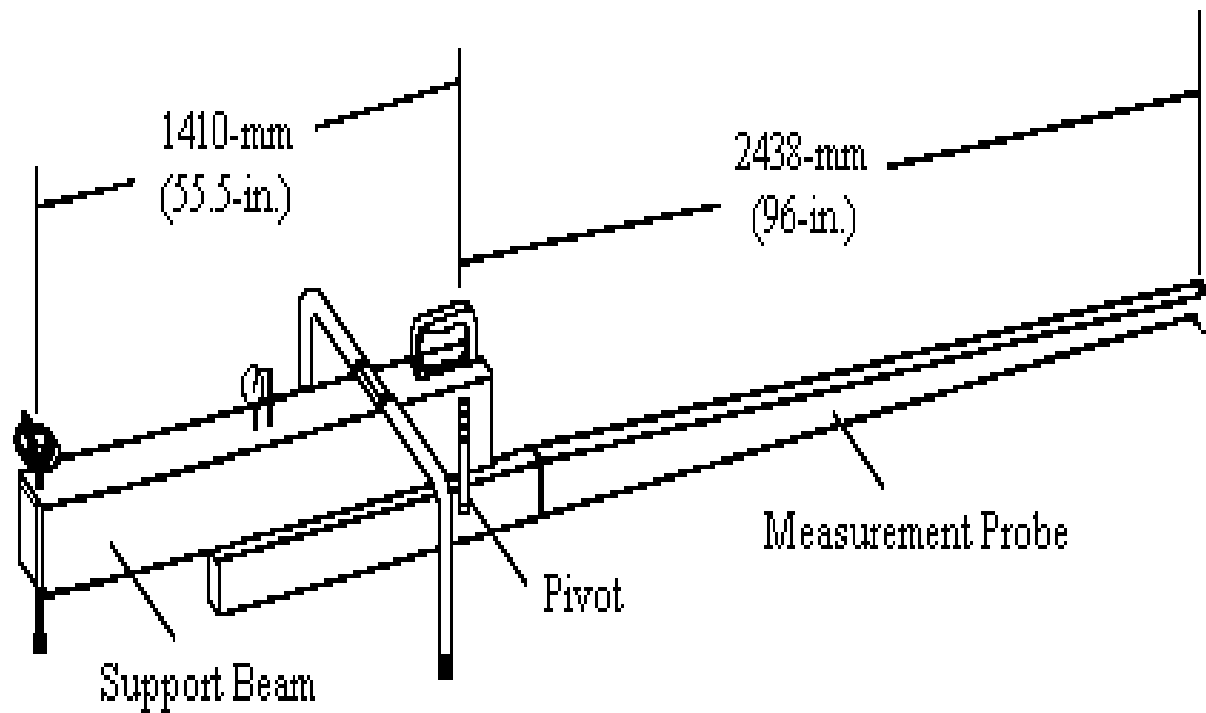


Figure 2.4 Benkelman Beam

SOURCE: Huang (1993)

point, the use of this device is limited to flexible pavements. In addition, the loads used to measure the surface deflection do not represent actual field conditions, impulse loads. Therefore, empirical correlations must be developed in order to use the results in any mechanistic pavement design procedure (Huang, 1993).

Steady state deflection systems, on the other hand, measure the pavement's response to loads applied by a vibratory device. Research has shown that the deflection at any specific driving frequency is approximately proportional to the amplitude of the load. However, at low frequencies, this factor approaches the value of the static pavement stiffness (Stoffels & Lytton, 1987). Therefore, the

vibratory device must apply a compressive force of varying magnitude, a dynamic force superimposed over a static force, in order to account for these effects. Two of the most common systems in this category include the Dynaflect and the Road Rater. Both devices use inertial motion sensors (geophones), placed at specific distances away from the point of loading, to record the surface deflection. This type of NDT device does not require a reference point like the static equipment. It is also a rapid method of analyzing a section's structural adequacy. Some of the disadvantages of this testing procedure include the inability to apply the actual loads in the form of steady-state vibration and the effect some large static loads may have on stress sensitive materials (Huang, 1993).

The third system, impulse load deflection, applies a transient force impulse to the pavement surface and records its response. This impulse is created by selecting a weight and dropping it a certain height. This type of NDT equipment is commonly called a Falling Weight Deflectometer (FWD). Three commonly used FWDs include: Dynatest, KUAB, and Phoenix. Figures 2.5 and 2.6 show pictures of the Wyoming DOT KUAB deflectometer. These testing devices allow another method of rapidly analyzing a section's structural adequacy for use in a mechanistic pavement design procedure. Overall, most pavement engineers agree that the FWD provides an accurate method of modeling actual moving loads in both magnitude and duration (Huang, 1993). This device also uses a relatively small static load compared to the impulse loading. However, these devices have some disadvantages. In many cases, it is difficult to obtain reliable results from the inertial motion sensors in the low frequency range. It is also difficult to produce force impulses that have a short duration to reliably measure the deflections in the significant frequency range of the pavement section (Stoffels & Lytton, 1987).

Back Calculation Computer Programs

There are several computer programs that can use deflection data to back calculate the strength of the different layers in a pavement structure. Some of the most widely used back calculation programs include: MODULUS, EVERCALC, and BOUSDEF. All of these programs compare the deflection basins from field data to theoretical basins to determine back calculated M_R values. However, each program computes these moduli by using different methodologies and



Figure 2.5 KUAB 2m-FWD

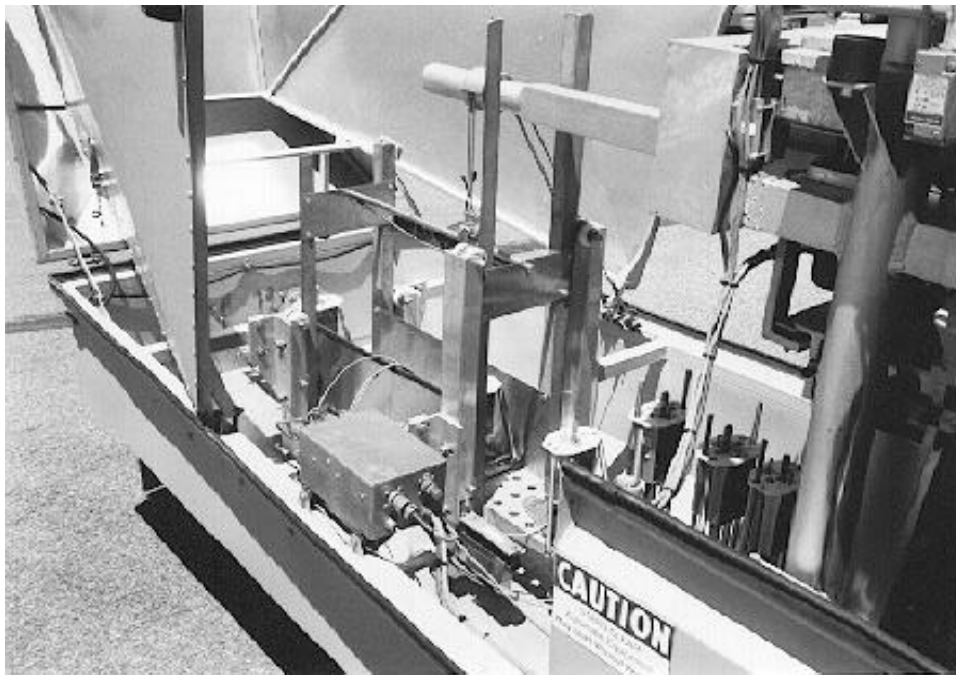


Figure 2.6 Sensors from KUAB 2m-FWD

assumptions. The first program, MODULUS, was developed at Texas A & M University. MODULUS determines M_R values based upon a layered elastic code called WES5. This code creates a large database of theoretical deflection basins and matches, through interpolation, the best basin to the field data. The second program, EVERCALC, was developed at the University of Washington. In this program, theoretical deflections are based on CHEVRON, another layered elastic code. The third program, BOUSDEF, was developed at Oregon State University. This program uses the method of equivalent thicknesses, assuming one thick, uniform layer of material, and the Boussinesq theory to determine theoretical basins. Overall, by matching the deflection basin measured in the field, a M_R value is calculated for the surface, base, and subgrade layer.

Even though these computer programs provide pavement engineers with a quick method of obtaining M_R values, the following problems associated with back calculation procedures must be taken into consideration (Uddin, 1984):

1. The nonuniqueness of the resilient modulus back calculated from the measured deflection basin.
2. Errors due to possible variation in thickness of pavement layers.
3. Errors involved in assuming a semi-infinite subgrade.
4. Time involved in the iterative process.
5. Errors in back calculated moduli because of the nonlinear behavior of granular layers and subgrade.
6. Errors involved in using input values out of the range for which the model was calibrated.

In addition, three factors can influence the deflection measurements used in these computer programs: loading, climate and pavement condition. Loading should simulate the conditions used in the design process, typically, a 40-kN (9000-lbs.) wheel load. Climate factors such as temperature and moisture can also affect pavement deflections. These conditions should be recorded so that corrections can be

made to the deflection measurements before using them in a computer program. Finally, pavement conditions influence the deflection measurements. During testing, careful selection of test sections should be made in order to avoid testing over a distress such as cracking or rutting (Huang, 1993).

M_R DETERMINATION FROM CORRELATION STUDIES

In many cases, agencies lack the large capital required for the laboratory M_R equipment and/or their pavement engineers are unfamiliar with this new subgrade soil property. As a result, correlation charts and equations have been created to convert values from some of the commonly used soil tests to resilient modulus values. Figure 2.7 presents a correlation chart for most common soil tests. This chart was developed using data from the AASHO Road Test and several design curves from California, Washington, and Kentucky (Van Til et al., 1972).

The soil support scale, on the far left, has values ranging from 1 to 10 and was developed using AASHO Road Test data. A 3.0 on the scale represents the silty clay roadbed soil while a 10.0 represents the crushed rock base material. In order to use this scale, highway agencies developed relationships between their commonly used material characterization test and the soil support scale. As a result, each state usually adopted a different test which caused variations in selecting subgrade strength. This problem contributed to the adoption of the M_R value as the material property used to design pavement structures. Through several research projects on the AASHO roadbed soil, it was shown that the soil support value (S) of 3.0 had a M_R value of 20,684-kPa (3000-psi). The rest of the correlations for converting soil support values to M_R values were based on this relationship.

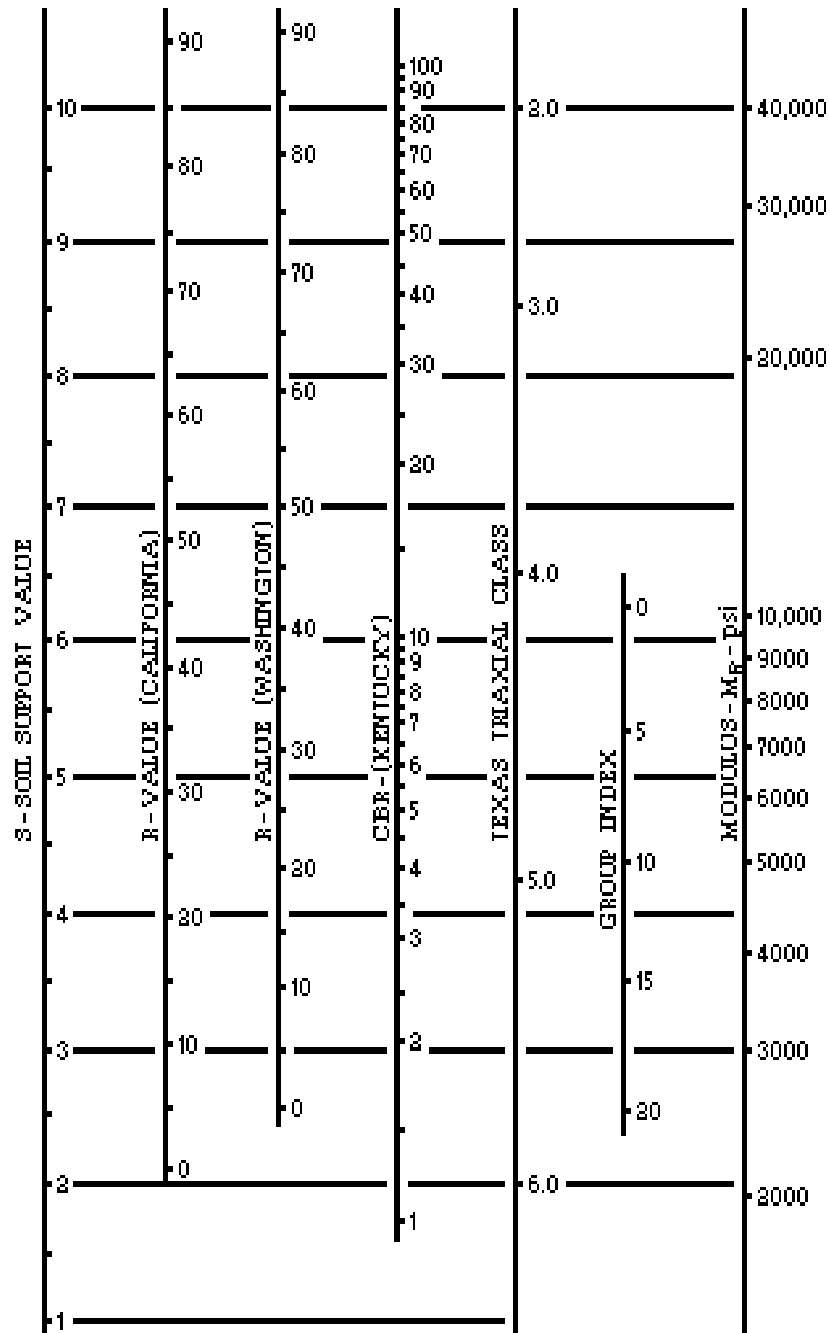


Figure 2.7 Correlation Chart for Common Soil Tests

SOURCE: Van Til et al. (1972)

Besides these correlations, two well known equations have also been developed through research to convert values from the strength tests to resilient modulus values. Heukelom and Klomp (1972) developed the following equation to convert CBR values to M_R values:

$$M_R = 1500 (\text{CBR}) \quad (2.3)$$

On the other hand, the Asphalt Institute (1982) developed the following equation to calculate resilient modulus from R-values:

$$M_R = 1155 + 555 (\text{R-value}) \quad (2.4)$$

Other equations have also been developed by state highway agencies. One example is Nebraska. Woolstrum (1990) reported a method to reliably determine the resilient modulus value based on the Nebraska Group Index (NGI). This index is similar to the group index developed by AASHTO because it uses the percent retained on the No. 200 sieve, the liquid limit, and the plasticity index. However, the NGI allows negative values for granular materials. Through a regression analysis, fourth-order equations were developed under three moisture conditions: optimum, wet, and dry. These equations correlated well with M_R values obtained in the laboratory. Even though the use of the correlation charts and equations to obtain resilient modulus values is acceptable, AASHTO (1993) recommends that "user agencies acquire the necessary equipment to measure M_R ."

SELECTION OF A DESIGN M_R VALUE

Because of the importance of material characterization, several factors must be taken into consideration when selecting a M_R value for pavement design. According to Darter et al. (1992),

“regardless of the method used, the design subgrade M_R value must be consistent with the value used in the design performance equation for the AASHTO Road Test subgrade.” The 1993 AASHTO guide uses a value of 20,684-kPa (3000-psi), but does not justify its selection. This value is one of the underlying assumptions of the flexible pavement performance model. Based on a study by Thompson and Robnett (1976), this value is appropriate when the AASHTO soil is about 1% wet of optimum and subjected to a deviator stress of about 41.4-kPa (6-psi) or more. In addition, these results were based on laboratory tests using zero confining pressure, and they reported little effect when testing the samples using a confining pressure of 20.7 to 34.5-kPa (3 to 5-psi). Therefore, when selecting a M_R value from laboratory testing, a zero confining pressure and a 41.4-kPa (6-psi) deviator stress is suggested (Elliott, 1992).

Besides the above considerations, other factors such as water content, soil type, and sample condition must be accounted for when selecting an M_R value from the laboratory testing. First, water content is important because of its effects on M_R values obtained either above or below the optimum value. In 1989, Elfino and Davidson reported variations in the resilient modulus value of 7-41% from soils at different water contents. Second, whether the sample is undisturbed or disturbed will influence the M_R . Third, soil type may influence the M_R because of the differences in quality and soil strength. Overall, by considering these variations, an appropriate M_R value will be selected to represent the design field conditions.

The above observations also play an important part when determining a back calculated M_R value. In order to make a non-destructive testing value consistent with the 20,684-kPa (3000-psi) value, the calculated M_R value is multiplied by a correction factor. The need for a correction factor

resulted from the fact that most NDT programs assume the measure deflection, at a certain distance away from the loading plate is attributable solely to the subgrade. In many cases, the amount of stress at this point is less than 41.4-kPa (6-psi), giving a higher resilient modulus value. Therefore, by reducing the back calculated resilient modulus value, one of the underlying assumptions in the flexible pavement performance model is satisfied.

UTILIZATION OF SOIL M_R IN THE AASHTO OVERLAY DESIGN PROCEDURES

Over the years, several highway agencies developed their own overlay design procedures. In addition, AASHTO recently released the 1993 AASHTO Guide for Design of Pavement Structures. In the AASHTO guide, the determination of the subgrade resilient modulus value is essential for designing both new pavements and overlay thicknesses. If the design resilient modulus value is too high, the thickness of the pavement layer will be insufficient. If the design resilient modulus value is too low, the thickness will be conservative and not cost-effective. The implications of selecting resilient modulus values in designing new pavements will not be discussed here since the objective of this research project is to evaluate the new AASHTO overlay design procedure for asphalt pavements.

The 1993 AASHTO Overlay Design Procedure

The 1993 AASHTO Guide for Design of Pavement Structures outlines an eight step procedure for determining the overlay thickness. These steps include evaluating the existing pavement design and construction, traffic analysis, condition surveys, deflection testing, coring and materials testing, determination of required structural number for future traffic (SN_f), determination of effective structural

number (SN_{eff}) of the existing pavement, and determination of the overlay thickness (D_{ol}). Each of these steps provides valuable information to determine an appropriate overlay design.

In the first step, evaluating the existing pavement design and construction, thicknesses of each layer and material types and characterization should be determined. Next, in the traffic analysis, the past cumulative 80-kN (18-kip) equivalent single-axle loads (ESALs) (N_p) and the future 80-kN (18-kip) ESALs (N_f) should be estimated. This traffic information is important in determining the SN_f value and overlay thickness. Third, pavement condition surveys provide information needed to determine the structural coefficients for each pavement layer. Fourth, deflection testing provides the basic information needed in the AASHTO overlay design procedures. Some type of NDT device, usually a FWD, provides this type of data. The AASHTO guide recommends using the following formula for determining the resilient modulus value of the subgrade soil based on the deflection measurements:

$$M_R = \frac{0.24P}{d_r} \quad (2.5)$$

where: M_R = subgrade resilient modulus, psi
 P = applied load, pounds
 d_r = deflection at a distance r from the center of the load, inches
 r = distance from center of load (sensor location), inches

Fifth, coring and materials testing provides additional information to confirm the values obtained from reviewing construction records. Laboratory testing of the subgrade soil is recommended if deflection testing is not completed on a pavement section. In addition, the thicknesses of all the layers in the pavement structure can be confirmed by coring. Sixth, the SN_f value is determined by using several pieces of information. These items include: the effective design subgrade resilient modulus, design

present serviceability index (PSI) loss, overlay design reliability (R), and the overall standard deviation (S_o) for flexible pavement. Seventh, the SN_{eff} value is determined using one or more of the following three methods: non-destructive testing (NDT), pavement condition surveys (PCS), and remaining life (RL). Finally, the overlay thickness (D_{ol}) is determined by taking the difference between the SN_f and SN_{eff} values and dividing this quantity by the layer coefficient for new asphalt pavement ($D_{ol} = (SN_f - SN_{eff})/a_{ol}$).

Determining the Need for an Overlay

Structural deterioration is any condition that reduces the load-carrying capacity of the pavement (Darter et al., 1992). As time and the number of loads applied (traffic) to a pavement section increase, the structural capacity (SC) of the section decreases from its initial state, SC_o , as shown in Figure 2.8. When an evaluation for an overlay is conducted, the section's structural

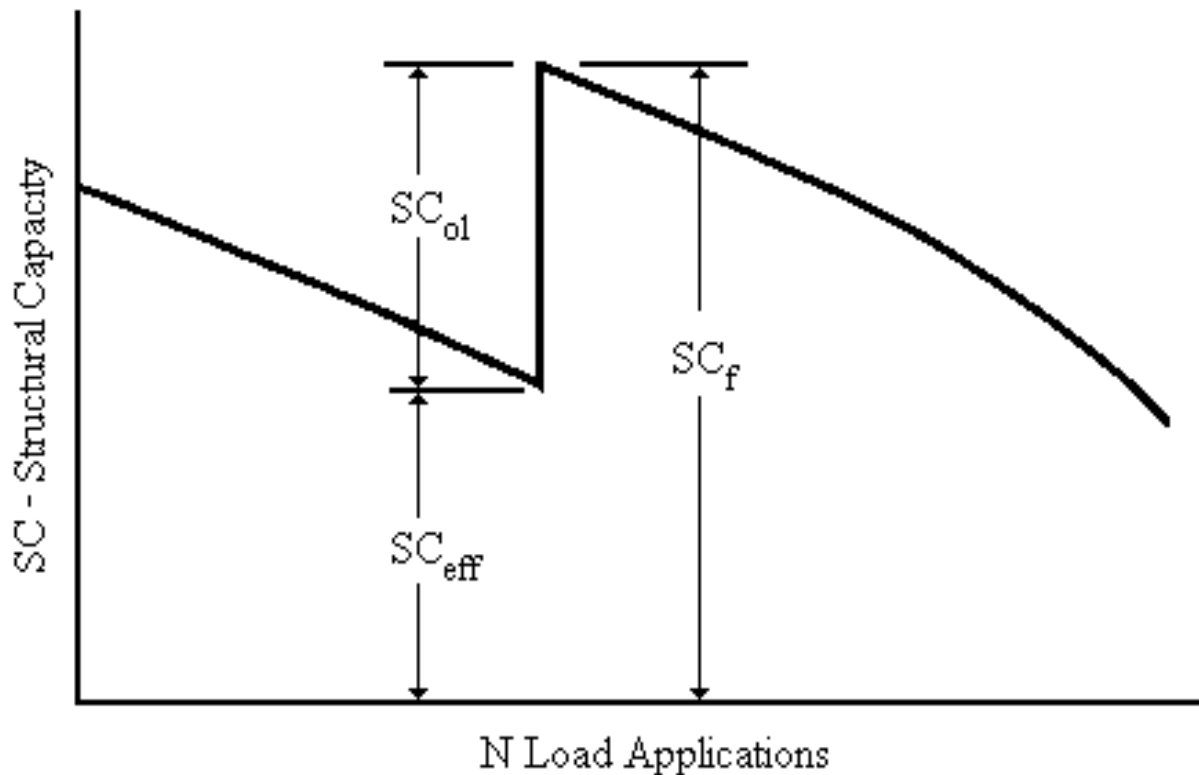


Figure 2.8 Structural Capacity Loss Over Time and with Traffic

SOURCE: Darter et al. (1992)

capacity is evaluated and denoted by SC_{eff} . In order to repair the section and return it to its original or higher capacity, SC_f , an overlay is placed with a value of SC_{ol} (Note: $SC_f = SC_{eff} + SC_{ol}$). This method of evaluation is known as the structural deficiency approach.

In order to obtain the “correct” thickness of the overlay, the evaluation of the effective structural capacity must be accurate by examining the existing pavement conditions and determining how the pavement materials will behave in the future. However, this is very difficult since the declining relationship is not well defined. It is often assumed by many agencies that a section’s structural capacity is linear in order to simplify calculations and provide a conservative measurement. As a result, the

AASHTO guide uses three different methods to determine a section's asphalt overlay thickness: non-destructive testing (NDT), pavement condition surveys (PCS), and remaining life (RL).

The first method, NDT, involves determining the effective structural capacity, expressed as the effective structural number (SN_{eff}) for flexible pavements, based on non-destructive deflection measurements. This data is often obtained using a Falling Weight Deflectometer (FWD). The SN_{eff} is determined with the following formula as a function of the total thickness and overall stiffness of a section:

$$SN_{\text{eff}} = 0.0045D\sqrt[3]{E_p} \quad (2.6)$$

where: SN_{eff} = effective structural number
D = total thickness of all pavement layers above the subgrade, inches
 E_p = effective modulus of pavement layers above the subgrade, psi

The E_p value is based on a back calculation procedure for resilient modulus described in the AASHTO guide.

The second method involves using pavement condition surveys. This type of visual survey determines the SN_{eff} value based on the distress conditions observed in the field, drainage surveys, and maintenance history. For flexible pavements, the following distress types should be examined: alligator cracking, rutting, transverse and longitudinal cracks, and localized failing areas. Each distress type is converted to a layer coefficient based on the percentage of the surface condition. The following formula is then used to determine the SN_{eff} value:

$$SN_{\text{eff}} = a_1D_1 + a_2D_2m_2 + a_3D_3m_3 \quad (2.7)$$

where: SN_{eff} = effective structural number
 a_1, a_2, a_3 = corresponding structural layer coefficients
 D_1, D_2, D_3 = thicknesses of existing pavement surface, base, and subbase layers
 m_2, m_3 = drainage coefficients for granular base and subbase

Remaining life is the third procedure to determine the effective structural capacity. This method determines the SN_{eff} value based on fatigue damage from traffic. As the name implies, the amount of load-carrying capacity remaining in the pavement section is determined. This procedure requires the knowledge of past traffic (N_p) and estimates the total traffic the pavement could be expected to carry to “failure” ($N_{1.5}$). This failure is often assumed to be 1.5 on the Present Serviceability Index (PSI). In general, a new pavement has a PSI between 4 and 5, and repair is usually needed when the PSI is between 1.5 and 2.5. The following formula determines the remaining life:

$$RL = 100 \left[1 - \left(\frac{N_p}{N_{1.5}} \right) \right] \quad (2.8)$$

where: RL = remaining life, percent
 N_p = total traffic to date, 18-kip ESAL
 $N_{1.5}$ = total traffic to pavement “failure”, 18-kip ESAL

The RL value is then converted to a condition factor (CF) ranging from 0.5 to 1.0 using a graph of CF versus RL . The SN_{eff} value is then computed with the following formula:

$$SN_{\text{eff}} = CF * SN_o \quad (2.9)$$

where: SN_{eff} = effective structural number
CF = condition factor
 SN_o = structural number of the pavement if it were newly constructed

Darter et al. (1992) cites the following four major sources of error in this procedure: the predictive capability of the AASHO Road Test equations, the large variations in performance typically observed even among pavements of seemingly identical designs, estimation of the past 18-kip ESALs, and the inability to account for the amount of preoverlay repair to the pavement. Overall, this evaluation procedure should only be used for pavement sections which have very little visible deterioration and no previous overlays.

CHAPTER SUMMARY

Material characterization is important in designing pavement sections. The traditional methods for examining the characteristics of the subgrade, CBR and R-value, do not provide information that directly represent field conditions. However, the resilient modulus test measures a subgrade's ability to recover after loading. Therefore, this value is expected to improve the modeling of actual field conditions and to provide a better basis for pavement designs. A soil's M_R value may be measured by using the following three techniques: laboratory testing, back calculation, and correlation charts/equations. Once a M_R value is determined for a section, this value can be used to calculate an overlay design thickness.

CHAPTER 3

DESIGN OF EXPERIMENT

INTRODUCTION

In this research project, extensive data were collected in the field and laboratory to fulfill the objectives of the study presented in Chapter 1. Figure 3.1 shows the seven basic steps performed in this research. These steps were: site selection, data collection, resilient modulus determinations, data base preparation, data analysis, effect of M_R on overlay thicknesses, and conclusions. In this chapter, each one of the above evaluation strategies will be discussed.

SITE SELECTION

Nine pavement test sections were selected in the State of Wyoming. These sections represent typical cohesive subgrade soil conditions throughout the state (refer to Figure 3.2). Overall, a typical cross-section of the pavement structure included an asphalt concrete layer, a granular or treated base (asphalt or cement), and the underlying subgrade soil. Because of the relatively low traffic volumes in Wyoming, pavement structures do not normally have a subbase layer.

DATA COLLECTION

In the summer of 1992 and spring of 1993, extensive field data were collected on all test sections included in the experiment. This field evaluation included pavement and subgrade coring,

deflection measurements, and condition surveys. At each site, three pavement cores and three Shelby tubes of subgrade soil were obtained. Table 3.1 summarizes the locations of the nine test sections. The Wyoming DOT's KUAB 2m-Falling Weight Deflectometer was used to take

TABLE 3.1 Location of Test Sections

Number on State Map	Route	Roadway	Test Site Milepost
1	P-12	US-30	48
2	P-12	US-30	70
3	P-23	US-287	416
4	P-30	US-26	108
5	P-34	US-20/26	15
6	P-34	US-20	163
7	P-44	US-16	229
8	P-44	US-16	244
9	F-25	US-85	197.4

deflection measurements at each site using three different levels of loading: 26.7, 40.0, 53.4-kN (6000, 9000, and 12000-lbs.). Figure 3.3 shows the locations of the sensors used to take the deflection measurements in this research. Other important data, such as pavement and air temperatures, were recorded for later use in correcting the temperature to the standard value of

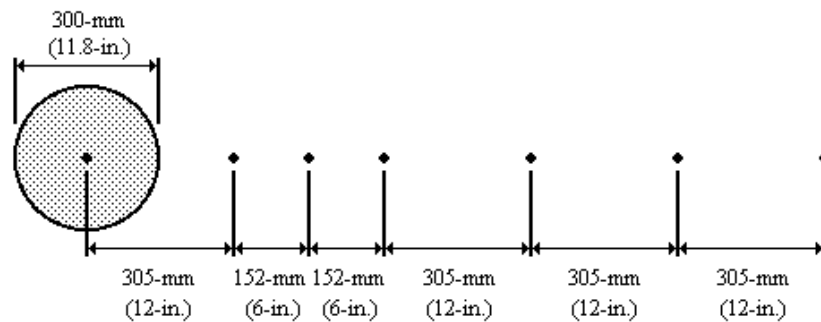


Figure 3.3 Layout of FWD Sensors

21° Celsius (70° Fahrenheit). Pavement condition surveys were completed on each test section to examine pavement surface conditions.

LABORATORY TESTING AND RESILIENT MODULUS DETERMINATION

After obtaining the soil samples from the field, several laboratory tests were initially conducted to determine the soil classification of the subgrade at each test section. These preliminary tests included: sieve analysis, Atterburg Limits, water content determinations, and R-values. The AASHTO Soil Classification system was later used to determine the soil type at each test section. The equation below, occasionally used by the Wyoming DOT, was used in estimating the optimum water content for each sample:

$$\omega = 0.477(LL) + 2 \tag{3.1}$$

where: ω = optimum water content (%),
 LL = liquid limit

All laboratory tests were conducted in accordance with their respective AASHTO specification. Table 3.2 summarizes these testing specifications.

TABLE 3.2 AASHTO Specification Summary

Property	Specification
Standard R-value	AASHTO T 190
Liquid Limit (LL)	AASHTO T 89 (WYO MOD)
Plastic Limit (PL)	AASHTO T 90 (WYO MOD)
Sieve Analysis	AASHTO T 88

Resilient modulus values were then determined for each test section from: laboratory testing based on 41.4-kPa (6-psi) deviator stress, laboratory testing based on actual field stress conditions, and from deflection measurements.

Laboratory Testing for Resilient Modulus

Laboratory soil resilient modulus tests were performed on the Wyoming DOT machine manufactured by the Interlaken Technology Corporation. The system has a Series 3300 98-kN (22-kip) capacity test frame, a Series 3230, 16 channel data acquisition system, and a Series 3200 controller. This device is located in the Materials Branch at the Wyoming Department of Transportation. Figures 3.4 and 3.5 show the resilient modulus testing device. All samples tested were 71-mm (2.8-in.) in diameter and 152-mm (6-in.) in height. These measurements were selected in accordance with the specifications, a height not less than two times the diameter and a minimum diameter of 71-mm (2.8-in.) or five times the nominal particle size (AASHTO, 1992). In this research project, deformation readings were recorded at two different locations during laboratory testing. First, from 2 LVDT's located outside of the triaxial cell on the loading piston (referred to as the actuator in this report) and second, from three LVDT's located on the rings inside of the testing chamber. Even though some testing programs available for M_R testing automatically average the signals from the LVDT's, individual measurements were saved in a computer file in this research project. This procedure was used to identify and eliminate inconsistent deformation measurements coming from the LVDT's. All applied load and deformation readings were also stored in a computer file for later analysis.



Figure 3.4 Interlaken Resilient Modulus Testing Equipment



Figure 3.5 Resilient Modulus Testing Chamber

Subgrade cores obtained from the summer of 1992 were tested in two conditions, undisturbed and disturbed. Due to the fact that the soils were left in the Shelby tubes for several months prior to testing, normal extraction could not be performed without disturbing the samples. The solution to this problem was to freeze the tubes, extract the soil samples, and let the cores thaw for twenty-four hours prior to testing. Overall, this procedure was successful and allowed soil samples to be removed from the tubes with minimal disturbance. Resilient modulus testing was performed on some of the frozen samples, but reasonable M_R values were difficult to obtain because of the low stresses applied to the samples and the high stiffness of the frozen cores. As a result, the frozen condition was not considered in the analysis. After testing was completed for the undisturbed samples, the disturbed (remolded) samples were prepared by crumbling the sample and re-compacting it to 152-mm (6-in.) using five lifts and static compaction. This procedure is fully described in the specifications (refer to Appendix A).

Subgrade cores from the spring of 1993 were tested for resilient modulus shortly after obtaining them from the field. Again, two conditions were considered, undisturbed and disturbed (remolded). Unlike the first set of subgrade cores, samples were easily removed from the Shelby tubes and, therefore, they did not require freezing prior to extraction. LVDT measurements during M_R testing were also taken outside and inside the testing chamber.

After the laboratory M_R testing was completed, deformation and applied load readings from the last five cycles of loading condition were retrieved from the data files created during the tests. Several spreadsheets were developed to accept these data as well as the length and diameter of each sample. By entering this information, the resilient modulus values for all nine test sections were calculated automatically for each testing condition. Plots were then constructed by using the \log_{10} (resilient

modulus) versus the deviator stress. Simple regression analysis was then performed to estimate resilient modulus based on deviator stresses. As suggested in the literature, a design M_R value was determined by substituting a deviator stress of 41.4-kPa (6-psi) into this resulting equation.

Besides using this suggested deviator stress, the actual stresses in the subgrade were computed by using the computer program BISAR. This computer program, developed by De Jong et al. (1973), computes the stresses in a n-layer pavement structure by considering the vertical and horizontal loads. Information obtained in the field evaluation, specifically the thicknesses of each pavement layer, and certain material properties were entered into this program. Table 3.3 summarizes the values for the material properties commonly used by the Wyoming DOT. These computed deviator stresses were then substituted into the linear equations developed in the laboratory testing to determine another design resilient modulus value based on actual field stress conditions.

TABLE 3.3 Summary of Typical Material Properties in Wyoming

Layer in Pavement Structure	Young's Modulus (MPa)	Unit Weight (kN/m ³)	Poisson's Ratio
Asphalt Cement Mix	2758	23.1	0.35
Granular Base	124	22.8	0.40
Cement Treated Base	5516	22.0	0.25
Asphalt Treated Base	2413	23.1	0.37

Back Calculation of M_R

In addition to the laboratory analysis, the deflection data collected were used to determine subgrade M_R values with the following three back calculation programs: MODULUS, EVERCALC, and BOUSDEF. All of these programs compare the deflection basins based on field data to theoretical

basins in order to determine the back calculated resilient modulus values. Deflection measurements used in these three computer programs were corrected to a standard temperature of 21° Celsius (70° Fahrenheit) with a computer program called TAFFY. This program was developed by the Colorado Department of Transportation (1988) to determine temperature adjustment factors. The average air temperature, the surface temperature, and the mean pavement temperature all affect this adjustment factor.

DATA BASE PREPARATION AND DATA ANALYSIS

All field and laboratory data were summarized in a computerized data base. Statistical analyses were then performed to determine how fundamental soil properties, linear variable differential transducer (LVDT) placement during M_R testing, and sample condition influence resilient modulus values. Further analyses were completed to examine the relationship between laboratory and back calculated M_R values. These results and analyses will be presented in Chapter 4.

EVALUATING THE EFFECT OF M_R SELECTION ON OVERLAY THICKNESSES

Three different sets of M_R values were used to complete this analysis. In addition to the values calculated using a 41.4-kPa (6-psi) deviator stress and the actual field deviator stress, the AASHTO equation based on deflection measurements was used to determine the third set of M_R values. Overlay thicknesses were then determined by using the 1993 AASHTO NDT overlay design procedure for asphalt overlay on asphalt pavements. Finally, several analyses were performed to determine the effects of selected M_R values on the resulting overlay thicknesses. These results and analyses will be presented

and discussed further in Chapter 5.

CHAPTER SUMMARY

This chapter presented the data collection and overall evaluation strategies followed in this research project. Site selection, data collection, resilient modulus calculations, data base preparation, data analysis, effect of M_R on overlay thickness, and conclusions were the seven steps. Overall, each step provided a way to thoroughly satisfy the objectives of this research project.

CHAPTER 4

RESULTS FROM LABORATORY AND FIELD EVALUATIONS

INTRODUCTION

As described earlier, laboratory and back calculation tests were performed on subgrade soils gathered from primary roads in the State of Wyoming. Laboratory testing consisted of determining the soil's resilient modulus, R-value, water content, optimum water content, plasticity index, soil classification, and group index. Back calculation tests consisted of substituting values obtained from the field deflection tests into three computer programs to obtain a second set of resilient modulus values. This chapter presents the results from the above tests and provides a comprehensive discussion and analysis of the data gathered.

SITE CHARACTERISTICS

The pavement structure for each of the nine sites included in this research consisted of an asphalt concrete (AC) layer, a granular or treated base, and the underlying subgrade soil. Four of the nine sites had a treated base, three with an asphalt treated base (ATB) and one with a cement treated base (CTB). Table 4.1 summarizes the thicknesses of the AC and base layers for all test sections. All of these thicknesses were determined from the pavement cores at each test section. In addition, approximately 610 to 762-mm (24 to 30-in.) of the underlying subgrade soil was removed in each Shelby tube for laboratory testing.

TABLE 4.1 Thicknesses of Test Sections

Route	Mile Post	Pavement Thicknesses	
		Surface (mm)	Base (mm)
P-12	48	305	305
P-12	70	140	152
P-23	416	152	152
P-30	108	127	152
P-34	15	127	203
P-34	163	76	64*
P-44	229	152	203*
P-44	244	58	191*
F-25	197.4	152	203**

*Asphalt Treated Base (ATB)

**Cement Treated Base (CTB)

RESULTS FROM SOIL PROPERTY TESTS

Several fundamental soil property tests were conducted after performing the resilient modulus test on the soil samples. These tests included: sieve analysis, liquid and plastic limits (LL and PL, respectively), and water content determination. Tables 4.2 and 4.3 summarize the values for these soil property tests from the summer of 1992 and spring of 1993, respectively. A high percentage of the sites had water contents below the optimum water content. All of the soils were classified as A-4, A-6, or A-7-6 based on the AASHTO soil classification system. However, one sample from the summer of 1992 and two samples from the spring of 1993 had different soil classifications, A-1-B, A-2-4, and A-2-6, respectively. In addition, several of the sites had large group index (GI) values, indicating poor quality soils. Finally, the plasticity index (PI) values were moderate to high as shown in the tables.

TABLE 4.2 Fundamental Soil Properties for Samples Collected in the Summer of 1992

Route	Mile Post	Tube #	Layer #	Actual Water Content (%)	Optimum Water Content (%)	LL	PL	PI	AASHTO Soil Class.
P-12	70	2	1	11.8	15.8	29	15	14	A-6(3)
P-12	70	3	1	14.5	15.8	29	16	13	A-6(2)
P-12	70	3	2	13.8	15.4	28	16	12	A-6(2)
P-12	70	3	3	15.7	14.9	27	17	10	A-4(1)
P-23	416	1	1	14.4	12.5	22	NP	NP	A-4(0)
P-23	416	1	2	9.2	11.1	19	NP	NP	A-4(0)
P-23	416	1	3	14.6	13.4	24	19	5	A-4(0)
P-23	416	2	2	12.6	12.5	22	18	4	A-4(0)
P-23	416	2	3	11.9	12.0	21	18	3	A-4(0)
P-30	108	1	1	18.6	18.7	35	19	16	A-6(9)
P-30	108	1	2	16.6	16.3	30	17	13	A-6(4)
P-30	108	1	3	18.7	18.7	35	16	19	A-6(11)
P-30	108	2	1	18.5	19.6	37	18	19	A-6(11)
P-30	108	2	2	16.4	16.8	31	16	15	A-6(6)
P-30	108	2	3	19.0	19.2	36	18	18	A-6(8)
P-30	108	3	1	19.1	18.2	34	17	17	A-6(10)
P-30	108	3	2	17.6	16.8	31	16	15	A-6(7)
P-34	15	1	2	25.8	25.4	49	20	29	A-7-6(29)
P-34	15	1	3	21.1	21.1	40	23	17	A-6(16)
P-34	15	2	1	4.7	11.1	19	13	6	A-1-B(0)
P-34	15	2	2	19.5	24.4	47	23	24	A-7-6(23)
P-34	15	3	2	17.6	23.5	45	19	26	A-7-6(25)
P-34	15	3	3	17.9	29.7	58	24	34	A-7-6(32)
P-34	163	1	1	15.5	15.4	28	15	13	A-6(4)
P-34	163	1	2	17.2	14.4	26	14	12	A-6(5)
P-34	163	1	3	19.0	13.4	24	16	8	A-4(3)
P-34	163	2	2	15.4	13.0	23	14	9	A-4(3)
P-44	244	1	1	15.2	18.2	34	16	18	A-6(11)
P-44	244	1	2	15.2	17.3	32	17	15	A-6(7)
P-44	244	1	3	18.3	20.6	39	19	20	A-6(14)
P-44	244	2	1	12.2	18.7	35	16	19	A-6(10)
P-44	244	2	2	12.2	18.2	34	16	18	A-6(9)
P-44	244	2	3	14.4	20.1	38	17	21	A-6(14)

TABLE 4.3 Fundamental Soil Properties for Samples Collected in the Spring of 1993

Route	Mile Post	Tube #	Layer #	Actual Water Content (%)	Optimum Water Content (%)	LL	PL	PI	AASHTO Soil Class.
P-12	48	1	1	14.2	13.9	25	15	10	A-4(1)
P-12	48	1	2	15.9	18.7	35	7	22	A-6(13)
P-12	48	2	1	11.9	14.4	26	13	13	A-6(1)
P-23	416	1	1	10.8	14.9	27	20	7	A-4(0)
P-23	416	1	2	11.5	14.4	26	20	6	A-4(0)
P-23	416	2	1	13.2	14.4	26	18	8	A-4(0)
P-23	416	2	2	13.9	14.4	26	21	5	A-4(0)
P-23	416	3	1	12.8	14.4	26	19	7	A-4(0)
P-23	416	3	2	11.3	15.4	28	19	9	A-2-4(0)
P-30	108	1	1	15.5	17.7	33	15	18	A-6(9)
P-30	108	1	2	15.8	13.0	23	14	9	A-4(3)
P-30	108	2	1	16.8	15.4	28	14	14	A-6(6)
P-30	108	3	1	15.4	20.1	38	15	23	A-6(13)
P-34	15	1	1	19.7	25.9	50	24	26	A-7-6(26)
P-34	15	1	2	20.1	28.2	55	27	28	A-7-6(30)
P-34	15	2	1	18.7	26.3	51	22	29	A-7-6(29)
P-34	15	2	2	20.7	22.0	42	26	16	A-7-6(16)
P-34	15	3	1	20.6	23.5	45	23	22	A-7-6(23)
P-34	15	3	2	20.8	22.5	43	28	15	A-7-6(15)
P-34	163	1	1	9.8	21.1	40	20	20	A-2-6(2)
P-34	163	1	2	15.9	17.7	33	16	17	A-6(7)
P-44	229	1	1	23.7	19.6	37	24	13	A-6(7)
P-44	229	1	2	20.7	24.4	47	24	23	A-7-6(21)
P-44	229	2	1	25.3	19.6	37	26	11	A-6(3)
P-44	229	2	2	21.1	17.3	52	24	28	A-7-6(26)
P-44	229	3	1	20.9	24.4	47	21	26	A-7-6(23)
P-44	229	3	2	19.8	26.3	51	22	29	A-7-6(27)
P-44	244	1	1	11.3	21.1	40	17	23	A-6(16)
P-44	244	1	2	17.4	22.5	43	18	25	A-7-6(20)
P-44	244	2	1	12.8	21.6	41	18	23	A-7-6(17)
P-44	244	2	2	15.5	22.0	42	19	23	A-7-6(17)
P-44	244	3	1	19.3	21.1	40	18	22	A-6(16)
P-44	244	3	2	16.2	22.0	42	20	22	A-7-6(16)
F-25	197.4	1	1	15.2	23.5	45	20	25	A-7-6(21)
F-25	197.4	1	2	13.8	14.9	27	18	9	A-4(5)
F-25	197.4	2	1	17.4	25.9	50	19	31	A-7-6(25)
F-25	197.4	2	2	12.7	15.4	28	21	7	A-4(4)

LABORATORY RESILIENT MODULUS VALUES BASED ON 41.4-kPa (6-psi) DEVIATOR STRESS

Samples for laboratory M_R testing were primarily selected from the middle portion of each Shelby tube because of the visible disturbance on both ends. This selection process provided samples that were rigid and “undisturbed.” After extracting each sample from the tube, diameter and height measurements were taken and recorded on laboratory data sheets. Next, a rubber membrane was placed over the soil sample and two porous stones which were on both ends of the sample. After aligning the sample in the testing device, the resilient modulus tests were conducted by following the AASHTO specifications. During each of the fifteen loading conditions, the last five cycles were saved on disk for future retrieval to determine the resilient modulus value. After completing the first set of tests on undisturbed samples, another set of tests were conducted on disturbed samples using a zero confining pressure and deviator stresses ranging from 13.8 to 69.0-kPa (2 to 10-psi) in 13.8-kPa (2-psi) increments. The disturbed samples were prepared by destroying the original undisturbed samples and re-compacting them in five, equal lifts under a static load.

For each of the testing conditions described earlier, the peak deformation and applied load readings were retrieved from the files created during testing. By entering these values into a special spreadsheet, resilient modulus values were calculated from each of the different confining and deviator stress conditions. Figure 4.1 shows an example of the M_R summary spreadsheet used in this research (Note: values in metric units). The upper half of the spreadsheet shows the measured values under different testing conditions. These values include: mean deviator load, mean applied deviator stress, mean recoverable deformation from each LVDT, mean resilient strain, and mean M_R value. A logarithmic plot of resilient modulus versus deviator stress was

CHAMBER CONFINING PRESSURE (kPa)	NOMINAL DEVIATOR STRESS (kPa)	MEAN DEVIATOR LOAD (N)	STANDARD DEVIATION OF LOAD (N)	MEAN APPLIED DEV. STRESS (kPa)	MEAN RECOV. DEF. LVDT #1 (m)	MEAN RECOV. DEF. LVDT #2 (m)	MEAN RECOV. DEF. LVDT #3 (m)	MEAN RECOV. DEF. (m)	STD. DEV. OF RECOV. DEF. (m)	MEAN OF RESILIENT STRAIN (m/m)	MEAN OF Mr (kPa)	STD. DEV. OF Mr (kPa)
41.4	13.8	70.2855	0.3634	16.92	4.40282E-05	4.37182E-05		4.38732E-05	6.93317E-07	2.88122E-04	58720	802.1
41.4	27.6	103.9076	0.3634	25.01	9.70484E-05	9.58081E-05		9.64283E-05	8.83790E-07	6.33295E-04	39492	285.3
41.4	41.4	161.5088	0.1942	38.87	2.89595E-04	2.83394E-04		2.86494E-04	1.69844E-06	1.88148E-03	20661	138.8
41.4	55.2	201.9946	0.3071	48.01	4.29400E-04	4.11139E-04		4.18299E-04	1.29739E-06	2.74894E-03	17698	49.8
41.4	68.9	235.7035	0.4953	56.73	5.24820E-04	5.06076E-04		5.15318E-04	1.38639E-06	3.38417E-03	16763	33.0
20.7	13.8	67.5922	0.4952	16.27	4.03075E-05	4.09277E-05		4.08176E-05	4.24519E-07	2.06742E-04	60990	651.5
20.7	27.6	101.9964	0.5664	24.55	9.85986E-05	9.89087E-05		9.87537E-05	1.03994E-06	6.48530E-04	37854	385.2
20.7	41.4	201.9076	0.6444	48.59	3.95834E-04	3.88194E-04		3.91914E-04	1.29739E-06	2.57376E-03	18880	57.2
20.7	55.2	212.2464	0.5828	51.08	4.32841E-04	4.23540E-04		4.28181E-04	1.28632E-06	2.81199E-03	18166	90.7
20.7	68.9	246.9111	0.7892	59.42	5.40943E-04	5.37022E-04		5.41983E-04	8.48641E-07	3.55928E-03	10090	94.5
0.0	13.8	62.1187	0.4344	14.95	3.53466E-05	3.72069E-05		3.62768E-05	1.49104E-06	2.38239E-04	62833	2443.4
0.0	27.6	93.6561	0.3887	22.54	1.36426E-04	1.33325E-04		1.34875E-04	1.09622E-06	8.85747E-04	25449	138.3
0.0	41.4	150.5618	0.9517	36.24	3.05435E-04	3.06338E-04		3.07888E-04	3.39598E-06	2.02195E-03	17922	84.1
0.0	55.2	205.6435	0.5827	49.49	4.89194E-04	4.80513E-04		4.94853E-04	2.28954E-06	3.24978E-03	15230	57.8
0.0	68.9	251.6026	0.7139	60.95	6.59185E-04	6.46102E-04		6.52203E-04	1.59007E-06	4.28621E-03	14128	16.6

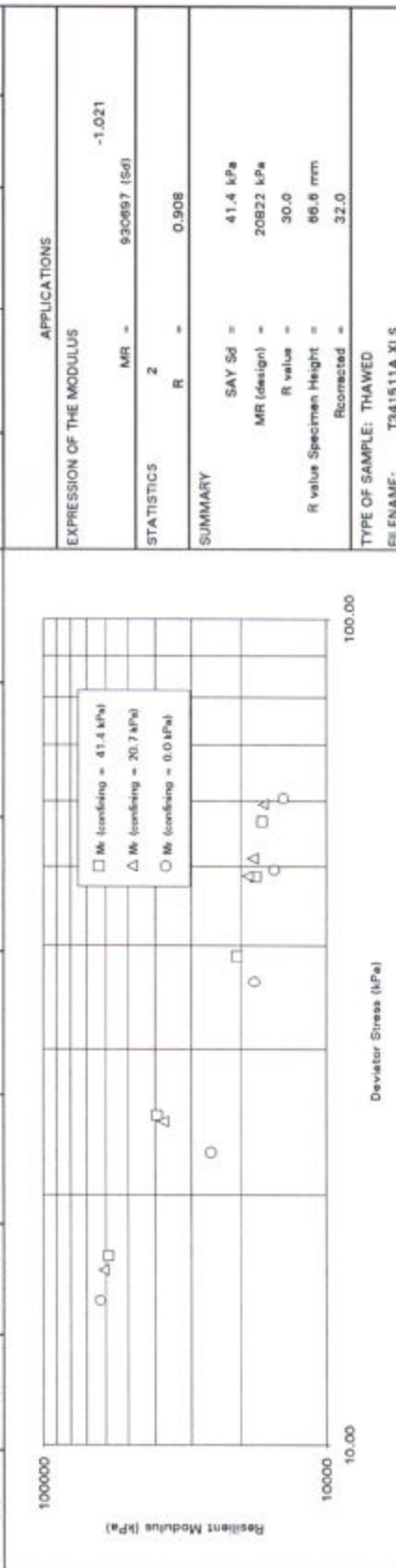


Figure 4.1 M_R Summary Spreadsheet

created using these values as shown in the lower left hand corner. In addition, a simple linear regression analysis was performed to develop a general equation for determining the resilient modulus value as a function of the deviator stress ($M_R = f(\sigma_d)$). After obtaining the equation, a deviator stress of 41.4-kPa (6-psi), suggested in the literature, was substituted into the equation to determine a design resilient modulus value. The lower right hand portion of the spreadsheet summarizes other important laboratory information, such as the R-value specimen height, the linear regression equation for M_R , the coefficient of determination (R^2), and the condition of the sample. Similar spreadsheets were created for each test site and sample condition (undisturbed ring, undisturbed actuator, disturbed ring, and disturbed actuator). An example of the M_R calculation sheet used to create the summary sheet is presented in Appendix B.

Tables 4.4 and 4.5 summarize all the laboratory subgrade soil resilient modulus values. Some of the tests were not completed, shown by a blank space, because testing could not be performed on an undisturbed sample or the values obtained from the test were significantly beyond the range of realistic subgrade M_R values.

BACK CALCULATED RESILIENT MODULUS VALUES

Information obtained from pavement coring and deflection testing were used in this part of the research. First, the deflection measurements were corrected to a standard temperature of 21° Celsius (70° Fahrenheit) using a computer program called TAFFY (1988). This computer program produces temperature adjustment factors for the deflection readings based on an algorithm recommended by the Asphalt Institute. This program requires the following information: thicknesses of asphalt cement and untreated base layers, surface temperature, and previous 5-day air temperature history. Since the

previous 5-day air temperature history was not

TABLE 4.4 M_R values for Samples Collected in the Summer of 1992

Route	Mile Post	Tube #	Layer #	Undisturbed M _R Ring LVDT's (kPa)	Undisturbed M _R Actuator LVDT's (kPa)	Disturbed M _R Ring LVDT's (kPa)	Disturbed M _R Actuator LVDT's (kPa)
P-12	70	2	1	81409		102273	
P-12	70	3	1	55494		53121	
P-12	70	3	2	56124		103358	
P-12	70	3	3	30437		23413	34676
P-23	416	1	1	40523	32013	3828	3747
P-23	416	1	2	40359	38045	22385	19998
P-23	416	1	3	78239	56686	43649	27530
P-23	416	2	2			34984	30535
P-23	416	2	3			25282	25289
P-30	108	1	1	33609	28116	89353	69766
P-30	108	1	2	20216	21729	70828	127373
P-30	108	1	3	21037	19099	122068	41855
P-30	108	2	1	25969	25570	183266	78644
P-30	108	2	2	27086	23497	154540	68416
P-30	108	2	3	42013	13461	103127	69262
P-30	108	3	1			102582	68795
P-30	108	3	2			88844	57949
P-34	15	1	2			53590	29824
P-34	15	1	3			102586	26874
P-34	15	2	1			60861	35786
P-34	15	2	2	45331	22623	202782	71616
P-34	15	3	2			118568	40237
P-34	15	3	3				50588
P-34	163	1	1	15557	14986	78856	48163
P-34	163	1	2	22208	17596	8156	6461
P-34	163	1	3	4615	4682	1482	1647
P-34	163	2	2			41275	34597
P-44	244	1	1	28029	40635	244357	94602
P-44	244	1	2	62172	53025	150085	85572
P-44	244	1	3	69564	44153	259907	78390
P-44	244	2	1	136841	102037		
P-44	244	2	2			194743	80376
P-44	244	2	3	137073	84348		

TABLE 4.5 M_R values for Samples Collected in the Spring of 1993

Route	Mile Post	Tube #	Layer #	Undisturbed M _R Ring LVDT's (kPa)	Undisturbed M _R Actuator LVDT's (kPa)	Disturbed M _R Ring LVDT's (kPa)	Disturbed M _R Actuator LVDT's (kPa)
P-12	48	1	1	35746	27290	69153	33115
P-12	48	1	2	86113	24374	227127	65801
P-12	48	2	1			48012	31919
P-23	416	1	1			23214	15871
P-23	416	1	2	42306	38031	46812	26866
P-23	416	2	1			30008	25822
P-23	416	2	2			25057	18387
P-23	416	3	1	44618	40685	27969	22010
P-23	416	3	2	61275	51161	65048	41937
P-30	108	1	1			33053	28513
P-30	108	1	2	16499	14259	73173	49957
P-30	108	2	1			370022	81867
P-30	108	3	1	94825	68252	577153	96091
P-34	15	1	1	25316	20822	200771	53147
P-34	15	1	2	13846	10976	94507	42501
P-34	15	2	1	19046	17268	177558	59079
P-34	15	2	2			116548	57137
P-34	15	3	1	16594	12681	148110	56176
P-34	15	3	2	16304	13585	136508	78748
P-34	163	1	1			41063	32757
P-34	163	1	2			73476	40652
P-44	229	1	1			44362	27866
P-44	229	1	2			209110	65484
P-44	229	2	1			29354	22639
P-44	229	2	2	127381	84834	424881	56291
P-44	229	3	1	44407	36517	89182	51909
P-44	229	3	2	129986	67978	207017	56342
P-44	244	1	1	85595	53809	457417	65533
P-44	244	1	2	56043	38112	441268	63933
P-44	244	2	1	64007	51445	276190	52213
P-44	244	2	2	54198	37799	639168	56793
P-44	244	3	1	84936	66139	312769	68665
P-44	244	3	2	49654	39157	249198	69109
F-25	197.4	1	1	274532	64340	237480	89856
F-25	197.4	1	2	80866	55988	215178	66024
F-25	197.4	2	1			208297	76290
F-25	197.4	2	2			134948	66712

known in this research, an average high and low temperature were entered into the program. These values were obtained by taking the recorded air temperature during the deflection testing and adding or subtracting 8.33 degrees to obtain Celsius temperatures (15 degrees to obtain Fahrenheit temperatures), respectively.

The computer programs, MODULUS, EVERCALC, and BOUSDEF were used in this analysis. All three programs require the following input parameters: magnitude of the load creating the deflection basin, the FWD load plate radius, distance of the sensors from center of the load plate, corrected deflection measurements, layer thicknesses, and the estimated Poisson's ratio values for all layers. Tables 4.6 and 4.7 summarize the back calculated resilient modulus values obtained with these three programs.

RESULTS FROM R-VALUE TESTS

After completing all resilient modulus tests, the soil samples were re-compacted and prepared for the R-value test. Each soil sample was compacted in a 102-mm (4-in.) diameter and 64-mm (2.5-in.) high mold by using static load compaction. The specifications for Resistance R-Value and Expansion Pressure of Compacted Soils AASHTO T 190 (ASTM D 2844) outline these testing procedures. In addition, the final R-values were corrected for variations in specimen height. Tables 4.8 and 4.9 summarize the results of the R-value tests for both sampling periods.

STATISTICAL ANALYSIS

As mentioned earlier, data were obtained from nine different sites during two different time periods, the summer of 1992 and the spring of 1993. Five of these sites were common to

TABLE 4.6 Back Calculated Resilient Modulus Values (Summer of 1992)

Route	Mile Post	Project Number	M _R from MODULUS Program (kPa)	M _R from EVERCALC Program (kPa)	M _R from BOUSDEF Program (kPa)
P-12	70	22	71016	91838	87977
P-23	416	2	129621	196294	181677
P-30	108	18	21374	94458	45505
P-34	15	14	57916	98319	90183
P-34	163	11	91011	87426	114936
P-44	244	7	106179	172989	151409

TABLE 4.7 Back Calculated Resilient Modulus Values (Spring 1993)

Route	Mile Post	Project Number	M _R from MODULUS Program (kPa)	M _R from EVERCALC Program (kPa)	M _R from BOUSDEF Program (kPa)
P-12	48	22	80669	133758	81358
P-23	416	2	170990	195811	175816
P-30	108	18	60674	134448	56537
P-34	15	14	74463	95837	79290
P-34	163	11	102732	96527	97906
P-44	229	8	184779	184090	174437
P-44	244	7	124106	156511	132379
F-25	197.4	5	188906	200637	197190

TABLE 4.8 R-Values for Samples Collected in the Summer of 1992

Route	Mile Post	Tube #	Layer #	R value
P-12	70	2	1	45
P-12	70	3	1	47
P-12	70	3	2	52
P-12	70	3	3	48
P-23	416	1	1	47
P-23	416	1	2	57
P-23	416	1	3	56
P-23	416	2	2	44
P-23	416	2	3	48
P-30	108	1	1	31
P-30	108	1	2	37
P-30	108	1	3	39
P-30	108	2	1	38
P-30	108	2	2	37
P-30	108	2	3	35
P-30	108	3	1	32
P-30	108	3	2	38
P-34	15	1	2	32
P-34	15	1	3	34
P-34	15	2	1	42
P-34	15	2	2	38
P-34	15	3	2	38
P-34	15	3	3	37
P-34	163	1	1	32
P-34	163	1	2	24
P-34	163	1	3	NA
P-34	163	2	2	39
P-44	244	1	1	45
P-44	244	1	2	41
P-44	244	1	3	49
P-44	244	2	1	83
P-44	244	2	2	43
P-44	244	2	3	68

TABLE 4.9 R-Values for Samples Collected in the Spring of 1993

Route	Mile Post	Tube #	Layer #	R value
P-12	48	1	1	45
P-12	48	1	2	32
P-12	48	2	1	36
P-23	416	1	1	46
P-23	416	1	2	45
P-23	416	2	1	45
P-23	416	2	2	48
P-23	416	3	1	52
P-23	416	3	2	59
P-30	108	1	1	35
P-30	108	1	2	40
P-30	108	2	1	33
P-30	108	3	1	36
P-34	15	1	1	32
P-34	15	1	2	28
P-34	15	2	1	33
P-34	15	2	2	28
P-34	15	3	1	31
P-34	15	3	2	32
P-34	163	1	1	42
P-34	163	1	2	32
P-44	229	1	1	37
P-44	229	1	2	29
P-44	229	2	1	42
P-44	229	2	2	30
P-44	229	3	1	30
P-44	229	3	2	31
P-44	244	1	1	50
P-44	244	1	2	43
P-44	244	2	1	42
P-44	244	2	2	39
P-44	244	3	1	44
P-44	244	3	2	40
F-25	197.4	1	1	47
F-25	197.4	1	2	50
F-25	197.4	2	1	39
F-25	197.4	2	2	49

both time periods, one was specific to the summer of 1992, and three were specific to the spring of 1993. Table 4.10 summarizes the sites analyzed in each time period.

TABLE 4.10 Summary of Test Sites Included in Each Period

Route	Mile Post	Summer of 1992	Spring of 1993
P-12	48		X
P-12	70	X	
P-23	416	X	X
P-30	108	X	X
P-34	15	X	X
P-34	163	X	X
P-44	229		X
P-44	244	X	X
F-25	197.4		X

As a result of the laboratory and back calculation tests, several measured variables were available for analysis. These variables included: the resilient modulus (measured under four conditions), R-value, and certain soil characteristics (actual and optimum water contents, plasticity index, soil classification, and group index). Because the nine sites had a variety of soil classifications, statistical analyses were completed by taking into account these differences as necessary. In addition, all analyses were based upon $\log_{10}(M_R)$, abbreviated as LMR, instead of M_R itself because this minimized the differences between high and low resilient modulus values obtained at each test site.

Relationship Between Resilient Modulus and R-Value

Because the resilient modulus and the R-value provide similar information on a section's subgrade, one would assume that a relationship exists between these two laboratory tests. As a result,

correlations were obtained between the measured R-values and the four measured resilient modulus conditions for both time periods. These four conditions were the undisturbed M_R from the ring, the undisturbed M_R from the actuator, the disturbed M_R from the ring, and the disturbed M_R from the actuator. Recall, the ring refers to the LVDT's placed inside of the testing chamber and the actuator refers to the LVDT's placed on the loading piston. Table 4.11 presents the correlations obtained from these laboratory measurements. Comparisons can be made within the

TABLE 4.11 Correlations Between LMR¹ and R-Value

	Undisturbed		Disturbed		Sample Size
	Ring	Actuator	Ring	Actuator	
Summer of 1992	0.630	0.749	-0.041	-0.089	16
Spring of 1993	0.334	0.437	-0.219	-0.273	23
Pooled ²	0.380	0.509	-0.136	-0.142	39

¹Log₁₀ (Resilient Modulus Values)

²Pooled (1992 & 1993)

rows of this table because they are based on the same soil samples. However, differences in the soil classifications between Periods A and B may distort comparisons between rows. Overall, this table shows that the disturbed soil LMR's were not significantly correlated with the R-value, but that the undisturbed soil LMR's were correlated with the R-value. Correlations between undisturbed and disturbed LMR's (not shown) were modest to nonexistent. Therefore, samples should remain undisturbed if the resilient modulus is to be a meaningful measure for pavement design. Only undisturbed LMR's were used in remaining analyses, unless noted otherwise.

The Effect of Sensor Locations on M_R Measurements

The correlations shown in Table 4.11 also favor the placement of the LVDT's outside the testing chamber on the loading piston (actuator) instead of on the rings inside the chamber. However, observed differences in the correlations with the R-values were not extreme, and placements were also compared on the basis of measurement precision. In order to ensure that all variability measured was attributable to differences in measurement methods, values were adjusted for site, period, and sample tube. The test for differences in variances for paired data (Snedecor & Cochran, 1989) showed the ring variance to be greater than the actuator variance ($t = 2.238$, $df = 20$, $p = 0.0368$). The greater variation in ring measurements can be explained by the fact that it is difficult to obtain good contact between the LVDT's on the ring and the soil sample. Therefore, the remaining analyses were completed using actuator measurements only.

Although measurements at the actuator appear to be preferable, the possible relationship between actuator and ring measures was examined. Table 4.12 shows a high correlation between actuator and ring measurements of LMR. In addition, a t-test of paired differences indicates that ring measurements were on average higher than actuator measurements. For undisturbed

TABLE 4.12 Relations Between LMRR¹ and LMRA²

	Correlation	Mean Diff.	t	df	p-value
Summer of 1992	0.858	0.0987	2.94	17	0.009
Spring of 1993	0.906	0.1576	5.11	22	<0.0001
Pooled	0.885	0.1317	5.75	40	<0.0001

¹Log₁₀ (Resilient Modulus Value for Ring Measurement)

²Log₁₀ (Resilient Modulus Value for Actuator Measurement)

samples, a repeated measures analysis indicates a similarity in differences between ring and actuator measurements ($p = 0.206$).

The Effect of Sample Locations on M_R Values

Sample selection from the Shelby tubes is an important issue when determining the resilient modulus value. If the layers within a tube systematically differ from each other, with the upper portion consistently having higher or lower values than the lower portion, one would expect a noticeable difference in the values obtained from the selected samples. However, available data do not yield evidence of such differences (repeated measures analysis $F_{2,13} = 1.27$, $p = 0.3126$). On the other hand, if one assumes the layers are similar to each other, averaging the LMR values will give more reliable results than using the value from a single layer. Overall, it is not possible with the available data in this research to select one layer over another without an additional reference criterion.

Relationship Between Back Calculated and Laboratory M_R Values

Besides laboratory testing, M_R values can also be determined by back calculations using information from non-destructive tests. As mentioned earlier, the following three back calculation computer programs were utilized in the research: MODULUS (MP), EVERCALC (EP), and BOUSDEF (BP). In order to consider the quality of these programs, logs of back calculated values (designated as LMR-MP, LMR-EP, and LMR-BP, respectively) were compared to laboratory LMR values. The site-by-period mean LMR from undisturbed samples measured on the actuator was used as the best available value for the “true” resilient modulus, the one exception being a single site for which only ring measurements were available in Period A. Because means were calculated from a different number of observations, a weighted analysis was used (weight = sample size). Table 4.13 presents the results of this analysis. Note that the EVERCALC program appears to be slightly superior to the other two back calculation programs. In general, all back calculated values match better with each other than they do with the laboratory measurements.

Assuming constant differences between logs of back calculated and laboratory values, the best estimated differences appear in Table 4.14, along with implied relationships between laboratory and back calculated values of M_R . A 95% confidence interval for the appropriate correction factor (C) for subgrade soils in Wyoming, based on the EVERCALC program, is [0.20, 0.32], where $M_R = C * [\text{back calculated } M_R \text{ value}]$.

Relationship Between M_R Values and Soil Properties

Another important question to consider when selecting a M_R value is the relationship with common soil properties. The possible relationship between LMR and four factors, moisture = (actual % water content - optimum % water content), plasticity index, soil classification, and group index were analyzed. Because the group and plasticity indices were highly correlated, only one was ultimately considered for describing soil- M_R relationships, group index (GI).

Moisture and LMR were related, and their relationship depended on soil type. Similar strengths of the relationship between soil factors and responses were found for both undisturbed and disturbed (remolded) samples, and also for R-values (refer to Table 4.15). All of the test sections had one or more of the following types of AASHTO subgrade soil: A-4, A-6, and A-7-6. For each of these classifications correlations were developed to determine the effect of moisture on the measured values. Overall, values for undisturbed and remolded M_R values and R-values from A-4 and A-6 soils decreased as water content increased. The A-7-6 subgrade soils, however, showed very little change in the measured values (refer to Table 4.16).

TABLE 4.13 Back Calculation Correlations (N = 13)

	Weighted Correlations with LMR ¹	Cross-correlations		
		LMR-MP	LMR-EP	LMR-BP
LMR-MP ²	0.526	1.000	0.744	0.941
LMR-EP ³	0.735	0.744	1.000	0.799
LMR-BP ⁴	0.590	0.941	0.799	1.000

¹Log₁₀(Resilient Modulus Values)

²Log₁₀(Resilient Modulus Values from MODULUS Program)

³Log₁₀(Resilient Modulus Values from EVERCALC Program)

⁴Log₁₀(Resilient Modulus Values from BOUSDEF Program)

TABLE 4.14 Back Calculation Relationships (N = 13)

Computer Program	Diff.	Standard Error	95% CI	Relation	Bounds on C (M _R = C * [X])
MODULUS (MP)	0.408	0.073	(0.249, 0.567)	M _R = 0.39MP	(0.27, 0.56)
EVERCALC (EP)	0.599	0.049	(0.492, 0.706)	M _R = 0.25EP	(0.20, 0.32)
BOUSDEF (BP)	0.503	0.059	(0.374, 0.632)	M _R = 0.31BP	(0.23, 0.42)

CHAPTER SUMMARY

In this chapter, the results from the laboratory tests and back calculation computer programs were presented. Several statistical analyses were also conducted and summarized to evaluate the factors influencing the determination of the M_R value used in designing new pavements or overlays. In general, these analyses indicated that the design resilient modulus value should be chosen based on laboratory tests using undisturbed soil samples and the actuator LVDT's. Multiple M_R values obtained from the same Shelby tube should also be averaged to give a better representation of the subgrade soil. The M_R values calculated from the equations based on the actuator LVDT deformation readings will be used to determine overlay thicknesses at each test site. This analysis will be presented and analyzed in the following chapter.

TABLE 4.15 Coefficients of Determination for Soil-M_R Relations

Models (linear models with interaction)	Undisturbed samples LMR ¹	Remolded samples LMR ¹	R-value
Moisture and Soil Classification	0.427	0.436	0.478
Moisture and Group Index	0.479	0.286	0.321

¹Log₁₀(Resilient Modulus Values)

TABLE 4.16 Parameter Estimates ± Standard Error for Model with GI

Soil Classification	Parameter Estimates	Undisturbed Samples LMR ¹	Remolded Samples LMR ¹	R-value
A-4	Intercept	4.50 ± 0.0740	4.35 ± 0.0893	47.1 ± 1.21
	Slope(Moisture)	-0.102 ± 0.0286	-0.0803 ± 0.0383	-0.845 ± 0.619
A-6	Intercept	4.38 ± 0.0548	4.685 ± 0.0524	37.9 ± 1.96
	Slope(Moisture)	-0.0682 ± 0.0148	-0.0401 ± 0.0162	-2.04 ± 0.570
A-7	Intercept	4.54 ± 0.151	4.73 ± 0.0435	31.0 ± 1.97
	Slope(Moisture)	0.0110 ± 0.0250	-0.00492 ± 0.00699	-0.762 ± 0.316

¹Log₁₀(Resilient Modulus Values)

CHAPTER 5

EFFECT OF M_R SELECTION ON OVERLAY THICKNESSES

INTRODUCTION

In order to design overlays for existing pavement sections using the AASHTO design guide, a M_R value must be selected to represent the characteristics of the subgrade soil, specifically, the stress conditions. When laboratory testing is completed to determine this value, a single deviator stress is often chosen to represent the design conditions. The deviator stress suggested in the literature is 41.4-kPa (6-psi). However, the actual deviator stress may be determined by using data from the field. If the actual field deviator stress is less than 41.4-kPa (6-psi), then the selected M_R value is conservative which may result in a thick overlay. On the other hand, if the field deviator stress is higher than 41.4-kPa (6-psi), then the selected M_R value is higher than the actual one which can result in a thin overlay. This chapter presents an evaluation of how three different procedures for determining resilient modulus (laboratory with a 41.4-kPa [6-psi] deviator stress, laboratory with actual deviator stress, and the AASHTO equation with field deflection measurements) affect the resulting overlay thicknesses.

LABORATORY RESILIENT MODULUS VALUES BASED ON ACTUAL FIELD STRESSES

The design resilient modulus values computed from the laboratory analysis in Chapter IV were based on a deviator stress of 41.4-kPa (6-psi). Since the thicknesses of each pavement

section were available from the field evaluation, this information was used to compute the “actual” deviator stresses in the subgrades. The computer program, BISAR, was used in this analysis, assuming a 40-kN (9000-lbs.) wheel load, a 689-kPa (100-psi) tire pressure, and a three layer pavement structure (refer to Figure 5.1). The thicknesses of the AC and base layers along

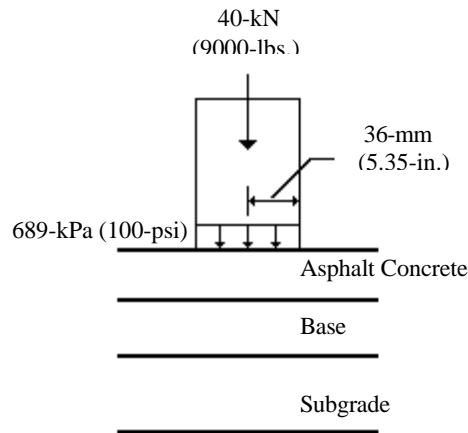


Figure 5.1 Assumptions Made in Calculating Actual Field Stresses

with typical Young’s Modulus values used by the Wyoming DOT (refer to Table 3.3) were used in this analysis. In addition, the undisturbed actuator M_R value, calculated by using a 41.4-kPa (6-psi) deviator stress, was entered into this program as the first seed moduli. Several iterations were then completed by taking the resulting deviator stress and substituting this value into the regression equation developed from the laboratory tests using undisturbed samples and the actuator LVDT’s. The M_R value computed from the previous trial was inputted each time as the seed moduli until the resulting deviator stress changed by less than 3.5-kPa (0.5-psi). Tables 5.1 and 5.2 summarize the undisturbed actuator M_R values based on 41.4-kPa (6-psi) and actual field deviator stresses. M_R values for undisturbed samples, based on actuator LVDT measurements,

TABLE 5.1 M_R Values Based on 41.4-kPa (6-psi) and Actual Field Deviator Stresses for Summer of 1992 Data

Route	Mile Post	Field Deviator Stress (kPa)	M_R (kPa) field	M_R (kPa) 41.4-kPa
P-12	70	51.7	59302	81409
P-12	70	49.8	47656	55494
P-12	70	50.2	51376	56124
P-12	70	39.9	31375	30437
P-23	416	38.3	34501	32013
P-23	416	39.9	39315	38045
P-23	416	46.6	54946	56686
P-30	108	41.5	28000	28116
P-30	108	38.0	22753	21729
P-30	108	35.5	23001	19099
P-30	108	40.7	25959	25570
P-30	108	40.1	24290	23497
P-30	108	31.5	18707	13461
P-34	15	32.5	23435	22623
P-34	163	41.3	15007	14986
P-34	163	42.4	17117	17596
P-44	244	48.1	34691	40635
P-44	244	54.4	46460	53025
P-44	244	49.8	39099	44153
P-44	244	78.6	98883	102037
P-44	244	66.7	69506	84348

were used in this part of the analysis simply because it was found in Chapter 4 that they best represent the characteristics of subgrade soil samples.

Comparison of Laboratory M_R Values

Because laboratory M_R values were calculated using two different stress conditions, one would want to know if there is any statistical difference between using the actual deviator stress and the

assumed value of 41.4-kPa (6-psi). Therefore, the test for differences for paired data was performed.

The data were placed in two groups, granular and treated, because four of the nine

TABLE 5.2 M_R Values Based on 41.4-kPa (6-psi) and Actual Field Deviator Stresses for Spring of 1993 Data

Route	Mile Post	Field Deviator Stress (kPa)	M_R (kPa) field	M_R (kPa) 41.4-kPa
P-12	48	15.7	73656	27290
P-12	48	11.2	48202	24374
P-23	416	39.6	39183	38301
P-23	416	40.8	41286	40685
P-23	416	43.6	49157	51161
P-30	108	31.3	17104	14259
P-30	108	60.4	55196	68252
P-34	15	32.8	26410	20822
P-34	15	26.9	15761	10976
P-34	15	33.4	21942	17268
P-34	15	29.5	17681	12681
P-34	15	29.7	18541	13585
P-44	229	28.3	129331	84834
P-44	229	21.4	71972	67978
P-44	244	53.8	44040	53809
P-44	244	46.9	31429	38112
P-44	244	50.1	42370	51445
P-44	244	46.6	33409	37799
P-44	244	56.7	51936	66139
P-44	244	46.5	36358	39157
F-25	197.4	12.1	43693	64340
F-25	197.4	17.0	73102	55988

sites had some type of treated base, either ATB or CTB. After completing this separation, comparisons were completed on all the sites within each group and then by each site individually. Tables 5.3 and 5.4 summarize the tests for the granular and treated sites, respectively.

Both tables show that there is not a statistical difference between the two data sets. However, by examining the variances, one would favor using the computed field deviator stresses over the assumed 41.4-kPa (6-psi) value because of the reduction in the amount of variance or

TABLE 5.3 Testing Significance of Differences for Granular Base Sites (Summer of 1992 & Spring of 1993)

Route	Mile Post	M _R (41.4-kPa) Variance	M _R (field) Variance	t	df	p-value
P-12	48	4.25E+06	3.24E+08	3.11	1	0.198
P-12	70	4.33E+08	1.38E+08	1.71	3	0.184
P-23	416	8.53E+07	5.68E+07	0.403	5	0.744
P-30	108	3.08E+08	1.44E+08	0.064	7	0.951
P-34	15	2.20E+07	1.60E+07	6.06	5	0.002
Pooled		3.50E+08	2.49E+08	1.07	25	0.293

TABLE 5.4 Testing Significance of Differences for Treated Base Sites (Summer of 1992 & Spring of 1993)

Route	Mile Post	M _R (41.4-kPa) Variance	M _R (field) Variance	t	df	p-value
P-34	163	3.41E+06	2.23E+06	0.916	1	0.528
P-44	229	1.42E+08	1.65E+09	1.197	1	0.443
P-44	244	4.37E+08	4.00E+08	6.084	10	0.0001
F-25	197.4	3.49E+07	4.32E+08	0.094	1	0.941
Pooled		5.31E+08	8.53E+08	0.630	16	0.537

variability. By determining actual deviator stresses, the resulting M_R values were more consistent within each test site.

OVERLAY THICKNESS RESULTS

Several spreadsheets were developed to determine the overlay thicknesses for each test site. An example of this spreadsheet is shown in Appendix D. After entering the applied loads and corrected deflection measurements from the field FWD tests into the spreadsheet, several equations were solved in order to determine the SN_{eff} value of each test site. The first set of equations determined the M_R value based on the AASHTO equation (refer to Section 2.8.1). These M_R values were calculated by

using the corrected deflection measurements taken at the following sensor locations: 305, 457, 609, 914, 1219, and 1524-mm (12, 18, 24, 36, 48, and 60-in., respectively). The underlying assumption for the AASHTO equation is that at a certain distance away from the loading plate, the measured deflection is attributable solely to the subgrade. In order to determine this distance and the resulting M_R value which will be used for design purposes, several checks must be completed. The minimum distance from the loading plate is determined with the following formula:

$$r \geq 0.7a_e \quad (5.1)$$

where: r = distance from center of load, inches
 a_e = radius of the stress bulb at the subgrade-pavement interface, inches

Two additional equations provide values related to this condition. First, the value of a_e is determined from the following formula:

$$a_e = \sqrt{a^2 + \left(D^3 \sqrt{\frac{E_p}{M_R}} \right)^2} \quad (5.2)$$

where: a_e = radius of the stress bulb at the subgrade-pavement interface, inches
 a = NDT load plate radius (5.91-in.)
 D = total thickness of pavement layers above the subgrade, inches
 E_p = effective modulus of all pavement layers above the subgrade, psi

Second, the value of E_p is determined from the following formula:

$$d_o = 1.5pa \left\{ \frac{1}{M_R \sqrt{1 + \left(\frac{D}{a} \sqrt{\frac{E_p}{M_R}} \right)^2}} + \frac{1 - \frac{1}{\sqrt{1 + \left(\frac{D}{a} \right)^2}}}{E_p} \right\} \quad (5.3)$$

- where:
- d_o = deflection measured at the center of the load plate, inches
 - p = NDT load plate pressure, psi
 - a = NDT load plate radius (5.91-in.)
 - D = total thickness of pavement layers above the subgrade, inches
 - M_R = subgrade resilient modulus, psi
 - E_p = effective modulus of all pavement layers above the subgrade, psi

These three constraints must be satisfied in order to determine the minimum distance. Once this distance is determined, the M_R value can then be adjusted with a correction factor before it is used to determine the SN_f value. In this research study, a correction factor of 0.33 was used. For each test site, nine M_R and E_p values were calculated because nine different loads were applied to each section. Final design values for both of these parameters were determined by taking a logarithmic average. Tables 5.5 and 5.6 summarize the M_R values from the three different methods: 41.4-kPa

TABLE 5.5 Summary of M_R Values from 3 Methods (Summer of 1992)

Route	Mile Post	M_R AASHTO (kPa)	M_R LAB (kPa)	M_R FIELD (kPa)
P-12	70	26193	52704	46202
P-23	416	45774	41024	42086
P-30	108	13548	21339	23601
P-34	15	22794	22622	23435
P-34	163	28758	10728	16030
P-44	244	38307	60626	53372

TABLE 5.6 Summary of M_R Values from 3 Methods (Spring of 1993)

Route	Mile Post	$M_{R \text{ AASHTO}}$ (kPa)	$M_{R \text{ LAB}}$ (kPa)	$M_{R \text{ FIELD}}$ (kPa)
P-12	48	22180	25793	59584
P-23	416	41562	42941	43003
P-30	108	17099	31199	30723
P-34	15	20119	14672	19733
P-44	229	51014	59495	96478
P-44	244	33577	46664	39328
F-25	197.4	59805	60019	56516

(6-psi) deviator stress, field deviator stress, and deflection measurements (referred to as LAB, FIELD, and AASHTO, respectively).

The three sets of M_R values were then used to compute the effective structural number (SN_f) using the flexible pavement design equation developed by AASHTO shown below:

$$\log_{10} W_{18} = z_R * S_o + 9.36 * \log_{10} (SN_f + 1) - 0.20 + \frac{\log_{10} \left[\frac{\Delta PSI}{4.2 - 1.5} \right]}{0.40 + \frac{1094}{(SN_f + 1)^{5.19}}} + 2.32 * \log_{10} M_R - 8.07 \quad (5.4)$$

- where:
- W_{18} = estimated future traffic, 18-kip ESALs
 - z_R = standard normal deviate (based on reliability factor)
 - S_o = overall standard deviation
 - SN_f = future design structural number
 - ΔPSI = design present serviceability index (PSI) loss
 - M_R = design resilient modulus value, psi

In this research project, the following three different estimated future levels of traffic (W_{18}) were used in the above equation: 800,000, 3,000,000, and 5,000,000 ESALs corresponding to low, medium, and high traffic levels, respectively. The following values were assumed for the rest of

the variables in the above equation: 85% reliability factor, 0.45 standard deviation, and 2.5 as the change in PSI (Δ PSI).

The SN_f values were determined for all test sections based on the three calculated M_R values and three different traffic levels. This analysis resulted in a total of nine SN_f values for each test site. Tables 5.7 and 5.8 summarize all SN_f results. Next, the SN_{eff} values were determined using the NDT overlay procedure and the averaged M_R (based on deflection measurements) and E_p values calculated earlier for each site. Tables 5.9 and 5.10 present the SN_{eff} values. Recall from Chapter 2, the SN_{eff} for the NDT procedure is calculated with the following formula:

$$SN_{eff} = 0.0045D^3\sqrt{E_p} \quad (5.5)$$

Finally, the overlay design equation was used to determine the resulting overlay thicknesses (D_{ol}) for each section. These values were obtained by taking the difference between the SN_f and SN_{eff} values ($SN_{ol} = SN_f - SN_{eff}$) and dividing this quantity by 0.44, the layer coefficient (a_{ol}) for new asphalt pavement. Tables 5.11 and 5.12 summarize the D_{ol} values obtained in this analysis.

STATISTICAL ANALYSIS

With three different methods for determining M_R (AASHTO, LAB, and FIELD), it would be of interest to know if there are any statistical differences in the calculated overlay thicknesses due to the method used. The negative thicknesses were left in the analysis in order to provide a better indication of the differences among methods. A repeated measures analysis showed no evidence of differences (null hypothesis) among the methods at low, medium, or high traffic levels ($F_{2,24} = 2.16$, p-value = 0.1367, $F_{2,24} = 2.18$, p-value = 0.1351, and $F_{2,24} = 2.18$, p-value =

0.1349,

TABLE 5.7 SN_f Values for the Summer of 1992 Data

Route	Mile Post	Traffic Level											
		Low			Medium			High					
		AASHTO M _R	LAB M _R	FIELD M _R	AASHTO M _R	LAB M _R	FIELD M _R	AASHTO M _R	LAB M _R	FIELD M _R	AASHTO M _R	LAB M _R	FIELD M _R
P-12	70	3.81	3.02	3.16	4.55	3.65	3.81	4.87	3.92	4.24	4.21	4.09	
P-23	416	3.17	3.29	3.26	3.82	3.96	3.92	4.10	5.18	5.03	5.04	5.64	
P-30	108	4.68	4.06	3.94	5.56	4.85	4.70	5.93	5.08	5.09	5.04	5.64	
P-34	15	3.98	3.99	3.94	4.75	4.76	4.71	5.08	5.09	5.09	5.04	5.64	
P-34	163	3.69	5.03	4.44	4.42	5.95	5.29	4.74	6.35	6.35	5.64	5.64	
P-44	244	3.36	2.88	3.01	4.04	3.49	3.63	4.33	3.75	3.75	3.90	3.90	

TABLE 5.8 SN_f Values for the Spring of 1993 Data

Route	Mile Post	Traffic Level											
		Low			Medium			High					
		AASHTO M _R	LAB M _R	FIELD M _R	AASHTO M _R	LAB M _R	FIELD M _R	AASHTO M _R	LAB M _R	FIELD M _R	AASHTO M _R	LAB M _R	FIELD M _R
P-12	48	4.01	3.83	2.90	4.79	4.58	3.51	5.12	4.90	4.18	4.18	3.77	
P-23	416	3.27	3.24	3.24	3.94	3.90	3.90	4.22	4.18	4.18	4.18	4.18	
P-30	108	4.36	3.60	3.61	5.19	4.31	4.33	5.54	4.62	4.64	4.64	4.64	
P-34	15	4.14	4.57	4.17	4.94	5.43	4.97	5.28	5.79	5.31	5.31	5.31	
P-44	229	3.06	2.90	2.45	3.69	3.51	2.99	3.96	3.77	3.22	3.22	3.22	
P-44	244	3.51	3.15	3.33	4.22	3.80	4.01	4.52	4.07	4.30	4.30	4.30	
F-25	197.4	2.90	2.89	2.95	3.50	3.50	3.57	3.76	3.76	3.76	3.83	3.83	

TABLE 5.9 Summary of SN_{eff} Values for the Summer of 1992 Data

Route	Mile Post	SN _{eff}
P-12	70	2.29
P-23	416	2.65
P-30	108	1.71
P-34	15	1.99
P-34	163	2.13
P-44	244	1.48

TABLE 5.10 Summary of SN_{eff} Values for the Spring of 1993 Data

Route	Mile Post	SN _{eff}
P-12	48	6.31
P-23	416	2.44
P-30	108	1.53
P-34	15	2.05
P-44	229	4.84
P-44	244	1.48
F-25	197.4	6.42

TABLE 5.11 Summary of D_{ol} (in mm) for the Summer of 1992 Data

Route	Mile Post	Traffic Level											
		Low				Medium				High			
		AASHTO D_{ol}	LAB D_{ol}	FIELD D_{ol}	AASHTO D_{ol}	LAB D_{ol}	FIELD D_{ol}	AASHTO D_{ol}	LAB D_{ol}	FIELD D_{ol}	AASHTO D_{ol}	LAB D_{ol}	FIELD D_{ol}
P-12	70	87	42	50	131	78	88	149	94	104			
P-23	416	30	37	35	68	76	74	84	92	90			
P-30	108	172	136	129	222	181	173	244	201	192			
P-34	15	115	115	113	160	160	157	179	179	176			
P-34	163	90	167	134	132	221	182	150	243	203			
P-44	244	109	81	88	148	116	124	165	131	140			

TABLE 5.12 Summary of D_{ol} (in mm) for the Spring of 1993 Data

Route	Mile Post	Traffic Level											
		Low				Medium				High			
		AASHTO D_{ol}	LAB D_{ol}	FIELD D_{ol}	AASHTO D_{ol}	LAB D_{ol}	FIELD D_{ol}	AASHTO D_{ol}	LAB D_{ol}	FIELD D_{ol}	AASHTO D_{ol}	LAB D_{ol}	FIELD D_{ol}
P-12	48	-133	-144	-198	-88	-100	-162	-69	-82	-147			
P-23	416	48	46	46	87	84	84	103	101	100			
P-30	108	163	120	121	211	161	162	232	179	180			
P-34	15	121	146	122	167	195	169	186	216	188			
P-44	229	-103	-112	-138	-67	-77	-107	-51	-62	-94			
P-44	244	118	97	107	158	134	146	176	150	163			
F-25	197.4	-203	-203	-200	-168	-168	-165	-153	-154	-149			

respectively). Huynh-Feldt epsilon values were calculated in order to account for any model violations and to make adjustments to the denominator degrees of freedom. Values were near one, indicating that violations were minor: 0.8690, 0.8725, and 0.8733 for the low, medium, and high traffic levels, respectively.

Even though there were no differences among the methods, it might also be of interest to know, at a given difference in thickness, if one could detect that the methods were not the same. Therefore, the power of the F test was performed to determine the probability of accepting the alternative hypothesis (H_a) that the methods are different. Suppose, one is interested in determining if a maximum difference of 25.4-mm (1.0-in) could be detected. At the low traffic level, there was about 92 chances in 100 that differences would be detected among the 3 different methods. At 12.7-mm (0.5-in.) differences, this detection dropped to 34 chances in 100. Overall, 19.1-mm (0.75-in.) maximal differences could be detected with 80% probability. Detecting differences of 12.7-mm (0.5-in) would not be very easy with the given data set.

Besides the above test, the Tukey procedure for pairwise comparisons was also completed. The following 95% confidence intervals were obtained ($\mu_{.3}$ is the treatment mean for AASHTO, $\mu_{.2}$ is the treatment mean for LAB, and $\mu_{.1}$ is the treatment mean for FIELD) for the low traffic level:

$$-0.49 \leq \mu_{.3} - \mu_{.2} \leq 1.01$$

$$-0.13 \leq \mu_{.3} - \mu_{.1} \leq 1.37$$

$$-0.40 \leq \mu_{.2} - \mu_{.1} \leq 1.11$$

These intervals suggest that AASHTO M_R values give the lowest overlay thicknesses. There is also a slight indication that field M_R values give different results than the other two procedures. However, with

the current sample size, these differences are not statistically significant. Similar results were obtained at the medium and high levels of traffic as shown below by the 95% confidence intervals, respectively.

$$-0.56 \leq \mu_{.3} - \mu_{.2} \leq 1.17$$

$$-0.15 \leq \mu_{.3} - \mu_{.1} \leq 1.58$$

$$-0.45 \leq \mu_{.2} - \mu_{.1} \leq 1.27$$

$$-0.59 \leq \mu_{.3} - \mu_{.2} \leq 1.23$$

$$-0.16 \leq \mu_{.3} - \mu_{.1} \leq 1.66$$

$$-0.48 \leq \mu_{.2} - \mu_{.1} \leq 1.34$$

CHAPTER SUMMARY

In this chapter, an analysis was presented using the 1993 AASHTO guide for overlay pavements with three different sets of M_R values calculated throughout this research. Overlay thicknesses were calculated using the non-destructive testing (NDT) method for determining the SN_{eff} value of a pavement section. Three different statistical analyses were then conducted to evaluate the results: a repeated measures analysis, the power of the F test, and Tukey procedure for pairwise comparisons.

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

SUMMARY

In this research project, comprehensive field and laboratory evaluations were performed on subgrade soils at nine different sites representing typical primary roads in the State of Wyoming. Resilient modulus (M_R) values were obtained from: laboratory testing based on 41.4-kPa (6-psi) and actual field deviator stresses, back calculation based on three different computer programs, and the AASHTO equation based on deflection measurements. In addition, several laboratory tests were conducted to examine the fundamental soil properties of the subgrade soils included in this study. These soil properties included: water content (actual and optimum), AASHTO soil classification, group index, and plasticity index. Three different M_R values (AASHTO, LAB, and FIELD) obtained at each site were then used to determine the required overlay thicknesses. Finally, all of the resulting data were used in conducting comprehensive data analyses. The following conclusions can be drawn from this research:

1. Subgrade soil samples should be extracted from Shelby tubes shortly after obtaining them from the field.
2. M_R measurements made with the LVDT's on the ring located inside the testing chamber consistently gave higher values compared to the actuator LVDT's located on the loading piston.

3. The EVERCALC back calculation program appears to give somewhat better M_R values than do the MODULUS and BOUSDEF programs.
4. Some fundamental soil properties do influence the measured M_R value. Resilient modulus values for type A-4 and A-6 subgrade soils in this study decreased as water content increased.
5. Layers within Shelby tubes do not differ significantly from one another. Therefore, averaging the resilient modulus values from all layers will give more reliable results compared to the value from one layer.
6. The recommended correction factor (C) of 0.33 or less appears to be adequate for cohesive subgrade soils in the State of Wyoming.
7. M_R values based on actual deviator stresses did not statistically differ from values based on the assumed deviator stress of 41.4-kPa (6-psi). However, by computing actual deviator stresses, the resulting M_R values within each testing site were more consistent.
8. The three M_R values calculated based on AASHTO equation, laboratory with 41.4-kPa (6-psi) deviator stress, and laboratory with actual deviator stress did not result in significantly different overlay thicknesses. Among the three, however, the AASHTO M_R value gave the lowest overlay thicknesses.

RECOMMENDATIONS FOR NEEDED FUTURE RESEARCH

1. Because this study was limited to cohesive subgrade soils, it would be of interest to conduct a similar research project on granular subgrade soils.

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2. The effect of resilient modulus selection on the design of new pavement structures should also be evaluated.

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APPENDICES ON FILE