MICHIGAN STATE UNIVERSITY

Pavement Subgrade MR Design Values for Michigan's Seasonal Changes

Final Report

July 22, 2009

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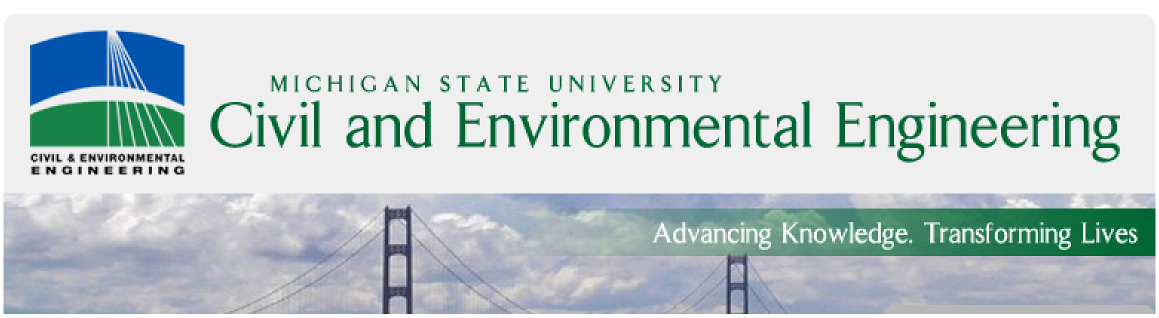

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The resilient modulus (MR) of roadbed soil plays an integral role in the design of pavement systems. Currently, the various regions of the Michigan Department of Transportation (MDOT) use different procedures to determine the MR values. Most of these procedures are applicable to the new Mechanistic-Empirical Pavement Design Guide (M-E PDG) level 3 design only and the 1993 AASHTO design guide. Therefore, a consistent, uniform, and implementable procedure that meets the requirements of the M-E PDG for level 1, 2, and 3 designs and the 1993 AASHTO Design Guide was developed.

In this study, the State of Michigan was divided into fifteen clusters where the physical and engineering characteristics of the soils were similar. The clusters were then divided into ninety nine areas to narrow down the ranges of the engineering and physical characteristics of the soils. Disturbed roadbed soil samples were collected from seventy five areas, and twelve undisturbed soil samples (Shelby tubes) were collected from areas with CL and SC roadbed soils. The soil samples were subjected to simple tests (moisture content, grain size distribution, and Atterberg limits when applicable), and cyclic load triaxial tests to determine the soils resilient modulus values. Predictive correlation equations were developed to estimate the MR values of the roadbed soil based on the results of the simple tests.

Deflection data from Falling Weight Deflectometer (FWD) tests conducted throughout the state were obtained from MDOT. The deflection database consisted of hundreds of previous FWD tests that were conducted on various projects over the last 20^+ years and eighty FWD tests conducted as part of this study. All FWD data files with sufficient data were analyzed to backcalculate the roadbed soil MR values. Comparison between the laboratory and the backcalculated MR values indicated that the two values are almost equal when the stress boundaries used in the laboratory tests matched those of the FWD tests.

The report includes the recommended MR design values are provided on state map and in table format..</td></tr>
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EXECUTIVE SUMMARY

The resilient modulus (MR) of roadbed soil and the modulus of subgrade reaction (k) play a major role in the design of asphalt and rigid pavement systems. Currently, the various Regions of the Michigan Department of Transportation (MDOT) use different empirical procedures to determine the MR and k values of the roadbed soils. These procedures vary from one Region to another and are applicable to the 1993 AASHTO Guide for Design of Pavement Structures and to only design level 3 of the new AASHTO Mechanistic-Empirical Pavement Design Guide (M-E PDG). Therefore, consistent, uniform, economical and implementable procedures for determining MR and k values of the roadbed soils that meet the requirements of the three design levels of the M-E PDG were developed in this study and presented in this report.

The research plan for this study, which was developed and modified jointly by the Michigan State University (MSU) research team and the members of the MDOT Research Advisory Panel (RAP), is presented in Appendix A. The problem statement and the objectives of the study, on the other hand, are presented in Chapter 1. All of the study objectives were satisfied.

The AASHTO M-E PDG specifies three design levels depending on the class of the road. The commonality between the three levels, all require MR and k values as inputs. The differences are in the required level of accuracy and the required procedures for determining the MR and k values. The existing MDOT practices for determining the MR and k values of the roadbed soils are presented in Chapter 2. The procedures for determining the MR and k values of the roadbed soils in each design level of the M-E PDG are also detailed in Chapter 2. On the other hand, the AASHTO 1993 pavement design procedures do not include design levels. The required inputs to these procedures include the effective MR or k value, which can be determined based on damage factors calculated for the various environmental seasons.

In this study, the State of Michigan was divided into fifteen clusters and seventy-five areas based on the similarity of the physical and engineering characteristics of the roadbed soils. Soil samples were then obtained from each area in the Upper and Lower Peninsulas of Michigan. The soil samples were tested in the laboratory using cyclic load triaxial tests and their MR and k values were determined. Nondestructive deflection tests were conducted using the MDOT Falling Weight Deflectometer (FWD). The deflection data that were measured during this study along with those measured by MDOT over the last twenty year period were used to backcalculate the MR and k values of the various roadbed soils. The results are presented and discussed in Chapters 3 and 4 of this report. Results of the laboratory and FWD tests are presented, discussed and compared in Chapter 5. It is shown that based on the test results, the roadbed soils in the State of Michigan were divided into eight soil types based on the Unified Soil Classification System (USCS); gravelly sand (SG), poorly graded sand (SP), which was divided into two groups SP1 and SP2 based on the percent fine contents, silty sand (SM), poorly graded sand – silty sand (SP-SM), clayey sand – silty sand (SC-SM), clayey sand (SC), low plasticity clay (CL) and low plasticity silt (ML)

To satisfy the requirements for design levels 2 and 3 of the M-E PDG, predictive correlation equations (see Table 6.1) were developed to estimate the MR and k values of the roadbed soil based on the results of simple and economical tests such as moisture content, degree of saturation, Atterberg limits, dry unit weight, specific gravity, and grain size distribution data. The predictive equations are presented and discussed in Chapter 6. Further, a procedure that satisfies the requirements of design level 1 of the M-E PDG was developed and is also presented and discussed in Chapter 6.

The soil type and the design MR and k values obtained in this study are incorporated into a state map for a quick reference and easy implementation. Based on the data obtained and analyzed during the course of the study, several conclusions and recommendations were made and are included in Chapter 6.

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

The state of Michigan is geographically located within the glaciated section of North America and most of its soil has developed from glacial deposits. The ice sheet advanced over the state in three lobes; one along Lake Michigan, one along Lake Huron, and the third along Lake Erie. A branch from the Lake Huron lobe advanced southwesterly and connected to the other two lobes. During the advance of ice a large amount of soil and bedrock along the path of each ice lobe were pulverized and incorporated into the ice sheet to be later redeposited. When the Wisconsin ice sheet retreated to the north, these materials (known as glacial drift) were superimposed on sedimentary rock of the Michigan Basin in the Lower Peninsula and the Eastern part of the Upper Peninsula and on igneous and metamorphic rocks in the Western part of the Upper Peninsula. The thickness and composition of the deposit varies from one location to another. For example, the thickness of the deposit in the Alpena area is only few inches whereas it is more than 1200 ft thick in the Cadillac area. The glacial drift varies from clay to gravel with some scattered pockets of organic (peat) materials that were formed later. The granular texture may be segregated or mixed heterogeneously with boulders and clays. Because of these complex arrangements, about one hundred sixty-five different soil types were formed and are being used for engineering purposes by the Michigan Department of Transportation (MDOT) (MDOT 1970). The engineering and physical characteristics of these soils vary significantly from those of gravel and sand in the western side of the Lower Peninsula, to clay in the eastern side and to varved clay in the western part of the Upper Peninsula.

For a given type of roadbed soil, its mechanical (engineering) properties (the resilient modulus (MR) and the plastic properties) are a function of the physical parameters (moisture content, grain size, grain angularity, Atterberg limits, etc.) of the soil and have a major impact on the performance of pavement structures. In the past, MDOT has funded several research projects to study the engineering properties of certain types of roadbed soils (Goitom 1981, Lentz 1979). The results of those studies were incorporated into this research.

In this study, the MR values of various roadbed soil types were determined in the laboratory using cyclic load triaxial tests and in the field using the Falling Weight Deflectometer (FWD) deflection data. Statistical predictive equations were obtained with the main goal being to determine whether or not the MR value of a given roadbed soil type could be estimated using the results of simple tests.

In the laboratory, the soil samples are typically subjected to cyclic load triaxial tests. The MR values of a given soil type are then calculated as the ratio of the applied deviatoric stress, σ_d, (the difference between the axial and lateral stresses) to the recoverable axial strain (ε_r) of the soil (Goitom 1981, Lentz 1979, Young and Baladi 1977, and Yau and Von Quintus 2002). Mathematically, the MR is expressed as follows:

$$MR = \frac{\sigma_d}{\varepsilon_r} \qquad \text{Equation 1.1}$$

1.2 PROBLEM STATEMENT

The roadbed soil in the state of Michigan consists mainly of glacial soils with distinct seasonal stiffness changes due to temperature (possible frozen condition) and moisture levels. The MDOT's current flexible and rigid pavement design process follows the procedures outlined in the 1993 American Association of State Highway and Transportation Officials (AASHTO) Guide for Design of Pavement Structures. One of the inputs of said procedures is the effective value of the resilient modulus of the roadbed soil, which is a function of seasonal changes. The pending new AASHTO Mechanistic-Empirical Pavement Design Guide (M-E PDG) procedure is even more stringent for defining MR in terms of seasonal effects. Currently, the various regions of MDOT provide the "adjusted" MR value used for pavement design. The MR value is derived from either backcalculated pavement deflection data or from correlation with known soil parameters such as the Soil Support Value (SSV). Chapter 2 of this report addresses the various practices used by the MDOT regions to estimate the resilient modulus of the roadbed soil for flexible pavement design and the modulus of subgrade reaction for rigid pavement design.

1.3 OBJECTIVES

The objectives of this study as stated in the proposal are to:

1. Evaluate the existing processes used by all regions of MDOT for determining the MR value of the roadbed soil for flexible pavement design and the modulus of subgrade reaction (k) for rigid pavement design.
2. Determine the needed modifications to make the process compatible with the M-E PDG.

In the process of satisfying the above objectives, several steps were taken during the study period. These include:

- Classify the various types of roadbed soils that exist throughout the State of Michigan.
- Conduct laboratory tests to determine the physical and mechanistic characteristics of the various roadbed soils.
- Determine the resilient modulus of the roadbed soils using cyclic load triaxial testing in the laboratory and nondestructive deflection test data.
- Establish procedures (equations) for obtaining the resilient modulus of the roadbed soils for levels 1, 2, and 3 design of the M-E PDG.
- Develop relationships between the backcalculated roadbed modulus values and the ones obtained from the laboratory tests.

To accomplish the objectives, a research plan was developed and implemented. For convenience, the plan is included in Appendix A of this report.

1.4 REPORT LAYOUT

This final report consists of six chapters and five appendices as follows:

Chapter 1 – Introduction
Chapter 2 – Review of MDOT Practices & M-E PDG
Chapter 3 – Laboratory Investigation and Data Analysis

Chapter 4 – FWD Investigation and Data Analysis
Chapter 5 – Comparison of Laboratory and FWD Data
Chapter 6 – Summary, Conclusions, and Recommendations
Appendix A – Research Plan
Appendix B – Literature Review
Appendix C – Soil Classification Systems
Appendix D – Laboratory Results
Appendix E – FWD Results

CHAPTER 2

REVIEW OF MDOT PRACTICES & M-E PDG

2.1 GENERAL

During the course of the study, the following three general topics regarding the resilient modulus of roadbed soils and the modulus of subgrade reaction were reviewed.

1. Review of the existing MDOT practices regarding the determination of the resilient modulus (MR) values of roadbed soils and the modulus of subgrade reaction (k) values.
2. The role of the MR and k values of the roadbed soils in the new AASHTO Mechanistic-Empirical Pavement Design Guide (M-EPDG).
3. Literature review regarding:

    - The existing, national state of the practice in determining the MR and k values of the roadbed soils using laboratory and field tests.
    - Past and on-going efforts to develop correlation equations between the MR and k values and other roadbed soil parameters.
    - Existing correlation equations relating laboratory obtained and backcalculated MR values using pavement deflection data.

The first two topics are presented below whereas the last topic (the literature review) is included in Appendix B of this report.

2.2 REVIEW OF MDOT PRACTICES

MDOT divides the State of Michigan into seven regions; Superior, North, Bay, Grand, University, Southwest, and Metro as shown in Figure 2.1. Table 2.1 summarizes the practices and/or procedures used by each region to estimate the MR and k values of the roadbed soils. As can be seen, the practices differ slightly from one region to another and need to be unified to satisfy the second objective of this study. In general, the soil engineer in each Region uses Figure 2.2 to estimate the MR values of the roadbed soils based on other known or estimated parameters of the soils such as soil support value, soil classification or the type of soil usage (subbase versus base materials). The chart in Figure 2.2 is based on the Soil Support Values (SSV) and the United States Department of Agriculture (USDA) soil classification system (for completion, review of various soil classification systems is included in Appendix C of this report). Figure 2.2 also provides correlations between the SSV, the American Association of State Highway and Transportation Officials (AASHTO) layer coefficient for base (a_2) and subbase (a_3) materials, and the resilient modulus.

2.3 ROLE OF ROADBED RESILIENT MODULUS IN THE M-E PDG

The required inputs to the M-EPDG can be broadly classified under four main categories - general, traffic, climatic, and structural inputs. The Design Guide uses a three level hierarchical design approach for the selection of traffic and structural inputs. This provides the pavement designers with the flexibility of using specific or general input data in the design process depending on the agency's resources and the requirements of each specific design project (Coree et. al 2005).

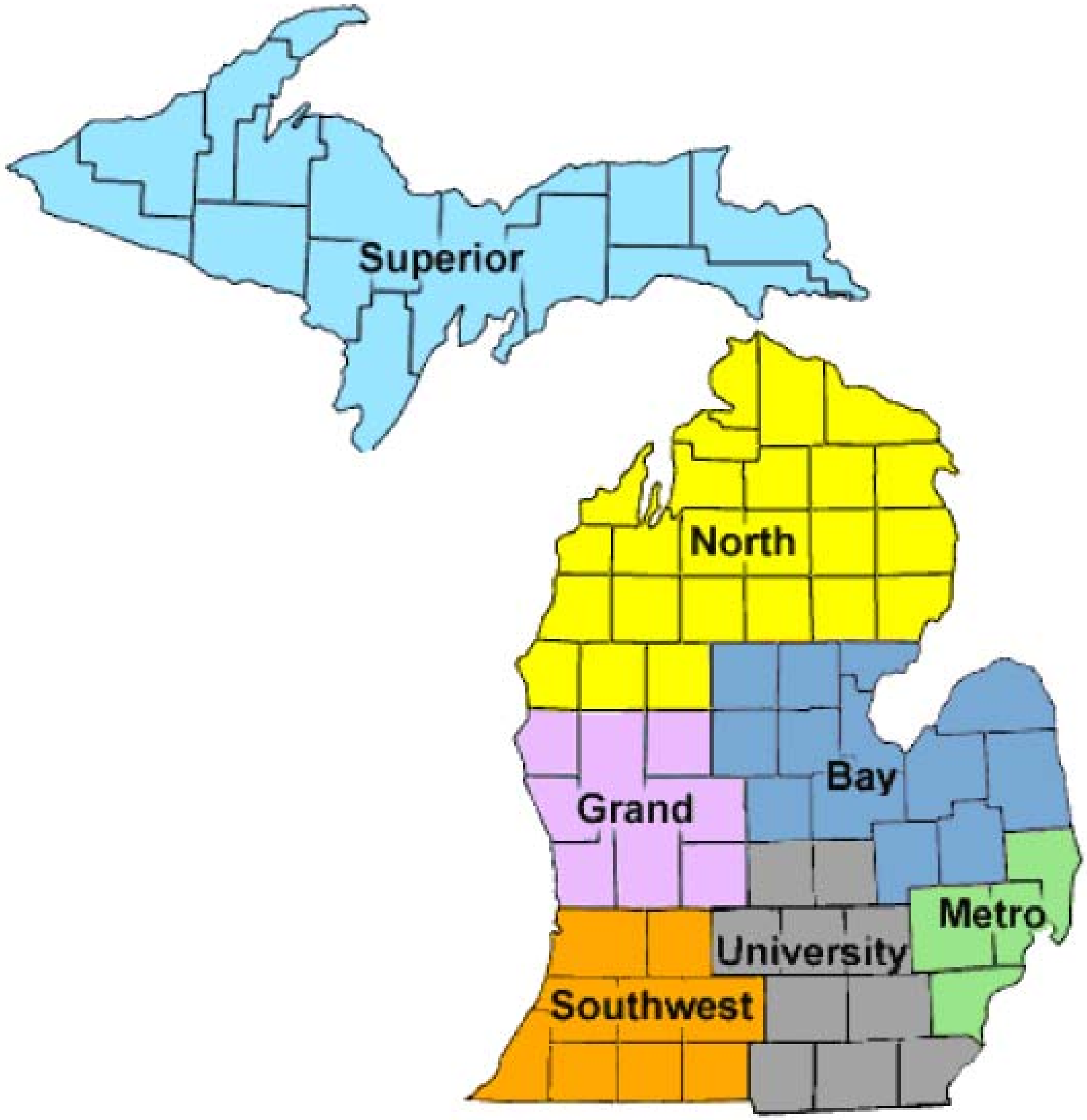


Figure 2.1 MDOT regions (MDOT)

Table 2.1 Regional MDOT practices and/or procedures for determining the resilient modulus (MR) of the roadbed soils

Region	Procedure	Typical MR values (psi)
Bay	Soil boring & visual identification	3600
Grand	FWD data (if available) or soil boring & visual identification	2700 - 8600
Metro	Soil boring & visual identification	3000 - 4500
North	FWD data (if available) or soil boring & visual identification	2500 - 6000
Southwest	California Bearing Ratio correlations	
Superior	Soil boring & visual identification	4500 - 7000
University	Soil boring & visual identification	3000 - 4000

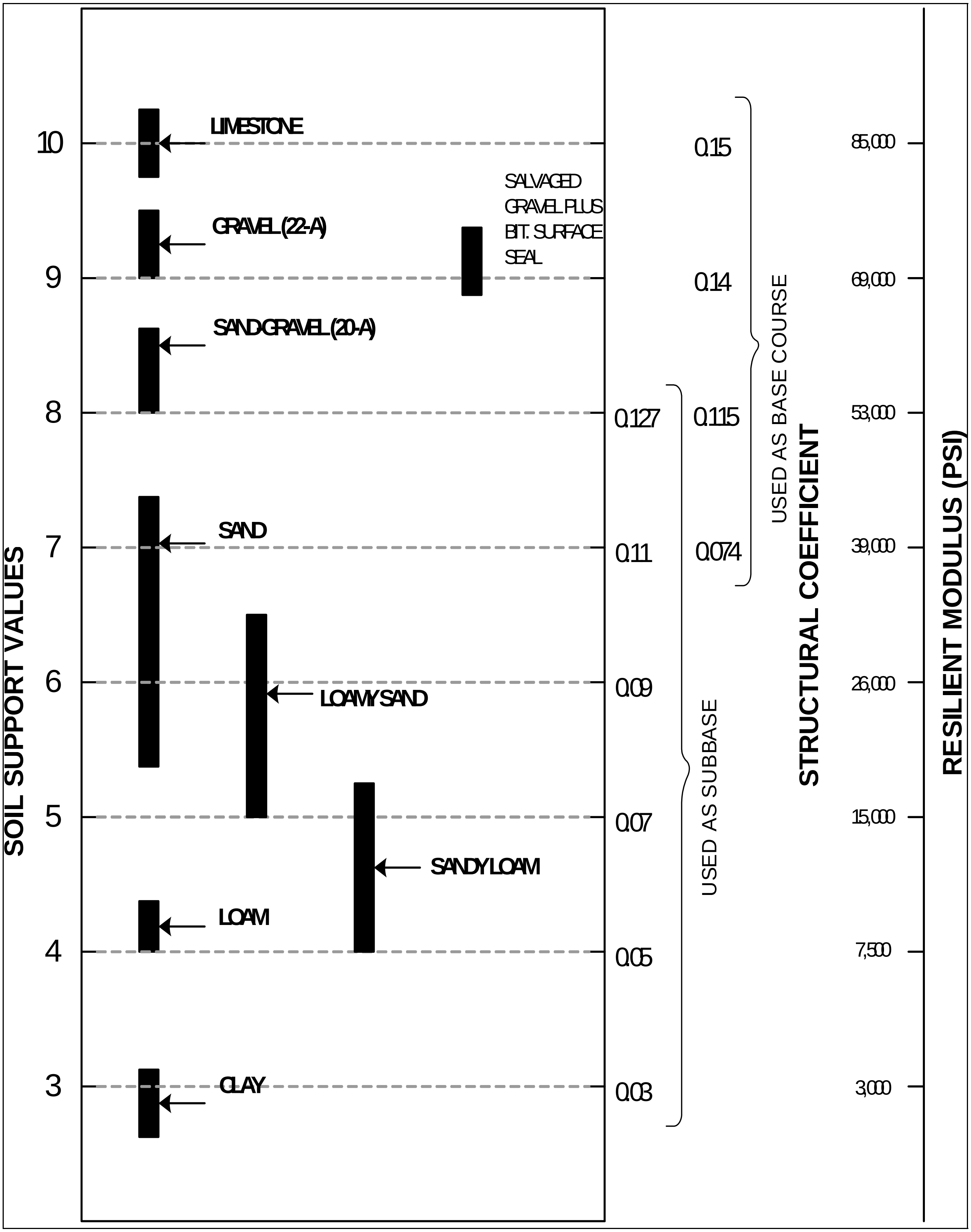


Figure 2.2 Range of soil support values, structural coefficients, and resilient modulus for various materials (MDOT)

The procedure for design level 1 can be thought of as "first class" and requires high accuracy inputs. The level 1 procedure will typically be used for obtaining inputs for the design of pavement sections subjected to heavy traffic or wherever there are significant safety and/or economic consequences of early failure. The procedure requires laboratory or field testing, such as dynamic modulus testing of hot-mixed AC or Falling Weight Deflectometer (FWD) deflection testing. Hence, the inputs for design level 1 require more resources and time than the other two levels.

Design level 2 is an intermediate design level whose required inputs are similar to those used for many years in the earlier editions of the AASHTO design guide. This level is used when resources and/or testing equipment are not available to obtain level 1 input. The required design data inputs for level 2 could be selected from an agency database, derived from a limited testing program, or estimated through correlations. Examples would be dynamic modulus value estimated from binder, aggregate and mix properties, or PCC elastic modulus estimated from unconfined compressive strength tests and so forth.

The required pavement design inputs for design level 3 have the lowest level of accuracy. This design level might be used for pavement sections where there are minimal consequences of early failure (low volume roads). The data inputs consist of typical default or average values used by the agency.

Further, the input data requirements can vary from one input parameter to another, which makes the procedure more flexible. For example, on a given project, the pavement designer could use level 1 for the roadbed soil resilient modulus input and level 3 for the traffic distribution data. Regardless of the selected input level, the 2002 design process is the same (Prozzi and Hong 2006).

Finally, regardless of the design level used, the resilient modulus of the roadbed soil is a required input to the pavement structural response model. It has a significant effect on the computed pavement response and on the dynamic modulus of subgrade reaction, k-value, which is computed internally by the Design Guide software.

The resilient modulus values of the roadbed soil can be measured directly from the laboratory or obtained through correlations with other material parameters such as California Bearing Ratio (CBR) and Dynamic Cone Penetrometer (DCP). The procedures for obtaining the resilient modulus for the various design levels are presented in the next three subsections (NCHRP 2004).

2.3.1 Procedures for Determining the Resilient Modulus of the Roadbed Soil for Design Level One

For design level 1, the resilient modulus values of the roadbed soil are determined using cyclic load triaxial tests in accordance with one of the following standard test methods:

- NCHRP 1-28A, “Harmonized Test Methods for Laboratory Determination of Resilient Modulus for Flexible Pavement Design.”
- AASHTO T307, “Determining the Resilient Modulus of Soil and Aggregate Materials.”

The stress conditions used in the laboratory testing must represent the range of stress states likely to be developed beneath the pavements. Stress states used for modulus testing are based upon the depth at which the material will be located within the pavement system (i.e., the stress states for specimens to be used as base, subbase, or roadbed soil may differ considerably).

The M-E PDG recommends Equation 2.1 for calculating MR values. The nonlinear elastic coefficients and exponents of the model are determined by using linear or nonlinear regression analyses to fit the model to laboratory generated MR test data (NCHRP 2004):

$$MR = k_1 P_a \left(\frac{\theta}{P_a} \right)^{k_2} \left(\frac{\tau_{oct}}{P_a} + 1 \right)^{k_3} \quad \text{Equation 2.1}$$

$$\tau_{oct} = \frac{1}{3} \sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_1 - \sigma_3)^2 + (\sigma_2 - \sigma_3)^2} \quad \text{Equation 2.2}$$

Where, MR = resilient modulus (psi), θ = bulk stress = $\sigma_1 + \sigma_2 + \sigma_3$ (psi), σ_1 = major principal stress (psi), σ_2 = intermediate principal stress (psi), σ_3 = minor principal stress/confining pressure (psi), P_a = atmospheric pressure (psi), and K_1, K_2, K_3 = regression constants

Please note that the above described procedure for design level 1 applies equally to reconstruction and rehabilitation where destructive material samples can be obtained. Alternatively, for pavement rehabilitation, nondestructive deflection tests can be performed and the MR values can be determined using a backcalculation routine.

2.3.2 Procedures for Determining the Resilient Modulus of the Roadbed Soil for Design Level Two

The MR values for pavement design level 2 can be estimated using existing direct or indirect correlation equations between MR values and other material parameters. An example of direct correlation is that between the CBR and the MR values. An example of indirect correlation is that between soil parameters (such as plasticity index, water content, or density) and CBR values and then CBR and MR values. Table 2.2 provides the list of direct and indirect correlation equations included in the M-E PDG.

In addition, the M-E PDG software allows the use of the following two options to estimate the design MR value of the roadbed soils:

- Input a representative value of MR and use the Enhanced Integrated Climatic Model (EICM) to adjust the MR value for the effect of seasonal climate (i.e., the effect of freezing, thawing, and so on).
- Input an MR value for each month (season) of the year (total of 12 months) and the software will calculate the effective design MR value.

The primary use of the EICM in this design procedure is to estimate the temperature and moisture profiles within the pavement system throughout its design life. The estimated temperature and moisture profiles within the roadbed soil layers can also be used to modify the representative MR value to account for the effects of climate. The procedure for pavement design level 2 is applicable to new, reconstruction, and rehabilitation design.

Table 2.2 Models relating material index and strength properties to MR values (NCHRP 2004)

Strength/index property	Model	Comments	Test standard
CBR	$MR = 2555\left(CBR\right)^{0.64}$	CBR = California Bearing Ratio	AASHTO T193
R-value	$MR = 1155 + 555\,R$	R = R-value	AASHTO T190
AASHTO layer coefficient	$MR = 30{,}000\left(\dfrac{ai}{0.14}\right)$	ai = AASHTO layer coefficient	AASHTO guide for the design of pavement structures
PI and gradation	$CBR = \dfrac{75}{1+0.728\left(wPI\right)}$	wPI = P200*PI P200 = percent passing sieve No. 200 PI = plasticity index	AASHTO T27, AASHTO T90
DCP	$CBR = \dfrac{292}{DCP^{1.12}}$	DCP = DCP index, mm/blow	ASTM D6951

2.3.3 Procedures for Determining the Resilient Modulus of the Roadbed soil for Design Level Three

For the M-E PDG design level 3, the MR value is determined based on the classification of the soil. Table 2.3 provides a list of MR values that are recommended in the M-E PDG. For this design level, typical, representative MR value at the optimum moisture content is required. Users have the option to use the EICM to modify the MR value for the effect of climate.

Design level 3 could be used for new, reconstruction, and rehabilitation projects. The material type can be obtained from historical boring record, material reports, or county soil maps.

It is important to note that the MR values presented in Table 2.3 are approximate and should be cautiously used. The reason is that these values are based on the assumption of a semi-infinite media. For a finite roadbed soil thickness (less than 5ft), the MR values of the lower and weaker materials should be used to obtain a composite MR value.

Table 2.3 Range and typical resilient modulus values for unbound granular and roadbed soil materials (NCHRP 2004)

Classification system	Material classification	pounds/square inch	
		MR Range	Typical MR
AASHTO	A-1-a	38,500 - 42,000	40,000
	A-1-b	35,500 - 40,000	38,000
	A-2-4	28,000 - 37,500	32,000
	A-2-5	24,000 - 33,000	28,000
	A-2-6	21,500 - 31,000	26,000
	A-2-7	21,500 - 28,000	24,000
	A-3	24,500 - 35,500	29,000
	A-4	21,500 - 29,000	24,000
	A-5	17,000 - 25,500	20,000
	A-6	13,500 - 24,000	17,000
	A-7-5	8,000 - 17,500	12,000
	A-7-6	5,000 - 13,500	8,000
USCS	CH	5,000 - 13,500	8,000
	MH	8,000 - 17,500	11,500
	CL	13,500 - 24,000	17,000
	ML	17,000 - 25,500	20,000
	SW	28,000 - 37,500	32,000
	SP	24,000 - 33,000	28,000
	SW - SC	21,500 - 31,000	25,500
	SW - SM	24,000 - 33,000	28,000
	SP - SC	21,500 - 31,000	25,500
	SP - SM	24,000 - 33,000	28,000
	SC	21,500 - 28,000	24,000
	SM	28,000 - 37,500	32,000
	GW	39,500 - 42,000	41,000
	GP	35,500 - 40,000	38,000
	GW - GC	28,000 - 40,000	34,500
	GW - GM	35,500 - 40,500	38,500
	GP - GC	28,000 - 39,000	34,000
	GP - GM	31,000 - 40,000	36,000
	GC	24,000 - 37,500	31,000
	GM	33,000 - 42,000	38,500

CHAPTER 3

LABORATORY INVESTIGATIONS AND DATA ANALYSIS

3.1 INTRODUCTION

At the outset, field and laboratory investigation plans were designed to accomplish the objectives of this study. The plans consisted of the following activities and tests:

- Soil delineation in the State of Michigan
- Soil sampling
- Field tests which consist of:
    - Penetration resistance using pocket size penetrometer
    - Shear strength using pocket vane shear tester
    - Deflection using Falling Weight Deflectometer (FWD), which is discussed in Chapter 4
- Laboratory tests which consist of:
    - Moisture content
    - Wet and dry sieving and grain analysis
    - Atterberg limits (liquid and plastic limits and plasticity index)
    - Hydrometer analysis
    - Cyclic load triaxial test
- Data Analysis

3.2 SOIL DELINEATION

As stated in Chapter 1, the State of Michigan is geographically located within the glaciated section of North America and most of its soil has developed from glacial drifts/deposits. The composition and thickness of the deposits vary from clay to gravel and from thin to thick depending on the location. Because of the complexity of the glacial drifts, about one hundred sixty-five different soil types were formed and are being used for engineering purposes by MDOT. To characterize the resilient modulus of the glacial drifts in an economical and practical manner, the State of Michigan was divided into 15 clusters where the soil in each cluster has similar (not the same) engineering and physical characteristics. The boundaries of the 15 clusters were established based on the 1982 Quaternary Geology map of Michigan (DEQ 1982), inputs from members of the Research Advisory Panel (RAP) of MDOT, and from the soil engineers in the various MDOT Regions. The boundaries of the 15 clusters are shown in Figure 3.1. After establishing the cluster boundaries, each cluster was preliminarily divided into areas based on the percentages of each soil type within the area (see Table D.1 in Appendix D) reported by the Natural Resources Conservation Service (NRCS) Web Soil Survey (Web Soil Survey 2007). Once again, the preliminary boundaries of each area were slightly modified based on inputs from the RAP members and from the soil engineers in the various MDOT Regions. The final divisions consisted of 15 clusters divided into 99 areas. Figure 3.2 depicts the boundaries of the 15 clusters (shown by the dashed lines) and the boundaries of the 99 areas (shown by the solid lines).

Figure 3.1 Cluster Boundaries of State of Michigan

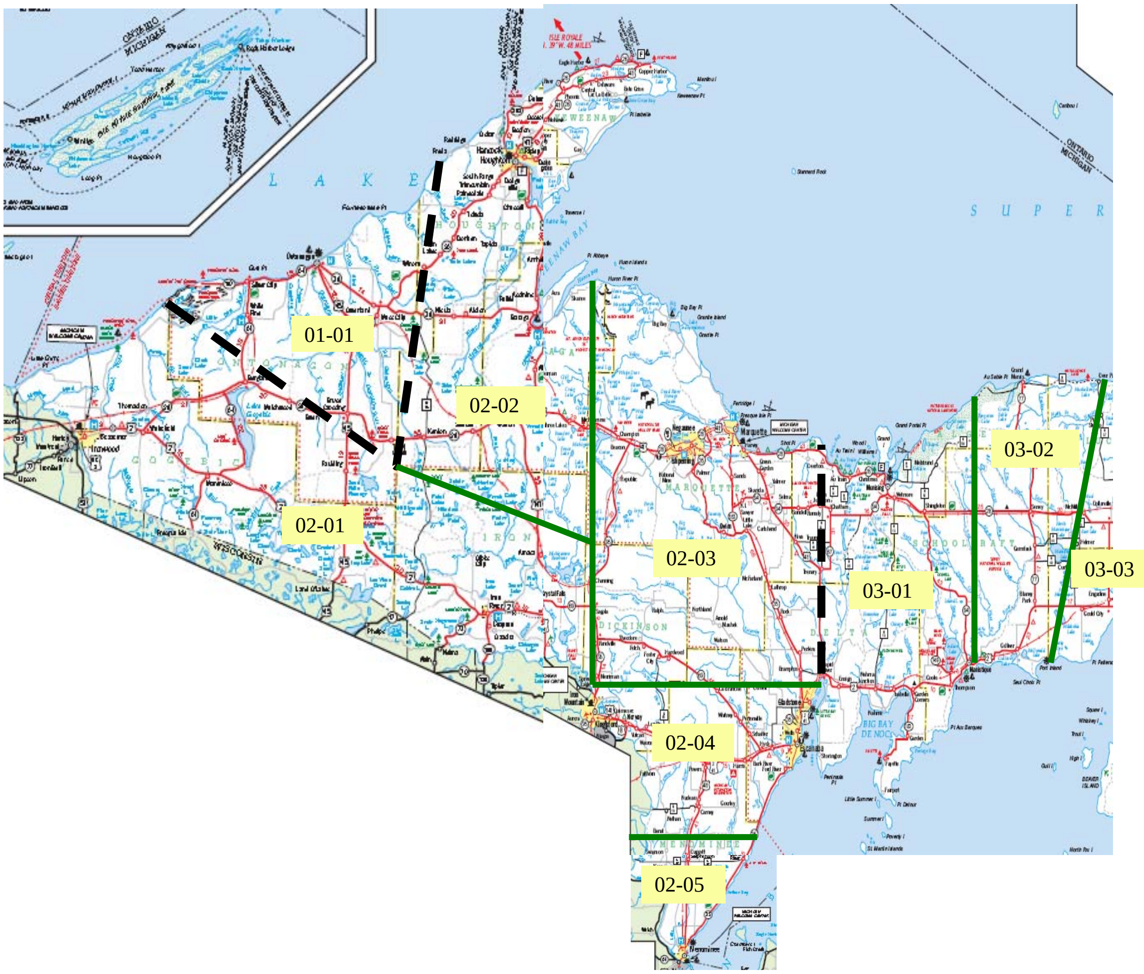
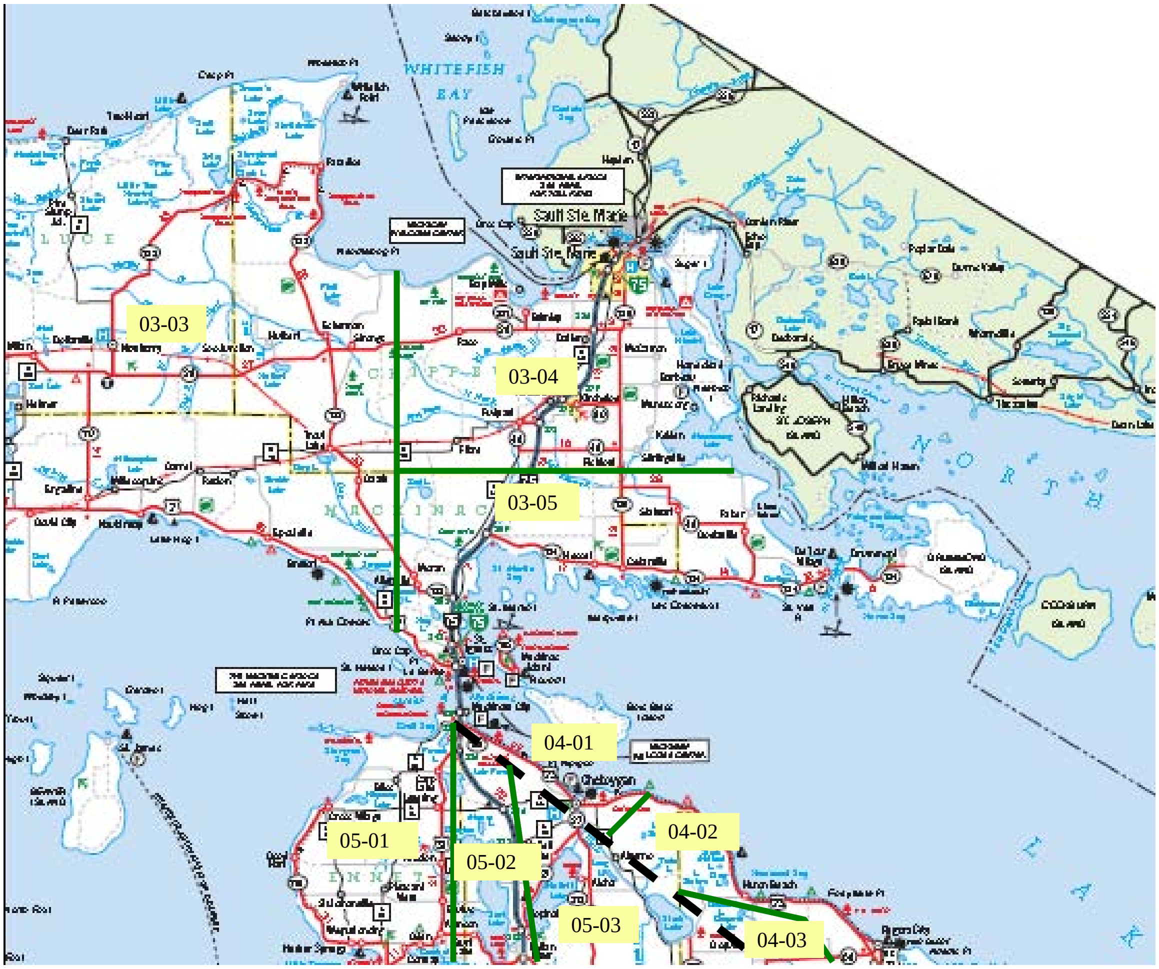


Figure 3.2 Cluster and area boundaries in the State of Michigan



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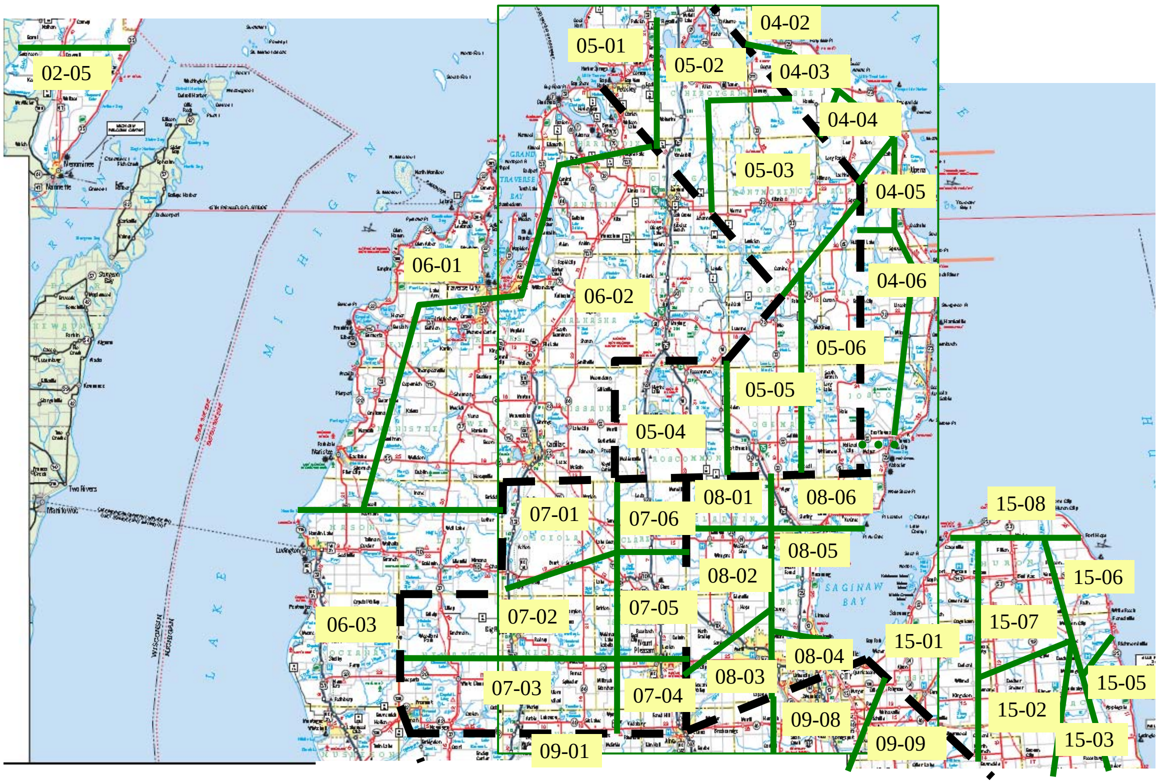

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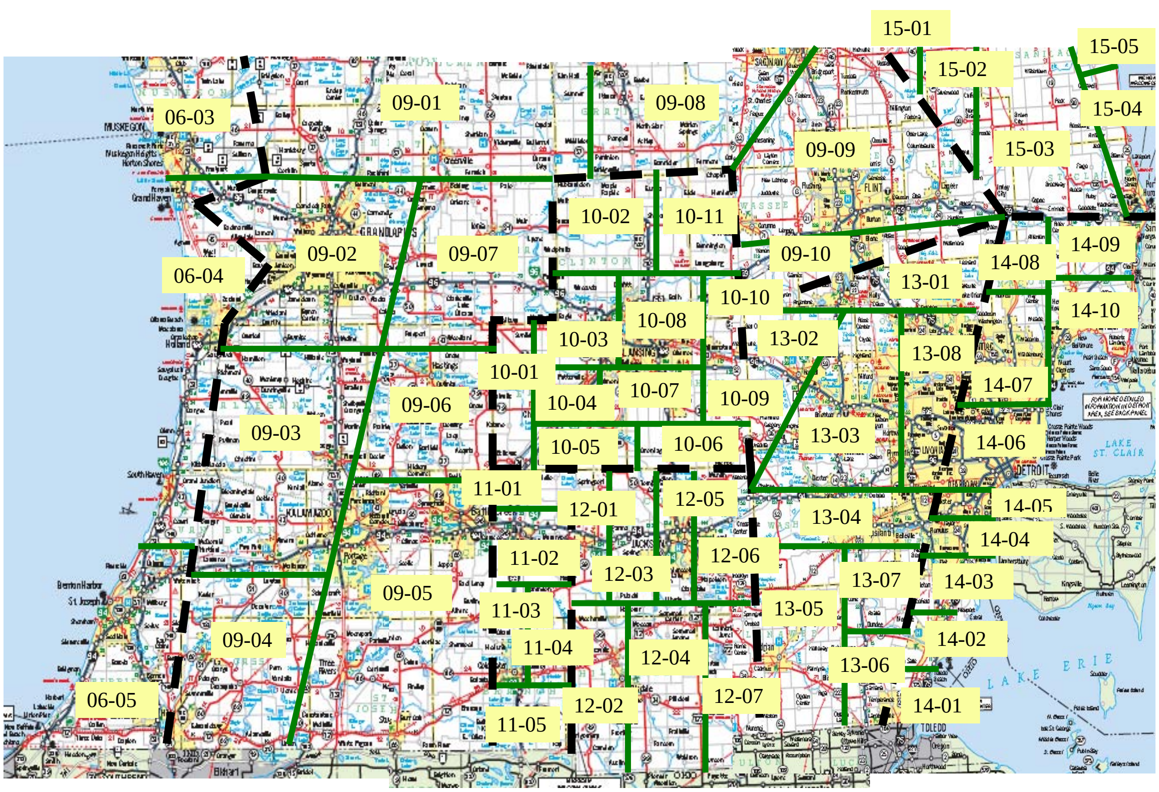


Figure 3.2 (cont'd)

The numbers (such as 02-03) in the lightly shaded areas in Figure 3.2 are the cluster number followed by the area number. It should be noted that for convenience, Figures 3.1 and 3.2 are also included in Appendix D as Figures D.1 and D.2 accompanying Table D.1. Once again it is important to note that the division of the state to 15 clusters is based on similar (not the same) soil types having a wide range of soil parameters. The boundaries between the areas, on the other hand, are based on narrower ranges of the soil parameters within each cluster.

3.3 ROADBED SOIL SAMPLING

After dividing the State of Michigan into 15 clusters and 99 areas, the percent of each soil type (sand, clay, silt, etc) in each area was obtained from the Natural Resources Conservation Service (NRCS) Web Soil Survey (Web Soil Survey 2007). Table 3.1 lists an example of the percentages of each soil type in the 15 clusters. The soil data for the remainder of the clusters and areas are listed in Table D.1 of Appendix D. Because of budget constraints and based on similar soil makeup, some areas within some clusters were grouped together which reduced the number of areas to be sampled from 99 to 75. The combined areas are collectively marked by the letter “X” in Tables 3.1 and D.1 of Appendix D. For each of the 75 areas, at least one disturbed roadbed soil sample was obtained. The soils were classified by the Unified Soil Classification System (USCS) and AASHTO soil classification systems, as discussed later in this chapter.

3.4 FIELD TESTING

The field testing consisted of measuring the soils penetration resistance using hand-held pocket penetrometer and the shear strength resistance using pocket size vane shear tester. The pocket penetrometer and pocket vane shear tests were conducted at the same time and at the same location where some of the disturbed roadbed soil samples were obtained.

3.4.1 Penetration Resistance Using Pocket Penetrometer

The pocket penetrometer is a small hand held device that consists of a spring loaded probe that slides into a cylinder. To perform the test the maximum pressure required to push the probe 0.25 inches into the soil is recorded. Pocket penetrometer is typically used to estimate the bearing capacity of the soil surface (Liu and Evett 2008). However, for this project the penetration resistance was recorded and correlations between MR values and penetration resistance were developed (when possible). A total of 67 pocket penetrometer tests were conducted. The test results and the test locations are listed in Table D.2 of Appendix D. An example of the data is provided in Table 3.2. Table D.2 also provides a list of the designation number and the location of each of the disturbed roadbed soil samples. The designation number consists of 9 characters A-BCD-E-(FG-HI) where A designates the road type (I=interstate, U=US road, and M=Michigan road), BCD represents the route number, E shows the traveling direction (N=North, E=East, S=South, and W=West), FG is the cluster number (01, 02, …15), and HI is the area number (01, 02, … 10). For example, the sample designation number M-059-W-(13-02) means that the sample was obtained from Westbound M-59 in cluster 13 and area 02.

Table 3.1 Soil type percentages for each area within 4 clusters

Cluster	Area	Muck (%)	Sand (%)	Loamy sand (%)	Silty loam (%)	Sandy loam (%)	Clayey loam (%)	Loam (%)	Mucky sand (%)	Clay (%)	Silty clay (%)	Proposed sampling
01	01	NO DATA										X
02	02	12.8	18.6	38	18.7	9						X
	03	24.3	30.6	12	3	13.4	9.1					X
	04	24.2	12	9.4	9.3	24.9	18.3					
	05	25	12.8	15.3	5.6	28.1	12.5					
	01	NO DATA										X
03	01	21	37	9.2	7.3	6.7	11.3					X
	02	20	29.1	8.6	12.2	5.3	14.8					
	04	9.2	8	16	63							X
	05	14.8	37	33.1	13							X
	03	29.2	29.6	9.4	11.5	16						
04	06	20	13.2	9.4		37.4			10			X
	02	25	15.2	34.4	16.2							
	01	58.4	33.3	4.1								X
	05	37.4	35	4.4	10	6.5						X
	03	16.1	50			28.4						X
	04	24.8				59.9		14.5				X

Table 3.2 Example of test results and location of pocket penetrometer and vane shear tests

Sample number	Location	Vane shear test (psi)	Pocket penetrometer (psi)
M-045-S (01-01)	405 feet South of Ontonagon River	did not fail	did not penetrate
U-002-E (02-01)	385 feet East of M-45	2	2.3
M-028-W (02-02)	~1000 feet West of M-141	4	3.5
M-028-W (02-03)	~2000 feet East of M-35	0.25	1.4
U-002-E (02-04)	765 feet East of Spalding Rd	did not fail	did not penetrate
U-002-E (03-01)	400 feet East of Hwy 13	0.26	0.4
M-028-W (03-02)	1500 feet North of M-77	0.5	0.7
M-028-W (03-03)	500 feet West of Basnau Rd	0.25	3
U-002-E (03-03)	200 feet East of M-117	1	1.3
I-075-N (03-04)	mile marker 380	0.25	0.4
I-075-N (03-05)	mile marker 368	did not fail	did not penetrate
U-023-S (04-01)	320 feet North of F 05 Co Rd	0.5	0.9
M-068-W (04-02)	180 feet West of US-23	0.5	2
M-068-W (04-03)	150 feet West of Little Ocqueoc River	0.5	0.9
M-065-S (04-04)	160 feet South of Elm Hwy	did not fail	did not penetrate
M-032-W (04-05)	220 feet East of Herron Rd	3	5.1
U-131-N (05-01)	200 feet South of Michigan Fisheries Visitor Center	0.25	1
U-127-N (05-04)	120 feet North of Co Rd 300	0.25	0.4
M-033-S (05-05)	750 feet South of Peters Rd	did not fail	did not penetrate
M-072-W (05-06)	330 feet West of M-32	6	3.7
M-132-N (06-01)	1000 feet North of Addis Rd (paved rd)	0.75	1.4

3.4.2 Pocket Vane Shear Test

The field vane shear test is used to estimate the undrained shear strength of the soils. To perform the test the full depth of the vane is inserted into the soil. The vane is then rotated by applying torque at the top of the rod until the soil fails (Das 2004). The maximum torque required to fail the sample was used in an attempt to develop correlations between the vane shear and the MR values. The test results and the locations of the 67 tests are listed in Table D.2 of Appendix D. An example is provided in Table 3.2.

3.5 LABORATORY TESTING

Once the disturbed and undisturbed Shelby tube samples were received in the laboratory, they were subjected to a battery of tests that include: natural moisture content, grain size tests that included dry and wet sieving and hydrometer testing, Atterberg limits (liquid and plastic limits and plasticity index), and cyclic load triaxial tests. These tests and the test results are presented and discussed in the next few subsections.

3.5.1 Moisture Content

All 81 collected soil samples underwent natural moisture content tests according to ASTM C 29 standard test procedure. The results of the moisture content tests can be found in Table D.3 of Appendix D, and an example of the test data are listed in Table 3.3. Samples with an "X" under the Shelby tube column were taken from undisturbed Shelby tubes and those with empty cells are from disturbed samples. The effects of moisture content on the MR values in this study are discussed later in this chapter.

3.5.2 Grain Size Distribution

The grain size distribution for soils with more than 10 percent passing sieve number 200 was determined using sieve and hydrometer tests and data analyses. The reason is that at fine contents of more than 10 percent, the size of the clay and silt particles or plates affect the mechanical behavior of the soils. For soils with less than 10 percent passing sieve number 200 the grain size distribution was determined by sieve analysis only.

3.5.2.1 Sieving Tests

All bag samples were subjected to either wet (see Figure 3.3) or dry sieve (see Figure 3.4) tests according to ASTM C 117 and ASTM C136 standard test procedures, respectively. First all soils were subjected to dry sieving. When the test results showed more than 10 percent passing sieve number 200, the soil was subjected to wet sieve and hydrometer analyses. The purpose of the tests was to determine the particle size distribution and the classification of the roadbed. In all analyses, the sieves were arranged as follows: 3/8 inch, #4, #10, #20, #40, #100, and #200. A total of 81 dry sieve and 56 wet sieve tests were conducted. Results of the dry and wet sieve analyses can be found in Table D.3 of Appendix D, and an example is provided in Table 3.3.

3.5.2.2 Hydrometer Tests

Soil samples with more than 10 percent passing sieve #200 were subjected to hydrometer tests according to the AASHTO T 88 standard test procedure. A total of 56 hydrometer tests were conducted.

3.5.3 Atterberg Limits Tests

Soil samples with more than eight percent passing the #200 sieve were subjected to Atterberg limit tests. This consisted of liquid and plastic limit testing, and calculation of plasticity index. The liquid limit of a soil is the water content at which soils change behavior from plastic to liquid. Whereas the plastic limit is the water content at which soils possess plastic behavior (Liu and Evett 2008). Both the liquid and the plastic limit tests were conducted according to the AASHTO T 89 standard test procedure. Figure 3.5 shows the devices for both tests. After obtaining the liquid and plastic limits, the plasticity index was calculated as the difference between the two limits. A total of 60 Atterberg limit tests were performed. Results of the Atterberg limit tests are listed in Table D.3 of Appendix D, and an example is provided in Table 3.3.

Table 3.3 Example of laboratory test results

Sample number	Shelby tube	Natural water content (%)	Sample weight (g)	Percent passing sieve #							Atterberg limits			D_{10}	D_{30}	D_{60}	$C_U = D_{60}/D_{10}$	$C_C = D_{30}^2/(D_{60})(D_{10})$	Classification	
				3/8 inch 9.500	4 4.750	10 2.000	20 0.850	40 0.425	100 0.150	200 0.075	LL	PL	PI						AASHTO	USCS
M-045-S (01-01)		11.5	298.8	99.5	99.3	98.9	96.8	96.7	77.2	66.7	26	16	10	0.0030	0.006	0.040	13.33	0.30	A-6	CL
U-002-E (02-01)		16.8	303.3	99.1	97.8	96.6	92.3	68.1	46.4	39.2	18	-	NP	0.008	0.040	0.300	37.50	0.67	A-4	SM
M-028-W (02-02)		21.0	200.0	100.0	99.4	98.0	93.4	83.2	64.5	56.1	23	-	NP	0.0080	0.024	0.110	13.75	0.65	A-4	ML
M-028-W (02-03)		6.6	535.8	100.0	99.3	97.2	92.1	81.8	23.4	6.1	16	-	NP	0.091	0.175	0.285	3.13	1.18	A-1-b	SP-SM
U-002-E (02-04)		10.8	200.0	100.0	99.4	98.0	93.4	83.2	64.5	54.1	19	-	NP	0.0100	0.050	0.110	11.00	2.27	A-4	ML
U-002-E (03-01)		5.0	525.3	100.0	99.8	99.6	98.5	92.6	15.8	6.5	13	-	NP	0.130	0.190	0.275	2.12	1.01	A-3	SP-SM
M-028-W (03-02)		3.1	519.1	99.9	99.6	99.3	97.9	89.7	14.0	3.0	NA	NA	NP	0.150	0.190	0.280	1.87	0.86	A-3	SP
U-002-E (03-03)		13.1	222.9	100.0	96.8	93.7	88.7	77.8	31.7	25.1	15	-	NP	0.002	0.120	0.300	150.00	24.00	A-2-4	SM
M-028-W (03-03)		4.8	520.2	94.1	87.5	82.6	71.2	45.5	11.1	6.4	21	-	NP	0.140	0.285	0.600	4.29	0.97	A-3	SP-SM
I-075-N (03-04)		9.4	549.2	99.9	99.8	99.5	98.4	91.3	10.0	1.5	NA	NA	NP	0.160	0.200	0.280	1.75	0.89	A-3	SP
I-075-N (03-05)		21.2	197.8	100.0	99.9	94.1	92.4	80.9	60.3	48.2	55	22	33	0.001	0.002	0.150	150.00	0.03	A-7-6	SC
U-023-S (04-01)		22.0	547.2	98.8	98.8	98.5	96.4	90.3	10.3	4.3	NA	NA	NP	0.170	0.200	0.280	1.65	0.84	A-3	SP
M-068-W (04-02)		4.0	205.0	99.9	98.6	91.0	51.3	25.2	16.0	14.1	18	12	6	0.040	0.500	1.000	25.00	6.25	A-2-4	SC-SM
M-068-W (04-03)		33.3	515.6	100.0	100.0	99.7	98.7	89.8	14.3	3.7	NA	NA	NP	0.160	0.190	0.280	1.75	0.81	A-3	SP
M-065-S (04-04)		8.1	201.5	99.3	95.4	91.3	87.5	72.7	30.4	21.5	30	-	NP	0.001	0.150	0.300	300.00	75.00	A-2-4	SM
M-032-W (04-05)		9.6	203.4	100.0	99.8	99.6	99.0	95.0	64.6	48.7	19	12	7	0.001	0.006	0.130	130.00	0.28	A-4	SC-SM
U-131-N (05-01)		13.1	199.4	99.8	99.2	96.4	95.0	78.7	43.5	29.2	14	-	NP	0.016	0.140	0.280	17.50	4.38	A-2-4	SM
U-127-N (05-04)		8.9	527.6	91.8	84.4	79.1	73.3	53.6	6.4	3.7	NA	NA	NP	0.180	0.260	0.500	2.78	0.75	A-3	SP
M-033-S (05-05)		3.5	525.7	63.1	57.5	45.4	35.7	26.7	7.8	4.6	NA	NA	NP	0.185	0.510	6.000	32.43	0.23	A-1-a	SG
M-072-W (05-06)		14.3	201.0	100.0	99.6	98.8	97.3	91.4	56.1	39.9	22	11	11	0.0070	0.035	0.160	22.86	1.09	A-6	SC
M-132-N (06-01)		15.0	521.7	99.5	99.0	98.5	96.8	78.7	8.8	4.2	NA	NA	NP	0.160	0.220	0.320	2.00	0.95	A-3	SP
I-075-N (06-02)		3.4	518.0	95.1	93.7	92.8	90.4	63.4	5.8	4.1	NA	NA	NP	0.170	0.260	0.400	2.35	0.99	A-3	SP
U-031-N (06-03)		5.8	1060.3	99.5	99.1	98.4	97.4	87.2	7.9	0.5	NA	NA	NP	0.170	0.210	0.300	1.76	0.86	A-3	SP
I-196-N (06-05)		10.5	1085.6	99.6	98.4	96.2	91.2	84.4	26.5	5.9	15	-	NP	0.089	0.160	0.275	3.09	1.05	A-2-4	SP-SM
M-020-W (07-02)		4.2	1003.7	99.6	99.3	98.7	97.9	88.0	2.1	0.8	NA	NA	NP	0.180	0.220	0.300	1.67	0.90	A-3	SP
M-020-E (07-03)		4.5	513.3	99.2	97.9	96.8	94.5	89.6	21.2	3.3	NA	NA	NP	0.110	0.190	0.280	2.55	1.17	A-3	SP
U-127-N (07-04)		10.9	200.8	100.0	98.8	96.6	95.4	90.3	38.3	26.9	22	12	10	0.001	0.100	0.230	230.00	43.48	A-2-6	SC
U-127-N (07-05)	X	11.2	203.9	100.0	98.3	92.6	87.3	79.9	53.7	40.5	23	14	9	0.0011	0.006	0.190	172.73	0.17	A-6	SC
U-127-N (07-05)		14.4	213.7	99.8	98.2	85.2	81.0	74.8	52.1	43.7	24	14	10	0.0010	0.008	0.210	210.00	0.30	A-6	SC
M-061-E (07-06)		22.1	198.5	100.0	98.8	93.3	84.7	59.3	23.7	17.9	19	-	NP	0.040	0.190	0.430	10.75	2.10	A-2-4	SM
M-061-E (08-02)		20.3	223.1	100.0	99.7	93.9	77.8	51.9	26.1	23.2	11	-	NP	0.050	1.000	0.520	10.40	38.46	A-2-4	SM
U-010-W (08-03)		21.4	200.2	100.0	100.0	99.8	99.7	97.6	61.0	55.2	32	14	18	0.001	0.002	0.140	140.00	0.02	A-6	CL
U-010-W (08-04)		8.2	200.1	99.9	99.9	98.8	96.6	84.5	48.8	36.7	29	13	16	0.001	0.011	0.200	200.00	0.61	A-6	SC

Figure 3.3 Wet sieve testing apparatus

Figure 3.4 Dry sieve testing apparatus

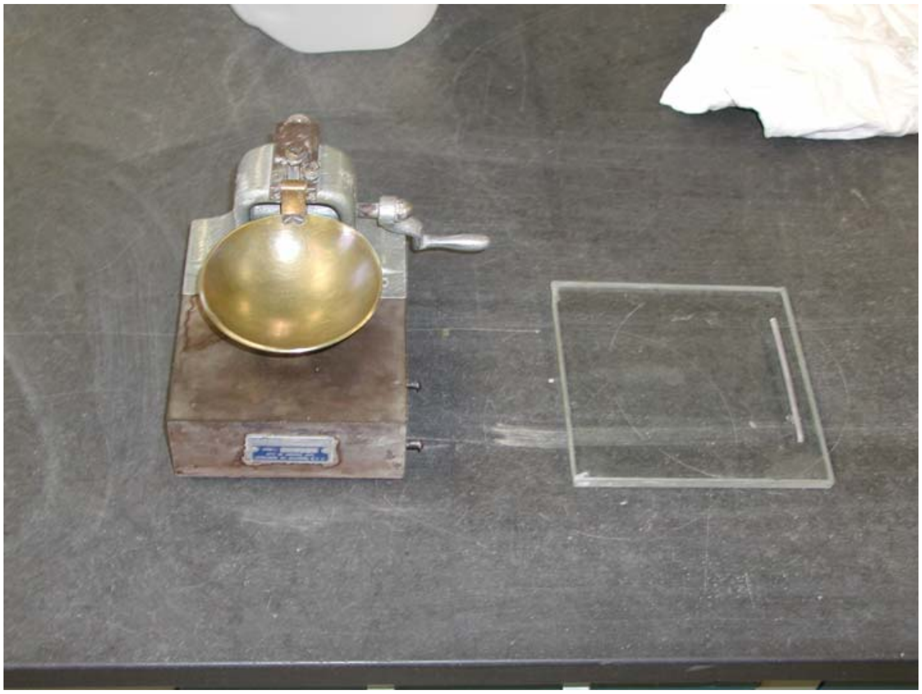


Figure 3.5 Liquid and plastic limit testing apparatus

3.5.4 Cyclic Load Triaxial Test

Cyclic load triaxial tests were conducted to determine the resilient modulus of the disturbed and undisturbed soil samples that were collected from 75 areas within the 15 clusters. The sample preparation procedure for the disturbed and undisturbed samples and the cyclic load test procedures and parameters are addressed below.

Laboratory Preparation of Sand Samples - All sand samples were compacted in a 2.125 inch diameter, 4.8 inch high split mold using 10 pound static load and vibrating table. The split mold has two outlets connected on the outside of the mold by small diameter drainage tubes and is protected on the inside by two small porous stones tightly fit to the holes. The sand sample preparation procedure consisted of the following steps:

1. Measure the total weight of the sand sample.
2. Assemble the split mold.
3. Stretch a rubber membrane along the interior walls of the mold and flip the membrane over the upper and lower rims of the split mold.
4. Apply vacuum to the space between the rubber membrane and the interior walls of the mold. The vacuum would force the membrane to stick to the wall. Care should be taken to eliminate wrinkling of the rubber membrane.
5. Place the split mold and the rubber membrane on the base pedestal of the triaxial cell and place a paper filter on top of the pedestal.
6. Place the entire assembly on top of a vibrating table (an ELE International 60 Hz. vibrating table model CT 164 was used in this study).
7. Placed sand in the split mold to a height of about 22 percent of the mold height using a spoon (the sand sample will be placed in the mold in five lifts).

8. Compact the sand in five lifts by placing a ten pound static load on top of each lift.
9. Turn the vibrating table on and vibrate the entire assembly for a period of three minutes.
10. Stop the vibrating table and remove the static load. Care should be taken not to disturb the compacted sand.
11. Place the second sand lift and repeat steps 7 through 11 until all five lifts are compacted.
12. Place a paper filter inside the split mold on top of the fifth sand lift after compaction.
13. Place the top pedestal of the triaxial cell on top of the paper filter.
14. Secure the rubber membrane around the top and bottom pedestals using rubber bands.
15. Remove the vacuum lines from the split mold and connect them to the drainage lines at the bottom of the triaxial cell to apply vacuum to the compacted sand sample.
16. Remove the split mold and finish assembling the triaxial cell.
17. Apply a confining pressure (air confining pressure of 7.5 psi was used in this study).
18. Disconnect the vacuum lines from the triaxial cell.
19. Weigh the left over sand and calculate the weight of the sand sample as the difference between the initial weight of the sand (step 1) and the weight of the left over sand.
20. Place the triaxial cell on the load cell of the MTS system for cyclic load testing.

Figure 3.6 shows the vibrating table, the static load, the vacuum pump, and the split mold seated on top of the base pedestal of a triaxial test apparatus.

Figure 3.6 Vibrating table setup

Preparation of Clay Samples - Disturbed clay samples were compacted according to the AASHTO standard proctor test procedure T99. After compaction, the samples were then trimmed to the desired length of 4.5 inches and diameter of 2.25 inches. The samples were then placed in a rubber membrane (which was stretched tightly around the interior of a split mold). The sample was then sealed from the atmosphere by securing the rubber membrane to the bottom and top pedestals. The entire mold assembly was then transferred to the triaxial cell where the cell was assembled and a confining pressure of 7.5 psi was applied to the sample.

Preparation of the Undisturbed Shelby Tube Samples – First, the Shelby tubes were cut to several 6 inch long segments. The clay soil was then extracted from the tube segment and was trimmed to a height of 5.6 inches. The diameter of the soil was kept the same as the interior of the Shelby tube (2.8 inches). A rubber membrane was placed around the soil (see clay samples above) and the sample was placed in the triaxial cell and subjected to 7.5 psi confining pressure.

After placing the (sand, clay, or Shelby tube) sample in the triaxial cell and subjecting it to 7.5 psi confining pressure, the cyclic load test commenced. The details of the cyclic load triaxial test are presented below.

Cyclic Load Test Procedure – All cyclic load triaxial tests were mainly conducted according to the AASHTO T 307 standard test procedure. Because of the type of tests and equipment available, the following three modifications to the AASHTO standard test procedure were made:

- The load cell of the available Material Testing System (MTS) is located below the triaxial cell assembly, whereas the AASHTO T 307 standard test procedure call for a load cell to be placed on top of the same or triaxial cell.
- The AASHTO T 307 standard test procedure specifies loading and unloading time of 0.1 seconds and relaxation time of 0.9 seconds. In this study, a loading and unloading time of 0.5 seconds and relaxation time of 0.9 seconds were used to more accurately simulate the duration of the load pulse experienced by a roadbed soil located about 30 inches below the pavement surface (see Figure 3.7).
- The sample was conditioned under 498 load cycles instead of the laborious conditioning sequences outlined in the AASHTO T 307 standard test procedure.

The cyclic load triaxial test parameters used throughout this study are:

- A sustained load of 10 pounds was applied to maintain contact between the MTS actuator and the piston of the triaxial cell.
- All samples were subjected to a confining pressure of 7.5 psi.
- A load unload frequency of 2 Hz and a total test frequency (load unload and relaxation period) of 0.71 hertz.
- Cyclic axial stress of 10 psi followed immediately by cyclic axial stress of 15 psi.
- For each axial stress level, the samples were conditioned to 498 load cycles.
- The axial sample deformations were measured using two linear variable differential transducers (LVDTs) located at 180 degrees across the diameter of the test sample.
- The resilient modulus of each test sample was calculated as the average of the resilient modulus values obtained at load cycles 499, 500, 501, 799, 800, 801, 999, 1000 and 1001. At each of the above load cycle, the resilient modulus was calculated using the measured cyclic stress and the average sample deformation measured by the LVDTs.

It is important to note that cyclic load triaxial tests are difficult to conduct and require extreme care and patience. The resulting MR values obtained from the test are typically affected by several test and sample variables including: confining pressure, deviatoric stress, loading frequency, soil type, moisture content, and specimen conditioning.

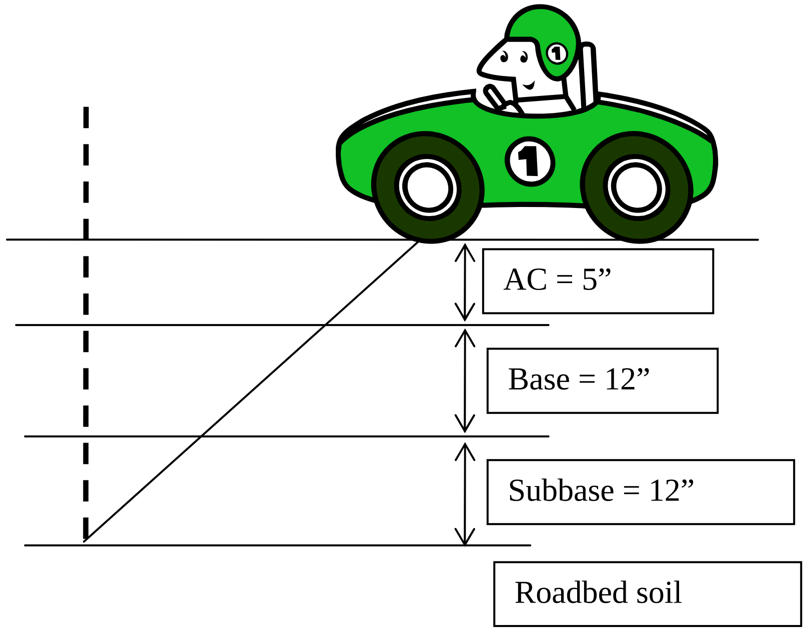


Figure 3.7 Load duration increases from A to B as a function of depth

The test setup used for the cyclic load tests is shown in Figure 3.8. Figure 3.9 shows two example hysteresis loops from load cycles 800 and 1000. The figure shows that as the load increases the sample deformation increases and vise versa. The shift in the two loops represents the cumulative plastic deformation that took place in the sample between load cycles 800 and 1000.

Figure 3.8 Cyclic load test setup

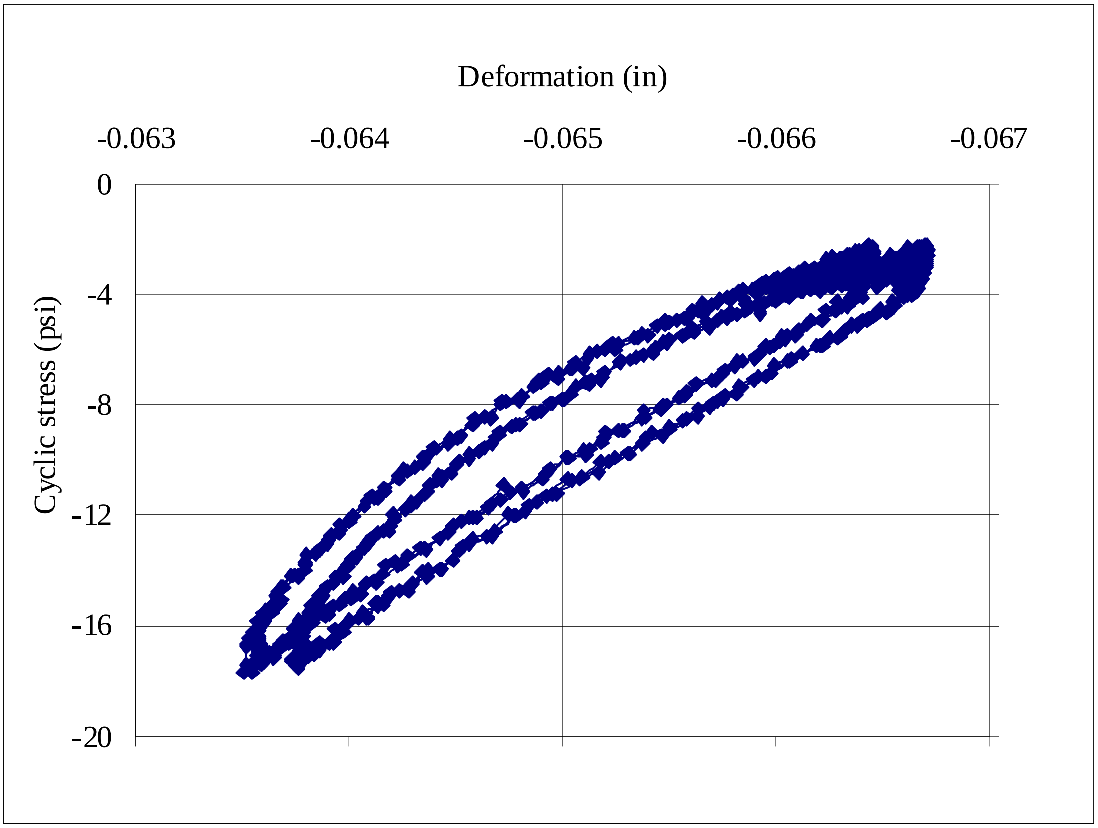


Figure 3.9 Typical cyclic load test results

For all soil samples, the resilient modulus values obtained from the 10 and 15 psi cyclic axial stresses are listed in Table D.4 of Appendix D. An example listing of the data is provided in Table 3.4. The highlighted rows of data in the tables indicate the results of those tests that were conducted to verify the developed resilient modulus prediction models. The verification is discussed later in the chapter. Table 3.4 and D.4 also include a listing of the moisture content, the degree of saturation, the dry unit weight of the test sample, the USCS and the AASHTO soil classifications, and the sample type (disturbed or undisturbed). It should be noted that the data in the tables are sorted based on the USCS. During the study, eighty-seven test samples were subjected to cyclic load triaxial tests. The number of test samples of each USCS soil classification is listed below.

- Twenty-six poorly graded sand (SP)test samples
- Seventeen silty sand (SM)test samples
- Eight poorly graded silty sand (SP-SM) test samples
- Nine clay (CL) test sample
- Sixteen clayey sand (SC) test samples
- Four low plasticity silt (ML) test samples.
- Seven clayey sand – silty sand (SC-SM) test samples

Table 3.4 Example of laboratory and MR test results

Sample number	Sample type		Classification		Dry unit weight (lb/ft^3)	Water content for cyclic test	Saturation	MR at cyclic stress (psi)	
	Shelby tube	Disturbed	AASHTO	USCS				10.0	15.0
M-028-W (02-03)		X	A-1-b	SP-SM	113.4	8.5	47.3	19,195	17,845
U-002-E (03-01)		X	A-3	SP-SM	108.7	4.5	22.1	22,787	19,592
M-028-W (03-03)		X	A-3	SP-SM	105.5	2.0	9.0	16,895	15,941
I-196-N (06-05)		X	A-2-4	SP-SM	111.5	3.7	19.5	23,009	21,964
I-069-N (10-01)		X	A-3	SP-SM	116.1	9.9	59.2	15,858	15,682
I-069-N (11-01)		X	A-3	SP-SM	118.0	7.0	44.2	30,701	28,120
I-094-W (12-03)		X	A-3	SP-SM	121.6	11.4	79.8	18,122	15,961
U-023-N (13-07)		X	A-3	SP-SM	115.4	6.5	38.2	22,608	20,574
M-068-W (04-03)		X	A-3	SP	100.9	20.0	80.6	9,969	10,004
M-020-W (07-02)		X	A-3	SP	110.5	11.5	59.2	29,418	28,566
M-059-W (13-02)		X	A-3	SP	107.7	9.0	43.1	24,840	23,788
M-059-W (13-02)		X	A-3	SP	104.4	7.9	34.6	23,195	
U-127-N (05-04)		X	A-3	SP	112.6	6.9	37.5	37,123	29,921
I-075-N (03-04)		X	A-3	SP	111.7	6.9	36.6	26,115	24,378
I-094-W (11-02)		X	A-3	SP	116.7	6.2	37.7	44,479	27,346
I-094-W (13-04)		X	A-3	SP	114.3	6.0	34.2	21,449	18,842
U-024-S (14-04)		X	A-3	SP	108.2	10.0	48.5	22,768	21,924
M-020-W (07-02)		X	A-3	SP	109.2	5.3	26.4	30,244	24,872
I-069-E (09-10)		X	A-3	SP	116.9	5.1	31.2	28,636	26,070
M-132-N (06-01)		X	A-3	SP	112.9	4.7	25.8	31,711	28,970
M-053-S (14-07)		X	A-3	SP	113.9	3.9	22.0	25,714	22,275

3.6 DATA ANALYSIS

The hand held pocket penetrometer and the pocket vane shear tester data as well as all the laboratory test data were analyzed. Results of the analyses are presented and discussed in the next subsections.

Hand-Held Pocket Penetrometer and Vane Shear Tester - The locations where pocket vane shear and pocket penetrometer tests were conducted are listed in Table D.2 of Appendix D. Results of the pocket penetrometer (PPR) and pocket vane shear (VSR) tests were compared as shown in Figure 3.10. The best fit curve (trendline), the coefficient of correlation and the resulting correlation equation (Equation 3.1) are also shown in the figure.

$$PPR = 0.9888 \ln(VSR)^{0.4685} \qquad \text{Equation 3.1}$$

The poor correlation between the two sets of data was expected because each set expresses the shear strength of the soils at a given location. The scatter of the data is due mainly to the geometry of the instruments (the pocket penetrometer has much less contact area with the soil than the pocket vane shear). When the two sets of data were compared to the laboratory obtained resilient modulus of the same soil, almost no correlation was found. This observation was also expected because the resilient modulus tests were conducted using low stress level (in the elastic range) whereas the pocket penetrometer and vane shear tests were conducted mainly in the plastic zone near or at failure.

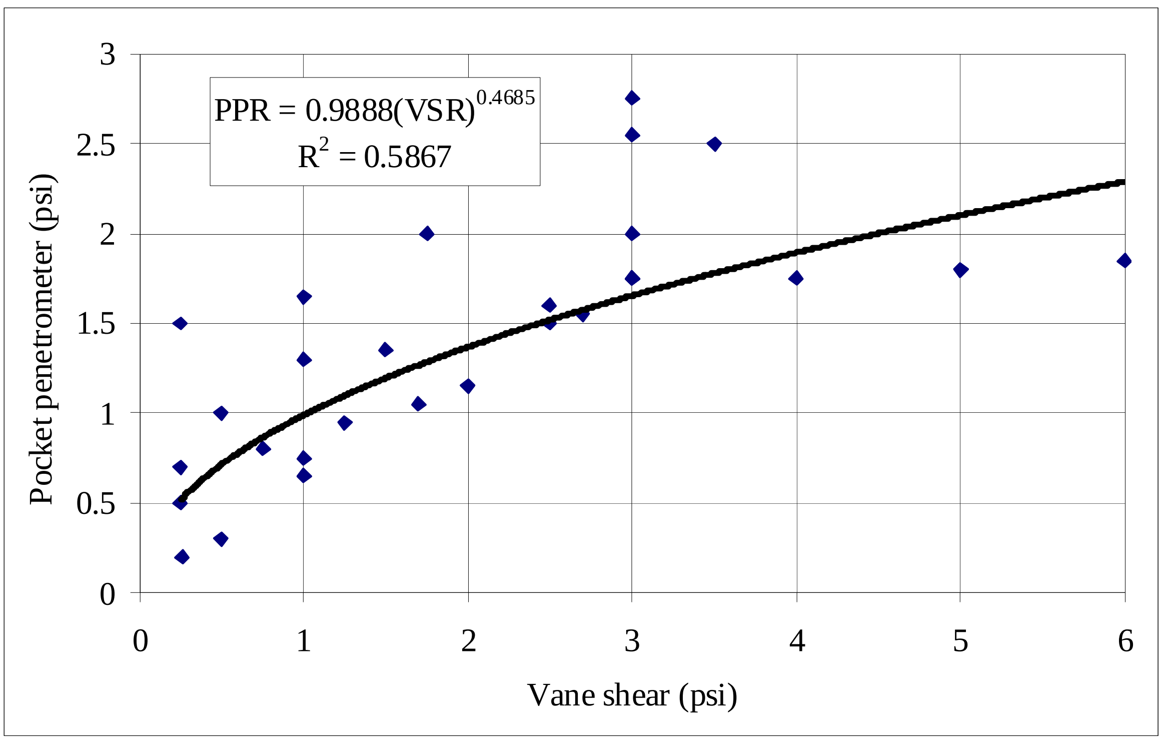


Figure 3.10 Pocket penetrometer versus vane shear test data

3.6.1 Soil Classification

For each disturbed soil sample the dry and/or wet sieve test data, the hydrometer test data, and the Atterberg Limits data were analyzed to determine:

- The grain size distribution curve and the coefficients of uniformity and curvature.
- The soil classification according to the USCS and the AASHTO system.

Results of the analyses are listed in Table D.3 of Appendix D, and an example is listed in Table 3.3. Table 3.5, provides a list of the number of disturbed soil samples that were classified as one soil type according to the AASHTO system and the USCS. As can be seen, the majority of the roadbed soils in the State of Michigan can be divided, in general, into 8 soil classification types for both the USCS and the AASHTO soil classification system. Figure 3.11 depict the grain size distribution curves for 6 soil samples. The numbers in the legend of the figure express the cluster and area numbers where the soil samples were obtained. It is very important to note that although soil samples obtained from different areas showed the same classification, the soils have a wide range of grain size distribution parameters that fall within the classification boundaries. The vast range of gradation parameters within a given soil type or classification is the direct result of the glaciations and glacial deposits in the State of Michigan. Finally, details of the AASHTO soil classification system and the USCS are included in Appendix C.

Table 3.5 Number of samples per soil type

USCS		AASHTO Classification	
Soil classification	Number of samples	Soil classification	Number of samples
SP	20	A-1-a	2
SM	16	A-1-b	1
CL	8	A-2-4	21
ML	2	A-2-6	3
SC	18	A-3	26
SC-SM	7	A-4	10
SP-SM	8	A-6	12
SG	2	A-7-6	6

3.6.2 Cyclic Load Triaxial Test Results

Recall that, in the laboratory, most soil samples were tested to determine their natural moisture content, grain size distribution, Atterberg limits, and resilient modulus using cyclic load triaxial tests. As stated earlier, all cyclic load tests were conducted using confining pressure of 7.5 psi. In each test, after applying the confining pressure, the soil samples were subjected to 10 psi cyclic axial stress and deformations were recorded at 5 intervals (one at each of load cycles 100, 200, 500, 800, and 1000). Each interval consisted of data from three consecutive load cycles, for example, the interval at load cycle number 100 consists of the axial cyclic load and deformations data at cycle numbers 99, 100, and 101. For each load cycle within a given interval, the resilient modulus was calculated and the average resilient modulus from the three consecutive load cycles

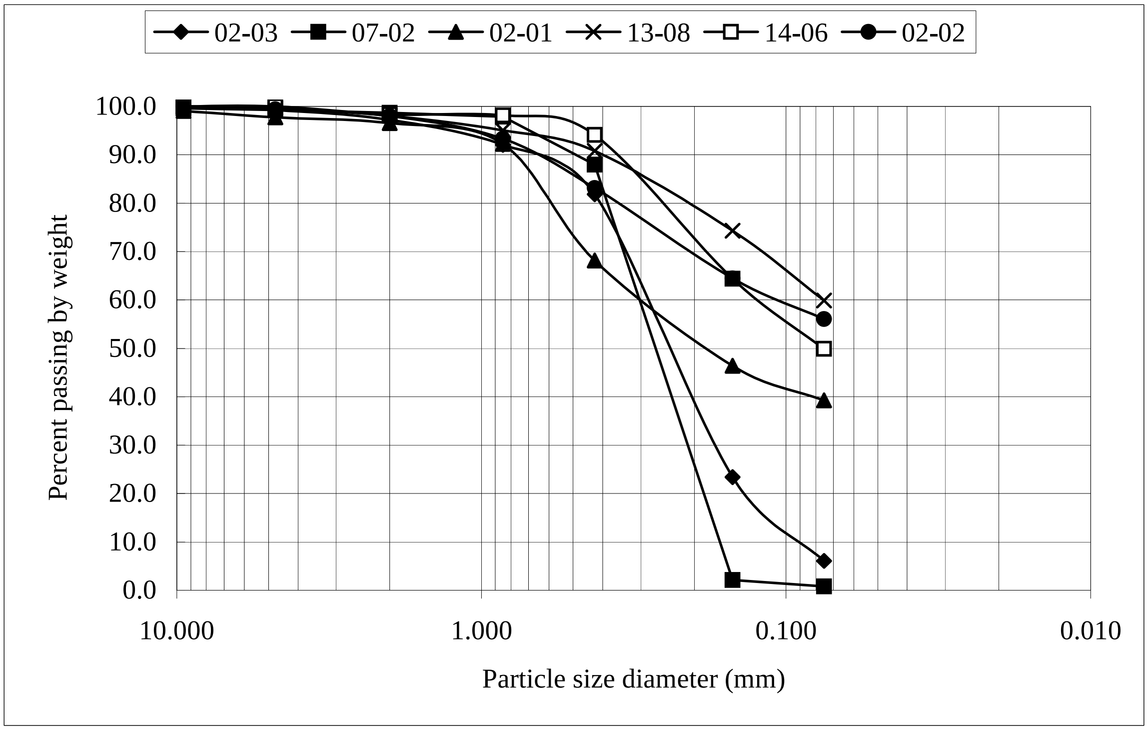


Figure 3.11 Typical particle size distribution curves

was determined. The test was terminated at load cycle number 1001, the number of load cycles was reset, the axial cyclic stress was increased to 15 psi and the test restarted again. The resilient modulus for the 15 psi axial cyclic stress was calculated in the same manner as that of the 10 psi axial cyclic stress test. For each axial stress level, Table D.5 of Appendix D provides a list of:

- The sample designation number
- The AASHTO and USCS soil classification
- The average cyclic stress and the average cyclic load
- The average deformation and the calculated average resilient modulus value for each load cycle interval of 100, 200, 500, 800, and 1000.
- The calculated average resilient modulus value at the three load cycle intervals of 500, 800, and 1000.

Table 3.6 provides an example of the data listed above for four test samples. The data for all test samples are listed in Table D.5.

Effects of Axial Stress Level on MR Values - Figure 3.12 depicts the resilient modulus values obtained at 10 and 15 psi cyclic axial stress levels for all soil classification. The line of equality between the two sets of MR values is also shown in the figure. The data in the figure indicates that, in general, the MR values decrease slightly with increasing cyclic axial stress level, which indicates slight non-linearity. This observation agrees with that reported by Young and Baladi (1977). It should be noted that the above observation does not necessarily disagree with the bulk stress model stated in the M-E PDG (see Equation 2.1 in Chapter 2). In this model, as the axial stress increases, the bulk stress (the sum of the axial stress and twice the confining pressure) and

Table 3.6 Example of laboratory resilient modulus results

Sample number	Soil Type		Cycle number	Cyclic stress (psi)							
				10				15			
	AASHTO	USCS		Average cyclic load (lbs)	Average deformation (mils)	Average resilient modulus (psi)	Average MR (psi) at load cycles 500, 800 and 1000	Average cyclic load (lbs)	Average deformation (mils)	Average resilient modulus (psi)	Average MR (psi) at load cycles 500, 800 and 1000
M-045-S (01-01)	A-6	CL	100	31.6	2.304	35,043	36,543	49.0	3.740	31,266	31,503
			200	32.1	2.202	36,823		50.3	3.774	31,862	
			500	32.2	2.262	36,639		50.1	3.663	31,747	
			800	32.5	2.205	37,056		50.1	3.817	31,297	
			1000	32.8	2.227	35,934		50.4	3.872	31,465	
U-002-E (02-01)	A-4	SM	100	32.5	3.729	13,894	15,352	50.3	5.850	12,872	13,818
			200	32.9	3.592	14,285		50.1	5.727	13,150	
			500	32.7	3.442	15,044		50.4	5.551	13,686	
			800	32.7	3.325	15,708		50.4	5.496	13,826	
			1000	33.3	3.415	15,305		49.9	5.364	13,942	
M-028-W (02-02)	A-4	ML	100	32.0	1.741	48,422	53,824	50.7	2.777	45,310	41,516
			200	32.5	1.650	50,092		51.0	2.801	44,090	
			500	32.7	1.569	53,892		51.3	2.969	42,510	
			800	32.7	1.600	53,350		51.3	3.047	41,331	
			1000	33.0	1.598	54,230		51.3	3.087	40,707	
M-028-W (02-03)	A-1-b	SP-SM	100	33.9	2.675	19,996	19,195	51.4	4.042	16,997	17,845
			200	33.8	2.698	20,013		51.4	3.956	16,510	
			500	33.7	2.821	19,057		52.6	3.873	17,649	
			800	33.8	2.796	19,502		51.7	3.733	17,942	
			1000	34.0	2.792	19,025		51.5	3.774	17,945	

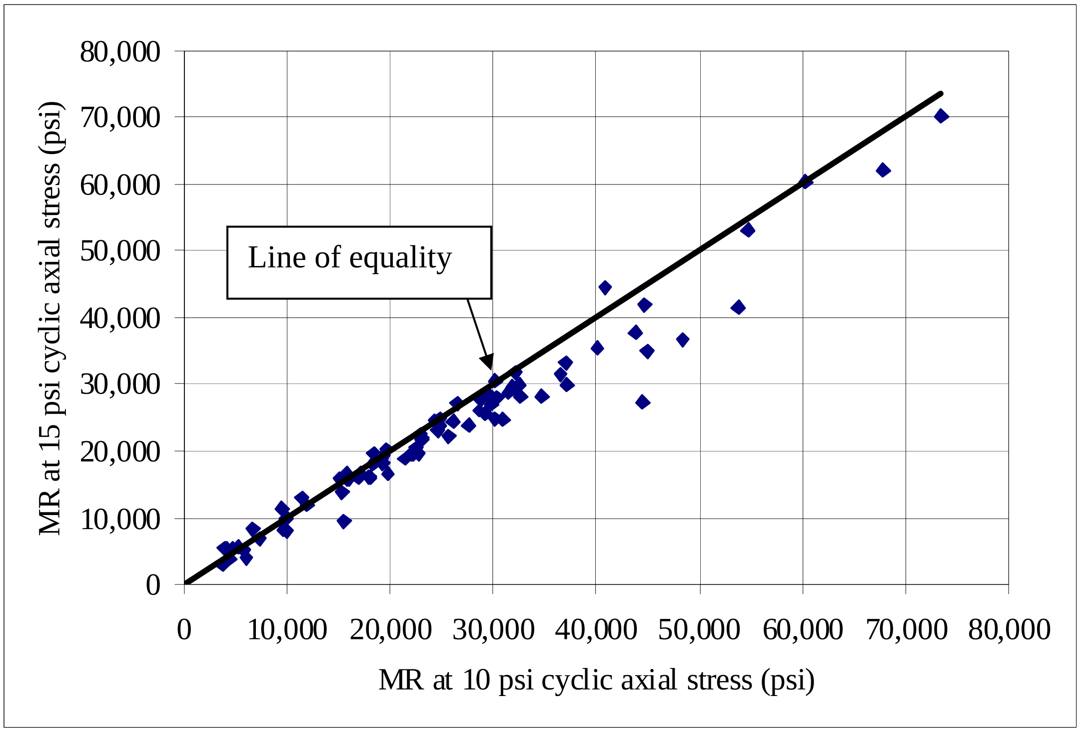


Figure 3.12 Resilient moduli at 10 and 15 psi cyclic axial stresses

the octahedral shear stresses (the sum of the differences between the principal stresses) increase. The effect of the latter is typically greater than that of the former. Since k_3 in Equation 2.1 is negative, it implies increasing the octahedral stress leads to decreases in the MR values.

Effects of the Sample Variables on MR Values - Given the usual pavement cross-section used in the State of Michigan (the roadbed soil is located between 18 to 36 inches below the pavement surface), the roadbed soil is typically subjected to 4 to 7 psi vertical stress due to half of an 18,000 pound single axle load. Therefore, the effects of the sample variables on MR values are discussed for the axial stress level of 10 psi only. In addition, the effects of the sample variables on the MR values of the roadbed soils were studied in a two step procedure as follows:

1. In the first step, the soils were divided into six groups according to their USCS classification listed below.

   - Poorly graded sand (SP)
   - Silty sand (SM)
   - Clayey sand (SC), low plasticity clay (CL), and low plasticity silt (ML)
   - Poorly graded sand – silty sand (SP-SM)
   - Clayey sand – silty sand (SC-SM)
   - Gravelly sand (SG)

2. In the second step, univariate and multivariate analyses were conducted to determine the relationships (if any) between the resilient modulus values and the sample parameters. Results of these analyses for each of the six soil types are discussed below.

3.6.2.1 Poorly Graded Sand (SP)

Table 3.7 lists the sample designation number, the location, and the USCS and the AASHTO soil classifications of twenty disturbed soil samples that were collected from various clusters and areas throughout the State of Michigan. Results of the soil classification conducted in step 1 of the analyses indicated that eighteen samples can be designated as A-3, one as A-1-a, and one sample as A-1-b according to the AASHTO soil classification system, whereas all twenty samples were classified as SP according to the USCS. In Table 3.7, eleven soil samples are labeled SP1 and nine SP2. The reason for that is stated below in the subsection titled “effects of sample grain size.” It should be noted that more than twenty rows are presented in Table 3.7; this is due to the fact that some samples were tested more than once. For example, sample M-020-W (07-02) is listed 5 times because it was tested as part of the main study as well as to study the effects of moisture on the MR values. In step 2 of the analyses, univariate and multivariate analyses were conducted to determine the relationships between the sample variables and the resilient modulus of the roadbed soils. Results of these analyses are presented in the next two subsections. The cyclic load triaxial test results of four samples (the shaded data in Table 3.7) were used for model verification (discussed later in the subsection).

Univariate Analyses - In the univariate analysis, the effects of each individual test sample variables on the MR values of SP soils were studied. These included: the moisture content of the samples, the laboratory compacted dry unit weight, and grain size distribution parameters. The discussion of the effects of each variable is presented below.

Effect of Sample Grain Size – To study the effects of sample gradation on the MR values of SP soils, sieve analyses were conducted and grain size distribution curves were plotted to determine the coefficients of curvature and uniformity. Observation of the grain size distribution curves indicate that the twenty poorly graded roadbed soil samples can be divided into two categories according to the slope of the gradation curve between the percent passing sieves number 40 and sieve number 200 as shown in Figure 3.13. Soils having the steep curves are labeled SP1 whereas the others are labeled SP2 (see Table 3.7). When the locations of SP1 and SP2 soils were studied, it was clear that all SP2 soils are located in the eastern half of the State of Michigan while SP1 soils in the western half. Examination of the sieve analyses data of both soils indicated that:

a) The percent passing sieve number 40 of the SP1 soil is about 90 percent whereas it is 50 percent for the SP2.
b) The fine materials (passing sieve number 40) of the SP1 soils are mainly silt while they are a combination of clay and silt for the SP2 soils.

The possible causes of these differences include:

- The two soils have different origins, the SP2 soils were deposited as the glacial lobe, which was advanced along Lake Huron, retreated. The SP1 soils, on the other hand, were deposited when the glacial lobe, which was advanced along the Lake Michigan trough, retreated.
- The SP1 soils were deposited by gently flowing melted water (it contains higher percent of fine sand) while the SP2 soils were deposited by relatively faster moving melted water.

Table 3.7 Location of SP roadbed soils

Sample designation number	Location	AASHTO	USCS
M-028-W (03-02)	1500 feet North of M-77	A-3	SP1
I-075-N (03-04)	mile marker 380	A-3	SP1
M-132-N (06-01)	1000 feet North of Addis Rd (paved rd)	A-3	SP1
I-075-N (06-02)	160 feet North of Co Rd 662	A-3	SP1
U-031-N (06-03)	307 feet North of M-46	A-3	SP1
M-020-W (07-02)	~.5 mile East of 13 Mile Rd	A-3	SP1
M-020-W (07-02)	~.5 mile East of 13 Mile Rd	A-3	SP1
M-020-W (07-02)	~.5 mile East of 13 Mile Rd	A-3	SP1
M-020-W (07-02)	~.5 mile East of 13 Mile Rd	A-3	SP1
M-020-W (07-02)	~.5 mile East of 13 Mile Rd	A-3	SP1
M-020-E (07-03)	~500 feet East of Cottonwood Ave	A-3	SP1
U-131-S (09-01)	160 feet South of Lake Montcalm Rd	A-3	SP1
U-131-S (09-03)	105 feet South of 110th Ave	A-3	SP1
U-131-S (09-05)	60 feet South of 'Reduce Speed 55 MPH' sign right where it turns from interstate to limited access	A-3	SP1
I-094-W (11-02)	132 feet West of exit 110 on ramp	A-1-a	SP1
U-023-S (04-01)	320 feet North of F 05 Co Rd	A-3	SP2
M-068-W (04-03)	150 feet West of Little Ocqueoc River	A-3	SP2
U-127-N (05-04)	120 feet North of Co Rd 300	A-3	SP2
I-069-E (09-10)	172 feet East of Grand River Rd	A-1-b	SP2
M-059-W (13-02)	Station 131+29	A-3	SP2
M-059-W (13-02)	Station 131+29	A-3	SP2
I-094-W (13-04)	Station 75+02	A-3	SP2
U-024-S (14-04)	150 feet North of Pardee	A-3	SP2
M-053-S (14-07)	1500 feet South of Canal Rd	A-3	SP2
M-025-S (15-05)	200 feet North of Day Rd	A-3	SP2

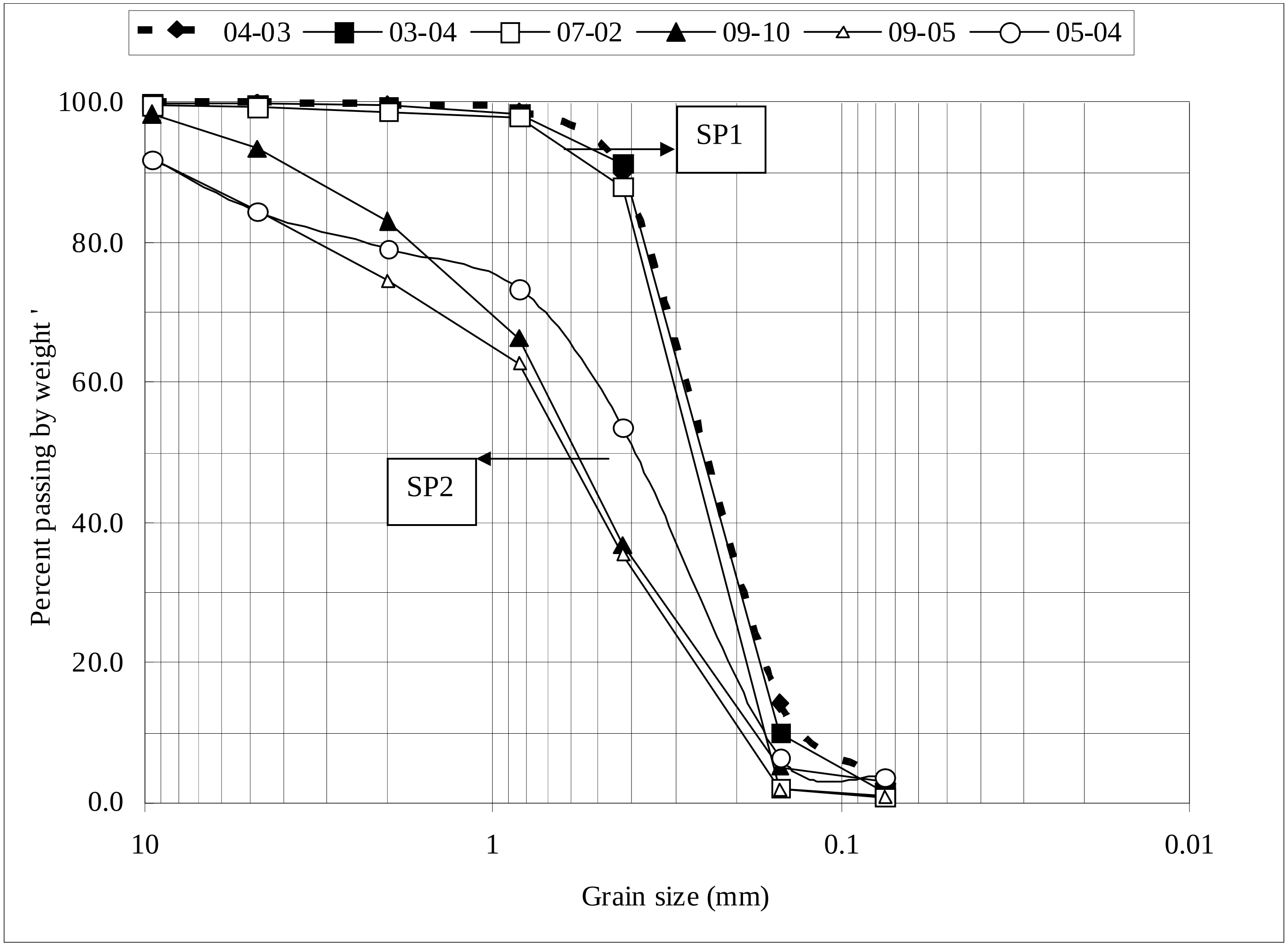


Figure 3.13 Grain size distribution curves for SP soils

- The differences in the soil origin and deposition may have created different angularity of the coarse materials. Unfortunately, the soil angularity was not measured due to lack of proper equipment nor was it a part of this study.

The effects of the sample variables of both the SP1 and SP2 soils on their MR values were studied by plotting the MR values versus each of the following grain size parameters:

- Percent passing sieves number 200, 100, 40, 20, 10, 4, and 3/8 inch
- The coefficients of curvature and uniformity (C_C and C_U)
- The average particle size at ten, thirty, and sixty percent passing (D_{10}, D_{30}, and D_{60}, respectively)
- The coarse sand content (percent passing sieve number 4 – percent passing sieve number 40) and the fine sand content (percent passing sieve number 40 – percent passing sieve number 200)

The data in most plots were scattered indicating no relationship between the grain size data and the MR values. An example plot between the percent passing sieve number 10 and the MR values is shown in Figure 3.14. The scenario however, was drastically different when the data were separated into two groups SP1 and SP2 as shown in Figure 3.15, where some of the data showed moderate degree of correlation (R^2 for SP1 soils of 0.61 and 0.15 for SP2 soils). Given this observation, it was decided to include the grain size distribution parameters in the multivariate analyses of the SP1 and SP2 soils. A similar, but stronger scenario was found when the MR values of the SP soils were plotted against the laboratory compacted dry density of the samples as discussed in the next subsection.

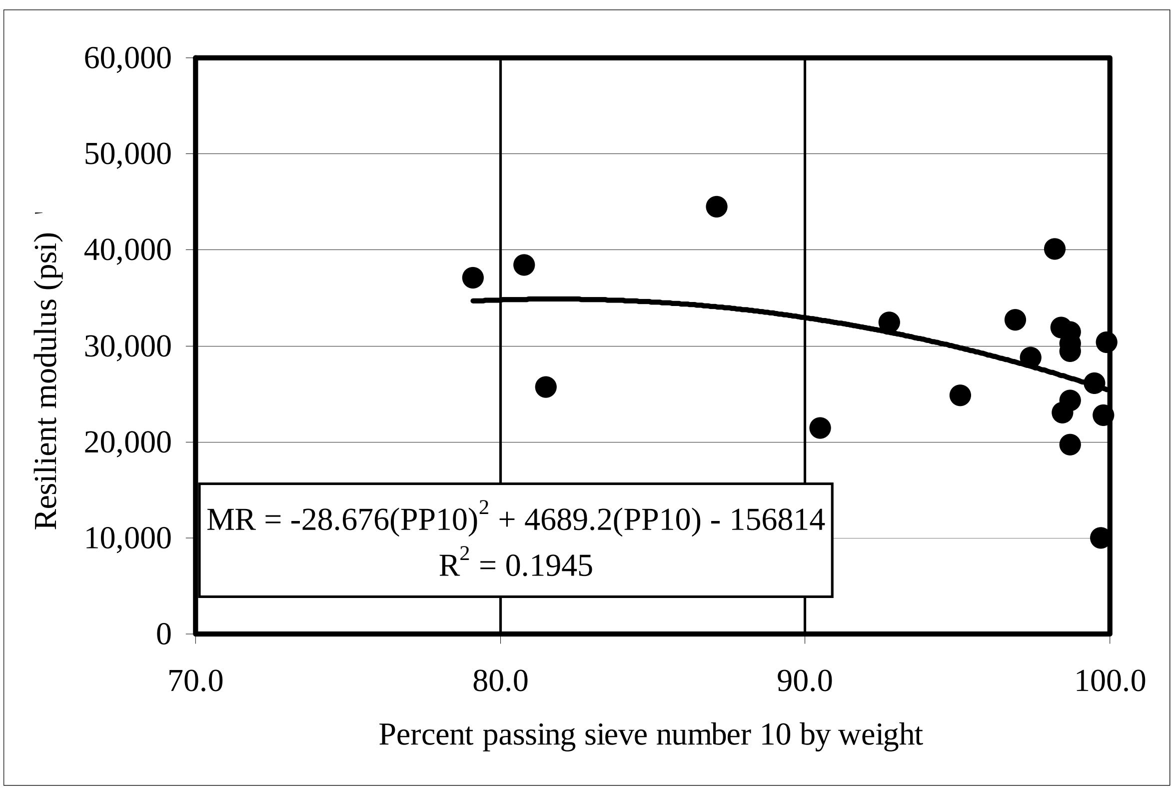


Figure 3.14 Resilient modulus versus the percent passing sieve number 10 for SP soils

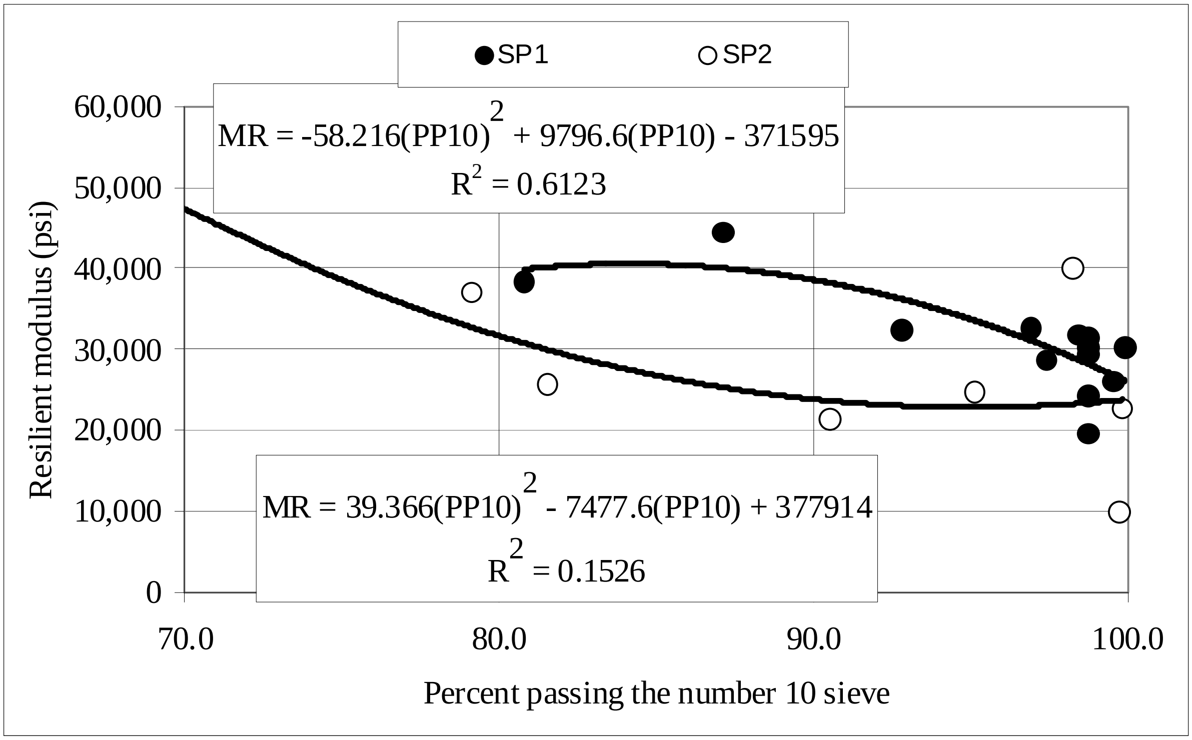


Figure 3.15 Resilient modulus versus the percent passing sieve number 10 for SP1 & SP2

Effect of Sample Dry Unit Weight - The effect of the dry unit weight of the laboratory compacted test samples on the MR values for SP soils was studied by testing three soil samples obtained from the same location and compacted at the same water content using different compaction effort (vibrating times of 1, 2, and 3 minutes). The resulting dry unit weights were 104.1, 106.6, and 109.8 pcf. After compaction, the three samples were subjected to cyclic load tests and the results were used to calculate the MR values of the soil. Figure 3.16 shows the MR values plotted as a function of the dry unit weight. As can be seen from the figure, higher dry unit weights yield higher MR values. This result was expected and similar ones were reported by score of researchers including Maher et al. 2000.

Effect of the Sample Moisture Content – To study the effects of moisture content on the MR values of SP soils, the soil sample with the lowest natural water content (0.2 percent) was selected for testing. This selection allowed the addition of water to increase the water content of the sample from 0.2 percent to 5.3 and to 11.5 percent. The three water contents were selected such that the highest degree of saturation of the samples would be less than eighty percent. The eighty percent saturation level may cause liquefaction and a total loss of shearing resistance (Richart et al 1970). The three selected water contents correspond to degrees of saturation of 1, 26.4, and 59.3 percent, respectively. For each of the three water contents a soil sample was compacted using the same static load and vibrating table described in section 3.5.4 of this chapter. All three samples were compacted using the same compaction effort.

After compaction, the sample was subjected to cyclic load test. Results of the cyclic load tests were used to calculate the resilient modulus of the sample. Figure 3.17 depicts the resilient modulus of the three soil samples plotted as a function of their water contents. The figure shows that for the given range of moisture content, their effect on MR values is insignificant. This was

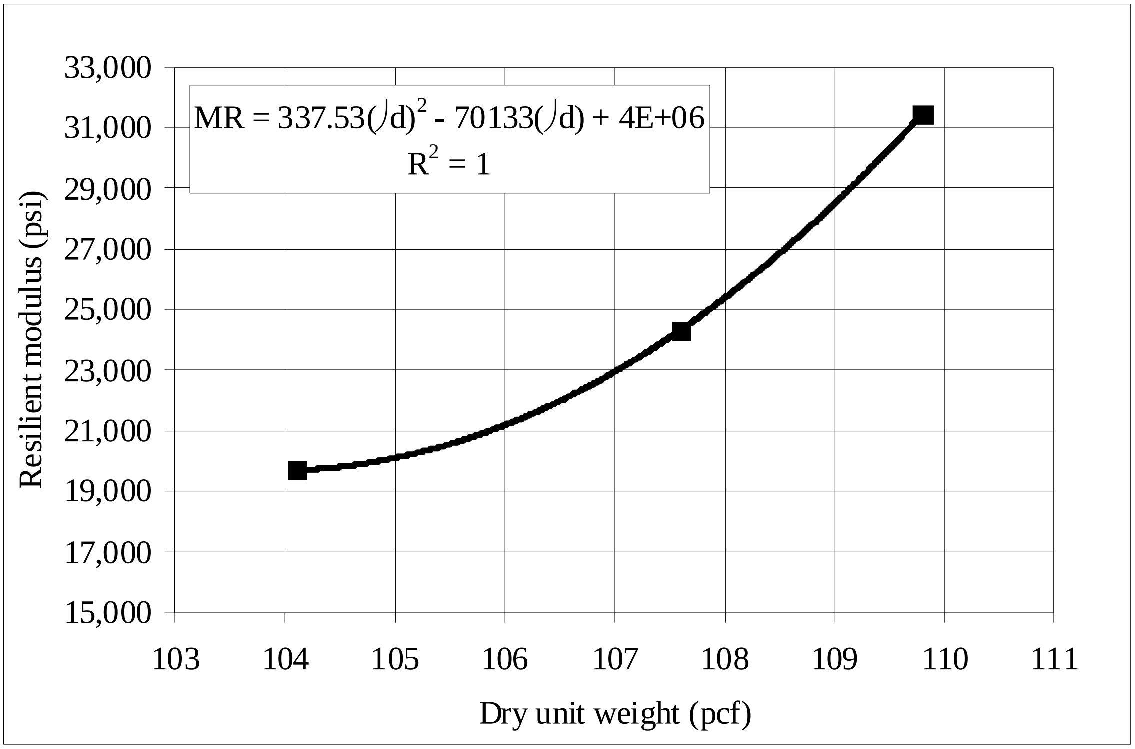


Figure 3.16 Resilient modulus versus the dry unit weights for one SP soil

expected because, in general, the strength and stiffness of sand soils are only slightly affected by the water content of the soils. At low water contents, the capillary between the sand particles slightly increases the normal stress and hence, the friction between particles. As the moisture content increases, the lubrication between the sand particles increases, which overcomes the capillary effect and the strength begins to decrease. When the sample is at 80 percent saturation level or higher, the pore water pressure increases during shearing and the effective stress decreases causing decreases in the angle of internal friction. At or near saturation, the strength drops to zero and the sand liquefies. Similar results were reported by (Young and Baladi 1977, Holtz and Kovacs 1981, and Richart et al 1970).

Multivariate Analyses - Multivariate analyses were conducted to study the combined effects of several independent Sample Variables of the SP (SVSP) soils on the dependent variable (MR values) of those soils. The term SVSP was divided into two terms; SVSP1 and SVSP2 to express the two SP soil subgroups; SP1 and SP2. During the multivariate analyses:

- Various models (equation forms) were used in an attempt to maximize the value of the coefficient of determination (R^2).
- Special care was taken to:

    - Ensure that the resulting equation satisfies the previously reported trends between each of the independent variables and the dependent variable (MR values.)
    - Avoid any significant co-linearity between the independent variables.
    - Decrease the number of independent variables in the equation.

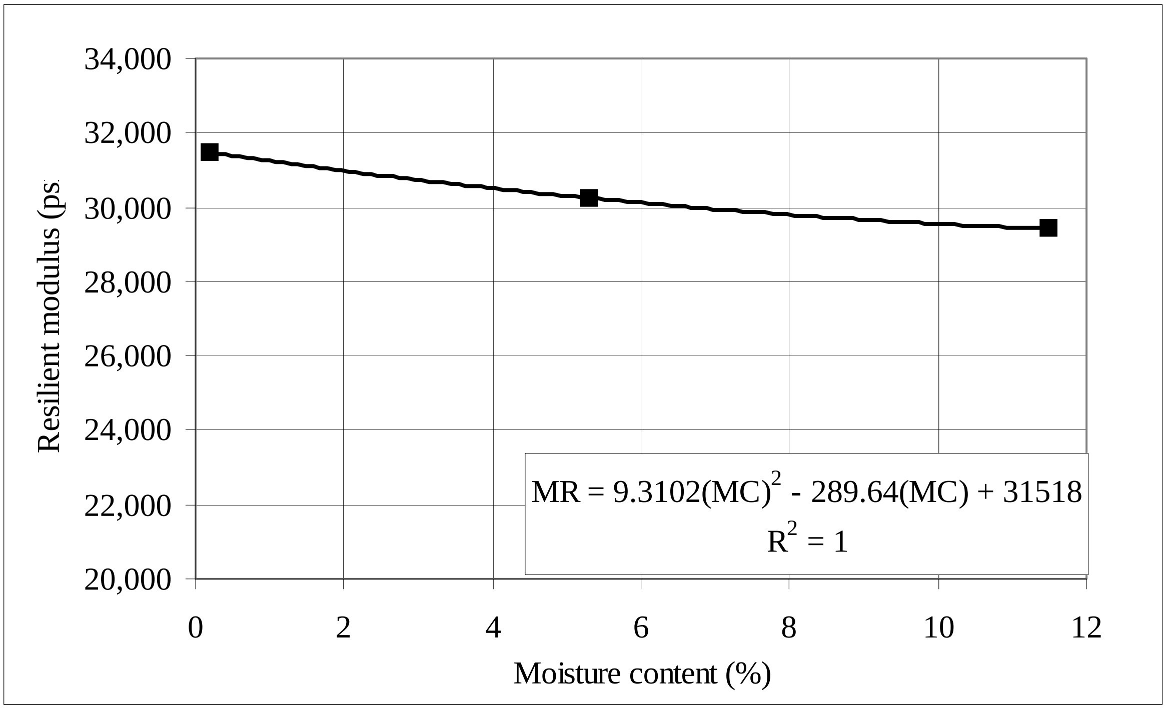


Figure 3.17 Resilient modulus versus moisture content of one SP soil sample

The multivariate analyses yielded Equations 3.2 and 3.3 for SP1 soils and Equations 3.4 and 3.5 for SP2 soils.

$$MR = 89.825(SVSP1^{2.9437}), \quad R^2 \approx 0.78 \qquad \text{Equation 3.2}$$

$$SVSP1 = \frac{\gamma_d^{1.15}}{\left(P_4^{1.5} - P_{40}^{0.25}\right)^{0.5}} \qquad \text{Equation 3.3}$$

$$MR = 0.8295(SVSP2^{3.6006}); \quad R^2 \approx 0.81 \qquad \text{Equation 3.4}$$

$$SVSP2 = \frac{\gamma_d^{1.35} * P_{200}^{-0.1}}{\left(P_4^{1.5} - P_{40}^{0.25}\right)^{0.5}} \qquad \text{Equation 3.5}$$

Where, γ_d is the dry unit weight (pcf) of the laboratory compacted sample, and P_4, P_{40}, and P_{200} are the percent by weight passing sieve numbers 4, 40, and 200, respectively.

Examination of Equations 3.2 and 3.4 indicates that the resilient modulus values of the SP1 and SP2 soil groups are a function of the dry unit weight of the soil and the parameter $(P_4^{\alpha} - P_{40}^{\beta})^{\omega}$, which represents the coarse sand content in the soil. Further, the resilient modulus values of the SP2 soil group are also a function of the percent passing sieve number 200. Hence, the data and results of the multivariate analyses reflect the shape of the gradation curves of SP1 and SP2 soils shown in Figure 3.13. The shapes of the gradation curves imply that the mechanistic behavior of

SP1 and SP2 soils are not the same. The SP1 soils are deficient in their coarse sand content; hence, coarse sand particles are floating in the fine sand matrix. For SP2 soils, the opposite is true. The SP2 soils contain relatively large amounts of coarse sands; hence, the coarse sand particles are likely in contact with each other while the fine sand particles are filling the voids between the coarse particles.

Figure 3.18 shows the MR values of SP1 and SP2 soils plotted against the predicted sample variables; SVSP1 and SVSP2, respectively. The equations of the best fit lines, the coefficients of determination, and the standard errors are also shown in the figure. Examination of the figure indicates that for SP1 and SP2 soils, higher values of SVSP1 and SVSP2 produce higher MR values. This was expected because higher dry unit weights imply denser particle packing, higher relative density, higher friction, higher stiffness, and hence, higher MR values. On the other hand, for SP2 soils, higher percent fine (passing sieve number 200) yields lower MR. Once again; this was expected and agreed with most findings reported in the literature.

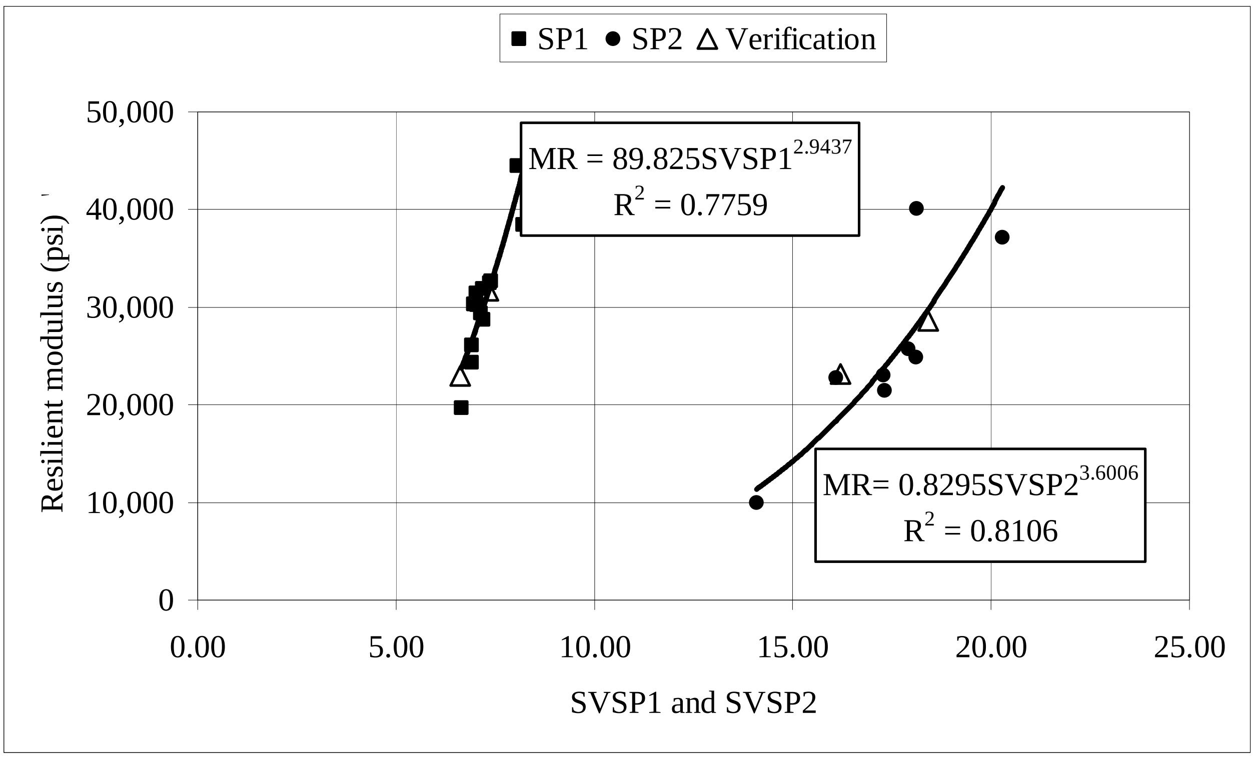


Figure 3.18 Resilient modulus versus SVSP1 and SVSP2

It should be noted that numerous attempts were made to include all SP soils in the models by conducting various univariate analyses using all sample variables. None of the attempts yielded significant increases in the value of R^2.

Equations 3.2 and 3.4 apply to samples with dry unit weight values ranging from 100.9 to 120.3 pcf and percent passing sieve number 4 values between 84.5 and 100.0 percent while the percent passing sieve number 40 is between 36.8 and 97.2 percent. The range in percent passing sieve number 200 is 0.5 to 4.7 percent. This range is the entire allowable range of fine contents of SP soils. The use of the two equations outside the stated ranges may yield unrealistic MR values.

Verification - In order to check the validity of Equations 3.2 and 3.4, the two SP1 and the two SP2 samples highlighted in Table 3.7, which were not included in the development of the equations, were subjected to grain size analysis, and cyclic load triaxial tests. The test results were used to calculate the resilient modulus values of the four samples and the sample variables (SVSP1 and SVSP2). The data are shown in Figure 3.18 as open triangles. It can be seen that the data for the four samples are very close to the best fit curve. Further, the three SVSP1 values for SP1 samples and the two SVSP2 values for SP2 samples were used in Equations 3.2 and 3.4, respectively, and the resilient modulus of the four samples were predicted. Figure 3.19 shows the measured and the predicted MR values of the four samples. The straight line in the figure is the line of equality between the predicted and the measured MR values. As can be seen from the figure, the predicted MR values are almost equal to the laboratory measured values. Hence, one can conclude that the developed models (Equations 3.2 and 3.4) are relatively accurate and can be used to estimate the resilient modulus of the soils based on knowledge of the dry unit weight of the soils and their grain size distribution.

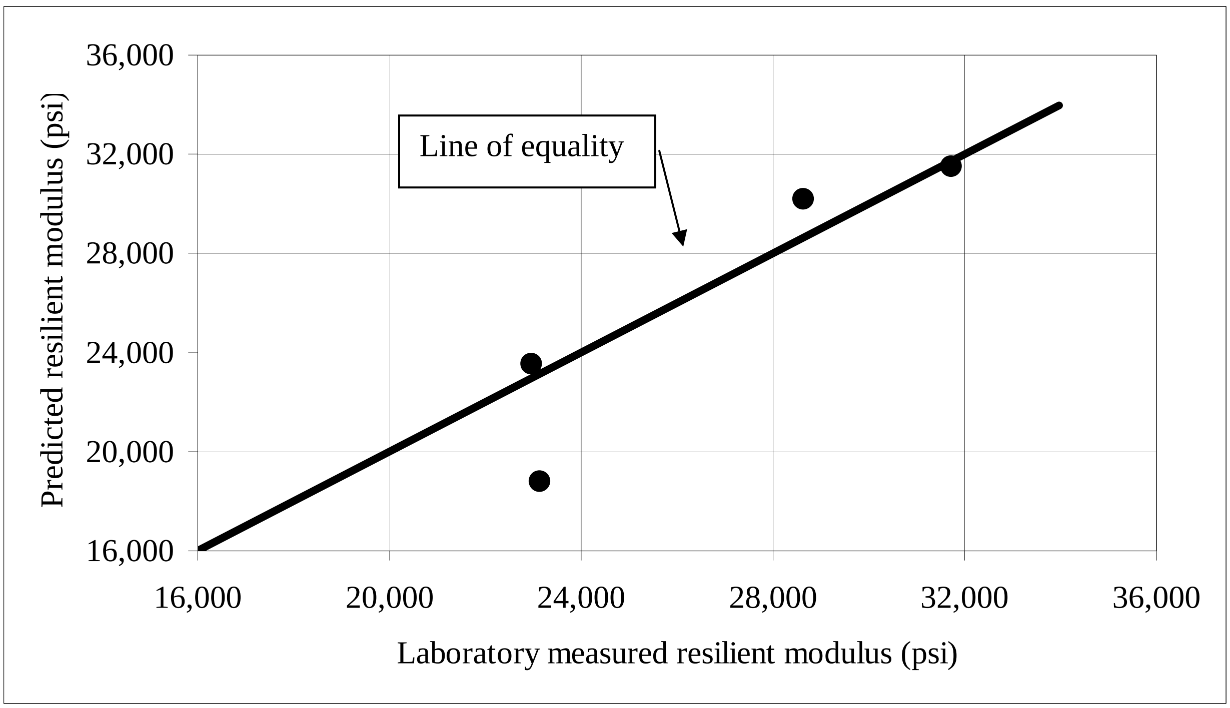


Figure 3.19 Predicted versus laboratory measured resilient modulus

3.6.2.2 Silty Sand (SM)

Table 3.8 lists the locations, the USCS and the AASHTO soil classifications, and the sample designation number of sixteen disturbed soil samples that were collected from various clusters and areas throughout the State of Michigan. The commonality between the sixteen samples is that all of them were classified in step 1 of the analyses as silty sand (SM) soils according to the USCS. Recall that the USCS specifies that SM soils may contain anywhere between 12 and 49.9 percent passing sieve number 200 (Holtz and Kovacs 1981). Hence, the fine materials play a major role in the mechanistic behavior and the resilient modulus (MR) values of the soil. In step 2 of the analyses, univariate and multivariate analyses were conducted to develop correlations between the test sample variables and their resilient modulus values. The results are presented and discussed below. It should be noted that the five rows highlighted in Table 3.8 were not included in the development of the correlation equations between the sample variables and their MR values. They were tested and the test data were used to verify the developed models. It should be noted that there are more than sixteen rows in Table 3.8. This is due to the fact that some samples were tested more than once in order to verify the models presented later in the section.

Table 3.8 Locations of SM roadbed soils

Sample number	Location	AASHTO	USCS
U-002-E (02-01)	385 feet East of M-45	A-4	SM
U-002-E (03-03)	200 feet East of M-117	A-2-4	SM
M-065-S (04-04)	160 feet South of Elm Hwy	A-2-4	SM
U-131-N (05-01)	200 feet South of Michigan Fisheries Visitor Center	A-2-4	SM
M-061-E (07-06)	420 feet East of left hand turn on M-61 (off US-127)	A-2-4	SM
M-061-E (08-02)	165 feet West of Hockaday	A-2-4	SM
M-044-E (09-07)	Station 137+10	A-2-4	SM
M-024-S (09-09)	20 feet North of Burley Rd	A-2-4	SM
I-069-N (10-04)	150 feet North of Island Hwy	A-2-4	SM
I-069-N (10-05)	100 feet North of Five Points Hwy	A-2-4	SM
I-096-W (10-09)	140 feet West of Dietz Rd	A-2-4	SM
U-012-E (12-04)	100 feet East of Emarld Rd	A-2-4	SM
I-094-W (12-06)	53 feet West of Mt Hope Rd	A-2-4	SM
I-094-W (12-06)	53 feet West of Mt Hope Rd	A-2-4	SM
I-094-W (12-06)	53 feet West of Mt Hope Rd	A-2-4	SM
M-024-S (13-01)	250 feet North of Best Rd	A-4	SM
M-053-S (15-02)	300 feet South of M-46	A-2-4	SM
M-019-S (15-07)	650 feet South of Thompson Rd 1 mile South of M-142	A-2-4	SM

Univariate Analysis - In the univariate analyses, the effects of each of several sample variables on the MR values of SM soils were studied. These sample variables include: the moisture content of the samples, the degrees of saturation, the liquid limits, the dry unit weight after compaction, and the grain sizes. The effects of each variable on the resilient modulus values are presented and discussed below.

Effect of the Sample Moisture Content – For SM soils, it was hypothesized that because of the high range of fine content, the water content should play a major role in determining the elastic response of the soil to the applied loads. Figure 3.20 shows the MR values of thirteen SM soil samples (the five shaded samples in Table 3.8 are not included) plotted against the samples moisture contents. As it was expected, the figure shows increases in the sample moisture content cause significant decreases in the MR values. This observation tends to validate the hypothesis stated above. Similar results were also reported by many researchers including (Maher et. al 2000, George 2000, and 2003). One observation is important to note herein is that the effect of moisture content on the resilient modulus values of SM soils is much higher than that for the SP soils reported in the previous section. This is mainly due to the much higher fine content in the SM soils compared to the fine content of the SP soils (less than 5 percent). Since the water content is strongly correlated to the MR values of SM soils, it will be included in the multivariate analyses.

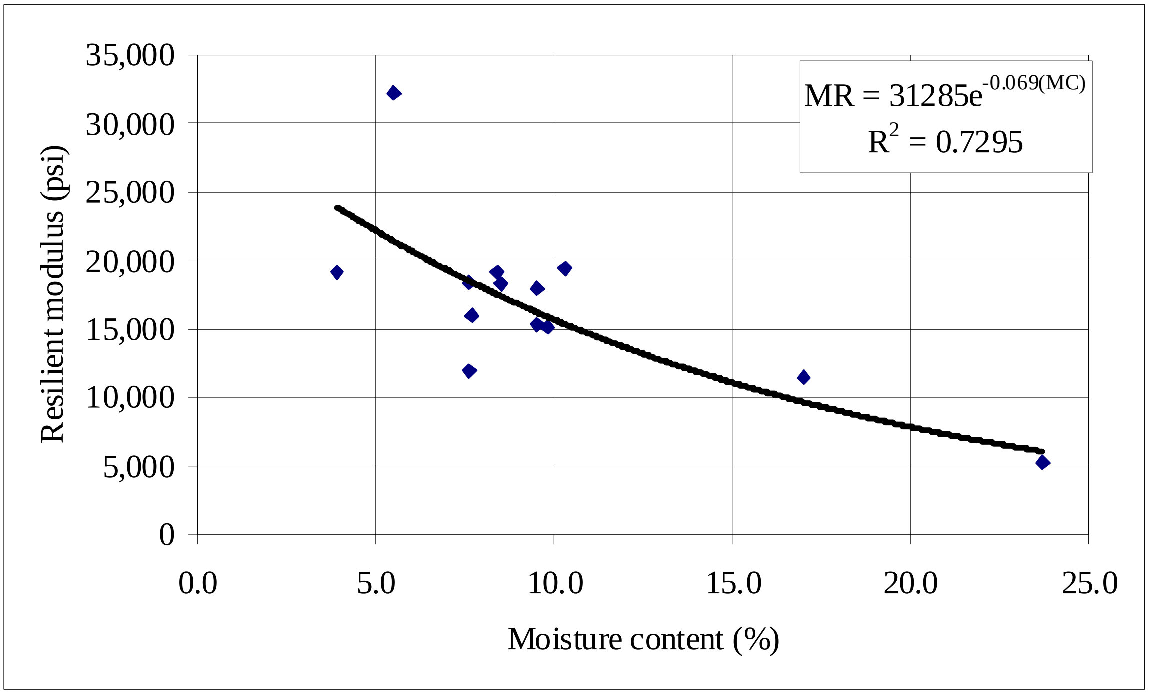


Figure 3.20 Resilient modulus versus water contents of the SM soil samples

Effect of Sample Dry Unit Weight - The effect of the test sample dry unit weight on the MR values of SM soils was studied by plotting the MR values as a function of the sample dry unit weight as shown in Figure 3.21. As was expected, the figure shows a weak correlation between the dry unit weight and the MR values of the test samples. The main reason for the weak correlation is that the water contents of the test samples vary from about 3 to about 23 percent. Such variation in the water content, (when examined in perspective of the compaction curve) covers both the wet and the dry sides of the curve. This implies that two soil samples having the same dry unit weight value may have two significantly different water contents. One is located on the wet side of the optimum moisture content and the other on the dry side. Test samples compacted on the dry side of optimum would have higher strength and stiffness and display a more brittle behavior than those compacted wet of optimum. The latter would have lower strength, higher plastic deformation, and softer behavior under loads. The differences in the behavior are directly related to differences in the degrees of lubrication caused by the water and the particle arrangement in the soil. The soil particles of a soil sample compacted on the dry side of optimum tend to stay in a flocculated arrangement whereas on the wet side of optimum, they are dispersed (they line up), (Holtz and Kovacs 1981).

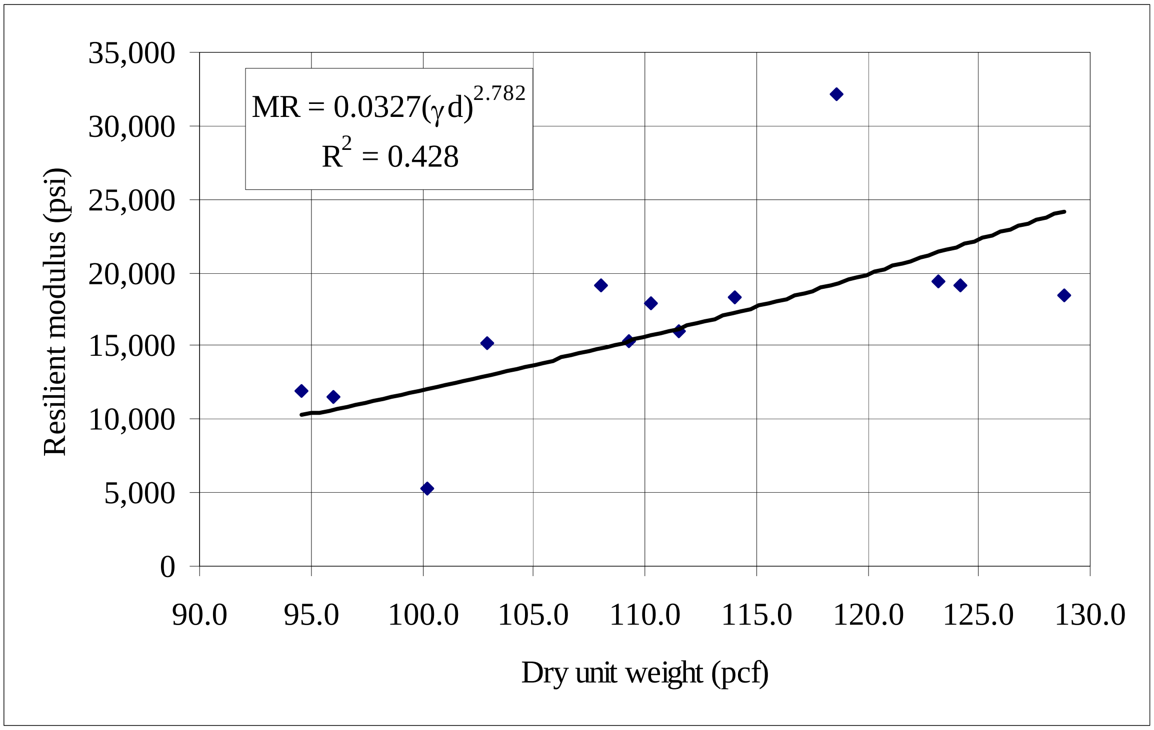


Figure 3.21 Resilient modulus versus dry unit weight of 13 SM soil samples

The above discussion implies that the true effect of the dry unit weight on MR values cannot be separated from the effect of the water content unless the latter is held constant and the former is changed using different compaction effort. In the multivariate analyses presented in the next subsection, the effect of dry unit weight on the MR values were analyzed in conjunction with the effect of water content of the test samples.

Affect of the Sample Degree of Saturation – For each test sample, after the conclusion of the cyclic load test, the sample moisture content and dry unit weight was determined and the degree of saturation (S) was calculated using Equation 3.6. Please note that for all SM soil samples, a typical value of the specific gravity of the solid (G_s) of 2.7 was assumed and used in Equation 3.6.

$$S = \left[\frac{G_s * (MC/100) * \gamma_d}{G_s * \gamma_w - \gamma_d} \right] * 100 \qquad \text{Equation 3.6}$$

Where, S = degree of saturation (%), MC = moisture content (%), G_s = specific gravity of the soil solid = 2.7, γ_d = dry unit weight of the sample (pcf), and γ_w = unit weight of water = 62.4 pcf

Figure 3.22 shows the MR values plotted against the degree of saturation of the test samples. As it was expected and reported by Maher et al. (2000), the MR values decrease significantly with increasing degree of saturation. One may argue that the data in Figure 3.22 is repetitive and are the same as the data in Figure 3.20; hence, Figure 3.22 can be eliminated. In reality, the water content of a soil sample is an independent variable whereas the degree of saturation is a function of the dry unit weight and the water content of the soil. Hence, the data in Figure 3.22 show the combined effects of the dry unit weight and the water content of the test samples on their MR values. The degree of saturation will be included with other sample variables in the multivariate analyses to determine their combined effect on the MR values.

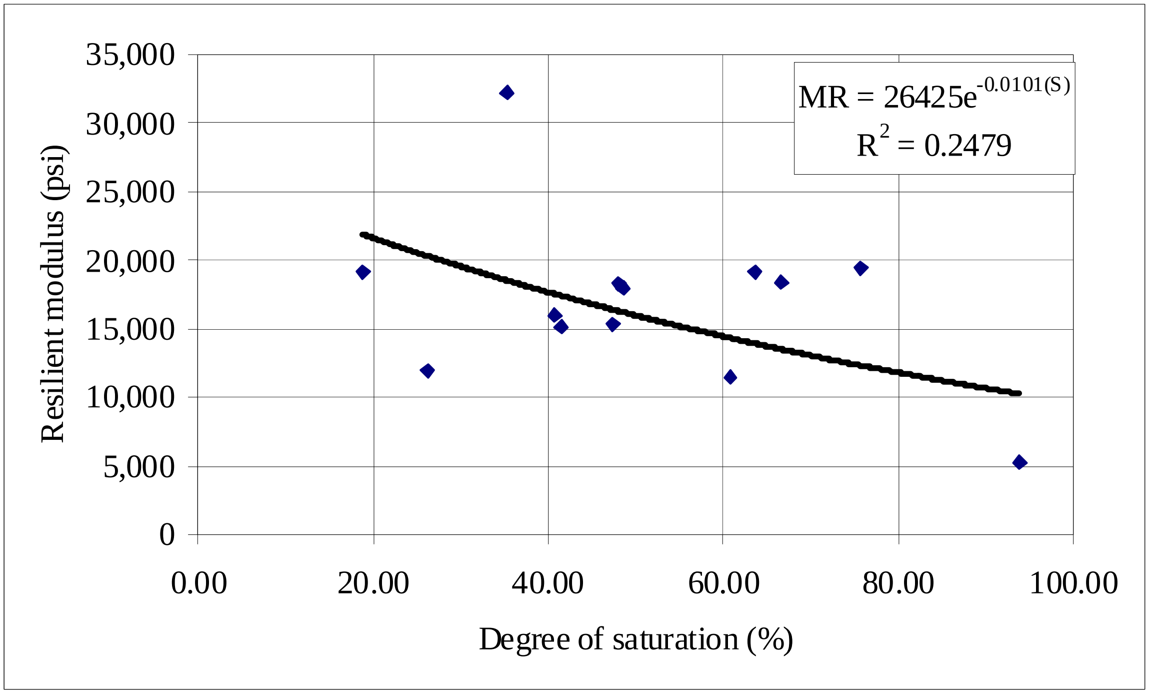


Figure 3.22 Resilient modulus versus the degree of saturation of SM soil samples

Effect of the Sample Liquid Limit - For each SM soil sample, the Atterberg limits for all materials passing sieve number 40 were determined in order to classify the type of fine materials (silt or clay). The liquid limit tests resulted in the classification of 15 soil samples as non-plastic (the plastic limit test failed repeatedly). The plastic limit test was successful for only one soil sample and the plastic limit of the soil was very low. Hence, the effects of the plastic limit and plasticity index on the MR values were not analyzed. However, the effects of the liquid limit on the MR values of the soils were analyzed. Figure 3.23 depicts the influence of the liquid limits on the MR values of SM soils. The data in the figure indicate that the MR values of SM soils having higher liquid limits are lower than those having lower liquid limits. Such observation was expected and has been reported by many researchers for various soil types including silty and clayey sands, silt, and clay (Gudishala 2004). Given the strong correlation between the liquid limit of the material passing sieve number 40 and the soils MR values, the liquid limit data were included in the multivariate analyses presented in the next subsection.

Effect of Sample Grain Size – Because of high fine contents, all SM soils were subjected to wet sieving and hydrometer data analyses to determine their grain size distribution. The effects of sample gradation on MR values were assessed through the following gradation parameters:

- Percent passing sieves 200, 100, 40, 20, 10, 4 and 3/8 inch
- The coefficients of curvature and uniformity (C_C and C_U)
- Average particle size at ten, thirty, and sixty percent passing (D_{10}, D_{30}, and D_{60}, respectively)

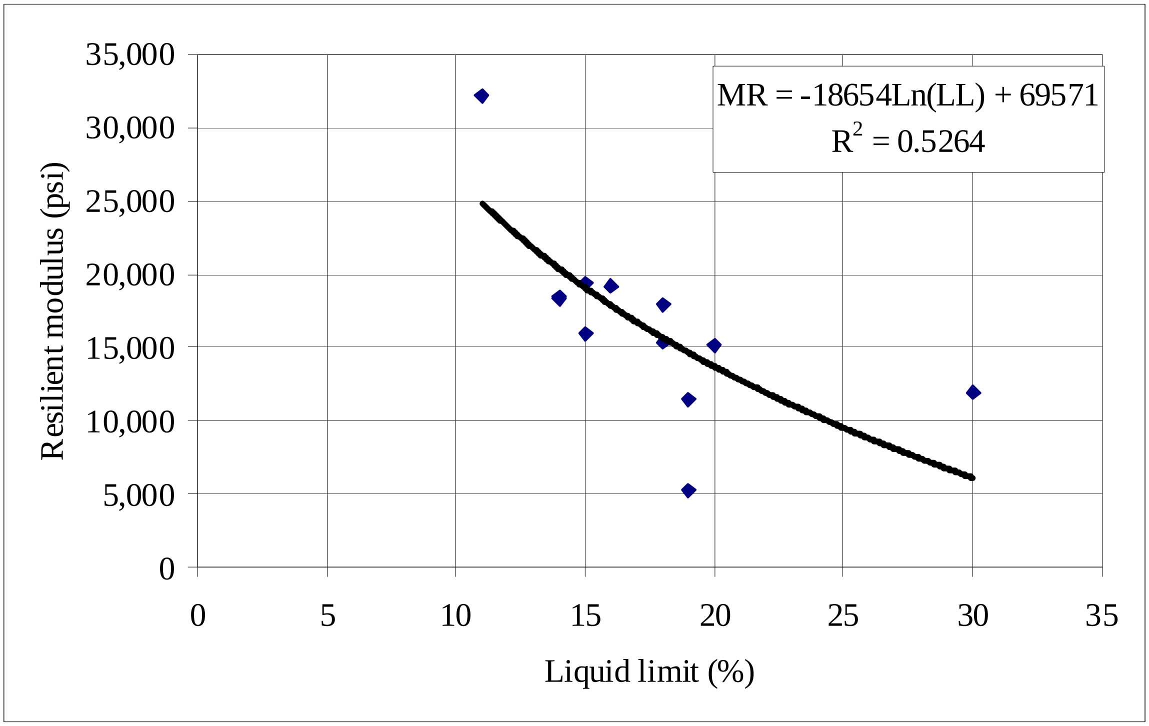


Figure 3.23 Resilient modulus versus the liquid limits of SM soils

The effects of each gradation parameter on the MR values were analyzed by plotting the MR values of the soil samples as a function of that parameter. Only three gradation parameters, the average particle size at 10, 30, and 60 percent passing showed minor correlation to MR values, the others (C_U and C_C) showed no correlation. Figure 3.24 shows the correlation between the average particle size at thirty percent passing and the MR values. Based on these observations, the average particle size at 10, 30, and 60 percent passing were included in the multivariate analyses, which are presented in the next subsection.

Multivariate Analysis - Multivariate analyses were conducted to study the combined effects of several sample variables on the dependent variable MR of SM soils. During the analyses:

- Various models (equation forms) were used in an attempt to maximize the value of the coefficient of determination (R^2).
- Special care was taken to:
  - Ensure that the resulting equation satisfies the previously reported trends between each of the independent variable and the dependent variable MR.
  - Avoid any significant co-linearity between the independent variables.
  - Minimize the number of independent variables in the equation.

Results of the analyses yielded two models having relatively high R^2 values. The first model is based on two sample variables of the SM (SVSM) soils (the dry unit weight and the degree of saturation) as stated in Equation 3.7.

$$MR = 0.0303(SVSM)^{4.1325} \qquad \text{Equation 3.7}$$

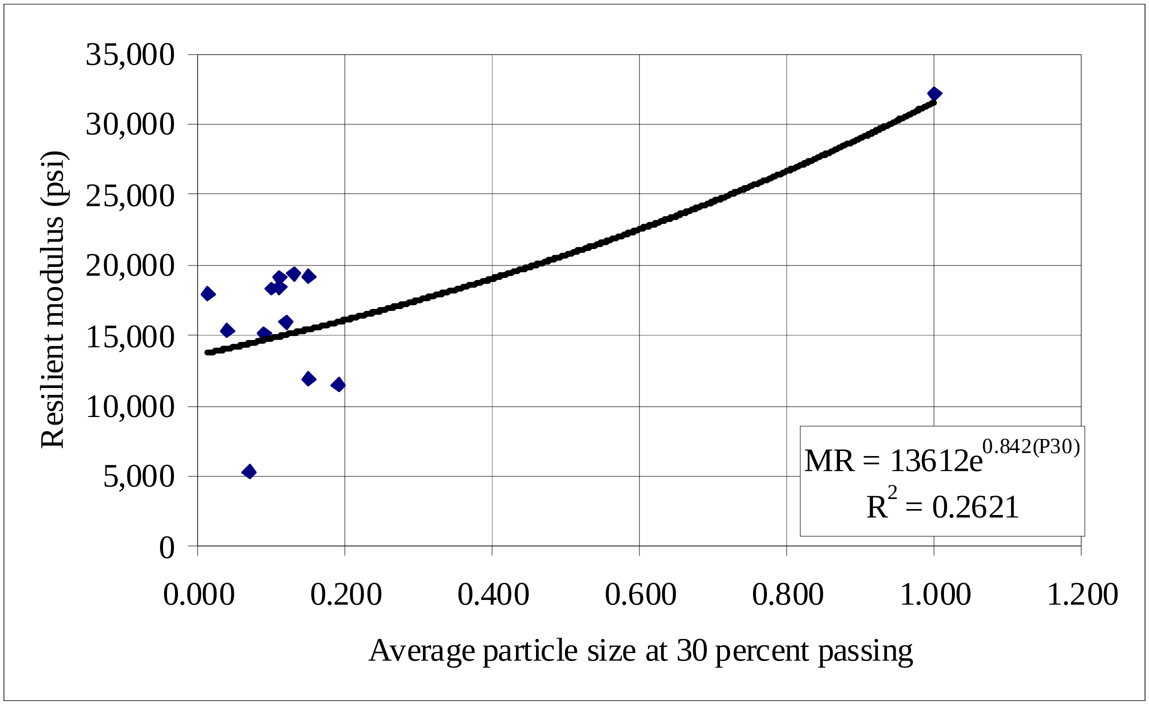


Figure 3.24 Resilient modulus versus the average particle size at thirty percent passing

$$\mathrm{SVSM} = \frac{\gamma_d^{0.8}}{S^{0.15}} \qquad \text{Equation 3.8}$$

Where, γ_d = dry unit weight (pcf) and S = degree of saturation (%)

Although the two variables are some how co-linear (both are a function of the water content of the soil), the interpretation of the model agrees with most literature. To illustrate, consider the data in Figure 3.25, in which the resilient modulus is plotted as a function of the SVSM. It can be seen from the figure and from Equations 3.7 and 3.8 that increases in the dry unit weight cause increases in the SVSM values and hence, increases in the MR values. Further, increases in the degree of saturation cause decreases in the SVSM and the MR values. That is, the MR value can be increased by either increasing the dry unit weight (i.e., higher compaction effort) or by decreasing the degree of saturation or by combination thereof. The reason is that, as the dry unit weight of the sample increases, the relative density increases and the particle to particle contact in the sample increases causing higher internal friction and hence, higher stiffness (Perloff and Baron 1976). On the other hand, decreasing degree of saturation implies decreasing moisture content and decreasing the degree of lubrication between the soil particles. This causes increases in the soil internal friction, soil stiffness, and MR values. Similar results were also reported by Maher et al. (2000).

The two important points that should be noted herein are:

1. The data for the five open symbols in Figure 3.25 are those of the five samples used to verify Equation 3.7. They are discussed later in the subsection.

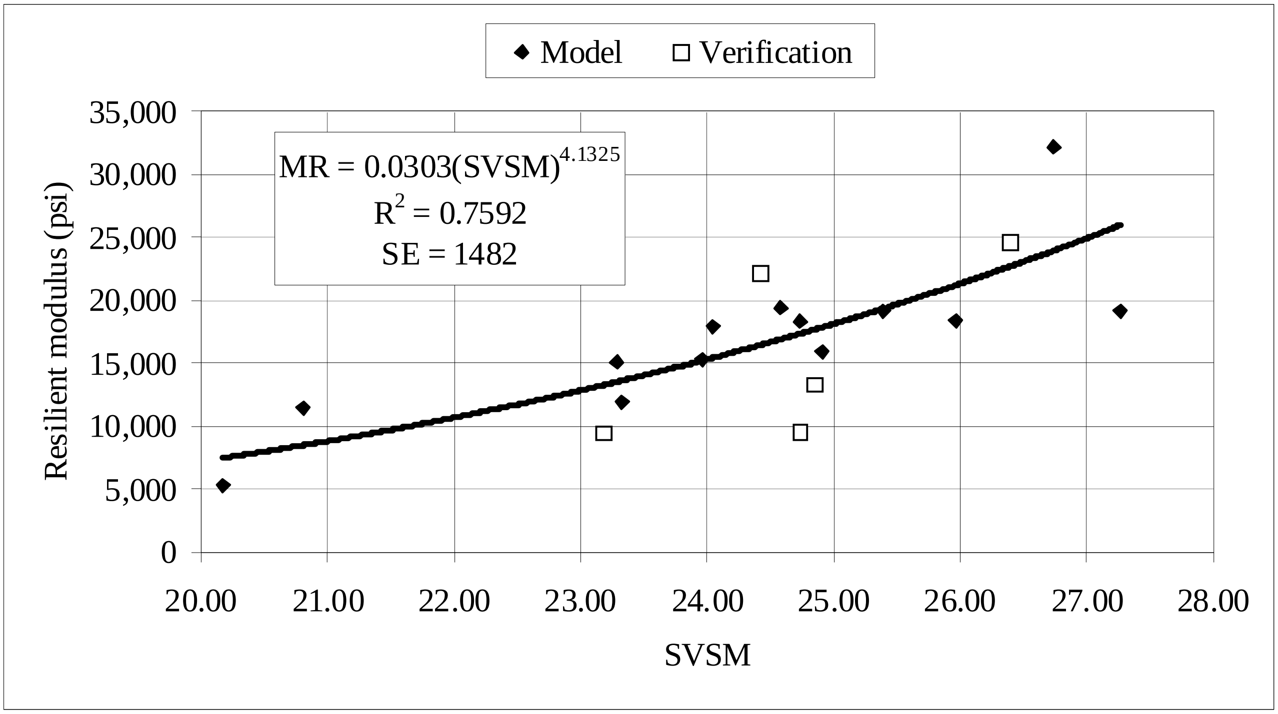


Figure 3.25 Resilient modulus versus the sample variable for SM (SVSM) roadbed soil

2. The test data used in support of Equation 3.7 have the following ranges: degrees of saturation from 18.8 to 93.9 percent and dry unit weight from 94.6 to 128.8 pounds per cubic foot. The use of the equation outside these two ranges is not recommended.

Nevertheless, the multivariate analyses of the SM soils yielded a second model (Equation 3.9) based on two independent variables; the moisture contents (MC) of the test samples and the liquid limits (LL) of the soils passing sieve number 40. In this study, the two independent variables were combined into one parameter, which was named the moisture index (MI) as stated in Equation 3.10.

$$MR = 45722\exp(-0.0258 * MI) \qquad \text{Equation 3.9}$$

$$MI = \text{Moisture Index} = LL^{1.1} + MC^{1.25} \qquad \text{Equation 3.10}$$

Where, LL = liquid limit and MC = moisture content (%)

Figure 3.26 depicts the resilient modulus values of the thirteen SM soils plotted against the moisture index. Inspection of the figure indicates that, in general, higher MI values produce lower MR values. That is, increasing either the moisture content or the liquid limit of the soils causes increases in the MI values and hence, decreases in the MR values of the soils. These observations were expected because higher MI values due to higher moisture contents cause softening of the SM soils and hence lower resilient modulus values. Likewise, soils having higher liquid limits tend to be more plastic and have higher softening potential due to changes in their water contents.

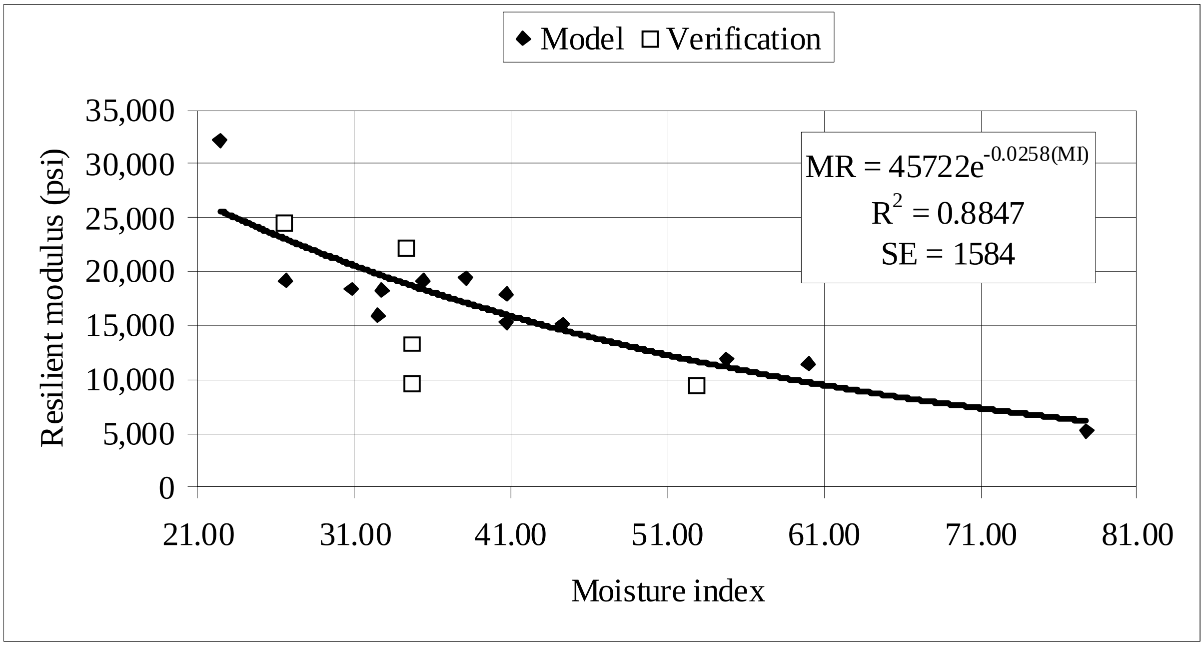


Figure 3.26 Resilient modulus versus the moisture index of the test sample

Equation 3.9 allows the user to estimate the MR values of SM roadbed soils using the results of two simple tests: the natural water content of the entire soil and the liquid limit of the particles passing sieve number 40. It should be noted that, in the field, the natural moisture content of a soil varies from one season to another whereas its liquid limit is constant. Hence, if the LL of a given roadbed soil is known, one needs only to determine the moisture content of the soil before using Equation 3.9. It is important to note that the data used to generate Equation 3.9 include a range in moisture contents from 3.9 to 23.7 percent and a range in liquid limits from 11 to 30 percent. The use of Equation 3.9 outside the two ranges is not recommended. The decision on which equation to use will be based on the type of available data. If the dry density and the degree of saturation data are available; Equation 3.7 can be used. On the other hand, Equation 3.9 can be used if the liquid limit and moisture content data are available. Both equations would yield similar results.

Verification - The five shaded SM soil samples in Table 3.8 were not included in the development of the two SM soil models (Equations 3.7 and 3.9). The samples were subjected to wet sieve analysis, Atterberg limit tests, and cyclic load triaxial tests to determine the physical parameters and the MR values of the soils. After the laboratory tests were completed, the data for the five SM soils were plotted in Figures 3.25 and 3.26 as open symbols. It can be seen that the open symbols in both figures are located in the vicinity of the best fit curves.

Finally, Equations 3.7 and 3.9 were used to predict the resilient modulus values of the five SM soils based on their parameters (dry unit weight and degree of saturation and moisture content and the liquid limit of the soils). The predicted and the laboratory measured resilient modulus values are plotted in Figures 3.27 and 3.28. As can be seen from the figures, the data for both equations are located close to the line of equality. Based on this observation, one may conclude that the two models presented in Equations 3.7 and 3.9 can be used to estimate the resilient modulus values of SM soils based on the soil parameters. Such parameters can be obtained from simple tests; water content, liquid limit, and dry unit weight.

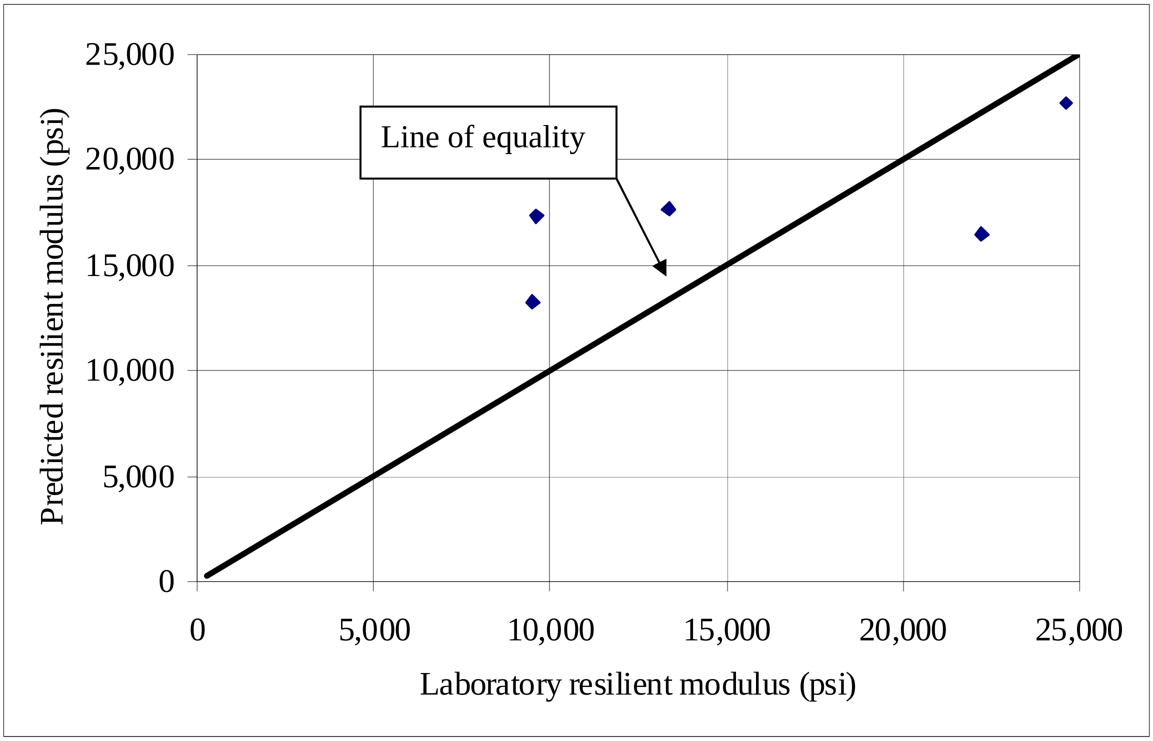


Figure 3.27 Predicted versus laboratory measured resilient modulus values, Equation 3.7

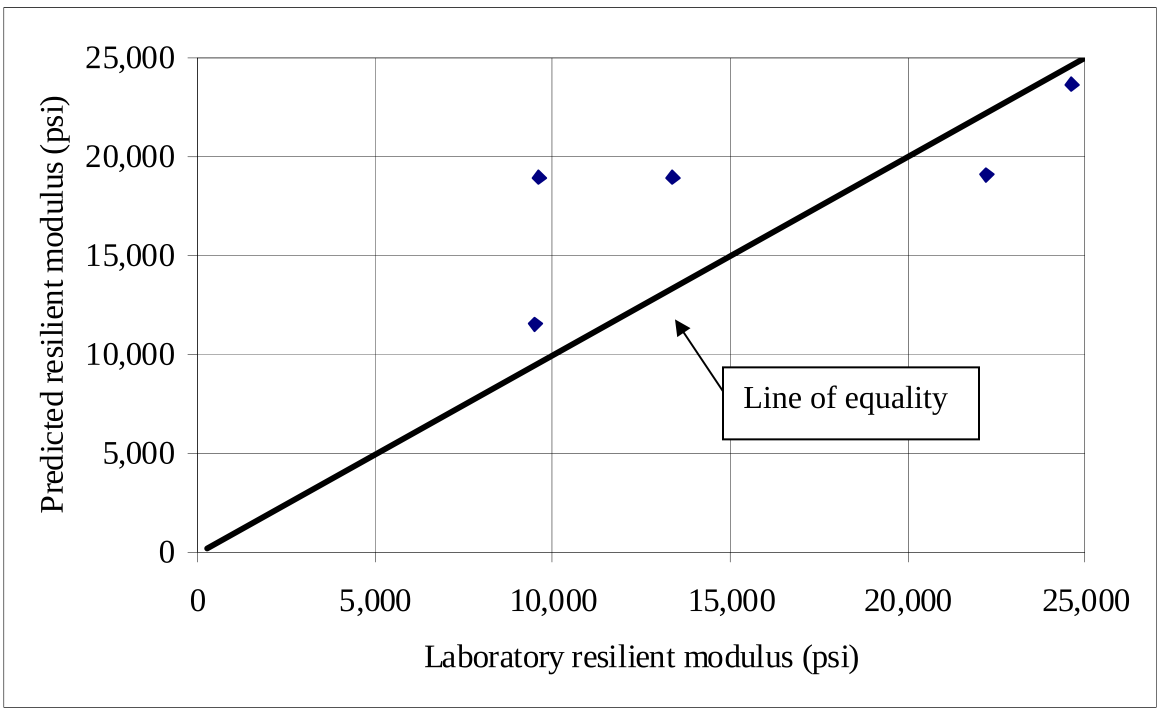


Figure 3.28 Predicted versus laboratory measured resilient modulus values, Equation 3.9

3.6.2.3 Clayey Sand (SC), Low Plasticity Clay (CL), and Low Plasticity Silt (ML)

Table 3.9 lists the locations, the USCS and the AASHTO soil classifications, Atterberg limits, and the sample designation number of 17, eleven disturbed and six Shelby tube (marked with S in the table), soil samples that were collected from various clusters and areas throughout the State of Michigan. According to the USCS, nine samples consist of clayey sand (SC), six clay (CL), and two low plasticity silt (ML) samples. According to the USCS, SC soils may contain anywhere between 12 and 49.9 percent by weight fine materials and the plasticity index and liquid limit of the material passing sieve number 40 plot above the A-line on the plasticity chart. Clay (CL) soils contain more than 50 percent by weight passing sieve number 200 and the plasticity index and liquid limit of the soil plot above the A-line on the plasticity chart. Finally, the ML soils contain more than 50 percent by weight passing sieve number 200 and the plasticity index and liquid limit data plot below the A-line on the plasticity chart (Holtz and Kovacs 1981). It should be noted that more than 17 rows are presented in Table 3.9, this is due to the fact that some samples were tested more than once in order to study the effects of moisture content and to verify the model presented later in the section (shown by gray highlight).

All soil samples listed in Table 3.9 were subjected to wet sieve and hydrometer data analyses, Atterberg limit tests, and cyclic load tests. The reason that the three types of soils are housed in Table 3.9 is that, after the completion of the cyclic load tests, the resilient modulus values of the samples were plotted against the samples moisture contents. The three soil types showed the same type relationship between the MR values and the sample moisture content.

A total of 16 cyclic load tests were conducted on SC soils, 11 tests on undisturbed soil samples and 5 tests on disturbed soil samples. Nine cyclic load tests were conducted on CL soils, five tests on undisturbed soil samples and four on disturbed samples. Finally, four disturbed ML soil samples were subjected to cyclic load triaxial tests.

In step 2 of the analyses, univariate and multivariate analyses were conducted simultaneously on the three soil types to study the effects of the sample variables on their MR values. Results of the analyses are presented and discussed below.

Univariate Analyses - In the univariate analyses, the effects of each of several sample variables on the MR values of SC, CL, and ML soils were studied. These sample variables include: the moisture content of the samples, the degrees of saturation, the dry unit weight after compaction, and the grain sizes. Results of the analyses are presented and discussed below.

Effect of Sample Moisture Contents – As is the case for the SM soils, because of the high fine contents of SC, CL, and ML soils, it was hypothesized that the water content of the samples would play a major role in determining the elastic response of the soil to the applied loads. Figure 3.29 shows the MR values plotted against the samples moisture contents. As it was hypothesized and expected, the figure shows increases in the sample moisture content cause significant decreases in the MR values. Similar results were also reported by many researchers including (Maher et. al 2000, George 2000, and 2003). Further, the data in Figure 3.29 also show that the three soil types have similar, if not the same, relationship between the sample moisture content and the MR values. Hence, the moisture content or the degree of saturation will be considered in the multivariate analyses.

Table 3.9 Location of SC, CL, and ML roadbed soils

Sample designation number	Shelby tube samples	Location	Atterberg limits		AASHTO	USCS
			Liquid limit	Plastic limit		
M-072-W (05-06)		330 feet West of M-32	22	11	A-6	SC
U-127-N (07-05)	S	65 feet North of Vernon Rd	24	14	A-6	SC
U-127-N (07-05)	S	65 feet North of Vernon Rd	24	14	A-6	SC
U-127-N (07-05)	S	65 feet North of Vernon Rd	24	14	A-6	SC
U-127-N (07-05)	S	65 feet North of Vernon Rd	24	14	A-6	SC
U-127-N (07-05)		65 feet North of Vernon Rd	24	14	A-6	SC
U-010-W (08-04)	S	145 feet West of Mackinaw Rd	29	13	A-6	SC
U-010-W (08-04)	S	145 feet West of Mackinaw Rd	29	13	A-6	SC
I-096-W (10-03)		210 feet West of bridge before exit 97	29	14	A-2-6	SC
I-075-S (14-01)	S	60 feet South of Gaynier Rd	45	19	A-7-6	SC
I-075-S (14-01)	S	60 feet South of Gaynier Rd	45	19	A-7-6	SC
I-075-S (14-01)	S	60 feet South of Gaynier Rd	45	19	A-7-6	SC
I-075-S (14-01)		60 feet South of Gaynier Rd	45	19	A-7-6	SC
M-153-E (14-06)	S	~800 feet East of Greenfield Rd	51	19	A-7-6	SC
M-153-E (14-06)	S	~800 feet East of Greenfield Rd	51	19	A-7-6	SC
M-153-E (14-06)		~800 feet East of Greenfield Rd	52	20	A-7-6	SC
M-045-S (01-01)		405 feet South of Ontonagon River	26	16	A-6	CL
M-010-E (13-08)		Station 38+00	24	14	A-6	CL
M-010-E (13-08)	S	Station 38+00	24	14	A-6	CL
M-010-E (13-08)	S	Station 38+00	24	14	A-6	CL
M-010-E (13-08)	S	Station 38+00	24	14	A-6	CL
I-094-W (14-09)	S	350 feet West of Wadhams Rd	44	21	A-7-6	CL
I-094-W (14-09)	S	350 feet West of Wadhams Rd	44	21	A-7-6	CL
I-094-W (14-09)		350 feet West of Wadhams Rd	44	21	A-7-6	CL
M-090-E (15-04)		200 feet East of Bobcock St 37 feet East of Village Limit sign	24	15	A-4	CL
M-028-W (02-02)		1053 feet West of M-141	23	NA	A-4	ML
M-028-W (02-02)		1053 feet West of M-141	23	NA	A-4	ML
M-028-W (02-02)		1053 feet West of M-141	23	NA	A-4	ML
U-002-E (02-04)		765 feet East of Spalding Rd	19	NA	A-4	ML

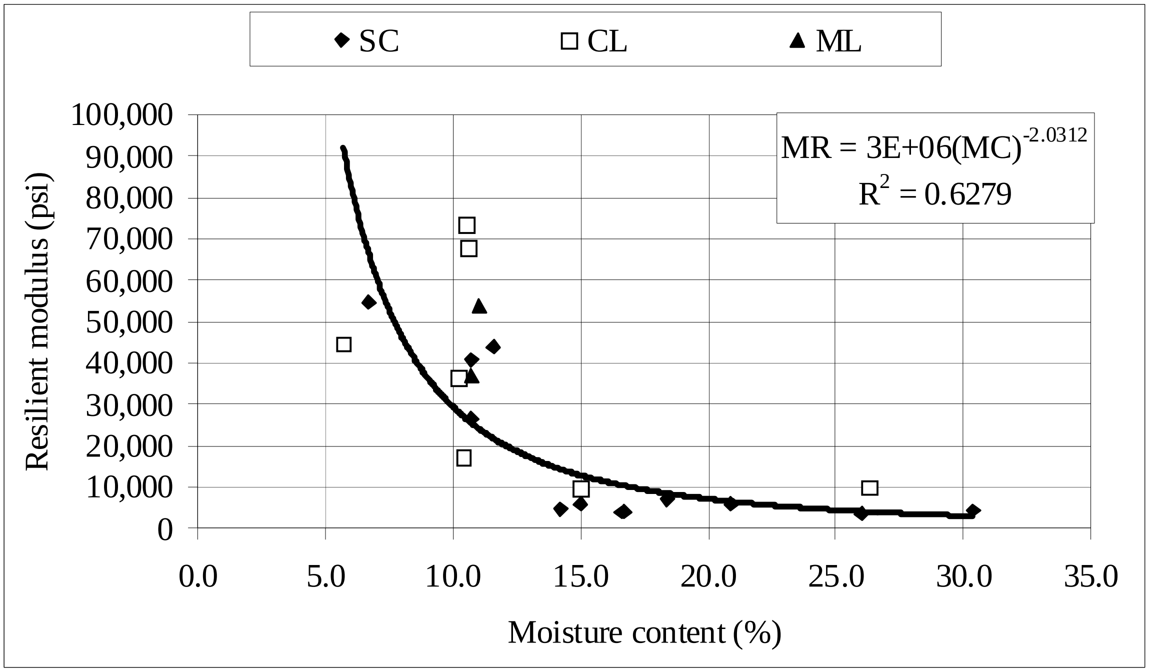


Figure 3.29 Resilient modulus versus the moisture contents of the samples

Effect of the Degree of Saturation – At the conclusion of the cyclic load test, the sample moisture content and dry unit weight were determined and the degree of saturation (S) was calculated using Equation 3.6 (which is repeated below for convenience). It should be noted that for all test samples, a typical value of the specific gravity of the solid (G_s) of 2.7 was assumed and used in Equation 3.6.

$$S = \left[\frac{G_s * (MC/100) * \gamma_d}{G_s * \gamma_w - \gamma_d} \right] * 100 \qquad \text{Equation 3.6}$$

Where, S = degree of saturation (%), MC = moisture content (%), G_s = specific gravity of the soil solid, γ_d = dry unit weight of the sample (pcf), and γ_w = unit weight of water = 62.4 pcf

Figure 3.30 shows the MR values plotted against the degree of saturation of the test samples. As it was expected and reported by Maher et al. (2000), the MR values decrease with increasing degree of saturation. The difference between this observation and the previous one regarding the sample water content is that the degree of saturation is a function of both the water content of the sample and its dry unit weight. Said functionality caused the coefficient of determination to increase from about 0.63 in Figure 3.29 to about 0.88 in Figure 3.30. Therefore, the degree of saturation will be included in the multivariate analyses.

Effect of Sample Dry Unit Weight - The effect of the test sample dry unit weight on the MR values of SC, CL, and ML soils was studied by plotting the MR values as a function of the sample dry unit weight as shown in Figure 3.31. As it was expected, the figure shows a very weak correlation between the dry unit weight and the MR values of the test sample. The main reason for the weak correlation is that the water contents of the test samples vary from about 6.7

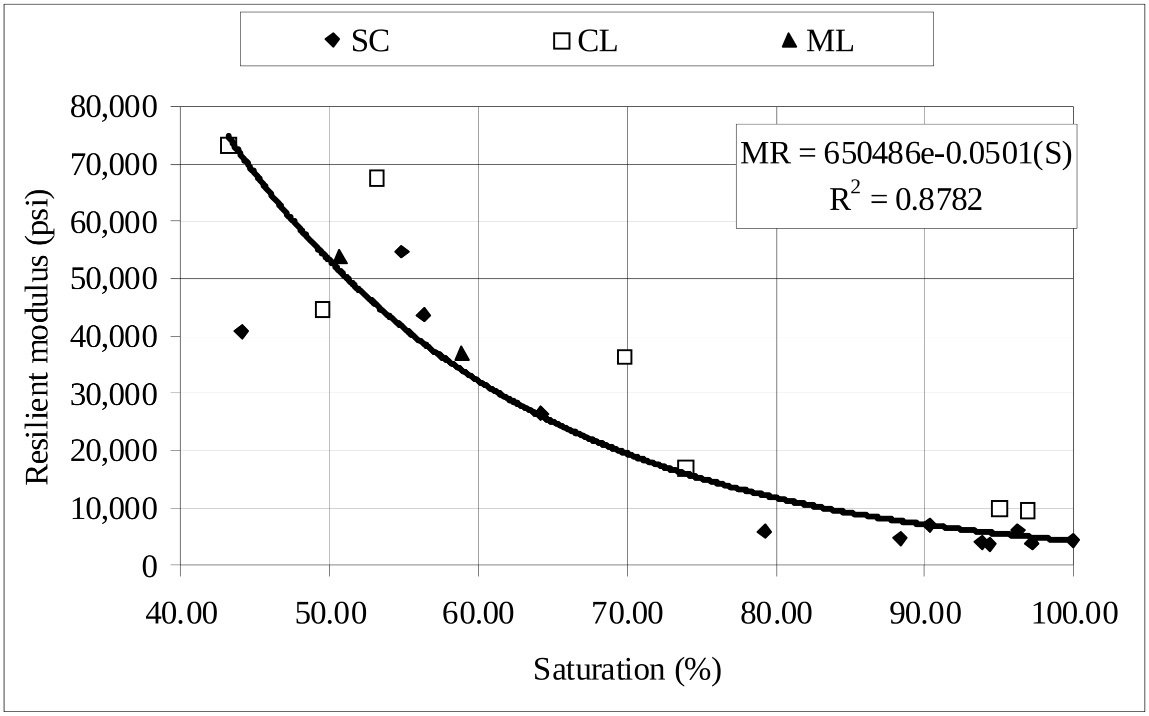


Figure 3.30 Resilient modulus versus degree of saturation for SC, CL, and ML soils

to 30.4 percent. Relative to the compaction curve, this variation causes some test samples to be on the wet side while others on the dry side of optimum. This implies that two compacted samples having the same dry unit weight may or may not have the same water content. Soil samples compacted dry of optimum would have brittle behavior, higher strength, and higher MR values than samples compacted wet of optimum. The latter would have lower strength, higher plastic deformation, and softer behavior under loads. In addition, when soil samples are compacted on the dry side of optimum, the soil particles tend to stay in a flocculated arrangement. Whereas, for samples compacted wet of optimum; the particles tend to disperse (line up) due to the extra water lubrication (Holtz and Kovacs 1981). Nevertheless, the dry unit weight will be included in the multivariate analyses to determine if the dry unit weight interacts with other variables to significantly affect the MR values.

Effect of Sample Grain Size – Because of high fine contents, all SC, CL, and ML soils were subjected to wet sieving and hydrometer data analyses to determine their grain size distributions. The effects of sample gradation on MR values were assessed through the following gradation parameters:

- Percent passing sieves 200, 100, 40, 20, 10, 4, and 3/8 inch
- The coefficients of curvature and uniformity (C_C and C_U)
- Average particle size at ten, thirty, and sixty percent passing (D_{10}, D_{30}, and D_{60}, respectively)

The effects of each gradation parameter on the MR values were analyzed by plotting the MR values of the soil samples as a function of that parameter. However, as it was expected, none of the variables showed a good correlation to the MR values.

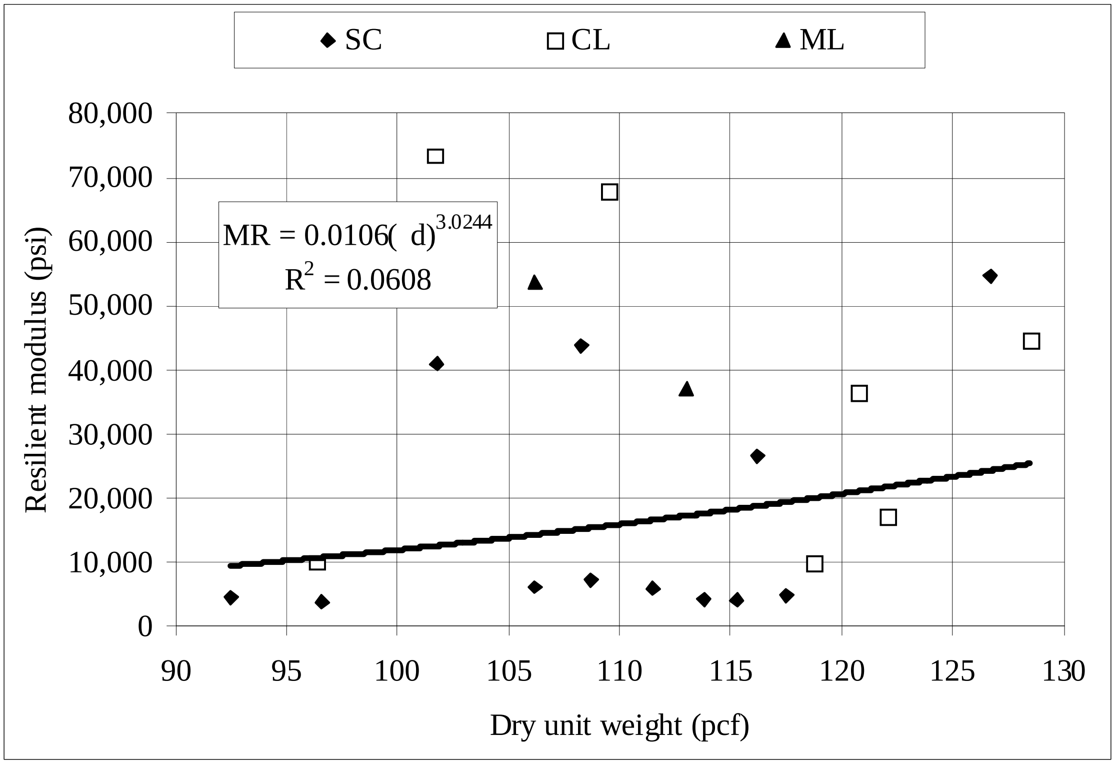


Figure 3.31 Resilient modulus versus dry unit weight for SC, CL, and ML soils

Multivariate Analysis - Multivariate analyses were conducted to study the combined effects of several independent sample variables on the dependent variable MR values of SC, CL, and ML soils. During the analyses:

- Various models (equation forms) were used in an attempt to maximize the value of the coefficient of determination (R^2).
- Special care was taken to:
  - Ensure that the resulting equation satisfies the previously reported trends between each of the independent variable and the dependent variable MR values.
  - Avoid any significant co-linearity between the independent variables.
  - Minimize the number of independent variables in the equation.

Results of the analyses yielded models having relatively high R^2 values. However, none of the models produced a better correlation than the degree of saturation (S) alone. Therefore, it is recommended that the MR values of SC, CL, and ML soils be predicted using the degree of saturation (S) alone through Equation 3.11.

$$MR = 650486 \exp - 0.0501(S) \qquad \text{Equation 3.11}$$

Where, S = degree of saturation (%)

The data used to develop Equation 3.11 have saturation values ranging from 43.2 to 99.9 percent, with a corresponding range of dry unit weight from 92.5 to 128.5 pcf and moisture content range from 5.7 to 30.4 percent. The use of Equation 3.11 outside those ranges of values is not recommended.

Figure 3.32 shows the MR values plotted against the degree of saturation. The figure consists of two sets of data. The first set (solid symbols) is the data that was used to develop the model (Equation 3.11) with R^2 value of about 0.88. The second set (open symbols) is the data that was used to verify the developed model. The latter data were obtained from testing five disturbed and three undisturbed soil samples. The sequential procedure used to test the samples is presented in the next subsection.

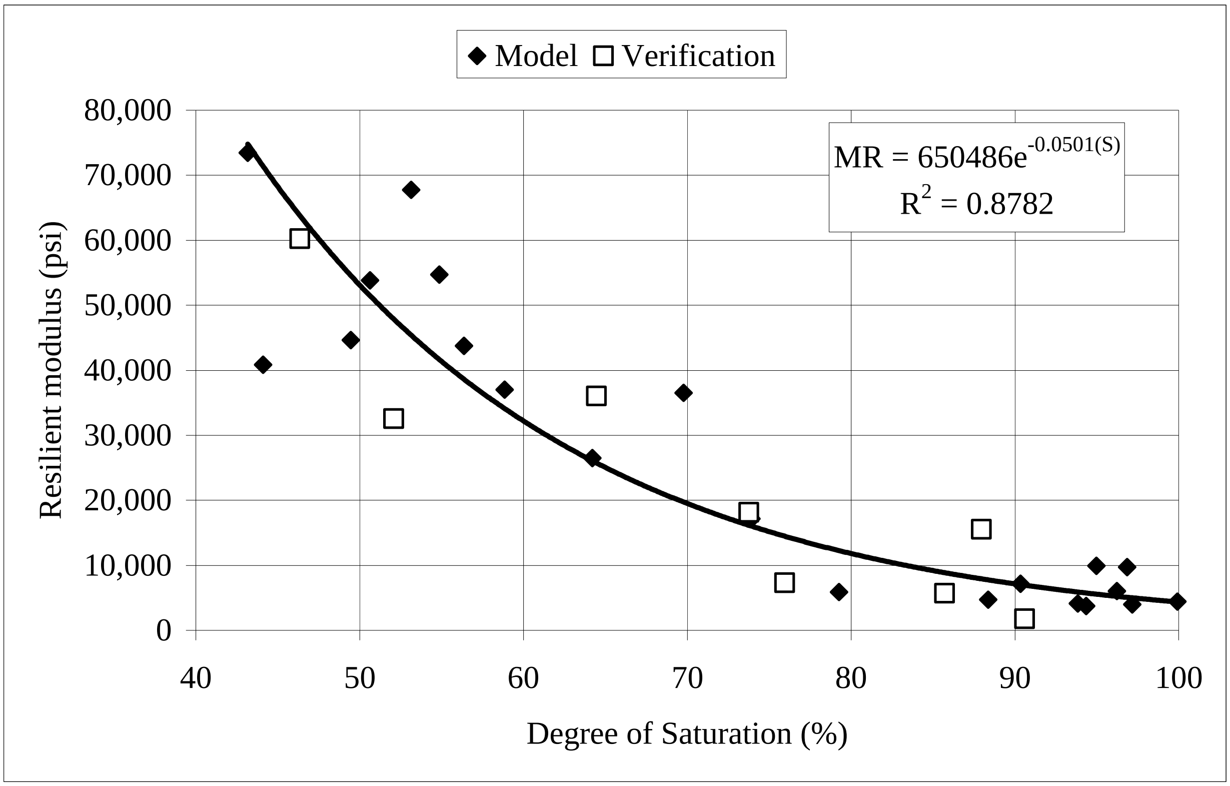


Figure 3.32 Resilient modulus versus degree of saturation

One important point should be noted herein is that the data used to generate Equation 3.11 were obtained from both disturbed and undisturbed samples. Hence, the equation applies equally to both types of samples. The implication of this is that, for the same degree of saturation, the elastic behavior of laboratory compacted samples is similar to that of undisturbed samples.

Verification - As stated above, eight additional soil samples (five disturbed samples and three undisturbed soil samples) were tested to verify the model presented in Equation 3.11. Three of the disturbed soil samples were allowed to dry from their natural water contents of 14.4, 25.4, and 21.9 percent to 10.3, 11.3 and 18.8 percent, respectively. After drying, the soils were compacted using standard proctor and standard compaction mold. The compacted soil was then extracted from the compaction mold, trimmed to the size of the test sample, and subjected to cyclic load triaxial tests. When the tests were terminated, the water content and the dry unit weight of each sample were then measured and their respective degrees of saturation were calculated. The natural water contents of the three undisturbed Shelby tube samples that were tested for verification purposes were 12.3, 18.4 and 11.2 percent. One test sample was extracted from each of the three Shelby tubes. Two test samples were subjected to cyclic load triaxial tests at their natural water contents of 12.3 and 11.2 percent. The third sample was dried at room temperature for two days, then it was sealed in a plastic bag for one week to make the moisture in the sample consistent, and then it was tested. After the cyclic load triaxial tests were terminated, the moisture content of each sample and its dry unit weight were determined.

For the eight verification samples, results of the cyclic load triaxial test data were used to calculate the resilient modulus of the soil. The data are shown in Figure 3.32 by the open symbols. After measuring the test sample water content and dry unit weight, the data were used in Equation 3.11 to estimate the resilient modulus values of the soils. Figure 3.33 shows the laboratory measured resilient modulus values plotted against the MR values predicted using Equation 3.11 and the degree of saturation of the test samples. It can be seen from Figure 3.33 that all eight data points are located in the vicinity of the line of equality. Hence, the eight data points verify the accuracy of Equation 3.11.

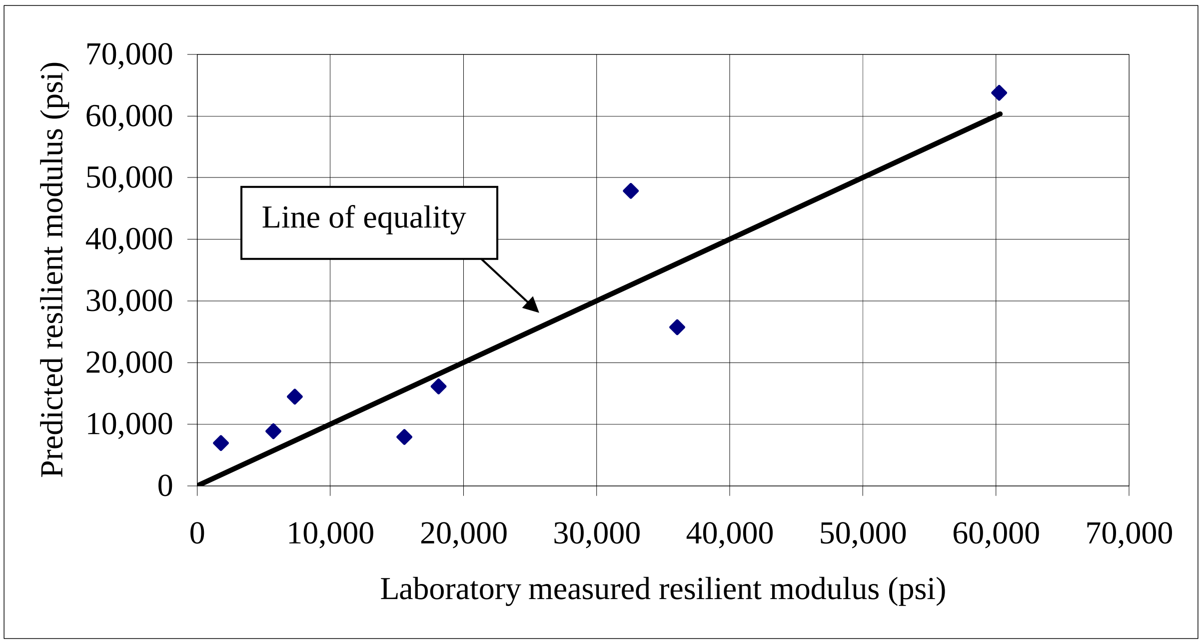


Figure 3.33 Predicted versus laboratory measured MR values for SC, CL, and ML soils

3.6.2.4 Poorly Graded Sand – Silty Sand (SP-SM)

Table 3.10 lists the locations, the USCS and the AASHTO soil classifications, and the sample designation number of eight disturbed soil samples that were collected from various clusters and areas throughout the State of Michigan. The commonality between the eight samples is that all of them were classified in step 1 of the analyses as SP-SM according to the USCS. SP-SM soils may contain anywhere between 5 and 12 percent passing sieve number 200 materials and a plasticity index less than 4 (Holtz and Kovacs 1981). In step 2 of the analyses, univariate and multivariate analyses were conducted and are discussed below.

Table 3.10 Locations of SP-SM roadbed soils

Sample number	Location	AASHTO	USCS
M-028-W (02-03)	~2000 feet East of M-35	A-3	SP-SM
U-002-E (03-01)	400 feet East of Hwy 13	A-3	SP-SM
M-028-W (03-03)	500 feet West of Basnau Rd	A-2-4	SP-SM
I-196-N (06-05)	110 feet North of Schmuhl Rd	A-3	SP-SM
I-069-N (10-01)	75 feet North of Base Line Hwy	A-1-b	SP-SM
I-069-N (11-01)	160 feet North of mile marker 42	A-3	SP-SM
I-094-W (12-03)	36 feet West of bridge after exit 135	A-3	SP-SM
U-023-N (13-07)	60 feet North of Sherman	A-3	SP-SM

Univariate Analysis - In the univariate analyses, the effects of each of several sample variables on the MR values of SP-SM soils were studied. These sample variables include: the moisture contents of the samples, the dry unit weight after compaction, and the grain sizes. Discussion of the effects of each variable is presented below.

Effect of the Sample Moisture Content – Because of the narrow range in fine content, it is expected that the water content would have minimal effects on the elastic response of the soil to the applied loads. Figure 3.34 shows the MR values of all SP-SM soil samples plotted against the samples moisture contents. The figure shows that the sample moisture content has no effect on the MR values. This result was not expected and it contradicts findings by many researchers including (Maher et. al 2000, George 2000, and 2003) who stated that the MR value decreases with increasing moisture content. One possible explanation of the above result is that the effects of moisture content on the MR values interact with other variables that are not included in the equation. This issue is addressed in the multivariate analyses subsection.

Effect of Sample Dry Unit Weight - The effect of the test sample dry unit weight on the MR values of SP-SM soils was studied by plotting the MR values as a function of the sample dry unit weight, as shown in Figure 3.35. As it was expected, the figure shows a weak correlation between the dry unit weight and the MR values of the test samples.

The main reason for the weak correlation is that it is possible for two test samples to have the same dry unit weight but significantly different elastic behavior under load. This scenario is certain if one sample was compacted dry of optimum and the second wet of optimum. A sample compacted dry of optimum has higher strength and stiffness and displays more brittle behavior than the one compacted wet of optimum. The latter would have lower strength, higher plastic deformation, and softer behavior under loads (Holtz and Kovacs 1981). For the SP-SM soils, the water contents of the test samples varied from about 2 to about 11 percent. Such range in the water content extends from the dry side to the wet side of the optimum moisture content on the compaction curve. To overcome the problem, the dry unit weight was included in the multivariate analyses to determine whether or not it interacts with other variables to significantly affect the MR values.

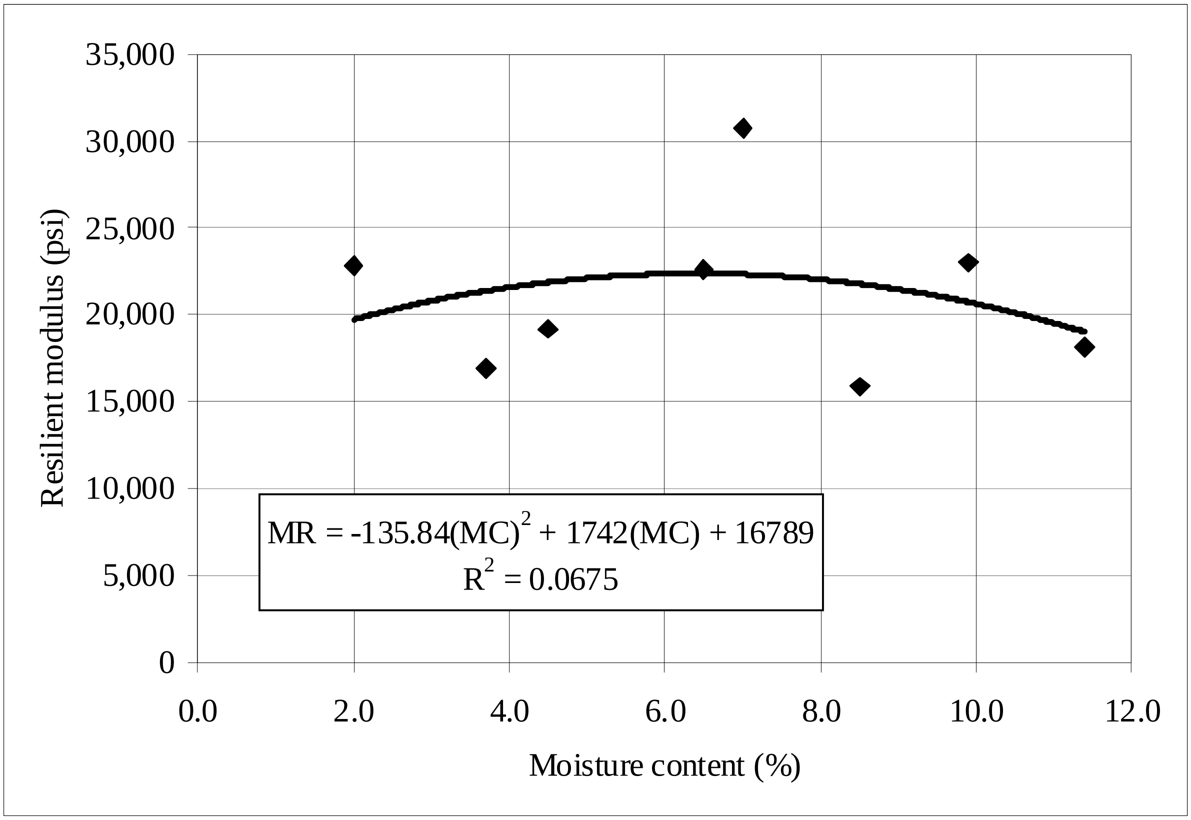


Figure 3.34 Resilient modulus versus moisture content for SP-SM soils

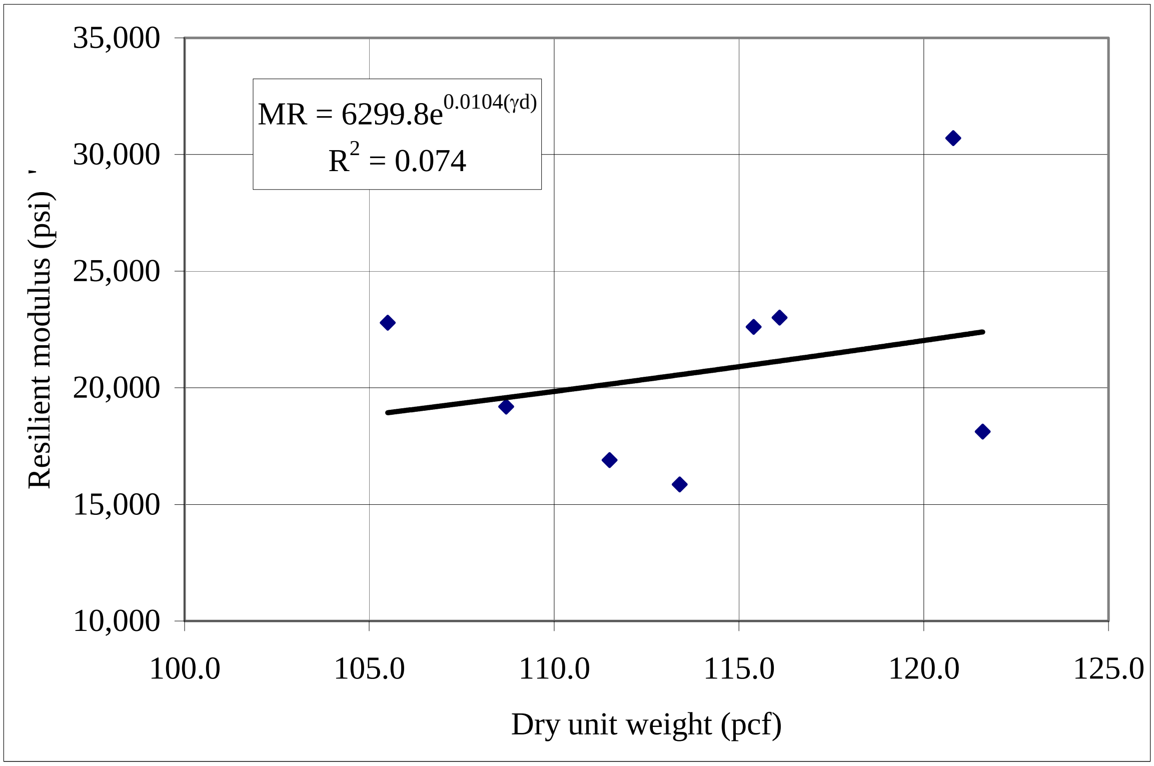


Figure 3.35 Resilient modulus versus the dry unit weight of the test samples

Effect of Sample Grain Size – Because of low fine contents of SP-SM soils, all soil samples were subjected to dry sieving to determine the grain size distribution shown in Figure 3.36. Examination of the figure indicates that the SP-SM roadbed soil samples contain variable amounts of fine and coarse sands. Since the fine and coarse sand contents are co-linear or dependent, both variables should not be included in the analyses. Hence, the effect of either the fine or coarse sand contents on the MR values should be included in the analyses. The effects of sample gradation on MR values were also assessed through the following gradation parameters:

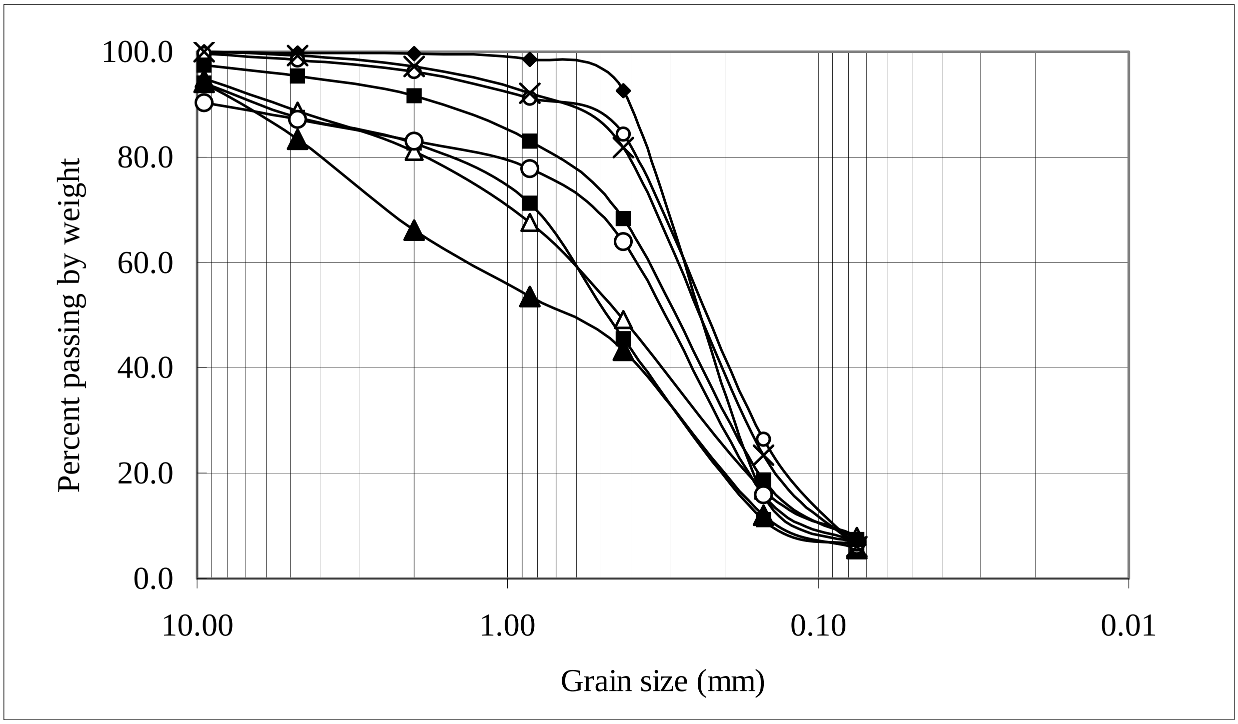


Figure 3.36 Eight gradation curves of the SP-SM roadbed soil samples

- Percent passing sieves 200, 100, 40, 20, 10, 4 and 3/8 inch
- The coefficients of curvature and uniformity (C_C and C_U)
- Average particle size at ten, thirty, and sixty percent passing (D_{10}, D_{30}, and D_{60}, respectively)

The effects of each gradation parameter on the MR values were analyzed by plotting the MR values of the soil samples as a function of that parameter. Few gradation parameters, the percent passing sieve number 200, the percent fine sand content, and the coefficients of uniformity and curvature, showed poor correlation to MR values. For example, the effect of the percent passing sieve number 200 and the MR values is shown in Figure 3.37. Although the data in the figure shows that the percent passing sieve number 200 has insignificant effects on the MR values, it also shows that increasing the percent fine materials (passing sieve number 200) causes decreases in the MR values.

Multivariate Analysis - Multivariate analyses were conducted to study the combined effects of several independent sample variables on the dependent variable MR values of SP-SM soils. During the analyses:

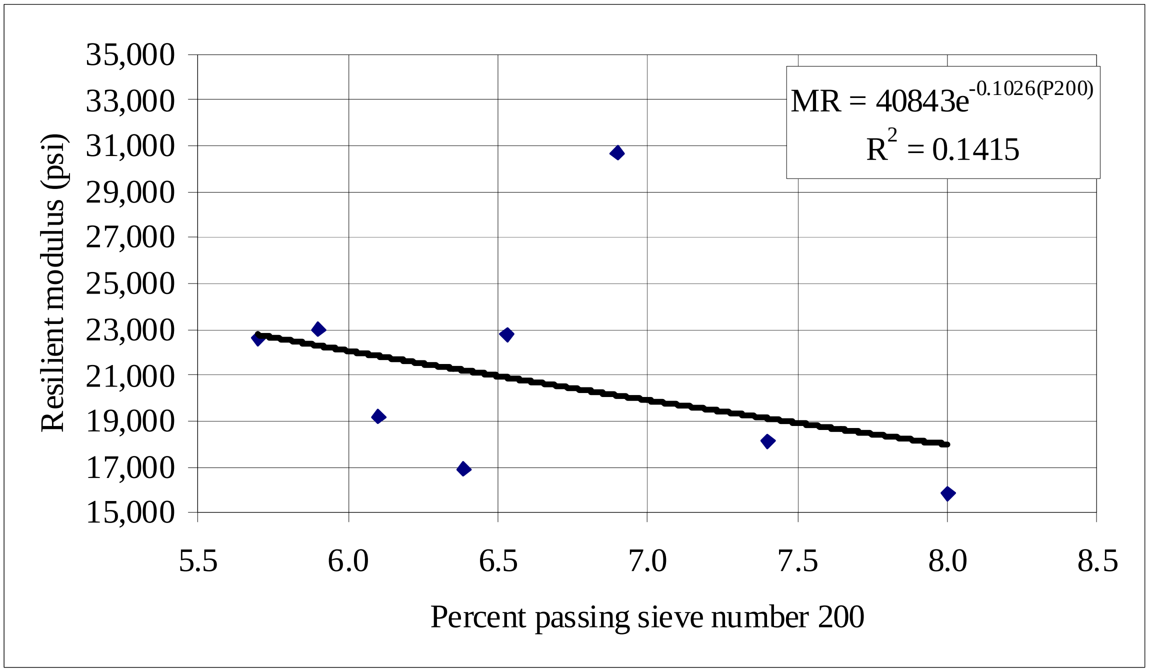


Figure 3.37 Resilient modulus versus percent passing sieve number 200 for SP-SM soils

- Various models (equation forms) were used in an attempt to maximize the value of the coefficient of determination (R^2).
- Special care was taken to:
  - Ensure that the resulting equation satisfies the previously reported trends between each of the independent variable and the dependent variable MR values.
  - Avoid any significant co-linearity between the independent variables.
  - Minimize the number of independent variables in the equation.

Results of the analyses yielded several different models with coefficient of determination values from 0.62 to about 0.93. For example, the model shown in Figure 3.38 and Equation 3.12 has a coefficient of determination of slightly higher than 0.93.

$$\mathrm{MR} = 1357.5(SVSP - SM)^2 - 6145.3(SVSP - SM) + 23613 \qquad \text{Equation 3.12}$$

$$SVSP - SM = \frac{\gamma^{3.95}}{10^7\left(D_{30}^{2.66}\right)\left(D_{10}^{2.25}\right)\left(D_{60}^{0.41}\right)\left(S^{0.03}\right)} \qquad \text{Equation 3.13}$$

Where, γ_d = dry unit weight of the test sample (pcf), D_{30}, D_{10}, D_{60} = the particle diameter at 30, 10 and 60 percent passing (mm), and S = degree of saturation (%)

The model was not accepted although the value of the coefficient of determination is relatively high. Three reasons can be cited for rejection; first the model has too many variables for a sample size of 8, second, the model interpretation (higher particle size yield lower modulus) cannot be physically supported, and third, the three particle sizes in the denominator are not truly independent variables, which makes the model very sensitive to small changes in the values of the variables.

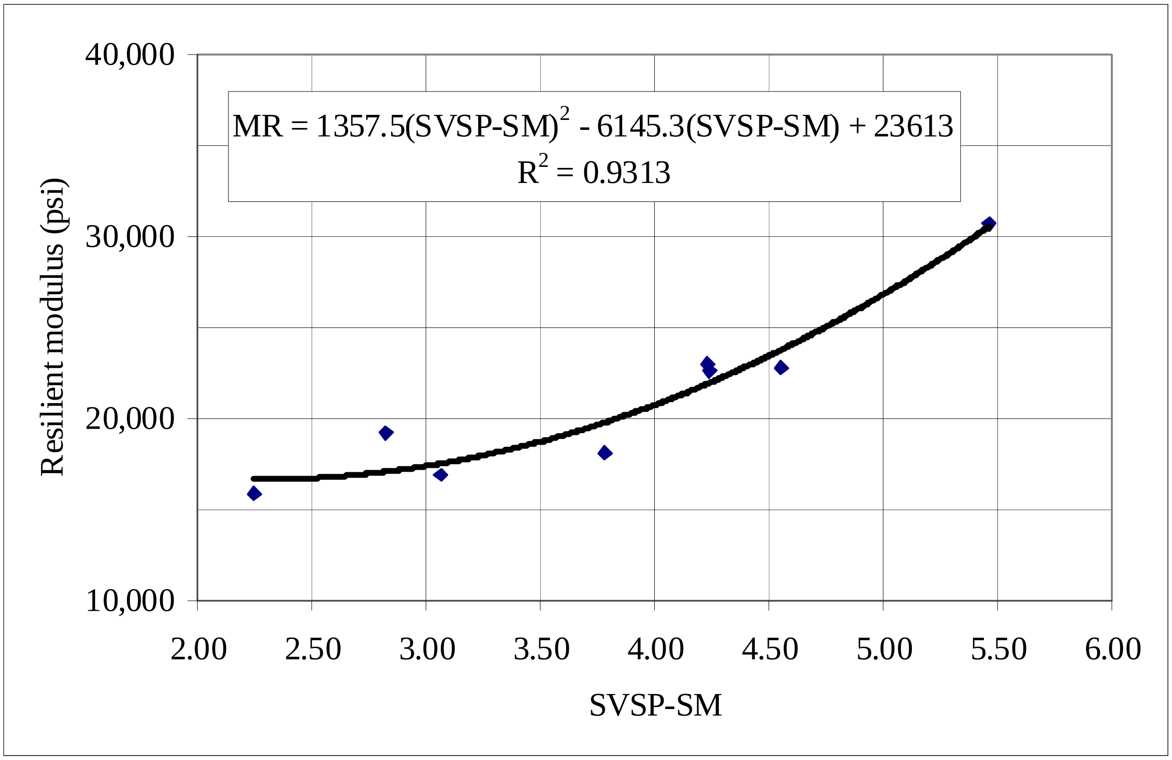


Figure 3.38 Resilient modulus versus the sample variable model, SP-SM soils

Several other models were also rejected for similar reasons. Finally the model presented in Figure 3.39 and expressed in Equation 3.14 was accepted although the value of the coefficient of determination is moderate (0.74).

$$MR = 1749.6 \exp 0.0054(\mathrm{SVSP\text{-}SM}) \qquad \text{Equation 3.14}$$

$$\mathrm{SVSP\text{-}SM} = \frac{\gamma_d^{1.75}}{MC^{0.5} + LL^{0.6} + (P_{40} - P_{200})^{0.01}} \qquad \text{Equation 3.15}$$

Where, γ_d = dry unit weight (pcf), LL = liquid limit, MC = moisture content (%), P_{40}, P_{200} = percent passing sieves number 40 and 200, respectively (%)

The term (P_{40} - P_{200}) expresses the percent fine sand content in the soils. Examination of the data in Figure 3.39 and Equation 3.14 indicates that increases in the values of the dry unit weight cause increases in the MR values, whereas increases in the values of either the moisture content, liquid limit, or the percent fine sand content cause decreases in the resilient modulus values.

It is important to note that the data used in support of Equation 3.14 includes soil samples having dry unit weight ranging from 105.5 to 121.6 pcf, percent passing sieve number 40 from 43.2 to 92.6, percent passing sieve number 200 from 5.7 to 8, liquid limit from 13 to 21, and moisture content from 2.0 to 11.4 percent. The use of Equation 3.14 outside those ranges of values is not recommended.

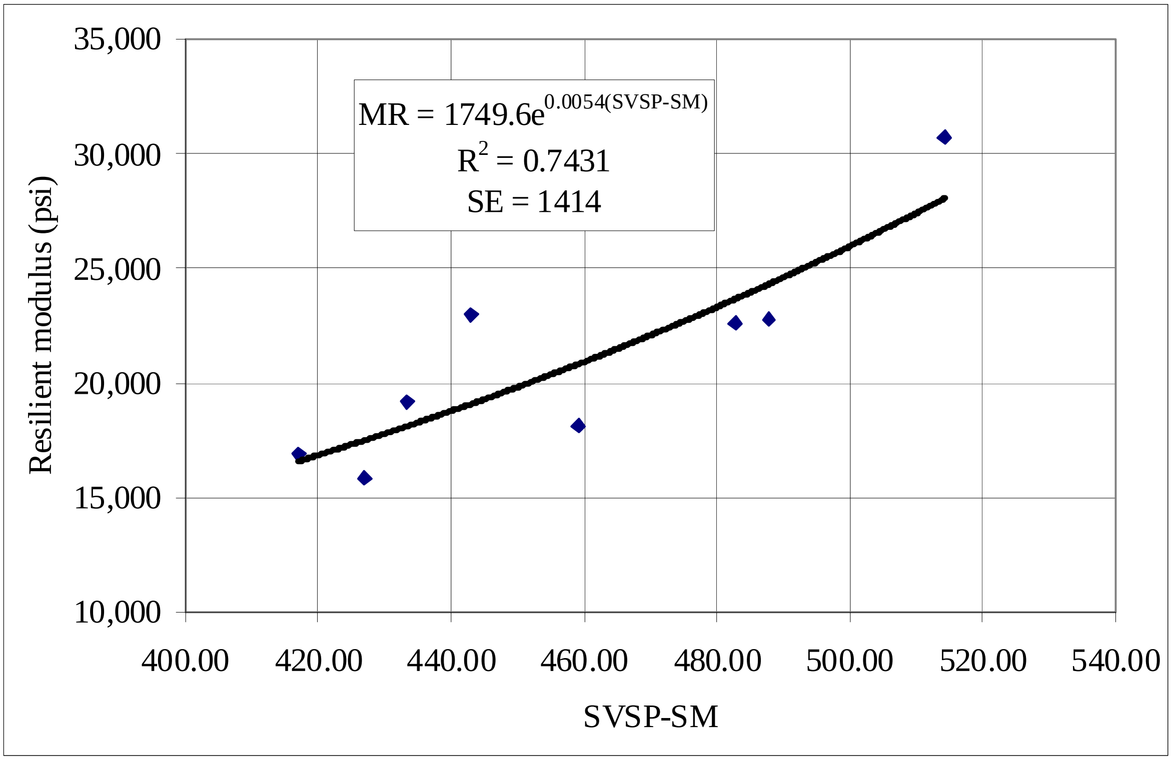


Figure 3.39 Resilient modulus versus SVSP-SM soils

3.6.2.5 Clayey Sand – Silty Sand (SC-SM)

Table 3.11 lists the locations, the USCS and the AASHTO soil classifications, and the sample designation number of seven disturbed soil samples that were collected from various clusters and areas throughout the State of Michigan. The commonality between the seven samples is that all of them were classified in step 1 of the analyses as SC-SM according to the USCS. SC-SM soils may contain anywhere between 12 and 49.9 percent passing sieve number 200 and a plasticity index between 4 and 7 (Holtz and Kovacs 1981). Hence, the fine materials may play a major role in the mechanistic behavior of the soil. In step 2 of the analyses, univariate and multivariate analyses were conducted and are discussed below.

Table 3.11 Locations of SC-SM roadbed soils

Sample number	Location	AASHTO	USCS
M-068-W (04-02)	180 feet West of US-23	A-2-4	SC-SM
M-032-W (04-05)	220 feet East of Herron Rd	A-4	SC-SM
I-075-N (08-06)	80 feet North of bridge after exit 195	A-2-4	SC-SM
I-096-W (09-02)	141 feet West of Morse Lake Ave	A-2-4	SC-SM
M-060-W (11-03)	135 feet West of Southbound I-69 overpass	A-2-4	SC-SM
I-069-S (11-05)	95 feet South of Bridge after exit 10	A-4	SC-SM
I-094-W (12-01)	95 feet West of 29 Mile Rd	A-2-4	SC-SM

Univariate Analysis - In the univariate analyses, the effects of each of several sample variables on the MR values of SC-SM soils were studied. These sample variables include: the moisture content of the samples, the degrees of saturation, the liquid limit, and the grain sizes. The effects of each variable are presented and discussed below.

Effect of the Sample Moisture Contents - Because of the high range in fine content, water content may play a significant role in determining the elastic response of the soil to the applied loads. Figure 3.40 shows the MR values of all seven SC-SM soil samples plotted against the samples moisture contents. As it was expected, the figure shows increases in the sample moisture content cause decreases in the MR values. Similar results were also reported by many researchers including (Maher et. al 2000, George 2000, and 2003). The low value of the coefficient of determination of about 0.25 implies that MR values cannot be explained accurately by the moisture content alone. Therefore, it will be included with other sample variables in the multivariate analyses to determine their combined effect.

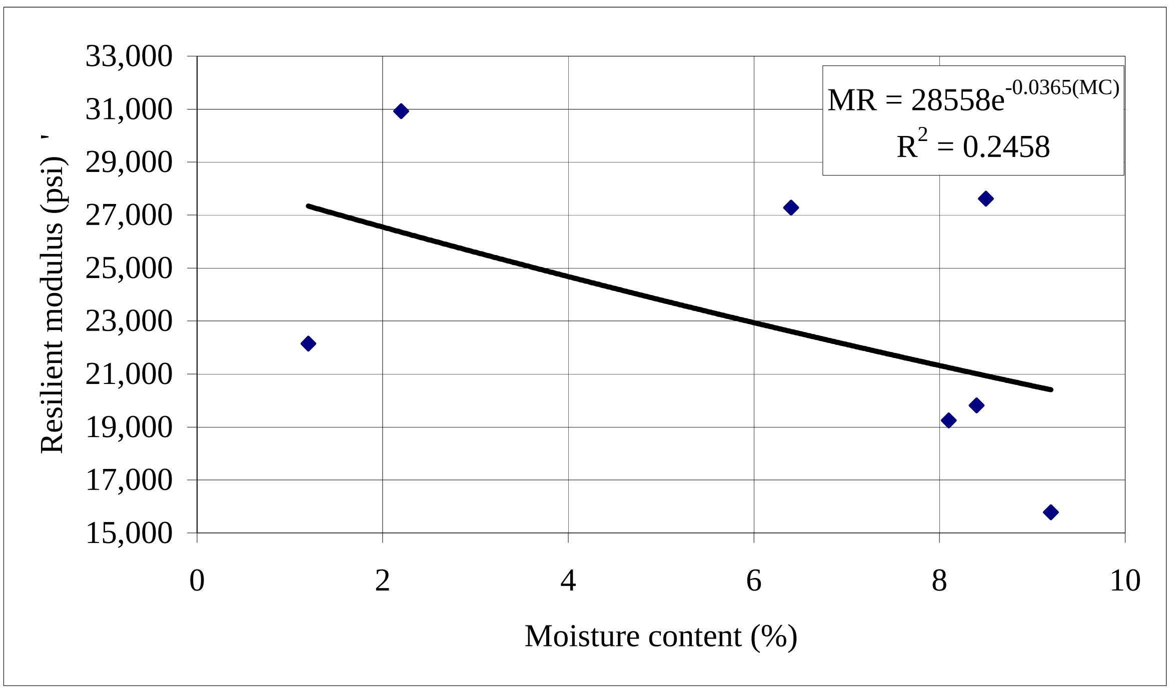


Figure 3.40 Resilient modulus versus moisture content for SC-SM soils

Effect of the Degree of Saturation – At the conclusion of the cyclic load test, the sample moisture content and dry unit weight were determined and the degree of saturation (S) was calculated using Equation 3.6 (which is repeated below for convenience) assuming a specific gravity of the solids of 2.7.

$$S = \left[\frac{G_s * (MC/100) * \gamma_d}{G_s * \gamma_w - \gamma_d} \right] * 100 \qquad \text{Equation 3.6}$$

Where, S = degree of saturation (%), MC = moisture content (%), G_s = specific gravity of the soil solid, γ_d = dry unit weight of the sample (pcf), and γ_w = unit weight of water = 62.4 pcf

Figure 3.41 shows the MR values plotted against the degree of saturation of the test samples. The data in the figure indicates an insignificant correlation between the MR values and the degree of saturation although increasing degree of saturation causes decreases in the MR values as it was reported by Maher et al. (2000). When the data and the values of the coefficients of determination of Figure 3.41 are compared to those in Figure 3.40, it becomes clear that the effects of moisture contents on the MR values can be better expressed using the water content. The reason is that the water content is an independent variable whereas the degree of saturation is a function of both the water content and the dry unit weight.

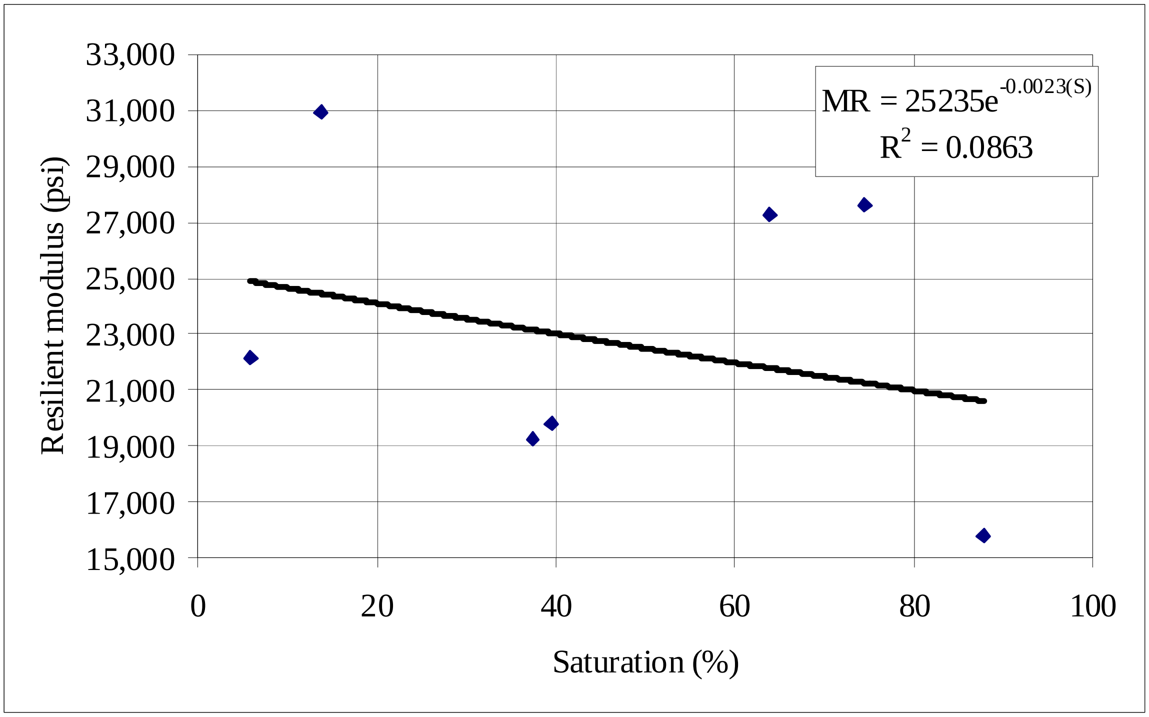


Figure 3.41 Resilient modulus versus saturation for SC-SM soils

Figure 3.42 depicts the MR values plotted against the dry unit weights of the test samples. It can be seen that correlation between them is insignificant although, as it was expected, increasing the values of the dry unit weight cause increases in the MR values. The reason for the insignificant correlation is that the dry unit weight of the test sample is a function of its water content. Further, it is possible for two test samples having the same dry unit weight to have drastically different mechanistic behavior. The scenario is possible provided that the two samples were compacted at two different water contents on the opposite sides of the optimum moisture content (Holtz and Kovacs 1981).

Effect of the Sample Liquid Limit - For each SC-SM soil sample, the Atterberg limits for all materials passing sieve number 40 were determined in order to classify the type of fine materials (silt or clay). The test results indicate that for all samples the liquid limit varied from 15 to 22, the plastic limit from 10 to 15, and the plasticity index from 4 to 7. The effects of the liquid limit on the MR values of the soils were analyzed. Figure 3.43 depicts the influence of the liquid limits on the MR values of SC-SM soils. The data in the figure indicate that the MR values of SC-SM soils decrease as the liquid limit of the material passing sieve number 40 increases. This

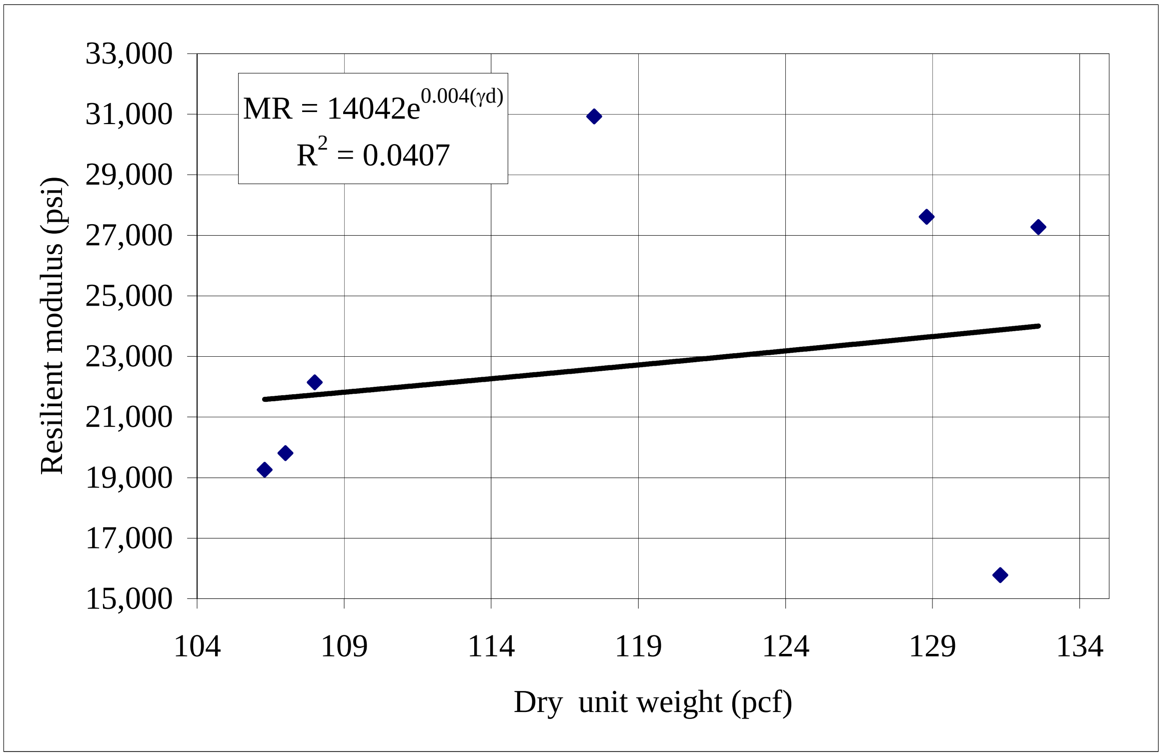


Figure 3.42 Resilient modulus versus the dry unit weight for SC-SM soils

observation was expected and has been reported by many researchers for various soil types including silty and clayey sands, silt, and clay (Gudishala 2004). Therefore, the liquid limit will be included in the multivariate analyses.

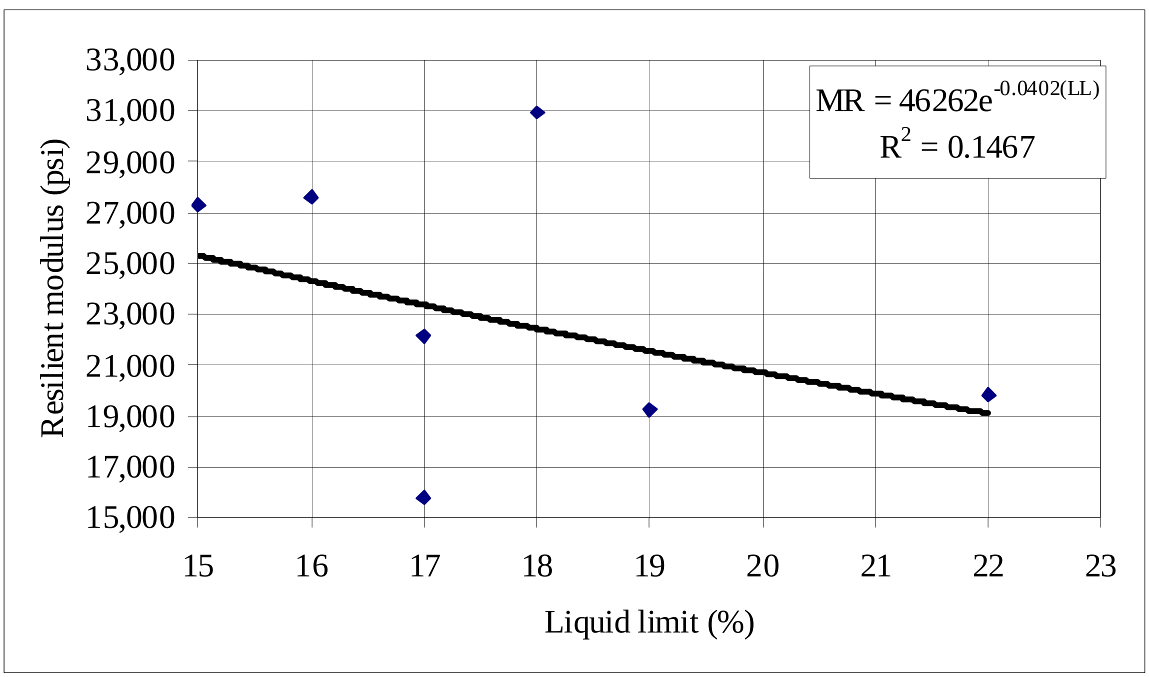


Figure 3.43 Resilient modulus versus liquid limit for SC-SM soils

Effect of Sample Grain Size – Because of the relatively high fine contents, all SC-SM soils were subjected to wet sieving and hydrometer analyses to determine their grain size distribution. The effects of sample gradation on the MR values were assessed through the following gradation parameters:

- Percent passing sieves 200, 100, 40, 20, 10, 4, and 3/8 inch
- The coefficients of curvature and uniformity (C_C and C_U)
- The average particle size at ten, thirty, and sixty percent passing (D_{10}, D_{30}, and D_{60}, respectively)

The effects of each gradation parameter on the MR values were analyzed by plotting the MR values of the soil samples as a function of that parameter. All gradation parameters showed minor correlations to the MR values. Despite this, several attempts were made to include gradation variables in the multivariate analyses as presented below.

Multivariate Analyses - Multivariate analyses were conducted to study the combined effects of several independent sample variables on the dependent variable MR values of SC-SM soils. During the analyses:

- Various models (equation forms) were used in an attempt to maximize the value of the coefficient of determination (R^2).
- Special care was taken to:
  - Ensure that the resulting equation satisfies the previously reported trends between each of the independent variable and the dependent variable MR values.
  - Avoid any significant co-linearity between the independent variables.
  - Minimize the number of independent variables in the equation.

Results of the analyses yielded a model having a relatively high R^2 value. The model is based on three sample variables of the SC-SM (SVSC-SM) soils (the water content, liquid limit, and the coefficient of uniformity) as stated in Equation 3.16. Figure 3.44 depicts the resilient modulus values plotted as a function of the SVSC-SM. It can be seen from the figure that higher SVSC-SM values yield lower MR values.

$$\mathrm{MR} = 39638 \exp - 0.0037(\mathrm{SVSC\text{-}SM}) \qquad \text{Equation 3.16}$$

$$\mathrm{SVSC\text{-}SM} = C_u^{0.2} * \left(\mathrm{LL}^{1.15} + \mathrm{MC}^{1.3}\right) \qquad \text{Equation 3.17}$$

Where, LL = liquid limit, MC = moisture content (%), and C_u = coefficient of uniformity

The liquid limit and coefficient of uniformity are constant for a given soil. Therefore, lower SVSC-SM can be obtained by decreasing the moisture content of the test sample. As the moisture content of the sample increases, the lubrication between the sand particles increases, thus reducing the MR values (Perloff and Baron 1976). The term ($LL^{1.15} + MC^{1.3}$) can be thought of as the moisture index of the SC-SM roadbed soils.

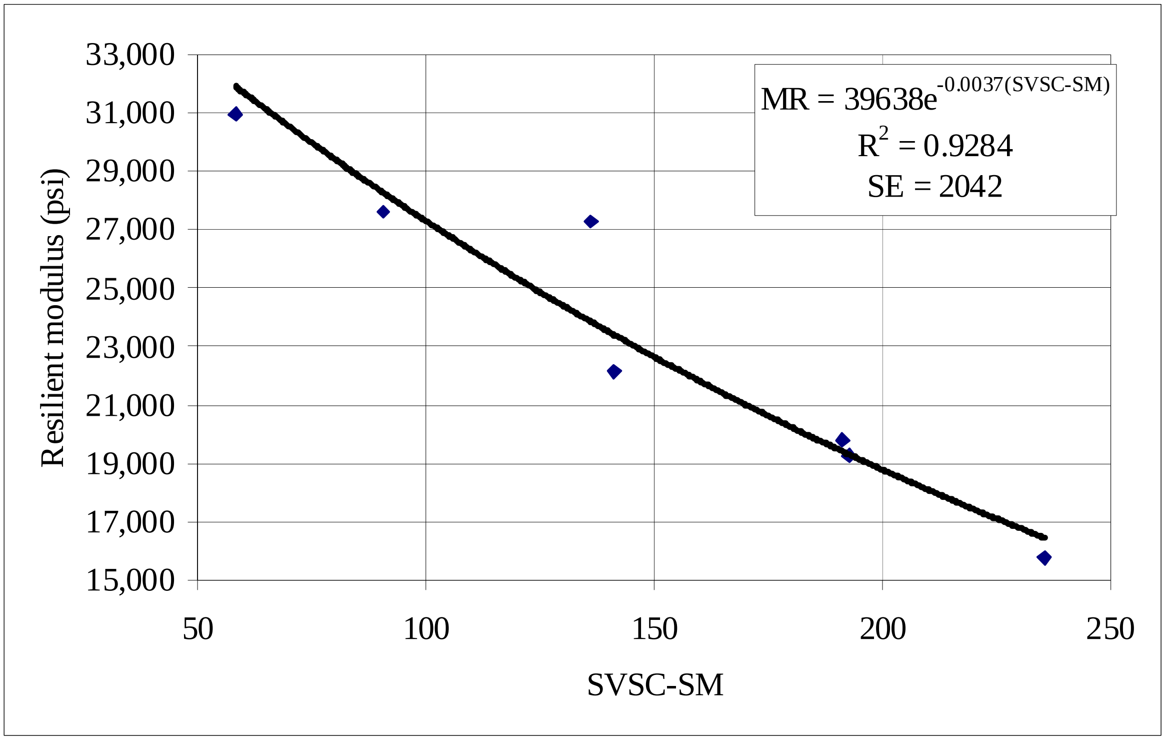


Figure 3.44 Resilient modulus versus SVSC-SM soils

It is important to note that the data used in support of Equation 3.16 includes soil samples having liquid limit values ranging from 15 to 22, coefficients of uniformity between 14.74 and 270, and moisture contents from 1.2 to 9.2 percent. The use of the equation outside any of these ranges is not recommended.

3.6.2.6 Gravely Sand (SG)

Table 3.12 lists the locations, the USCS and the AASHTO soil classifications, and the sample designation number of two disturbed soil samples that were collected from two clusters and areas in the State of Michigan. The two samples were classified in step 1 of the analyses as SG. However, due to the large gravel particles, cyclic load tests could not be performed on these samples. The AASHTO standard test procedure that was used requires that the diameter of the sample be at least four times larger than the largest particle. For this to be satisfied the sample diameter would have to about 9 inches. Because of the limited number of samples and because this type of roadbed material is very much limited to small areas in the State of Michigan, no further analyses were conducted on the two samples.

Table 3.12 Locations of SG roadbed soils

Sample number	Location	AASHTO	USCS
M-033-S (05-05)	750 feet South of Peters Rd	A-1-a	SG
U-012-E (12-07)	120 feet West of Person Hwy	A-1-a	SG

3.7 CLIMATIC DAMAGE MODELS

The State of Michigan is located in the AASHTO wet-freeze region. The average annual rainfall and snowfall in the State varies from one location to another. In the Lansing area, the average annual rainfall is about 32 inches and the average annual snow fall is about 56 inches. Further, the frost depth varies from about 7 feet in the Upper Peninsula to about 3 feet in the southern part of the Lower Peninsula. These climatic data affect the behavior of the paving materials and roadbed soils. Because of the variability of the climatic conditions, the resilient modulus of any given soil is dynamic in nature and it changes seasonally with changing water content and temperatures below and above freezing.

For most pavement structures, the roadbed material is the weakest part of the structure. Hence, the 1993 AASHTO Design Guide calls for protecting the roadbed soils under flexible pavements by providing adequate structural number. For rigid pavement, the modulus of subgrade reaction is typically integrated with that of the base layer to yield a composite modulus of subgrade reaction. In addition, the 1993 AASHTO Design Guide includes damage assessment scenario that must be used to obtain the design modulus of the roadbed soils. Finally, the guide does not provide pavement layer thicknesses based on frost penetration. That is, the guide does not provide any protection (significant depth to provide prevention from freezing) to the roadbed soils against freezing. The new M-EPDG also includes climatic model to reduce the given roadbed soil modulus and hence, account for the seasonal damage.

For any pavement structure, the moisture content and percent saturation of the roadbed materials is a function of many variables including:

- The duration and frequency of rainfall
- The geometry of the pavement and the flow regime of the adjacent ground
- The conditions and permeability of the surface layers and the number of unsealed cracks
- The permeability of the roadbed materials
- The type of drainage system installed in the pavement structure
- The initial crown of the roadbed soils created during rolling
- The elevation of the ground water table

Based on the above list, a given roadbed soil located in the State of Michigan could be saturated during spring thaw season, during extended periods of rainfall (more than few hours), or during frequent rainfall. Based on this scenario, one can conclude that the roadbed soil is highly likely saturated during spring thaw, and it could be occasionally saturated during the summer and fall seasons. The roadbed soil is likely frozen during the late winter season. Based on this scenario and on weather data, it was assumed that, on average, for a given year period, the roadbed soil in Michigan is subjected to:

- Four months of near saturation water contents
- Four months of near optimum water contents
- Four months of dry of optimum water contents

Given the variability of the degree of saturation of the roadbed soil during the pavement service life and the corresponding variation in its resilient modulus, the question becomes what roadbed soil resilient modulus should be used in the design of new pavement or the rehabilitation of existing pavements. The answer depends on the type of damage model used in the analysis.

In general, two types of damage models are available; empirical and mechanistic-empirical. An example of the former is the 1993 AASHTO Design Guide. Examples of the latter are the fatigue and rut models in the MICHPAVE computer program, in VESYS computer program, in the Asphalt Institute pavement design, and in almost every other mechanistic-empirical computer program. The 1993 AASHTO Design Guide damage model is presented in Equation 3.18.

$$Uf = 1.18 * 10^{8} * MR^{-2.32} \qquad \text{Equation 3.18}$$

Where, Uf = damage factor and MR = resilient modulus of the roadbed soils (psi)

Equation 3.18 is used by entering different values of the roadbed soils that correspond to different season or degrees of saturation. For each MR value, the damage factor (Uf) is calculated. The average damage factor ($Uf_{average}$) is then calculated by dividing the sum of all damage factors by the number of MR values. The ($Uf_{average}$) is then used as input into equation 3.18 and the design MR is calculated.

In this study, the reduction factor to reduce the resilient modulus value from its summer and fall season values was obtained using the following procedure:

- For each of the eight soil types except the SP, the applicable correlation equation was used to predict three resilient modulus values corresponding to three water contents; dry of optimum, near optimum and wet of optimum (degree of saturation between 93 and 99 percent) of the applicable standard proctor compaction curve. The three MR values are listed in Table 3.13.
- Moisture contents or the degrees of saturation are not input variables to the predictive equations (Equations 3.2 through 3.5) of the SP soils. The reason is that the two variables have insignificant effects on MR for all moisture contents below the 80 percent saturation level. At water contents corresponding to saturation levels of 80 percent or higher, water drains out of the SP test samples during the cyclic load triaxial test. Hence, the water content of the test sample changes during the test. For that reason, the reduction factor between the resilient modulus at the optimum water content and that near saturation for SP soils was obtained from Holtz and Kovacs (1981).
- For each of the eight soil types, two damage factor values were calculated as stated below and are listed in Table 3.13.

    - The ratio of the MR value near the optimum water content (the highest MR value in Table 3.13) divided by the MR value near saturation (the lowest MR value).
    - The ratio of the MR value at saturation levels between 75 and 85 divided by the MR value near saturation (the lowest MR value).

- A pavement cross-section of 12 inch thick subbase, 9 inch base, and 7 inch asphalt layer was used to calculate the service life of the pavement in terms of ESALs using the MICHPAVE computer program. The same pavement cross-section was analyzed twice, once for roadbed soil modulus value of 16,000 psi (an average modulus near the optimum water content) and once for an average modulus value near saturation (4000 psi). Results of the analysis yielded two expected pavement lives in term of ESALs. The ratio of the expected ESAL at the high modulus was then divided by the ratio of ESAL at the low modulus. The results yielded a damage factor of about 4.5.

- The same standard pavement cross-section and procedure used in MICHPAVE were used in the 1993 AASHTO Pavement Design Guide; the calculated damage factor was about 5.
- The recommended roadbed soil design resilient modulus values for each of the eight soil types was then calculated by dividing the resilient modulus corresponding to 75-85 percent saturation, per soil type, by the medium/low reduction factor listed in Table 3.13. The values of the recommended design resilient modulus are listed in Tables 3.13 and 3.14.

It should be noted that the recommended design resilient modulus values listed in Table 3.14 are to be used in the M-E PDG design level 3. For design level 2, the correlation equations should be used. And for design level 1, FWD tests should be conducted and the design modulus value backcalculated (as discussed in Chapter 4).

Table 3.13 Damage factor calculation and design resilient modulus

USCS classification	Water content (%)			Resilient modulus (psi) corresponding to saturation range of			Average MR (psi) using the highest 2 degrees of saturation	Reduction Factor		Design MR Value (psi)	Damage factor	
	Near optimum	Moderate	High	45-75	75-85	85-100		High/ Low	Medium/ Low		AASHTO	MICHPAVE
SC, CL, ML	15	22	30	30,543	6,879	4,430	5655	6.9	1.55	4,430		
SC-SM	8.5	15	28	27,276	10,000	5,100	7550	5.3	1.96	5,100		
SP-SM	8	15	20	23,009	11,000	7,000	9000	3.3	1.57	7,000		
SM (MI)	12	20	23	18,416	11,480	5,290	8385	3.5	2.17	5,290		
SP1	11	20	25	29,418	11,798	7,100	9449	4.1	1.66	7,100		
SP2	10	20	25	22,768	9,969	6,500	8235	3.5	1.53	6,500		
Average								4.4	1.74		5	4.5
Lightly shaded cells represent moisture contents outside those of the sampled soil type												

Table 3.14 Design resilient modulus values for M-E PDG design level 3

USCS Classification	Design MR (psi)
SC, CL, ML	4,430
SC-SM	5,100
SP-SM	7,000
SM	5,290
SP1	7,100
SP2	6,500

CHAPTER 4

FWD INVESTIGATIONS AND DATA ANALYSIS

4.1 DEFLECTION TESTS

Several thousand deflection tests using Falling Weight Deflectometer (FWD) were conducted and analyzed during this study. All NDT were conducted by MDOT personnel using the MDOT KUAB FWD. The weight and the height of drop for all NDT were adjusted to produce 9000 pound load. For each test, the pavement surface deflections were measured at the distances of 0.0, 8.0, 12.0, 18.0, 24.0, 36.0 and 60.0-inch from the center of the loaded area. To analyze the roadbed soils of the entire state, FWD tests must be conducted on the entire state road network. MDOT has been conducting FWD tests for over 20 years and has collected deflection data from most of the state road network. A total of five hundred five data files were obtained from MDOT and scrutinized for possible inclusion in the backcalculation of the roadbed modulus. All data files were tested relative to the information available in the data file and MDOT records. All files that passed the tests were included in the analysis. The tests consisted of the following:

- The FWD data files contain the proper date and location reference information.
- The pavement type and the pavement cross-section data at the time of the FWD tests are available in (and can be obtained from) the MDOT project files and records.
- The FWD tests were conducted on Interstate (I), United State (US), and/or Michigan (M) roads.
- The FWD tests were conducted on either flexible or rigid pavement types (composite pavements were not analyzed).

One hundred one FWD data files containing 6,246 FWD tests satisfied the above requirements, and therefore they were included in the analyses. These files were examined to determine the NDT test locations (see solid squares in Figure 4.1, which are repeated for convenience in Figure E.1 of Appendix E). The tests were conducted along twenty one roads (eleven M roads, six I roads, and four U.S. roads) spanning twelve clusters and thirty two areas. Table 4.1 lists the distribution of the FWD data files by pavement type (flexible or rigid pavement) and by roadbed soil USCS classification.

As can be seen from Figure 4.1, certain areas of the state lack sufficient NDT tests. Hence, 217 additional FWD test sites were requested from MDOT to fill up the gap and to cover different environmental seasons (see open circles and triangles in Figure 4.1). Due to several constraints, the number of requested FWD tests was reduced several times. Finally, 57 additional FWD tests were conducted spanning fifteen roads (four M roads, four I roads, and seven U.S. roads) in eleven clusters and nineteen areas by MDOT; the locations of these tests are indicated by the open triangles in Figure 4.1, and detailed in Table E.1 of Appendix E.

The data from a typical deflection test are shown in Table 4.2. The example tests were conducted on September 15th, 1998 on a flexible pavement section of northbound M-24 north of Hoppe Road. All deflection data is presented in Tables E.2, E.3, E.4, and E.5 Appendix E. It should be noted herein that Tables E.4 and E.5 are on the accompanying CD only; examples of the tables are shown in Tables 4.3 and 4.4.

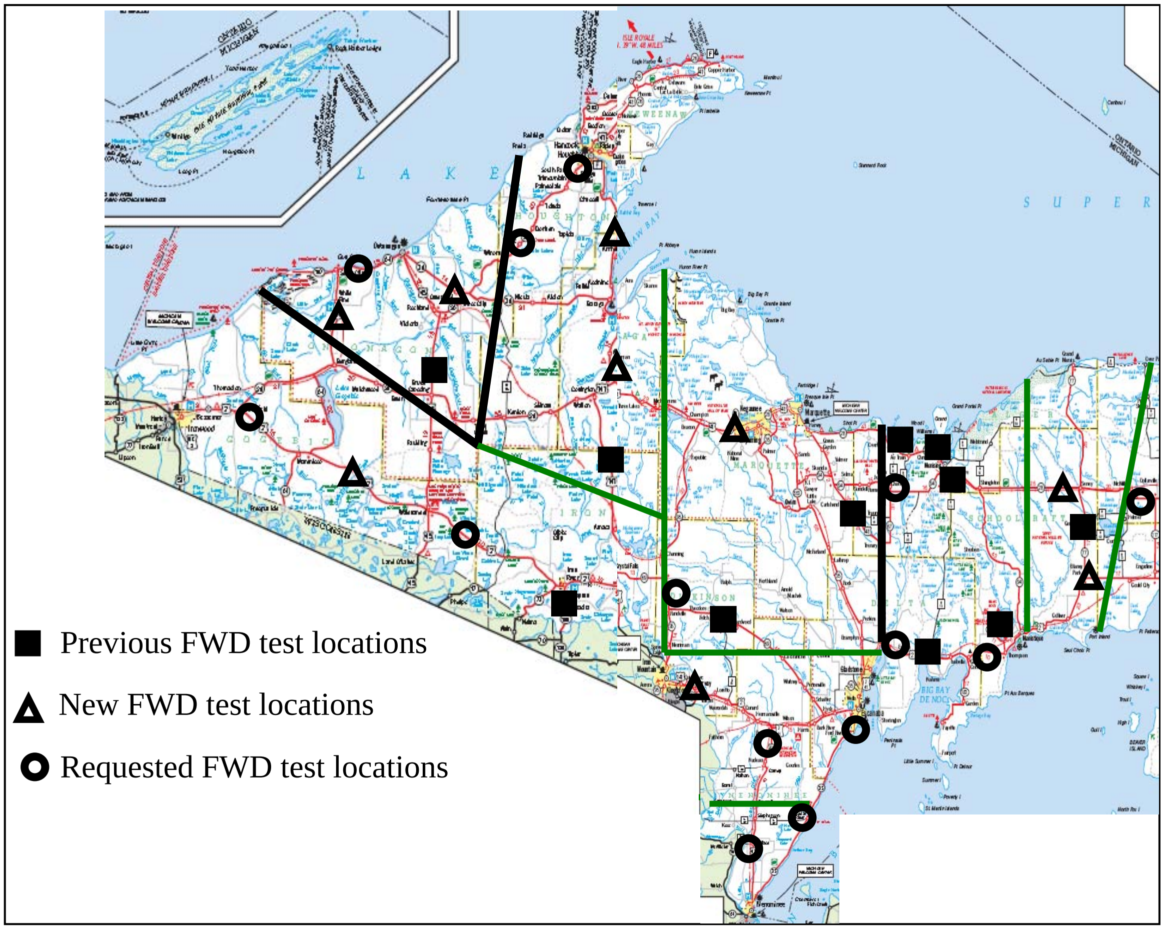


Figure 4.1 FWD test locations in the State of Michigan

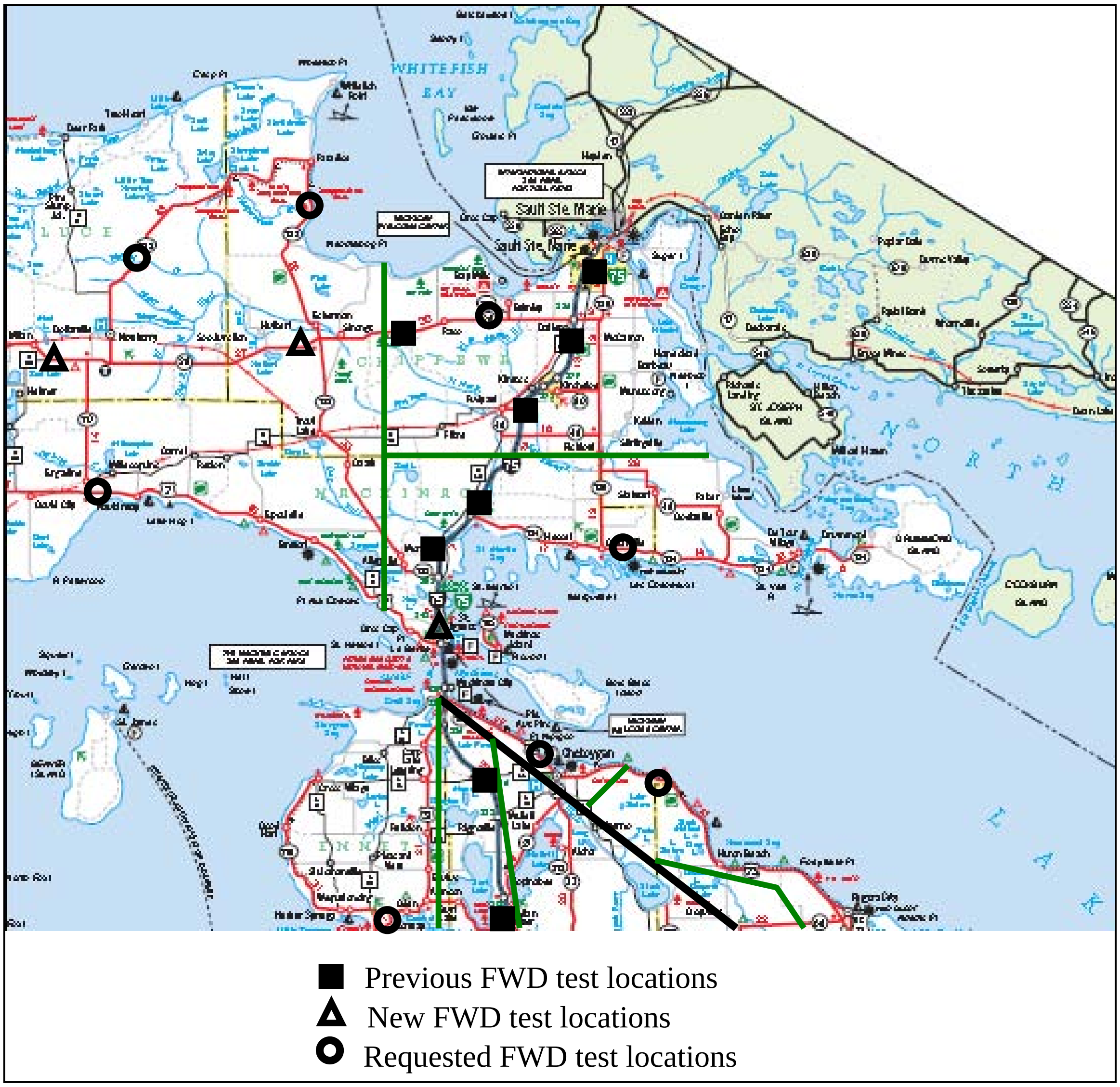


Figure 4.1 (cont'd)

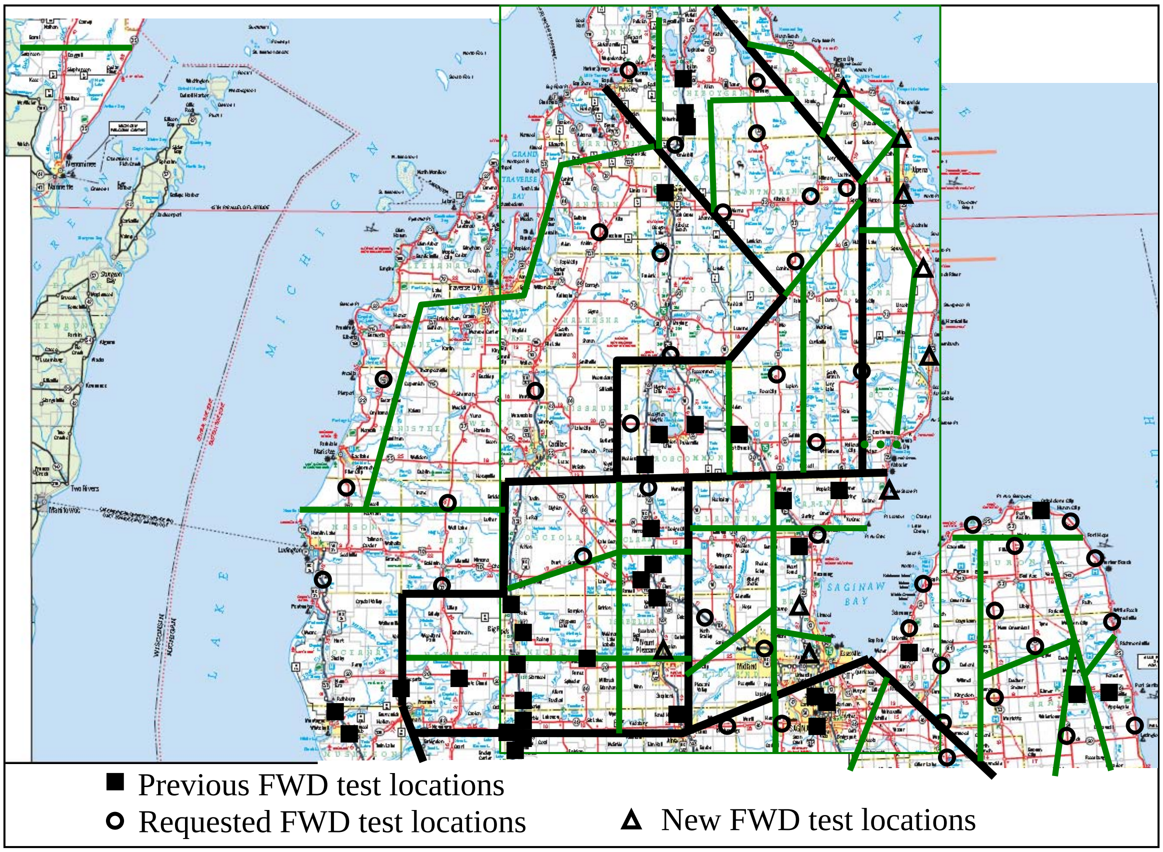


Figure 4.1 (cont'd)

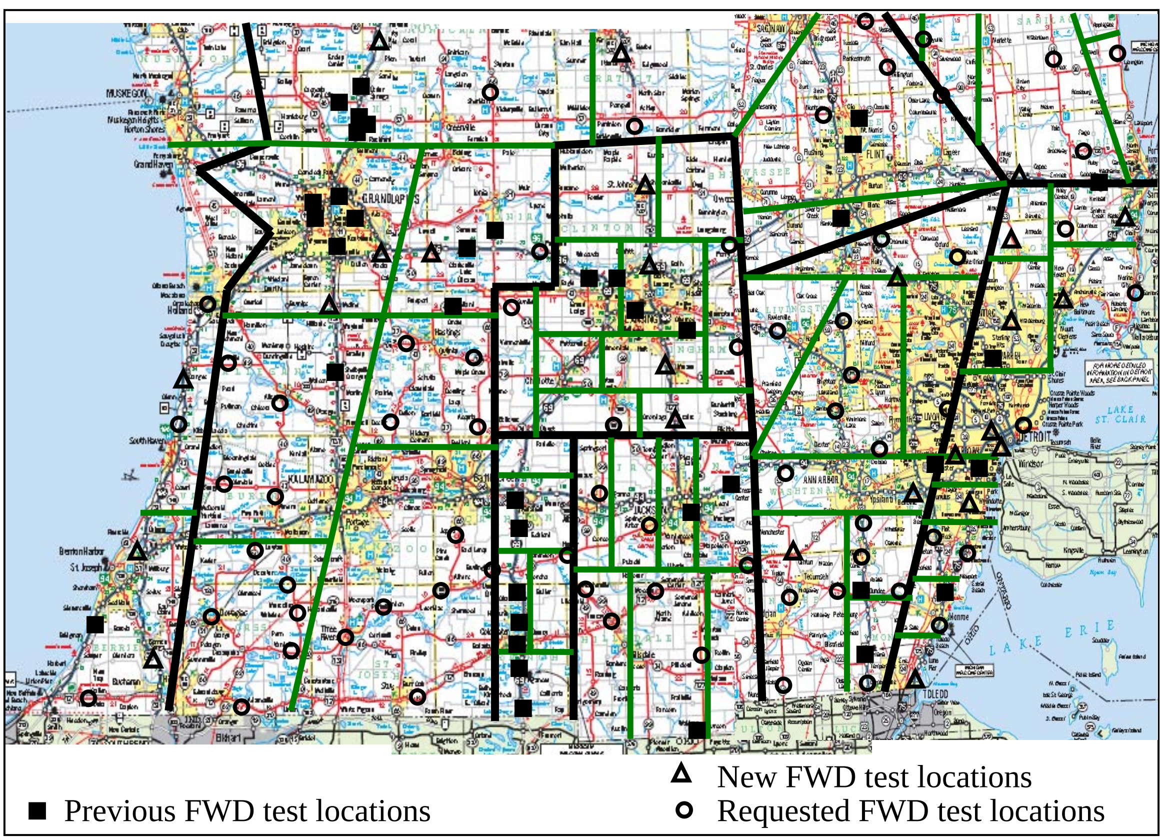


Figure 4.1 (cont'd)

Table 4.1 Distribution of old FWD files

		Rigid pavement		Flexible pavement		Total	
USCS		Files	Tests	Files	Tests	Files	Tests
	Total	295	-	140	-	435	-
	Usable	64	4,684	37	1,562	101	6,246
SM		6	244	1	79	7	323
SP1		9	494	22	1,027	31	1,521
SP2		8	575	2	67	10	642
SP-SM		9	379	0	0	9	379
SC-SM		11	1,967	0	0	11	1,967
SC		19	941	12	389	31	1,330
CL		2	84	0	0	2	84
ML		0	0	0	0	0	0

Table 4.2 Typical FWD deflection file

Distance (ft)	Average load (lbs)	Average deflection in mils at the specified distance from the center of the loaded area in inch						
		0	8	12	18	24	36	60
0	9402	11.56	9.33	8.00	6.53	5.09	3.23	1.29
20	9407	10.76	8.72	7.50	6.14	4.69	2.88	1.10
40	9379	11.68	9.35	8.03	6.46	4.98	3.07	1.16
60	9371	14.02	11.71	10.19	8.57	6.82	3.83	1.62
80	9400	11.72	9.68	8.49	7.16	5.75	3.73	1.46
100	9333	11.68	9.59	8.35	7.13	5.77	3.80	1.57
120	9412	11.31	9.29	8.00	6.65	5.23	3.27	1.23
140	9361	12.73	10.17	8.73	6.96	5.36	3.25	1.15
160	9367	11.99	9.58	8.35	6.89	5.47	3.39	1.22
180	9354	12.12	9.91	8.57	7.14	5.68	3.46	1.31
200	9341	11.71	9.51	8.33	6.97	5.62	3.64	1.37
220	9345	10.60	8.55	7.34	6.05	4.74	2.90	1.02
240	9286	11.60	9.56	8.36	7.18	5.83	3.92	1.66
260	9314	10.58	8.60	7.40	6.09	4.75	2.91	1.03
280	9373	11.90	9.85	8.57	7.23	5.80	3.74	1.42
300	9324	12.26	10.11	8.91	7.51	5.98	3.92	1.55

Table 4.3 Flexible pavement test and backcalculation of layer moduli results

Location			Roadbed type USCS	FWD file information		Pavement layer thickness (in)		Deflection in mils at the specified distance from the center of the loaded area in inch							Backcalculation				Resilient modulus (psi)		
															Error RMS (%)	Converged?		Depth to stiff layer (in)			
Region	Road	Cluster-area		Date	File title	Asphalt concrete	Base/ subbase	0	8	12	18	24	36	60		Yes	No		Asphalt concrete	Base/ subbase	Roadbed
Superior	US-2	02-01	SM	5/20/2008	flex-Su-US2-CS27022-05-20-2008	3.5	26.5	8.31	5.84	4.68	3.62	3.07	2.42	1.56	1.94	1		250	655,235	107,874	18,118
Superior	US-2	02-01	SM	5/20/2008	flex-Su-US2-CS27022-05-20-2008	3.5	26.5	13.09	9.28	6.93	4.76	3.50	2.10	1.04	1.20	1		250	947,354	35,020	25,165
Superior	US-2	02-01	SM	5/20/2008	flex-Su-US2-CS27022-05-20-2008	3.5	26.5	13.71	9.78	7.35	4.92	3.59	2.12	1.04	0.92	1		250	931,590	32,275	25,169
Superior	US-2	02-01	SM	5/20/2008	flex-Su-US2-CS27022-05-20-2008	3.5	26.5	13.48	10.02	8.06	5.98	4.67	2.99	1.55	1.58	1		250	1,356,105	34,541	17,121
Superior	US-2	02-01	SM	5/20/2008	flex-Su-US2-CS27022-05-20-2008	3.5	26.5	12.24	8.65	6.64	4.71	3.56	2.18	1.12	1.63	1		250	1,088,984	38,870	23,762
Superior	US-2	02-01	SM	5/20/2008	flex-Su-US2-CS27022-05-20-2008	3.5	26.5	13.93	10.12	7.78	5.47	4.09	2.52	1.40	1.78	1		250	887,769	35,293	19,730
Superior	US-2	02-01	SM	5/20/2008	flex-Su-US2-CS27022-05-20-2008	3.5	26.5	14.48	10.50	8.11	5.75	4.38	2.70	1.42	1.22	1		250	942,671	32,916	18,862
Superior	US-2	02-01	SM	5/20/2008	flex-Su-US2-CS27022-05-20-2008	3.5	26.5	7.39	5.46	4.58	3.88	3.44	2.78	1.67	2.80		1	250			
Superior	US-2	02-01	SM	5/20/2008	flex-Su-US2-CS27022-05-20-2008	3.5	26.5	9.28	6.19	4.65	3.53	3.08	2.61	1.77	5.90		1	250			
Superior	US-2	02-01	SM	5/20/2008	flex-Su-US2-CS27022-05-20-2008	3.5	26.5	12.08	8.28	6.01	4.01	3.00	2.03	1.31	5.76		1	250			
Superior	US-2	02-01	SM	5/20/2008	flex-Su-US2-CS27022-05-20-2008	3.5	26.5	13.40	9.48	7.15	4.78	3.40	1.92	0.84	2.56		1	250			
North	US-131	07-01	SM	5/1/2002	flex-N-US131-CS67017-05-01-2002	7.25	22	6.30	5.68	5.19	4.52	3.89	2.84	1.58	0.20	1		700	2,273,076	29,426	25,071
North	US-131	07-01	SM	5/1/2002	flex-N-US131-CS67017-05-01-2002	7.25	22	8.07	7.21	6.50	5.55	4.64	3.20	1.42	0.66	1		700	1,807,213	11,116	30,083
North	US-131	07-01	SM	5/1/2002	flex-N-US131-CS67017-05-01-2002	7.25	22	7.53	6.77	6.19	5.35	4.54	3.19	1.58	0.08	1		700	2,006,488	14,768	26,207
North	US-131	07-01	SM	5/1/2002	flex-N-US131-CS67017-05-01-2002	7.25	22	10.87	8.11	6.93	5.58	4.62	3.12	1.76	1.76	1		700	429,536	36,980	20,693
North	US-131	07-01	SM	5/1/2002	flex-N-US131-CS67017-05-01-2002	7.25	22	7.90	7.11	6.52	5.71	4.89	3.54	1.85	0.19	1		700	2,035,012	15,852	22,285
North	US-131	07-01	SM	5/1/2002	flex-N-US131-CS67017-05-01-2002	7.25	22	6.76	6.29	5.88	5.13	4.44	3.31	1.82	0.80	1		700	2,737,602	17,573	23,167
North	US-131	07-01	SM	5/1/2002	flex-N-US131-CS67017-05-01-2002	7.25	22	6.90	6.18	5.60	4.81	4.08	2.88	1.56	0.41	1		700	1,834,574	26,364	25,270
North	US-131	07-01	SM	5/1/2002	flex-N-US131-CS67017-05-01-2002	7.25	22	6.08	5.49	5.06	4.46	3.84	2.84	1.55	0.19	1		700	2,745,326	23,557	26,436
North	US-131	07-01	SM	5/1/2002	flex-N-US131-CS67017-05-01-2002	7.25	22	6.77	6.09	5.55	4.84	4.13	2.80	1.54	1.58	1		700	1,944,166	24,414	26,019
North	US-131	07-01	SM	5/1/2002	flex-N-US131-CS67017-05-01-2002	7.25	22	5.91	5.25	4.79	4.15	3.55	2.57	1.41	0.20	1		700	2,295,196	30,795	27,913
North	US-131	07-01	SM	5/1/2002	flex-N-US131-CS67017-05-01-2002	7.25	22	6.44	5.85	5.38	4.78	4.14	3.07	1.75	0.29	1		700	2,538,022	26,136	22,896
North	US-131	07-01	SM	5/1/2002	flex-N-US131-CS67017-05-01-2002	7.25	22	9.03	8.03	7.38	6.46	5.54	3.95	2.00	0.48	1		700	1,798,903	12,261	20,974
North	US-131	07-01	SM	5/1/2002	flex-N-US131-CS67017-05-01-2002	7.25	22	6.70	6.00	5.57	4.94	4.26	3.11	1.60	0.57	1		700	2,775,105	14,208	27,162
North	US-131	07-01	SM	5/1/2002	flex-N-US131-CS67017-05-01-2002	7.25	22	5.38	4.85	4.48	3.95	3.44	2.60	1.53	0.10	1		700	2,836,815	41,551	25,564
North	US-131	07-01	SM	5/1/2002	flex-N-US131-CS67017-05-01-2002	7.25	22	8.07	7.22	6.56	5.72	4.87	3.46	1.68	0.55	1		700	1,978,408	12,350	25,199
North	US-131	07-01	SM	5/1/2002	flex-N-US131-CS67017-05-01-2002	7.25	22	7.91	7.10	6.48	5.62	4.72	3.29	1.54	0.24	1		700	1,958,727	11,183	27,965
North	US-131	07-01	SM	5/1/2002	flex-N-US131-CS67017-05-01-2002	7.25	22	7.81	7.15	6.62	5.87	5.15	3.94	2.20	0.52	1		700	2,466,243	17,152	18,680
North	US-131	07-01	SM	5/1/2002	flex-N-US131-CS67017-05-01-2002	7.25	22	8.52	7.72	7.06	6.07	5.13	3.60	1.91	0.75	1		700	1,596,168	17,463	20,914
North	US-131	07-01	SM	5/1/2002	flex-N-US131-CS67017-05-01-2002	7.25	22	8.36	7.42	6.72	5.68	4.70	3.10	1.46	0.64	1		700	1,463,826	14,380	27,254
North	US-131	07-01	SM	5/1/2002	flex-N-US131-CS67017-05-01-2002	7.25	22	6.37	5.70	5.21	4.51	3.84	2.78	1.35	0.77	1		700	2,558,032	15,648	31,317
North	US-131	07-01	SM	5/1/2002	flex-N-US131-CS67017-05-01-2002	7.25	22	7.36	6.58	6.02	4.84	4.11	2.94	1.50	1.68	1		700	1,513,192	24,600	25,531
North	US-131	07-01	SM	5/1/2002	flex-N-US131-CS67017-05-01-2002	7.25	22	7.76	6.59	5.83	4.93	4.12	2.81	1.48	0.72	1		700	1,222,334	28,513	25,251

Table 4.4 Rigid pavement test and backcalculation of layer moduli results

Location			Roadbed type USCS	FWD file information		Concrete slab thickness (in)	Average deflection in mils at the specified distance from the center of the loaded area in inch							Roadbed K (pci)	Roadbed MR (psi)	Regular deflection basin
Region	Road	Cluster-area		Date	File title		0	8	12	18	24	36	60			number = yes blank = no
Grand	I-96	09-07	SM	6/27/2001	rigid-G-I96-CS34044-06-27-2001	9	7.05	5.87	5.24	4.61	4.13	3.43	2.32	262	20,341	20,341
Grand	I-96	09-07	SM	6/27/2001	rigid-G-I96-CS34044-06-27-2001	9	8.07	6.61	5.87	4.96	4.29	3.31	2.17	285	22,118	22,118
Grand	I-96	09-07	SM	6/27/2001	rigid-G-I96-CS34044-06-27-2001	9	7.91	6.46	5.67	4.84	4.17	3.27	2.13	295	22,893	22,893
Grand	I-96	09-07	SM	6/27/2001	rigid-G-I96-CS34044-06-27-2001	9	7.83	6.42	5.67	4.80	4.13	3.23	2.09	296	22,946	22,946
Grand	I-96	09-07	SM	6/27/2001	rigid-G-I96-CS34044-06-27-2001	9	5.43	4.69	4.25	3.78	3.43	2.72	1.73	295	22,915	22,915
Grand	I-96	09-07	SM	6/27/2001	rigid-G-I96-CS34044-06-27-2001	9	7.52	6.26	5.47	4.69	3.94	2.83	1.61	332	25,793	25,793
Grand	I-96	09-07	SM	6/27/2001	rigid-G-I96-CS34044-06-27-2001	9	7.52	6.26	5.55	4.72	3.98	2.87	1.61	327	25,386	25,386
Grand	I-96	09-07	SM	6/27/2001	rigid-G-I96-CS34044-06-27-2001	9	7.48	6.22	5.55	4.69	3.98	2.83	1.57	330	25,617	25,617
Grand	I-96	09-07	SM	6/27/2001	rigid-G-I96-CS34044-06-27-2001	9	6.34	5.51	4.96	4.33	3.78	2.91	1.46	306	23,759	23,759
Grand	I-96	09-07	SM	6/27/2001	rigid-G-I96-CS34044-06-27-2001	9	3.78	3.27	2.95	2.64	2.28	1.81	0.98	471	36,582	36,582
Grand	I-96	09-07	SM	6/27/2001	rigid-G-I96-CS34044-06-27-2001	9	3.78	3.27	2.95	2.60	2.28	1.81	0.98	472	36,633	36,633
Grand	I-96	09-07	SM	6/27/2001	rigid-G-I96-CS34044-06-27-2001	9	3.82	3.27	2.95	2.60	2.24	1.81	0.98	483	37,499	37,499
Grand	I-96	09-07	SM	6/27/2001	rigid-G-I96-CS34044-06-27-2001	9	8.46	7.17	6.42	5.59	4.96	3.98	2.44	223	17,282	17,282
Grand	I-96	09-07	SM	6/27/2001	rigid-G-I96-CS34044-06-27-2001	9	8.19	6.89	6.18	5.39	4.76	3.86	2.36	233	18,071	18,071
Grand	I-96	09-07	SM	6/27/2001	rigid-G-I96-CS34044-06-27-2001	9	8.19	6.89	6.22	5.39	4.80	3.90	2.40	228	17,694	17,694
Grand	I-96	09-07	SM	6/27/2001	rigid-G-I96-CS34044-06-27-2001	9	3.50	3.15	2.91	2.64	2.40	2.05	1.38	332	25,783	25,783
Grand	I-96	09-07	SM	6/27/2001	rigid-G-I96-CS34044-06-27-2001	9	4.45	3.70	3.31	2.91	2.60	2.09	1.38	420	32,563	32,563
Grand	I-96	09-07	SM	6/27/2001	rigid-G-I96-CS34044-06-27-2001	9	4.45	3.66	3.31	2.95	2.64	2.17	1.34	407	31,618	31,618
Grand	I-96	09-07	SM	6/27/2001	rigid-G-I96-CS34044-06-27-2001	9	6.14	4.92	4.37	3.66	3.11	2.40	1.42	413	32,076	32,076
Grand	I-96	09-07	SM	6/27/2001	rigid-G-I96-CS34044-06-27-2001	9	5.83	4.61	4.06	3.43	2.91	2.24	1.34	450	34,903	34,903
Grand	I-96	09-07	SM	6/27/2001	rigid-G-I96-CS34044-06-27-2001	9	5.75	4.61	4.09	3.43	2.95	2.28	1.38	431	33,470	33,470
Bay	US-23	09-09	SM	10/21/1998	rigid-B-US23-CS25031-10-21-1998	9	4.17	3.66	3.46	3.15	2.68	2.24	1.42	363	28,182	28,182
Bay	US-23	09-09	SM	10/21/1998	rigid-B-US23-CS25031-10-21-1998	9	4.25	3.74	3.50	3.23	2.72	2.28	1.42	358	27,779	27,779
Bay	US-23	09-09	SM	10/21/1998	rigid-B-US23-CS25031-10-21-1998	9	4.17	3.70	3.50	3.19	2.76	2.28	1.46	342	26,565	26,565
Bay	US-23	09-09	SM	10/21/1998	rigid-B-US23-CS25031-10-21-1998	9	4.61	2.83	2.72	2.56	2.20	1.89	1.26	698	54,161	54,161
Bay	US-23	09-09	SM	10/21/1998	rigid-B-US23-CS25031-10-21-1998	9	4.65	2.87	2.72	2.60	2.28	1.93	1.22	684	53,087	53,087
Bay	US-23	09-09	SM	10/21/1998	rigid-B-US23-CS25031-10-21-1998	9	4.65	2.87	2.68	2.60	2.24	1.89	1.22	698	54,159	54,159
Bay	US-23	09-09	SM	10/21/1998	rigid-B-US23-CS25031-10-21-1998	9	3.54	3.19	3.07	2.95	2.56	2.20	1.42	300	23,303	23,303
Bay	US-23	09-09	SM	10/21/1998	rigid-B-US23-CS25031-10-21-1998	9	3.54	3.27	3.11	2.95	2.64	2.20	1.42	287	22,296	22,296
Bay	US-23	09-09	SM	10/21/1998	rigid-B-US23-CS25031-10-21-1998	9	3.58	3.23	3.11	2.99	2.64	2.24	1.46	287	22,276	22,276
Bay	US-23	09-09	SM	10/21/1998	rigid-B-US23-CS25031-10-21-1998	9	3.58	3.31	3.19	2.99	2.60	2.17	1.42	296	22,941	22,941
Bay	US-23	09-09	SM	10/21/1998	rigid-B-US23-CS25031-10-21-1998	9	3.62	3.43	3.19	2.99	2.64	2.13	1.42	302	23,420	23,420
Bay	US-23	09-09	SM	10/21/1998	rigid-B-US23-CS25031-10-21-1998	9	3.62	3.39	3.19	3.07	2.72	2.24	1.42	281	21,778	21,778

The deflection data from the existing FWD files and from the new FWD tests were used to:

- For each soil type, backcalculate layer moduli of flexible and rigid pavements
- Evaluate the variability in roadbed soil MR values along and across the pavement network
- Assess the seasonal effects on roadbed soil MR values

4.2 BACKCALCULATION OF LAYER MODULI

For each existing FWD data file, the test location reference was obtained and the MDOT project files and records were searched to obtain the pavement cross-section data that existed at the time when the FWD tests were conducted. All FWD test data where pavement cross-section data were not found were eliminated from further analyses.

Each deflection basin in the remaining and new FWD data files was examined for possible irregularities by plotting the pavement surface deflections as a function of distance from the center of the applied load as shown in Figure 4.2. Irregular deflection basins were removed and stored in different data files and were not included in the backcalculation of layer moduli. For some FWD data files, as much as 75% of the deflection basins were irregular while others didn't contain any irregular basins.

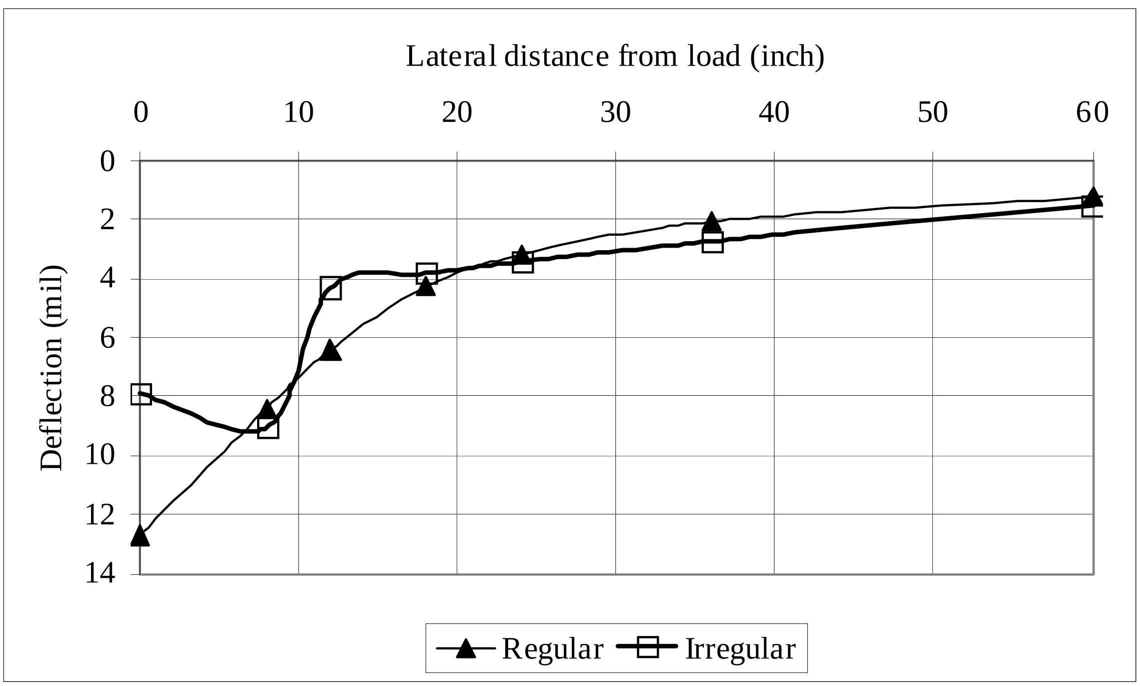


Figure 4.2 Regular and irregular deflection basins

4.2.1 Flexible Pavement

For flexible pavements, the deflection data were used along with the appropriate pavement cross-section data to backcalculate the pavement layer moduli using the MICHBACK iterative computer program. The program, which was developed at Michigan State University, uses the Chevronx computer program (a five layer elastic program) as the forward engine to calculate the pavement deflections for a given set of layer moduli, Poisson ratios, layer thicknesses, and load magnitude. The MICHBACK program utilizes a modified Newtonian algorithm to calculate a gradient matrix by incrementing the estimated layer modulus values and calculating the differences between the measured and the calculated pavement deflection in three consecutive cycles. When the convergence criteria (specified by the program user) are satisfied, the iteration process stops and the final set of backcalculated layer moduli are recorded. In this study, the following convergence criteria were used:

1. Modulus Tolerance - Maximum modulus tolerance (the difference between two successive backcalculated modulus values) of 0.2 percent.
2. Root Mean Square (RMS) error - Maximum RMS error tolerance (the square root of the sum of squared errors between measured and calculated deflections) of 0.2 percent.

The MICHBACK is a user-friendly computer program. The program was used with some of the available default values (such as Poisson's ratios for the various pavement layers) when appropriate. The sensitivity of the backcalculated layer moduli using the MICHBACK computer program to some of the input parameters is presented in the subsection 4.2.1.1. Results of the backcalculation are presented and discussed in subsection 4.2.1.2.

4.2.1.1 Sensitivity of the Backcalculated Moduli

The output of the MICHBACK computer program is sensitive to some of the inputs used in the backcalculation procedure. Several MICHBACK computer program sensitivity analyses were conducted by forward calculating pavement response to applied loads with the Chevronx computer program and then backcalculating layer moduli, from the calculated deflection, with the MICHBACK computer program. The error between the layer moduli used in forward calculation and the backcalculated layer moduli were than studied. The analyses are discussed in this subsection.

Number of Layers - In all backcalculation of layer moduli of flexible pavements, a two layer and roadbed soil system was used. The reason is that the objective of the backcalculation is to determine the roadbed modulus only. The moduli of the asphalt, aggregate base, and sand subbase layers were not included in this study. Hence, the aggregate base and sand subbase layers were combined into one granular base layer. This significantly decreased the number of iterations required to satisfy the convergence criteria, and yet yielded smaller error in roadbed modulus values. This procedure was tested by using forward calculation of pavement response to applied loads and backcalculating the layer moduli. It should be noted that a typical flexible pavement section, in the State of Michigan, consists of three layers (asphalt, aggregate base, sand subbase) and the roadbed soil, and the MICHBACK program is capable of handling a total of five layers, including the roadbed soil. However, the accuracy of the backcalculated moduli of a five layer system is questionable. Figure 4.3 illustrates the effects of using three and four layered systems on the value of the backcalculated layer moduli when combining the base and subbase layers. As can be seen in the figure, the error in the MR values of the roadbed soil remain near zero when a single granular base layer is used. Therefore, the base/subbase combination is appropriate when backcalculating roadbed soil MR values.

Pavement Layer Thickness - The thickness of the pavement layers used in backcalculation can have a significant impact on backcalculated MR values; especially for the AC layer. Constant pavement layer thickness is used for each layer in the backcalculation of layer moduli. However,

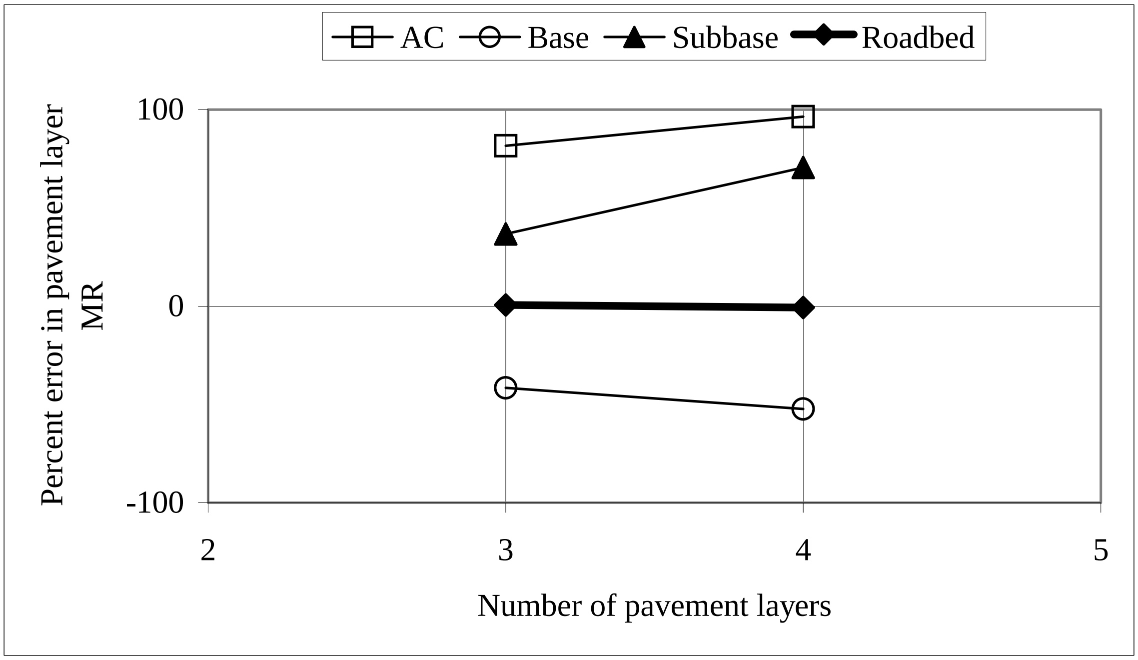


Figure 4.3 Effect of number of pavement layers

due to construction practices the AC thickness may vary +/- 1 inch from the average. Figure 4.4 shows that when the AC thickness is varied, to reflect possible conditions, the backcalculated AC MR values are drastically affected, while the error in the other layers remain generally constant. The roadbed soil MR values are more or less unaffected by changes in the AC layer thickness.

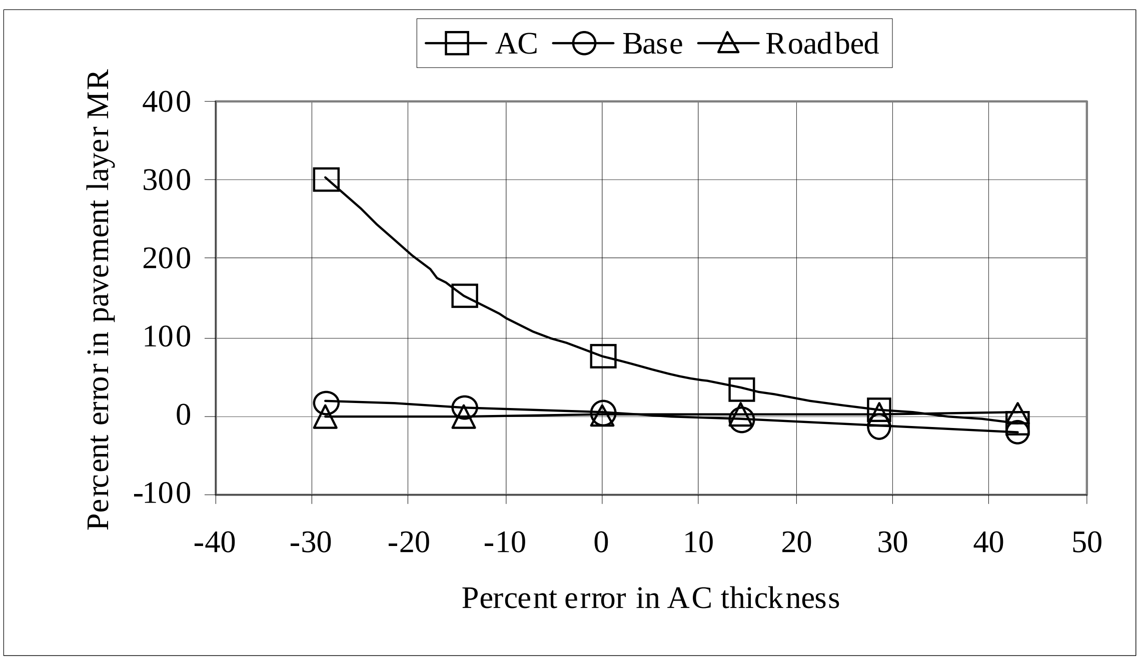


Figure 4.4 Effect of AC layer thickness on MR values

Similarly, Figure 4.5 shows that varying base thickness does not have much affect on the backcalculated roadbed soil MR values. However, the backcalculated MR values of the base and AC layers are affected by varying the base thickness.

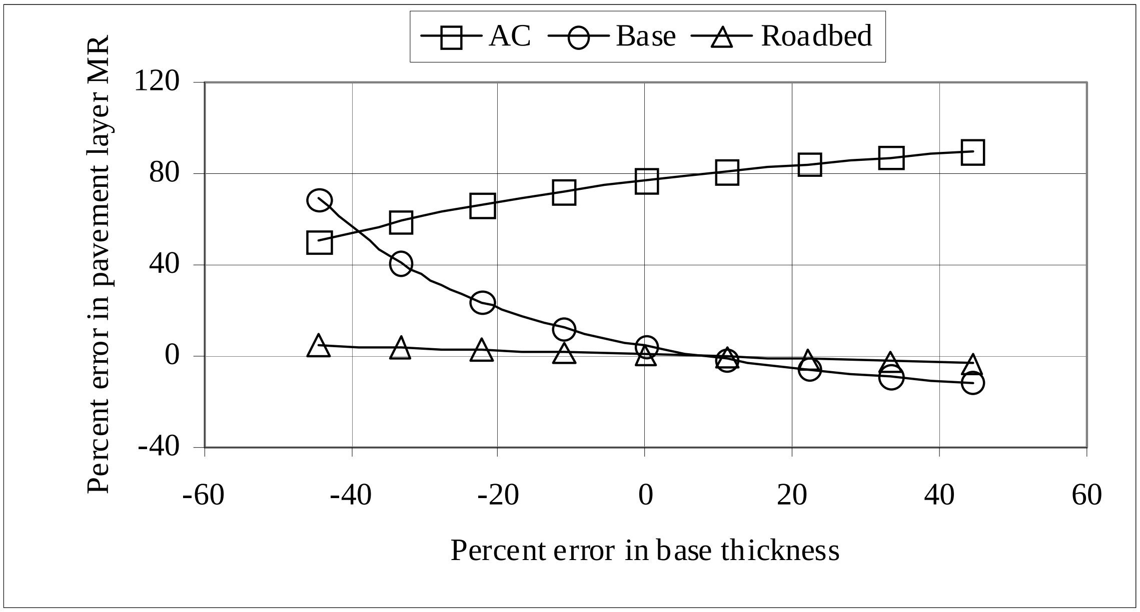


Figure 4.5 Effect of base layer thickness on MR values

Stiff Layer - The effects of stiff layer depth are accounted for in the MICHBACK computer program. In the analyses, the depth to stiff layer was estimated using Equations 4.1 and 4.2 of the Boussinesq equivalent modulus procedure.

$$E_o(0) = 2\frac{(1-\mu^2)\sigma_0 \times a}{d(0)} \qquad \text{Equation 4.1}$$

$$E_o(r) = \frac{(1-\mu^2)\sigma_0 \times a^2}{r \times d(r)} \qquad \text{Equation 4.2}$$

Where, $E_o(r)$ = surface modulus at a distance r (in) from the center of the FWD loading plate (psi), μ = Poisson's ratio (0.5 assumed), σ_o = contact stress under the loading plate (82 psi), d(r) = deflection (mil) at a distance r (in), and a = radius of loading plate (5.91 inch)

By calculating E_o for each sensor in a deflection basin and plotting them against the distance between the sensor and the load, four possible outcomes may occur. Examples of the four outcomes are listed below and shown in Figures 4.6 through 4.9.

a) No stiff layer exists
b) A stiff layer at a shallow (~ 100 inch) depth exists
c) A stiff layer at a deep (~ 400+ inch) location exists
d) A soft layer at a deep (~ 400+ inch) location exists

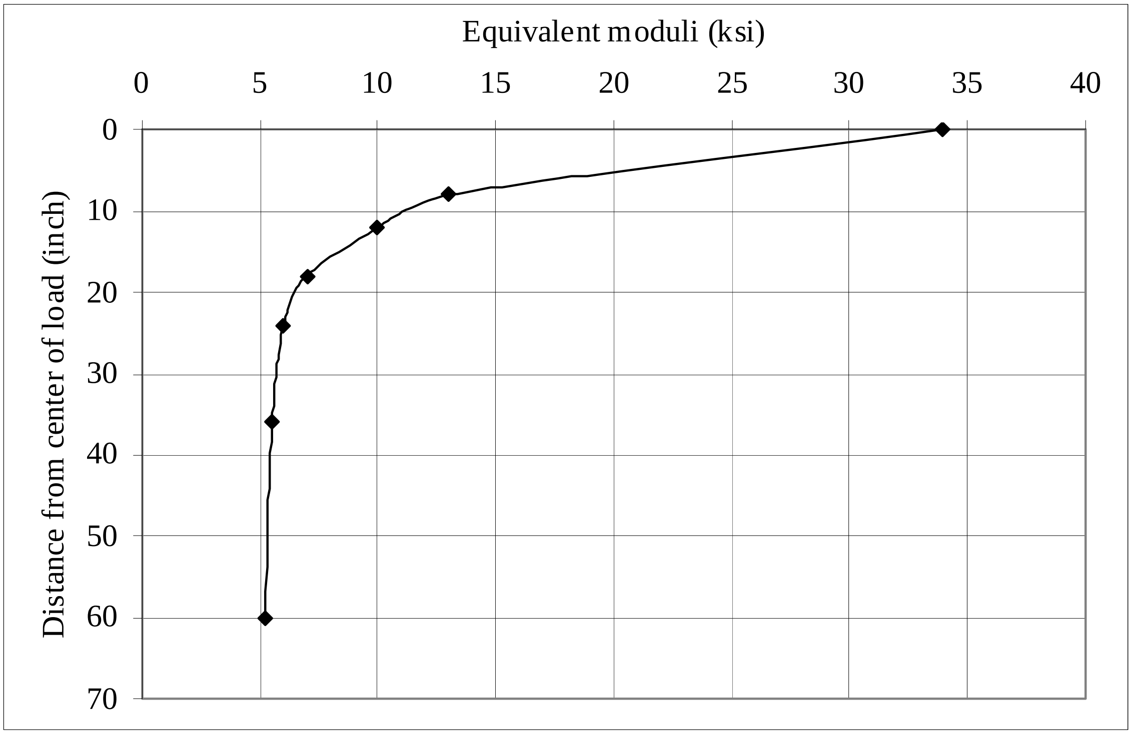


Figure 4.6 No stiff layer

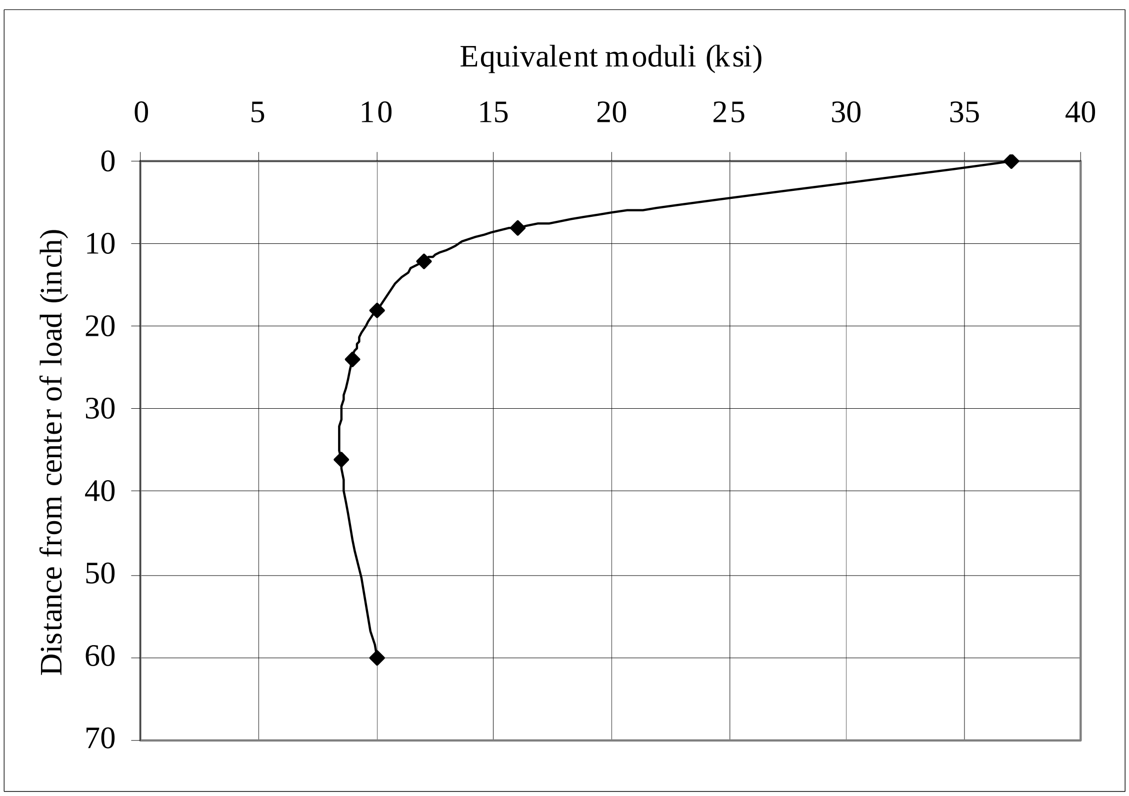


Figure 4.7 Stiff layer at shallow depth

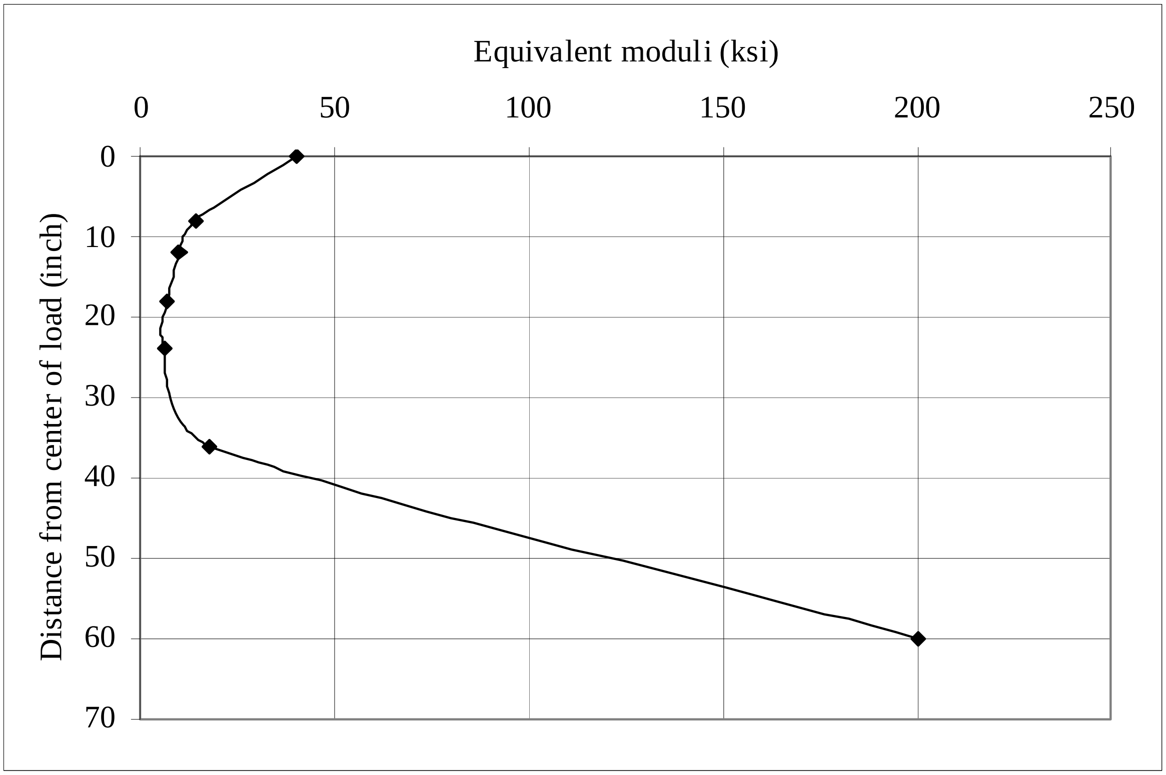


Figure 4.8 Stiff layer at deep location

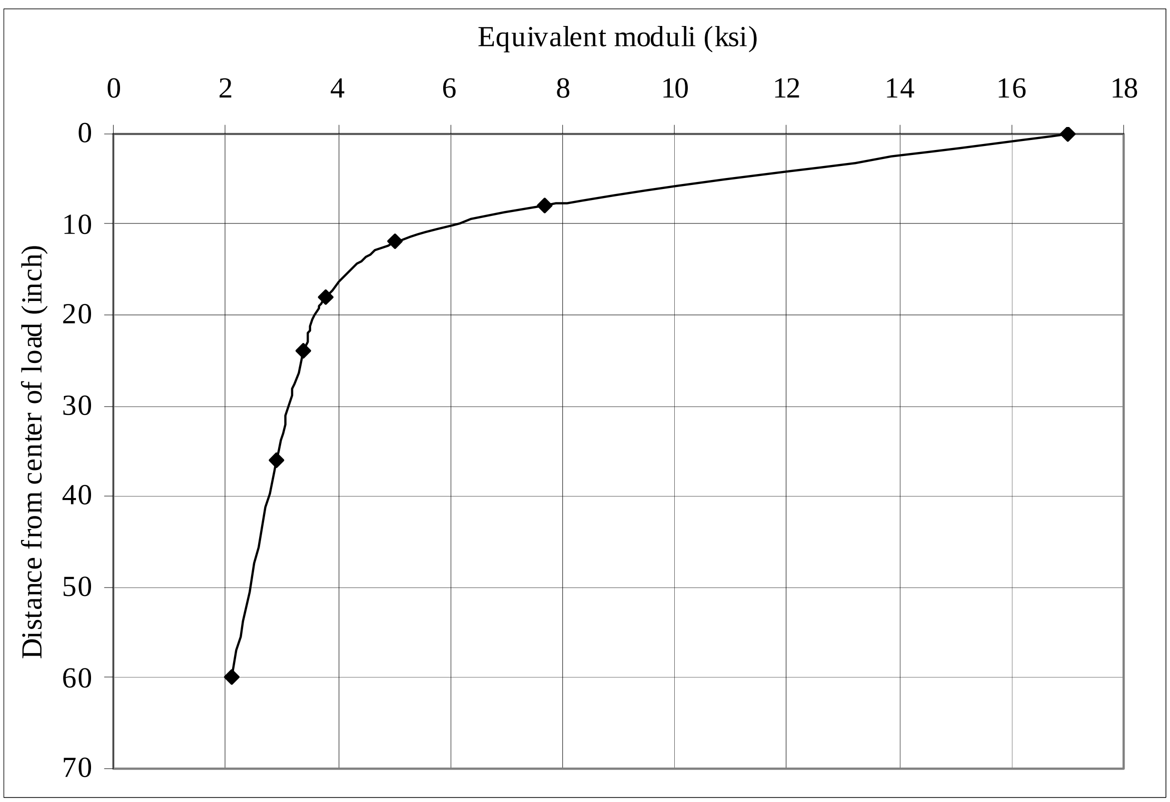


Figure 4.9 Soft layer at deep location

Based on the Boussinesq procedure, the depth to stiff layer is estimated and then changed incrementally to minimize the root mean square error between the measured and the calculated deflections. If the depth to stiff layer used in the backcalculation is not relatively close to the actual depth, the MR values of the roadbed soil can be greatly affected. This procedure was tested by using forward calculation of pavement response to applied loads and backcalculating the layer moduli. Figure 4.10 illustrates the effects of errors in the estimated depth to stiff layer on the backcalculated MR values for four true depths to stiff layer (100, 300, 500 and 700-inch). It can be seen that negative errors in the estimates (shallower estimated depths) cause negative errors (decreases) in the MR values and visa versa.

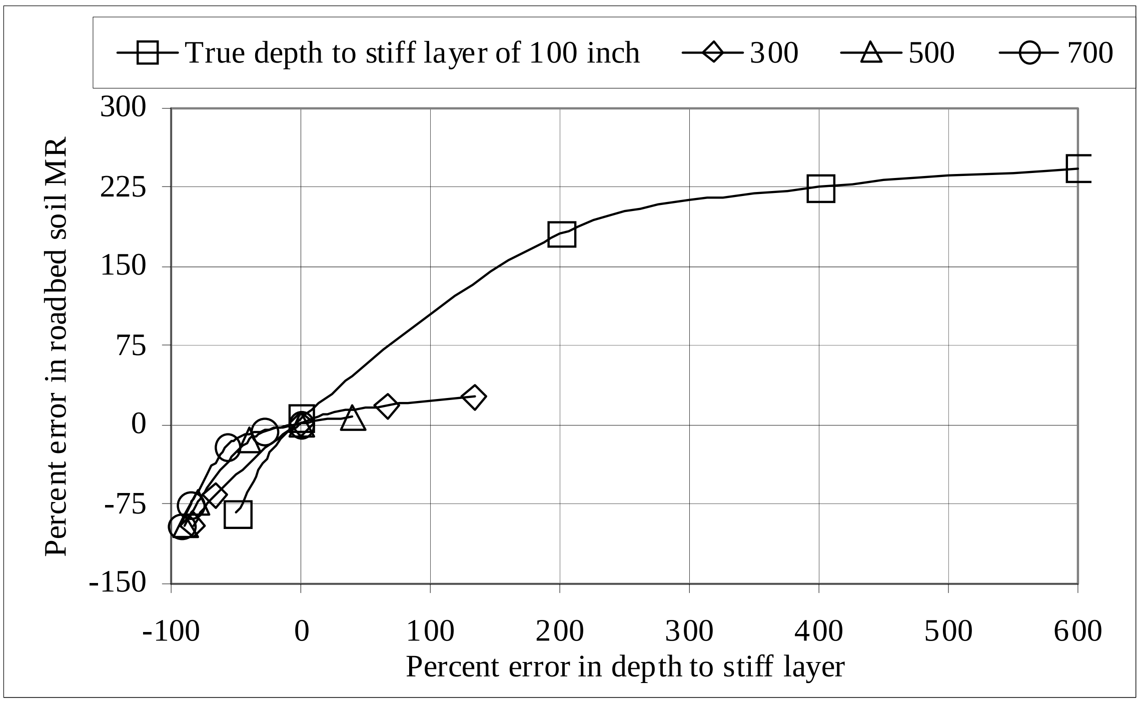


Figure 4.10 Effect of stiff layer depth

Figure 4.11 illustrates that the MR values of the stiff layer have almost no effect on the backcalculated layer moduli. To be considered a stiff layer the MR value must be several hundred thousand psi, and anything more stiff has nearly the same effect.

Roadbed Soil Seed Modulus - The MICHBACK begins its iterative process with a seed MR value for each layer. Figure 4.12 shows that variation in the roadbed seed modulus does not have much impact on the backcalculated MR values. However, the range of MR values specified is important, as the values must be within a reasonable range for each pavement layer. The minimum, seed, and maximum MR values used in this study were:

- AC = (minimum = 100,000, seed = 1,000,000, maximum = 4,000,000 psi)
- Base = (minimum =10,000, seed = 50,000, maximum = 500,000 psi)
- Roadbed = (minimum = 3,000, seed = 7,500, maximum = 100,000 psi)

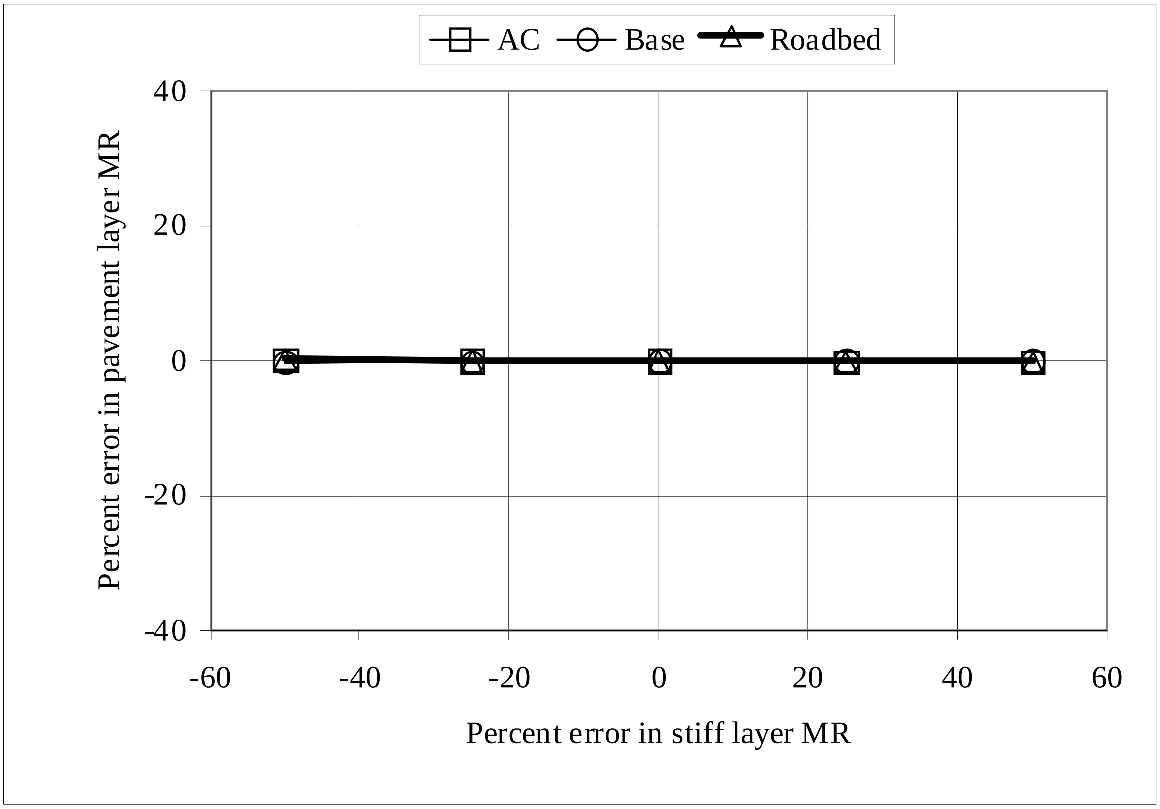


Figure 4.11 Effect of stiff layer MR values

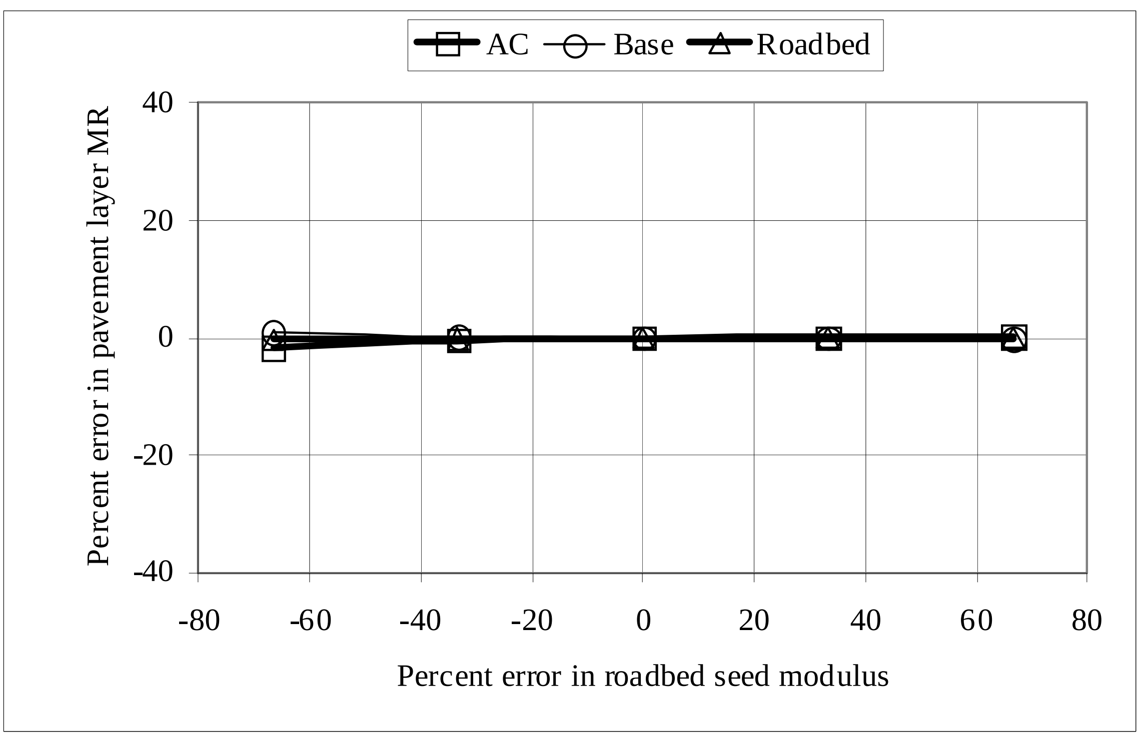


Figure 4.12 Effect of roadbed seed MR values

4.2.1.2 Analysis of Backcalculated Data from MICHBACK

The accuracy of the backcalculated results were verified in the following ways:

- After deflection data were backcalculated using the MICHBACK the data was scrutinized to make sure that all results with greater than 2% RMS error were eliminated. A maximum RMS error of 2% was established for acceptance of the backcalculated MR values because errors above this threshold are much less accurate.
- The deflection measured at the sensor 60 inch (d_{60}) from the load most closely corresponds to the deflection of the roadbed soil. This is due to the arching effects of soil as stress is distributed downward and away from an applied load. The deflection measured at sensors closer to the load (36 inch (d_{36}) and less) are not as closely related to the MR values of roadbed soils. This is illustrated in Figure 4.13 where the open triangles represent the backcalculated MR values versus the measured deflection at d_{60} and the open squares represent the deflection measured at d_{36}. The R^2 of the correlation between MR values and d_{60} is much greater than that of d_{36}, as can be seen in the figure. Due to this relationship, the accuracy of the MICHBACK results can be scrutinized based on the strength of the correlation between d_{60} and the MR values of roadbed soils.
- The deflection measured at the sensor 60 inch from the load is inversely proportionate to the backcalculated roadbed soil MR values. An increase in measured deflection corresponds to a decrease in backcalculated MR values and vise versa, as illustrated by Figure 4.14. Due to this relationship, the accuracy of the MICHBACK results can be scrutinized based on an observation of this trend.

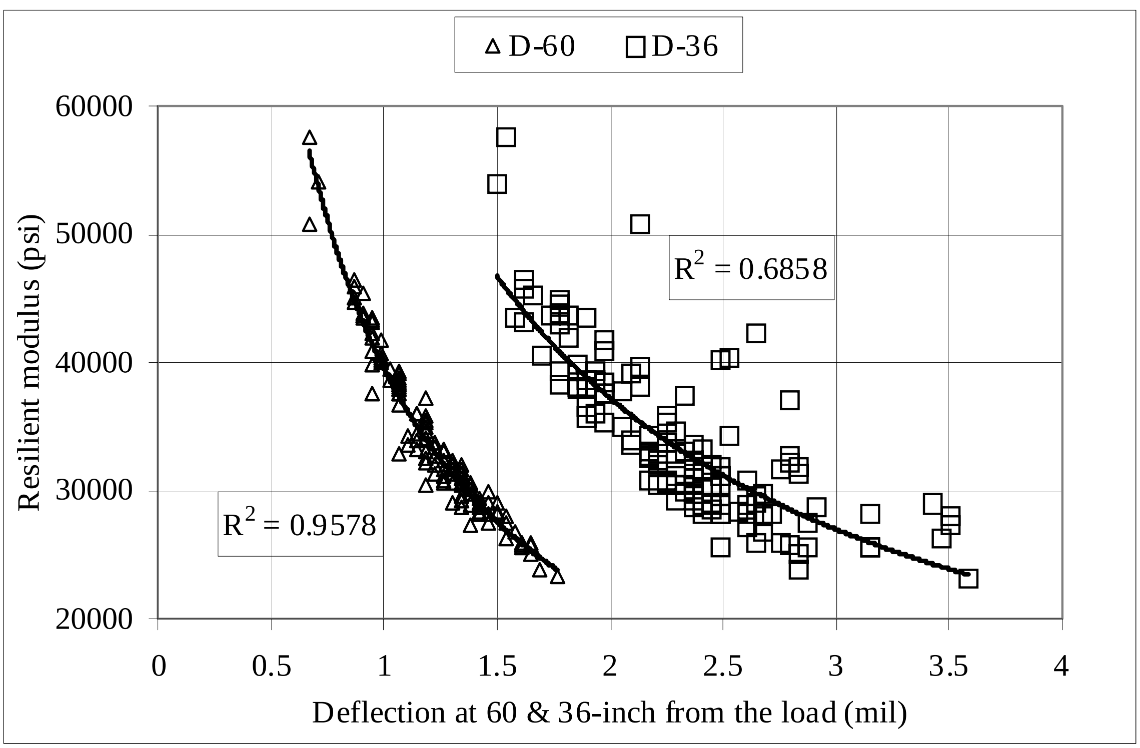


Figure 4.13 Example MR values versus d_{60} and d_{36}

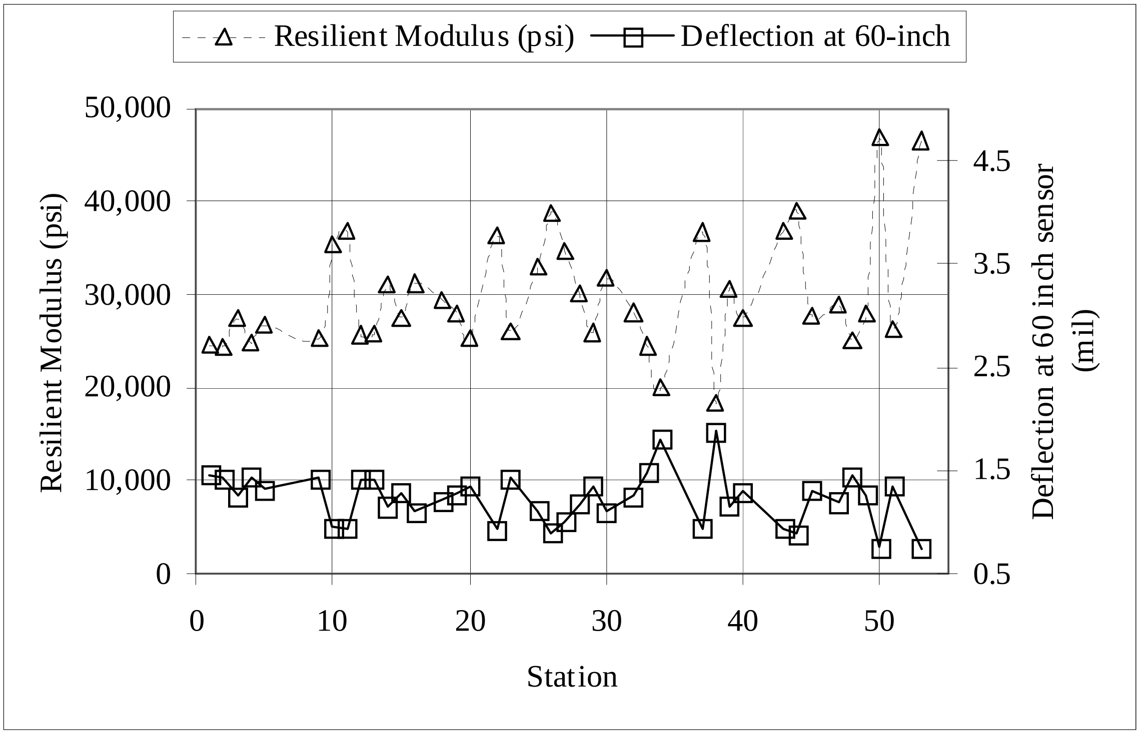


Figure 4.14 MR values versus deflection

Results - Only the backcalculated MR values of roadbed soil were further analyzed. The raw results of base/subbase and AC MR values are listed in Table E.4 of Appendix E (on the accompanying CD).The average, maximum, minimum, and standard deviation of MR values of backcalculated roadbed soil supporting flexible pavements are listed in Table 4.5.

Table 4.5 Backcalculated roadbed soil MR values supporting flexible pavement

Roadbed type USCS	MR results (psi)			
	Average	Maximum	Minimum	Std. dev.
SM	22,976	32,319	16,115	3,373
SP1	30,707	70,138	13,154	7,562
SP2	23,042	28,602	19,243	3,036
SP-SM	21,292	30,666	15,623	3,740
SC-SM	18,734	31,218	7,088	5,847
SC	24,704	67,793	11,728	6,695
CL	22,226	32,804	4,640	6,539
ML	15,976	31,279	8,711	6,394

4.2.2 Rigid Pavement

The rigid pavements layer moduli were backcalculated using the measured deflection data and the empirical AREA method. The method uses the measured deflection at 7 sensors and Equation 4.3 to estimate the parameter "AREA", Equation 4.4 to calculate the radius of relative stiffness (*l*) of the concrete slab, Equations 4.5 and 4.6 to calculate the elastic modulus of the concrete (E_c), and Equation 4.7 to calculate the modulus of subgrade reaction (k) which can be converted into MR value using Equation 4.8 (AASHTO 1993).

$$AREA = \left[4 + 6\left(\frac{\delta_8}{\delta_0}\right) + 5\left(\frac{\delta_{12}}{\delta_0}\right) + 6\left(\frac{\delta_{18}}{\delta_0}\right) + 9\left(\frac{\delta_{24}}{\delta_0}\right) + 18\left(\frac{\delta_{36}}{\delta_0}\right) + 12\left(\frac{\delta_{60}}{\delta_0}\right)\right] \qquad \text{Equation 4.3}$$

$$l = \left[LN\left(\frac{60 - AREA}{289.708}\right) / (-0.698)\right]^{2.566} \qquad \text{Equation 4.4}$$

$$\delta_r^* = a \exp^{\left[-b \exp^{(-cl)}\right]} \qquad \text{Equation 4.5}$$

$$E_c = \frac{12(1-\nu^2) P l^2 \delta_r^*}{\delta_r h^3} \qquad \text{Equation 4.6}$$

$$k = \frac{E_c h^3}{12(1-\nu^2) l^4} \qquad \text{Equation 4.7}$$

$$MR = 19.4k \qquad \text{Equation 4.8}$$

Where, AREA = deflection basin area (inch), δ_r = deflection of the r^th sensor (inch), *l* = radius of relative stiffness (inch), E_c = elastic modulus of the concrete (psi), υ = Poisson's ratio for concrete = .15, *P* = FWD load (pound), δ_r^* = non-dimensional regression coefficient at distance "r (inch)", *h* = concrete slab thickness (inch) (use 9" if unknown), *a ,b, and c* = regression coefficients (see Table 4.6), *k* = modulus of subgrade reaction (pci), and MR = resilient modulus (psi)

Table 4.6 Regression coefficients for δ_r^* (Smith et al 1997)

Radial distance, r (inches)	a	b	c
0	0.12450	0.14707	0.07565
8	0.12323	0.46911	0.07209
12	0.12188	0.79432	0.07074
18	0.11933	1.38363	0.06909
24	0.11634	2.06115	0.06775
36	0.10960	3.62187	0.06568
60	0.09521	7.41241	0.06255

Equation 4.8 was developed based on k values backcalculated from plate load bearing tests. The tests were conducted to simulate a pavement system where the slab is placed directly on top of the subgrade. The FWD tests in this study were conducted on a pavement system consisting of concrete slabs, granular base/subbase and roadbed soil. When Equation 4.8 was used, the resulting MR values were substantially lower than the backcalculated resilient modulus of the same roadbed soils under flexible pavements. Hence, Equation 4.8 was modified by adding a correction factor (CF), as a multiplier, as shown in Equation 4.9.

$$MR = (CF)19.4k \qquad \text{Equation 4.9}$$

The value of the correction factor (CF) of Equation 4.9 was estimated using the three step procedure enumerated below.

I. In the first step, Figure 4.15 was used to estimate the values of the modulus of subgrade reaction (k) corresponding to California Bearing Ratio (CBR) values from 1 to 100. The estimates were then plotted and the best fit curve and equation were obtained as shown in Figure 4.16 and stated in Equation 4.10.

$$k = 51.495(CBR)^{0.5835} \qquad \text{Equation 4.10}$$

II. In this step, Equation 4.11 (a known correlation between MR and CBR values) was divided by Equation 4.10, which resulted in Equation 4.12 as follows:

$$MR = 1500(CBR) \qquad \text{Equation 4.11}$$

$$\frac{MR}{k} = \frac{1500}{51.495}\frac{CBR}{CBR^{0.584}} = 29.13(CBR)^{0.41} \qquad \text{Equation 4.12}$$

III. Since the CBR value of each roadbed soil type in the State of Michigan is not known, an average value of 11 (MR of 16,500 psi, which is slightly lower than the average backcalculated or the average laboratory measured MR values) was assumed. Substituting CBR of 11 in Equation 4.12, arranging terms, and substituting in Equation 4.9, yielded Equation 4.13, which was used throughout this study for the backcalculation of roadbed modulus under concrete pavements.

$$MR = (CF)(19.4)(k) = (29.13)(2.67)(k) = (4)(19.4)(k) \qquad \text{Equation 4.13}$$

Analysis of Backcalculated Data from the AREA Method -All deflection basins which had a d_0 of 10 mils or greater were not included in the analyses. This threshold was set because rigid pavements FWD tested at mid-slab should not experience more than 10 mils of deflection under the center of a 9,000 pound load.

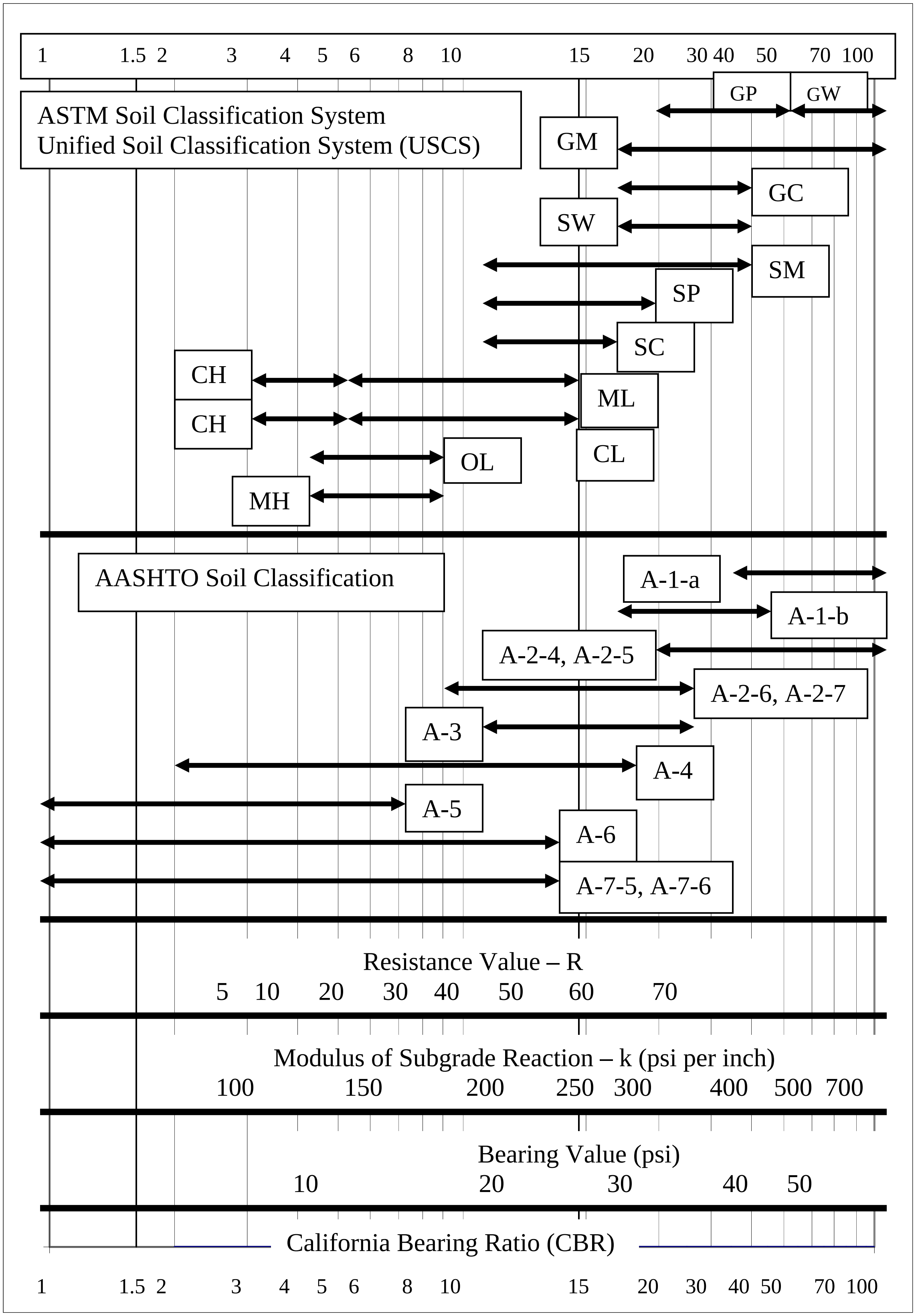


Figure 4.15 Soil classification related to strength parameters (NHI 1998)

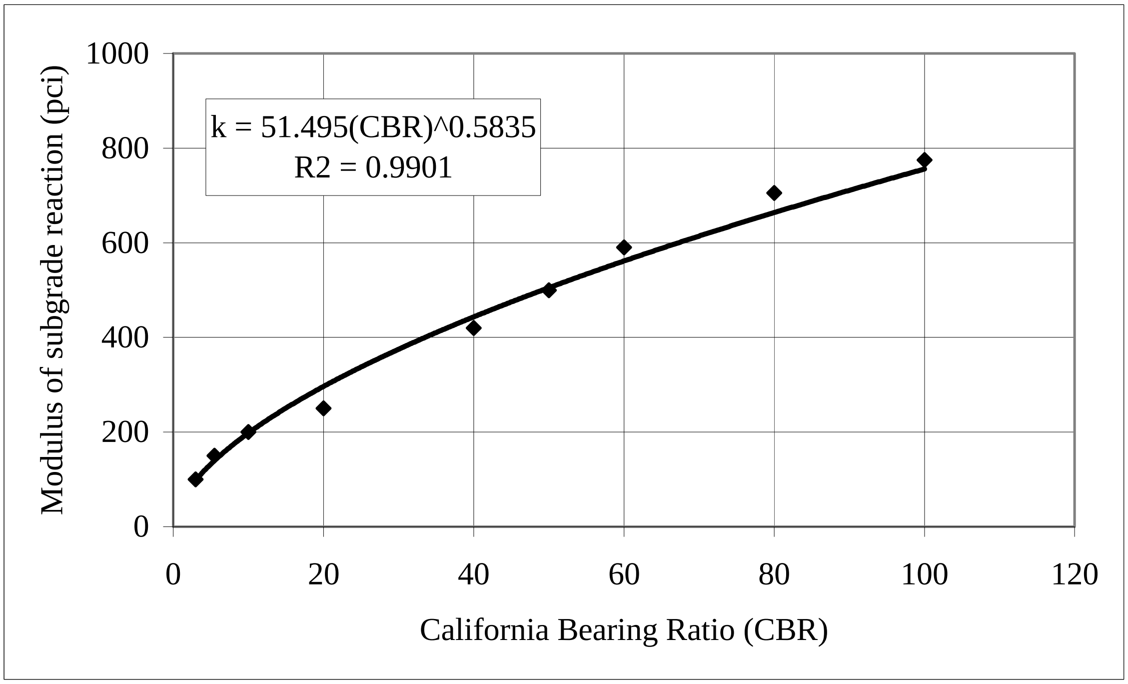


Figure 4.16 Modulus of subgrade reaction versus California Bearing Ratio (after NHI 1998)

Results - Only the backcalculated results of roadbed soil MR values were further analyzed. The average, maximum, minimum, and standard deviation of MR values of backcalculated roadbed soil supporting rigid pavements are listed in Table 4.7, and the detailed results are listed in Table E.5 of Appendix E (on the accompanying CD). It should be noted that no ML soils supporting rigid pavements were FWD tested.

Table 4.7 Backcalculated roadbed soil MR values supporting rigid pavement

Roadbed type USCS	MR results (psi)			
	Average	Maximum	Minimum	Std. dev.
SM	25,306	55,200	4,537	8,944
SP1	20,731	37,209	11,811	4,240
SP2	25,393	41,941	9,495	7,364
SP-SM	20,317	38,035	10,226	5,879
SC-SM	20,350	47,655	3,875	6,613
SC	20,578	35,830	7,462	6,050
CL	14,295	37,358	4,304	5,636
ML	-	-	-	-

4.3 COMPARISON BETWEEN BACKCALCULATED RESILIENT MODULUS VALUES OF ROADBED SOILS SUPPORTING FLEXIBLE AND RIGID PAVEMENTS

The resilient modulus (MR), for a given soil classification, is a fundamental soil property reflecting its response to the applied stresses. For a given soil type, density and water content, its resilient modulus is more or less constant and independent of the pavement type (flexible or rigid) and the types of pavement layers. The MR values of roadbed soils are dependent only on the soil type, water content, dry density, particle gradation, Atterberg limits, and stress states. The roadbed soil response to load is dependent on the stress level applied to the roadbed soil, which is a function of the thicknesses (not type) of the pavement layers. For each soil classification, the average values of the backcalculated MR of the roadbed soils supporting flexible and rigid pavements as well as the average between flexible and rigid pavements are listed in Table 4.8. The average value was calculated by giving each NDT conducted equal weight, as opposed to simply using the average between flexible and rigid pavements. The number of NDT for each pavement and soil type is also given in the table. Please note that no NDT were conducted on rigid pavements supported on ML roadbed soils. The reason is that, for most cases, the ML soils are removed and replaced by another soil type.

The average ratio of backcalculated roadbed soil MR values supporting flexible pavements to rigid pavements was 1.15. The distribution of this ratio by soil type can be seen in Figure 4.17. As indicated by Figure 4.17, for all soil types except the SP1 and CL roadbed soils, the backcalculated resilient modulus is roughly the same regardless if the soils are supporting flexible or rigid pavement sections. This was expected because, for the same soil classification, the resilient modulus is a fundamental soil property reflecting its response to the applied stresses. Such a response is dependent on the stress level applied to the roadbed soil, not the type of the pavement layers. For the SP1 roadbed soils, the flexible pavement sections that were FWD tested are located mainly on the western side of the state where the sand deposit varies from more than 500 feet in the Cadillac area to about 200 feet in the Grand Rapids area. On the other hand, the SP1 roadbed soils that are found under rigid pavement are located along I-75 in the Upper Peninsula and the northern part of the Lower Peninsula of the State of Michigan where the bedrock is located at shallow depths (in some locations rock outcrop can be seen on both sides of I-75). The significant point is that, the algorithm of the AREA method does not account for a shallow stiff layer or bedrock.

The 1993 AASHTO pavement design guide suggests modifying k values when a stiff layer is present within ten feet from the pavement surface. Figure 4.18 depicts the modified k values due to three stiff layer depths versus the k values for an infinite stiff layer depth and the equation of each trend line. The data in the figure were developed based on the 1993 AASHTO Guide for Design of Pavement Structures. The noteworthy observation is that the affect of a stiff layer on the k values increases as the depth to stiff layer decreases. The implication of this is that the backcalculated k values for rigid pavements are artificially low for those cases where the stiff layer is located at shallow depths; the AREA method assumes an infinite depth to stiff layer.

The difference between the backcalculated MR values of the SP1 roadbed soils supporting flexible and rigid pavements is mainly related to the effects of the depths to stiff layer. To account for the presence of a shallow stiff layer under the rigid pavements supported by SP1 soil the equations shown in Figure 4.18 were utilized to modify the average MR value for SP1 soil supporting rigid pavement sections. Two and five foot depth to stiff layer were assumed and the average resultant was 30,303 psi. This results in the ratio between backcalculated roadbed soil MR values supporting flexible pavements to rigid pavements of 1.15.

Table 4.8 Backcalculated roadbed soil MR values supporting flexible and rigid pavements

Roadbed type USCS	Pavement Type	Number of NDT	MR results (psi)				Ratio (flexible/ rigid)
			Average	Maximum	Minimum	Std. dev.	
SM	Flexible	86	22,976	32,319	16,115	3,373	0.91
	Rigid	284	25,306	55,200	4,537	8,944	
	Combined	370	24,764	55,200	4,537	6,715	
SP1	Flexible	1,053	30,707	70,138	13,154	7,562	1.48
	Rigid	446	20,731	37,209	11,811	4,240	
	Combined	1,499	27,739	70,138	11,811	6,573	
SP2	Flexible	67	23,042	28,602	19,243	3,036	0.91
	Rigid	496	25,393	41,941	9,495	7,364	
	Combined	563	25,113	41,941	9,495	6,849	
SP-SM	Flexible	31	21,292	30,666	15,623	3,740	1.05
	Rigid	333	20,317	38,035	10,226	5,879	
	Combined	364	20,400	38,035	10,226	5,697	
SC-SM	Flexible	43	18,734	31,218	7,088	5,847	0.92
	Rigid	1,881	20,350	47,655	3,875	6,613	
	Combined	1,924	20,314	47,655	3,875	6,645	
SC	Flexible	393	24,704	67,793	11,728	6,695	1.20
	Rigid	1,124	20,578	35,830	7,462	6,050	
	Combined	1,517	21,647	67,793	7,462	4,931	
CL	Flexible	86	22,226	32,804	4,640	6,539	1.55
	Rigid	688	14,295	37,358	4,304	5,636	
	Combined	774	15,176	37,358	4,304	4,386	
ML	Flexible	23	15,976	31,279	8,711	6,394	-
	Rigid	-	-	-	-	-	
	Combined	23	15,976	31,279	8,711	6,394	
Average							1.15

The difference between the backcalculated MR values of the CL roadbed soils supporting flexible and rigid pavements is mainly related to the time when the tests were conducted. The majority of the FWD tests on rigid pavements supported by CL roadbed soils were conducted while attempting to study the seasonal effects on the pavement strength. The roadbed soil MR values were expected to be low since the data came from attempts to capture low strength conditions. The seasonal effects are discussed in section 4.4.

4.4 SEASONAL EFFECTS

The State of Michigan is located in the AASHTO wet-freeze region. The average annual rainfall and snowfall in the State varies from one location to another. In the Lansing area, the average annual rainfall and snowfall are about 32 and 56 inches, respectively. Further, the frost depth

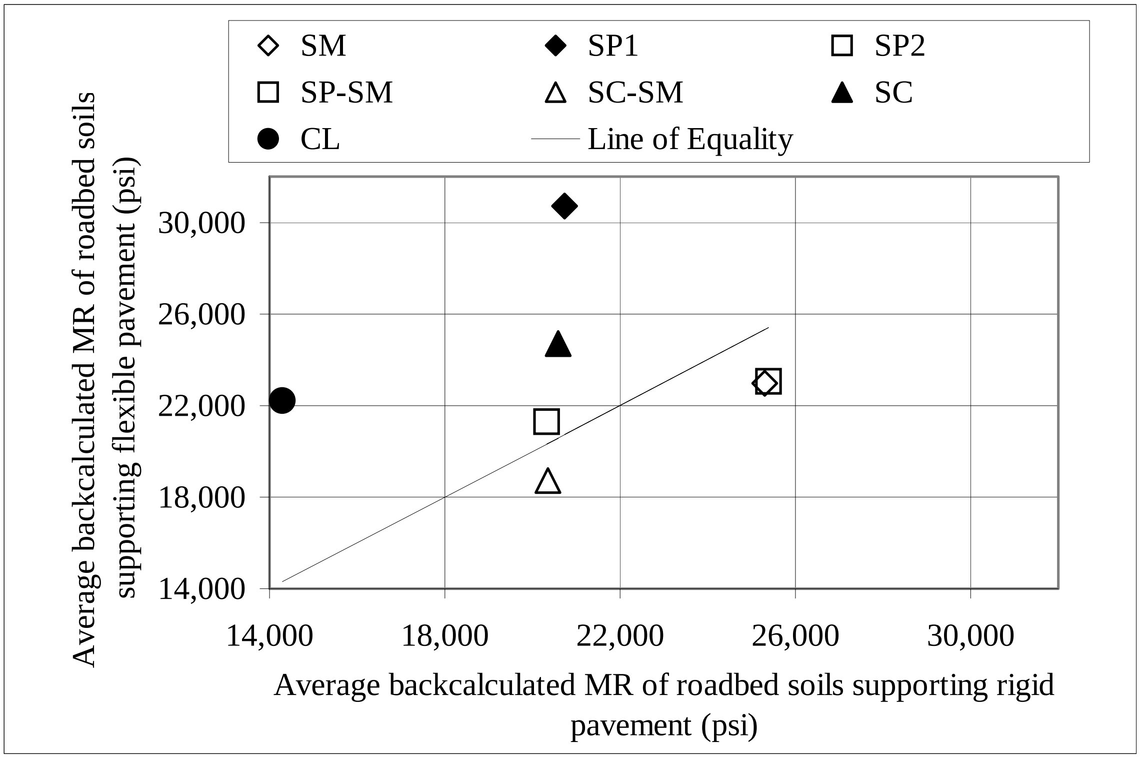


Figure 4.17 Flexible versus rigid backcalculated roadbed soil MR values

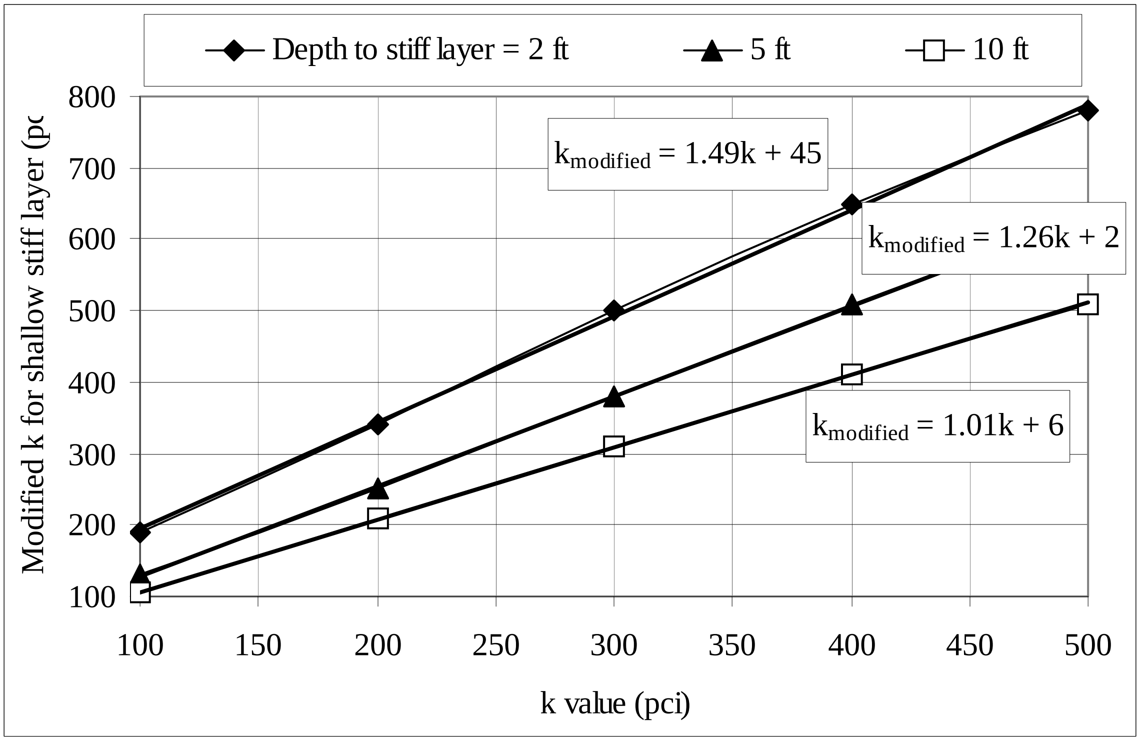


Figure 4.18 Stiff layer effects on backcalculated k

varies from about 7 feet in the Upper Peninsula to about 3 feet in the Lower Peninsula. These climatic data affect the behavior of the paving materials and roadbed soils. Because of the variability of the climatic conditions, the MR of any given soil is dynamic in nature and it changes seasonally with changing water content and temperatures below and above freezing.

One of the objectives of this study was to investigate the effects of seasonal variations on roadbed soil MR values. In order to study the effects; FWD tests were to be conducted once in the summer/fall season and once again at the same location during the spring season. The factor between backcalculated roadbed soil MR values during the summer/fall and spring seasons would be the seasonal damage factor. It should be noted that testing could not be conducted at exactly the same locations, due to lack of exact positioning equipment, however the tests were conducted as close as possible to the original test locations. Unfortunately, the spring season was not captured. Only six sets of data represent partial reduction in backcalculated roadbed soil MR values or increases in measured deflection between the two seasonal tests. Table 4.9 lists the average deflection at each sensor and the backcalculated roadbed soil MR values for each set of tests. All tests were conducted and analyzed as discussed in this chapter.

One would expect the measured deflection in the spring to be higher than in the fall. However the data indicates that often the deflection measured at the sensors located closer to the load (i.e. d_0) are less during the spring season than the fall. This is likely due to the fact that the pavement is saturated. Water is incompressible, and therefore when the load is applied to the roadbed soil it reflects some of the stress back upward toward the pavement surface. In fact, if time sequence deflection measurements were available, it is highly likely that negative deflection may exist. For similar reasons, it should also be noted that the deflection measured at d_{60} often exhibits the greatest difference between the fall and spring tests. This is likely due to the fact that the stress delivered to the roadbed soil is felt for a longer period of time than at the surface. Therefore the roadbed soil can expel some of its water and deflect more.

The data indicates that only partial spring condition was captured. Therefore, no conclusions can be drawn based on the limited data.

Table 4.9 Seasonal effects

Road	Cluster-area	Control section	location	Pavement type	Roadbed soil type	Season	Deflection (mil) at sensor located (in) from load							Roadbed soil MR (psi)
							0	8	12	18	24	36	60	
I-75	14-01	58151	50 ft North of Mile Marker 7	PCC	SC	Fall	4.62	4.08	3.98	3.80	3.55	3.08	0.79	23,071
I-75	14-01	58151	50 ft North of Mile Marker 7	PCC	SC	Spring	4.59	4.43	4.23	4.04	3.79	3.25	2.21	11,041
Factor (Spring/Fall)							0.99	1.09	1.06	1.06	1.07	1.06	2.80	0.48
M-53	14-08	50015	245 feet north of centerline of 32 mile rd	AC	CL	Fall	8.98	8.00	7.22	6.12	8.05	3.40	1.56	22,259
M-53	14-08	50015	245 feet north of centerline of 32 mile rd	AC	CL	Spring	12.59	10.86	9.51	7.61	6.03	3.53	1.41	22,508
Factor (Spring/Fall)							1.40	1.36	1.32	1.24	0.75	1.04	0.90	1.01
I-69	10-04	23063	Station 1557	PCC	SM	Fall	5.69	4.98	4.74	4.48	4.12	3.53	0.93	23,132
I-69	10-04	23063	Station 1557	PCC	SM	Spring	3.86	3.66	3.50	3.30	3.05	2.58	1.73	16,470
Factor (Spring/Fall)							0.68	0.73	0.74	0.74	0.74	0.73	1.86	0.71

Table 4.9 (Cont'd)

Road	Cluster-area	Control section	location	Pavement type	Roadbed soil type	Season	Deflection (mil) at sensor located (in) from load							Roadbed soil MR (psi)
							0	8	12	18	24	36	60	
US-127	10-02	19034	M-21 North to end of freeway	PCC	SC-SM	Fall	4.19	3.66	3.54	3.35	3.09	2.63	0.68	29,442
US-127	10-02	19034	M-21 North to end of freeway	PCC	SC-SM	Spring	3.58	3.48	3.34	3.15	2.93	2.48	2.13	12,261
Factor (Spring/Fall)							0.85	0.95	0.94	0.94	0.95	0.93	3.13	0.42
I-75	14-01	58151	S of Gaynier Rd. truck scales	PCC	SC	Fall	4.62	4.08	3.98	3.80	3.55	3.08	0.79	23,071
I-75	14-01	58151	S of Gaynier Rd. truck scales	PCC	SC	Spring	4.80	4.67	4.34	4.18	3.87	3.28	2.11	11,640
Factor (Spring/Fall)							1.04	1.14	1.09	1.10	1.09	1.06	2.67	0.50
US-23	04-02	71073	At Beach hwy to the west	AC	SC-SM	Fall	11.64	9.11	7.62	5.96	4.72	3.14	1.76	20,751
US-23	04-02	71073	At Beach hwy to the west	AC	SC-SM	Spring	11.55	9.47	8.18	6.58	5.38	3.62	1.97	18,584
Factor (Spring/Fall)							0.99	1.04	1.07	1.10	1.14	1.15	1.22	0.90

CHAPTER 5

COMPARISON OF LABORATORY AND FWD DATA

5.1 COMPARISON BETWEEN BACKCALCULATED AND LABORATORY DETERMINED RESILIENT MODULUS VALUES

For a given soil classification, the resilient modulus is a fundamental soil property controlling its response to the applied stresses. However, this property changes with changing soil type, water content, dry density, particle gradation, Atterberg limits, and stress states. Therefore, in order to compare the backcalculated and the laboratory measured MR values special care must be taken to match the conditions of the soils in question. In this study, the MICHPAVE computer program was used to analyze the stresses induced in a 25-inch thick pavement section and the roadbed soil due to 9000 pound (half the standard single axle load of 18,000 pounds) simulating the FWD load. It should be noted that, in the analyses, a lateral earth pressure coefficient of 2.0 was used to simulate the locked-in lateral stress due to compaction. The results indicate that the roadbed soil is subjected to 8 psi vertical stress and to about 7.5 psi lateral stress. Hence, all laboratory cyclic load tests were conducted using 7.5 psi confining pressure and 10 and 15 psi vertical axial cyclic stress. These boundary conditions yielded stress ratios of 1.33 and 2.0, respectively. Nevertheless, as stated earlier, for all soil types, the laboratory resilient modulus values obtained from cyclic stress of 10 psi and confining pressure of 7.5 psi were used in the analyses.

For each soil classification, Table 5.1 provides a list of the average MR value obtained in the laboratory and the average backcalculated MR value using the measured deflection data. The two sets of MR values and the line of equality between the two average values are plotted in Figure 5.1.

The data in Table 5.1 and Figure 5.1 indicate that the ratio of the two averages of the MR values for the SP1, SP2, SP-SM, SC-SM, and SC are close to one. Therefore, no conversion is necessary to relate laboratory determined to backcalculated MR values, in satisfaction of objective 7 on page 4. However, the ratios for the other three soil types (SM, CL, and ML) vary from 1.45 to 0.41. These values were expected because:

- For the SM soils, the average laboratory MR values were obtained as the average MR values of soil samples compacted at water contents corresponding to degrees of saturation from about 25 to about 95 percent (which simulate the water contents throughout one year period). However, the FWD tests were mainly conducted in the summer and fall seasons where the water contents of roadbed soils are on the dry side of optimum. Hence, the backcalculated values are expected to be higher than the laboratory obtained values as shown in Table 5.1 and Figure 5.1

- For the CL and ML soils on the other hand, the majority of the laboratory tests were conducted on soil samples that were on the dry side or near the optimum water content. The water contents of only four out of thirteen test samples were near or above the optimum water content, whereas the water contents of the other nine test samples were well below the optimum water content. Therefore, the average laboratory MR values should be expected to be high. Since, the FWD tests were conducted in the spring (the water content of the roadbed soil is near or above the optimum) the backcalculated MR value is relatively low. Hence, the

Table 5.1 Laboratory determined and backcalculated roadbed soil MR values

USCS	AASHTO	Laboratory results		Backcalculation results		Average of backcalculated to average laboratory MR
		Number of tests	Average MR (psi)	Number of tests	Average MR (psi)	
SP1	A-1-a A-3	16	28,942	1,499	27,739	0.96
SP2	A-1-b A-3	10	25,685	563	25,113	0.98
SP-SM	A-1-b A-2-4 A-3	8	21,147	364	20,400	0.96
SC-SM	A-2-4 A-4	7	23,258	1,924	20,314	0.88
SC	A-2-6 A-6 A-7-6	16	18,756	1,517	21,467	1.16
SM	A-2-4 A-4	17	17,028	370	24,764	1.45
CL	A-4 A-6 A-7-6	9	37,225	774	15,176	0.41
ML	A-4	4	24,578	23	15,976	0.65
Average						0.93

average MR value obtained from the laboratory tests is higher than the average backcalculated value.

The two reasons are related to the effects of the moisture content of the test samples on the MR values. To explore such relationship for the ML soils, four cyclic load tests were conducted on ML soils at four different moisture contents. The test results are plotted in Figure 5.2. As can be seen from the figure, increasing the water content from about 11 percent (dry of optimum) to about 24 percent (wet of optimum) causes decreases in the MR value from about 40,000 to less than 2,000 psi. This trend agrees with most results reported in the literature.

Once again, the test results in this research indicate that, if the roadbed soil samples were tested in the laboratory at similar water contents as the field water contents at the time when the FWD tests were conducted, then the ratios of the backcalculated to the laboratory obtained modulus values are close to unity. This finding contradicts those reported in the literature where the ratio between the backcalculated and the laboratory determined MR values vary from almost 1.6 to almost 5.0. The discrepancy between the finding in this study and the literature can be mainly related to the stress boundary conditions used in this study. Most previous studies were conducted using much higher confining pressure (10 to 50 psi) and much lower stress ratio (σ_1/σ_3).

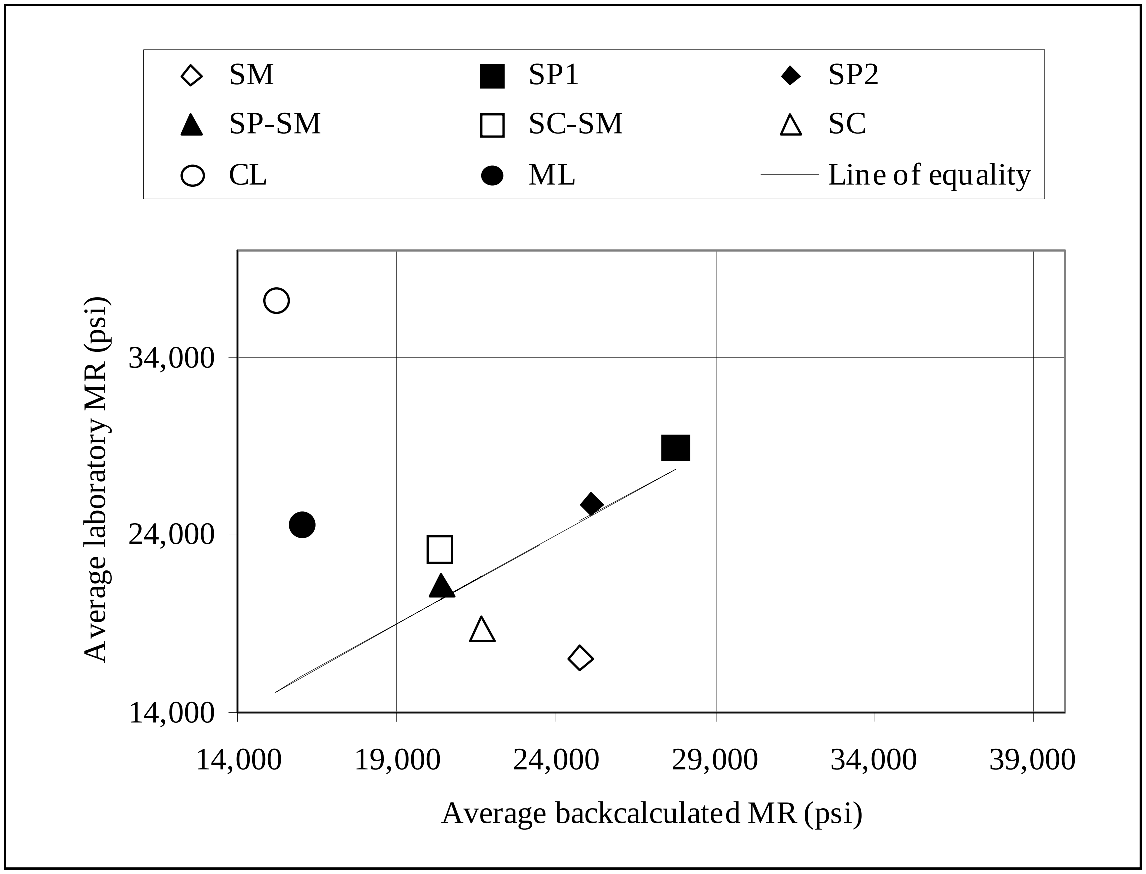


Figure 5.1 Laboratory determined and backcalculated roadbed soil MR values

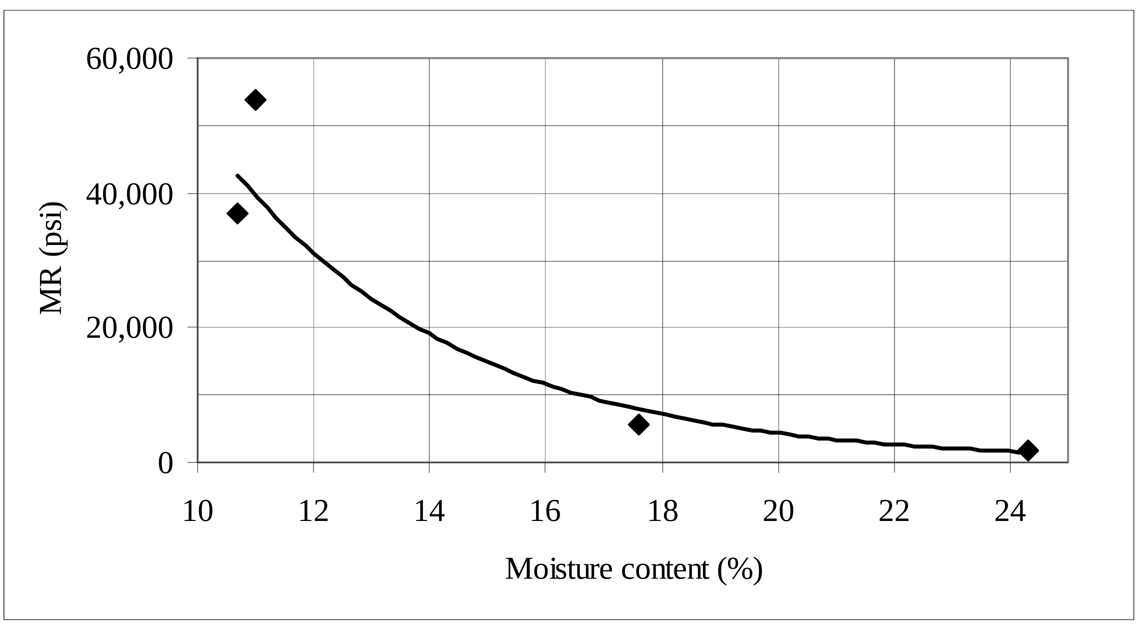


Figure 5.2 Moisture content affect on MR values of ML soils

Most laboratory test data reported in the literature are based on stress ratio (the ratio between the axial cyclic stress and the confining pressure) of 2.0 or higher. As stated at the beginning of this section, two stress ratios were used in the laboratory testing program of this research study, 1.33 and 2.0. However, all analyses were conducted on the resilient modulus values obtained from a stress ratio of 1.33.

Increasing the cyclic stress while keeping the confining pressure at a constant level yields higher stress ratio and lower resilient modulus values. In this study, the effects of the stress ratio on the resilient modulus values were analyzed by conducting tests at different stress ratios. Results of said tests are depicted in Figure 5.3. The figure shows the resilient modulus value as a function of the stress ratio. It can be seen, from the figure, that increasing stress ratios result in lower MR values. This in turn would yield higher ratios between the backcalculated and the laboratory determined MR values. The important point herein is that the resilient modulus test should be conducted at similar boundary conditions as those expected in the field. That is, the applied stresses in the laboratory should resemble those delivered to the roadbed soil due to 9,000 pound load traveling over the pavement section in question. Higher stress ratios should be used when testing the base and subbase materials.

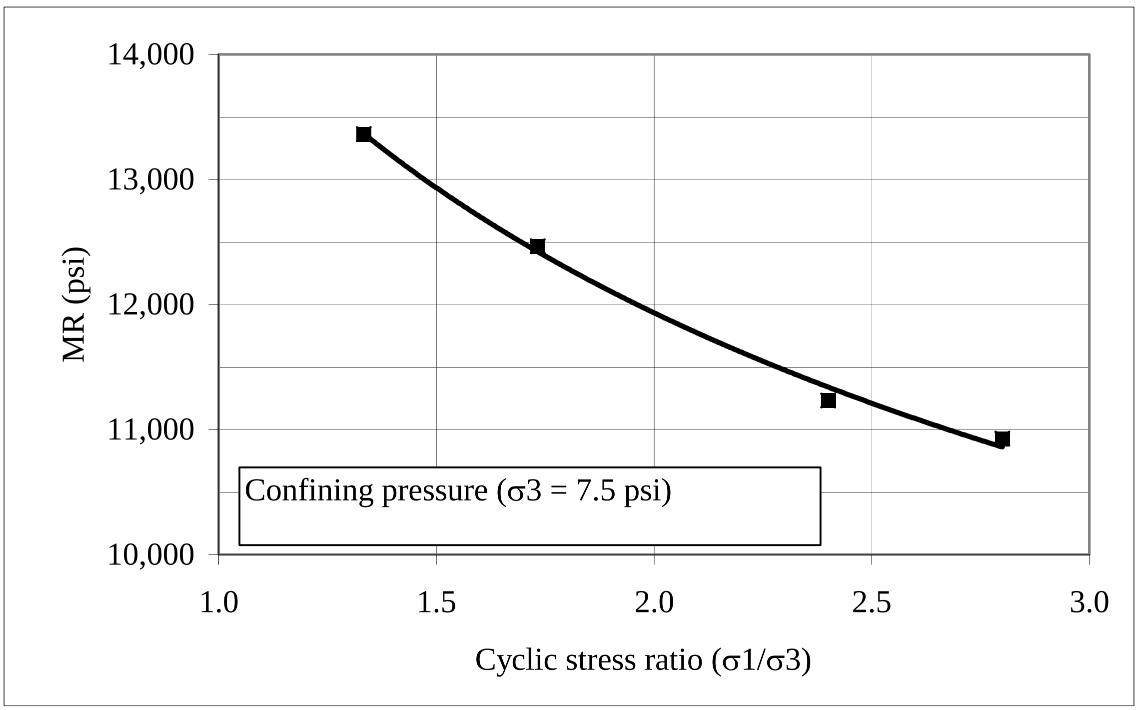


Figure 5.3 Laboratory obtained resilient modulus versus the cyclic stress level

5.2 SEASONAL DAMAGE

The State of Michigan is located in the AASHTO wet-freeze region. The average annual rainfall and snowfall in the State varies from one location to another. In the Lansing area, the average annual rainfall is about 32 inches and the average annual snowfall is about 56 inches. Further, the frost depth varies from about 7 feet in the Upper Peninsula to about 3 feet in the Lower Peninsula. These climatic data affect the behavior of the paving materials and roadbed soils. Because of the variability of the climatic conditions, the resilient modulus of any given soil is dynamic in nature and changes seasonally with changing water content and temperatures fluctuating below and above the freezing point.

The seasonal effects were studied as part of the laboratory and FWD analyses, presented in sections 3.7 and 4.4. However, due to the lack of sufficient spring FWD data, as discussed in section 4.4, only the laboratory analysis was completed successfully. Therefore, the seasonal damage factor developed in section 3.7, will be used in this study. Table 5.2 presents the damage factors calculated as part of the laboratory and FWD analyses. Table 5.3 presents the average roadbed soil MR values for each soil classification and the seasonal damage factor (7) reduced MR values.

It should be noted that the recommended design resilient modulus values listed in Table 5.3 are to be used in the M-E PDG design level 3. For design level 2, the correlation equations should be used. And for design level 1, FWD tests should be conducted and the design modulus value backcalculated.

Table 5.2 Damage factor calculation and design resilient modulus

USCS classification	Water content (%)			Resilient modulus (psi) corresponding to saturation range of			Average MR (psi)	Reduction Factor		Design MR Value (psi)	Damage factor	
	Near optimum	Moderate	High	45-75	75-85	85-100		High/ Low	Medium/ Low		AASHTO	MICHPAVE
SC, CL, ML	15	22	30	30,543	6,879	4,430	5655	6.9	1.55	4,430		
SC-SM	8.5	15	28	27,276	10,000	5,100	7550	5.3	1.96	5,100		
SP-SM	8	15	20	23,009	11,000	7,000	9000	3.3	1.57	7,000		
SM (MI)	12	20	23	18,416	11,480	5,290	8385	3.5	2.17	5,290		
SP1	11	20	25	29,418	11,798	7,100	9449	4.1	1.66	7,100		
SP2	10	20	25	22,768	9,969	6,500	8235	3.5	1.53	6,500		
Average								4.4	1.74		5	4.5
Lightly shaded cells represent moisture contents outside those of the sampled soil type												

Table 5.3 Design resilient modulus values for M-E PDG design level 3

USCS Classification	Design MR (psi)
SC, CL, ML	4,430
SC-SM	5,100
SP-SM	7,000
SM	5,290
SP1	7,100
SP2	6,500

CHAPTER 6

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

6.1 SUMMARY

The resilient modulus of roadbed soil plays an integral role in the design of pavement systems. Currently, the various regions of MDOT use different procedures to determine the MR values. Most of these procedures are applicable to M-E PDG design level 3 only. Therefore, a consistent, uniform, and implementable procedure that meets the requirements of M-E PDG for level 1, 2, and 3 designs, must be developed.

In order to accomplish the objectives of this study and to develop a uniform methodology for estimating the MR of roadbed soil, the State of Michigan was divided into fifteen clusters where the physical and engineering characteristics of the soil were similar. The clusters were then divided into ninety nine areas to narrow down the ranges of the engineering and physical characteristics of the soils. Disturbed roadbed soil samples were collected from seventy five areas, and fourteen undisturbed soil samples (Shelby tubes) were collected from areas with CL and SC roadbed soils. The soil samples were then tested to determine their moisture content, grain size distribution, Atterberg limits (when applicable), and resilient modulus values using cyclic load triaxial tests. Correlation equations were then developed to estimate the MR values of the roadbed soil based on the results of the moisture content, degree of saturation, Atterberg limits, dry unit weight, specific gravity, and grain size distribution data.

Deflection data from FWD tests conducted throughout the state were obtained from MDOT. The test database consisted of hundreds of FWD tests from previous projects spanning the last 20+ years as well as eighty tests conducted as part of this study. All FWD data files where sufficient pavement cross-section data were available, were analyzed to backcalculate the roadbed soil MR values. The backcalculated values were then compared to the laboratory determined MR values.

Although the objectives of this study stated in Chapter 1 were accomplished, the benefits to MDOT will not be materialized until the results of this study are implemented. The risk associated with the implementation of the findings of this study is significantly lower than the risk associated with the existing practice used by the various regions to estimate the MR values.

6.2 CONCLUSIONS

Based on the field and laboratory investigations and the data analyses, the following conclusions were drawn:

1. The Michigan Department of Transportation's (MDOT) current procedure for determining the MR values of roadbed soils is not consistent between the various regions of MDOT.
2. The roadbed soils in Michigan were classified based on the USCS and the AASHTO soil classification systems. In most cases, subgrade soils having similar elastic behavior under loads tend to fall within one classification (designation) of the USCS but within several AASHTO classification groups. Hence, based on the elastic behavior of the subgrade soils in Michigan, the USCS produces much better soil grouping than the AASHTO classification system.

3. Most of the roadbed soils in the State of Michigan can be divided into the following eight soil types:
   - Gravelly sand (SG)
   - Poorly graded sand (SP), which can be divided into two groups SP1 and SP2 based on the percent fine content
   - Silty sand (SM)
   - Poorly graded sand – silty sand (SP-SM)
   - Clayey sand – silty sand (SC-SM)
   - Clayey sand (SC)
   - Low plasticity clay (CL)
   - Low plasticity silt (ML)
4. Grain size distribution tests on soils with more than 10 percent passing sieve number 200 should be performed by a combination of dry and wet sieving.
5. For SC and CL subgrade soils the resilient modulus values obtained from testing undisturbed Shelby tube samples were compatible to those from disturbed samples.
6. An average seasonal damage factor of about 4.5 was calculated using the results of the laboratory tests, the correlation equations, the MICHPAVE computer program and the 1993 AASHTO design procedure. The damage factor accounts for the seasonal effects on the resilient modulus values of the subgrade soils in the State of Michigan.
7. The backcalculated and the laboratory determined MR values, in this study, may be used with the M-E PDG for design levels 1, 2, and 3.
8. Correlation equations between the laboratory obtained resilient modulus values and some of the soil parameters were developed and are summarized in Table 6.1.
9. MR values obtained from the correlation equations listed in Table 6.1 may be used with the M-E PDG for design levels 2 and 3.
10. The design resilient modulus value for each soil type, except the SG, and for the two SP soil groups were developed and are listed in Table 6.2 and presented in Figure 6.1.
11. The MR values in Figure 6.1 may be used with the M-E PDG for design level 3.
12. In general, the backcalculated MR values of roadbed soil are similar to those of the same soil type obtained from triaxial cyclic load laboratory testing.
13. In general, the backcalculated MR values of roadbed soil supporting flexible pavement sections are similar to those of the same soil type supporting rigid pavement sections.
14. The AREA method does not account for the effects of shallow stiff layers.
15. Equation 6.1 should be used when converting k values, backcalculated from the AREA method, to MR of roadbed soils.

$$MR = (4)(19.4)k \qquad \text{Equation 6.1}$$

6.3 RECOMMENDATIONS

Based on the results and conclusions of this study, it is strongly recommended that:

- MDOT implements the findings of this study by using deflection data collected at the project level to backcalculate the resilient modulus of the roadbed soil for use in the M-E PDG design level 1 or in the 1993 AASHTO Design Guide.
- MDOT implements the findings of this study by adopting the correlation models presented in Table 6.1 for M-E PDG design levels 2 and 3 or in the 1993 AASHTO Design Guide.

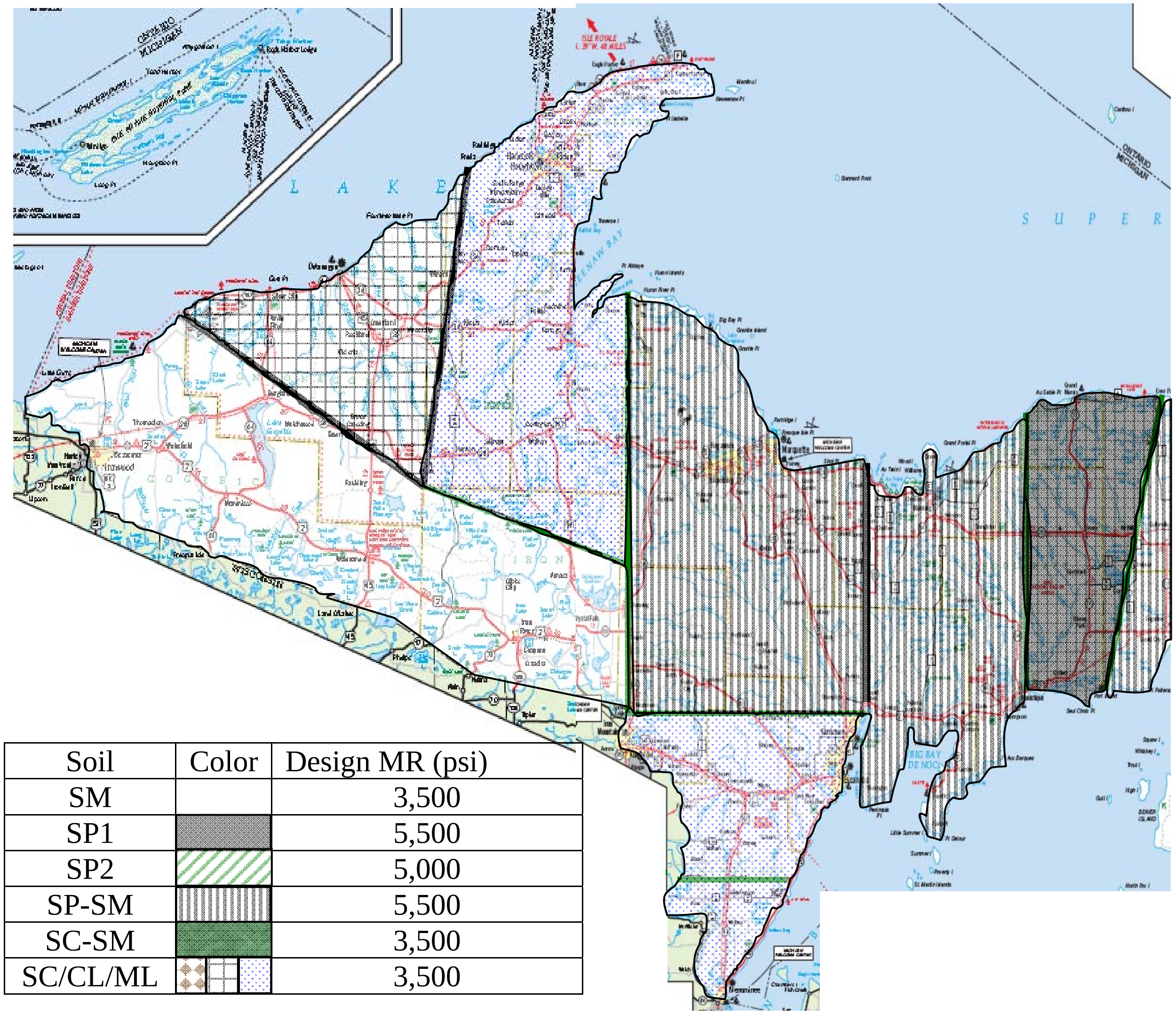
- MDOT implements the findings of this study by adopting the data presented in Figure 6.1 for M-E PDG design level 3 or in the 1993 AASHTO Design Guide.
- For rigid pavements, MDOT uses Equation 6.1 to convert backcalculated k values of roadbed soil to MR and vice versa.
- The above three recommendations apply equally to a newly planned and designed pavement sections and to some of the planned rehabilitation actions such as overlay.
- Additional deflection data should be collected during spring conditions and used to calibrate the seasonal damage factors that were developed based on laboratory data.
- Subject to the findings of the previous statement, the seasonal damage factor developed in this study should be implemented by all regions when using the 1993 AASHTO design guide. The reason is that, except for the SP1 and SP2 soils, the developed correlation equations, which are recommended to be used by all regions, include the natural water contents of the soils as variable. Such water contents increase during the spring season and decrease during dry periods.
- The backcalculated MR values need not be converted to laboratory MR values, the two are similar if the laboratory test boundary conditions are similar to those under FWD in the field.
- The subgrade soils in the nineteen areas, where neither disturbed nor undisturbed soil samples were obtained, should be subjected to the full testing schemes presented in Chapter 3 of this report.

Table 6.1 Summary of predictive equations for each soil type

USCS	Number of		Predictive equation	Variable equation
	Clusters	Areas		
SP1	6	8	$MR = 89.825(SVSP1)^{2.9437}$	$SVSP1 = \frac{\gamma_d^{1.15}}{\left(P_4^{1.5} - P_{40}^{0.25}\right)^{0.5}}$
SP2	6	12	$MR = 0.8295(SVSP2)^{3.6006}$	$SVSP2 = \frac{\gamma_d^{1.35} * P_{200}^{-0.1}}{\left(P_4^{1.5} - P_{40}^{0.25}\right)^{0.5}}$
SM	11	16	$MR = 0.0303(SVSM)^{4.1325}$	$SVSM = \frac{\gamma_d^{0.8}}{S^{0.15}}$
			$MR = 45722\exp[(-0.0258)(MI)]$	$MI = LL^{1.1} + MC^{1.25}$
SC,CL, ML	10	28	$MR = 650486\exp - 0.0501(S)$	$S = \left[\frac{G_s * (MC/100) * \gamma_d}{G_s * \gamma_w - \gamma_d}\right] * 100$
SP-SM	7	8	$MR = 1749.6\exp 0.0054(SVSP\text{-}SM)$	$SVSP\text{-}SM = \frac{\gamma_d^{1.75}}{MC^{0.5} + LL^{0.6} + \left(P_{40} - P_{200}\right)^{0.01}}$
SC-SM	5	7	$MR = 39638\exp - 0.0037(SVSC\text{-}SM)$	$SVSC\text{-}SM = C_u^{0.2} * \left(LL^{1.15} + MC^{1.3}\right)$
γ_d = dry unit weight (pcf), P_4, P_{40}, P_{200} = percent passing sieves number 4, 40, and 200, S = saturation (%), LL = liquid limit, MC = moisture content, G_s = specific gravity of the solid ≈ 2.7, γ_w = unit weight of water = 62.4 pcf, C_u = coefficient of uniformity				

Table 6.2 Average roadbed soil MR values

Roadbed type		Average MR (psi)			
USCS	AASHTO	Laboratory Determined	Backcalculated	Design value (psi)	Recommended design MR value (psi)
SM	A-2-4, A-4	17,028	24,764	5,290	5,200
SP1	A-1-a, A-3	28,942	27,739	7,100	7,000
SP2	A-1-b, A-3	25,685	25,113	6,500	6,500
SP-SM	A-1-b, A-2-4, A-3	21,147	20,400	7,000	7,000
SC-SM	A-2-4, A-4	23,258	20,314	5,100	5,000
SC	A-2-6, A-6, A-7-6	18,756	21,647	4,430	4,400
CL	A-4, A-6, A-7-6	37,225	15,176	4,430	4,400
ML	A-4	24,578	15,976	4,430	4,400
SC/CL/ML	A-2-6, A-4, A-6, A-7-6	26,853	17,600	4,430	4,400



Soil	Color	Design MR (psi)
SM		3,500
SP1		5,500
SP2		5,000
SP-SM		5,500
SC-SM		3,500
SC/CL/ML		3,500

Figure 6.1 State of Michigan average MR distribution

Soil	Color	Design MR (psi)
SM		3,500
SP1		5,500
SP2		5,000
SP-SM		5,500
SC-SM		3,500
SC/CL/ML		3,500

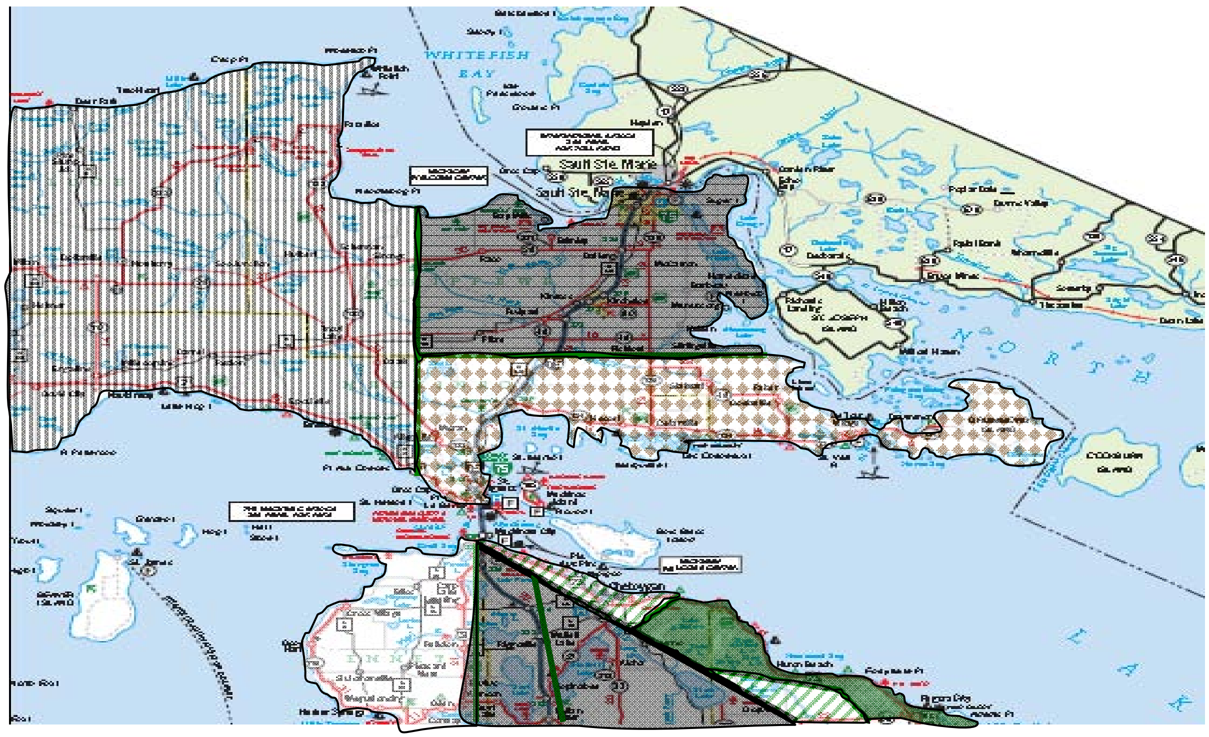


Figure 6.1 (cont’d)

Soil	Color	Design MR (psi)
SM		3,500
SP1		5,500
SP2		5,000
SP-SM		5,500
SC-SM		3,500
SC/CL/ML		3,500

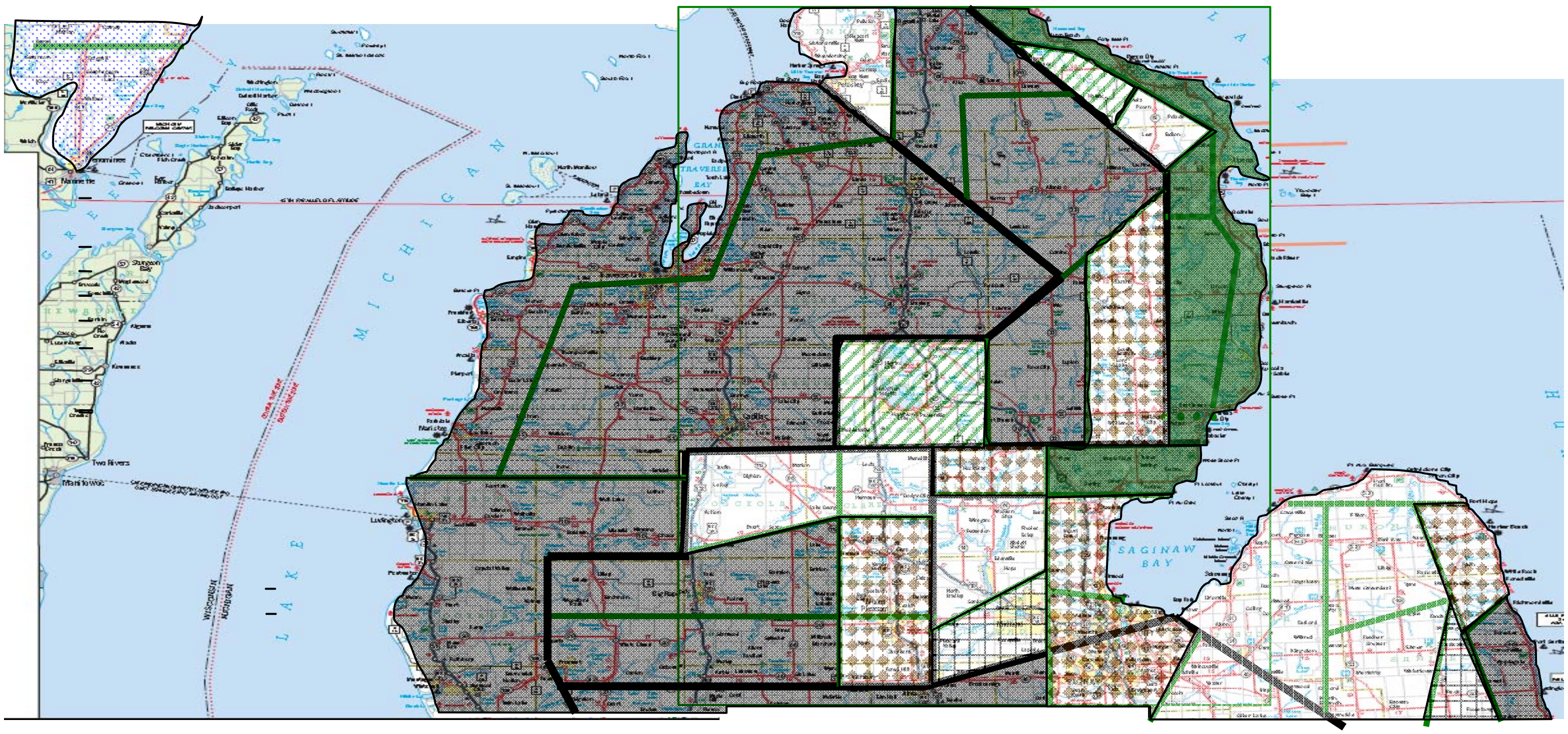


Figure 6.1 (cont'd)

Soil	Color	Design MR (psi)
SM		3,500
SP1		5,500
SP2		5,000
SP-SM		5,500
SC-SM		3,500
SC/CL/ML		3,500

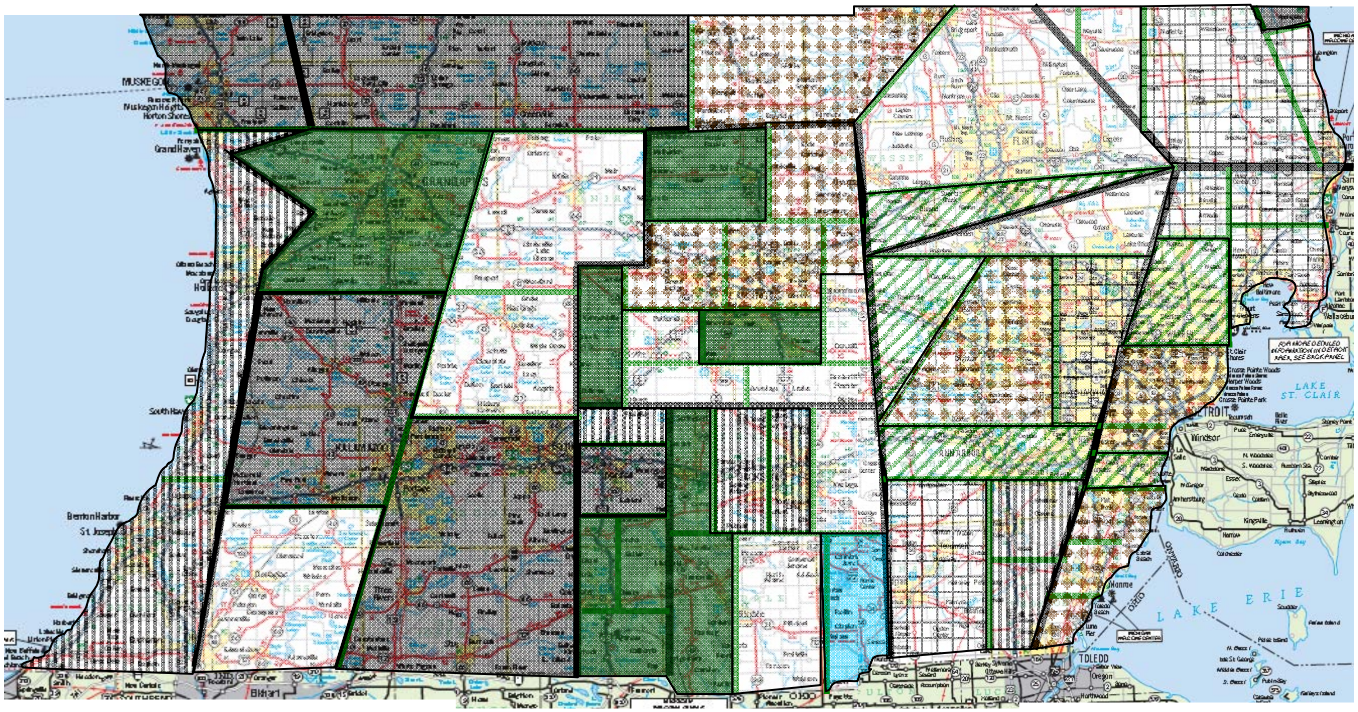


Figure 6.1 (cont'd)

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