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thesis entitled

BACKCALCULATED SUBGRADE RESILIENT MODULUS
DESIGN VALUES FOR THE STATE OF MICHIGAN

presented by

Tyler Allen Dawson

has been accepted towards fulfillment
of the requirements for the

Master of Science degree in Civil Engineering

Gilbert Baladi

Major Professor's Signature

12/10/08

Date

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BACKCALCULATED SUBGRADE RESILIENT MODULUS DESIGN VALUES FOR THE STATE OF MICHIGAN

By

Tyler Allen Dawson

A THESIS

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ABSTRACT

BACKCALCULATED SUBGRADE RESILIENT MODULUS DESIGN VALUES FOR THE STATE OF MICHIGAN

By

Tyler Allen Dawson

The resilient modulus (MR) of roadbed soils is an important input required for the design of pavement structures. The MR is a fundamental soil property reflecting the soil response to the applied stresses. The MR of a given roadbed soil is dependent on the soil type, water content, dry density, particle gradation and angularity, and stress states. The latter is a function of the pavement layer thicknesses and stiffness. The implication of the above is that for a given soil type and stress level, the MR of the soil is independent of the type of pavement surface (such as concrete, asphalt, or composite) and the type of testing procedure conducted (triaxial cyclic loading or Falling Weight Deflectometer (FWD) testing).

The Michigan Department of Transportation (MDOT) sponsored this study to characterize the MR of the roadbed soils in the State of Michigan. Laboratory tests were conducted to develop average MR values, by soil type, and correlations to simple tests (Sessions 2008). FWD tests were conducted and pavement layer moduli were backcalculated to determine roadbed soil MR. The MR results were very similar between laboratory and field testing, and between flexible and rigid pavements.

TO EMILY & ZUMAYA

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CHAPTER 1

INTRODUCTION

1.1 Background

A brief summary of the geography of the State of Michigan is presented below. The detailed geography can be found in (Sessions 2008).

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 was pulverized and incorporated into the ice sheet to later be re-deposited. 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 drift varies from one location to another. For example, the thickness of the drift in the Alpena area is only a few inches whereas it is more than 1200 ft thick in the Cadillac area. The glacial drift also varies from clay to gravel; the granular texture may be segregated or mixed heterogeneously with boulders and clays. Because of these complex arrangements, about 165 different soil types were formed and are being used for engineering purposes by the Michigan Department of Transportation (MDOT) (MDSH 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 this study, the MR of various roadbed soil types will be determined in the field using Falling Weight Deflectometer (FWD) deflection data and in the laboratory using cyclic load triaxial tests.

1.2 Problem Statement

The roadbed soils in the State of Michigan consist of glacial soils with distinct seasonal stiffness changes due to temperature (possible frozen condition) and moisture levels. MDOT's current pavement design process follows the procedure outlined in the 1993 American Association of State Highway and Transportation Officials (AASHTO 1993) Design Guide. One of the inputs of said procedure 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, MDOT's various regions provide the "adjusted" MR value used for pavement design. The MR value is derived from either backcalculated deflection data or a correlation with known Soil Support Values (SSV).

1.3 Objectives

The main objectives of this study are to:

- Evaluate the existing processes used by all regions of MDOT for determining the MR value of roadbed soil.
- Determine the needed modifications to make the MR selection process compatible with the new M-E PDG.
- Develop procedures, equations, and values for roadbed soil MR for use in any (1, 2, or 3) level design of the M-E PDG and the current AASHTO design guide.

1.4 Research Plan

To accomplish the objectives, a research plan consisting of five tasks was developed and is presented below.

Task 1— Review and Information Gathering

In this task, the research team will become familiar with MDOT current and historical processes/procedures for selecting MR and k values for the design of flexible and rigid pavements. The information could be obtained from the soil engineers in the various regions. The research team will also obtain information from MDOT that is needed for the other tasks in this study. These include:

1. Collection of deflection data from previously conducted FWD tests with known pavement cross-sections.
2. Tabulation of the procedures used by the various regions for selecting MR and k values and the basis of such selection.
3. Tabulation of the range and typical MR and k values used by the regional soil engineers for the various soil types.
4. Assessment of the adequacy and sufficiency of the existing process for estimating MR values to be used in the new M-E PDG.

Task 2— Partitioned State Map

Based on the MDOT Field Manual of Soil Engineering, the information obtained from the various regions in Task 1, the trunkline locations, and the soil maps of the US Soil Conservation Services (USCS), the state will be partitioned into geological zones for the purpose of field testing and soil sampling. The state will be divided into a maximum of 15 coarse clusters where the soil within any given cluster would have a similar range of engineering and physical characteristics. Each coarse cluster will then be further divided into areas to narrow the range of the soil characteristics. A maximum of 99 areas will be produced. The results will be presented to members of the Research Advisory Panel (RAP) for review and possible modification. The main use of the partitioned soil map will be to determine the locations for field testing and soil sampling.

Task 3— Field and Laboratory Testing and Soil Sampling

In this task, the research team will finalize the field sampling locations and the laboratory testing plans based upon the information obtained in Tasks 1 and 2. The total number of tests to be conducted will be based purely on cost and available budget. The field sampling and the laboratory testing plans are presented in three subtasks below.

Subtask 3.1 - Soil Sampling Plan

From each area on the state partitioned map, roadbed soil samples will be obtained. In areas where the roadbed soil is predominantly sand, only disturbed bag samples will be collected. In areas where the roadbed soil is composed of mostly clay, both disturbed and undisturbed (Shelby tube) samples will be obtained.. All samples will be transported to the laboratory at Michigan State University (MSU) for testing as presented in Subtask 3.2 below.

Subtask 3.2 – Laboratory Testing Plan

The laboratory testing plan consists of moisture content, sieve analysis, Atterberg limits, and cyclic load triaxial tests. All tests will be conducted according to MDOT, AASHTO or American Society for Testing and Materials (ASTM) standard test procedures. Results of the laboratory testing will be analyzed (see Task 4) to determine:

1. Soil classification - For each soil sample the soil will be subjected to sieve analyses to determine its gradation. Plastic and liquid limit tests (Atterberg Limits) will also be conducted on any sample where the fine fraction (passing sieve number 200) is more than seven percent. Results of the sieve analyses and Atterberg limit tests will be used to:
   - Classify the soil according to the USCS and the AASHTO soil classification systems.
   - Develop, if possible, statistical correlations between the resilient modulus of the roadbed soils and the gradation and Atterberg limits of the material.
2. Resilient modulus (MR) - For each soil sample, at least one triaxial cyclic load test will be conducted to determine the MR. Since, the resilient moduli of roadbed soils are heavily dependent upon the deviatoric stress; the laboratory tests will be conducted at three stress states which will be estimated through mechanistic analyses to simulate the probable in-situ field conditions.

Subtask 3.3 –Field Test Plan

This plan consists of FWD tests. The FWD tests will be conducted at the network- and project-levels. At the network level, one FWD tests will be conducted at 500 foot intervals along the state trunkline. At the project level, 20 FWD tests will be conducted within $\pm$ 50 ft from all locations where Shelby tubes (undisturbed soil samples) will be extracted.

All FWD tests will be conducted, at the same location, once in the spring and again in the late summer – early fall seasons. For those areas where FWD tests were conducted in the past and the deflection and pavement cross-section data are available from MDOT, the data will be used and the number of FWD tests (to be conducted in those areas in this study) will be reduced depending on the availability of spring and fall deflection data.

It should be noted that analyses of various damage models, including AASHTO, indicate that the two point FWD testing (spring and fall seasons) is adequate to assess the relative pavement damage related to the roadbed soil due to different degrees of saturation.

Task 4 – Data Analyses

The data analysis, in this study, will be accomplished according to the three subtasks presented below. First, it should be noted that for all soil types, the relationship between the MR and k found in the M-E PDG will be used.

Subtask 4.1 – Backcalculation of Pavement Layer Moduli

All deflection data, whether collected during this study or other studies, will be used (depending on the availability of pavement cross-section data) to backcalculate the pavement layer moduli. The MICHBACK computer program will be used for flexible pavements and the AREA method for rigid pavements. Although the moduli of all pavement layers will be backcalculated, only the resilient modulus of the roadbed soils will be subjected to further analyses. The moduli of the other pavement layers will be reported without further analyses. For each test area on the partitioned map, two sets of moduli will be backcalculated; one set will be based on the spring deflection data and the other on the late summer-early fall data. The two sets will be further analyzed to estimate the seasonal damage factor as presented in task five below.

Subtask 4.2 – Laboratory Test Data

Results of the cyclic load tests conducted on Shelby tube and reconstituted bag samples at various moisture contents will be analyzed to determine the laboratory values of the resilient modulus of the roadbed soil. Results of the analyses will be used to assess the impact of moisture (season) on pavement damage and to compare the values to those obtained from backcalculation.

The Atterberg limits (liquid limit, plastic limit, and plasticity index) and sieve analysis data will be used to classify the soil and to develop correlations to MR whenever possible.

Subtask 4.3 – Backcalculated and Laboratory Determined MR Comparison

MR results of roadbed soil from backcalculated FWD deflection data will be compared with results from cyclic load tests in the laboratory. Any variation between results, by soil type, will be analyzed. A correlation, if possible, will be made between backcalculated and laboratory determined MR results.

Task 5— Damage Assessment Analyses

The damage assessment analyses (noted in subtask 4.1) will be conducted based on the seasonal MR values obtained from the backcalculation of the FWD deflection data. The purpose of the analyses is to determine the effective MR values to be used in the design and rehabilitation of flexible and rigid pavements. The effective roadbed resilient modulus is an equivalent modulus that would result in the same damage as if the various seasonal resilient modulus values were used (Huang 2004).

The research plan was accomplished in two parts, laboratory testing and analyses and field testing and analyses. The former was presented in (Sessions 2008) whereas the latter is presented in this thesis.

1.5 Thesis Layout

This thesis is composed of six chapters as follows:

Chapter 1 – Introduction

Chapter 2 – Literature Review

Chapter 3 – Laboratory and Field Investigation

Chapter 4 – Data Analysis & Discussion

Chapter 5 – Summary, Conclusions, & Recommendations

Appendix A – Laboratory and field test results

Appendix B – NDT data test results

CHAPTER 2

LITERATURE REVIEW

2.1 Review of MDOT Practices

The Michigan Department of Transportation (MDOT) has divided the State of Michigan into seven autonomous regions as shown in Figure 2.1. Each region has developed its own practice (see Table 2.1) to estimate the resilient modulus (MR) of the roadbed soils. MDOT has its own soil classification system, based on other systems like the USCS to classify soil by grain size and other visual properties. Several MDOT regions use a correlation between soil support values (SSV), AASHTO layer coefficients, and MR based on the USDA soil classification system which can be seen in Figure 2.2.

Figure 2.1 Regional divisions for MDOT

Table 2.1 Typical MDOT testing procedures and values for MR

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

2.2 Soil Classification Systems

A brief summary of soil classification systems is presented in this section. A detailed review can be found in (Sessions 2008).

There are several soil classification systems used by various agencies and organizations. The three most popular include; the United States Department of Agriculture (USDA), the Unified Soil Classification System (USCS), and the American Association of State Highway and Transportation Officials (AASHTO) soil classification system (Holtz and Kovacs 1981). MDOT also has its own system, Uniform Field Soil Classification System, which was created to be applicable on site by visual identification. In order for various highway organizations to compare their roadbed soils with other agencies, that use other classification systems, a comparison must be made. Table 2.2 below compares USDA, USCS, and AASHTO.

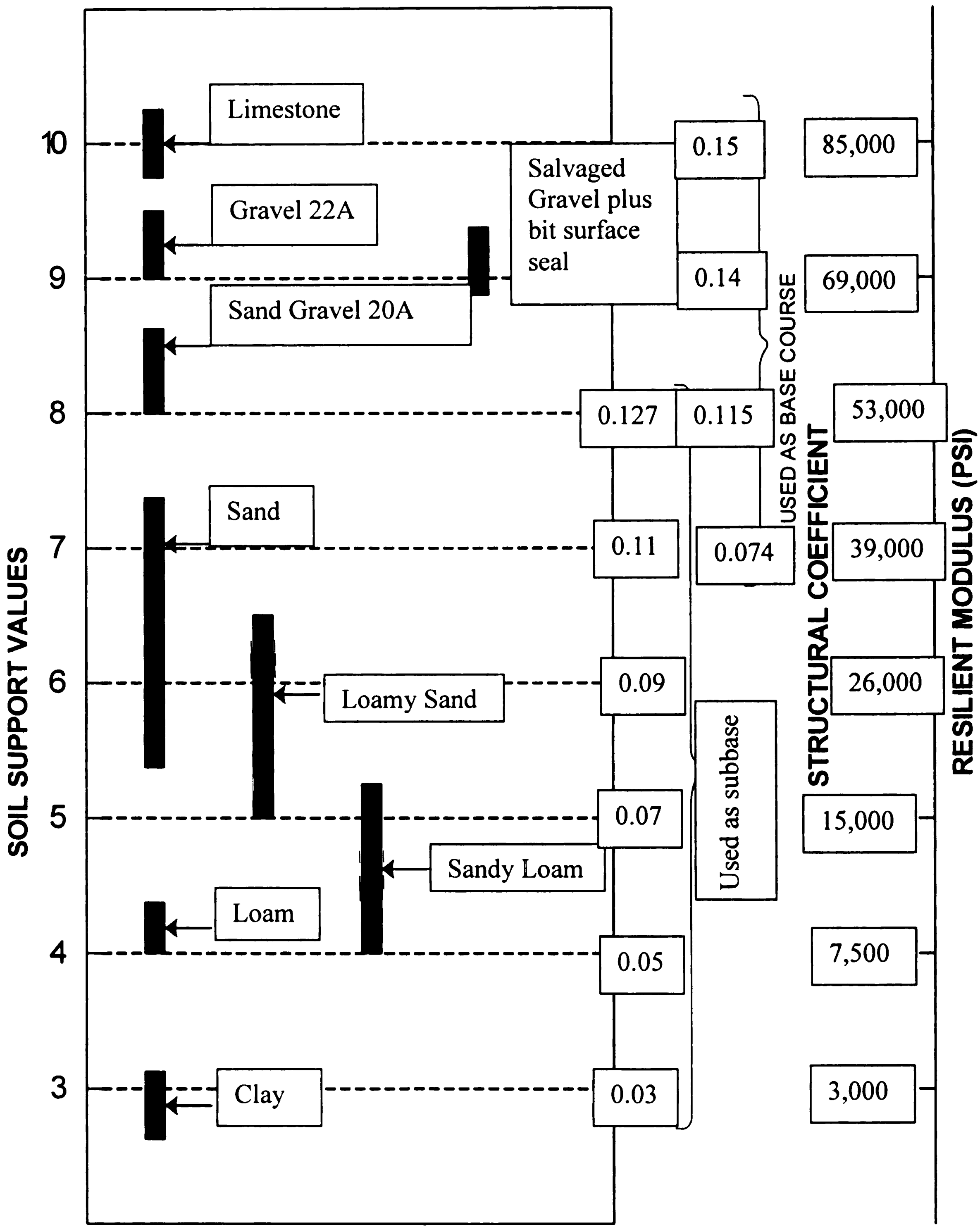


Figure 2.2 Soil support, structural coefficient, and MR correlations (MDSH)

Table 2.2 Comparison between three soil classification systems (USDA 1992)

USDA texture	Classification		Percent Passing Sieve Number				Liquid Limit	Plastic Limit
	USCS	AASHTO	4	10	40	200		
Muck	PT	A-8	100	100	90-100	40-100	0-14	NP
Sand	SP-SM, SM, SP, GP, GP-GM, GM	A-2-4, A-3, A-1-b, A-2, A-3, A-2	40-100	25-100	15-90	0-35	<25	NP
Loamy Sand	SM, SC-SM, ML, CL-ML, SP-SM, SP	A-2, A-4, A-1-b, A-1, A-2-4, A-3	85-100	60-100	30-90	3-55	<30	NP
Silty Loam	ML, CL, CL-ML, SC, SM, CH	A-4, A-6, A-7, A-2	95-100	85-100	60-100	30-95	<45	NP/P
Sandy Loam	SM, SC-SM, ML, CL-ML, SC, CL	A-2-4, A-4, A-2, A-1, A-1-b, A-6	70-100	60-100	35-90	15-75	<35	NP
Clay Loam	CL, CL-ML, SC, SC-SM	A-6, A-4, A-7, A-2	95-100	75-100	70-100	35-90	25-45	NP/P
Loam	CL, CL-ML, ML	A-4, A-6, A-7	90-100	75-100	70-100	50-90	15-45	NP/P
Mucky Sand	SM, SP, SP-SM	A-1-b, A-2-4, A-3	95-100	75-100	30-70	0-15	0-14	NP
Clay	CH, CL	A-6, A-7-6	90-100	85-100	65-95	45-95	30-65	P
Silty Clay	CL, SC, CL-ML	A-4, A-6, A-7	85-100	60-100	50-100	30-90	25-50	NP/P
NP = non-plastic, plastic limit<10 P = plastic soil, plastic limit>10								

Several correlations between the soil classification systems and the resilient modulus of roadbed soils can be found. Some regions of MDOT use the correlations found in Figure 2.2 currently. For use with the M-E PDG, Table 2.3 recommends ranges and typical MR values by AASHTO soil classification and USCS. Figure 2.3 also provides estimations of various roadbed soil parameters correlating to the AASHTO classification system and USCS (NHI 1998). The tables and figures previously mentioned only provide ranges and it is up to the engineer to decide upon a value to use in design.

Table 2.3 Typical resilient modulus values for unbound granular and roadbed 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

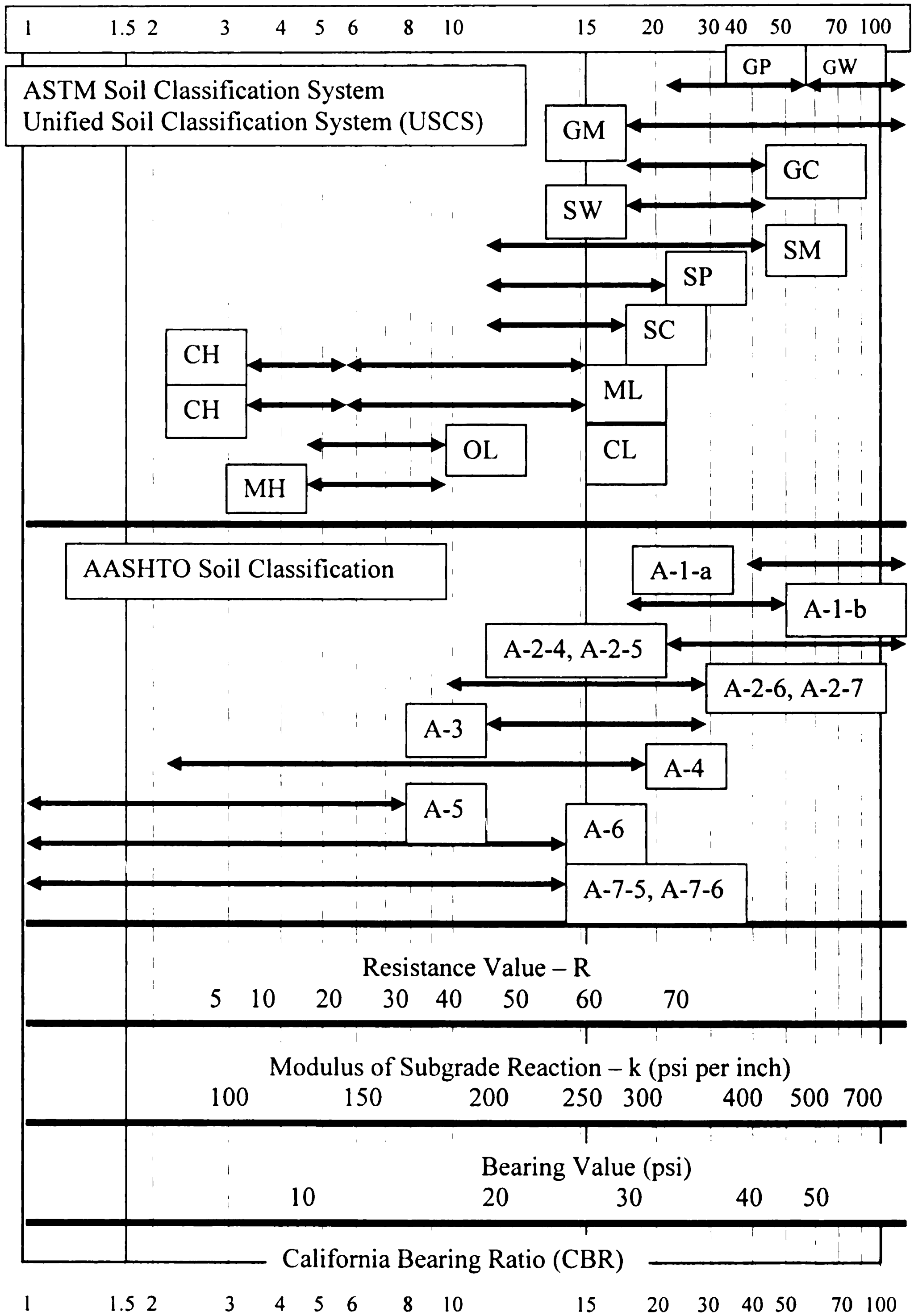


Figure 2.3 Soil classification related to strength parameters (NHI 1998)

2.3 The Role of Roadbed MR in Pavement Design for the M-E PDG

A brief summary the M-E PDG procedure with regard to roadbed MR is presented in this section. A detailed review can be found in (Sessions 2008).

The M-E PDG allows users to be flexible with the specificity they have in the input data to the design guide. Depending on the resources available and requirements of any given design project, the user can choose how general or specific the input data will be. Structural inputs, such as the roadbed MR, can be selected from any of a three level hierarchy (Coree et. al 2005).

Level one design is the most precise of the levels and requires the most accurate inputs. Roadbed MR must be determined by field or laboratory tests, such as FWD deflection testing or cyclic load triaxial testing. Pavement sections that are of greater importance to the agency, such as high traffic roads or those with economic and social significance, will be designed at level one. This level is more expensive and time consuming, but will lead toward a more dependable result.

Level two and three designs require less specific data. Roadbed MR could be estimated, for level two, from simple soil tests. Typical default values could be used with level three designs. The lower levels will be less expensive to design with but will yield less reliable results. The design level used does not have to stay constant between different inputs; general, traffic, climatic and structural. However, the design process will be the same regardless of the input level (Prozzi and Hong 2006).

In all three levels of design the roadbed MR is a required input to the pavement structural response model. No matter what design level is used, roadbed MR plays a significant role in computing pavement response and dynamic modulus of subgrade reaction, k-value, which is computed internally by the Design Guide software (NCHRP 2004).

2.4 Engineering Evaluation of Roadbed Soils

The engineering evaluation of roadbed soils can be achieved using several techniques that can be divided, in general, into two categories: destructive and nondestructive. Destructive tests include:

- Coring
- Drilling and/or Shelby tube extraction

Nondestructive tests include:

- Ground penetrating radar (GPR) to estimate the pavement layer thicknesses
- Nondestructive deflection tests (NDT) to measure the pavement response to loads
- Surface wave application to measure pavement response

Literature review regarding NDT and the use of the deflection data in pavement evaluation processes are addressed in the next sections.

2.4.1 Nondestructive Deflection Tests (NDT)

The nondestructive deflection test (NDT) is the most popular test used in pavement evaluation. Relative to destructive testing, NDT are fast and require minimum lane closure time. In recent years, the use of NDT has become an integral part of the structural evaluation and rehabilitation of pavement structures.

The NDT results (the pavement deflections at various distances from the center of the load) are used to:

- Backcalculate the pavement layer moduli
- Assess the variability of the pavement response to loads along and across the pavement and hence, the variability of the pavement structural capacity
- Estimate load transfer efficiency of dowel bars
- Evaluate the presence of voids beneath the pavement surface
- Design the thickness of pavement overlays

Various types of NDT devices are available and being used by various State Highway Agencies (SHA). These are presented in the next subsection.

2.4.1.1 NDT Devices

NDT devices are used by state highway agencies to apply patterns of loading and record deflection data along the pavement surface. The deflection data measured along the pavement surface at different distances from the center of the load are typically used to backcalculate the modulus values of the various pavement layers and the roadbed soil. Numerous backcalculation software packages are available either in the public domain or can be purchased. Most of these use more or less the common procedures presented in the next sections.

Various types of NDT equipment are available. A brief summary of the available equipment is presented in this subsection. Details on the equipment can be found in (Mahmood 1993).

- Static deflection equipment including: the Benkelman Beam, which can be seen in Figure 2.4, (Moore et al 1978; Asphalt Institute 1977; Epps et al 1989), the plate bearing test (Moore et al 1978; Nazarian et al 1989), the Dehlen Curvature Meter (Gouzheng 1982), the Pavement Deflection Logging Machine (Keneddy et al 1978), and the C.E.B.T.P. Curviameter (Paquet 1978).

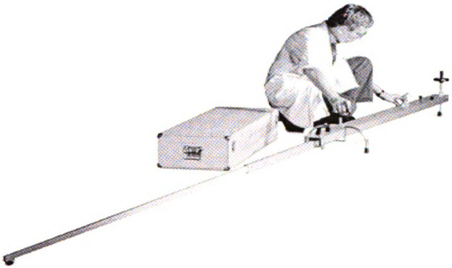


Figure 2.4 Benkelman beam

- Automated deflection equipment including: the La Croix Deflectograph, which can be seen in Figure 2.5, (Hoffman et al 1982; Keneddy 1978), and the California Travelling Deflectometer (Roberts 1977).
- Steady-State dynamic deflection equipment including: the Dynaflect, which can be seen in Figure 2.6, the Road Rater, the Cox Device, the Waterways Experiment Station (WES) Heavy Vibrator, and the Federal Highway Administration (FHWA) Thumper (Scrivner et al 1969; Smith et al 1984; Moore et al 1978).

Figure 2.5 La Croix deflectograph

Figure 2.6 Dynaflect

- Impulse deflection equipment including: the Dynatest FWD, which can be seen in Figure 2.7, KUAB FWD, and the Phoenix FWD (Nazarian et al 1989; Hoffman and Thompson 1981; Bohn et al 1972; Crovetti et al 1989; Claessan et al 1976).

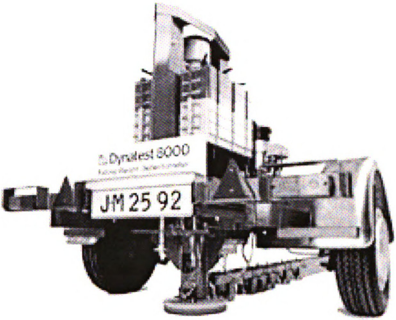


Figure 2.7 Dynatest Falling Weight Deflectometer

2.4.1.2 Falling Weight Deflectometer (FWD) Test

Falling Weight Deflectometers (FWD) are used to apply load to the pavement and measure deflection on the pavement surface at several longitudinal distances from the applied load. The FWD is often preferred over laboratory testing for several reasons including: the nondestructive nature of the tests, low operational cost per test, short test duration, tests can be designed to provide more coverage of the pavement network, and the roadbed soils are being tested under in-situ boundary conditions. The disadvantages include the difficulty to determine or control the water content of the roadbed soils, determine the roadbed soil density, and to control the applied normal and shear stress levels (Houston et. al 1992).

The FWD operates on two basic assumptions; the force of impact due to a falling load is considered a static load, and the roadbed soil acts as an elastic body. The weight of the falling mass can be calculated as follows, as presented in (Kim et al 2006).

$$W_1(H + \delta_{max}) - .5K\delta^2{}_{max} = 0 \qquad \text{Equation 2.1}$$

Where, W_1 = weight corresponding to the mass M

H = height M was dropped from

δ_{max} = maximum pavement deflection

K = spring constant

δ_{max}/δ_{st} = the impact factor, which can be found by equation 2.2.

$$\delta_{max}/\delta_{st} = 1 + \left(1 + \left(\frac{2H}{\delta_{st}}\right)\right)^{\frac{1}{2}} \qquad \text{Equation 2.2}$$

Where, δ_{st} = static deflection

The impact load is calculated using equation 2.3, by multiplying the static load by the impact factor.

$$P_{dyn} = W_1\left(1 + \left(1 + \left(\frac{2H}{\delta_{st}}\right)\right)^{\frac{1}{2}}\right) \qquad \text{Equation 2.3}$$

Due to the difficulty in measuring impact load, force is calculated by multiplying weight by height.

$$F = WH \qquad \text{Equation 2.4}$$

Where, F = force

The uniformly distributed load can be obtained from equation 2.5.

$$q = \frac{F}{A} \qquad \text{Equation 2.5}$$

Where, q = applied load to plate

A = loading plate area

A series of FWD tests are usually preformed in order to obtain more accurate results. Consecutive tests are conducted at regular intervals along a pavement surface. At each interval four drops of the weight are conducted. The first drop is not used in analysis, and the following three are averaged to create one set of data for each interval. This allows for average values along the pavement to be calculated. Averages are taken in order to capture the range of deflections as well as the most common values over a pavement section. The variations in deflection are due to non-constant roadbed soils and construction practices which often result in varying densities and thicknesses of the pavement layers. A typical asphalt concrete (AC) surface can range from plus or minus 1 inch of thickness from the design thickness. This can affect MR results because a constant layer thickness and Poisons' ratio is used for the entire pavement section tested. An example of how measured deflections at each sensor vary along a pavement section is shown in Figure 2.8.

The KUAB brand FWD is used by several state agencies, including MDOT and other agencies around the world; the device can be seen in Figure 2.9. The system applies a dynamic impulse load to the pavement surface with a two mass system that simulates a

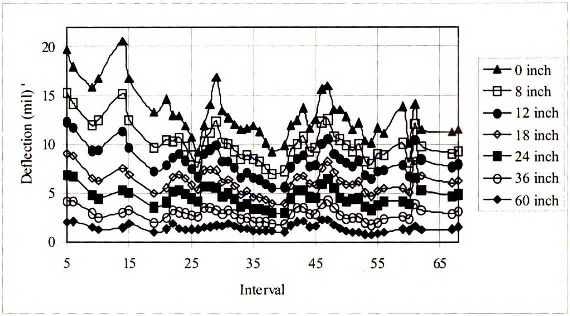


Figure 2.8 Typical deflections at all sensors

moving tire load. Seismometers set at specific distances along the pavement surface measure acceleration and double integrate to determine vertical deformation or deflection. The entire system is housed in a trailer and can be operated remotely from the truck cab, which allows for quick and easy execution of tests in any weather.

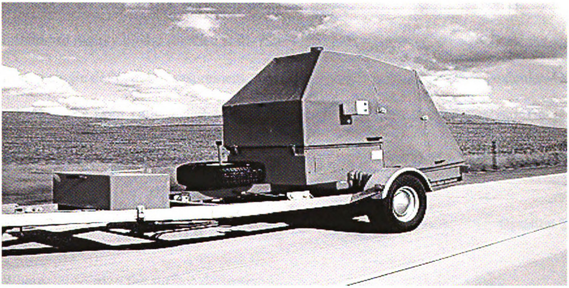


Figure 2.9 KUAB falling weight deflectometer

2.5 Backcalculation of Layer Moduli of Flexible Pavement

Flexible pavement layer moduli are backcalculated using deflection data from FWD tests. Deflection data is analyzed using computer programs to iteratively forward calculate deflection based on layer moduli, Poisson ratios, and thicknesses and load magnitude. Then the layer moduli are incremented until the calculated deflection is very close to the measured deflection. When the absolute or Root Mean Squared (RMS) error between the measured and calculated deflection is minimized results are the most accurate. There are 5 categories of assumptions that have been used to create the various computer programs; linear elastic-static, nonlinear elastic-static, linear-dynamic using frequency domain fitting, linear-dynamic using time domain fitting, and nonlinear-dynamic (Uzan 1994). Each category utilizes different assumptions and techniques.

2.5.1 Backcalculation Methods for Flexible Pavement

The roadbed soil modulus can be determined by using the pavement surface deflection measured at distances of 48-inches or more from the center of the load. Because of arching effects, at these distances, the pavement surface deflection is influenced mainly by the roadbed soils. Hence, the subgrade MR can be backcalculated from a single deflection measurement. The most widely used routine to backcalculate the subgrade MR from a single deflection measurement is the Boussinesq equation (George 2003).

$$d_r = \frac{CP(1-\nu^2)}{\pi r MR} \; or \; MR = \frac{CP(1-\upsilon^2)}{\pi r d_r} \qquad \text{Equation 2.6}$$

Where, d_r = the surface deflection (in) at a distance r (in) from the load

P = applied load (lbs)

C = correlation/adjustment factor that accounts for the difference between the backcalculated and the laboratory obtained MR value

MR = resilient modulus (psi)

ν= poison's ratio of the asphalt layer

By assuming a Poisson's ratio of 0.5, equation 2.6 can be reduced to the following equation (AASHTO 1993).

$$MR = \frac{0.24CP}{d_r r} \qquad \text{Equation 2.7}$$

AASHTO recommends the use of a *C* value no greater than 0.33

The minimum distance (r) in Equations 2.6 and 2.7 is given by the following relationship.

$$r = 0.7\sqrt{a_2 + \left(D \times \sqrt[3]{\frac{E_p}{MR}}\right)} \qquad \text{Equation 2.8}$$

Where, a_2 = radius of load plate (in)

D = total thickness of pavement layers above the roadbed (in)

E_p = effective modulus of all layers above the roadbed (psi)

E_p in equation 2.8 can be calculated by using the following equation:

$$\frac{MR \times d_o}{q \times a} = 1.5\left\{\frac{1}{\sqrt{1 + \left[\frac{D}{a}\sqrt[3]{\frac{E_p}{MR}}\right]^2}} + \frac{1 - \frac{1}{\sqrt{1 + \left(\frac{D}{a}\right)^2}}}{\left(\frac{E_p}{MR}\right)}\right\} \qquad \text{Equation 2.9}$$

Where, d_0 = deflection measured at the center of the load plate after adjustment to

a = temperature of 68 °F

q = pressure on load plate

D = total thickness of pavement layers above the subgrade

E_p = effective modulus of all layers above the subgrade

The Washington State Department of Transportation (WSDOT) developed, for asphalt pavements, Equations 2.10 through 2.12 and, for concrete pavements, Equation 2.13 to estimate the subgrade modulus from deflection sensors located at various distances from the center of the load (Pierce 1999).

$$MR\ (psi) = 9000\frac{0.2892}{24d_{24}} \qquad \text{Equation 2.10}$$

$$MR\ (psi) = -466 + 9000\frac{0.00762}{d_{36}} \qquad \text{Equation 2.11}$$

$$MR\ (psi) = -198 + 9000\frac{0.0.00567}{d_{48}} \qquad \text{Equation 2.12}$$

And for concrete pavements,

$$MR\ (psi) = -111 + 9000\frac{0.00577}{d_{48}} \qquad \text{Equation 2.13}$$

Where, d_{24}, d_{36} and d_{48} are the pavement surface deflections in inches measured at 24, 36, and 48 inches from the center of the load.

There are several different computer programs that utilize the before mentioned backcalculation methods, each with varying assumptions, routines, and methods. Table 2.4 below lists many of the available backcalculation programs.

Table 2.4 Backcalculation programs

Program name	Develop By	Forward calculation method	Forward calculation subroutine	Backcalculation subroutine	Non-linear analysis	Seed modulus	Comments
BISDEF	A. Bush USACE-WES	Mulit-Layer elastic theory	BISAR	ITERATIVE	Non-linear analysis	Required	Sensitive to seed modulus. Uses gradient search method
BOUSEDEF	Zhou, et al.	Equivalent layer thickness	MET	ITERATIVE	Yes	Required	Program logic similar to BISDEF
CHEVDEF	A. Bush USACE-WES	Mulit-Layer elastic theory	CHEVRON	ITERATIVE	Non-linear analysis	Required	Sensitive to seed modulus.
COMDEF	M Anderson	Mulit-Layer elastic theory	DELTA	DATA BASE	Non-linear analysis	Required	For composite pavements only.
DBCONPAS	M. Tia, et al.	Finite element	FEACONSIII	DATA BASE	Yes		For rigid pavements only.
ELMOD	P. Ulditz	Equivalent layer thickness	MET	ITERATIVE	Yes roadbed only	Not required	Fast, but has limitation inherent to MET program
ELSDEF	Texas A&M University	Mulit-Layer elastic theory	ELSYM5	ITERATIVE	No	Required	Sensitive to seed modulus.
EMOD		Mulit-Layer elastic theory	CHEVRON	ITERATIVE	Yes roadbed only	Required	

Table 2.4 (cont'd)

Program name	Develop By	Forward calculation method	Forward calculation subroutine	Backcalculation subroutine	Non-linear analysis	Seed modulus	Comments
EVERCALC	Mahoney, J., et al.	Mulit-Layer elastic theory	CHEVRON	ITERATIVE	Yes	Not required for up to 3 layers	Primarily for flexible pavements.
FPEDDI	W. Uddin	Mulit-Layer elastic theory	BASNIF	ITERATIVE	Yes	Not required	
ISSEM4	P. Ulidtz	Mulit-Layer elastic theory	ELSYM5	ITERATIVE	Yes	Required	Uses deflections at five point to calculate moduli for three layers.
MICHBACK	Michigan State University	Mulit-Layer elastic theory	CHEVRON	ITERATIVE	No	Required	
MODOMP2	L. Irvin	Mulit-Layer elastic theory	CHEVRON	ITERATIVE	Yes	Required	More oriented for research work.
MODULUS	J. Uzan	Mulit-Layer elastic theory	WESLEA	DATA BASE	Yes	Required	Used in an expert system frame work.
PADAL	**S. F. Brown**	**Finite element**		**ITERATIVE**	**Yes**	**Required**	
RPEDDI	W. Uddin	Mulit-Layer elastic theory	BASINR	ITERATIVE	Yes	Not required	For rigid pavements only.
WESDEF	USACE-WES	Mulit-Layer elastic theory	WESLEA	ITERATIVE	No	Required	Sensitive to seed modulus.

2.5.1.1 MICHBACK

The program for backcalculation of layer moduli of flexible pavement used in this report is MICHBACK, developed at Michigan State University (MSU). The MICHBACK uses the Chevronx (a multilayer elastic program) as the forward engine to calculate the pavement deflections for a given set of data (layer moduli and Poisson ratios, layer thicknesses, and load magnitude). The MICHBACK program utilizes a modified Newtonian algorithm to increment the layer modulus values based on the differences between the measured and the backcalculated pavement deflections (George 2003).

A brief summary of the MICHBACK program is presented in below. A detailed flow-sheet can be found in (Mahmood 1993).

1) Input initial data (pavement location, file name, layer information, etc...)
2) Upload FWD file, or manually input deflection data
3) Input modulus seed values and stiff layer depth
4) Perform backcalculation
5) View or print results

MICHBACK uses a linear-elastic model, as mentioned previously. In order for the program to work correctly, and converge, the deflection basin must be uniform with an elastic system. The main contributing factor leading to non-convergence is the degree of irregularity of the deflection basin. For the backcalculation of layer moduli to be successful, the shape of the deflection basin must be smooth and compatible with the elastic layer theory. Highly irregular measured deflection basins (such as that shown in Figure 2.10) cannot be matched to that calculated using the layer elastic theory. Irregularities in the deflection basins could be caused by an uneven contact between one or more deflection sensors and the pavement surface, debris (such as sand particles) between the deflection sensors and the pavement surface, and/or cracks or other structural distresses in the pavement that adversely impact the continuity of the stress dissipation with depth and distance from the load.

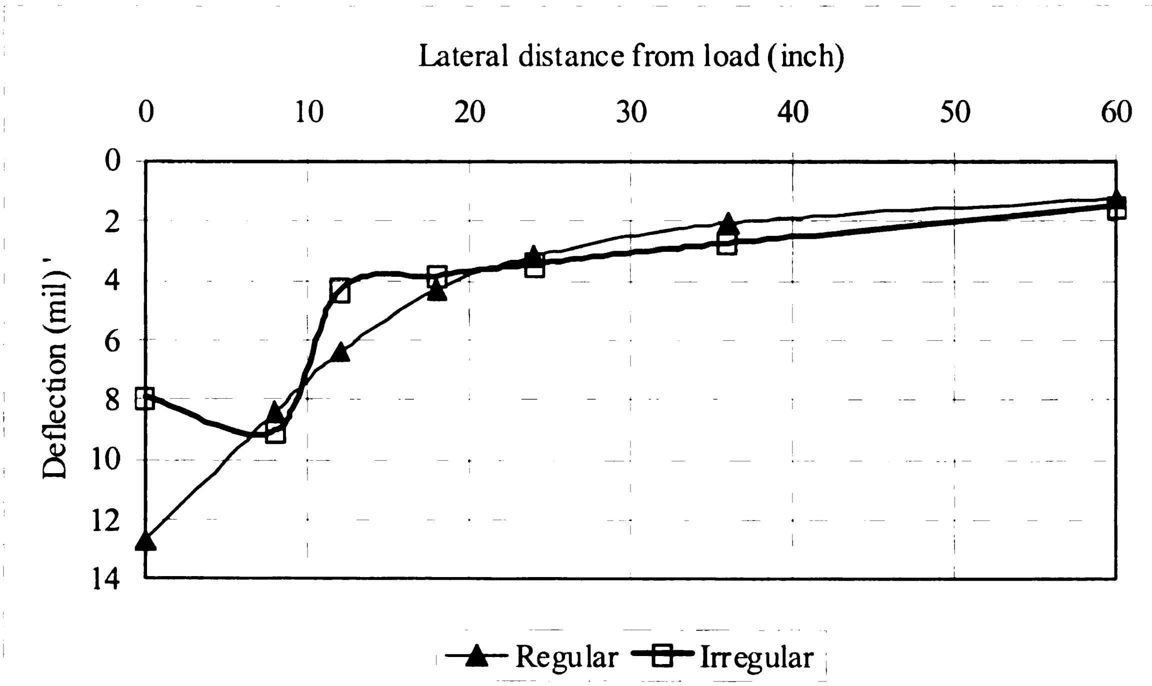


Figure 2.10 Regular and irregular deflection basins

2.5.2 Flexible Pavement Temperature Effect on Resilient Modulus

The Asphalt Concrete (AC) layer MR of a pavement system is greatly affected by temperature. It is common for the temperature of the AC layer to vary by 30° F in a given day. This temperature fluctuation can result in a 500,000 psi variation in AC MR, which will significantly affect the MR of other pavement layers when backcalculating. The ideal AC temperature for FWD testing is between 40° and 100° F. It can be difficult to backcalculate AC stiffness when the MR is above 2 million or below 200,000 psi, therefore flexible pavements should only be FWD tested within the recommended temperatures (Kathleen et al 2001).

A procedure was developed by (the Asphalt Institute 1977) to correct for temperature of AC pavement layers. This process requires the following data: The high and low temperature for the previous 5 days leading up to the NDT, pavement surface temperature at exact time of NDT, frequency of loading and time duration of load impulse, as well as percent asphalt content by weight. If all of this data is available, then AC temperature at the top, middle, and bottom of the layer can be determined, and the mean of the three temperatures is used as the corrected pavement temperature.

2.5.3 Depth to Stiff Layer Effect on Resilient Modulus

Roadbed soil is assumed to be uniformly stiff and infinitely thick, when using linear elastic models such as the one utilized by MICHBACK. This assumption is incorrect as roadbed soil tends to become denser with depth, due to stress increases. At some depth a "stiff layer" will be present, which can be composed of either bedrock or a very dense layer of roadbed soil. To account for this, an additional layer is incorporated in the backcalculation procedure. A stiff layer can be included in several ways:

- Assignment of a very high modulus to the lowest layer in the pavement system; however the depth to this layer will be unknown.
- Assignment of a 20 ft. depth to stiff layer for all FWD analysis (Bush 1980).
- Use of measured velocity of compression waves and frequency of loading (Uddin et al 1986).
- Application of trial and error method carried out until a minimum RMS error is reached (Chou 1989).

The above mentioned methods make various assumptions about depth, which is often unknown. Regression models have also been developed to estimate depth to stiff layer from deflections and layer thicknesses (Brown 1991). This method is used in MODULUS and EVERCALC, but does not accurately predict depth for medium to deep layers (Rohde and Scullion 1990; Mahoney et al 1993). The MICHBACK program uses a regression equation developed by (Baladi 1993) which iteratively improves the depth as described in (Mahmood 1993).

2.6 Backcalculation of Layer Moduli of Rigid Pavement

The modulus of subgrade reaction (k) can be determined from deflection testing conducted at the center of a Portland Cement Concrete (PCC) slab. An empirical set of equations known as the AREA method can be used to backcalculate k as well as the elastic modulus (E_C) of the concrete. Correlation equations have been developed to convert k to MR (AASHTO 1993).

2.6.1 Backcalculation Methods for Rigid Pavement

The AREA method for calculating the radius of relative stiffness and dynamic foundation k is presented in this subsection. A summary of various other methods can be found in (Sessions 2008).

The method for backcalculation of layer moduli of rigid pavement used in this study is based on (Frabizzio 1998). The method is based on calculating the area of the deflection basin, the radius of relative stiffness (l), the elastic modulus of the concrete (E_c), and the modulus of subgrade reaction using the measured deflection data as shown in equations 2.14 through 2.18.

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

Equation 2.14

$$l = \left[LN\left(\frac{60 - AREA}{289.708} \right) / (-0.698) \right]^{2.566}$$ Equation 2.15

$$E_c = \frac{12(1 - \nu^2) P l^2 \delta_r^*}{\delta_r h^3}$$ Equation 2.16

$$\delta_r^* = a \exp^{\left[-b \exp^{(-cl)}\right]}$$ Equation 2.17

$$k = \frac{E_c h^3}{12(1 - \nu^2) l^4}$$ Equation 2.18

Where, AREA = deflection basin area, inches

δ_r = deflection of the rth sensor, inches

l = radius of relative stiffness, in

E_c = elastic modulus of the concrete, psi

υ = Poisson's ratio for concrete = .15

P = FWD load, pounds

δ_r^* = non-dimensional deflection coefficient at distance "r"

h = concrete slab thickness, inches

a, b and c = regression coefficients (see Table 2.5)

k = modulus of subgrade reaction, pci

AREA is the cross-sectional area of the deflection basin between the center of the FWD load plate and the outer most deflection sensor. The radius of relative stiffness (l) characterizes the stiffness of the slab-foundation system. It should be noted that the final

Table 2.5 Regression coefficients for δ_r^*

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

elastic modulus of the concrete slab is the average of the seven elastic modulus values (one for each deflection sensor) obtained from equation 2.17 (Frabizzio 1998).

Equation 2.19 is used in the AASHTO pavement design guide to convert k into MR.

$$MR = k * 19.4 \qquad \text{Equation 2.19}$$

2.6.2 Rigid Pavement Temperature Effect on Resilient Modulus

Temperature can play a huge roll in the accuracy of deflection testing of concrete pavements. A concrete slab experiencing a temperature gradient can curl and come out of contact with the underlying material. Curling is more likely to occur on slabs supported by high-strength stabilized bases than those supported by soft bases. To avoid possible slab curling, testing the middle of the slab should be avoided during the day when the surface is hotter than the bottom of the slab and upward curling is taking place. Likewise, testing the corners and edges of the slab should be avoided at night when the slab surface is colder than the bottom of the slab and downward curling is taking place (Kathleen et al 2001).

2.6.3 Slab Location Selection for NDT

Conducting NDT at different positions on a PCC slab can be done to test for different pavement properties and conditions. Discussion of mid-slab, edge, and corner slab loading follows.

2.6.3.1 Mid-Slab Loading

The middle of the slab, in the outer lane, is usually where FWD tests are conducted for backcalculation of roadbed k-values. An infinite horizontal layer is assumed when considering rigid pavements, due to the evenly distributed load under a loaded slab. However, the standard 12-ft highway lane width is smaller than that required for the assumption of an infinite horizontal layer, but this is often ignored. The middle of the slab is tested to create the largest distance from pavement joints and edges, and from any distresses at these locations (Kathleen et al 2001).

2.6.3.2 Joint Loading

Loading near the joint of a concrete slab is usually done to calculate load transfer efficiency (LTE). One sensor can be placed on the loaded slab and all others on the unloaded slab. The ratio between the approach and leave slab deflection is used in calculation of LTE. The deflection measured from the 60-inch sensor can be used for roadbed MR backcalculation (Kathleen et al 2001).

2.6.3.3 Edge Loading

Loading the edge of a concrete slab is done to estimate the slab support to its adjacent structure, shoulder, or lane, as well as the presence of voids underneath the slab. This testing location is not normally used for backcalculation purposes.

2.7 Comparison of Backcalculated and Laboratory Determined MR Values

A brief summary of the comparison between backcalculated and laboratory determined MR value is presented in this section. The detailed comparison can be found in (Sessions 2008).

The primary purpose of establishing relationships between backcalculated and laboratory determined MR values are for pavement overlay design. The MR values are stress dependent. Therefore, in order to compare the different modulus values, the stress state in which the FWD test was performed must be known (George 2003). However, the grain size distribution, water content, saturation, dry unit weight, and other factors are often unknown in FWD testing. For this reason, it is very difficult to compare backcalculated to laboratory determined MR values.

Whether the backcalculated or laboratory determined MR of the roadbed soil is used in the pavement design and analysis depends on the input required for the model being used. For example, the original American Association of State Highway Officials (AASHO) road test was calibrated to the laboratory MR of the soil present at the test, clay. Therefore, when using the 1993 AASHTO pavement design or overlay procedures the appropriate input for the roadbed soil is the laboratory MR (AASHTO 1993).

In many other cases, MR values obtained from laboratory tests may be considerably lower than the backcalculated MR values. This is due largely in part to differences in the magnitudes of the deviatoric stress, confining pressure, and loading rate (George 2003). Similarly, field MR values for fine grained soils, obtained by backcalculation from FWD deflections, have been reported in a number of studies to exceed the laboratory MR values by factors between 3 and 5 (AASHTO 1993).

Several correlations have been developed to compare backcalculated and laboratory determined roadbed MR values. These correlations identify and allow correction for various factors which can lead to inflated or deflated MR values. Another study found similar results when layer theory was employed for the analysis of the stress state under a 9000 pound FWD load. It was found that a reasonable correlation exists between FWD backcalculated moduli and the laboratory moduli based on the in-situ conditions with identical stress states (Ping et al 2002).

$$MR_{FWD} = 1.6539 MR_{lab} \qquad \text{Equation 2.20}$$

The FWD backcalculated moduli were about 1.65 times higher than the laboratory MR. This ratio is in agreement with the suggestion by the AASHTO design guide (AASHTO 1993), which suggests that the FWD backcalculated moduli are approximately two to three times higher than the laboratory determined moduli. It must be remembered to consider that the AASHTO relationships were based primarily on clay soils. In addition, for this comparison the FWD tests were performed under in-situ soil conditions and the laboratory determined MR were obtained from the reconstituted soil samples; simulating the in-situ moisture and density conditions under identical stress states. The possible causes for the difference between the lab *MR* and backcalculated values as reported in the study (Ping et al 2002) were:

- The FWD backcalculation program is based on the assumption of linear elastic theory of multiple layer pavement structures, while the pavement materials are not purely elastic.

- The FWD backcalculation method does not lead to a unique solution; therefore, different layer moduli could be obtained from the same FWD data.
- The lab specimens were tested almost immediately after they were compacted, and the confining pressure for the triaxial test was applied by air; the in-situ soil had been there for many years, and the confining pressure was caused by vertical load and soil weight.

Von Quintus and Killingsworth (1998) reported that, for unbound granular materials, the ratio of the backcalculated to the laboratory determined MR ranged from 0.1 to 3.5. They stated that the reasons for the differences between the backcalculated and the laboratory determined MR values are related to the inability to simulate the in-situ boundary conditions in the laboratory.

2.8 Seasonal Changes

Pavement layers have varying properties and characteristics dependant on the time of the year. Pavements residing in areas that undergo freeze-thaw cycles are subject to seasonal effects. A pavement system can become very weak during the spring thaw season, then rapidly recover strength leading into summer, slowly recover over the summer and fall, and then reach a maximum stiffness when frozen during the winter (Shepherd and Vosen 1997). A typical annual range in deflection is shown in Figure 2.11.

2.8.1 Spring Season

During the winter season, un-drained water within the pavement, along with water from shallow water tables, can freeze and create ice lenses. Due to this, the surface can experience frost heave. When the spring season begins, and the lenses start to melt, the pavement layers can become saturated if not properly drained. Also, additional water can

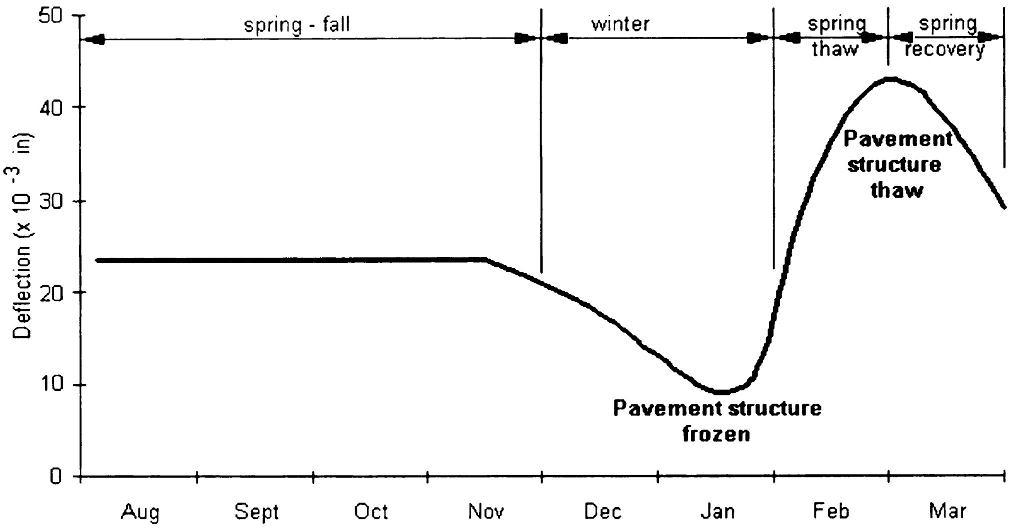


Figure 2.11 Typical pavement deflections illustrating seasonal pavement strength changes (PTC 2008)

enter the system from rain and snow melt. All pavement layers can experience a reduction in bearing capacity as a result of this. It is estimated that 90% of damage to pavements occurs during the spring thaw season (Janoo and Greatorex 2002). A diagram showing the formation of ice lenses can be seen in Figure 2.12.

2.8.2 Summer Season

The summer season is considered to start after the conclusion of the spring-thaw season which is defined by the time when moisture conditions, within the pavement system, return to normal and the ambient temperatures begin to rise. The date when this occurs changes from year to year and from location to location. The summer-fall season ends when the ground starts to freeze, but is often considered to last until the spring-thaw season begins and the ice starts to melt. In Michigan, summer season is typically from May to December.

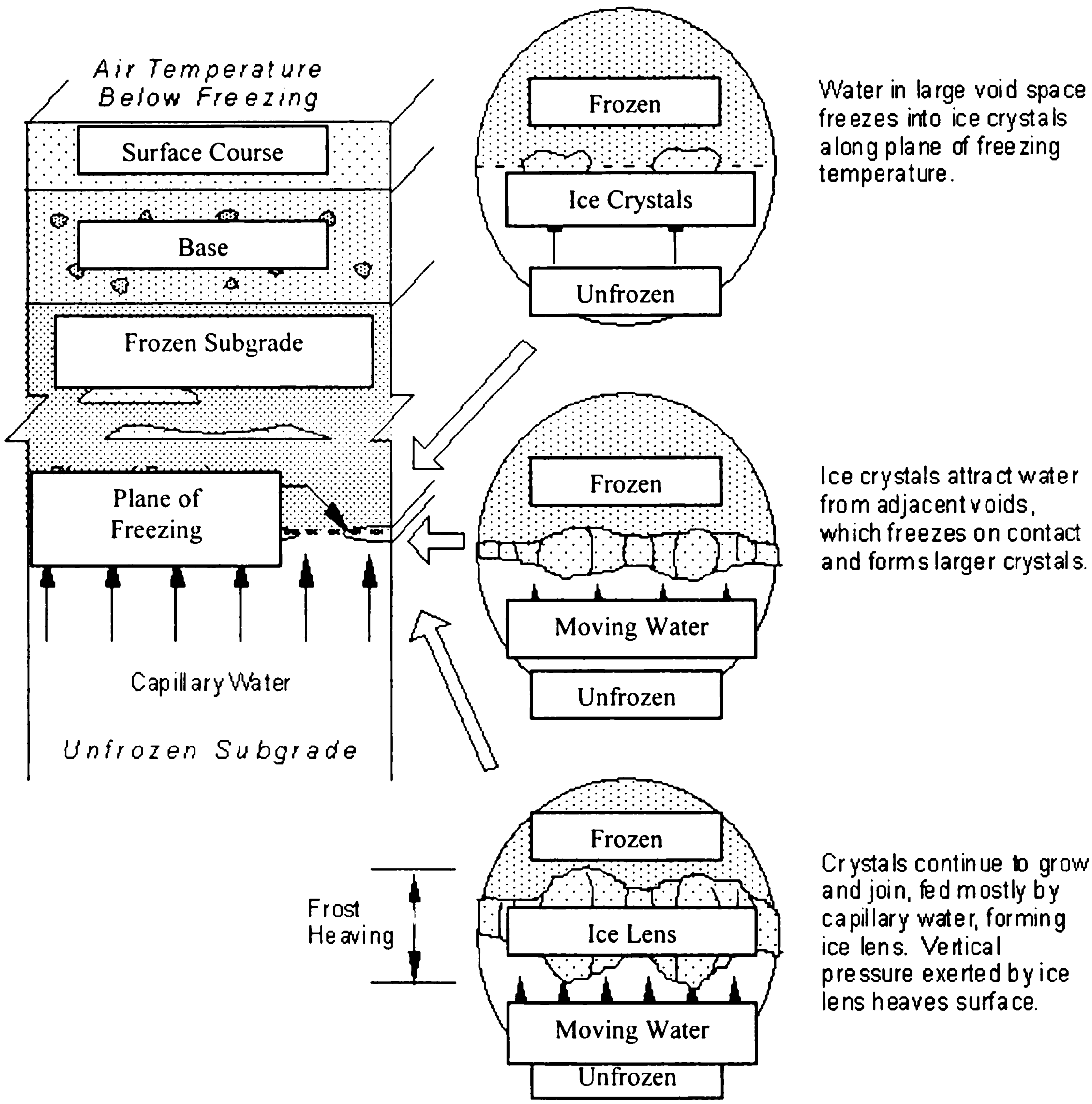


Figure 2.12 Formations of ice lenses in a pavement structure (PTC 2008)

2.9 Distribution of Resilient Modulus of Roadbed Soils in the State of Michigan

The entire state of Michigan is within the North American glaciated section. This implies that all soils have been deposited by glaciers. The action of the moving glaciers pulverized soil and bedrock while moving south, and then re-deposited the soil upon its melting and retreat. This created the topography of the state as well as the locations and variations of Michigan's soils. Within the state, 165 different soil classifications can be found. This includes everything from clay to boulders, and combinations throughout (MDSH 1970). All of these various classifications of soil deposited by glacial drifts can have different MR values and need to be classified and distinguished.

2.9.1 State Partitioning

A brief summary of the state partitioning process is presented in this subsection. The detailed process can be found in (Sessions 2008).

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 engineering and physical characteristics. The boundaries of the 15 clusters were established based on the 1982 Quaternary Geology map of Michigan, inputs from members of the Research Advisory Panel (RAP) of MDOT, and inputs from the soil engineers in the various MDOT Regions. After establishing the cluster boundaries, each cluster was divided into areas based on the percentages of each soil type found in the Natural Resources Conservation Service (NRCS) Web Soil Survey (Web Soil Survey 2007). Once again, the 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 state divisions consisted of 99 areas within the 15 clusters. Figure 2.13 depicts the boundaries of the clusters shown by the dashed lines and the boundaries of the 99 areas shown by the solid lines. Once again it should be noted that the division between the clusters was based on similar (not the same) soil types whereas the boundaries between the areas were based on narrowing the range of the soil parameters within each cluster.

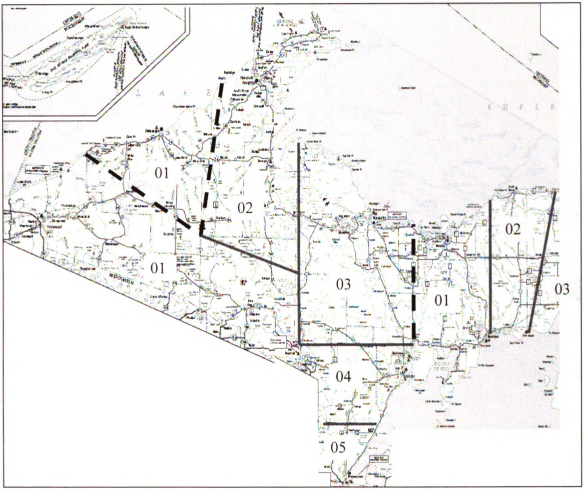


Figure 2.13 Areas and clusters of the State of Michigan

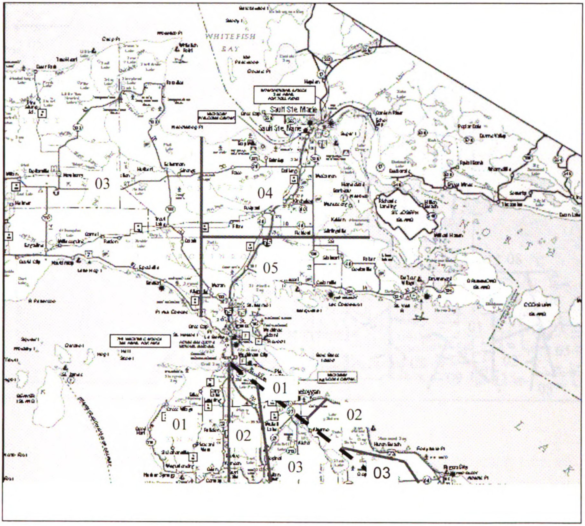


Figure 2.13 (cont'd)

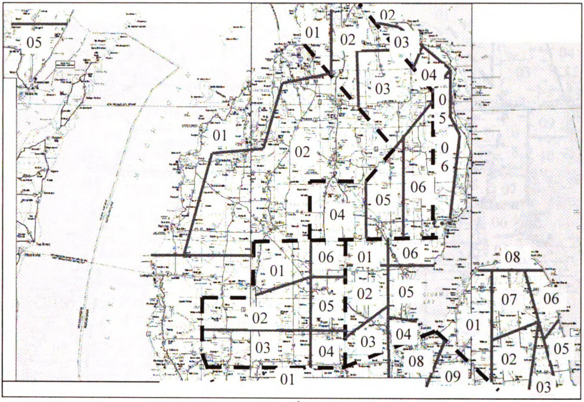


Figure 2.13 (cont'd)

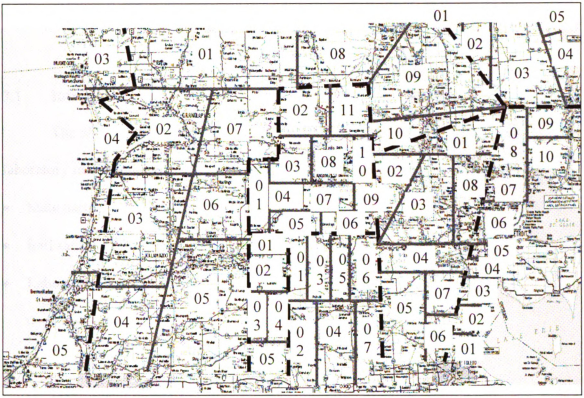


Figure 2.13 (cont'd)

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 quantified from the Natural Resources Conservation Service (NRCS) Web Soil Survey (Web Soil Survey 2007). Table A.1, in Appendix A, lists the percentages of each soil type in each of the 99 areas.

2.9.2 Soil Sample Collection

Of the 99 areas listed above, 75 have had disturbed soil samples collected from near the roadway. Areas with similar soils to each other were lumped together, for economic reasons, and only one sample was collected to represent both areas. The soil samples were analyzed at Michigan State University (MSU) for natural moisture content, Atterberg Limits (liquid and plastic limits and plasticity index), grain size distribution (wet and dry sieving and hydrometer analysis), and cyclic load triaxial tests.

CHAPTER 3

LABORATORY & FIELD INVESTIGATION

3.1 Introduction

The objectives of this study were achieved by carrying out several field and laboratory investigations. These investigations include:

- State partitioning
- Soil sampling
- Laboratory tests which consist of:
  - Moisture content
  - Sieve analysis (wet and dry sieving)
  - Hydrometer analysis
  - Atterberg limits (liquid and plastic limits and plasticity index)
  - Cyclic load triaxial test
- Field tests which consist of:
  - Penetration resistance using pocket size penetrometer
  - Shear strength using pocket vane shear tester
  - Deflection using Falling Weight Deflectometer (FWD)

3.2 State Partitioning and Soil Sampling

The state was divided into 15 clusters and 99 areas as discussed in subsection 2.9.1. At every location (75) where a disturbed soil sample was collected, as discussed in subsection 2.9.2, penetration resistance and vane shear tests were conducted on site. The roadbed soil samples were taken to the Geotechnical laboratory at MSU for further testing. In addition to the disturbed soil samples, 10 undisturbed (Shelby tube) samples were collected by MDOT and taken to the laboratory at MSU for testing. State partitioning and soil sampling is discussed in detail elsewhere (Sessions 2008).

3.3 Laboratory Tests and Procedures

All 81 disturbed soil samples, as well as the 10 Shelby tube samples were analyzed in the Geotechnical Laboratory at MSU. Each sample was subjected to a battery of tests to determine its moisture content, particle gradation, Atterberg limits, soil classification (both USCS and AASHTO soil classification system), and MR. A brief description of each test administered follows, a full explanation of each test can be found in (Sessions 2008).

3.3.1 Moisture Content, Particle Gradation, and Atterberg Limits

All 81 soil samples collected underwent natural moisture content, particle gradation (dry and wet sieve and hydrometer), and Atterberg limit analyses. The following standard test procedures were followed:

- Moisture content analysis - ASTM C 29
- Dry sieving - ASTM C 117
- Wet sieving - ASTM C136
- Hydrometer analysis - AASHTO T 88
- Atterberg limit analysis - AASHTO T 89

A detailed review of the tests and their effects on the MR values can be found in (Sessions 2008).

3.3.2 Cyclic Load Triaxial Test

Cyclic load triaxial tests were conducted to determine the resilient modulus of laboratory compacted sand and clay samples as well as Shelby tube samples. The sand samples were compacted in a split mold by vibration and static load. The clay samples were compacted according to AASHTO standard proctor test procedure T99 and then trimmed to the correct diameter. Shelby tube samples were simply cut into sections of proper length. All samples were contained within a rubber membrane. All cyclic load triaxial tests were mainly conducted according to the AASHTO T307 standard test procedure. Because of the type of tests and equipment available some modifications to the procedure are detail in (Sessions 2008).

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.

3.4 Field Tests

Several thousand deflection tests using Falling Weight Deflectometer (FWD) were conducted and analyzed during this study. In addition, all 81 disturbed soil samples were tested in the field using pocket penetration resistance and vane shear testers.

3.4.1 FWD Tests

In 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 six thousand two hundred forty six 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 3.1). 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 3.1 shows the distribution of the FWD data files by pavement type (flexible or rigid pavement) and by roadbed soil USCS.

As can be seen from the Figure 3.1, 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 squares and triangles in Figure 3.1). Due to several constraints, the number of requested FWD tests was reduced several times. Finally, 56 additional FWD tests were conducted spanning fifteen roads (four M

Table 3.1Distribution of old FWD files

		Rigid pavement		Flexible pavement		Total	
		Files	Tests	Files	Tests	Files	Tests
USCS	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

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 3.1, and detailed in Table 3.2.

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

- Backcalculate layer moduli of flexible and rigid pavements
- Evaluate the variability in roadbed soil MR along and across the pavement network
- Study roadbed soil MR as a function of soil type
- Assess the seasonal effects on roadbed soil MR

The results of these analyses are presented and discussed in chapter 4.

Analysis of seasonal effects on roadbed soil MR could not be accomplished due to a lack of deflection data reflecting spring conditions. FWD tests were not performed during the spring season in this study due to equipment breakdown and MDOT limited resources.

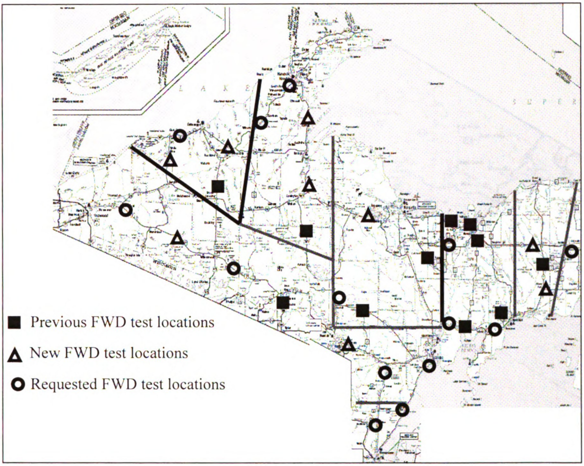
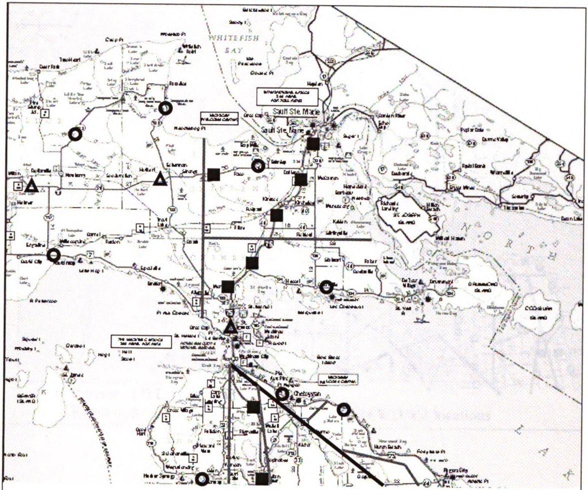


Figure 3.1 FWD test locations in the State of Michigan



■ Previous FWD test locations
△ New FWD test locations
○ Requested FWD test locations

Figure 3.1 (cont'd)

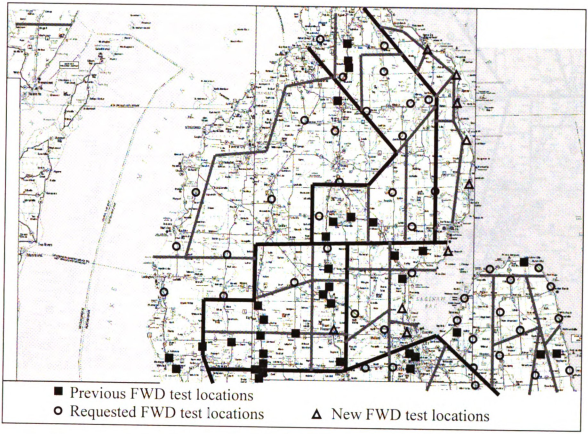


Figure 3.1 (cont'd)

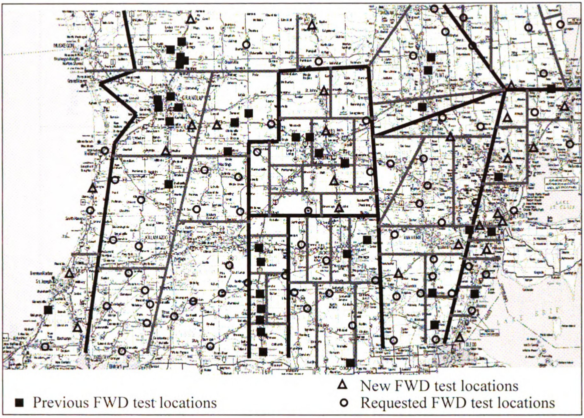


Figure 3.1 (cont'd)

3.4.2 Penetration Resistance and Vane Shear Test

Penetration resistance and vane shear tests were conducted in the field using hand held devices in order to capture in-situ conditions. The tests were conducted by measuring the soils penetration resistance using pocket penetrometer, and the shear strength resistance using pocket size vane shear tester. The results and analyses of these tests are discussed in (Sessions 2008) and can be seen in Table A.2 of Appendix A.

Table 3.2 New FWD test locations

Region	County	Control Section	Control Section BMP	Location	Pavement Type
Bay	Arenac	06073	7.439	Arenac/Osco Co Line South	Composite
Bay	Bay	09035	6.232	North of Beaver Rd	Rigid
Bay	Bay	09101	8.413	2048' West of Mackinaw Rd	Rigid
Bay	Gratiot	29011	13.609	North of Ithaca North City Limit	Rigid
Bay	Isabella	37014		100' North of Vernon Rd	Flexible
Grand	Ionia	34044	5.852	West of Portland	Rigid
Grand	Kent	41024	11.400	West of Ionia/Kent County line	Rigid
Grand	Montcalm	59012	3.592	North of Cannonsville Rd	Composite
Metro	Macomb	50112	4.664	West of Macomb/St. Clair Co Line	Composite
Metro	Macomb	50015	5.642	North of the end of the divided hwy (NB direction)	Flexible
Metro	Macomb	50013	0.807	North of Utica NCL	Rigid
Metro	Oakland	63173	10.282	Oakland/Genesee Co Line South	Composite
Metro	St. Clair	77024	0.000	East of Lapeer/St. Clair Co Line	Rigid
Metro	St. Clair	77111	12.536	450' West of Wadhams Road	Flexible
Metro	Wayne	82194	0.000	North of M-39	Rigid

Table 3.2 (cont'd)

Region	County	Control Section	Control Section BMP	Location	Pavement Type
Metro	Wayne	82022	7.620	West of M-39	Rigid
Metro	Wayne	82022	15.514	East of Rotunda Dr	Rigid
Metro	Wayne	82111		Station 38+00, 290' North of mile marker 13.2	Rigid
Metro	Wayne	82081	17.980	362' East of Greenfield Road	Flexible
North	Alcona	01052	11.728	0.2 miles South of Black River Rd North	Flexible
North	Alpena	04032	0.000	North of M-32	Flexible
North	Iosco	35032	16.482	North of Co Rd F-41 (Fullerton St)	Composite
North	Presque Isle	71073	14.481	Cheboygan/Presque Isle Co Line East	Flexible
North	Presque Isle	71073	0.000	West of M-66	Flexible
Southwest	Allegan	03033	0.000	North of Van Buren/Allegan County line	Rigid
Southwest	Allegan	03111	1.993	North of M-89	Rigid
Southwest	Berrien	11111	0.000	North of I-94	Composite
Southwest	Berrien	11057	8.929	North of Berrien Springs	Rigid
Superior	Baraga	07022	1.030	North of M-28 (area with passing lane)	Flexible
Superior	Baraga	07013	3.147	Baraga/Houghton Co Line South	Flexible

Table 3.2 (cont'd)

Region	County	Control Section	Control Section BMP	Location	Pavement Type
Superior	Baraga	07013	3.147	Baraga/Houghton Co Line South	Flexible
Superior	Chippewa	17061	7.250	West of M-123	Flexible
Superior	Gogebic	27022	0.797	East of Pierce St	Flexible
Superior	Luce	48041	11.068	West of M-117	Flexible
Superior	Mackinac	49025	0.607	North of US-2	Flexible
Superior	Marquette	52042	12.085	West of M-35	Composite
Superior	Menominee	55021	9.903	East of Potter Dr; WCL Powers	Composite
Superior	Ontonagon	66042	11.654	West of M-26	Flexible
Superior	Ontonagon	66012	0.000	North of M-28	Flexible
Superior	Schoolcraft	75061	13.250	West of M-77	Flexible
Superior	Schoolcraft	75021	2.285	section with passing lane	Flexible
University	Clinton	19033	0.000	North of I-69	Rigid
University	Clinton	19034	0.000	M-21 North to end of freeway	Rigid
University	Ingham	33031	11.600	North of M-36	Flexible
University	Ingham	33035	0.000	South of M-36	Composite
University	Monroe	58151		50 feet North of mile marker 7	Rigid
University	**Washtenaw**	**81031**	**11.560**	**1068 ' West of S Industrial Road**	Flexible

CHAPTER 4

DATA ANALYSIS & DISCUSSION

4.1 Analysis of Laboratory Test Data

A brief summary of the analysis of the laboratory test data is presented in the next two subsections. The detailed analyses can be found in (Sessions 2008).

4.1.1 Soil Classification

For each soil sample, the natural water content, the dry and wet sieve, the hydrometer, and the Atterberg Limits tests data were obtained and are listed in Table A.3 of Appendix A. The data were used to classify the soils according to the USCS and the AASHTO soil classification system. Results of the classification are also listed in Table A.3 of Appendix A. As can be seen in the table, the roadbed soils in the State of Michigan were divided into eight soil types according to the USCS; SM, SP, SC, SP-SM, SP-SC, ML, CL, and GW.

4.1.2 Cyclic Load Triaxial Test

For each disturbed and Shelby tube soil sample, at least one cyclic load triaxial test was conducted. In all tests a confining pressure of seven and a half psi, a sustained load of ten pounds and cyclic stresses of ten and fifteen psi were used. Some of the test parameters and the test results (the resilient modulus at load cycles 100, 200, 500, 800, and 1,000 and the average resilient modulus of at load cycles 500, 800, and 1,000) for each cyclic load are listed in Tables A.4. Table A.5 lists the sample parameters and the average resilient modulus for the ten and fifteen psi cyclic stresses. For each soil type, the average resilient modulus value at load cycles 500, 800 and 1000 was correlated to the soil physical parameters (moisture content, particle gradation, coefficient of uniformity, coefficient of curvature, liquid limit, plastic limit, plasticity index, dry density, percent passing certain sieves, degree of saturation, penetration resistance, and vane shear strength) using univariate and multivariate statistical analyses. Results of the analyses (the correlation equations) for all eight soil types are listed in Table 4.1.

Three important points should be noted herein are:

1. For the SC, CL, and ML soil types; similar trend between their parameters and MR values was found. Therefore, in the analyses they were grouped together.
2. For the SP soils, two distinctive trends between the MR values and the soil parameters were found. Hence, the SP soils were divided into two groups SP1 (the soil samples were obtained from the west side of the State of Michigan) and SP2 (the soil samples were obtained from the east side). The main difference between SP1 and SP2 is the course sand content. On average, the SP1 soil contains 90 percent passing sieve number 40 whereas SP2 soil, 50 percent.
3. As can be seen from table 4.1, the predictive equations apply to one or more soil types according to USCS. For example, the SP-SM soil has one predictive equation. Since for this soil, the AASHTO soil classification system yields three types of soil (A-1-b, A-2-4, and A-3), the USCS was used throughout the remainder of this thesis.

Two sets of additional cyclic load triaxial tests were conducted. The MR values and the sample parameters of both sets of tests were used to verify the MR predictive equations presented in Table 4.1. The results of the first set of additional tests are shown by the open squares in Figures 4.1 through 4.4. The results of the second set of tests, on the other hand, are shown by the open triangles in the figures. As can be seen from the figures, the results of both sets of tests are located relatively close to the solid curve

Table 4.1 MR Predictive equations

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	$MR = 1749.6\exp 0.0054(SVSP\text{-}SM)$	$SVSP\text{-}SM = \frac{\gamma_d^{1.75}}{MC^{0.5} + LL^{0.6} + (P_{40} - P_{200})^{0.01}}$
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

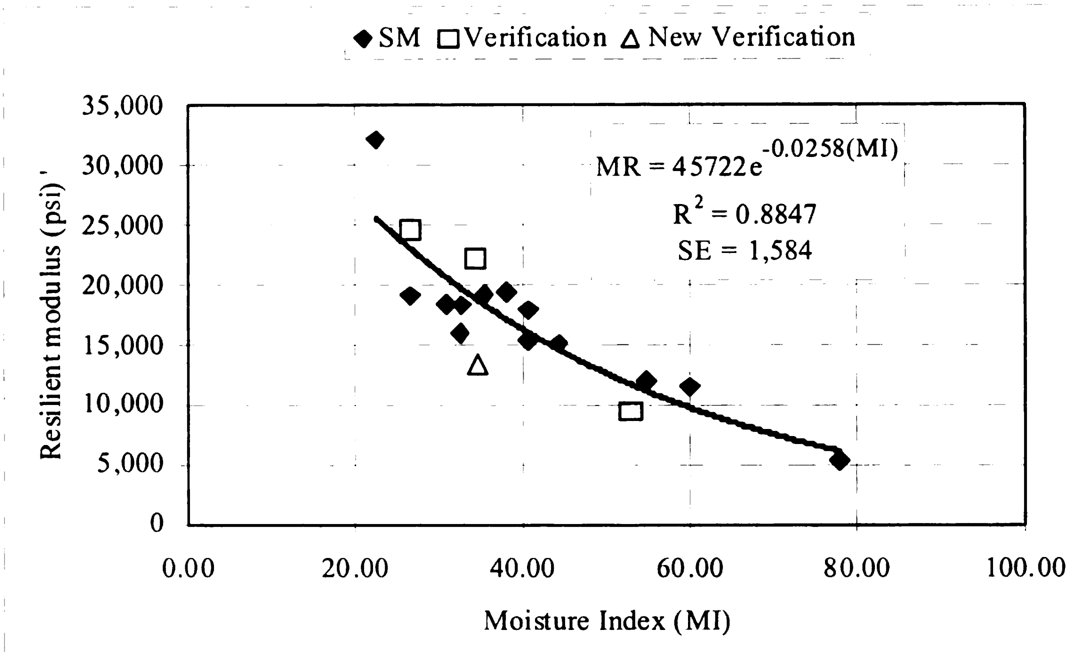


Figure 4.1 MR versus moisture index (see table 4.1) for the SM soils

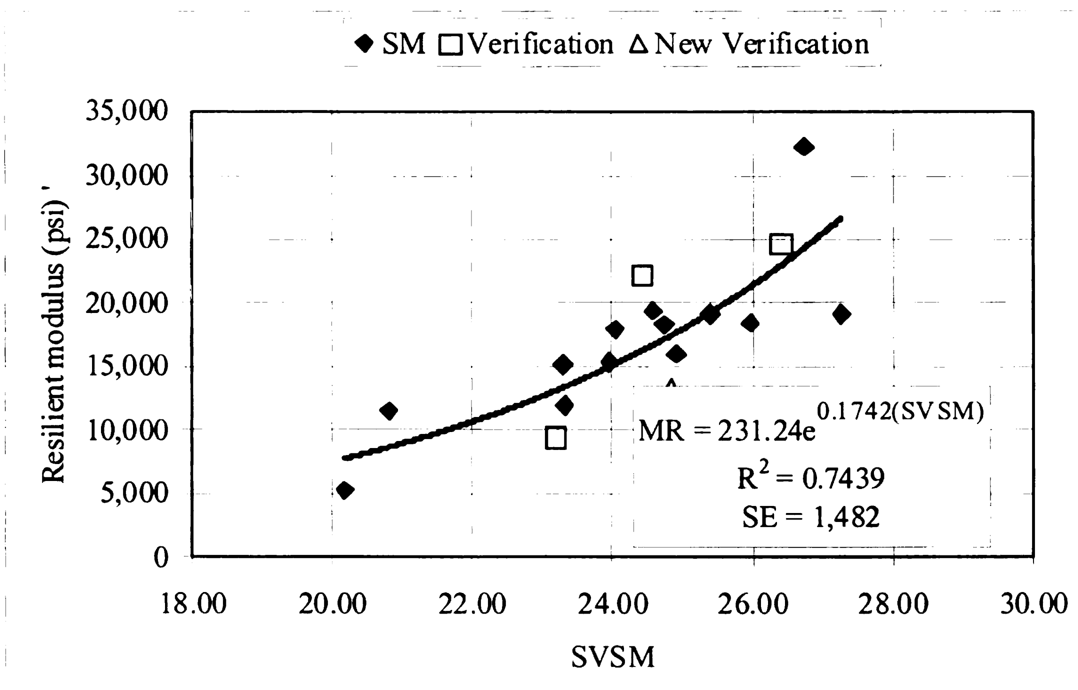


Figure 4.2 MR versus the SVSM (see table 4.1) for the SM soils

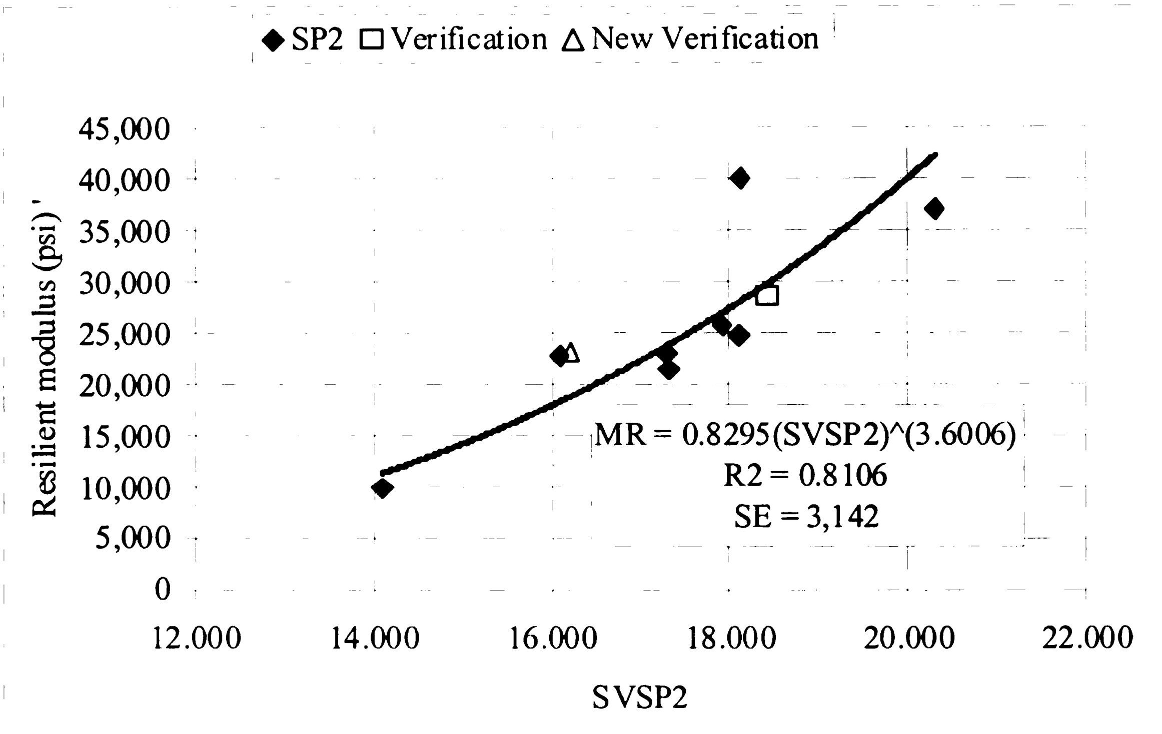


Figure 4.3 MR versus SVSP2 (see table 4.1) for the SP2 soils

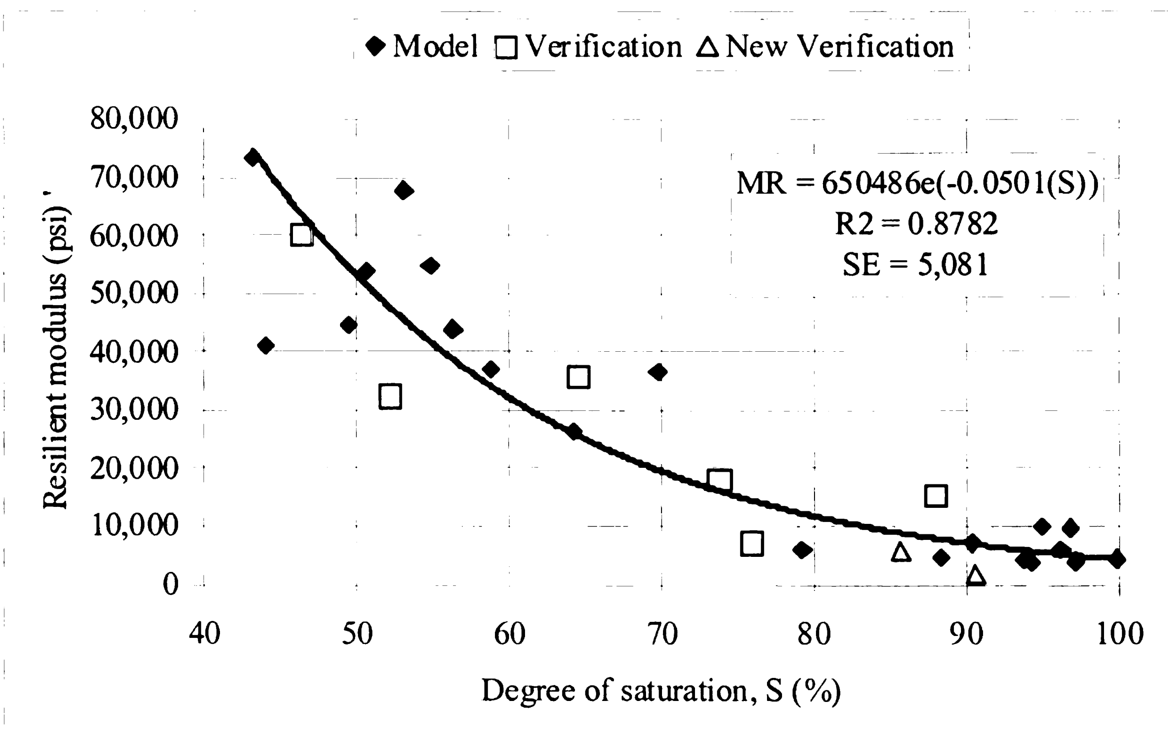


Figure 4.4 MR versus the degree of saturation (S, see table 4.1) for the SC, CL and ML soils

representing the predicted MR values. This implies that the MR predictive equations for the SM, SP2, and ML soils are reliable and relatively accurate. It should be noted that the correlation equation and the values of R^2 and standard error (SE) stated in Figures 4.1 through 4.4 were obtained based on the original data. When the additional data from the verification tests were included, the values of the statistical parameters of the equations were changed, and the values of R^2 and SE decreased.

Nevertheless, the results of the second set of additional cyclic load triaxial tests (verification tests) were also used to assess the impact of the applied stress boundary conditions and the sample moisture contents on the MR values of the test samples. These results are discussed in Section 4.4.

4.2 Backcalculation of Layer Moduli

As noted in Chapter 3, all existing FWD data files for flexible and rigid pavements were requested and obtained from MDOT. The locations of these tests were marked on a state map. Additional FWD test locations were then determined to fill the gap. Consequently, MDOT conducted the FWD tests at the new locations. These along with the old FWD tests provided good coverage of the pavement network in the State of Michigan (see Chapter 3).

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 4.5. 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.

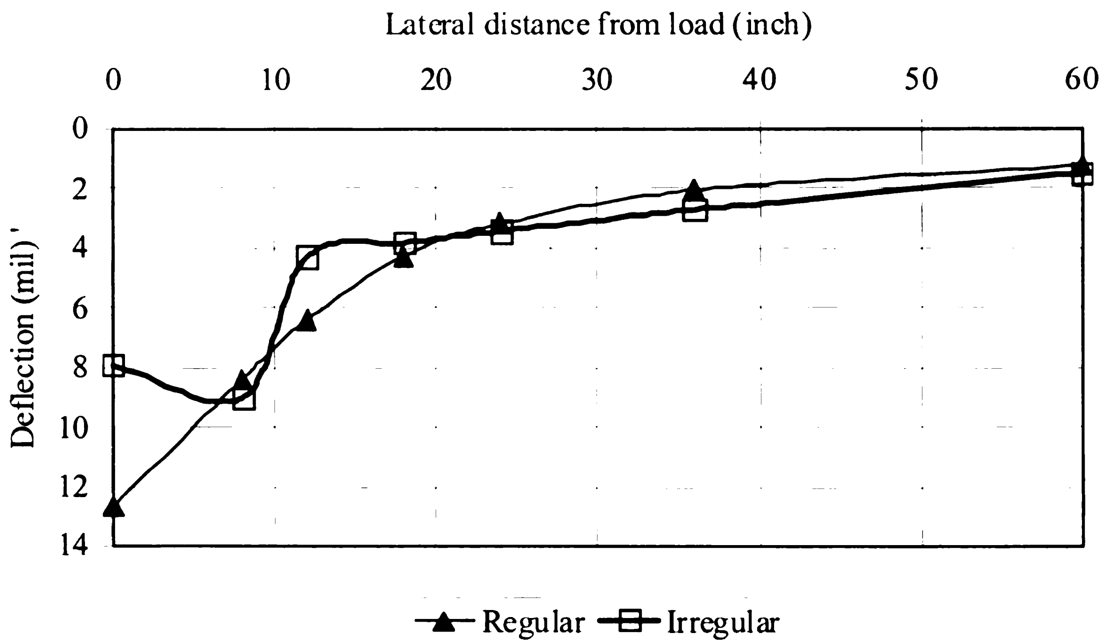


Figure 4.5 Regular and irregular deflection basins

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 backcalculations are presented and discussed in subsection 4.2.1.2.

4.2.1.1 Sensitivity of the Backcalculated Moduli

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 more accurate 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 4.6 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 MR of the roadbed soil is not affected much when a single granular base layer is used. Therefore, the base/subbase combination is appropriate when backcalculating roadbed soil MR.

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, due to construction practices the AC thickness may vary +/- 1 inch from the average. Figure 4.7 shows that when the AC thickness is

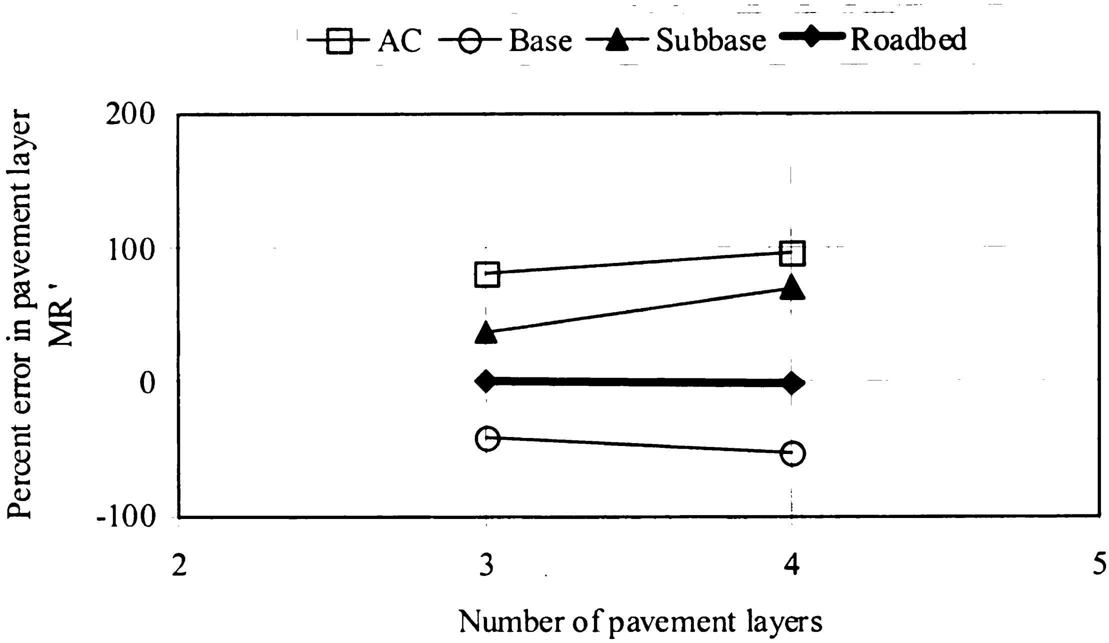


Figure 4.6 Effect of number of pavement layers

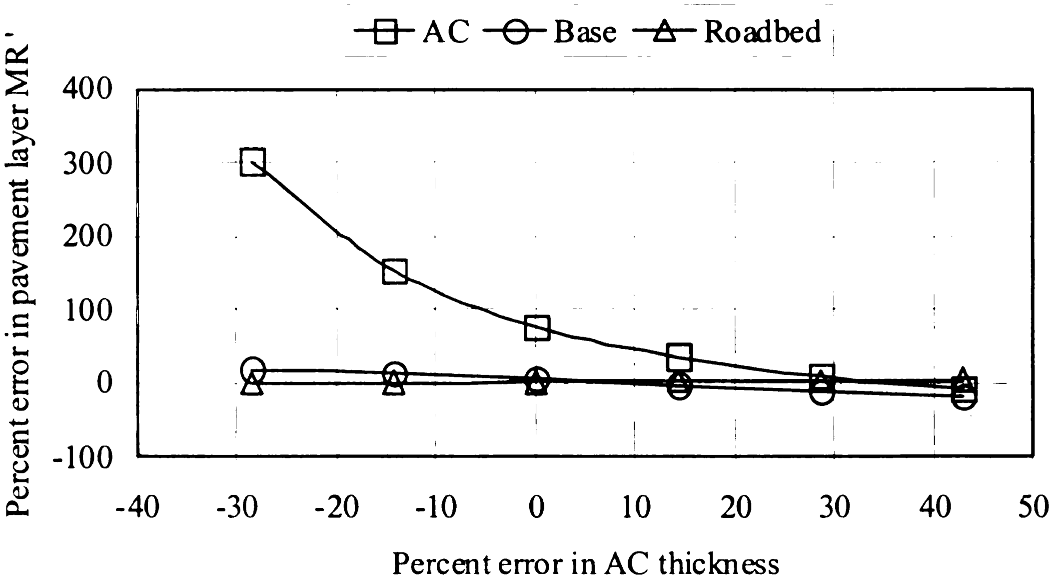


Figure 4.7 Effect of AC layer thickness on MR

varied, to reflect possible conditions, the backcalculated AC MR is drastically affected, while the other layers remain generally constant. The roadbed soil MR is more or less unaffected by changes in the AC layer thickness.

Similarly, Figure 4.8 shows that varying base thickness does not have much effect on the backcalculated roadbed soil MR. However, the backcalculated MR of the base and AC layers is affected by varying the base thickness.

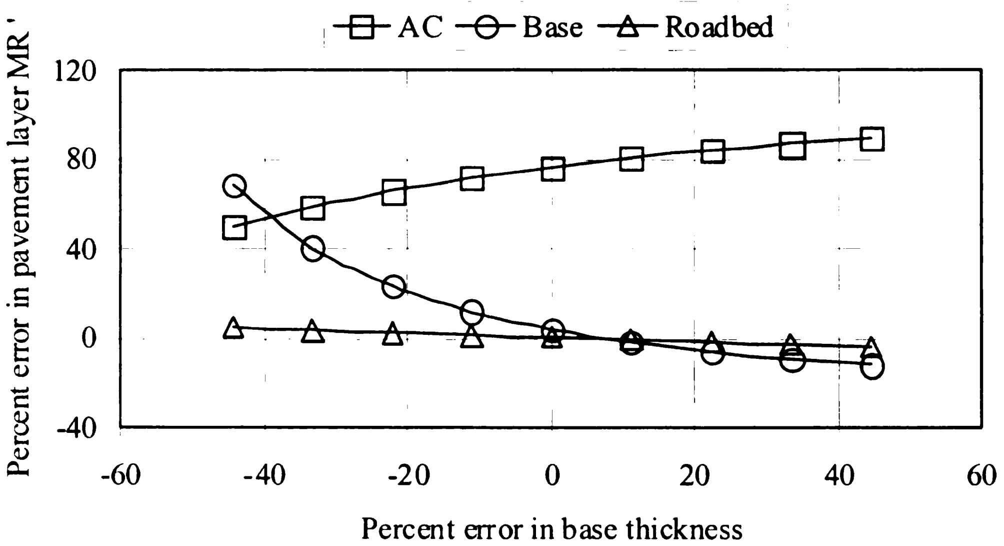


Figure 4.8 Effect of base layer thickness on MR

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.

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

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

Where, $E_O(r)$ = surface modulus at a distance r from the center of the FWD loading plate

μ = Poisson's ratio (0.5 assumed)

σ_o = contact stress under the loading plate (82 psi) d(r) = deflection at a distance r (inch)

a = radius of loading plate (5.91 inch)

By calculating E_0 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 4.9 through 4.12.

a) No stiff layer exists

b) A stiff layer at a shallow depth exists

c) A stiff layer at a deep location exists

d) A soft layer at a deep location exists

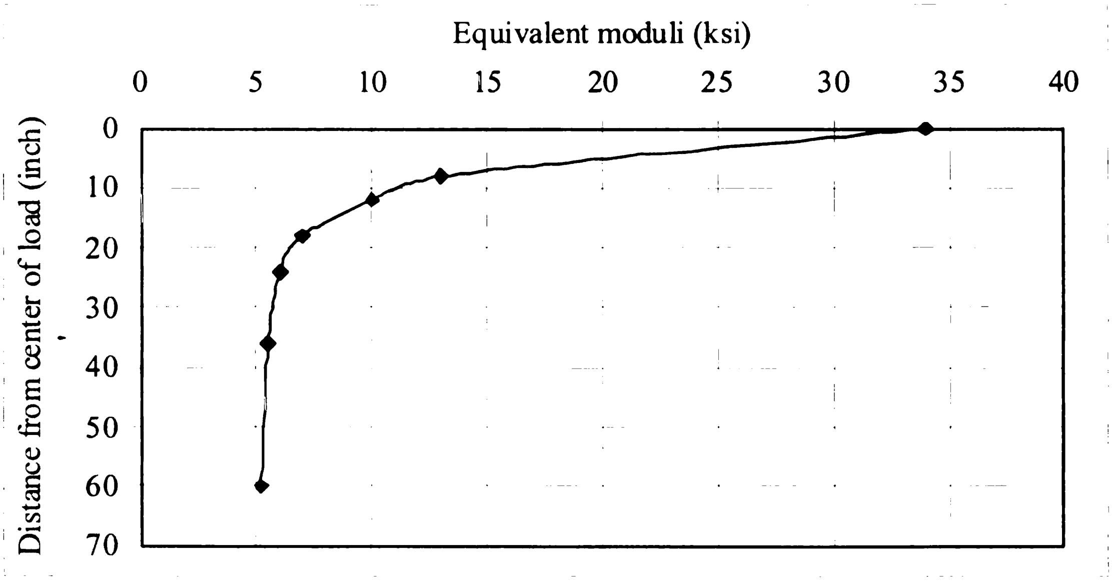


Figure 4.9 No stiff layer

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 of the roadbed soil can be greatly affected.

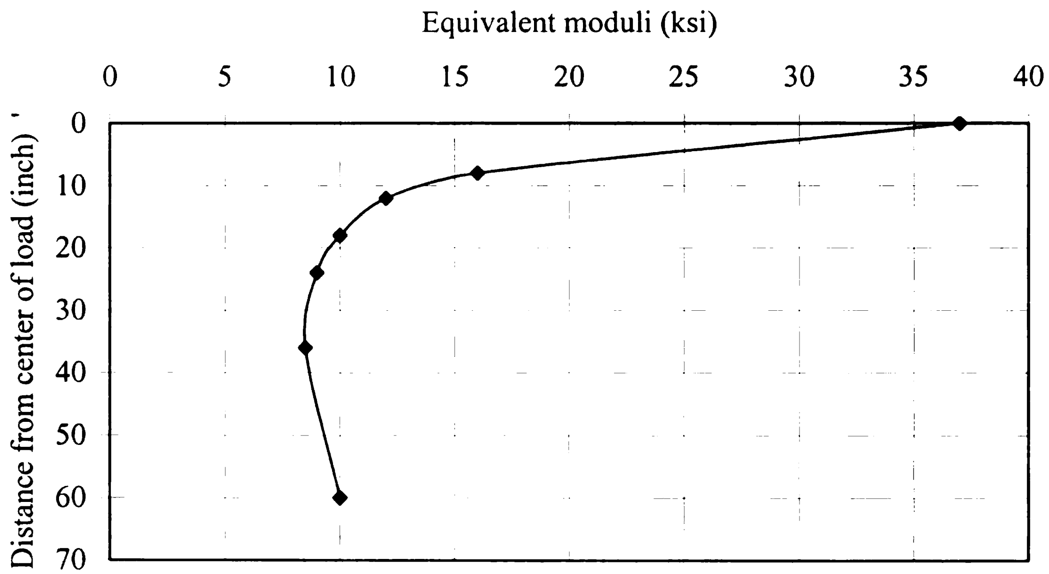


Figure 4.10 Stiff layer at shallow depth

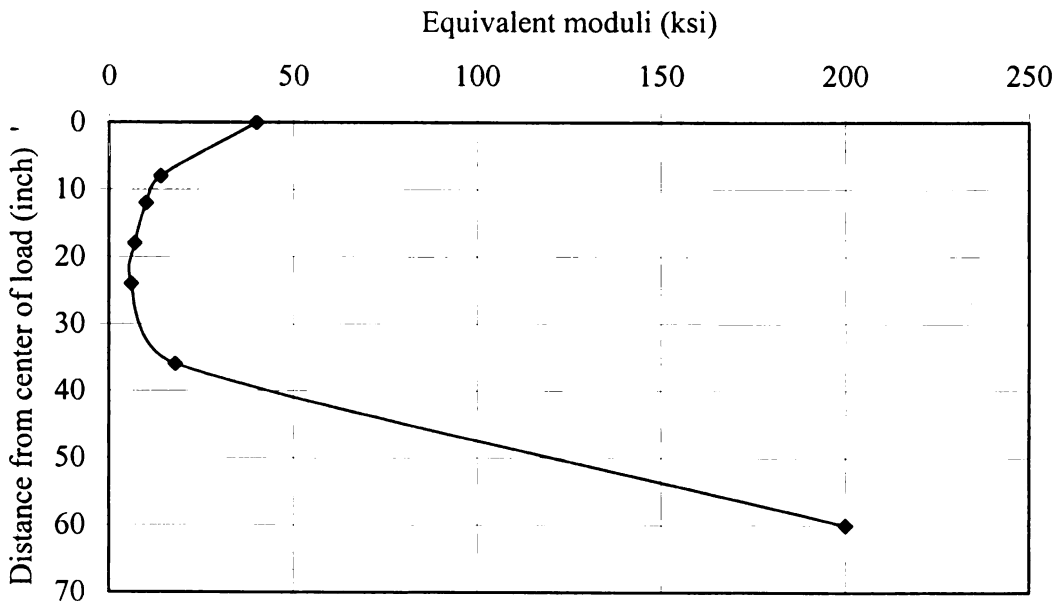


Figure 4.11 Stiff layer at deep location

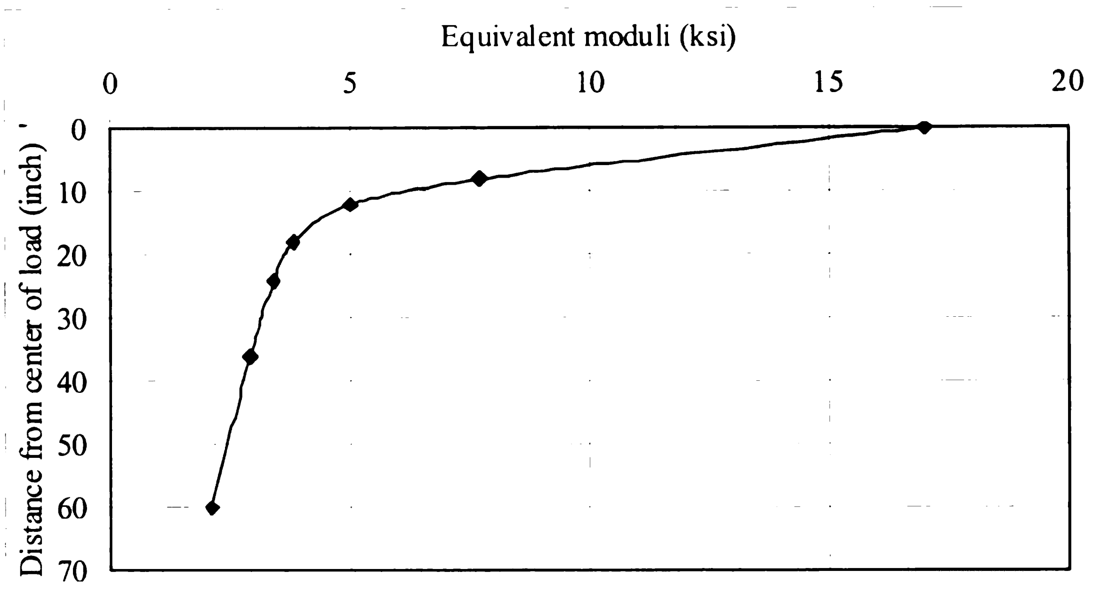


Figure 4.12 Soft layer at deep location

This procedure was tested by using forward calculation of pavement response to applied loads and backcalculating the layer moduli. Figure 4.13 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 4.14 illustrates that the MR of the stiff layer has almost no affect on the backcalculated layer moduli. To be considered a stiff layer the MR 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 4.15 shows that variation in the roadbed seed modulus does not have much impact on the backcalculated MR values.

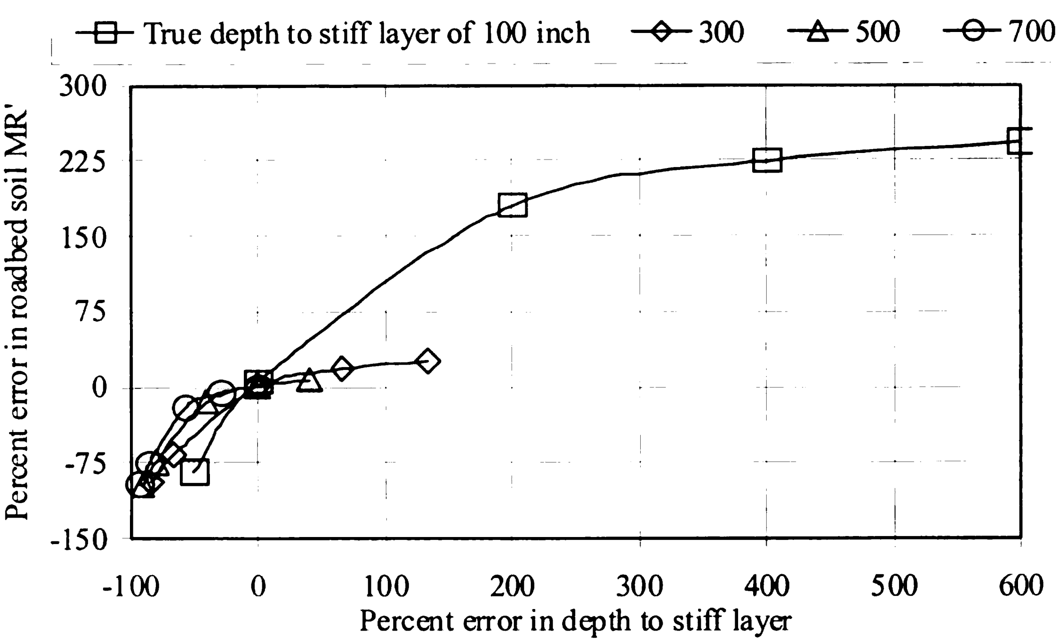


Figure 4.13 Effect of stiff layer depth

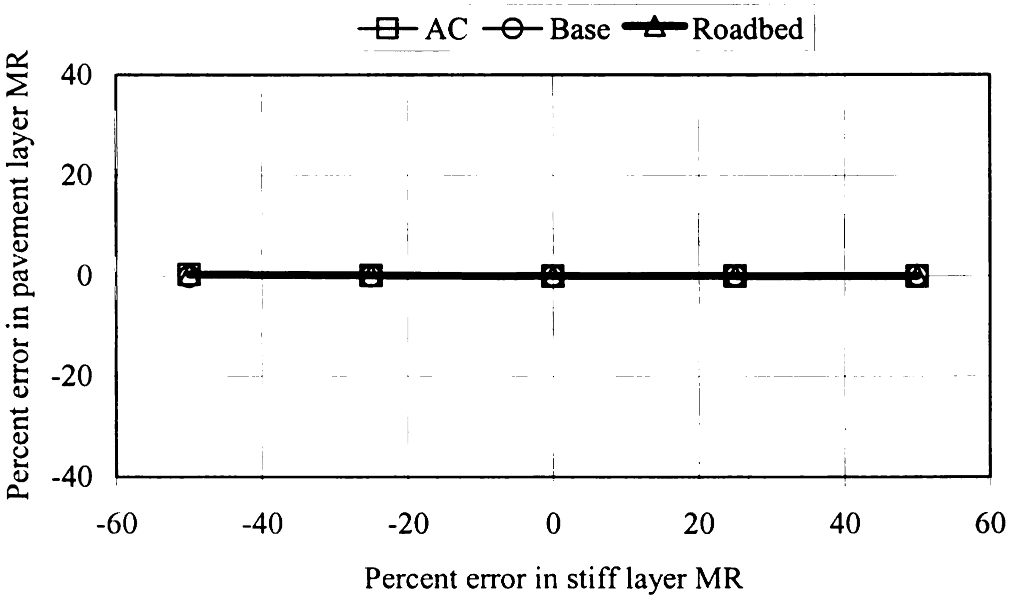


Figure 4.14 Effect of stiff layer MR

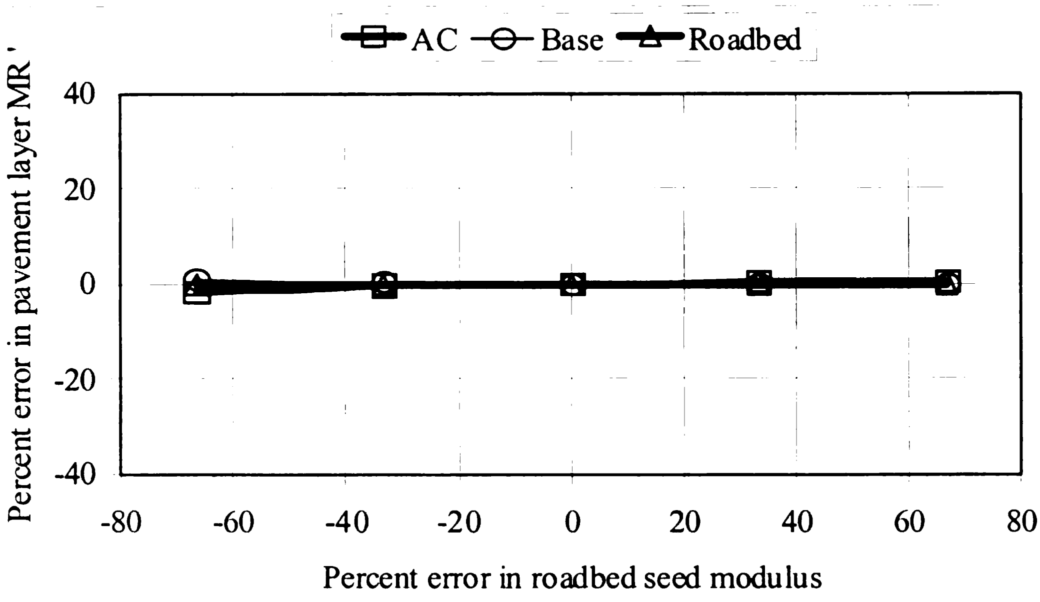


Figure 4.15 Effect of roadbed seed MR

The range of MR values specified is important, as the values used 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)

4.2.1.2 Analysis of Backcalculated Data from MICHBACK

The accuracy of the backcalculated results were also 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 a 2% RMS error were eliminated. A maximum RMS error of 2% was established for acceptance of the backcalculated MR results because errors above this threshold are much less accurate.

- The deflection measured at the sensor 60 inch 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 and less) are not as closely related to the MR of roadbed soils. This is illustrated in Figure 4.16 where the open triangles represent 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 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 accuracy of the correlation between d_{60} and the MR of roadbed soils.

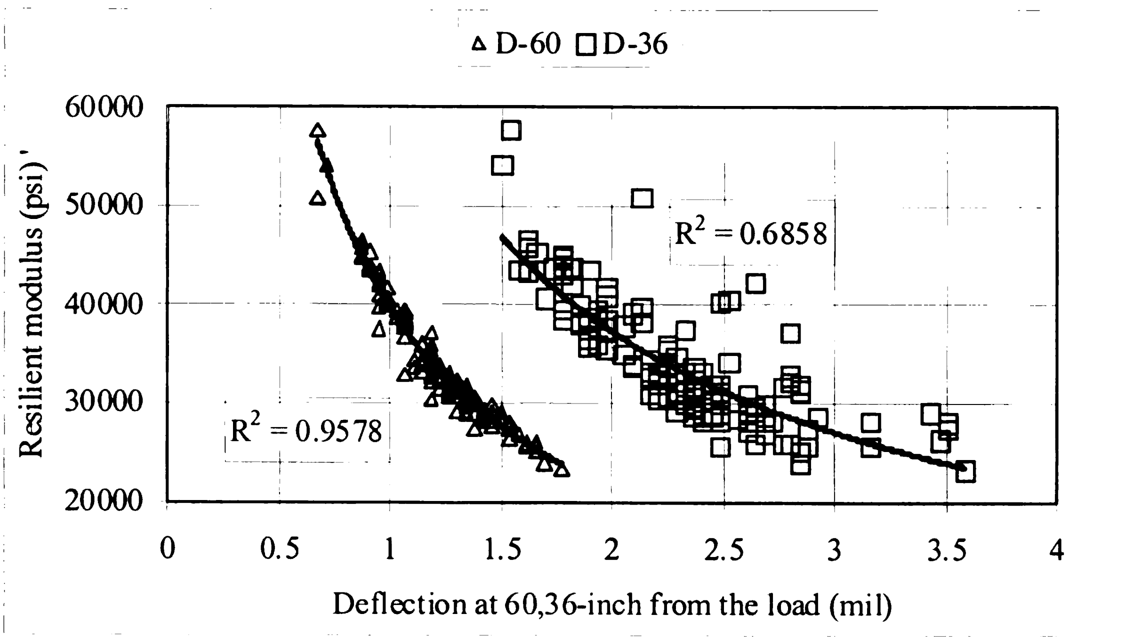


Figure 4.16 MR vs. d_{60} and d_{36}

- The deflection measured at the sensor 60 inch from the load is inversely proportionate to the backcalculated roadbed soil MR. An increase in measured deflection corresponds to a decrease in backcalculated MR and vise versa, as illustrated by Figure 4.17. Due to this relationship, the accuracy of the MICHBACK results can be scrutinized based on an observation of this trend.

Results - Only the backcalculated results of roadbed soil MR were further analyzed. The raw results of base/subbase and AC MR are listed in Table B.1 of Appendix B, and will be further discussed as part of the upcoming unbound material MR project. The average, maximum, minimum, and standard deviation of MR of backcalculated roadbed soil supporting flexible pavements are listed in Table 4.2, and the detailed results are listed in Table B.1. The full set of NDT data and backcalculated results of flexible pavements is available on the accompanying compact disc.

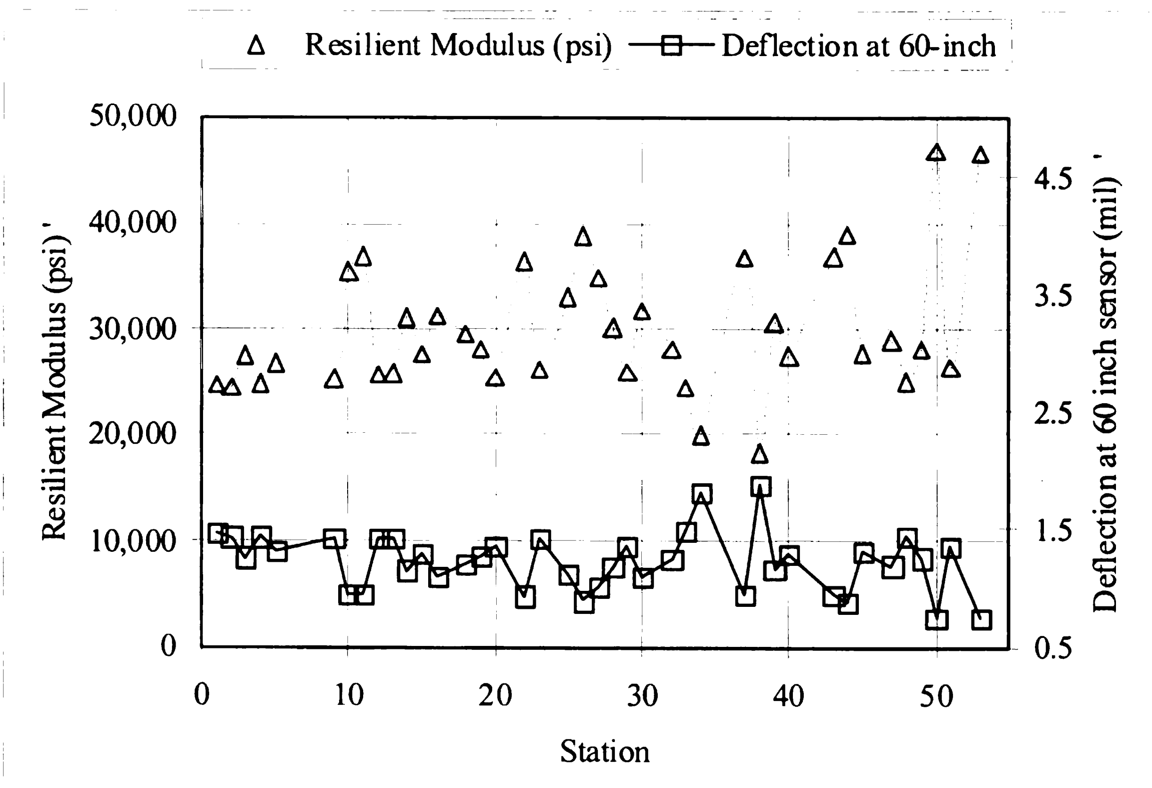


Figure 4.17 MR vs. deflection

Table 4.2 Backcalculated roadbed soil MR 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	18,989	31,218	7,088	6,541
SC	24,704	67,793	11,728	6,695
CL	20,100	28,849	11,996	4,326
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). The following equations were repeated from subsection 2.6.1 for the reader's convenience.

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

Equation 4.3

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

Equation 4.4

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

$$E_c = \frac{12\left(1-\nu^2\right)Pl^2\delta_r^*}{\delta_r h^3}$$ Equation 4.6

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

$$MR = 19.4k$$ Equation 4.8

Where, AREA = deflection basin area, inches

δ_r = deflection of the rth sensor, inches

l = radius of relative stiffness, inches

E_c = elastic modulus of the concrete, psi

υ = Poisson's ratio for concrete = .15

P = FWD load, pounds

δ_r^* = non-dimensional regression coefficient at distance "r"

h = concrete slab thickness, inches (use 9" if unknown)

a ,b, and c = regression coefficients (see Table 4.3)

k = modulus of subgrade reaction, pci

MR = resilient modulus, psi

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

Table 4.3 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

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 4.18 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 4.19 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) was divided by Equation 4.10, which resulted in Equation 4.12 as follows:

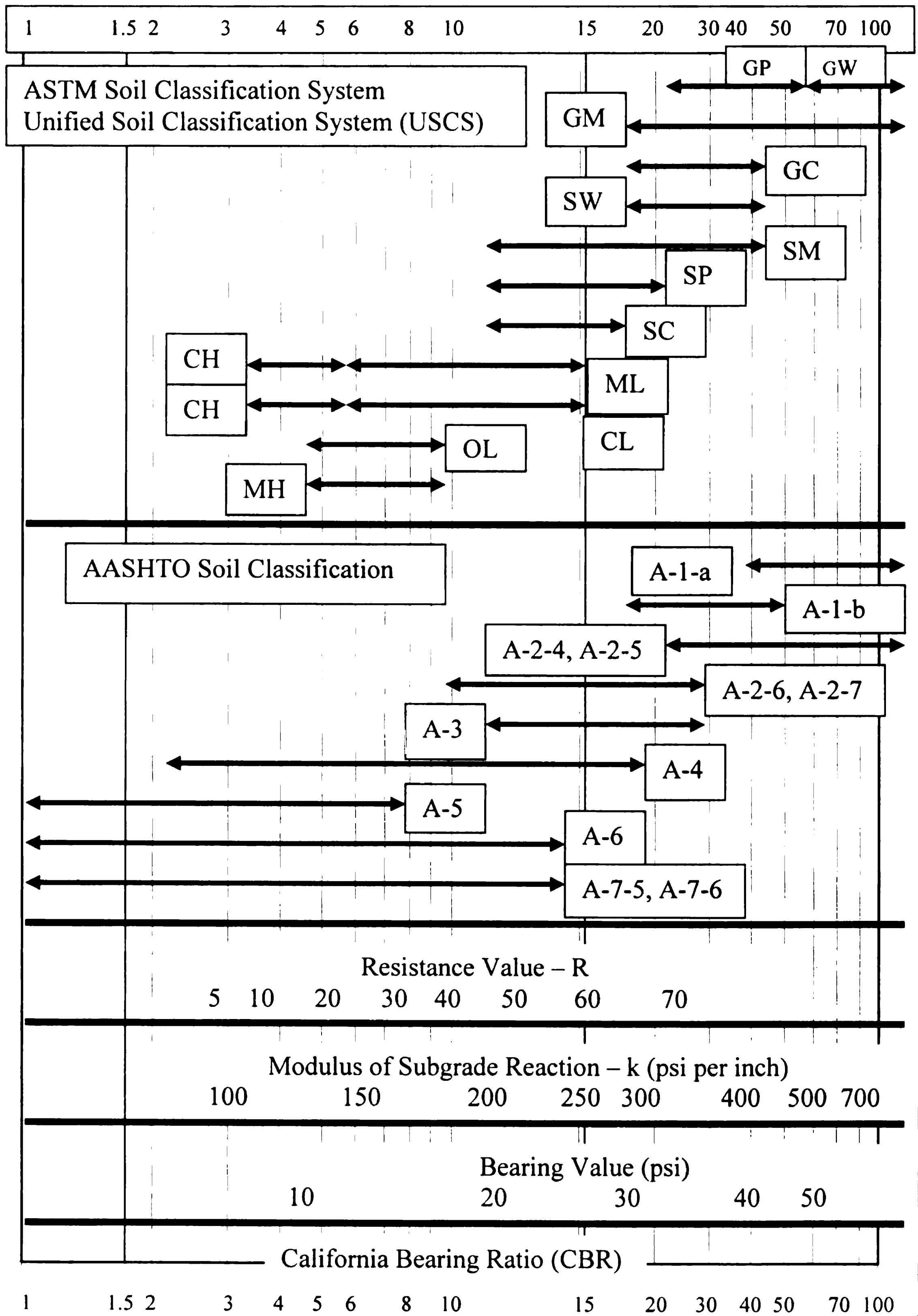


Figure 4.18 Soil classification related to strength parameters (NHI 1998)

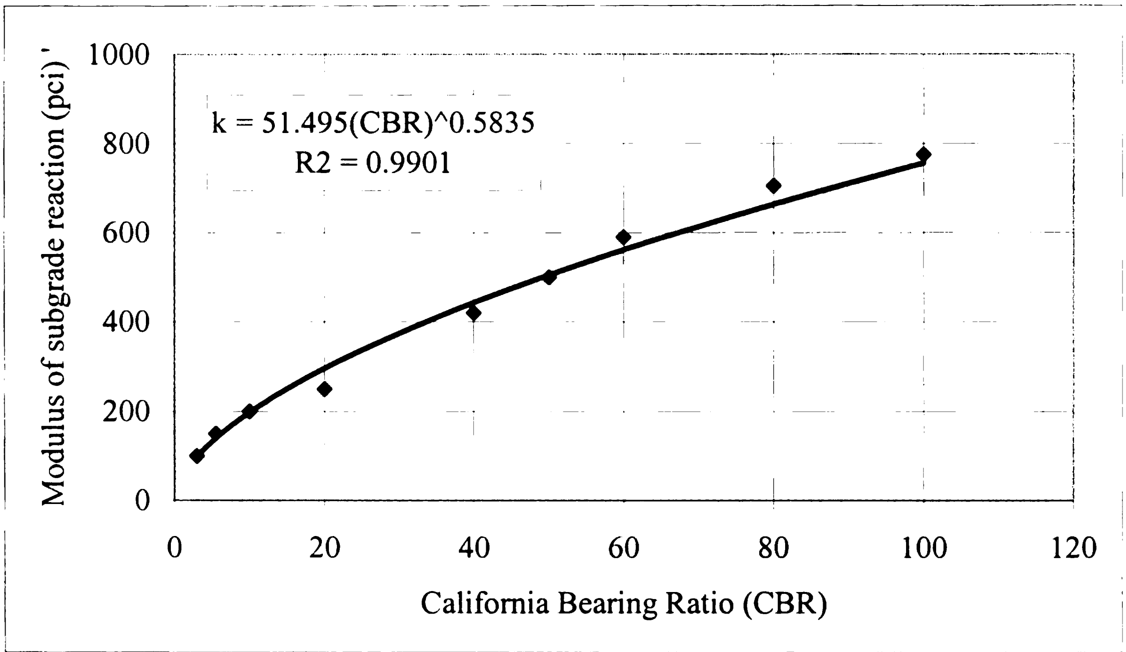


Figure 4.19 Modulus of subgrade reaction versus California Bearing Ratio (after NHI 1998)

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

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

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.

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

4.2.2.1 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.

Results - Only the backcalculated results of roadbed soil MR were further analyzed. The average, maximum, minimum, and standard deviation of MR of backcalculated roadbed soil supporting rigid pavements are listed in Table 4.4, and the detailed results are listed in Table B.2. The full set of NDT data and backcalculated results of rigid pavements is available on the accompanying compact disc. It should be noted that no ML soil supporting rigid pavements was FWD tested.

Table 4.4 Backcalculated roadbed soil MR supporting rigid pavement

Roadbed type USCS	MR results (psi)			
	Average	Maximum	Minimum	Std. dev.
SM	26,637	55,200	14,292	8,033
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	20,435	47,655	3,875	6,647
SC	23,034	35,830	11,662	4,147
CL	24,964	37,358	16,431	4,399
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. The resilient modulus of roadbed soils is more or less constant regardless if the soils are supporting flexible or rigid pavements. The MR of roadbed soils is dependent only on the soil type, water content, dry density, particle gradation, Atterberg limits, and stress states. Roadbed soil response to load is dependent on the stress level applied to the roadbed soil and the thickness, not the 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 4.5. 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 average ratio of backcalculated roadbed soil MR supporting flexible pavements to rigid pavements was 1.02. The distribution of this ratio by soil type can be seen in Figure 4.20. The frequency of the backcalculated MR values of roadbed soils supporting both flexible and rigid pavements are shown in Figure 4.21.

As indicated by Figure 4.20, for all soil types except the SP1 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

Table 4.5 Backcalculated roadbed soil MR 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	0.86
	Rigid	218	26,637	55,200	14,292	8,033	
	Combined	304	25,602	55,200	14,292	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	34	18,989	31,218	7,088	6,541	0.93
	Rigid	1,838	20,435	47,655	3,875	6,647	
	Combined	1,872	20,409	47,655	3,875	6,645	
SC	Flexible	393	24,704	67,793	11,728	6,695	1.07
	Rigid	884	23,034	35,830	11,662	4,147	
	Combined	1,277	23,548	67,793	11,662	4,931	
CL	Flexible	18	20,100	28,849	11,996	4,326	0.81
	Rigid	79	24,964	37,358	16,431	4,399	
	Combined	97	24,062	37,358	11,996	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							1.02

state where the sand deposit varies from more than 500 feet in the Cadillac area to about 200 feet in the Grand Rapid area. On the other hand, the SP1 roadbed soils under the rigid pavement along I-75 is located 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 AREA method algorithm doesn't account for shallow stiff layer or bedrock.

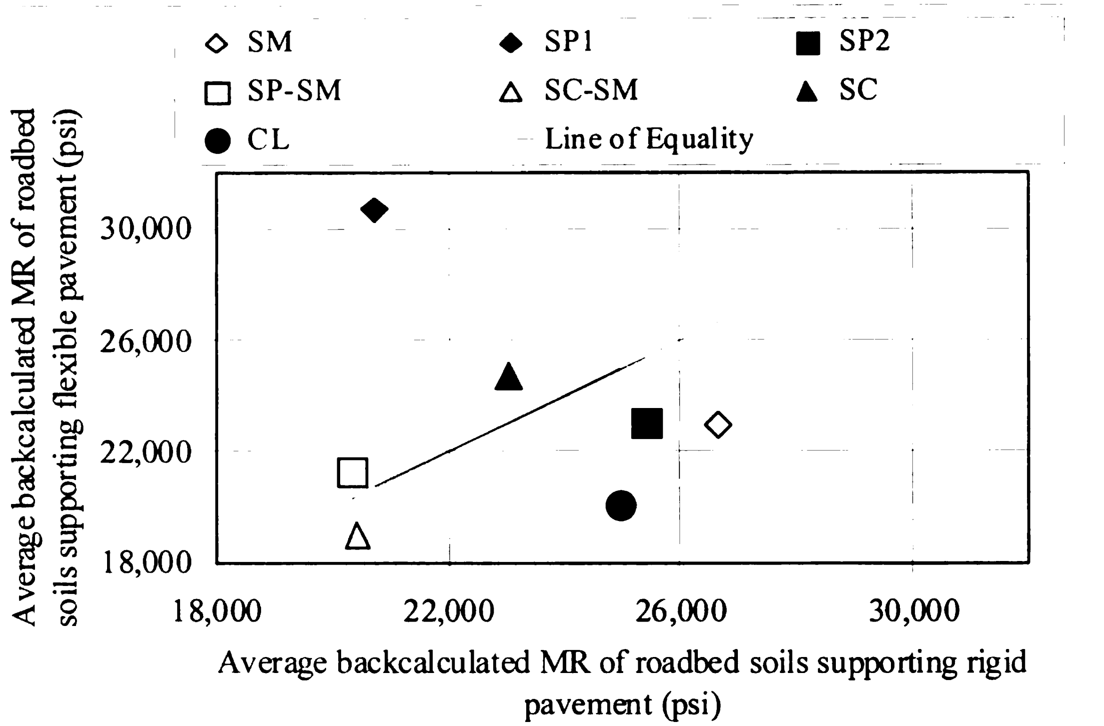


Figure 4.20 Flexible vs. rigid backcalculated roadbed soil MR

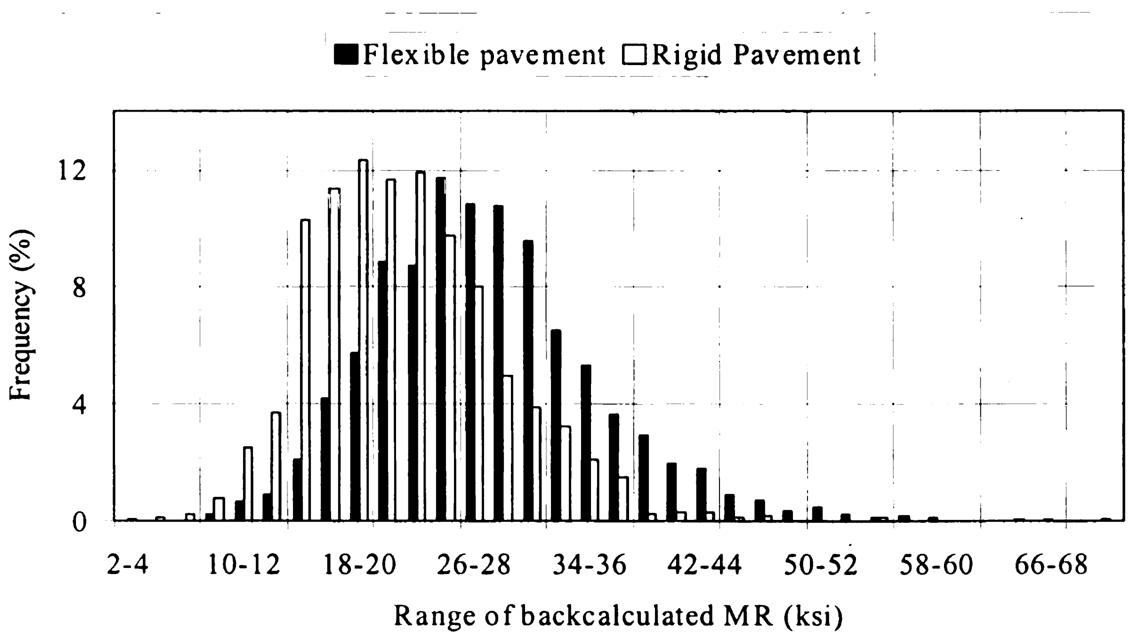


Figure 4.21 Frequency of backcalculated MR of roadbed soils under flexible and rigid pavements

The 1993 AASHTO pavement design guide suggests modifying k values when a stiff layer is present within ten feet from the pavement surface. Figure 4.22 depicts the modified k value due to three stiff layer depths versus the k value 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.

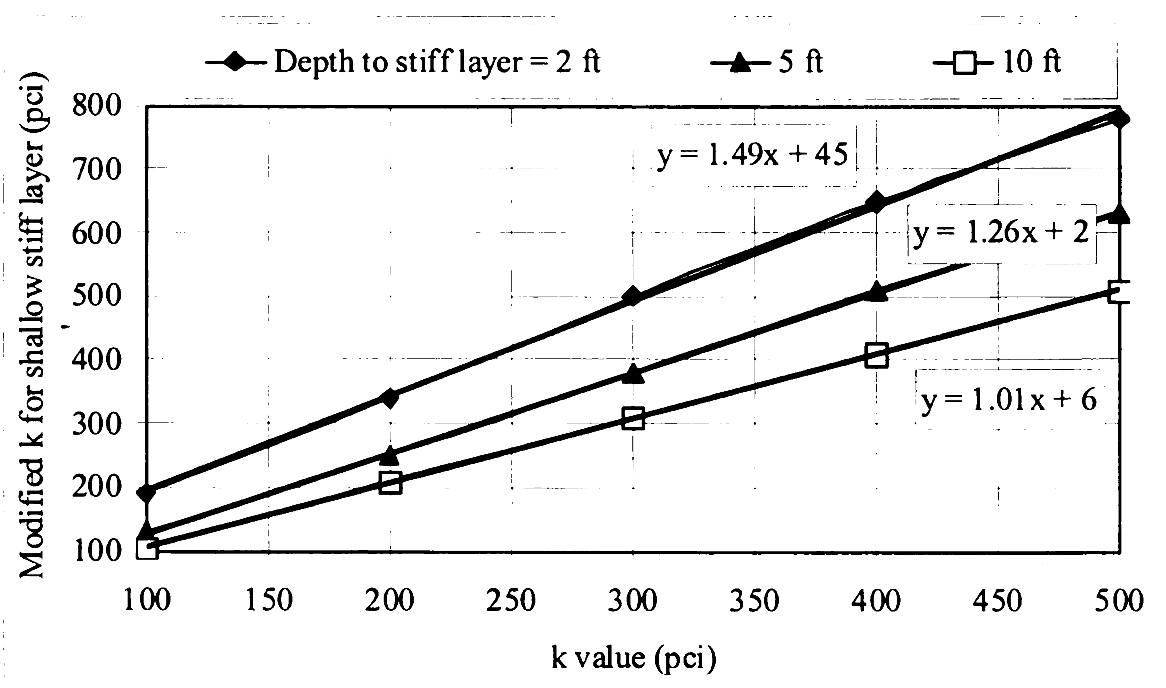


Figure 4.22 Stiff layer effects on backcalculated k

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 4.21 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 supporting flexible pavements to rigid pavements of 1.01.

Ranges - The maximum, minimum and the average backcalculated MR values roadbed soils supporting flexible and rigid pavements are shown in Figures 4.23 and 4.24. It can be seen that the ranges of the backcalculated MR of soils supporting flexible pavements are, for most soil types, less than those of the same soils supporting rigid pavements. This is mainly due to the dates (month and year) when the FWD tests were conducted. For most rigid pavements, the FWD tests were conducted over several year period and from early summer to late fall. Whereas, most of the FWD tests on flexible pavement were conducted during the same year and within few months.

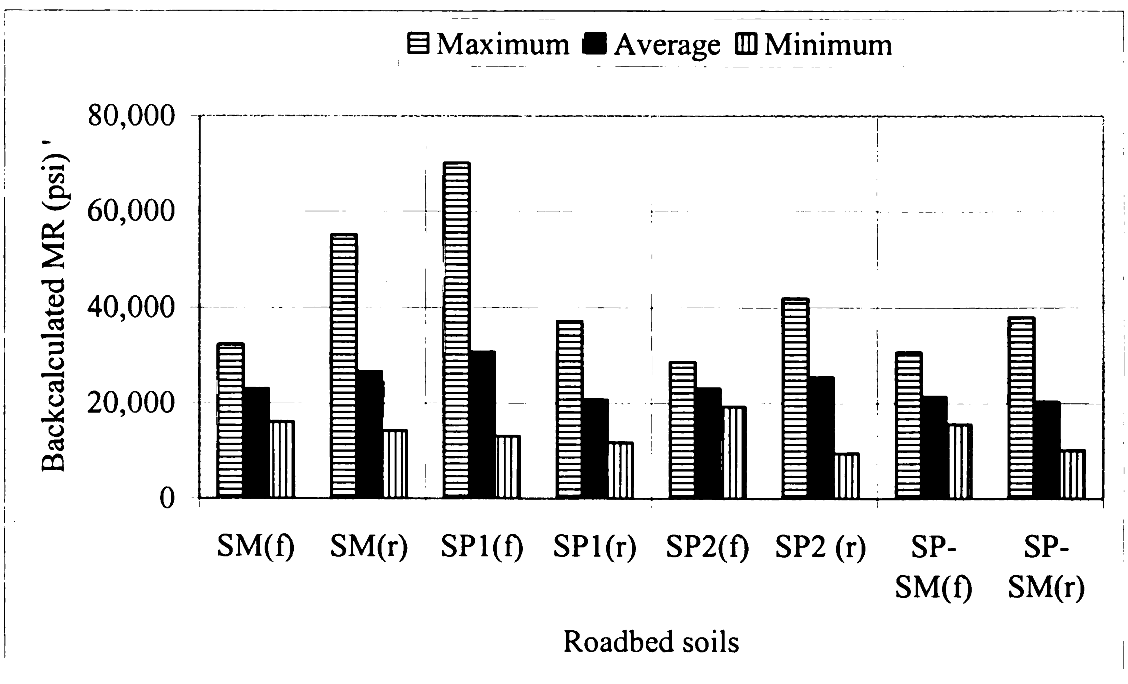


Figure 4.23 Range of backcalculated MR for SM, SP1, and SP2 soils

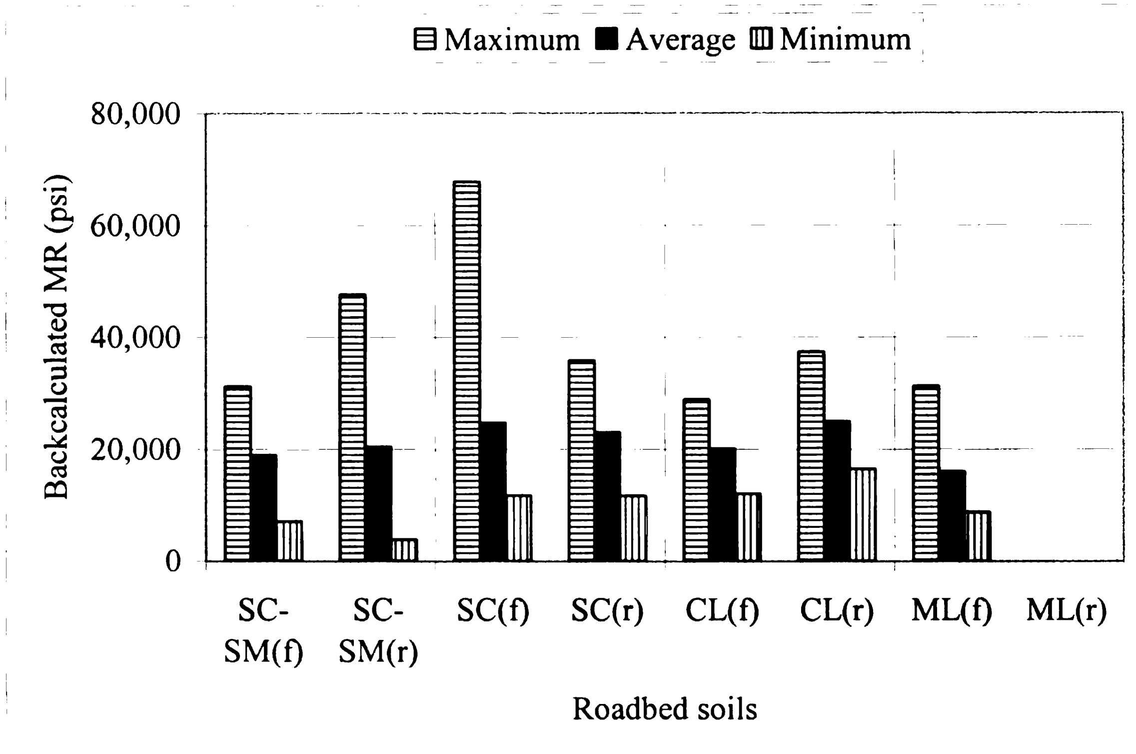


Figure 4.24 Range of backcalculated MR (SP-SM, SC-SM, SC, and CL soil)

The significance of the above scenario is that, for most rigid pavements, the range in the roadbed soil moisture contents is likely higher than that for flexible pavements. The larger variation in water content resulted in a larger variation in the backcalculated MR values. Further, for pavements supported by SP1 soils, the FWD tests conducted on flexible pavement sections were conducted over more environmental seasons and years than those on rigid pavements. Therefore, the range of the backcalculated MR values for the flexible pavement sections is larger than that for the rigid sections. Finally, it should be noted that no FWD tests were conducted on rigid pavements supported by ML soil.

4.4 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, all laboratory tests were conducted under a stress state that is compatible to that experienced by the soils in the field during the FWD tests. These conditions are discussed later in this section.

For each soil classification, Table 4.6 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 4.25.

Table 4.6 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	1,872	20,409	0.88
SM	A-2-4 A-4	17	17,028	304	25,602	1.50
SC	A-2-6 A-6 A-7-6	16	18,756	1,277	23,548	1.26
CL	A-4 A-6 A-7-6	9	37,225	97	24,062	0.65
ML	A-4	4	24,578	23	15,976	0.65
Average						1.03

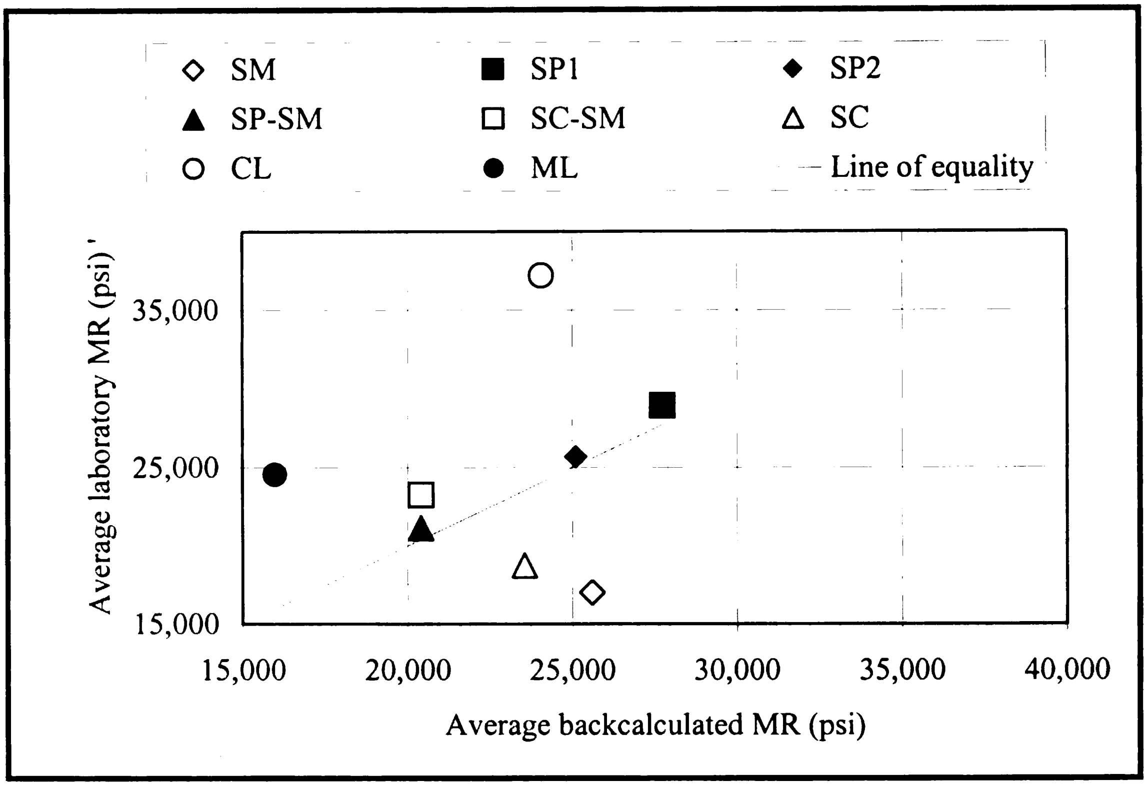


Figure 4.25 Laboratory determined and backcalculated roadbed soil MR

The data in Table 4.6 and Figure 4.25 indicate that the ratio of the two averages of the MR values for the SP1, SP2, SP-SM, and SC-SM are close to one. Whereas the ratios for the other four soil types (SM, SC, CL, and ML) vary from 1.5 to 0.65. These values were expected because:

- For the SM and SC 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 99 percent (which simulate the water contents throughout one year period). 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 4.6 and Figure 4.25.

- 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 should be expected to be high. Since, the FWD tests were conducted in the summer and fall (the water content of the roadbed soil is near the optimum) the backcalculated MR value is relatively low. Hence, the average MR value obtained from the laboratory tests is higher than the average backcalculated value.

The two reasons are related to the effects of moisture contents 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 using four different moisture contents. The test results are plotted in Figure 4.26. 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 more or less 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

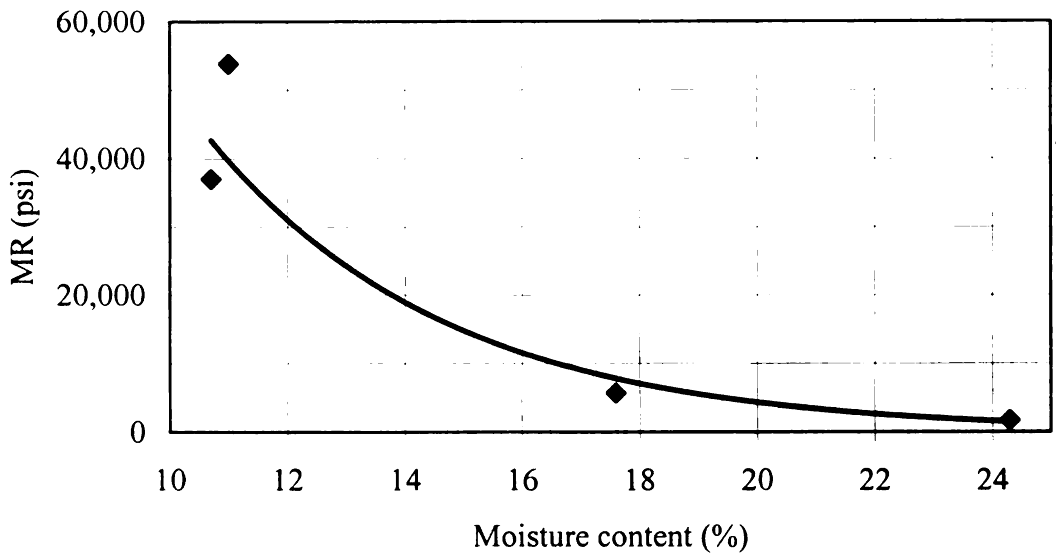


Figure 4.26 Moisture content affect on MR of ML soils

conditions used in this study. 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. 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. This ratio was obtained by conducting analyses of the stresses and strains delivered to the roadbed soil of a 25-inch thick pavement section due to 9000 pound wheel load (half the standard single axle load of 18000 pounds). The MICHPAVE finite element computer program, which is based on layered elastic theory, was used in the analyses. Results of the MICHPAVE computer program indicate that the roadbed soil is subjected to 8 psi vertical stress and to about 7.5 psi lateral stress. 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. 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. 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 4.27. 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 9000 pound load traveling over the pavement section in question. Higher stress ratios should be used when testing the base and subbase materials.

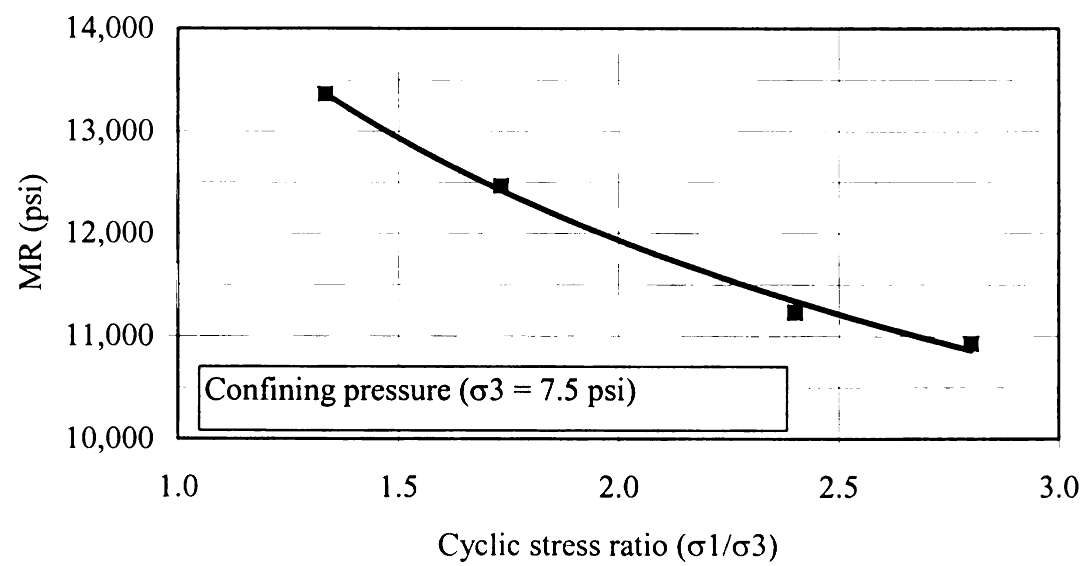


Figure 4.27 Laboratory obtained resilient modulus versus the cyclic stress level

Ranges – For each soil type, the ranges of the backcalculated and the laboratory determined MR values are shown in Figures 4.28 and 4.29. The backcalculated ranges of MR represent the variability in the soil moisture contents from early summer to late fall over several years. The ranges in the laboratory determined MR values, on the other hand, reflect variability in the water content of the soils and the compacted density. As it was expected, for fine soils (CL, ML, and SC), the effect of the water contents of the laboratory compacted test samples is higher than the variability in the density of the soils. For granular samples (e.g., SP1, SP2 and so forth), the effect of the density is higher than that of the water content.

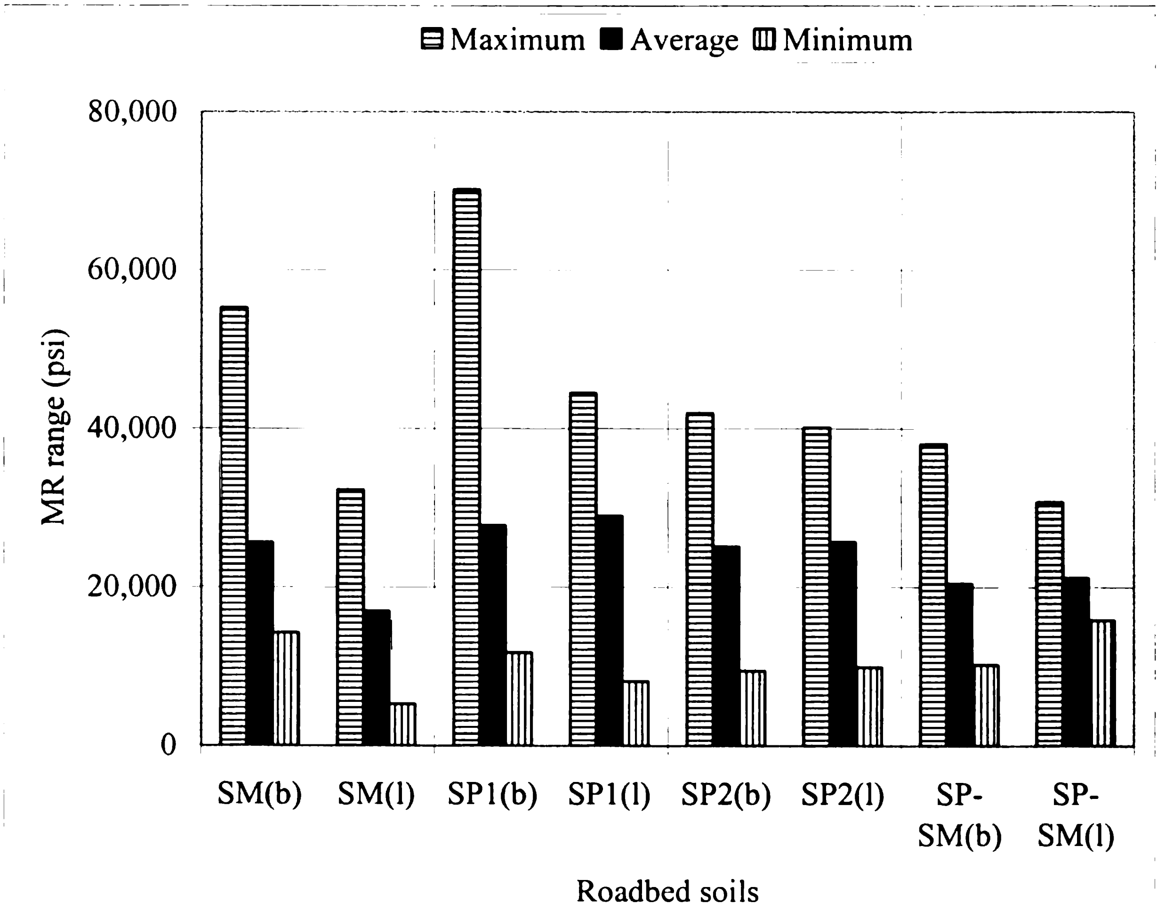


Figure 4.28 Range of MR values (SM, SP1, SP2, and SP-SM soil)

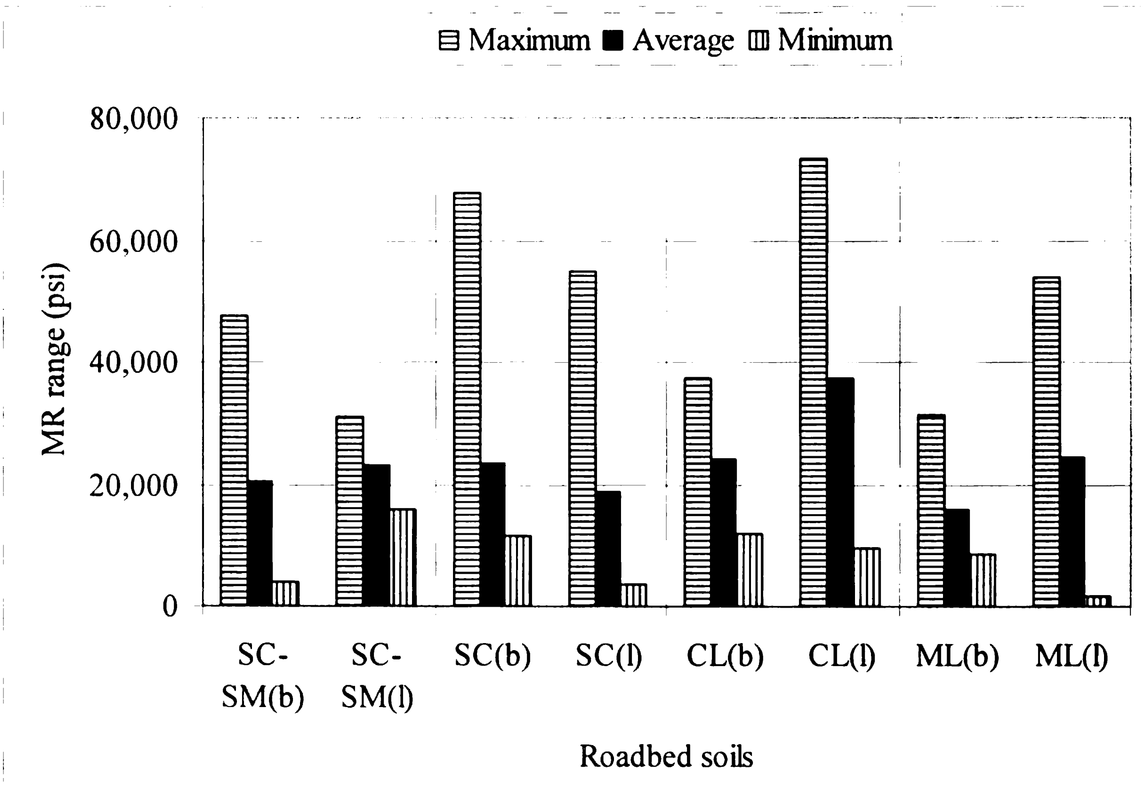


Figure 4.29 Range of MR values (SC-SM, SC, CL, and ML soil)

4.5 Damage

A brief summary of seasonal effects on roadbed soil is presented in the next two subsections. The detailed analyses can be found in (Sessions 2008).

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-inch and the average annual snowfall is about 56-inch. 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.

One of the objectives of this study was to investigate the affects of seasonal variations on roadbed soil MR. In order to study the affects; FWD tests were to be conducted once in the summer/fall season and once during the spring season. The factor between backcalculated roadbed soil MR during the summer/fall and spring seasons would be the seasonal damage factor. However, due to MDOT budget and equipment restraints only two sets of FWD tests were conducted during both seasons. Figure 4.30 indicates that the data represents partial spring conditions with only 40% and 15% reductions in MR respectively. The closed symbols represent the summer/fall tests and the open symbols represent the spring like conditions, while the arrows indicate the reduction in roadbed soil MR. No reasonable conclusions can be drawn based on the limited data.

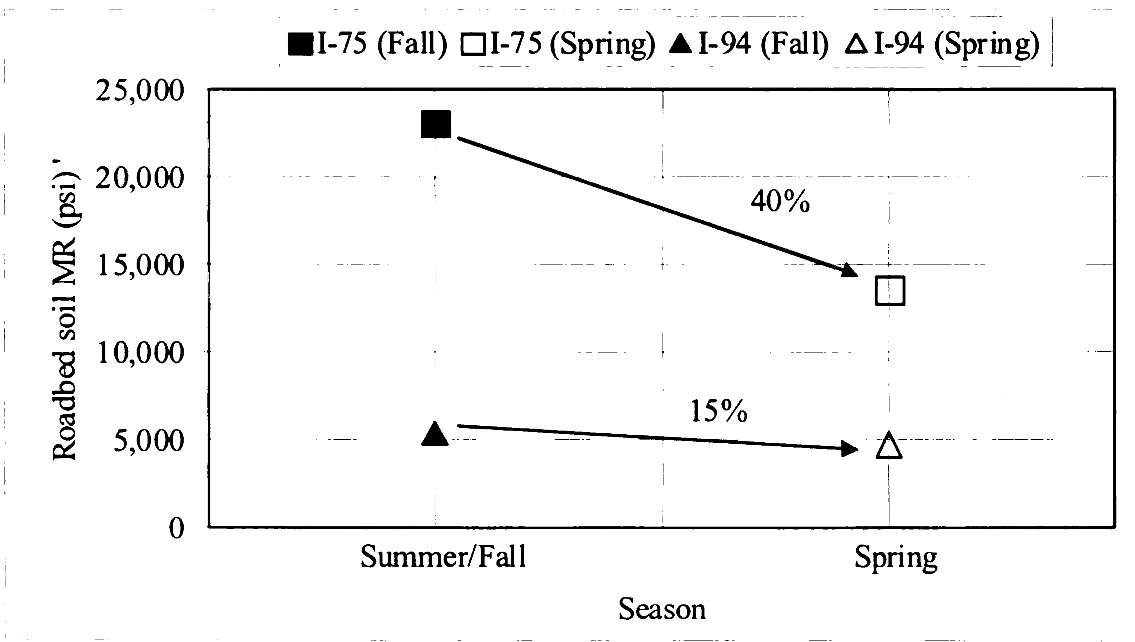


Figure 4.30 Partial spring condition FWD testing

CHAPTER 5

SUMMARY, CONCLUSIONS, & RECOMMENDATIONS

5.1 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. Most of these procedures are applicable to M-E PDG level 3 designs. 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.

To do this in this study, 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 twelve 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 contents, grain size distributions, Atterberg limits (when applicable), and resilient modulus using cyclic load triaxial tests. Correlation equations (see Table 5.1) 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 fifty six tests conducted as part of this

Table 5.1 Summary of predictive equations for each soil type

USCS	Number of		Predictive equation	Variable equation
	Clusters	Areas		
SP1	6	8	$MR = 89.825(SVSP1)^{2.9437}$	$SVSP1 = \dfrac{\gamma_d^{1.15}}{\left(P_4^{1.5} - P_{40}^{0.25}\right)^{0.5}}$
SP2	6	12	$MR = 0.8295(SVSP2)^{3.6006}$	$SVSP2 = \dfrac{\gamma_d^{1.35} * P_{200}^{-0.1}}{\left(P_4^{1.5} - P_{40}^{0.25}\right)^{0.5}}$
SM	11	16	$MR = 0.0303(SVSM)^{4.1325}$	$SVSM = \dfrac{\gamma_d^{0.8}}{S^{0.15}}$
			$MR = 45722\exp[(-0.0258)(MI)]$	$MI = LL^{1.1} + MC^{1.25}$
SC,CL, ML	10	28	$MR = 650486\exp - 0.0501(S)$	$S = \left[\dfrac{G_s * (MC/100) * \gamma_d}{G_s * \gamma_w - \gamma_d}\right] * 100$
SP-SM	7	8	$MR = 1749.6\exp 0.0054(SVSP\text{-}SM)$	$SVSP\text{-}SM = \dfrac{\gamma_d^{1.75}}{MC^{0.5} + LL^{0.6} + (P_{40} - P_{200})^{0.01}}$
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 study. FWD data files with sufficient accompanying data were analyzed to backcalculate the roadbed soil MR.

5.2 Conclusions

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

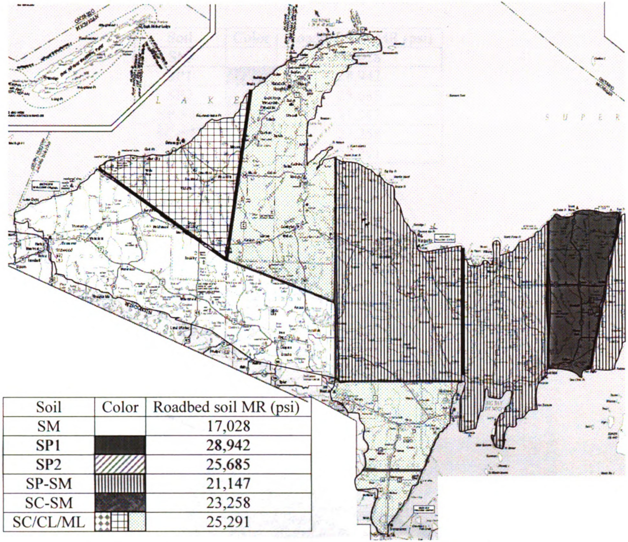
1. Most of the roadbed soils in the State of Michigan can be divided into the following eight soil types:
   - Gravelly sand (SG)
   - Poorly graded sand (SP), which can be divided into two groups SP1 and SP2 based on the percent fine contents.
   - Silty sand (SM)
   - Poorly graded sand – silty sand (SP-SM)
   - Clayey sand – silty sand (SC-SM)
   - Clayey sand (SC)
   - Low plasticity clay (CL)
   - Low plasticity silt (ML)
2. 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.
3. 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.
4. The backcalculated MR values, in this thesis, satisfy the M-E PDG requirements for level 1, 2, and 3 design.

5. Relatively accurate correlation equations between the laboratory obtained resilient modulus values and some of the soil parameters were developed and are summarized in Table 5.1.
6. MR values obtained from the correlation equations listed in Table 5.1 satisfy the M-E PDG requirements for level 2 and 3 design.
7. An average resilient modulus value for each soil type, except the SG, and for the two SP groups were developed and are listed in Table 5.2 and presented in Figure 5.1.
8. The MR values in Figure 5.1 satisfy the M-E PDG requirements for level 3 design.
9. The AREA method does not account for the effects of shallow stiff layers.
10. Equation 5.1 should be used when converting k, backcalculated from the AREA method, to MR of roadbed soils.

$$MR = (4)(19.4)k \qquad \text{Equation 5.1}$$

Table 5.2 Average roadbed soil MR values

Roadbed type		Average MR (psi)	
USCS	AASHTO	Laboratory Determined	Backcalculated
SM	A-2-4, A-4	17,028	25,602
SP1	A-1-a, A-3	28,942	27,739
SP2	A-1-b, A-3	25,685	25,113
SP-SM	A-1-b, A-2-4, A-3	21,147	20,400
SC-SM	A-2-4, A-4	23,258	20,409
SC	A-2-6, A-6, A-7-6	18,756	23,548
CL	A-4, A-6, A-7-6	37,225	24,062
ML	A-4	24,578	15,976
SC/CL/ML	A-2-6, A-4, A-6, A-7-6	25,291	23,459



Soil	Color	Roadbed soil MR (psi)
SM		17,028
SP1		28,942
SP2		25,685
SP-SM		21,147
SC-SM		23,258
SC/CL/ML		25,291

Figure 5.1 State of Michigan backcalculated MR distribution

Soil	Color	Roadbed soil MR (psi)
SM		17,028
SP1		28,942
SP2		25,685
SP-SM		21,147
SC-SM		23,258
SC/CL/ML		25,291

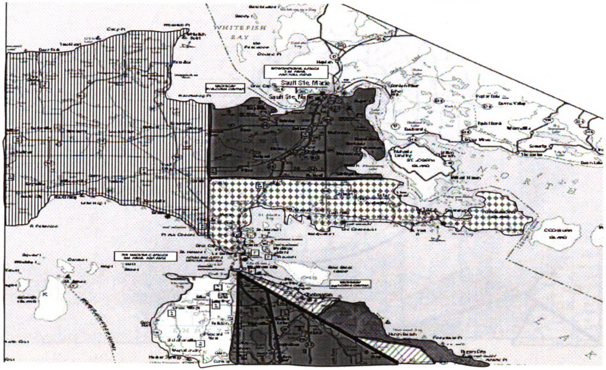


Figure 5.1 (cont'd)

Soil	Color	Roadbed soil MR (psi)
SM		17,028
SP1		28,942
SP2		25,685
SP-SM		21,147
SC-SM		23,258
SC/CL/ML		25,291

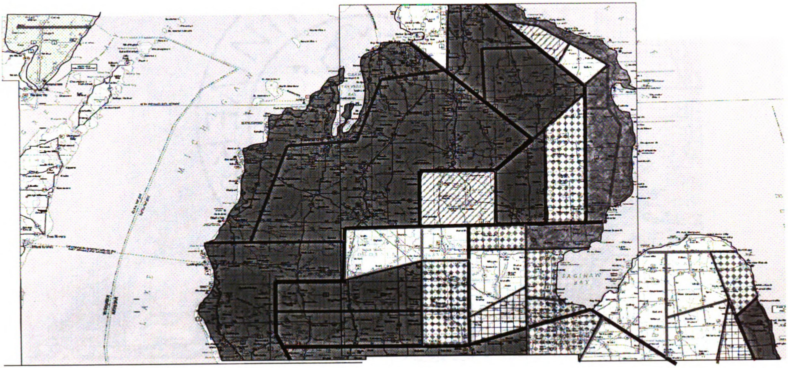


Figure 5.1 (cont'd)

Soil	Color	Roadbed soil MR (psi)
SM		17,028
SP1		**28,942**
SP2		**25,685**
SP-SM		21,147
SC-SM		23,258
SC/CL/ML		25,291
SG		-

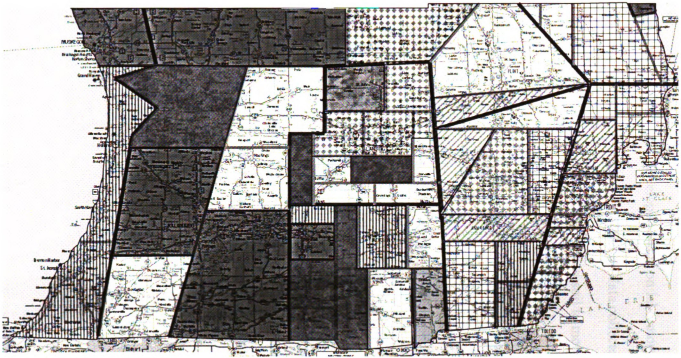


Figure 5.1 (cont'd)

5.3 Recommendations

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

- Additional deflection data should be collected during spring conditions and used to calibrate the seasonal damage factors that were developed based on laboratory data.
- 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 to meet the requirements of M-E PDG design levels 1, 2, and 3.
- MDOT implements the findings of this study by adopting the correlation models presented in Table 5.2 for M-E PDG design levels 2 and 3.
- MDOT implements the findings of this study by adopting the data presented in Figure 5.1 for M-E PDG design level 3.
- For rigid pavements, MDOT uses Equation 5.1 to convert backcalculated k of roadbed soil to MR and vice versa.
- Backcalculated MR needs not be converted to laboratory MR values, the two are the similar if the laboratory test boundary conditions are similar to those under FWD in the field.

APPENDICES

APPENDIX A

Laboratory and field test results

This appendix houses the laboratory and field test results arranged in table format as follows:

- For each of the 15 clusters, Table A.1 provides a list of the various percentile of soil types found in each area within the clusters.
- .Table A.2 provides a lists of the results of pocket penetrometer and vane shear tests for each of the 99 areas within the 15 clusters.
- Table A.3 provides a list of the moisture content, sieve analyses, and Atterberg limit test results for each soil type within the 99 areas.
- Table A.4 lists the results of the triaxial cyclic load tests.
- Table A.5 lists the MR results for each triaxial cyclic load test.

Table A.1 Soil percentages for each area within the 15 clusters (Sessions 2008)

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

Note: empty cells indicate 0 percent of that soil type.

* Loam contains sand, silt and clay. The breakdown of the loam is unknown at this point.

Table A.1 (cont'd)

Cluster	Area	Muck (%)	Sand (%)	Loamy Sand (%)	Silty Loam (%)	Sandy Loam (%)	Clayey Loam (%)	Loam* (%)	Mucky Sand (%)	Clay (%)	Silty Clay (%)	Proposed Sampling
05	01	22	5	72.4								X
	02	13.1	41.5	39.3								X
	03	14.3	74.7	7.8		2						
	04	26.3	51.4	17.4								
	05		97.9	1.7								X
	06	4.4	14.4	25.2		13		39.2				X
06	01	3.3	53.5	30.4		9.3						X
	02	8.1	71.8	7.5		8.2						X
	03	8.1	75.6	5.9		4.7						X
	04		25.7	39.5		8		26				X
	05		23.6	41.6		12.2		17.1				
07	04		14.9	6.8		11.4		65.3				X
	02		63.2	7					25.1			X
	05	2	18.4	18.6		7.9		48				X
	03	6.2	53.9	26.3		12.7						X
	01	15.1	32.1	28.6		10		10.1				X
	06	13	34.3	36.6		7.5		7.1				

Note: empty cells indicate 0 percent of that soil type.

* Loam contains sand, silt and clay. The breakdown of the loam is unknown at this point.

Table A.1 (cont'd)

Cluster	Area	Muck (%)	Sand (%)	Loamy Sand (%)	Silty Loam (%)	Sandy Loam (%)	Clayey Loam (%)	Loam* (%)	Mucky Sand (%)	Clay (%)	Silty Clay (%)	Proposed Sampling
08	03		2	24.9				72.1				X
	04		2	8.6		2.6		86.8				X
	01		12.4	15.6		11.3		58.8				X
	05		15	29.2		6.1		49.7				
	02		29.8	44.9				21.1				X
	06	19.2	39.4	22.2		11		8				X
09	10			8.2	43.3	3.2	9.6	33.4				X
	01	16	6.7	64.4		3.3		7.3				X
	08		12.1	19.1		3.4		62.8				X
	05	8.9				6.4		83				X
	03	33.4		19.4		18.1		21.4				X
	02		22.8	40.5		10.1	6.3	12.4				X
	09	4.3	14.8	43.3		22.4		9.6				
	07	4.8	3	11.2		23.2		52				X
	04	12.2		22.5		28.3		32.9				
	06	2.2		8.6		44.5		41.9				

Note: empty cells indicate 0 percent of that soil type.

* Loam contains sand, silt and clay. The breakdown of the loam is unknown at this point.

Table A.1 (cont'd)

Cluster	Area	Muck (%)	Sand (%)	Loamy Sand (%)	Silty Loam (%)	Sandy Loam (%)	Clayey Loam (%)	Loam (%)	Mucky Sand (%)	Clay (%)	Silty Clay (%)	Proposed Sampling
10	08					4.1		94.8				X
	10			8.5		12.7		75.8				
	11					13.2	11.8	73.8				X
	06	10.8		11.5		15.3		60.9				X
	05	3.7		11.4		29.3		55				
	09	30.1		27.6		29.8		11.1				X
	04	16.7				31.3		48.5				X
	03	10.6		11.1		34.4		38.1				X
	01			6.2		34.4		57.5				X
	07	3.2		8		39.9		46.1				
	02	7.1				40.6		48.4				
11	01	17		34.5		27.6		15.6				X
	02	6.9		5.8		54.8		25.3				X
	05	11.4		11.6		65.4		10.3				X
	03					67.2		30.5				X
	04	6				72.6		18.5				

Note: **empty cells indicate 0 percent of that soil type.**

* Loam contains sand, silt and clay. The breakdown of the loam is unknown at this point.

Table A.1 (cont'd)

Cluster	Area	Muck (%)	Sand (%)	Loamy Sand (%)	Silty Loam (%)	Sandy Loam (%)	Clayey Loam (%)	Loam (%)	Mucky Sand (%)	Clay (%)	Silty Clay (%)	Proposed Sampling
12	01	9.2	3.9	18.2		36.4		24.7				X
	02	7.9	9.2	5.6	11.4	53.2		9.4				
	06	22.7	17.4			34.3	17.9					X
	04	5.4		12		16.5	26.5	37.6				X
	07	6.2				7		84				X
	03	18.5				56.5		17.7				X
	05	9.2		5.8		62.5		19				
13	08		4.4	5.7		28.1	39.8	14.8				X
	07		5.1	28.5		30.5	28.8	5.8				X
	06		7.2	36.2		34.3	17.9	4.4				
	05					6.6	37.8	55.5				X
	04	6.2		4	13.6		14.2	60				X
	02	12		11.6		37.1		34.3				X
	03	13.1		4.3	28.6	20	24.9					X
	01	12.4		31.4		44		4.8				X

Note: **empty cells indicate 0 percent of that soil type.**

* Loam contains sand, silt and clay. The breakdown of the loam is unknown at this point.

Table A.1 (cont'd)

Cluster	Area	Muck (%)	Sand (%)	Loamy Sand (%)	Silty Loam (%)	Sandy Loam (%)	Clayey Loam (%)	Loam (%)	Mucky Sand (%)	Clay (%)	Silty Clay (%)	Proposed Sampling
14	01	8.2		29		20.3	18.7	14.8				X
	02	9.5	3	28.4		21.4	35.5					X
	03	5.2		11	25.5		53.2	3.5				X
	04				28.4		13.2	50				X
	07		21.4	7.2	12.3	28.9	13.5	12.9				X
	08			9		36.7		50.9				X
	09		11.2	3.5		16.2		64.3				
	10			5.8	4.6	9	57			7	12.5	X
	05	NO DATA										X
	06	NO DATA										X
15	04		7.4	6.1	22.5	4.9	33.7	17.7				X
	06		5.1	13.8	3.8	7		62.8				X
	02	20.9	6.4	5.5	14.9	15.3		33.1				X
	07			36		16.2		39.1				X
	01		15	28		23.5		31				
	03			4.8	60.3	23.9		8.5				X
	08			7.8	10.4	37.5		37				X
	05			4.4		38.6		56				

Note: empty cells indicate 0 percent of that soil type.

* Loam contains sand, silt and clay. The breakdown of the loam is unknown at this point.

Table A.2 Pocket penetrometer and vane shear test results

Sample number	Location	Vane shear test (Kg/cm^2)	Pocket penetrometer (Kg/cm^2)
M-045-S (01-01)	405 feet South of Ontonagon River	didn't fail	didn't 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	didn't fail	didn't 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	didn't fail	didn't 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	didn't fail	didn't 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	didn't fail	didn't 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

Table A.2 (cont'd)

Sample number	**Location**	**Vane shear test (Kg/cm^2)**	**Pocket penetrometer (Kg/cm^2)**
I-075-N (06-02)	160 feet North of Co Rd 662	0.25	1.8
U-031-N (06-03)	307 feet North of M-46	-	-
I-196-N (06-05)	110 feet North of Schmuhl Rd	1	3.3
M-020-W (07-02)	~.5 mile East of 13 Mile Rd	-	-
M-020-E (07-03)	~500 feet East of Cottonwood Ave	-	-
U-127-N (07-04)	100 feet North of Jefferson Rd	didn't fail	didn't penetrate
U-127-N (07-05)	65 feet North of Vernon Rd	5	3.6
M-061-E (07-06)	420 feet East of left hand turn on M-61 (off US-127)	-	-
M-061-E (08-02)	165 feet West of Hockaday	0.5	0.6
U-010-W (08-03)	65 feet West of bridge before Stark Rd	didn't fail	2.3
U-010-W (08-04)	145 feet West of Mackinaw Rd	didn't fail	didn't penetrate
I-075-S (08-05)	115 feet South of Prevo Rd	didn't fail	didn't penetrate
I-075-N (08-06)	80 feet North of bridge after exit 195	1.7	2.1
U-131-S (09-01)	160 feet South of Lake Montcalm Rd	-	-
I-096-W (09-02)	141 feet West of Morse Lake Ave	-	-
U-131-S (09-03)	**105 feet South of 110th Ave**	0.5	1.1
U-131-S (09-05)	**60 feet South of 'Reduce Speed 55 MPH' sign** right where it turns from interstate to freeway	0.5	1.1
M-044-E (09-07)	Station 137+10	-	-
I-075-S (09-08)	650 feet South of Wadsworth Rd	-	-
M-024-S (09-09)	20 feet North of Burley Rd	1.25	1.9

Table A.2 (cont'd)

Sample number	Location	Vane shear test (Kg/cm^2)	Pocket penetrometer (Kg/cm^2)
I-069-E (09-10)	172 feet East of Grand River Rd	0.5	1
I-069-N (10-01)	75 feet North of Base Line Hwy	3	4
I-096-W (10-03)	210 feet West of bridge before exit 97	1	1.3
I-069-N (10-04)	150 feet North of Island Hwy	1.75	4
I-069-N (10-05)	100 feet North of Five Points Hwy	2.5	3.2
I-096-W (10-09)	140 feet West of Dietz Rd	2.7	3.1
I-069-E (10-10)	120 feet East of Britton Rd	didn't fail	didn't penetrate
M-021-E (10-11)	800 feet East of Shepards Rd	3	2.6
I-069-N (11-01)	160 feet North of mile marker 42	-	-
I-094-W (11-02)	132 feet West of exit 110 on ramp	1.5	2.5
M-060-W (11-03)	135 feet West of Southbound I-69 overpass	3	5.5
I-069-S (11-05)	95 feet South of Bridge after exit 10	-	-
I-094-W (12-01)	95 feet West of 29 Mile Rd	1	2.6
I-094-W (12-03)	36 feet West of bridge after exit 135	1	1.5
U-012-E (12-04)	100 feet East of Emarld Rd	didn't fail	6
I-094-W (12-06)	53 feet West of Mt Hope Rd	3	3.5
U-012-E (12-07)	120 feet West of Person Hwy	0.5	0.9
M-024-S (13-01)	250 feet North of Best Rd	3.5	5
M-059-W (13-02)	Station 131+29	-	-
M-014-W (13-03)	255 feet West of Napler Rd	didn't fail	4.5

Table A.2 (cont'd)

Sample number	Location	Vane shear test (Kg/cm^2)	Pocket penetrometer (Kg/cm^2)
I-094-W (13-04)	Station 75+02	-	-
U-012-E (13-05)	Between Maple Rd and Industrial Ave	didn't fail	didn't penetrate
U-023-N (13-07)	60 feet North of Sherman	0.75	1.6
M-010-E (13-08)	Station 38+00	didn't fail	3.5
I-075-S (14-01)	60 feet South of Gaynier Rd	3	3.5
I-075-S (14-02)	40 feet South of Nadeau Rd	4.5	3.5
U-024-S (14-03)	~1000 feet South of Ready Rd	4.5	5
U-024-S (14-04)	150 feet North of Pardee	0.25	0.5
I-075-S (14-04)	Station 23+00	-	-
I-094-W (14-05)	300 feet West of Monroe Blvd	3.5	4.3
M-153-E (14-06)	~800 feet East of Greenfield Rd	didn't fail	4.5
M-053-S (14-07)	1500 feet South of Canal Rd	-	-
I-094-W (14-09)	350 feet West of Wadhams Rd	2.5	3
I-094-W (14-10)	227 feet West of Palms Rd	didn't fail	4
M-053-S (15-02)	300 feet South of M-46	2	2.3
M-090-E (15-03)	210 feet East of Murray Rd	0.5	0.7
M-090-E (15-04)	**200 feet East of Bobcock St 37 feet East of Village Limit sign**	**didn't fail**	**didn't penetrate**
M-025-S (15-05)	200 feet North of Day Rd	1.5	2.8
M-25-N (15-06)	170 feet North of North Huron Dr	1.3	2.2
M-019-S (15-07)	650 feet South of Thompson Rd 1 mile South of M-142	1.5	2.7

Table A.3 Moisture content, sieving, and Atterberg limit 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	4	10	20	40	100	200	LL	PL	PI						AASHTO	USCS
				9.500	4.750	2.000	0.850	0.425	0.150	0.075										
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

Table A.3 (cont'd)

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	4	10	20	40	100	200	LL	PL	PI						AASHTO	USCS
				9.500	4.750	2.000	0.850	0.425	0.150	0.075										
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
U-010-W (08-04)	X	15.0	205.1	98.0	98.9	96.5	95.8	80.3	42.5	33.3	27	13	14	0.0009	0.018	0.200	222.22	1.80	A-6	SC
I-075-S (08-05)		8.9	201.0	100.0	99.9	97.7	94.5	69.4	40.3	33.5	25	12	13	0.001	0.011	0.300	300.00	0.40	A-2-6	SC
I-075-N (08-06)		11.8	201.5	100.0	99.2	96.8	93.7	85.4	36.6	26.2	17	10	7	0.001	0.011	0.270	270.00	0.45	A-2-4	SC-SM
U-131-S (09-01)		4.6	1056.3	99.0	98.0	97.4	97.0	83.7	2.5	0.5	NA	NA	NP	0.180	0.220	0.300	1.67	0.90	A-3	SP
I-096-W (09-02)		9.9	206.2	100.0	99.0	97.3	93.8	82.7	40.9	30.5	17	13	4	0.001	0.075	0.240	240.00	23.44	A-2-4	SC-SM
U-131-S (09-03)		1.9	530.4	100.0	100.0	99.9	99.8	97.2	6.0	0.4	NA	NA	NP	0.180	0.200	0.290	1.61	0.77	A-3	SP
U-131-S (09-05)		3.6	1025.6	97.5	90.2	80.8	69.5	45.8	3.1	1.3	NA	NA	NP	0.185	0.295	0.605	3.27	0.78	A-3	SP
M-044-E (09-07)		8.7	206.5	100.0	99.5	97.7	94.1	85.5	37.7	26.7	14	-	NP	0.020	0.110	0.250	12.50	2.42	A-2-4	SM
I-075-S (09-08)		20.2	216.1	99.1	96.1	91.8	89.7	85.5	62.3	45.8	31	14	17	0.001	0.004	0.140	140.00	0.11	A-4	SC
M-024-S (09-09)		13.3	198.6	100.0	99.6	97.6	95.4	93.2	45.0	24.1	20	-	NP	0.012	0.090	0.200	16.67	3.38	A-2-4	SM
I-069-E (09-10)		7.1	527.8	98.3	93.4	83.0	66.3	36.8	5.2	3.1	NA	NA	NP	0.190	0.340	0.700	3.68	0.87	A-3	SP
I-069-N (10-01)		10.1	534.1	94.9	88.7	81.1	67.6	49.2	16.7	8.0	16	11	5	0.093	0.230	0.600	6.45	0.95	A-3	SP-SM
I-096-W (10-03)		14.7	199.7	100.0	98.4	93.9	90.1	82.0	29.5	17.5	29	14	15	0.0600	0.150	0.280	4.67	1.34	A-2-6	SC
I-069-N (10-04)		11.1	198.5	100.0	99.3	94.1	86.4	74.9	30.1	17.6	16	-	NP	0.010	0.150	0.200	20.00	11.25	A-2-4	SM
I-069-N (10-05)		24.0	204.0	100.0	100.0	97.8	87.6	54.9	43.2	37.3	19	-	NP	0.010	0.070	0.500	50.00	0.98	A-2-4	SM
I-096-W (10-09)		15.1	200.9	100.0	99.6	93.7	91.0	61.1	38.0	30.4	19	-	NP	0.006	0.075	0.410	68.33	2.29	A-2-4	SM
I-069-E (10-10)		12.8	204.9	98.0	96.1	92.4	90.5	84.7	57.2	37.7	26	15	11	0.001	0.009	0.170	170.00	0.48	A-6	SC
M-021-E (10-11)		15.0	230.2	99.4	92.1	85.9	79.5	72.2	46.3	33.8	23	14	9	0.001	0.030	0.270	270.00	3.33	A-2-4	SC
I-069-N (11-01)		9.1	1032.9	90.3	87.1	83.0	77.8	63.9	15.9	6.9	14	-	NP	0.120	0.210	0.390	3.25	0.94	A-3	SP-SM
I-094-W (11-02)		7.1	1022.7	95.0	91.7	87.1	77.5	51.2	6.2	2.7	NA	NA	NP	0.170	0.270	0.510	3.00	0.84	A-3	SP
M-060-W (11-03)		10.5	199.3	99.7	99.0	97.4	90.6	67.0	37.6	31.1	22	15	7	0.004	0.025	0.330	82.50	0.47	A-2-4	SC-SM
I-069-S (11-05)		6.6	201.1	100.0	99.1	93.9	86.9	77.3	49.3	38.6	15	11	4	0.002	0.034	0.210	105.00	2.75	A-4	SC-SM
I-094-W (12-01)		8.6	199.8	100.0	95.2	81.8	73.9	51.8	26.5	20.0	16	12	4	0.038	0.180	0.560	14.74	1.52	A-2-4	SC-SM
I-094-W (12-03)		13.2	527.4	97.4	95.4	91.6	83.0	68.3	18.7	7.4	16	-	NP	0.095	0.195	0.345	3.63	1.16	A-3	SP-SM
U-012-E (12-04)		4.9	200.4	99.9	98.9	94.2	89.4	73.7	36.6	23.0	16	-	NP	0.003	0.110	0.300	100.00	13.44	A-2-4	SM

Table A.3 (cont'd)

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
I-094-W (12-06)		12.1	213.7	100.0	99.8	92.2	90.5	86.0	35.2	23.8	15	-	NP	0.005	0.130	0.250	50.00	13.52	A-2-4	SM
U-012-E (12-07)		7.0	513.8	67.5	57.0	42.2	25.8	16.0	10.0	8.1	18	-	NP	0.160	1.000	6.000	37.50	1.04	A-1-a	SG
M-024-S (13-01)		10.6	196.0	100.2	98.4	93.4	90.2	85.2	59.4	45.1	18	15	3	0.001	0.013	0.150	150.00	1.13	A-4	SM
M-059-W (13-02)		11.6	1033.3	99.4	97.9	95.1	91.2	65.7	8.9	1.7	NA	NA	NP	0.160	0.220	0.380	2.38	0.80	A-3	SP
M-014-W (13-03)		9.3	198.1	100.0	99.1	94.0	90.0	85.7	62.7	49.2	22	13	9	0.001	0.006	0.130	130.00	0.28	A-4	SC
I-094-W (13-04)		8.0	1005.6	98.1	95.8	90.5	82.8	65.9	13.1	3.5	NA	NA	NP	0.140	0.210	0.390	2.79	0.81	A-3	SP
U-012-E (13-05)		14.9	205.0	100.0	99.9	99.0	97.8	95.5	65.6	56.7	33	17	16	0.001	0.002	0.100	111.11	0.04	A-6	CL
U-023-N (13-07)		9.8	529.5	94.1	83.4	66.2	53.5	43.3	12.0	5.7	13	-	NP	0.130	0.280	1.350	10.38	0.45	A-3	SP-SM
M-010-E (13-08)		14.0	201.0	100.0	99.7	98.1	95.0	90.8	74.3	59.9	24	14	10	0.0010	0.003	0.075	75.00	0.12	A-6	CL
M-010-E (13-08)	X	12.3	207.0	100.0	98.0	95.6	93.5	88.3	72.6	54.8	23	14	9	0.0009	0.015	0.090	100.00	2.78	A-6	CL
I-075-S (14-01)	X	18.4	204.5	100.0	99.9	89.4	87.9	67.6	54.2	48.2	42	21	21	0.0090	0.015	0.250	27.78	0.10	A-7-6	SC
I-075-S (14-01)		25.4	200.6	100.0	96.9	78.9	76.2	68.4	47.8	41.2	45	19	26	0.0007	0.003	0.270	385.71	0.05	A-7-6	SC
I-075-S (14-02)		18.7	201.0	100.0	98.3	97.6	92.6	85.5	64.1	46.1	41	19	22	0.001	0.003	0.190	211.11	0.06	A-7-6	SC
U-024-S (14-03)		19.2	202.3	100.0	99.4	98.8	91.8	79.7	55.3	41.4	40	13	27	0.001	0.003	0.190	271.43	0.07	A-6	SC
I-075-S (14-04)		15.8	200.8	100.0	99.9	99.8	99.7	96.4	59.4	46.9	34	17	17	0.001	0.003	0.260	288.89	0.04	A-6	SC
U-024-S (14-04)		22.2	543.7	100.0	100.0	99.8	99.6	96.3	23.3	2.5	NA	NA	NP	0.100	0.170	0.255	2.55	1.13	A-3	SP
I-094-W (14-05)		21.6	199.0	99.7	97.6	97.5	89.7	78.0	56.7	46.7	34	21	13	0.001	0.013	0.160	160.00	1.06	A-6	SC
M-153-E (14-06)	X	26.0	209.4	100.0	99.8	99.0	98.3	92.7	70.1	51.1	51	19	32	0.0090	0.018	0.100	11.11	0.36	A-7-6	SC
M-153-E (14-06)		21.6	202.9	100.0	100.0	98.4	98.1	94.1	64.4	49.9	52	20	32	0.0007	0.001	0.140	200.00	0.02	A-7-6	SC
M-053-S (14-07)		5.9	529.1	93.1	87.5	81.5	70.3	55.0	9.3	4.7	NA	NA	NP	0.170	0.240	0.500	2.94	0.68	A-3	SP
I-094-W (14-09)	X	26.3	205.1	100.0	100.0	98.5	97.9	85.2	59.8	55.8	42	23	19	0.0010	0.010	0.150	150.00	0.67	A-7-6	CL
I-094-W (14-09)		21.9	197.3	99.7	99.2	97.7	96.6	90.8	66.8	60.9	44	21	23	0.0010	0.002	0.075	75.00	0.05	A-7-6	CL
I-094-W (14-10)		21.5	198.9	100.0	99.5	93.3	91.6	80.3	65.2	56.3	42	19	23	0.001	0.002	0.100	166.67	0.07	A-7-6	CL
M-053-S (15-02)		17.2	200.4	100.0	99.5	96.8	94.4	87.5	42.8	26.2	14	-	NP	0.008	0.100	0.210	26.25	5.95	A-2-4	SM
M-090-E (15-03)		38.0	204.1	100.0	99.9	98.8	96.1	90.7	73.1	55.8	35	20	15	0.001	0.005	0.088	88.00	0.28	A-6	CL
M-090-E (15-04)		12.4	199.6	100.0	99.7	97.4	95.0	90.6	67.4	52.8	24	15	9	0.0010	0.006	0.100	100.00	0.36	A-4	CL
M-025-S (15-05)		4.4	532.8	99.3	98.7	98.2	97.3	84.4	1.9	1.1	NA	NA	NP	0.180	0.210	0.300	1.67	0.82	A-3	SP
M-25-N (15-06)		16.4	206.4	100.0	98.9	94.0	90.8	85.1	54.2	42.3	24	13	11	0.001	0.007	0.190	190.00	0.26	A-4	SC
M-019-S (15-07)		11.4	199.4	99.9	95.1	83.9	76.4	61.5	29.0	17.2	14	-	NP	0.065	0.160	0.400	6.15	0.98	A-2-4	SM

Table A.4 Triaxial cyclic load 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	
U-002-E (02-04)	A-4	ML	100	32.8	2.499	31,653	37,012	50.0	3.944	29,991	33,191
			200	32.8	2.471	33,225		49.8	3.855	30,881	
			500	33.7	2.322	36,319		50.0	3.724	31,614	
			800	33.1	2.219	36,874		50.1	3.560	33,569	
			1000	33.1	2.207	37,843		50.5	3.516	34,390	
U-002-E (03-01)	A-3	SP-SM	100	33.3	2.393	22,822	22,830	51.0	4.295	18,193	19,629
			200	33.9	2.412	23,466		50.2	4.135	18,644	
			500	33.9	2.441	23,426		51.6	4.005	19,685	
			800	34.1	2.522	22,465		52.0	4.114	19,323	
			1000	34.6	2.560	22,598		51.7	3.990	19,880	

Table A.4 (cont'd)

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-028-W (03-02)	A-3	SP	100	33.1	2.429	22,556	23,003	50.3	3.861	20,301	22,536
			200	33.3	2.428	22,706		50.9	3.783	21,157	
			500	34.1	2.460	23,286		51.3	3.731	21,613	
			800	33.9	2.374	23,167		51.3	3.644	22,811	
			1000	33.8	2.483	22,555		52.0	3.582	23,185	
M-028-W (03-03)	A-3	SP-SM	100	32.7	3.357	15,294	16,911	49.8	5.258	14,150	15,956
			200	33.4	3.283	16,085		50.3	5.113	14,855	
			500	33.0	3.059	16,876		50.5	4.774	15,866	
			800	33.8	3.094	16,885		50.9	4.876	15,840	
			1000	33.9	3.175	16,971		50.9	4.722	16,162	
U-002-E (03-03)	A-2-4	SM	100	31.7	3.230	15,364	15,984	50.3	5.163	14,590	15,833
			200	32.2	3.208	15,857		50.5	5.056	15,302	
			500	32.7	3.167	16,240		50.3	4.973	15,412	
			800	32.7	3.251	15,919		51.1	4.873	15,966	
			1000	32.6	3.286	15,793		51.0	4.793	16,120	
I-075-N (03-04)	A-3	SP	100	33.8	2.198	25,827	26,140	51.4	3.419	24,035	24,401
			200	33.8	2.255	25,821		51.9	3.423	24,209	
			500	34.0	2.203	25,887		51.7	3.456	24,040	
			800	34.3	2.156	26,592		52.3	3.402	24,474	
			1000	34.2	2.263	25,940		52.3	3.348	24,689	
U-023-S (04-01)	A-3	SP	100	33.6	2.390	23,852	23,060	51.5	3.757	21,584	21,735
			200	33.9	2.392	24,136		51.7	3.775	21,768	
			500	33.7	2.472	23,456		51.5	3.807	21,526	
			800	33.8	2.440	22,395		51.7	3.700	21,852	
			1000	33.7	2.508	23,330		51.7	3.814	21,828	
M-068-W (04-02)	A-2-4	SC-SM	100	34.1	2.034	29,159	30,958	51.3	3.595	22,151	24,764
			200	33.0	2.006	28,338		52.3	3.505	23,435	
			500	34.1	1.883	30,987		52.1	3.377	24,481	
			800	34.4	1.861	30,960		51.6	3.383	24,598	
			1000	34.6	1.980	30,927		51.9	3.299	25,212	

Table A.4 (cont'd)

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-068-W (04-03)	A-3	SP	100	29.4	5.139	8,572	9,979	46.6	8.078	8,440	10,013
			200	30.7	4.944	9,491		47.1	7.723	8,966	
			500	31.3	4.879	9,725		48.6	7.368	9,822	
			800	31.7	4.685	10,215		48.8	7.052	10,308	
			1000	31.6	4.806	9,996		48.5	7.166	9,910	
M-065-S (04-04)	A-2-4	SM	100	31.7	4.463	10,722	11,943	48.7	6.850	10,728	11,909
			200	31.5	4.383	10,903		49.7	6.524	11,210	
			500	32.1	4.101	11,945		50.0	6.427	11,637	
			800	32.3	4.157	11,833		50.3	6.268	11,995	
			1000	32.5	4.190	12,050		49.6	6.038	12,096	
M-032-W (04-05)	A-4	SC-SM	100	32.5	2.880	17,806	19,255	50.5	4.474	16,758	18,161
			200	32.3	2.805	17,979		50.7	4.478	17,269	
			500	32.9	2.739	18,915		50.6	4.283	18,002	
			800	33.2	2.725	19,303		50.9	4.267	18,269	
			1000	33.1	2.708	19,546		50.7	4.208	18,211	
U-131-N (05-01)	A-2-4	SM	100	32.1	2.263	23,769	24,651	51.4	3.311	24,414	25,604
			200	32.2	2.284	23,491		51.3	3.281	24,858	
			500	33.3	2.257	24,948		51.4	3.239	26,201	
			800	33.6	2.314	24,548		50.9	3.247	25,241	
			1000	33.8	2.266	24,456		51.9	3.298	25,370	
U-127-N (05-04)	A-3	SP	100	34.1	1.757	35,135	37,158	52.6	2.846	29,879	29,949
			200	34.3	1.708	36,388		51.9	2.861	29,921	
			500	34.5	1.687	37,843		52.9	2.866	30,325	
			800	35.3	1.727	36,438		51.9	2.901	29,588	
			1000	35.2	1.766	37,194		52.6	2.850	29,935	
M-072-W (05-06)	A-6	SC	100	30.6	3.211	22,916	26,492	48.9	4.652	24,395	27,193
			200	31.3	3.095	24,442		49.0	4.593	25,076	
			500	31.7	2.941	26,104		49.8	4.403	26,622	
			800	31.9	2.888	27,204		49.8	4.321	27,183	
			1000	31.2	2.958	26,168		50.2	4.252	27,774	

Table A.4 (cont'd)

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-132-N (06-01)	A-3	SP	100	33.8	2.004	27,798	31,741	52.1	3.079	26,746	28,997
			200	33.9	2.055	29,008		52.7	3.027	27,574	
			500	34.1	1.984	30,874		52.6	2.977	28,591	
			800	34.4	1.960	31,899		53.2	2.989	29,341	
			1000	34.0	1.857	32,449		52.0	2.917	29,059	
I-075-N (06-02)	A-3	SP	100	33.4	1.616	26,603	32,450	51.6	2.882	26,762	31,187
			200	33.3	1.535	28,072		51.9	2.855	28,300	
			500	33.8	1.461	32,068		52.3	2.552	31,485	
			800	34.5	1.499	32,023		52.1	2.508	31,026	
			1000	34.4	1.417	33,260		52.7	2.450	31,049	
U-031-N (06-03)	A-3	SP	100	34.1	1.933	30,572	31,867	52.5	2.814	29,633	29,636
			200	33.8	1.884	31,084		52.1	2.832	29,692	
			500	34.6	1.808	32,659		52.6	2.897	29,123	
			800	35.0	1.908	31,347		52.9	2.861	30,084	
			1000	35.4	1.937	31,594		52.6	2.842	29,701	
I-196-N (06-05)	A-2-4	SP-SM	100	33.5	2.456	22,276	23,030	51.9	3.650	21,630	21,985
			200	33.7	2.428	23,097		51.7	3.641	21,694	
			500	34.2	2.386	23,190		51.5	3.618	22,017	
			800	33.6	2.395	22,525		51.7	3.675	21,801	
			1000	33.9	2.371	23,375		52.1	3.637	22,136	
M-020-W (07-02)	A-3	SP	100	33.6	2.112	26,636	31,489	51.2	3.135	25,897	31,766
			200	33.8	2.001	29,046		51.6	3.012	27,403	
			500	34.3	1.969	30,442		51.8	2.891	28,918	
			800	34.0	1.902	31,795		52.5	2.649	33,296	
			1000	34.1	1.893	32,230		52.7	2.550	33,084	
M-020-W (07-02)	A-3	SP	100	32.8	2.029	27,192	30,272	50.6	3.326	23,445	24,896
			200	33.6	2.032	28,722		50.5	3.436	23,201	
			500	33.8	1.969	30,147		51.7	3.361	25,035	
			800	34.0	1.971	30,271		51.7	3.315	24,950	
			1000	33.9	1.965	30,399		51.6	3.408	24,702	

Table A.4 (cont'd)

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-020-W (07-02)	A-3	SP	100	33.7	2.016	28,992	29,446	51.2	3.080	25,785	28,593
			200	33.9	2.064	28,380		51.3	3.000	27,443	
			500	33.9	1.961	29,237		51.4	2.948	28,001	
			800	34.0	2.042	29,357		51.6	2.893	28,741	
			1000	34.3	1.984	29,743		51.7	2.938	29,037	
M-020-W (07-02)	A-3	SP	100	33.2	1.883	19,618	19,693	51.0	3.025	16,754	20,257
			200	33.1	1.884	19,462		51.0	2.949	18,301	
			500	33.6	1.901	20,010		52.1	2.779	19,917	
			800	33.8	1.922	19,736		52.0	2.722	20,529	
			1000	33.6	1.876	19,334		51.7	2.723	20,325	
M-020-W (07-02)	A-3	SP	100	31.7	2.516	20,521	24,320	49.5	4.178	17,679	24,552
			200	32.8	2.321	22,926		50.3	3.959	19,876	
			500	33.1	2.329	24,360		51.4	3.364	23,953	
			800	33.3	2.344	24,091		51.3	3.310	24,799	
			1000	33.8	2.284	24,508		51.4	3.307	24,904	
M-020-E (07-03)	A-3	SP	100	33.9	1.963	29,497	32,696	52.6	3.106	27,496	28,182
			200	34.0	1.935	30,737		52.2	2.999	27,797	
			500	34.4	1.881	32,158		51.9	2.992	28,321	
			800	34.3	1.941	31,958		52.3	3.019	27,995	
			1000	34.2	1.810	33,972		51.9	3.081	28,230	
U-127-N (07-05)	A-6	SC	100	42.5	11.329	3,466	3,984	72.2	15.047	4,432	5,481
			200	43.7	10.944	3,698		73.5	7.386	4,716	
			500	44.3	10.593	3,897		75.1	6.798	5,246	
			800	44.4	10.260	4,015		75.4	6.602	5,455	
			1000	44.2	10.170	4,041		75.4	12.596	5,742	
U-127-N (07-05)	A-6	SC	100	52.0	2.068	47,427	54,737	82.9	3.013	49,924	53,030
			200	51.7	1.926	50,103		82.3	2.944	50,430	
			500	52.8	1.878	53,735		81.6	2.806	51,951	
			800	51.5	1.860	54,842		82.5	2.821	53,516	
			1000	52.3	1.860	55,634		82.0	2.761	53,623	

Table A.4 (cont'd)

Sample number	Soil Type AASHTO	Soil Type USCS	Cycle number	10 psi: Average cyclic load (lbs)	10 psi: Average deformation (mils)	10 psi: Average resilient modulus (psi)	10 psi: Average MR (psi) at load cycles 500, 800 and 1000	15 psi: Average cyclic load (lbs)	15 psi: Average deformation (mils)	15 psi: Average resilient modulus (psi)	15 psi: Average MR (psi) at load cycles 500, 800 and 1000
U-127-N (07-05)	A-6	SC	100	46.3	6.202	7,133	7,323	75.8	10.783	6,642	6,925
			200	46.7	6.185	7,244		76.7	10.605	6,765	
			500	46.7	6.248	7,223		77.2	10.557	6,875	
			800	47.2	6.196	7,353		77.5	10.534	6,936	
			1000	47.9	6.195	7,395		77.9	10.543	6,965	
U-127-N (07-05)	A-6	SC	100	44.7	9.544	4,319	4,713	74.4	14.108	4,852	5,358
			200	45.8	9.458	4,487		75.7	13.880	5,081	
			500	46.2	9.290	4,651		75.8	13.294	5,339	
			800	46.5	9.278	4,718		75.7	13.131	5,338	
			1000	46.4	9.113	4,770		76.4	13.119	5,398	
U-127-N (07-05)	A-6	SC	100	30.4	2.658	31,474	36,054	47.9	4.349	28,993	27,729
			200	30.9	2.535	33,999		48.7	4.299	30,151	
			500	31.5	2.333	36,628		49.1	4.290	27,523	
			800	31.6	2.285	36,290		49.0	4.221	27,851	
			1000	32.5	2.231	35,243		50.2	4.333	27,814	
M-061-E (07-06)	A-2-4	SM	100	28.0	7.229	10,631	11,483	41.8	12.878	11,660	12,907
			200	28.6	7.191	10,855		42.3	12.581	11,834	
			500	29.3	6.855	11,362		43.2	12.189	13,155	
			800	29.6	6.630	11,709		44.2	11.636	12,831	
			1000	28.8	6.826	11,377		44.2	11.476	12,736	
M-061-E (08-02)	A-2-4	SM	100	33.1	1.937	29,999	32,231	51.2	2.807	30,104	31,763
			200	32.9	1.864	30,417		51.7	2.743	30,517	
			500	33.8	1.897	32,344		52.1	2.713	31,551	
			800	33.8	1.930	32,106		51.9	2.701	31,719	
			1000	34.0	1.875	32,242		52.3	2.728	32,020	
U-010-W (08-04)	A-6	SC	100	41.4	10.662	3,592	4,134	70.9	14.499	4,516	5,268
			200	42.3	10.310	3,829		71.7	13.972	4,771	
			500	42.9	9.824	4,076		72.8	13.214	5,099	
			800	43.2	9.691	4,164		73.7	12.850	5,323	
			1000	42.9	9.608	4,163		73.8	12.675	5,382	

Table A.4 (cont'd)

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
U-010-W (08-04)	A-6	SC	100	43.2	7.759	5,212	5,873	71.7	13.897	4,831	5,106
			200	44.4	7.585	5,439		72.2	13.671	4,937	
			500	45.6	7.430	5,791		73.0	13.508	5,049	
			800	45.9	7.266	5,946		73.6	13.342	5,130	
			1000	46.1	7.252	5,884		73.4	13.283	5,138	
I-075-N (08-06)	A-2-4	SC-SM	100	32.4	3.493	14,265	15,798	51.1	4.992	15,346	16,577
			200	32.9	3.499	14,932		50.9	4.807	15,758	
			500	33.3	3.406	15,448		51.4	4.785	16,290	
			800	33.3	3.297	15,986		51.2	4.676	16,606	
			1000	33.1	3.263	15,960		51.3	4.594	16,836	
U-131-S (09-01)	A-3	SP	100	33.8	2.123	26,881	28,793	52.3	3.220	26,018	27,732
			200	33.9	2.126	27,797		52.8	3.183	26,982	
			500	34.5	2.127	29,155		52.8	3.200	27,204	
			800	34.3	2.052	29,674		52.6	3.078	27,868	
			1000	34.6	2.152	27,550		53.2	3.081	28,124	
I-096-W (09-02)	A-2-4	SC-SM	100	33.5	2.694	20,127	22,163	52.0	4.254	18,607	19,597
			200	33.8	2.624	21,049		51.6	4.205	18,942	
			500	33.7	2.566	21,588		51.5	4.164	19,295	
			800	34.0	2.530	22,509		52.0	4.085	19,756	
			1000	33.8	2.473	22,392		51.2	3.999	19,740	
U-131-S (09-03)	A-3	SP	100	33.9	1.997	28,736	30,368	51.1	3.381	23,393	28,022
			200	34.3	2.019	29,283		51.9	3.341	24,843	
			500	34.9	1.990	30,848		52.1	3.006	27,525	
			800	34.0	1.948	30,648		52.4	3.010	27,615	
			1000	34.4	1.983	29,608		52.6	2.953	28,925	
U-131-S (09-05)	A-1-b	SP	100	34.0	2.036	36,835	38,498	52.5	2.817	33,838	35,390
			200	33.9	2.070	37,497		52.0	2.833	34,449	
			500	34.5	1.978	38,818		52.5	2.776	35,389	
			800	34.9	1.943	38,902		52.5	2.796	35,440	
			1000	34.8	2.007	37,773		52.5	2.856	35,340	

Table A.4 (cont'd)

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-044-E (09-07)	A-2-4	SM	100	33.4	2.943	17,969	18,434	51.4	4.275	18,433	19,654
			200	33.5	2.928	18,261		52.0	4.175	19,068	
			500	33.9	2.948	18,534		51.6	4.086	19,511	
			800	33.7	2.841	18,744		51.6	4.043	19,663	
			1000	34.0	3.046	18,023		51.9	4.041	19,788	
M-024-S (09-09)	A-2-4	SM	100	33.1	3.415	15,148	15,156	50.4	5.279	14,639	15,854
			200	33.1	3.421	15,097		50.5	5.143	15,004	
			500	33.6	3.458	15,204		50.6	4.889	15,786	
			800	32.9	3.427	14,945		50.8	4.853	15,880	
			1000	33.3	3.378	15,318		51.0	4.891	15,897	
I-069-E (09-10)	A-3	SP	100	34.0	1.985	29,263	28,663	52.2	3.321	25,428	26,095
			200	34.1	2.074	29,172		52.2	3.248	25,709	
			500	34.4	2.071	28,746		52.3	3.163	26,255	
			800	34.3	2.079	29,232		51.9	3.231	25,802	
			1000	34.8	2.160	28,012		52.4	3.192	26,227	
I-069-N (10-01)	A-3	SP-SM	100	33.5	3.483	14,917	15,873	51.2	4.732	16,473	17,394
			200	33.5	3.542	14,864		51.1	4.694	16,512	
			500	33.1	3.344	15,551		51.3	4.485	17,528	
			800	33.8	3.457	15,312		50.8	4.533	17,144	
			1000	34.0	3.162	16,756		51.7	4.526	17,509	
I-096-W (10-03)	A-2-6	SC	100	32.9	2.180	41,549	43,824	50.1	3.266	40,469	37,712
			200	33.4	2.210	42,092		50.5	3.275	41,776	
			500	33.8	2.175	43,219		51.5	3.167	42,767	
			800	34.1	2.196	44,499		49.2	3.779	34,806	
			1000	34.1	2.203	43,754		48.7	2.686	35,563	
I-069-N (10-04)	A-2-4	SM	100	33.4	2.846	18,890	19,190	51.4	4.400	18,049	18,963
			200	33.5	2.771	19,221		51.8	4.330	18,293	
			500	33.5	2.839	19,530		51.0	4.232	18,653	
			800	33.4	2.862	19,049		51.5	4.107	18,952	
			1000	33.9	2.802	18,990		51.6	4.076	19,284	

Table A.4 (cont'd)

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
I-069-N (10-05)	A-2-4	SM	100	25.4	8.607	4,273	5,295	37.6	15.895	3,469	5,646
			200	25.9	8.326	4,542		41.3	12.494	4,766	
			500	27.0	7.715	5,123		43.1	11.611	5,377	
			800	27.2	7.630	5,241		43.7	11.104	5,712	
			1000	27.6	7.381	5,521		43.8	11.027	5,850	
I-096-W (10-09)	A-2-4	SM	100	30.1	5.580	8,027	9,518	47.9	7.602	9,168	11,394
			200	30.9	5.176	8,832		48.1	7.430	9,504	
			500	31.2	5.002	9,361		49.5	6.721	10,908	
			800	31.0	4.917	9,419		49.5	6.363	11,495	
			1000	31.6	4.875	9,775		49.8	6.307	11,778	
I-069-N (11-01)	A-3	SP-SM	100	32.9	1.119	30,534	30,733	51.7	1.595	29,788	28,147
			200	33.8	1.119	32,960		52.0	1.567	30,484	
			500	34.0	1.063	30,406		52.0	2.994	28,203	
			800	34.0	1.119	30,967		52.2	2.974	28,154	
			1000	34.8	1.119	30,827		51.9	2.995	28,083	
I-094-W (11-02)	A-3	SP	100	33.6	1.694	36,073	44,521	51.2	3.205	25,752	27,372
			200	33.3	1.627	37,965		52.2	3.144	26,314	
			500	34.1	1.487	45,141		52.8	3.102	27,442	
			800	32.6	1.432	42,908		51.9	3.136	26,857	
			1000	34.1	1.453	45,513		52.6	3.020	27,817	
M-060-W (11-03)	A-2-4	SC-SM	100	31.9	2.614	19,615	19,812	50.2	4.354	17,216	16,639
			200	31.3	2.561	19,255		50.7	4.426	17,262	
			500	32.0	2.553	19,808		50.8	4.481	16,817	
			800	32.4	2.561	19,861		50.7	4.560	16,601	
			1000	32.2	2.563	19,768		50.9	4.669	16,498	
I-069-S (11-05)	A-4	SC-SM	100	33.7	2.252	23,451	27,303	52.3	3.358	24,923	25,645
			200	33.8	2.220	24,393		52.6	3.317	25,291	
			500	34.0	2.095	25,903		52.5	3.274	25,489	
			800	34.2	2.014	26,908		51.6	3.267	25,632	
			1000	34.3	1.931	29,098		52.2	3.245	25,814	

Table A.4 (cont'd)

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
I-094-W (12-01)	A-2-4	SC-SM	100	33.3	1.963	28,024	27,636	51.3	3.519	22,990	23,872
			200	32.5	2.028	27,129		52.0	3.594	23,024	
			500	34.1	2.041	28,985		51.8	3.438	23,554	
			800	33.6	2.129	26,615		52.0	3.414	24,001	
			1000	33.9	2.113	27,308		52.4	3.534	24,060	
I-094-W (12-03)	A-3	SP-SM	100	33.6	2.783	19,527	18,139	50.8	4.851	15,566	15,977
			200	33.6	2.766	19,827		50.9	4.881	15,796	
			500	33.4	2.814	18,886		50.4	4.789	16,090	
			800	33.9	3.021	17,820		50.8	4.831	15,893	
			1000	34.1	3.066	17,711		50.8	4.798	15,947	
U-012-E (12-04)	A-2-4	SP-SM	100	33.3	2.862	18,848	19,234	50.8	4.340	18,416	18,343
			200	33.3	2.930	19,047		51.3	4.323	18,683	
			500	33.6	2.781	19,237		51.2	4.264	18,191	
			800	34.1	2.881	19,210		51.4	4.312	18,324	
			1000	34.1	2.766	19,255		51.3	4.266	18,515	
I-094-W (12-06)	A-2-4	SM	100	33.9	2.675	19,996	19,425	51.4	4.042	19,797	21,382
			200	33.8	2.698	20,013		51.4	3.956	20,110	
			500	33.7	2.821	19,357		52.6	3.873	21,249	
			800	33.8	2.796	19,802		51.7	3.733	21,552	
			1000	34.0	2.792	19,115		51.5	3.774	21,346	
M-024-S (13-01)	A-4	SM	100	34.4	3.172	17,093	17,950	51.9	5.000	15,746	16,175
			200	34.0	3.101	17,359		50.0	4.846	15,814	
			500	34.8	3.149	17,853		51.5	4.878	16,213	
			800	34.6	3.049	17,891		51.6	4.844	16,042	
			1000	35.1	3.052	18,106		51.8	4.844	16,271	
M-059-W (13-02)	A-3	SP	100	33.6	2.042	28,216	24,863	51.7	3.362	23,959	23,810
			200	33.5	2.112	27,648		51.9	3.351	24,234	
			500	33.9	2.186	26,464		52.1	3.478	23,699	
			800	33.9	2.368	24,623		51.7	3.453	23,882	
			1000	33.8	2.439	23,502		51.9	3.436	23,849	

Table A.4 (cont'd)

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
I-094-W (13-04)	A-3	SP	100	32.8	2.693	20,399	21,470	51.2	4.441	17,533	18,859
			200	32.5	2.615	20,565		50.7	4.353	17,856	
			500	33.2	2.592	21,384		50.8	4.245	18,355	
			800	33.5	2.589	21,598		50.8	4.133	18,836	
			1000	33.4	2.648	21,427		51.2	4.040	19,387	
U-023-N (13-07)	A-3	SP-SM	100	34.0	2.515	22,197	22,629	51.6	3.907	20,649	20,593
			200	33.8	2.443	23,009		51.4	3.846	20,641	
			500	34.0	2.573	22,214		51.2	3.961	20,201	
			800	33.9	2.428	22,768		52.4	3.910	20,900	
			1000	34.7	2.477	22,904		52.2	3.952	20,678	
M-010-E (13-08)	A-6	CL	100	29.7	4.124	16,710	17,012	45.5	6.531	16,006	16,345
			200	30.1	4.182	16,898		46.1	6.637	15,991	
			500	30.2	4.256	16,855		46.3	6.562	16,218	
			800	30.7	4.226	16,995		46.5	6.433	16,417	
			1000	30.5	4.202	17,186		46.6	6.492	16,399	
M-010-E (13-08)	A-6	CL	100	48.6	3.375	14,374	15,561	77.7	7.441	9,934	9,553
			200	49.6	3.334	15,053		78.1	7.453	9,867	
			500	49.6	3.271	15,423		78.2	7.743	9,627	
			800	49.8	3.258	15,631		78.2	7.779	9,528	
			1000	49.8	3.257	15,629		78.3	7.849	9,504	
M-010-E (13-08)	A-6	CL	100	51.5	1.331	31,968	44,641	83.3	1.796	36,929	41,989
			200	51.8	1.291	36,534		82.8	1.731	38,488	
			500	52.7	1.218	43,564		82.5	1.808	40,155	
			800	52.3	1.211	45,089		82.2	1.629	42,399	
			1000	51.8	1.152	45,271		82.4	1.793	43,414	
M-010-E (13-08)	A-6	CL	100	39.6	7.658	8,407	9,713	46.6	8.078	8,440	8,280
			200	41.3	7.157	9,399		47.1	7.723	8,966	
			500	42.2	6.971	9,004		48.6	7.368	8,822	
			800	43.5	6.473	9,818		48.8	7.052	8,108	
			1000	44.0	6.376	10,317		48.5	7.166	7,910	

Table A.4 (cont'd)

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
I-075-S (14-01)	A-7-6	SC	100	30.3	2.283	11,369	18,221	48.9	3.899	14,813	17,842
			200	31.0	2.259	13,560		48.7	3.804	15,893	
			500	31.3	2.074	17,389		48.9	3.721	16,938	
			800	31.7	1.971	18,449		49.6	3.586	18,253	
			1000	32.0	2.067	18,825		49.6	3.573	18,336	
I-075-S (14-01)	A-7-6	SC	100	51.3	2.172	32,901	32,510	82.3	2.390	32,808	29,860
			200	51.2	1.994	36,098		82.8	2.299	31,287	
			500	52.3	1.815	31,799		82.7	2.045	29,226	
			800	52.3	1.481	32,377		82.4	1.858	30,295	
			1000	51.7	1.417	33,354		82.3	1.668	30,060	
I-075-S (14-01)	A-7-6	SC	100	35.0	10.936	5,114	7,187	61.1	14.155	6,907	8,386
			200	35.9	9.896	5,982		61.6	13.624	7,285	
			500	36.6	9.349	7,441		62.5	12.617	7,928	
			800	36.7	8.808	7,284		63.5	12.002	8,545	
			1000	37.3	8.616	6,835		64.1	11.896	8,685	
U-024-S (14-04)	A-3	SP	100	32.7	2.313	21,994	22,765	51.7	3.526	19,968	21,913
			200	33.0	2.337	22,227		52.0	3.540	20,296	
			500	33.5	2.305	22,633		52.3	3.336	21,874	
			800	33.6	2.300	22,813		52.1	3.396	21,707	
			1000	33.6	2.297	22,849		52.4	3.329	22,159	
M-153-E (14-06)	A-7-6	SC	100	42.5	10.621	3,715	3,732	65.6	19.895	3,023	3,015
			200	42.7	10.593	3,736		65.4	19.906	3,021	
			500	42.8	10.681	3,717		66.4	20.023	3,036	
			800	42.9	10.628	3,745		66.4	20.120	3,014	
			1000	43.1	10.729	3,733		66.0	20.123	2,995	
M-153-E (14-06)	A-7-6	SC	100	29.2	13.397	3,483	4,430	65.6	19.995	4,023	3,915
			200	30.3	12.818	3,798		65.4	19.996	3,821	
			500	32.1	12.062	4,285		66.4	20.017	3,936	
			800	32.8	11.875	4,471		66.4	21.120	3,814	
			1000	33.0	11.708	4,535		66.0	21.123	3,995	

Table A.4 (cont'd)

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-153-E (14-06)	A-7-6	SC	100	33.8	2.378	38,348	40,902	51.2	3.050	42,427	44,483
			200	34.4	2.254	39,970		51.5	2.964	42,728	
			500	34.5	2.223	40,365		51.4	3.004	43,684	
			800	34.0	2.119	41,453		51.5	2.965	44,394	
			1000	33.7	2.120	40,889		52.2	2.876	45,372	
M-053-S (14-07)	A-3	SP	100	33.6	2.237	25,772	25,738	51.5	3.727	21,646	22,296
			200	33.6	2.150	25,870		51.7	3.731	21,643	
			500	33.9	2.249	26,465		51.9	3.688	22,217	
			800	34.0	2.258	25,493		52.0	3.646	22,403	
			1000	33.7	2.315	25,255		51.8	3.622	22,268	
I-094-W (14-09)	A-7-6	CL	100	49.1	5.308	8,870	9,955	77.2	9.347	7,782	8,080
			200	49.1	5.217	9,211		77.2	9.253	7,846	
			500	49.2	4.966	9,690		77.1	9.107	7,995	
			800	48.8	4.777	9,943		77.8	9.010	8,089	
			1000	49.2	4.675	10,234		77.8	8.918	8,156	
I-094-W (14-09)	A-7-6	CL	100	51.2	2.114	45,953	73,344	81.9	2.609	57,985	70,094
			200	51.1	1.853	52,917		82.8	2.466	61,580	
			500	52.2	1.602	67,009		82.5	2.327	67,663	
			800	52.7	1.426	75,719		82.2	2.190	70,504	
			1000	51.2	1.383	77,304		81.8	2.205	72,116	
I-094-W (14-09)	A-7-6	CL	100	33.0	1.604	53,229	60,217	50.7	2.211	57,722	60,303
			200	32.7	1.585	55,517		51.7	2.228	59,950	
			500	33.8	1.530	60,326		51.7	2.152	60,448	
			800	34.1	1.462	60,280		51.8	2.104	60,142	
			1000	34.2	1.459	60,046		52.3	2.044	60,318	
M-053-S (15-02)	A-2-4	SM	100	33.3	2.923	18,400	18,342	51.1	4.424	18,022	18,060
			200	33.2	2.914	18,486		51.0	4.470	18,018	
			500	33.4	2.921	18,171		51.4	4.471	17,918	
			800	33.4	2.963	18,372		51.3	4.416	18,113	
			1000	33.4	2.894	18,483		51.0	4.372	18,149	

Table A.4 (cont'd)

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-090-E (15-04)	A-4	CL	100	34.6	1.494	65,657	67,841	51.5	2.170	60,204	62,065
			200	34.2	1.492	65,191		51.9	2.192	61,455	
			500	34.6	1.487	67,087		51.7	2.159	61,666	
			800	34.6	1.510	68,335		51.7	2.128	62,105	
			1000	34.5	1.398	68,102		52.0	2.212	62,423	
M-025-S (15-05)	A-3	SP	100	34.0	1.585	37,971	40,152	52.6	2.503	35,506	35,481
			200	34.0	1.601	38,716		52.2	2.445	35,369	
			500	34.1	1.588	39,705		51.7	2.500	35,195	
			800	34.9	1.643	40,506		52.3	2.468	35,680	
			1000	35.0	1.595	40,246		52.0	2.437	35,567	
M-019-S (15-07)	A-2-4	SM	100	34.3	2.740	19,702	22,233	51.3	4.328	18,630	19,500
			200	35.7	2.770	20,960		51.9	4.203	18,904	
			500	35.0	2.584	21,859		51.7	4.118	19,310	
			800	35.2	2.539	22,379		51.4	4.096	19,441	
			1000	34.6	2.572	22,462		53.2	4.183	19,750	

Table A.5 MR results

Sample number	Sample type		Classification		Dry unit weight (lb/ft^3)	Water content for cyclic test	Saturation	Average 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
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

Table A.5 (cont'd)

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
U-023-S (04-01)		X	A-3	SP	117.8	3.3	20.7	23,039	21,715
U-031-N (06-03)		X	A-3	SP	111.5	3.3	17.4	31,870	29,609
M-020-E (07-03)		X	A-3	SP	113.0	3.2	17.6	32,666	28,156
M-025-S (15-05)		X	A-3	SP	110.3	3.0	15.4	40,115	35,447
U-131-S (09-01)		X	A-3	SP	110.6	2.7	13.9	28,766	27,706
I-075-N (06-02)		X	A-3	SP	110.2	2.0	10.2	32,457	31,187
U-131-S (09-05)		X	A-3	SP	117.3	1.0	23.3	38,423	35,319
M-028-W (03-02)		X	A-3	SP	104.0	1.3	5.7	22,959	22,494
U-131-S (09-03)		X	A-3	SP	108.6	0.5	2.4	30,340	27,995
M-020-W (07-02)		X	A-3	SP	109.1	0.2	1.0	31,460	28,705
M-020-W (07-02)		X	A-3	SP	104.1	0.2	0.9	19,692	20,267
M-020-W (07-02)		X	A-3	SP	107.6	0.2	1.0	24,319	24,552
U-002-E (02-01)		X	A-4	SM	109.3	9.5	47.4	15,352	13,818
U-002-E (03-03)		X	A-2-4	SM	111.5	7.7	40.7	15,969	15,818
M-065-S (04-04)		X	A-2-4	SM	94.6	7.6	26.3	11,932	11,898
U-131-N (05-01)		X	A-2-4	SM	112.9	5.4	29.6	24,627	23,092
M-061-E (07-06)		X	A-2-4	SM	96.0	17.0	60.8	11,480	12,958
M-061-E (08-02)		X	A-2-4	SM	118.6	5.5	35.3	32,200	31,733
M-044-E (09-07)		X	A-2-4	SM	128.8	7.6	66.6	18,416	19,636

Table A.5 (cont'd)

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-024-S (09-09)		X	A-2-4	SM	102.9	9.8	41.5	15,142	15,839
I-069-N (10-04)		X	A-2-4	SM	124.2	8.4	63.6	19,172	18,945
I-069-N (10-05)		X	A-2-4	SM	100.2	23.7	93.9	5,290	5,641
I-096-W (10-09)		X	A-2-4	SM	117.9	14.1	88.7	9,509	11,383
U-012-E (12-04)		X	A-2-4	SM	108.0	3.9	18.8	19,152	18,377
I-094-W (12-06)		X	A-2-4	SM	123.2	10.3	75.7	19,406	19,305
M-024-S (13-01)		X	A-4	SM	110.3	9.5	48.6	17,933	16,160
M-053-S (15-02)		X	A-2-4	SM	114.0	8.5	48.0	18,325	18,043
M-019-S (15-07)		X	A-2-4	SM	113.7	9.2	51.6	22,213	19,482
M-068-W (04-02)		X	A-2-4	SC-SM	117.5	2.2	13.7	30,928	24,740
M-032-W (04-05)		X	A-4	SC-SM	106.3	8.1	37.4	19,255	18,161
I-075-N (08-06)		X	A-2-4	SC-SM	131.3	9.2	87.7	15,783	16,562
I-096-W (09-02)		X	A-2-4	SC-SM	108.0	1.2	5.8	22,142	19,579
M-060-W (11-03)		X	A-2-4	SC-SM	107.0	8.4	39.5	19,812	16,639
I-069-S (11-05)		X	A-4	SC-SM	132.6	6.4	63.9	27,276	25,621
I-094-W (12-01)		X	A-2-4	SC-SM	128.8	8.5	74.5	27,610	23,849

Table A.5 (cont'd)

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-045-S (01-01)		X	A-6	CL	120.8	10.2	69.8	36,543	31,503
M-010-E (13-08)	X		A-6	CL	118.8	15.0	96.8	9,714	8,235
M-010-E (13-08)		X	A-6	CL	122.1	10.4	73.9	17,150	16,572
M-010-E (13-08)	X		A-6	CL	128.5	5.7	49.5	44,634	41,942
M-010-E (13-08)	X		A-6	CL	122.3	12.3	88.0	15,561	9,553
I-094-W (14-09)	X		A-7-6	CL	101.7	10.5	43.2	73,444	70,095
I-094-W (14-09)		X	A-7-6	CL	101.6	11.3	46.3	60,247	60,327
I-094-W (14-09)	X		A-7-6	CL	96.4	26.3	95.0	9,955	8,080
M-090-E (15-04)		X	A-4	CL	109.5	10.6	53.1	67,778	62,006
M-072-W (05-06)		X	A-6	SC	116.2	10.7	64.2	26,492	27,193
U-127-N (07-05)	X		A-6	SC	120.5	11.2	75.9	7,323	6,925
U-127-N (07-05)	X		A-6	SC	117.5	14.2	88.4	4,713	5,338

Table A.5 (cont'd)

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
U-127-N (07-05)	X		A-6	SC	126.7	6.7	54.9	54,737	53,030
U-127-N (07-05)	X		A-6	SC	115.3	16.6	97.2	3,984	5,382
U-127-N (07-05)		X	A-6	SC	117.7	10.3	64.5	36,047	27,746
U-010-W (08-04)	X		A-6	SC	111.5	15.0	79.3	5,879	5,105
U-010-W (08-04)	X		A-6	SC	113.8	16.7	93.8	4,134	5,268
I-096-W (10-03)		X	A-2-6	SC	108.3	11.6	56.4	43,783	37,688
I-075-S (14-01)	X		A-7-6	SC	115.7	8.8	52.1	32,569	29,839
I-075-S (14-01)	X		A-7-6	SC	108.7	18.4	90.3	7,187	8,386
I-075-S (14-01)	X		A-7-6	SC	106.2	20.9	96.2	6,069	4,007
I-075-S (14-01)		X	A-7-6	SC	99.8	18.8	73.8	18,147	17,831
M-153-E (14-06)	X		A-7-6	SC	96.6	26.0	99.1	3,731	3,015
M-153-E (14-06)	X		A-7-6	SC	92.5	30.4	99.9	4,430	3,921
M-153-E (14-06)		X	A-7-6	SC	101.8	10.7	44.1	40,864	44,442
M-028-W (02-02)		X	A-4	ML	106.2	11.0	50.6	53,824	41,516
U-002-E (02-04)		X	A-4	ML	113.0	10.7	58.8	37,012	33,191

APPENDIX B

NDT data test results

This appendix contains two tables; Table B.1 provides a list of the results of backcalculated layer moduli of roadbed soils supporting flexible pavements.

Table B.2, on the other hand, provides the results of the backcalculated layer moduli of roadbed soils supporting rigid pavements

Table B.1 Backcalculated results of flexible pavement

Location			Roadbed type USCS	FWD File Information	Pavement layer thickness (in)		Backcalculation				Resilient modulus (psi)		
Region	Road	Cluster-area		File title	Asphalt concrete	Base/ subbase	Error RMS (%)	Converged? Yes	Converged? No	Depth to stiff layer (in)	Asphalt concrete	Base/ subbase	Roadbed
North	US-131	07-01	SM	flex-N-US131-CS67017-05-01-2002	7.25	22	0.63	79	14	700	1696003	26912	23263
Superior	US-2	02-01	SM	flex-Su-US2-CS27022-05-20-2008	3.5	26.5	1.47	7	4	250	972815	45256	21132
Grand	M-57	09-01	SP1	flex-G-M57-CS41122-8-23-1994	3	26	1.20	81	27	700	2446547	44131	29428
Grand	M-57	09-01	SP1	flex-G-M57-CS28021-5-23-1995	3	26	1.38	20	63	700	3871662	33601	32384
Grand	M-57	09-01	SP1	flex-G-M57-CS59021-08-23-1994	3	26	1.21	40	13	700	2756285	42908	29943
Grand	M-57	09-01	SP1	flex-G-M57-CS59021-08-23-1994-(2)	3	26	1.27	43	10	700	2680244	43546	29965
Grand	M-57	09-01	SP1	flex-G-M57-CS59021-08-23-1994-(3)	3	26	1.32	34	20	700	2657574	44229	32430
Grand	M-57	09-01	SP1	flex-G-M57-CS59021-08-23-1994-(4)	3	26	1.27	33	20	700	2663880	43749	28829
Grand	US-131	07-03	SP1	flex-G-US131-CS54013-08-18-1994	7.25	24	1.13	43	22	700	370266	38527	27557
Grand	US-131	07-03	SP1	flex-G-US131-CS54013-08-18-1994-(2)	7.5	22	1.49	16	16	700	372701	38341	31242
Grand	US-131	07-03	SP1	flex-G-US131-CS54013-08-18-1994-(3)	7.5	22	1.51	18	15	700	351947	38155	30740
Grand	US-131	07-03	SP1	flex-G-US131-CS54013-08-18-1994-(4)	7.5	22	1.15	47	18	700	354711	37817	27911
Grand	US-131	07-03	SP1	flex-G-US131-CS54013-08-18-1994-(5)	7.5	22	1.52	18	15	700	351862	38153	30744
Grand	M-120	06-03	SP1	flex-G-M120-CS61012-07-23-1998	7	24	1.43	28	46	700	191605	28836	18572
		07-02	SP1										
Grand	US-131	09-01	SP1	flex-G-US131-CS59012-06-25-1998	8	24	1.46	20	5	700	529653	38384	29069
Grand	M-37	07-03	SP1	flex-G-M37-CS62032-05-18-2000	8	25	1.28	25	15	700	433004	21019	20953
Grand	US-131	07-02	SP1	flex-G-US131-CS54013-08-18-1994	7.25	22	1.26	130	51	700	937313	30697	34067
Grand	M-20	07-03	SP1	flex-G-M20-CS54041-04-09-2002	6	24	1.09	48	20	700	592526	24492	25668
North	I-75	06-02	SP1	flex-N-I75-CS69014-11-12-1997	6.25	24	1.09	61	36	700	1073344	37124	30358
North	I-75	06-02	SP1	flex-N-I75-CS69014-08-03-1999	6.25	24	1.34	62	28	700	460044	49111	31074
North	I-75	06-02	SP1	flex-N-I75-CS69014-08-03-1999-(2)	6.25	24	1.36	61	29	700	516468	55316	40422
North	I-75	06-02	SP1	flex-N-I75-CS69014-11-23-1997	6.25	24	1.09	71	22	700	976787	38138	30700
North	I-75	06-02	SP1	flex-N-I75-CS69014-08-04-1999	6.25	24	1.36	65	27	700	364430	48793	30134
North	I-75	06-02	SP1	flex-N-I75-CS69014-08-04-1999-(2)	6.25	24	1.53	63	48	700	440817	52962	40183
North	M-55	05-04	SP2	flex-N-M55-CS77022-8-20-2001	7	23	1.75	32	44	700	253762	29680	23140

Table B.1 (cont'd)

Location			Roadbed type USCS	FWD File Information	Pavement layer thickness (in)		Backcalculation				Resilient modulus (psi)		
Region	Road	Cluster-area		File title	Asphalt concrete	Base/ subbase	Error RMS (%)	Converged? Yes	Converged? No	Depth to stiff layer (in)	Asphalt concrete	Base/ subbase	Roadbed
North	M-55	05-04	SP2	flex-N-M55-CS77022-8-20-2001-(2)	7	23	1.74	35	41	700	250435	29872	22953
Superior	M-28	03-01	SP-SM	flex-Su-M28-CS75061-05-21-2008	5.5	24.5	0.65	11	0	700	1719937	27402	21272
Superior	US-2	03-01	SP-SM	flex-Su-US2-CS75021-05-22-2008	5.5	24.5	1.28	10	1	300	3987549	60149	20953
Superior	M-28	03-03	SP-SM	flex-Su-M28-CS17061-05-22-2008	5	25	0.66	10	0	700	2255609	23293	21652
North	US-23	04-02	SC-SM	flex-N-US23-CS4032-06-03-2008	5	25	1.49	9	2	150	2126531	59749	14388
North	US-23	04-02	SC-SM	flex-N-US23-CS71073-06-04-2008-(2)	5.5	24.5	1.60	8	3	300	2175426	63071	23691
North	US-23	04-02	SC-SM	flex-N-US23-CS71073-06-04-2008	6.5	23.5	0.87	11	0	700	540464	32021	20721
North	US-23	04-02	SC-SM	flex-N-US23-CS1052-06-03-2008	3.5	26.5	1.77	6	5	200	1292560	62082	16445
Bay	M-57	09-08	SC	flex-B-M57-CS29022-08-30-1994	5.5	25	1.41	43	23	700	278977	26637	26310
Bay	M-57	09-08	SC	flex-B-M57-CS29022-08-30-1994-(2)	5.5	25	1.39	41	25	700	278587	26544	26385
Bay	M-57	09-08	SC	flex-B-M57-CS29022-08-30-1994-(3)	5.5	25	1.70	25	42	700	265828	27872	29292
Bay	M-57	09-08	SC	flex-B-M57-CS29022-08-30-1994-(4)	5.5	25	1.69	24	43	700	266435	27982	29611
Bay	M-57	09-08	SC	flex-B-M57-CS29022-01-28-1993	5.5	26	1.41	65	69	700	269243	26067	25798
Bay	M-84	09-08	SC	flex-B-M84-CS9011-10-03-2005	4	25	1.52	8	8	200	412843	35590	22034
Bay	M-84	09-08	SC	flex-B-M84-CS9011-05-17-2005	4	25	1.07	30	9	300	1058438	24543	19099
Bay	M-84	09-08	SC	flex-B-M84-CS9011-05-17-2005-(2)	4	25	1.15	52	17	400	1224141	23099	20129
Bay	M-84	09-08	SC	flex-B-M84-CS9011-10-10-2005	4	25	0.97	16	0	275	1179156	34888	25923
Bay	M-84	09-08	SC	flex-B-M84-CS9011-09-11-2005	4	25	1.40	30	2	160	2959369	47342	19575
Bay	M-84	09-08	SC	flex-B-M84-CS9011-09-13-2005-(2)	4	25	0.91	16	0	250	774323	32298	27322
Superior	I-75	03-05	SC	flex-Su-I75-CS49025-05-22-2008	7.5	22.5	1.41	4	7	150	783951	35186	55215
University	M-52	10-10	SC	flex-U-M52-CS33051-11-13-2002	6	24	1.14	39	5	250	776627	23650	23582
Metro	M-53	14-08	CL	flex-M-M53-CS50015-04-04-2008	8	24	1.07	9	2	300	1122329	16031	22259
Metro	I-94	14-09	CL	flex-M-I94-CS77111-04-02-2008	4.2	17.5	1.78	5	19	100	1937770	11660	4763
Superior	M-38	01-01	CL	flex-Su-M28-CS66042-05-20-2008	3.5	26.5	1.58	10	1	350	1468878	32573	18372
Superior	US-41	02-04	ML	flex-Su-US41-CS7013-05-19-2008	2.5	27.5	1.28	11	0	150	2059728	29874	10531
Superior	US-141	02-02	ML	flex-Su-US141-CS7022-05-19-2008	4.5	25.5	1.10	12	1	700	1075462	16677	21265

Table B.2 Backcalculated results of rigid pavement

Location			Roadbed type USCS	FWD File Information			Concrete slab thickness (in)	Number of tests	Slab (Ec)	Roadbed K (pci)	Roadbed MR (psi)
Region	Road	Cluster-area		Season	Date	File title					
Bay	US-23	09-09	SM	Summer	10/21/1998	rigid-B-US23-CS25031-10-21-1998	9	34	4,320,980	437	33,920
Bay	US-23	09-09	SM	Summer	5/30/2001	rigid-B-US23-CS25031-05-30-2001	9	46	2,748,666	351	27,250
Bay	US-23	09-09	SM	Summer	8/23/2005	rigid-B-US23-CS25031-08-23-2005-(2)	9	17	1,792,001	392	30,384
Bay	I-475	09-09	SM	Summer	6/26/1997	rigid-B-I475-CS25132-06-26-1997	9	60	3,104,905	283	21,958
Bay	I-475	09-09	SM	Summer	6/24/2001	rigid-B-I475-CS25132-06-24-2001	9	66	2,263,476	307	23,832
Grand	I-96	09-07	SM	Summer	6/27/2001	rigid-G-I96-CS34044-06-27-2001	9	21	1,268,974	347	26,950
University	I-69	10-04	SM	Summer	9/10/2007	rigid-U-I69-CS23063-09-10-2007	10	30	2,199,553	298	23,132
Grand	US-131	07-03	SP1	Summer	4/9/1998	rigid-G-US131-CS59012-04-09-1998	9	22	782,808	338	26,229
North	I-75	05-02	SP1	Summer	9/17/2001	rigid-N-I75-CS16091-09-17-2001	9	69	1,501,914	259	20,106
North	I-75	05-02	SP1	Summer	10/26/2001	rigid-N-I75-CS16091-10-26-2001	9	53	1,779,633	267	20,708
North	I-75	05-02	SP1	Summer	9/18/2001	rigid-N-I75-CS16092-09-18-2001	9	98	1,283,968	289	22,414
North	I-75	05-02	SP1	Summer	9/27/2001	rigid-N-I75-CS160992-09-27-2001	9	86	1,370,042	273	21,186
Superior	M-28	03-04	SP1	Summer	5/8/2001	rigid-Su-M28-CS17062-05-08-2001	8	80	1,874,921	235	18,226
Superior	I-75	03-04	SP1	Summer	5/31/2000	rigid-Su-I75-CS17033-05-31-2000	9	49	1,199,440	218	16,931
Superior	I-75	03-04	SP1	Summer	5/25/2000	rigid-Su-I75-CS17034-05-25-2000	9	16	1,330,163	247	19,141
Superior	I-75	03-04	SP1	Summer	5/22/2000	rigid-Su-I75-CS17034-05-22-2000	9	21	1,523,738	224	17,367
Bay	US-23	09-10	SP2	Summer	8/30/2005	rigid-B-US23-CS25031-08-30-2005	10	19	1,371,563	407	31,592
Bay	US-23	09-10	SP2	Summer	8/23/2005	rigid-B-US23-CS25031-08-23-2005	10	27	1,487,442	386	29,920
Bay	US-23	09-10	SP2	Summer	11/15/2005	rigid-B-US23-CS25031-11-15-2005	10	68	1,082,723	339	26,345
Bay	US-23	09-10	SP2	Summer	11/16/2005	rigid-B-US23-CS25031-11-16-2005	10	31	1,025,612	234	18,134
Bay	US-23	09-10	SP2	Summer	11/16/2005	rigid-B-US23-CS25031-11-16-2005-(2)	10	48	958,993	259	20,107
North	I-75	05-04	SP2	Summer	8/30/1997	rigid-N-I75-CS65041-08-30-2001	9	20	1,333,695	304	23,622
North	I-75	05-04	SP2	Summer	9/14/2001	rigid-N-I75-CS65041-09-14-2001	9	29	1,159,426	245	19,038
University	I-94	13-04	SP2	Summer	11/19/2006	rigid-U-I94-CS82021-11-19-2006	10	333	2,764,869	311	24,146
Southwest	I-94	12-05	SP-SM	Summer	11/18/2002	rigid-So-I94-CS11081-11-18-2002	9	84	1,402,982	216	16,759
Southwest	I-94	12-05	SP-SM	Summer	10/28/2004	rigid-So-I94-CS11081-10-28-2004	9	66	1,285,711	223	17,299
Southwest	I-94	12-05	SP-SM	Summer	10/30/2001	rigid-So-I94-CS11081-10-30-2001	9	12	1,247,204	192	14,871
Southwest	US-31	06-05	SP-SM	Summer	10/9/2001	rigid-So-US31-CS11057-10-09-2001	9	28	819,421	194	15,028
Southwest	US-31	06-05	SP-SM	Summer	10/30/2001	rigid-So-US31-CS11057-10-30-2001	9	27	1,909,724	192	14,879
Southwest	US-31	06-05	SP-SM	Summer	6/6/2003	rigid-So-US31-CS11057-06-06-2003	9	16	1,398,211	240	18,636
Southwest	US-31	06-05	SP-SM	Summer	4/18/2008	rigid-So-US31-CS11057-05-14-2008	9	33	3,943,545	218	16,897

Table B.2 (cont'd)

Location			Roadbed type USCS	FWD File Information			Concrete slab thickness (in)	Number of tests	Slab (Ec)	Roadbed K (pci)	Roadbed MR (psi)
Region	Road	Cluster-area		Season	Date	File title					
Southwest	US-31	06-05	SP-SM	Summer	11/9/2007	rigid-So-US31-CS3032-11-09-2007	10	33	2,444,743	390	30,295
Southwest	I-196	06-04	SP-SM	Summer	5/14/2008	rigid-So-I196-CS3033-05-14-2008	9	33	7,774,538	303	23,527
Southwest	I-196	06-04	SP-SM	Summer	9/11/2007	rigid-So-I196-CS3033-11-09-2007	9	36	5,170,572	445	34,562
Superior	M-28	03-01	SP-SM	Summer	8/23/2001	rigid-Su-M28-CS02041-08-23-2001	10	21	1,288,074	259	20,073
Superior	M-28	03-01	SP-SM	Summer	8/23/2001	rigid-Su-M28-CS02041-08-23-2001-(2)	10	46	1,006,652	209	16,199
University	US-23	13-06	SP-SM	Summer	9/14/2006	rigid-U-US23-CS58034-09-14-2006	10	79	931,042	365	28,310
Bay	I-75	08-06	SC-SM	Summer	9/13/2001	rigid-B-I75-CS6111-09-13-2001	9	57	1,617,746	286	22,182
Grand	US-131	09-02	SC-SM	Summer	11/7/1996	rigid-G-US131-CS41131-07-11-1996-(2)	9	9	2,055,282	313	24,279
Grand	M-6	09-02	SC-SM	Summer	9/15/2004	rigid-G-M6-CS41064-09-15-2004	10	57	2,657,345	392	30,406
Grand	M-6	09-02	SC-SM	Summer	9/8/2004	rigid-G-M6-CS41064-09-29-2004	10	653	6,913,721	262	20,344
Grand	M-6	09-02	SC-SM	Summer	9/8/2004	rigid-G-M6-CS41064-09-08-2004	10	665	6,929,648	262	20,329
Grand	M-6	09-02	SC-SM	Summer	11/15/2001	rigid-G-M6-CS41064-11-15-2001	10	159	3,091,380	253	19,654
Southwest	I-69	11-03	SC-SM	Summer	9/11/2001	rigid-So-I69-CS12034-09-11-2001	9	39	2,551,331	384	29,825
Southwest	I-69	11-03	SC-SM	Summer	10/8/1998	rigid-So-I69-CS12034-10-08-1998	9	7	1,224,345	342	26,522
Southwest	I-69	11-03	SC-SM	Summer	10/9/1998	rigid-So-I69-CS12034-10-09-1998	9	7	1,385,348	313	24,319
Southwest	I-69	11-05	SC-SM	Summer	12/18/2001	rigid-So-I69-CS12033-12-18-2001	9	65	2,286,690	253	19,637
University	US-127	10-02	SC-SM	Summer	6/15/1998	rigid-G-US27-CS19033-06-15-1998	10	249	3,688,356	222	17,215
University	US-127	10-02	SC-SM	Summer	11/7/2007	rigid-U-US127-CS19034-11-07-2007	10	31	7,943,405	379	29,442
Bay	US-127	09-08	SC	Summer	6/27/2008	rigid-B-US127-CS29011-06-27-2008	9	33	5,013,736	255	19,785
Bay	I-75	09-08	SC	Summer	8/15/2001	rigid-BI75-CS73101-08-15-2001	9	47	1,249,188	244	18,953
Bay	I-75	09-08	SC	Summer	11/30/1999	rigid-BI75-CS73101-11-30-1999	9	19	2,514,512	257	19,970
Bay	I-75	08-04	SC	Summer	7/2/2008	rigid-B-I75-CS3035-07-02-2008	9	36	4,624,514	272	21,138
Bay	I-675	09-08	SC	Summer	10/24/2003	rigid-B-I675-CS73101-10-24-2003	9	72	1,298,419	291	22,548
Bay	I-675	09-08	SC	Summer	5/26/2004	rigid-B-I675-CS73101-05-26-2004	9	49	1,322,471	285	22,091
Bay	I-675	09-08	SC	Summer	10/14/2004	rigid-B-I675-CS73101-10-14-2004	9	75	986,784	225	17,439
Bay	I-675	09-08	SC	Summer	12/5/2005	rigid-B-I675-CS73101-12-05-2005	9	63	1,634,031	281	21,767
Bay	US-10	08-04	SC	Summer	12/18/2007	rigid-B-US10-CS9101-12-18-2007	7.3	36	4,976,290	290	22,537
Bay	US-127	07-05	SC	Summer	12/19/2007	rigid-B-US127-CS37014-12-19-2007	8	45	2,490,183	475	36,844
Metro	M-5	13-03	SC	Summer	11/29/2006	rigid-M-M5-CS00000-11-29-2006	10	69	2,378,364	304	23,566
Metro	M-10	14-06	SC	Summer	10-3-2007	rigid-M-M10-CS82111-10-03-2007	10	44	2,770,674	663	51,486
Metro	I-94	14-05	SC	Summer	10/6/2005	rigid-M-I94-CS82022-10-06-2005	10	37	2,104,643	340	26,411

Table B.2 (cont'd)

Location			Roadbed type USCS	FWD File Information			Concrete slab thickness (in)	Number of tests	Slab (Ec)	Roadbed K (pci)	Roadbed MR (psi)
Region	Road	Cluster-area		Season	Date	File title					
Metro	I-94	14-05	SC	Summer	10/13/2005	rigid-M-I94-CS82022-10-13-2005	10	38	2,104,704	327	25,381
Metro	I-94	14-05	SC	Summer	10/26/2005	rigid-M-I94-CS82022-10-26-2005	10	49	2,226,098	320	24,813
Metro	I-94	14-05	SC	Summer	10/31/2005	rigid-M-I94-CS82022-10-31-2005	10	34	2,289,527	354	27,468
Metro	I-94	14-05	SC	Summer	9/30/2005	rigid-M-I94-CS82022-09-30-2005	10	64	2,240,000	303	23,523
Metro	I-94	14-05	SC	Summer	11/1/2005	rigid-M-I94-CS82022-11-01-2005	10	79	1,980,028	258	20,003
Superior	I-75	03-05	SC	Summer	6/13/2000	rigid-Su-I75-CS49025-06-13-2000	9	63	1,751,835	278	21,555
Superior	I-75	03-05	SC	Summer	6/2/2000	rigid-Su-I75-CS49025-06-02-2000	9	40	1,115,443	240	18,646
University	I-75	14-03	SC	Summer	10/6/2006	rigid-U-I75-CS58152-10-06-2006	10	21	1,085,465	228	17,695
University	I-75	14-01	SC	Summer	12/4/2007	rigid-U-I75-CS58151-12-04-2007	9.3	83	4,002,494	297	23,071
University	I-69	10-08	SC	Summer	6/25/2001	rigid-U-I69-CS19043-06-25-2001	9	14	1,163,467	217	16,805
University	I-69	10-08	SC	Summer	5/14/2002	rigid-U-I69-CS19043-05-14-2002	9	98	1,895,200	326	25,285
University	I-69	10-08	SC	Summer	9/18/1998	rigid-U-I69-CS19042-09-18-1999	9	10	1,687,833	339	26,277
University	I-75	14-01	SC	?	3/31/2008	rigid-U-I75-CS58151-03-31-2008	10	57	4,346,544	175	13,580
Metro	I-94	14-05	SC	Summer	9/16/2008	rigid-M-I94-CS82022-09-16-2008	12.25	78	2,982,552	310	24,056
Metro	I-94	14-06	SC	Summer	9/16/2008	rigid-M-I94-CS82022-09-16-2008-(2)	12.5	66	3,536,866	373	28,945
Metro	I-75	14-06	SC	Summer	9/16/2008	rigid-M-I75-CS82194-09-16-2008	12	62	3,246,240	300	23,300
Metro	M-14	13-08	CL	Summer	11/29/2006	rigid-M-M14-CS82102-11-29-2006	10	66	4,335,070	335	25,963
Metro	I-69	14-09 15-03	CL	Summer	7/2/1997	rigid-M-I69-CS77023-07-02-1997	9	18	2,283,001	233	18,107
Metro	I-69	15-03	CL	?	4/2/2008	rigid-M-I69-CS77024-04-02-2008	10	39	2,442,439	118	9,181

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