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Layer Moduli of Nebraska Pavements for the New Mechanistic Empirical Pavement Design Guide

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16. Abstract

As a step-wise implementation effort of the Mechanistic-Empirical Pavement Design Guide (MEPDG) for the design and analysis of Nebraska flexible pavement systems, this research develops a database of layer moduli — dynamic modulus, creep compliance, and resilient modulus — of various pavement materials used in Nebraska. The database includes all three design input levels. Direct laboratory tests of the representative Nebraska pavement materials are conducted for Level 1 design inputs, and surrogate methods, such as the use of Witczak's predictive equations and the use of default resilient moduli based on soil classification data, are evaluated to include Level 2 and/or Level 3 design inputs. Test results and layer modulus values are summarized in Appendices. Modulus values characterized for each design level are then input into the MEPDG software to investigate level-dependent performance sensitivity of typical asphalt pavements. The MEPDG performance simulation results then reveal any insights into the applicability of different modulus input levels for the design of typical Nebraska pavements. Significant results and findings are presented in this report.

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CHAPTER 1
INTRODUCTION

A new Mechanistic-Empirical Pavement Design Guide (MEPDG) has been developed and validated by many researchers and practitioners. This procedure was developed by the National Cooperative Highway Research Program (NCHRP) under sponsorship by the American Association of State Highway and Transportation Officials (AASHTO). The design guide represents a challenging innovation in the way pavement design is performed: design inputs include traffic (full load spectra for various axle configurations), material and subgrade characterization, climatic factors, performance criteria, and many others. One of the most interesting aspects of the design procedure is its hierarchical approach; i.e., the consideration of different levels of inputs. Level 1 requires the engineer to obtain the most accurate design inputs (e.g., direct testing of materials, on-site traffic load data, etc.). Level 2 requires testing, but the use of correlations is allowed (e.g., subgrade modulus estimated through correlation with another test), and Level 3 generally uses estimated values. Thus, Level 1 has the least possible error associated with inputs, Level 2 uses estimated values or correlations, and Level 3 is based on the default values.

Although evaluation of this new design procedure is still underway, many state transportation agencies have already begun adaptation and local calibration of this procedure for better and more efficient implementation with their local pavements. The Nebraska Department of Roads (NDOR) has also initiated this implementation process for a new design for Nebraska pavements with a research project, MPM-04 *"Toward Implementation of Mechanistic-Empirical Pavement Design in Nebraska"* funded in 2006. This project is primarily aimed at identification of the significant design factors involved and the development of a road map for step-by-step transition to the new design guide.

Among design factors involved in the new design guide, the key factors, from a materials aspect, include the layer moduli represented by *dynamic modulus* and *creep compliance*

for asphalt layers in flexible pavements and the *resilient modulus* for soils and unbound aggregate layers. These all represent mandatory design inputs that serve as stiffness indicators of the pavement system. Recent research has clearly emphasized the importance of accurate evaluation of layer moduli, because these moduli significantly affect overall pavement performance and they are typically quite dependent on local materials and regional environments. Evaluation of layer moduli is therefore viewed as a primary and most urgent implementation step.

1.1 RESEARCH OBJECTIVES

The primary objective of this research is to develop a database by performing tests of dynamic modulus, creep compliance, and resilient modulus in various pavement materials used in Nebraska. In addition to the direct laboratory testing of the representative Nebraska pavement materials for Level 1 design inputs in the modulus database, surrogate methods such as the use of Witczak's predictive equations and the use of default resilient moduli based on Nebraska soil classification data are also evaluated to include Level 2 and/or Level 3 design inputs. This allows investigation of their applicability for the design of pavements that are normally subject to low traffic volume. Modulus values characterized for each design level are then input into the MEPDG software to investigate level-dependent performance sensitivity of typical asphalt pavements. Findings from this study can also be related and/or compared to other studies that have already been conducted in other states, so that better and more reliable implementation of the new design concept can be accomplished for Nebraska asphalt pavements.

1.2. RESEARCH SCOPE

To accomplish the objectives, four primary tasks are performed in this research. Task 1 consists of a careful review of the recent literature related to MEPDG implementation, putting particularly more emphasis on the development of a layer modulus database. The second task is to establish mechanical testing facilities and analysis programs for the modulus characterization of various pavement materials (asphalt mixtures and soils). The

UTM-25kN mechanical testing equipment at the University of Nebraska-Lincoln (UNL) geomaterials laboratory was used for this effort, with several additions of testing accessories and new devices. The third task in this research is the selection and laboratory testing of local materials and mixtures to identify layer modulus characteristics that lead to the modulus database. The database includes all three design input levels. Task 4 uses the layer modulus database to perform sensitivity analyses by MEPDG simulations to investigate the effects of modulus input levels on overall pavement performance. The MEPDG performance simulation results can then be used to search for any insights into the applicability of different modulus input levels for the design of typical Nebraska pavements.

1.3. ORGANIZATION OF THE REPORT

This report is composed of six chapters. Following this introduction (Chapter 1), Chapter 2 presents background information related to the new design guide, MEPDG and its local implementation efforts, focusing in particular on the development of the modulus database. Chapter 3 presents detailed descriptions of material selection and the testing facilities used in this research. Chapter 4 shows the results of the laboratory tests conducted, which led to the MEPDG design input database for each design level. The design input database is tabulated for individual asphalt mixtures and soil samples and is located in the Appendices. Chapter 5 provides a discussion of sensitivity analyses of pavement performance conducted with different MEPDG input levels. Finally, Chapter 6 provides a summary and conclusions of this study. NDOR implementation plans are also presented in that chapter.

CHAPTER 2
BACKGROUND

This chapter presents background information related to the new design guide, MEPDG, and its local implementation efforts made by other researchers. The discussion focuses in particular on the development of the modulus database and its application to local practices to investigate design input sensitivity.

2.1 MEPDG ANALYSIS

The MEPDG is an analysis tool that enables prediction of pavement performances over time for a given pavement structure subjected to variable conditions, such as traffic and climate. The mechanistic-empirical design of new and reconstructed flexible pavements requires an iterative hands-on approach by the designer. The designer must select a trial design and then analyze the design to determine if it meets the performance criteria established by the designer. If the trial design does not satisfy the performance criteria, the design is modified and reanalyzed until the design satisfies the performance criteria (NCHRP 1-37A 2004).

The procedure for use of the MEPDG depends heavily on the characterization of the fundamental engineering properties of paving materials. It requires a number of input data in four major categories: traffic, materials, environmental influences, and pavement response and distress models. As shown in Figure 2-1, the design procedure accounts for the environmental conditions that may affect pavement response. These pavement responses are determined by mechanistic procedures. The mechanistic method determines structural response (i.e., stresses and strains) in the pavement structure. The transfer function is utilized for direct empirical calculation of individual distresses such as top-down cracking, bottom-up cracking, thermal cracking, rutting, and roughness.

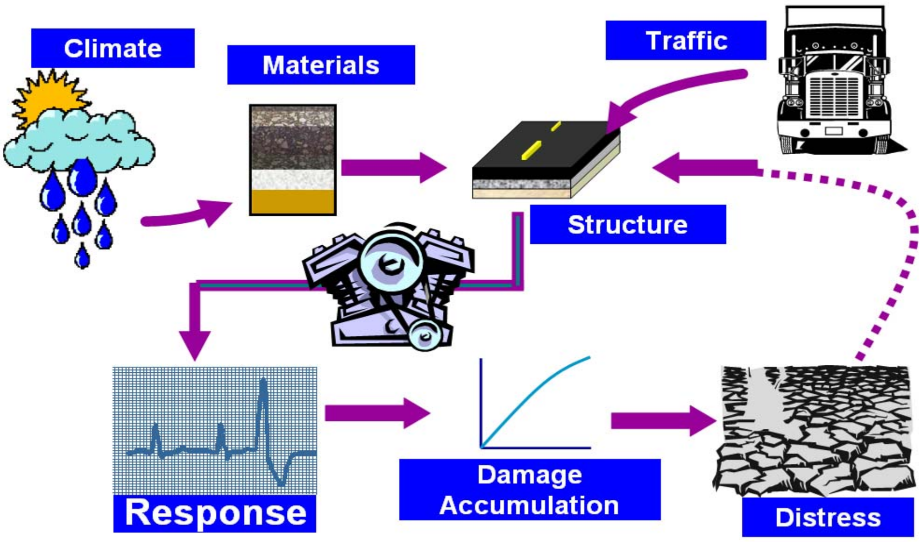

Figure 2-1. MEPDG Design Procedure (NCHRP 1-37A 2004)

2.2 MEPDG INPUTS

The MEPDG represents a challenging innovation in the way that pavement design is performed; design inputs include traffic (full load spectra for various axle configurations), material characterization, climatic factors, performance criteria, and many other factors. One of the most interesting aspects of the design procedure is its hierarchical approach; that is, the consideration of different levels of inputs. Level 1 requires the engineer to obtain the most accurate design inputs (e.g., direct testing of materials, on-site traffic load data, etc.). Level 2 requires testing, but the use of correlations is allowed (e.g., subgrade modulus estimated through correlation with another test). Level 3 generally uses estimated values. Thus, Level 1 has the least possible error associated with inputs, Level 2 uses estimated values or correlations, and Level 3 is based on the default values. This hierarchical approach enables the designer to select the design input depending on the degree of significance of the project and the availability of resources. The three levels of inputs are described as follows (NCHRP 1-37A 2004):

- Level 1 input provides the highest level of accuracy and, accordingly, would have the lowest level of uncertainty or error. Level 1 design generally requires project-specific input such as material input measured by laboratory or field testing, site-specific axle load spectra data, or nondestructive deflection testing. Because these types of inputs require additional time and resources to obtain, Level 1 inputs are generally used for research, forensic studies, or projects in which a low probability of failure is important.
- Level 2 input supplies an intermediate level of accuracy that is closest to the typical procedures used with earlier editions of the AASHTO guide. Level 2 input would most likely be user-selected from an agency database, derived from a limited testing program, or estimated through correlations. Examples of input include estimations of asphalt concrete dynamic modulus from binder, aggregate, and mix properties; estimations of Portland cement concrete elastic moduli from compressive strength tests; or use of site-specific traffic volume and traffic classification data in conjunction with agency-specific axle load spectra. Level 2 input is most applicable for routine projects with no special degree of significance.
- Level 3 input affords the lowest level of accuracy. This level might be used for designs where the consequences of early failure are minimal, as with lower volume roads. Inputs typically would be user-selected values or typical averages for the region. Examples include default unbound materials, resilient modulus values, or the default Portland cement concrete coefficient of thermal expansion for a given mix class and aggregates used by an agency.

2.2.1 Climatic Inputs

In the 1993 AASHTO design guide, the climatic variables were handled with seasonal adjustments and application of drainage coefficients. In the MEPDG, however, temperature changes and moisture profiles in the pavement structure and subgrade over the design life of a pavement are fully considered by using a sophisticated climatic modeling tool called the Enhanced Integrated Climatic Model (EICM). The EICM model simulates changes in behavior and characteristics of pavement and subgrade materials, in conjunction with climatic conditions, over the design life of the pavement. To use this

model, a relatively large number of input parameters are needed as follows (NCHRP 1-37A 2004):

- General information
- Weather-related information
- Groundwater table depth
- Drainage and surface properties
- Pavement structure materials

2.2.2 Traffic Inputs

For traffic analysis, the inputs for the MEPDG are much more complicated than are those required by the 1993 AASHTO design guide. In the 1993 design guide, the primary traffic-related input was the total design 80 kN equivalent single axle loads (ESALs) expected over the design life of the pavement. In contrast, the more sophisticated traffic analysis in the MEPDG uses axle load spectral data. The following traffic related input is required for the MEPDG (NCHRP 1-37A 2004):

- Base year truck-traffic volume (the year used as the basis for design computation)
- Vehicle (truck) operational speed
- Truck-traffic directional and lane distribution factors
- Vehicle (truck) class distribution
- Axle load distribution factors
- Axle and wheel base configurations
- Tire characteristics and inflation pressure
- Truck lateral distribution factors
- Truck growth factors

2.2.3 Material Inputs

There are a number of material inputs for the design procedure and various types of test protocols to measure material properties. Table 2-1 summarizes different types of materials involved in the MEPDG, and Table 2-2 shows the material properties of the hot-mix asphalt (HMA) layer and test protocols to characterize the HMA materials.

Table 2-1. Major Material Types for the MEPDG (AASHTO 2008)

<u>Asphalt Materials</u>
• Stone Matrix Asphalt (SMA)
• Hot Mix Asphalt (HMA)
    o Dense Graded
    o Open Graded Asphalt
    o Asphalt Stabilized Base Mixes
    o Sand Asphalt Mixtures
• Cold Mix Asphalt
    o Central Plant Processed
    o In-Place Recycled
 	
<u>PCC Materials</u>
• Intact Slabs – PCC
    o High Strength Mixes
    o Lean Concrete Mixes
• Fractured Slabs
    o Crack/Seat
    o Break/Seat
    o Rubblized
 	
<u>Chemically Stabilized Materials</u>
• Cement Stabilized Aggregate
• Soil Cement
• Lime Cement Fly Ash
• Lime Fly Ash
• Lime Stabilized Soils
• Open graded Cement Stabilized Aggregate
	<u>Non-Stabilized Granular Base/Subbase</u>
	• Granular Base/Subbase
	• Sandy Subbase
	• Cold Recycled Asphalt (used as aggregate)
	    o RAP (includes millings)
	    o Pulverized In-Place
	• Cold Recycled Asphalt Pavement (HMA plus aggregate base/subbase)
	
	<u>Sub-grade Soils</u>
	• Gravelly Soils (A-1;A-2)
	• Sandy Soils
	    o Loose Sands (A-3)
	    o Dense Sands (A-3)
	    o Silty Sands (A-2-4;A-2-5)
	    o Clayey Sands (A-2-6; A-2-7)
	• Silty Soils (A-4;A-5)
	• Clayey Soils, Low Plasticity Clays (A-6)
	    o Dry-Hard
	    o Moist Stiff
	    o Wet/Sat-Soft
	• Clayey Soils, High Plasticity Clays (A-7)
	    o Dry-Hard
	    o Moist Stiff
	    o Wet/Sat-Soft
	
	<u>Bedrock</u>
	• Solid, Massive and Continuous
	• Highly Fractured, Weathered

Table 2-2. Asphalt Materials and Their Test Protocols (AASHTO 2008)

Design Type	Measured Property	Source of Data		Recommended Test Protocol and/or Data Source
		Test	Estimate	
New HMA (new pavement and overlay mixtures), as built properties prior to opening to truck traffic	Dynamic modulus	X		AASHTO TP 62
	Tensile strength	X		AASHTO T 322
	Creep Compliance	X		AASHTO T 322
	Poisson's ratio		X	National test protocol unavailable. Select MEPDG default relationship
	Surface shortwave absorptivity		X	National test protocol unavailable. Use MEPDG default value.
	Thermal conductivity	X		ASTM E 1952
	Heat capacity	X		ASTM D 2766
	Coefficient of thermal contraction		X	National test protocol unavailable. Use MEPDG default values.
	Effective asphalt content by volume	X		AASHTO T 308
	Air voids	X		AASHTO T 166
	Aggregate specific gravity	X		AASHTO T 84 and T 85
	Gradation	X		AASHTO T 27
	Unit Weight	X		AASHTO T 166
	Voids filled with asphalt (VFA)	X		AASHTO T 209
Existing HMA mixtures, in-place properties at time of pavement evaluation	FWD backcalculated layer modulus	X		AASHTO T 256 and ASTM D 5858
	Poisson's ratio		X	National test protocol unavailable. Use MEPDG default values.
	Unit Weight	X		AASHTO T 166 (cores)
	Asphalt content	X		AASHTO T 164 (cores)
	Gradation	X		AASHTO T 27 (cores or blocks)
	Air voids	X		AASHTO T 209 (cores)
	Asphalt recovery	X		AASHTO T 164/T 170/T 319 (cores)
Asphalt (new, overlay, and existing mixtures)	Asphalt Performance Grade (PG), OR	X		AASHTO T 315
	Asphalt binder complex shear modulus (G^*) and phase angle (ϕ), OR	X		AASHTO T 49
	Penetration, OR	X		AASHTO T 53
	Ring and Ball Softening Point Absolute Viscosity Kinematic Viscosity Specific Gravity, OR	X		AASHTO T 202 AASHTO T 201 AASHTO T 228
	Brookfield Viscosity	X		AASHTO T 316

Note: The global calibration factors included in version 1.0 of the MEPDG software for HMA pavements were determined using the NCHRP 1-37A viscosity based predictive model for dynamic modulus.

2.3 MEPDG IMPLEMENTATION EFFORTS

Table 2-3 summarizes some of the MEPDG implementation efforts attempted by several state DOTs. As is evident from the table, most implementation studies were based on the development of a layer modulus database for local pavement materials and mixtures as a first step. Sensitivity or parametric analyses of design input variables related to local pavement performance were also pursued. Sensitivity analysis can identify how each design input parameter affects pavement performance.

Table 2-3. Summary of Implementation Efforts Pursued by Several State DOTs

Literature	Research Purpose	Significant Findings
Williams (2007)	- Evaluation of 21 HMA mixtures - Development of pavement structures using the MEPDG	- Most of the predictive models of version 0.8 need further refinement.
Witczak and Bari (2004)	- Development of database of dynamic modulus for lime modified asphalt mixtures	- Higher dynamic modulus from lime modified HMA mixtures than unmodified mixtures - Recommendation of testing protocol-
Khazanovich et al. (2006)	-Development of level 1 and Level 2 inputs	- Significant effect of thickness and stiffness of the AC and base layers on the predicted subgrade moduli
Coree et al. (2005)	- Investigation of sensitivity of input parameters to performance prediction	- Categorized the inputs for all distresses as highly significant and significant and not significant - Identified critical factors affecting predicted pavement performance from the MEPDG
Schwartz (2007) Kesiraju et al. (2007) Velasquez et al. (2009) Fernando et al. (2007) Ali (2005)	- Investigation of sensitivity of input parameters to performance prediction	- Identified critical factors affecting predicted pavement performance from the MEPDG
Daniel and Chehab. (2008)	- Investigation of sensitivity of predicted performance to assumed PG grade using level 1, 2, and 3	- Level 1 analysis is least conservative for the structure and mixtures
McCracken et al. (2008)	- Investigation of impact of using different input level on Pavement design	- Using different hierarchal levels for the critical inputs can have an effect on the design thickness

Flintsch et al. (2007, 2008) evaluated HMA characteristics based on the testing procedure established by the MEPDG to support its practical implementation in Virginia. They examined the dynamic modulus, creep compliance, and tensile strength of eleven HMA mixtures produced with PG 64-22 binder from different plants across Virginia. Test results indicated that Level 1 design inputs are necessary for HMA pavement projects with high significance, whereas Level 2 design could be used for design of pavements where low or medium traffic volumes are expected. The predicted HMA moduli obtained from the Level 2 approach were relatively close to the Level 1 measured values as shown in Figure 2-2. A ratio of the predicted to measured dynamic modulus values varied between 0.5 and 0.9.

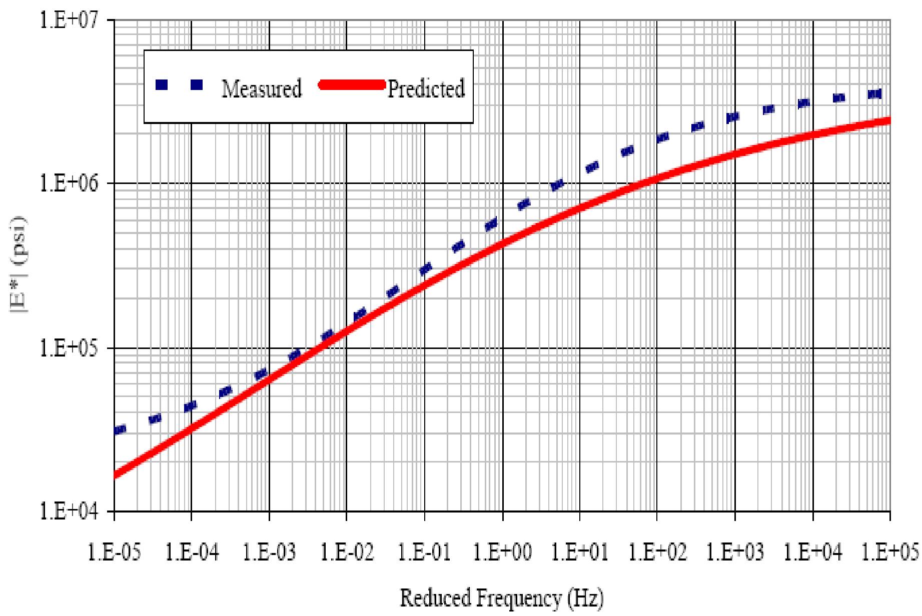

Figure 2-2. Measured vs. Predicted Dynamic Modulus Curves (Flintsch et al. 2008)

In 2005, Kim et al. conducted an experimental study on the dynamic modulus testing of typical North Carolina HMA mixtures in two different testing modes: uniaxial compression and indirect tension (IDT). The study included 42 HMA mixtures with varying aggregate sources, aggregate gradations, asphalt sources, asphalt grades, and asphalt contents. They found that the binder variables (i.e., the source, performance grade, and content) have a much more significant effect on the dynamic modulus than do

the aggregate variables (i.e., source and gradation). They also compared the dynamic modulus database (Level 1) developed from the uniaxial compression testing mode to predicted values by using two dynamic modulus predictive models: Witczak's equation (Level 2 implemented in the MEPDG) and another phenomenological model, the Hirsch model. Figure 2-3 illustrates a relatively good prediction using Witczak's model in the (a) and (b) graphs, whereas the (c) and (d) graphs show a mixture with a relatively poor prediction. It appeared that Witczak's prediction at cooler temperatures is more accurate than at warmer temperatures. The Hirsch model, as shown in Figure 2-3(b), performed very poorly at 10°C and approximately the same as Witczak's model at the remaining temperatures. The poorer prediction of the Hirsch model at 10°C could be due to the fact that the binder data at this temperature were extrapolated.

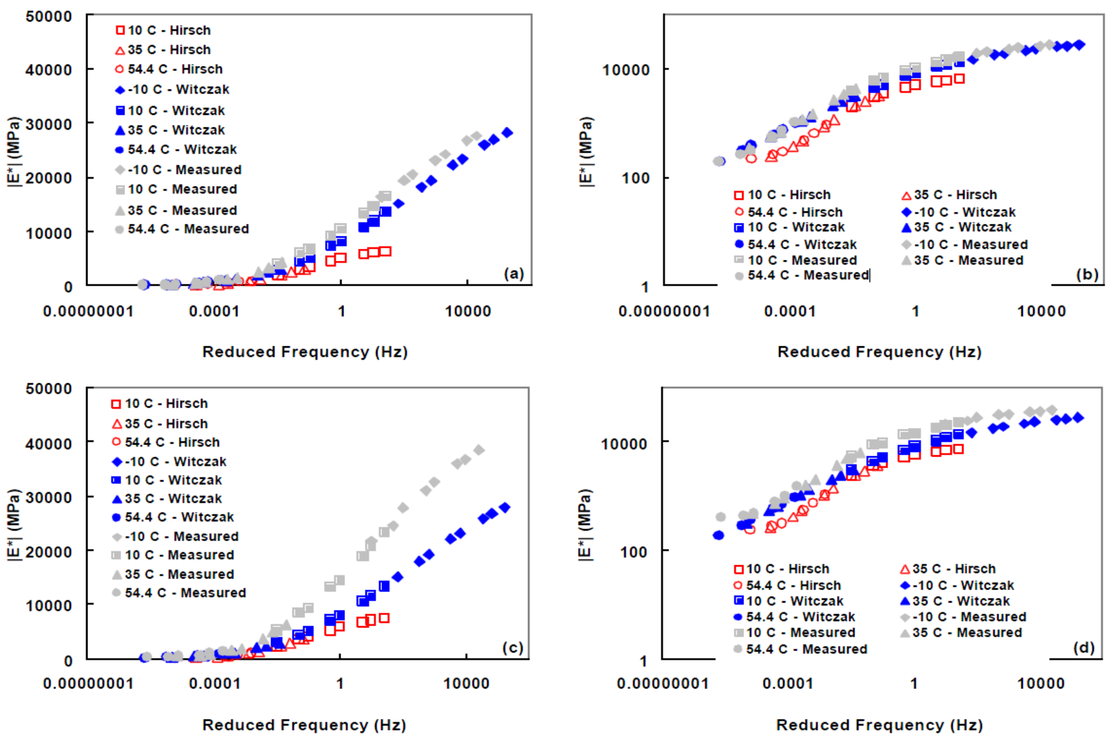

Figure 2-3. Measured Moduli Compared to Predicted Moduli (Kim et al. 2005)

Tashman et al. (2007) developed a database of dynamic modulus values of typical Superpave HMA mixes that are widely used in the State of Washington. The database was used to investigate the sensitivity of the dynamic modulus to HMA mix properties.

They compared performance predictions by the MEPDG with field performance data and reported that the MEPDG over-predicted the longitudinal cracking compared to field performance data, and level 3 analysis predicted distresses higher than Level 1 distresses.

Richardson et al. (2009) evaluated the resilient moduli for common Missouri subgrade soils and typical unbound granular base materials. Their testing program included 27 common subgrade soils and 5 unbound granular base materials. The tests were performed at their optimum water content and at elevated water content. They concluded that the material source and fines content were highly significant for the level of attained resilient modulus.

A similar study was conducted by Nazzal et al. (2008) to develop a database of resilient modulus values of subgrade soils commonly used in Louisiana at different moisture content levels. They also developed resilient modulus prediction models for Louisiana subgrade soils and found a good agreement between the measured resilient modulus coefficient values and those predicted using the developed regression models. They reported a significant difference between the measured resilient modulus values of A-4 and A-6 soils and those recommended by the MEPDG.

As mentioned earlier, sensitivity analysis of design input parameters can be used to identify important input parameters that significantly affect pavement performance among the entire design inputs. Therefore, sensitivity analysis of design input parameters is considered an important task that should be performed before implementing the new design guide into actual practice. This is because the analysis results can provide useful and relevant information for pavement design engineers in determining their appropriate level of effort for each design input to be specified.

Hoerner et al. (2007) selected inputs associated with five typical types of South Dakota asphaltic pavements for sensitivity analyses. A total of 56 MEPDG simulations for new asphalt pavement design were conducted with two representative climatic conditions. They ranked design inputs in an order of their significance to the pavement performance.

Table 2-4 presents sensitivity analysis results demonstrating design input parameters that are most significantly related to each performance indicator (i.e., longitudinal cracking, alligator cracking, and total rutting).

Table 2-4. Summary of Sensitivity Analysis Results (Hoerner et al. 2007)

Input Parameter/Predictor	Rankings for Individual Performance Indicators			Overall Order of Significance
	Longitudinal Cracking	Alligator Cracking	Total Rutting	
Average annual daily truck traffic	2	1	1	1
AC layer thickness	1	3	2	2
AC binder grade	4	2	5	3
Base resilient modulus	3	4	6	4
Subgrade resilient modulus	9	6	3	5
Traffic growth rate	6	5	8	6
Base layer thickness	5	8	10	7
Climate location	10	7	7	8
Tire Pressure	7	9	9	9
Depth of water table	12	14	4	10
Vehicle class distribution	8	10	13	11
AC mix gradation	11	11	12	12
AC creep compliance	13	12	14	13
Base plasticity index	15	15	11	14
Coef. of thermal contraction	14	13	15	15
Subgrade type	16	16	16	16
Truck hourly distribution factors	17	17	17	17

* Note: shaded cells indicate those variables that were found to be insignificant

CHAPTER 3
MATERIALS AND TESTING FACILITY

This chapter presents the local materials and mixtures selected for this research. A total of twenty hot-mix asphalt (HMA) mixtures paved during year 2008 and 2009 were collected from asphalt field projects, and three unbound soils (loess, loess/till, and sandy silt) typically used for roadway foundations in Nebraska pavements were obtained to characterize their physical properties and resilient moduli. In addition to the testing of the three unbound soils, nine stabilized soils (loess, till, and shale stabilized with hydrated lime, fly ash and cement kiln dust) that were tested by Hensley et al. (2007) for a previous NDOR research project were also analyzed for their resilient modulus characteristics.

One of the major milestones planned for this research was to develop a mechanical testing system to perform various modulus (stiffness) tests of different paving materials. The UNL research team has installed and used the UTM-25kN (Universal Testing Machine with a 25kN load cell) mechanical testing station and related devices in the UNL geomaterials laboratory for various mechanical tests of asphalt mixtures. The current UTM-25kN mechanical testing-analysis facility has been used for this study, but some improvements were necessary, such as an installation of a triaxial cell with associated measuring devices to evaluate stress-dependent modulus characteristics of soils.

3.1 HMA MIXTURES

Based on the literature reviews and discussions with NDOR Technical Advisory Committee (TAC) members, two major issues were considered for the testing of asphalt mixtures: 1) the number of mixture types; and 2) combination of materials of each mixture type. In this research, a total of twenty HMA mixtures from field projects were collected for two years: 2008 to 2009. Figure 3-1 shows the locations where each HMA mixture was collected. As seen in the figure, five different types of HMA mixtures (i.e.,

HRB, SPL, SP4(0.375), SP4(0.5), and SP5) among eleven existing HMA mixture types (SPS, SPL, SP1 to SP6, SP4 Special, RLC, and LC) were the focus of this study, since they are primary types often used for Nebraska asphalt pavements. For each type of mixture, four field projects were collected, which resulted in a total of twenty HMA mixtures.

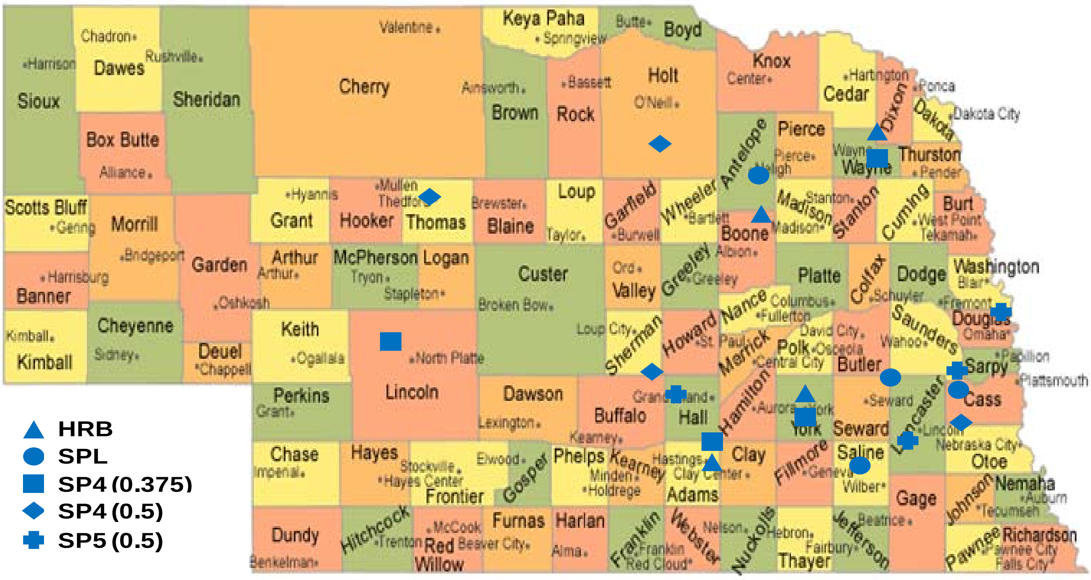

Figure 3-1. Project Locations of Collected HMA Mixtures

Table 3-1 summarizes mixture information such as project identification, contractor, binder grade and source of each mixture, and construction year. Table 3-2 summarizes the aggregate gradation of each mixture. The gradation values are used as crucial information to conduct MEPDG analysis such as predicting dynamic modulus characteristics of HMA mixtures for Level 2 or Level 3 pavement design.

Each HMA mixture was tested in the uniaxial compressive cyclic loading mode for the dynamic modulus (AASHTO TP62) and in the indirect tensile (IDT) mode for creep compliance at low temperatures (AASHTO T322).

Table 3-1. Summary of Mixture Information

Mix Type	Project Identification	Contractor	Binder Grade	Asphalt Source	Construction Year
HRB	RD 9-4(1012)	Werner Construction	PG 58-28	FLINT HILLS	2008
	RD 81-2(1037)	Paulsen Inc.	PG 58-28	FLINT HILLS	2008
	STP 14-4(110)	Knife River Midwest	PG 58-28	JEBRO	2008
	NH 6-4(125)	VONTZ Paving	PG 58-34	FLINT HILLS	2009
SPL	STPD 6-6(156)	Constructors Inc.	PG 58-28	FLINT HILLS	2008
	STPD 79-2(102)	Dobson Brothers	PG 58-28	FLINT HILLS	2008
	STP 91-3(107)	Paulsen Inc.	PG 58-34	FLINT HILLS	2009
	NH 80-9(832)	Constructors Inc.	PG 64-28	MONARCH	2009
SP4 (0.375)	RD 81-2(1037)	Paulsen Inc.	PG 64-28	FLINT HILLS	2008
	RD 9-4(1012)	Werner Construction	PG 64-28	FLINT HILLS	2008
	NH 6-4(125)	VONTZ PAVING	PG 64-28	FLINT HILLS	2009
	RD 25-2(1014)	Paulsen Inc.	PG 64-28	JEBRO	2009
SP4(0.5)	PEP 183-1(1020)	Paulsen Inc.	PG 64-28	MONARCH	2008
	STPD-NFF 11-2 (115)	Werner Construction	PG 64-28	SEM	2008
	NH 281-4(119)	CHAMBERS JCT. NORTH	PG 64-28	JEBRO	2009
	NH 83-3(107)	Werner Construction	PG 64-28	JEBRO	2009
SP5	RD 75-2(1055)	U.S. ASPHALT	PG 70-28	FLINT HILLS	2008
	STPD 6-7(178)	Constructors Inc.	PG 64-28	JEBRO	2008
	RD 77-2(1057)	PAVERS COMPANIES	PG 70-28	FLINT HILLS	2008
	IM 80-6(97)	VONTZ PAVING	PG 70-28	FLINT HILLS	2009

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Table 3-2. Summary of Aggregate Gradation of Each Mixture

Mix Type	Project Number	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#200
HRB	RD 9-4 (1012)	100.0	93.7	91.5	82.5	62.0	42.0	32.6	19.9	6.0
	RD 81-2 (1037)	100.0	98.3	96.2	85.4	60.8	40.5	27.5	18.5	7.7
	STP 14-4 (110)	100.0	99.5	95.5	88.2	59.4	39.8	27.8	16.9	5.9
	NH 6-4 (125)	99.2	96.0	91.6	77.8	52.2	38.2	23.0	17.0	5.8
SPL	STPD 6-6 (156)	98.9	92.1	86.4	72.9	47.5	32.6	23.8	15.9	7.5
	STPD 79-2 (102)	100.0	90.0	81.5	69.2	49.4	33.3	22.3	14.4	6.9
	STP 91-3 (107)	100.0	88.9	83.4	71.8	52.2	35.5	25.2	15.9	5.5
	NH 80-9 (832)	98.5	91.9	85.6	76.9	54.5	43.4	30.6	18.9	7.7
SP4 (0.375)	RD 81-2 (1037)	99.9	98.8	96.5	82.9	53.1	34.1	22.4	15.2	6.9
	RD 9-4 (1012)	100.0	97.8	95.3	84.1	67.4	46.9	31.4	18.2	4.6
	NH 6-4 (125)	100.0	99.6	96.4	87.2	56.7	39.3	23.3	15.8	5.4
	RD 25-2 (1014)	100.0	99.4	98.3	87.1	62.2	42.5	29.3	19.1	7.7
SP4 (0.5)	PEP 183-1 (1020)	100	92.9	88.9	75.1	47	28.8	18.4	11.8	4.4
	STPD-NFF 11-2 (115)	99.6	93.4	87.7	69.4	45.2	30.2	20.5	12.3	5.5
	NH 281-4 (119)	99.8	96.3	90.7	83	57.2	35	23.3	14.8	5.7
	NH 83-3 (107)	100.0	94.8	91.1	69.1	41.5	25.6	17.0	10.4	5.0
SP5	RD 75-2 (1055)	100.0	94.0	89.5	75.9	50.8	34.6	23.5	14.8	6.1
	STPD-6-7 (178)	99.0	89.9	89.9	79.6	54.4	36.2	25.2	15.9	6.8
	RD-77-2 (1057)	100.0	99.1	93.8	77.7	54.2	35.1	22.0	10.5	3.8
	IM 80-6 (97)	100.0	97.0	91.2	80.5	55.8	37.4	23.2	14.5	5.4

3.2 SUBGRADE SOILS

Three different native soils (loess, loess/till, and sandy silt) presented in Figure 3-2 were collected and tested to evaluate their comprehensive physical properties and resilient modulus characteristics. Based on discussions with NDOR TAC members, the three soils are considered representative subgrade materials often used in Nebraska pavements. In order to characterize physical properties of the soils, various laboratory tests were performed, including the specific gravity test (AASHTO T100), Atterberg limit tests (AASHTO T89, T90), sieve analysis (AASHTO T88), and hydrometer analysis (ASTM D422). For mechanical characterization of the soils, the resilient modulus test designated in AASHTO T307 was performed with soil specimens that were compacted at the maximum dry unit weight with an optimum moisture content which was pre-determined from a standard proctor test (AASHTO T99).

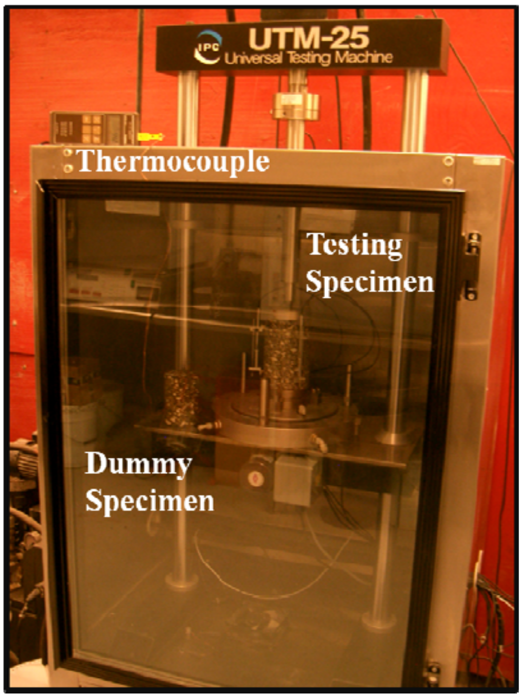
Sandy Silt (SS) **Loess (L)** **Loess/Till (LT)**

Figure 3-2. Three Native Soils Selected for This Research

In addition to the comprehensive testing of the three unbound native soils, nine stabilized soils (loess, till, and shale stabilized with hydrated lime, fly ash and cement kiln dust, respectively) that were studied by Hensley et al. (2007) for a previous NDOR research project were also analyzed for their resilient modulus characteristics. This effort was attempted to provide a more general and comprehensive resilient modulus database of the subgrade soils that are often stabilized with cementing agents in various pavement projects. Hensley et al. (2007) reported resilient modulus test results of the nine soils that were compacted with an optimum amount of different types of pozzolans.

3.3 TESTING FACILITY

All three layer modulus tests (i.e., the dynamic modulus test and creep compliance test for HMA mixtures and the resilient modulus test for soils) were conducted using UTM-25kN mechanical test station. This equipment is capable of applying loads up to 25 kN static or 20 kN dynamic over a wide range of loading frequencies. An environmental chamber is incorporated with the loading frame, as presented in Figure 3-3, to control testing temperatures. The chamber can control temperatures ranging from 5 ºF to 140 ºF. Better achievement of the target testing temperatures of specimens was obtained by using a dummy specimen with a thermocouple embedded in the middle of the specimen, as presented in the figure. Figure 3-3 also presents other key features and specifications of the UTM-25kN test station.



Specifications

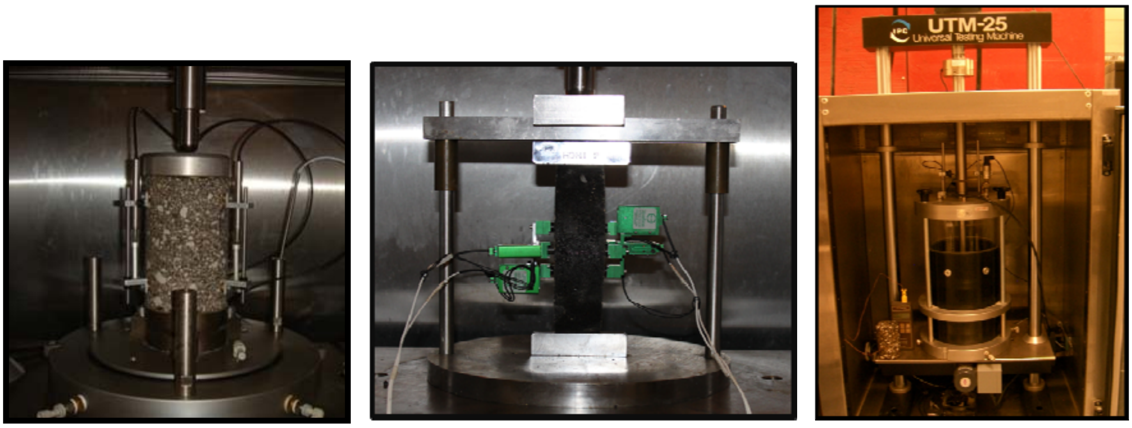
Load Frame	
Size:	185(H) x 58(D) x 60(W) cm
Weight:	130kg
Load Capacity:	25kN static, 20kN dynamic
Between columns:	45cm
Vertical space:	80cm
Stroke:	50mm

Hydraulic Power Supply	
Size:	81(H) x 40(D) x 70(W) cm
Weight:	75kg (excluding oil)
Flow rate	5 litres/min
High pressure:	160 Bar
Low pressure:	2 to 160 Bar (adjustable)
Mains power	208V / 230V, 50 or 60Hz; 2.6kW
Noise level:	less than 70db at 2m

Figure 3-3. UTM-25kN Mechanical Test Station and Its Key Specifications

Figure 3-4(a) presents a cylindrical specimen (100 mm in diameter and 150 mm high) with three linear variable differential transducers (LVDTs) attached on the surface to measure vertical linear deformations in the uniaxial compressive cyclic loading mode for

the dynamic modulus test of HMA mixtures. In order to conduct the creep compliance test of HMA mixtures at low temperature, two cross extensometers were attached to both faces of the indirect tensile specimen as shown in Figure 3-4(b). In order to perform the resilient modulus test of soil specimens, a universal triaxial cell with associated measuring devices was developed so as to evaluate stiffness characteristics of subgrade soils that are stress-dependent. Figure 3-4(c) presents the triaxial testing system.



(a) (b) (c)

Figure 3-4. Testing Specimens with Associated Measuring Devices Installed

CHAPTER 4

LABORATORY TESTS AND RESULTS

This chapter describes laboratory tests conducted for this study and presents their results. Determination of layer stiffness characteristics of HMA mixtures for each MEPDG design level requires various tests of asphalt binder and HMA mixture as summarized in Table 4-1. Similarly, Table 4-2 presents soil laboratory tests necessary to perform each level of MEPDG design. As previously mentioned, the triaxial resilient modulus test was conducted for Level 1, whereas basic physical properties of soils such as specific gravity, Atterberg limits, and gradations were identified for Level 2 or 3 inputs. Test results obtained from individual asphalt mixtures and soil samples were then tabulated in the form of an MEPDG design input database and are presented in the Appendices.

Table 4-1. Various Tests of Asphalt Binder and Mixture for Each Input Level

Material	Parameter	Level 1	Level 2	Level 3
Asphalt Mixture	IE*I Master Curve	Mix Specific	Not Required	Not Required
	IDT-Creep/Strength	Mix Specific	Reduced Testing	Reduced Testing
	Air Voids	Not Required	Mix Design	Specification
Asphalt Binder	G*/Phase Angle	AASHTO MP1 Binder Test	AASHTO MP1 Binder Test	Not Required
	Pen./Vis./PG	Not Required	Mix Design	Not Required
	Type (PG, Vis.)	Not Required	Not Required	Specification
Aggregate	Effective SG.	Not Required	Mix Design	Quarry Specific
	Gradation	Not Required	Mix Design	Specification

Table 4-2. Various Tests of Soils and Unbound Materials for Each Input Level

Parameter	Input Level 1	Input Level 2	Input Level 3
Resilient Modulus	Site/Material Specific	Not Required	Not Required
Gradation	Not Required	Material Specific	Not Required
Hydrometer Analysis	Not Required	Material Specific	Not Required
Atterberg Limits	Not Required	Material Specific	Not Required
M-D Relations	Not Required	Material Specific	Not Required
DCP – Base CBR, R-Value - Soil	Not Required	Material Specific	Not Required
Classification	Not Required	Not Required	Default, Material Specific

4.1 TESTS AND RESULTS OF ASPHALT MATERIALS

4.1.1 Binder tests

As presented in Table 4-1, for Level 1 and Level 2 designs, the MEPDG requires measurements of binder viscoelastic stiffness data (i.e., binder complex shear modulus G^* and binder phase angle ϕ) at several different temperatures. The binder stiffness data obtained at different temperatures are then used to calculate binder viscosity (η), as presented in Equation [4.1]. Using the binder test data, two regression parameters (A and VTS), which represent the temperature susceptibility of asphalt binder, are then found by the curve fitting of Equation [4.2].

$$\eta = \frac{G^*}{10}\left(\frac{1}{\sin\phi}\right)^{4.8628} \quad [4.1]$$

$$\log(\log\eta) = A + VTS\log T_R \quad [4.2]$$

where G^* = asphalt binder complex shear modulus (Pa),

ϕ = asphalt binder phase angle (degree),

η = viscosity of asphalt binder (centi poise),

T_R = temperature (Rankine) at which the viscosity was estimated, and

A and VTS = regression parameters.

Binders were evaluated with a dynamic shear rheometer (DSR) in oscillatory shear loading mode using parallel plate test geometry. The DSR binder testing was performed at three different temperatures (70 ºF, 85 ºF, and 100 ºF). Binder test results and the two corresponding regression parameters (*A* and *VTS*) for each HMA mixture are summarized in Appendix 1. For Level 3 MEPDG analysis, no testing is required for the two parameters. Default values of *A* and *VTS* embedded in the MEPDG software are generated when one specifies the grade (either traditional or Superpave performance) of the binder (NCHRP 1-37A 2004).

4.1.2 Dynamic modulus test (AASHTO TP62)

The dynamic modulus test is a linear viscoelastic test for asphalt concrete. The dynamic modulus is an important input when evaluating pavement performance related to the temperature and speed of traffic loading. The loading level for the testing was carefully adjusted until the specimen deformation was between 50 and 75 microstrain, which was considered to be a level that would not cause nonlinear damage to the specimen, so that the dynamic modulus would represent the intact stiffness of the asphalt concrete.

A Superpave gyratory compactor was used to produce cylindrical samples with a diameter of 150 mm and a height of 170 mm. The samples were then cored and cut to produce cylindrical specimens with a diameter of 100 mm and a height of 150 mm. The target air void of the cored and cut specimens was 4% ± 0.5%. Figure 4-1 demonstrates the specimen production process using the Superpave gyratory compactor, core, and saw machines, and the resulting cylindrical specimen used to conduct the dynamic modulus test.

Figure 4-1. Specimen Production Process for the Dynamic Modulus Testing

Table 4-3 summarizes air voids, bulk specific gravity (G_{mb}), maximum specific gravity (G_{mm}), asphalt content, and compaction temperature of each dynamic modulus testing specimen. As shown in the table, two specimens were tested for each mixture. It should also be noted that the volumetric characteristics presented in the table are used to provide necessary model inputs, such as effective binder content (%), air voids (%), and total unit weight, for MEPDG analysis. The model inputs that are related to the mixture volumetric properties are summarized in Appendix 1.

Table 4-3. Summary of Volumetric Characteristics of Specimens for Dynamic Modulus

Mix Type	Project Number	Specimen Number	Air Void (%)	G_{mb}	Asphalt Content (%)	Compaction Temperature (ºF)
HRB	RD 9-4(1012)	#1	4.18	2.323	5.62	275
		#2	4.26	2.321		
	RD 81-2(1037)	#1	3.90	2.326	5.78	275
		#2	4.01	2.323		
	STP 14-4(110)	#1	3.85	2.322	5.88	280
		#2	3.86	2.322		
	NH 6-4(125)	#1	3.74	2.328	5.56	280
		#2	3.75	2.328		
SPL	STPD 6-6(156)	#1	3.57	2.362	5.02	275
		#2	4.06	2.350		
	STPD 79-2(102)	#1	4.30	2.360	5.15	275
		#2	3.96	2.368		
	STP 91-3(107)	#1	4.31	2.338	5.12	285
		#2	4.37	2.336		
	NH 80-9(832)	#1	4.14	2.352	5.31	280
		#2	4.06	2.354		
SP4 (0.375)	RD 81-2(1037)	#1	3.93	2.334	5.27	293
		#2	3.96	2.334		
	RD 9-4(1012)	#1	3.63	2.322	6.10	293
		#2	4.38	2.304		
	NH 6-4(125)	#1	3.83	2.330	5.71	280
		#2	3.76	2.332		
	RD 25-2(1014)	#1	4.16	2.315	5.86	285

		#2	4.17	2.315		
SP4(0.5)	PEP 183-1(1020)	#1	4.10	2.340	6.27	285
		#2	4.09	2.340		
	STPD-NFF 11-2 (115)	#1	3.60	2.341	5.19	298
		#2	359	2.342		
	NH 281-4(119)	#1	3.90	2.335	5.62	290
		#2	3.94	2.334		
	NH 83-3(107)	#1	4.26	2.324	5.23	275
		#2	4.17	2.326		
SP5	RD 75-2(1055)	#1	4.07	2.348	6.27	278
		#2	3.73	2.357		
	STPD-6-7(178)	#1	3.70	2.351	5.60	278
		#2	4.17	2.339		
	RD-77-2(1057)	#1	4.00	2.365	6.10	280
		#2	4.19	2.361		
	IM 80-6(97)	#1	3.60	2.338	5.58	270
		#2	3.75	2.334		

To measure the axial displacement of the testing specimens, mounting studs were glued to the surface of the specimen so that three linear variable differential transformers (LVDTs) could be installed on the surface of the specimen through the studs at 120^{o} radial intervals with a 100-mm gauge length. Figure 4-2 illustrates the studs affixed to the surface of a specimen. The specimen was then mounted onto the UTM-25kN equipment for testing, as shown in Figure 4-3.

Figure 4-2. Studs Fixing on the Surface of a Cylindrical Specimen

Figure 4-3. A Specimen with LVDTs mounted in UTM-25kN Testing Station

The test was conducted at five temperatures (14, 40, 70, 100, and 130 °F). At each temperature, six frequencies (25, 10, 5, 1, 0.5, and 0.1 Hz) of load were applied to the specimens. The axial forces and vertical deformations were recorded by a data acquisition system and were converted to stresses and strains. Figure 4-4 presents typical test results of axial stresses and strains from the dynamic modulus test.

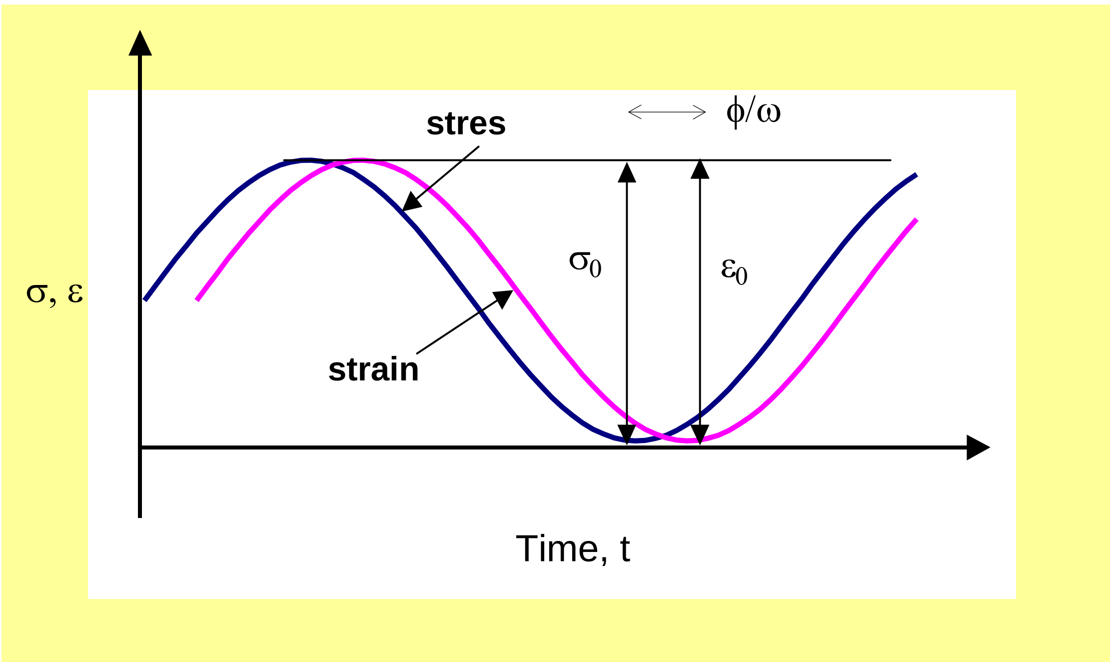


Figure 4-4. Typical Test Results of Dynamic Modulus Test

The dynamic modulus was then obtained by dividing the maximum (peak-to-peak) stress by the recoverable (peak-to-peak) axial strain, as expressed by the following equation:

$$\left|E^*\right| = \frac{\sigma_o}{\varepsilon_o} \qquad [4.3]$$

where $|E^*|$ = dynamic modulus,

σ_o = (peak-to-peak) stress magnitude, and

ε_o = (peak-to-peak) strain magnitude.

As presented in Figure 4-4, viscoelastic materials such as HMA mixtures normally produce a delay between input loading (i.e., repeated stress) and output response (i.e., repeated strain) under cyclic loading conditions. The time delay between two signals is expressed as a phase angle as follows:

$$\phi = \omega \cdot t_d = (2\pi f) \cdot t_d \qquad [4.4]$$

where ϕ = phase angle (degree),

ω = angular frequency (radian/sec.),

f = loading frequency (Hz), and

t_d = time delay between stress and strain.

As mentioned, two replicates were tested and average values of dynamic modulus and phase angle were obtained for each mixture. As an example, Table 4-4 presents the dynamic modulus and phase angle data of two replicates and their averaged values obtained from a SP4(0.5) mixture. The averaged values of dynamic modulus and phase angle at each different testing temperature over the range of loading frequencies are plotted in Figure 4-5 and Figure 4-6, respectively.

As expected, the dynamic modulus increases as the loading frequency increases, while it decreases as the testing temperature increases. For phase angle, it decreases as the frequency increases at temperatures of 10, 40, and 70 ºF. However, the behavior of the phase angle at 100 ºF and 130 ºF seems more complex. Similar results have been

reported in many other studies including that by Flintsch et al. (2008). All twenty mixtures tested in this study showed similar behavior.

Table 4-4. Dynamic Moduli and Phase Angles of SP4(0.5) NH281-4(119) Mixture

Temp. (ºF)	Freq (Hz)	#1		#2		Average	
		$\|E^*\|$ (psi)	ϕ (º)	$\|E^*\|$ (psi)	ϕ (º)	$\|E^*\|$ (psi)	ϕ (º)
14	25	3706833.2	4.3	4158437.9	7.2	3932635.5	5.8
	10	3649624.3	6.2	4029779.4	9.1	3839701.8	7.7
	5	3276894.6	8.6	3768305.8	9.1	3522600.2	8.9
	1	2927421.9	10.3	3319492.8	11.6	3123457.3	11.0
	0.5	2774197.8	9.1	3140589.5	12.2	2957393.6	10.6
	0.1	2681577.9	11.5	3024835.7	13.5	2853206.8	12.5
40	25	2705128.7	8.2	2469577.0	7.2	2587352.8	7.7
	10	2596081.3	14.4	2279307.6	10.6	2437694.5	12.5
	5	2366518.9	17.3	2067985.7	12.5	2217252.3	14.9
	1	1779580.4	21.1	1628127.8	17.3	1703854.1	19.2
	0.5	1537555.3	24.0	1439686.4	19.2	1488620.8	21.6
	0.1	1326416.4	26.4	1246506.8	22.6	1286461.6	24.5
70	25	1081550.8	18.7	1103120.2	17.8	1092335.5	18.2
	10	887793.4	23.4	914184.5	24.6	900989.0	24.0
	5	702660.5	27.4	745089.1	23.3	723874.8	25.3
	1	380178.6	33.1	410632.8	32.4	395405.7	32.8
	0.5	271310.4	35.4	303462.3	32.8	287386.3	34.1
	0.1	192383.6	32.7	216222.3	31.7	204302.9	32.2
100	25	283236.2	39.8	361721.7	27.4	322478.9	33.6
	10	199252.3	30.8	269312.8	23.8	234282.6	27.3
	5	148747.9	34.8	199533.1	28.9	174140.5	31.9
	1	77095.0	35.0	97100.0	35.3	87097.5	35.2
	0.5	64520.3	29.9	82343.5	32.2	73431.9	31.0
	0.1	53189.2	27.4	64971.7	28.3	59080.4	27.8
130	25	83076.2	42.2	84895.4	36.0	83985.8	39.1
	10	60024.0	29.8	65426.9	24.6	62725.5	27.2
	5	50290.8	27.1	53320.8	27.0	51805.8	27.1
	1	36749.1	27.0	39599.0	25.1	38174.1	26.1
	0.5	33430.4	26.4	35626.5	26.8	34528.4	26.6
	0.1	36346.9	25.2	37166.2	23.2	36756.5	24.2

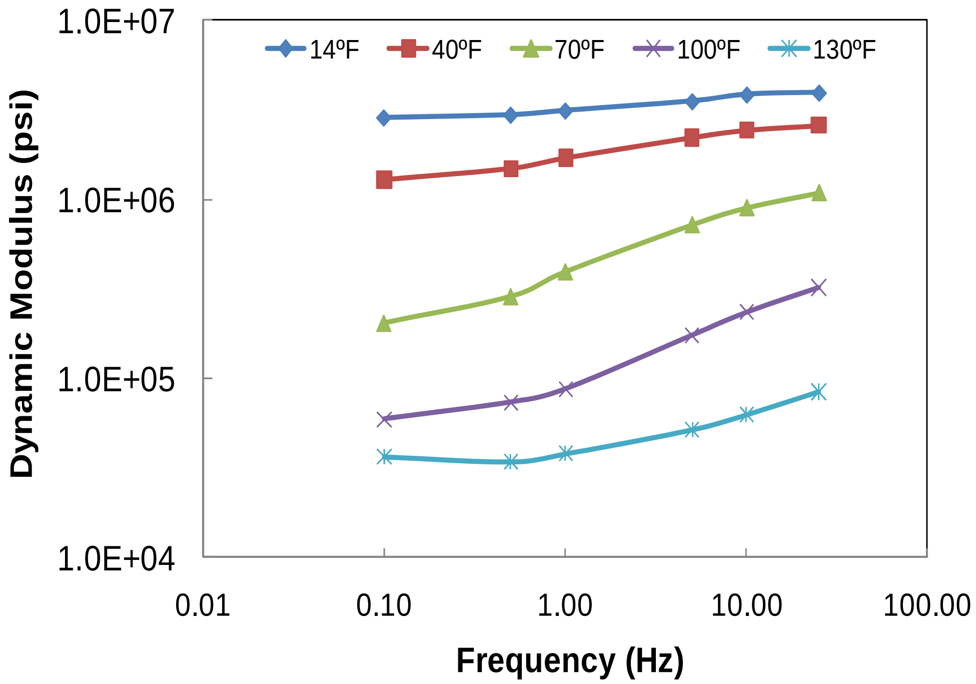


Figure 4-5. Plot of Averaged Dynamic Moduli: SP4(0.5) NH281-4(119) Mixture

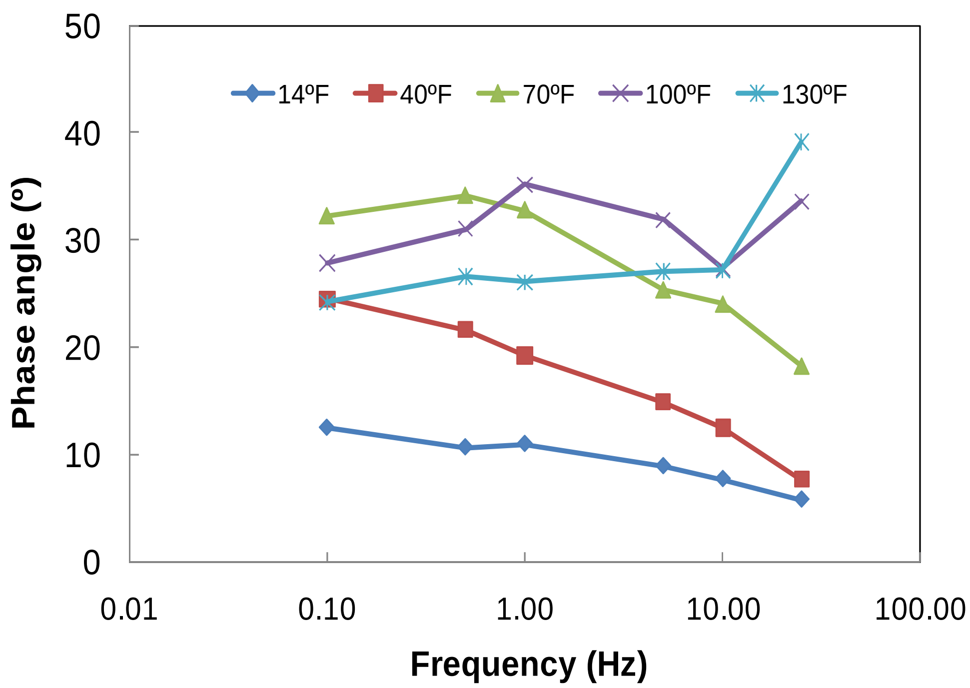
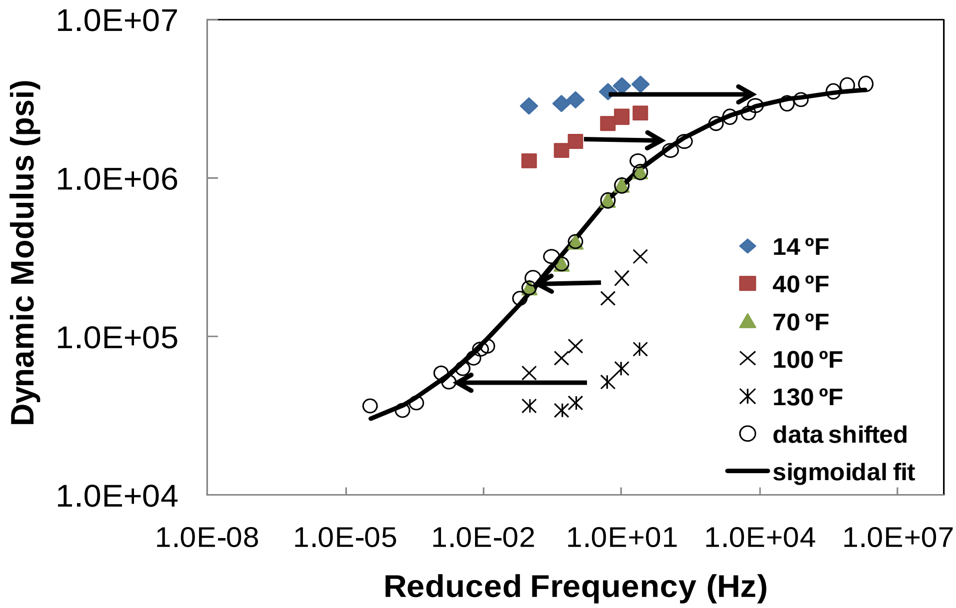


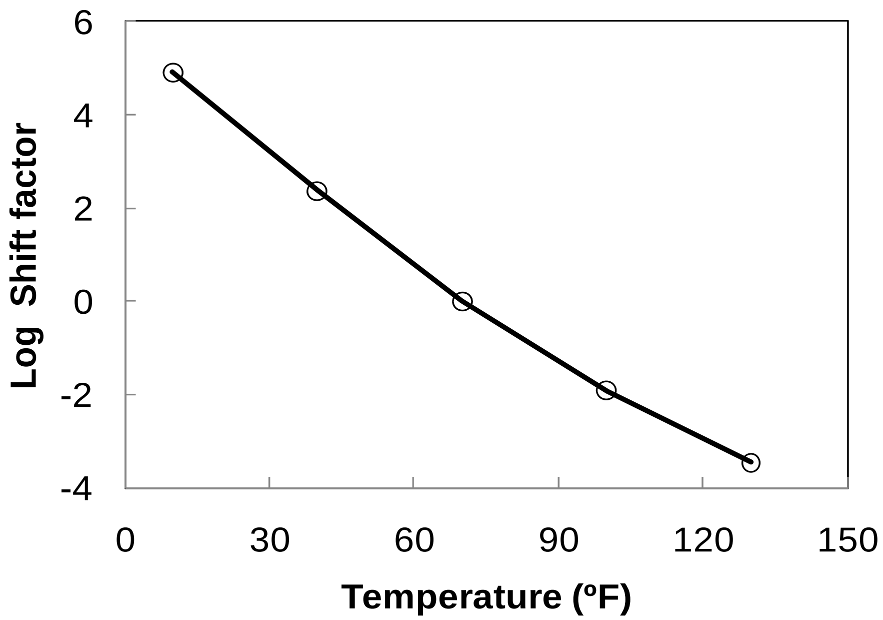
Figure 4-6. Plot of Averaged Phase Angles: SP4(0.5) NH281-4(119) Mixture

MEPDG requires the dynamic moduli for 30 temperature-frequency combinations (i.e., five temperatures and six frequencies) to conduct Level 1 design analysis. Therefore, the dynamic modulus values of the 30 temperature-frequency combinations are presented in Appendix 1.

With the 30 individual dynamic moduli at all levels of temperature and frequency, the MEPDG determines a stiffness master curve constructed at a reference temperature (generally taken as 70 °F). The master curve represents the stiffness of the material in a wide range of loading frequencies (or loading times, equivalently). Master curves are constructed using the principle of time (or frequency) - temperature superposition. The data at various temperatures are shifted with respect to loading frequency until the curves merge into a single smooth function. The master curve of the dynamic modulus as a function of time (or frequency), formed in this manner, describes the time (or loading rate) dependency of the material. The amount of shifting at each temperature required to form the master curve describes the temperature dependency of the material. As an example, Figure 4-7 shows a constructed master curve and its shift factors for a mixture: SP4(0.5) NH281-4(119).



(a) Construction of a Master Curve



(b) Shift Factors

Figure 4-7. Example of Developing a Master Curve and Its Shift Factors

As illustrated in Figure 4-7(a), the modulus master curve can be mathematically modeled by a sigmoidal function (Pellinen and Witczak 2002) described as follows:

$$\log\left|E^*\right| = \delta + \frac{\alpha}{1+e^{\beta-\gamma\log f_r}} \qquad [4.5]$$

where $\log|E^*|$ = log of dynamic modulus,

δ = minimum modulus value,

f_r = reduced frequency,

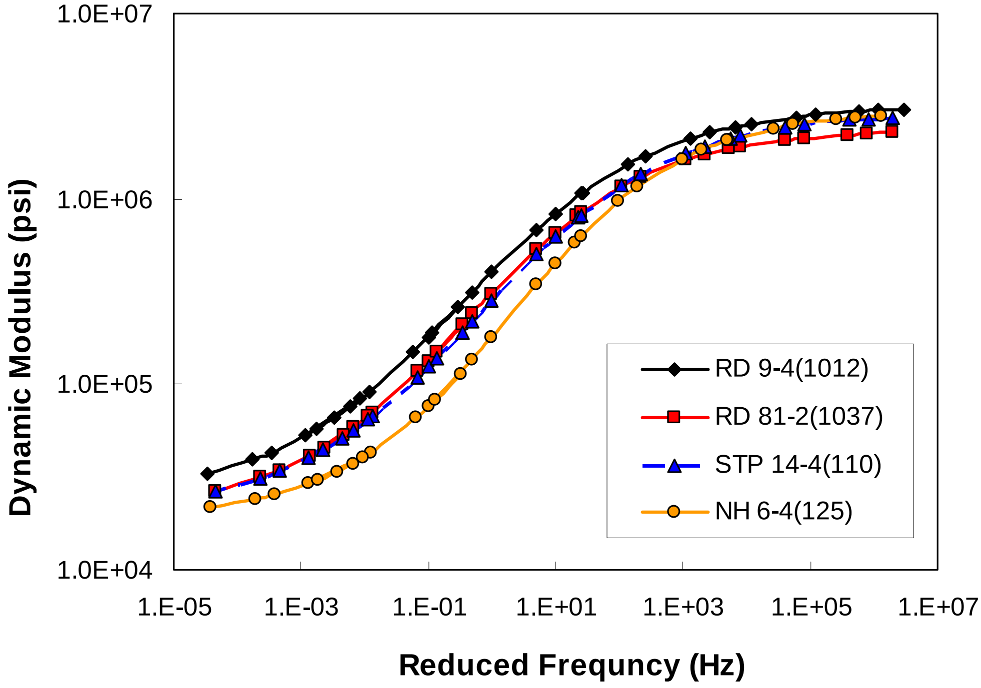
α = span of modulus values, and

β, γ = shape parameters.

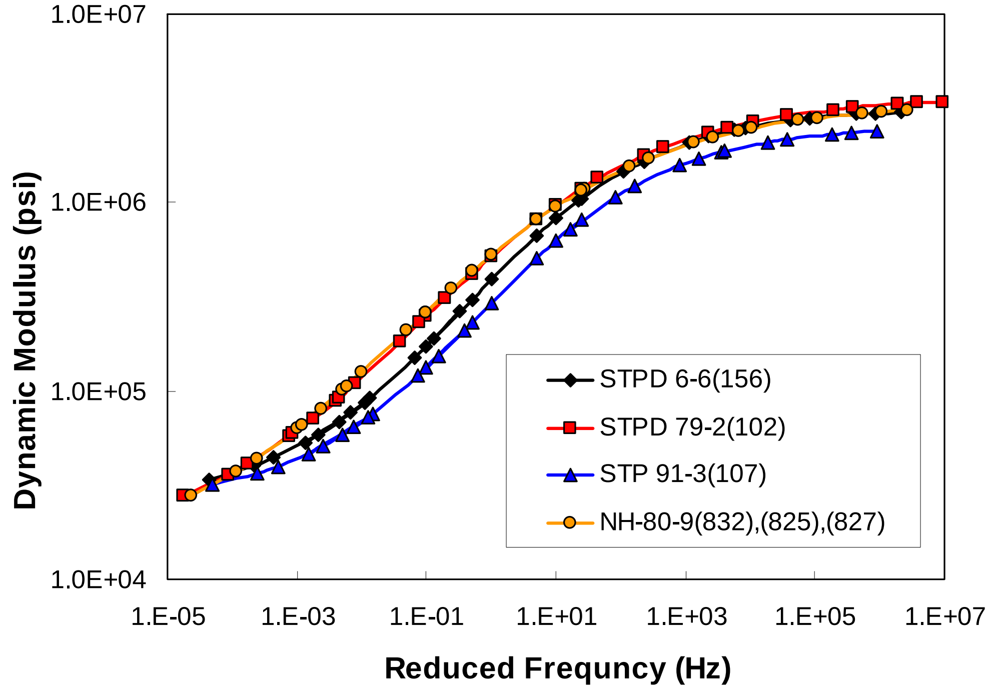
For Level 1 MEPDG analysis, the master curve and sigmoidal function parameters of each mixture were determined using measured dynamic modulus test data as mentioned above. Figures 4-8(a) through 4-8(e) present master curves of all twenty HMA mixtures: four HRB, four SPL, four SP4(0.375), four SP4(0.5), and four SP5, respectively.

Legends in each graph indicate field project identifications as previously shown in Table 3.1. From the figures, variations in dynamic modulus values among mixtures can be observed even though they are the same type of mixtures. This implies that mixture stiffness characteristics are related to properties and proportioning of mixture constituents. Individual mixtures in the same mixture type were produced by blending different mixture components.

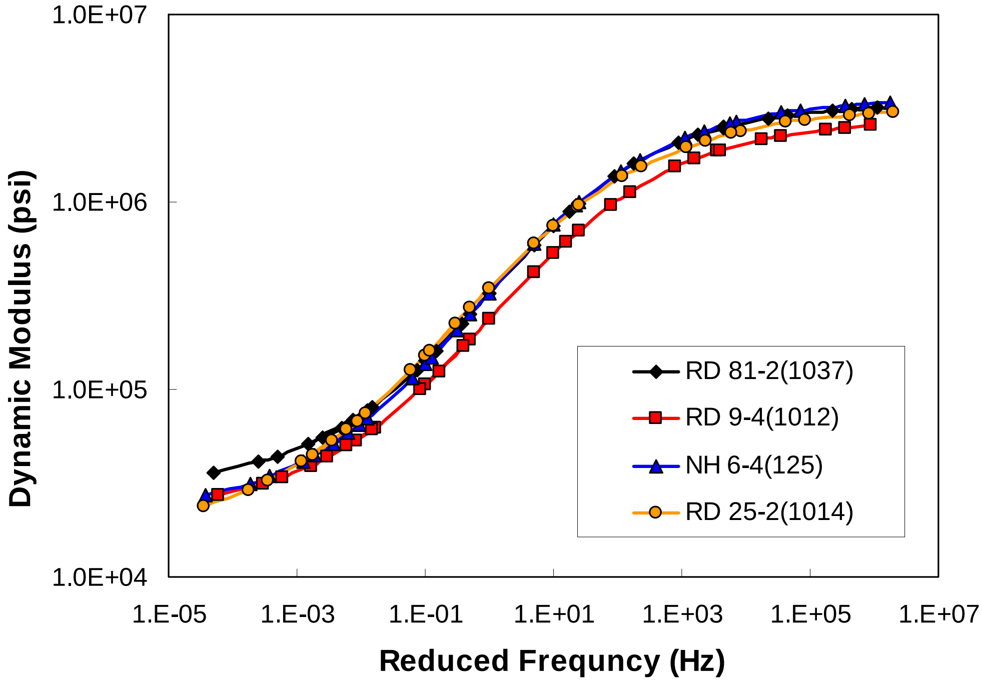
Table 4-5 presents sigmoidal function parameters and shift factors for each mixture. These model parameters and shift factors were utilized to develop master curves of each HMA mixture. Using the values presented in the table, a new master curve at an arbitrary reference temperature can be identified by simply moving the whole master curve in the horizontal direction.



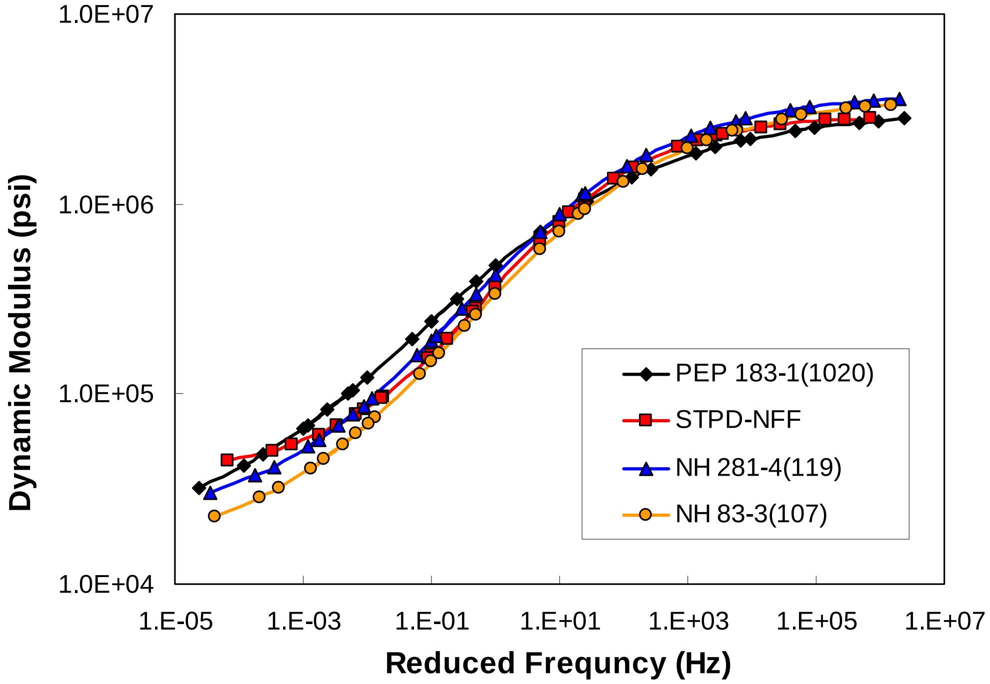
(a) HRB Mixtures



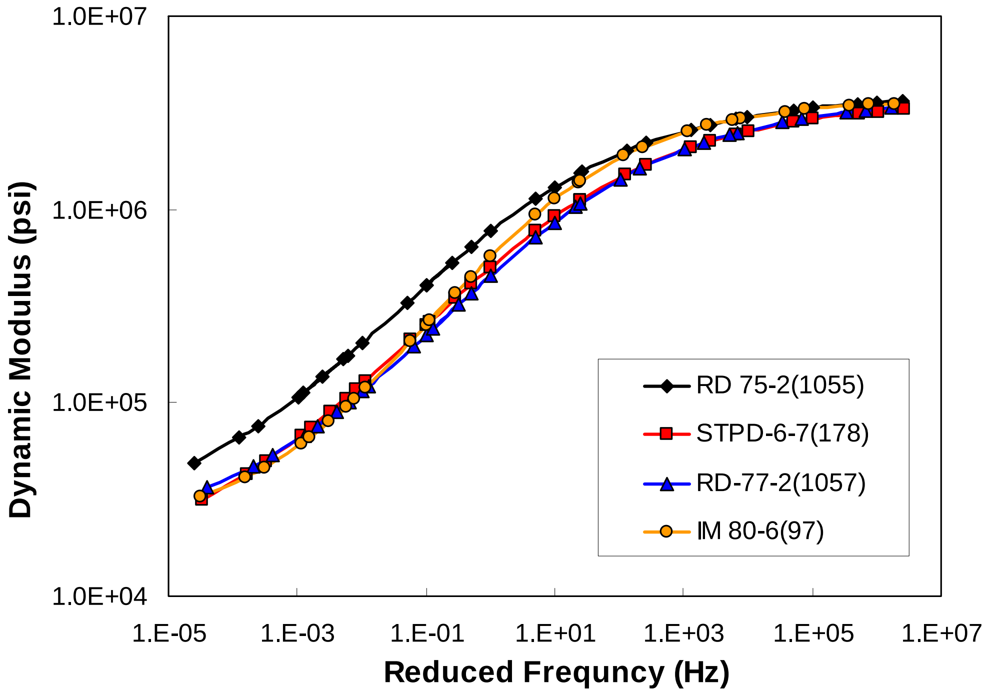
(b) SPL Mixtures



(c) SP4(0.375) Mixtures



(d) SP4(0.5) Mixtures



(e) SP5 Mixtures

Figure 4-8. Master Curves of Each Mixture at a Reference Temperature (70 °F)

Table 4-5. Sigmoidal Function Parameters and Shift Factors of All Mixtures

Mix Type	Project Number	δ	α	β	γ	log a(14)	log a(40)	log a(70)	log a(100)	log a(130)	A	VTS
HRB	RD 9-4 (1012)	4.385	2.120	-0.304	0.668	5.072	2.423	0	-1.937	-3.467	9.513	-3.155
	RD 81-2 (1037)	4.308	2.065	-0.290	0.711	4.897	2.337	0	-1.864	-3.335	9.611	-3.190
	STP 14-4 (110)	4.301	2.167	-0.126	0.673	4.909	2.344	0	-1.871	-3.347	9.587	-3.180
	NH 6-4 (125)	4.277	2.203	0.232	0.745	4.723	2.291	0	-1.887	-3.420	8.059	-2.631
SPL	STPD 6-6 (156)	4.393	2.111	-0.272	0.675	4.926	2.352	0	-1.878	-3.359	9.579	-3.177
	STPD 79-2 (102)	4.158	2.404	-0.604	0.548	5.581	2.655	0	-2.105	-3.756	9.910	-3.299
	STP 91-3 (107)	4.396	2.004	-0.140	0.705	4.565	2.213	0	-1.821	-3.300	8.107	-2.646
	NH 80-9 (832)	4.055	2.475	-0.726	0.523	5.035	2.439	0	-2.001	-3.623	8.254	-2.688
SP4 (0.375)	RD 81-2 (1037)	4.473	2.054	-0.023	0.733	4.652	2.246	0	-1.832	-3.307	8.549	-2.799
	RD 9-4 (1012)	4.330	2.111	0.020	0.691	4.560	2.197	0	-1.786	-3.220	8.708	-2.859
	NH 6-4 (125)	4.322	2.233	-0.136	0.693	4.855	2.340	0	-1.902	-3.430	8.699	-2.856
	RD 25-2 (1014)	4.207	2.302	-0.322	0.636	4.914	2.365	0	-1.917	-3.453	8.836	-2.906
SP4 (0.5)	PEP 183-1 (1020)	4.187	2.307	-0.595	0.522	4.968	2.415	0	-1.996	-3.625	7.897	-2.560
	STPD-NFF 11-2 (115)	6.473	-1.907	0.094	-0.770	4.454	2.153	0	-1.760	-3.181	8.438	-2.757
	NH 281-4 (119)	4.293	2.297	-0.329	0.615	4.904	2.362	0	-1.919	-3.458	8.741	-2.872
	NH 83-3 (107)	6.567	-2.432	0.291	-0.595	4.776	2.302	0	-1.871	-3.374	8.699	-2.856
SP5	RD 75-2 (1055)	4.319	2.277	-0.795	0.528	4.999	2.423	0	-1.993	-3.610	8.161	-2.656
	STPD-6-7 (178)	4.115	2.453	-0.603	0.509	5.020	2.408	0	-1.940	-3.483	9.154	-3.019
	RD-77-2 (1057)	4.279	2.296	-0.406	0.539	4.835	2.326	0	-1.884	-3.392	8.874	-2.920
	IM 80-6 (97)	4.309	2.261	-0.574	0.643	4.884	2.363	0	-1.936	-3.502	8.335	-2.721

4.1.3 Dynamic modulus characterization for Level 2 and Level 3 analysis

As mentioned in Chapter 2, one of the most interesting aspects of the MEPDG design procedure is its hierarchical approach, i.e., the consideration of different levels of inputs. This hierarchical approach enables the designer to select the design input level depending on the degree of significance of the project and availability of resources. Each input level needs different testing efforts and procedures to determine mixture dynamic modulus characteristics as presented in Table 4-6.

Table 4-6. Dynamic Modulus Estimation at Various Hierarchical Input Levels

Input Level	Description
1	• Conduct $\lvert E^*\rvert$ (dynamic modulus) laboratory test at loading frequencies and temperatures of interest for the given mixture • Conduct binder complex shear modulus (G^*) and phase angle (ϕ) testing on the proposed asphalt binder (AASHTO T315) at ω=1.59 Hz (10 rad/s) over a range of temperatures • From binder test data estimate *A-VTS* for mix-compaction temperature • Develop master curve for the asphalt mixture that accurately defines the time-temperature dependency including aging
2	• No $\lvert E^*\rvert$ laboratory test required • Use $\lvert E^*\rvert$ predictive equation • Conduct binder complex shear modulus (G^*) and phase angle (ϕ) testing on the proposed asphalt binder (AASHTO T315) at ω=1.59 Hz (10 rad/s) over a range of temperatures. The binder viscosity or stiffness can also be estimated using conventional asphalt test data such as Ring and Ball Softening Point, absolute and kinematic viscosities, or using the Brookfield viscometer. • Develop *A-VTS* for mix-compaction temperature • Develop master curve for the asphalt mixture that accurately defines the time-temperature dependency including aging
3	• No $\lvert E^*\rvert$ laboratory test required • Use $\lvert E^*\rvert$ predictive equation • Use typical *A-VTS* values provided in the Design Guide software based on PG, viscosity, or penetration grade of the binder • Develop master curve for the asphalt mixture that accurately defines the time-temperature dependency including aging

As shown in the table, Level 1 MEPDG design needs mixture dynamic modulus tests at different temperatures and loading frequencies, while Levels 2 and 3 do not require physical modulus testing. Dynamic modulus master curves for Level 2 and 3 analyses are developed using Witczak's dynamic modulus predictive equation. This equation has the ability to predict the dynamic modulus of asphalt mixtures over a range of

temperatures, rates of loading, and aging conditions by using information that is readily available from the volumetric mixture design.

The first version of Witczak's predictive equation (Fonseca and Witczak 1996) was used in the first development of MEPDG interim guide (Andrei et al. 1999). In the interim guide, MEPDG considered mixture volumetric properties and gradation, binder viscosity, and loading frequency as input variables to predict the dynamic modulus of asphalt concrete mixtures. Multivariate regression analysis of 2,750 experimental data was used to construct the 1999 version of the predictive $|E^*|$ expression. Later, the 1999 version of the predictive equation was revised with more test data, which resulted in replacements of several model coefficients. The predictive equation implemented in the current MEPDG version (NCHRP 1-37A 2004) is shown in the following equation:

$$\begin{aligned}\log\left|E^*\right| &= 3.750063 + 0.02932\rho_{200} - 0.001767(\rho_{200})^2 \\ &- 0.002841\rho_4 - 0.058097V_a - 0.802208\left(\frac{V_{beff}}{V_{beff} + V_a}\right) \\ &+ \frac{3.871977 - 0.0021\rho_4 + 0.003958\rho_{38} - 0.000017(\rho_{38})^2 + 0.005470\rho_{34}}{1 + e^{(-0.603313 - 0.31335\log f - 0.393532\log\eta)}}\end{aligned} \qquad [4.6]$$

where $|E^*|$ = dynamic modulus of mixture (psi),

ρ_{200} = % passing the No.200 sieve,

ρ_4 = cumulative % retained on the No.4 sieve,

ρ_{38} = cumulative % retained on the 3/8-in. sieve,

ρ_{34} = cumulative % retained on the 3/4-in. sieve,

V_a = air void content (%),

V_{beff} = effective binder content (% by volume),

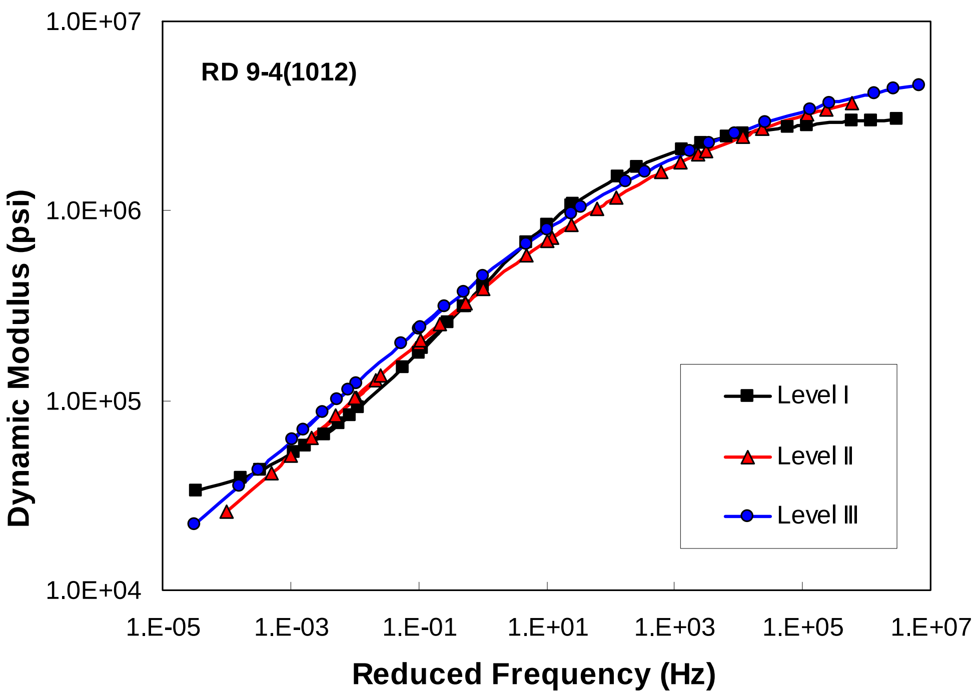
f = loading frequency (Hz), and

η = bitumen viscosity (10^6 Poise).

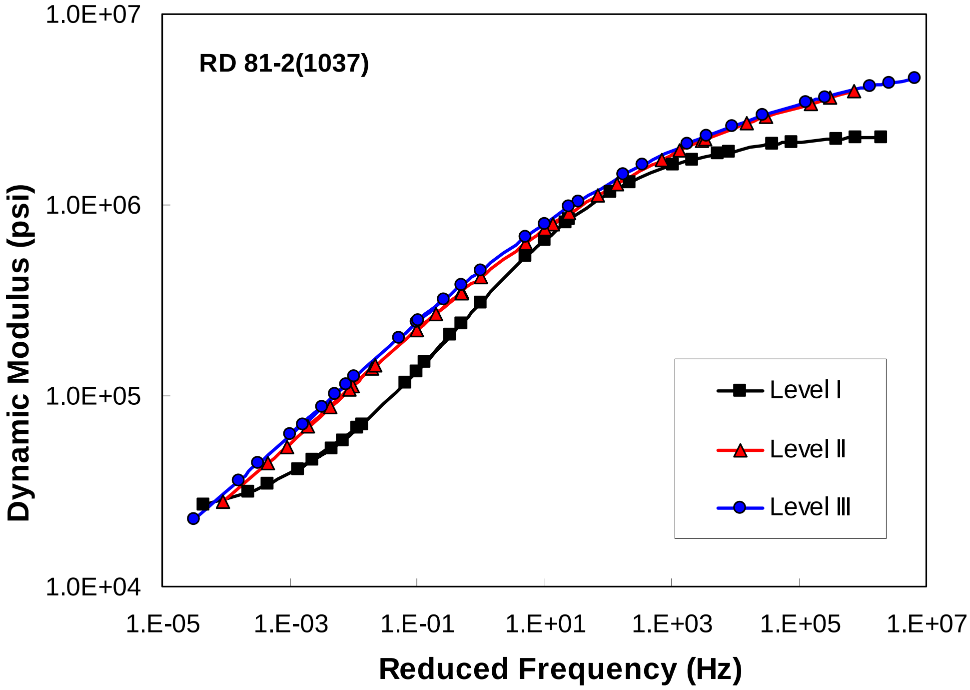
The viscosity of the asphalt binder at the temperature of interest is a critical input parameter for the dynamic modulus characterization and the determination of shift factors as presented in Table 4-6. For Level 1 and Level 2 design, the MEPDG requires

conducting binder complex shear modulus (G^*) and phase angle (ϕ) testing on at ω=1.59 Hz (10 rad/s) over a range of temperatures. The binder stiffness data obtained at different temperatures are then used to calculate binder viscosity (η) and correspondingly two regression parameters (A and VTS) which represent temperature susceptibility of the asphalt binder as previously described in Equations [4.1] and [4.2]. On the other hand, Level 3 MEPDG analysis uses typical *A-VTS* values provided in the Design Guide software based on PG, viscosity, or penetration grade of the binder.

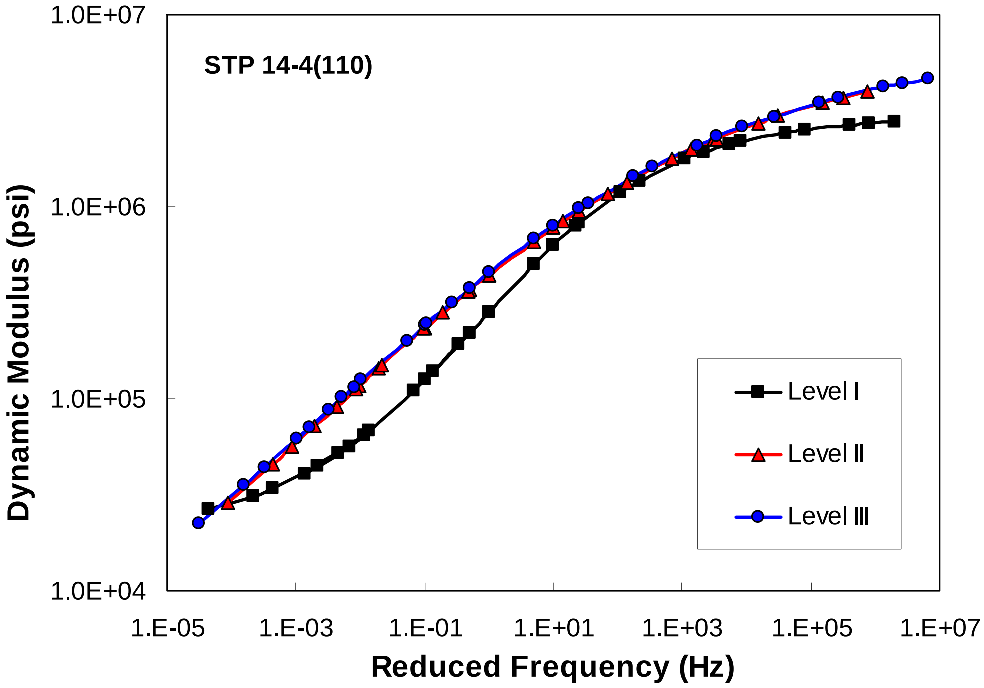
Figure 4-9 shows constructed master curves for Level 2 and 3 design analyses for all HMA mixtures. For comparison, Level 1 master curves are also plotted in each graph. A discrepancy between the Level 1 (measured) master curves and Level 2 or 3 (predicted) master curves can be observed. The level of discrepancy between curves was mixture-specific, and was generally larger at lower or higher loading frequencies. Differences between Level 2 and Level 3 master curves were not significant, since Witczak's predictive model in Equation [4.6] was used for both cases.



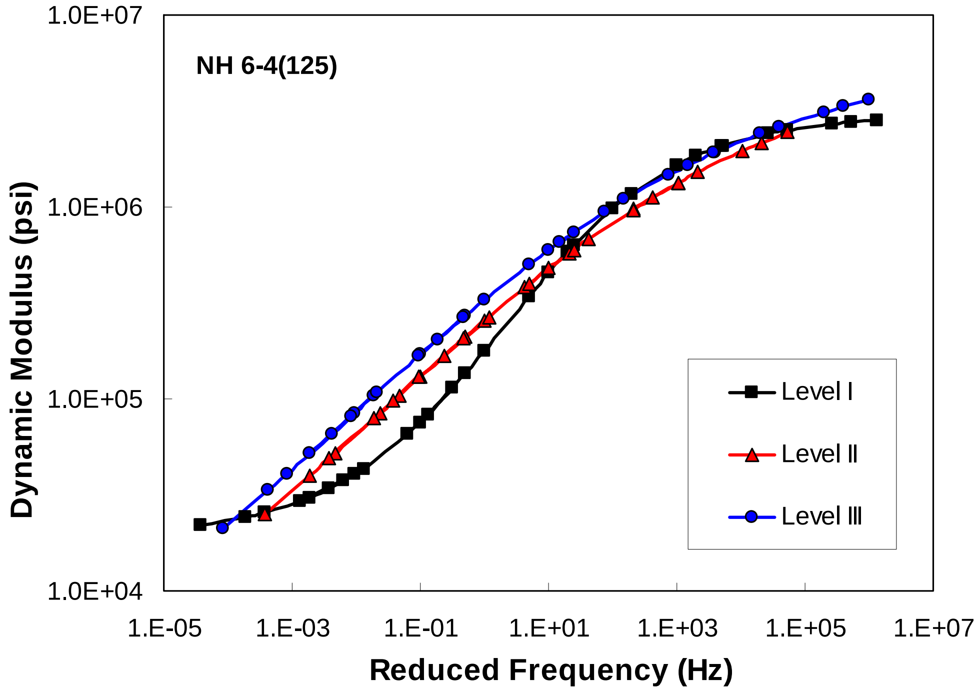
(a) HRB: RD 9-4(1012)



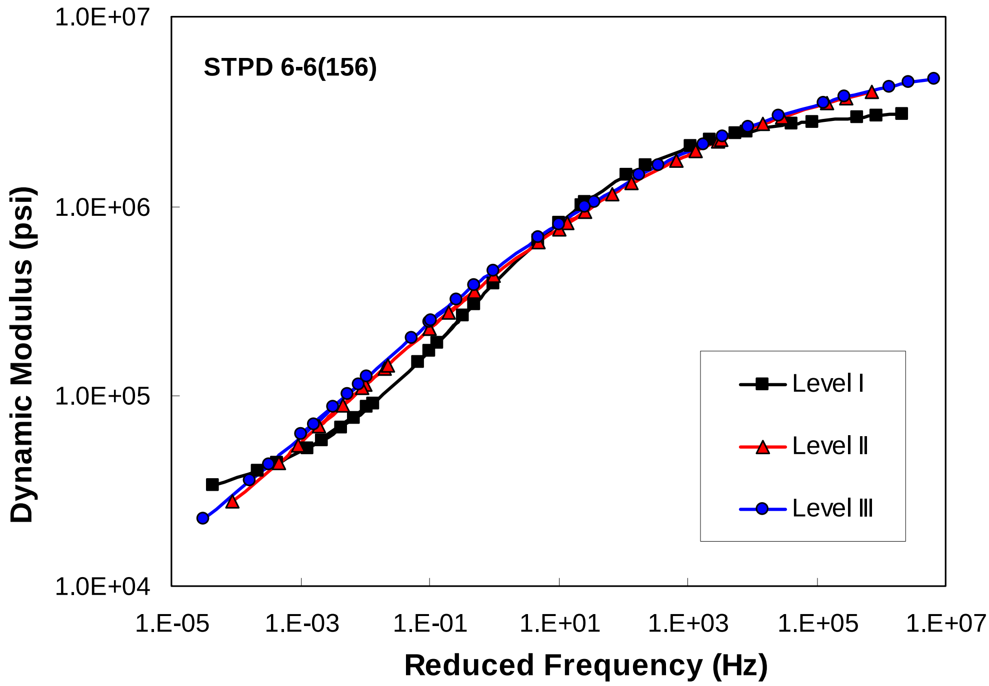
(b) HRB: RD 81-2(1037)



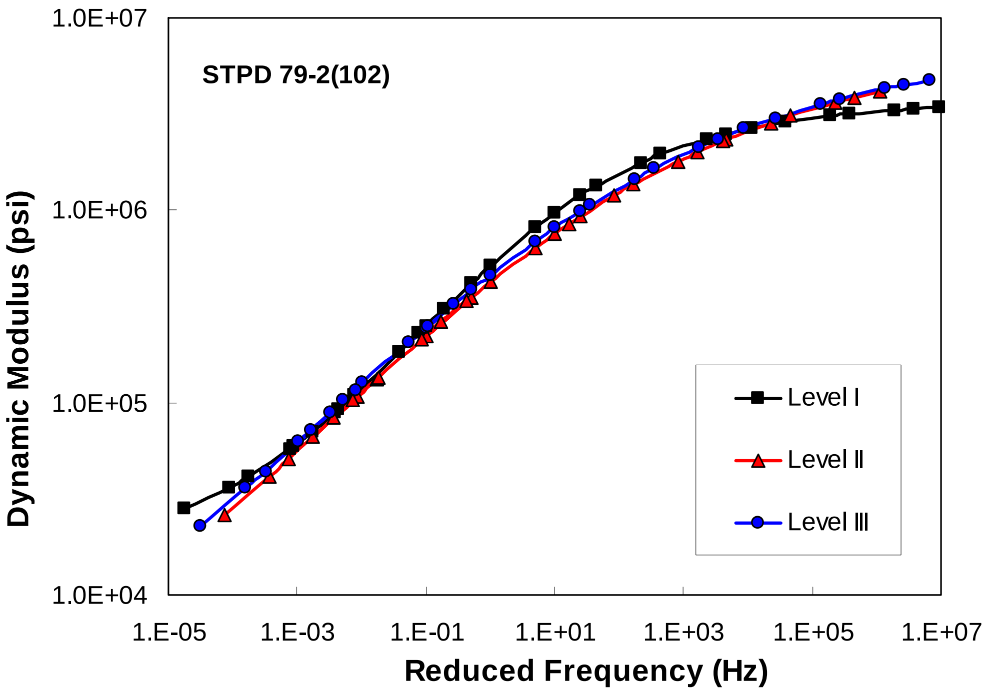
(c) HRB: STP 14-4(110)



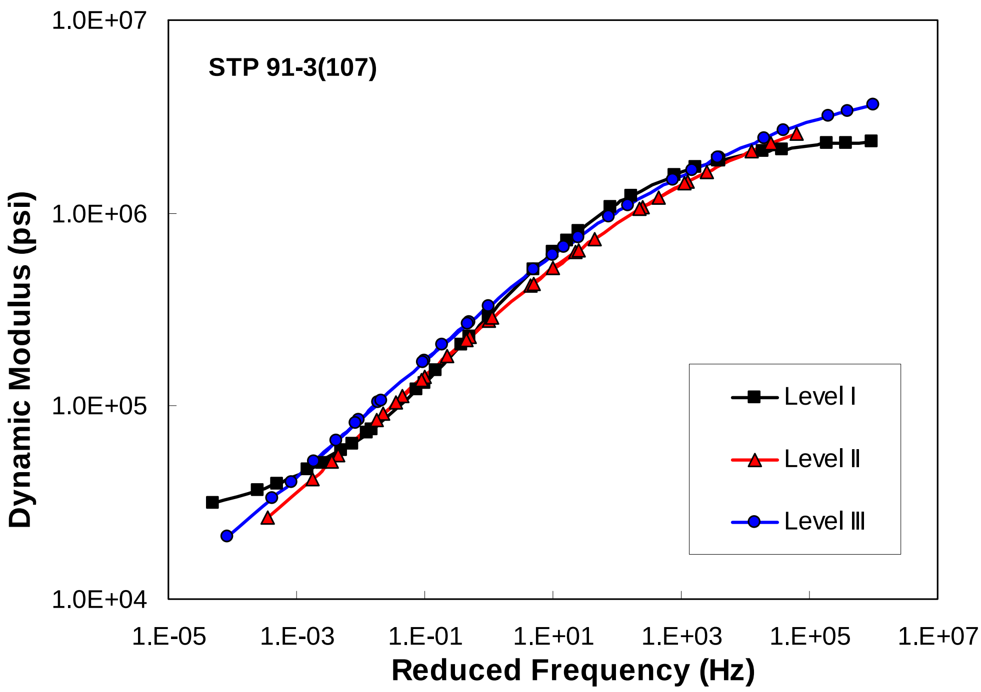
(d) HRB: NH6-4(125)



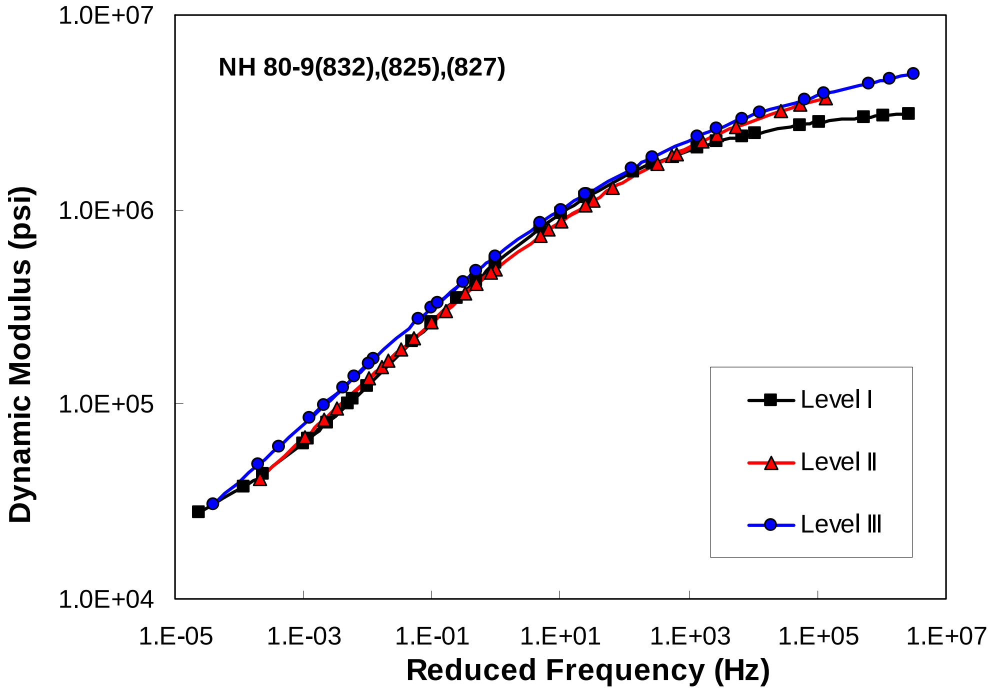
(e) SPL: STPD 6-6(156)



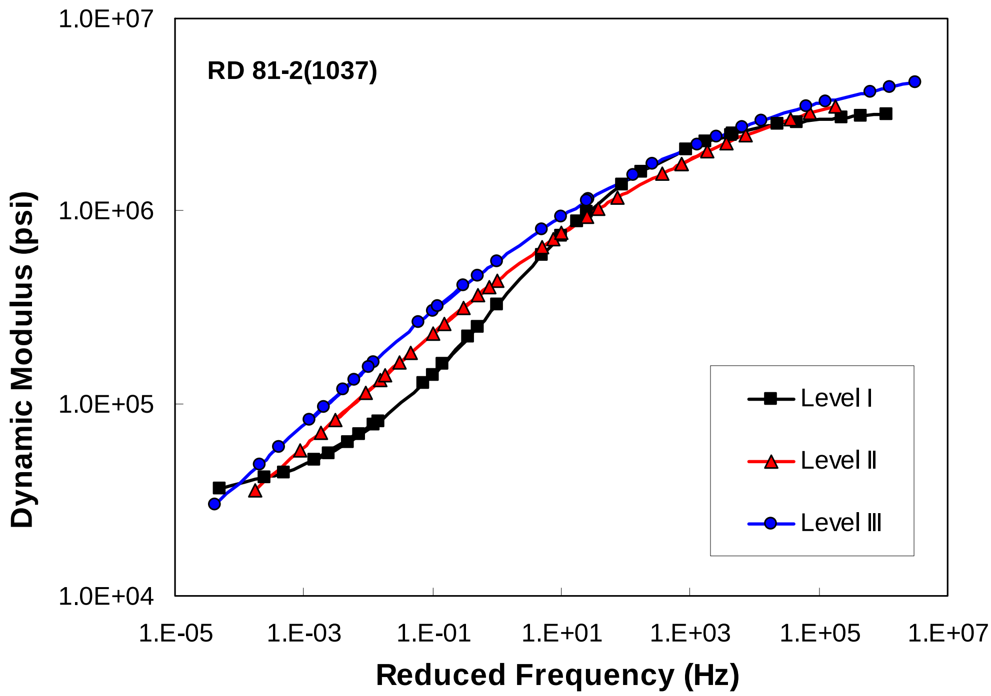
(f) SPL: STPD 79-2(102)



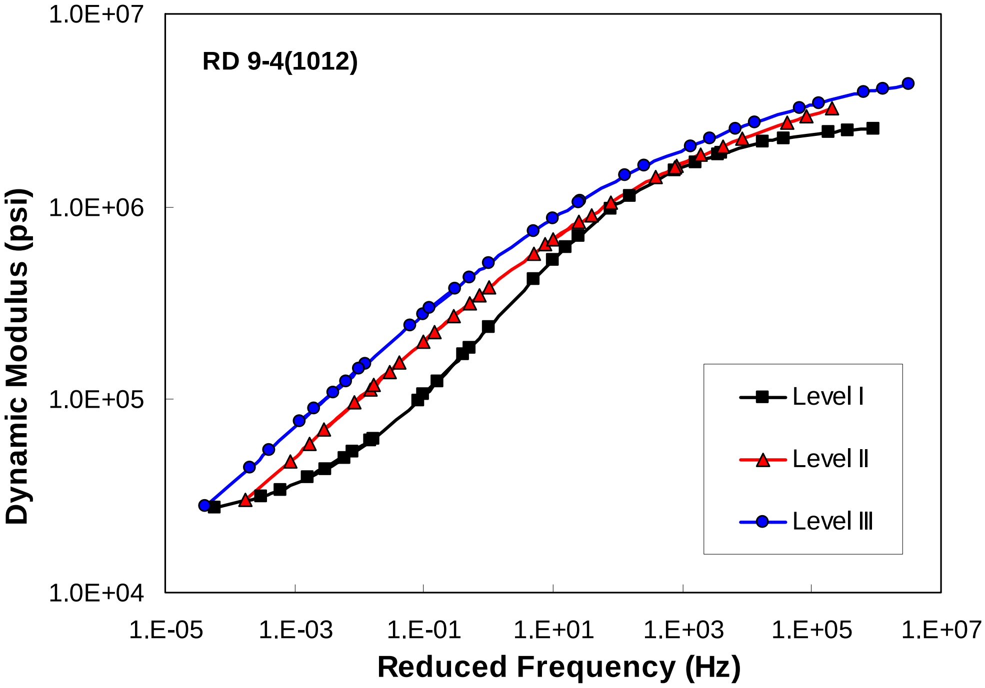
(g) SPL: STP 91-3(107)



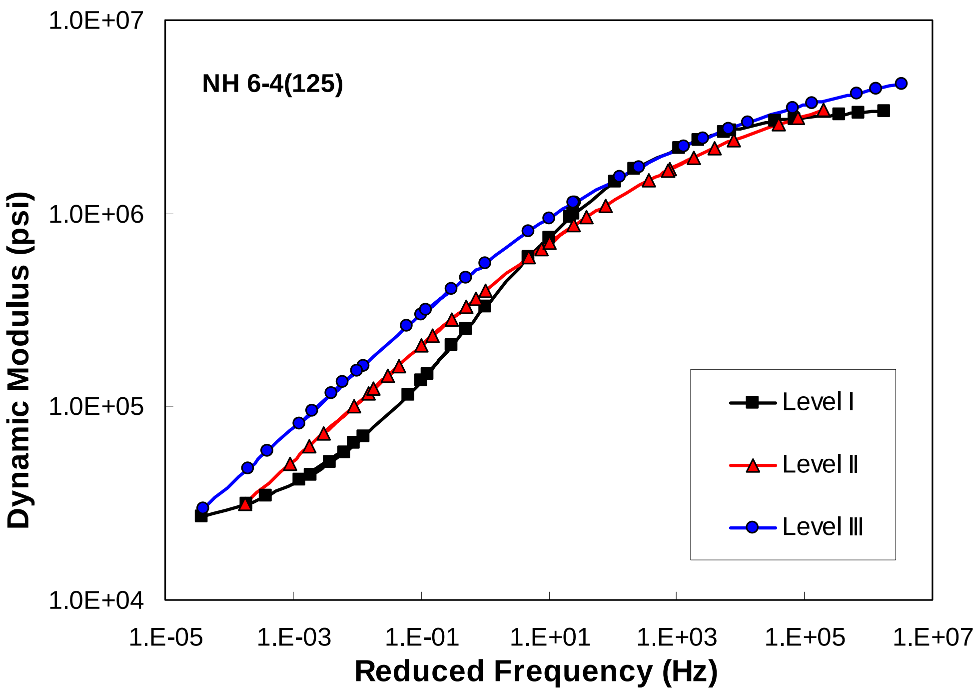
(h) SPL: NH 80-9 (832), (825), (827)



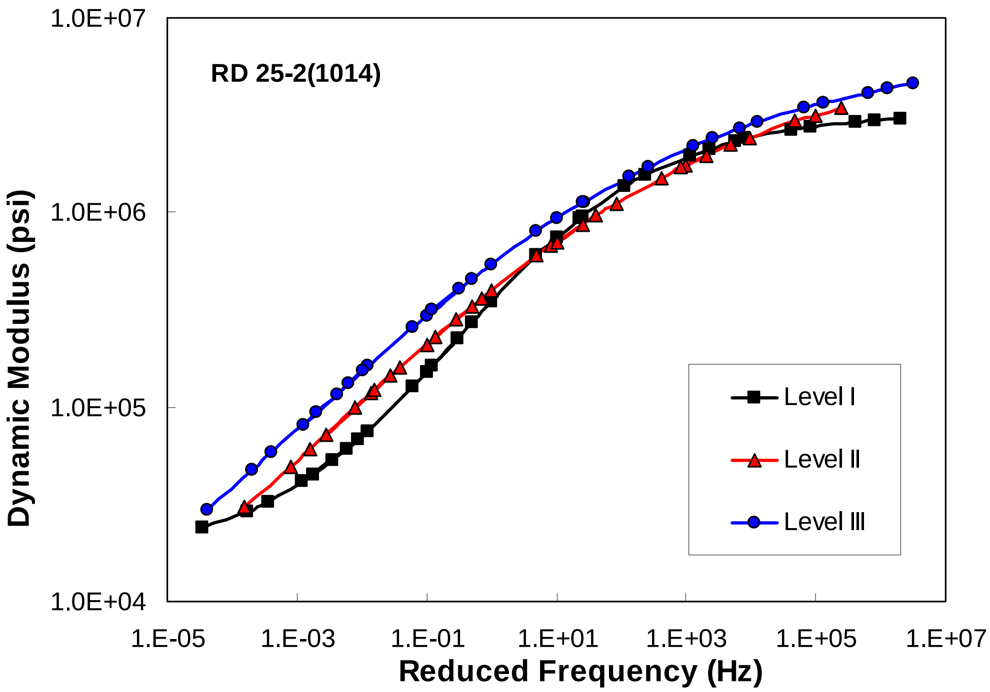
(i) SP4(0.375): RD 81-2(1037)



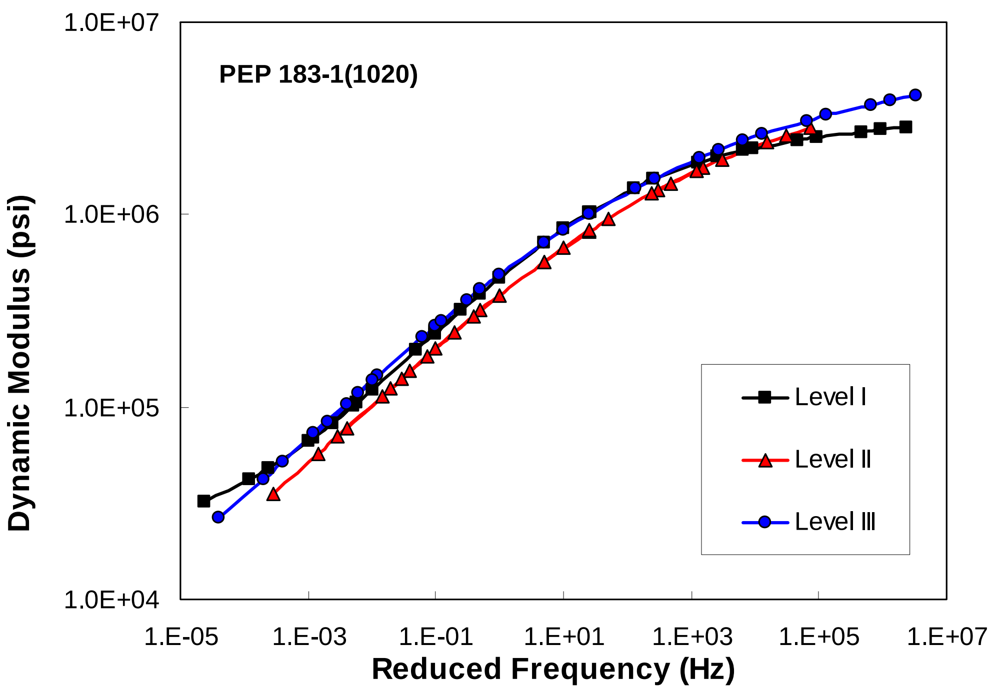
(j) SP4(0.375): RD 9-4(1012)



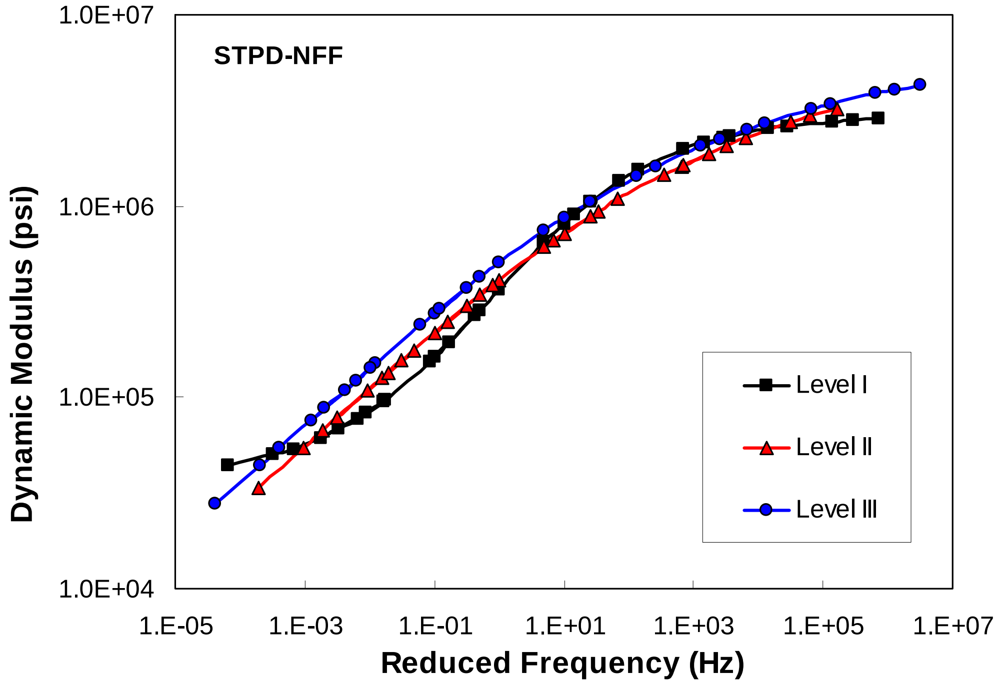
(k) SP4(0.375): NH 6-4(125)



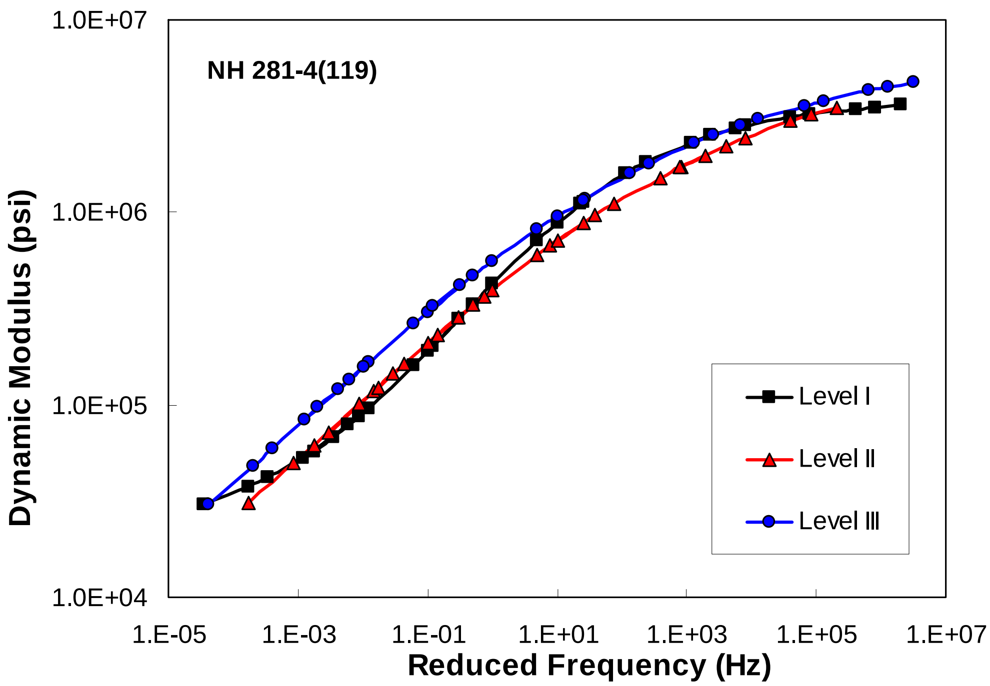
(l) SP4(0.375): RD 25-2(1014)



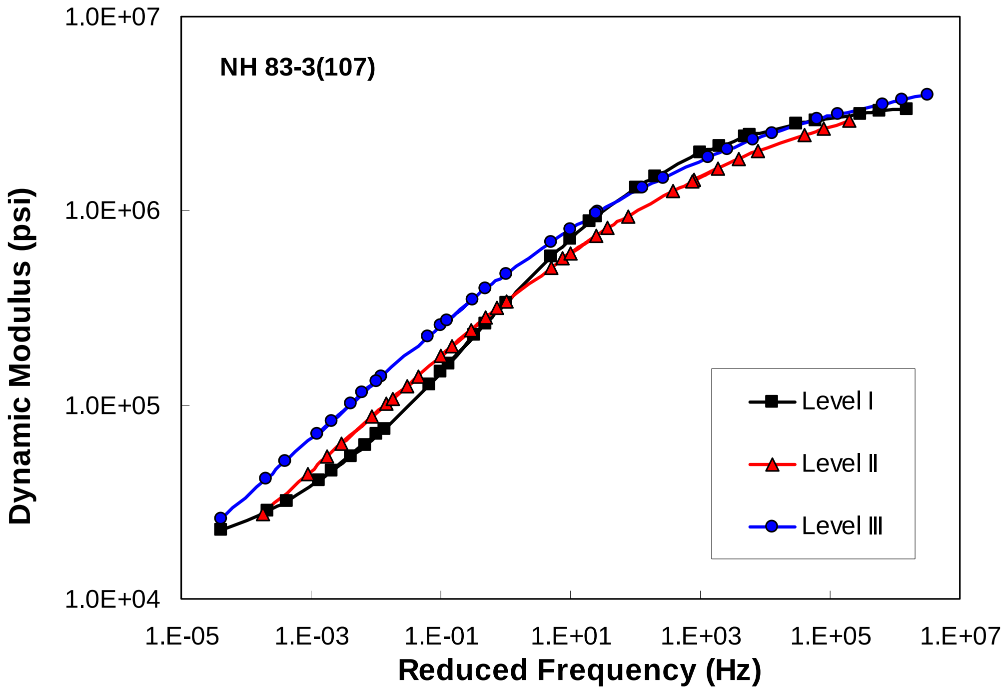
(m) SP4(0.5): PEP 183-1(1020)



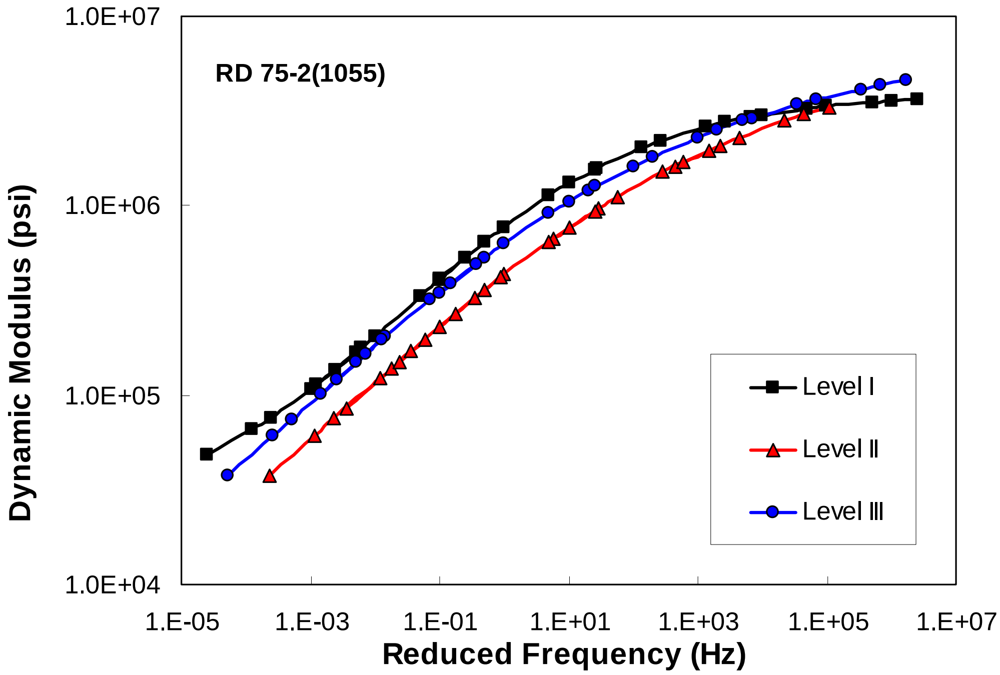
(n) SP4(0.5): STPD-NFF 11-2(115)



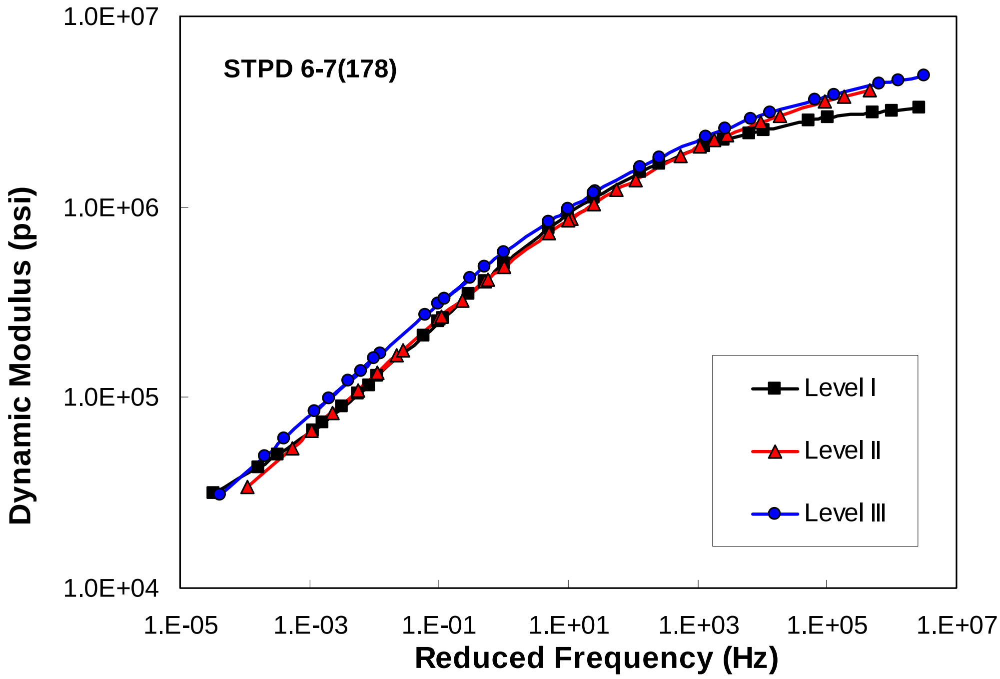
(o) SP4(0.5): NH 281-4(119)



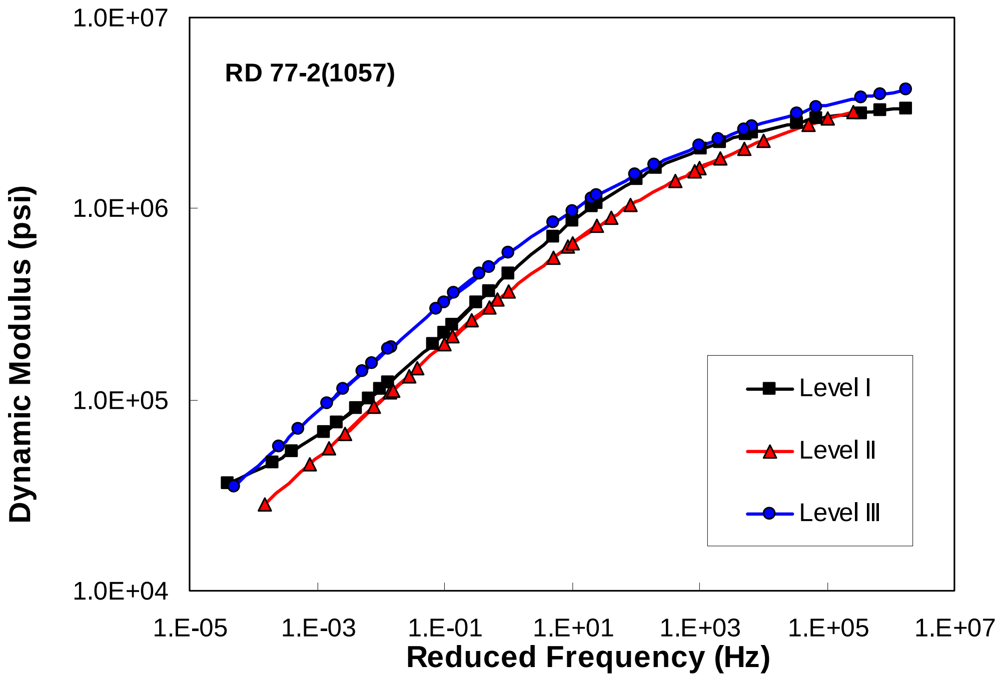
(p) SP4(0.5): NH 83-3(107)



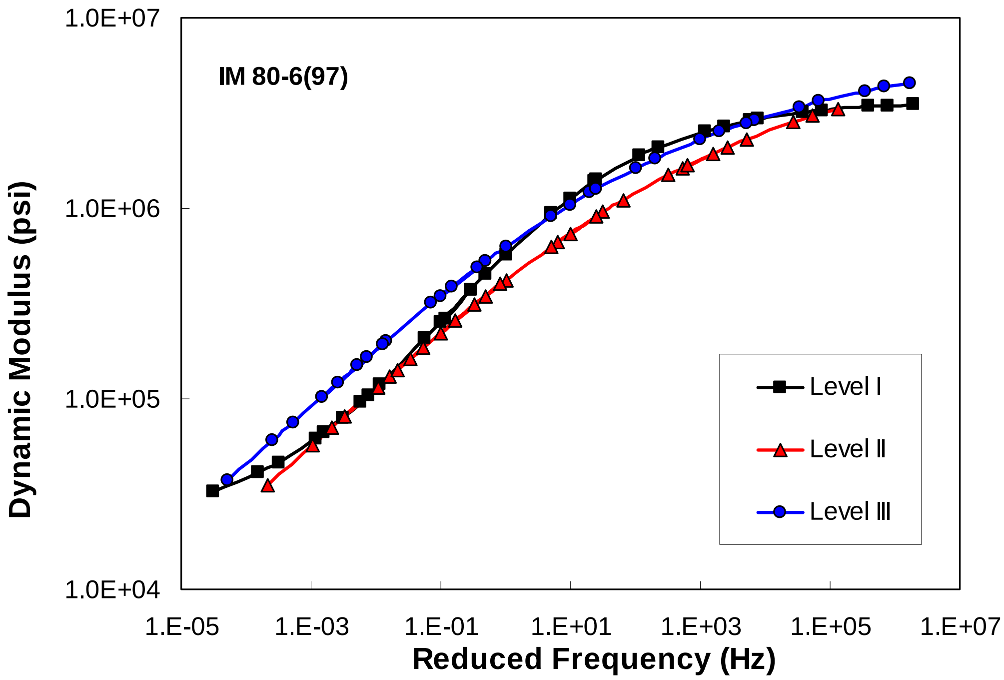
(q) SP5: RD 75-2(1055)



(r) SP5: STPD 6-7(178)



(s) SP5: RD 77-2(1057)



(t) SP5: IM 80-6(97)

Figure 4-9. Master Curves of All Twenty HMA Mixtures at Different Input Levels

4.1.4 Creep compliance test (AASHTO T322)

The creep compliance test is used to describe the low-temperature behavior of asphalt mixtures. It is the primary input for predicting thermal cracking in asphalt pavements over their service lives. This test procedure is described in AASHTO T322. The current standard method used in the United States to determine the creep compliance of asphalt mixtures is the indirect tensile (IDT) test. In this study, the creep compliance test was conducted at 14 °F.

A Superpave gyratory compactor was used to produce cylindrical samples with a diameter of 150 mm and a height of 115 mm. The samples were then cut into specimens with a diameter of 150 mm and a thickness of 38 mm. The target air void of testing specimens was 4% ± 0.5%. Figure 4-10 demonstrates the specimen production process using the Superpave gyratory compactor, a saw machine, and the resulting specimen used to conduct the creep compliance test.

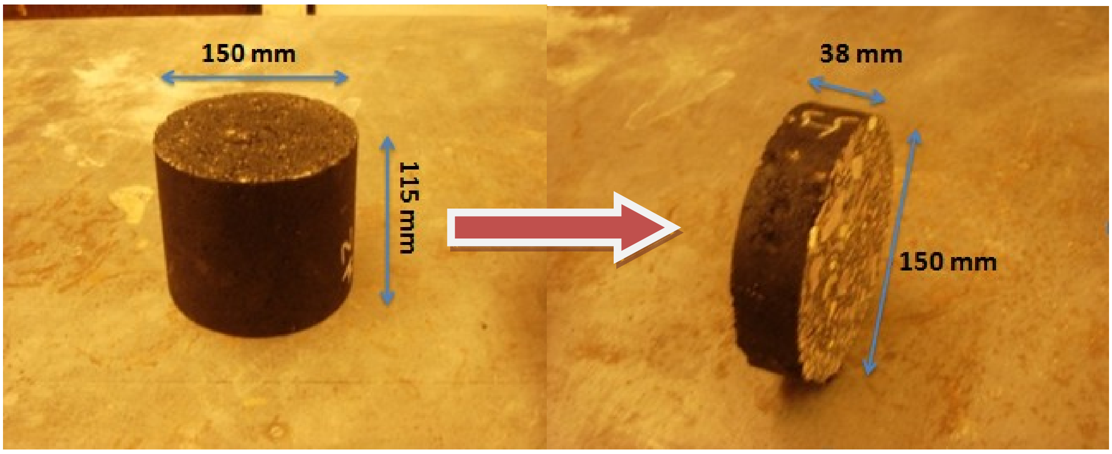


Figure 4-10. Specimen Preparation Process for Creep Compliance Test

Table 4-7 summarizes air voids, bulk specific gravity (G_{mb}), and maximum specific gravity (G_{mm}) of each creep compliance testing specimen. As shown in the table, three replicates were tested for each mixture.

Table 4-7. Air Voids, G_{mb}, and G_{mm} of Creep Compliance Testing Specimens

Mix Type	Project Number	Sample Number	Air Void (%)	G_{mb}	G_{mm}
HRB	RD 9-4(1012)	#1	4.10	2.325	2.424
		#2	4.22	2.322	
		#3	4.15	2.323	
	RD 81-2(1037)	#1	3.68	2.331	2.420
		#2	3.51	2.335	
		#3	3.56	2.334	
	STP 14-4(110)	#1	3.62	2.328	2.415
		#2	4.22	2.313	
		#3	4.09	2.316	
	NH 6-4(125)	#1	4.41	2.312	2.419
		#2	4.30	2.315	
		#3	4.43	2.312	
SPL	STPD 6-6(156)	#1	3.57	2.362	2.449
		#2	3.69	2.359	
		#3	3.68	2.359	
	STPD 79-2(102)	#1	3.69	2.375	2.466
		#2	4.02	2.367	
		#3	4.26	2.361	
	STP 91-3(107)	#1	4.32	2.337	2.443
		#2	4.31	2.338	
		#3	4.38	2.336	
	NH 80-9(832)	#1	4.39	2.346	2.454
		#2	4.38	2.347	
		#3	4.44	2.345	

SP4 (0.375)	RD 81-2(1037)	#1	3.83	2.337	2.430
		#2	3.94	2.334	
		#3	3.68	2.341	
	RD 9-4(1012)	#1	4.33	2.305	2.409
		#2	4.28	2.306	
		#3	4.28	2.306	
	NH 6-4(125)	#1	4.16	2.322	2.423
		#2	3.88	2.329	
		#3	4.13	2.323	
	RD 25-2(1014)	#1	3.90	2.322	2.416
		#2	4.00	2.319	
		#3	3.92	2.321	
SP4(0.5)	PEP 183-1(1020)	#1	4.00	2.342	2.440
		#2	3.84	2.346	
		#3	4.32	2.355	
	STPD-NFF 11-2 (115)	#1	3.54	2.343	2.429
		#2	4.02	2.331	
		#3	4.22	2.326	
	NH 281-4(119)	#1	3.93	2.335	2.430
		#2	3.96	2.334	
		#3	3.85	2.336	
	NH 83-3(107)	#1	4.24	2.324	2.427
		#2	3.75	2.336	
		#3	4.34	2.322	
SP5	RD 75-2(1055)	#1	3.58	2.360	2.448
		#2	4.17	2.346	
		#3	4.37	2.341	
	STPD-6-7(178)	#1	3.77	2.349	2.441
		#2	4.14	2.340	
		#3	4.13	2.340	
	RD-77-2(1057)	#1	3.93	2.367	2.464
		#2	3.77	2.371	
		#3	3.96	2.366	
	IM 80-6(97)	#1	4.05	2.327	2.425
		#2	4.29	2.321	
		#3	4.24	2.322	

On each flat face of the specimen, two studs were placed along the vertical and two along the horizontal axes, with a center-to-center spacing of 38 mm, so that two cross extensometers could be mounted on the surfaces of the specimens (shown in Figure 4-11). The vertical and horizontal displacements were recorded using the two cross extensometers during the test.

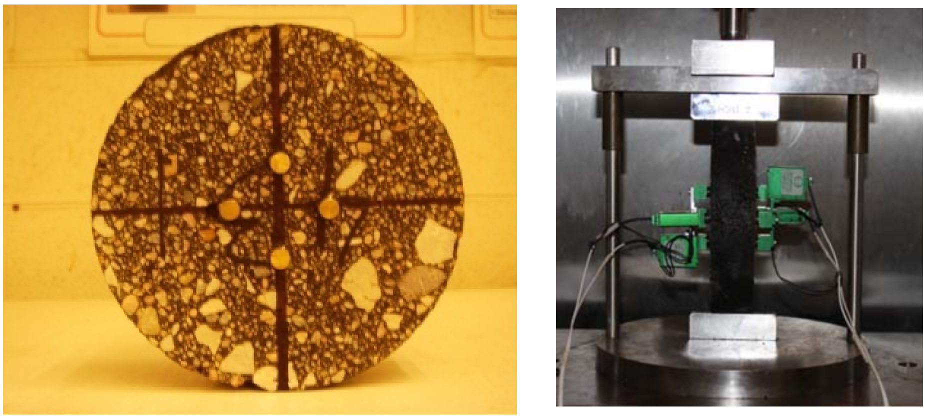


Figure 4-11. A Specimen with Extensometers Mounted in Testing Station

Once all three replicates of each mixture were tested, horizontal and vertical deformation measurements of the six faces (three specimens and two faces per specimen) were recorded for each specimen. The highest and lowest measurements of horizontal and vertical deformation were then excluded so that four middle measurements could be averaged. Finally, the creep compliance of each mixture was determined by using the following equation, incorporating the averaged measurements:

$$D(t) = \frac{X \cdot d \cdot b}{P \cdot GL} \cdot \left\{ 0.6354 \cdot \left(\frac{X}{Y} \right)^{-1} - 0.332 \right\} \quad [4.7]$$

where $D(t)$ = creep compliance,

X = averaged horizontal deformation,

Y = averaged vertical deformation,

d = specimen diameter,

b = specimen thickness,

P = creep load, and

GL = gauge length.

In order to achieve the Level 1 MEPDG design, three temperatures (32 °F, 14 °F, and −4 °F) are used to determine the creep compliance of mixtures, and a tensile strength test at 14 °F is also performed. For the Level 2 MEPDG design, only one temperature (14 °F) is involved for the creep compliance and tensile strength testing of mixtures. On the other

hand, Level 3 analysis does not require physical testing at low temperatures. Creep compliance values at three different temperatures (−4, 14, and 32 °F) and the tensile strength at 14 °F are automatically generated by the MEPDG software based on correlations with mixture volumetric characteristics and binder properties.

In this study, only the Level 2 creep compliance tests at 14 °F were conducted. Level 1 creep compliance testing and the tensile strength test at 14 °F could not be performed because of the limited capability of the UTM-25kN testing equipment, which allows a loading level up to 25 kN and testing temperatures from 5 °F to 140 °F. The resulting Level 2 creep compliances at 14 °F of all twenty HMA mixtures are presented in Figure 4-12. As can be observed from the figure, and similar to the dynamic modulus test results, variations in creep compliance values among mixtures exist even though the mixtures are of the same type. Since creep compliance values at different loading times (i.e., 1, 2, 5, 10, 20, 50, and 100 s) were used as inputs for the MEPDG simulations to predict the thermal cracking potential of pavements, the creep compliance data at the seven discrete loading times were included in the database presented in Appendix 1. Tensile strength value at 14 °F presented in the database was calculated using the following regression equation, which has been implemented in the current MEPDG software:

$$TS = 7416.712 - 114.016(V_a) - 0.304(V_a)^2 - 122.592(VFA) + 0.704(VFA)^2 + 405.71\log(Pen77F) - 2039.296\log(A) \qquad [4.8]$$

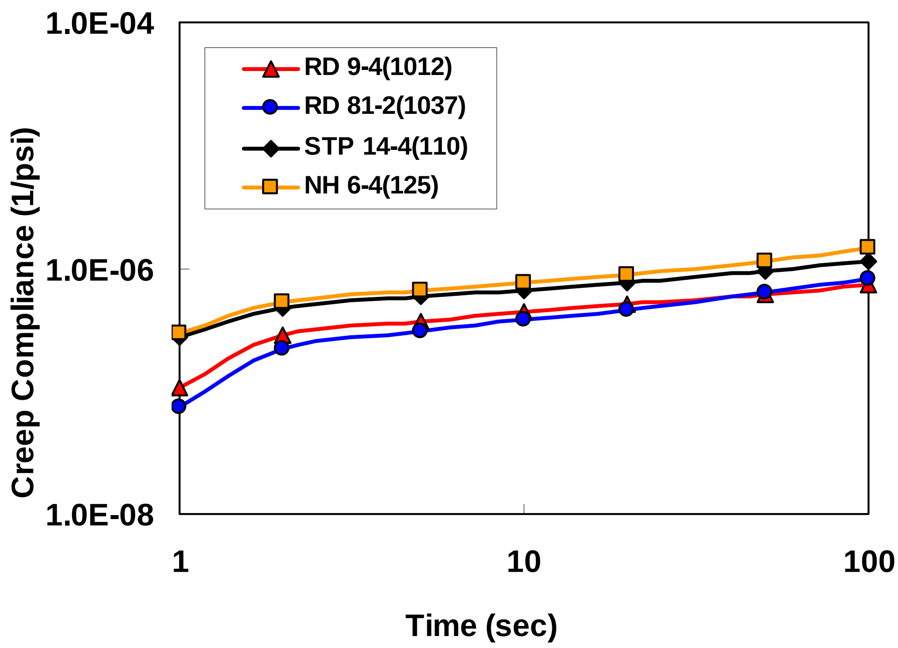
where TS = indirect tensile strength (psi) at 14 ºF,

V_a = air void content (%),

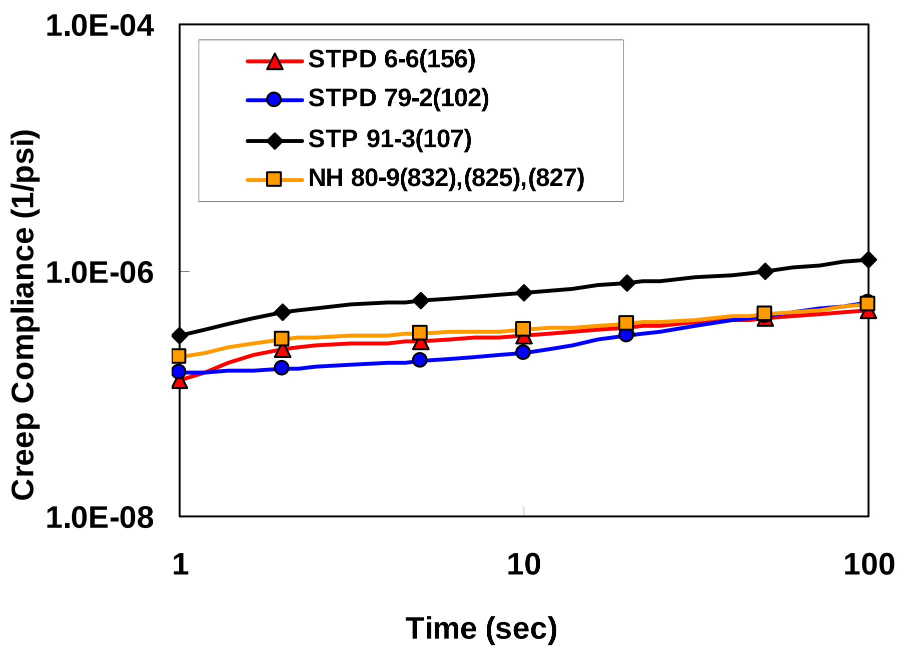
VFA = voids filled with asphalt (%),

$Pen77F$ = binder penetration at 77 ºF (dmm), and

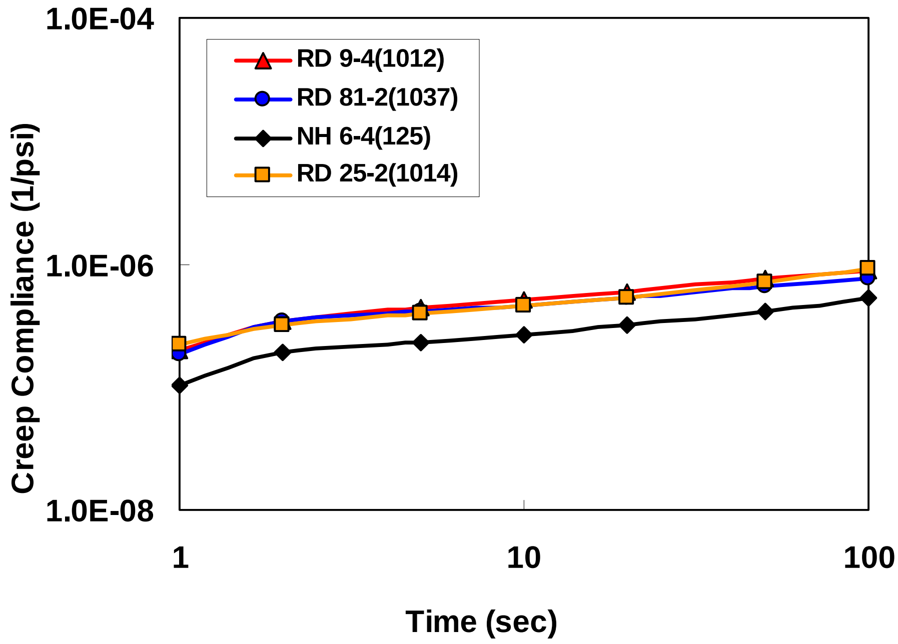
A = viscosity – temperature susceptibility intercept.



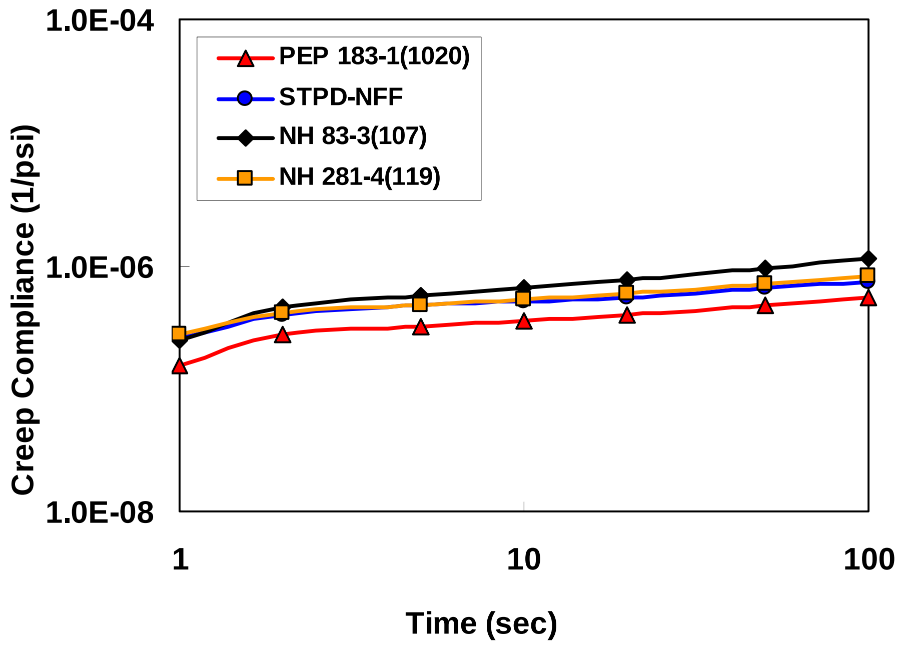
(a) HRB Mixtures



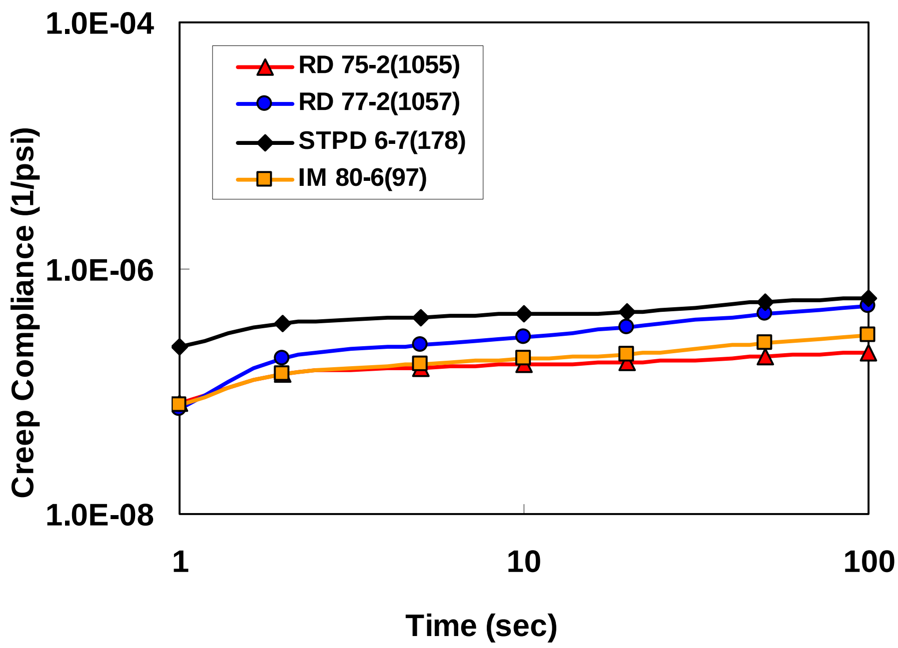
(b) SPL Mixtures



(c) SP4(0.375) Mixtures



(d) SP4(0.5) Mixtures



(e) SP5 Mixtures

Figure 4-12. Creep Compliance at 14 °F of All HMA Mixtures

As previously mentioned, the Level 3 analysis can also be conducted using creep compliance and tensile strength data that are produced by MEPDG software based on correlations with mixture volumetric characteristics and binder properties. Similar to the regression equation for the tensile strength of mixture, time-varying creep compliance data are obtained by the following equations:

$$D(t) = D_1 t^m \quad [4.9]$$

$$\log D_1 = -8.524 + 0.01306(T) + 0.7957\log(V_a) + 2.0103\log(VFA) - 1.923\log(A) \quad [4.10]$$

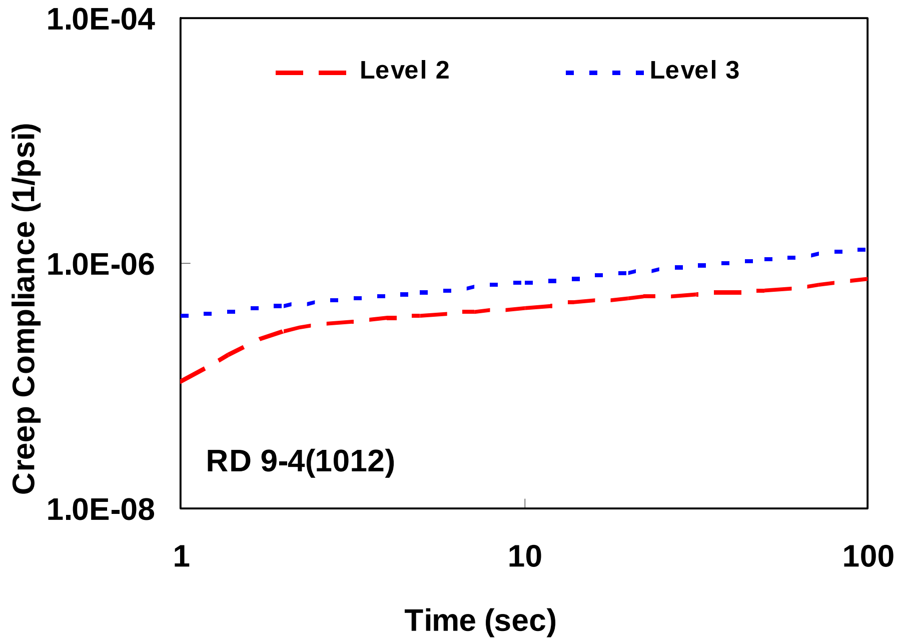
$$m = 1.1628 - 0.00185(T) - 0.04596(V_a) - 0.01126(VFA) + 0.00247(Pen77F) + 0.001638(T)(Pen77F)^{0.4605} \quad [4.11]$$

where $D(t)$ = creep compliance (1/psi),

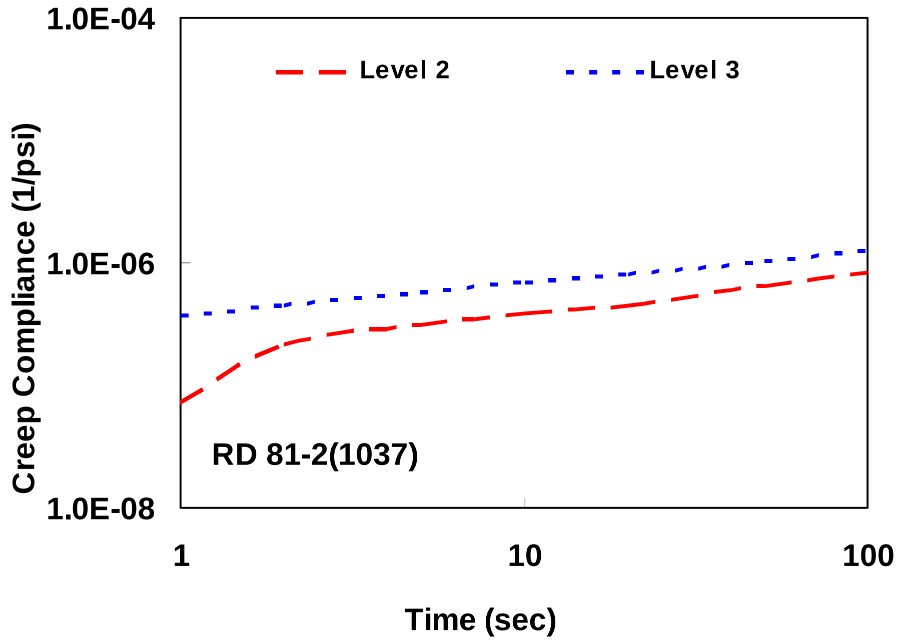
D_1 and m = creep compliance model parameters, and

T = testing temperature (F).

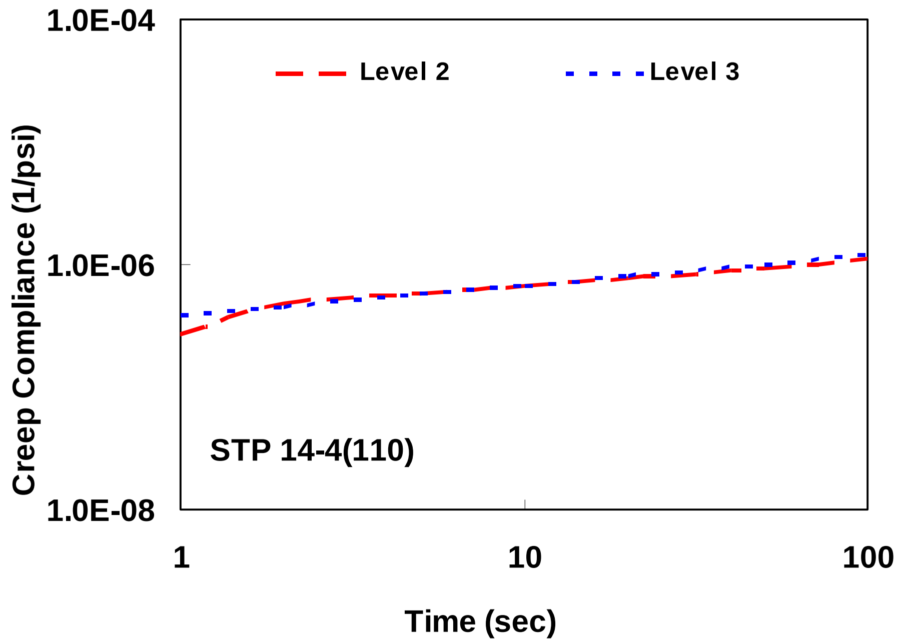
Figure 4-13 compares creep compliance results obtained from the Level 2 testing to the calculated creep compliance values using Equation [4.11] for Level 3 analysis. A mixture-specific discrepancy can be observed between Level 2 (measured) curves and Level 3 (calculated) curves. Differences between Level 2 and Level 3 shown in the figure would affect low temperature cracking performance of pavements.



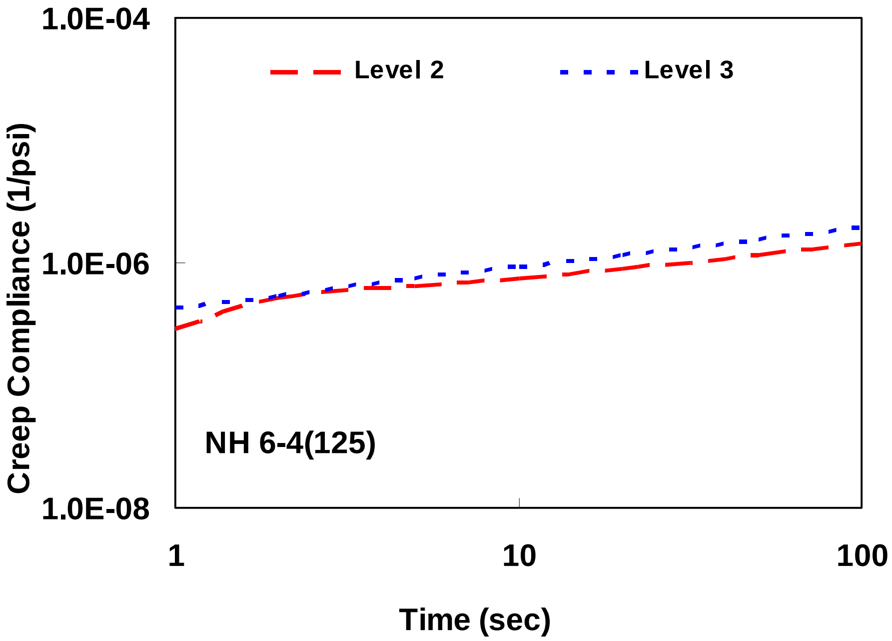
(a) HRB: RD 9-4(1012)



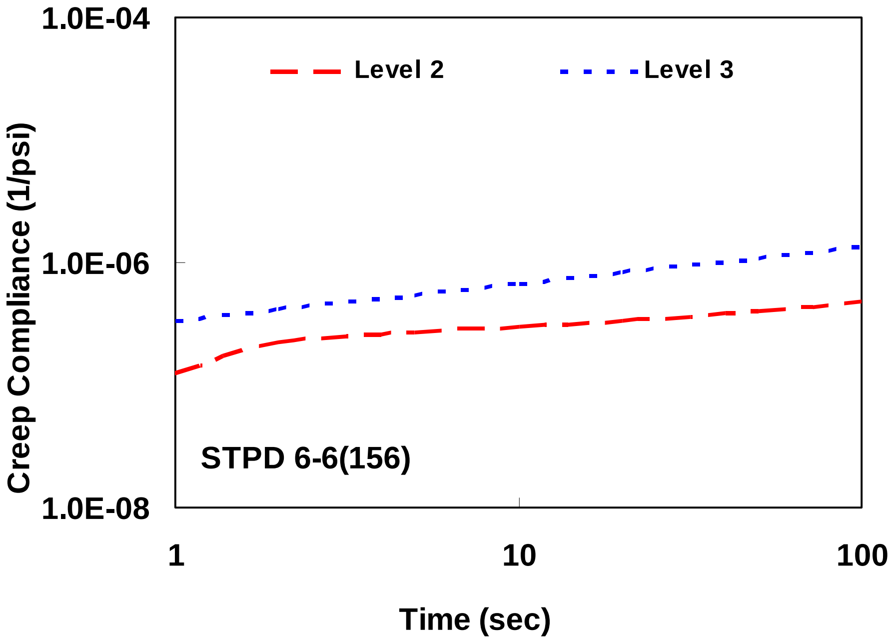
(b) HRB: RD 81-2(1037)



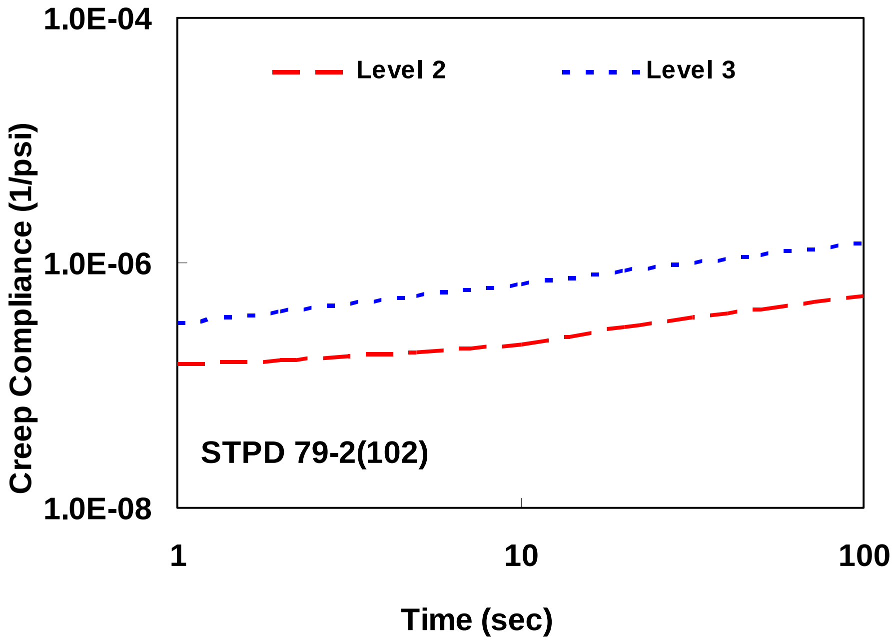
(c) HRB: STP 14-4(110)



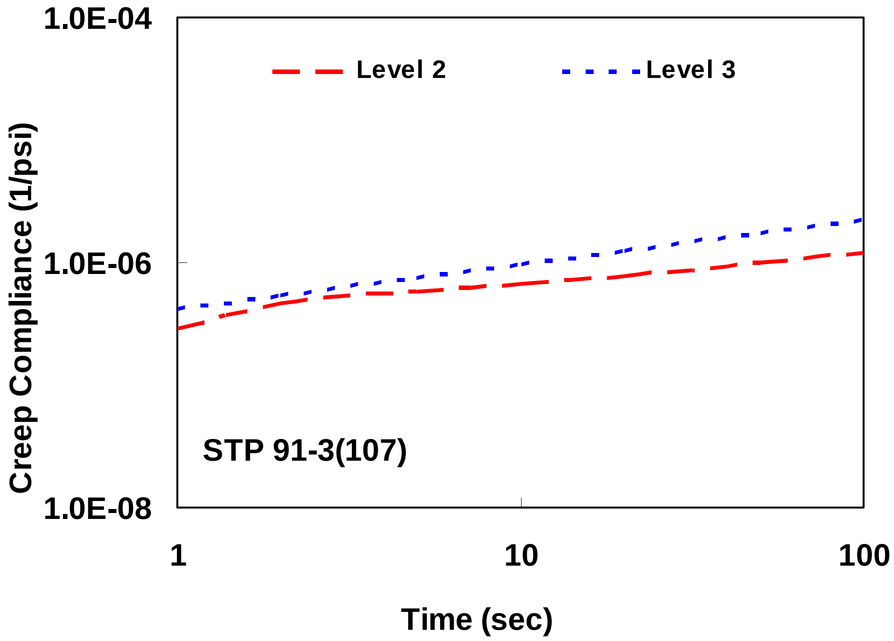
(d) HRB: NH6-4(125)



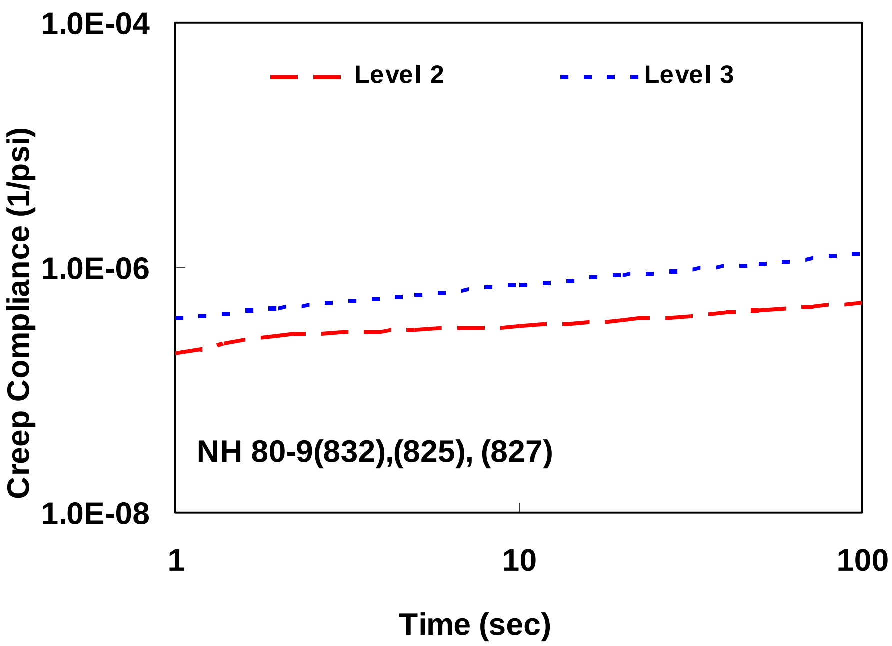
(e) SPL: STPD 6-6(156)



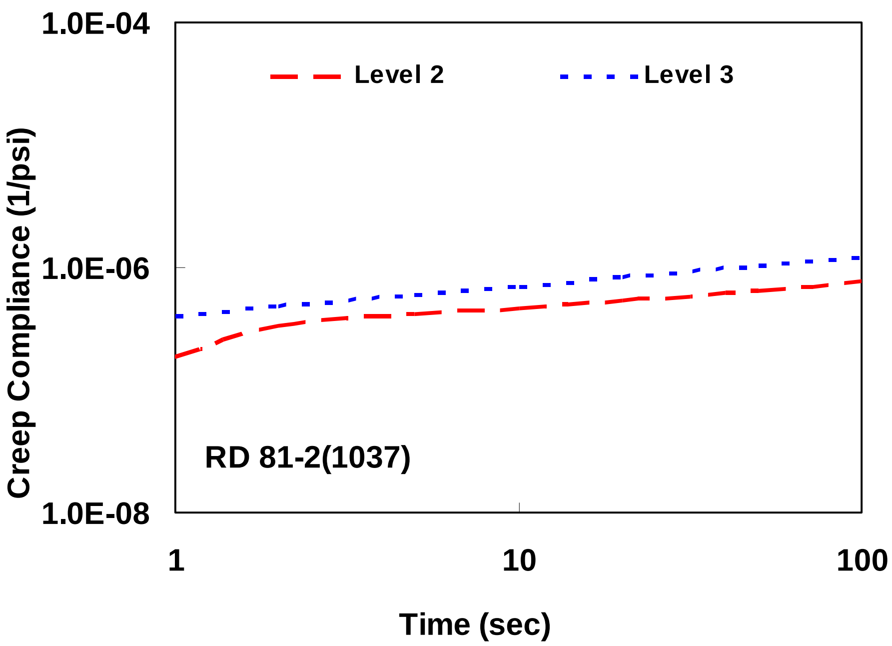
(f) SPL: STPD 79-2(102)



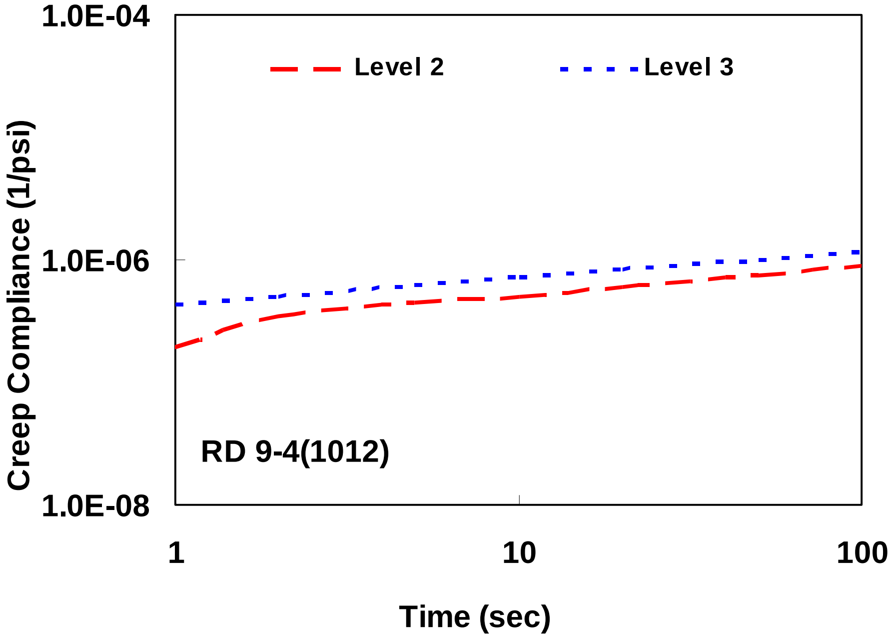
(g) SPL: STP 91-3(107)



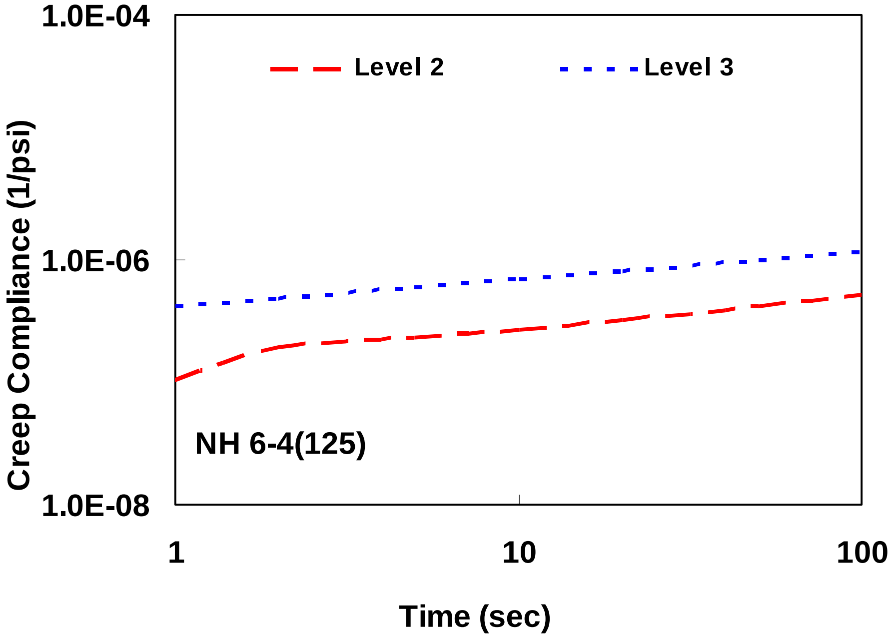
(h) SPL: NH 80-9 (832), (825), (827)



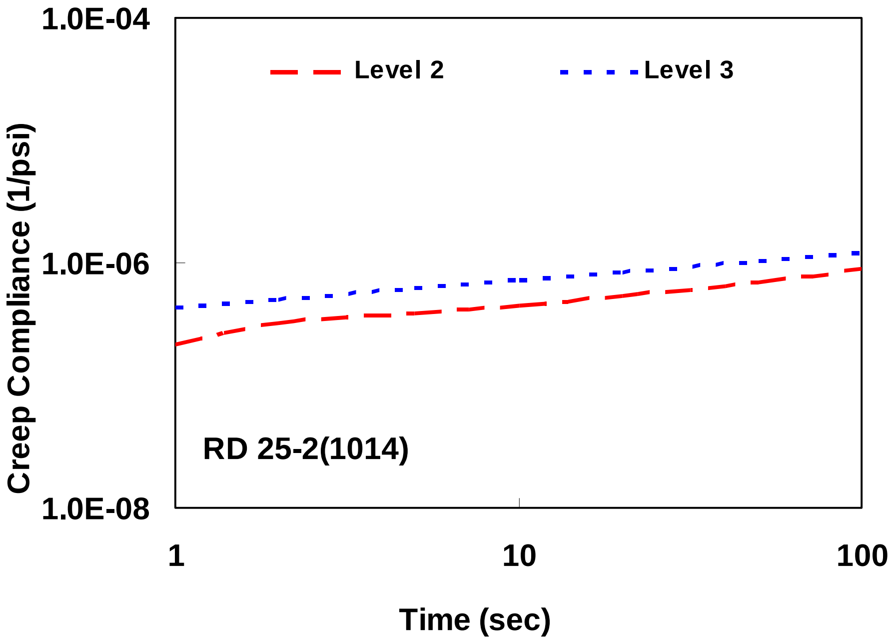
(i) SP4(0.375): RD 81-2(1037)



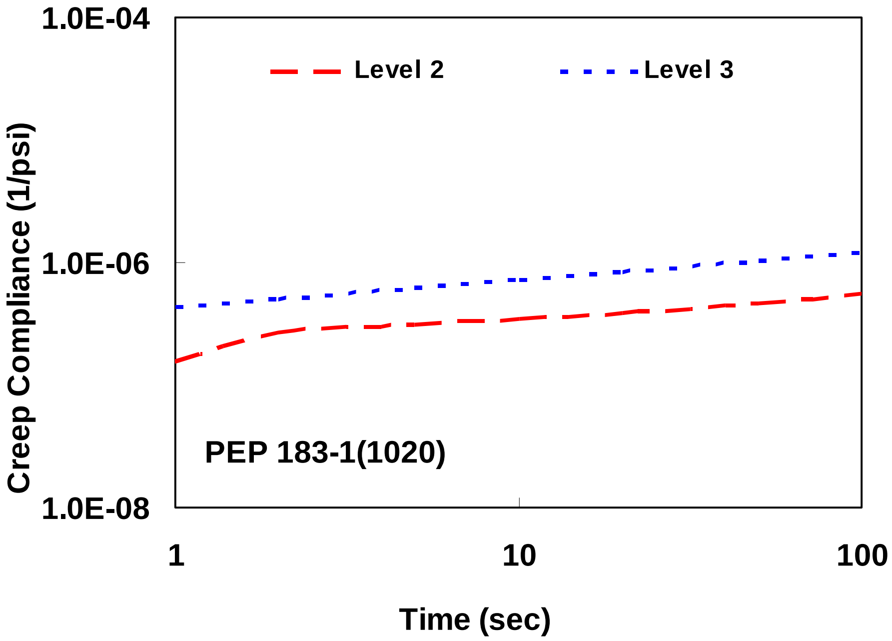
(j) SP4(0.375): RD 9-4(1012)



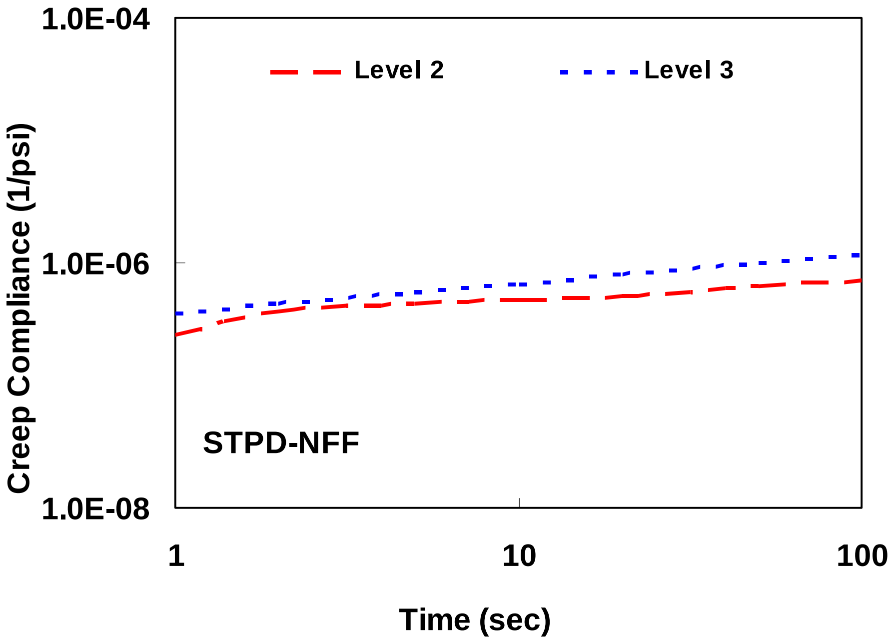
(k) SP4(0.375): NH 6-4(125)



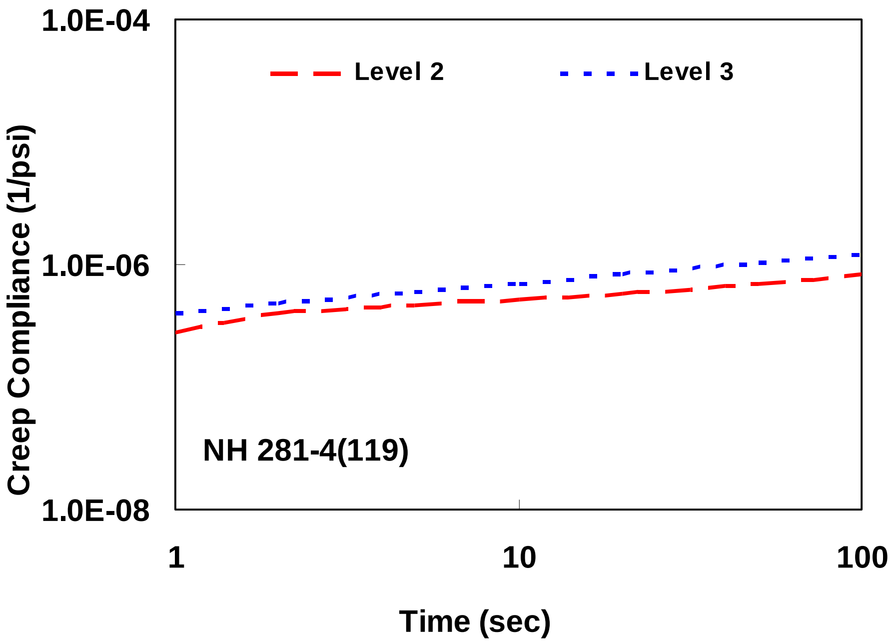
(l) SP4(0.375): RD 25-2(1014)



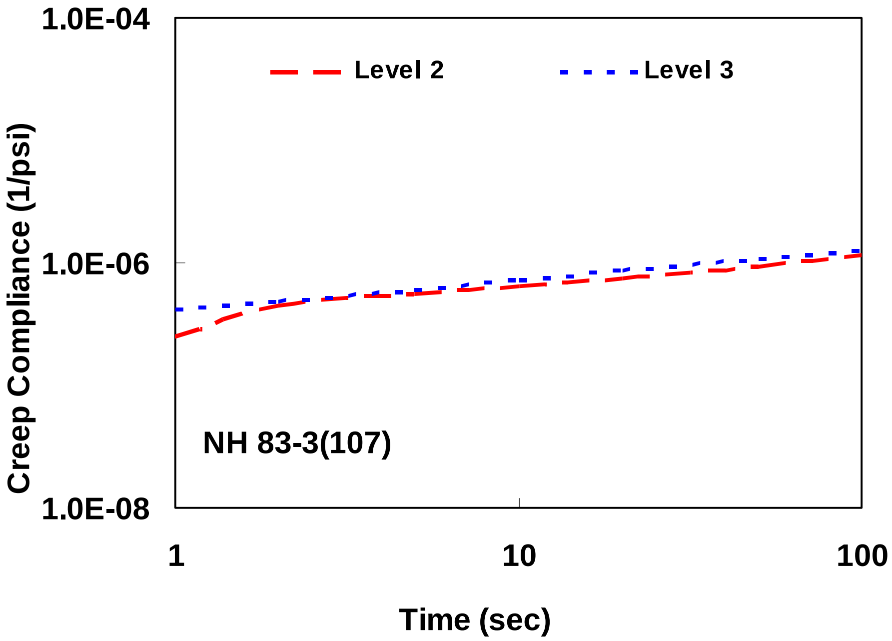
(m) SP4(0.5): PEP 183-1(1020)



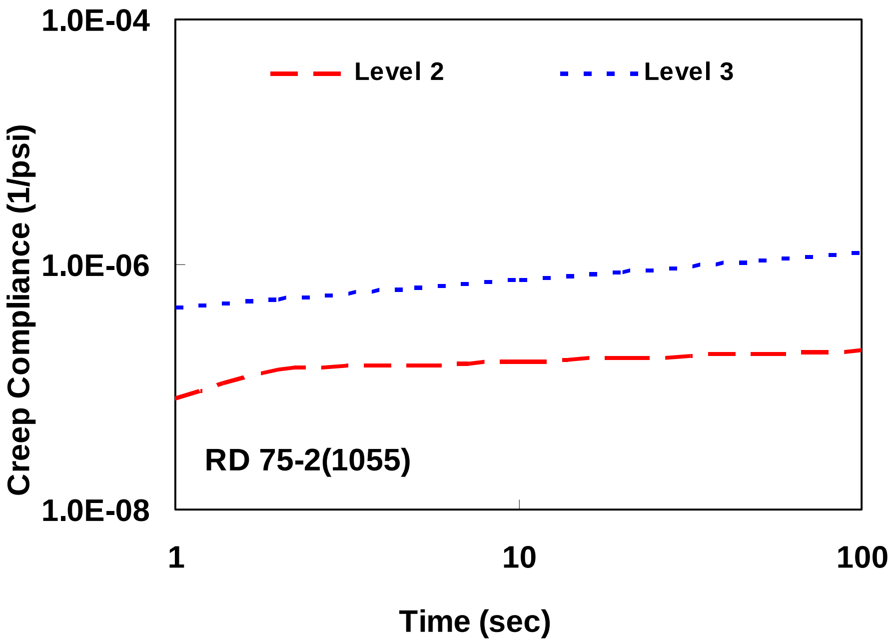
(n) SP4(0.5): STPD-NFF 11-2(115)



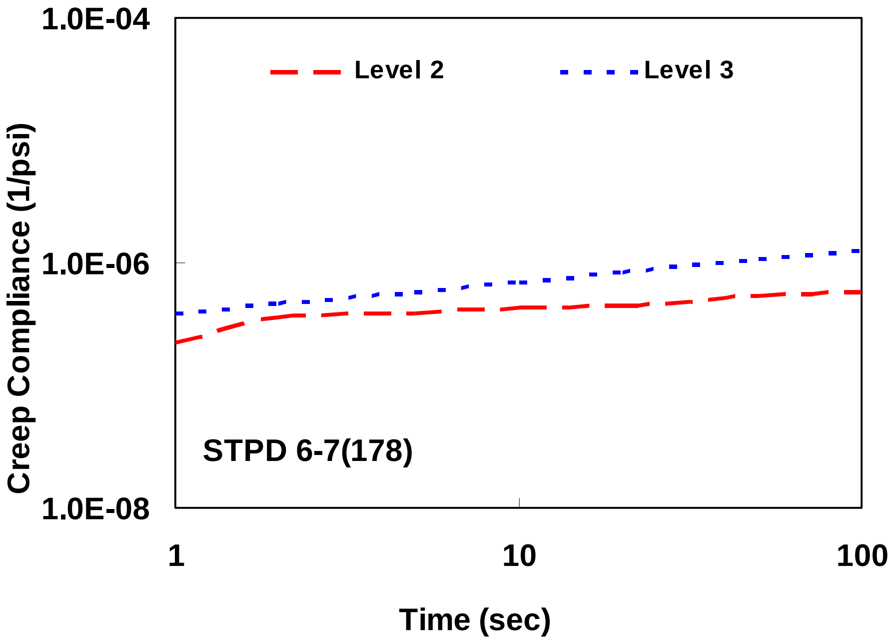
(o) SP4(0.5): NH 281-4(119)



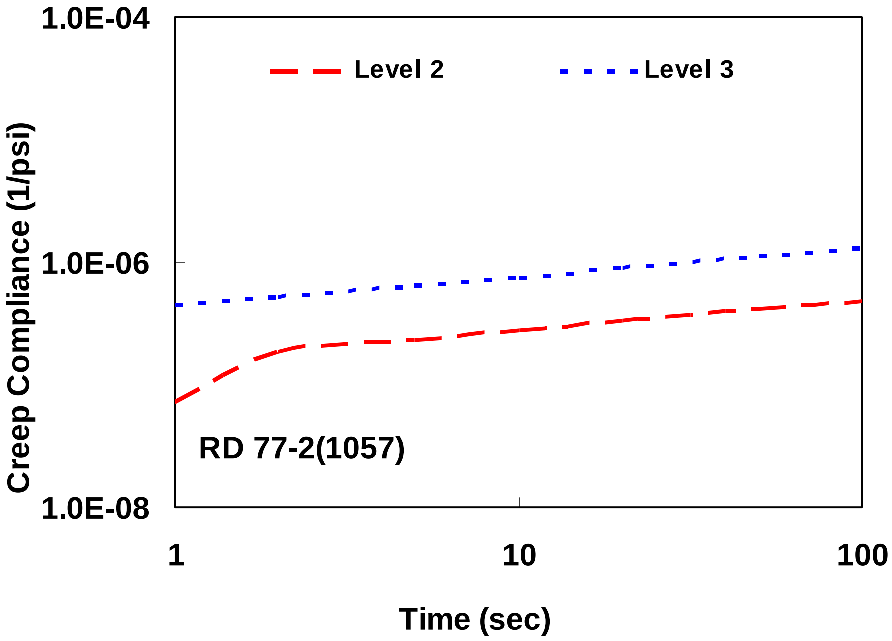
(p) SP4(0.5): NH 83-3(107)



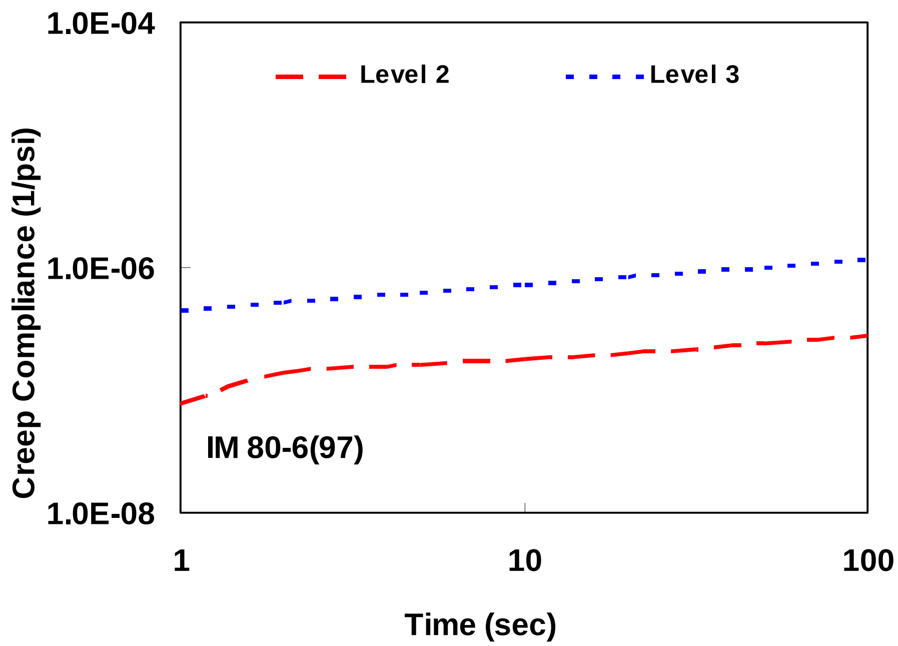
(q) SP5: RD 75-2(1055)



(r) SP5: STPD 6-7(178)



(s) SP5: RD 77-2(1057)



(t) SP5: IM 80-6(97)

Figure 4-13. Creep Compliance Results: Level 2 vs. Level 3

4.2 TESTS AND RESULTS OF SUBGRADE SOILS

Layer stiffness characteristics of subgrade soils in the MEPDG analysis is represented by resilient modulus. As mentioned earlier, the triaxial resilient modulus test is conducted for Level 1 analysis, whereas basic physical properties of soils such as specific gravity, Atterberg limits, and particle size gradations are used as necessary information to conduct Level 2 or 3 analysis.

Three native unbound soils (loess, loess/till, and sandy silt) were selected for this research as representative subgrade soils often used in Nebraska pavements. They were tested to evaluate all aforementioned physical properties and resilient modulus characteristics so that all three levels of MEPDG analysis could be performed. In addition to the three unbound soils, nine stabilized soils (loess, till, and shale stabilized with hydrated lime, fly ash and cement kiln dust, respectively) that were studied by Hensley et al. (2007) for a previous NDOR research project were also included in this

study to characterize their resilient modulus properties. Hensley et al. (2007) tested the nine stabilized soils compacted with an optimum amount of different types of pozzolans. The three unbound soils and the nine stabilized soils are expected to provide a more general and comprehensive resilient modulus database of the types of subgrade soils that are often applied to various Nebraska pavement projects.

4.2.1 Physical properties of unbound soils

Table 4-8 summarizes the physical property tests considered, their standard methods used, and test results for the three unbound soils: loess, loess/till, and sandy silt. All tests were performed at the UNL soils laboratory, and representative soil samples were then sent to NDOR geotechnical laboratory for validation. As can be seen in the table, physical properties obtained from UNL laboratory were very close to NDOR measurements.

Table 4-8. Summary of Physical Property Tests and Results of Three Unbound Soils

Physical Property	Standard Method	Sandy Silt		Loess		Loess/Till	
		UNL	NDOR	UNL	NDOR	UNL	NDOR
Specific Gravity	AASHTO T100	2.61	N/A	2.65	N/A	2.71	N/A
Liquid Limit	AASHTO T89	28	29	25	25	40	41
Plastic Limit	AASHTO T90	20	21	22	23	19	20
Plasticity Index	AASHTO T90	8	8	3	2	21	21
Ret. % Sieve No.200	AASHTO T88	37	40	9	10	0.5	1
Group Classification	AASHTO M145	A-4	A-4	A-4	A-4	A-6	A-6

4.2.2 Standard proctor test results of unbound soils

The optimum moisture content and the maximum dry unit weight were determined by performing compaction tests on each soil based on the standard testing method, AASHTO T99: Moisture-Density Relations of Soils Using a 5.5 lb Rammer and a 12 in. Drop. Soils were compacted using a mechanical compactor to produce cylindrical specimens of 4 in. (100 mm) in diameter and 4 in. (100 mm) high. The test results were then plotted on a dry unit weight vs. moisture content diagram as shown in Figure 4-14.

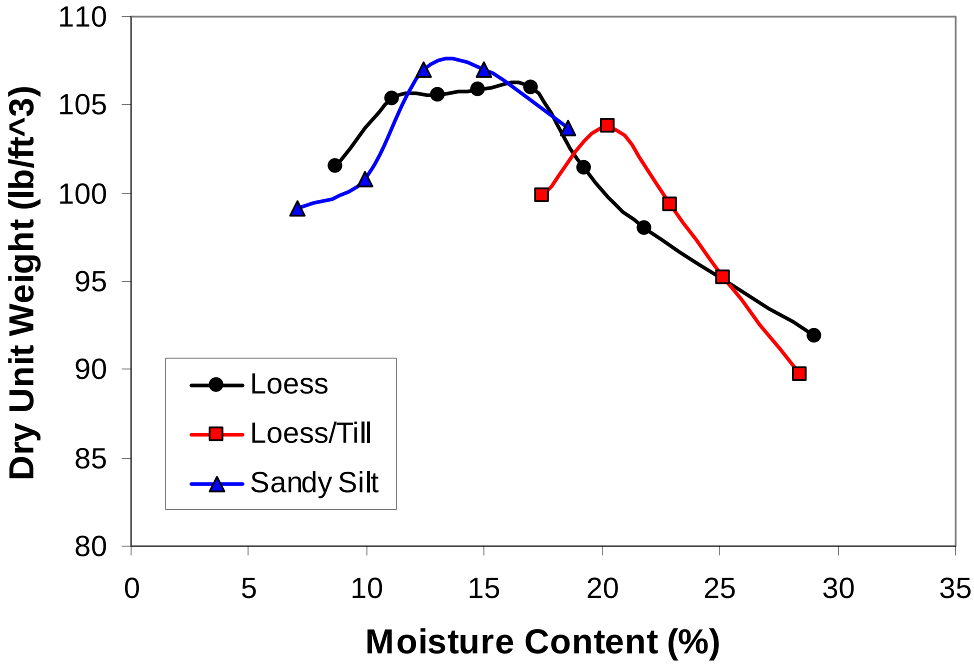


Figure 4-14. Plots of Compaction Curves

The curve connecting the data points represents the dry unit weight achieved by compacting the soil at various moisture contents. Higher dry unit weight values indicate higher quality fill, so there is a certain moisture content, known as the optimum moisture content that produces the greatest dry unit weight. The greatest dry unit weight is called the maximum dry unit weight. Table 4-9 presents the optimum moisture content and the corresponding maximum dry unit weight of the three unbound soils, determined from Figure 4-14.

Table 4-9. Summary of Standard Proctor Test Results

Unbound Soil	Loess	Loess/Till	Sandy Silt
Optimum Moisture Content (%)	16.5	20.3	13.0
Maximum Dry Unit Weight (lb/ft^3)	106	104	108

4.2.3 Resilient modulus test of unbound soils

The resilient modulus represents the elastic response of a material under simulated repeated traffic loading. Most paving materials are known not to be elastic but instead they deform plastically after each load application. However, if the load is small compared to the strength of the material and is repeated for a large number of times, the deformation under each load application is almost completely recoverable and proportional to the load, so that it can be considered as elastic (Huang 1993). The response of a soil specimen under repeated load is illustrated in Figure 4-15. As shown in the figure, the total strain is composed of plastic strain, which is called permanent strain, and elastic strain. Considerable plastic strain occurs during the initial loading stage, but as the number of repetition increases, the increasing rate of plastic strain decreases. After 150 to 200 load repetitions, the cumulative plastic strain approaches a constant level. The resilient modulus is defined as elastic modulus based on recoverable (resilient) strain under repeated loads, expressed by:

$$M_R = \frac{\sigma_d}{\varepsilon_r} \qquad [4.12]$$

where M_R = resilient modulus,

σ_d = deviator stress, and

ε_r = recoverable (resilient) strain.

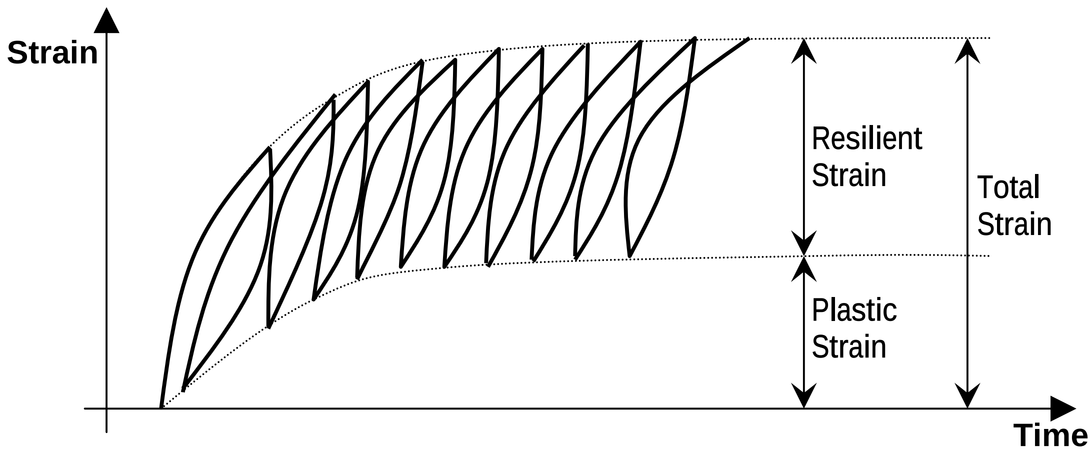


Figure 4-15. General Response of a Soil Specimen under Repeated Load

The deviator stress is the axial stress in an unconfined compression test or the axial stress in excess of the confining pressure in a triaxial compression test. Figure 4-16 shows confining pressure (σ_c) and deviator stress (σ_d) for a cylindrical specimen in a triaxial test.

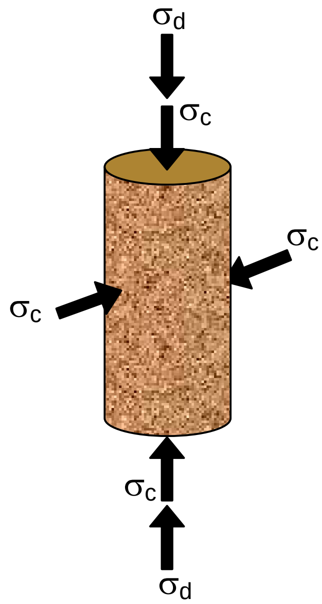


Figure 4-16. Confining Stress and Deviator Stress on a Triaxial Cylindrical Specimen

In the laboratory, the resilient modulus can be determined from triaxial, repeated load testing at a given confining pressure and temperature. Figure 4-17 shows the resilient modulus testing setup for cylindrical specimens (4 inch in diameter and 8 inch in height). The testing specimens were compacted at the optimum moisture content, which was pre-determined from the standard proctor compaction test (Table 4-9). The resilient modulus test was performed following the standard test method, AASHTO T307-99: Determining the Resilient Modulus of Soils and Aggregate Materials. It should be noted that difficulties were encountered in performing the resilient modulus test of loess soil. As presented in Figure 4-18, loess specimens were significantly deformed during the test, which resulted in erroneous measurements. The large deformation of specimens is not desirable since the resilient modulus test is to capture elastic stiffness characteristics of soils. Therefore, resilient modulus test was performed only for the two unbound soils, loess/till and sandy silt.

Figure 4-17. Resilient Modulus Testing Setup (AASHTO T307)

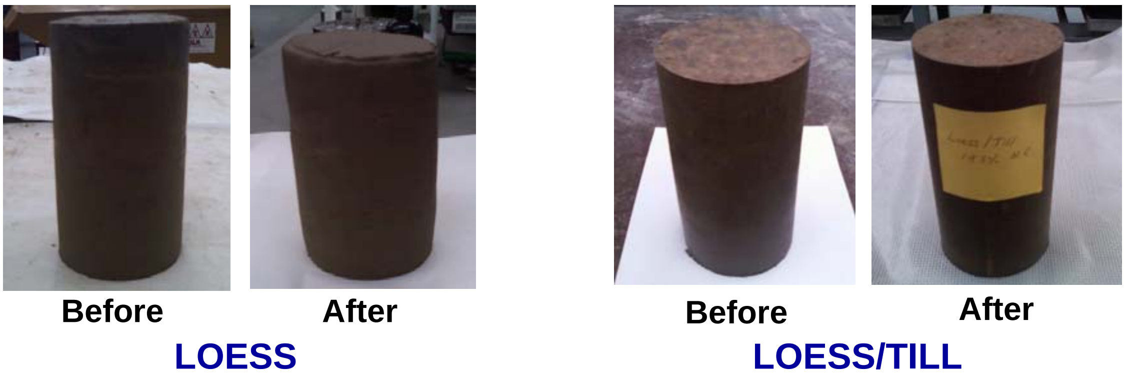


Figure 4-18. Specimens before and after Resilient Modulus Testing

Following the standard method, AASHTO T307-99, each soil specimen was prepared by hand mixing at the optimum moisture content. The moistened soil was then cured for 24 hours in a sealed plastic bag before it was compacted to produce cylindrical specimens. After compaction, a latex membrane was sealed onto the specimen surface to apply pre-conditioning process and designated series of confining pressure and deviator stress. For each specimen, the resilient modulus was determined for fifteen consecutive stress states at confining pressure ranged from 2 to 6 psi and deviator stress between 2 and 10 psi. Table 4-10 presents the fifteen combinations of confining pressure and deviator stress specified in the testing protocol: AASHTO T307.

Table 4-10. Combinations of Confining Pressure and Deviator Stress Applied

Sequence No.	Confining Pressure (psi)	Deviator Stress (psi)	Cyclic Stress (psi)	Constant Stress (psi)	No. of Load Applications
1	6.0	2	1.8	0.2	100
2	6.0	4	3.6	0.4	100
3	6.0	6	5.4	0.6	100
4	6.0	8	7.2	0.8	100
5	6.0	10	9.0	1.0	100
6	4.0	2	1.8	0.2	100
7	4.0	4	3.6	0.4	100
8	4.0	6	5.4	0.6	100
9	4.0	8	7.2	0.8	100
10	4.0	10	9.0	1.0	100
11	2.0	2	1.8	0.2	100
12	2.0	4	3.6	0.4	100
13	2.0	6	5.4	0.6	100
14	2.0	8	7.2	0.8	100
15	2.0	10	9.0	1.0	100

4.2.4 Resilient modulus test results of unbound soils

Figure 4-19 shows representative resilient modulus test results from specimen No. 1 of loess/till soil. The figure clearly demonstrates that the resilient modulus of the soil is a function of both the confining pressure and the deviator stress, which infers that the soil stiffness is stress state dependent.

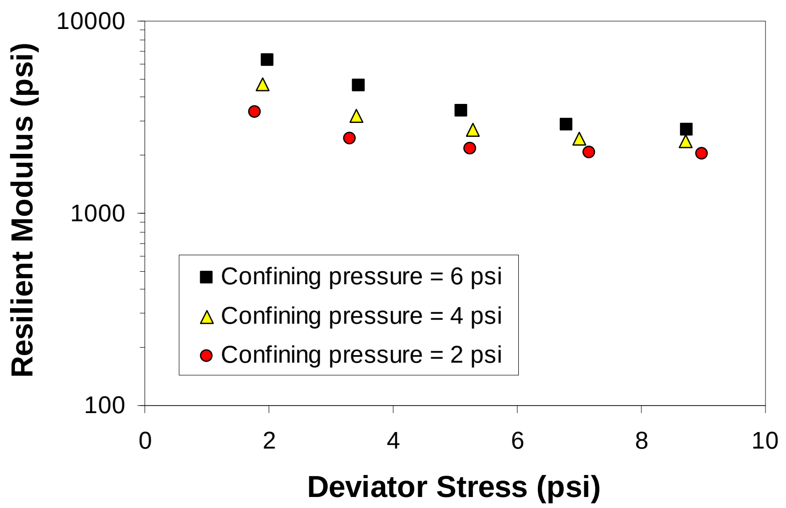


Figure 4-19. Resilient Modulus Test Results of Loess/Till Soil Specimen

Stress states (i.e., confining pressure and deviator stress) used for the resilient modulus test are based on the depth at which the soils are located within the pavement structure and the traffic loads applied to the pavement structure. In the MEPDG, the stress-dependent resilient modulus of soils is characterized using a generalized constitutive model. The nonlinear elastic coefficients and exponents of the generalized constitutive model are determined through nonlinear regression analyses by fitting the model to laboratory resilient modulus test results. The generalized constitutive model used in the MEPDG design procedure is as follows:

$$M_R = k_1 P_a \left(\frac{\theta}{P_a} \right)^{k_2} \left(\frac{\tau_{oct}}{P_a} + 1 \right)^{k_3} \qquad [4.13]$$

where M_R = resilient modulus,

θ = 1st stress invariant = $3\sigma_c + \sigma_d$,

σ_c and σ_d = confining stress and deviator stress, respectively,

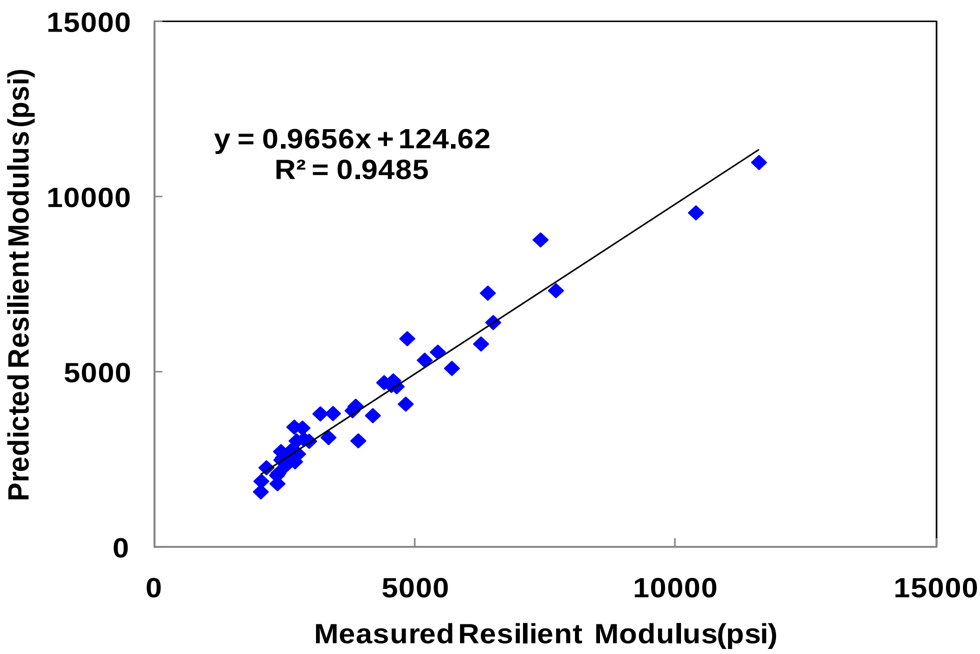
P_a = atmospheric pressure (101.3 kPa or 14.7 psi),

τ_{oct} = octahedral shear stress which is equal to $\left(\sqrt{2}/3\right)\sigma_d$, and

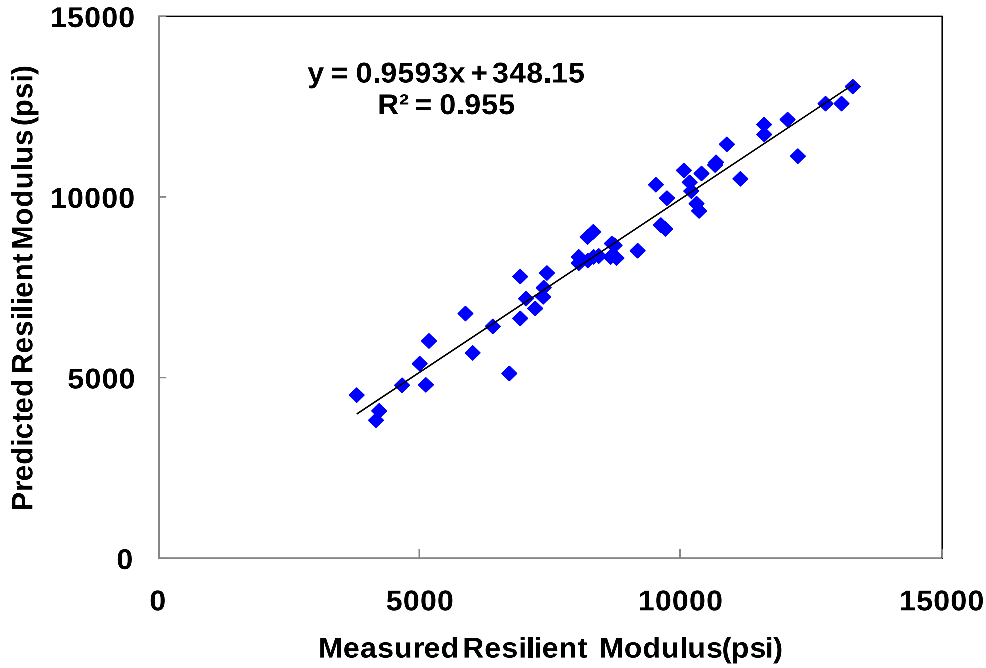
k_1, k_2, k_3 = model parameters.

The constitutive model parameters (k_1, k_2, k_3) for each test material should be determined with a high level of correlation to test data. Generally, R^2-value (called a coefficient of determination) exceeding 0.90 is recommended. To obtain model parameters of each soil in a more general sense, resilient modulus test results of multiple specimens (i.e., three for loess/till and four specimens for sandy silt) were used together. Figure 4-20 presents cross-plots between measured moduli and predicted moduli using the model (Equation [4.13]) after finding the three model parameters. As indicated by the coefficient of determination (R^2) values, the model fits test results very well, which implies that the model can be appropriately used to represent stress-dependent behavior of each soil in a pavement structure.

Table 4-11 presents resulting model parameters. The parameter k_2 is positive, indicating that an increase in confinement causes an increase in the modulus, while the parameter k_3 is negative, indicating that an increase in the deviator stress causes a reduction in the resilient modulus. The work by Uzan (1985) has shown that the decrease in resilient modulus with an increase in deviator stress occurs when the ratio of the major principal stress to minor principal stress is lower than 2 or 3 depending on the soil type. Notably, the input data required for the Level 1 MEPDG analysis are not the actual resilient modulus test data but the three model parameters. Therefore, the nonlinear regression process to identify the model parameters needs to be conducted to operate the Level 1 analysis.



(a) Loess/Till



(b) Sandy Silt

Figure 4-20. Predicted Moduli vs. Measure Moduli

Table 4-11. Resulting Model Parameters and R^2-value of Each Soil

	k_1	k_2	k_3	R^2
Loess/Till	723.3492	0.580731	-6.79546	0.949
Sandy Silt	772.2054	0.474492	-2.12098	0.955

In addition to the two native unbound soils (i.e., loess/till and sandy silt) tested for the Level 1 resilient modulus characterization, as previously mentioned, the resilient modulus characteristics were also determined for the nine stabilized soils (loess, till, and shale stabilized with hydrated lime (HL), fly ash (FA), and cement kiln dust (CKD), respectively) that were studied by Hensley et al. (2007) for a previous NDOR research project. Raw test data presented in Hensley et al. (2007) were used, and the resulting Level 1 model parameters are summarized in Table 4-12. The database presented in Tables 4-11 and 4-12 and in Appendix 2 is expected to provide a general input set of subgrade soils that are often used in Nebraska pavement projects.

Table 4-12. Level 1 Resilient Modulus Model Parameters of Nine Stabilized Soils

	Loess			Till			Shale		
	7% CKD	12% FA	5% HL	7% CKD	12% FA	5% HL	7% CKD	14% FA	6% HL
k_1	1985.2	802.4	1109.3	2564.5	1864.2	2061.8	2007.3	1063.5	1823.0
k_2	0.367	0.392	0.414	0.467	0.420	0.311	0.395	0.455	0.364
k_3	-1.081	-2.597	-2.601	-0.975	-0.917	-0.843	-0.744	-2.431	-1.219
R^2	0.971	0.995	0.951	0.857	0.936	0.969	0.970	0.930	0.970

4.2.5 Resilient modulus values for Level 2 MEPDG analysis

When the Level 1 resilient modulus laboratory test (AASHTO T307) is not performed, the user is then able to consider Level 2 analysis using the relationships between resilient modulus and other soil properties, such as the California bearing ratio (CBR) or R-value. Table 4-13 shows these types of correlations with other soil characteristics. Accordingly, the Level 2 resilient modulus is not stress-dependent, but instead is a constant value. Table 4-14 presents a single resilient modulus value for each soil considered in this research for the Level 2 MEPDG analysis.

Table 4-13. Models Relating Material Properties to M_R (NCHRP 1-37A 2004)

	Model	Comments	Test Standard
CBR	M_R (psi) = 2555(CBR)$^{0.64}$(TPL)	CBR = California Bearing Ratio	AASHTO T193
R-value	M_R (psi) = 1155+555R(20)	R = R value	AASHTO T190
AASHTO layer coefficient	M_R (psi) = 30000(a_i/0.14)(20)	a_i = AASHTO layer coefficient	AASHTO Guide for the Design of Pavement
PI and gradation	CBR (%) = 75/{1+0.728(wPI)}	wPI = P200*PI P200 = % passing No. 200 sieve, PI = plasticity index (%)	AASHTO T27, AASHTO T90
DCP	CBR (%) = 292/DCP$^{1.12}$	CBR= California Bearing Ratio, DCP = DCP index (mm/blow)	ASTM D6951

Table 4-14. Level 2 Resilient Modulus Value of Each Soil

				Loess/Till			Sandy Silt		
M_R (psi)				3098.9			7170.5		
	Loess			Till			Shale		
	7% CKD	12% FA	5% HL	7% CKD	12% FA	5% HL	7% CKD	14% FA	6% HL
M_R (psi)	22370.5	7051.6	9688.4	28652.4	21273.9	24479.7	23698.6	9445.4	20108.9

4.2.6 Resilient modulus values for Level 3 MEPDG analysis

For input Level 3, typical resilient modulus values presented in Table 4-15 are provided by MEPDG software as national default values. Table 4-15 summarizes default resilient modulus values of each soil based on its classification (standard AASHTO and USC: unified soil classification). As mentioned in the guide (NCHRP 1-37A 2004), significant caution is advised for the use of the resilient modulus values in the table since they are very approximate. Levels 1 and 2 testing are preferred, if possible.

Table 4-15. Typical M_R Values for Unbound Granular and Subgrade Materials

Soil Classification	M_R Range (psi)	Typical M_R (psi)
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
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	25,000
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	39,500 - 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

Table 4-16 summarizes resilient modulus values of five unbound soils (three native soils primarily tested in this research and the two soils studied by Hensley et al. 2007) determined based on their classification. Group classifications of individual soils are also presented in the table.

Table 4-16. Level 3 Resilient Modulus Values Based on Group Classification

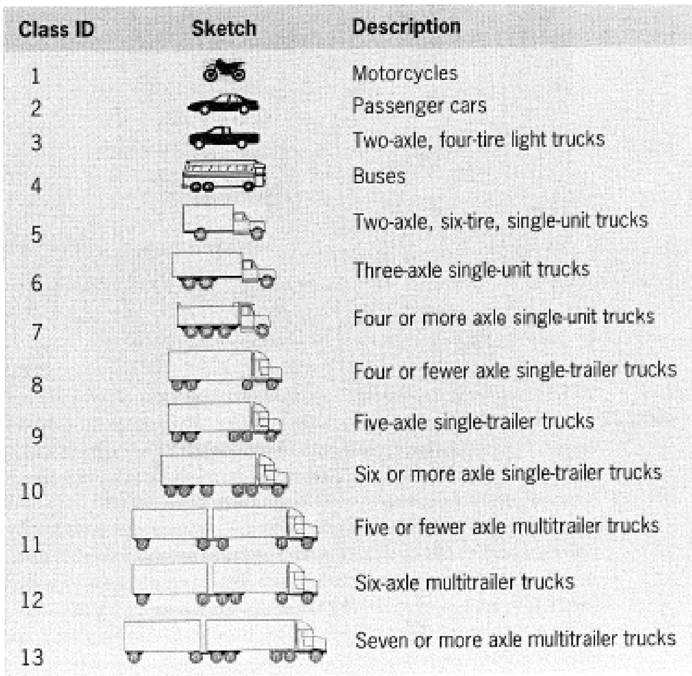
Type of Soil	Sandy Silt	Loess	Loess/Till	Till	Shale
Group Classification	A-4	A-4	A-6	A-6	A-7-5
M_R (psi)	16,500	16,500	14,500	14,500	13,000

CHAPTER 5
MEPDG SENSITIVITY ANALYSIS

In this chapter, MEPDG sensitivity analyses were conducted to evaluate the effects of using different design level inputs on MEPDG performance predictions of asphalt pavement structures. Each design level input of asphalt and soil materials presented in the previous chapter was used for the MEPDG analyses, and resulting performance between levels were compared to examine sensitivity of MEPDG performance prediction depending on input levels. To this end, the sensitivity analysis was conducted for typical full-depth flexible pavement structures that have usually been implemented in Nebraska. Different levels of layer properties and material characteristics presented in the database were incorporated with the typical full-depth pavement structures to examine MEPDG performance sensitivities relating to the input level of layer moduli. The most recent version (1.10) of MEPDG software was used for simulations.

5.1 SENSITIVITY ANALYSIS OF TYPICAL PAVEMENT STRUCTURES

Nebraska flexible pavements are generally full-depth pavements with a design based on the 1993 AASHTO Pavement Design Guide. When a new flexible pavement is designed, the volume of heavy trucks (vehicle Class 4 to 13 shown in Figure 5-1) expected on the specific project site is the primary factor considered for determining the pavement structure geometry with its type of HMA mixture. In cases where fewer than 200 heavy trucks per day are expected, a minimum HMA layer thickness of 8 in. is usually applied. If more than 200 heavy trucks per day are expected, the minimum HMA layer thickness is 10 in., while a minimum HMA layer thickness of 12 in. is necessary for the cases with more than 1,500 heavy trucks per day.



Class ID	Sketch	Description
1		Motorcycles
2		Passenger cars
3		Two-axle, four-tire light trucks
4		Buses
5		Two-axle, six-tire, single-unit trucks
6		Three-axle single-unit trucks
7		Four or more axle single-unit trucks
8		Four or fewer axle single-trailer trucks
9		Five-axle single-trailer trucks
10		Six or more axle single-trailer trucks
11		Five or fewer axle multitrailer trucks
12		Six-axle multitrailer trucks
13		Seven or more axle multitrailer trucks

Figure 5-1. FHWA Vehicle Classification

The type of HMA mixture is also based on the volume of heavy trucks. In general, SPR mixtures have been used as base asphalt mixtures or surface layer mixtures for Nebraska highways subject to fewer than 200 trucks per day. SP4 Special mixtures are typically used for surface layers for low volume highways with 200-500 trucks per day, and SP4 mixtures are applied to asphalt surface layers of pavements where 500-1,500 trucks are expected per day. SP5 mixtures are typically used for high volume highways with more than 1,500 heavy trucks traveling daily. Finally, SPL and HRB mixtures are usually used as base layer materials. An approximately 8 in. thick subgrade layer is then placed under the asphalt layers. The subgrade materials are usually stabilized with fly ash or hydrated lime.

Figure 5-2 presents three typical full-depth asphalt pavement structures in Nebraska for the three different levels of traffic volume (i.e., fewer than 200, 200-1,500, and more than 1,500). Two pavement structures, (b) and (c) shown in Figure 5-2, were selected in order to conduct the first sensitivity analysis which is to investigate MEPDG performance predictions resulting from different input levels for typical full-depth pavement structures.

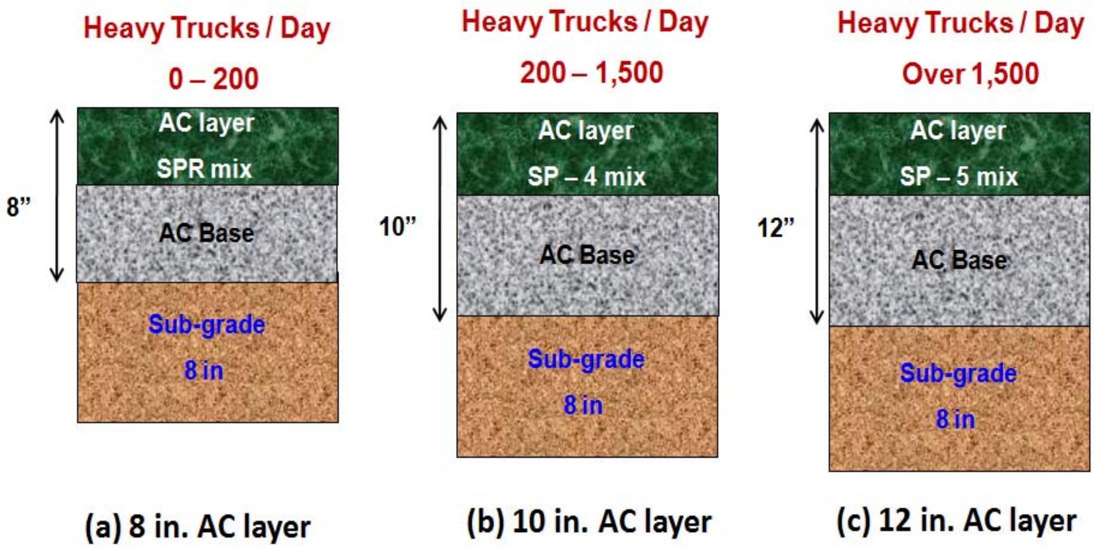


Figure 5-2. Typical Full-Depth Asphalt Pavement Structures Used in Nebraska

5.1.1 Design inputs for the sensitivity analysis

Table 5-1 shows a summary of design input parameters used for the sensitivity analysis. All pavement performance indicators, such as fatigue cracking, thermal cracking, rutting, and IRI, were predicted for the 20-year design period with a design reliability level of 90%. The same operation speed of 60 mph was chosen for each simulation with a total of 1,500 trucks and 3,000 trucks per day applied to pavement structures (b) and (c), respectively. The location of project sites was assumed to be Lincoln, Nebraska. One SP4(0.5) mixture (i.e., NH 281-4(119) project) in the asphalt database was selected to represent the 4 in. thick HMA surface layer of the pavement structure (b), and a SP5 mixture (IM 80-6(97) project) was used to represent the 4 in. surface layer of pavement structure (c). For an asphalt base layer of both structures, one of HRB mixtures in the database was used with different layer thicknesses (6 in. for structure (b) and 8 in. for structure (c)) as shown in the table. To represent the subgrade layer, resilient modulus values of shale stabilized with 14% fly ash were used for pavement (b), while resilient moduli of till with 12% fly ash were used for the analysis of pavement structure (c). Table 5-1 also shows performance criteria.

Table 5-1. Design Input Parameters for MEPDG Sensitivity Analysis

	Pavement Structure (b)	Pavement Structure (c)
Design Period (year)	20	
Operation Speed (mph)	60	
Design Reliability (%)	90	
Project Location	Lincoln, NE	
Daily Heavy Trucks	1,500	3,000
Surface Asphalt Mixture	SP4(0.5) mixture NH 281-4(119) project 4-in. thickness	SP5 mixture IM 80-6(97) project 4-in. thickness
Base Asphalt Mixture	HRB mixture NH 6-4(125) project 6-in. thickness	HRB mixture NH 6-4(125) project 8-in. thickness
Type of Subgrade	Shale with fly ash of 14% - M_R = 9,445 psi (Level 2) - M_R = 13,000 psi (Level 3)	Till with fly ash of 12% - M_R = 21,274 psi (Level 2) - M_R = 14,500 psi (Level 3)
Performance Criteria	• Initial IRI (in/mile): 63 • Terminal IRI (in/mile): 172 • AC surface down cracking (ft/mile): 2,000 • AC bottom up cracking (%): 25 • AC thermal cracking (ft/mile): 1,000 • AC Permanent deformation (in): 0.25 • Total permanent deformation (in): 0.75	

5.1.2 MEPDG simulations and results

As can be implied from Table 5-1, all three hierarchical levels of inputs can be applied to each layer for the MEPDG sensitivity simulations. However, Level 1 simulations for subgrade soils were not conducted in this study, because it is not recommended by the MEPDG software: it needs more than 40 hours to complete a 20-year design analysis. Thus, a total of 36 simulations (18 simulations for each structure) were accomplished as presented in Table 5-2. Simulation results for various pavement performance indicators, including the longitudinal cracking, alligator cracking, thermal cracking, asphalt rutting, total rutting, and IRI, were compared to investigate input level dependent performance of the two typical Nebraska flexible pavement structures.

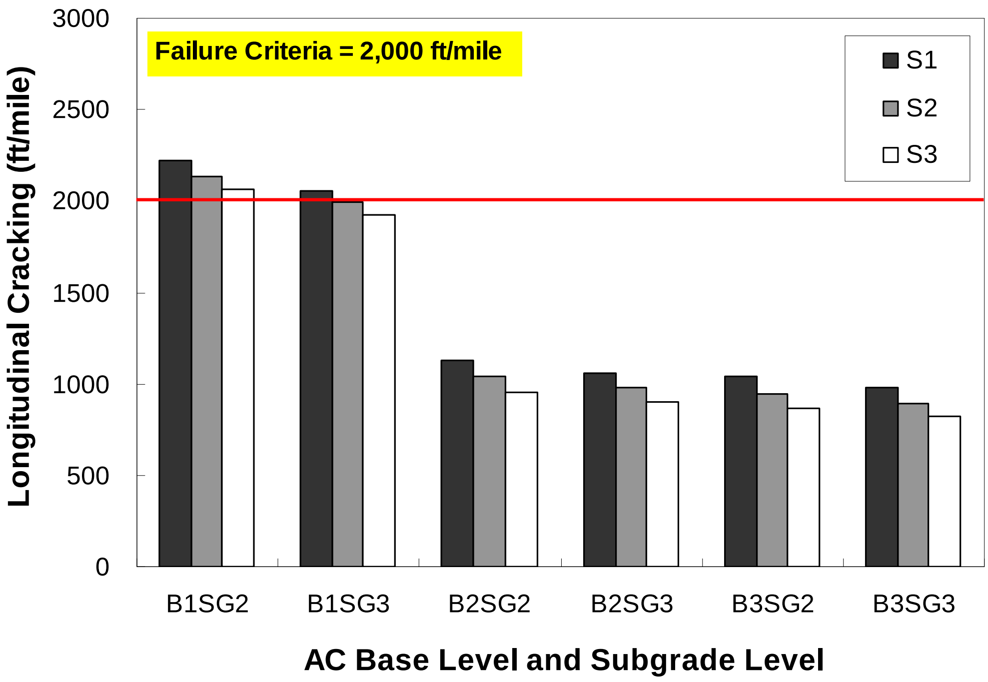
Table 5-2. Input Level Combinations Applied to Original Structures

<table>
<tr><th>Case</th><th>Level of Surface HMA</th><th>Level of Base HMA</th><th>Level of Subgrade</th></tr>
<tr><td>1</td><td rowspan="6">1 (denoted as S1)*</td><td rowspan="2">1 (denoted as B1)</td><td>2 (denoted as SG2)</td></tr>
<tr><td>2</td><td>3 (denoted as SG3)</td></tr>
<tr><td>3</td><td rowspan="2">2 (denoted as B2)</td><td>2</td></tr>
<tr><td>4</td><td>3</td></tr>
<tr><td>5</td><td rowspan="2">3 (denoted as B3)</td><td>2</td></tr>
<tr><td>6</td><td>3</td></tr>
<tr><td>7</td><td rowspan="6">2 (denoted as S2)</td><td rowspan="2">1</td><td>2</td></tr>
<tr><td>8</td><td>3</td></tr>
<tr><td>9</td><td rowspan="2">2</td><td>2</td></tr>
<tr><td>10</td><td>3</td></tr>
<tr><td>11</td><td rowspan="2">3</td><td>2</td></tr>
<tr><td>12</td><td>3</td></tr>
<tr><td>13</td><td rowspan="6">3 (denoted as S3)</td><td rowspan="2">1</td><td>2</td></tr>
<tr><td>14</td><td>3</td></tr>
<tr><td>15</td><td rowspan="2">2</td><td>2</td></tr>
<tr><td>16</td><td>3</td></tr>
<tr><td>17</td><td rowspan="2">3</td><td>2</td></tr>
<tr><td>18</td><td>3</td></tr>
</table>

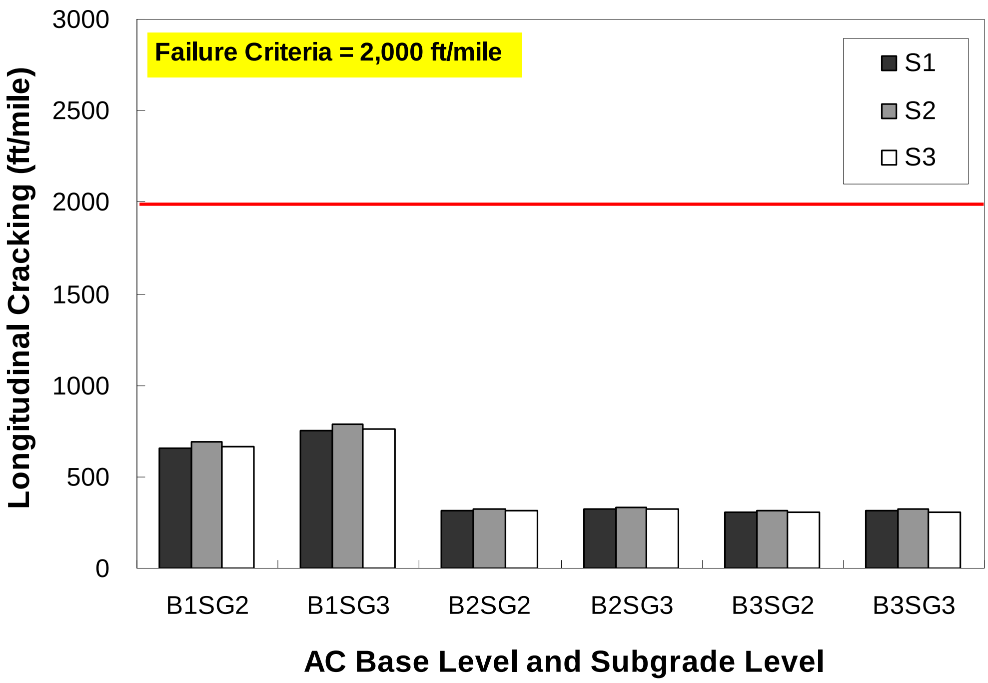
Note: *level 1 simulation of thermal cracking was not conducted because the creep compliance testing and the tensile strength test at 14 °F could not be performed, as mentioned earlier.

MEPDG simulation results are presented in Figure 5-3 to Figure 5-8 for each different performance indicator. In each figure, the predicted amount of pavement distress resulting from different combinations of design input levels (S, B, and SG as shown in Table 5-2) is plotted for the two different pavement structures: (b) and (c).

Figure 5-3 shows the amount and variation of predicted longitudinal cracking between different combinations of input levels. The longitudinal cracking performance was sensitively affected by the design inputs in this particular example. For both structures, the longitudinal cracking was strongly related to the input level of asphalt base layer, HRB mixture: NH 6-4(125). Simulation results from B1 cases clearly presented higher level of cracking than cases with B2 or B3. Based on the performance predictions shown in Figure 5-3 and the level-dependent dynamic modulus curves presented in Figure 4-9, it can be implied that surface cracking is not merely affected by surface layer properties, but also influenced by interlayer relationships.



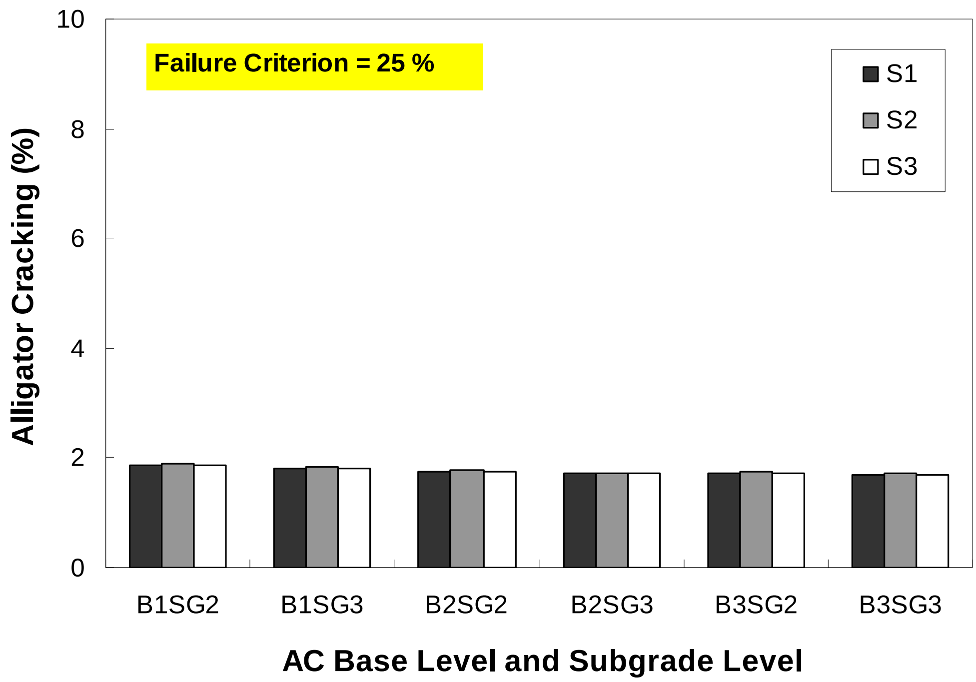
(a) Pavement Structure (b)



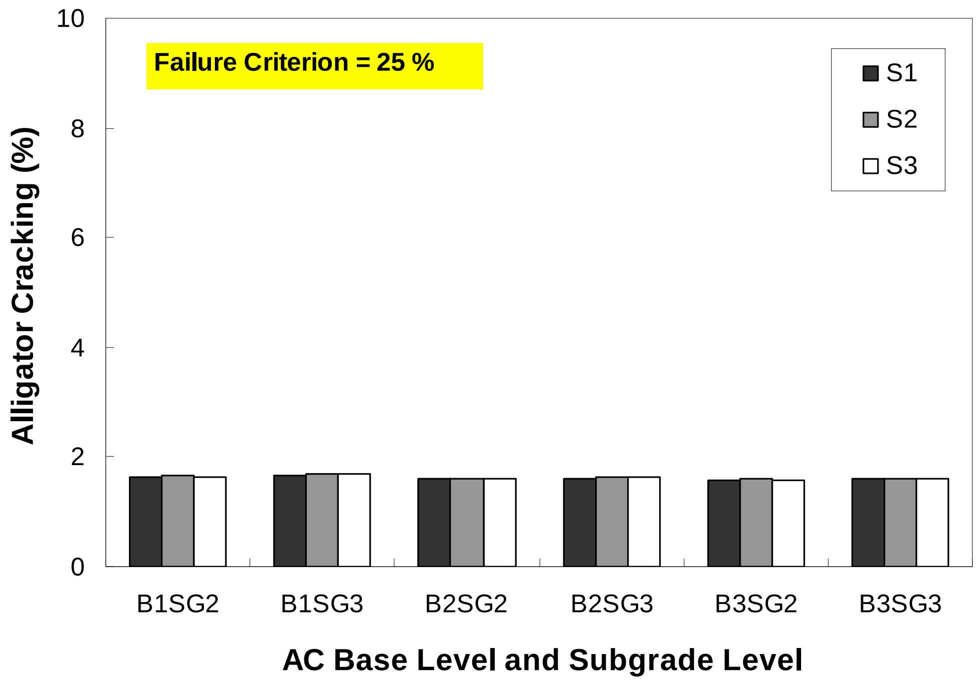
(b) Pavement Structure (c)

Figure 5-3. MEPDG Simulation Results of Longitudinal Cracking

Figure 5-4 shows simulation results of alligator cracking over a 20-year service. Alligator cracking is known to be sensitively affected by the stiffness and thickness of the asphalt surface layer. This is because the tensile strain at the bottom of the asphalt surface layer is used to estimate the predicted level of fatigue cracking in the MEPDG. Increasing the surface layer thickness can significantly reduce the tensile strain at the bottom of the surface layer and this consequently mitigates bottom-up fatigue cracking. As can be observed from the figure, for both structures, the amount of predicted alligator cracking at the 90% design reliability was very small compared to the typical alligator cracking failure criterion: 25%. In addition, no clear variation was observed with different combinations of input levels for the alligator cracking.



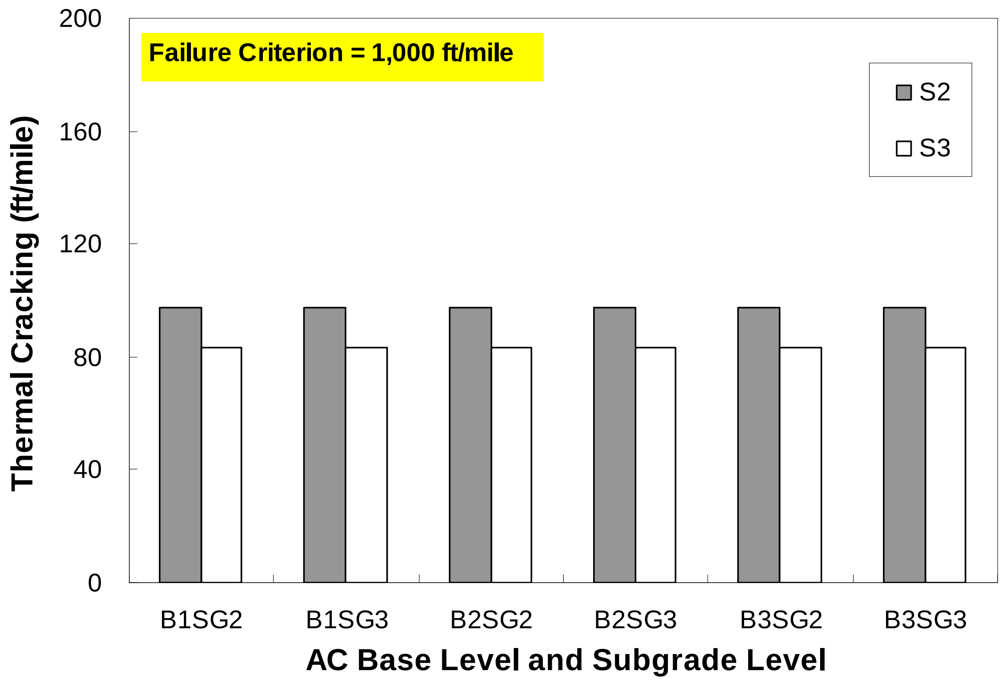
(a) Pavement Structure (b)



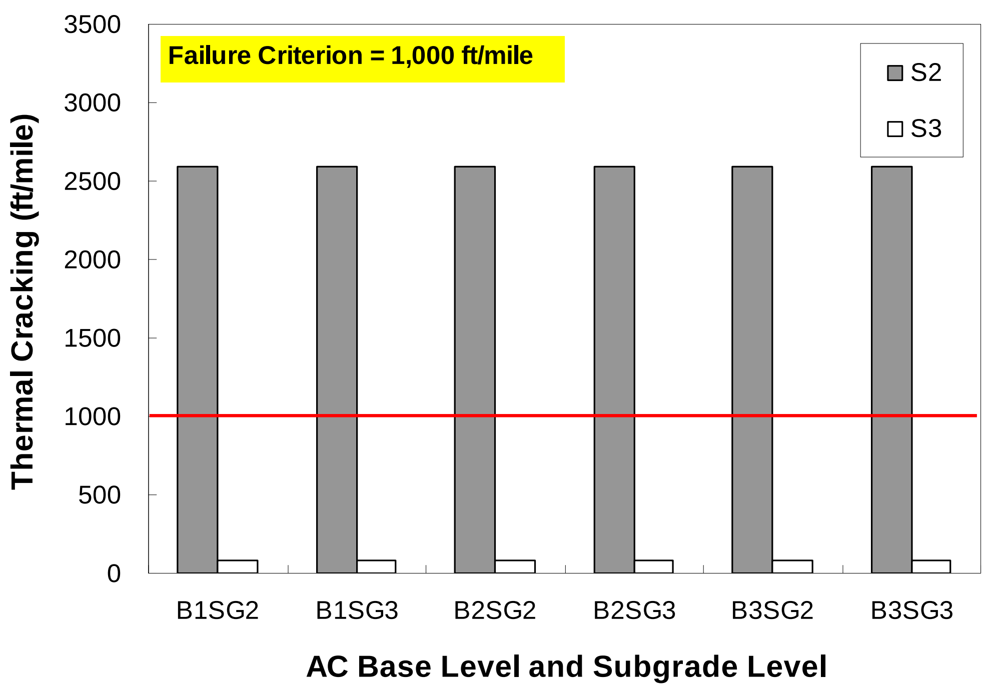
(b) Pavement Structure (c)

Figure 5-4. MEPDG Simulation Results of Alligator Cracking

MEPDG simulation results of thermal cracking over a 20-year service are presented in Figure 5-5. As shown in the figure and mentioned earlier, Level 1 simulation of surface layer was not conducted because the creep compliance testing was performed at only one temperature, 14 °F, which provided inputs for Level 2 design. Level 3 simulation of surface layer could also be conducted using creep compliance and tensile strength data that were produced by MEPDG software based on correlations with mixture volumetric characteristics and binder properties. Therefore, the figure compares thermal cracking predicted from the two input levels of the asphalt surface layer that were incorporated with different input level combinations of the base and subgrade layers. It is evident, for both structures, that layer modulus properties of the asphalt base and subgrade layers were not sensitively related to the thermal cracking performance, whereas the asphalt surface layer characteristics sensitively affected the thermal cracking, as particularly demonstrated in Figure 5-5(b). The high sensitivity observed from pavement structure (c) seems to be related to the large discrepancy in the creep compliance between the two input levels, as previously shown in Figure 4-13.



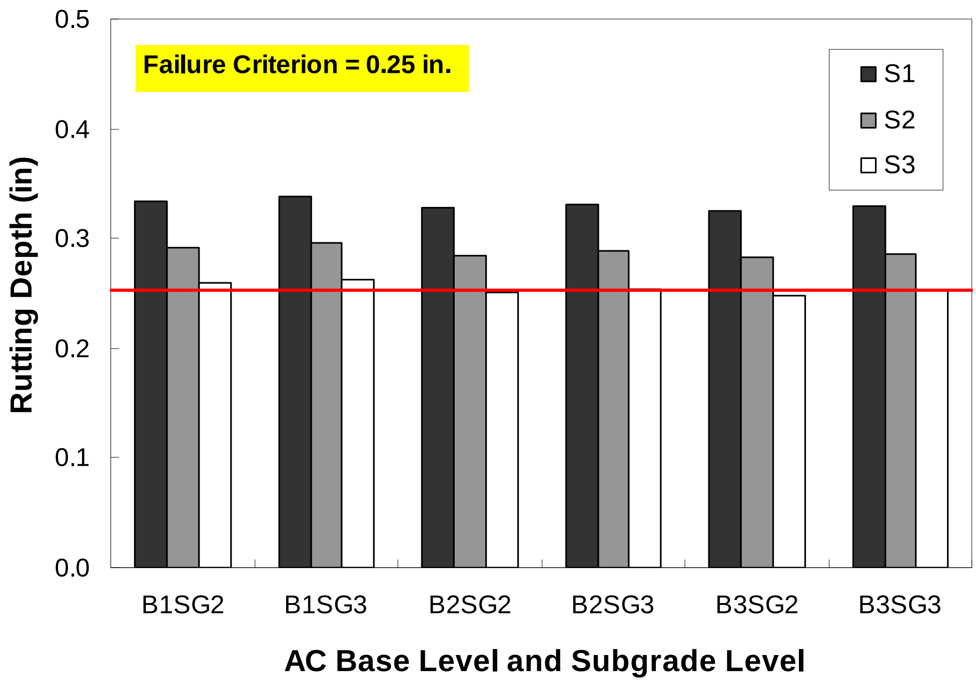
(a) Pavement Structure (b)



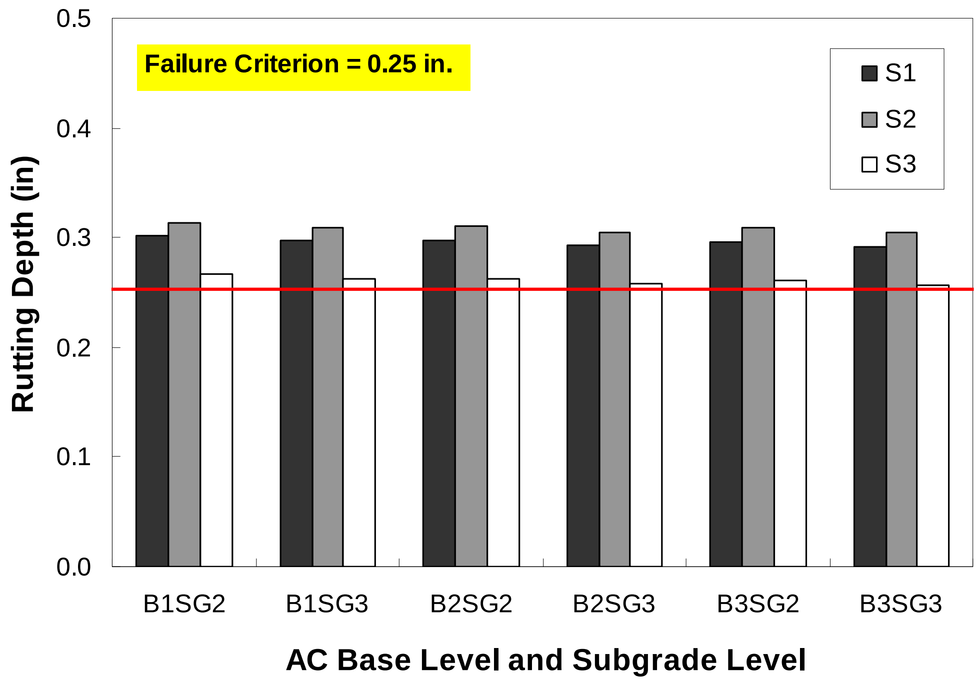
(b) Pavement Structure (c)

Figure 5-5. MEPDG Simulation Results of Thermal Cracking

MEPDG simulation results of rut performance are plotted in Figures 5-6 and 5-7 for the surface layer rutting and for the total rutting, respectively. Contrary to the previous case presenting alligator cracking performance, the magnitude of rut depth was not negligible. At the end of a 20-year service, the surface layer rutting was generally more than the typical rut failure criterion of 0.25 in., and the total pavement rutting was close to the typical failure criterion of 0.75 in. Another interesting observation from those two figures is that the pavement rutting was sensitively influenced by the dynamic modulus input level of the asphalt surface layer, while layer modulus properties of the asphalt base and subgrade were not sensitively related to the rutting performance. For each input level of asphalt surface layer, no clear deviation in the predicted rutting was evident with different combinations of base-subgrade moduli inputs.

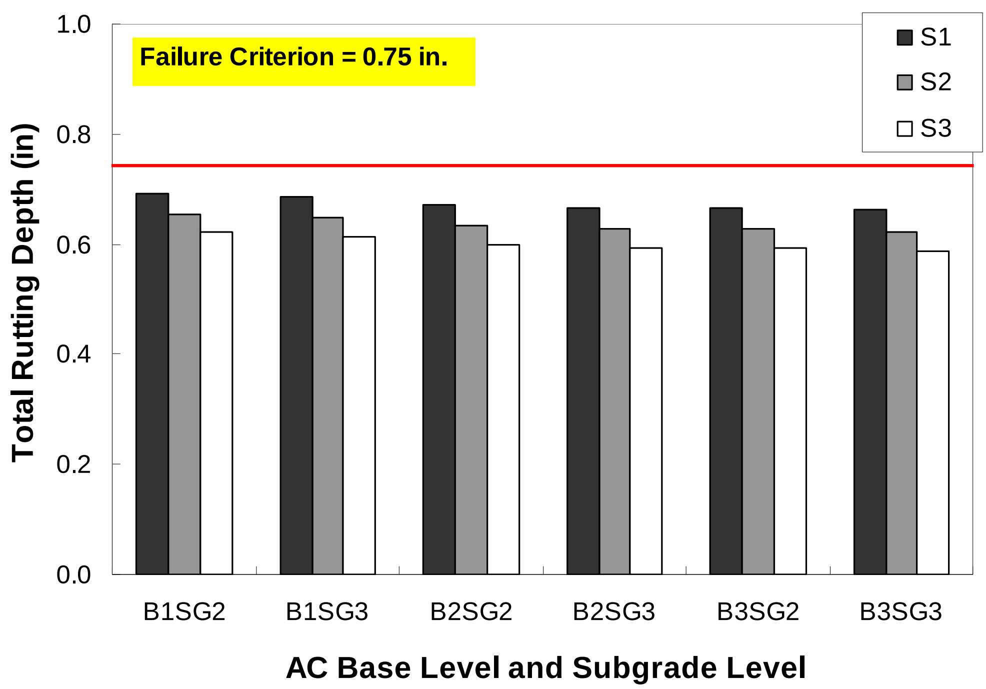


(a) Pavement Structure (b)

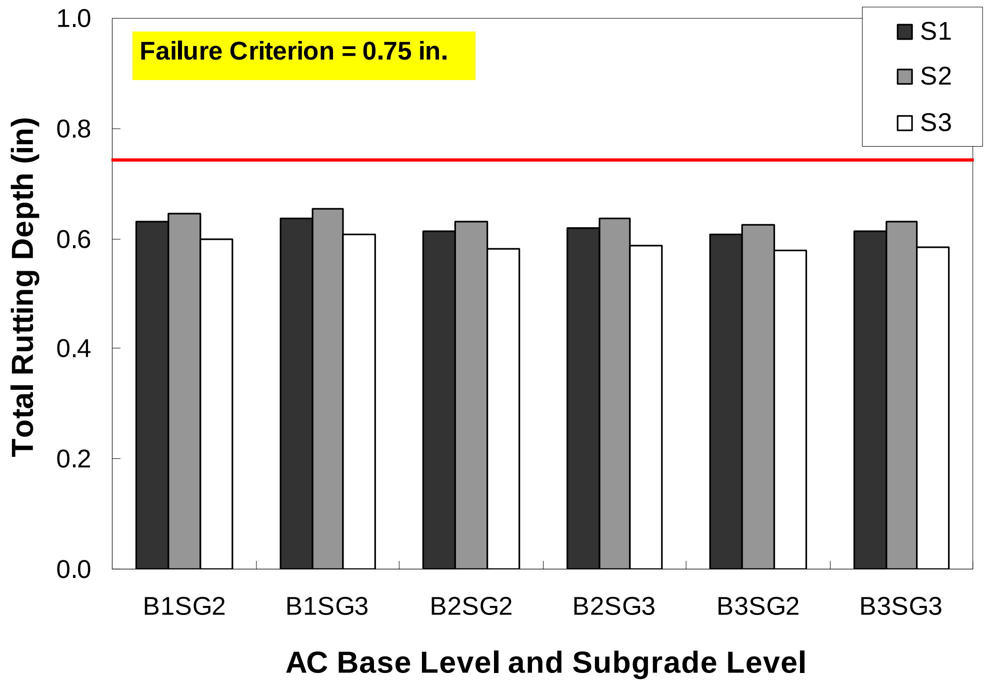


(b) Pavement Structure (c)

Figure 5-6. MEPDG Simulation Results of Surface Rutting



(a) Pavement Structure (b)

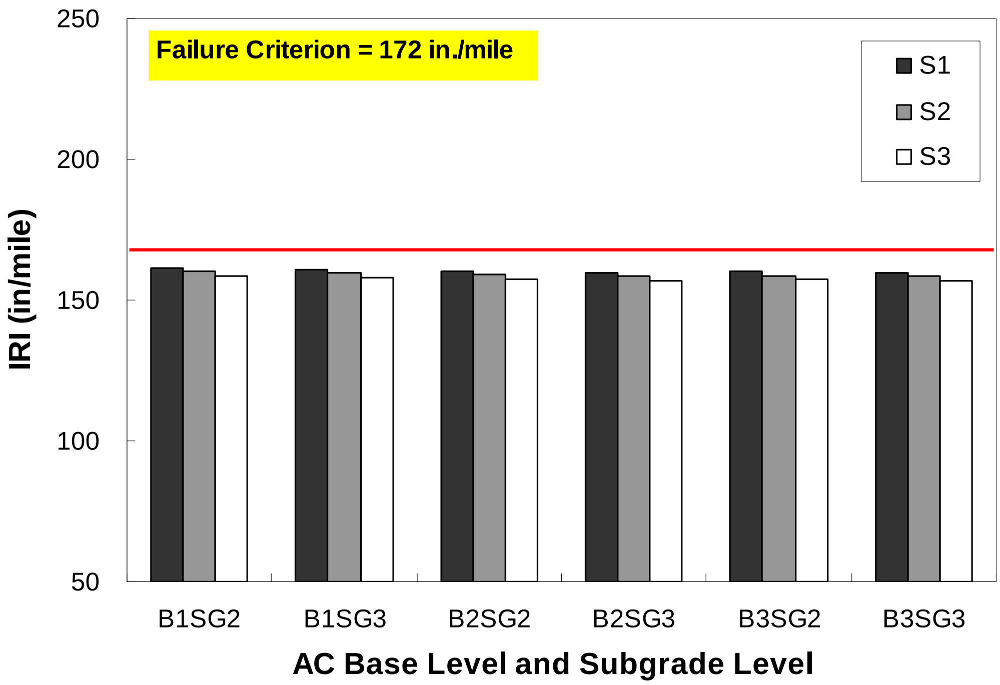


(b) Pavement Structure (c)

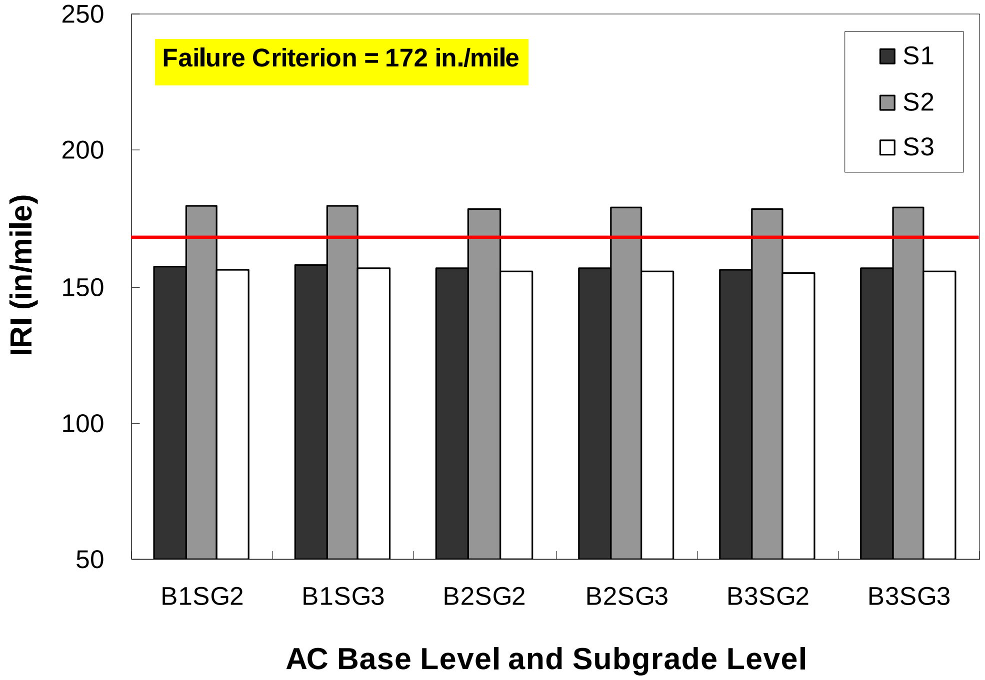
Figure 5-7. MEPDG Simulation Results of Total Rutting

Finally, Figure 5-8 presents the predicted performance of IRI from each combination of layer moduli. No evident performance sensitivity was observed among different input level combinations of the base layer and subgrade layer, while pavement structure (c) presented the effect of surface layer characteristics on overall pavement roughness.

For both pavement structures analyzed in this study, the performance variation related to the stiffness of subgrade layer was not significant for all type of distresses, although the resilient modulus values used for the Level 2 and Level 3 differed by around 70 percent. Similar results can also be found in several studies (Hoerner et al. 2007, McCracken et al. 2008, and Kim et al. 2005). They reported that the resilient modulus of subgrade shows minimal effects to the pavement performance. Based on the observed analysis results herein and the research outcomes presented in other studies, it can be concluded that the effect of the hierarchical subgrade modulus input on the overall predicted pavement performance is not significant.



(a) Pavement Structure (b)



(b) Pavement Structure (c)

Figure 5-8. MEPDG Simulation Results of IRI

CHAPTER 6
SUMMARY AND CONCLUSIONS

The layer modulus database of various pavement materials used in Nebraska was developed at all three hierarchical levels for a step-wise implementation of the new Mechanistic-Empirical Pavement Design Guide (MEPDG). The database presents inputs of twenty HMA mixtures, two native soils, and nine stabilized soils typically used in Nebraska pavements for use with the MEPDG design-analysis software. Modulus values for each design level were then applied to the MEPDG software to perform sensitivity analyses. The sensitivity analyses investigated level-dependent performance predictions obtained from the MEPDG simulations of typical Nebraska asphalt pavement structures. Based on the test results and analyses, the following conclusions can be drawn.

6.1 CONCLUSIONS

- From the laboratory dynamic modulus test results of twenty HMA mixtures, variations in dynamic modulus values among mixtures were found to exist, even though these are the same type of mixtures. This implies that mixture stiffness characteristics are related to properties and proportioning of mixture constituents. Individual mixtures of the same mixture type were produced with different blends of components.
- When comparing dynamic modulus master curves among levels, a discrepancy was evident between Level 1 (measured) master curves and Level 2 or 3 (predicted) master curves. The level of discrepancy between curves was mixture-specific, while it was generally larger at lower or higher loading frequencies. Differences between Level 2 and Level 3 master curves were not significant, which may be because Witczak's predictive model was used for both levels.
- Creep compliance test results for all twenty HMA mixtures presented similar observations with dynamic modulus testing. Variations in creep compliance values were apparent among the mixtures, even though they were the same type of mixtures. Comparison of creep compliance results obtained from the Level 2 testing to the

Level 3 estimation demonstrated a mixture-specific discrepancy between the two levels.

- The resilient modulus test was performed only for the two unbound soils, loess/till and sandy silt. Testing difficulties were encountered in performing the resilient modulus test of loess soil because of significant plastic deformation during the test. In addition to the two native unbound soils tested for the Level 1 resilient modulus characterization, the resilient modulus characteristics were also determined for the nine stabilized soils (loess, till, and shale stabilized with hydrated lime, fly ash and cement kiln dust, respectively) that were studied by Hensley et al. (2007) for a previous NDOR research project.
- Resilient modulus test results for the Level 1 inputs clearly demonstrated that resilient modulus of soils is stress state dependent. The stress-dependent resilient modulus of soils was characterized by identifying the three model parameters (k_1, k_2, k_3) in the generalized constitutive model. On the other hand, Level 2 and 3 resilient modulus inputs are stress-independent values and therefore different from the Level 1 characterization.
- MEPDG sensitivity analyses were conducted to evaluate the effects of using different design input levels on MEPDG performance predictions of asphalt pavement structures. Sensitivity analysis results conducted for typical full-depth flexible pavement structures showed somewhat strong effects of design input levels. For the particular example case in this research, pavement performance indicators such as the longitudinal cracking, thermal cracking, and rutting were sensitively affected by the design inputs of surface and/or base layer. However, the performance variation related to the stiffness of subgrade layer was not significant for all type of distresses.

6.2 NDOR IMPLEMENTATION PLAN

The primary focus of this research was to obtain the layer moduli of various asphalt types currently used in Nebraska. This research has provided those moduli values which will be utilized in our current pavement design procedures. This research also provided valuable data about the prediction models that are internal to the Mechanistic-Empirical

Pavement Design Guide (MEPDG) software. This data will be used for future development of Nebraska's implementation of the Mechanistic-Empirical design procedures.

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APENDIX 1

HMA DATABASE FOR MEPDG

Project Number: **RD 9-4(1012)**
Name of Road: EMERSON TO WAKEFIELD NORTH
Type of Asphalt Concrete: HRB (2008 yr)

Asphalt Cement
Source: Flint Hills
Grade: PG 58-28

Aggregate Gradation

Cumulative % Retained 3/4 inch	Cumulative % Retained 3/8 inch	Cumulative % Retained #4 sieve	% Passing #200 sieve
0	8.5	17.5	6

Dynamic Modulus (psi)

Temperature (°F)	Frequency (Hz)					
	0.1	0.5	1	5	10	25
10	2543899.6	2660061.2	2782350.1	3080025.9	3207938.2	3300223.4
40	1291776.1	1516767.2	1728366.8	1969848.8	2148699.8	2257414.1
70	192506.7	295673.4	382967.3	680259.3	846756.6	1017114.1
100	50985.6	66159.2	83160.6	154296.8	192571.1	271091.7
130	36088.4	37897.1	41584.8	59512.5	70513.5	89270.5

Binder Properties

Superpave binder data (at Short Term Aging - RTFO)						
Temperature (°F)	At 70 (°F)		At 85 (°F)		At 100 (°F)	
Properties	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)
	1122000	64.37	274800	69.46	72530	73.5

Volumetric Properties

Effective Binder Content (%)	Air Voids (%)	Total Unit Weight (pcf)	Maximum Specific Gravity
10.5	4.2	144.9	2.424

Thermal Cracking

Creep Compliance (1/psi)						
Loading Time (sec) at Medium Temperature (14°F) with **Level 2**						
1	2	5	10	20	50	100
1.07E-07	2.80E-07	3.72E-07	4.36E-07	5.09E-07	6.04E-07	7.40E-07

Average tensile strength at 14 (°F)	439.05

Project Number: **RD 81-2(1037)**

Name of Road: IN YORK

Type of Asphalt Concrete: HRB (2008 yr)

Asphalt Cement

Source: Flint Hills

Grade: PG 58-28

Aggregate Gradation

Cumulative % Retained 3/4 inch	Cumulative % Retained 3/8 inch	Cumulative % Retained #4 sieve	% Passing #200 sieve
0	3.8	14.6	7.7

Dynamic Modulus (psi)

Temperature (°F)	Frequency (Hz)					
	0.1	0.5	1	5	10	25
10	1831067.3	1951430.7	2094582.9	2339753.9	2479891.6	2502839.6
40	886394.9	1001843.0	1177350.6	1557480.7	1692684.9	1838489.7
70	150607.4	224404.4	302152.0	565929.2	691838.3	818614.8
100	42802.7	50973.8	66487.3	125697.1	160454.5	231657.4
130	32271.7	31103.6	32572.1	44036.8	47692.5	63445.0

Binder Properties

Superpave binder data (at Short Term Aging - RTFO)						
Temperature (°F)	At 70 (°F)		At 85 (°F)		At 100 (°F)	
Properties	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)
	1330000	63.12	325400	68.01	82040	72.24

Volumetric Properties

Effective Binder Content (%)	Air Voids (%)	Total Unit Weight (pcf)	Maximum Specific Gravity
11.2	4.0	145.0	2.420

Thermal Cracking

Creep Compliance (1/psi)						
Loading Time (sec) at Medium Temperature (14°F) with **Level 2**						
1	2	5	10	20	50	100
7.31E-08	2.18E-07	3.09E-07	3.87E-07	4.54E-07	6.37E-07	8.25E-07

Average tensile strength at 14 (°F)	417.48

Project Number: **STP 14-4(110)**
Name of Road: ELGIN TO US-20 & PLAINVIEW WEST ON US-20
Type of Asphalt Concrete: HRB (2008 yr)

Asphalt Cement
Source: JEBRO
Grade: PG 58-28

Aggregate Gradation

Cumulative % Retained 3/4 inch	Cumulative % Retained 3/8 inch	Cumulative % Retained #4 sieve	% Passing #200 sieve
0	4.5	11.8	5.9

Dynamic Modulus (psi)

Temperature (°F)	Frequency (Hz)					
	0.1	0.5	1	5	10	25
10	2190029.5	2291099.6	2493332.2	2909207.5	2940971.4	3034239.1
40	912895.3	1077580.9	1241729.3	1632184.5	1803570.8	1955574.8
70	153183.6	210606.4	281419.8	531234.3	651579.7	800466.1
100	37573.6	42036.7	56476.7	108609.1	142988.2	205805.3
130	31385.8	32022.6	32662.8	48260.0	54281.0	65851.9

Binder Properties

Superpave binder data (at Short Term Aging - RTFO)						
Temperature (°F)	At 70 (°F)		At 85 (°F)		At 100 (°F)	
Properties	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)
	1311000	59.37	328500	65.59	88560	69.92

Volumetric Properties

Effective Binder Content (%)	Air Voids (%)	Total Unit Weight (pcf)	Maximum Specific Gravity
11.8	3.9	144.9	2.415

Thermal Cracking

Creep Compliance (1/psi)						
Loading Time (sec) at Medium Temperature (14°F) with **Level 2**						
1	2	5	10	20	50	100
2.71E-07	4.75E-07	5.84E-07	6.59E-07	7.63E-07	9.35E-07	1.12E-06

Average tensile strength at 14 (°F)	405.15

Project Number: **NH 6-4(125)**
Name of Road: HASTINGS WEST
Type of Asphalt Concrete: HRB (2009 yr)

Asphalt Cement
Source: Flint Hills
Grade: PG 58-34

Aggregate Gradation

Cumulative % Retained 3/4 inch	Cumulative % Retained 3/8 inch	Cumulative % Retained #4 sieve	% Passing #200 sieve
0.8	8.4	22.2	5.8

Dynamic Modulus (psi)

Temperature (°F)	Frequency (Hz)					
	0.1	0.5	1	5	10	25
10	2097054.5	2297631.2	2438765.0	2842385.8	2927245.5	3235682.7
40	680455.4	918934.5	1036122.6	1517179.0	1735601.9	1919331.4
70	83270.0	121914.5	178761.3	362690.3	476509.8	640031.9
100	28363.4	32445.5	36428.9	64777.8	86759.1	125428.8
130	25876.5	24214.5	25875.5	30196.1	34511.5	38537.6

Binder Properties

Superpave binder data (at Short Term Aging - RTFO)						
Temperature (°F)	At 70 (°F)		At 85 (°F)		At 100 (°F)	
Properties	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)
	377200	64.05	116000	64.98	35870	65.84

Volumetric Properties

Effective Binder Content (%)	Air Voids (%)	Total Unit Weight (pcf)	Maximum Specific Gravity
11.0	3.7	145.3	2.424

Thermal Cracking

Creep Compliance (1/psi)						
Loading Time (sec) at Medium Temperature (14°F) with **Level 2**						
1	2	5	10	20	50	100
2.93E-07	5.22E-07	6.51E-07	7.58E-07	8.93E-07	1.15E-06	1.44E-06

Average tensile strength at 14 (°F)	569.34

Project Number: **STPD 6-6(156)**
Name of Road: DORCHESTER TO MILFORD
Type of Asphalt Concrete: SPL (2008 yr)

Asphalt Cement
Source: Flint Hills
Grade: PG 58-28

Aggregate Gradation

Cumulative % Retained 3/4 inch	Cumulative % Retained 3/8 inch	Cumulative % Retained #4 sieve	% Passing #200 sieve
1.1	13.6	27.1	7.5

Dynamic Modulus (psi)

Temperature (°F)	Frequency (Hz)					
	0.1	0.5	1	5	10	25
10	2656874.4	2728711.3	2880849.8	3146278.7	3303297.2	3271087.3
40	1101444.3	1274602.1	1445819.5	1869871.5	2008415.8	2206527.7
70	177780.0	307653.3	413332.0	735147.6	865229.5	997621.2
100	53737.8	60183.6	80531.6	181242.3	208254.7	286939.4
130	37725.3	39297.5	53771.4	59228.5	59300.9	74802.9

Binder Properties

Superpave binder data (at Short Term Aging - RTFO)						
Temperature (°F)	At 70 (°F)		At 85 (°F)		At 100 (°F)	
Properties	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)
	1373000	63.17	340200	67.89	85470	72.44

Volumetric Properties

Effective Binder Content (%)	Air Voids (%)	Total Unit Weight (pcf)	Maximum Specific Gravity
9.5	3.8	147.0	2.449

Thermal Cracking

Creep Compliance (1/psi)						
Loading Time (sec) at Medium Temperature (14°F) with **Level 2**						
1	2	5	10	20	50	100
1.26E-07	2.25E-07	2.65E-07	2.97E-07	3.37E-07	4.07E-07	4.77E-07
Average tensile strength at 14 (°F)					488.18	

Project Number: **STPD-79-2(102)**

Name of Road: RAYMOND SOUTH

Type of Asphalt Concrete: SPL (2008 yr)

Asphalt Cement

Source: Flint Hills

Grade: PG 58-28

Aggregate Gradation

Cumulative % Retained 3/4 inch	Cumulative % Retained 3/8 inch	Cumulative % Retained #4 sieve	% Passing #200 sieve
0	18.5	30.8	6.9

Dynamic Modulus (psi)

Temperature (°F)	Frequency (Hz)					
	0.1	0.5	1	5	10	25
10	2734224.0	3034270.5	3294642.6	3619511.1	3749186.1	3484809.0
40	1340550.5	1649962.9	1862052.9	2254756.1	2386535.4	2451876.1
70	245471.4	417205.4	524915.2	850792.2	966330.5	1156902.8
100	50798.7	88522.3	108551.3	204110.3	258207.9	356502.8
130	29918.9	36478.9	43402.2	66509.3	72062.5	60003.6

Binder Properties

Superpave binder data (at Short Term Aging - RTFO)						
Temperature (°F)	At 70 (°F)		At 85 (°F)		At 100 (°F)	
Properties	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)
	1298000	63.5	326700	68.56	74640	73.67

Volumetric Properties

Effective Binder Content (%)	Air Voids (%)	Total Unit Weight (pcf)	Maximum Specific Gravity
8.6	4.1	147.5	2.466

Thermal Cracking

Creep Compliance (1/psi)						
Loading Time (sec) at Medium Temperature (14°F) with **Level 2**						
1	2	5	10	20	50	100
1.50E-07	1.61E-07	1.86E-07	2.14E-07	2.98E-07	4.22E-07	5.46E-07
Average tensile strength at 14 (°F)					545.27	

Project Number: **STP-91-3(107)**
Name of Road: TAYLOR EAST
Type of Asphalt Concrete: SPL (2009 yr)

Asphalt Cement
Source: Flint Hills
Grade: PG 58-34

Aggregate Gradation

Cumulative % Retained 3/4 inch	Cumulative % Retained 3/8 inch	Cumulative % Retained #4 sieve	% Passing #200 sieve
0	16.6	28.2	5.5

Dynamic Modulus (psi)

Temperature (°F)	Frequency (Hz)					
	0.1	0.5	1	5	10	25
10	1856757.2	1964578.5	2088753.4	2424598.8	2514705.4	2557477.4
40	786061.0	930340.6	1083982.7	1502493.0	1656655.9	1797114.1
70	149985.3	218513.3	290692.9	537784.9	675369.3	869362.4
100	51053.7	58510.2	65429.8	122193.1	152756.4	217889.8
130	35465.4	36812.2	37173.8	48469.5	58175.1	70915.8

Binder Properties

Superpave binder data (at Short Term Aging - RTFO)						
Temperature (°F)	At 70 (°F)		At 85 (°F)		At 100 (°F)	
Properties	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)
	469500	64.03	139800	64.74	41900	65.29

Volumetric Properties

Effective Binder Content (%)	Air Voids (%)	Total Unit Weight (pcf)	Maximum Specific Gravity
9.4	4.3	145.8	2.443

Thermal Cracking

Creep Compliance (1/psi)						
Loading Time (sec) at Medium Temperature (14°F) with **Level 2**						
1	2	5	10	20	50	100
2.91E-07	4.61E-07	5.73E-07	6.67E-07	7.82E-07	9.91E-07	1.22E-06

Average tensile strength at 14 (°F)	633.29

Project Number: **NH-80-9(832),(825),(827)**

Name of Road: GREENWOOD TO MAHONEY

Type of Asphalt Concrete: SPL (2009 yr)

Asphalt Cement

Source: MONARCH

Grade: PG 64-28

Aggregate Gradation

Cumulative % Retained 3/4 inch	Cumulative % Retained 3/8 inch	Cumulative % Retained #4 sieve	% Passing #200 sieve
1.5	14.4	23.1	7.7

Dynamic Modulus (psi)

Temperature (°F)	Frequency (Hz)					
	0.1	0.5	1	5	10	25
10	2591924.9	2691213.4	2823077.4	3093657.7	3190453.1	3236438.5
40	1300154.1	1429971.3	1581567.0	1968816.8	2094123.0	2261920.0
70	288516.3	391251.3	494634.2	824001.9	970045.9	1121245.9
100	68716.7	90706.1	111405.9	232127.5	296792.1	396367.0
130	32810.6	34252.6	39033.0	64506.0	79339.8	101739.2

Binder Properties

Superpave binder data (at Short Term Aging - RTFO)						
Temperature (°F)	At 70 (°F)		At 85 (°F)		At 100 (°F)	
Properties	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)
	1415000	56.7	440600	61.5	130500	63.21

Volumetric Properties

Effective Binder Content (%)	Air Voids (%)	Total Unit Weight (pcf)	Maximum Specific Gravity
9.5	4.1	146.8	2.454

Thermal Cracking

Creep Compliance (1/psi)						
Loading Time (sec) at Medium Temperature (14°F) with **Level 2**						
1	2	5	10	20	50	100
1.98E-07	2.75E-07	3.05E-07	3.31E-07	3.69E-07	4.42E-07	5.23E-07

Average tensile strength at 14 (°F)	495.41

Project Number: **RD-81-2 (1037)**
Name of Road: IN YORK
Type of Asphalt Concrete: SP4 (0.375) (2008 yr)

Asphalt Cement
Source: Flint Hills
Grade: PG 64-28

Aggregate Gradation

Cumulative % Retained 3/4 inch	Cumulative % Retained 3/8 inch	Cumulative % Retained #4 sieve	% Passing #200 sieve
0.1	3.5	17.1	6.9

Dynamic Modulus (psi)

Temperature (°F)	Frequency (Hz)					
	0.1	0.5	1	5	10	25
10	2568877.8	2683453.8	2862536.0	3204519.3	3323408.8	3416495.9
40	1054986.6	1270094.0	1481885.4	1961818.4	2175465.3	2453835.1
70	163956.2	236487.0	312133.9	604859.0	736558.3	912207.1
100	43222.9	54299.7	70769.1	130867.4	175560.7	248213.6
130	42989.5	43793.7	42421.6	58221.0	64192.8	77575.5

Binder Properties

Superpave binder data (at Short Term Aging - RTFO)						
Temperature (°F)	At 70 (°F)		At 85 (°F)		At 100 (°F)	
Properties	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)
	1270000	59.27	354300	63.47	103600	65.19

Volumetric Properties

Effective Binder Content (%)	Air Voids (%)	Total Unit Weight (pcf)	Maximum Specific Gravity
10.4	3.9	145.6	2.430

Thermal Cracking

Creep Compliance (1/psi)						
Loading Time (sec) at Medium Temperature (14°F) with **Level 2**						
1	2	5	10	20	50	100
1.85E-07	3.36E-07	4.08E-07	4.65E-07	5.32E-07	6.46E-07	7.62E-07

Average tensile strength at 14 (°F)	457.52

Project Number: **RD-9-4(1012)**

Name of Road: EMERSON TO WAKEFIELD

Type of Asphalt Concrete: SP4 (0.375) (2008 yr)

Asphalt Cement

Source: Flint Hills

Grade: PG 64-28

Aggregate Gradation

Cumulative % Retained 3/4 inch	Cumulative % Retained 3/8 inch	Cumulative % Retained #4 sieve	% Passing #200 sieve
0	4.7	15.9	4.6

Dynamic Modulus (psi)

Temperature (°F)	Frequency (Hz)					
	0.1	0.5	1	5	10	25
10	1828347.1	2135366.8	2286956.6	2581680.4	2632575.3	2626224.0
40	691676.3	947599.5	1112873.2	1484841.0	1593921.3	1683507.7
70	109317.9	176762.3	235497.8	440129.9	549142.3	684896.4
100	33200.5	49927.9	58642.9	102503.8	123459.6	177119.3
130	28620.4	32868.0	35891.3	44994.1	53763.6	59647.5

Binder Properties

Superpave binder data (at Short Term Aging - RTFO)						
Temperature (°F)	At 70 (°F)		At 85 (°F)		At 100 (°F)	
Properties	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)
	1184000	61.57	294200	65.08	86920	66.06

Volumetric Properties

Effective Binder Content (%)	Air Voids (%)	Total Unit Weight (pcf)	Maximum Specific Gravity
11.5	4.0	144.3	2.409

Thermal Cracking

Creep Compliance (1/psi)						
Loading Time (sec) at Medium Temperature (14°F) with **Level 2**						
1	2	5	10	20	50	100
1.95E-07	3.47E-07	4.46E-07	5.05E-07	5.95E-07	7.49E-07	8.90E-07

Average tensile strength at 14 (°F)	416.17

Project Number: **NH-6-4(125)**
Name of Road: HASTINGS WEST
Type of Asphalt Concrete: SP4 (0.375) (2009 yr)

Asphalt Cement
Source: Flint Hills
Grade: PG 64-28

Aggregate Gradation

Cumulative % Retained 3/4 inch	Cumulative % Retained 3/8 inch	Cumulative % Retained #4 sieve	% Passing #200 sieve
0	3.6	12.8	5.4

Dynamic Modulus (psi)

Temperature (°F)	Frequency (Hz)					
	0.1	0.5	1	5	10	25
10	2736367.7	2960234.9	3079546.3	3418065.4	3327987.5	3657522.8
40	1170982.4	1427465.3	1538830.7	2040142.7	2230419.7	2404839.5
70	146071.5	221605.9	308553.6	602689.7	758221.1	956026.3
100	42116.1	49160.4	66077.7	118184.4	157125.8	230277.6
130	30583.4	30643.2	32494.8	45625.8	52408.7	69506.6

Binder Properties

Superpave binder data (at Short Term Aging - RTFO)						
Temperature (°F)	At 70 (°F)		At 85 (°F)		At 100 (°F)	
Properties	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)
	1124000	62.74	281800	65.2	79630	66.05

Volumetric Properties

Effective Binder Content (%)	Air Voids (%)	Total Unit Weight (pcf)	Maximum Specific Gravity
11.1	3.8	145.5	2.423

Thermal Cracking

Creep Compliance (1/psi)						
Loading Time (sec) at Medium Temperature (14°F) with **Level 2**						
1	2	5	10	20	50	100
1.04E-07	1.90E-07	2.32E-07	2.67E-07	3.16E-07	4.09E-07	5.23E-07

Average tensile strength at 14 (°F)	435.09

Project Number: **RD-25-2(1014)**

Name of Road: WALLACE SOUTH

Type of Asphalt Concrete: SP4 (0.375) (2009 yr)

Asphalt Cement

Source: JEBRO

Grade: PG 64-28

Aggregate Gradation

Cumulative % Retained 3/4 inch	Cumulative % Retained 3/8 inch	Cumulative % Retained #4 sieve	% Passing #200 sieve
0	1.7	12.9	7.7

Dynamic Modulus (psi)

Temperature (°F)	Frequency (Hz)					
	0.1	0.5	1	5	10	25
10	2352905.0	2504044.6	2636020.4	2952482.2	3096571.9	3276636.2
40	1106325.1	1282804.9	1466726.7	1929211.2	2106181.2	2329040.7
70	164030.8	240708.0	326227.2	586287.3	728898.5	930941.0
100	43697.7	56565.6	68178.8	135264.2	177667.1	256495.8
130	27723.6	27179.6	30738.6	43616.6	50841.8	66251.9

Binder Properties

Superpave binder data (at Short Term Aging - RTFO)						
Temperature (°F)	At 70 (°F)		At 85 (°F)		At 100 (°F)	
Properties	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)
	1162000	62.51	291400	65.51	82730	67.31

Volumetric Properties

Effective Binder Content (%)	Air Voids (%)	Total Unit Weight (pcf)	Maximum Specific Gravity
11.5	4.2	144.5	2.416

Thermal Cracking

Creep Compliance (1/psi)						
Loading Time (sec) at Medium Temperature (14°F) with **Level 2**						
1	2	5	10	20	50	100
2.19E-07	3.23E-07	3.90E-07	4.51E-07	5.35E-07	7.05E-07	8.99E-07

Average tensile strength at 14 (°F)	412.51

Project Number: **PEP-183-1 (1020)**

Name of Road:

Type of Asphalt Concrete: SP4 (0.5) (2008 yr)

Asphalt Cement

Source: MONARCH

Grade: PG 64-28

Aggregate Gradation

Cumulative % Retained 3/4 inch	Cumulative % Retained 3/8 inch	Cumulative % Retained #4 sieve	% Passing #200 sieve
0	11.1	24.9	4.4

Dynamic Modulus (psi)

Temperature (°F)	Frequency (Hz)					
	0.1	0.5	1	5	10	25
10	2214166.3	2623301.7	2697566.0	2573689.2	2810799.0	2434479.3
40	1019577.6	1338239.3	1462163.6	1903738.1	1940090.5	1796865.8
70	237148.5	353805.8	457334.5	754418.9	865924.8	1017322.7
100	62104.4	92709.1	120265.6	208645.6	250617.9	338105.2
130	35944.7	45470.7	50658.2	71624.6	82675.7	103470.4

Binder Properties

Superpave binder data (at Short Term Aging - RTFO)						
Temperature (°F)	At 70 (°F)		At 85 (°F)		At 100 (°F)	
Properties	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)
	1176000	58.15	379200	63.43	126300	65.04

Volumetric Properties

Effective Binder Content (%)	Air Voids (%)	Total Unit Weight (pcf)	Maximum Specific Gravity
11.2	4.10	146.0	2.440

Thermal Cracking

Creep Compliance (1/psi)						
Loading Time (sec) at Medium Temperature (14°F) with **Level 2**						
1	2	5	10	20	50	100
1.56E-07	2.70E-07	3.15E-07	3.49E-07	3.93E-07	4.71E-07	5.49E-07

Average tensile strength at 14 (°F)	423.10

Project Number: **STPD-NFG-11-2(115)**
Name of Road: CAIRO TO BOELUS (2008 yr)
Type of Asphalt Concrete: SP4(0.5) *W/1.0% H. LIME

Asphalt Cement
Source: SEM
Grade: PG 64-28

Aggregate Gradation

Cumulative % Retained 3/4 inch	Cumulative % Retained 3/8 inch	Cumulative % Retained #4 sieve	% Passing #200 sieve
0.4	12.3	30.6	5.5

Dynamic Modulus (psi)

Temperature (°F)	Frequency (Hz)					
	0.1	0.5	1	5	10	25
10	2252845.5	2628856.3	2763260.5	3007106.8	3049967.9	3024647.4
40	888492.9	1209823.3	1406420.3	1781655.0	1958605.3	2063308.1
70	169488.2	325842.0	435214.3	720145.4	859749.2	1029385.9
100	41793.5	69843.4	86210.2	150909.9	189436.2	257282.8
130	43379.4	62578.2	69115.7	74435.6	76913.1	89472.5

Binder Properties

Superpave binder data (at Short Term Aging - RTFO)						
Temperature (°F)	At 70 (°F)		At 85 (°F)		At 100 (°F)	
Properties	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)
	1409000	60.1	379800	63.14	112300	64.91

Volumetric Properties

Effective Binder Content (%)	Air Voids (%)	Total Unit Weight (pcf)	Maximum Specific Gravity
10.1	3.6	146.1	2.429

Thermal Cracking

Creep Compliance (1/psi)						
Loading Time (sec) at Medium Temperature (14°F) with **Level 2**						
1	2	5	10	20	50	100
2.54E-07	4.02E-07	4.66E-07	5.08E-07	5.44E-07	6.52E-07	7.30E-07
Average tensile strength at 14 (°F)					472.33	

Project Number: **NH-281-4(119)**

Name of Road: CHAMBERS JCT. NORTH

Type of Asphalt Concrete: SP4 (0.5) (2009 yr)

Asphalt Cement

Source: JEBRO

Grade: PG 64-28

Aggregate Gradation

Cumulative % Retained 3/4 inch	Cumulative % Retained 3/8 inch	Cumulative % Retained #4 sieve	% Passing #200 sieve
0.2	9.3	17	5.7

Dynamic Modulus (psi)

Temperature (°F)	Frequency (Hz)					
	0.1	0.5	1	5	10	25
10	2853206.8	2957393.6	3123457.3	3522600.2	3839701.8	3932635.5
40	1286461.6	1488620.8	1703854.1	2217252.3	2437694.5	2587352.8
70	204302.9	287386.3	395405.7	723874.8	900989.0	1092335.5
100	59080.4	73431.9	87097.5	174140.5	234282.6	322478.9
130	36756.5	34528.4	38174.1	51805.8	62725.5	83985.8

Binder Properties

Superpave binder data (at Short Term Aging - RTFO)						
Temperature (°F)	At 70 (°F)		At 85 (°F)		At 100 (°F)	
Properties	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)
	1068000	63.14	284400	65.63	78250	67.59

Volumetric Properties

Effective Binder Content (%)	Air Voids (%)	Total Unit Weight (pcf)	Maximum Specific Gravity
10.3	3.92	145.7	2.430

Thermal Cracking

Creep Compliance (1/psi)						
Loading Time (sec) at Medium Temperature (14°F) with **Level 2**						
1	2	5	10	20	50	100
2.78E-07	4.03E-07	4.70E-07	5.23E-07	5.86E-07	7.00E-07	8.24E-07
Average tensile strength at 14 (°F)					459.17	

Project Number: **NH-83-3(107)**
Name of Road: THEDFORD SOUTH
Type of Asphalt Concrete: SP4(0.5) (2009 yr)

Asphalt Cement
Source: JEBRO
Grade: PG 64-28

Aggregate Gradation

Cumulative % Retained 3/4 inch	Cumulative % Retained 3/8 inch	Cumulative % Retained #4 sieve	% Passing #200 sieve
0	8.9	30.9	5

Dynamic Modulus (psi)

Temperature (°F)	Frequency (Hz)					
	0.1	0.5	1	5	10	25
10	2615108.1	2711784.6	2886458.5	3195422.2	3358592.5	3786092.9
40	1021748.0	1192144.1	1364688.5	1799610.7	1992517.1	2319529.1
70	169663.7	240858.1	330839.8	589823.6	738174.2	909841.2
100	39871.3	52045.1	63820.6	136974.2	183080.9	258038.8
130	26576.4	28011.0	31581.6	43472.8	52610.9	68886.0

Binder Properties

Superpave binder data (at Short Term Aging - RTFO)						
Temperature (°F)	At 70 (°F)		At 85 (°F)		At 100 (°F)	
Properties	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)
	1124000	62.74	281800	65.2	79630	66.05

Volumetric Properties

Effective Binder Content (%)	Air Voids (%)	Total Unit Weight (pcf)	Maximum Specific Gravity
10.4	4.2	145.1	2.429

Thermal Cracking

Creep Compliance (1/psi)						
Loading Time (sec) at Medium Temperature (14°F) with **Level 2**						
1	2	5	10	20	50	100
2.47E-07	4.52E-07	5.63E-07	6.50E-07	7.55E-07	9.43E-07	1.14E-06

Average tensile strength at 14 (°F)	452.25

Project Number: **RD-75-2(1055)**

Name of Road: FORT STR. SOUTH, OMAHA

Type of Asphalt Concrete: SP5(0.5) (2008 yr)

Asphalt Cement

Source: Flint Hills

Grade: PG 70-28

Aggregate Gradation

Cumulative % Retained 3/4 inch	Cumulative % Retained 3/8 inch	Cumulative % Retained #4 sieve	% Passing #200 sieve
0	10.5	24.1	6.1

Dynamic Modulus (psi)

Temperature (°F)	Frequency (Hz)					
	0.1	0.5	1	5	10	25
10	3063296.6	3169743.8	3415507.2	3540164.8	3814552.6	3995832.1
40	1828143.5	1842210.1	2024905.9	2446564.4	2640186.7	2791958.1
70	489704.5	625164.8	755084.5	1119519.9	1288385.3	1468226.7
100	97661.3	139433.3	185838.4	352278.5	438929.3	555331.5
130	55786.4	63579.1	71412.4	115612.1	142841.2	182549.4

Binder Properties

Superpave binder data (at Short Term Aging - RTFO)						
Temperature (°F)	At 70 (°F)		At 85 (°F)		At 100 (°F)	
Properties	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)
	1410000	60.12	363000	62.98	113500	62.9

Volumetric Properties

Effective Binder Content (%)	Air Voids (%)	Total Unit Weight (pcf)	Maximum Specific Gravity
9.7	3.9	146.8	2.448

Thermal Cracking

Creep Compliance (1/psi)						
Loading Time (sec) at Medium Temperature (14°F) with **Level 2**						
1	2	5	10	20	50	100
7.92E-08	1.38E-07	1.52E-07	1.63E-07	1.73E-07	1.89E-07	2.02E-07

Average tensile strength at 14 (°F)	495.37

Project Number: **STPD-6-7(178)**

Name of Road: GREENWOOD TO ASHLAND

Type of Asphalt Concrete: SP5(0.5) (2008 yr)

Asphalt Cement

Source: JEBRO / Flint Hills

Grade: PG 64-28

Aggregate Gradation

Cumulative % Retained 3/4 inch	Cumulative % Retained 3/8 inch	Cumulative % Retained #4 sieve	% Passing #200 sieve
1	10.1	20.4	6.8

Dynamic Modulus (psi)

Temperature (°F)	Frequency (Hz)					
	0.1	0.5	1	5	10	25
10	2781317.9	2670536.5	3003834.4	3188212.0	3221113.8	3523836.0
40	1253042.1	1417307.3	1611422.4	2002855.3	2165463.3	2264449.0
70	260805.5	357472.6	462744.5	800013.9	932103.0	1102407.6
100	75315.6	93470.5	126910.7	230936.1	291956.4	385186.9
130	35212.4	40221.6	46524.3	68718.8	86644.4	112588.8

Binder Properties

Superpave binder data (at Short Term Aging - RTFO)						
Temperature (°F)	At 70 (°F)		At 85 (°F)		At 100 (°F)	
Properties	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)
	1630000	59.8	377900	64.55	105800	66.03

Volumetric Properties

Effective Binder Content (%)	Air Voids (%)	Total Unit Weight (pcf)	Maximum Specific Gravity
9.6	3.9	146.3	2.441

Thermal Cracking

Creep Compliance (1/psi)						
Loading Time (sec) at Medium Temperature (14°F) with **Level 2**						
1	2	5	10	20	50	100
2.25E-07	3.53E-07	3.92E-07	4.25E-07	4.44E-07	5.34E-07	5.78E-07

Average tensile strength at 14 (°F)	491.72

Project Number: **RD-77-2(1057)**
Name of Road: Lincoln South
Type of Asphalt Concrete: SP5(0.5) (2008 yr)

Asphalt Cement
Source: Flint Hills
Grade: PG 70-28

Aggregate Gradation

Cumulative % Retained 3/4 inch	Cumulative % Retained 3/8 inch	Cumulative % Retained #4 sieve	% Passing #200 sieve
0	6.2	22.3	3.8

Dynamic Modulus (psi)

Temperature (°F)	Frequency (Hz)					
	0.1	0.5	1	5	10	25
10	2878547.1	2876997.0	2959608.0	3076199.5	3072403.4	3639388.4
40	1144281.8	1327891.8	1515637.5	1978565.4	2105509.7	2294792.5
70	242589.6	326246.5	420104.8	726675.8	882497.3	1046170.7
100	72683.9	90038.8	116725.6	208464.8	269761.7	360267.7
130	40969.7	42019.7	50828.8	78592.3	87908.1	108862.1

Binder Properties

Superpave binder data (at Short Term Aging - RTFO)						
Temperature (°F)	At 70 (°F)		At 85 (°F)		At 100 (°F)	
Properties	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)
	1178000	62.3	293200	65.86	83730	67.45

Volumetric Properties

Effective Binder Content (%)	Air Voids (%)	Total Unit Weight (pcf)	Maximum Specific Gravity
9.6	4.10	147.5	2.464

Thermal Cracking

Creep Compliance (1/psi)						
Loading Time (sec) at Medium Temperature (14°F) with **Level 2**						
1	2	5	10	20	50	100
7.23E-08	1.85E-07	2.36E-07	2.78E-07	3.32E-07	4.19E-07	4.84E-07

Average tensile strength at 14 (°F)	502.55

Project Number: **IM-80-6(97)**

Name of Road: WOOD RIVER TO GRAND ISLAND

Type of Asphalt Concrete: SP5(0.5) (2009 yr)

Asphalt Cement

Source: Flint Hills

Grade: PG 70-28

Aggregate Gradation

Cumulative % Retained 3/4 inch	Cumulative % Retained 3/8 inch	Cumulative % Retained #4 sieve	% Passing #200 sieve
0	8.8	19.5	5.4

Dynamic Modulus (psi)

Temperature (°F)	Frequency (Hz)					
	0.1	0.5	1	5	10	25
10	3055089.1	3031140.5	3137813.7	3534049.5	3526444.1	3805294.2
40	1562122.9	1773741.2	1984390.8	2529887.1	2687549.3	2891389.6
70	283628.6	408648.4	543788.9	927070.5	1140891.9	1358296.2
100	66338.3	83353.9	105174.6	218045.9	282249.8	391417.8
130	36009.2	37474.8	43851.6	67281.5	82205.4	106902.6

Binder Properties

Superpave binder data (at Short Term Aging - RTFO)						
Temperature (°F)	At 70 (°F)		At 85 (°F)		At 100 (°F)	
Properties	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)	G*(Pa)	Delta (°)
	1332000	61.1	337300	63.4	101400	63.76

Volumetric Properties

Effective Binder Content (%)	Air Voids (%)	Total Unit Weight (pcf)	Maximum Specific Gravity
10.7	3.7	145.8	2.425

Thermal Cracking

Creep Compliance (1/psi)						
Loading Time (sec) at Medium Temperature (14°F) with **Level 2**						
1	2	5	10	20	50	100
7.81E-08	1.39E-07	1.63E-07	1.82E-07	2.00E-07	2.43E-07	2.83E-07

Average tensile strength at 14 (°F)	462.32

APENDIX 2

SOILS DATABASE FOR MEPDG

Resilient Modulus Testing - AASHTO T 307-99

Sample's ID: OMC Case #1

Soil Type:	Loess-Till	Compacted Moisture Content (%):	19.10		
Max. Dry Density (pcf):	104.0	Final Sample Diameter (in):	3.99	Liquid Limit:	40
Moisture Content (%):	20.3	Final Sample Height (in):	8.03	Plasticity Index:	21
Weight of Wet Soil (lb)	7.26	Final Sample Wet Weight (lb):	7.25		
Initial Sample Diameter (in):	3.94			k1:	453.988
Initial Sample Height (in):	8.07			k2:	0.696
Initial Sample Area (in^2):	12.2			k3:	-5.808
Sample Volume (in^3):	98.3				

Seq.	Confine stress (psi)	Nominal Maximum Axial Stress (Scyclic) (psi)	Actual Applied Max. Axial Load (Pmax) (lb)	Actual Applied Cyclic Load (Pcyclic) (lb)	Actual Applied Contact Load (Pcontact) (lb)	Actual Applied Max. Axial Stress (Smax) (psi)	Actual Applied Cyclic Stress (Scyclic) (psi)	Actual Applied Contact Stress (Scontact) (psi)	Recov. Def. LVDT #1 reading (in)	Recov. Def. LVDT #2 reading (in)	Average Recov. Def. LVDT 1 and 2 reading (in)	Resilient Strain (in/in)	Resilent Modulus (psi)
1	6.0	1.8	28.8	26.5	2.2	2.2	2.0	0.2	0.0026	0.0026	0.0026	0.000317	6272.2
2	6.0	3.6	52.8	47.4	5.4	3.9	3.4	0.5	0.0061	0.0061	0.0061	0.000751	4587.9
3	6.0	5.4	77.6	69.9	7.6	5.7	5.1	0.6	0.0120	0.0120	0.0120	0.001488	3432.1
4	6.0	7.2	101.4	92.2	9.2	7.6	6.8	0.8	0.0191	0.0191	0.0191	0.002366	2875.5
5	6.0	9.0	126.1	116.2	9.9	9.5	8.7	0.8	0.0260	0.0260	0.0260	0.003224	2706.1
6	4.0	1.8	27.2	25.2	2.0	2.1	1.9	0.2	0.0033	0.0033	0.0033	0.000407	4654.3
7	4.0	3.6	51.3	46.3	4.9	3.8	3.4	0.4	0.0086	0.0086	0.0086	0.001066	3189.2
8	4.0	5.4	76.0	70.1	5.8	5.8	5.3	0.5	0.0156	0.0156	0.0156	0.001934	2734.1
9	4.0	7.2	100.3	92.6	7.6	7.6	7.0	0.6	0.0231	0.0231	0.0231	0.002861	2443.2
10	4.0	9.0	124.5	115.3	9.2	9.5	8.7	0.8	0.0298	0.0298	0.0298	0.003695	2359.0
11	2.0	1.8	25.6	23.6	2.0	1.9	1.8	0.2	0.0043	0.0043	0.0043	0.000532	3347.3
12	2.0	3.6	50.1	45.2	4.9	3.7	3.3	0.4	0.0109	0.0109	0.0109	0.001356	2434.4
13	2.0	5.4	74.2	69.0	5.2	5.7	5.2	0.4	0.0196	0.0196	0.0196	0.002432	2157.2
14	2.0	7.2	98.5	92.8	5.6	7.6	7.2	0.5	0.0281	0.0281	0.0281	0.003483	2057.0
15	2.0	9.0	122.7	116.0	6.7	9.5	9.0	0.6	0.0353	0.0354	0.0353	0.004378	2049.7

Resilient Modulus Testing - AASHTO T 307-99

Sample's ID: OMC Case #2

Soil Type:	Loess-Till	Compacted Moisture Content (%):	19.03		
Max. Dry Density (pcf):	104.0	Final Sample Diameter (in):	3.98	Liquid Limit:	40
Moisture Content (%):	20.3	Final Sample Height (in):	8.01	Plasticity Index:	21
Weight of Wet Soil (lb)	7.28	Final Sample Wet Weight (lb):	7.27		
Initial Sample Diameter (in):	3.98			k1:	946.510
Initial Sample Height (in):	8.07			k2:	0.507
Initial Sample Area (in^2):	12.4			k3:	-6.419
Sample Volume (in^3):	100.2				

Seq.	Confine stress (psi)	Nominal Maximum Axial Stress (Scyclic) (psi)	Actual Applied Max. Axial Load (Pmax) (lb)	Actual Applied Cyclic Load (Pcyclic) (lb)	Actual Applied Contact Load (Pcontact) (lb)	Actual Applied Max. Axial Stress (Smax) (psi)	Actual Applied Cyclic Stress (Scyclic) (psi)	Actual Applied Contact Stress (Scontact) (psi)	Recov. Def. LVDT #1 reading (in)	Recov. Def. LVDT #2 reading (in)	Average Recov. Def. LVDT 1 and 2 reading (in)	Resilient Strain (in/in)	Resilent Modulus (psi)
1	6.0	1.8	29.2	27.0	2.2	2.2	2.0	0.2	0.0014	0.0014	0.0014	0.000171	11600.9
2	6.0	3.6	53.7	47.7	6.1	3.8	3.3	0.5	0.0036	0.0037	0.0036	0.000451	7412.6
3	6.0	5.4	78.9	72.6	6.3	5.8	5.3	0.5	0.0066	0.0066	0.0066	0.000820	6504.7
4	6.0	7.2	103.2	92.2	11.0	7.4	6.5	0.9	0.0102	0.0102	0.0102	0.001261	5190.4
5	6.0	9.0	128.1	116.2	11.9	9.4	8.4	1.0	0.0140	0.0140	0.0140	0.001737	4827.2
6	4.0	1.8	27.7	24.5	3.1	2.0	1.7	0.3	0.0013	0.0013	0.0013	0.000166	10391.6
7	4.0	3.6	52.4	47.9	4.5	3.9	3.5	0.4	0.0044	0.0044	0.0044	0.000546	6401.6
8	4.0	5.4	77.3	71.5	5.8	5.7	5.3	0.5	0.0078	0.0078	0.0078	0.000968	5442.2
9	4.0	7.2	101.8	91.0	10.8	7.3	6.4	0.9	0.0118	0.0118	0.0118	0.001461	4413.4
10	4.0	9.0	126.6	113.3	13.3	9.1	8.0	1.1	0.0155	0.0155	0.0155	0.001917	4198.1
11	2.0	1.8	26.1	22.5	3.6	1.8	1.5	0.3	0.0016	0.0016	0.0016	0.000198	7707.8
12	2.0	3.6	50.8	44.7	6.1	3.6	3.1	0.5	0.0052	0.0052	0.0052	0.000641	4856.5
13	2.0	5.4	75.8	69.2	6.5	5.6	5.1	0.5	0.0089	0.0090	0.0090	0.001110	4553.7
14	2.0	7.2	100.7	89.9	10.8	7.2	6.4	0.9	0.0135	0.0135	0.0135	0.001671	3806.4
15	2.0	9.0	125.2	114.2	11.0	9.2	8.3	0.9	0.0171	0.0171	0.0171	0.002122	3917.7

Resilient Modulus Testing - AASHTO T 307-99

Sample's ID: OMC Case #3

Soil Type:	Loess-Till	Compacted Moisture Content (%):	19.60		
Max. Dry Density (pcf):	104.0	Final Sample Diameter (in):	3.98	Liquid Limit:	40
Moisture Content (%):	20.3	Final Sample Height (in):	8.04	Plasticity Index:	21
Weight of Wet Soil (lb)	6.94	Final Sample Wet Weight (lb):	6.94		
Initial Sample Diameter (in):	3.96			k1:	469.446
Initial Sample Height (in):	8.07			k2:	0.664
Initial Sample Area (in^2):	12.3			k3:	-5.361
Sample Volume (in^3):	99.5				

Seq.	Confine stress (psi)	Nominal Maximum Axial Stress (Scyclic) (psi)	Actual Applied Max. Axial Load (Pmax) (lb)	Actual Applied Cyclic Load (Pcyclic) (lb)	Actual Applied Contact Load (Pcontact) (lb)	Actual Applied Max. Axial Stress (Smax) (psi)	Actual Applied Cyclic Stress (Scyclic) (psi)	Actual Applied Contact Stress (Scontact) (psi)	Recov. Def. LVDT #1 reading (in)	Recov. Def. LVDT #2 reading (in)	Average Recov. Def. LVDT 1 and 2 reading (in)	Resilient Strain (in/in)	Resilent Modulus (psi)
1	6.0	1.8	29.0	26.1	2.9	2.1	1.9	0.2	0.0004	0.0004	0.0004	0.000046	40821.5
2	6.0	3.6	53.7	47.9	5.8	3.9	3.4	0.5	0.0048	0.0048	0.0048	0.000598	5712.3
3	6.0	5.4	78.5	72.6	5.8	5.9	5.4	0.5	0.0114	0.0113	0.0113	0.001402	3863.3
4	6.0	7.2	103.2	93.7	9.4	7.6	6.8	0.8	0.0195	0.0193	0.0194	0.002402	2846.0
5	6.0	9.0	127.7	119.8	7.9	9.7	9.1	0.6	0.0267	0.0263	0.0265	0.003280	2764.2
6	4.0	1.8	27.7	25.0	2.7	2.0	1.8	0.2	0.0011	0.0011	0.0011	0.000139	12993.7
7	4.0	3.6	51.9	48.1	3.8	3.9	3.6	0.3	0.0076	0.0074	0.0075	0.000929	3867.8
8	4.0	5.4	77.1	69.7	7.4	5.6	5.1	0.6	0.0153	0.0150	0.0152	0.001878	2690.2
9	4.0	7.2	100.9	94.4	6.5	7.7	7.1	0.5	0.0222	0.0219	0.0220	0.002727	2618.9
10	4.0	9.0	126.6	116.9	9.7	9.5	8.7	0.8	0.0279	0.0275	0.0277	0.003429	2535.1
11	2.0	1.8	26.1	22.7	3.4	1.8	1.6	0.3	0.0012	0.0012	0.0012	0.000146	10673.1
12	2.0	3.6	50.6	45.0	5.6	3.6	3.2	0.5	0.0087	0.0085	0.0086	0.001068	2975.4
13	2.0	5.4	75.1	69.5	5.6	5.6	5.2	0.5	0.0170	0.0167	0.0169	0.002090	2470.7
14	2.0	7.2	99.8	95.1	4.7	7.7	7.3	0.4	0.0249	0.0245	0.0247	0.003063	2387.5
15	2.0	9.0	124.5	118.0	6.5	9.6	9.1	0.5	0.0311	0.0306	0.0308	0.003822	2368.7

Resilient Modulus Testing - AASHTO T 307-99

Soil Type:	Sandy-Silt	Compacted Moisture Content (%):	13.3	Sample's ID:	OMCCase #1
Max. Dry Density (pcf):	108.0	Final Sample Diameter (in):	3.97	Liquid Limit:	28
Moisture Content (%):	13.0	Final Sample Height (in):	7.99	Plasticity Index:	8
Weight of Wet Soil (lb)	7.45	Final Sample Wet Weight (lb):	7.45		
Initial Sample Diameter (in):	3.97			k1:	688.626
Initial Sample Height (in):	8.03			k2:	0.709
Initial Sample Area (in^2):	12.4			k3:	-4.127
Sample Volume (in^3):	99.3				

Seq.	Confine stress (psi)	Nominal Maximum Axial Stress (Scyclic) (psi)	Actual Applied Max. Axial Load (Pmax) (lb)	Actual Applied Cyclic Load (Pcyclic) (lb)	Actual Applied Contact Load (Pcontact) (lb)	Actual Applied Max. Axial Stress (Smax) (psi)	Actual Applied Cyclic Stress (Scyclic) (psi)	Actual Applied Contact Stress (Scontact) (psi)	Recov. Def. LVDT #1 reading (in)	Recov. Def. LVDT #2 reading (in)	Average Recov. Def. LVDT 1 and 2 reading (in)	Resilient Strain (in/in)	Resilent Modulus (psi)
1	6.0	1.8	28.8	26.3	2.5	2.1	1.9	0.2	0.0015	0.0015	0.0015	0.000186	10315.2
2	6.0	3.6	53.7	50.6	3.1	4.1	3.8	0.3	0.0035	0.0035	0.0035	0.000436	8780.9
3	6.0	5.4	78.7	73.3	5.4	5.9	5.5	0.4	0.0061	0.0059	0.0060	0.000743	7376.4
4	6.0	7.2	103.2	94.6	8.5	7.7	7.0	0.7	0.0089	0.0085	0.0087	0.001086	6411.0
5	6.0	9.0	128.1	116.7	11.5	9.4	8.5	0.9	0.0117	0.0110	0.0113	0.001412	6025.3
6	4.0	1.8	27.4	23.6	3.8	1.9	1.6	0.3	0.0019	0.0019	0.0019	0.000230	6936.1
7	4.0	3.6	52.2	47.7	4.5	3.9	3.5	0.4	0.0048	0.0048	0.0048	0.000596	5887.1
8	4.0	5.4	76.4	69.9	6.5	5.7	5.1	0.5	0.0081	0.0078	0.0080	0.000990	5189.1
9	4.0	7.2	101.6	92.2	9.4	7.5	6.7	0.8	0.0109	0.0105	0.0107	0.001333	5009.0
10	4.0	9.0	126.3	114.7	11.7	9.3	8.3	1.0	0.0134	0.0127	0.0130	0.001623	5128.1
11	2.0	1.8	26.1	24.5	1.6	2.0	1.8	0.1	0.0022	0.0022	0.0022	0.000275	6726.4
12	2.0	3.6	50.6	47.0	3.6	3.8	3.5	0.3	0.0061	0.0060	0.0060	0.000750	4673.3
13	2.0	5.4	75.3	67.0	8.3	5.4	4.8	0.7	0.0102	0.0099	0.0100	0.001250	3802.7
14	2.0	7.2	99.8	91.5	8.3	7.4	6.7	0.7	0.0130	0.0126	0.0128	0.001593	4234.0
15	2.0	9.0	124.1	111.3	12.8	9.0	8.0	1.0	0.0157	0.0150	0.0153	0.001907	4173.0

Resilient Modulus Testing - AASHTO T 307-99

Sample's ID: OMCCase #2

Soil Type:	Sandy-Silt	Compacted Moisture Content (%):	13.1		
Max. Dry Density (pcf):	108.0	Final Sample Diameter (in):	3.96	Liquid Limit:	28
Moisture Content (%):	13.0	Final Sample Height (in):	7.98	Plasticity Index:	8
Weight of Wet Soil (lb)	7.53	Final Sample Wet Weight (lb):	7.53		
Initial Sample Diameter (in):	3.96			k1:	726.877
Initial Sample Height (in):	8.05			k2:	0.350
Initial Sample Area (in^2):	12.3			k3:	-1.996
Sample Volume (in^3):	99.0				

Seq.	Confine stress (psi)	Nominal Maximum Axial Stress (Scyclic) (psi)	Actual Applied Max. Axial Load (Pmax) (lb)	Actual Applied Cyclic Load (Pcyclic) (lb)	Actual Applied Contact Load (Pcontact) (lb)	Actual Applied Max. Axial Stress (Smax) (psi)	Actual Applied Cyclic Stress (Scyclic) (psi)	Actual Applied Contact Stress (Scontact) (psi)	Recov. Def. LVDT #1 reading (in)	Recov. Def. LVDT #2 reading (in)	Average Recov. Def. LVDT 1 and 2 reading (in)	Resilient Strain (in/in)	Resilent Modulus (psi)
1	6.0	1.8	29.0	26.5	2.5	2.2	2.0	0.2	0.0004	0.0003	0.0003	0.000039	50114.6
2	6.0	3.6	53.5	48.6	4.9	3.9	3.5	0.4	0.0022	0.0020	0.0021	0.000262	13547.8
3	6.0	5.4	78.2	71.0	7.2	5.8	5.2	0.6	0.0044	0.0042	0.0043	0.000538	9631.5
4	6.0	7.2	103.0	93.1	9.9	7.6	6.8	0.8	0.0064	0.0061	0.0062	0.000775	8744.8
5	6.0	9.0	127.2	114.9	12.4	9.3	8.3	1.0	0.0085	0.0082	0.0083	0.001034	8058.7
6	4.0	1.8	27.4	25.0	2.5	2.0	1.8	0.2	0.0004	0.0002	0.0003	0.000037	49872.7
7	4.0	3.6	51.9	47.0	4.9	3.8	3.4	0.4	0.0022	0.0020	0.0021	0.000267	12840.4
8	4.0	5.4	76.7	69.2	7.4	5.6	5.0	0.6	0.0052	0.0049	0.0050	0.000626	8057.8
9	4.0	7.2	101.6	91.7	9.9	7.4	6.6	0.8	0.0073	0.0070	0.0072	0.000892	7448.7
10	4.0	9.0	125.9	113.5	12.4	9.2	8.2	1.0	0.0091	0.0088	0.0090	0.001112	7387.4
11	2.0	1.8	26.1	23.6	2.5	1.9	1.7	0.2	0.0004	0.0003	0.0003	0.000042	41203.3
12	2.0	3.6	50.4	45.4	4.9	3.7	3.3	0.4	0.0026	0.0024	0.0025	0.000313	10509.0
13	2.0	5.4	75.3	67.9	7.4	5.5	4.9	0.6	0.0057	0.0055	0.0056	0.000699	7046.9
14	2.0	7.2	99.8	89.9	9.9	7.3	6.5	0.8	0.0074	0.0071	0.0073	0.000902	7222.9
15	2.0	9.0	124.3	112.0	12.4	9.1	8.1	1.0	0.0095	0.0093	0.0094	0.001169	6936.8

Resilient Modulus Testing - AASHTO T 307-99

				Sample's ID:	OMCCase #3
Soil Type:	Sandy-Silt	Compacted Moisture Content (%):	13.2		
Max. Dry Density (pcf):	108.0	Final Sample Diameter (in):	3.98	Liquid Limit:	28
Moisture Content (%):	13.0	Final Sample Height (in):	8.03	Plasticity Index:	8
Weight of Wet Soil (lb)	7.44	Final Sample Wet Weight (lb):	7.44		
Initial Sample Diameter (in):	3.98			k1:	860.532
Initial Sample Height (in):	8.04			k2:	0.562
Initial Sample Area (in^2):	12.4			k3:	-1.703
Sample Volume (in^3):	99.9				

Seq.	Confine stress (psi)	Nominal Maximum Axial Stress (Scyclic) (psi)	Actual Applied Max. Axial Load (Pmax) (lb)	Actual Applied Cyclic Load (Pcyclic) (lb)	Actual Applied Contact Load (Pcontact) (lb)	Actual Applied Max. Axial Stress (Smax) (psi)	Actual Applied Cyclic Stress (Scyclic) (psi)	Actual Applied Contact Stress (Scontact) (psi)	Recov. Def. LVDT #1 reading (in)	Recov. Def. LVDT #2 reading (in)	Average Recov. Def. LVDT 1 and 2 reading (in)	Resilient Strain (in/in)	Resilent Modulus (psi)
1	6.0	1.8	29.2	27.2	2.0	2.2	2.0	0.2	0.0079	0.0080	0.0080	0.000991	2043.9
2	6.0	3.6	54.0	49.0	4.9	4.0	3.6	0.4	0.0022	0.0021	0.0021	0.000267	13310.7
3	6.0	5.4	78.9	71.5	7.4	5.8	5.2	0.6	0.0033	0.0032	0.0032	0.000404	12787.0
4	6.0	7.2	103.6	93.7	9.9	7.6	6.8	0.8	0.0045	0.0045	0.0045	0.000560	12060.3
5	6.0	9.0	128.4	116.0	12.4	9.3	8.3	1.0	0.0058	0.0057	0.0058	0.000717	11614.0
6	4.0	1.8	27.7	25.6	2.0	2.1	1.9	0.2	0.0079	0.0080	0.0080	0.000989	1929.5
7	4.0	3.6	52.4	47.4	4.9	3.8	3.4	0.4	0.0026	0.0026	0.0026	0.000321	10672.7
8	4.0	5.4	77.3	69.9	7.4	5.6	5.0	0.6	0.0039	0.0039	0.0039	0.000485	10410.7
9	4.0	7.2	102.5	92.6	9.9	7.4	6.7	0.8	0.0053	0.0052	0.0053	0.000653	10182.9
10	4.0	9.0	126.6	114.2	12.4	9.2	8.2	1.0	0.0065	0.0065	0.0065	0.000803	10214.8
11	2.0	1.8	26.3	24.3	2.0	2.0	1.8	0.2	0.0076	0.0077	0.0077	0.000954	1892.3
12	2.0	3.6	51.0	45.9	5.2	3.7	3.3	0.4	0.0032	0.0032	0.0032	0.000399	8232.0
13	2.0	5.4	76.0	68.6	7.4	5.5	4.9	0.6	0.0048	0.0047	0.0047	0.000590	8342.7
14	2.0	7.2	100.7	90.8	9.9	7.3	6.5	0.8	0.0062	0.0062	0.0062	0.000771	8442.7
15	2.0	9.0	125.4	113.1	12.4	9.1	8.1	1.0	0.0075	0.0075	0.0075	0.000935	8668.8

Resilient Modulus Testing - AASHTO T 307-99

Sample's ID: OMCCase #4

Soil Type:	Sandy-Silt	Compacted Moisture Content (%):	13.3		
Max. Dry Density (pcf):	108.0	Final Sample Diameter (in):	3.99	Liquid Limit:	28
Moisture Content (%):	13.0	Final Sample Height (in):	8.06	Plasticity Index:	8
Weight of Wet Soil (lb)	7.44	Final Sample Wet Weight (lb):	7.31		
Initial Sample Diameter (in):	3.98			k1:	846.294
Initial Sample Height (in):	8.06			k2:	0.351
Initial Sample Area (in^2):	12.4			k3:	-1.580
Sample Volume (in^3):	100.1				

Seq.	Confine stress (psi)	Nominal Maximum Axial Stress (Scyclic) (psi)	Actual Applied Max. Axial Load (Pmax) (lb)	Actual Applied Cyclic Load (Pcyclic) (lb)	Actual Applied Contact Load (Pcontact) (lb)	Actual Applied Max. Axial Stress (Smax) (psi)	Actual Applied Cyclic Stress (Scyclic) (psi)	Actual Applied Contact Stress (Scontact) (psi)	Recov. Def. LVDT #1 reading (in)	Recov. Def. LVDT #2 reading (in)	Average Recov. Def. LVDT 1 and 2 reading (in)	Resilient Strain (in/in)	Resilent Modulus (psi)
1	6.0	1.8	29.2	26.8	2.5	2.2	2.0	0.2	0.0012	0.0012	0.0012	0.000149	13094.0
2	6.0	3.6	54.0	49.0	4.9	3.9	3.5	0.4	0.0025	0.0024	0.0025	0.000305	11611.1
3	6.0	5.4	78.9	71.5	7.4	5.8	5.2	0.6	0.0039	0.0037	0.0038	0.000474	10897.3
4	6.0	7.2	103.6	93.7	9.9	7.6	6.8	0.8	0.0052	0.0050	0.0051	0.000632	10689.0
5	6.0	9.0	128.4	116.0	12.4	9.3	8.3	1.0	0.0062	0.0058	0.0060	0.000747	11156.4
6	4.0	1.8	27.7	25.2	2.5	2.0	1.8	0.2	0.0012	0.0012	0.0012	0.000149	12256.4
7	4.0	3.6	52.4	47.4	4.9	3.8	3.4	0.4	0.0028	0.0026	0.0027	0.000339	10073.6
8	4.0	5.4	77.6	70.1	7.4	5.6	5.1	0.6	0.0044	0.0041	0.0043	0.000530	9537.8
9	4.0	7.2	102.3	92.4	9.9	7.4	6.6	0.8	0.0057	0.0053	0.0055	0.000681	9749.8
10	4.0	9.0	126.8	114.4	12.4	9.2	8.2	1.0	0.0066	0.0062	0.0064	0.000793	10365.4
11	2.0	1.8	26.3	23.8	2.5	1.9	1.7	0.2	0.0015	0.0014	0.0014	0.000176	9718.1
12	2.0	3.6	51.0	46.1	4.9	3.7	3.3	0.4	0.0033	0.0031	0.0032	0.000396	8338.7
13	2.0	5.4	76.0	68.6	7.4	5.5	4.9	0.6	0.0050	0.0047	0.0048	0.000598	8226.8
14	2.0	7.2	100.7	90.8	9.9	7.3	6.5	0.8	0.0062	0.0059	0.0060	0.000750	8694.0
15	2.0	9.0	125.2	112.9	12.4	9.1	8.1	1.0	0.0073	0.0069	0.0071	0.000881	9187.4