Develop Typical Material Input Values for Mechanistic – Empirical Pavement Design in Tennessee

Phase I

Submitted to the Tennessee Department of Transportation

Research Development and Technology Program

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CHAPTER 1 INTRODUCTION

1.1 Problem Statement

In 2007, the AASHTO Joint Technical Committee on Pavements, Subcommittee on Design and Subcommittee on Materials recommended the Mechanistic-Empirical Pavement Design Guide (MEPDG) as an interim guide for the design of pavement structures. The development of such a procedure was conducted by the National Cooperative Highway Research Program (NCHRP) under sponsorship by the American Association of State Highway and Transportation Officials (AASHTO). The MEPDG and related software provide capabilities for the analysis and performance prediction of different types of flexible and rigid pavements.

The MEPDG uses mechanistic-empirical numerical models to analyze input data for traffic, climate, materials and proposed structure. The models estimate damage accumulation over service life. The concept of pavement performance accounts for structural and functional performance, which the Guide is primarily concerned with. Performance predictions are made in terms of pavement distresses and ride quality.

The Design Guide is an important innovation in pavement design. Inputs include traffic (full load spectra for single, tandem, tridem, and quad axles), material and subgrade characterization, climatic factors, performance criteria and others. Many state transportation agencies began the evaluation of this procedure with the ultimate goal of its adaptation and calibration for local conditions. An important part of this process is the evaluation of the performance prediction models and sensitivity of the predicted distresses to various input parameters for local conditions and, if necessary, re-calibration of the performance prediction models.

As part of the transition, TDOT has contracted with independent consultant to develop implementation strategies for the state. Two main objects of this study are to establish typical material input values for the new MEPDG (Phase I), and to validate and/or calibrate the MEPDG design procedure with Pavement Management System (PMS) in Tennessee (Phase II).

1.2 Scope

In order to start the smooth transition of this significant change, it is necessary to develop input parameter values for the typical materials used in Tennessee pavements. In the first phase, the proposed research project will collect the typical TDOT material input values for the MEPDG by following the new MEPDG test protocol. Laboratory experiments will be conducted to evaluate the subgrade soils and other materials according to the MEPDG protocols. Properties of materials obtained in the lab will be put into MEPDG as input values for predicting pavement performance of major routes in Tennessee. Results from MEPDG software will be compared with the results from AASHTO 1993 design procedure to see the difference between these two design tools. Sensitivity analysis will be also carried out in asphalt pavement and rigid pavement to identify factors that influence pavement performance significantly. Figure 1 shows main work in Phase I.

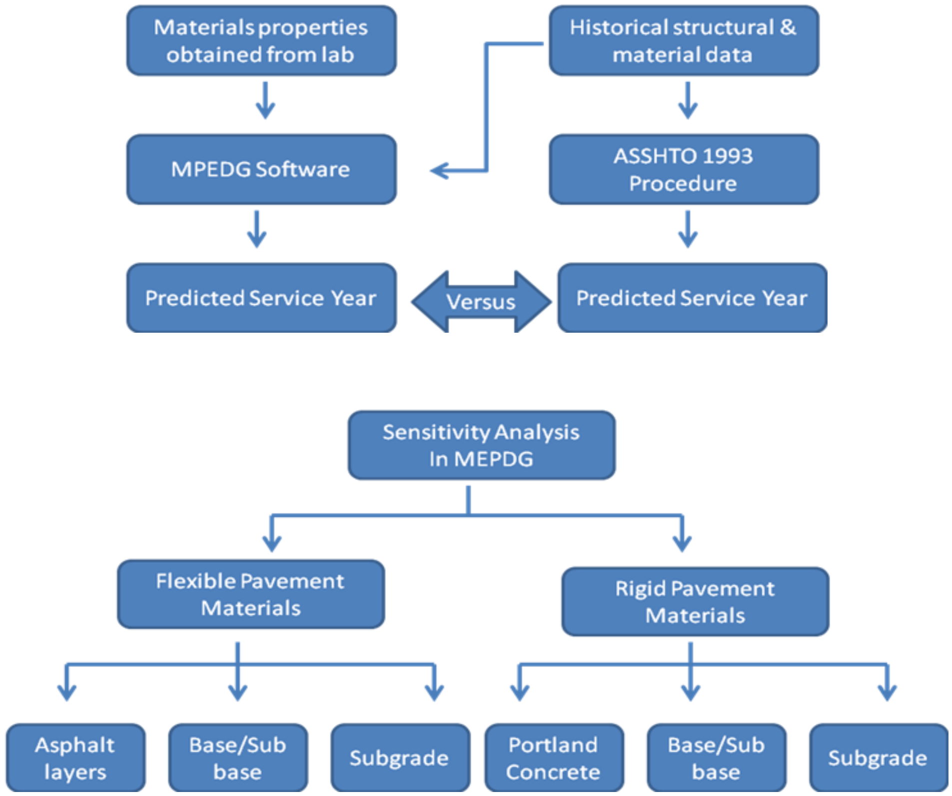


Figure 1 Main Work in Phase I

In the second phase, validation and/or calibration of MEPDG will be held based on the PMS in Tennessee. Collected pavement performance data from major routes in Tennessee will be used to compare with the predicted pavement performance, while input values of local materials obtained from lab and experience are adopted in MEPDG. The main effort will be put on permanent deformation model and roughness model. The research process during the second phase is shown in Figure 2.

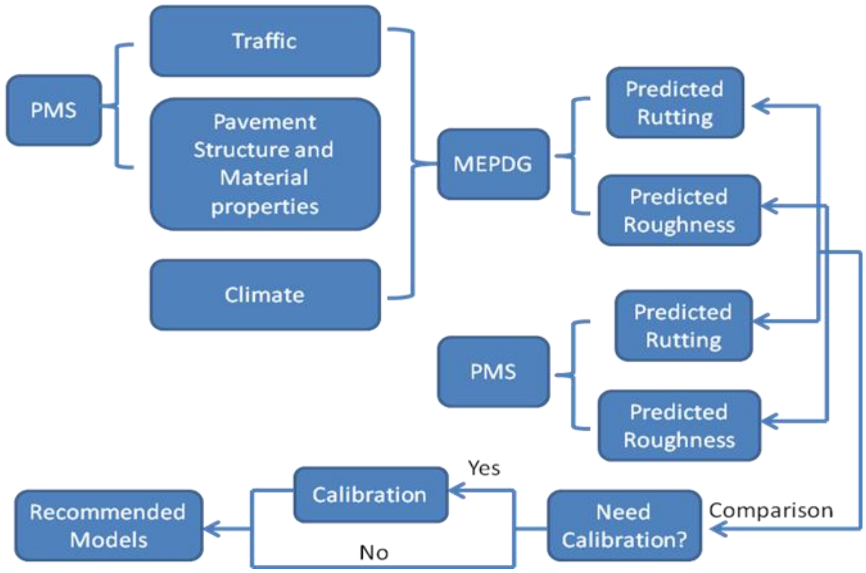


Figure 2 Research Flowchart in Phase II

1.3 Objective

The objective of the proposed research is to develop typical material input values necessary for the implementation of AASHTO MEPDG in Tennessee, and to validate and/or calibrate MEPDG based on local weather and material conditions for its complementation in Tennessee.

1.4 Research methodology

The research methodology for this study can be divided into the following tasks:

Task 1: Literature Review

This task will involve a comprehensive literature search of published materials and on-going research projects to obtain the latest information on the MEPDG implementation in the US, especially in the Southeast. Databases of TRB, TRIS, COMPENDIX, and UMI THESIS AND DISSERTATIONS will be searched.

Task 2: Select Typical TDOT Materials

Typical TDOT materials will be selected by region. Subgrade soils and asphalt mixtures will be considered and collected from the job site of field projects. These materials will be transported to the Laboratory of Infrastructural Materials of the University of Tennessee for laboratory testing.

Task 3: Build up the Basic Materials Reference Library for Typical TDOT Materials

Laboratory tests will be performed on the typical TDOT materials to obtain their input values required for the MEPDG analysis. The results from the laboratory tests will then be used to build up the basic materials reference library for typical TDOT materials. The following tests will be used to obtain the material inputs for MEPDG according to the MEPDG protocols:

- Resilient modulus test for base and subgrade soils
- Dynamic modulus test for asphalt mixtures

Task 4: Pavement Structural Analyses Using MEPDG Software

The pavement structural analyses will be performed using the latest version of MEPDG software. The established input values for typical TDOT materials will be used in the MEPDG analyses.

Task 5: Comparison between the MEPDG Analyses and the Current TDOT Pavement Design Procedures

The current TDOT pavement design procedures will also be utilized to perform the pavement structural analyses with the available material inputs from the same materials. The results from the MEPDG analyses will be compared with those from the current TDOT pavement design procedures to identify the advantages and benefits of MEPDG.

Task 6: Recommendations for Future MEPDG Implementation Plans in Tennessee

The plans for future implementation of MEPDG in Tennessee will be recommended based on the results and findings from this proposed study. The recommendations will lay a basis for a smooth transition from the current TDOT pavement design practice to the new AASHTO MEPDG.

CHAPTER 2 LITERATURE REVIEW

2.1 Flexible Pavement Design Procedures

Prior to the 1920s, pavement design basically consisted of determining thicknesses of layered materials that would provide strength to protect a weak subgrade. Pavements were designed against subgrade shear failure.

The Shell method (Claussen et al., 1977) and the Asphalt Institute method (ShookF et al., 1982) were the first to use linear-elastic theory of mechanics to compute structural responses in combination with empirical models to predict number of loads to failure for flexible pavements. However, frequently pavement materials do not exhibit the simple behavior assumed in isotropic linear-elastic theory. Nonlinearities, time and temperature dependency, and anisotropy are some examples of complicated features often observed in pavement materials. In this case, advanced modeling is required to predict performance mechanistically. The mechanistic design approach is based on the theories of mechanics and relates pavement structural behavior and performance to traffic loading and environmental influences.

The mechanistic-empirical approach is a hybrid method. Empirical models fill in the gaps between the theory of mechanics and the performance of pavement structures. Simple mechanistic responses are easy to compute with assumptions and simplifications, but they cannot be used to predict performance directly by themselves; some types of empirical models are required to make the appropriate correlation. Mechanistic-empirical methods are considered an intermediate step between empirical and fully mechanistic methods.

In 1958, American Association of State Highway Officials (the AASHO) built a test road. And based on it, empirical-based pavement design procedures were developed. Although they may exhibit good accuracy, empirical methods are valid only for the materials and climate conditions for which they were developed. Some of the limitations of the AASHO road test are one climate region, limited traffic, single vehicle type, and single subgrade type.

Several studies over the past 20 years have improved mechanistic-empirical techniques. Most of work, however, was based on variants of the same two strain-based criteria developed by Shell and the Asphalt Institute. The Departments of Transportation of the North Carolina, Washington State, and Minnesota, to name a few, developed their own M-E procedures. The NCHRP 1-26 project report, Calibrated Mechanistic Structural Analysis Procedures for Pavements (1990), provided the basic framework for most of the efforts attempted by state DOTs. WSDOT and NCDOT developed similar M-E frameworks incorporating environmental variables (e.g., asphalt concrete temperature to determine stiffness) and cumulative damage model using Miner's Law with the fatigue cracking criterion.

The 1993 AASHTO is a largely empirical method based primarily on the AASHO Road Test conducted in the late 1950s. Over the years adjustments and modifications have been made in an effort to upgrade and expand the limits over which the AASHTO guide is valid (HRB, 1962; AASHTO, 1972, 1986, 1993). A 1996 workshop meant to develop a framework for improving the 1993 Guide recommended instead the development of a new guide based as much as possible on mechanistic principles. The

M-E PDG developed in NHCRP 1-37A is the result of this effort. Following independent reviews and validations that have been ongoing since its initial release in April, 2004, the M-E PDG is expected to be adopted by AASHTO as the new national pavement design guide. The NCHRP 1-37A project (NCHRP, 2004) delivered the most recent M-E-based method that incorporates nationally calibrated models to predict distinct distresses induced by traffic load and environmental conditions. The NCHRP 1-37A methodology also incorporates vehicle class and load distributions in the design, a step forward from the Equivalent Single Axle Load (ESAL) approach used in the AASTHO design equation and other methods. The performance computation is done on a seasonal basis to incorporate the effects of climate conditions on the behavior of materials.

AASHTO 1993 Guide for Design of Pavement Structures

The 1993 AASHTO Guide specifies the following empirical design equation for flexible pavements:

$$\log W_{18} = Z_R \cdot S_0 + 9.36 \cdot \log(SN+1) - 0.20 + \frac{\log(\Delta PSI)/(4.2-1.5)}{0.4+1094/(SN+1)^{5.19}} + 2.32 \cdot \log(MR) - 8.07 \qquad (1)$$

where:

W_{18} = accumulated 18 kip equivalent single axle load for the design period
Z_R = reliability factor
S_0 = standard deviation
SN = structural number
ΔPSI = initial PSI – terminal PSI
MR = subgrade resilient modulus (psi)

The structural number is calculated using the layer thicknesses, material drainage properties, and layer coefficients, which are used to express the relative strength contribution of each pavement layer to the overall pavement structure. (AASHTO, 1993) While the respective layer thicknesses and drainage conditions are relatively easy to quantify, the layer coefficients are not so straightforward. No direct method exists for establishing new layer coefficients as new HMA mix types are created, and they are dependent upon many different parameters including material stiffness, tensile strength, compressive strength, moisture conditions, and even the layer's position within the pavement cross section. (AASHTO, 1993; Pologruto, 2001). Layer coefficients, resilient modulus are part of most important factors that affects the flexible design. (NCAT 09-03 Recalibration of the asphalt layer coefficient) Therefore, usually resilient modulus of base and subgrade soil and modulus of asphalt mixture attract the interests of engineers.

Despite of the adjustments made over the years to the design equation in attempts to expand its suitability to different climate regions and materials, the design of flexible pavements in the AASHTO 1993 Guide still lacks accuracy in performance predictions and in ability to include different materials and their complex behavior.

2002 Mechanical-Empirical Pavement Design Guide

The objective of NCHRP 1-37A is to develop a pavement design tool based on mechanistic-empirical principles. The resulting pavement design tool, called the 2002 Design Guide (2002DG), is intended to be a user-friendly software for analysis and design of new, reconstructed, and rehabilitated flexible, rigid, and composite pavements. The 2002 Design Guide is a result of coordinated effort of NCHRP Project Panel C1-37 and AASHTO JTFP. The models in the design guide were calibrated using data from LTPP sections from all over the nation.

Based on existing mechanistic-empirical technology, accompanied by the necessary computational software, AASHTO developed the Guide for Design of New and Rehabilitated Pavements Structures. The key advantage of M-E design is that it is a comprehensive design procedure not just only focusing on layer thickness. M-E models straightly consider true influences and interactions of inputs, i.e. climate, traffic, structure, on structural distress and ride quality. With minimizing all distress types, an optimal design could be reached possibly. The design process of MEPDG (NCHRP, 2004) is shown in Figure 3.

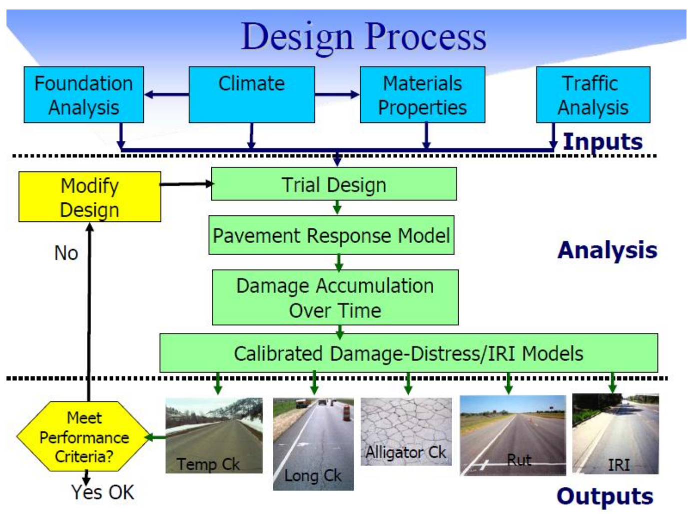


Figure 3 MEPDG Design Process (NCHRP 2004)

The MEPDG has a hierarchical approach for the design inputs, defined by the quality of data available and importance of the project. There are three levels: Level 1 – Laboratory measured material properties are required. Project-specific traffic data is also required (e.g., vehicle class and load distributions); Level 2 – Inputs are obtained through empirical correlations with other parameters; Level 3 – Inputs are selected from a database of national or regional default values according to the material type or highway class (e.g., soil classification to determine the range of resilient modulus, highway class to determine vehicle class distribution). According to the NCHRP 1-37A report, level 1 is recommended for heavily trafficked highways where premature failure is economically undesirable. Level 2 can be used for intermediate projects, while level 3 is recommended for minor projects, usually low traffic roads. In addition, Multi Layer Linear Elastic Theory (MLET) is used to predict mechanistic responses in the pavement structure in the MEPDG software. When level 1 nonlinear stiffness inputs for unbound material are selected, a nonlinear Finite Element Method (FEM) is utilized instead.

Three parts of the design process require material properties: the climate model, the pavement response models, and the distress models. Climate-related properties determine temperature and moisture variations inside the pavement structure. The pavement response models use material properties (corrected as appropriate for temperature and moisture effects) to compute the state of stress/strain at critical locations in the structure due to traffic loading and temperature changes. These structural responses are used by the distress models along with complementary material properties to predict pavement performance.

Table 1 summarizes the flexible pavement material properties required by the M-E PDG. (Recall that measured properties are level 1 inputs, correlations with other parameters are level 2, and default values selected from typical ranges are level 3.)

Table 1 Material Inputs Requirement for Flexible Pavements (NCHRP 2004)

Material Category	Material inputs required		
	Climatic models	Response models	Distress models
Asphalt concrete	- mixture: surface shortwave absorptivity, thermal conductivity, and heat capacity - asphalt binder: viscosity (stiffness) characterization to account for aging	- time-temperature dependent dynamic modulus (E*) of HMA mixture - Poisson’s ratio	- tensile strength, - creep compliance - coefficient of thermal expansion
Unbound materials	- plasticity index - gradation parameters - effective grain sizes - specific gravity - saturated hydraulic conductivity - optimum moisture	- resilient modulus (Mr) at optimum density and moisture content - Poisson’s ratio - unit weight - coefficient of lateral pressure	- gradation parameters

	content - parameters to define the soil -water characteristic curve		

2.2 Key Inputs of Materials

Two material properties required in the MEPDG, the dynamic modulus for asphalt concrete and the nonlinear stiffness model for unbound materials, are considered innovative for pavement design methods. Time- and temperature-dependency of asphalt mixtures is characterized by the dynamic modulus. The dynamic modulus master curve models the variation of asphalt concrete stiffness due to rate of loading and temperature variation. The nonlinear elastic behavior of unbound granular materials is modeled by a stress-dependent resilient modulus included in level 1 input.

Dynamic Modulus of Asphalt Mixture

The complex dynamic modulus |E*| is the principal material property input for asphalt concrete. It is a function of asphalt mixture characteristics (binder, aggregate gradation, and volumetrics), rate of loading, temperature, and age. For level 1 inputs, the dynamic modulus master curve is constructed based on time-temperature superposition principles (Huang, 2004) by shifting laboratory frequency sweep test data. Level 1 also requires binder viscosity measured using the dynamic shear rheometer (DSR). Aging effects on binder viscosity are simulated using the Global Aging System, which considers short term aging from mix/compaction and long term aging from oxidation (NCHRP, 2004). For level 2 and 3 inputs, the dynamic modulus master curve is obtained via an empirical predictive equation. The |E*| predictive equation is an empirical relationship between |E*| and mixture properties:

$$
\begin{aligned}
\log E^* &= 3.750063 + 0.02932 \cdot \rho_{200} - 0.001767 \cdot (\rho_{200})^2 - 0.002841 \cdot \rho_4 - 0.058097 \cdot V_a \\
&- 0.802208 \cdot \left(\frac{V_{beff}}{V_{beff} + V_a}\right) \\
&+ \frac{3.871977 + 0.0021 \cdot \rho_4 + 0.003958\rho_{38} - 0.000017 \cdot (\rho_{38})^2 + 0.005470 \cdot \rho_{34}}{1 + e^{(-0.603313 - 0.313351 \cdot \log(f) - 0.393532 \cdot \log(\eta))}}
\end{aligned}
\quad (2)
$$

where:

E^* = dynamic modulus, 10^5 psi

η = binder viscosity, 10^6 Poise

f = loading frequency, Hz

V_a = air void content, %

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

ρ_{34} = cumulative % retained on the 19-mm sieve

ρ_{38} = cumulative % retained on the 9.5-mm sieve

ρ_4 = cumulative % retained on the 4.75-mm sieve

ρ_{200} = % passing the 0.075-mm sieve

The binder's viscosity at any temperature is given by the binder's viscosity-temperature relationship:

$$\log\log\eta = A + VTS \cdot \log T_R \quad (3)$$

where:

η = bitumen viscosity, cP

TR = temperature, Rankine (TR=$T_{Fahrenheit}$+460)

A = regression intercept

VTS = regression slope of viscosity temperature susceptibility.

For level 2 asphalt concrete inputs, binder parameters A and VTS are determined from DSR testing. For level 3, default A and VTS values are based on the binder grading.

Additional asphalt concrete material properties are required to predict thermal cracking: (1) tensile strength, (2) creep compliance, (3) coefficient of thermal expansion, (4) surface shortwave absorptivity, and (5) thermal conductivity and heat capacity.

Tensile strength and creep compliance are determined in the laboratory using the indirect tensile test for level 1 and 2 inputs. At level 3, these properties are correlated with other material parameters.

In recent years, the Dynamic Modulus E* test, conducted per AASHTO TP 62-03 has gained wider use in the pavement community. This is because it is a major input into the Mechanistic Empirical Pavement Design Guide (MEPDG), and is also being used as a simple performance test indicator.

Timothy R. Clyne, X. Li, et al. in 2003 tested four types of asphalt mixture and obtained dynamic moduli of them in Minnesota. They found some measured data fit well with the predicted dynamic moduli from the Witczak model, while some did not. The revised NCHRP 1-37A model (Bari et al., 2006.) has improved the overall prediction of E* but has not significantly improved the over prediction of the lower moduli. Gabriel Garcia and Marshall Thompson (Garcia et al., 2007) offered a review on HMA dynamic modulus predictive models and they claimed that the Witczak predictive equation and the Hirsch model (Christensen et al, 2003) both generate sufficiently accurate and reasonable dynamic modulus estimates adequate for use in mechanistic-empirical design.

Resilient Modulus of Base Materials and Subgrade

Resilient modulus is the primary unbound material property required for the structural response model. The resilient modulus of fine-grain soils depends on a lot of factors such as stress state, soil type and its structure (Li et al., 1994), while the resilient modulus of coarse-grain soils is mainly influenced by the stress state, degree of saturation, and compactive effort. Various forms of correlation equations including constitutive models and soil properties were developed. K.P. George (George, 2004) described them in detail and compared them with lab test results. And George recommended using the LTPP equation to predict resilient modulus for coarse-grain soil and use the average value of predicted values from the LTPP equation and the Mississippi equation to describe resilient modulus of fine-grain soil. However, Munir D. Nazzal (Nazzal et al., 2008) claimed that LTPP correlation equations tend to underestimate the values of resilient modulus coefficients of their samples of subgrade soils in Louisiana. They developed models using different physical properties which could lead to a better prediction of measured resilient moduli than LTPP models.

Models predicting resilient modulus of soils are highly dependent of soil type, gradation, stress state, and so on. They are all highly region-related. It is hard to find a general model that fits all kinds of soils. Thus, without sufficient data of basic physic properties, a direct rest is still necessary to obtain resilient modulus of soils.

In MEPDG level 1 resilient modulus values are determined from laboratory test data as fitted to the stress-dependent stiffness model:

$$M_R = k_1 P_a \left(\frac{\theta}{P_a}\right)^{k_2} \left(\frac{\tau_{oct}}{P_a} + 1\right)^{k_3} \tag{4}$$

where:

M_R = resilient modulus

θ = bulk stress = $\sigma_1 + \sigma_2 + \sigma_3$

σ_1= *major principal stress*

σ_2 = intermediate principal stress = $\sigma 3$ for MR test on cylindrical specimens

σ_3 = minor principal stress/confining pressure

τ_{oct} = octahedral shear stress = $\frac{1}{3}\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2}$

Pa = atmospheric pressure (used to normalize the equation)

k_1, k_2, k_3 = regression constants determined from the laboratory tests

The resilient modulus is correlated with other parameters (e.g., CBR, R-value, AASHTO layer coefficient) at level 2. The resilient modulus can be selected from a range of default values that are typical for the material type and/or soil classification at level 3. The input resilient modulus data at all levels are assumed to be at optimum moisture content and density; this value is adjusted by the EICM for seasonal climate variations. There is also an option for direct entry of a best estimate for the seasonally-adjusted unbound resilient modulus, in which case the EICM is bypassed. Poisson's ratio is also required for the structural response model. It can be determined from laboratory testing, correlations with other properties, or estimated from ranges of typical values. The Atterberg limits, gradation, hydraulic conductivity, maximum dry unit weight, specific gravity, optimum moisture, and degree of saturation are additional unbound material inputs used for determining the effect of seasonal climate variations on resilient modulus.

2.3 SUMMARY

In this chapter, the historical development of flexible pavement design procedures was briefly summarized. And two critical material inputs, i.e. dynamic modulus of asphalt materials and resilient modulus of soils were described on prediction models and test methods of them. Prediction models for these two parameters require local validation and calibration before being utilized to estimate E* and Mr. In this research, lab tests were carried out to obtain these two properties of asphalt materials and soils, respectively, which is a prime step to establish the database of typical materials input values for the implementation of MEPDG in Tennessee.

CHAPTER 3 DYNAMIC MODULI OF ASPHALT MATERIALS

In this chapter, the asphalt mixtures used by the Tennessee Department of Transpiration (TDOT) were tested for dynamic modulus using the Asphalt Mixture Performance Tester (AMPT). The master curves of dynamic modulus were constructed for these asphalt mixtures.

Dynamic modulus of asphalt mixture is highly related to the physical properties and gradation of aggregates, which are different from one region to another. Therefore, asphalt mixtures were collected from all over the whole state, as shown in Figure 4, and tested in the laboratory.

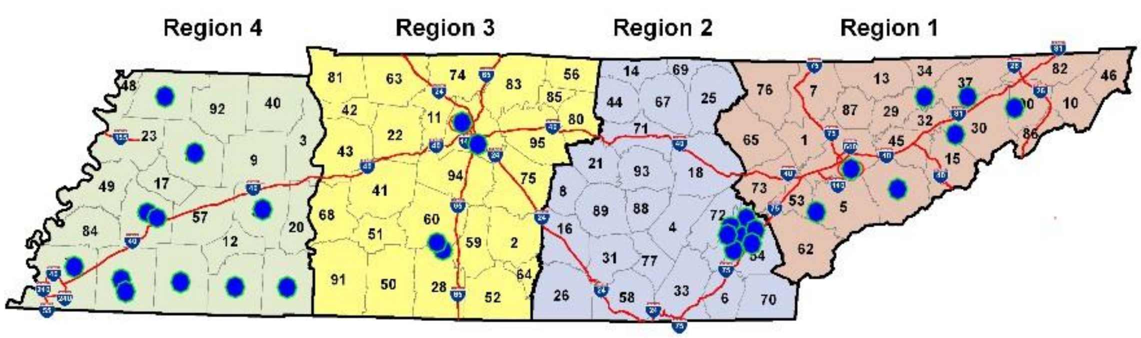


Figure 4 Asphalt Mixtures Tested in the STudy

Three replicate cylindrical specimens 150 mm in diameter and 170 mm high were compacted to an air void content of 4 ± 0.5% for each mixture. Dynamic modulus test, as shown in Figure 5, was conducted on these specimens at three temperatures (10, 25, and 45 °C) and loading frequencies ranging from 0.01 to 25 Hz (0.01, 0.1, 0.2, 0.5, 1, 2, 5, 10, 20, 25Hz). Table 1 presents the dynamic modulus results of the six HMA mixtures. Figure 2 compares the dynamic modulus results of the six mixtures at 10 Hz. Figure 3 shows the master curves of the dynamic modulus of the mixtures. Partial test results were shown in the Appendix.

Figure 5 Dynamic Modulus Test Setup

The MEPDG software requires dynamic modulus of asphalt materials under 10~20F and under 120~150°F. It is hard to conduct the SPT test under these temperatures. However, we can obtain dynamic modulus of asphalt materials based on time temperature equivalence principle.

The modulus properties of asphalt concrete are known to be a function of temperature, rate of loading, age, and mixture characteristics such as binder stiffness, aggregate gradation, binder content, and air voids. To account for temperature and rate of loading effects, the modulus of the asphalt concrete at all analysis levels will be determined from a master curve constructed at a reference temperature of 70°F.

The dynamic modulus master curve can be represented by the sigmoidal function described by Equation (5) listed below.

$$\log|E^*| = \delta + \frac{\alpha}{e^{\beta+\gamma \log f_r}} \quad (5)$$

where E* = dynamic modulus.

t_r = time of loading at the reference temperature.

δ,α= fitting parameters; for a given set of data, δ represents the minimum value of E* and δ+α represents the maximum value of E*.

β, γ= parameters describing the shape of the sigmoidal function.

The fitting parameters δ and α depend on aggregate gradation, binder content, and air void content. The fitting parameters β and γ depend on the characteristics of the asphalt binder and the magnitude of δ and α. The sigmoidal function describes the time dependency of the modulus at the reference temperature. The shift factors describe the temperature dependency of the modulus. Equations (6) and (7) provide the general form of the shift factors.

$$t_r = \frac{t}{a(T)} \quad (6)$$

$$\log(t_r) = \log(t) - \log\,[a(T)] \quad (7)$$

where

tr = time of loading at the reference temperature.

t = time of loading at a given temperature of interest.

a(T) = shift factor as a function of temperature.

T = temperature of interest.

Therefore, the time of loading at the reference temperature can be calculated for any time of loading at any temperature. Then the appropriate modulus can be calculated from Equation (5) using the time of loading at the reference temperature. Figure 6 is an example of the master curve.

Microsoft EXCEL SOLVER template was used to do nonlinear regressions of those dynamic moduli obtained in the SPT tests. With nonlinear regressions, we obtained dynamic moduli of asphalt materials under 15°F, 50°F, 77°F and 130°F, as shown in Appendix C.

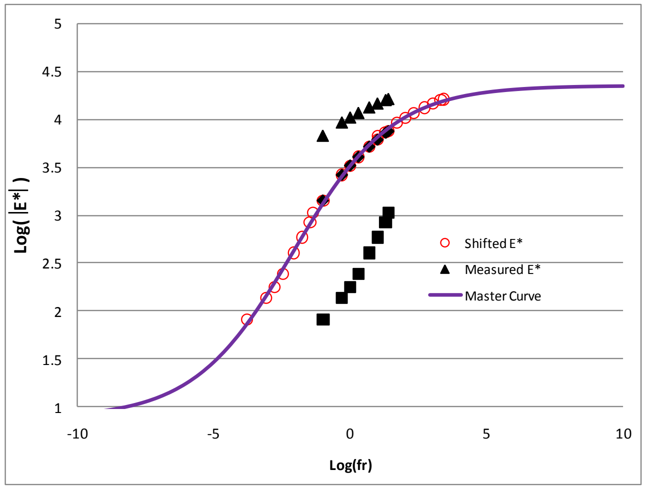


Figure 6 Master Curve Example

CHAPTER 4 RESILIENT MODULI OF GRANULAR MATERIALS

The laboratory resilient modulus test was conducted on five gravel base materials at their optimum moisture content (OMC) and maximum dry density (MDD) in accordance with AASHTO T307-99. Table 2 presents the testing sequences for these base materials. The cyclic load was applied over a period of 0.1 second followed b a 0.9 second rest period. The resilient modulus is defined as follows:

$$M_r = \frac{S_{cyclic}}{\varepsilon_r} \tag{8}$$

where, M_r = resilient modulus, S_{cyclic} = cyclic (resilient) axial stress, and ε_r = resilient (recovered) axial strain.

Table 2 Testing Sequences for Base Materials in AASHTO T307-99

Sequence No.	Confining Pressure, S_3	Max. Axial Stress, S_{max}	Cyclic Stress, S_{cyclic}	Contact Stress, $0.1S_{max}$	No. of Load Applications
	psi	psi	psi	psi	
0	15	15	13.5	1.5	500-1000
1	3	3	2.7	0.3	100
2	3	6	5.4	0.6	100
3	3	9	8.1	0.9	100
4	5	5	4.5	0.5	100
5	5	10	9	1	100
6	5	15	13.5	1.5	100
7	10	10	9	1	100
8	10	20	18	2	100
9	10	30	27	3	100
10	15	10	9	1	100
11	15	15	13.5	1.5	100
12	15	30	27	3	100
13	20	15	13.5	1.5	100
14	20	20	18	2	100
15	20	40	36	4	100

The resilient modulus results were fitted to Equation (4) and the regression constants are presented in Table 3. The comparison of the measured resilient moduli and those predicted with the regression equations is presented in

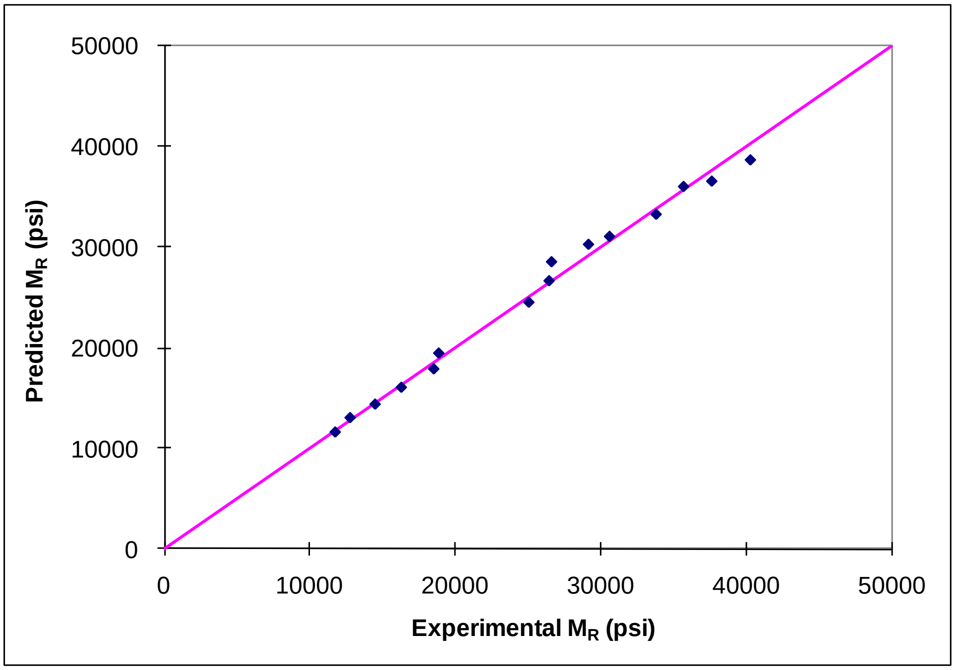
Table 4 and also shown in Figure 7.

Table 3 Regression Constants for Gravel Base Materials

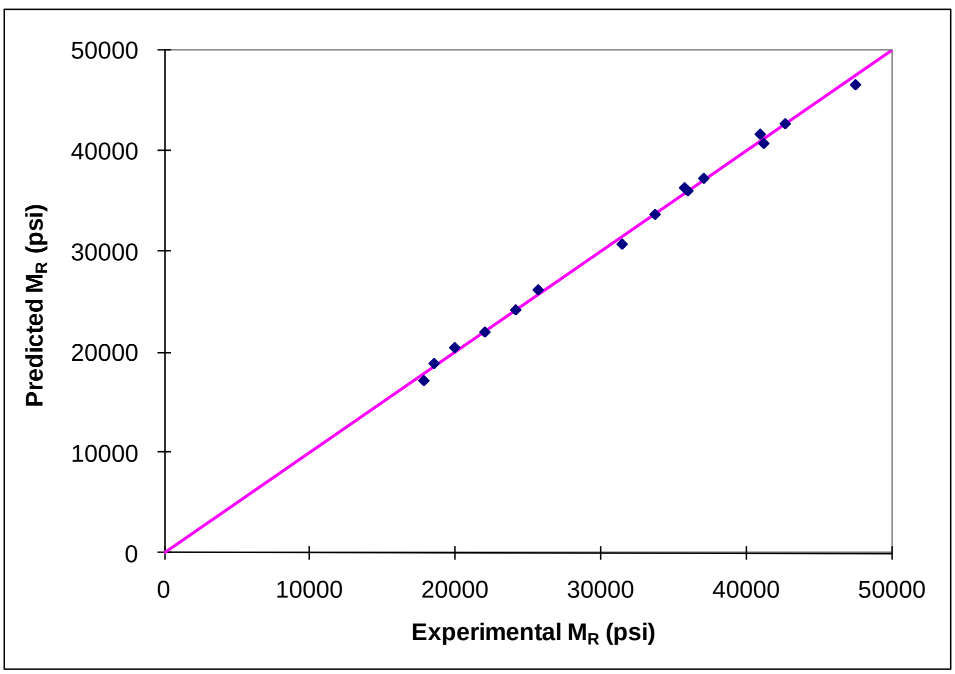
Base Materials	k_1	k_2	k_3	R^2
Ford Construction-Troy	936.8484	0.652599	-0.25414	0.99437
JR Hayes-Paris	1317.709	0.486874	-0.04735	0.997031
Memphis Stone-North Plant	1245.798	0.543191	0.015553	0.988769
Standard Construction-FR RD	1058.419	0.559816	0.056372	0.992948
Standard Construction-Stantonville	1064.024	0.569743	0.021645	0.995821

Table 4 Comparison of Measured and Predicted Resilient Moduli

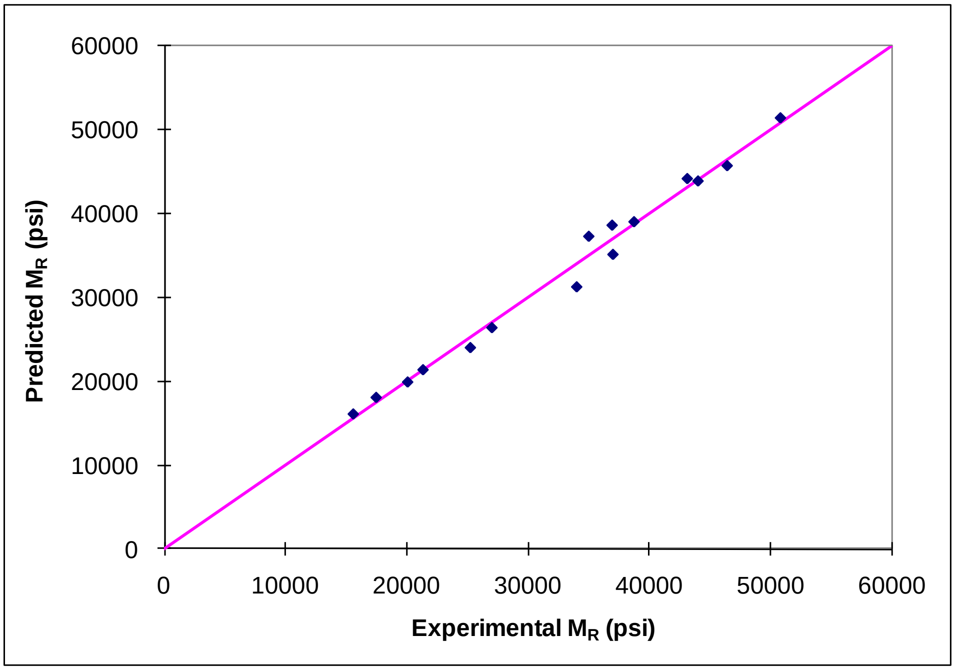
Sequence	Ford Constr. -Troy		JR Hayes - Paris		Memphis Stone -Deadfall		Standard Constr.- FR RD		Standard Constr.- Stantonville	
	Meas.	Pred.	Meas.	Pred.	Meas.	Pred.	Meas.	Pred.	Meas.	Pred.
1	11720	11575	17892	17201	15588	16141	13758	13696	14328	13678
2	12742	13006	18595	18910	17479	18118	15375	15445	15050	15390
3	14464	14344	20009	20487	20079	19956	16400	17082	16576	17052
4	16265	16019	22086	22032	21349	21421	19566	18380	18695	18388
5	18495	17846	24202	24232	25255	24063	21150	20822	20734	20787
6	18836	19417	25747	26226	27036	26442	21895	22993	22507	23022
7	25037	24467	31515	30768	34037	31316	28303	27335	28254	27448
8	26430	26607	33778	33722	37030	35187	32022	31037	31376	31034
9	26597	28491	35801	36373	36962	38674	33089	34429	33329	34376
10	29139	30221	36031	36063	35034	37345	32078	32781	31771	33050
11	30586	31011	37123	37306	38769	39085	33955	34504	34236	34673
12	33793	33206	41248	40778	44055	43952	39949	39345	40056	39294
13	35680	35966	41002	41700	43166	44234	38042	39216	39367	39495
14	37616	36500	42722	42740	46460	45777	40849	40798	41934	40963
15	40274	38611	47550	46608	50857	51485	48396	46641	47194	46417



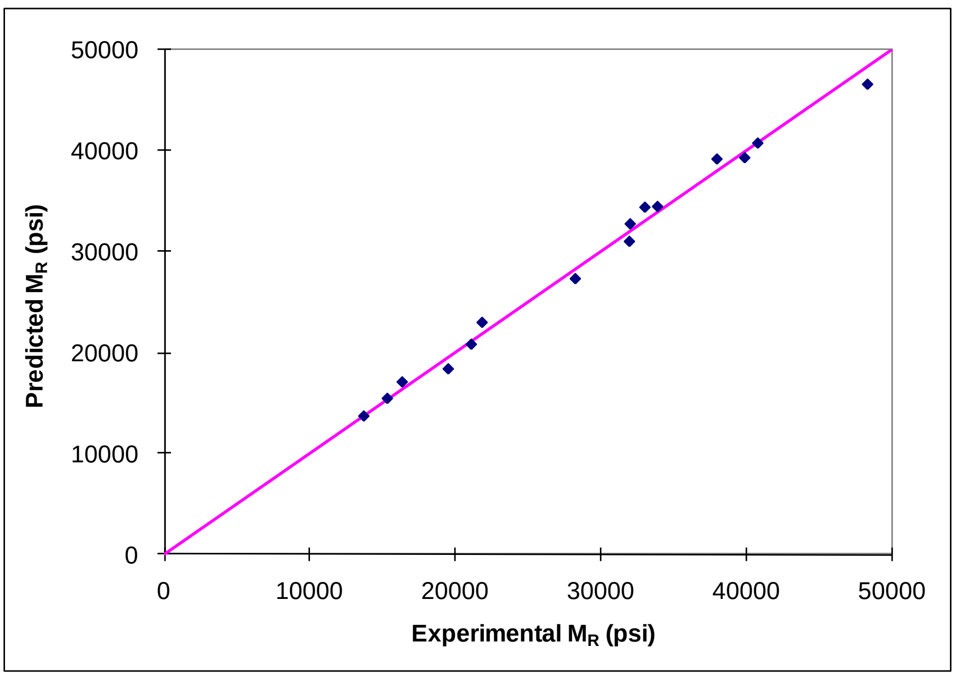
(a) Ford Construction - Troy



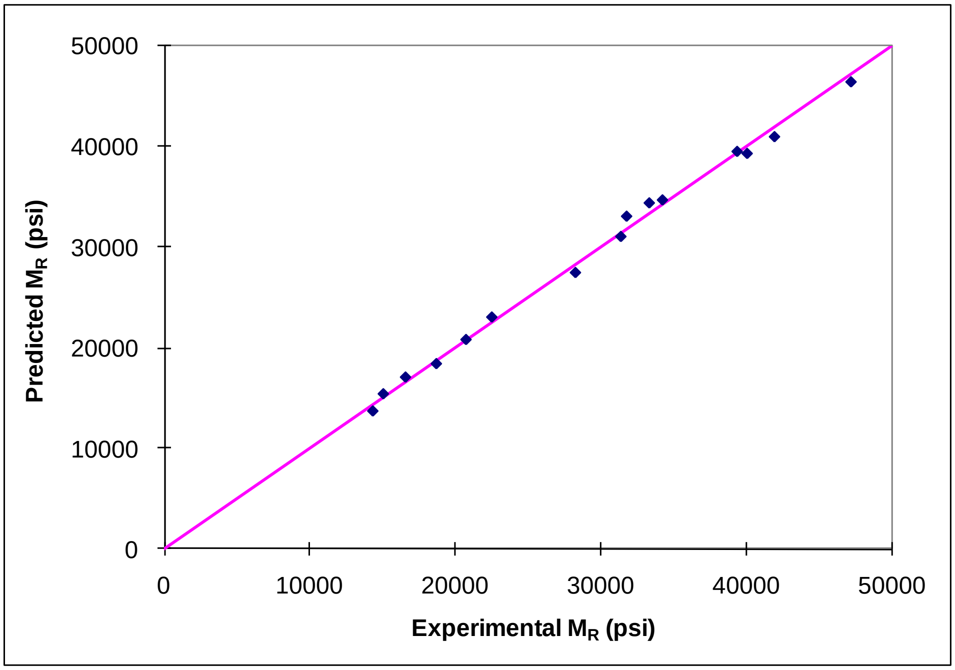
(b) JR Hayes - Paris



(c) Memphis Stone - Deadfall



(d) Standard Construction – FR RD



(e) Standard Construction - Stantonville

Figure 7 Comparison of Measured and Predicted Resilient Moduli

CHAPTER 5 COMPARISON OF PREDICTED PAVEMENT LIVES FROM AASHTO 1993 DESIGN GUIDE AND MEPDG

5.1 Introduction

The mechanistic-empirical models included in the new design software can directly consider effects and interactions of inputs on structural distress and ride quality. To apply the new design guide, many states have dedicated to data collecting/upgrading (Wang et al., 2008; Aguiar-Moya, et al., 2008), model testing (Garcia et al., 2007; Saxena et al., 2010; Banerjee et al., 2009), sensitivity analysis (Ayyala et al., 2010; Aguiar-Moya et al., 2009), and software evaluation, validation and calibration.

Maryland (Schwartz et al., 2007) analyzed the sensitivity of the MEPDG performance prediction to input parameters, including traffic, environmental conditions, and material properties. They found MEPDG was very sensitive to climate variations and different material properties. They recommended local calibrations for different materials and every region. Minnesota (Velasquez et al., 2009) and Iowa (Sunghwan et al., 2010) utilized field performance data to evaluate the MEPDG performance predictions. Minnesota used MnROAD pavement sections as well as other pavement sections located in Minnesota and neighboring states as performance data sources. Ohio (Mallela et al., 2009), Arizona (Souliman et al., 2010) and Washington (Li et al., 2009, 2010) developed their guidelines for MEPDG procedures.

It can be summarized from the above literatures that local calibration for MEPDG is necessary in that the national calibrated models for distress and/or roughness either under-predicted or over-predicted pavement performance for each specific state. Besides, there are still some issues of roughness and longitudinal cracking models of MEPDG software. The frequently utilized pavement performance data sources include the Minnesota MnROAD test roads, states' PMS system and LTPP database. Since materials, climate, and traffic significantly influence the pavement performance prediction, it is of great importance to calibrate the MEPDG models for local transportation agencies. In the phase I the predicted lives of pavements form MEPDG were compared with those lives from AASHTO 1993 Design Guide, which would demonstrate the advantage of the new design/analysis procedure. The phase II would focus on the validation and calibration of MEPDG.

5.2 Data Collection

Traffic

The concept of axle load spectra was introduced into the MEPDG which requires truck counts by week days and months for all truck types from Class 4 to Class 13 (FHWA). The traffic volume adjustment factors for truck distribution, vehicle class distribution and axle load distribution factors are required. Some factors such as axle load distribution factor and percentage of vehicles in the design lane are very sensitive inputs (Oman, 2010). However, due to the unavailability of the detailed axle load distribution in HPMA, national default axle load spectra were used in this paper. The Equivalent Single Axle Load (ESAL) collected from HPMA was selected as a traffic level indicator.

Climate

Intrastate variations in climate conditions have a non-negligible influence on the MEPDG performance prediction (Schwartz, 2007). The default climate data of weather stations located in Tennessee was tested, inspected and judged acceptable for the validation efforts. Stations with incomplete data cannot be used alone in MEPDG. Utilizing these stations when creating a virtual weather station through interpolation may only decrease the quality of prediction (Johanneck et al., 2010). It is observed that the Knoxville station in Tennessee missed some data of some months. The nearest weather station with complete data was used instead of this station in the analysis. According to Tennessee Water Science Center, the groundwater table is 6 ft deep or more. Since distress predictions for asphalt concrete pavement sections are not affected by depths greater than 4 ft (Witczak et al., 2006; Zapata, 2009), the depth to groundwater table was assumed to be 6 ft for all road sections.

Pavement Structures and Material Properties

The main highways in Tennessee were selected, as shown in Appendix, including 16 interstate highway sections and 4 state route sections.

Since material characteristics are factors our research focused on, we use default climate and traffic parameters in the analysis. And in order to estimate whether or not the MEPDG software underestimates pavements' lives and their performance, we used default values (Level 3) of structural layers as their inputs. Furthermore, TDOT offered CBR and/or R-value (which belong to input level 2) of subgrade, which are adopted in the analysis.

Select Proper AADT

Since MEPDG uses the full axle-load spectrum data for each axle type for both new pavement and rehabilitation design procedures, The ESAL approach used in traffic characterization in previous versions of the Guide for Pavement Design (AASHTO 1993) needs to be converted into cumulative heavy truck numbers, which is an output of MEPDG software.

There is a relationship between ESAL and accumulative heavy truck numbers:

ESAL=average truck factor*accumulative heavy truck numbers

Where average truck factor=(sum (number of axles*load equivalency factor))/number of vehicles

According to the default axle spectra distribution, get the average truck factor valued 0.682. In the design life of 20years, after trials, proper AADT were got, as shown in Table 1, and calculated ESALs and real ESALs offered by TDOT are shown in Figure 8. It indicates that AADTs in

Table 5 is proper to reflect traffic conditions in the routes.

Table 5 AADT Inputs for Selected Routes

Route	I-24	I-24	I-26	I-40	I-40	I-40	I-40	I-65	I-75	I-75
County	Rutherford	Coffee	Sullivan	Cocke	Cumberland	Davidson	Shelby	Williamson	Anderson	Hamilton-1
AADT	800	1200	2700	850	1100	2800	2900	7500	2600	18500
Route	I-75	I-81	I-155	I-240	I-440	I-540	SR-15	SR-385	SR-30	SR-331
County	Hamilton-2	Greene	Dyer	Shelby	Davidson	Knox	Lawrence/Giles	Shelby/ Fayette	Mcminn	Knox
AADT	12600	1420	1050	2200	3400	2750	900	4300	660	330

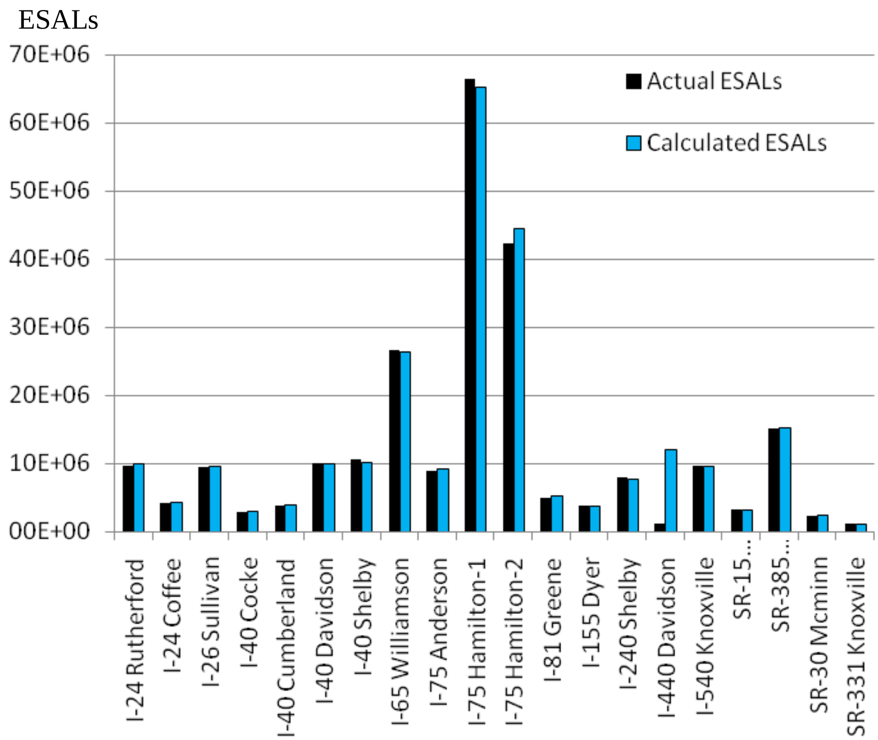


Figure 8 Calculated ESALs vs. Actual ESALs

5.3 Predicted Life vs. Design Life

The pavement life is defined as the length of time a pavement structure is being designed for, including the time from construction to major rehabilitation. Figure 9) and Figure 10) show predicted lives of several asphalt pavements and cement concrete separately. It is indicated that no matter to asphalt pavement or cement concrete pavement, MEPDG highly underestimated pavements' lives comparing to AASHTO 1993, which was used for designing these routes before.

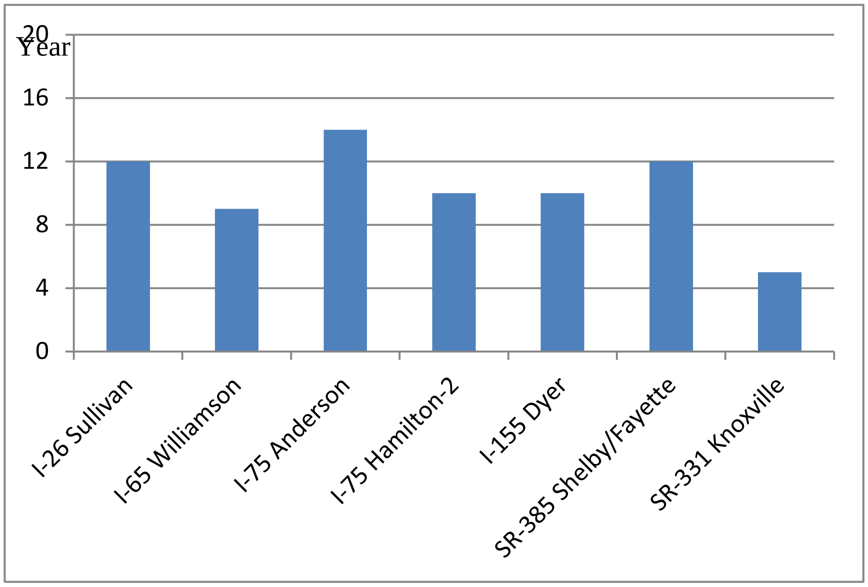


Figure 9 Predicted Lives of Several Asphalt Pavements

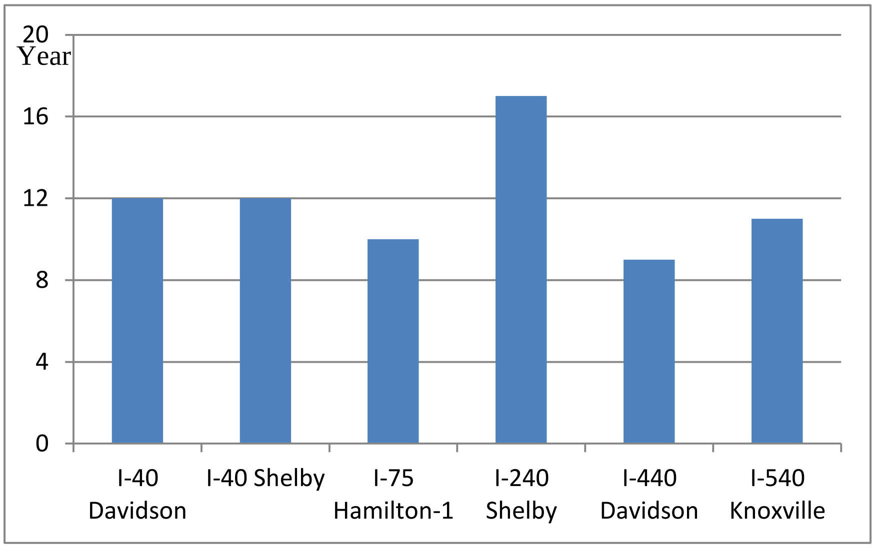


Figure 10 Predicted Lives of Cement Concrete Pavements

Typical pavement performance

For cement concrete pavements, Mean Joint Faulting is the major failure, as shown

in

Table 6.

Table 6 Performance of PCC Routes Predicted by MEPDG at Their Predicted Life

Route	County	Year	Terminal IRI (in/mi)	Transverse Cracking (% slabs cracked)	Mean Joint Faulting (in)
I-40	Davidson	12	101.3	0	0.07
		13	104.1	0	0.077/87.28
I-40	Shelby	12	100.6	0.1	0.071
		13	103.2	0.1	0.076/87.98
I-75	Hamilton-1	10	100.5	0	0.07
		11	102.9	0	0.074/88.91
I-240	Shelby	17	100.7	0.1	0.071
		18	102.6	0.1	0.074/88.79
I-440	Davidson	9	99	0.1	0.068
		10	102.6	0.1	0.074/88.85
I-540	Knoxville	11	99.3	0.1	0.068
		12	103	0.1	0.075/88.57

For asphalt pavements, among all of distresses and terminal IRI, permanent deformation of Asphalt concrete usually first pass its limit, as shown in **Error! Not a valid bookmark self-reference.**. Other distresses were very far away from their limits.

Table 7 Performance of Asphalt Pavement Routes Predicted by MEPDG at Their Predicted Life

Route	**County**	**Year**	**Terminal IRI (in/mi)**	**AC Surface Down Cracking (Long. Cracking) (ft/mile)**	**AC Bottom Up Cracking (Alligator Cracking) (%)**	**AC Thermal Fracture (Transverse Cracking) (ft/mi)**	**Permanent Deformation (AC Only) (in)**	**Permanent Deformation (Total Pavement) (in)**
I-24	Rutherford	20	108.8	0	0.1	1	0.17	0.37
		21	111.5	0	0.2	1	0.17/89.98	0.38
I-26	Sullivan	12	96.1	0	0.2	1	0.17	0.43
		13	98.5	0	0.3	1	0.18/88.64	0.44
I-65	Williamson	9	90.2	0	0.2	1	0.17	0.42
		10	92.4	0	0.3	1	0.18/87.74	0.43
I-75	Anderson	14	94.8	0	0.1	1	0.17	0.42
		15	96.8	0	0.2	1	0.18/87.12	0.43
I-75	Hamilton-2	3	86	0	0.1	1	0.15	0.53
		4	88.8	0	0.1	1	0.18/88.34	0.57
I-155	Dyer	10	85.5	0	0	1	0.17	0.34
		11	87.2	0	0	1	0.18/88.00	0.35
SR-385	Shelby/ Fayette	12	94.7	0	0.2	1	0.17	0.41
		13	97.2	0	0.2	1	0.18/87.52	0.42
SR-331	Knoxville	5	79.4	7.1	0.2	1	0.06	0.31
		6	81.3	9.7/89.48	0.2	1	0.06	0.32

Note: The number x/y mean IRI or some a distress x has y percent probabilities to stay below its criterion.

The trends of several distresses and IRI of I-26 Sullivan asphalt pavement in its life, i.e. 12 years, are listed below Figures (11) to (14). These figures indicated permanent deformation due only to asphalt concrete is the main reason for asphalt pavement's failure.

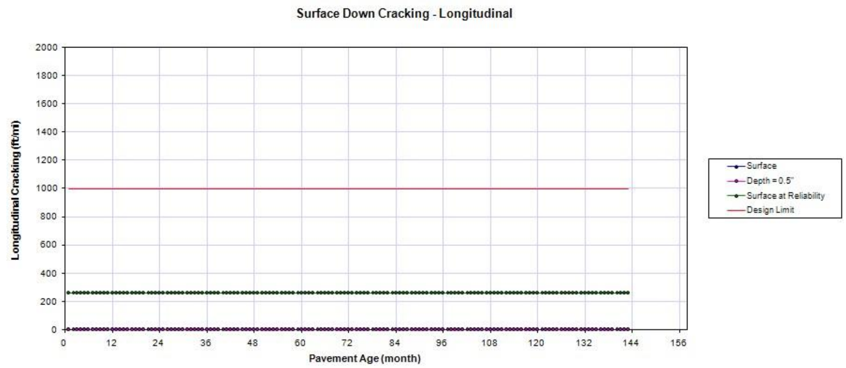


Figure 11 Surface-Down Cracking (Longitudinal) Development Trend of I-26 Sullivan Asphalt Pavement

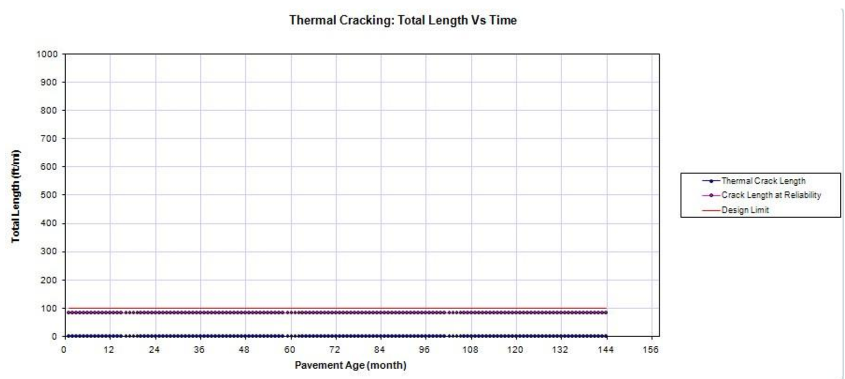


Figure 12 Thermal Cracking Development Trend of I-26 Sullivan Asphalt Pavement

Figure 13 Rutting Development trend of I-26 Sullivan Asphalt Pavement

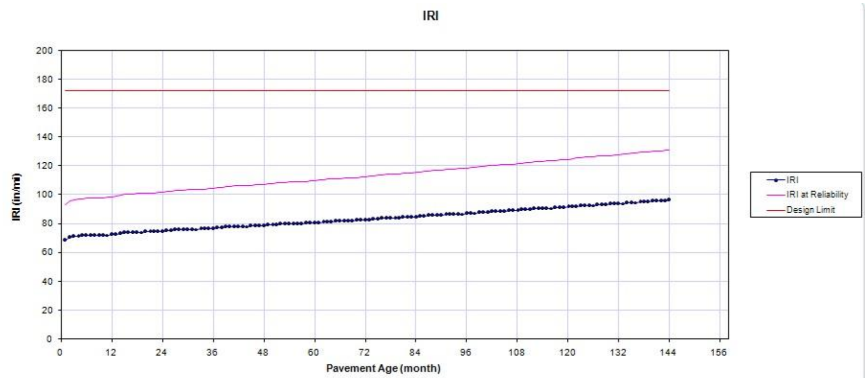


Figure 14 IRI Development trend of I-26 Sullivan Asphalt Pavement

The predicted faulting, load transfer efficiency, and IRI of I-44 Davison cement concrete pavement in its life, i.e. 9 years, are listed below Figures (15) to (17). Mean Joint Faulting quickly increased to the limit, while load transfer efficiency decrease as time went by.

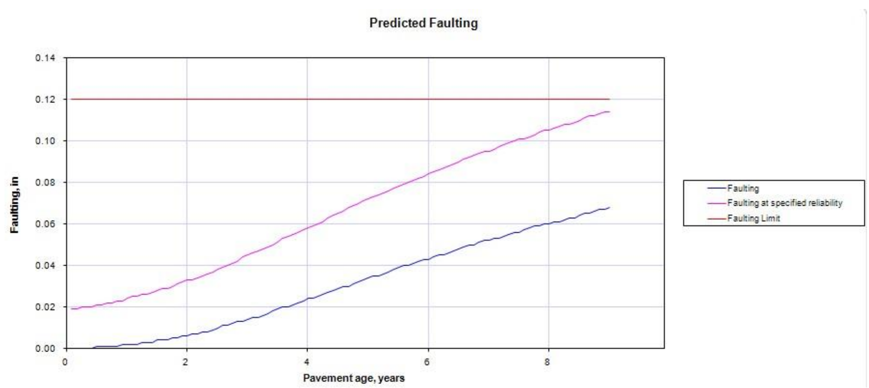


Figure 15 Predicted Mean Joint Faulting of I-440 Davison Cement Concrete Pavement

Figure 16 Predicted load Transfer Efficiency of I-440 Davison Cement Concrete Pavement

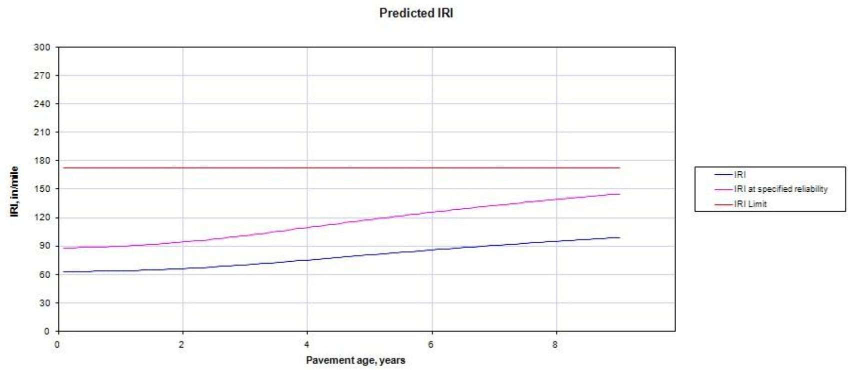


Figure 17 Predicted IRI I-440 Davison Cement Concrete Pavement

Empirical equations are used to relate observed or measurable phenomena (pavement characteristics) with outcomes (pavement performance). The 1993 AASHTO Guide basic design equation (Equation 1) for flexible pavements is widely used.

Resilient modulus (M_R) is not a fixed number. MR varies greatly with the state of stress and moisture, for all types of materials. Therefore, one CBR value cannot be correlated with the multitude of values M_R can take as a function of moisture and state of stress. Even if moisture is fixed, say to optimum moisture content, still the state of stress and the compactive effort (density) will make a difference. Therefore, none of the CBR-M_R relationships can be correct. Several researchers tried to develop different CBR- M_R relationships for different types of materials but generally no one was successful in finding unique and accurate relationships. However, of the several CBR-MR relationships available, the one used in the M-E Guide gives the most reasonable estimates of MR as a function of CBR (MR(psi)=1500*CBR).

According to the ESALs of these Routes in the end of the design year (20 years) offered by TDOT, with assumed traffic increase rate 4%, we can calculate the service lives of these routes can take under AASHTO 1993, shown in

Table 8.

Table 8 Predicted Service Lives of These Asphalt Pavement Routes

Route	County	Estimated Service Life (Year)	
		MEPDG 2002	AASHTO 1993
I-24	Rutherford	20	40
I-24	Coffee	26	38
I-26	Sullivan	12	25
I-40	Cocke	28	55
I-40	Cumberland	28	49
I-65	Williamson	9	24
I-75	Anderson	14	41
I-75	Hamilton-2	10	17
I-81	Greene	30	
I-155	Dyer	10	23
SR-15	Lawrence/giles	20	23
SR-385	Shelby/Fayette	12	31
SR-30	Mcminn	26	33
SR-331	Knoxville	5	23

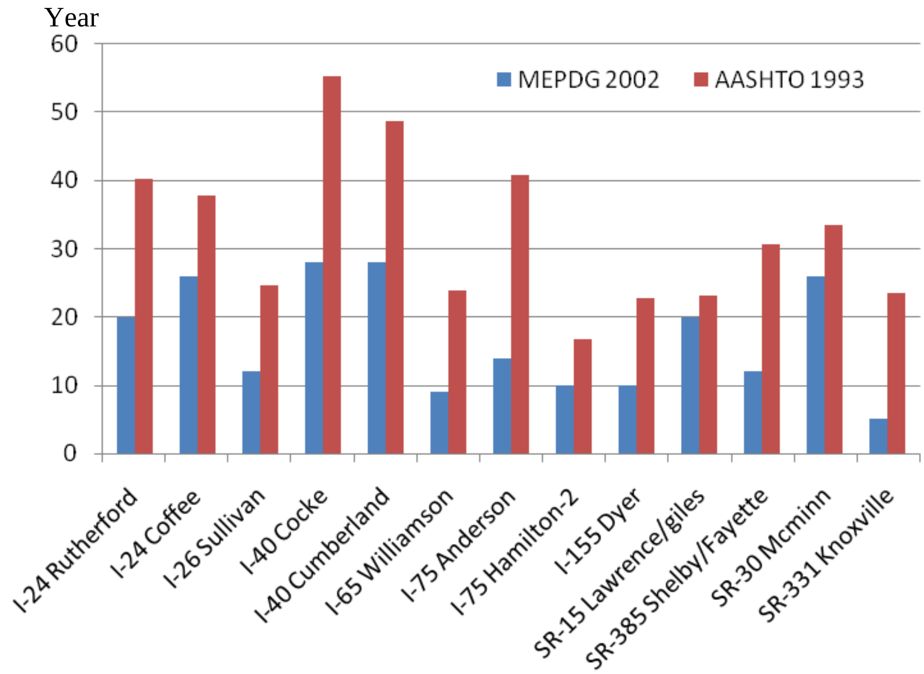


Figure 18 AASHTO 1993 Predicted Lives vs. MEPDG 2002 Predicted Lives of Asphalt Pavements

From Figure 18, the AASHTO 1993 indicated that most of these routes could lead to a longer life than 20 years. However, with the national default material input, the MEPDG 2002 software predicted a much shorter life for each route.

5.4 Asphalt Layer Input Level 1 vs. Level 3

The MEPDG software was run with dynamic moduli as the input of asphalt materials. Life and performance of asphalt pavements offered by TDOT were evaluated. Table 9 summarizes asphalt pavement's lives. Also a contrast is shown in Figure 19 Predicted Lives of Asphalt Pavements under Asphalt Layer Input Level 3 vs. Level 1Figure 19. It is indicated that asphalt pavements can last longer when we adopt dynamic moduli from lab comparing to adopt default values in the software. In the last quarterly report, it is concluded that using default values the MEPDG offers relatively conservative prediction on the service life of asphalt pavements.

Table 9 Predicted Lives of Asphalt Pavements under Ashpalt Layer Input 3 vs. Input 1

Section	Material inputs	
	Asphalt Layer Input Level 1	Asphalt Layer Input Level 3
I-24 Rutherford	24	25
I-24 Coffee	23	24
I-26 Sullivan	8	22
I-40 Cocke	25	26
I-40 Cumberland	23	24
I-65 Williamson	9	14
I-75 Anderson	16	27
I-75 Hamilton-2	1	3
I-81 Greene	20	27
I-155 Dyer	12	17
SR-15 Lawrence/Giles	16	23
SR-385 Shelby/Fayette	16	19
SR-30 McMinn	21	25
SR-331 Knoxville	10	10

It should be note that Case 1 uses dynamic modulus E*as asphalt mixture inputs, complex modulus G* and phase angle δ as asphalt binder inputs, uses layer coefficient as base inputs, and uses CBR or k value as subgrade inputs. Case 2uses national default values as asphalt layers' inputs, uses layer coefficient as base inputs, and uses CBR or k value as subgrade inputs. The only difference between case 1 and case 2 is the asphalt layer's inputs that the former uses level 1 inputs while the later uses level 3.

Because the dynamic modulus is obtained from lab, the software offers relatively accurate prediction for pavement life and performance. Therefore, according to the results we got, it indicates that with dynamic modulus of asphalt materials involved in the MEPDG software, these asphalt pavements lead to longer lives, which means TDOT can save the budget on pavement maintenance or get a relatively pavement thickness which reduces construction cost.

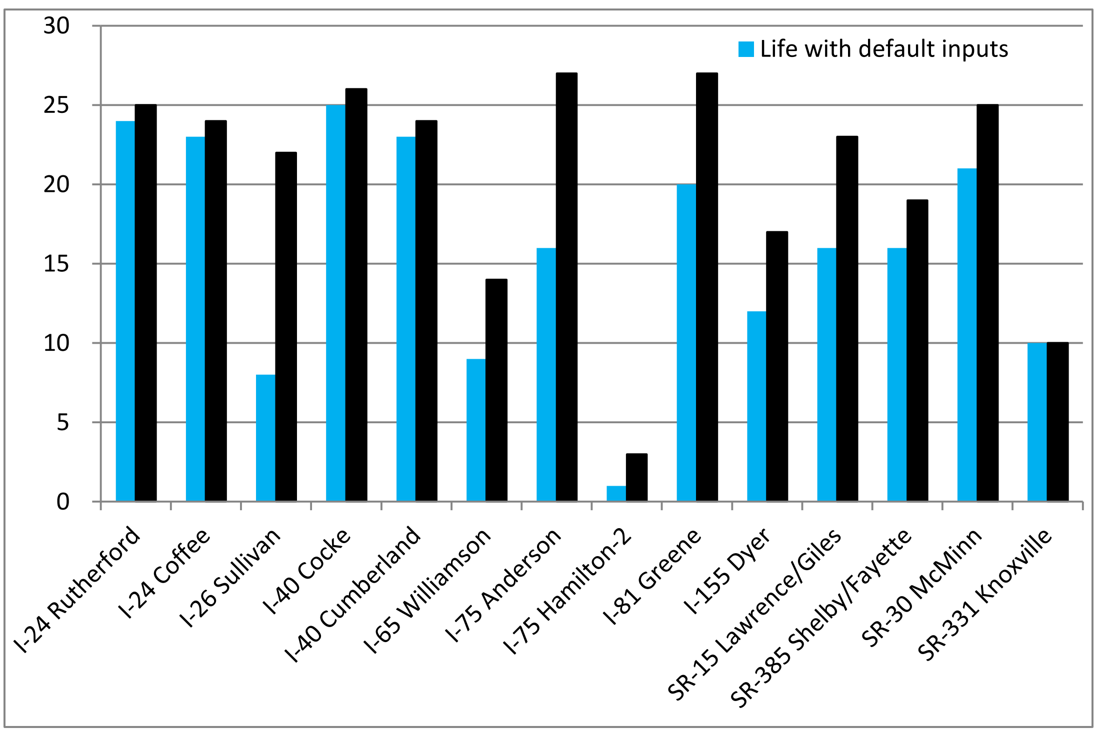


Figure 19 Predicted Lives of Asphalt Pavements under Asphalt Layer Input Level 3 vs. Level 1

CHAPTER 6 CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

This study is focused on a major step toward the implementation of MEPDG in Tennessee - developing typical material input values for mechanistic-empirical pavement design. To reach this goal, two important input values, dynamic moduli of asphalt concrete (28 types) and resilient moduli of base materials (8 types) were obtained through laboratory testing.

Based on the time-temperature superposition principle, the master curves of dynamic modulus were constructed for the asphalt mixtures. Using dynamic modulus master curve, the dynamic modulus inputs for asphalt mixtures can be obtained for the MEPDG analysis on level 1.

The regression coefficients, i.e. k_1, k_2, $k_{3,}$ of Uzan Model for base materials were acquired. Trials have been made to use the regression coefficient in the MEPDG analysis on level 1 for base materials. However, the trials were not successful. Further trials will be made to use the coefficients for base materials in MEPDG analysis in Phase II study.

The AASHTO 1993 Design Guide and the MEPDG were compared in this study. For the comparison, 20 pavement sections were selected from the state of Tennessee. The predicted lives from the MEPDG with national default input values were generally lower than the predicted lives from the AASHTO 1993 Design Guide. With national default input values, the MEPDG software could not accurately predict the lives of TDOT, regardless of asphalt concrete pavement or portland cement concrete pavement. Therefore, the MEPDG software could not be used for design purpose Without local material database. It is imperative to establish a local material characteristic database and to validate the inputs of pavement materials in Tennessee.

With the laboratory-measured dynamic modulus values of the TDOT asphalt mixtures as level 1 input, the MEPDG software predicted longer service lives, which indicates that some budgets on pavement maintenance or construction might be saved.

6.2 Recommendations

The implementation of MEPDG in Tennessee requires not only the development of property database of local materials but also validation and calibration of MEPDG with the pavement management system (PMS). Therefore, the following tasks are suggested in the Phase II study:

- Adding Coefficient of Thermal Expansion (CTE) of PCC into the property database.
- For evaluation and calibration of overlay pavements, non-destructive tests, such as falling weight deflectometer can be carried out on existing pavements.
- Data collection, filtration and standardization from PMS for comparison with the results from MEPDG analysis.
- Validation and/or calibration of MEPDG prediction models for local conditions in Tennessee.

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APPENDICES

APPENDIX A Dynamic Modulus Results of Several HMA Concrete

(MPa)

Mixture	Asphalt Cement	Coarse Aggregate	Location	|E*|	Frequencies (Hz) at 10 °C									
					25	20	10	5	2	1	0.5	0.2	0.1	0.01
411D	PG 67-22	Gravel	Jackson, TN	Avg.	16925	16570	15390	14236	12693	11384	10356	8877	7792	4639
				S.D.	907	860	784	727	674	612	484	381	304	53
				%CV	5.4	5.2	5.1	5.1	5.3	5.4	4.7	4.3	3.9	1.1
	PG 70-22	Gravel	Dresden, TN	Avg.	15732	15310	14091	12899	11388	10293	9230	7884	6951	4396
				S.D.	661	605	560	527	497	508	537	553	529	459
				%CV	4.2	4.0	4.0	4.1	4.4	4.9	5.8	7.0	7.6	10.4
		Gravel	Jackson, TN	Avg.	16978	16604	15446	14305	12758	11591	10415	8915	7802	4651
				S.D.	668	650	588	520	435	388	337	280	270	239
				%CV	3.9	3.9	3.8	3.6	3.4	3.3	3.2	3.1	3.5	5.1
	PG 76-22	Gravel	Memphis, TN	Avg.	15628	15156	13858	12577	10967	9784	8641	7263	6317	3737
				S.D.	1043	1022	937	845	760	673	613	531	440	228
				%CV	6.7	6.7	6.8	6.7	6.9	6.9	7.1	7.3	7.0	6.1
		Gravel	Mullins, TN	Avg.	16140	15714	14480	13256	11636	10464	9293	7805	6798	3950
				S.D.	307	286	277	276	278	273	232	180	177	146
				%CV	1.9	1.8	1.9	2.1	2.4	2.6	2.5	2.3	2.6	3.7
411E	PG 67-22	Limestone	Troy, TN	Avg.	15398	14950	13648	12363	10701	9494	8344	6887	5884	3197
				S.D.	1352	1311	1217	1124	969	905	819	675	565	308
				%CV	8.8	8.8	8.9	9.1	9.1	9.5	9.8	9.8	9.6	9.6

(MPa)

Mixture	Asphalt Cement	Coarse Aggregate	Location	|E*|	Frequencies (Hz) at 25 °C									
					25	20	10	5	2	1	0.5	0.2	0.1	0.01
411D	PG 67-22	Gravel	Jackson, TN	Avg.	8526	8229	7152	6131	4898	4073	3336	2497	1979	833
				S.D.	377	425	411	384	353	322	295	245	193	40
				%CV	4.4	5.2	5.7	6.3	7.2	7.9	8.8	9.8	9.8	4.8
	PG 70-22	Gravel	Dresden, TN	Avg.	7426	7099	6104	5206	4159	3481	2873	2221	1857	1010
				S.D.	384	398	331	262	206	167	140	122	126	115
				%CV	5.2	5.6	5.4	5.0	5.0	4.8	4.9	5.5	6.8	11.4
		Gravel	Jackson, TN	Avg.	7898	7541	6469	5505	4368	3624	2973	2258	1847	952
				S.D.	758	745	710	654	575	512	438	349	275	112
				%CV	9.6	9.9	11.0	11.9	13.2	14.1	14.7	15.5	14.9	11.8
	PG 76-22	Gravel	Memphis, TN	Avg.	7195	6869	5845	4941	3869	3165	2557	1885	1488	652
				S.D.	1061	1032	938	849	707	627	540	433	341	137
				%CV	14.7	15.0	16.0	17.2	18.3	19.8	21.1	23.0	22.9	21.1
		Gravel	Mullins, TN	Avg.	7384	7005	5945	4984	3886	3174	2564	1904	1525	726
				S.D.	752	737	695	632	547	482	413	315	235	50
				%CV	10.2	10.5	11.7	12.7	14.1	15.2	16.1	16.5	15.4	6.9
411E	PG 67-22	Limestone	Troy, TN	Avg.	6634	6292	5249	4318	3268	2590	2034	1456	1136	523
				S.D.	733	699	597	504	413	341	276	204	159	71
				%CV	11.1	11.1	11.4	11.7	12.6	13.2	13.6	14.0	14.0	13.5

(MPa)

Mixture	Asphalt Cement	Coarse Aggregate	Location	|E*|	Frequencies (Hz) at 45 °C									
					25	20	10	5	2	1	0.5	0.2	0.1	0.01
411D	PG 67-22	Gravel	Jackson, TN	Avg.	2182	1957	1493	1107	729	516	380	255	193	94
				S.D.	80	39	80	68	54	42	32	22	15	8
				%CV	3.7	2.0	5.4	6.2	7.4	8.2	8.3	8.6	7.9	8.1
	PG 70-22	Gravel	Dresden, TN	Avg.	1975	1792	1386	1065	734	540	418	296	234	117
				S.D.	102	100	96	86	68	58	49	38	33	21
				%CV	5.1	5.6	6.9	8.1	9.3	10.7	11.6	12.7	14.3	17.8
		Gravel	Jackson, TN	Avg.	2487	2268	1740	1317	896	655	499	348	272	137
				S.D.	133	128	114	101	83	67	58	44	38	26
				%CV	5.3	5.6	6.5	7.7	9.3	10.3	11.5	12.8	13.9	19.0
	PG 76-22	Gravel	Memphis, TN	Avg.	1857	1665	1212	928	619	450	340	241	189	99
				S.D.	254	238	258	165	125	103	82	59	47	26
				%CV	13.7	14.3	21.2	17.8	20.1	22.9	24.1	24.3	24.8	26.0
		Gravel	Mullins, TN	Avg.	2312	2091	1567	1149	757	541	402	272	208	100
				S.D.	551	508	382	269	175	126	93	55	39	9
				%CV	23.9	24.3	24.4	23.4	23.2	23.3	23.0	20.2	18.7	9.3
411E	PG 67-22	Limestone	Troy, TN	Avg.	1476	1287	947	710	468	337	257	183	140	74
				S.D.	82	73	59	61	52	47	42	25	15	9
				%CV	5.5	5.7	6.2	8.6	11.2	14.0	16.3	13.5	10.5	12.1

(MPa)

Mixture	Asphalt Cement	Coarse Aggregate	Location	|E*|	Frequencies (Hz) at 10 °C									
					25	20	10	5	2	1	0.5	0.2	0.1	0.01
BM2	PG 64-22	Gravel	Mullins, TN	Avg.	19325	18845	17566	16242	14526	13190	11876	10143	8925	5395
				S.D.	1733	1736	1750	1725	1710	1700	1660	1507	1314	632
				%CV	9.0	9.2	10.0	10.6	11.8	12.9	14.0	14.9	14.7	11.7
	PG 67-22	Gravel	Jackson, TN	Avg.	19382	19275	18057	17026	15531	14198	13058	11553	10476	7169
				S.D.	3595	3917	3588	3624	3469	3024	2900	2714	2491	1567
				%CV	18.5	20.3	19.9	21.3	22.3	21.3	22.2	23.5	23.8	21.9
		Gravel	Savannah, TN	Avg.	18131	17762	16770	15706	14229	13109	11982	10497	9387	6035
				S.D.	1031	966	754	694	724	733	693	611	525	307
				%CV	5.7	5.4	4.5	4.4	5.1	5.6	5.8	5.8	5.6	5.1
		Gravel	Parsons, TN	Avg.	17159	16864	15850	14842	13483	12274	11357	10019	9027	6179
				S.D.	119	103	206	351	574	992	931	1029	1115	1413
				%CV	0.7	0.6	1.3	2.4	4.3	8.1	8.2	10.3	12.4	22.9
		Gravel	Troy, TN	Avg.	17207	16756	15522	14241	12605	11343	10152	8611	7513	4409
				S.D.	1557	1516	1420	1349	1225	1161	1087	976	861	583
				%CV	9.0	9.0	9.1	9.5	9.7	10.2	10.7	11.3	11.5	13.2
	PG 70-22	Gravel	Stantonville, TN	Avg.	15591	15224	14197	13088	11765	10806	9687	8467	7542	4793
				S.D.	544	544	528	468	525	562	475	524	491	418
				%CV	3.5	3.6	3.7	3.6	4.5	5.2	4.9	6.2	6.5	8.7

(MPa)

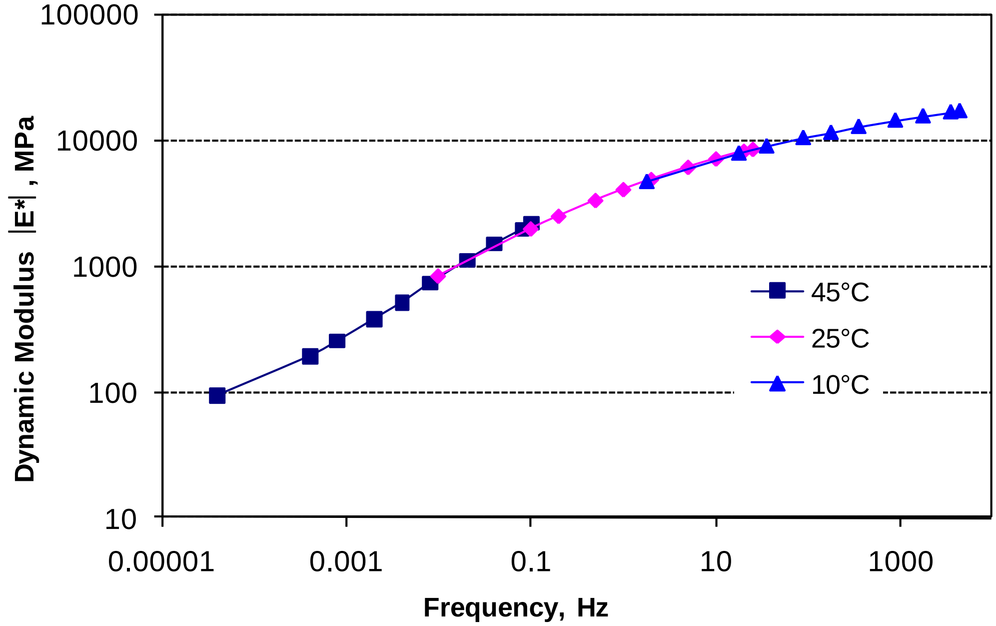
Mixture	Asphalt Cement	Coarse Aggregate	Location	|E*|	Frequencies (Hz) at 25 °C									
					25	20	10	5	2	1	0.5	0.2	0.1	0.01
BM2	PG 64-22	Gravel	Mullins, TN	Avg.	9719	9339	8026	6828	5393	4421	3536	2570	2006	876
				S.D.	200	177	277	291	262	266	250	179	112	41
				%CV	2.1	1.9	3.4	4.3	4.9	6.0	7.1	7.0	5.6	4.7
	PG 67-22	Gravel	Jackson, TN	Avg.	9714	9382	8310	7313	6099	5261	4495	3597	3053	1672
				S.D.	1149	1103	1019	951	875	790	708	598	504	242
				%CV	11.8	11.8	12.3	13.0	14.4	15.0	15.8	16.6	16.5	14.5
		Gravel	Savannah, TN	Avg.	10154	9724	8545	7462	6162	5281	4478	3542	2959	1551
				S.D.	1471	1394	1270	1158	1039	905	778	609	467	189
				%CV	14.5	14.3	14.9	15.5	16.9	17.1	17.4	17.2	15.8	12.2
		Gravel	Parsons, TN	Avg.	10880	10512	9470	8484	7192	6286	5473	4428	3712	1800
				S.D.	273	284	322	396	456	492	533	514	479	207
				%CV	2.5	2.7	3.4	4.7	6.3	7.8	9.7	11.6	12.9	11.5
		Gravel	Troy, TN	Avg.	7985	7579	6459	5441	4279	3519	2857	2132	1713	791
				S.D.	932	840	707	582	468	404	341	270	221	120
				%CV	11.7	11.1	11.0	10.7	10.9	11.5	11.9	12.7	12.9	15.2
	PG 70-22	Gravel	Stantonville, TN	Avg.	7509	7169	6231	5390	4361	3659	3052	2377	1985	1069
				S.D.	1248	1222	1148	1066	940	841	744	613	505	211
				%CV	16.6	17.0	18.4	19.8	21.6	23.0	24.4	25.8	25.4	19.8

(MPa)

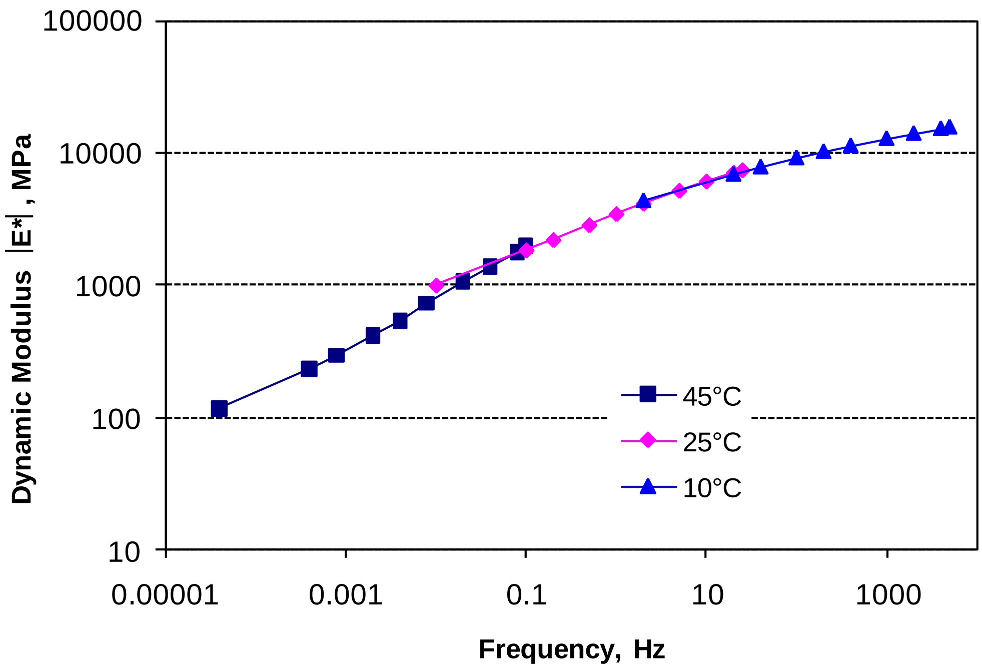
Mixture	Asphalt Cement	Coarse Aggregate	Location	|E*|	Frequencies (Hz) at 45 °C									
					25	20	10	5	2	1	0.5	0.2	0.1	0.01
BM2	PG 64-22	Gravel	Mullins, TN	Avg.	2938	2669	1965	1417	903	629	457	298	224	105
				S.D.	128	138	101	94	64	41	29	24	20	10
				%CV	4.4	5.2	5.1	6.6	7.0	6.5	6.4	8.1	9.0	9.3
	PG 67-22	Gravel	Jackson, TN	Avg.	3640	3360	2665	2102	1522	1158	884	618	477	230
				S.D.	588	514	383	299	221	171	124	87	66	46
				%CV	16.2	15.3	14.4	14.2	14.5	14.8	14.1	14.1	13.9	20.1
		Gravel	Savannah, TN	Avg.	3847	3604	2890	2276	1625	1221	927	633	474	198
				S.D.	500	476	405	343	266	209	159	109	84	27
				%CV	13.0	13.2	14.0	15.1	16.4	17.1	17.1	17.3	17.7	13.4
		Gravel	Parsons, TN	Avg.	3784	3546	2821	2196	1547	1125	832	549	402	160
				S.D.	202	200	185	170	177	139	102	73	57	24
				%CV	5.3	5.6	6.6	7.7	11.5	12.3	12.3	13.3	14.3	14.7
		Gravel	Troy, TN	Avg.	2170	1982	1494	1110	732	517	378	259	194	91
				S.D.	90	82	64	44	29	28	29	26	26	20
				%CV	4.2	4.1	4.3	4.0	4.0	5.5	7.6	10.1	13.2	22.0
	PG 70-22	Gravel	Stantonville, TN	Avg.	2844	2653	2098	1626	1139	845	647	444	336	142
				S.D.	148	150	127	110	89	75	64	50	43	28
				%CV	5.2	5.6	6.1	6.8	7.8	8.9	9.9	11.2	12.7	19.8

APPENDIX B Dynamic Modulus Master Curves of HMA

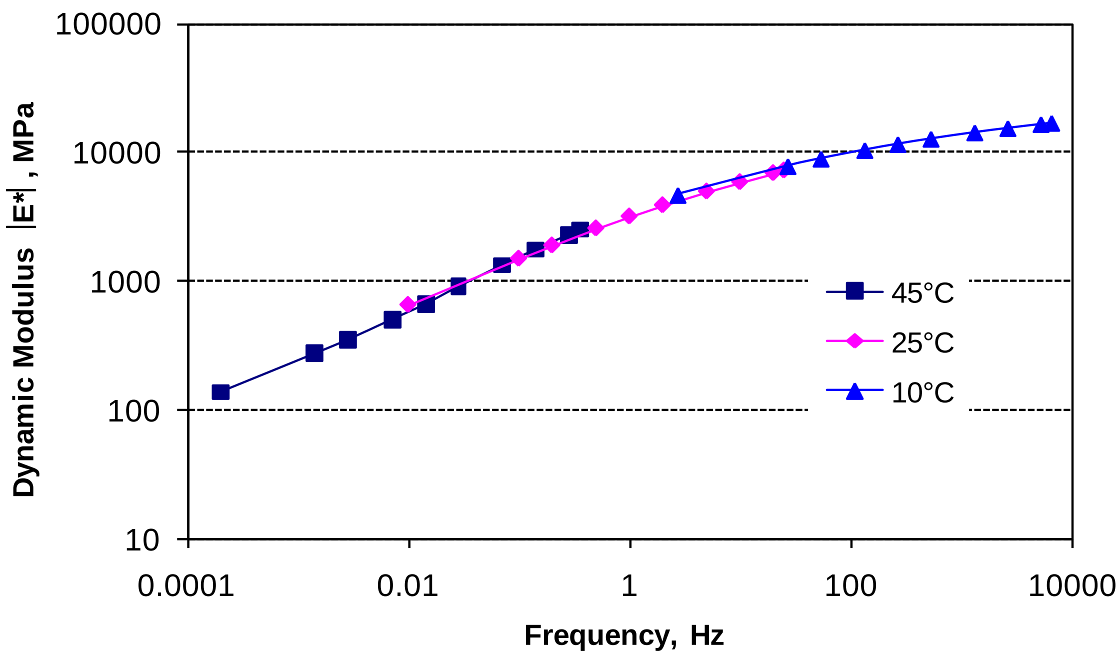
411D-PG 67-22- Jackson



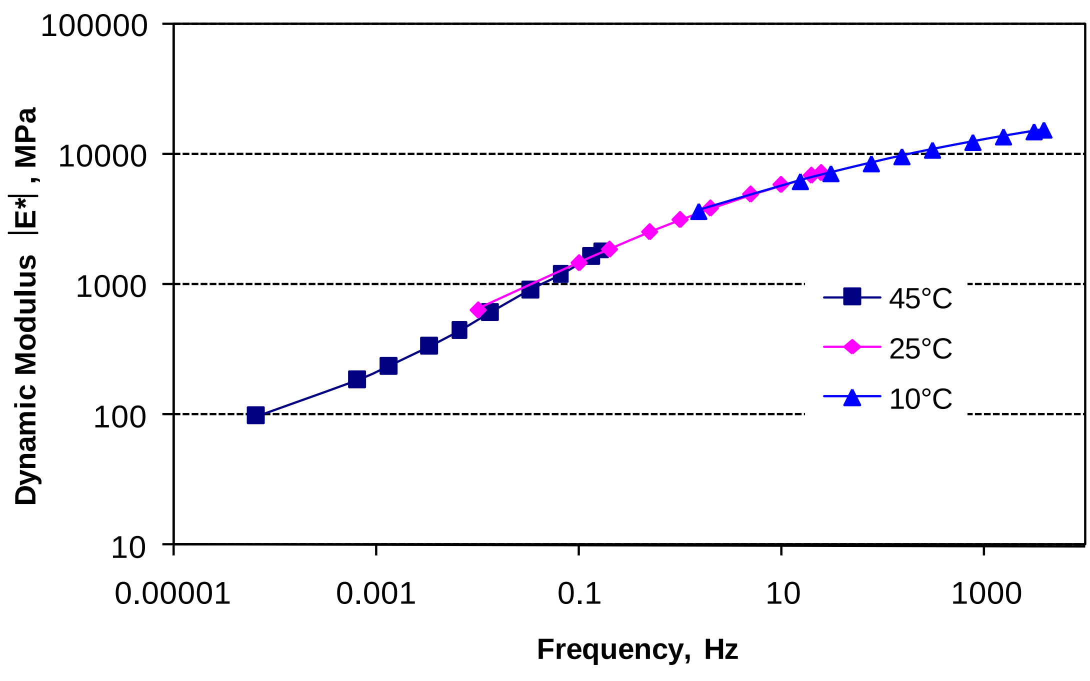
411D-PG 70-22-Dresden



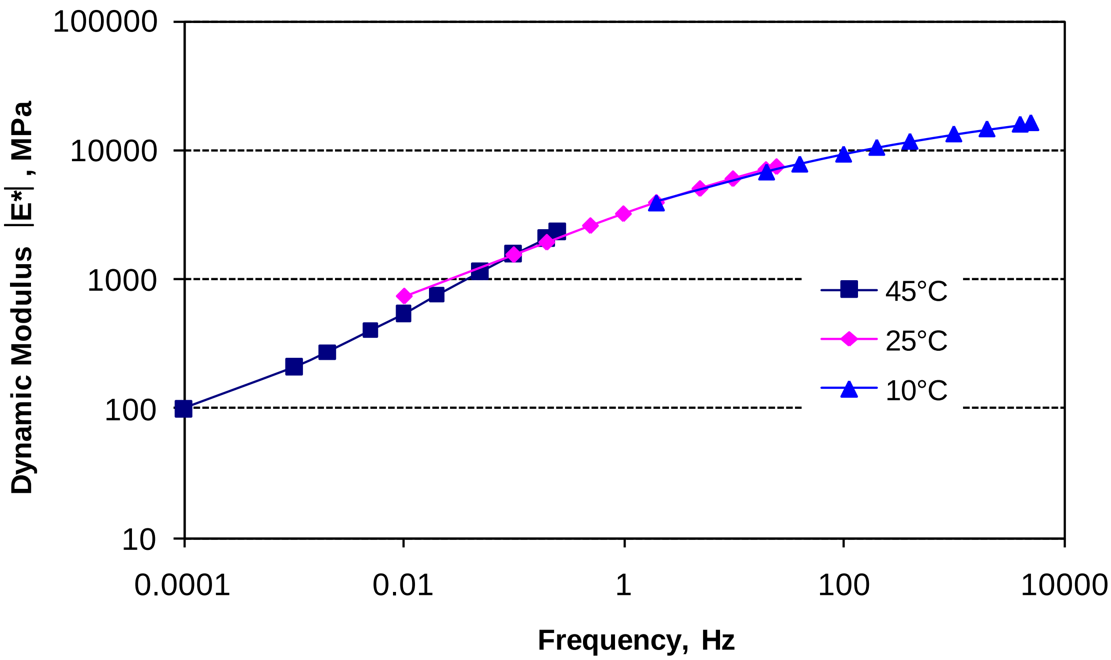
411D-PG 70-22-Jackson



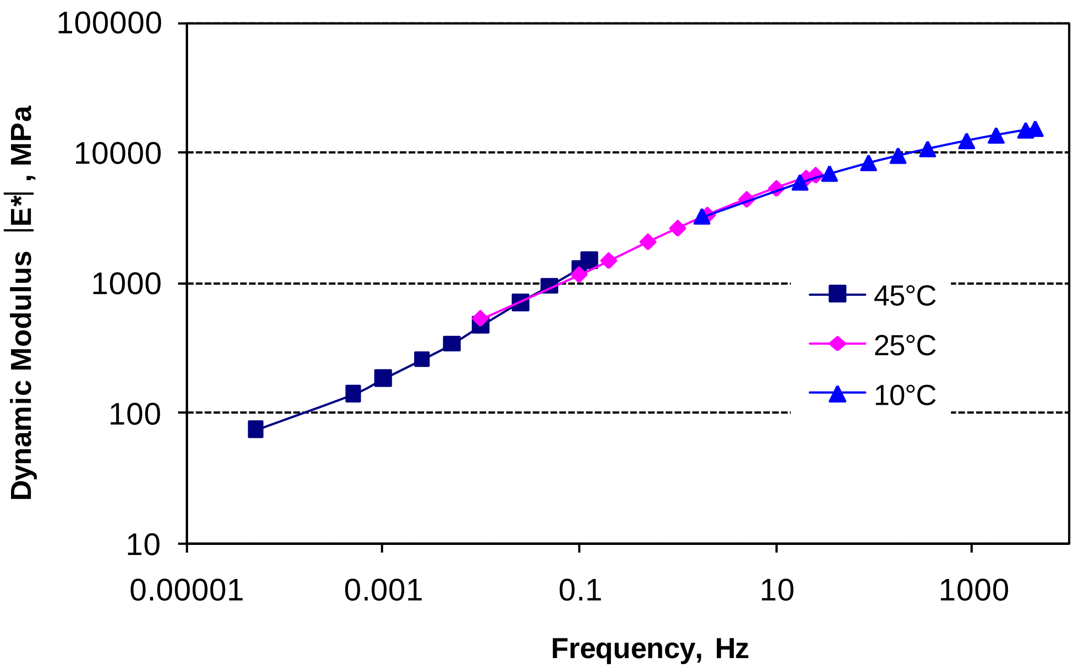
411D-PG 76-22-Memphis

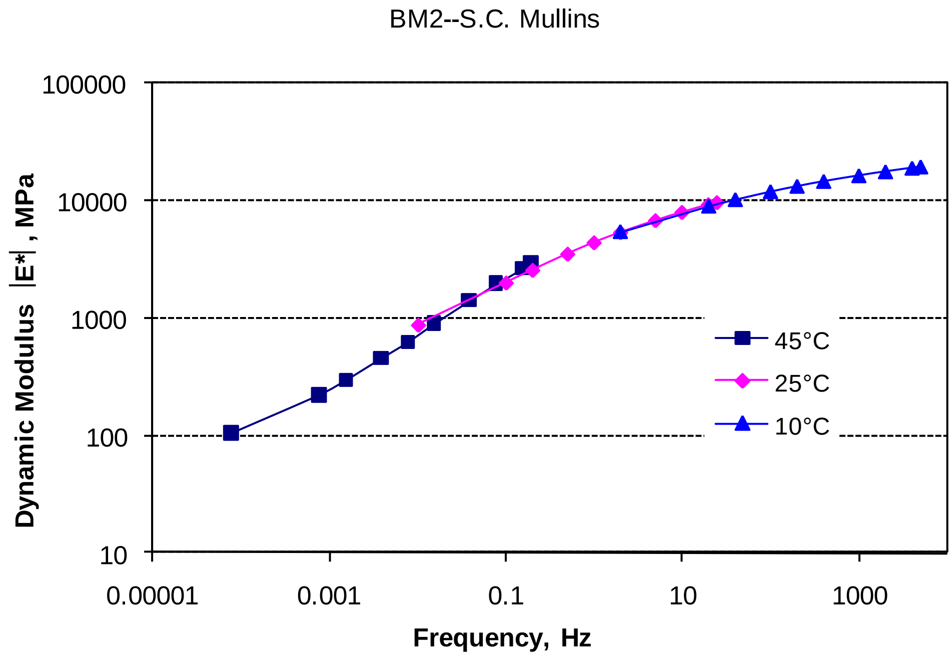


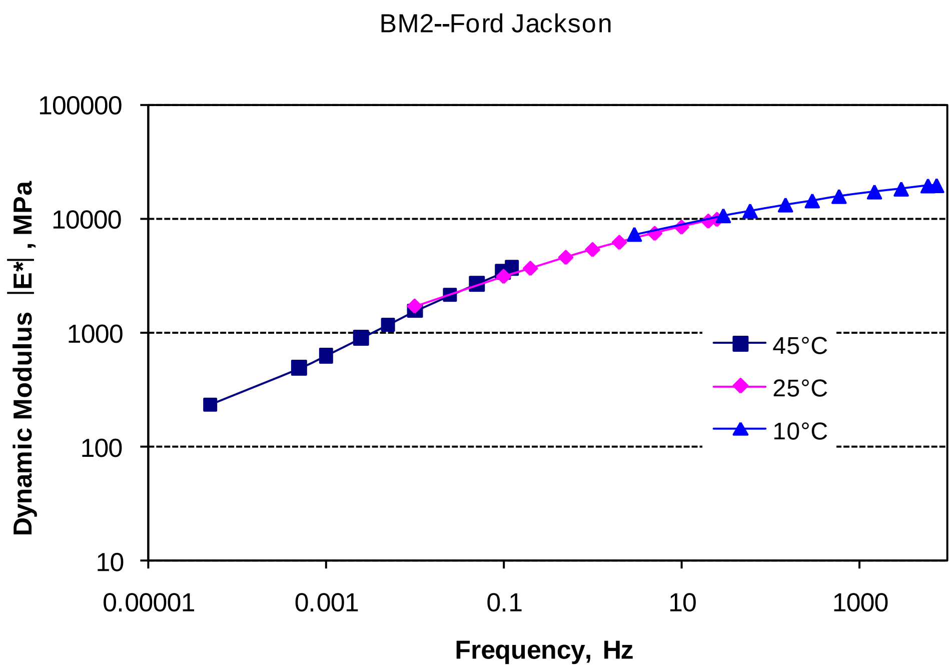
411D-PG 76-22-Mullins

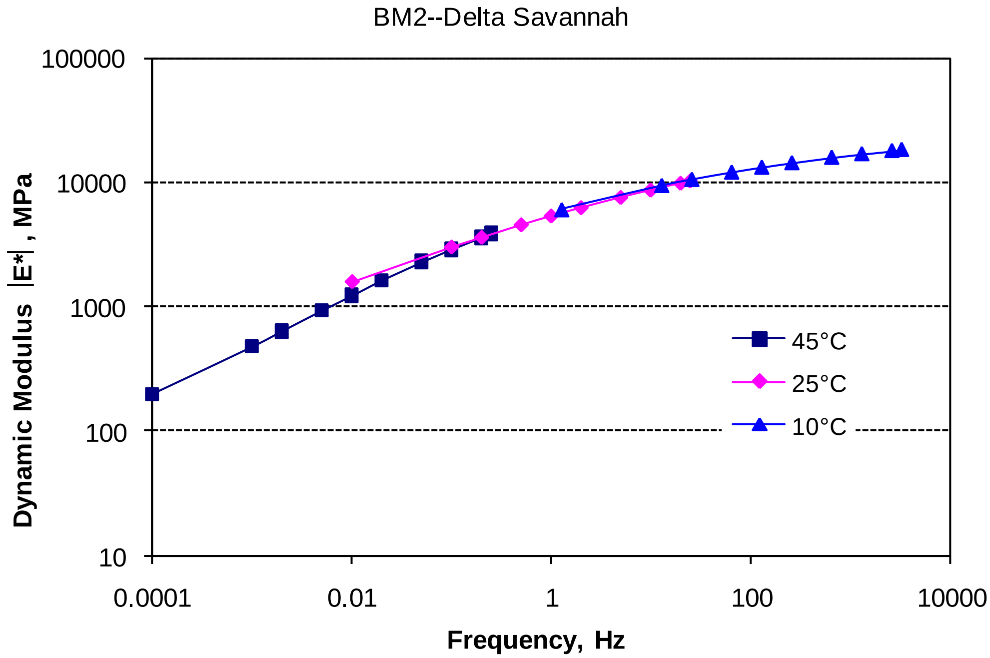


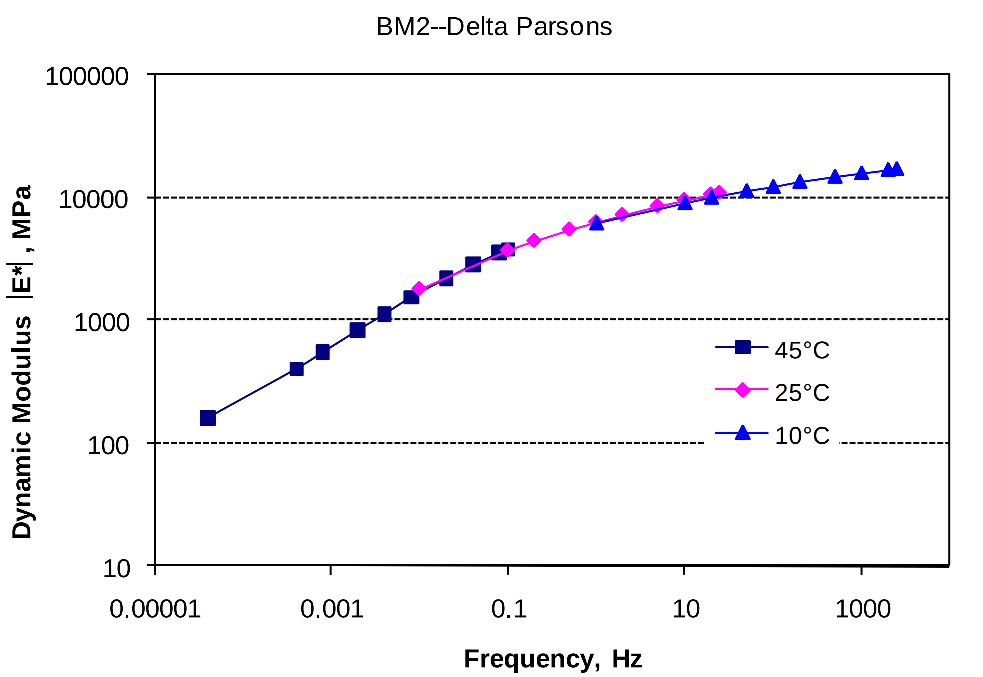
411E-PG 67-22-Troy

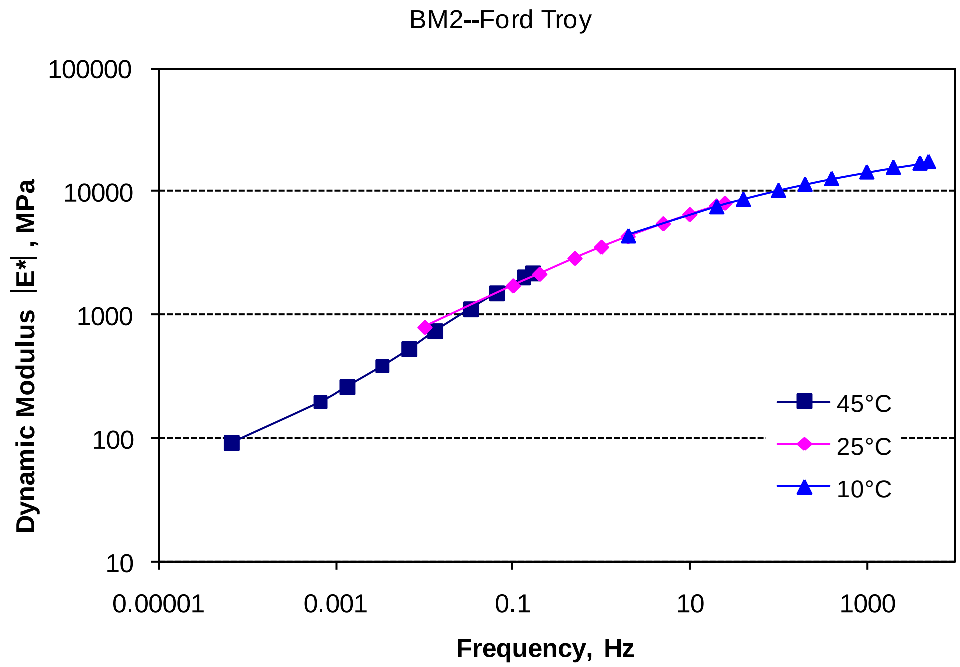


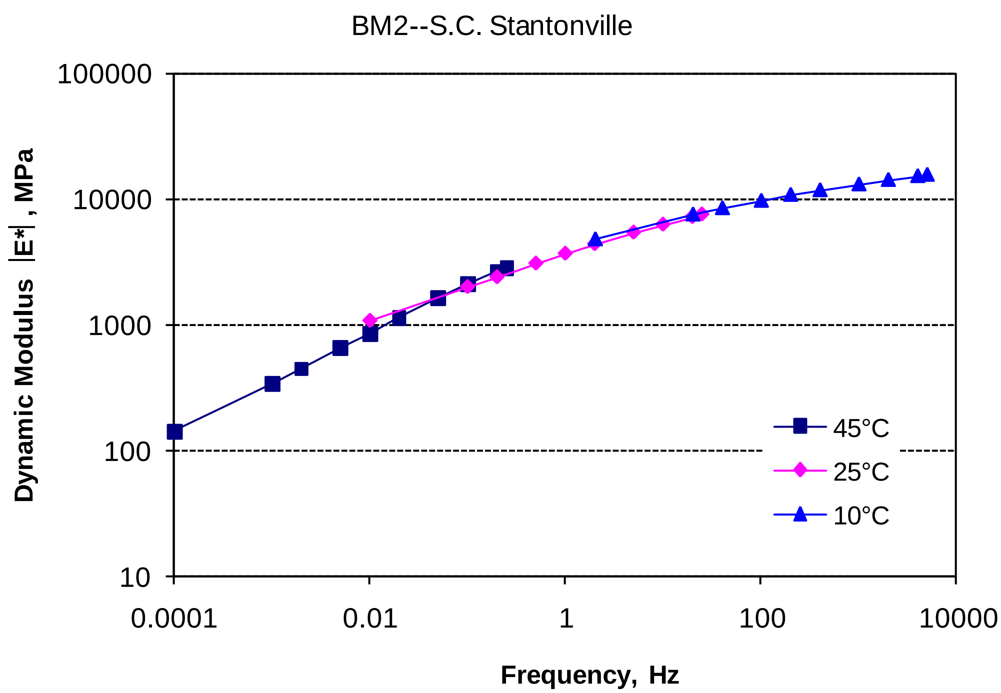

BM2--S.C. Mullins
100000
10000
1000
100
10
Dynamic Modulus |E*|, MPa
0.00001
0.001
0.1
10
1000
Frequency, Hz
45°C
25°C
10°C


BM2--Ford Jackson
100000
10000
1000
100
10
Dynamic Modulus |E*|, MPa
0.00001
0.001
0.1
10
1000
Frequency, Hz
45°C
25°C
10°C


BM2--Delta Savannah
Dynamic Modulus |E*|, MPa
100000
10000
1000
100
10
0.0001
0.01
1
100
10000
Frequency, Hz
45°C
25°C
10°C


BM2--Delta Parsons
Dynamic Modulus |E*|, MPa
100000
10000
1000
100
10
0.00001
0.001
0.1
10
1000
Frequency, Hz
45°C
25°C
10°C


BM2--Ford Troy
100000
10000
1000
100
10
Dynamic Modulus |E*| , MPa
0.00001
0.001
0.1
10
1000
Frequency, Hz
45°C
25°C
10°C


BM2--S.C. Stantonville
100000
10000
1000
100
10
Dynamic Modulus |E*| , MPa
0.0001
0.01
1
100
10000
Frequency, Hz
45°C
25°C
10°C

APPENDIX C Database of Dynamic Moduli for Asphalt Concrete Input Level 1

(psi)

Mixture	Asphalt Cement	Coarse Aggregate	Location	Temp.(F)	Frequency (Hz) 25	10	5	2	0.5	0.1	δ	α	β	γ	a(T)
411D	PG 67-22	Gravel	Jackson, TN	15	316744	311436	306762	299592	286267	266427	0.917	3.456	-1.274	-0.494	222829
				50	245412	223155	206422	184048	150162	112984					134.861
				77	123627	103704	888995	710210	483720	286955					1
				130	125544	85431	63590	43070	24287	13144					0.00051
411D	PG 70-22	Gravel	Dresden, TN	15	342055	332888	325208	314008	294674	268512	0.56	3.886	-1.22	-0.382	554777
				50	228114	204319	187035	165126	133835	100789					187.036
				77	107677	885080	754870	603055	416585	269265					1
				130	122367	88059	68311	48653	29120	16304					0.00037
411D	PG 70-22	Gravel	Jackson, TN	15	376820	369226	362775	353230	336371	312796	0.571	3.896	-1.215	-0.399	212009
				50	246181	223967	207422	184991	151017	113129					226.071
				77	114521	938005	798225	633360	431085	267815					1
				130	194243	139756	108010	76185	44501	23926					0.0015
411D	PG 76-22	Gravel	Memphis, TN	15	363068	352072	342824	329296	305890	274246	0.578	3.899	-1.099	-0.406	304406
				50	226606	200941	182366	159021	125294	915965					140.699
				77	104327	847525	716445	561005	370765	215760					1
				130	116391	81738	62273	43361	25187	13745					0.00072
411D	PG 76-22	Gravel	Mullins, TN	15	351148	343924	337723	328452	311850	288268	0.562	3.871	-1.174	-0.432	952589
				50	234030	209960	192212	168722	134748	985710					168.308
				77	107068	862025	722680	563470	371780	221125					1
				130	174873	122454	92644	63525	35629	18419					0.00211
411E	PG 67-22	Limestone	Troy, TN	15	383932	372004	361955	347238	321737	287241	0.597	3.904	-0.964	-0.413	475867
				50	223271	197896	179263	155164	120988	853180					159.206
				77	961930	761105	626110	473860	294930	164720					1
				130	91294	63297	47878	33141	19236	10612					0.00078
BM2	PG 64-22	Gravel	Mullins, TN	15	344044	340380	337078	331885	321862	306127	1.053	3.342	-1.265	-0.54	587600
				50	280212	254707	235509	210627	172202	129412					151.044
				77	140925	116377	990060	781985	512720	290870					1
				130	203289	136780	100438	66456	35909	18461					0.00165

(psi)

Mixture	Asphalt Cement	Coarse Aggregate	Location	Temp.(F)	Frequency (Hz)						δ	α	β	γ	a(T)
					25	10	5	2	0.5	0.1					
BM2	PG 67-22	Gravel	Jackson, TN	15	357900	354435	351420	346840	338417	325926	1.068	3.347	-1.373	-0.435	841785
				50	281039	261826	246877	225199	189341	151902					344.196
				77	140853	120495	106038	884355	651775	442685					1
				130	318089	235826	186170	134807	81651	45631					0.00148
BM2	PG 67-22	Gravel	Savannah, TN	15	363083	356175	350305	341615	326245	304687	0.048	4.401	-1.65	-0.395	561113
				50	262899	243165	227737	206320	173739	136111					149.496
				77	147233	123902	108199	893490	649310	429055					1
				130	323674	237395	185013	130701	74744	37728					0.00161
BM2	PG 67-22	Gravel	Parsons, TN	15	299076	292122	286157	277249	261342	238879	0.027	4.342	-1.914	-0.437	16073.6
				50	248805	229825	215209	195503	164676	130891					67.626
				77	157760	137315	123018	104284	793585	538240					1
				130	211551	147310	110221	73743	38963	18188					0.00026
BM2	PG 67-22	Gravel	Troy, TN	15	393898	383963	375595	363321	341953	312695	0.015	4.388	-1.471	-0.382	384015
				50	249501	225069	206494	182772	147204	108938					241.987
				77	115782	936555	788945	620455	414265	248385					1
				130	266064	195933	153370	109167	63372	32693					0.00309
BM2	PG 70-22	Gravel	Stantonville, TN	15	329587	323825	318943	311731	299009	281197	0.048	4.401	-1.65	-0.395	561113.
				50	226069	205856	189776	170592	140461	109359					149.496
				77	108880	903495	781550	632345	442540	287825					1
				130	265531	195516	153026	108903	63196	32584					0.00306
D	PG64-22	Gravel	Northeast	15	282055	278910	276017	271374	262158	247210	1.118	3.189	-1.104	-0.602	221153
				50	234088	210119	191632	167837	133748	975850					110.134
				77	109605	890590	751970	586815	379320	203435					1
				130	154135	85115	58725	35670	20010	11890					0.00188
D	PG64-22	Limestone	Northeast	15	297836	289287	281436	268933	244750	208011	1.131	3.224	-0.895	-0.662	5793.00
				50	229999	198606	175450	146334	106560	675265					37.4
				77	112998	877830	701945	510110	279850	125570					1
				130	87580	46545	32770	20590	11455	7250					0.00161

(psi)

Mixture	Asphalt Cement	Coarse Aggregate	Location	Temp.(F)	Frequency (Hz)						δ	α	β	γ	a(T)
					25	10	5	2	0.5	0.1					
D	PG64-22	Granite	Northeast	15	237335	233566	230125	224653	213967	197089	0.567	3.673	-1.138	-0.595	96086.0
				50	185643	164241	148712	128296	992960	683530					84.6204
				77	844335	667580	548245	411220	247805	121655					1
				130	94395	47560	32915	19140	9425	5075					0.00201
D	PG70-22	Gravel	Northeast	15	268178	262457	257325	249313	234115	211105	0.641	3.663	-1.403	-0.564	16208.7
				50	219182	197229	180380	158557	126295	917705					54.641
				77	118856	980200	837810	667145	445585	246210					1
				130	145000	78155	53650	32190	16385	8700					0.00104
D	PG70-22	Limestone	Athens, TN	15	339668	334179	329290	321701	307361	285569	0.672	3.728	-1.02	-0.525	605802
				50	233015	209336	191327	167910	131399	914370					148.79
				77	100151	787930	645830	480820	286230	143115					1
				130	113245	63075	44370	27695	15370	8410					0.00193
D	PG76-22	Gravel	Athens, TN	15	241183	234723	229057	220419	204602	181761	0.586	3.688	-1.211	-0.517	28412.9
				50	175290	158122	144898	127252	101021	651630					64.74
				77	882325	717895	604215	471250	305370	165155					1
				130	103095	57855	41180	25665	14065	7685					0.00105
D	PG76-22	Limestone	Athens, TN	15	430264	415800	403498	385324	353523	310190	0.749	3.801	-1.054	-0.451	71787.4
				50	271048	241773	219979	192487	151409	107010					90.216
				77	129499	104458	871740	668595	421660	229680					1
				130	133835	78590	56695	37120	22040	14210					0.00075
BM-2	PG64-22	Limestone	Athens, TN	15	430264	415800	403498	385324	353523	310190	0.749	3.801	-1.054	-0.451	71787.4
				50	317593	293552	274920	248182	207408	157673					90.216
				77	150684	125106	106734	838100	541720	287100					1
				130	167765	99180	68005	40455	21170	11020					0.00075
BM-2	PG70-22	Limestone	Athens, TN	15	515166	507929	501520	491619	473017	444850	0.778	3.8	-1.253	-0.501	760458
				50	384119	348870	324321	290957	236712	174493					178.03
				77	176813	147175	124714	980055	632490	334805					1
				130	197200	116580	81635	51040	27260	15805					0.00095

(psi)

Mixture	Asphalt Cement	Coarse Aggregate	Location	Temp.(F)	Frequency (Hz)						δ	α	β	γ	a(T)
					25	10	5	2	0.5	0.1					
BM-2	PG76-22	Limestone	Athens, TN	15	419025	412004	405827	396357	378773	352607	0.737	3.758	-1.156	-0.491	622059.
				50	294074	266046	246297	218515	177001	128963					158.899
				77	135183	110127	923795	713835	452110	239250					1
				130	169795	99760	70180	43355	24360	14790					0.00133
BM-2	PG82-22	Gravel	Athens, TN	15	315294	303040	292601	277180	250285	214040	0.705	3.718	-1.039	-0.474	21054.2
				50	198780	175131	157905	135430	103501	713255					50.49
				77	102471	821860	682370	505905	322190	165735					1
				130	177190	99470	72210	47270	26970	16240					0.0029
A	PG70-22(East)	Limestone	Knoxville	15	351904	342943	334981	322686	299763	265943	0.71	3.721	-1.314	-0.55	14675.1
				50	278530	247094	223851	194314	151090	106053					52.31
				77	143477	118276	100122	776040	501410	279270					1
				130	161240	89755	62640	37845	19140	10000					0.00109
A	PG70-22(Mid.)	Limestone	Davison	15	647647	628417	612474	589476	550398	498595	0.889	3.848	-1.002	-0.362	179471
				50	342562	316897	296916	269207	224590	169635					240.653
				77	167504	139185	117812	919880	582320	303050					1
				130	170085	164575	122090	84390	54955	38425					0.00072
A	PG76-22	Limestone	East	15	349266	345200	341388	335152	322457	301270	1.395	3.004	-1.24	-0.66	35397.9
				50	299207	273615	254663	228070	185527	135575					64.61
				77	165836	138243	132834	934380	608130	312185					1
				130	223300	126005	87435	55245	30305	17690					0.00163
A-S	PG70-22(East)	Limestone	Sevierville	15	290282	290249	290207	290114	289815	288902	2.088	2.214	-0.757	-1.474	157596
				50	332818	311068	292465	266002	222256	167359					72.124
				77	168968	142491	122800	982810	652065	38280					1
				130	217065	129050	92510	59160	31175	17835					0.01484
A-S	PG70-22(Mid.)	Limestone	Nashville	15	395426	392796	390329	386285	377992	363885	1.196	3.249	-1.3	-0.642	380939
				50	342287	314534	294930	268612	225794	175290					133.441
				77	178161	152873	129485	102616	638435	322625					1
				130	219385	125425	86275	52780	26825	14935					0.00157

APPENDIX D Database of Resilient Moduli for Base Materials Input Level 1

Base Materials	k_1	k_2	k_3	R^2
Ford Construction-Troy	936.848	0.653	-0.254	0.99
JR Hayes-Paris	1317.709	0.487	-0.047	0.997
Memphis Stone-North Plant	1245.798	0.543	0.016	0.99
Standard Construction-FR RD Collierville	1058.419	0.560	0.056	0.99
Standard Construction-Stantonville	1064.024	0.570	0.022	0.99
Limestone-Cement	2364.025	0.734	0.021	0.84
Gravel-Cement	1855.515	0.702	0.409	0.77
Limestone-Fly Ash-Lime	1906.295	0.576	0.579	0.81

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

where:

M_R = resilient modulus

θ = bulk stress = $\sigma_1 + \sigma_2 + \sigma_3$

σ_1= *major principal stress*

σ_2 = intermediate principal stress = $\sigma 3$ for MR test on cylindrical specimens

σ_3 = minor principal stress/confining pressure

τ_{oct} = octahedral shear stress = $\frac{1}{3}\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2}$

Pa = atmospheric pressure (used to normalize the equation)

k_1, k_2, k_3 = regression constants determined from the laboratory tests

APPENDIX E Selected Routes around TN for Comparison of Predicted Lives between AASHTO 1993 Design Guide and MEPDG 2002

Route	County	CBR	Surface" D/E" (0.4)	PCC	BM2 (0.4)	C (0.4)	A-Mix (0.4)	A-S Mix (0.4)	TPB	CTB (0.23)	LFAB (0.28)	30301 (0.14)
I-24	Rutherford	13.3	1.25		3.5	1.5	7					11
I-24	Coffee	6	1		3.5	1	7					8
I-26	Sullivan	3	1		3.5	1	3.5	3.5				8
I-40	Cocke	18	1		3.5	1	7					8
I-40	Cumberland	18	1		3.5	1	7					8
I-40	Davidson	4.8		10								9
I-40	Shelby	4.8		10						6		
I-65	Williamson	4	1.25		2		7	3				10
I-75	Anderson	7	1.25		3.5	1.5	7					8
I-74	Hamilton	4		13					4			4
I-75	Hamilton	4	1.25		2		8	3.5				10
I-81	Greene		1		3.5	1.25	7					8
I-155	Dyer	10	1.25		2			3.5				
I-240	Shelby	5		10								8
I-440	Davidson	13.3		10								5
I-540	Knoxville	8.2		10								5
SR-15	Lawrence/Giles	3	1.25		2		3	3				16
SR-385	Shelby/Fayette	5	1.25		2		6	4				11
SR-30	Mcminn	7	1.25		2		3	3				8
SR-331	Knoxville	7	1.25		2		3					8