

ABSTRACT

Title of Document: CATALOG OF MATERIAL PROPERTIES
FOR MECHANISTIC-EMPIRICAL
PAVEMENT DESIGN

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Engineering

The new pavement design methodology is based on mechanistic-empirical principles that are expected to be used in parallel with and eventually replace the current empirical pavement design procedures. The new mechanistic-empirical pavement design guide (MEPDG) requires greater quantities and quality of input data. Material characterization for the mechanistic-empirical approach, the focus of this thesis, is significantly more fundamental and extensive than in the current empirically-based AASHTO Design Guide. The objective of the thesis is to develop an organized database of material properties for the most common paving materials used in Maryland. A comprehensive material property database in Microsoft Access 2007 has been developed. The database is initially populated with all information received from SHA. It provides complete data management tools for adding and managing future data as well as data display screens for MEPDG. Recommendations for future material testing for Maryland are also provided.

CATALOG OF MATERIAL PROPERTIES FOR MECHANISTIC-EMPIRICAL
PAVEMENT DESIGN

By

Rui Li

Thesis submitted to the Faculty of the Graduate School of the
University of Maryland, College Park, in partial fulfillment
of the requirements for the degree of
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2011

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Dedication

To my wife, Junjian, and my coming baby.

Acknowledgements

I would like to acknowledge Dr. Charles Schwartz for his generosity in providing me with a research assistantship and bringing me to the University of Maryland. Also, I thank him for his valuable advice and warm encouragement throughout this first phase of my graduate studies and for keeping me motivated for the challenges ahead.

I would also like to express my thanks to Dr. Goulias for his valuable advice and encouragement.

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CHAPTER 1: INTRODUCTION

The new pavement design methodology developed in NCHRP Project 1-37A, refined in NCHRP Project 1-40D, and subsequently adopted by AASHTO (AASHTO, 2008) is based on mechanistic-empirical principles that are expected to be used in parallel with and eventually replace the current empirical pavement design procedures derived from the AASHO Road Test conducted in the late 1950's (HRB, 1962). The new mechanistic-empirical pavement design guide (MEPDG) requires greater quantities and quality of input data in four major categories: traffic; material characterization; environmental factors; and pavement performance (for local calibration/validation). Material characterization for the mechanistic-empirical approach, the focus of this thesis, is significantly more fundamental and extensive than in the current empirically-based AASHTO Design Guide (AASHTO, 1993).

The implementation plan developed by the University of Maryland (UMD) for the Maryland State Highway Administration (SHA) recommended a range of research projects to be completed in preparation for the MEPDG (Schwartz, 2007). One of the higher priority efforts identified in the plan was to catalog and compile existing material properties. This thesis addresses this need.

A hierarchical input data scheme has been implemented in the MEPDG to permit varying levels of sophistication for specifying material properties, ranging from laboratory measured values (Level 1) to empirical correlations (Level 2) to default values (Level 3). It is expected that most states, including Maryland, will begin implementation of the new design procedure using Level 3 default inputs or Level 2 correlations that are relevant to their local materials and conditions and will, over time, supplement these with

typical Level 1 measured data for their most common materials. To accomplish this, databases or libraries of typical material property inputs must be developed for the following categories:

Binder properties (e.g., binder dynamic modulus G^* and phase angle δ or binder viscosities η)

Hot mix asphalt (HMA) mechanical properties (e.g., dynamic modulus E^* master curves—either measured directly or predicted empirically)

Portland cement concrete (PCC) mechanical properties (e.g., elastic modulus E_c , modulus of rupture MOR)

Unbound mechanical properties (e.g., resilient modulus M_r or k_1 - k_3 values)

Thermohydraulic properties (e.g., saturated hydraulic conductivity k_{sat})

The objective of the thesis is to develop this type of organized database of material properties for the most common paving materials used in Maryland. Note that this study provides an essential prerequisite for an eventual full local calibration/validation of the MEPDG for Maryland conditions.

The work plan for accomplishing the research objective was organized into seven tasks:

Task 1: Database Design

Task 2: Binder Properties

Task 3: HMA Mechanical and Physical Properties

Task 4: PCC Mechanical and Physical Properties

Task 5: Unbound Mechanical and Physical Properties

Task 6: Thermohydraulic Properties

Task 7: Workshop and Final Report

The organization of this thesis closely mirrors the work plan. The principal difference is that the findings on thermohydraulic properties from Task 6 have been merged with the coverage of the mechanical and physical properties for each material type. The organization of the chapters of this thesis is thus:

- 1: Introduction
- 2: Binder Data
- 3: HMA Data
- 4: PCC Data
- 5: Unbound Material Data
- 6: Material Properties Database
- 7: Summary of Recommendations
- 8: References

Each of the specific material Chapters 2 through 5 generally follows the same consistent organization:

- MEPDG Input Requirements
 - New Construction/Reconstruction/Overlays
 - Rehabilitation (Existing Layers)
- Data Available from Maryland SHA
- Analyses of MEDPG Inputs
 - Level 1 vs. Level 2 vs. Level 3
 - Sensitivity Analyses
- Summary
 - Testing Recommendations
 - Recommended MEPDG Inputs

The final Chapter 7 compiles in one location all of the detailed testing recommendations from each of the specific material Chapters 2 through 5.

A comprehensive material property database developed in Microsoft Access 2007 accompanies this thesis. This database is initially populated with all information received from SHA. It provides complete data management tools for adding future data as well as data display screens for MEPDG inputs that mirror the input screens in the MEPDG Version 1.100 software. Documentation of this database is provided in Chapter 6.

In addition to this thesis, results from this study have appeared/will appear in part in published articles by Schwartz (2009), Schwartz and Li (2010), and Schwartz *et al.* (2011). Complete citations for these articles can be found in the reference list at the end of this thesis.

CHAPTER 2: BINDER DATA

MEPDG Input Requirements

The binder properties required at each of the input levels in the MEPDG are as follows:

- Level 1: Shear stiffness G^* and phase angle δ at multiple temperatures at a frequency of $\omega = 10$ radians/sec (AASHTO T315)
- Level 2: Same as Level 1
- Level 3: Default A-VTS viscosity temperature susceptibility parameters based on Superpave Performance Grade (PG)

The required binder inputs are the same for new construction, rehabilitation, and reconstruction. Note that only Superpave binder properties are considered here. The conventional softening point, Brookfield viscosity, kinematic viscosity, and penetration properties used in the past have not been included in this study since SHA stopped measuring these once it had moved to the Superpave mix design system.

Binder Data Received and Preliminary Analysis

A large set of binder properties was provided by SHA for initial population of the material properties database. The scope of the provided data is described in Table 1. All of the SHA testing data was collected for Superpave PG acceptance purposes. The data received for SHA represented test results from early 2002 through mid-September 2008.

Table 1. Number of test records received from SHA.

PG Grade	Number of Test Records
58-28	15
64-22	3685
64-28	150
70-22	864
76-22	1540

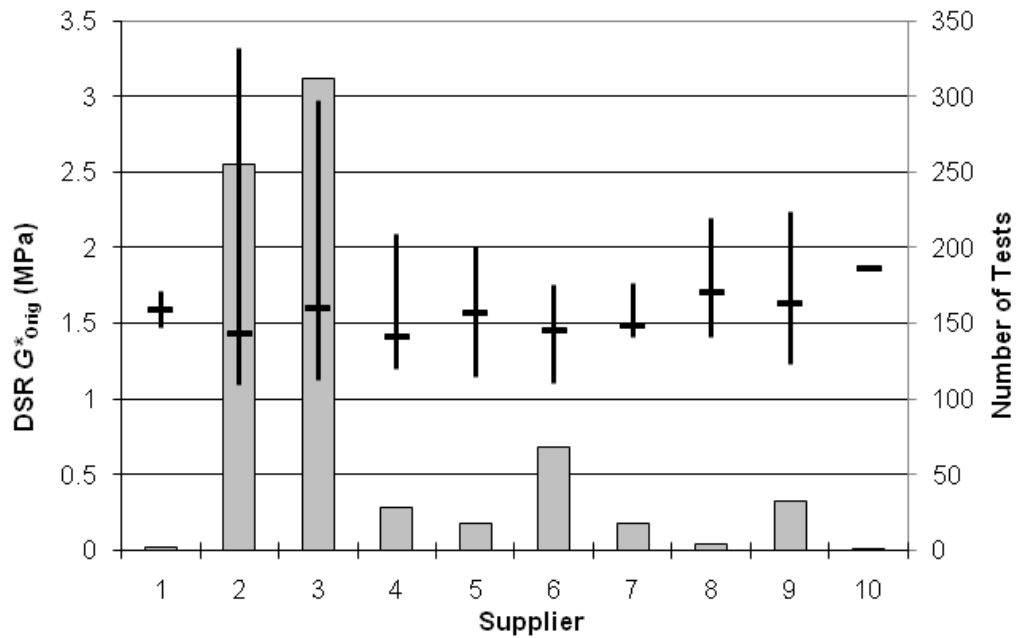
Considerable effort was devoted to identify incorrect or inconsistent information in the data provided by SHA. Incorrect or inconsistent data were corrected when possible and eliminated when not.

The variability of the acceptance test data was also carefully evaluated. Values of binder stiffness (G^*) and phase angle (δ) at original, RTFO and PAV conditions, BBR stiffness and BBR m value were reported in the test data received from SHA. Figure 1, Figure 2 and Figure 3 summarize the variability of property values by supplier for the PG 64-22, PG 70-22, and PG 76-22 performance grades, respectively (the PGs most commonly used in Maryland). Definitions of the code numbers used in these figures are listed in Table 2. The black lines in these figures indicate the minimum, average, and maximum property values (left axis), the gray bars summarize the number of test data in each category (right axis). Note that the thick dashed lines in Chart (c), Chart (f) and Chart (i) in each figure indicate the specification limits for $G^*/\sin\delta$ at each aging condition. There are no specification values for BBR stiffness or BBR m value. From these figures it can be seen that nearly all data fall within the specification limits for the original and RTFO conditions. Some data are significantly above the maximum limit for the PAV aged condition (i.e. Supplier 2 and 6 in Figure 2.1(i)). The reasons and consequences of these violations of the acceptance specification conditions are unknown. However, since the stiffness properties at PAV condition represent binder performance at

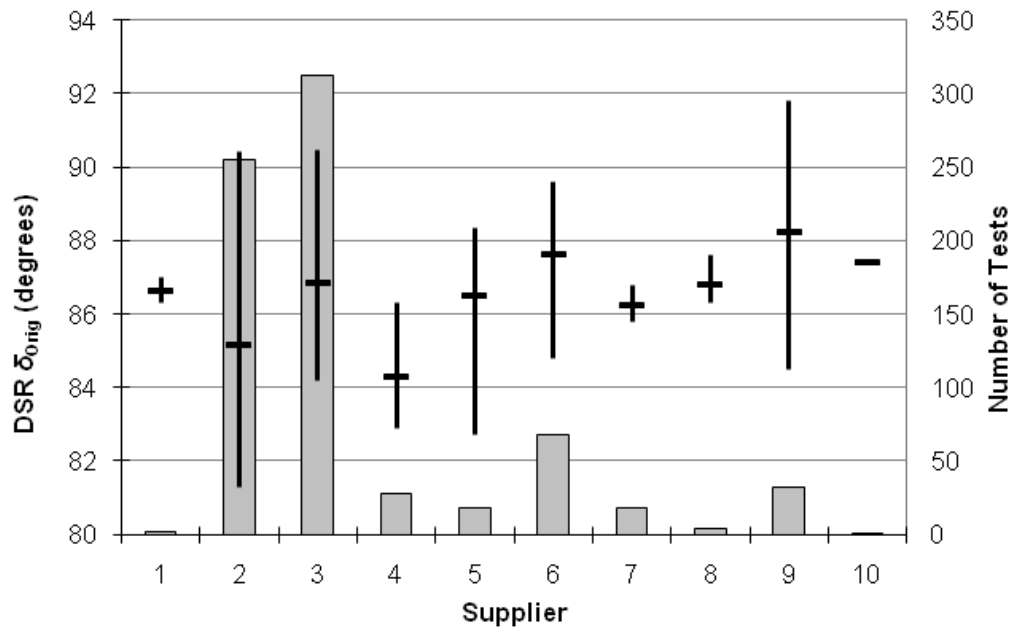
low temperature, this should not have much practical significance in Maryland where low temperature cracking is not a problem. Furthermore, since binder data at the PAV condition is not an input in MEPDG, it will not affect the MEPDG predictions.

Table 2. Legend for supplier code numbers in Figure 1 to Figure 3.

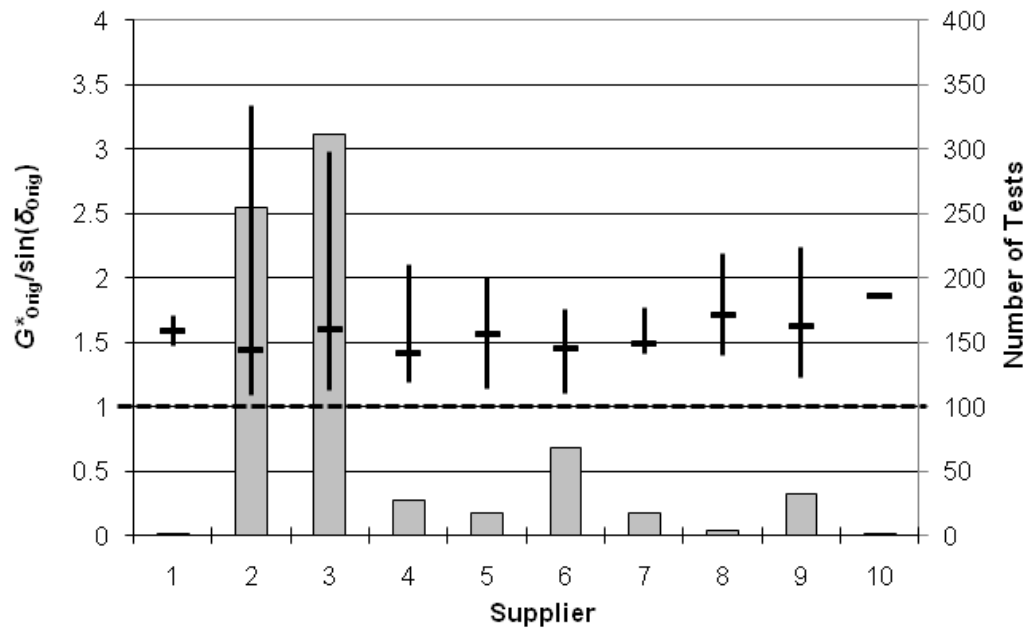
Code	Figure 1	Figure 2	Figure 3
1	Associated Asphalt	Chevron	Associated Asphalt
2	Chevron	Citgo	Chevron
3	Citgo	Marathon Ashland	Citgo
4	ESM ASPHALT, LLC	NuStar Asphalt Refining, LLC	Conoco Phillips
5	Koch	Valero	ESM ASPHALT, LLC
6	Marathon Ashland		Koch
7	NuStar Asphalt Refining, LLC		Marathon Ashland
9	United		NuStar Asphalt Refining, LLC
10	Valero		SEM Materials



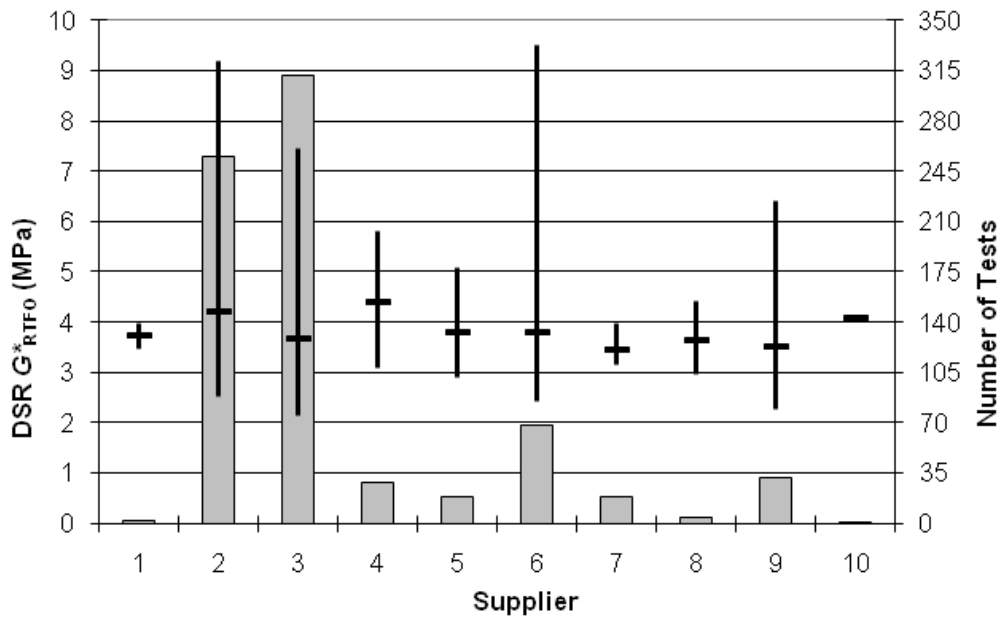
(a)



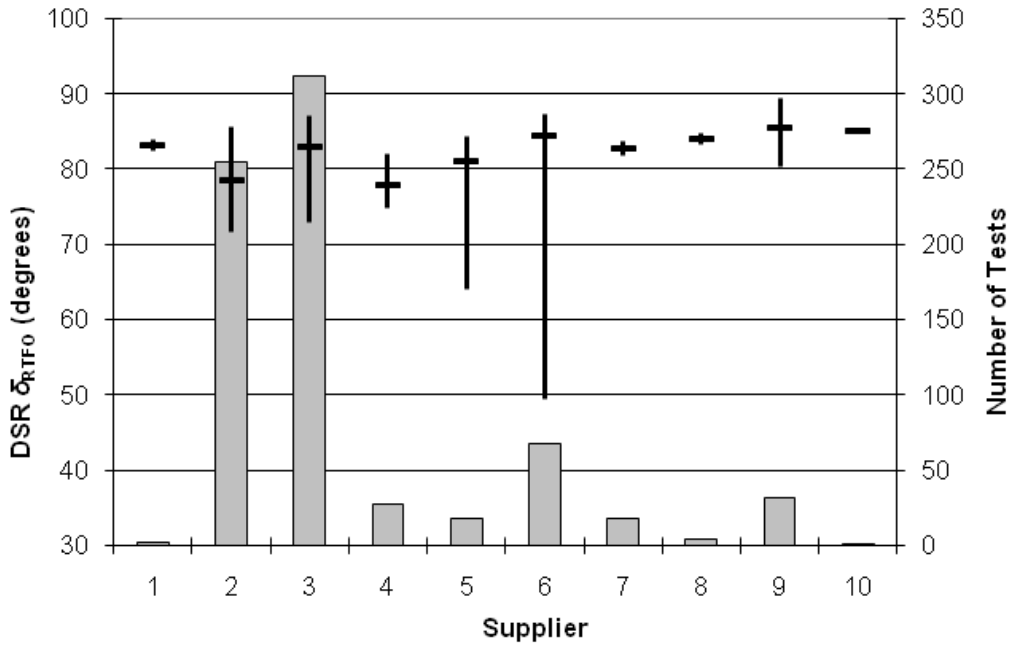
(b)



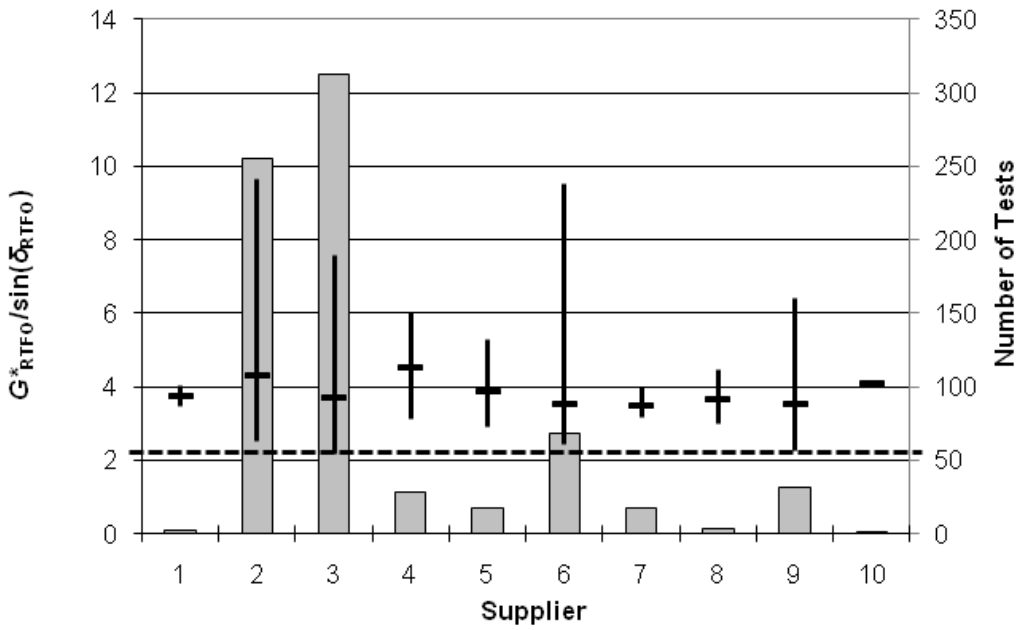
(c)



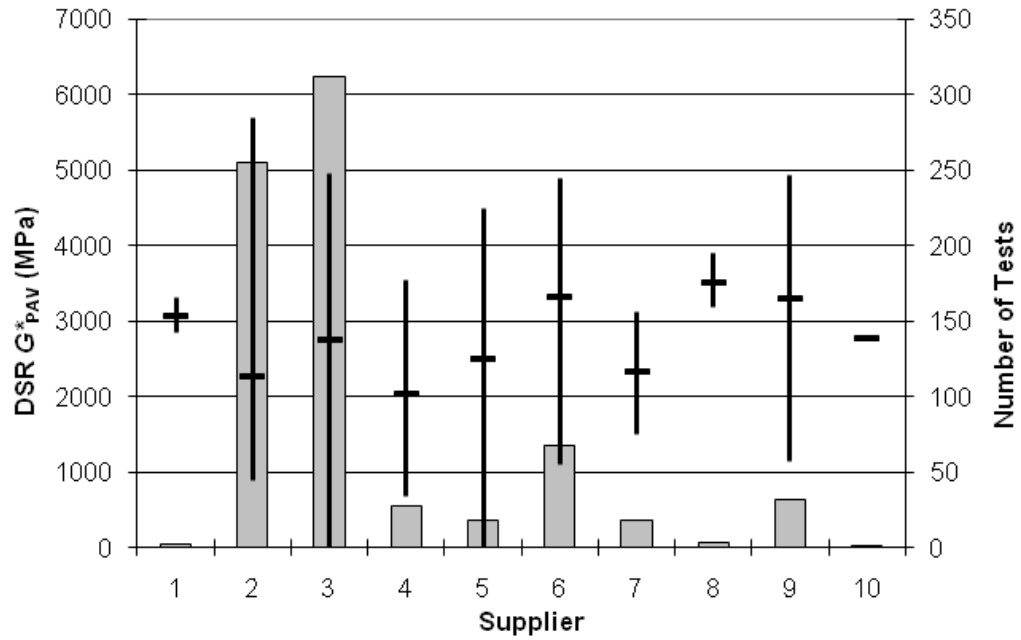
(d)



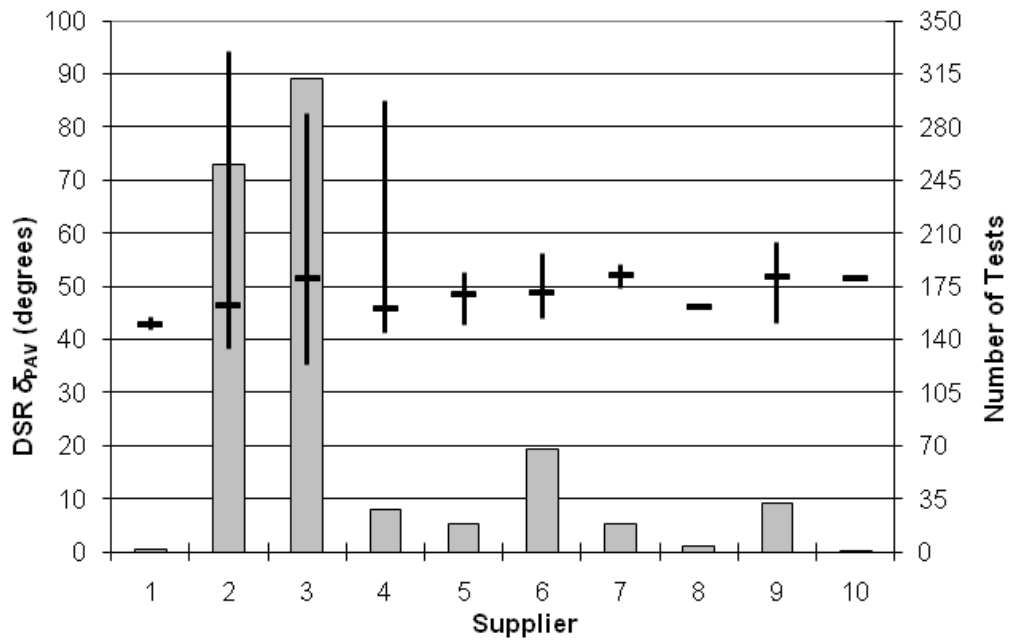
(e)



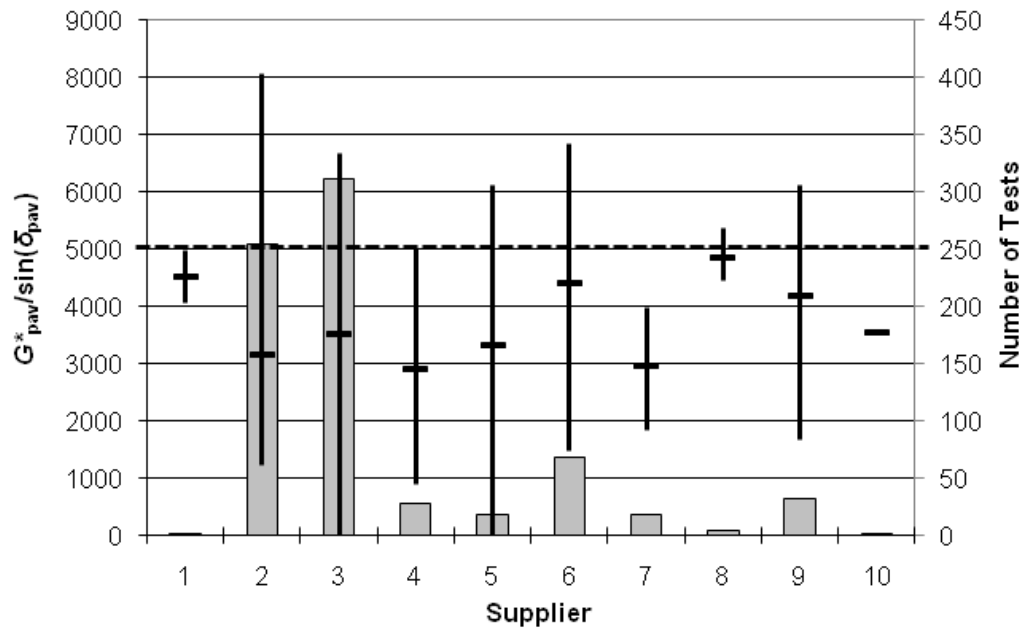
(f)



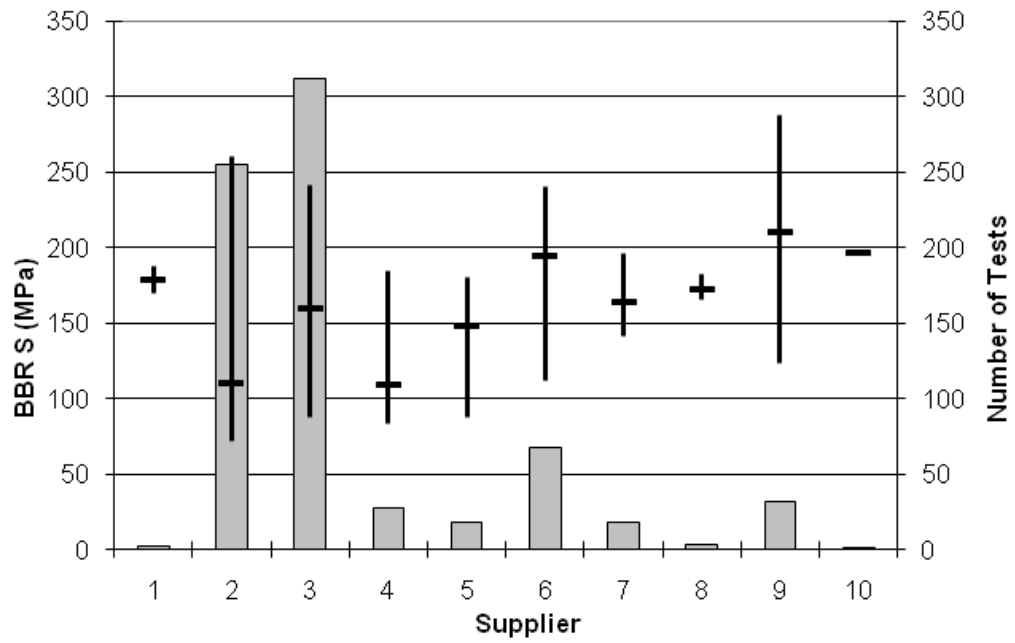
(g)



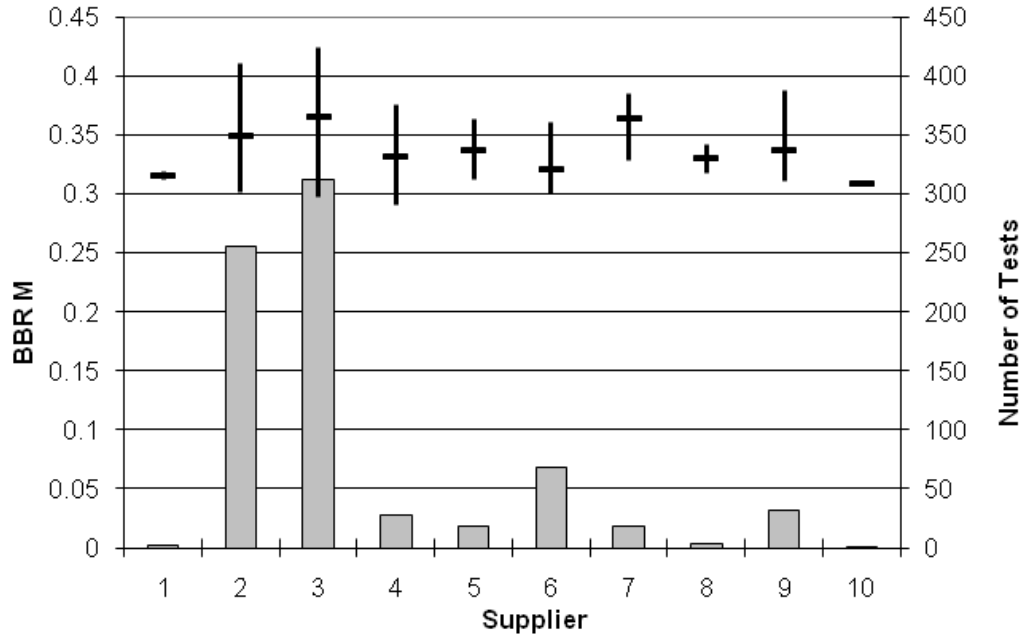
(h)



(i)

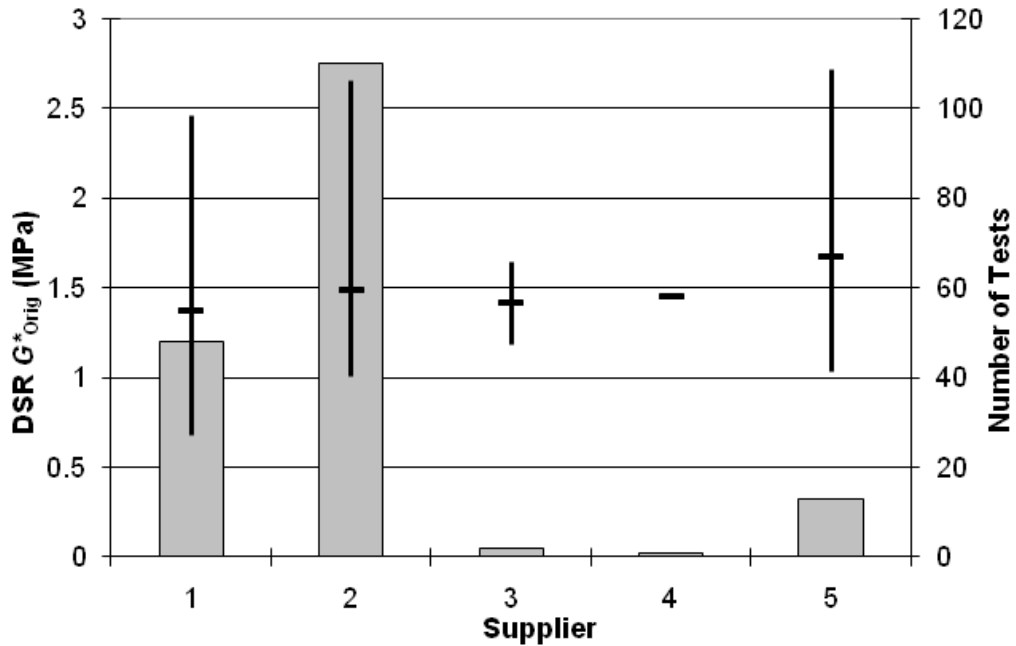


(j)

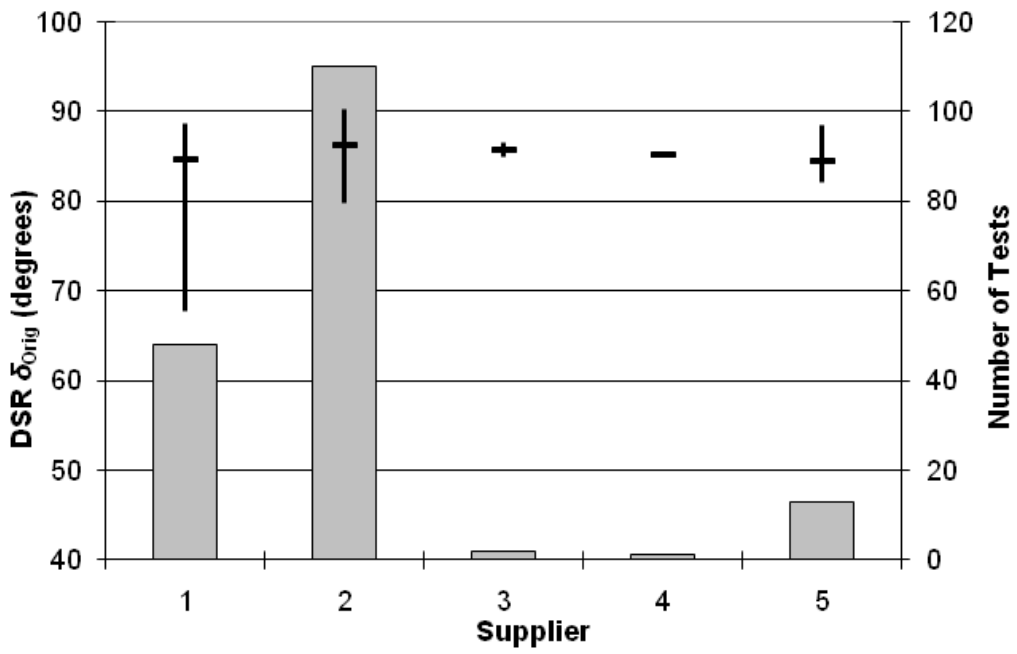


(k)

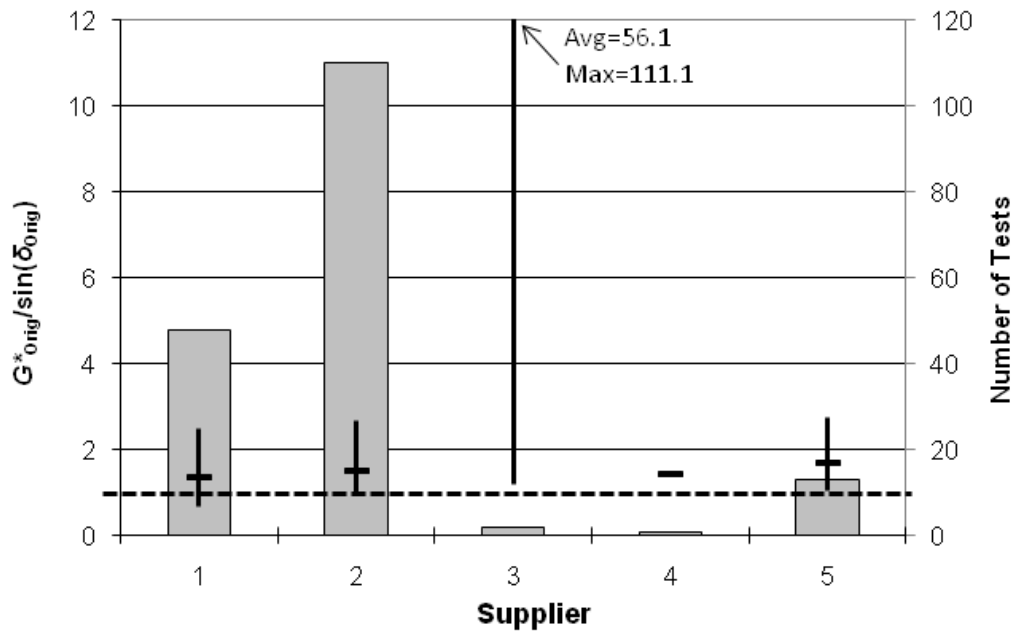
Figure 1. High/low/average/volume plots for PG 64-22 binder acceptance properties: (a) binder stiffness G^*_{Orig} , (b) phase angle δ_{Orig} , (c) ratio of $G^*_{Orig}/\sin(\delta_{Orig})$ at original conditions; (d) binder stiffness G^*_{RTFO} , (e) phase angle δ_{RTFO} , (f) ratio of $G^*_{RTFO}/\sin(\delta_{RTFO})$ at RTFO aged conditions; (g) binder stiffness G^*_{PAV} , (h) phase angle δ_{PAV} , (i) ratio of $G^*_{PAV}/\sin(\delta_{PAV})$ for PAV aged conditions, (j) BBR Stiffness and (k) BBR m value. Test temperature is 64 °C for original and RTFO aged conditions, 25 °C for PAV aged condition, and -12 °C for BBR.



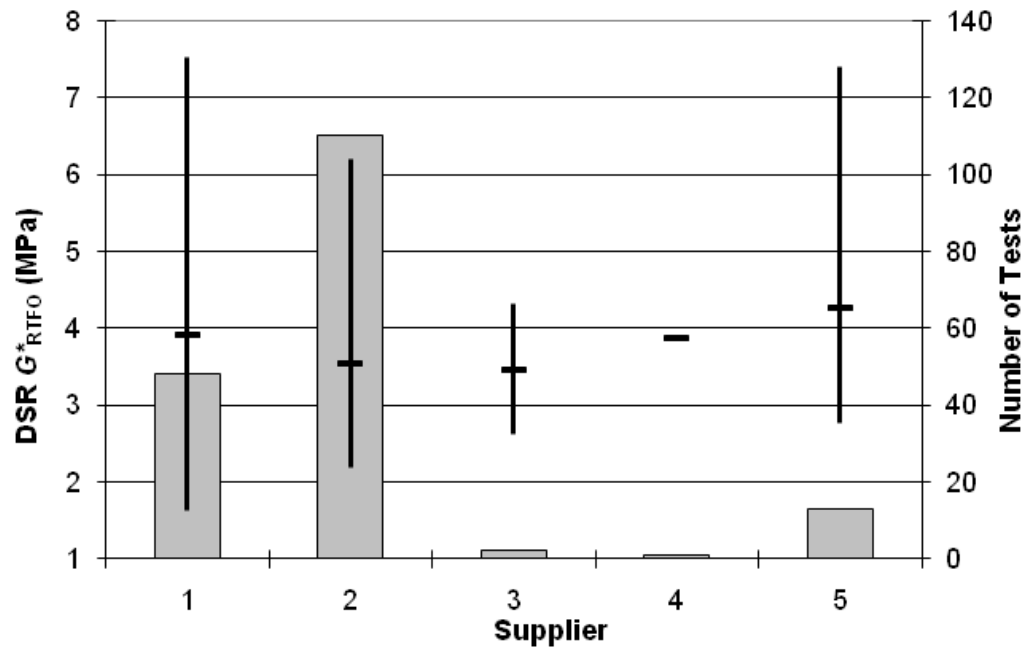
(a)



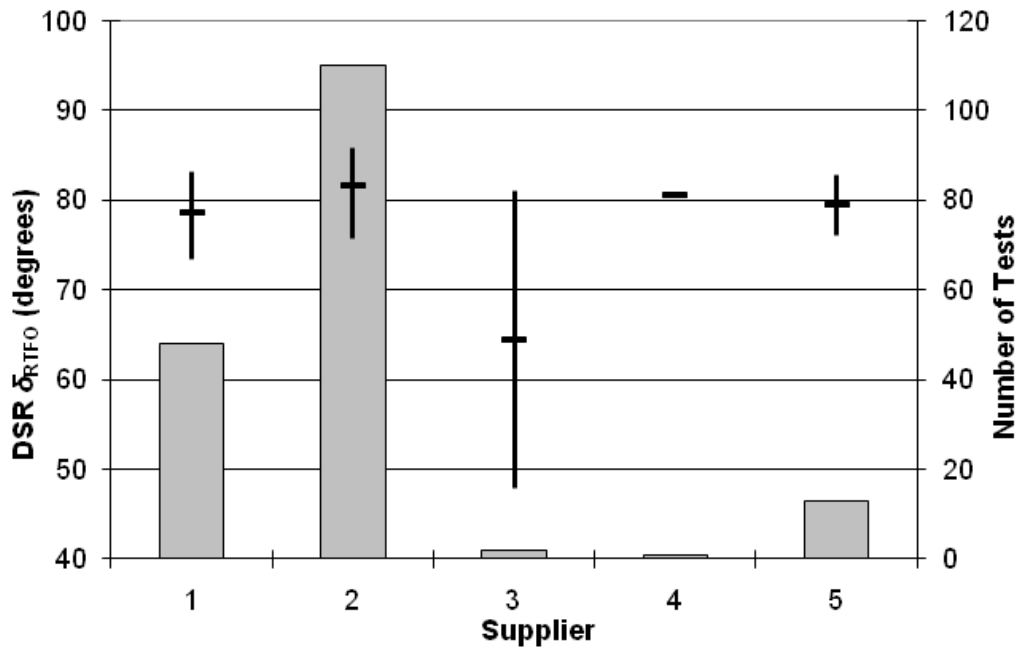
(b)



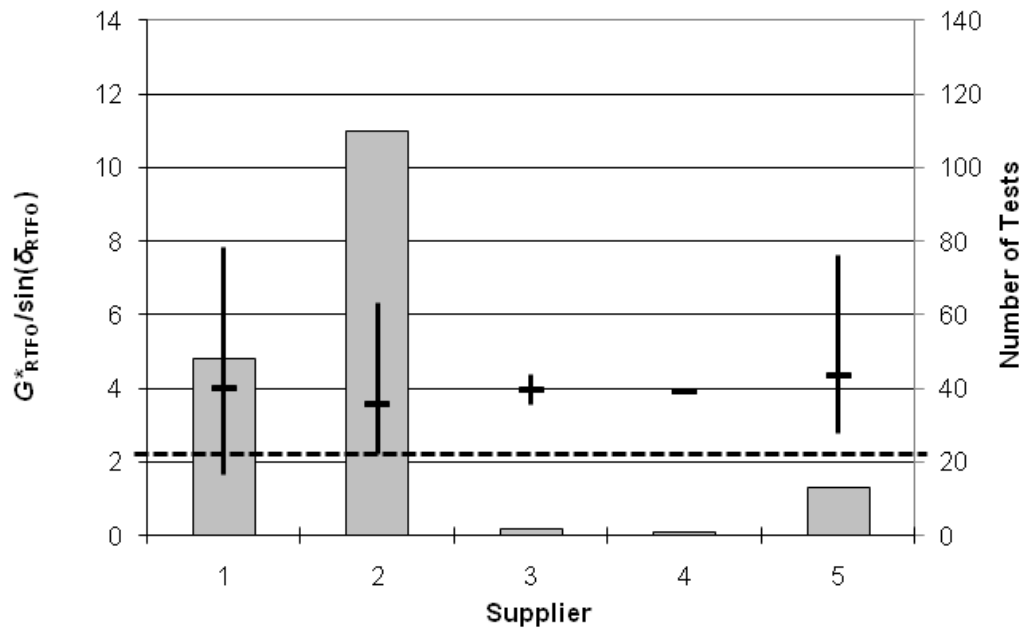
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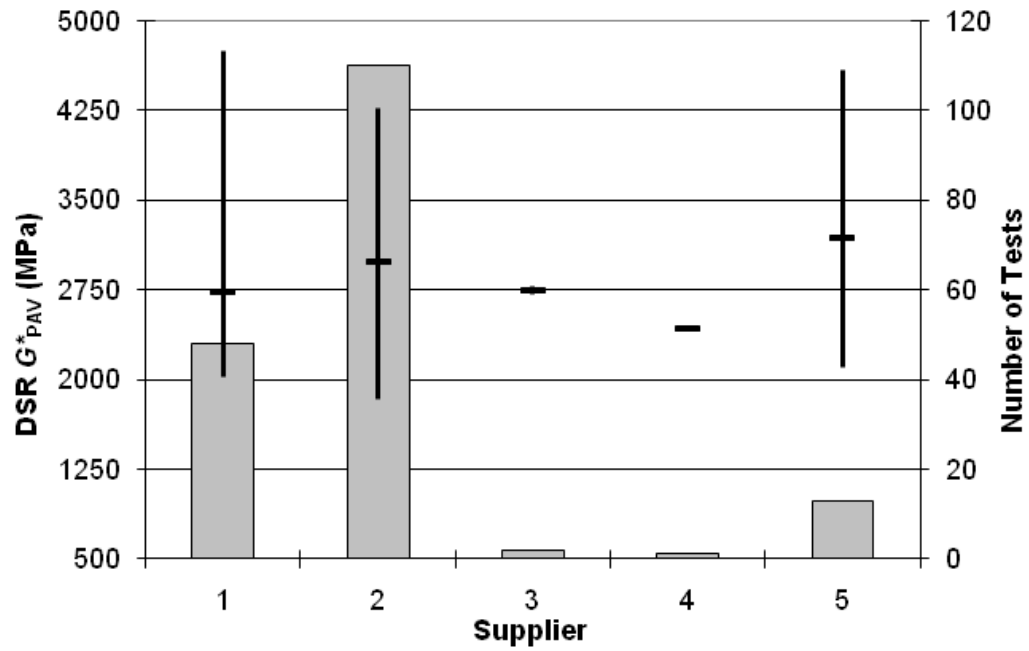
(d)



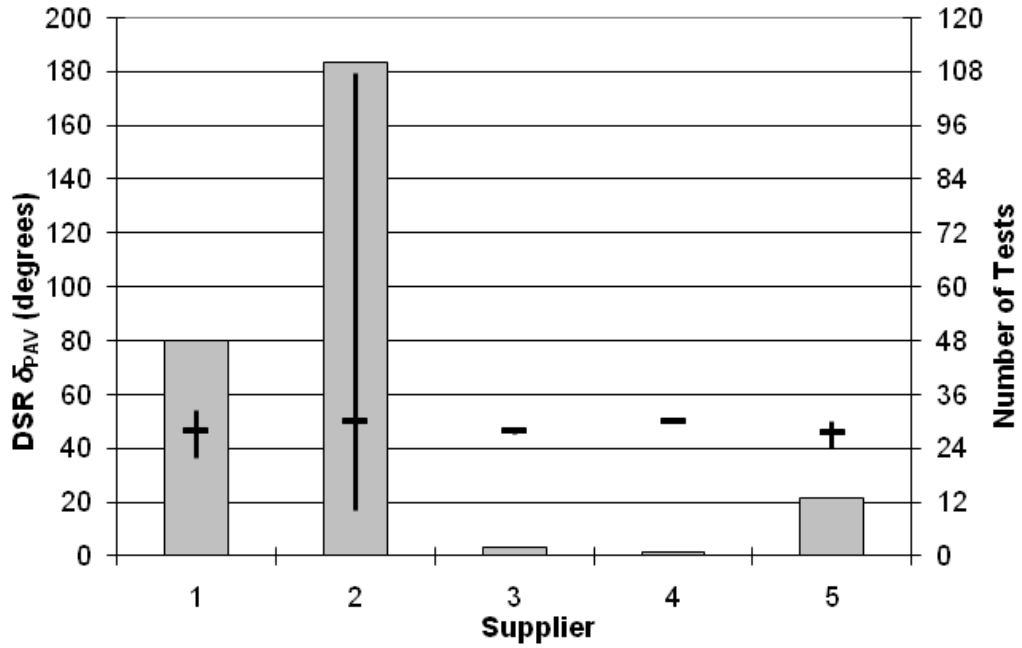
(e)



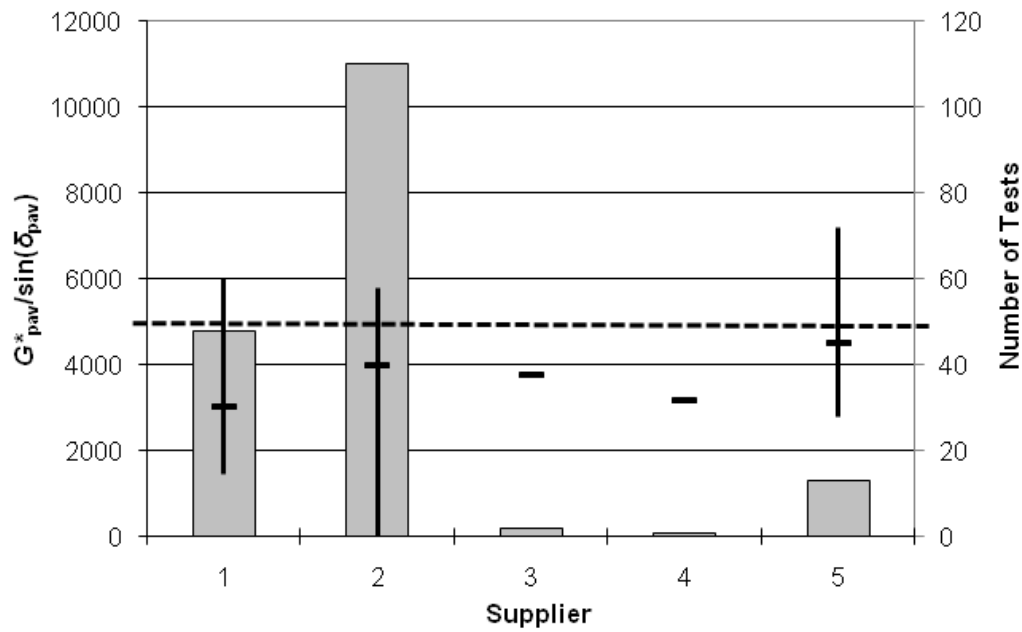
(f)



(g)



(h)



(i)

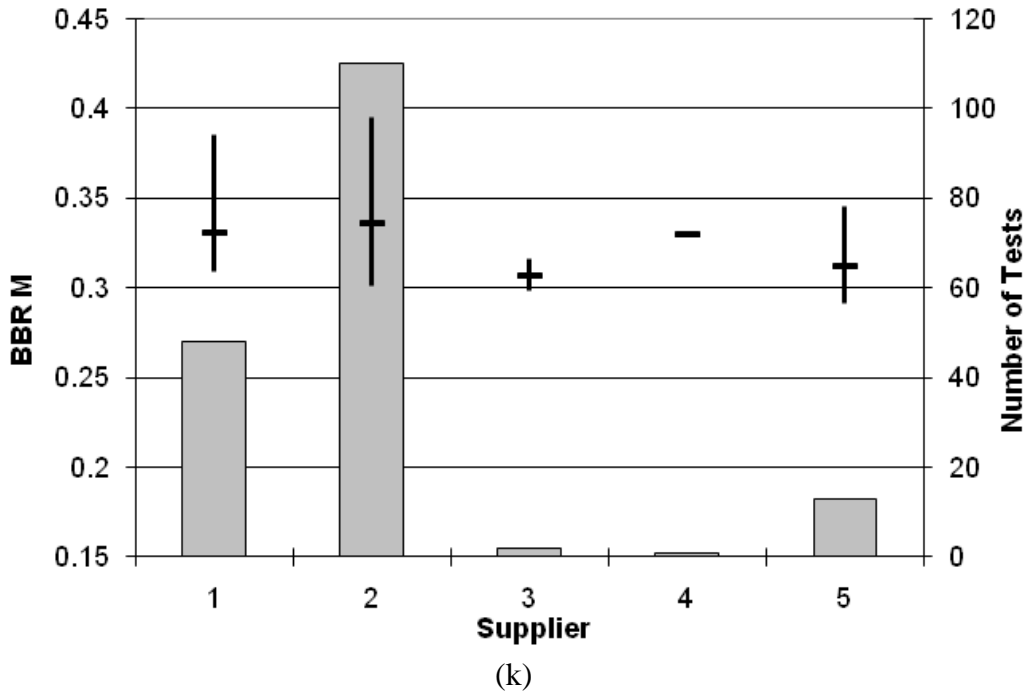
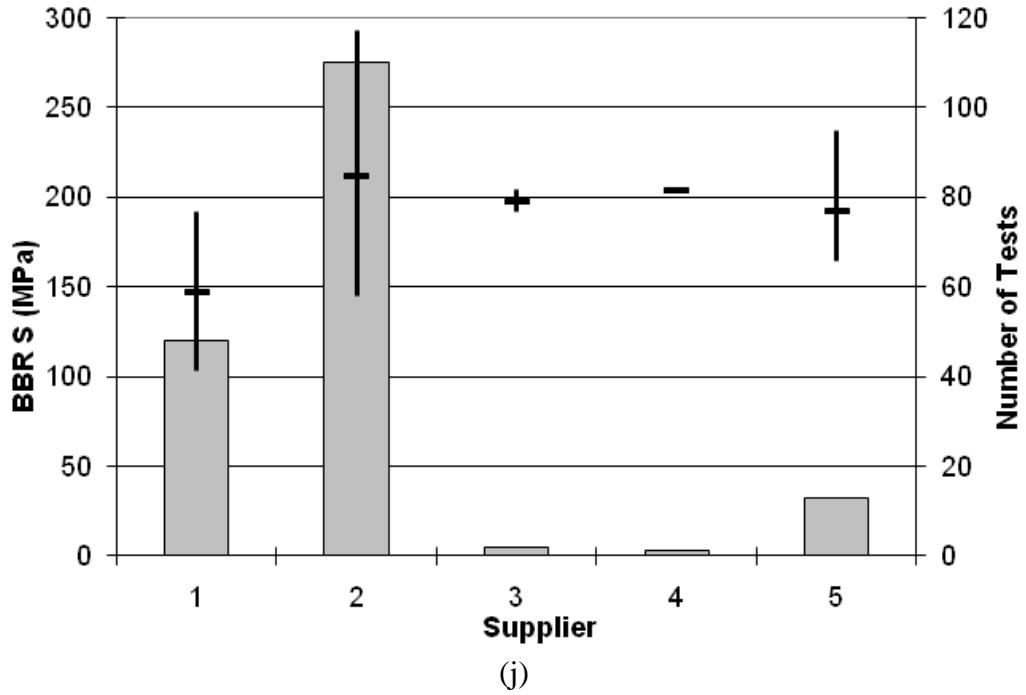
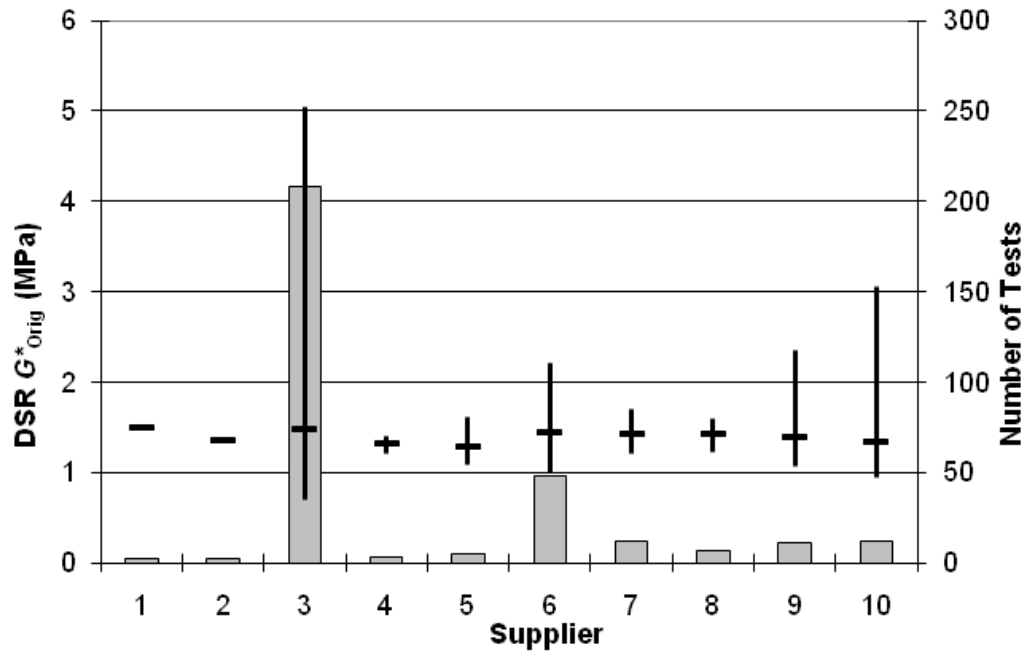
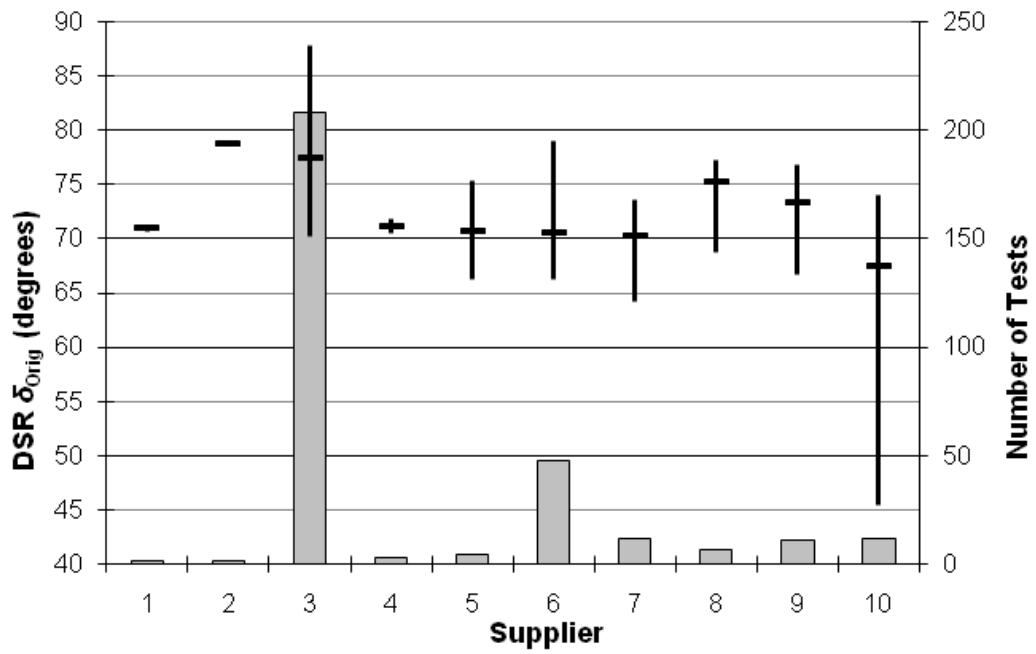


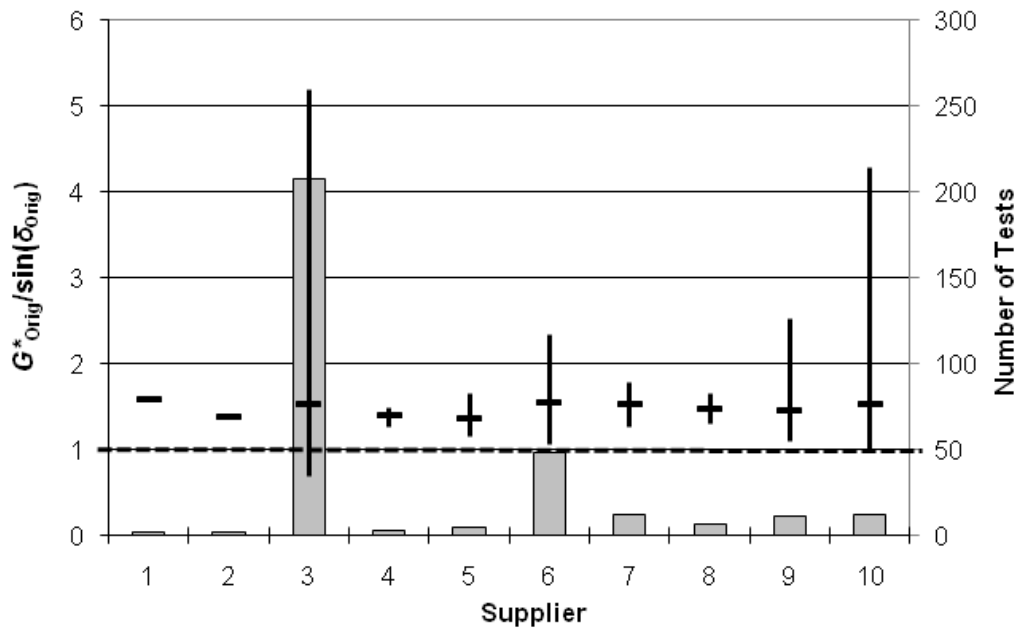
Figure 2. High/low/average/volume plots for PG 70-22 binder acceptance properties: (a) binder stiffness G^*_{orig} , (b) phase angle δ_{Orig} , (c) ratio of $G^*_{Orig}/\sin(\delta_{Orig})$ at original conditions; (d) binder stiffness G^*_{RTFO} , (e) phase angle δ_{RTFO} , (f) ratio of $G^*_{RTFO}/\sin(\delta_{RTFO})$ at RTFO aged conditions; (g) binder stiffness G^*_{PAV} , (h) phase angle δ_{PAV} , (i) ratio of $G^*_{PAV}/\sin(\delta_{PAV})$ for PAV aged conditions; (j) BBR Stiffness and (k) BBR m value. Test temperature is 70 °C for original and RTFO aged conditions, 25 °C for PAV aged conditions and -12 °C for BBR.



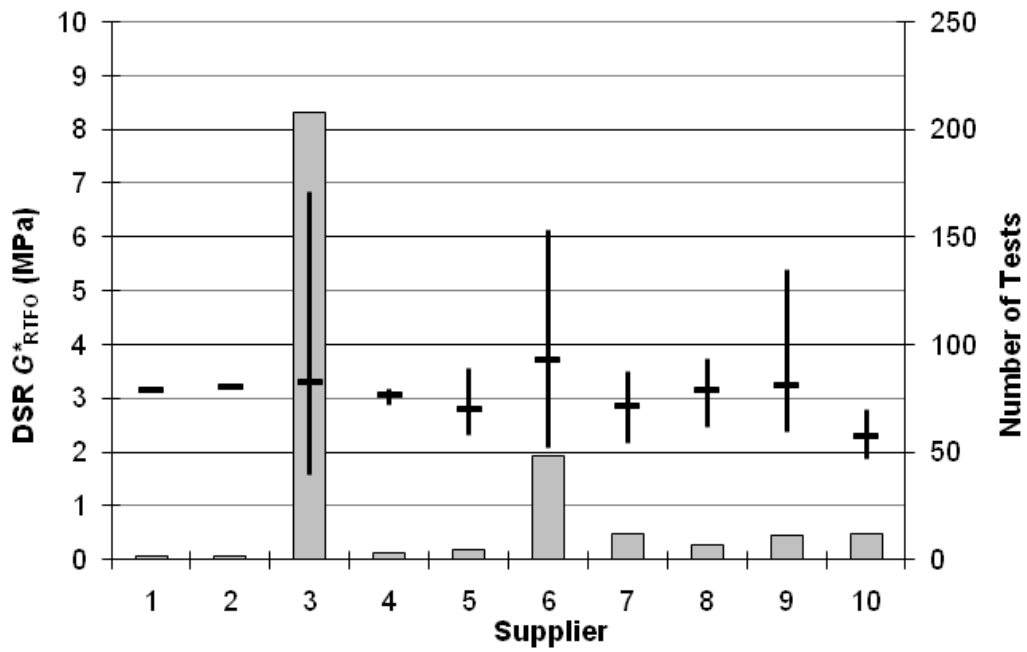
(a)



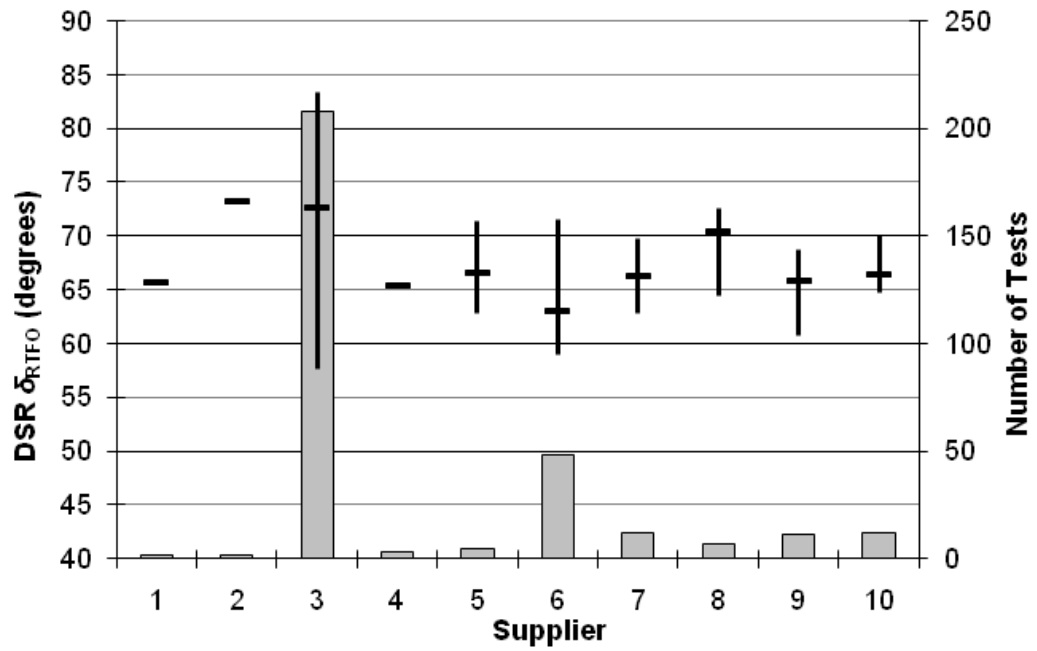
(b)



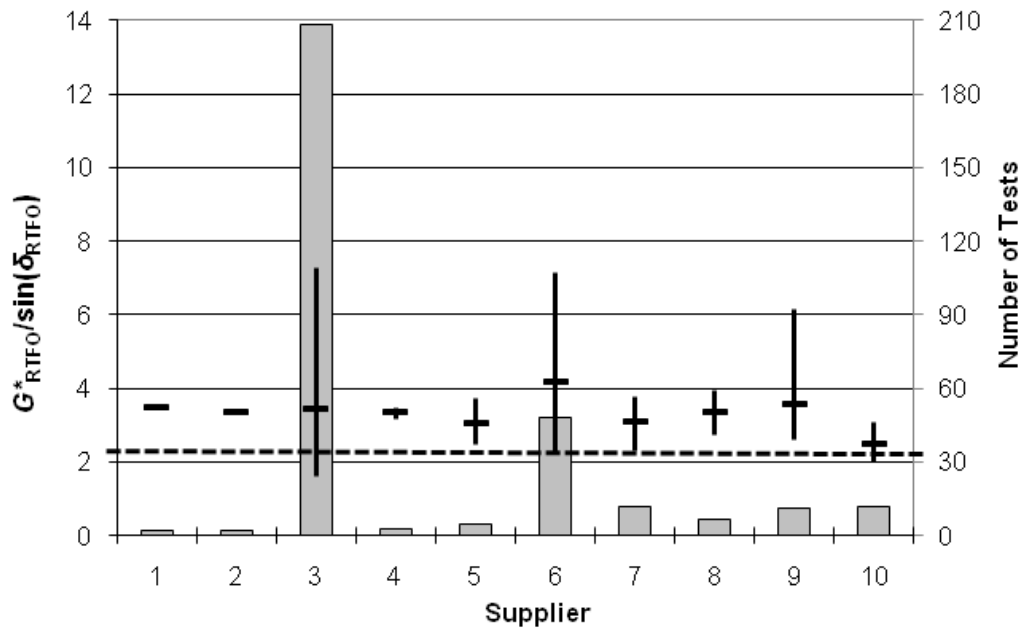
(c)



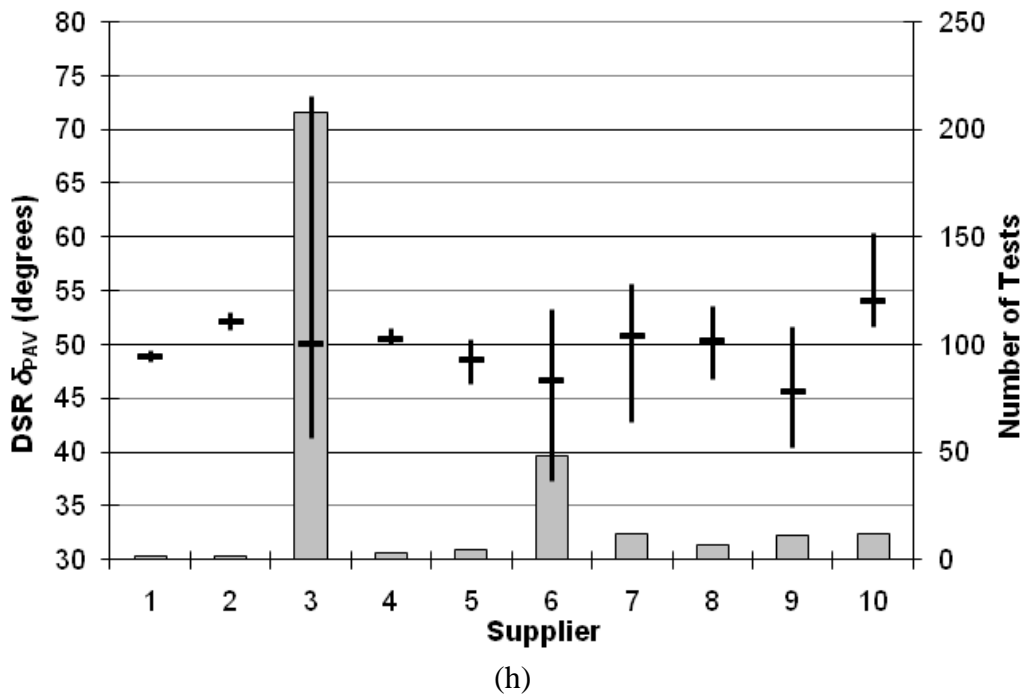
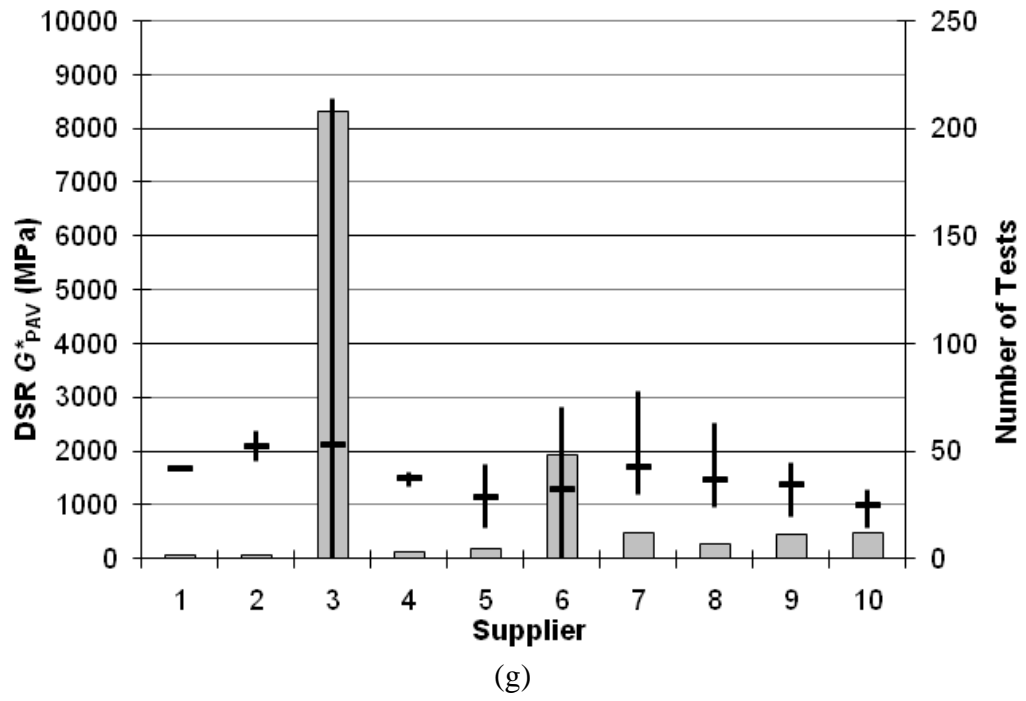
(d)

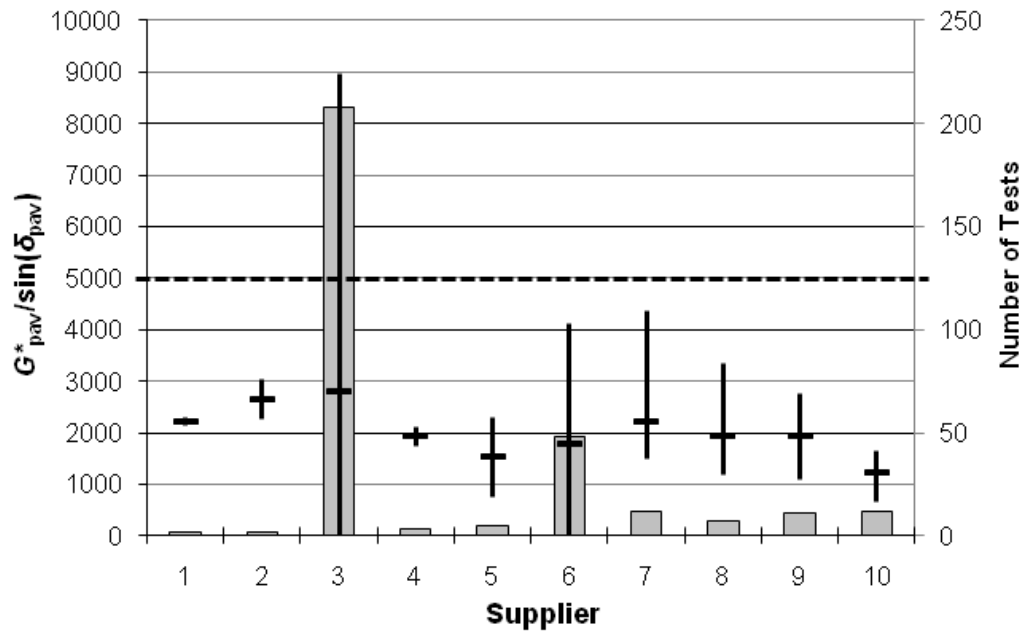


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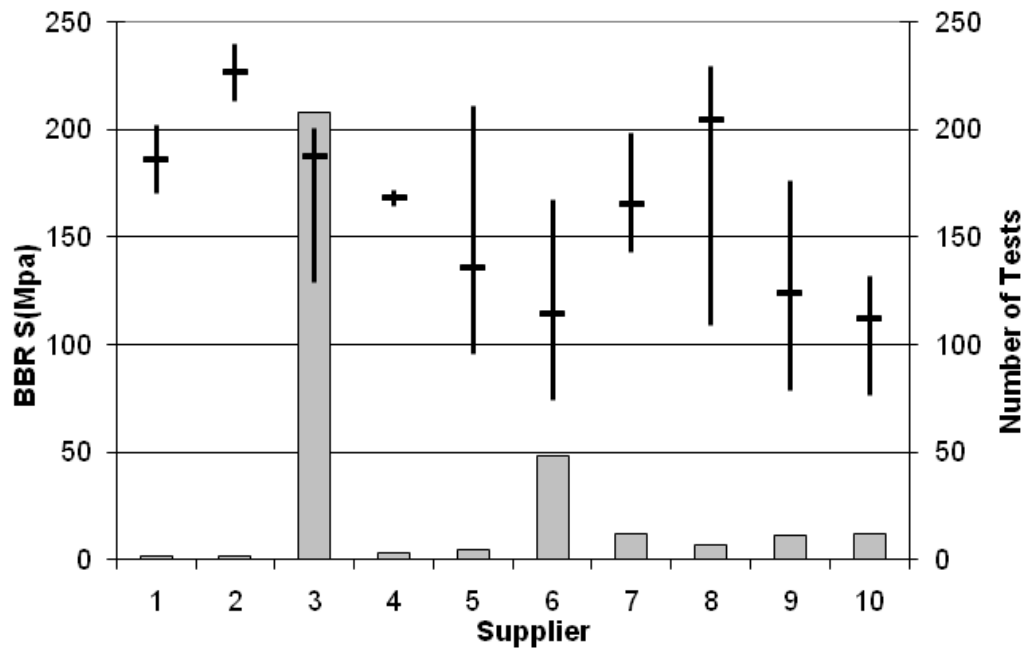


(f)





(i)



(j)

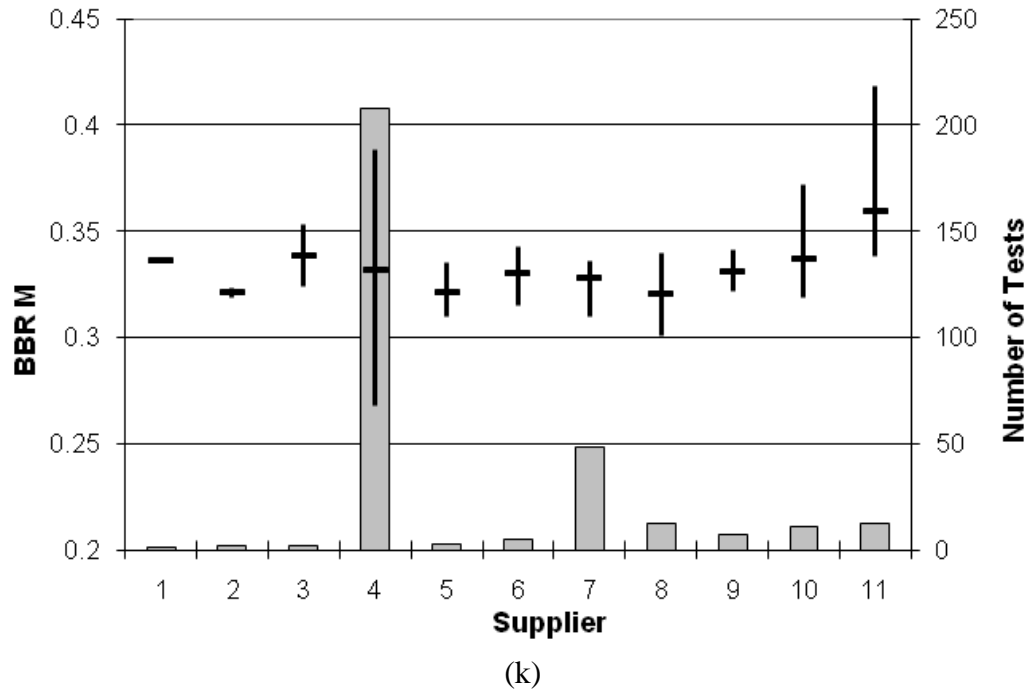


Figure 3. High/low/average/volume plots for PG 76-22 binder acceptance properties: (a) binder stiffness G^*_{orig} , (b) phase angle δ_{orig} , (c) ratio of $G^*_{Orig}/\sin(\delta_{Orig})$ at original conditions; (d) binder stiffness G^*_{RTFO} , (e) phase angle δ_{RTFO} , (f) ratio of $G^*_{RTFO}/\sin(\delta_{RTFO})$ at RTFO aged conditions; (g) binder stiffness G^*_{PAV} , (h) phase angle δ_{PAV} , (i) ratio of $G^*_{pav}/\sin(\delta_{pav})$ for PAV aged conditions; (j) BBR Stiffness and (k) BBR m value. Test temperature is 76 °C for original and RTFO aged conditions, 25 °C for PAV aged conditions and -12 °C for BBR.

As documented later in Chapter 6, binder property tables for the *MatProp* database have been designed to accommodate both the current Superpave acceptance testing data provided by SHA and to permit future entry of full Superpave characterization data—i.e., DSR at multiple temperatures at RTFO conditions, BBR, etc.

No conventional binder viscosity data (e.g., Brookfield viscosity, penetration, etc.) were provided by SHA. Therefore, no provisions for storing these older superseded viscosity characteristics have been included in the database design.

Sensitivity Analysis of Level 1/2 vs. Level 3 Binder Property Data

Since acceptance testing is performed at a single temperature, it does not provide sufficient information for Level 1 or Level 2 Superpave binder characterization in the MEPDG. Therefore, only Level 3 inputs—PG grade—can be provided for the binders based on the data received from SHA.

The major question regarding appropriate input levels for binder property data is: “Are there significant differences in predicted performance from the MEPDG using Level 1, 2, or 3 binder property data?” A review of the literature found no published studies that specifically addressed this question. Therefore, a very limited comparison analysis using the MEPDG for typical Maryland conditions was conducted. The analysis scenario was a simple pavement section consisting of 6 inches of HMA (19mm dense graded, PG 76-22) over 15 inches of granular base (A-1-b) over subgrade (A5, upper 12 inches compacted). The HMA was based on the control mixture at the FHWA ALF test, which was designed using aggregates and unmodified binders similar to those commonly used in dense graded mixtures in Maryland. Level 2 binder test data was extracted from the FHWA ALF research reports. Level 3 defaults were assumed for all other material properties. Traffic was set at 950 trucks per day in the design lane (TTC4 for Principal Arterials – Interstates and Defense Highways) and Baltimore (BWI) weather history (interpolated with DC and IAD weather history) was taken as the climate input. Reliability was set at the MEPDG default 90% level for all distresses.

The distresses predicted by the MEPDG using Level 3 vs. Level 1 inputs for this scenario are summarized in Table 3 (recall that Level 2 binder inputs are the same as Level 1). The MEPDG consistently predicts slightly higher distress magnitudes using

Level 1 than Level 3 inputs for this scenario, but the differences are very small. Although this comparison is extremely limited (i.e., just one binder, albeit of a type commonly used in Maryland), a reasonable conclusion is that, based just on the binder influence alone, it does not seem worthwhile for SHA to embark on a large-scale Level 1/Level 2 binder testing program. However, as will be shown in Chapter 3, this conclusion is superseded when considering Level 1 vs. Level 3 HMA properties. Level 1 stiffness data for the binder must be entered into the MEPDG if Level 1 dynamic modulus data is entered for the mixture. The Level 1 binder data is used by the global aging model in the MEPDG along with the Level 1 mixture data.

Table 3. Differences in predicted distresses using MEPDG Level 3 vs. Level 1 binder inputs.

Distress Type	Distress Magnitude		
	Level 3 Inputs	Level 1 Inputs	Δ (%) Level 3→1
Longitudinal Cracking (ft/mile)	470	501	+6.2
Alligator Cracking(% wheelpath)	2.31	2.43	+4.9
Transverse Cracking (ft/mile)	0	0	--
Subgrade Rutting (in)	0.2655	0.2663	+0.3
Base Rutting(in)	0.0998	0.1015	+1.7
HMA Rutting(in)	0.250	0.265	+5.7
Total Rutting (in)	0.615	0.633	+2.8
IRI (in/mile)	120.2	121.0	+0.7

Summary

Testing Recommendations

The sensitivity of predicted pavement performance to Level 1 vs. Level 2 vs. Level 3 binder inputs appears slight. Therefore, based only on this criterion there would be little purpose for SHA collection of Level 1 or 2 binder data. As will be shown in Chapter 3, however, predicted pavement performance can be substantially different using MEPDG Level 1 vs. Level 2 vs. Level 3 HMA mixture inputs. There is consequently a

motivation for SHA collection of Level 1 HMA dynamic modulus values. However, input of Level 1 HMA properties also requires input of Level 1/2 binder data.

It is recommended that SHA develop a policy of full binder characterization on major projects and that the test results be entered into the material property database so that typical Level 1/2 properties can be input into the MEPDG in the future. The testing frequency for full binder characterization should match the recommendations for HMA dynamic modulus testing as detailed in Chapter 3.

Recommended MEPDG Inputs

Only binder acceptance data has been collected by SHA to date. This is insufficient for Level 1 or Level 2 inputs in the MEPDG. Consequently, only Level 3 binder data can be input at this time. Until Level 1 binder data become available, it is recommended that the PG grade for Level 3 input be selected according to the binder recommendations in the *SHA/OMT Pavement Design Guide*:

1. All HMA layers other than wearing course/surface layer: PG 64-22
2. HMA wearing courses/surface layers other than gap-graded: See Table 4.
3. Gap-graded HMA wearing courses/surface layers: PG 76-22

Table 4. Recommended Level 3 binder grade inputs for wearing courses/surface layers (OMT, 2006).

(a) Wearing surface for all counties except Garrett

		< 0.3		0.3 to 30		> 30	
		No Rut	Rut	No Rut	Rut	No Rut	Rut
< 1000 tons	Standard	64-22	64-22	64-22	70-22	70-22	70-22 P
	Slow	64-22	64-22	70-22	70-22 P	70-22	70-22 P
	Standing	64-22	70-22	70-22 P	70-22 P	70-22 P	70-22 P
> 1000 tons	Standard	64-22	64-22	64-22	70-22	70-22	76-22
	Slow	64-22	64-22	70-22	76-22	70-22	76-22
	Standing	64-22	70-22	76-22	76-22	76-22	76-22

(b) Wearing surface for Garrett county

		< 0.3		0.3 to 30		> 30	
		No Rut	Rut	No Rut	Rut	No Rut	Rut
< 1000 tons	Standard	64-28	64-28	64-28	64-28	70-22	70-22 P
	Slow	64-28	64-28	70-22	70-22 P	70-22	70-22 P
	Standing	64-28	64-28	70-22 P	70-22 P	70-22 P	70-22 P
> 1000 tons	Standard	64-28	64-28	64-28	64-28	70-22	76-22
	Slow	64-28	64-28	70-22	76-22	70-22	76-22
	Standing	64-28	64-28	76-22	76-22	76-22	76-22

Standing Traffic - where the average traffic speed is less than 12 mph (20 km/h).

Slow Traffic - where the average traffic speed ranges from 12 to 43 mph (20 to 70 km/h).

Standard Traffic - where the average traffic speed is greater than 43 mph (70 km/h).

CHAPTER 3: HMA DATA

MEPDG Input Requirements

New Construction/Reconstruction/Overlays

Dynamic modulus is the principal mechanical property input for HMA in the MEPDG. The methods for specifying dynamic modulus at each of the input levels in the MEDPG are as follows:

- Level 1: Laboratory-measured dynamic modulus $|E^*|$ at multiple temperatures and loading frequencies (AASHTO TP62). In addition, Level 1/2 binder stiffness and phase angle data are required for the global aging model.
- Level 2: Gradation and volumetric data for use in the Witczak $|E^*|$ predictive model: percent retained above the 3/4" sieve; percent retained above the 3/8" sieve; percent retained above the #4 sieve; percent passing the #200 sieve; effective volumetric binder content (%); and in-place air voids (%). In addition, Level 1/2 stiffness and phase angle data are also required for the binder.
- Level 3: Gradation and volumetric data for use in the Witczak $|E^*|$ predictive model. Default binder stiffness properties are based on the Superpave Performance Grade for the binder.

Creep compliance and low temperature tensile strength are additional mechanical properties required in the MEPDG for predicting thermal cracking distress. The methods for specifying these properties at each of the input levels in the MEPDG are as follows:

- Level 1: Laboratory-measured creep compliance at three temperatures and various loading times and laboratory-measured tensile strength at 14°F (AASHTO T322).

- Levels 2 and 3: Default creep compliance and low temperature tensile strength determined from empirical relations built into the MEPDG; empirical relations are functions of mix volumetric and binder viscosity properties.

HMA thermal properties required by the MEPDG include:

- Thermal conductivity and heat capacity: see Table 5.
- Surface shortwave absorptivity (SSA), which quantifies the fraction of available solar energy that is absorbed by a given surface. Lighter and more reflective surfaces have lower SSA values. The recommended methods for determining SSA at each of the input levels are:
 - Level 1: Estimate through laboratory testing. However there is no AASHTO certified testing standards for this.
 - Levels 2 and 3: Default values based on surface characteristics:
 - Weathered asphalt (gray) 0.80-0.90
 - Fresh asphalt (black) 0.90-0.98
- Aggregate coefficient of thermal expansion (also sometimes called coefficient of thermal contraction): see Table 6.

Additional physical mixture properties required for all input levels are Poisson's ratio and total unit weight. Both of these properties have relatively small influence on predicted pavement performance. There is no national test protocol for measuring Poisson's ratio for HMA; the default Level 3 values recommended in the MEDPG are given in

Table 7. HMA total unit weight can be measured in the laboratory according to AASHTO T166 or estimated based on previous construction records.

Table 5. MEPDG thermal conductivity and heat capacity inputs. (NCHRP, 2004).

Material Property	Input Level	Description
Thermal Conductivity, K	1	A direct measurement is recommended at this level (ASTM E 1952, "Standard Test Method for Thermal Conductivity and Thermal Diffusivity by Modulated Temperature Differential Scanning Calorimetry").
	2	Correlations are not available. Use default values from Level 3.
	3	User selects design values based upon agency historical data or from typical values shown below: <ul style="list-style-type: none"> • Typical values for asphalt concrete range from 0.44 to 0.81 Btu/(ft)(hr)(°F).
Heat Capacity, Q	1	A direct measurement is recommended at this level (ASTM D 2766, "Specific Heat of Liquids and Solids").
	2	Correlations are not available. Use default values from Level 3.
	3	User selects design values based upon agency historical data or from typical values shown below: <ul style="list-style-type: none"> • Typical values for asphalt concrete range from 0.22 to 0.40 Btu/(lb)(°F).

Table 6. Typical coefficient of thermal expansion ranges for common aggregates (NCHRP, 2004).

Material Type	Coefficient of Thermal Expansion, 10 ⁻⁶ /°F
Aggregates	
Marbles	2.2-3.9
Limestones	2.0-3.6
Granites & Gneisses	3.2-5.3
Syenites, Diorites, Andesite, Basalt, Gabbros, Diabase	3.0-4.5
Dolomites	3.9-5.5
Blast Furnace Slag	
Sandstones	5.6-6.7
Quartz Sands & Gravels	5.5-7.1
Quartzite, Cherts	6.1-7.0

Table 7. Typical Poisson's ratio values for HMA mixtures (from NCHRP, 2004; AASHTO, 2008).

Reference Temperature (°F)	Dense Graded	Open Graded
< 0	0.15	0.35
0 – 40	0.20	0.35
40 – 70	0.25	0.40
70 – 100	0.35	0.40
100 – 130	0.45	0.45
> 130	0.48	0.45

Rehabilitation

The primary difference between characterizing new and existing HMA layers is that the dynamic modulus for existing HMA layer must be adjusted for the damage caused to the pavement by traffic loads and environmental effects. Table 8 summarizes the stated methods for determining dynamic modulus for existing layers at each of the input levels in the MEPDG. However, only Level 3 (specification of damage indirectly via pavement condition rating) is implemented in the current Version 1.100 of the MEPDG software.

Table 8. Asphalt dynamic modulus determination for rehabilitation design at different input levels. (NCHRP, 2004).

Material Group Category	Type Design	Input Level	Description
Asphalt Materials (existing layers)	Rehab	1	<ul style="list-style-type: none"> • Use NDT-FWD backcalculation approach. Measure deflections, backcalculate (combined) asphalt bound layer modulus at points along project. • Establish backcalculated E_i at temperature-time conditions for which the FWD data was collected along project. • Obtain field cores to establish mix volumetric parameters (air voids, asphalt volume, gradation, and asphalt viscosity parameters to determine undamaged Master curve). • Develop undamaged Master curve with aging for site conditions by sigmoidal function: $\log(E^*) = \delta + \frac{\alpha}{1 + e^{\beta + \gamma \log t_r}}$ <ul style="list-style-type: none"> • where • t_r = Time of loading at the reference temperature • δ = Minimum value of E^* • $\delta + \alpha$ = Maximum value of E^* • β, γ = Parameters describing the shape of the sigmoidal function • Estimate damage, d_j, by: <ul style="list-style-type: none"> • $d_j = E_{i(NDT)} / E^*_{(Pred)}$

			<ul style="list-style-type: none"> • In sigmoidal function, δ is minimum value and α is specified range from minimum. • Define new range parameter α' to be: • $\alpha' = (1-dj) \alpha$ • Develop field damaged master curve using α' rather than α
	2		<ul style="list-style-type: none"> • Use field cores to establish mix volumetric parameters (air voids, asphalt volume, gradation, and asphalt viscosity parameters to define Ai-VTSi values). • Develop by predictive equation, undamaged master curve with aging for site conditions from mix input properties determined from analysis of field cores. • Conduct indirect Mr laboratory tests, using revised protocol developed at University of Maryland for NCHRP 1-28A from field cores. • Use 2 to 3 temperatures below 70 °F • Estimate damage, dj, at similar temperature and time rate of load conditions: • $dj = Mr_i/E^*_{(Pred)}$ • In sigmoidal function, δ is minimum value and α is specified range from minimum. Define new range parameter α' to be: • $\alpha' = (1-dj) \alpha$ • Develop field damaged master curve using α' rather than α
	3		<ul style="list-style-type: none"> • Use typical estimates of mix modulus prediction equation (mix volumetric, gradation and binder type) to develop undamaged master curve with aging for site layer. • Using results of distress/condition survey, obtain estimate for pavement rating (excellent, good, fair, poor, very poor) • Use a typical tabular correlation relating pavement rating to pavement layer damage value, dj. • In sigmoidal function, δ is minimum value and α is specified range from minimum. Define new range parameter α' to be: • $\alpha' = (1-dj) \alpha$ • Develop field damaged master curve using α' rather than α

Other existing HMA layer properties are specified in the MEPDG as follows:

- Creep compliance and low temperature tensile strength: Not required for existing HMA layers.
- Thermal conductivity and heat capacity: Same as for new construction (Table 5).
- Surface shortwave absorptivity: Not required for existing HMA layers.
- Aggregate coefficient of expansion: Not required for existing HMA layers.

Unit weight and Poisson’s ratio: Same as for new construction (

- **Table 7).**

HMA Data Summary and Preliminary Analysis

A large set of asphalt mixture design properties was provided by SHA for initial population of the material properties database. The scope of the provided data is described in Table 9. The date range for these data is unknown, other than that they were received from SHA in Fall 2008. As for the binder data in Chapter 2, considerable effort was devoted to identifying incorrect or inconsistent information in the data. Incorrect or inconsistent data were corrected when possible and eliminated when not.

Table 9. Number of mixtures in database for each mixture size and type. Mixtures in bold italics were included in the correlation analyses.

HMAS	Mix Type	Number
4.75mm	High Polish	1
	Virgin	25
9.5mm	Gap Graded	10
	High Polish	122
	RAP	126
	Shingle	5
	Virgin	68
12.5mm	Gap Graded	40
	High Polish	56
	RAP	84
	Shingle	6
	Virgin	86
19.0mm	Gap Graded	1

	High Polish	24
	RAP	122
	Shingle	8
	Virgin	37
25.00mm	RAP	50
	Virgin	20
37.5mm	RAP	6

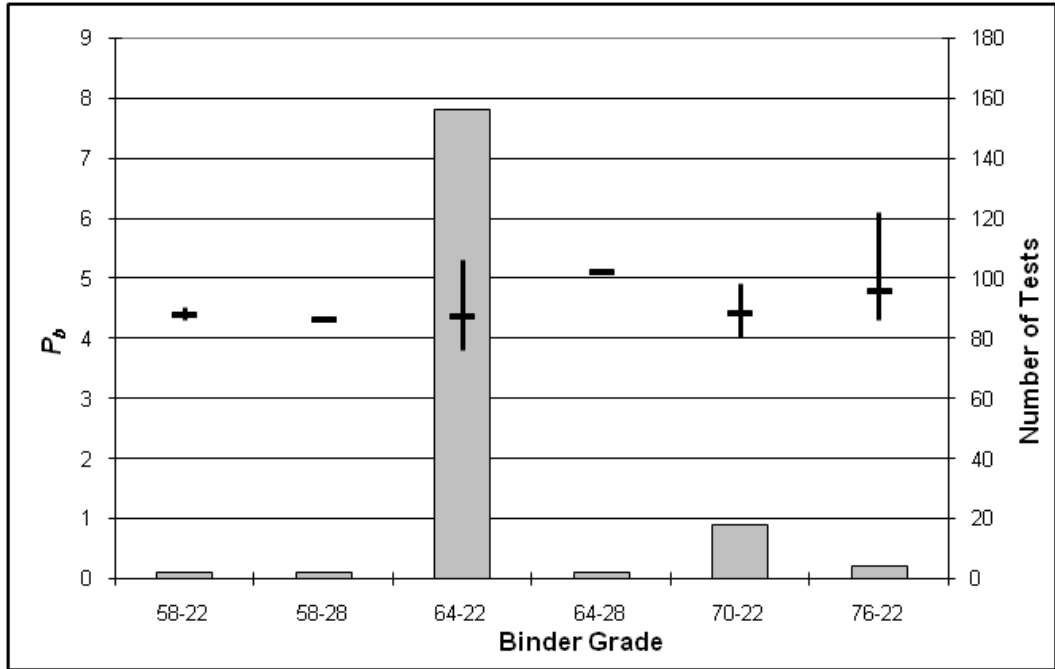
The HMA mixture information provided by SHA is limited to volumetric and gradation data suitable for Level 3 input to the MEPDG. No measured dynamic modulus, creep compliance, low temperature tensile strength, or thermal property values suitable for Level 1 inputs were provided. Although the volumetric and gradation data provided by SHA are sufficient for Level 2 inputs, the required corresponding Level 2 binder data are absent.

The simplest way to categorize typical Level 3 volumetric and gradation MEDPG inputs for Maryland materials is to define them as a function only of mix type (e.g., gap- vs. dense-graded) and mix size (e.g., 19 mm nominal maximum aggregate size). To explore whether this is possible, trends in volumetric and gradation data for a given mix type and mix size as a function of binder grade and/or traffic level were examined, as illustrated in Figure 4 and Figure 5 respectively for 19 mm dense-graded mixtures. In these figures, the grey bars indicate the number of tests included in the database for each subset of data (right axis), the heavy black vertical lines indicate the ranges of the data (left axis), and the heavy black short horizontal lines indicate the mean values (left axis). Noteworthy observations regarding the data in these figures include the following:

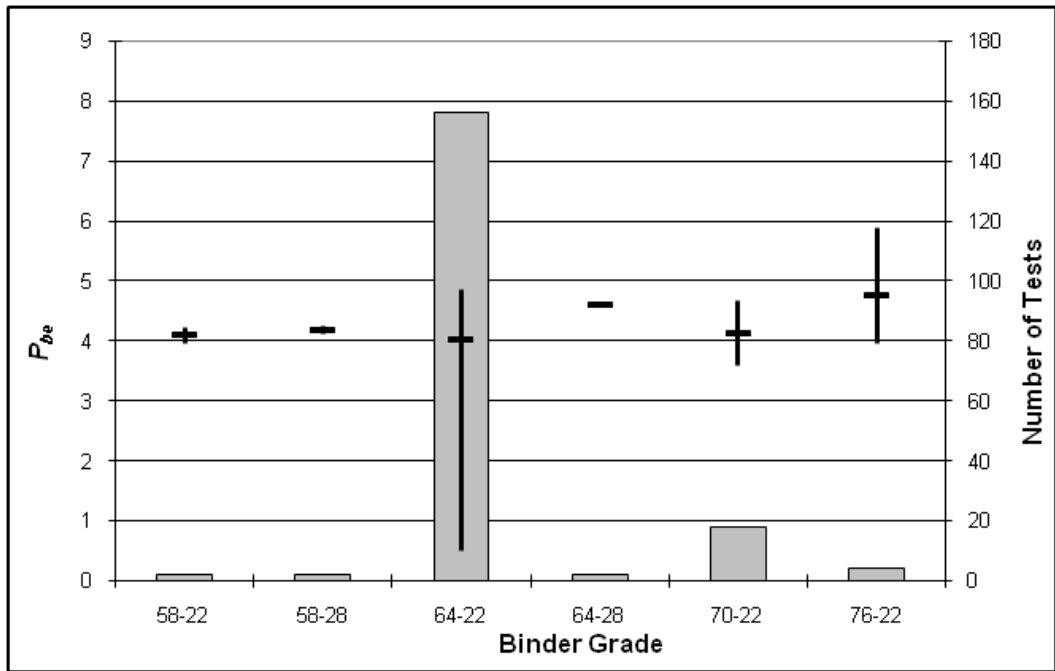
- The PG 64-22 is the most common binder in the data set (Figure 4). This is not surprising, as this is the recommended binder for Maryland environmental conditions under all but the heaviest traffic conditions.

- The ranges of the volumetric properties are largest for the PG 64-22 mixtures (Figure 4). This is most likely because these are the most common mixtures, and thus the opportunity for encountering especially high or low values is large.
- The ranges of the volumetric properties for the PG 70-22 and PG 76-22 mixtures (Figure 4), although not as large as for the PG 64-22 data, are still surprisingly large, especially given that the number of mixtures using these binders is comparatively small. Air voids V_a is the only exception to this trend (Figure 4h). Note that the PG 70-22 and PG 76-22 binders are generally specified by SHA for its premium mixtures—e.g., SMA surface mixtures on heavily trafficked interstate highways.
- The 0.3-3M ESAL traffic category is the most common design condition (Figure 5). Very few mix designs fall into the >30M ESAL very high traffic condition.
- Overall, the range of the volumetric properties is moderate to large for the four lowest traffic categories (Figure 5). There are insufficient mixtures in the highest traffic category to portray the property ranges accurately.

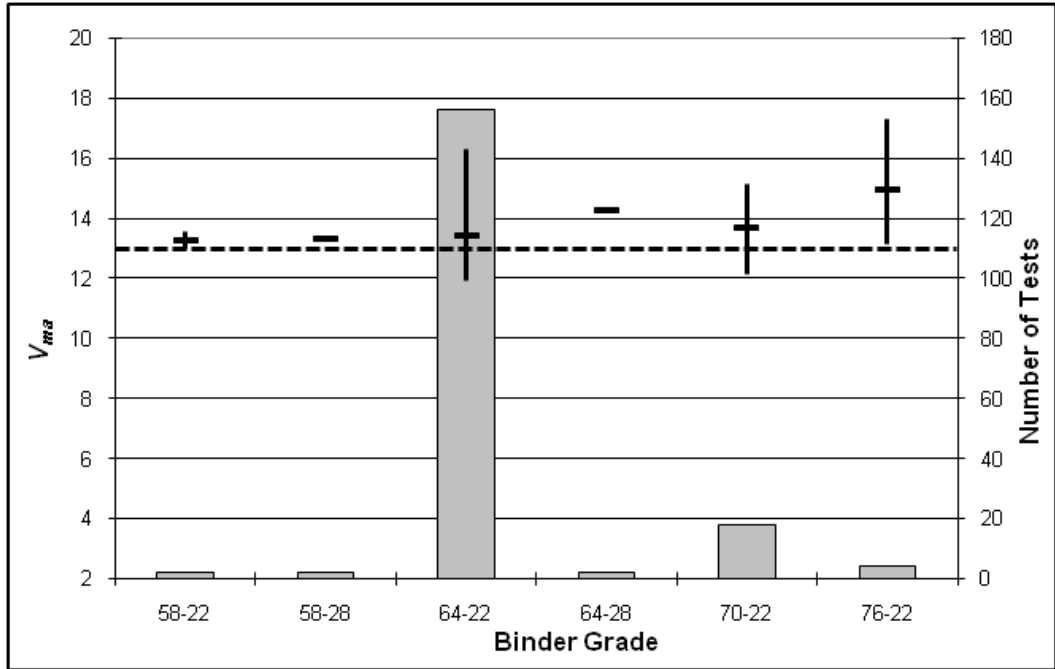
There were no consistent overall trends in the mean values for the volumetric and gradation properties either with regard to binder grade or traffic level. This is consistent with expectations, as the SHA mix design specifications for these properties are not functions of binder grade or traffic level.



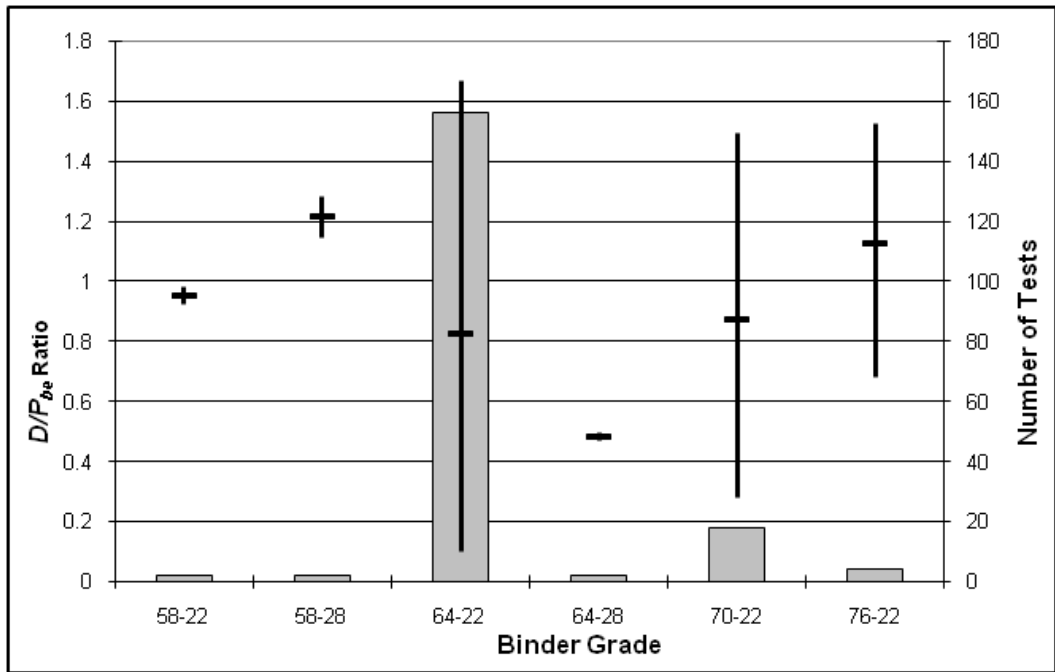
(a) Binder content by weight of mixture P_b .



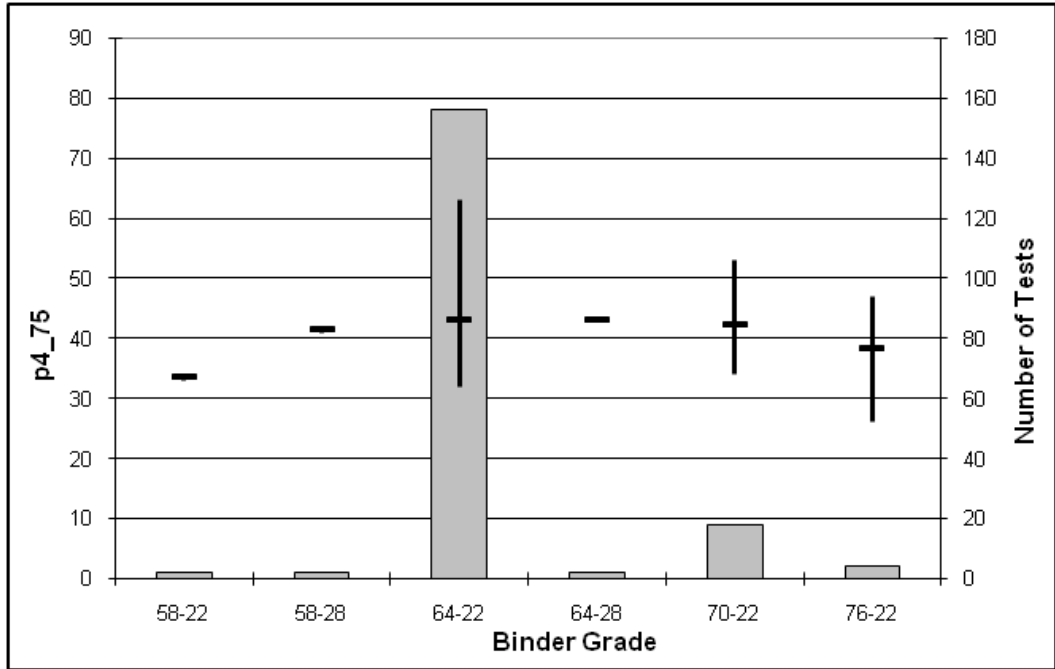
(b) Effective binder content by weight of mixture P_{be} .



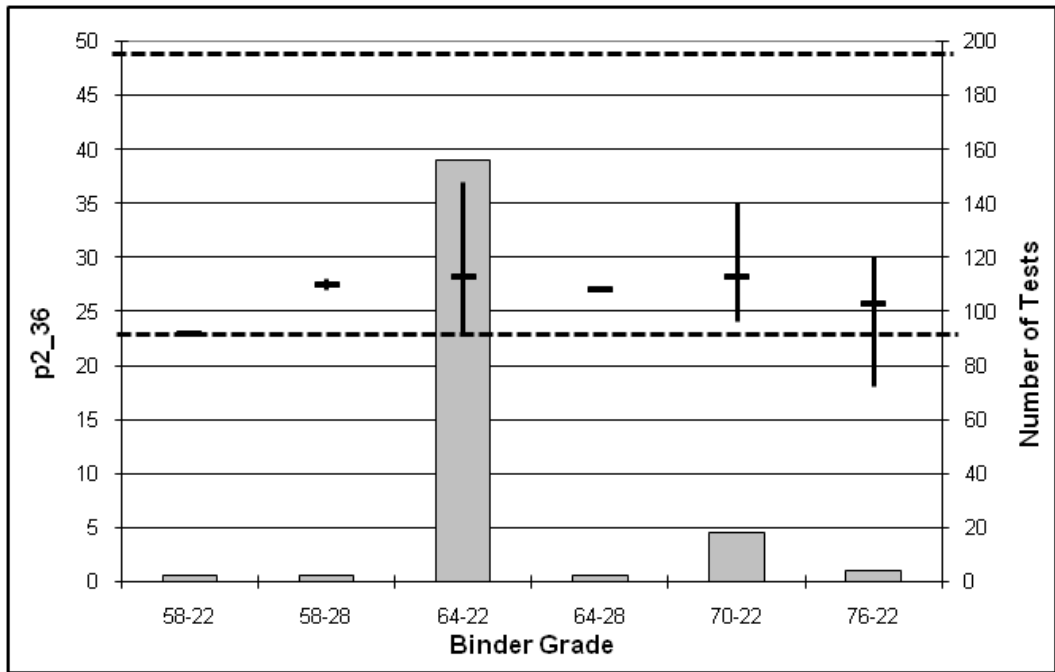
(c) Voids in mineral aggregates VMA . Minimum VMA is 13.



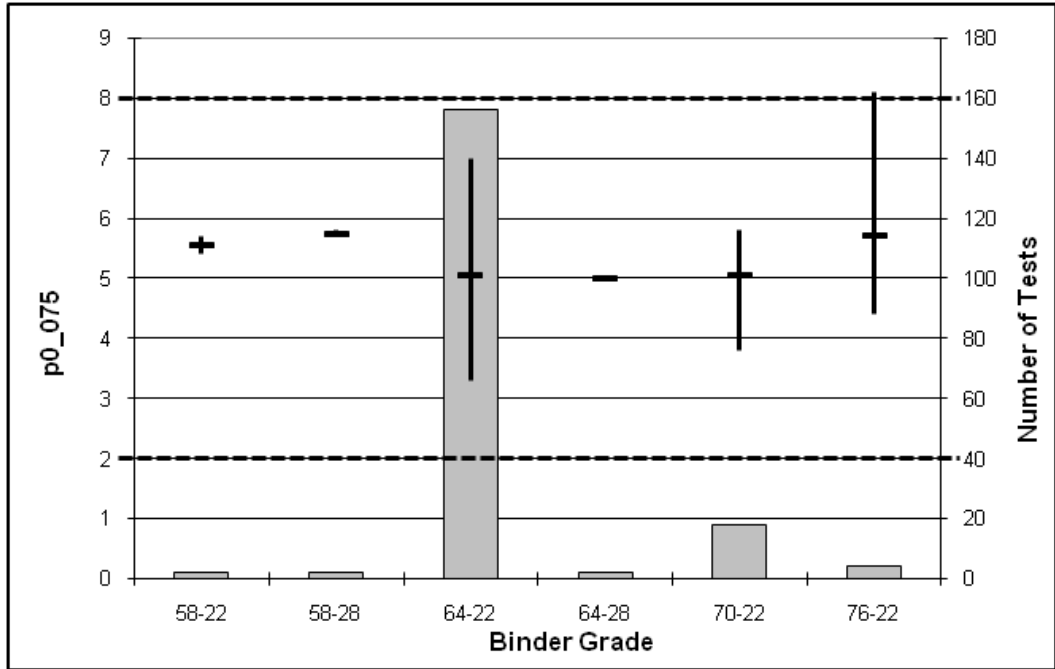
(d) Dust to effective binder ratio D/P_{be} .



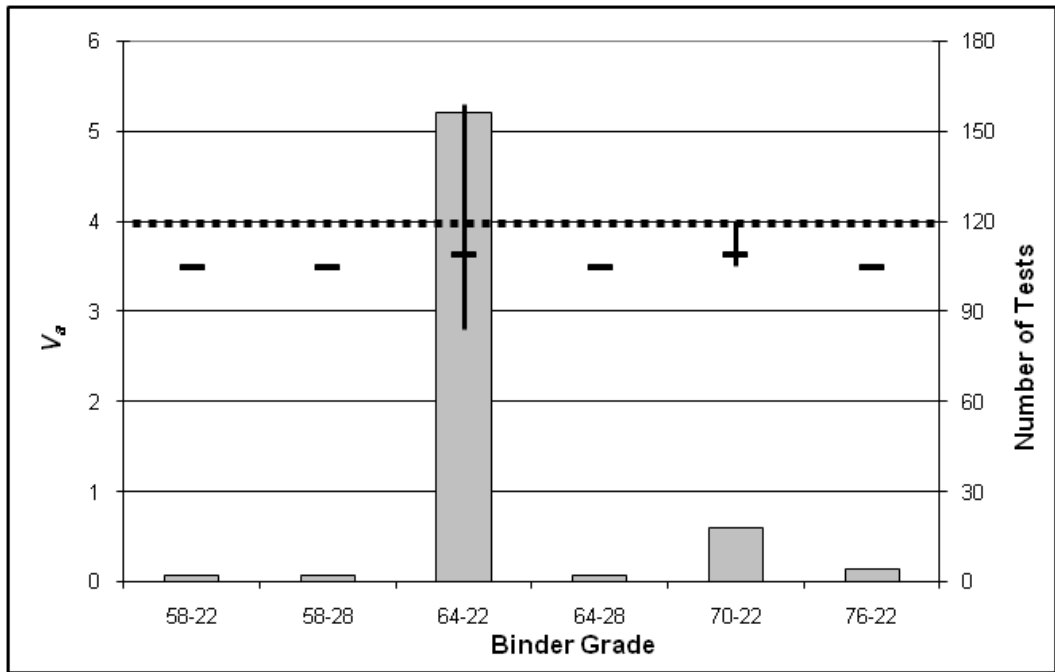
(e) Percent passing 4.75mm sieve size.



(f) Percent passing 2.36mm sieve size. Specific limits are 23 and 49.

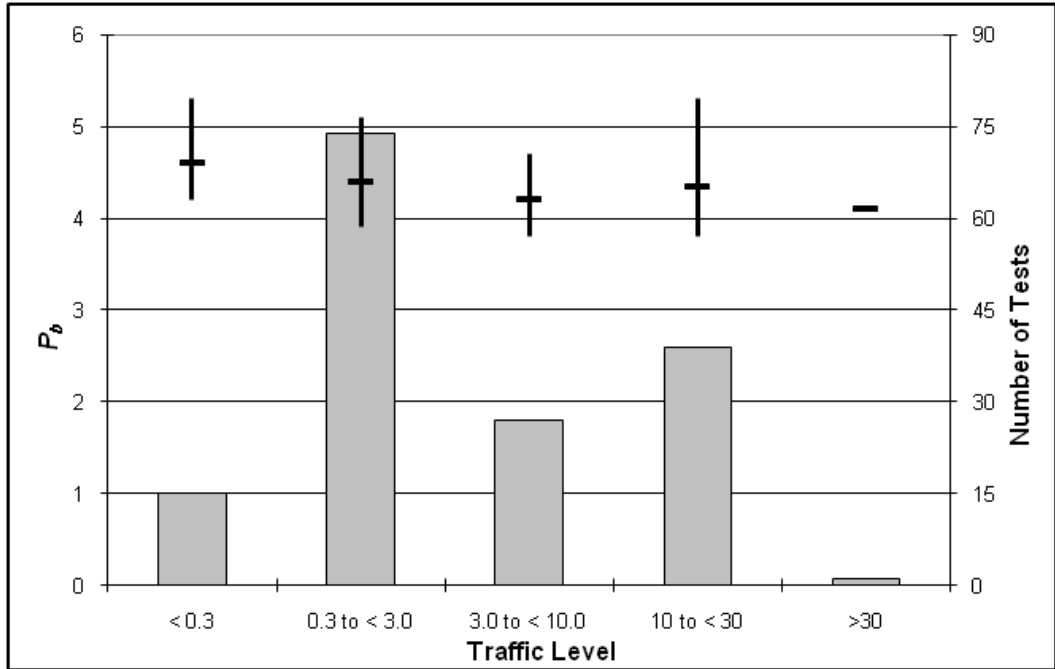


(g) Percent passing 0.075mm sieve size. Specific limits are 2 and 8.

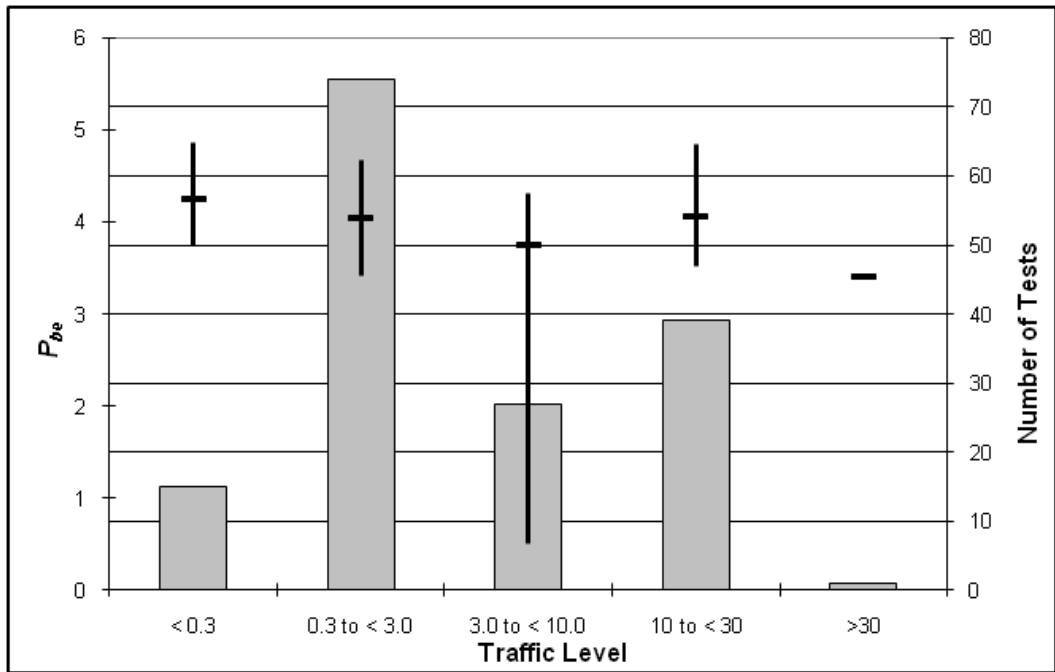


(h) Air voids V_a . Target value is 4.

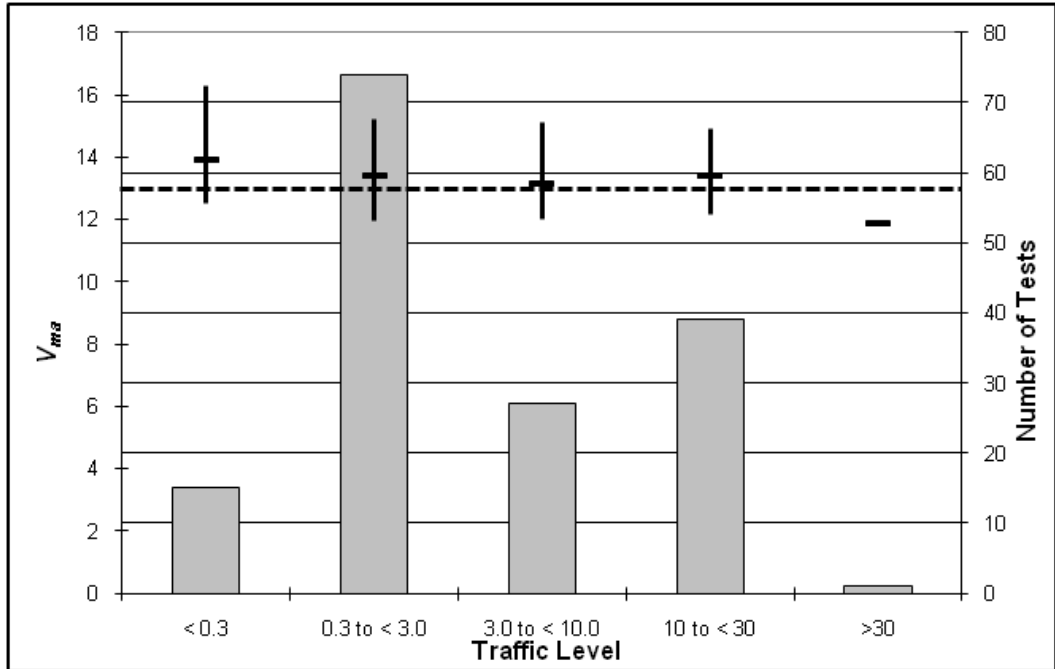
Figure 4. High/low/average/volume plots for 19mm dense graded mixtures. Data include all traffic volume categories.



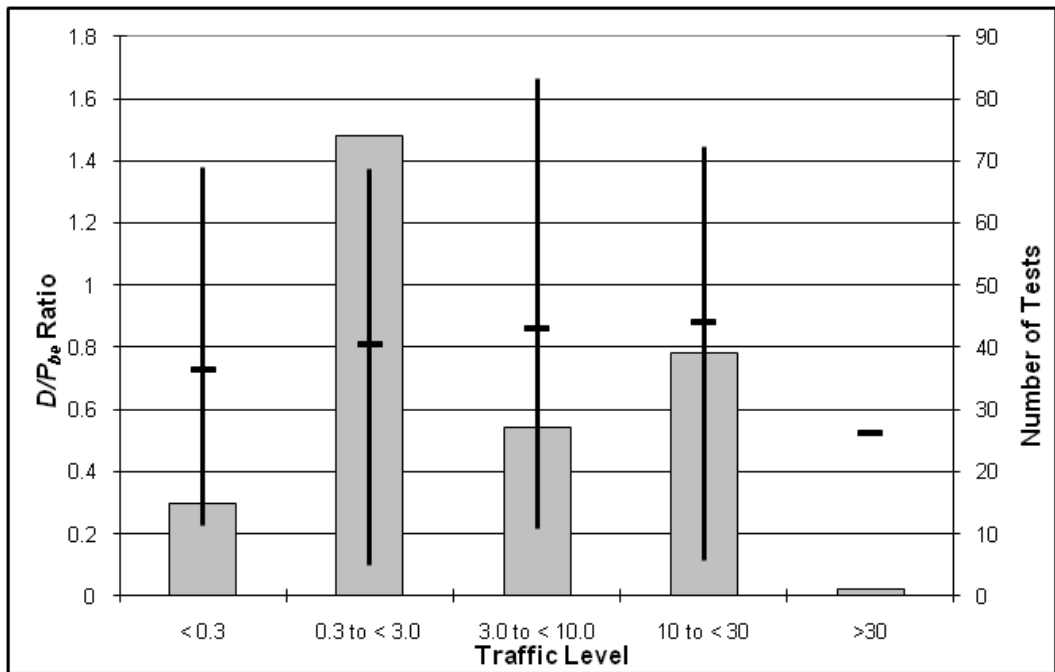
(a) Binder content by weight of mixture P_b .



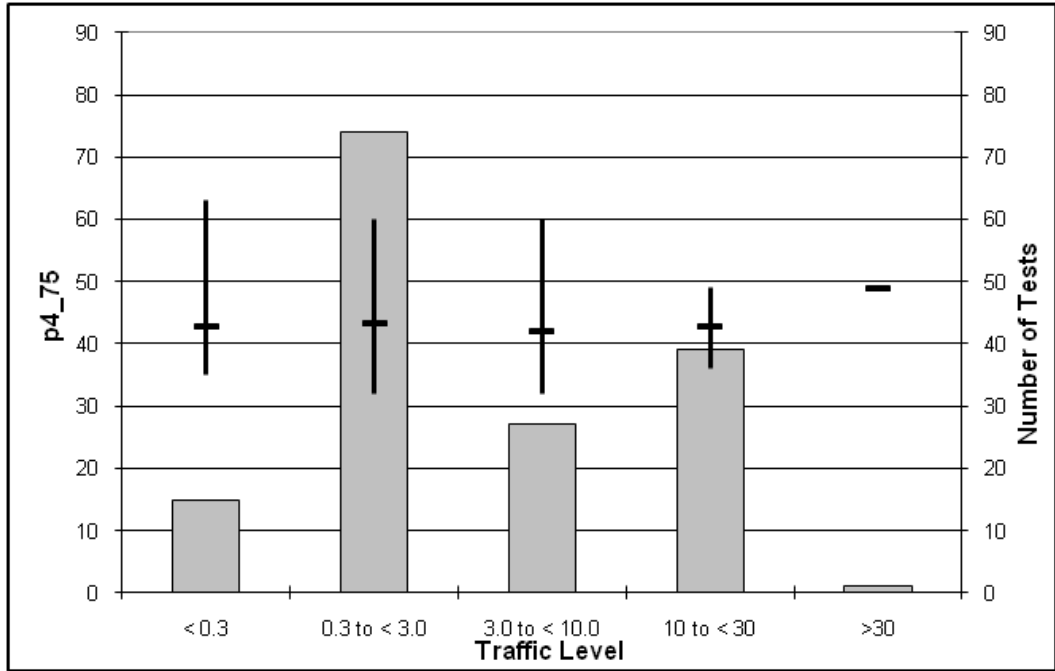
(b) Effective binder content by weight of mixture P_{be} .



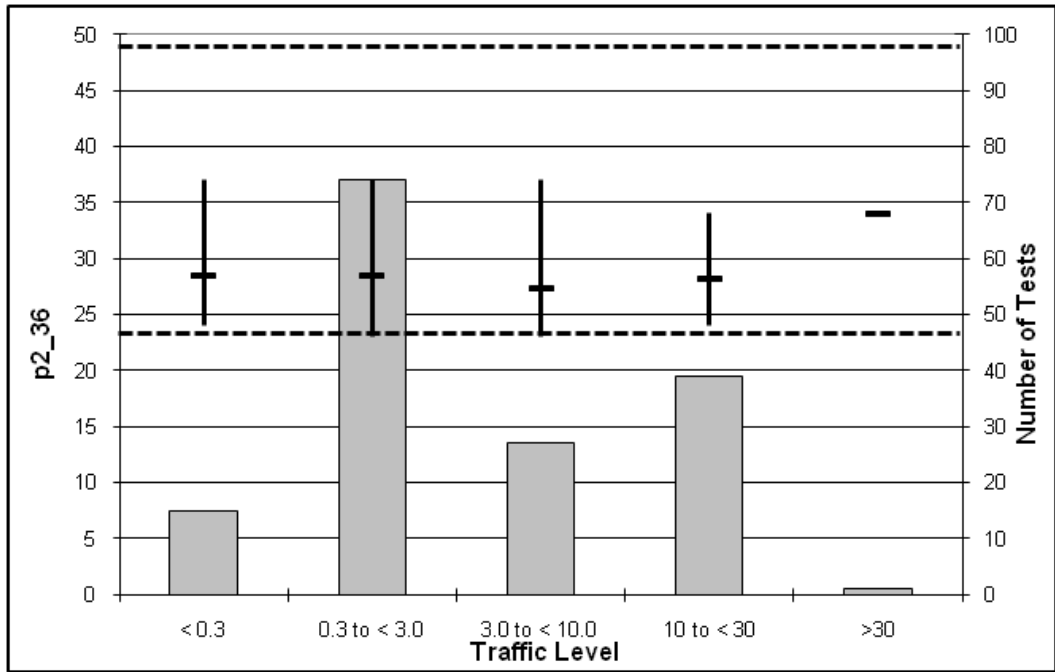
(c) Voids in mineral aggregates VMA . Minimum value is 13.



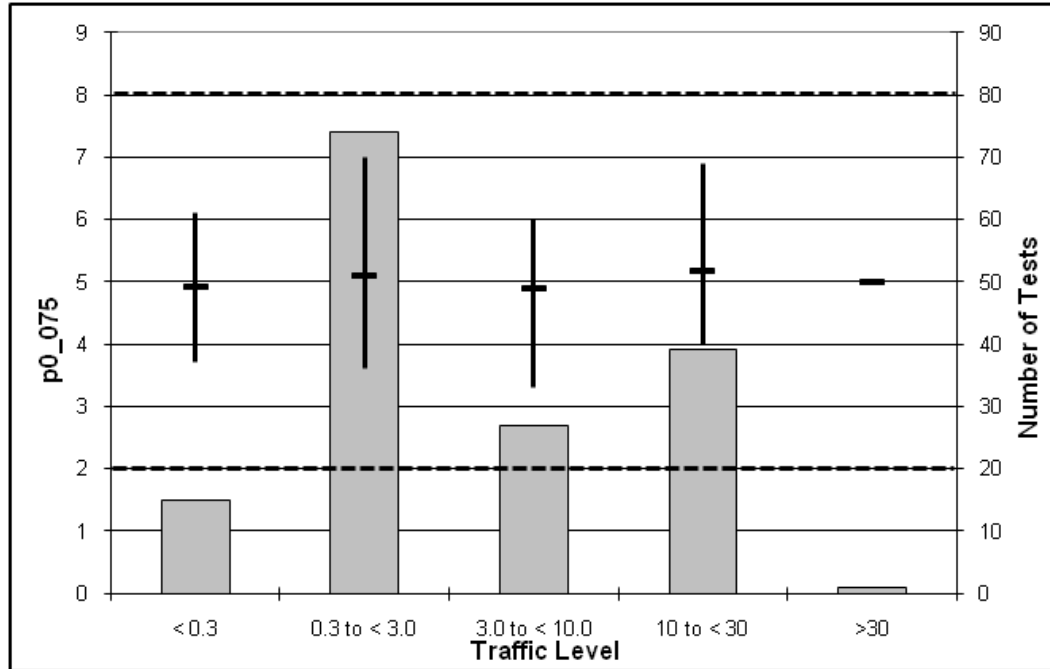
(d) Dust to effective binder ratio D/P_{be} .



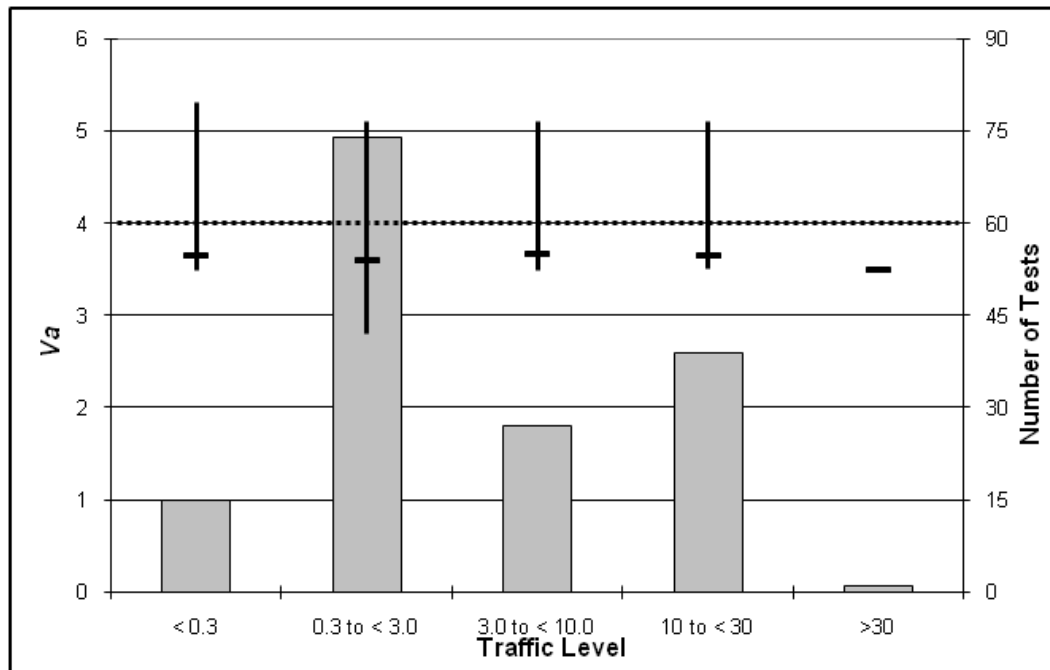
(e) Percent passing 4.75mm sieve size.



(f) Percent passing 2.36mm sieve size. Specific limits are 23 and 49.



(g) Percent passing 0.075mm sieve size. The specific limits are 2 and 8.



(h) Air voids V_a . Target value is 4.

Figure 5. High/low/average/volume plots for PG 64-22 19mm dense graded mixture. Data include all traffic volume categories.

The large amount of HMA mixture property data provided by SHA can be used to develop Maryland-specific average values for use as Level 3 inputs in the MEPDG. In

order to develop these average properties, however, the appropriate level of data aggregation must be determined.

Clearly, mixture gradation, and possibly volumetric properties, will be direct functions of nominal maximum aggregate size (NMAS, termed “band” in the SHA data set). Gradation and volumetric properties will also be functions of mix type (e.g., dense vs. gap graded). However, volumetric properties might also vary significantly with respect to other categorizations such as binder grade and/or traffic. Although Figure 4 and Figure 5 suggest that there were no consistent overall trends in the mean values for the gradation and volumetric properties either with regard to binder grade or traffic level, a correlation analysis was conducted to examine more thoroughly whether volumetric properties are functions of binder grade or traffic.

The number of mixtures in the MDSHA database corresponding to each mixture type is summarized in Table 9. Since there are many different mixture types, only a representative subset was considered for the correlation analyses. These, indicated in bold italic font in Table 9, were selected to provide a range of mix size and type subsets having large numbers of data points for statistical validity.

In order for the correlation results to be credible, there must be a reasonable distribution of binder grades and traffic levels in each analysis data set. As shown in Figure 6 and Figure 7, this was achieved in most of the data sets. The results from the correlation analyses for the selected mixture types are summarized in

Table 10 through Table 14. The binder grades and traffic Levels corresponding to the binder and traffic code columns are defined in Table 15. The following observations can be drawn from these results:

- The volumetric properties are insensitive to binder grade. Only four correlation coefficients were greater than 0.2. The largest coefficient was 0.47 for the correlation of binder grade and

traffic for 12.5 mm mixtures. This simply reflects the fact that MDSHA uses stiffer binders for both dense and gap graded surface mixes on high volume roadways.

- The volumetric properties are insensitive to traffic Level. Eight correlation coefficients were greater than 0.2 but none exceeded 0.35.

Based on these findings, it was determined that grouping mixtures by NMAS and mix type is sufficient for determining average Level 3 input properties. A built-in query was implemented in the *MatProp* database to determine these average values.

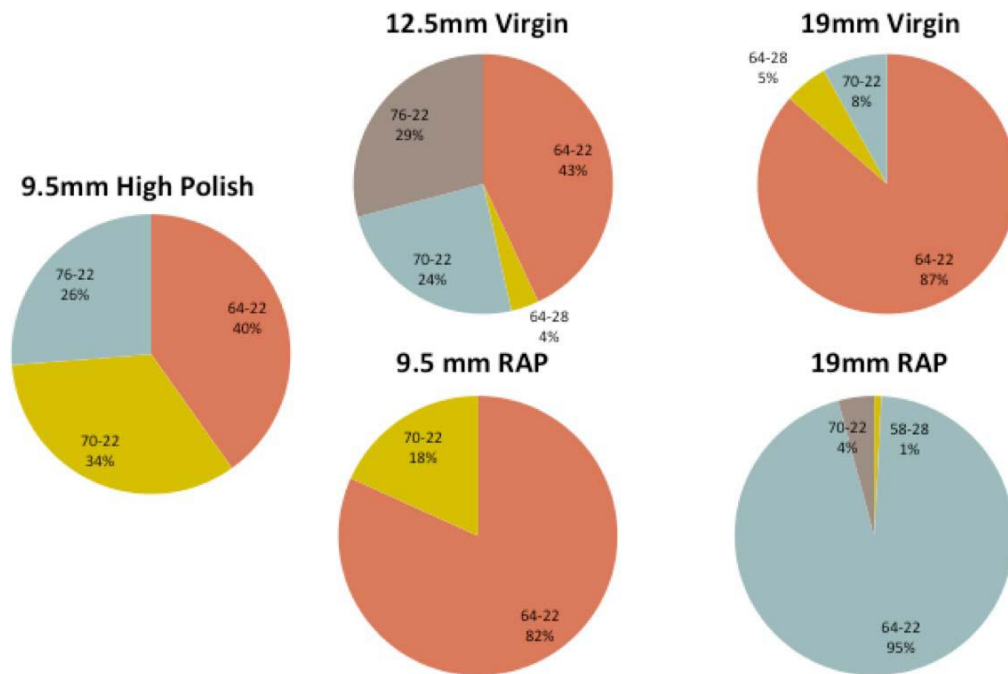


Figure 6. Distribution of binder grades for mixture data sets in correlation analyses.

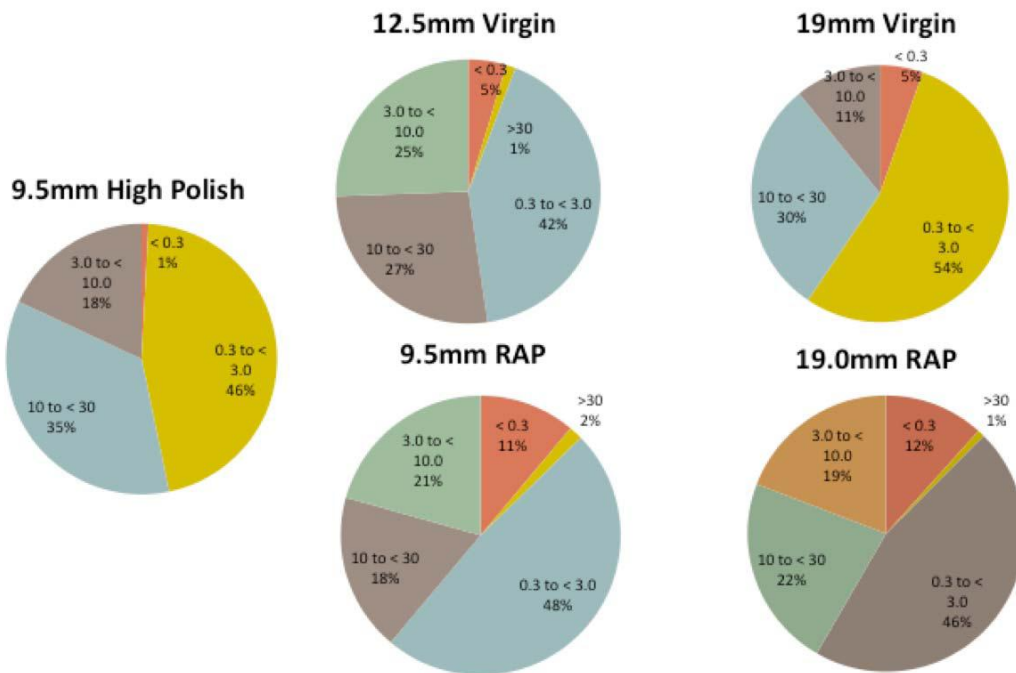


Figure 7. Distribution of traffic Levels for mixture data sets in correlation analyses.

Table 10. Correlation analysis result for 9.5mm high polish mixes.

	BG Code Traffic Code	
BG Code	1.00	
Traffic Code	0.16	1.00
Gmm	0.23	0.04
Gmb	-0.07	-0.10
Gse	0.22	0.00
Pb	-0.12	-0.19
Pba	0.09	-0.03
Pbe	-0.18	-0.11
Va	0.11	0.12
Vma	0.10	0.11
Vfa	-0.14	-0.15
D/Pbe Ratio	-0.15	0.01
D/B Ratio	-0.16	-0.03

Table 11. Correlation analysis result for 12.5mm virgin mixes.

	BG Code	Traffic Code
BG Code	1.00	
Traffic Code	0.47	1.00
Gmm	0.19	0.15
Gmb	0.16	-0.11
Gse	0.16	0.17
Pb	-0.20	0.00
Pba	-0.07	0.01
Pbe	-0.14	-0.01
Va	-0.12	0.15
Vma	-0.12	0.15
Vfa	0.09	-0.14
D/Pbe Ratio	0.02	-0.05
D/B Ratio	-0.07	-0.09

Table 12. Correlation analysis for 19.0mm virgin mixes.

	BG Code	Traffic Code
BG Code	1.00	
Traffic Code	-0.05	1.00
Gmm	0.14	-0.10
Gmb	0.13	-0.09
Gse	0.14	-0.19
Pb	0.00	-0.26
Pba	-0.12	0.03
Pbe	0.09	-0.29
Va	0.11	-0.13
Vma	0.14	-0.32
Vfa	0.03	-0.18
D/Pbe Ratio	-0.03	0.01
D/B Ratio	-0.23	0.06

Table 13. Correlation analysis for 9.5mm RAP mixes.

	BG Code	Traffic Code
BG Code	1.00	
Traffic Code	0.24	1.00
Gmm	0.07	0.01
Gmb	0.09	0.02

Gse	0.09	0.00
Pb	0.08	-0.04
Pba	0.21	0.15
Pbe	-0.17	-0.19
Va	-0.08	-0.04
Vma	-0.08	-0.24
Vfa	-0.02	-0.20
D/Pbe Ratio	-0.19	-0.06
D/B Ratio	0.04	0.31

Table 14. Correlation analysis for 19.0mm RAP mixes.

	BG Code	Traffic Code
BG Code	1.00	
Traffic Code	0.11	1.00
Gmm	0.15	0.15
Gmb	0.13	0.13
Gse	0.16	0.09
Pb	0.00	-0.30
Pba	0.00	-0.13
Pbe	0.00	-0.18
Va	0.09	0.10
Vma	0.06	0.00
Vfa	-0.01	-0.01
D/Pbe Ratio	0.00	0.22
D/B Ratio	-0.01	0.21

Table 15. Definitions of binder and traffic codes.

Binder Code	Binder Grade	Traffic (MESALs)
0	PG 58-22	N/A
1	PG 58-28	<0.3
2	PG 64-22	0.3 to <3
3	PG 64-28	3 to < 10
4	PG 70-22	10 to < 30
5	PG 76-22	>30

Level 1 vs. Level 2 vs. Level 3 Dynamic Modulus

The Maryland SHA has not to date collected any Level 1 property data for any of its HMA mixtures. The SHA laboratories contain an Asphalt Mixture Performance Tester (AMPT) and a UTM-25 general purpose test system, both of which could be employed for measuring Level 1 dynamic modulus, creep compliance, and low temperature tensile strength properties. The question is whether there is a compelling reason to perform this testing.

Level 1 testing by SHA for creep compliance, low temperature tensile strength, and aggregate coefficient of thermal contraction is not recommended. These properties are used only for thermal cracking prediction, which is not a major problem in Maryland except perhaps for the western mountains in Garrett, Allegany, and Washington counties. The MEPDG generally does not predict any significant thermal cracking in Maryland provided an appropriate binder grade is specified. Given this, the Level 3 relationships built into the MEPDG code for converting dynamic modulus and other mixture properties to creep compliance, low temperature tensile strength, and aggregate coefficient of thermal contraction are judged sufficient for Maryland purposes.

The recommendation for Level 1 dynamic modulus testing is different, however. Past studies using earlier versions of the MEPDG code have found significant differences in predicted performance using Level 1 vs. Level 3 dynamic modulus data (Azari *et al.* 2008) and some inability of Level 2/3 inputs to differentiate between different mixes adequately (e.g., Flintsch *et al.*, 2008; Ceylan *et al.*, 2009) A limited comparison analysis was conducted to confirm these general findings using the current version of the MEPDG

software for Maryland conditions. The analysis scenario was a simple pavement section consisting of 10 inches of HMA (19mm dense graded, PG 76-22) over 20 inches of crushed stone base over subgrade (A-7-5). The HMA was based on the control mixture at the FHWA ALF test, which uses aggregates and binders similar to those commonly used for dense graded mixtures in Maryland. The Level 1 dynamic modulus test data was extracted from the FHWA ALF research reports. Level 3 defaults were assumed for all other material properties. Traffic was set at 950 trucks per day in the design lane and Baltimore (BWI) weather history (interpolated with DC and IAD) was taken as the climate input. Reliability was set at the MEPDG 90% default level for all distresses.

The distresses predicted by the MEPDG using Level 1 vs. Level 2 vs. Level 3 dynamic modulus inputs for this scenario are summarized in Figure 8. The predicted rutting for the HMA layer is slightly larger for the Level 2 and 3 inputs than for the Level 1 value. However, the predicted total rutting using Level 2 and 3 inputs is significantly larger than when using the Level 1 inputs. Although this comparison is extremely limited (i.e., just one mixture, albeit of a type commonly used in Maryland), the findings are broadly comparable with those by others.

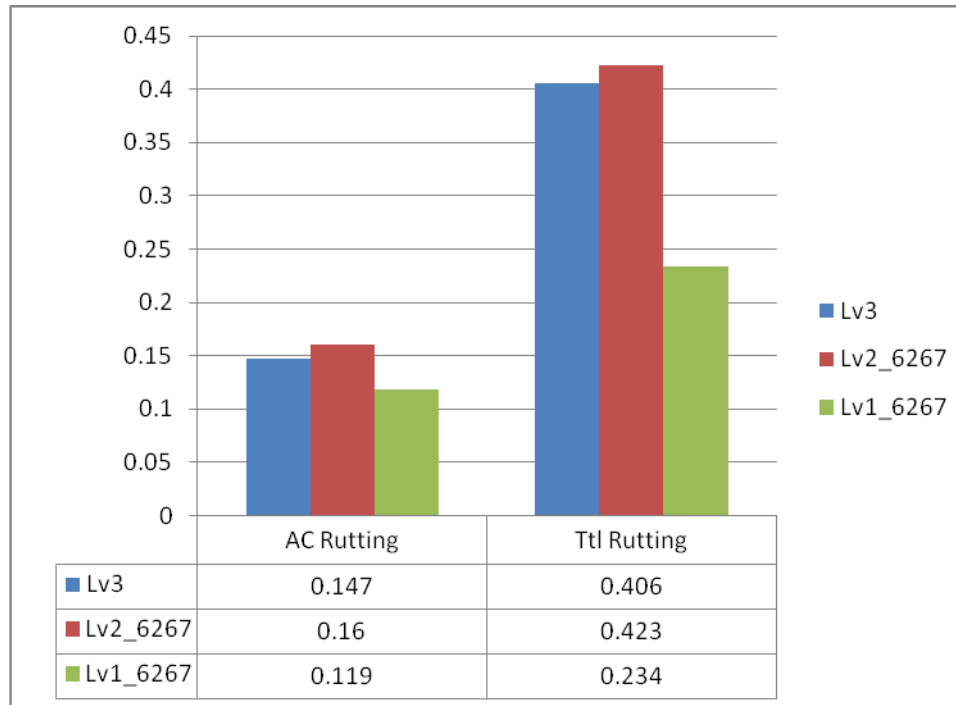


Figure 8. Comparison of predicted rutting using Level 1 vs. Level 2 vs. Level 3 HMA dynamic modulus data.

Past investigations have consistently found that the Witczak predictive model used for Level 3 dynamic modulus inputs is dominated by temperature influences and does not do a good job of ranking mixtures in terms of their measured stiffness values at a given temperature and loading frequency (e.g., Ceylan *et al.* 2009). In addition, the databases used to develop and calibrate the Witczak and other similar dynamic modulus predictive models contain very few gap graded SMA mixtures of the type commonly used on high volume roads in Maryland.

Given all of these issues, it is recommended that SHA begin a program of measuring Level 1 dynamic modulus data for its more commonly used mixtures. It is envisioned that this could be done as part of the project design and/or quality assurance activities. The testing, which could be done in-house in the SHA laboratories or outsourced to commercial and/or University testing facilities, should focus on larger and/or more important projects employing mixtures having the largest tonnage

production in Maryland or being placed on the highest traffic volume roadways. If this type of testing regimen were adopted as routine for large/important paving projects, SHA could amass a large body of Maryland specific Level 1 dynamic modulus data in a relatively short period of time.

Note that this recommendation for Level 1 dynamic modulus testing of the HMA mixture implicitly includes Level 1 testing of the binder as well. Although it was previously concluded in Chapter 2 that Level 3 vs. Level 1/2 binder property data had little effect on predicted performance (when coupled with Level 2/3 predicted dynamic modulus), Level 1 stiffness data for the binder must be entered into the MEPDG if Level 1 dynamic modulus data is entered for the mixture. The Level 1 binder data is used by the global aging model in the MEPDG along with the Level 1 mixture data.

Thermal Properties

The MEPDG requires input values for the HMA thermal conductivity, heat capacity and the surface shortwave absorptivity (SSA). HMA thermal conductivity is the capability of HMA material to transmit heat, heat capacity is the capability of a HMA material to store heat, and SSA is the capability of HMA surface to absorb solar thermal radiation. These HMA thermal properties are expected to have significant effects on pavement performance. These properties are not commonly measured in the laboratory, and literature data on typical values are sparse. The MEPDG recommends values in the range of about 0.4 to 0.8 BTU/hr-ft-°F for HMA thermal conductivity, 0.2 to 0.4 BTU/lb-°F for HMA heat capacity, and 0.8 to 1.0 (dimensionless) for SSA. The basis for these recommended ranges is not known, but the ranges are reasonably narrow.

In order to evaluate whether more effort needs to be devoted to better quantify these properties, a limited sensitivity study was conducted to evaluate the impact of HMA thermal conductivity and heat capacity on pavement performance (Schwartz and Li, 2010). Typical pavement sections were evaluated for College Park MD climate conditions as well as for Seattle WA, Caribou ME, and Phoenix AZ in order to evaluate more extreme climate cases. Sensitivity of performance to material inputs was quantified using the following normalized index S_{ji}

$$S_{ji} = \frac{\partial D_j}{\partial X_i} \left(\frac{X_{iR}}{D_{jR}} \right) \quad (1)$$

which quantifies the variation of distress magnitude D_j about some baseline reference condition D_{jR} caused by varying an analysis input X_i around its reference condition X_{iR} . The normalized sensitivity index S_{ji} can be interpreted as the percentage change in distress D_j caused by a given percentage change in input X_i .

Figure 9 provides an overall summary of the normalized sensitivity indices as averaged (in absolute value terms) across all distresses. As shown, SSA has nearly the same normalized influence on overall performance at all four sites as does subgrade stiffness—i.e., a very high sensitivity. However, the normalized sensitivity indices for HMA thermal conductivity and heat capacity are about an order of magnitude lower than those for SSA.

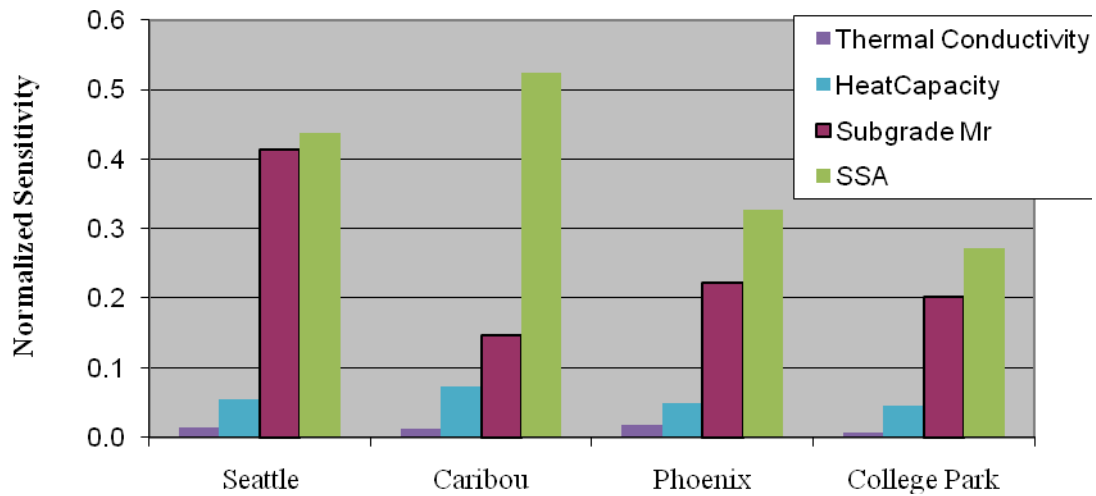


Figure 9. Average normalized sensitivity indices for thermal conductivity, heat capacity, surface shortwave absorptivity and subgrade modulus across all distresses. Legend entries from top to bottom correspond to vertical bars from left to right for each location.

Figure 10 through Figure 12 summarized the normalized sensitivity indices by distress for SSA, HMA thermal conductivity and HMA heat capacity. On the horizontal axis of these figures “LT Crk” means longitudinal cracking, “All Crk” means alligator cracking, “SG RD” means subgrade rutting, “GB RD” means granular base rutting, “AC RD” means asphalt concrete rutting, “Total RD” means total rutting and IRI means international roughness index. SSA (Figure 10) has the largest influence on HMA rutting and, by extension, on total rutting. It has moderate influence on longitudinal and transverse cracking, alligator fatigue cracking, and granular base rutting, most likely due to the differences in temperature and thus stiffness/load spreading ability of the HMA layer at different SSA values.

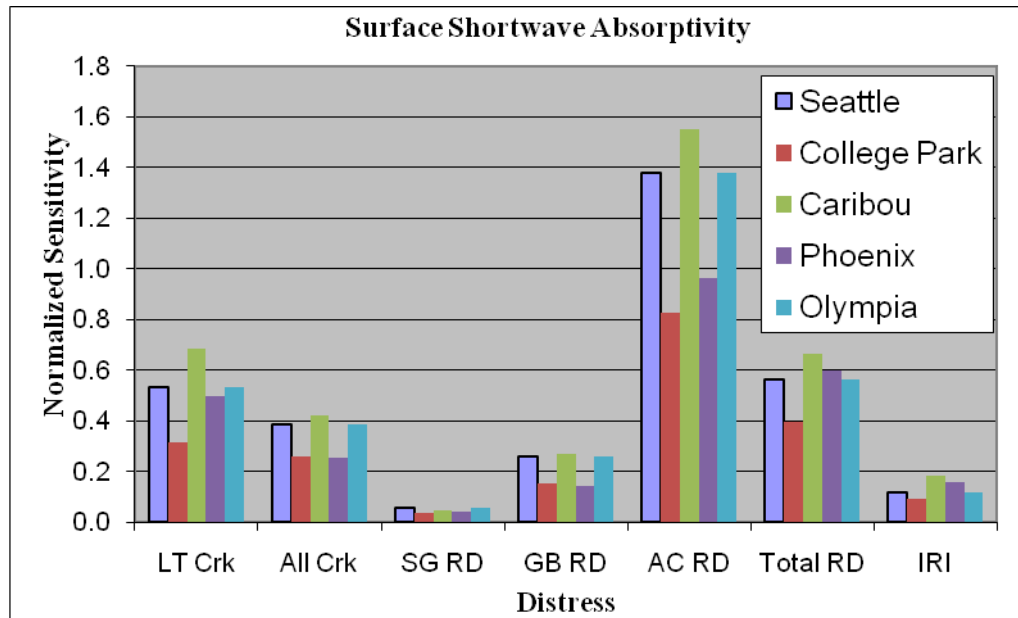


Figure 10. Normalized sensitivity indices by distress for SSA. Legend entries from top to bottom correspond to vertical bars from left to right for each location.

HMA thermal conductivity (Figure 11) has a negligible influence on all distresses. HMA heat capacity (Figure 12) also has a small but somewhat larger influence. Interestingly, the largest effect of heat capacity is on subgrade and base rutting, with rutting in these materials decreasing as HMA heat capacity increases. The mechanism for this is unclear. Increased heat capacity increases the thermal inertia of the HMA layer and thus smoothes out some of the temperature fluctuations in the layers, which may play a role in reducing the rutting in the unbound layers.

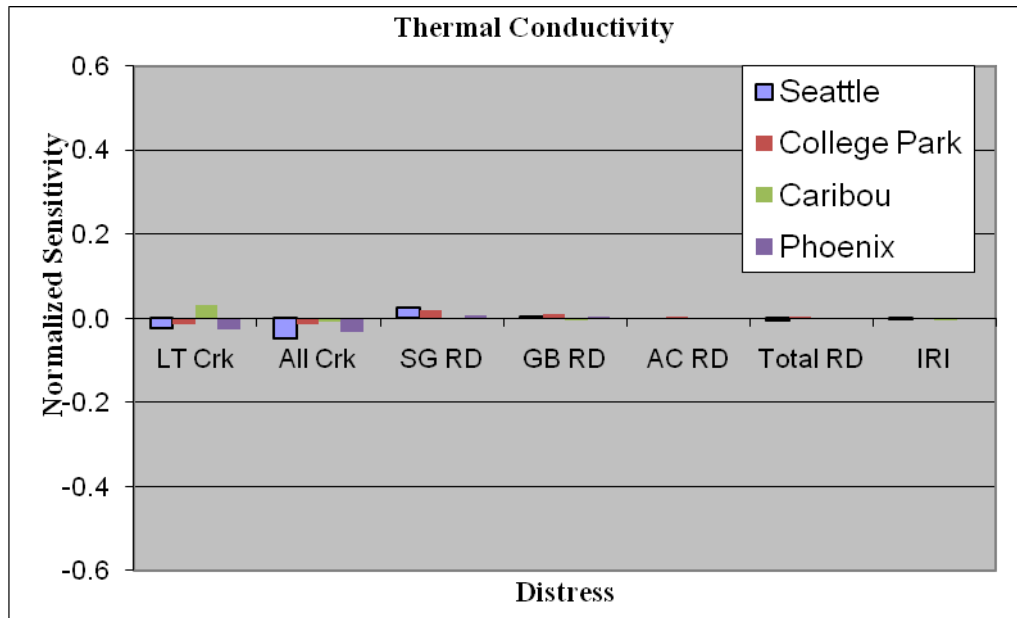


Figure 11. Normalized sensitivity indices by distress for thermal conductivity. Legend entries from top to bottom correspond to vertical bars from left to right for each distress.

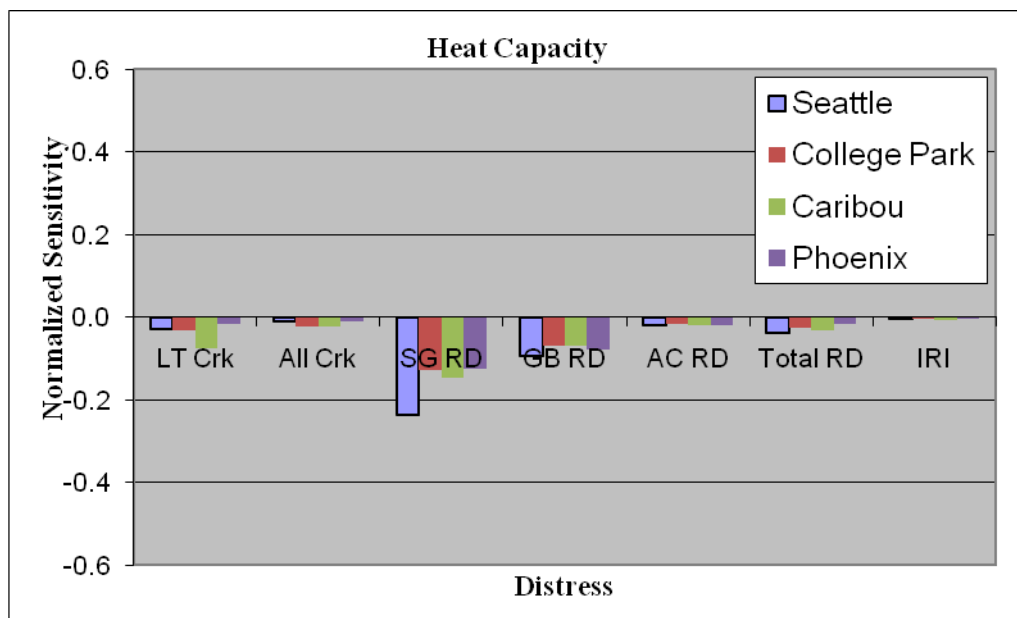


Figure 12. Normalized sensitivity indices by heat capacity. Legend entries from top to bottom correspond to vertical bars from left to right for each distress.

The overall conclusion that SSA is the only environment-related HMA material input showing a strong impact on predicted performance. HMA thermal conductivity and heat capacity all show normalized sensitivity indices (averaged across all distresses in

absolute value terms) an order of magnitude lower than SSA. SSA is not an easily measured parameter, however, and it changes significantly over the pavement life (e.g., as the asphalt surface bleaches and lightens with time). Consequently, there is no good alternative to using the MEPDG Level 3 default values for this input.

Summary

Testing Recommendations

The principal findings and recommendations relevant to HMA material property testing by SHA are as follows:

1. The HMA mixture information provided by SHA is limited to volumetric and gradation data suitable for Level 3 input to the MEPDG. No measured dynamic modulus values suitable for Level 1 inputs are available. There is the potential for significant differences in predicted performance using Level 1 vs. Level 2/3 dynamic modulus data. In addition, the Witzak predictive equation used to generate the Level 2/3 dynamic modulus data is not intended for SMA mixtures, a common premium mixture type in Maryland, and often does not differentiate among different dense graded mixtures adequately. Therefore, SHA should plan to begin measuring Level 1 dynamic modulus data over time for the most commonly used mixture types in conjunction with major paving projects. Level 1 dynamic modulus testing of HMA mixtures will also require companion Level 1 characterization of the asphalt binders.

It is recommended that SHA develop a policy requiring Level 1 HMA dynamic modulus and binder characterization testing for all major projects. Major projects could be defined by SHA in terms of a minimum placement tonnage, minimum traffic volume, or some other measure of project/mix importance. This testing could be done in-house using either the UTM-25 or AMPT test systems in the SHA laboratories; however, some equipment repair and/or

- calibration would be required as both of these systems are currently nonoperational. This testing could also be outsourced to local commercial testing facilities (e.g., Advanced Asphalt Technologies, LLC) and/or the University of Maryland (HMA dynamic modulus testing only).
2. There is no perceived need for measuring Level 1 creep compliance, low temperature tensile strength, and aggregate coefficient of thermal contraction properties. These properties are used only for predictions of thermal cracking, which is not a major distress type in Maryland. The Level 3 relationships built into the MEPDG code for converting dynamic modulus and other mixture inputs to creep compliance, low temperature tensile strength, and aggregate coefficient of thermal contraction are judged as sufficient for Maryland purposes.
 3. HMA thermal conductivity and heat capacity generally have a very slight influence on pavement performance predicted by the MEPDG. Consequently, the Level 3 default values built into the MEPDG software are sufficient and laboratory measurement of these properties is not warranted.

Although SSA has a much more significant influence on predicted performance, there at present is no easy method for measuring this parameter, either initially after construction or over the pavement life. Therefore, the Level 3 defaults values built into the MEPDG software should be used.

Recommended MEPDG Inputs

The recommended HMA dynamic modulus and asphalt inputs to the MEPDG for Maryland conditions are summarized in Table 16 and Table 17 for new and existing HMA layers, respectively. Table 18 summarizes the recommendations for creep compliance and low temperature tensile strength inputs for both new and existing HMA layers.

Table 16. Recommendation material property inputs for new HMA layers for Maryland conditions.

Property	Input Level	Value	Comment
Asphalt material type	All	Asphalt concrete	Only option available.
Layer thickness	All	Project specific	
<i>Asphalt Mix</i>			
Dynamic Modulus Table	1	Mixture specific	Recommended for future collection.
Aggregate gradation and volumetric properties ¹	2/3	Mixture specific	See Table 19 for typical values for Maryland mixtures.
<i>Asphalt Binder</i>			
Superpave binder dynamic stiffness data	1/2	Binder specific	Recommended for future collection.
Superpave binder grade	3	Mixture specific	See Table 4 for Maryland SHA binder recommendations.
<i>Asphalt General</i>			
Reference temperature	All	70	Does not influence predictions.
Effective binder content	All	Mixture specific	See Table 19 for typical values for Maryland mixtures.
In-Place Air Voids	All	Project specific	See Table 19 for typical values for Maryland mixtures.
Total unit weight	All	Project specific	See Table 20 for typical values for Maryland mixtures. (Note: Values in table should be adjusted for in-place air voids percentage.)
Poisson's ratio	All	0.35	MEPDG default
Thermal conductivity	All	0.67	MEPDG default (global calibration value).
Heat capacity asphalt	All	0.23	MEPDG default (global calibration value).
Short wave absorption	All	0.85	MEPDG default (global calibration value).

¹Percent retained above the 3/4" sieve; percent retained above the 3/8" sieve; percent retained above the #4 sieve; percent passing the #200 sieve; effective volumetric binder content (%); and in-place air voids (%).

Table 17. Recommendation material property inputs for existing HMA layers for Maryland conditions.

Property	Input Level	Value	Comment
Asphalt material type	All	Asphalt concrete	
Layer thickness	All	Project specific	
<i>Asphalt Mix</i>			
Aggregate gradation and volumetric properties ¹	All	Mixture specific	See Table 19 for typical values for Maryland mixtures.
Superpave binder dynamic stiffness data	1/2	Binder specific	Recommended for future collection.

Superpave binder grade	3	Mixture specific	See Table 4 for Maryland SHA binder recommendations.
<i>Asphalt General</i>			
Reference temperature	All	70	Does not influence predictions.
Effective binder content	All	Mixture specific	See Table 19 for typical values for Maryland mixtures.
In-Place Air Voids	All	Project specific	See Table 19 for typical values for Maryland mixtures.
Total unit weight	All	Project specific	
Poisson's ratio	All	0.35	MEPDG default
Thermal conductivity	All	0.67	MEPDG default (global calibration value).
Heat capacity asphalt	All	0.23	MEPDG default (global calibration value).
Pavement condition rating	All	Project specific	

¹Percent retained above the 3/4" sieve; percent retained above the 3/8" sieve; percent retained above the #4 sieve; percent passing the #200 sieve; effective volumetric binder content (%); and in-place air voids (%).

Table 18. Recommendation thermal cracking inputs for new HMA layers for Maryland conditions.

Property	Input Level	Value	Comment
Average tensile strength at 14 °C	3	Mixture specific	MEPDG calculated value
Creep compliance	3	Mixture specific	MEPDG calculated value
Mixture coefficient of thermal contraction	3	Mixture specific	MEPDG calculated value
Aggregate coefficient of thermal contraction	3	Project specific	See Table 6

Table 19. Level 3 inputs for Maryland HMA mixtures (based on material properties database at time of report).

NMAS (mm)	Mix Type	% Retained above 3/4" sieve	% Retained above 3/8" sieve	% Retained above #4 sieve	% Passing #200 sieve	Effective Volumetric Binder Content (%)	In-Place Air Voids (%)
4.75	Virgin	0.0	0.0	6.9	7.7	14.06	6.54
9.5	Shingle	0.0	4.8	39.2	6.4	11.61	6.47
9.5	RAP	0.0	4.7	38.1	5.5	11.59	6.47
9.5	Virgin	0.0	3.7	34.5	6.0	11.88	6.47
9.5	GAP	0.0	10.7	61.3	9.2	14.85	6.47
9.5	High Polish	0.0	3.4	36.7	5.5	11.76	6.47
12.5	High	0.0	14.0	49.1	5.1	11.09	6.47

	Polish						
12.5	RAP	0.0	13.0	50.0	5.4	10.70	6.47
12.5	Shingle	0.0	14.2	50.7	6.1	10.73	6.47
12.5	Virgin	0.0	15.6	45.3	5.3	11.14	6.47
12.5	GAP	0.0	21.7	66.8	8.6	14.31	6.47
19	GAP	5.0	44.0	74.0	8.1	13.84	6.47
19	RAP	3.5	26.6	57.4	5.0	9.69	6.47
19	Shingle	4.1	29.5	58.8	5.6	9.72	6.47
19	High Polish	2.5	30.4	58.3	5.3	10.20	6.47
19	Virgin	5.5	33.5	55.5	4.9	10.08	6.47
25	RAP	11.7	40.6	65.2	4.7	9.10	6.47
25	Virgin	15.0	47.0	63.5	4.2	9.46	6.47
37.5	RAP	23.0	52.2	70.8	4.4	8.38	6.47
4.75	Virgin	0.0	0.0	6.9	7.7	14.06	6.54

Table 20. SHA historical unit weights for Superpave mixes at 4% air voids (OMT, 2006).

Material	Average Unit Weight (lbs/ft³)
4.75 mm	153.2
9.5 mm	147.5
12.5 mm	148.5
19.0 mm	149.9
25.0 mm	150.9
12.5 mm Gap Graded	152.1
19.0 mm Gap Graded	150.2
Non GG Surface Mixes	149.7
Base Mixes	150.4
All Mixes	150.3

CHAPTER 4: PCC DATA

MEPDG Input Requirements

New Construction/Reconstruction/Overlays

The key project-specific PCC stiffness and strength properties required for new construction/reconstruction/overlay designs in the MEPDG are the elastic modulus (E_c) and the modulus of rupture (MOR). The methods for determining these properties at each of the input levels in the MEPDG are summarized in Table 21 and Table 22. The corresponding required user inputs at each level are:

- Level 1: E_c and MOR at 7, 14, 28, and 90 days and the estimated ratio of 20 year to 28 day values.
- Level 2: Compressive strength (f'_c) at 7, 14, 28 and 90 days and the estimated ratio of 20 year to 28 day values. The corresponding E_c and MOR values are estimated using the standard empirical relationships shown in Table 21 and Table 22.
- Level 3: Either the 28-day MOR or the 28-day f'_c . The corresponding 28-day E_c modulus is then either estimated by the MEPDG software using the standard empirical relationship shown in Table 21 or optionally provided by the user. The values of E_c and MOR are determined at other time values using the default aging relationships shown in Table 21 and Table 22, respectively.

Additional PCC properties required at all input levels include:

- Mix properties: Unit weight; Poisson's ratio; cement type; cementitious material content; water cement ratio; aggregate type; curing method.

- Thermal properties: thermal conductivity; heat capacity; surface shortwave absorptivity; coefficient of thermal expansion; PCC zero-stress temperature
- Shrinkage properties: ultimate shrinkage at 40% relative humidity; reversible shrinkage; time to develop 50% of ultimate shrinkage.

The methods for determining thermal conductivity, heat capacity, and surface shortwave absorptivity at each of the input levels in the MEPDG are summarized in

Table 23. Coefficient of thermal expansion can be measured using AASHTO TP60 (Level 1), approximated using mixture theory (Level 2), or estimated from historical values (Level 3). Default values are provided in the MEPDG software for all of these additional PCC properties. These default values may be overridden by the user if desired.

Table 21. PCC elastic modulus estimation for new, reconstruction, and overlay design (NCHRP, 2004).

Material Group	Type of Design	Input Level	Description
PCC (Slabs)	New	1	<ul style="list-style-type: none"> PCC modulus of elasticity, E_c, will be determined directly by laboratory testing. This is a chord modulus obtained from ASTM C 469 at various ages (7, 14, 28, 90-days). Estimate the 20-year to 28-day (long-term) elastic modulus ratio. Develop modulus gain curve using the test data and long-term modulus ratio to predict E_c at any time over the design life.
		2	<ul style="list-style-type: none"> PCC modulus of elasticity, E_c, will be determined indirectly from compressive strength testing at various ages (7, 14, 28, and 90 days). The recommended test to determine f'_c is AASHTO T22. The E_c can also be entered directly if desired. Estimate the 20-year to 28-day compressive strength ratio. Convert f'_c to E_c using the following relationship: $E_c = 33\rho^{3/2} (f'_c)^{1/2} \text{ psi}$ Develop modulus gain curve using the test data and long-term modulus ratio to predict E_c at any time over the design life.
		3	<ul style="list-style-type: none"> PCC modulus of elasticity, E_c, will be determined indirectly from 28-day estimates of flexural strength (MR) or f'_c. MR can be determined from testing (AASHTO T97) or from historical records. Likewise, f'_c can be estimated from testing (AASHTO T22) or from historical records. The E_c can also be entered directly. If 28-day MR is estimated, its value at any given time, t, is determined using: $MR(t) = (1 + \log_{10}(t/0.0767) - 0.01566 * \log_{10}(t/0.0767)^2) * MR_{28\text{-day}}$ Estimate $E_c(t)$ by first estimating $f'_c(t)$ from MR(t) and then converting $f'_c(t)$ to $E_c(t)$ using the following relationships: $f'_c = (MR/9.5)^2 \text{ psi}$ $E_c = 33\rho^{3/2} (f'_c)^{1/2} \text{ psi}$ If 28-day f'_c is estimated, first convert it to an MR value using equation above and then project MR(t) as noted above and from it $E_c(t)$ over time.

Table 22. PCC modulus of rupture estimation for new or reconstruction design and PCC overlay design (NCHRP, 2004).

Material Group Category	Type Design	Input Level	Description
PCC (Slabs)	New	1	<ul style="list-style-type: none"> PCC MR will be determined directly by laboratory testing using the AASHTO T97 protocol at various ages (7, 14, 28, 90-days). Estimate the 20-year to 28-day (long-term) MR ratio. Develop strength gain curve using the test data and long-term strength ratio to predict MR at any time over the design life.
		2	<ul style="list-style-type: none"> PCC MR will be determined indirectly from compressive strength testing at various ages (7, 14, 28, and 90 days). The recommended test to determine f'_c is AASHTO T22. Estimate the 20-year to 28-day compressive strength ratio. Develop compressive strength gain curve using the test data and long-term strength ratio to predict f'_c at any time over the design life. Estimate MR from f'_c at any given time using the following relationship: $MR = 9.5 * (f'_c)^{1/2} \quad \text{psi}$
		3	<ul style="list-style-type: none"> PCC flexural strength gain over time will be determined from 28-day estimates of MR or f'_c. If MR is estimated, use the equation below to determine the strength ratios over the pavement design life. The actual strength values can be determined by multiplying the strength ratio with the 28-day MR estimate. $F_STRRATIO = 1.0 + 0.12\log_{10}(AGE/0.0767) - 0.01566[\log_{10}(AGE/0.0767)]^2$ If f'_c is estimated, convert f'_c to MR using equation 2.2.31 and then use the equation above to estimate flexural strength at any given pavement age of interest.

Table 23. Estimation of PCC thermal conductivity, heat capacity, and surface absorptivity at various hierarchical input levels (NCHRP, 2004).

Input Level	Required Properties	Options for Input Estimation					
1	Thermal conductivity	Estimate using laboratory testing in accordance with ASTM E 1952.					
	Heat capacity	Estimate using laboratory testing in accordance with ASTM D 2766.					
	Surface short wave absorptivity	Laboratory estimation is recommended ¹ .					
2	Thermal conductivity	Same as level 1					
	Heat capacity						
	Surface short wave absorptivity						
3	Thermal conductivity	Reasonable values range from 1.0 to 1.5 Btu/(ft)(hr)(°F). A typical value of 1.25 Btu/(ft)(hr)(°F) can be used for design.					
	Heat capacity	Reasonable values range from 0.2 to 0.28 Btu/(lb)(°F). A typical value of 0.28 Btu/(lb)(°F) can be used for design.					
	Surface short wave absorptivity	However, default property values are available for user convenience: <table style="margin-left: 20px; border: none;"> <tr> <td>Fresh snow cover</td> <td>0.05 – 0.25</td> </tr> <tr> <td>Old snow cover</td> <td>0.30 – 0.60</td> </tr> <tr> <td>PCC pavement</td> <td>0.70 – 0.90</td> </tr> </table> <p>A typical value of 0.85 can be used for PCC pavements.</p>	Fresh snow cover	0.05 – 0.25	Old snow cover	0.30 – 0.60	PCC pavement
Fresh snow cover	0.05 – 0.25						
Old snow cover	0.30 – 0.60						
PCC pavement	0.70 – 0.90						

¹ Currently, there are no available AASHTO or ASTM procedures to estimate these quantities for concrete materials. Other protocols may be used as appropriate.

Rehabilitation

There are two primary differences between characterizing new concrete layers and existing layers: (a) the E_c and MOR values for existing PCC slabs to be overlaid need to be adjusted for the damage caused to the pavement by traffic loads and environmental effects; (b) gains in E_c and MOR over time are not considered for the old existing PCC. The material properties required by the MEPDG at each input level for existing PCC in rehabilitation projects are as follows:

- Level 1:
 - Elastic modulus E_{TEST} is measured from cores taken from the existing pavement in accordance with ASTM C 46. Alternatively, E_{TEST} can be determined via FWD nondestructive evaluation at mid-slab. E_{TEST} is then adjusted for pavement condition to determine the E_c value of the existing

pavement to be used in design:

$$E_c = C * E_{TEST} \quad (2)$$

in which C is the pavement condition factor given in Table 24.

- In-place MOR is measured from prismatic beams cut from the existing concrete pavement in accordance with AASHTO T97.

- Level 2:
 - In-place f_c' is measured from cores taken from the existing pavement in accordance with AASHTO T22.
 - In-place f_c' is converted to E_{TEST} internally in the MEPDG software using the standard empirical relationship (see Table 21). E_{TEST} is then adjusted for pavement condition to determine the design E_c value using Eq. (2) and Table 24.
 - In-place f_c' is converted to MOR internally in the MEPDG software using the standard empirical relationship (see Table 22).

- Level 3:
 - The in-place E_c of the existing PCC as a function of pavement condition is estimated using the guidelines in Table 25.
 - Either 28-day MOR or f_c' is estimated based on past historical records or local experience; f_c' is converted to MOR internally in the MEPDG software using the standard empirical relationship (see Table 22).

Table 24. Recommended condition factor values used to adjust moduli of intact slabs (from NCHRP, 2004).

Qualitative Description of Pavement Condition ¹	Recommended Condition Factor, C
Good	0.42 to 0.75
Moderate	0.22 to 0.42
Severe	0.042 to 0.22

¹Table 2.5.15 in PART 2, Chapter 5 of NCHRP (2004) presents guidelines to assess pavement condition.

Table 25. Level 3 guidelines for in-place PCC elastic modulus (from NCHRP, 2004).

Qualitative Description of Pavement Condition ¹	Typical Modulus Ranges, psi
Adequate	3 to 4 x 10 ⁶
Marginal	1 to 3 x 10 ⁶
Inadequate	0.3 to 1 x 10 ⁶

¹Table 2.5.15 in PART 2, Chapter 5 of NCHRP (2004) presents guidelines to assess pavement condition.

PCC Data Summary

The only PCC properties available from SHA were 201 QC/QA data records from the Salisbury bypass project on the Eastern Shore of Maryland. These data are for a single Mix No. 7 design consisting of #57 limestone coarse aggregate, sand, 580 lb/cy cementitious material (377 lb/cy Type I cement plus 203 lb/cy ground iron blast furnace slag), and a design water-to-cement ratio of 0.44. Material properties included in the QC/QA records were the 28-day split cylinder tensile strength, slump, and water-to-cement ratio. Summaries of these properties are provided in Figure 13 through Figure 15.

Note that the material property data from the Salisbury Bypass project are insufficient for direct input to the MEPDG. However, the *MOR* can be estimated from the split cylinder tensile strength f_t using a standard empirical relationship (see, e.g., Papagiannakis and Masad, 2008):

$$MOR = 1.35 f_t \quad (3)$$

Using this relation and the data in Figure 13, the average *MOR* for the Salisbury bypass mix is estimated at 685 psi.

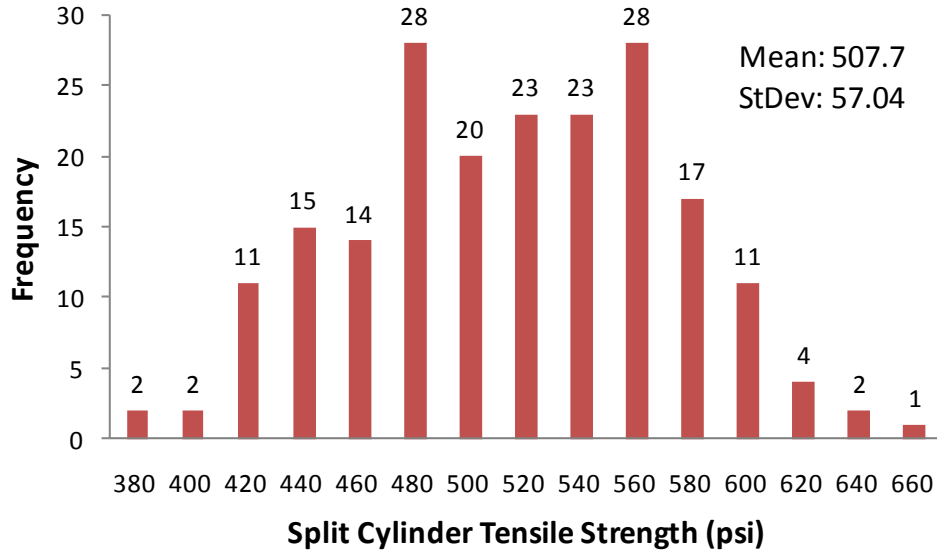


Figure 13. Summary of split cylinder tensile strength data provided by SHA.

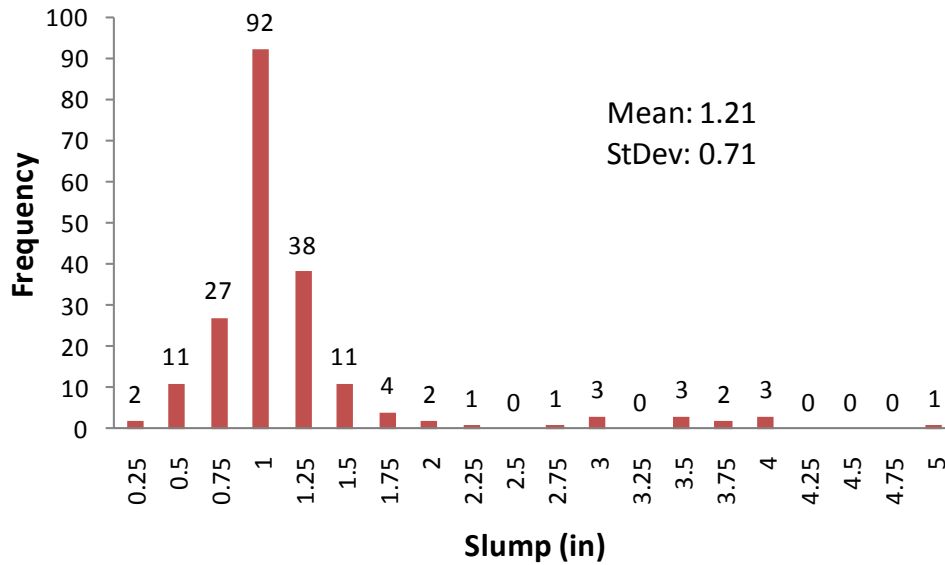


Figure 14. Summary of slump data provided by SHA.

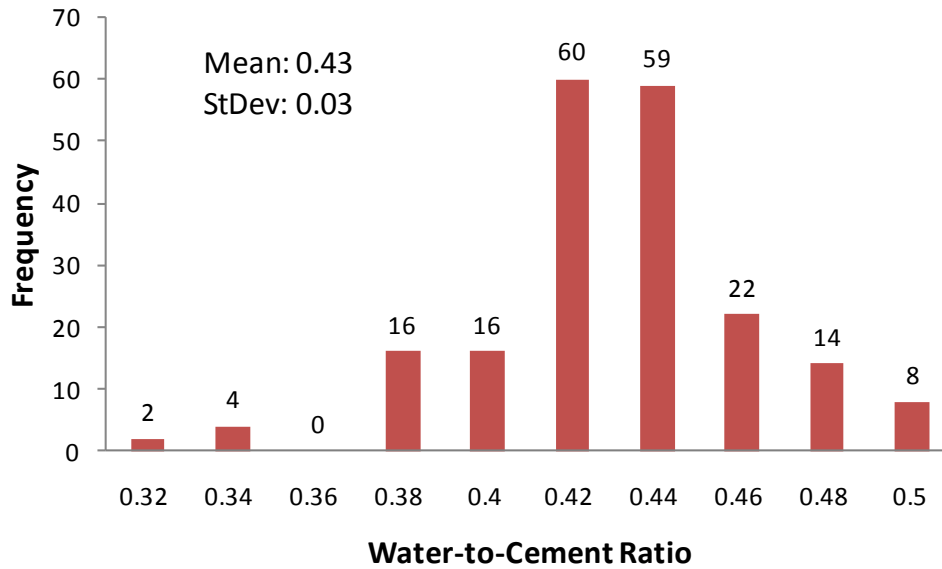


Figure 15. Summary of water-to-cement ratio data provided by SHA.

Sensitivity Analyses for PCC Inputs

Strength and Stiffness Properties

Effect of Input Level

The major question regarding the appropriate input level for PCC strength and stiffness data is: “Are there significant differences in predicted performance from the MEPDG using Level 1, 2 or 3 PCC material inputs?” A review of the literature found a few relevant studies, but they either did not definitively address the question or they were for climate conditions significantly different from Maryland. Therefore, a very limited comparison of MEPDG predictions using Level 1 vs. Level 2 vs. Level 3 PCC inputs for typical Maryland conditions was conducted.

Since no Level 1 PCC material properties were available from SHA, several alternatives for obtaining the required data were explored. In the first attempt, 28-day PCC compressive strength, 28-day modulus of rupture, and 28-day elastic modulus data collected by the University of Maryland as part of the Salisbury bypass project

were used along with the property vs. time relations incorporated in the MEPDG to estimate the missing Level 1 and Level 2 PCC properties. No significant differences were found among the Level 1, 2, and 3 predicted performances. However, this was expected and merely shows that PCC material inputs portion of the MEPDG software is bug-free and internally consistent.

The second attempt capitalized on a study by Hall and Beam (2005) that detailed Level 1, 2 and 3 PCC property data for a specific concrete mixture. Using these data, the MEPDG predicted performance again showed little difference between Level 1, Level 2, and Level 3 inputs. However, the Hall and Beam paper did not describe how the Level 2 and Level 3 data were obtained. It is quite likely that these data were generated using the same approach as our initial attempt with the Salisbury bypass data.

The third and best attempt was based on measured Level 1, 2, and 3 PCC material input data acquired from Missouri DOT for five PCC mixes (ARA, 2009). All five mixes used Type I cement, limestone coarse aggregate, and flyash¹. The mix composition data are described in Table 26 and the corresponding measured stiffness and strength data are summarized in Table 27. Plots of E_c and MOR are shown in Figure 16 and Figure 17. The E_c and MOR values when normalized by their 28-day values are re-plotted in Figure 18 and Figure 19 along with the MEPDG Level 3 default strength and stiffness gains. Note that although these are the best PCC material property data that could be found, there are still some anomalies:

¹ Fly ash affects the rate of strength gain.

- high 14-day E_c and MOR values (or alternatively a counterintuitive dip in the 28-day E_c and MOR values) for the Gradation B mixture;
- an elevated 7-day MOR for the Gradation B Opt mixture at 7 days;
- anomalously high 28-day E_c and MOR values and/or anomalously low 90-day values for the Gradation D Opt mixture;
- consistently higher MOR values for Gradation F as compared to the other mixtures at all ages but particularly at 90 days;
- measured stiffness and strength changes with time (Figure 18 and Figure 19) that are greater than those predicted by the MEPDG Level 3 default aging relations (with the exception of the Gradation D Opt mixture, but this may be because of the anomalously high 28-day E_c and MOR values used to normalize the trends).

Table 26. Composition of Missouri DOT PCC mixes (ARA, 2009).

Location	Gradation	Cement Content	Flyash Content	Total Cementitious Materials Content	Percent Flyash	Total Water	w/cm Ratio
I-44 in Laclede County	Gradation B	479	85	564	15%	215	0.38
US 412 in Dunklin County	Gradation B	445	111	556	20%	229	0.41
I-435 in Jackson County	Gradation B Optimized	510	90	600	15%	258	0.43
MO 367 in St. Louis County	Gradation D	441	110	551	20%	215	0.39
US 63 in Randolph County	Gradation D Optimized	432	108	540	20%	212	0.39
I-35 in Clinton County	Gradation F	517	91	608	15%	231	0.38

Table 27. Mixture Properties from Missouri DOT.

MoDOT Mix Designation	Sample Age, days	Compressive Strength, psi	Flexural Strength (MOR), psi	Modulus of Elasticity, psi
Gradation B	3	3343	477	3775772
	7	4001	550	4172195
	14	4390	654	4318238
	28	4902	626	4290195
	90	5421	674	4757531
Gradation B Opt	3	3472	564	3729516

	7	3936	634	3972549
	14	4474	652	4164558
	28	4857	718	4266237
	90	5606	788	4632843
Gradation D	3	3756	587	3835707
	7	4472	595	4291245
	14	4848	640	4271614
	28	5082	655	4452082
	90	5875	725	4974852
Gradation D Opt	3	3884	540	4049615
	7	4382	583	4239712
	14	4810	637	4347735
	28	5120	744	4958388
	90	5970	699	4785520
Gradation F	3	3243	566	3348184
	7	3847	654	3767819*
	14	4502	739	4101783
	28	4886	772	4320960
	90	5643	897	4635612

*Missing data; interpolated via regression analysis from other Gradation F E_c values.

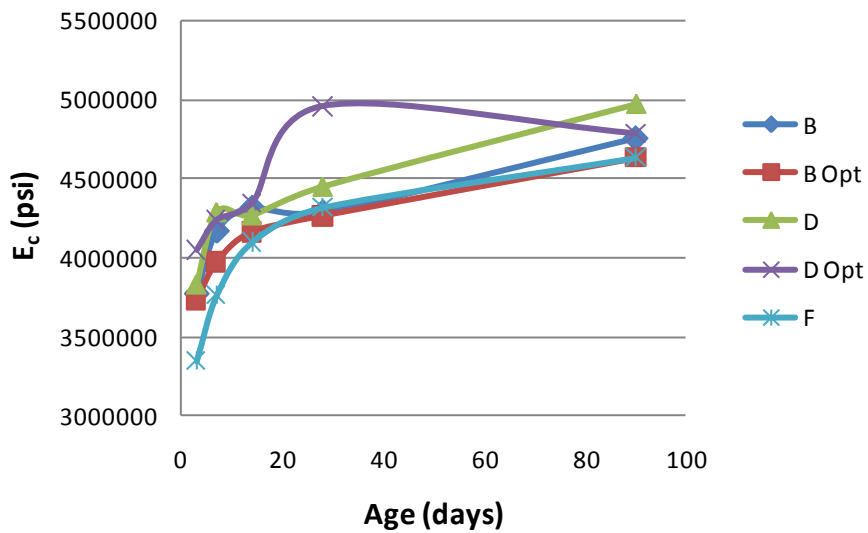


Figure 16. Measured E_c for Missouri PCC mixes.

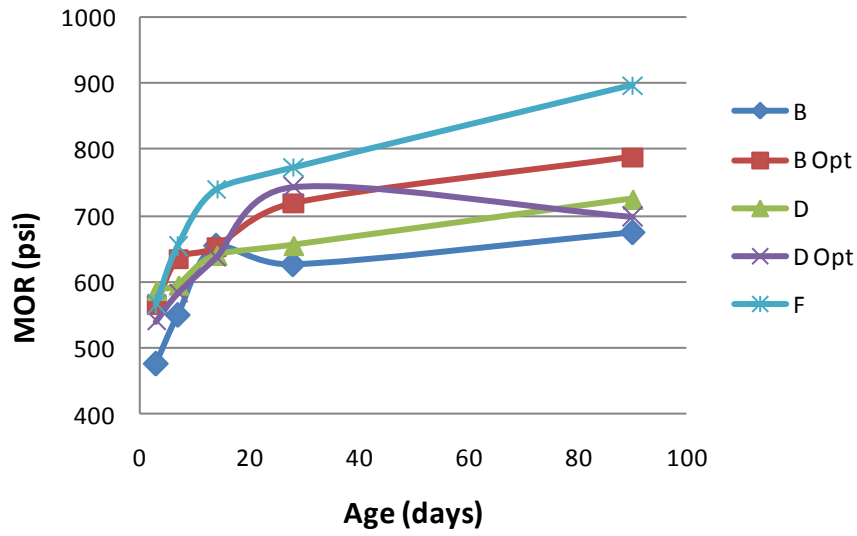


Figure 17. Measured MOR for Missouri PCC mixes.

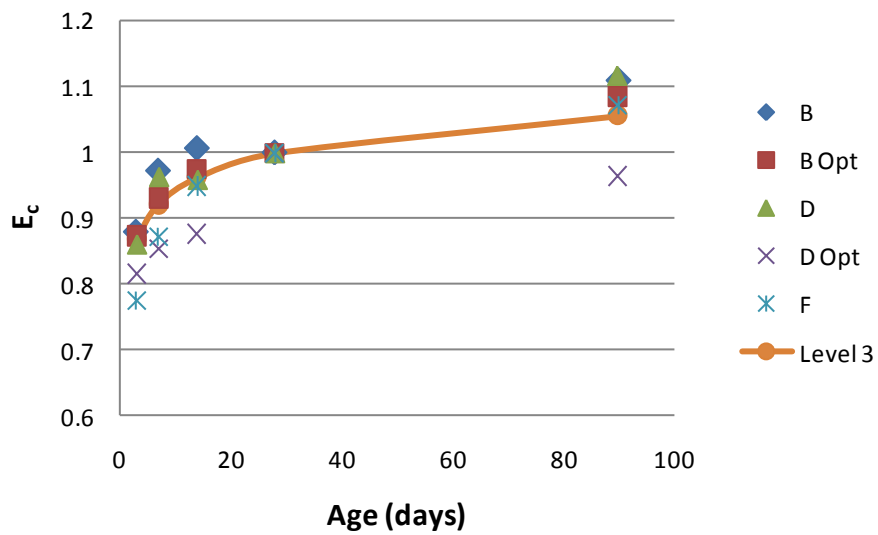


Figure 18. Normalized E_c data for Missouri PCC mixes with default Level 3 aging relationship.

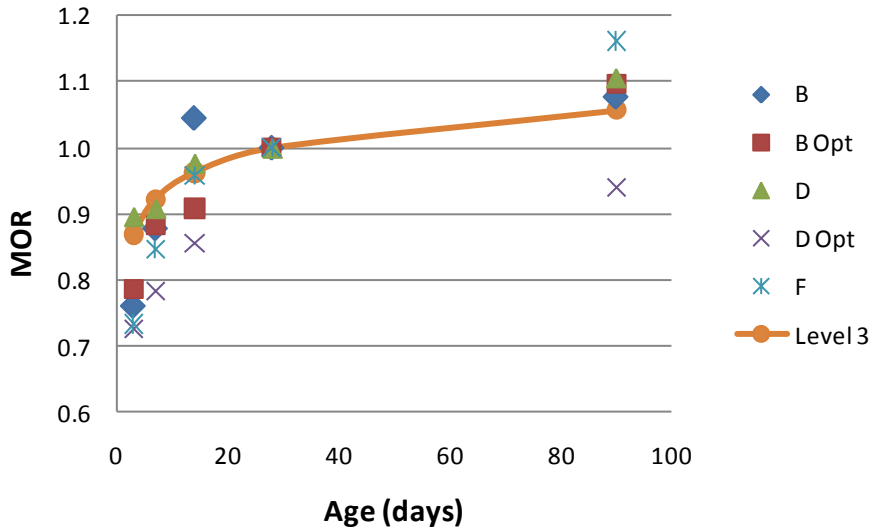


Figure 19. Normalized MOR data for Missouri PCC mixes with default Level 3 aging relationship.

The data in Table 27 were used in analysis scenarios of 10-inch, 9-inch and 8 inch thick PCC slabs over 6 inches of granular base. Joint spacing was set at 15 ft with a 1.25 in dowel diameter and 12 in dowel bar spacing. The measured PCC strength and stiffness properties were taken from the Missouri DOT study, and all other thermal and mix properties were taken either from the Maryland Salisbury bypass project or set equal to the MEPDG Level 3 defaults.² Initial two-way AADTT was set at 4000 (Principal Arterials – Interstate and Defense, TTC 1), with all other traffic variables set equal to the Level 3 default values in MEPDG. Baltimore (BWI) weather history was interpolated with Washington Dulles (IAD) data for the climate input to the MEPDG. This interpolation was required because the BWI weather history has gaps. The MEPDG default reliability of 90% was used for all predicted distresses.

² CTE values were measured for the Missouri mixes, but these data are judged to be unreliable.

The distresses predicted by the MEPDG using Level 1, 2 and 3 inputs for these scenarios are summarized in Table 28 to Table 32. Each table corresponds to one set of mix properties in Table 27 as applied to each of the three slab thicknesses (only the 8 inch slab thickness was analyzed for Gradation F because of the small levels of cracking predicted). The four alternatives for Level 3 are: Level 3a – 28-day f_c' only; Level 3b – 28-day MOR only; Level 3c – 28-day f_c' and E_c ; and Level 3d – 28-day MOR and E_c . The Missouri DOT report did not specify the estimated 20-year to 28-day property ratio. Therefore the Level 3 default value of 1.2 was used.

Table 28. Predicted distresses using MEPDG Level 1, 2 and 3 PCC material property inputs based on “Gradation B” in Missouri DOT (ARA, 2009).

Distress Type	Joint opening (in)		LTE (%)		Faulting (in)		Cracked Slabs (%)		IRI (in/mile)	
	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1
Level 1	0.053	0%	91.2	0%	0.068	0%	77.3	0%	165.6	0%
Level 2	0.052	-2%	91.2	0%	0.062	-9%	47.8	-38%	137.5	-17%
Level 3a	0.052	-2%	91.2	0%	0.064	-6%	27	-65%	121.1	-27%
Level 3b	0.053	0%	91.2	0%	0.061	-10%	47.7	-38%	137.4	-17%
Level 3c	0.053	0%	91.2	0%	0.068	0%	80.9	5%	168	1%
Level 3d	0.052	-2%	91.2	0%	0.067	-2%	41.9	-46%	135	-18%
CoV	0.010		0.000		0.048		0.392		0.129	

Layer thickness = 8 inch

Distress	Joint opening (in)	LTE (%)	Faulting (in)	Cracked Slabs (%)	IRI (in/mile)
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Type										
Input Level	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1
Level 1	0.052	0%	89.2	0%	0.069	0%	20.1	0%	118.9	0%
Level 2	0.051	-2%	89.3	0%	0.066	-4%	5.3	-74%	104.4	-12%
Level 3a	0.051	-2%	89.3	0%	0.066	-4%	2	-90%	101.9	-14%
Level 3b	0.052	0%	89.2	0%	0.065	-6%	5.1	-75%	104.2	-12%
Level 3c	0.052	0%	89.2	0%	0.069	0%	22.8	13%	120.8	2%
Level 3d	0.051	-2%	89.3	0%	0.068	-1%	4.7	-77%	105.2	-12%
CoV	0.011		0.001		0.026		0.899		0.076	

Layer thickness = 9 inch

Distress Type	Joint opening (in)		LTE (%)		Faulting (in)		Cracked Slabs (%)		IRI (in/mile)	
Input Level	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1
Level 1	0.049	0%	87.2	0%	0.063	0%	3.5	0%	104.1	0%
Level 2	0.05	2%	86.8	-1%	0.066	5%	0.6	-83%	100.3	-4%
Level 3a	0.05	2%	86.8	-1%	0.066	5%	0.2	-94%	99.9	-4%
Level 3b	0.051	4%	86.7	-1%	0.065	3%	0.5	-86%	100.4	-4%
Level 3c	0.051	4%	86.7	-1%	0.068	8%	4.1	17%	104.4	0%
Level 3d	0.05	2%	86.8	-1%	0.067	6%	0.6	-83%	100.8	-3%
CoV	0.015		0.002		0.026		1.095		0.020	

Layer thickness = 10 inch

Table 29. Predicted distresses using MEPDG Level 1, 2 and 3 PCC material property inputs based on “Gradation B Opt” in Missouri DOT (ARA, 2009).

Distress Type	Joint opening (in)		LTE (%)		Faulting (in)		Cracked Slabs (%)		IRI (in/mile)	
	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1
Level 1	0.052	0%	91.2	0%	0.067	0%	6.8	0%	105.8	0%
Level 2	0.053	2%	91.2	0%	0.063	-6%	44.4	554%	135.3	28%
Level 3a	0.053	2%	91.2	0%	0.064	-4%	28.4	318%	122.5	16%
Level 3b	0.052	0%	91.2	0%	0.067	0%	10.2	50%	107.8	2%
Level 3c	0.052	0%	91.2	0%	0.066	-2%	9.5	40%	99.5	-6%
Level 3d	0.053	2%	91.2	0%	0.067	0%	43.4	538%	136.6	29%
CoV	0.010		0.000		0.027		0.730		0.135	

Layer thickness = 8 inch

Distress Type	Joint opening (in)		LTE (%)		Faulting (in)		Cracked Slabs (%)		IRI (in/mile)	
	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1
Level 1	0.051	0%	89.3	0%	0.068	0%	0.3	0%	100.9	0%
Level 2	0.051	0%	89.2	0%	0.066	-3%	4.7	1470%	104.1	3%
Level 3a	0.051	0%	89.2	0%	0.066	-3%	2.1	600%	102	1%
Level 3b	0.051	0%	89.3	0%	0.068	0%	0.5	67%	101.1	0%
Level 3c	0.051	0%	89.3	0%	0.068	0%	0.5	67%	100.9	0%
Level 3d	0.051	0%	89.2	0%	0.068	0%	5	1570%	105.4	4%
CoV	0.000		0.001		0.015		0.993		0.019	

Layer thickness = 9 inch

Distress	Joint opening	LTE	Faulting	Cracked	IRI
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Type	(in)		(%)		(in)		Slabs (%)		(in/mile)	
	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 2*	Result	Δ based on Level 1
Level 1	0.05	0%	86.9	0%	0.066	0%	0	-100%	99.6	0%
Level 2	0.05	0%	86.8	0%	0.066	0%	0.5	0%	100.4	1%
Level 3a	0.05	0%	86.8	0%	0.066	0%	0.2	-60%	99.9	0%
Level 3b	0.05	0%	86.9	0%	0.066	0%	0.1	-80%	99.6	0%
Level 3c	0.05	0%	86.9	0%	0.066	0%	0	-100%	99.5	0%
Level 3d	0.05	0%	86.8	0%	0.067	2%	0.6	20%	100.8	1%
CoV	0.000		0.001		0.006		1.107		0.005	

*Zero slab cracking predicted using Level 1 inputs.

Layer thickness = 10 inch

Table 30. Predicted distresses using MEPDG Level 1, 2 and 3 PCC material property inputs based on “Gradation D” in Missouri DOT (ARA, 2009).

Distress Type	Joint opening (in)		LTE (%)		Faulting (in)		Cracked Slabs (%)		IRI (in/mile)	
	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1
Level 1	0.053	0%	91.2	0%	0.07	0%	55.6	0%	148.1	0%
Level 2	0.052	-2%	91.2	0%	0.063	-10%	39.9	-28%	131.7	-11%
Level 3a	0.052	-2%	91.2	0%	0.064	-9%	21.6	-61%	117.2	-21%
Level 3b	0.053	0%	91.2	0%	0.063	-10%	31.3	-44%	124.8	-16%
Level 3c	0.053	0%	91.2	0%	0.069	-1%	62.5	12%	153.4	4%
Level 3d	0.052	-2%	91.2	0%	0.069	-1%	39.4	-29%	134	-10%
CoV	0.010		0.000		0.050		0.363		0.102	

Layer thickness = 8 inch

Distress Type	Joint opening (in)		LTE (%)		Faulting (in)		Cracked Slabs (%)		IRI (in/mile)	
	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1
Level 1	0.051	0%	89.2	0%	0.07	0%	9	0	109.7	0%
Level 2	0.051	0%	89.3	0%	0.066	-6%	3.9	-57%	103.4	-6%
Level 3a	0.051	0%	89.3	0%	0.067	-4%	1.5	-83%	101.6	-7%
Level 3b	0.051	0%	89.2	0%	0.066	-6%	2.5	-72%	102.3	-7%
Level 3c	0.051	0%	89.2	0%	0.07	0%	11.3	26%	111.4	2%
Level 3d	0.051	0%	89.3	0%	0.069	-1%	4.5	-50%	105.4	-4%
CoV	0.000		0.001		0.028		0.708		0.038	

Layer thickness = 9 inch

Distress Type	Joint opening (in)		LTE (%)		Faulting (in)		Cracked Slabs (%)		IRI (in/mile)	
	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1
Level 1	0.05	0%	86.8	0%	0.068	0%	1.4	0%	102.1	0%
Level 2	0.05	0%	86.8	0%	0.066	-3%	0.5	-64%	100.2	-2%
Level 3a	0.05	0%	86.8	0%	0.066	-3%	0.1	-93%	99.8	-2%
Level 3b	0.05	0%	86.8	0%	0.066	-3%	0.2	-86%	100	-2%
Level 3c	0.05	0%	86.8	0%	0.068	0%	1.7	21%	102.3	0%
Level 3d	0.05	0%	86.8	0%	0.067	-2%	0.6	-57%	100.9	-1%
CoV	0.000		0.000		0.015		0.871		0.011	

Layer thickness = 10 inch

Table 31. Predicted distresses using MEPDG Level 1, 2 and 3 PCC material property inputs based on “Gradation D Opt” in Missouri DOT (ARA, 2009).

Distress Type	Joint opening (in)		LTE (%)		Faulting (in)		Cracked Slabs (%)		IRI (in/mile)	
	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1
Level 1	0.052	0%	91.3	0%	0.071	0%	18.5	0%	117.3	0%
Level 2	0.052	0%	91.2	0%	0.064	-20%	36.7	98%	129.3	10%
Level 3a	0.052	0%	91.2	0%	0.065	-8%	21	14%	116.8	0%
Level 3b	0.052	0%	91.3	0%	0.068	-4%	6.1	-67%	105.7	-10%
Level 3c	0.052	0%	91.3	0%	0.072	1%	14.5	-22%	114.9	-2%
Level 3d	0.052	0%	91.2	0%	0.073	3%	69.1	274%	160.7	37%
CoV	0.000		0.001		0.055		0.819		0.157	

Layer thickness = 8 inch

Distress Type	Joint opening (in)		LTE (%)		Faulting (in)		Cracked Slabs (%)		IRI (in/mile)	
	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1
Level 1	0.051	0%	89.3	0%	0.07	0%	1.5	0%	102.7	0%
Level 2	0.051	0%	89.3	0%	0.067	-4%	3.4	127%	103.1	0%
Level 3a	0.051	0%	89.3	0%	0.067	-4%	1.4	-7%	101.5	-1%
Level 3b	0.051	0%	89.3	0%	0.068	-3%	0.3	-80%	101	-2%
Level 3c	0.051	0%	89.3	0%	0.071	1%	1.1	-27%	103	0%
Level 3d	0.051	0%	89.3	0%	0.072	3%	15.9	960%	116.1	13%
CoV	0.000		0.000		0.031		1.513		0.055	

Layer thickness = 9 inch

Distress Type	Joint opening (in)		LTE (%)		Faulting (in)		Cracked Slabs (%)		IRI (in/mile)	
	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1
Level 1	0.05	0%	87.0	0%	0.067	0%	0.2	0%	100	0%
Level 2	0.05	0%	86.8	0%	0.066	-2%	0.4	100%	100.2	0%
Level 3a	0.05	0%	86.8	0%	0.066	-2%	0.1	-50%	99.8	9%
Level 3b	0.05	0%	87.0	0%	0.066	-2%	0	-100.0%	99.5	-1%
Level 3c	0.05	0%	87.0	0%	0.068	2%	0.1	-50%	100.3	0%
Level 3d	0.05	0%	86.8	0%	0.069	3%	2.8	1300%	103.4	3%
CoV	0.000		0.001		0.019		1.810		0.014	

Layer thickness = 10 inch

Table 32. Predicted distresses using MEPDG Level 1, 2 and 3 PCC material property inputs based on “Gradation F” in Missouri DOT (ARA, 2009).

Distress Type	Joint opening (in)		LTE (%)		Faulting (in)		Cracked Slabs (%)		IRI (in/mile)	
	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1	Result	Δ based on Level 1
Level 1	0.052	0%	91.3	0%	0.067	0%	0.6	0%	100.3	0%
Level 2	0.053	2%	91.2	0%	0.063	-6%	41.2	677%	132.9	32%
Level 3a	0.053	2%	91.2	0%	0.064	-4%	27.1	4420%	121.5	21%
Level 3b	0.052	0%	91.3	0%	0.069	3%	3.4	467%	104	4%
Level 3c	0.052	0%	91.3	0%	0.066	-2%	2	233%	101.2	1%
Level 3d	0.053	2%	91.2	0%	0.067	0%	44.4	7300%	137.6	37%
CoV	0.010		0.001		0.036		0.853		0.139	

Layer thickness = 8 inch

Differences in predicted joint opening displacement and load transfer efficiency (LTE) were less than 5% for all input levels, mixtures, and slab thicknesses. Differences in predicted faulting were slightly greater, but the largest discrepancies between the Level 2/3 and Level 1 were still less than about 10% in all cases.

In contrast, extremely large differences in the percentage of cracked slabs were found between the different input levels. As is evident from the data in the tables, Level 2 and the four alternatives for specifying Level 3 inputs produced wildly varying predictions of slab cracking, all of which were significantly different from the predictions using the Level 1 inputs. The differences in predicted slab cracking as compared to the Level 1 predictions ranged up to many thousands of percent.

Differences in IRI predictions using the various input levels were also significant, in large part because predicted slab cracking is one of the major inputs to the IRI model. The IRI discrepancies among the input levels increase as layer thickness decreases. The largest discrepancy in IRI was 74% for Gradation B in an 8-inch slab.

The coefficient of variation (CoV) is a good overall measure for the range of predicted performance across input levels. CoV was calculated across Level 1, Level 2, and the four cases of Level 3 for each distress for each mixture; these are summarized in the bottom row of Table 28 to Table 32. The ranges of CoV values for each distress across all mixtures are summarized in Figure 20. The CoV values for LTE, joint opening, and faulting do not exceed 0.06, which means the standard deviations are all within 6% of the average. The CoV values for IRI are higher but

still less than 0.2. In contrast, the CoV values for slab cracking are extremely large, with an average value of about 100% and lower and upper bounds of 0.36 and 1.81, respectively.

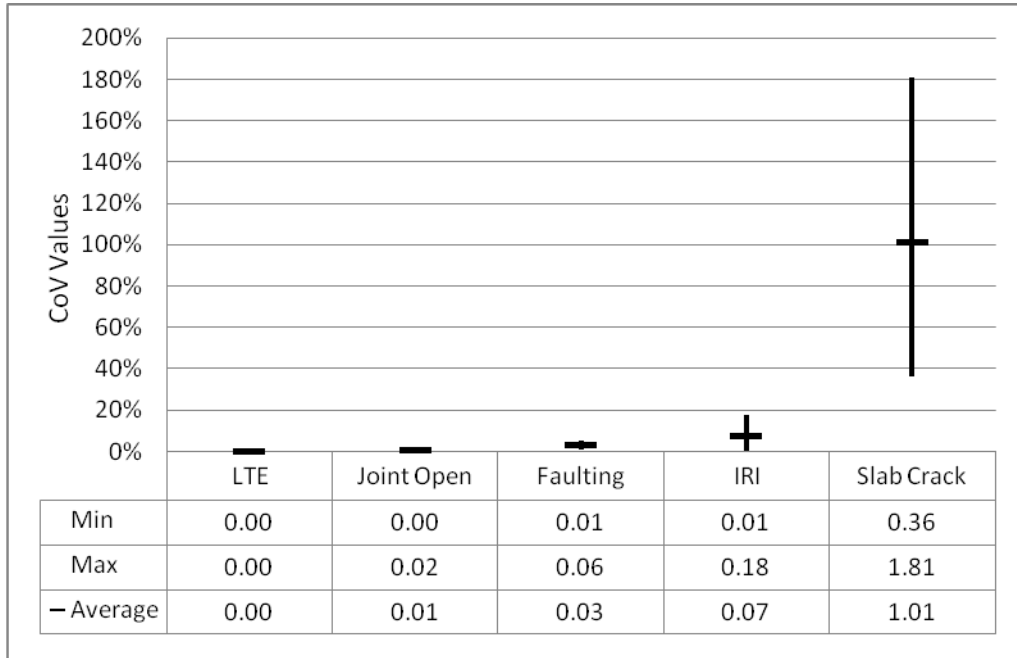


Figure 20. High/low/average plots of coefficient of variation by distress type.

Figure 20 merely shows that predicted slab cracking varies greatly by input level. Figure 21, Figure 22, Figure 23 and Figure 24 examine whether there are any trends in predicted slab cracking by input level for each slab thickness for the Gradation B, Gradation B Opt, Gradation D, and Gradation D Opt mixtures, respectively. Several observations can be drawn from the figures. First, for each individual mixture the general trends in the variations of predicted slab cracking with input level are qualitatively similar for all layer thicknesses, although the absolute magnitude of cracking sensibly increases with decreasing slab thickness. Second, there are no consistent trends for the variations of slab cracking over different input levels. For some mixtures, Level 1 produces the largest amount of predicted slab

cracking while for others it produces the smallest. Third, predicted slab cracking using Level 3c inputs (28-day E_c and MOR) consistently matches the Level 1 predictions most closely; there is generally poor agreement between Level 2 or the other Level 3 predictions and the reference Level 1 values.

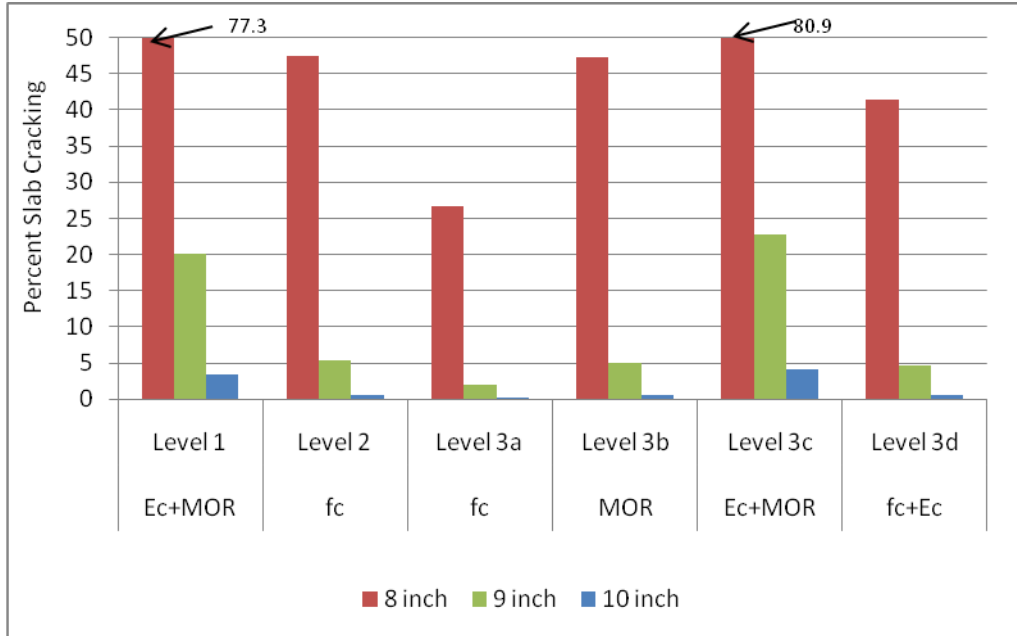


Figure 21. Predicted slab cracking value for different input levels and slab thicknesses for Gradation B mixture.

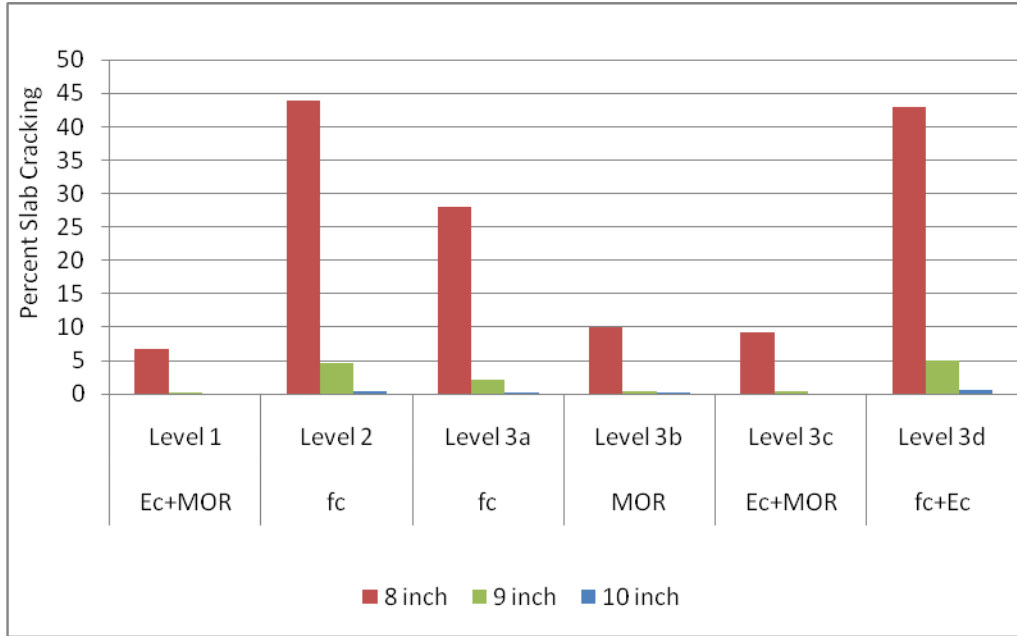


Figure 22. Predicted slab cracking value for different input levels and slab thicknesses for Gradation B Opt mixture.

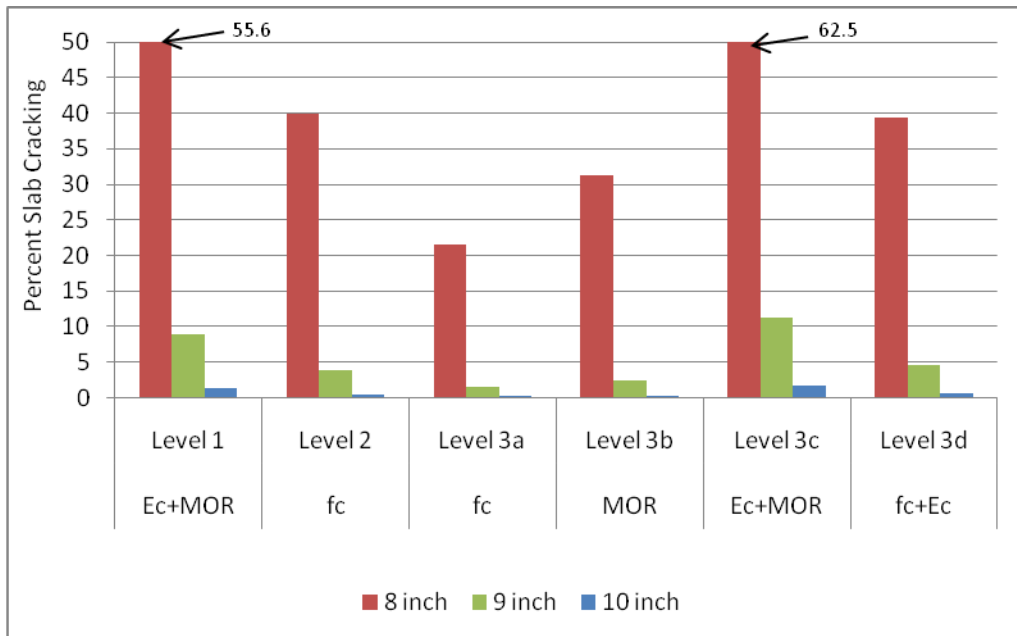


Figure 23. Predicted slab cracking value for different input levels and slab thicknesses for Gradation D mixture.

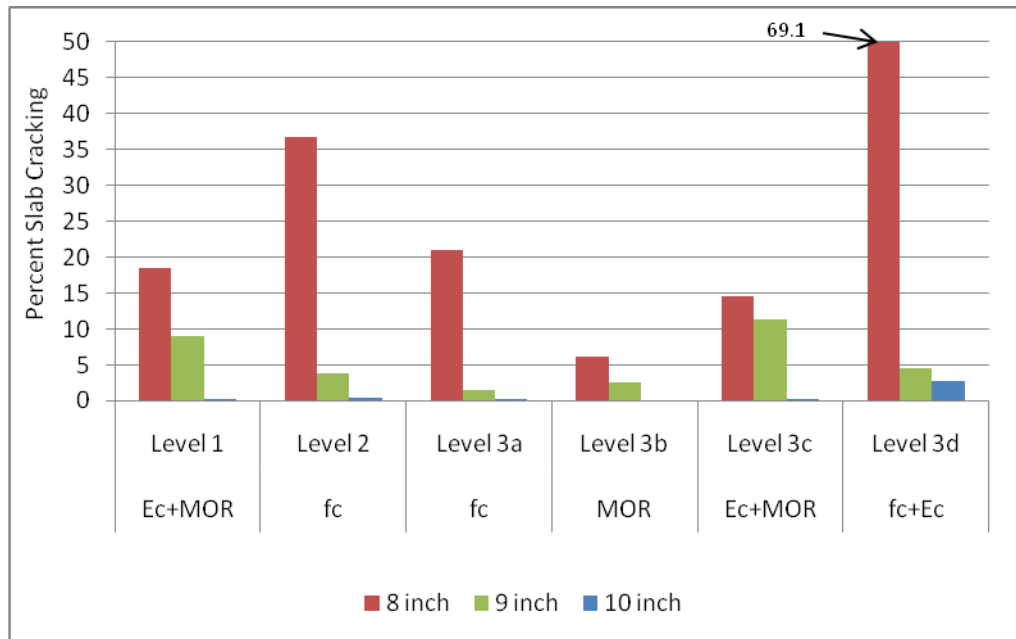


Figure 24. Predicted slab cracking value for different input levels and slab thicknesses for Gradation D Opt mixture.

Figure 25 summarizes the ratio of predicted slab cracking at Level 2 and different Level 3 options to the reference Level 1 predictions for the 8 inch slab thickness. The Level 3c predictions are generally the most consistently similar to the reference Level 1 values. The only exception to this is the Gradation F mixture, most likely because of its exceptionally high 90-day measured *MOR* strength value. The Level 2, 3a, 3b, and 3d predictions are erratic, often giving substantially larger or smaller predictions compared to the Level 1 reference depending upon the specific PCC mixture; no rational trends are observed. The reasons underlying these observations may be that both Level 1 and Level 3c use E_c and *MOR* data while Level 2 and Level 3a use only f_c' data, Level 3b is missing E_c information, and Level 3d is missing *MOR* information. These results suggest that the strength and stiffness aging

relationships built into the MEPDG in combination with 28-day measured values of both E_c and MOR may be adequate for design.

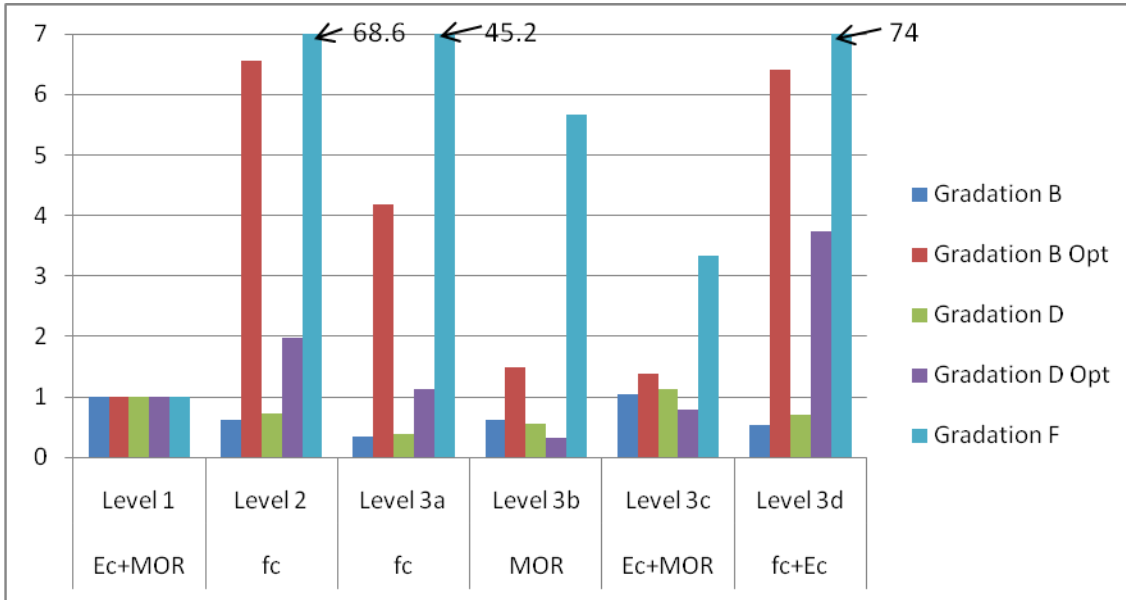


Figure 25. Predicted slab cracking compared to Level 1. Legend entries from top to bottom correspond to vertical bars from left to right for each input level option.

Sensitivity Analysis

A one-at-a-time (OAT) sensitivity analysis was also conducted to explore the relative importance of the various Level 1 strength and stiffness inputs. Base cases of 7, 14, 28, 90 and 20year/28day E_c and MOR data were generated using 28-day MOR and E_c values for Gradation B and D Opt and the Level 3 PCC aging relations, as summarized in Table 33. The Gradation B and D Opt mixes were selected because they respectively have among the lowest and highest 28-day E_c and MOR values. Bold entries in Table 33 are the measured values for Gradation B or D Opt and the non-bold values are calculated values using the default Level 3 PCC aging relations built into the MEPDG. The values in Table 33 were then used as Level 1 input values for the MEPDG. Each Level 1 input value was then increased or decreased by a given

percentage and the impact on predicted distress was evaluated. Similar to the sensitivity analyses in Chapter 3, the normalized sensitivity index for each output distress was calculated using Eq. (1) for each input parameter. Table 34 summarizes the computed normalized sensitivity indices of predicted distresses to input E_c and MOR values at different ages. It was seen that small variations in the 14-day E_c and MOR values have little impact on any of the predicted distresses. The sensitivities of the distresses to the stiffness and strength values at other ages are larger but more variable. The sensitivity indices for the 28-day, 90-day, and 20-year/28-day stiffness and strength values are generally larger than those for 7-day stiffness and strength. Overall, faulting is least sensitive to all of the stiffness and strength inputs, slab cracking is the most sensitive, and IRI exhibits intermediate sensitivity.

Table 33. Baseline cases in OAT sensitivity analysis. Bold values are measured.

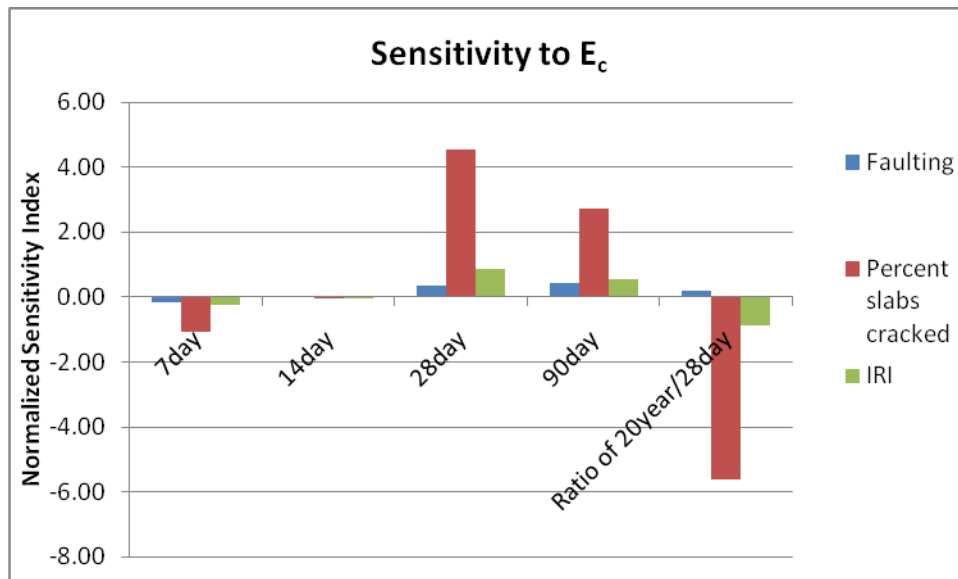
Age	From Gradation B		From Gradation D Opt	
	E_c (psi)	MOR (psi)	E_c (psi)	MOR (psi)
7day	3955929	577	4572061	686
14day	4129168	602	4772282	716
28day	4290231	626	4958430	744
90day	4534010	662	5240177	786
Ratio of 20year/28day	1.2	1.2	1.2	1.2

Table 34. Generalized sensitivity indices of E_c and MOR at different ages to predictions.

Age	From Gradation B			Normalized Sensitivity to E_c			Normalized Sensitivity to MOR		
	Faulting	Percent slabs cracked	IRI	Faulting	Percent slabs cracked	IRI	Faulting	Percent slabs cracked	IRI
7day	-0.19	-0.47	-0.23	0.04	1.04	0.42			
14day	0.04	-0.02	-0.01	0.00	0.05	0.02			
28day	0.41	1.70	0.76	-0.11	-2.76	-1.20			
90day	0.45	0.94	0.47	-0.04	-2.23	-0.90			
Ratio of 20year/28day	0.22	-2.09	-0.80	0.22	-2.11	-0.81			

From Gradation D Opt	Normalized Sensitivity to E_c			Normalized Sensitivity to MOR		
	Faulting	Percent slabs cracked	IRI	Faulting	Percent slabs cracked	IRI
7day	-0.14	-1.67	-0.24	0.00	4.25	0.48
14day	0.00	-0.06	0.00	0.00	0.18	0.02
28day	0.31	7.42	0.93	-0.10	-15.64	-1.82
90day	0.38	4.52	0.62	-0.03	-11.03	-1.23
Ratio of 20year/28day	0.14	-9.17	-0.97	0.14	-9.13	-0.96

To better illustrate these trends, Figure 26 summarizes the normalized sensitivity averaged across the Gradation B and D Opt values in Table 34. This figure clearly shows that (a) faulting is least sensitive to E_c and MOR; (b) slab cracking is most sensitive; and (c) the stiffness and strength properties at 7 and 14 days have less influence on predicted distress than do the values at 28, 90, and 20 years. (Note that varying the 28-day stiffness or strength values will also change the corresponding 20-year values for a fixed 20-year/28-day ratio.)



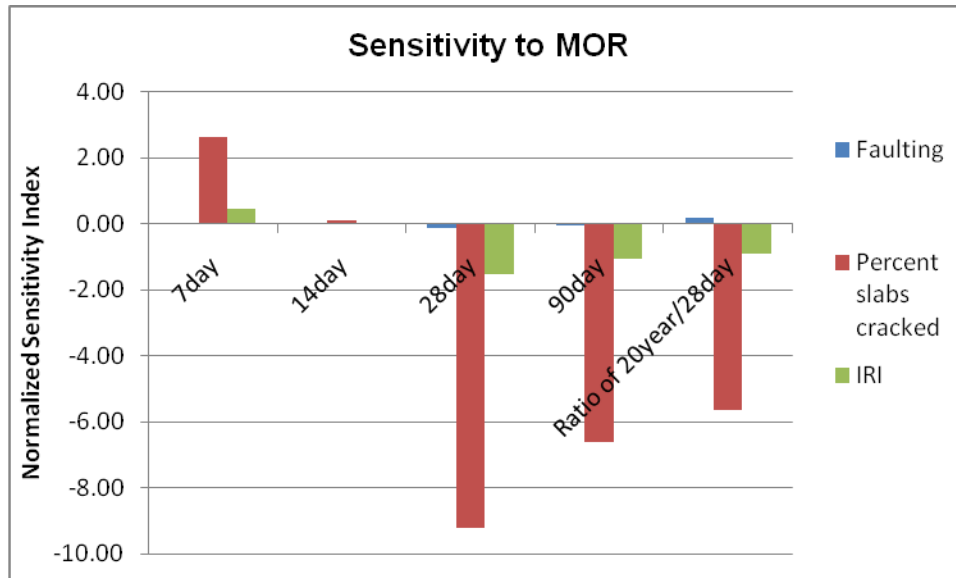


Figure 26. Normalized sensitivity of predicted distresses to E_c and MOR values at different ages.

Summary and Recommendations

The key findings from the analyses reported in this section are as follows:

1. Predicted slab cracking for jointed plain concrete pavements (JPCP) is highly sensitive to the input level for E_c and MOR . IRI is also sensitive to input level, primarily because it is a function a slab cracking. Predicted joint faulting and load transfer efficiency are essentially the same at all input levels.
2. Performance predictions using Level 3 inputs of 28-day E_c and MOR closely agree with those using the full Level 1 inputs. Therefore, this Level 3 input combination should be suitable for most SHA designs.
3. For full Level 1 inputs, predicted performance is most sensitive to 28-day, 90-day, and 20year/28day E_c and MOR inputs and less sensitive to 7-day and 14-day values.

Thermal Properties

Previous reviews of the literature (Schwartz and Ceylan, 2010) have documented that rigid pavement performance is very sensitive to surface shortwave absorptivity and the coefficient of thermal expansion, moderately sensitive to thermal conductivity, and insensitive to heat capacity. As described in

Table 23, there is no accepted method for measuring surface shortwave absorptivity, so the Level 3 guidelines should be followed for this input. Thermal conductivity can be measured in the laboratory, but as indicated in

Table 23 this property is relatively fixed for PCC and therefore the Level 3 default value should suffice for most designs.

The strong influence of the coefficient of thermal expansion (CTE) on JPCP performance has been demonstrated in several prior studies (Tanesi *et al.*, 2007; Buch *et al.*, 2008; Kampmann, 2008; Oh and Fernando, 2008; Haider *et al.*, 2008, 2009; Velasquez *et al.*, 2009). Results from these studies suggest that this influence may be inconsistent across different climates. Therefore, a limited sensitivity analysis of predicted pavement performance to CTE for typical Maryland conditions was conducted.

The baseline inputs for this sensitivity study are the same as for 8 inch thick Gradation D Opt PCC calculation (the results are shown in first subtable in Table 33). Climate conditions correspond to the Baltimore metropolitan area. A CTE value of $5.5 \times 10^{-6}/^{\circ}\text{F}$ was taken as a baseline, and this value was then adjusted $\pm 0.1 \times 10^{-6}/^{\circ}\text{F}$ to evaluate the sensitivity of predicted distress to CTE. The sensitivity of pavement performance to CTE input is defined in terms of a normalized sensitivity index S :

$$S = \frac{\Delta D}{\Delta CTE} \frac{CTE}{D} \quad (4)$$

in which ΔD is the change in distress caused by ΔCTE and D and CTE are the corresponding values for the baseline conditions.

The results from the CTE sensitivity study are summarized in Figure 27. Faulting and IRI were found to have a high sensitivity to CTE, with sensitivity indices averaging 2.9 and 1.6, respectively. Slab cracking was found to be extremely sensitive to CTE, reaching normalized sensitivity index values of up to 7.3; this

means that a 10% increase in CTE will cause a 73% increase in the predicted percentage of cracked slabs. These findings confirm the literature findings that CTE is a critical input for PCC performance predictions in the MEPDG. Consequently, accurate values of CTE will be required for design.

Because CTE is important but difficult to measure, a literature review was conducted in an attempt to find good predictive models for estimating CTE. No suitable model was found in this search. The weighted average method incorporated in the MEPDG appears to be the best model currently available.

As an added complication, a recent position paper issued by the FHWA (2009) cautions that the current AASHTO TP60 test protocol overestimates CTE by about 15%. Based on the limited sensitivity analysis in the present study, a 15% overestimate of CTE corresponds to about a 25% increase in predicted IRI, an approximately 50% increase in faulting, and an over 100% increase in slab cracking. The issues raised in the FHWA position paper have serious implications current CTE testing and future modifications of the AASHTP TP60 test protocol. They also have implications for the global calibration of the rigid pavement performance models in the current version of the MEPDG, as these calibrations are based on the erroneously overestimated CTE values. This issue is ongoing should be monitored by SHA. In the meantime, it is certainly premature to embark on any testing program for CTE for local Maryland mixtures. The Level 3 defaults for CTE in the current MEPDG software should be used in the interim until these issues are clarified and resolved.

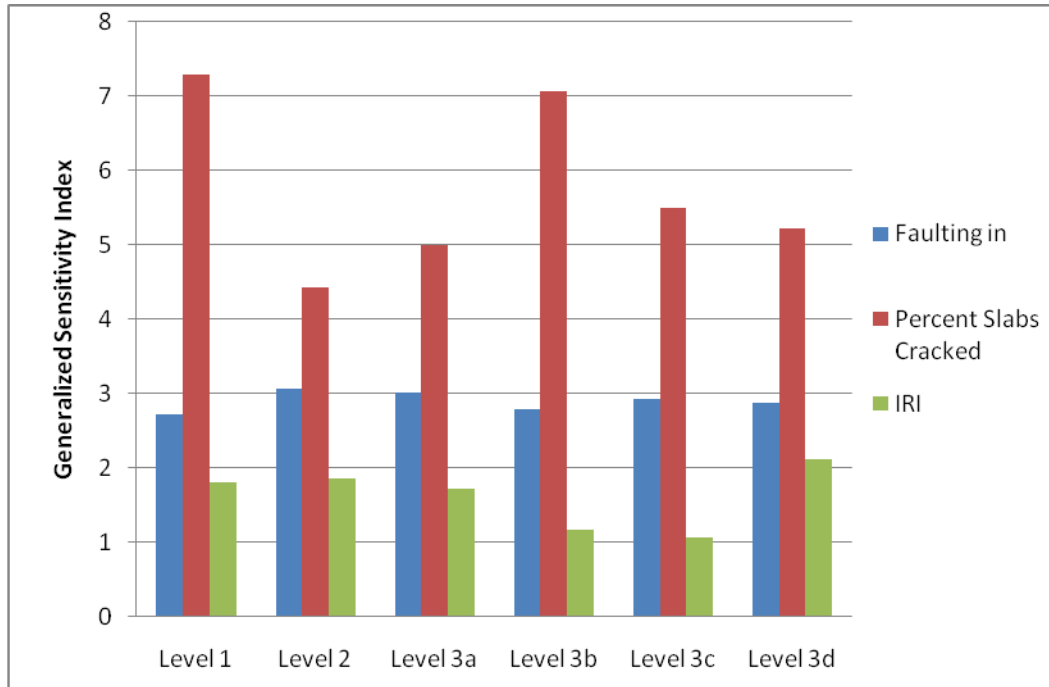


Figure 27. Generalized Sensitivity Index of CTE of different Levels in MEPDG

Shrinkage Properties

The MEPDG documentation provides little guidance on measurement of project-specific shrinkage properties for PCC mixes. For many of these properties (e.g., ultimate shrinkage strain, time to 50% shrinkage), there are no acceptable practical test protocols. The best recommendation at present is to use the Level 3 defaults for these properties built into the MEPDG software.

Summary

Testing Recommendations

There is very little data on the physical and mechanical properties Maryland PCC mixes to be incorporated into the database at this time. Much of the physical data required by the MEPDG (e.g., cement type, cementitious material content and water/cement ratio) is routinely available for individual projects and should be collected and entered into the database. Continued measurement of split cylinder tensile strength should be discontinued, as this is not a primary input to the MEPDG (or to the 93 AASHTO Design Guide). Instead, 28-day PCC elastic modulus and modulus of rupture should be measured for JPCP paving projects in the future, incorporated into the database, and used for Level 3 inputs to the MEPDG. There is no documented need to perform additional laboratory testing to determine the full Level 1 stiffness and strength inputs for PCC.

Given the lack of practical accepted test standards, ongoing test protocol issues, and other reasons, it is recommended that SHA not embark on any additional testing for thermal or shrinkage properties at this time. The current version of the MEPDG has been calibrated using the default Level 3 values for these properties, and these default values should continue to be used until accepted testing standards are available.

Recommended MEPDG Inputs

The recommended PCC inputs for the MEPDG are summarized in Table 35 through Table 38 below.

Table 35. Recommended PCC thermal and shrinkage property inputs for Maryland conditions (all JPCP construction types).

Property	Input Level	Value	Comment
<i>General Properties</i>			
PCC Material	3	JPCP	Only option available.
Layer thickness	1	Project specific	
Unit weight	3	150 pcf	MEPDG default
Poisson's ratio	3	0.2	MEPDG default
<i>Thermal Properties</i>			
Coefficient of thermal expansion	3	$5.5 \times 10^{-2}/^{\circ}\text{F}$	MEPDG default (global calibration value).
Thermal conductivity	3	1.25 BTU/hr-ft- $^{\circ}\text{F}$	MEPDG default (global calibration value).
Heat capacity	3	0.28 BTU/lb-ft	MEPDG default (global calibration value).

Table 36. Recommended PCC mix property inputs for Maryland conditions.

Property	Input Level	Value	Comment
Cement type	1	Type 1	Based on Mix 7 data from Salisbury bypass; should be replaced by mix-specific value if available
Cementitious material content	1	580 lb/cy	Based on Mix 7 data from Salisbury bypass; should be replaced by mix-specific value if available
Water/cement ratio	1	0.44	Based on Mix 7 data from Salisbury bypass; should be replaced by mix-specific value if available
Aggregate type	1	Limestone	Based on Mix 7 data from Salisbury bypass; should be replaced by mix-specific value if available
PCC zero stress temperature	3	Project specific	Default calculated by MEDPG as function of site weather conditions and cementitious material content
Ultimate shrinkage	2	Project specific	Default calculated by MEDPG as function of cement type, cement content, water/cement ratio, 28-day compressive strength, and curing conditions
Reversible shrinkage	3	50%	Value used in global calibration of distress models
Time to develop 50% shrinkage	3	35 days	Value used in global calibration of distress models
Curing method	1	Project specific	

Table 37. Recommended strength and stiffness input properties for new PCC for Maryland conditions (new/reconstruction/rehabilitation designs).

Property	Input Level	Value	Comment
28-day PCC modulus of rupture	3	685 psi	Based on Mix 7 split cylinder tensile strength from Salisbury bypass and empirical conversion; should be replaced by mix-specific value if available
28-day PCC elastic modulus	3	4,371,000 psi	Based on Mix 7 split cylinder tensile strength from Salisbury bypass and empirical conversion; should be replaced by mix-specific value if available

Table 38. Recommended strength and stiffness input properties for existing PCC for Maryland conditions (rehabilitation designs).

Property	Input Level	Value	Comment
28-day PCC compressive strength	2	Project specific	Obtained from cores of existing PCC slab.

CHAPTER 5: UNBOUND MATERIAL DATA

MEPDG Input Requirements

The principal mechanical property for unbound materials is the resilient modulus at the reference condition of optimum moisture and in-place density (AASHTO T180). The input requirements for resilient modulus vary by input level:

- Level 1
 - *Laboratory Measurement (New Construction/Reconstruction)*: The regression coefficients k_1 , k_2 , and k_3 for the stress-dependent resilient modulus relationship (AASHTO T307 or NCHRP 1-28A):

$$M_R = k_1 p_a \left(\frac{\theta}{p_a} \right)^{k_2} \left(\frac{\tau_{oct}}{p_a} + 1 \right)^{k_3} \quad (5)$$

in which:

M_R = resilient modulus

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

τ_{oct} = octahedral shear stress

$$= \frac{1}{3} \sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_1 - \sigma_3)^2}$$

$\sigma_1, \sigma_2, \sigma_3$ = principal stresses

p_a = atmospheric pressure (normalizing factor)

- *Field Measurement (Rehabilitation/Overlay Design)*: FWD backcalculated E_{FWD} values (AASHTO T256/ASTM D5858). These field backcalculated E_{FWD} value must be converted to an equivalent laboratory M_R value using the adjustment factors in Table 39.

- Level 2 - M_R determined from correlations with California Bearing Ratio, R -value, structural layer coefficient a_i , or plasticity index and gradation as summarized in Table 40.
- Level 3 - Default M_R at optimum moisture and density as a function of AASHTO soil type as summarized in Table 41.

In addition to stiffness, hydraulic properties for the partially saturated unbound materials in the base, subbase, and subgrade layers are required as inputs for the Enhanced Integrated Climate Model (EICM) built into the MEPDG.³ The principal hydraulic properties for unbound materials are the saturated hydraulic conductivity (permeability) and the soil water characteristic curve (SWCC). The input requirements for these vary by input level:

- Level 1 – Measured saturated hydraulic conductivity (AASHTO T215) and measured soil water characteristic curve (ASTM D6836) for determining parameters of the Fredlund-Xing (1994) model.
- Level 2/3 – Default saturated hydraulic conductivity as a function of gradation and plasticity index; default Fredlund-Xing SWCC parameters as a function of gradation (nonplastic/coarse soils) or gradation and plasticity (plastic/fine-grained soils).

Additional mechanical and physical property data required at all input levels in the MEPDG include:

- Gradation (AASHTO T88)
- Atterberg limits (AASHTO T89, T90)

³ Note that the details of the relations for the EICM inputs in the current version of the MEPDG have changed slightly from the descriptions in the NCHRP 1-37A final report. The updated formulation is described in Zapata and Houston (2008) and in Zapata *et al.* (2009).

- Specific gravity of solids G_s (AASHTO T100)
- Maximum dry density and optimum moisture content (AASHTO T180)
- Poisson's ratio ν
- Coefficient of lateral earth pressure K_0

Default values for these properties for each AASHTO soil class are provided in the MEPDG software. These default values can be replaced by project-specific data if available.

Table 39. Ratio of laboratory M_R to field backcalculated E_{FWD} modulus values for unbound materials (AASHTO, 2008).

Layer Type	Location	M_R/E_{FWD}
Aggregate Base/Subbase	Between a stabilized and HMA layer	1.43
	Below a PCC layer	1.32
	Below an HMA layer	0.62
Subgrade/Embankment	Below a stabilized subgrade/embankment	0.75
	Below an HMA or PCC layer	0.52
	Below an unbound aggregate base	0.35

Table 40. Models relating material index and strength properties to resilient modulus (NCHRP, 2004).

Strength/Index Property	Model	Comments	Test Standard
CBR	$M_r = 2555(\text{CBR})^{0.64}$ (TRL) Mr, psi	CBR = California Bearing Ratio, percent	AASHTO T193, "The California Bearing Ratio"
R-value	$M_r = 1155 + 555R$ (20) Mr, psi	R = R-value	AASHTO T190, "Resistance R-Value and Expansion Pressure of Compacted Soils"
AASHTO layer coefficient	$M_r = 30000 \left(\frac{a_i}{0.14} \right)$ (20) Mr, psi	a_i = AASHTO layer coefficient	AASHTO Guide for the Design of Pavement Structures
PI and gradation*	$\text{CBR} = \frac{75}{1 + 0.728(\text{wPI})}$ (see Appendix CC)	wPI = P200*PI P200= percent passing No. 200 sieve size PI = plasticity index, percent	AASHTO T27, "Sieve Analysis of Coarse and Fine Aggregates" AASHTO T90, "Determining the Plastic Limit and Plasticity Index of Soils"
DCP*	$\text{CBR} = \frac{292}{\text{DCP}^{1.12}}$	CBR = California Bearing Ratio, percent DCP =DCP index, mm/blow	ASTM D 6951, "Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications"

*Estimates of CBR are used to estimate M_r .

Table 41. MEPDG Level 3 default resilient moduli values at optimum moisture and density (AASHTO, 2008).

AASHTO Soil Classification	Recommended Resilient Modulus at Optimum Moisture (AASHTO T 180), psi		
	Base/Subbase for Flexible and Rigid Pavements	Embankment and Subgrade for Flexible Pavements	Embankment and Subgrade for Rigid Pavements
A-1-a	40,000	29,500	18,000
A-1-b	38,000	26,500	18,000
A-2-4	32,000	24,500	16,500
A-2-5	28,000	21,500	16,000
A-2-6	26,000	21,000	16,000
A-2-7	24,000	20,500	16,000
A-3	29,000	16,500	16,000
A-4	24,000	16,500	15,000
A-5	20,000	15,500	8,000
A-6	17,000	14,500	14,000
A-7-5	12,000	13,000	10,000
A-7-6	8,000	11,500	13,000

Summary of Data and Preliminary Analysis

A total of 85 acceptable sets of unbound properties were provided by SHA for initial population of the material properties database. Each set of data contained the following unbound material properties: AASHTO class and Group Index; Atterberg limits (PI and LL); percents passing the No. 4 and No. 200 sieves; moisture content, saturation, and dry density at optimum conditions (AASHTO T180 assumed) and at other resilient modulus testing conditions; one or more sets of laboratory-measured resilient modulus data.

Considerable effort was devoted to identify incorrect or inconsistent information in the data provided by SHA. Incorrect or inconsistent data were corrected when possible and eliminated when not.

The key mechanical property required for unbound base/subbase/subgrade materials is the resilient modulus, M_R , at optimum moisture content and in-place density. For Level 1 inputs, the stress dependence of M_R must also be included as determined from the AASHTO T307, LTPP P46, or NCHRP 1-28A test protocols.

The scope of the provided data is described in Table 42. Many of the soils had laboratory measured M_R data at multiple moisture contents (typically optimum and optimum \pm 2%). There are multiple M_R testing records for each moisture condition since M_R is measured at different stress states. Gradation information was limited to the percents passing the No. 4 and No. 200 sieves. Other properties received included the PI , LL , maximum dry unit weight, optimum gravity water content, saturation

degree at optimum condition were also received. No hydraulic properties (e.g., hydraulic conductivity) were provided.

Table 42. Number of test records received from SHA.

Classification	Number of Property Sets	Number of Acceptable M_R Records
A-1-b	3	44
A-2-4	17	575
A-2-6	6	103
A-3	1	56
A-4	33	1331
A-5	1	42
A-6	13	463
A-7-5	4	168
A-7-6	2	84
Classification Not Mentioned	5	45
TOTAL	85	2911

The mean and ranges of values for the unbound material supplied by SHA are summarized in Figure 28 through Figure 31. These summaries include M_R at 95% compaction and optimum moisture content (Figure 28), optimum moisture content (Figure 29), degree of saturation at optimum moisture (Figure 30), and maximum dry unit weight (Figure 31) for each soil type. The grey bars (right axis) indicate the number of test records included in the database, the heavy black vertical lines (left axis) indicate the ranges of the data, and the heavy black short horizontal lines indicate the mean values. Noteworthy observations regarding the data in these figures include the following:

- The A-2-4, A-4 and A-6 are the most common unbound material in the data set. There is not much data for the A-1-b, A-2-6, A-3, A-5, A-7-5, and A-7-6 soil classes.
- The ranges of the M_R value are large for all soil types. This is because all stress states are included in the chart.

- The ranges of optimum moisture content, saturation at optimum, and maximum dry unit weight are within reasonable limits.

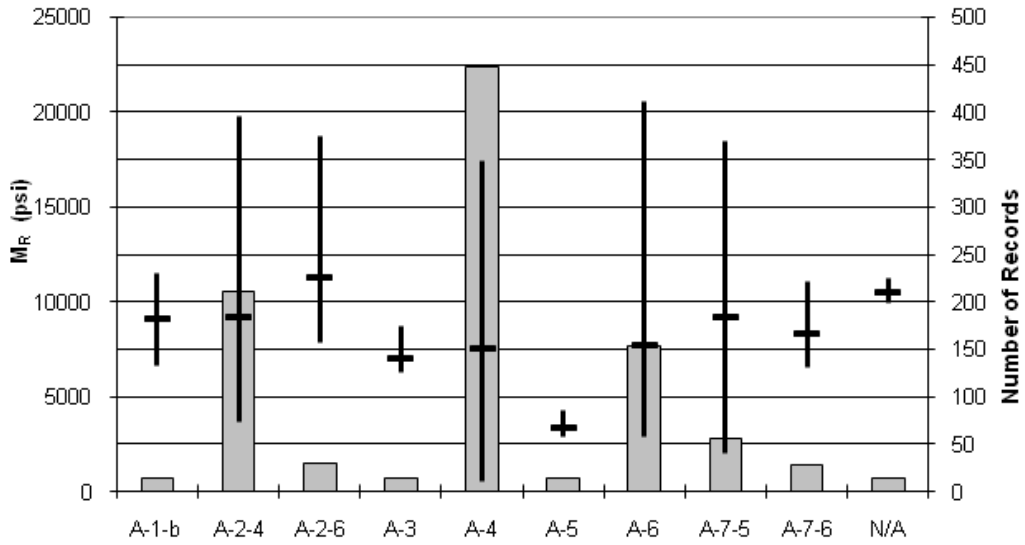


Figure 28. Averages and ranges of resilient modulus values at 95% compaction and optimum moisture content (includes all stress states).

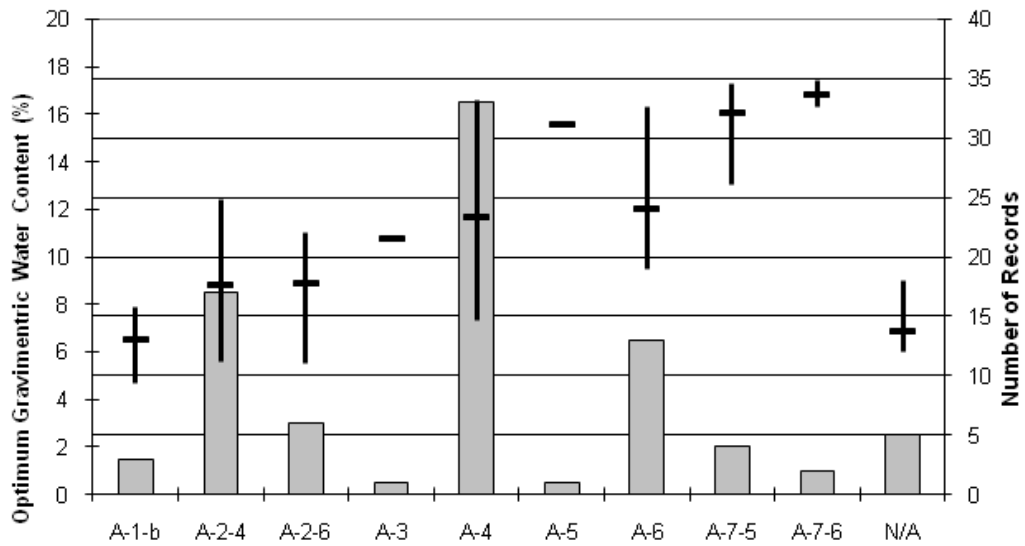


Figure 29. Averages and ranges of optimum water contents.

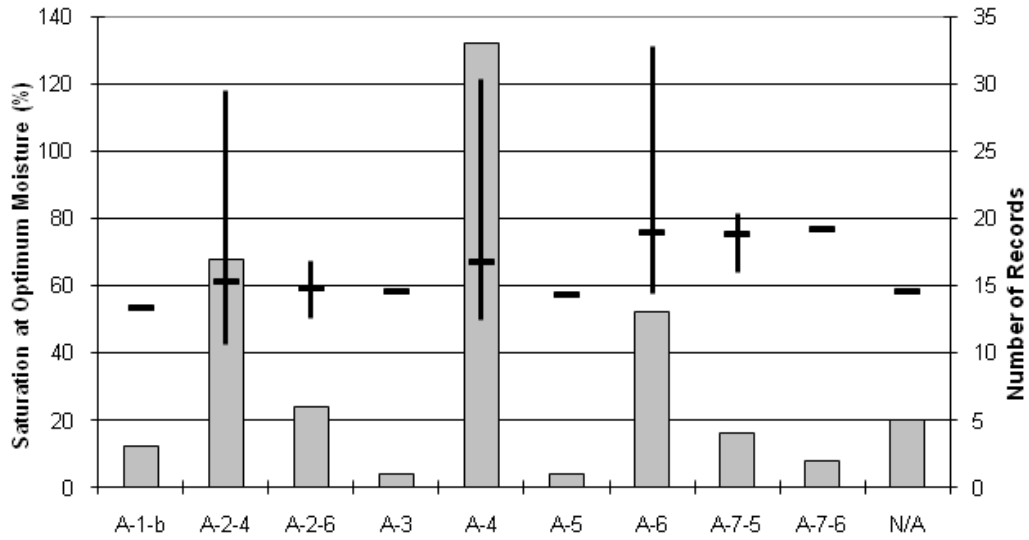


Figure 30. Averages and ranges of saturation levels at optimum moisture.

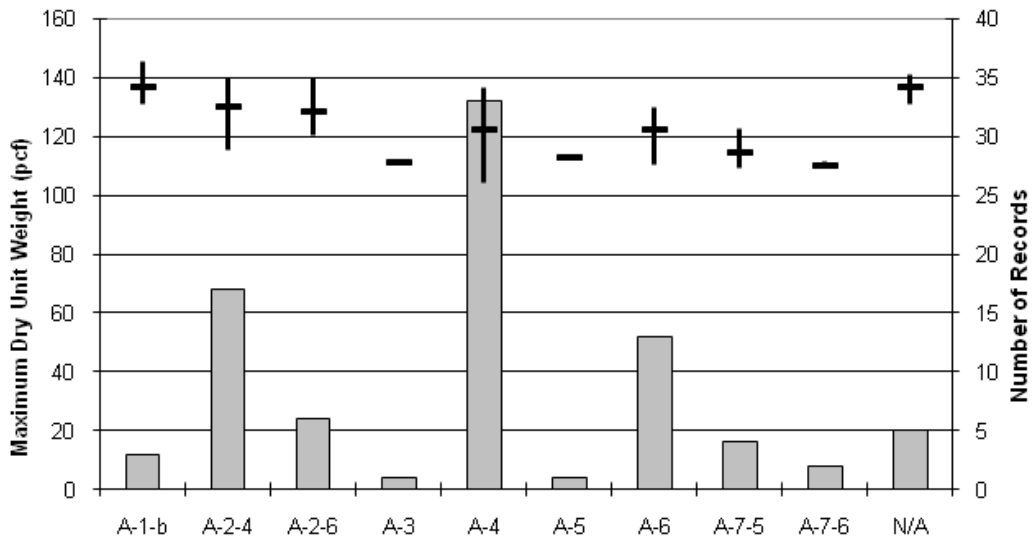


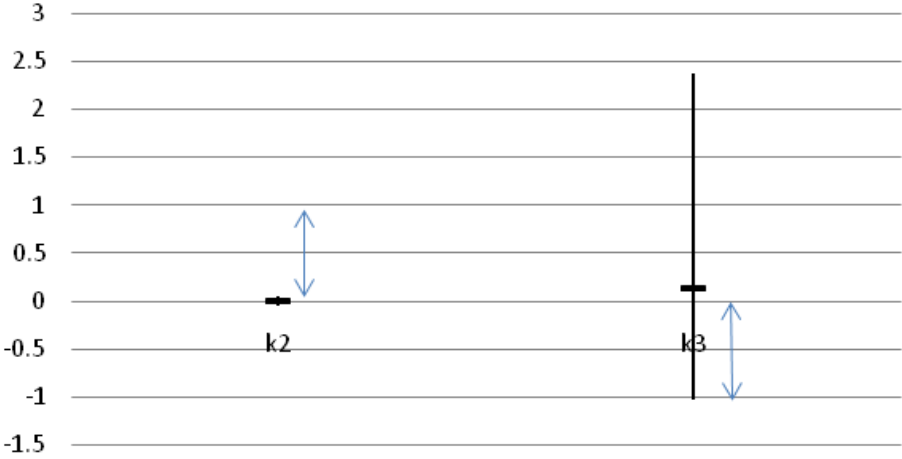
Figure 31. Averages and ranges of maximum dry unit weights.

The k_1 , k_2 and k_3 values for each soil property set were calculated using nonlinear regression of the laboratory M_R test records. Figure 32 shows the minimum, maximum, and average values of k_2 and k_3 for 95% compaction at optimum moisture content for A-2-4 and A-7-5 soils, the coarsest and finest grained soils respectively in

the database. The heavy black vertical lines indicate the ranges of the data, and the heavy black short horizontal lines indicate the mean values. The double arrows in the figure show the typical expected range for each parameter for each soil type. As defined in Eq. (5), k_2 is the confining stress stiffening exponent and k_3 is the shear stress softening exponent. The k_2 exponent is typically around 0.5 to 0.8 for coarse-grained soils and near 0 for fine-grained cohesive soils, while the k_3 exponent is always negative, slightly negative for coarse-grained soils and more strongly negative for fine-grained cohesive soils. It is clear from Figure 32 that the k_2 and k_3 values for the coarse-grained A-2-4 soil lie mostly outside their expected ranges; the k_3 values for the fine-grained A-7-5 are mostly positive, contrary to physical reasoning. These material parameters nevertheless provide good predictions of measured M_R values; as shown in Figure 33, the predicted vs. measured values fall nearly along the line of equality with minimal scatter.

The explanation for these anomalous findings is unclear. Closer examination of the data reveals that many of the property sets have some measured M_R records that do not follow the expected trends of increasing M_R as chamber pressure increases or decreasing M_R as deviator stress increases. It is not known whether these anomalies are due to testing or material issues.

k2 and k3 for A-2-4 soil



k2 and k3 for A-7-5 soil

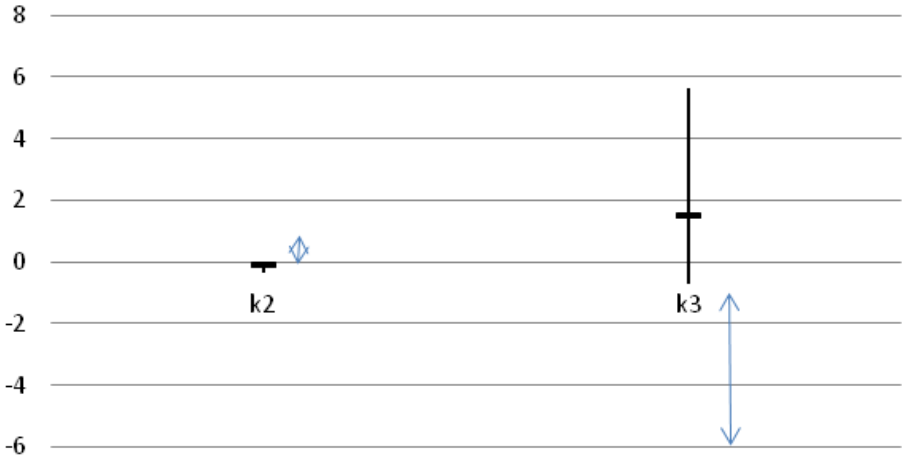
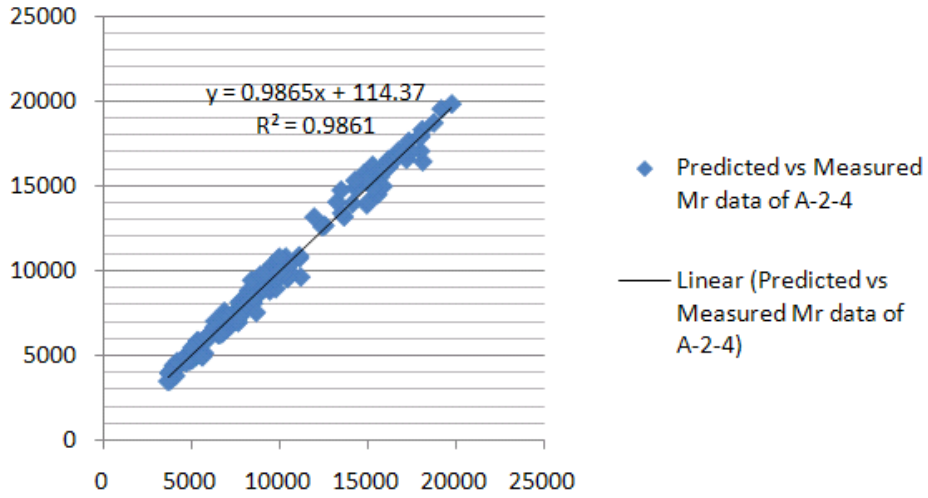


Figure 32. Averages and ranges of k_2 and k_3 for A-2-4 and A-7-5 soils.

Predicted vs Measured Mr (A-2-4)



Predicted vs Measured Mr (A-7-5)

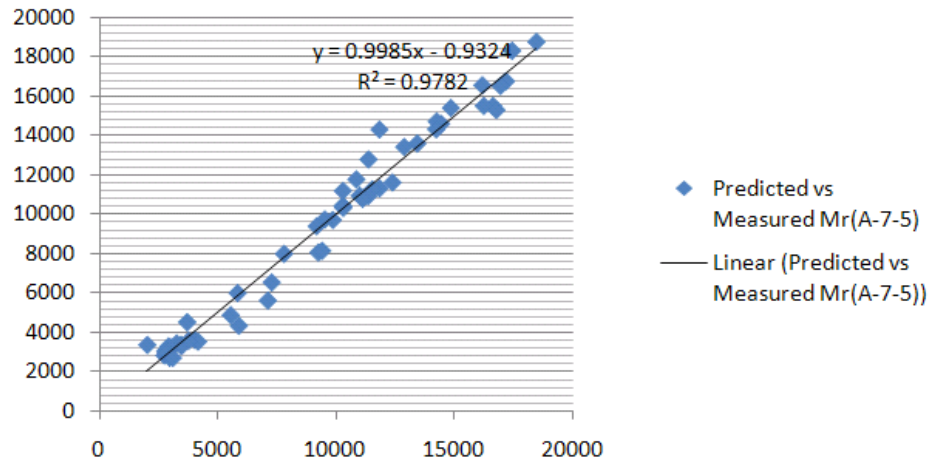


Figure 33. Predicted vs. measured resilient moduli for A-2-4 and A-7-5 soils.

In addition to the laboratory resilient modulus test data, the *SHA Pavement Design Guide* provides recommended moduli for unbound materials. These are summarized in Table 43. Note that these values are intended for use with the 1993

AASHTO Pavement Design Guide and therefore implicitly represent seasonally averaged values after adjustment for drainage.

Table 43. Recommended moduli for unbound materials from SHA *Pavement Design Guide*.

Material	Modulus (psi)		
	Minimum	Typical	Maximum
<i>Base/Subbase Materials</i>			
Graded Aggregate Base	15,000	25,000	45,000
Gravel	10,000	15,000	30,000
Soil Contaminated Aggregate Base	3,000	10,000	20,000
Capping Borrow	10,500	10,500	10,500
<i>Subgrade Soils</i>			
Silts and Clays (w/ high compressibility)		1,000 – 2000	
Fine Grained Soils with Silts and Clays (w/ low compressibility)		2,000 – 3,000	
Poorly Graded Sands		3,000 – 4,500	
Gravelly Soils, Well Graded Sands, and Sand/Gravel Mixtures		4,500 – 10,000	

Analyses of Unbound Material Properties

Stiffness Properties

Input Level

Level 1 unbound resilient modulus inputs are not recommended for use in the MEPDG at this time for the following reasons:

1. Input of Level 1 properties for unbound materials automatically switches the MEPDG structural analysis model from multilayer elastic theory to finite element analysis. The execution time for the flexible pavement finite element calculations in the current version of the MEPDG is far too long for practical usage.
2. Performance predictions using Level 1 unbound material properties have not been calibrated in the current version of the MEPDG.

Both of these issues will likely change in future versions of the MEPDG.

However, to date there have been no published studies using Level 1 unbound material inputs to provide any guidance on the sensitivity of predicted performance to resilient modulus input level.

Unlike many agencies, the Maryland SHA is well-equipped to perform Level 1 laboratory characterization of unbound materials, and the SHA testing effort to date has practical value despite the recommendations against Level 1 unbound resilient modulus inputs in the current version of the MEPDG. Laboratory measurements can be evaluated for expected in-service stress states to develop improved estimates of M_R values for Level 2/3 input. However, this requires estimates of typical *in situ* stress states for granular base layers and subgrades.

The 1993 AASHTO Design Guide provides typical values for the bulk stress θ for granular base layers. These are summarized in Table 44. Since the 1993 AASHTO Guide does not consider shear stress effects for granular base layers, no typical values for the octahedral shear stress τ_{oct} are provided. For typical Maryland pavement conditions, the data in Table 44 suggest a bulk stress θ in the range of 5 to 10 psi for granular base layers.

Table 44. Suggested bulk stress θ (psi) values for use in design of granular base layers (AASHTO, 1993).

Asphalt Concrete Thickness (inches [*])	Subgrade Resilient Modulus (psi [*])		
	3,000	7,500	15,000
< 2	20	25	30
2 – 4	10	15	20
4 – 6	5	10	15
> 6	5	5	5

^{*} 1 inch = 25.4 mm; 1 psi = 6.9 kPa

Von Quintus and Killingsworth (1997a, 1997b) provide examples for estimating the stress state in granular base and subgrade for an individual pavement structure. The representative conditions are taken at one-quarter depth into the granular base layer and 18 inches below the top of subgrade for a 9 kip wheel load. In situ stresses at these locations based on the material unit weights and the coefficients of lateral stress are combined with the load-induced stresses computed using multilayer elastic theory and reasonable preliminary estimates of the layer moduli. Seasonal effects due to moisture variations can also be included in the calculations.

Table 45 summarizes for typical Maryland conditions the stress states in the granular base and subgrade layers computed using Von Quintus and Killingsworth's approach. The HMA stiffness was assumed as 250,000 psi in all cases and the base layer stiffness was estimated as 25,000 psi. The load consisted of a 9000 lb tire having a 120 psi pressure. Full-slip conditions were assumed at the layer interface. All stress states also include the influence of the in situ stresses. The average computed stress states over all HMA and granular base thickness and subgrade M_R conditions were $\theta \cong 40$ psi, $\tau_{oct} \cong 3.5$ psi for the granular base layer and $\theta \cong 10$ psi, $\tau_{oct} \cong 2$ psi for the subgrade.

Table 45. Stress states for various typical Maryland pavement structures.

D ₁ (in)	D ₂ (in)	Subgrade M _R (psi)	Base - Quarter Depth					Subgrade - 18 in Depth				
			z (in)	σ_v (psi)	σ_h (psi)	θ (psi)	τ_{oct} (psi)	z (in)	σ_v (psi)	σ_h (psi)	θ (psi)	τ_{oct} (psi)
4	8	2000	6	31.005	25.418	82.84	2.75	30	3.858	0.605	10.07	2.12
		5000	6	34.031	20.609	76.25	6.45	30	5.308	0.582	11.47	2.82
	12	2000	7	33.919	19.598	74.28	6.89	34	2.932	0.571	9.74	1.78
		5000	7	35.264	15.931	68.29	9.25	34	4.205	0.606	11.08	2.36
10	8	2000	12	5.007	4.796	16.60	0.34	36	1.427	0.415	8.26	1.18
		5000	12	6.43	4.281	16.99	1.25	36	2.215	0.517	9.25	1.51
	12	2000	13	6.364	5.142	18.81	0.83	40	1.328	0.403	8.80	1.22
		5000	13	7.416	4.408	18.40	1.67	40	2.048	0.508	9.73	1.51
	16	2000	14	7.163	4.969	19.43	1.31	44	1.196	0.383	9.30	1.25
		5000	14	7.914	4.152	18.55	2.05	44	1.843	0.49	10.16	1.50
MINIMUM						16.60	0.34				8.26	1.18
MAXIMUM						82.84	9.25				11.47	2.82
AVERAGE						41.05	3.28				9.79	1.73

Richter (Richter, 2002; Richter and Schwartz, 2002) used multilayer elastic theory to estimate stress states at various locations within granular base/subbase layers and subgrades for field sections in the LTPP Seasonal Monitoring Program. These calculations were part of an effort to evaluate stress dependency of backcalculated layer moduli. Her calculated stresses are summarized in Figure 34 for granular base/subbase layers and in Figure 35 and Figure 36 for coarse and fine grained subgrades, respectively. Ranges of stress states encompassing most of the data points in these figures are summarized in Table 46 after conversion to U.S. Customary units.

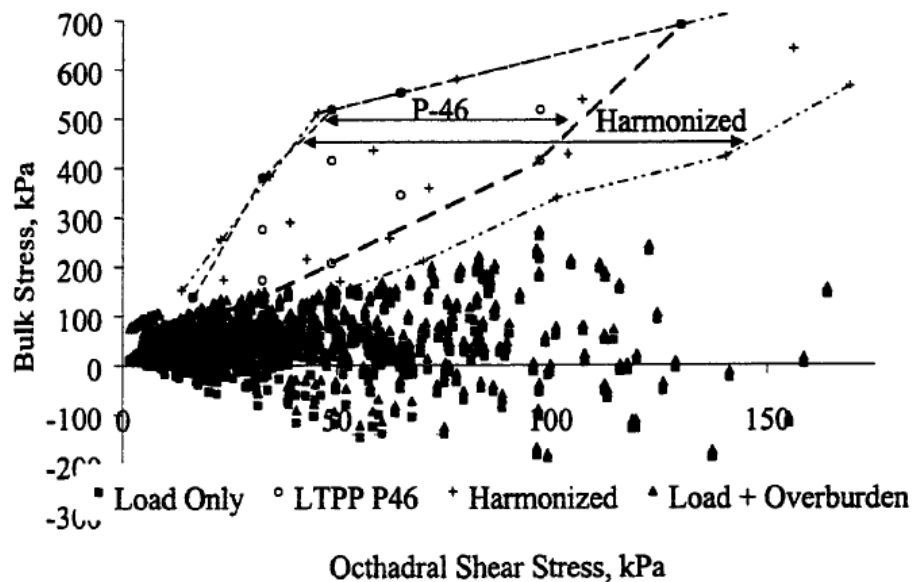


Figure 34. Calculated stress states for granular base and subbase layers (Richter, 2002).

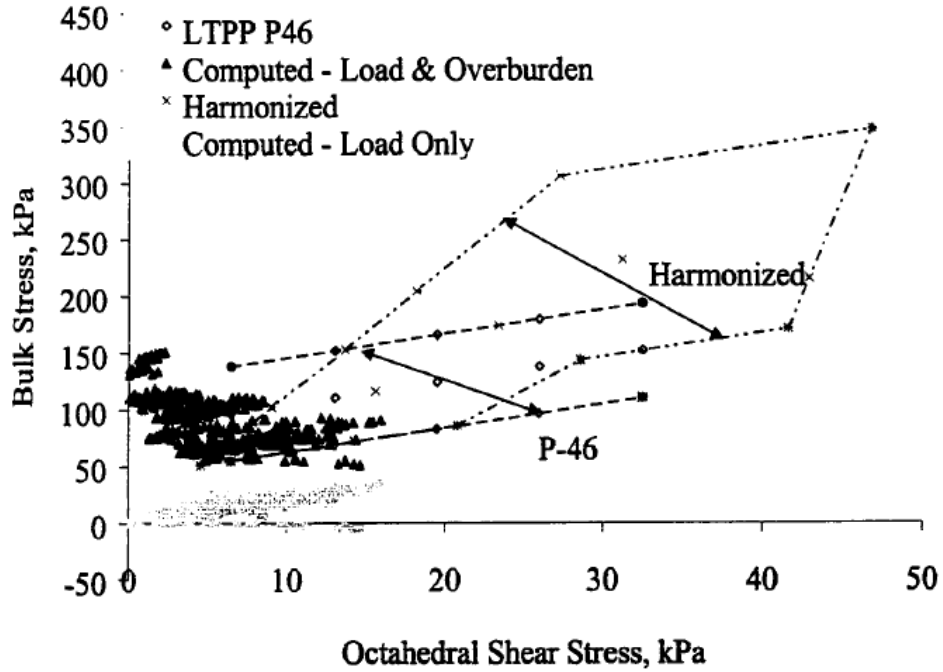


Figure 35. Calculated stress states for coarse grained subgrade soils (Richter, 2002).

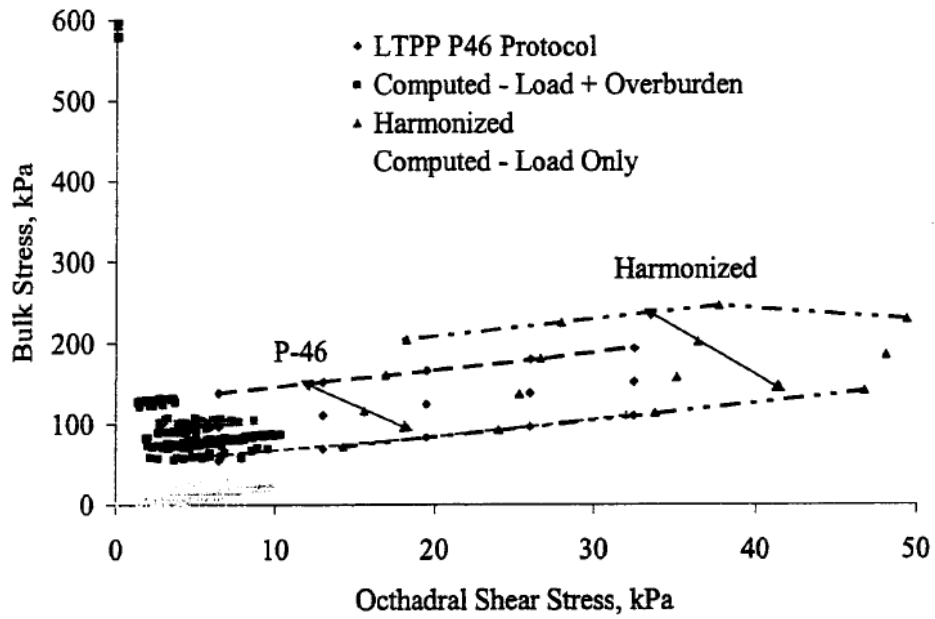


Figure 36. Calculated stress states for fine grained subgrade soils (Richter, 2002).

Table 46. Summary of stress state ranges from Richter (2002).

Layer/Soil Type	θ (psi)	τ_{oct} (psi)
Granular base/subbase	0 - 30	0 - 15
Coarse subgrades	7 - 20	0 - 2
Fine subgrades	7 - 20	0 - 1.5

Andrei (2003) estimated typical stress states for 30 LTPP test sections in Arizona. He found that the typical stress states for granular base layers and subgrades varied significantly with stiffness of the asphalt layer and thus with season. His values for typical stress states summarized in Table 47 are based on the assumption of an asphalt stiffness of 50 ksi during the hot summer months (Arizona conditions) and 1000 ksi during the cold winter. The values in Table 47 corresponding to hot conditions should be revised downward slightly for the more temperate Maryland summer climate. Adjusting for typical Maryland pavement conditions, the data in Table 47 suggest average stress states of approximately $\theta = 35$ psi, $\tau_{oct} = 10$ psi for granular base layers and $\theta = 10$ psi, $\tau_{oct} = 3$ psi for subgrades.

Table 47. Typical states of stress for Arizona flexible pavement sections (Andrei, 2003).

AC Layer Temperature Condition	E_{AC} (ksi)	Granular Base		Subgrade	
		θ (psi)	τ_{oct} (psi)	θ (psi)	τ_{oct} (psi)
Hot	50	44	16	13	5
Cold	1000	13	7	5	1

Von Quintus *et al.* (2004) compared backcalculated vs. laboratory measured M_R values in the LTPP database. In order to make this comparison, they estimated typical stress states beneath the FWD to use in determining the correct laboratory modulus value. Their estimated stress states for subgrade soils were $\sigma_v = 4$ psi, $\sigma_h = 4$ psi, $\theta = 12$ psi, and $\tau_{oct} = 0$ psi; the corresponding estimated stresses for granular base

layers were $\sigma_v = 15$ psi, $\sigma_h = 10$ psi, $\theta = 35$ psi, and $\tau_{oct} = 2.4$ psi. These values are in general agreement with those suggested by Andrei (2003). However, no backup calculations or other justifications for these values are provided.

Table 48 consolidates the typical stress states estimated for Maryland conditions based on all of the studies described above. In the absence of a detailed analysis of a specific pavement structure, the values listed as “best estimates” in the last row of the table can be used to determine an appropriate laboratory M_R value. In evaluating these “best estimates,” it is important to remember that most granular base materials should be relatively insensitive to τ_{oct} and most Maryland subgrade soils (other than on the Eastern Shore) should be relatively insensitive to θ .

Table 48. Consolidated estimates of pavement stress states for Maryland conditions.

Source	Granular Base		Subgrade	
	θ (psi)	τ_{oct} (psi)	θ (psi)	τ_{oct} (psi)
1993 AASHTO	5 – 10			
Von Quintus and Killingsworth (1997a, 1997b) – Table 45	40	3.5	10	2
Richter (2002)	0 – 30	0 – 15	7 – 20	0 – 1.5
Andrei (2003)	35	10	10	3
Von Quintus <i>et al.</i> (2004)	35	2.4	12	0
Best Estimate	30	5	12	2

The k_1 , k_2 , and k_3 resilient modulus parameters for Eq. (5) were evaluated for the laboratory M_R measurements provided by SHA and then Eq. (5) was evaluated for the “best estimate” stress states in Table 48. The results from this analysis are summarized in Table 49 by AASHTO soil class and moisture condition. In cases where multiple sets of test records are available, both the mean and the range of values are reported. Some soils are typical only of base conditions (e.g., A-1-b), some could be either base/subbase or subgrade soils (e.g., A-2-4), and some are

encountered only in subgrades (e.g., A-7-5). The appropriate M_R values at the appropriate stress state are given for each case. The shaded entries in the table indicate values that appear to be excessively high or low for the given soil class and moisture condition.

Note that the measured M_R values in Table 48 for subbase materials (A-2-4, A-2-6) are slightly lower on average but within the range of values in Table 43 that SHA currently uses in its 1993 AASHTO designs (e.g., soil contaminated granular base). Conversely, the measured M_R values in Table 48 for fine-grained subgrade materials (A-4, A-6) are higher than those in Table 43. However, the values in Table 43 are intended for use in the 1993 AASHTO Pavement Design Guide and implicitly include seasonal effects and drainage influences.

Table 49. SHA resilient modulus data evaluated at representative stress states. Shaded entries represent values that are anomalously low or high.

Class	Condition	N	Average M_R (Range of M_R) (psi)	
			Granular Base	Subgrade
A-1-b	Optimum	1	9851	
	Saturated	1	7526	
A-2-4	Optimum -2%	6	9842 (4894-13308)	9023 (5207-12382)
	Optimum	15	9370 (3870-18146)	8919 (4637-15871)
	Optimum+2%	1	7519	8717
	Saturated	3	7922 (4064-12188)	7582 (5088-10687)
A-2-6	Optimum -2%	1	2737	2438
	Optimum	2	11929 (8209-15650)	10022 (8232-11812)
A-3	Optimum	1	6670	7410
A-4	Optimum -2%	13		8258 (2940-15798)
	Optimum	12		5923 (2966-8462)
	Optimum+2%	1		15798
	Saturated	12		3964

				(2580-6457)
A-6	Optimum -2%	3		6688 (3937-8464)
	Optimum	3		5556 (3114-8668)
	Saturated	3		3050 (2134-3653)
A-7-5	Optimum -2%	2		8180 (6946-9415)
	Optimum	4		8438 (3477-13893)
	Saturated	1		5091
A-7-6	Optimum -2%	1		11555
	Optimum	2		7498 (7092-7904)
	Saturated	1		5361

Once the laboratory-measured M_R value at the appropriate stress state has been determined, it can be entered directly as a Level 2 or Level 3 input into the MEPDG. There are two options at Levels 2 and 3: “ICM Inputs,” for which M_R is entered at optimum moisture content and the EICM adjusts for seasonal moisture fluctuations, and “Representative value (design value),” which bypasses the EICM and instead uses an externally-determined seasonally adjusted M_R (similar in concept to 1993 AASHTO approach). The equation used by the EICM for moisture effects is given as (Andrei, 2003):

$$\log_{10} \frac{M_R}{M_{R,opt}} = a + \frac{b-a}{1 + e^{(\beta+k_s(S-S_{opt}))}} \quad (6)$$

in which:

M_R = resilient modulus at field saturation S

$M_{R,opt}$ = resilient modulus at maximum dry density and optimum

moisture

a = minimum value of the log of the modulus ratio

b = maximum value of the log of the modulus ratio

β = location parameter = $\ln(-b/a)$

k_s = regression parameter

$(S-S_{opt})$ = deviation of field saturation from optimum (decimal)

Values for the coefficients in Eq. (6) as implemented in the MEPDG are given in Table 50. Equation (6) can also be used for external estimation of a seasonally adjusted M_R .

Table 50. MEPDG values of a , b , and k_s for Eq. (6).

Parameter	Coarse-Grained Materials	Fine-Grained Materials
a	-0.3123	-0.5934
b	0.3	0.4
k_s	6.8157	6.1324

Yau and Von Quintus (2001) performed an extensive analysis of the LTPP database to determine regressions between k_1 , k_2 , and k_3 in Eq. (5) as a function of gradation, Atterberg limits, and other physical properties. However, these regressions are more appropriately used when no laboratory resilient modulus test data are available.

Sensitivity Analysis

The strong impact of base and subgrade stiffness on pavement performance is well-known from practical experience, from the 1993 AASHTO Design Guide, and from numerous MEPDG sensitivity studies reported in the literature (e.g., Masad and Little, 2004; El-Basyouny and Witczak 2005a, 2005b, 2005c; Carvalho and Schwartz, 2006; Hoerner *et al.*, 2007; Schwartz, 2009; Ayyala *et al.*, 2010). For example, Schwartz (2009) examined predicted service life as a function of granular base and

subgrade properties for typical flexible pavement sections using both the 1993 AASHTO and MEPDG procedures. Figure 37 and Figure 38 summarize the sensitivity of predicted service life to base and subgrade stiffness, respectively, for a granular base layer thickness $D_2 = 12$ inches and three HMA layer thicknesses $D_1 = 3, 6,$ and 9 inches. Reliability was set at 50% for both the AASHTO and MEPDG procedures. The strong impact of base and subgrade stiffness on predicted service life is clearly evident in all cases. There also is good agreement in the trends of the two design methods for the thin (3 inch) and medium (6 inch) asphalt layer cases. However, for the thick (9 inch) asphalt case, the sensitivity of service life to base modulus for the AASHTO design procedure is much greater than that for the MEPDG, as indicated by the steeper slope in the curve in Figure 38. Moreover, there is a crossing point for the thick asphalt case; the AASHTO procedure predicts much longer service life for pavements with high quality thick bases than does the MEPDG, but the reverse is true for low quality bases in this scenario. These trends and other similar results from the literature emphasize that good estimates of the resilient modulus of the unbound layers are important for accurate pavement performance prediction.

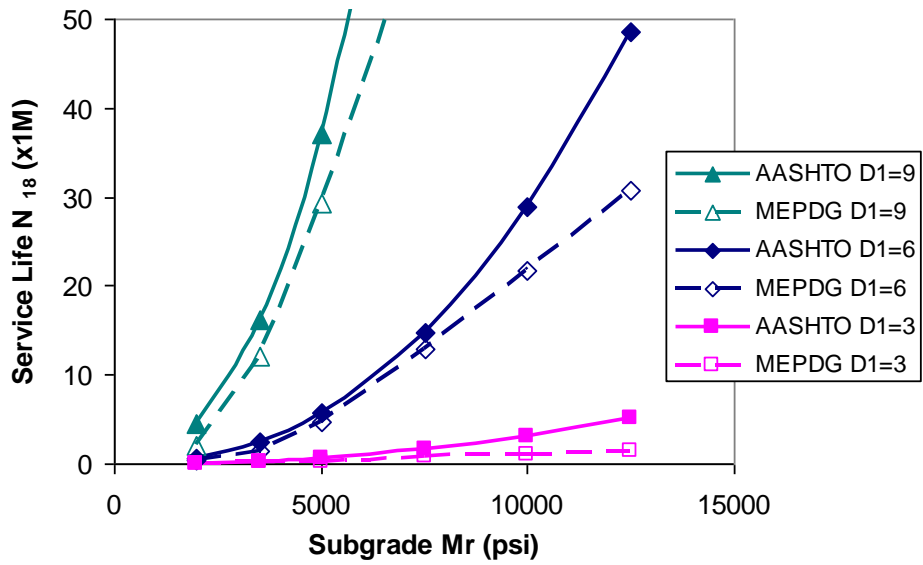


Figure 37. Predicted service life vs. subgrade resilient modulus; base modulus = 30,600 psi (Schwartz, 2009).

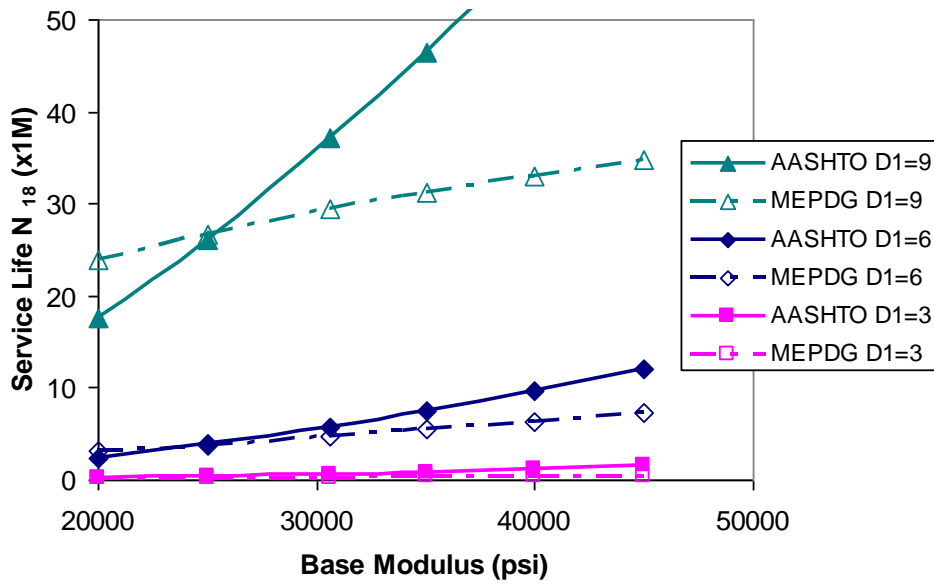


Figure 38. Predicted service life vs. granular base modulus; subgrade modulus = 5000 psi (Schwartz, 2009).

Hydraulic Properties

Input Level

The environmental model in the MEPDG is a one-dimensional formulation for vertical heat and partially-saturated moisture flows in the pavement system. Some of the assumptions in the MEPDG analyses include: zero rainfall infiltration through the pavement surface; no lateral flow to edge drains; liquid flow only—i.e., no vapor flow; uncoupled heat and fluid flow; and unbound material thermal conductivity and heat capacity values set internally to typical default values.

The principal hydraulic inputs in the MEDPG are the saturated hydraulic conductivity or permeability (k_s) and the parameters defining the soil-water characteristic curve (*SWCC*) for unsaturated soil conditions typical of unbound materials beneath pavements. Saturated hydraulic conductivity (permeability) is a familiar property to most geotechnical and pavement engineers. The *SWCC* generally is not. The *SWCC* is a fundamental property of unsaturated soils that soil matric suction (in conceptual terms, the negative porewater pressure in a partially saturated soil) and water content as shown in Figure 39. The *SWCC* is required for analyses of water movement under partially saturated conditions. It is also used in characterizing the shear strength and compressibility of unsaturated soils, and the unsaturated hydraulic conductivity of soil is often estimated using properties from the *SWCC* together with the saturated hydraulic conductivity.

The issue of Level 1 vs. Level 2 vs. Level 3 inputs for these properties is less significant than for other MEPDG inputs for several reasons:

- Few geotechnical laboratories, including the one at SHA, are equipped with the pressure plate apparatus (ASTM D3152-72) required to determine the level 1 *SWCC* inputs. In addition, this test procedure is exacting, time-consuming, and expensive to perform.
- The developers of the MEPDG expended considerable energy to develop a simplified yet accurate approach for specifying the *SWCC* in terms of the empirical Fredlund and Xing (1994) model. This model requires just four parameters to define the *SWCC*. The developers of the MEPDG developed correlations between these four parameters and grain size characteristics (for coarse soils) or grain size characteristics and plasticity (for fine-grained cohesive soils).
- As will be shown in the next subsection, predicted pavement performance is relatively insensitive to the *SWCC* and other hydraulic properties.

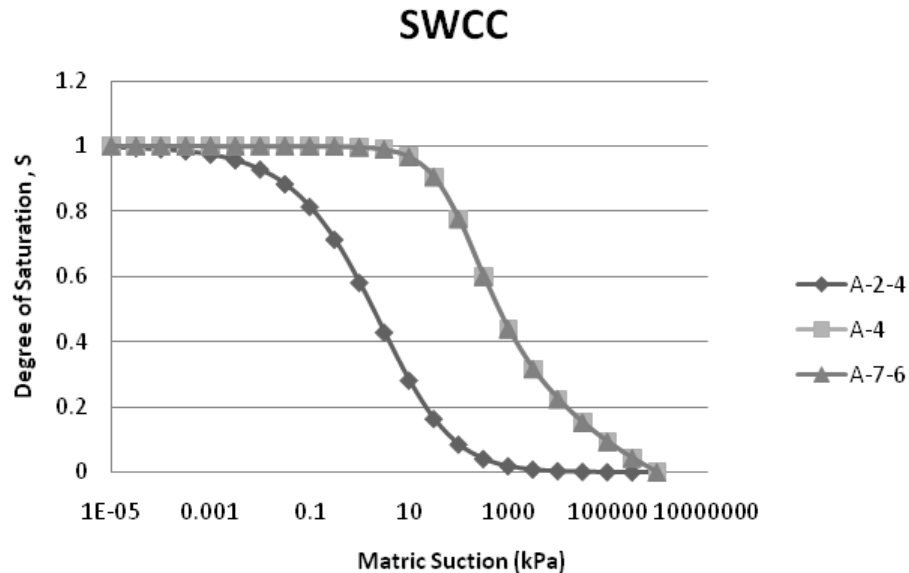


Figure 39. Examples of *SWCC* curves from the MEPDG.

Sensitivity Analyses

The impacts of thermohydraulic properties for the bound layers on pavement performance predictions in the MEPDG have been well documented. For flexible pavements, for example, the thermal properties of the asphalt concrete layer have a direct and pronounced influence on thermal contraction and low temperature cracking during sharp temperature drops in the winter and on softening and permanent deformations during high summer temperatures (e.g., Masad and Little, 2004; El-Basyouny *et al.*, 2005a, 2005b; Carvalho and Schwartz, 2006). The sensitivity of predicted pavement distresses to variations in the hydraulic properties of the unbound pavement materials—or even typical ranges of values for many of these properties—are much less well established. For example, it is expected qualitatively that increasing subgrade moisture content will tend to reduce subgrade stiffness and increase rutting and fatigue cracking. What is not known are the quantitative magnitudes of these changes with respect to a given percentage change in subgrade moisture content or the changes in subgrade moisture for different sets of soil water characteristic curve parameters.

A limited number of sensitivity analyses were performed as part of a reconnaissance study to quantify the influence of unbound hydraulic properties on predicted pavement performance. Four sites representing different climate extremes were considered: Seattle WA (wet-no freeze; PG 52-16), Caribou ME (wet-freeze; PG 52-34), Phoenix AZ (dry-no freeze; PG 76-10); and College Park MD (temperate; PG 64-22). Three pavement sections were analyzed at each site: 2 inches, 4 inches, and 6 inches of hot mix asphalt (HMA) over 12 inches of A-1-a base over a reference

A-4 subgrade. Traffic levels were adjusted to give a service life of approximately 15 years for the reference conditions at a 50% reliability level for all distresses. After a preliminary study, AADTT values of 300, 1000, and 2000 were used with 2, 4, and 6 inches of HMA, respectively. HMA material properties were typical for a 19 mm dense graded Superpave mixture. All other reference inputs were set equal to the Level 3 defaults. Input parameters varied for the sensitivity analyses included subgrade type (A-2-4, A-4, and A-7-6), groundwater table (*GWT*) depth (2, 7, and 12 feet), *SWCC* parameters (weighted plasticity index wPI varied by $\pm 50\%$ from reference), saturated hydraulic conductivity k_s ($\log k_s$ varied by $\pm 50\%$ from reference), traffic ($\pm 50\%$ from reference) and subgrade resilient modulus M_R ($\pm 50\%$ from reference).

Figure 40 summarizes the distress magnitudes predicted by the MEPDG for the reference analysis conditions. Because of the disparate units and magnitudes of the distress measures, they are expressed as a percentage of the MEPDG default design limits: 2000 ft/mi for longitudinal/top-down fatigue cracking; 25% of wheel path area for alligator/bottom-up fatigue cracking; 1000 ft/mi for transverse/thermal cracking; 0.25 in. for AC rutting; 0.75 in. for total rutting, and 172 in/mi for *IRI*. As is clear from

Figure 40, rutting was the controlling distress at all locations, followed by *IRI*. Very little fatigue cracking was predicted in any of the analyses and no thermal cracking was predicted.

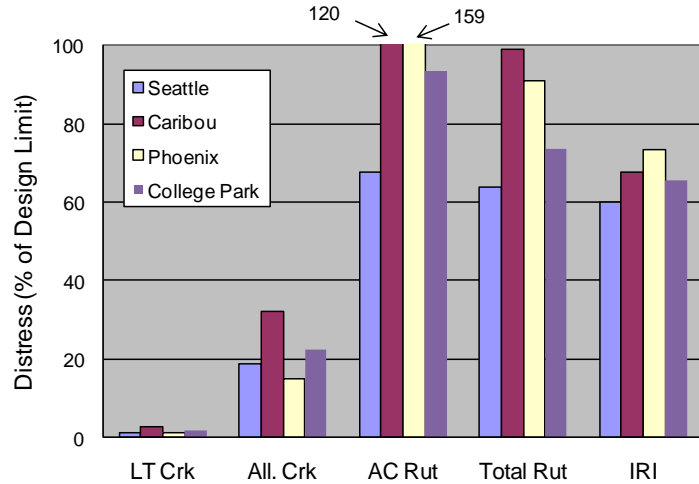


Figure 40. Predicted distresses for reference conditions (4 in. HMA, A-4 subgrade, 7 ft GWT depth, medium traffic). Legend entries from top to bottom correspond to vertical bars from left to right for each distress.

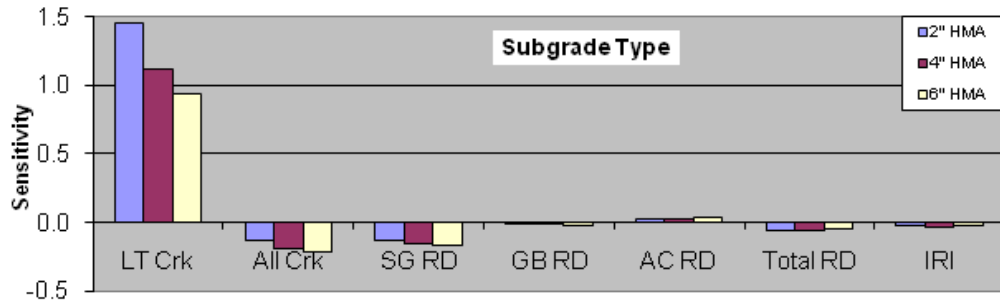
A local sensitivity index S_i for quantifying the effect of subgrade type on performance can be defined in normalized terms as:

$$S_i = \frac{D_{+1} - D_{-1}}{2D} \quad (7)$$

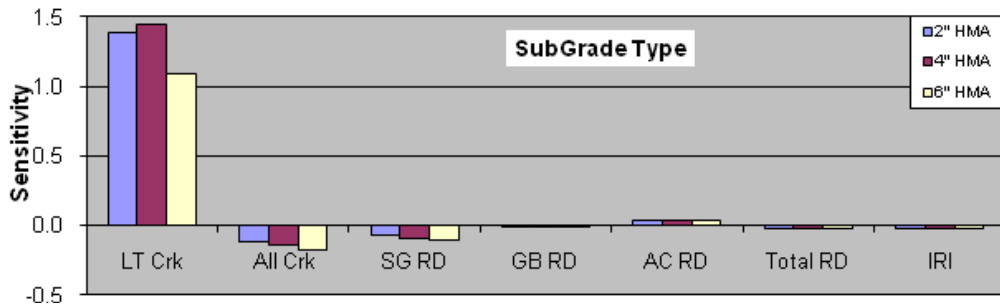
in which D is the distress magnitude predicted using the reference subgrade type (A-4) and D_{+1} and D_{-1} are the distress magnitude predicted using stronger (A-2-4) and weaker (A-7-6) subgrade, respectively.

Figure 41 summarizes the effect of subgrade type on performance under different climate condition in terms of the sensitivity index defined in Eq. (7). Several trends can be observed in these results: (a) longitudinal cracking (LT Crk) is most sensitive to subgrade type; (b) the sensitivities of alligator cracking (All Crk) and subgrade rutting (SG RD) are similarly low in magnitude; (c) granular base rutting (GB RD), asphalt concrete rutting (AC RD), and IRI are not sensitive to subgrade

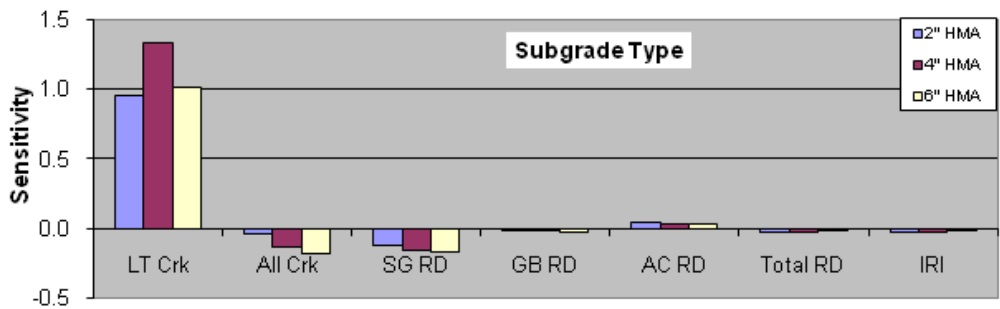
type; and (d) the sensitivity index values do not appear to be a function of climate type.



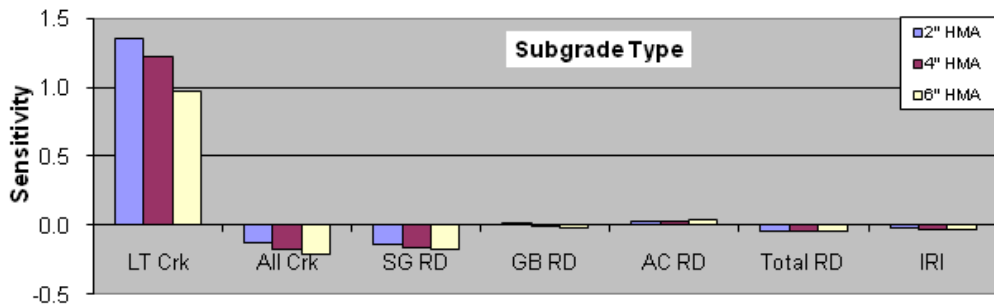
Seattle



Caribou



Phoenix



College Park

Figure 41. Sensitivity of distresses to subgrade type at each location.

Figure 42 summarizes the sensitivity of alligator cracking and total rutting to subgrade type variations at each of the four climate locations. The trends in Figure 42 are generally sensible. Alligator cracking decreases when going from a poor (A-7-6) to good (A-2-4) subgrade, and the rate of decrease is approximately the same at all four locations. Total rutting is less sensitive to subgrade type, although the trends from poor to good subgrades are physically reasonable. The fact that subgrade deformations are only one part of total rutting may be responsible for the relatively lower sensitivities.

Figure 42 shows that Caribou exhibited both the highest amount of alligator cracking and total rutting for all subgrade types among all of the climate locations. Examination of the average modulus of the top two feet of subgrade vs. time in Figure 43 sheds some insight into this. The annual freeze-thaw cycles at Caribou are very much evident for all three subgrade types. When frozen, the subgrade in Caribou is vastly stiffer than at any of the other locations. However, during the spring thaw and recovery the subgrade at Caribou has only about half the stiffness as at the other locations. The influence of subgrade stiffness on performance is not linear; this is true

even in the AASHTO empirical pavement design procedure. The detrimental effects of very soft subgrades far outweigh the beneficial effects of stiff subgrades. In other words, the spring thaw at Caribou is more significant for performance than is the frozen winter. This certainly conforms to real-world experience—e.g., the posting of load limits on roads in northern climes during spring thaw conditions.

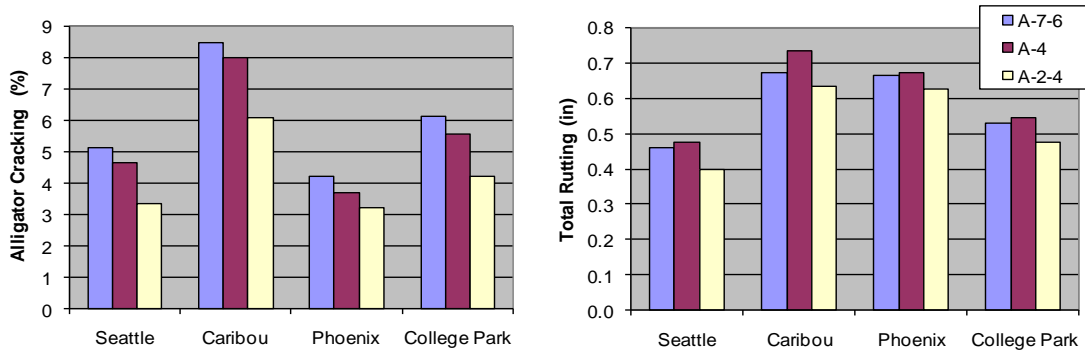


Figure 42. Influence of subgrade type on selected predicted distresses at all four climate locations (4 in. HMA thickness, 7 ft. GWT depth, medium traffic).

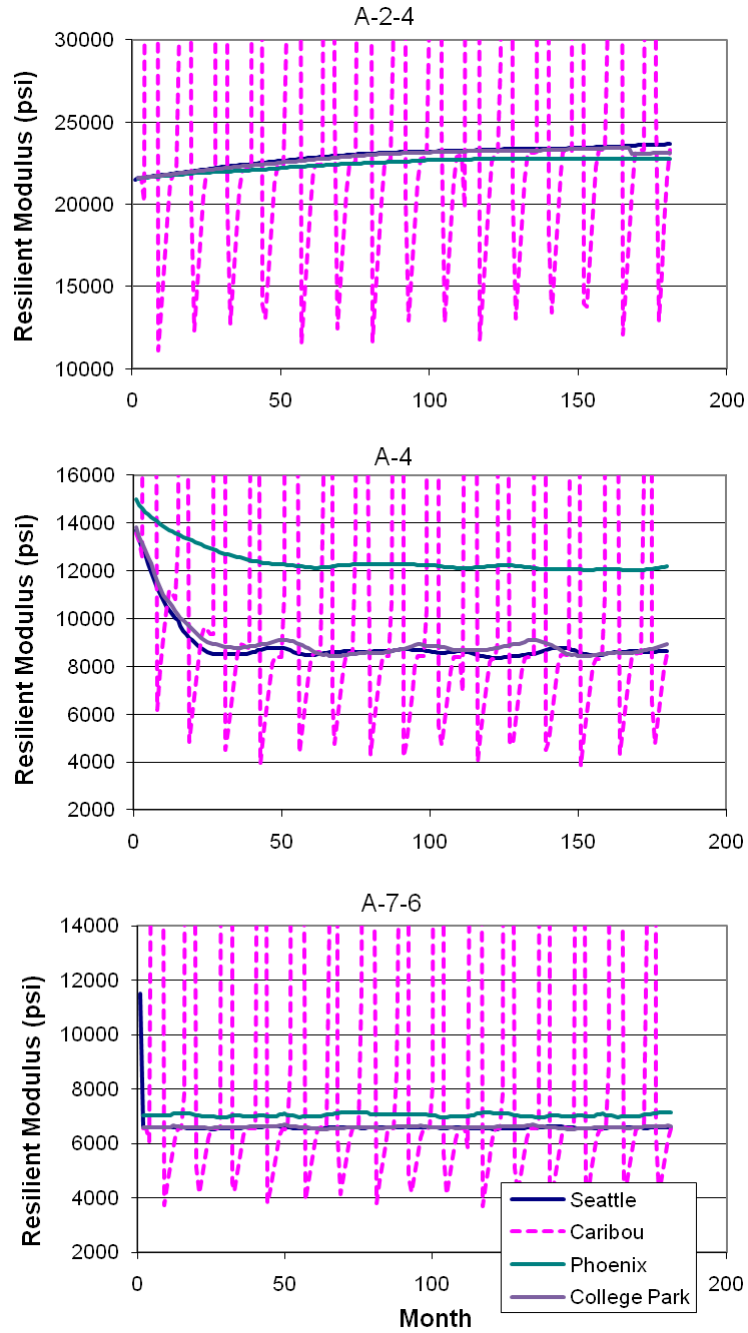
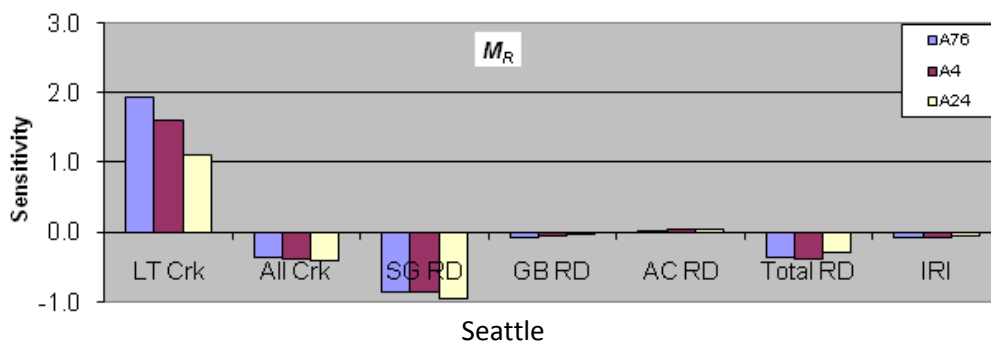


Figure 43. Average modulus of top 2 feet of subgrade vs. time (4 in. HMA thickness, 7 ft. GWT depth, medium traffic).

A local sensitivity index S_{ji} for quantifying the effect on performance of varying an analysis input X_i (where X is the vector of analysis inputs) around some reference condition X_{iR} can be defined in normalized terms as (Saltelli *et al.*, 2000):

$$S_{ji} = \frac{\partial D_j}{\partial X_i} \left(\frac{X_{iR}}{D_{jR}} \right) \quad (8)$$

in which D_j is the magnitude of distress j predicted using the input X_i and D_{jR} is the distress magnitude predicted using the reference input X_{iR} . Equation (8) can be interpreted as the percentage change in distress D_j caused by a given percentage change in input X_i . Figure 40 summarizes for all predicted distresses (except transverse thermal cracking) the normalized S_{ji} values computed using Eq. (8) for varying subgrade resilient modulus $M_R \pm 50\%$ from reference. The reference resilient modulus values are 11500 psi for A-7-6, 16500 psi for A-4, and 21500 psi for A-2-4. Several trends can be observed in these results: (a) longitudinal cracking (LT Crk) is most sensitive to M_R , and subgrade rutting (SG RD) is second; (b) longitudinal cracking is more sensitive in warm climates (e.g., Phoenix) than in cold (e.g., Caribou); (c) longitudinal cracking is more sensitive to M_R at higher reference moduli for all locations except Caribou.; and (d) granular base rutting (GB RD), asphalt concrete rutting (AC RD) and IRI are insensitive to M_R .



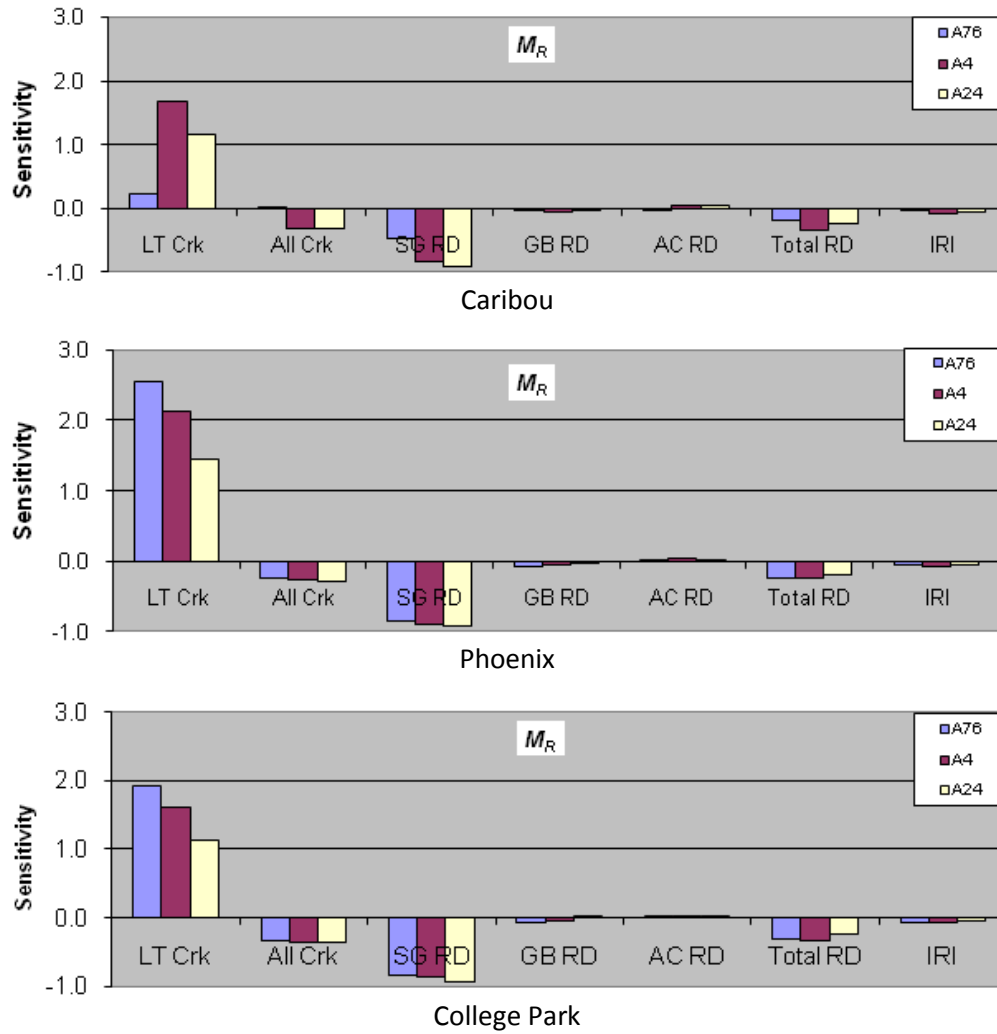


Figure 44. Sensitivity to M_R for all distresses at all locations.

Figure 45 and Figure 46 illustrate the influence of climate conditions in terms of average absolute sensitivity values—i.e., the average of the absolute values of the sensitivity indices across all distresses. The average sensitivity to subgrade modulus (Figure 45) shows some variation among sites, but this is sensible. The highest sensitivity in Seattle is likely due to the fact that the moisture-adjusted moduli there are the lowest of the four sites. Conversely, the relatively low sensitivity of average performance to subgrade modulus for Caribou may be attributed to the frozen stiff

state of the subgrade for much of the year. Subgrade conditions in Phoenix and College park are arguably intermediate between these two extremes, which is consistent with their intermediate sensitivity values.

The variations in sensitivity of average performance to environmental inputs (Figure 46) are all much smaller than for subgrade modulus (Figure 45). Depths to the groundwater table, soil-water characteristic curve parameters, and saturated hydraulic conductivity all have minimal influence on the predicted pavement performance.

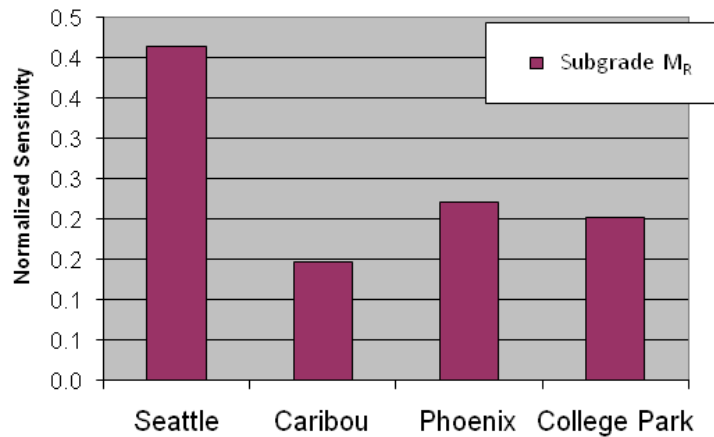


Figure 45. Average absolute sensitivity to subgrade M_R .

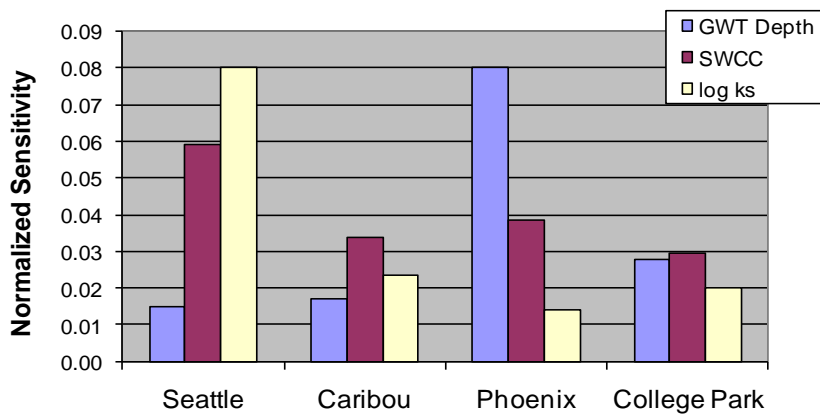


Figure 46. Average absolute sensitivity to environmental variables.

The sensitivity study presented here had a very confined scope, and therefore one must be cautious in drawing any far-reaching conclusions. However, based upon the limited scenarios investigated here, the following observations can be made:

- M_R was the input of those studied that had the largest impact on predicted distresses;
- The unbound material hydraulic inputs (GWT depth, *SWCC* parameters, and k_s) all had slight to negligible influence on the predicted distresses;
- Variations of performance with climate location and subgrade type were sensible.

These findings will need to be supplemented by those from other scenarios before any truly robust conclusions can be drawn. However, even these limited sensitivity studies serve the valuable purpose of confirming that the Level 3 defaults for the unbound material hydraulic properties (GWT depth, *SWCC* parameters, and k_s) should be suitable for design. This is fortunate, as these properties (except perhaps for project-specific GWT depth) are not currently measured by SHA.

Summary

Testing Recommendations

Recommendations for future testing of unbound materials by SHA are as follows:

1. SHA should perform further investigations to determine why the k_1 , k_2 , and k_3 values computed from their laboratory-measured resilient modulus test data do not follow the expected physical trends. The causes may be either due to testing issues or unusual characteristics of the specific materials included in the database (e.g., cemented sands, highly overconsolidated clays, or other extreme /unusual soil conditions).

2. SHA should continue to perform laboratory resilient modulus tests on common unbound materials in the state to augment and fill gaps in the database. The current database has a reasonable amount of measured resilient modulus data for subbase materials (e.g., A-2-4) and some subgrade soils (A-4, A-6). However, it is deficient in measured data for granular base materials (e.g., A-1-a and A-1-b) and the poorer subgrade soils (A-7-5, A-7-6). Similar to the recommendations for asphalt binder and HMA testing, the resilient modulus testing for unbound materials could be included as a matter of policy (perhaps as part of the contract requirements) for large/important/expensive paving projects in the state. This testing could continue to be performed by SHA in its own laboratories or outsource to third-party laboratories.

3. There is no need for SHA to begin any testing program for the hydraulic properties of unbound materials. These properties have very little impact of predicted pavement performance, and the empirical correlations in terms of gradation and plasticity parameters built into the MEPDG provide sufficient accuracy.

Recent results from NCHRP Project 9-23A “A National Catalog of Subgrade Soil-Water Characteristic Curve (SWCC) Default Inputs and Selected Soil Properties for Use with the ME-PDG” were made available to the public as NCHRP Web Document 153 in September 2010 after the draft final report had been submitted to SHA for review (Zapata, 2010). Although the hydraulic properties of unbound materials have been found to have little influence on predicted pavement performance for Maryland conditions, the information in this thesis could be mined for additional default values if desired. Key primary information in the NCHRP 9-23A database include: AASHTO soil class; limited gradation data; Atterberg limits; saturated hydraulic conductivity; saturated volumetric water content; and volumetric water

content at various levels of matric suction (used to compute soil water characteristic curve--SWCC). In addition to this primary information, there are also many secondary quantities in the database that are derived from the primary information, e.g., SWCC parameters and CBR estimated from correlations with gradation and plasticity, M_R estimated from (estimated) CBR, etc. Data are organized by “map unit,” a geographical area over which the soil properties are assumed roughly uniform. There are a total of 568 map units for the state of Maryland; 79 of these have no information in the database. Of the remainder, it is expected that many will have only partial information. For example, on a national level only 66% of the map units have measured soil water characteristic curve data. Unfortunately, detailed evaluation of the Maryland data could not be performed in this study because the information became available to the public only at the very end of the project, approximately 2 months after the draft final report was submitted to SHA.

Recommended MEDPG Inputs

The recommended unbound material inputs to the MEDPG for Maryland conditions are summarized in Table 51. Average gradation, plasticity, and volumetric information for Maryland materials as obtained from the data supplied by SHA is summarized in Table 54. The limited number of measured values for subgrade M_R in Table 54 correspond to the data in Table 49 at optimum moisture and 95% compaction. Note that these values are almost all significantly lower than the MEDPG level 3 defaults and should therefore be used with caution.

Table 51. Recommendation material property inputs for unbound materials for Maryland conditions.

Property	Input Level	Value	Comment
Unbound material	All	Project specific	Material class (e.g., AASHTO)
Thickness	All	Project specific	
<i>Strength Properties (ICM Calculated Modulus)</i>			
Poisson's ratio	3	Material specific	Use level 3 defaults. Table 52 provides additional guidance.
Coefficient of lateral pressure	3	Material specific	Use level 3 defaults. Table 53 provides additional guidance.
Modulus	2/3	Material specific	Use level 3 defaults or values from Table 54 where available.
<i>ICM (Mean Values)</i>			
Gradation	2/3	Material specific	Use values from Table 54 or level 3 defaults.
Plasticity Index	2/3	Material specific	Use values from Table 54 or level 3 defaults.
Liquid Limit	2/3	Material specific	Use values from Table 54 or level 3 defaults.
Compacted	All	Project/layer specific	
Maximum dry unit weight	2/3	Material specific	Use values from Table 54 or level 3 defaults.
Specific gravity	2/3	Material specific	Use level 3 defaults.
Saturated hydraulic conductivity	3	Material specific	Use level 3 defaults.
Optimum gravimetric water content	2/3	Material specific	Use values from Table 54 or level 3 defaults.
Degree of saturation at optimum	2/3	Material specific	Use values from Table 54 or level 3 defaults.
Soil-water characteristic curve parameters (a_f, b_f, c_f, h_f)	3	Material specific	Use level 3 defaults.

Table 52. Typical Poisson's ratio values for unbound granular and subgrade materials (NCHRP, 2004).

Material Description	μ_{Range}	μ_{Typical}
Clay (saturated)	0.4—0.5	0.45
Clay (unsaturated)	0.1—0.3	0.2
Sandy clay	0.2—0.3	0.25
Silt	0.3—0.35	0.325
Dense sand	0.2—0.4	0.3
Coarse-grained sand	0.15	0.15
Fine-grained sand	0.25	0.25
Bedrock	0.1—0.4	0.25

Table 53. Typical coefficient of lateral pressure for unbound granular, subgrade, and bedrock materials (NCHRP, 2004).

Material Description	Angle of Internal Friction, ϕ	Coefficient of Lateral Pressure, k_s
Clean sound bedrock	35	0.495
Clean gravel, gravel-sand mixtures, and coarse sand	29 to 31	0.548 to 0.575
Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	24 to 29	0.575 to 0.645
Clean fine sand, silty or clayey fine to medium sand	19 to 24	0.645 to 0.717
Fine sandy silt, non plastic silt	17 to 19	0.717 to 0.746
Very stiff and hard residual clay	22 to 26	0.617 to 0.673
Medium stiff and stiff clay and silty clay	19 to 19	0.717

Table 54. Average properties for Maryland unbound materials (based on material property database at time of report).

Class	N	LL (%)	PI (%)	% < No. 4	% < No. 200	OMC (%)	S at OMC (%)	Max Dry Unit Weight (pcf)	Subgrade M_R ¹ (psi)
A-1-b	5	23.8	7.0	60	15	6.7	53.6	135.3	
A-2-4	42	24.2	8.7	97	28	8.9	60.3	129.5	10,000
A-2-6	12	26.0	13.6	89	29	8.9	59.0	128.3	10,000
A-3	4			99	8	10.8	58.2	111.3	
A-4	96	29.5	8.0	99	48	11.8	67.8	122.5	6,000
A-5	3	41.0	8.0	99	71	15.6	57.2	112.6	
A-6	34	31.0	12.1	99	54	12.3	76.1	121.5	5,500
A-7-5	12	46.0	14.5	100	57	16.0	75.1	114.3	8,000
A-7-6	6	49.5	22.0	100	64	16.8	76.8	110.1	7,500

¹These values are significantly lower than the MEPDG level 3 defaults in most cases.

CHAPTER 6: MATERIAL PROPERTIES DATABASE

Introduction

MatProp is a MEPDG data management system based on Microsoft Access[®] 2007. It incorporates data entry, editing, and storing functionality for the material property inputs required by the MEPDG as well as additional data maintained by SHA. *MatProp* displays the data in a format similar to the MEPDG Version 1.100 data entry screens. The overall organization of *MatProp* is diagrammed in Figure 47.

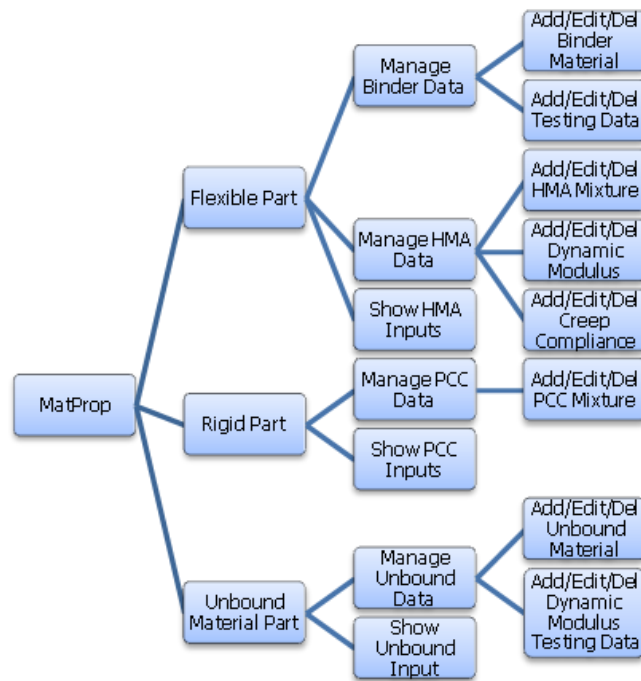


Figure 47. Organization of *MatProp* database.

MatProp is composed of 3 main sections: flexible, rigid, and unbound materials. The flexible section includes both binder and HMA related data; the rigid section includes PCC related data, and the unbound section includes material property data for granular base and subgrade materials.

Instructions for Using MatProp

Installation of *MatProp* consists of simply unzipping the “MatProp.zip” archive. This creates a folder named “MatProp System” containing 3 files: “MatProp.mdb”, the actual database; “Mouse Hook.dll,” a utility for increasing mouse functionality while within the database; and “Readme.txt,” which contains any release and/or installation notes. Double clicking “MatProp.mdb” opens *MatProp*. Depending upon the security settings of the host computer, the security warning shown in Figure 48 may be displayed (it may be hidden behind the main menu); if so, simply click “Options...”, choose “Enable this content” as shown in Figure 49, and click OK. After that, the main menu appears as shown in Figure 50.

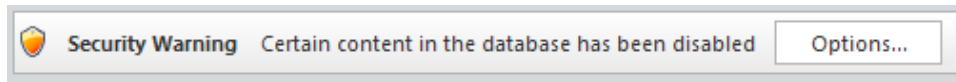


Figure 48. Security warning.

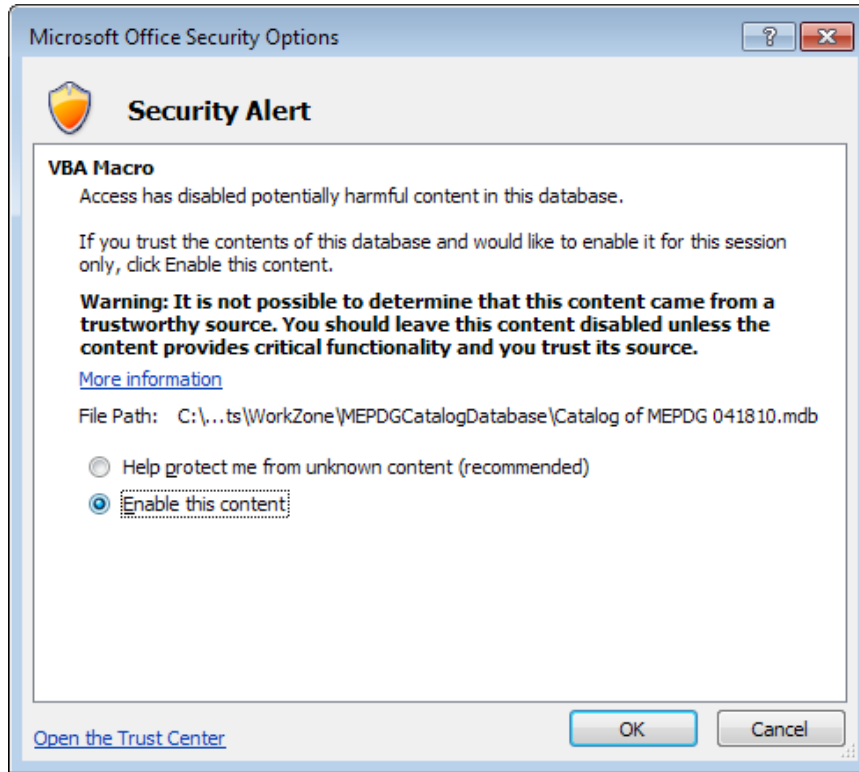


Figure 49. Security alert.

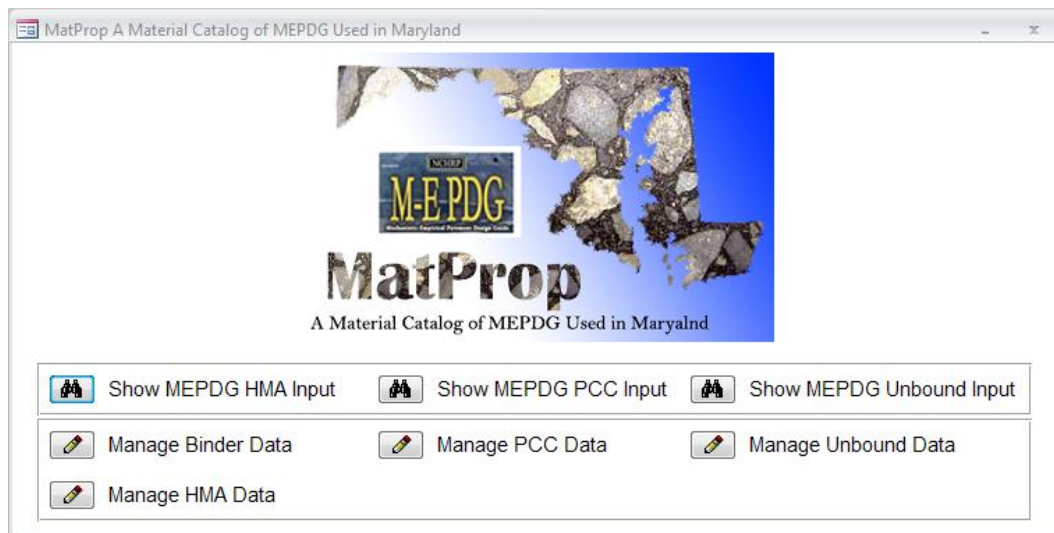


Figure 50. Main menu.

There are 3 buttons in the upper rectangle portion of the main menu that open screens that display the input data required by the MEPDG. Examples of these are shown in Figure 51 to Figure 55 for HMA materials after clicking “Show MEPDG HMA Input.” Note that the data display screens closely mirror the appearance of the corresponding data entry screen in the MEDPG Version 1.100. Similar MEPDG input screens are provided for the PCC and unbound material categories.

The 4 buttons in the lower rectangle portion of the main menu are used for data entry and management. The data entry and management functionality for the different material categories is described in the following subsections.

User Interface for Flexible Pavement Material Management

Clicking “Manage Binder Data” will bring up binder edit form shown in Figure 56. Binder data can be added or edited using the form in Figure 57. For consistency with the data received from SHA, the ID of binder data is numeric. Suppliers and terminals can also be added/edited/deleted as shown from Figure 58 to Figure 62. Data integrity checking is enforced as shown in Figure 63 and Figure 64. Binders can be edited by clicking “Edit” besides binder list in Figure 56 as shown in Figure 65. If “Delete” is clicked without selecting a record, a warning (Figure 66) will pop up.

Clicking “Manage HMA Data” brings up the HMA data management form shown in Figure 67. New HMA mixtures can be added as shown in Figure 68. Dynamic modulus testing data can be managed as shown in Figure 69. Creep compliance data management is shown in Figure 71. If temperature is not specified beforehand, a warning will pop up as shown in Figure 70. Note that records can be

excluded from calculations of average mixture properties (for a given nominal maximum aggregate size and mix type) by simply changing the “excluded” control to “Yes”.

Please choose an Asphalt Mix and an input level from below:

Level: Choose an Asphalt Mix:

Asphalt Mix | Asphalt Binder | AsphaltGeneral

Dynamic Modulus Table

For Number of Temperature and Frequency, please count in the table below.

Temp. (°F)	Modulus E*(psi)							
	1.0	2.0	3.0					
1	1.100E+01	1.200E+01	1.300E+01					
2			2.300E+01					
3			3.300E+01					

Thermal Cracking

Loading Time sec	Creep Compliance (1/psi)		
	LowTemp(C)	MedTemp(C)	HighTemp(C)
	0	0	0

In most cases, the MEPDG automatically calculates the Thermal Cracking.

For Level 1 and 3, simply input as shown on the left. For Level 2, just input MedTemp data.

Figure 51. “Show MEDPG HMA Input” screen for level 1 Asphalt Mix properties.

MEPDG HMA Input

Please choose an Asphalt Mix and an input level from below:

Level: Type: Max Aggr. Size: mm

Asphalt Mix | Asphalt Binder | AsphaltGeneral

Aggregate Gradation

Cumulative % Retained 3/4 inch	<input type="text" value="0.0"/>
Cumulative % Retained 3/8 inch	<input type="text" value="15.4"/>
Cumulative % Retained #4	<input type="text" value="45.0"/>
% Passing #200 sieve:	<input type="text" value="5.3"/>

Thermal Cracking

Loading Time sec	Creep Compliance (1/psi)		
	LowTemp(C)	MedTemp(C)	HighTemp(C)
	0	1	0

In most cases, the MEPDG automatically calculates the Thermal Cracking.

For Level 1 and 3, simply input as shown on the left. For Level 2, just input MedTemp data.

Figure 52. “Show MEPDG HMA Input” screen for level 2/3 Asphalt Mix properties.

MEPDG HMA Input

Please choose an Asphalt Mix and an input level from below:

Level: Choose an Asphalt Mix:

Asphalt Mix | **Asphalt Binder** | AsphaltGeneral

Please choose a Binder ID.
BinderID

Please choose from data listed below.

Temperature (C)	Angular frequency = 10 rad/sec		Count
	G* (Pa)	Delta (degree)	

Thermal Cracking

Loading Time sec	Creep Compliance (1/psi)		
	LowTemp(C)	MedTemp(C)	HighTemp(C)
	0	0	0

In most cases, the MEPDG automatically calculates the Thermal Cracking.
For Level 1 and 3, simply input as shown on the left. For Level 2, just input MedTemp data.

Figure 53. “Show MEPDG HMA Input” screen for level 1/2 Asphalt Binder properties.

MEPDG HMA Input

Please choose an Asphalt Mix and an input level from below:

Level: Type: Max Aggr. Size: mm

Asphalt Mix | **Asphalt Binder** | AsphaltGeneral

Click the appropriate Binder Grade.

Thermal Cracking

Loading Time sec	Creep Compliance (1/psi)		
	LowTemp(C)	MedTemp(C)	HighTemp(C)
	0	1	0

In most cases, the MEPDG automatically calculates the Thermal Cracking.

For Level 1 and 3, simply input as shown on the left. For Level 2, just input MedTemp data.

Figure 54. “Show MEPDG HMA Input” screen for level 3 Asphalt Binder properties.

MEPDG HMA Input

Please choose an Asphalt Mix and an input level from below:

Level: Choose an Asphalt Mix:

Asphalt Mix | Asphalt Binder | **AsphaltGeneral**

General
Reference temperature

Gravimetric Properties (Mx Design)
Binder content by
OBC (%)
Design air voids used to select OBC (%)

Volumetric Properties as Built
Effective binder
Air Void
Total unit weight

Poisson's Ratio
Poisson's Ratio:
Parameter
Parameter

Thermal Properties
Thermal conductivity
Heat capacity

Thermal Cracking

Loading Time sec	Creep Compliance (1/psi)		
	LowTemp(C)	MedTemp(C)	HighTemp(C)
	<input type="text" value="0"/>	<input type="text" value="0"/>	<input type="text" value="0"/>

In most cases, the MEPDG automatically calculates the Thermal Cracking.
For Level 1 and 3, simply input as shown on the left. For Level 2, just input MedTemp data.

Figure 55. "Show MEDPG HMA Input" screen for Asphalt General properties (all input levels).

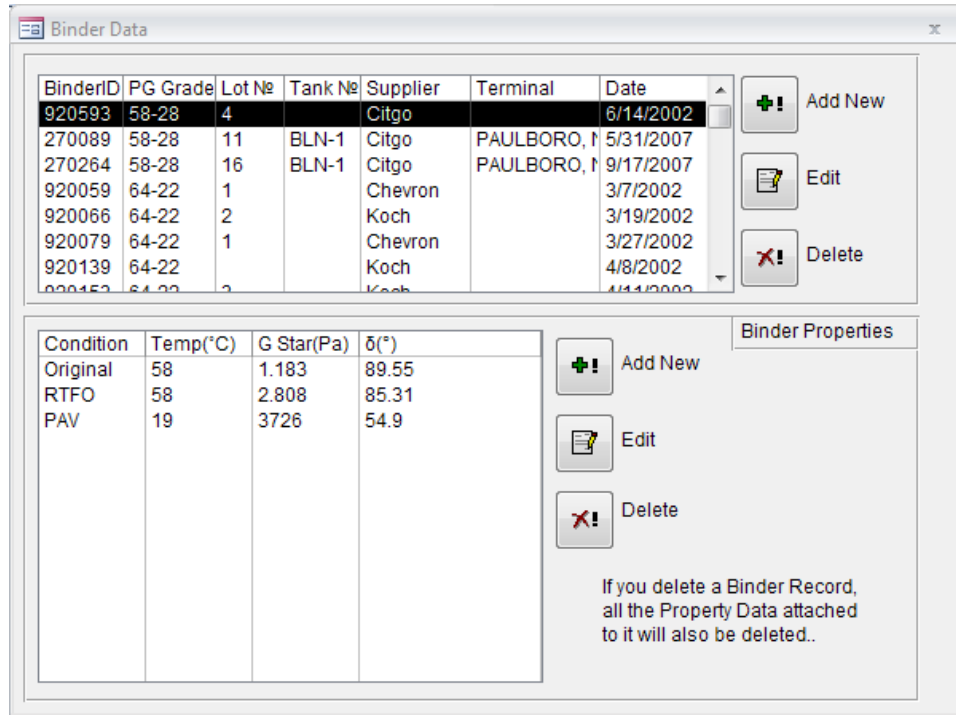


Figure 56. “Manage Binder Data” – main screen.

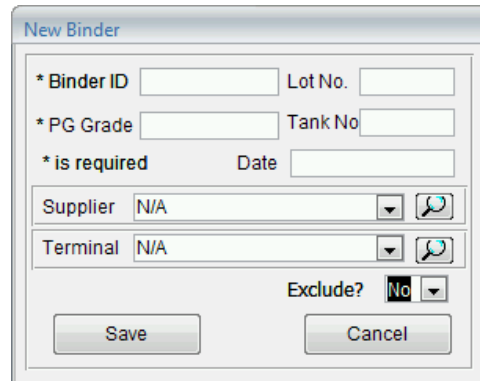


Figure 57. Add new binder.

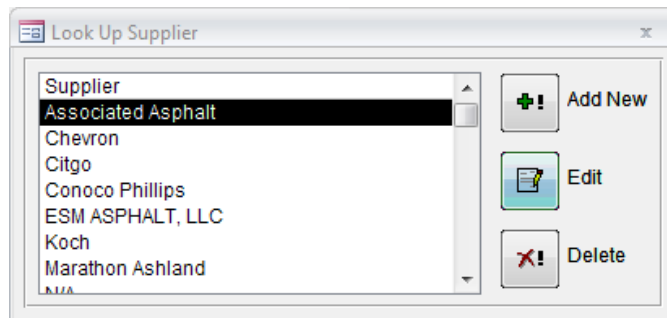


Figure 58. Look up binder supplier.

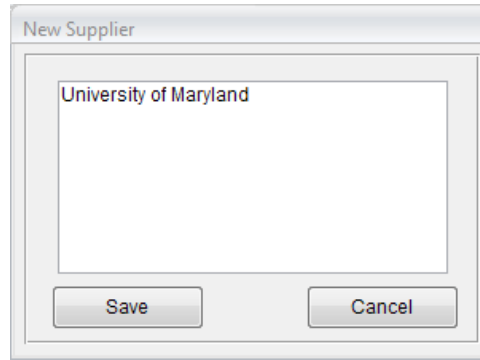


Figure 59. Add new supplier.

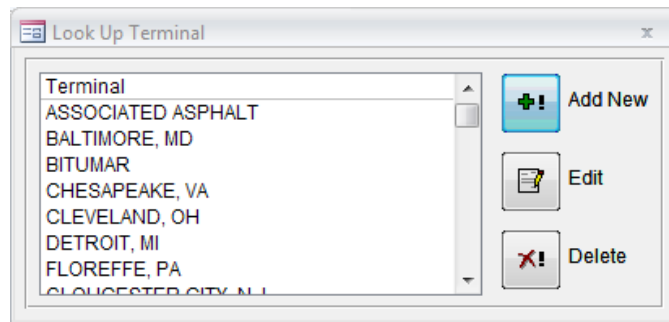


Figure 60. Look up terminal.

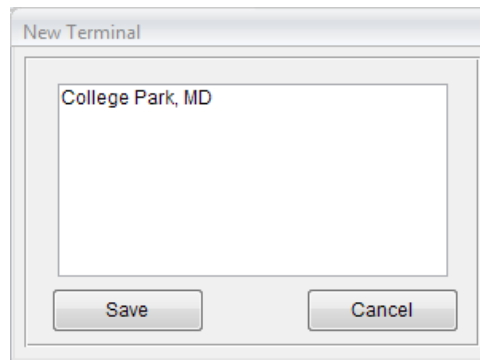


Figure 61. Add new terminal.

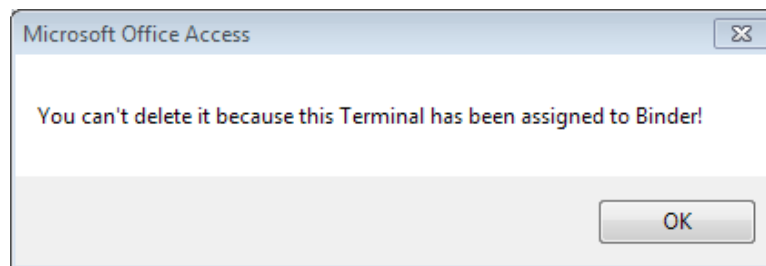


Figure 62. Data integrity checking before deleting terminal.

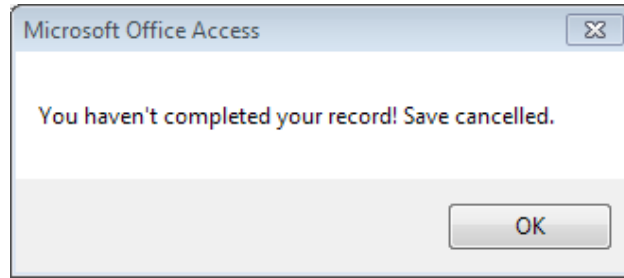


Figure 63. Saving without completion.

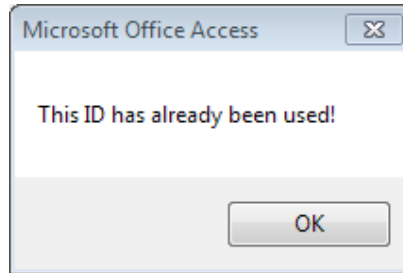


Figure 64. ID integrity checking.

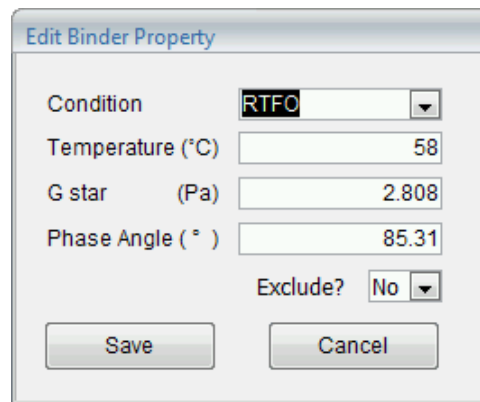


Figure 65. "Edit Binder Property" screen.

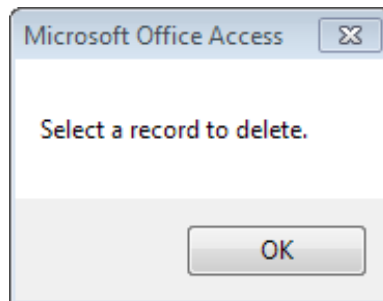


Figure 66. Delete without selection.

HMA Data

MixDesignID	Plant	Band	Type	Traffic	BinderGrade	MixNumber	Ninitial	NDesign	NMaximum
W13704V1C01	137	4.75	Virgin	< 0.3	64-22	01	6	50	75
N09004V1C01	090	4.75	Virgin	< 0.3	64-22	01	6	50	75
N13904V1C01	139	4.75	Virgin	< 0.3	64-22	01	6	50	75
W13704V1E01	137	4.75	Virgin	< 0.3	70-22	01	6	50	75
W16904V2C02	169	4.75	Virgin	0.3 to < 3.0	64-22	02	7	75	115
E13104V2C06	131	4.75	Virgin	0.3 to < 3.0	64-22	06	7	75	115
E13104V2C04	131	4.75	Virgin	0.3 to < 3.0	64-22	04	7	75	115
E13104V2C00	131	4.75	Virgin	0.3 to < 3.0	64-22	00	7	75	115

Add New
 Edit
 Delete

Aqqr. Gradation
 50.0mm
 37.5mm
 25.0mm
 19.0mm
 12.5mm
 9.5mm
 4.75mm
 2.36mm
 1.18mm
 0.60mm
 0.30mm
 0.15mm
 .075mm

Volumetric Information
 Gmm Pb
 Gse Pba
 Gmb Pbe
 Testing Va Inplace Va
 Vbe Vma
 yt(pcf) Vfa
 D/PbeRatio
 D/BRatio

Other Properties
 Poisson's Ratio
 Heat Capacity: (BTU/lb-ftF)
 Thermal Conductivity (BTU/hr-ftF)

Temp(°F) | Freq | Modulus E* (psi) | PhaseAngle(°)
 Add New
 Edit
 Delete

Loading Time (sec) | Creep Compliance (1/psi)
 LowTemp(C) | MedTemp(C) | HighTemp(C)
 0 | 1 | 0
 Add Nev
 Edit
 Delete

Tensile strength at 14 degree (psi):
 Aggregate coef. of thermal contraction:

Figure 67. "Manage HMA Data" – main screen.

New Mix Desing ID

* Mix Design ID Plant

* Binder Grade Mix No

N Initial N Maximum

N Design * is requird

Type

Traffic

Exclude?

Figure 68. Add HMA mixture.

Figure 69. Edit dynamic modulus data.

Figure 70. Attempt to add creep data without providing temperature.

Figure 71. Add creep compliance data.

User Interface for Rigid Pavement Material Management

The main PCC data management screen is shown in Figure 72. Integrity checking is the same as in other management forms. New PCC material can be added and edited as shown in Figure 73. The ID for PCC material is a character string. As for binders and HMA mixtures, integrity checking is enforced and no duplicate IDs

are allowed. Records can be excluded from average property calculations by simply changing the “excluded” control to “Yes”.

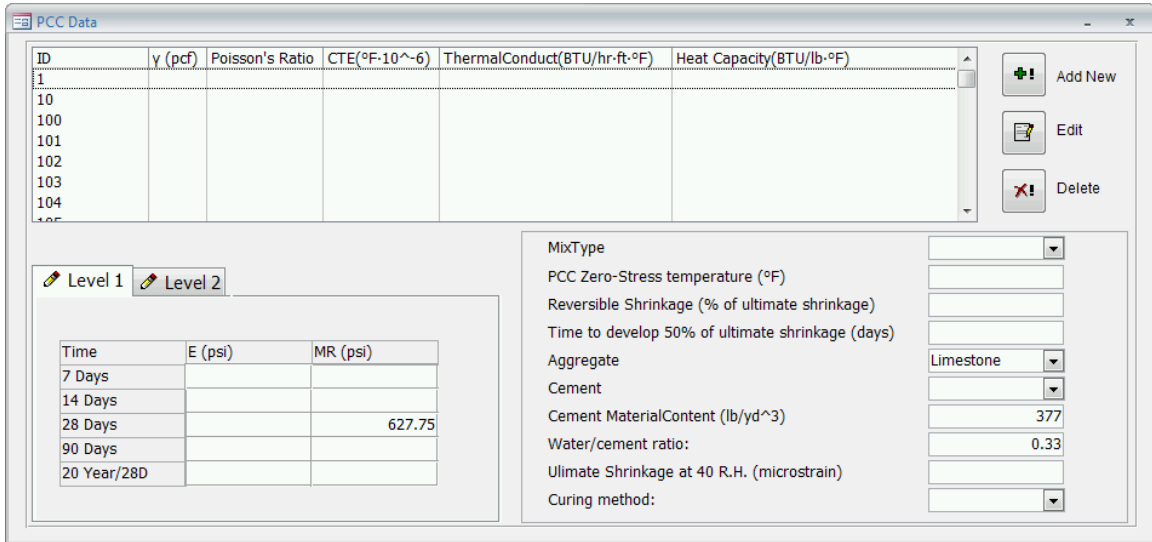


Figure 72. “Manage PCC Data” – main screen.

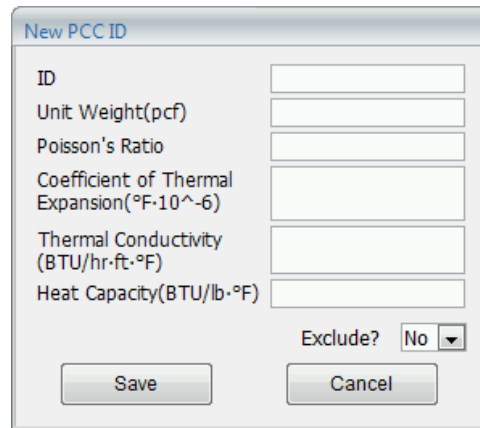


Figure 73. New PCC mixture.

User Interface for Unbound Material

Figure 74 shows the main data management screen for unbound materials. Integrity checking is similar to that in the other data management forms. The stress-dependent modulus *k* values can be calculated using the “Calculate Parameters” button. If the calculated values are not acceptable, they can be modified manually by

directly inputting the preferred values. Unbound material can be added and edited as shown in Figure 75 and Figure 76. Resilient modulus testing conditions (e.g., moisture and density as tested) can be added and edited as shown in Figure 77 and Figure 78. Resilient modulus data can be added and edited as shown in Figure 79 and Figure 80. Individual records can be excluded from the calculation of average parameter values for the soil class by changing the “excluded” control to “Yes”.

The screenshot displays the 'Unbound Data' software interface with several data tables and control panels:

- Master Record Table:**

Material ID	Class	Group	PRatio	Ko
PG260	A-1-b			
S141038	A-1-b			
SPG149	A-1-b			
SMO255	A-2-4	0		
SMO210	A-2-4	0		
SPG48	A-2-4	0		
SPG51	A-2-4	0		
SHO14	A-2-4	0		
- Testing Conditions Table:**

ConditionName	DryDensity	MoistureContent
AT 2% BELOW OPT II	109.9483	10.25641
AT OPTIMUM	109.2802	12.2342
- Mr Records Table:**

Chamber P (psi)	Deviator σ (psi)	Mr (psi)
2	2.585	6416
2	4.739	5390
2	6.752	4988
5	3.356	6530
5	4.848	5738
5	6.673	5107
5	9.677	4730
- ICM Mr Parameters:**

K1	K2	K3	Calculate Parameters
453.161	0.028	-1.121	
- Strength ICM Value:**

Mr (psi)	CBR (psi)
- Right Panel:** Contains input fields for parameters like PI, LL, Maximum Dry UnitWeight (pcf) (123.1), Gs (2.7), Sat. Hydraulic Conductivity (ft/hr), Opt. Gravity Water Content (%) (12.4), SatDegreeOpt (60.97629), and a list of testing conditions (#200 to #31/2).

Figure 74. “Manage Unbound Data” – main screen.

The 'New Unbound Material' dialog box contains the following fields and controls:

- ID:** Text input field.
- Classification:** Dropdown menu.
- PassoinRatio:** Text input field.
- Ko:** Text input field.
- Date:** Text input field with value '7/20/2010'.
- Exclude?:** Dropdown menu with 'No' selected.
- Group:** Text input field.
- Buttons:** 'Save' and 'Cancel' buttons.

Figure 75. Add unbound material.

Edit Unbound Material

ID: Group:

Classification:

PassoinRatio:

Ko:

Date:

Exclude?

Figure 76. Edit unbound material.

New Testing Condition

Condition Name:

Dry Density(pcf)

% Moisture

Exclude?

Figure 77. New testing condition.

Edit Testing Condition

Condition Name:

Dry Density(pcf)

% Moisture

Exclude?

Figure 78. Edit testing condition.

New Resilient Modulus Data

Chamber P (psi)

Deviator Stress(psi)

Mr (psi)

Exclude?

Figure 79. Add M_R data.

Edit Resilient Modulus Data	
Chamber P (psi)	2
Deviator Stress(psi)	7.653
Mr (psi)	6795
Exclude?	No
<input type="button" value="Save"/> <input type="button" value="Cancel"/>	

Figure 80. Edit M_R data.

Database Structure

The *MatProp* relational database contains a total of 33 tables. The relations among the binder tables are illustrated in Figure 81 to Figure 84. The tblBinderMaster and tblBinderProperties tables in Figure 81 store the main information about binder material. The tblBinderMaster table stores basic information for the binder while the tblBinderProperties table stores the binder testing data. One record in the tblBinderMaster table may have many related records in tblBinderProperties table. This is shown by the relationship lines that designate one-to-one, one-to-many, or many-to-many relationships between tables and the fields (e.g., ID fields) used to define these relationships. Deleting a record in tblBinderMaster table will also delete all the related record in the tblBinderProperties table. Other tables prefixed with c_ are look-up tables used to define the data entry codes in the main tables.

Figure 82 to Figure 84 similarly show relations among the HMA mixture, PCC mixture, and unbound materials tables, respectively. Again, table names having c_ prefixes are look-up tables while all other tables are main data tables. The relationship lines have the same meanings as in Figure 81.

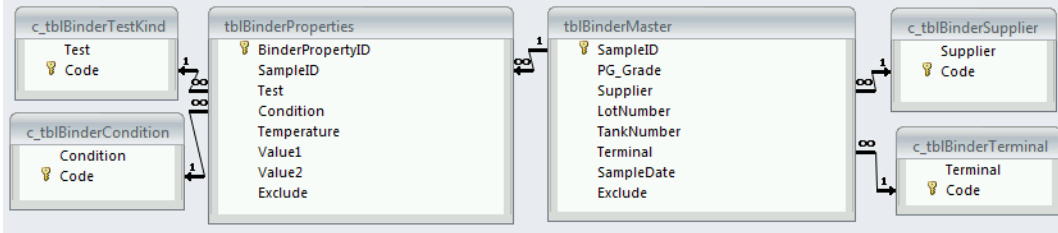


Figure 81. Tables and relations for binder material data.

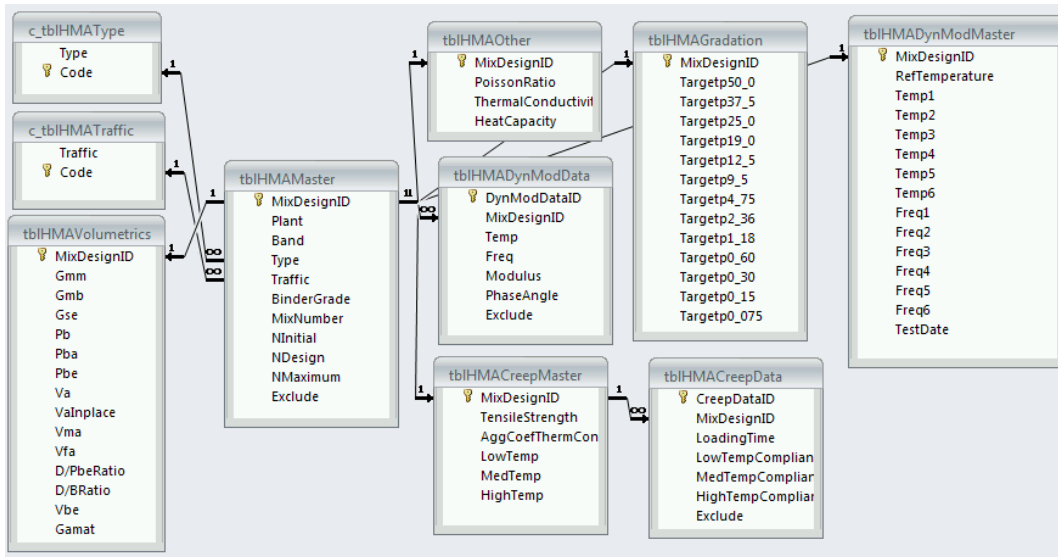


Figure 82. Tables and relations for HMA mixture data.

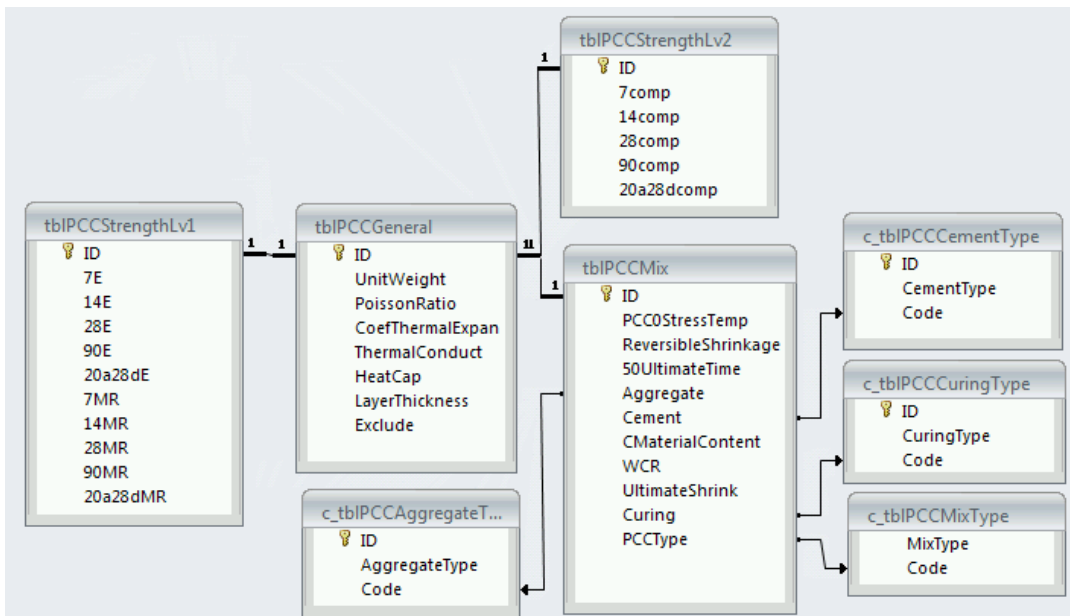


Figure 83. Tables and relations for PCC mixture data.

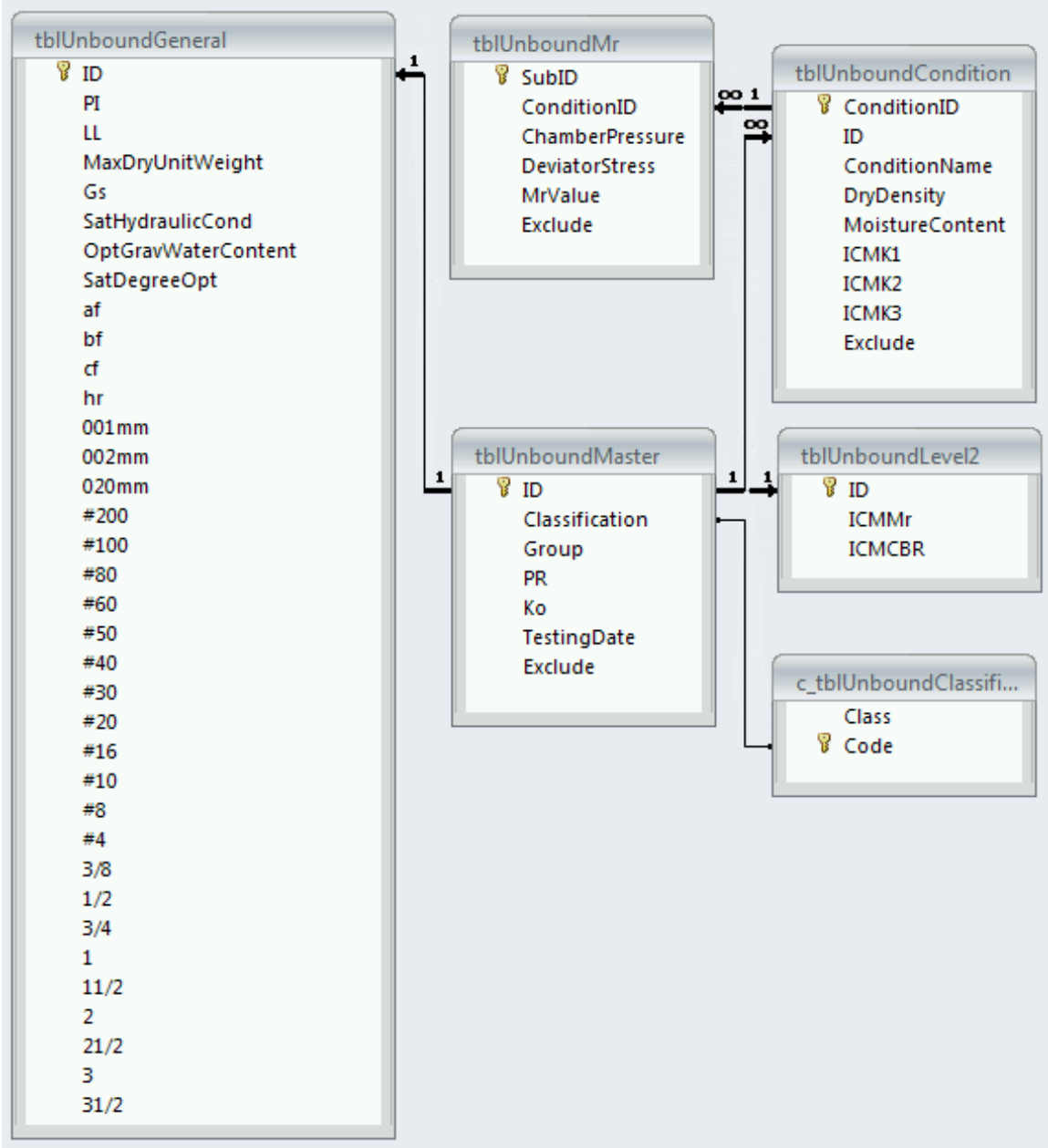


Figure 84. Tables and relations for unbound material data.

All tables and fields used in the database are summarized in the listing below:

Table Name	Field Name	Data Type	Size	Description
c_tblBinderCondition	Condition	Text	10	
	Code	Byte	1	
c_tblBinderSupplier	Supplier	Text	35	Supplier name should be less than 35 letters

	Code	Byte	1	
c_tblBinderTerminal	Terminal	Text	30	Terminal name should be less than 35 letters
	Code	Byte	1	
c_tblBinderTestKind	Test	Text	25	
	Code	Byte	1	
c_tblHMATraffic	Traffic	Text	13	
	Code	Byte	1	
c_tblHMAType	Type	Text	15	
	Code	Byte	1	
c_tblPCCAggregateType	ID	Long Integer	4	Auto Number so use Long Integer
	AggregateType	Text	10	
	Code	Byte	1	
c_tblPCCCementType	ID	Long Integer	4	Auto Number so use Long Integer
	CementType	Text	8	
	Code	Byte	1	
c_tblPCCCuringType	ID	Long Integer	4	Auto Number so use Long Integer
	CuringType	Text	16	
	Code	Byte	1	
c_tblPCCMixType	MixType	Text	10	
	Code	Byte	1	
c_tblUnboundClassification	Class	Text	40	
	Code	Byte	1	
tblBinderMaster	SampleID	Long Integer	4	The ID of test sample.
	PG_Grade	Text	6	The PG Grade of test sample.
	Supplier	Byte	1	Supplier of the test. See tblSupplierCode for detail.
	LotNumber	Text	30	Lot Number of test sample.
	TankNumber	Text	30	Tank Number of test sample.
	Terminal	Byte	1	Terminal of the test. See tblTerminalCode for detail.
	SampleDate	Date/Time	8	The date when test processed.
	Exclude	Yes/No	1	
tblBinderProperties	BinderPropertyID	Long Integer	4	The ID for the test data. Cannot be duplicate.
	SampleID	Long Integer	4	The ID for the test sample. CAN be duplicate.
	Test	Byte	1	Test kind of the data. See

				tblTestKindCode for detail.
	Condition	Byte	1	Testing condition. See tblCondition Code for detail.
	Temperature	Integer	2	The temperature in which test proceed.
	Value1	Single	4	RotationalViscosity in test No.1; GStar in test No.2; Stiffness in test No.3
	Value2	Single	4	Spindle size in test No. 1; Phase in test No.2; Mvalue in test No. 3.
	Exclude	Yes/No	1	Default value is No. If yes, the record is excluded from being used.
tblHMALCreepData	CreepDataID	Long Integer	4	The ID number of the data. Cannot be duplicate.
	MixDesignID	Text	11	The ID of the sample. CAN be duplicate here.
	LoadingTime	Single	4	In the same unit.eg: second.
	LowTempCompliance	Single	4	Compliance in low temperature.
	MedTempCompliance	Single	4	Compliance in medium temperature.
	HighTempCompliance	Single	4	Compliance in high temperature.
	Exclude	Yes/No	1	
tblHMALCreepMaster	MixDesignID	Text	11	The ID number of the sample. Cannot be duplicate.
	TensileStrength	Single	4	Unit:(psi)
	AggCoefThermContr	Single	4	Low temp in which creep compliance is tested.
	LowTemp	Single	4	Low temp in which creep compliance is tested.
	MedTemp	Single	4	Medium temp in which creep compliance is tested.
	HighTemp	Single	4	High temp in which creep compliance is tested.
tblHMALDynModData	DynModDataID	Long Integer	4	The ID number of the property data. Cannot be duplicate.
	MixDesignID	Text	11	The ID number of the mix. CAN be duplicate.
	Temp	Single	4	Temperature of the data testing.Related to tblHMALDynModMaster. = 1,2,3,4,5,6

	Freq	Single	4	Frequency of the data testing. Related to tblHMADynModMaster. = 1,2,3,4,5,6
	Modulus	Single	4	Mr of the sample.
	PhaseAngle	Single	4	Phase Angle of the sample.
	Exclude	Yes/No	1	
tblHMADynModMaster	MixDesignID	Text	11	The ID number of the mix.
	RefTemperature	Single	4	The temperature of testing. (F)
	Temp1	Single	4	Temperature appears at 1 st place in form
	Temp2	Single	4	Temperature appears at 2 nd place in form
	Temp3	Single	4	Temperature appears at 3 rd place in form
	Temp4	Single	4	Temperature appears at 4 th place in form
	Temp5	Single	4	Temperature appears at 5 th place in form
	Temp6	Single	4	Temperature appears at 6 th place in form
	Freq1	Single	4	Frequency appears at 1 st place in form
	Freq2	Single	4	Frequency appears at 2 nd place in form
	Freq3	Single	4	Frequency appears at 3 rd place in form
	Freq4	Single	4	Frequency appears at 4 th place in form
	Freq5	Single	4	Frequency appears at 5 th place in form
	Freq6	Single	4	Frequency appears at 6 th place in form
	TestDate	Date/Time	8	
tblHMAGradation	MixDesignID	Text	11	The ID number of the mix.
	Targetp50_0	Single	4	Volumetric Property
	Targetp37_5	Single	4	Volumetric Property
	Targetp25_0	Single	4	Volumetric Property
	Targetp19_0	Single	4	Volumetric Property
	Targetp12_5	Single	4	Volumetric Property
	Targetp9_5	Single	4	Volumetric Property
	Targetp4_75	Single	4	Volumetric Property
	Targetp2_36	Single	4	Volumetric Property
	Targetp1_18	Single	4	Volumetric Property

	Targetp0_60	Single	4	Volumetric Property
	Targetp0_30	Single	4	Volumetric Property
	Targetp0_15	Single	4	Volumetric Property
	Targetp0_075	Single	4	Volumetric Property
tblHMAMaster	MixDesignID	Text	11	The ID number of the record. Unique in this table.
	Plant	Text	3	The code of the Plant.
	Band	Single	4	The band length of the sample.
	Type	Byte	1	The Type of the sample. See tblType for detail.
	Traffic	Byte	1	The traffic level. See tblTraffic for detail.
	BinderGrade	Text	5	The binder grade of the Mix.
	MixNumber	Text	2	This is Mix Number.
	NInitial	Text	1	This is N initial.
	NDesign	Text	3	This is N design.
	NMaximum	Text	3	This is N maximum.
	Exclude	Yes/No	1	
tblHMAOther	MixDesignID	Text	11	The ID number of the record. Unique in this table.
	PoissonRatio	Single	4	
	ThermalConductivity	Single	4	unit: BTU/(hr*ft*F)
	HeatCapacity	Single	4	unit: BTU/(lb*F)
tblHMAVolumetrics	MixDesignID	Text	11	The ID number of Mix.
	Gmm	Single	4	bulk-specific gravity of a compacted asphalt mixture
	Gmb	Single	4	theoretical maximum specific gravity
	Gse	Single	4	aggregate-effective specific gravity
	Pb	Single	4	asphalt content with respect to the total mixture weight
	Pba	Single	4	asphalt absorbed with respect to the total mixture weight
	Pbe	Single	4	effective asphalt content with respect to the total mixture weight
	Va	Single	4	Air void as tested
	VaInplace	Single	4	Air void as in place condition
	Vma	Single	4	Voids in the mineral aggregate
	Vfa	Single	4	Voids Filled with Asphalt

	D/PbeRatio	Single	4	The ratio of D over Pbe
	D/BRatio	Single	4	The ratio of D over B
	Vbe	Single	4	Volume of effective asphalt binder
	Gamat	Single	4	total unit weight
tblLevels	ID	Long Integer	4	
	Field1	Byte	1	Levels used in forms.
tblPCCAggregateCTE	AggregateID	Byte	1	Aggregate code, see <u>c_tblPCCAggregateType</u>
	CTE	Single	4	CTE of that aggregate
tblPCCGeneral	ID	Text	11	PCC ID
	UnitWeight	Single	4	in pcf
	PoissonRatio	Single	4	
	CoefThermalExpansion	Single	4	Thermal Conductivity
	ThermalConduct	Single	4	Heat Capacity
	HeatCap	Single	4	Heat Capacity
	LayerThickness	Long Integer	4	May not be used
	Exclude	Yes/No	1	
tblPCCMix	ID	Text	11	PCC ID
	PCC0StressTemp	Single	4	hard to test
	ReversibleShrinkage	Single	4	hard to test
	50UltimateTime	Single	4	hard to test
	Aggregate	Byte	1	Aggregate type used in the PCC
	Cement	Byte	1	Cement type used
	CMaterialContent	Single	4	Cement content
	WCR	Single	4	Water Cemenet Ratio
	UltimateShrink	Single	4	hard to test
	Curing	Byte	1	Curing Method.
	PCCType	Byte	1	Regular or Recycle
tblPCCStrengthLv1	ID	Text	11	ID of PCC
	7E	Single	4	E at 7 day
	14E	Single	4	Similar to above
	28E	Single	4	Similar to above
	90E	Single	4	Similar to above
	20a28dE	Single	4	E Ratio of 20year over 28 day
	7MR	Single	4	Modulus of Rupture at 7 day
	14MR	Single	4	Similar to above
	28MR	Single	4	Similar to above

	90MR	Single	4	Similar to above
	20a28dMR	Single	4	Modulus of Rupture Ratio of 20year over 28 day
tblPCCStrengthLv2	ID	Text	11	PCC ID
	7comp	Single	4	Compressive Strength at 7 day
	14comp	Single	4	Similar to above
	28comp	Single	4	Similar to above
	90comp	Single	4	Similar to above
	20a28dcomp	Single	4	Compressive Strength Ratio of 20year over 28 day
tblUnboundCondition	ConditionID	Long Integer	4	ID for each unbound material condition
	ID	Text	11	ID according to tblUnboundMaster(master)
	ConditionName	Text	30	Name of the conditon i.e. Optimum
	DryDensity	Single	4	The dry density of this condition
	MoistureContent	Single	4	The dry density of this condition
	ICMK1	Single	4	Regression factor K1
	ICMK2	Single	4	Regression factor K2
	ICMK3	Single	4	Regression factor K3
	Exclude	Yes/No	1	
tblUnboundGeneral	ID	Text	11	A unique ID for each unbound material sample.
	PI	Single	4	Plastic Index
	LL	Single	4	Liquid limit
	MaxDryUnitWeight	Single	4	As field name(pcf)
	Gs	Single	4	Specific Gravity
	SatHydraulicCond	Single	4	Saturation Hydraulic Conductivity. (ft/hr)
	OptGravWaterContent	Single	4	In %
	SatDegreeOpt	Single	4	In %
	af	Single	4	Parameters defining SWCC
	bf	Single	4	Parameters defining SWCC
	cf	Single	4	Parameters defining SWCC
	hr	Single	4	Parameters defining SWCC
	001mm	Single	4	Volumetric Property
	002mm	Single	4	Volumetric Property
	020mm	Single	4	Volumetric Property
	#200	Single	4	Volumetric Property

	#100	Single	4	Volumetric Property
	#80	Single	4	Volumetric Property
	#60	Single	4	Volumetric Property
	#50	Single	4	Volumetric Property
	#40	Single	4	Volumetric Property
	#30	Single	4	Volumetric Property
	#20	Single	4	Volumetric Property
	#16	Single	4	Volumetric Property
	#10	Single	4	Volumetric Property
	#8	Single	4	Volumetric Property
	#4	Single	4	Volumetric Property
	3/8	Single	4	Volumetric Property
	1/2	Single	4	Volumetric Property
	3/4	Single	4	Volumetric Property
	1	Single	4	Volumetric Property
	1 1/2	Single	4	Volumetric Property
	2	Single	4	Volumetric Property
	2 1/2	Single	4	Volumetric Property
	3	Single	4	Volumetric Property
	3 1/2	Single	4	Volumetric Property
tblUnboundLevel2	ID	Text	11	A unique ID for each unbound material sample.
	ICMMr	Single	4	Resilient Modulus of Integrated Climate Model
	ICMCBR	Single	4	CBR of Integrated Climate Model
tblUnboundMaster	ID	Text	11	A unique ID for each unbound material sample.
	Classification	Byte	1	Code for Classification
	Group	Byte	1	Group number for SHA testing convenience.
	PR	Single	4	Poisson's Ratio
	Ko	Single	4	Coefficient of lateral pressure
	TestingDate	Date/Time	8	
	Exclude	Yes/No	1	
tblUnboundMr	SubID	Replication ID	16	A unique ID for each Mr testing.
	ConditionID	Long Integer	4	ID according to the testing condition
	ChamberPressure	Single	4	ChamberPressure (psi)
	DeviatorStress	Single	4	DeviatorStress (psi)
	MrValue	Single	4	Average Resilient Modulus

				(psi)
	Exclude	Yes/No	1	
tblYesNo	codeYesNo	Yes/No	1	0→No, -1→Yes
	txtYesNo	Text	3	text "Yes" or "No"

CHAPTER 7: SUMMARY OF RECOMMENDATIONS

Project Summary

The objective of the study described in this thesis was to develop this type of organized database of material properties for the most common paving materials used in Maryland. Separate chapters for each of the major pavement material types (asphalt binder, HMA mixtures, PCC mixtures, unbound materials) detail the following essential information for understanding and using the MEPDG:

MEPDG Input Requirements

- New Construction/Reconstruction/Overlays

- Rehabilitation (Existing Layers)

Data Available from Maryland SHA

Analyses of MEDPG Inputs

- Level 1 vs. Level 2 vs. Level 3

- Sensitivity Analyses

Summary

- Testing Recommendations

- Recommended MEPDG Inputs

A comprehensive material property database developed in Microsoft Access 2007 accompanies this thesis. This database is initially populated with all information receive from SHA. It provides complete data management tools for adding future data as well as data display screens for MEDPG inputs that mirror the input screens in the MEPDG Version 1.100 software.

The detailed recommendations from this study for the Maryland-specific MEDPG input values are summarized in the last sections of Chapters 2 through 5 for asphalt binder, HMA mixtures, PCC mixtures, and unbound materials, respectively. These recommendations will not be repeated here. However, the recommendations for future MEDPG-related materials testing by SHA are compiled below for the convenience of SHA materials engineers and policy makers.

Testing Recommendations

Asphalt Binders

1. Maryland SHA currently has no Level 1/2 input data for asphalt binders. The sensitivity of predicted pavement performance to Level 1 vs. Level 2 vs. Level 3 binder inputs appears slight. Based only on this criterion there would be little purpose for SHA collection of Level 1 or 2 binder data. However, input of Level 1 HMA properties also requires input of Level 1/2 binder data. It is therefore recommended that SHA develop a policy of full binder characterization on major projects and that the test results be entered into the material property database so that typical Level 1/2 properties can be input into the MEDPG in the future. The testing frequency for full binder characterization should match the recommendations for HMA dynamic modulus testing.

HMA Mixtures

2. Maryland SHA currently has no Level 1 input data for HMA mixtures. There is the potential for significant differences in predicted performance using Level 1 vs. Level 2/3 dynamic modulus data. In addition, the Witczak predictive equation used to generate the Level 2/3 dynamic modulus data is not intended for SMA mixtures, a common premium mixture type in Maryland, and often does not differentiate among different dense graded

mixtures adequately. Therefore, SHA should plan to begin measuring Level 1 dynamic modulus data over time for the most commonly used mixture types in conjunction with major paving projects. Level 1 dynamic modulus testing of HMA mixtures will also require companion Level 1 characterization of the asphalt binders.

It is recommended that SHA develop a policy requiring Level 1 HMA dynamic modulus and binder characterization testing for all major projects. Major projects could be defined by SHA in terms of a minimum placement tonnage, minimum traffic volume, or some other measure of project/mix importance. This testing could be done in-house using either the UTM-25 or AMPT test systems in the SHA laboratories; however, some equipment repair and/or calibration would be required as both of these systems are currently nonoperational. This testing could also be outsourced to local commercial testing facilities (e.g., Advanced Asphalt Technologies, LLC) and/or the University of Maryland (HMA dynamic modulus testing only).

3. There is no need for measuring Level 1 creep compliance, low temperature tensile strength, and aggregate coefficient of thermal contraction properties. These properties are used only for predictions of thermal cracking, which is not a major distress type in Maryland. The Level 3 relationships built into the MEPDG code for converting dynamic modulus and other mixture inputs to creep compliance, low temperature tensile strength, and aggregate coefficient of thermal contraction are judged as sufficient for Maryland purposes.
4. HMA thermal conductivity and heat capacity generally have only slight influence on pavement performance predicted by the MEPDG. Consequently, the Level 3 default values built into the MEPDG software are sufficient and laboratory measurement of these properties is not warranted.

5. Although SSA has a much more significant influence on predicted performance, there at present is no easy widely-used method for measuring this parameter, either initially after construction or over the pavement life. Therefore, no testing is recommended for this property. The Level 3 defaults values built into the MEPDG software should be used.

PCC Mixtures

6. There is very little data on the physical and mechanical properties Maryland PCC mixes to be incorporated into the database at this time. Much of the physical data required by the MEPDG (e.g., cement type, cementitious material content, water/cement ratio) is routinely measured for individual projects. These data should be collected and entered into the database on a project-by-project basis.
7. Maryland SHA has no Level 1 input data for PCC mixtures. However, there is no documented need to perform additional laboratory testing to determine the full Level 1 stiffness and strength inputs for PCC. Instead, 28-day PCC elastic modulus and modulus of rupture should be measured for JPCP paving projects in the future, incorporated into the database, and used for Level 3 inputs to the MEPDG. Continued measurement of split cylinder tensile strength should be discontinued, as this is not a primary input to the MEPDG (or to the 93 AASHTO Design Guide).
8. Given the lack of practical accepted test standards, ongoing test protocol issues, and other reasons, it is recommended that SHA not embark on any additional testing for thermal or shrinkage properties at this time. The current version of the MEPDG has been calibrated using the default Level 3 values for these properties, and these default values should continue to be used until accepted testing standards are available.

Unbound Materials

9. Maryland SHA already has significant Level 1 resilient modulus data for unbound materials. SHA should perform further investigations to determine why the k_1 , k_2 , and k_3 values computed from their laboratory-measured resilient modulus test data do not follow the expected physical trends. The causes may be either due to testing issues or unusual characteristics of the specific materials included in the database (e.g., cemented sands, highly overconsolidated clays, or other extreme /unusual soil conditions).
10. SHA should continue to perform laboratory resilient modulus tests on common unbound materials in the state to augment and fill gaps in the database. The current database has a reasonable amount of measured resilient modulus data for subbase materials (e.g., A-2-4) and some subgrade soils (A-4, A-6). However, it is deficient in measured data for granular base materials (e.g., A-1-a and A-1-b) and the poorer subgrade soils (A-7-5, A-7-6). Similar to the recommendations for asphalt binder and HMA testing, the resilient modulus testing for unbound materials could be included as a matter of policy (perhaps as part of the contract requirements) for large/important/expensive paving projects in the state. This testing could continue to be performed by SHA in its own laboratories or outsource to third-party laboratories.
11. There is no need for SHA to begin any testing program for the hydraulic properties of unbound materials. These properties have very little impact of predicted pavement performance, and the empirical correlations in terms of gradation and plasticity parameters built into the MEPDG Level 2/3 inputs provide sufficient accuracy.

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