Final Report

Guidelines for Implementing NCHRP 1-37A M-E Design Procedures in Ohio: Volume 2 —Literature Review

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Prepared in cooperation with
the Ohio Department of Transportation, and
the U.S. Department of Transportation,
Federal Highway Administration
State Job Number 134300

November, 2009

<table>
<tr><td>1. Report No.
FHWA/OH-2009/9B</td><td>2. Government Accession No.</td><td>3. Recipient's Catalog No.</td></tr>
<tr><td colspan="2" rowspan="2">4. Title and Subtitle
Guidelines for Implementing NCHRP 1-37A M-E Design Procedures in Ohio: Volume 2—Literature Review</td><td>5. Report Date
November, 2009</td></tr>
<tr><td>6. Performing Organization Code</td></tr>
<tr><td colspan="2">7. Authors
Jagannath Mallela and Leslie Titus Glover—Applied Research Associates, Inc.

Dr. Robert Y. Liang—University of Akron
Dr. Eddie Y. Chou—University of Toledo</td><td>8. Performing Organization Report No.</td></tr>
<tr><td colspan="2" rowspan="2">9. Performing Organization Name and Address
Applied Research Associates, Inc.
100 Trade Centre Dr., Suite 200
Champaign, IL 61820-7233</td><td>10. Work Unit No. (TRAIS)</td></tr>
<tr><td>11. Contract or Grant No.
134300</td></tr>
<tr><td colspan="2" rowspan="2">12. Sponsoring Agency Name and Address
Ohio Department of Transportation
1980 West Broad Street
Columbus, Ohio 43223</td><td>13. Type of Report and Period Covered</td></tr>
<tr><td>14. Sponsoring Agency Code</td></tr>
<tr><td colspan="3">15. Supplementary Notes</td></tr>
<tr><td colspan="3">16. Abstract
The development of the Mechanistic-Empirical Pavement Design Guide (MEPDG) under National Cooperative Highway Research Program (NCHRP) projects 1-37A and 1-40D has significantly improved the ability of pavement designers to model and simulate the effects of the combination of traffic and climate on future pavement damage, distress, and smoothness. With the adoption of the MEPDG as an American Association of State Highway and Transportation Officials (AASHTO) Interim Guide for Pavement Design, the next step is to integrate the MEPDG into the mainstream of pavement design procedures of State highway agencies across the U.S. The objective of this project is to implement the MEPDG for the Ohio Department of Transportation. To successfully accomplish this objective, it was important to review significant literature published on the MEPDG to identify issues related to the implementation of the MEPDG as a design standard and review feedback on how effective the MEPDG is as a pavement design tool among others. Since the completion of the MEPDG in 2004, there have been significant efforts by State highway agencies (SHA), transportation organizations (e.g., FHWA, NCHRP), and others to evaluate the MEPDG design procedure (e.g., models, algorithms) and implement the MEPDG as a pavement design standard, or to adopt the MEDPG as part of existing or new pavement design, evaluation, and analysis procedure. A detailed overview of some of the key activities undertaken or in the process of being undertaken nationwide and in Ohio is presented in this report, volume 2 of four volumes.</td></tr>
<tr><td>17. Key Words
NCHRP 1-37A, mechanistic empirical pavement design, distress, smoothness, performance prediction, reliability, implementation, flexible pavement, rigid pavement, overlays</td><td colspan="2">18. Distribution Statement
No restriction. This document is available to the public through the National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161</td></tr>
</table>

19. Security Classif.(of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	109	

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

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ACKNOWLEDGEMENTS

The authors express their deep gratitude to the Ohio Department of Transportation (ODOT). We are particularly grateful to Mr. Roger Green, Mr. Aric Morse, Mr. David Powers, Ms. Vicki Fout, and Ms. Monique Evans for their useful, timely, and constructive comments throughout the planning and execution of this research project.

The authors also gratefully acknowledge the contributions of Dr. Suri Sadasivam of ARA Inc. for his assistance with preparing the project reports and Dr. Susanne Aref, the project's statistical consultant, for her guidance in model recalibration.

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CHAPTER 1. INTRODUCTION

Since the completion of the MEPDG in 2004, State highway agencies (SHA), transportation organizations, and others have made significant efforts to evaluate the Mechanistic-Empirical Pavement Design Guide (MEPDG) procedure and either implement the MEPDG as a pavement design standard or adopt it as part of existing or new pavement design, evaluation, and analysis procedures. A detailed overview of some of the key activities is presented in the following sections.

The National Cooperative Highway Research Program (NCHRP), Federal Highway Administration (FHWA), and others have undertaken a number of research studies related to the MEPDG. Some examples are presented in table 1. In addition to the projects shown in this table, over 200 papers and reports have been published on various aspects of implementing the MEPDG. Expected outcomes of these studies includes (1) calibration and validation of the MEPDG procedures, performance prediction models, etc., (2) identification of deficiencies, and (3) recommendations for improvements.

The FHWA has established a Design Guide Implementation Team (DGIT) charged with coordinating national agencies/organizations such as the NCHRP, National Highway Institute (NHI), and Turner-Fairbank Highway Research Center to set up research projects and workshops to promote the use of MEPDG at the State level.

Various States (including Alabama, Arizona, Colorado, Georgia, Florida, Indiana, Iowa, Kansas, Maine, Mississippi, Missouri, Montana, New Jersey, New Mexico, New York, Texas, Utah, Virginia, Washington, and Wisconsin) have established or are in the process of establishing research programs to assess the feasibility/need for implementing the MEPDG. Based on the results of some of these studies, some States have initiated projects to begin implementing the MEPDG, while others have adopted a longer term approach to implementation.

Internationally, the Transportation Association of Canada has sponsored studies to help develop a version of the MEPDG calibrated to Canadian conditions, while pavement researchers in Argentina have reviewed the MEPDG jointed plain concrete pavement (JPCP) performance prediction models and recalibrated the models for local conditions.

Expected outcomes of these implementation efforts include roadmaps for implementation of the MEPDG, identification of potential difficulties along with good and bad implementation practices, and recommendations for improvements to the MEPDG to make it more suitable for local conditions.

Table 1. Summary of research studies undertaken to review/evaluate the MEPDG.

Research Objective	Research Title	Agency
MEPDG review/evaluation	• NCHRP 1-40A – Independent Review of the Recommended Mechanistic-Empirical Design Guide and Software.	NCHRP
Improved performance modeling	• NCHRP 1-42 – Top-Down Fatigue Cracking of Hot Mix Asphalt Layers.	NCHRP
	• NCHRP 9-38 – Endurance Limit of Hot Mix Asphalt Layers to Prevent Fatigue Cracking in Flexible Pavements.	NCHRP
	• NCHRP 1-41 – Selection, Calibration, and Validation of a Reflective Cracking Model for Hot Mix Asphalt Overlays.	NCHRP
	• NCHRP 9-30A – Rutting Performance Model for HMA Mix & Structural Design.	NCHRP
Development of support tools	• NCHRP 1-39 – Traffic Data Collection, Analysis, and Forecasting for Mechanistic Pavement Design.	NCHRP
	• NCHRP 9-33 – A Mixture Design Manual for Hot Mix Asphalt.	NCHRP
Implementation (into new or existing tools)	• NCHRP 9-30(01) – Expand Population of the M-E Database and Conduct Two Pre-Implementation Studies.	NCHRP
	• NCHRP 9-22 – Beta Testing and Validation of HMA Performance Related Specifications.	NCHRP
	• Modification of FHWA Highway Performance Data Collection System and Pavement Performance Models	FHWA
	• Adapting The Improved Models To NAPCOM	FHWA
	• Implementation and Support of New Pavement Equations For Highway Economic Requirements System	FHWA
	• Creation of Reports for Pavements Remaining Service Life Using the Improved Pavement Performance Models Developed for HERS	FHWA
Technology transfer	• FHWA-NHI-131109 — Analysis of New and Rehabilitated Pavement with M-E Design Guide Software	NHI
	• FHWA-NHI-131064 — Introduction to Mechanistic Design for New and Rehabilitated Pavements	NHI
	• FHWA Design Guide Implementation Team (DGIT) workshops on materials, climate, traffic, local calibration etc.	FHWA

CHAPTER 2. SYNTHESIS OF NATIONAL LITERATURE

Over 200 publications exploring various aspects of the MEPDG have been published to date. The studies collectively provide a vast reservoir of information that is key to the smooth and successful implementation of the MEPDG. The information will help prevent avoidable problems and pitfalls that may have been experienced in the past by agencies in similar situations, as well as provide ready answers and solutions to problems that may be common to all agencies attempting to implement the MEPDG.

Key issues of interest for researchers at the national and State levels include:

- Validation of the pavement distress prediction models that are essential to local implementation of the MEPDG procedure. Predicted pavement distress and smoothness for a given pavement type must match reasonably well with the field measured conditions, in order to establish confidence in the design process and facilitate its acceptance and implementation locally.
- Characterization of MEPDG input parameters, since accuracy of the distress and smoothness predictions depends on the accuracy of inputs such as traffic loading, layer material and subgrade foundation properties, climate, and design features.
- Agency business practices and strategies for local implementation of the MEPDG.

This chapter summarizes key conclusion and recommendations from published national literature relevant to this study.

Traffic

Research related to MEPDG traffic characterization has focused on estimation of the axle load spectra, forecasting of traffic volume growth, and seasonal traffic patterns. A summary of information available is presented in the following sections.

<u>Timm et al. 2006</u>

Evaluated load spectra from 12 sites in Alabama against a statewide load distribution. It was concluded that statewide load spectra can be used for pavement design in most cases.

Tran and Hall 2007

Compared default MEPDG single, tandem, and tridem axle load distributions to statewide axle load distributions developed for Arkansas, to develop statewide axle load spectra and evaluate the significance of the developed inputs on predicted MEPDG distress and International Roughness Index (IRI). Concluded the following:

- Statewide axle load spectra in Arkansas were different from the default values contained in the MEPDG.
- The influence of the differences between the statewide and default axle load spectra on pavement performance predictions provided by the MEPDG software is significant.
- The difference in predicted pavement life can be more than 25 percent.
- There is a significant difference in predicted rutting and fatigue cracking when using the statewide vehicle class distribution factors rather than the MEPDG suggested default values.
- The influence of statewide monthly and hourly distribution factors on predicted rutting and fatigue cracking was not significant compared to the default MEPDG values.

Based on the conclusions, the following was recommended:

- Weight data collected at weigh-in-motion (WIM) stations should be checked carefully for errors before using for design purposes.
- Where there are significant differences in statewide and MEPDG default axle load distributions, the use of statewide axle load spectra instead of default values is recommended.
- Statewide axle load spectra should be updated periodically unless no significant changes are observed in the future.
- Of the 17 (truck traffic classification (TTC)) groups recommended in the MEPDG software, only seven (TTC groups 3, 6, 7, 9, 10, and 13) are applicable in Arkansas. Therefore, guidance for selecting an appropriate TTC group for a given design project provided in the MEPDG software should be State-specific.
- MEPDG default monthly and hourly distribution factors may be used for the pavement design in Arkansas.
- Statewide vehicle class distribution factors should be reviewed periodically and updated as necessary.

Lu et al. 2007

Conducted a study in California to analyze the growth pattern of truck traffic volume, sensitivity of pavement responses to errors in growth rate estimation, and potential contributing predictors that can be used to predict truck traffic growth rate based on roadway characteristics and socio-economic data. Significant findings of the study were as follows:

- Different truck classes were found to have different growth trends.
- Selection of growth model should consider expected pattern of future growth in truck traffic while checking its reasonableness using the regional economic growth rate. Moreover, when using linear growth rate in percentage, the reference year should be clearly stated.
- Traffic data from a minimum of 6 years should be used to reduce the variance in truck volume predictions.
- Roadway characteristics and socio-economic factors cannot be used to directly predict truck traffic growth rate with high accuracy.
- Factors such as population density, population density growth rate, land use, and highway functional classification can significantly influence traffic growth rates.

Prozzi and Hong 2006

Conducted a study to develop mathematical models to incorporate the long-term traffic volume growth trends and short-term seasonal variations simultaneously using seasonal time series techniques. The longer term growth trend was developed using a time series model that predicts traffic as a function of time in years, while trigonometric functions are used to simulate monthly (seasonal) variations in traffic within the year. The results of the analysis showed that the two model alternatives, linear trend plus time series and compound trend plus time series models, developed herein accurately capture volume growth and seasonal variations. However, for traffic predictions over 20 years, the authors recommended using the linear trend model. The growth factors varied among different vehicle classes, as did the seasonal variation characteristics. This methodology, if incorporated, can provide a simplified traffic input and lead to more efficient running of the MEPDG program.

Zaghloul et al. 2006

Performed a sensitivity analysis study to evaluate the impact of using Level 3 traffic data versus Level 1 on accumulated damage and corresponding MEPDG predicted pavement service life. The results of the sensitivity analysis showed that the use of Level 1 or Level 3 traffic data does affect predicted hot mix asphalt

(HMA) pavement rutting significantly. However, its impact on longitudinal "top-down" cracking was not significant.

<u>Li et al. 2007</u>

Conducted a multifaceted study to investigate the impact of factors such as duration of WIM data collection and sampling, data input level, traffic count accuracy, and vehicle operation speed on flexible pavement performance. The analysis was performed on actual WIM traffic data from various sites in Indiana. The following were concluded:

- The duration of traffic data collection had negligible effect on the following MEPDG inputs:
    - Percent of trucks in design direction.
    - Percent of trucks in design lane.
    - Percent of heavy trucks.
    - Average number of axles per truck.
    - Vehicle operational speed.
- The duration of traffic data collection had significant effect on the following MEPDG inputs:
    - Average annual daily truck traffic (AADTT).
    - Vehicle class distribution.
    - Hourly truck traffic distribution.
- The duration of traffic data collection had some effect on predicted rutting and cracking (particularly significant effect on longitudinal cracking and its time of occurrence). Over estimation of truck traffic produced higher predictions of rutting and cracking.
- Applying lower levels of vehicle operational speeds resulted in higher predictions of distress and IRI with longitudinal cracking showing the greatest increase.

A summary of the effects of traffic input level, accuracy of truck count, and vehicle operational speed on predicted flexible pavement distresses is presented table 2.

Table 2. Summary of the effects of traffic input level, accuracy of truck count, and vehicle operational speed on predicted flexible pavement distresses.

MEPDG Traffic Input Parameter	Effect on Predicted Flexible Pavement Distress			
	Rutting	Longitudinal Cracking	Alligator Cracking	IRI
Vehicle class distribution	Fair	High	Medium	No
Monthly adjustment factors	Fair	Medium	Fair	No
Hourly distribution	No	No	No	No
Axle load distribution	Medium to High	High	High	Medium to High
Number of axles per truck	No	No	No	No
Truck count accuracy	Fair	Medium	Medium	No
Operational speed	Fair	Medium	Medium	No

Thus, it was recommended that traffic data used for developing MEPDG inputs must be collected for at least 3 full random months. This will help avoid estimates with extreme values and provide more reasonable pavement performance predictions.

<u>Cottrell et al. 2003</u>

Proposed a plan (based on the FHWA Traffic Monitoring Guide approach) to collect traffic and truck axle weight data to support Level 2 pavement designs in Virginia. Key aspects of the plan were:

- Develop truck weight groups.
- Develop criteria for site selection.
- Develop a site selection plan.
- Estimate the cost to implement the plan.
    - Select most suitable technology.
    - Calculate the configuration and installation costs.
    - Outline and estimate personnel requirements and costs.
    - Estimate the annual operating and maintenance costs.
- Define benefits of implementing the traffic data plan.
    - Estimate the potential savings from improved pavement designs.
    - Compare the annual savings with the cost of the implemented program.

Al-Yagout et al. 2005

Presented efforts in Washington State towards improving traffic characterization, with the primary objective being developing statewide truck axle load spectra. The study reported the following:

- Significantly underestimating or overestimating axle load spectra could result in rehabilitation occurring within 5 to 7 years of the actual axle load spectra.
- Slightly underestimating or overestimating axle load spectra could result in rehabilitation occurring within 2.5 to 4 years of the actual axle load spectra.
- Thus, the MEPDG is especially sensitive to extreme load spectra (i.e., significantly underestimated or overestimated spectra) and is moderately sensitive to slightly underestimated or overestimated load spectra.
- The MEPDG rather than statistics must be used to evaluate whether different load spectra are significantly different.
- Highway geographic location (urban or rural) and functional class (Interstate or otherwise) did not significantly impact axle load spectra.
- Statewide axle load spectra developed were similar to MnROAD and MEPDG axle load spectra.

Haider and Harichandran 2007

Developed a practical method for estimating single and tandem axle load spectra using truck weight and volume data. The important aspect of this research was the fact that truck weight (measured as gross vehicle weight) and volume data are more readily available than axle load spectra. It is argued that the use of site-specific or regional truck weight and volume-generated axle load spectra in pavement design will be better than using MEPDG Level 3 defaults.

Materials

Researchers at the national level have conducted studies on all the key MEPDG materials groups. Significant findings are summarized in the following sections.

Hot Mix Asphalt

Loulizi et al. 2006

- The Witczak E* prediction equation produces reasonable estimates of E* that are of the same order of magnitude as the measured lab-tested values.

- Witzak's sigmoidal function provides a very good fit to the dynamic modulus master curve. However, since the sigmoidal parameters are obtained through regression analysis, they are only valid for the range of testing frequencies for which lab data are available. Extrapolating the sigmoidal function to cover frequencies outside that used for determining the sigmoidal parameters is not encouraged.
- Using the average values for the backcalculated modulus may not provide the best estimates for existing HMA layer damage factor. Reasonable estimates of HMA damage factor may be obtained using Falling Weight Deflectometer (FWD) test location specific backcalculated E* and lab tested E* obtained from lab-tested cores.
- The use of Level 3 pavement performance characterization ranking may be misleading if the latest HMA surface condition does not reflect the overall condition of underlying HMA layers.

Flintsch et al. 2007

- Recommends Level 1 testing to characterize HMA for pavement projects of significant impact. Level 1 characterization could be implemented by developing a catalog of required HMA properties for typical mixes. The catalog would provide a better characterization of HMA properties than just using default inputs and the MEPDG E* prediction equation.
- Level 2 characterization of HMA (based on the default Witczak prediction equation) is recommended for projects with less significant impact.
- A sensitivity analysis to determine effect of E* on predicted pavement performance is recommended. This would help determine how the use of lab-tested E* and level E* estimates will impact pavement performance prediction. Results will be key for developing recommendations on when full scale E* lab testing is required versus the use of typical values in catalogs or Level 3 defaults. Also, if predicted pavement performance is sensitive to E* of special thin surface layers, lab-tested E* or E* characterization in catalogs of these materials is required.

Lundy et al. 2005

- Comparisons between laboratory-tested E* values and predictions from the MEPDG E* model (Level 3) show reasonable agreement (the average percent difference in predictions was about 30 percent). MEPDG estimated E* values might be improved if the binder viscosity were tested over a range of temperatures to allow measured values to be directly input into the regression equations (Level 2).

Azari et al. 2007

- The MEPDG E* prediction model produced reasonably good E* estimates overall. However, for lower bound E* values (which correspond to lower bound loading frequencies and higher temperatures), E* predictions were somewhat overestimated. HMA subjected to lower bound loading frequency and high temperatures are susceptible to rutting, and this could adversely impact MEPDG rutting predictions, leading to designs prone to premature rutting failure.
- Since aggregate properties have a significant impact on lower bound E* estimates, the improvement of lower bound E* estimates would depend on better characterization of HMA mix aggregate properties, including gradation.
- Correlation analysis conducted to examine the effect of binder parameters on MEPDG predicted E* equation showed the following:
  - Lab-tested binder shear modulus (G*) is highly correlated with E*. Thus, using Level 2 inputs such as G* to predict E* results in increased accuracy of E* estimates.
  - Binder phase angle, δ, was not found to have a significant impact on E*.
  - Overall, the MEPDG E* estimates for intermediate to high E* values are more accurate that the lower bound values.

Mohammad et al. 2006

- E* test is sensitive to the nominal maximum aggregate size of the HMA mix. Larger aggregates combined with recycled asphalt pavement (RAP) materials tend to have higher E* values at high temperatures.

<u>Portland Cement Concrete (PCC)</u>

Tanesi et al. 2007

Analyzed (coefficient of thermal expansion (CTE)) test results from 1800 PCC samples representing a variety of mixtures that are representative of pavements across the United States. The analysis reported the following:

- There is no correlation between CTE variability and CTE. The difference between replicate test results on the same specimen ranged from 0 to 2.4 x 10-6 in/in/°F, with an average of 0.4 x 10-6 in/in/°F.
- Sensitivity of CTE to predicted JPCP distress and smoothness (for a typical JPCP design) showed the following (1) the higher the CTE, the higher the effect of the variability on predicted distresses and smoothness and (2) the

mean CTE variability of 0.4 x 10-6 in/in/°F for a PCC with CTE of 6.5 x 10-6 in/in/°F would result in a difference in (1) percent slabs cracked of 10 percent, (2) mean joint faulting of 0.019-in, and (3) IRI of 18 in/mile.

- For a trial PCC mixture that does not represent the worst-case scenario in terms of pavement design, the CTE variability could lead to significant discrepancies in the predicted IRI, percent slabs cracked, and faulting.
- The results show that a single CTE test result may not necessarily be representative of the CTE of a mixture due to test variability. As a consequence, it is important to decrease the test variability by ensuring high testing standards and quality assurance/quality control (QA/QC) in the process of qualifying a mixture.

Khanum et al. 2006

In this study, the effect of predicted performance of typical Kansas JPCP using the MEPDG on the current Kansas Department of Transportation (KDOT) QA/QC specifications was evaluated to identify the acceptable levels of PCC strength. The results show that current KDOT percent within limits (PWL) specifications are more sensitive to the PCC strength than to the slab thickness. PCC slab thickness, concrete strength, and truck traffic significantly influence the distresses predicted by MEPDG. The interactions among these factors are also significant. For each JPCP project, there would be an optimal combination of PCC strength and slab thickness that is the most economical design. The optimum PCC slab thickness appears to be 9.5 to 10 in for 3,000-psi concrete and 8.5 to 9.0 in for 5,000-psi concrete, irrespective of traffic levels. Any thickness or strength increase beyond these levels would be conservative according to the MEPDG analysis. It was recommended that each MEPDG JPCP design analysis be studied for sensitivity toward the design concrete strength. An upper specification limit should be considered for the concrete strength in KDOT PWL specifications.

Hossain et al. 2006

In this study, CTE results from Long Term Pavement Performance (LTPP) projects in Iowa, Kansas, and Missouri were reviewed. Based on this study, the following conclusions were made:

- There was a wide range of measured PCC CTE values in Kansas. This was partly due to variations in aggregates and PCC mix composition.
- The calculated PCC CTE value as a weighted average of the coefficients of thermal expansion of the aggregates and hardened portland cement paste is always higher than the measured CTE value. The average of calculated and measured CTE values was close for limestone.

- Use of Level 2 PCC CTE input would result in more conservative JPCP design than that using Level 1 input.
- The detrimental effect of high PCC CTE value can be mitigated using higher slab thickness, larger dowel bar diameter, or widened lane with a tied concrete shoulder.

Unbound Aggregate Material and Subgrade Soils

Khogali and Mohamed 2007

The reasonableness of the MEPDG default Mr for unbound materials and subgrade soils was evaluated. The study revealed that the proposed MEPDG default values frequently overestimate the Mr. Furthermore, American Association of State Highway and Transportation Officials (AASHTO) soil class and material mechanical properties do not correlate well, suggesting the need for a different approach for adequately estimating Mr.

Kim et al. 2007

As part of implementation of the MEPDG, the MEPDG subgrade soil characterization procedure was reviewed. The following is a summary of results:

- The MEPDG assumes that the subgrade is compacted to optimum moisture content, leading to over-estimation of the pavement performance. The use of the average Mr value is recommended to obtain a conservative design.
- Estimates of Mr wet of optimum can be used to reasonably characterize Mr of subgrade soils during thawing.
- MEPDG frozen subgrade soil Mr values must be used with caution, as they tend to be greater than lab-tested values.

Performance Modeling and Reliability

National level research has been conducted on all key aspects of the MEPDG, with particular focus on reliability and performance modeling. Significant findings are summarized in the following sections.

Carvalho and Schwartz 2006

Compared flexible pavement designs and performance derived from the empirical 1993 AASHTO pavement design methodology and MEPDG for a range of locations within the United States, each with its own climate, subgrade, material properties, and local design preferences. The results suggest that, relative to the MEPDG predicted performance, the 1993 AASHTO Guide overestimates performance (i.e., underestimates distress) for pavements in warm locations and at high traffic levels. Trends of pavement performance with reliability level were similar for both methodologies. The results suggest that the default design criteria incorporated in the MEPDG software are consistent with what would be observed historically from pavements designed using the 1993 AASHTO Guide.

Schwartz 2007

Discrepancies between M-E model predictions and observed field performance conventionally are attributed to the predicted pavement distresses—i.e., to model error. Although there are many sources of uncertainty in the model predictions, there also is inherent uncertainty in the measured pavement distresses due to spatial variability, sampling errors, and measurement error. Using statistical arguments, this uncertainty in the measured pavement distresses can be incorporated into a corrected field calibration and an improved estimate of the "true" errors in the model predictions. Examples presented for the MEPDG rutting and faulting models were coefficient of determination (R^2) values increased by 10 to 40 percent and normalized standard errors decreased by 5 to 15 percent.

Schram and Abdelrahman 2006

This study presents the results of statewide calibration of the MEPDG JPCP IRI model for Nebraska. The comparison between local and nationally calibrated JPCP IRI models smoothness predictions was done at two levels, using network-level pavement data and using project-specific information. The results of the comparison showed considerable improvements in accuracy. However, although statewide calibrations showed accuracy improvements from the MEPDG model, they still represent the entire State pavement management network. Statewide calibration of the JPCP IRI model resulted in a 15 percent reduction in model standard error of estimates (SEE) when compared to the nationally calibrated model. The reduction nearly doubled to 29 percent when project-level data defining materials properties, traffic, and design features are used. For IRI of HMA overlays of existing PCC pavements, SEE was reduced by 43 percent when using network-level data. Acceptance of the MEPDG weighs heavily on its ability to predict distress and IRI reasonably. Only through local calibration will model error be minimized.

Timm et al. 2000

The primary objective of this research was to develop a rational means of accounting for the variability of the M-E input design parameters. This was accomplished by incorporating Monte Carlo simulation into the MEPDG. The main product of this research was a new edition of ROADENT (3.0), which incorporates the findings of this study into a user-friendly computer program that performs a comprehensive reliability analysis. A summary of the research findings is presented as follows:

- Monte Carlo simulation is an effective means of incorporating reliability analysis into the M-E design process for flexible pavements.
- The resulting distributions of fatigue life and rutting life, obtained from Monte Carlo simulation using the various MEPDG input distributions, are governed by an extreme value type I function.
- For most practical design scenarios, the number of Monte Carlo cycles should be set at 5,000.
- The input parameters having the greatest influence on the fatigue performance variability are HMA E* and HMA thickness. Likewise, the input parameters having the greatest influence on the rutting performance variability are base thickness, HMA thickness, and subgrade modulus.
- The axle weight variability has an overwhelming effect on the variability of both fatigue and rutting performance predictions. Therefore, careful load characterization is critical to the pavement design.

Gramajo et al. 2007

The comparison of the measured distresses with those predicted by the MEPDG in Virginia showed fair to poor agreement. There was not enough evidence to determine whether this was due to errors in the prediction models or because of the use of MEPDG default Level 3 material properties, especially for the HMA layers. The results suggest that significant calibration and validation will be required before implementation of the MEPDG in Virginia. The national calibration factors used on the distress prediction models do not seem to apply to the structures considered for this study.

Graves and Mahboub 2006

Conducted a pilot study to perform sensitivity analysis of the MEPDG flexible pavement distress/IRI prediction models. The results showed that AADTT, HMA thickness, and subgrade strength have significant impact on predicted distress/IRI, while HMA gradation, climate, and vehicle class distribution have less impact.

Graves and Mahboub 2007

Evaluated the reasonableness of the MEPDG approach for incorporating reliability into pavement design. This was done by utilizing a simulation based technique (Monte-Carlo) to evaluate the variation in predicted distress/smoothness based on the variability of a set of input parameters for a specific pavement design section. Simulation involved (1) assuming reasonable levels of variability in selected key MEPDG input (up to 100 design scenarios), (2) running the design scenarios through the MEPDG to predict distress/smoothness at the mean and 90 percent reliability levels, (3) characterizing predicted distress/smoothness variability and estimating predicted distress/smoothness at 90 percent reliability, and (4) comparing MEPDG determined and simulated distress/smoothness levels at 90 percent reliability.

The selected key MEPDG input were AADTT, HMA (surface layer) mix properties including moduli, HMA base mix properties, and layer thicknesses (i.e., HMA base and crushed stone thickness). The study revealed the following:

- Variability of the predicted distress/smoothness was not necessarily normally distributed. In general, the areas which did not match the normal characteristics were primarily in the lower tails of the distributions. This has the potential to be significant, since many high type facilities are designed with high reliability but with lower distress levels.
- Predicted distress/smoothness at the 90 percent reliability based on simulation of the variation in input parameters is less in all cases (virtually equal for total rutting) than that predicted by the MEPDG. This would indicate some of the other errors in the prediction model (measurement error, pure error or model fitting error) have had significant impact on the MEPDG reliability.
- A comparison of 90 percent reliability given by the MEPDG for mean inputs of the variables in this study and the reliability given from the simulation results is as follows:

Performance Indicator	MEPDG	Monte-Carlo Simulation
Alligator cracking, percent area	20.1	11.0
Transverse cracking, ft/mile	1,249	592
Total rutting, in	1.05	1.06
IRI, in/mi	160.5	126.2

- There were significant differences between the MEPDG and simulation reliability at the 90 percent level for most of the distresses and smoothness. The differences may be partly explained by the fact that prediction models that generally had less model error as indicated by higher coefficient of determination and lower SEE values had a closer match than those with lower coefficient of determination and higher SEE values. Additionally, the observed departure from normality of various distress modes may impact the reliability of the design.

- A full understanding of this difference is necessary, so that designers can have the best possible confidence in the predictions which are provided by the MEPDG software.
    - High type pavements are designed to perform without distresses exceeding a low threshold. It is indeed at these low thresholds levels where significant departures from normality take place under the current models. Therefore, any reliability based analysis for high type pavement facilities would seem suspect at this time.
    - A similar argument can be made for the low-volume roads, in which distress thresholds is much higher. Only the mid-ranges of the distress modes seem to follow normal distributions.

<u>Kim et al. 2006</u>

Conducted a study to determine the relative sensitivity of MEPDG HMA, traffic, and climate input parameters for Iowa. Based on the results of the sensitivity analysis, the following conclusions were drawn:

- Input parameters that had a significant influence on predicted distress/IRI were as follows:

Longitudinal Cracking	Alligator Cracking	Transverse Cracking	Rutting
• Binder type • AADTT • Tire pressure Subgrade and modulus	• Base thickness • Base type and modulus	• Binder type • HMA air voids and voids in mineral aggregate • Climate	• AADTT

- There is no input parameter that is sensitive to all the MEPDG performance measures in this study. Few input parameter used in this study affect all the predicted performance measures for flexible pavements.
- Compared to other performance measures, the predicted longitudinal cracking was influenced by most input parameters. A reasonable design concept to reduce longitudinal cracking should be considered in pavement designs with a relatively thick asphalt concrete (AC) layer.
- Alligator cracking does not seem to be a critical distress in flexible pavement structures with relatively thick AC layers as considered in this study.

- The input parameters related to material properties and climate was especially sensitive to predicted transverse cracking.
- The AC surface layer rutting dominated the total rutting in this study. This may be due to the relatively thick AC layers used in the pavements considered in this study.
- The IRI was not sensitive to most input parameters. This was probably due to the nature of the MEPDG IRI model, which is a function of initial IRI, IRI due to distress, frost heave and subgrade swelling.

Climate

Research has been conducted on various aspects of the Integrated Climatic Model (ICM) used in the MEPDG for predicting climate-related data required for analysis. Significant findings are summarized in the following sections.

<u>Zaghloul et al. 2006</u>

Discussed using data from eight weather stations near LTPP projects in New Jersey to evaluate the accuracy of the ICM on MEPDG and the impact of potential inaccuracies in climate data predictions on MEPDG predicted damage and pavement distress and IRI. Significant variations in distress and smoothness predictions for weather stations located in relatively close proximity.

<u>Sadasivam et al. 2006</u>

Studied the effects of ground water table (GWT) predicted by the enhanced ICM (EICM) on MEPDG predictions of pavement performance. The study concluded that the depth of GWT affects predicted top-down cracking, bottom up fatigue cracking, and subgrade rutting. Thus, it is important to model and predict this accurately for MEPDG design.

<u>Richter 2006</u>

Demonstrated the capability of the ICM to predict in situ moisture content and developed an empirical model to predict seasonal variations in the moduli of unbound pavement layer. It was concluded that the overall accuracy of the modulus predictions achieved was not fully acceptable due to fundamental discrepancies between layer moduli backcalculated using linear-layered elastic theory and the laboratory resilient modulus test conditions.

Oh et al. 2006

Demonstrated that unbound aggregate material and subgrade moisture contents predicted by the EICM compared reasonably well with field measurements in Texas. The researchers found that in most pavements, the predicted moisture contents reached an equilibrium stage some time after construction as expected. Also, variations in soil moisture contents due to changes in environmental and soil characteristics were as expected.

Puccinelli and Jackson 2007

Demonstrated that predicted long-term pavement performance differed significantly for pavements subjected to deep frost penetration remaining throughout the winter months and pavements exposed to repeated freeze-thaw cycles during winter.

Local Calibration

Researchers at the national level have studied the local calibration of the MEPDG performance models. Significant findings are summarized in the following sections.

Kim et al. 2006

Performed a sensitivity study to assess the effect of MEPDG design input parameters such as material properties, traffic, and climate on the performance of two existing flexible pavements in Iowa. The results showed that the predicted longitudinal cracking was influenced by most input parameters. Alligator cracking, roughness, and rutting in unbound layers remained insensitive to most input parameters.

Gramajo et al. 2007

Conducted a study for Virginia DOT to validate the MEPDG performance models by comparing the predicted pavement distresses with measured distresses on three flexible and four composite pavements. In general, agreement ranged from fair to poor. For many of the input parameters, only Level 3 data were available. The study concluded that significant calibration and validation are required before implementation of the MEPDG.

Salama et al. 2007

Reported that the MEDPG procedure significantly underestimates predicted rutting due to multiple axles. The researchers concluded that (1) the best method for calculating rut depths under multiple axle groups seems to be the integration of the entire strain pulse due to all the axles and (2) rutting damage is proportional to the number of axles within an axle group. These results were confirmed in the laboratory for the HMA layer. The study also indicated that while the MEPDG rut models are superior to the previous rut models in that they are able to dissect the total surface rutting between all pavement layers, their prediction of the individual layer rutting contributions does not always agree with the results from the measured transverse profiles.

Wu et al. 2007

Tested at the Louisiana Accelerated Pavement Research Facility (APRF), three full-scale asphalt pavement sections (with three base types blended calcium sulfate (BCS)/slag, BCS/flyash, and foam asphalt material), each having a 2-in thin HMA surface layer. After 225,000 accelerated wheel loading repetitions, one of the test sections failed due to rutting (rut depth > 0.5-in). Although the other two sections developed considerable amounts of rutting, they did not fail (rut depth < 0.5-in). The MEPDG was used to predict the total rutting developed in the three test sections evaluated. The study concluded that MEPDG generally overestimated the rut depths developed in all three test sections of this study.

Wang et al. 2007

Under a multi-year study supported by the Florida Department of Transportation (FDOT) and University of Florida developed a top-down HMA cracking model based on the HMA fracture mechanics. The key features of the model were:

- Damage in asphalt mixture is equal to the accumulated dissipated creep strain energy (DCSE).
- There exists a damage threshold (called DCSE threshold or DCSE limit) illustrated in asphalt mixture that is independent of loading model or loading history.
- Damage under the cracking threshold is fully healable;
- Once the damage (accumulated DCSE) exceeds the damage threshold (DCSE limit), a macro-crack will initiate, or propagate if the crack is already present.
- A macro-crack is not healable.

The calculation of DCSE needs the structural properties (used to determine the tensile stress) and the material properties D_1 and m-value, which are parameters in the creep compliance function $D(t) = D_0 + D_1 t_m$. This model is currently being used in a Windows-based program to evaluate/optimize pavement design in Florida. Key aspects of this model (i.e., predictive models for the material properties) are currently being reevaluated and refined in the NCHRP project 1-42A.

Ker et al. 2007

Used LTPP database to develop improved fatigue cracking models for flexible pavements. The study found that the assumptions of normality of random errors and constant variance were not appropriate. Therefore, several modern regression techniques, including generalized linear model and generalized additive model, along with the assumption of Poisson distribution and quasi-likelihood estimation, were used for the modeling. The resulting predictive models showed reasonable agreement with the LTPP pavement performance data.

Schram and Abdelrahman 2006

Used Nebraska Department of Roads (NDOR) pavement management data to calibrate two MEPDG smoothness models at the local project level. The dataset was categorized by annual daily truck traffic and surface layer thickness. Results showed that project level calibration reduced default model prediction error to nearly half of that of network-level calibration.

CHAPTER 3. SYNTHESIS OF OHIO SPECIFIC MEPDG LITERATURE

Several publications exploring various aspects of the mechanistic empirical pavement design MEPDG in Ohio have been published to date. The studies, sponsored mainly by ODOT, collectively provide a good amount vast reservoir of information that is key to a smooth and successful implementation of the MEPDG in Ohio. However, to date, MEPDG- related research in Ohio has mainly focused on the characterization of Ohio paving materials for the MEPDG. Thus, the presentation of key conclusion and recommendations regarding ODOT materials (including subgrade soils) characterization will be the focus of this chapter. Note that only the pavement materials presented in the 2005 Construction and Materials Specifications Manual and still utilized in Ohio will be described. Literature on ODOT MEPDG related traffic, climate, and performance studies have also been presented. It must be noted, however, that very limited amount of effort has been expended to date on traffic related studies under ODOT's research program.

Recommended Tests and Test Protocols for Materials Characterization Recommended by the MEPDG

Tables 3 and 4 present the types of tests and test protocols recommended by the MEPDG arranged by material type and hierarchical input level for new and rehabilitated pavements. This table is used as a backdrop to compare the testing performed under the various ODOT materials testing studies summarized herein. In the MEPDG, Level 1 input data represents the highest quality data available for use in design. Such data may be obtained at the project level from laboratory or field testing as the case may be. Level 3 represents inputs configured from agency's historical testing databases or approximate engineering estimates based prior knowledge. Similar to level 1 inputs, level 2 inputs are also based on testing; however, the tests involved are less rigorous are as associated with the main property of interest through standard default or agency specific correlations.

Hot Mix Asphalt Concrete (HMA) Materials Studies

The primary input parameter is the HMA dynamic modulus (E*). At Level 1, the MEPDG recommends HMA dynamic modulus testing in the lab following guidelines presented in the NCHRP 1-28A report. Also, required at Level 1 is the asphalt binder complex shear modulus and phase angle testing (AASHTO T315). These are used to develop an HMA E* master curve.

Table 3. Testing requirements and corresponding protocols at various hierarchical input levels required for new pavement design.

Materials Category	**Measured Property**	**Recommended Test Protocol**	**Hierarchical Input Level**		
			3	**2**	**1**
Hot-mix asphalt mixture	Dynamic modulus (E*)	AASHTO TP62			X
	Tensile strength	AASHTO T322		X	X
	Creep Compliance	AASHTO T322		X	X
	Mixture gradation	AASHTO T27	X	X	
	Mixture volumetrics (as-built) : Effective asphalt content Air voids Voids filled with asphalt (VFA)	AASHTO T308 AASHTO T166 & T209	X	X	X
	Penetration at 77°F	AASHTO T49	X		
	Unit weight	AASHTO T166	X	X	X
	Short term oven aging	AASHTO R30			X
Asphalt binder	Asphalt binder complex shear modulus (G*) and phase angle (δ) OR *Conventional binder test data:* Penetration OR Ring and Ball Softening Point Absolute Viscosity Kinematic Viscosity Specific Gravity OR Brookfield Viscosity	AASHTO T315 AASHTO T49 OR AASHTO T53 AASHTO T 202 AASHTO T201 AASHTO T228 OR AASHTO T316		X	X
	Asphalt binder grade: PG Grade OR Viscosity Grade OR Penetration Grade	AASHTO M320 OR AASHTO M226 OR AASHTO M20	X		
	Rolling thin film oven aging	AASHTO T315		X	X
Portland cement concrete mixture	Elastic modulus (chord modulus)	ASTM C469	X[1]		X
	Poisson's ratio	ASTM C469			X
	Flexural Strength	AASHTO T97	X[1]		X
	Indirect tensile strength (CRCP only)	AASHTO T198			X
	Compressive strength	AASHTO T22	X[1]	X	
	Unit weight	AASHTO T121	X	X	X
	Coefficient of thermal expansion	AASHTO TP60	X		
	Elastic modulus and Poisson's ratio	ASTM C469			X
Lean concrete & Cement-treated aggregate	Flexural strength[2]	AASHTO T97			X
	Compressive strength	AASHTO T22		X	X
Lime-cement-flyash	Unconfined compressive strength	ASTM C593		X	
Soil cement	Unconfined compressive strength	ASTM D1633		X	
	Resilient modulus	Mixture Design and Testing Protocol (MDTP) in conjunction with AASHTO T307[3]			X
Lime stabilized soil	Unconfined compressive strength	ASTM D5102		X	

[1]Testing requirements are much reduced compared to Level 1, e.g., historical 28-day values suffice.
[2]Required when lean concrete or cement treated aggregate layers are used in HMA pavement design only.
[3]MDTP is described by Little (2000); an equivalent test can be used in lieu based on local experience.

Table 3. Testing requirements and corresponding protocols at various hierarchical input levels required for new design (continued).

Material Category	Measured Property	Recommended Test Protocol	Hierarchical Input Level		
			3	2	1
Unbound materials	Regression coefficients k_1, k_2, k_3 for the generalized constitutive model[4] that define resilient modulus as a function of stress state[5]	AASHTO T307 or NCHRP 1-28A			X
	Resilient modulus (M_r): M_r at optimum moisture (OMC) and maximum dry density (MDD) or design value OR California Bearing Ratio (CBR) OR R-value OR Gradation and Atterberg limit parameters OR Dynamic Cone Penetrometer (DCP)	AASHTO T307 or NCHRP 1-28A OR AASHTO T193 OR AASHTO T190 OR AASHTO T27, T89 & T90 OR ASTM D6951		X	
	MDD & OMC: Direct testing OR Estimated from gradation & Atterberg limits	AASHTO T99 or T180 OR AASHTO T27, T89 & T90		X (estimated)	X (direct testing)
	Specific gravity: Direct testing OR Estimated from gradation & Atterberg limits	AASHTO T100 OR AASHTO T27, T89 & T90		X (estimated)	X (direct testing)
	Saturated hydraulic conductivity: Direct testing OR Estimated from gradation & Atterberg limits	AASHTO T215 OR AASHTO T27, T89 & T90		X (estimated)	X (direct testing)
	Degree of saturation	AASHTO T27, T89 & T90		X	
	Soil water characteristic curve parameters: Direct Testing OR Estimated from MDD, OMC, gradation, and Atterberg limits	Pressure plate, filter paper, and/or Tempe cell testing; AASHTO T99 or T180; AASHTO T100 OR AASHTO T180 or T99 AASHTO T100; AASHTO T27 AASHTO T90 OR AASHTO T27 AASHTO T90	X (last option)	X (estimated)	X (direct testing)

[4]See section 2.2.5.1 of the ME PDG Part 2, Chapter 2.
[5]Level 1 inputs for resilient modulus are not needed for JPCP and CRCP; current HMA models not calibrated with level 1 inputs.

Table 4. Testing requirements and corresponding protocols at various hierarchical input levels required for pavement rehabilitation design.

Materials Category	Measured Property	Recommended Test Protocol	Hierarchical Input Level		
			3	2	1
Hot-mix asphalt base pavement	FWD backcalculated pavement modulus	ASTM D4694			X
	Indirect resilient modulus, M_{ri}	NCHRP 1-28A protocol		X	
	Mixture volumetrics (as-built) : Asphalt content and gradation Air voids	 AASHTO T164[1] AASHTO T166 & T209		X	X
	Penetration at 77°F	AASHTO T49	X		
	Unit weight	AASHTO T166	X	X	X
Asphalt binder in base pavement	Asphalt recovery	ASTM D5404		X	X
	Asphalt binder complex shear modulus (G*) and phase angle (☐) OR *Conventional binder test data:* Penetration OR Ring and Ball Softening Point Absolute Viscosity Kinematic Viscosity Specific Gravity OR Brookfield Viscosity	AASHTO T315 AASHTO T49 OR AASHTO T53 AASHTO T 202 AASHTO T201 AASHTO T228 OR AASHTO T316		X	X
	Asphalt binder grade: PG Grade OR Viscosity Grade OR Penetration Grade	 AASHTO M320 OR AASHTO M226 OR AASHTO M20	X		
	Rolling thin film oven aging	AASHTO T315		X	X
Portland cement concrete base pavement	Elastic modulus (chord modulus) of cores[1] OR FWD backcalculated modulus	ASTM C469 OR ASTM D4694	X\		X
	Poisson's ratio	ASTM C469			X
	Flexural Strength of base pavement[2]	AASHTO T97			X[2]
	Indirect tensile strength[3]	AASHTO T198			X
	Compressive strength of base pavement	AASHTO T22	X	X	
	Unit weight of base pavement	AASHTO T271 (cores)	X	X	X
Lean concrete & Cement-treated aggregate	FWD backcalculated modulus	ASTM D4694			X
	Compressive strength	AASHTO T22 (cores)		X	
Lime-cement-flyash	FWD backcalculated modulus	ASTM D4694			X
	Unconfined compressive strength	ASTM C593 (cores)		X	
Soil cement	FWD backcalculated modulus	ASTM D4694			X
	Unconfined compressive strength	ASTM D1633		X	
Lime stabilized soil	FWD backcalculated modulus	ASTM D4694			X
	Unconfined compressive strength	ASTM D5102		X	
Unbound materials	FWD backcalculated modulus[4]	ASTM D4694			X
	California Bearing Ratio (CBR) OR Gradation and Atterberg limit parameters OR Dynamic Cone Penetrometer (DCP)	AASHTO T193 OR AASHTO T27, T89 & T90 OR ASTM D6951		X	

[1]An equivalent test method to AASHTO T164 such as AASHTO T308 can be used. However, if the latter is used, it will still be necessary to run an asphalt extraction and recovery tests to determine the properties of the binder.
[2]Required only for JPCP bonded overlays or restoration projects.
[3]Required for bonded PCC overlays of CRCP only.
[4]The backcalculated modulus will be at in situ conditions. It needs to be adjusted for OMC and MDD and to reflect lab testing.

For MEPDG Levels 2 and 3, Witczak's dynamic modulus prediction model, which requires HMA gradation, air voids, volumetric binder content, and asphalt binder type as inputs, is used to estimate E* and develop the Master Curve. Additional testing is necessary to characterize HMA for predicting thermal cracking. The additional testing includes

- Tensile strength (AASHTO T322).
- Creep compliance (AASHTO T322).
- Thermal conductivity and heat capacity (ASTM E 1952 and ASTM D2766).

<u>HMA Mix Dynamic Modulus Test (E*)</u>

At Level 1, the MEPDG requires HMA E* values for 3 test temperatures at 3 corresponding loading frequencies. Dynamic modulus (E*) for typical ODOT HMA mixes have been determined as part of three ODOT MEPDG- related studies, conducted by namely Liang (2001), Sargand et al. (1991), and Masada and Sargand (2002). In addition, mixture testing data from the WAY30 experimental project was also made available by ODOT (2007).

In the study conducted by Liang (2001), E* was tested for one SuperPave and four polymer modified asphalt (PMA) mixes as follows:

- Superpave PG 58-28.
- SBS Goodyear (polymer modified).
- SBR Butanol (polymer modified).
- SBS Ashland modifier (polymer modified).
- SBS Kock (polymer modified).

Testing was done at three loading frequency (i.e., 16 Hz, 4Hz, and 1Hz) and a single test temperature of 104^{o}F (40°C). Testing was done in accordance to ASTM 3497. Table 5 presents the test results of testing conducted by Liang 2001. This test data set is limited and cannot be used for characterizing typical ODOT HMA mix E* at Level 1 for the MEPDG, since the testing was done at a single test temperature. Further the test protocol used is very different from AASHTO TP62.

Sargand et al. (1991) conducted E* tests at a single test frequency of 8 Hz on some selected ODOT HMA mixes. Again, the testing done was not sufficient to satisfy the MEPDG Level 1 E* requirements. Masada and Sargand (2002), using ASTM D3497, determined E* for composite HMA specimens recovered from the Ohio-

SHRP test road. For the recovered specimens, testing was done at three loading frequencies (16 Hz, 4Hz, and 1Hz) and three temperatures (41°F, 68°F, and 104°F). Table 6 present a summary of composite HMA specimens E* results.

Table 5. HMA dynamic modulus results from Liang 2001.

Binder Type	HMA Dynamic Modulus (psi)		
	16 Hz	4 Hz	1 Hz
Unmodified PG 58-28	2.34 E+05	5.09E+05	1.17E+05
2% SBR Goodyear + PG 58-28	3.21E+05	9.96E+05	1.27E+04
3% SBR Goodyear + PG 58-28	2.54E+05	8.89E+05	1.19E+04
4% SBR Goodyear + PG 58-28	2.61E+05	7.39E+05	1.19E+04
5% SBR Goodyear + PG 58-28	2.66E+05	7.87E+05	1.24E+04
2% SBR Butanol + PG 58-28	3.15E+05	6.43E+05	1.43E+04
3% SBR Butanol + PG 58-28	3.34E+05	6.43E+05	1.46E+04
4% SBR Butanol + PG 58-28	2.96E+05	5.31E+05	1.43E+04
2% SBS Ashland + PG 58-28	3.22E+05	1.58E+06	2.14E+04
3% SBS Ashland + PG 58-28	3.12E+05	2.05E+06	1.95E+04
4% SBS Ashland + PG 58-28	3.59E+05	1.35E+06	2.23E+04
5% SBS Ashland + PG 58-28	3.69E+05	1.29E+06	2.23E+04
2% SBS Koch + PG 58-28	3.35E+05	1.41E+06	2.26E+04
3% SBS Koch + PG 58-28	3.48E+05	2.15E+06	2.36E+04
4% SBS Koch + PG 58-28	3.46E+05	2.73E+06	2.40E+04
5% SBS Koch + PG 58-28	3.20E+05	2.37E+06	2.17E+04

Table 6. HMA Dynamic modulus from Masada and Sargand (2002).

Source of Data	Test Temperature, °C	Ave. │E*│ (million psi) @ Loading Frequency of:			
		16Hz	10Hz	4Hz	1Hz
Masada and Sargand (2002) w/Strain Gages	5	1.44	N/A	1.27	0.90
	25	0.42	N/A	0.27	0.16
	40	0.11	N/A	0.07	0.05
Masada and Sargand (2002) w/Extensometer	5	1.98	1.75	1.56	1.36
	25	1.29	1.08	0.93	0.72
	40	0.06	0.04	0.03	0.02

Additionally, phase angle was reported for the HMA dynamic modulus test conducted for typical ODOT mixes tested by Masada and Sargand (2002). Table 7 presents the phase angle of the tested asphalt concrete specimens.

Information on E* testing performed using protocols compatible with the MEPDG was provided by ODOT in 2007 (ODOT 2007) from the Ohio SHRP Test Road and Wayne 30 experimental projects. Figures 1 through 5 present the data and master curves developed from this testing. This information can be very useful in building HMA input libraries for ODOT.

Table 7. Phase angle of Ohio SHRP Test Road asphalt mixes tested by Masada and Sargand (2002).

Source of Data	Test Temperature, °C	Phase Angle (deg.) @ Loading Frequency of:			
		16Hz	10Hz	4Hz	1Hz
Masada and Sargand (2002) w/Strain Gages	5	20.2	N/A	17.7	16.5
	25	28.8	N/A	27.4	28.8
	40	34.5	N/A	33.5	21.9
Masada and Sargand (2002) w/Extensometer	5	27.0	30.6	20.3	19.8
	25	29.3	25.3	22.1	18.7
	40	33.2	31.1	27.8	19.2

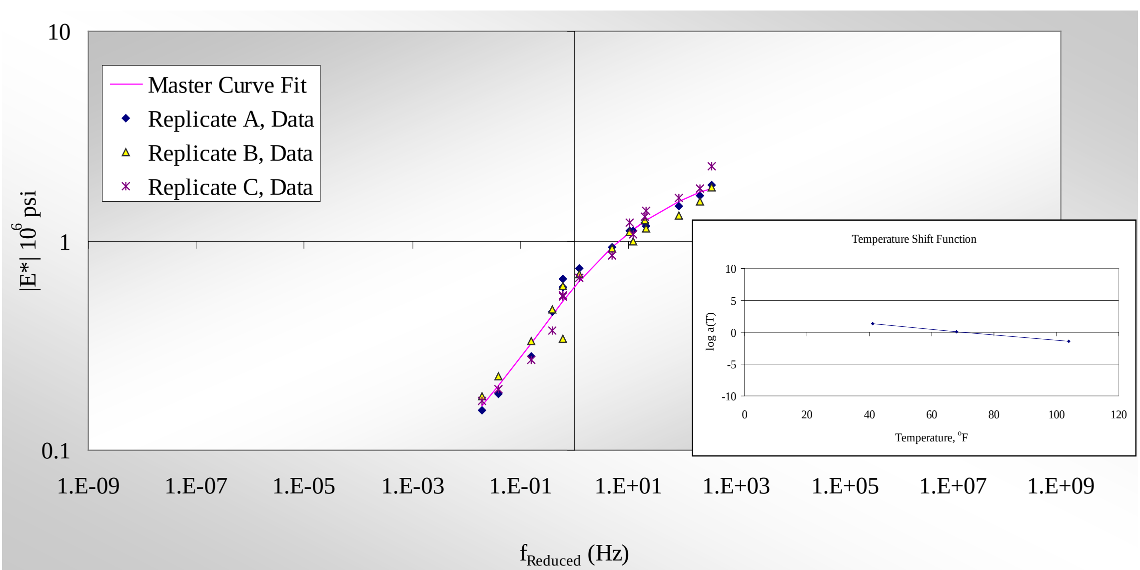

Figure 1. Dynamic modulus test data, master curve, and temperature shift function for HMA materials from the Ohio SHRP Test Road.

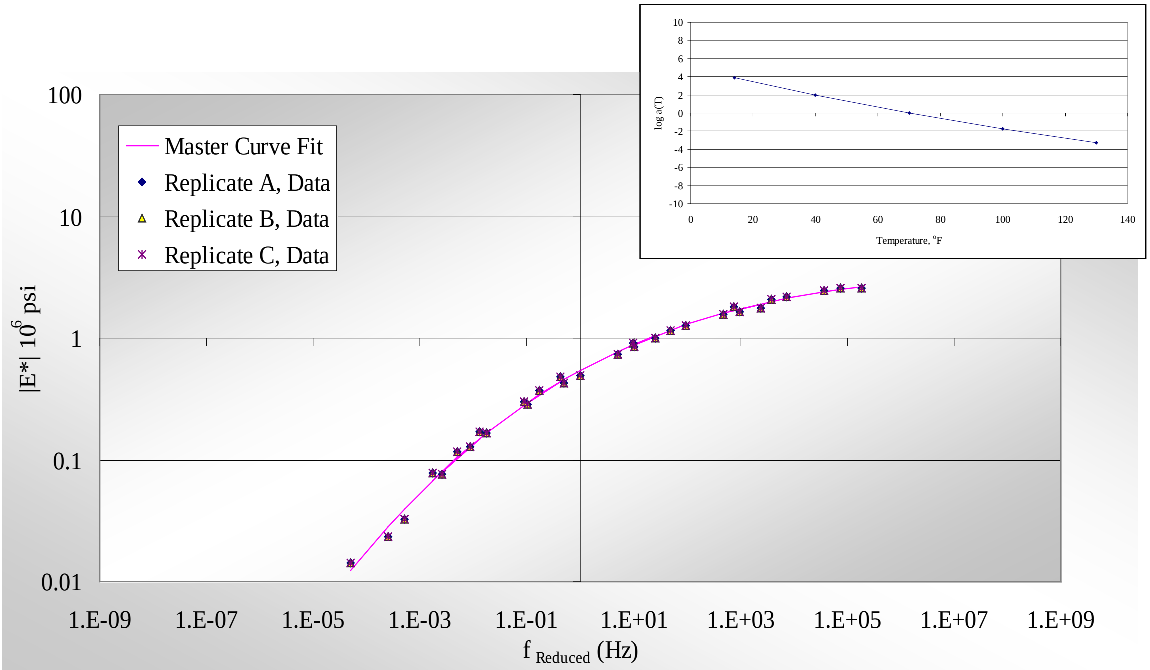

Figure 2. Dynamic modulus test data, master curve, and temperature shift function for HMA materials from the WAY30 442 (Superpave intermediate course) layer.

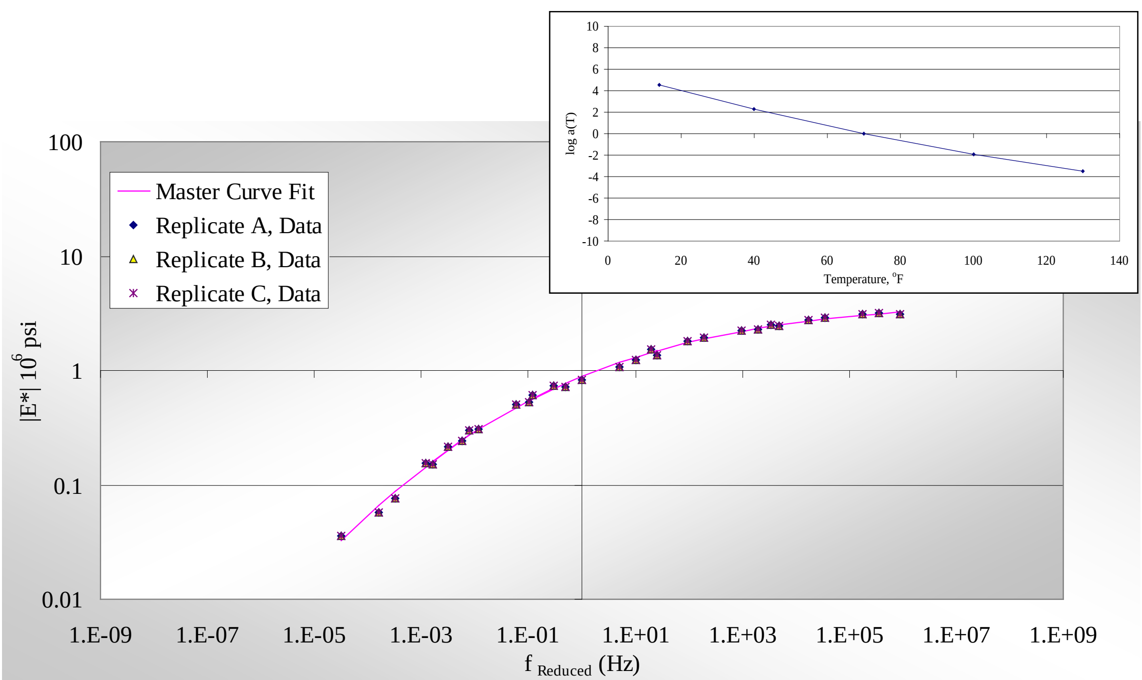

Figure 3. Dynamic modulus test data, master curve, and temperature shift function for HMA materials from the WAY30 443 (SMA surface course) layer.

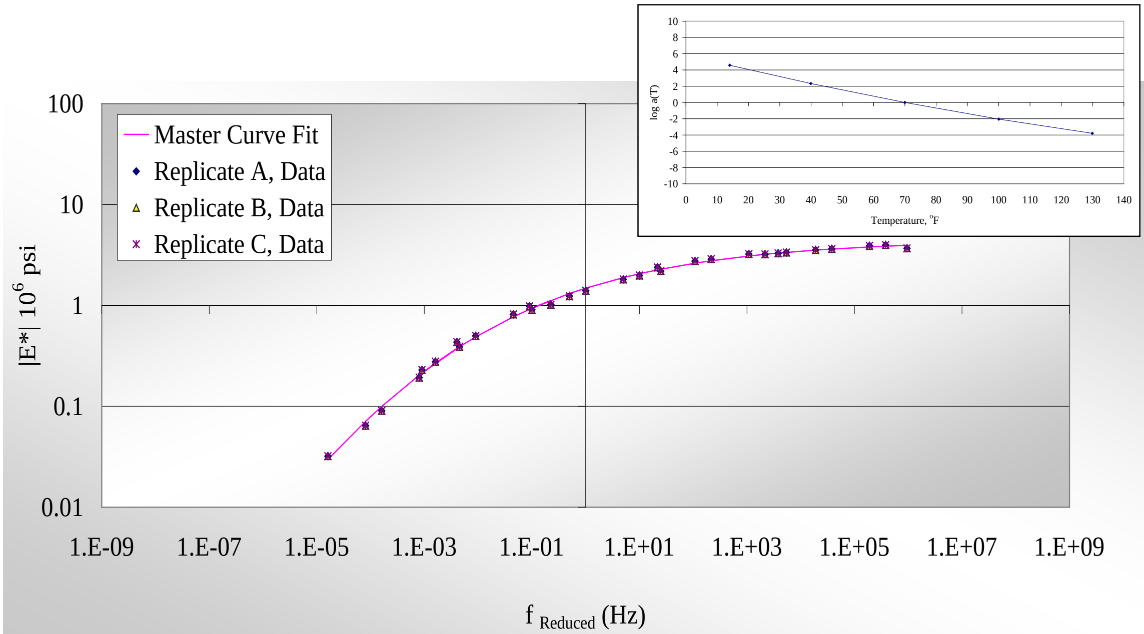

Figure 4. Dynamic modulus test data, master curve, and temperature shift function for HMA materials from the WAY30 302(HMA base course) layer.

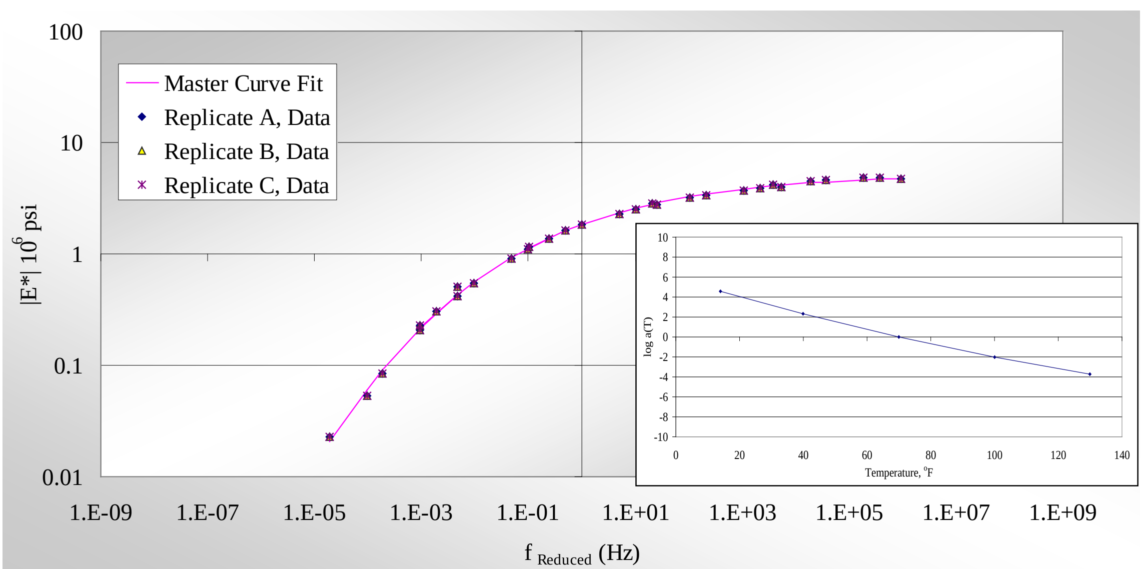

Figure 5. Dynamic modulus test data, master curve, and temperature shift function for HMA materials from the WAY30 fatigue resistant layer (binder rich bottom HMA lift).

At Level 1, the MEPDG requires asphalt binder complex shear modulus (G*) and phase angle (deg) tested at a frequency loading of 10 radians/sec (i.e., 10 Hz). No binder complex shear modulus and phase angle testing was in any of the studies reviewed. Additional testing is thus required to obtain a full set of MEPDG Level 1 input requirements for typical ODOT HMA mixes.

<u>HMA Mix Indirect DT Tensile Strength Testing</u>

The indirect tensile strength (ITS) values of asphalt concrete were determined in four studies conducted for ODOT, namely:

- Liang (1998): Tested indirect tensile strength of typical ODOT mixes. Mix binder type was AC-20 and testing was done at room temperature and in accordance with SHRP test protocol P06.
- Liang (2001) measured the indirect tensile strength of asphalt concrete specimens with five binder types, namely PG 58-28 and four polymer modified asphalt binders.
- Abdulshafi (2002) measured the indirect tensile strength of ODOT HMA specimens with different percentages of recycled asphalt pavement (RAP). The binder type represented in the mixes was PG 64-28.
- Masada and Sargand (2002): Tested indirect tensile strength of typical ODOT mixes. Testing was done at room temperature in accordance with SHRP test protocol P06.

Table 8 presents IDT summary of the test results from these studies. At Level 1, the MEPDG requires HMA mix IDT tested at 14 ^{o}F. The data presented in tables 6 and 7 shows testing at room temperature (approximately 70 to 80 ^{o}F and 25 ^{o}C [77 ^{o}F]). Thus, additional testing is thus required to obtain MEPDG Level 1 input requirements for typical ODOT HMA mixes.

Masada and Sargand (2002) additionally determined in the lab the Poisson's ratio of HMA specimens obtained from the Ohio-SHRP Test Road. The predictive equation that relates the Poisson's ratio with temperature is shown below:

$$\mu = -0.00004(T)^2 - 0.012(T) - 0.2837 \qquad (1)$$

<u>HMA Mix IDT Unconfined Creep and Recovery Testing</u>

The creep test values of asphalt concrete were determined in three studies conducted for ODOT by:

Table 8. HMA indirect tensile strength results.

Data Source	Mix Type	Mixture Type	Indirect Tensile Strength, psi @ ___^{o}C
Liang (2001)	Not Available	Unmodified PG 58-28	105.0 @ Room Temp.
		2% SBR Goodyear + PG 58-28	103.4 @ Room Temp.
		3% SBR Goodyear + PG 58-28	108.7 @ Room Temp.
		4% SBR Goodyear + PG 58-28	102.2 @ Room Temp.
		5% SBR Goodyear + PG 58-28	105.3 @ Room Temp.
		2% SBR Butanol + PG 58-28	118.7 @ Room Temp.
		3% SBR Butanol + PG 58-28	103.7 @ Room Temp.
		4% SBR Butanol + PG 58-28	121.9 @ Room Temp.
		2% SBS Ashland + PG 58-28	114.6 @ Room Temp.
		3% SBS Ashland + PG 58-28	126.4 @ Room Temp.
		4% SBS Ashland + PG 58-28	113.8 @ Room Temp.
		5% SBS Ashland + PG 58-28	116.6 @ Room Temp.
		2% SBS Koch + PG 58-28	107.6 @ Room Temp.
		3% SBS Koch + PG 58-28	134.2 @ Room Temp.
		4% SBS Koch + PG 58-28	136.4 @ Room Temp.
		5% SBS Koch + PG 58-28	128.4 @ Room Temp.
Abdulshafi (2002)	ODOT Item 441 Type 2, Limestone aggregate	0% RAP; 100% PG 64-28	125.7
		10% RAP D;90% PG 64-28	106.5
		20% RAP D;80% PG 64-28	145.1
		30% RAP D;70% PG 64-28	149.2
		10% RAP E;90% PG 64-28	127.7
		20% RAP E;80% PG 64-28	142.9
		30% RAP E;70% PG 64-28	154.6
		10%RAP F; 90% PG 64-28	120.1
		20% RAP F;80% PG 64-28	117.1
		30% RAP F;70% PG 64-28	122.3
	ODOT Item 441 Type 2, Gravel aggregate	0% RAP; 100% PG 64-28	189
		10% RAP D;90% PG 64-28	207
		20% RAP D;80% PG 64-28	202
		30% RAP D;70% PG 64-28	258
		10% RAP E;90% PG 64-28	188
		20% RAP E;80% PG 64-28	187
		30% RAP E;70% PG 64-28	209
		10%RAP F; 90% PG 64-28	174
		20% RAP F;80% PG 64-28	191
		30% RAP F;70% PG 64-28	226

Table 8. HMA indirect tensile strength results, continued.

Data Source	AC Mix	Type	No. of Data Points	ITS (psi) of AC @ 25 °C			
				Min.	Ave.	Max.	Std. Dev.
Liang (1998)	AC20	No aging	3	—	101.7	—	—
		Short-term aging	3	—	145.6	—	—
		Long-term aging	3	—	254.4	—	—
Liang (2001)	PG58-28	Marshall type	3	—	105.0	—	—
Abdulshafi (2002)	ODOT Item 441 Type 2	Intermediate Layer, (Limestone Aggregate)	—	—	125.7	—	—
		Intermediate Layer, (Gravel Aggregate)	N/A	—	189	—	—
Masada & Sargand (2002)	ODOT Item 446	Surface Layer	11	63.3	102.0	139.9	—
		Intermediate Layer	11	74.1	108.0	142.4	—

- Liang (2001).
- Sargand and Kim (2001).
- Masada and Sargand (2002).

Liang (2001) measured the creep compliance of HMA specimens using PG 58-28 and four various polymer modified binders. The test results are summarized in table 9. Sargand and Kim (2001) measured the static creep on the asphalt concrete specimens prepared with three gradations of limestone and two binders (unmodified PG 70-22 and SBS binders) at 41° F (5 °C) and 140 °F (60 °C). The test results are shown in table 10. Masada et al. (2002) measured the creep modulus of 23 core specimens, having dimensions of 4 inches in diameter by 4 inches in height using the SHRP P06 protocol. The creep modulus was measured at the following temperatures: 41 °F (5 °C), 77 °F (25 °C), 104 °F (40 °C), and 140 °F (60 °C). A summary of the creep compliance test results are presented in table 11 for the standard thickness specimens.

At Level 1, the MEPDG requires HMA creep compliance at 3 test temperatures and 7 loading times as indicated in table 2. At Level 2, creep compliance is required at a single test temperature and 7 loading times. The data presented in tables 9 through 11 do not meet these requirements. Also, the test protocols used are different from those required by the MEPDG. Thus, additional testing is required to obtain a full set of MEPDG Level 1 and 2 input requirements for typical ODOT HMA mixes.

Table 9. Static creep test results from Liang (2001).

Source	Mixture Type	Max. Strain after One Hour Loading, in/in	Final Strain after Unloading, in/in	Applied Stress, psi	Creep Compliance, sq. in/lb
Liang (2001)	Unmodified PG 58-28	0.00582	0.00550	2.10	0.00262
	2% SBR Goodyear + PG 58-28	0.00928	0.00918	1.13	0.00814
	3% SBR Goodyear + PG 58-28	0.00461	0.00448	1.06	0.00421
	4% SBR Goodyear + PG 58-28	0.00413	0.00395	1.08	0.00367
	5% SBR Goodyear + PG 58-28	0.00448	0.00433	1.07	0.00405
	2% SBR Butanol + PG 58-28	0.00901	0.00867	2.09	0.00415
	3% SBR Butanol + PG 58-28	0.00686	0.00648	2.07	0.00313
	4% SBR Butanol + PG 58-28	0.00805	0.00769	2.08	0.00370
	2% SBS Ashland + PG 58-28	0.00748	0.00669	4.13	0.00162
	3% SBS Ashland + PG 58-28	0.00739	0.00650	4.15	0.00157
	4% SBS Ashland + PG 58-28	0.00676	0.00579	4.09	0.00142
	5% SBS Ashland + PG 58-28	0.00679	0.00587	4.09	0.00143
	2% SBS Koch + PG 58-28	0.00845	0.00751	5.16	0.00145
	3% SBS Koch + PG 58-28	0.00628	0.00556	5.10	0.00109
	4% SBS Koch + PG 58-28	0.00453	0.00322	6.15	0.00052
	5% SBS Koch + PG 58-28	0.00904	0.00746	6.12	0.00122

Table 10. Static creep test results from Sargand and Kim (2001).

Source	Test Temp, deg C	Agg. Type	Asphalt Type	Void Change, percent	Strain @ 1 h, percent	Permanent Strain, percent	Recovery, percent	Stiffness @ 1 hr, MPa
Sargand and Kim (2001)	60	Coarse	Unmodified	0.3	0.553	0.372	32.7	75.2
			Unmodified	0.2	0.595	0.413	30.6	69.3
			SBS	0.2	0.5	0.354	29.2	82.6
			SBS	0.1	0.453	0.216	52.3	90.2
		Inter	Unmodified	0.2	0.446	0.267	40.2	92.7
			Unmodified	0.4	0.422	0.244	42.2	97.9
			SBS	0.2	0.429	0.254	40.9	96.1
			SBS	-0.1	0.464	0.240	48.2	88.5
		Fine	Unmodified	0.2	0.439	0.219	50.0	94.6
			Unmodified	0.2	0.42	0.280	33.3	98.1
			SBS	0.2	0.39	0.235	39.8	105.8
			SBS	0.2	0.414	0.294	29.0	100.4
	40	Coarse	Unmodified	0.0	0.525	0.350	33.3	78.8
			Unmodified	0.1	0.485	0.341	29.6	85.3
			SBS	0.1	0.428	0.281	34.3	96.7
			SBS	0.0	0.418	0.224	46.5	99.1
		Interm ediate	Unmodified	0.0	0.425	0.203	52.2	97.6
			Unmodified	0.1	0.413	0.336	18.6	99.9
			SBS	0.0	0.370	0.223	39.7	112.1
		Fine	Unmodified	0.1	0.356	0.215	39.5	116.5
			Unmodified	0.1	0.338	0.187	44.7	122.5
			SBS	0.1	0.343	0.217	36.7	119.2
			SBS	0.0	0.309	0.131	57.6	134.0

Table 11. Static creep test results from Masada et al. (2002).

Source	Specimen ID	Creep Modulus, psi @ 1hr			
		41 °F	77 °F	104 °F	140 °F
Masada et al. (2002)	390101	2.694E+04	1.165 E+04	1.758 E+04	3.110 E+04
	390107	2.576 E+04	1.196 E+04	1.403 E+04	2.870 E+04
	390112	3.303 E+04	6.722 E+03	5.890 E+03	5.050 E+02
	390901	2.772 E+04	9.370 E+03	1.057 E+04	9.441 E+03
	390902	4.025 E+04	1.017 E+04	1.074 E+04	2.416 E+04
	390903	2.702 E+04	1.436 E+04	1.673 E+04	3.450 E+04
	Average	3.012 E+04	1.071 E+04	1.259 E+04	2.140 E+04

HMA Mix Georgia Loaded Wheel Testing (GLWT)

In the current ODOT specifications, a "torture" test is required if more than 15 percent of fine aggregate is not meeting fine aggregate angularity (FAA) criteria in Superpave specifications. The standard test method is listed in ODOT Supplemental Specification 1057, "Loaded Wheel Tester Asphalt Mix Rut Testing Method."

The rutting potential of asphalt concrete mixtures was determined in two studies conducted for ODOT— (i.e., Liang (2001) and, Sargand and Kim (2001)). Liang (2001) measured the rut depth of the asphalt concrete specimens using PG 58-28 and four various polymer modified binders. The test results are summarized in table 12. Sargand and Kim (2001) measured the rut depth of the asphalt concrete specimens using PG 70-22 and two various polymer modified binders (SBS and SBR). The rut depth was measured in dry and wet conditions. A test temperature of 140^{o}F (60°C) was used in this study, which is higher than the temperature specified in the ODOT specifications because of very low rutting potential of the heavy-duty mixes used in this study. The test results are summarized in table 13.

The test data from the Georgia Wheel Load Tester provides mix-specific rut susceptibility information which can be used to locally calibrate the MEPDG rutting models.

HMA Mix General Properties

At Level 1, HMA air void, effective binder content, and total unit weight are required by the MEPDG which are then combined with the laboratory E* data to determine its temperature susceptibility and age hardening properties. At Level 3, HMA aggregate gradation along with binder type and mix air void, effective binder content, and total unit weight are required to predict E* and its properties with time and temperature.

These basic HMA properties are typically are obtained from Job Mix Formula (JMF) of typical HMA mixtures used by ODOT. The JMF's of different asphalt pavement mixtures, provided by ODOT engineers, are summarized in table 14. The information presented in table 14 along with selected binder type as specified by ODOT. This information is valuable for Level 3 design.

Table 12. Georgia loaded wheel test results from Liang (2001).

Source	Mixture Type	Rut Depth, mm				
		500 Cycle	1000 Cycle	2000 Cycle	4000 Cycle	8000 Cycle
Liang (2001)	Unmodified PG 58-28	4.45	5.31	6.13	7.19	8.60
	2% SBR Goodyear + PG 58-28	4.65	5.67	6.78	>9.91	>11.00
	3% SBR Goodyear + PG 58-28	3.64	4.81	5.97	7.45	9.37
	4% SBR Goodyear + PG 58-28	3.32	4.47	5.46	6.58	8.14
	5% SBR Goodyear + PG 58-28	4.51	5.60	6.85	8.49	9.86
	2% SBR Butanol + PG 58-28	0.96	1.41	2.02	2.50	3.97
	3% SBR Butanol + PG 58-28	1.48	1.87	2.52	3.81	5.18
	2% SBS Ashland + PG 58-28	1.36	2.07	3.27	3.77	4.94
	3% SBS Ashland + PG 58-28	1.90	2.47	3.20	3.89	4.95
	4% SBS Ashland + PG 58-28	0.93	1.19	1.58	2.37	3.37
	5% SBS Ashland + PG 58-28	0.70	0.88	1.07	1.47	2.33
	2% SBS Koch + PG 58-28	1.45	1.65	1.93	2.00	2.61
	3% SBS Koch + PG 58-28	0.86	1.18	1.15	2.04	2.95
	4% SBS Koch + PG 58-28	0.39	0.53	0.60	0.69	0.84
	5% SBS Koch + PG 58-28	0.55	0.62	0.76	0.83	0.94

Table 13. Georgia loaded wheel tester results from Sargand and Kim (2001).

Source	Gradation	Asphalt Type	Dry Rut Depth, mm @ 8000 cycles	Wet Rut Depth, mm @ 8000 cycles	Dry/Wet Ratio
Sargand and Kim (2001)	Coarse	Unmodified	0.86	1.13	0.76
		SBS	0.72	0.96	0.75
		SBR	1.12	1.09	1.03
	Intermediate	Unmodified	0.99	0.79	1.25
		SBS	0.68	0.90	0.76
		SBR	0.95	1.06	0.90
	Fine	Unmodified	0.81	0.69	1.17
		SBS	0.54	1.01	0.53
		SBR	0.81	1.33	0.61
	Gravel	Unmodified	6.11	4.49	1.36
		SBS	4.73	3.19	1.48
		SBR	4.86	4.16	1.17

Asphalt Binder

The series of tests listed below were conducted to quantify the rheological and mechanical properties of the asphalt binders:.

- Brookfield Rotational Viscometer.
- Dynamic Shear Rheometer (DSR).
- Bending Beam Rheometer (BBR).
- Direct Tension Test (DTT).

Results are discussed in the following sections.

Brookfield Rotational Viscometer

Brookfield rotational viscometer with thermosel was used to measure the viscosity of asphalt binders in two research projects completed by Liang (2001) and Sargand (2001) for ODOT.

Liang (2001) measured the viscosity of polymer modified binders. Testing was done based on ASTM D4402. The viscosity was measured at four different shearing rates (12, 20, 50, and 100 rpm) and at 7 different temperatures (from 250°F to 400°F at 25°F increments). The viscosity values have been typically documented for a representative shear rate of 20 rpm. The four binder modifiers tested were (1) SBS Goodyear modifier, (2) SBR Butanol modifier, (3) SBS Ashland modifier, and (4) SBS Kock modifier. In the study conducted by Sargand (2001), the viscosity was measured for two polymer modified binders, namely SBS and SBR modified PG 70-22, unmodified PG 70-22 and PG 64-22. Table 15 presents a summary of the test results. The short term aged binder data are needed for characterizing the asphalt binders in the MEPDG at Level 2.

Dynamic Shear Rheometer (DSR)

The DSR dynamic shear rheometer is used to characterize the viscous and elastic behavior of asphalt binders at high and intermediate temperatures. Dynamic shear rheometer testing of typical ODOT asphalt binders was done as part of three studies conducted by Liang (2001), Abdulshafi (2002), and Sargand and Kim (2001). The typical test data from the DSR is asphalt binder complex shear modulus G^* and phase angle δ. In the study conducted by Liang (2001), testing was done according to AASHTO TP5. The unaged, short-term aged, and long term-aged polymer modified binders were tested at 8 different temperatures from 34°C to 76°C at 10 rad/s using 25- mm parallel plates to investigate the binder's ability to resist rutting. For more resistance to rutting, a high value of G^* and lower value of δ is desirable. The $G^*/\sin\delta$ (rutting) parameter was chosen as a Superpave asphalt binder specification to indicate rutting resistance, while the G*sinδ (fatigue) parameter was chosen to evaluate the fatigue cracking. The measured values of G* and, δ, are tabulated in table 16 for the four different modified binders (SBS Goodyear modifier, SBR Butanol modifier, SBS Ashland modifier, and SBS Kock modifier).

Table 14. ODOT HMA materials general properties.

HMA Mix Type	HMA General Properties						
	Gradation						
	Percent Retained on ¾-in Sieve	Percent Retained on ⅜-in Sieve “	Percent Retained on No. 4 Sieve	Percent Passing on No. 200 Sieve	Total Unit Weight, pcf	Air Void, percent	Effective Volumetric Binder Content , percent
Item 448-1H	0	21	53	4.8	147	3.5	11.9
Item 441 – Type 1	0	0	46	3.3	145	3.5	12.6
Item 442 – 9.5 mm	0	4	43	4.5	147	4	11.1
Item 441 –Type 2	6	25	52	3.7	149	4	9.8
Item 442 – 19.00 mm	4	29	53	3.9	145	4	10.2
Item 441 – 12.5 mm	0	2	38	4.6	152.9	4.0	10.4
Item 441 – 1H (0.3 pct Type b Fiber)	0	15	52	3.0	147	3.5	11.2
Item 302	15	30	52	3.7	147.6	4.5	9.1

Table 15. Summary of Brookfield rotational viscometer test results.

Source	Asphalt Type	Aging	Viscosity, cP (test temperature, °F)						
			250	**275**	**300**	**325**	**350**	**375**	**400**
Liang (2001)	PG 58-28	Unaged	595	307.5	175	113	75	50	37.5
		Short Term Aged	812	393	213	125	75	50	37.5
		Long Term Aged	1370	610	307	172	106	73	37.5
	PG 58-28 + 2% SBR Goodyear	Unaged	912.5	493.8	313	188	131.3	75	50
		Short Term Aged	1244	596.3	325	188	121.3	87.5	56.3
		Long Term Aged	1987	900	450	263	175	106	75
	PG 58-28 + 3% SBRGoodyear	Unaged	1031	537.5	313	188	125	87.5	62.5
		Short Term Aged	1437	718.8	394	244	162.5	106	75
		Long Term Aged	2362	1019	514	288	187.5	119	81.3
	PG 58-28 + 4% SBRGoodyear	Unaged	1281	612.5	394	238	156.3	106	75
		Short Term Aged	1694	931.3	463	256	162.5	106	75
		Long Term Aged	2818	1240	644	363	225	138	93.8
	PG 58-28 + 5% SBR Goodyear	Unaged	1731	893.8	500	313	206.3	144	93.8
		Short Term Aged	2419	1156	686	419	287.5	181	131
		Long Term Aged	3650	1612	813	456	281.3	188	125
	PG 58-28	Unaged	595	308	175	113	75	50	37.5
		Short Term Aged	812	393	213	125	75	50	37.5
		Long Term Aged	1370	610	307	172	106	73	37.5
	PG 58-28 + 2% SBR Butanol	Unaged	1680	960	680	475	240	125	63
		Short Term Aged	2000	9500	500	300	250	NA	NA
		Long Term Aged	3870	1700	850	550	NA	NA	NA
	PG 58-28 + 3% SBR Butanol	Unaged	2080	1220	800	485	260	250	100
		Short Term Aged	3160	1620	975	685	495	300	240
		Long Term Aged	NA	2450	1250	800	550	440	
	PG 58-28 + 4% SBR Butanol	Unaged	3500	2080	1480	1000	800	505	320
		Short Term Aged	4500	2100	1200	700	500	250	245
		Long Term Aged	NA	NA	NA	NA	NA	NA	NA
	PG 58-28	Unaged	595	307.5	175	112.5	75	50	37.5
		Short Term Aged	812	393	212.5	125	75	50	37.5
		Long Term Aged	1370	610	307	172	106	73	37.5
	PG 58-28 + 2% SBS Ashland	Unaged	1200	637.5	350	225	137.5	100	70
		Short Term Aged	1712	750	390	237.5	145.2	100	70
		Long Term Aged	2500	1025	490	268	162.5	100	75
	PG 58-28 + 3% SBS Ashland	Unaged	1612	837.5	462.5	287.5	187.5	130	95
		Short Term Aged	2490	1083	512.5	300	187.5	125	87.5
		Long Term Aged	3295	1270	587.5	325	200	135	95
	PG 58-28 + 4% SBS Ashland	Unaged	2375	1137	680	400	250	165	112.5
		Short Term Aged	4020	1575	750	412.5	250	165	112.5
		Long Term Aged	5380	1987	812.5	405	250	163	112.5
	PG 58-28 + 5% SBS Ashland	Unaged	4637	2175	1137	617	437.5	300	207
		Short Term Aged	6162	2490	1087	587.5	332	213	142
		Long Term Aged	8890	3112	1150	545	325	213	148

*Test standards are ASTM D4402 and AASHTO TP48.

Table 15. Summary of Brookfield rotational viscometer test results, continued.

Source	Asphalt Type	Aging	Viscosity, cP (test temperature, °F)						
			250	275	300	325	350	375	400
Liang (2001)	PG 58-28	Unaged	595	307.5	175	112.5	75	50	37.5
		Short Term Aged	812	393	212.5	125	75	50	37.5
		Long Term Aged	1370	610	307	172	106	73	37.5
	PG 58-28 + 2% SBS Kock	Unaged	1369	575	337.5	200	125	93.75	62.5
		Short Term Aged	1612	725	375	225	137.5	100	62.5
		Long Term Aged	1706	1118.5	550	300	175	112.5	75
	PG 58-28 + 3% SBS Kock	Unaged	3556	856.25	506.25	318.75	200	137.5	87.5
		Short Term Aged	3663	1412	600	350	206.3	137.5	100
		Long Term Aged	2444	1443.5	693.75	368.5	225	143.75	93.75
	PG 58-28 + 4% SBS Kock	Unaged	4862	1268.5	700	400	250	137.5	87.5
		Short Term Aged	5075	1700	777	400	250	162.5	106.3
		Long Term Aged	4600	1706	950	468.8	337.5	187.5	125
	PG 58-28 + 5% SBS Kock	Unaged	–	2025	887.5	556.25	350	225	150
		Short Term Aged	7844	3300	1225	562.5	350	225	150
		Long Term Aged	–	2768.5	1181	606.3	381.3	243.8	150
Sargand and Kim (2001)	Base PG 64-22	Unaged	–	513	–	148	–	–	–
		Short Term Aged	–	–	–	–	–	–	–
		Long Term Aged	–	–	–	–	–	–	–
	Unmodified PG 70-22	Unaged	–	628	–	16.8	–	–	–
		Short Term Aged	–	–	–	–	–	–	–
		Long Term Aged	–	–	–	–	–	–	–
	SBS PG 70-22	Unaged	–	2172	–	602	–	–	–
		Short Term Aged	–	–	–	–	–	–	–
		Long Term Aged	–	–	–	–	–	–	–
	SBR PG 70-22	Unaged	–	1735	–	500	–	–	–
		Short Term Aged	–	–	–	–	–	–	–
		Long Term Aged	–	–	–	–	–	–	–

*Test standards are ASTM D4402 and AASHTO TP48.

Table 16. Summary of dynamic shear rheometer test results (AASHTP TP 5). (Liang, 2001)

Asphalt Type	Aging	Temperature, °C								
		22	34	40	46	52	58	64	70	76
		Complex Modulus (G*), Pa								
PG 58-28	Unaged	–	–	25128	9179.4	3633.9	1579	737.1	369	–
	STA*	–	–	42458	16047	6368.8	2707.8	1231	594.6	–
	LTA**	–	–	124050	51346	20466.	8495.4	3681.4	1681	–
PG 58-28 + 2% SBR Butanol	Unaged	–	–	41244	17203	7623.0	3545.5	2061.8	1049	555.6
	STA	–	–	104090	42565	18732	8659.7	4188.5	2114.2	1110
	LTA	–	–	284480	133770	65282	35919	21504	16430	–
PG 58-28 + 3% SBR Butanol	Unaged	–	–	43327	21528	10346	4927.2	2450.7	1266.4	680.29
	STA	–	–	122360	49141	22000	10385	5124.1	2631.4	1398.8
	LTA	–	–	325055	134400	64313	31076	14919	7293.9	3694.4
PG 58-28 + 4% SBR Butanol	Unaged	–	–	62961	26478	14228	7024.5	3624.6	1939.7	1077.1
	STA	–	–	66852	37959	16755	8218.4	4058.5	2054.1	1068.9
	LTA	–	–	–	–	–	–	–	–	–
		Phase Angle (δ), degrees								
PG 58-28	Unaged	–	–	80.7	83.3	85.1	86.5	87.6	88.5	
	STA	–	–	76.8	80.2	83	85.1	86.6	87.9	
	LTA	–	–	68.48	72.37	76.38	79.95	82.84	85.16	
PG 58-28 + 2% SBR Butanol	Unaged	–	–	73.17	75.88	78.56	80.65	82.93	85.1	86.65
	STA	–	–	68.19	71.33	74.008	76.71	79.2	81.5	83.32
	LTA	–	–	59.45	63.03	66.98	71.43	75.77	78.85	
PG 58-28 + 3% SBR Butanol	Unaged	–	–	71.32	74.13	75.99	78.69	81.38	83.67	85.37
	STA	–	–	67.72	70.39	72.52	74.71	77.15	79.82	82.34
	LTA	–	–	59.81	63.89	67.02	70.26	73.44	76.39	78.99
PG 58-28 + 4% SBR Butanol	Unaged	–	–	67.83	70.18	72.44	75.15	77.94	80.44	82.33
	STA	–	–	67.86	69.19	71.3	73.75	76.34	79.81	82.63
	LTA	–	–	–	–	–	–	–	–	–
		Complex Modulus (G*), Pa								
PG 58-28	Unaged	–	–	25128	9179.4	3633.8	1579	737.1	369	–
	STA	–	–	42457	16047	6368.8	2707.8	1231	594.6	–
	LTA	–	–	124050	51346	20466	8495.4	3681.4	1681	–
PG 58-28 + 2% SBS Ashland	Unaged	–	61875	26381	11874	5510.6	2688.2	1376.3	739.3	–
	STA	–	–	41669	18722	8917.9	4190.6	2121	1121.9	–
	LTA	–	314200	127990	55557	24524	11292	5447.5	2745.5	1445.3
PG 58-28 + 3% SBS Ashland	Unaged	–	64457	28188	13235	6766.3	3541	1877.4	1019	–
	STA	–	–	44042	20690	10221	5187.8	2729.2	1503.7	–
	LTA	–	269000	111000	50871	23971	11722	5978.5	3164.1	1729.8
PG 58-28 + 4% SBS Ashland	Unaged	–	–	31162	16021	8857.2	4900	2693.5	1510.6	–
	STA	–	–	49579	24213	12503	6698	3727.8	2116.5	–
	LTA	–	283000	117000	56242	28003	14529	7791.5	4315.5	24771
PG 58-28 + 5% SBS Ashland	Unaged	–	–	37716	19601	10785	6186	3592.1	2091.5	1236.3
	STA	–	–	62334	31040	16618	9209	5219.5	3006.9	1693.4
	LTA	–	256000	107000	54042	28279	15612	8892.6	5204	3123.9

*STA: Short Term Aging
**LTA: Long Term Aging

Table 16. Summary of dynamic shear rheometer test results (AASHTP TP 5), continued. (Liang, 2001)

Asphalt Type	Aging	Temperature (°C)								
		22	34	40	46	52	58	64	70	76
		Phase Angle , δ								
PG 58-28	Unaged	–	–	80.7	83.3	85.1	86.5	87.6	88.5	–
	STA	–	–	76.8	80.2	83	85.1	86.6	87.9	–
	LTA	–	–	68.48	72.37	76.38	79.95	82.84	85.16	–
PG 58-28 + 2% SBS Ashland	Unaged	–	71.7	72.3	73.1	74.3	76	77.7	79	–
	STA	–	–	70.1	71.5	72.8	74.2	75.7	77.1	–
	LTA	–	61.3	64.95	67.12	69.31	71.41	73.52	75.71	77.99
PG 58-28 + 3% SBS Ashland	Unaged	–	68.9	68.2	67.4	68.1	70.7	74.3	77.4	–
	STA	–	–	67.2	67.9	68.5	69.5	70.8	72	–
	LTA	–	60.22	63.03	64.55	66.07	67.64	69.44	71.46	73.75
PG 58-28 + 4% SBS Ashland	Unaged	–	–	63.5	61.9	62.3	65	69.2	72.9	–
	STA	–	–	63.2	62.8	62.7	63.6	65.8	68.7	–
	LTA	–	58.39	60.66	61.47	62.27	63.2	64.38	65.86	67.74
PG 58-28 +5% SBS Ashland	Unaged	–	–	60.8	59.6	59.5	61	63.8	67	69.9
	STA	–	–	60.6	60.3	60	60.5	62.5	65.6	68.7
	LTA	–	55.95	57.54	57.79	58.07	58.46	59.1	60.1	61.6
		Complex Modulus (G*), Pa								
PG 58-28	Unaged	–	–	25128	9179.4	3633.8	1579	737.1	369	–
	STA	–	–	42457	16047	6368.8	2707.8	1231	594.6	–
	LTA	–	–	124050	51364	20466	8495.4	3681.4	1681	–
PG 58-28 + 2% SBS Kock	Unaged	–	–	26232	11618	5206.4	2456.2	1300.1	689.95	394.46
	STA	–	–	43952	18222	8092.1	3824.7	1907.8	1002.5	553.4
	LTA	–	–	142120	57959	25052	11401	5431.0	2718.8	1435.6
PG 58-28 + 3% SBS Kock	Unaged	–	–	31572	13350	6479.2	3365.5	1832.6	1031.2	601.5
	STA	–	–	37326	18040	8736.9	4585.4	2532.3	1456.9	867.4
	LTA	–	–	127010	56787	25772	12189	6069.7	3163.9	1727
PG 58-28 + 4% SBS Kock	Unaged	–	–	28726	15012	8085.6	4570.1	2665.3	1601.3	992.5
	STA	–	–	36805	18891	9860.7	5445.3	3177.6	1947.0	1237.
	LTA	–	–	129990	58423	27182	13241	6818.8	3688.8	2073
PG 58-28 + 5% SBS Kock	Unaged	–	–	33424	18510	10470	6195.1	3822.7	2410.9	1537
	STA	–	–	26794	14341	7844.6	4609.5	2842.2	1824.7	1238
	LTA	–	–	112420	53178	25646	13066	7022.7	3943.5	230
		Phase Angle (δ), degrees								
PG 58-28	Unaged	–	–	80.70	83.3	85.1	86.5	87.6	88.5	–
	STA	–	–	76.80	80.2	83	85.1	86.6	87.9	–
	LTA	–	–	68.48	72.37	76.38	79.95	82.84	85.16	–
PG 58-28 + 2% SBS Kock	Unaged	–	–	72.4	73.78	75.4	77.11	78.56	79.75	80.47
	STA	–	–	70.74	72.51	74.03	75.52	77.09	78.62	80.08
	LTA	–	–	63.82	67.09	69.56	71.76	73.94	76.14	78.5
PG 58-28 + 3% SBS Kock	Unaged	–	–	65.34	67.6	68.18	69.5	71.27	72.92	74.11
	STA	–	–	66.23	66.39	66.81	67.27	67.89	68.61	69.44
	LTA	–	–	62.82	65.1	66.83	68.43	70.08	71.96	74.15
PG 58-28 + 4% SBS Kock	Unaged	–	–	60.96	60.11	60	60.47	61.2	62.02	62.7
	STA	–	–	62.68	62.07	61.61	61.41	61.27	61.14	61.14
	LTA	–	–	61.05	62.86	64	65.08	66.27	67.75	69.64
PG 58-28 +5% SBS Kock	Unaged	–	–	57.95	56.46	55.28	54.67	54.77	55.7	57.39
	STA	–	–	58.42	57.06	55.97	55.23	54.69	54.2	52.48
	LTA	–	–	59.95	61.07	61.65	62.18	62.87	63.84	65.32

*STA: Short Term Aging, **LTA: Long Term Aging

In the study conducted by Abdulshafi et al. (2002), the dynamic shear rheometer tests were performed on seven asphalt binders: one virgin binder and six binders that came from RAP. Levels of each RAP addition were 10, 20, and 30percent. Table 17 shows the average dynamic shear rheometer test results at the test temperature of 22°C.

In the study conducted by Sargand and Kim (2001), two polymer modified binder (SBS and SBR modified PG 70-22) were tested. The unmodified PG 70-22 asphalt was also used. Table 18 shows the test results in this study.

The MEPDG requires G* and δ for a minimum of 3 test temperatures to characterize asphalt binders at Level 1. Based on the testing performed, adequate information appears to be available to characterize typical ODOT binders for the MEPDG at Level 1.

Bending Beam Rheometer (BBR)

The BBR ending Beam Rheometer tests were conducted by Liang (2001), Abdulshafi (2002), and Sargand and Kim (2001). Table 19 presents a summary of the test results. All testing was performed in accordance with AASHTO TP1. In the study conducted by Liang (20012), the creep stiffness, which is a measure of how the asphalt binder resists the constant creep loading, and the m-value, which is a measure of the rate at which the creep stiffness changes with the loading time, wereas measured at three temperatures: -12°C, -18°C and -24°C. The temperature that satisfy the Superpave requirement is -18°C, for SBR Goodyear, SBS Ashland and SBS Kock modified binders; while it is -12°C for SBR Butanol. The creep stiffness and m-values were measured at -18°C and -24°C in the study conduced by Abulshafi (2002) and at -12°C and -18°C in the study conducted by Sargand and Kim (2001).

The MEPDG requires creep compliance at seven loading times and three test temperatures (-4 ^{o}F, 14 ^{o}F, and 32 ^{o}F) at Level 1 and at seven loading times and one1 test temperature (14 ^{o}F) at Level 2. Adequate information appears to have been provided by the test data presented to develop default creep compliance values for the typical ODOT binders tested.

Table 17. Average dynamic shear rheometer test results for binders used with both gravel and limestone aggregate.

Source	Asphalt Type	Temperature (°C) @ 22 °C					
		Binders used with Gravel Aggregate			Binders used with Limestone Aggregate		
		G*, kPa	δ deg.	G*sin δ, kPa	G* kPa	δ, deg	G*sin δ, kPa
Abdulshafi et al (2002)	0% RAP; 100% PG 64-28*	2884	49	2522	2884	49	2522
	10% RAP D;90% PG 64-28	5368	47.4	3953	5935	47.8	4395
	20% RAP D;80% PG 64-28	6480	44.7	4562	7276	44.9	5135
	30% RAP D;70% PG 64-28	9300	43	6436	N/A	N/A	7050
	10% RAP E;90% PG 64-28	4645	49.2	3518	5343	48.6	4006
	20% RAP E;80% PG 64-28	5086	46.4	3682	6775	45.2	4805
	30% RAP E;70% PG 64-28	7385	44.8	5200	6723	43.8	5601
	10%RAP F; 90% PG 64-28	4261	49.4	3234	10620**	44.3	7418**
	20% RAP F;80% PG 64-28	5498	46.2	3971	6724	43.4	4624
	30% RAP F;70% PG 64-28	7007	44.6	4921	7481	44.9	5284

* Virgin Binder is PG 64-28.
** Value has to be erroneous.

Table 18. Dynamic shear rheometer test results.

Source	Binder Properties	Base PG 64-22	Unmodified PG 70-22	SBS PG 70-22	SBR PG 70-22
	Original Binder				
Sargand and Kim (2001)	**Pass DSR Temp, °C**	**64**	**70**	**76**	**76**
	G*/sinδ (Minimum 1 KPa)	1.927	1.345	1.671	1.268
	FailDSR Temp, °C	**70**	**76**	**82**	**82**
	G*/sinδ (Minimum 1 KPa)	0.880	0.650	0.927	0.689
	RTFO Residual				
	Pass RTFO Temp,°C	**64**	**70**	**76**	**76**
	G*/sinδ(Minimum 2.2 KPa)	4.629	2.989	2.542	4.306
	Fail RTFO Temp, °C	**70**	**76**	**82**	**82**
	G*/sinδ (Minimum 2.2 KPa)	2.002	1.388	1.395	2.187
	PAV Residual				
	Pass PAV Temp, °C	**25**	**25**	**19**	**19**
	G*/sinδ (Maximum 5000 KPa)	3636	4928	4479	4904
	Fail PAV Temp, °C	**22**	**22**	**16**	**16**
	G*/sinδ (Maximum 5000 KPa)	5225	6902	6365	6825

Table 19. Bending beam rheometer (BBR) test results (AASHTO TP1).

Data Source	Asphalt Type	Temperature, °C					
		-12	-18	-24	-12	-18	-24
		Creep Stiffness [S(t)], MPa			m-value, MPa		
Liang (2001)	PG 58-28	106.122	215.724	434.371	0.3534	0.3019	0.2443
	PG 58-28 + 2% SBR Goodyear	86.354	201.758	365.371	0.3585	0.296	0.2732
	PG 58-28 + 3% SBRGoodyear	86.048	148.236	379.971	0.3535	0.2595	0.2474
	PG 58-28 + 4% SBRGoodyear	74.65	156.36	330.936	0.3647	0.3036	0.2632
	PG 58-28 + 5% SBR Goodyear	81.658	155.288	282.644	0.3561	0.3166	0.2806
		Creep Stiffness [S(t)], MPa			m-value, MPa		
	PG 58-28	106.122	215.724	434.371	0.3534	0.3019	0.2443
	PG 58-28 + 2% SBS Ashland	84.659	183.647	404.739	0.3641	0.3084	0.2627
	PG 58-28 + 3% SBS Ashland	76.063	171.64	355.862	0.3725	0.3148	0.2714
	PG 58-28 + 4% SBS Ashland	73.163	157.585	332.31	0.36554	0.3076	0.2707
	PG 58-28 + 5% SBS Ashland	71.965	147.591	309.657	0.3564	0.3064	0.2744
		Creep Stiffness [S(t)], MPa			m-value, MPa		
	PG 58-28	106.12	215.724	434.371	0.3534	0.3019	0.2443
	PG 58-28 + 2% SBS Kock	35.231	65.0077	93.2887	0.41188	0.40018	0.35244
	PG 58-28 + 3% SBS Kock	33.246	58.7839	86.0489	0.41352	0.37669	0.34869
	PG 58-28 + 4% SBS Kock	30.869	54.0588	88.6888	0.42784	0.37243	0.36162
	PG 58-28 + 5% SBS Kock	28.690	57.0736	282.644	0.43099	0.38129	0.48683
Sargand and Kim (2001)		Creep Stiffness [S(t)], MPa			m-value, MPa		
	Base PG 64-22	155.5	314.5	N/A	0.321	0.270	N/A
	Unmodified PG 70-22	206.0	394	N/A	0.306	0.248	N/A
	SBS PG 70-22	103.5	232	N/A	0.340	0.293	N/A
	SBR PG 70-22	120.0	259	N/A	0.317	0.266	N/A

Table 19. Bending beam rheometer (BBR) test results (AASHTO TP1), continued.

Source	Asphalt Type	Temperature (°C)					
		-12	-18	-24	-12	-18	-24
		Creep Stiffness [S(t)], MPa, with Gravel Aggregate			m-value, MPa, with Gravel Aggregate		
Abdulshafi et al. (2002)	0% RAP; 100% PG 64-28*	N/A	262	N/A	N/A	0.316	N/A
	0% RAP; 100% PG 64-28*	N/A	N/A	585	N/A	N/A	0.254
	10% RAP D;90% PG 64-28	N/A	287	N/A	N/A	0.315	N/A
	20% RAP D;80% PG 64-28	N/A	340	N/A	N/A	0.287	N/A
	30% RAP D;70% PG 64-28	N/A	330	N/A	N/A	0.286	N/A
	10% RAP E;90% PG 64-28	N/A	261	N/A	N/A	0.325	N/A
	20% RAP E;80% PG 64-28	N/A	284	N/A	N/A	0.302	N/A
	30% RAP E;70% PG 64-28	N/A	313	N/A	N/A	0.286	N/A
	10%RAP F; 90% PG 64-28	N/A	N/A	566	N/A	N/A	0.240
	20% RAP F;80% PG 64-28	N/A	N/A	541	N/A	N/A	0.250
	30% RAP F;70% PG 64-28	N/A	N/A	586	N/A	N/A	0.238
		Creep Stiffness [S(t)], MPa, with Limestone Aggregate			m-value, MPa, with Limestone Aggregate		
	0% RAP; 100% PG 64-28*	N/A	262	N/A	N/A	0.316	
	0% RAP; 100% PG 64-28*	N/A	N/A	585	N/A	N/A	0.254
	10% RAP D;90% PG 64-28	N/A	287	N/A	N/A	0.313	N/A
	20% RAP D;80% PG 64-28	N/A	342	N/A	N/A	0.286	N/A
	30% RAP D;70% PG 64-28	N/A	406	N/A	N/A	0.263	N/A
	10% RAP E;90% PG 64-28	N/A	N/A	607	N/A	N/A	0.235
	20% RAP E;80% PG 64-28	N/A	N/A	621	N/A	N/A	0.234
	30% RAP E;70% PG 64-28	N/A	N/A	643	N/A	N/A	0.228
	10%RAP F; 90% PG 64-28	N/A	411	N/A	N/A	0.261	N/A
	20% RAP F;80% PG 64-28	N/A	413	N/A	N/A	0.252	N/A
	30% RAP F;70% PG 64-28	N/A	395	N/A	N/A	0.267	N/A

Direct Tension (DT)

The direct tension (DT) test was used to evaluate the low service temperature resistance to thermal cracking for stiff, ductile asphalt binders with stiffness between 300MPa and 600MPa. The low temperature ultimate tensile strain of the asphalt binder were measured at test temperature -12°C, -18°C, -24°C, and -30°C. The test procedure is given in AASHTO TP3. The asphalt binder must exhibit a failure strain of at least 1.0 percent to meet Superpave binder specifications.

The direct tension failure strain, used to evaluate the low service temperature resistance to thermal cracking, was determined in a study completed by Liang (2001). In this study AASHTO TP3 was used. The low temperature ultimate tensile strain of the asphalt binders were measured at test temperature -12°C, -18°C, -24°C, and -30°C. Table 20 presents a summary of the test results.

Table 20. Direct tension test (DTT) (AASHTO TP3).

Source	**Asphalt Type**	**Temperature (C) at 1%Failure Strain**		
Liang (2001)		**Unaged**	**Short Term Aged**	**Long Term Aged**
	PG 58-28	-25.75	-23.25	-22.5
	PG 58-28 + 2% SBR Goodyear	N/A	-20	-23.5
	PG 58-28 + 3% SBRGoodyear	N/A	-21	-23.25
	PG 58-28 + 4% SBRGoodyear	N/A	N/A	-25.25
	PG 58-28 + 5% SBR Goodyear	N/A	N/A	-26.25
		Unaged	**Short Term Aged**	**Long Term Aged**
	PG 58-28	-25.75	-23.25	-22.25
	PG 58-28 + 2% SBR Butanol	-26.25	-25	-24.5
	PG 58-28 + 3% SBR Butanol	-23.25	-23.75	-25.75
	PG 58-28 + 4% SBR Butanol	-24.5	-23.75	-22.25
	PG 58-28 + 5% SBR Butanol	-23.5	-23.75	-20
		Unaged	**Short Term Aged**	**Long Term Aged**
	PG 58-28	-25.75	-32.25	-22.25
	PG 58-28 + 2% SBS Ashland	-29	-29	-23.25
	PG 58-28 + 3% SBS Ashland	-31	-29.75	-29.25
	PG 58-28 + 4% SBS Ashland	-30.25	-29.75	-28.75
	PG 58-28 + 5% SBS Ashland	-30.5	-31.5	
		Unaged	**Short Term Aged**	**Long Term Aged**
	PG 58-28	-25.75	-23.25	-22.25
	PG 58-28 + 2% SBS Kock	-23.25	-25.5	-16.5
	PG 58-28 + 3% SBS Kock	-30	-26.5	-19.75
	PG 58-28 + 4% SBS Kock	-29.5	-23.5	-16.5
	PG 58-28 + 5% SBS Kock	-28.75	-27.25	-19.25

Asphalt Treated Base (ATB) Materials

Asphalt Treated Base Resilient Modulus

The functional as well as the structural performance of asphalt pavement materials is highly dependent on the temperature to which these materials are exposed. Although HMA pavement is closer to being elastic at low temperatures, it would become viscoelastic at higher temperatures. The engineering properties of the asphalt-treated base were measured in several studies conducted for ODOT—Abdulshafi et al. (1994), Figeroa (2002), Masada and Sargand (2002), and Sargand and Edwards (2002).

Abdulshafi et al. (1994) measured the resilient modulus of ATB (ODOT Item 301) specimens at the normal room temperature in the laboratory according to the AASHTO test method. In the study conducted by Figueroa (2004), the resilient modulus of ATB (ODOT Item 301) and PATB (ODOT Item 302) specimens were measured in the laboratory using the indirect tension test mode according to ASTM D4123 test standard. Masada and Sargand (2002) conducted resilient modulus and indirect tension tests on ATB (ODOT Item 301) core specimens taken from the Ohio-SHRP Test Road site. They found good correlations between the resilient modulus at room temperature and the indirect tensile strength (ITS).

In the study conducted by Sargand and Edwards (2002), the resilient modulus values of ATB (ODOT Item 301) and PATB (ODOT Item 302) placed at the Ohio-SHRP Test Road was estimated through backcalculation using FWD test data. They applied a multiple-elastic layer model to FWD test data to backcalculate the summer and fall resilient modulus values. Table 21 summarizes all the test results reported by the aforementioned researchers.

The MEPDG does not require resilient modulus of asphalt treated materials. MEPDG input requirements are similar to those of HMA mixes.

Asphalt Treated Base Unit Weight

The unit weight of the asphalt stabilized base was reported in two studies—Masada and Sargand (2002) and Figueroa (2004). Table 22 presents the test results reported by the researchers. Unit weight is required by the MEPDG at all levels of inputs. The data provided could be used to develop defaults inputs for the MEPDG based on material type.

Table 21. Resilient modulus results for asphalt treated base (ATB) materials reported in Ohio.

Data Source	ATB Type (Site)	No. of Data Points	Test Temp.	$M_R = K_1 \theta^{K_2}$		$M_R = k_1 P_a (\frac{\theta}{P_a})^{k_2} (\frac{\tau_{oct}}{P_a})^{k_3}$			Resilient Modulus Mr, psi (millions)		
				k_1	k_2	k_1	k_2	k_3	Min.	Max.	Ave.
Abdulshafi et al. (1994)	Item 301 (Jac-35)	21	NA (Std. Room)	NA	NA	NA	NA	NA	NA	NA	0.6
	Item 301 (LIC-70)	21	NA (std. room)	NA	NA	NA	NA	NA	NA	NA	0.43
Masada & Sargand (2002)	Item 301 (Ohio-SHRP)	9	41°F	NA	NA	NA	NA	NA	0.81	1.4	1.03
		9	77°F	NA	NA	NA	NA	NA	0.41	0.79	0.59
		9	104°F	NA	NA	NA	NA	NA	0.18	0.46	0.29
Figueroa (2004)	Item 301	10	41°F	NA	NA	NA	NA	NA	Na	Na	0.61
		10	77°F	NA	NA	NA	NA	NA	NA	NA	0.17
		10	104°F	NA	NA	NA	NA	NA	NA	NA	0.08
Figueroa (2004)	Item 302	10	41°F	NA	NA	NA	NA	NA	NA	NA	0.36
		10	77°F								0.17
		10	104°F								0.12
Sargand & Edwards (2002)	Item 302	150	Summer & Fall FWD	NA	NA	NA	NA	NA	NA	NA	0.25 (as subbase)
Sargand & Edwards (2002)	Item 301 (Ohio-SHRP)	150	Summer & Fall FWD	NA	NA	NA	NA	NA	NA	NA	0.44 (as subbase)
	Item 301 (OHIO-SHRP)	150	Summer & Fall FWD	NA	NA	NA	NA	NA	NA	NA	0.75 (as subbase)

Table 22. Unit weight results for asphalt treated base materials reported in Ohio.

Source	ODOT Materials	No. of Data Points	Temperature	Unit Weight, pcf		
				Min.	Max.	Avg.
Masada and Sargand (2002)	Item 301	9 to 12	77° F	139.3	143.5	142.7
	Item 302		77° F	NA	NA	145.0
Figueroa (2004).	Item 301		77° F	NA	NA	140.0
	Item 302		77° F	NA	NA	140.0

Asphalt Treated Base Poisson's Ratio

The Poisson's ratio of the asphalt treated base was reported in two studies conducted by Masada and Sargand (2002) and Sargand and Edward (2002). The results of the two projects are presented in table 23. Poisson's ratio is required by the MEPDG at all levels of inputs. The data provided could be used to develop defaults inputs for the MEPDG based on material type.

Table 23. Poisson's ratio results for asphalt treated base materials reported in Ohio.

Source	ODOT Materials	No. of Data Points	Temperature	Poisson's Ratio		
				Min.	Max.	Ave.
Sargand and Edward (2002)	Item 301	FWD	T = Temperature, °F	$\mu = 0.00004(T^2 + T) + 0.0345$		
Masada and Sargand (2002)	Item 301	9	41° F	0.1	0.12	0.1
		9	77° F	0.13	0.42	0.26
		9	104° F	0.33	0.5	0.45

Permeable Asphalt Treated Base (PATB) Materials

The engineering properties of the PATBpermeable asphalt-treated base were measured in several studies conducted for ODOT—Liang (2007), Figeroa (2002), Masada and Sargand (2002), and Sargand and Edwards (2002).

In the study conducted by Liang (2007), the resilient modulus of ODOT Item 308 specimens were tested according to AASHTO T294-94I procedure. A haversine load waveform was applied with load pulse duration of 0.1 second and a rest period of 0.9 second. ODOT Item 308 is a mixture of asphalt binder with aggregate in an amount equal to 1.5 to 3.5 percent by weight of the mix. ODOT specifies a PG64-22 asphalt binder for this mix.

In the study conducted by Figueroa (2002), the resilient modulus tests were performed, according to ASTM D4123, on a series of PATB (ODOT Item 302) laboratory test specimens in the indirect tension mode. The average unit weight of the specimens was 140 pcf. The following correlation was established to correlate the resilient modulus with temperature.

$$M_R \text{ (million psi)} = 0.00005(T)^2 - 0.0117(T) + 0.7481$$

where T = temperature in Fahrenheit (°F)

Masada and Sargand (2002) measured the unit weight of the PATB (ODOT Item 302) specimens in the laboratory. The average unit weight of the PATB specimens was measured to be 145.0 pcf. Sargand and& Edwards (2002) estimated both summer and fall resilient modulus values of PATB (ODOT Item 302) placed at the Ohio-SHRP Test Road. They applied a multi-elastic layer model to FWD test data to backcalculate the summer and fall resilient modulus. Table 24 summarizes the results obtained by the above researchers.

Table 24. Resilient modulus results for permeable asphalt treated base (PATB) materials reported in Ohio.

Data Source	**ATB Type (Site)**	**No. of Data Points**	**Test Temp.**	$M_R = K_1 \theta^{K_2}$		$M_R = k_1 P_a (\frac{\theta}{P_a})^{k_2} (\frac{\tau_{oct}}{P_a})^{k_3}$			**Resilient Modulus Mr, psi (million)**		
				k_1	**k_2**	**k_1**	**k_2**	**k_3**	**Min.**	**Max.**	**Ave.**
Liang (2007)	Item–308 (I-90)	3	32 °F	389.41	0.254	12.396	0.26	-0.003	0.174	0.3	0.237
		3	77 °F	275.17	0.208	6.021	0.294	-0.092	0.1	0.1547	0.129
		3	104 °F	37.54	0.44	2.396	0.553	-0.109	0.039	0.1	0.069
		3	Soaking	182.1	0.277	5.895	0.319	-0.052			
Figueroa (2002)	Item 302	10	41°F	NA	NA	NA	NA	NA	NA	NA	0.36
		10	77°F	NA	NA	NA	NA	NA	NA	NA	0.17
		10	104°F	NA	NA	NA	NA	NA	NA	NA	0.12
Sargand & Edwards (2002)	Item 302	150	Summer & Fall FWD	NA	NA	NA	NA	NA	NA	NA	0.25 (as subbase)

Portland Cement Concrete (PCC) Testing Studies

As noted in table 3, the material characterization input parameters required for the new MEPDG to design/analysis of the rigid pavement includes: the elastic constants (elastic modulus, Poisson's ratio) of the PCC to compute the developed stresses and strains in the concrete slab, the modulus of rupture or flexural strength (MR) to estimate the fatigue life of the concrete, the thermal coefficient of expansion (CTE) to calculate the joint opening and curling-induced stresses in the slab, and the composite modulus of subgrade reaction (k-value) to calculate the surface deflections and joint faulting. The following sections describe the various studies that have investigated some of these input parameters for typical ODOT PCC mixes.

PCC Unit Weight

The PCC unit weight (γ) was determined in two studies conducted for ODOT— - Sehn (2002) and Masada and Sargand (2002). In the study conducted by Sehn (2002), the unit weight of 18 PCC eighteen Portland cement concrete specimens were measured according to ASTM C469 in the laboratory. Masada and Sargand (2002) measured the unit weight of 103 PCC core specimens recovered from the Ohio-SHRP Test Road.

Table 25 summarizes the unit weight results obtained in the two studies. From table 25, it can be seen that the unit weight for PCC Class C and Class S are about the same (143 pcf), while for Class F it is slightly lower (138 pcf).

Unit weight is required by the MEPDG at all levels of inputs. The data provided could be used to develop defaults inputs for the MEPDG based on material type.

Table 25. Summary of PCC unit weight test results.

Source	**PCC Mix Type**	**No. of Data Points**	**Unit Weight of PCC, pcf**			
			Min.	**Max.**	**Ave.**	**Std. Dev.**
Sehn (2002)	Class C (Case 1)	10	141.0	145.0	143.0	1.10
	Class S (Case 2)	8	141.0	148.0	144.0	2.88
	HP	NA	NA	NA	NA	NA
Masada & Sargand (2002)	Class F (Case 3)	9	135.9	138.2	138.0	0.79
	Class C (Case 4)	54	140.2	144.8	142.2	1.45
	Class S (Case 5)	40	139.9	145.0	143.1	1.63

Class F = Low Strength Mix; Class C = Regular Strength Mix; Class S = High Strength Mix; HP = High Performance Mix (containing micro-silica).

PCC Elastic Modulus

The 28-day PCC elastic modulus (E_c) was determined in five studies conducted for ODOT—Abdulshafi et al. (1994), Sargand and Cinadr (1997), Sargand et al. (2001b), Masada and Sargand (2002), and Sargand & Edwards (2002).

In the study conducted by Abdulshafi et al. (1994), the static modulus of 80 PCC core specimens obtained from four different rigid pavement sites were measured in the laboratory following ASTM C469 standard. In the study conducted by

Sargand and Cinadr (1997), the elastic modulus was reported without giving any detailed information. The value of the elastic modulus for the high performance mix was also reported by Sargand et al. (2001b). In the study conducted by Masada and Sargand (2002), the modulus of elasticity of 103 PCC core specimens were measured following the ASTM C469 standard.

Table 26 summarizes the static modulus results obtained in the above-referenced studies. Caution should be taken while dealing with the values presented in the table 25, since aging was not clearly mentioned except for the study conducted by Masada and Sargand (2002). From this study, one can notice that the elastic modulus of PCC could be increased by 25 to 30 percent between the ages of 28 days and 1 year for any given mixture. Furthermore, at the age of 28 days, the elastic modulus of Class S mix was about 18 percent higher than that of Class C mix.

The 28-day PCC elastic modulus is a key MEPDG input requirement at Level 3. Also, testing was done in accordance with the MEPDG recommended test protocol (i.e., ASTM C 469). Therefore, the test data presented can be used to develop MEPDG PCC elastic modulus default data for the typical ODOT PCC mixes tested. However, for this data to be more useful and accurate, it should be correlated to the strength test results for the same PCC mixes, if available.

Table 26. Summary of PCC modulus of elasticity results.

Source	PCC Mix Type (Site)	No. of Data Points	28-Day Elastic Modulus, psi (million)			
			Min.	Max.	Ave.	Std. Dev.
Abdulshafi et al. (1994)	Class C (GEA-422)	20	3.2	4.8	3.94	0.398
	Class C (HAM-126)	20	5.0	6.8	5.96	0.484
	Class C (JEF-22)	20	3.7	6.5	4.70	0.167
	Class C (LOR-20)	20	3.0	5.2	4.04	0.665
Abdulshafi et al. (1994)	Class C (ATH-33)	NA	NA	NA	4.00	NA
Sargand (2001b)	HP (ATH-50)	NA	NA	NA	3.70	NA
Masada & Sargand (2002)	Class F (Ohio-SHRP)	1	NA	NA	1.14	NA
	Class C (Ohio-SHRP)	12	2.5	4.38	3.31	0.712
	Class S (Ohio-SHRP)	6	2.87	4.74	3.91	0.649

PCC Poisson's Ratio

The Poisson's ratio (μ) values were determined in a study conducted by Masada and Sargand (2002) for ODOT. In this study, Poisson's ratio of PCC core specimens was measured in the laboratory. This was done with a ring-compressometer attached onto each specimen. Table 27 summarizes the Poisson's ratio value for different types of mixes. It can be seen that the Poisson's ratio values did not vary significantly among the different mixtures. The average value was 0.2 for all mix types. Poisson's ratio is a key MEPDG input requirement. The data presented in table 27 could be used with the MEPDG.

PCC Modulus of Rupture

The modulus of rupture values were determined in three studies conducted for ODOT— – Abdulshafi et al. (1994), Sargand (2001a), and Sehn (2002). Furthermore, moduli of rupture test data produced by ODOT were summarized in the report by Masada and Sargand (2002). In the study conducted by Abdulshafi et al. (1994) the modulus of rupture of PCC specimens obtained from four rigid pavement sites in Ohio were measured in the laboratory following ASTM C78 (AASHTO T 97). Sargand (2001a) measured the modulus of rupture of HP-concrete beam specimens cured in the field following ASTM C78 test standard. Sehn (2002) measured the modulus of rupture in the laboratory following ASTM C78 test standard. The test results produced by ODOT by performing a laboratory test following ASTM C78 standard on beam specimens during the construction of the Ohio-SHRP Test Road were supplied to Masada et al. (2002).

Table 28 reports the results obtained from the aforementioned studies. From this table, it can be seen that the modulus of rupture average value for Class C PCC (regular strength mixture) is about 760 psi which is within a typical range (650 to 800 psi) mentioned in the literature. For Class S (High strength mixtures), the modulus of rupture value is about 880 psi. The result on the HP (high performance mixture) reported by Sargand (2001a) does not provide us with the modulus of rupture value expected for this type of mixes. This is because of the presence of the freezing temperature in the initial stage of the PCC curing. The 28-day PCC modulus of rupture is a key MEPDG input requirement at Level 3. Also, testing was done in accordance with the MEPDG recommended test protocol (i.e., ASTM C78). Therefore, the test data presented can be used to develop MEPDG PCC modulus of rupture default data at Level 3 for the typical ODOT PCC mixes tested.

Table 27. Summary of PCC Poisson's ratio test results.

Source	PCC Mix Type (Site)	No. of Data Points	Poisson's Ratio of PCC			
			Min.	Max.	Ave.	Std. Dev.
Masada & Sargand (2002)	Class F (Ohio-SHRP)	3	0.144	0.213	0.19	0.04
	Class C (Ohio-SHRP)	11	0.133	0.329	0.19	0.05
	Class S (Ohio-SHRP)	6	0.166	0.293	0.21	0.04

Table 28. Summary of PCC modulus of rupture test results.

Source	PCC Mix Type (Site)	No. of Data Points	28-Day Modulus of Rupture, psi			
			Min.	Max.	Ave.	Std. Dev.
Abdulshafi et al. (1994)	Class C (GEA-422)	20	520	737	657	79
	Class C (HAM-126)	20	462	994	814	99
	Class C (JEF-22)	20	526	913	625	42
	Class C (LOR-20)	20	610	960	812	57
Sargand (2001a)	HP (ATH-50)	NA	NA	NA	400	NA
Masada & Sargand (2002)	Class C (Ohio-SHRP)	5	702	880	782	70
	Class S (Ohio-SHRP)	4	784	890	834	43
Sehn (2002)	Class C (Lab)	3	840	865	850	13
	Class S (Lab)	7	770	975	880	74

PCC Compressive Strength

The 28-day compressive strength was measured in four studies conducted for ODOT— Sargand (2001a), Sargand et al. (2001b), Masada and Sargand (2002), and Sehn (2002).

In the study conducted by Sargand (2001a), the 28-day compressive strength for both Class C and HP (High Performance Concrete Mix, containing GGBFS) PCC core specimens were tested at the age of 28 days for the U.S. 50 project in Athens, Ohio. Also, the 28-day compressive strength for HP PCC cores was measured by Sargand et al. (2001b). Following the ASTM C469 test standard, Masada and Sargand (2002) measured the compressive strength of PCC cores from the Ohio-SHRP Test Road. In the study conducted by Sehn (2002), the 28-day compression strength of PCC core specimens were measured in the laboratory following ASTM C39. Table 29 provides a summary of all the test results obtained in these four studies.

Table 29. Summary of PCC compressive strength test results.

Source	PCC Mix Type (Site)	No. of Data Points	28-Day Compressive Str., ksi			
			Min.	Max.	Ave.	Std. Dev.
Sargand (2001a)	Class C (ATH-50)	1	NA	NA	4.4	NA
	HP (ATH-50)	6			4.0	NA
Sargand et al. (2001b)	HP (ATH-50)	1	NA	NA	4.0	NA
Masada & Sargand (2002)	Class C (Ohio-SHRP)	13	4.265	6.340	5.62	0.60
	Class S (Ohio-SHRP)	6	6.020	8.165	7.02	0.95
Sehn (2002)	Class C (Lab)	12	5.4	6.9	6.0	0.55
	Class S (Lab)	13	5.2	6.6	6.0	0.45

The 28-day PCC compressive strength is a key MEPDG input requirement at Level 2. Also, testing was done in accordance with the MEPDG recommended test protocol. Therefore, the test data presented can be used to develop MEPDG PCC compressive strength default data for the typical ODOT PCC mixes tested.

<u>PCC 28-Day Split-Tensile Strength</u>

The 28-day split-tensile strength was measured in five studies conducted for ODOT— - Abdulshafi et al. (1994), Sargand et al. (2001b), Sargand (2001a), Masada and Sargand (2002), and Sehn (2002). In these studies, the ASTM C496 (AASHTO T 198) standard test procedure was followed in conducting the split tensile strengths of PCC core specimens. The results are summarized in table 30.

From table 30, it can be seen that, in most cases, the 28-day split-tensile strength values for Class C mix is ranging between 500 and 560 psi. However, values outside this range were reported. For Class S and Class F mixes, the average values of 540 and 498 were reported, respectively. The result on the HP (high performance mixture) reported by Sargand (2001b) does not provide us with the split-tensile strength values expected for this type of mixes. This is because of the presence of the freezing temperature in the initial stage of the PCC curing.

The 28-day PCC split-tensile strength is a key MEPDG input requirement for continuously reinforced concrete pavement (CRCP) design. However, since this pavement type is not in the current pavement designs of interest to ODOT, the data my not be immediately useful. However, if this pavement type becomes more viable to ODOT in the future, the test data presented here will be useful.

Table 30. Summary of PCC split tensile strength test results.

Source	PCC Mix Type (Site)	No. of Data Points	28-Day PCC Split Tensile Strength, psi			
			Min.	Max.	Ave.	Std. Dev.
Abdulshafi et al. (1994)	Class C (GEA-422)	20	410	627	547	79
	Class C (HAM-126)	20	352	884	704	99
	Class C (JEF-22)	20	416	803	515	42
	Class C (LOR-20)	20	500	850	702	57
Sargand (2001a)	Class C (ATH-50)	NA	NA	NA	553	NA
Sargand et al. (2001b)	HP (ATH-50)	NA	NA	NA	360	NA
Masada & Sargand (2002)	Class F (Ohio-SHRP)	1	NA	NA	498	NA
	Class C (Ohio-SHRP)	12	580.2	580.2	482.8	55
	Class S (Ohio-SHRP)	7	704.8	704.8	516.3	129
Sehn (2002)	Class C (Lab)	3	865	865	850	13
	Class S (Lab)	6	595	595	566	21

The testing herein was done in accordance with the MEPDG's recommended test protocol (i.e., AASHTO T 198). The data quality appears reasonable, with the exception of test data for the HP high performance PCC. Therefore, the test data presented could potentially be used to develop MEPDG PCC split-tensile strength default data for the typical ODOT PCC mixes tested.

PCC Coefficient of Thermal Expansion (CTE)

The thermal expansion coefficient (CTE) values were measured in only one study conducted by Masada and Sargand (2002) for ODOT. The average test results for PCC containing crushed limestone aggregate were reported in this study. For regular strength Class C mix, the value of the thermal expansion coefficient was 6.3×10^{-6} /°F (11.3×10^{-6} /°C), while for the high strength Class S mixes, it was 6.4×10^{-6} /°F (11.6×10^{-6} /°C). These values appear higher than those reported nationally from the LTPP program for limestone aggregate based PCC mixtures. Since the JPCP design is very sensitive to this input, there is the need for additional testing for confirmation as well as to capture the characteristics of other PCC mix types and coarse aggregate types to obtain a full library of CTE default values for pavement design in Ohio.

PCC Coefficient of Drying Shrinkage

The drying shrinkage coefficient (α) values were measured in only one study conducted by Sehn (2002) for ODOT. The coefficient was measured after 64 weeks of drying test in the laboratory following ASTM C157 test standard. For regular strength Class C mix, the value of the dry shrinkage coefficient was 0.05 percent, while for the high strength Class S mixes, it was 0.04 percent. Table 31 presents the test results measured by Sehan (2002).

Table 31. Summary of PCC coefficient of drying shrinkage test results.

Source	PCC Mix Type (Site)	No. of Data Points	PCC Coeff. Of Drying Shrinkage, percent			
			Min.	Max.	Ave.	Std . Dev.
Sehn (2002)	Class C (Lab)	3	0.047	0.051	0.05	0.0023
	Class S (Lab)	5	0.035	0.043	0.04	0.0037

Although, the MEPDG does not require actual test drying shrinkage test values, the use of lab test data (180 days or beyond) to develop confidence in Level 2 and 3 estimates obtained through correlations or defaults is recommended. The data presented in table 30 can be used for reasonableness checks. Additional testing is, however, required to cover all other typical ODOT PCC mix types.

Unbound Base/Subbase/Subgrade Materials

For unbound materials including subgrade soils, the primary inputs required by the MEPDG are:

1. Resilient modulus at optimum moisture content (OMC) and maximum dry density (MDD).
2. OMC and MDD.
3. Specific gravity.
4. Saturated hydraulic conductivity.
5. Soil water characteristics curve (SWCC) parameters.

The resilient modulus input can be obtained as a function of stress state at level 1 for HMA pavements. However, this approach is not recommended at this time in the MEPDG. Table 3 lists the various protocols that can be used to obtain the aforementioned inputs. At level 1, the inputs can be obtained for each soil/unbound material class through direct testing. At levels 2 and 3, they can be estimated with other, more easily obtainable soil properties. For example, at Level 2, the unbound layer M_r can be estimated through correlations with several other commonly tested soil properties noted in table 32. At Level 3, the resilient modulus of unbound materials is selected based on the unbound material classification (AASHTO or USC) either from agency-specific testing or by adopting the MEPDG defaults (provided the agency is satisfied with these values). The MEPDG provides a general range of typical modulus values (based on LTPP averages) for each unbound material classification at their optimum moisture content and maximum dry density.

Table 32. Summary of correlations for the determination of unbound aggregate materials resilient modulus.

Test	Correlation Equation	Test Procedure
California Bearing Ration (CBR)	$M_r = 2555(CBR)^{0.64}$ M_r, psi	AASHTO T193, "The California Bearing Ratio"
R-value	$M_r = 1155 + 555R$ (*20*) M_r, psi	AASHTO T190, "Resistance R-Value and Expansion Pressure of Compacted Soils"
AASHTO layer coefficient	$M_r = 30000\left(\frac{a_i}{0.14}\right)$ M_r, psi	AASHTO Guide for the Design of Pavement Structures
Plasticity Index (PI) and Percent of Material Passing the 75 μm sieve size (w)	$CBR = \frac{75}{1+0.728(wPI)}$	AASHTO T27. "Sieve Analysis of Coarse and Fine Aggregates" AASHTO T90, "Determining the Plastic Limit and Plasticity Index of Soils"
Dynamic Cone Penetrometer (DCP) value	$CBR = \frac{292}{DCP^{1.12}}$	ASTM D 6951, "Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications"

Note: 1 MPa ~ 145 psi, 1 psi ~ 0.0069 MPa.

Unbound Aggregate Base Material Resilient Modulus

Resilient modulus of the base materials were determined in several studies conducted for ODOT— – Liang (2007), Sargand et al. (1991), Abdulshafi et al. (1994), Randolph et al. (2000), Sargand et al. (2001a), Figueroa (2001), Masada and Sargand (2002), and Sargand (2002).

In the study conducted by Liang (2007), three gradations of the ODOT 304 Limestone and 307-IA, median grading of AASHTO No. 57, and ODOT 307 NJ and CE were tested for resilient modulus, M_r, under drained condition in accordance with AASHTO T294. The aggregates were obtained from a project site on Highway I-90 in Ashtabula County, Ohio. Two resilient modulus models were used in this study for characterizing the unbound aggregate behavior: - $K - \theta$ *Model* (Hicks and Monismith 1971) and Uzan Model (1992). Sargand et al. (1991) conducted the resilient modulus test on DGAB (ODOT Item 304). Abulshafi et al. (1994) measured the resilient modulus of DGAB subbase (ODOT Item 304) in the laboratory according to the AASHTO test method.

Randolph et al. (2000) measured the resilient moduli of five different types of base materials according to SHRP P46, SHPR P31, and AASHTO T274. The five base materials are: ODOT Item 304 (limestone), ODOT Item 307-NJ (limestone), ODOT Item 307-IA (limestone), ODOT Item 310 (limestone), and AASHTO #57 (limestone). Sargand et al. (2000a) conducted the resilient modulus testing on ODOT Item 304 (DGAB), ODOT Item 307 (NJ), and ODOT Item 307 (Iowa).

In the study conducted by Sargand et al. (2000b), both laboratory and three nondestructive tests were performed on ODOT Item 304 (DGAB). Masada and Sargand (2002) performed the resilient modulus testing on ODOT Item 304 (DGAB) according to SHRP P 46 Protocol. Figueroa et al (2001) performed the resilient modulus testing on ODOT Item 304 (DGAB). Sargand et al. (2007) measured the resilient modulus for both unbound base materials and stabilized base materials. Laboratory and backcalculated resilient modulus were measured in this study. Tables 33 and 34 summarize the test results reported by the aforementioned studies.

<u>Subgrade Soils Resilient Modulus</u>

Subgrade resilient moduli for various soils were determined in several studies conducted for ODOT— - Liang (2007), Sargand et al. (1991), Figueroa (1994), Sargand (1998), Sargand et al. (1999), Sargand et al. (2000), Masada and Sargand (2002), Figueroa (2004), and Wolfe and Butalia (2004).

Based on his work, Figueroa (1994) proposed simple formulae for predicting the breakpoint resilient modulus of the subgrade soil found in Ohio. Those formulae are presented in table 35 and the model coefficients in table 36.

Wolfe and Butalia (2004) developed a regression model to relate resilient modulus to stress state under which the soil specimen was tested. This model is presented in table 35 and the means to estimate the model coefficients from soil properties is presented in table 37.

Table 33. Summary of constitutive model parameters for typical Ohio unbound aggregate base materials.

Source	ODOT Material	No. of Data Points	$M_R(ksi) = K_1\ \theta^{K_2}$		$M_R(ksi) = P_a k_1 \left[\frac{\theta}{P_a}\right]^{k_2} \left[\frac{\tau_{oct}}{P_a}\right]^{k_3}$		
			k1	**k2**	**k1**	**k2**	**k3**
Laing (2007)	304 – Fine	3	2.9	0.504	1.846	0.559	-0.057
	304 – Median	3	1.23	0.631	1.453	0.664	-0.034
	304 – Coarse	3	2.2	0.600	2.289	0.633	-0.034
	304 – Saturated	3	1.5	0.587	1.469	0.660	-0.074
	No.57	3	2.3	0.554	1.629	0.718	-0.167
	307 – NJ Median	3	2.1	0.589	2.007	0.661	-0.103
	307 – IA Fine	3	21.22	0.562	2.216	0.588	-0.050
	307 – IA Median	3	15.022	0.574	1.953	0.628	-0.540
	307 – IA Coarse	3	19.712	0.546	0.981	0.636	-0.051
Sargand et al (1991)	304 - DGAB	13	5.0	0.5	–	–	–
Sargand et al (2000a)	304 (DGAB)	2	2.8	0.5	–	–	–
	307 (NJ)	4	2.7	0.46	–	–	–
	307 (IA)	4	2.5	0.46	–	–	–
Sargand et al (2000b)	304 - DGAB	10	2.50	0.45	–	–	–
Figueroa (2001)	304 - DGAB	15	9.00	0.33	–	–	–
Masada & Sargand (2002)	304 - DGAB	4	5.00	0.34	–	–	–

Table 34. Summary of unbound aggregate base materials resilient modulus test results.

Source	ODOT Materials	No. of Data Points	Resilient Modulus, ksi		
			Min.	Ave.	Max
Abdulshafi et al. (1994)	304 (DGAB)	20	N/A	15.2 (JAC-35) 14.4 (LIC-70)	17.5
Randolph et al (2000)	304 (DGAB)	11	5.3	34.0	45.7
	307 (NJ)	11	8.5	42.0	59.8
	307 (IA)	11	6.0	30.0	44.6
	310 (DGAB)	15	6.9	32.0	43.3
	57 (AASHTO)	11	5.3	34.0	45.7
Sargand et al (2000a)	304 (DGAB)	2	4.5	10.4	12.5
	307 (NJ)	4	3.6	7.3	10.5
	307 (IA)	4	3.5	6.9	10.0
Sargand et al (2000b)	304 (DGAB)	SSG	13.3	26.9	43.5
		FWD	3.3	32.8	65.0
		GPL	2.0	16.1	40.0
Sargand et al (2001)	307 (NJ)	5	9.0		13.0
Sargand et al (2002)	Item 304 (DGAB)	FWD	5.7 5.6	26.0 (base) 31.0 (subbase)	43 51.2

[Note] SSG: Soil Stiffness gage; FWD: Falling Weight Deflectometer ; GPL: German Plate Load

Table 35. Summary of correlations for the determination of subgrade soils resilient modulus.

Source	Soil Type	Predictor Variables	Prediction Equation
Figueroa (1994)	A-4 , relatively undisturbed	Degree of Saturation, S* $S(\%)=\dfrac{w(\%)}{\left(\frac{\gamma_w}{\gamma_d}\right)-\left(\frac{1}{SG}\right)}$	$M_R(ksi)=-0.3289S+37.908$
	A-4, disturbed-recompacted		$M_R(ksi)=-0.4135S+43.290$
	A-6 , relatively undisturbed		$M_R(ksi)=-0.1206S+17.245$
	A-6, disturbed-recompacted		$M_R(ksi)=-0.2126S+22.263$
	A-7 , relatively undisturbed		$M_R(ksi)=-0.2549S+28.010$
	A-7, disturbed-recompacted		$M_R(ksi)=-0.4031S+41.137$
Figueroa (2002)	A-4, A-6 And A-7-6	FWD deflection (in): δ AC layer thickness: t_{AC} Granular Base layer thickness (in): t_B AC resilient modulus(ksi): $M_{R\text{-}AC}$ A0 through A4: regression constant given in table 35	$M_R(ksi)=\begin{bmatrix}\log(\delta)-A0-A1*(t_{AC})-\\ A2*(t_B)-A3*(M_{R-AC})\end{bmatrix}/A4$ *See table 36 for models to estimate A1 through A4.*
Wolfe and Butalia (2004)	A-4, A-6 And A-7-6	Deviator Stress (kPa), (σ_d) Minor Principal Stress or Confining Stress (kPa), ($\sigma3$)	$\dfrac{M_r}{P_a}=k_1\left[\dfrac{9P_a}{2}\left(\dfrac{1}{3\sigma_d}+\dfrac{\sigma_3}{\sigma_d{}^2}\right)\right]^{k_2}$ *See table 37 for models to estimate k_1 and k_2*
ODOT, Pavement Design Concepts, (1999)	A-4, A-6, and A-7-6	GI (percent passing No. 200 sieve, LL, PI), CBR	M_r = 1200 * CBR

Table 36. Figueroa (1994) M_r prediction model regression coefficients.

Soil Type	Load P, kN	A0	A1	A2	A3	A4
A-4	40.0	-0.67328	-0.01775	-3.9954 E(-4)	-4.6293 E(-5)	-2.9088 E(-3)
	53.4	-0.55789	-0.01746	-4.9750 E(-4)	-4.5820 E(-5)	-2.9211 E(-3)
	66.7	-0.46776	-0.01726	-5.7782 E(-4)	-4.5423 E(-5)	-2.9271 E(-3)
A-6	40.0	-0.67834	-0.01761	-5.6275 E(-4)	-4.5594 E(-5)	-3.2383 E(-3)
	53.4	-0.56192	-0.01731	-6.6143 E(-4)	-4.5079 E(-5)	-3.2862 E(-3)
	66.7	-0.47122	-0.01711	-7.3800 E(-4)	-4.4741 E(-5)	-3.2981 E(-3)
A-7	40.0	-0.61095	-0.01719	-9.1448 E(-4)	-4.3979 E(-5)	-4.4855 E(-3)
	53.4	-0.49853	-0.01674	-1.1505 E(-3)	-4.3822 E(-5)	-4.4056 E(-3)
	66.7	-0.40663	-0.01670	-1.0849 E(-3)	-4.3071 E(-5)	-4.4847 E(-3)

Table 37. Summary of correlations for the determination of model coefficients for Wolfe and Butalia (2004) M_r models in table 35.

Source	Soil Type	Predictor Variables	Prediction Equation

Note: k_1 and k_2 regression parameters as developed by Wolfe and Butalia (2004) are as follow:

$$k_1 = a_1\sigma_3^{a_2} + a_3\left(\frac{S}{100}\right)^{a_4} + a_5 q_u + a_6 PI + a_7(LL - w) + a_8(w_{opt} - w) + a_9(\%Pas\sin g\#200 -$$

$$k_2 = b_1\sigma_3^{b_2} + b_3\left(\frac{S}{100}\right)^{b_4} + b_5 q_u^{b_6} + b_7 PI + b_8 LL$$

where

$$a_1 = a_{11} + a_{12}\left(\frac{w_{opt} - w}{w_{opt}}\right)$$

$$b_1 = b_{11} + b_{12}(w - w_{opt})$$

w_{opt} = optimum moisture content, percent
w = sample moisture content, percent
σ_3 = confining stress, kPa
S = degree of saturation, percent
q_u = unconfined compressive strength, kPa
PI = plasticity index
LL = liquid limit
% passing No. 200 = percent soil particles finer than 0.075mm.

In the study conducted by Liang (2007), the resilient modulus of A-4a at OMC at 2 percent over the OMC, A -6a at OMC and at 2 percent over the OMC were measured in the laboratory following AASHTO T294 procedure. Also, seasonal nondestructive Falling Weight Deflectometer (FWD) tests were conducted at the pavement sections to gain insights into the seasonal variation of the subgrade resilient modulus. Sargand et al. (1991) analyzed the seasonal nondestructive tests data conducted at four pavement sites in Ohio to backcalculate the subgrade resilient modulus using finite element software.

Figueroa (2004) analyzed seasonal FWD test data accumulated at six pavement sites throughout Ohio . FWD data were used by Figueroa (2004) to predict the subgrade resilient modulus of the subgrade soil found in Ohio.

Sargand (1998) performed dynamic cone penetrometer (DCP) tests to estimate the resilient modulus of subgrade soil (A-7-6) in SPS Section 390101 during a forensic study on a failed flexible pavement section at the Ohio-SHRP Test Road site.

The DCP test results were converted to the resilient modulus.

Sargand et al. (1999) analyzed the FWD test data to estimate the subgrade resilient modulus using the backcalculation approach. FWD tests were performed on the top of the subgrade soils at the Ohio-SHRP Test Road. Sargand et al. (2000) measured the resilient modulus of A-6 subgrade soil. Both laboratory and field tests were conducted. In the study conducted by Masada and Sargand (2002), the subgrade resilient modulus was performed on a series of soil samples taken from the Ohio-SHRP Test Road site according to SHRP P-46. Sargand and Edwards (2002) applied a multi-elastic layer model to FWD test data to backcalculate the summer and fall resilient modulus values of the subgrade soils at the Ohio-SHRP Test Road.

In the study conducted by Wolfe and Butalia (2004), the resilient modulus of A-6, A-6 and A-7-6 subgrade soils were measured in the laboratory according to AASHTO T294-94 procedure for unsaturated samples and modified T294-94 procedure (Huang, 2001) for saturated samples. Models used to predict the resilient modulus of cohesive subgrade soils typical of those found in Ohio were evaluated. Also, an improved and more accurate resilient modulus prediction model was been developed and validated in this study.

Table 38 summarizes the subgrade resilient modulus test results obtained by the above researchers. Table 39 shows the resilient modulus prediction using the CBR equations as presented by Liang (2007).

Table 38. Summary of subgrade soil materials resilient modulus test results.

Data Source	Test Method	Resilient Modulus, ksi		
		Min.	Average	Max.
A-1a Soils				
Sargand et al. (1991)	FWD	17.3	31.6	39.0
A-1b Soils				
Sargand et al. (1991)	FWD	32.5	46.5	61.5
A-4 Soils				
Liang (2007)	Lab (AASHTO T294-92I)	7.53	10	14
Sargand et al. (1991)	FWD	13.0	17.4	18.6
Masada & Sargand (2002)	Lab (SHRP P-46)	2.2	11.3	35.7
Sargand & Edwards (2002)	FWD	20.0	22.5	25.0
Wolfe and Butalia (2004)	Lab (AASHTO T294-94)	2.8	6	10.3
Figueroa (2004)	FWD	2.8	4.2	6.1
		33.6	34.8	36.6
A-6 Soils				
Liang (2007)	Lab (AASHTO T294-92I)	7.8	6	7.7
Sargand et al. (1991)	FWD	32.1	24.9	26.0
Sargand et al. (2000)	SSG	9.3	19.4	36.3
	FWD-S. load	2.7	19.3	68.0
	FWD-L. load	3.6	21.3	67.7
	GPL	2.0	12.2	27.9
Masada & Sargand (2002)	Lab (SHRP P-46)	1.7	10.3	29.4
Sargand & Edwards (2002)	FWD	20.0	22.5	25.0
Wolfe and Butalia (2004)	Lab (AASHTO T294-94)	4.8	9.4	15.8
Figueroa (2004)	FWD	20.7	21.8	22.6
		4.1	5.1	6.5
		6.2	10.1	11.9

Table 38. Summary of subgrade soil materials resilient modulus test results, continued.

Data Source	Test Method	Resilient Modulus, ksi		
		Min.	**Ave.**	**Max.**
A-7-6 Soils				
Sargand et al. (1991)	FWD	30.0	36.9	50.0
Sargand (1998)	DCP	2.0	15.0	35.0
Sargand et al. (1999)	FWD	NA	16.8	NA
Masada & Sargand (2002)	Lab (SHRP P-46)	1.6	11.7	25.2
Sargand & Edwards (2002)	FWD	20.0	22.5	25.0
Wolfe and Butalia (2004)	Lab (AASHTO T294-94)	5.3	8.2	16.4
Figueroa (2004)	FWD	8.2	9.9	11.9

Table 39. Summary of resilient modulus prediction using CBR equations.

Material	CBR	Resilient Modulus, MPa	M_R=1500CBR, ksi (MPa)	M_R=2555CBR$^{0.64}$, ksi (MPa)
304-Fine	94	196-570	141 (974)	47 (323)
304-Median	90	164-558	135 (930)	46 (313)
304-Coarse	150	223-778	225(1551)	63 (435)
No.57	N/A	214-590	N/A	N/A

Summary for Unbound Materials and Subgrade Soils Resilient Modulus

Resilient modulus values for unbound materials and subgrade soils have been presented from several sources. Most of the level 2 M_r testing did not follow the protocols required by the MEPDG since they were not available at the time when the research studies reviewed were conducted. However, some of the information is certainly useful for building materials related libraries. A more thorough review of the data sources will be required to determine the usefulness of all the information presented for developing default MEPDG inputs.

Modulus of Subgrade Reaction, k-value

The modulus of subgrade reaction (k) was measured in three studies conducted for ODOT— - Abdulshafi et al. (1994), Sargand et al. (1998), and Sargand et al. (2001). In the study conducted by Abdulshafi et al. (1994), the modulus of subgrade reaction values were determined by conducting a plate load test according to AASHTO T222 test standard. Table 40 summarizes the test results.

Table 40. Summary of modulus of subgrade reaction for typical Ohio subgrade soils.

Site Location	k-value, psi/in
GEA-422	77.5
HAM-126	38.3
JEF-22	23.3
LOR-20	246

In the study conducted by Sargand et al. (1998), the Falling-Weight-Deflectometer (FWD) tests were performed on the subgrade soil at Ohio-SHRP Test Road. Another FWD tests also were also performed by Sargand et al. (2001), on the subgrade soil at U.S. Rt. 35 site in Green County.

Based on the results obtained from these studies, the k-values were estimated by dividing the applied stress (P) by the deflection (δ) under the loading plate. Table 41 summarizes the calculation results. According to the table, it can be seen that the averages of the subgrade reaction modules values are almost the same regardless of the subgrade soil type. The average k values are 2156 pci, 1925 pci and 2065 pci for A-4, A-6 and A-7-6 soils, respectively. However, it must be noted that the k-value estimation method is not what is typically recommended by ASTM and the test values are an order of magnitude higher than the static k-values. These numbers, therefore, cannot be used in design.

The MEPDG requires k-values only as a direct input for the rehabilitation designs of existing rigid pavements. The information presented may be useful in developing default values for rigid pavement rehabilitation design. In addition, the values reported in the literature can be used to check the k-values computed by the MEPDG for new JPCP designs.

Table 41. Summary of backcalculated modulus of subgrade reaction for typical Ohio subgrade soils.

Soil Type and Source	SHRP Section	No. of Data Points	Backcalculated k Value, pci			
			Min.	Ave.	Max.	Std. Dev.
A-4 Sargand et al.(1998)	390110	20	599	1,593	2,842	669
	390160	20	1,293	2,292	3,753	689
	390902	20	596	1,907	3,964	853
A-4 Sargand et al. (2001)	Sta. 410 to 430 @ U.S. Rt. 35 in Green Co.	41 41	225(S)* 445 (L)*	2,379(S) 2,611(L)	8,417(S) 8,327(L)	2,235(S) 2,433(L)
A-6 Sargand et al. (1998)	390111	20	491	2,225	4,540	1,106
	390202	20	423	2,201	4,549	1,249
	390205	20	301	1,147	2,704	662
	390207	20	1,201	2,101	3,281	646
	390211	20	1,099	1,950	2,590	378
	390262	20	739	1,923	3.,967	760
A-7-6 Sargand et al. (1998)	390107	20	978	2,062	3,484	703

* S = Small Load; L = Large Load.

General Physical and Volumetric Properties of Aggregate Materials and Subgrade Soils

Unit Weight and Moisture Content

Subgrade material unit weight and moisture content were measured in several studies conducted for ODOT; Sargand et al. (1999), Sargand et al. (2000), and Masada and Sargand (2002). Table 42 summarizes the unit weight and moisture content measurements for different subgrade materials. The subgrade materials measured were classified based on AASHTO's classification scheme as A-4, A-6, and A-7-6.

Hydraulic Properties

The hydraulic conductivity of the drainable base materials was reported in two studies conducted for ODOT— – Liang (2007), and Randolph (2000). In the study conducted by Liang (2007), the hydraulic conductivity of granular materials was evaluated by the constant head method, as described in ASTM D2434-68 (2000) and AASHTO T215-70 (1993). All of the materials used by Liang (2007) are expected to have a hydraulic conductivity larger than 0.01 cm/s. As a result, the constant head rigid wall permeameter is used in his study. Three different empirical methods were used to predict the hydraulic conductivity of the materials tested. All of these methods are based on the grain size distribution of the materials.

Table 42. Summary of unit weight results for subgrade materials reported in Ohio.

Source	Soil Type	Moisture Content, percent	Unit Weight, pcf
Sargand et al. (1999)	A-4 (Ohio SHRP Test Road)	8.4 to 10.8	118.6 to 123.4
	A-6 (Ohio SHRP Test Road)	8.0 to 10.7	118.2 to 124.2
	A-7-6 (Ohio SHRP Test Road)	7.3	120.6
Sargand et al. (2000)	A-6 (U.S. Rt. 35)	5.8	136.5
	A-6 (U.S. Rt. 35)	8.5	128.5
Masada and Sargand (2002)	A-4 (Ohio SHRP Test Road)	10.0 to 20.0	111.0 to 116.2
	A-6 (Ohio SHRP Test Road)	8.0 to 20.0	100.8 to 120.5
	A-7-6 (Ohio SHRP Test Road)	10.5 to 21.8	111.0 to 116.2

- Hazen Equation, (Hazen, 1974): $K = CD_{10}^2$ where D_{10} represents the particle size at which 10 percent by weight of the sample is smaller, in mm, and C is an empirical coefficient ranging from 1 to 1.5.
- Sherard Equation, (Sherard, 1984): $K = 0.35D_{15}^2$ where D_{15} represents the particles size at which 15 percent by weight of the material is smaller, in mm.
- Moulton Equation, (Moulton, 1980): $K = \frac{6.214x10^5 \, D_{10}^{1.478} \, n^{6.654}}{P_{200}^{0.597}}$ where n is the porosity of the material, P_{200} is the percent of the material finer than No. 200 sieve, the hydraulic conductivity has a unit of ft/day.

Table 43 shows the measured and predicted hydraulic conductivity for the tested materials. As shown in the table, the Moulton and Hazen equations tend to over-estimate the hydraulic conductivity of open graded materials by a factor of four. The best prediction is given by Sherard equation. Randolph et al. (2000) determined the hydraulic conductivity of AASHTO No. 57, AASHTO No.67, ODOT No.304, Iowa DOT mix and New Jersey mix. Table 44 summaries the results of the hydraulic conductivity tests conducted by the above researchers.

Unbound materials and subgrade soils maximum dry unit weight, specific gravity, saturated hydraulic conductivity, and optimum gravimetric water content are computed internally by the MEPDG using gradation and other index properties at level 2 or 3 or can be a direct input at level 1. The information provided can be used to develop defaults for direct inputs or for evaluating estimates generated by the MEPDG.

Table 43. Summary of hydraulic conductivity values (estimated using model).

Material Type	Hydraulic Conductivity, cm/s			
	Measured	Sherard	Hazen	Moulton
No.57 (fine)	-	9.3551	23.0400	18.3922
No.57 (median)	9.37	12.8957	28.3769	32.4501
No.57 (coarse)	-	23.1055	49.0000	-
304 (fine)	0.073	0.0020	0.0018	0.0001
304 (median)	0.50	0.0211	0.0136	0.0016
304 (coarse)	1.92	0.8229	0.5236	-

Table 44. Summary of lab tested hydraulic conductivity values.

Material Type	Critical Hydraulic Gradient	Hydraulic Conductivity	
		k (cm/s)	k (ft/day)
304 - Fine	0.20	0.073	206
304 - Median	0.111	0.50	1417
304 - Coarse	0.034	1.92	5443
No. 57	0.0164	9.37	26563
307 – NJ Fine	0.141	0.788	2234
307 – NJ Median	0.070	1.349	3824
307 –NJ Coarse	0.039	2.8	7850
307 – IA Fine	0.211	0.308	873
307 – IA Median	0.075	0.803	2277
307 –IA Coarse	0.041	2.89	8210
307 – CE Fine	0.079	0.937	2654
307 – CE Median	0.088	1.307	3703
307 –CE Coarse	0.038	3.07	8720
Cement Stabilized	0.015	8.94	25345
Asphalt Stabilized	0.0155	8.84	25061

Chemically Treated Base Materials Testing Studies

Chemically stabilized materials covered in the MEPDG include lean concrete, cement stabilized, cement treated open graded drainage layers, soil cement, lime, cement, and fly ash treated layers. The elastic modulus of the layer is the primary input parameter for chemically stabilized materials. For lean concrete and cement treated materials in new pavements, the elastic modulus is determined using ASTM C 469. For lime stabilized materials, AASHTO T 307 protocols apply.

Permeable Cement Treated Base (PCTB) Materials

PCTB Resilient Modulus

The resilient modulus of the cement-treated base were determined in three studies for Ohio— – Liang (2007), Masada and Sargand (2002), and Sargand and Edwards (2002). In the study conducted by Liang (2007), the resilient modulus of ODOT Item 306 specimens were measured conducted in accordance with the standard testing procedure AASHTO T294I-94, using the MTS 810 testing system. ODOT specification material 306 is a cement treated base material, consisting of a mixture of durable aggregate, portland cement, and water. The minimum cement content according to Ohio Department of Transportation (ODOT) specifications should be 250 lb per cubic yard. The water cement ratio (w/c) shall be approximately 0.36. Masada and Sargand (2002) measured the resilient modulus of ODOT Item 306 core specimens, taken from the Ohio-SHRP Test Road site, following a test procedure similar to ASTM D3496 (Dynamic Modulus Test). In the study conducted by Sargand and Edwards (2002), both summer and fall resilient modulus values of ODOT Item 306, placed at Ohio-SHRP Test Road, were estimated using the MODULUS4.2 backcalculation program. Table 45 summarizes all the test results reported by the aforementioned researchers.

PCTB Poisson's Ratio

The Poisson's ratios of the PCTB were reported in two studies conducted by Masada and Sargand (2002) and Sargand and Edward (2002). In the study conducted by Masada and Sargand (2002), Poisson's Ratio (μ) equal to 0.22 was measured for PCTB (ODOT Item 306). In the study conducted by Sargand and Edward (2002), the Poisson's Ratio (μ) ranged was ranging from 0.15 to 0.20.

PCTB Unit Weight

The unit weight of the PCTB was reported in a study conducted by Masada and Sargand (2002). In this study, PCTB (ODOT Item 306) core specimens taken from the Ohio-SHRP Test Road site were tested for their unit weight values. It was found that the unit weight of the PCTB (ODOT Item 306) ranging from 121.1 pcf and 135.1 pcf.

Table 45. Summary of resilient modulus test results for permeable cement-treated base (PCTB) reported in Ohio.

Source	Material Type and Project	No. of Data Points	No. of Freezing/ Thawing Cycles	$M_R = K_1 \theta^{K_2}$		$M_R = k_1 P_a (\frac{\theta}{P_a})^{k_2} (\frac{\tau_{oct}}{P_a})^{k_3}$			Resilient Modulus Mr (million psi)		
				k1	k2	k1	k2	k3	Min	Max	Ave.
Liang (2007)	Item – 306 (I-90)	3	0	549.22	0.230	16.398	0.212	0.022	0.223	0.366	0.293
		3	5	356.68	0.318	15.223	0.324	-0.003	0.219	0.424	0.314
		3	15	133.8	0.415	8.602	0.438	-0.023	0.125	0.292	0.205
		3	25	113.57	0.384	6.828	0.373	0.021	0.096	0.208	0.146
		3	35	24.15	0.601	3.103	0.718	-0.097	0.0528	0.182	0.11
Masada & Sargand (2002)	Item – 306 (Ohio-SHRP Test Road Site	2	NA	NA	NA	NA	NA	NA	NA	NA	1.7 & 0.6*
Sargand & Edwards (2002)	Item – 306 (Ohio-SHRP Test Road Site		NA	NA	NA	NA	NA	NA	NA	NA	1.25

Traffic Studies

Traffic data are key inputs for the analysis and design of pavement structures in the MEPDG. In the past, the AASHTO Design Guides quantified traffic in terms of equivalent single axle loads (ESALs). However, the MEPDG requires a lot more detailed traffic data. Essentially, the MEPDG requires the raw traffic data used to estimate ESALs, namely, the load distribution on each axle for each month of the design period. In addition, the MEPDG requires other traffic inputs not often considered in pavement design such as wheel wander, 24-hour truck counts, wheelbase distribution (i.e., distance between the drive axle and the first axle on the trailer), etc. The traffic data needs are summarized in the original MEPDG documentation (ARA, 2004) and the MEPDG Manual of Practice (Von Quintus et al., 2007).

ODOT typically collects two categories of traffic data: (a) Weigh-in-motion (WIM) data, (b) Automatic vehicle classification (AVC) data, and (c) Traffic Volume sites. ODOT has approximately the following number of these sites:

- 44 permanent WIM sites
    - 2 Bending plate WIM sites.
    - 31 Piezo WIM sites.
    - 11 WIM sites through LTPP (these have been analyzed in this study and reported in Volume 3).
- Approximately 50 AVC sites to determine length based class.
- Approximately 50 volume sites.

However, much of this information has not been analyzed to date for MEPDG purposes. Recently, Sargand et al. (2007) completed a research project titled "Evaluation of Pavement Performance on DEL 23" for ODOT. The research results documented in this report constitute the latest effort by ODOT to continue monitoring the response and performance of many of the LTPP SPS experimental test sections built on US 23 in Delaware, Ohio. Data in this report cover the years 2000 - 2005.

A Mettler-Toledo WIM system was installed to monitor traffic loading in all four lanes of the test road. The system started collecting useful axle weight data since the end of October 1996 and is still in operation. Sargand et al. (2007), obtained unadjusted monthly summaries of axle weight data from the site and created traffic summary tables grouped by axle type, vehicle class, and number of axles within a weight bin. Only single, tandem and tridem axles were considered.

The report also extends the analysis of WIM data from W-cards collected through April 2005 using one week of good data each month to represent the loading rate for that month. This procedure improved the estimate of accumulated traffic loading carried by these SPS test sections from August 1996 to April 2005. Excel spreadsheets were developed to review the quality of WIM data, to select the best daily files, to fill in missing data when necessary, and to provide the required output.

Three EXCEL spreadsheets were developed to calculate the following five traffic parameters from the daily WIM files: 1) volume by hour and lane, 2) classifications by hour for all four lanes, 3) total weight by hour and lane, 4) total ESALs by hour and lane, and 5) modified daily load spectra of single, tandem, tridem and quad/penta/hex axles for all truck classifications.

Table 46 provides an assessment of traffic data in relation to the MEPDG traffic inputs in terms of sufficient, in-sufficient, or none existent. It can be seen that ODOT needs to continue to compile and analyze traffic data in a format compatible with MEPDG traffic inputs. Further analysis of the traffic data presented is warranted for use in with the MEPDG. This was performed for the local validation/calibration exercise undertaken in this study and documented in Chapter 4 of this Volume of the report as well as in Volume 4 of the report.

Pavement Performance Data

A literature review reveals that there are several studies conducted for ODOT to document typical pavement performance in Ohio, e.g., Sargand, et al. (1998), Sargand and Edwards (2000), Sargand, et al. (2006), Sargand et al. (2007), Liang (2007), and Chou, et al. (2008).

In the study conducted by Sargand, et al. (1998), a forensic investigation on section 390101 of Ohio SHRP test pavement was conducted to obtain critical data relevant to the performance and cause of excessive rutting at a limited number of locations of this section. Non-destructive tests were conducted on each section, including Falling Weight Deflectometer, transverse profiling, Dynamic Cone Penetration tests, and Cone Penetration Test. Trenches were excavated at locations with various levels of distress to measure transverse layer profiles, to determine the thickness of individual material layers, and to obtain material samples for laboratory testing. Analysis of all the collected data was utilized to determine the causes of the localized distresses.

In the study conducted by Sargand and Edwards (2000), the effects of six base types (from two aggregate sources, #57 from Martin-Marietta in Woodville, Ohio considered resistant to D-cracking and #57 from Sandusky Crushed Stone in Parkertown, Ohio considered susceptible to D-cracking) and various design features on the performance of PCC pavement were investigated. The six bases considered included ODOT 304, 310, 307 IA, 307 NJ, and asphalt - and cement-treated free draining bases. Falling Weight Deflectometer (FWD) tests were conducted to determine load transfer on the test sections. Cracks in slabs were also evaluated through inspection and taking concrete cores. These core samples indicated that most of the cracks were initiated at the pavement surface and propagated downward. No D-cracking has been observed in the test sections. An extensive series of laboratory tests has also been completed to determine resilient modulus and strength of each base type. To date, the sections with base 307 NJ and CTFDB are performing poorly and have developed a substantial number of cracks. The asphalt treated free draining (ATFD) base is performing the best of all the test bases. Additional monitoring was deemed necessary to assess the overall performance of each base type and to address potential D-cracking.

Table 46. Traffic data inputs.

Traffic Category	Traffic Parameters	Sufficient	Non-Existent
General Traffic Inputs	Design Period	X	
	AADTT	X	
	Number of Lanes in Design Direction	X	
	Percent of Truck in Design Direction, %	X	
	Percent of Truck in Design Lane, %	X	
	Operational Speed, mph	X	
	Lateral Traffic Wander		X
	Number of Axles /Truck		X
	Axle Configuration		X
	Wheelbase		X
Traffic Volume Adjustment	Monthly Adjustment		X
	Vehicle Class Distribution		X
	Hourly Distribution		X
	Traffic Growth Factor		X
Axle Load Distribution Factors	Single Axle		X
	Tandem Axle		X
	Tridem Axle		X
	Quad Axle		X

In the study conducted by Sargand, et al. (2006), a forensic investigation was performed on sections 390103, 390108, 390109, and 390110 of Ohio SHRP U.S. RT. 23 Test Pavement. A forensic study of Sections 390103, 390108, 390109, and 390110 in the SPS-1 experiment was completed through a series of non-destructive and destructive tests to determine the cause of rutting and localized distresses that had developed in these four pavement sections. Non-destructive testing included photographs of selected areas and referenced by station, distress surveys conducted according to SHRP-P-338 Distress Identification Manual for LTPP. FWD tests and transverse profiles were taken as well. Destructive testing included dynamic cone penetration (DCP) tests, trenching and recovering cores of sections of HMA for laboratory testing. The collected data was utilized to determine the causes of the localized distresses.

In the study conducted by Sargand et al. (2007), the response and the performance of many of the original 40 test sections and several sections constructed later to replace the lighter designs on DEL-23 were monitored. Data in this report cover the years 2000 - 2005. Performance data include observations of various parameters indicative of overall condition and serviceability, such as roughness, rut depth, cracking, skid resistance, and faulting. Visual distress survey at SPS-1, SPS-2, SPS-8 and SPS-9 was conducted. This data could be useful for MEPDG models validation.

ODOT has been monitoring the performance of three other experimental pavements in Ohio during the past few years. These pavements included sections of ATH 50, LOG 33 and ERI/LOR 2. A brief description of the nature of each project follows:

- In 1997, an experimental high-performance jointed concrete pavement was constructed on US 50 east of Athens, Ohio (ATH 50). Dynamic Cone Penetrometer (DCP) profiles were collected in the eastbound driving lane between Stations 381 and 463 in 2004 to determine the cause of some severe slab cracking after two years of service. As a result, seventeen DCP profiles were obtained for this task. Also, in 2004, a comprehensive set of FWD measurements was made to provide additional insights on the performance of various experimental features incorporated into the ATH 50 project.
- Five test sections were constructed on LOG 33 to evaluate the effects of different drainable bases on the overall performance of AC pavement. All sections had a HMA layer thickness of 11 inches. Base materials included: asphalt-treated free-draining base (ATFDB), cement-treated free-draining base (CTFDB), ODOT 307 aggregate with a New Jersey gradation (307NJ), ODOT 307 aggregate with an Iowa gradation (307IA), and ODOT 304 aggregate. Monitoring was halted after Novachip was placed on all sections after the 2001 evaluation. As expected, the CTFDB section had the lowest deflection followed by the 307IA section as shown by the results of FWD measurements taken on April 11, 2002 and May 17, 2004. Serviceability trends in the five test sections, as measured by PSI, have remained relatively constant in all five test sections between 1994 and 2001. Pavement Condition Rating (PCR) performed in all five sections from 1994 to 1999 decreased about the same for all sections. In 2000, the PCR in CTFDB and ATFDB sections increased slightly, while the PCR in the 304 section continued to decrease, and the PCR in 307NJ and 307 IA sections remained steady. In 2001, the PCR in all sections dropped with the ATFDB section having a 15 point structural deduction for extensive cracking. The increased PCR values in 2003 were conjectured to be due to

the application of Novachip on all test sections, which appeared to cover the earlier surface distresses observed in 2001.

- ERI/LOR 2 test pavement was constructed in the westbound lanes of ERI/LOR 2 to evaluate the combined effects of 13- and 25-foot joint spacing with different types of base materials on the performance of PCC pavements. Among the materials used in the bases were asphalt treated free-draining base (ATFDB), cement-treated free-draining base (CTFDB), and ODOT 304, 310, 307IA and 307NJ aggregates. FWD measurements were obtained on the ERI/LOR 2 test pavements in 2002, 2003 and 2004. Slab crack surveys were performed in 1999, 2002, 2003 and 2004. Sections with the 13 foot slabs performed better than the sections with 25 foot slabs on all bases. The 13 foot slabs with ATFDB and 310 performed better than the 13 foot slabs with 307NJ, CTFDB, 304, and 307IA bases. Smoothness was monitored by ODOT with a non-contact profilometer through 2002. The roughness results were consistent with other performance parameters, in that sections with CTB showed early degradation which continued into 2002, and sections with 307NJ base showed a later decline which brought both sections to a lower PSI than the sections with 304, 307IA, 310, and ATB base.

In the study conducted by Liang (2007), both FWD and Profiling tests were performed by ODOT on the asphalt pavement sections built with different permeable base types. The drainable bases include: (a) ODOT 307 base, including IA, NJ, and CE types, (b) ODOT 306 Cement Treated Base, (c) ODOT 308 Asphalt Treated Base.

In the study conducted by Chou, et al. (2008), individual regression, family regression, and Markov probabilistic models were developed based on available data in the ODOT pavement database to forecast future pavement conditions and to determine remaining service life of pavements based on the forecasted conditions. By comparing the predicted conditions with the actual observed conditions, it was found that the Markov model have the highest overall prediction accuracy. It can also predict future distresses as well as the PCR values. Consequently, future rehabilitation needs, planning, and management decision-makings at both project and network levels can be determined.
As a product of this study, the Infrastructure Information System Laboratory at the University of Toledo has developed a Pavement Database for ODOT. The database is in Microsoft Access database format. The ODOT pavement management information system (PMIS) includes the database and a set of reporting tools to extract the data necessary for pavement performance analysis.

The Ohio specific pavement performance data reviewed in the above could be used for both validating and calibrating the MEPDG approach. Tables 47 and 48 provide a summary of the availability of the pavement performance indicators extracted from the cited ODOT sponsored studies for both flexible and rigid pavements. It can be seen that there is no performance data was available for Continuous Reinforced Concrete Pavements (CRCP); this is of little consequence since at this time ODOT is not planning to build CRCP. The data provided by Sargand and his associates (2007) could be considered sufficient enough as a starting point to conduct both MEPDG validation and calibration.

It is worthwhile to mention here that Ohio also has several LTPP GPS (General Pavement Studies) test sections for rigid pavements. The data from these sections was used in addition to the LTPP SPS sections located on the Ohio SHRP Test Road to validate and calibrate the MEPDG distress prediction models.

Table 47. Flexible and rigid pavement performance indicators collected by ODOT.

Pavement Type	Performance Indicator	Sources				
		Sargand, et al. (1998)	Sargand and Edwards (2000)	Sargand, et al. (2006)	Sargand and ORITE staff (2007)	Liang (2007)
HMA Pavement	Rut Depth (Total, HMA, & Unbound Layers)	x		x	x	
	Transverse Cracking					
	Alligator Cracking (Fatigue Cracking)	x			x	
	Top-Down Cracking			x	x	
	Reflective Cracking					
	Smoothness				x	x

Table 48. Flexible and rigid pavement performance indicators, continued.

Pavement Type	Performance Indicator	Sources				
		Sargand, et al. (1998)	Sargand and Edwards (2000)	Sargand, et al. (2006)	Sargand et al (2007)	Liang (2007)
JPCP	Mean Joint Faulting					
	Transverse Cracking		x		x	
	Smoothness					
	Load Transfer efficiency					
	Punchouts					
	Smoothness					

Pavement Construction Data

For the construction database, ODOT contract manager, Mr. Roger Green, provided the project team with an extensive construction database for exploration. Using this database, the project team established a good construction time frame for flexible pavement, rigid pavement, granular base, and subgrade materials. Frequency charts were constructed to explore various aspects of the database.

For example, figure 6 presents shows that the peak construction month for the HMA pavements is September. However, most of the construction actives are performed between June and October. Figure 7 shows that the peak construction period for the concrete pavements is somewhere between August and September. However, most of the construction actives are performed between May and November. Figure 8 shows that the peak construction period for the aggregate base material is August, however, most of the base construction actives are performed between May and November. From Figure 9, it can be seen that the peak construction period for the subgrade material is August. However, most of the subgrade preparation actives are performed between June and October.

Based on the above analysis, we can assume that the typical ODOT new pavement construction season begins in the later part of spring and ends in early fall. Peak construction can be assumed to be September for both asphalt and rigid pavement and August for the unbound materials for a given year. Also, October can be assumed to be the month when the traffic is allowed to use the newly constructed pavement structure.

Climate Related Studies

Climate conditions have a significant effect on the performance of both flexible and rigid pavements. Climatic factors, such as precipitation, temperature, freeze-thaw cycles, and frost penetration depth, play a key role in affecting the material properties and the performance of the pavement. Consequently, the susceptibility of the pavement materials to moisture and freeze-thaw induced deterioration, the drainability of the paving layers, the infiltration potential of the pavement, also define the extent to which the pavement will react to the climatic conditions.

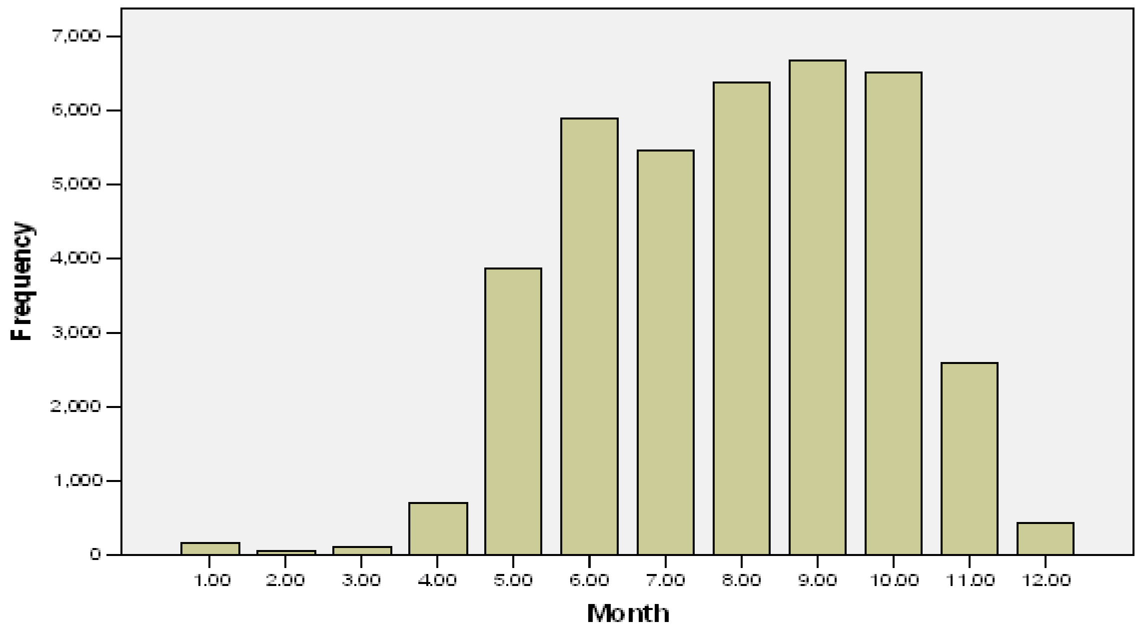

Figure 6. Chart showing frequency distribution of monthly HMA surface course placement (1 = January; 12 = December).

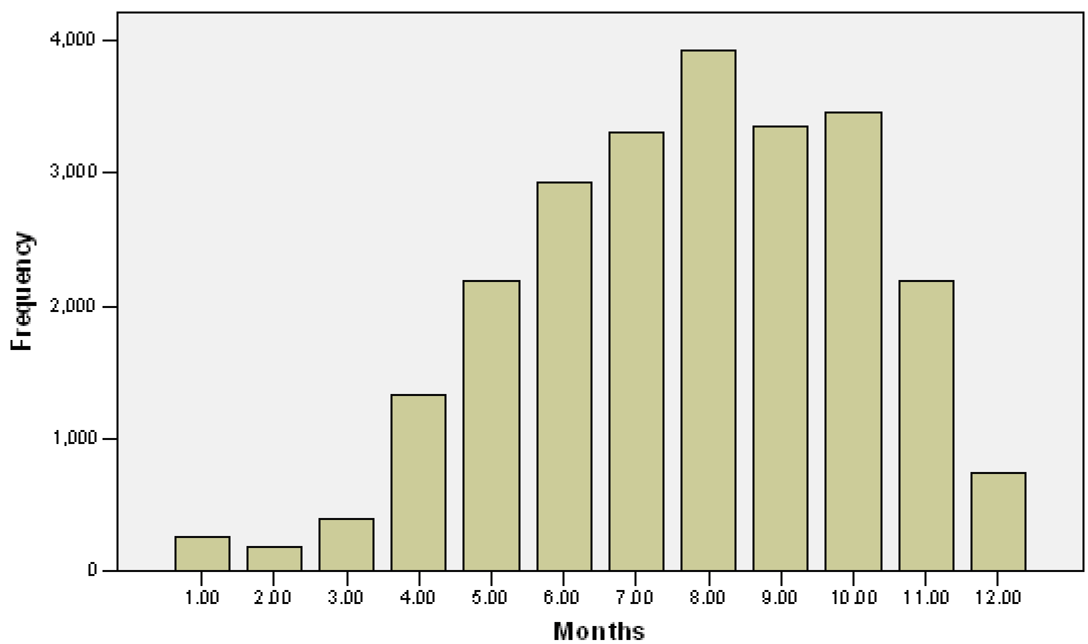

Figure 7. Concrete pavement construction frequency chart.

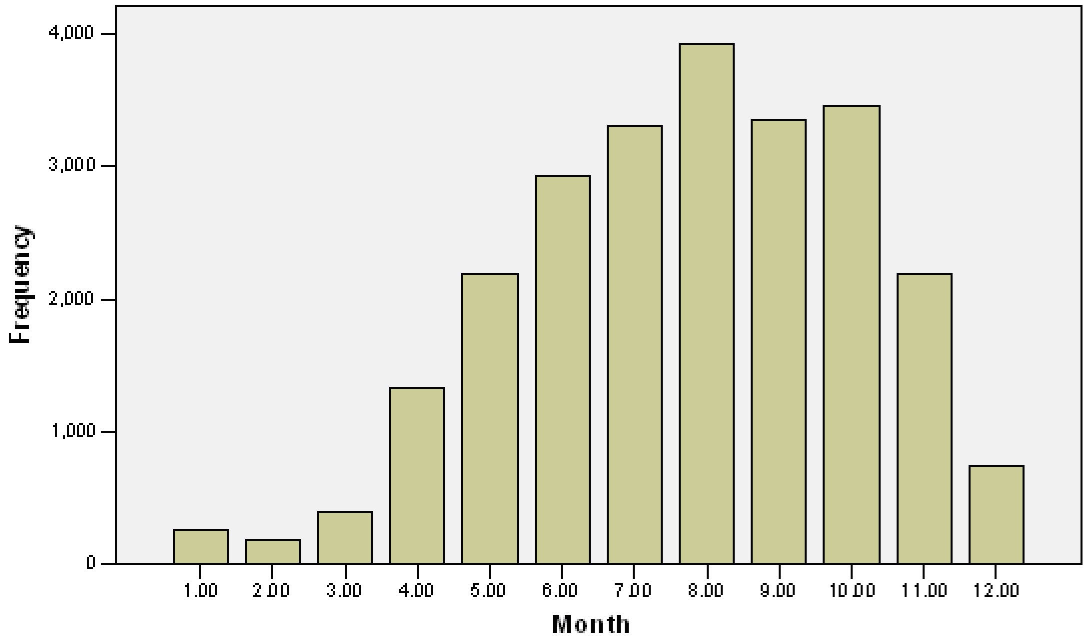

Figure 8. Aggregate base construction frequency chart.

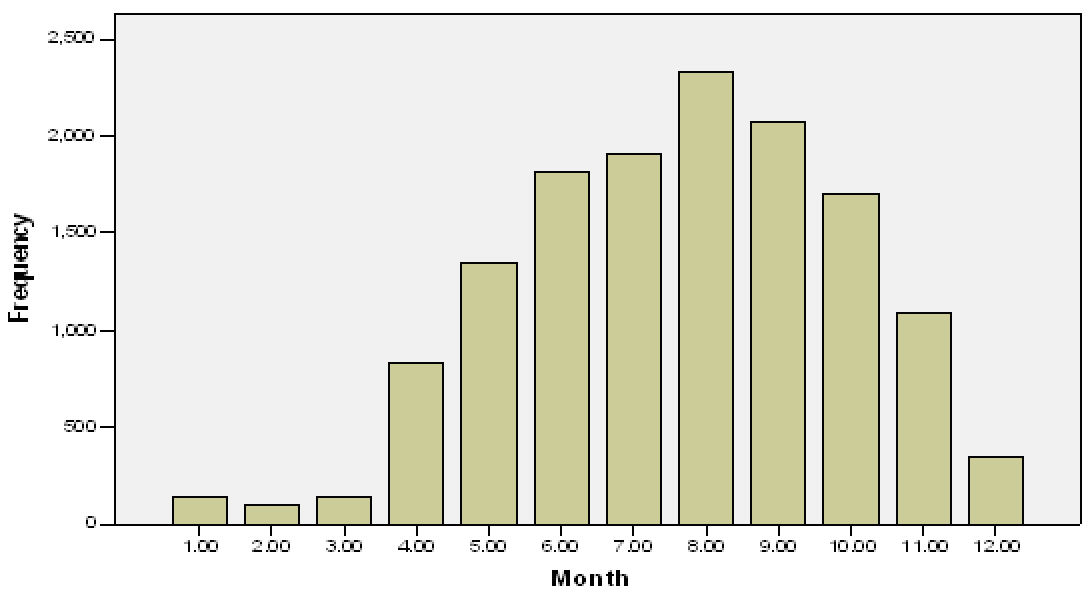

Figure 9. Subgrade construction frequency chart.

As part of the MEPDG, the pavement engineers can carry out numerical simulations using the Enhanced Integrated Climatic Model (EICM) software to compute the moisture, temperature, frost depth of a pavement under the prescribed climatic conditions. Two options to specify the climate file are available. The climate file can be one of the following two sources:

- Import a previously generated climate .icm-file
- Generate the .icm-file using the weather data available in EICM for several weather stations across the United States.

There are several ODOT sponsored studies related to climate condition in Ohio and its effects on pavement performance – Figueroa (2004), Sargand and his associate (2007), and Liang (2007). In the study conducted by Figueroa (2004) "Long-Term Monitoring of Seasonal and Weather Stations and Analysis of Data from SHRP Pavements", field data pertaining to moisture content, pavement and soil temperature and resistivity, as well as weather –related parameters were collected at all instrumented sections at the Strategic Highway Research Program (SHRP) on U.S. 23 north of Delaware, Ohio. Analysis of these field data has led to meaningful findings concerning the relationship between solar radiation and temperature, the development of the predictive equations for asphalt concrete temperature versus air temperature, the better understanding of temperature differentials on PCC slabs, and in-depth understanding of the moisture content in the subgrade soil and depth of frost penetration.

Sargand et al. (2007) completed a research titled "Evaluation of Pavement Performance on Del 23". The research documented in this report was the latest effort by ODOT to continue monitoring the response and performance of many of the original 40 test sections and several sections constructed later to replace the lighter design which, as anticipated, showed early distress. To assist in monitoring climatic changes along the test road, a weather station was installed near the north end of the project and along the east side of the test road to monitor solar radiation, air temperature, wind speed, wind direction, relative humidity, and rainfall. Air temperature and relative humidity were monitored with one probe containing a thermistor and a capacitive relative humidity sensor. An on-site datalogger stores all weather-related measurements.

Liang (2007) conducted a study titled "Evaluation of Drainable Bases under Asphalt Pavements". The objectives of the instrumentation project were to measure the water content, temperature, and frost depth for the asphalt concrete pavement built with different drainable base sections at the Interstate Highway 90 (I-90) in Ashtabula County, Ohio. The instrumentation was designed to provide long term monitoring data. In addition to the embedded sensors, a weather station at the site was installed to monitor the weather conditions, including air temperature, rainfall, wind speed and direction, and solar radiation. Ground water table depth (GWT) was monitored using a piezometer device. Table 49 provides a summary of the weather-related data contained in these three research reports.

The depth of the water table along with the latitude and longitude coordinate of several sections at Ohio SHRP test road were provided by Mr. Roger Green. Table 50 represents the water table depth and location coordinate. Figure 10 shows the variation of water table with time for three sections.

Table 49. Climatic data.

Sources	Weather Station Parameters						Ground Water Table Depth, GWT, ft
	Air Temperature, °C	Wind speed, mph	Wind Direction	Solar Radiation	Precipitation, in	Relative Humidity, percent	
Liang (2007)	X	X	X	X	X		X
Sargand and his associates (2007)	X	X	X	X	X	X	X
Figueroa (2004)	X	X	X	X	X	X	

Table 50. Ohio SHRP Test Road Coordinates and the Depth of Water Table (Office of Pavement Engineering, ODOT).

SHRP Section No.	Section Coordinates			Average Depth of Water Table, ft
	Latitude, (degree. minutes)	Longitude, (degree. minutes)	Elevation, ft	
390102	40º 24' 46" N	83º 04' 32" W	953.7	5.20
390103	40º 25' 32" N	83º 04' 31" W	955.4	8.75
390104	40º 24' 13" N	83º 04' 32" W	956.0	3.50
390108	40º 25' 05" N	83º 04' 35" W	953.4	6.7
390901	40º 23' 16" N	83º 04' 31" W	955.5	8.48
390201	40º 24' 15"N	83º 04' 27" W	954.9	5.31
390204	40º 23' 04"N	83º 04' 29" W	955.6	8.61
390208	40º 25' 16"N	83º 04' 33" W	954.4	8.49
390212	40º 23' 23"N	83º 04' 29" W	957.2	5.55

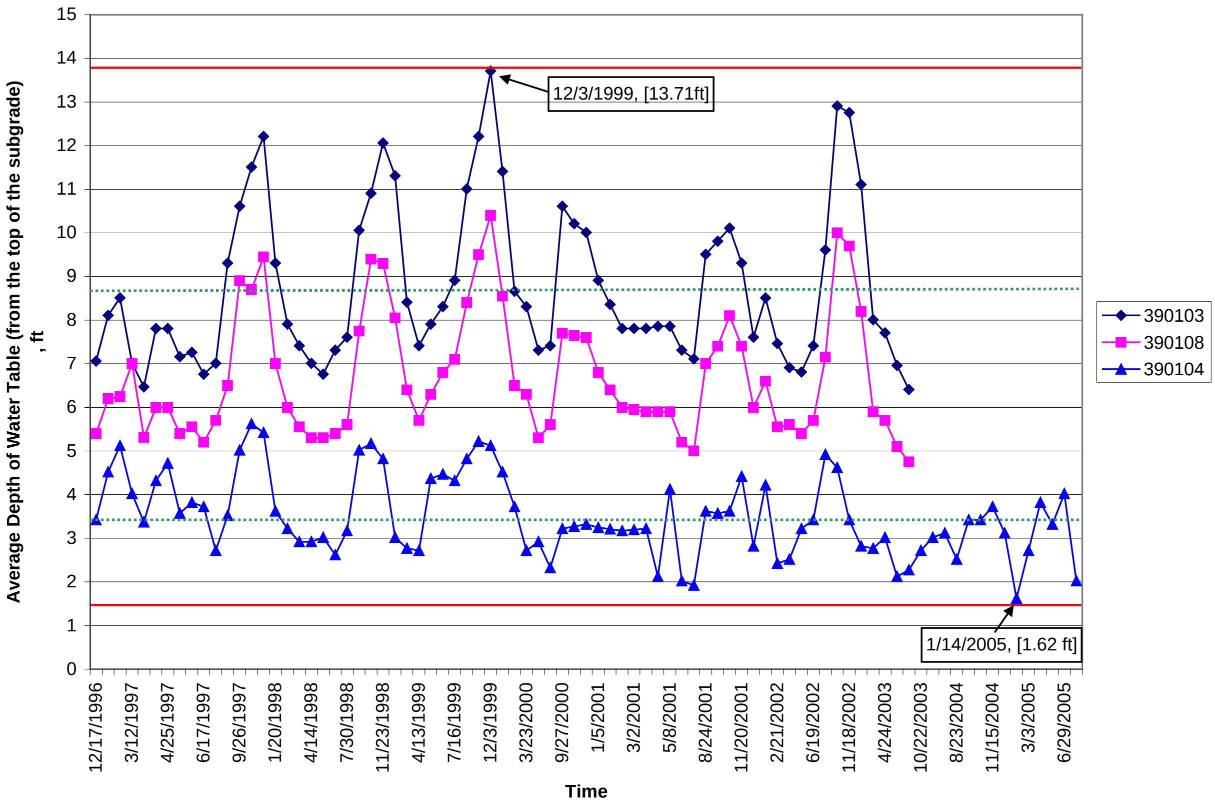

Figure 10. Water depth variation with time.

CHAPTER 4. SUMMARY OF ODOT'S MEPDG INPUT DEFAULTS AND DEFAULT LIBRARIES

The MEPDG procedure requires greater quantity and quality of input data than the current ODOT design procedure in four major categories: traffic, material characterization and properties, environmental influences, and pavement response and distress models. However, the design guide uses a hierarchical approach for inputs which allows the designer flexibility in selecting the design inputs based on the importance of the project and available resources or information. The three hierarchical inputs levels are as follows:

- Level 1 (highest) - Level 1 input requires the highest quality of data. The input data is obtained from direct testing on the actual project material in question; e.g., dynamic modulus testing of an asphalt concrete mix.
- Level 2 (intermediate) - Level 2 input is used when direct test results for a given parameter cannot be obtained but results from other related tests are available which can then be correlated to the required input. For example, if the required Level 1 data parameter is the resilient modulus, M_r of soil but it is not available, then the resilient modulus values are determined through correlations with other more standard testing procedures, such as California Bearing Ratio (CBR) or from soil gradation and plasticity index, etc.
- Level 3 (lowest) - This level of data input requires the lowest level of accuracy and intended for use for lower volume roadways. At Level 3, not only are direct test results (Level 1) unavailable, but secondary test results (e.g., CBR) (Level 2) are also not available. Level 3 permits the user to enter an estimated input value for a given parameter (based on historical agency specifications, test results, or MEPDG supplied national defaults). Typical material property default values derived from the Ohio-specific LTPP database or Ohio construction or PMIS databases can be used for this level of input.

It is possible for a designer to mix and match the levels of input for a specific project during design or even local calibration efforts.

In production type application of the MEPDG design procedure, agencies are expected to use libraries of inputs to define typical project materials, foundation conditions, traffic factors, or climatic variables encountered in their respective States. Level 1 testing of actual project materials will perhaps be done only for the most critical projects. To this end, several agencies are currently undertaking efforts to define these properties as part of their MEPDG implementation activities. These data are being stored in databases for future use as well to confirm/reject national MEPDG defaults.

Flexible Pavement Input Defaults Library

Levels 2 and 3 inputs for flexible pavement are presented in tables 51 and 52.

Rigid Pavement Input Defaults Library.

Based on review of the ODOT sponsored research reports, correlation equations and/or values are recommended to be used as Level 2 or Level 3 inputs in the MEPDG for rigid pavement design. Levels 2 and 3 inputs are presented in tables 53 and 54. The default general PCC pavement input values are presented in table 55.

EICM Input Defaults Library

The MEPDG approach fully considers the effects of the change of temperature and moisture profiles in the pavement structure and subgrade over the design life of a pavement, through the use of climatic modeling software referred to as the Enhanced Integrated Climatic Model (EICM). The EICM predicts variations of temperature and moisture throughout the seasons and within the pavement structure that can be used to adjust the material property for that particular environmental condition. Many of the material properties required by the EICM were reported in throughout several research project reports conducted for ODOT. Table 55 provides a summary of representative EICM input values specific to ODOT specification materials

Climate Input Default Library

In the MEPDG, the variations in temperature and moisture profiles within the pavement structure and subgrade are simulated through an analysis tool called the Enhanced Integrated Climatic Model (EICM). The EICM requires a relatively large number of input parameters. As with all other design inputs, EICM input parameters can be provided at any of the hierarchical levels (1, 2, or 3). Tables 56 and 2.38 present the climatic default values to be used in the MEPDG for flexible and rigid pavement design.

Table 51. Level 2 ODOT Flexible Pavement Properties for MEPDG.

Material Category	M-E Property	Description
HMA	Dynamic Modulus, |E*|	At F = 16 Hz: |E*| (in million psi) = - 0.0005(T)2 – 0.0328(T) + 3.0059 At F = 4 Hz |E*| (in million psi) = 0.0001(T)2 – 0.0420(T) + 2.9224 At F = 1 Hz |E*| (in million psi) = 0.0001(T)2 – 0.0391(T) + 2.5006 where T = Ave. AC temperature (°F).
	Poisson's Ratio, μ	$\mu = -0.00004(T)^2 - 0.012(T) - 0.2837$
	Resilient Modulus	For Surface Layer (ODOT Item 446 - Type 1): MR (ksi) = 3.286*(ITS) + 86.13 For Intermediate Layer (ODOT Item 446 - Type 2): MR (ksi) = 3.020*(ITS) + 149.59 ITS = indirect tensile strength (psi) at 25 °C (77 °F).
	Tensile Strength	N/A "The data available were tested on temperature ranges outside the ranges required by the MEPDG"
	Creep Compliance	N/A "The data available were tested on temperature ranges outside the ranges required by the MEPDG"
	Thermal Conductivity, k & Heat Capacity, Q	N/A
	Surface shortwave absorptivity	N/A
Asphalt Binder	Superpave Binder Test Data	Both dynamic complex modulus (G*) and phase angle (δ) are required for both level 1 and 2. See Chapter 3.
	Conventional Binder Test Data	For PG 58-28 η (cp) = 178901.17 EXP(-0.0229 * T) where T = Temperature in °F. For Other Grades N/A
Base/ Subbase	Resilient Modulus, Mr	For Unbound Materials: MR (ksi) = 1.2(CBR) For Asphalt Treated Base (ATB): $M_R(psi,10^6) = 0.00005(T)^2 - 0.0116(T) + 1.2627$ $M_R(MPa) = 3.0827(T)^2 - 243.47(T) + 5332.6$ $M_R(ksi) = 2.9643(ITS) + 183.06$ Where T = Temperature in ^{o} F ITS = Indirect Tensile Strength For Permeable Asphalt Treated Base (PATB): $M_R(psi,10^6) = 0.00000247(T)^2 - 0.003(T) + 0.320$ Where T = Temperature in ^{o} F

Table 51. Level 2 ODOT Flexible Pavement Properties for MEPDG, continued.

Material Category	M-E Property	Description
Base/Subase	Resilient Modulus, Mr	For Portland Cement Treated Base (PCTB): $M_R(psi,10^6) = 0.000034(FT)^2 - 0.007(FT) + 0.315$ Where FT= Freeze/thaw cycles For Lean Concrete Base (LCB): N/A
	Poisson's Ratio, μ	For Unbound Materials: N/A For Asphalt Treated Base (ATB): $\mu = 0.00004(T^2 + T) + 0.0345$, Where T = Temp, ºF For Permeable Asphalt Treated Base (PATB): $\mu = 0.00004(T^2 + T) + 0.0345$, Where T= Temp, ºF For Portland Cement Treated Base (PCTB): N/A For Lean Concrete Base (LCB): N/A
	Coefficient of Lateral Pressure, k_o	N/A
Subgrade	Resilient Modulus, Mr	See Chapter 3.
	Poisson's Ratio, μ	N/A
	Coefficient of Lateral Pressure, k_o	N/A

Table 52. Level 3 ODOT Flexible Pavement Properties for MEPDG.

Material Category	M-E Property	Description
HMA	Dynamic Modulus, |E*|	At 5 °C (41 °F) |E*| = 1.8 (16 Hz), 1.6 (4 Hz), and 1.2 (1 Hz) million psi At 25 °C (77 °F) |E*| = 0.7 (16 Hz), 0.5 (4 Hz), and 0.3 (1 Hz) million psi At 40 °C (104 °F) |E*| = 0.16 (16 Hz), 0.10 (4 Hz), and 0.07 (1 Hz) million psi
	Poisson's Ratio, μ	At 5 °C (41 °F) Poisson's Ratio (μ) = 0.14 At 25 °C (77 °F) Poisson's Ratio (μ) = 0.35 At 40 °C (104 °F) Poisson's Ratio (μ) = 0.48
	Tensile Strength	N/A "The data available were tested on temperature ranges outside the ranges required by the MEPDG"
	Creep Compliance	N/A "The data available were tested on temperature ranges outside the ranges required by the MEPDG"
	Thermal Conductivity, k & Heat Capacity, Q	See Chapter 3.
	Surface shortwave absorptivity.	N/A
Asphalt Binder	Superpave Binder Grading	PG 64-22, PG 64-28, PG 76-22& PG 70-22M
Base/ Subbase	Resilient Modulus, Mr	For Unbound Materials: Use a typical value of 22.0 ksi For Asphalt Treated Base (ATB): Use 0.55 million psi @ 77 ºF For Permeable Asphalt Treated Base (PATB): Use 0.15 million psi @ 77 ºF For Portland Cement Treated Base (PCTB): Use a typical values of 0.75 million psi. For Lean Concrete Base (LCB): Use a typical value of 1.0 million psi
	Poisson's Ratio, μ	For Unbound Materials: The typical value of 0.35 could be used For Asphalt Treated Base (ATB): Use the following range of typical values: μ = 0.1 to 0.26 For Permeable Asphalt Treated Base (PATB): Use the following range of typical values: μ = 0.1 to 0.26 For Portland Cement Treated Base (PCTB): Use the following range of typical values: μ = 0.15 to 0.20. For Lean Concrete Base (LCB): Use the following range of typical values: μ = 0.15 to 0.20.
	Coefficient of Lateral Pressure, k_o	N/A

Table 52. Level 3 ODOT Flexible Pavement Properties for MEPDG, continued.

Material Category	M-E Property	Description
Subgrade	Resilient Modulus, Mr	• A-1 soil , M_R =32 ksi • A-4 soil, M_R =11 ksi • A-6 soil, M_R =10 ksi • A-7-6 soil, M_R =11 ksi
	Poisson's Ratio, μ	N/A
	Coefficient of Lateral Pressure, k_o	N/A

Table 53. Level 2 ODOT Rigid Pavement Properties for MEPDG.

<table>
<tr><th>Material Category</th><th>M-E Property</th><th>Description</th></tr>
<tr><td rowspan="8">Portland Cement Concrete (PCC)</td><td>Elastic Modulus, Ec</td><td>N/A</td></tr>
<tr><td>Poisson's Ratio, μ</td><td>N/A</td></tr>
<tr><td>Modulus of Rupture, Sc</td><td>N/A</td></tr>
<tr><td>Compressive Strength, fc'</td><td>N/A</td></tr>
<tr><td>Thermal Expansion Coefficient , β</td><td>N/A</td></tr>
<tr><td>Drying Shrinkage, α</td><td>N/A</td></tr>
<tr><td>Thermal Conductivity, k & Heat Capacity, Q</td><td>N/A</td></tr>
<tr><td>Unit Weight, γ</td><td><u>Class C:</u>
γ = [140.2 to 144.8] pcf, Avg. =142.6 pcf
<u>Class S:</u>
γ = [139.9 to 148] pcf, Avg. =143.6 pcf
<u>Class F:</u>
γ = [135.9 to 138.2] pcf, Avg. =138.0 pcf
<u>HP:</u>
N/A</td></tr>
<tr><td>Base/Subbase</td><td>Presented in flexible pavement section</td><td>Presented in flexible pavement section</td></tr>
<tr><td rowspan="2">Subgrade</td><td>Dynamic Modulus of Subgrade Reaction, k</td><td>N/A</td></tr>
<tr><td>Others presented in flexible pavement section</td><td>Others presented in flexible pavement section</td></tr>
</table>

Table 53. Level 3 ODOT Rigid Pavement Properties for MEPDG, continued.

Material Category	M-E Property	Description
Portland Cement Concrete (PCC)	Elastic Modulus, Ec	Class C: Ec = 4.3 E+06, psi Class S: Ec = 4.0 E+06, psi Class F: Ec = 1.1 E+06, psi HP: Ec = 3.7 E+06, psi
	Poisson's Ratio, μ	Class C: μ = [0.133 – 0.329], Avg. = 0.19 Class S: μ =[0.133 – 0.329], Avg. = 0.21 Class F: μ = [0.133 – 0.329], Avg. = 0.19 HP: N/A
	Modulus of Rupture, Sc	Class C: Sc = [625 - 850] psi, Avg. = 750 psi Class S: Sc = [834 - 880] psi, Avg. = 850 psi Class F: N/A HP: N/A
	Compressive Strength, fc'	Class C: fc' = [4.2 – 6.9] ksi, Avg. = 5.3 ksi Class S: fc' = [5.2– 8.1] ksi, Avg. = 6.5 ksi Class F: N/A HP: fc' = 4.0 ksi

Table 53. Level 3 ODOT Rigid Pavement Properties for MEPDG, continued.

Material Category	M-E Property	Description
PCC	Thermal expansion coefficient , β	Class C: $\beta = 6.3 \times 10^{-6}$ / °F Class S: $\beta = 6.4 \times 10^{-6}$ / °F Class F: N/A HP: N/A
	Drying shrinkage, α	Class C: $\alpha =$ [0.047 – 0.051] %, Avg. = 0.05 % Class S: $\alpha =$ [0.035 – 0.043] %, Avg. = 0.04 % Class F: N/A HP: N/A
	Thermal conductivity, k & heat capacity, Q	See Table A.25 (Appendix A)
	Unit weight, γ	The same as Level 2
Base/subbase	Presented in flexible pavement section	Presented in flexible pavement section
Subgrade	Dynamic modulus of Subgrade reaction, k	A-4: k (min) = 632 pci A-6: k (min) = 709 pci A-7-6: k (min) = 978 pci
	Others presented in flexible pavement section	Others presented in flexible pavement section

Table 54. General PCC pavement inputs.

General Inputs	Type of Design	
	JPCP	CRCP
Slab Thickness, in	10	N/A
Slab Width, ft	12	N/A
Dowel Bar Diameter, in	1.25 or 1.5	N/A
Dowel Bar Spacing, ft	12	N/A
Joint Spacing, ft	15	N/A
Sealant Type	- hot-applied sealants - silicone sealants - preformed compression seals	N/A
Steel Reinforcement, percent	N/A	N/A
Edge Support	Tied PCC Shoulder	N/A
Pavement cur/wrap Effective Temperature Difference, °F	N/A	N/A
Base/Subbase Friction Coefficient	N/A	N/A
Erodibility Index	N/A	N/A
PCC-Base Interface	N/A	N/A
Loss of Full Friction (in months)	N/A	N/A
Load Transfer Efficiency (LTE), %	87%	N/A
Cement Type	Type I	N/A
Cementitious Material Content (lb/yd ^{3})	Regular Strength 510 lb/yd ^{3} High strength 750 lb/yd ^{3} Low Strength 350 lb/yd ^{3}	N/A
w/c	0.44	N/A
Aggregate Type	Limestone	N/A
PCC Zero-Stress Temperature, °F	N/A	N/A

Table 55. EICM ODOT default input library.

<table>
<tr><th rowspan="2">Source</th><th rowspan="2">Layer</th><th rowspan="2">Layer Type</th><th colspan="2">Atterberg Limits</th><th colspan="3">Sieve Analysis*, Percent passing</th><th rowspan="2">Maximum Dry Unit Weight (MDD), (pcf)</th><th rowspan="2">Specific Gravity of Solids</th><th rowspan="2">Saturated Hydraulic Conductivity, (ft/hr)</th><th rowspan="2">Optimum Gravimetric Moisture Content (OMC), %</th><th rowspan="2">SWCC*** Parameters</th></tr>
<tr><th>PI</th><th>LL</th><th>No. 200</th><th>No. 4</th><th>D60</th></tr>
<tr><td rowspan="9">Liang (2007)</td><td rowspan="6">Base</td><td>ODOT Item 304</td><td>0</td><td>N/A</td><td>6.5</td><td>45.0</td><td>11.9</td><td>130</td><td>2.59</td><td>F** =8.5, M=59, C=226</td><td>N/A</td><td>N/A</td></tr>
<tr><td>ODOT Item 307, NJ</td><td>0</td><td>N/A</td><td>1.2</td><td>47.0</td><td>12.2</td><td>127</td><td>2.6</td><td>F=93, M=159, C=327</td><td>2</td><td>N/A</td></tr>
<tr><td>ODOT Item 307, IA</td><td>0</td><td>N/A</td><td>0.7</td><td>40.0</td><td>11.9</td><td>128</td><td>2.6</td><td>F=36, M=95, C=342</td><td>2</td><td>N/A</td></tr>
<tr><td>ODOT Item 307, CE</td><td>0</td><td>N/A</td><td>1.18</td><td>28.0</td><td>16.1</td><td>127</td><td>2.6</td><td>F=111, M=154, C=363</td><td>2</td><td>N/A</td></tr>
<tr><td>ODOT Item 308</td><td>0</td><td>N/A</td><td>0.5</td><td>5.0</td><td>16</td><td>106</td><td>2.6</td><td>1056</td><td>N/A</td><td>N/A</td></tr>
<tr><td>ODOT Item 306</td><td>0</td><td>N/A</td><td>0.5</td><td>5.0</td><td>16</td><td>106</td><td>2.65</td><td>1044</td><td>N/A</td><td>N/A</td></tr>
<tr><td>Subbase</td><td>ODOT 304</td><td>0</td><td>N/A</td><td>4.4</td><td>33.0</td><td>22.4</td><td>122</td><td>2.6</td><td>266</td><td>N/A</td><td>N/A</td></tr>
<tr><td rowspan="2">Subgrade</td><td>A-4a</td><td>8</td><td>27.8</td><td>56.3</td><td>94.7</td><td>N/A</td><td>113</td><td>2.70</td><td>N/A</td><td>14.2</td><td>N/A</td></tr>
<tr><td>A-6a</td><td>12.3</td><td>30.8</td><td>68.8</td><td>94.1</td><td>N/A</td><td>112.7</td><td>2.75</td><td>6.1×10^{-6}</td><td>16.5</td><td>N/A</td></tr>
<tr><td rowspan="6">Randolph et al. (2000)</td><td rowspan="6">Base</td><td>ODOT Item 304</td><td>0</td><td>N/A</td><td>7.0</td><td>45.0</td><td>N/A</td><td>130.6</td><td>2.61</td><td>F**=4.5, M=8.7, C=49</td><td>N/A</td><td>N/A</td></tr>
<tr><td>ODOT Item 307, NJ</td><td>0</td><td>N/A</td><td>0</td><td>47.5</td><td>N/A</td><td>108.4</td><td>2.61</td><td>310.6</td><td>N/A</td><td>N/A</td></tr>
<tr><td>ODOT Item 307, IA</td><td>0</td><td>N/A</td><td>3.0</td><td>NA</td><td>N/A</td><td>118.1</td><td>2.62</td><td>F**=55, M=105, C=410</td><td>N/A</td><td>N/A</td></tr>
<tr><td>ODOT Item 310</td><td>0</td><td>N/A</td><td>6.0</td><td>62.5</td><td>N/A</td><td>128.2</td><td>2.59</td><td>F**=0.8, M=4.25, C=525</td><td>N/A</td><td>N/A</td></tr>
<tr><td>AASHTO No.57</td><td>0</td><td>N/A</td><td>0</td><td>5.0</td><td>N/A</td><td>99.2</td><td>2.67</td><td>1444</td><td>N/A</td><td>N/A</td></tr>
<tr><td>AASHTO No. 67</td><td>0</td><td>N/A</td><td>0</td><td>5.0</td><td>N/A</td><td>99.4</td><td>2.67</td><td>2002</td><td>N/A</td><td>N/A</td></tr>
</table>

* Details Sieve Analysis is presented in Table 2.48, **F= Fine gradation, M= Median, C= Coarse, *** SWCC = Soil water Characteristic Curve

Table 55. EICM ODOT default input library, continued.

Source	Percent Passing, %	Base Course							Subbgrade	
		AASHTO No. 57	**ODOT Item 304**	**ODOT Item 307, NJ**	**ODOT Item 307, IA**	**ODOT Item 307, CE**	**ODOT Item 308**	**ODOT Item 306**	**A-4**	**A-6**
Liang (2007)	2″	100	100	100	100	100	100	100		
	1 ½″	100		100	100	100	100	100		
	1″	97.5	85	97.5	100	85	97.5	97.5		
	¾″		70			72.5				
	½″	42.5		70	65	57.5	42.5	42.5		
	3/8″					47.5				
	No.4	5	45	47.5	22.5	27.5	5	5	94.7	94
	No.8	2.5		15		15	2.5	2.5	88	88
	No.16			4		6				
	No.30		21							
	No.40								76	81
	No.50			2.5	7.5	3				
	No.100								66	75
	No.200	0.5	7.5	1	3		0.5	0.5	56	68

Table 56. Climatic input default library.

Input Category	Input Parameters	Input Values
General information	Base/Subbase construction completion date	August
	Pavement construction date	September
	Traffic opening date	October
Weather-related information	Hourly air temperature	Ohio weather stations included in MEPDG database*
	Hourly precipitation	
	Hourly wind speed	
	Hourly relative humidity	
	Hourly percentage sunshine	
Ground water related information	Ground water table depth	Site specific (e.g., in the study conducted by Liang (2007) the water table depth was 20 ft.
Asphalt and Portland cement concrete Pavement materials	Thermal conductivity and Heat capacity	For Asphalt Cement: See Table A.50 For Portland Cement N/A
Unbound materials Pavement materials	Specific gravity	Refer to EICM default input library
	Maximum dry unit weight	
	Optimum moisture content	
Unbound materials Pavement materials	Soil water characteristic curve parameters (SWCC)	Refer to EICM default input library
	Hydraulic conductivity	
Surface properties	Surface shortwave absorptivity	N/A

* Weather stations monitored by other research for ODOT could be added to the climatic database.

Traffic Input Default Library

ODOT has no specific studies for traffic that can be readily used as default traffic input. However, ODOT has 13 sections in the LTPP database. Those data are not easy to use and require significant efforts to organize it in such a way that they could be used in the MEPDG. It is recommended for ODOT to initiate a new study to characterize the Ohio traffic data that could be directly compatible with the MEPDG traffic data format.

REFERENCES

Abdulshafi, O., Mukhtar, H., and Kedzierski, B. *Reliability of AASHTO Design Equation for Predicting Performance of Flexible and Rigid Pavements in Ohio*. State Job No. 14447(0), Report No. FHWA/OH-95/006, Submitted to Ohio Department of Transportation, CTL Engineering Inc, 1994.

Abdulshafi, O., Kedzierski, B., and Fitch, M. G. *Determination of Recycled Asphalt Pavement (RAP) Content in Asphalt Mixes Based on Expected Mixture Durability*. State Job No. 14734(0), Report No. FHWA/OH-2002/037, Submitted to Ohio Department of Transportation, Ohio State University, December 2002.

Al-Yagout, M. A., Mahoney, J. P., Pierce, L. M., and Hallenbeck, M. E. 2005. *Improving Traffic Characterization to Enhance Pavement Design and Performance: Load Spectra Development*. Research Report No. WA-RD 600.1, Submitted to Washington Department of Transportation, University of Washington, 2005.

Azari, H., Al-Khateeb, G. G., Shenoy, A., and Gibson, N. H. "Comparison of Measured Simple Performance Test E* of ALF Mixtures with Predicted E* Using NCHRP 1-37A Model and Witczak's New Equations." *Transportation Research Record No. 1998*, Journal of the Transportation Research Board, Washington, D.C., 2007.

Bonaquist, R., and Christensen, D. W. "Practical Procedure for Developing Dynamic Modulus Master Curves for Pavement Structural Design." *Transportation Research Record No. 1929*, Journal of the Transportation Research Board, Washington, D.C., 2005.

Buchanan, M. S. *Traffic Load Spectra Development for the 2002 AASHTO Design Guide*. Report No. FHWA/MS-DOT-RD-04-165, Mississippi State University, 2004.

Carvalho, R. L., and Schwartz, C. W. "Comparisons of Flexible Pavement Designs: AASHTO Empirical Vs. NCHRP Project 1-37A Mechanistic–Empirical." *Transportation Research Record No. 1947*, Journal of the Transportation Research Board, Washington, D.C., 2006.

Ceylan, H., Gopalakrishnan, K., and Coree, B. "A Strategic Plan for Implementing the Mechanistic-Empirical Pavement Design Guide in Iowa." *Proceedings of the 85th Annual Meeting of the Transportation Research Board*, Washington, D.C., 2006.

Chehab, G. R., and Daniel, J. S. "Evaluating Recycled Asphalt Pavement Mixtures with Mechanistic–Empirical Pavement Design Guide Level 3 Analysis." *Transportation Research Record No. 1962*, Journal of the Transportation Research Board, Washington, D.C., 2006.

Chou, E., Fournier, L., Luo, Z., and Wielinski, J. *Structural Support of Lime or Cement Stabilized Subgrade Used with Flexible Pavements*. State Job No. 14746(0), Report No. FHWA/OH-2004/017, Submitted to Ohio Department of Transportation, University of Toledo, November 2004.

Chou, E.Y., Pulugurta, H., Datta, D. *Pavement Forecasting Models*, State Job No. 134148, Report No. FHWA/OH-2008/3, Submitted to Ohio Department of Transportation, University of Toledo, 2008.

Coree, B., Ceylan, H., and Harrington, D. *Implementing the Mechanistic-Empirical Pavement Design Guide: Technical Report.* IHRB Project TR-509, Center for Transportation Research and Education, Iowa State University, May 2005. Document: http://www.ctre.iastate.edu/reports/mepdg%5Ftechnical.pdf.

Cottrell, Jr., B.H., Schinkel, T.O., and Clark, T.M. A *Traffic Data Plan for Mechanistic-Empirical Pavement Designs (2002 Pavement Design Guide)*. Report no. VTRC 04-R3, Virginia Transportation Research Council, Virginia Department of Transportation, October, 2003. Document: http://www.virginiadot.org/VTRC/main/online_reports/pdf/04-r3.pdf

David Powers, Office of Materials Management, Ohio Department of Transportation, david.powers@dot.state.oh.us.

El-Basyouny, M. M., and Witczak, M. "Calibration of the Alligator Fatigue Cracking Model for the 2002 Design Guide." *Transportation Research Record No. 1919*, Journal of the Transportation Research Board, Washington, D.C., 2005.

FHWA's LTPP website, http://www.ltpp-products.com/index.asp

Figueroa, J. L. *Characterization of Ohio Subgrade Types*. State Job No. 14498 (0), Report No. FHWA/OH- 94/006, Submitted to Ohio Department of Transportation, Case Western Reserve University, August 1994.

Figueroa, J. L. *Extended Monitoring and Analysis of Moisture-Temperature Data*. May 2001." State Job No. 14694 (0), Report No. FHWA/OH-2001/10, Submitted to Ohio Department of Transportation, Case Western Reserve University, October 2001.

Figueroa J. L. *Long Term Monitoring of Seasonal Weather Stations and Analysis of Data from SHRP Pavement*. State Job. No. 14693(0), Report No. FHWA/OH-2004/044, Submitted to Ohio Department of Transportation, Case Western Reserve University, March 2004.

Flintsch, G. W., Loulizi, A., Diefenderfer, S. D., Galal, K. A., and Diefenderfer, B. K. *Asphalt Materials Characterization in Support of Implementation of the Proposed Mechanistic-Empirical Pavement Design Guide*. Report No. VTRC 07-CR10, Virginia Transportation Research Council, Virginia Department of Transportation, 2007. Document: http://www.virginiadot.org/vtrc/main/online_reports/pdf/07-cr10.pdf.

Gramajo, C. R., Flintsch, G. W., Loulizi, A., and Izeppi, E. D. de L. "Verification of Mechanistic-Empirical Pavement Deterioration Models Based on Field Evaluation of In-Service High-Priority Pavements." *Proceedings of the 86th Annual Meeting of the Transportation Research Board*, Washington, D.C., 2007.

Graves II, R. C., and Mahboub, K. C. "Sampling-Based Flexible Pavement Design Reliability Evaluation of NCHRP Mechanistic-Empirical Pavement Design Guide." Paper # 07-3155, *Proceedings of the 86th Annual Meeting of the Transportation Research Board*, Washington, D.C., 2007.

Graves, R. C., and Mahboub, K. C. "Pilot Study in Sampling-Based Sensitivity Analysis of NCHRP Design Guide for Flexible Pavements." *Transportation Research Record No. 1947*, Journal of the Transportation Research Board, Washington, D.C., 2006.

Haider, S. W., and Harichandran, R. S. “Characterizing Axle Load Spectra By Using Gross Vehicle Weights and Truck Traffic Volumes.” *Proceedings of the 86thAnnual Meeting of the Transportation Research Board*, Washington, D.C., 2007.

Hong, F., Pereira, M. F., and Prozzi A. J. “Comparison of Equivalent Single Axle Loads from Empirical and Mechanistic-Empirical Approaches.” *Proceedings of the 85th Annual Meeting of the Transportation Research Board*, Washington, D.C., 2006.

Hossain, M., Khanum, T., Tanesi, J., Schieber, G., and Montney, R.A. "Portland Cement Concrete Coefficient of Thermal Expansion Input for Mechanistic-Empirical Pavement Design Guide," *Proceedings of the 85th Annual Meeting of the Transportation Research Board*, Washington, D.C., 2006.

Kapoor, K., and Prozzi, A. J. *Methodology for Quantifying Pavement Damage Caused by Different Axle and Load Configurations*. Report No. SWUTC/05/167827-1, South West University Transportation Center, University of Texas at Austin, 2005.

Ker, H. W., Lee, Y. H., and Wu, P. H. “Development of Fatigue Cracking Performance Prediction Models for Flexible Pavements Using LTPP Database.” *Proceedings of the 86th Annual Meeting of the Transportation Research Board*, Washington, D.C., 2007.

Khanum, T., Hossain, M., Barezinsky, R. and Romanaschi, S.A. “Influence of Selected Design and Construction Features on JPCP Performance," *Proceedings of the 85th Annual Meeting of the Transportation Research Board*, Washington,D.C., 2006.

Khogali, W.E.I and E.H.H. Mohamed. “Mechanistic Classification of Unbound Materials,” *Transportation Research Record No. 2016*, Journal of the Transportation Research Board, Washington, D.C., 2007.

Kim, H. B. “Development of Reliability-Based Safety Factors for the Mechanistic-Empirical Flexible Pavement Design.” Proceedings of the *86th* Annual Meeting of the Transportation Research Board, Washington, D.C., 2007.

Kim, S., Ceylan, H., Gopalakrishnan, K., and Heitzman, M. “Sensitivity Study of Iowa Flexible Pavements Using the Mechanistic Empirical Pavement Design Guide.” Proceedings of the *85th* Annual Meeting of the Transportation Research Board, Washington, D.C., 2006.

Kvasnak, A., Robinette, C. J., and Williams, R C. “Statistical Development of a Flow Number Predictive Equation for the Mechanistic-Empirical Pavement Design Guide.” Paper #07-1000, Proceedings of the *86th* Annual Meeting of the Transportation Research Board, Washington, D.C., 2007.

Kwigizile, V., Mussa, R. N., and Selekwa, M. “Connectionist Approach to Improving Highway Vehicle Classification Schemes: The Florida Case.” Transportation Research Record No. 1917, Journal of the Transportation Research Board, Washington, D.C., 2005.

Lacroix, A., Khandan, A. A. M., and Kim, Y. R. “Predicting the Resilient Modulus of Asphalt Concrete from the Dynamic Modulus.” *Transportation Research Record No. 2001*, Journal of the Transportation Research Board, Washington, D.C., 2007.

Lee, S. J., Seo, Y., and Kim, Y. R. “Validation of Material Level Performance Models Using the Third-Scale Model Mobile Loading Simulator.” *Transportation Research*

Record No. 1949, Journal of the Transportation Research Board, Washington, D.C., 2006.

Li, S., Jiang, Y., Zhu, K. Q., and Nantung, T. E. "Truck Traffic Characteristics for Mechanistic-Empirical Flexible Pavement Design: Evidences, Sensitivities, and Implications." Paper #07--0101, *Proceedings of the 86th Annual Meeting of the Transportation Research Board*, Washington, D.C., 2007.

Li, S., Nantung, T., and Jiang, Y. "Assessing Issues, Technologies, and Data Needs to Meet Traffic Input Requirements by Mechanistic-Empirical Pavement Design Guide: Implementation Initiatives." *Transportation Research Record No. 1917*, Journal of the Transportation Research Board, Washington, D.C., 2005.

Liang, R. Y. *Mixture Design and Performance Prediction of Rubber-Modified Asphalt in Ohio*. State Job No. 14572(0), Report No. FHWA/OH-98/003, Submitted to Ohio Department of Transportation, University of Akron, August 1998.

Liang, R. Y. *Durability and Performance Characteristics of Hot Mix Asphalt Containing Polymer Additives - Phase I: Laboratory Study*. State Job No. 14701(0), Report No.FHWA/OH-2001/18, Submitted to Ohio Department of Transportation, University of Akron, December 2001.

Liang, R. Y., and Saleeb, A. F. *Mechanistic Evaluation of the Georgia Loaded Wheel Tester for Superpave Asphalt Mixtures*. State Job No. 14751(0), Report No. FHWA/OH-2004/008, Submitted to Ohio Department of Transportation, University of Akron, June 2004.

Liang, R. Y. *Evaluation of Drainable Bases under Asphalt Pavements*. State Job No. 148040(0), Report No. FHWA/OH-2007/10, Submitted to Ohio Department of Transportation, University of Akron, May 2007.

Loulizi, A., Flintsch, G. W., Al-Qadi, I. L., and Mokarem, D. (2006). "Comparing Resilient Modulus and Dynamic Modulus of Hot-Mix Asphalt as Material Properties for Flexible Pavement Design." *Transportation Research Record No. 1970*, Journal of the Transportation Research Board, Washington, D.C., 2006.

Loulizi, A., Flintsch, G. W., and McGhee, K. K. "Determination of In-Place Hot-Mix Asphalt Layer Modulus for Rehabilitation Projects Using a Mechanistic-Empirical Procedure." *Transportation Research Record No. 2037*, Journal of the Transportation Research Board, Washington, D.C., 2007.

Lu, Q., and Harvey, J. T. (2006). "Characterization of Truck Traffic in California for Mechanistic Empirical Design." *Transportation Research Record No. 1945*, Journal of the Transportation Research Board, Washington, D.C., 2006.

Lu, Q., Zhang, Y., and Harvey, J. T. "Analysis of Truck Traffic Growth for Mechanistic-Empirical Pavement Design Guide." Paper #07-1592, *Proceedings of the 86th Annual Meeting of the Transportation Research Board*, Washington, D.C., 2007.

Lundy, J. R., Sandoval-Gil, J., Brickman, A., and Patterson, B. *Asphalt Mix Characterization Using Dynamic Modulus and APA Testing*. Oregon State University, 2005. Document: http://www.oregon.gov/ODOT/TD/TP_RES/docs/Reports/DynamicModulus.pdf.

Masada, T., and Sargand, S. M. *Laboratory Characterization of Materials & Data Management for Ohio-SHRP Projects*. State Job No. 14695(0), Report No. FHWA/OH-2001/07, Submitted to Ohio Department of Transportation, Ohio University, January 2002.

Mohammad, L. N., Wu, Z., Obulareddy, S., Cooper, S., and Abadie, C. "Permanent Deformation Analysis of Hot-Mix Mixtures Using Simple Performance Tests and 2002 Mechanistic-Empirical Pavement Design Software." *Transportation Research Record No. 1970*, Journal of the Transportation Research Board, Washington, D.C., 2006.

Neidigh, J.D., and Crawford, J.A. "Expanding the Texas Weigh-In-Motion Program to Meet AASHTO 2002 Traffic Data Needs." *North American Travel Monitoring Exhibition and Conference*, San Diego, 2004.

Oh, J., Ryu, D., Fernando, E. G., and Lytton, R. L. "Estimation of Expected Moisture Contents for Pavements Using Environmental and Soil Characteristics." *Transportation Research Record No. 1967*, Journal of the Transportation Research Board, Washington, D.C., 2006.

Pavement Design and Rehabilitation Manual, Ohio Department of Transportation, 1999.

Perkins, S. W., Christopher, B. R., Cuelho, E. L., Eiksund, G. R., Hoff, I., Schwartz, C W., Svano, G., and Watn, A. *Development Of Design Methods For Geosynthetic Reinforced Flexible Pavements*. Western Transportation Institute, Montana State University, 2004. Document: http://www.coe.montana.edu/wti/wti/pdf/426202_Final_Report.pdf.

Priest, A. L., and Timm D. H. *Methodology and Calibration of Fatigue Transfer Functions for Mechanistic-Empirical Flexible Pavement Design*. National Center for Asphalt Technology Report No. 06-03, Auburn University, 2006.

Prozzi, J. A., and Hong, F. "Seasonal Time Series Models to Support Traffic Input Data for Mechanistic-Empirical Design Guide." *Transportation Research Record No. 1947*, Journal of the Transportation Research Board, Washington, D.C., 2006.

Puccinelli, J., and Jackson, N. "Development of Pavement Performance Models to Account for Frost Effects and their Application to M-E Design Guide Calibration." *Transportation Research Record No. 1990*, Journal of the Transportation Research Board, Washington, D.C., 2007.

Randolph, B. W., Heydinger, A. G., and Gupta, J. D. *Permeability and Stability of Base and Subbase Materials*. State Job No.14512(0), Report No. FHWA/OH 2000/017, Submitted to Ohio Department of Transportation, University of Toledo, August 2000.

Rao, C., Selezneva, O., and Darter, M. I. "Calibration of A Mechanistic Empirical Performance Model for CRCP Punchouts." *Transportation Research Record No. 1896*, Journal of the Transportation Research Board, Washington, D.C., 2004.

Richter, C. A. *Seasonal Variations in the Moduli of Unbound Pavement Layers*. Report No. FHWA-HRT-04-079. Federal Highway Administration, 2006.

Sadasivam, S., and Morian, D. "Effects of Ground Water Table Depths on Predicted Performance of Pavements." *Proceedings of the 85th Annual Meeting of the Transportation Research Board*, Washington, D.C., 2006.

Salama, H. K., Haider, S. W., and Chatti, K. "Evaluation of the New Mechanistic-Empirical Pavement Design Guide Rutting Models For Multiple Axle Loads." *Transportation Research Record No. 2005*, Journal of the Transportation Research Board, Washington, D.C., 2007.

Sargand, S. M., and Cinadr, E. *Field Instrumentation of Dowels*. State Job No. 14580(0), Report No. ST/SS/97-002, Submitted to Ohio Department of Transportation, Ohio University, May 1997.

Sargand, S. M., Edward, W., and Khoury, I. *Application of High Performance Concrete in the Pavement System: Structural Performance of High Performance Concrete Pavements.* State Job No. 14666(0) &14696(0), Report No. FHWA/OH-2001/15 & FHWA/OH-2001/16, Submitted to Ohio Department of Transportation, Ohio University, March 2002.

Sargand, S. M. *Performance of Dowel Bars and Rigid Pavement*. State Job No. 14667(0), Report No. FHWA/HWY-01/2001, Submitted to Ohio Department of Transportation, Ohio University, June 2001.

Sargand, S. M., and Edwards, W. F. *Determination of Pavement Layer Stiffness on Ohio-SHRP Test Road Using Backcalculation Techniques*. Submitted to Ohio Department of Transportation, Ohio University, 2002.

Sargand, S. M., Edwards, W. F., and Salimath, S. *Evaluation of Soil Stiffness Via Non-Destructive Testing*. Ohio Research Institute for Transportation and the Environment, Submitted to Ohio Department of Transportation, Ohio University, June 2001.

Sargand, S. M., and Masada, T. *Resilient Modulus of Highway Subgrade Soils*. Report to University of Cincinnati, Ohio University, June 1993.

Sargand, S. M., and Hazen, G. A. *Evaluation of Pavement Joint Performance*. State Job No. 14474(0), Report No. FHWA/OH-93/021, Submitted to Ohio Department of Transportation, Ohio University, August 1993.

Sargand, S. M., and Hazen, G. A. *Instrumentation of a Rigid Pavement System*. State Job No. 14515(0), Report No. FHWA/OH-97/001, Submitted to Ohio Department of Transportation, Ohio University, April 1997.

Sargand, S. M., and Hazen, G. A. *Evaluation of Resilient Modulus by Back-Calculation Technique*. State Job No. 14425(0), Report No. FHWA/OH-91/005, Submitted to Ohio Department of Transportation, Ohio University, Oct. 1991.

Sargand, S. M., Young, B. A., Khoury, I. S., Wasniak, D. L., and Goldsberry, B. M. *Final Report on Forensic Study for Section 390101 of Ohio SHRP Test Pavement*. Submitted to Ohio Department of Transportation, Ohio University, 1998.

Sargand, S. M., Wasniak, D. L., Masada, T., and Beegle, D. *Evaluation of Initial Subgrade Variability on the Ohio SHRP Test Road*, Submitted to Ohio Department of Transportation, Ohio University, 1999.

Sargand, S. M., and Edwards, W. F. *Effectiveness of Base Type on the Performance of PCC Pavement on ERI/LOR 2*. State Job No. 14652(0), Report No. FHWA/OH-2000/005, Submitted to Ohio Department of Transportation, Ohio University, April 2000.

Sargand, S. M., and Edwards, W. F. *Accelerated Testing of Ohio SHRP Section 390101, 390102, 390105, and 390107*. State Job. No. 14744 (0), Report No. FHWA/OH-2004/012, Submitted to Ohio Department of Transportation, Ohio University, 2004.

Sargand, S. M., and Morrison, J. *Truck/Pavement/Economic Modeling and In-Situ Field Test Data Analysis Applications – Volume 4: Effects of Slab Shape and Load Transfer Mechanisms on Portland Cement Concrete Pavement*." State Job No. 147700-SPR2 (203), Report No. FHWA/OH-2006/3D, Submitted to Ohio Department of Transportation, Ohio University, Sep.2007.

Sargand, S. M and ORITE Staff. *Evaluation of Pavement Performance of DEL-23* State Job. No. 14768(0), Report No. FHWA/OH-2007/05, University of Ohio, March, 2007.

Sargand, S. M., Khoury, I. S., Harrigal, A. G., Sargand, L. M., and Kim, Huntae. *Evaluation of Pavement Performance on DEL-23 – Interim Report – Forensic Study for Sections 390103, 390108, 390109, and 390110 of Ohio SHRP U.S. Rt. 23 Test Pavement*. State Job. No. 14768(0), Submitted to Ohio Department of Transportation, Ohio University, Jan. 2006.

Sargand, S. M., Hazen, G. A., Wilson, B. E., and Russ, A. C. *Evaluation of Resilient Modulus by Back-Calculation Technique*. State Job No. 14425(0), Report No. FHWA/OH-91/005, Submitted to Ohio Department of Transportation, Ohio University, Oct. 1991.

Sargand, S. M., and Kim, S. *Pavement Performance Testing*. State Job No. 14702(0), Report No. FHWA/OH-2001/14, Ohio University, Dec. 2001.

Scharm, S., and Abdelrahman, M. "Improving Prediction Accuracy in the Mechanistic-Empirical Pavement Design Guide." *Transportation Research Record No. 1947*, Journal of the Transportation Research Board, Washington, D.C., 2006.

Schwartz, C.W. "Implications of Uncertainty in Distress Measurement for Calibration of Mechanistic–Empirical Performance Models," *Transportation Research Record No. 2037*, Journal of the Transportation Research Board, Washington, D.C., 2007.

Sehn, A. L. *Evaluation of Portland Cement Concretes Containing Ground Granulated Blast Furnace Slag*. State Job No. 14559(0), Report No. FHWA/OH-2002/022, Submitted to Ohio Department of Transportation, University of Akron, May 2002.

Sharp, S. R., Galal, K. A., and Elfino, M. K. *Using Fiber-Optic Sensor Technology to Measure Strains Under the Asphalt Layer of a Flexible Pavement Structure*. Virginia Transportation Research Council, Virginia Department of Transportation, 2006. Document: http://www.virginiadot.org/vtrc/main/online_reports/pdf/07-r5.pdf.

Tam, W. O., and Von Quintus, H. "Use of Long Term Pavement Performance Data to Develop Traffic Defaults in Support of Mechanistic-Empirical Pavement Design Procedures." *Transportation Research Record No. 1855*, Journal of the Transportation Research Board, Washington, D.C., 2003.

Tanesi, J., Kutay, M.E., Abbas, A. R. and Meininger, R.C. "Effect of Coefficient of Thermal Expansion Test Variability on Concrete Pavement Performance as Predicted by Mechanistic-Empirical Pavement Design Guide," Transportation Research Record No.2020, Journal of the Transportation Research Board, 2007.

Timm, D. H., Bower, J. M., and Turochy, R. E. "Effect of Load Spectra on Mechanistic-Empirical Flexible Pavement Design." *Transportation Research Record No. 1947*, Journal of the Transportation Research Board, Washington, D.C., 2006.

Timm, D. H., Newcomb, D. E., and Galambos, T. V. "Incorporation of Reliability into Mechanistic-Empirical Pavement Design." *Transportation Research Record No. 1730*, Journal of the Transportation Research Board, Washington, D.C., 2000.

Tran, N. H., and Hall, K. D. "Evaluation of Testing Protocols for Dynamic Modulus of Hot-Mix Asphalt." *Transportation Research Record No. 1960*, Journal of the Transportation Research Board, Washington, D.C., 2006.

Tran, N. H., and Hall, K. D. "Evaluation of Weigh-In-Motion Data for Developing Axle Load Distribution Factors for Mechanistic Empirical Pavement Design Guide." *Proceedings of the 86th Annual Meeting of the Transportation Research Board*, Washington, D.C., 2007.

Tran, N. H., and Hall, K. D. "Development and Influence of Statewide Axle Load Spectra on Flexible Pavement Performance." *Transportation Research Record No. 2037*, Journal of the Transportation Research Board, Washington, D.C., 2007.

Tran, N. H., and Hall, K. D. "Development and Significance of Statewide Volume Adjustment Factors in Mechanistic-Empirical Pavement Design Guide." *Transportation Research Record No. 2037*, Journal of the Transportation Research Board, Washington, D.C., 2007.

Von Quintus, H.L., Darter, M.I., and Mallela, J. Manual of Practice – Interim Mechanistic Empirical Pavement Design Guide, NCHRP Project 1-40B, Transportation Research Board, Washington, D.C., 2007.

Wang, J., Birgisson, B., and Roque, R. "Windows-Based Top-Down Cracking Design Tool for Florida: Using the Energy Ratio Concept." *Transportation Research Record No. 2037*, Journal of the Transportation Research Board, Washington, D.C., 2007.

Wolfe, R. K., Heath, G. L., and Colony, D. C. *University of Toledo Time-Temperature Model Laboratory and Field Validation*. Report # FHWA/OH-80/006, Submitted to Ohio Department of Transportation, University of Toledo, April 1980.

Wolfe, W., and Butalia, T. *Continued Monitoring of SHRP Pavement Instrumentation Including Soil Suction and Relationship with Resilient Modulus*. State Job No. 14691(0), Report No. FHWA/OH-2004/007, Submitted to Ohio Department of Transportation, Ohio State University, June 2004.

Wu, Z., Zhang, Z., King, B., and Mohammad, L. N. "Evaluating Structural Performance of Base and Subbase Materials at Louisiana Accelerated Pavement Research Facility." Paper # 07-0773, *Proceedings of the 86th Annual Meeting of the Transportation Research Board*, Washington, D.C., 2007.

Zaghloul, S., Abd El Halim, A., Ayed, A., Vitillo, N., and Sauber, R. W. "Sensitivity Analysis of Input Traffic Levels on MEPDG Predictions." *Proceedings of the 85th Annual Meeting of the Transportation Research Board*, Washington, D.C., 2006.

Zaghloul, S., Ayed, A., Abd El Halim, A., Vitillo, N. P., and Sauber, R. W. "Investigations of Environmental and Traffic Impacts on Mechanistic-Empirical Pavement Design Guide Predictions". *Transportation Research Record No. 1967*, Journal of the Transportation Research Board, Washington, D.C., 2006. Order Document: http://www.trb.org/news/blurb_detail.asp?id=7506.