Verification and Local Calibration/Validation of the MEPDG Performance Models for Use in Georgia

Literature Search and Synthesis / Task 1—Interim Report

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July 2013
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VERIFICATION AND LOCAL CALIBRATION/VALIDATION OF THE MEPDG PERFORMANCE MODELS FOR USE IN GEORGIA

Literature Search and Synthesis

Task 1 – Interim Report

Submitted to

Georgia Department of Transportation
Office of Research and Office of Materials Testing
15 Kennedy Drive
Forest Park, Georgia 30297

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July 16, 2013
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The Georgia Department of Transportation (GDOT) is transitioning from empirical design procedures to the MEPDG procedure for designing new and rehabilitated highway pavements. GDOT currently uses the 1972 AASHTO Interim Guide for Design of Pavement Structures as the standard pavement design procedure. As a part of the implementation process, GDOT has undertaken a project to verify the MEPDG global distress models and locally calibrate these models for local field conditions of Georgia, if determined to be necessary by the verification process, using the Long-Term Pavement Performance (LTPP) and non-LTPP sections in Georgia.

A comprehensive literature search was conducted at the beginning of the project to collect, review, and summarize available information on both completed and ongoing MEPDG implementation activities by various federal and state agencies. The literature search covered a full spectrum of the implementation process from material characterization research to local calibration and full-scale deployment.

This report synthesizes the current and completed implementation activities by the State DOTs and identifies outcomes from those implementation studies that will be of benefit to GDOT in their implementation activities. More importantly, the interim report summarizes the lessons learned from various calibration studies that are directly applicable to Georgia for use in determining appropriate design inputs, setting up a sampling matrix to verify, locally calibrate, and validate the MEPDG transfer functions, and selecting design reliability and performance criteria.
## SI* (MODERN METRIC) CONVERSION FACTORS

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**NOTE:** Volumes greater than 1000 L shall be shown in m³.

| **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") |
| °F | Fahrenheit | 5 (F-32)/9 | Celsius | °C |

| **ILLUMINATION** | | | | |
| fc | foot-candles | 10.76 | lux | lx |
| fl | foot-Lamberts | 3.426 | candela/m² | cd/m² |

| **FORCE and PRESSURE or STRESS** | | | | |
| lbf | poundforce | 4.45 | newtons | N |
| lbf/in² | poundforce per square inch | 6.89 | kilopascals | kPa |

### APPROXIMATE CONVERSIONS FROM SI UNITS

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| **ILLUMINATION** | | | | |
| lx | lux | 0.0829 | foot-candles | fc |
| cd/m² | candela/m² | 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/in² |

*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)
EXECUTIVE SUMMARY

This synthesis report captures the status of current and completed implementation activities by various State agencies. The information compiled in this report will serve as a reference document to GDOT to see what other States are doing with regards to implementation and help prevent avoidable problems experienced by other agencies during their implementation effort. The other intent of the synthesis report is to provide a summary of the results from other agencies calibration efforts in planning the sampling matrix and experimental factorial, if a local calibration for Georgia is determined to be required (the results from Task 2 or Work Order #1).

Most State DOT studies have focused on building design input data libraries for key material types and traffic loadings, and evaluate the ability of lower hierarchical input levels to produce reasonable predictions for the agency-specific material types. Numerous studies have focused on HMA mixtures; the evaluation of Level 3 Witzak dynamic modulus model indicates that the model predictions appeared to be acceptable for mixtures with conventional asphalt, significant deviations were observed for binders with higher PG grades. Use of measured binder test data (i.e. input level 2) in Witzak model has greatly improved the accuracy of dynamic modulus predictions. Studies on PCC mixtures have particularly emphasized on the CTE measurement and its significance in rigid pavement performance. Although the results are diverse, most studies have concluded the Witzak dynamic modulus regression equation is reasonable.

Most surrounding states, including the Florida, Louisiana, Mississippi, and Virginia, have conducted studies on resilient modulus of unbound materials and soils. These studies when combined with GDOT studies on resilient modulus can be used to provide recommended values for a range of soils and their in place condition for establishing inputs to the ME Design software. The results from all of these studies, as well as from FHWA sponsored studies on resilient modulus, can help provide local default values for GDOT to use in calibration, as well as in design.

The findings of national level studies, including NCHRP 1-40B, NCHRP 9-30A and NCHRP 1-47, are directly applicable to Georgia. More importantly, the lessons learned from various calibration studies are directly applicable to Georgia for use in determining appropriate design inputs, setting up a sampling matrix to verify, locally calibrate, and validate the MEPDG transfer functions, and selecting design reliability and performance criteria. The following lists some of the more important findings from the literature and projects reviewed under this Task.

1. The key findings of the sensitivity analyses conducted under NCHRP 1-47 will be used in the GDOT MEPDG implementation study to select sites for the local calibration and in evaluating the residual error of the predicted distress values.

2. Selection of design reliability and design performance criteria requires sensitivity analyses to show how the resulting design depends on these critical inputs. Selection of too high of reliability and performance criteria will result in unreasonable and costly designs. Selection of reliability and performance criteria should be done together and not independently.
3. The local calibration factors determined from State calibration studies for PCC pavements are reasonably consistent with the global coefficients. However, several important advantages were obtained through State local calibration for PCC pavements.
   a. More accurate design inputs were established through the local calibration process. For example, the estimate of the number of months of full friction between the slab and base was improved using local data.
   b. The use of the correct CTE input for the PCC (as measured by AASHTO T336) was found to verify the national coefficients determined under NCHRP 20-07 in 2010 for several states. This makes it possible for the State to test the CTE of PCC and then use directly the value in design.
   c. Modifications to some the calibration coefficients were found to be valuable in reducing the standard error of prediction which is used directly in reliability design.

4. The following are some consistent findings from flexible pavement calibration studies:
   a. MEPDG over predicts rutting in the HMA and unbound layers based on using laboratory equivalent resilient modulus values.
   b. Dynamic modulus does not explain the difference in rutting between HMA and PMA mixtures.

5. The following local calibration coefficients were found to be significantly different between many of the studies reviewed for flexible pavements:
   a. $B_{11}$ for the fatigue cracking transfer function.
   b. $B_{13}$ (exponent to the number of load cycles term) and $B_{11}$ (the intercept term) for the HMA rut depth transfer function.
   c. $C1$ or coefficient of the rutting term in the IRI regression equation.

6. The procedures outlined in the NCHRP 1-40B and NCHRP 9-30A can be used to develop field adjustment factors for fatigue cracking and rutting models that will remove the over and under prediction and reduce the standard error of prediction.
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1. **INTRODUCTION**

The Mechanistic-Empirical Pavement Design Guide (MEPDG) was developed under the National Cooperative Highway Research Program (NCHRP) project 1-37A with an objective to provide the highway community with a state-of-the-practice tool for the design for new and rehabilitated pavement structures. This effort produced a mechanistic-empirical (M-E) pavement design guide that includes (a) a Guide for M-E design and analysis, (2) companion research-grade software with a user manual, and (3) training materials.

Following the development phase is the implementation of MEPDG by the State Departments of Transportation (DOT). To aid the State DOTs and other agencies in MEPDG implementation, the MEPDG Manual of Practice and Local Calibration Guidance were prepared under NCHRP Project 1-40B, which became the American Association of State Highway and Transportation Officials (AASHTO)' balloted and approved design procedure for the MEPDG. The Manual of Practice presents information to guide pavement design engineers in making decisions and using the MEPDG for new pavement and rehabilitation design. The local calibration manual provides guidance in making a decision to recalibrate the MEPDG to local conditions, policies, and materials, and in conducting the local calibration process. The AASHTOWare® Pavement ME Design™ software, shortly referred as the M-E Design, was released in April 2011 for production-level pavement designs. A software user-manual was released to provide users with in-depth guidance on the operation and application of the M-E Design software.

Georgia Department of Transportation (GDOT) is transitioning from empirical design procedures to the MEPDG procedure for designing new and rehabilitated highway pavements. GDOT currently uses the 1972 AASHTO Interim Guide for Design of Pavement Structures as the standard pavement design procedure. As a part of the implementation process, GDOT has undertaken a project to verify the MEPDG global distress models and locally calibrate these models for local field conditions of Georgia, if determined to be necessary by the verification process, using the Long-Term Pavement Performance (LTPP) and non-LTPP sections in Georgia. Under this project, a comprehensive literature search was conducted to collect, review, and summarize available information on both completed and ongoing MEPDG implementation activities by various federal and state agencies. The literature search covered a full spectrum of the implementation process from material characterization research to local calibration and full-scale deployment. This report synthesizes the current and completed implementation activities by the State DOTs.

This synthesis will serve as a reference document to GDOT to see what other States are doing with regards to implementation and use and help preventing avoidable problems experienced by other agencies during the implementation effort.

2. **OVERVIEW OF THE LITERATURE REVIEW**

The project team conducted a literature survey using bibliographic databases, such as Transportation Research Information Services (TRIS), Research in Progress (RiP), State DOTs Planning and Research websites, and the ASCE research library. The literature search identified documents/projects that are directly or indirectly related to the MEDPG implementation effort such as the laboratory testing and input
databases for key materials, studies related to traffic and environment, validation and local calibration of performance models, and deployment. The documents and projects reviewed included both on-going and completed research studies.

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 implementation activities were documented in one of the following topics:

- Pavement material characterization
  - Asphalt materials
  - Portland Cement Concrete (PCC) materials
  - Chemically stabilized materials
  - Unbound materials (includes environmental effects)
  - Material characterization for rehabilitation
- Climate and environmental effects
  - Climate data
  - Environmental effects on unbound materials
- Traffic inputs
- Instrumentation and sensitivity analysis
  - Instrumentation & Accelerated Pavement Testing (APT) studies
  - Sensitivity analysis
- Local calibration and deployment of the MEPDG
  - Local calibration
  - Deployment

3. **Pavement Materials Characterization**

The implementation activities in the pavement materials characterization have focused on identifying data needs, as well as, gathering the necessary material properties for use in the MEPDG.

3.1 **Asphalt Materials**

Several State Departments of Transportation (DOT) including Arizona, Idaho, Colorado, Florida, Idaho Kansas, Minnesota, Missouri, North Carolina, Ohio, Oklahoma, Virginia, and Wisconsin have completed a significant portion of the implementation effort for asphalt materials through research contracts or in-house studies. These activities generally focus on one or more of the following objectives:

- Developing an input data library for asphalt materials that represents the typical materials widely used for both new/reconstruction and rehabilitation designs.
- Evaluating the sensitivity of inputs at different hierarchy for reliability assessment and understand their relationships to field performance.
- Developing site-specific inputs for validation and calibration of performance prediction models used in the MEPDG.
• Including specialty mixtures, such as stone-matrix asphalt, cold-recycled and mixtures with high reclaimed or recycled asphalt pavement (RAP) material content, that were not included in the original material database used in developing Level 3 models and defaults.
• Developing policy guidance on determining the level of effort required for projects of varying size, cost, and overall importance.

Several studies have focused on laboratory testing to measure fundamental properties of typical mixtures used in their states. Laboratory testing typically included dynamic modulus measurements, but creep compliance, indirect tensile strength, Poisson’s ratio, plastic deformation properties, and resilient modulus testing were also included.

The completed activities in the area of asphalt materials characterization include:

• FHWA has developed a program, Artificial Neural Networks for Asphalt Concrete Dynamic Modulus Prediction (ANNACAP), to aid in populating the LTPP database with dynamic modulus data (Kim et al, 2011). This computed parameter database has been found to be very useful in estimating and demonstrating the in place damage of HMA layers through the deflection basins in accordance with the MEPDG Manual of Practice.

• Arizona DOT conducted a comprehensive study of HMA material characterization through a series of projects with Arizona State University (Witczak, 2011). Eleven typical ADOT conventional lab blended hot mix asphalt (HMA) mixtures using five different aggregates were used in this study. ADOT has developed an AC (Asphalt Concrete) Binder Characterization Database that contains properties of six typical AC binders commonly used in Arizona. The agency also developed separate comprehensive databases for dynamic modulus properties, thermal fracture properties, mixture fracture (fatigue) properties and permanent-strains collected from repeated load dynamic tests.

• Colorado DOT conducted a similar study of HMA materials characterization to support the MEPDG implementation efforts. This study included nine HMA mixtures typically used in Colorado. HMA characterization tests include dynamic modulus, repeated shear tests at constant height, creep compliance, indirect tensile strength and mix volumetrics. The repeated shear test results were then utilized to calibrate the HMA rutting model, while other properties were used in developing materials input libraries.

• Florida DOT has developed a database for referencing available resilient modulus and dynamic modulus values by funding a laboratory testing program for select agency-specific mixtures (Ping and Xiao, 2007). This study found that the dynamic modulus values measured at a loading frequency of 4 Hz was comparable with the resilient modulus obtained from the indirect diametrical test at the same temperature level. By comparing the lab-measured modulus with the predicted values, this study found Witczak’s dynamic modulus prediction model comparable for agency-specific mixtures used in this study.

• Florida DOT sponsored a similar study to develop dynamic modulus capabilities for HMA mixtures in compression, torsion, and tension (Birgisson et al, 2004). This study proposed a new approach to determine creep compliance parameters from a combination of complex modulus and static creep tests. In addition, this study also evaluated the relationships between the dynamic modulus measurements and the performance of mixtures in rutting and fracture.
Idaho Transportation Department (ITD), through its research contract with University of Idaho, has developed material inputs library for HMA (Bayomy et al, 2012). The input library was developed based on the laboratory test results of 27 Idaho HMA mixtures that covered 6 different binder types (PG [Performance Grade] 58-28, PG58-34, PG64-28, PG64-34, PG70-28, and PG76-28), varied mix aggregate gradation, and mix volumetric properties. Laboratory testing included dynamic modulus and volumetric properties of HMA mixtures, and Brookfield rotational viscosity and dynamic shear rheometer tests for asphalt binders. The HMA inputs library contains inputs at all MEPDG hierarchical input levels for HMA mixtures and binders typically used in Idaho.

Illinois DOT conducted dynamic modulus testing for twenty IDOT mixtures at 7 and 4 percent air voids (Carpenter, 2007). The test results appeared satisfactory from a structural design standpoint, with measured dynamic modulus values at 20 °C ranging from 1,000,000 psi to 2,000,000 psi.

Iowa Highway Research Board (IHRB) funded a study that undertook an experimental plan for characterizing the cold recycled mixtures with foamed asphalt and emulsions. The experimental plan included dynamic modulus test, dynamic creep test, flow number and raveling test (Lee and Kim, 2007; Lee et al, 2009). Iowa DOT is currently funding a research project to develop the asphalt dynamic modulus master curve directly from falling weight deflectometer (FWD) testing for use in MEPDG flexible pavement analysis and rehabilitation design (Contract Number IHRB-12-06).

Kansas DOT conducted a study to evaluate if the HMA dynamic modulus could be achieved in the field construction (Gedafa et al, 2009). This study performed a statistical comparison of dynamic modulus at different temperatures (40, 70 and 95°F) determined from laboratory testing, backcalculation of FWD measurements and predictive models of Witczak and Hirsh. This study concluded that no two approaches provided statistically consistent results. i.e. some approaches tended to give similar moduli for a certain site but not for all sites. When compared with the lab measurements, this study observed, the Witczak model underestimated the dynamic modulus at low temperatures and overestimated them at high temperatures.

Kansas DOT sponsored another study to develop a database of material inputs required by the ME Design software for HMA mixes (Romanoschi et al, 2009). This study conducted laboratory testing to measure dynamic modulus at 7.0% and 4.0% air voids, creep compliance and indirect tensile strength values. This study observed both the Witczak and Hirsch models underestimated dynamic modulus in comparison to the measured values.

Another study by Romanoschi et al (2006) evaluated dynamic modulus, bending stiffness and fatigue properties of four representative Superpave mixtures used in the construction of base layers of Kansas flexible pavements. When compared with the lab-based fatigue measurements, the MEPDG fatigue model was found to over-predict the fatigue lives for the mixtures with virgin binder and severely under-predict for those with SBS polymer modified asphalt (PMA).

Maryland State Highway Agency (MDSHA) has assembled a database of material properties primarily involving asphalt binder properties and HMA dynamic modulus. (Schwartz, C.W. and R. Li, 2011).

Clyne and Marasteanu (2004) developed an inventory of the rheological properties of certified asphalt binders used in Minnesota. This study conducted a suite of lab tests to evaluate the rheological properties of nine different asphalt binders. Marasteanu et al (2003) also conducted laboratory testing on four different HMA mixtures obtained from the Mn/ROAD site to determine
Level 1 properties. This study observed that the Witczak model provided higher estimates of dynamic modulus at high temperatures than the measured values.

- Mississippi DOT has developed a library of dynamic modulus inputs for selected HMA mixtures. The research study by White et al (2007) conducted dynamic modulus characterization of twenty-five mixtures with different combinations of aggregate type, maximum nominal size of aggregates, binder grades and compaction levels. In addition, this study also conducted Asphalt Pavement Analyzer (APA) tests to provide MDOT a relative comparison of the mixture’s potential in-service performance.

- Missouri DOT has developed a library of creep compliance and indirect tensile (IDT) strength inputs for select plant-produced surface course mixtures at different air void levels (Richardson and Lushar, 2008).

- Nebraska Department of Roads (DOR) recently developed a database of layer moduli (dynamic modulus, creep compliance, and resilient modulus) of various agency-specific pavement materials through a research contract (Im et al, 2010). This study identified some discrepancies between measured and predicted dynamic modulus. At lower and/or higher loading frequencies, the discrepancies or differences are mix-dependent.

- New England Transportation Consortium funded a research project to establish default dynamic modulus for New England States. This study investigated if there is a significant difference between dynamic modulus values for materials from throughout the region. This study compared the dynamic modulus of laboratory and plant produced mixes with MEPDG Level 3 models (Jackson et al, 2011).

- New Jersey DOT has developed a catalog of dynamic modulus inputs for plant-produced and laboratory-compacted samples of various HMA mixtures (Bennert, 2009). Evaluating the precision of the predictive models, this study reported dynamic modulus derived from the Witczak model was found to compare better with the measured dynamic modulus values than those from the Hirsch model. The precision of the predictions were found to be better for the PG64-22 asphalt binders than for the polymer modified PG76-22 asphalt binders; however, the precision improved when the actual test data of rolling thin film oven (RTFO) aged asphalt binders were used in lieu of Level 3 binder properties.

- For New Jersey DOT, Maher and Bennert (2008) compared the lab-measured values of Poisson’s Ratio with those estimated with the prediction equation provided in the MEPDG. The authors found some discrepancies between the measured and predicted values, especially when higher PG asphalt binder grades were used.

- North Carolina DOT has developed a library of dynamic modulus inputs for commonly used HMA mixtures (Kim et al, 2005). This study included forty-two mixtures with varying aggregate sources, aggregate gradations, asphalt sources, asphalt grades, and asphalt contents. Evaluating the precision of Witczak model, Kim et al observed that the predictions compared better at lower temperatures than higher temperatures. A parametric study was conducted to study the effects of mixture variables on dynamic modulus. This study developed an analytical solution in accordance with the theory of linear viscoelasticity to determine the dynamic modulus, phase angle, and Poisson’s ratio using the indirect tensile (IDT) test.

- North Carolina has currently undertaken a research project that focuses on MEPDG inputs for warm mix asphalt (WMA). This study intends to perform dynamic modulus tests for stiffness characterization, direct tension cyclic tests for fatigue performance characterization, and triaxial
repeated load plastic deformation (TRLPD) tests for rutting characterization. This study also plans to develop recommendations for MEPDG input parameters for various WMA mixtures.

- Ohio DOT has developed a database containing mechanical properties of a wide variety of pavement materials utilized in each of the 28 pavement-related research projects conducted within the last two decades (Masada et al, 2004).

- Oklahoma DOT sponsored a research study to develop an improved procedure of predicting dynamic modulus for agency’s use to minimize the need for performing detailed lab testing for each mix in a pavement system (Cross et al, 2007). This study conducted dynamic modulus testing for twenty-one mixes representing different mix types, region placed aggregates, binder grades and mixtures with and without recycled materials. Another study conducted rheological tests on commonly used asphalt binders in Oklahoma and to determine their shear modulus ($G^*$) and phase angle values at a given range of temperatures (Hossain et al, 2011). Phase 2 of this study evaluated the dynamic modulus of SMA mixtures and compared the measured values with MEPDG predicted values (Cross et al, 2011).

- Oklahoma DOT is currently developing lab and field data vital for HMA mixtures with high RAP content. The testing plan involves performance testing of high RAP mixtures and rheological properties of the blended (virgin and recovered) binders (Oklahoma DOT contract number: DTRT06-G-0016).

- Lundy et al (2005) conducted dynamic modulus tests of typical asphalt mixes for Oregon DOT. Evaluating the precision of the Witczak model, the study concluded the predicted values did not agree well with the measured ones.

- South Dakota DOT has funded a study to obtain HMA dynamic modulus and subgrade resilient modulus (Contract Number: SD2008-10).

- Texas DOT has developed a HMA test database for use with the MEPDG through a suite of performance tests (Bhasin et al, 2005). This study included several plant and laboratory-produced mixtures. Tests included in the experimental plan were APA, Hamburg, Dynamic Modulus, Flow Time, Flow Number, and Simple Shear at Constant Height for a comparison of test results and an evaluation of rutting.

- Virginia Tech Transportation Institute (VTRC) conducted dynamic modulus, creep compliance and tensile strength tests of eleven mixtures (4 base, 4 intermediate, and 3 surface mixes) collected from different plants across the state. In addition, resilient modulus tests were also performed to find possible correlations with the dynamic modulus. Witczak model predictions were found reasonable with lab measured binder properties; however, it did not account for some of the differences between the mixes captured by the lab measurements. Virginia DOT has also developed a catalog of binder properties. This study investigates the effect of changes in asphalt-binder properties on the predicted distress levels for trial pavement designs evaluated using the MEPDG (Diefenderfer, S.D., 2011). Addition testing was conducted for twelve mixtures identified among the most used contract items of VDOT’s Maintenance Division (Apeagyei A.K. and S.D. Diefenderfer, 2011).

- Washington DOT has developed a database of dynamic modulus values for typical Superpave mixes widely used in the state. Seven job mix formula (JMF) mixes were selected for the study (Tashman, L and M.A. Elangovan, 2007).

- Wisconsin DOT (WisDOT) has completed three research projects on HMA material characterization. The first project evaluated the stiffness and permanent deformation properties of
twelve WisDOT mixtures using the Asphalt Mixture Performance Tester (AMPT). A database containing dynamic modulus master curve and flow numbers was assembled to support the MEPDG implementation efforts (Bonaquist, 2010). The second project focused on establishing a range of tensile strength and creep compliance properties (Bonaquist, 2011), and the third project focused on creating a database of flow numbers for representative mixtures (Bonaquist, 2012).

Most of the studies focused on building a data library of Level 1 asphalt material inputs (primarily dynamic modulus) with agency-specific mixtures. These studies have invariably evaluated the Witczak model for its ability to reasonably predict dynamic modulus values of the agency-specific mixtures. While the model predictions appeared to be acceptable for mixtures with conventional binders, significant deviations were observed for binders with higher PG grades. Use of measured binder test data (i.e. input level 2) in the Witczak model has greatly improved the accuracy of dynamic modulus predictions.

Fewer studies have focused on material characterization (dynamic modulus) of non-conventional mixtures, such as mixtures with high RAP content, stone-matrix asphalt, cold-recycled mixtures; and measuring the plastic strain and fatigue or fracture properties of HMA mixtures. Similarly, fewer studies have focused on the characterization of asphalt materials for rehabilitation designs, but there are a few exceptions.

The Asphalt Institute sponsored a study to compare the predicted and observed performance between HMA and PMA mixtures. The Asphalt Institute found a significant difference in performance (rutting and fatigue cracking), which was not explained by the MEPDG procedure when using input level 3. Figures 1 and 2 provide a comparison of the predicted and measured distresses for rutting and fatigue cracking, respectively. As shown, the MEPDG over predicted the distresses rutting and fatigue cracking for PMA mixtures. This finding is similar to the finding from the Romanoschi, et al., study.

NCHRP Project 9-30A (Von Quintus, et al., 2012) focused on evaluating multiple rut depth transfer functions in comparison to the Kaloush-Witczak transfer function included in the ME Design software. Repeated load constant height shear and triaxial tests, as well as dynamic modulus tests were performed on a range of dense-graded neat and polymer modified mixtures with the purpose of relating the laboratory-derived or measured plastic strain constants of the transfer function to field-derived constants. Figures 3 and 4 show the resulting relationships between laboratory and field-derived values that can be used with the Kaloush-Witczak rut depth transfer function in the ME Design software.
Figure 1  Comparison of the predicted and measured rut depths using the locally calibrated equation for the companion sites and those with PMA mixtures.

Figure 2  Comparison of the predicted and measured fatigue cracking for the companion sites and those sections with PMA mixtures.
Figures 3 and 4 permit an agency to adjust the laboratory-derived repeated load plastic strain constants of different mixtures for use in the MEPDG rut depth prediction methodology – input level 1. The other important finding from NCHRP project 9-30A was the use of repeated load plastic strain tests was significantly more important than dynamic modulus tests. In other words, the plastic strain constants from repeated load triaxial tests explained the differences in rutting of HMA mixtures more
accurately, than the use of dynamic modulus; similar to the finding from the Asphalt Institute study and comparison of HMA and PMA mixtures. This finding and the results are directly applicable to Georgia.

Similar studies for fatigue and low temperature cracking have yet to be sponsored on a national level. Results from studies sponsored by the Asphalt Institute, FHWA, and NCHRP will be used to develop similar field adjustment factors or methods for using laboratory generated fracture values to predict the cracking performance of HMA mixtures in Georgia.

3.2 PCC Materials

Implementation activities pertinent to the characterization of PCC materials have primarily focused on the following objectives:

- Building a data library of material properties that include both strength and fresh concrete properties.
- Determining thermal properties of PCC with a special emphasis on the coefficient of thermal expansion (CTE).

The key activities of State agencies are summarized as follows:

- Arkansas State Highway and Transportation Department (AHTD) has developed a catalog of PCC material properties for the MEPDG inputs such as the CTE, Poisson’s ratio, and elastic modulus (Hall and James, 2009). This effort included testing of 24 concrete/cement paste mixtures at ages ranging from 7 to 90 days with various local aggregate types. This study also updated the regression equations of the MEPDG strength gain curve for local conditions.
- Colorado DOT developed a catalog of material properties for four PCC mixtures typically used in Colorado. The testing plan included compressive strength, flexural strength, splitting tensile strength, Young’s modulus, Poisson’s ratio, CTE and shrinkage tests.
- Florida DOT has developed a similar catalog with test results of three agency-specific concrete mixtures (Ping and Kampmann, 2008). The testing plan included compressive strength, flexural strength, splitting tensile strength, Young’s modulus, Poisson’s ratio, and CTE. This study also conducted a sensitivity analysis to understand the effects of CTE on PCC behavior for magnitudes expected in Florida. The sensitivity matrix indicated the MEPDG performance predictions are not CTE sensitive to load transfer efficiency, minimally CTE sensitive to faulting, CTE sensitive to bottom-up damage (for thin PCC layers), and extremely CTE sensitive to top-down damage, cracking, and smoothness. This study concluded two out of three pavement performance criteria appeared to be highly susceptible to CTE in Florida JPCP structures.
- Under MEPDG Work Plan Task 4, Iowa DOT synthesized and analyzed over 20,000 data sets obtained from various sources to identify typical Iowa-specific inputs for the MEPDG Level 3 concrete pavement design. Most of these data includes typical fresh concrete and strength properties. Their statistical parameters were compared with their corresponding MEPDG level 3 inputs (Wang et al, 2008). Under MEPDG Work Plan Task 5, the thermal properties of typical
Iowa concrete materials, such as CTE and thermal conductivity were analyzed (Wang et al, 2008).

- Louisiana Transportation Research Center (LTRC) completed a study that measured the CTE of typical concrete mixes used in Louisiana. Three types of aggregates (Kentucky limestone, gravel, and Mexican limestone) were used in this study. CTE was measured with various ages (3, 5, 7, 14, 28, 60, and 90 days), coarse aggregate proportions (20, 64, and 80 percent of coarse aggregate), and relative humidity values of specimen to identify the factor that has the most critical impact on CTE (Shin, H.C. and Y. Chung, 2011).

- Michigan DOT funded a research study to measure the CTE of typical PCC mixtures made with eight different sources of coarse aggregates (Buch and Jahangirnejad, 2008). Evaluating the sensitivity of the MEPDG performance predictions, this study found the PCC transverse cracking sensitive to CTE values, slab thickness and joint spacing; and further observed that, from a practical perspective, a combination of thinner slab, longer joint spacing, and higher CTE values could prove detrimental to pavement performance.

- Mississippi DOT has completed developing a library of the MEPDG inputs for PCC materials through a research contract (Al-Ostaz, 2007). The testing plan will include a range of concrete mixtures with typically encountered aggregate types and cement blends. Extensive testing of all key PCC materials properties was conducted.

- Similarly, Pennsylvania DOT is also developing a library of PCC inputs for use in the MEPDG through a research contract (Nassiri, S and J. M. Vandenbossche, 2011).

- WisDOT has developed a database of splitting tensile strength and CTE values for PCC materials. The experimental plan represented various types of coarse aggregates from 15 sources and various combinations of cementitious materials (Naik et al, 2006). WisDOT also funded another project that evaluated local aggregates and cementitious materials for fresh concrete, thermal and strength properties (Effinger et al, 2012). This study included fifteen sources of coarse aggregate, two sources of fine aggregate, two sources of ordinary Portland cement, two sources of slag cement, and three sources of fly ash.

While most of the activities focused on building a data library of PCC material properties, several agencies have shown considerable interest in measuring the CTE values of typical PCC mixtures with local aggregates and understanding the significance of CTE in performance predictions.

### 3.3 Chemically Stabilized Materials

There have been very few studies related to determining the material properties of chemically stabilized materials for implementing the MEPDG. One reason for this observation could be that the fatigue cracking transfer function for semi-rigid pavements was not calibrated under NCHRP projects 1-37A or 1-40D. The Montana DOT is the only agency to-date that completed a material characterization and local calibration study of the fatigue cracking transfer function of semi-rigid pavements, which was done using version 0.9 of the MEPDG. The following summarizes the findings from that study for use in semi-rigid pavement design.
• For High Strength Cement Aggregate Mixtures (CAM)—(intact cores can be recovered and mixture has cement contents greater than 6 percent, with compressive strengths generally greater than 1,000 psi:
  o $B_{c1} = 0.85$.
  o $B_{c2} = 1.10$.
• For CAM with moderate levels of cement—intact cores can be recovered and mixture has cement contents greater than 4 percent but less than 6 percent, with compressive strengths generally greater than 300 psi but less than 1,000 psi:
  o $B_{c1} = 0.75$.
  o $B_{c2} = 1.10$.
• For Low Strength CAM—intact cores cannot be recovered and mixture has cement contents generally less than 4 percent, with compressive strengths less than 300 psi:
  o $B_{c1} = 0.65$.
  o $B_{c2} = 1.10$.

Version 0.9 of the MEPDG, however, contained an error in the software where the elastic modulus of the chemically stabilized layer was hard-coded and could not be changed at the time. Thus, any difference in elastic modulus between the chemically stabilized materials had to be considered through the local calibration coefficient, as summarized above. Other on-going studies related to the chemically stabilized layers of semi-rigid pavements are listed below:

• Mississippi DOT funded a study to quantity the effects of compaction and moisture conditions on the strength of chemically stabilized soils (James et al, 2009). The findings will be used in conjunction with the new MEPDG to optimize pavement structural sections and to provide data to improve construction specifications. Results from this study should be directly applicable to Georgia, if chemically stabilized layers are going to be considered as a design strategy alternate.
• Although not a part of MEPDG implementation, Oklahoma DOT funded a project to characterize chemically stabilized materials. This study focused on eight common fine-grained soils (A-4 through A-7-6) stabilized with four different chemical additives (hydrated Lime, cement kiln dust and 2 sources of Class C Fly Ash) in varying amounts (Cerato et al, 2011).
• NCHRP Project 4-36 is evaluating different fatigue cracking transfer functions for chemically stabilized materials in comparison to the current transfer function included in the MEPDG. The objective of the project is to recommend a fatigue cracking transfer function for the semi-rigid pavements that can be included in the ME Design software, if found to be more accurate than the current transfer function. Multiple test sections have been sampled and are being monitored to measure various properties of these layers, in addition to those required as inputs to the MEPDG. The project, however, has yet to be completed.

3.4 Unbound Materials

The implementation activities pertinent to the characterization of unbound materials have primarily focused on the following objectives:

• Developing a resilient modulus data library for typical granular aggregate base materials and local subgrade soils.
• Developing a resilient modulus prediction model based on soil parameters.
Utilizing FWD and other non-destructive tests to determine the resilient modulus.

The key activities of State agencies are summarized as follows:

- Florida DOT has developed a database of resilient modulus for local soils (Ping et al, 2003).
- As a part of ITD’s MEPDG research project, default Level 3 resilient modulus values for Idaho unbound materials and subgrade soils (Bayomy et al, 2012). This study utilized a database of ITD historical test results collected from the different Idaho districts. This study also developed a correlation or regression model between resilient modulus and R-value for Level 2 inputs of fine grained soils in Idaho.
- Indiana DOT conducted an in-house testing program to assess the resilient and plastic strain behavior of involving 14 cohesive subgrade soils and five cohesionless soils commonly encountered in the state (Kim and Siddiki, 2006). The experimental plan included resilient modulus tests, physical property tests, unconfined compressive tests, and Dynamic Cone Penetrometer (DCP) tests. This study proposed regression models for predicting resilient modulus based on unconfined compressive strengths, and the k1, k2 and k3 coefficients of the resilient modulus constitutive model based on basic soil properties; such as moisture content, dry density and Atterberg limits.
- Another Indiana DOT in-house study investigated the relationship between the FWD modulus and lab measured resilient modulus of the subgrade soils under four flexible pavement sections (Dai et al, 2010). This study found that the lab resilient modulus of the soil, on average, was 0.48 times the FWD modulus; slightly higher than the value reported or recommended for use by the MEPDG Manual of Practice (0.35).
- Two decades earlier, although not directly related to the MEPDG implementation, Indiana DOT developed a set of resilient modulus data of six commonly found soils (Lee et al, 1992). This study developed a set of correlations between the resilient modulus and the unconfined compression test results for normal and thawed subgrade conditions, based on the test results on the specimen sampled from existing subgrades.
- Through MEPDG Work Plan Task 5 contract, Iowa DOT has also developed a library of Iowa typical unbound pavement materials containing resilient modulus inputs established using the repeated load triaxial resilient modulus test results (Ceylan et al, 2009). This library includes the non-linear, stress-dependent resilient modulus model coefficients values for input level 1, the unbound material properties values correlated to resilient modulus for input level 2, and default resilient modulus values for input level 3.
- Kentucky Transportation Cabinet has developed a prediction model based on resilient modulus tests on typical types of crushed stone aggregate bases (Hopkins et al, 2007). A windows based computer program was developed to make the resilient modulus data and the new model readily available to agency design personnel, and is embedded in the Kentucky Geotechnical Database. Earlier, the same team developed a similar model for predicting the resilient modulus of typical Kentucky soils (Hopkins et al, 2001).
- LTRC conducted field and laboratory testing programs to develop resilient modulus prediction models for Louisiana’s subgrade soils using the test results of DCP, CIMCPT, FWD, Dynaflect, and soil properties (Mohammed et al, 2007). Four soil types and nine overlay rehabilitation pavement projects in Louisiana were selected. LTRC is conducting another project that intends to
validate the prediction of seasonal variation strengths in the base course and subgrade (Contract Number: 30000425).

- Michigan DOT evaluated whether the processes used by its regions in determining the roadbed resilient modulus and modulus of subgrade reaction are compatible with the new MEPDG requirements (Baladi et al, 2009). Another study developed the resilient moduli of the various granular base and subbase materials using FWD backcalculation (Baladi et al, 2011).
- Minnesota DOT funded a study to investigate the strength and deformation characteristics of base material produced from various proportions of RAP and aggregate base (Kim and Labuz, 2007). Resilient modulus and compaction characteristics were also evaluated.
- Mississippi DOT has developed several predictive models to estimate resilient modulus of typical Mississippi soils from their soil index properties (George, 2004). A similar study investigated the viability of using FWD data for deriving resilient modulus through empirical correlations (George et al, 2003).
- Mississippi DOT has recently completed the testing of 34 subgrade soils, 13 granular base/subbase materials and 16 stabilized soils for developing the pavement materials library. The agency is currently documenting the valuable practical experience, lessons and observations that were gained during the testing and review of the data.
- Missouri DOT has developed a library of resilient modulus values for granular base materials and subgrade soils (Richardson et al, 2007). The experimental plan included 27 subgrade soils and five granular base materials commonly found in Missouri. This study also presents regression models to estimate k1, k2 and k3 coefficients using basic soil properties.
- Montana DOT compared over thirty different resilient modulus prediction models available in the literature and evaluated those with laboratory data for two soils sampled in Montana (Mokwa and Akin, 2009). This study discouraged the general use of such models without prior testing and verifying the reliability of the model estimates until additional studies suggest otherwise.
- A New England Transportation Consortium (NETC) study developed regression equations to estimate k1, k2 and k3 coefficients for typical soils encountered in New England states (Malla and Joshi, 2006). Using the data extracted from the LTPP database, this study developed prediction equations for six AASHTO soil types and further validated with lab measurements.
- Cold Regions Research and Engineering Laboratory (CRREL) conducted resilient modulus tests on five subgrade soils commonly found in the state of New Hampshire (Janoo et al, 1999).
- New Jersey DOT funded a laboratory program to determine the resilient modulus of typical New Jersey subgrade soils (Bennert et al, 2000). Laboratory results were used to calibrate a statistical model for predicting the resilient modulus at different moisture contents and stress ratios.
- An Oklahoma DOT study evaluated the effect of post-compaction moisture content on the resilient modulus of selected soil types (Zaman and Khoury, 2007). The findings of this study were later used to improve the existing database of resilient modulus and suction values for selected soil types. Another Oklahoma DOT study compiled resilient modulus data for Oklahoma subgrade soils and aggregates (Hossain et al, 2011). This study also conducted statistical analyses of the collected data to evaluate selective stress-based models for unbound and stabilized subgrade soils, and develop correlations between resilient modulus and other routine soil parameters.
Another Oklahoma study undertook an experimental plan to investigate engineering properties of chemically stabilized subgrades. The plan included resilient modulus, modulus of elasticity, unconfined compressive strength, moisture susceptibility and three-dimensional swell. Four different types of soils treated with three stabilizers (hydrated lime, class C fly ash and cement kiln dust) were used (Solanki et al, 2009).

VTRC undertook a program to develop a library of resilient modulus values for Virginia's subgrade soils for use in the MEPDG (Hossain, 2008). More than 100 soil samples from all over Virginia representing every physiographic region were collected and tested for resilient modulus, soil index properties, standard Proctor, and California Bearing Ratio testing. Resilient modulus values and regression coefficients (k-values) of constitutive models for resilient modulus for typical Virginia soils were computed. This study observed that only quick shear test was found to have statistically significant correlations with resilient modulus. Another VTRC study focused on developing a database of resilient modulus values (or k-values) for typical unbound base materials and subgrade soils (Hossain, 2010)

WisDOT funded a laboratory testing program to evaluate their physical and compaction properties of commonly found subgrade soils (Titi et al, 2006). This study developed statistical correlations to estimate k1, k2 and k3 coefficients from basic soil properties. Another WisDOT study undertook an experimental plan to develop a resilient modulus predictive model for typical crushed aggregate base materials encountered in Wisconsin (Eggen and Brittnacher, 2004). The plan included 37 aggregate sources and a wide range of influencing variables, such as physical characteristics, material type, source lithology and regional factors, were evaluated for their effect on resilient modulus.

Most agencies have undertaken comprehensive laboratory studies to measure resilient modulus properties of typically encountered materials and soils. These studies have then used these experimental results to either build a data library of typical values or develop statistical models for estimating resilient modulus from basic soil properties.

Some studies have developed empirical models to derive k1, k2 and k3 coefficients for the resilient modulus constitutive model. Although the MEPDG recommends the use of lab measured resilient modulus properties, Khazanovich et al (2006) observed that the Guide does not provide adequate guidance on using the test data in multilayer elastic theory (MLET) analysis. In their 2006 TRB paper, the authors provide a detailed step-by-step guidance on how to determine resilient modulus using the model coefficients by taking stress states into account.

FHWA and various State pavement associations, however, have sponsored numerous studies to determine the in place resilient modulus of aggregate base layers and subgrades using the results from repeated load resilient modulus tests included in the LTPP database and other sources. A few of these include:

Yau and Von Quintus (2002) developed regression equations to estimate the k1, k2 and k3 coefficients of the universal resilient modulus equation recommended for use in the MEPDG Manual of Practice. The authors suggested, however, that the regression equations be used with caution because of the poor statistics from the regression analyses.
Von Quintus and Killingsworth (1998) developed and recommended values for the AASHTO c-factor in relating laboratory-derived resilient modulus for the in place unbound layers in comparison to the field-derived elastic modulus backcalculated from deflection basins. The c-factors were found to be dependent on pavement structure and independent of soil or material type. The variation in values for the c-factors for each data set, however, was high in some cases – exceeding a coefficient of variation of 30 percent.

These studies combined with the Florida, Louisiana, Mississippi, and Virginia studies and initial studies on resilient modulus sponsored by Georgia can be used to provide recommended values for a range of soils and their in place condition for establishing inputs to the ME Design software.

GDOT currently uses Soil Support Values (SSV) for characterizing subgrade in pavement design. The SSV, which typically ranges between 2.0 and 4.5, is determined based on its correlation with CBR. The MEPDG allows using CBR as a Level 2 input for subgrade characterization. The MEPDG uses the following relationship to convert CBR to resilient modulus:

$$M_r = 2555(CBR)^{0.6}$$

While CBR test is typically conducted at a range of moisture contents and compactive effort, the design CBR is selected based on the degree of compaction and moisture content expected in the field. In the ME Design software, when the design CBR is used an input to determine subgrade resilient modulus, the moisture content and density values associated with the input CBR must also be used.

The AASHTO MEPDG Interim Manual of Practice provides recommendations of Level 3 default resilient modulus values for use in the ME Design software (see Table 1). Note that the Level 3 resilient modulus values presented in this table represent optimum moisture condition and maximum dry density typically anticipated in the field at the time of construction.

<table>
<thead>
<tr>
<th>AASHTO Soil Classification</th>
<th>Recommended Resilient Modulus at Optimum Moisture (AASHTO T 180), psi</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Base/Subbase for Flexible and Rigid Pavements</td>
</tr>
<tr>
<td>A-1-a</td>
<td>40,000</td>
</tr>
<tr>
<td>A-1-b</td>
<td>38,000</td>
</tr>
<tr>
<td>A-2-4</td>
<td>32,000</td>
</tr>
<tr>
<td>A-2-5</td>
<td>28,000</td>
</tr>
<tr>
<td>A-2-6</td>
<td>26,000</td>
</tr>
<tr>
<td>A-2-7</td>
<td>24,000</td>
</tr>
<tr>
<td>A-3</td>
<td>29,000</td>
</tr>
<tr>
<td>A-4</td>
<td>24,000</td>
</tr>
<tr>
<td>A-5</td>
<td>20,000</td>
</tr>
</tbody>
</table>
Table 1. MEPDG Level 3 Resilient Modulus for Unbound Materials.

<table>
<thead>
<tr>
<th>AASHTO Soil Classification</th>
<th>Recommended Resilient Modulus at Optimum Moisture (AASHTO T 180), psi</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Base/Subbase for Flexible and Rigid Pavements</td>
</tr>
<tr>
<td>A-6</td>
<td>17,000</td>
</tr>
<tr>
<td>A-7-5</td>
<td>12,000</td>
</tr>
<tr>
<td>A-7-6</td>
<td>8,000</td>
</tr>
</tbody>
</table>

For rehabilitation designs, the resilient modulus of each unbound layer and embankment may be backcalculated from the FWD deflection basin data. A key debate with the use of backcalculated unbound moduli values is related to the relationship between field-derived FWD modulus and laboratory-measured resilient modulus of unbound materials has yet to be resolved. The debate is whether the AASHTO c-factor or ratio (i.e. C = Mr/EFWD) between the laboratory-derived resilient modulus and elastic modulus backcalculated from deflection basins is a reality, as well as the accuracy and appropriateness of the ratios that have been reported to date. This debate is significant particularly in the context of rehabilitation design, as the MEPDG requires lab measured resilient moduli at optimum moisture content (at the time of construction) as inputs for unbound layers, while the in-situ condition may not represent the same.

The MEPDG Interim Manual of Practice recommends the use of c-factors presented in Table 2 to adjust FWD backcalculated unbound layer modulus to an equivalent laboratory derived resilient modulus. These factors represent the moisture content and associated dry density of the in-situ materials; therefore, these factors should only be used in conjunction with the in-situ moisture contents and dry densities measured from materials recovered from field sampling such as borings.

Table 2. C-factors Recommended in the MEDPG Manual of Practice to Convert FWD Backcalculated Subgrade Modulus to Laboratory Derived Resilient Modulus.

<table>
<thead>
<tr>
<th>Layer Type</th>
<th>Location</th>
<th>C-Value or Mr/EFWD Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aggregate Base/Subbase</td>
<td>Between a Stabilized &amp; HMA Layer</td>
<td>1.43</td>
</tr>
<tr>
<td></td>
<td>Below a PCC Layer</td>
<td>1.32</td>
</tr>
<tr>
<td></td>
<td>Below an HMA Layer</td>
<td>0.62</td>
</tr>
<tr>
<td>Subgrade-Embankment</td>
<td>Below a Stabilized Subgrade/Embankment</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td>Below an HMA or PCC Layer</td>
<td>0.52</td>
</tr>
<tr>
<td></td>
<td>Below an Unbound Aggregate Base</td>
<td>0.35</td>
</tr>
</tbody>
</table>

However, during global calibration of MEPDG performance models, a c-factor of 0.55 and 0.67 was used for fine-grained and coarse-grained soils. These factors represent the optimum moisture content and associated maximum dry density of the materials; therefore, these factors should only be used in conjunction with optimum moisture content and maximum dry density of the subgrade material.
An on-going FHWA study is expected to shed more light on the relationship between field-derived and laboratory-measured modulus values. This project is currently investigating the fundamental principles underlying the observed differences between backcalculated layer moduli and laboratory resilient moduli. This study is expected to demonstrate how to use interchangeably lab and field derived resilient moduli or throw light on why it is not possible to derive such relationships (Contract Number: DTFH61-09-C-00007).

### 3.5 Material Characterization for Rehabilitation

The implementation efforts in this area have focused on identifying critical material parameters and setting up laboratory programs and field evaluation for rehabilitation design. FHWA is currently funding a project to perform backcalculation of all deflection basin data that are stored in the LTPP database for flexible, semi-rigid, rigid, composite, and rehabilitated pavement structures (Contract Number: DTFH61-11-R-00019). Texas DOT has developed a framework (Tex-ME) that documents the laboratory and field procedures to be used in material characterization for rehabilitation design; recommendations for Level 1 characterization of existing pavement damage i.e. the rutting potential of asphalt, granular, and soil layers and the cracking potential of asphalt layers (Zhou et al, 2009).

### 4. Climate and Environmental Effects

#### 4.1 Climate Data

While several agencies including Colorado, Florida, Iowa, Minnesota, Mississippi and Wyoming, are either expanding or considering expansion of climatic data sets by adding the coverage period (e.g., 30 to 50 year data sets) and/or coverage area, fewer instances are recorded in the literature. The implementation activities published in the literature primarily focus on the following objectives:

- Identifying climatic sub regions within a state.
- Assessing the quality of climatic data.
- Evaluating the validity of climate-related inputs (e.g. thermal conductivity).
- Analyzing the impacts on the use of virtual weather stations on performance predictions.

Examples include:

- California DOT (Caltrans) evaluated the impact of pavement temperatures and precipitation of seven distinct climatic regions on distress mechanisms for rigid, flexible and composite pavements. Based on the findings, this study summarized the expected effects of climate region on specific pavement distress mechanisms (Harvey et al, 2000).
- Florida DOT’s Phase I implementation study assembled a database of key climatic variables (e.g. air temperatures, precipitation, relative humidity, Thornthwaite Moisture Index) and grouped statewide counties into four climatic sub regions using cluster analysis (Fernando et al, 2007). The missing data were reconciled with interpolation of corresponding hourly records from neighboring stations.
- Johanneck and Khazanovich (2010) evaluated the effect of climate on predicted performance of a composite pavement (HMA over PCC), and the effect of climate file generation. Some
inconsistencies in the PCC cracking predictions were observed due to incomplete or questionable records in the MEPDG hourly climatic data.

- Another study by Johanneck et al (2011) examined the measured and MEPDG modeled temperature distributions in the composite pavement structures at the MnRoad facility. The EICM simulations of the MEPDG produced temperature distributions smaller than the measured distributions when the MEPDG default thermal conductivity value of PCC, \( k = 1.25 \text{ BTU} / \text{hr-ft-°F} \), was used. A sensitivity analysis of PCC thermal conductivity indicated that a value of 0.94 BTU / hr-ft-°F resulted in closest agreement between modeled and measured data for a 6-in PCC test section.

- Saha and Bayat (2011) compared the predicted performance of flexible pavements using actual station-based and virtual station-based climatic data. The virtual weather station is created by interpolating climatic data (as a function of geographic distance) from surrounding locations. This study observed that the differences in predicted total rut depth between virtual and actual stations ranged from -20 to 50 percent. Total rut depth and HMA rutting appeared to be more sensitive to these differences, while the IRI appeared to be less sensitive. These differences were primarily attributed to missing hourly records in some stations and poor quality data.

- Li et al (2010) compared the virtual climate data generated using the MEPDG with the LTPP Automated Weather Station data. This study observed when using a greater number of nearby weather stations provided more accurate results than using the closest weather station. Rather than the distance between the stations, elevation appeared to have significantly affected the accuracy of the virtual climatic data. Similar to the observations made by Saha and Bayat (2011), this study also concluded that the variations in climatic data appeared to have greater influence on AC rutting and lesser influence on the predicted IRI.

- Through a research contract with the University of Maryland at College Park, FHWA is currently examining current and emerging needs in climate data collection and engineering indices for use in MEPDG calibration, changes in Superpave binder performance grading, and development of future mechanistic based infrastructure management including pavement, bridge, and other types of asset management models (Contract Number: DTFH61-11-C-00030).

By evaluating the sensitivity of expected performance to climatic data, these studies reinforce the importance of good-quality hourly climatic records. Missing hourly records and errors in the raw climatic data files are likely to have adverse impact on the precision and reasonableness of the predicted performance. To account for missing records, the interpolation of data from neighboring stations is typically used (e.g. Florida). Such techniques should be used with caution by considering influencing factors such as elevation differences between the stations.

It should be understood and mentioned that most of the climate studies referenced above were completed prior to a major weather station data clean-up in releasing the latest version of the M-E Design software.

4.2 Environmental Effects of Unbound Materials

The implementation activities in this area have focused on the following objectives:
• Validating the predictions of MEPDG-based Enhanced Integrated Climatic Model (EICM) with actual test data.
• Examining the validity of EICM-related inputs.
• Investigating the response of resilient modulus to seasonal changes.
• Analyzing the behavior of unsaturated soils – i.e. the impact of changes in moisture content on measured resilient modulus.

The key activities of State agencies are summarized as follows:

• Arkansas HTD is funding a study to validate the estimations of the EICM model (Contract Number: TRC-0902). This study intends to monitor changes in moisture content and stiffness during a wet and dry season at select sites and compare the results with the predictions of the EICM model.
• A Florida DOT study analyzed the response of drying and wetting cycles on resilient modulus properties of soils to understand the impact of changes in moisture on the effective confining pressure of the material (Toros et al, 2008).
• Minnesota DOT funded a study to explore the applications of unsaturated soil behavior concepts in pavement design. This study investigated the effects of soil suction on shear strength and resilient modulus of four soils, each representing different regions of Minnesota (Gupta et al, 2007). This study developed models for incorporating suction effects in shear strength and resilient modulus measurements of highly compacted subgrade soils.
• New York State DOT is funding a research program to model the effects of seasonal variations on layer moduli of unbound (subbase and subgrade) materials. This study intends to cover the climatological and materials conditions of approximately 90 percent of the geographic area of the state (Contract Number: RF 55505-03-03).
• Ohio DOT has investigated resilient modulus and hydraulic conductivity properties of various drainable base materials (Liang, 2007). The author evaluated the sensitivity of EICM 3.2 estimations to materials thermal properties (surface short-wave absorptivity, thermal conductivity, and heat capacity), porosity, and fine contents (percent passing #200 sieve). The predictions of EICM were then compared with the environmental field data gathered at the Ohio DOT’s ATB 90 project site. The authors concluded that EICM 3.2 exhibited high sensitivity to the soil porosity and fines contents for moisture predictions, high to moderate sensitivity to the surface short-wave absorptivity and low sensitivity to the asphalt thermal conductivity and heat capacity for temperature predictions.
• Oklahoma DOT is currently investigating the effects of soil suction hysteresis on resilient modulus for commonly encountered subgrade soils through a laboratory testing program (Contract Number: DTRT-06-G-0016). A similar study conducted earlier evaluated the effects of wetting and drying cycles on resilient modulus of eight soils using soil-water characteristics curves (SWCC). Based on its findings, the study suggested further testing with an expanded scope to characterize the behavior of soils subjected to cyclical seasonal changes (Zaman and Khoury, 2007).
• NCHRP Project 9-23A has developed a national database of pedologic soil families that contains the soil properties needed as inputs to the MEPDG. This database focuses upon the parameters describing the SWCC, but also includes measured soil index properties needed by the EICM in all
three hierarchical levels of pavement design (Zapata, 2010). This database is being implemented and used in Mississippi to establish site specific input values. The product being produced for Mississippi can be applied to Georgia to facilitate the implementation process and used in local calibration of the transfer functions.

The MEPDG uses the concepts of unsaturated soil behavior i.e. relationship between water content and matric suction as defined by the SWCC to model the effects of moisture on resilient modulus of unbound models. The models used in the MEPDG were largely drawn from the national LTPP database and limited field testing of sections located throughout North America (Zapata, 2009). Zapata suggested a more local or regional calibration may be needed for the EICM, as the national calibration may not be adequate for specific regions of the country.

5. **Traffic Inputs**

The implementation efforts in this area primarily have focused on the following:

- Analyzing Weigh-in-Motion (WIM) data with appropriate quality checks to develop traffic inputs for the MEPDG.
- Sensitivity analysis of traffic inputs on the MEPDG distress predictions and final pavement design thickness.
- Applications of statistical models and techniques such as Cluster Analysis in identifying homogenous traffic patterns.
- Review of current traffic collection infrastructure and practices to meet the requirements of the MEPDG.

GDOT is sponsoring a traffic study to develop truck traffic data in support of the MEPDG procedure. Currently, GDOT has many sites with axle weight data, but the predominant data was collected with portable weight scales. The accuracy of this data has been analyzed to determine whether it can be used to develop regional or state-wide averages in developing default normalized axle weight values in support of the MEPDG. The preliminary results from this analysis suggest that the WIM data collected by these portable scales are insufficient in providing the default or site specific truck traffic inputs. Thus, a plan is being developed to collect this data over time. The following activities and plans summarize the key efforts by the State agencies from which some of these results can possibly be used to generate default values to be used by GDOT in the interim:

- Alabama DOT has developed traffic factors and axle load distribution models using WIM data from thirteen sites (Turochy et al, 2005). A sensitivity analysis was also performed to determine the effects the variation in truck factors on the final pavement design thickness. Statistical models, such as some combination of log normal and normal distributions, were developed for axle load spectra. ALDOT is currently funding a research project to develop Alabama Traffic Factors for Use in M-E Pavement Design. The agency plans to develop materials reference library and concurrent designs and perform local calibration in the near future (UTCA Project Number: 12415).
• Arizona DOT has evaluated the way traffic data are acquired and compiled to make it compatible with the requirements of the MEPDG (Project Number: SPR 672). This project investigated the existing traffic data collection infrastructures, such as weigh in motion (WIM) stations, determined their validity and usefulness for use with the MEPDG, and developed a detailed action plan for ADOT to continuously obtain all necessary traffic data and compile that information for effective use in the MEPDG. (Darter, et al 2013)

• Arkansas HTD has developed traffic inputs for initial implementation and a procedure for updating these inputs in the future (Tran and Hall, 2006). Classification and weight data collected at 55 WIM stations were used in this study. Quality control checks were performed to ensure accurate interpretation of the data. A sensitivity analysis was also performed to evaluate the use of default traffic values in place of statewide vehicle class distribution factors and axle load spectra. Through a follow-up research project, Arkansas HTD has also developed a software program to pre-process raw traffic data, import, check data quality, and generate the required traffic inputs for the MEPDG (Wang et al, 2009).

• Caltrans developed traffic inputs and axle load spectra from WIM data collected during 1991 to early 2001 on the State highway network (Lu and Harvey, 2006; Lu et al, 2002). This study also evaluated the possibility of extrapolating available truck traffic data to sites where WIM stations were not installed. Cluster analysis was applied to traffic data to extract influential factors and homogenous traffic patterns to ensure the preservation of useful information during the analysis.

• Another Caltrans study focused on the truck traffic growth patterns, sensitivity of pavement responses to variation in growth rates, and potential contributing predictors that can be used to predict truck traffic growth rates (Lu et al, 2007). While both linear and compound growth functions can model growth trends, this study observed the linear growth function fit the data slightly better. This study further recommends that at least six-year traffic observations should be used for estimating growth rates to reduce variance in truck volume predictions and their significant effect on pavement design.

• Colorado DOT has completed characterizing LTPP and non-LTPP traffic data for use in the MEPDG. This study used cluster analysis techniques for identifying similarities and dissimilarities among data sources. This study developed a catalog of traffic inputs for use in MEPDG (Mallela et al, 2010).

• Under the MEPDG implementation research project, Idaho has developed site-specific and State wide traffic inputs using classification and weight data from 25 WIM sites in Idaho (Bayomy, 2011).

• Indiana DOT has developed a Visual Basic program to process WIM data and prepare traffic inputs for the MEPDG.

• Louisiana is evaluating its current traffic characterization techniques for their compatibility with the MEPDG requirements. The agency intends to develop traffic load spectra from available truck traffic data sources, update its load equivalency factors, and make recommendations for its future implementation efforts (Contract Number: 736-99-1411).
Michigan DOT has developed a data library of site-specific traffic inputs using WIM data from 44 stations (Buch et al, 2009). This study utilized cluster analysis to group sites with similar characteristics and subsequently discriminant analysis to develop regional traffic inputs. Data from all sites were averaged to establish the statewide Level 3 inputs. The effects of the developed hierarchical traffic inputs on the predicted performance of rigid and flexible pavements were also investigated.

Mississippi DOT’s has developed a software program to automate the processing of raw traffic data and prepare inputs for the MEPDG (Jiang and Saeed, 2007). This study also provided additional support in the form of technical documentation, user’s guide, on-site software installation, and training. Earlier, Buchanan (2004) reviewed traffic data obtained from LTPP sites located within the state and developed baseline data for internal use. The agency is currently funding a project to establish procedures for quantifying the effects of changing traffic conditions on pavement performance and to enable traffic personnel in Mississippi to perform the traffic analysis for implementing and using the M-E Design software. It also involves a software upgrade component so that the traffic analysis tool is compatible with M-E Design software and future Windows platforms (Contract Number: 257).

Missouri DOT has completed the quality analysis of WIM data and preparation of inputs for the MEPDG.

North Carolina DOT has developed a database of traffic data with Level 1, 2 and 3 inputs (Stone et al, 2011). This study developed an implementation plan that identified the resources needed for traffic data collection, reviewed existing infrastructure and practices and identified homogenous groups of traffic patterns for regional inputs and training. Preliminary findings of this study indicate multi-dimensional hierarchical clustering analysis and decision trees are applied in generating regional values of axle load and monthly adjustment factors (Sayyady et al, 2010). This study utilized an MS-Access based quality control procedure to review the WIM volume and weight data. A MEPDG-damage based sensitivity analysis was also conducted to identify sensitive traffic factors.

Oregon DOT has developed state-specific traffic inputs through a traffic characterization study (Elkins and Higgins, 2008). Four weigh-in-motion (WIM) sites representing high, moderate, and low average daily truck traffic (ADTT) volumes, were selected to characterize axle weight and spacing spectra on Oregon State highways.

VTRC developed traffic inputs for Virginia DOT using traffic data from eight interstate and seven primary route WIM sites (Smith and Diefenderfer, 2010). This study evaluated the statistical significance of differences in predicted distresses for flexible and rigid pavements between site-specific and default traffic inputs. This study provided recommendations for using different hierarchical level of traffic inputs for different roadway functional classes. Currently, VTRC is reviewing VDOT’s plan to collect traffic and truck-axle weight data and propose revisions, if needed. The review will assess the data obtained from the Division of Motor Vehicle (DMV) sites and the appropriateness of the truck-weight groups and compile information on truck-travel patterns and characteristics (Contract Number: CSC 1118012-00092722-50012).
• FHWA through the LTPP program has also sponsored multiple studies to improve on the default normalized axle load distributions and other inputs required by the MEPDG procedure. The “best” WIM sites within the program were identified and selected to generate and recommend default axle load distributions to be used in design with the ME Design software. Recommended default distributions were developed but have yet to be incorporated into the M-E Design software. Those default distributions are being considered for use in Task 2 of Task Order #1 for the GDOT MEPDG implementation project. The limitation of these default distributions is that most of the “best” LTPP WIM sites are located on the interstate system with a portion of these WIM sites located on primary arterials.

• Truck wheel base was estimated through studies in Utah, Arizona, and a national based LTPP study conducted by Selenzneva, et al, 2012). The national mean truck wheel base for trucks was established from 25 WIM stations to be the following mean values:
  - Short = 17%;
  - Medium = 22%;
  - Long = 61%.
In addition, new axle load distributions for all types of axles and vehicle classes were established in the same study based on many years of WIM measurements.

In summary, most studies have focused on building data libraries for traffic inputs and axle load spectra. Other related efforts include: developing customized software programs to derive MEDPG inputs from WIM data, and evaluating the impact of using MEDPG defaults (input level 3) in place of agency-specific or site-specific (input level 1) traffic data. Some agencies (e.g. Arizona, California, Colorado, Michigan and North Carolina), have utilized statistical techniques such as cluster analysis to identify homogenous groups of traffic patterns for developing regional traffic inputs. As noted above, Georgia has also sponsored traffic studies to determine the truck traffic inputs to the MEPDG. These are being completed by ARA and will be included and used within the local calibration process. Specifically, results from that study have provided the default normalized axle load spectra for each LTPP test section for comparing the predicted to measured distress values under Task Order #1, Task 2.

6. INSTRUMENTATION AND SENSITIVITY ANALYSIS

6.1 Instrumentation & Accelerated Pavement Testing Studies
Several State agencies have funded pavement instrumentation studies or accelerated pavement testing studies to conduct full-scale investigation of pavement responses to climate, traffic and changes in material properties. Data collected from these studies are typically used for verification and/or local calibration of the MEPDG performance prediction models. Some examples of these studies include:

• KDOT has constructed five new pavement sections and used four pavement test sections on the Kansas perpetual pavement project on US-75 as test sections to conduct field verification of Superpave mixtures. Both field and laboratory tests were conducted for material characterization (Gedafa et al, 2009). KDOT has continued collecting pavement distress data even after the project was completed. KDOT intends to utilize both material characterization and distress data for local calibration.
In the North East, Maine DOT is funding a pavement instrumentation study to obtain in-place data necessary for adopting the MEPDG. New Hampshire DOT is also constructing a fully instrumented pavement section to collect data for local calibration of the MEPDG.

Oklahoma DOT is funding a similar study that focuses on monitoring and modeling of the test section on I-35 to facilitate collection of MEPDG related data in an accelerated manner (Contract Number: SP&R 2200).

Pennsylvania DOT sponsored a multi-year project called Superpave In-Situ Stress/Strain Investigation (SISSI) encompassing eight different instrumented full-depth HMA pavement sections located across the state (Solaimanian et al, 2006). This project focused on pavement instrumentation, response measurement to vehicle loading and environment, distress evaluation and data collection for the MEPDG model validation. The data collected from these sites is also being used towards the calibration and validation of the MEPDG transfer functions and other models.

6.2 Sensitivity analysis

State highway agencies and other organizations have conducted or currently funding several studies to assess the relative sensitivity of the MEPDG performance predictions to various inputs. Some examples of these studies include:

- NCHRP Project 1-47 has completed the sensitivity studies of the MEPDG performance predictions to variability of input parameter values. Global sensitivity analyses were performed for five pavement types under five climate conditions and three traffic levels. Design inputs evaluated in the analyses included traffic volume, layer thicknesses, material properties (e.g., stiffness, strength, HMA and PCC mixture characteristics, subgrade type), groundwater depth, geometric parameters (e.g., lane width), and others. This study found that, for all pavement types and distresses, the sensitivities of the design inputs for the bound surface layers were consistently the highest. (Schwartz, 2011). Results from NCHRP 1-47 will be used in the Georgia implementation study to select sites for the local calibration and in evaluating the residual error of the predicted distress values.

- Earlier, Schwartz (2007) conducted a study for MDSHA that compared flexible pavement designs and performance between the empirical 1993 AASHTO pavement design guide and the MEPDG, and performed a sensitivity analysis of various input parameters.

- For Arkansas HTD, Hall et al (2006) analyzed the sensitivity of the MEPDG performance predictions to various design inputs.

- Kannekanti and Harvey (2006) developed a sample catalog of simple design tables (catalog) for rigid pavement design based on the ME Design software that are being used for design by Caltrans.

- For Georgia DOT, Watson et al (2009) compared the design results of the AASHTO 1972 Guide with the MEPDG distress predictions at different hierarchical input levels.
For Michigan DOT, Buch et al (2008) conducted a sensitivity analysis of rigid and flexible pavement models. For rigid pavements, the results showed the effect of PCC slab thickness and edge support on performance were significant among design variables while CTE, flexural, base type and subgrade played an important role among material related properties. For flexible pavements, significant variables include HMA layer thickness, HMA mix characteristics, binder grade, base, subbase and subgrade moduli, and base and subbase thickness. Significant interactions were found among several of the variables in affecting all the performance measures.

Ala et al (2009) conducted a parametric study for Nebraska DOR to identify the parameters that are important and level of sophistication that is needed at the input level.

Nebraska DOR is currently funding a study that focuses on investigating the impact of heavy truck loading on damage of flexible pavements. This study intends to compare the MEPDG analysis results to the results from the purely mechanistic approach based on the finite element method (FEM).

Won (2009) conducted a sensitivity analysis of the MEPDG punchout model using project data of 27 continuously reinforced concrete pavement (CRCP) sections obtained from the TxDOT Rigid Pavement Database.

Freeman et al (2006) conducted a sensitivity analysis of the MEPDG for TxDOT to estimate to what degree the input parameters affect the performance of the initial design.

7. **Local Calibration and Deployment of MEPDG**

7.1 **Validation and Calibration**

The MPEDG software includes the global and local calibration factors for distress and IRI prediction models. These calibration factors are used to make adjustments to the predicted values so that the difference between the measured and predicted values, defined as the residual error, is minimized. The global calibration coefficients are based on hundreds of pavement sections located throughout the US and Canada and are the defaults in the software. Some agencies conduct local calibration and adjust these global coefficients as needed to improve on their prediction capability. To further the implementation process, NCHRP sponsored a project under NCHRP 1-40B to develop a local calibration guide to provide guidance to agencies deciding to conduct a local calibration-validation effort. That local calibration guide was adopted and published by AASHTO (AASHTO, 2010). Figures 5 and 6 show the different steps suggested for calibrating and validating the MEPDG to local conditions, policies, and materials. A total of eleven steps are defined within that local calibration guide.

Table 3 lists the coefficients of the MEPDG transfer functions or distress and IRI prediction models that should be considered for revising the predictions to eliminate model bias for flexible pavements and HMA overlays. The model bias is the summation of the individual residual errors. Table 3 was prepared to provide guidance in eliminating any local model bias in the predictions. The distress specific parameters can be dependent on site factors, layer parameters, or more importantly, the operational and management policies of the agency. All transfer functions in the ME Design software were calibrated to measured values at a representative spectrum of pavement test sites around North America. The LTPP-General Pavement Study (GPS) and Special Pavement Study (SPS) test sections were used extensively in the
Figure 5. Flow Chart of the Procedure and Steps Suggested for Local Calibration;
Steps 1 Through 5 (after NCHRP Project 1-40B, 2008a)

6 – Conduct Field Investigations of Test Sections to Define Missing Data

Accept MEPDG Assumptions; Forensic investigations NOT required – only field tests to obtain missing data.

Conduct field testing and materials sampling plan to define missing data.

Re-evaluate experimental matrix to ensure hypothesis can be properly evaluated; accept or reject the hypothesis; optional activity.

Determine inputs for each roadway segment and execute MEPDG – distress predictions.

PMS Segments; only PMS distress data

Roadway PMS segments with more detailed (research grade) surveys (LTPP)

Roadway segments, research grade condition surveys (LTPP); and/or APT Sites

8 – Determine Local Calibration Value to Eliminate Bias of Transfer Function

Use local calibration coefficient to predict distress & calculate standard error of the estimate.

Reject Hypothesis; local error too large

Accept/Reject hypothesis related for

Accept Hypothesis

9 – Assess Standard Error for Transfer Function

Accept/Reject hypothesis for standard error?

Accept Hypothesis

10 – Improve Precision of Model; Modify coefficients & exponents of transfer functions or develop calibration function.

11 – Interpretation of Results; Decide on Adequacy of Calibration Values.

Figure 6. Flow Chart of the Procedure and Steps Suggested for Local Calibration; Steps 6 Through 11 (after NCHRP Project 1-40B, 2008a)
calibration effort, because of the consistency in the monitored data over time and the diversity of test sections spread throughout North America.

Table 3. Calibration Parameters to be Adjusted for Eliminating Bias and Reducing the Standard Error of the Flexible Pavement Transfer Functions (after NCHRP Project 1-40B, 2008a)

<table>
<thead>
<tr>
<th>Distress</th>
<th>Eliminate Bias</th>
<th>Reduce Standard Error</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Rutting</td>
<td>Unbound Materials &amp; HMA Layers</td>
<td>$k_1$, $\beta_{s1}$, or $\beta_{r1}$</td>
</tr>
<tr>
<td>Load Related Cracking</td>
<td>Alligator Cracking</td>
<td>$C_2$ or $k_1$</td>
</tr>
<tr>
<td></td>
<td>Longitudinal Cracking</td>
<td>$C_2$ or $k_1$</td>
</tr>
<tr>
<td></td>
<td>Semi-Rigid Pavements</td>
<td>$C_2$ or $\beta_{c1}$</td>
</tr>
<tr>
<td>Non-Load Related Cracking</td>
<td>Transverse Cracking</td>
<td>$\beta_{t3}$</td>
</tr>
<tr>
<td>IRI</td>
<td></td>
<td>$C_4$</td>
</tr>
</tbody>
</table>

Irrespective of the calibration process, however, the final calibrated transfer functions have an error associated with them. In fact, all distress prediction models (transfer functions) have errors. This error is often termed the standard error of the estimate ($S_e$) and can be used to establish confidence intervals for the transfer function. In other words, this error explains the scatter of the data around the 1:1 line between the predicted and observed distress quantities. The standard error of the estimate of a transfer function is an important factor that must be understood and quantified in making a decision on whether to try and increase the precision of a transfer function.

Results from various calibration studies are directly applicable to Georgia for use in setting up a sampling matrix to verify, locally calibrate, and validate the MEPDG transfer functions.

The implementation efforts in this area have included:

- Identification of suitable pavement sections involving a wide range of pavement types for model verification. State PMS sections and the LTPP sites located within the state or neighboring states are typically selected for verification and local calibration efforts.
- Selection of proper MEPDG inputs for all of the selected sections to be used in the local calibration. This includes material and subgrade properties, design factors, local climatic conditions, and traffic loadings.
- Verification of the MEPDG performance prediction models to estimate distress and IRI in the State.
- Recalibration and validation of performance prediction models to local conditions, if the nationally calibrated models are statistically biased or inaccurate.

Multiple studies have been completed regarding the use of the MEPDG to confirm the accuracy and precision of the MEPDG transfer functions. The following summarizes some of the key efforts by State agencies:
Arizona DOT has been implementing the MEPDG since 2002 through extensive materials testing (Witczak, 2008) and traffic load characterization. (Darter, et al 2013) A major local calibration effort was then conducted using LTPP and PMS data from throughout the State by ARA, Inc. (Darter, 2012) The local calibration resulted in the obtaining of new calibration coefficients for most of the distress and IRI models for use in Arizona that improved their prediction accuracy. The NCHRP 20-07 calibration coefficients for JPCP cracking, faulting, and IRI using the AASHTO T336 test for coefficient of thermal expansion were verified.

Arkansas HTD is currently assessing the suitability of the nationally calibrated MEPDG models for Arkansas conditions. This study intends to locally calibrate the MEPDG model coefficients, if necessary, to provide more accurate distress predictions for local conditions (Contract Number: TRC1003).

Florida DOT completed a cooperative effort to establish and characterize field test sections for the purpose of compiling a database of materials, geometric, and traffic-related design variables to verify the predictions from the MEPDG program and perform local model calibrations as warranted (Oh and Fernando, 2008). From these efforts, researchers and FDOT engineers established thickness tables for flexible and rigid pavement designs.

Colorado DOT has completed their local calibration of the transfer functions for flexible and rigid pavements in pursuing implementation and use of the MEPDG.

The Indiana DOT has implemented the MEPDG and has used the procedure on many projects over the past 4 years. Materials testing and traffic analysis was conducted. Some local verification was also accomplished. (Tommy Nantung, et al, 2010)

Through a research contract with Iowa State University (ISU), Iowa DOT is currently performing local calibration for the MEPDG performance models using the Pavement Management Information System (PMIS) data (RiP Project 28337). Earlier Iowa DOT evaluated the PMIS data retrieved for interstate and primary roads for completeness and definition compatibility with respect to the MEPDG input requirements and outputs. Recommendations were made to update the existing PMIS and add new parameters that are currently unavailable, but are required for MEPDG rehabilitation design. (Ceylan at al, 2009)

Kansas DOT is currently conducting local calibration of the MEPDG performance models through a research contract with University of Kansas (Contract Number: RE-0610-01).

Kentucky Transportation Cabinet is currently funding a project to evaluate and recalibrate the nationally calibrated MEPDG models for local conditions (Contract Number: P10-396). This study also intends to develop a detailed implementation plan for integration of the MEPDG into the current design process.

Midwest Regional University Transportation Center is developing a regional pavement performance database for use in validation and calibration process of the MEPDG models. This research effort intends to collect data from multiple states including Illinois, Indiana, Iowa, Michigan, Minnesota, Ohio, and Wisconsin. It is expected that the extended database could result in more precise calibration factors for the Midwest region (Contract Number: 07-01).

Michigan DOT has a research contract with Michigan State University for the preparation of MEPDG implementation in Michigan. This project includes the HMA characterization (Contract Number: RC-1593), evaluation of rehabilitation designs (Contract Number: RC-1594), and the local calibration and validation of performance models (Contract Number: RC-1595). This
project is scheduled to be completed in 2014. Michigan DOT is expecting to begin the final transition to using the M-E Design software upon the completion of this project.

- Minnesota DOT and the Local Road Research Board (LRRB) initiated a study to evaluate the adoption of the nationally calibrated MEPDG models to local conditions (Velasquez et al, 2007). This study evaluated the use of default inputs, identified software deficiencies in version 1.003, and analyzed the results with local conditions for model recalibration.

- The Missouri DOT was the first agency to begin a major implementation effort of the MEPDG. Extensive materials testing (HMA, PCC, unbound) and traffic characterization was accomplished. Local calibration of all flexible and rigid pavement models was then performed and improved model predictions obtained. (Mallela, et al, 2009). The calibrated MEPDG has been used on many major projects including design/build projects.

- The Mississippi DOT is conducting a major implementation of the MEPDG. Extensive materials testing (HMA, PCC, unbound) has been conducted and local calibration is underway.

- Through a research contract with North Carolina State University, North Carolina DOT has developed local calibration factors for MEPDG flexible pavement designs (Kim et al, 2011). Earlier, it developed an implementation plan containing detailed recommendations for the steps necessary for the local calibration and validation of the MEPDG procedures.

- Ohio DOT has developed a comprehensive roadmap for the MEPDG implementation that included an assessment of Ohio’s needs for the implementation effort, established default values (means and ranges) for those inputs that have adequate data from previous research, and validated nationally calibrated distress and smoothness prediction model using readily available state-specific pavement section data (Mallela et al 2009).

- The Oregon DOT is in the process of implementing the MEPDG for new pavement sections. Forty-four pavement sections throughout Oregon were included for calibration. Oregon initially used some of those sites to verify the global calibration coefficients. Results from the verification study found the MEPDG predicted distresses were significantly different than the measured distresses. Thus, a local calibration of the transfer functions was initiated. Four distress prediction models (rutting, alligator, longitudinal, and thermal cracking) of the HMA overlays were calibrated for Oregon conditions (Williams and Shaidur).

- Virginia DOT is currently performing local calibration of MEPDG performance models. Virginia started with training its district personnel on the use of the MEPDG. It has been using the MEPDG as a shadow design procedure to AASHTO 1993 until full implementation of MEPDG is completed (Elfino, 2012).

- Vermont DOT is currently pursuing an in-house study evaluating the MEPDG rutting and IRI predictions with observed field data for possible recalibration (Contract / Grant Number: SPR 711).

- WisDOT initiated two implementation projects. The first generation implementation project included LTPP sections only. Under this effort, WisDOT developed default data libraries for HMA, concrete, unbound materials and subgrade and traffic. Verification and local calibration of MEPDG performance models using limited LTPP sections were conducted as the next step. WisDOT then initiated a second MEPDG implementation project in 2009 based on review of a much wider dataset of statewide projects to generate a more robust set of local calibration factors (WisDOT, 2010).
Li et al (2009) presented the Washington State DOT’s latest efforts on calibrating the flexible and rigid pavement portion of the MEPDG with data obtained from the Washington State Pavement Management System.

Minnesota DOT has conducted a comprehensive evaluation of the MEPDG performance predictions (Yut et al, 2007). The cracking model was re-calibrated using the design and performance data for 65 pavement sections located in Minnesota, Iowa, Wisconsin, and Illinois. A prototype of the catalog of recommended design features for Minnesota low volume PCC pavements was developed using the MEPDG version 0.910. The catalog offers a variety of feasible design alternatives (PCC and base thickness, joint spacing and PCC slab width, edge support type, and dowel diameter) for a given combination of site conditions (traffic, location, and subgrade type).

Montana DOT was the first agency to develop performance criteria (e.g., ride quality, rutting, fatigue cracking, transverse cracking) of flexible pavements, and used these characteristics in the verification and calibration of the distress prediction models included in the ME Design software using version 0.9000 (Von Quintus and Moulthrop, 2007). The work conducted within this study included using the ME Design software to develop local calibration factors in the use of that software for Montana climate, structures, and materials for flexible pavements. This study developed a reference manual that included selection of distress prediction models, traffic characterization and analyses, and a database for calibration of distress prediction models. In addition, a calibration and user’s Guide for making future refinements to either the regional or local MDT calibration factors was prepared.

Utah DOT has completed a major local calibration effort to enable the agency to implement the MEPDG in routine or day-to-day design practice (Darter et al, 2009). In this study, the nationally calibrated MEPDG distress and smoothness prediction models were validated and calibrated for Utah’s local conditions using the LTPP and UDOT PMS data by ARA, Inc. This study also suggested modifications to some UDOT standard procedures and pavement design protocols such as lab testing procedures, equipment, traffic data reporting, software issues and design output interpretation. The NCHRP 20-07 calibration coefficients for JPCP cracking, faulting, and IRI using the new AASHTO T336 test for coefficient of thermal expansion were verified. A comprehensive User’s Guide was prepared. (Darter, et al, 2009)

Wyoming DOT is currently pursuing the implementation of the MEPDG. The study being conducted by ARA, Inc. will also prepare a design manual/user's guide of recommended procedures for the agency use (Contract Number: RS03(209))

In summary, tables 4 and 5 present the national and local calibration coefficients developed by various States for flexible and rigid pavements, respectively. The remainder of this section provides a discussion and comparison of results from the studies completed to date. This summary will be used in judging the results from the GDOT verification study using the LTPP sites located in Georgia under Task 2 of Work Order #1.

Comparison of Results

Rut Depth Transfer Function

The MEPDG has been found to over predict the rutting in the unbound layers in several States. The over prediction of rut depths in the unbound layers was confirmed for projects where forensic investigations
were conducted to measure the rutting in the unbound and HMA layers. In fact, the rutting predicted in the subgrade was greater than the measured rutting at the surface for more than just a few of the test sections. As such, local adjustment or calibration values were determined for the unbound layers and are summarized in Table 6 for the different projects. These values were determined by limiting the rutting in the unbound layers to the values reported from the forensic investigations. For the most part, Table 4 for the fine and coarse-grained materials/soils are within the same range of values summarized in Table 2.

Most of the unbound material local calibration values are less than 1.0 (the global value), with the exception for the condition where construction anomalies occurred; high water contents and low densities (refer to Table 6). The unbound material local calibration values for the Montana and northwest sections (located in states adjacent to Montana) were found to be lower than the mid-west sections—probably because most of these have heavier truck traffic and thicker HMA layers. The unbound layers in the northwest sites were also found to have lower water contents in the subgrade soils over time.

HMA mixture specific factors, documented under NCHRP Project 1-40B, were used to modify or adjust the MEPDG global calibration factors where sufficient data was available. These projects included all test sections located in Montana and selected SPS-1 and SPS-5 projects. Mixture specific calibration parameters were not used with the Kansas pavement management segments for demonstrating use of the local calibration guide, because insufficient mixture data were unavailable for those segments. As summarized in Table 6, a significant difference exists between the local calibration values for the Montana and Kansas examples, especially for $\beta_{c1}$ and $\beta_{c3}$. The values for the local calibration coefficient for the exponent of the number of load repetitions terms, $B_{c3}$, summarized in Table 4 are slightly different than those values summarized in Table 2. The similarity between the two tables, however, is all studies to date found non-unity for at least one of the local calibration coefficients for the HMA rutting transfer function.

Table 7 summarizes the diagnostic statistics (bias and standard error) for each of the facilities and types of experiments, while figure 7 shows a comparison of the predicted and measured rut depths for all data sets and projects. No systematic difference in the standard error, bias, and other statistics was found between the different experiments. This suggests that the MEPDG rut depth transfer function and model adequately account for many different factors; including HMA volumetric properties, HMA layer thickness, truck loading condition, and climate. Specifically, the MEPDG rut depth prediction model with the NCHRP 1-40B mixture adjustments adequately accounted for differences between the asphalt grades and HMA mixtures – fine to coarse graded aggregate blends from well compacted to poorly compacted mixtures. These data include results from different wheel loads, test temperatures, and mixtures.

More importantly, previous studies have found significant differences between the test results from APT simulated truck traffic, APT full-scale truck traffic, and actual roadway sections with mixed truck traffic. A question that has been continually raised by industry is how to combine the results from different APT experiments – simulated truck loads to full-scale truck loads. These significant differences have been adequately accounted for or normalized through the MEPDG rut depth prediction model.
Table 4. Local Calibration Factors for Flexible Pavements

<table>
<thead>
<tr>
<th>Distress</th>
<th>Coefficient</th>
<th>National</th>
<th>MO</th>
<th>UT</th>
<th>AZ</th>
<th>CO**</th>
<th>WY**</th>
<th>WI</th>
<th>OH</th>
<th>OR</th>
<th>WA</th>
<th>Midwest</th>
<th>NC</th>
</tr>
</thead>
<tbody>
<tr>
<td>AC Fatigue</td>
<td>Bf1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>249.0087</td>
<td>130.3674</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>-3.3</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Bf2</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>-40</td>
<td>1.2</td>
</tr>
<tr>
<td></td>
<td>Bf3</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1.2334</td>
<td>1.217799</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>20</td>
<td>1.5</td>
<td></td>
</tr>
<tr>
<td>AC Bottom Up Cracking</td>
<td>c1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>0.07</td>
<td>0.4951</td>
<td>1</td>
<td>1</td>
<td>0.56</td>
<td>1</td>
<td>0.4372</td>
<td></td>
</tr>
<tr>
<td></td>
<td>c2</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>4.5</td>
<td>2.35</td>
<td>1.469</td>
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<td>1</td>
<td>0.225</td>
<td>0</td>
<td>0.1505</td>
<td></td>
</tr>
<tr>
<td></td>
<td>c3</td>
<td>6000</td>
<td>6000</td>
<td>6000</td>
<td>6000</td>
<td>6000</td>
<td>6000</td>
<td>6000</td>
<td>6000</td>
<td>6000</td>
<td>6000</td>
<td>~0</td>
<td></td>
</tr>
<tr>
<td>AC Rutting</td>
<td>Br1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1.07</td>
<td>0.56</td>
<td>0.69</td>
<td>1.34</td>
<td>1.0896</td>
<td>1.0157</td>
<td>0.51</td>
<td>1.48</td>
<td>0.6</td>
</tr>
<tr>
<td></td>
<td>Br2</td>
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<td>1</td>
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<td>20.6</td>
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</tr>
<tr>
<td></td>
<td>Br3</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>0.9</td>
<td>8.9</td>
<td>1</td>
</tr>
<tr>
<td>Base Rutting; Coarse-Grained Materials/Soils</td>
<td>Bs1</td>
<td>1</td>
<td>0.01</td>
<td>0.604</td>
<td>0.14</td>
<td>0.4</td>
<td>0.9475</td>
<td>0.01</td>
<td>0.32</td>
<td>0</td>
<td>0.7785</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Subgrade Rutting; Fine-Grained Materials/Soils</td>
<td>Bs1</td>
<td>1</td>
<td>0.4375</td>
<td>0.4</td>
<td>0.37</td>
<td>0.84</td>
<td>0.6897</td>
<td>0.5731</td>
<td>0.33</td>
<td>0</td>
<td>0.6616</td>
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<td></td>
</tr>
<tr>
<td>Thermal Fracture</td>
<td>Level 1 K</td>
<td>1.5</td>
<td>0.625</td>
<td>1.5</td>
<td>1.5</td>
<td>7.5</td>
<td>7.5</td>
<td>0.625</td>
<td>1.5</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td></td>
<td>Level 2 K</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Level 3 K</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>7.5</td>
<td>0.3</td>
<td>1.5</td>
<td>10</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>IRI</td>
<td>C1 (Rutting)</td>
<td>40</td>
<td>17.7</td>
<td>40</td>
<td>1.2281</td>
<td>35</td>
<td>20.53</td>
<td>18.71</td>
<td>17.6</td>
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</tr>
<tr>
<td></td>
<td>C2 (Fatigue)</td>
<td>0.4</td>
<td>0.975</td>
<td>0.4</td>
<td>0.1175</td>
<td>0.3</td>
<td>0.4094</td>
<td>0.04</td>
<td>1.37</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C3 (Thermal)</td>
<td>0.008</td>
<td>0.008</td>
<td>0.008</td>
<td>0.008</td>
<td>0.02</td>
<td>0.00179</td>
<td>0.085</td>
<td>0.01</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C4 (Site Factor)</td>
<td>0.015</td>
<td>0.01</td>
<td>0.015</td>
<td>0.028</td>
<td>0.019</td>
<td>0.015</td>
<td>0.0197</td>
<td>0.066</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reflection Cracking (AC over AC only)</td>
<td>C</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>2.55</td>
<td>2.5489</td>
<td>0.75</td>
<td>1</td>
<td>1</td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td></td>
<td>D</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1.23</td>
<td>1.2341</td>
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<td>1</td>
<td>1</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notes:
** Local calibration coefficients for Colorado and Wyoming are not final estimates and are subject to revisions.
Table 5. Local Calibration Factors for Rigid Pavements

<table>
<thead>
<tr>
<th>Distress</th>
<th>Coefficients</th>
<th>National</th>
<th>NCHRP 20-07 Task 288*</th>
<th>MO*</th>
<th>UT*</th>
<th>AZ*</th>
<th>CO*+</th>
<th>WY*+</th>
<th>WI*</th>
<th>OH*</th>
</tr>
</thead>
<tbody>
<tr>
<td>JPCP Transverse or Fatigue Cracking</td>
<td>C1</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
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<tr>
<td></td>
<td>C2</td>
<td>1.22</td>
<td>1.22</td>
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<td>1.22</td>
<td>1.22</td>
<td>1.22</td>
<td>1.22</td>
<td>1.22</td>
<td>1.22</td>
</tr>
<tr>
<td></td>
<td>C4</td>
<td>1</td>
<td>0.6</td>
<td>1</td>
<td>0.6</td>
<td>0.19</td>
<td>1</td>
<td>0.6</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>C5</td>
<td>-1.98</td>
<td>-2.05</td>
<td>-1.98</td>
<td>-2.05</td>
<td>-2.067</td>
<td>-1.98</td>
<td>-2.05</td>
<td>-1.98</td>
<td>-1.98</td>
</tr>
<tr>
<td>Joint Faulting</td>
<td>C1</td>
<td>1.0184</td>
<td>0.5104</td>
<td>1.0184</td>
<td>0.0355</td>
<td>1.0184</td>
<td>0.5104</td>
<td>1.15</td>
<td>1.0184</td>
<td></td>
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<tr>
<td></td>
<td>C2</td>
<td>0.91656</td>
<td>0.00838</td>
<td>0.91656</td>
<td>0.1147</td>
<td>0.91656</td>
<td>0.00838</td>
<td>0.91656</td>
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<tr>
<td></td>
<td>C3</td>
<td>0.0021848</td>
<td>0.00147</td>
<td>0.002185</td>
<td>0.00436</td>
<td>0.002185</td>
<td>0.00147</td>
<td>0.004</td>
<td>0.002185</td>
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</tr>
<tr>
<td></td>
<td>C4</td>
<td>0.000883739</td>
<td>0.008345</td>
<td>0.000884</td>
<td>0.000884</td>
<td>1.10E-07</td>
<td>0.000884</td>
<td>0.08345</td>
<td>0.000884</td>
<td>0.000884</td>
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<td></td>
<td>C5</td>
<td>250</td>
<td>5999</td>
<td>250</td>
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<td>250</td>
</tr>
<tr>
<td></td>
<td>C6</td>
<td>0.4</td>
<td>0.8404</td>
<td>0.4</td>
<td>0.4</td>
<td>2.0389</td>
<td>0.4</td>
<td>0.504</td>
<td>0.4</td>
<td>0.4</td>
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<tr>
<td></td>
<td>C7</td>
<td>1.83312</td>
<td>5.9293</td>
<td>1.83312</td>
<td>0.189</td>
<td>1.83312</td>
<td>5.9293</td>
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<td>400</td>
<td>400</td>
<td>400</td>
<td>400</td>
<td>400</td>
</tr>
<tr>
<td>IRI</td>
<td>C1 (Cracks)</td>
<td>0.8203</td>
<td>0.8203</td>
<td>0.82</td>
<td>0.8203</td>
<td>0.6</td>
<td>0.8203</td>
<td>1.7</td>
<td>4.0567</td>
<td>0.82</td>
</tr>
<tr>
<td></td>
<td>C2 (Spall)</td>
<td>0.4417</td>
<td>0.4417</td>
<td>1.17</td>
<td>0.4417</td>
<td>3.48</td>
<td>0.4417</td>
<td>1.32</td>
<td>1.6275</td>
<td>3.7</td>
</tr>
<tr>
<td></td>
<td>C3 (Fault)</td>
<td>1.4929</td>
<td>1.4929</td>
<td>1.43</td>
<td>1.4929</td>
<td>1.22</td>
<td>1.4929</td>
<td>1.8</td>
<td>0.7236</td>
<td>1.711</td>
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<tr>
<td></td>
<td>C4 (Site Factor)</td>
<td>25.24</td>
<td>25.24</td>
<td>66.8</td>
<td>25.24</td>
<td>45.2</td>
<td>25.24</td>
<td>35</td>
<td>45.2388</td>
<td>5.703</td>
</tr>
</tbody>
</table>

Notes:
* Local calibration coefficients for Colorado and Wyoming are not final estimates and may subject to revisions.
* National calibration coefficients were adjusted to correct for previous errors in the measurement of CTE under the NCHRP project 20-07 Task 288. Proper CTE input is that measured by AASHTO T336. The NCHRP 20-07 derived calibration coefficients using correct CTE values were found to be adequate for Arizona, Colorado, Utah, and Wyoming.
+ National calibration coefficients presented in the AASHTO software (not corrected for errors in CTE measurements) were used in the local calibration for Missouri, Ohio and Wisconsin. Utah was originally calibrated using the National coefficients but later the NCHRP 20-07 coefficients were found to be unbiased and are now used as CTE inputs from the AASHTO T336 test can be used directly.
Table 6. Summary of Local Calibration Values for the Rut Depth Transfer Function

<table>
<thead>
<tr>
<th>Project Identification</th>
<th>Unbound Materials/Soils, $\beta_{s1}$</th>
<th>HMA Calibration Values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Fine-Grained</td>
<td>Coarse-Grained</td>
</tr>
<tr>
<td>NCHRP Projects 9-30 &amp; 1-40B; Verification Studies, Version 0.900 of the MEPDG.</td>
<td>0.30</td>
<td>0.30</td>
</tr>
<tr>
<td>Montana DOT; Based on version 0.900 of the MEPDG</td>
<td>0.30</td>
<td>0.30</td>
</tr>
<tr>
<td>Kansas DOT; PM Segments; HMA Overlay Projects; All Mixtures (Version 1.0)</td>
<td>0.50</td>
<td>0.50</td>
</tr>
<tr>
<td>Kansas PM Segments; New Construction</td>
<td>0.50</td>
<td>0.50</td>
</tr>
<tr>
<td>Kansas DOT; PM Segments; New Construction</td>
<td>Convent. Superpave</td>
<td>0.50</td>
</tr>
<tr>
<td>LTPP SPS-1 &amp; SPS-5 Projects built in accordance with specification; conventional HMA mixtures (Version 1.0).</td>
<td>0.50</td>
<td>0.50</td>
</tr>
<tr>
<td>LTPP SPS-1 Projects with anomalies or construction difficulties, unbound layers.</td>
<td>0.50 to 1.25</td>
<td>0.50 to 3.0</td>
</tr>
<tr>
<td>NCHRP Project 9-30A; based on extensive laboratory tests; Mixture Specific</td>
<td>0.3 to 0.75</td>
<td>0.3 to 0.75</td>
</tr>
</tbody>
</table>

Table 7. Summary of the Bias and Standard Error for the Rut Depth Transfer Function from Independent Data Sets

<table>
<thead>
<tr>
<th>Facility/Project Identification</th>
<th>No. of Points</th>
<th>Bias, in.</th>
<th>Standard Error, in.</th>
<th>$R^2$ Term</th>
<th>$s_e/s_y$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Global Calibration Statistics</td>
<td>334</td>
<td>0.107</td>
<td>0.577</td>
<td>0.818</td>
<td></td>
</tr>
<tr>
<td>FHWA ALF – Simulated Loading APT</td>
<td>28</td>
<td>+0.053</td>
<td>0.1830</td>
<td>0.769</td>
<td>0.57</td>
</tr>
<tr>
<td>Florida ALF – Simulated Loading APT</td>
<td>40</td>
<td>+0.085</td>
<td>0.0945</td>
<td>0.524</td>
<td>0.58</td>
</tr>
<tr>
<td>GTI ALF – Simulated Loading APT</td>
<td>8</td>
<td>+0.146</td>
<td>0.1643</td>
<td>Poor</td>
<td>1.99</td>
</tr>
<tr>
<td>NCAT Test Track; Full Scale APT, Round 1</td>
<td>28</td>
<td>-0.001</td>
<td>0.0377</td>
<td>0.857</td>
<td>0.41</td>
</tr>
<tr>
<td>NCAT Test Track; Full Scale APT, Round 2</td>
<td>24</td>
<td>+0.062</td>
<td>0.0434</td>
<td>0.978</td>
<td>0.41</td>
</tr>
<tr>
<td>WesTrack; Full Scale APT</td>
<td>76</td>
<td>+0.142</td>
<td>0.0844</td>
<td>0.900</td>
<td>0.34</td>
</tr>
<tr>
<td>MnRoads Test Facility – Mixed Traffic</td>
<td>60</td>
<td>+0.038</td>
<td>0.0652</td>
<td>0.791</td>
<td>0.42</td>
</tr>
<tr>
<td>SPS-1 Projects; General; Mixed Traffic</td>
<td>108</td>
<td>-0.0178</td>
<td>0.1339</td>
<td>0.673</td>
<td>0.61</td>
</tr>
<tr>
<td>HMA Overlay Experiments; Mixed Traffic</td>
<td>46</td>
<td>+0.062</td>
<td>0.0426</td>
<td>0.673</td>
<td>0.31</td>
</tr>
</tbody>
</table>
Table 7. Summary of the Bias and Standard Error for the Rut Depth Transfer Function from Independent Data Sets

<table>
<thead>
<tr>
<th>Facility/Project Identification</th>
<th>No. of Points</th>
<th>Bias, in.</th>
<th>Standard Error, in.</th>
<th>$R^2$ Term</th>
<th>$s_e/s_y$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Montana DOT</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>New Construction, Conventional</td>
<td>67</td>
<td>+0.0069</td>
<td>0.0536</td>
<td>0.888</td>
<td>0.342</td>
</tr>
<tr>
<td>New Construction, Semi-Rigid</td>
<td>18</td>
<td>-0.0103</td>
<td>0.0457</td>
<td>0.664</td>
<td>0.662</td>
</tr>
<tr>
<td>HMA Overlays</td>
<td>50</td>
<td>+0.0126</td>
<td>0.0520</td>
<td>0.873</td>
<td>0.359</td>
</tr>
<tr>
<td>Northwest, Adjacent to Montana; Project Sites</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Conventional</td>
<td>72</td>
<td>+0.0108</td>
<td>0.0539</td>
<td>0.763</td>
<td>0.418</td>
</tr>
<tr>
<td>Semi-Rigid</td>
<td>32</td>
<td>-0.0023</td>
<td>0.0472</td>
<td>0.866</td>
<td>0.384</td>
</tr>
<tr>
<td>HMA Overlays</td>
<td>75</td>
<td>+0.0059</td>
<td>0.0501</td>
<td>0.704</td>
<td>0.499</td>
</tr>
<tr>
<td>Kansas DOT</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>PM Segments; Full-Depth Projects</td>
<td>77</td>
<td>+0.0249</td>
<td>0.00397</td>
<td>0.650</td>
<td>0.522</td>
</tr>
<tr>
<td>PM Segments; HMA Overlays</td>
<td>35</td>
<td>+0.0278</td>
<td>0.0725</td>
<td>Poor</td>
<td>0.841</td>
</tr>
<tr>
<td>Mid-West; Project Sites</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>LTPP SPS-1 Sites</td>
<td>122</td>
<td>+0.028</td>
<td>0.134</td>
<td>0.676</td>
<td>0.640</td>
</tr>
<tr>
<td>LTPP SPS-5 Sites</td>
<td>158</td>
<td>-0.031</td>
<td>0.0642</td>
<td>0.357</td>
<td>0.768</td>
</tr>
</tbody>
</table>

In summary, the prediction model is believed to be a reasonable simulation of the experiments and is reasonably accurate in predicting rutting over a diverse range of site conditions, wheel or truck loads, design features, volumetric properties, and HMA mixtures. Using local calibration values that are dependent on the HMA volumetric properties provide a closer estimate to the measured values. Thus, it is concluded that the majority of the error (difference between the measured and predicted HMA rut depths) is a result of distress measurement error and not a lack-of-fit modeling error.

**Fatigue Cracking Transfer Function for Flexible Pavements and HMA Overlays – Alligator Cracking or Bottom-Up Fatigue Cracking**

The HMA mixture adjustment factors documented under NCHRP Project 1-40B were used to modify the MEPDG global calibration parameters where sufficient data was available. The projects where the mixture adjustment factors were and were not used was the same as for rutting. Table 8 lists the overall average values or range of values that were determined for the different data sets. As summarized, the local calibration values between the Montana and Kansas sites are different. The $\beta_f$ values for the Kansas sites are so low, however, the data become questionable. In other words: Did the reported area fatigue cracking actually initiate at the surface?

Unlike for rutting, the local calibration coefficients for fatigue cracking are highly variable between the different studies summarized in Table 2 and 6, with the largest variability occurring for the fatigue cracking coefficient of $B_f$. Currently, it is unclear whether or how the results from previous local calibration studies will be used for the Georgia flexible pavements, because of the diverse difference in results.
Table 9 summarizes the diagnostic statistics (bias, standard error, and RMSE) for each of the facilities and types of experiments for area fatigue cracking. Figure 8 shows a comparison of the predicted and measured fatigue cracking for all data sets from those sections with fatigue cracking—assumed to be bottom-up cracking. In summary, it is believed that the area fatigue cracking transfer function provides a reasonable estimate of fatigue cracking.

It should be noted that the test sections with longitudinal cracking in the wheel paths measured for the MnRoads and FHWA APT experiments are included in Table 9. The length of longitudinal cracking was converted to an area basis – assuming that a longitudinal crack affects the mixture response within 6 inches either side of the crack. As summarized, the correlation between the predicted and measured values is very poor.

The experiments or facilities with the greatest $\frac{s_e}{s_y}$ and greater bias are those with longitudinal cracking. This suggests that the area fatigue cracking prediction model for bottom-up cracking does not accurately predict the occurrence of these longitudinal cracks that initiate at the surface of the HMA layers. Even excluding those sites, with longitudinal cracking, the fatigue cracking transfer function has much less precision for the Kansas sections (including both the pavement management segments and SPS-1 and SPS-5 projects), than for the Montana and northwest project sites. The major difference between these two data sets is that many of the Kansas and mid-west sites have low levels of fatigue cracking and many of those sites with higher levels of fatigue cracking have very high asphalt viscosities. This observation
suggests a bias in the HMA mixture properties between the two data sets, as well as a difference between pavement preservation strategies.

Table 8. Summary of Local Calibration Values for the Area Fatigue Cracking Transfer Function

<table>
<thead>
<tr>
<th>Project Identification</th>
<th>$\beta_{f1}$</th>
<th>$\beta_{f2}$</th>
<th>$\beta_{f3}$</th>
<th>$C_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>NCHRP Projects 9-30 &amp; 1-40B; Verification Studies, Version 0.900 of the MEPDG</td>
<td></td>
<td></td>
<td></td>
<td>Values dependent on the volumetric properties.</td>
</tr>
<tr>
<td>Montana DOT; Based on version 0.900 of the MEPDG, with pavement preservation treatments</td>
<td></td>
<td></td>
<td></td>
<td>0.75 to 10.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.70 to 1.35</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.0 to 3.0</td>
</tr>
<tr>
<td>NorthWest Sites; Located in States Adjacent to Montana, without pavement preservation treatments</td>
<td></td>
<td></td>
<td></td>
<td>Values dependent on the volumetric properties.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.0 to 5.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.0 to 3.0</td>
</tr>
<tr>
<td>Kansas DOT; PM Segments; HMA Overlay Projects; All HMA Mixtures</td>
<td>0.05</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>Kansas DOT; PM Segments; New Construction</td>
<td>Conventional HMA Mixes</td>
<td>0.05</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>PMA</td>
<td>0.005</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>Superpave</td>
<td>0.0005</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>Mid-West Sites</td>
<td>LTPP SPS-1 Projects built in accordance with specifications</td>
<td>0.005</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>LTPP SPS-1 Projects with anomalies or production difficulties</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>LTPP SPS-5 Projects; Debonding between HMA Overlay and Existing Surface</td>
<td>0.005</td>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>

The MEPDG fatigue cracking transfer function was found to be a reasonable estimate of the measured magnitudes over a diverse range of mixtures and structures. The standard error for the area fatigue cracking prediction model was found to be relatively large (less precision) but reasonable for a distress that exhibits high variability measurements. These errors consist of both measurement and lack-of-fit modeling errors. It is believed that the measurement errors are the greater of the two. The lack-of-fit error can be explained by a potential loss of bond between the HMA overlay and existing surface and/or production difficulties identified in the construction reports where the asphalt was severely hardened during production.
### Table 9. Summary of the Bias and Standard Error for the Area Fatigue Cracking Transfer Function from Independent Data Sets

<table>
<thead>
<tr>
<th>Facility/Project Identification</th>
<th>No. of Points</th>
<th>Bias, %</th>
<th>Standard Error, %</th>
<th>$R^2$ Term</th>
<th>$s_e/s_y$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Global Calibration Statistics</td>
<td>405</td>
<td>---</td>
<td>5.01</td>
<td>0.275</td>
<td>0.815</td>
</tr>
<tr>
<td>FHWA APT, Simulated Truck Loading</td>
<td>28</td>
<td>+5.85</td>
<td>8.30</td>
<td>Poor</td>
<td>1.42</td>
</tr>
<tr>
<td>WestTrack APT, Full Scale Truck Loading</td>
<td>58</td>
<td>+0.70</td>
<td>9.40</td>
<td>0.893</td>
<td>0.35</td>
</tr>
<tr>
<td>NCAT, Full Scale Truck Loading</td>
<td>24</td>
<td>-1.96</td>
<td>4.726</td>
<td>0.998</td>
<td>0.338</td>
</tr>
<tr>
<td>MnRoads, Mixed Truck Traffic</td>
<td>60</td>
<td>+2.20</td>
<td>4.75</td>
<td>Poor</td>
<td>3.10</td>
</tr>
<tr>
<td>Roadway Sections, Mixed Truck Traffic</td>
<td>100</td>
<td>-0.98</td>
<td>6.938</td>
<td>0.999</td>
<td>0.53</td>
</tr>
<tr>
<td>Montana DOT</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>New Construction</td>
<td>58</td>
<td>+1.11</td>
<td>2.34</td>
<td>0.573</td>
<td>0.401</td>
</tr>
<tr>
<td>Semi-Rigid Pavements</td>
<td>16</td>
<td>0.00</td>
<td>0.000</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>HMA Overlays</td>
<td>50</td>
<td>-0.02</td>
<td>8.17</td>
<td>0.913</td>
<td>0.318</td>
</tr>
<tr>
<td>Northwest Sites, Adjacent to Montana</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>New Construction</td>
<td>76</td>
<td>+0.15</td>
<td>2.45</td>
<td>0.900</td>
<td>0.315</td>
</tr>
<tr>
<td>Semi-Rigid Pavements</td>
<td>51</td>
<td>+0.51</td>
<td>1.51</td>
<td>0.234</td>
<td>0.532</td>
</tr>
<tr>
<td>HMA Overlays</td>
<td>76</td>
<td>+0.67</td>
<td>7.67</td>
<td>0.877</td>
<td>0.318</td>
</tr>
<tr>
<td>Kansas DOT</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>PM Segments; Full-Depth Projects</td>
<td>177</td>
<td>+0.383</td>
<td>2.154</td>
<td>0.322</td>
<td>1.399</td>
</tr>
<tr>
<td>PM Segments; HMA Overlay Projects</td>
<td>35</td>
<td>+1.272</td>
<td>1.441</td>
<td>Poor</td>
<td>0.806</td>
</tr>
<tr>
<td>Mid-West Project Sites</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>LTPP SPS-1 Sections</td>
<td>122</td>
<td>+1.363</td>
<td>1.433</td>
<td>0.360</td>
<td>0.885</td>
</tr>
<tr>
<td>LTPP SPS-5 Sections</td>
<td>158</td>
<td>-1.150</td>
<td>4.900</td>
<td>0.683</td>
<td>0.890</td>
</tr>
</tbody>
</table>

**Fatigue Cracking Transfer Function for Semi-Rigid Pavements**

Sites were selected in Montana and adjacent States to calibrate the semi-rigid transfer function for fatigue cracking. Unfortunately most of these sites exhibited little load related cracking, as reported in the LTPP database. Only one of the Montana sections had any fatigue cracking recorded on the semi-rigid pavement structures.

The MEPDG-Version 0.900 was used to predict the fatigue cracking of this pavement design strategy by varying the local calibration parameters. These local calibration coefficients were found to be mixture quality dependent, as expected. Based on the data available for regional calibration refinement, the following are the local calibration values for use in predicting the fatigue cracking of semi-rigid pavements (based on version 0.900 of the MEPDG).

- For High Strength CAM Mixtures (intact cores can be recovered and mixture has cement contents greater than 6 percent, with compressive strengths generally greater than 1,000 psi):
  - $B_{c1} = 0.85$.
  - $B_{c2} = 1.10$. 

---

**Table 9. Summary of the Bias and Standard Error for the Area Fatigue Cracking Transfer Function from Independent Data Sets**

<table>
<thead>
<tr>
<th>Facility/Project Identification</th>
<th>No. of Points</th>
<th>Bias, %</th>
<th>Standard Error, %</th>
<th>$R^2$ Term</th>
<th>$s_e/s_y$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Global Calibration Statistics</td>
<td>405</td>
<td>---</td>
<td>5.01</td>
<td>0.275</td>
<td>0.815</td>
</tr>
<tr>
<td>FHWA APT, Simulated Truck Loading</td>
<td>28</td>
<td>+5.85</td>
<td>8.30</td>
<td>Poor</td>
<td>1.42</td>
</tr>
<tr>
<td>WestTrack APT, Full Scale Truck Loading</td>
<td>58</td>
<td>+0.70</td>
<td>9.40</td>
<td>0.893</td>
<td>0.35</td>
</tr>
<tr>
<td>NCAT, Full Scale Truck Loading</td>
<td>24</td>
<td>-1.96</td>
<td>4.726</td>
<td>0.998</td>
<td>0.338</td>
</tr>
<tr>
<td>MnRoads, Mixed Truck Traffic</td>
<td>60</td>
<td>+2.20</td>
<td>4.75</td>
<td>Poor</td>
<td>3.10</td>
</tr>
<tr>
<td>Roadway Sections, Mixed Truck Traffic</td>
<td>100</td>
<td>-0.98</td>
<td>6.938</td>
<td>0.999</td>
<td>0.53</td>
</tr>
<tr>
<td>Montana DOT</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>New Construction</td>
<td>58</td>
<td>+1.11</td>
<td>2.34</td>
<td>0.573</td>
<td>0.401</td>
</tr>
<tr>
<td>Semi-Rigid Pavements</td>
<td>16</td>
<td>0.00</td>
<td>0.000</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>HMA Overlays</td>
<td>50</td>
<td>-0.02</td>
<td>8.17</td>
<td>0.913</td>
<td>0.318</td>
</tr>
<tr>
<td>Northwest Sites, Adjacent to Montana</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>New Construction</td>
<td>76</td>
<td>+0.15</td>
<td>2.45</td>
<td>0.900</td>
<td>0.315</td>
</tr>
<tr>
<td>Semi-Rigid Pavements</td>
<td>51</td>
<td>+0.51</td>
<td>1.51</td>
<td>0.234</td>
<td>0.532</td>
</tr>
<tr>
<td>HMA Overlays</td>
<td>76</td>
<td>+0.67</td>
<td>7.67</td>
<td>0.877</td>
<td>0.318</td>
</tr>
<tr>
<td>Kansas DOT</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>PM Segments; Full-Depth Projects</td>
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<td>2.154</td>
<td>0.322</td>
<td>1.399</td>
</tr>
<tr>
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<td>1.441</td>
<td>Poor</td>
<td>0.806</td>
</tr>
<tr>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>LTPP SPS-1 Sections</td>
<td>122</td>
<td>+1.363</td>
<td>1.433</td>
<td>0.360</td>
<td>0.885</td>
</tr>
<tr>
<td>LTPP SPS-5 Sections</td>
<td>158</td>
<td>-1.150</td>
<td>4.900</td>
<td>0.683</td>
<td>0.890</td>
</tr>
</tbody>
</table>
For CAM Mixtures with moderate levels of cement (intact cores can be recovered and mixture has cement contents greater than 4 percent but less than 6 percent, with compressive strengths generally greater than 300 psi but less than 1,000 psi):
  o $B_{c1} = 0.75$.
  o $B_{c2} = 1.10$.

For Low Strength CAM Mixtures (intact cores cannot be recovered and mixture has cement contents generally less than 4 percent, with compressive strengths less than 300 psi):
  o $B_{c1} = 0.65$.
  o $B_{c2} = 1.10$.

Figure 8. Comparison of the measured and predicted fatigue cracking for all fatigue cracking data combined, excluding those sections with longitudinal cracking.

*Fatigue Cracking Transfer Function for Longitudinal Cracking or Top-Down Cracking*

Longitudinal cracking in the wheel paths were calculated for the different test sections and found to be inaccurate for test sections located in Montana and Kansas, as well as those built in adjacent States. The longitudinal cracking predicted for the test sections was found to be significantly greater for some test sections and significantly lower for others. In fact, significant lengths of longitudinal cracks were predicted for those sections that have yet to exhibit this type of cracking.

No consistent trend in the predictions could be identified to reduce the bias (improvement in accuracy) of this transfer function. It is believed that there is a significant lack-of-fit modeling error for the occurrence of longitudinal cracks. Thus, the bias (residual error) is considered too large for use in structural design. The top-down fatigue cracking model is not recommended for use in making design decisions until it is further refined based on work completed under NCHRP Project 1-42A.
In summary, no consistent result could be identified to adjust the local calibration factors to improve on the accuracy of the transfer function. It is believed that the transfer function is not using the critical response parameter and material properties that affect the occurrence of cracks that initiate at the HMA surface. This difference is considered a lack-of-fit modeling error. It is believed by the author that more than just some of this type of cracking reported or documented in the LTPP database and in other experiments is a result of construction deficiencies (such as inadequate bond between HMA layers and longitudinal segregation), rather than any HMA mixture property.

The Arizona fatigue local calibration was conducted using substantial data from the State PMS, research sections, and LTPP. The results produced a significantly different calibration curve than the national based curve.

**Transverse (Low Temperature) Cracking Transfer Function**

The MEPDG transverse cracking transfer function was used to calculate the length of thermal cracks for all test sections. In general, the MEPDG over-predicted the length of transverse cracks for all test sections where pavement preservation treatments had been used, even for those sections where indirect tensile strength and creep compliance tests were performed on the HMA. An average local calibration factor of 0.25 was determined from the test sections located in Montana, with pavement preservation treatments applied within a short time period after construction. Conversely, it significantly under predicted the transverse cracking of most sections without any pavement preservation treatment. The local calibration factor was found to be agency dependent for the test sections, without any pavement preservation treatment.

Table 10 lists the overall range of values that were determined from the local calibration effort for the transverse cracking transfer function, while Table 11 summarizes the diagnostic statistics for the different projects or sets of data. These values generally cover the same range of values summarized in Table 2, but were found to be highly mixture and/or production specific. Figure 9 compares the predicted and measured values of transverse cracking. The MEPDG has a maximum length of thermal cracking of 2,200 ft./mi. Obviously, some of the sites have exhibited much greater lengths of transverse cracking (refer to figure 9). This limit in the software can result in a relatively large bias, which cannot be eliminated.

The MEPDG transfer function with the local calibration factor was found to be reasonable for predicting transverse cracks in HMA pavements and overlays. However, the standard error is relatively large. In summary, it is believed that there are both lack-of-fit and measurement errors in terms of predicting the crack growth with time.

**Smoothness or IRI Regression Equation for Flexible Pavements**

The MEPDG regression equation for predicting smoothness or increasing roughness was developed from a regression analysis of hundreds of test sections included in the LTPP program. This prediction model is not based on mechanistic principles so it can only be revised using regression-based procedures. Table 12 summarizes the diagnostic statistics (bias, standard error, and RMSE) for each of the facilities and types of experiments for IRI. Figure 10 compares the measured and predicted IRI values for all sites. In summary, the IRI regression equation has been found to be adequate, both in terms of accuracy and precision, for all conditions.
### Table 10. Summary of the Local Calibration Values for the Thermal Cracking Transfer Function

<table>
<thead>
<tr>
<th>Project Identification</th>
<th>$\beta_{t1}$</th>
<th>$\beta_{t2}$</th>
<th>$\beta_{t3}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Montana DOT; application of pavement preservation treatments.</td>
<td>---</td>
<td>---</td>
<td>0.25</td>
</tr>
<tr>
<td>Northwest Sites, located in states adjacent to Montana, but without pavement preservation treatments; appears to be agency dependent.</td>
<td>---</td>
<td>---</td>
<td>1.0 to 5.0</td>
</tr>
<tr>
<td>Kansas PM Segments; Full-Depth Projects</td>
<td>PMA</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td></td>
<td>Conventional</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td></td>
<td>Superpave</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>Kansas PMS Segments; HMA Overlay Projects</td>
<td>PMA</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td></td>
<td>Conventional</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td></td>
<td>Superpave</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>LTPP Projects; HMA produced in accordance with specifications</td>
<td>Conventional</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>LTPP Projects; Severely aged asphalt</td>
<td>Conventional</td>
<td>---</td>
<td>---</td>
</tr>
</tbody>
</table>

### Table 11. Summary of the Bias and Standard Error for the Thermal Cracking Transfer Function from Independent Data Sets

<table>
<thead>
<tr>
<th>Facility/Project Identification</th>
<th>No. of Points</th>
<th>Bias, in./mi.</th>
<th>Standard Error, in./mi.</th>
<th>$R^2$ Term</th>
<th>$S_e/S_y$</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Global Calibration Statistics</strong></td>
<td>28</td>
<td>---</td>
<td>---</td>
<td>0.064</td>
<td>---</td>
</tr>
<tr>
<td>Montana DOT; New Construction &amp; Overlay Projects</td>
<td>110</td>
<td>-26.5</td>
<td>353.1</td>
<td>0.763</td>
<td>0.634</td>
</tr>
<tr>
<td>Kansas PM Segments; Full-Depth Pavement Projects</td>
<td>177</td>
<td>-59.4</td>
<td>313.6</td>
<td>0.595</td>
<td>0.829</td>
</tr>
<tr>
<td>Kansas PM Segments; HMA Overlay Projects</td>
<td>35</td>
<td>-43.7</td>
<td>410.2</td>
<td>0.736</td>
<td>1.136</td>
</tr>
<tr>
<td>LTPP SPS-1 Projects</td>
<td>122</td>
<td>+23.53</td>
<td>287.4</td>
<td>0.696</td>
<td>0.583</td>
</tr>
<tr>
<td>LTPP SPS-5 Projects</td>
<td>158</td>
<td>-100.1</td>
<td>606.6</td>
<td>0.639</td>
<td>0.979</td>
</tr>
</tbody>
</table>
Figure 9. Comparison of Predicted and Measured Thermal (Transverse) Cracking

Table 12. Summary of the Bias and Standard Error for the IRI from Independent Data Sets

<table>
<thead>
<tr>
<th>Facility/Project Identification</th>
<th>No. of Points</th>
<th>Bias, in./mi.</th>
<th>Standard Error, in./mi.</th>
<th>R² Term</th>
<th>sₑ/sᵧ</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Global Calibration Statistics</strong></td>
<td>1926</td>
<td>18.9</td>
<td>0.560</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Montana DOT</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>New Construction</td>
<td>110</td>
<td>+0.27</td>
<td>6.08</td>
<td>0.887</td>
<td>0.417</td>
</tr>
<tr>
<td>HMA Overlays</td>
<td>120</td>
<td>-2.65</td>
<td>6.91</td>
<td>0.892</td>
<td>0.352</td>
</tr>
<tr>
<td><strong>Kansas PM Segments; Full-Depth Pavement Projects</strong></td>
<td>177</td>
<td>-2.87</td>
<td>15.0</td>
<td>0.703</td>
<td>0.632</td>
</tr>
<tr>
<td><strong>Kansas PM Segments; HMA Overlay Projects</strong></td>
<td>35</td>
<td>+0.38</td>
<td>14.3</td>
<td>0.402</td>
<td>0.646</td>
</tr>
<tr>
<td><strong>Mid-West Sites</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>LTPP SPS-1 Projects</td>
<td>122</td>
<td>+2.804</td>
<td>14.348</td>
<td>0.668</td>
<td>0.631</td>
</tr>
<tr>
<td>LTPP SPS-5 Projects</td>
<td>158</td>
<td>+4.900</td>
<td>14.077</td>
<td>0.121</td>
<td>0.773</td>
</tr>
</tbody>
</table>

Maximum length of transverse cracks predicted by the MEPDG.
Transverse Fatigue Cracking of JPCP

The local calibration of transverse fatigue cracking of JPCP was performed by Arizona, Colorado, Wyoming, and Utah. The Arizona calibration utilized the correct CTE values as measured by AASHTO T336 testing and Arizona pavement management sections and LTPP sections including the SPS-2 experiment. Findings showed the following results:

- The slab to base (for Lean Concrete Bases or LCB) friction value should be set at 0 months. The specifications used to construct these bases (e.g., smooth LCB surface, two coatings of wax based curing compounds) resulted in immediate breaking of the bond and reduction in interfacial friction. This resulted in rapid cracking of many of these sections. When the MEPDG was set at 0 months friction, their cracking was accurately predicted.

- The local calibration coefficients tested were those developed under NCHRP 20-07 for the correct CTE values of the PCC. The Arizona calibration verified these coefficients and made only small changes in the recommended values. A plot of the calibration curve is given in Figure 11.
Figure 11. Arizona Recalibration of Transverse (fatigue) Cracking Model using Correct CTE Values (all LCB sections have zero months friction).

Other studies performed in Utah, Colorado, and Wyoming (using a combination of LTPP sections from surrounding States) found that the NCHRP 20-07 calibration coefficients along with correct CTE values produce unbiased predictions and was adopted.

**Transverse Joint Faulting of JPCP**

The local calibration of transverse joint faulting of JPCP was performed by Arizona, Colorado, Wyoming, and Utah. The Arizona calibration utilized the correct CTE values as measured by AASHTO T336 testing and Arizona pavement management sections and LTPP sections including the SPS-2 experiment. Findings showed the following results:

- The local calibration coefficients tested were those developed under NCHRP 20-07 for the correct CTE values of the PCC. The Arizona calibration verified these coefficients but made some changes to further minimize the prediction error.

Other studies performed in Utah, Colorado, and Wyoming (using a combination of LTPP sections from surrounding States) found that the NCHRP 20-07 calibration coefficients along with correct CTE values produce unbiased predictions and was adopted.

**IRI of JPCP**

The local calibration of IRI of JPCP was performed by Arizona, Colorado, Wyoming, and Utah. The Arizona local calibration utilized the revised transverse cracking and joint faulting coefficients and Arizona pavement management sections and LTPP sections including the SPS-2 experiment. The IRI revised model coefficients are shown in Table 11 and resulted in a significant reduction of prediction error as shown. The final Arizona predicted versus measured IRI is shown in Figure XX. Similar results
were obtained for other states including Utah, Colorado, and Wyoming (WY used a combination of LTPP sections from surrounding States).

Table XX. Summary of changes in the IRI calibration coefficients for Arizona local calibration.

<table>
<thead>
<tr>
<th>Model Coefficients</th>
<th>Global Calibration Coefficient</th>
<th>Arizona Local Calibration Coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>J1 (Cracking)</td>
<td>0.8203</td>
<td>0.60</td>
</tr>
<tr>
<td>J2 (Spalling)</td>
<td>0.4417</td>
<td>3.48</td>
</tr>
<tr>
<td>J3 (Faulting)</td>
<td>0.4929</td>
<td>1.22</td>
</tr>
<tr>
<td>J4 (Site Factor)</td>
<td>25.24</td>
<td>45.2</td>
</tr>
<tr>
<td>Standard Error of Prediction</td>
<td>24.6 in/mile</td>
<td>9.8 in/mile</td>
</tr>
</tbody>
</table>

Figure XX. Predicted IRI (using recalibrated coefficients) versus measured IRI for Arizona.

This local calibration of IRI for Arizona clearly shows the benefits obtained by reducing the standard error of prediction. This will impact design reliability and construction costs.

**Punchouts and IRI of CRCP**

Arizona made a comparison of the performance of two CRCP sections over 20 years to the MEPDG predictions. Results indicated that the MEPDG global calibration predicted well for these sections. No other local calibration of CRCP punchouts or IRI model have been accomplished to date. The calibration coefficients obtained from the NCHRP 20-07 study using the correct CTE values produced an unbiased punchout prediction model.

**Summary**

A summary of the significant findings from the calibration refinement studies are listed below, as they relate to the MEPDG transfer functions and their predictive capability.

- The local calibration for a State is critically important and can be used to adjust the predicted performance indicators to minimize the over and under prediction bias and reduce prediction error.
• Some of the local calibration parameters are dependent on material properties and material types. Accounting for this effect reduced the standard error or increased the precision of the transfer function.

• The error component that has the greatest effect or impact on the standard error or precision of the transfer function and model is the measurement error of the performance indicator. This error can be observed from plots of cracking, rutting, or faulting over time where significant variations occur for the same section from year to year. Some of these measurement errors are large and until they are reduced, reducing the lack-of-fit modeling error component (lack of model ability to accurately characterize the distress) will have a small effect on the overall standard error of the estimate term.

• The other item found to have a significant effect on the standard error and bias of a transfer function (primarily load related cracking) is construction anomalies (errors in mixture properties or thickness). If not properly identified, there can be a large bias between the predicted and measured values.

• With adequate local calibration, the following transfer functions are considered appropriate for use in day-to-day designs for flexible pavements.
  
  a. Rut depth transfer function.
  b. Area fatigue cracking (bottom initiated) transfer function.
     i. The fatigue cracking transfer function for semi-rigid pavements needs additional confirmation work to support the local calibration values using the latest version of the MEPDG.
  c. Thermal cracking transfer function for cold climates. The transverse cracking transfer function does not have the capability to predict transverse cracks caused by other mechanisms, other than low temperatures. These other mechanisms include: severe aging or hardening, asphalt absorption or shrinkage, and other long term conditions that can occur in hotter climates.
  d. Smoothness or IRI regression equation.

• With adequate local calibration, the following transfer functions are considered appropriate for use in day-to-day designs for rigid pavements.
  
  a. Transverse (fatigue) cracking of JPCP.
  b. Transverse joint faulting of JPCP.
  c. Smoothness or IRI of JPCP.
  d. Edge punchouts of CRCP.
  e. Smoothness or IRI of CRCP.

• No consistent trend in the prediction of LCWP could be identified within these studies to reduce the bias and standard error, and improve on the accuracy and precision of this transfer function. The LCWP prediction model is considered inaccurate for the projects and test sections used in the calibration refinement study. It is believed that there is a significant lack-of-fit modeling error (theoretical assumptions) for the occurrence of LCWP.
7.2 Partial or Full Scale Deployment

Few State agencies have either completed the calibration effort for some or all aspects of the MEPDG. These agencies have either developed design catalogues (e.g. California, Florida and Washington State) or User Guides (e.g. Missouri and Indiana) that provides guidance on input selection and design procedures. Examples include:

- The MEPDG is fully implemented in Indiana since January 1, 2009. Guidance for using MEPDG in routine pavement design is published in the INDOT 2013 Design Manual Chapter 52. (INDOT, 2013). INDOT has documented the cost savings with efficient pavement designs resulting from the implementation of MEPDG (Nantung, 2010).
- Missouri DOT has completed the full-scale research study enabling the agency to facilitate a transition from the current pavement design methodology to MEPDG. This effort resulted in development of a User’s Guide with default data libraries for MEPDG inputs of relevance to local materials, soils and traffic, recommended input levels to be used for different conditions and roadways, performance criteria, reliability levels and calibration factors. This effort also conducted the verification, validation and recalibration of relevant MEPDG models for use in Missouri and provided recommendations for the MEPDG deployment in Missouri (Mallela et al, 2009).
- Utah DOT has completed a major local calibration effort from 2004 to 2009 and along with extensive staff training has allowed the agency to utilize the MEPDG for pavement design. A comprehensive User’s Guide was developed that detailed the selection of inputs, design performance and reliability criteria, and procedures to achieve passing designs. The major calibration adjustment was to eliminate the over prediction of rutting. One aspect of implementation that was discussed was selection of design reliability since the MEPDG generally resulted in thinner HMA and JPCP designs at the same level of reliability.

7.3 Design Reliability and Performance Criteria

Selection of design reliability and design performance criteria was studied by Arizona. A sensitivity analysis was conducted that showed how the resulting design depends on these critical inputs. Selection of too high of reliability and/or performance criteria resulted in unreasonable and costly designs. It was recommended that the selection of reliability and performance criteria should be done together and not independently. These factors can affect the design as much as any other inputs and need to be more fully considered in the implementation effort.

8. SUMMARY

This synthesis report intends to capture the status of current and completed implementation activities by various State agencies. The information compiled in this report will serve as a reference document to GDOT to see what other States are doing with regards to implementation and help prevent avoidable problems experienced by other agencies during their implementation effort. The other intent of the synthesis report is to provide a summary of the results from other agencies calibration efforts in planning the sampling matrix and experimental factorial, if a local calibration for Georgia is believed to be required (the results from Task 2 or Work Order #1).
Most State DOT studies have focused on building data libraries for key material types, and evaluate the ability of lower hierarchical input levels to produce reasonable predictions for the agency-specific material types. Numerous studies have focused on HMA mixtures; the evaluation of Level 3 Witczak dynamic modulus model indicates that the model predictions appeared to be acceptable for mixtures with conventional binder, significant deviations were observed for binders with higher PG grades. Use of measured binder test data (i.e. input level 2) in Witczak model has greatly improved the accuracy of dynamic modulus predictions. Studies on PCC mixtures have particularly emphasized on the CTE measurement and its significance in rigid pavement performance. Although the results are diverse, most studies have concluded the Witczak dynamic modulus regression equation is reasonable.

Most surrounding states, including the Florida, Louisiana, Mississippi, and Virginia, have conducted studies on resilient modulus. These studies when combined with GDOT studies on resilient modulus can be used to provide recommended values for a range of soils and their in place condition for establishing inputs to the ME Design software. The results from all of these studies, as well as from FHWA sponsored studies on resilient modulus, can help provide local default values for GDOT to use in calibration, as well as in design.

The findings of national level studies, including NCHRP 1-40B, NCHRP 9-30A and NCHRP 1-47, are directly applicable to Georgia. More importantly, the lessons learned from various calibration studies are directly applicable to Georgia for use in setting up a sampling matrix to verify, locally calibrate, and validate the MEPDG transfer functions. The following lists some of the more important findings from the literature and projects reviewed under this Task.

1. Selection of design reliability and design performance criteria requires sensitivity analyses to show how the resulting design depends on these critical inputs. Selection of too high of reliability and performance criteria will result in unreasonable and costly designs. Selection of reliability and performance criteria should be done together and not independently.

2. The key findings of the sensitivity analyses conducted under NCHRP 1-47 will be used in the GDOT MEPDG implementation study to select sites for the local calibration and in evaluating the residual error of the predicted distress values.

3. The local calibration factors determined from State calibration studies for PCC pavements are reasonably consistent with the global coefficients. However, several important advantages were obtained through State local calibration for PCC pavements.
   a. More accurate design inputs were established through the local calibration process. For example, the estimate of the number of months of full friction between the slab and base was improved using local data.
   b. The use of the correct CTE input for the PCC (as measured by AASHTO T336) was found to verify the national coefficients determined under NCHRP 20-07 in 2010 for several states. This makes it possible for the State to test the CTE of PCC and then use directly the value in design.
   c. Modifications to some the calibration coefficients were found to be valuable in reducing the standard error of prediction which is used directly in reliability design.
4. The following are some consistent findings from flexible pavement calibration studies:
   a. MEPDG over predicts rutting in the unbound layers based on using laboratory equivalent resilient modulus values.
   b. Dynamic modulus does not explain the different in rutting between HMA and PMA mixtures.

5. The following local calibration coefficients were found to be significantly different between many of the studies reviewed:
   a. $B_{f1}$ for the fatigue cracking transfer function.
   b. $B_{r3}$ (exponent to the number of load cycles term) and $B_{r1}$ (the intercept term) for the HMA rut depth transfer function.
   c. $C_1$ or coefficient of the rutting term in the IRI regression equation.

6. The procedures outlined in the NCHRP 1-40B and NCHRP 9-30A will be used to develop field adjustment factors for fatigue cracking and rutting models.

9. References

Completed Projects, Reports and Papers


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**On-going Research Projects**


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6. Inputs of Portland Cement Concrete Parameters Needed for the Design of New and Rehabilitated Pavements in Mississippi, Contract/Grant No. 177, Sponsored by Mississippi Department of Transportation, Jackson, MS. http://rip.trb.org/browse/dproject.asp?n=10002

7. Instrumented Pavement Construction, Contract/Grant No. SPR Item 2200, Sponsored by Oklahoma Department of Transportation, Oklahoma City, OK. http://rip.trb.org/browse/dproject.asp?n=12549


13. MEPDG Inputs for Warm Mix Asphalts, Contract/Grant No. FHWA/NC/2012-01, Sponsored by North Carolina Department of Transportation, Raleigh, NC. https://apps.dot.state.nc.us/Projects/Research/ProjectInfo.aspx?ID=2721


15. Improved Characterization of Truck Traffic Loading for MDOT Pavement Design, Contract/Grant No. 257, Sponsored by Mississippi Department of Transportation, Jackson, MS. http://rip.trb.org/view/2012/P/1231596

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