Local Calibration of the Mechanistic-Empirical Pavement Design Software for Wisconsin

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to:

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16. Abstract

The Wisconsin Department of Transportation is in the process of implementing the mechanistic-empirical pavement design process. Considerable resources have been invested in local material characterization. An initial implementation study sponsored by WisDOT focused on Long-Term Pavement Performance (LTPP) sites for model verification and local calibration. LTPP sites were used because they had readily available traffic, climate, subgrade, materials, pavement structure and performance data. A more robust local calibration is being undertaken in Wisconsin to fully realize the potential benefits of the mechanistic-empirical design process. Factors affecting the performance of high-type, rural JPCP and HMA pavements were investigated during this study using data contained in the WisDOT pavement information files and the Meta Manager traffic database. Key parameters were identified and a targeted filed study of nine hot mix asphalt (HMA) pavement sections and nine jointed concrete pavement (JCP) sections was implemented to provide data necessary for the local calibration of the Mechanistic Empirical Pavement Design software for high-type rural facilities in Wisconsin. Modifications to the default calibration factors relating to HMA rutting and alligator cracking and JCP slab cracking were shown to provide pavement performance predictions which were in line with observed field performance at the 50% and 90% reliability levels. The results presented in this report were generated using the evaluation version 1.0 of the mechanistic empirical pavement design software. Additional calibration efforts using the recently released DARWin-ME software will be required to extend these results to routine pavement design evaluations.

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Chapter 1 – Introduction

1.1 Background and Problem Statement

The Wisconsin Department of Transportation (WisDOT) is in the process of implementing AASHTO's Mechanistic-Empirical Pavement Design Guide (MEPDG). Considerable resources have been invested in local material characterization. An initial implementation study, conducted by WisDOT, focused on LTPP sites for model verification and local calibration program outputs. LTPP sites were used because they had readily available traffic, climate, subgrade, materials, pavement structure and performance data. However, the initial implementation study did not include Wisconsin-specific pavement projects that would require field and laboratory testing to establish the required MEPDG inputs. A more robust local calibration is necessary if Wisconsin is to fully realize the potential benefits of the MEPDG.

1.2 Research Objectives

The objectives of this research were to develop a database of information on Wisconsin pavements, use that database to examine the MEPDG performance prediction models, and either validate the adequacy of the national calibration factors used in the models, or, to the extent that the national calibration factors do not yield sufficiently adequate predictions, determine new calibration factors that better represent Wisconsin conditions and the performance of Wisconsin's pavements.

1.3 Scope of Work

The work conducted for this study encompassed the following activities:

- Development of an experimental sampling plan that covered different materials, ages, pavement types, and designs, to allow for a Wisconsin calibration of the MEPDG.
- Identification of specific highway sections that filled out the experimental sampling plan.
- Identification of sites for which data collection was feasible within the available budget.
- Collection of all field information required for local calibration such as materials and pavement distresses.
- Performing material characterization as necessary.
- Development of a database of the project information obtained.
- Use of the database developed, together with existing information on Wisconsin LTPP sites, to examine the MEPDG performance prediction models and either validate the adequacy of

the national calibration factors used in the models, or, to the extent that the national calibration factors do not yield sufficiently adequate predictions, determine new calibration factors that better represent Wisconsin conditions and the performance of Wisconsin's pavements.

1.4 Organization of Final Report

This final report is organized as follows:

Chapter 1 – Introduction

Chapter 2 – MEPDG Overview

Chapter 3 – Project Selection and Field Studies

Chapter 4 – Laboratory Testing of Pavement Materials

Chapter 5 – MEPDG Performance Model Calibration and Sensitivity

Chapter 6 – MEPDG Analysis of Wisconsin Sections

Chapter 7 – Conclusions and Recommendations

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Appendix – Annotated Bibliography

Chapter 2 – MEPDG Overview

The Mechanistic-Empirical Pavement Design Guide (MEPDG) is a tool for analyzing designs for new and rehabilitated pavement structures. The MEPDG software does not generate a pavement design for a set of input conditions, nor does it search through a database of feasible designs. Rather, it evaluates a trial design that is input by the user, and predicts the performance of that trial design. Structural responses (stresses, strains, and deflections) of the trial pavement to loading are calculated using multilayer elastic theory or finite element methods, as a function of input material properties, climate conditions, and traffic loading characteristics. Temperature and moisture distributions in the pavement structure are calculated using a computer program known as the Enhanced Integrated Climatic Model. The calculated responses are used to compute incremental damage over time, and empirically relate the cumulative damage to observable pavement distresses.

2.1 Methodology

In a review of the MEPDG conducted by Iowa State University for the Iowa DOT, Coree et al.[1] detailed as follows the steps in the process by which the adequacy of a trial pavement design is assessed using the MEPDG software. The process is illustrated in Figure 2.1 (from AASHTO's *Guide for the Local Calibration of the Mechanistic-Empirical Pavement Design Guide*[2]).

- The temperature and moisture profiles through the pavement are generated for the conditions at time = t.
- The spectrum of traffic loadings in the next time increment (Δt) is defined.
- The elastic properties and the thickness of each layer (E, μ, h) are determined from the initial inputs, the age since construction, the temperature and moisture profiles, and the speed (duration or frequency) of each load.
- A structural analysis is performed to estimate critical stresses and strains within the structure.
- An ancillary analysis is performed to determine the non-load-related stresses and strains (i.e., due to thermal and moisture gradients).
- The load-related and non-load-related critical stresses and strains are combined.
- The incremental changes in performance indicators are computed based on the critical stresses and strains (or their increments). These include the basic set of distresses, such as rutting, faulting, transverse cracking, etc. Incremental changes in IRI as function of incremental changes in distress are also computed. These may be computed based on calibrated deterministic or empirical models.

- Changes in initial material parameters (E, μ) resulting from the computed incremental damage are estimated. For example, if a cement-stabilized layer (e.g., E = 2,400,000 psi) is found to have been overstressed and cracked during this time interval, its effective modulus may be reduced, e.g., to 1,200,000 psi for the ensuing time interval).
- The time scale is incremented to t = ti + Δt, and the cycle is repeated.

2.2 Performance Models

The performance indicators predicted by the MEPDG models are the following.

For hot-mix asphalt (HMA) pavements and overlays:

- Total rut depth and rutting in the HMA, unbound aggregate base, and subgrade.
- Non-load-related transverse cracking.
- Load-related alligator cracking.
- Load-related longitudinal cracking.
- HMA overlay reflection cracking of joints and cracks in existing flexible, semi-rigid, composite, and rigid pavements.
- Smoothness (International Roughness Index, or IRI).

For Portland cement concrete (PCC) pavements and overlays:

- Jointed plain concrete pavement (JPCP) mean joint faulting.
- JPCP load-related transverse slab cracking.
- Continuously reinforced concrete pavement (CRCP) crack spacing and crack width.
- CRCP load transfer efficiency.
- CRCP punchouts.
- JPCP and CRCP smoothness (IRI).

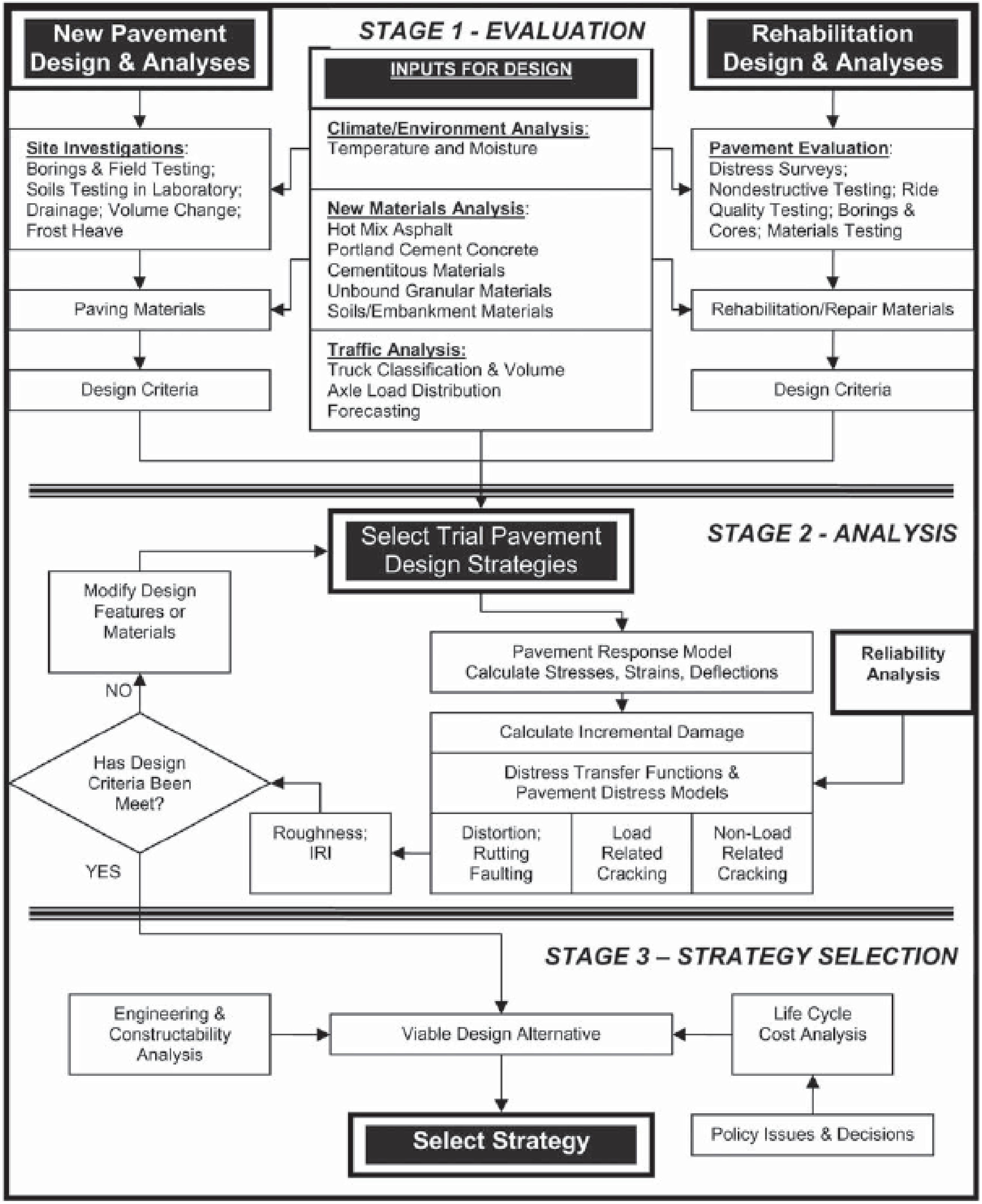

Figure 2.1 Conceptual Flowchart of the MEPDG Design Analysis Process[2]

2.3 Revisions to MEPDG Models and Software

The first version (0.7) of the MEPDG software was developed under National Cooperative Highway Research Program (NCHRP) Project 1-37A. The performance prediction models developed for the MEPDG under NCHRP Project 1-37A are documented in the *Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures*[3], the final report for NCHRP Project 1-37A.

NCHRP subsequently oversaw an independent review of the 1-37A final report and version 0.7 of the MEPDG software, conducted by three teams of consultants, under NCHRP Project 1-40A. One of the consultant teams reviewed the MEPDG's guidelines, procedures, models, and software for new flexible pavement design; a second team reviewed the same for new rigid pavement design. The third team reviewed the MEPDG's guidelines, procedures, models, and software for pavement rehabilitation design, and also reviewed the MEPDG's approach to design reliability. The three teams' findings are presented in References 4, 5, and 6, respectively, and are summarized in NCHRP Research Results Digest 307.[7] Also presented in Digest 307 are the MEPDG development team's responses to the comments from the NCHRP Project 1-40A technical panel and independent review teams, including indications of whether specific recommended changes were or were not planned for future versions of the software.

Version 0.8 of the MEPDG software was released in November 2005, and version 0.9 was released in July 2006. Changes to the MEPDG software through version 0.9 are summarized in NCHRP Research Results Digest 308,[8] which is considered the interim report for NCHRP Project 1-40D. Version 1.0 of the MEPDG software was released in April 2007, and version 1.1 of the MEDPG software was released in September 2009. Installation instructions and a list of changes to the software from version 1.0 to version 1.1 are provided in Reference 9.

NCHRP also oversaw, under Project 1-40B, the development of a local calibration guide and manual of practice for the MEPDG and software. The local calibration guide[2] and manual of practice[10] have been published by AASHTO.

Version 1.1 of the MEPDG software was available for download from the Transportation Research Board website through 30 September 2011. The DARWin-ME software version 2.0, released by AASHTO in April 2011, is based on the models incorporated in version 1.1 of the MEPDG software. Licenses for use of DARWin-ME 2.0 may be purchased from AASHTO.

2.4 Reliability

There is an element of uncertainty associated with the performance predictions obtained from each of these models. Some of this uncertainty is associated with design factors (such as environment, traffic, and materials), some is associated with construction factors (such as equipment, procedures, and ambient conditions), and some is associated with aspects of the predictive models themselves (such as the adequacy of the model forms and the adequacy of the data used for calibration).

In the early stages of the planning and development of the MEPDG, a probabilistic approach using Monte Carlo simulation was envisioned for use in dealing with the uncertainty associated with the many inputs to the performance models. Concerns existed, however, about the computational demands of Monte Carlo simulation and other such methods, and the difficulty of achieving sufficiently precise solutions when dealing with a large number of uncertain inputs.

The approach taken instead to address the variability of the inputs was to calibrate the distress models using data from the national Long-Term Pavement Performance (LTPP) database, compare predicted to actual values, and calculate the standard error of the residuals (differences between the predicted and actual values). This standard error quantifies the spread of the failure probability distribution around the mean predicted value, as illustrated in Figure 2.2 below using cracking as an example. A desired reliability level may thus be specified for each predicted distress type. The reliability of the design with respect to the predicted performance indicator is defined as the probability that the predicted distress (e.g., CRK_{avg} in Figure 2.2) will be less than the critical distress level (e.g., $CRK_{failure}$ in Figure 2.2). The design reliability levels selected may vary by distress, although it is recommended that the same reliability level be used for all.

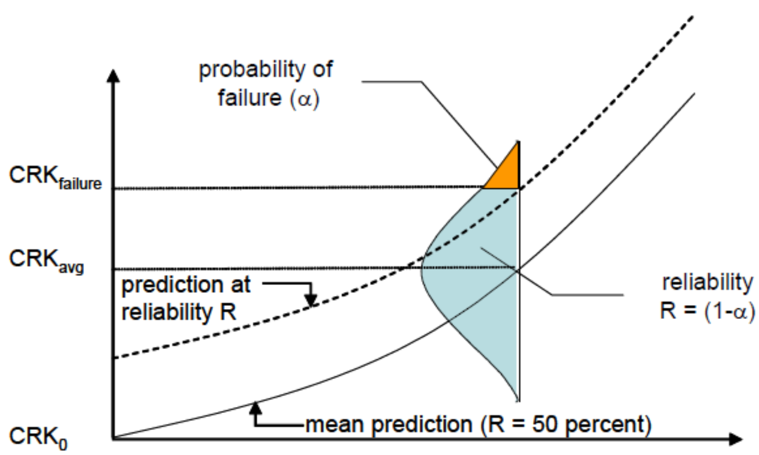

Figure 2.2 Illustration of Reliability Concept for Predicted Cracking[3]

The variability associated with smoothness is handled differently in the MEPDG from the variability associated with each of the distress predictions. The variance of the computed IRI is calculated as the sum of the variances of the initial IRI, the distresses that contribute to the IRI for the pavement type in question, the variances in the site factors that enter into the IRI models, and the variance of the overall model error for the pavement type in question.

2.5 Input Levels

The MEPDG introduces to the field of pavement engineering a formalized hierarchical approach to design. The inputs for a single design analysis do not need to be all at the same level. Input levels can be mixed, depending on the data available. The input levels do not affect in any way the models and procedures used to compute predicted pavement distresses and smoothness. The damage calculations performed by the MEPDG software are exactly the same regardless of the input level or levels used. Inputs may be categorized according to the following levels:

> ***Input Level 1***: Site-specific or project-specific measured input values. This level has the highest data collection and testing costs of the three input levels. According to the MEPDG *Manual of Practice*, Level 1 should be used "for pavement designs having unusual site features, materials, or traffic conditions that are outside the inference space used to develop the correlations and defaults included for input Levels 2 and 3." Level 1 inputs are often described as being appropriate for design of projects with high visibility, high importance, and high costs, as well as for research and forensic purposes.
>
> ***Input Level 2***: Input values estimated from correlations or regression equations. The input values may be estimated from other, easier-to-measure site-specific values, or may be non-project-specific estimates. Level 2 inputs are often described as being appropriate for routine but significant real-world projects.
>
> ***Input Level 3***: Default values used for inputs. Level 3 inputs are often described as being appropriate for routine minor projects.

Presumably, the extra time and effort expended to determine higher-level values for inputs should be justified by the greater precision (smaller standard error) of the performance prediction obtained. As the final report on the development of the MEPDG puts it: "the reliability of a design should logically increase when the level of the engineering effort used to obtain inputs is increased. This would logically lead to a reduction in life-cycle costs." However, this presumption has yet to be validated for any of the MEPDG models except the HMA thermal fracture model, and it remains, again as the MEPDG final report puts it: "very important to illustrate to the engineering community that additional time, effort, and design funding will actually result in lower cost and a longer-performing product."

In reality, it falls to every State highway agency seeking to implement the MEPDG to assess, for its own network of highways, its own range of conditions, and its own pavement data resources, how best to expend its time and effort to determine the various input values for the MEPDG models at the levels most appropriate to (a) the models' sensitivity to the inputs and (b) the importance of the various distress types to the actual performance of pavements in that State.

2.6 Climate

Predicting pavement performance with the MEPDG requires detailed and extensive climatic data, including hourly inputs for temperature, precipitation, wind speed, relative humidity, and cloud cover. The climatic inputs are used to compute temperature and moisture distributions in the pavement layers and foundation, and also enter into the computation of the site parameter in the IRI prediction models. The climatic data needed to analyze a trial pavement design at a particular site are available in the climatic database that is distributed with the MEPDG software. This database contains data from thousands of weather stations around the US, many of them located at airports. Given the latitude and longitude of the site of interest, the MEPDG software will retrieve the climate data from the six weather stations closest to that site, and the user can select which one or more of those weather stations to use. From among the weather stations selected, the software will interpolate the climatic inputs for a "virtual weather station" at the site. If a single weather station is selected, only data from that location will be used.

2.7 Traffic

The types of traffic data required for use with the MEPDG are base-year truck traffic volume, truck traffic volume adjustment factors, axle load distribution factors, and general traffic inputs such as number of axles per truck, axle configuration, and wheel base. Traffic inputs at Level 1 are obtained from site-specific weigh-in-motion (WIM) and automated vehicle classification (AVC) data. Traffic inputs at Level 2 rely on regional WIM and AVC data, and traffic inputs at Level 3 use regional or statewide default values. Some of the MEPDG models–particularly the HMA rutting model and the JPCP cracking model–have been found to be very sensitive to traffic inputs, and all of the models are considered to be sensitive to overloads in the load spectra. For important design situations, characterizing traffic with the highest-level inputs possible is recommended.

2.8 Sensitivity Analysis

An evaluation of the sensitivity of each of the MEPDG's pavement performance models with respect to their inputs, and for a range of traffic levels and climates, was conducted under NCHRP Project 1-47, and published at the end of 2011 as NCHRP Report No. 372, entitled *Sensitivity Evaluation of MEPDG Performance Prediction*.[11]

NCHRP Report No. 372 identifies seven reports of sensitivity analyses of the MEPDG rigid pavement models published between 2004 and 2006, and sixteen reports of sensitivity analysis of the MEDPG flexible pavement models published between 2004 and 2007, all conducted using versions of the MEPDG software prior to Version 1.0, which was released in 2007. The relevance of the findings of these early sensitivity analyses to the current version of the MEPDG software are limited, due to the many changes made to the software and recalibration of the models between 2004 and 2007. More relevant to the current MEPDG software are another eighteen reports on sensitivity analyses of the MEPDG rigid pavement models published between 2007 and 2010, and eleven reports on sensitivity analyses of the MEPDG flexible pavement models published between 2008 and 2010, all of which were conducted using Version 1.0 or the very similar Version 1.1 of the MEPDG software.

The authors of NCHRP Report 372 point out the following additional limitations of many of the MEPDG sensitivity studies conducted between 2004 and 2011, including both those conducted with earlier versions of the MEPDG software and those conducted with the current version of the software:

1. Many of the studies were focused largely on confirming that the predicted pavement performance trends were consistent with expectations based on engineering experience.
2. Past studies typically have involved varying only a small subset of inputs judged subjectively to be most important. It is not always true, however, that the inputs judged subjectively to be most important are in fact most sensitive in the prediction of performance, nor that other inputs not judged to be important are in fact sensitive. Indeed, in the research conducted for NCHRP Project 1-47, initial subjective assessments of the sensitivity of each performance model to its inputs were made, and later found to agree with objectively computed sensitivities only about 65 to 85 percent of the time, depending on the pavement type.
3. Most of the studies employed a methodology in which the values of individual inputs are varied one at time for one or more baseline cases. Depending on the range of baseline cases evaluated and the ranges over which the inputs are varied, this type of approach might not yield a truly global characterization of a model's sensitivity to its inputs.
4. Correlations and interactions among inputs have largely been ignored in past sensitivity studies. Correlations are situations in which a change in one input tends to coincide with a change in another input (e.g., concrete modulus of rupture and modulus of elasticity). Interactions are situations in which simultaneous changes in two or more inputs result in a change in the predicted performance greater than the sum of the changes in predicted performance resulting from changes in the individual inputs. However, while several cases of correlated inputs have significant practical effects on performance predictions, NCHRP Project 1-47 found that interactions among inputs to the MEPDG models were generally not significant.
5. Multivariate linear regression (MLVR) response surface methods (RSM), although widely employed in the sensitivity analysis literature, were found to be inadequate to the task of characterizing sensitivity in MEPDG pavement performance predictions. In the research conducted for NCHRP Project 1-47, MVLR RSMs yielded only poor to fair goodness-of-fit statistics.

While observing that there is no one quantitative metric of sensitivity that is ideal for all variables and all purposes, in the research conducted for NCHRP Project 1-47, it was found useful to employ a normalized sensitivity index (NSI), as well as a mean NSI plus or minus two standard deviations ($NSI\mu\pm2\sigma$) to express the relative sensitivities of inputs to performance predictions. The following categories of MEPDG model input sensitivity were defined based on this latter parameter:

- Hypersensitive (HS): $NSI\mu\pm2\sigma$ greater than 5,
- Very sensitive (VS): $NSI\mu\pm2\sigma$ between 1 and 5,
- Sensitive (S): $NSI\mu\pm2\sigma$ between 0.1 and 1,
- Non-sensitive (NS): $NSI\mu\pm2\sigma$ less than 0.1.

Details of the sensitivity results obtained in NCHRP Project 1-47 for each of the inputs to the MEPDG HMA and JPCP performance models are presented in the chapter of this report on MEPDG model sensitivity. Among the overall conclusions presented in NCHRP Report 372 are the following:

- For all pavement types and distresses, the most sensitive design inputs were those related to the bound surface layers (HMA, PCC).
- The sensitivity values for each combination of distress and design input did not vary substantially or systematically by climatic zone. The magnitudes of predicted distresses may vary by climatic zone, but the sensitivities of the predicted distresses did not.
- For the HMA performance prediction models, only the HMA property inputs (the upper and lower limits of the HMA dynamic modulus master curve, HMA thickness, surface shortwave absorptivity, and Poisson's ratio) were consistently in the highest sensitivity categories. None of the base, subgrade, or other inputs were as consistently in the two highest sensitivity categories.
- The sensitivity values for the inputs to the HMA longitudinal cracking, rutting, and alligator cracking were consistently and substantially higher than those for the inputs to the HMA thermal cracking and IRI models.
- Little or no thermal cracking was predicted in any climate when the correct binder grade for the climate was used with the HMA thermal cracking model. The low-temperature binder grade had to be shifted 2 or 3 grades stiffer (warmer) to generate sufficient predicted thermal cracking distress to quantify the sensitivity of the model inputs.
- For JPCP, slab width was consistently the most sensitive design input, followed by PCC properties (unit weight, coefficient of thermal expansion, strength, stiffness, and surface

shortwave absorptivity), PCC thickness, and other geometric properties (lane width and joint spacing).

- The magnitudes of the sensitivity values for the inputs to JPCP faulting, transverse cracking, and IRI were similar. However, the range of sensitivity values for the inputs to the faulting model was significantly larger than for those to the transverse cracking and IRI models.

NCHRP Report 372 also provides some practical guidance to designers on how to address the sensitivity of the various inputs to the MEPDG models. For example, some highly sensitive inputs, such as PCC thickness, can be specified very precisely. The report advises that the careful characterization of HMA and PCC stiffness and strength properties, including mix-specific laboratory measurement of HMA dynamic modulus or PCC stiffness and strength properties may be appropriate in some cases. However, other highly sensitive material properties, such as the PCC coefficient of thermal expansion, are very difficult to measure, and testing protocols are still evolving. The report also points out that the high sensitivity of distresses predictions to surface shortwave absorptivity for both HMA and PCC pavements is problematic, as this property cannot be readily measured and guidance on realistic values for specific paving materials is lacking. For these as well as other highly sensitive inputs, NCHRP Report 372 recommends project-specific design sensitivity studies to evaluate the consequences of uncertain input values.

2.9 Annotated Bibliography

Hundreds of papers and reports have been written on aspects of the MEPDG's performance predictions and how they compare with field measurements of pavement distress and roughness, and dozens more research studies related to implementing the MEPDG are in progress. A broad selection of references for completed studies and research in progress related to the MEPDG are summarized in an annotated bibliography provided as an appendix to this report. These references are presented in the following categories: sensitivity analysis, traffic, climate, materials, and other topics.

CHAPTER 3 - Database Development and Project Selection

3.1 Databases

Databases from WisDOT were obtained for design, new construction reports, traffic, and performance data. The purpose of collecting these databases was to assemble higher-trafficked doweled jointed plain concrete pavements (JPCP) pavements (Type 8) and hot mix asphalt (HMA) pavements having a flexible base (Type 1), then develop a factorial design to determine the most appropriate projects for calibration. The databases in Table 3.1 were accessed with assistance from the WisDOT Pavement Management Unit.

Table 3.1 Databases Accessed in Study

Database (1)	Description (2)
Meta Manager	This database compiles traffic data and forecasts anticipated traffic levels. Traffic data from each of the 5 WisDOT regions were combined into one dataset with pavement segments. Key data fields obtained were highway number, pavement sequence number, Reference Point (RP), termini of segment, pavement type, functional class, number of lanes, projected AADT for 2009, and percent trucks.
Pavement Inventory Files (PIF).	Descriptions and pavement distress data for each RP segment were obtained, including PDI, IRI, and both extent and severity of individual pavement distresses (slab cracking, rutting, edge raveling, etc.). This database also included highway number, termini description, directional lane of measurement, year of measurement, region number, and county.
New Construction Reports	Attributes of projects constructed in a given year are detailed, including such fields as prime contractor, thickness of PCC or HMA, base type and/or preparation (DGBC, OGBC, milled, pulverized, etc.), lane-miles of paving, and project identification number. This database was used to verify the paving year of RP segments in the Meta Manager and PIF databases.

3.1.1 Meta-Manager Database

Meta-Manager is a comprehensive, integrated database system for conducting needs and performance analyses for pavements and bridges. It is updated and distributed quarterly. It is comprised of independent databases organized by region for all five regions. Each of WisDOT region's data consists of one Excel spreadsheet workbook with multiple datasheets. The workbook datasheets include information on base, roadway, unimproved pavement condition, improved pavement condition, safety, pavement treatment scoping, mobility, unimproved bridge condition, and improved bridge condition. The primary datasheet accessed was 'roadway'. The roadway datasheet provides the most current traffic volume data using sequence numbers, traffic segment identification numbers, and from-and-to reference points. Other relevant fields include highway number by direction, projected 2-way AADT, and percent trucks for 1, 5, 10, 15, and 20-year periods from a base year.

3.1.2 Pavement Inventory Files

The pavement performance database, commonly referred to as PIF, is a relational database model designed to store pavement inventory information, capture distress characteristics, and summarize continuous ride/rutting data. The key datasheets include the descriptive (DESC), pavement distress index (PDI) history file, and International Roughness Index (IRI) data.

Descriptive datasheet identifies pavement segments by sequence numbers, county name, county number, district, from-to reference points, from feature, highway number, highway direction, functional class number, national highway system designation, surface year and original construction year. In addition, the datasheet has fields for the segment length, cumulative mileage, and roadbed soil type.

PDI history datasheet has separate text files for both rigid and flexible pavement segments tested between 1984 and 2009. It lists the segment sequence number, inverse year, test day-month- year, surface year, and distress type, severity, and extent for quantifying PDI.

IRI datasheet contains records representing segments tested between 1980 and 2008. Approximately three of four records pertain to flexible pavements. The datasheet lists fields representing the sequence number, inverse year, day-month-year segment was tested, the surface year, surface type, air temperature, average values for IRI, PSI, and Rut. In addition, it lists the speed and test vehicle number used to conduct the tests.

3.1.3 New Construction Reports

The New Construction database consists of spreadsheets organized by year from 1989 to present. Prior to 2000, spreadsheets were created in Microsoft Excel. Since 2000, the New Construction Reports can be found in Microsoft Access files organized by year from 2000 to present. Each file has two key datasheets, namely, the Office (ACOffice or PCCOffice) and Field (ACField or PCCField). The Office datasheet has a shows pavement location (rural or urban, district, county, termini by descriptive start and end points), construction style (reconstruction, resurfacing, rehabilitation), contract identification numbers (contract1, contract2), project length, pavement surface thickness, milling depth for HMA, base type (DGBC, CABC, OGBC2, pulverized, etc.), pavement surface paved over (Pvdovr), flexible/rigid pavement type, surface year (pvmntyr). For the ACOffice field, additional detail is provided such as mix type (Hv, Mv, Lv), case type (Standard, Superpave, SMA, HMA Warranty), and design ESAL magnitude for more recent reports. The ACField and PCCField datasheet has fields representing site identification number (site), sequence number (Sqno) for 2000 to present, beginning reference point (RP), contract identification number (contract2), highway name by direction, survey length (Survlen), lane, direction, Asphalt or PCC, set value, measured IRI, and rut depth (Rut) immediately after construction.

3.2 Database Merge

The Meta Manager, PIF, and New Construction databases were merged by pavement Sequence Number (ISEQNO) to yield a single composite database for every pavement segment in Wisconsin constructed from 1989 to 2002. The year 1989 was chosen as the earliest year since electronic reports are readily available. The year 2002 was choase as the latest year to allow the pavement to undergo traffic loading for at least 5 years. For pavement constructed before 2000, no Sequence Number is provided in the New Construction Reports, so a merge procedure was developed using the hierarchy of surface year, county number, and highway number. Then, a manual verification was performed to determine accuracy.

The merged database was then reduced to pavement segments that were either HMA paved over flexible or CABC base (Type 1) or JPCP with dowels (Type 8). These pavements are appropriate for MEPDG calibration. HMA pavement removed from consideration included Road Mix (Type 2) and HMA paved over undisturbed PCC (Type 3). These individual segments were then combined to yield a full description and data for each Sequence Number. The follow sections describe the pavement sections included for the JPCP and HMA calibration study, which are illustrated in Figure 3.1.

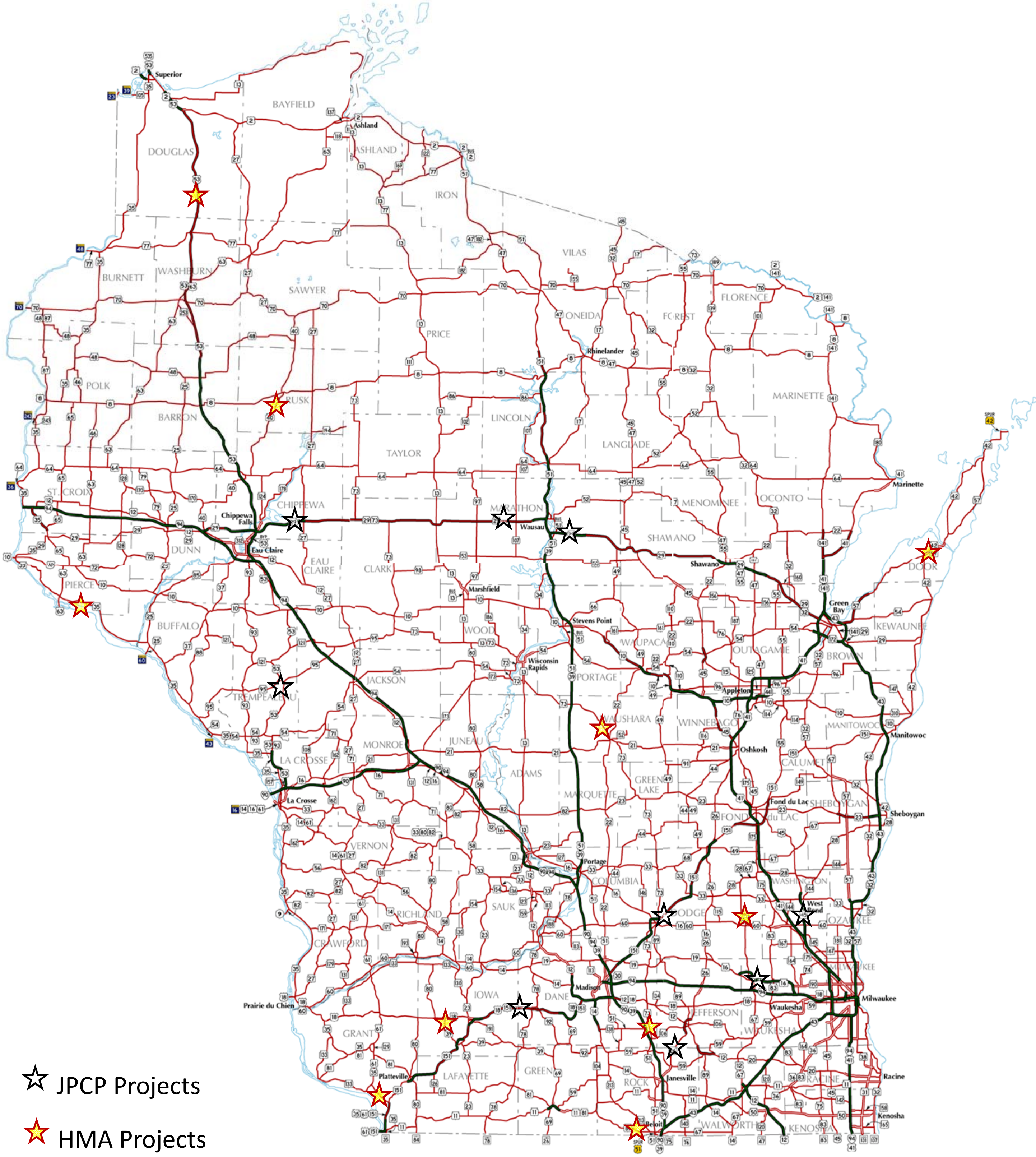

Figure 3.1 Project Locations

3.3 JPCP Database Development

The merged database identified a number of candidate pavement sections for inclusion into this study. Consultation with WisDOT and Industry personnel affirmed their desire to focus on JPCP pavements with a range of slab cracking percentages that would be considered representative of performance trends in Wisconsin. Table 3.2 provides a listing of the nine JPCP sections included in this study. Table 3.3 provides information on the traffic attributes for these projects obtained from the title sheets of project plans. Table 3.4 provides data on the percentage of slab cracking recorded for each project during the most recent survey.

Table 3.2 JPCP Test Sections

Test Location	Project ID	Project Limits	Year of Constr	Length mi	Included Sequence
STH 16 Waukesha Co	1371-03-77	Oconomowoc River to CTH C	1990	2.9	19620 – 19610
USH 18 Dane Co	1204-04-74	CTH E to STH 78	1988	1.0	21140
STH 26 Rock/Jefferson Co	1390-02-73	Milton to Ft Atkinson Rd	1986	8.6	29960 – 30040
STH 29a Marathon Co	1055-04-73	STH 97 to STH 107	1994	5.0	35290 – 35330
STH 29b Marathon Co	1054-07-77	USH B51 to CTH Q	1990	10.5	35460 – 35540
STH 29 Chippewa Co	1052-07-79	Stillson Creek to STH 27	1993	9.6	34670 – 34730
USH 45 Washington Co	2221-08-71	Paradise Dr to CTH D	1989	6.7	62610 – 62550
USH 53 Trempealeau	1638-01-71	STH 95 to Whitehall	1989	5.6	70930 – 70970
USH 151 Dodge Co	1111-07-79	STH 73 to CTH D	1993	8.4	125170 – 125230

3.4 HMA Database Development

The HMA database was created by merging the PIF, MetaManager, and New Construction files by Sequence Number. Construction data pre-2000 was merged using surface year, county, and highway number. Consultation with WisDOT and Industry personnel affirmed their desire to focus on HMA pavements with a range of rutting and alligator cracking that would be considered representative of performance trends in Wisconsin. Table 3.5 provides a listing of the nine HMA sections selected for inclusion in this study. Table 3.6 provides information on the traffic attributes for these projects obtained from the title sheets of project plans. Table 3.7 provides rutting and alligator performance data recorded for each project during the most recent survey.

Table 3.3 JPCP Test Section Traffic Attributes

Test Location	CY(1)	CY ADT	DY(2)	DY ADT	D-D(3)	% Trucks	Design Speed	Year 1 AADTT	Growth Rate
STH 16 Waukesha Co	1990	29000	2010	40000	60/40	4.3	55	1720	1.6%
USH 18 Dane Co	1988	8000	2008	12500	60/40	15	60	1875	2.3%
STH 26 Rock/Jefferson Co	1989	5980	2009	7180	60/40	12.8	60	919	0.9%
STH 29a Marathon Co	1985	10100	2010	14700	60/40	30	65	4410	1.5%
STH 29b Marathon Co	1988	8600	2010	11800	60/40	21.1	65	2490	1.4%
STH 29 Chippewa Co	1992	8550	2012	11150	60/40	13	70	1450	1.3%
USH 45 Washington Co	1989	11900	2009	13600	60/40	12	65	1632	0.7%
USH 53 Trempealeau	1988	2400	2008	3400	60/40	17	60	578	1.8%
USH 151 Dodge Co	1993	11400	2013	14600	60/40	15	65	2190	1.2%

(1)Construction Year; (2)Design Year; (3)Directional Distribution

Table 3.4 JPCP Test Section Cracking Data

Test Location	Pavement Age	Average % of Cracked Slabs	90% Reliability % Cracked Slabs
STH 16 Waukesha Co	18	6.7	14.1
USH 18 Dane Co	19	10.0	n.a.
STH 26 Rock/Jefferson Co	18	10.0	28.1
STH 29a Marathon Co	14	32.0	75.8
STH 29b Marathon Co	18	6.7	21.0
STH 29 Chippewa Co	14	45.7	79.5
USH 45 Washington Co	19	3.3	9.6
USH 53 Trempealeau	18	35.0	74.8
USH 151 Dodge Co	14	71.4	100.0

Table 3.5 HMA Test Sections

Test Location	Project ID	Project Limits	Year of Constr	Length mi	Included Sequence
STH 35 Pierce Co	7181-07-71	Pepin Co Line To USH 63	1992	14.5	44570 – 44700
STH 39 Iowa Co	5952-00-71	Linden to Mineral Point Rd	1991	10.4	48070 – 48140
STH 40 Rusk Co	8580-04-71	South Co Line To USH 8	1991	12.1	50640 – 50740
USH 53 Douglas Co	1198-01-74	CTH AA to Kent Rd	1994	9.2	72460 – 72540
USH 61 Grant Co	1650-01-76	Dickeyville to Lancaster Rd	1991	7.8	83380 – 83450
STH 67 Dodge Co	3031-02-71	STH 60 to STH 109	1993	3.0	89600 – 89620
STH 73 Dane Co	0013-13-30	IH 90 to CTH B	1991	4.0	93480 – 93510
STH 73 Waushara Co	6310-05-71	Wautoma to Plainfield	1992	14.4	94340 – 94440
STH 81 Rock Co	5341-03-71	CTH K to Beloit Rd	1990	5.2	100450 – 100490

Table 3.6 HMA Test Section Traffic Attributes

Test Location	CY	CY ADT	DY	DY ADT	D-D	% Trucks	Design Speed	Year 1 AADTT	Growth Rate
STH 35 Pierce Co	1996	2120	2016	3100	60/40	5.8	55	180	1.9%
STH 39 Iowa Co	1992	940	2012	1150	60/40	7	55	81	1.0%
STH 40 Rusk Co	1978	450	1998	630	50/50	7.3	55	46	1.7%
USH 53 Douglas Co	1991	4120	2011	6650	60/40	12	65	798	2.4%
USH 61 Grant Co	1991	3880	2011	4860	60/40	11	60	535	1.1%
STH 67 Dodge Co	1992	1200	2012	1400	60/40	9.7	60	136	0.8%
STH 73 Dane Co	1990	3300	2010	4200	60/40	9	60	378	1.2%
STH 73 Waushara Co	1992	3030	2012	4280	60/40	16	60	685	1.7%
STH 81 Rock Co	1990	3075	2010	3750	60/40	8	55	300	1.0%

Table 3.7 HMA Test Section Performance Data

Test Location	Pavement Age	Average Rut Depth inch	90% Reliability For Rutting	Average % Area of Alligator Cracking	90% Reliability For Alligator Cracking
STH 35 Pierce Co	15	0.11	0.18	10	41
STH 39 Iowa Co	16	0.15	0.22	2	10
STH 40 Rusk Co	16	0.19	0.26	0	0
USH 53 Douglas Co	13	0.09	0.15	0	0
USH 61 Grant Co	17	0.23	0.29	0	0
STH 67 Dodge Co	14	0.19	0.24	0	0
STH 73 Dane Co	16	0.20	0.23	0	0
STH 73 Waushara Co	16	0.20	0.25	0	0
STH 81 Rock Co	17	0.38	0.47	0	0

3.5 Project Site Visits

Project site visits were conducted from September 16 – October 23, 2009. For each project location, a representative portion was identified which would allow for safe traffic control operations during material sampling. A visual survey of existing distress, following the Pavement Condition Index (PCI) survey procedures, was completed for each project location. A minimum of one 528-ft sample was surveyed, covering the full lane width. Where traffic control provisions allowed, an additional 528-ft sample unit was surveyed for distress.

For the JPCP test sections, full-depth concrete samples were extracted from the outside edge of a selected pavement slab. A total of 3 beam samples were extracted by cutting through the PCC layer with a portable, parallel blade cut and break saw. Two full-depth beam samples, each approximately 6 inches x 24 inches, were extracted to provide sufficient materials for conducting modulus of rupture, unconfined compression, and coefficient of thermal expansion testing. A third full-depth beam, approximately 8 inches x 24 inches, was extracted to provide material for conducting modulus of elasticity and unconfined compression testing. All beam samples were extracted side-by-side to minimize the number of full-depth saw cuts. After the PCC materials were extracted, samples of base, subbase and subgrade were obtained for laboratory testing of gradation and Atterberg limits. The open concrete hole was then patched with rapid setting paving materials and opened to traffic within 2 hours after placement.

For the HMA sections, a full-depth block sample approximately 12 inches x 24 inches was extracted from the center of the lane using the parallel blade cut and break saw. This block specimen provided sufficient material for conducting dynamic modulus, aggregate gradation, and binder characterization tests. After the HMA materials were extracted, samples of base, subbase and subgrade were obtained for laboratory testing of gradation and Atterberg limits. A full-depth transverse cut, approximately 3 inches wide x 6 ft long was also made across the outer wheel path location. This extracted sample provided a cross-sectional view of the HMA layer that was examined to determine if any surface layer rutting was present. The pavement openings were then overfilled cold mix and compacted with a vibrating plate compactor. The pavement was re-opened to traffic immediately following sampling operations.

Chapter 4 – Laboratory Testing of Pavement Materials

This chapter presents the results of laboratory tests conducted on the concrete and HMA paving materials recovered from the test pavement sections. The primary purpose of the laboratory tests was to develop Level 1 inputs for the PCC and HMA surface layers. Laboratory tests were conducted at Marquette University, the University of Missouri–Kansas City, and Iowa State University. All field samples were initially transported to Marquette University for processing and testing. PCC core samples extracted from modulus of rupture (MOR) beams were transported to the University of Missouri–Kansas City for determination of the coefficient of thermal expansion (CTE) of the cured concrete materials. Recovered samples of HMA materials were transported to Iowa State University for dynamic modulus testing and mixture characterizations.

4.1 Modulus of Rupture of Concrete Materials

The modulus of rupture (MOR), or flexural strength, is determined from laboratory testing of concrete beam samples (AASHTO T97) and represents the maximum bending stress prior to rupture for a simply supported beam under third-point loading. The MOR value is directly related to the fatigue cracking potential of a PCC slab. For any given magnitude of repeated flexural stress, σ, developed from the combination of traffic and thermal loadings, a higher MOR value results in a lower stress ratio (σ/MOR) and a longer estimated fatigue life. For input Level 1, it is recommended that MOR tests be conducted at 7, 14, 28, and 90 days after placement to develop the early-age strength gain trend for a PCC mixture. The ratio of 20-year to 28-day MOR is also required to characterize the long-term strength gain, which is used in incremental damage analysis. The MEPDG software recommends a maximum ratio of 1.20 be used to estimate 20-year MOR from 28-day MOR values.

For this calibration effort, the JPCP test sections included were constructed between 1988 and 1994, which results in pavement ages of 15 to 21 years at the time of sampling and testing. Hence, the early-age MOR values at 7, 14, 28, and 90 days must be estimated from the long-term MOR values. For this analysis, an early-age MOR estimation process was developed based on standard maturity trends for concrete material compressive strength developments.

The compressive strength development for concrete up to 1 year may be estimated from the Plowman relationship, as shown:

$$\%\ of\ 28\text{-}day\ f'_c = 45.3 \log Maturity - 106.4 \tag{4.1}$$

where:

% of 28-day f'_c = percent of 28-day compressive strength

$$Maturity = 1{,}272 * curing\ days \tag{4.2}$$

The above relationship assumes a datum temperature of 11°F and a constant curing temperature of 64°F (64 – 11 = 53 * 24 = 1,272 °F hours per day). Equation 4.1 yields the following multipliers for estimating early-age compressive strength as a function of measured 28-day compressive strength:

7-day compressive strength = 45.3 Log (1,272*7) – 106.4 = 0.73*28-day f'_c

14-day compressive strength = 45.3 Log (1,272*14) – 106.4 = 0.86*28-day f'_c

28-day compressive strength = 45.3 Log (1,272*28) – 106.4 = 1.00*28-day f'_c

90-day compressive strength = 45.3 Log (1,272*90) – 106.4 = 1.23*28-day f'_c

The MEPDG software recommends that the ratio of long-term (15- to 20-year) to 28-day compressive strength be limited to a maximum value of 1.35. Using this recommended maximum ratio, the following multipliers would be appropriate to estimate early-age compressive strength as a function of the measured 20-year compressive strength:

7-day strength = 0.73/1.35 = 0.54*long-term strength

14-day strength = 0.86/1.35 = 0.64*long-term strength

28-day strength = 1.00/1.35 = 0.74*long-term strength

90-day strength = 1.23/1.35 = 0.91*long-term strength

The relationship between compressive strength and MOR is of the following form:

$$MOR = A\,(f'_c)^{0.5} \qquad (4.3)$$

Based on this general model form, the following multipliers would be appropriate to estimate early-age MOR as a function of the measured long-term (15- to 20-year) MOR:

7-day MOR = 0.73*long-term MOR

14-day MOR = 0.80*long-term MOR

28-day MOR = 0.86*long-term MOR

90-day MOR = 0.95*long-term MOR

For the purposes of this study, the above multipliers will be considered applicable for all JPCP test sections.

4.1.1 Sample Preparation and Testing

The recovered beam samples were first trimmed to approximately 6 inches in depth, resulting in two beam specimens from each test section, each approximately 6 in. x 6 in. x 20 in. These beam specimens were then tested in third-point loading using a portable beam loading frame equipped with an automated servo-motor to control the applied loading rate. Figures 4.1 and 4.2 provide photographs of the test frame and the custom data collection computer screen. Figure 4.3 provides a sample plot of the MOR test data obtained from the STH 16 Waukesha County beam tests.

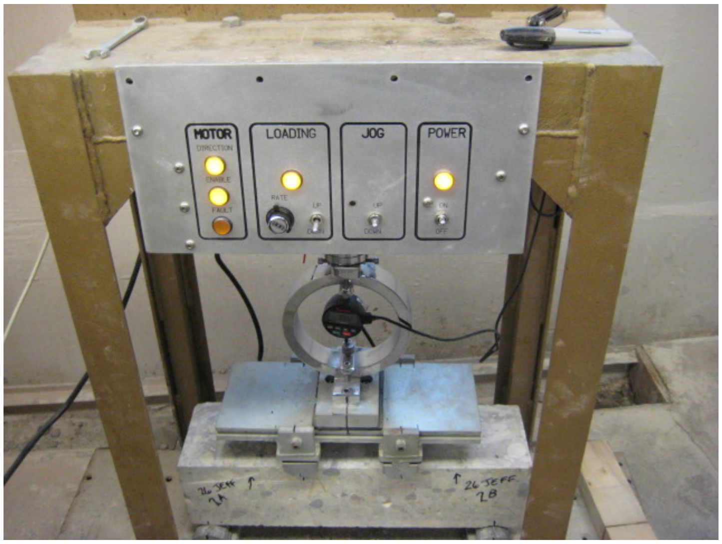

Figure 4.1 Portable Beam Load Frame

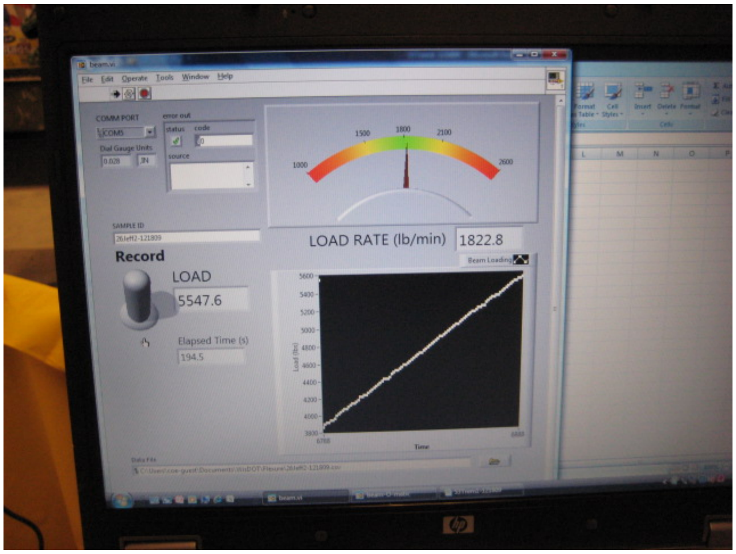

Figure 4.2 Data Collection Screen

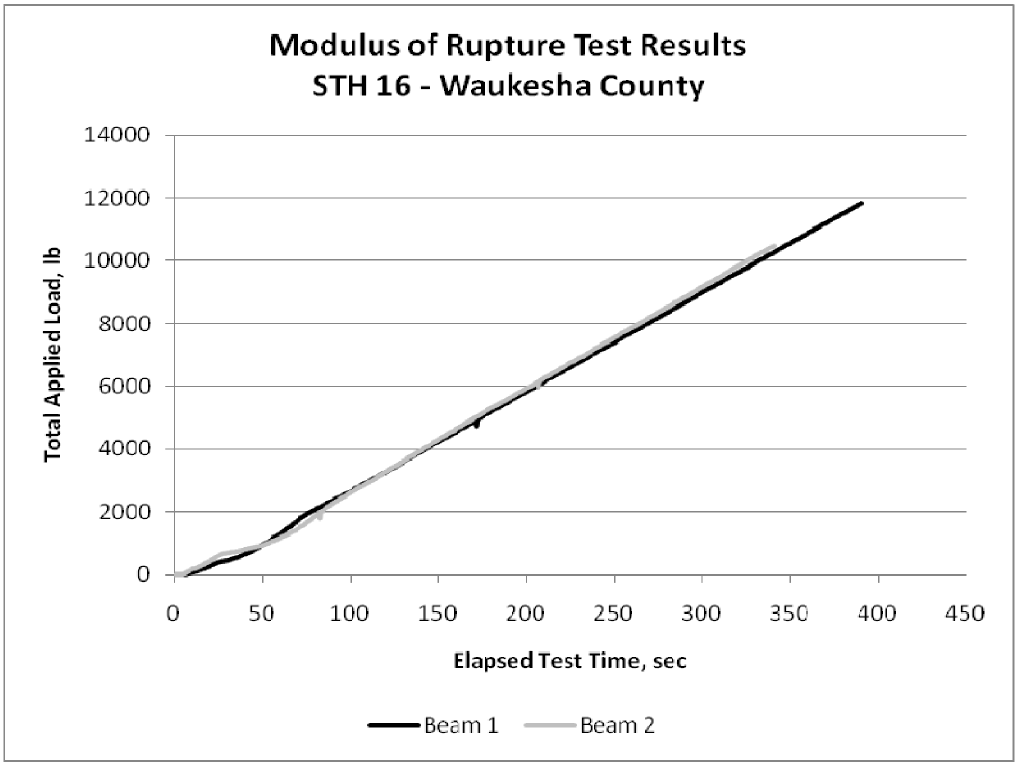

Figure 4.3 Sample MOR Data – STH 16, Waukesha County

After specimen rupture, the average width and depth of the fracture face were measured and the MOR was computed as follows:

$$MOR = \frac{PL}{bd^2} \tag{4.4}$$

where:

P	=	maximum applied load, lb
L	=	span length = 18 inches
b	=	average width of fracture face, inches
d	=	average depth of fracture face, inches

Table 4.1 provides the results of MOR testing and the early-age estimates for the nine JPCP test sections.

Table 4.1 Modulus of Rupture Test Results

Test Section	Modulus of Rupture, psi						
	Beam 1	Beam 2	Average	Early-Age Estimates			
				7-day	14-day	28-day	90-day
STH 16 - Waukesha	904	880	892	651	714	767	847
USH 18 - Dane	1063	883	973	710	778	837	924
STH 26 - Jefferson	662	758	710	518	568	611	675
STH 29 - Chippewa	816	698	757	553	606	651	719
STH 29a - Marathon	444	612	528	385	422	454	502
STH 29b - Marathon	752	688[1]	720	526	576	619	684
USH 45 - Washington	772	830[1]	801	585	641	689	761
USH 53 - Trempealeau	715	948	832	607	665	715	790
USH 151 - Dodge	849	856[1]	853	622	682	733	810

[1] Break was outside middle third of specimen

4.2 Compression Testing

The compressive strength is determined from laboratory testing of concrete cylinder samples (AASHTO T22) and represents the maximum compressive stress prior to rupture. The compressive strength is not required as a Level 1 input; however, this value is commonly obtained and used to predict the MOR and/or elastic modulus (E_c) of the concrete materials if no other test data are available. To better understand whether compressive strength would serve adequately as a surrogate for direct measures of MOR and/or E_c, compression testing was conducted on concrete cylinders, nominally 4 in x 8 in and 6 in x 12 in, fabricated from the recovered beam specimens.

Initially, the two portions of the broken beam specimens from the MOR tests were cored horizontally to obtain two cylinder specimens, each with a nominal diameter of 4 inches, as shown schematically in Figure 4.4. One of these cores was randomly selected and trimmed to provide a nominal L/D ratio of 2.0. The remaining core was preserved for conducting coefficient of thermal expansion testing.

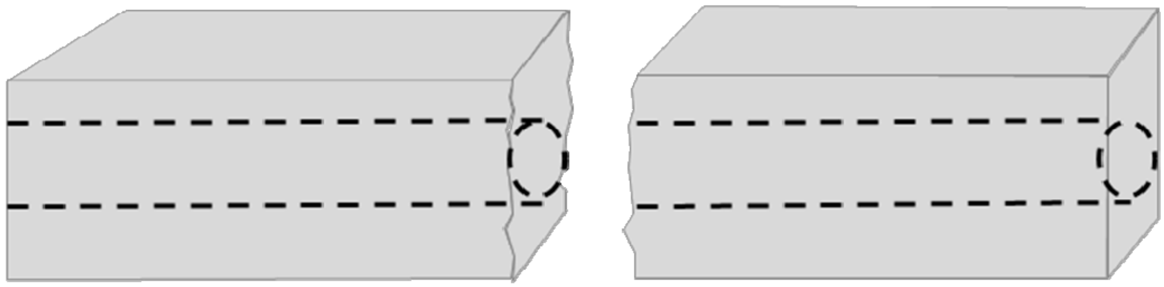

Figure 4.4 Schematic Illustration of Cores Extracted from Broken MOR Beam

The larger, unbroken beam specimens recovered from the field were cored horizontally to obtain cylindrical specimens with a nominal diameter of 6 inches. Each of these cylindrical specimens was cut to a nominal length of 12 inches (L/D = 2) and tested to obtain the long-term unconfined compressive strengths given in Table 4.2. Estimates of the early-age compressive strengths, given in Table 4.3, were computed from the equations presented earlier. (Note: Compressive strength testing was completed after the modulus of elasticity testing discussed in the next section.)

A comparison between the average 4 x 8 cylinder strengths and the 6 x 12 cylinder strengths is provided in Figure 4.5. The 6 x 12 cylinder strengths are, on average, about 5% lower than the 4 x 8 cylinder strengths.

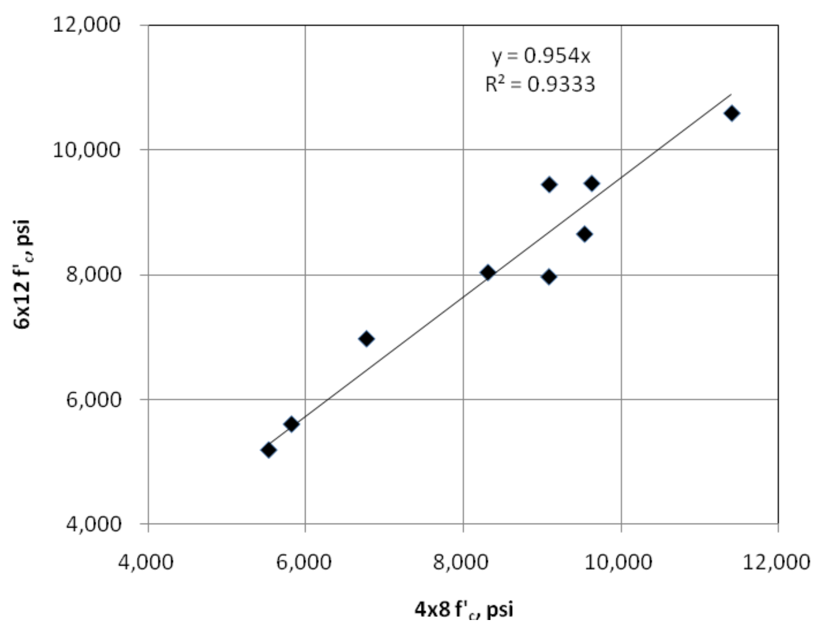

Figure 4.5 Comparison of 6 x 12 and 4 x 8 PCC Cylinder Strengths

Table 4.2 Long-Term Compressive Strength Test Results

Test Section	Unit Weight pcf	Compressive Strength, psi			
		4 x 8 Cylinder 1	4 x 8 Cylinder 2	4 x 8 Average	6 x 12 Cylinder
STH 16 - Waukesha	148.0	9,487	9,765	9,626	9,462
USH 18 - Dane	147.5	11,809	10,996	11,402	10,587
STH 26 - Jefferson	149.1	8,864	10,207	9,535	8,650
STH 29 - Chippewa	134.7	6,186	5,451	5,818	5,602[2]
STH 29a - Marathon	139.5	5,913	5,145	5,529	5,188
STH 29b - Marathon	143.4	7,050	6,489	6,769	6,971
USH 45 - Washington	151.9	9,135	9,038[1]	9,087	9,444
USH 53 - Trempealeau	145.0	7,371	9,247	8,309	8,035
USH 151 - Dodge	144.8	8,811	9,356	9,083	7,965

[1] L/d ratio = 1.74 [2] L/d ratio = 1.78

Table 4.3 Early-Age Compressive Strength Estimates

Test Section	6x12 Cyl. Long-Term Strength, psi	Early-Age Compressive Strength Estimate, psi			
		7-Day	14-Day	28-Day	90-Day
STH 16 - Waukesha	9,462	5,109	6,056	7,002	8,610
USH 18 - Dane	10,587	5,717	6,776	7,834	9,634
STH 26 - Jefferson	8,650	4,671	5,536	6,401	7,872
STH 29 - Chippewa	5,602[1]	3,025	3,585	4,145	5,098
STH 29a - Marathon	5,188	2,802	3,321	3,839	4,721
STH 29b - Marathon	6,971	3,765	4,462	5,159	6,344
USH 45 - Washington	9,444	5,100	6,044	6,988	8,594
USH 53 - Trempealeau	8,035	4,339	5,142	5,946	7,311
USH 151 - Dodge	7,965	4,301	5,098	5,894	7,248

[1] L/d ratio = 1.78

4.3 Modulus of Elasticity Testing

The concrete modulus of elasticity, E_c, is used in mechanistic analyses to characterize the concrete slab's stress responses to load- and temperature-induced strains. For input Level 1, E_c is determined by laboratory testing (ASTM C469) at ages of 7, 14, 28, and 90 days, and these values are used to estimate the long-term modulus gain. The MEPDG software recommends a maximum ratio of 1.20 be used for the estimation of long-term E_c values from 28-day test results.

For this calibration effort, the results of the 4 x 8 cylinder compression tests were used to estimate the ultimate strengths of the companion 6 x 12 cylinders. These estimated ultimate strengths were then used to establish the target maximum load to be applied during the modulus test, which by protocol is fixed at 40% of the ultimate failure stress.

The chord modulus was determined from two successive load cycles ranging from an initial longitudinal strain value of 50 millionths to a compressive stress of approximately 40% of the estimated ultimate failure stress (computed as the average compressive strength of the matching 4x8 cylinders). Figure 4.6 provides an example stress-strain plot for the STH 26 Jefferson County test site. For this site, the average compressive strength of the 4 x 8 cylinders was 9,535 psi, which yielded a target load of 108,500 lbs (3,800 psi) on the 6 x 12 cylinder to reach approximately 40% of this ultimate stress.

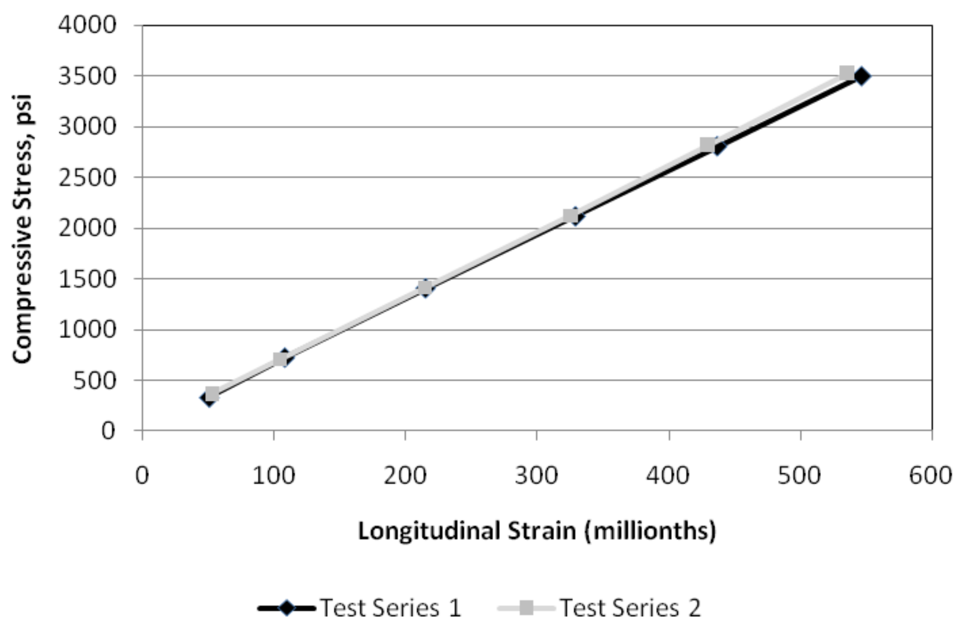

Figure 4.6 Stress-Strain Plots for STH 26, Jefferson County

The chord modulus was computed using the following formula:

$$E_c = \frac{(S_2 - S_1)}{\varepsilon_2 - 0.000050} \tag{4.5}$$

where:

E_c = chord modulus, psi
S_2 = stress corresponding to 40% of ultimate load
S_1 = stress corresponding to a longitudinal strain ε_1 of 50 millionths, psi
ε_2 = longitudinal strain produced by stress S_2

The Poisson's ratio was also computed, using the standard formula:

$$\mu = \frac{(\varepsilon_{t2} - \varepsilon_{t1})}{(\varepsilon_2 - 0.000050)} \tag{4.6}$$

where:

μ = Poisson's ratio
ε_{t2} = mid-height transverse strain produced by stress S_2
ε_{t1} = mid-height transverse strain produced by stress S_1

Table 4.4 provides the calculated long-term chord modulus values and Poisson's ratio for the included JPCP test sections. Table 4.5 provides the estimated 28-day E_c values based on the long-term values, assuming a constant reduction factor of 1.20.

Table 4.4 Computed Modulus of Elasticity and Poisson's Ratio

Test Section	Modulus of Elasticity, Mpsi			Poisson's Ratio		
	Load Cycle 1	Load Cycle 2	Average	Load Cycle 1	Load Cycle 2	Average
STH 16 - Waukesha	7.07	7.23	7.15	0.26	0.26	0.26
USH 18 - Dane	6.18	6.23	6.20	0.25	0.24	0.24
STH 26 - Jefferson	6.37	6.55	6.46	0.24	0.25	0.24
STH 29 - Chippewa	3.45	3.60	3.53	0.19	0.19	.019
STH 29a - Marathon	3.56	3.72	3.64	0.18	0.18	0.18
STH 29b - Marathon	4.78	5.08	4.93	0.22	0.23	0.22
USH 45 - Washington	6.54	6.56	6.55	0.12	0.11	0.12
USH 53 - Trempealeau	5.20	5.52	5.36	0.23	0.23	0.23
USH 151 - Dodge	4.90	5.23	5.16	0.22	0.21	0.22

Table 4.5 Estimated 28-day Modulus of Elasticity Values

Test Section	Long-Term Average E_c, Mpsi	28-Day Estimated E_c, Mpsi
STH 16 - Waukesha	7.15	5.96
USH 18 - Dane	6.20	5.17
STH 26 - Jefferson	6.46	5.38
STH 29 - Chippewa	3.53	2.94
STH 29a - Marathon	3.64	3.03
STH 29b - Marathon	4.93	4.11
USH 45 - Washington	6.55	5.46
USH 53 - Trempealeau	5.36	4.47
USH 151 - Dodge	5.16	4.30

4.3.1 Modulus of Elasticity Correlations

Modulus of elasticity values, when needed, are commonly estimated from compression strength test results of normal weight concrete using the following relationship:

$$E_{c\text{-}est} \;=\; 33\, w^{1.5} {f'_c}^{0.5} \qquad (4.7)$$

where:

$E_{c\text{-}est}$ = estimated modulus of elasticity, psi
w = unit weight of concrete, pcf

The results of compression strength and modulus testing for the 6 x 12 cylinders were examined to determine how well this general equation fit the data. Figure 4.7 presents the results for the nine cylinders tested. As shown, there is a good correlation of the data (R^2 = 0.834) with the measured E_c values being, on average, about 8% higher than estimated values.

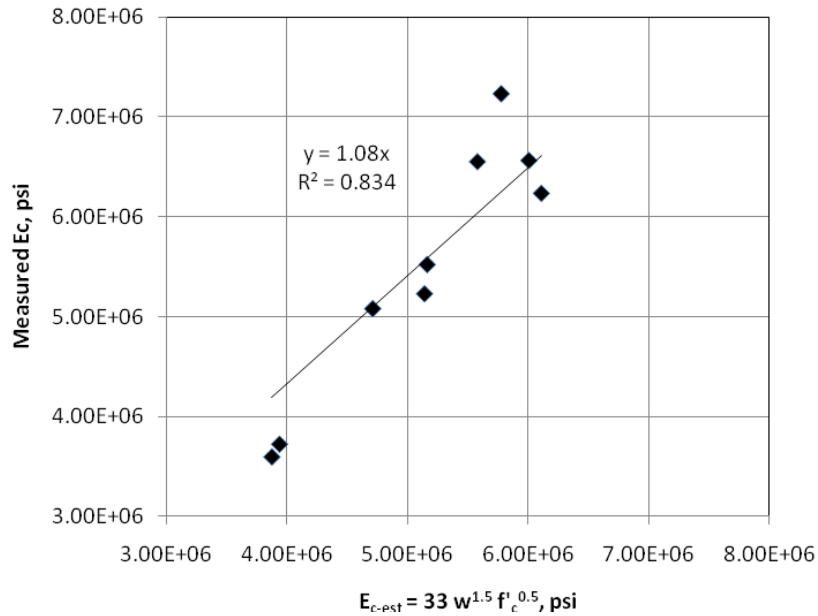

Figure 4.7 Measured versus Estimated Modulus of Elasticity Values

4.4 Coefficient of Thermal Expansion Testing

The coefficient of thermal expansion (CTE) is a required Level 1 input used to calculate thermal stresses due to diurnal slab temperature gradients and the magnitudes of joint openings in response to seasonal temperature changes. All CTE tests were performed at the University of Missouri–Kansas City (UMKC) according to AASHTO standard test method T 336-09, *Coefficient of Thermal Expansion of Hydraulic Cement Concrete*. CTE values are reported assuming a CTE of 304 Stainless Steel (SS) of 17.3 x $10^{-6}/^{o}C$, per AASHTO TP 60 recommendations given in the literature (Assumed TP60) and the CTE value determined according to ASTM E 228-06, *Standard Test Method for Linear Thermal Expansion of Solid Materials With a Push-Rod Dilatometer* (Actual E228). A recent FHWA inter-laboratory study has shown that the assumed CTE from TP 60 overestimates CTE and causes bias in the level of distress predicted by the Mechanistic-Empirical Pavement Design Guide (MEPDG).

The randomly selected remnants from the flexural tests were initially cored to obtain a nominal 4-inch-diameter cylindrical specimen. These specimens were then trimmed to a length of seven inches at Marquette University and transported to UMKC for CTE testing. All tests were performed according to AASHTO standard test method T336-09, *Coefficient of Thermal Expansion of Hydraulic Cement Concrete*. The concrete core specimens were saturated in a lime bath for at least 48 hours before testing. Saturated specimens were placed in an invar metal frame. The frame and the concrete specimen were then placed in a water bath. The temperature of the water bath was changed from 10°C to 50°C (50°F to 122°F) and then from 50°C to 10°C (122°F to 50°F), and the length change of the concrete specimen was measured with a linear variable differential transformer (LVDT) mounted to the frame (Figure 4.8). The measured change in the length of the concrete was corrected for change in the length of the frame for the same temperature range. The CTE of the concrete specimen was then calculated by dividing the corrected length change by the change in temperature and specimen length:

$$CTE = \frac{\left(\Delta L_a / L_0\right)}{\Delta T} \tag{4.8}$$

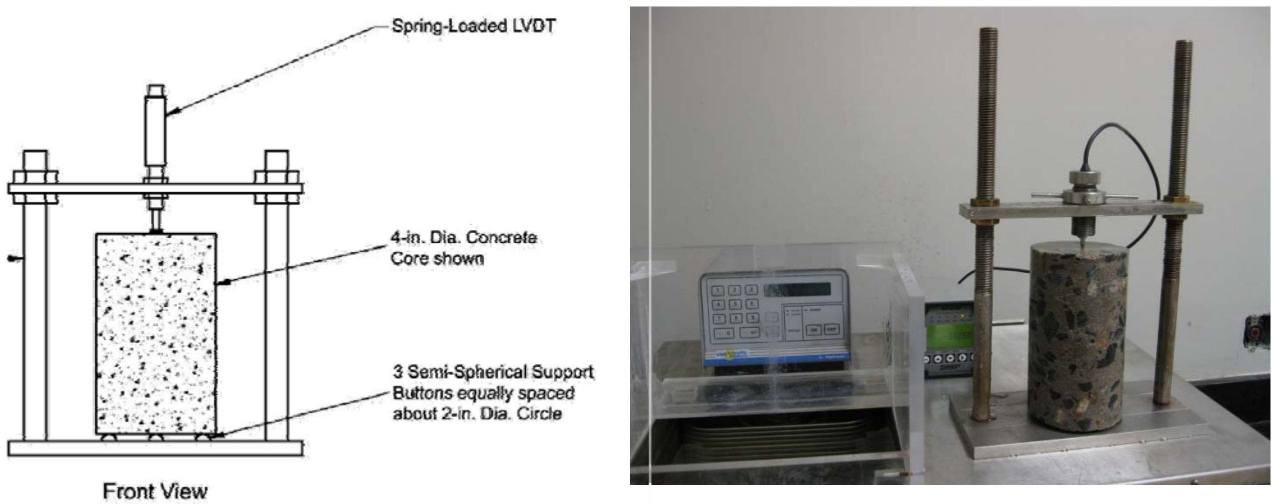

Figure 4.8 Schematic and Photo of the CTE Test Frame and Specimen

The length change of the frame was determined by testing a metal specimen (called a calibration specimen) of known CTE. Both 304 stainless steel (304 SS) and 410SS were used as calibration specimens. The C_f for the frame, assuming a 304SS CTE of 17.3 x 10^{-6}/^{o}C, was 2.34 x 10^{-6}/^{o}C. The C_f for the frame using the actual 410SS CTE of 10.3 x 10^{-6}/^{o}C was 1.62 x 10^{-6}/^{o}C.

The CTE test results are shown in Table 4.6. All values reported are averages, determined, to within 0.3 με/°C, from the 10°C to 50°C heating phase and the 50°C to 10°C cooling phase. All specimens had lengths similar to the required 180 ± 2 mm required in the standard, except for the specimens from USH 151, Dodge County, which had a length of 154 mm. Figure 4.9 provides a comparison plot of the CTE values computed per ASTM E228 versus AASHTO TP60. As shown, the CTE values from TP60 are, on average, about 6.5% higher than E228 values.

Table 4.6 Computed Coefficient of Thermal Expansion (VTE) Values

Test Section	Assumed TP60, $\mu\varepsilon/^{o}F$			Actual E228, $\mu\varepsilon/^{o}F$		
	Specimen		Average	Specimen		Average
	1	2		1	2	
STH 16 - Waukesha	6.6	6.3	6.4	6.2	5.9	6.1
USH 18 - Dane	5.9	5.5	5.7	5.5	5.1	5.3
STH 26 - Jefferson	6.6	6.4	6.5	6.2	5.9	6.1
STH 29 - Chippewa	5.8	6.0	5.9	5.4	5.6	5.5
STH 29a - Marathon	6.0	5.9	6.0	5.6	5.6	5.6
STH 29b - Marathon	5.6	5.4	5.5	5.2	4.9	5.1
USH 45 - Washington	6.1	6.2	6.1	5.7	5.8	5.8
USH 53 - Trempealeau	6.4	6.4	6.4	5.9	6.0	6.0
USH 151 - Dodge	6.4	6.5	6.5	6.1	6.1	6.1

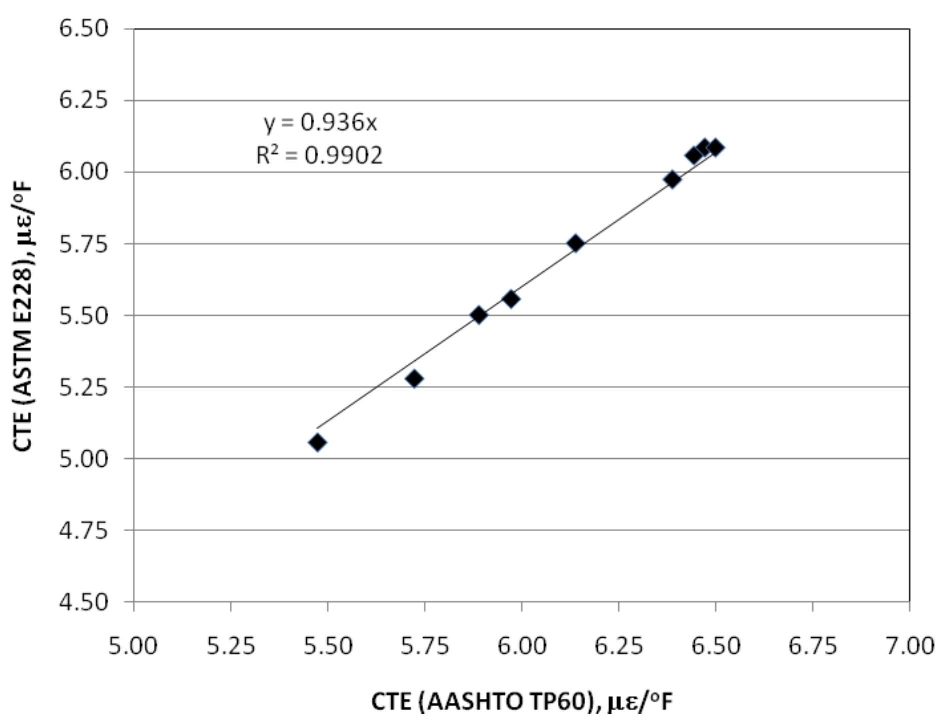

Figure 4.9 Comparison of Computed CTE Values

4.5 HMA Testing

All HMA tests were completed at Iowa State University under the direction of R. Christopher Williams. Iowa State University received saw-cut pavement samples for laboratory testing. The samples were catalogued and marked to denote the different lifts of asphalt mixtures. The surface course and base course were carefully chiseled from the sample. If a pavement sample contained more than two asphalt mixtures, then only the top two lifts were separated from the sample and denoted as the surface course and leveling course. The different asphalt courses were then tested for their bulk specific gravity (Gmb).

A portion of the samples were sent to the Minnesota Department of Transportation (MNDOT) for asphalt binder extraction. The asphalt binder was recovered from the pavement samples following AASHTO T-164 Method A (Centrifuge Method) by using toluene as the extraction method. The fines were removed from the binder extract by using a centrifuge at high speeds. Solvent was removed from the extract by following the rotavapor recovery process in ASTM D5404. At Iowa State University, the Performance Grade (PG) of the extracted asphalt binders was determined by following AASHTO M-320 “Standard Specification for Performance-Graded Asphalt Binder”. However, when the asphalt binder samples were tested in the DSR using the 25mm plate for the high PG temperature determination, the samples were so stiff they would fracture during the test. Therefore, in lieu of high temperature DSR testing, viscosity testing was conducted at four different temperatures using the rotational viscometer. Washed gradations of the aggregates after extractions were also conducted at Iowa State University by following AASHTO T27.

Dynamic modulus testing was then conducted on the asphalt pavement samples. The top two asphalt material lifts of pavement sample were heated in an oven for 3 hours at 150°C. The heated asphalt mixtures were then compacted in a gyratory compactor to 150mm in height using a 100mm diameter mold. The dynamic modulus samples were compacted to the same density of the in-situ density of the pavement samples by using the Gmb of the pavement samples. Between three and five dynamic modulus samples were compacted for each asphalt mixture depending upon the amount of material available. All pavement samples were successfully heated and recompacted, except for the sample from STH 35 which did not melt down into a consistency suitable for compacting, even when heated to 180°C.

AASHTO TP62 “Standard Test Method for Dynamic Modulus of Asphalt Concrete Mixtures” was followed to test the dynamic of the asphalt pavements samples. Each dynamic modulus sample was tested in a hydraulically powered Universal Testing Machine (UTM) where an actuator applied the cyclical load to the sample. The resulting strains were measured from three linear variable differential transformers (LVDT’s) that were attached to the sample. Each sample was tested at three temperatures (40^{o}, 70^{o}, and 99^{o}F) and nine frequencies of cyclical loading

(0.1, 0.3, 0.5, 1, 3, 5, 10, 15, and 25 Hz) to capture the time and temperature dependency of the HMA mixes.

Table 4.7 provides the MEPDG Level 1 dynamic modulus test data developed from the test data. Dynamic modulus values at 10 and 130 ^{o}F were estimated from the provided dynamic modulus data. Table 4.8 provides the result of the mix characterization tests. Mixture characterization data were used to generate MEPDG Level 3 inputs.

Table 4.7. MEPDG Level 1 Dynamic Modulus Input Values

Temp F	Freq Hz	39Iowa Surface	39Iowa Lower	40Rusk Surface	40Rusk Lower	53Doug Surface	53Doug Lower	61Grant Surface	67Dodge Surface	67Dodge Lower	73Waush Surface	73Dane Surface	73Dane Lower	81Rock Surface	81Rock Lower
10	0.1	3,978	4,224	3,276	2,391	2,102	3,029	2,540	2,865	2,328	2,759	3,028	1,417	3,483	2,673
	1	4,794	4,900	3,968	3,086	2,724	3,656	2,922	3,230	3,189	3,662	3,628	1,634	3,995	3,249
	10	4,950	4,950	4,292	3,425	3,086	4,185	3,278	3,479	3,652	4,103	4,032	1,743	4,277	3,569
	25	4,999	4,999	4,818	3,574	3,274	4,441	3,437	3,555	3,804	4,501	4,286	1,819	4,577	3,984
40	0.1	1,970	2,158	1,682	1,126	1,028	1,590	1,423	1,607	995	1,238	1,547	753	1,925	1,194
	1	2,574	2,808	2,205	1,591	1,423	2,106	1,753	1,996	1,498	1,759	2,018	933	2,435	1,683
	10	3,106	3,436	2,643	2,065	1,822	2,602	2,076	2,360	2,072	2,245	2,456	1,108	2,880	2,217
	25	3,500	3,573	2,661	2,230	1,912	2,705	2,159	2,381	2,300	2,585	2,548	1,117	3,027	2,404
70	0.1	653	770	606	329	326	579	615	687	208	308	557	306	807	291
	1	1,028	1,198	933	580	535	915	850	997	429	544	862	437	1,205	607
	10	1,574	1,831	1,356	1,007	872	1,323	1,128	1,364	890	940	1,253	628	1,678	1,111
	25	1,737	2,144	1,579	1,220	1,030	1,505	1,545	1,534	1,097	1,121	1,432	699	1,846	1,346
99	0.1	183	224	175	93	80	113	210	215	60	78	175	127	261	71
	1	338	402	303	174	166	223	318	352	108	159	295	204	450	148
	10	667	791	580	376	345	499	546	610	246	341	563	359	813	381
	25	812	975	700	505	431	683	628	714	338	434	671	416	964	510
130	0.1	116	142	111	59	51	82	135	112	38	49	111	80	178	45
	1	225	267	196	131	125	168	239	265	81	120	206	181	338	111
	10	311	349	262	214	181	284	295	347	140	305	319	271	462	216
	25	364	418	328	264	210	334	305	355	110	365	358	283	514	266

Table 4.8 HMA Mixture Properties

		Aggregate Gradation Data															
Project	HMA Layer	1.5"	1"	3/4"	1/2"	3/8"	#4	#8	#10	#16	#30	#40	#50	#100	#200	%AC	Density (g/cm^3)
STH 39 Iowa Co	Surface	100	100	98	79	68	50	41	40	37	33	28	22	11	6.9	4.88	2.409
STH 39 Iowa Co	Lower	100	100	100	96	86	61	45	43	37	32	29	24	14	8.6	6.29	2.434
STH 40 Rusk Co	Surface	100	100	93	79	59	47	45	38	28	23	17	10	6	5.9	4.65	2.359
STH 40 Rusk Co	Lower	100	100	100	93	83	63	50	48	41	30	24	18	10	6.1	4.66	2.377
USH 53 Douglas Co	Surface	100	100	99	99	93	70	48	45	36	27	23	18	9	5.4	5.02	2.455
USH 53 Douglas Co	Lower	100	100	100	98	96	76	52	48	38	28	23	18	10	5.4	5.47	2.468
USH 61 Grant Co	Surface	100	100	100	94	83	59	45	43	39	35	30	23	11	5.9	5.18	2.418
STH 67 Dodge Co	Surface	100	100	100	95	86	70	59	58	52	42	34	25	12	6.4	5.39	2.285
STH 67 Dodge Co	Lower	100	100	99	89	81	63	53	51	44	32	25	18	9	5.5	5.23	2.371
STH 73 Waushara Co	Surface	100	100	100	97	90	62	43	42	38	32	26	18	8	4.4	6.60	2.365
STH 73 Dane Co	Surface	100	100	100	96	88	69	54	53	47	38	28	20	10	5.9	4.46	2.334
STH 73 Dane Co	Lower	100	100	100	92	82	64	53	51	46	37	28	19	9	5.3	5.03	2.352
STH 81 Rock Co	Surface	100	99	91	80	71	51	37	35	29	23	19	16	12	9.5	4.91	2.326
STH 81 Rock Co	Lower	100	100	100	95	83	57	40	38	32	26	22	18	13	9.8	4.74	2.381

Chapter 5 – MEPDG Performance Model Calibration and Sensitivity

The discussion that follows in this chapter focuses on the two pavement types of highest priority to WisDOT, namely, conventional HMA and JPCP, and on the performance prediction models for these two pavement types as they function in version 1.1 of the MEPDG software. The predicted values shown in all of the figures in this chapter relating to calibration of the MEPDG's pavement models were obtained by modeling the performance over 30 years of the MEPDG software's built-in example HMA and JPCP designs, using climatic inputs for Wausau, Wisconsin.

5.1 Rutting in HMA Pavements

The MEPDG software predicts wheelpath rutting in HMA pavements by predicting the permanent deformation in each rut-susceptible layer (asphaltic or unbound) and in the subgrade as a function of time and traffic over the analysis period, and summing the predicted permanent deformations to compute the predicted total permanent deformation of the pavement surface.

The MEPDG text contains a recommendation from the MEPDG development team that trench studies be completed on certain LTPP flexible pavement test sections for the purpose of calibrating the layer rutting models in the MEPDG software. This is described as "critically important" because "without trenching data, it is physically impossible to accurately calibrate any type of rutting model for flexible pavement systems."

5.1.1 Permanent Deformation in Asphalt Mixtures

The form of the MEPDG model for AC permanent deformation is as follows:

$$\frac{\varepsilon_p}{\varepsilon_r} = k_z * \beta_{r1} 10^{k_1} \; T^{k_2 \beta_{r2}} \, N^{k_3 \beta_{r3}} \tag{5.1}$$

where:

- ε_p = accumulated plastic strain at N repetitions of load (in/in)
- ε_r = resilient strain as a function of asphalt mix properties, temperature, and rate of loading (in/in)
- k_z = a function to correct for effect of depth on confining pressure
- T = temperature (°F)
- N = number of load repetitions
- k_i = nonlinear regression coefficients
- β_{ri} = calibration factors

The resilient strain ε_r at any given depth in the AC layer is defined by the three-dimensional stress state at that depth and the elastic properties (dynamic modulus E* and Poisson's ratio μ) of the AC layer. The dynamic modulus E* varies with temperature and loading rate, and is also highly dependent on the viscosity characteristics of the binder. The general form of the Witczak equation for E* used in the MEPDG is the following:

$$\log(E^*) = \delta + \frac{\alpha}{1 + e^{\beta + \gamma \log(t_r)}} \quad (5.2)$$

where:

E* = dynamic modulus (psi)

δ = lower limit for E*

α = range of E* ($\delta + \alpha$ = upper limit for E*)

β, γ = parameters for the position and slope of the E* curve in the range between the lower and upper limits

Given the vertical resilient strain at any depth, and the ratio of plastic strain to resilient strain (from Equation 5.1), the plastic strain at any depth can be calculated, and the depth of rutting in the AC layer can be calculated as a function of the plastic strain and the layer thickness (h_{ac}, in).

The correction for the effect of depth on confining pressure is the following function of the total thickness of AC layers and the depth in inches to the computational point (i.e., the middepth of the AC structural layer):

$$k_z = (C_1 + C_2\, depth) * 0.328196^{depth} \quad (5.3)$$

$$C_1 = -0.1039\, {h_{ac}}^2 + 2.4868\, h_{ac} - 17.342 \quad (5.4)$$

$$C_2 = 0.0172\, {h_{ac}}^2 - 1.7331\, h_{ac} + 27.428 \quad (5.5)$$

The text of the MEPDG (in Section 3.3 and in Appendix GG) gives the following values of the β_{ri} coefficients and the k_i regression coefficients for the nationally calibrated AC rutting model:

β_{r1} = 0.509

β_{r2} = 0.9

β_{r3} = 1.2

k_1 = -3.15552

k_2 = 1.734

k_3 = 0.39937

With the β_{ri} calibration factors applied to the k_i regression coefficients as follows:

$$\beta_{r1} * 10^{k_1} = 0.509 * 10^{-3.15552} = 0.000356 = 10^{-3.4488}$$

$$\beta_{r2} * k_2 = 0.9 * 1.734 = 1.5606$$

$$\beta_{r3} * k_3 = 1.2 * 0.39937 = 0.479244$$

The nationally calibrated AC rutting model, shown as Equation 3.3.7 of the MEPDG text, is:

$$\frac{\varepsilon_p}{\varepsilon_r} = k_z * 10^{-3.4488}\, T^{1.5606}\, N^{0.479244} \tag{5.6}$$

The coefficient of determination (R^2) of this model is reported in the text to be 0.648, which means that 64.8 percent of the variation in the data used is accounted for by the model.

After recalibration, the model appears in version 1.1 of the MEPDG software with different values for the k_i and β_{ri} terms, as shown in Figure 5.1. The screen in Figure 5.1 is accessed in the MEPDG software by selecting the menu options Tools > Calibration Settings > Flexible New, as shown in Figure 5.2.

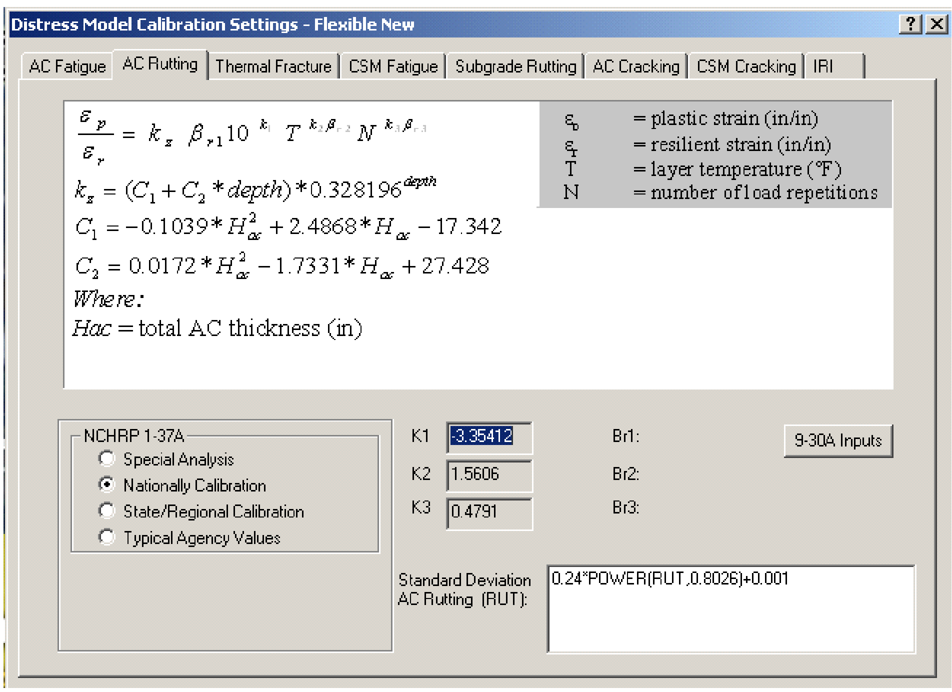

Figure 5.1 National Calibration of AC Rutting Model for New Flexible Pavement in MEPDG

Figure 5.2 MEPDG Menu Selection to View and Modify Model Calibration Settings

Thus, the recalibrated national AC rutting model is:

$$\frac{\varepsilon_p}{\varepsilon_r} = k_z * 10^{-3.35412}\ T^{1.5606}\ N^{0.4791} \tag{5.7}$$

To modify the national calibration coefficients for the AC rutting model to reflect local calibration of the model, an MEPDG software user would click on the "State/Regional Calibration" option, as shown in Figure 5.3, and vary the β_{ri} calibration factors as needed from their default values of 1.0.

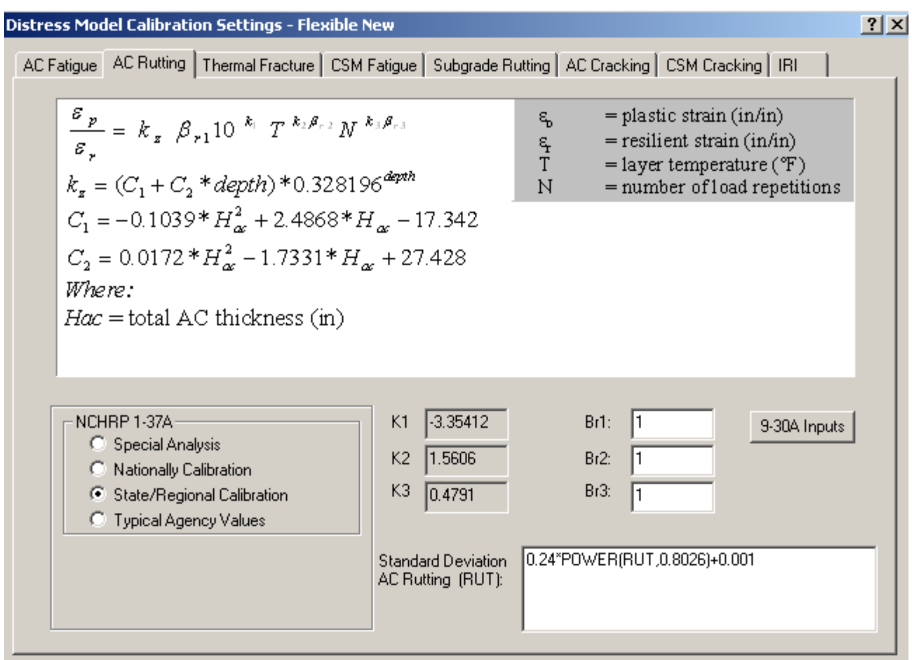

Figure 5.3 State/Regional Calibration Screen for MEPDG AC Rutting Model

The allowable range for the β_{ri} values on this screen is -10 to 10. A value entered outside this range will display in red rather than black and will cause the display of an error message such as the one shown in Figure 5.4.

Although the MEPDG software will allow β_{r1} values between -10 and 10, in fact any value for β_{r1} between -10 and 0 results in reported AC rutting values of 0 for the entire analysis period. Since, as can be seen from Equation 5.1, a β_{r1} value of 0 would remove the $k_z\, 10^{k1}$ term from the AC rutting prediction, and negative values of β_{r1} would produce negative AC rutting values, it follows that the software is constrained to display AC rutting as 0 when β_{r1} is less than or equal to 0. When β_{r1} is a positive number, predicted AC rutting increases linearly with β_{r1}, as shown in Figure 5.5.

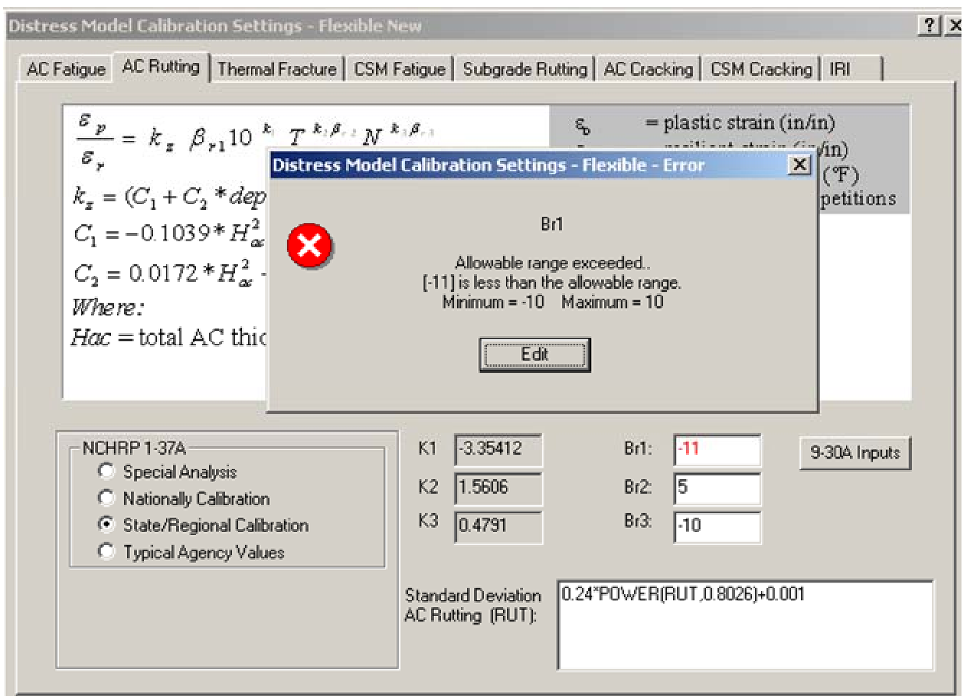

Figure 5.4 Out-of-Range Error Message on State/Regional Calibration Screen for MEPDG AC Rutting Model

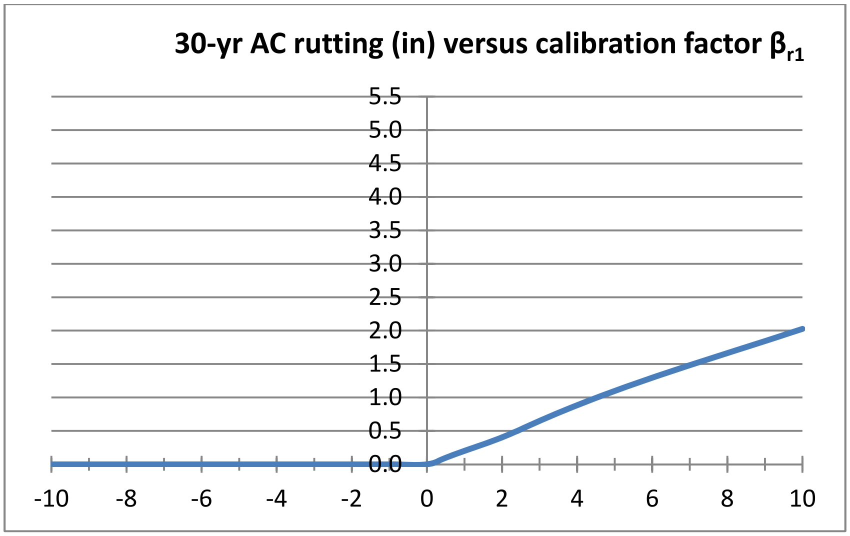

Figure 5.5 MEPDG Predicted AC Rutting versus Calibration Factor β_{r1}

As Equation 5.1 shows, AC rutting is expected to increase with increasing layer temperature (T) and increasing load repetitions (N). Although the MEPDG software will allow β_{r2} and β_{r3} values between -10 and 10, in fact any value for β_{r2} or β_{r3} less than zero would invert these trends. The software is constrained to display AC rutting as 0 when either β_{r2} or β_{r3} is less than zero. For values of β_{r2} and β_{r3} greater than or equal to zero, predicted AC rutting increases as shown in Figures 5.6 and 5.7. Predicted AC rutting is very sensitive to changes in the β_{r2} and β_{r3} coefficients in the range between 1 and 2.

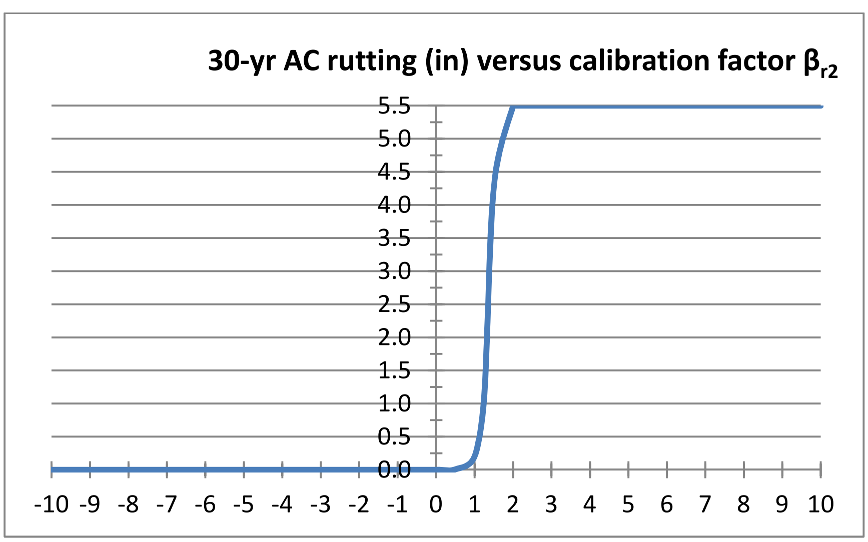

Figure 5.6 MEPDG Predicted AC Rutting versus Calibration Factor β_{r2}

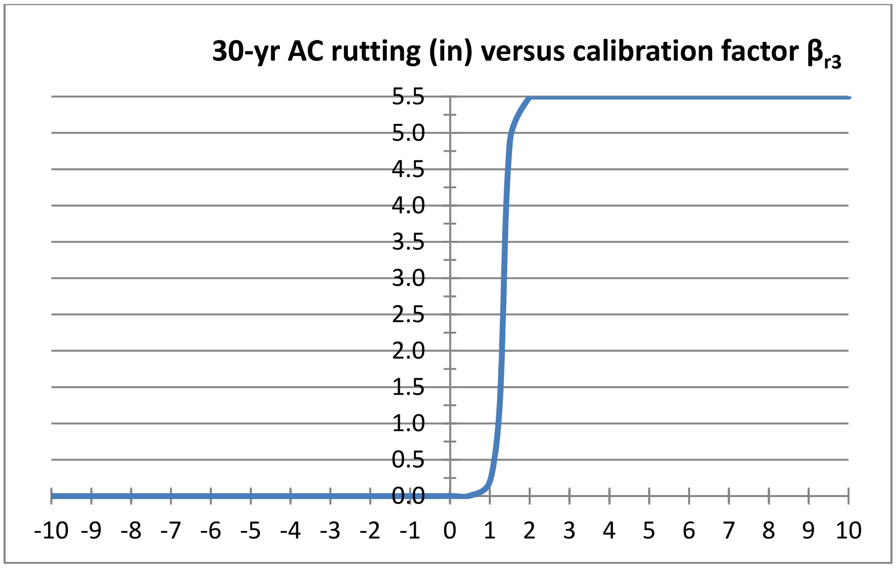

Figure 5.7 MEPDG Predicted AC Rutting versus Calibration Factor β_{r3}

NCHRP Report 372, *Sensitivity Evaluation of MEPDG Performance Prediction*, presents the results of sensitivity analyses of each of the MEPDG models with respect to its design inputs. The results are quantified in terms of a mean normalized sensitivity index plus or minus two standard deviations ($NSI_{\mu\pm2\sigma}$). A positive $NSI_{\mu\pm2\sigma}$ value means that the predicted pavement response increases as the value of the input increases, and a negative $NSI_{\mu\pm2\sigma}$ value means that the predicted pavement response decreases as the value of the input increases.

The results of the NCHRP Report 372 sensitivity analysis of the MEPDG AC rutting model are illustrated in Figure 5.8. Note that many of the inputs shown do not appear directly in the AC rutting model (Equation 5.1); rather, they enter into the elastic layer subroutine and/or the Enhanced Integrated Climatic Model (EICM) used to calculate resilient strain in the AC as a function of asphalt mix properites, temperature, and rate of loading. Remember also that this model predicts only the rutting that occurs within the AC layer. The total predicted rutting is the sum of the predicted AC rutting and the predicted rutting in the underlying base and subgrade, the model for which is discussed subsequently.

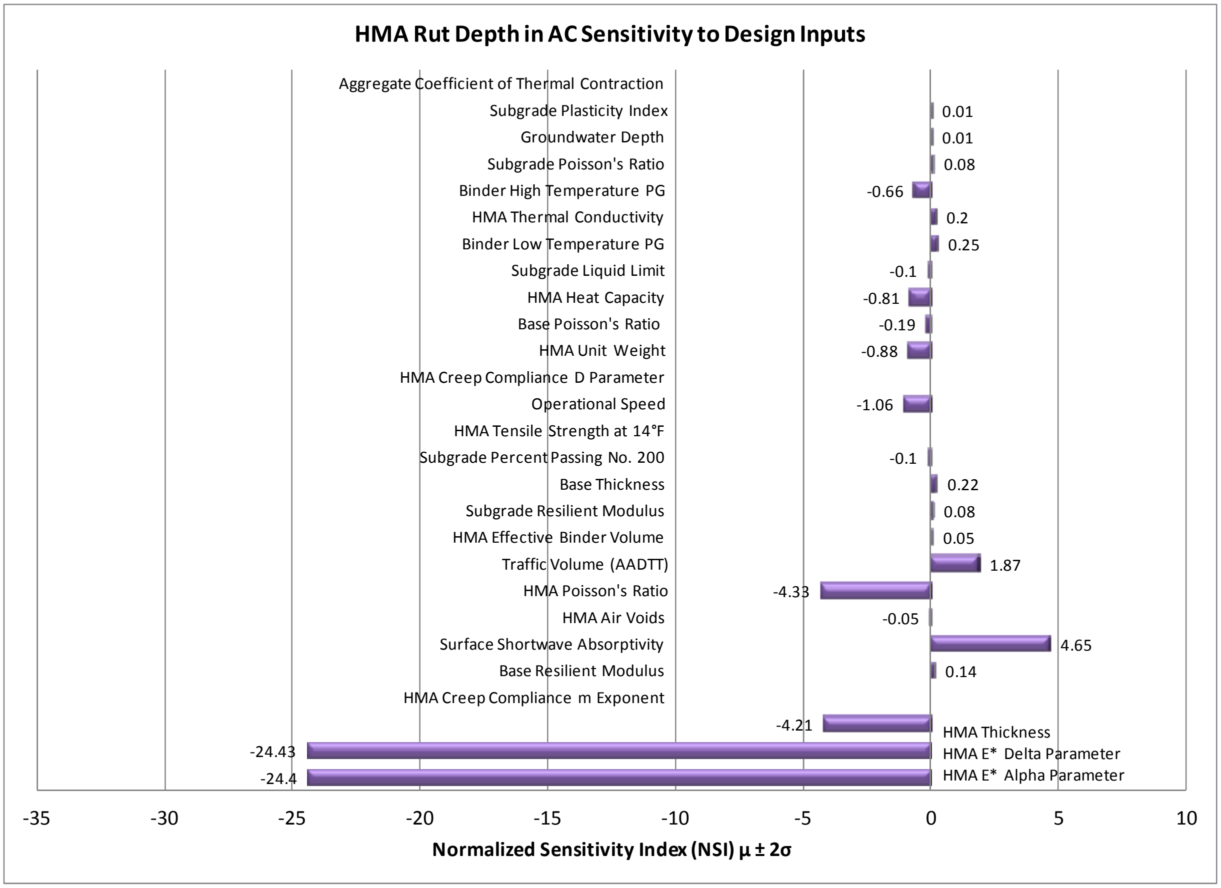

Figure 5.8 MEPDG AC Rutting Sensitivity to Design Inputs (Data from NCHRP Report 372)

These results illustrate that the MEPDG AC rutting model is extremely sensitive to the delta (δ) and alpha (α) parameters in the E* model (Equation 5.2). The next most sensitive inputs to the AC rutting model are the surface shortwave absorptivity, HMA Poisson's ratio, and HMA thickness. The next most sensitive inputs after these five–and the first not associated with the properties or thickness of the AC layer–is the truck traffic level, expressed in terms of the average annual daily truck traffic (AADTT).

About the two most sensitive inputs to the AC rutting model, NCHRP Report 372 remarks:

> *"When interpreting the very large sensitivity values for the HMA E* alpha and delta parameters, it is important to note that the typical ranges for these parameters are very narrow. The standard deviations for α and δ are only 1.6% and 3.5% of their mean values, respectively. The high sensitivity of most predicted distresses to the HMA E* alpha and delta parameters suggests a careful Level 1 characterization of HMA dynamic modulus for important projects."*

Surface shortwave absorptivity is used in the EICM, along with historical climatic data and the groundwater depth input, to calculate temperature and moisture distributions with depth and over time. These distributions are used to calculate seasonal moisture contents and freeze-thaw cycles in unbound pavement layers. The MEPDG software provides the following definition of, and typical values for, surface shortwave absorptivity:

> *"This input parameter pertains to AC and PCC surface layers, and is a measure of the amount of available solar energy that is absorbed by the pavement surface. The lighter and more reflective the surface, the lower the surface shortwave absorptivity. The suggested ranges for this value are:*
>
> | | |
> |---|---|
> | *Aged PCC layer:* | *0.70-0.90* |
> | *Weathered asphalt (gray):* | *0.80-0.90* |
> | *Fresh asphalt (black):* | *0.90-0.98"* |

According to the reference manual for the National Highway Institute's training course on *Geotechnical Aspects of Pavements*[1], although laboratory procedures exist for measuring shortwave absorptivity, there currently are no AASHTO protocols for measuring this parameter for paving materials.

About the sensitivity of predicted AC rutting to HMA Poisson's ratio, NCHRP Report 372 remarks:

> *"Poisson's ratio was an unexpectedly sensitive input for HMA and to a lesser extent, for the subgrade. Poisson's ratio is conventionally thought to have only a minor effect on pavement performance and consequently its value is usually assumed for design. These findings suggest a need for reexamination."*

5.1.2 Permanent Deformation in the Granular Base and Subgrade

The form of the MEPDG model for permanent deformation in unbound granular bases and subgrade soils, as it appears in the MEPDG text (Section 3.3 and Appendix GG), is as follows:

$$\delta_a\,(N) = \;\beta_i\,\varepsilon_v\;h\,\left(\frac{\varepsilon_0}{\varepsilon_r}\right)\,e^{-\left(\frac{\rho}{N}\right)^{\beta}} \tag{5.8}$$

where:

δ_a	=	permanent deformation in the layer (in)
N	=	number of load repetitions
ε_v	=	average vertical strain in the layer, from elastic layer analysis (in/in)
h	=	thickness of the layer (in)
ε_0, β, ρ	=	parameters whose values depend on material properties

ε_r = resilient strain imposed in the laboratory to obtain material properties (in/in)
β_i = calibration factors for unbound granular layer or subgrade soil
= β_{GB} for granular base
= β_{SG} for subgrade soil

The exponent β shown in Equation 5.8 is not a calibration factor; it is a material property parameter that is a function of the water content of the material (degree of saturation, in percent). The value of the ρ term also depends on the water content of the material. The MEPDG software uses equations for estimating the ratio $\varepsilon_o/\varepsilon_r$ for different types of materials as a function of water content of the material, the resilient modulus of the material, and the bulk stress (for granular materials) or deviator stress (for fine-grained soils) applied to the material.

The plastic strain in the material is represented by the following portion of Equation 5.8:

$$\varepsilon_p = \varepsilon_v \left(\frac{\varepsilon_0}{\varepsilon_r}\right) e^{-\left(\frac{\rho}{N}\right)^{\beta}} \tag{5.9}$$

The permanent deformation in granular base layer or subgrade soil due to N load repetitions is predicted as a function of the predicted plastic strain (which varies with depth) and the thickness of the layer, multiplied by the k_i and β_i calibration factors.

The MEPDG text gives the following values for the β_i calibration factors for the unbound materials permanent deformation model:

β_{GB} = 1.673
β_{SG} = 1.35

For these values for β_{GB} and β_{SG}, the R^2 values for the permanent deformation models for granular bases and subgrades, respectively, are reported in the MEPDG text as 0.677 and 0.136. The permanent deformation model for unbound materials is shown in Figure 5.9 and Equation 5.10.

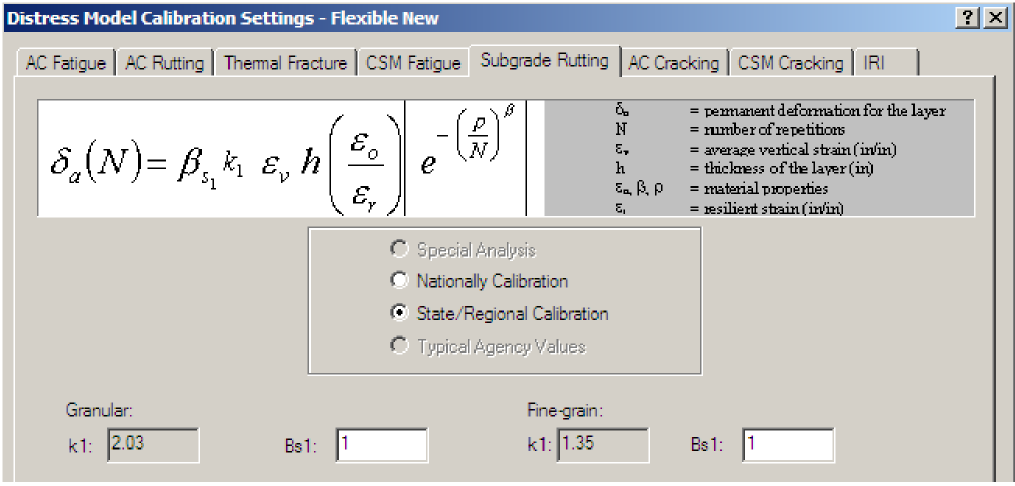

Figure 5.9 Unbound Materials Permanent Deformation Model in the MEPDG Software

$$\delta_a\,(N) = \ \beta_i\,k_i\ \varepsilon_v\ h\left(\frac{\varepsilon_0}{\varepsilon_r}\right) e^{-\left(\frac{\rho}{N}\right)^\beta} \qquad (5.10)$$

The values that had been determined for the β_i calibration factors are shown in the software as applying to regression constants k_i, with the value of the k_{GB} regression constant having been changed from 1.673 to 2.03 after recalibration, and the β_i calibration factors are both given as 1.0 for the nationally recalibrated models. Local calibration is accomplished by varying the values of the β_{GB} and β_{SG} calibration factors from their default values of 1.0.

Although the allowable range for both β_{GB} and β_{SG} is -10 to 10, rutting values of 0 are displayed when values less than or equal to zero are used for either or both of these calibration factors. For positive values of β_{GB} and β_{SG}, predicted base and subgrade rutting increase as shown in Figures 5.10 and 5.11. Note that the horizontal and vertical scales in these two figures are the same as in Figures 5.5, 5.6, and 5.7, to illustrate that the MEPDG's prediction of rutting in the AC layers of an AC pavement is far more sensitive to variation in the AC model's calibration factors than the prediction of granular base and subgrade rutting are to variation in the β_{GB} and β_{SG} calibration factors.

The predicted total rut depth in an HMA pavement over a granular foundation is computed as the sum of the predicted rutting in the AC plus the predicted rutting in the granular base plus the predicted rutting in the subgrade. The MEPDG text reports that for the nationally calibrated models (prior to recalibration reflected in version 1.1 of the software), the R^2 of predicted total rutting versus actual total rutting was 0.399. The user manual for the MEPDG software reports the R^2 of predicted total rutting versus actual total rutting (after recalibration) as 0.577.

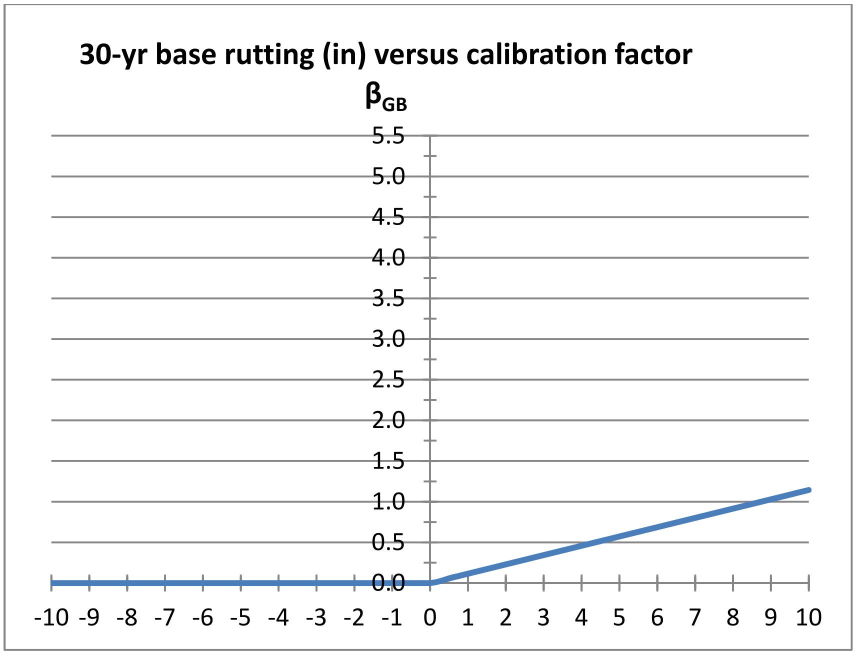

Figure 5.10 MEPDG Predicted Base Rutting versus Calibration Factor β_{GB}

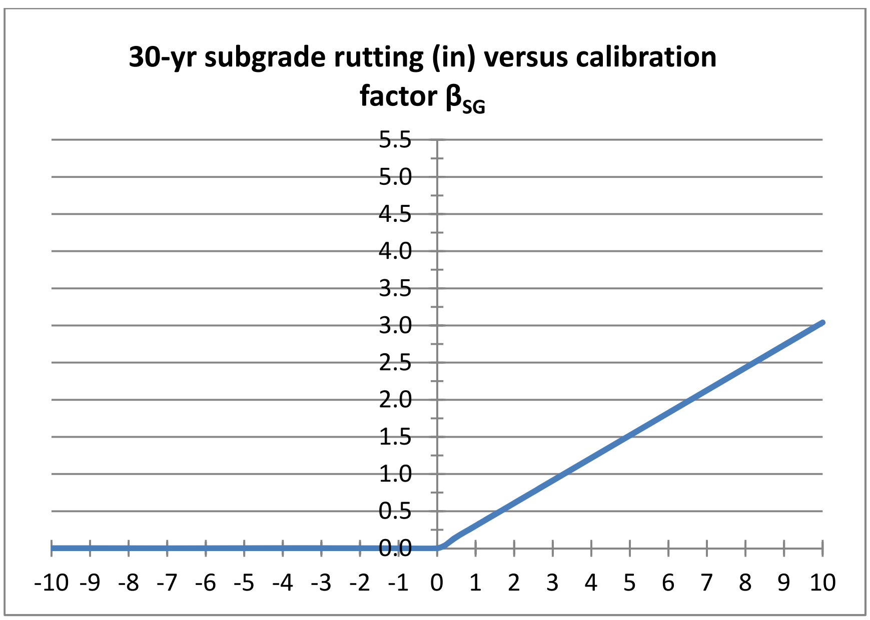

Figure 5.11 MEPDG Predicted Subgrade Rutting versus Calibration factor β_{SG}

The results of the sensitivity analysis of total rut depth conducted for NCHRP Report 372 are illustrated in Figure 5.12. The prediction of total rutting is extremely sensitive to the delta (δ) and alpha (α) parameters in the E* model (Equation 5.2). The next most sensitive inputs are the surface shortwave absorptivity, HMA thickness, and HMA Poisson's ratio, and HMA thickness. The remarks quoted from NCHRP Report 372 concerning the sensitivity of the prediction of AC rutting to these inputs apply to the prediction of total rutting as well.

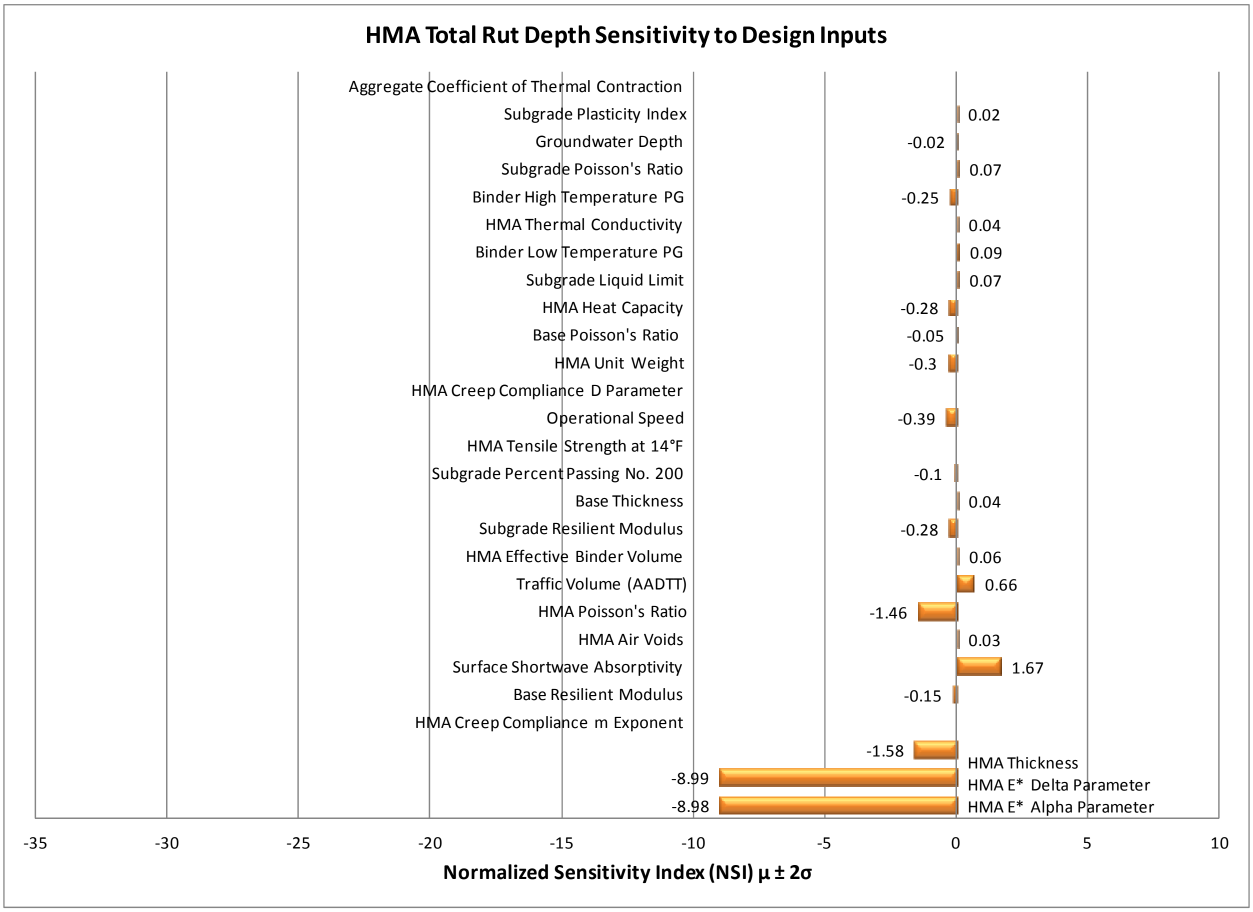

Figure 5.12 MEPDG Total HMA Rutting Sensitivity to Design Inputs
(Data from NCHRP Report 372)

5.2 Load-Related Cracking in HMA Pavements

The MEPDG uses the Asphalt Institute's fatigue model[2] in the prediction of two types of load-related cracking in HMA pavements. Alligator cracking in the wheelpaths is assumed to propagate upward from the bottom of the AC layer due to fatigue damage, and is referred to in the MEPDG as bottom-up cracking. Longitudinal cracking in the wheelpaths is assumed to propagate downward from the top of the AC layer due to fatigue damage, and is referred to in the MEPDG as top-down cracking.

The Asphalt Institute's model for prediction of the number of repetitions to fatigue cracking asphalt mixtures is based on laboratory testing of beam samples in constant stress mode (in which the magnitude of the applied load is held constant and repeated until failure occurs). The form of the model is as follows:

$$N_f \; = \; 0.00432 * \; 10^{4.84\left(\frac{V_b}{V_a+V_b} - 0.69\right)} \; \beta_{f1} \; k_1 \; \left(\frac{1}{\varepsilon_t}\right)^{\beta_{f2}\, k_2} \; \left(\frac{1}{E}\right)^{\beta_{f3}\, k_3} \tag{5.11}$$

where:

N_f = number of repetitions to fatigue cracking
ε_t = tensile strain at the critical location
E = stiffness (dynamic modulus) of the material
k_i = laboratory regression coefficients
β_i = calibration factors
V_a = air voids (percent)
V_b = effective binder content (percent)

The cumulative fatigue damage done to the HMA layers of the pavement is determined by summing the ratio of applied loads to allowable loads (from Equation 5.11) with respect to time and the spectrum of axle loads applied:

$$D \; = \; \sum\left(\frac{n}{N_f}\right)_{j,m,l,p.T} \tag{5.12}$$

where:

D = cumulative damage index for each critical location analyzed in HMA layers
n = number of loads within a given time period
N_f = number of allowable loads, from Equation 5.11
j = axle load interval
m = axle load type (single, tandem, tridem, quad, or special)
l = truck type from truck classification groups
p = month
T = median temperature for the five intervals used to subdivide each month

The predicted quantity of longitudinal (top-down) cracking is computed using the following transfer function:

$$FC_{top} \; = \; 10.56 \; \left(\frac{C_4}{1+ e^{\left(C_1 - C_2 * log\,(D_t * 100)\right)}}\right) \tag{5.13}$$

where:

FC_{top} = top-down fatigue cracking, percent of lane area

D_t = top-down cumulative fatigue damage index, from Equation 5.12

C_1, C_2, C_4 = transfer function regression constants, with the following values for the nationally calibrated model: $C_1 = 7$, $C_2 = 3.5$, $C_4 = 1000$

The predicted quantity of alligator (bottom-up) cracking is computed using the following transfer function:

$$FC_{bottom} = \left(\frac{1}{60}\right)\left(\frac{C_4}{1+e^{\left(C_1 C_1{}' + C_2 C_2{}' * \log\left(D_b * 100\right)\right)}}\right) \tag{5.14}$$

where:

FC_{bottom} = bottom-up fatigue cracking, percent of lane area

D_b = bottom-up cumulative fatigue damage index, from Equation 5.12

C_1, C_2, C_4 = transfer function regression constants, with the following values for the nationally calibrated model: $C_1 = 7$, $C_2 = 1$, $C_4 = 6000$

C_1' = $-2 * C_2'$

C_2' = $-2.40874 - 39.748 * (1+h_{ac})^{-2.856}$

h_{ac} = total thickness of AC layers, inches

According to the MEPDG *Manual of Practice*[10], the predictions of longitudinal cracking and alligator cracking also require multiplication of the results of Equations 13 and 14, respectively, by correction factors that are a function of the thickness of the HMA layers. For longitudinal (top-down) cracking, the equation for the thickness correction factor is as follows:

$$C_H = \frac{1}{0.01 + \frac{12.00}{1+e^{(15.676-2.8186\,h_{ac})}}} \tag{5.15}$$

For alligator (bottom-up) cracking, the equation for the thickness correction factor is as follows:

$$C_H = \frac{1}{0.00398 + \frac{0.003602}{1+e^{(11.02-3.49\,h_{ac})}}} \tag{5.16}$$

These thickness correction factors appear in the 2004 MEPDG text but do not appear in the screens that present the MEPDG models and calibration factors in Versions 1.0 and 1.1 of the MEPDG software. Presumably these equations as shown are embedded in the code for Version 1.1 of the MEPDG software.

The fatigue model (Equation 5.11) appears in the MEPDG software as shown in Figure 5.14. As the figure shows, the AI model has been adjusted by the following values of the k_i regression coefficients for national calibration purposes:

$k_1 = 0.007566$
$k_2 = 3.9492$
$k_3 = 1.281$

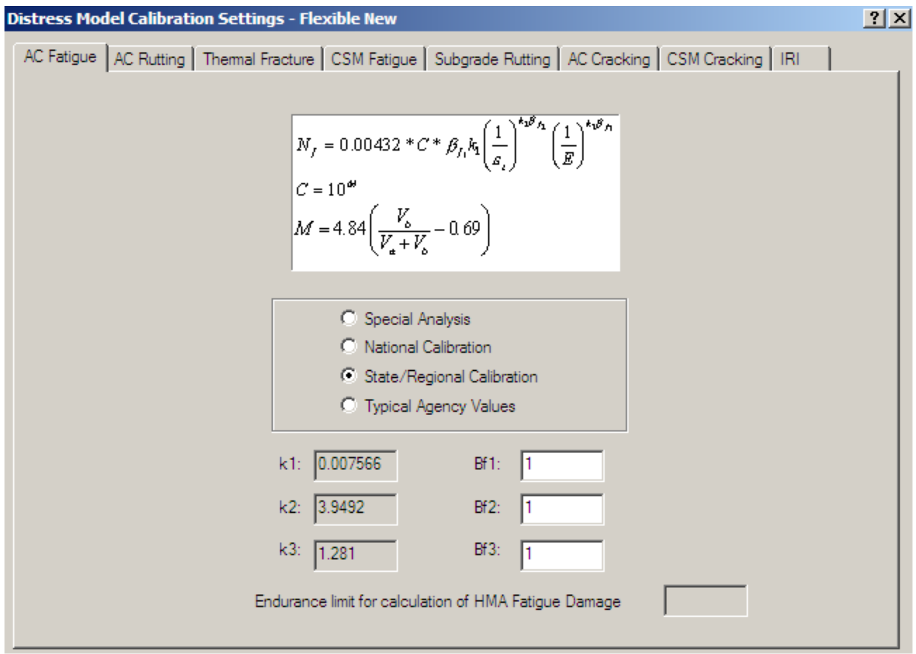

Figure 5.13 Asphalt Institute Fatigue Model used in MEPDG

Local calibration of the fatigue model is accomplished by varying the values of the β_{f1}, β_{f2}, and β_{f3} calibration factors from their default values of 1.0.

The allowable range of values for β_{f1} is -10 to 10, but fatigue and cracking values of 0 are displayed in the software output for β_{f1} values of 0 or less. For positive values of β_{f1}, the predicted 30-year top-down and bottom-up fatigue damage levels (applied loads as a percentage of allowable) are shown in Figure 5.14. The corresponding predicted 30-year top-down cracking (ft/mile) and bottom-up cracking (percent of lane area) levels are shown in Figure 5.15.

Both the longitudinal (top-down) fatigue damage and cracking predictions are highly sensitive to values of β_{f1} close to 0, with predicted longitudinal fatigue damage and predicted longitudinal cracking decreasing rapidly as β_{f1} increases from 0 to 1. For higher positive values of β_{f1}, the magnitudes of predicted longitudinal fatigue damage and longitudinal cracking are greatly diminished and increasingly insensitive to the value of β_{f1}. The predictions of alligator (bottom-up) fatigue damage and cracking exhibit similarly declining trends.

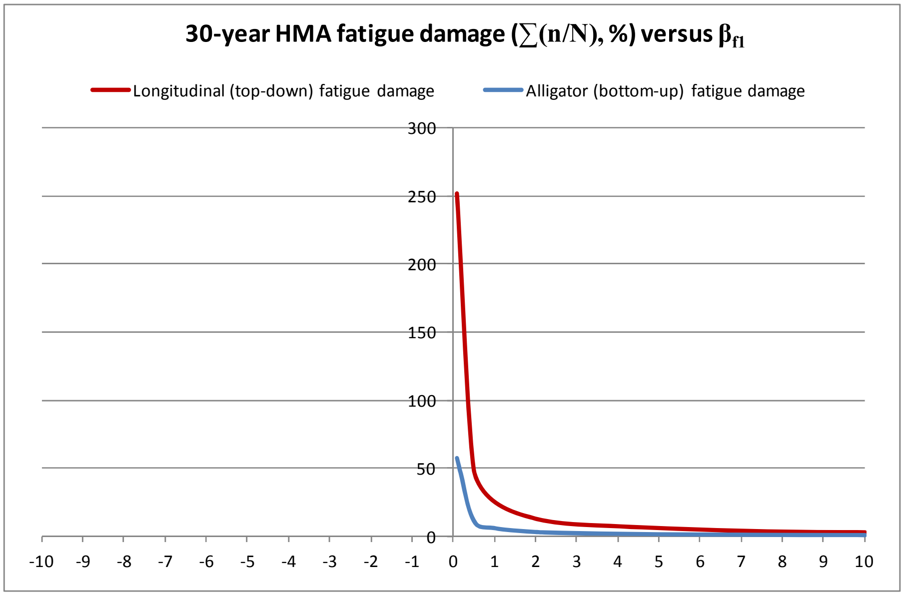

Figure 5.14 MEPDG Predicted Fatigue Damage versus Calibration Factor β_{f1}

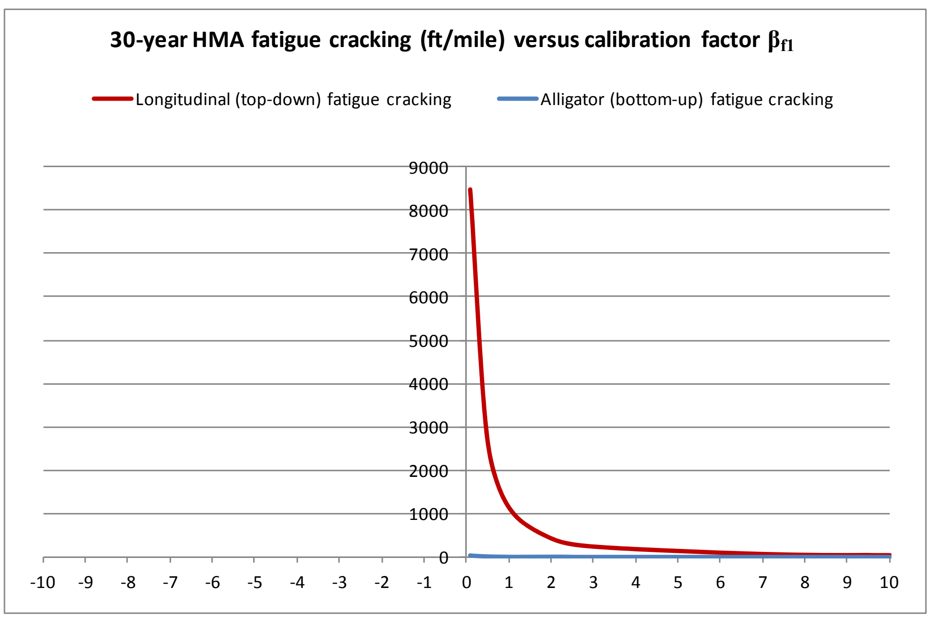

Figure 5.15 MEPDG Predicted Fatigue Cracking versus Calibration Factor β_{f1}

The allowable range of values for β_{f2} is -10 to 10, but the MEPDG software only predicts realistic values for fatigue damage and cracking for a very narrow range of β_{f2} values around the default value of 1.0, as shown in Table 5.1. In the analyses conducted for this study, the software failed to reach a solution when values less than 0 were entered for β_{f2}. Positive values of β_{f2} close to 0 produce extraordinarily high predicted values of fatigue damage, and positive values of β_{f2} greater than 1 produce extraordinarily small predicted values of fatigue damage.

Table 5.1 30-year Predicted Fatigue Damage and Cracking as a Function of Calibration Factor β_{f2}

β_{f2}	Longitudinal (top-down)		Alligator (bottom-up)	
	Damage (%)	Cracking (ft/mile)	Damage (%)	Cracking (% area)
-10	*software did not reach a solution*			
-1	*software did not reach a solution*			
0	2 E+21	10,600	8.72 E+17	100
0.5	6.73 E +10	10,600	8.93 E+8	100
0.9	1,740	10,400	225	71.1
1	25.2	1160	5.7	3.96
1.1	0.379	2.2	0.151	0.007
2	5.31 E-17	0	3.75 E-15	0

The allowable range of values for β_{f3} is -10 to 10, but, as with β_{f2}, the MEPDG software only predicts realistic values for fatigue damage and cracking for a very narrow range of β_{f3} values around the default value of 1.0, as shown in Table 5.2.

Table 5.2 30-year Predicted Fatigue Damage and Cracking as a Function of Calibration Factor β_{f3}

β_{f2}	Longitudinal (top-down)		Alligator (bottom-up)	
	Damage (%)	Cracking (ft/mile)	Damage (%)	Cracking (% area)
-10	2.4 E-81	0	1.83 E-82	0
-1	1.61 E-14	0	3.24 E-15	0
0	5.65 E-7	0	1.24 E-7	0
0.5	0.00347	0	0.000773	0
0.9	4.26	86.5	0.965	0.57
1	25.2	1160	5.7	3.96
1.1	149	6840	33.8	23
2	1.65 E+9	10,600	3.66 E+9	100

5.2.1 Longitudinal (Top-Down) Cracking in HMA Pavements

The top-down cracking model (Equation 5.13) appears in the MEPDG software as shown in Figure 5.16. Note that a C_3 term is shown with a value of 0, but there is no C_3 term in the model.

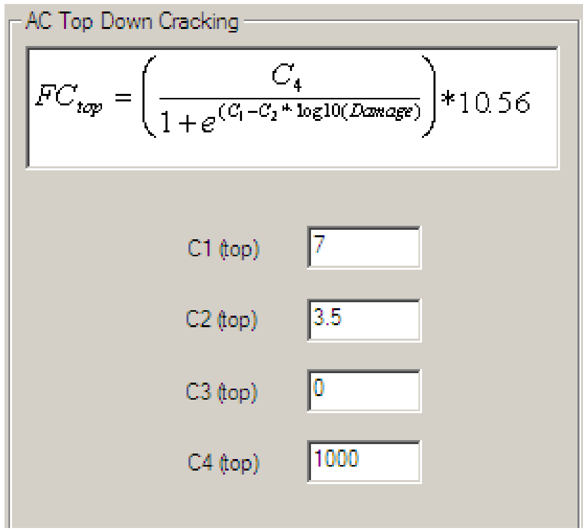

Figure 5.16 Longitudinal (Top-Down) HMA Cracking Model in the MEPDG Software

The values of C_1, C_2, and C_4 terms in this model can be varied as part of an agency's local calibration efforts. The MEPDG software does not appear to place any limits on the ranges over which the values of these terms can be varied. The sensitivity of predicted top-down cracking to the C_1, C_2, and C_4 terms is illustrated in Figures 5.17, 5.18, and 5.19, respectively.

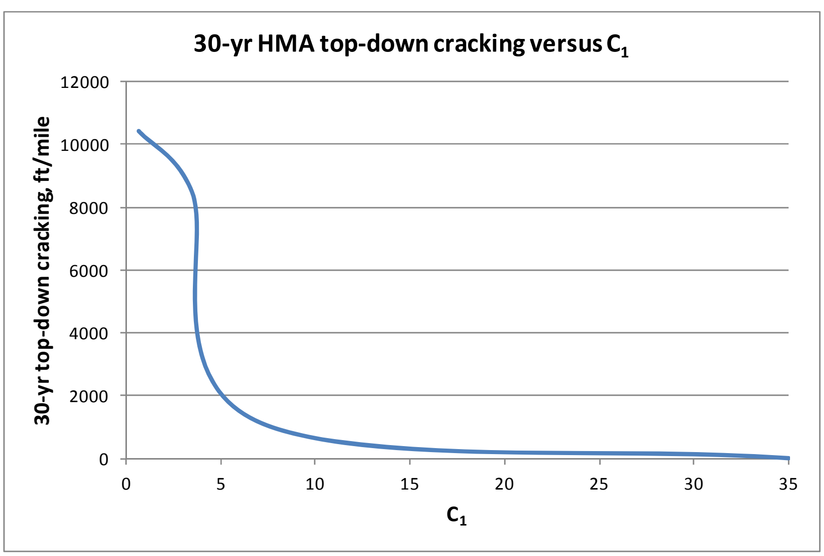

Figure 5.17 Predicted 30-Year Top-Down Cracking versus C_1

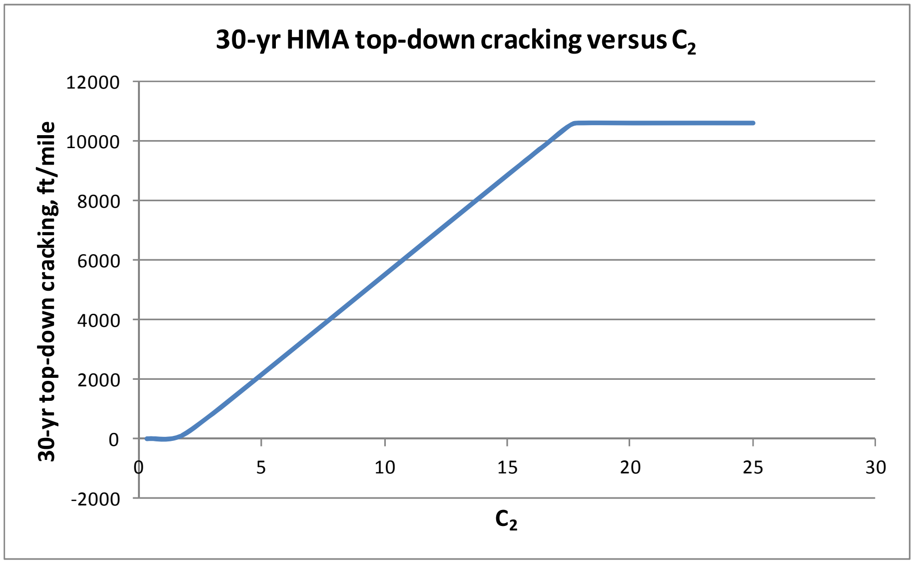

Figure 5.18 Predicted 30-Year Top-Down Cracking versus C_2

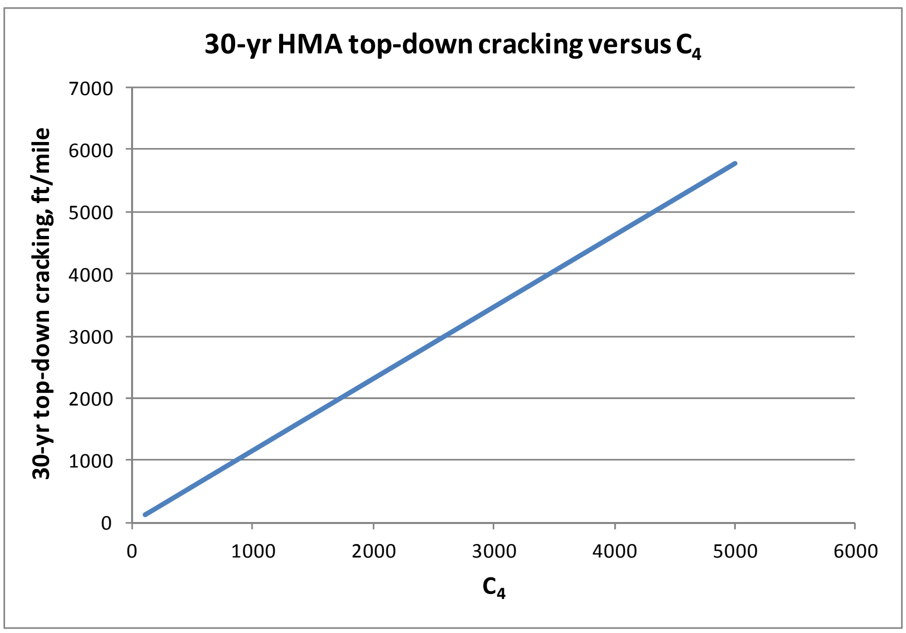

Figure 5.19 Predicted 30-Year Top-Down Cracking versus C_4

The results of the sensitivity analysis of longìtudinal (top-down) HMA cracking conducted for NCHRP Report 372 are illustrated in Figure 5.20. The prediction of longitudinal cracking is extremely sensitive to the delta (δ) and alpha (α) parameters in the E* model (Equation 5.2) and the HMA thickness. The next most sensitive inputs are the traffic volume (AADTT), subgrade resilient modulus, HMA air voids, HMA effective binder volume, and base resilient modulus.

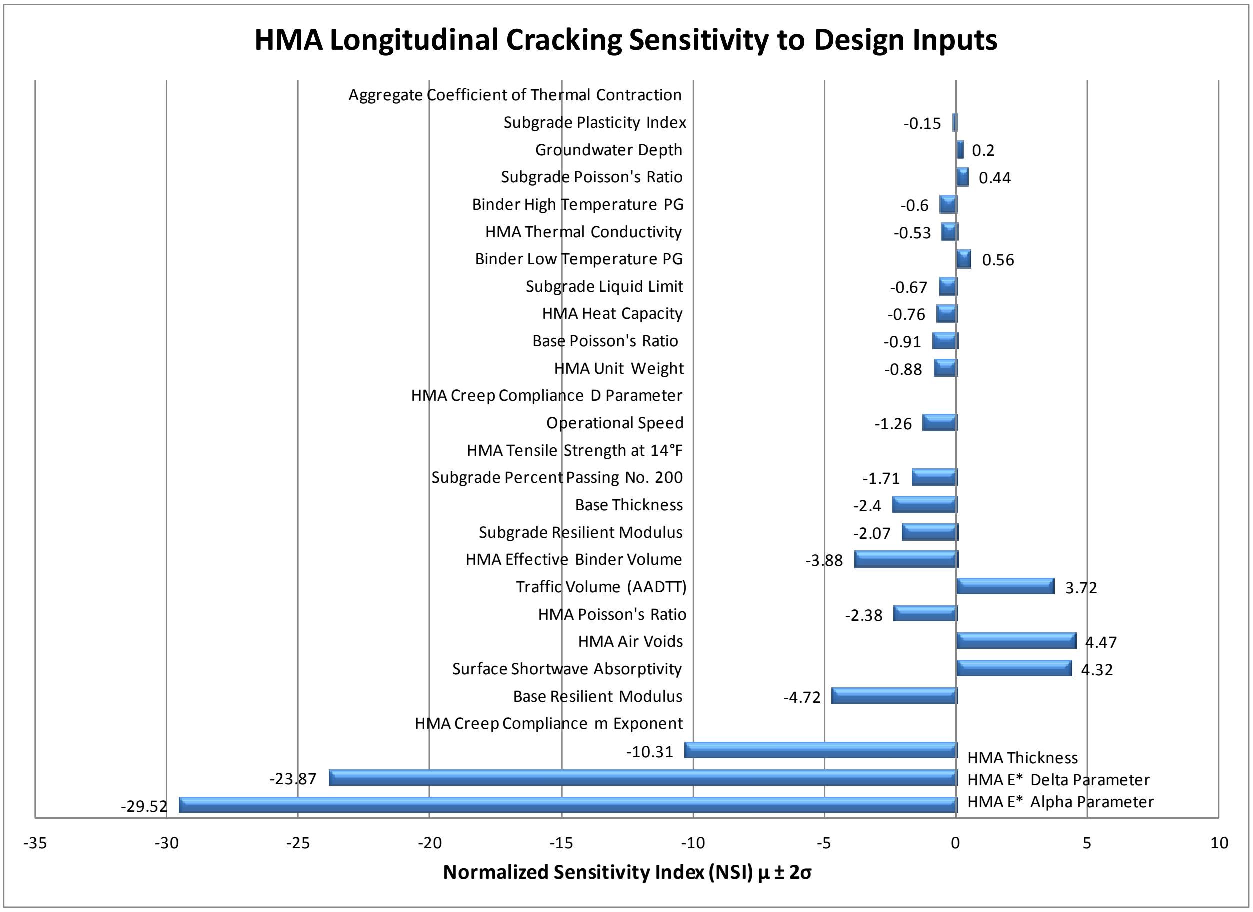

Figure 5.20 MEPDG HMA Longitudinal (Top-Down) Cracking Sensitivity to Design Inputs (Data from NCHRP Report 372)

The remarks quoted from NCHRP Report 372 earlier concerning the sensitivity of the prediction of AC rutting to the delta (δ) and alpha (α) parameters in the E* model apply to the prediction of longitudinal (top-down) cracking as well. Concerning the HMA air voids and HMA effective binder volume parameters, NCHRP Report 372 remarks,

> *"The high sensitivities for HMA air voids and HMA effective binder volume are in addition to any influence they many have on HMA dynamic modulus and/or low-temperature strength and creep compliance. The simulations [conducted for NCHRP Report 372] used synthetic Level 1 inputs for the HMA dynamic modulus and low-temperature properties. Formulating these properties in terms of the Level 3 empirical relations would increase the sensitivities attributable to HMA air voids and effective binder volume."*

5.2.2 Alligator (Bottom-Up) Cracking in HMA Pavements

The bottom-up cracking model (Equation 5.14) appears in the MEPDG software as shown in Figure 5.21. As with the prediction of top-down cracking, there is no C_3 term in the model, and for bottom-up cracking, no C_3 term appears on the MEPDG software screen.

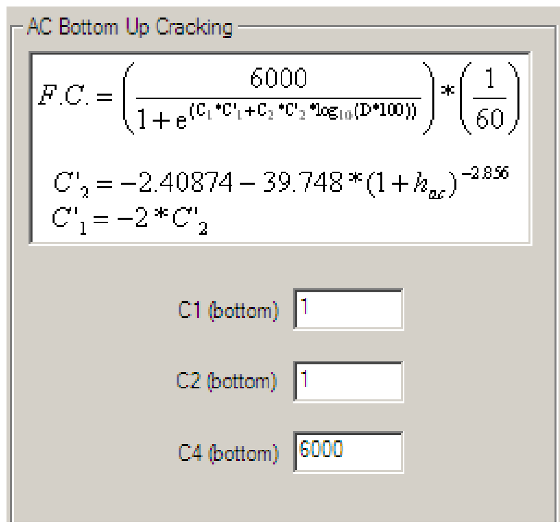

Figure 5.21 Alligator (Bottom-Up) HMA Cracking Model in the MEPDG Software

The values of the C_1, C_2, and C_4 terms in this model can be varied as part of an agency's local calibration efforts. The MEPDG software does not appear to place any limits on the ranges over which the values of these terms can be varied, although program execution times do increase noticeably for values considerably different from the default values, and some extreme values will produce program execution error messages. The sensitivity of predicted top-down cracking to the C_1, C_2, and C_4 terms is illustrated in Figures 5.22, 5.23, and 5.24, respectively.

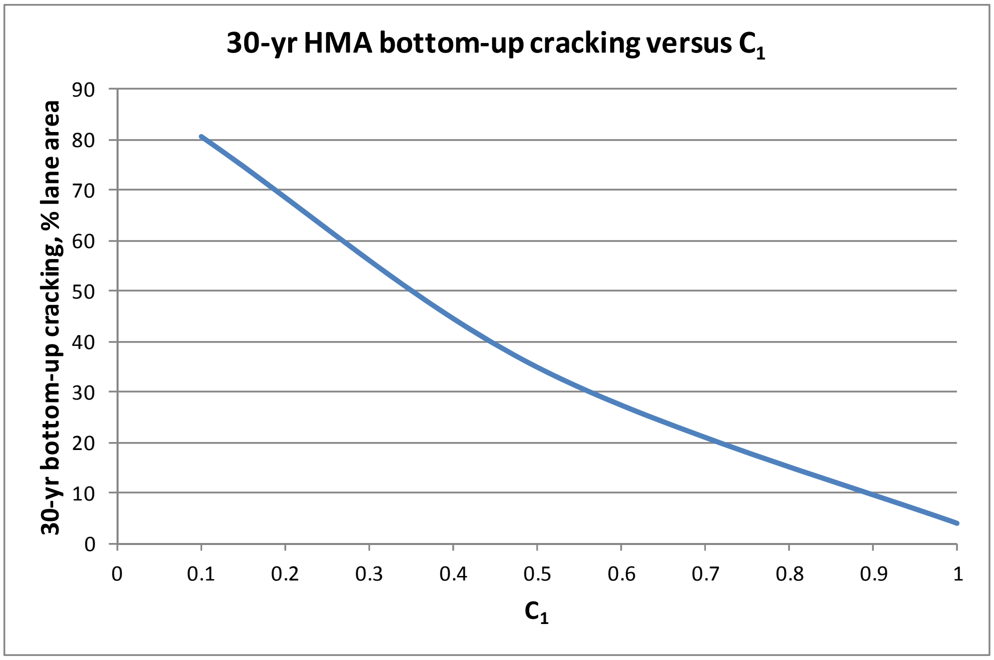

Figure 5.22 Predicted 30-Year Bottom-Up Cracking versus C_1

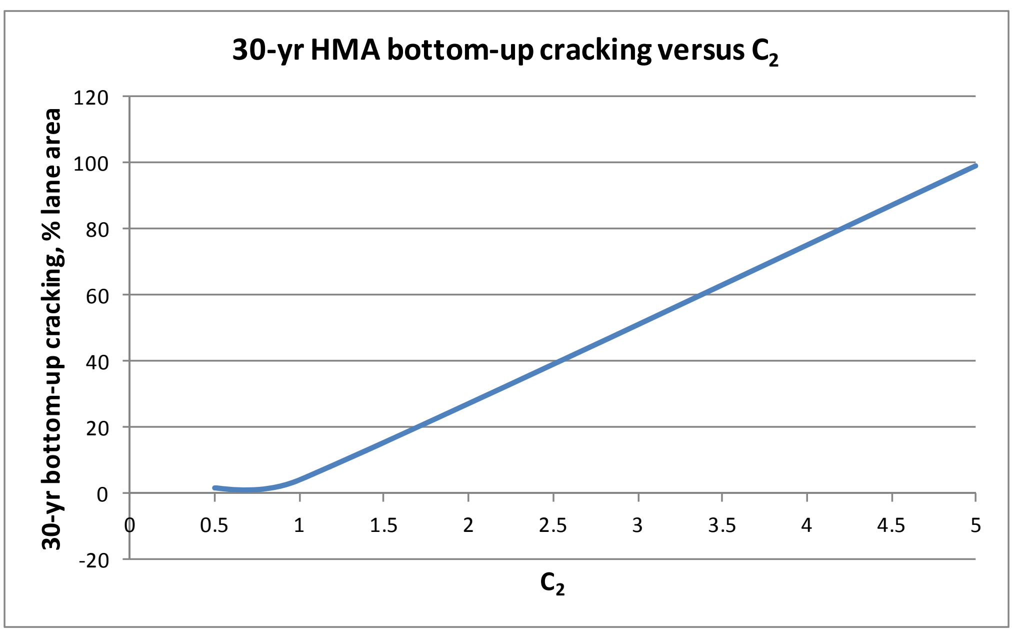

Figure 5.23 Predicted 30-Year Bottom-Up Cracking versus C_2

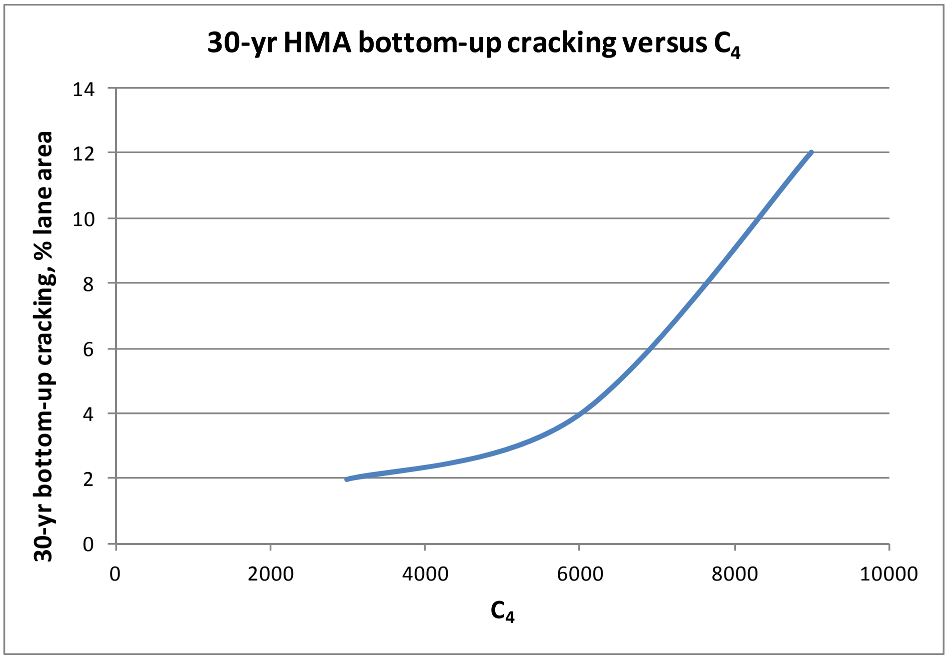

Figure 5.24 Predicted 30-Year Bottom-Up Cracking versus C_4

The results of the sensitivity analysis of alligator (bottom-up) HMA cracking conducted for NCHRP Report 372 are illustrated in Figure 5.25. The prediction of alligator cracking is very sensitive to the delta (δ) and alpha (α) parameters in the E* model (Equation 5.2) and the HMA thickness. The next most sensitive inputs are the traffic volume (AADTT), subgrade resilient modulus, HMA air voids, HMA effective binder volume, and base resilient modulus.

The remarks quoted from NCHRP Report 372 earlier concerning the sensitivity of the prediction of AC rutting and longitudinal (top-down) cracking to the delta (δ) and alpha (α) parameters in the E* model apply to the prediction of alligator (bottom-up) cracking as well, as do the remarks concerning the HMA air voids and HMA effective binder volume parameters quoted previously with respect to longitudinal (top-down) cracking.

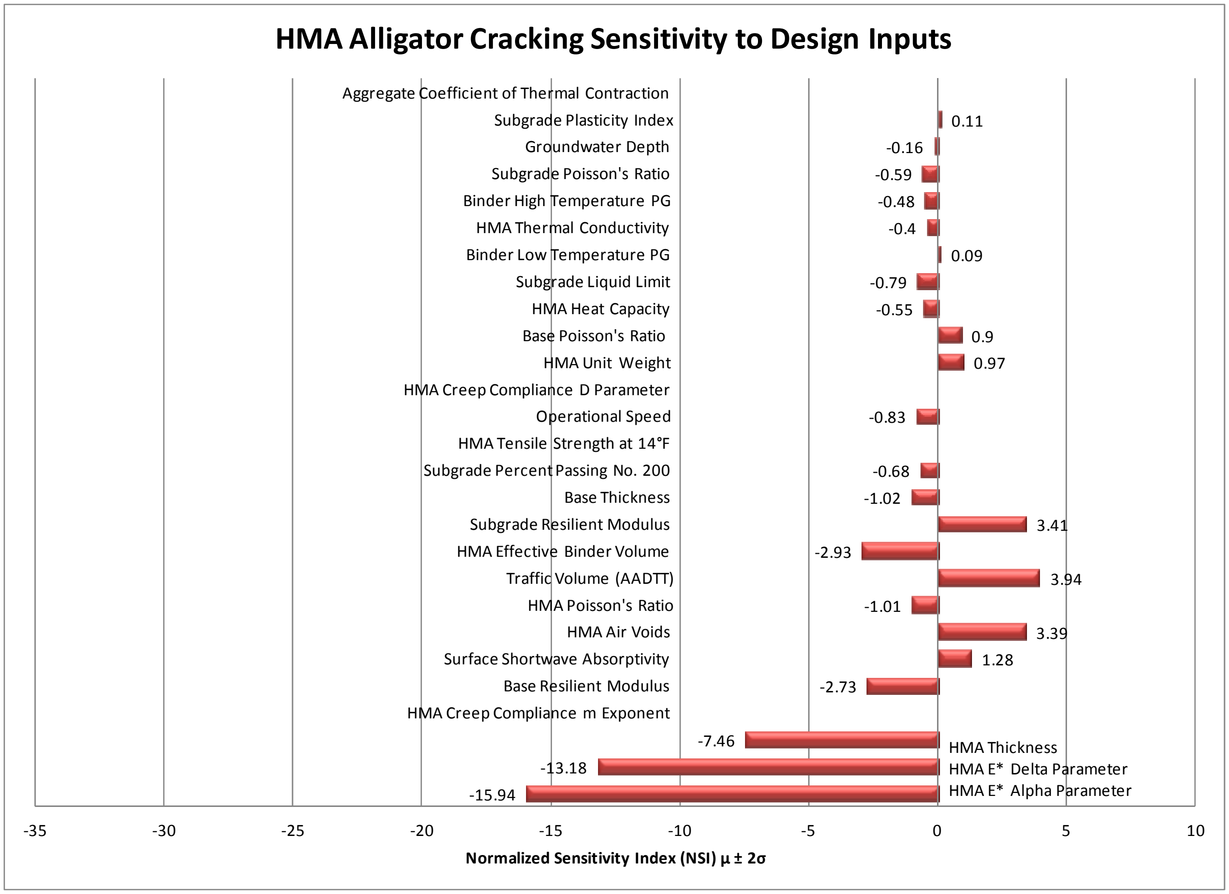

Figure 5.25 MEPDG HMA Alligator (Bottom-Up) Cracking Sensitivity to Design Inputs (Data from NCHRP Report 372)

5.3 Thermal Cracking in HMA Pavements

The general form of MEPDG model for the amount of transverse thermal cracking in an HMA pavement relates the probability distribution of the log of the ratio of the crack depth to the HMA layer thickness and the percent of cracking as follows:

$$TC = \beta_{t1} * N\left[\frac{\log C/_{h_{ac}}}{\sigma}\right] \tag{5.17}$$

where:

- TC = observed amount of thermal cracking, ft/500 ft
- βt_1 = regression coefficient determined through field calibration, fixed at 400
- $N(z)$ = standard normal distribution evaluated at z
- σ = standard deviation of the log of the depth of cracks in the pavement
- C = crack depth
- h_{ac} = thickness of asphalt layer

The crack depth, C, is determined using the Paris law of crack propagation to determine the amount of crack propagation induced by a thermal cooling cycle, as given by the following formula:

$$\Delta C = A\,(\Delta K)^n \tag{5.18}$$

where:

ΔC	=	change in crack depth due to a cooling cycle
ΔK	=	change in the stress intensity factor due to a cooling cycle
A, n	=	fracture parameters for the AC mixture

Estimates for A and n can be obtained from the indirect tensile creep compliance and strength of the HMA, using the following two formulas:

$$A = 10^{k_i\,\beta_t\,[4.389 - 2.52\log(E\,\sigma_m\,n)]} \tag{5.19}$$

where:

$$n = 0.8\left(1 + \frac{1}{m}\right) \tag{5.20}$$

k_i	=	field calibration determined for each input level
σ_m	=	mixture tensile strength, psi
β_t	=	local or mixture calibration factor

The thermal cracking model and related equations appear in the MEDPG software as shown in Figure 5.26.

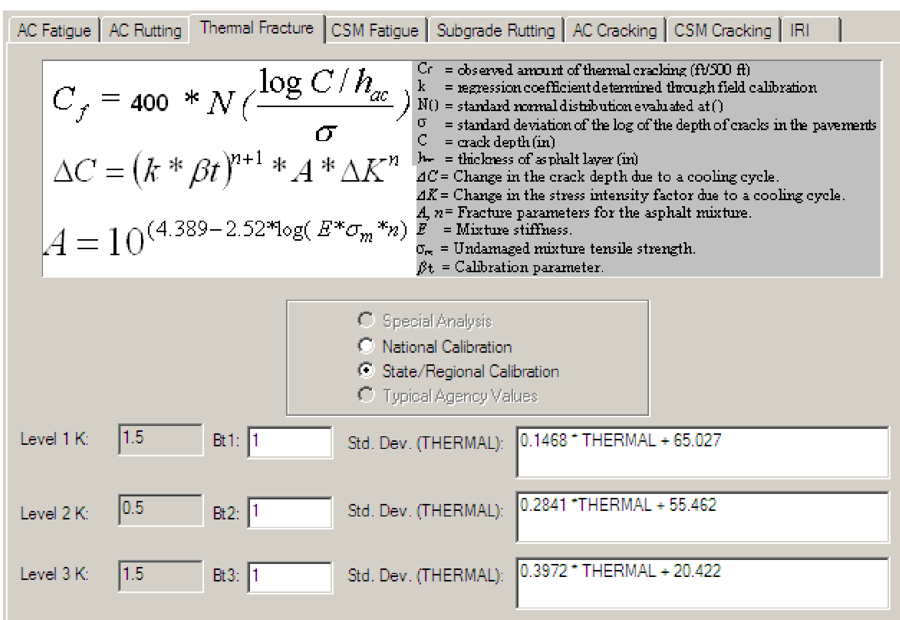

Figure 5.26 Thermal Cracking Model in the MEPDG Software

Agency calibration of the thermal cracking model is accomplished by varying the β_{ti} term that corresponds to the selected level of design. The sensitivity of predicted thermal cracking to the β_{t3} term for Level 3 design cannot be determined for the trial site (Wausau, Wisconsin) used for these analyses, because no thermal cracking is predicted over the 30-year analysis for this site, as shown in Figure 5.27. As explained below, in the sensitivity analyses conducted for NCHRP Report 372, magnitudes of thermal cracking sufficient to assess the sensitivity of the MEPDG thermal cracking model could only be generated by the MEPDG software by modelling an HMA mix with inappropriate binder characteristics for the site. Whatever the shape of the curve of the sensitivity of predicted thermal cracking to the β_{t3} term would be for such conditions, the shapes of the sensitivity curves for the β_{t1} and β_{t2} terms for Level 1 and Level 2 would be the same, since they enter into the thermal cracking model in the same way.

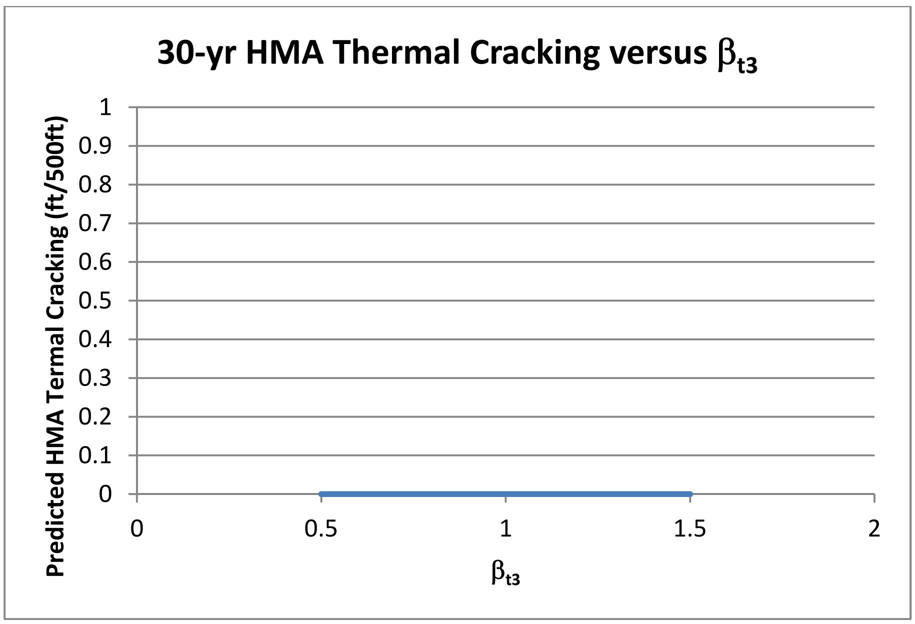

Figure 5.27 Sensitivity of Predicted Thermal Cracking to β_{t3} Calibration Factor

The results of the NCHRP Report 372 sensitivity analysis of the MEPDG thermal cracking model are illustrated in Figure 5.28. It is important to note how levels of thermal cracking sufficient to assess the sensitivity of the model to its inputs were generated in the analyses conducted for NHCRP Report 372:

> *"Little or no thermal cracking was predicted when using the correct binder grade recommended by LTPPBind (98% reliability). The low-temperature binder grade had to be shifted 2 to 3 grades stiffer (warmer) in order to generate sufficient thermal cracking distress for evaluating the sensitivity metrics."*

The only inputs to the MEPDG thermal cracking model to which it displays a notable sensitivity are the HMA creep compliance parameter m, the HMA E* delta parameter, and the HMA tensile strength. With respect to the creep compliance parameter and tensile strength, NCHRP Report 372 remarks:

> *"The key HMA low-temperature properties (tensile strength, creep compliance) are correlated with other HMA and binder properties. The low-temperature creep compliance in particular is correlated, albeit in a complex way, with dynamic modulus."*

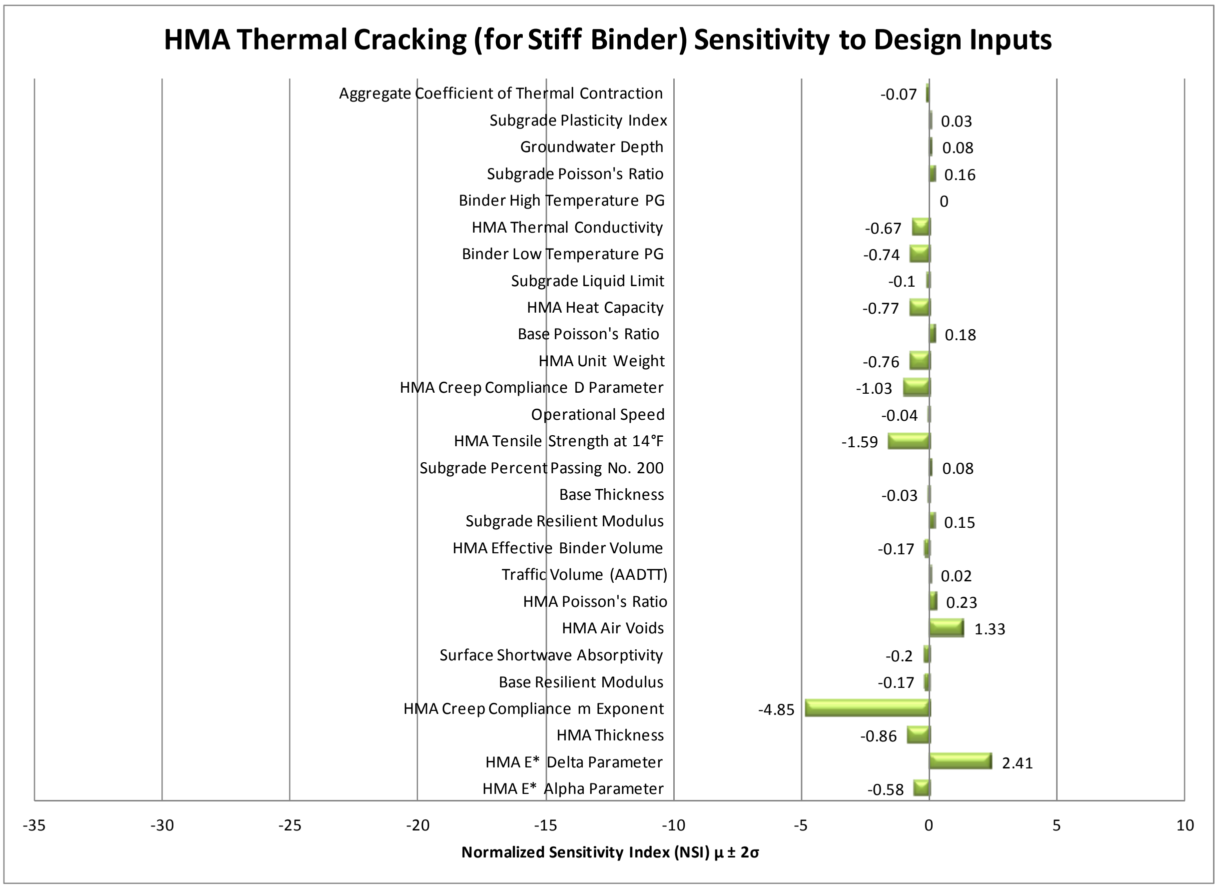

Figure 5.28 MEPDG Thermal Cracking Sensitivity to Design Inputs
(Data from NCHRP Report 372)

With respect to the differences in the sensitivity analysis results for the thermal cracking model and for the other HMA performance models, NCHRP Report 372 remarks:

> *"The most sensitive design inputs for longitudinal cracking, alligator cracking, AC rutting, total rutting, and IRI had very little overlap with the most sensitive design inputs*

for thermal cracking. Clearly this is because the former are primarily load-related distresses while thermal cracking is exclusively environment-driven."

With respect to the sensitivity of the thermal cracking model to the E* delta parameter, NCHRP Report 372 remarks:

"The sensitivity of thermal cracking to the HMA E delta parameter (lower shelf of the dynamic modulus master curve) is larger in absolute value terms than its sensitivity to the HMA E* alpha parameter (the offset of the upper shelf above delta). In addition, the influence of HMA E* delta is positive; as the lower shelf stiffness increases (which also increases the upper shelf stiffness, for fixed HMA E* alpha), thermal cracking also increases. This appears contrary to engineering expectations."*

5.4 IRI in HMA Pavements

The MEPDG model for IRI in HMA pavements is as follows:

$$IRI = IRI_o + 40\,RD + 0.4\,FC + 0.008\,TC + 0.015\,SF \tag{5.21}$$

where:

IRI = predicted IRI, in/mile
IRI_o = initial IRI after construction, in/mile
RD = average rut depth, inches
FC = combined longitudinal and alligator cracking, percent of lane area
TC = transverse thermal cracking, ft/mile
SF = site factor, from Equation 5.22:

$$SF = Age\,[0.02003\ (PI+1) + 0.007947\,(Rain+1) + 0.000636\,(FI+1)\,] \tag{5.22}$$

Age = pavement age, years
PI = plasticity index of the subgrade soil
FI = average annual Freezing Index, Fahrenheit degree-days
Rain = Average annual rainfall, inches

The IRI model for HMA pavements appears in the MEPDG software as shown in Figure 5.29, which is to say, the model itself does not appear at all; only the coefficients for the rut depth, fatigue cracking, thermal cracking, and site factor terms in the IRI model appear.

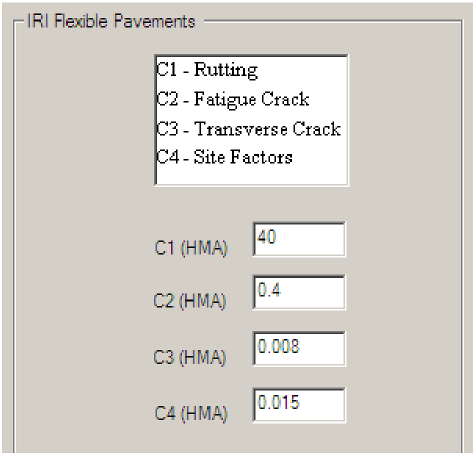

Figure 5.29 HMA IRI Model in the MEPDG Software

The sensitivity of predicted IRI in HMA pavement to the values of the C coefficients is illustrated in Figures 5.30, 5.31, 5.32, and 5.33. These figures show that the HMA IRI model is sensitive to the C_1 coefficient for rut depth and the C_2 coefficient for fatigue cracking. The model displays no sensitivity to the C_3 coefficient for thermal cracking–at least, for this example site in Wausau, Wisconsin, for which, assuming climate-appropriate binder properties, no thermal cracking is predicted. The IRI model displays a very slight nonzero sensitivity to the C4 coefficient for the site factor. These results suggest that an agency's efforts at local calibration of the HMA IRI model should focus on varying the C_1 and C_2 coefficients only.

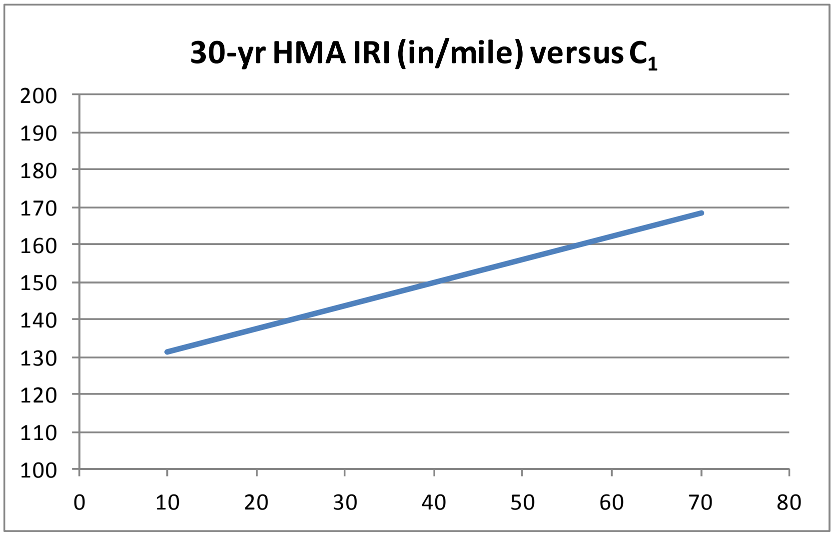

Figure 5.30 Sensitivity of Predicted IRI in HMA Pavement to C_1

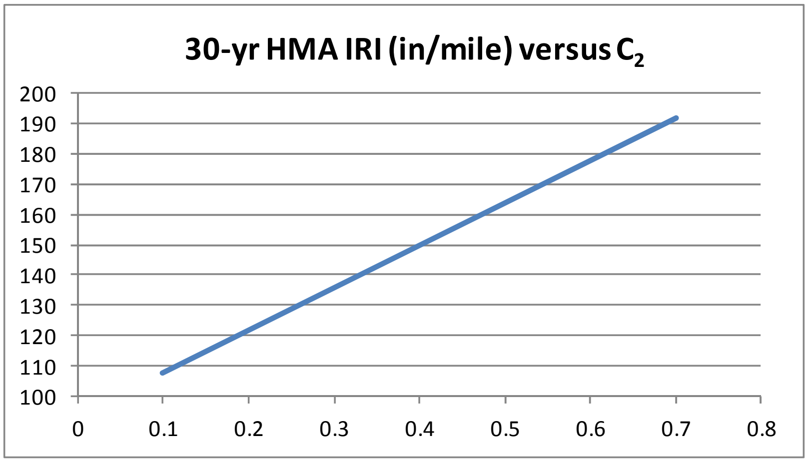

Figure 5.31 Sensitivity of Predicted IRI in HMA Pavement to C_2

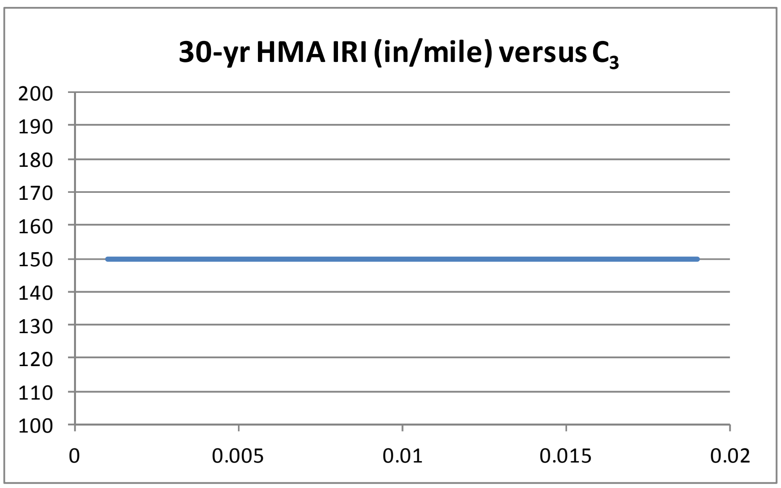

Figure 5.32 Sensitivity of Predicted IRI in HMA Pavement to C_3

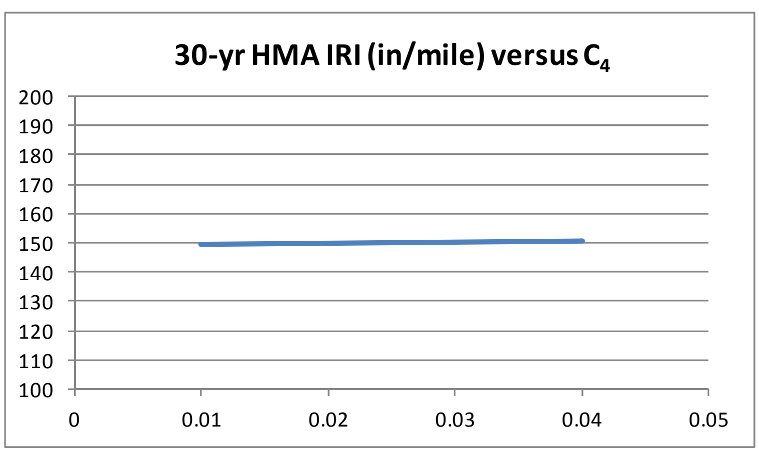

Figure 5.33 Sensitivity of Predicted IRI in HMA Pavement to C_4

The results of the NCHRP Report 372 sensitivity analysis of the MEPDG HMA IRI model are illustrated in Figure 5.34. The results indicate that the prediction of IRI in HMA pavements is somewhat sensitive to the E* alpha and delta parameters, and to a lesser extent, to the HMA thickness, surface shortwave absorptivity, traffic volume (AADTT), and subgrade resilient modulus. It is only slightly sensitive, if at all, to the other factors listed in Figure 5.34.

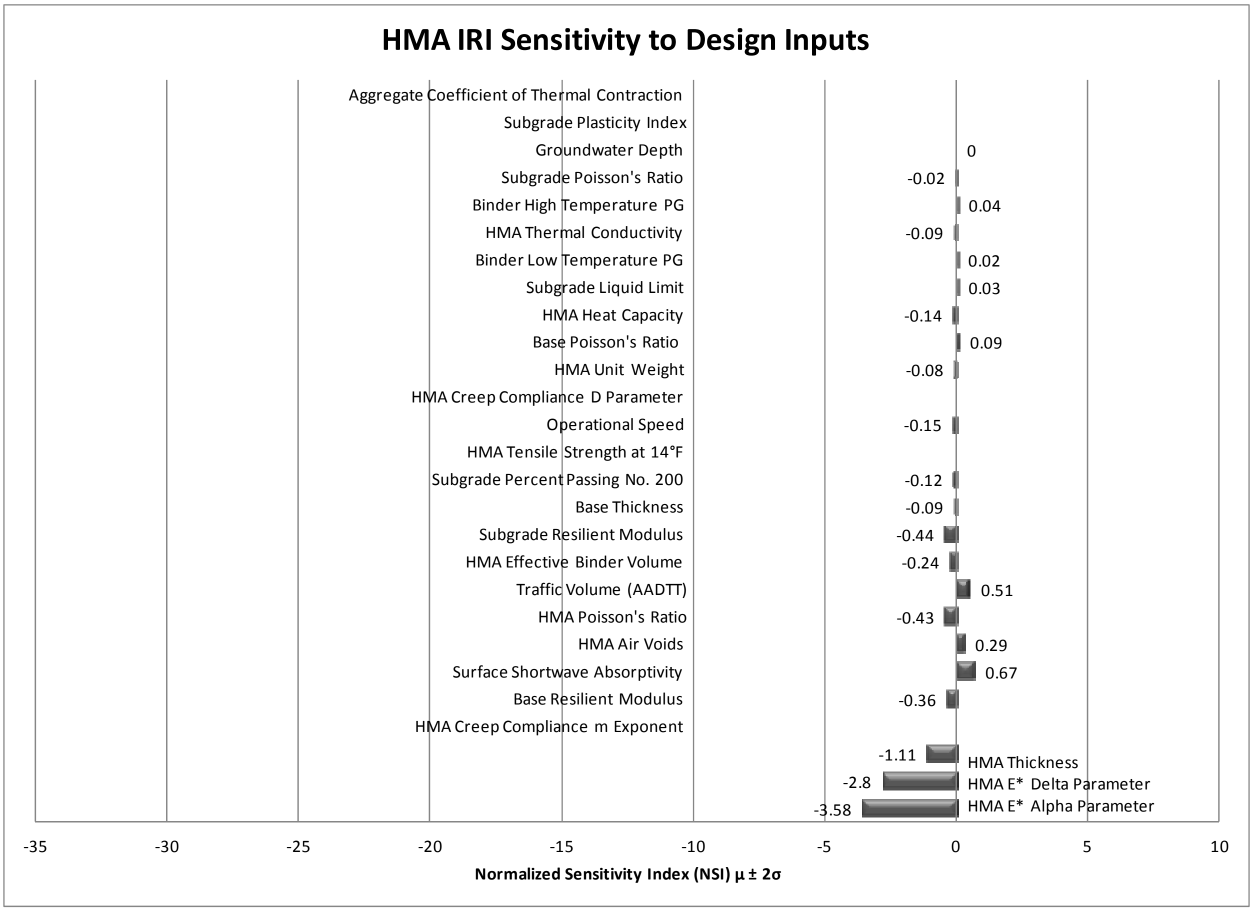

Figure 5.34 MEPDG HMA IRI Sensitivity to Design Inputs
(Data from NCHRP Report 372)

5.5 Joint faulting in JPCP

Joint faulting in JPCP is predicted in the MEPDG software by computing the faulting predicted to occur in each month and summing the incremental monthly faulting amounts. The MEPDG model for mean joint faulting in JPCP is as follows:

$$Fault_m = \sum_{i=1}^{m} \Delta Fault_i \tag{5.23}$$

$$\Delta Fault_i = C_{34} * (FAULTMAX_{i-1} - FAULT_{i-1})^2 * DE_i \quad (5.24)$$

$$FAULTMAX_i = FAULTMAX_0 + C_7 * \sum_{j=1}^{m} DE_j * log(1 + C_5 * 5^{EROD})^{C_6} \quad (5.25)$$

$$FAULTMAX_0 = C_{12} * \delta_{curling} * \left[\log(1 + C_5 * 5^{EROD}) * \log\left(\frac{P_{200} * WetDays}{p_s}\right)\right]^{C_6} \quad (5.26)$$

where:

$FAULT_m$	=	mean joint faulting at the end of month m, inches
$\Delta FAULT_i$	=	incremental monthly change in mean transverse joint faulting during month i, inches
$FAULTMAX_i$	=	maximum mean transverse joint faulting for month i, inches
$FAULTMAX_0$	=	initial maximum mean transverse joint faulting, inches
EROD	=	base/subbase erodibility factor
DE_i	=	differential deformation energy accumulated during month i
$\delta_{curling}$	=	maximum mean monthly upward corner deflection of concrete slab due to temperature curling and moisture warping
P_{200}	=	percent subgrade material passing the No. 200 sieve
WetDays	=	annual number of wet days (greater than 0.1 inch of rainfall)
p_s	=	overburden on subgrade, lb
C_{12}	=	$C_1 + C_2 * FR^{0.25}$
C_{34}	=	$C_3 + C_4 * FR^{0.25}$
FR	=	base freezing index, defined as the percentage of time that the temperature at the top of the base is below freezing (32°F)

The faulting model appears in the MEPDG software as shown in Figure 5.35. The values shown for the coefficients C_1 through C_8 are those obtained after national recalibration of the model. Note that the coefficient C_8 is a calibration factor in an internal model for incremental dowel damage. The sensitivity of predicted JPCP faulting to the coefficients C_1 through C_8 is illustrated in Figures 5.36 through 5.43. The JPCP faulting model is more sensitive to the values assigned to C_1, C_2, and C_6 than to the values assigned to the other five C_i coefficients.

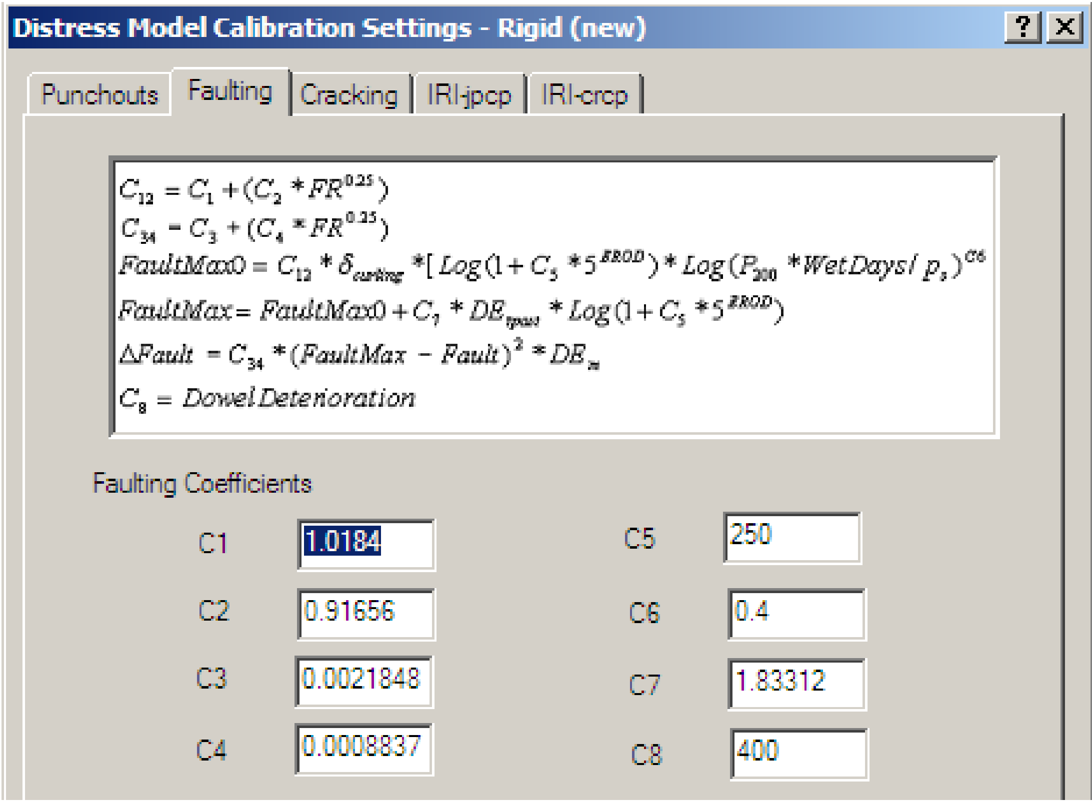

Figure 5.35 JPCP Faulting Model in the MEPDG Software

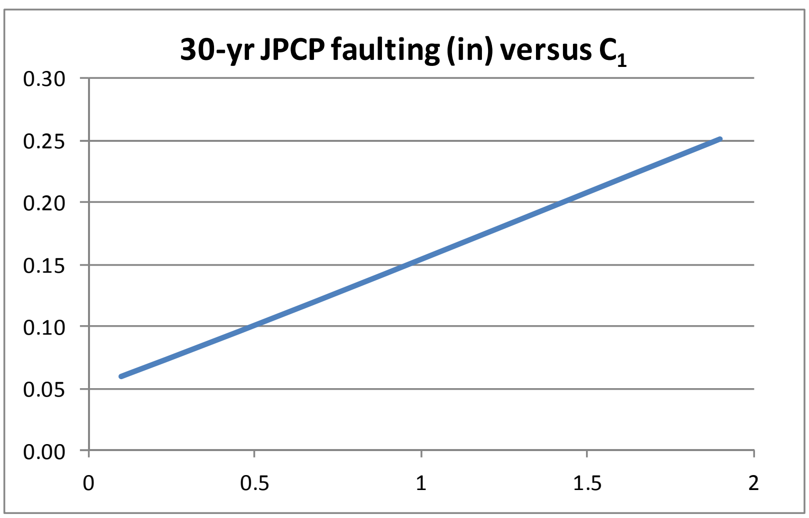

Figure 5.36 Sensitivity of JPCP Faulting to C_1

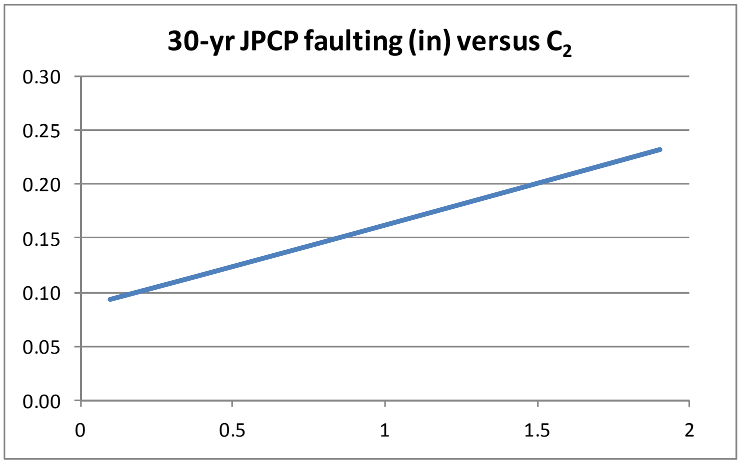

Figure 5.37 Sensitivity of JPCP Faulting to C_2

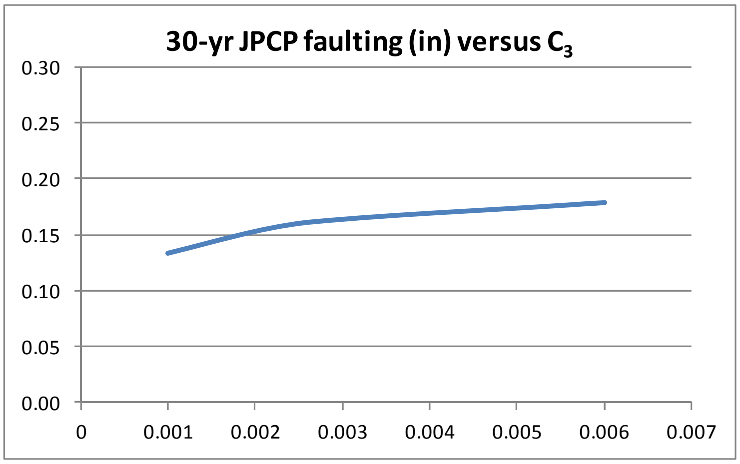

Figure 5.38 Sensitivity of JPCP Faulting to C_3

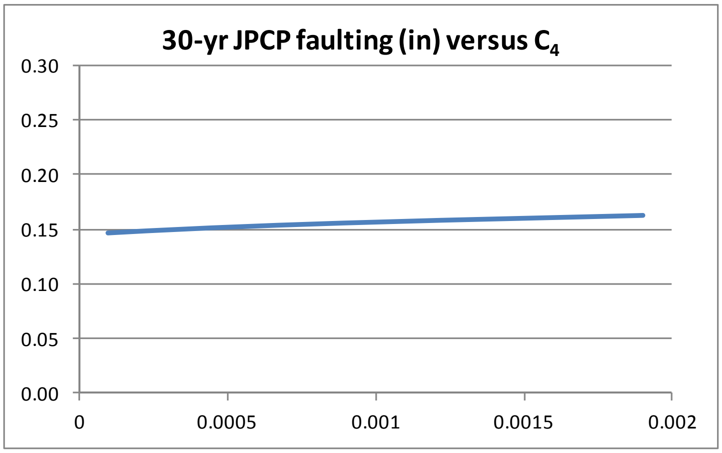

Figure 5.39 Sensitivity of JPCP Faulting to C_4

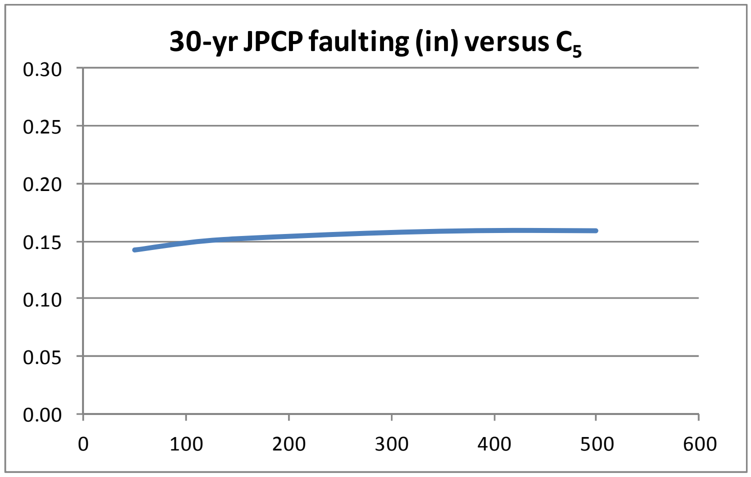

Figure 5.40 Sensitivity of JPCP Faulting to C_5

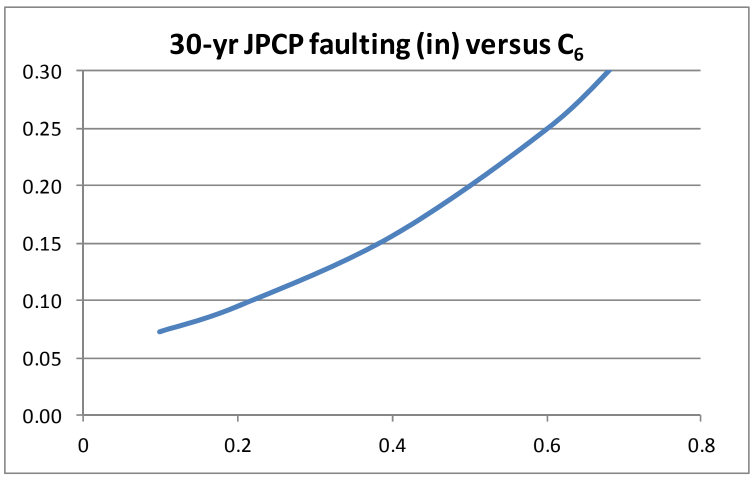

Figure 5.41 Sensitivity of JPCP Faulting to C_6

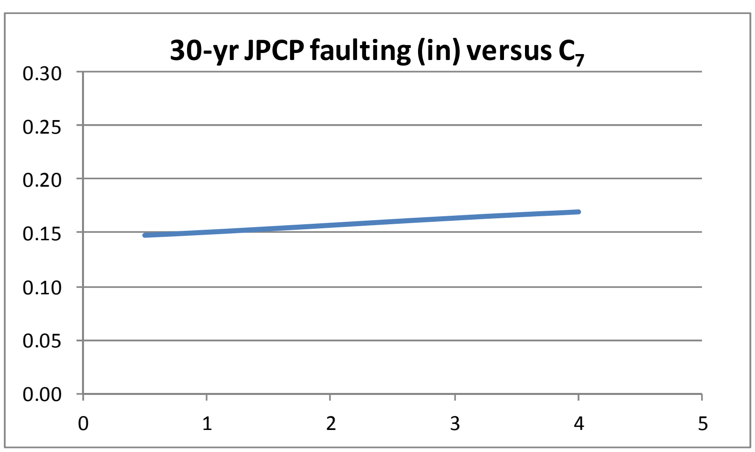

Figure 5.42 Sensitivity of JPCP Faulting to C_7

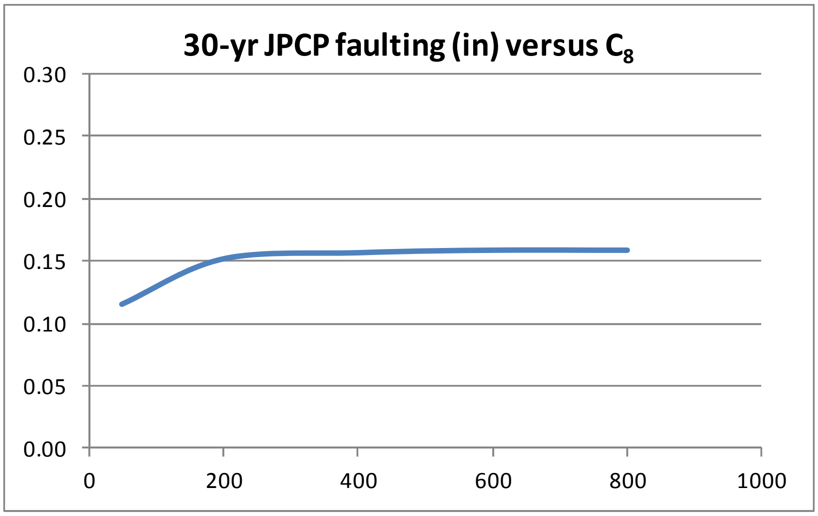

Figure 5.43 Sensitivity of JPCP Faulting to C_8

The results of the NCHRP Report 372 sensitivity analysis of the MEPDG JPCP faulting model are illustrated in Figure 5.44. Concrete slab width is by far the most sensitive input in the prediction of JPCP faulting. The next most sensitive inputs are the concrete unit weight, the concrete coefficient of thermal expansion, and design lane width.

About the sensitivity of the unit weight of the concrete, NCHRP Report 372 remarks:

> *"PCC unit weight is an unexpectedly sensitive input. The PCC unit weight is an important factor in the calculation of critical responses in the rigid pavement structural response models employed in MEPDG through its influence on curling deflections (faulting) and curling stresses (transverse cracking)."*

5.6 Cracking in JPCP

The percentage of slabs with transverse cracks in a given traffic lane is predicted using the following model for both top-down and bottom-up cracking:

$$CRK = \frac{100}{1+ C_4\, FD^{C_5}} \tag{5.27}$$

where:

CRK = predicted percentage of slabs with top-down or bottom-up cracking

FD = fatigue damage:

$$FD = \sum \frac{n_{i,j,k,l,m,n}}{N_{i,j,k,l,m,n}} \tag{5.28}$$

$n_{i,j,k,l,m,n}$ = applied loads at age i, month j, axle type k, load level l, temperature difference m, and traffic path n

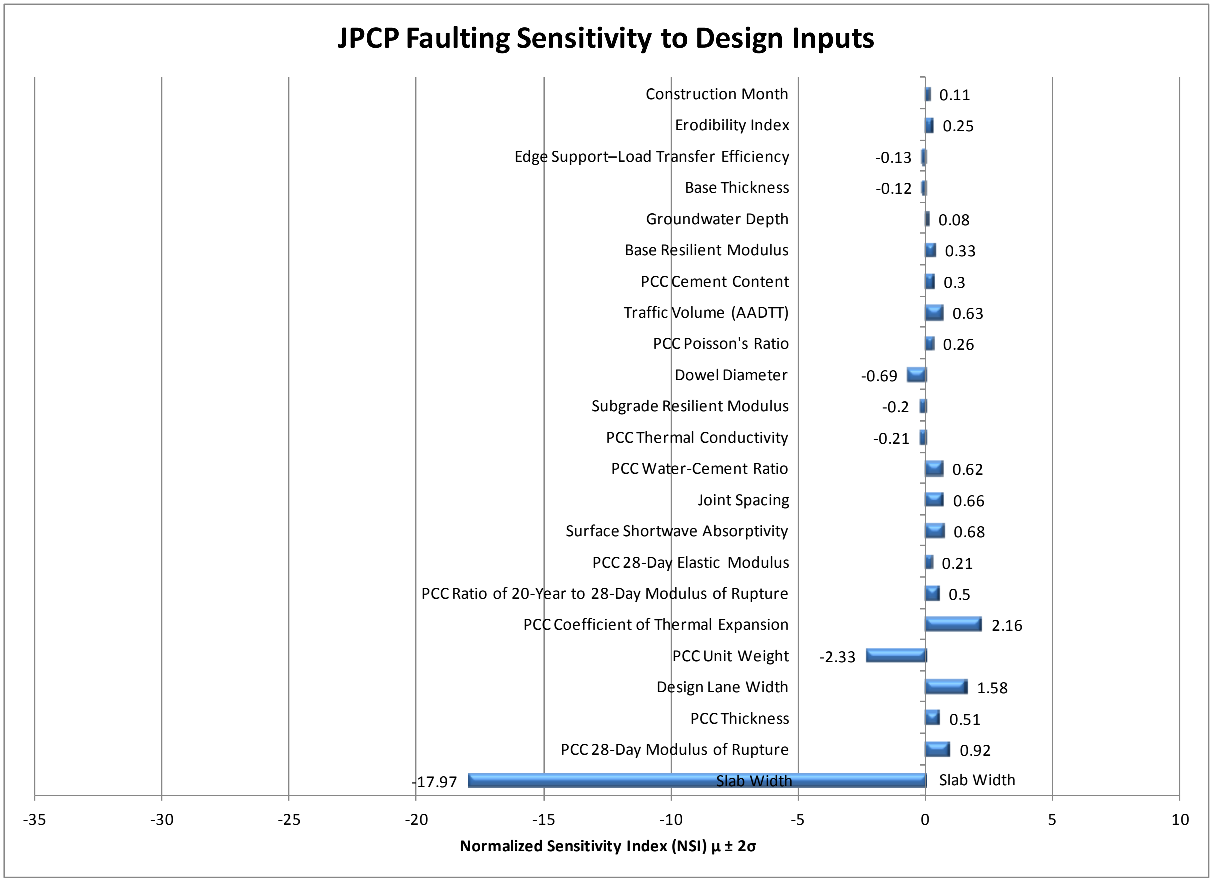

Figure 5.44 MEPDG JPCP Faulting Sensitivity to Design Inputs
(Data from NHCRP Report 372)

$N_{i,j,k,l,m,n}$ = allowable loads at age i, month j, axle type k, load level l, temperature difference m, and traffic path n

The fatigue model used to predict allowable loads is the following:

$$\log N = C_1 \left(\frac{MR_i}{\sigma_{i,j,k,l,m,n}} \right)^{C_2} \tag{5.29}$$

where:

MR_i = concrete modulus of rupture at age i, psi

$\sigma_{i,j,k,l,m,n}$ = applied stress at age i, month j, axle type k, load level l, temperature difference m, and traffic path n

The total accumulated top-down and bottom-up fatigue damage amounts are computed by summing the incremental amounts of top-down and bottom-up damage computed from the fatigue model, and these total fatigue damage amounts are used to compute the amount of top-down and bottom-up cracking predicted. The total amount of cracking predicted is computed as follows:

$$TCRACK = (CRK_{bu} + CRK_{td} - CRK_{bu} * CRK_{td}) * 100\% \quad (5.30)$$

where:

TCRACK = total percentage of slabs cracked

CRK_{bu} = predicted amount of bottom-up cracking (fraction)

CRK_{td} = predicted amount of top-down cracking (fraction)

The cracking model appears in the MEPDG software as shown in Figure 5.45. The values shown for the coefficients C_1, C_2, C_4, and C_5 are those obtained after national recalibration of the model. The sensitivity of predicted JPCP cracking to C_1, C_2, C_4, and C_5 is illustrated in Figures 5.46, 5.47, 5.48, and 5.49, respectively. Predicted JPCP cracking is very sensitive to the values assigned to the two damage model coefficients, C_1 and C_2, and is somewhat sensitive to the value assigned to the cracking model coefficient C_5, but is not very sensitive to the value assigned to the other cracking model coefficient, C_4.

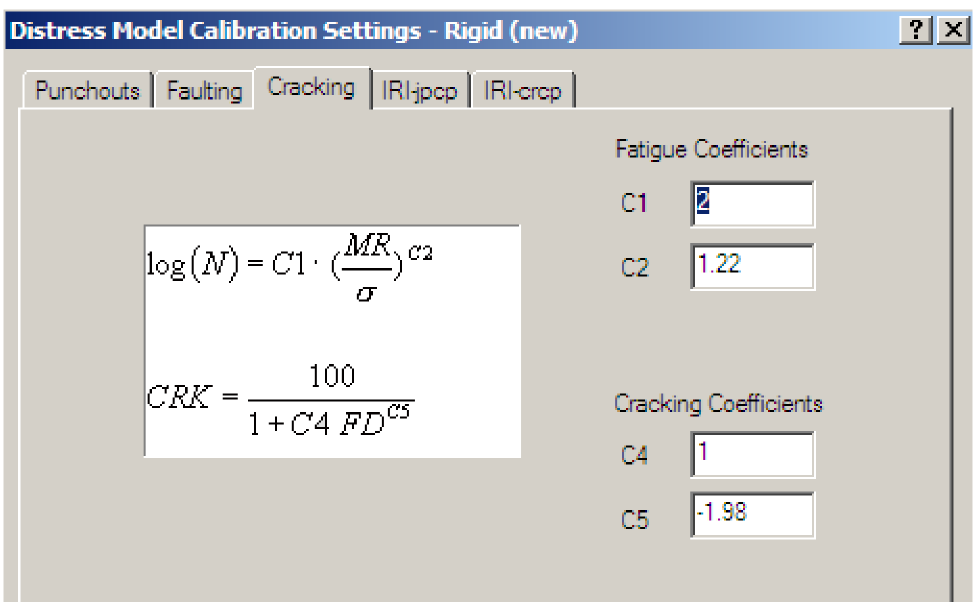

Figure 5.45 JPCP Cracking Model in the MEPDG Software

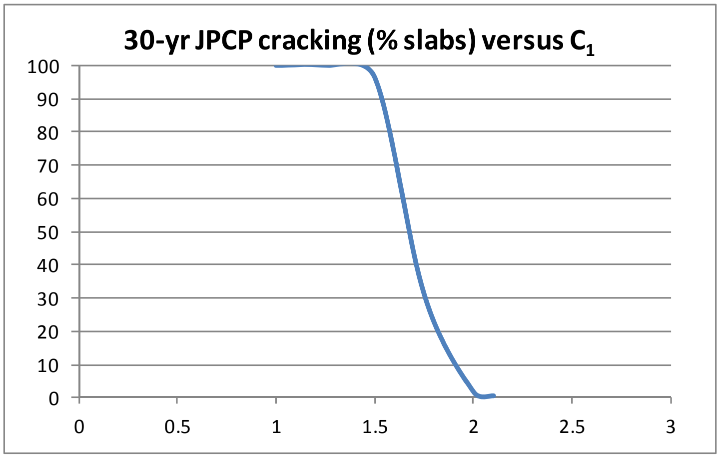

Figure 5.46 Sensitivity of Predicted JPCP Cracking to C_1

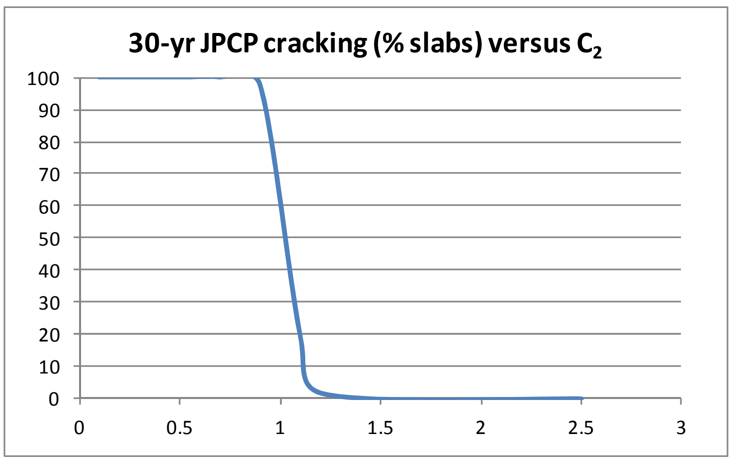

Figure 5.47 Sensitivity of Predicted JPCP Cracking to C_2

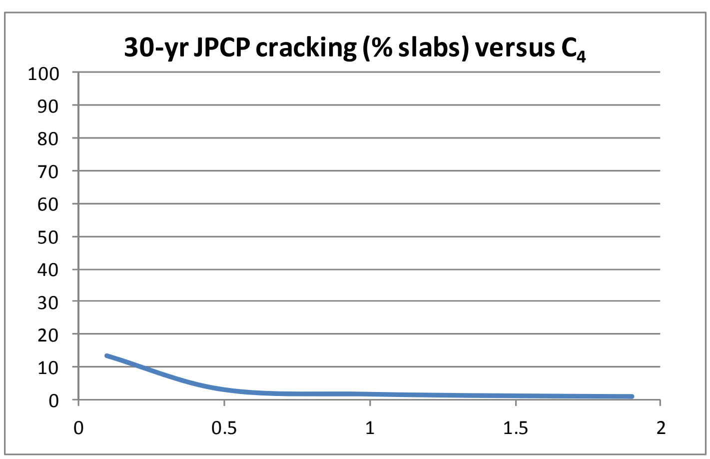

Figure 5.48 Sensitivity of Predicted JPCP Cracking to C_4

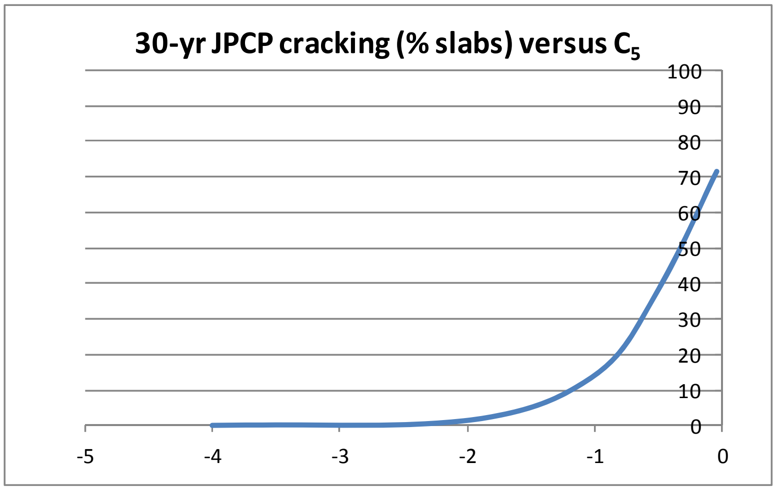

Figure 5.49 Sensitivity of Predicted JPCP Cracking to C_5

The results of the NCHRP Report 372 sensitivity analysis of the MEPDG JPCP cracking model are illustrated in Figure 5.50. The most sensitive inputs are the slab width, the concrete 28-day modulus of rupture, the slab thickness, the design lane width, the concrete unit weight, the concrete coeficient of thermal expansion, the ratio of the 20-year to the 28-day concrete modulus of rupture, the 28-day concrete elastic modulus, and the surface shortwave absorptivity.

About the sensitivity of the design lane width to the prediction of cracking, NCHRP Report 372 remarks:

> *"When interpreting the sensitivity of design lane width, it is important to note that it was evaluated under three different edge support conditions (no edge support, tied shoulder edge support with 80% LTE [load transfer efficiency] and widened slab edge support. Design lane width under widened slab edge support showed high sensitivity for transverse cracking but design lane width under either no edge support or tied shoulder edge support was not sensitive."*

5.7 IRI in JPCP

The MEPDG model for IRI in JPCP is as follows:

$$IRI = IRI_0 + 0.8203\, CRK + 0.4417\, SPALL + 1.4929\, TFAULT + 25.24\, SF \tag{5.31}$$

where:

IRI = predicted IRI, in/mile

IRI_o = initial IRI after construction, in/mile

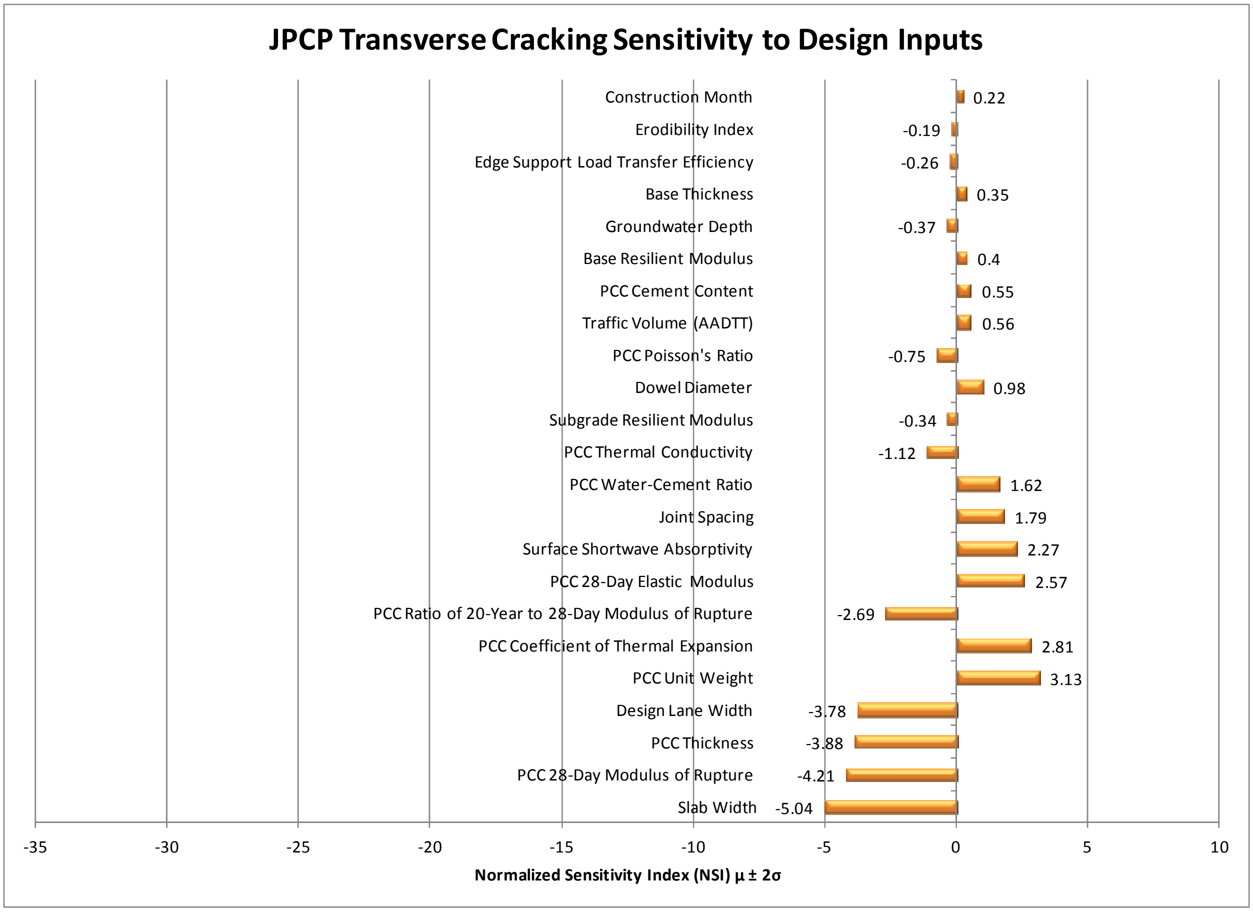

Figure 5.50 MEPDG JPCP Faulting Sensitivity to Design Inputs
(Data from NHCRP Report 372)

CRK = percentage of slabs with transverse cracks, all severities, from JPCP cracking model

SPALL = percentage of joints with medium- or high-severity spalling:

$$SPALL = \left(\frac{AGE}{AGE+0.01}\right)\left[\frac{100}{1+1.005^{(-12*AGE+SCF)}}\right] \tag{5.32}$$

SCF = spalling prediction scaling factor:

$$SCF = -1400 + 350\,AIR\,(0.5 + PREFORM) + 3.4\,{f_c{}'}^{0.4} - 0.2\,(FTCYC * AGE) + 43\,h_{pcc} - 536\,WCratio \tag{5.33}$$

AIR = concrete air content, percent, fixed in software at 6 percent

PREFORM = 1 if preformed sealant is used in joints, 0 if not

f_c' = concrete compressive strength, psi

FTCYC = average annual number of freeze-thaw cycles

h_{pcc} = concrete slab thickness, inches

WCratio = concrete water/cement ratio

TFAULT = total joint faulting, in/mile, from JPCP faulting model

SF = site factor:

$$SF = Age\,(1 + 0.5556\,FI)(1 + P200) * 10^{-6} \tag{5.34}$$

Age = pavement age, years

FI = average annual Freezing Index, Fahrenheit degree-days

P200 = percent subgrade soil passing the No. 200 sieve

Concrete air content is not in fact a variable in the scaling factor equation, but rather is fixed in the MEPDG software at a value of 6 percent. Hall has shown that the joint spalling predicted by Equation 5.32, with the scaling factor (Equation 5.33) as an input, is fairly sensitive to air content.[3]

The IRI model for JPCP appears in the MEPDG software as shown in Figure 5.51, which is to say, as with the HMA IRI model, the model itself does not appear at all; only the coefficients for the cracking, spalling, total faulting, and site factor terms in the IRI model appear. The sensitivity of predicted JPCP IRI to C_1, C_2, C_3, and C_4 is illustrated in Figures 5.52, 5.53, 5.54, and 5.55. The prediction of JPCP IRI is most sensitive to the value assigned to the C_3 coefficient and is also sensitive to the value assigned to the C_4 coefficient, but is not very sensitive to the values assigned to the C_1 and C_2 coefficients.

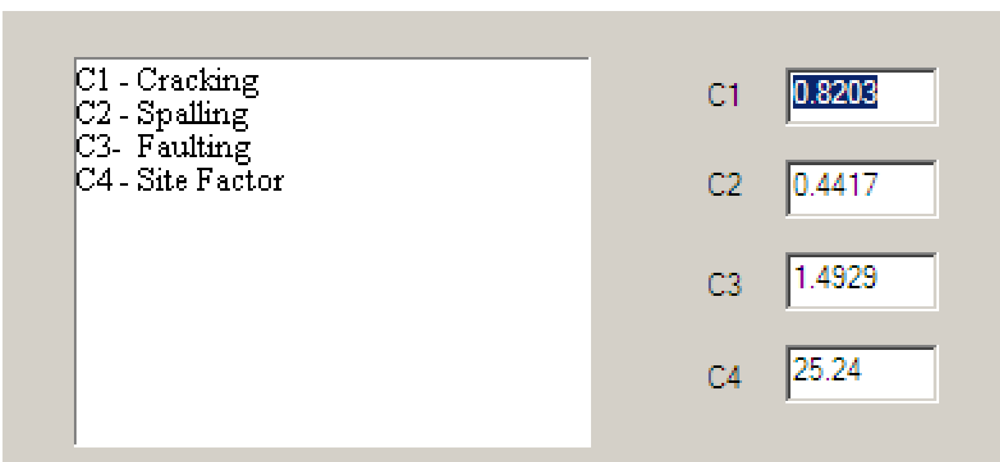

Figure 5.51 JPCP IRI Model in the MEPDG Software

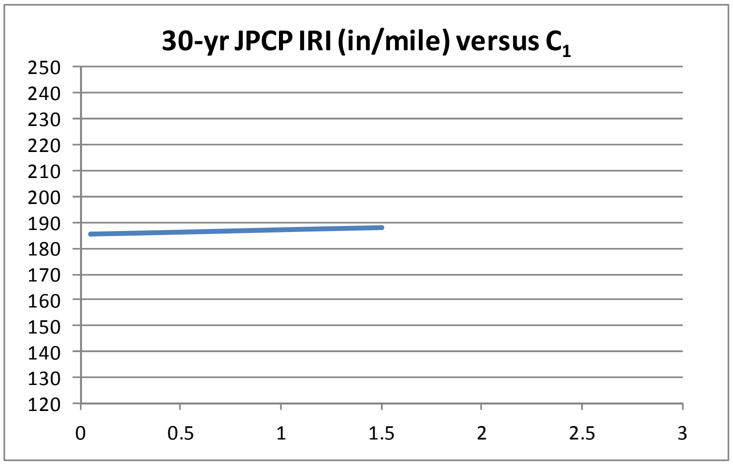

Figure 5.52 Sensitivity of Predicted JPCP IRI to C_1

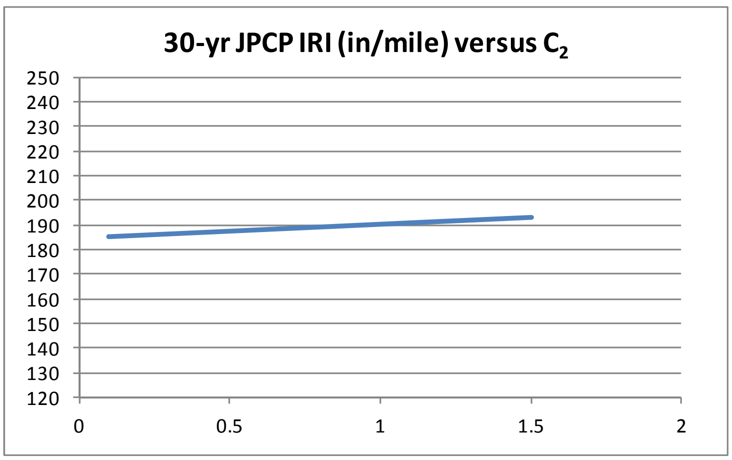

Figure 5.53 Sensitivity of Predicted JPCP IRI to C_2

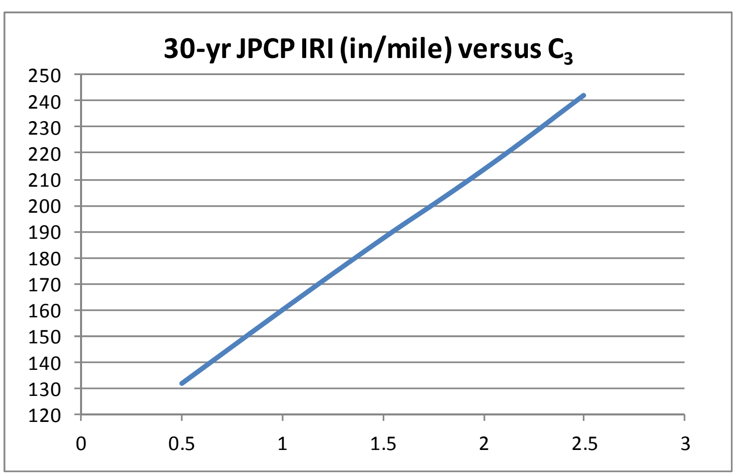

Figure 5.54 Sensitivity of Predicted JPCP IRI to C_3

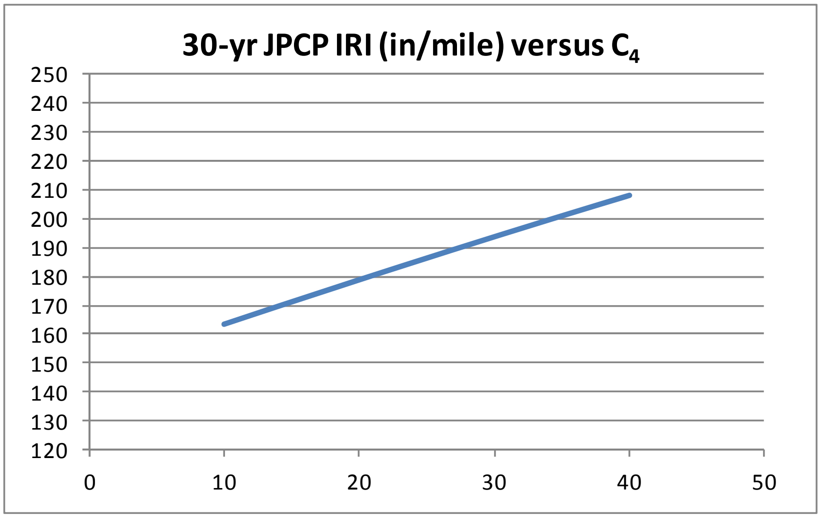

Figure 5.55 Sensitivity of Predicted JPCP IRI to C_4

The results of the NCHRP Report 372 sensitivity analysis of the MEPDG JPCP IRI model are illustrated in Figure 5.56. Note that many of the inputs shown do not appear directly in the JPCP IRI model; some are inputs to the models for cracking, faulting, and spalling model from which IRI is predicted. Concrete slab width is the only input analyzed that appears to be of notable sensitivity in the prediction of JPCP IRI.

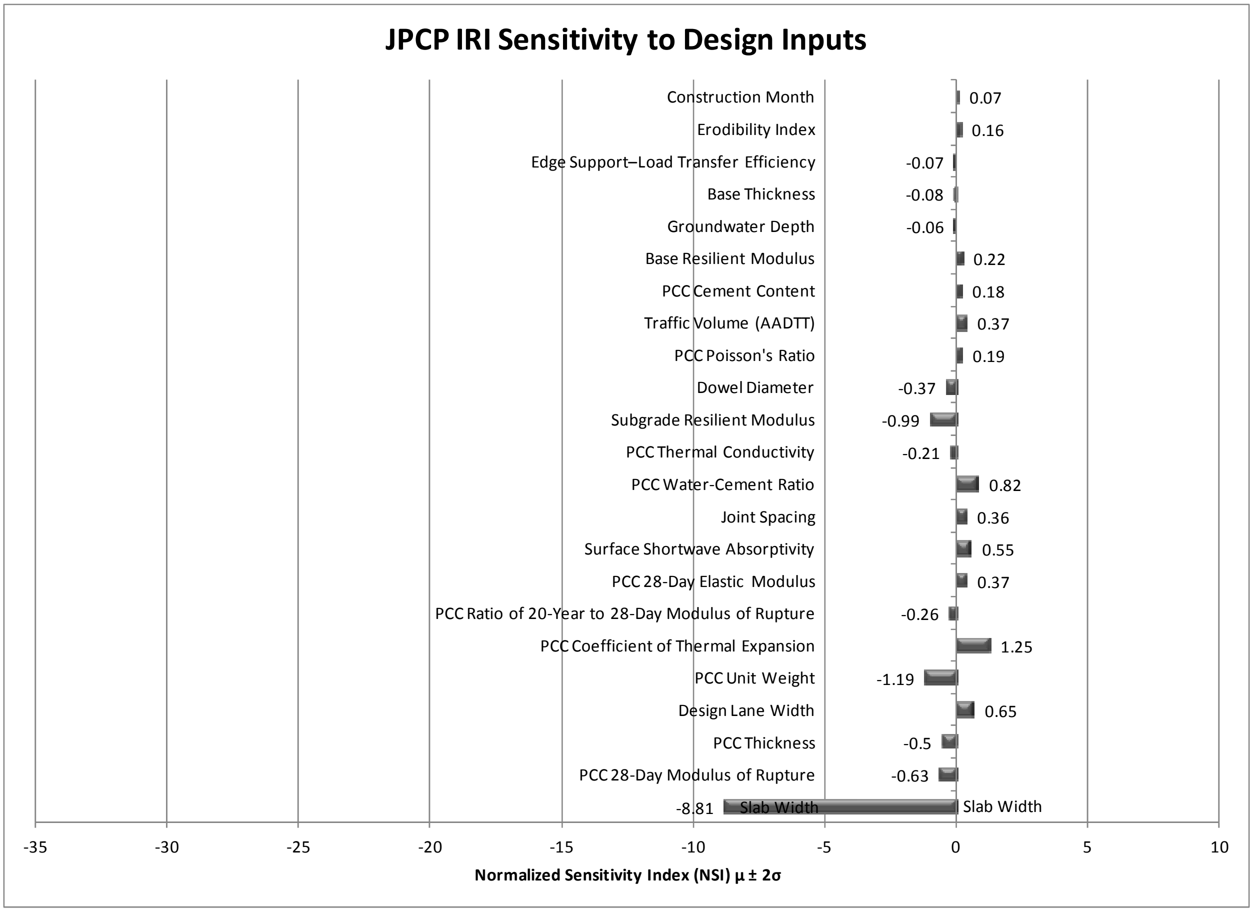

Figure 5.56 MEPDG JPCP IRI Model Sensitivity to Design Inputs
(Data from NCHRP Report 372)

CHAPTER 6 - MEPDG Calibration for Wisconsin Sections

6.1 Introduction

The following sections present the analysis of the Wisconsin test sections using the MEPDG Design Software, version 1.0. For all analyses, the MEPDG software was first utilized to provide distress projections using default values for all calibration settings. Output trends for the mean and 90% reliability were then compared to the field performance data. Where necessary, the calibration settings were adjusted to bring the output trends into agreement with recorded field data.

6.2 JPCP Calibration

JPCP calibration efforts were focused on predicted slab cracking. Figures 6.1 and 6.2 provide example comparisons of predicted and actual slab cracking for STH 29 – Chippewa County and STH 29b – Marathon County, respectively. As shown for these extreme cases, the MEPDG predictions using default calibration settings produced performance predictions that were substantially different than actual field performance measures. Adjustments to the fatigue coefficient C1 and the cracking coefficient C5 were necessary to bring the predicted performance in line with field measurements.

For the cracking model,

$$CRK = \frac{100}{1 + C4FD^{C5}}$$

decreasing the C5 coefficient to a value of -4.0 to -4.5 was necessary to delay the onset of cracking to latter years, which is the typical performance trend for this distress.

For the fatigue model,

$$Log(N) = C1\left(\frac{MR}{\sigma}\right)^{C2}$$

setting the C1 coefficient to a value between 1.45 to 2.80 was necessary to bring the amount of cracking in line with field observations. Figures 6.3 and 6.4 illustrate the trends for the calibrated models for STH 29 – Chippewa County and STH 29b – Marathon County, respectively. As shown, adjustments to the calibration coefficients successfully brought the MEPDG predictions into line with field performance. Figures 6.5 through 6.11 illustrate the calibrated model trends for the remaining pavement sections.

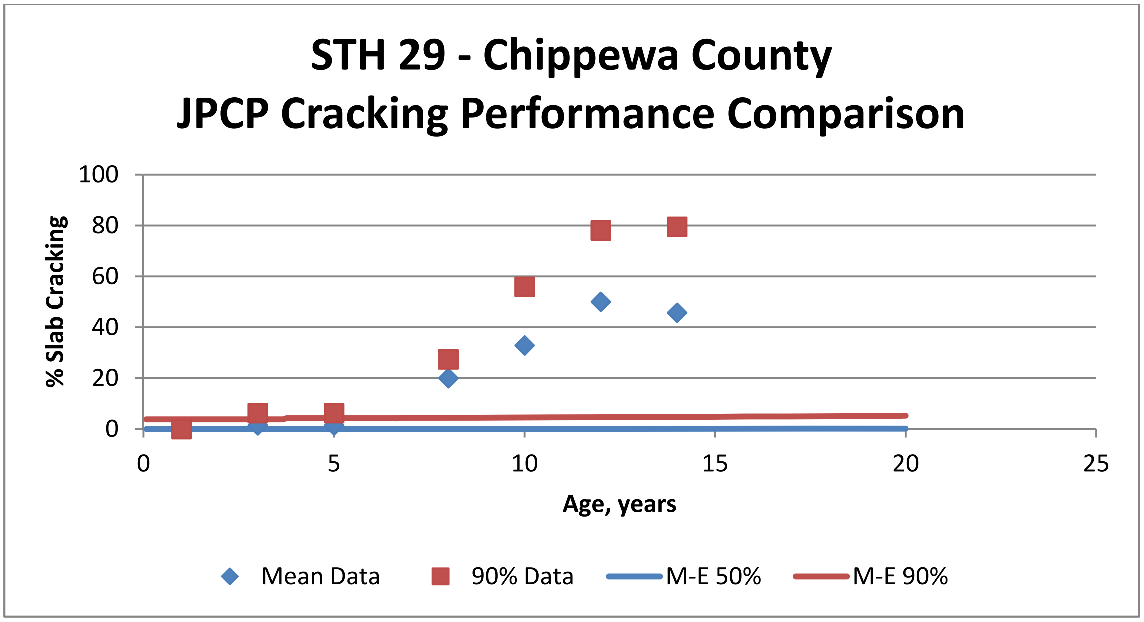

Figure 6.1 Observed and Predicted Slab Cracking Trends Using Default MEPDG Calibration Settings for STH 29 – Chippewa County

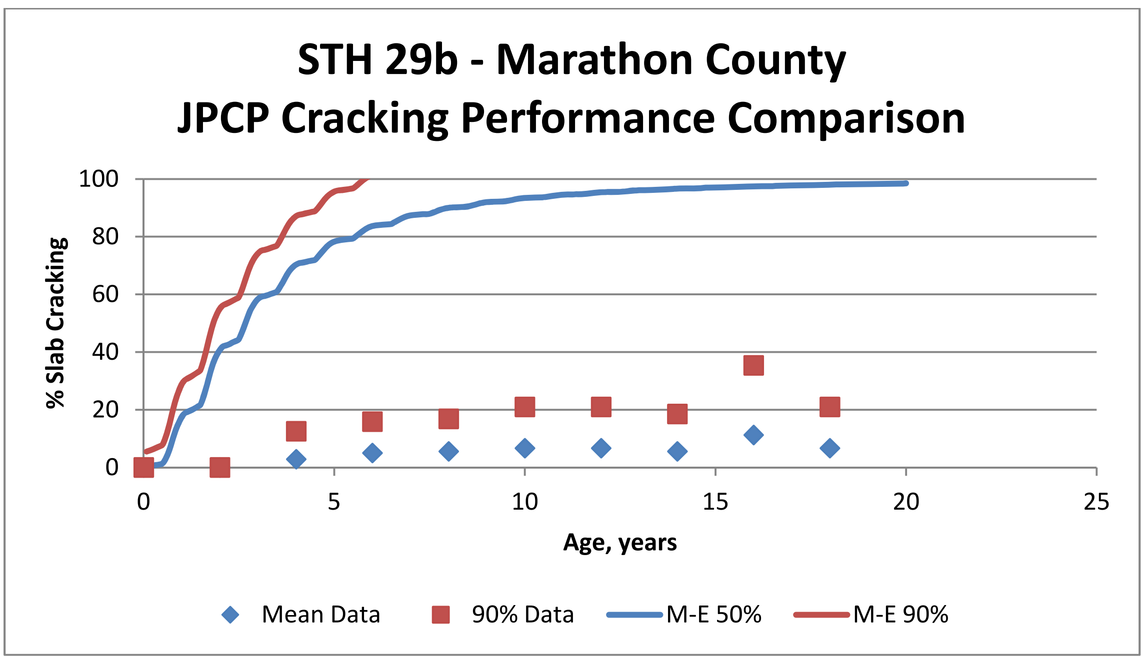

Figure 6.2 Observed and Predicted Slab Cracking Trends Using Default MEPDG Calibration Settings for STH 29b – Marathon County

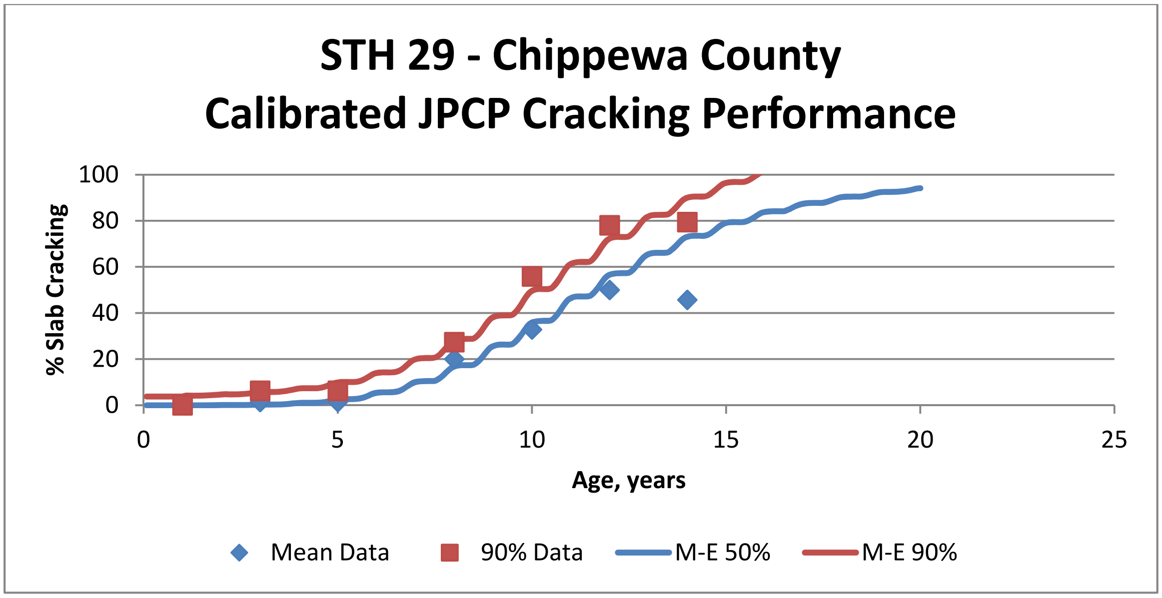

Figure 6.3 Observed and Predicted Slab Cracking Trends Using Adjusted MEPDG Calibration Settings for STH 29 – Chippewa County

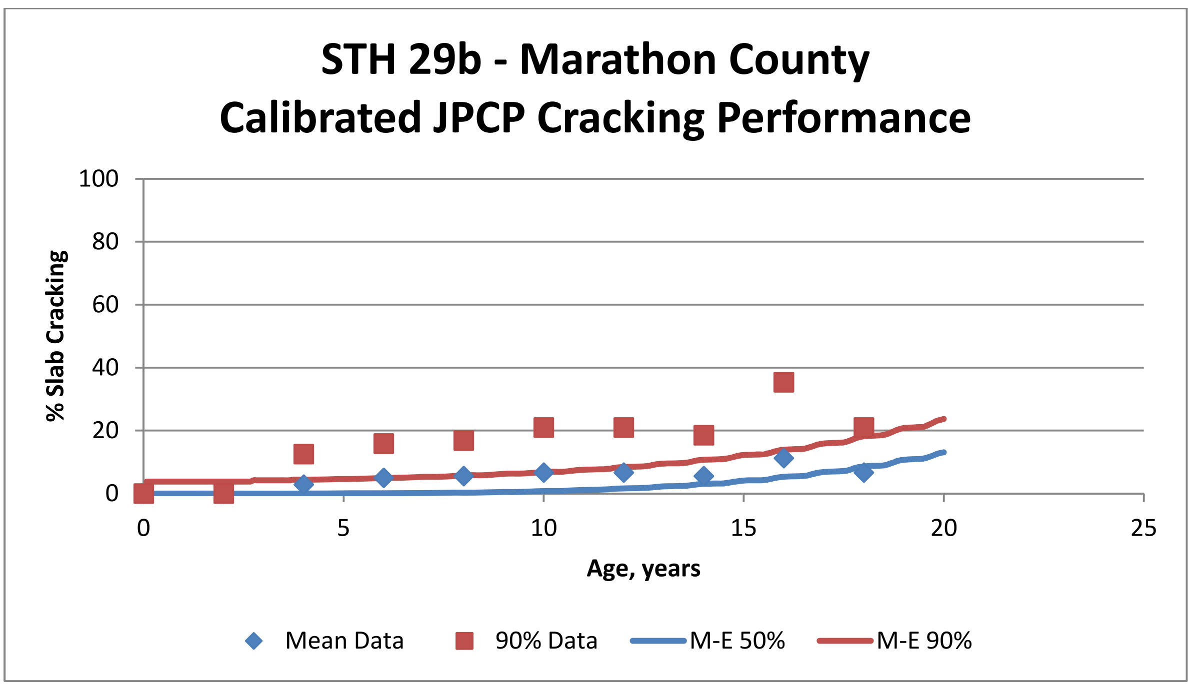

Figure 6.4 Observed and Predicted Slab Cracking Trends Using Adjusted MEPDG Calibration Settings for STH 29b – Marathon County

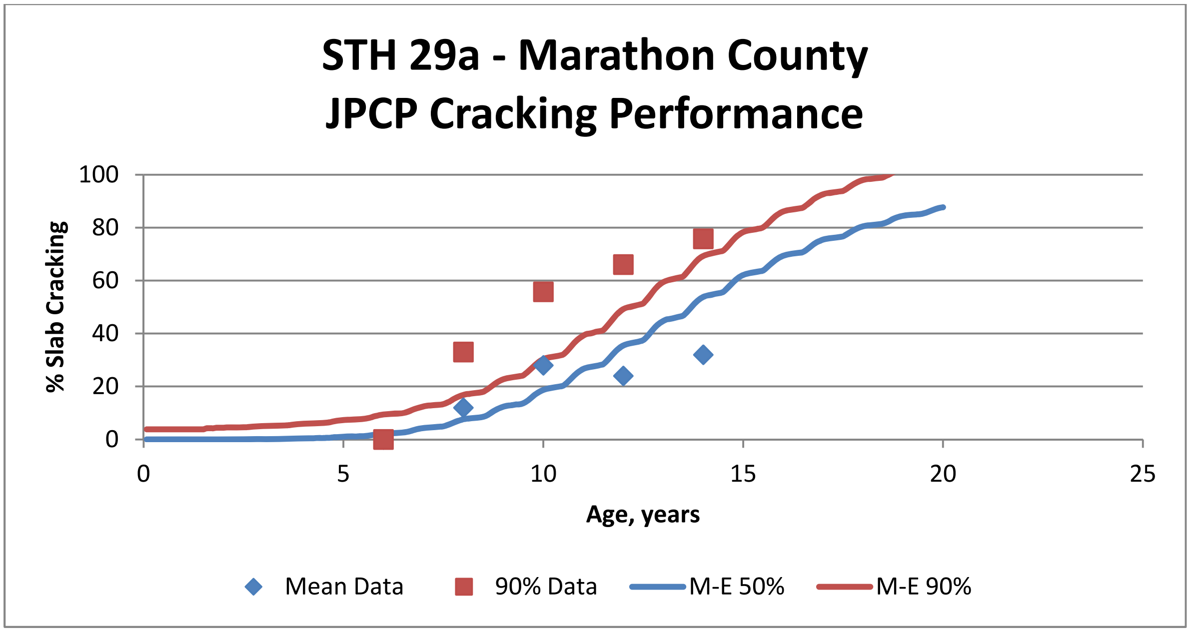

Figure 6.5 Observed and Predicted Slab Cracking Trends Using Adjusted MEPDG Calibration Settings for STH 29a – Marathon County

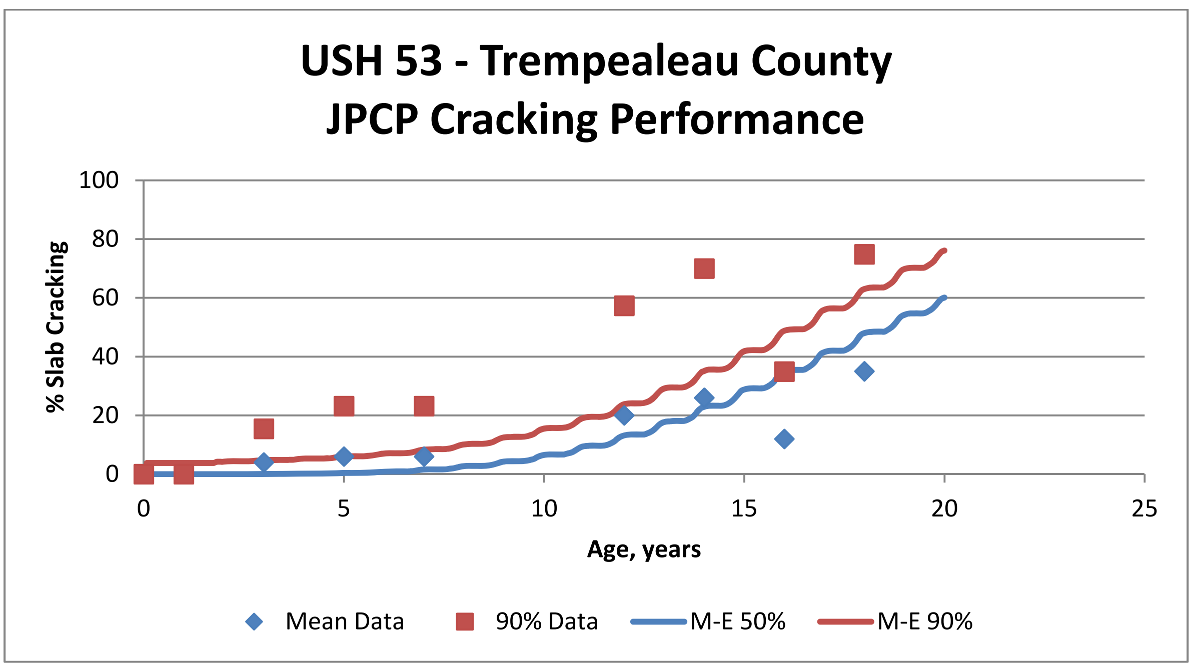

Figure 6.6 Observed and Predicted Slab Cracking Trends Using Adjusted MEPDG Calibration Settings for USH 53 – Trempealeau County

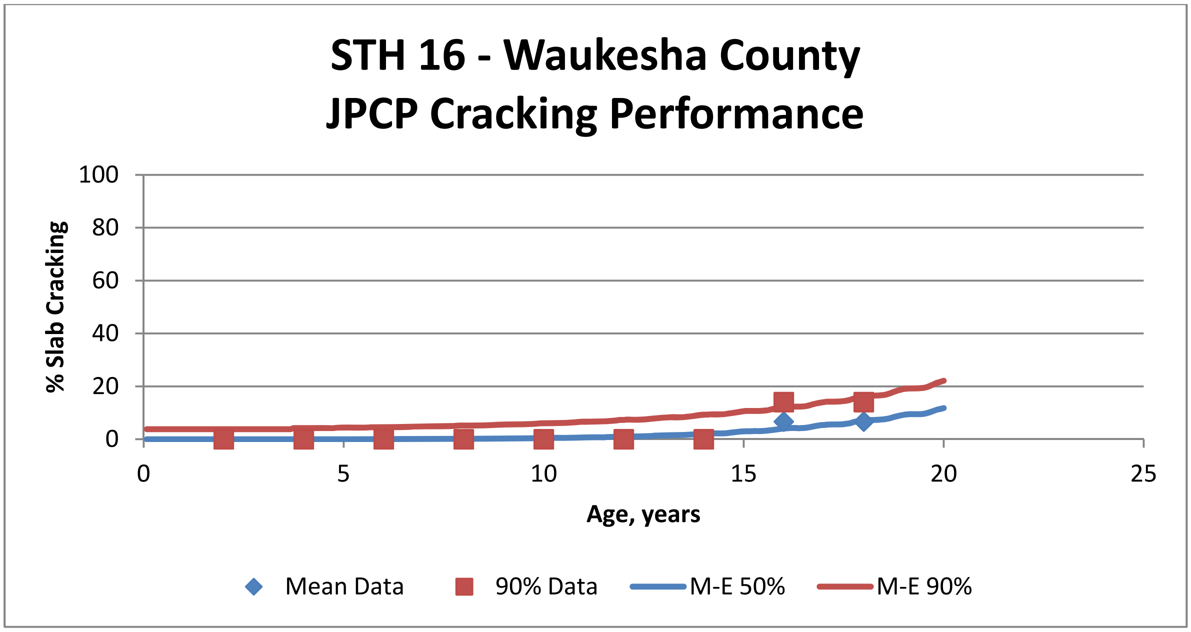

Figure 6.7 Observed and Predicted Slab Cracking Trends Using Adjusted MEPDG Calibration Settings for STH 16 – Waukesha County

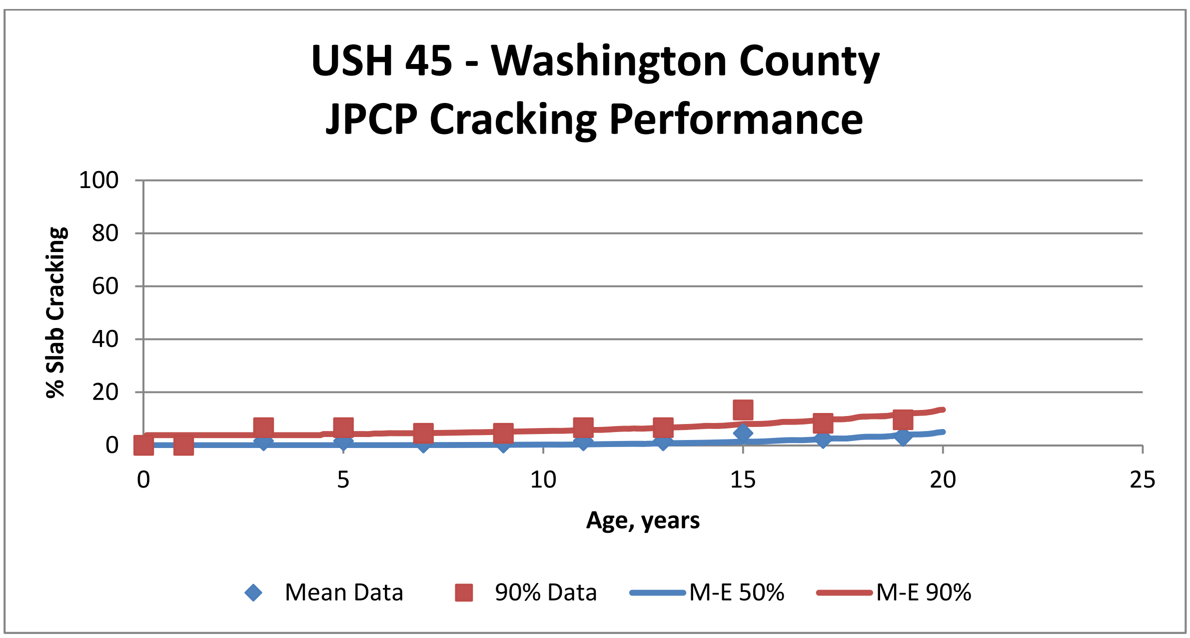

Figure 6.8 Observed and Predicted Slab Cracking Trends Using Adjusted MEPDG Calibration Settings for USH 45 – Washington County

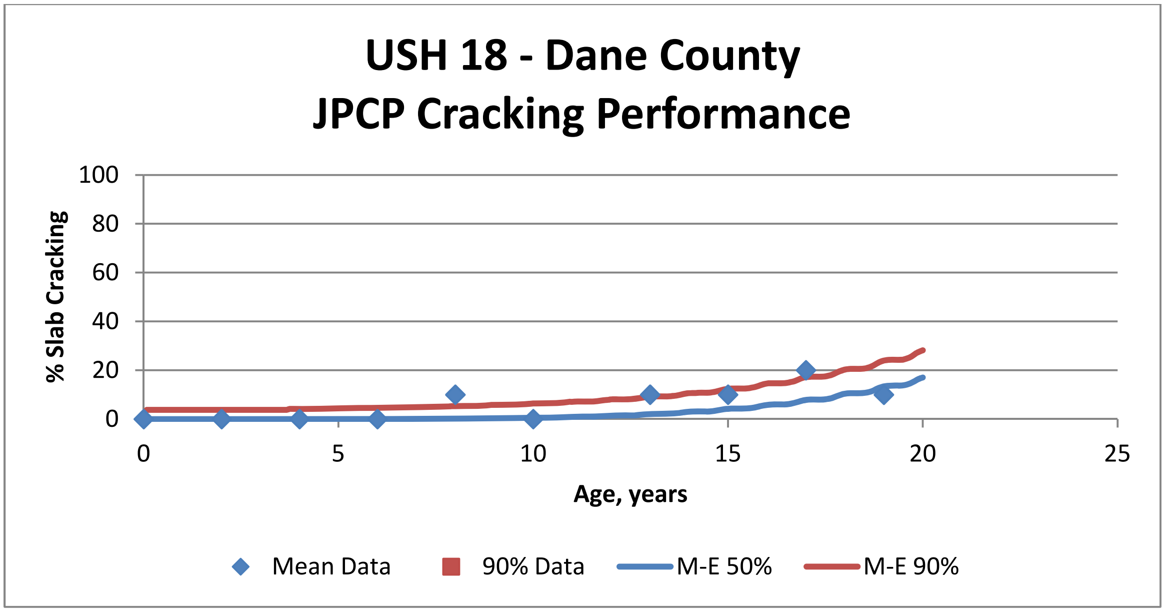

Figure 6.9 Observed and Predicted Slab Cracking Trends Using Adjusted MEPDG Calibration Settings for USH 18 – Dane County

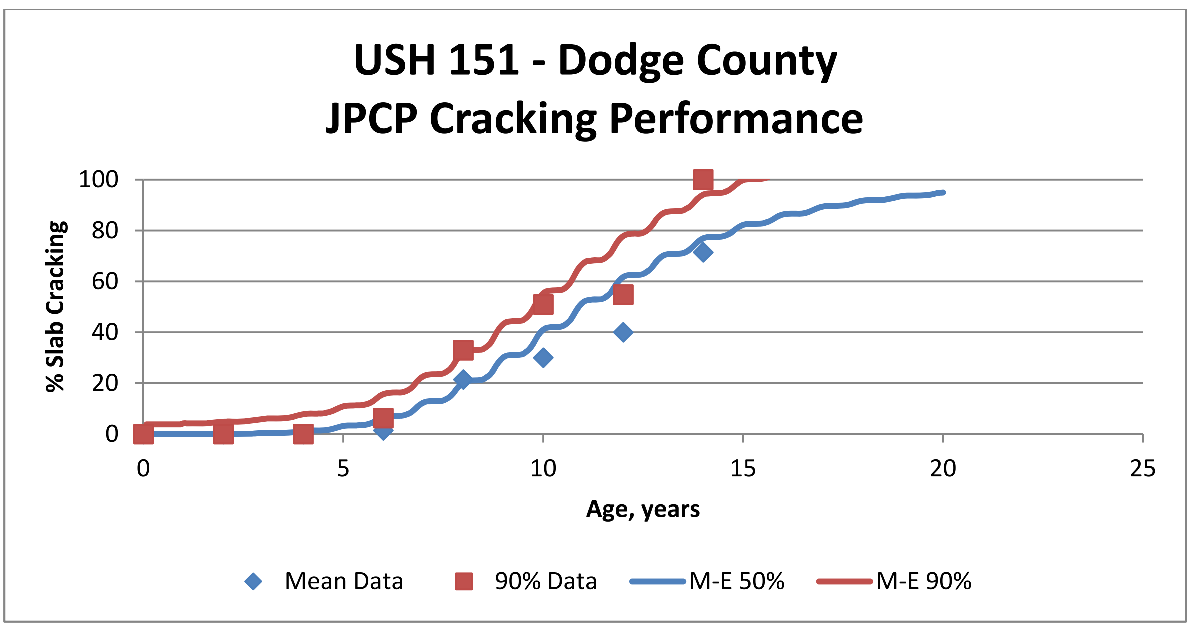

Figure 6.10 Observed and Predicted Slab Cracking Trends Using Adjusted MEPDG Calibration Settings for USH 151 – Dodge County

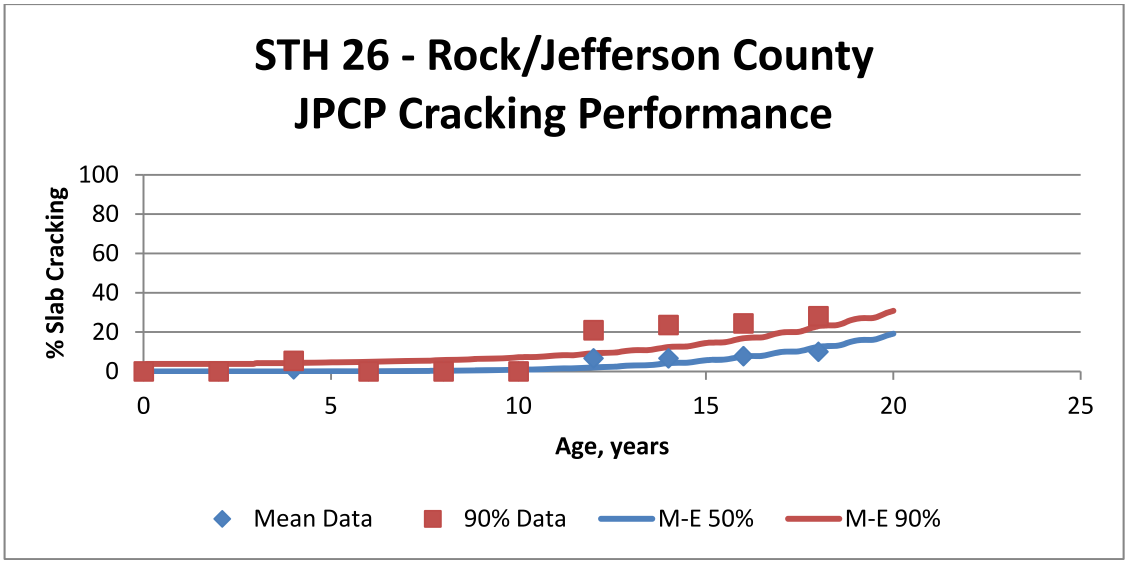

Figure 6.11 Observed and Predicted Slab Cracking Trends Using Adjusted MEPDG Calibration Settings for STH 26 – Rock/Jefferson County

Table 6.1 provides a summary of the calibration settings for each of the included pavement sections. Figures 6.12 and 6.13 provide plots of the C1 calibration setting versus the back-estimated 28-day modulus of rupture (from long-term MOR measurement) and the TP60 coefficient of thermal expansion.

Table 6.1 Final Calibration Settings for JPCP Test Sections

Test Section	C1	C5	28-Day MOR psi	TP60 CTE με/F
STH 29 Chippewa Co	1.45	-4.5	651	5.9
STH 29a Marathon Co	2.35	-4.5	454	6
STH 29b Marathon Co	2.50	-4.0	619	5.5
USH 53 Trempealeau Co	1.90	-4.0	715	6.4
STH 16 Waukesha	2.70	-4.5	767	6.4
USH 18 Dane Co	1.98	-4.5	837	5.7
USH 45 Washington Co	2.80	-4.5	689	6.1
STH 26 Rock/Jefferson Co	2.77	-4.5	611	6.5
USH 151 Dodge Co	2.00	-4.5	733	6.5

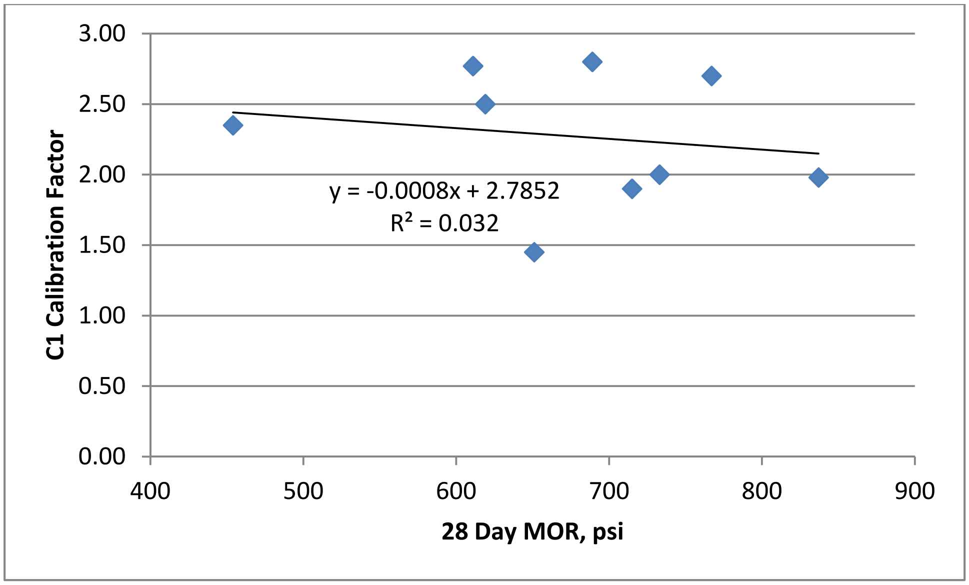

Figure 6.12 Final C1 Calibration Setting Versus Back-Estimated 28-Day Modulus of Rupture

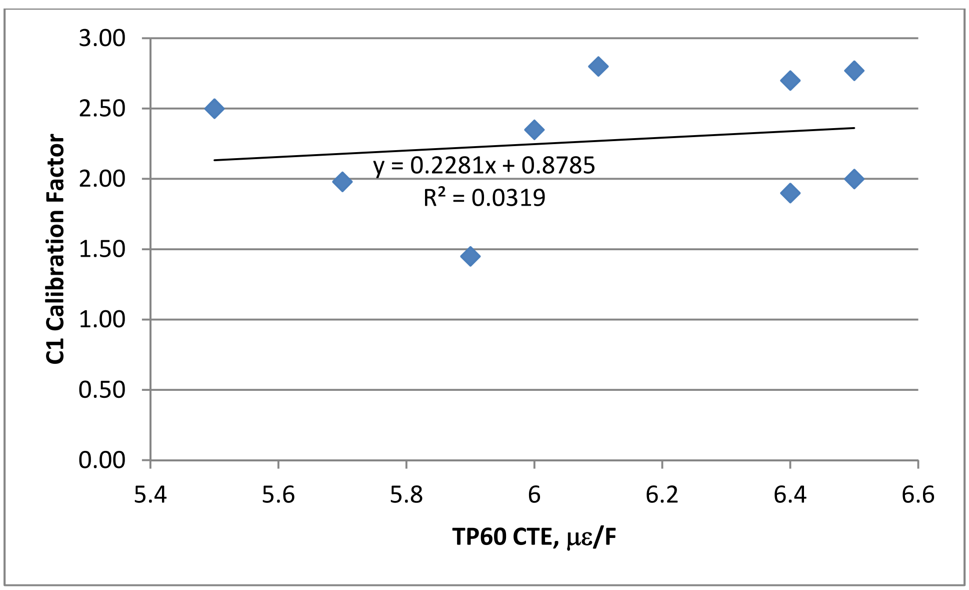

Figure 6.13 Final C1 Calibration Setting Versus TP60 Coefficient of Thermal Expansion

The C1 calibration values have a much greater variation than the C5 values. In lieu of determining a C1 value based on expected or measured MOR or CTE values, it may be desired to use a single input for this value during the pavement design phase. Considering the STH 29 Chippewa County project, adjusting C1 to the average value of 2.27 results in a prediction of no slab cracking after 20 years, which is a dramatic under prediction of the actual performance of this section. While the trends provided in Figures 6.12 and 6.13 have poor R^2 values, the regression equations represent the first-order method for estimating C1 values a priori using available PCC data. However, for the aforementioned STH 29 Chippewa County, the calculated values for C1 using the trends displayed in Figures 6.12 and 6.13 are 2.26 and 2.22, respectively, both yielding predictions of no slab cracking, again being far from observed cracking trends. These calibration models will require further verification and refinement using the recently released DARWin ME software to ensure their applicability for general usage.

6.3 HMA Calibration

HMA calibration efforts were focused on predicted rutting and alligator cracking performance. Figure 6.14 and 6.15 provide example comparisons of predicted and actual rutting and alligator cracking for STH 35 – Pierce County using default MEPDG calibration settings. As shown, the default MEPDG predictions produced rutting performance predictions that were substantially higher than actual field performance measures and alligator cracking performance predictions that were slightly lower than actual field performance measures.

For the rutting calibrations, measurements of HMA layer thicknesses across the outer wheel path did not indicate surface layer rutting was present. Adjustments to the HMA rutting coefficient Br1 and the base layer rutting coefficient Bs1 were necessary eliminate rutting in these layers. The remaining subgrade rutting coefficient Bs1 was then adjusted as necessary to bring MEPDG rutting predictions in line with field performance. For the alligator cracking calibrations, adjustments to the HMA fatigue coefficient Bf2 were necessary to bring MEPDG alligator cracking predictions in line with field performance. In some instances, the default value for Bf2 adequately predicted alligator cracking performance for sections with no observed cracking. Figures 6.16 and 6.17 provide calibrated rutting and alligator cracking performance comparisons for STH 35 – Pierce County. Figures 6.18 through 6.33 provide calibrated rutting and alligator performance trends for the remaining pavement sections.

Table 6.2 provides the rutting and alligator cracking calibration settings for each included pavement section.

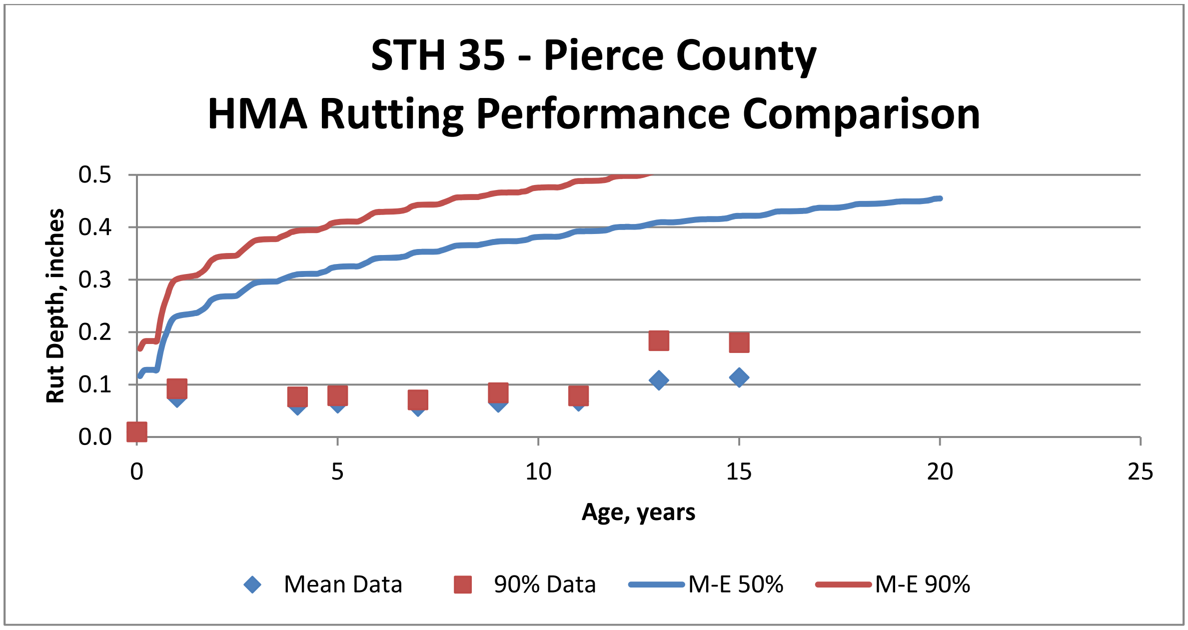

Figure 6.14 Observed and Predicted Rutting Trends Using Default MEPDG Calibration Settings for STH 35 – Pierce County

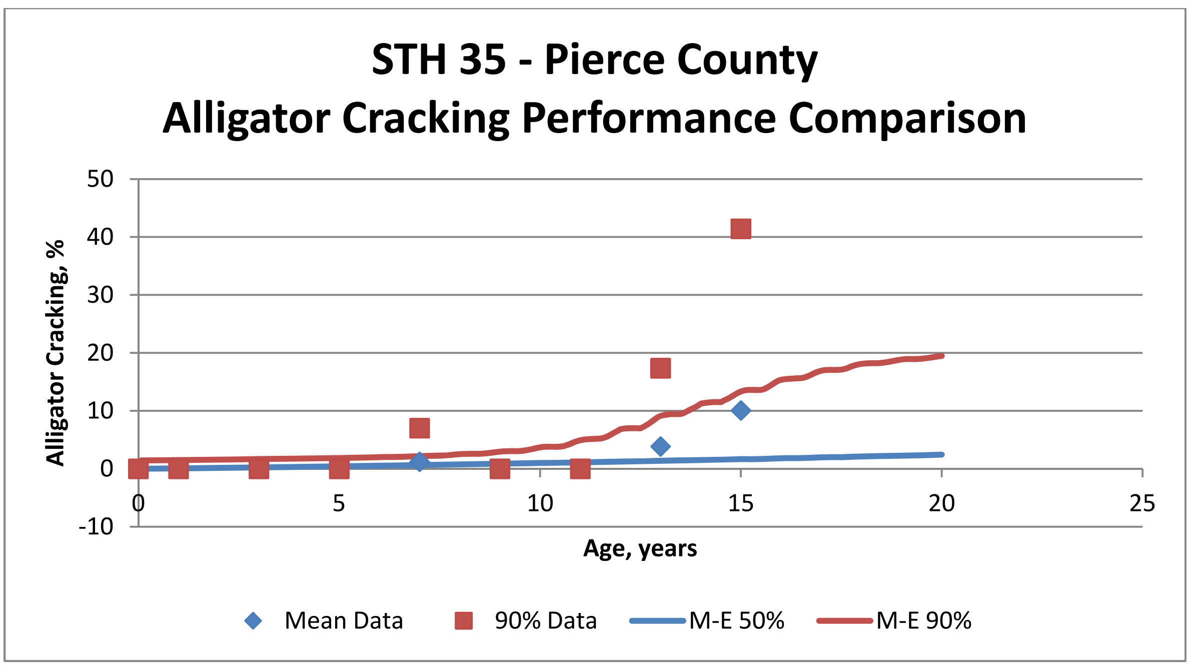

Figure 6.15 Observed and Predicted Alligator Cracking Trends Using Default MEPDG Calibration Settings for STH 35 – Pierce County

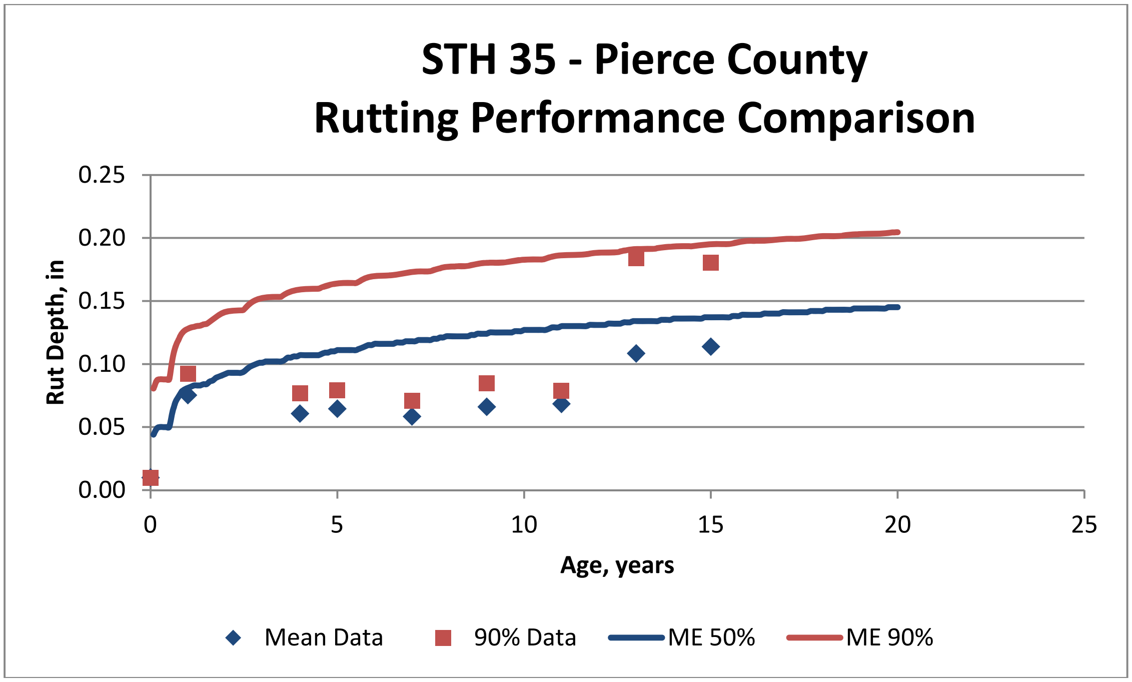

Figure 6.16 Observed and Calibrated Rutting Trends for STH 35 – Pierce County

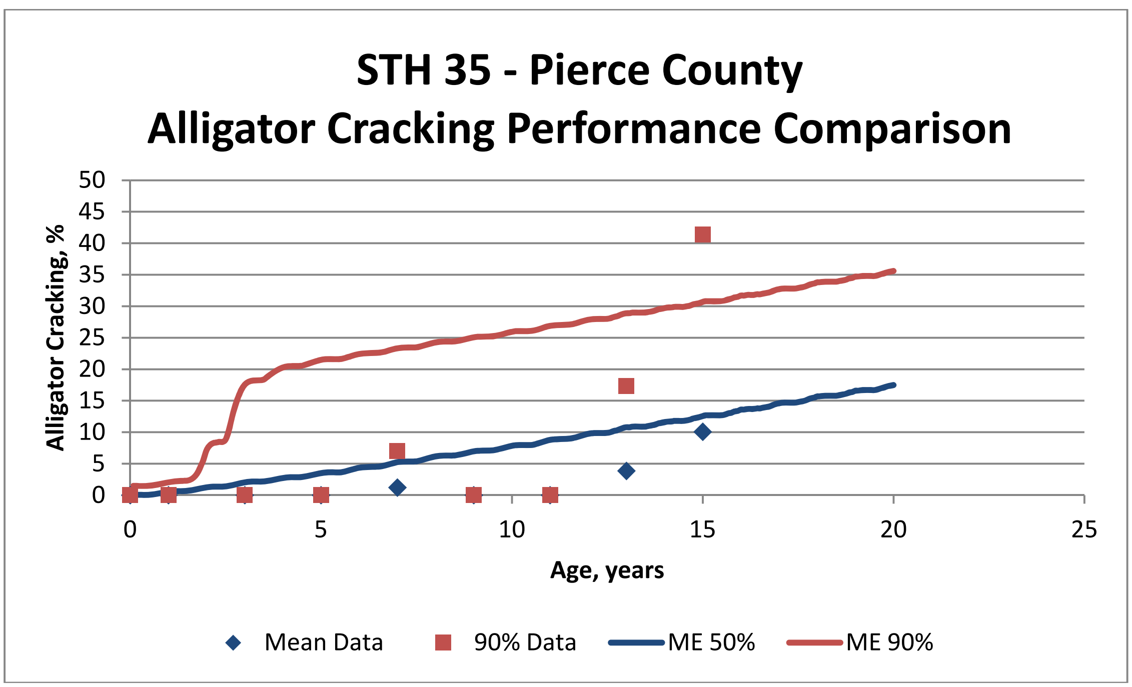

Figure 6.17 Observed and Calibrated Alligator Cracking Trends for STH 35 – Pierce County

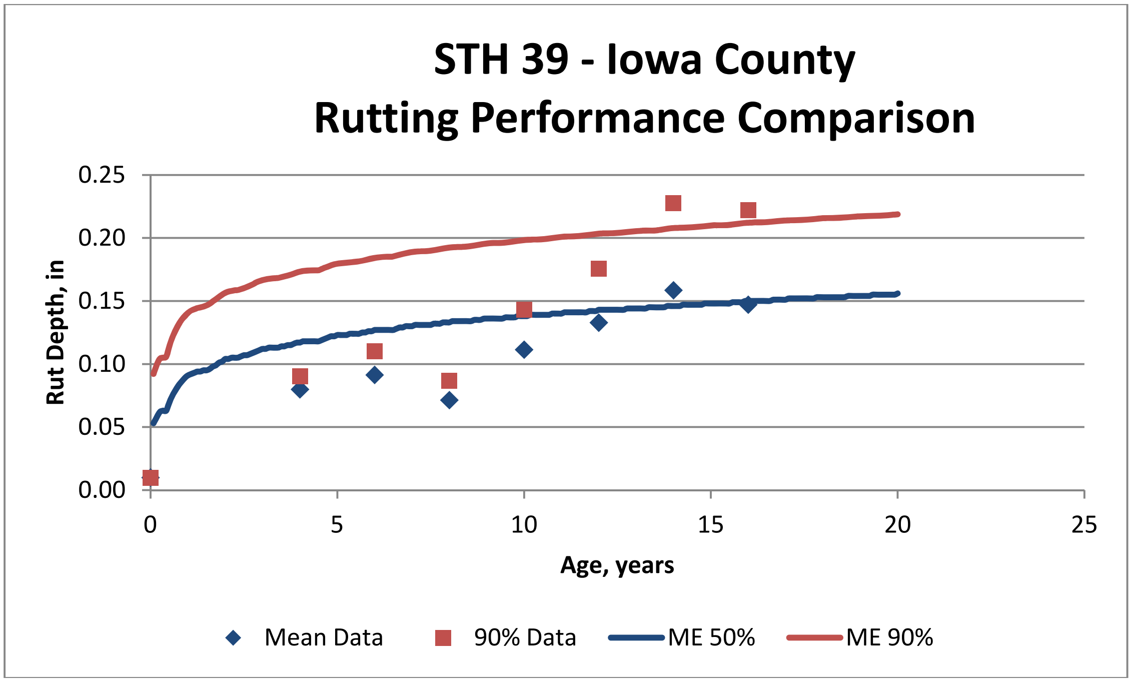

Figure 6.18 Observed and Calibrated Rutting Trends for STH 39 - Iowa County

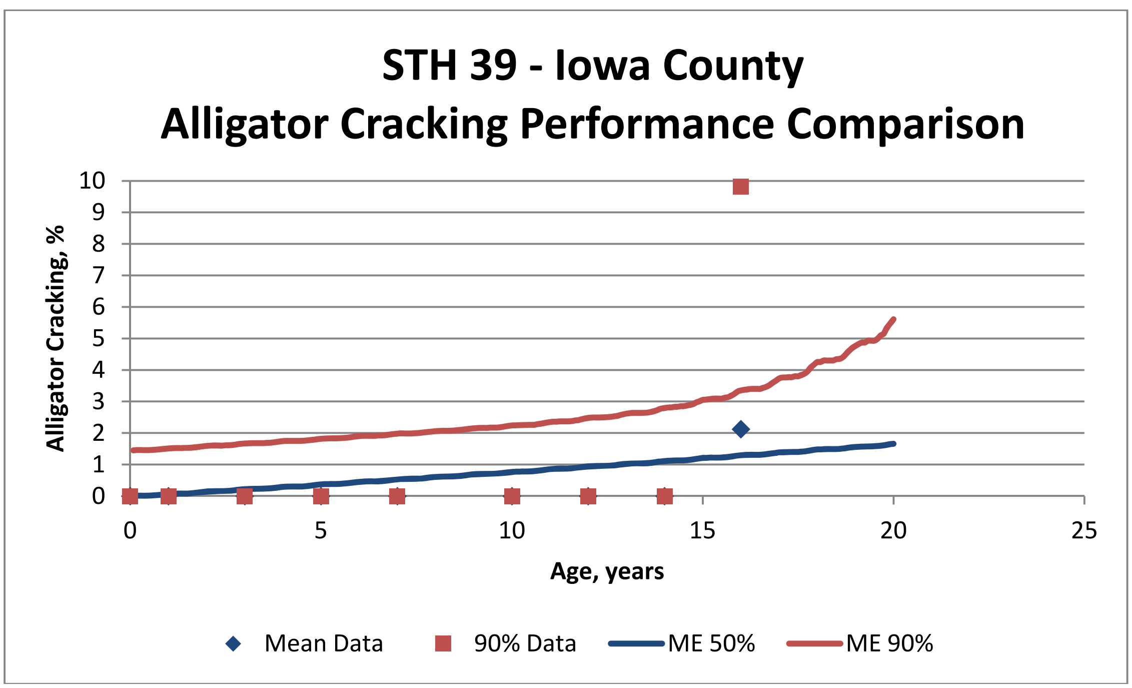

Figure 6.19 Observed and Calibrated Alligator Cracking Trends for STH 39 - Iowa County

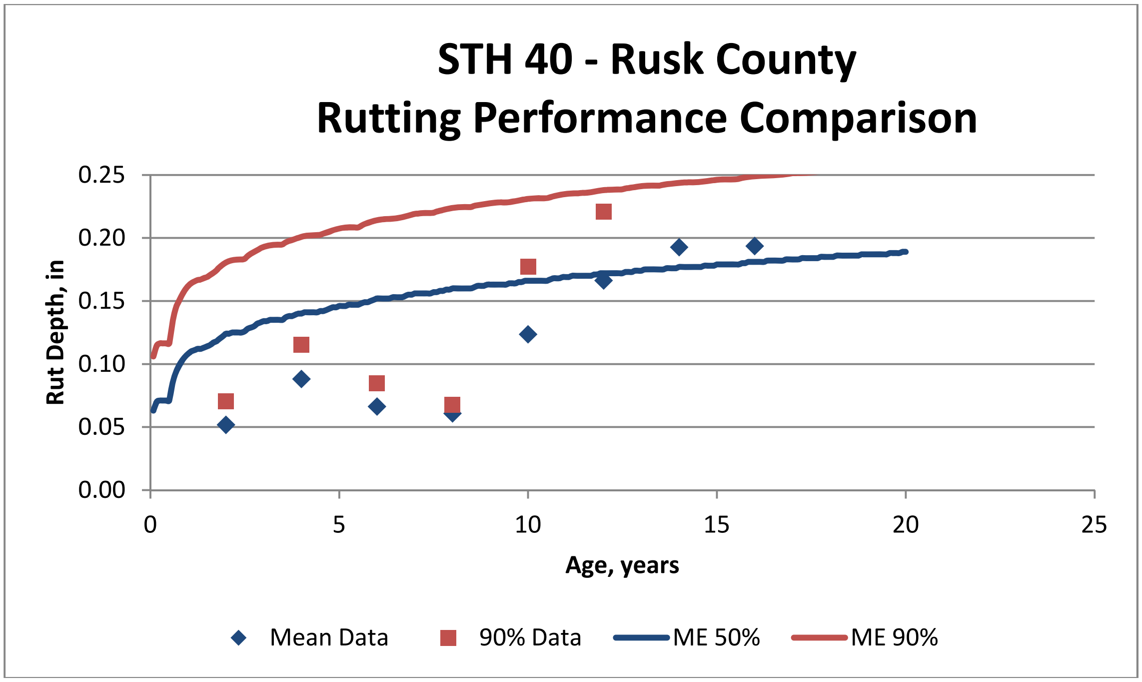

Figure 6.20 Observed and Calibrated Rutting Trends for STH 40 – Rusk County

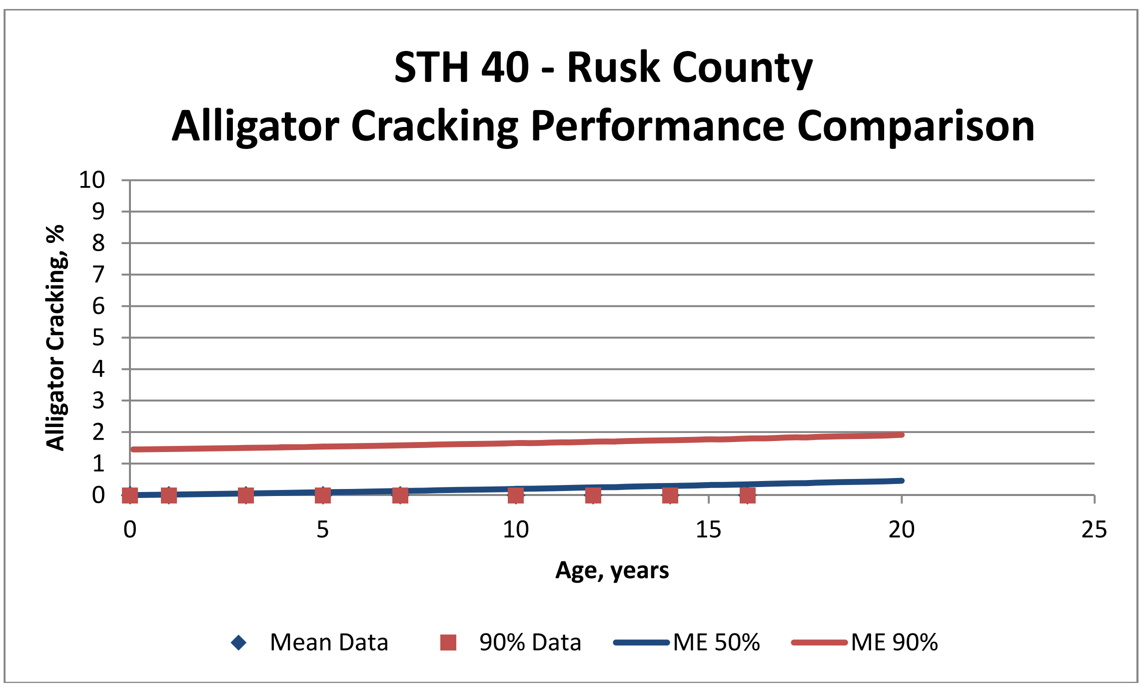

Figure 6.21 Observed and Calibrated Alligator Cracking Trends for STH 40 – Rusk County

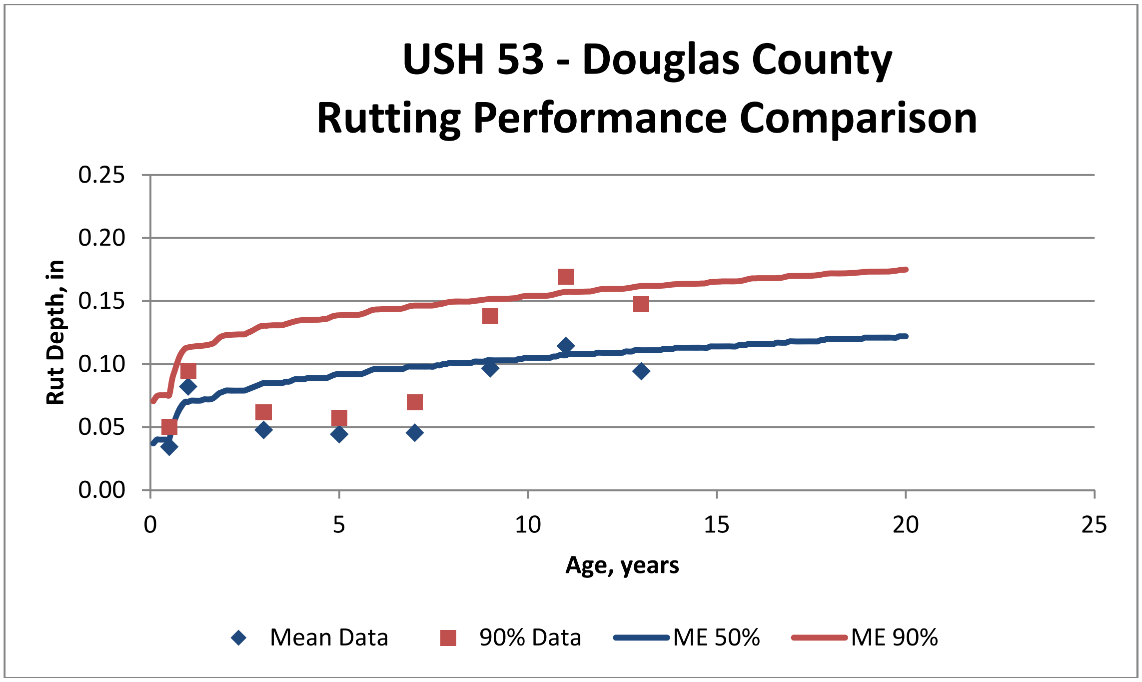

Figure 6.22 Observed and Calibrated Rutting Trends for USH 53 – Douglas County

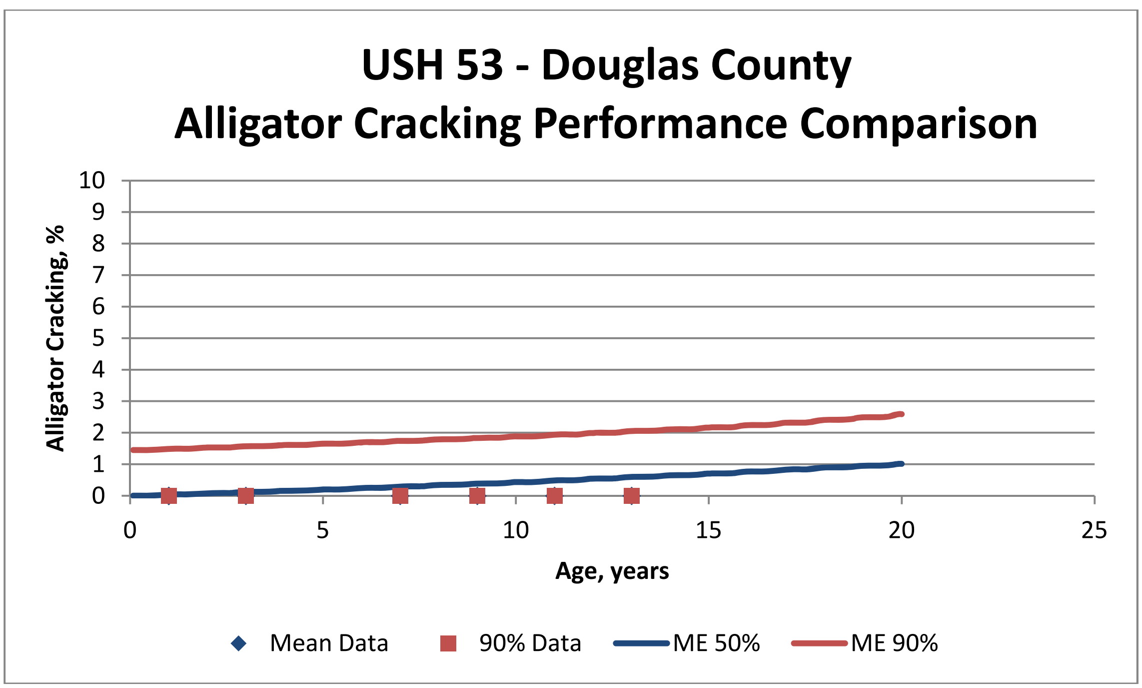

Figure 6.23 Observed and Calibrated Alligator Cracking Trends for USH 53 – Douglas County

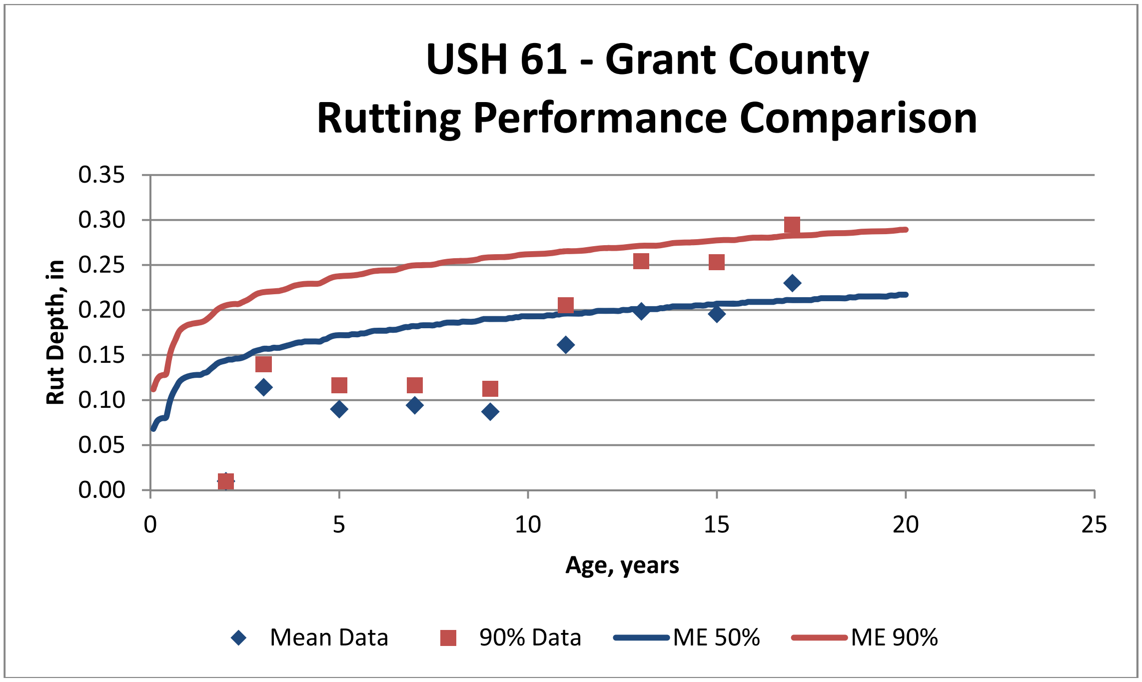

Figure 6.24 Observed and Calibrated Rutting Trends for USH 61 – Grant County

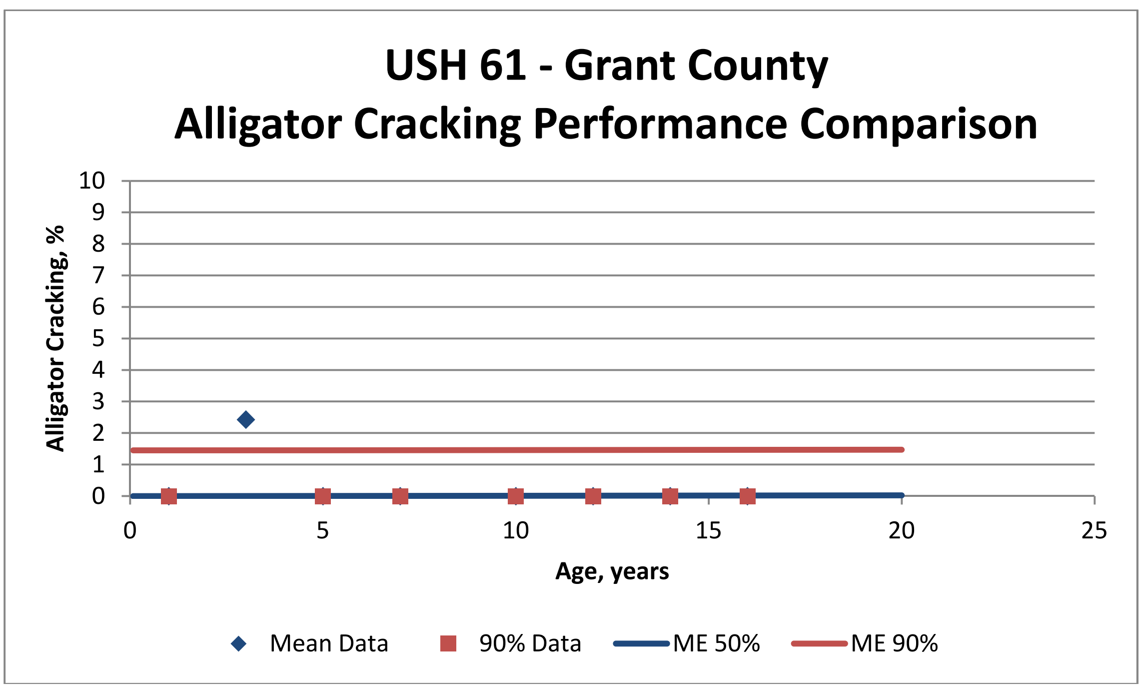

Figure 6.25 Observed and Calibrated Alligator Cracking Trends for USH 61 – Grant County

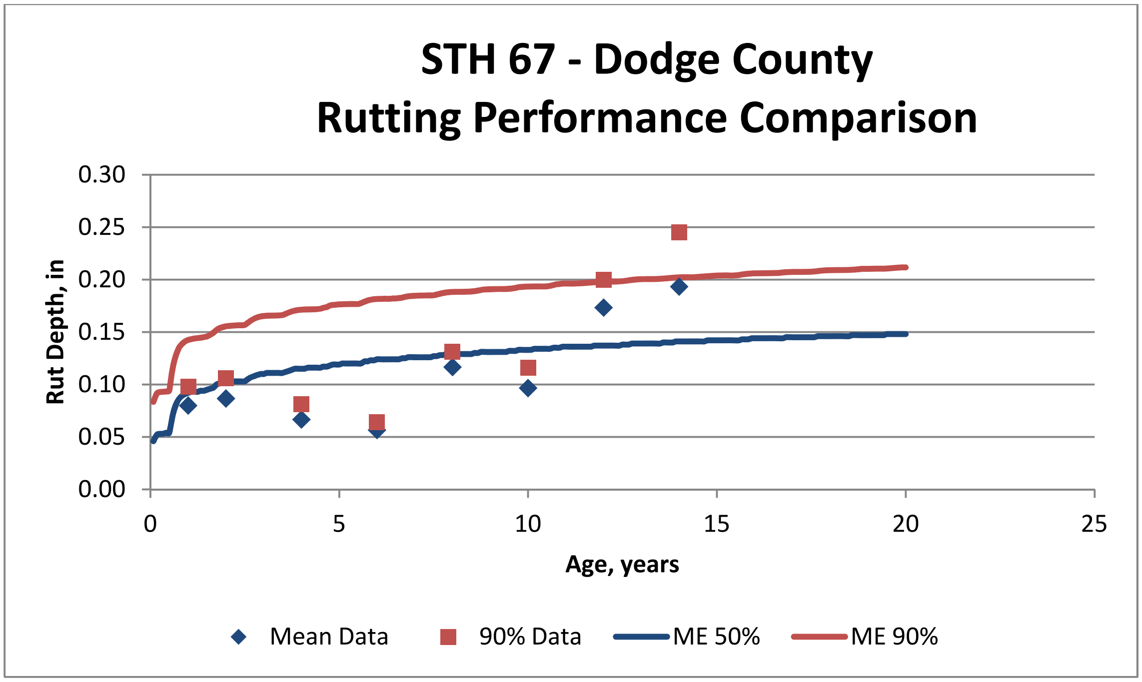

Figure 6.26 Observed and Calibrated Rutting Trends for STH 67 – Dodge County

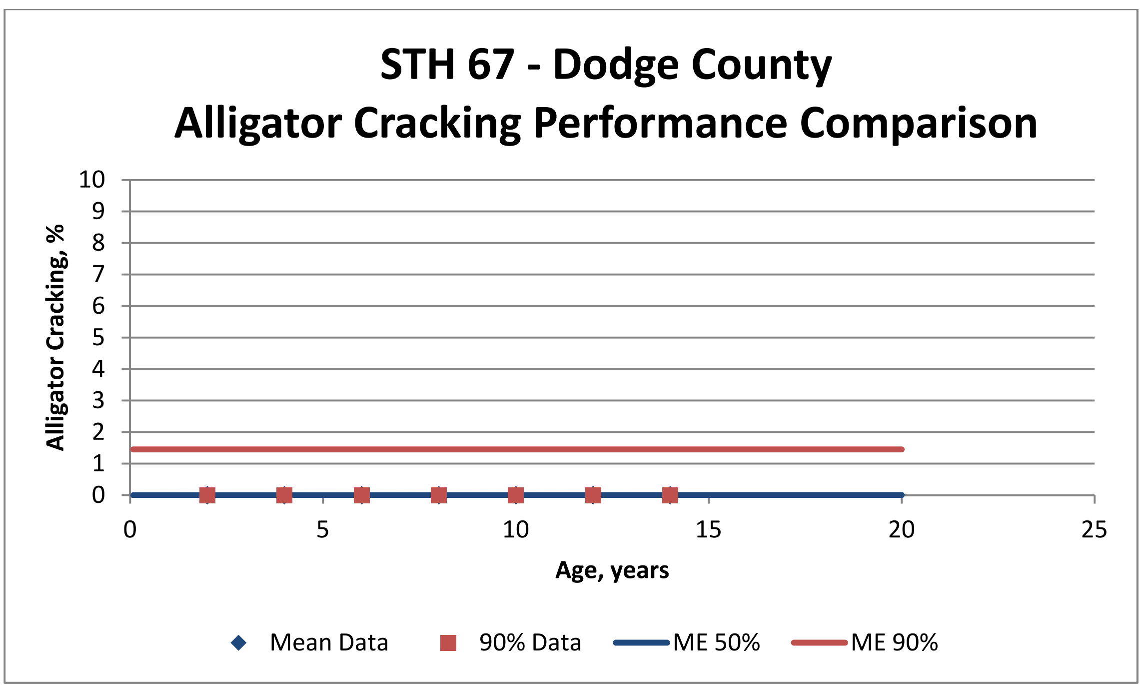

Figure 6.27 Observed and Calibrated Alligator Cracking Trends for STH 67 – Dodge County

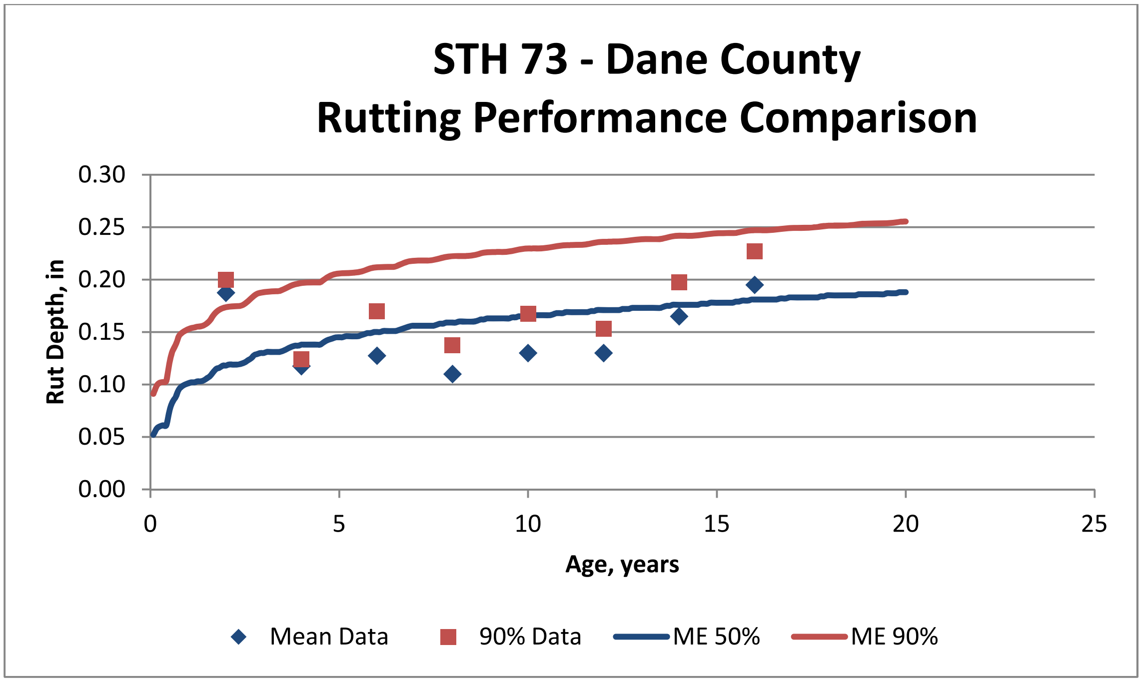

Figure 6.28 Observed and Calibrated Rutting Trends for STH 73 – Dane County

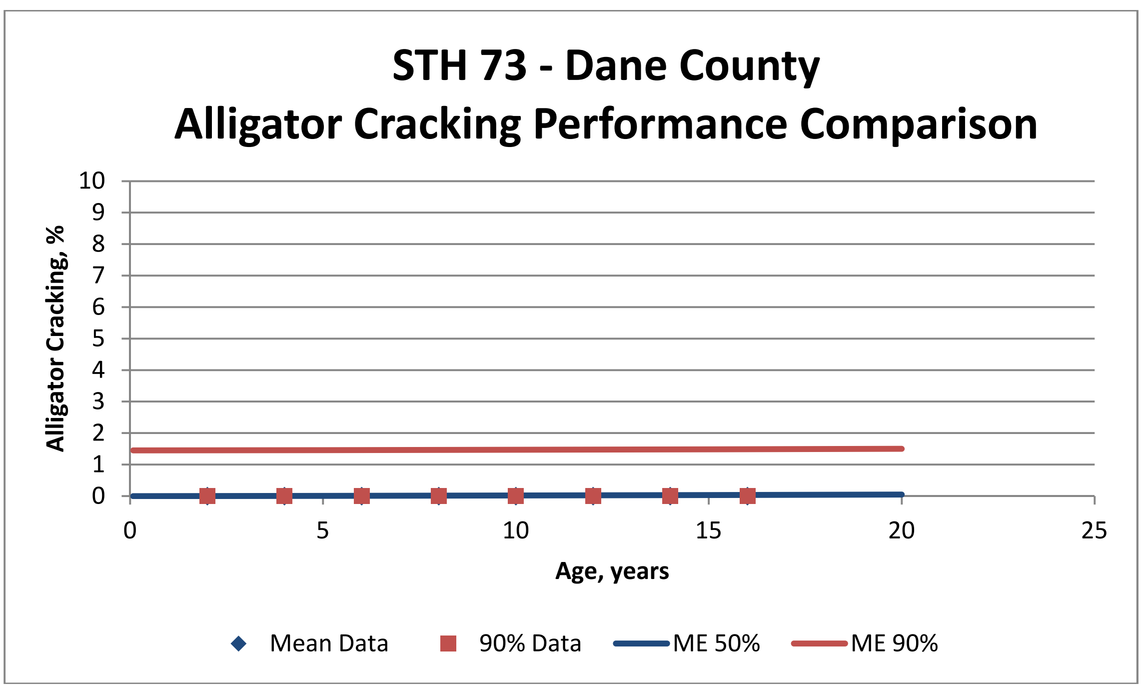

Figure 6.29 Observed and Calibrated Alligator Cracking Trends for STH 73 – Dane County

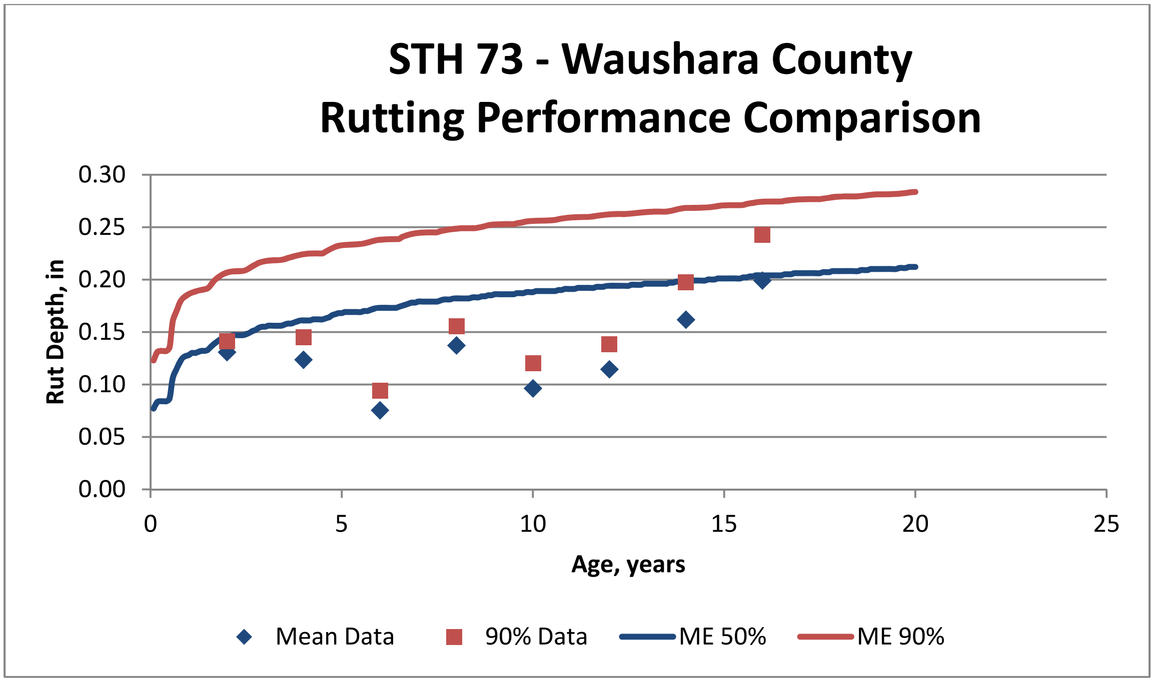

Figure 6.30 Observed and Calibrated Rutting Trends for STH 73 – Waushara County

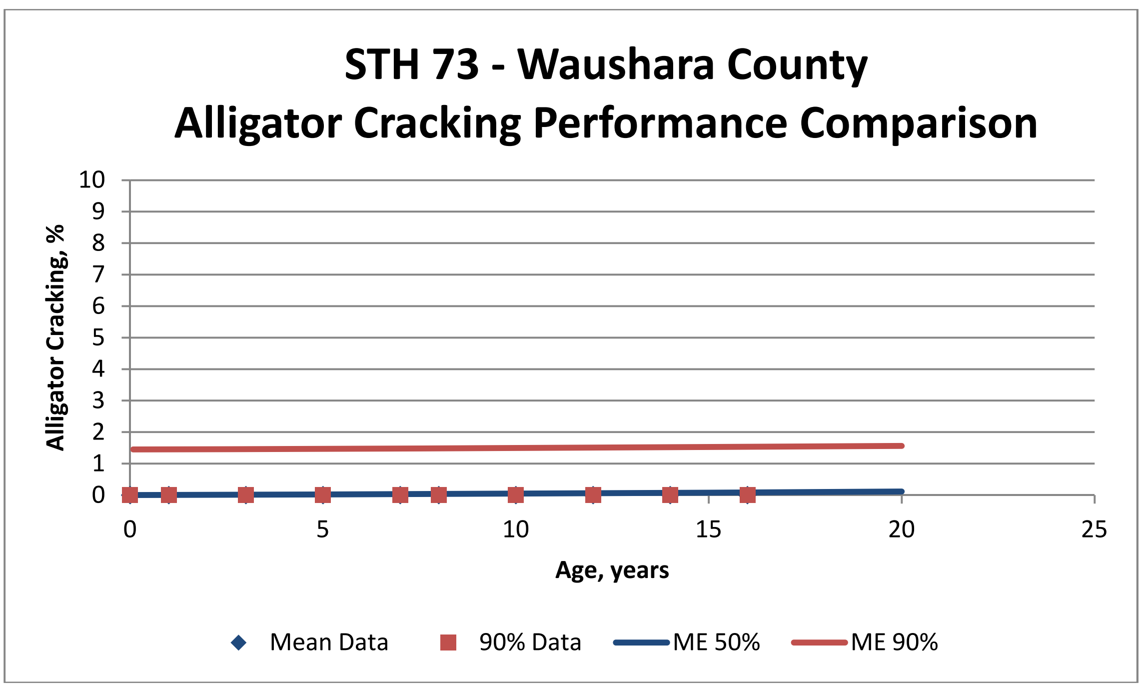

Figure 6.31 Observed and Calibrated Alligator Cracking Trends for STH 73 – Waushara County

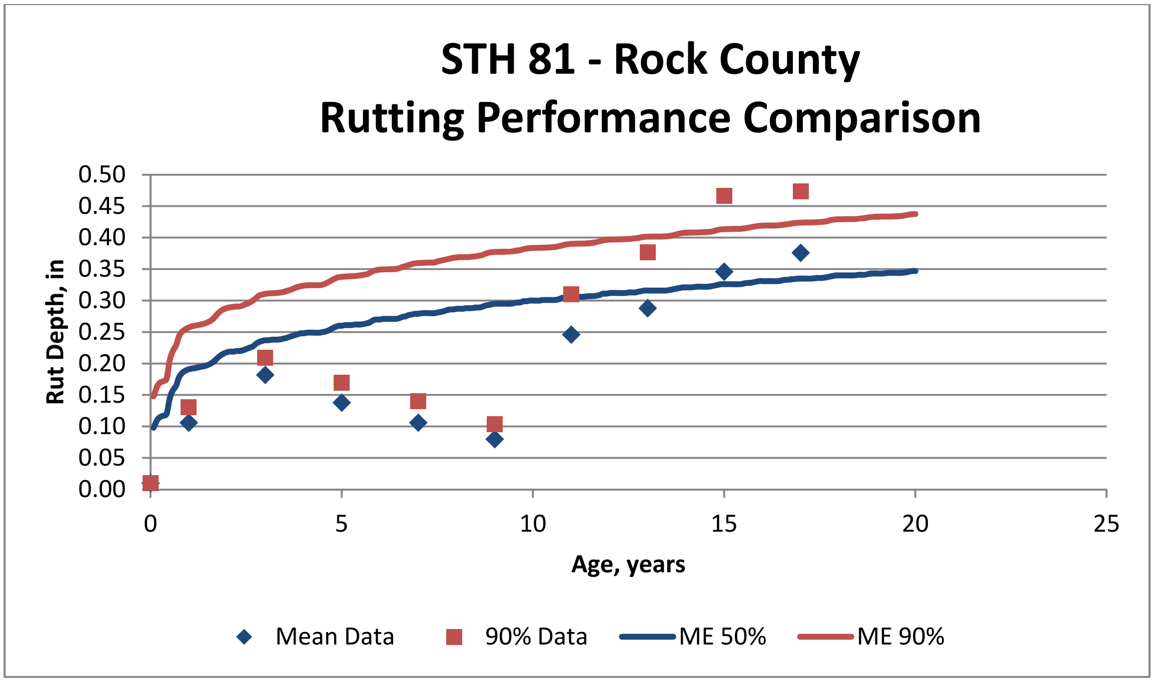

Figure 6.32 Observed and Calibrated Rutting Trends for STH 81 – Rock County

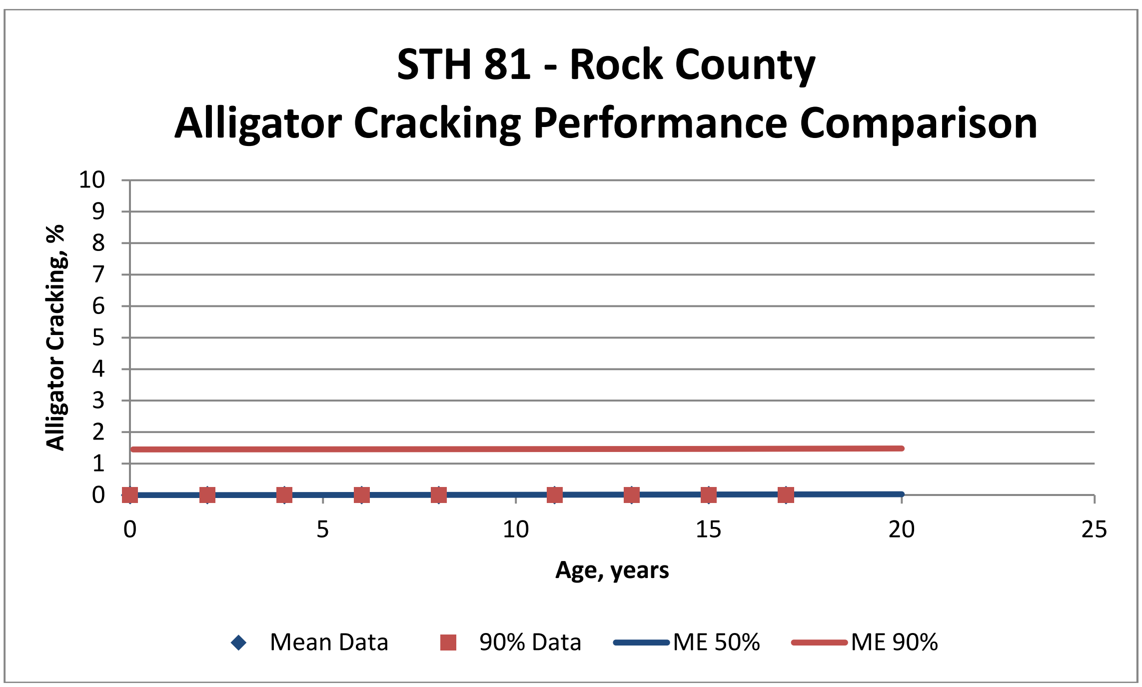

Figure 6.33 Observed and Calibrated Alligator Cracking Trends for STH 81 – Rock County

Table 6.2 Final Calibration Settings for HMA Sections

Test Section	HMA Rutting Calibration Factors			Alligator Cracking Calibration Factor
	Br1 - HMA	Bs1 - Base	Bs1 - Subgrade	
STH 35 Pierce Co	0.1	0.05	0.5	0.95
STH 39 Iowa Co	0.1	0.05	1.0	0.92
STH 40 Rusk Co	0.1	0.05	0.9	1.0
USH 53 Douglas Co	0.1	0.05	0.4	1.0
USH 61 Grant Co	0.1	0.05	0.9	1.1
USH 67 Dodge Co	0.1	0.05	0.9	1.1
STH 73 Dane Co	0.1	0.05	0.6	1.1
STH 73 Waushara Co	0.1	0.05	0.6	1.1
STH 81 Rock Co	0.4	0.20	1.5	1.1

Chapter 7 – Conclusions and Recommendations

The Wisconsin Department of Transportation (WisDOT) is in the process of implementing AASHTO's Mechanistic-Empirical Pavement Design Guide (MEPDG). Considerable resources have been invested in local material characterization. The objectives of this research were to develop a database of information on Wisconsin pavements, use that database to examine the MEPDG performance prediction models, and either validate the adequacy of the national calibration factors used in the models, or, to the extent that the national calibration factors do not yield sufficiently adequate predictions, determine new calibration factors that better represent Wisconsin conditions and the performance of Wisconsin's pavements.

The MEPDG software is a tool for analyzing designs for new and rehabilitated pavement structures. The MEPDG software does not generate a pavement design for a set of input conditions, nor does it search through a database of feasible designs. Rather, it evaluates a trial design that is input by the user, and predicts the performance of that trial design. The complexity of the MEPDG's pavement performance prediction models, load response models, and climatic models necessitate the use of computer software, the current version of which is available as AASHTOWare DARWin-ME™.

Many sensitivity analyses of the MEPDG performance prediction models have been conducted in the past 8 years, although a good number of these were conducted using early versions of the software. The most recent and most comprehensive sensitivity analyses of the MEPDG models was conducted for NCHRP Project 1-47 and documented in NCHRP Report 372. Among its key findings are the following:

- For all pavement types and distresses, the most sensitive design inputs were those related to the bound surface layers (HMA, PCC).
- The sensitivity values for each combination of distress and design input did not vary substantially or systematically by climatic zone.
- For the HMA performance prediction models, only the HMA property inputs (the upper and lower limits of the HMA dynamic modulus master curve, HMA thickness, surface shortwave absorptivity, and Poisson's ratio) were consistently in the highest sensitivity categories.
- Little or no thermal cracking was predicted in any climate when the correct binder grade for the climate was used with the HMA thermal cracking model.
- For JPCP, slab width was consistently the most sensitive design input, followed by PCC properties (unit weight, coefficient of thermal expansion, strength, stiffness, and surface shortwave absorptivity), PCC thickness, and other geometric properties (lane width and joint spacing).

- Some of the models are surprisingly sensitive to inputs that are difficult to determine, such as the coefficient of thermal expansion of concrete, and the surface shortwave absorptivities of asphalt and concrete surfaces, and/or not usually measured, such as the Poisson's ratios of asphalt and concrete.

Similar observations by other researchers on the sensitivity of the MEPDG models to their inputs and other design factors are documented in the annotated bibliography to this report. The bibliography also documents the findings of various researchers on the sensitivity of MEPDG predictions to the level and accuracy of traffic inputs, on the role of various climatic factors in performance prediction, and other topics related to the use, agency implementation, and local calibration of the MEPDG.

Eighteen pavement sections, nine HMA and nine JPCP, were identified for inclusion in this study. Pavement performance data was obtained from WisDOT databases and construction materials were obtained by direct sampling after long-term trafficking and environmental exposure. The extracted materials were analyzed to develop Level 1 inputs to the MEPDG software; however, only Level 3 inputs were used for the HMA project analyses due to the unstable behavior of the MEPDG software to Level 1 inputs.

An examination of the sensitivity of the MEPDG performance prediction models to their calibration factors was conducted during this research. Many of the calibration factors exhibit reasonable sensitivity trends that indicate the direction in which and degree to which a model can be adjusted to achieve a better fit between observed and predicted distress or IRI values. However, some of the calibration factors for some models exhibit much more sensitive trends that only allow changes to the calibration factor values within a very small range and produce very large change in predicted distress values for those small changes to the calibration factor values. In the most extreme cases, the software fails to execute correctly and reach a solution when certain calibration factors are varied considerably from their default values.

Repeated trials using the MEPDG software resulted in the development of project specific calibration factors which were able to bring the MEPDG predictions of JPCP cracking, and HMA rutting and alligator cracking in line with observed field performance. Correlations between these calibration factors and pavement material properties were investigated in an attempt to generate a rational approach for establishing these inputs a priori during the pavement design/analysis stage. While rough predictive models were developed, more work needs to be done in this area to minimize performance prediction errors during routine usage.

Additional calibration efforts should be completed using the recently released DARWin-ME software to refine the results of this study and to extend their applicability to current design standards. Trial runs completed with the DARWin-ME software, using the MEPDG calibrated

JPCP coefficients and the site average E228 coefficient of thermal expansion (CTE = 5.5 $\mu\varepsilon/^{o}F$), resulted in a significantly different pavement performance prediction for slab cracking, as illustrated in Figure 7.1 for STH 29 – Chippewa County. The sensitivity of the DARWin-ME predictions, based on the input CTE value, is provided in Figure 7.2. As shown, increasing the CTE value to 6.0 $\mu\varepsilon/^{o}F$, which is slightly higher than the value of 5.9 $\mu\varepsilon/^{o}F$ used during MEPDG calibrations, still resulted in a mean cracking prediction lower than field observations.

Pavement design standards for jointed concrete pavements in Wisconsin have changed, resulting in dowel bars sizes and joint spacings significantly different than those used during the construction of the sections used for these calibration efforts. A sensitivity analysis using current design standards over a range of traffic intensities, soil support values and environmental conditions should be completed to fully assess the impacts of the design standards.

For the HMA pavements with level 3 HMA inputs, performance predictions using DARWin-ME were in good agreement with those generated with the MEPDG software. DARWin-ME performance predictions using Level 1 HMA inputs are similar to the predictions using Level 3 HMA inputs, as shown in Figure 7.3 for STH 40 – Rusk County. Further refinement to the local calibration factors may be needed if Level 1 HMA inputs will be utilized for routine pavement design evaluations.

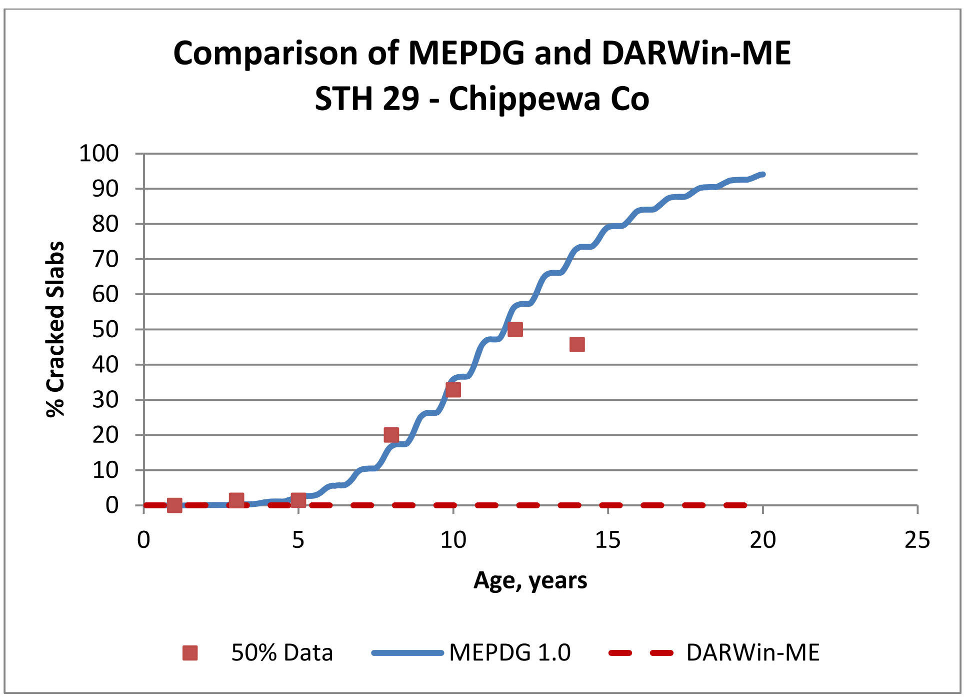

Figure 7.1 Comparison of MEPDG and DARWin-ME Outputs for STH 29 – Chippewa County

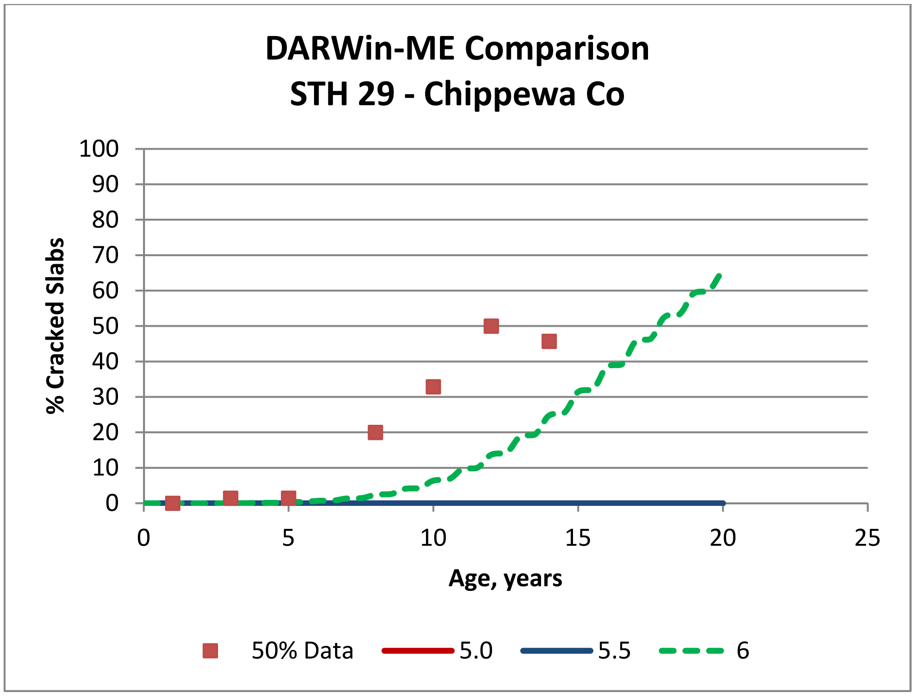

Figure 7.2 Sensitivity of DARWin-ME Outputs to Input CTE Value

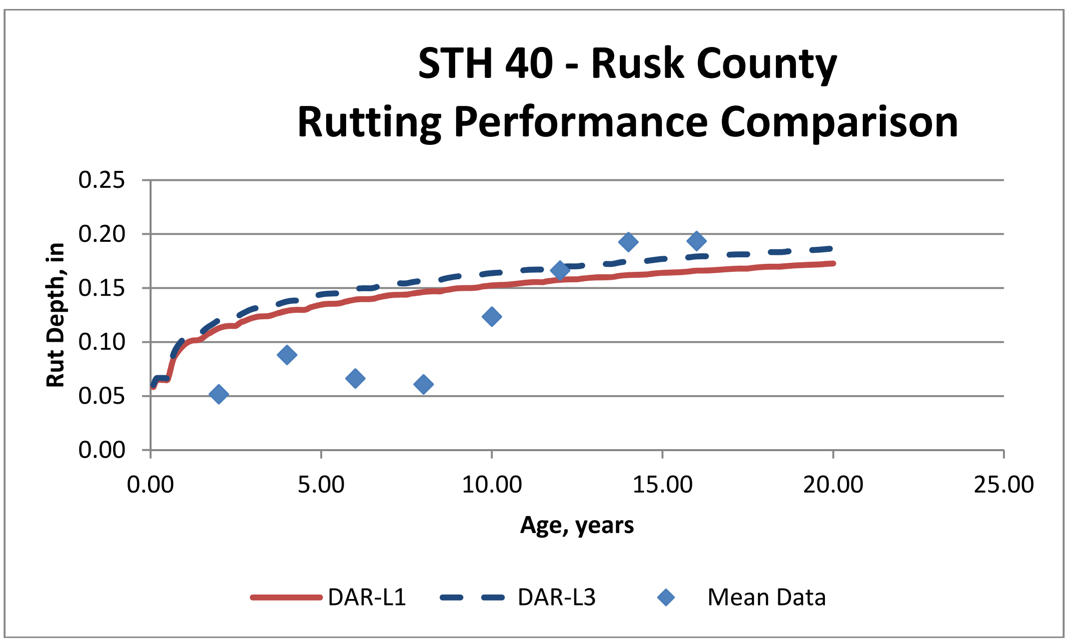

Figure 7.3 Comparison of DARWin-ME Outputs with Level 1 and Level 3 HMA Inputs

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Appendix A– Annotated Bibliography

Hundreds of papers and reports have been written on aspects of the MEPDG's performance predictions and how they compare with field measurements of pavement distress and roughness, and dozens more research studies related to implementing the MEPDG are in progress. A broad selection of the most recent references for completed studies and research in progress related to the MEPDG are summarized in an annotated bibliography provided as an appendix to this report. These references are presented in the following categories: sensitivity analysis, traffic, climate, materials, and other topics.

Sensitivity Analysis

Key Reference: Schwartz, C. R., Li, R., Kim, S. H., and Ceylan, H., *Sensitivity Evaluation of MEPDG Performance Prediction*, Report No. 372, National Cooperative Highway Research Program, Transportation Research Board, National Research Council, Washington, DC, 2011. This report documents and presents the results of a study to evaluate the sensitivity of pavement performance predicted by the Mechanistic-Empirical Pavement Design Guide to the values of the design inputs. 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. Detailed traffic inputs were not considered. Depending on the base case, approximately 25 to 35 design inputs were evaluated in the analyses. Correlations among design inputs (e.g., between PCC elastic modulus and modulus of rupture) were considered where appropriate. A normalized sensitivity index defined as the percentage change of predicted distress relative to its design limit caused by a given percentage change in the design input. The analyses revealed that, for all pavement types and distresses, the sensitivities of the design inputs for the bound surface layers were consistently the highest. Additional findings are also reported for each specific pavement type.

Pre-Version 1.0 Sensitivity Analyses: The findings of the sensitivity analyses of the MEPDG pavement models listed in this section, published between 2004 and 2007, were based on versions of the MEPDG software prior to Version 1.0. Summary comments are not included, as the findings from these studies are not reliably relevant to Versions 1.0 and 1.1 of the software.

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Khazanovich, L., Darter, M. I., and Yu, H. T., “Mechanistic-Empirical Model to Predict Transverse Joint Faulting,” Transportation Research Record 1896, Transportation Research Board, National Research Council, Washington, DC, 2004, pp. 34–45.

Kim, S., Ceylan, H., Gopalakrishnan, K., and Heitzman, M., “Sensitivity Study of Iowa Flexible Pavements Using Mechanistic-Empirical Pavement Design Guide,” Annual Meeting of the Transportation Research Board, National Research Council, Washington, DC, 2006.

Kim, S., Halil, C., and Kasthurirangan, G., “Effect of M-E Design Guide Inputs on Flexible Pavement Performance Predictions,” *Road Materials and Pavement Design*, Vol. 8, No. 3, 2007, pp. 375–397.

Lee, M. C., *Mechanistic-Empirical Pavement Design Guide: Evaluation of Flexible Pavement Inputs*, M.S thesis, University of Arkansas, AK, 2004.

Masad, S. A., *Sensitivity Analysis of Flexible Pavement Response and AASHTO 2002 Design Guide for Properties of Unbound Layers*, M.S. Thesis, Texas A&M University, College Station, TX, 2004.

Masad, S., and Little, D. N., *Sensitivity Analysis of Flexible Pavement Response and AASHTO 2002 Design Guide for Properties of Unbound Layers*, Project No. ICAR 504-1, International Center for Aggregates Research, Austin, TX, 2004.

Mohammad, L. N., Wu, Z., Obulareddy, S., Cooper, S., and Abadie, C., “Permanent Deformation Analysis of Hot-Mix Mixtures Using Simple Performance Tests and 2002 Mechanistic- Empirical Pavement Design Software,” Transportation Research Record 1970, Transportation Research Board, National Research Council, Washington, DC, 2006, pp. 133–142.

Rao, C., Selezneva, O., Darter, M. I., Titus-Glover, L., and Khazanovich, L., “Calibration of Mechanistic-Empirical Performance Model for Continuously Reinforced Concrete Pavement Punch-outs,” Transportation Research Record 1896, Transportation Research Board, National Research Council, Washington, DC, 2004, pp. 15–22.

Schwartz, C. W., “Evaluation of the Witczak Dynamic Modulus Prediction Model,” Annual Meeting of the Transportation Research Board, National Research Council, Washington, DC, 2005.

Selezneva, O., Rao, C., Darter, M. I., Zollinger, D., and Khazanovich, L., “Development of a Mechanistic-Empirical Structural Design Procedure for Continuously Reinforced Concrete Pavements,” Transportation Research Record 1896, Transportation Research Board, National Research Council, Washington, DC, 2004, pp. 46–56.

Yin, H., Chehab, G. R., and Stoffels, S. M., "A Case Study: Assessing the Sensitivity of the Coefficient of Thermal Contraction of AC mixtures on Thermal Crack Prediction," Asphalt Concrete: Simulation, Modeling, and Experimental Characterization–Proceedings of the Symposium on Mechanics of Flexible Pavements, part of the 2005 Joint ASME/ASCE/SES Conference on Mechanics and Materials, Geotechnical Special Publication, No. 146, 2006, pp. 115–123.

Zaghloul, S., Ayed, A., Halim, A, A, E., Vitillo, N., and Sauber, R., "Investigations of Environmental and Traffic Impacts on Mechanistic-Empirical Pavement Design Guide Predictions," Transportation Research Record 1967, Transportation Research Board, National Research Council, Washington, DC, 2006, pp. 148–159.

Versions 1.0/1.1 Sensitivity Analyses: The findings of the sensitivity analyses of the MEPDG pavement models listed in this section, published between 2007 and 2010, were based on Versions 1.0 or 1.1 of the MEPDG software. Abstracts and/or summary comments are included for some of these references. Summary comments for some references are taken from NCHRP Report 372, *Sensitivity Evaluation of MEPDG Performance Prediction,* 2011.

Aguiar-Moya, J.P., Banerjee, A., and Prozzi, J.A., "Sensitivity Analysis of the MEPDG Using Measured Probability Distributions of Pavement Layer Thickness," Annual Meeting of the Transportation Research Board, National Research Council, Washington, DC, 2009. Aguiar-Moya et al. used ground penetrating radar to determine the thickness distributions of the HMA surface layer, binder course, and granular base layer for LTPP SPS-1 sections in Texas. The vast majority of the analyzed pavement layers were found to have normally distributed thicknesses. The variations in thickness along the lane centerline and under the right wheel-path were also determined. MEPDG sensitivity analyses were performed based on the measured coefficients of variation for the thicknesses of the HMA surface and granular base layers. The results showed considerable changes in distress, especially fatigue cracking, for layer thicknesses variations within ±3 standard deviations from the mean. (Summary comments from NCHRP Report 372.)

Agauiar-Moya, J. P. and Prozzi, J. A., "**Effect of Field Variability of Design Inputs on the MEPDG," Paper 11-1202, Annual Meeting of the Transportation Research Board, National Research Council, Washington, DC, 2011.** There are many sources of variability that have an effect on how a given pavement structure is going to perform in the field. There is variability associated with the material properties, environmental conditions, traffic loading, structural layout, and construction practices. This paper assesses the variability of several design parameters that have been identified to have an important effect on the deterioration of pavement structures as predicted by the current version of the Mechanistic-Empirical Pavement Design Guide (MEPDG). The design variables that are analyzed include thickness of the hot-mix asphalt (HMA) layer, asphalt binder content, air void content, thickness of the base, resilient modulus of the HMA layer, modulus of the base, and modulus of the subgrade. Actual field information on each of these properties has been obtained from FHWA's Long-Term Pavement Performance Database (LTPP). The variability of the previous variables is measured by means of statistics such as mean, standard deviation, and coefficients of variation (CV). Additionally, the datasets obtained for each one of the previous design variables have been tested to evaluate the feasibility that the sample data follow a normal distribution or otherwise. It was identified that most of these design variables have considerable variability from their mean values (high standard deviations), mainly in the case of the modulus of the HMA layers. Also, it was concluded that it is likely that most of the studied design characteristics are likely to follow normal distributions. Finally, the effect of the variability of the selected design variables on the deterioration of a pavement structure as predicted by the MEPDG was evaluated. A large number of simulations based on the MEPDG predictions by means of response surfaces indicated that the typical variability in the design variables can result in doubling of the rutting and fatigue cracking, and up to six times increase in terminal IRI, as compared to the mean design values. This conclusion emphasizes the urgent need for continuing the research on the MEPDG to incorporate sound reliability estimations.

Ahn, S., Kandala, S., Uzan, J., and El-Basyouny, M., "Comparative Analysis of Input Traffic Data and MEPDG Output for Flexible Pavements in the State of Arizona," Annual Meeting of the Transportation Research Board, National Research Council, Washington, DC, 2009. Ahn et al. focused this study on the effects of input traffic parameters on the MEPDG pavement performance. Traffic inputs studied were annual average daily truck traffic (AADTT), monthly adjustment factors (MAF), and axle load distribution factors. AADTT was found to have a significant effect on predicted performance, especially fatigue cracking; MAF was not found to have a significant influence. The accuracy of pavement prediction increased when using WIM-collected axle load distributions from Arizona as compared to the MEPDG default values. (Summary comments from NCHRP Report 372.)

Ayyala, D., Chehab, G. R., and Daniel, J. S., "Sensitivity of MEPDG Level 2 and 3 Inputs Using Statistical Analysis Techniques for New England States," Paper 10-3694, Annual Meeting of the Transportation Research Board, National Research Council, Washington DC, 2010. Ayyala et al. evaluated the sensitivity of thirteen MEPDG inputs for base cases consisting of one LTPP section each from NH and CT. MEPDG Version 1.0 was used for all distresses except thermal cracking; Version 0.91 was used for thermal cracking evaluation as no distress was predicted by Version 1.0. Sensitivity was quantified in terms of correlation ratios between inputs and outputs. Design inputs were assigned to quantitative sensitivity categories tabulated by distress. (Summary comments from NCHRP Report 372.)

Buch, N., Chatti, K., Haider, S. W., and Manik, A., *Evaluation of the 1-37A Design Process for New and Rehabilitated JPCP and HMA Pavements*, Research Report RC-1516, Michigan State University, East Lansing, MI, 2008. Buch et al. and Haider et al. conducted comprehensive sensitivity analyses for rigid pavements. The approach in these studies included: one-at-a-time analyses to investigate the effect of individual input variables on performance (preliminary sensitivity analyses), and (2) full factorial analyses to evaluate the interaction effects of input variables on performance (detailed sensitivity analyses). The preliminary sensitivity results identified 23 sensitive input variables for the three-level full factorial design matrix. This list was subsequently reduced using various criteria to decrease the number of MEPDG runs. The analysis results revealed that the effect of PCC slab thickness and edge support on performance were most significant among the design inputs and that CTE, modulus of rupture (MOR), base type and subgrade type material inputs also played an important role. In terms of input variable interaction effects, slab thickness-CTE-MOR, CTE-MOR-subgrade soil type-climate, and slab thickness-CTE-subgrade soil type-climate interactions were found to be significant for JPCP cracking, faulting, and roughness (IRI), respectively. (Summary comments from NCHRP Report 372.)

Ceylan, H., Schwartz, C.W., Kim, S., and Gopalakrishnan, K., "Accuracy of Predictive Models for Dynamic Modulus of Hot-Mix Asphalt," ***Journal of Materials in Civil Engineering*****, ASCE, Vol. 21, No. 6, 2009, pp. 286–293.** Ceylan et al. identified curl/warp effective temperature difference and the PCC coefficient of thermal expansion (CTE) as being the most sensitive inputs influencing predicted rigid pavement performance using all three versions (0.7, 0.9, and 1.0) of MEPDG software. (Summary comments from NCHRP Report 372.)

Clark, T., "Sensitivity Analyses for Flexible Pavement Design with the Mechanistic–Empirical Pavement Design Guide–Conclusion," ***Engineering Circular*** **155, Transportation Research Board, National Research Council, Washington, DC, 2011.** This workshop's mission was to inform the pavement engineering community on the completed and on-going efforts related to assessing the sensitivity of the Mechanistic-Empirical Pavement Design Guide (MEPDG). Specifically, the workshop was concerned with those parameters that had an impact on flexible pavement analysis and design. Many transportation agencies have been involved in various studies to look at particular parts of the MEPDG, but much of this work had not been compiled into a single document. As such, a workshop was proposed by the Transportation Research Board (TRB) Flexible Pavement Design Committee (AFD60) and approved by TRB to look at the flexible pavement sensitivity analysis in the MEPDG. Once accepted, a planning team was established to develop the workshop by collecting and disseminating the work done by transportation agencies. The workshop planning team had two primary goals: (1) Take a snapshot of the current implementation status of transportation agencies through a questionnaire and reporting on workshops hosted by the Federal Highway Administration and (2) Invite transportation agencies based on their responses to the questionnaire to present on a specific subject or overall research implementation effort. Additionally, the planning team wanted to capture and present current National Cooperative Highway Research Program research related to flexible pavement analysis and performance. Workshop Session 143, held in January 2010, met these goals by providing presentations on various efforts related to understanding the sensitivity of flexible pavement performance using the MEPDG inputs. http://onlinepubs.trb.org/onlinepubs/circulars/ec155.pdf.

Haider, S. W., Buch, N., and Chatti, K., "Evaluation of MEPDG for Rigid Pavements–Incorporating the State of the Practice in Michigan," Proceedings of the 9th International Conference on Concrete Pavements, San Francisco, California, August 17-21, 2008, pp. 111–133. Buch et al. and Haider et al. conducted comprehensive sensitivity analyses for rigid pavements. The approach in these studies included: one-at-a-time analyses to investigate the effect of individual input variables on performance (preliminary sensitivity analyses), and (2) full factorial analyses to evaluate the interaction effects of input variables on performance (detailed sensitivity analyses). The preliminary sensitivity results identified 23 sensitive input variables for the three-level full factorial design matrix. This list was subsequently reduced using various criteria to decrease the number of MEPDG runs. The analysis results revealed that the effect of PCC slab thickness and edge support on performance were most significant among the design inputs and that CTE, modulus of rupture (MOR), base type and subgrade type material inputs also played an important role. In terms of input variable interaction effects, slab thickness-CTE-MOR, CTE-MOR-subgrade soil type-climate, and slab thickness-CTE-subgrade soil type-climate interactions were found to be significant for JPCP cracking, faulting, and roughness (IRI), respectively. (Summary comments from NCHRP Report 372.)

Haider, S. W., Buch, N., and Chatti, K., "Simplified Approach for Quantifying Effect of Significant Input Variables and Designing Rigid Pavements using M-E PDG," Annual Meeting of the Transportation Research Board, National Research Council, Washington, DC, 2009. Haider et al. proposed simplified JPCP performance models for use during the initial design stage to quantify the impact of design-related input variables on expected performance. (Summary comments from NCHRP Report No. 372.)

Hall, K. D., Xiao, X. D., and Wang, K. C. P., "Thickness Estimation of Existing Pavements Using Nondestructive Techniques: Matching Accuracy to Application," Annual Meeting of the Transportation Research Board, National Research Council, Washington, DC, 2010. Hall et al. examined HMA overlay designs (HMA over HMA and HMA over PCC) using the MEPDG to assess the relative sensitivity to variations in surface layer thickness. The analyses indicated that overlaid HMA bottom-up fatigue and rutting will not likely be sensitive to variations in existing (underlying) surface layer thickness (existing HMA or PCC). On the other hand, the current models for top-down cracking and reflection cracking in MEPDG appear to be more sensitive to existing surface layer thickness. (Summary comments from NCHRP Report 372.)

Hiller, J. E., and Roesler, J. R., "Comparison of Mechanistic-Empirical Thickness Design Methods and Predicted Critical Fatigue Locations," Proceedings of the 9th International Conference on Concrete Pavements, San Francisco, California, August 17-21, 2008, pp. 171–188. Hiller and Roesler demonstrated the sensitivity of input parameters such as joint spacing, shoulder type, traffic level, climate, and built-in curl level on JPCP fatigue failure mechanisms predicted by MEPDG as well as by several other design methods. They found that the level of built-in curl significantly affected the required slab thickness in the MEPDG and that the required thickness increased as the built-in curl level became more negative. (Summary comments from NCHRP Report 372.)

Johanneck, L., and Khazanovich, L., "A Comprehensive Evaluation of the Effect of Climate in MEPDG Predictions," Annual Meeting of the Transportation Research Board, National Research Council, Washington, DC, 2010. Johanneck and Khazanovich conducted a comprehensive evaluation of the effect of climate in MEPDG predictions for an asphalt-over-concrete (AC/PCC) composite pavement. As expected, it was found that environment has a significant impact on predicted pavement performance. They also found that some of the weather stations included in the MEPDG database may be of insufficient quality to obtain reliable pavement performance predictions. (Summary comments from NCHRP Report 372).

Kapmann, R., *Engineering Properties of Florida Concrete Mixes for Implementing the AASHTO Recommended Mechanistic-Empirical Rigid Pavement Design Guide*, M.S. Thesis, Florida State University, FL, 2008. This study by Kampmann focused exclusively on the effect of CTE on MEPDG-predicted JPCP performance measures, based on experimental analysis of three typical Florida concrete mixtures. This study found that CTE was least sensitive for JPCP faulting, reasonably sensitive for bottom-up damage for thin PCC layers, and extremely sensitive for top-down damage, cracking, and smoothness. Laboratory test results also revealed that CTE rapidly increases within the first week after paving but stabilizes after 28 days. (Summary comments from NCHRP Report 372.)

Mallela, J. and Donahue, J., "Sensitivity Analysis as Decision Support Tool for Missouri Department of Transportation MEPDG Implementation," ***Engineering Circular*** **155, Transportation Research Board, National Research Council, Washington, DC, 2011.** This presentation overviews the concepts of sensitivity analysis and applications of the results in guiding decisions during state-specific implementation of AASHTO's interim Mechanistic-Empirical Pavement Design Guide (MEPDG). The results from the sensitivity studies performed during the local calibration of the MEPDG models for the Missouri Department of Transportation (DOT) are specifically referenced. The presentation underscores the importance of sensitivity analysis as a decision support tool in locally calibrating the MEPDG performance models and illustrates an approach for setting up a successful sensitivity study. Results from the sensitivity analysis conducted for new jointed plain concrete pavements and asphalt concrete pavements are presented. In addition, a discussion of how the results were used by Missouri DOT is provided.

McCarthy, L. M. and Liang, J., "Sensitivity Analysis for Flexible Pavement Design Using the Mechanistic-Empirical Pavement Design Guide," ***Engineering Circular*** **155, Transportation Research Board, National Research Council, Washington, DC, 2011.** Over the last few years, transportation agencies have had the opportunity to use AASHTO's interim Mechanistic–Empirical Pavement Design Guide (MEPDG) software. This software allows users to assess the impacts of traffic, climate, materials properties, etc. on the predicted pavement performance. Several transportation agencies have begun the process of implementing the design process. However, many agencies are just starting the implementation process or are waiting to see the results from other states. As such, the Transportation Research Board Flexible Pavement Design Committee (AFD60) requested assistance from state agencies in collecting and disseminating information and results related to sensitivity analysis of flexible pavement designs performed by transportation agencies. A survey similar to the Federal Highway Administration (FHWA) MEPDG survey used earlier in the decade was circulated via electronic mail during the summer of 2009. The survey questions and summary of responses are provided in this presentation. A total of 52 agencies participated in the study, including 48 of the 50 U.S. states. The other agencies were the District of Columbia Department of Transportation, Puerto Rico, the FHWA Federal Lands Division, and the Ontario Ministry of Transportation.

McCracken, J. K., Vandenbossche, J. M., and Asbahan, R. E., "Effect of the MEPDG Hierarchal Levels on the Predicted Performance of a Jointed Plain Concrete Pavement," Proceedings of the 9th International Conference on Concrete Pavements, San Francisco, California, August 17-21, 2008, pp. 153–170. In this study evaluating the effect of the MEPDG hierarchical levels on JPCP predicted performance, McCracken et al. concluded that the PCC CTE and MOR were the most sensitive inputs on predicted JPCP performance. The hierarchal level of individual design inputs was also found to be significant in some cases. For example, it was noted that the use of different levels (1, 2, and 3) for CTE input changed the design thickness by up to 51 mm (2 in). On the other hand, they did not find that the slab thickness was significantly influenced by the input levels used in defining the PCC strength and subgrade resilient modulus or the use of different climatic weather stations. (Summary comments from NCHRP Report 372.)

Moon, W., *Evaluation of MEPDG with TxDOT Rigid Pavement Database,* Research Report FHWA/TX-09/0-5445-3, Center for Transportation Research, The University of Texas at Austin, Austin, TX, 2009. Moon evaluated the Texas DOT rigid pavement database with the MEPDG and found that the zero-stress temperature and built-in curling have substantial effects on CRCP punch-out development and thus on the design slab thickness. (Summary comments from NCHRP Report 372.)

Oh, J., and Fernando, E. G., *Development of Thickness Design Tables Based on the M-E PDG*, Research Report BDH10-1, Texas Transportation Institute, College Station, TX, 2008. As part of their research on developing thickness design tables based on the MEPDG under Florida conditions, Oh and Fernando conducted rigid pavement sensitivity studies that identified PCC CTE and compressive strength as the most significant variables impacting projected performance measures. Other sensitive variables identified were joint spacing, dowel diameter, and slab width. However, the moduli of the unbound materials and MOR were found to have minimal effect on the projected JPCP performance measures. (Summary comments from NCHRP Report 372.)

Oman, M. S., "MnROAD Traffic Characterization for the Mechanistic-Empirical Pavement Design Guide Using Weigh-in-Motion Data," Annual Meeting of the Transportation Research Board, National Research Council, Washington, DC, 2010. Oman conducted a MEPDG traffic characterization study using the weigh-in-motion data acquired at the MnRoad research facility. The study results revealed that the MEPDG predicted distresses are sensitive to the axle load spectra and the percentage of vehicles in the design lane. Further, the Monthly Adjustment Factors (MAF) were found to have moderate impact on the predicted distresses, whereas the axle groups per vehicle showed least sensitivity in terms of predicted performance. The hourly distribution factors seem to influence cracking in rigid pavements. (Summary comments from NCHRP Report 372.)

Orobio, A. and Zaniewski, J. P., "Sampling-Based Sensitivity Analysis of the Mechanistic-Empirical Pavement Design Guide Applied to Material Inputs," *Transportation Research Record* 2226, Transportation Research Board, National Research Council, Washington, DC, 2011. The Mechanistic-Empirical Pavement Design Guide (MEPDG) is the result of NCHRP Project 1-37A. This pavement design procedure uses mechanistic and empirical models calibrated with national data. Calibration of MEPDG is required to improve the accuracy of the models for local conditions. The data needed for calibration require numerous tests to characterize materials. The fieldwork for collecting the data to verify the models is laborious. This research studied the sensitivity of MEPDG to material input parameters. The main goal of this research was to determine the influence of material parameters on pavement performance as predicted by MEPDG. Results from sensitivity analyses can be used for planning data collection for calibration, implementation, and general understanding of MEPDG. However, MEPDG is so complex that the sensitivity analysis methodology was carefully designed to identify the relative importance of input variables. This research employed space-filling computer experiments with Latin hypercube sampling, standardized regression coefficients, and Gaussian stochastic processes to categorize the relative importance of material inputs in MEPDG for two flexible pavement structures that used Level 3 analysis. The methodology worked well for analyzing the sensitivity of material inputs in MEPDG, with the advantages of varying all parameters at once and requiring a relatively small number of runs for a completed analysis of the entire input space. Effective binder content, as-built air voids, Poisson's ratio, surface shortwave absorption of asphalt layers, and resilient modulus of subgrade had a significant effect on pavement performance.

Puertas, J. J. G., *Evaluating the JPCP Cracking Model of the Mechanistic-Empirical Pavement Design Guide*, M.S. Thesis, University of Pittsburgh, PA, 2008. Puertas performed several sensitivity analyses of the MEPDG JPCP cracking model. The study found that anomalies exist in the MEPDG prediction of JPCP top-down fatigue cracking and that the cracking results contradict current engineering understanding of this failure mechanism. This was thought to be due to inadequate treatment of certain newly introduced inputs such as the permanent built-in temperature gradient (or permanent curl-warp effective temperature difference). Recommendations from this study include use of the response surface methodology (RSM) to support more exhaustive sensitivity analyses covering a wider range of input values. (Summary comments from NCHRP Report 372.)

Retherford, J. Q., "Estimation and Validation of Gaussian Process Surrogate Models for Sensitivity Analysis and Design Optimization Based on the Mechanistic-Empirical Pavement Design Guide," *Transportation Research Record* 2226, Transportation Research Board, National Research Council, Washington, DC, 2011. The Mechanistic-Empirical Pavement Design Guide (MEPDG) is a powerful predictor of pavement distress, but it is computationally expensive to evaluate. Analyses that require many MEPDG evaluations, such as sensitivity analysis and design optimization, become impractical because of the computational expense. These applications are important in achieving robust, reliable, and cost-effective pavement designs. This paper develops Gaussian process (GP) surrogate models that, with a trivial amount of computational expense, accurately approximate the results of the MEPDG for each relevant distress mode. The GP is validated in accordance with three model metrics: average predictive percent error, predictive coefficient of determination, and Bayes factor. The GP models are then exploited for sensitivity analysis and design optimization, making these tasks computationally affordable.

Schwartz, C. W. and Li, R., "Sensitivity of Predicted Flexible Pavement Performance to Unbound Material Hydraulic Properties, " *Advances in Analysis, Modeling, and Design* (GeoFlorida 2010), ASCE, 2010. Schwartz and Li quantified the sensitivity of predicted pavement distresses to subgrade type, groundwater table depth, saturated hydraulic conductivity, and soil water characteristic curve parameters analyses for three flexible pavement sections in four climate locations. In overall terms, traffic volume and subgrade resilient modulus were the two inputs of those studied that had the largest impact on predicted distresses. The environmental inputs related to groundwater depth, soil water characteristic curve parameters, and saturated hydraulic conductivity all had slight to negligible influence on the predicted distresses.

Variations of performance with climate location and subgrade type were sensible. (Note: Additional unpublished analyses found that predicted distresses were moderately to highly sensitive to surface shortwave absorptivity and generally negligibly sensitive to HMA thermal conductivity and heat capacity.) (Summary comments from NCHRP Report 372.)

Tanesi, J., Kutay, M. E., Abbas, A. R., and Meininger, R. C., "Effect of Coefficient of Thermal Expansion Test Variability on Concrete Pavement Performance as Predicted by Mechanistic- Empirical Pavement Design Guide," ***Transportation Research Record*** **2020, Transportation Research Board, National Research Council, Washington, DC, 2007, pp. 40–44.** Tanesi et al. investigated the effect of MEPDG hierarchical input levels for CTE on predicted JPCP performance. The effect of PCC CTE input on predicted IRI was more pronounced for JPCP with thinner slabs or lower PCC strengths. A combination of high cement factor and higher PCC CTE produced higher JPCP faulting. In general, faulting was very sensitive to this input. PCC CTE also had a very significant effect on slab cracking. However, it did not affect the predicted IRI for a JPCP with widened lane and tied PCC shoulder. A level 2 CTE input may result in a more conservative JPCP design than that obtained using a Level 1 input. The authors suggested that the detrimental effects of high CTE value could be mitigated using higher PCC slab thickness, larger-diameter dowel bars or a widened lane with a tied PCC shoulder. They also recommended not using a single test result as representative of the CTE of a mixture and implementing a specification for the minimum number of tests and the acceptable test variability. (Summary comments from NCHRP Report 372.)

Thyagarajan, S., Sivaneswaran, N., Muhunthan, B., and Petros, K., "Statistical Analysis of Critical Input Parameters in the Mechanistic Empirical Pavement Design Guide," ***Journal of the Association of Asphalt Paving Technologists*****, Vol. 79, 2010, pp. 635–662.** Sensitivity studies conducted by Thyagarajan et al. focused on the influence of air voids, effective binder content, and gradation inputs to the Witczak |E*| predictive model on predicted performance. Only one pavement scenario (structural section, binder grade, traffic level) was considered. Sensitivity was characterized via tornado plots of Spearman correlation coefficients and by extreme tail analysis. Air voids were found to be the most sensitive parameter affecting all predicted distress modes. The percents passing the No. 200 sieve and retained on the No. 4 sieve were also found to have a significant influence on rutting, while effective binder content had a significant influence on fatigue cracking. Khazanovich et al. (2008) performed a very similar study. (Summary comments from NCHRP Report 372.)

Tran, N. H., Hall, K. D., and James, M., "Coefficient of Thermal Expansion of Concrete Materials: Characterization to Support Implementation of the Mechanistic-Empirical Pavement Design Guide," ***Transportation Research Record*** **2087, Transportation Research Board, National Research Council, Washington, DC, 2008, pp. 51–56.** Tran et al. focused on characterizing CTE in this study. They found that aggregate type has a significant influence on both the laboratory-measured CTE as well as on MEPDG pavement performance predictions. The MEDPG-recommended default CTE values were found appropriate for PCC mixtures with limestone and sandstone aggregates but not for PCC mixtures with gravel. (Summary comments from NCHRP Report 372.)

Transportation Research Board, ***Sensitivity Analyses for Flexible Pavement Design with the Mechanistic-Empirical Pavement Design Guide*****,** ***Engineering Circular*** **155, National Research Council, Washington, DC, 2011.** This circular contains the proceedings of a workshop that was held in conjunction with the 2010 TRB 89th Annual Meeting. The workshop was developed to provide information for transportation agencies in the process of, or considering the implementation of, the interim American Association of State Highway and Transportation Officials Mechanistic-Empirical Pavement Design Guide (MEPDG). The workshop shared experiences from transportation agencies that have performed various sensitivity analyses using the MEPDG software, primarily related to flexible pavement analysis. The participants explored which input factors are important to the final pavement designs, so that agencies can focus their research accordingly during the implementation process.

Yin, H., Chehab, G. R., Stoffels, S. M., Kumar, T., and Premkumar, L., "Use of Creep Compliance Interconverted from Complex Modulus for Thermal Cracking Prediction Using the M-E Pavement Design Guide," ***International Journal of Pavement Engineering*****, Vol. 11, No. 2, 2010, pp. 95–105.** Yin et al. (2010) compared thermal crack predictions from Level 1 vs. Level 2 vs. Level 3 creep compliance inputs as well as directly measured creep compliance vs. converted-from-E* values for Level 1/2 inputs. This paper did not evaluate sensitivity per se, but significant differences in predicted thermal cracking for the various input options suggest that thermal cracking is sensitive to creep compliance. (Summary comments from NCHRP Report 372.)

Traffic

Ahammed, M. A., Kass, S., Hilderman, S., and Tang, W. K. S., "MEPDG Implementation–Manitoba Experience," 2011 Conference and Exhibition of the Transportation Association of Canada, Edmonton, Canada, 2011. The new Mechanistic Empirical Pavement Design Guide (MEPDG) has been developed based on fundamental properties of materials and the physical observations of performance. It can be used for all truck volume and axle load scenarios. However, for a more reliable design, local material properties, climate data, truck volume and distributions, and axle load spectra (ALS) are critical. This paper presents the experience of Manitoba Infrastructure and Transportation (MIT) with the MEPDG in using local truck traffic data with an example of a flexible pavement design. The sensitivity of the program for changes in truck volume, ALS and truck distributions are presented. Analysis/experience showed that MEPDG produces designs with similar or thinner pavement structures for low truck volume but it overestimates the pavement structures for moderate to high truck volumes compared to the AASHTO 1993 and surface deflection methods. A significant variation in required structure was also noted for a within-province variation in the truck class distribution. This emphasizes the importance of calibrating the performance models to local conditions. The issues and challenges in calibrating the MEPDG performance models are also discussed.

Ahn, S., Kandala, S., Uzan, J., and El-Basyouny, M., "Impact of Traffic Data on the Pavement Distress Predictions using the Mechanistic Empirical Pavement Design Guide" ***Road Materials and Pavement Design*****, Volume 12, Number 1, 2011.** This study examines the adequacy of using conventional traffic data and national default values in the absence of weigh-in-motion (WIM) data for pavement design. A comparative study was conducted on 14 sections in Arizona, where WIM data is available through the Long Term Pavement Performance (LTPP) program. The study consists of 2 parts: 1) comparisons of input traffic data and 2) comparisons of pavement distresses predicted by the Mechanistic Empirical Pavement Design Guide (MEPDG). The traffic related input parameters include average design-lane truck volumes, Vehicle Classification Percentages (VCP), Monthly Adjustment Factors (MAF), axle load distribution factors, and the number of axles per truck. The truck volumes and VCPs are available through the Arizona Department of Transportation (ADOT) while only national average values are available for the other traffic inputs in the absence of WIM data. Comparisons of the input variables showed that the truck volumes for a design lane estimated from the ADOT database and default MAFs differed significantly from those in the LTPP database. The national default axle load distribution factors differed somewhat from the site-specific values. The differences in input data were reflected in the pavement distress values that were predicted by MEPDG. The outputs of the design guide reveal large prediction errors, particularly for longitudinal cracking, exceeding 40% in absolute percent error on average. The large difference in design-lane truck volume was the major source of the large prediction errors. The national default factors also generated moderate prediction errors, and performance improved slightly with the use of the Arizona average factors. Finally, the differences in MAF had little impact on predictions of pavement distress.

Cottrell, B. H. and Kweon, Y. J., ***Review of the Virginia Department of Transportation's Truck Weight Data Plan for the Mechanistic-Empirical Pavement Design Guide,*** **Report No. VCTIR 12-R4, Virginia Center for Transportation Innovation and Research, Charlottesville, VA, 2011.** In 2003, staff of the Virginia Transportation Research Council (now the Virginia Center for Transportation Innovation and Research) and the Virginia Department of Transportation (VDOT) developed a plan to collect traffic and truck axle weight data to support the Mechanistic-Empirical Pavement Design Guide (MEPDG). The purpose of this study was to review VDOT's traffic data plan for the MEPDG and revise it as needed. The review included an assessment of the data obtained from the VDOT and Virginia Department of Motor Vehicles weigh-in-motion (WIM) sites and the appropriateness of the truck weight groups in VDOT's traffic data plan. Information on truck travel patterns and characteristics was compiled. There is very little literature that provides specific information on the structure of a traffic data plan for the MEPDG. Guidance provided by the Federal Highway Administration allows for much flexibility in the development of such a plan. Most states are working to develop the plan, and such plans that are already in place vary considerably. The Corridors of Statewide Significance in Virginia's statewide long-range multimodal transportation plan represent the routes where truck traffic is most prominent and therefore represent routes on which the VDOT plan should focus. The study recommends that VDOT continue with its current truck weight data plan for the MEPDG. With this plan, VDOT is positioned to implement the MEPDG from a truck data perspective. The WIM data comprise an important input to the MEPDG process that is expected to provide VDOT with more accurate pavement designs based on actual traffic loadings in Virginia. Further, staff of VDOT's MEPDG Traffic Data Team and staff of the VDOT Traffic Engineering Division's Traffic Monitoring Program should work together to develop a strategic plan for the continuing incremental expansion of the WIM program. The plan should include consideration of the resources needed not only to add sites but also to administer an expanded WIM program. VDOT's Chief Engineer and Chief of System Operations should encourage the addition of WIM sites when major projects are planned in locations that are part of the strategic plan for WIM. Site characteristics required for acceptable WIM sensor performance should be specified by VDOT's MEPDG Traffic Data Team. Implementation of the recommendations provided in this study will assist VDOT in using the MEPDG to advance pavement design and improve its cost-effectiveness. The likelihood of implementation is high.

Diefenderfer, B., "Virginia Mechanistic-Empirical Pavement Design Guide Research: Influence of Traffic and Materials Research," ***Engineering Circular*** **155, Transportation Research Board, National Research Council, Washington, DC, 2011.** This presentation gives an overview of the research underway at the Virginia Transportation Research Council related to traffic and materials inputs for use with the Mechanistic- Empirical Pavement Design Guide (MEPDG). The presentation lists and shows examples of those traffic and materials inputs that are considered significant with respect to the MEPDG-predicted pavement conditions. Various methods to determine the significance of the inputs (by statistical or practical consideration of the predicted pavement conditions) are discussed. Two methods of calculating a normalized difference statistic are presented along with a brief description of regression analyses that could serve as examples for statistical-based analysis. Practical-based methods were suggested to include consideration of the time to failure as it relates to the timing of pavement maintenance activities. The presentation discussed preliminary findings in terms of a comparison of the predicted pavement condition to expected values based on field experience. The need for local calibration was discussed as a future need.

Haider, S. W., Buch, N., Chatti, K., and Brown, J., "Development of Traffic Inputs for Mechanistic-Empirical Pavement Design Guide in Michigan," ***Transportation Research Record*** **2256, Transportation Research Board, National Research Council, Washington, DC, 2011.** Characterizing traffic and developing accurate and desirable traffic inputs for the new Mechanistic–Empirical Pavement Design Guide (MEPDG) are critical but challenging activities. The purpose of this study was to develop a process for characterizing traffic inputs in support of the new MEPDG for the state of Michigan. These traffic characteristics include monthly distribution factors, hourly distribution factors, truck traffic classifications, axle groups per vehicle, and axle load distributions for different axle configurations. Axle weight and vehicle classification data were obtained from 44 weigh-in-motion and classification stations located throughout the state of Michigan to develop Level 1 (site-specific) traffic inputs. Cluster analyses were conducted to group sites with similar characteristics for development of Level 2 (regional) inputs. Finally, 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 pavements were investigated with the MEPDG models. An algorithm based on discriminant analysis was developed to acquire the appropriate Level 2 traffic characteristic inputs for pavement design. For pavement analysis and design, it is recognized that site-specific data should be used wherever available. For projects in which site-specific data are not available, it is necessary to know whether Level 2 or Level 3 data are acceptable at a minimum for design. The MEPDG was used to investigate the impact of traffic input levels on predicted pavement performance for rigid pavements. The results of the analysis showed that for pavement design in Michigan, statewide averages should be used instead of MEPDG Level 3 data.

Ishak, S., Shin, H. C., and Sridhar, B. K., ***Characterization and Development of Truck Load Spectra and Growth Factors for Current and Future Pavement Design Practices in Louisiana*****, Report No. FHWA/LA.11/445, Louisiana Department of Transportation and Development, Baton Rouge, LA, 2011.** For pavement design practices, several factors must be considered to ensure good pavement performance over the anticipated life cycle. Such factors include, but are not limited to, the type of paving materials, traffic loading characteristics, prevailing environmental conditions, and others. Traditional pavement design practices have followed the standards set by the American Association of State Highway and Transportation Officials (AASHTO) which require the use of an equivalent single axle load (ESAL), 18-kip single-axle load, for design traffic input. The new mechanistic-empirical pavement design guide (MEPDG) was developed to improve pavement design practices. The guide, however, requires the development of truck axle load spectra, which are expressed by the number of load applications of various axle configurations (single, dual, tridem, and quad) within a given weight classification range. This raises the need for more axle load data from new and existing traffic data sources. Such additional data requirements pose a challenge for many states including Louisiana. This research study was conducted for LADOTD to address traffic data needs and requirements for the adoption of the new pavement design guide. The study reviewed current practices of traffic data collection processes adopted by LADOTD as well as existing and newly proposed traffic data collection procedures followed by other states. The study developed a strategic plan for Louisiana to meet the MEPDG traffic data requirements. Two alternative plans were proposed for the addition of new permanent Weigh-in-Motion (WIM) stations on major truck routes as well as utilizing axle load data from the existing weight enforcement sites. Cost estimates were also provided for each plan. In addition, the study developed axle load spectra and vehicle class distributions using screened traffic data collected by portable WIM sites from 2004 to 2006. For current design practices, the study also utilized portable WIM data to update load equivalency factors (LEF) using the Vehicle Travel Information System (VTRIS) software.

Khanum, T., Hossain, M., and Schieber, G., "Influence of Traffic Inputs on Rigid Pavement Design Analysis Using Mechanistic-Empirical Pavement Design Guide," Paper 06-2621, Annual Meeting of the Transportation Research Board, January 2006. In a comparison of the MEPDG default axle load spectra with Kansas-specific axle load spectra, Khanum et al. found that the MEPDG rigid pavement performance prediction models did not seem particularly sensitive to differences in axle load spectra inputs. They found that the Kansas-specific axle distributions were different from the MEDPG default distributions for some functional classes. They also found that monthly adjustment factors for truck traffic are necessary to properly characterize the heavier truck traffic on Kansas highways in the winter and spring months.

Kweon, Y. J. and Cottrell, B. H., "Analysis of Weigh-in-Motion Data for Truck Weight Grouping in the Mechanistic-Empirical Pavement Design Guide," ***Transportation Research Record*** **2256, Transportation Research Board, National Research Council, 2011.** The purpose of this study was to evaluate the Virginia Department of Transportation's traffic data plan for implementation of the Mechanistic-Empirical Pavement Design Guide (MEPDG) with weigh-in-motion (WIM) data from 22 sites in Virginia for 2007 and 2008. The evaluation included an assessment of the WIM data for pavement design and for enforcement of overloaded trucks and the appropriateness of the truck weight groups. Grouping the sites on the basis of average equivalent single-axle loads was notably different from the current truck weight groups and grouping results based on traffic characteristics such as truck volume. Thus, further efforts to suggest a better grouping scheme are needed. For calculating monthly traffic factors, an input to the MEPDG, volume data from WIM sites could lead to biased results. Thus, vehicle classification count data are a better source than WIM data for the factors. The enforcement sites were found to carry heavier trucks in terms of average equivalent single-axle loads than the sites installed for pavement data collection. Thus, the concern that truck weights collected at the enforcement sites might be inappropriate for pavement design because of possible avoidance of the sites by overloaded trucks seems unwarranted. However, because of several factors and limitations, a definitive conclusion regarding this result could not be drawn.

Li, S., Jiang, Y., Zhu, K., and Nantung, T., "Truck Traffic Characteristics for Mechanistic-Empirical Flexible Pavement Design: Evidences, Sensitivities, and Implications," Paper 07-0101, Annual Meeting of the Transportation Research Board, January 2007. In an Indiana study reported by Li et al. summarized the relative sensitivity of the MEPDG's flexible pavement performance prediction models to various traffic input parameters as shown in the following table.

Sensitivity of MEPDG flexible pavement models to traffic inputs (Li et al. 2007).

Truck Traffic Characteristics	Pavement Distress			
	Roughness (IRI)	Rutting	Longitudinal Cracking	Alligator Cracking
Class Distribution	No	Fair	High	Medium
Monthly Distribution	No	Fair	Medium	Fair
Hourly Distribution	No	No	No	No
Axle Load Distribution	Medium ~ High	Medium ~ High	High	Fair ~ High
No. of Axles per Truck	No	No	No	No
Truck Count Accuracy	No	Fair	Medium	Fair
Operational Speed	No	Fair	Medium	Fair

Lu, Q. and Harvey, J., "Characterization of Truck Traffic in California for Mechanistic-Empirical Design," Paper 06-0389, Annual Meeting of the Transportation Research Board, January 2006. Lu and Harvey found in analysis of California truck traffic data that rural and urban areas had distinctly different truck traffic characteristics. Two default axle load spectra, based on the proportion of volumes of Class 5 and Class 9 vehicles (the two classes accounting for the majority of truck traffic) were identified for use with the MEPDG software. The spectrum with a higher ratio of Class 5 to Class 9 vehicles was judged more appropriate for use in analysis of urban (short-haul) sites, while the spectrum with a lower ration of Class 5 to Clas 9 vehicles was judged more appropriate for use in analysis of rural (long-haul) sites.

Ohio Department of Transportation, *Improved Characterization of Truck Traffic Volumes and Axle Loads for Mechanistic Empirical Pavement Design*, research in progress. The recently introduced Mechanistic-Empirical Pavement Design Guide (MEPDG) and related software provide capabilities for the analysis and performance prediction of different types of flexible and rigid pavements. In order to utilize MEPDG, project specific data must be entered into the application. In some instances, all of the required data may not be available. To advance the implementation of the MEPDG in Ohio, there is an urgent need for an automated tool to assemble traffic volume and axle load information from operational traffic monitoring systems within the state. This tool should be capable of accounting for missing information and summarizing traffic inputs in a format that can be directly imported into the MEPDG. The objectives of this project are to (1) develop a methodology to obtain the required MEPDG traffic inputs using available project-specific and regional traffic data and then (2) implement the developed methodology into user-friendly software.

Papagiannakis, A. T., Bracher, M., Li, J., and Jackson, N. C., "Sensitivity of NCHRP 1-37A Pavement Design to Traffic Input," Paper 06-0191, Annual Meeting of the Transportation Research Board, January 2006. In a study conducted for the Washington State DOT, Papagiannakis et al. analyzed the sensitivity of the MEPDG to the type of traffic data inputs used. The objective of this study was to establish the minimum traffic data collection requirements for predicting pavement life within an acceptable margin of error, for a given reliability level. The study was conducted using data from 30 LTPP sites (15 flexible pavement sites and 15 rigid pavement sites), selected to provide a wide range of pavement thicknesses and truck traffi c volumes, from among 178 sites identified in the LTPP database as having WIM data coverage exceeding 299 days per year. The following conclusions were reached: (1) Discontinuous traffic data collection involving site-specific WIM data is inferior to continuous site-specific AVC (automated vehicle classification) data. The reason for this is that partial WIM coverage does not yield site-specific monthly adjustment factors, which are necessary for accurately modelling seasonal damage with the MEPDG. (2) Pavement performance analysis conducted using continuous site-specific AVC is capable of predicting pavement life with errors lower than 10, 16, and 27 percent, for confidence levels of 75, 85, and 95 percent, respectively. (3) Where continuous site-specific traffic counts are combined with regional classification and load data, pavement life prediction errors may range from 25% to 64%, depending on the reliability level. (4) Where continuous site-specific traffic counts are combined with regional classification and national load data, pavement life prediction errors may range from 27% to 68%, depending on the reliability level. (5) Where continuous site-specific traffic counts are combined with national classification and load data, pavement life prediction errors may range from 30% to 76%, depending on the reliability level.

Prozzi, J. A. and Hong, F., "Seasonal Time Series Models for Supporting Traffic Input Data for Mechanistic-Empirical Design Guide," Paper 06-1450, Annual Meeting of the Transportation Research Board, January 2006. The level of effort required by the MEPDG to obtain detailed information on truck volumes by truck class, and the seasonal variation in those truck class volumes, is daunting. Prozzi and Hong proposed an interesting approach to this problem, using seasonal time series models. The growth trend term in a time series model predicts traffic as a function of time in years, while trigonometric functions are used to capture seasonal variations over the course of a year. WIM data collected on I-37 in Texas were used to develop and test a variety of time series models for truck traffic growth and seasonal variation for use with the MEPDG software. Among the key findings from the study were the following: (1) Two model alternatives, linear trend plus time series and compound trend plus time series, accurately captured volume growth and seasonal variation. Their seasonality-related parameter estimates were close to each other. For traffic predictions over 20 years, the linear trend model is recommended. (2) Growth factors varied among different vehicle classes. The seasonal variation characteristics among those truck classes were also different. Growth factor and seasonality of the aggregated truck classes was not equal to that of any individual truck class evaluated in this study. (3) Generally, two cycle lengths are sufficient to capture traffic volume seasonality. For simplicity, one cycle length of 12 months can be adopted in practice without significant loss of accuracy.

Ramachandran, A. N., Taylor, K. L., Stone, J. R., and Sajjadi, S. S., "NCDOT Quality Control Methods for Weigh-in-Motion Data," ***Public Works Management and Policy*****, Volume 16, Number 1, 2011.** The North Carolina Department of Transportation (NCDOT) collects weigh-in-motion (WIM) data using procedures and systems consistent with recommended industry practices. The NCDOT WIM systems are designed to estimate static vehicle axle weights based on dynamic traffic measurements. Regardless of the technology used, data errors and poor quality data are captured, which makes a quality control (QC) process an important part of all WIM data systems. This article describes the NCDOT WIM QC procedures. WIM data must undergo a series of sequential and well-defined QC procedures to ensure that the data meet the federal requirements and new standards for the Mechanistic Empirical Pavement Design Guide (MEPDG) process. After a literature review and consideration of prototype procedures, the authors concluded that the most efficient method of performing the WIM QC at NCDOT included structured query language (SQL) queries in a front-end database system applied to raw data stored in live back-end databases. The QC technique uses a combination of rule-based checks and manual audits of plots and reports. The NCDOT WIM QC process was applied to 45 WIM stations, which were checked for class and weight data anomalies. Findings show that the proposed process provided reliable data sets for use in developing the MEPDG traffic inputs for the NCDOT. Recommendations for future research are given.

Regehr, J. D., "Understanding and Anticipating Truck Fleet Mix Characteristics for Mechanistic-Empirical Pavement Design," Paper 11-0206, Annual Meeting of the Transportation Research Board, National Research Council, Washington, DC, 2011. This paper analyzes vehicle classification data to support the implementation of the Mechanistic-Empirical Pavement Design Guide (MEPDG). A cluster analysis and expert judgment are applied to vehicle classification data from Manitoba to produce six jurisdiction-specific truck traffic classification groups (TTCGs). These groups are used to estimate truck volumes by class at locations where no site-specific classification data exist. The unique vehicle classification distributions evident from these groups, particularly the relative predominance of six-axle tractor semitrailers and multiple-trailer trucks within the fleet, demonstrate the importance of developing truck traffic data inputs based on local conditions and expertise. Aspects of the analysis are specific to Manitoba; however, the general approach is transferable to other jurisdictions. Although this analysis provides a current understanding of truck fleet mix, there is a need to also understand the dynamic nature of fleet mix so that future changes may be anticipated. Based on a 40-year perspective of fleet mix changes in the Canadian Prairie Region, the impacts of truck size and weight regulations (among other influencing factors) on fleet mix are revealed. While this historical perspective is uniquely Canadian, the lessons learned provide insight into the potential fleet mix impacts that may be anticipated from plausible changes in U.S. truck size and weight policies—namely the introduction of a tridem axle group on a six-axle tractor semitrailer and the expansion of longer combination vehicle operations. This insight is relevant to a wide range of transportation contexts, including pavement design.

Romanoschi, S. A., Momin, S., Bethu, S., and Bendaña,"Development of Traffic Inputs for New Mechanistic-Empirical Pavement Design Guide: Case Study," *Transportation Research Record* 2256, Transportation Research Board, National Research Council, Washington, DC, 2011. Vehicle classification and axle load data are required for the structural design of new and rehabilitated flexible and rigid pavements with the new "Mechanistic–Empirical Pavement Design Guide" (MEPDG) developed under NCHRP Project 1-37A. The axle load spectra are determined from traffic data collected at weigh-in-motion (WIM) stations, and vehicle count and class data are recorded by vehicle classification stations. Some preliminary results are presented for an extensive traffic data-processing effort conducted to develop traffic inputs required by the MEPDG to design pavements in New York State. The data collected by classification and WIM sites from 2004 to 2009 were processed with the TrafLoad software developed in NCHRP Project 1-39. The discussion focuses on the variability of the major traffic input variables required by the MEPDG, as obtained from data collected in New York State, and on the differences between the data obtained from individual stations, state average values, and the default values recommended by the MEPDG, where applicable. The effect of variability of the major traffic input variables on the performance predicted by the MEPDG for a typical flexible pavement structure is also discussed.

Sayyady, F., Kim, Y. R., and Stone, J. R., "Damage-Based Analysis to Guide the Development of MEPDG Axle Load Distribution Factors and Clustering for North Carolina," Paper 11-3928, Annual Meeting of the Transportation Research Board, National Research Council, Washington, DC, 2011. This paper shows ways that the Mechanistic-Empirical Pavement Design Guide (MEPDG) can be utilized to develop damage factors to guide the development of MEPDG Axle Load Distribution Factor (ALDF) clusters. Pavement designers using the MEPDG must select the most appropriate ALDF clusters, in addition to other traffic data, for a particular project location before they can run any trial design. The MEPDG considers four axle types in ALDF input: single, tandem, tridem, and quad. A total of 140 axle type-load combinations are associated with these four axle types. In this paper, damage factors are developed for each of these combinations based on bottom-up fatigue damage as the failure criterion. Furthermore, bilinear functions are developed to enable partial factorial runs using the MEPDG to save time and effort. In this study, the MEPDG traffic input tables are adjusted to force the MEPDG to apply a certain axle type-load combination. Results show that the 82-kip tandem axle has the highest damage factor among the axles. Based on the damage-based analysis, tridem and quad axles, in addition to some heavy single and tandem axles, are excluded from the ALDF clustering process because these axle type-load combinations do not significantly affect pavement performance but can bias the results of ALDF clusters because of the infrequent occurrence of these axles.

Sayyady, F., Stone, J. R., and Kim, Y. R., "Effects of Sampled Weigh-in-Motion Data on Axle Load Distribution for Mechanistic-Empirical Pavement Design in North Carolina," Paper 11-2122, Annual Meeting of the Transportation Research Board, National Research Council, Washington, DC, 2011. Resource and budget constraints may restrict departments of transportation (DOTs) from collecting and reporting complete data. Also, technical and/or equipment problems may lead to incomplete or intermittent weigh-in-motion (WIM) data. To address these issues, DOTs have begun to investigate WIM data sampling procedures, focusing on axle load distribution factors (ALDFs) and the effectiveness of the sampling procedures for estimating the ALDF accurately. This paper proposes sampling schemes using two dimensions: frequency (annual, semiannual, quarterly, and monthly) and duration (two consecutive weekdays and five consecutive weekdays). The effectiveness of the sampling schemes is evaluated using the sum of the relative error (SRE) in estimating the ALDF derived from sampled WIM data compared to ALDF derived from annual WIM data. In addition, the paper investigates the relationship between data sampling and seasonal variations for traffic data in circumstances where annual WIM data are not available. The three regions in North Carolina, which have different climatic characteristics, are studied for this purpose. Findings show that a direct correlation between seasonal variations and the accuracy of the sampling schemes exists for truck traffic. When truck traffic is fairly stable, the annual sampling schemes for two or five consecutive weekdays generate encouraging results. In locations with high seasonal variability, semiannual and quarterly sampling schemes are required to capture the seasonal variations in terms of the axle load distribution. The results of this paper will provide guidance to DOTs in situations where annual WIM data that are incomplete or intermittent and must be sampled in order to produce ALDF inputs for the Mechanistic-Empirical Pavement Design Guide (MEPDG).

Sayyady, F., Stone, J. R., List, G. F., Jadoun, F. M., and Kim, Y. R., "Axle Load Distribution for Mechanistic-Empirical Pavement Design in North Carolina: Multidimensional Clustering Approach and Decision Tree Development," *Transportation Research Record* 2256, Transportation Research Board, National Research Council, Washington, DC, 2011. A multidimensional clustering approach to generate regional average truck axle load distribution factors (ALDFs) for North Carolina is presented. The results support the "Mechanistic–Empirical Pavement Design Guide" (MEPDG). Weigh-in-motion data collected on North Carolina roadways are used in the analysis. A multidimensional clustering analysis based on ALDF data develops representative clusters for different highway functional classifications. Findings show that ALDF clusters have distinct characteristics for primary roads, secondary roads, collectors, and local roads. An easy-to-use decision tree based on available traffic parameters and local knowledge helps the pavement designer select the proper ALDF input. Specific contributions include a multidimensional clustering analysis that is guided by MEPDG damage-based analysis, well-defined ALDF clusters that represent specific traffic patterns in North Carolina, and a decision tree characterized by its simplicity to help pavement designers select ALDF inputs.

Tran, N. H. and Hall, K. D., "Evaluation of Weigh-In-Motion Data for Developing Axle Load Distribution Factors for Mechanistic Empirical Pavement Design Guide," Paper 07-0698, Annual Meeting of the Transportation Research Board, January 2007. Tran and Hall used WIM data from 25 sites in Arkansas to assess the adequacy of the WIM data for development of statewide axle load spectra for use with the MEPDG. They found that traffic data from WIM sites, especially those using temperature-dependent piezoelectric sensors, often have errors, and that pavement performance predictions obtained from the MEPDG software were sensitive to these errors. The sensitivity of predicted fatigue cracking to overestimated WIM data was the most pronounced of the prediction errors identified. The authors recommended frequent equipment calibration and application of the WIM data quality control checks recommended by LTPP to minimize the contribution of WIM data error to MEPDG performance prediction error.

Wang, C. P. K., Li, Q., Hall, K. D., and Nguyen, V., "Traffic Data Support for the Mechanistic-Empirical Pavement Design Guide (MEPDG), Paper 09-3755, Annual Meeting of the Transportation Research Board, January 2009. In a study conducted for the Arkansas Highway and Transportation Department, Wang et al. described the development of computer software to take the voluminous raw WIM data collected in Arkansas, apply quality control checks to the data, and generate traffic data files in a format that can be used directly with the MEPDG software.

Zaghloul, S., El Halim, A. A., Ayed, A., Vitillo, N. P.., and Sauber, R. W., "Sensitivity Analysis of Input Traffic Levels on Mechanistic-Empirical Design Guide Predictions," Paper 06-0937, Annual Meeting of the Transportation Research Board, January 2006. In a study conducted using data from three SPS sites in New Jersey, Zaghloul et al. found that the impact of using Level 1 traffic inputs versus Level 3 traffic inputs can be very significant. For the sites analyzed, and for some performance indicators, such as rutting, the predicted damage was significantly increased, and sometimes even unrealistic high, when Level 1 traffic inputs were used. For some other performance indicators, however, such as top-down cracking, the differences in predicted damage using Level 1 versus Level 3 traffic were slight. The authors offered the following additional observations and recommendations: (1) Mixing Level 1 and Level 3 traffic for the same run may result in misleading results, and should be done with caution. (2) More investigation of the stability of the performance models should be conducted so that unrealistic results can be avoided. (3) The performance models should be calibrated for each material type (binder) and layer thickness combination. (4) Performance indices that were successfully calibrated using Level 3 traffic showed reasonable results for Level 1 traffic, but better predictions may be achieved if the calibration is done with Level 1 traffic.

Climate

Dzotepe, G. A. and Ksaibati, K., ***The Effect of Environmental Factors on the Implementation of the Mechanistic-Empirical Pavement Design Guide (MEPDG)*****, Report No. 10-225B, Mountain-Plains Consortium, North Dakota State University, 2011.** This study summarizes the challenges that are likely to impede implementation of the MEPDG within the Northwest Region and how these can be overcome. The study also investigates the effects of climate variables on the predicted pavement performance indicators and, in addition, evaluates the adequacy of using interpolated climate data on pavement performance in the state of Wyoming.

Heitzman, M., Timm, D., Tackle, E. S., Herzmann, D. E., and Traux, D. D., ***Developing MEPDG Climate Data Input Files for Mississippi*****, Report Number FHWA/MS-DOT-RD-11-232, Missouri Department of Transportation, Jackson, MS, 2011.** Prior to this effort, Mississippi's Mechanistic-Empirical Pavement Design Guide (MEPDG) climate files were limited to 12 weather stations in only 10 countries, and only seven weather stations had over 8 years (100 months) of data. Hence, building MEPDG climate input datasets improves modeling accuracy and geographic coverage. The new historic climate files created by this project use hourly data from 23 automated surface observation system (ASOS) and automated weather observation system (AWOS) stations and daily data from over 100 Cooperative Observer Program (COOP) sources combined to generate a more accurate 40-year historic climate input data file for the 82 counties in Mississippi. This represents over 30 times more climate input data for MEPDG analyses in the state. The study then built viral (future) climate files by applying global and regional climate models to the 40-year historic data. These virtual files were limited to nine climate zones across Mississippi due to the nature of long-range climate prediction. The temperature and precipitation data were adjusted in the virtual files, and the 82 historic and nine virtual climate data files were checked for logical errors and using the MEPDG program as part of the development process. The sensitivity analysis examined how the improved climate data input files (MEPDG, historic, and virtual) on three common types of pavements (jointed PCC, thick hot mix asphalt (HMA), and thin HMA) used in Mississippi. The analysis showed that repeating the limited data in the MEPDG climate input files to predict pavement distress over a typical 20 to 40 year analysis period resulted in significantly higher predicted distress in some cases. The sensitivity study determined that the resources used to built the improved climate files were an appropriate effort with a measurable long-term benefit.

Johanneck, L., Tompkins, D., Clyne, T., and Khazanovich, L., "Minnesota Road Research Data for Evaluation and Local Calibration of the Mechanistic-Empirical Pavement Design Guide's Enhanced Integrated Climatic Model," ***Transportation Research Record*** **2226, Transportation Research Board, National Research Council, Washington, DC, 2011.** This paper describes research to evaluate modeling of the thermal behavior of concrete and composite pavements by the Enhanced Integrated Climatic Model (EICM), the climate-modeling package used in the "Mechanistic–Empirical Pavement Design Guide" (MEPDG). First, the study uses temperature data collected at the Minnesota Road Research Project (MnROAD) facility from portland cement concrete (PCC) and asphalt concrete (AC)–PCC pavements to investigate benefits of AC overlays on the thermal characteristics of PCC slabs. Furthermore, the study validates EICM predictions of thermal gradients through the slabs and investigates the effect of MEPDG-user inputs for thermal conductivity of PCC. Overall, the paper examines measured data from MnROAD for AC-PCC pavements and their single-layer PCC counterparts and attempts to explain how similar pavement systems and their thermal characteristics are taken into account in the MEPDG. The paper concludes that evaluation of the material thermal inputs should be part of a process of local calibration and adaptation of the MEPDG.

Li, Q., Mills, L., and McNeil, S., ***The Implications of Climate Change on Pavement Performance and Design*****, Final Report, Delaware Center for Transportation, Newark, DE, 2011**. Pavements are designed based on historic climatic patterns, reflecting local climate and incorporating assumptions about a reasonable range of temperatures and precipitation levels. Given anticipated climate changes and the inherent uncertainty associated with such changes, a pavement could be subjected to very different climatic conditions over the design life and might be inadequate to withstand future climate forces that impose stresses beyond environmental factors currently considered in the design process. This research explores the impacts of potential climate change and its uncertainty on pavement performance and therefore pavement design. Two tools are integrated to simulate pavement conditions over a variety of scenarios. The first tool, MAGICC/SCENGEN (Model for the Assessment of Greenhouse-Gas-Induced Climate Change: A Regional Climate Scenario Generator), provides estimates of the magnitude of potential climate change and its uncertainty. The second tool, the Mechanistic-Empirical Pavement Design Guide (MEPDG) software analyzes the deterioration of pavement performance. Three important questions are addressed: (1) How does pavement performance deteriorate differently with climate change and its uncertainty? (2) What is the risk if climate change and its uncertainty are not considered in pavement design? and (3) How do pavement designers respond and incorporate this change into pavement design process? This research develops a framework to incorporate climate change effects into the mechanistic-empirical based pavement design. Three test sites in the North Eastern United States are studied and the framework is applied. It demonstrates that the framework is a robust and effective way to integrate climate change into pavement design as an adaptation strategy.

Oklahoma Department of Transportation, *Evaluation of Enhanced Integrated Climatic Model for Oklahoma Pavements*, research in progress. The Enhanced Integrated Climatic Model (EICM) is an integral component of the Mechanistic Empirical Pavement Design Guide (MEPDG) that involves analysis of water and heat flow through pavement layers in response to climatic, soil, and boundary conditions above and below the ground surface in pavement structures. The performance of a pavement depends on many factors such as the structural integrity, the material properties, traffic loading, construction method, and climatic conditions. Since unbound materials (subgrade soils and base course) are a significant portion of the construction of pavements, much of the distress can be traced to problems in these materials. The goal of the MEPDG is to provide a quantitative and site-specific assessment of the pavement section needed to resist the traffic loading during the design lifetime. The EICM plays a significant role in defining the material properties in the design guide. In this regard, the moisture and temperature variations are the paramount parameters that determine the behavior of pavement structures. For instance, the water content and temperature are closely associated with the mechanical properties of the subgrade soils which govern deformation response of the pavement under traffic loading. AASHTO has recommended that the MEPDG be adopted by state departments of transportation in pavement design. However, the Oklahoma Department of Transportation (ODOT) has noticed that the EICM model in the MEPDG does not contain sufficient and site-specific climatic data for realistic predictions of moisture and temperature changes in pavement layers in Oklahoma. Several states in the U.S. have conducted independent studies to validate the EICM, and assess the effects of water content change on the short- and long-term pavement performance. Some of these states are Minnesota, Idaho, New Jersey, Ohio, and Arkansas. All these states have encountered difficulties in matching the predictions made by the EICM for moisture content with field observations. The proposed research study is specifically focused on a detailed evaluation of the EICM for Oklahoma in order to reduce the sources of uncertainty in the MEPDG design. The EICM plays an important role in defining the short and long-term properties of pavement materials used in the MEPDG. The proposed study will help determine the appropriateness of the EICM for the Oklahoma climatic conditions. This study will lead to the estimation of site specific variation in environmental factors that are used in predicting seasonal and long-term variations of moduli of unbound materials. This project will advance the strategic plan of the Oklahoma Transportation Center and greatly benefit one of its key stakeholders, ODOT.

Saha, J. and Bayat, A., "Evaluation of Canadian Climate Information and Its Effect on Pavement Performance Through MEPDG Prediction," Paper 11-2081, Annual Meeting of the Transportation Research Board, National Research Council, Washington, DC, 2011. This paper aims to investigate the quality of recently developed Canadian climate data and the effects of climate on flexible pavement performance using the Mechanistic-Empirical Pavement Design Guide (MEPDG). Two hundred and six (206) weather stations were categorized into six weather zones to better understand climate distribution. The analysis was carried out for all weather stations across Canada, and sensitivities of pavement performances in terms of the International Roughness Index (IRI), Asphalt Concrete (AC) rutting and total pavement deformation were studied. Pavement performances of weather stations in close proximity were studied to investigate the consistency of Canadian climate data. Also, pavement performances for Virtual Weather Station (VWS) data and actual station data were compared.

University of Maryland and MACTEC Engineering, *Evaluation of LTPP Climatic Data for Use in MEPDG Calibration and Other Pavement Analysis*, research in progress. Climate is a major factor influencing the performance of pavements and pavement materials. The Long-Term Pavement Performance (LTPP) studies have performed pioneering work to characterize and summarize site-specific climatic data for its General Pavement Studies (GPS) and Specific Pavement Studies (SPS) test sections. However, improvements in climatic data collection are needed to support current and future research on climate effects on pavement materials, design, and performance. The calibration and enhancement of the new Mechanistic-Empirical Pavement Design Guide is just one example of these emerging needs. To address these needs, this study is designed around the following objectives: (1) Examine 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. (2) Develop a methodology for characterizing location-specific historic climate indices that includes temporal changes in the position and measurement characteristics of the operating weather stations (OWS) used for the computation. This new methodology will include an estimate of the variability or uncertainty caused by the spatial averaging process used to develop the baseline indices. (3) Apply this new methodology to update the climate statistics in the LTPP database. (4) Examine the need to add a climate-soils parameter such as the Thornthwaite Moisture Index (TMI) to the LTPP database. Examine the applicability of TMI to other transportation infrastructure applications. (5) Examine the need for continued location-specific solar radiation measurements to capture the effect of climate change on pavement and other infrastructure performance. Determine if other existing data sources can be used to fulfill this need.

Materials

Apeagyei, A. K., and Diefenderfer, S. D., *Asphalt Material Design Inputs for Use with the Mechanistic-Empirical Pavement Design Guide in Virginia*, Report No. FHWA/VCTIR 12-R6, Virginia Center for Transportation and Innovation Research, Charlottesville, VA, 2011. The Guide for the Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures (MEPDG), developed under NCHRP Project 1-37A and recently adopted by the American Association of State Highway and Transportation Officials (AASHTO) offers an improved methodology for pavement design and evaluation. To achieve this improved prediction capability, the MEPDG procedure requires fundamental material properties in addition to certain empirically determined binder and mixture properties as design inputs. One of the key tasks identified by the Virginia Department of Transportation's (VDOT) Asphalt Concrete MEPDG Committee was the laboratory characterization of asphalt mixtures commonly used in Virginia to generate a catalog of MEPDG-required design inputs. The purpose of this study was to evaluate, compile, and present asphalt material properties in a format that could be readily used in the MEPDG software and to develop a comprehensive catalog of MEPDG design input parameters for pavement design in Virginia. To achieve this objective, eighteen asphalt concrete mixtures, sampled from seven of the nine VDOT districts, were tested using a battery of MEPDG-required tests, including dynamic modulus (|E*|), flow number (FN), creep compliance, tensile strength, and beam fatigue tests. Testing involving binder and volumetric properties of the mixtures was also conducted. Finally, rut tests using the asphalt pavement analyzer (APA), a standard VDOT test protocol, were conducted to allow a direct comparison of the APA and FN test results. On the basis of these tests, suggestions for additional studies were made. The results of the study were presented in a form matching the MEPDG input format, and a catalog of design input parameters was developed for the 18 asphalt concrete mixtures. Included in the catalog were binder stiffness, mixture |E*|, mixture gradation, and mixture volumetric properties that would enable a designer the flexibility to select the desired input level (1, 2, or 3) depending on the pavement type. An illustrative example of how the developed inputs could be implemented using the MEPDG software was also provided. The results showed that |E*| master curves of asphalt mixtures obtained using the five standard testing temperatures described in AASHTO TP 62 could be obtained by testing at only three temperatures, which could result in a substantial reduction of testing time. The results also showed that the FN test was a sensitive test for evaluating rutting susceptibility of asphalt mixtures in the laboratory. The FN test was found to be sensitive to binder stiffness, mixture stiffness, mixture volumetric properties, aggregate gradation, and amount of recycled asphalt pavement (RAP) for the mixtures considered in this study. The study recommends that the catalog of input data for typical asphalt mixtures developed in this study be considered for pavement design in Virginia. The data followed expected trends and compared quite well with those reported in previous studies. Further studies should be conducted to evaluate the FN test as an additional tool for evaluating rutting in asphalt mixtures. Mixtures containing higher amounts of RAP (greater than 20%) exhibited comparatively lower rutting resistance than those with 20% RAP or less. This phenomenon was unexpected, since it is generally believed that adding more RAP should result in stiffer and hence more rut-resistant mixtures. Additional research should be conducted to investigate this phenomenon further.

Attia, M. and Abdelrahman, M., "Effect of State of Stress on the Resilient Modulus of Base Layer Containing Reclaimed Asphalt Pavement," ***Road Materials and Pavement Design*****, Volume 12, Number 1, 2011.** The use of reclaimed asphalt pavement (RAP) as a base layer is a sustainable rehabilitation method and reduces local agency cost. Proper characterizations of the stress-dependent behavior of pavement layers have significant impact on the accuracy of pavement response predictions. This research examines which constitutive model is the most appropriate for predicting the resilient behavior of RAP as a base layer. The resilient modulus (MR) was examined in the laboratory for specimens containing different ratios of RAP and aggregate. The MR of RAP/aggregate blends were higher, less sensitive to bulk stress, and more sensitive to confining pressure compared to base aggregate. The MEPDG model that presented the nonlinear resilient behavior of unbound layers fit the RAP material and was mathematically stable.

Awed, A., El-Badawy, S. M., Bayomy, F. M., and Santi, M., "Influence of MEPDG Binder Characterization Input Level on Predicted Dynamic Modulus for Idaho Asphalt Concrete Mixtures," Paper 11-1268, Annual Meeting of the Transportation Research Board, National Research Council, Washington, DC, 2011. The current version of the Mechanistic-Empirical Pavement Design Guide (MEPDG) includes two different models for the hot-mix asphalt (HMA) dynamic modulus (E*) prediction. In MEPDG levels 2 and 3 hot mix asphalt (HMA) stiffness characterization; users can choose either one of these two models. The first model is the NCHRP1-37A viscosity-based model while the second one is the NCHRP1-40D binder shear modulus-based model. The main difference between the two models is the binder stiffness characterization parameter. Moreover, MEPDG allows users to choose between three hierarchical input levels for binder stiffness characterization. This paper focuses on the evaluation of the influence of the binder characterization input level on the prediction accuracy of the MEPDG E* predictive models. In this study, a total of 22 HMA mixtures commonly used in Idaho with six binder grades were investigated. Binder characterization data were obtained from Brookfield viscosity testing for MEPDG level 1 analysis for conventional binders, Dynamic Shear Rheometer (DSR) testing for MEPDG level 1 analysis for Superpave binders, and typical MEPDG values (MEPDG level 3 analysis). Comparison between laboratory measured and predicted E* values along with the three investigated binder characterization cases, revealed that both MEPDG E* predictive models resulted in reasonable E* predictions. The 1-37A E* model along with MEPDG level 3 binder inputs yielded the most accurate and least biased E* estimates for Idaho mixtures. Nevertheless, the lowest prediction accuracy and highest bias in the E* estimates, especially at the higher temperatures, resulted from the 1-40D model with MEPDG level 3 binder characterization.

Ayithi, A., Hiltunen, D. R., and Roque, R., *Base Course Resilient Modulus for the Mechanistic-Empirical Pavement Design Guide*, Final Report, Florida Department of Transportation, Tallahassee, FL, 2011. The Mechanistic-Empirical Pavement Design Guide (MEPDG) recommends use of modulus in lieu of structural number for base layer thickness design. Modulus is nonlinear with respect to effective confinement stress, loading strain, and moisture. For design purposes, a single effective modulus of a base layer is desirable, and this modulus should be able to approximately account for nonlinearities. However, the MEPDG does not describe a procedure for determining this single modulus value. This research focused on laboratory characterization of base modulus nonlinearity, developing a nonlinear response model using laboratory data for nonlinear pavement analysis, and a methodology to determine a single effective modulus for a base layer via the nonlinear response model. Resonant column tests were conducted on two base materials used in Florida to characterize shear modulus (G) nonlinearity under different confinements and moisture contents. The suction effect increases G in the strain range of 10 to the negative 5th power to 10 to the negative 1st power, with very significant increases at strain levels below 10 to the negative 3rd power. Using laboratory nonlinear modulus data, a nonlinear response model was developed via the Plaxis finite element methodology. The model is an effective means for assessing the effects of unbound material nonlinearity on the response of pavements. A representative modulus can be determined by a backcalculation procedure in which surface deflections from a nonlinear analysis are matched via an equivalent linear analysis. The single effective modulus varies over a range of conditions, including the moisture content of the base, pavement layer thicknesses, and the modulus of the subgrade. There is a significant effect of moisture on the effective modulus of limerock base materials used in Florida and the modulus/moisture relationship employed in the MEPDG underpredicts this increase. An equivalent linear analysis using effective moduli for both an unbound base and the subgrade can predict the structural response of an asphalt surface layer in a flexible pavement. It should be possible to utilize these structural response predictions in the assessment of cracking performance of the surface layer. However, caution is warranted in predicting the structural response of the unbound base and subgrade layers using an equivalent linear analysis. Use of an effective modulus for a nonlinear base layer appears reasonable for very thick pavement structures, but appears to underpredict vertical strain at the top of subgrade as the nonlinearity increases. Use of effective moduli for both a nonlinear base and subgrade appears to underpredict top of subgrade vertical strain even for very thick pavements.

Bonaquist, R., ***Characterization of Wisconsin Mixture Low-Temperature Properties for the AASHTO Mechanistic-Empirical Pavement Design Guide*****, Special Report 0092-10-07, Wisconsin Department of Transportation, Madison, WI, 2011.** This research evaluated the low-temperature creep compliance and tensile strength properties of Wisconsin mixtures. Creep compliance and tensile strength data were collected for 16 Wisconsin mixtures representing commonly used aggregate sources and binder grades. Engineering and statistical analyses were performed on the data to provide recommendations for using measured mechanical properties in thermal cracking analyses with the Mechanistic-Empirical Pavement Design Guide (MEPDG), and to evaluate the thermal fracture resistance of Wisconsin mixtures.

Breakah, T. M., Bausano, J. P., Willliams, R. C., and Vitton, S., "The Impact of Fine Aggregate Characteristics on Asphalt Concrete Pavement Design Life," ***International Journal of Pavement Engineering*****, Volume 12, Number 2, 2011.** The development of the Mechanistic-Empirical Pavement Design Guide (MEPDG) provides an opportunity to simulate the performance of pavements. This paper considers the impact of fine aggregate on the predicted performance of pavements by simulating the performance differences between pavement mixes prepared with different sources of fine aggregate with different gradations using the MEPDG. A natural and four manufactured sands from parent material consisting of dolomite, limestone, traprock (TR) and a glacial gravel (GG), and five gradations were utilised in this study. This resulted in 19 different sand/stone combinations being tested for dynamic modulus to enable level 1 analysis in the MEPDG. The results indicate that the fine aggregate angularity (FAA) test adequately ranks aggregates from the same source, but does not appropriately rank aggregates from different sources. TR and GG were identified as the best performers within the investigated aggregate sources and that the FAA, aggregate source and gradation are not significant in determining mixture performance.

Cross, S. A., Gibbe, R., and Aryal, N., ***Development of a Flexible Pavement Database for Local Calibration of the MEPDG, Part 2–Evaluation of ODOT SMA Mixtures*****, Report No. FHWA-OK-11-06(2), Oklahoma Department of Transportaion, Oklahoma City, OK, 2011.** There has been some reluctance on the part of some in Oklahoma to use SMA mixtures. There are several factors that could be involved in the slow acceptance of SMA mixtures in Oklahoma.

These factors are 1) the extra expense associated with the higher binder contents and better quality aggregates required, 2) a lack of data indicating that SMA mixtures perform substantially better than conventional Superpave mixtures and 3) a lack guidance on thickness design benefits, including appropriate input parameters for the MEPDG. The objectives this study are to evaluate the performance of SMA mixes compared to S-4 mixes and to determine the performance benefits. Testing included Hamburg Rut Tests and dynamic modulus testing. Hamburg rut testing indicated that SMA resists permanent deformation better than ODOT S-4 mixes made with the same source and grade of asphalt cement. Both measured and predicted dynamic modulus of SMA was less than ODOT S-4 mixes. The Asphalt Institutes fatigue equation indicated longer fatigue life for SMA compared to S-4 mixes. MEPDG prediction models contradict these findings.

Dave, E. V., Leon, S., and Park, K., "Thermal Cracking Prediction Model and Software for Asphalt Pavements," First T&DI Congress 2011: Integrated Transportation and Development for a Better Tomorrow, American Society of Civil Engineers, Chicago, IL, 2011. Thermally induced cracking in asphalt pavements remains to be one of the prominent distress mechanisms in regions with cooler climates. At present, the AASHTO Mechanistic-Empirical Pavement Design Guide (MEPDG) is the most widely deployed pavement analysis and design procedure. For thermal cracking predictions, MEPDG utilizes a simplified one-dimensional stress evaluation model with a simple Paris law (i.e., linear elastic fracture mechanics)-based crack propagation procedure. The user-friendly graphical interface for MEPDG makes it an attractive design procedure of choice, however, the over simplicity of the model and lack of a physics-based representation to accurately capture the nonlinear fracture behavior of rate-dependent asphalt concrete reduce(s) the reliability of predictions. This study presents an interactive thermal cracking prediction model that utilizes a nonlinear finite element based thermal cracking analysis engine which can be easily employed using a user-friendly graphical interface. The analysis engine is comprised of (1) the cohesive zone fracture model for accurate simulation of crack initiation and propagation due to thermal loading and (2) the viscoelastic material model for time and temperature dependent bulk material behavior. The graphical user interface (GUI) is designed to be highly interactive and user-friendly in nature, and features screen layouts similar to those used in the AASHTO MEPDG, thus minimizing transition time for the user. This paper describes the individual components of the low temperature cracking prediction software (called LTC Model) including details on the graphical user interface, viscoelastic finite element analysis, cohesive zone fracture model, and integration of various software components for thermal cracking predictions.

Diefenderfer, S. D., *Analysis of the Mechanistic-Empirical Pavement Design Guide Performance Predictions: Influence of Asphalt Material Input Properties*, Report No. FHWA/VTRC 11-R3, Virginia Transportation Research Council, 2010. The Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures (MEPDG) is an improved methodology for pavement design and the evaluation of paving materials. The Virginia Department of Transportation (VDOT) is expecting to transition to using the MEPDG methodology in the near future. The purpose of this research was to support this implementation effort. A catalog of mixture properties from 11 asphalt mixtures (3 surface mixtures, 4 intermediate mixtures, and 4 base mixtures) was compiled along with the associated asphalt binder properties to provide input values. The predicted fatigue and rutting distresses were used to evaluate the sensitivity of the MEPDG software to differences in the mixture properties and to assess the future needs for implementation of the MEPDG. Two pavement sections were modeled: one on a primary roadway and one on an interstate roadway. The MEPDG was used with the default calibration factors. Pavement distress data were compiled for the interstate and primary route corresponding to the modeled sections and were compared to the MEPDG-predicted distresses. Predicted distress quantities for fatigue cracking and rutting were compared to the calculated distress model predictive errors to determine if there were significant differences between material property input levels. There were differences between all rutting and fatigue predictions using Level 1, 2, and 3 asphalt material inputs, although not statistically significant. Various combinations of Level 3 inputs showed expected trends in rutting predictions when increased binder grades were used, but the differences were not statistically significant when the calibration model error was considered. Pavement condition data indicated that fatigue distress predictions were approximately comparable to the pavement condition data for the interstate pavement structure, but fatigue was over-predicted for the primary route structure. Fatigue model predictive errors were greater than the distress predictions for all predictions. Based on the findings of this study, further refinement or calibration of the predictive models is necessary before the benefits associated with their use can be realized. A local calibration process should be performed to provide calibration and verification of the predictive models so that they may accurately predict the conditions of Virginia roadways. Until then, implementation using Level 3 inputs is recommended. If the models are modified, additional evaluation will be necessary to determine if the other recommendations of this study are impacted. Further studies should be performed using Level 1 and Level 2 input properties of additional asphalt mixtures to validate the trends seen in the Level 3 input predictions and isolate the effects of binder grade changes on the predicted distresses. Further, additional asphalt mixture and binder properties should be collected to populate fully a catalog for VDOT's future implementation use. The implementation of these recommendations and use of the MEPDG are expected to provide VDOT with a more efficient and effective means for pavement design and analysis. The use of optimal pavement designs will provide economic benefits in terms of initial construction and lifetime maintenance costs.

Diefenderfer, S. D., "Evaluation of the MEPDG Using Asphalt Material Inputs Obtained From Plant Mix," ***Journal of the Association of Asphalt Paving Technologists*****, Volume 80, 2011.** Decisions must be made regarding the hierarchical level of desired asphalt material inputs and the extent to which asphalt mixture characterization must be performed, because the Virginia Department of Transportation (VDOT) is expecting to transition to using the methodology described in the Guide for the MechanisticEmpirical Design of New and Rehabilitated Pavement Structures (MEPDG) in the near future. A study was conducted to examine the effects of these variables utilizing input values measured from 11 typical plant mixtures (i.e., three surface, four intermediate, and four base mixtures). The predicted fatigue and rutting distresses were used to evaluate the response of the MEPDG to differences in the mixture properties and to evaluate the future needs for implementation. Two example pavement sections were modeled: a primary and an interstate roadway section. Pavement distress data were compiled for an interstate and primary route corresponding to the modeled sections and were compared to the MEPDG-predicted distresses. Predicted distress quantities for fatigue cracking and rutting were compared to the calculated distress model predictive errors to determine if the material property input levels were statistically different. There were differences between all rutting and fatigue predictions using level 1, 2, and 3 asphalt material inputs, but they were not statistically significant. Various combinations of level 3 inputs showed expected trends in rutting predictions when increased binder grades were used, but the differences were not statistically significant when the calibration model error was considered. Fatigue distress predictions were approximately comparable to the pavement distress data, but the model predictive errors were greater than the distress predictions. Although this was a limited study, it suggested several steps VDOT should take before implementing the MEPDG. Additional work to identify the source of predictive insensitivity to changes in both asphalt mixture properties and the level of input values is necessary. The comparisons of predicted rutting and fatigue distresses to pavement distress data for the interstate and primary pavement indicate the necessity of considering local verification and calibration of the predictive models.

Dongre, R., "Utah's Efforts to Implement the Mechanistic-Empirical Pavement Design Guide: Asphalt Binder Uniformity Study," ***Engineering Circular*** **155, Transportation Research Board, National Research Council, Washington, DC, 2011.** In this study, the effect on pavement performance of day-to-day production uniformity of asphalt binder supply during construction was determined. The latest available version (0.900, August 2006) of the newly developed NCHRP project 1-37A, AASHTO's Mechanistic– Empirical Pavement Design Guide (MEPDG), was used for this purpose, and results are described in this report. Utah Department of Transportation (DOT) engineers wanted to limit the amount of performance grade (PG) variation of the asphalt binder supply during construction if the results showed significant effects on predicted pavement performance. Two existing pavement structures (weak and strong) were selected by Utah DOT for this study. Original asphalt binder grades for each structure were recreated along with additional formulations that simulated variation in grades. Two suppliers were asked to formulate six PG binders each, (three each for strong and weak structures) giving a total of 12 asphalt binders. Aggregates from the same quarry as the original aggregates were collected and hot-mix samples were compacted in the gyratory compactor using the appropriate mix designs. From these compacted samples, smaller simple performance test (SPT) specimens were cored and tested to obtain dynamic modulus (E*) values for MEPDG analysis. Binder properties required for MEPDG were also determined in the laboratory. Traffic and climate data was obtained from Utah DOT. A total of 366 different designs were analyzed to complete the MEPDG portion of this study. All levels of MEPDG (Level 1, Level 2, and Level 3) were used in the analysis. Analysis showed that PG uniformity of the asphalt binder supply does not show a significant sensitivity to predicted performance of regular or value engineered pavements. This finding is based on evaluation of all distresses predicted by MEPDG, such as, but not limited to, rutting, fatigue and thermal cracking. Consequently there was no justification found to develop limits on uniformity of PG of the asphalt binder supply. New hot-mix asphalt (HMA) mix-design requirement cannot be justified for the within-PG variation of asphalt binder supply observed at Utah DOT in the past 4 years.

El-Badawy, S. M., Awed, A., and Bayomy, F. M., "Evaluation of the MEPDG Dynamic Modulus Prediction Models for Asphalt Concrete Mixtures," First T&DI Congress 2011: Integrated Transportation and Development for a Better Tomorrow, American Society of Civil Engineers, Chicago, IL, 2011. HMA dynamic modulus (E*) is one of key inputs to the Mechanistic-Empirical Pavement Design Guide (MEPDG). There are two different E* models in the MEPDG; the NCHRP 1-37A viscosity-based model, and the NCHRP 1-40D, which is based on the binder shear modulus. This paper focuses on evaluating the influence of the binder characterization input level on the predicted E* in MEPDG. Laboratory E* tests were conducted on samples from 15 different HMA plant-produced mixtures. The shear modulus (G*) and phase angle (&dgr;) for each binder were also determined in the laboratory. Results showed that MEPDG levels 1and 3 binder characterization inputs with both E* predictive models yielded E* values that are in excellent to fair agreement with laboratory measured E* . However, the 1-37A model showed better results than the 1-40D model. On the other hand, high bias in E* values was observed when level 1 binder characterization data was used.

Georgia Department of Transportation, *Determination of Coefficient of Thermal Expansion for Portland Cement Concrete for MEPDG Implementation*, research in progress. The objectives of this project are (1) to develop the framework for a statewide database for coefficient of thermal expansion (CTE) input values, and (2) to develop a decision tree to aid Georgia Department of Transportation (GDOT) designers in selecting appropriate CTE inputs.

Giuliana, G., Nicolosi, V., and Festa, B., "Predictive Formulas of Complex Modulus for High Air Void Content Mixes," Paper 12-0878, Annual Meeting of the Transportation Research Board, National Research Council, Washington DC, 2012. Over the last 10 years, there have been a number of trials and developments of porous asphalt (according to the definition of European Standard EN 13108-7) or open-graded friction course (according to the definition used in USA), which has led to the use of high-air-void-content mixes with better acoustic performance. Mixes with air void contents greater than 20% are used at present in Europe and their use is spreading in other countries (USA, New Zealand, etc.). The dynamic modulus (|E*|) is one of the most important performance parameters for characterizing bituminous mixes, and it is used as an input in mechanistic and mechanistic-empirical design methods (MEPDG). Since the measurement of stiffness in the laboratory is not straightforward, a commonly used approach is to estimate dynamic modulus using predictive models. Several models are available in the literature for estimating |E*|, but they were developed and calibrated using hot mixes with air void contents less than 15%. This paper describes research carried out to evaluate whether some predictive dynamic modulus equations work well for porous asphalt with high air void contents. Two predictive models were analyzed: the Witczak-Andrei model, and the Witczak-Bari model. The experimental values of |E*| in some high-air-void-content mixes are compared with values obtained using previously mentioned predictive formulas. The results showed that the proposed formulas underpredict (Witczak-Andrei) or overpredict (Witczak-Bari) the |E*|, but should work well if they are recalibrated. Based on experimental results, a methodology of calibration of the predictive models was proposed for use with high-air-void-content mixes.

Grebenshikov, S. and Prozzi, J. A., "Enhancing Mechanistic-Empirical Pavement Design Guide Rutting Performance Predictions with Hamburg Wheel-Tracking Results," *Transportation Research Record* 2226, Transportation Research Board, National Research Council, Washington, DC, 2011. To analyze the effects of mix production variability on pavement rutting, this paper compares predictions from the Mechanistic-Empirical Pavement Design Guide (MEPDG) with test results from the Hamburg wheel-tracking device (HWTD). An experiment was conducted during research to establish correlations effectively between the two methods. The experiment consisted of using volumetric data to model asphalt mixes and pavement structures in the MEPDG and to evaluate the sensitivity of these mixes in comparison to results obtained in the laboratory under the HWTD. The research study analyzed two limestone mixes: a coarse, dense-graded hot-mix asphalt (Type C) and a fine, dense-graded hot-mix asphalt (Type D), as defined in the 2004 Texas Department of Transportation "Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges." The research found that the permanent deformation model embedded in the MEPDG supported the general performance trend observed in the laboratory under the HWTD. Both the MEPDG and the HWTD showed better performance for the Type C mix than for the Type D mix. However, the HWTD showed that small variations in mixture properties, such as mixture gradation within construction tolerance specifications and minor deviations from the optimum in binder content, had significant effects on pavement performance that were not captured by the MEPDG at a Level 2 mixture design. This finding suggests that laboratory results under the HWTD could be used to fine-tune the rutting performance models in the MEPDG until actual field performance data become available.

Hossain, M. S., ***Characterization of Unbound Materials for Use in the New Mechanistic-Empirical Pavement Design Procedure Pavement Materials From Virginia Sources,*** **Report No. FHWA/VTRC 11-R6, Virginia Transportation Research Council, Charlottesville, VA, 2010.** The implementation of mechanistic-empirical pavement design requires mechanistic characterization of pavement layer materials. The subgrade and base materials are used as unbound, and their characterization for Virginia sources was considered in this study as a supplement to a previous study by the Virginia Transportation Research Council. Resilient modulus tests were performed in accordance with AASHTO T 307 on fine and coarse soils along with base aggregates used in Virginia. The degree of saturation as determined by moisture content and density has shown significant influence on the resilient behavior of these unbound materials. The resilient modulus values, or k-values, are presented as reference for use by the Virginia Department of Transportation (VDOT). The results of other tests were analyzed for correlation with the results of the resilient modulus test to determine their use in estimating resilient modulus values. The results of the triaxial compression test, referred to as the quick shear test in AASHTO T 307, correlated favorably with the resilient modulus. Although the complexity of such a test is similar to that of the resilient modulus test for cohesionless coarse soil and base aggregate, fine cohesive soil can be tested with a simpler triaxial test: the unconfined compression test. In this study, a model was developed to estimate the resilient modulus of fine soil from the initial tangent modulus produced on a stress-strain diagram from an unconfined compression test. The following recommendations are made to VDOT's Materials Division: (1) implement the use of the resilient modulus test for pavement design along with the implementation of the MEPDG; (2) use the universal constitutive model recommended by the MEPDG to generate the k-values needed as input to MEPDG Level 1 design/analysis for resilient modulus calculation; (3) develop a database of resilient modulus values (or k-values), which could be used in MEPDG design/analysis if a reasonable material match were to be found; (4) use the initial tangent modulus from an unconfined compression test to predict the resilient modulus values of fine soils for MEPDG Level 2 input and the 1993 AASHTO design; and (5) continue to collect data for the unconfined compression test and update the prediction model for fine soil in collaboration with the Virginia Transportation Research Council. Implementing these recommendations would support and expedite the implementation efforts under way by VDOT to initiate the statewide use of the MEPDG. The use of the MEPDG is expected to improve

VDOT's pavement design capability and should allow VDOT to design pavements with a longer service life and fewer maintenance needs and to predict maintenance and rehabilitation needs more accurately over the life of the pavement.

Im, S., Kim, Y. R., and Ban, H., ***Layer Moduli of Nebraska Pavements for the New Mechanistic-Empirical Pavement Design Guide (MEPDG),*** **Report MPM-08, Nebraska Department of Roads, 2010.** As a stepwise implementation effort of the Mechanistic-Empirical Pavement Design Guide (MEPDG) for the design and analysis of Nebraska flexible pavement systems, this research develops a database of layer moduli–dynamic modulus, creep compliance, and resilient modulus–of various pavement materials used in Nebraska. The database includes all three design input levels. Direct laboratory tests of the representative Nebraska pavement materials are conducted for Level 1 design inputs, and surrogate methods, such as the use of Witczak's predictive equations and the use of default resilient moduli based on soil classification data, are evaluated to include Level 2 and/or Level 3 design inputs. Test results and layer modulus values are summarized in Appendices. Modulus values characterized for each design level are then input into the MEPDG software to investigate level-dependent performance sensitivity of typical asphalt pavements. The MEPDG performance simulation results then reveal any insights into the applicability of different modulus input levels for the design of typical Nebraska pavements. Significant results and findings are presented in this report.

Jackson, E., Li, J., Zofka, A., Yut, I., and Mahoney, J., ***Establishing Default Dynamic Modulus Values For New England,*** **Final Report, University of Connecticut, Storrs, CT, 2011.** The primary objective of this research is to test commonly used Hot Mix Asphalt (HMA) mixtures throughout New England to determine their respective dynamic modulus master curves. Four mixes were requested from each of the New England states for modulus testing. Physical testing consisted of two replicates of each mix, outfitted with 3, linear variable differential transformers (LVDTs). AASHTO TP 62 was followed for the testing of these samples. Comparisons of plant produced mix vs. lab produced mix shows no significant difference between the two methods. Thus indicating lab produced samples are analogous to real-world pavements for dynamic modulus testing. Furthermore, the results of physical modulus testing were compared to predicted modulus values from three different theoretical modulus models. Comparisons of Predicted |E*| values from the Mechanistic-Empirical Pavement Design Guide (MEPDG) and physical testing indicates the predicted |E*| values may be off by as much as 100% for New England Mixes. Through this research scaling factors were developed for all the mixes tested to allow state DOTs to forgo expensive and labor intensive physical testing. Furthermore, the minimal range and standard deviation of scaling factors for the Hirsh and Witczak models indicates there is potentially a constant scaling factor that could be applied to all New England mixes, regardless of aggregate source, and binder type. However, further testing may be required to determine if a uniform scaling factor for our region is truly valid.

Kim, Y. R., "GIS-Based Implementation Methodology for the NCHRP Project 9-23A Recommended Soil Parameters for Use as Input to the MEPDG in North Carolina," Paper 11-3963, Annual Meeting of the Transportation Research Board, National Research Council, Washington, DC, 2011. The product of the NCHRP 9-23A project is a comprehensive nationwide soils database that includes Soil Water Characteristics Curve (SWCC) parameters and other soil properties that are required by the Enhanced Integrated Climatic Module (EICM)

within the Mechanistic-Empirical Pavement Design Guide (MEPDG). The SWCC represents a measure of the water-holding capacity of a given soil for different suction values. Soil suction and water content are important parameters that control permeability, volume change, deformability and the shear strength of unsaturated soils. The EICM is a model through which the MEPDG considers the effects of moisture and temperature profiles on the performance of bound and unbound materials. The NCHRP 9-23A product includes Geographic Information System (GIS)-based soil maps for all states. However, these maps were transformed into image files and stored as PDF documents. The main challenge in the implementation of the NCHRP 9-23A product is the absence of a method that can be used easily and reliably to superimpose any road section on a soil map and, consequently, select the most accurate soils type for that road section. In this paper, a GIS-based methodology is presented that can accurately superimpose any road section on 9-23A soil maps. Moreover, a simple Excel-based Visual Basic for Application (VBA) code is developed to generate Level 1 subgrade materials input that can be imported directly through the MEPDG interface. AASHTO soil classifications extracted from the NCHRP 9-23A database are compared to those extracted from the Long-Term Pavement Performance (LTPP) database and are found to be comparable.

Kim, Y. R., Underwood, B., Far, M. S., Jackson, N., and Puccinelli, J., *LTPP Computed Parameter: Dynamic Modulus*, Report Number FHWA-HRT-10-035, Federal Highway Administration, McLean, VA, 2011. The dynamic modulus, |E*|, is a fundamental property that defines the stiffness characteristics of hot-mix asphalt (HMA) mixtures as a function of loading rate and temperature. In spite of the demonstrated significance of |E*|, it is not included in the current Long-Term Pavement Performance (LTPP) materials tables, because the database structure was established before |E*| was identified as the main HMA property in the Mechanistic-Empirical Pavement Design Guide (MEPDG). The objective of this study was to use readily available binder, volumetric, and resilient material properties in the LTPP database to develop |E*| estimates. This report provides a thorough review of existing prediction models. In addition, several models have been developed using artificial neural networks for use in this study. This report includes assessments of each model, quality control checks applied to the data, and the final structure and format of the dynamic modulus data added to the LTPP database. A program was also developed to assist in populating the LTPP database, and the details of the program are provided in this report.

Kutay, M. E., Chatti, K., and Lei, Ligang, "Backcalculation of Dynamic Modulus Master Curve from Falling Weight Deflectometer Surface Deflections," *Transportation Research Record* 2227, Transportation Research Board, National Research Council, Washington, DC, 2011. The need to characterize the structural condition of existing pavements accurately has increased with the recent development, release, and ongoing implementation of the new "Mechanistic–Empirical Pavement Design Guide" (MEPDG). There is a strong need to identify and evaluate the way that falling weight deflectometer (FWD) testing is operated and integrated into the new design procedure. One of the key inputs in the MEPDG for asphalt pavements is the dynamic modulus (|E*|) mastercurve. If the damaged |E*| mastercurve of the asphalt concrete in an existing pavement can be obtained from FWD deflections, a more accurate prediction of its remaining service life can be achieved. A methodology backcalculates the |E*| mastercurve of the asphalt pavement layer by using the time history of FWD surface deflections. The method uses a layered viscoelastic forward algorithm in an iterative backcalculation procedure for linear

viscoelastic characteristics of asphalt pavements. With deflection time histories from a typical FWD test, it was possible to backcalculate the relaxation modulus curve, E(t), up to about t ~ 10 to the -1 power s and the complex modulus curve, |E*|, from f = 10 to the -3 power Hz and above. Recommendations to improve the accuracy of the backcalculated |E*| mastercurve are provided in the context of enhancing current FWD technology and test procedures.

Lee, H. S., Kim, S., Choubane, B., and Upshaw, P., "Construction of Dynamic Modulus Master Curves Using Resilient Modulus and Creep Test Data," Paper 12-0724, Annual Meeting of the Transportation Research Board, National Research Council, Washington DC, 2012. For the past few decades, the stiffness of materials used for roadway design and construction has been commonly characterized by the resilient modulus, defined as the ratio of the applied stress to the recoverable strain. However, the resilient modulus is not a fundamental material property of a viscoelastic material, and hence, the concept of resilient modulus has been subsequently diminished in the latest Mechanistic-Empirical Pavement Design Guide (MEPDG). Although the MEPDG could not endorse the use of the resilient modulus test protocol as the primary means of asphalt concrete modulus characterization in the design of flexible pavements, it has been a primary mixture test, and a considerable amount of laboratory testing has been completed to date. In this paper, analysis methodologies are introduced for backcalculating the dynamic modulus from the resilient modulus test data. To assess the usefulness of the proposed algorithm, laboratory experiments in both the uniaxial compression and indirect tensile test modes were carried out on asphalt specimens compacted using the Superpave gyratory compactor. The backcalculated dynamic modulus was used to generate the master curve, and the creep test data was used to enhance the accuracy of the master curve. The advantage of such a methodology is that the existing resilient modulus and creep test data can be leveraged for estimating the dynamic modulus. The approach would significantly save time and effort in reevaluating the dynamic modulus of asphalt mixture when the resilient modulus and creep test data are available.

Loria, L. G., Badilla, G., Jimenez Acuna, M., Elizondo, F., and Agauiar-Moya, J. P., "Experiences in the Characterization of Materials Used in the Calibration of the AASHTO Mechanistic-Empirical Pavement Design Guide (MEPDG) for Flexible Pavement for Costa Rica," Paper 11-3359, Annual Meeting of the Transportation Research Board, National Research Council, Washington, DC, 2011. The AASHTO Mechanistic-Empirical Pavement Design Guide (MEPDG) includes empirical distress models that have been calibrated using the North American conditions. But, the differences of material properties, traffic information, and environmental conditions for Latin American countries make necessary to calibrate these models using local conditions. This paper presents an overview of Costa Rica's experience in the calibration of the flexible pavement component of the AASHTO MEPDG performed by the National Laboratory of Materials and Structural Models at the University of Rica (In Spanish, LanammeUCR). First, the paper deals with the importance of using mechanistic-empirical (ME) analysis and design models, as opposed to the purely empirical models that have been traditionally used in Latin America and the world. In second place, it discusses the data and testing requirements to perform proper ME analysis, and the feasibility of performing some of the more complex material characterization tests in Costa Rica. Then, a dynamic modulus (E*) model is developed in order to assess the improvement in accuracy provided by the local calibration (Witczak-Lanamme Model). Finally, this gives rise to future work in calibration of other performance models. This paper also serves as a guide to identify potential problems to highway agencies in their MEPDG calibrations.

Louisiana Transportation Research Center, *Assessment of Environmental, Seasonal and Regional Variations in Pavement Base and Subgrade Properties*, Louisiana Department of Transportation and Development, Baton Rouge, LA, research in progress. The objectives of this project are to: (1) validate the prediction of seasonal variation in base course and subgrade strengths, (2) validate the Mechanistic-Empirical Pavement Design Guide (MEPDG)-provided soil properties and strengths, (3) validate soil properties and locations from Soil Unit Maps, (4) link soil unit maps with the Louisiana Department of Transportation and Development (LADOTD) geotechnical data base, (5) document water table depths, and (6) obtain Level 2 modulus inputs with data from the Falling Weight Deflector (FWD) and Dynamic Cone Penetrometer (DCP). A companion study will be conducted through the southeast Superpave pooled fund study to refine the historical climatic model and build new future climatic models to be utilized in the MEPDG.

Mogawer, W. S., Austerman, A. J., Daniel, J. S., Zhou, F., and Bennert, T., "Evaluation of the Effects of Hot Mix Asphalt Density on Mixture Fatigue Performance, Rutting Performance and MEPDG Distress Predictions," *International Journal of Pavement Engineering*, Volume 12, Number 2, 2011. The purpose of this study was to evaluate the effect of density on the fatigue cracking and rutting performance of hot mix asphalt mixtures. Two plant-produced Superpave mixtures, 9.5 and 12.5 mm, were utilised to fabricate specimens to target density levels of 88, 91, 94 and 97% of the theoretical maximum specific gravity. The specimens were used to evaluate the mixture stiffness in the asphalt mixture performance test device, fatigue cracking characteristics utilising the beam fatigue test and the overlay test-based fatigue cracking analysis and rutting potential using the asphalt pavement analyser and the flow number test. Additionally, the mechanistic-empirical pavement design guide (MEPDG) distress prediction equations were used to predict the mixture performance as function of density. Overall, the testing analysis and MEPDG predictions indicated that higher density specimens yielded improved fatigue and rutting performance.

Nassiri, S. and Vandenbossche, J. M., *Establish Inputs for the New Rigid Component of the Mechanistic-Empirical Pavement Design Guide (MEPDG)*, Report No. FHWA-PA-2011-006-PIT013, Pennsylvania Department of Transportation, Harrisburg, PA, 2011. Each design input in the Mechanistic-Empirical Design Guide (MEPDG) required for the design of jointed plain concrete pavements (JPCPs) is introduced and discussed in this report. The best values for Pennsylvania conditions were established and recommended for each input by considering: (1) typical values suggested in the Pennsylvania Department of Transportation (PennDOT) publications and databases, (2) recommendations in the MEPDG documentation based on nationwide data, and (3) laboratory/field tests performed over the three-year duration of this study.

North Carolina Department of Transportation, *MEPDG Inputs for Warm Mix Asphalts*, research in progress. Warm-mix asphalt (WMA) technologies have emerged as major components of asphalt production and placement that may save fuel costs and lower emissions. Several paving contractors in North Carolina are already using WMA technologies; hence, understanding of the characteristics and long-term performance of various WMA mixtures is urgently needed in order for the NCDOT pavement design engineers to adequately design WMA pavements. Specifically, the NCDOT is preparing for the implementation of the AASHTO Mechanistic-Empirical Pavement Design Guide (MEPDG). Thus, determining appropriate MEPDG input values for various types of WMA mixtures is critical, because MEPDG inputs developed for standard hot-mix asphalt (HMA) mixes may not be applicable to WMA mixtures. Inputs that are not appropriate for WMA mixtures could lead to pavements that have a shorter life than expected or are unnecessarily costly. The objectives of the research project are: (1) to determine the dynamic moduli, fatigue characteristics, and rutting characteristics of WMA mixtures that are currently used in North Carolina as a function of moisture conditioning and aging levels; (2) to compare the material properties of WMA mixtures with their HMA counterparts; and (3) to develop recommendations for MEPDG input parameters for the various WMA mixtures. These objectives will be accomplished by performing dynamic modulus tests for stiffness characterization, direct tension cyclic tests for fatigue performance characterization, and triaxial repeated load permanent deformation (TRLPD) tests for rutting characterization. These tests will be performed on various WMA and HMA mixtures subjected to varying moisture conditioning and aging levels in order to address these two major factors in the different behaviors between the HMA and WMA mixtures. All the test methods and analyses will be the same as those performed under previous NCDOT projects (HWY-2003-09 Typical Dynamic Moduli for North Carolina Asphalt Concrete Mixes and HWY-2007-07 Local Calibration of the MEPDG for Flexible Pavement Design) for HMA characterization so that consistency can be ensured between the existing HMA database and the WMA database to be developed from this study. It should be noted that the Principal Investigator (PI) of the proposed study is currently evaluating various WMA mixtures used for the National Center for Asphalt Technology (NCAT) Test Track and Manitoba WMA projects. The knowledge and experience gleaned from the ongoing WMA study are expected to be significant and will help reduce the time and costs for the proposed study. More importantly, the collaboration with experts at the national level ensures that the outcomes from the proposed study are based on state-of-the-art information. The products of this research will allow a comparison of the performance and costs of WMA pavements with those of HMA pavements. Use of accurate input parameters and calibration factors for WMA mixtures in the MEPDG will help prevent premature failure of pavements and will help prevent the development of unnecessarily costly pavement designs.

Oscarsson, E., "Evaluation of the Mechanistic–Empirical Pavement Design Guide Model for Permanent Deformation in Asphalt Concrete," ***International Journal of Pavement Engineering*****, Volume 12, Number 1, 2011.** The model for permanent deformation in asphalt concrete (AC) layers used in the Mechanistic-Empirical Pavement Design Guide (MEPDG) was evaluated. Two instrumented flexible pavements were subjected to heavy vehicle simulator testing at three constant temperatures. Permanent deformation modelling was carried out using the MEPDG software v1.003 at the highest accuracy level. The nationally calibrated model resulted in underprediction by 35-45%. Model results were successfully calibrated to fit observation using field calibration factors that increased temperature susceptibility by 45-50% and decreased the dependency of the number of loadings by 30-40%. The field calibration factors should be confirmed by studying existing pavements before being employed. The model did not assess the permanent deformation contribution of each AC layer correctly. Therefore, the model should be refined with further trench studies using in-service pavements.

Rao, C., Titus-Glover, L., Bhattacharya, B. B., and Darter, M. I., "Estimation of DeltaT Input for JPCP Design Using the MEPDG," Paper 11-3938, Annual Meeting of the Transportation Research Board, National Research Council, Washington, DC, 2011. In recent years, highway agencies have focused on adopting pavement design practices that meet agency-specified performance requirements. The American Association of State Highway and Transportation Officials (AASHTO) Mechanistic-Empirical Pavement Design Guide, Interim Edition (MEPDG) uses various material property inputs as well as construction and design feature inputs to predict performance. The permanent curl/warp equivalent temperature difference, commonly referred to as ƒ´T or deltaT in rigid pavement design, is a critical input to the jointed plain concrete pavements (JPCP) design procedure, and is often difficult to estimate for a given project. The magnitude of the deltaT procedure is considered to vary with the paving weather, mix design and pavement design features. During the calibration of the MEPDG JPCP distress models, the value of the deltaT parameter was set at -10¢XF. While it is recognized that this assumption is not necessarily accurate for all conditions (or all projects), this was considered the optimum value yielding the closest match to field performance. This paper presents the development of a predictive model to estimate this key JPCP design parameter for a given project based on portland cement concrete (PCC) material index properties, pavement design features, and local climate parameters. The JPCP deltaT negative gradient was found to increase with an increase in temperature range at the project location for the month of construction and slab width, and also with a decrease in slab thickness, PCC unit weight, w/c ratio, and latitude of the project location. This model is valid only for use with the MEPDG Version 1.1 that was utilized in the deltaT model development.

Robbins, M. M. and Timm, D. H., "Evaluation of Dynamic Modulus Predictive Equations for Southeastern United States Asphalt Mixtures," ***Transportation Research Record*** **2210, Transportation Research Board, National Research Council, Washington, DC, 2011.** Mechanistic-empirical pavement design has recently come to the forefront of design, with many states looking to the Mechanistic–Empirical Pavement Design Guide (MEPDG) as their future primary design method. Central to any flexible pavement mechanistic–empirical design framework is the characterization of hot-mix asphalt (HMA) through the use of dynamic modulus E*. Because expensive and specialized equipment is required to measure E* in the laboratory, predictive E* equations that use binder and mixture properties to estimate E* at various frequencies and temperatures must be evaluated. There are currently three global E* predictive equations, two of which are used at Levels 2 and 3 in the MEPDG. These three models (Witczak 1-37A, Witczak 1-40D, and Hirsch) were evaluated with the use of 18 HMA plant-produced, lab-compacted mixtures (representative of general-use mixtures used in the southeastern United States) that were placed at the 2006 National Center for Asphalt Testing test track. E* predictions were made at three temperatures and three frequencies for direct comparison with measured values. The Witczak models had the greatest deviation from measured values, and the Witczak 1-40D model overestimated E* by approximately 61%. The Hirsch model most accurately predicted the moduli for the 2006 test track mixtures. Calibration of the Hirsch model for these mixtures indicated that the Poisson ratio selected for the asphalt binder had little effect on its prediction capabilities. The little improvement resulting from calibration proves that this step is unnecessary.

Roberts, L., Romero, P., VanFrank, K., and Ferrin, R., "Evaluation of the Asphalt Mixture Performance Tester (AMPT): Utah Experience" Paper 12-1793, Annual Meeting of the Transportation Research Board, National Research Council, Washington DC, 2012. For over 5 years, the Utah Department of Transportation (UDOT) has been collecting field material and testing it using the Asphalt Mixture Performance Tester (AMPT). Thirty-four field mixtures were evaluated as part of this study. The mixtures were produced according to UDOT standard volumetric specifications and using 2 different binder grades, PG 64-34 and PG 70-28, from three different suppliers. It was found that the AMPT produced data with a coefficient of variation below 15%, indicating good repeatability. Mixtures prepared with different binder grades were easily separated. The dynamic modulus master curves of mixtures prepared using the same binder grade were essentially the same regardless of the aggregate source (same type of mixture was used). It was shown that it is possible to obtain asphalt mixture level 1 MEPDG input parameters based on historic data and knowledge of the binder grade used and testing at only 1 temperature. The single temperature test can be used for quality control and to verify that the mixture is of the same kind as the historical data available. This approach can significantly reduce the time and effort required to obtain AMPT data without any loss in performance prediction capacity; thus making adoption of this device more appealing to state DOTs.

Rodezno, M. C. and Kaloush, K. E., "Implementation of Asphalt-Rubber Mixes into the Mechanistic-Empirical Pavement Design Guide," *Road Materials and Pavement Design*, Volume 12, Number 2, 2011. This study discusses how the Mechanistic-Empirical Pavement Design Guide (MEPDG) developed by the National Cooperative Highway Research Program (NCHRP) utilizes material properties to predict distresses in pavement structures. This new MEPDG is in the process of replacing the traditional pavement design based on the AASHTO 1993 Design Guide. The Arizona Department of Transportation (ADOT) uses Asphalt- Rubber (AR) mixes statewide. These mixes include both gap and open gradation designs. However, the national calibration process that was undertaken for the MEPDG did not include asphalt-rubber mixes. Because of their unique characteristics, this paper addresses steps and efforts undertaken in a recent study to implement these AR mixes into the MEPDG. There were several issues and limitations identified pertaining to the implementation of AR mixes in the MEPDG. Short and long term recommendations were provided. An important outcome includes a modified Dynamic Modulus predictive equation for AR mixes that is expected to be used in a future implementation of AR mixes in MEPDG.

Rowe, G. M. and Sharrock, M. J., "Alternate Shift Factor Relationship for Describing Temperature Dependency of Viscoelastic Behavior of Asphalt Materials," *Transportation Research Record* 2207, Transportation Research Board, National Research Council, Washington, DC, 2011. Traditionally, various forms of shift factors such as Arrhenius, Williams-Landel–Ferry (WLF), and polynomials have been used with asphalt materials. Shift factors have also been estimated with binder viscosity parameters. Successful extrapolation of viscoelastic functions requires a robust form of shift factor–temperature relationship. This form is important for performing calculations at the extremes of temperature found in practice. A preliminary analysis of complex modulus E* data of mixtures obtained from the "Mechanistic-Empirical Pavement Design Guide" (MEPDG) database demonstrated that the Kaelble form of shift factor could describe the functional form of the shift factor more accurately than the

Arrhenius, WLF, or polynomial-fitting functions. However, the Kaelble shift function as originally described uses the same temperature as a reference temperature and as an inflection temperature. This factor creates a problem when attempts are made to implement the function in a design method or when materials are compared at a given temperature. Since 2008, additional work has investigated the use of this shift function to describe the properties of asphalt materials, particularly mixes and materials that require a wide range of property description (both above and below the glass transition or some other defining point). A modified form of the Kaelble function has been implemented in analysis software and thus makes multiple calculations more rapid. Additional analysis working with MEPDG E* database materials has shown that shifting works best with the Kaelble modification of the WLF equation. The same method has been applied to other asphalt materials.

Salama, H. K. and Chatti, K., "Evaluation of Fatigue and Rut Damage Prediction Methods for Asphalt Concrete Pavements Subjected to Multiple Axle Loads," ***International Journal of Pavement Engineering*****, Volume 12, Number 1, 2011.** The new Mechanistic-Empirical Pavement Design Guide (MEPDG) developed under NCHRP 1-37A does away with the American Association of State Highway and Transportation Officials-derived equivalent single axle load concept and calculates damage caused by various axle configurations directly. However, the calculation of pavement damage caused by multiple axles (tandem, tridem, etc.) is significantly affected by the summation method used to describe the response of the pavement due to the passage of a given axle type or truck configuration. In this paper, the results from different methods of accounting for the passage of a given axle group are compared using laboratory fatigue and rut data from repeated cyclic load tests. The evaluation criterion for these different summation methods was the degree of agreement with the measured laboratory performance. The results show that for fatigue damage, dissipated energy and strain area methods have an excellent agreement with the laboratory values, whereas peak and peak mid-way methods have poor agreement. This implies that it is important to consider the entire strain pulse when calculating fatigue damage under multiple axles. For rutting damage, the peak strain method has the best agreement with the laboratory values, whereas dissipated energy and peak mid-way methods underestimate the rutting damage. Finally, the MEPDG procedure for calculating the strain under multiple axles considerably underestimates the damage for both fatigue and rutting. However, model calibration in the MEPDG does improve the prediction of the damage due to multiple axles for fatigue and to a lesser extent for rutting.

Schwartz, C. W. and Li, R., ***Catalog of Material Properties for Mechanistic-Empirical Pavement Design*****, Report No. MD-11-SP808B4F, Maryland State Highway Administration, Baltimore, MD, 2011.** The new Mechanistic-Empirical Pavement Design Guide (MEPDG) adopted by AASHTO represents a fundamental advance over the current 50-year-old empirical pavement design procedures derived from the AASHO Road Test. The goal is to provide more cost-effective and better-performing pavement designs for the traffic volumes, vehicle characteristics, pavement materials, construction/rehabilitation techniques, and performance demands of today and the future. The MEPDG design procedures are implemented in the new DARWin-ME software currently under development and scheduled for release in April 2011. Material characterization for the MEPDG, the focus of this report, is significantly more fundamental and extensive than in the previous empirically-based AASHTO pavement design methodology. A hierarchical input data scheme has been implemented in the MEPDG to permit varying levels of sophistication for specifying material properties, ranging from laboratory measured values (Level 1) to empirical correlations (Level 2) to default values (Level 3). The development of this type of organized database of material properties for the most common paving materials in Maryland was the primary objective of this study. The database that was developed was populated with information received from the Maryland State Highway Administration (SHA). It provides complete data management tools for adding future data as well as data display screens for MEPDG inputs that mirror the input screens in the MEPDG Version 1.100 software. These data display screens can be easily modified to mirror the DARWin-ME input screens once the DARWin-ME software has been finalized and released to the public. All of the detailed testing recommendations for each of the specific materials are compiled in the summary.

Schwartz, C. W., Li, R., Kim, S., Ceylan, H., and Gopalkrishnan, K., "Effect of Concrete Strength and Stiffness Characterization on Predictions of Mechanistic-Empirical Performance for Rigid Pavements," ***Transportation Research Record*** **2226, Transportation Research Board, National Research Council, Washington, DC, 2011.** The hierarchical approach for specifying design inputs is a key feature of the new "Mechanistic–Empirical Pavement Design Guide" (MEPDG). The three levels of design input for the strength and stiffness characterization of portland cement concrete (PCC) range from a Level 1 laboratory measurement of modulus of elasticity and modulus of rupture at 7, 14, 28, and 90 days to a Level 3 estimation of the 28-day unconfined compressive strength. This paper evaluates the effect of design input level for PCC strength and stiffness properties on MEPDG performance predictions for jointed plain concrete pavements (JPCPs). The effects of the different PCC stiffness and strength design input levels on predicted faulting, transverse cracking, and international roughness index (IRI) are evaluated for eight PCC mixtures in several JPCP design scenarios. Faulting was found to be insensitive to the MEPDG PCC input level, transverse cracking was extremely sensitive, and IRI was only moderately sensitive. In particular, the results showed that the Level 3 input combination of a measured 28-day modulus of rupture and a measured 28-day modulus of elasticity yielded predicted distresses that were consistently in closest agreement with predictions that used Level 1 inputs. Reliable and accurate 28-day modulus of rupture and modulus of elasticity values can therefore be used as less-expensive and more-practical alternatives to full Level 1 stiffness and strength characterization in JPCP analysis and design. When full Level 1 characterization is performed, high-quality testing is mandatory for avoiding small anomalies in measured values that can cause physically unrealistic predictions by the MEPDG of stiffness and strength gains over time.

Shin, H. C., ***Determination of Coefficient of Thermal Expansion Effects on Louisiana's PCC Pavement Design*****, State Project Number 736-99-1450, Louisiana Department of Transportation, Baton Rouge, LA, 2011.** With the development of the Mechanistic-Empirical Pavement Design Guide (MEPDG) as a new pavement design tool, the coefficient of thermal expansion (CTE) is now considered an important design parameter in estimating concrete pavement performance in terms of cracking, faulting, and International Roughness Index (IRI). This study was conducted to measure typical CTE values of Portland cement concrete (PCC) pavements having various aggregates used in Louisiana and to investigate the relationship between CTE and other critical variables such as aggregate types, age of concrete, dimension of specimen, amount of course aggregate in mixture, relative humidity, and concrete mechanical properties. AASHTO TP 60-00 was used for measuring concrete CTE and a recently new standard test method, AASHTO T 336-09, was adopted to replace the TP 60-00. A calibration factor was developed to convert the CTE values measured by AASHTO TP 60-00 to that of the new standard testing method. From the analysis of measured data, it was found that aggregate types, coarse aggregate proportion, and relative humidity have a significant influence on CTE. This finding was confirmed with a statistical analysis of variance (ANOVA). CTE tests and mechanical property tests were also performed at different ages to provide input data for Level 1 design of MEPDG. Based on the results of the MEPDG analysis, current maximum joint spacing [20 ft. (6.1 m)] in jointed plain concrete pavement (JPCP) can be adjusted to 15 or 18 ft. (4.6 or 5.5 m) when Kentucky limestone is used as a coarse aggregate.

Singh, D., Zaman, M., and Commuri, S., "Evaluation of Dynamic Modulus of Modified and Unmodified Asphalt Mixes for Different Input Levels of the MEPDG," ***International Journal of Pavement Research and Technology,*** **Volume 5, Number 1, 2012.** The Mechanistic-Empirical Pavement Design Guide (MEPDG) provides three levels of input for the design and analysis of flexible pavements. The selection of a particular level of input depends on the amount of information available to the designer and the criticality of the project. For all three input levels, the dynamic modulus (|E*|) of hot-mix asphalt (HMA) is used to evaluate the stress-strain characteristics of an asphalt layer associated with its performance (i.e., rutting and fatigue cracking). This study was undertaken to compare |E*| for these three levels of inputs for modified and unmodified HMA mixes. Two different mixes having a similar nominal maximum aggregate size of 19 mm were collected from a production plant. The mixes were prepared with a styrene-butadiene-styrene (SBS)-modified binder of performance grade (PG)70-28 and an unmodified binder of PG64-22. Specimens were prepared for each mix at four different levels of air voids, namely, 6%, 8%, 10%, and 12%. For Level 1, |E*| values were measured in the laboratory at different temperatures and frequencies in accordance with the AASHTO TP62-06 standard. |E*| values for Level 2 and Level 3 were predicted using the Witczak 1999 model provided in the MEPDG. Analyses of the results show that the prediction accuracy of this model for Level 2 and Level 3 varies with the type of mix, temperature, and level of air voids. In addition, it was discovered that this model performs differently for modified and unmodified HMA mixes. To address this variability, correction factors were developed for each type of mix, resulting in more accurate |E*| values comparable to those obtained at Level 1.

Utah Department of Transportation, ***Mechanistic Characterization of Soils and Aggregates,*** **research in progress.** As the Utah Department of transportation (UDOT) moves to implement the Mechanistic-Empirical Pavement Design Guide (MEPDG), new information about the properties of individual pavement layers will be needed. Particular to soils and aggregates, the mechanistic-empirical pavement design process requires knowledge of Poisson's ratio, the coefficient of lateral earth pressure, and resilient modulus as strength properties. While resilient modulus can be estimated in the MEPDG software from CBR and selected other parameters, the application of the correlations to typical Utah materials warrants investigation. Therefore, research to establish resilient modulus values for typical Utah materials is needed, and existing correlations among these test results and those of more commonly performed tests need to be evaluated and new correlations developed as needed; for example, developing correlations between modulus and CBR for typical Utah materials would be very useful to UDOT pavement and materials engineers. In addition, understanding the effects of freezing on the strength properties of soils and aggregates would be useful in determining representative properties to use as inputs in the MEPDG software. The objective of this study is to conduct mechanistic characterizations of typical Utah soils and aggregates to support implementation of the new MEPDG. Tasks for this study include (1) Conduct a literature review and prepare a summary of available information related to mechanistic characterization of soils and aggregates for the MEPDG and correlations between modulus and other properties; (2) Identify and obtain approximately 10 samples of soils and aggregates of greatest interest to UDOT; (3) Design laboratory experimentation to measure resilient modulus (at normal and freezing temperatures), as well as other selected strength properties of each material; (4) Perform analyses of test results, including evaluations of correlations used within the MEPDG software and calculations of representative modulus values for given climates within Utah; (5) Develop an implementation plan; and (6) Prepare a research report documenting the full project.

VonQuintus, H., "Calibration of Rutting Models for Hot-Mix Asphalt Structural and Mix Design: Update on NCHRP Project 9-30A," ***Engineering Circular*** **155, Transportation Research Board, National Research Council, Washington, DC, 2011.** This presentation includes an overview and comparison of different rut depth transfer functions that were included in a modified version of the Mechanistic–Empirical Pavement Design Guide (MEPDG) software identified as Version 9-30A, prepared under NCHRP project 9-30A. The presentation is organized into four topics: (1) an overview of the project objectives and rut depth transfer functions, (2) comparison of predicted and measured rut depths using the different transfer functions, (3) assessment and effectiveness of the transfer functions, and (4) a summary of the observations and findings from the project.

Wen, H., "**Development of a Damage-based Phenomenological Fatigue Model for Asphalt Pavement," Paper 11-4100, Annual Meeting of the Transportation Research Board, National Research Council, Washington, DC, 2011.** Bottom-up fatigue cracking is one of the major distresses for asphalt pavements. Accurate prediction of fatigue cracking for asphalt pavement is of paramount importance for a cost-effective pavement design. The fatigue model in the mechanistic empirical pavement design guide (MEPDG) is based on original Asphalt Institute model, with some modifications. However, there are some controversies about the effectiveness of fatigue model in the MEPDG. The major concern exists on the use of dynamic modulus as key parameter and there is no damage property of asphaltic mix to predict fatigue which is induced by damage to the material. This study developed a damage-based phenomenological fatigue model. The pavements at the Federal Highway Administration's accelerated loading facility (ALF) were used to test the effectiveness of existing models, including the MEPDG fatigue model, and validity of the damage-based fatigue model. The data used in this study included dynamic modulus and critical strain energy density of hot mix asphalt (HMA), and tensile strain at bottom of HMA layer and the fatigue life of ALF pavements. It was found that the damage-based model significantly improved the accuracy of the prediction, when compared to the MEPDG fatigue model and other conventional models.

Wiser, L., ***LTPP Computed Parameter: Dynamic Modulus*****, Report No. FHWA-HRT-11-018, Federal Highway Administration, McLean, VA, 2010.** This document is a technical summary of the Federal Highway Administration (FHWA) report, LTPP Computed Parameter: Dynamic Modulus, FHWA-HRT-10-035. The primary objective of this project was to develop estimates of the dynamic modulus, |E*|, of hot-mix asphalt (HMA) layers on Long-Term Pavement Performance (LTPP) program test sections following the models used in the Mechanistic-Empirical Pavement Design Guide (MEPDG). These data will provide a means of linking MEPDG inputs (for HMA analysis) to known field performance as measured on LTPP test sections. As part of this project, existing models used to estimate |E*| values were evaluated, and additional models were developed based on the use of Artificial Neural Networks (ANNs). The models utilize readily available mixture and binder information to estimate dynamic modulus.

Witczak, M. W., El-Basyouny, M., and Uzan, J., ***Adapting Specification Criteria for Simple Performance Tests to HMA Mix Design*****, Project 9-33A, National Cooperative Highway Research Program, Transportation Research Board, National Research Council, Washington, DC, 2011.** The objective of this project was to develop a software program for evaluating the potential performance of hot mix asphalt (HMA) mix designs in combination with their intended pavement structures. This "Program for Integrated Analysis of HMA Mix And Structural Designs" is coded as a Microsoft Excel spreadsheet (9-33A(Sep10).xlsm) and supporting files. The program incorporates the Mechanistic-Empirical Pavement Design Guide (MEPDG) spreadsheet solutions developed in NCHRP Projects 9-19 and 9-22. Predictions of permanent deformation (rutting) and fatigue cracking are made on the basis of the estimated HMA dynamic modulus, E*; thermal cracking predictions are based on estimates of the HMA creep compliance, D. The program will serve as a multi-purpose tool for HMA mix and structural design engineers. First, it provides an easy graphical check that a prospective job mix formula (JMF) falls within the acceptable limits of air voids and effective binder volume established by the project's HMA specification. Second, using powerful, pre-solved solutions of the MEPDG, it provides rapid estimates of the performance of the JMF over the design life of the HMA pavement and whether the JMF will satisfy specific pavement distress criteria established by the agency. Third, it can test "what-if" scenarios by estimating how changes in the JMF, pavement structure, or both may affect performance. Finally, it can be used in forensic investigations of pavement distresses, by assessing the potential contributions of the HMA and pavement structure to distress development before any testing is conducted. This report presents (1) a description of the program's inputs and outputs, (2) a brief review of the underlying performance prediction models, and (3) examples illustrating the use of the program to analyze specific mix-structure combinations. Technical familiarity with the MEPDG design principles, the E* Simple Performance Test (SPT) Specification Criteria Program, and the Quality-Related Specification Software (QRSS) will enhance the user's understanding of the program.

Zborowski, A. and Kaloush, K. E., "A Fracture Energy Approach to Model the Thermal Cracking Performance of Asphalt Rubber Mixtures," ***Road Materials and Pavement Design*****, Volume 12, Number 2, 2011.** The existing thermal cracking model (TCMODEL) that is currently an integral part of the Mechanistic-Empirical Pavement Design Guide (MEPDG) has been shown to adequately predict low temperature cracking of asphalt concrete mixtures utilizing conventional binders. This study concluded that the existing TCMODEL in the MEPDG has limitations in adequately predicting low temperature cracking in asphalt rubber mixtures. This paper presents development and findings of a new method for thermal cracking potential evaluation in asphalt mixtures, with a focus on asphalt rubber mixtures. Refinements of the existing indirect iensile (IDT) test protocol are presented, and a new crack depth fracture model is proposed to include a fracture energy parameter. The new fracture energy model developed in this study was evaluated with laboratory test results and rationality corresponding with field observations. The new model proved to be satisfactory in predicting the thermal cracking performance for both conventional and asphalt rubber mixtures. Its use in future MEPDG models is recommended.

Other Topics

Aguiar-Moya, J. P. and Prozzi, J., *Development of Reliable Pavement Models*, Report No. SWUTC/11/161025-1, Southwest Regional Transportation Center, Texas Transportation Institute, Texas A&M University, College Station, TX, 2011. This report presents a framework for estimating the reliability of a given pavement structure as analyzed by the Mechanistic-Empirical Pavement Design Guide (MEPDG). The methodology uses a previously fit response surface, in place of the time-demanding implicit limit state functions used within the MEPDG, in combination with an analytical approach to estimating reliability using first-order and second-order reliability methods (FORM and SORM). In addition, to assess the accuracy of the FORM and SORM reliability estimates, Monte Carlo simulations are also performed. A case study based on a three-layer pavement structure is used to demonstrate the methodology. Several pavement design variables are treated as random; these include HMA and base layer thicknesses, base and subgrade modulus, and HMA layer binder and air void content. Information on the variability of and correlation among these variables are obtained from the Long-Term Pavement Performance (LTPP) database. Response surfaces for limit states dealing with HMA rutting failure are fit using several runs of the MEPDG, based on a factorial design of combinations among the aforementioned random variables, as well as traffic, structural, and climatic considerations. These response surfaces are then used to analyze the reliability of the given pavement structure. Using the second moment and simulation techniques, it was found that on average the reliability estimate by the MEPDG is very conservative. Additionally, the validity of the methodology is verified by means of direct simulation using the MEPDG. Finally, recommendations on fitting the response surface are provided to ensure the applicability of the methodology.

Archilla, A. R., "Hawaii's Efforts to Implement the Mechanistic–Empirical Pavement Design Guide," *Engineering Circular* 155, Transportation Research Board, National Research Council, Washington, DC, 2011. This presentation includes an overview of several efforts directed to aid in the implementation of the AASHTO Mechanistic–Empirical Pavement Design Guide (MEPDG) in the state of Hawaii. The presentation discusses a few issues with existing software designed for manipulating traffic loading information (TRAFLOAD). An important problem identified is that in some situations the derived number of axles per vehicle is erroneous. A summary of research aimed at developing models for characterization of pavement material behavior, including unbound materials and hot-mix asphalt (HMA), is provided. In addition, the presentation also discusses some locally developed pavement management system (PMS) tools that could provide useful information for the local calibration of the MEPDG.

Banerjee, A., Prozzi, J. A., and Freiman, A., "Regional Calibration of Permanent Deformation Performance Models for Rehabilitated Flexible Pavements," Paper 11-1025, Annual Meeting of the Transportation Research Board, National Research Council,

Washington, DC, 2011. The Mechanistic-Empirical Pavement Design Guide (MEPDG) is a pavement analysis system that has been gaining popularity as an analysis tool for new and rehabilitated pavement structures. A handful of states are already using the MEPDG for design. The performance models used in the MEPDG, developed under the National Cooperative Highway Research Program Projects NCHRP 1-37A and 1-40D, have been calibrated using sections spread throughout the United States. It is necessary to calibrate these models for specific states and regional conditions due to the differences in terms of materials, environmental conditions and construction practices. In general, a pavement design based on the nationally calibrated MEPDG will result in either an overestimate or an underestimate of the pavement layer thicknesses because of systematic errors due to local differences. An average does not necessary represent any specific condition. This deficiency calls for a local calibration of the performance models in the MEPDG, so that it can be used to design pavements at a regional level. Several states have already done so; others are in the process. This paper documents some of the calibration work in Texas. This study focuses on finding bias correction factors for systematic differences due to traffic, climatic conditions, and material properties that govern the initial condition as well as the progression of rutting over time. Bias correction factors for rutting in the subgrade were assumed based on inputs from other similar studies, which are properly referenced. The paper proposes a set of Level 2 bias correction factors which are good for analysis of rehabilitated pavements for the regions specified. It has been also observed that pavements rehabilitated in warmer climatic areas will experience lower initial rutting, but a higher rate of increase in the rut depth with time compared to those constructed in colder climatic areas. It should be emphasized that calibration factors for new and rehabilitated pavements are significantly different.

Bustos, M., Cordo, O., Girardi, P., and Pereyra, M., "Calibration of Distress Models from the Mechanistic-Empirical Pavement Design Guide for Rigid Pavement Design in Argentina," ***Transportation Research Record*** **2226, Transportation Research Board, National Research Council, Washington, DC, 2011.** This paper presents the main results of a calibration of distress models from the Mechanistic-Empirical Pavement Design Guide (MEPDG) to local conditions in Argentina to be used in rigid pavement structural design. Test sections with rigid pavements were selected to cover a wide range of climatic conditions in the central region of the country. Local road agencies provided information about pavement structure, traffic volume, and load spectra. Field distress data and international roughness index (IRI) values were collected and processed with MEPDG software; calibration factors were determined for the different distress models of rigid pavements. The accuracy of distress prediction was significantly improved if calibration factors that considered additional influence of climatic conditions, soil and base type, slab length, and pavement age were incorporated into the transverse joint-faulting and transverse-cracking models. Through the use of calibrated factors instead of default values provided in MEPDG models, the errors in distress predictions were reduced by more than one-half in all cases. Construction procedures seemed to influence the IRI roughness prediction. Better calibration results were obtained if different after-construction IRI values were considered for pavements constructed before and after the 1990s to reflect the difference between older and newer constructive techniques.

Heitzman, M., "Summary of Mechanistic-Empirical Pavement Design Guide Regional User Group Meetings," ***Engineering Circular*** **155, Transportation Research Board, National**

Research Council, Washington, DC, 2011. In 2007, nine states in the north-central region proposed to meet and share their expertise, challenges and accomplishments toward successful implementation of the Mechanistic-Empirical Pavement Design Guide (MEPDG). The Federal Highway Administration's (FHWA's) Design Guide Implementation Team (DGIT) agreed to sponsor the meeting, and the National Center for Asphalt Technology (NCAT) and the Concrete Preservations Technology (CPTech) Center facilitated the February 2008 two-day event. Based on the success of the north-central meeting, FHWA's DGIT sponsored four additional regional meetings in late 2008 and early 2009 to cover the balance of the country. All five meetings were planned independently, but a common agenda emerged. Each meeting started with a general overview of individual states' MEPDG implementation plans. The second session examined general implementation issues, such as how to calibrate the performance models (empirical transfer functions) and how to coordinate the effort of multiple department of transportation (DOT) offices. The third session allowed the participants to examine more specific technical details regarding traffic and material inputs. The meeting ended with a discussion of software limitations and regional challenges and opportunities. This report summarizes the limitations, challenges, and opportunities identified by the participants.

Ceylan, H. and Gopalakrishnan, K., "Computationally Efficient Surrogate Response Models for Mechanistic-Empirical Pavement Analysis and Design," ***Structure and Infrastructure Engineering*****, Volume 7, Number 4, 2011. T**his paper proposes the use of neural network- (NN-) based pavement structural analysis tools as surrogates for the flexible pavement response analysis in the new mechanistic empirical pavement design guide (MEPDG) developed for the American State Highway and Transportation Officials (AASHTO). Some of the recent successful applications of NN-based structural analysis models for predicting critical flexible pavement responses and nonlinear pavement layer moduli from falling weight deflectometer (FWD) deflection basins are highlighted. Because NNs excel at mapping in higher-order spaces, such models can go beyond the existing univariate relationships between pavement structural responses and performance (such as the subgrade strain criteria for considering flexible pavement rutting). The NN-based rapid prediction models could easily be incorporated into the newer versions of the MEPDG, which will continue to be updated. This can lead to better performance prediction and also reduce the risk of premature pavement failure.

Delgadillo, R., Wahr, C., and Alarcón, J. P., "Toward Implementation of the Mechanistic–Empirical Pavement Design Guide in Latin America: Preliminary Work in Chile," ***Transportation Research Record*** **2226, Transportation Research Board, National Research Council, Washington, DC, 2011.** The Mechanistic-Empirical Pavement Design Guide (MEPDG) and MEPDG software are tools that provide more rational pavement designs than previous guides. The application of the mechanistic-empirical method, however, requires careful consideration of the input data and proper calibration to local conditions. Since 2004, several efforts have been made in Chile to study implementation of MEPDG in that country. The work has included obtaining local input data (axle load distributions, weather, and material characteristics), calibration efforts for distress prediction, and comparison of pavement designs by using MEPDG and the local design method. Axle load distributions were obtained for three important highways in the most populated area in the country. Statistical analysis was performed to compare the default load spectra from MEPDG with the local spectra. Significant differences were observed. Weather information was obtained for six cities that represent broadly the different weather conditions in the country. Master curves were developed for Level 2 design on the basis of the aggregate and binder properties of the local materials. Preliminary local calibration factors were obtained for transverse cracking and faulting of jointed plain concrete pavement. These local data were employed to develop pavement designs with the MEPDG software; these designs were then compared with designs developed with the current Chilean design methodology, which is based on AASHTO 93 and 98 methods. The work presented shows that the implementation of the MEPDG is possible for countries of Latin America. Some recommendations are provided for future versions of the MEPDG to make use of the software in the Southern Hemisphere easier and friendlier.

El-Badaway, S. M., Bayomy, F. M., Santi. M., and Clawson, C. W., "Comparison of Idaho Pavement Design Procedure with AASHTO 1993 and MEPDG Methods," First T&DI Congress 2011: Integrated Transportation and Development for a Better Tomorrow, American Society of Civil Engineers, Chicago, IL, 2011. Several in service pavements located in different regions of Idaho that have been designed according to the ITD design method were redesigned using the AASHTO 1993 as well as the Mechanistic-Empirical Pavement Design Guide (MEPDG) procedures. All designs were conducted at a 50% reliability level. The nationally calibrated MEPDG (version 1.1) was used to predict the performance of the three design methods. Level 2 subgrade material characterization inputs were used in the MEPDG analysis. All other MEPDG inputs were level 3. Performance indicators predicted using MEPDG related to the three design methods were compared to each other. Results showed that, relative to AASHTO 1993 and MEPDG procedures, ITD design method significantly overestimates the thickness of the pavement structure, and particularly the thickness(s) of the unbound layer(s). On the other hand, the AASHTO 1993 and MEPDG guides show reasonable agreement on the resulting pavement structure.

Gedafa, D. S., Khanum, T., Hossain, M., and Schieber, G. "Effect of Construction Environment on JPCP Performance," First T&DI Congress 2011: Integrated Transportation and Development for a Better Tomorrow, Chicago, IL, 2011. Some properties of newly paved jointed plain concrete pavements (JPCP) are known to influence long-term performance. The traditional empirical design procedures for JPCP were unable to take into account most of these factors. However, the new Mechanistic-Empirical Pavement Design Guide (MEPDG) accounts for climatic conditions, local materials, selected construction practices, and actual highway traffic distribution. In this study, performance (in terms of International Roughness Index (IRI), faulting, and percent slab cracked) of six typical JPCP pavements in Kansas corresponding to alternative inputs of Portland cement concrete (PCC) strength development, PCC shrinkage, and "zero-stress" temperature has been evaluated using MEPDG. The results show that predicted JPCP roughness (IRI) and faulting by MEPDG are not very sensitive to the PCC strength. However, slab cracking decreases with higher PCC strength. In general, PCC shrinkage does not affect predicted IRI. Higher shrinkage strain results in higher faulting. Long-term cracking appears to be fairly insensitive to the shrinkage strain. MEPDG-predicted IRI and percent slabs cracked are fairly insensitive to the zero-stress temperature but the faulting is severely affected except on a JPCP pavement section with widened lane and tied PCC shoulder. Percent slab cracked highly depends on the PCC slab thickness. April and October are the best months for JPCP construction (paving) in Kansas.

Gedafa, D. S., Mulandi, J., Hossain, M. and Schieber, G., "Comparison of Pavement Design Using AASHTO 1993 and NCHRP Mechanistic-Empirical Pavement Design Guides," First T&DI Congress 2011: Integrated Transportation and Development for a Better Tomorrow, American Society of Civil Engineers, Chicago, IL, 2011. The new Mechanistic-Empirical Pavement Design Guide (MEPDG) provides methodologies for mechanistic-empirical pavement design as opposed to the empirical methodology used in the 1993 American Association of State Highway and Transportation Officials (AASHTO) pavement design guide. The objective of this study was to compare the pavement designs obtained using the 1993 AASHTO and the new MEPDG methods for typical Portland Cement Concrete (PCC) and Asphalt Concrete (AC) pavements in Kansas. Five in-service Jointed Plain Concrete Pavement (JPCP) projects were reanalyzed as equivalent JPCP and AC projects using both approaches at the same reliability level. The results show that the new MEPDG analysis yielded thinner AC sections for all projects than those obtained from the 1993 AASHTO design guide analysis. Four of the PCC sections, designed using the 1993 AASHTO design guide, were thicker than the sections obtained with MEPDG. The MEPDG analysis resulted in thicker PCC slab for the fifth project. Effect of change in performance criteria on the thickness of AC and PCC sections has also been investigated. It has been found that AC sections are more sensitive to change in performance criteria as compared to PCC sections using MEPDG versions 1.0 and 1.1. In general, difference in thickness using both versions is not significant for all practical purposes.

Graves, R. C. and Mahboub, K. C., "Sampling-Based Flexible Pavement Design Reliability Evaluation of the Mechanistic Empirical Pavement Design Guide (MEPDG)," 8th International Conference on Managing Pavement Assets, Santiago, Chile, 2011. The Mechanistic-Empirical Pavement Design Guide (MEPDG) Version 1.1 developed under NCHRP 1-40 is a very complex tool for the evaluation of pavement structures. The guide provides a means to utilize a variety of input parameters to characterize the materials and construction techniques utilized to build roadway pavements. It provides a prediction of the performance of a variety of distresses. This prediction is considered to be a mean (50% reliability) based upon the input parameters utilized. In addition, it allows the designer the option to select design reliability. This reliability is based upon the prediction error (standard error) of the distress prediction equations, and it is assumed to be normally distributed about the mean predicted distress. Additionally, it is assumed that this reliability would account for all the variation in the design inputs and model parameters. This research provides a method for evaluating the variation of the predicted outputs based upon the assumed variability of the input parameters. This is accomplished by selecting typical variability of the input parameters and then processing them through the MEPDG. The study varied the following input parameters: Average Annual Daily Truck Traffic (AADTT), hot mix asphalt (HMA) surface mix properties, HMA base mix properties, HMA base, and crushed stone thicknesses and moduli. More than 100 design scenarios were randomly sampled from these input matrices. The results of this study indicated that the variability of the predicted performance is not necessarily normally distributed through typical ranges of input parameters. An alternative method for addressing reliability in MEPDG may involve utilizing the actual distribution of errors.

Gaurav, G., Wojtkiewicz, S. F., and Khazanovich, L., "Optimal Design of Flexible Pavements Using a Framework of DAKOTA and MEPDG," ***International Journal of Pavement Engineering*****, Volume 12, Number 2, 2011.** Pavement design optimization is an active area of research. Due to a large number of parameters, such as thickness of layers, material properties, climatic conditions, affecting pavement performance, it is usually not feasible to determine an optimal design using a trial and error approach. In order to make the design calculation computationally tractable, the process can be posed as an optimisation problem. Previous investigations in this vein have suffered from the limitations of a specific pavement analysis tool, specific design goals and specific optimisation algorithms. This paper presents a general computational framework, combining the Mechanistic-Empirical Pavement Design Guide and the Design Analysis Kit for Optimization and Terascale Applications (DAKOTA), to overcome these shortcomings. The framework's promise is demonstrated through its application to a minimum cost pavement design problem using both direct and surrogate-based optimisation (SBO) approaches. The SBO formulation is shown to achieve significant savings in required computational time with a minimal loss of accuracy in the determined optimal design.

Guthrie, W. S. and Butler, M. J., "Field Evaluation of Asphalt Overlays on State Route 30 in Northern Utah," Paper 11-2008, Annual Meeting of the Transportation Research Board, National Research Council, Washington, DC, 2011. The purpose of this research was to compare the rutting, cracking, and development of roughness of two asphalt overlay types commonly used in northern Utah and to evaluate how well the Mechanistic-Empirical Pavement Design Guide (MEPDG) can predict the observed results. AC-10 and PG 64-34 asphalt overlay materials were paved in a checkerboard pattern at a test site on State Route 30 near Logan, Utah, and observed for 3 years at 6-month intervals. Primary data included rutting, cracking, and roughness. At the conclusion of the 3-year period, the AC-10 overlay exhibited more rutting but less transverse, longitudinal, and fatigue cracking and lower roughness than the PG 64-34 overlay. Although the MEPDG predictions for rutting were within the range of observed rut depths, the MEPDG overestimated the AC-10 rut depth while underestimating the PG 64-34 rut depth. The MEPDG predicted negligible cracking for both overlay types for the duration of the 3-year analysis period. While the MEPDG cracking models appear to be unsuitable for predicting cracking at this site, the MEPDG predictions for roughness are shown to be within the range of observed values. Given the findings of this study, the researchers recommend that Utah Department of Transportation (UDOT) engineers consider specifying the AC-10 asphalt overlay product for pavement treatments in conditions similar to those evaluated in this investigation. Even though the MEPDG predictions of rutting and roughness were generally correct, further evaluation of the MEPDG cracking models should be completed before the MEPDG is fully adopted by UDOT.

Hall, K. D., Wang, K. C. P., and Xiao, D. X, "Calibration of the Mechanistic-Empirical Pavement Design Guide for Flexible Pavement Design in Arkansas," ***Transportation Research Record*** **2226, Transportation Research Board, National Research Council, Washington, DC, 2011.** Because of potential differences between national and local conditions, the Mechanistic-Empirical Pavement Design Guide (MEPDG) should be calibrated to a local level. Arkansas has invested heavily in efforts to implement the MEPDG. This paper summarizes the initial local calibration of flexible pavement models in the MEPDG for Arkansas. Data from the Long-Term Pavement Performance (LTPP) database and local pavement management system (PMS) were used. The solver function in Microsoft Excel was used to optimize the coefficients for alligator cracking. Iterative runs of the MEPDG by means of discrete calibration coefficients were conducted to optimize rutting models. In general, the alligator cracking and rutting models are improved by calibration. However, a question remains about the suitability of the calibrated models for routine design. Many default values were used in the MEPDG because of a lack of data. It is recommended that additional sites be established and a more robust data collection procedure be implemented for future calibration efforts. The difference in the definitions of transverse cracking between the MEPDG and the LTPP may be critical to data collection and identification. Thermal cracking should be specifically identified in a transverse cracking survey to calibrate the transverse cracking model in MEPDG. The procedure using LTPP and PMS data for local calibration of the MEPDG in Arkansas is established. Additional development of database software for data manipulation, preprocessing, and quality control—under way in Arkansas—will significantly streamline the calibration process.

Idaho Department of Transportation, *Calibration of the MEPDG Performance Models for Flexible Pavements in Idaho*, research in progress. This project will develop local calibration (adjustment) factors for the mechanistic-empirical pavement design guide (MEPDG) predictive models for flexible pavement design. The project will ensure a successful implementation of MEPDG in Idaho.

Iowa Department of Transportation, *Iowa Calibration of MEPDG Performance Prediction Models*, research in progress. The primary benefit of this research will be improved accuracy of pavement performance predictions for Iowa pavement systems in addition to implementing the use of Mechanistic-Empirical Pavement Design Guide (MEPDG) in Iowa, with the proposed local calibration factors. A systematic and thorough evaluation of the Iowa Department of Transportation's (Iowa DOT) existing Pavement Management Information System (PMIS) will be valuable in identifying the current status of the database and the information that may need to be collected in the future as part of the routine pavement evaluation. The knowledge gathered during this research will also be helpful in developing advanced distress prediction models based on past experience.

Li., J., Uhlmeyer, J. S., Mahoney, J. P., and Muench, S. T., *Use of the 1993 AASHTO Guide, MEPDG and Historical Performance to Update the WSDOT Pavement Design Catalog*, Report No. WA-RD 779.1, Washington State Department of Transportation, Tumwater, WA, 2011. This report describes the preparation of a revised pavement thickness design catalog for the Washington State Department of Transportation (WSDOT) using the 1993 American Association of State Highway and Transportation Officials (AASHTO) Guide, the Mechanistic-Empirical Pavement Design Guide (MEPDG), and WSDOT historical pavement performance data.

Li, Q., Xiao, D. X., and Hall, K. D., "Mechanistic-Empirical Pavement Design Guide-Based Pavement Design Catalog for Low-Volume Roads in Arkansas," *Transportation Research Record 2203*, Transportation Research Board, National Research Council, Washington, DC, 2011. Historically, low-volume roads in Arkansas were typically constructed by use of a standard section, that is, a double surface treatment over a specified thickness of granular base. Subsequent analysis indicated that these sections were structurally inadequate in many cases. In recent years, the Arkansas State Highway and Transportation Department has invested significant research dollars to implement the "Mechanistic–Empirical Pavement Design Guide"

(MEPDG), which is widely believed to be a quantitative leap over the 1993 AASHTO design guide. However, the MEPDG research efforts mostly target high-volume roads. In this paper, a design catalog for low-volume roads (LVRs) in Arkansas was developed with MEPDG software, Version 1.10. The catalog offers a variety of feasible design alternatives for a comprehensive combination of site conditions. The factors considered include the five geographical regions in Arkansas and the typical Arkansas load spectrum for LVRs with three traffic levels, three subgrade types, and six potential aggregate types available in Arkansas that can be used as granular base and surface layer aggregates. All the MEPDG design inputs needed for the development of the design catalog are generated on the basis of the variety of previous MEPDG implementation research conducted in the state of Arkansas. It is anticipated that the design catalog will serve as a simplified and rational design process for the LVRs in Arkansas.

Li, Q. J., Xiao, D. X., McNeil, S., and Wang, K. C. P., "Benchmarking Sustainable Mechanistic-Empirical Based Pavement Design Alternatives Using Data Envelopment Analysis (DEA)," 8th International Conference on Managing Pavement Assets, Santiago, Chile, 2011. The selection of alternative pavement designs is not an exact science but one in which the highway engineer uses judgment to balance various factors. Integrating these factors into the selection process is challenging. In this paper, Data Envelopment Analysis (DEA), a methodology based on linear programming, is used to measure the efficiency of alternative designs. Eight alternative designs, including both rigid and flexible pavements, for the proposed I-49 Bella Vista bypass in Arkansas, are generated following the Mechanistic- Empirical Pavement Design Guide (MEPDG). The assessment of the overall impacts of these designs on the environment during road construction and maintenance operations are conducted using the Pavement Life-cycle Assessment Tool for Environmental and Economic Effects (PaLATE). Economic output in terms of Net Present Value (NPV) and environmental outputs including energy consumption and gaseous emissions during initial construction and pavement maintenance are quantified and used as the inputs and outputs for the DEA process, from which the efficiency of each alternative is determined and therefore the most sustainable design is benchmarked. The analysis shows that the conventional deep strength flexible pavement design using Reclaimed Asphalt Pavement materials as the base, and rigid pavements designs using cement treated base or granular base are the most efficient among the eight designs for this project. It is anticipated that the proposed framework can assist highway agencies in achieving sustainable pavement designs.

Mehta, Y., and Bennert, T., "New Jersey's Efforts to Implement the Mechanistic–Empirical Pavement Design Guide," ***Engineering Circular*** **155, Transportation Research Board, National Research Council, Washington, DC, 2011.** This presentation provides an overview of the effort conducted by the state of New Jersey towards implementation of the AASHTO Mechanistic–Empirical Pavement Design Guide (MEPDG). New Jersey is one of the lead states in implementing the MEPDG. Leading up to the implementation, Rutgers University conducted an extensive evaluation of the paving materials used in New Jersey and developed a materials database. The database included resilient modulus of subgrades and base-subbase materials. In addition, a dynamic modulus database of mixtures was developed. Rutgers University assisted in conducting Design Guide Implementation Workshops. Beginning in fall 2008, Rowan University conducted Level 3 input verification of all distresses. Rowan University also calibrated and validated the fatigue cracking model based on 29 field pavement sections in

New Jersey. Then a pavement catalog was developed in the form of a user-friendly Microsoft Access database. This catalog will help to identify candidate pavement structures that will meet failure criteria during the design life, based on MEDPG Level 3 inputs.

Oklahoma Department of Transportation*, Implementation of MEPDG for Asphalt Pavements with RAP*, research in progress. Hot-mix asphalt (HMA) is the most widely used paving material in the U.S. Each year as much as 100 million tons of HMA are milled during road resurfacing and widening projects. About 80 million tons (80%) of this amount are reused as recycled asphalt pavement (RAP). Recent studies show that in addition to preserving the natural environment, significant cost savings are realized with increased use of RAP ($3.7 per ton of HMA for each 10 percent increase in RAP, based on Virginia data). In Oklahoma, the current state of practice is to allow up to 25% RAP for base courses, while none for surface courses. Comparatively, a number of neighboring states including Arkansas, Louisiana and Texas allow 30% or more RAP in base courses and 10% or more in surface courses. Some of the major reasons for using a lower percentage of RAP in Oklahoma are variations in RAP quality and lack of field/laboratory performance data on mixes with high RAPs. Additionally, the implementation of the new mechanistic-empirical pavement design guide (MEPDG) requires mechanistic input parameters for asphalt mixes with RAP and rheological properties of the blended (virgin and recovered) binders. In this collaborative study between the University of Oklahoma (OU) and Langston University (LU), we will generate laboratory and field data vital to the implementation of the new MEPDG for asphalt mixes with high RAP. In addition to technical and economic merits, this study will strengthen collaboration between two OTC institutions (OU and LU) as well as stakeholders (Oklahoma Department of Transportation, Oklahoma Asphalt Paving Association).The study seeks to generate useful laboratory and field data for the accelerated use of RAP in asphalt paving in Oklahoma. Specifically, this study seeks to evaluate two RAPs, two virgin aggregates, one virgin binder and one anti-stripping agent through comprehensive laboratory testing. The laboratory study will involve PG grading of the virgin, recovered and blended binders pertaining to the percentages of RAP used in the mix design. The MEPDG input parameters for the blended binders will be evaluated. In addition, elemental analyses of the blended binders will be performed. Physical and mechanical properties including gradation, soundness and insoluble residue of virgin and extracted (from RAP) aggregates will be determined. Asphalt mixes with varying percentages of RAPs (up to 40% for base course (S3) and up to 10% for surface course (S4)) will be designed. Also, their MEPDG input parameters (dynamic modulus, creep compliance and indirect tensile strength) and performance relative to rut, moisture damage and fatigue will be evaluated. In addition to laboratory compacted specimens, field compacted specimens will be collected from selected sites and tested. Two field sites, preferably involving temporary pavements (e.g., detours) and/or shoulders will be identified in collaboration with ODOT and OAPA for this purpose. Outreach activities include organizing two technology transfer workshops, preferably one in collaboration with ODOT Division 6 and the other in collaboration with OAPA, for effective dissemination of results and recycling awareness.

Pierce, L. M., Zimmerman, K. A., and Saadatmand, N., "Use of Pavement Management Data for Calibration of the Mechanistic-Empirical Pavement Design Guide," 8th International Conference on Managing Pavement Assets, Santiago, Chile, 2011. Implementation of the Mechanistic-Empirical Pavement Design Guide (MEPDG) is expected to improve the efficiency of pavement designs and enhance the abilities of highway agencies to predict pavement performance, which will thereby improve their ability to assess maintenance and rehabilitation needs over the life of the pavement structure. Before the MEPDG can be fully implemented, verification and if necessary, calibration using actual pavement design input and response data to ensure its validity and accuracy to local conditions is needed. The MEPDG has been nationally calibrated using data contained within the Long-Term Pavement Program (LTPP) database. Although the LTPP database represents a valuable resource, the enormous variability between the states in terms of geography, climatic conditions, construction materials, construction practices, traffic compositions and volumes, and numerous other pavement design variables make it desirable to calibrate the MEPDG at the local level using local field performance data. Collection of data needed to support the local calibration effort is expensive, time consuming, and resource intensive, but significant savings could be realized by highway agencies if existing pavement management system data could be used for MEPDG performance prediction model calibration. This paper will discuss the development of a framework for using existing pavement management data to calibrate the MEPDG performance models. The framework identifies the data collection and storage requirements for using data contained within a highway agencies pavement management system. The feasibility of the framework will be demonstrated using actual data from a highway agencies pavement management system.

Souliman, M. I., Mamlouk, M., Zapata, C. E., and Cary, C. E., "Data Collection to Support Implementation of the Mechanistic-Empirical Pavement Design Guide for County Roads," *Transportation Research Record* 2225, Transportation Research Board, National Research Council, Washington, DC, 2011. Evaluation and calibration of the Mechanistic-Empirical Pavement Design Guide (MEPDG) has been attempted by various agencies throughout the United States. Agencies interested in adopting the MEPDG procedure must prepare a practical implementation plan that fits local conditions. The first step in the implementation plan is collection of design input data and establishment of a database for inputs. A 3-year study was conducted at Arizona State University to establish a database to support MEPDG implementation for the Maricopa County, Arizona, Department of Transportation. The implementation program included testing of asphalt binders, hot-mix asphalt dynamic modulus, and unbound-materials resilient modulus; development of climatic weather stations and training material; and collection of traffic data. The collected information can be used to calibrate the MEPDG distress models to county conditions and verify such models. The input parameters can serve as a framework for similar highway agencies and help ensure the successful implementation of the MEPDG.

Texas A&M Research Foundation, *Quantifying the Influence of Geosynthetics on Pavement Performance*, Project 01-50, College Station, TX, research in progress. Geosynthetics are available in a wide range of forms and materials and are used in many applications. Geosynthetics are often used by highway agencies in conjunction with unbound base layers (i.e., within the layer or as a subgrade/base interface layer) as a means for enhancing the performance of flexible and rigid pavements. Although a great deal of research has been performed on the properties of these materials and their use in pavement structures, limited research has dealt with the methodologies of quantifying their influence on pavement performance in a manner that would allow incorporation into the mechanistic-empirical pavement design and analysis procedures. The AASHTO Interim Mechanistic-Empirical Pavement Design Guide Manual of Practice (MEPDG) developed under NCHRP Project 01-37A provides a methodology for the analysis and performance prediction of pavements. However, use of geosynthetics in pavement layers and their influence on distress models have not been addressed in the MEPDG. Procedures that quantify the influence of geosynthetics on pavement performance will help in determining the payoff obtained by using these materials and selecting the appropriate material for a specific application. Such information is not readily available. Therefore, research is needed to (1) evaluate those tests currently used for characterizing geosynthetics and, if necessary, identify new tests that relate to performance and (2) develop a methodology for quantifying the influence of geosynthetics on pavement performance for use in pavement design and analysis. This information can be incorporated into the MEPDG thus allowing a rational analysis and design procedure of flexible and rigid pavements in which geosynthetics are used in conjunction with unbound bases/subbases. The objective of this research is to develop a methodology for quantifying the influence of geosynthetics on pavement performance for use in pavement design and analysis. The methodology shall be consistent with the MEPDG framework to facilitate incorporation into the MEPDG. This research is concerned with the use of geosynthetics in conjunction with unbound base/subbase layers (i.e., within the layer or as a subgrade/base interface layer) for flexible and rigid pavements.

Thyagarajan, S., Muhunthan, B., Sivaneswaran, N., and Petros, K., "Efficient Simulation Techniques for Reliability Analysis of Flexible Pavements Using the Mechanistic-Empirical Pavement Design Guide," ***Journal of Transportation Engineering*****, Volume 137, Number 11, American Society of Civil Engineers, 2011.** Many sources of uncertainty are inherent in pavement design. These uncertainties must be incorporated systematically in a reliability analysis to compute their combined effects on the probability of failure of a given pavement structure. Monte Carlo simulation has been the technique of choice in the past to simulate the effects of uncertainties in input parameters on pavement distress and the resultant reliability analyses. The impractical computational time associated with a Monte Carlo scheme, however, has prompted the deferral of the implementation of similar techniques in the current Mechanistic-Empirical Pavement Design Guide (MEPDG). Instead, the reliability analysis implemented in the current MEPDG is performed on the basis of a simple assessment of the overall standard error of the predicted distress compared to the observed distress of the long-term pavement performance sections. It relies on a set of predetermined variability values obtained from a performance database instead of the site-specific input parameters that induce such uncertainty in distress predictions. Past efforts found that techniques (such as the Latin hypercube method) that require a substantially reduced number of simulations compared with Monte Carlo accuracy still suffered from the need for repeated execution of the MEDPG calculations. This study proposes to combine an efficient numerical scheme to conduct statistical simulations with the MEPDG calculations. It makes use of the concept of the representative linear elastic structure to minimize the number of repeated executions involved in simulations. The numerical scheme can be combined with any simulation technique of random variables to perform a reliability analysis of flexible pavements. The relative merits of Monte Carlo simulation, Latin hypercube simulation, and Rosenblueth's 2K+1 point-estimate method are compared. The simulations show that the Latin hypercube method is an efficient alternative to the computationally intensive Monte Carlo technique. On the other hand, although Rosenblueth's 2K+1 point-estimate method is much simpler, it is not capable of capturing the important attributes of the distribution of either input or output variables.

Vandenbossche, J. M, Mu, F., and Burnham, T. R., "Comparison of Measured vs. Predicted Performance of Jointed Plain Concrete Pavements Using the Mechanistic-Empirical Pavement Design Guide," ***International Journal of Pavement Engineering*****, Volume 12, Number 3, 2010.** This research evaluates the ability of the Mechanistic-Empirical Pavement Design Guide (MEPDG) to accurately predict the performance of jointed plain concrete pavements (JPCP). This is accomplished by comparing predicted performances with observed performances for the in-service mainline test cells at Mn/ROAD. These comparisons indicate that MEPDG performance predictions for JPCP are most accurate when the default (constant) built-in equivalent temperature difference of -5.5 degrees C is used instead of a site-dependent value. It appears that significant portions of the error of estimation can be explained by the sensitivity of the performance models to variability in hardened concrete properties (modulus of rupture, modulus of elasticity and coefficient of thermal expansion) and pavement structural features (slab thickness, joint spacing, subbase type and bond condition). Predictions of slab cracking were found to be highly sensitive to these parameters. In addition, the MEPDG cracking model seemed not to fit local cracking observations for the Minnesota test cells. New calibration factors are needed to more accurately predict Minnesota JPCP slab cracking. This study also included comparisons of predicted service lives for the Mn/ROAD test cells using different design methodologies and as-built input parameters. In most cases considered, the MEPDG predicted longer service lives than did the 1993 AASHTO procedure. The MEDPG also predicted longer service lives than the PCA procedure for the 5-year cells but shorter service lives for the 10-year cells. This infers that, when holding service life constant, the MEPDG generally results in thinner concrete pavement sections than the 1993 AASHTO procedure.

Vandenbossche, J. M., Mu, F., Gutierrez, J. J., and Sherwood, J., "An Evaluation of the Built-In Temperature Difference Input Parameter in the Jointed Plain Concrete Pavement Cracking Model of the Mechanistic-Empirical Pavement Design Guide," ***International Journal of Pavement Engineering*****, Volume 12, Number 3, 2010.** This paper evaluates the implementation of the built-in temperature difference input parameter in the Mechanistic-Empirical Pavement Design Guide (MEPDG) for the design of jointed plain concrete pavements (JPCPs). The pavement distress, in terms of transverse cracking, is expected to be minimised when the transient temperature difference is equal in magnitude to the built-in temperature difference but of the opposite sign. However, this study shows that a built-in temperature difference of -6.5 degrees C minimises the cracking prediction for JPCPs. This optimum value of -6.5 degrees C coincides with the default value in the MEPDG of -5.5 degrees C, which was established through the nationwide calibration. The cause of this phenomenon is further investigated by taking into account the traffic loading time, slab thickness, joint spacing and reversible shrinkage, but none of these factors are able to explain this anomaly. The results from this study indicate that the built-in gradient should not be an input but is merely a calibration constant. A comparison between predictions using the measured and default built-in temperature difference again supports that it is better characterised as a calibration constant.

Vandenbossche, J. M., Nassiri, S., Ramirez, L., and Sherwood, J. A., "Evaluating Continuously Reinforced Concrete Pavement Performance Models of Mechanistic-Empirical Pavement Design Guide," Paper 11-2611, Annual Meeting of the Transportation Research Board, National Research Council, Washington, DC, 2011. The reasonableness of the models utilized internally by the Mechanistic-Empirical Pavement Design Guide (MEPDG) to predict the performance of the continuously reinforced concrete pavements (CRCPs), was evaluated in this study through a comprehensive sensitivity analysis. This study focused on evaluating the punchout, crack width and spacing and load transfer efficiency (LTE) models through a factorial sensitivity study including variables such as Portland cement concrete (PCC) mechanical and thermal properties, CRCP design features, traffic and climate. The input matrix was defined in a way that each individual scenario would reflect a "real-world" design situation. While doing so, all the correlations and interactions between the parameters were also considered. Approximately 2,600 simulations of the MEPDG revealed that all the models were sensitive to the base type and the longitudinal steel content. The punchout and crack spacing prediction models were also sensitive to the PCC mixture. The interactions between the predicted distress and the performance indicators were investigated in regards to these significant variables. As a result, it was found out that, in contrary to the 1993 AASHTO Design Guide, the crack width must be below 0.02 in (0.5 mm) to maintain adequate performance. Additionally, based on the performance prediction equations within the MEPDG, the crack spacing should be less than 6 ft (1.8 m) to help ensure a crack width of less than 0.02 in (0.5 mm). This is another contradiction with the approach used by the 1993 Guide (the crack spacing is designed to be between 3.5 and 8 ft [1.0 and 2.4 m]).

Wang., K. C. P. and Li, Q., Pavement Smoothness Prediction Based on Fuzzy and Gray Theories, *Computer-Aided Civil and Infrastructure Engineering*, Volume 26, Number 1, 2011. Pavement smoothness has been recognized as one of the measures of pavement performance. In the Mechanistic-Empirical Pavement Design Guide (MEPDG), pavement smoothness indicated by the International Roughness Index (IRI) was predicted based on various distresses using traditional regression analysis approaches. Recognizing the limitations of linear regression methods, a gray theory-based technique was previously proposed by the authors for the development of pavement smoothness prediction models. In this article, instead of using the conventional least squares method to determine the coefficients for gray prediction models, fuzzy regression method is proposed to solve this gray problem. With pavement IRI and distresses data exported from the Long-Term Pavement Performance (LTPP) database, fuzzy and gray model (FGM)-based smoothness predictions are established using influencing factors similar to those in MEPDG. Based on the comparisons among results originated from MEPDG models, conventional GM models, FGM models, and actual LTPP data, it is shown that the gray theory-based prediction methods with fuzzy regression for estimating model coefficients provide promising results and are useful for modeling pavement performance.

Wojtkiewicz, S. F., Khazanovich, Gaurav, G., and Velasquez, R., "Probabilistic Numerical Simulation of Pavement Performance using MEPDG," *Road Materials and Pavement Design*, Volume 11, Number 2, 2010. It is widely acknowledged that accurate simulation of complex engineering systems, such as nuclear power reactors, modern weapon systems, and aircraft, requires probabilistic analysis due to inherent uncertainties in their models' parameters. The demand for and complexity of probabilistic analysis prompted Sandia National Laboratories to develop a versatile software toolkit, DAKOTA, adaptable to various engineering applications. Pavements are another example of a complex engineering system requiring probabilistic modeling due to the uncertain nature of most of the pavement performance models parameters, including traffic, climate, material properties, and pavement structure. The deterministic pavement performance models vary from simplistic empirical relationships to complex mechanistic-empirical computational algorithms. Due to DAKOTA's independence of choice of analysis tool, it is a natural candidate to perform probabilistic aspects of pavement performance prediction. The paper presents a software framework for probabilistic modeling of pavement performance, which combines deterministic performance prediction models from the MEPDG and probabilistic analysis tools from DAKOTA. The power of this approach is demonstrated by analyzing the effect of variability in the asphalt concr[\ete AC mix design on the variability of the pavement performance prediction.

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