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Development of Local Calibration Factors and Design Criteria Values for Mechanistic-Empirical Pavement Design

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This study compared the measured performance of asphalt and continuously reinforced concrete pavements (CRCP) from VDOT's Pavement Management System (PMS) records to the predicted performance in AASHTOware Pavement ME Design. Model coefficients in the software were adjusted to match the predicted asphalt pavement permanent deformation, asphalt bottom-up fatigue cracking, and CRCP punchout outputs to the measured values from PMS records. Values for reliability, design life inputs, and distress limits were identified as a starting point for VDOT to consider when using AASHTOware Pavement ME Design through consideration of national guidelines, existing VDOT standards, PMS rating formulas, typical pavement performance at time of overlay, and the data used for local calibration.

The model calibration coefficients and design requirement values recommended in this study can be used by VDOT with AASHTOware Pavement ME Design as a starting point to implement the software for design, which should allow for more optimized pavement structures and improve the long-term performance of pavements in Virginia.

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FINAL REPORT

DEVELOPMENT OF LOCAL CALIBRATION FACTORS AND DESIGN CRITERIA VALUES FOR MECHANISTIC-EMPIRICAL PAVEMENT DESIGN

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ABSTRACT

A mechanistic-empirical (ME) pavement design procedure allows for analyzing and selecting pavement structures based on predicted distress progression resulting from stresses and strains within the pavement over its design life. The Virginia Department of Transportation (VDOT) has been working toward implementing ME design by characterizing traffic and materials inputs, training with the models and design software, and analyzing current pavement designs in AASHTOware Pavement ME Design software.

This study compared the measured performance of asphalt and continuously reinforced concrete pavements (CRCP) from VDOT's Pavement Management System (PMS) records to the predicted performance in AASHTOware Pavement ME Design. Model coefficients in the software were adjusted to match the predicted asphalt pavement permanent deformation, asphalt bottom-up fatigue cracking, and CRCP punchout outputs to the measured values from PMS records. Values for reliability, design life inputs, and distress limits were identified as a starting point for VDOT to consider when using AASHTOware Pavement ME Design through consideration of national guidelines, existing VDOT standards, PMS rating formulas, typical pavement performance at time of overlay, and the data used for local calibration.

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INTRODUCTION

The Virginia Department of Transportation (VDOT) maintains a roadway network of more than 126,000 lane-miles. With this large roadway network and a limited budget, it is critical that appropriate pavement structures are constructed that can efficiently withstand traffic loading and weathering effects over the design life.

VDOT's current pavement design procedure (VDOT, 2008) for all new and rehabilitated pavements is based on the 1993 American Association of State Highway and Transportation Officials (AASHTO) *Guide for Design of Pavement Structures* (hereinafter 1993 AASHTO design guide) (AASHTO, 1993). This empirical design procedure is based on the results of the AASHO Road Test of the late 1950s in which the designed pavement thickness was found to be primarily a function of the anticipated service life, the serviceability of the pavement, and the number of equivalent traffic loads applied.

AASHTO has since released the *Guide for the Mechanistic-Empirical Design of New & Rehabilitated Pavement Structures* (MEPDG) that uses the calculated mechanistic response combined with empirical results from pavement test sections in the Long-Term Pavement Performance (LTPP) Program to predict the performance of pavement structures (Applied Research Associates, Inc. [ARA, Inc.], 2004). The mechanistic-empirical (ME) design process presents a major change in pavement design from the 1993 AASHTO design guide. It calculates pavement responses (stresses, strains, and deflections) based on inputs such as traffic, climate, and materials parameters to predict the pavement damage over time for both asphalt and concrete (Portland cement concrete) pavements. After this step, transfer functions relate computed pavement responses (e.g., pavement damage) to observed pavement distresses. The ME design procedure will be able to improve upon the pavement design methodology from the 1993 AASHTO design guide because of the mechanistic component; further, the pavement test sections more closely resemble the pavements being designed and constructed in today's environment than those constructed for the AASHO Road Test. This ME design procedure was incorporated into AASHTOware Pavement ME Design software (hereinafter Pavement ME Design) to provide a functional tool for developing pavement designs using ME principles.

VDOT currently uses Pavement ME Design to analyze pavement designs developed in accordance with the 1993 AASHTO design guide, and no adjustment is being made to the pavement structure based on the ME outcome.

VDOT developed a plan to implement ME pavement design procedures in 2007 that outlined steps to provide a functional version for VDOT (VDOT, 2007). The tasks involved developing traffic inputs, characterizing material properties, calibrating and validating the models, and providing training. Researchers at the Virginia Center for Transportation Innovation and Research (VCTIR) have conducted many studies that helped identify traffic data collection needs and develop traffic loading inputs that can be used for design (Cottrell and Kweon, 2011; Cottrell et al., 2003; Smith and Diefenderfer, 2010). Other studies considered various asphalt mixtures and existing pavement to initiate a catalog of asphalt material properties for use with ME design (Apeagyei and Diefenderfer, 2011; Diefenderfer, 2010; Flintsch et al., 2007; Loulizi et al., 2006). Studies also characterized unbound and subgrade materials and identified test methods to correlate with resilient modulus (Hossain, 2008; Hossain, 2010; Hossain and Kim, 2014). Based on these studies, ongoing research, discussions with experts, and testing in Pavement ME Design, a draft manual for using Pavement ME Design that details how designers should enter project information was developed (VDOT, 2013a).

One of the final tasks in implementing the ME design process is to perform validation, calibration, and verification of the models to substantiate that the predicted pavement performance matches what is observed in Virginia for the distress and ride quality values. In addition, requirement values for pavement design should be reviewed so that pavement structures are expected to achieve an acceptable level of performance throughout their design life with a cost-effective design. AASHTO (2010) highly recommends that each agency conduct an analysis of the results of Pavement ME Design to determine if the nationally calibrated performance models accurately predict field performance, as the performance prediction models used in ME design may require calibration to local conditions.

PURPOSE AND SCOPE

The purpose of this study was to perform local calibration of the distress models included in Pavement ME Design so that they would better match VDOT's observed performance. In addition, preliminary values for performance targets, reliability, and design life criteria were to be developed to provide a full set of inputs for VDOT to use to develop pavement designs using Pavement ME Design.

The study included a review of both asphalt and concrete distress prediction models for pavements in Virginia to develop a set of calibration factors applicable for the entire state. The calibration was based on a comparison of predicted permanent deformation, cracking, punchouts, and International Roughness Index (IRI) values and the measured values from VDOT's Pavement Management System (PMS). It was expected that further refinement of the calibration coefficients might be necessary and that model improvement would continue beyond the initial implementation.

METHODS

Three tasks were conducted to fulfill the purpose of the study:

1. A literature review was conducted to document the experiences of other transportation agencies in calibrating and implementing ME design procedures.

2. Local calibration was performed to remove bias and assess standard error of distress models for asphalt and concrete pavements.

3. Suggested values for design requirements were developed.

Literature Review

The literature search was conducted by searching various databases such as TRID, the Catalog of Transportation Libraries (TLCat), the Catalog of Worldwide Libraries (WorldCat), and the Transportation Research Board's Research in Progress (RiP) and Research Needs Statements (RNS) databases.

Local Calibration of Distress Models

The local calibration process for this study was similar to the procedure outlined by AASHTO in *Guide for Local Calibration of the Mechanistic-Empirical Pavement Design Guide* (hereinafter AASHTO local calibration guide) (AASHTO, 2010). The guide offers an 11-step procedure for local calibration as follows:

1. Select input level for each parameter.
2. Develop local experimental plan and sampling template.
3. Estimate sample size for specific distress prediction models.
4. Select roadway segments.
5. Extract and evaluate distress and project data.
6. Conduct field investigations.
7. Assess local bias.
8. Eliminate local bias of distress and IRI prediction models.
9. Assess the standard error of the estimate.
10. Reduce the standard error of the estimate.
11. Interpret the results.

Select Input Level for Each Parameter

VDOT's *Pavement ME User Manual—Draft* (VDOT, 2013a) describes VDOT's current procedure for inputting project details to perform pavement analysis using Pavement ME Design. Material, subgrade, traffic, and climate inputs used in the local calibration are based on these procedures to mimic what would be used in design.

Select Roadway Segments

VDOT began the local calibration effort in 2009, prior to the publication of the AASHTO calibration guide, by identifying test sections to use for calibration. Although a detailed sampling template and minimum sample size requirements were not established, VDOT recognized that a large, varied sample of projects would be necessary to provide a representative account for pavement performance in Virginia. The initial goal was to identify five asphalt pavement sites from each VDOT district that were built after 1999 and that were more than 0.5 mi long with more than 8 in of asphalt.

PMS data were used to identify concrete pavement sites for calibration of concrete pavement models. Sites with both continuously reinforced concrete pavements (CRCP) and jointed plain concrete pavements (JPCP) were sought; projects with construction dates after 1985 with a minimum section length of 0.5 mi were included.

Extract and Evaluate Pavement Distress and Project Data

The project information for each calibration site required two aspects: field performance records and project details. The field performance records were extracted from VDOT's PMS network level distress data. VDOT has automated distress data available beginning in 2007 at yearly intervals for all interstate and primary roadways. Distress data are measured at 0.1-mi intervals; the distresses at each interval within the project section were averaged to obtain the average distress for each site per year.

For asphalt pavement distress data, the rutting depth (inches), fatigue cracking—labeled as alligator cracking in PMS (square feet, three severity levels), and IRI (inches/mile) were used in calibration. Longitudinal cracks are recorded in PMS; however, these cracks are defined as outside the wheelpath and different from the longitudinal cracks predicted in Pavement ME Design that are assumed to be loading induced from the top of the pavement. Instead, the low severity (Level 1) fatigue cracks were assumed to be longitudinal cracks in Pavement ME Design predictions, and medium and high severity (Levels 2 and 3) alligator cracks were matched with the Pavement ME Design fatigue cracking predictions. The AASHTO local calibration guide suggests combining cracking types if the location where cracking initiated is not known and adjusting the bottom-up fatigue cracking model to fit the data (AASHTO, 2010).

CRCP distresses recorded in PMS include punchouts (count and square feet), cluster cracks (count and square feet, two severity levels), concrete patches (square feet, three severity levels), asphalt concrete patches (square feet), and IRI (inches/mile). The main distress that is predicted for CRCP in Pavement ME Design is punchouts (Pavement ME Design also classifies cluster cracks as punchouts). The measured area of punchouts was related to the number of punchouts by assuming the area of each punchout to be 25 ft^2 based on the typical lane width and definitions of crack spacing for punchouts (Miller and Bellinger, 2003). The number of punchouts from area distress measurement was used because it appeared more consistent year to year than the PMS count of punchouts. Some of the CRCP were known to have premature deterioration attributable to the method used to place the reinforcement. These sections had patching to repair failures soon after construction but still remain in service; these sites were included, and patched areas were disregarded.

The project-specific data entered into Pavement ME Design to produce predicted distresses were collected from a combination of sources. Asphalt pavement structure information including layer types, layer thicknesses, and year of construction information was provided by VDOT district materials personnel. The available subgrade information varied by project; when available, the subgrade classification was combined with records of resilient modulus testing of similar local materials. Otherwise, subgrade properties were taken from records of typical materials encountered in the area on recent projects with resilient modulus test data. Concrete pavement structure information, including base type, depth of concrete, and shoulder type, was obtained from construction records in PMS and current pavement images from pavement management; all CRCP were modeled with 0.7% steel at mid-depth.

Traffic count records were averaged at each section beginning with the year of construction to obtain the average daily traffic (ADT) over the analysis period. A zero growth rate was used for all vehicle classes because average ADT values already accounted for changes in traffic volume over the period. The percent truck traffic was selected from the design year to determine the average annual daily truck traffic (AADTT) for input into Pavement ME Design.

A single weather station was selected near the project location. Statewide average values were used for asphalt mixture properties (by mixture type), concrete material properties, aggregate base properties, vehicle class distribution, axle load spectra, and axles per truck in accordance with VDOT's *Pavement ME User Manual—Draft* (VDOT, 2013a). Other inputs were left at national default values.

Checks were performed on the distress and construction records to remove data points that seemed unreasonable. Project sites that indicated pavement layers with zero thickness were removed if the appropriate pavement structure could not be ascertained. PMS data were not considered at sites after rehabilitation was performed. The year of rehabilitation was identified for asphalt sites when the PMS data showed an improvement in the Critical Condition Index (CCI) or IRI of 10% or greater. For concrete pavement sites with rehabilitation by asphalt overlay, PMS data would no longer show distress rating criteria required for concrete-surfaced pavements. Concrete pavements with significant patching rehabilitation were identified by a 10% or greater CCI improvement that withstood two consecutive years; this was done because the concrete pavement CCI data showed high year-to-year variability that did not always represent rehabilitation. Data on sites prior to rehabilitation were still included for use in calibration.

Assess and Reduce Local Bias and Standard Error of the Estimate

The main parameters to evaluate the fit of the distress or IRI prediction models are the bias and standard error of the estimate (S_e). These terms are defined as the systematic offset between predicted and observed values and the variability between the predicted and measured values, respectively (AASHTO, 2010). The residual error represents the difference between the measured and predicted values for each data point; the bias was calculated as the average of the individual residual errors, and the S_e was the standard deviation of the residual error. Another way used to evaluate the residual error is to compare the S_e to the standard deviation of the measured distress (S_y); the S_e/S_y ratio should decrease with local calibration. These values were calculated by entering the predicted and measured performance in a spreadsheet. Adjustments to the calibration coefficients for each model were made using a generalized reduced gradient non-linear tool to eliminate or reduce the bias and improve the S_e.

The AASHTO local calibration guide notes the importance of both the calibration and validation steps being a part of the local calibration effort; calibration is the process to minimize the residual error (difference between observed and predicted values), and validation is the process of applying the model to data that were not used in calibration to ensure the model statistics are similar to those from calibration, confirming the robustness of the model. If the model statistics are similar, the calibration and validation datasets are recombined to refine the model coefficients based on the entire available set of data (AASHTO, 2010). One of two procedures may be used for validation: (1) withhold 20% of the sites from the calibration data (called split-sample validation), or (2) remove a single site from the calibration data (called the jackknife procedure) (AASHTO, 2010).

The total rutting and bottom-up fatigue cracking were the primary models of interest for asphalt pavements, with IRI also being considered. Pavement ME Design also includes models to predict top-down fatigue cracking, thermal cracking, and chemically stabilized fatigue; these models were not considered for calibration at this time because either the models have revisions pending based on research under the auspices of the National Cooperative Highway Research Program (NCHRP) to improve the model or the distresses lack sensitivity in predictions for Virginia sections. CRCP pavement calibrations focused on the punchout and IRI predictions. Pavement ME Design has models for JPCP percent of cracked slabs and faulting distress, which were not considered because of the lack of sites with that pavement type in Virginia.

The calibration coefficients considered for adjustment to improve the model fit are shown in Table 1.

Table 1. Recommendation From AASHTO (2010) for Transfer Function Calibration Coefficients to Be Adjusted for Eliminating Bias and Reducing the Standard Error

Distress		Eliminate Bias	Reduce Standard Error
Asphalt Pavements			
Total rutting	Unbound materials and asphalt layers	β_{s1} or β_{r1}	β_{r2}, β_{r3}
Load-related cracking	Alligator cracking	C_2 or β_{f1}	β_{f2}, β_{f3}, and C_1
	Longitudinal cracking	C_2 or β_{f1}	β_{f2}, β_{f3}, and C_1
	Semi-rigid pavements	C_2 or β_{c1}	C_1, C_2, C_4
Non–load-related cracking	Transverse cracking	β_{f3}	β_{f3}
IRI		C_4	C_1, C_2, C_3
Concrete Pavements			
JPCP faulting		C_1	C_1
JPCP fatigue cracking		C_1 or C_4	C_2, C_5
CRCP punchouts	Fatigue	C_1	C_2
	Punchouts	C_3	C_4, C_5
	Crack widths	C_6	C_6
IRI	JPCP	C_4	C_1
	CRCP	C_4	C_1, C_2

IRI = International Roughness Index; JPCP = jointed plain concrete pavement; CRCP = continuously reinforced concrete pavement.

The distress and IRI models with the coefficients from Table 1 are shown here; variable definitions and other details for the models can be found in the AASHTO MEPDG Manual of Practice (AASHTO, 2008).

$$\text{Asphalt rutting} = \beta_{r1} k_z 10^{k_{r1}} n^{k_{r2}\beta_{r2}} T^{k_{r3}\beta_{r3}}$$

$$\text{Base/Subgrade rutting} = \beta_{s1} k_{s1} \varepsilon_v \left(\frac{\varepsilon_0}{\varepsilon_r}\right) e^{-\left(\frac{p}{n}\right)^{\beta}}$$

$$N_{f-HMA} = k_{f1}(C)(C_H)\beta_{f1}(\varepsilon_t)^{k_{f2}\beta_{f2}}(E_{HMA})^{k_{f3}\beta_{f3}}$$

$$\text{Fatigue cracking}_{\text{Bottom}} = \left(\frac{1}{60}\right)\left(\frac{C_4}{1 + e^{\left(C_1 C_1^* + C_2 C_2^* \text{Log}(DI_{\text{Bottom}} * 100)\right)}}\right)$$

$$IRI_{\text{Flexible}} = IRI_0 + C_1 * RD + C_2 * FC_{\text{total}} + C_3 * TC + C_4 * SF$$

$$\text{Punchouts} = \frac{C_3}{1 + C_4 * DI_{PO}^{C_5}}$$

$$IRI_{CRCP} = IRI_I + C_1 * PO + C_2 * SF$$

Development of Suggested Values for Design Requirements

Another key area of concern for VDOT prior to implementing ME design procedures is selecting design requirement properties—specifically design life, reliability level, and target performance values. These values are an important component of the transition from analysis of pavement structures with Pavement ME Design to development of pavement designs that can efficiently balance cost and pavement performance. The decision on selecting these values reflected a few different perspectives:

- national guidelines
- previous VDOT design standards
- data from end-of-service pavements in Virginia
- relationships between distress in serviceability used in PMS
- values in local calibration site data
- experience of VDOT district and field personnel.

Example pavement designs were developed for hypothetical scenarios to demonstrate how the revised local calibration coefficients and design criteria values relate to a pavement structure. Three design scenarios were considered with low, medium, and high traffic levels combined with test records of subgrade properties and climate data from different locations in Virginia.

RESULTS AND DISCUSSION

Literature Review

A research report from the Federal Highway Administration (FHWA) on the use of PMS data when ME distress models are calibrated identified some recommendations for agencies to consider when using these data to calibrate locally (FHWA, 2010). The first recommendation was to evaluate the measured distress data and ensure they are consistent with the distress definitions when Pavement ME Design was developed; further, the material, traffic, and climate parameters should be reviewed to determine changes from the default inputs that are necessary to model the project sections accurately. Some of the other challenges identified were the lack of distress or pavement material information, limited ranges of distress values and pavement service life, and fewer sites than needed for statistically meaningful calibration.

In preparation for local calibration of ME distress models, the Georgia Department of Transportation (DOT) commissioned a synthesis study of local calibration activities being conducted by state highway agencies (Von Quintus et al., 2013). The study showed that many states were working toward calibration by focusing on building design input libraries for material and traffic inputs. Further, the study showed that states that performed local calibration of asphalt pavements consistently found that the global predictions from the ME design method overpredicted rutting and developed local calibration factors to improve the prediction. The local calibration of the asphalt fatigue cracking transfer function showed more variability than the rutting model, but it reasonably estimated the measured levels of cracking over a broad range of pavement structures. One state, Arizona, had performed local calibration of CRCP punchout and IRI models and found the global calibration coefficients for the CRCP models to be reasonable. Utah, Colorado, and Wyoming found the global calibration coefficients for JPCP transverse cracking and joint faulting to be acceptable unbiased predictions when correct materials inputs were used; Arizona found the JPCP global distress models to match field observations reasonably but made slight adjustments to reduce the error.

The Missouri DOT also commissioned a local calibration of ME models (ARA, Inc., 2009). To evaluate the models, their study used LTPP data combined with sections from the state PMS data split into 500-ft-long sections with both statistical and non-statistical approaches. The statistical approach considered the R^2, standard error of the estimate, and bias; hypothesis testing on the model intercept being equal to 0, and the model slope being equal to 1; and a paired *t*-test of measured and predicted distress/IRI values. A non-statistical approach was used for some models that showed little or no measured distress (i.e., asphalt pavement bottom-up fatigue cracking and JPCP joint faulting). The study found the default models for predicting bottom-up fatigue cracking of asphalt pavements and IRI predictions of asphalt overlays on JPCP to be reasonable, but transverse cracking, total rutting, and IRI prediction for asphalt pavements were all acceptable after local calibration of the model coefficients. The Missouri JPCP investigation showed that the national models for slab cracking and joint faulting were adequate but recalibrated the IRI prediction model.

A report documenting the implementation of the ME design procedure by the Colorado DOT covered identifying LTPP and PMS sites for calibration, developing traffic and material inputs, locally calibrating models, analyzing sensitivity, and comparing design outputs to previous pavement design results (Mallela et al., 2013). The distress model validation compared pavement thicknesses on projects determined by the locally calibrated ME models to Colorado's previous design strategy using the 1993/1998 AASHTO design procedures (AASHTO, 1993; AASHTO, 1998). The comparison showed good agreement between the two design methods, with all the example projects showing thickness values within 1 in. A comparison of pavement designs from the Indiana DOT showed that thicknesses developed from the ME design procedure were less than those developed from the 1993 AASHTO procedure for all examples of both asphalt and concrete pavements; this thickness reduction translated to an estimated cost savings of more than $10 million (Nantung, 2010).

In a recent NCHRP synthesis project, Pierce and McGovern (2014) surveyed highway transportation agencies to collect information on implementing the MEPDG. Of the 57 agencies surveyed, 3 had already implemented the MEPDG at the time of the survey; 8 agencies reported local calibration of at least some of the asphalt and concrete models (Arizona, Colorado, Florida, Indiana, Missouri, North Dakota, and Oregon). Agencies reported threshold levels for design that showed varying distress or IRI performance levels, design lives, and reliability levels based on the agency; functional classification; traffic level; or distress type considered. These values were developed from pavement management data, engineering judgment by pavement managers and designers, sensitivity analysis, previous design standards, and ranges provided in Pavement ME Design. Agencies that had conducted local calibration work identified the need to reanalyze when further data were available; a database to maintain all of the data necessary for calibrating ME models was recommended.

Local Calibration of Distress Models

The asphalt pavement sites considered for local calibration are listed in Table 2. These locations cover eight of the nine VDOT districts; no sites were identified in the Hampton Roads District because many of the roads there that have had significant construction involve overlays on concrete pavements, which were not included in this calibration. The Site ID tag is an identifier that was used to keep track of the projects during calibration. As may be seen, some asphalt sites were withheld from the calibration dataset to be used for validation. Some sites are listed as being both directions, and others are listed as being single direction and may contain parallel routes as separate numbers; directions were combined when the pavement structure and year of construction were the same for both directions; other cases may have had different years of construction and therefore were separate sites.

Table 3 shows the CRCP sites used for concrete pavement calibration. A limited number of concrete pavement sections were available for local calibration, especially for JPCP with only four projects identified. Therefore, the JPCP models were not reviewed in this study and the concrete pavement calibration was focused on CRCP. A jackknife approach (n – 1) was used for validation instead of split-sampling to achieve an independent check on the model; therefore, no concrete sites were marked as validation sites.

Table 2. Asphalt Pavement Calibration Sites

Site ID	County	Route Type	Route	Direction	From Mile Post	To Mile Post	Length	Year Paved
Br-1	Lee	US	58	East	22.03	25.27	3.24	2002
Br-2	Washington	SR	91	North	14.6	16.2	1.6	2002
Br-3	Grayson	US	58	East	37.137	41.2	4.063	2000
Br-4[a]	Grayson	US	58	East	41.2	44.6	3.4	2008
Br-5	Russell	US	19	North	8.01	13.77	5.76	2000
Sa-1	Pulaski	SR	100	North	19.81	22.54	2.73	2000
Sa-2	Montgomery	IS	81	North	9.5	14.17	4.67	2001
Sa-3	Montgomery	IS	81	South	9.5	14.17	4.67	2001
Sa-4[a]	Patrick	US	58	East	0.99	3.31	2.32	2005
Sa-5	Patrick	US	58	East	16.84	18	1.16	2002
Ly-1	Pittsylvania	US	58	Both	9.05	16.98	7.93	2004
Ly-2	Pittsylvania	US	29	North	7.72	15.02	7.3	2004
Ly-3	Pittsylvania	SR	41	Both	0	2.8	2.8	2003
Ly-4[a]	Amherst	US	29	Both	11.22	12.56	1.34	2003
Ly-5	Halifax	US	360	Both	20.01	21.3	1.29	2006
Ly-6	Amherst	SR	130	Both	23.87	25.21	1.34	2002
Ri-1	Goochland	SR	288	North	0	6.04	6.04	2004
Ri-2	Hanover	SR	30	East	1.25	2.2	0.95	2007
Ri-3	Hanover	SR	30	West	1.25	2.2	0.95	2007
Ri-4[a]	Henrico	SR	895	East	0	7.88	7.88	2002
Ri-5	Henrico	SR	895	West	0	7.88	7.88	2003
Ri-6	Mecklenburg	US	58	East	6.95	10.22	3.27	2005
Ri-7	Mecklenburg	US	58	West	6.95	10.22	3.27	2005
Ri-8	Goochland	IS	64	West	11.99	15.9	3.91	1992
Ri-9	Goochland	IS	64	East	25.1	26.34	1.24	1993
Ri-10	Goochland	IS	64	East	21.24	23.07	1.83	2003
Fr-1	Stafford	US	1	North	3	3.96	0.96	2005
Fr-2	Spotsylvania	SR	208	North	19.8	21.71	1.91	2008
Fr-3	Spotsylvania	SR	208	South	19.8	21.71	1.91	2008
Fr-4[a]	Caroline	SR	30	East	0	1.15	1.15	2007
Fr-5	Stafford	US	17	North	6.96	8.86	1.9	1992
Cu-1	Culpeper	SR	299	North	0	0.62	0.62	1999
Cu-2	Culpeper	SR	299	South	0	0.62	0.62	1999
Cu-3	Culpeper	SR	3	East	9.358	10.932	1.574	1999
Cu-4[a]	Fauquier	SR	28	North	4.16	5.04	0.88	2000
Cu-5	Albemarle	US	29	North	3.86	4.29	0.43	2001
Cu-6	Greene	US	33	East	8.066	9.47	1.404	2001
Cu-7	Greene	US	33	East	9.47	6.6	2.87	2001
Cu-8	Culpeper	SR	3	East	1.51	3.19	1.68	2007
Cu-9	Louisa/Fluvanna	US	15	North	0.29	-0.25	0.54	2004
Cu-10	Fauquier	IS	66	East	14.66	20.96	6.3	2004
Cu-11	Fauquier	IS	66	West	14.83	20.45	5.62	2003
Cu-12	Fauquier	US	15	South	11.44	12.23	0.79	2005
St-1	Rockbridge	IS	81	North	4.86	5.66	0.8	2001
St-2	Rockbridge	IS	81	South	4.86	5.66	0.8	2003
St-3	Alleghany	IS	64	East	16.47	18.67	2.2	2004
St-4[a]	Alleghany	IS	64	West	16.47	18.67	2.2	2003
NO-1	Fairfax	SR	642	East	2.105	2.84	0.735	2006
NO-3	Fairfax	SR	608	North	2.64	4.44	1.8	2006
NO-4[a]	Prince William	SR	234	North	8.77	11.92	3.15	2003
NO-5	Prince William	SR	234	North	2.48	6	3.52	2006
NO-6	Fairfax	SR	611	North	9.31	10.97	1.66	2002

US = U.S. route; SR = state route; IS = interstate route.
[a] Validation site.

Table 3. Concrete Pavement Calibration Sites

Site ID	County	Route Type	Route	Direction	From Mile Post	To Mile Post	Length	Year Paved
PCC-2	York	IS	64	Both	28.820	31.250	2.43	2006
PCC-5	Chesterfield	IS	295	East	1.250	2.110	0.86	1992
PCC-6	Henrico	IS	295	Both	25.080	37.830	12.75	1990
PCC-7	Prince George	IS	295	Both	0.000	5.490	5.49	1992
PCC-8	Prince George	IS	295	Both	5.490	12.330	6.84	1992
PCC-9	Norfolk	IS	564	Both	1.360	2.110	0.75	1991
PCC-10	Nansemond	IS	664	East	2.210	5.630	3.42	1991
PCC-12	Norfolk	IS	664	East	0.000	3.920	3.92	1991
PCC-13	York	IS	664	Both	1.120	4.030	2.91	1987
PCC-14	Chesterfield	SR	76	Both	0.000	9.760	9.76	1988
PCC-15	Nansemond	SR	164	West	0.200	1.240	1.04	1991
PCC-16	Chesterfield	SR	288	North	0.000	0.630	0.63	1989
PCC-17	Chesterfield	SR	288	South	0.000	15.900	15.9	1988
PCC-18	Chesterfield	SR	288	Both	16.110	22.620	6.51	2004
PCC-19	Amherst	US	29	Both	0.460	11.680	11.22	2005
PCC-20	Greensville	US	58	East	9.790	11.290	1.5	1988
PCC-21	Greensville	US	58	East	11.290	4.450	6.84	1990

US = U.S. route; SR = state route; IS = interstate route.

Once the projects were selected, an experimental testing matrix was completed to show the range of base types and pavement thicknesses covered by the calibration sites. This sampling matrix is shown in Table 4 for asphalt pavement sites and in Table 5 for concrete pavement sites showing the type of base layer and the thickness of the asphalt or concrete. Some of the pavement sections also included open-graded drainage layers, large stone base layers, or stabilized subgrade layers that are not shown in the testing matrix tables but were accounted for in modeling the pavement structures.

Table 4. Testing Matrix for Asphalt Pavement Sites

Base Type	Asphalt Thickness (in)				
	5-7	7.1-9	9.1-11	11.1-13	>13
Graded aggregate base	3	8	6	8	5
Cement-treated aggregate	2	5	7	2	0
Select materials	0	1	2	2	0
Stabilized subgrade	0	0	2	0	0
Full-depth asphalt	0	0	0	2	0

Table 5. Testing Matrix for Portland Concrete Pavement Sites

Base Type	Concrete Thickness (in)			
	8	9	10	12
Graded aggregate base	5	3	1	0
Cement-treated aggregate	0	0	0	1
Select materials	0	4	0	0
Stabilized subgrade	0	2	0	1

Asphalt Pavement Rutting Calibration

The first model considered in VDOT's local calibration was the predicted rutting on asphalt pavements. A comparison of the measured and predicted values of total rutting when the default global calibration coefficients in Pavement ME Design were used is shown in Figure 1. This figure shows an overprediction in the amount of rutting, with most of the points falling above the line of equality.

Statistics for the global and local calibration data are shown in Table 6. The global calibration values showed a large bias, with the Pavement ME Design models predicting more than 0.2 in greater rutting than was measured in the field on average. In addition, the standard error of the estimate indicated a large amount of variability in the differences between measured and predicted rutting values.

The model intercept factors for asphalt and subgrade rutting were adjusted with the use of Solver to meet the constraints of a minimum standard error of the estimate and zero bias for total rutting of the calibration dataset. Table 6 shows the calibration coefficients that were obtained and indicators of how well the revised model fit the data for the calibration and validation datasets. The AASHTO local calibration guide suggests that a reasonable limit for the standard error of the estimate is 0.10 in, based on the typical amounts of rutting that were encountered nationally (AASHTO, 2010). Both datasets showed little or no bias and acceptable standard error values. Although the validation dataset showed statistically significant results at an alpha level of 0.05 for the paired *t*-test, which suggests the predicted rutting did not match the measured rutting, the researchers decided to accept the calibration since the validation still showed a low bias and a S_e lower than that of the calibration dataset. Therefore, the model was recalibrated with the combined calibration and validation datasets to refine the model coefficients.

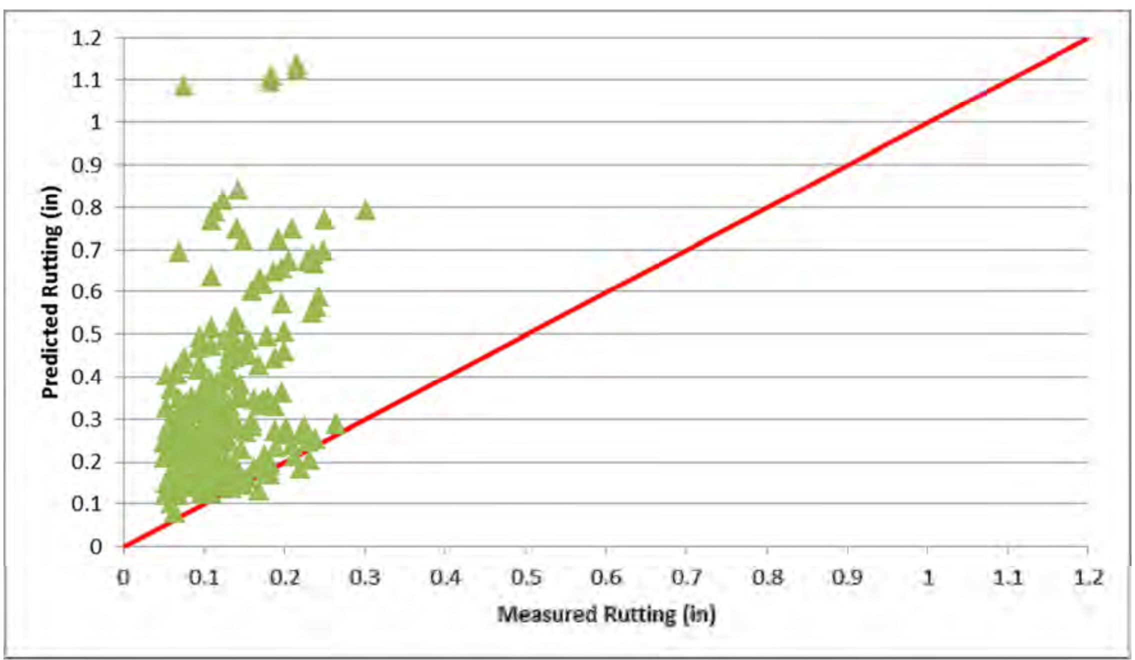

Figure 1. Asphalt Pavement Rutting Comparison With Global Calibration Coefficients

Table 6. Rutting Local Calibration Results

Statistic	Global Calibration	Local Calibration	Validation	Combined
Count	236	198	38	236
Bias, in	-0.214	0.000	0.023	0.000
S_e, in	0.183	0.079	0.033	0.076
R^2, %	16.5	22.2	42.8	23.7
p-value (paired *t*-test)	0.00	1.00	0.0001	1.00
Regression slope	1.546	0.792	0.492	0.812
p-value (slope)	0.017	0.050	0.000	0.050
Regression intercept	0.144	0.027	0.033	0.024
p-value (intercept)	0.000	0.069	0.005	0.069
S_e/S_y	3.52	1.50	0.76	1.47
β_{r1}	1.000	0.664	0.664	0.687[a]
β_{s1}–fine subgrade	1.000	0.151	0.151	0.153[a]
β_{s1}–granular subgrade	1.000	0.151	0.151	0.153[a]

[a] Coefficients used to generate Figure 2.

All of the data points were recombined to refine the model coefficients after the validation model showed a reasonable fit. The resulting calibration coefficients were used to graph the predicted and measured rutting value comparison with local calibration in Figure 2. One concern with regard to the local calibration models is the poor fit shown by the coefficient of determination and S_e/S_y; however, this may be partially attributable to the fairly small range over which measured rutting values were recorded for the sites. Despite a slope that is statistically similar to unity for the calibrated model, care should be taken when predicted distress values, at 50% reliability, that are above the range of values used for calibration are considered.

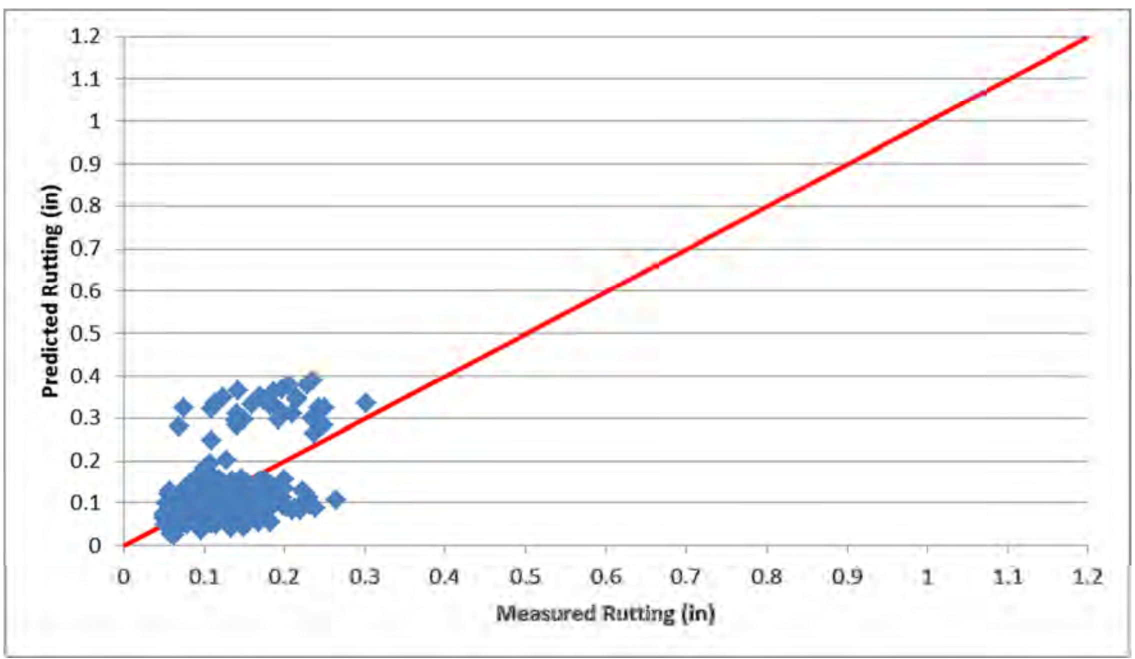

Figure 2. Asphalt Pavement Rutting With Locally Calibrated Coefficients

Asphalt Pavement Bottom-up Fatigue Cracking Calibration

The next consideration for asphalt pavements was calibration of the fatigue cracking models. A graph of the measured (combined severity Level 2 and Level 3) alligator cracking and predicted bottom-up fatigue distress using the global calibration parameters is shown in Figure 3. As shown in the figure, none of the sites had very high levels of predicted fatigue distress, with the maximum value being 1.7% cracking. The measured fatigue cracking values were also fairly low, with 265 data points (78% of the measurements) less than 2%.

The adjustment of calibration coefficients was performed by varying the β_{f1} value for determining fatigue damage and both C_1 and C_2 used in the transfer function for bottom-up cracking prediction. Initial calibration attempts through Solver optimization found calibration coefficients that effectively would predict the same amount of cracking for all pavement designs. To ensure that Pavement ME Design would still predict a range of predicted distress values depending on the situation, the parameters C_1 and C_2 were set to be equal and data points with greater than 2% measured cracking were used first to set the starting point for readjusting the calibration coefficients. Table 7 shows the fit parameters for global and local calibration coefficients adjusted for either all the data or the subset with greater than 2% cracking. For both cases, the validation results showed acceptable results, with a paired *t*-test showing a *p*-value greater than 0.05, indicating the difference between predicted and measured values would not be considered statistically significant.

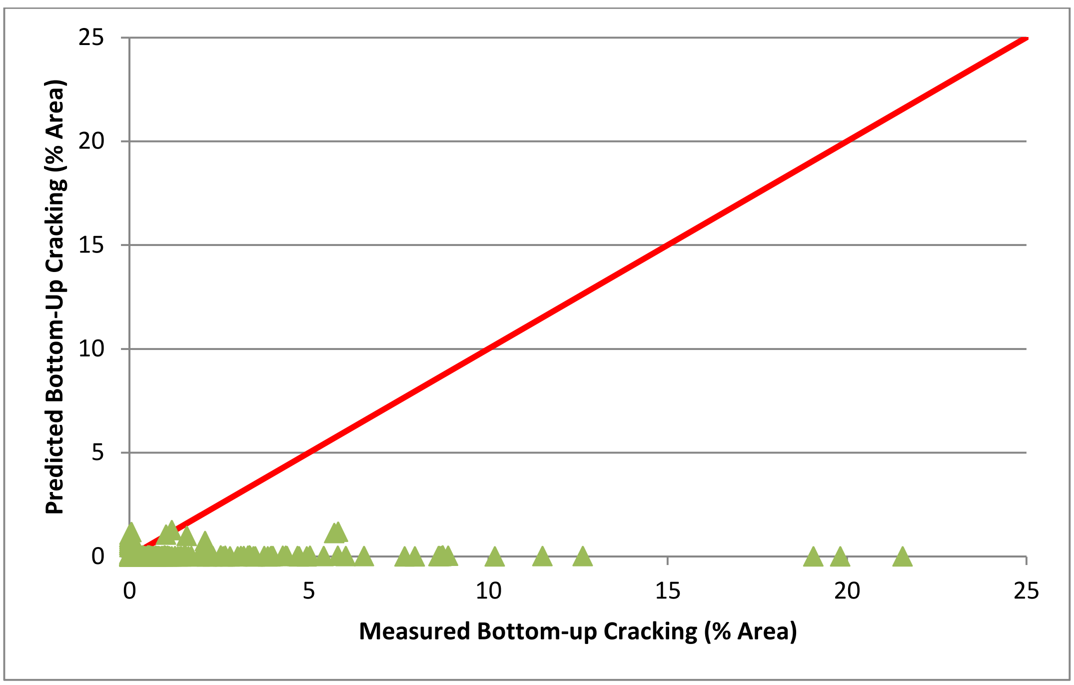

Figure 3. Asphalt Pavement Bottom-Up Cracking Comparison With Global Calibration Coefficients

Table 7. Bottom-Up Fatigue Cracking Local Calibration Results

Statistic	All Data Points				Measured Cracking > 2%			
	Global Calibration	Local Calibration	Validation	Combined	Global Calibration	Local Calibration	Validation	Combined
Count	233	195	38	233	51	44	7	51
Bias, %	1.486	0.000	0.003	0.000	5.725	0.000	0.426	0.000
S_e, %	3.10	3.52	2.21	3.34	4.47	5.49	1.98	5.15
R^2, %	0.51%	3.34%	6.42%	3.04%	0.78%	8.06%	25.32%	7.75%
p-value (paired *t*-test)	0.0000	1.0000	0.9939	1.0000	0.0000	1.0000	0.5895	1.0000
p-value (slope)	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0869	0.0000
p-value (intercept)	0.0000	0.0000	0.0000	0.0000	0.1096	0.0000	0.1611	0.0000
S_e/S_y	1.005	1.095	0.993	1.085	1.004	1.169	0.781	1.157
β_{f1}	1.0000	42.87	42.87	42.87[a]	1.0000	36.63	36.63	36.63
C_1	1.0000	0.3190	0.3190	0.3190[a]	1.0000	0.2218	0.2218	0.2209
C_2	1.0000	0.3190	0.3190	0.3190[a]	1.0000	0.2218	0.2218	0.2209

[a] Coefficients used to generate Figure 4.

The bias that was present in the global calibrated model was removed in the locally calibrated models. The standard error of the estimate did increase slightly but was still below 7%, which is the recommended acceptable level in the AASHTO local calibration guide (AASHTO, 2010). This recommended value is based on the cracking levels observed in national calibration, which are above the range observed in this local calibration. Although the paired *t*-test showed no bias, the slope and intercept terms in the model had a statistically significant difference from the expected values of 1.0 and 0.0. Further, the coefficient of determination and S_e/S_y values suggested a poor fit to the data in all cases. Despite these drawbacks, the local calibration coefficients were able to remove bias and can be considered a better fit to Virginia's field performance than the global calibration models. Figure 4 shows the data comparison after the local calibration coefficients were applied.

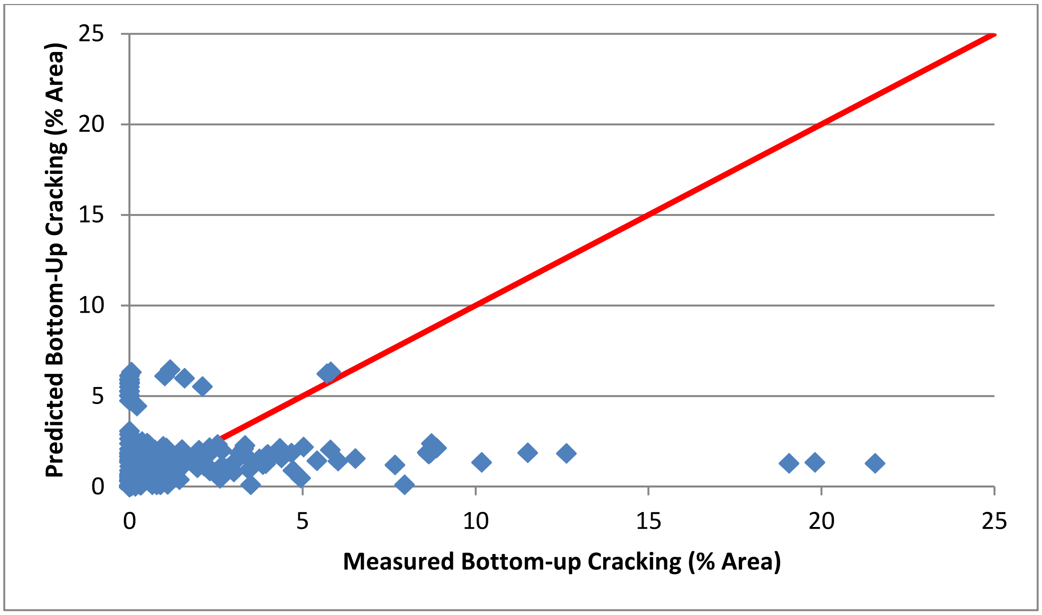

Figure 4. Asphalt Pavement Bottom-Up Cracking Calibration Coefficients With Local Calibration

Asphalt Pavement IRI Calibration

The final model considered for asphalt pavement calibration was the IRI model. Because the IRI model is dependent on the other distresses predicted, this model must be calibrated after the coefficients for the other models are adjusted. Similar to the fatigue cracking values, the predicted IRI values for all of the projects were similar whereas the range of measured values was much larger. Figure 5 shows the measured and predicted values for the globally calibrated model. The AASHTO local calibration guide does not provide a suggested value for the standard error of the estimate for IRI predictions, but the Pavement ME Design models showed a standard error of the estimate of 18.9 in/mi in national calibrations (AASHTO, 2008). These data show that the global model is underpredicting the IRI measured in the field and the S_e is above the suggested level of the reliability model.

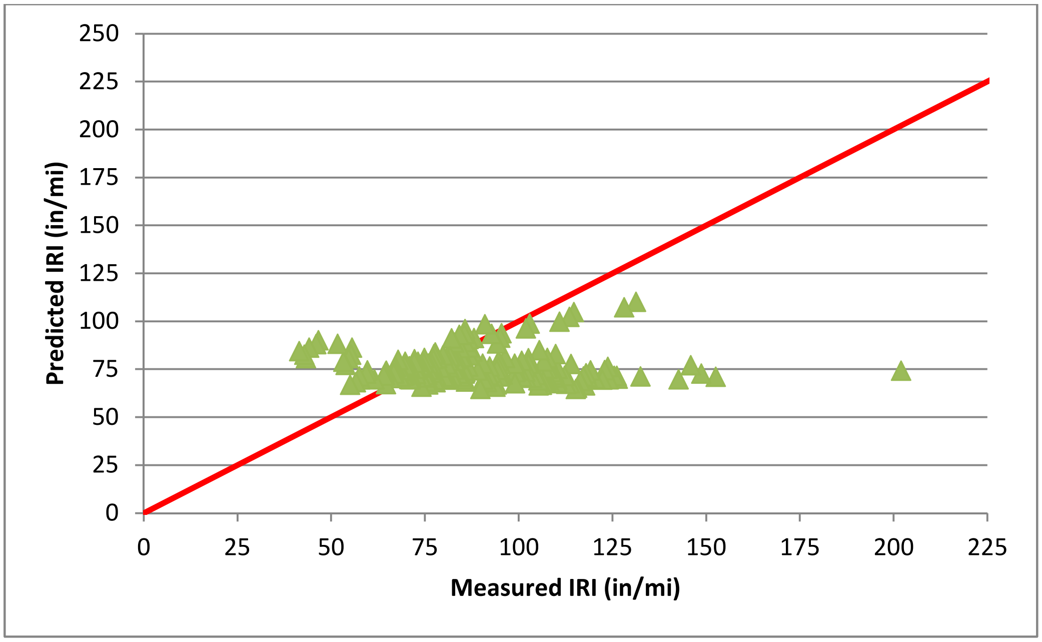

Figure 5. Asphalt Pavement International Roughness Index (IRI) With Global Calibration Coefficients

A comparison of the fit parameters from the global model with a revised model that sought to remove the bias by adjusting the site factor coefficient, C_4, is shown in Table 8, and the local calibration model is graphed in Figure 6. One difficultly in recalibrating the model coefficients to Virginia conditions was the lack of initial construction IRI values. The initial value of IRI is an important component of the predicted IRI in Pavement ME Design because it is used as a starting point for progression of IRI. The national default value of 63 in/mi was left in place; however, if the sites in calibration start with higher initial IRI values, for instance, closer to 73, then the model would show little or no bias. Without the initial IRI values, it is difficult to assume that the inaccuracy of the model prediction is caused by improper calibration coefficients. Further, it is likely that variability in the after construction IRI could reduce some of the prediction error observed. Therefore, it is recommended that VDOT maintain the global calibration coefficients at this time.

Table 8. Asphalt Pavement IRI Local Calibration Results

Parameter	Global Calibration	Local Calibration
Count	236	236
Bias, %	11.641	0.000
S_e, %	23.99	27.51
R^2, %	2.35%	4.91%
p-value (paired *t*-test)	0.0000	1.0000
p-value (slope)	0.0000	0.0000
p-value (intercept)	0.0000	0.0000
S_e/S_y	1.081	1.239
C_1	40	40[a]
C_2	0.4	0.4[a]
C_3	0.008	0.008[a]
C_4	0.0150	0.0392[a]

IRI = International Roughness Index.
[a] Coefficients used to generate Figure 6.

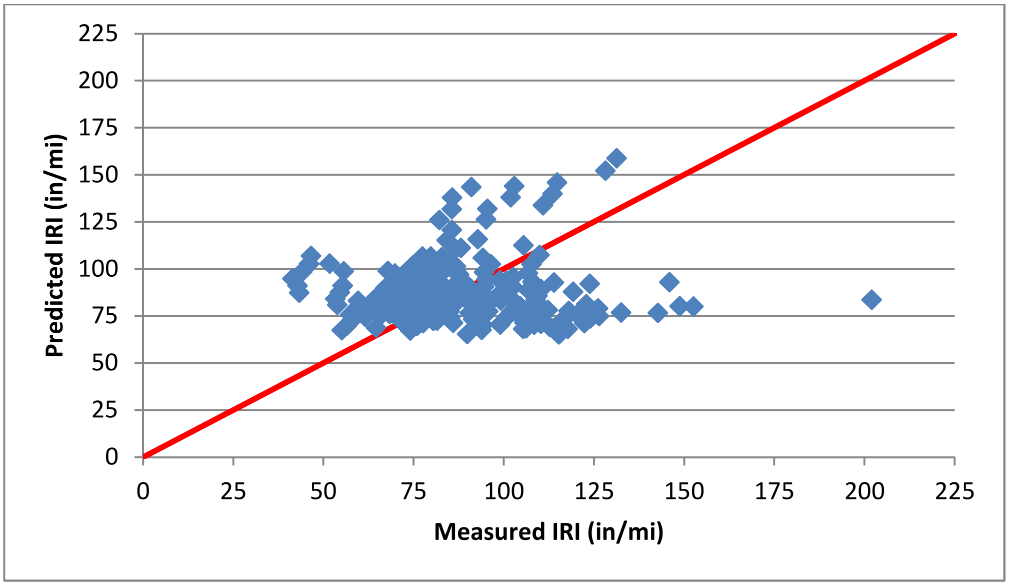

Figure 6. Asphalt Pavement International Roughness Index (IRI) With Local Calibration Coefficients

Evaluation of Revised Asphalt Local Calibration Coefficients

To evaluate further the revised rutting and fatigue local calibration coefficients and their effect on different pavement types, the calibration sites were split based on different factors and the error terms from the data points were reviewed. This type of residual analysis can be used to help determine if there are any situations where the models may have bias. The evaluation was performed by plotting the error with a subjective review to identify any factors that might need further consideration when performing design. All of the plots showed residual error (measured minus predicted) on the vertical axes; thus, negative values indicate model overprediction and positive values indicate underprediction. Although the calibration work minimized the variability of the residual error and ensured the values were on average near zero, these plots can help identify areas where the ME models may have residual error values that are grouped together, showing a pattern of underprediction or overprediction.

Figure 7 shows box plots of the rutting error and fatigue error for the different types of base under the asphalt pavement calibration sections. The select materials base type showed noticeable overprediction for both distresses, although this was on a limited number of locations (count of number of sites shown in Table 4) that also had other unique project factors. The line representing no bias is between the 25th and 75th percentile of the rutting and fatigue cracking error terms for the other base types. This figure suggests the local calibration is fairly reasonable across the different pavement base types encompassed in the sites.

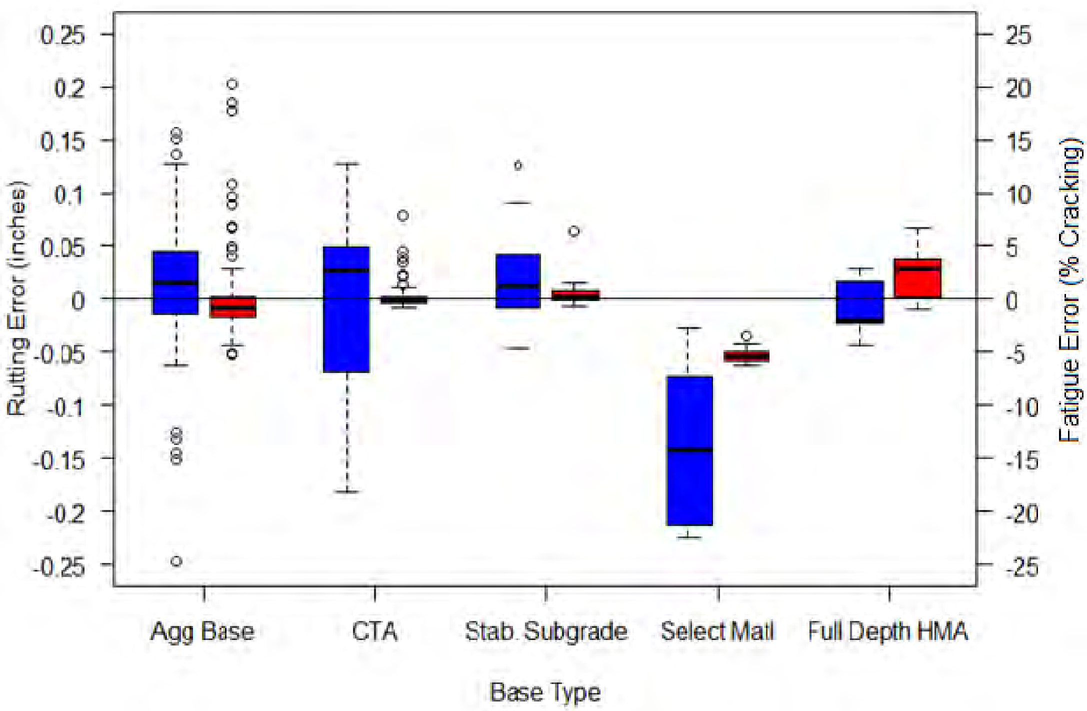

Figure 7. Asphalt Pavement Calibration Residual Error Box Plots by Base Type (rutting error shown in blue on left for each pair; fatigue error shown in red on right). Agg Base = Aggregate Base; CTA = Cement Treated Aggregate Base; Stab. Subgrade = Stabilized Subgrade; Select Matl = Select Material Base; Full Depth HMA = Full Depth Asphalt.

Figure 8 shows a scatter plot of the residual error with the subgrade resilient modulus on the horizontal axis. Based on this figure, the error values for both rutting and fatigue cracking appear centered at zero over the range of resilient modulus values with fairly consistent variance. This figure shows that observed error is not related to the subgrade modulus, suggesting that the Pavement ME Design models effectively account for the different performance related to different subgrade conditions.

Figure 9 shows the observed error with the AADTT for each section. Based on this graph, it appears the rutting is overpredicted at high truck volumes. Four sites (all located on I-81) had significantly more loading with more than twice as much truck traffic as any of the other calibration sites; two of these sites were also the only two sites with select material base that are shown as underpredicting in Figure 7. This observation should be considered when results from Pavement ME Design on highly trafficked areas are reviewed, i.e., the locally calibrated results may still be overpredicting the rutting response of the asphalt pavement. The fatigue error appears reasonable over the range of truck traffic. The box plot of error by route classification, shown in Figure 10, shows similar results with a larger skew toward negative values for interstate sites, which were largely influenced by the I-81 sites.

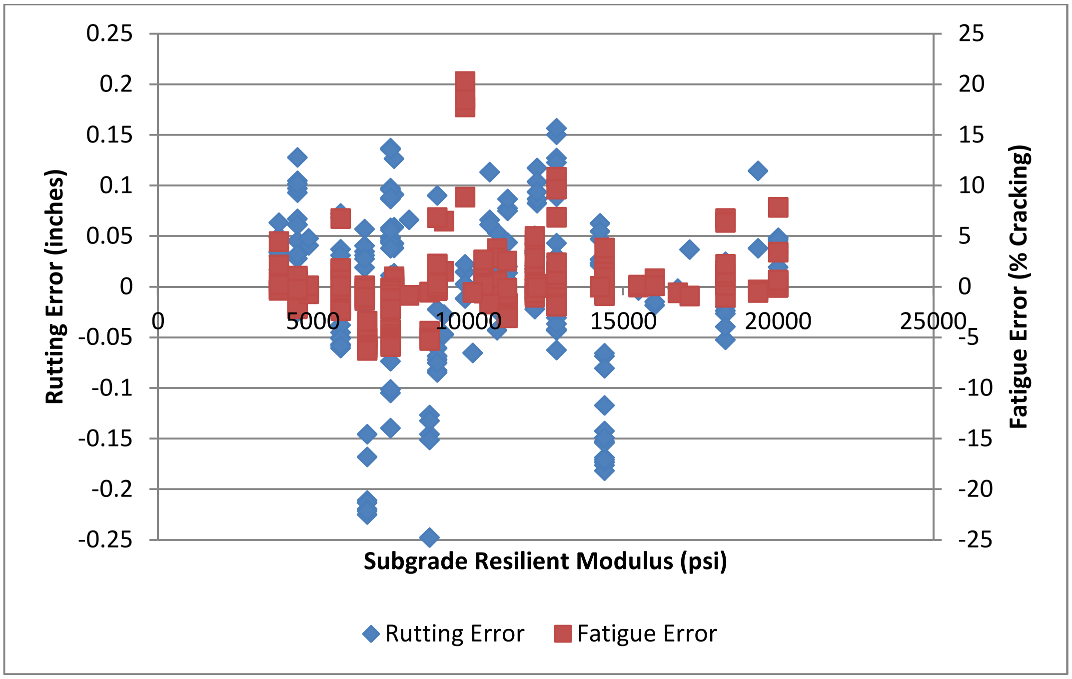

Figure 8. Asphalt Pavement Calibration Residual Error Plot by Subgrade Resilient Modulus

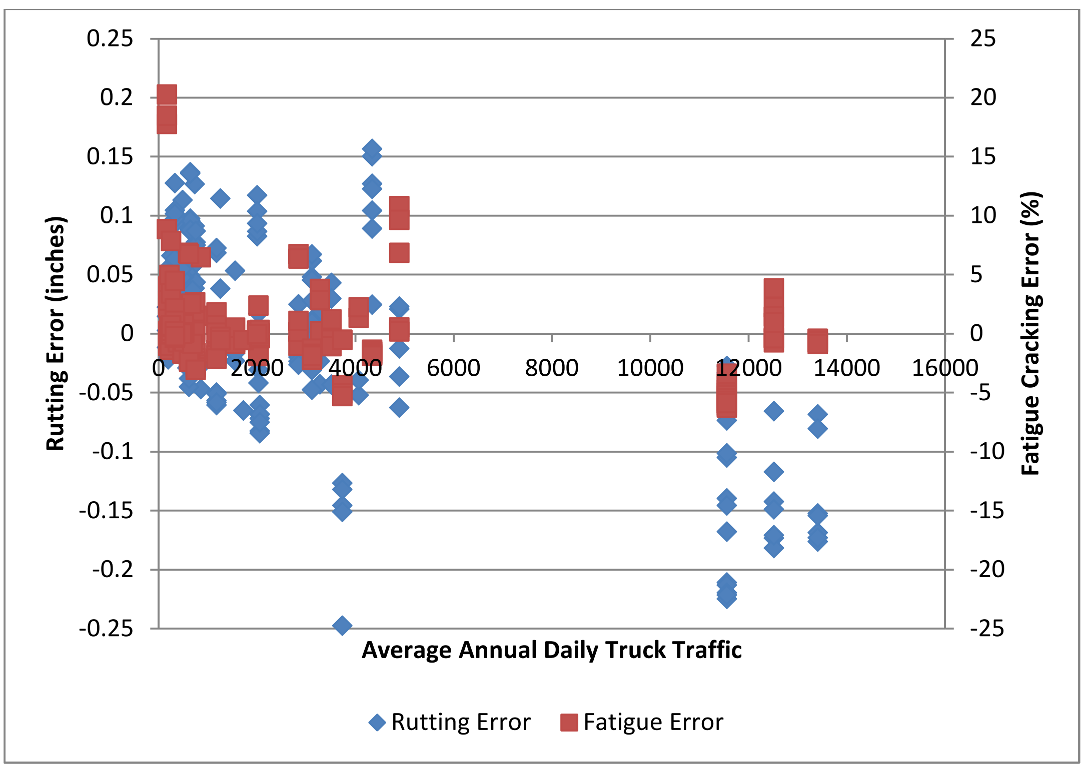

Figure 9. Asphalt Pavement Calibration Residual Error Plot by Average Annual Daily Truck Traffic

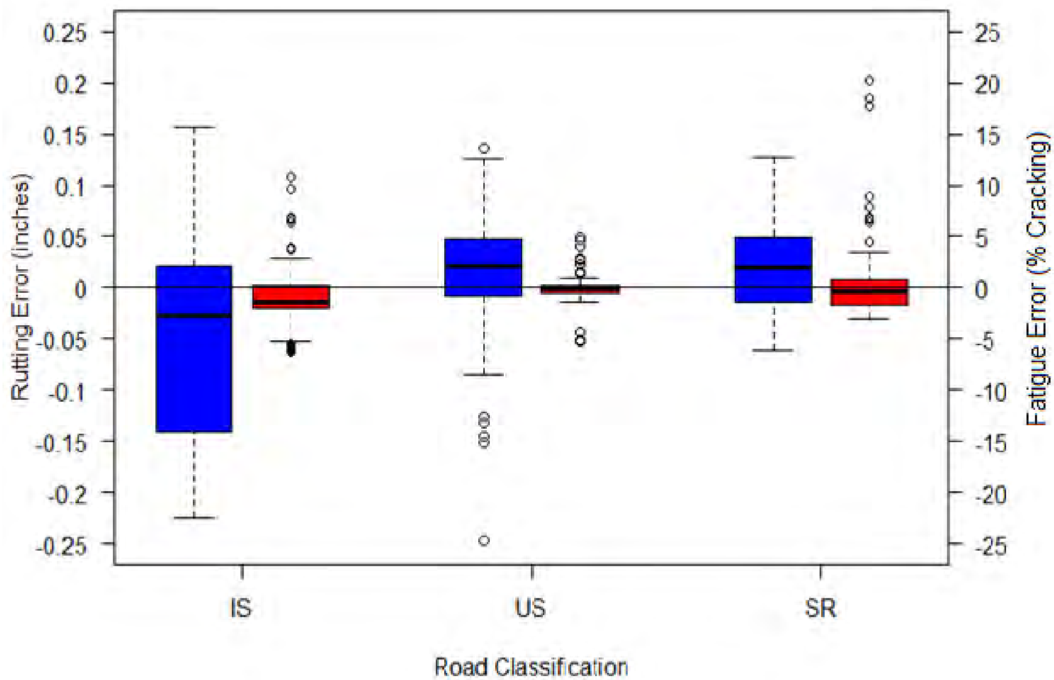

Figure 10. Asphalt Pavement Calibration Residual Error Box Plots by Road Classification (rutting error shown in blue on left for each pair; fatigue error shown in red on right). IS = interstate route; US = U.S. route; SR = state route.

Figure 11 shows the error based on the number of years since the site was constructed. Multiple years of distress data were used for calibration, so data points from the same sites are shown progressing in years as the pavement ages. Based on this figure, the bias appears consistently centered at zero, with no large differences in variance as the age progresses.

Figure 11. Asphalt Pavement Calibration Residual Error Plot by Age

Figure 12 shows box plots of the residual error as based on the asphalt thickness ranges identified in Table 4. The rutting values for the asphalt sections showed an underprediction of rutting for most of the five sites with less than 7 in of asphalt. The asphalt sections with greater than 13 in of asphalt also showed underprediction with five sites. This again shows the need for caution when applying the local calibration results beyond the ranges used in the calibration sites. The fatigue error shows little bias over the range of asphalt thicknesses and has small boxes for the interquartile range, indicating that most of the data are consistent for the different asphalt ranges.

The box plots in Figure 13 show the rutting and fatigue error for the asphalt calibration sites based on stone matrix asphalt (SMA) and standard dense-graded asphalt surface mixtures. Seven of the sites were constructed with a gap-graded SMA surface layer that responds to pavement loading differently than the dense-graded asphalt used on the rest of the sites. The residual error between these two surface mixture types showed similar results, which suggests that Pavement ME Design sufficiently accounts for the difference between the two mixture types when predicting rutting and fatigue distress.

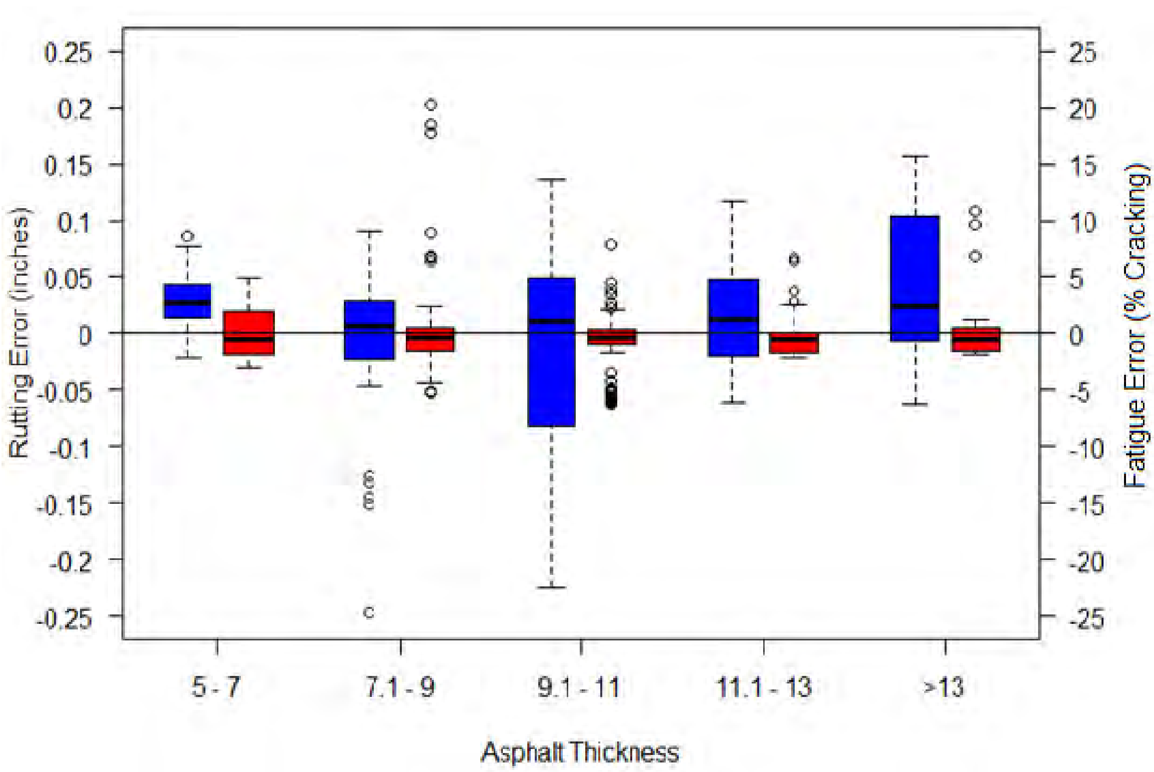

Figure 12. Asphalt Pavement Calibration Residual Error Box Plots by Asphalt Thickness (in inches) (rutting error shown in blue on left for each pair; fatigue error shown in red on right).

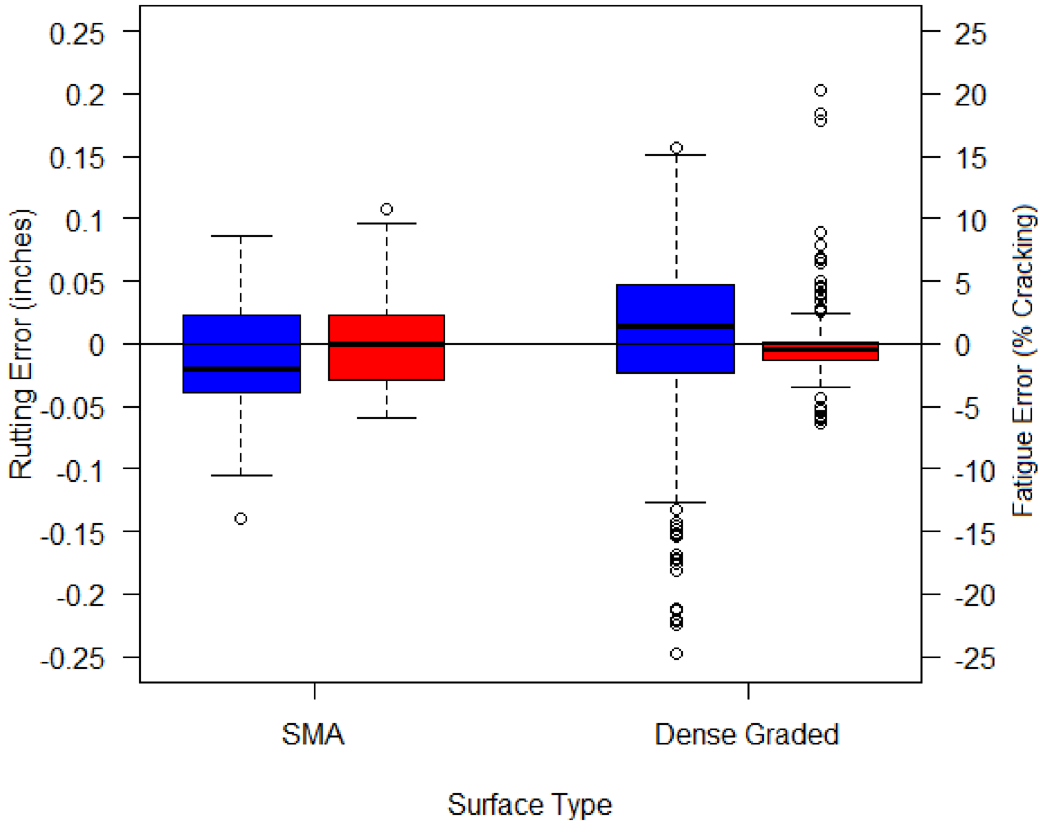

Figure 13. Asphalt Pavement Calibration Residual Error Box Plots for Stone Matrix Asphalt (SMA) Versus Dense-Graded Asphalt Surface Type (rutting error shown in blue on left for each pair; fatigue error shown in red on right).

CRCP Punchout Calibration

The main distress that is used for CRCP in Pavement ME Design is the prediction of punchouts (count/mile). The comparison of predicted to measured punchouts based on the global calibration coefficients is shown in Figure 14. The global calibration coefficients used represent the default coefficients in Pavement ME Design (Version 1.3).

Table 9 shows the C_3 factor and fit statistics for the global and calibration datasets. With the global calibration coefficients, the punchouts were overpredicted by 8/mi. The rows labeled for each site show the validation results from that site after C_3 was calibrated based on the remaining 15 sites. Thus, the statistics in these rows are independent of the model calibration. For 13 of the 16 cases, the paired *t*-test was not significant at an alpha level of 0.05, suggesting that there was no statistically significant difference between the measured and predicted punchout values. The standard error was improved from the global value for 14 of the 16 validation sites.

The combined calibration data showed no bias and an improved standard error from the global calibration coefficient; therefore, the model was assumed to be acceptable, despite the fact that the standard error of the estimate value (21.8/mi) from the local calibration was above the value of 4/mi recommended in the AASHTO local calibration guide (AASHTO, 2010).

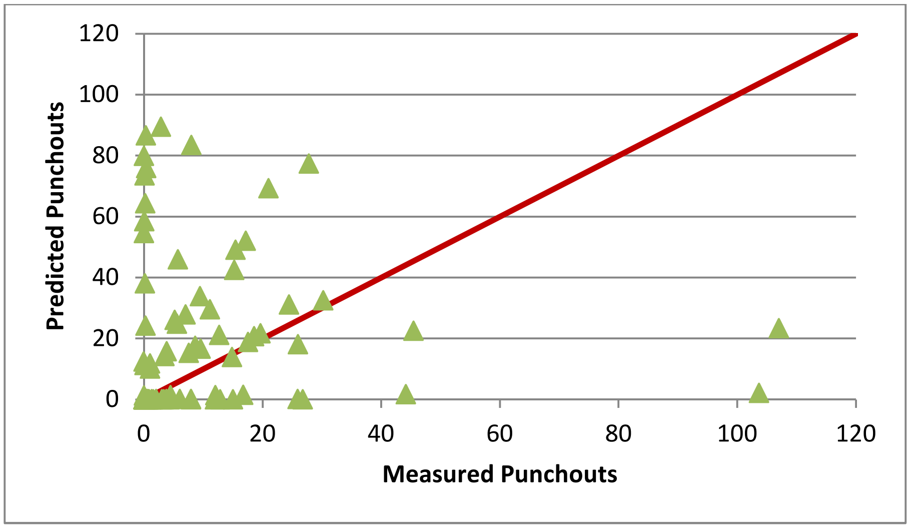

Figure 14. Concrete Pavement Punchout Comparison With Global Calibration Coefficients

Table 9. Punchout Local Calibration Jackknife Results

Dataset	C_3	Count	Bias	Standard Error	S_e/S_y	Significance Test *p*-Value		
						Paired-*t*	Slope	Intercept
Global	216.84	87	-8.47	30.3	1.705	0.01	0.00	0.00
PCC-2	114.52	6	0.29	0.4	1.000	0.14	0.00	0.01
PCC-5	100.75	5	20.21	42.8	0.964	0.35	0.00	0.00
PCC-6	118.05	7	-3.38	9.6	0.886	0.39	0.00	0.00
PCC-7	108.87	6	7.09	14.5	0.888	0.28	0.00	0.00
PCC-8	90.16	6	29.57	39.1	0.997	0.12	0.00	0.00
PCC-9	112.69	4	3.74	6.1	1.000	0.31	0.00	0.00
PCC-10	109.29	6	6.58	6.2	1.000	0.05	0.00	0.00
PCC-12	109.90	4	8.78	11.9	1.000	0.24	0.00	0.00
PCC-13	113.00	6	2.12	2.4	0.996	0.08	0.00	0.00
PCC-14	114.16	7	0.62	8.7	0.948	0.86	0.00	0.00
PCC-15	111.03	2	13.46	18.9	1.000	0.50	N/A	N/A
PCC-17	130.89	6	-19.38	8.2	1.033	0.00	0.02	0.00
PCC-18	114.19	6	0.69	1.3	1.000	0.26	0.00	0.00
PCC-19	114.00	6	0.91	1.2	1.000	0.11	0.00	0.00
PCC-20	154.00	5	-56.61	4.3	1.283	0.00	0.35	0.00
PCC-21	154.00	5	-56.61	4.3	1.283	0.00	0.35	0.00
Combined calibration	114.76[a]	87	0.00	21.8	1.226	1.00	0.00	0.00

N/A/ = Not available because of too few data points.
[a] Coefficient used to generate Figure 15.

Additional calibration attempts to reduce further the standard error and achieve slope and intercept values statistically similar to 1.0 and 0.0, respectively, by adjusting C4 and C5 with C3 did not result in a practical set of calibration coefficients. The model with C3 calibrated based on the full dataset from the 16 sites was selected for use as it removed the bias and improved on the standard error from the global calibration model. Figure 15 shows the measured and predicted distress comparison based on the locally calibrated model.

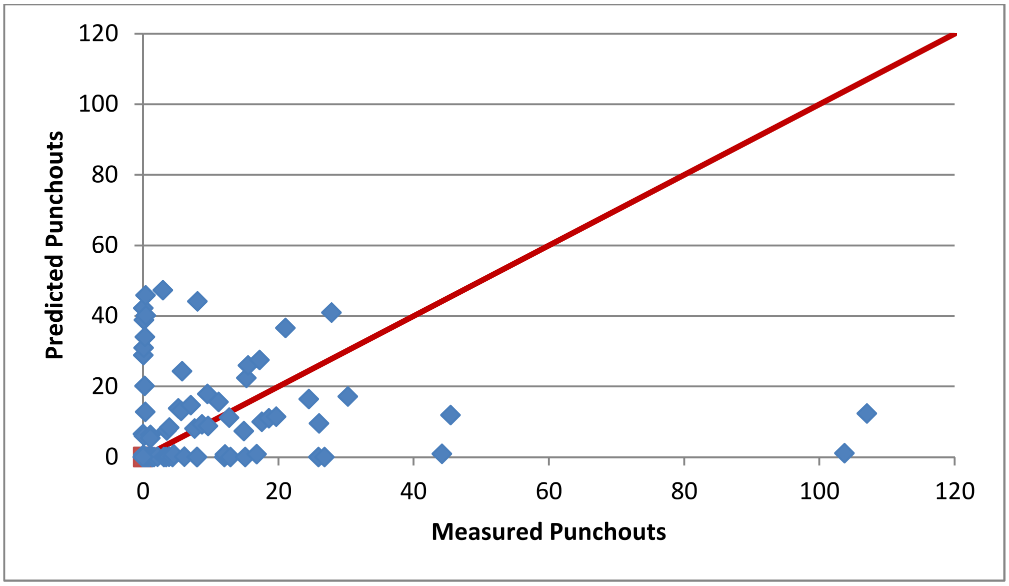

Figure 15. Concrete Pavement Punchouts With Local Calibration Coefficient

Concrete IRI Calibration

After the punchout prediction model was adjusted, the CRCP ride quality model was evaluated. Figure 16 shows the global predicted IRI values with the measured values from PMS. This figure shows a large number of points with a predicted IRI near 63 in/mi, which was the initial IRI value used when the analysis was performed in Pavement ME Design for the projects. These points show that when no punchout distress is predicted, the IRI of the CRCP is expected to stay fairly constant. The graph also shows that Pavement ME Design is underpredicting IRI values for the sites considered.

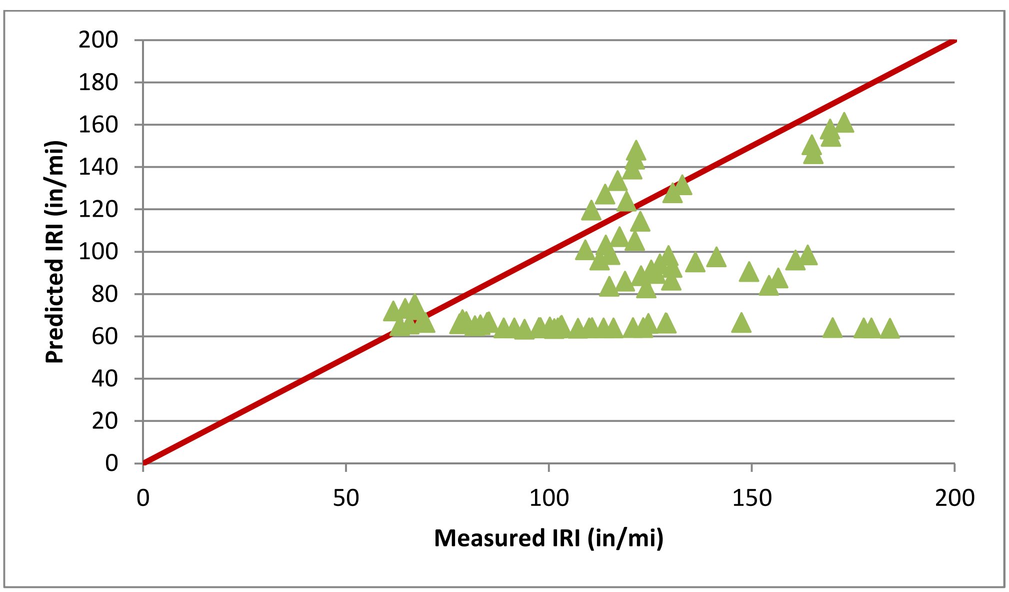

Figure 16. Concrete Pavement International Roughness Index (IRI) Comparison With Global Calibration Coefficients

Table 10 shows the data fit using the global calibration coefficients and local calibration coefficients adjusting C_1 and C_2 to reduce the bias and minimize the standard error; the local calibration comparison is graphed in Figure 17. The number of data points differs from the total amount shown in Table 9 because the PMS data did not contain IRI results for data collection year 2013. The AASHTO local calibration guide does not provide a suggested value for the standard error of the estimate for IRI predictions, but a value of 14.6 in/mi was observed in the national calibration dataset (AASHTO, 2008). Although the bias was lowered to an insignificant level, the standard error increased significantly. Similar to the evaluation of IRI predictions with asphalt pavement, the unknown initial IRI value may have a pronounced effect on the model agreement. Therefore, the global coefficient values are preferable to locally adjusted values until more information is available.

Table 10. Concrete Pavement IRI Local Calibration Results

Parameter	Global Calibration	Local Calibration
Count	82	82
Bias, in/mi	28.53	1.44
S_e, %	31.08	40.97
R^2, %	21.3	11.6
p-value (paired *t*-test)	0.0000	0.7500
p-value (slope)	0.0000	0.0000
p-value (Intercept)	0.0004	0.0000
S_e/S_y	0.99	1.30
C_1	3.15	9.55[a]
C_2	28.35	172.55[a]

IRI = International Roughness Index.
[a] Coefficients used to generate Figure 17.

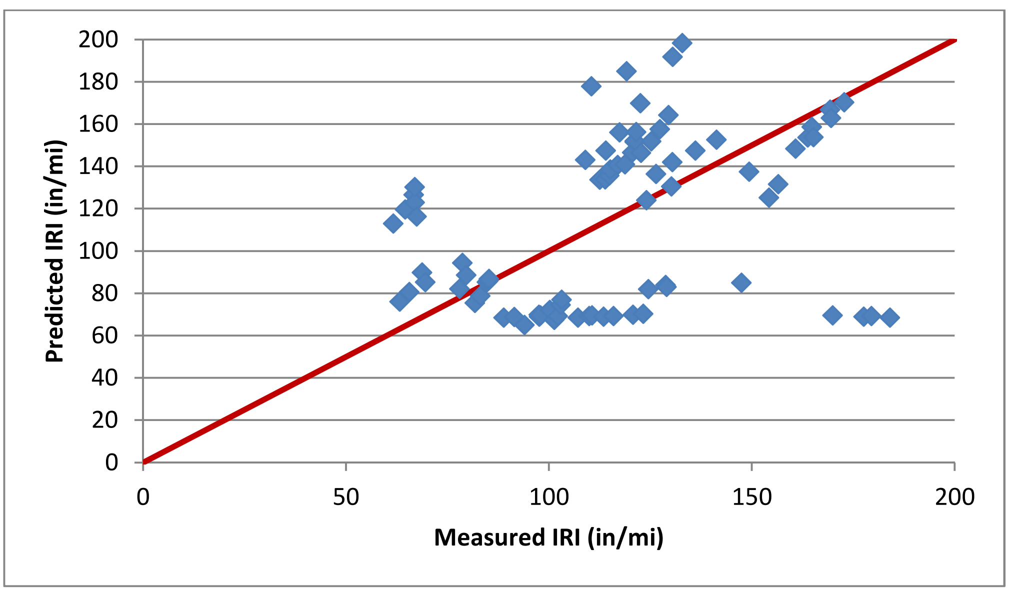

Figure 17. Concrete Pavement International Roughness Index (IRI) Comparison With Local Calibration Coefficients

Evaluation of Revised Concrete Local Calibration Coefficients

Similar to the asphalt pavement analysis, an evaluation of the concrete pavement local calibration fit was performed by grouping the projects by different features and comparing the residual errors (measured minus predicted) from each data point. The only CRCP distress that was locally calibrated was the punchout prediction since IRI or JPCP distresses were not calibrated against field measurements. Two sites (both on I-295) each contained a data point with a punchout residual error greater than 90/mi; because these outlier error values were well beyond all the rest of the data points, the axis limits for the graphs were set at ±40/mi to show the majority of the data points better.

The punchout residual error is shown in Figure 18 by base type as described in Table 5. This graph shows that only pavements with aggregate base material showed overprediction of distress; the sections with different base types typically showed very small underpredictions, likely attributable to models predicting little damage on these sections. The sections on an aggregate base also showed larger variability in the results; the two residual points above 90 were also in the aggregate base group. Although there is a limited number of data points for the three base types other than aggregate base, significant bias was not identified relating to the base type under CRCP.

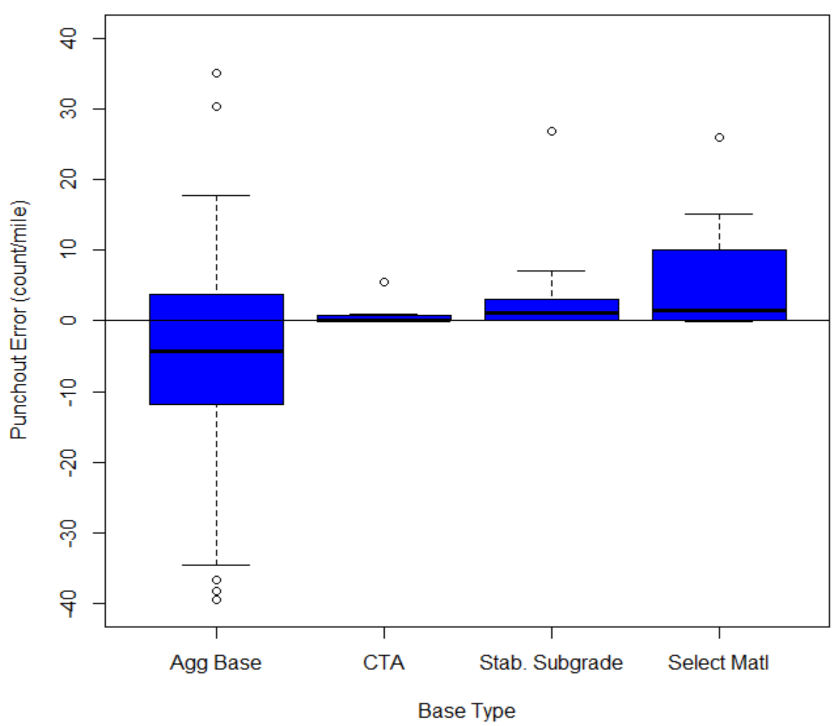

Figure 18. Concrete Pavement Calibration Residual Error Box Plot by Base Type. Agg Base = Aggregate Base; CTA = Cement Treated Aggregate; Stab. Subgrade = Stabilized Subgrade; Select Matl = Select Material Base.

Figure 19 shows the punchout error based on the subgrade resilient modulus for each project. Because the CRCP projects were located in only select areas in the state and the pavement structure inputs were developed based on database values, most of the subgrade inputs were similar for the projects. The two outlier values above 90/mi are located at the band of data points around 16,500 psi. The values appear to show no change in the bias related to subgrade resilient modulus.

A scatterplot of concrete pavement calibration residual error versus AADTT is shown in Figure 20. This chart does not show any trends between the truck traffic volume and model prediction error for punchout distress. The two outlier points at approximately 90/mi did have different truck traffic volumes: one project had an AADTT of 3,930, and the other 6,808.

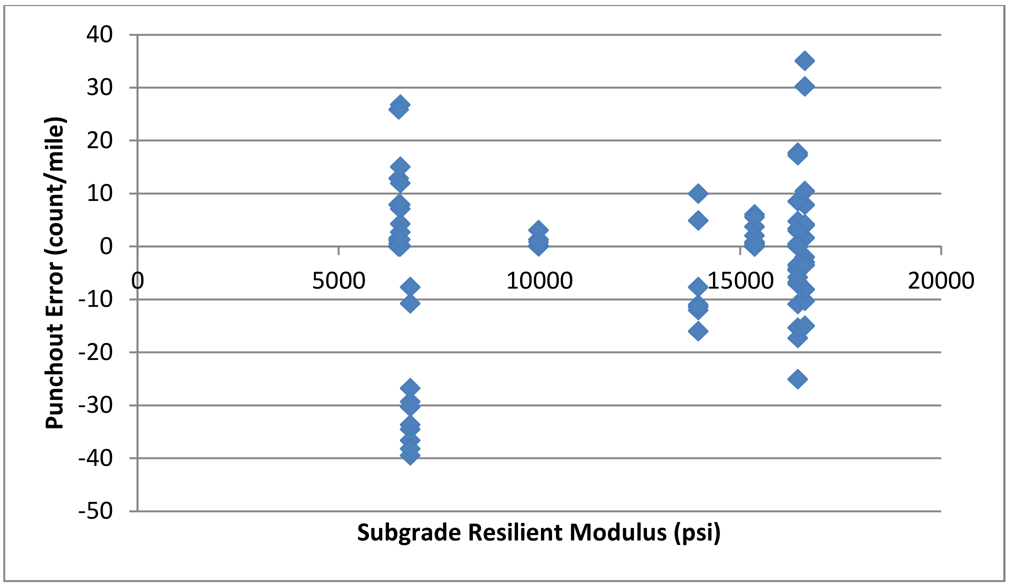

Figure 19. Concrete Pavement Calibration Residual Error Plot by Subgrade Resilient Modulus

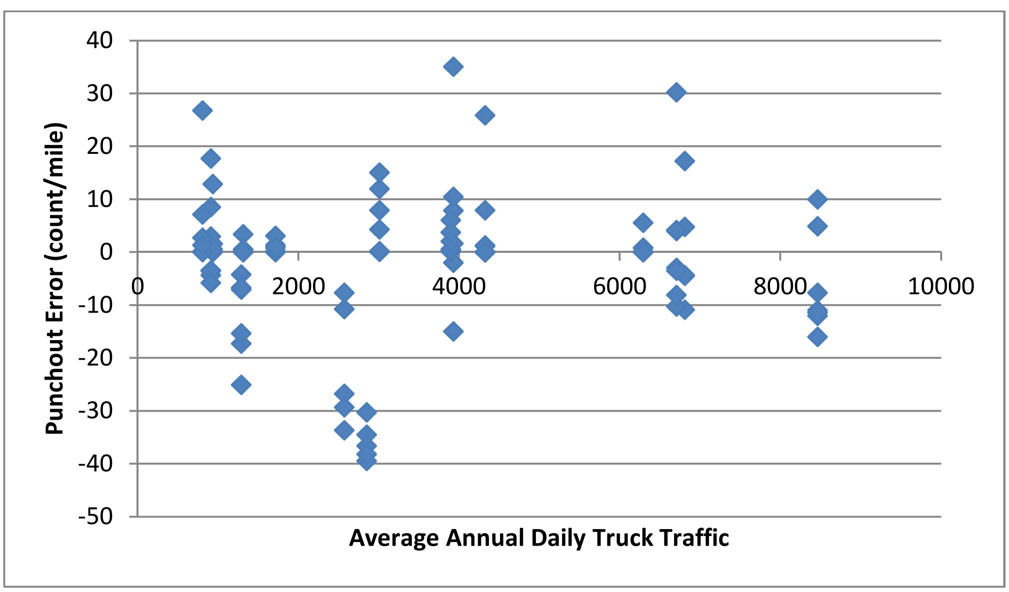

Figure 20. Concrete Pavement Calibration Residual Error Plot by Average Annual Daily Truck Traffic

Figure 21 shows the age of the concrete pavement versus the error of the punchout calibration. There is a noticeable difference between the residual error for CRCP for three projects in the first 10 years and for CRCP at 15 to 25 years of age. This difference may be related to the more recent projects having little to no distresses predicted and measured. In addition to having less time in service, the more recent pavements were also built thicker and with improved construction methods, both of which are expected to help improve the pavement performance. The error values from the data points with greater age values have a larger variance; the two outlier points are at the age of 20 years. Because the data are centered at zero error for both sets of age ranges, it appears the locally calibrated model is appropriate for more recent CRCP sections as well as those that have been in service for a longer period.

A corresponding factor to the age of the sites is the thickness of the pavement structure, shown in box plots of the residual error in Figure 22. The three projects built after 2000 were the only projects constructed with a concrete thickness of 10 or 12 in, whereas all of the 8- and 9-in projects represented projects that have 15 to 25 years of aging. The 8-in pavement sections were all built on aggregate base sections and their punchout distress was similarly overpredicted, as shown in Figure 18. One outlier data point each was in the 8-in and 9-in concrete thickness ranges. Based on the relationships between base type, age, and concrete thickness among the sites, it is hard to determine if one of these factors contributed more or less than the others to the model prediction error. Overall, the results appear reasonable over the range of CRCP thicknesses included in the dataset.

Figure 23 shows box plots of the CRCP residual error based on the road classification. All of these CRCP sites were built as new construction with fairly high volumes of truck traffic expected, regardless of the road classification. The U.S. routes show primarily negative error values, whereas the other two classifications had average values very close to an error of zero. The U.S. routes represented only three different projects, two of which were near the same location with matching designs constructed 2 years apart.

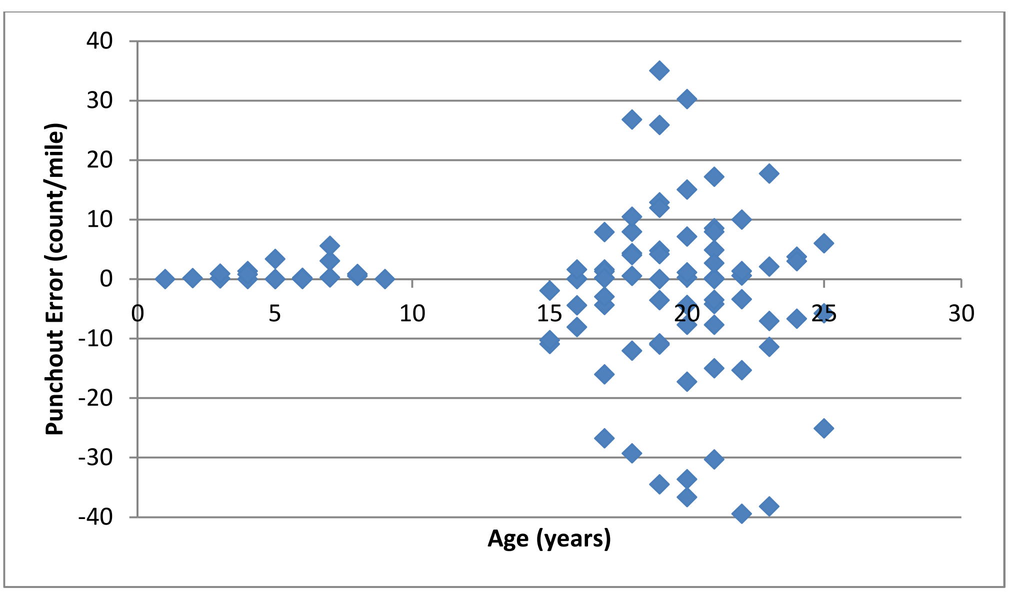

Figure 21. Concrete Pavement Calibration Residual Error Plot by Age

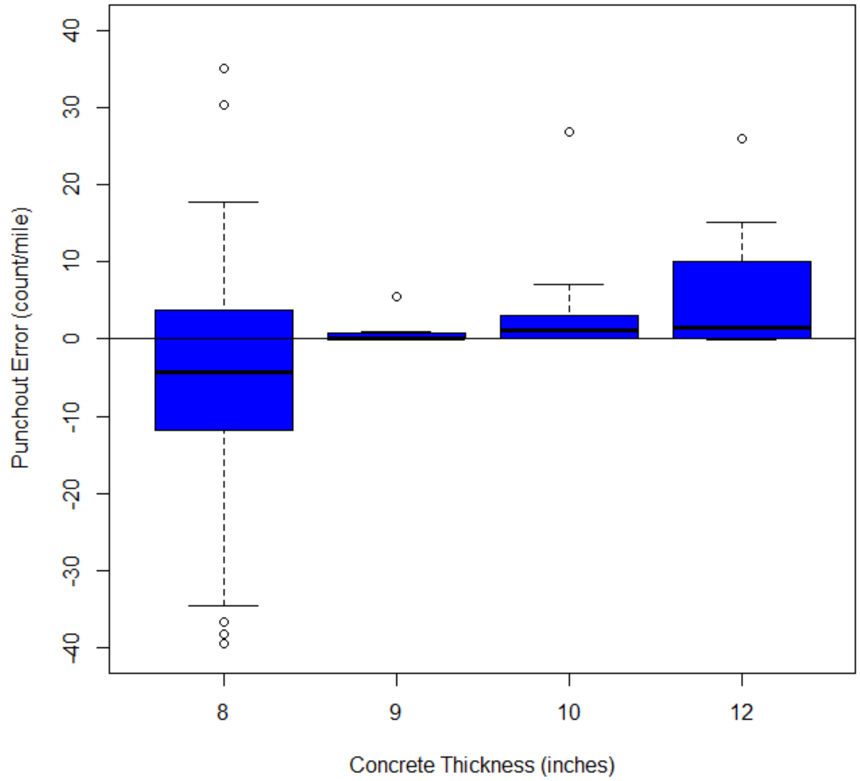

Figure 22. Concrete Pavement Calibration Residual Error Box Plot by Concrete Thickness

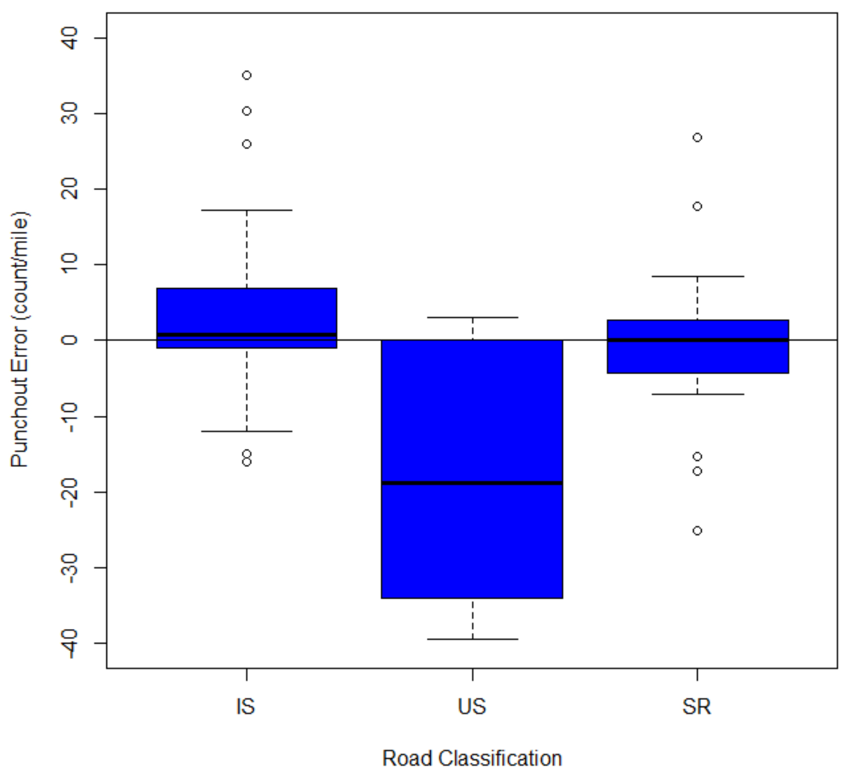

Figure 23. Concrete Pavement Calibration Residual Error Box Plot by Road Classification. IS = interstate route; US = U.S. route; SR = state route.

Suggested Values for Design Requirements

Another essential step toward implementing the ME design procedure for VDOT is reviewing requirement values for design. Values such as the design life, reliability level, and performance limit (or target value) can all have a significant effect on whether a pavement design is suitable for a project or if adjustments are necessary. VDOT's current pavement design policy, i.e., following the 1993 AASHTO design guide, specifies the design life and reliability values for the different road classifications and pavement types shown in Table 11. The performance limit with the 1993 AASHTO design policy is based on terminal serviceability; the values used for VDOT pavement designs, found in the VDOT Materials Division *Manual of Instructions*, Chapter 6, also vary by highway classification (VDOT, 2008). The design life and reliability in the 1993 AASHTO design guide resemble criteria used for Pavement ME Design; although they are integrated into the two design methods differently, the previously established values are helpful in identifying values for Pavement ME Design. Performance limits based on distress in Pavement ME Design are harder to compare to existing criteria in the 1993 AASHTO design guide based on serviceability, so target values for Pavement ME Design will need to be considered separately from existing values.

The design life values shown in Table 11 are for new asphalt pavement designs; concrete pavements are designed for a 30-year life for all classifications. It is acknowledged that rehabilitation work is likely to be performed on a pavement before the end of the design life to maintain its functional characteristics, whereas the objective of the design life is to prevent structural repairs from being required during the design life period. This distinction is implicit in the 1993 AASHTO pavement designs, whereas Pavement ME Design predicts pavement performance in terms of both functional and structural criteria. Therefore, some performance measures are better evaluated on a shorter time frame that better represents when a functional repair will be scheduled. A longer design life (e.g., 30 years) is still needed to evaluate distresses that identify an insufficient pavement structure. A similar structural design life is recommended for lower highway classifications since the functional characteristics can be separated in design and any additional costs to achieve a lasting pavement structure are expected to be outweighed by reducing the need for major rehabilitation.

Table 11. VDOT Design Life and Reliability Level With 1993 AASHTO Pavement Design Guidelines

Highway Classification	Design Life (years)	Reliability (%)	
		Urban	Rural
Interstate	30	95	95
Divided primary	30	90	90
Undivided primary	20	90	85
High-volume secondary	20	90	85
Farm to market secondary	20	85	75
Subdivision	20	75	70

In both the 1993 AASHTO pavement design and Pavement ME Design, reliability values are used to account for variability that is expected from design and construction of the pavement. The reliability levels work differently in the two programs in that the 1993 AASHTO pavement design reliability factors add additional loading (or damage) whereas Pavement ME Design increases the predicted distress or IRI based on the variability of the prediction. The variability for each performance measure in Pavement ME Design is determined from standard deviation values calculated based on the level of distress (greater distress equals greater variability), although the standard deviation for IRI remains constant regardless of predicted ride quality. Reviewing the standard deviation models was not considered as part of this study, so the globally calibrated error models were maintained for variability calculations.

The reliability level should be considered in conjunction with the performance criteria, as a high reliability level can be especially difficult or costly if the distress limits are also low (AASHTO, 2008). With regard to the reliability levels VDOT uses with 1993 AASHTO design, a 95% reliability is recommended for interstate projects. A 90% reliability value is recommended for both divided and undivided primary highways regardless of whether the setting is urban or rural. This should simplify some of the design considerations and is not expected to change the pavement design outputs greatly. For secondary routes being designed with Pavement ME Design, a reliability value of 85% is recommended. VDOT policy on when to apply Pavement ME Design (based on functional classification, etc.) was outside the scope of this study. These reliability levels were taken into consideration in determining appropriate values for the performance targets discussed here.

Asphalt Pavement Design Requirements

The default asphalt pavement performance limit criteria in Pavement ME Design (Version 1.3) and recommended values from the MEPDG Manual of Practice (AASHTO, 2008) are shown in Table 12 along with the associated reliability recommendations. As mentioned previously, some of the models shown will not be considered for VDOT pavement designs at this time.

Table 12. AASHTO (2008) Recommendations for Asphalt Pavement ME Design Criteria

Performance Criterion	Program ME Design Default	MEPDG Manual of Practice		
		Interstate	Primary	Secondary
Terminal IRI (in/mi)	172	160	200	200
Asphalt top-down fatigue cracking (ft/mi)	2,000	N/A	N/A	N/A
Asphalt bottom-up fatigue cracking (%)	25	10%	20%	35%
Asphalt thermal cracking (ft/mi)	1,000	500	700	700
Chemically stabilized layer–fatigue fracture (%)	25	N/A	N/A	N/A
Permanent deformation–total pavement (in)	0.75	0.4	0.5	0.65
Permanent deformation–asphalt only (in)	0.25	N/A	N/A	N/A
Reliability level (%)	90	95	85-90	75-80

IRI = International Roughness Index.

These recommendations from national guidelines may need to be adjusted to represent typical distress levels observed by VDOT. Based on VDOT's *State of the Pavement 2013*, 12.5% of the primary miles in Virginia had an IRI greater than 140 in/mi, with only 2% with an IRI greater than 200 in/mi (VDOT, 2013b). This suggests that a threshold limit lower than the 200 in/mi suggested by AASHTO (2008) for primary routes or the 172 in/mi program default value would be more consistent with experience. Distress ratings from different VDOT-specific sources that were used to help identify performance limits for asphalt pavements are shown in Table 13. Column 1 shows the amount of distress that would result in a pavement being rated as deficient either by IRI or through CCI using deduct equations for Level 3 alligator cracking and permanent deformation. These values are assuming that the particular distress is the only deficiency, which would be an extreme case. Column 2 shows the average distress measured prior to rehabilitation based on VDOT's PMS data. Column 3 shows the average of the maximum IRI or distress that was observed at each site used in local calibration.

A consistent pattern for the IRI, asphalt bottom-up cracking, and permanent deformation is that the distress from the PMS deduct is greater than the average prior to rehabilitation, which is greater than the average distress from the calibration sites. This pattern is not irrational considering pavement resurfacing is scheduled to prevent a pavement being rated as deficient in many cases. This may be especially true for calibration sites, which were all constructed relatively recently and mostly show good performance, although some have already been resurfaced. As noted previously, it is important to consider the effect of the reliability when performance limits are selected. It would be unreasonable to select performance targets in accordance with the average observed distress and then combine them with a high value of reliability; that would create a change in the expected level of performance, which is not a desired result of changing pavement design methodologies.

A value of 6% is recommended for the bottom-up fatigue cracking performance limit primarily because of the distress levels observed in this calibration study. Because the predicted cracking at calibration sites does not represent a very robust model over a large range of measured cracking values, it is important to ensure that the performance criteria are well within the range of predicted distress. The recommended value is based on the typical peak distress of calibration sites and accounts for variability between 75% and 95%, depending on highway type.

Table 13. VDOT Distress Measurements for Asphalt Pavement Performance Limit Selection

Performance Criterion	Distress to Reach "Deficient" PMS Rating	Average Prior to Rehabilitation	Average Maximum Distress of Calibration Sites
Terminal IRI (in/mi)	140[a]	110	101
Asphalt bottom-up fatigue cracking (%)	15[b]	11	4.6
Permanent deformation–total pavement (in)	0.37	0.18	0.16

PMS = Pavement Management System; IRI = International Roughness Index.
[a] 140 in/mi represents "deficient" for interstate and primary.
[b] Calculation for bottom-up fatigue cracking deficient rating taken from Level 3 alligator cracking.

Because both the rutting (occurring predominately in the asphalt surface) and IRI for asphalt pavements are expected to be improved when a functional rehabilitation is performed on a project, the predicted performance of these distresses at the end of a 30-year structural design life may not match experience. It is recommended that the predicted rutting distress and IRI be considered at a 15-year period for all pavement designs. This shorter time frame will show values that are more in line with VDOT's experience for pavements nearing resurfacing.

For permanent deformation, again using total amount, which is predominately the rutting in asphalt, the average prior to rehabilitation and peak values of the calibration sites were similar. A target value of 0.26 in is recommended for use in pavement design. Based on the data that were considered, a rutting value performance target of 0.26 would match what is typically observed prior to overlay, accounting for variability included for higher reliability designs.

For terminal IRI, a good portion of the interstate and primary routes are resurfaced while still in good condition and many of the rest are in fair condition. However, the large adjustment attributable to reliability (standard error of 18.9 in/mi for new asphalt pavements) makes the 140 in/mi limit of distress, coinciding with deficient ride quality for interstate and primary routes, an appropriate performance limit. Secondary pavements show higher levels of IRI that may be attributable to the higher initial IRI values (typically VDOT has not applied rideability specifications for these projects) and deferred resurfacing on the sections. The current recommendation is to use the same performance limit and time frame for secondary routes as for other highway classifications, although a longer service life or adjusted initial/terminal IRI values may be considered in the future.

Concrete Pavement Design Requirements

Target values were considered for CRCP and JPCP. Limits for distress or IRI values at the end of the design life as recommended for concrete pavements are shown in Table 14. The reliability level recommendations from ME Pavement Design and AASHTO's Manual of Practice (AASHTO, 2008) are also listed (these are the same as shown in Table 12). Since IRI is consistent for both pavement types, the performance limit should be the same between the two pavement types.

Table 14. MEPDG Recommendations for Concrete Pavement Design Criteria

Performance Criterion	Pavement ME Design Default	MEPDG Manual of Practice (AASHTO, 2008)		
		Interstate	Primary	Secondary
Terminal IRI (in/mi)	172	160	200	200
CRCP punchouts (count/mile)	10	N/A	N/A	N/A
JPCP transverse cracking (% slabs)	15	10	15%	20%
JPCP mean joint faulting (in)	0.12	0.15	0.20	0.25
Reliability level (%)	90	95	85-90	75-80

IRI = International Roughness Index; CRCP = continuously reinforced concrete pavement; JPCP = jointed plain concrete pavement.

The values shown in Table 15 were based on VDOT's PMS data collection on concrete pavements. The average distress prior to rehabilitation values were based on historical PMS data for the count of punchouts per mile in CRCP and the percentage of divided slabs in JPCP. The other values shown for CRCP punchouts represent the number per mile based on the area of distress.

For CRCP punchouts, the value of 6/mi is suggested for a target value for VDOT designs. This value is less than the default target and the magnitude of distress observed for many of the calibration sites but is near the typical distress level measured prior to rehabilitation. The suggested initial target of 6/mi agrees with some of the most recent CRCP designs that are expected to have a long service life with little damage (and have shown very little distress thus far in service).

Table 15 also shows some distress values for JPCP pavements, although based on only four sites that were identified in the state. Based on the deduct equations for jointed pavements, the program default target value of 15% cracked slabs appears reasonable. Based on the limited data on JPCP in Virginia, the faulting predictions may be lower than what has been observed in PMS, although since the model has not been locally calibrated, it is recommended that the default value of 0.12 in be maintained.

Table 15. VDOT Distress Measurements for Concrete Pavement Performance Limit Selection

Performance Criterion	Distress to Reach "Deficient" PMS Rating	Average Prior to Rehabilitation	Average Maximum Distress of Calibration Sites
Terminal IRI (in/mi)	140	N/A	124
CRCP punchouts (count/mile)	65	7	29
JPCP transverse cracking (% slabs)	15	2	5.3
Mean joint faulting (in)	N/A	N/A	0.158

IRI = International Roughness Index; CRCP = continuously reinforced concrete pavement; JPCP = jointed plain concrete pavement.

Example Pavement Structure with ME Design Process

To demonstrate the pavement structures that are developed using the ME design process, some example designs were performed using hypothetical pavement design scenarios. Table 16 shows the route type, traffic, location, and subgrade inputs that were used to develop the ME design. Scenarios A, B, and C were selected based on the 25th, 50th, and 75th percentile of AADTT levels, respectively, from interstate and primary traffic link data (VDOT, 2012).

Table 16. Example Pavement Design Scenario Inputs

Scenario	Route Type	Virginia County	No. of Lanes (Directional)	Annual Daily Traffic (2-way)	% Trucks	AASHTO Soil Classification	Resilient Modulus (psi)
A	Primary	Nelson	1	2,200	8%	A-4	3,882
B	Primary	Chesapeake	2	9,300	11%	A-2-4	6,533
C	Interstate	Montgomery	3	22,000	25%	A-6	8,853

Pavement designs were developed for each of the scenarios using Pavement ME Design with the local calibration coefficients and performance limits identified in this study and project inputs based on Table 16 in accordance with VDOT's *Pavement ME User Manual—Draft* (VDOT, 2013a). The pavement structures that were developed for each of the hypothetical design situations are shown in Table 17 for asphalt, CRCP, and JPCP sections. The layer thickness values represent the necessary structure assuming a typical base structure and do not reflect project-specific design features that could be used on a specific project (i.e., stabilized subgrade or widened-edge concrete pavement).

Table 17. Example Pavement Structure From ME Design Process

Scenario	Asphalt Design	CRCP Design	JPCP Design
A	1.5 in SM-9.5A 2.5 in IM-19.0A 3.5 in BM-25A 8 in 21A	8.0 in CRCP 8 in 21A	8.0 in JPCP 8 in 21A
B	1.5 in SM-12.5D 3.0 in IM-19.0A 4.0 in BM-25A 8 in 21B	9.0 in CRCP 8 in 21B	9.0 in JPCP 8 in 21B
C	2.0 in SMA-12.5 (76-22) 2.5 in SMA-19 (70-22) 7.5 in BM-25D 2 in OGDL 6 in CTA	10 in CRCP 2 in OGDL 6 in CTA	11.5 in JPCP 2 in OGDL 6 in CTA

SM = Surface Mix; IM = Intermediate Mix; BM = Base Mix; SMA = Stone Matrix Asphalt; CRCP = continuously reinforced concrete pavement; JPCP = jointed plain concrete pavement; OGDL = open graded drainage layer; CTA = cement treated aggregate.

Limitations

The adjustments to local calibration coefficients presented in this report are highly dependent on the setting in which they were developed. The pavement structure, materials, traffic, and climate inputs represent VDOT's current practice when using Pavement ME Design; the calibration coefficients may need to be revisited as these input parameters are further developed and improved upon. In addition to the ongoing work by VDOT to augment Pavement ME Design, changes are anticipated at a national level to enhance the models that will require VDOT to recalibrate. Some future changes include revisions to the global calibration coefficients for concrete pavements, improvements to the top-down asphalt cracking model, adjustment of cement-stabilized material failure models, and revision of the reflective cracking models.

The PMS data provide convenient, consistent information on the pavement performance; they do not provide the detailed project information that would be more helpful in comparing the measured to predicted distress data in Pavement ME Design. One reason a forensic investigation of calibration sites was not included in the scope of this study was that many of the sites either had been rehabilitated or indicated a minimal amount of distress. In addition, some categories of pavement type, base type, pavement thickness, distress level, and age may be underrepresented in this study. Expanding the pool of project sites used for calibration can help provide more robust calibration coefficients. In some cases, such as JPCP, this may require partnering with surrounding states to calibrate models if enough sites in Virginia are not available.

CONCLUSIONS

- *The local calibration values identified in this study offer improved pavement performance predictions compared to the global calibration coefficients, but they should be used with caution with pavement types that differ from those included in this study and distress levels that are beyond the measured values in this study.*

- *Rutting model local calibration coefficients remove an overprediction from the global model; the adjusted values show no bias and lower the standard error of the estimate within an acceptable range.*

- *The global model for bottom-up fatigue cracking underpredicts the Virginia dataset, although most sites had very little fatigue cracking damage measured and predicted. Local calibration removes the bias and maintains a reasonable standard error of the estimate. Bottom-up fatigue cracking model local calibration coefficients remove an underprediction from the global model and maintain a reasonable standard error of the estimate.*

- *The asphalt pavement IRI model local calibration coefficient corrects an underprediction from the global model but increases the standard error of the estimate.*

- *The CRCP punchout local calibration coefficient removes an overprediction bias in the global model and decreases the standard error of the estimate.*

- *The CRCP IRI local calibration coefficient removes a large underprediction from the global model but also increases the standard error of the estimate.*

- *Global calibration coefficients for JPCP transverse cracking and faulting are considered appropriate despite too few project sites to evaluate. Study findings from Arizona, Utah, Colorado, and Wyoming verified that the global coefficients were acceptable (Von Quintus et al., 2013).*

RECOMMENDATIONS

1. *VDOT's Materials Division should incorporate the locally adjusted calibration coefficients shown in Table 18 into Pavement ME Design when analyzing or designing asphalt or CRCP pavement structures.*

Table 18. VDOT Pavement ME Design Coefficients Adjustments From Local Calibration

Model	β_{r1}	β_{S1} (fine subgrade)	β_{S1} (granular subgrade)	β_{f1}	C_1	C_2	C_3
Asphalt pavement permanent deformation	0.687	0.153	0.153				
Asphalt pavement bottom-up cracking				42.87	0.3190	0.3190	
CRCP punchouts							114.76

Coefficients not noted should remain at the global default values included with Pavement ME Design, Version 1.3. CRCP = continuously reinforced concrete pavement.

2. *VDOT's Materials Division should use the design requirement values shown in Table 19 as a starting point for evaluating the pavement designs using Pavement ME Design.* Further assessment of these values through design comparisons, discussion, and training with district pavement designers is necessary prior to establishing values for VDOT's pavement design procedure guidelines.

Table 19. Reliability Level, Design Life, and Performance Target Recommendations for VDOT's Use With Pavement ME Design

Pavement ME Design Requirement Parameter	Design Life (years)	Highway Classification			
		Interstate	Divided Primary	Undivided Primary	Secondary
Reliability Level		95	90	90	85
Performance Measure					
Asphalt pavement—Total permanent deformation (in)	15	0.26	0.26	0.26	0.26
Asphalt pavement—Bottom-up fatigue cracking (%)	30	6	6	6	6
Asphalt and concrete pavement—IRI (in/mi)	15	140	140	140	140
CRCP punchouts (count/mile)	30	6	6	6	6

IRI = International Roughness Index; CRCP = continuously reinforced concrete pavement.

3. *VCTIR, with the assistance of VDOT's Materials Division, should develop a database of project details for future local calibration studies.* This database should be updated as new sites are identified within a sampling template and additional years of field distress data are available.

BENEFITS AND IMPLEMENTATION

Benefits

Applying the ME design procedure can help develop pavement structures that are optimized to provide the necessary performance in a cost-effective manner. The Indiana DOT estimated an average cost savings of $450,000 on new construction projects (Nantung, 2010). Having a pavement design tool that predicts performance in measurable quantities that VDOT already uses for network level distress measures can also help VDOT develop better estimates for future rehabilitation needs for a pavement section. This application can begin to improve the data available for planning and forecasting future pavement needs. In addition to design work, forensic pavement investigations can be performed with locally calibrated Pavement ME Design analysis and use project-specific materials testing information to obtain better estimates of pavement performance.

Implementation

The local calibration coefficient and design value recommendations from this study will be implemented by VDOT's Materials Division by incorporating them into VDOT's *Pavement ME User Manual*. In addition, the revised manual should be distributed to pavement design staff in VDOT districts along with training on incorporating the revised values into Pavement ME Design and developing a pavement design using the software.

The ME design method should be used with the revised inputs for comparison designs with VDOT's current design procedure to continue to evaluate the new design process and ensure that the output is consistent with experience and engineering judgment. Prior to switching to a ME design procedure, VDOT will also need to provide training opportunities to external partners. Ongoing research needs related to ME design will continue to be identified and managed through VCTIR as appropriate. The data used for this study will be organized and maintained to provide a basis for future revisions to local calibration coefficients that will be necessary as VDOT gains more experience with ME design.

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