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Implementation of AASHTOWare Pavement ME Design Software for Asphalt Pavements in Kansas

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Abstract

Many highway agencies are transitioning from the 1993 AASHTO pavement design guide to the AASHTOWare Pavement ME Design (PMED). Pavement performance models embedded in the PMED software need to be calibrated for new and reconstructed hot-mix asphalt (HMA) pavements. Twenty-seven newly constructed HMA pavements were used to calibrate the prediction models—twenty-one for calibration and six for validation. Local calibration for permanent deformation, top-down fatigue cracking, and the International Roughness Index (IRI) models was done using the traditional split-sample method. Comparison with the results from the 1993 AASHTO design guide for ten new HMA pavement sections with varying traffic levels was done. The results show that the thicknesses obtained from locally calibrated PMED are within 1 inch of the AASHTO 1993 design guide prediction for low to medium-low traffic. For sections with high traffic level, the 1993 AASHTO design guide yielded higher thickness than PMED. The PMED implementation strategies adopted in Kansas and relevant concerns are discussed. Finally, an automated calibration technique has been proposed to help highway agencies to perform periodic in-house calibration of the performance models.

US state highway agencies have predominantly been using the American Association of State Highway Transportation Officials (AASHTO) Guide for Design of Pavement Structures (1993 version) and the associated DARWin software to design highway pavements. Many agencies, including the Kansas Department of Transportation (KDOT), are planning to adopt the recently developed mechanistic-empirical pavement design guide (MEPDG) for new and reconstructed flexible pavements. The MEPDG design approach has been incorporated in proprietary pavement design software, commonly known as AASHTOWare Pavement ME Design (PMED). Version 2.5 is the latest version of the AASHTOWare series.

After the release of MEPDG several states have attempted implementation of the software for routine pavement design. NCHRP synthesis 457 conducted a survey in 2014 among fifty-seven highway transportation agencies across North America and reported that three agencies had implemented MEPDG approaches and forty-six agencies were evaluating MEPDG models (*1*). The technical report of the AASHTO Pavement ME national user group in 2017 stated that nine highway agencies (out of twenty-one responding) have successfully implemented PMED software for asphalt pavements (*2*). The report also listed several challenges faced by the state highway agencies in implementing the PMED software. Local calibration and verification of PMED performance models topped the list. Other challenges include availability of performance data, characterizing bound and unbound layer material properties, compatibility of performance measures and threshold criteria, and so forth.

One of the prerequisites of implementing the PMED software for routine design is to calibrate and validate the performance models to local conditions (*3*). In addition, truck-traffic characterization, developing a material inputs database, and establishing performance criteria and distress-wise reliability levels are also required (*4*). Nantung et al. proposed a six-step MEPDG implementation plan for the Indiana Department of Transportation (*5*). These steps include reviewing

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existing state-of-knowledge in pavement engineering and management, documenting hierarchical design input parameters, reviewing long-term pavement performance data pertinent to the agency, assessing laboratory and field investigation required for higher-level design inputs, executing local calibration and validation of MEPDG distress models, and providing necessary technology and training to implement ME design approaches at the district, local agency, and contractor levels (*5*).

The primary objective of this research is to develop locally calibrated pavement performance models embedded in the AASHTOWare PMED software for new asphalt concrete (AC) pavements in Kansas. This paper discusses the issues that were encountered both in the local calibration process as well in the implementation of the AASHTOWare PMED software.

Study Overview

An earlier study in Kansas completed the verification and local calibration of performance models in the pavement ME software for new/reconstructed pavements. This study was undertaken to facilitate implementation of AASHTOWare PMED software for new AC pavement design in Kansas. Local calibration was carried out for permanent deformation, top-down fatigue cracking, and the International Roughness Index (IRI) model. Level 2 truck-traffic volumetric factors were developed from data collected at eleven automatic vehicle classification stations across the state. In addition, traffic load spectra were developed from ten weigh-in-motion stations. A database of level 3 material input parameters was developed for AC pavements from the construction records, quality control and quality assurance database, and as-build construction plans. After local calibration and validation of the PMED software models, comparative analysis was done to determine the differences in design between the AASHTO 1993 and the AASHTOWare PMED software so that the latter could be adopted for routine design in Kansas. The purpose was to investigate whether the PMED-designed thicknesses made sense given historical performance of in-service pavement sections.

One of the concerns about implementation of the PMED software is the need for calibration after any update of distress models. In addition, the PMED software needs to be recalibrated periodically with updated performance data (*6*). In this study, a framework has been presented to automate the calibration and validation processes of the PMED performance models.

Local Calibration of MEPDG Models

The primary goal of distress and IRI model calibration for AC pavements is to reduce or eliminate bias. A biased model in the AASHTOWare PEMD software will produce either over-designed or under-designed pavements (*7*). Twenty-seven AC pavement projects were selected for local calibration and validation. The traditional split-sample method was used to determine the accuracy of the calibrated model and to verify the goodness-of-fit statistics. The traditional split-sampling approach requires random splitting of the sample projects into two subsets (*3*). An 80–20 division of calibration and validation set was used in this study. AASHTOWare PMED version 2.2 (latest version available at the time of this analysis) was used for calibrating the performance models. Table 1 lists the general characteristics of the projects used for calibration.

AC Layer Properties

PMED software requires dynamic modulus (E*), creep compliance, and indirect tensile strength of the asphalt mix for level 1 input. Dynamic shear modulus (G*) and phase angle (δ) values of the asphalt binder are also required to generate dynamic modulus master curves for the asphalt mixes. As these data were not available for the selected projects, the researchers extracted level 3 input values (aggregate gradation, binder grade, and mix volumetric properties) from the mix design database of KDOT for the surface, intermediate, and base AC layers.

The unit weight of hot-mix asphalt (HMA) mix for all projects was taken as 140 pcf (2,243 kg/m^3). The air void of the surface course mixture was taken to be 8% whereas that for the binder and base course mixtures was 7%. The Poisson's ratio for the HMA mix was assumed to be 0.35. Project-specific asphalt binder grade, aggregate gradation, and mix volumetric properties were extracted from the KDOT construction management database. Project-specific asphalt content and binder grade information are provided in Table 2 for each HMA layer.

Unbound Layer Properties

Two of the twenty-seven projects selected for this study were designed with unbound base course layers (AB-3). The rest of the projects were full-depth AC pavements. The resilient modulus (M_R) of the base course layer was set at 31,000 psi (214 MPa).

All projects except one were built with 6-inch (150 mm) treated subgrade layer on top of natural subgrade. In most cases this 6-inch (150 mm) layer was treated with lime. In a few projects, fly-ash and mechanically stabilized layers were used. The resilient modulus of the mechanically stabilized layer was set at 25,000 psi (172 MPa). For the lime/fly-ash-treated layers, the M_R was calculated according to the formula below (*8*):

Table 1. Selected Projects for New Flexible Pavement Local Calibration and Validation

No.	Route	County	Length (mile)	Pavement cross section: HMA Surface course	HMA intermediate course	Base course	Treated subgrade
1	US-40	Douglas	1.20	1.5”	2.5”	8.7” HMA	6” LTSG
2	US-50	Finney	9.48	1.5”	2.5”	11.0” HMA	6” LTSG
3	US-54	Butler	8.22	1.5”	2.5”	8.7” HMA	6” LTSG
4*	US-54	Seward	3.87	1.5”	2.5”	13.4” HMA	6” FATSG
5	US-56	Morton	2.12	1.5”	2.5”	9.4” HMA	6” SUBMOD
6	US-56	Stevens	2.55	1.5”	2.5”	9.4” HMA	6” FATSG
7	US-69	Cherokee	2.99	1.5”	2.5”	6.3” HMA	6” LTSG
8*	US-73	Atchison	4.14	1.5”	2.5”	6.5” HMA	6” LTSG
9	US-73	Leavenworth	2.47	1.5”	2.5”	6.5” HMA	6” LTSG
10	US-75	Brown	6.63	1.5”	2.5”	11.8” HMA	6” LTSG
11	US-77 (1)	Butler	12.71	1.5”	2.5”	10.2” HMA	6” LTSG
12	US-77 (2)	Butler	9.56	1.5”	2.5”	8.7” HMA	6” LTSG
13	US-77 (3)	Butler	7.23	1.5”	2.5”	6.5” HMA	6” LTSG
14	US-160	Crawford	4.85	1.5”	2.5”	7.1” HMA & 6.0” AB-3	N/A
15	US-183	Rooks	5.92	1.5”	2.5”	9.4” HMA	6” FATSG
16	US-283	Graham	13.40	1.5”	2.5”	9.4” HMA	6” SUBMOD
17	US-283	Norton	10.50	1.5”	2.5”	7.1” HMA	6” SUBMOD
18*	US-283	Trego	11.46	1.5”	2.5”	8.7” HMA	6” LTSG
19	K-7 (1)	Crawford	4.97	1.5”	2.5”	7.9” HMA	6” LTSG
20	K-7 (2)	Crawford	6.02	1.5”	2.5”	6.3” HMA	6” LTSG
21*	K-7	Doniphan	5.79	1.5”	2.5”	4.7” HMA & 11.0” AB-3	6” FATSG
22	K-18	Geary	2.00	1.5”	2.5”	10.2” HMA	6” LTSG
23	K-27	Morton	2.67	1.5”	2.5”	7.0” HMA	6” SUBMOD
24*	K-27	Sherman	4.19	1.5”	2.5”	8.7” HMA	6” SUBMOD
25*	K-39	Wilson	1.96	1.5”	2.5”	5.5” HMA	6” SUBMOD
26	K-99	Elk	8.80	1.5”	2.5”	7.1” HMA	6” LTSG
27	K-156	Ellsworth	12.77	1.5”	2.5”	11.0” HMA	6” LTSG

Note: *validation project; AB-3 = aggregate base course; LTSG = lime-treated subgrade; FATSG = fly ash-treated subgrade; SUBMOD = mechanically stabilized subgrade.

$$\mathrm{LTSGM_R(psi)} = (2.03\mathrm{xsoilM_R, psi}) + 225 \quad (1)$$

Beneath the lime-treated subgrade (LTSG) layer, a 12-in. (300 mm) compacted subgrade layer was used for all projects. AASHTOWare PMED software requires soil resilient modulus and gradation data. The resilient modulus data were provided by KDOT. Subgrade soil gradation and Atterberg limit values were obtained from the soil survey reports of the Soil Conservation Service for each county.

Calibration of the PMED Performance Models

KDOT collects four types of distress data on flexible pavements: (i) total pavement permanent deformation, (ii) load-related fatigue cracking, (iii) transverse cracking, and (iv) IRI. In addition to the four types of distresses collected, KDOT tracks overall performance level of the existing flexible pavement based on examination of all four collected distresses. KDOT recognizes all load-related cracking as top-down cracking, because most of the existing full-depth flexible pavements are roughly 13–30 inches (325–750 mm) thick, which makes the occurrence of bottom-up fatigue cracking unlikely. Historically, no bottom-up fatigue cracking has been observed on existing full-depth flexible pavements. Thus, local calibration of the bottom-up fatigue cracking model was not conducted.

Calibration of the Permanent Deformation Model. The existence of local bias was determined by conducting hypothesis testing for the total permanent deformation model with the globally calibrated coefficients. The globally calibrated model showed significant bias in the paired *t*-test and the null hypothesis was rejected at a 95% confidence level.

To eliminate local bias in the rutting model, the AASHTOWare software was run numerous times to adjust the model coefficients. The combination of coefficients that resulted in the smallest sum of square of error (SSE) between measured and predicted rutting was finally selected for the rutting model. The results of paired *t*-test with globally and locally calibrated coefficients are shown in Table 3.

Table 2. Project-Specific Material Properties for the Selected Flexible Pavements

No.	Route	County	HMA Layer PG binder grade and % binder content by volume					
			Surface course		Binder course		Base course	
1	US-40	Douglas	PG 76-28	11.5	PG 76-28	9.5	PG 64-22	9.1
2	US-50	Finney	PG 70-28	11.8	PG 64-22	11	PG 64-22	11
3	US-54	Butler	PG 64-28	12.3	PG 64-28	10.7	PG 64-22	10.1
4*	US-54	Seward	PG 70-28	10.9	PG 70-28	9.6	PG 64-22	9.8
5	US-56	Morton	PG 70-28	11.2	PG 64-22	9.2	PG 64-22	9.6
6	US-56	Stevens	PG 76-28	13.1	PG 64-22	10.5	PG 76-28	9.9
7	US-69	Cherokee	PG 64-28	12	PG 64-28	10.9	PG 64-22	11.1
8*	US-73	Atchison	PG 64-28	10.9	PG 64-28	9.4	PG 64-22	9.3
9	US-73	Leavenworth	PG 64-28	10.8	PG 64-28	9.5	PG 64-22	9.2
10	US-75	Brown	PG 70-28	11.6	PG 64-22	11	PG 70-22	10.1
11	US-77 (1)	Butler	PG 64-28	12.2	PG 64-28	9.9	PG 64-28	9.9
12	US-77 (2)	Butler	PG 64-28	12.4	PG 64-28	9.9	PG 64-28	9.7
13	US-77 (3)	Butler	PG 64-28	11.6	PG 64-28	10.2	PG 64-22	9.2
14	US-160	Crawford	PG 64-22	10.0	PG 64-22	9.2	PG 64-22	9.2
15	US-183	Rooks	PG 64-28	12.3	PG 64-28	10.3	PG 64-28	10.1
16	US-283	Graham	PG 64-28	11.7	PG 64-22	10.3	PG 64-22	10.3
17	US-283	Norton	PG 64-28	11.0	PG 64-28	9.1	PG 64-28	9.2
18*	US-283	Trego	PG 64-28	11.9	PG 64-28	10.9	PG 64-22	10.5
19	K-7 (1)	Crawford	PG 64-28	10.3	PG 64-28	10.9	PG 64-22	10.9
20	K-7 (2)	Crawford	PG 64-28	12.4	PG 64-28	11.1	PG 64-22	10.5
21*	K-7	Doniphan	PG 64-28	11.6	PG 64-28	9.8	PG 64-22	9.9
22	K-18	Geary	PG 70-28	11.9	PG 70-28	10.3	PG 64-22	10.1
23	K-27	Morton	PG 58-28	11.9	PG 58-28	9.0	PG 58-28	9.0
24*	K-27	Sherman	PG 64-28	11.6	PG 64-28	9.8	PG 64-22	9.9
25*	K-39	Wilson	PG 64-28	13.6	PG 64-28	9.7	PG 64-28	9.7
26	K-99	Elk	PG 64-28	12.9	PG 64-22	10.3	PG 64-28	9.8
27	K-156	Ellsworth	PG 70-28	11.9	PG 70-28	10.3	PG 64-22	10.1

Table 3. Statistical Analysis Results for Rutting Model

	Bias (in.)	S_e (in.)	R^2	*P*-value	Hypothesis, Ho: $\sum$(Measured-Predicted)=0
Global Calibration	−17.9	0.06	0.29	<0.0001	Rejected
Local Calibration	−0.81	0.05	0.49	0.13	Accepted
Validation	−0.47	0.49	0.34	0.063	Accepted

Results in Table 3 show that the standard error of estimate (S_e) has decreased slightly after local calibration (0.05 in or 1.25 mm). The local calibration guide for the PMED software recommends that the S_e of the permanent deformation model be within 0.10 in (2.5 mm). Figure 1 presents the measured versus predicted total rutting with global and local coefficients.

Calibration of the Load-Related Cracking Model. KDOT does not consider any load-related cracking to initiate from the bottom of the HMA layer. The reason behind such philosophy is the predominant construction of thick full-depth HMA pavements across the state. The KDOT Pavement Management Information System (PMIS) database also does not differentiate between bottom-up or top-down cracks. All load-related cracks, therefore, were considered to be top-down, and only the longitudinal cracking model was calibrated.

The extracted top-down cracking data from the PMIS database showed a high degree of variability. The average top-down cracking on these projects varied from 13,000 ft/mile (2,462 m/km) to none. To reduce the variability, the top-down cracking model was calibrated excluding projects with no top-down cracking. Thus, nine out twenty-seven projects were omitted for calibrating the top-down cracking model.

The globally calibrated top-down fatigue cracking model showed significant bias in the paired *t*-test and the null hypothesis was rejected. To reduce bias the β_{f1}

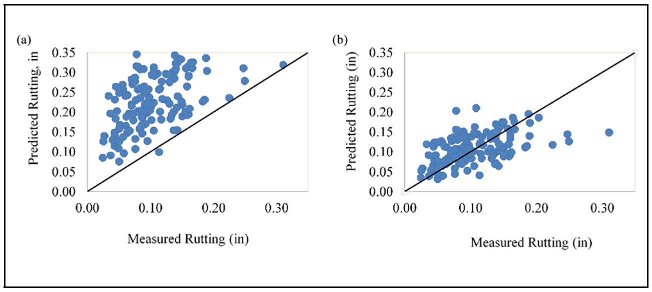

Figure 1. Measured versus predicted rut depth with (*a*) global coefficients and (*b*) local coefficients.

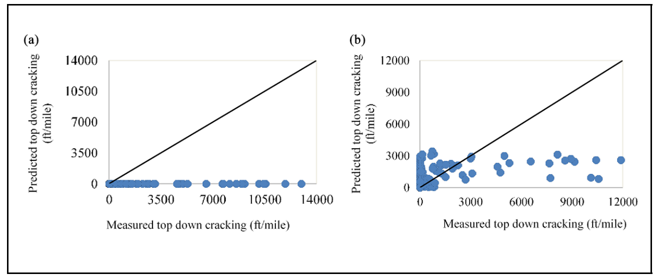

Figure 2. Measure versus predicted top-down cracking with (*a*) global coefficients and (*b*) local coefficients.

Table 4. Statistical Analysis Results of the Top-down Cracking Model

Calibration	Bias (ft/mile)	S_e (ft/mile)	R^2	*P*-value	Hypothesis, Ho: $\sum$(Measured-Predicted)=0
Global	174,395	2,899	0.21	<0.0001	Rejected
Local	59,986	2,750	0.36	0.061	Accepted

parameter in the fatigue ($N_{f\text{-}HMA}$) model and the C_1 & C_2 coefficients of the top-down cracking transfer function were adjusted using the generalized reduced gradient optimization technique in Microsoft Excel Solver. Table 4 shows the summary statistics before and after calibration.

After calibration, the bias was reduced from 174,395 ft/mile (33,030 m/km) to 59,986 ft/mile (11,361 m/km). The calibrated model still showed a very high standard error of 2,750 ft/mile (521 m/km). It should be noted that the local calibration guide for the AASHTOWare software recommends that S_e of the top-down cracking model should be within 600 ft/mile (114 m/km). However, the null hypothesis in the *t*-test was accepted. This model could not be validated because all projects were used for calibration. Figure 2 presents the measured versus predicted top-down cracking with global and local coefficients.

Calibration of the Thermal Cracking Model. In this study, only thirteen out of twenty-seven projects showed transverse cracking. Also, Level 1 input (creep compliance and indirect tensile strength) data for this study were not available. An attempt was made to calibrate the AC thermal cracking model with Level 3 inputs. As the AASHTOWare PMED software did not predict transverse cracking for any project with the global coefficients, calibration of the model to force the predicted data to match the measured data yielded a model that generated high AC thermal cracking for all projects. Thus it was decided that the AC thermal cracking model should be calibrated regionally, and only regions showing AC transverse cracking will be included in the calibration process in the future.

Calibration of the IRI Model. The IRI model was defined in the Excel spreadsheet as a summation of rut depth, load-related cracking, thermal cracking, and site factor multiplied with coefficient C1, C2, C3, and C4, respectively. The residual errors were obtained as the difference between the measured and predicted IRI. Microsoft Solver was used to adjust the coefficients to minimize SSE for the full dataset. These adjusted coefficients were used as the calibrated coefficients of the IRI model in the AASHTOWare PMED software, and a paired *t*-test was conducted between the measured and the predicted data. Table 5 shows the summary statistics before and after calibration.

Results in Table 5 show that S_e increased slightly after local calibration of the IRI model. Nonetheless S_e of the IRI model was well within the range of the calibration guide (17 in/mile or 0.26 m/km).

Comparison of New Flexible Pavement Design Using AASHTO 1993 and Locally Calibrated PMED Software

Many studies have been conducted to compare AC pavement designs using the PMED software and the AASHTO 1993 guide. In most cases, both designs were applied to a pavement structure in a project. However, such direct comparison could be misleading as the input

Table 5. Statistical Analysis Results of the IRI Model

	Bias (in./mile)	S_e (in./mile)	R^2	*P*-value	Hypothesis, Ho: ∑(Measured-Predicted)=0
Global calibration	−1487.6	9.72	0.24	0.043	Rejected
Local calibration	−215.8	10.13	0.28	0.077	Accepted
Validation	−8.65	14.7	0.911	0.30	Accepted

Table 6. General Features of Projects for Thickness Comparison

Project no.	Route	County	Subgrade soil type	Subgrade modulus (psi)	Initial AADT	% Truck	Traffic level
1	K-99	Wabaunsee	A-6	2,600	343	55	Low
2	US-281	Russell	A-6	2,600	300	50	Low
3	US-77	Geary	A-6	3,000	396	60	Low-medium
4	K-383	Norton	A-6	3,600	378	60	Low-medium
5	K-14	Reno	A-6	4,174	684	60	Low-medium
6	US-56	Gray	A-7-6	2,800	893	55	Low-medium
7	US-50	Ford	A-7-6	3,000	1,660	60	Medium-high
8	US-50	Gray	A-6	4,600	1,580	55	Medium-high
9	I-70	Thomas	A-6	5,000	3,991	50	High
10	I-70	Gove	A-6	5,400	3,990	60	High

parameters and design criteria required by two design methods are quite different (*9*). Carvalho et al. used an alternative approach to evaluate whether both methods consistently predict performances for a range of design conditions. Several pavement sections were first designed using the AASHTO 1993 design guide and then reanalyzed with the PMED software for the same design period (*10*).

In this study, a side-by-side comparison of the AASHTO 1993 guide and PMED software was made. Ten AC pavement sections were used in this analysis. The 1993 AASHTO-designed sections were reanalyzed using the PMED software to predict pavement performance. Based on the 10-year cumulative 18-kip equivalent single axle loads (ESALs), the study sections were divided into four traffic categories: high (>6 million ESALs), high-medium (3–6 million ESALs), low-medium (1–3 million) and low (<1 million ESALs). Table 6 lists the general features of the projects selected for thickness comparison.

Inputs

To make a better comparison of the two design procedures, it is important that the input parameters are somewhat equivalent. Common inputs for both design procedures are design traffic, reliability, design period, and subgrade properties. Specific design inputs of the AC pavement design using the 1993 AASHTO Guide are HMA layer thickness, structural layer coefficient, subgrade resilient modulus, and drainage coefficient. AASHTOWare PMED version 2.5 was used in this analysis. The key input parameters are traffic, climate, structural details, and layer material properties.

Design Traffic. In the AASHTO 1993 design guide, design traffic is estimated based on the cumulative expected 18-kip (80-kN) ESALs in the design lane over the design period. For AASHTOWare PMED, site-specific (Level 1) traffic inputs, such as, average annual daily truck-traffic data, operational speed, number of lanes, vehicle class distribution, traffic growth rate, and the percentage of trucks in the design direction or lane were used. All other traffic inputs were chosen from the list of the AASHTOWare default values. Kansas regional traffic inputs (Level 2) were used for monthly adjustment factors, axles per truck, and axle load spectra.

Serviceability. Present serviceability index (PSI) quantifies pavement performance in the 1993 AASHTO Guide. An initial PSI of 4.2 is typically used for AC pavements in Kansas. The terminal PSI was 2.5 for all projects in this study.

Failure Criteria. In this study, KDOT-recommended failure criteria were used for each distress type. These failure criteria are based on the traffic level and functional class of highway. They were derived by analyzing historical performance data extracted from the KDOT PMIS database.

Table 7. KDOT Distress-Wise Failure Criteria and Design Reliability Level

Distress	Functional class	Failure criteria	% Reliability
Terminal IRI	Interstate	160 in./mile	85
	Principal arterial	180 in./mile	75
	Minor arterial	190 in./mile	65
	Collector & local	200 in./mile	60
AC top-down fatigue cracking	Interstate	1500 ft./mile	95
	Principal arterial	2000 ft./mile	85
	Minor arterial	2000 ft./mile	75
	Collector & local	2000 ft./mile	70
AC bottom-up fatigue cracking	Interstate	10% lane area	95
	Principal arterial	20% lane area	85
	Minor arterial	25% lane area	75
	Collector & local	30% lane area	70
AC thermal cracking	Interstate	750 ft./mile	85
	Principal arterial	750 ft./mile	75
	Minor arterial	750 ft./mile	65
	Collector & local	750 ft./mile	60
Permanent deformation	Interstate	0.35 in.	95
	Principal arterial	0.45 in.	85
	Minor arterial	0.50 in.	75
	Collector & local	0.60 in.	70

Reliability. The AASHTO Guide reliability level is a function of functional classification and intended use of the roadway. It is applied in the design by selecting a standard normal value multiplied by a standard deviation. For this study, a standard deviation value of 0.45 was used for all projects.

The reliability concept of MEPDG, which is more sophisticated than 1993 design guide reliability, allows a designer to design a pavement with an acceptable level of distress for each predicted distress type at the end of design life. In this study, KDOT-recommended reliability level based on traffic level and functional class of highway was used. These design reliability levels were established based on the engineering experience and state-of-the-practice in Kansas. Table 7 presents the KDOT distress-wise design reliability level and failure criteria established for the AASHTOWare PMED software analysis.

Structural Layer Coefficient. KDOT-recommended composite structural coefficient of 0.367 was used for all projects in this study.

Drainage Coefficient. Depending on whether the subsurface drainage is present or not, a value of 1.0 or 1.2 is typically used as the drainage coefficient (C_d) in Kansas. A value of 1.0 was used in this study.

Climate. Because PMED version 2.5 was used, modern-era retrospective analysis for research and application (MERRA) climatic files were available. Among all climatic inputs for PMED, the coordinates (latitude and longitude) and elevations for all pavement sections were given inputs as site-specific values, or Level 1 inputs.

Structural Details. For this analysis, the baseline pavement structure for new pavement design consisted of a full-depth HMA construction over a 6-in. (150 mm) LTSG or fly-ash-treated subgrade (FATSG). The last layer is the natural compacted subgrade. Typical Kansas full-depth HMA sections usually have a 1.5-inch (37.5 mm) surface course and a 2.5-inch (62.5 mm) intermediate course followed by the base course.

Comparison Results

Initially the pavement sections were designed using the 1993 AASHTO design guide. Then these sections were reanalyzed using the AASHTOWare PMED software. Thickness design in AASHTOWare was performed based on a strategy that, if a section passed all failure criteria for the smoothness (IRI) and other distresses, the pavement thickness would be reduced by 0.5 inch (12.5 mm), and the analysis will be repeated. This process was continued until the section failed to pass one of the failure criteria. Both local calibration coefficients for Kansas and national calibration coefficients of the prediction models were used in this study to assess the variations in the pavement design.

Design Thickness Comparison

The thicknesses of the projects obtained from the AASHTO 1993 and AASHTOWare ME are presented in Figure 3. The design thicknesses predicted with locally calibrated PMED software were always found to be lower than the 1993 AASHTO Guide. For projects with low and low-medium traffic, this difference is not more than 1 inch. The failure mode for all projects with locally calibrated PMED models was top-down fatigue cracking, whereas the failure mode of all projects with globally calibrated factors was bottom-up fatigue cracking.

To investigate performance of these projects designed with the 1993 AASHTO method, the sections were reanalyzed using the PMED software. When the 1993 AASHTO Guide-designed thicknesses are used, all projects satisfied the failure criteria for the design period when locally calibrated models were used. No significant rutting, thermal, or bottom-up fatigue cracking was predicted. The highest top-down fatigue cracking (1,375 ft / mile or 260 m/km) was predicted for K-383 in Norton County. However, when globally calibrated models were used, both US-50 projects in Ford and Gray Counties failed in bottom-up fatigue cracking at the specified

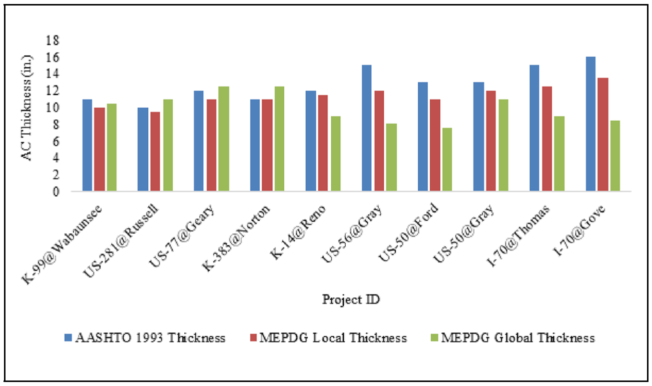

Figure 3. Comparison of AC thickness using AASHTO 1993 and AASHTOWare ME.

reliability. No significant rutting, thermal, or top-down fatigue cracking was predicted with the globally calibrated PMED models.

Automated Calibration Technique

State highway agencies have long been trying to implement the PMED software for routine pavement design. One of the challenges of implementing the software is that it needs to be calibrated locally to reflect the agency's design and construction practices, materials, and climate. From the inception, the software went through several improvements. These improvements also necessitated recalibration of the performance models in the software (*11*). As new information is constantly becoming available in the form of performance data and additional test results, the performance models in PMED must be continually verified to see if recalibration is needed (*12*). KDOT has a long-term concern regarding the time, effort, and resources that would be needed each time to repeat the local calibration. In this study, a framework has been proposed to automate the calibration and validation processes of the PMED software.

Methodology

The primary goal of this task is to employ a suitable optimization technique to determine the effective calibration parameters to minimize model bias and standard error. A systematic method has been developed to automatically search for the optimal value of calibration parameters in the prediction model transfer functions.

To calibrate performance model transfer functions using this method, the user must first successfully run the PMED software. The automated technique requires inputs such as number of projects, measured distresses and corresponding distress collection dates. The user also needs to browse the directory where the PMED software outputs are stored. The automated technique then randomly divides the projects into calibration and validation sets. The user can select a split of 80–20 or 70–30 in this division. One of the major challenges in automating the PMED software calibration process is to recognize relevant software outputs for a particular project at a particular time. The developed automated calibration technique has the capability to search PMED output files and identify mechanistic damages and distresses for a project on a particular date. After obtaining relevant damage data from the PMED output files, the application predicts distresses with a set of global model calibration coefficients (Φ_{Global}) to verify the prediction model. A paired *t*-test is then conducted to determine the initial bias between the actual data and the AASHTOWare PMED software predicted data at 95% confidence level. The coefficient of determination (R^2), S_e, and SSE are also recorded. Based on the verification results model calibration is conducted. The optimization technique is applied to the calibration dataset to obtain a set of model calibration coefficients (Φ_{Local}) that minimizes the SSE between measured and predicted distresses. Bias, R^2, S_e, and SSE are also generated for the calibrated model. The application also generates graphs for measured versus predicted distress data. The damage versus measured cracking data is also generated for fatigue cracking. At this time, the congruent gradient (CG) optimization technique is used to minimize the SSE between measured and predicted data. The advantage of the CG technique is that it is computationally cheap and converges quickly (*13*). Figure 4 presents key steps carried out in the automated calibration technique.

One of the concerns of applying the automated calibration technique is to identify the bounds of the models coefficients. A simple optimization technique usually searches for a local minimum of the objective function (SSE), whereas the robust optimization technique employed has the capability of finding a global minimum in which the function value is smaller than the local optima (*14*). No study has been conducted until now to identify the lower and upper bounds of the model coefficients in the PMED models beyond which realistic predictions would not be possible. Currently the developed automated technology in this study allows users to select a lower and upper bound of these coefficients.

Figure 5 presents the top-down cracking model verification and calibration results completed with the automated calibration technology using the highway segments listed in Table 1. AASHTOWare PMED version 2.5 was used in this analysis. After local calibration, bias was reduced to –499.7 in/mile (–7.89 m/km). The R^2 value was reported to be 0.42. The null hypothesis in the paired *t*-test was also accepted.

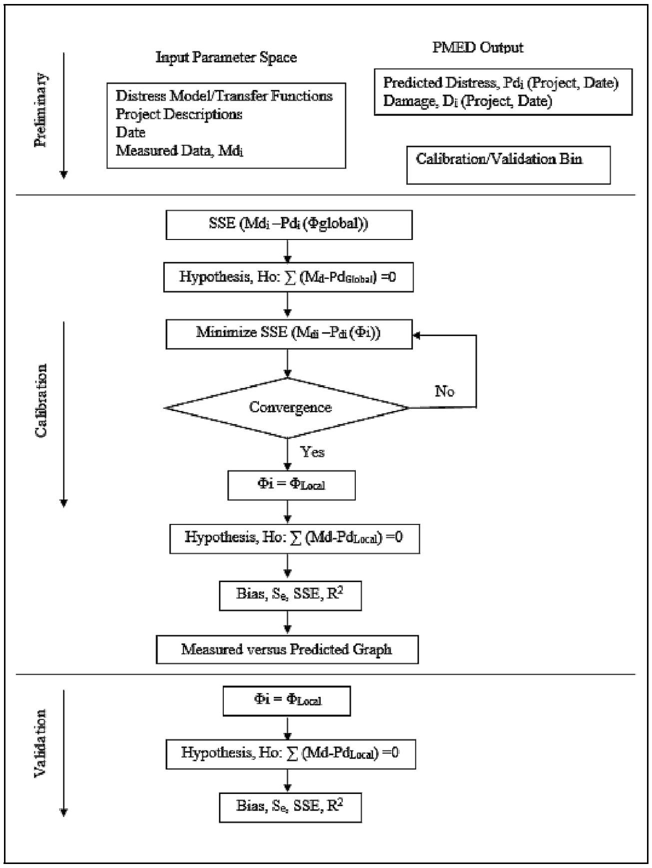

Figure 4. Analysis process for automated calibration technique.

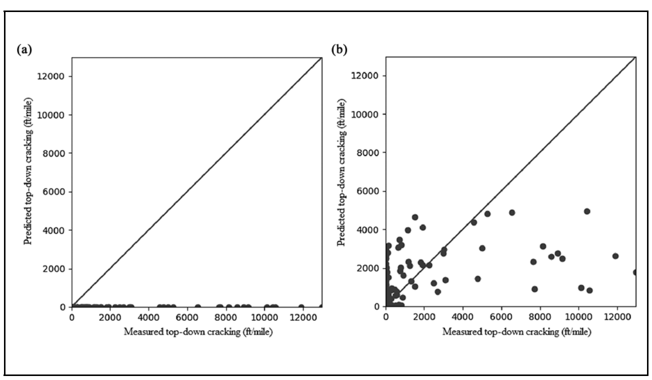

Figure 5. Measured versus predicted top-down cracking with (*a*) global coefficients and (*b*) local calibration coefficients using the automated technique.

Limitation of the Automated Calibration Technique

Currently the automated technique can only calibrate the parameters that do not require multiple simulations of the PMED software. For AC pavements, the permanent deformation, IRI, bottom-up and top-down fatigue cracking transfer functions can be calibrated. However, β_{2r} and β_{2r} coefficients of the permanent deformation model cannot be calibrated using this technology. Similarly, the thermal cracking model cannot be calibrated using this technique. Currently the automated method can only employ the traditional split-sampling technique.

Challenges in Implementation of PMED Software

KDOT has identified some long-term concerns in implementing the AASHTOWare PMED software. The key concerns are listed below:

- Continuous investment of resources for updating traffic data, refinement of measured data, identification of additional data requirements, and laboratory testing for better and consistent results;
- Recognizing differences in design and service life between AASHTO 1993 and PMED methodologies;
- Investigating differences in life-cycle costs between AASHTO 1993 and PMED methods;
- Ensuring design of equivalent structural pavement sections that meet industry expectations for competitiveness;
- Making surface distress measurements and data collection systems compatible with the PMED software for ease of use and efficiency; and
- Training for monitoring and analyzing distress data for the experienced staff in the agency and to update design libraries by integrating new distress data with historical performance data.

Conclusions

This study presents the local calibration process of the performance models of AASHTOWare PMED software and subsequent implementation in design. The thicknesses obtained following the 1993 AASHTO design guide and PMED for ten prospective projects in Kansas were compared. Finally an automated calibration technique has been proposed to help highway agencies in periodic calibration of the performance models. Based on this study, the following conclusions can be made:

- Thermal cracking model in the PMED software could not be successfully calibrated in this study because of extreme variation in thermal cracking in the selected projects.
- For flexible pavement sections with low to low-medium traffic, locally calibrated PMED thicknesses were within 1 inch of what AASHTO 1993 design guide predicted. For sections with high

traffic level, 1993 AASHTO design guide yielded higher pavement thickness than the AASHTOWare ME.

- The automated calibration technique developed in this study could be extremely beneficial to help highway agencies to do periodic in-house calibration of the performance models.
- One of the major challenges in local calibration and implementation of the PMED software is affording the extensive time and effort needed. The agency needs to invest resources for traffic data update, refinement of collected measured data, and laboratory testing for consistent results.

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Author Contributions

Study Conception and design: Shuvo Islam, Mustaque Hossain, Ryan Barrett, and Nat Velasquez, Jr.; data collection: Ryan Barrett, Nat Velasquez, Jr., and Shuvo Islam; analysis and interpretation of results: Shuvo Islam, Avishek Bose, Mustaque Hossain, and Christopher A. Jones; draft preparation of manuscript: Shuvo Islam, Mustaque Hossain, Ryan Barrett, and Nat Velasquez, Jr.

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