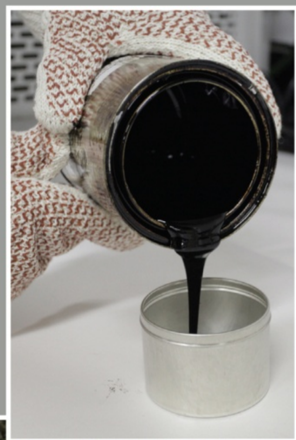

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PAVEMENT ME DESIGN – IMPACT OF LOCAL CALIBRATION, FOUNDATION SUPPORT, AND DESIGN AND RELIABILITY THRESHOLDS

Dr. Nam Tran, P.E.
Dr. Mary M. Robbins
Dr. Carolina Rodezno
Dr. David H. Timm, P.E.

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National Center for Asphalt Technology
NCAT
at AUBURN UNIVERSITY

277 Technology Parkway ■ Auburn, AL 36830

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Dr. Nam Tran, P.E.
Associate Research Professor
National Center for Asphalt Technology

Dr. Mary M. Robbins*
Research Engineer
Ohio Research Institute for Transportation and the Environment
(*Work completed while at National Center for Asphalt Technology)

Dr. Carolina Rodezno
Assistant Research Professor
National Center for Asphalt Technology

Dr. David H. Timm, P.E.
Brasfield and Gorrie Professor of Civil Engineering
Principal Investigator

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DISCLAIMER

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1 INTRODUCTION

The Mechanistic-Empirical Pavement Design Guide (MEPDG) and the accompanying AASHTOWare Pavement ME Design software (hereafter referred to as the ME Design software) have been developed to replace the empirical AASHTO Pavement Design Guides. The MEPDG represents a quantum leap forward from the empirical pavement design procedures (*1, 2*). As indicated in a survey of state agencies conducted in 2013, 43 agencies were evaluating the MEPDG, and 15 agencies planned to implement the new design procedure in the next two years (*3*). The implementation plans of these agencies include, among other elements, important steps for (1) conducting local calibration to account for differences in state practices, policies, and local conditions, and (2) selecting design thresholds and reliability levels for acceptable pavement designs (*4*, *5*). Without properly conducting these important implementation steps, the adoption of the MEPDG will not make the pavement design process "better." In fact, it has been suggested that use of the globally calibrated ME Design software may potentially yield inaccurate asphalt pavement designs (*6*, *7*).

Recognizing the importance of local calibration and selection of design thresholds and reliability levels, this study provides information and evidence to support the need for local calibration of the MEPDG and careful consideration of design thresholds and reliability levels in the implementation process. The results of this research effort are presented in two reports. A previous report was prepared to discuss the general approach to local calibration undertaken by state agencies and to summarize results of their local calibration efforts and recommendations for implementing the locally calibrated MEPDG (*8*). This (second) report presents results of a case study that compares pavement designs conducted with global and local calibration coefficients to illustrate the importance of conducting local calibration of the MEPDG in the implementation process. In addition, it provides results of sensitivity analyses that show the effect of performance criteria, reliability levels, and foundation support on pavement design.

In the following sections, the MEPDG and local calibration methodologies are briefly discussed, followed by case studies and results of sensitivity analyses. Finally, key findings of this research effort are synthesized, and recommendations are offered.

2 MECHANISTIC-EMPIRICAL PAVEMENT DESIGN GUIDE

The MEPDG was developed to design new and rehabilitated pavement structures based on mechanistic-empirical principles. Figure 1 illustrates basic steps for conducting a pavement design using the ME Design software. Based on the inputs and trial pavement design information, the ME Design software "mechanistically" calculates pavement responses (stresses and strains) and uses those responses to compute incremental damage over time. The program then utilizes the cumulative damage to "empirically" predict pavement distresses for each trial pavement structure. A trial pavement structure is accepted as the final design when its predicted pavement distresses meet the design criteria at the selected reliability levels.

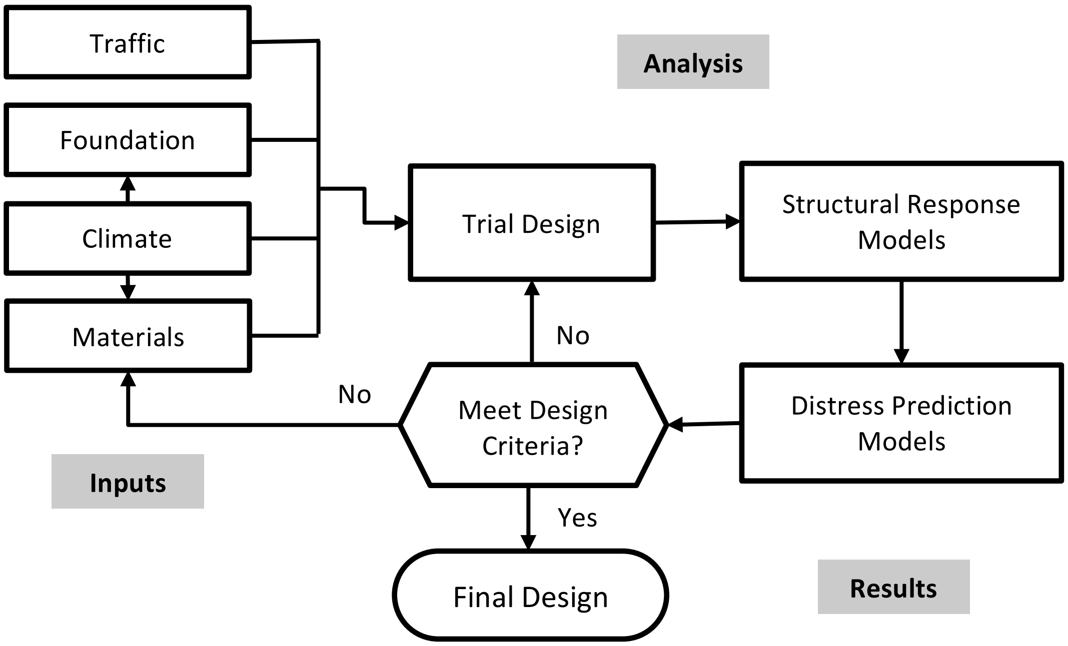

Figure 1. Basic Steps of Pavement ME Design (*7*)

The mechanistic analysis utilizes the Enhanced Integrated Climatic Model, structural response models, and time-dependent material property models. The empirical analysis uses the distress prediction (regression) models, sometimes called transfer functions, representing relationships between the cumulative damage and observed pavement distresses. While the mechanistic models are assumed to be accurate and to correctly simulate field conditions, inaccuracies still exist and affect the results of distress prediction function computations and final distress predictions (*5*). The local calibration process, which is briefly discussed in the following section, is often related to the distress prediction functions, but it essentially addresses the errors of both the mechanistic and empirical analyses.

3 LOCAL CALIBRATION OF MEPDG

Under the NCHRP 1-37A and 1-40 projects, the MEPDG was "globally" calibrated using a representative database of pavement test sites across North America. Most of these test sites have been monitored through the Long-Term Pavement Performance (LTPP) program. They were used because of the consistency in the monitored data over time and the diversity of test sections spread throughout North America. However, construction and material specifications, pavement preservation and maintenance practices, and materials and climatic conditions vary across North America. These differences are not currently considered directly in the ME Design software but are indirectly considered through local calibration in which the calibration coefficients of distress prediction functions in the ME Design software can be adjusted (*5*).

Due to the differences in the local practices and conditions, the distresses predicted by the globally calibrated distress prediction models may have higher bias and/or lower precision when compared with the locally measured pavement distresses. As illustrated in Figure 2, through local calibration, the coefficients of these distress prediction models may be adjusted to improve the bias and precision of the models in the ME Design software. In practical terms, bias is the difference between the 50% reliability prediction and the measured mean. Precision dictates how far the predicted values at a specified design reliability level would be from the corresponding predicted values at the 50% reliability. The locally calibrated models are then validated using an independent set of data. The models are considered successfully validated to the local conditions if the bias and precision statistics of the models are similar to those obtained from model calibration when applied to the validation dataset.

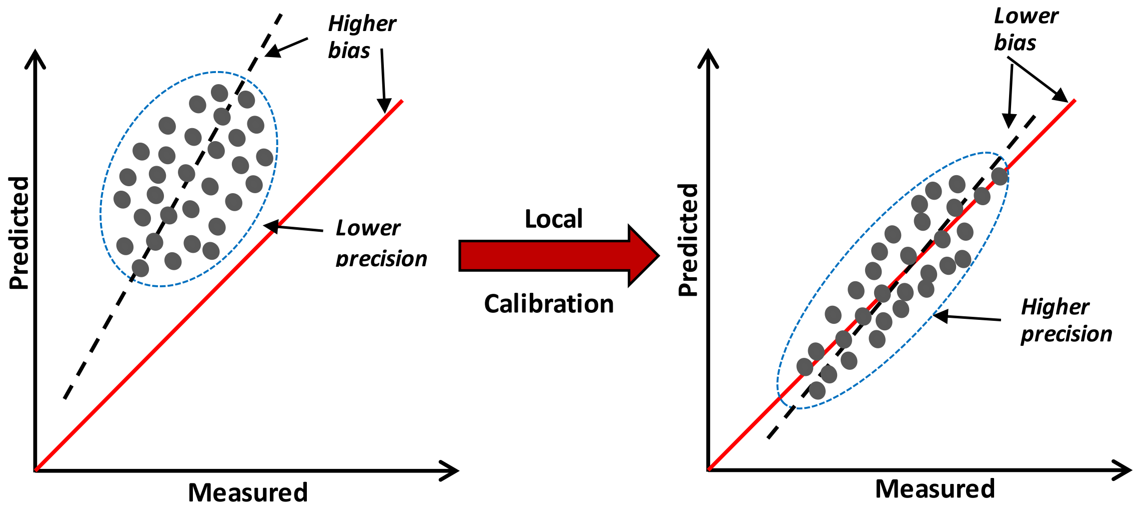

Figure 2. Improvement of Bias and Precision through Local Calibration

Basic steps for calibrating a distress prediction model to improve its bias and precision are shown in Figure 3. The ME Design software with global calibration factors is first conducted to design pavements at 50% reliability using the inputs available from the pavement segments that have been selected for local calibration. The predicted distresses are then compared with the measured distresses of the selected pavement segments, and diagnostic statistics, including R-square, bias, and the standard error of the estimate (S_e), are determined. If the diagnostic statistics are not acceptable, the model calibration coefficients are adjusted, and the analysis is repeated using the adjusted coefficients until the diagnostic statistics are deemed acceptable.

A step-by-step procedure for local calibration is described in the Guide for the Local Calibration of the MEPDG (*5*). The procedure includes detailed steps for (1) selecting and collecting inputs for local calibration, (2) determining local calibration coefficients to improve the bias and precision of each distress prediction function, and (3) reviewing the calibration results to make sure the expected pavement design life is "reasonable" for the performance criteria and reliability levels selected for future use by the agency.

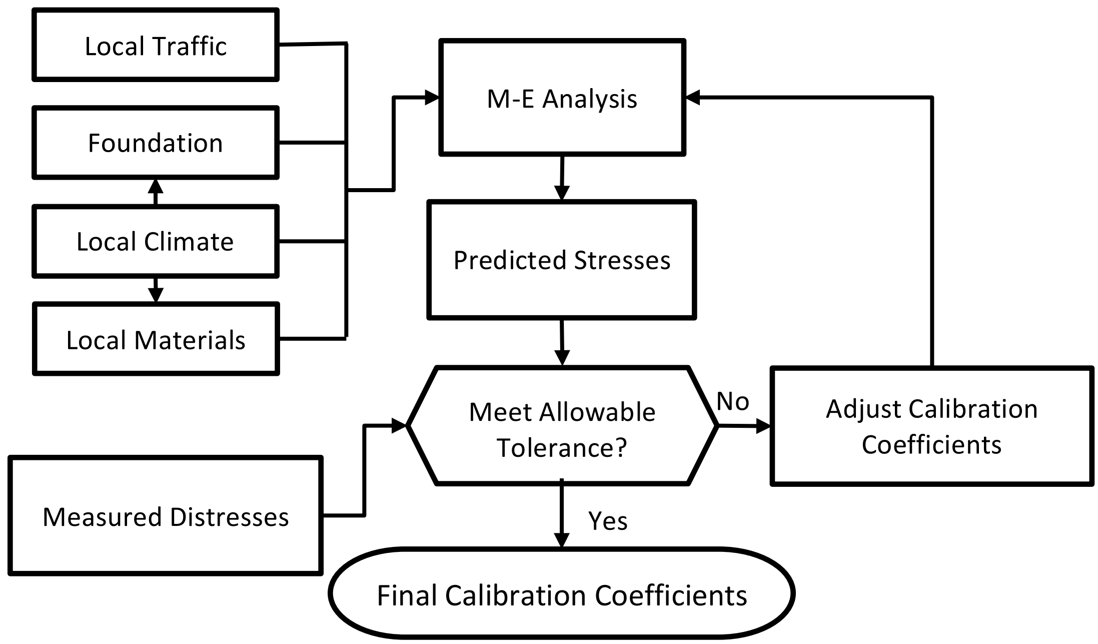

Figure 3. Local Calibration of Pavement ME Design

Since the first release of the MEPDG, several states have sponsored studies to verify and calibrate the MEPDG to local materials and conditions. Detailed results of the local calibration efforts completed through 2015 were reviewed in this study and presented in a previous report (*8*). Despite the availability of the Guide for the Local Calibration of the MEPDG (*5*), many calibration efforts did not follow the step-by-step procedure or the terminology used in the guide, in part due to the timing of the publication (i.e., in 2010) relative to the timing of calibration efforts in each state and the time dedicated to such efforts. In the following section, results of the local calibration efforts in two states, Missouri and Colorado, are summarized in two case studies that compare pavement designs conducted with global and local calibration coefficients to illustrate the impact of local calibration on MEPDG pavement designs.

4 CASE STUDIES

After completing the local calibration of the MEPDG, some states have considered taking the next step of the implementation process, which is adopting the locally calibrated design procedure for some routine pavement designs. Among the state agencies that have completed the local calibration, the Missouri Department of Transportation (MoDOT) and Colorado Department of Transportation (CDOT) have implemented the MEPDG for routine pavement designs. Both MoDOT and CDOT have used the MEPDG to design new asphalt pavements and jointed plain concrete pavements (JPCPs) as well as asphalt and concrete overlays. In the following sections, results of the local calibration sponsored by these agencies are first discussed, followed by performance and reliability limits selected by the states for future designs. Finally, a comparison of pavement designs conducted with global and local calibration coefficients is presented to illustrate how the local calibration coefficients affect the Pavement ME Design results.

4.1 Local Calibration Results

Both MoDOT and CDOT hired a consultant to calibrate the MEPDG distress prediction models to local conditions. The two agencies conducted local calibration using Version 1.0 of the Pavement ME Design software. The pavement types considered for local calibration include (1) new asphalt pavement and asphalt overlay over existing asphalt and concrete pavements; and (2) new JPCP, JPCP overlay over existing asphalt pavement, and unbonded JPCP overlay over existing JPCP. A summary of the local calibration efforts for both new asphalt pavements and asphalt overlays over existing asphalt and concrete pavements is presented in this section. More detailed information about the local calibration efforts in these states can be found in other reports (*8-10*).

The local calibration efforts sponsored by the agencies included two main activities—preparing required information for local calibration and calibrating performance prediction models. Both states determined from previous sensitivity studies the inputs that have a major impact on distress and International Roughness Index (IRI) predictions; therefore, they should be characterized accurately using the highest possible hierarchical input level. Using more accurate inputs would help reduce predicted distress/IRI standard error or deviation, which is a key component of the variability terms used in calculating design reliability. A higher standard error would result in higher predicted distress/IRI at a specified reliability level greater than 50 percent, which in turn would require a thicker pavement structure or better materials, both of which can affect the economy of the pavement.

To prepare inputs essential to the local calibration of performance prediction models, several research efforts were conducted or sponsored by the states to (1) characterize traffic inputs, (2) determine laboratory material properties of typical asphalt mixtures, aggregate bases, and subgrade soils, (3) conduct in-situ testing of pavements, and (4) analyze pavement performance data. Table 1 summarizes the hierarchical input levels used in local calibration in each state. In Table 1, Level 1 inputs require the highest level of accuracy and are laboratory- and/or field-measured data. Level 2 inputs require an intermediate level of accuracy and are determined using procedures (or correlations) similar to those used in the empirical AASHTO Pavement Design Guides. Level 3 inputs require the lowest level of accuracy and are default inputs previously used by state or provided in the ME Design software (*9*, *10*).

In addition to the inputs listed in Table 1, automated and manual distress surveys of the selected pavement sections were conducted by each state for local calibration. The data were used to characterize pavement condition (measured fatigue cracking, transverse cracking, rutting, and IRI). Table 2 provides the number of 500-ft and 1,000-ft pavement sections selected by MoDOT and CDOT, respectively, for local calibration.

Table 1. Input Levels for Local Calibration (*9*, *10*)

Input Group	MoDOT	CDOT
All Traffic Inputs (Except as Noted)	Level 3	Level 3
AADTT	Level 1	Level 1
Axle Load Distribution	Level 1	Level 1
Vehicle Class Distribution	Level 1	Level 1
Truck Wheel Base Percentages	Level 3	Level 1
Climatic Inputs	Level 2	Level 2
All Asphalt Layer Inputs (Except as Noted)	Level 3	Level 3
Mixture Volumetrics	Level 1	Level 3 (CDOT Defaults)
Mechanical Properties	Level 2	Level 2
All Unbound Aggregate Layer Inputs (Except as Noted)	Level 3	Level 3
Classification	Level 3	Level 1*
Resilient Modulus	Level 2	Level 2
Moisture-Density Relationships	Level 2	Level 2
All Subgrade Layer Inputs (Except as Noted)	Level 3	Level 3
Classification	Level 3	Level 1*
Resilient Modulus	Level 2	Level 2

* Laboratory-measured data utilized

Table 2. Pavement Sites Selected for Local Calibration (*9*, *10*)

Pavement	MoDOT (500-ft)			CDOT (1,000-ft)		
	LTPP	Agency	Total	LTPP	Agency	Total
New Asphalt	14	6	20	30	16	46
Asphalt Overlay over Asphalt	11	0	11	20	21	41
Asphalt Overlay over JPCP	9	0	9	2	5	7
Total	34	6	40	52	42	94

The results of laboratory and field evaluation, layer thickness measurement, and field performance survey of the selected pavement sections were used to evaluate and calibrate the performance prediction models in the ME Design software to local conditions in Missouri and Colorado. A summary of the global and local calibration coefficients for MoDOT and CDOT is shown in Table 3 through Table 6. For MoDOT, four models were locally calibrated, including asphalt rutting, total rutting, transverse cracking, and IRI models. The fatigue cracking model was found to be appropriate for use in Missouri; thus, its coefficients were not adjusted during local calibration. Five models were calibrated by CDOT, including fatigue cracking, asphalt rutting, total rutting, transverse cracking, and IRI. The information presented in Table 3 through Table 6 was used later in case studies to compare pavement designs conducted with global and local calibration coefficients. More information about the performance models and their coefficients is presented in Appendix A.

Table 3. Local Calibration Results for Fatigue Cracking Model (*8*, *9*, *10*)

Performance Indicator	Coefficient	Global*	MoDOT	CDOT
Fatigue Cracking	k_{f1}	0.007566	Not Calibrated	0.007566
	k_{f2}	-3.9492		-3.9492
	k_{f3}	-1.281		-1.281
	β_{f1}	1		130.3674
	β_{f2}	1		1
	β_{f3}	1		1.218
	C_1	1		0.07
	C_2	1		2.35
	C_4	6000		6000
Goodness of Fit	R^2, %	27.5		62.7
	S_e (%)	5.01		9.4
	N	405		56
Bias	p-value (paired t-test)	Not Reported		0.7566
	p-value (slope)	Not Reported		0.3529

*Globally calibrated coefficients shown in ME Design software

Table 4. Local Calibration Results for Rutting Models (8, 9, 10)

Performance Indicator	Coefficient	Global*	MoDOT	CDOT
Asphalt Rutting	k_{r1}	-3.35412	Not Reported	-3.3541
	k_{r2}	1.5606		1.5606
	k_{r3}	0.4791		0.4791
	β_{r1}	1	1.07	1.34
	β_{r2}	1	1	1
	β_{r3}	1	1	1
Fine Graded Submodel	k_{s1}	1.35	Not Reported	0.84
	β_{s1}	1	0.01	Not Reported
Granular Submodel	k_{s1}	2.03	Not Reported	0.4
	β_{b1}	1	0.4375	Not Reported
Goodness of Fit	R^2, %	57.7	52	41.7
	S_e (in)	0.107	0.051	0.147
	N	334	183	137
Bias	p-value (paired t-test)	Not Reported	0.943	0.4306
	p-value (intercept)		0.05	0.0898
	p-value (slope)		0.322	Not Reported

*Globally calibrated coefficients shown in ME Design software

Table 5. Local Calibration Results for (Thermal) Cracking Model (*8*, *9*, *10*)

Performance Indicator	Coefficient	Global*	MoDOT	CDOT
Transverse Cracking	K (Level 1)	1.5	0.625	7.5
	K (Level 2)	0.5		
	K (Level 3)	1.5		
Goodness of Fit	R^2, %	Not Reported	91 (Level 1)	43.1
	S_e (ft/mi)		51.4	194
	N		49	12
Bias	p-value (paired t-test)		0.0041	0.529
	p-value (intercept)		0.907	Not Reported
	p-value (slope)		<0.0001	0.339

*Globally calibrated coefficients shown in ME Design software

Table 6. Local Calibration Results for International Roughness Index Model (*8*, *9*, *10*)

Performance Indicator	Coefficient	Global*	MoDOT	CDOT
Top-Down (Longitudinal) Cracking	C_1	40	17.7	35
	C_2	0.4	0.975	0.3
	C_3	0.008	0.008	0.02
	C_4	0.015	0.01	0.019
Goodness of Fit	R^2, %	56	53	64.4
	S_e (in/mi)	18.9	13.2	17.2
	N	1,926	125	343
Bias	p-value (paired t-test)	Not Reported	0.6265	0.1076
	p-value (intercept)		0.0092	0.3571
	p-value (slope)		0.225	Not Reported

*Globally calibrated coefficients shown in ME Design software

4.2 Selection of Performance and Reliability Limits

After the performance prediction models were calibrated, the states selected performance criteria and reliability levels that met their needs for designing new asphalt pavements and asphalt overlays. Using the selected performance criteria and reliability levels, they conducted trial designs using the local calibration coefficients to ensure that the expected pavement design life was reasonable for future use.

Table 7 lists the performance criteria and reliability limits selected by MoDOT for designing new asphalt pavements and asphalt overlays. Even though four performance models were locally calibrated, including asphalt rutting, total rutting, transverse cracking, and IRI, MoDOT currently designs pavements based only on fatigue cracking and rutting in asphalt layers. The performance criterion for fatigue cracking was set to minimize/eliminate bottom up cracking in asphalt layers, and the criterion for asphalt rutting was determined based on the approximate depth to reduce the potential for hydroplaning. MoDOT has not adopted the IRI criteria in the pavement design process.

Table 7. MoDOT Performance Criteria and Reliability Limits (*11*)

Performance Indicator	Years	Performance Criteria	Reliability (%)
Fatigue Cracking	30	2% Lane Area Maximum	50
Asphalt Rut Depth	20	0.5 in. Maximum	50

Table 8 shows the reliability levels and performance criteria chosen by CDOT for the implementation of Pavement ME Design in Colorado. The selected thresholds are similar to those recommended in the Manual of Practice (*4*). The reliability and performance limits vary based on the functional classification with the thresholds for higher traffic roadways being more stringent. The criteria are also different for new pavement and overlay designs. For new pavement designs, the thresholds for terminal IRI, total rutting, AC rutting, and top-down fatigue cracking are only required for the years to the first rehabilitation (with a minimum initial performance period of 12 years) while the criteria for bottom-up fatigue cracking and thermal cracking are required for the entire design life of the new pavement. For overlay designs, all of the criteria are required for the years to the end of the overlay design life, and the minimum age for the overlays under rehabilitation consideration is 10 years.

4.3 Comparison of Design Results Using Global and Local Calibration Coefficients

4.3.1 Design Projects

To illustrate the impact of local calibration on pavement designs, the research team contacted CDOT and MoDOT to obtain information for case studies. CDOT suggested a reconstruction project on I-25 at Cimarron Boulevard in Colorado Springs, Colorado, and MoDOT recommended a new realignment project on US-50 in Osage County, Missouri. Both agencies conducted initial flexible pavement and jointed plain concrete pavement (JPCP) designs for these sections using the Pavement ME Design software with their local calibration coefficients. CDOT and MoDOT shared the design files for concrete and asphalt pavements with the research team to conduct additional designs using the global calibration coefficients for comparison.

4.3.2 Design Inputs

Table 9 shows the design life and basic traffic inputs for new asphalt and concrete pavement designs conducted by MoDOT and CDOT. MoDOT designed both the new flexible pavement and JPCP for 45 years. The years shown in Table 9 (30 years for flexible pavement and 25 years for JPCP) represented the last time within the design period when MoDOT would perform rehabilitation. CDOT designed the new asphalt pavement for 20 years and the new JPCP for 30 years. In addition to the traffic inputs summarized in Table 9, MoDOT used default values for the other traffic inputs in the Pavement ME Design software, whereas CDOT used its specific traffic inputs for vehicle class distribution, monthly adjustment, and axle load distribution factors determined during local calibration. More information about these inputs is described in CDOT's M-E Pavement Design Manual (*12*).

Table 8. CDOT Performance Criteria and Reliability Limits (*12*)

Classification	Reliability (%)	To Determine the Years to First Rehabilitation*				Maximum Value at the End of the Design Life		
		Terminal IRI (in/mi)	Total Rutting (in)	Rutting in AC layers (in)	Top-Down Cracking (ft/mi)	Bottom-Up Cracking (% lane)	Thermal Cracking (ft/mi)	Reflective Cracking (% lane)
Interstate	80-95	160	0.40	0.25	2,000	10	1,500	5
Principal Arterials (Freeways/Expressways)	75-95	200	0.50	0.35	2,500	25	1,500	10
Principal Arterials (Others)	75-95	200	0.50	0.35	2,500	25	1,500	10
Minor Arterial	70-95	200	0.65	0.50	3,000	35	1,500	15
Major Collectors	70-90	200	0.65	0.50	3,000	35	1,500	15

* Maximum value used to determine the years to the first rehabilitation for new pavement designs or maximum value at the end of the design life for overlay designs. The minimum age to the first rehabilitation for flexible pavements shall be 12 years.

Table 9. General and Traffic Inputs

Design Inputs	MoDOT's Inputs		CDOT's Inputs	
	New Flexible	New JPCP	New Flexible	New JPCP
Design Life	30 Years*	25 Years*	20 Years	30 Years
Traffic Inputs				
Two-way AADTT	1056		11,975	
No. of Lanes	2		3	
Trucks in Design Direction	50%		50%	
Trucks in Design Lane	95%		60%	
Operational Speed	60 mph		60 mph	
Growth Rate	2.2% Compound		1.43% Linear	

* These years represented the last time within the design period when MoDOT would perform rehabilitation. Both pavements were designed for 45 years.

The climate files for Jefferson City, Missouri and Colorado Springs, Colorado were selected for utilization in the MoDOT and CDOT designs, respectively, to predict pavement temperature and moisture. The software used these predictions to modify the asphalt concrete modulus as a function of temperature and the granular materials as a function of moisture content. The annual average depth of water table was 3 ft. for the MoDOT design and 10 ft. for the CDOT design.

Table 10 through Table 13 shows the pavement structures selected for the new flexible and rigid pavement designs by MoDOT and CDOT. MoDOT used Level 3 material inputs while CDOT used Level 1 site-specific material inputs in their designs.

Table 10. Pavement Structure for MoDOT's New Flexible Pavement Design

Pavement Structure	MoDOT's Inputs
Layer 1: Asphalt Concrete Thickness: 1.8 in.	12.5-mm Superpave, PG 70-22 Level 3 Inputs
Layer 2: Asphalt Concrete Thickness: 3 in.	25-mm Superpave, PG 70-22, Level 3 Inputs
Layer 3: Asphalt Concrete Thickness: Varied to Meet Design Criteria	25-mm Superpave, PG 64-22, Level 3 Inputs
Layer 4: Crushed Stone Thickness: 18 in.	Classification: A-1-a M_r = 30,000 psi Gradation & Other Properties for A-1-a
Layer 5: Subgrade	Subgrade A-7-6 M_r = 8,000 psi Gradation & Other Properties for A-7-6

Table 11. Pavement Structure for MoDOT's New JPCP Design

Pavement Structure	MoDOT's Inputs
Layer 1: Portland Cement Concrete Thickness: Varied to Meet Design Criteria	Limestone with Type I Cement Coefficient of Thermal Expansion = $5.5\text{x}10^{-6}$ in/in/^{o}F Level 3 Inputs
Layer 2: Crushed Stone Thickness: 18 in.	Classification: A-1-a M_r = 30,000 psi Gradation & Other Properties for A-1-a
Layer 3: Subgrade	Classification: A-7-6 M_r = 8,000 psi Gradation & Other Properties for A-7-6

Table 12. Pavement Structure for CDOT's New Flexible Pavement Design

Pavement Structure	CDOT's Inputs
Layer 1: Asphalt Concrete Thickness: 2 in.	Stone Matrix Asphalt, PG 76-28, Mix #FS1919-2 Level 1 Inputs
Layer 2: Asphalt Concrete Thickness: Varied to Meet Design Criteria	Superpave, PG 64-22, Mix #FS1938-1 Level 1 Inputs
Layer 3: Crushed Gravel Thickness: 6 in.	Classification: A-1-a M_r = 41,424 psi Gradation & Other Properties for A-1-a
Layer 4: Subgrade Thickness: 120 in.	Classification: A-2-4 M_r = 13,808 psi Gradation & Other Properties for A-2-4
Layer 5: Bedrock	Highly Fractured and Weathered E = 500,000 psi

Table 13. Pavement Structure for CDOT's New JPCP Design

Pavement Structure	CDOT's Inputs
Layer 1: Portland Cement Concrete Thickness: Varied to Meet Design Criteria	Granite with Type I Cement, Mix #2009105 Coefficient of Thermal Expansion = 4.86×10^{-6} in/in/^{o}F Level 1 Inputs
Layer 2: Crushed Gravel Thickness: 6 in.	Classification: A-1-a M_r = 44,445 psi Gradation & Other Properties for A-1-a
Layer 3: Subgrade Thickness: 120 in.	Classification: A-2-4 M_r = 28,905 psi Gradation & Other Properties for A-2-4
Layer 4: Bedrock	Highly Fractured and Weathered E = 500,000 psi

4.3.3 Design Simulation and Evaluation

MoDOT's and CDOT's new flexible and rigid pavement designs were conducted using global and local calibration coefficients in this study, resulting in eight unique designs. For each design, layer thicknesses were varied while the other design inputs were kept constant. For a particular set of layer thicknesses, the software performed simulations to predict pavement performance over the selected design period. The simulation output consisted of incremental pavement damage and distresses over time at the 50-percent reliability level and at the reliability level selected for each design. A pavement cross-section was acceptable if all distresses predicted at the selected reliability level were below the design criteria. The criteria and reliability levels for the designs in the case studies are presented in Table 14.

Table 14. Design Criteria and Reliability Levels

Performance Criteria	MoDOT		CDOT	
	Limit	Reliability	Limit	Reliability
New Flexible				
Initial IRI (in./mi)	63	NA	50	NA
Terminal IRI (in./mi)	172	50	160	90
AC Top-down Fatigue Cracking (ft/mile)	NA	NA	2,000	90
AC Bottom-up Fatigue Cracking (percent)	2	50	10	90
AC Thermal Cracking (ft/mile)	NA	NA	1,500	90
Permanent Deformation - Total Pavement (in.)	0.75	50	0.4	90
Permanent Deformation - AC Only (in.)	NA	NA	0.25	90
New JPCP				
Initial IRI (in./mi)	63	NA	75	NA
Terminal IRI (in./mi)	172	50	160	90
JPCP Transverse Cracking (Percent Slabs)	1.5	50	7	90
Mean Joint Faulting (in.)	0.15	50	0.12	90

*NA = Not Available

4.3.4 Design Results

Table 15 shows the final pavement designs obtained from the ME Design software using the global and local calibration coefficients. For MoDOT's designs, the final thicknesses are the same for new flexible pavements using global and local calibration coefficients, but the design thickness using the global calibration coefficients is slightly greater than the one using the local calibration coefficients for the new JPCP designs. For CDOT's designs, the local calibration coefficients yielded a thinner asphalt concrete layer but the same Portland cement concrete structure as the global calibration coefficients. A detailed analysis of the performance prediction results follows.

Table 15. Comparison of Design Thicknesses using Global and Local Calibration Coefficients

Pavement Layer	MoDOT		CDOT	
	Global	Local*	Global	Local*
New Flexible				
Asphalt Concrete (in.)	8	8	11.5	10.5
Crushed Aggregate Base (in.)	18	18	6	6
New Rigid (JPCP)				
Portland Cement Concrete (in.)	8.5	8**	7.5	7.5
Crushed Aggregate Base (in.)	18	18	6	6

*Based on MoDOT's and CDOT's designs
**Due to changes in locally calibrated IRI model

Figure 4 and Figure 5 compare the predicted distresses and IRI results for flexible and rigid pavement designs for the realignment project on US-50 in Osage County, Missouri using global and local calibration coefficients. Based on these results, the following observations can be drawn.

- The flexible pavement designs using the global and local calibration coefficients have the same thickness designs (Table 15) because the designs are governed by the predicted bottom-up cracking distresses, as shown in Figure 4. The predicted bottom-up cracking results are the same for the two designs since the coefficients for the bottom-up cracking model were not adjusted during local calibration.
- Also, shown in Figure 4, the predicted total rutting and IRI results based on the local calibration coefficients are lower than those based on the global calibration coefficients as the performance prediction models for asphalt layer rutting, base layer rutting, and IRI were locally calibrated. Not shown in Figure 4 are the predicted transverse (thermal) cracking results for the flexible pavement designs since transverse cracking is not used as a design criterion in Missouri.
- The local calibration JPCP design (Figure 5) is 0.5 in. thinner than the global calibration JPCP design, as shown in Table 15. With a thinner thickness design, the local calibration coefficients yield similar predicted PCC cracking and faulting results but higher IRI predictions than the global calibration coefficients.

Similar to Figure 4 and Figure 5, Figure 6 and Figure 7 compare the predicted distresses and IRI results for flexible and rigid pavement designs using global and local calibration coefficients for the reconstruction project on I-25 at Cimarron Boulevard in Colorado Springs, Colorado. The following observations can be drawn from these results.

- With a 1-in. thinner asphalt layer (Table 15), the local-calibration design yields lower rutting and cracking predictions and slightly higher IRI results (Figure 6). More flexible pavement design results are shown in Table 16 and Table 17. As shown in these tables, all of the performance predictions pass the corresponding design criteria except the predicted total rutting and AC rutting results. The asphalt pavement design was still accepted by CDOT as rutting had not been found to be a performance issue in similar pavements in the area. This suggests that the CDOT design is largely governed by the predicted bottom-up cracking results.
- For the JPCP designs (Figure 7), the global and local calibration coefficients yield similar performance predictions, suggesting that the local calibration has a minimum effect on JPCP design for this project.

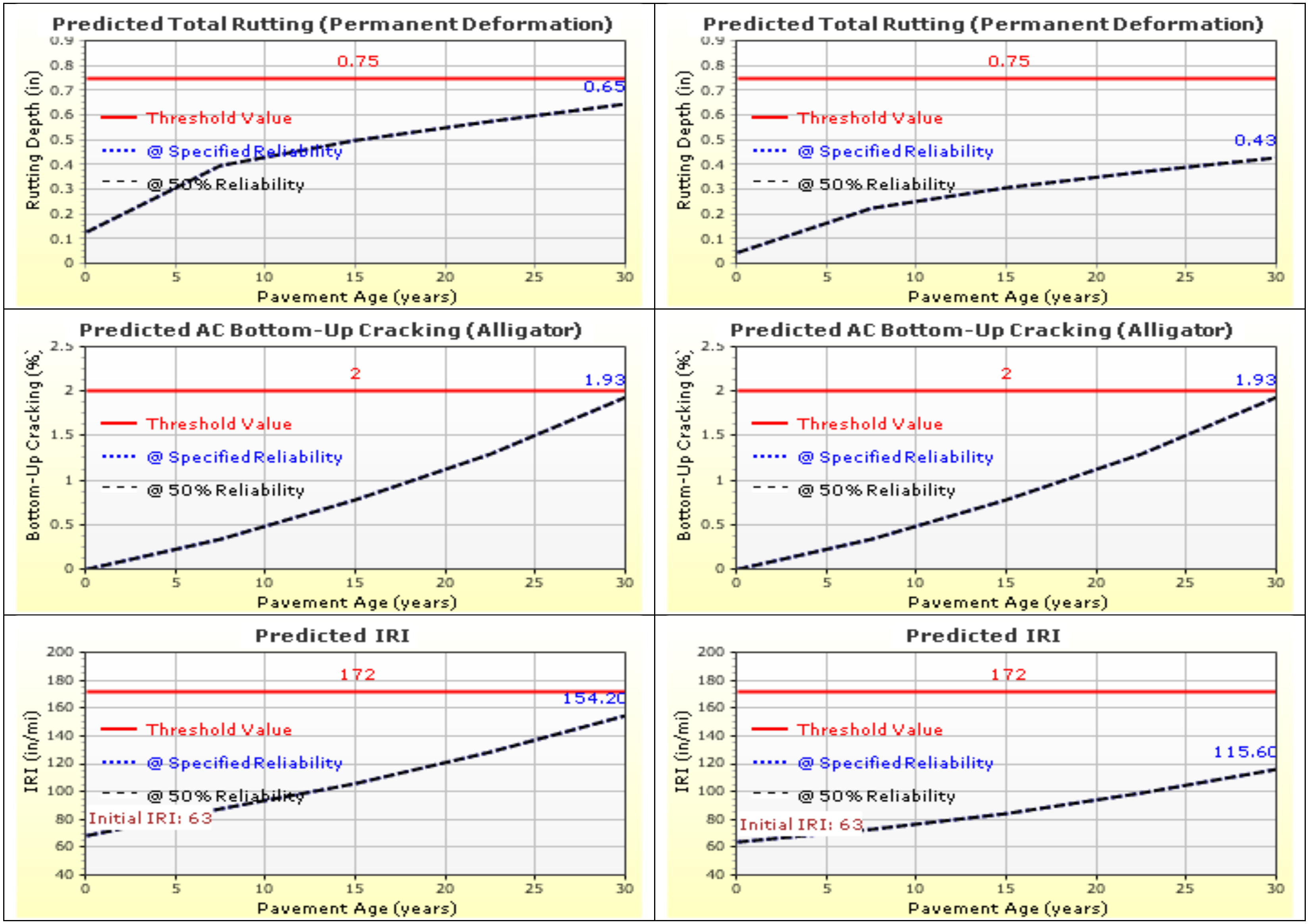

Figure 4. MoDOT's Flexible Pavement ME Design Results using Global (Left) and Local (Right) Calibration Coefficients

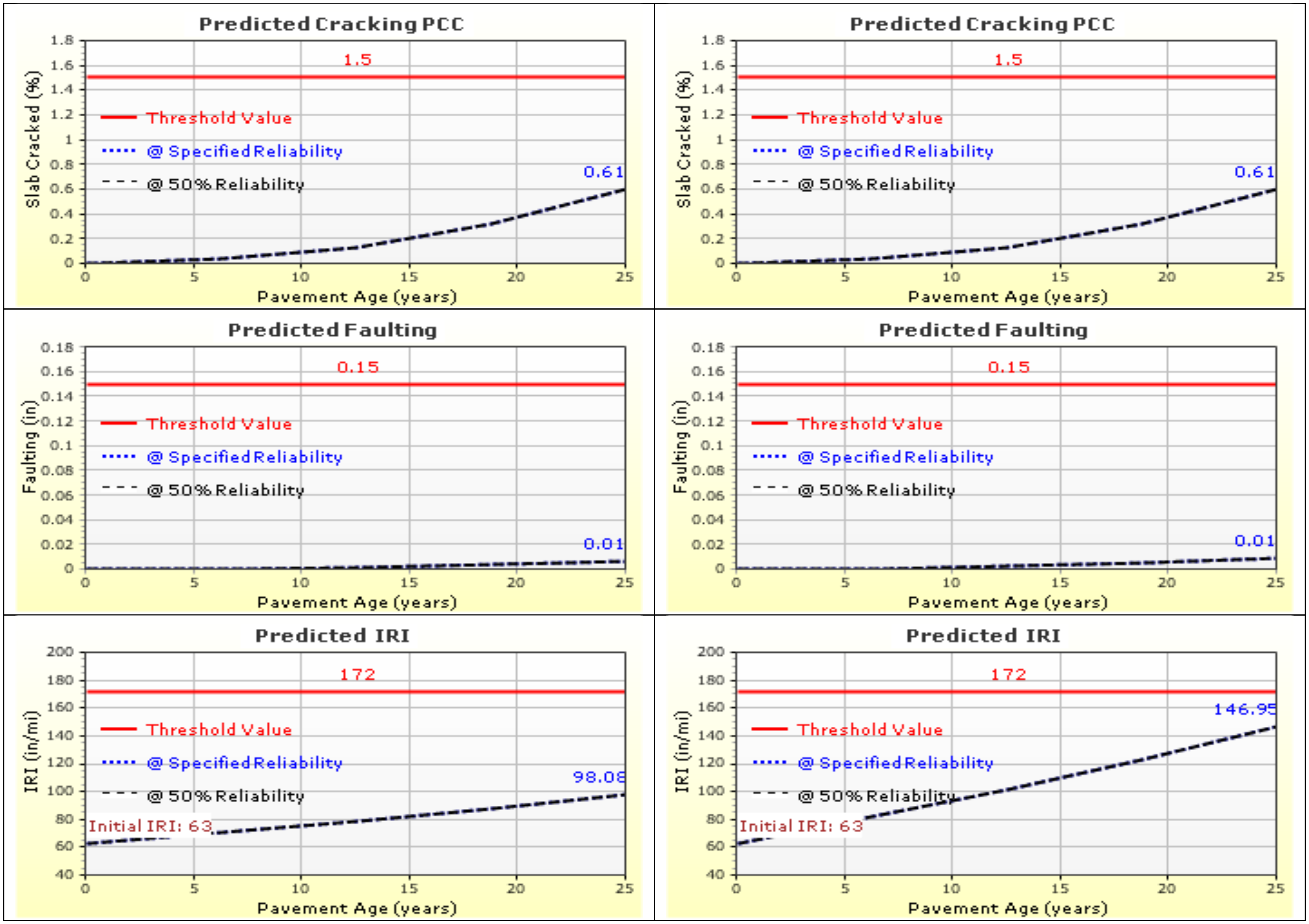

Figure 5. MoDOT's Rigid Pavement ME Design Results using Global (Left) and Local (Right) Calibration Coefficients

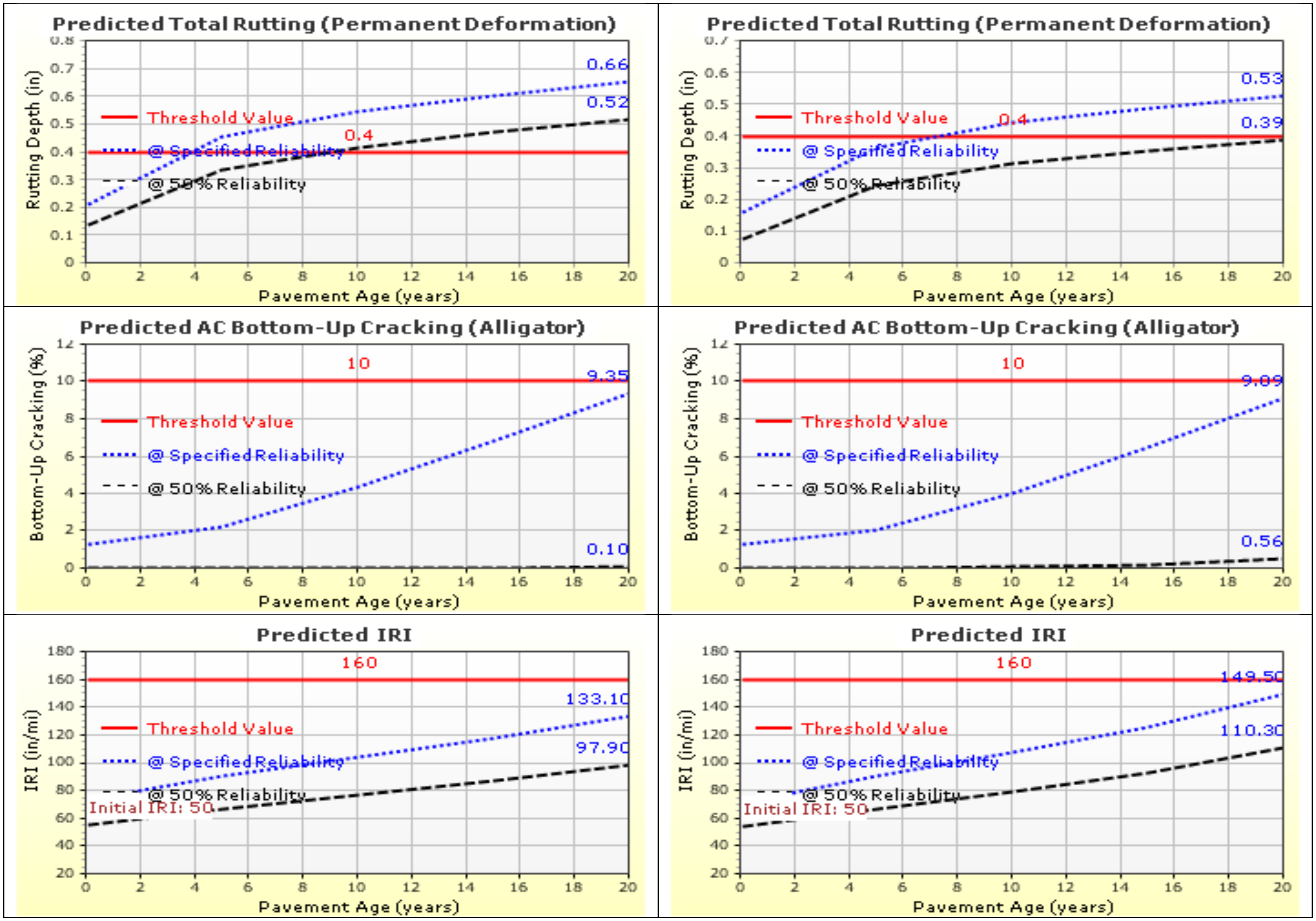

Figure 6. CDOT's Flexible Pavement ME Design Results using Global (Left) and Local (Right) Calibration Coefficients

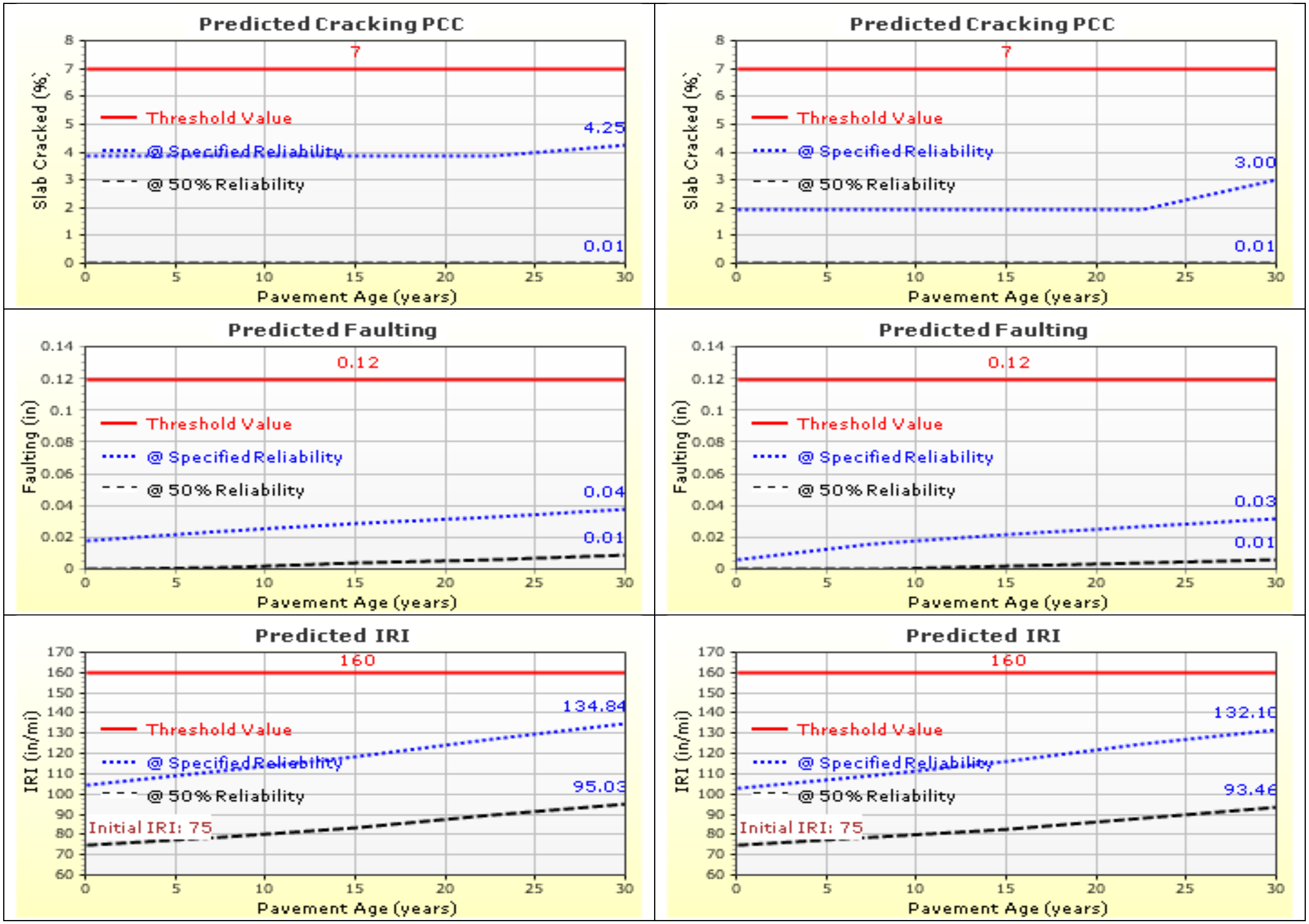

Figure 7. CDOT's Rigid Pavement ME Design Results using Global (Left) and Local (Right) Calibration Coefficients

Table 16. Distress Prediction Results for CDOT Flexible Pavement ME Design using Global Calibration Coefficients

Distress Type (AC Thickness = 11.5 in.)	Distress at Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	160.00	133.12	90.00	98.81	Pass
Permanent Deformation - Total Pavement (in.)	0.40	0.66	90.00	13.86	Fail
AC Bottom-up Fatigue Cracking (percent)	10.00	9.35	90.00	91.48	Pass
AC Thermal Cracking (ft/mile)	1500.00	84.34	90.00	100.00	Pass
AC Top-down Fatigue Cracking (ft/mile)	2000.00	309.69	90.00	100.00	Pass
Permanent Deformation - AC Only (in.)	0.25	0.42	90.00	37.67	Fail

Table 17. Distress Prediction Results for CDOT Flexible Pavement ME Design using Local Calibration Coefficients

Distress Type (AC Thickness = 10 in.)	Distress at Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	160.00	149.52	90.00	94.78	Pass
Permanent Deformation - Total Pavement (in.)	0.40	0.53	90.00	53.04	Fail
AC Bottom-up Fatigue Cracking (percent)	10.00	9.09	90.00	92.19	Pass
AC Thermal Cracking (ft/mile)	1500.00	537.38	90.00	100.00	Pass
AC Top-down Fatigue Cracking (ft/mile)	2000.00	330.61	90.00	100.00	Pass
Permanent Deformation - AC Only (in.)	0.25	0.43	90.00	32.52	Fail

4.4 Summary

The case studies were conducted for two pavement design projects, one in Missouri and the other in Colorado. The two states completed their local calibration processes and have utilized the Pavement ME Design software for routine pavement designs. For the local calibration processes completed thus far, CDOT spent more time and effort on determining more accurate design inputs and selected more pavement sites. As more information is available since the last local calibration, both CDOT and MoDOT have planned for recalibrating their MEPDG procedures.

MoDOT calibrated four models for asphalt rutting, total rutting, transverse cracking, and IRI; the fatigue cracking was found to be appropriate for use in Missouri. CDOT calibrated five models for fatigue cracking, asphalt rutting, total rutting, transverse cracking, and IRI.

Even though four performance models were locally calibrated, MoDOT currently designs pavements based only on fatigue cracking and rutting in asphalt layers. The performance criteria for these distresses were set to minimize/eliminate bottom up cracking in asphalt layers and to reduce the potential for hydroplaning. MoDOT conducts all flexible pavement designs at a 50% reliability level.

CDOT selected its design criteria and reliability levels similar to those recommended in the Manual of Practice (*4*); they vary based on the functional classification with the thresholds for higher traffic roadways being more stringent. The criteria are also different for new pavement and overlay designs. For new pavement designs, the thresholds for IRI, total rutting, AC rutting, and top-down fatigue cracking are required for the years to the first rehabilitation whereas the criteria for bottom-up fatigue cracking and thermal cracking are required for the entire design life of the new pavement. For overlay designs, all of the criteria are required for the years to the end of the overlay design life.

The local calibration results were used in a comparative analysis to illustrate the impact of local calibration on pavement designs for the two pavement design projects in Missouri and Colorado. For both the projects, flexible pavement designs were conducted using the global and local calibration coefficients. In addition, JPCP designs were also conducted for these projects for comparison.

For the project in Missouri, the new flexible pavement designs were governed by the predicted bottom-up cracking distresses. The bottom-up cracking model was not adjusted during local verification as its predictions agreed with the early age performance of deep-strength flexible pavements. Thus, the final thicknesses were the same for new flexible pavements using global and local calibration coefficients. For the new JPCP designs, the design using the global calibration coefficients was 0.5 in. thicker than that using the local calibration coefficients.

For the project in Colorado, the local calibration coefficients yielded a 1-in. thinner asphalt structure but the same Portland cement concrete structure as the global calibration coefficients. The new flexible pavement designs for the Colorado project were determined by the predicted bottom-up cracking results. The design failed the rutting performance criteria, but it was accepted by CDOT as rutting was not a performance issue in similar pavements in the area.

The above comparative analysis was conducted for one type of aggregate base and subgrade; further analysis was conducted to determine the effect of base and subgrade support on asphalt pavement design, and it is presented in the next section.

5 EFFECT OF FOUNDATION SUPPORT

A flexible pavement structure typically consists of surface, base, and subbase layers placed on subgrade to carry traffic loads. The performance of a flexible pavement relies upon the performance of individual layers. Since the stresses induced by traffic loads in a pavement structure are highest in the top layers and decrease with depth, higher quality materials are generally used in the upper layers, and lower quality materials are used in the lower layers.

At the bottom, the subgrade provides a platform and supports the pavement structure. When subgrade is of low quality and proper base material is not locally available, a subbase layer is often required. Constructed on top of the subgrade, the aggregate subbase can be unbound or treated with cement, lime, or fly ash to improve its strength characteristics. The subbase layer contributes to the structural capacity of the pavement, prevents intrusion of fine grained subgrade soils into the base layer, minimizes the damaging effect of frost action, and provides drainage for free water that may enter the pavement structure.

The base layer is placed on the subbase or directly on the subgrade to provide support for the surface course and other functions similar to those of the subbase. High quality material such as gravel and crushed stone is normally used for this layer. The material may also be stabilized with cement, lime, or other admixtures.

Built on top of the base layer, the surface layer consists of one or more asphalt layers that provide structural capacity to support traffic loads, distribute the loads to the lower layers, minimize water infiltration, and provide a smooth and skid resistant surface.

The performance of an asphalt pavement is usually based upon observations of surface distresses, which can be due to deficiencies in the surface layers, but very often, they are caused by deficiencies in the underlying layers. Table 18 shows how a foundation-related problem can cause other surface distresses for an asphalt pavement (*17*).

Table 18. Surface Distresses Potentially Caused by Foundation-Related Problems (*17*)

Foundation-Related Problems	Surface Distresses Resulted from Foundation Problems						
	Cracking	Rutting	Corrugation	Bumps	Depressions	Potholes	Roughness
Insufficient Strength	x	x	x		x		x
Moisture/Drainage	x	x			x	x	x
Freeze/Thaw	x	x		x	x	x	x
Swelling				x			x
Contamination	x	x			x		x

The Pavement ME Design software allows an asphalt pavement foundation support to be designed with unbound aggregate or chemically stabilized materials for base and subbase layers and allows for the selection of different soil types for subgrade. In the following sections, a brief summary of foundation materials inputs required in the Pavement ME Design is first presented followed by a sensitivity analysis to illustrate the effect of foundation support on asphalt pavement design.

5.1 Foundation Materials Inputs Required in Pavement ME Design

5.1.1 Inputs for Unbound and Subgrade Layers

The hierarchical design approach in the ME Design software also applies to foundation materials inputs. The required inputs for unbound layers and subgrade include resilient modulus, physical/engineering properties (such as soil classification, moisture content, and dry density), and hydraulic properties. Even though three input levels were planned, only Level 2 and 3 inputs are allowed for these layers in the current Pavement ME Design software (Version 2.0). Table 19 summarizes the properties required for Level 2 and 3 inputs for unbound aggregate and subgrade soils.

Table 19 Inputs for Unbound Aggregate and Subgrade Soils (*4*)

Property	Description	Level 2	Level 3
Resilient Modulus of Unbound Layers and Subgrade			
M_r	Measured or Estimated Resilient Modulus	✓	✓
CBR	California Bearing Ratio	✓	
R	R-value	✓	
a_i	Layer Coefficient	✓	
DCP	Dynamic Cone Penetration Index	✓	
PI	Plasticity Index	✓	
P_{200}	Percentage Passing No. 200 Sieve	✓	
Soil class	AASHTO or USCS Soil Class		✓
ν	Poisson's Ratio	✓	✓
Physical Properties			
G_s	Specific Gravity	Default	
γ_{dmax}	Maximum Dry Density	Default	
w_{opt}	Optimum Moisture Content	Default	
PI	Plastic Index	✓	✓
Hydraulic Properties			
a_f, b_f, c_f, h_r	Soil Water Characteristic Curve Parameters	Default	
K_{sat}	Saturated Hydraulic Conductivity (Permeability)	Default	

The resilient modulus (M_r) input is required in the ME Design software to calculate the structural response of the pavement. It can be measured directly from laboratory testing or obtained through the use of correlations with other material strength properties such as CBR and R-value (Level 2) or by using typical values (Level 3). Table 20 shows models that can be used to estimate M_r based on other material and strength properties for Level 2 inputs. Listed in Table 21 are typical resilient moduli for unbound granular and subgrade for Level 3 inputs. Other physical properties for Level 2 and 3 inputs are shown in Table 22.

Table 20 Models for Estimating Level 2 M_r Inputs (*4*)

Property	Model	Test Standard
CBR	M_r (psi) = $2555(CBR)^{0.64}$	AASHTO T193 - California Bearing Ratio
R-Value	M_r (psi) = 1155 + 555R	AASHTO T190 - Resistance R-value
AASHTO Layer Coefficient	$M_R \text{ (psi)} = 30000\left(\frac{a_i}{0.14}\right)$ a_i = Layer Coefficient	1993 AASHTO Guide for the Design of Pavement Structures
PI and Gradation[1]	$CBR = \frac{75}{1 + 0.728(wPI)}$ $wPI = P_{200} * PI$	AASHTO T27 - Sieve Analysis of Aggregates AASHTO T90 - Plastic Limit and Plasticity Index of Soils
DCP[1]	$CBR = \frac{292}{DCP^{1.12}}$	ASTM D6951 - Dynamic Cone Penetrometer

[1] Estimates of CBR to estimate M_r.

Table 21 Level 3 M_r For Unbound Granular and Subgrade at Optimum Moisture Content (*4*)

AASHTO Soils Classification	Base/Subbase	Embankment and Subgrade
A-1-a	40,000	29,500
A-1-b	38,000	26,500
A-2-4	32,000	24,500
A-2-5	28,000	21,500
A-2-6	26,000	21,000
A-2-7	24,000	20,500
A-3	29,000	16,500
A-4	24,000	16,500
A-5	20,000	15,500
A-6	17,000	14,500
A-7-5	12,000	13,000
A-7-6	8,000	11,500

Table 22 Other Level 2 and 3 Inputs for Unbound Layers and Subgrade (*4*)

Property	Recommended Input
Specific Gravity	Estimated Using Gradation, Plasticity Index, and Liquid Limit
Maximum Dry Density	
Optimum Moisture Content	
Saturated Hydraulic Conductivity	
Soil Water Characteristic Curve Parameters	Selected Based on Aggregate/Subgrade Class

In addition, the Manual of Practice (*4*) recommends that when granular base/subbase layers are used, the resilient modulus of these layers be a function of the resilient modulus of the underlying layers, including subgrade layers. The initial resilient modulus of the granular layer should not exceed three times the resilient modulus of the supporting layers to avoid decompaction and tensile stresses in the unbound layers. Figure 8 shows the maximum resilient modulus of an unbound material layer as a function of its thickness and the resilient modulus of the supporting layers.

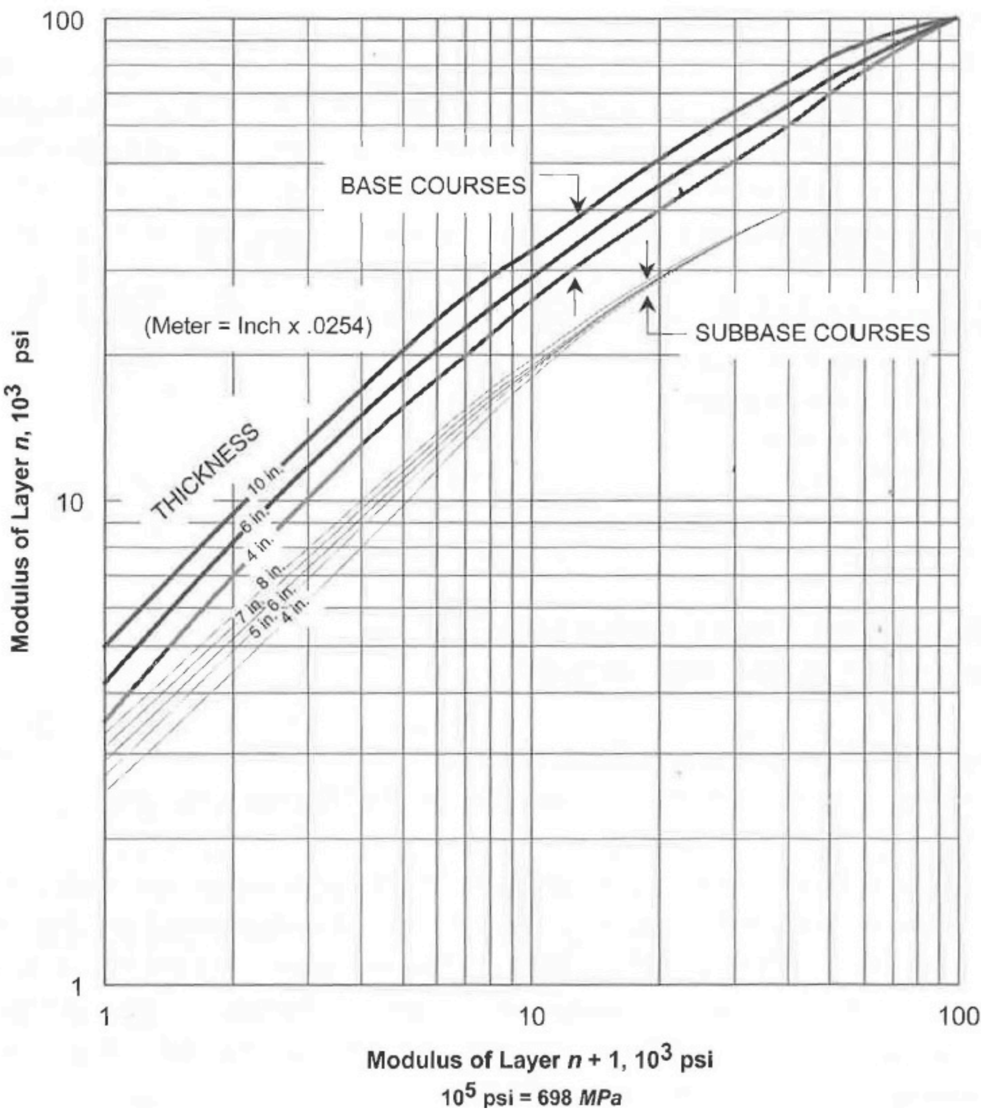

Figure 8 Limiting Modulus Criteria of Unbound Aggregate Base and Subbase Layers (*4*)

5.1.2 Inputs for Stabilized Layers

Some unbound materials and subgrade soils susceptible to fluctuations in strength and stiffness properties may require stabilization due to fluctuations in moisture content. The ME Design software allows inputs for chemically stabilized layers, including elastic/resilient modulus, flexural strength, and physical and thermal properties. Table 23 summarizes the properties required for each stabilized material at Levels 1, 2, and 3. Table 24 summarizes the recommended test protocols and relationships for Level 1 and 2 inputs.

Table 25 summarizes the recommended input values for elastic/resilient modulus and flexural strength when a Level 3 analysis is used. Finally, Table 26 presents the recommended unit weight and thermal properties values for Level 2 and 3 inputs for chemically stabilized layers.

Table 23 Chemically Stabilized Materials Input Requirements

Material Type	Property	Level		
		1	2	3
Lean Concrete and Cement Treated Aggregate	Elastic Modulus	✓	✓	✓
	Flexural Strength	✓	✓	✓
Lime Cement-Fly Ash	Elastic Modulus		✓	✓
	Flexural Strength	✓	✓	✓
Soil Cement	Elastic Modulus		✓	✓
	Flexural Strength	✓	✓	✓
Lime Stabilized Soil	Elastic Modulus	✓	✓	✓
	Flexural Strength		✓	✓
All	Unit Weight			✓
	Thermal Conductivity			✓
	Poisson's Ratio			✓
	Heat Capacity			✓

Table 24 Recommended Test Protocols and Relationships for Elastic Modulus and Flexural Strength values for Chemically Stabilized Layers (Level 1 and 2 Inputs)

Design Type	Material	Level 1 – Modulus	Level 1 – Flexural Strength	Level 2 – Relationship for Modulus	Level 2 – Relationship for Flexural Strength
New	Lean Concrete and cement treated aggregate	ASTM C 469	AASHTO T 97	$E=57,000(f'_c)^{0.5}$ For f'_c: AASHTO T 22	Use 20% of the Compressive Strength (Lab Samples or Cores)
	Lime-Cement-Fly Ash	N/A	AASHTO T 97	$E=500+q_u$ For q_u: ASTM C 593	
	Soil Cement	N/A	ASTM D 1635	$E=1200(q_u)$ For q_u: ASTM D 1633	
	Lime Stabilized Soil	AASHTO T 307	N/A	$M_r=0.124(q_u)+9.98$ For q_u: ASTM D 5102	
Existing	All	Modulus from FWD AASHTO T 256 and ASTM D 5858	N/A	Same as New Design	Same as New Design

Table 25 Recommended Elastic/Resilient Modulus and Flexural Strength Values for Chemically Stabilized Layers for Level 3 Analysis

Material	Modulus (psi)	Material	Flexural Strength (psi)
Lean Concrete, E	2,000,000	Chemically Stabilized Material Used as Base	750
Cement Stabilized Base, E	1,000,000		
Soil Cement, E	750,000	Chemically Stabilized Material Used as Subbase or Subgrade	250
Lime Stabilized Soil, M_r	45,000		

Table 26 Recommended Unit Weight and Thermal Properties Values for Level 2 and 3 Inputs for Chemically Stabilized Layers

Required Input	Recommended Value
Unit Weight	Default Value 150 pcf
Thermal Conductivity	Default Value 1.25 BTU/h-ft°F
Heat Capacity	Default Value 0.28 BTU/lb-°F

5.1.3 Effect of Foundation Support on Pavement ME Design Results

In this section, a sensitivity analysis is presented to evaluate the effect of foundation support on the Pavement ME Design results. The CDOT new flexible pavement design with local calibration coefficients was utilized for this analysis. Table 27 shows the pavement structure and material inputs that were varied in the sensitivity analysis. The inputs for the base and subgrade layers were selected to cover the typical ranges of these materials discussed in the previous sections. Other inputs, including traffic inputs, climatic inputs, reliability levels, and performance criteria, were kept the same as those in the CDOT pavement design shown earlier.

Table 27. Inputs for Sensitivity Analysis to Evaluate Effect on Foundation Support

Pavement Structure	Material	Inputs for Sensitivity Analysis
Layer 1: Asphalt Concrete	Stone Matrix Asphalt, PG 76-28	Thickness: 2 in. Level 1 Inputs
Layer 2: Asphalt Concrete	Superpave, PG 64-22	Thickness Optimized Level 1 Inputs
Layer 3: Base	Unbound and Cement Stabilized (CTB) Classification Other Properties for Unbound and CTB	Thickness: 6 & 12 in. Mr (Unbound): 30,000; 40,000 & 50,000 psi; Mr (Cement Treated Base): 100,000 psi
Layer 4: Subgrade	Classification: A-7-4, A-4 and A-2-4 Gradation & Other Properties for A-7-6, A-4 and A-2-4	Thickness: 120 in. (to Bedrock) M_r: 10,000; 15,000 & 20,000 psi
Layer 5: Bedrock	Highly Fractured and Weathered	Semi-infinite E = 500,000 psi

Table 28 shows results of the sensitivity analysis to evaluate the effect of foundation support on the thickness design, which in this case is the thickness of Layer 2 (Superpave asphalt base). Based on the analysis results, the following observations can be drawn:

- As the stiffness of subgrade increased from 10,000 psi to 20,000 psi and the stiffness of the unbound aggregate base was selected as a function of the subgrade stiffness and the thickness of the aggregate base layer (Figure 8), the thickness of the asphalt structure was reduced by 1 inch.
- When increasing the thickness of the aggregate base from 6 inches to 12 inches (all else equal), the thickness of the asphalt structure changed by 0.5 inches for only one scenario with subgrade resilient modulus of 15,000 psi and unbound base resilient modulus of 40,000 psi.
- Compared to a 6-inch unbound aggregate base, the 6-inch stabilized base could yield a 1.5-inch, 1.0-inch, and 0.5-inch thinner asphalt structure for 10,000 psi, 15,000 psi, and 20,000 psi subgrade, respectively.
- The thickness of the asphalt structure decreased by 3 inches when the thickness of the cement stabilized base increased from 6 inches to 12 inches.
- All of the designs with the unbound aggregate base were governed by the bottom-up fatigue cracking, and all of the designs with the stabilized base were governed by the IRI criteria. Table 29 and Table 30 show the final prediction results for Scenarios 1 and 12. Even though the rutting predictions failed the design criteria, the locally calibrated AC models were found by CDOT to over-predict the AC layer rutting in the field.

Table 28. Effect on Foundation Support on Layer 2 Thickness

Scenario	Layer 5: Bedrock	Layer 4: Subgrade		Layer 3: Base		Layer 2: Superpave	Layer 1: SMA	Design Governed By
	E (psi)	H (in)	Mr (psi)	H (in)	Mr (psi)	H (in)	H (in)	
	Unbound Aggregate Base							
1	500,000	120	10,000	6	30,000	**8.5**	2	Bottom-up
2	500,000	120	15,000	6	40,000	**8.0**	2	Bottom-up
3	500,000	120	20,000	6	50,000	**7.5**	2	Bottom-up
4	500,000	120	10,000	12	30,000	**8.5**	2	Bottom-up
5	500,000	120	15,000	12	40,000	**7.5**	2	Bottom-up
6	500,000	120	20,000	12	50,000	**7.5**	2	Bottom-up
	Cement Stabilized Base							
7	500,000	120	10,000	6	100,000	**7.0**	2	IRI
8	500,000	120	15,000	6	100,000	**7.0**	2	IRI
9	500,000	120	20,000	6	100,000	**7.0**	2	IRI
10	500,000	120	10,000	12	100,000	**4.0**	2	IRI
11	500,000	120	15,000	12	100,000	**4.0**	2	IRI
12	500,000	120	20,000	12	100,000	**4.0**	2	IRI

Table 29. Distress Prediction Results for Scenario 1 Sensitivity Analysis

Distress Type	Distress at Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	160.00	149.06	90.00	94.94	Pass
Permanent Deformation - Total Pavement (in.)	0.40	0.53	90.00	53.54	Fail
AC Bottom-up Fatigue Cracking (percent)	10.00	9.96	90.00	90.10	Pass
AC Thermal Cracking (ft/mile)	1500.00	494.69	90.00	100.00	Pass
AC Top-down Fatigue Cracking (ft/mile)	2000.00	297.51	90.00	100.00	Pass
Permanent Deformation - AC Only (in.)	0.25	0.41	90.00	40.63	Fail

Table 30. Distress Prediction Results for Scenario 12 Sensitivity Analysis

Distress Type	Distress at Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	160.00	151.61	90.00	93.99	Pass
Permanent Deformation - Total Pavement (in.)	0.40	0.65	90.00	19.51	Fail
AC Bottom-up Fatigue Cracking (percent)	10.00	1.28	90.00	100.00	Pass
Total Cracking (Reflective + Fatigue) (percent)	15	4.48	-	-	Pass
AC Thermal Cracking (ft/mile)	1500.00	189.22	90.00	100.00	Pass
AC Top-down Fatigue Cracking (ft/mile)	2000.00	645.01	90.00	100.00	Pass
Permanent Deformation - AC Only (in.)	0.25	0.61	90.00	3.70	Fail
Chemically Stabilized Layer - Fatigue Fracture (percent)	25.00	0.84	-	-	-

6 PERFORMANCE CRITERIA AND RELIABILITY

6.1 Performance Criteria and Reliability Levels

As for the empirical AASHTO Guides, the MEPDG allows the user to set the performance criteria and reliability levels to evaluate the adequacy of each design. Table 31 and Table 32 show the performance criteria and reliability levels recommended in the Manual of Practice (*4*). A brief description of the reliability concept employed in the MEPDG is presented below.

Table 31. Performance Criteria Recommended for Flexible Pavement Design (*4*)

Performance Indicator	Maximum Value at End of Design Life
Terminal IRI (Smoothness)	Interstate: 160 in/mi Primary: 200 in/mi Secondary: 200 in/mi
Total Rutting (In Wheel Paths)	Interstate: 0.40 in. Primary: 0.50 in. Others (< 45mph): 0.65 in.

Performance Indicator	Maximum Value at End of Design Life
Fatigue Cracking	Interstate: 10% lane area Primary: 20% lane area Secondary: 35% lane area
Transverse (Thermal) Cracking	Interstate: 500 ft/mi Primary: 700 ft/mi Secondary: 700 ft/mi

Table 32. Levels of Reliability for Different Functional Classifications of the Roadway (*4*)

Functional Classification	Level of Reliability	
	Urban	Rural
Interstate/Freeways	95	95
Principal Arterials	90	85
Collectors	80	75
Local	75	70

In the Pavement ME Design software, reliability is applied to the individual performance indicators, which include total rutting, fatigue cracking, thermal cracking, IRI, and other performance predictions including longitudinal cracking, reflective cracking, and AC rutting. Reliability (R) is defined as the probability (P) that each of the key distress types and IRI will be less than a selected critical level over the design period, as shown in Equation 1.

$$R = P\ [\text{Distress over Design Period} < \text{Critical Distress Level}] \tag{1}$$

For each performance indicator, the Pavement ME Design software first determines the mean prediction using the corresponding performance (transfer) model. The software then increases the prediction by the prediction reliability, which is dependent on the standard error of the performance model determined when it was calibrated and the reliability level selected. This procedure is utilized for all the performance indicators.

6.2 Effect of Performance Criteria and Reliability on Pavement Design

Since the performance criteria and design reliability greatly affect the final design, initial construction costs, long-term performance, and life cycle costs, they should be carefully selected in balance with each other. For the same set of performance criteria, a lower reliability level would yield lower predicted distresses, resulting in a thinner pavement section. For the same design reliability level, a set of criteria with lower allowable cracking, rutting, and IRI would result in a thicker pavement section. For example, since MoDOT set lower allowable cracking and rutting, its reliability level was set lower than that of CDOT.

An analysis was conducted in this study to evaluate the sensitivity of pavement design thickness to the performance criteria and reliability levels recommended in the Manual of Practice (*4*). The sensitivity analysis was conducted for the CDOT's new flexible pavement structure shown in Table 12 using CDOT's local calibration coefficients. It was conducted for four roadway classifications including interstate, principal arterial, minor arterial, and major collector. The design life and traffic inputs selected for each roadway classification are shown in Table 33.

Table 33. Design Life and Traffic Inputs Selected for Sensitivity Analysis

Design Inputs	Interstate	Principal Arterial	Minor Arterial	Major Collectors
Design Life, years	20	20	20	20
Traffic Inputs				
Two-way AADTT	11500	8640	4320	1920
No. of Lanes	3	3	2	1
Trucks in Design Direction	50%	50%	50%	60%
Trucks in Design Lane	60%	60%	90%	100%
Operational Speed, mph	70	65	55	45
Growth Rate	1.43%	1.43%	1.43%	1.43%

Table 34 shows the performance criteria and reliability levels used in the analysis. The performance criteria were set based on the recommendations in the Manual of Practice (*4*), as shown in Table 31, and those utilized by CDOT, as shown in Table 8. Also, shown in Table 34 are the performance criteria for the unbound layers that were obtained by subtracting permanent deformation in the AC layer from the total permanent deformation in the pavement. These criteria are the same (0.15 in.) for all the roadway classifications.

As shown in Table 34, each design was conducted at several reliability levels starting from 50% to the reliability level recommended for each roadway classification shown in Table 32. In addition, the AC thickness was varied for each design, which also includes a 6-in. aggregate base, to evaluate the sensitivity of AC thickness to the performance criteria and reliability levels as follows:

- Interstate design: 9 – 12 in. AC, including 2 in. SMA
- Principal arterial design: 7.5 – 10.5 in. AC, including 2 in. SMA
- Minor arterial design: 7 – 9.5 in. AC, including 2 in. SMA
- Major collector design: 6.5 – 9.5 in. AC, including 2 in. SMA

Table 34. Performance Criteria and Reliability Levels Selected for Sensitivity Analysis

Performance Criteria	Interstate		Principal Arterial		Minor Arterial		Major Collectors	
	Limit	Reliability	Limit	Reliability	Limit	Reliability	Limit	Reliability
New Flexible								
Initial IRI (in./mi)	50		50		50		50	
Terminal IRI (in./mi)	160	50—95	200	50—90	200	50—85	200	50—80
AC Top-down Fatigue Cracking (ft/mile)	2,000	50—95	2,500	50—90	3,000	50—85	3,000	50—80
AC Bottom-up Fatigue Cracking (percent)	10	50—95	25	50—90	35	50—85	35	50—80
AC Thermal Cracking (ft/mile)	1,500	50—95	1,500	50—90	1,500	50—85	1,500	50—80
Permanent Deformation - Total Pavement (in.)	0.4	50—95	0.5	50—90	0.65	50—85	0.65	50—80
Permanent Deformation - AC Only (in.)	0.25	50—95	0.35	50—90	0.5	50—85	0.5	50—80
Permanent Deformation - Unbound (in.)	0.15	50—95	0.15	50—90	0.15	50—85	0.15	50—80

Based on the analysis, two types of pavement distresses, including bottom-up fatigue cracking and permanent deformation in the unbound layers, were found to be more sensitive to the changes in pavement design thickness. Thus, the sensitivity of these predicted distresses to the changes in pavement design thickness is further discussed in the following subsections.

6.2.1 Sensitivity of Permanent Deformation in Unbound Layers to Pavement Design Thickness

The Pavement ME Design software predicted total permanent deformation in the pavement and permanent deformation in the AC layer. Based on this information, the predicted permanent deformation in the unbound layers, including aggregate base and subgrade, was obtained by subtracting the permanent deformation in the AC from the total permanent deformation in the pavement. Figure 9 through Figure 12 shows the effect of AC thickness on predicted rutting in the unbound layers for the four roadway classifications. The dashed line shown in these figures is the design limit for maximum rutting in the unbound layers. Since each design was also conducted at several reliability levels, these figures also show the effect of reliability level on predicted rutting in the unbound layers. Based on these results, the following observations can be offered:

- The effect of AC thickness was slightly higher for the pavements with thinner AC layers. However, the difference in rutting in the unbound layers became practically insignificant when the AC thickness increased above 9 inches, approaching the design thickness of a perpetual asphalt pavement.
- The impact of reliability level on predicted rutting in the unbound layers was minimal for each design.
- The predicted permanent deformation in the unbound layers passed the design limit (0.15 in.), suggesting that the design AC layer could be thinner, that the design limit for rutting in the unbound layers could be lower, or that the design was governed by the other distress type.

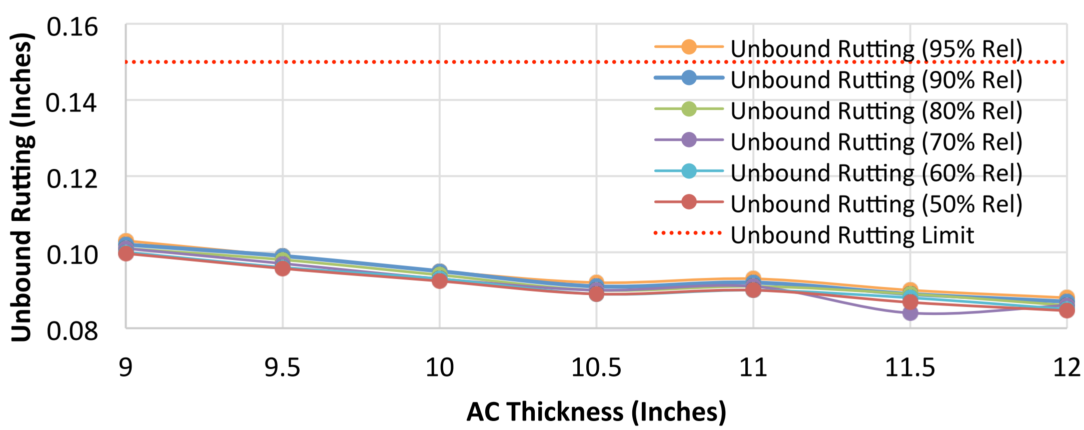

Figure 9 Effect of AC Thickness on Predicted Rutting in Unbound Layers (Interstate)

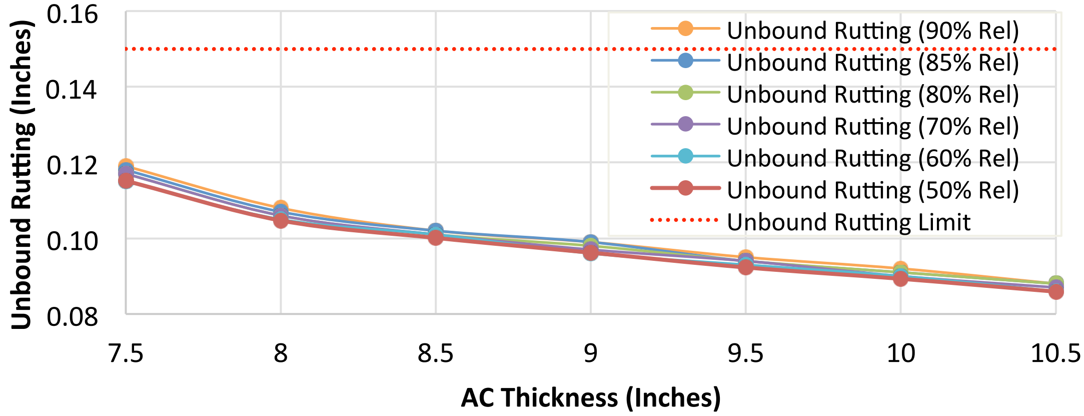

Figure 10 Effect of AC Thickness on Predicted Rutting in Unbound Layers (Principal Arterial)

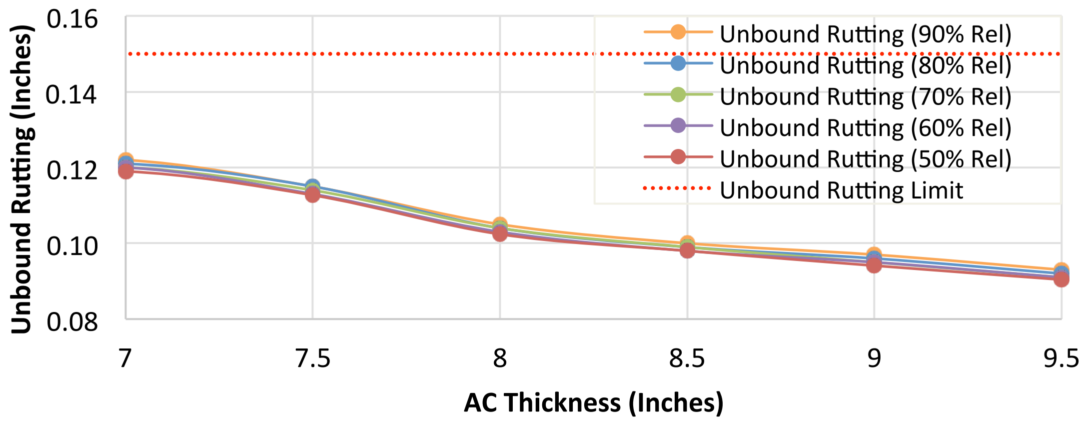

Figure 11 Effect of AC Thickness on Predicted Rutting in Unbound Layers (Minor Arterial)

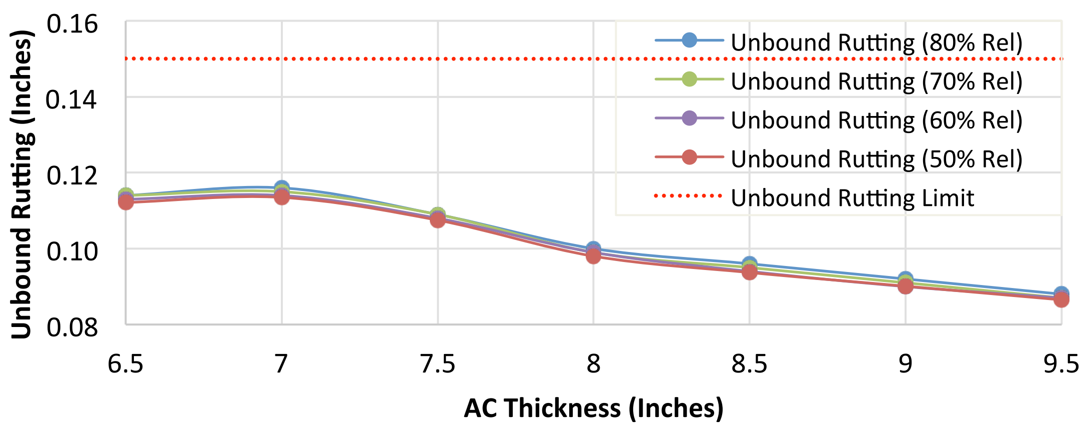

Figure 12 Effect of AC Thickness on Predicted Rutting in Unbound Layers (Major Collector)

6.2.2 Sensitivity of Bottom-Up Fatigue Cracking to Pavement Design Thickness

Figure 13 through Figure 16 shows the effect of AC thickness and reliability level on the bottom-up fatigue cracking using CDOT's local calibration coefficients. The dashed lines in these figures represent the performance criteria listed in Table 34. Based on these results, the following observations can be drawn:

- The effect of AC thickness on predicted bottom-up fatigue cracking was more significant for the pavements with thinner AC layers. Also, the effect of AC thickness on predicted bottom-up fatigue cracking was more significant than that on predicted rutting in the unbound layers.
- The impact of reliability level on predicted bottom-up fatigue cracking was more profound than that on predicted rutting in the unbound layers.

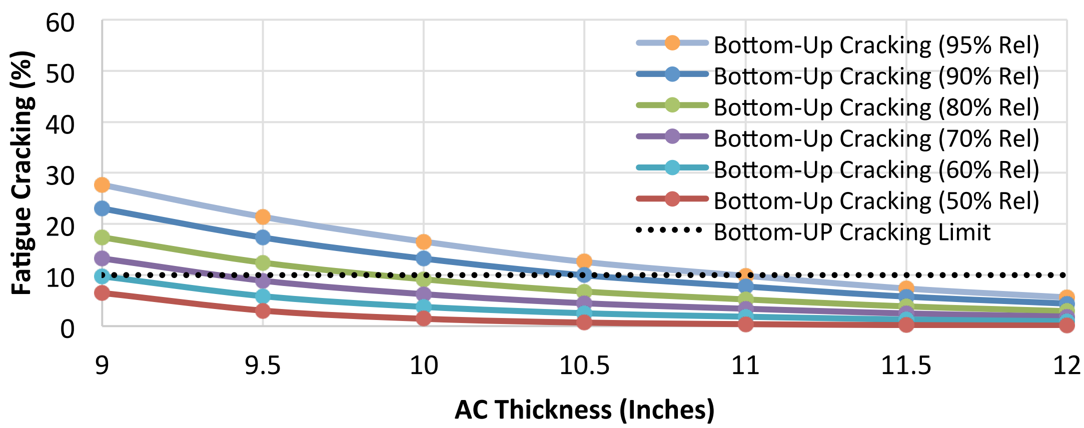

Figure 13 Effect of AC Thickness on Predicted Bottom-Up Cracking (Interstate)

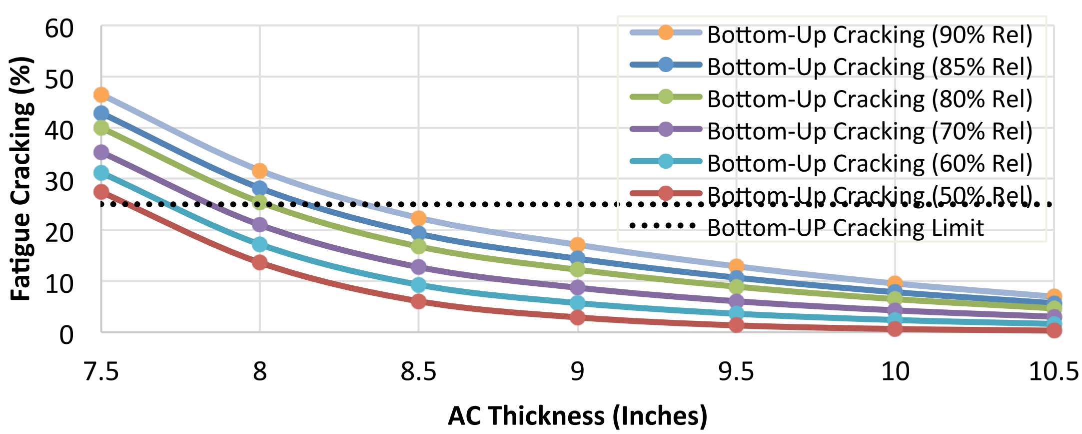

Figure 14 Effect of AC Thickness on Predicted Bottom-Up Cracking (Principal Arterial)

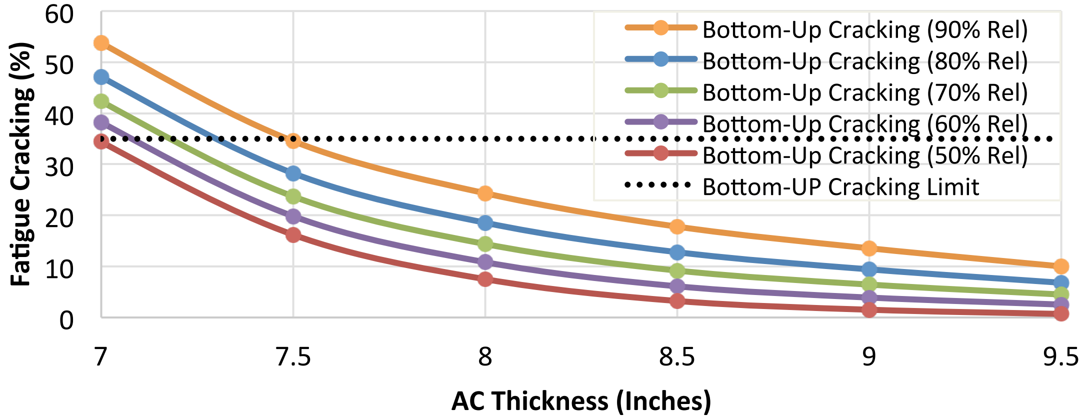

Figure 15 Effect of AC Thickness on Predicted Bottom-Up Cracking (Minor Arterial)

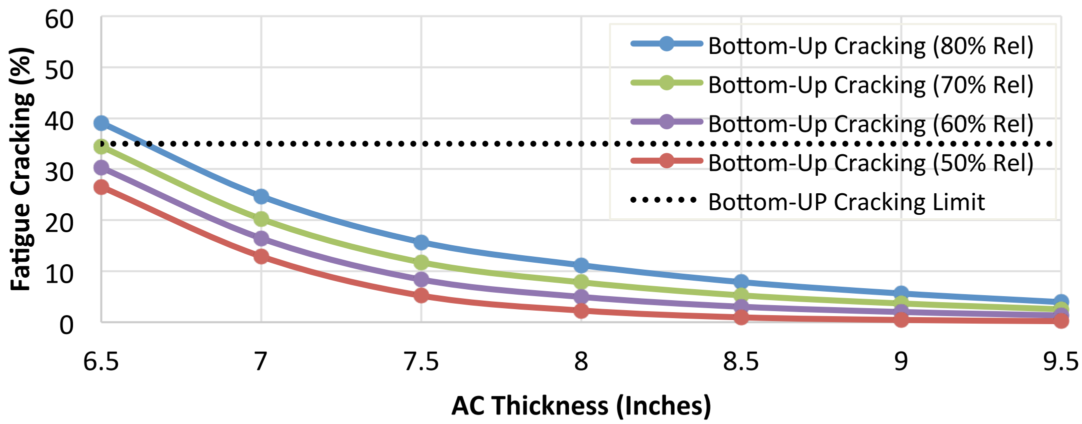

Figure 16 Effect of AC Thickness on Predicted Bottom-Up Cracking (Major Collector)

Figure 17 shows the levels of reliability that could be achieved for various AC thicknesses in this analysis if the performance limit for bottom-up fatigue cracking was set at 10% for interstates, 25% for principal arterials, and 35% for minor arterials and major collectors, as shown in Table 34. Based on these results, the following observations can be drawn:

- For the interstate design (Figure 17), when the reliability level achieved was between 80% and 90%, a 0.5-in. increase in AC thickness would improve the reliability level by approximately 10%. When the reliability level achieved was around 90%, a 0.5-in. increase in AC thickness would only improve the reliability level by approximately 5%. When the reliability level achieved was above 95%, a 0.5-in. increase in AC thickness would have a minimum effect on the design reliability level. This suggests that the reliability level for interstates could be set between 80% and 95%.
- For the principal arterial design (Figure 17), the design AC thickness would not be affected when the reliability level was selected between 80% and 90%.

- For the minor arterial and major collector design (Figure 17), the AC thickness for each design would be the same when the reliability level was selected between 70% and 90%.

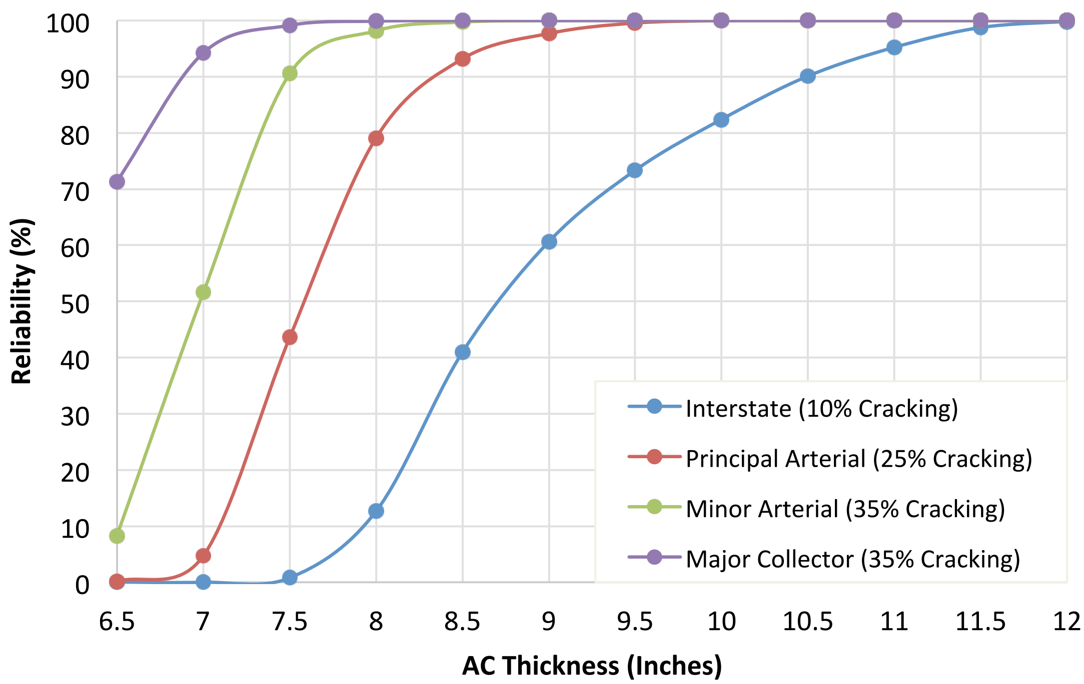

Figure 17 Reliability Level versus AC Thickness at Selected Performance Criteria

6.2.3 Proposed Performance Criteria and Reliability Levels for Pavement ME Design

Table 35 lists the proposed performance criteria and reliability levels that can be used in the future for Pavement ME Design. These values were selected based on the sensitivity analysis results and considering the performance criteria and reliability levels recommended in the Manual of Practice, shown in Table 31 and Table 32, and those adopted by CDOT, shown in Table 8. Further justification for the proposed design criteria and reliability levels follow:

- The proposed reliability levels are similar to those in the Manual of Practice (Table 32), except for rural interstate when a reliability level of 90% was found to be appropriate in the sensitivity analysis.
- The proposed performance limits for bottom-up fatigue cracking are the same as those adopted by CDOT.
- The proposed performance criteria for rutting in the lower layers were determined based on the total rutting and AC layer rutting criteria adopted by CDOT, except for interstate where a rut depth of 0.1 in. was found to be achievable in the sensitivity analysis. However, it should be noted that these criteria would only be utilized when the rutting models for the unbound layers have been properly calibrated. The nationally calibrated models have been found to over-predict rutting in the unbound layers.

- For the other distresses in Table 35, when the distresses predicted by Pavement ME Design are greater than the performance criteria, the designer may need to verify whether the levels of distress predicted are observed in the field or if other materials may be used to avoid these types of distress. In general, increasing the AC thickness would not reduce these distresses effectively. For example, a thicker AC layer would not result in lower rutting in the AC layer. Rather, choosing an asphalt binder with a lower low temperature grade would be more effective in mitigating thermal cracking than a thicker AC layer.

Table 35. Proposed Performance Criteria and Reliability Levels for Future Design

Classification	Reliability (%)		Performance Criteria						
	Urban	Rural	Terminal IRI (in/mi)	Rutting Total (in)	Rutting AC (in)	Rutting Unbound (in.)	Top-Down (ft/mi)	Bottom-Up (% lane)	Thermal Cracking (ft/mi)
Interstate	95	90	160	0.5	0.4	0.1	2,000	10	1,500
Principal Arterials	90	85	200	0.5	0.35	0.15	2,500	25	1,500
Minor Arterial	80	75	200	0.65	0.5	0.15	3,000	35	1,500
Major Collectors	75	70	200	0.65	0.5	0.15	3,000	35	1,500

7 CONCLUSIONS AND RECOMMENDATIONS

State agencies have considered implementing Pavement ME Design to replace the empirical Pavement Design Guides. Among other elements, their implementation plans often include two important steps—local calibration and selection of performance thresholds and reliability levels for accepting future designs. Past studies have shown that without locally calibrating Pavement ME Design, using the more sophisticated software would not yield better designs. Recognizing the importance of these steps, this report presents two case studies that compare pavement designs conducted with global and local calibration coefficients to illustrate the importance of conducting local calibration of Pavement ME Design in the implementation process. In addition, it discusses sensitivity analyses that show the effect of foundation support, performance criteria, and reliability levels on pavement design.

MoDOT and CDOT have completed their local calibration processes and adopted Pavement ME Design for routine pavement designs. The local calibration coefficients were used in a comparative analysis to illustrate the impact of local calibration on pavement designs for two pavement design projects in Missouri and Colorado. For both projects, flexible pavement designs were conducted using the global and local calibration coefficients. In addition, JPCP designs were also conducted for these projects for comparison. The following conclusions can be offered from this comparative analysis:

- For the project in Missouri, the new flexible pavement designs were governed by the predicted bottom-up cracking distresses. The bottom-up cracking model was found to be appropriate for pavements in Missouri, and its coefficients were not adjusted during local verification. Thus, the final thicknesses were the same for new flexible pavements using global and local calibration coefficients. For the new JPCP designs, the design using

the global calibration coefficients was 0.5 in. thicker than that using the local calibration coefficients.
- For the project in Colorado, the local calibration coefficients yielded a 1-in. thinner asphalt structure but the same Portland cement concrete structure as the global calibration coefficients. The final thickness designs were selected based on the predicted bottom-up cracking results. These designs failed the rutting criteria. However, rutting was not a performance issue in similar pavements in the area, so the designs were still accepted by CDOT.

The above comparative analysis was conducted for one type of aggregate base and subgrade; thus, a sensitivity analysis was later performed using CDOT's design to determine the impact of base and subgrade support on design AC thickness. Based on the analysis results, the following conclusions can be offered:

- As the stiffness of subgrade increased from 10,000 psi to 20,000 psi, the thickness of the asphalt structure would reduce by 1 inch. Also, when increasing the thickness of the aggregate base from 6 inches to 12 inches, the thickness of the asphalt structure changed by 0.5 inches.
- Compared to a 6-inch unbound aggregate base, the 6-inch stabilized base could yield 1.5-inch, 1.0-inch, and 0.5-inch thinner asphalt structure for 10,000 psi, 15,000 psi, and 20,000 psi subgrade, respectively. Also, the thickness of the asphalt structure decreased by 3 inches when the thickness of the cement stabilized base increased from 6 inches to 12 inches.
- All the designs with the unbound aggregate base were governed by the bottom-up fatigue cracking, and the designs with the stabilized base were governed by the IRI criteria.

Another sensitivity analysis was conducted using the CDOT's new flexible pavement structure and local calibration coefficients to evaluate the sensitivity of pavement design thickness to the performance criteria and reliability levels. Since these design parameters greatly affect the final design, they should be carefully selected in balance with each other. The analysis was conducted for four roadway classifications including interstate, principal arterial, minor arterial, and major collector. Based on the analysis, there were two types of pavement distresses that were found to be more sensitive to the changes in pavement design thickness, including bottom-up fatigue cracking and permanent deformation in the unbound layers. Key findings from this analysis are as follows:

- The effect of AC thickness on predicted bottom-up fatigue cracking and rutting in the unbound layers was more significant for thinner pavements. Also, the effect of AC thickness was more significant on bottom-up fatigue cracking than on rutting in the unbound layers. The impact of reliability level on predicted bottom-up fatigue cracking was also more profound than that on predicted rutting in the unbound layers.
- For the interstate design, a 0.5-in. increase in AC thickness would improve the reliability level from 80% to 90%. Or, it would improve the reliability level from 90% to 95%. However, it would have a minimum effect on the design reliability level when it was above 95%. For the principal arterial design, the design AC thickness would not be affected when the reliability level was selected between 80% and 90%. For the minor

arterial and major collector design, the AC thickness for each design would be the same when the reliability level was selected between 70% and 90%. This analysis was conducted with the performance limit for bottom-up fatigue cracking being set at 10% for interstates, 25% for principal arterials, and 35% for minor arterials and major collectors.

- Based on the sensitivity analysis results and considering the performance criteria and reliability levels recommended in the Manual of Practice and those adopted by CDOT, the performance criteria and reliability levels that can be used in the future for Pavement ME Design are proposed in Table 35.

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APPENDIX A PERFORMANCE MODELS FOR FLEXIBLE PAVEMENT DESIGN

A.1 Introduction

The MEPDG includes several performance (transfer) models to predict the following distresses:

- Rut depth—total, asphalt and unbound layers (in)
- Transverse (thermal) cracking (non-load related) (ft/mi)
- Fatigue (bottom-up fatigue) cracking (percent lane area)
- Longitudinal (top-down) cracking (ft/mi)
- Reflective cracking of asphalt overlays over asphalt, semi-rigid, composite and concrete pavements (percent lane area)
- International roughness index (IRI) (in/mi)

These models are presented in this appendix to facilitate the discussion of the local calibration results. The information is adapted from the Manual of Practice for the MEPDG (*3*) and the AASHTOWare Pavement ME Design software Version 2.0. When discrepancies are found between the two references, information in the software is presented.

A.2 Rut Depth for Asphalt and Unbound Layers

Two performance models are used to predict the total rut depth of flexible pavements and asphalt overlays: one for the asphalt layers and the other one for all unbound aggregate base layers and subgrades. Equation A.1 shows the asphalt rutting model developed based on laboratory repeated load plastic deformation tests.

$$\Delta_{p(AC)} = \varepsilon_{p(AC)} h_{(AC)} = \beta_{r1} k_z \varepsilon_{r(AC)} 10^{k_{r1}} n^{k_{r2}\beta_{r2}} T^{k_{r3}\beta_{r3}} \quad \text{(A.1)}$$

where:

- $D_{p(AC)}$ = Accumulated permanent or plastic vertical deformation in the asphalt layer or sublayer, in;
- $\varepsilon_{p(AC)}$ = Accumulated permanent or plastic axial strain in the asphalt layer or sublayer, in/in;
- $\varepsilon_{r(AC)}$ = Resilient or elastic strain calculated by the structural response model at the mid-depth of each asphalt layer or sublayer, in/in;
- $h_{(AC)}$ = Thickness of the asphalt layer or sublayer, in;
- n = Number of axle load repetitions;
- T = Mix or pavement temperature, °F;
- k_z = Depth confinement factor shown in Equation A.2;
- $k_{r1,r2,r3}$ = Global field calibration parameters (from the NCHRP 1-40D recalibration; k_{r1} = -3.35412, k_{r2} = 1.5606, k_{r3} = 0.4791); and
- $\beta_{r1,r2,r3}$ = Local or mixture field calibration constants; for the global calibration, these constants were all set to 1.0.

$$k_z = (C_1 + C_2 D)0.328196^D \quad \text{(A.2)}$$

$$C_1 = -0.1039(H_{HMA})^2 + 2.4868H_{HMA} - 17.342 \quad \text{(A.3)}$$

$$C_2 = 0.0172(H_{HMA})^2 - 1.7331H_{HMA} + 27.428 \quad \text{(A.4)}$$

where:

- *D* = Depth below the surface, in; and
- $H_{(AC)}$ = Total asphalt thickness, in.

Equation A.5 shows the field-calibrated transfer function for the unbound layers and subgrade.

$$\Delta_{p(soil)} = \beta_{s1} k_{s1} \varepsilon_v h_{soil} \left(\frac{\varepsilon_o}{\varepsilon_r}\right) e^{-\left(\frac{\rho}{n}\right)^{\beta}} \quad \text{(A.5)}$$

where:

- $D_{p(Soil)}$ = Permanent or plastic deformation for the layer or sublayer, in;
- *n* = Number of axle load applications;
- e_o = Intercept determined from laboratory repeated load permanent deformation tests, in/in;
- e_r = Resilient strain imposed in laboratory test to obtain material properties ε_o, β, and r, in/in;
- e_v = Average vertical resilient or elastic strain in the layer or sublayer and calculated by the structural response model, in/in;
- h_{soil} = Thickness of the unbound layer or sublayer, in;
- k_{sl} = Global calibration coefficients; k_{s1}=1.673 for granular materials and 1.35 for fine-grained materials; and
- β_{s1} = Local calibration constant for the rutting in the unbound layers; the local calibration constant was set to 1.0 for the global calibration effort.

$$log\beta = -0.6119 - 0.017638(W_c) \quad \text{(A.6)}$$

$$\rho = 10^9 \left(\frac{C_o}{1-(10^9)^{\beta}}\right)^{\frac{1}{\beta}} \quad \text{(A.7)}$$

$$C_o = Ln\left(\frac{a_1 M_r^{b_1}}{a_9 M_r^{b_9}}\right)^{\frac{1}{\beta}} = 0.0075 \quad \text{(A.8)}$$

where:

- W_c = Water content, percent;
- W_r = Resilient modulus of the unbound layer or sublayer, psi;

$a_{1,9}$ = Regression constants; a_1=0.15 and a_9=20.0; and
$b_{1,9}$ = Regression constants; b_1=0.0 and b_9=0.0.

A.3 Transverse (Thermal) Cracking

The amount of thermal cracking is estimated using Equation A.9 based on the probability distribution of the log of the crack depth to asphalt layer thickness ratio.

$$TC = \beta_{tl} N \left[\frac{1}{\sigma_d} log \left(\frac{C_d}{H_{HMA}} \right) \right] \tag{A.9}$$

where:

TC = Observed amount of thermal cracking, ft/mi;
β_{t1} = Regression coefficient determined through global calibration (400);
$N[z]$ = Standard normal distribution evaluated at [z];
σ_d = Standard deviation of the log of the depth of cracks in the pavement (0.769), in;
C_d = Crack depth, in; and
H_{AC} = Thickness of asphalt concrete layers, in.

The crack depth (C_d) induced by a given thermal cooling cycle is estimated using the Paris law of crack propagation, as shown in Equation A.10.

$$\Delta C = A(\Delta K)^n \tag{A.10}$$

where:

DC = Change in the crack depth due to a cooling cycle;
DK = Change in the stress intensity factor due to a cooling cycle; and
A, n = Fracture parameters for the HMA mixture, which are obtained from the indirect tensile creep-compliance and strength of the asphalt mixture using Equation A.11.

$$A = 10^{k_t \beta_t \left(4.389 - 2.52 Log(E_{AC} \sigma_m n)\right)} \tag{A.11}$$

where:

m = $0.8\left[1 + \frac{1}{m}\right]$;
k_t = Coefficient determined through global calibration for each input level (Level 1 = 5.0; Level 2 = 3.0; and Level 3 = 1.5);
E_{AC} = Asphalt concrete indirect tensile modulus, psi;
s$_m$ = Mixture tensile strength, psi;
m = M-value derived from the indirect tensile creep compliance curve; and
β_t = Local or mixture calibration factor (set to 1.0).

The stress intensity factor, K, is determined using Equation A.12.

$$K = \sigma_{tip}(0.45 + 1.99(C_o)^{0.56}) \quad \text{(A.12)}$$

where:

s $_{tip}$ = Far-field stress from pavement response model at depth of crack tip, psi; and
C_o = Current crack length, ft.

The following equations for transverse (thermal) cracking are according to the AASHTOWare Pavement ME Design software Version 2.1:

$$C_f = 400 \times N\left[\frac{\log\left(\frac{C}{h_{ac}}\right)}{\sigma}\right] \quad \text{(A.13)}$$

where:

C_f = Observed amount of thermal cracking, (ft/500 ft);
$N[z]$ = Standard normal distribution evaluated at [z];
C = Crack depth, in;
h_{ac} = Thickness of asphalt concrete layers, in; and
σ = Standard deviation of the log of the depth of cracks in the pavements.

The change in the crack depth due to a cooling cycle, ΔC, is calculated as shown in Equation A.14.

$$\Delta C = (k \times \beta_t)^{n+1} \times A \times \Delta K^n \quad \text{(A.14)}$$

where:

DC = Change in the crack depth due to a cooling cycle;
k = Regression coefficient determined through field calibration (Level 1 = 1.5; Level 2 = 0.5; and Level 3 = 1.5);
β_t = Calibration parameter;
DK = Change in the stress intensity factor due to a cooling cycle; and
A,n = Fracture parameters for the asphalt mixture, A is determined by Equation A.15.

$$A = 10^{\left(4.389 - 2.52 \times Log(E \times \sigma_m \times n)\right)} \quad \text{(A.15)}$$

where:

E = Mixture stiffness; and
s$_m$ = Undamaged mixture tensile strength.

A.4 Fatigue (Bottom-Up) Cracking

Fatigue cracking is assumed to initiate at the bottom of the asphalt concrete layers and propagate to the surface under truck traffic. The allowable number of axle load applications needed for the incremental damage index approach to predict both types of load related cracks (fatigue and longitudinal) is shown in Equation A.16 as it is shown in the Manual of Practice (*4*).

$$N_{f-AC} = k_{f1}(C)(C_H)\left(\beta_{f1}\right)(\varepsilon_t)^{k_{f2}\beta_{f2}}(E_{AC})^{k_{f3}\beta_{f3}} \tag{A.16}$$

where:

- $N_{f\text{-}AC}$ = Allowable number of axle load applications for a flexible pavement and asphalt overlays;
- ε_t = Tensile strain at critical locations and calculated by the structural response model, in/in;
- E_{AC} = Dynamic modulus of the HMA measured in compression, psi;
- $k_{f1,f2,f3}$ = Global field calibration parameters (from the NCHRP 1-40D re- calibration; k_{f1} = 0.007566, k_{f2} = -3.9492, and k_{f3} = -1.281);
- $\beta_{f1,f2,f3}$ = Local or mixture specific field calibration constants; for the global calibration effort, these constants were set to 1.0; and
- C_H = Thickness correction term, dependent on type of cracking.

$$C = 10^M \tag{A.17}$$

$$M = 4.84\left(\frac{V_{be}}{V_a+V_{be}} - 0.69\right) \tag{A.18}$$

where:

- V_{be} = Effective asphalt content by volume, %; and
- V_a = Percent air voids in the HMA mixture.

The allowable number of axle load applications as it is presented in the AASHTOWare Pavement ME Design software Version 2.1 is shown in Equation A.19. Equations A.17 and A.18 are applied in the same manner as in Equation A.16.

$$N_{f-AC} = 0.00432(C)\left(\beta_{f1}\right)(k_1)\left(\frac{1}{\varepsilon_1}\right)^{k_2\beta_{f2}}\left(\frac{1}{E}\right)^{k_3\beta_{f3}} \tag{A.19}$$

where:

- $N_{f\text{-}AC}$= Allowable number of axle load applications for a flexible pavement and asphalt overlays;

ε_t = Tensile strain at critical locations and calculated by the structural response model, in/in;
E = Dynamic modulus of the HMA measured in compression, psi;
$k_{1,2,3}$= Global field calibration parameters (k_1 = 0.007566, k_2 = 3.9492, and k_3 = 1.281); and
$\beta_{f1,f2,f3}$ = Local or mixture specific field calibration constants; for the global calibration effort, these constants were set to 1.0.

The allowable axle load applications were then used to determine the cumulative damage index (*DI*), which is a sum of the incremental damage indices over time as shown in Equation A.20.

$$DI = \sum(\Delta DI)_{j,m,l,p,T} = \sum\left(\frac{n}{N_{f-AC}}\right)_{j,m,l,p,T} \qquad \text{(A.20)}$$

where:

n = Actual number of axle load applications within a specific time period;
j = Axle load interval;
m = Axle load type (single, tandem, tridem, quad, or special axle configuration);
l = Truck type using the truck classification groups included in the MEPDG;
p = Month; and
l = Median temperature for the five temperature intervals or quintiles used to subdivide each month, ^{o}F.

The area of fatigue cracking is calculated from the cumulative damage index at the bottom of the AC layer over time using Equation A.21.

$$FC_{bottom} = \left(\frac{C_4}{1+e^{\left(C_1*C_1'+C_2*C_2'*log(D_k*100)\right)}}\right)*\left(\frac{1}{60}\right) \qquad \text{(A.21)}$$

where:

FC_{bottom} = Area of fatigue cracking that initiates at the bottom of the AC layers, percent of total lane area;
DI_{bottom} = Cumulative damage index at the bottom of the AC layers, percent; and
$C_{1,2,4}$ = Transfer function regression constants; C_4= 6,000; C_1=1; and C_2=1.

$$C_2' = -2.40874 - 39.748(1 + h_{AC})^{-2.856} \qquad \text{(A.22)}$$

$$C_1' = -2 * C_2' \qquad \text{(A.23)}$$

where:

h_{AC} = total thickness of asphalt layer, in.

A.5 Longitudinal (Top-Down) Cracking

Longitudinal cracks are assumed to initiate at the surface and propagate downward. The MEPDG uses Equations A.16 and A.20 to calculate the allowable number of axle load applications and cumulative damage index for fatigue and longitudinal cracks. The length of longitudinal cracking is then determined using Equation A.18.

$$FC_{top} = \left(\frac{C_4}{1+e^{\left(C_1 - C_2 * log\left(D_{top}\right)\right)}} \right) * 10.56 \tag{A.24}$$

where:

- FC_{top} = Length of longitudinal cracking that initiates at the surface, in;
- DI_{top} = Cumulative damage index at the surface, percent; and
- $C_{1,2,4}$ = Transfer function regression constants; C_4= 1,000; C_1=7; and C_2=3.5.

A.6 International Roughness Index (IRI)

The MEPDG uses Equation A.25 to predict IRI over time for AC pavements. This regression equation was developed based on data from the LTPP program.

$$IRI = IRI_0 + C_1(RD) + C_2(FC_{Total}) + C_3(TC) + C_4(SF) \tag{A.25}$$

where:

- IRI_0 = Initial IRI after construction, in/mi;
- RD = Average rut depth, in;
- FC_{Total} = Total area of load-related cracking (combined fatigue, longitudinal, and reflection cracking in the wheel path), percent of wheel path area;
- TC = Length of transverse cracking (including the reflection of transverse cracks in existing HMA pavements), ft/mi;
- $C_{1,2,3,4}$ = Regression constants; C_1 = 40; C_2 = 0.4; C_3 = 0.008; C_4= 0.015; and
- SF = Site factor (Equation A.26).

$$SF = Frosth + Swellp * \ Age^{1.5} \tag{A.26}$$

where:

- IRI_0 = Initial IRI after construction, in/mi; and
- Age = pavement age, year.

$$Frosth = Ln[(Precip + 1) * Fines * (FI + 1)] \tag{A.27}$$

$$Swellp = Ln[(Precip + 1) * Clay * (PI + 1)] \tag{A.28}$$

$$Fines = F_{sand} + Silt \quad \text{(A.29)}$$

where:

PI = subgrade soil plasticity index, percent
Precip = average annual precipitation or rainfall, in