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Local Calibration of AASHTOWare Pavement ME Design Coefficients for VDOT's Upgrade to Web-Based Software for New Pavement Design

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ELIZABETH TUROCHY, Ph.D., Research Scientist
Virginia Transportation Research Council

MESBAH AHMED, Ph.D., P.E., Senior Pavement Engineer
Materials Division, Virginia Department of Transportation

HARIKRISHNAN NAIR, Ph.D., P.E., Associate Director
Virginia Transportation Research Council

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530 Edgemont Road, Charlottesville, VA 22903-2454
vtrc.virginia.gov

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Author(s): Elizabeth Turochy, Ph.D., Mesbah Ahmed, Ph.D., P.E., Harikrishnan Nair, Ph.D., P.E.	
Performing Organization Name and Address: Virginia Transportation Research Council, 530 Edgemont Road, Charlottesville, VA 22903	

Sponsoring Agencies' Name and Address:
Virginia Department of Transportation
1221 E. Broad Street
Richmond, VA 23219

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Abstract:

The Virginia Department of Transportation (VDOT) maintains more than 130,000 lane- miles of roadway, making reliable and cost-effective pavement design essential to the long-term performance of the transportation network. Since adopting AASHTOWare Pavement ME Design version 2.2.6 in 2018, VDOT has relied on mechanistic-empirical (ME) pavement design to provide accurate distress predictions and optimized pavement structure designs for all new construction, reconstruction, and pavement widening projects. However, substantial advancements in the Pavement ME Design software—including updates to global calibration coefficients, revisions to distress models, enhancements to environmental data, and the transition from a desktop to a web-based platform—necessitate updated local calibration efforts to ensure continued design reliability under Virginia-specific conditions.

This study developed revised local calibration coefficients for distress prediction models under flexible (asphalt pavements with an unbound base or subbase), semi-rigid (asphalt pavements with a cement-treated base material), and continuously reinforced concrete pavement within the AASHTOWare Pavement ME web-based platform (version 3). Jointed plain concrete pavements were not considered in the research because of insufficient sites to use in local calibration. Using predicted versus measured pavement performance data extracted from VDOT's Pavement Management System, the research optimized calibration factors to align software predictions with observed field performance. These updated coefficients provide a reliable foundation for VDOT's transition from the desktop platform to the web-based application, supporting more accurate pavement designs and improved long-term performance.

The study recommends implementing the new calibration coefficients and associated design values directly within the web-based Pavement ME system. Because the updated platform includes a redesigned user interface and newly calibrated models, targeted training for VDOT users is strongly encouraged to ensure proper interpretation and application of design outputs. Periodic recalibration using current Pavement Management System performance data is also advised to maintain the long-term accuracy of the ME design models. Overall, this work enables VDOT to move confidently to the new web-based Pavement ME platform while ensuring that pavement designs continue to reflect Virginia's unique traffic, climate, and material conditions.

FINAL REPORT

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Elizabeth Turochy, Ph.D.
Research Scientist
Virginia Transportation Research Council

Mesbah Ahmed, Ph.D., P.E.
Senior Pavement Engineer / Materials Division
Virginia Department of Transportation

Harikrishnan Nair, Ph.D., P.E.
Associate Director
Virginia Transportation Research Council

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ABSTRACT

The Virginia Department of Transportation (VDOT) maintains more than 130,000 lane-miles of roadway, making reliable and cost-effective pavement design essential to the long-term performance of the transportation network. Since adopting AASHTOWare Pavement ME Design version 2.2.6 in 2018, VDOT has relied on mechanistic-empirical (ME) pavement design to provide accurate distress predictions and optimized pavement structure designs for all new construction, reconstruction, and pavement widening projects. However, substantial advancements in the Pavement ME Design software—including updates to global calibration coefficients, revisions to distress models, enhancements to environmental data, and the transition from a desktop to a web-based platform—necessitate updated local calibration efforts to ensure continued design reliability under Virginia-specific conditions.

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The study recommends implementing the new calibration coefficients and associated design values directly within the web-based Pavement ME system. Because the updated platform includes a redesigned user interface and newly calibrated models, targeted training for VDOT users is strongly encouraged to ensure proper interpretation and application of design outputs. Periodic recalibration using current Pavement Management System performance data is also advised to maintain the long-term accuracy of the ME design models. Overall, this work enables VDOT to move confidently to the new web-based Pavement ME platform while ensuring that pavement designs continue to reflect Virginia's unique traffic, climate, and material conditions.

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Mesbah Ahmed, Ph.D., P.E.
Senior Pavement Engineer / Materials Division
Virginia Department of Transportation

Harikrishnan Nair, Ph.D., P.E.
Associate Director
Virginia Transportation Research Council

INTRODUCTION

The Virginia Department of Transportation (VDOT) manages more than 130,000 lane-miles of roadway, making durable, cost-effective, and timely pavement design and maintenance essential to the performance of the roadway network. To support these goals, VDOT officially adopted the American Association of State Highway and Transportation Officials' (AASHTO) AASHTOWare Pavement ME Design (version 2.2.6) in 2018 as the standard methodology for all new construction, reconstruction, and pavement widening projects on interstate, primary, and secondary routes with an annual average daily traffic of more than 10,000. The *Mechanistic-Empirical Pavement Design Guide* and the Pavement ME software represent a significant improvement over previous empirical methods by using mechanistic-empirical (ME) principles. These tools enable more accurate pavement performance predictions, improved pavement structure design, and better consideration of traffic loading, climate, and material properties during a pavement's design life.

Prior to the initial implementation of AASHTOWare Pavement ME, VDOT performed research to establish Virginia-specific local calibration factors and design criteria values to ensure accurate ME pavement design (Smith and Nair, 2015). The study compared measured pavement performance—extracted from VDOT's Pavement Management System (PMS)—with the performance predicted by AASHTOWare Pavement ME for both flexible pavement and continuously reinforced concrete pavement (CRCP). Model coefficients within the software were adjusted to align predicted asphalt pavement performance with observed field performance. The resulting calibration coefficients and design requirements served as VDOT's baseline for adopting the ME design framework, supporting more optimized pavement structures and improved long-term performance across Virginia.

Since VDOT's adoption of Pavement ME Design desktop version 2.2.6, the software has undergone numerous updates and enhancements. These enhancements include improvements to software execution, revisions to global default layer and material properties, updates to global calibration coefficients, and the incorporation of new or revised distress prediction models. In addition, the underlying climate dataset has been updated to improve environmental inputs. A major milestone in this evolution was the release of the web-based application in July 2022 (AASHTO, 2024). With the desktop version of the software and its associated services being phased out—and given the central role that AASHTOWare Pavement ME plays in pavement design—ensuring continuity, accuracy, and operational readiness, as well as a seamless transition to the web-based platform, is essential. As with the original implementation, it is necessary to establish updated local calibration factors, verify or refine the design criteria, and refresh material properties and climatic inputs within the web-based platform. These updates ensure that the ME design models reflect current Virginia conditions and continue to support pavement designs that are cost effective, resilient, and consistent with performance expectations.

PURPOSE AND SCOPE

The purpose of this study was to develop updated local calibration coefficients for the distress transfer functions in the AASHTOWare Pavement ME web-based platform (version 3) that best represent observed pavement performance across VDOT's network. This work supports VDOT's transition from the desktop to the web-based version of the software by identifying relevant design scenarios and distress models, performing the required local calibrations, and developing guidelines for implementing these calibrations within the new platform.

METHODS

The purpose of this study was achieved through the following primary tasks:

1. Data compilation and data quality assessment.
2. Preliminary local calibration of distress models.
3. Refinement of local calibration coefficients.

Data Compilation and Data Quality Assessment

Within this task, pavement sections were selected, and measured distress data were extracted from VDOT's PMS database and from VDOT Materials Division for consideration in the local calibration effort. These sections were evaluated based on pavement type, service life, and their distribution across the state to achieve a set of data representative of the VDOT roadway network. They were also compared by route type, total pavement thickness, and subgrade soil. Once the roadway segments were selected for the local calibration effort, the pavement performance data from each section were evaluated to ensure that the data were consistent, practical, and suitable for analysis. The sampling matrix for flexible pavements, semi-rigid pavements, and CRCP were each evaluated for any potential gaps in the data, and if any were identified, they were addressed. For this task, researchers received support from Applied Research Associates (ARA), Inc.

Select Roadway Segments

Roadway segments from the Smith and Nair (2015) VDOT local calibration effort were selected as the initial set of test sections in this research. These segments provided a large, varied sample of projects to help provide a representative account for pavement types and performance across Virginia. Both asphalt (hereafter, referred to as *flexible*) pavement and CRCP were included in this selection; however, VDOT's upgrade from the currently used version of the Pavement ME Design software to the web-based version will include a calibration of the semi-rigid pavement design model in addition to flexible and CRCP models. The semi-rigid design covers flexible pavements containing a semi-rigid layer, such as cement-treated full-depth reclamation (FDR) and cement-treated aggregate (CTA). To calibrate the coefficients associated with this pavement type, a set of semi-rigid pavements was chosen. The semi-rigid design model was investigated prior to the initiation of this study and concluded that, because further refinement was necessary before implementation, it should be included in this local calibration study.

Extract and Evaluate Pavement Distress and Project Data

Once the roadway segments for calibration were selected, the associated pavement distress and project data were evaluated. Field performance records were extracted from PMS network-level distress data. These records provide distress data in yearly intervals, beginning in 2007, across all interstate and primary roadways. The distress data are collected and summarized at 0.1-mile increments. Distresses at each increment within a selected project were averaged to obtain an average distress measurement per site, per year.

For flexible pavement distress data, rut depth (inches) and fatigue cracking (% lane area) measurements were evaluated in the calibration process. For the calibration process, low-severity fatigue cracking (Level 1) within PMS was assumed to be top-down cracking within the Pavement ME Design cracking predictions. Medium- and high-severity fatigue cracking (Levels 2 and 3) were assumed to be bottom up cracking. These methods were followed per recommendations in the AASHTO local calibration guide that suggest combining cracking types if the location where cracking initiated is not known and adjusting the bottom up cracking model to fit the data (AASHTO, 2020).

For semi-rigid pavement distress data, total fatigue cracking, total transverse cracking, and total rutting were evaluated. The total fatigue cracking is a combination of reflective cracking from the cement-stabilized layer (either CTA or FDR) and bottom up cracking originating within the asphalt pavement layers. For the total transverse cracking, both the transverse cracking propagating from the cement-stabilized layer and any thermal transverse cracking originating from the asphalt pavement layers contribute to the distress values.

For CRCP distress data, punchouts (#/mile) were evaluated from the PMS database. The main distress that is predicted for CRCP in Pavement ME Design is punchouts, which are heavily influenced by the software-determined cracking spacing.

Preliminary Local Calibration of Distress Models

After Task 1 was completed, the pavement material, climate, and traffic data from the selected sections were input into the web-based AASHTOWare Pavement ME software to generate predicted pavement performance. These predicted values were then compared with measured distress values collected from PMS to assess the accuracy of the global coefficients within the pavement distress models. Based on these comparisons, the calibration coefficients were adjusted to better match predicted and actual pavement performance for the selected sections and to improve the bias and standard error of the estimate (SEE). For this task, researchers received support from ARA, Inc., and the local calibration process for this research followed similar methods to those detailed in the AASHTO Guide for Local Calibration, or local calibration guide (AASHTO, 2026).

Select Input Level for Each Parameter

VDOT's (2017) *Pavement ME User Manual* details VDOT's current procedure for inputting details needed to perform pavement analyses using the Pavement ME Design software. Climate, traffic, and material inputs used in this research are based on these procedures and data to mimic what would be used in design applications.

Assess and Reduce Local Bias and Standard Error of the Estimate

When evaluating the fit of the distress models, two primary parameters are considered—bias and SEE. These terms are defined as the systematic offset between predicted and observed values and the variability between the predicted and measured values, respectively (AASHTO, 2026). The bias is a calculation of the average of the individual residual errors, which is the difference between the measured and predicted distresses for each section. SEE is another way to evaluate the residual errors, being the standard deviation of the data points. These values are produced by the Calibration Assistance Tool (CAT) software during the calibration coefficient optimization process. For any optimization iterations conducted, these values were produced to evaluate the model suitability. During the calibration process, the goal is to minimize residual error and SEE. The validation process involves using the optimized coefficients that resulted in minimal residual error and SEE and applying them to data not used in the calibration process to ensure the validity of the model. The ratings (e.g., Very Good) associated with the bias and SEE statistics reported for each distress evaluated are based on a rating system produced by ARA, Inc., as guidance for interpreting these statistics as a part of the local calibration process. Appendix B provides a table detailing the rating systems.

Refinement of Local Calibration Coefficients

Once the preliminary local calibration coefficients were determined, a refinement process was conducted for both the flexible and semi-rigid distress models. This analysis evaluated the inputs and calibration coefficients used in the pavement distress models, quantified the level of impact each input had on predicted pavement performance, and refined the locally calibrated coefficients to best fit the needs in the VDOT roadway network.

RESULTS AND DISCUSSION

Data Compilation and Data Quality Assessment

Final Roadway Segments

After the sections from the previous VDOT local calibration effort were selected and additional sites were supplemented to create a representative sample of pavement types within the VDOT roadway network, 36 flexible sections, 35 semi-rigid sections, and 25 CRCP sections were deemed suitable to use in the local calibration effort. Figure 1 shows a map with the section distribution across the state.

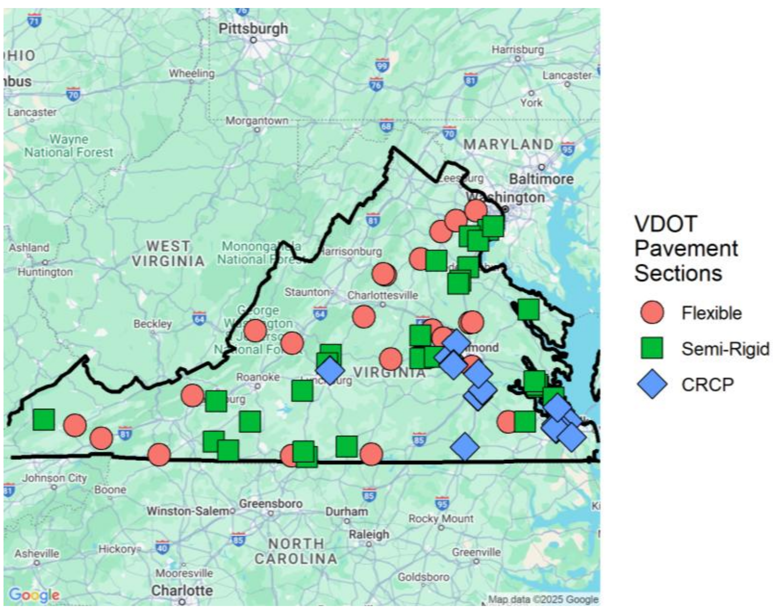

Figure 1. Map of Pavement Sections Selected to Use in the Local Calibration Effort. CRCP = continuously reinforced concrete pavement.

Table 1 lists the flexible pavement sites selected for local calibration. These locations cover eight of the nine VDOT districts; no sites were identified from the Hampton Roads District due to a significant amount of the roadways being composite (flexible pavement over concrete), which was not within the scope of this project. The Site ID tag is an identifier used to keep track of the projects during calibration. Throughout the flexible distress model calibration, sections were withheld occasionally because of outlying data points. Examples are detailed in the Flexible Pavement Calibration section.

Table 1. Flexible Pavement Calibration Sites

Site ID	District	County	Route	Route Type	Direction	Length (mile)
Br-1	Bristol	Lee	58	US	E	3.24
Br-2	Bristol	Washington	91	SR	N	1.60
Br-3	Bristol	Grayson	58	US	E	4.06
Br-5	Bristol	Russell	19	US	N	5.76
Sa-1	Salem	Pulaski	100	SR	N	2.73
Ly-1	Lynchburg	Pittsylvania	58	US	E	7.93
Ly-2	Lynchburg	Pittsylvania	58	US	W	7.93
Ri-1	Richmond	Goochland	288	SR	N	6.04
Ri-2	Richmond	Hanover	30	SR	E	0.95
Ri-4	Richmond	Henrico	895	SR	E	7.88
Ri-5	Richmond	Henrico	895	SR	W	7.88
Ri-6	Richmond	Mecklenburg	58	US	E	3.27
Ri-7	Richmond	Mecklenburg	58	US	W	3.27
Ri-8	Richmond	Goochland	64	IS	W	3.91
Ri-9	Richmond	Goochland	64	IS	E	1.24
Ri-10	Richmond	Goochland	64	IS	E	1.83
Ri-11	Richmond	Mecklenburg	58	US	E	3.18
Ri-12	Richmond	Mecklenburg	58	US	W	2.25
Fr-4	Fredericksburg	Caroline	30	SR	E	1.15
Cu-1	Culpeper	Culpeper	299	SR	N	0.62
Cu-2	Culpeper	Culpeper	299	SR	S	0.62
Cu-4	Culpeper	Fauquier	28	SR	N	0.88
Cu-5	Culpeper	Albemarle	29	US	N	0.43
Cu-6	Culpeper	Greene	33	US	E	1.40
Cu-7	Culpeper	Greene	33	US	E	2.87
Cu-9	Culpeper	Louisa\Fluvanna	15	US	N	0.54
Cu-12	Culpeper	Fauquier	15	US	B	0.79
St-1	Staunton	Rockbridge	81	IS	N	0.80
St-2	Staunton	Rockbridge	81	IS	S	0.80
St-3	Staunton	Alleghany	64	IS	E	2.20
St-4	Staunton	Alleghany	64	IS	W	2.20
NO-2	NOVA	Prince William	619	SC	E	3.07
NO-3	NOVA	Fairfax	608	SC	N	1.80
BR-F-6	Bristol	Washington	54	US	E	2.44
SA-F-8	Salem	Montgomery	81	IS	S	5.30
ST-F-7	Staunton	Rockbridge	81	IS	N	7.00

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

Table 2 lists the semi-rigid pavement sites selected for local calibration. All semi-rigid sections contained a flexible pavement surface over a cement-stabilized base, either CTA or FDR. The Site ID tag is an identifier used to keep track of the projects during calibration. Throughout the semi-rigid distress model calibration, sections were withheld occasionally because of outlying data points. Examples are detailed in the Semi-Rigid Pavement Calibration section.

Table 2. Semi-Rigid Pavement Calibration Sites

Site ID	District	County	Route	Route Type	Direction	Length (mile)
Br-1-SR	Bristol	Wise	58 (Alt.)	US - Alt	B	2.40
Sa-1-SR	Salem	Montgomery	81	IS	N	4.67
Sa-2-SR	Salem	Montgomery	81	IS	S	4.67
Sa-3-SR	Salem	Patrick	58	US	E	2.32
Sa-4-SR	Salem	Patrick	58	US	E	1.16
Sa-5-SR	Salem	Bedford	24	SR	B	2.99
Sa-6-SR	Salem	Franklin	40	SR	B	0.50
Ly-1-SR	Lynchburg	Pittsylvania	29	US	N	7.30
Ly-2-SR	Lynchburg	Pittsylvania	41	SR	B	2.80
Ly-3-SR	Lynchburg	Amherst	29	US	B	1.34
Ly-4-SR	Lynchburg	Halifax/Town of S. Boston	360	US	B	1.29
Ly-5-SR	Lynchburg	Amherst	130	SR	B	1.34
Ri-1-SR	Richmond	Powhatan	13	SR	B	3.56
Ri-2-SR	Richmond	Goochland	6	SR	B	3.63
Ri-3-SR	Richmond	Powhatan	13	SR	B	0.71
Ri-4-SR	Richmond	Powhatan	60	US	W	1.66
HR-1-SR	Hampton Roads	York	64 (Seg. II)	IS	E	4.51
HR-2-SR	Hampton Roads	York	64 (Seg. II)	IS	W	5.00
HR-3-SR	Hampton Roads	York	64 (Seg. III)	IS	W	2.00
HR-3-SR-Spl	Hampton Roads	York	64 (Seg. III)	IS	E	2.00
HR-4-SR	Hampton Roads	Isle of Wight	620	Secondary	B	1.37
HR-5-SR	Hampton Roads	York	620	Secondary	B	1.16
Fr-1-SR	Fredericksburg	Stafford	1	US	N	0.96
Fr-2-SR	Fredericksburg	Spotsylvania	208	SR	N	1.91
Fr-3-SR	Fredericksburg	Spotsylvania	208	SR	S	1.91
Fr-4-SR (E)	Fredericksburg	Richmond	3	SR	E	1.43
Fr-4-SR (W)	Fredericksburg	Richmond	3	SR	W	1.43
Cu-1-SR	Culpeper	Culpeper	3	SR	E	1.57
St-1-SR	Staunton	Augusta	81	IS	S	0.41
St-2-SR	Staunton	Augusta	81	IS	S	3.25
NO-1-SR	NOVA	Fairfax	642	Secondary	E	0.74
NO-2-SR	NOVA	Prince William	234	SR	N	3.15
NO-3-SR	NOVA	Prince William	234	SR	N	3.52
NO-4-SR	NOVA	Fairfax	611	Secondary	N	1.66
HR-SR-7	Hampton Roads	York	105	SR	W	3.16

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

Table 3 shows the CRCP sites used for concrete pavement calibration. Like the other pavement types, the Site ID tag is an identifier used to keep track of the projects during calibration. Throughout the CRCP distress model calibration, sections were withheld occasionally because of outlying data points or data that significantly influenced the distress model results when the predicted data did not reflect distress typically seen within the VDOT network. Examples are detailed in the Continuously Reinforced Concrete Pavement Calibration section. Because of site number limitations, jointed plain concrete pavements were not considered in the local calibration process.

Table 3. CRCP Calibration Sites

Site ID	County	Route	Route Type	Direction	Length (mile)
PCC-13 EB CRCP	York	664	IS	E	2.57
PCC-13 WB CRCP	York	664	IS	W	2.46
PCC-17-CRCP	Chesterfield	288	SR	S	7.99
PCC-20-CRCP	Greensville	58	US	E	1.50
PCC-6-CRCP-295 SB	Henrico	295	IS	S	1.22
PCC-9-NB	Norfolk	564	IS	N	0.53
PCC-9-SB	Norfolk	564	IS	S	0.53
PCC-10-CRCP	Nansemond	664	IS	E	1.61
PCC-11-CRCP	Nansemond	664	IS	W	1.57
PCC-12-CRCP	Norfolk	664	IS	E	3.85
PCC-15	Nansemond	164	SR	W	0.87
PCC-7-CRCP-295 NB	Prince George	295	IS	N	3.26
PCC-7-CRCP-295 SB	Prince George	295	IS	S	0.70
PCC-8-CRCP-295 NB	Prince George	295	IS	N	6.03
PCC-8-CRCP-295 SB	Prince George	295	IS	S	4.88
PCC-18-CRCP-NB	Chesterfield	288	SR	N	6.41
PCC-18-CRCP 288 SB	Chesterfield	288	SR	S	5.21
PCC-19 Lynchburg Route 29N	Amherst	29	US	N	7.04
PCC-19 Lynchburg Route 29S	Amherst	29	US	S	7.02
PCC-2-CRCP-64EB	York	64	IS	E	2.31
PCC-2-CRCP-64WB	York	64	IS	W	2.18
PCC-22-CRCP 64EB (Battle)	Norfolk Maintenance Area	64	IS	E	1.00
PCC-23-CRCP 64WB (Battle)	Norfolk Maintenance Area	64	IS	W	1.00
PCC-24-CRCP 295NB	Chesterfield	295	IS	N	2.00
PCC-25-CRCP 295SB	Chesterfield	295	IS	S	2.00

CRCP = continuously reinforced concrete pavement; IS = interstate route; SR = state route; US = U.S. route.

Data Quality Assessment

For each section, the 0.1-mile PMS distress data were evaluated to determine if any distress variability or outliers existed. This detailed evaluation provided a more detailed review of individual data measurements and visualized the variability within each pavement section on a year-by-year basis. Through this process, any distress variabilities were addressed, and outliers were removed. Once the quality assessment was completed, the data were deemed suitable to use in the local calibration process.

Preliminary Local Calibration of Distress Models

Flexible Pavement Calibration

ARA, Inc., conducted the preliminary flexible pavement local calibration effort using the AASHTOWare Pavement CAT program. The previously selected flexible pavement sections for local calibration and their corresponding measured bottom up cracking, top down cracking, and rutting were uploaded to CAT to start the local calibration process. For each distress, a separate CAT project was created to perform the verification, calibration, and validation. Once the initial data verification was complete, calibration coefficient optimization was conducted. The round of model calibration conducted using the CAT software served as the preliminary calibration of the model. After the recommended coefficients from this calibration were produced, further optimization was conducted to better refine the coefficients to meet pavement performance and design standards used by VDOT. This effort produced a final set of recommended coefficients suitable for implementation.

For the bottom up and top down cracking prediction models, optimization of the calibration coefficients began with the transfer function. Within these coefficients, some were prioritized to reduce bias, whereas others held a greater influence over improving SEE. The set of flexible roadway sections for calibration surpassed the minimum required for CAT; therefore, the sections were randomly split so that 80% of the sections could be used to optimize the model and the other 20% could be used for model validation. Appendix A presents the distress prediction equations along with their corresponding calibration coefficients.

Bottom Up Cracking

Initial verification results of the bottom up cracking model yielded almost no bias and fair SEE; however, the data distributed along the underprediction zone indicate a need for local calibration (Figure 2). Based on these initial verification results, the data were assessed for sections of significant over and underprediction of the bottom up cracking. Sections that were considered outliers were removed prior to calibration. Figure 2 shows the initial verification results after removing the outliers.

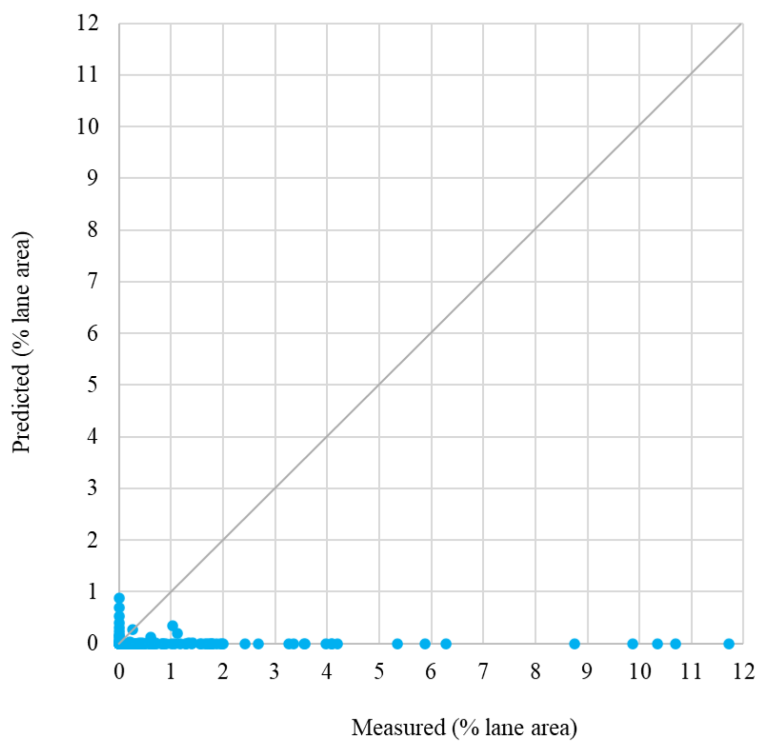

Figure 2. Initial Verification Results of the Bottom Up Cracking Model for Flexible Pavements, Excluding Outliers

Once initial verification was complete, the preliminary model coefficient optimization began. During this process, six of the selected sections were deemed outliers in the bottom up cracking distress model analysis, thus resulting in their removal from the calibration set for this distress type. Adjustments to the C1 coefficient of the transfer function, related to the level of predicted load-related cracking, were made in three iterations. Through the iterations, bias was eliminated, and SEE was reduced compared with the initial verification value. These adjustments proved sufficient for preliminary optimization, thus requiring no other coefficient adjustments. Figures 3 and 4 show the data from the optimization and the validation results for this process, respectively. The preliminary model reduced the overall bias while slightly increasing SEE compared with the initial verification results. Table 4 shows the model coefficient preliminary optimization results.

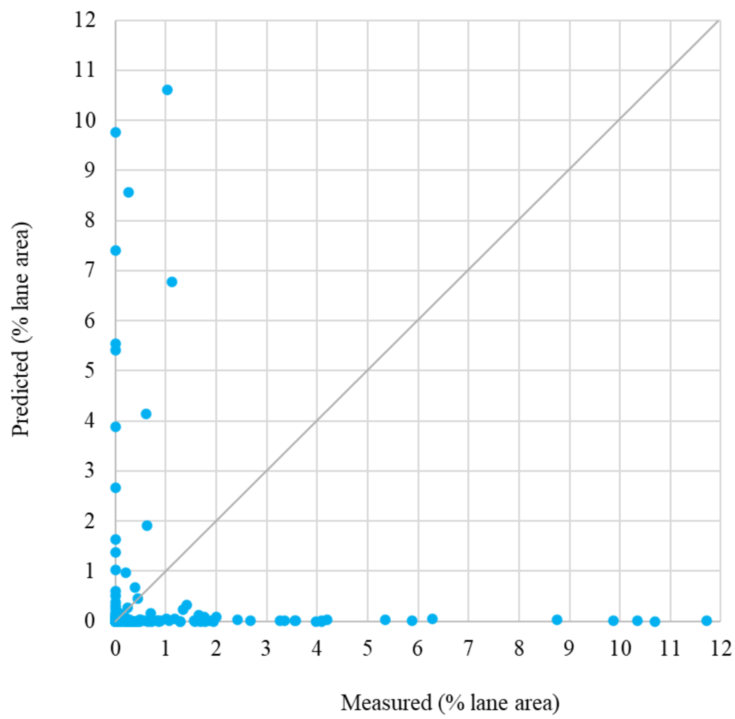

Figure 3. Measured Versus Predicted Bottom Up Cracking for Flexible Pavements from the Preliminary Optimization Process

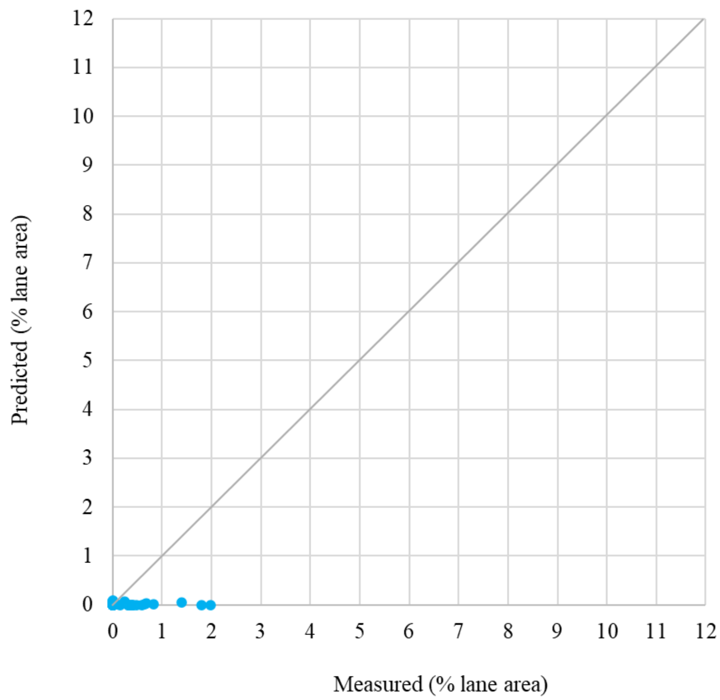

Figure 4. Measured Versus Predicted Bottom Up Cracking for Flexible Pavements from the Preliminary Validation Process

Table 4. Bottom Up Cracking Model Coefficient Preliminary Optimization Results

Calibration Iteration #	Adjusted Coefficient	Adjustment Range	Accepted Coefficient	Bias (% of area)	SEE (% of area)
Verification	N/A	N/A	N/A	-0.54	1.68
1	C1	0.3–0.8	0.5857	-0.01	3.37

N/A = not applicable; SEE = standard error of the estimate.

Table 5 shows the statistical adequacy ratings for the preliminary bottom up cracking model verification, calibration, and validation of the final selected coefficients. The calibration results show improved values for the bias and SEE, with a poor R^2 value.

Table 5. Bottom Up Cracking Model Coefficient Preliminary Optimization Statistics Ratings

Flexible Bottom Up Cracking Model Statistics Ratings	Verification		Optimization		Validation	
Test Statistic	**Value**	**Rating**	**Value**	**Rating**	**Value**	**Rating**
Bias	0.8	-----	-0.06	-----	-1.08	-----
SEE	6.01	Poor	1.97	Good	3.13	Fair
R^2	0.06	Poor	0.13	Poor	0.01	Poor
Hypothesis testing of slope of the linear measured vs. predicted: True if slope = 1.	0.57	Reject	0.00	Reject	0.00	Reject
Hypothesis testing of intercept of the linear measured vs. predicted: True if intercept = 0.	0.0	Reject	0.00	Reject	0.00	Reject
Paired t-test between measured and predicted: True if difference = 0.	0.016	Reject	0.612	Accept	0.011	Reject

SEE = standard error of the estimate.

Top Down Cracking

The top down cracking initial verification results indicated a positive bias leading to overpredicted measured top-down cracking. No outliers were identified in the initial verification. Figure 5 shows the results.

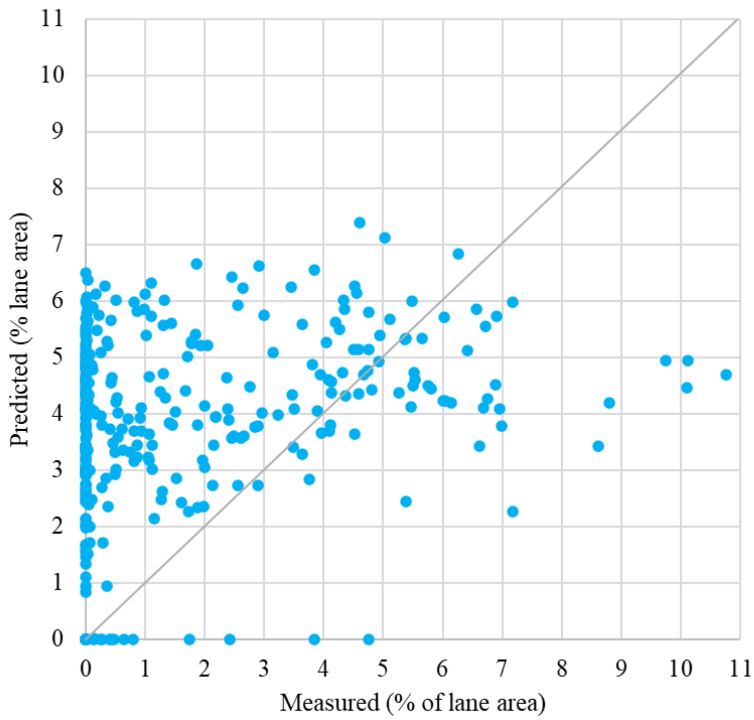

Figure 5. Initial Verification of the Top Down Cracking Model for Flexible Pavements

Once initial verification was complete, the preliminary model coefficient optimization began. Adjustments to the C2 coefficient of the transfer function were made in three iterations. Through these iterations, eliminating bias and reducing SEE were the primary objectives. Table 6 shows the results from the preliminary coefficient calibration effort, in which both bias and SEE were reduced. Figures 6 and 7 show the data from the optimization and the validation results for this process, respectively.

Table 6. Top Down Cracking Model Preliminary Optimization Results

Calibration Iteration #	Adjusted Coefficient	Adjustment Range	Accepted Coefficient	Bias (% of area)	SEE (% of area)
Verification	N/A	N/A	N/A	2.02	3.36
1	C2	0.6–1.0	-	0.37	2.39
2	C2	1.0–1.5	-	-0.40	2.34
3	C2	1.0–1.125	1.0625	-0.05	2.32

N/A = not applicable; SEE = standard error of the estimate.

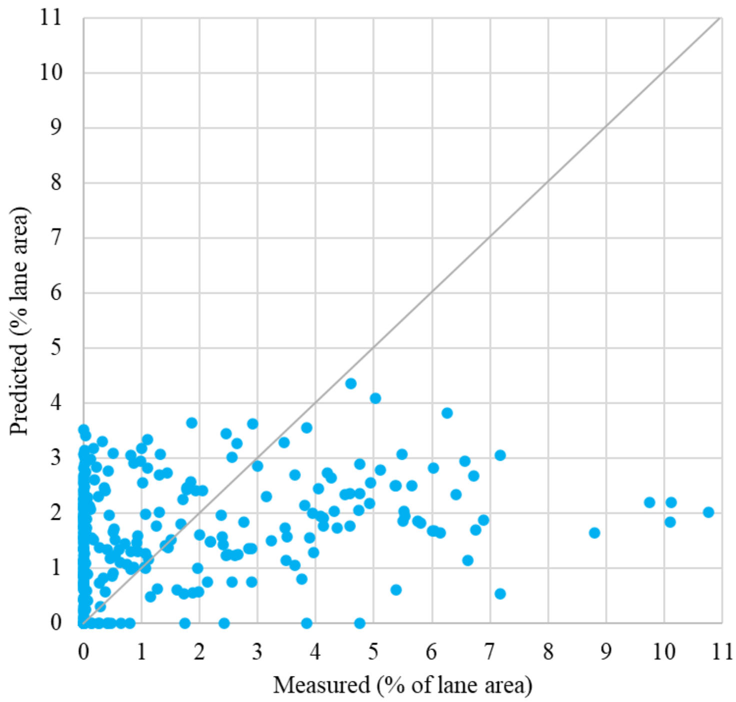

Figure 6. Measured Versus Predicted Top Down Cracking for Flexible Pavements from the Preliminary Optimization Process

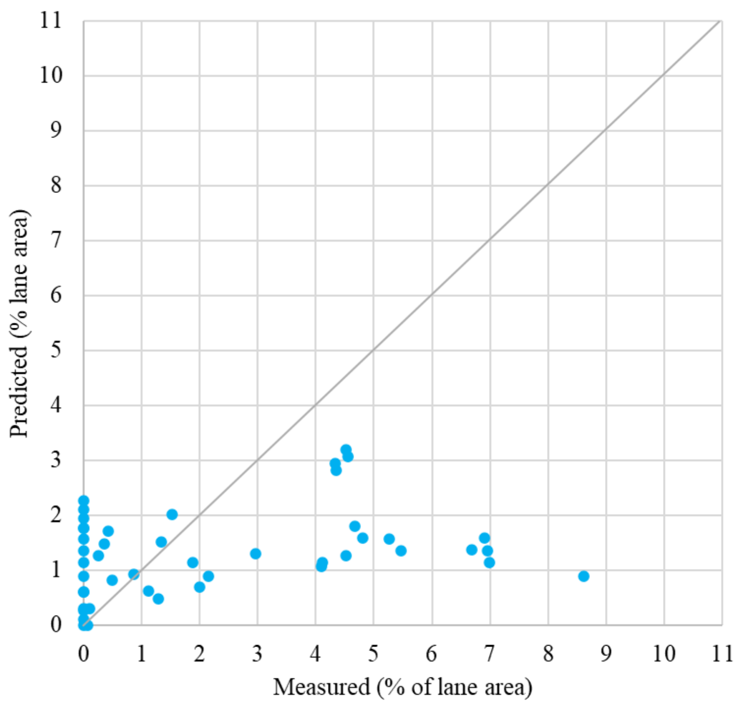

Figure 7. Measured Versus Predicted Top Down Cracking for Flexible Pavements from the Preliminary Validation Process

Table 7 shows the statistical adequacy ratings for the preliminary top down cracking model verification, optimization, and validation of the final selected coefficients. The optimization results show a slight improvement in SEE versus initial verification and an even greater improvement in the bias. However, the R^2 value remains poor.

Table 7. Top Down Cracking Model Coefficient Preliminary Optimization Statistics Ratings

Flexible Top Down Cracking Model Statistics Ratings	**Verification**		**Optimization**		**Validation**	
Test Statistic	Value	Rating	Value	Rating	Value	Rating
Bias	2.02	-----	-0.05	-----		-----
SEE	3.36	Good	2.32	Good	3.27	Good
R2	0.06	Very Poor	0.06	Poor	0.06	Poor
Hypothesis testing of slope of the linear measured vs. predicted: True if slope = 1.	0.00	Reject	0.00	Reject	0.00	Reject
Hypothesis testing of intercept of the linear measured vs. predicted: True if intercept = 0.	0.00	Reject	0.00	Reject	0.00	Reject
Paired t-test between measured and predicted: True if difference = 0.	0.00	Reject	0.699	Accept	0.01	Reject

SEE = standard error of the estimate.

Total Rutting

The initial verification results of the rutting model showed consistent overprediction in measured rutting, despite minimal bias and SEE. A detailed review was performed to determine if the overprediction of rutting is a function of total asphalt thickness, subgrade type, or both. Most of the selected sections for local calibration contain a substantial asphalt thickness, which should confine the measured rut depths to the asphalt layers and minimize the amount of rutting contributed to the base and subgrade layers. Figure 8 shows the initial verification results for total rutting.

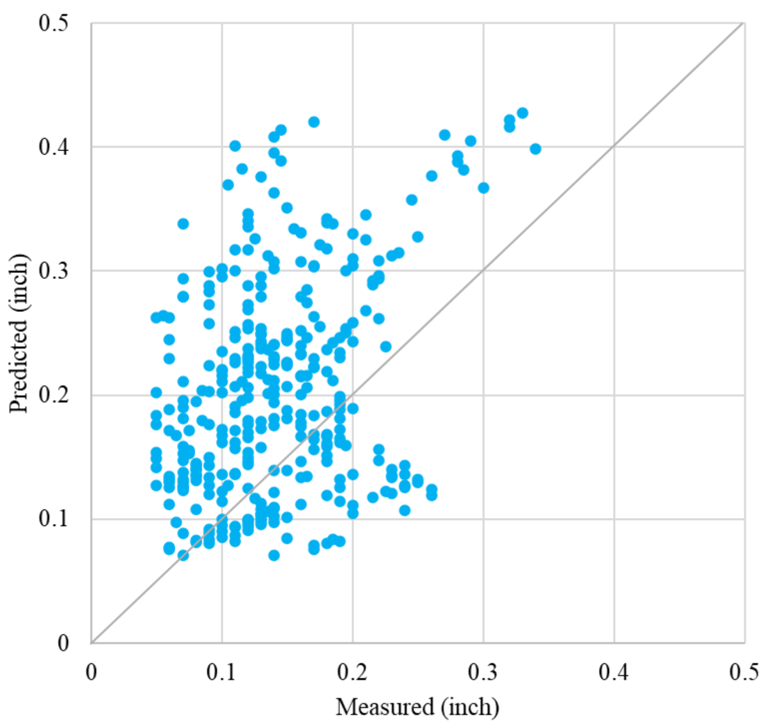

Figure 8. Initial Verification Results of the Total Rutting Model for Flexible Pavement

The data were subjected to additional analysis to compare pavements with an asphalt thickness of greater than 10 inches versus those with less than 10 inches to determine the influence of the total asphalt layer thickness on predicted rutting. In flexible pavements with a total asphalt layer thickness of greater than 10 inches, the model both overpredicts and underpredicts total rutting with a slight bias toward overprediction. However, sections with a total asphalt thickness of less than 10 inches showed a consistent overprediction of rutting across the entire pavement structure.

To address the overprediction of rutting, both the subgrade and the asphalt layer rutting model coefficients were investigated. First, Bs1, the subgrade rutting coefficient, was optimized to reduce the overall subgrade rutting predictions, which then affects the total rutting predictions. Preliminary optimization runs consisted of reducing the Bs1 coefficient from a value of 1 to a range of values between 0.25 and 0.75. Next, the Br1 intercept coefficient for the asphalt rutting model transfer function was adjusted within 0.1 to 0.35 to further optimize the bias and SEE results. Table 8 shows the preliminary optimization of the rutting model. Figures 9 and 10 show the data from the optimization and the validation results for this process, respectively.

Table 8. Total Rutting Model Preliminary Optimization Results

Calibration Iteration #	Adjusted Coefficient	Adjustment Range	Accepted Coefficient	Bias (in)	SEE (in)
Verification	N/A	N/A	N/A	0.06	0.11
1	Bs1	0.25–0.75	0.5	0.01	0.08
2	Br1	0.1–0.35	0.35	0.00	0.07

N/A = not applicable; SEE = standard error of the estimate.

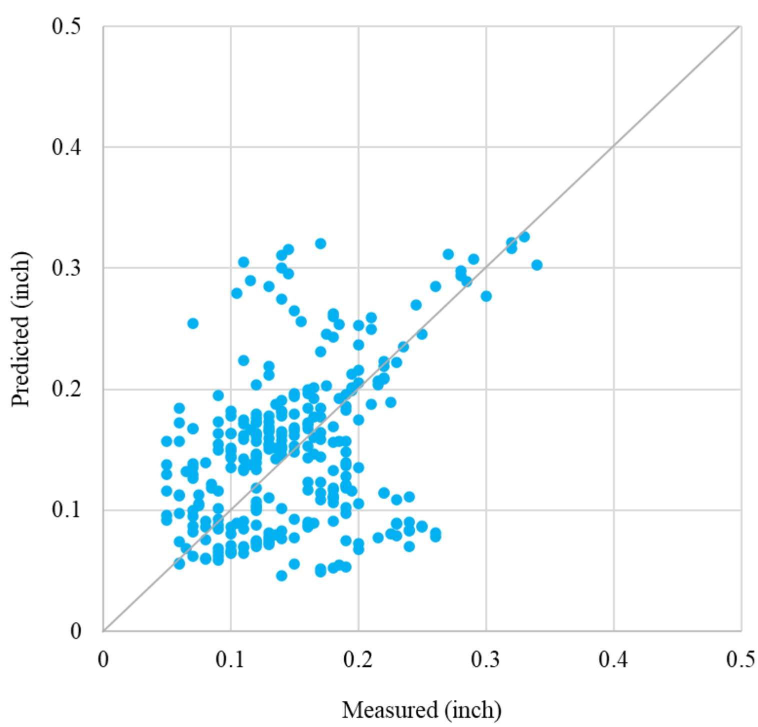

Figure 9. Measured Versus Predicted Total Rutting for Flexible Pavements from the Preliminary Optimization Process

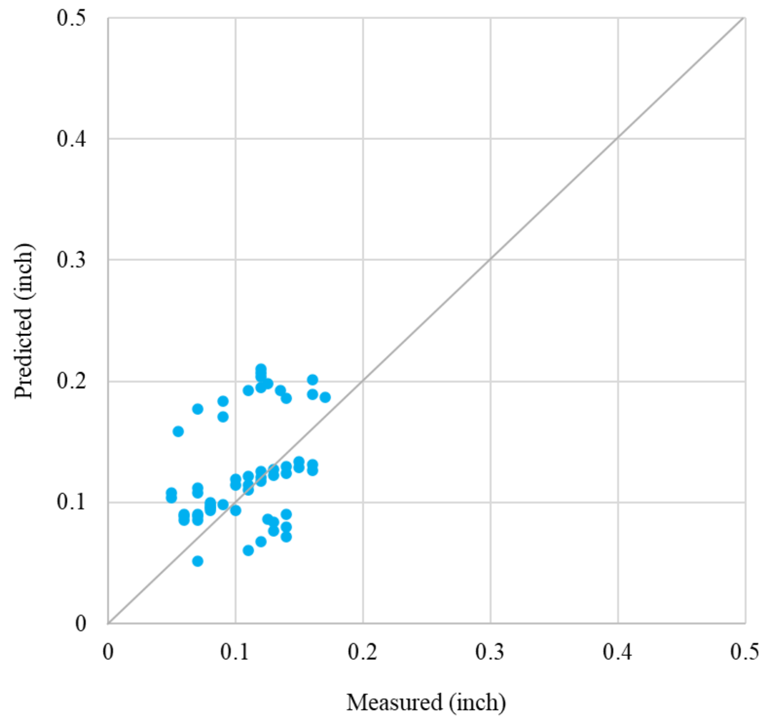

Figure 10. Measured Versus Predicted Total Rutting for Flexible Pavements from the Preliminary Validation Process

Table 9 shows the statistical adequacy ratings for the preliminary total rutting model verification, optimization, and validation of the final selected coefficients. Adjustment of the Bs1 and Br1 coefficients aided in a near elimination of the bias and a reduction in the SEE. However, the R^2 value remains poor.

Table 9. Total Rutting Model Coefficient Preliminary Optimization Statistics Ratings

Flexible Total Rutting Model Statistics Ratings	**Verification**		**Optimization**		**Validation**	
Test Statistic	Value	Rating	Value	Rating	Value	Rating
Bias	0.06	-----	0.00	-----	0.02	-----
SEE	0.11	Fair	0.07	Good	0.05	Very Good
R^2	0.11	Poor	0.14	Poor	0.12	Poor
Hypothesis testing of slope of the linear measured vs. predicted: True if slope = 1.	0.00	Reject	0.00	Reject	0.00232	Reject
Hypothesis testing of intercept of the linear measured vs. predicted: True if intercept = 0.	0.00	Reject	0.00	Reject	0.00013	Reject
Paired t-test between measured and predicted: True if difference = 0.	0.00	Reject	0.72894	Accept	0.00592	Reject

SEE = standard error of the estimate.

Semi-Rigid Pavement Calibration

ARA, Inc., conducted the preliminary semi-rigid pavement local calibration effort using the AASHTOWare Pavement CAT program. The selected semi-rigid pavement sections for local calibration and their corresponding measured total fatigue cracking, total transverse cracking, and total rutting data were uploaded to CAT to start the local calibration process. For each distress, a separate CAT project was created to perform the verification, calibration, and validation. Once the initial data verification was complete, the calibration coefficient optimization was conducted. The round of model calibration conducted using the CAT software served as the preliminary optimization of the model. After the recommended coefficients from this optimization were produced, further optimization was conducted to better refine the coefficients to meet pavement performance and design standards used by VDOT. This effort produced a final set of recommended coefficients suitable for implementation.

For semi-rigid pavements, two types of stabilized base materials were considered—CTA and cement-treated FDR. For the calibration process, both were treated as a single base type, cement-treated base (CTB).

Total Fatigue Cracking

The initial verification results of the total cracking model yielded a large overprediction of cracking with poor SEE (Figure 11). For the total fatigue cracking model, both reflective cracking from the CTB and bottom- up cracking originating within the asphalt pavement layers were considered. Due to the stiffness of the CTB, the model tends to predict minimal bottom up cracking and attributes most of the predicted distress to reflective cracking. Therefore, the preliminary calibration of the model focused on adjusting the CTB fatigue cracking model coefficients. The bottom up cracking coefficients remain the same as those used in the flexible pavement model.

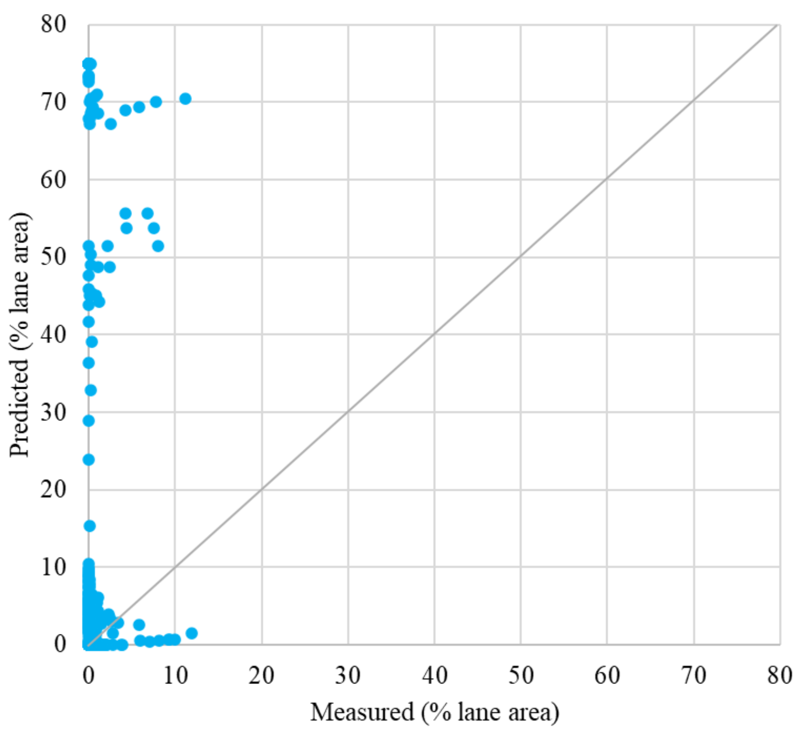

Figure 11. Initial Verification Results of the Total Fatigue Cracking Model for Semi-Rigid Pavements

Once initial verification was complete, the preliminary model coefficient optimization began. This process began with adjusting the CTB fatigue cracking model coefficients to help reduce the over amount of predicted CTB fatigue cracks. The first optimization attempt consisted of investigating the C2, C3, and C4 CTB coefficients. C2 directly affects the upper limit for predicted cracking, whereas C3 and C4 affect the initial increase rate in cracking. The second and third optimizations evaluated the C5 coefficient and revisited the C4 coefficient, adjusting both to reduce the bias and SEE. Ultimately, C5 did not affect the bias or SEE, and it was recommended to use the global coefficient. Figures 12 and 13 show the data from the optimization and the validation results for this process, respectively. Preliminary model optimization reduced the overall bias and SEE compared with the initial verification results. Table 10 shows the model coefficient preliminary optimization results.

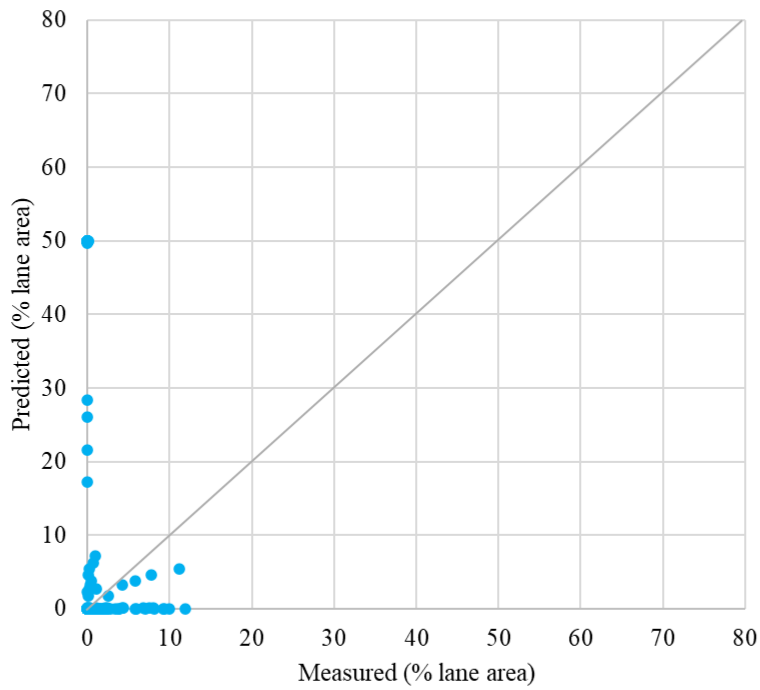

Figure 12. Measured Versus Predicted Total Fatigue Cracking for Semi-Rigid Pavements from the Preliminary Optimization Process

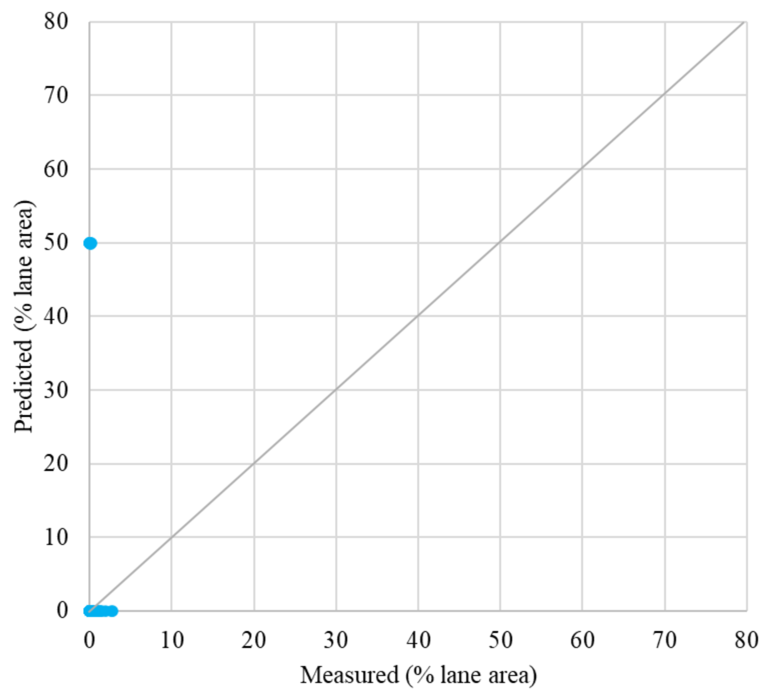

Figure 13. Measured Versus Predicted Total Fatigue Cracking for Semi-Rigid Pavements from the Preliminary Validation Process

Table 10. Total Fatigue Cracking Model Coefficient Preliminary Optimization Results

Calibration Iteration #	Adjusted Coefficient	Adjustment Range	Accepted Coefficient	Bias (% of area)	SEE (% of area)
Verification	N/A	N/A	N/A	16.36	32.16
1	CTB C2, C3, C4	C2: 50 C3: 12 C4: 4.2	C2: 50 C3: 12 C4: 4.2	4.184	15.198
2	C5	-600 – -150	-	-	-
3	C4	20–100	-	-	-

CTB = cement-treated base; N/A = not applicable; SEE = standard error of the estimate.

Table 11 shows the statistical adequacy ratings for the preliminary total fatigue cracking model verification, calibration, and validation of the final selected coefficients. The preliminary calibration results show improved values for calibration and validation versus the verification for both the bias and SEE, with a very poor R^2 value.

Table 11. Total Fatigue Cracking Model Coefficient Preliminary Optimization Statistics Ratings

Semi-Rigid Total Fatigue Cracking Model Statistics Ratings	**Verification**		**Optimization**		**Validation**	
Test Statistic	**Value**	**Rating**	**Value**	**Rating**	**Value**	**Rating**
Bias	16.36	-----	4.18	-----	8.76	-----
SEE	32.16	Poor	15.20	Poor	21.62	Poor
R^2	0	Very Poor	0.01	Very Poor	0.05	Very Poor
Hypothesis testing of slope of the linear measured vs. predicted: True if slope = 1.	0.98	Reject	0.00	Reject	0.03	Reject
Hypothesis testing of intercept of the linear measured vs. predicted: True if intercept = 0.	0.00	Reject	0.00	Reject	0.00	Reject

Semi-Rigid Total Fatigue Cracking Model Statistics Ratings	Verification		Optimization		Validation	
Paired t-test between measured and predicted: True if difference = 0.	0.00	Reject	0.00	Reject	0.00	Reject

SEE = standard error of the estimate.

Total Transverse Cracking

The total transverse cracking distress prediction is a function of both the transverse cracking propagating from the CTB and any thermal transverse cracking originating from the asphalt pavement layers. The transverse cracks propagating from the CTB are later defined by the crack spacing selected for each pavement section, which is then converted to a total length of transverse cracks per mile. All transverse cracks are assumed to have the potential to reflect through to the surface. To match the measured amount of transverse cracking at the pavement surface with the predicted amount, adjustments to the cracking spacing were made. The default value of 25-foot spacing was reduced to 15 feet for all semi-rigid sections, thus resulting in an increase in possible transverse cracks.

Notably, cracking is collected as a surface distress characteristic, which in turn makes it difficult to quantify whether a transverse crack is a reflective crack propagating from the CTB or from within the flexible asphalt layers. Initially, it is assumed that most of the measured transverse cracks for the semi-rigid pavement sections are reflective in nature. This assumption was based on a review of the measured data, pavement structure, and the inputs used to characterize the CTB layers. The optimization process consists of adjusting the transverse reflective cracking model coefficients first and then adjusting the thermal transverse cracking model calibration coefficients. Initial verification results of the total transverse cracking model showed a large underprediction of cracking with poor SEE (Figure 14).

After initial verification, the semi-rigid calibration sections were evaluated by mean annual air temperature (MAAT). Evaluating the sections in this way provides insight into what calibration coefficients to alter in the preliminary optimization process. MAAT is used to determine the magnitude of the global k-value coefficients within the thermal transverse cracking model. Based on the analysis, the sections with MAAT less than 57°F resulted in a significant underprediction of total transverse cracking versus the sections with MAAT greater than 57°F. Adjusting the thermal cracking model coefficients for the sections with MAAT greater than or equal to 57°F could help to increase the predicted cracking to better match the results with the measured distresses.

The preliminary model optimization consisted of three optimization runs to reduce the bias and SEE. The first step to optimize the model coefficients was to adjust the calibration coefficients that directly affect the rate at which transverse cracks from the CTB layers reflect through the asphalt layers above them. The second step then consisted of adjusting the thermal cracking model coefficients for the sections where MAAT is less than 57°F to help eliminate bias and reduce SEE further.

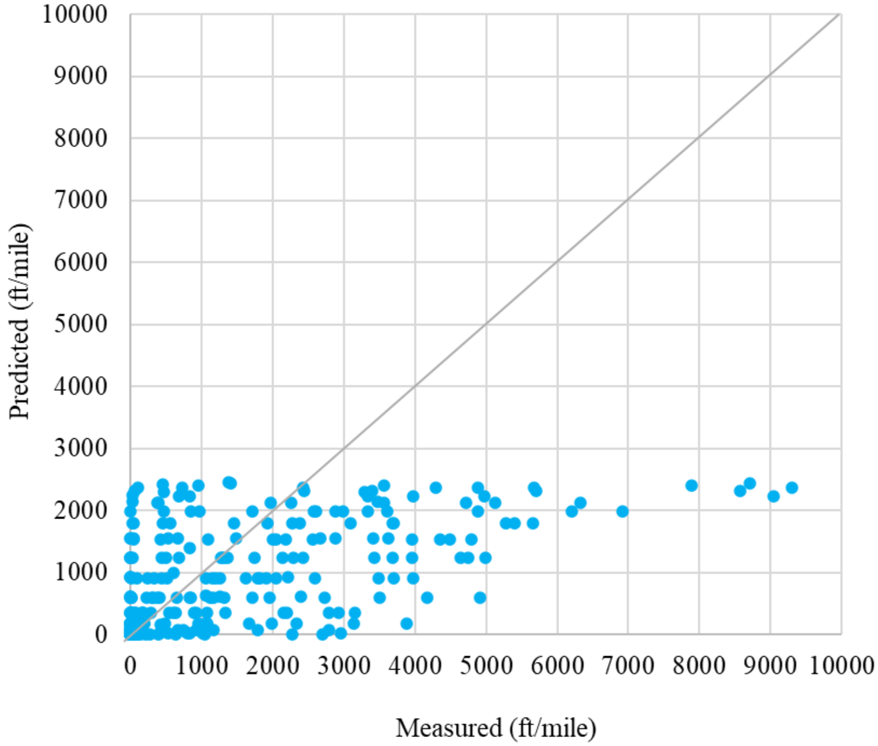

Figure 14. Initial Verification of the Total Transverse Cracking Model for Semi-Rigid Pavements

For the first optimization run, the C4 transverse reflection cracking coefficient was adjusted to determine if the transverse cracks in the stabilized layer need to reflect through the asphalt concrete layers faster or slower than the global model to best match the measured distresses in the selected calibration segments. A C4 value of 50 was determined to significantly reduced the bias between measured and predicted total transverse cracking. The second optimization run evaluated the C5 coefficient, which was observed to have little effect on the bias and SEE, and it was recommended to be left at the global coefficient value. The third optimization run investigated the thermal cracking coefficients for the sections with MAAT less than 57°F and resulted in a recommended constant k-value. This value, along with the results of the previous optimization trials, can be found in Table 12. Figures 15 and 16 show the data from the optimization and the validation results for this process, respectively.

Table 12. Total Transverse Cracking Model Coefficient Preliminary Optimization Results

Calibration Iteration #	Adjusted Coefficient	Adjustment Range	Accepted Coefficient	Bias (% of area)	SEE (% of area)
Verification	N/A	N/A	N/A		
1	C4	50–400	50	-164.98	1337.71
2	C5	-15 – -5.1048	-5.1048	-164.98	1337.71
3	AC Thermal Cracking Level 3K MAAT ≤ 57°F	5–25	11.67	-94.34	1387.66

AC = asphalt concrete; MAAT = mean annual air temperature; N/A = not applicable; SEE = standard error of the estimate.

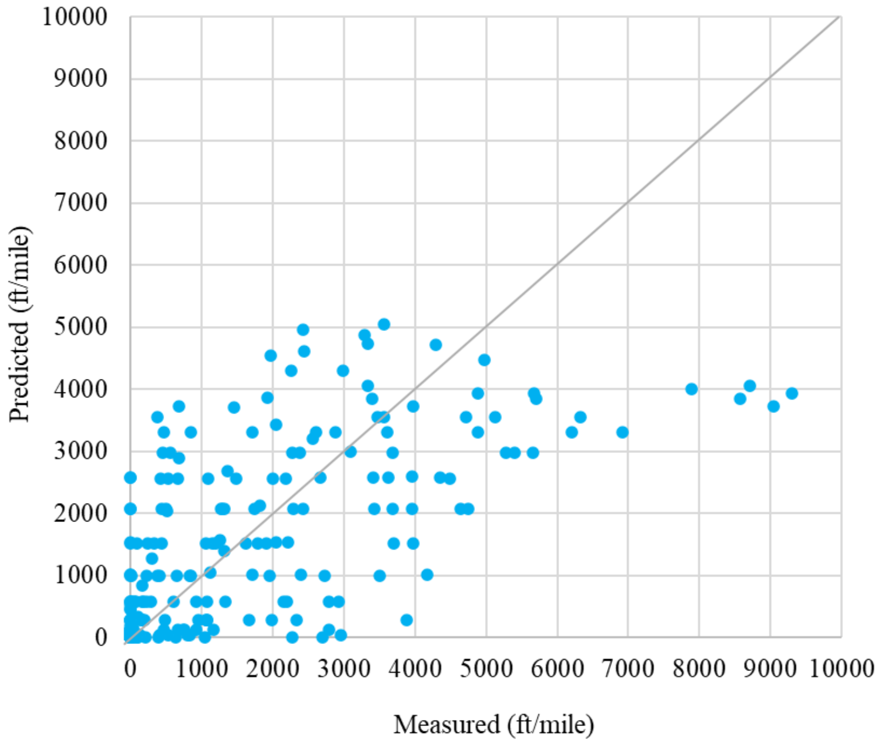

Figure 15. Measured Versus Predicted Total Transverse Cracking for Semi-Rigid Pavements from the Preliminary Optimization Process

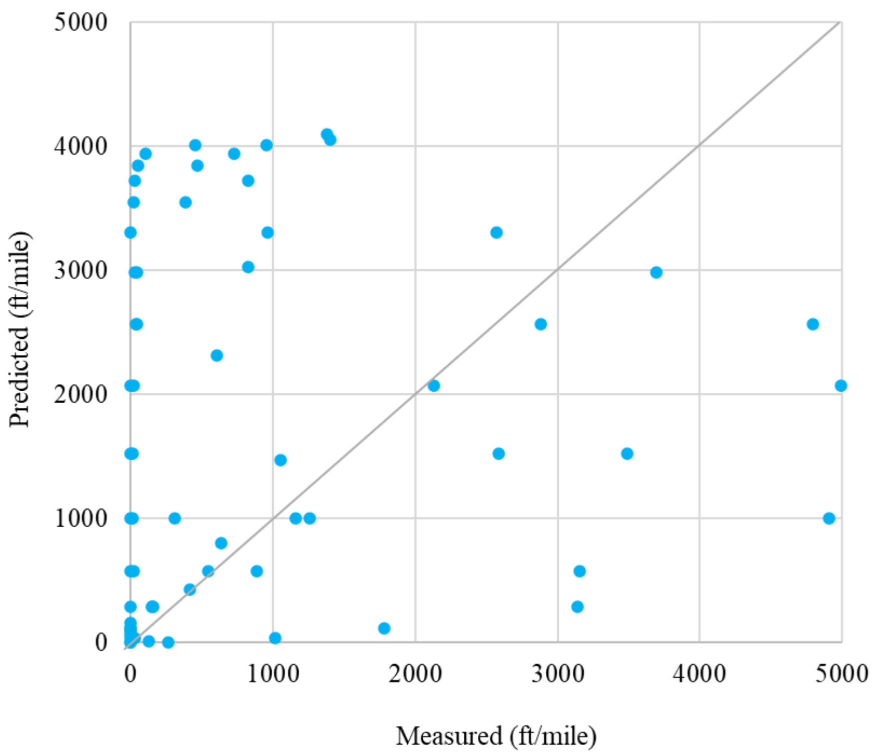

Figure 16. Measured Versus Predicted Total Transverse Cracking for Semi-Rigid Pavements from the Preliminary Validation Process

Table 13 shows the statistical adequacy ratings for the preliminary total transverse cracking model verification, optimization, and validation of the final selected coefficients. The preliminary calibration results show improved values for calibration and validation versus the verification for both the bias and SEE, with a poor R^2 value.

Table 13. Total Transverse Cracking Model Coefficient Preliminary Optimization Statistics Ratings

Semi-Rigid Total Transverse Cracking Model Statistics Ratings	Verification		Optimization		Validation	
Test Statistic	**Value**	**Rating**	**Value**	**Rating**	**Value**	**Rating**
Bias	-505.56	-----	-94.34	-----	800.83	-----
SEE	1,569.55	Poor	1,387.66	Poor	2,033.40	Poor
R^2	0.35	Poor	0.48	Poor	0.01	Very Poor
Hypothesis testing of slope of the linear measured vs. predicted: True if slope = 1.	0.00	Reject	0.00	Reject	0.00	Reject
Hypothesis testing of intercept of the linear measured vs. predicted: True if intercept = 0.	0.00	Reject	0.00	Reject	0.00	Reject
Paired t-test between measured and predicted: True if difference = 0.	0.00	Reject	0.276	Accept	0.001	Reject

SEE = standard error of the estimate.

Total Rutting

Figure 17 shows the rutting model initial verification results. For this model, the rutting coefficients determined in the flexible pavement models were used as a baseline versus the global coefficients. The rutting model did not result in a large bias or SEE. Additional comparisons were run to determine if the type of CTB (CTA versus FDR) resulted in differing rutting results, and no significant differences were identified.

The rutting model coefficient from the flexible pavement preliminary optimization provided both low bias and low SEE. However, in an effort to reduce the bias and SEE even further, various coefficient optimizations were attempted. First, the Br3 coefficient (traffic exponent) was decreased. In turn, this adjustment increased the amount of predicted rutting, reduced the bias, but increased SEE. A second optimization assessed the Br1 coefficient, which was decreased, resulting in a reduction in the total rutting, a reduced bias, but an increased SEE. The third and fourth optimization runs adjusted the Br2 (temperature exponent) coefficient. The model sensitivity to this coefficient was assessed, and it was noted that small changes in this coefficient can significantly affect the predicted total rutting, with lower Br2 values reducing the total rutting and high values increasing it. However, this optimization did not significantly improve the bias and SEE. Ultimately, the adjustments made within the previous four optimizations did not significantly improve the bias and SEE. In most cases, as bias approached zero, SEE increased and vice versa. Therefore, the preliminary calibration coefficients were recommended to remain the same as identified in the flexible pavement rutting model. Figures 18 and 19 show the data from the optimization and the validation results for this process, respectively. Table 14 shows the model coefficient preliminary optimization results.

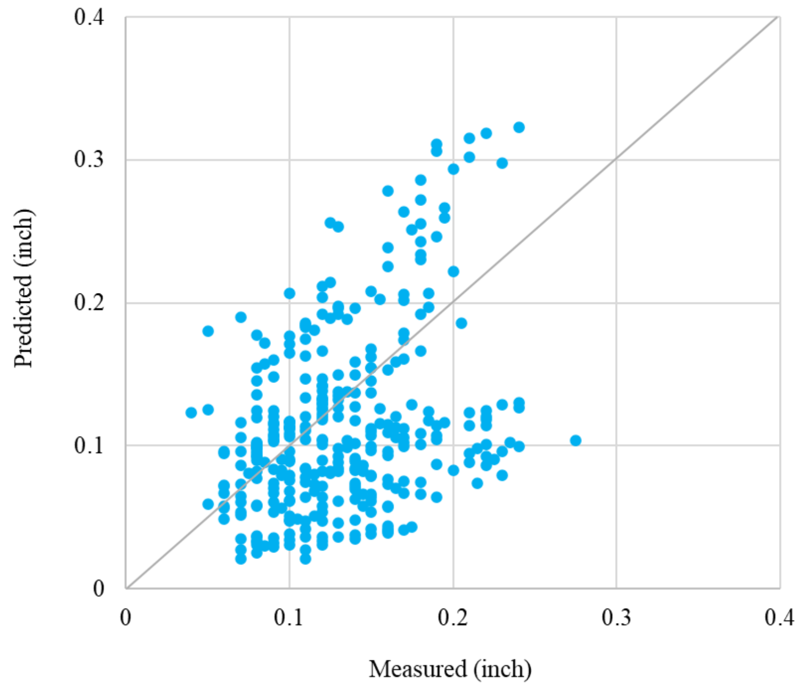

Figure 17. Initial Verification of the Total Rutting Model for Semi-Rigid Pavements

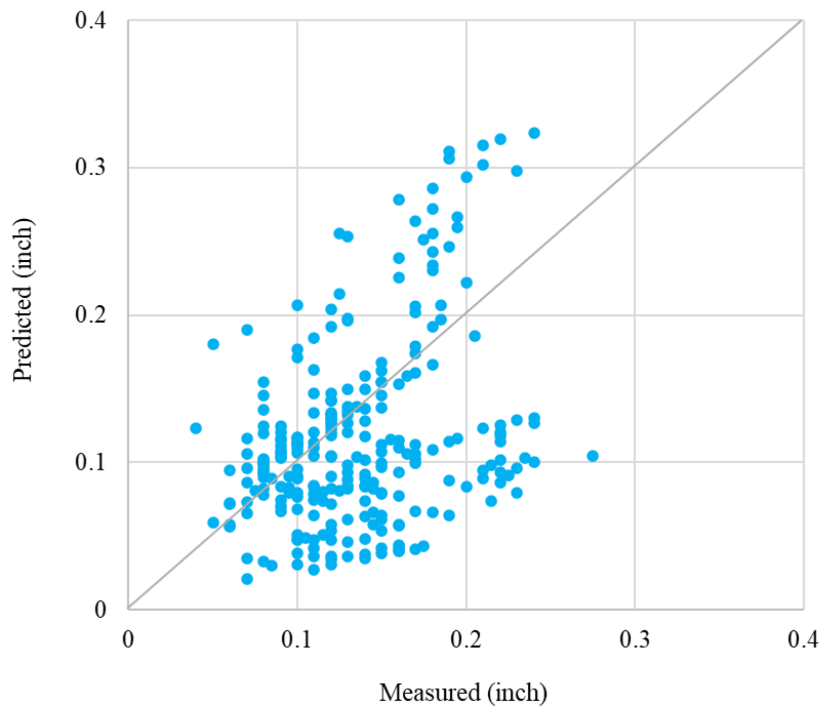

Figure 18. Measured Versus Predicted Total Rutting for Semi-Rigid Pavements from the Preliminary Optimization Process

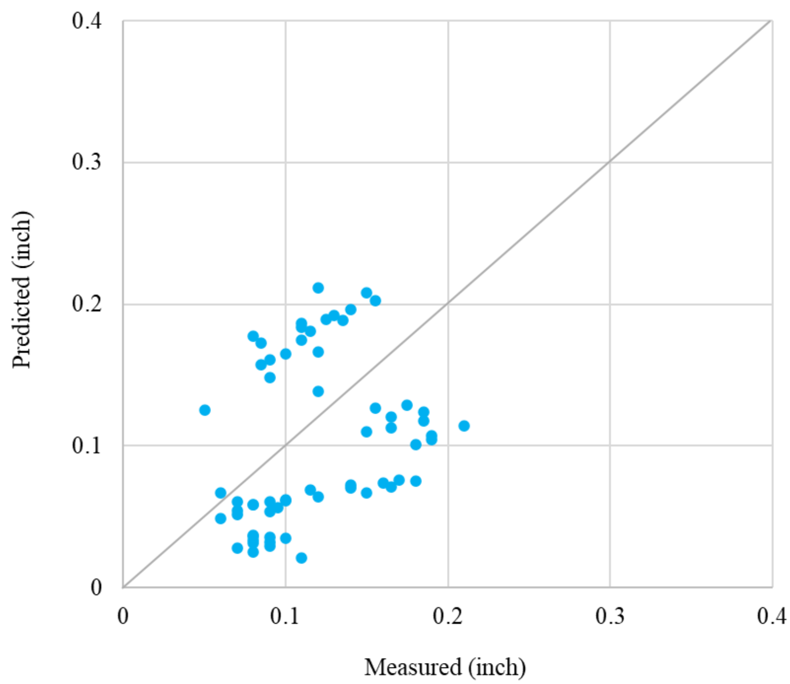

Figure 19. Measured Versus Predicted Total Rutting for Semi-Rigid Pavements from the Preliminary Validation Process

Table 14. Total Rutting Model Coefficient Preliminary Optimization Results

Calibration Iteration #	Adjusted Coefficient	Adjustment Range	Accepted Coefficient	Bias (in)	SEE (in)
Verification	N/A	N/A	N/A	-0.017	0.066
1	Br3	1.25–1.45	1.45	0.0065	0.081
2	Br1	0.05–0.30	0.3	-0.0091	0.072
3	Br2	0.42–0.62	-	-	-
4	Br2	0.52–0.58	-	-	-

N/A = not applicable; SEE = standard error of the estimate.

Statistical adequacy ratings for the preliminary total rutting model verification, optimization, and validation of the final selected coefficients can be found in Table 15 in the total rutting calibration for the semi-rigid pavement model.

Table 15. Semi-Rigid Total Rutting Model Coefficient Preliminary Optimization Statistics Ratings

Semi-Rigid Total Rutting Model Statistics Ratings	Verification		Optimization		Validation	
Test Statistic	Value	Rating	Value	Rating	Value	Rating
Bias	-0.02	-----	-0.02	-----	-0.02	-----
SEE	0.07	Very Good	0.07	Very Good	0.06	Very Good
R2	0.13	Very Poor	0.13	Very Poor	0.09	Very Poor
Hypothesis testing of slope of the linear measured vs. predicted: True if slope = 1.	0.00	Reject	0.00	Reject	0.00	Reject
Hypothesis testing of intercept of the linear measured vs. predicted: True if intercept = 0.	0.00	Reject	0.00	Reject	0.02	Reject
Paired t-test between measured and predicted: True if difference = 0.	0.00	Reject	0.00	Reject	0.05	Accept

SEE = standard error of the estimate.

Continuously Reinforced Concrete Pavement Calibration

ARA, Inc., conducted the preliminary CRCP pavement local calibration effort using the AASHTOWare Pavement CAT program. The previously selected CRCP pavement sections for local calibration and their corresponding measured punchout data were uploaded to CAT to start the local calibration process. For each distress, a separate CAT project was created to perform the verification, calibration, and validation. Once the initial data verification was complete, calibration coefficient optimization was conducted. The round of model calibration conducted using the CAT software served as the preliminary optimization of the model. After the recommended coefficients from this optimization were produced, further optimization was conducted to better refine the coefficients to meet pavement performance and design standards used by VDOT. This effort produced a final set of recommended coefficients suitable for implementation.

The VDOT CRCP local calibration dataset met the minimum number of sections required to automatically perform split sampling. The dataset was randomly split such that 80% of the sections were used for optimizing the calibration coefficients, whereas the remaining 20% were used for validation.

Punchouts

Initial verification of the punchout model results in consistent overprediction and high SEE. Because of the high bias and SEE, individual section results were investigated to determine any underlying causes. CRCP sections that contained an unbound granular base showed distinct differences in overprediction of punchout versus sections without an unbound granular base. It is suspected that the significant overprediction of punchouts in CRCP sections containing a granular base can be attributed to high cracking spacing predictions for the associated distress model. These high cracking spacing predictions do not reflect the performance seen from CRCP sections with an unbound base within the VDOT roadway network. With this result in mind, the predicted crack spacing was turned off for the sections containing a granular base, and a constant 48-inch cracking spacing was set. Once the cracking spacing for CRCP sections with an unbound base was set, a second initial verification was performed. This second verification showed a significant improvement in the overprediction of the punchout model. In addition to the cracking spacing change, some measured data points deemed as outliers were removed. Figure 20 shows the revised data verification after the previously mentioned adjustments.

The CRCP punchout model uses an indirect translation process that consists of a damage accumulation function, using key material properties and pavement structural inputs and a transfer function, which converts damage to a measure of physical distress observed at the pavement surface. The optimization of the CRCP punchout model calibration coefficients began with the transfer function coefficients. Some coefficients are prioritized to reduce bias, whereas others may have a greater influence on improving SEE. Adjustments within the transfer function coefficients were prioritized first.

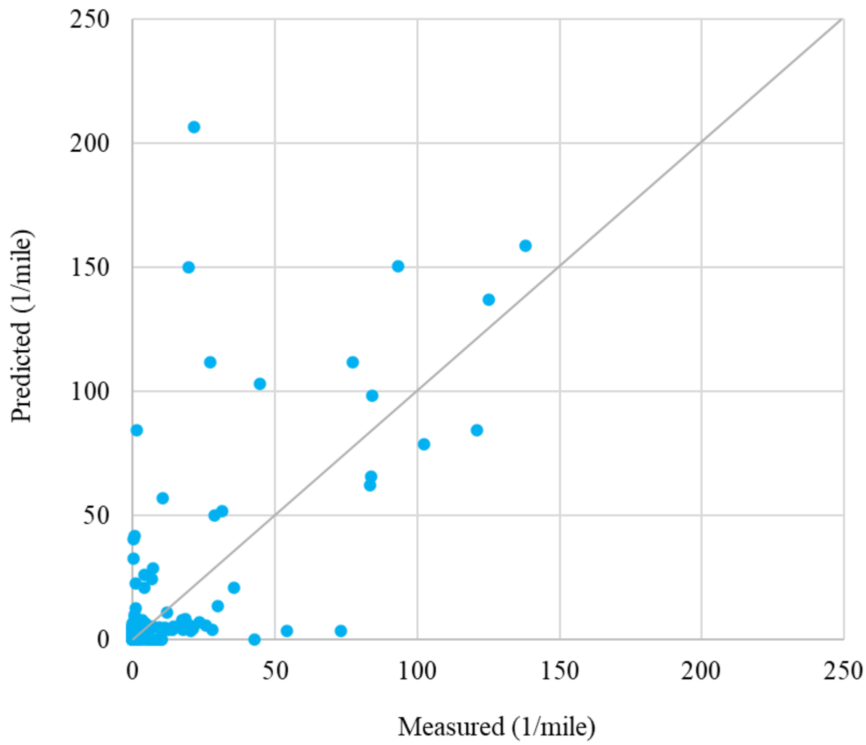

Figure 20. Revised Verification Results of the Punchout Model for Continuously Reinforced Concrete Pavements

The preliminary optimization process consisted of adjusting the C4 coefficient. The first iteration evaluated the coefficient over a wide range of potential values to determine the bias and SEE sensitivity and whether the values would increase or decrease when compared with the global model coefficient. It was concluded that higher values tended to improve the bias and SEE values; therefore, a second set of values was tested to further optimize the coefficient. A final coefficient of 70 was selected as the preliminary optimum value (Table 16). Figures 21 and 22 show the data from the optimization and the validation results for this process, respectively.

Table 16. CRCP Punchout Model Coefficient Preliminary Optimization Results

Calibration Iteration #	Adjusted Coefficient	Adjustment Range	Accepted Coefficient	Bias (% slabs cracked)	SEE (% slabs cracked)
Verification	N/A	N/A	N/A	3.58	27.44
1	C4	5–60	-	1.70	25.61
2	C4	60–80	70	0.28	22.81

CRCP = continuously reinforced concrete pavement; N/A = not applicable; SEE = standard error of the estimate.

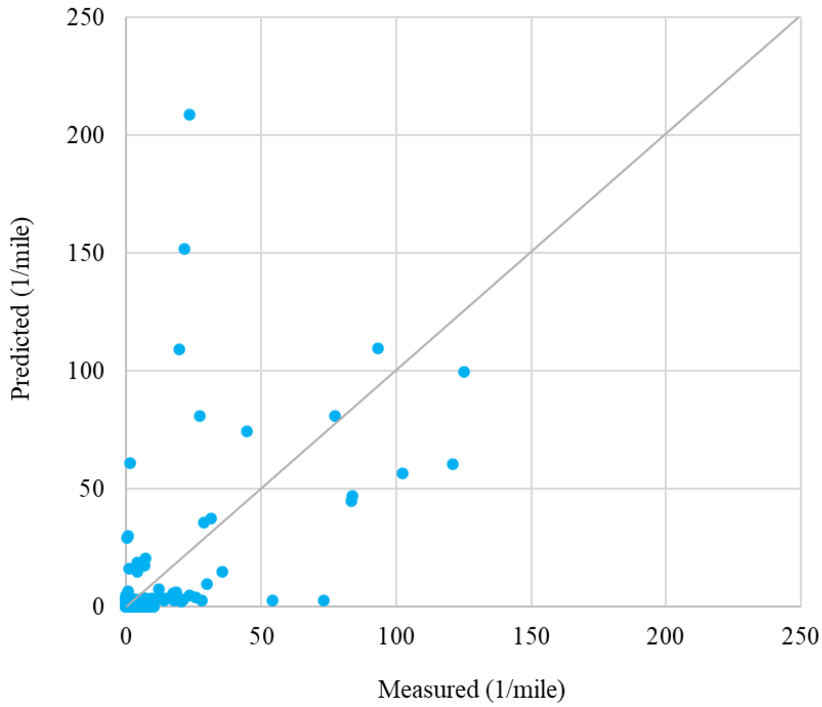

Figure 21. Measured Versus Predicted Punchouts for Continuously Reinforced Concrete Pavements from the Preliminary Optimization Process

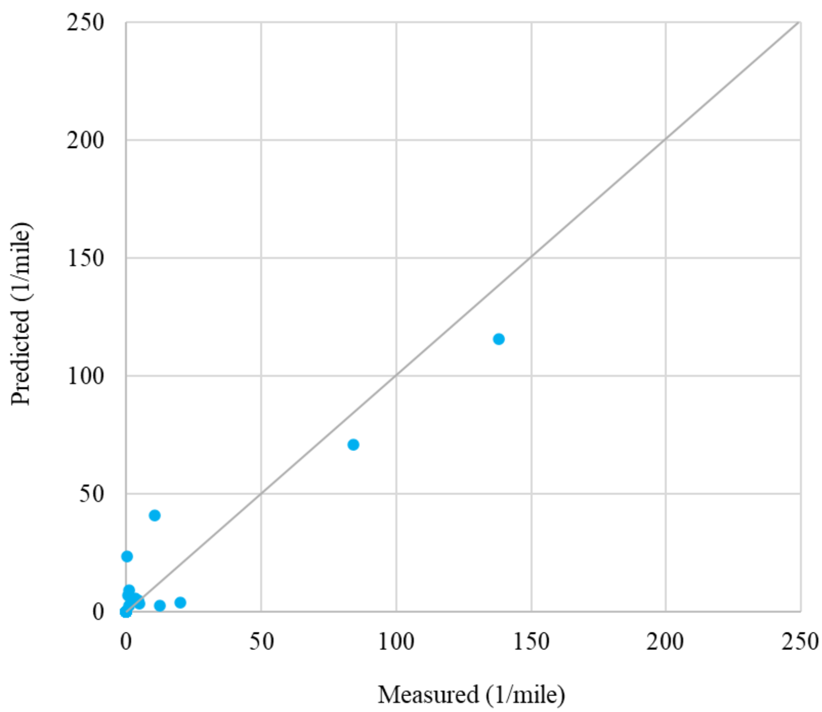

Figure 22. Measured Versus Predicted Punchouts for Continuously Reinforced Concrete Pavements from the Preliminary Validation Process

Table 17 shows the statistical adequacy ratings for the preliminary punchout model verification, optimization, and validation of the final selected coefficients. The preliminary calibration results showed a reduction in bias to nearly zero, with a poor SEE and R^2 value.

Table 17. Punchout Model Coefficient Preliminary Optimization Statistics Ratings

CRCP Punchout Model Statistics Ratings	Verification		Optimization		Validation	
Test Statistic	**Value**	**Rating**	**Value**	**Rating**	**Value**	**Rating**
Bias	3.58	-----	0.28	-----	0.31	-----
SEE	27.44	Poor	22.81	Poor	8.04	Good
R^2	0.40	Poor	0.32	Poor	0.9	Good
Hypothesis testing of slope of the linear measured vs. predicted: True if slope = 1.	0.76	Accept	0.00	Reject	0.00	Reject
Hypothesis testing of intercept of the linear measured vs. predicted: True if intercept = 0.	0.00	Reject	0.00	Reject	0.00	Reject
Paired t-test between measured and predicted: True if difference = 0.	0.05	Accept	0.87	Accept	0.80	Accept

CRCP = continuously reinforced concrete pavement; SEE = standard error of the estimate.

Refinement of Preliminary Local Calibration Coefficients

Local Calibration Coefficient Refinement: Flexible Pavements

The refinement of the local calibration coefficients for flexible pavement focused on the total rutting and bottom up cracking models because they are the distress models being used under VDOT's current design practice. Outcomes from the 'Initial Verification' and the input data were re-reviewed first to determine the areas for potential adjustment prior to performing the optimization in CAT. Based on this re-review, six pavement sections (St-1, St -2, Ri-11, Ri-12, Cu-1, and Cu-2) were identified as outliers. Among these sections, the temporal range of measured distress data was adjusted for two pavement sections (Ri-11 and Ri-12) because of the presence of highly inconsistent data during select years. After the adjustment, the distress data of those sections were updated in the CAT interface. The remaining four pavement sections (St-1, St-2, Cu-1, and Cu-2) were excluded from the re-optimization process because some of these pavement sections consist of asphalt layers directly over a subgrade material that does not align with VDOT's current practice and VDOT's current Pavement ME Design guidance. The sections also consisted of highly inconsistent distress trends with limited years of data.

After exclusion of the previously mentioned pavement sections and distress data adjustments for selected sections, 32 pavement sections were selected for the re-optimization of the select coefficients in CAT. These coefficients are the bottom up cracking C1 coefficient and the total rutting coefficient $\beta r1$ for asphalt rutting and $\beta s1$ for subgrade rutting. During the re-optimization process, several additional considerations were incorporated beyond reducing bias and SEE. These considerations included ensuring that the models appropriately reflected the influence of pavement foundation properties on rutting and maintaining consistency in the coefficients used in the rutting transfer functions between flexible and semi-rigid pavement design models.

Prior to the optimization process, CAT selected 26 pavement sections for the optimization step and 6 pavement sections for the validation step. The optimization step iterated the C1 coefficient associated with the bottom up cracking model, and from this iteration process, a C1 value of 0.5 was obtained with low bias and SEE. Figures 23 and 24 show the comparison between the predicted versus measured distress data obtained from the optimization and validation steps, respectively. Table 18 summarizes the statistical evaluation associated with the C1 coefficient of 0.5.

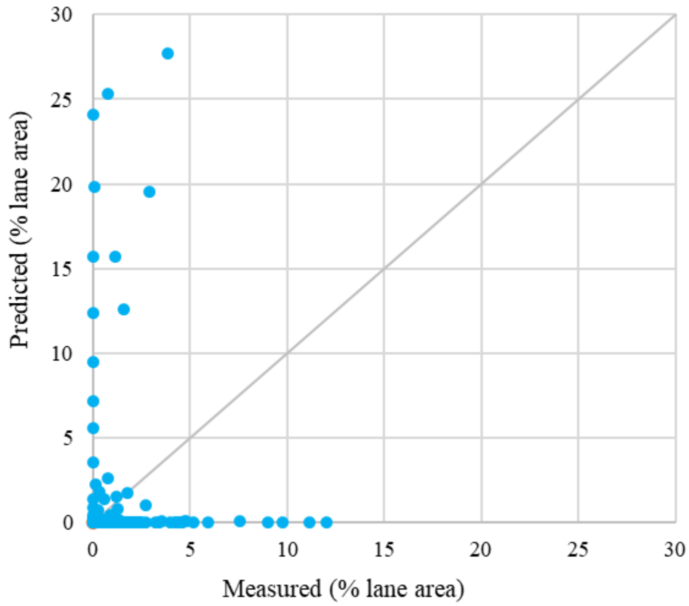

Figure 23. Measured Versus Predicted Bottom Up Cracking for Flexible Pavements from Refinement

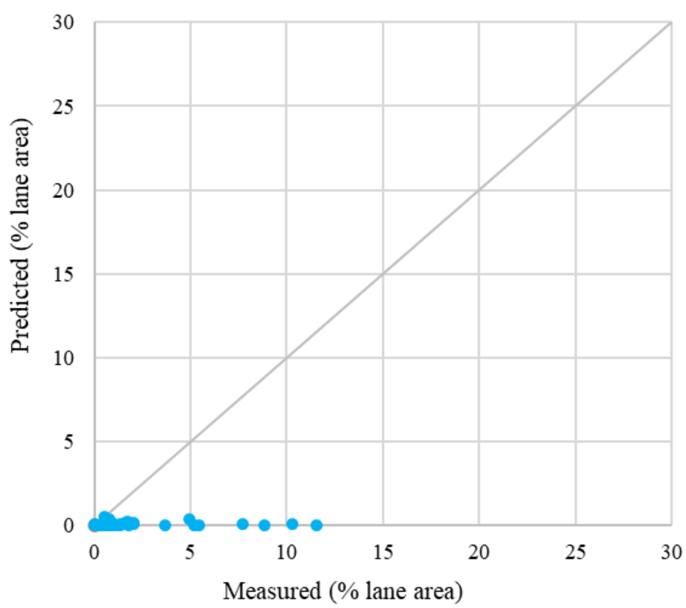

Figure 24. Measured Versus Predicted Bottom Up Cracking for Flexible Pavements from Refinement Validation

Table 18. Bottom Up Cracking Model Coefficient Preliminary Optimization Statistics Ratings

Flexible Bottom Up Cracking Model Statistics Ratings	Optimization		Validation	
Test Statistic	**Value**	**Rating**	**Value**	**Rating**
Bias	0.01	-----	-1.45	-----
SEE	4.17	Good	2.94	Good
R^2	0.064	Poor	0.106	Poor
Hypothesis testing of slope of the linear measured vs. predicted: True if slope = 1.	0.00	Reject	0.00	Reject
Hypothesis testing of intercept of the linear measured vs. predicted: True if intercept = 0.	0.00	Reject	0.00	Reject
Paired t-test between measured and predicted: True if difference = 0.	0.972	Accept	0.00	Reject

SEE = standard error of the estimate.

As previously mentioned, the βr1 coefficient for asphalt rutting and βs1 for subgrade rutting models were also evaluated. From this optimization step, a βr1 value of 0.55 and a βs1 value of 0.8 were obtained for the asphalt and subgrade rutting models, respectively, and helped to produce a model with low bias and SEE. Figures 25 and 26 show the comparison between the predicted versus measured distress data obtained from the optimization and validation steps, respectively. Table 19 summarizes the statistical evaluation associated with the βr1 of 0.55 and βs1 of 0.8.

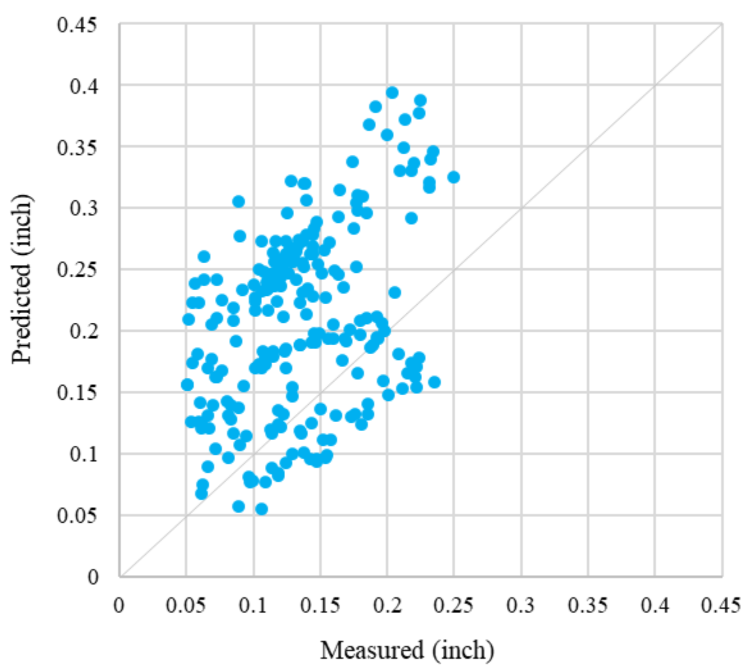

Figure 25. Measured Versus Predicted Total Rutting for Flexible Pavements from Refinement

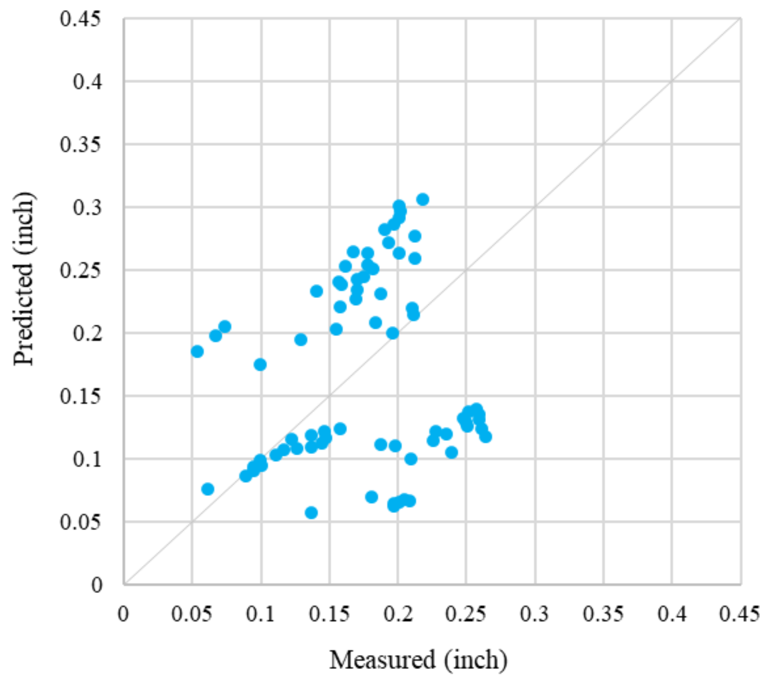

Figure 26. Measured Versus Predicted Total Rutting for Flexible Pavements from Refinement Validation

Table 19. Total Rutting Model Coefficient Preliminary Optimization Statistics Ratings

Flexible Total Rutting Model Statistics Ratings	**Optimization**		**Validation**	
Test Statistic	Value	Rating	Value	Rating
Bias	0.07	-----	-0.01	-----
SEE	0.1	Fair	0.09	Good
R^2	0.16	Poor	0.01	Poor
Hypothesis testing of slope of the linear measured vs. predicted: True if slope = 1.	0.00	Reject	0.00	Reject
Hypothesis testing of intercept of the linear measured vs. predicted: True if intercept = 0.	0.00	Reject	0.00	Reject
Paired t-test between measured and predicted: True if difference = 0.	0.00	Reject	0.53	Accept

SEE = standard error of the estimate.

Tables 20 and 21 summarize the coefficients from global, preliminary optimization calibration, and final optimization calibration values. Refinement was not performed for top down cracking or International Roughness Index (IRI) because the primary focus was on bottom up cracking and total rutting.

Table 20. Bottom Up Cracking Model Optimization Results

Flexible Bottom Up Cracking Model Coefficient	**PMED 3.0 Global Value**	**Preliminary Optimization Calibration Value**	**Final Optimization Calibration Value**
Bf1: < 5 in	0.02054	0.02054	0.02054
Bf1: 5 in ≤ Hac ≤ 12 in	(5.014 * Pow(hac, -3.416)) * 1 + 0	(5.014 * Pow(hac, -3.416)) * 1 + 0	(5.014 * Pow(hac, -3.416)) * 1 + 0
Bf1: > 12 in	0.001032	0.001032	0.001032
Bf2	1.38	1.38	1.38
Bf3	0.88	0.88	0.88

Flexible Bottom Up Cracking Model Coefficient	PMED 3.0 Global Value	Preliminary Optimization Calibration Value	Final Optimization Calibration Value
C1	1.31	0.5857	0.5
C2: < 5 in	2.1585	2.1585	2.1585
C2: 5 in ≤ h_{AC} ≤ 12 in	(0.867 + 0.2583 * hac) * 1 + 0	(0.867 + 0.2583 * hac) * 1 + 0	(0.867 + 0.2583 * hac) * 1 + 0
C2: > 12 in	3.9666	3.9666	3.9666

PMED = AASHTOWare Pavement ME Design.

Table 21. Total Rutting Model Optimization Results

Flexible Total Rutting Model Coefficient	PMED 3.0 Global Value	Preliminary Optimization Calibration Value	Final Optimization Calibration Value
Br1	0.4	0.35	0.55
Br2	0.52	0.52	0.52
Br3	1.36	1.36	1.36
Bs1: Granular Base	1	1	1
Bs1: Subgrade	1	0.5	0.8

PMED = AASHTOWare Pavement ME Design.

Local Calibration Coefficient Refinement: Semi-Rigid Pavements

The refinement of the local calibration coefficients for semi-rigid pavement focused on the total rutting and the total fatigue cracking model to be consistent with VDOT's current design approach for pavements with CTB. Like flexible pavement, outcomes from the initial verification and the input data were also re-reviewed for the semi-rigid model to determine any areas for potential adjustment prior to performing the optimization in CAT. Based on this re-review, four pavement sections (Sa-5-SR, Sa-6-SR, Cu-1-SR, and Ri-1-SR) were identified as outliers. These sections contained a thin asphalt layer (i.e., one 2-inch-thick lift of asphalt on stabilized base) that does not align with VDOT's current Pavement ME Design guidance or exhibited unusual distress predictions, likely because of a modeling issue. After exclusion of the previous pavement sections, 31 pavement sections were ultimately used for the re-optimization of select coefficients in CAT. The selected coefficients for re-optimization were C2, C3, and C4 associated with the CTB reflective fatigue cracking model, within the total fatigue cracking model, and the total rutting coefficient $\beta r1$ for asphalt rutting and $\beta s1$ for subgrade rutting.

Prior to the optimization process, CAT selected 25 pavement sections for the optimization step and 6 pavement sections for the validation step. The optimization step iterated the C4 coefficient associated with the reflective fatigue cracking model, and coefficients C2, C3, and C4 from the CTB cracking model were set to 45, 2, and 2, respectively. It should be noted that the C1 coefficient from the flexible pavement bottom up fatigue cracking model was used to ensure consistency between the flexible and semi-rigid pavement models. From this iteration process, a C4 value of 75 was obtained with low bias and SEE. Figures 27 and 28 show the comparison between the predicted versus measured distress data obtained from the optimization and validation steps, respectively. Table 22 summarizes the statistical evaluation associated with the C4 coefficient of 75 (under the Reflective Fatigue Cracking model) and CTB coefficients (C2, C3, and C4) of 45, 2, and 2.

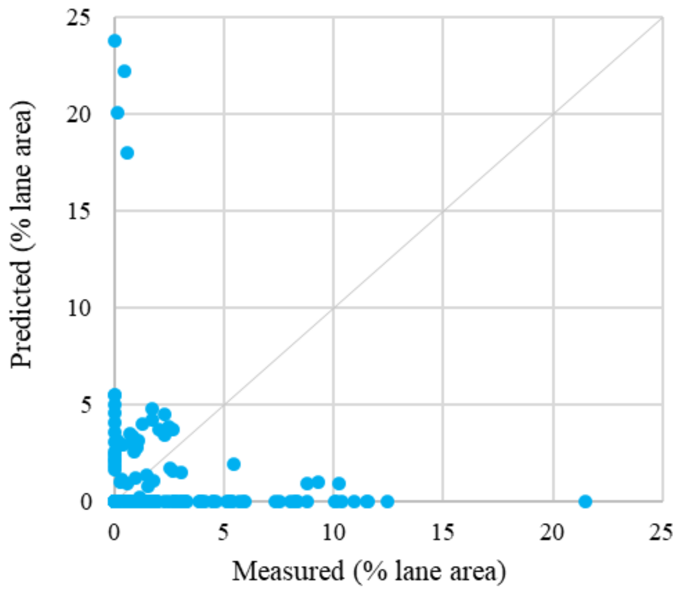

Figure 27. Measured Versus Predicted Total Fatigue Cracking for Semi-Rigid Pavements from Refinement

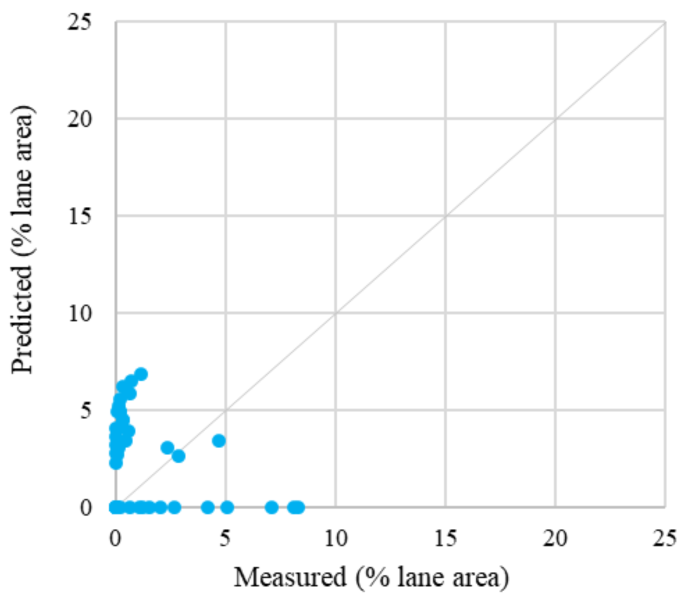

Figure 28. Measured Versus Predicted Total Fatigue Cracking for Semi-Rigid Pavements from Refinement Validation

Table 22. Total Fatigue Cracking Model Coefficient Preliminary Optimization Statistics Ratings

Flexible Bottom Up Cracking Model Statistics Ratings	Optimization		Validation	
Test Statistic	**Value**	**Rating**	**Value**	**Rating**
Bias	-0.11	-----	0.86	-----
SEE	6.10	Fair	3.60	Good
R^2	0.00	Poor	0.05	Poor
Hypothesis testing of slope of the linear measured vs. predicted: True if slope = 1.	0.00	Reject	0.00	Reject
Hypothesis testing of intercept of the linear measured vs. predicted: True if intercept = 0.	0.00	Reject	0.00	Reject
Paired t-test between measured and predicted: True if difference = 0.	0.77	Accept	0.08	Accept

SEE = standard error of the estimate.

Like the flexible pavement model, the same two coefficients (βr1 and βs1) were optimized for total rutting under the semi-rigid pavement model. From this optimization step, a βr1 value of 0.55 and a βs1 value of 0.8 were obtained for the asphalt rutting model and the subgrade model, respectively, with a low bias and SEE. Figures 29 and 30 show the comparison between the predicted versus measured distress data obtained from the optimization and validation steps, respectively. Table 23 summarizes the statistical evaluation associated with the βr1 coefficient of 0.55 and the βs1 coefficient of 0.8.

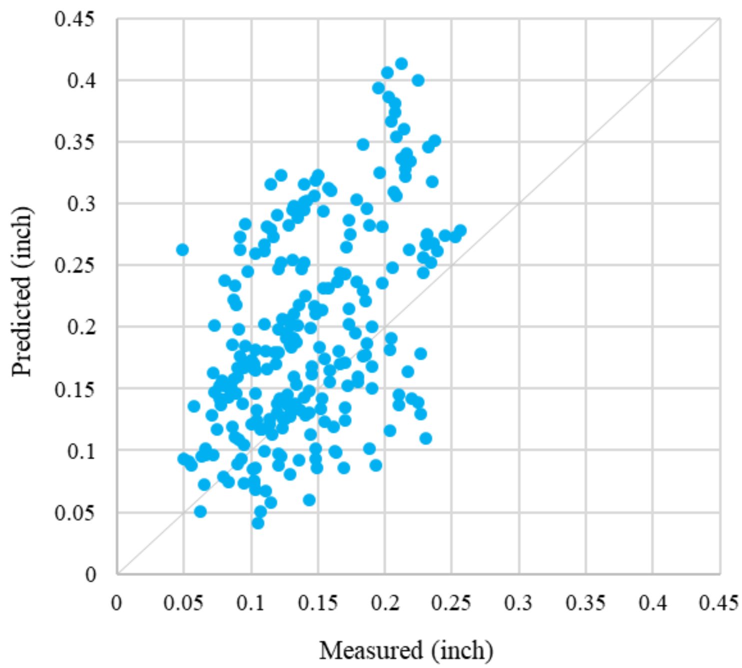

Figure 29. Measured Versus Predicted Total Rutting for Semi-Rigid Pavements from Refinement

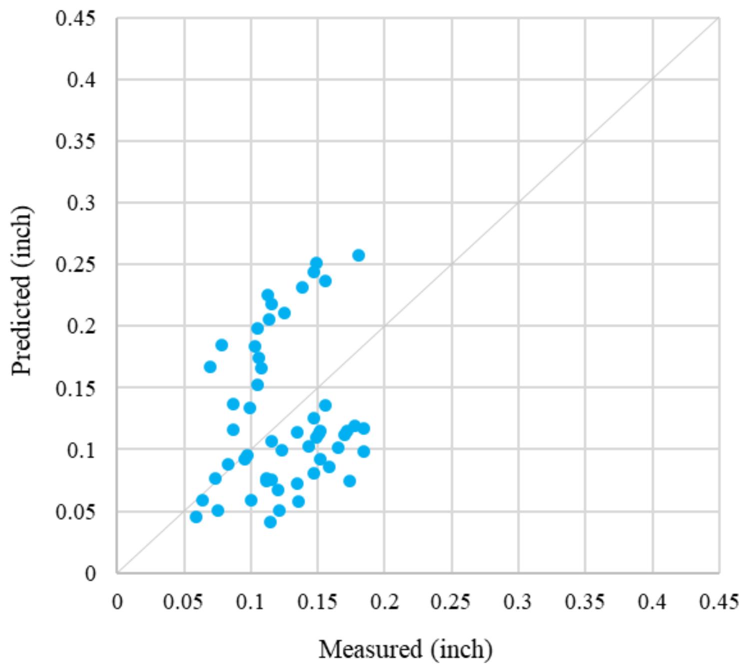

Figure 30. Measured Versus Predicted Total Rutting for Semi-Rigid Pavements from Refinement Validation

Table 23. Semi-Rigid Total Rutting Model Coefficient Preliminary Optimization Statistics Ratings

Semi-Rigid Total Rutting Model Statistics Ratings	**Optimization**		**Validation**	
Test Statistic	Value	Rating	Value	Rating
Bias	0.05	-----	0.001	-----
SEE	0.09	Good	0.06	Good
R2	0.25	Poor	0.02	Poor
Hypothesis testing of slope of the linear measured vs. predicted: True if slope = 1.	0.16	Accept	0.00	Reject
Hypothesis testing of intercept of the linear measured vs. predicted: True if intercept = 0.	0.00	Reject	0.00	Reject
Paired t-test between measured and predicted: True if difference = 0.	0.00	Reject	0.92	Accept

SEE = standard error of the estimate.

Tables 24 and 25 summarize the coefficients from global, preliminary optimization calibration, and final optimization calibration values. Refinement was not performed for the total transverse cracking model because the field data for model calibration are currently insufficient.

Table 24. CTB Cracking Model Optimization Results

Semi-Rigid CTB Cracking Model Coefficient	**PMED 3.0 Global Value**	**Preliminary Optimization Calibration Value**	**Final Optimization Calibration Value**
BC1	1	1	1
BC2	1	1	1
C1	0	0	0
C2	75	50	45
C3	2	12	2
C4	2	4.2	2

CTB = cement-treated base; PMED = AASHTOWare Pavement ME Design.

Table 25. Reflective Fatigue Cracking Model Optimization Results

Semi-Rigid Reflective Fatigue Cracking Model Coefficient	PMED 3.0 Global Value	Preliminary Optimization Calibration Value	Final Optimization Calibration Value
C1 – Bending	1.64	1.64	1.64
C2 – Shearing	1.1	1.1	1.1
C3 – Thermal	0.19	0.19	0.19
C4	62.1	62.1	75
C5	-404.6	-404.6	-404.6

PMED = AASHTOWare Pavement ME Design.

Tables 26 and 27 summarize the coefficients from global, preliminary optimization calibration, and final optimization calibration values for bottom up (fatigue) cracking and total rutting for the semi-rigid model. Notably, these values are identical to the values identified for flexible pavements.

Table 26. Bottom Up (Fatigue) Cracking Model Optimization Results

Semi-Rigid Bottom Up (Fatigue) Cracking Model Coefficient	PMED 3.0 Global Value	Preliminary Optimization Calibration Value	Final Optimization Calibration Value
Bf1: < 5 in	0.02054	0.02054	0.02054
Bf1: 5 in ≤ Hac ≤ 12 in	(5.014 * Pow(hac, -3.416)) * 1 + 0	(5.014 * Pow(hac, -3.416)) * 1 + 0	(5.014 * Pow(hac, -3.416)) * 1 + 0
Bf1: > 12 in	0.001032	0.001032	0.001032
Bf2	1.38	1.38	1.38
Bf3	0.88	0.88	0.88
C1	1.31	0.5857	0.5
C2: < 5 in	2.1585	2.1585	2.1585
C2: 5 in ≤ h_{AC} ≤ 12 in	(0.867 + 0.2583 * hac) * 1 + 0	(0.867 + 0.2583 * hac) * 1 + 0	(0.867 + 0.2583 * hac) * 1 + 0
C2: > 12 in	3.9666	3.9666	3.9666

PMED = AASHTOWare Pavement ME Design.

Table 27. Total Rutting Model Optimization Results

Semi-Rigid Total Rutting Model Coefficient	PMED 3.0 Global Value	Preliminary Optimization Calibration Value	Final Optimization Calibration Value
Br1	0.4	0.35	0.55
Br2	0.52	0.52	0.52
Br3	1.36	1.36	1.36
Bs1: Granular Base	1	1	1
Bs1: Subgrade	1	0.5	0.8

PMED = AASHTOWare Pavement ME Design.

Local Calibration Coefficient Refinement: Continuously Reinforced Concrete Pavements

Table 28 summarizes the coefficients from global and preliminary optimization calibration values. Refinement was not performed for the CRCP model coefficients, and the preliminary optimization coefficients were accepted as is. Although the SEE value was poor during the optimization, a notable decrease in the statistic was seen during the validation phase, resulting in a good SEE value. Bias was also considerably low for both optimization and validation. Based on the visual observation (Figures 21 and 22), most of the data points were distributed close to the line of equality, with a few exceptions. These data points, which represent significant overprediction in Figure 21, represent very few pavement sites, and therefore, minimal effect on pavement design is anticipated. For this reason, no further refinement was pursued for CRCP coefficients.

Table 28. CRCP Punchout Model Optimization Results

CRCP Punchout Model Coefficient	PMED 3.0 Global Value	Preliminary Optimization Calibration Value
C1	2.0	2.0
C2	1.22	1.22
C3	1760	1760
C4	49.7359	70
C5	-0.6009	-0.6009

CRCP = continuously reinforced concrete pavement; PMED = AASHTOWare Pavement ME Design.

Example Pavement Structure Design with the Pavement ME Web-Based Version

Example designs were developed using the Pavement ME Design web-based software, incorporating the refined coefficients and a set of hypothetical pavement design scenarios to illustrate representative pavement structures. Table 29 summarizes the route type, traffic level, project location, and subgrade inputs used in the simulations.

Table 29. Example Pavement Design Scenario Inputs

Scenario	Route Type	No. of Lanes (Directional)	Two-Way AADTT	AASHTO Soil Type	Resilient Modulus (psi)
A	Interstate	3	7,000	A-2-4	16,500
B	Interstate	3	16,000	A-7-6	14,000

AADTT = annual average daily truck traffic.

The example pavement designs shown for each scenario in Table 29 were developed using the inputs summarized in Table 30. When input values were missing, reasonable assumptions were applied. All designs follow the performance thresholds (i.e., distress limits at specific design years) outlined in VDOT's *Pavement ME User Manual* (VDOT, 2017). The layer thicknesses represent pavement structures appropriate for typical foundations (base layer and subgrade) and are not intended to reflect project-specific design details for any particular site.

Table 30. Example Pavement Design Thickness

Scenario	Flexible	Semi-Rigid	CRCP
A	2.0 in SM-12.5 2.0 in IM-19.0 8.5 in BM-25.0 6.0 in 21B	2.0 in SM-12.5 2.0 in IM-19.0 7.5 in BM-25.0 2.0 in OGDL 6.0 in CTA	11.0 in CRCP 6.0 in 21A
B	2.0 in SM-12.5 2.0 in IM-19.0 10.0 in BM-25.0 6.0 in Type I, No. 21B 12.0 in No. 1 Aggregate	2.0 in SM-12.5 2.0 in IM-19.0 9.0 in BM-25.0 2.0 in OGDL 6.0 in CTA	13.0 in CRCP 2.0 in OGDL 6.0 in CTA

BM = base mix; CRCP = continuously reinforced concrete pavement; CTA = cement-treated aggregate; IM = intermediate mix; OGDL = open-graded drainage layer; SM = surface mix.

Suggested Values for Design Requirements

The suggested values for design requirements remain the same as those established in the VDOT *Pavement ME User Manual* (VDOT, 2017). Tables 31 through 33 show the design requirements VDOT uses.

Table 31. Flexible Pavement Performance Criteria

Performance Criteria (Flexible)	Limit	Limit at Year
AC Bottom Up Cracking (% lane area)	6	30
Permanent Deformation—Total Pavement (in)	0.26	15

AC = asphalt concrete.

Table 32. Semi-Rigid Pavement Performance Criteria

Performance Criteria (Semi-Rigid)	Limit	Limit at Year
AC Fatigue Cracking (% lane area)	6	30
Permanent Deformation—Total Pavement (in)	0.26	15

AC = asphalt concrete.

Table 33. CRCP Pavement Performance Criteria

Performance Criteria (CRCP)	Limit	Limit at Year
CRCP Punchouts (#/mile)	6	30

CRCP = continuously reinforced concrete pavement.

Notable Changes from Current VDOT Pavement ME Design Methods

The VDOT upgrade from AASHTOWare Pavement ME Design version 2.2.6 to version 3 introduces updates to the transfer functions and to the locally calibrated coefficients. These changes are evident in the flexible pavement rutting model, particularly when comparing predicted rutting under high versus low traffic volumes.

In version 2.2.6 (currently used by VDOT), the flexible pavement rutting model predicts increasingly higher rut depths as truck traffic grows. However, these predictions do not consistently align with the levels of rutting observed across the VDOT network for comparable traffic volumes. As Figures 31 through 34 show, the solid line represents VDOT's rutting threshold of 0.26 inches, and the dotted and dash-dot lines (in blue) show the predicted rutting at the specified reliability of 95% for this design. At an annual average daily truck traffic of 8,000, the model in version 2.2.6 (Figure 32) produces a noticeably steeper rutting progression than in version 3 (Figure 34). This steep, overprediction of rutting is primarily driven by the elevated $\beta 3$ calibration coefficient, which amplifies the effect of traffic loading on rutting outcomes.

During the version 3 calibration, the rutting coefficients were adjusted to better match the measured distresses in VDOT's roadway network. The $\beta 3$ coefficient in the global rutting prediction model was reduced so that traffic no longer has as strong an influence as it did in version 2.2.6. versus version 3. To compensate for this reduction, the $\beta 1$ and $\beta 2$ coefficients were increased. Together, these changes improved the model's alignment with measured rutting data and produced lower bias and SEE values. Figures 33 and 34 show the updated rutting predictions for version 3.

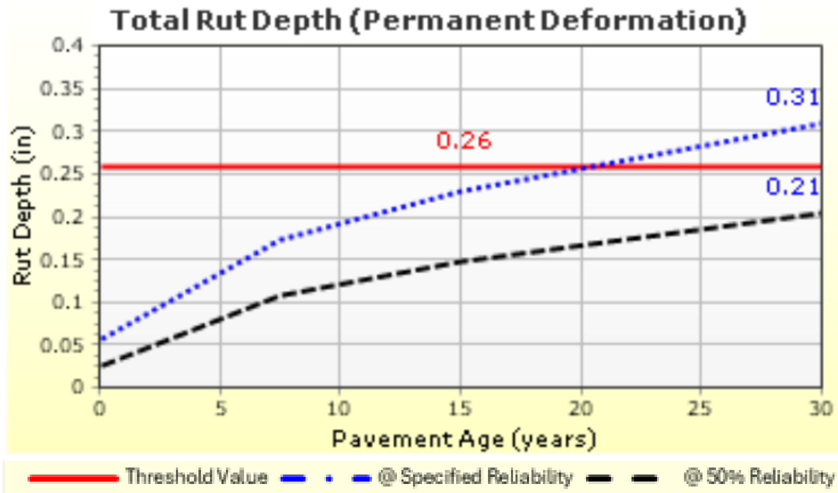

Figure 31. Predicted Rut Depth for a Pavement Designed for Annual Average Daily Truck Traffic of 2,000 in Version 2.2.6

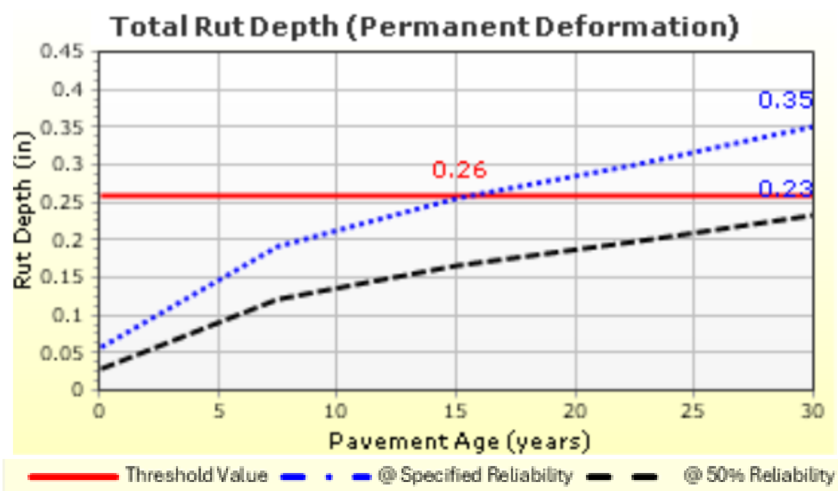

Figure 32. Predicted Rut Depth for a Pavement Designed for Annual Average Daily Truck Traffic of 8,000 in Version 2.2.6

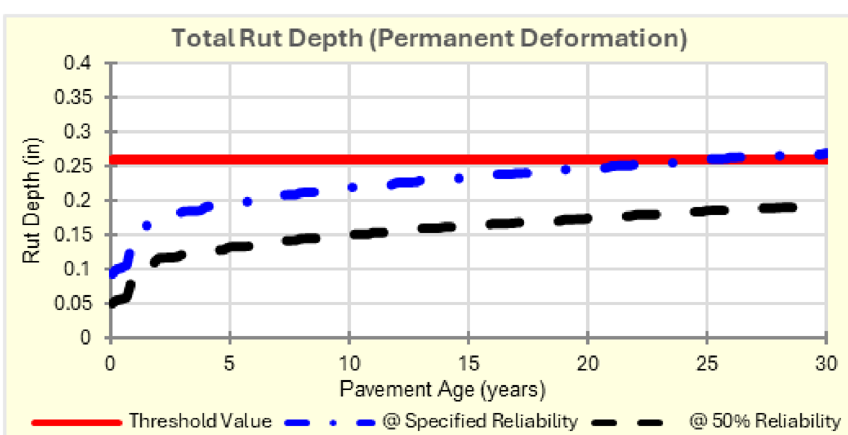

Figure 33. Predicted Rut Depth for a Pavement Designed for Annual Average Daily Truck Traffic of 2,000 in Version 3

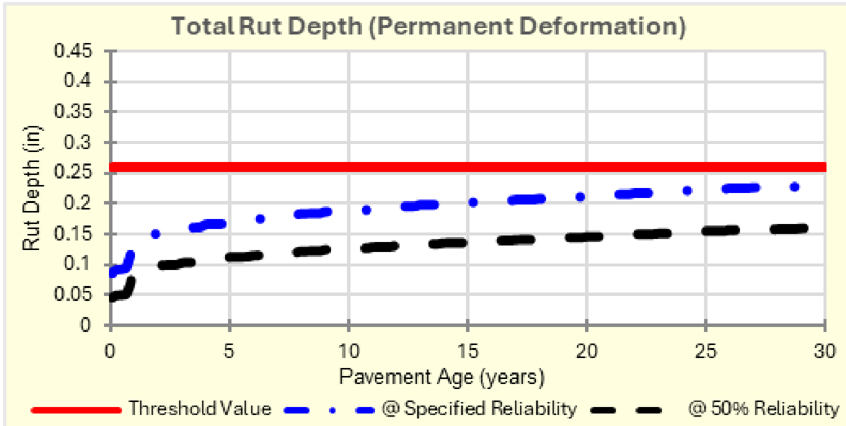

Figure 34. Predicted Rut Depth for a Pavement Designed for Annual Average Daily Truck Traffic of 8,000 in Version 3

Limitations

The locally calibrated coefficients presented in this report are specific to the conditions and data used in their development. The climate data, traffic data, pavement structure, and material inputs reflect VDOT's current design practices and procedures when using the AASHTOWare Pavement ME Design software. As these inputs evolve over time through refinement or improvements, recalibration of the coefficients for any design module (flexible, semi-rigid, or CRCP) may become necessary.

CONCLUSIONS

- *The recommended local calibration coefficients listed in this study offer improved pavement distress predictions for the VDOT roadway network over the global coefficients*. However, the applicability of these findings is limited to the pavement types and distress levels evaluated in this study. Extra consideration is warranted when interpreting results for pavement types and distress levels not included in this calibration effort.

- *The rutting model for flexible pavements using global coefficients consistently overpredicted distresses*. Local calibration of these coefficients helped to adjust the bias and SEE values within the preferred ranges while maintaining the other key considerations, such as consistency of the coefficients between flexible and semi-rigid for the same materials.

- *The global top down cracking model for flexible pavements indicated an overprediction of the distresses*. The coefficients were locally calibrated to result in reduced bias and SEE values.

- *The local calibration for the bottom up cracking model for flexible pavements yielded no bias and a low SEE value.*

- *For the semi-rigid model, the total cracking model resulted in a large overprediction of cracking with a poor SEE value*. This result was largely attributed to the predicted reflective cracking from the CTB; thus, the associated coefficients with this distress were optimized to better match the measured and predicted distresses.

- *The total transverse cracking distress prediction within the semi-rigid model resulted in underpredictions of cracking originating from the reflective cracking portion of the model*. To address this, the default value of 25-foot crack spacing was reduced to 15 feet, and the thermal cracking coefficients were adjusted based on MAAT.

- *CRCP over an unbound granular base resulted in unrealistic punchout predictions because of the predicted cracking within the model*. The crack spacing was set to a constant 48 inches, which reduced the predicted punchouts and brought bias within acceptable ranges.

- *IRI depends on multiple distress models and must be recalibrated when those models are updated*. Because several flexible and semi-rigid distress models were refined, the IRI model also requires adjustment. The statistical performance for CRCP was also inadequate; therefore, further refinement of the IRI model is recommended if it is intended to be pursued in the future.

- *Version 3 of the web-based AASHTOWare Pavement ME software introduces an updated interface that differs from the desktop version, creating a need for training to ensure users are familiar with the changes*.

RECOMMENDATIONS

1. *VDOT should incorporate the recommended calibration coefficients identified in this study into future new pavement designs within the AASHTOWare Pavement ME Design web-based software. Global coefficients should be used for jointed plain concrete pavement designs*.

2. *VDOT's Materials Division, with support from the Virginia Transportation Research Council (VTRC), should update the* Pavement ME User Manual *to reflect the calibration adjustments recommended in this study, as well as the interface and data input changes introduced between versions 2.2.6 and 3 of the AASHTOWare Pavement ME Design web-based software*.

3. *VDOT's Materials Division, with the support of VTRC, should provide training courses to ensure that VDOT users are informed about updates made to the web-based software interface, the semi-rigid design model, and any new design considerations*.

IMPLEMENTATION AND BENEFITS

The researcher and the technical review panel (listed in the Acknowledgments) for the project collaborate to craft a plan to implement the study recommendations and determine the benefits of doing so. This process is to ensure that the implementation plan is developed and approved with the participation and support of those involved with VDOT operations. The implementation plan and the accompanying benefits are provided here.

Implementation

Regarding Recommendations 1 and 2, VDOT's Materials Division will implement the local calibration coefficient and design value recommendations from this study by incorporating them into VDOT's Pavement ME User Manual by March 31, 2027, following a thorough internal review and comprehensive sensitivity analysis.

Regarding Recommendation 3, VDOT's Materials Division, with the support of VTRC, will provide training on incorporating the revised calibration values into the Pavement ME Design, developing pavement designs using the updated software, and utilizing the semi-rigid model. This training will be delivered by March 31, 2028. In addition, the ME design method, using the revised calibration coefficients, will be applied to comparison designs alongside VDOT's current design procedure to continue evaluating the updated process and to verify that the predicted outcomes remain consistent with engineering judgment and experience.

Benefits

Implementing the new Pavement ME Design software, procedure, and coefficients will support the development of pavement structures that are better optimized for the VDOT network based on cost and service life, versus those produced using global coefficients. Preliminary findings indicate that pavement thickness can be reduced by approximately 1 to 2 inches when using specific material properties, with the optimal reduction depending on traffic loading and climatic conditions. This thickness optimization translates into an estimated cost savings of $50,000 to $100,000 per mile for new pavement designs. With the planned sunset of the desktop software currently in use, integrating these newly developed models into the web-based version will also help ensure a smooth and efficient transition from the existing desktop platform. The transition to the web-based version will provide a more efficient design process and improved modeling approaches for roadway design within VDOT.

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APPENDIX A

Flexible Pavements

Asphalt Concrete Rutting

$$\frac{\varepsilon_p}{\varepsilon_r} = k_z \beta_{r1} 10^{k_1} T^{k_2 \beta_{r2}} N^{k_3 B_{r3}}$$
$$k_z = (C_1 + C_2 * depth) * 0.328196^{depth}$$
$$C_1 = -0.1039 * H_\alpha^2 + 2.4868 * H_\alpha - 17.342$$
$$C_2 = 0.0172 * H_\alpha^2 - 1.7331 * H_\alpha + 27.428$$

Where:

$$H_{ac} = total\ AC\ thickness(in)$$

$$\varepsilon_p = plastic\ strain\left({}^{in}/_{in}\right)$$
$$\varepsilon_r = resilient\ strain\left({}^{in}/_{in}\right)$$
$$T = layer\ temperature\left({}^{\circ}F\right)$$
$$N = number\ of\ load\ repetitions$$

Subgrade Rutting

$$\delta_a(N) = \beta_{s_1} k_1 \varepsilon_v h \left(\frac{\varepsilon_0}{\varepsilon_r}\right) \left| e^{-\left(\frac{\rho}{N}\right)^{\beta}} \right|$$

$$\delta_a = permanent\ deformation\ for\ the\ layer$$
$$N = number\ of\ repetitions$$
$$\varepsilon_v = average\ vertical\ strain(in/in)$$
$$\varepsilon_0, \beta, \rho = material\ properties$$
$$\varepsilon_r = resilient\ strain(in/in)$$

Fatigue Cracking

$$N_f = 0.00432 * C * \beta_{f1} k_1 \left(\frac{1}{\varepsilon_1}\right)^{k_2 \beta_{f2}} \left(\frac{1}{\mathrm{E}}\right)^{k_3 \beta_{f3}}$$

$$C = 10^M$$

$$M = 4.84 \left(\frac{V_b}{V_a + V_b} - 0.69\right)$$

$$FC_{Bottom} = \left(\frac{1}{60}\right)\left(\frac{C_4}{1 + e^{C_1 C_1^* + C_2 C_2^* Log(DI_{Bottom} \times 100)}}\right)$$

$$C_1^* = -2C_2^*$$

$$C_2^* = -2.40874 - 39.748(1 + H_{AC})^{-2.856}$$

Top Down Cracking

$$L(t) = L_{Max} e^{-\left(\frac{C_1 \rho}{t - C_3 t_o}\right)^{C_2 \beta}}$$

$$t_0(\mathrm{Days}) = \frac{k_{L1}}{1 + \mathrm{e}^{(k_{L2} \times 100 \times \mathrm{a}_0 / 2\mathrm{A}_0) + (k_{L3} \times \mathrm{HT}) + (k_{L4} \times \mathrm{LT}) + (k_{L5} \times \log_{10} \mathrm{AADTT})}}$$

Semi-Rigid Pavements

Asphalt Concrete Rutting

$$\frac{\varepsilon_p}{\varepsilon_r} = k_z \beta_{r1} 10^{k_1} T^{k_2 \beta_{r2}} N^{k_3 \beta_{r3}}$$
$$k_z = (C_1 + C_2 * depth) * 0.328196^{depth}$$
$$C_1 = -0.1039 * H_\alpha^2 + 2.4868 * H_\alpha - 17.342$$
$$C_2 = 0.0172 * H_\alpha^2 - 1.7331 * H_\alpha + 27.428$$

Where:

$H_{ac} = total\ AC\ thickness(in)$

$\varepsilon_p = plastic\ strain({}^{in}/_{in})$
$\varepsilon_r = resilient\ strain({}^{in}/_{in})$
$T = layer\ temperature({}^\circ F)$
$N = number\ of\ load\ repetitions$

Subgrade Rutting

$$\delta_a(N) = \beta_{s_1} k_1 \varepsilon_v h \left(\frac{\varepsilon_0}{\varepsilon_r}\right) \left| e^{-\left(\frac{\rho}{N}\right)^{\beta}} \right|$$

$\delta_a = permanent\ deformation\ for\ the\ layer$
$N = number\ of\ repetitions$
$\varepsilon_v = average\ vertical\ strain(in/in)$
$\varepsilon_0, \beta, \rho = material\ properties$
$\varepsilon_r = resilient\ strain(in/in)$

Fatigue Cracking

$$N_f = 0.00432 * C * \beta_{f1} k_1 \left(\frac{1}{\varepsilon_1}\right)^{k_2 \beta_{f2}} \left(\frac{1}{\mathrm{E}}\right)^{k_3 \beta_{f3}}$$
$$C = 10^M$$
$$M = 4.84\left(\frac{V_b}{V_a + V_b} - 0.69\right)$$

$$FC_{Bottom} = \left(\frac{1}{60}\right)\left(\frac{C_4}{1 + e^{C_1 C_1^* + C_2 C_2^* Log(DI_{Bottom} \times 100)}}\right)$$
$$C_1^* = -2C_2^*$$
$$C_2^* = -2.40874 - 39.748(1 + H_{AC})^{-2.856}$$

Cement-Treated Base Cracking

$$FC_{ctb} = C_1 + \frac{C_2}{1 + e^{C_3 - C_4 * log_{10}(Damage)}}$$

Cement-Treated Base Fatigue

$$N_f = 10^{\left(\frac{k_1 \beta_{c1} \left(\frac{\sigma_s}{M_r}\right)}{k_2 \beta_{c2}}\right)}$$

$N_f = number\ of\ repetitions\ to\ fatigue\ cracking$
$\sigma_s = Tensile\ stress(psi)$
$M_r = modulus\ of\ rupture(psi)$

Reflective Cracking

$$\Delta C = k_1\Delta_{bending} + k_2\Delta_{shearing} + k_3\Delta_{thermal}$$

$$\Delta D = \frac{C_1 k_1\Delta_{bending} + C_2 k_2\Delta_{shearing} + C_3 k_3\Delta_{thermal}}{h_{OL}}$$

$$\Delta_{Bending} = A(SIF)_B^n$$

$$\Delta_{Shearing} = A(SIF)_S^n$$

$$\Delta_{Thermal} = A(SIF)_T^n$$

$$D = \sum_{i=1}^{N} \Delta D$$

$$RCR = \left(\frac{100}{C4 + e^{C5 \log D}}\right) * EX\_CRK$$

Where

ΔC	=	Crack length increment, in
ΔD	=	Incremental damage ratio
$k_1, k_2, k_3, C_1, C_2, C_3, C_4, C_5$	=	Calibration factors (local and global)
$\Delta_{bending}$, $\Delta_{shearing}$, $\Delta_{thermal}$	=	Crack length increments caused by bending, shearing, and thermal loading
A, n	=	HMA material fracture properties
N	=	Total number of days
$(SIF)_B$, $(SIF)_S$, $(SIF)_T$	=	Stress intensity factors caused by bending, shearing, and thermal loading
D	=	Damage ratio
h_{OL}	=	Overlay thickness, in
RCR	=	Cracks in the underlying layers reflected, %
EX_CRK	=	Transverse cracking in underlying pavement layers, ft/mile (transverse cracking) Alligator cracking in underlying pavement layers, % lane area (alligator cracking)

Thermal Fracture

$$C_f = \beta_{t1} N\left[\frac{1}{\sigma_d} \log\left(\frac{C}{h_{AC}}\right)\right]$$

$$\Delta C = A(\Delta K)^n$$

$$A = k_t \beta_t 10^{[4.389 - 2.52\log(E_{HMA}\sigma_m n)]}$$

Cf = Observed amount of thermal cracking, ft. / 500ft.
β_{t1} = Regression coefficient determined through global calibration (400)
N[z] = Standard normal distribution evaluated at [z]
σ_d = Standard deviation of the logarithm of crack depth in the pavement (0.769), in.
C = Crack depth, in.
h_{AC} = Thickness of asphalt layer, in.
ΔC = Change in the crack depth due to a cooling cycle
ΔK = Change in the stress intensity factor due to a cooling cycle
A, n = Fracture parameters for the asphalt mixture
E = Asphalt mixture stiffness, MPa
σ_m = Undamaged mixture tensile strength, MPa
k_t = Regression coefficient determined through field calibration
β_t = Calibration parameter

Continuously Reinforced Concrete Pavements

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

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

APPENDIX B

Table B-1. Criteria for Assessing Adequacy of MEPDG Transfer Functions

Criterion of Interest	Test Statistic	Range of Statistical Value	Rating or Assessment
Goodness of fit	s_e/s_y, (all model)	< 0.35	Very good
		0.35 to 0.55	Good
		0.56 to 0.75	Fair
		0.76 to 1.0	Poor
		> 1.0	Very poor
	Coefficient of determination (R^2), percent (all models)	81 to 100	Very good (strong relationship)
		64 to 81	Good
		49 to 64	Fair
		< 49	Poor (weak relationship)
	Global rut depth model SEE	< 0.1 in.	Good
		0.1 to 0.2 in.	Fair
		> 0.2 in.	Poor
	Global fatigue cracking model SEE	< 5 percent	Good
		5 to 10 percent	Fair
		> 10 percent	Poor
	Global transverse cracking model SEE[3]	< 300 ft./mi.	Good
		300 to 600 ft./mi.	Fair
		> 600 ft./mi.	Poor
	Global IRI model SEE	< 20 in/mi	Good
		20 to 35 in/mi	Fair
		> 35 in/mi	Poor
Bias	Hypothesis testing of slope of the linear measured versus predicted values. H_0: slope = 1	p-value > 0.05 (i.e., a 5 percent significant level)	Accept
		p-value > 0.05 (i.e., a 5 percent significance level)[4]	Accept
	Hypothesis testing of intercept of the linear measured versus predicted values. H_0: Intercept = 0	p-value > 0.05	Accept
	Paired t-test between measured and predicted values. H_0: Bias = 0	p-value > 0.05	Accept

IRI = International Roughness Index; MEPDG = *Mechanistic Empirical Pavement Design Guide*; SEE = standard error of the estimate.

APPENDIX C

Flexible Pavement Additional International Roughness Index Analysis

The International Roughness Index (IRI) model's initial verification was performed once all the calibration coefficients had been optimized based on the results from the three previous distress models. Initial verification shows that global calibration coefficients resulted in a minor overprediction and a low standard error of the estimate (SEE). Figure C-1 shows the initial verification results. The study team hypothesized that the thermal cracking component within the IRI model may contribute to some of the IRI overprediction. The thermal transverse cracking model was not calibrated as part of the VDOT flexible pavement sections. As such, it was recommended to initially set the thermal transverse cracking coefficient (C3) to zero to minimize the effect of predicted thermal transverse cracking on the predicted IRI.

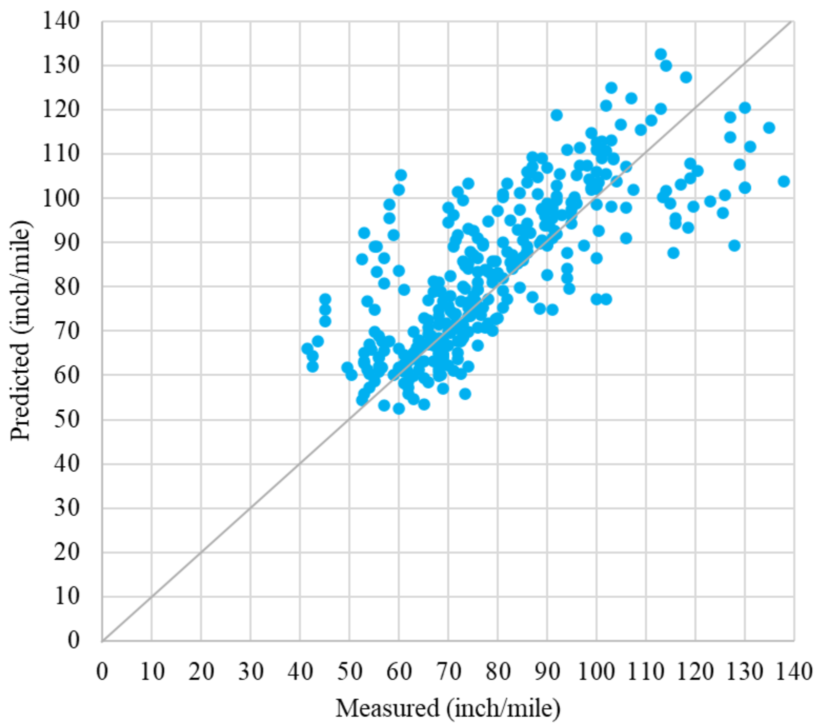

Figure C-1. Initial Verification Results of the International Roughness Index Model for Flexible Pavements

The preliminary optimization process began with setting the C3 coefficient to zero, as mentioned previously, followed by evaluating the total fatigue cracking (C2) component between 0.2 and 1.0. Generally, the lower the C2 value, the less impact fatigue cracking will have on the predicted IRI values. Selecting a C2 value on the lower end also produced lower SEE and bias. Next, the C1 (rutting) component of the IRI prediction was optimized. Like the C2 coefficient, the reduction of C1 will also reduce its effect on the IRI prediction. Ultimately, the C1 value selected was less than the global value in an effort to reduce bias and SEE. Table C-1 shows the results from the preliminary optimization of the IRI prediction model coefficients. Figures C-2 and C-3 show the data from the optimization and the validation results for this process, respectively.

Table C-1. International Roughness Index Model Preliminary Optimization Results

Calibration Iteration #	Adjusted Coefficient	Adjustment Range	Accepted Coefficient	Bias (inch/mile)	SEE (inch/mile)
Verification	N/A	N/A	N/A	3.95	13.60
1	C3 and C2	C3: 0 C2: 0.2–1.0	0 0.2	2.40	12.59
2	C1	10–35	22.5	-0.03	11.74

N/A = not applicable; SEE = standard error of the estimate.

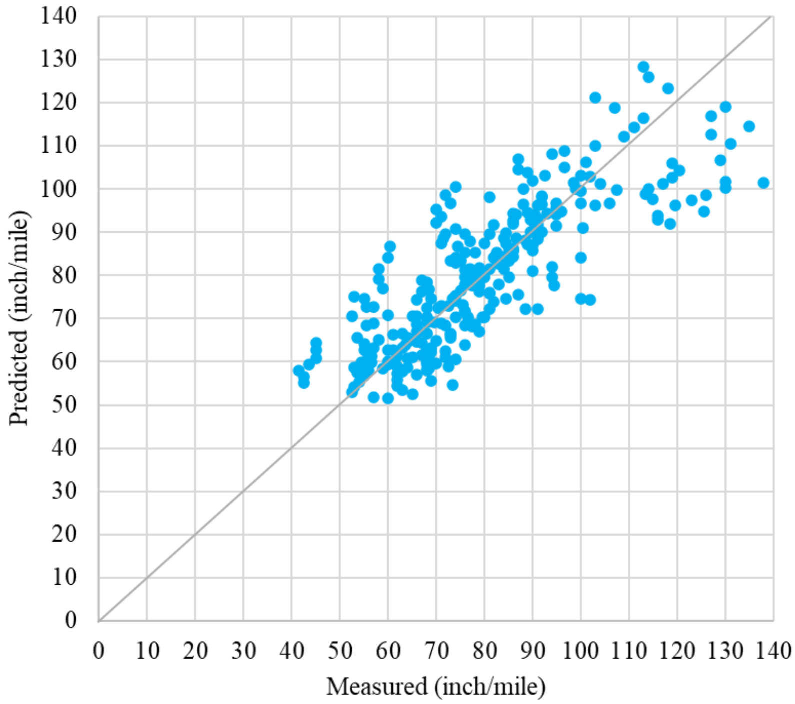

Figure C-2. Measured Versus Predicted International Roughness Index for Flexible Pavements from the Preliminary Optimization Process

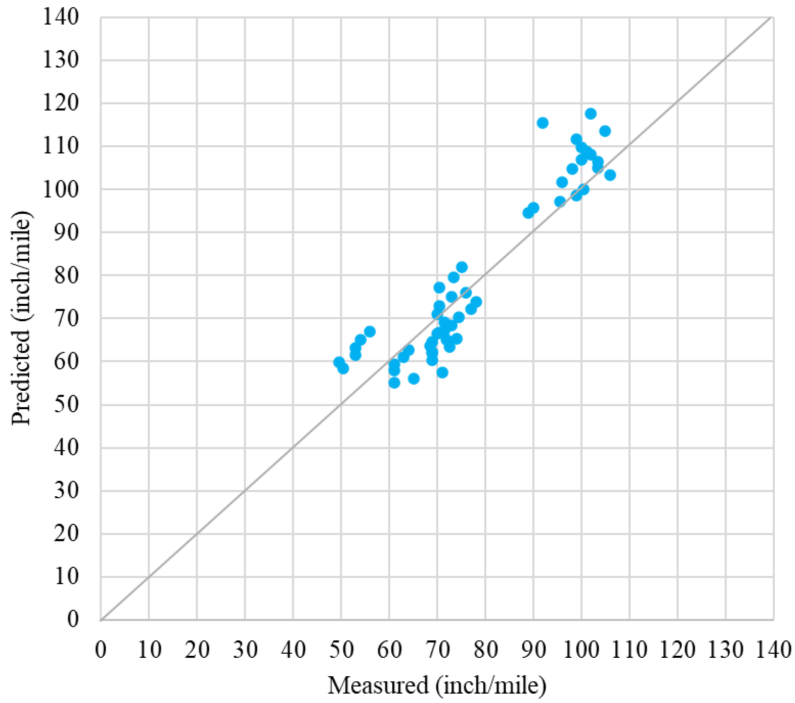

Figure C-3. Measured Versus Predicted International Roughness Index for Flexible Pavements from the Preliminary Validation Process

Table C-2 shows the statistical adequacy ratings for the preliminary IRI model verification, optimization, and validation of the final selected coefficients. The resulting coefficients from the preliminary optimization showed improved values for the SEE and bias. The R^2 parameter improved from fair to good after optimization.

Table C-2. IRI Model Coefficient Preliminary Optimization Statistics Ratings

Flexible IRI Model Statistics Ratings	Verification		Optimization		Validation	
Test Statistic	Value	Rating	Value	Rating	Value	Rating
Bias	3.95	-----	-0.03	-----	1.341	-----
SEE	13.60	Good	11.74	Good	7.64	Good
R^2	0.59	Fair	0.67	Good	0.87	Very Good
Hypothesis testing of slope of the linear measured vs. predicted: True if slope = 1.	0.00	Reject	0.00	Reject	0.06	Accept
Hypothesis testing of intercept of the linear measured vs. predicted: True if intercept = 0.	0.00	Reject	0.00	Reject	0.11	Accept
Paired t-test between measured and predicted: True if difference = 0.	0.00	Reject	0.97	Accept	0.17	Accept

IRI = International Roughness Index; SEE = standard error of the estimate.

Continuously Reinforced Concrete Pavement Additional IRI Analysis

The initial verification of the CRCP IRI model resulted in a very high bias and SEE. After further investigation of individual section data, a significant overprediction was found in four of the CRCP sections, far beyond what is shown in field distress data. Thus, they were removed from the optimization process. Figures C-4 and C-5 show the data from the optimization and the validation results for this process, respectively. Based on the new results, the IRI model tended to underpredict measured IRI and showed improved bias and SEE values.

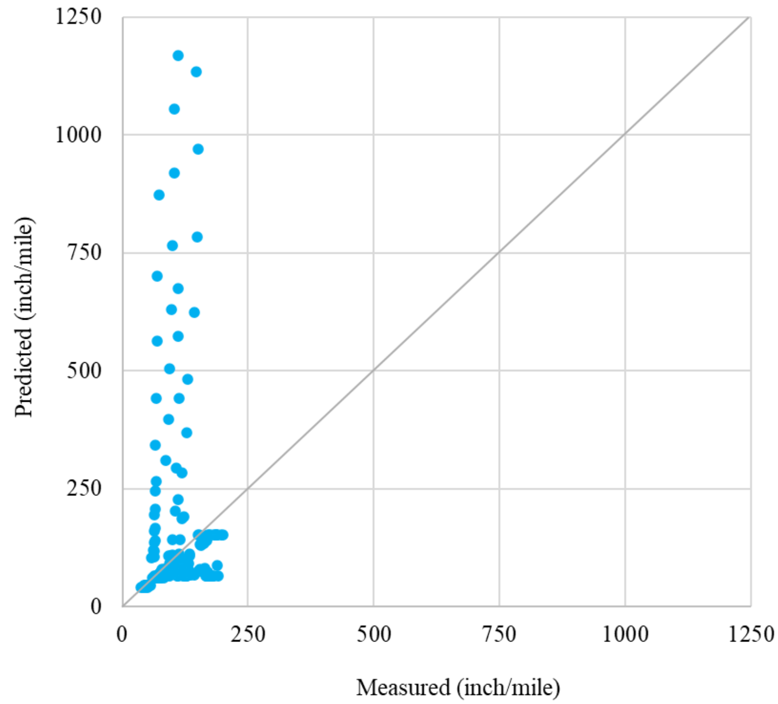

Figure C-4. Initial Verification Results of the International Roughness Index Model for Continuously Reinforced Concrete Pavements

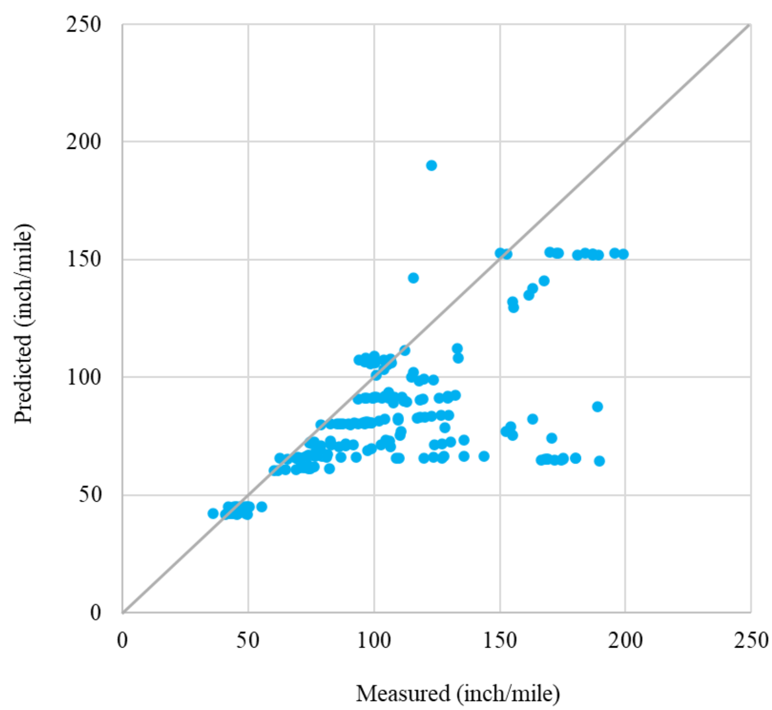

Figure C-5. Revised Verification Results of the International Roughness Index Model for Continuously Reinforced Concrete Pavements

The preliminary optimization of the IRI model was performed on the C1 coefficient from the punchout model, resulting in a significant reduction in the overall bias and a reduction in the SEE value. Two additional optimization iterations were conducted to further optimize the model. Figures C-6 and C-7 show the data from the optimization and the validation results for this process, respectively. Table C-3 shows the preliminary optimization iteration results.

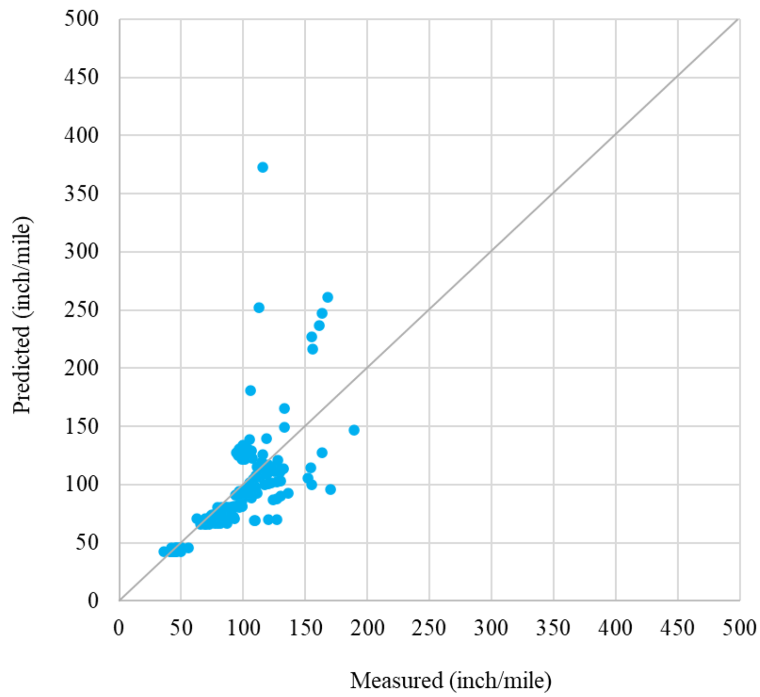

Figure C-6. Measured Versus Predicted International Roughness Index for Continuously Reinforced Concrete Pavements from the Preliminary Optimization Process

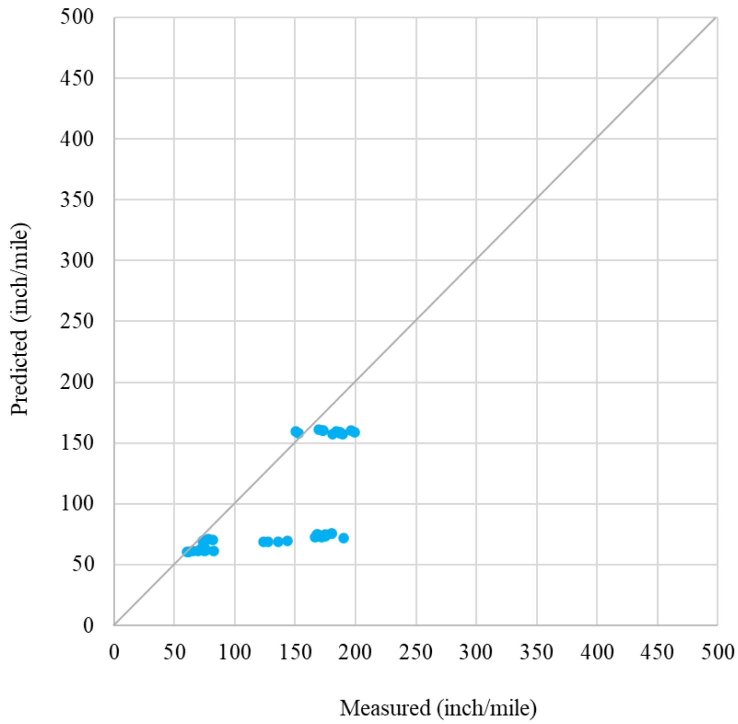

Figure C-7. Measured Versus Predicted International Roughness Index for Continuously Reinforced Concrete Pavements from the Preliminary Validation Process

Table C-3. CRCP International Roughness Index Model Coefficient Preliminary Optimization Results

Calibration Iteration #	Adjusted Coefficient	Adjustment Range	Accepted Coefficient	Bias (inches)	SEE (inches)
Verification	N/A	N/A	N/A	-22.197	36.497
1	C1	2–7	-	-15.341	26.548
2	C1	5–25	-	-4.290	38.519

CRCP = continuously reinforced concrete pavement; N/A = not applicable; SEE = standard error of the estimate.

Table C-4 shows the statistical adequacy ratings for the preliminary IRI model verification, calibration, and validation of the final selected coefficients. The preliminary calibration results showed a reduction in bias, an increase in SEE and a poor R^2 value.

Table C-4. CRCP IRI Model Coefficient Preliminary Optimization Statistics Ratings

CRCP IRI Model Statistics Ratings	Verification		Optimization		Validation	
Test Statistic	**Value**	**Rating**	**Value**	**Rating**	**Value**	**Rating**
Bias	-22.20	-----	0.25	-----	-36.20	-----
SEE	36.50	Poor	47.54	Poor	53.60	Good
R^2	0.45	Poor	0.34	Poor	0.45	Poor
Hypothesis testing of slope of the linear measured vs. predicted: True if slope = 1.	0.00	Reject	0.40	Accept	0.00	Reject
Hypothesis testing of intercept of the linear measured vs. predicted: True if intercept = 0.	0.00	Reject	0.42	Accept	0.04	Reject
Paired t-test between measured and predicted: True if difference = 0.	0.00	Reject	0.95	Accept	0.00	Reject

CRCP = continuously reinforced concrete pavement; IRI = International Roughness Index; SEE = standard error of the estimate.