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Developing an Implementation Strategy for Virginia Department of Transportation Pavement Rehabilitation Design Using Mechanistic-Empirical Concepts

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

BIPAD SAHA, P.E.
Pavement Engineer
Materials Division
Virginia Department of Transportation

GIRUM MERINE, P.E.
District Pavement Engineer
Hampton Roads District
Virginia Department of Transportation

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VIRGINIA TRANSPORTATION RESEARCH COUNCIL
530 Edgemont Road, Charlottesville, VA 22903-2454
vtrc.virginiadot.org

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16. Abstract:

The *Mechanistic-Empirical Pavement Design Guide* (MEPDG) was developed with an objective to provide the highway community with a state-of-the-practice tool for the design of new and rehabilitated pavement structures. The Virginia Department of Transportation (VDOT) officially adopted the MEPDG for new construction for interstate and primary routes effective January 1, 2018. For rehabilitation design, VDOT currently uses an earlier-generation AASHTO guide, the *1993 Guide for Design of Pavement Structures,* but expects eventually also to implement the MEPDG for the most common scenarios. To ensure a more effective overlay design, it is imperative to conduct a local calibration/validation of design procedures and to determine the proper material inputs for both the existing and any new pavement materials that may be used in the rehabilitation.

The purpose of this study was to assist VDOT in the implementation of AASHTOWare Pavement ME Design software (hereinafter "Pavement ME Design") for the design of overlays for existing flexible, rigid, and composite pavement. The study evaluated various input levels and the need for separate local calibration factors for rehabilitation of asphalt concrete (AC) over AC, AC over jointed concrete, and AC over continuously reinforced concrete pavements using Version 2.2.6 of Pavement ME Design. The study recommends implementation of the use of the current Version 2.2.6 for rehabilitation design only after a detailed sensitivity analysis with regard to various distresses using current calibration coefficients. Further, the study recommends the promotion of detailed forensic evaluation as part of rehabilitation design for restorative maintenance projects and that VDOT consider adopting V2.6 of Pavement ME Design for new and rehabilitation design.

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FINAL REPORT

DEVELOPING AN IMPLEMENTATION STRATEGY FOR VIRGINIA DEPARTMENT OF TRANSPORTATION PAVEMENT REHABILITATION DESIGN USING MECHANISTIC-EMPIRICAL CONCEPTS

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

Bipad Saha, P.E.
Pavement Engineer
Materials Division
Virginia Department of Transportation

Girum Merine, P.E.
District Pavement Engineer
Hampton Roads District
Virginia Department of Transportation

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ABSTRACT

The *Mechanistic-Empirical Pavement Design Guide* (MEPDG) was developed with an objective to provide the highway community with a state-of-the-practice tool for the design of new and rehabilitated pavement structures. The Virginia Department of Transportation (VDOT) officially adopted the MEPDG for new construction for interstate and primary routes effective January 1, 2018. For rehabilitation design, VDOT currently uses an earlier-generation AASHTO guide, the *1993 Guide for Design of Pavement Structures,* but expects eventually to also implement the MEPDG for the most common scenarios. To ensure a more effective overlay design, it is imperative to conduct a local calibration/validation of design procedures and to determine the proper material inputs for both the existing and any new pavement materials that may be used in the rehabilitation.

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INTRODUCTION

The *Mechanistic-Empirical Pavement Design Guide* (MEPDG) was developed under National Cooperative Highway Research Program (NCHRP) Project 1-37A with an objective to provide the highway community with a state-of-the-practice tool for the design of new and rehabilitated pavement structures (American Association of Highway and Transportation Officials [AASHTO], 2020). The MEPDG uses the calculated mechanistic response combined with empirical results from pavement test sections in the Long-Term Pavement Performance Program to predict the performance of pavement structures (Applied Research Associates, Inc., 2004). It calculates pavement responses (stresses, strains, and deflections) based on inputs such as traffic, climate, and material parameters to predict the pavement damage over time for asphalt pavements. After this step, transfer functions relate computed pavement responses (e.g., pavement damage) to observed pavement distresses. The mechanistic-empirical (ME) principles in the MEPDG were incorporated into analysis software commissioned by AASHTO and supported as AASHTOware Pavement ME Design software (hereinafter “Pavement ME Design”).

Implementation of the MEPDG has been proceeding throughout North America since its release. A 2020 report regarding the FHWA Pavement ME User Group meetings put the number of implementing agencies (using or conducting further review) at 18 and the number of agencies planning to implement the MEPDG in the future at 24 (FHWA Pavement ME User Group, 2020). VDOT officially adopted the MEPDG for new construction (new alignment, lane addition, and total reconstruction including full-depth reclamation projects) for interstate and primary routes effective January 1, 2018. VDOT completed several steps before implementing the ME pavement design procedures, including developing traffic inputs (Smith and

Diefenderfer, 2010); characterizing material properties (Apeagyei and Diefenderfer, 2011; Hossain, 2008; Hossain, 2010; Hossain et al., 2016); calibrating and validating the models (Smith and Nair, 2015); and providing training. Based on these studies and ongoing research, discussions with experts, and testing within Pavement ME Design, VDOT's *Pavement ME User Manual* that details how designers should enter project information for Virginia was developed (VDOT, 2021).

For rehabilitation design, VDOT currently uses an earlier-generation AASHTO guide, the 1993 *Guide for Design of Pavement Structures* (AASHTO, 1993) but also expects eventually to implement ME principles for the most common scenarios. To ensure a more effective design, it is imperative to conduct a local calibration/validation of design procedures and to determine the proper material inputs for both the existing and any new pavement materials that may be used in rehabilitation. Although pavement experts suggest that it is possible to implement Pavement ME Design for both new and rehabilitation designs simultaneously, the 2020 FHWA Pavement ME User Group meeting report recommended determining the cracking and rutting calibration coefficients for new design first and then applying them to the rehabilitation sections (FHWA Pavement ME User Group, 2020).

A good rehabilitation design must also start with an assessment of the overall condition of the existing pavement. AASHTO recommended that agencies collect and evaluate sufficient information about the existing pavement to minimize the chances of under- or over-designing the rehabilitated structure (AASHTO, 2020). A 2006 VDOT Instructional and Informational Memorandum addressed this by requiring appropriate evaluation and design for pavement sections identified as needing restorative maintenance and reconstruction (VDOT, 2006). In a subsequent study conducted at the Virginia Transportation Research Council (VTRC), Diefenderfer et al. (2018) observed widespread debonding and moisture damage in certain existing pavements and recommended further study to determine the causes and solutions to the structural issues observed.

As mentioned earlier, VDOT completed local calibration of the MEPDG distress models for new asphalt pavements, focusing on fatigue cracking and rutting and punchout outputs for continuously reinforced concrete pavement (CRCP) (Smith and Nair, 2015). This previous work culminated in a recommended strategy for applying ME design for new pavements. A similar exercise for rehabilitation designs has not been conducted.

PURPOSE AND SCOPE

The purpose of this study was to assist VDOT in the implementation of Pavement ME Design for the design of overlays for existing flexible, rigid, and composite pavement. The researchers were asked to address at least the following key points:

- types of rehabilitation scenarios that are going to be adopted by VDOT
- types of input (and input level) to be used during the design to characterize the existing pavement

- any need for separate local calibration factors for rehabilitation and new design
- any need for separate threshold values for new and rehabilitation design
- modeling of composite pavements in Pavement ME Design (existing CRCP, jointed plain concrete pavement [JPCP], and jointed reinforced concrete pavement [JRCP] with an asphalt concrete [AC] overlay).

The scope of the study was limited to analysis/investigation of selected projects representing VDOT's most common rehabilitation scenarios. Consistent with VDOT's approach for construction/reconstruction, all analyses were conducted using V2.2.6 of Pavement ME Design.

METHODS

Literature Search

A literature search was conducted to gather information on the implementation of Pavement ME Design for rehabilitation design by other transportation agencies.

Pavement Rehabilitation Scenarios

Several different rehabilitation options using hot mix asphalt (HMA) and concrete overlays can be applied to existing pavements to extend their useful life. Currently, VDOT does not plan to use all of the rehabilitation options available in the MEPDG, as shown in Figure 1. This study identified the most widely used VDOT rehabilitation strategies.

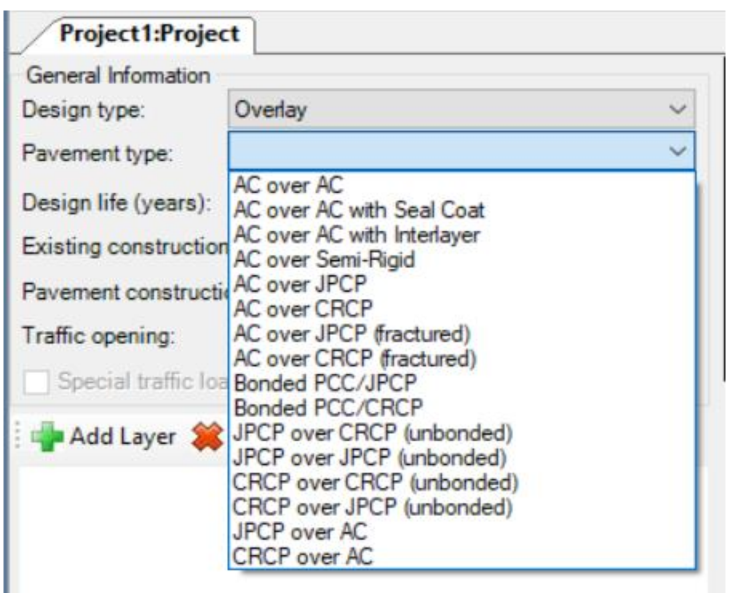

Figure 1. Overlay Options Available for Rehabilitation Design in Pavement ME Design

Types of Inputs Required During Design to Characterize Existing Pavement and Overlay

The MEPDG has different levels of input parameters that can be used depending on the availability and scope of the project. In general, Level 1 for analysis reflects the most accurate site-specific values; Level 3 inputs reflect the values estimated by using national correlation; and Level 2 inputs fall in between. Mixed and matched use of these input levels is allowed. This study evaluated different levels of distress inputs required by the MEPDG design procedure for typical rehabilitation design projects.

One of the initial objectives of this study was to compare Level 1 and Level 2 inputs for AC rehabilitation projects. Rehabilitation input Level 1 analysis requires backcalculated layer moduli values for each existing pavement layer as determined by testing with a falling weight deflectometer (FWD). The research team looked at the historical project data from the VDOT Materials Division database but was not able to obtain any that were suitable for this study. FWD data from an earlier VTRC project (US 60, Lynchburg District) was used to compare Level 1 and Level 2 input data for AC overlay rehabilitation design.

Assessment of Need for Separate Local Calibration Coefficients

The local calibration/validation process for this study was conducted in accordance with AASHTO's *Guide for Local Calibration of the Mechanistic-Empirical Pavement Design Guide* (hereinafter "AASHTO's *Local Calibration Guide*") (AASHTO, 2010), which gives details on developing an experimental plan and sampling template and estimating sample size for specific distress prediction models. The project information required for each calibration site had two aspects: field performance records, and project details. The field performance records were extracted from VDOT's Pavement Management System (PMS) network-level distress data, which include automated distress data beginning in 2007 and continuing at yearly intervals for all interstate and primary roadways. Distress data are measured at 0.1-mi intervals; the distresses at each interval within the project section were averaged to obtain the average rutting distress and added to obtain the bottom-up cracking percent for each site per year.

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

Traffic count records for the year of overlay construction were obtained from the VDOT traffic data jurisdiction report for the year of overlay construction. A growth rate was determined to calculate future design year traffic. The percent truck traffic was selected from the design year to determine the annual average daily truck traffic (AADTT) for input into Pavement ME Design. Statewide average values were used for vehicle class distribution, axle load spectra, and axles per truck in accordance with VDOT's *Pavement ME User Manual* (VDOT, 2021). Some of the inputs were left at national default values per the manual. A single weather station was selected near each project location to provide climatic data.

For existing asphalt pavement characterization, Level 1 input requires backcalculated modulus from FWD testing. These data were not available for the sites selected, so rehabilitation Level 2 inputs was used. Level 3 inputs were also evaluated, but based on the comparison of outputs to field performance data, the research team decided to use Level 2 inputs. Other pavement experts suggested that agencies should not use input Level 3 and that Level 1 and/or 2 should always be used (FHWA Pavement ME User Group, 2020**)**. There is no rehabilitation input level option for AC over CRCP and AC over JPCP. Full friction interface among layers was assumed. Global calibration efforts for flexible pavements were also completed assuming full friction between all layers (AASHTO, 2020).

The project-specific data entered into Pavement ME Design to produce predicted distresses were collected from a combination of sources. Asphalt pavement structure information including layer types, layer thicknesses, and year of construction was available from various sources including the PMS, network-level ground penetrating radar (GPR) data, project level GPR, and coring, and some information was provided by VDOT district materials personnel. The net thickness of the existing layer (coring depth minus milling depth) was entered as the layer thickness in accordance with AASHTO's *Mechanistic-Empirical Pavement Design Guide: A Manual of Practice* (hereinafter "*MEPDG Manual of Practice*") (AASHTO, 2020). For existing AC properties, performance grade (PG) and volumetric properties of VDOT's base mixture were used. Statewide average values were used for new asphalt mixture overlay properties. VDOT's typical values were used for aggregate base properties. The available subgrade information came from the VDOT Materials Division database, and some data were obtained from the Web Soil Survey (Natural Resources Conservation Service, n.d.). Checks were performed on the distress and construction records to remove data points that seemed unreasonable.

GPR testing was conducted by VDOT's non-destructive testing unit on 20 sites on interstate and primary routes. The main purpose for the GPR testing was to estimate the thickness of different pavement layers. The GPR data were collected only in the travel lane (right lane). In addition, GPR data were analyzed to find any anomalies within the pavement structure. GPR analysis can help to reduce the number of cores required for a project by segmenting the project by similar or different features identified and also helps in making decisions related to whether more detailed data collection efforts are needed (AASHTO, 2020). GPR testing was conducted with a 2-GHz horn antenna and SIR-30 controller manufactured by GSSI. Scans were collected at 1-ft intervals. VDOT's Materials Division processed the GPR data with RADAN-7 software. To verify the GPR thickness, pavement coring was performed in a few sites and the results were compared.

AC over JPCP and CRCP are modeled based on the available information obtained from the PMS and VDOT district materials staff. Some of the concrete sections also included JRCP and unique design features (e.g., a joint spacing of 61.5 ft was used). This information was obtained from district materials staff. AASHTO's *MEPDG Manual of Practice* suggests that JRCP can be modeled as JPCP (AASHTO, 2020). Full friction interface among layers was assumed. It was also assumed that existing distressed slabs were restored/repaired before overlay. A few existing composite pavement projects were also included. As per the AASHTO ME Design FY21 webinar series, existing composite pavements can also be modeled in Pavement ME Design (AASHTO, 2021). Most of the project surface layers were milled before overlay, so VDOT intermediate mixture (IM) properties were used to represent the existing asphalt layer for analysis. Guidance in AASHTO's *MEPDG Manual of Practice* (AASHTO, 2020) was used to predict reflection cracking for AC overlay over JPCP (in lieu of FWD testing). As per the manual, when dowels are present, the joints are rated as having good load transfer efficiency (LTE) (i.e., LTE > 60%) and when dowels are not present, the joints are rated as having poor LTE. Since the project list included AC over JPCP, AC over JRCP, and existing composite pavements, a value of 50% LTE was used as a conservative approach.

After the distress and Pavement ME Design predictions were developed, they were compared to evaluate the datasets. The main parameters to evaluate the fit of the distress or IRI prediction models are the bias and standard error of the estimate (Se). AASHTO defines these terms as the systematic offset between predicted and observed values and the variability between the predicted and measured values, respectively (AASHTO, 2010). The residual error represents the difference between the measured and predicted values for each data point; the bias was calculated as the average of the individual residual errors, and the Se was the standard deviation of the residual error. Another way used to evaluate the residual error is to compare the Se to the standard deviation of the measured distress (Sy); the Se/Sy ratio should decrease with local calibration. These values were calculated by entering the predicted and measured performance into a spreadsheet. The total rutting and bottom-up fatigue cracking were the primary models of interest for asphalt pavements, with IRI and bottom-up + reflective cracking also being considered. Pavement ME Design also includes models to predict top-down fatigue cracking and thermal + reflective cracking; top-down models were not considered for calibration because the updated models are included only in the latest versions (V2.5 and V2.6) of the software. The needs for further local calibration were identified for each distress; variable definitions and other details for the models are provided in AASHTO's *MEPDG Manual of Practice* (AASHTO, 2020) and AASHTO'S *Local Calibration Guide* (AASHTO, 2010). The guide also provides recommendations for transfer function calibration coefficients to be adjusted for eliminating bias and reducing the standard error.

Need for Separate Threshold Values for New and Rehabilitation Design

Another important factor for rehabilitation design procedures is the selection of design requirement properties including design life, reliability level, and target performance values. These values are an important component of the transition from analysis of pavement structures with Pavement ME Design to development of pavement designs that can efficiently balance cost and pavement performance. Initial recommendations were provided for rehabilitation design criteria based on the current criteria used for design of new pavements.

Modeling Approaches for Composite Pavement or Multiple Overlays of Portland Cement Concrete Pavements (AC Over Existing CRCP, JPCP, and JRCP)

Pavement ME Design does not include composite pavement as a design option. However, existing composite pavements can be analyzed in Pavement ME Design by conducting an AC over portland cement concrete (PCC) analysis (AASHTO, 2020).

RESULTS AND DISCUSSION

Literature Review

As per the 2020 FHWA Pavement ME User Group meeting report, only a few transportation agencies have implemented the rehabilitation design option in Pavement ME Design (FHWA Pavement ME User Group, 2020). The Indiana DOT (INDOT) performed their first major calibration for asphalt pavement rutting in 2017. INDOT is currently planning to transition from Pavement ME Design V2.3 to V2.6. Analysis of the permanent deformation model showed comparable levels of predicted total rutting between the 2017 locally calibrated model (using Pavement ME Design V2.3) and the Pavement ME Design V2.6 default model. The Pavement ME Design V2.6 default model predicted substantially more bottom-up cracking than the V2.3 default model, but the use of a lower in-place air-void content (7% instead of 8%) showed more reasonable predictions of bottom-up cracking. INDOT also evaluated the new top-down cracking model in V2.6 (FHWA Pavement ME User Group, 2020). A comparison of pavement designs from INDOT showed that thicknesses developed from the ME design procedure were less than those developed from the 1993 AASHTO procedure (AASHTO, 1993) for all examples of both asphalt and concrete pavements; this thickness reduction translated to an estimated cost savings of more than $10 million (Nantung, 2010).

The Missouri DOT completed a second local calibration study in 2020 using Pavement ME Design V2.5.5. The study looked at both new designs and rehabilitation designs (asphalt and concrete overlays); however, the primary focus was calibrations for new design. Although calibrations were performed for the various performance models, the Missouri DOT uses only bottom-up fatigue cracking and permanent deformation (AC-only and total rutting) as design criteria for full-depth HMA pavement and transverse cracking and joint faulting as design criteria for full-depth JPCP. The Missouri local calibration study recommended the use of Level 1 field data for future design (FHWA Pavement ME User Group, 2020; Titus-Glover et al., 2020).

The Utah DOT has been conducting pavement designs using Pavement ME Design since 2011. An initial local calibration for new asphalt pavement and overlays (focusing on the bottom-up fatigue cracking, transverse cracking, rutting, and IRI models) was performed in 2009. This was followed by a second local calibration study in 2013 that focused on the rutting models. With the agency's transition to Pavement ME Design V2.5, a third calibration involving all of the mentioned models was performed in 2019 (FHWA Pavement ME User Group, 2019). The New Jersey DOT uses Pavement ME Design V2.5.5 to design new and reconstructed asphalt pavements and also performs parallel designs with the 1993 AASHTO guide (AASHTO, 1993) for asphalt overlays (FHWA Pavement ME User Group, 2019).

Recognizing the importance of local calibration of flexible pavement performance models, the National Center for Asphalt Technology conducted a study to review the general approach undertaken for state highway agencies. The results of those efforts and recommendations for implementing the nationally or locally calibrated models are documented (Robbins et al., 2017). In preparation for local calibration of ME distress models, the Georgia DOT published a report on local calibration activities being conducted by state highway agencies (Von Quintus et al., 2013). The synthesis showed that many states are working toward calibration by focusing on building design input libraries for material and traffic inputs. Further, the synthesis showed that states that performed local calibration of asphalt pavements consistently found that the global predictions from the ME design method overpredicted rutting and developed local calibration factors to improve the prediction. The local calibration of the asphalt fatigue cracking transfer function showed more variability than the rutting model but reasonably estimated the measured levels of cracking over a broad range of pavement structures.

The Kansas DOT is currently participating in a pooled fund study, TPF-5(311): Implementation of the AASHTO Mechanistic-Empirical Design Guide for Pavement Rehabilitation Design, that is looking at the design of AC overlays for existing asphalt and concrete pavements. The calibration efforts, which are not yet available, will focus on V2.6 of Pavement ME Design.

Pavement Rehabilitation Scenarios

After an evaluation of VDOT's rehabilitation practices over the years and discussions with VDOT pavement experts, the research team decided to evaluate Pavement ME Design for AC over AC, AC over CRCP, and AC over JCP design options only.

Types of Input Levels to Characterize the Existing Pavement

The first step in the pavement rehabilitation process involves assessing the overall condition of the existing pavement. In Pavement ME Design, rehabilitation design considers distresses developing in the overlay and the continuation of damage from the existing pavement structure. The new overlay helps to reduce the rate at which distresses develop in the existing pavement (AASHTO, 2020). The design also provides for the reflection of the distresses from existing pavement through the overlay layers. Thus, the condition of the existing pavement has a major effect on the development of damage in the new AC layers. The pavement structural evaluation for determining the condition of the existing pavement layers can include visual distress surveys, coring, deflection tests, and other field and laboratory tests.

Pavement ME Design allows the designer to use different input levels (Levels 1, 2, and 3) based on the importance of the project and available resources (AASHTO, 2020). Rehabilitation input Level 1 analysis requires backcalculated layer moduli values for each existing pavement layer determined using FWD testing; values for transverse cracking (feet/mile) with severity level (low, medium, or high); and values for rutting in the existing pavement layers. The backcalculated modulus from the deflection basin is used to calculate the damage in the existing layers. Level 2 inputs require values for fatigue cracking (%) and transverse cracking (ft/mi) and rutting in the existing pavement layers. The software also requires values for the severity level of existing fatigue cracking (%) and transverse cracking, which are used for selecting values for LTE. For Level 2 input, the fatigue cracking (%) defines the level of damage for the existing layers. Rehabilitation input Level 3 requires structural and environmental ratings (excellent to poor) and the rut depth to characterize existing pavement damage. As mentioned previously, agencies are encouraged not to use input Level 3 if Level 1 and/or Level 2 inputs are available (FHWA Pavement ME User Group, 2020).

Information on many of the factors related to the existing pavement condition (fatigue cracking [%] and transverse cracking [feet/mile] and rutting in the existing pavement) can be obtained from the PMS. For example, Table 1 shows typical PMS data. Training for pavement design staff may include how to extract the PMS data for Level 1 and 2 inputs.

For Level 1, there is a need to supplement the PMS data with FWD deflection data that can be used to characterize the existing pavement structure through backcalculation, in which the in-situ layer moduli of the existing overlay and underlying base and subgrade modulus are estimated based on the measured surface deflections, the magnitude of the load, and information on the pavement layer thicknesses. The stand-alone software program, Deflection Data Analysis and Backcalculation Tool (BcT), is available with Pavement ME Design to generate backcalculation inputs (using the EVERCALC algorithm) from the FWD test for generating Level 1 inputs for rehabilitation design. Training for pavement design staff may also include analyzing FWD data with the BcT tool.

Table 1. VDOT PMS Distress Data

Year	CCI	IRI	Rutting	Cracking Severity 1	Cracking Severity 2	Cracking Severity 3	% Cracking	Transverse Cracking-Severity-I	Transverse Cracking-Severity-II
2007	48	134	0.06	2733	18764	3233	18.33	2945	8675
2008	33	133	0.10	1858	15080	8386	18.80	1875	7244
2009	100	94	0.06	0	2	0	0.00	0	0
2010	99	93	0.07	162	353	0	0.36	0	0
2011	97	97	0.09	249	426	0	0.48	54	0
2012	97	96	0.08	327	695	5	0.74	86	14
2013	93	103	0.08	190	646	83	0.67	110	21
2014	77	105	0.07	4339	3809	0	5.87	2163	89
2015	79	106	0.07	4070	3560	0	5.83	547	5
2016	82	104	0.10	5141	1084	0	4.37	1384	47
2017	75	110	0.10	6442	925	0	5.12	1867	165
2018	49	114	0.14	5492	815	0	4.45	1492	116
2019	49	96	0.14	5131	1840	0	4.98	1502	360

Data shaded in yellow indicated rutting and % cracking before overlay; PMS = Pavement Management System; CCI = Critical Condition Index; IRI = International Roughness Index.

Though surface distresses provide a valuable insight into a pavement's current structural condition, coring of existing pavement is required to assess the layer damage. For example, Figure 2 shows core photographs where delamination/damage is confined to the top layer of the surface course and requires removal of that layer. The depth of milling is an input in Pavement ME Design. The thickness of the existing AC layers represented in Pavement ME Design is the thickness of the AC layers measured from cores minus the depth of milling (AASHTO, 2020). Cores should be evaluated for moisture damage, mixture deterioration, and delamination, etc. If no moisture damage or mixture deterioration is observed through the asphalt core, it can be simulated as one layer (as shown in Figure 3). Figure 4 shows delamination in multiple layers and moisture damage at the bottom. In this situation, engineering judgment/experience is needed to select a proper rehabilitation design. Using the cores, the thickness of the individual layers can be examined to make a decision for grouping the different existing AC layers. The GPR and FWD deflection data can also be used to estimate the variability along a project and determine if the damage or layer thicknesses of the pavement structure are significantly different along the project.

Figure 2. Core Showing Delamination in Top Surface Layer (I-95 SB, Richmond District)

Figure 3. Intact Core on Rte. 58 EB, Richmond District

Figure 4. Core Showing Delamination in Multiple Layers and Moisture Damage at Bottom Layer (Rte. 250 EB, Richmond District)

Layer interface friction is an input parameter in Pavement ME Design, and cores and visual surveys can be used to determine if debonding exists along the project. Slippage cracks in the surface and separation of layers during the coring process may be an indication of low interface friction between AC layers (AASHTO, 2020). If debonding exists, the designer can assume no bond or a low interface friction during the rehabilitation design if those layers are to remain place and not be milled or removed. It is recommended that debonded layers be removed in practice (AASHTO, 2020).

In-place air voids, asphalt content, and gradation are required inputs for existing AC layers for undamaged modulus prediction. Air voids of existing layers can be obtained from project records, and the average effective asphalt content by volume and gradation measured during construction are used for the rehabilitation design. Cores from the project can be used to measure these properties if they are not available from construction records. The ignition oven can be used to measure the asphalt content, which can be followed by a gradation analysis that can be conducted on the remaining aggregate. Air voids can be calculated from bulk specific gravity and maximum theoretical specific gravity. Asphalt binder extract from the cores can be used to determine the PG of the recovered asphalt. Historical binder grade data can also be used. The asphalt grade and volumetric test results are used to determine the undamaged condition of the AC layer (AASHTO, 2020). Pavement ME Design provides the user with two options for estimating the undamaged dynamic modulus: a viscosity-based model, and the G* based model. The global calibration factors for all AC predictive equations were determined using the viscosity-based model (AASHTO, 2020). An undamaged modulus value is then compared to the average backcalculated modulus to estimate the amount of damage for Level 1 input. As mentioned previously for Level 2 input, fatigue cracking (%) is used to calculate damage in the existing asphalt layer. After this step, the software calculates damaged dynamic modulus. For existing layer material properties, PG and volumetric properties (gradation, asphalt content by volume, in-place air voids) must be entered. The volumetric properties of the lower AC layer should be used, since that is where fatigue cracking will initiate.

Properties (dry density, moisture content, resilient modulus, etc.) for the existing unbound and subgrade layers are also needed. If the resilient modulus values are determined by backcalculating elastic layer modulus values from deflection basin tests, based on AASHTO's *MEPDG Manual of Practice* (AASHTO, 2020), the values need to be adjusted to laboratory conditions by applying a correction factor.

Previous studies showed that use of Level 1 and Level 2 inputs for the existing asphalt pavement layer resulted in substantial differences in overlay design; the advantages of using Level 1 inputs instead of Level 2 inputs were demonstrated by Pierce and Smith (2015). According to national experts, rehabilitation input Levels 1 and 2 should result in comparable designs; otherwise, further investigation is needed (AASHTO, 2021). Ayyala et al. (2018) conducted a detailed study titled *Characterizing Existing Asphalt Concrete Layer Damage for Mechanistic Pavement Rehabilitation Design* and provided important information for Level 1 inputs. Their study suggested that backcalculated modulus using FWD data includes a bias relative to the laboratory E* and that bias is temperature dependent. The authors recommended that an adjustment factor be applied to backcalculated values entered into Pavement ME Design similar to the correlation factors for unbound layers and provided some recommendations for the same. Their study also showed that large differences in the predicted amount of fatigue cracking can be expected between MEPDG rehabilitation input Levels 1 and 2 when all other inputs are equal. They recommended that the backcalculated AC elastic moduli and damage index ratio be compared to the amount of cracking exhibited on the pavement surface when a rehabilitation input level to be used for design is selected. Zeng et al. (2021) conducted a study where two pavement structures in North Carolina were selected to evaluate the accuracy of the Pavement ME Design guide using its three levels of inputs. They found that Levels 1, 2, and 3 can each lead to significantly different damaged master curves. They recommended the Level 1 method if the existing pavement was a multilayered asphalt pavement and suggested that core extracted from all the layers can be used to generate the input properties.

To compare Level 1 and 2 inputs for VDOT, an example project from US 60 in the Lynchburg District was selected. Cores showed an existing pavement thickness of 6.5 in. Traffic volume was very low (two-way AADT of 760). Existing pavement also included an aggregate base of 6 in and subgrade. FWD testing was conducted to backcalculate the modulus. The backcalculated modulus was 225,000 psi (at 73°F); a loading frequency of 15 Hz was used. To see the effect of damage prediction between Level 1 and 2 inputs, the same modulus for aggregate base and subgrade was used (backcalculated modulus showed lower values for aggregate base and subgrade). Existing pavement had 11% fatigue cracking, 0.1-in rutting, and 2,100 ft/mi transverse cracking. The first analysis was conducted with a straight overlay with a 2-in surface mixture (SM). Then, an analysis was conducted with 2-in mill and fill. Results are shown in Tables 2 and 3. Results clearly showed that Level 1 input (backcalculated modulus of AC) predicted more distress compared to Level 2 input. This was due to more damage prediction of asphalt layer when using backcalculated modulus compared to damage prediction based on fatigue cracking (%) (Level 2 input) of the existing asphalt layer. It should be noted that the software uses 50% reliability for bottom-up fatigue cracking for rehabilitation design.

Table 2. Distress Comparison of Level 1 and 2 (Straight Overlay)

Distress Type	Distress at Specified Reliability		Reliability (%)		Distress at Specified Reliability	Reliability (%)
	Target	Predicted Using Level 1 Inputs	Target	Predicted Using Level 1 Inputs	Predicted Using Level 2 Inputs	Predicted Using Level 2 Inputs
Terminal IRI (in/mi)	140.00	200	95.00	51	152	88
Permanent deformation, total pavement (in)	0.26	0.48	95.00	20	0.09	100
AC bottom-up fatigue cracking (% lane area)	6.00	3.01	50.00	99.86	0.00	100
AC top-down fatigue cracking (ft/mi)	2000.00	932	95.00	99.98	332	100

IRI = International Roughness Index; AC = asphalt concrete.

Table 3. Distress Comparison of Level 1 and 2 (Mill and Fill)

Distress Type	Distress at Specified Reliability		Reliability (%)		Distress at Specified Reliability	Reliability (%)
	Target	Predicted Using Level 1 Inputs	Target	Predicted Using Level 1 Inputs	Predicted Using Level 2 Inputs	Predicted Using Level 2 Inputs
Terminal IRI (in/mi)	140.00	205	95.00	47	153	88
Permanent deformation - total pavement (in)	0.26	0.60	95.00	5.71	0.10	100
AC bottom-up fatigue cracking (% lane area)	6.00	4	50.00	97.72	0.00	100
AC top-down fatigue cracking (ft/mi)	2000.00	1390	95.00	99.12	378	100

IRI = International Roughness Index; AC = asphalt concrete.

Rehabilitation input level is not an option for AC over CRCP and AC over JPCP. In Pavement ME Design, the AC over PCC analysis considers continued damage of the PCC slab under the AC overlay using the rigid pavement performance models (AASHTO, 2020). For existing JPCP, the joints, existing cracks, and any new cracks that develop during the overlay period are reflected through the AC overlay using the reflection cracking models of the ME-based fracture mechanics approach (AASHTO, 2020). A primary design consideration for AC overlays of existing CRCP is to perform full-depth repair of all working cracks and existing punchouts. Sufficient AC overlay is then provided to increase the structural section, keep the cracks sufficiently tight, and exhibit little loss of crack LTE over the design period. A sufficient AC overlay is also needed to reduce the critical top-of-slab tensile stress and fatigue damage that leads to punchouts (AASHTO, 2020).

AASHTO's *MEPDG Manual of Practice* (AASHTO, 2020) provides recommended assessment practice for existing rigid and flexible pavements. It also provides information to relate the condition of the pavement surface to whether the pavement is structurally adequate, marginal, or inadequate. Further, the manual offers candidate repair and preventive treatments for flexible, rigid, and composite pavements. Appendix A provides a detailed description of various input level requirements in Pavement ME Design. Appendix A along with information from AASHTO's *MEPDG Manual of Practice* (AASHTO, 2020) can be used for developing a Pavement ME Design user manual for rehabilitation design.

Assessment of the Need for Separate Local Calibration Coefficients

The AC over AC pavement sites (26 sites) considered for the analysis are listed in Table 4. Initially, 53 sites were considered, but the research team was not able to collect all the project details required for the analysis. It should be noted that some of the projects were initially constructed in the 1960s and obtaining all the pavement structure and rehabilitation details was difficult. Collecting project and distress data was one of the time-consuming parts of this study; VDOT's Materials Division was instrumental in collecting project details. Project sites were selected from all nine VDOT districts. Of 26 project sites, 15 represented interstates, 6 represented state routes, and 5 represented U.S. routes. Rehabilitation years spanned from 2007-2012. For 16 sites, the rehabilitation included two lifts of asphalt (SM and IM). For IM mixtures, 10 sites used IM 19.0 and 6 sites used SMA 19. SM thickness ranged from 1.5 to 2 in and IM mixtures were mostly 2-in thick. Most of the projects included milling (1.5 to 4 in), but 5 straight overlay projects were also included. Total asphalt thickness ranged from 8 to 13 in. Original pavement construction dates ranged from 1966-1997. Two-way average daily truck traffic (AADTT) ranged from 36 to 16, 640 vehicles and the growth rate of vehicles ranged from 0% to 3%. More details of the projects are presented in Appendix B.

Fourteen projects were selected for AC over CRCP calibration. Except for three state route projects (SR 288 and two projects on SR 58) all were interstates. Ten of the routes were selected from I-64 (Richmond, Hampton Roads, and Culpeper districts), and one project was selected from I-295 (Richmond District). More details about the projects are presented in Table 5 and Appendix B. Except for one SM, most of the overlay mixtures were SMA (1.5 to 2 in thick). Ten projects also included an IM (8 projects with SMA 19 (6 projects 2 in thick, and 2 projects 3 in thick) and 2 projects with IM 19 (3 in thick). Total AC thickness ranged from 2 to 6 in. For one-half of the projects (6 projects), overlay thickness was 3.5 in. CRCP construction (average thickness of 8 in) years ranged from 1966-1990, with most of them (9 projects) constructed before 1980. Average two-way AADT for these sections ranged from 1,960 to 8,640, with an average value of 4,170 (from years 2008-2016). Growth rate of traffic ranged from 1% to 7%, with an average value of 2.9%. Some of the projects were existing composite pavements where mill and fill was used as part of rehabilitation activity. The rehabilitation year ranged from 2008-2015.

Nineteen projects were selected for AC over JPCP of which 15 represented interstates (I-81, I-95, I-85, I-495, I-395), 3 represented U.S. routes, and 1 represented a state route. More details are provided in Table 6 and Appendix B.

Table 4. AC Over AC Project Sites

Sl No.	Name	District	County	Route No.	Type	Direction	Begin MP	End MP	Length	Pavement Type	Last Rehab Year
1	Br-I	Bristol	Washington	81	IS	NB	26.14	30.26	4.1	BIT	2010
2	Br-II	Bristol	Wythe	77	IS	NB	2.36	5.16	2.8	BIT	2010
3	Cul-I	Culpeper	Fauquier	66	IS	EB	15.2	18.63	3.430	BIT	2010
4	Fred-II	Fredericksburg	Spotsylvania	3	SR	EB	1.83	3.41	1.6	BIT	2008
5	HR-I	Hampton Road	Norfolk Maintenance	664	IS	WB	3.97	5.34	1.370	BIT	2007
6	Lynch-I	Lynchburg	Nelson	151	SR	NB	12.26	14.71	2.450	BIT	2010
7	Nova-I	NOVA	Loudoun	7	SR	EB	14	15.03	1.0	BIT	2011
8	Nova-II	NOVA	Loudoun	7	SR	EB	13	14	1.0	BIT	2011
9	Nova-III	NOVA	Prince William	66	IS	EB	0.00	1.82	1.820	BIT	2007
10	Rich-X	Richmond	Brunswick	58	US	WB	13.67	17.36	3.7	BIT	2011
11	Rich-V	Richmond	Goochland	64	IS	EB	7.39	8.95	1.6	BIT	2009
12	Rich-VII	Richmond	Goochland	64	IS	WB	22.34	23.2	0.9	BIT	2009
13	Rich-IX	Richmond	Goochland	250	US	EB	9.47	10.32	0.9	BIT	2009
14	Rich-III	Richmond	Henrico	64	IS	WB	1.86	0	1.9	BIT	2011
15	Rich-IV	Richmond	Henrico	95	IS	NB	0.75	2.98	2.2	BIT	2010
16	Rich-VI	Richmond	Henrico	95	IS	NB	11.45	12.56	1.1	BIT	2009
17	Rich-II	Richmond	Henrico	64	IS	EB	2.04	2.82	0.8	BIT	2010
18	Rich-XI	Richmond	Mecklenburg	58	US	EB	2	4.72	2.7	BIT	2011
19	Rich-XVIII	Richmond	Chesterfield	95	IS	SB	12.50	14.50	2.0	BIT	2011
20	Rich-XIX	Richmond	Chesterfield	95	IS	SB	8.39	10.39	2.0	BIT	2011
21	STN-VI	Staunton	Augusta	81	IS	SB	20.10	21.94	1.8	BIT	2008
22	STN-I	Staunton	Frederick	37	SR	NB	5.67	9.1	3.4	BIT	2010
23	STN-II	Staunton	Frederick	37	SR	SB	5.67	9.1	3.4	BIT	2010
24	STN-III	Staunton	Rockingham	33	US	EB	0	3.93	3.9	BIT	2010
25	STN-XIV	Staunton	Rockingham	11	US	NB	3.71	4.74	1.03	BIT	2008
26	Salem-III	Salem	Carroll	77	IS	NB	2.36	3.36	1	BIT	2012

AC = asphalt concrete; IS = interstate, US = US route, SR = state route; MP = milepost; BIT = AC over AC pavement.

Table 5. AC Over CRCP Sites

No.	County	Route No.	Type	Direction	Begin MP	End MP	Length	Pavement Type	Last Rehab Year
1	Albemarle	64	IS	EB	26.77	31.2	4.43	BOC	2010
2	Norfolk	64	IS	WB	22.16	27.82	5.66	BOC	2011
3	Norfolk	64	IS	EB	22.24	27.82	5.58	BOC	2011
4	Southampton	58	SR	EB	2.41	5.31	2.9	BOC	2012
5	Southampton	58	SR	EB	13	15	2	BOC	2012
6	Henrico	64	IS	EB	5.83	10.21	4.38	BOC	2010
7	Henrico	64	IS	WB	5.84	10.11	4.27	BOC	2010
8	Hanover	295	IS	EB	2.28	4.56	2.28	BOC	2010
9	Chesterfield	288	SR	SB	8.12	8.85	0.73	BOC	2011
10	Henrico	64	IS	WB	10.5	11.19	0.69	BOC	2010
11	Albemarle	64	IS	EB	8.86	11.3	2.44	BOC	2008
12	Louisa	64	IS	EB	5.22	7.06	1.84	BOC	2015
13	New Kent	64	IS	EB	17	20	3	BOC	2015
14	New Kent	64	IS	EB	2.5	4.1	1.6	BOC	2015

AC = asphalt concrete; CRCP = continuously reinforced concrete pavement; IS = interstate; SR = state route; MP = milepost; BOC = AC over AC pavement.

Table 6. AC Over Jointed Concrete Pavement Sites

Sl No.	Name	District	County	Route No.	Type	Direction	Begin MP	End MP	Length	Pavement Type	Last Rehab Year
1	Salem-I	Staunton	Botetourt	81	IS	NB	4.35	6.67	2.32	BOJ	2009
2	Salem-II	Staunton	Botetourt	81	IS	SB	4.37	6.61	2.24	BOJ	2009
3	Rich-VII	Richmond	Brunswick	1	US	SB	8.37	10.57	2.2	BOJ	2005
4	Rich-I	Richmond	Dinwiddie	85	IS	NB	21.41	23.94	2.53	BOJ	2009
5	Rich-II	Richmond	Dinwiddie	1	US	NB	2.51	3.54	1.03	BOJ	2008
6	Nova-I	NOVA	Fairfax	95	IS	SB	8.1	9.41	1.31	BOJ	2009
7	Fred-IV	Fredericksburg	Spotsylvania	95	IS		6.02	8.6	2.58	BOJ	2009
8	Fred-I New	Fredericksburg	Stafford	95	IS	NB	10	11.1	1.1	BOJ	2010
9	Fred-II	Fredericksburg	Stafford	95	IS	SB	10	11.09	1.09	BOJ	2009
10	Fred-III	Fredericksburg	Stafford	95	IS	SB	12.92	15.54	2.62	BOJ	2008
11	HR-II	Hampton Roads	York	64	IS	EB	34.12	35.15	1.03	BOJ	2010
12	Rich-I-CIR	Richmond	Henrico	60	US	EB, WB	8.96	10.2	1.24	BOJ	2011
13	Rich-II-CIR	Richmond	Prince George	35	SR	EB, WB	0	3.76	3.76	BOJ	2011
14	Rich-IX	Richmond	063-New Kent	64	IS	WB	0	1.03	1.03	BOJ	2007
15	I-85 Dinwiddie	Richmond	Dinwiddie	85		IS	14.5	15.31	0.81	BOJ	2015
16	I-95 Prince George	NOVA	Prince George	95	IS	SB	0.9	1.9	1	BOJ	2015
17	I-495 Fairfax	NOVA	Fairfax	495	IS	SB	13.4	14.8	1.4	BOJ	2015
18	I-395 Fairfax	NOVA	Fairfax	395	IS	SB	1.34	2.15	0.81	BOJ	2015
19	I-95 Spotsylvania	Fredericksburg	Spotsylvania	95	IS	NB	10.25	11.29	1.04	BOJ	2015

AC = asphalt concrete; IS = interstate; US = US route; SR = state route; MP = milepost; BOJ = AC jointed concrete pavement.

For most of the projects (12) the SM used was SMA; 6 used an SM E mixture, and 1 used an SM 12.5D mixture. Eleven projects also had an IM mixture (6 projects with SMA 19 mixture). Most of the projects were existing composite pavements so mill and fill (2 to 5 in) was included in the rehabilitation activity. Total AC overlay thickness ranged from 3 to 9 in. The majority of the pavements included old JRCP construction (construction years from 1965-1989). Two-way AADT ranged from 80 to 17,460 (based on the 2008-2015 years of data). Growth rate of traffic ranged from 0% to 2.6%.

Table 7 shows pre-overlay distress details of AC over AC sites. The PMS annually rates each section of highway through automated digital video logging. Pavement condition ratings from zero (0) to 100 are calculated using a deduct value system for each type of distress based on these video images. A higher rating number represents better pavement condition. In general, a Critical Condition Index (CCI) of 60 or below indicates a very poor condition requiring a significant rehabilitation and/or reconstruction. The CCI for these sections ranged from 26 to 73. The pre-overlay IRI from the PMS ranged from 60 to 142 in/mi. Average rutting was 0.16 in (standard deviation [SD] of 0.48 in). Cracking was the major pre-overlay distress observed (average value of 11% [SD: 8%, ranged from 2% to 40%]). Of 26 sites, only 8 sites showed cracking more than 10%. Transverse cracking was also observed in all sections. VDOT's 2016 State of the Pavement Report also showed that average rutting was below 0.18 in among interstate projects and that cracking was the predominant distress (VDOT, 2016). Statewide pre-overlay IRI values from VDOT's ride specification database are shown in Table 8. Overall, VDOT interstate projects showed before IRI ranges of 77 to 86 in/mi and after IRI ranges of 49 to 55 in/mi. VDOT has a good ride specification in place for rehabilitation projects, and a big difference in IRI was not observed for pre-overlay and after overlay for projects. Thus, IRI is not a controlling design criterion for VDOT.

Bottom-up cracking is defined as a series of interconnected cracks that initiate at the bottom of the AC layers. Top-down cracking is a load related distress where the crack initiates at the pavement surface and propagates downward through the asphalt layer. Top-down and bottom-up cracks are difficult to differentiate from visual observations, and use of coring is needed to confirm these two types of cracks. The PMS data do not distinguish top-down and bottom-up cracking; it was assumed that severity Level 2 and 3 cracks from the PMS were bottom-up cracks and severity Level 1 cracks were top-down cracks. As mentioned previously, AC thickness ranged from 8 to 13 in; it may be possible that many cracks originate from the top surface unless there is lot of damage in the underlying layers. Forensic investigation is needed to differentiate cracking and assess the damage in pavements. Forensic evaluation was not included in the first VDOT local calibration study for new construction.

As part of the forensic evaluation for this study, GPR testing was conducted for 20 sites to confirm pavement thickness. A list of the GPR-tested sites is shown in Table 9.

Table 7. Summary of Pre-Overlay Distress (AC Over AC Projects)

Name	Route Name	Year	CCI	IRI	Rutting	Alligator Cracking Sev1 (Top-Down)	Alligator Cracking Sev2	Alligator Cracking Sev3	Total Alligator Cracking	Total Alligator Cracking %	Bottom Alligator Cracking (Sev2 and 3)	Bottom Alligator Cracking %
STN-III	33	2010	30	140	0.11	477	92320	1532	94329	40.4	93852	40.2
Rich-XVIII	95	2011	40	99	0.17	3968	22636	17	26621	22.0	22653	18.7
Rich-VII	64	2009	35	88	0.16	1424	5981	1985	9390	19.6	7966	16.6
Fred-II	3	2008	47	92	0.11	590	14741	21	15352	18.4	14762	17.7
Br-II	77	2010	38	78	0.13	4110	24635	550	29295	17.8	25185	15.3
Nova -II	7	2011	51	79	0.09	1185	8498	611	10294	17.6	9109	15.6
Rich-V	64	2009	44	89	0.22	4455	4798	5767	15020	16.5	10565	11.6
Nova-I	7	2011	43	91	0.15	2335	5992	845	9172	15.3	6837	11.4
Rich-XIX	95	2011	48	121	0.18	4105	12349	184	16638	12.8	12533	9.6
Br-I	81	2010	31	86	0.14	6914	20000	24	26938	11.0	20024	8.2
Rich-III	64	2011	58	85	0.21	3627	6408	62	10097	9.3	6470	5.9
Lynch-I	151	2010	42	142	0.15	2237	9750	223	12210	9.1	9973	7.5
Rich-IV	95	2010	56	118	0.15	4250	7953	72	12275	9.1	8025	6.0
HR-I	664	2007	68	94	0.13	5449	1166	0	6615	8.0	1166	1.4
Rich-X	58	2011	52	60	0.32	3582	104	14171	17857	8.0	14275	7.1
STN-II	37	2010	56	86	0.16	2895	12666	318	15879	8.0	12984	6.5
Rich-VI	95	2009	26	113	0.21	1274	3606	325	5205	7.7	3931	5.8
Rich-IX	250	2009	47	109	0.18	1103	1985	25	3113	7.7	2010	4.9
Cul-I	66	2010	39	96	0.09	3481	8145	0	11626	5.6	8145	3.9
Rich-XI	58	2010	72	107	0.17	4610	2709	20	7339	4.9	2729	1.8
Nova-III	66	2008	42	85	0.15	1390	2591	463	4444	4.1	3054	2.8
STN-VI	81	2008	72	85	0.15	3474	0	0	3474	3.2	0	0.0
STN-I	37	2010	70	92	0.17	2150	3608	39	5797	2.8	3647	1.8
Rich-II	64	2011	50	91	0.20	552	540	2	1094	2.3	542	1.2
STN-XIV	11	2008	73	103	0.13	543	554	11	1108	1.8	565	0.93
				Avg.	**0.16**	**2807.20**	**10949.40**	**1090.68**	**14847.28**	**11.31**	**12040.08**	**8.90**
				SD	**0.047927**	**1733.8091**	**18318.768**	**2975.811**	**18264.214**	**8.447986**	**18380.542**	**8.628961**

AC = asphalt concrete; CCI = Critical Condition Index; IRI= International Roughness Index; Sev = severity; SD = standard deviation.

Table 8. IRI Pre-Overlay (AC Over AC Projects)

	IRI Average by Year									
	2016		2017		2018		2019		2020	
System	**Before**	**After**	**Before**	**After**	**Before**	**After**	**Before**	**After**	**Before**	**After**
IS	87	55	86	53	72	52	80	49	77	55
US	89	59	92	69	92	65	93	65	99	67
SR	98	68	106	76	100	74	113	80	101	71
CR	N/A	N/A	89	96	114	88	180	104	N/A	121

IRI = International Roughness Index; AC = asphalt concrete; IS = interstate; US = U.S. routes; SR = state routes; CR = county roads; N/A = not available.

Table 9. List of GPR-Tested Sites

Serial No.	District	Route No.	Type	Direction	Begin MP	End MP	Length
1	Fredericksburg	360	US	EB	9.45	12.20	2.750
2	Fredericksburg	3	SR	EB	1.83	3.41	1.6
3	Lynchburg	151	SR	NB	12.26	14.71	2.450
4	Richmond	64	IS	EB	25.1	26.34	1.2
5	Richmond	64	IS	EB	2.04	2.82	0.8
6	Richmond	64	IS	EB	7.39	8.95	1.6
7	Richmond	64	IS	WB	22.34	23.2	0.9
8	Richmond	64	IS	WB	7.33	9.04	1.7
9	Richmond	250	US	EB	9.47	10.32	0.9
10	Richmond	58	US	WB	13.67	17.36	3.7
11	Richmond	58	US	EB	2	4.72	2.7
12	Richmond	33	US	EB	14.64	16.91	2.3
13	Richmond	6	SR	EB	0.00	1.05	1.1
14	Staunton	42	SR	NB	2.83	5.72	2.9
15	Staunton	211	US	WB	2.74	5.64	2.9
16	Staunton	11	US	NB	3.71	4.74	1.030
17	Staunton	33	US	EB	0	3.93	3.9
18	Staunton	250	US	EB	27.99	29.32	1.3
19	NOVA	7	SR	EB	13	14	1.0
20	NOVA	66	IS	EB	0.00	1.82	1.820

GPR = ground penetrating radar; US = U.S. route; SR = state route; IS = interstate; MP = milepost.

The thickness of AC and base pavement layers was captured in the analysis. Potential separation of layers or possible air voids were observed in the asphalt pavement for a few projects (around 15 projects), and these could be a possible concern for the future rehabilitation treatment type selection process. Analysis showed that the actual thickness of AC and base material varied along the project. To confirm the actual AC and base material thickness, pavement coring was performed at a few sites and GPR thickness was verified with core thickness. Pavement coring was not performed at all sites due to funding and other logistical and resource constraint reasons. The layer thickness was summarized by using the average of the data taken at 0.1-mi intervals. The data represented by each point were 0.05 mi on either side of the testing point. Each layer interface at the bottom of the scan image was marked, and AC bottom and aggregate base were marked separately. Deeper layers that could be the bottom of an aggregate base layer were not visible in the approximately 20 in within the GPR scan. An example of the marks in RADAN indicating the bottom of the layers is shown in Figure 5.

In addition to the bottom of the presumed asphalt layer, a faint interface was also seen in the GPR data, typically between the AC layers. This represents layer interfaces for different maintenance treatments over the years, as shown in Figure 6. It is unclear from the analysis if the varying strength of the signal of this layer is related to a potential issue at the layer interface in some locations. This might be an indication of weaker bonding between the AC layers. However, to confirm the delamination in the AC layers, pavement coring is needed.

Pavement coring was performed on a few sites to evaluate the existing pavement condition and to compare the AC and base depth between the GPR and from the cores. The average GPR thickness and average core thickness were compared. The thickness of the AC layer from the cores and from the GPR were comparable, as shown in Table 10.

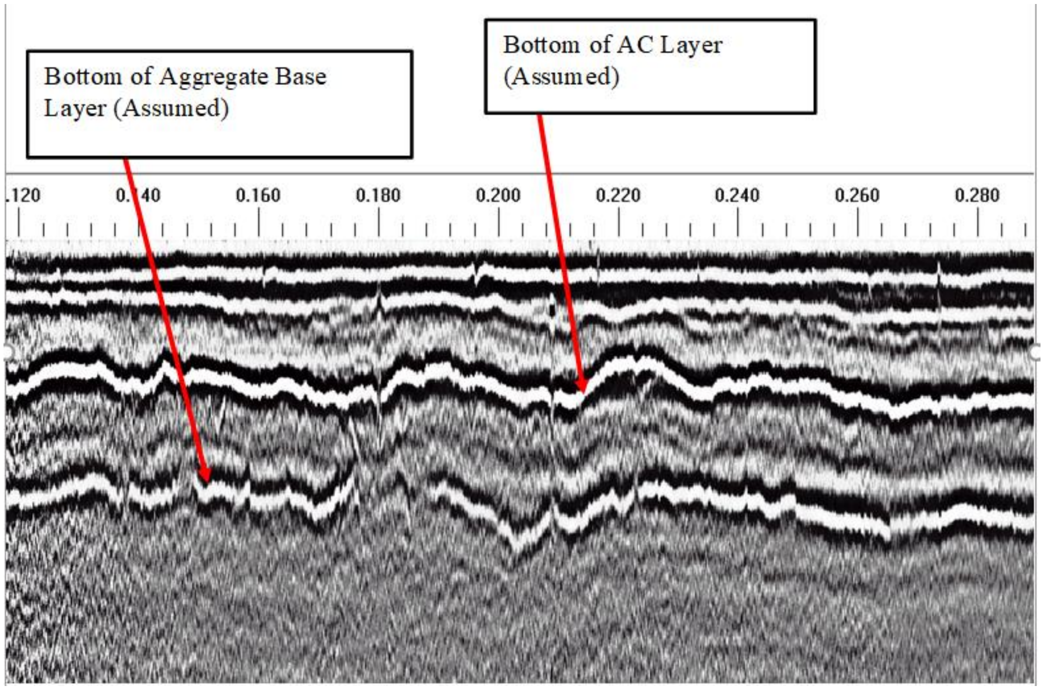

Figure 5. Example of GPR Scan. GPR = ground penetrating radar.

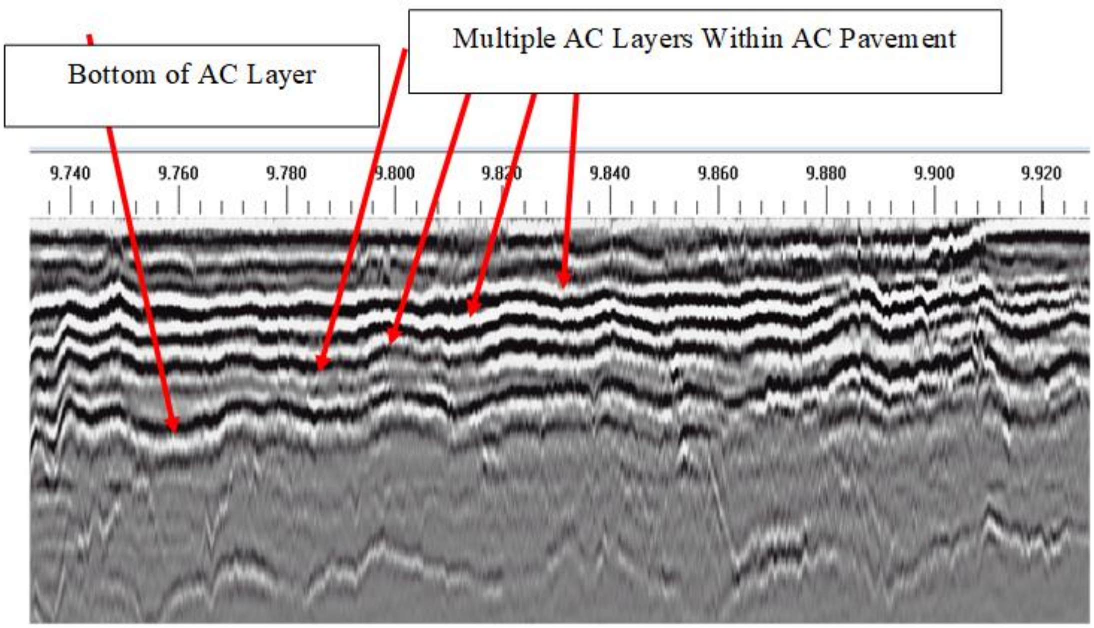

Figure 6. GPR Showing Possible Multiple AC Sublayers Within Asphalt or AC Pavement. GPR = ground penetrating radar; AC = asphalt concrete.

Table 10. Comparison of GPR and Core Thickness

Site No.	Coring Average AC Thickness, in	GPR Average AC Thickness, in	Core Average Base Thickness, in	GPR Average Base Thickness, in
1	12	11.3	8	7
2	7.5	7	4 to 8	8
3	11.6	10.8	4 to 8	7.1

GPR = ground penetrating radar; AC = asphalt concrete.

A detailed analysis of the pavement structural condition evaluation was conducted for Rte. 250 EB from GPR images. Rte. 250 EB is located in Goochland County, and the project limit was MP 9.47 to 10.32. This section was originally constructed in 1967 with AC on top of an aggregate base. Different rehabilitation treatments were applied over the years, as shown in Table 11 (maintenance history extracted from the PMS). The actual AC thickness was significantly higher than the PMS thickness, and this may be due to the details from the original construction and past treatments not being updated in the PMS.

Figure 7 shows a GPR-processed image. Processed images were evaluated carefully, and AC bottom and aggregate base bottom layers were marked. However, multiple faint AC layers were observed within the total AC layer. The presence of multiple layers may come from thin AC rehabilitation or also may be a potential indication of delamination within the AC layers. The GPR images were evaluated all along the project length, and the images show similar layers. Pavement coring was recommended to confirm the delamination within the AC layers.

Pavement coring was performed on Rte. 250 EB on July 23, 2020. Eight cores were taken from the wheel path and centerline locations. Most of the cores showed delamination and stripping, as shown from the GPR analysis. Core logs and images are shown in Appendix C for Rte. 250. Similar observations/results regarding delamination were also observed at two other pavement sites when GPR images were compared with coring. This shows that GPR can be used as an important tool in forensic investigation of asphalt pavements.

Table 11. Pavement Layer History From VDOT'S Pavement Management System

Year Completed	Treatment	Layer	Material Code	Thickness (in)
2020	BIT	1	Latex Modified Emulsion Type C	0.5
2009		1	SM-12.5A	1.0
2009		2	IM-19.0A	2.0
1989		1	S-5	0.6
1989		2	S-5	1.2
1987		1	S-5	1.3
1980		1	Surface Treatment	0.0
1967		1	S-5	0.0

BIT = AC over AC pavement; AC = asphalt concrete.

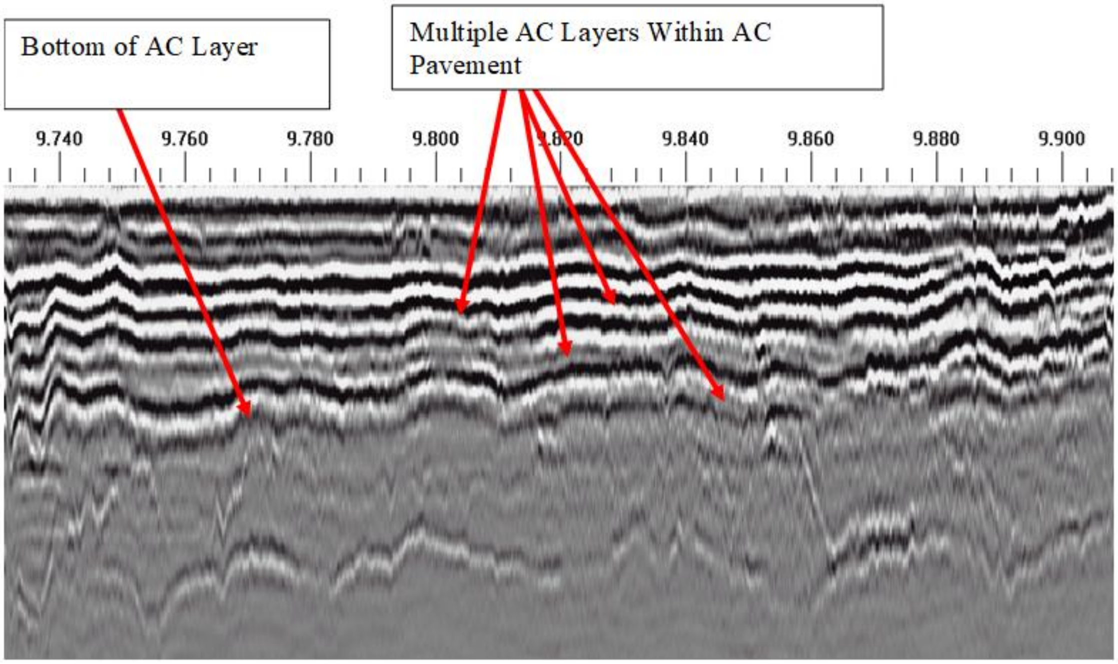

Figure 7. GPR Image of Rte. 250. GPR = ground penetrating radar.

Asphalt Pavement Rutting Calibration

The first model considered in the local calibration was the predicted rutting on asphalt pavements. Equation 1 shows the rutting model used in Pavement ME Design. The coefficients include K-values, which represent properties or values derived from laboratory testing, and β-values, which represent field shift values intended to remove the bias between predicted and measured distresses. A comparison of measured and predicted values of total rutting when the VDOT local calibration coefficients were used in Pavement ME Design is shown in Figure 8. Rutting prediction with VDOT current calibration values (for new design) showed a bias, with the Pavement ME Design models predicting less rutting than was measured in the field (0.03 in on average). However, it should be noted that Figure 8 compares only 8 to 9 years of the measured and predicted data. In general, measured rut depth was less than 0.2 in. There can be several reasons for this bias. Delamination in existing AC layers can result in some of the residual error or bias in the total rut depth. Moisture damage in layers will also result in bias. For the calibration effort, full friction was assumed in all layers. Some of the deeper rutting measured (>0.2 in) came from three projects with SMA mixture, as shown in Figure 9.

$$Asphalt\ rutting = \beta_r k_z 10^{k_1} T^{k_2 \beta_2} n^{k_3 \beta_3} \qquad \text{[Eq. 1]}$$

where

n = number of axle load repetitions
T = temperature in the asphalt sublayer, ºF
kz = depth correction factor
k_1, k_2, k_3 = laboratory-determined permanent deformation coefficients
β_1, β_2, β_3 = local calibration coefficients.

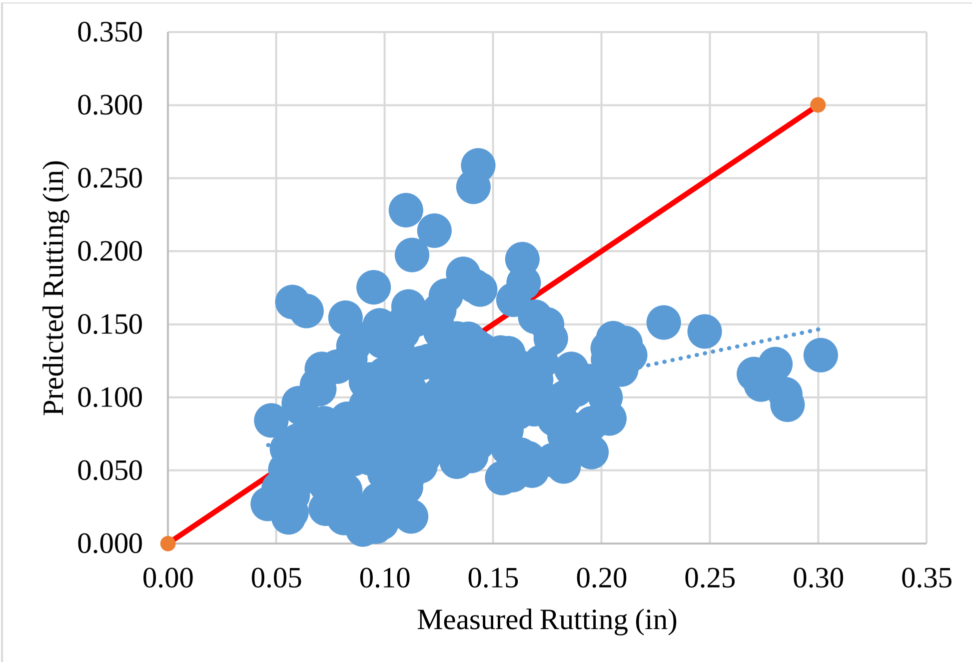

Figure 8. Asphalt Pavement Rutting Comparison With VDOT Calibration Coefficients

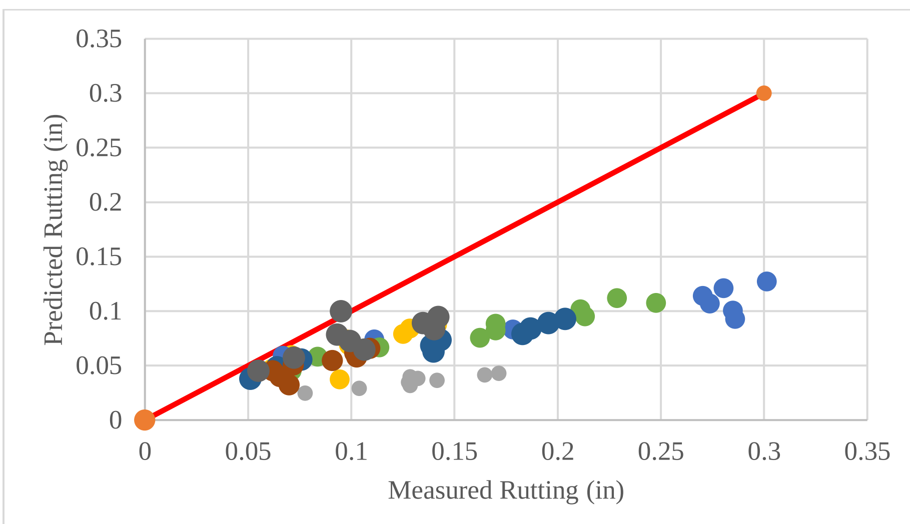

Figure 9. Rutting of Projects With SMA Mixtures. SMA = stone matrix asphalt.

Table 12 shows calibration statistics with current VDOT coefficients. The standard error of the estimate was acceptable as per the local calibration guide. The AASHTO local calibration guide suggests changing the β_1 value to remove the bias. Adjusting the β_1 value to 1.12 (instead of 0.687) will remove the bias and further reduce the standard error, as shown in Table 12. However, an earlier study conducted at VTRC (Nair and Saha, 2021) showed that this is not the right approach and VDOT needs to adjust both β_1 and β_3 values to match the measured and predicted values.

Table 12. Rutting Calibration Results

Name	Using VDOT Current Values	Local Calibration
Count	236	236
Bias, in	0.03	0.00
Se, in	0.06	0.04
R^2, %	11.4	34.1
p-value (paired t-test)	1.72E-18	0.0029
Regression slope	0.31	0.55
p-value (slope)	1.29E-26	7.68E-16
Regression intercept	0.05	0.04
p-value (intercept)	3.35E-11	6.41E-11
Se/Sy	1.31	0.90
βr1	0.687	1.12
βs1–fine subgrade	0.153	0.153
βs1–granular subgrade	0.153	0.153

As mentioned previously, VDOT currently uses V2.2.6 of Pavement ME Design. The latest available version of the software is V2.6. In V2.6, the laboratory and field coefficient values were separated out (the global values for β_1, β_2, and β_3 were 0.40, 0.52, and 1.36, respectively). A major observation from the earlier study (Nair and Saha, 2021) was that V2.2.6 uses a high coefficient for k_3* β_3, which will predict higher rutting as truck traffic increases. An explanation for this finding was that the k_3 global value was originally derived from unconfined repeated load plastic deformation tests. For NCHRP Project 9-30A, Von Quintus et al. (2012) recommended use of confined repeated load plastic deformation tests. The k_3 value derived from confined repeated load tests was included in the latest version of Pavement ME Design, including V2.6. VTRC laboratory testing showed a similar k_3* β_3 when compared to V2.6 of Pavement ME Design. However, it will still be necessary to calibrate/validate these coefficients if and when VDOT adopts a newer version.

VDOT uses a rutting criterion of 0.26 in for 15 years. Even with the current rutting coefficient, the interstate sections shown in Table 13 predicted higher rutting than 0.26 in (10 of 13 interstate projects). Adjusting just k_1* β_1 to remove bias (without changing k_3* β_3) will further increase the rutting prediction. Therefore, it is recommended that both k_1* β_1 and k_3* β_3 be calibrated for new design first and then that the same coefficients be applied to rehabilitation design. As noted previously, PMS data do not show rutting as a big concern for pavements. If VDOT chooses to implement Pavement ME Design V2.2.6 for rehabilitation design, careful consideration should be given to rutting prediction by either using a lower reliability level or adjusting the threshold criteria to avoid an unnecessary increase in AC thickness as part of the rehabilitation design. A detailed sensitivity analysis is needed to determine where the adjustments should be made for successful implementation with V.2.2.6. The sensitivity analysis for this task was outside the scope of this study.

Table 13. Rutting Prediction for Design Year 15 With Current Rutting Coefficients

Project	Cumulative Truck Traffic	Total Pavement Permanent Deformation (in)
Br-1	26,129,100	0.27
Br-11	29,007,800	0.29
Cul-1	10,359,900	0.27
I-77 Carroll County	27,173,500	0.27
Nova-III	9,807,460	0.29
Rich-III	17,624,900	0.36
Richmond-IV	21,579,400	0.36
Rich-VI	33,449,100	0.48
Rich-VII	15,307,100	0.28
STN-VI	33,887,800	0.32

Asphalt Pavement Bottom-up Fatigue Cracking Calibration

Figure 10 shows measured and predicted bottom-up fatigue cracking (from 8 to 9 years of performance data). Higher cracking (>10%) was observed in only one project. For most of the projects, measured cracking was less than 2%. Table 14 shows the calibration statistics, which showed bias. As mentioned earlier, debonding and pavement distress such as moisture damage are not accounted for in the calibration process and will increase the cracking prediction if considered. In general, projects with such features should not be included in the calibration study. Because of this, national experts suggest that rutting and cracking calibration should first be conducted using performance data from new pavement construction and then that the same coefficients be applied to rehabilitation design.

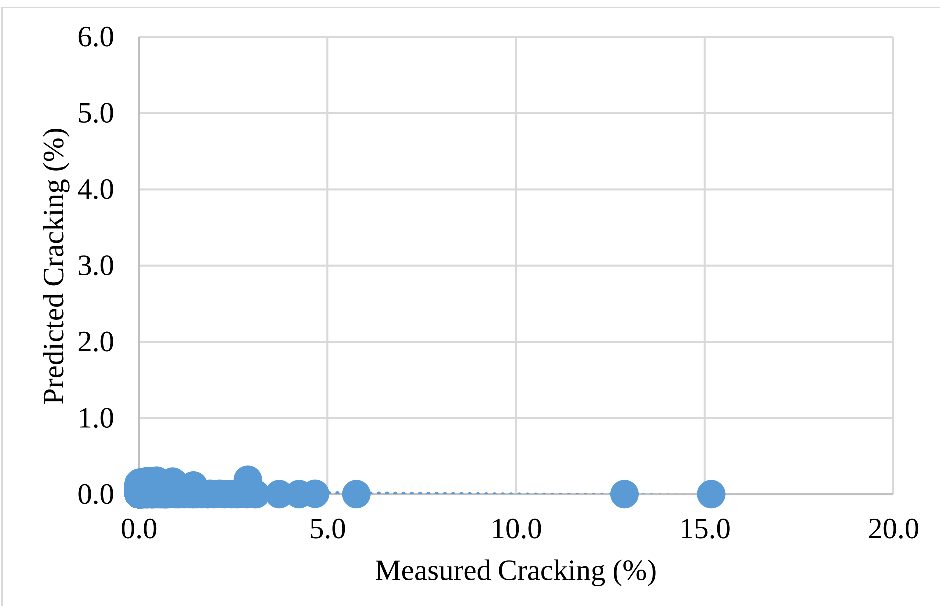

Figure 10. Predicted vs. Measured Bottom-Up Cracking With Current Coefficients

Table 14. Cracking Calibration With Current Coefficients

Count	236
Bias, in	0.48
Se, in	1.59
R^2, %	0.011
p-value (paired t-test)	2.20E-06
Regression slope	0.00
p-value (slope)	0
Regression intercept	0.028
p-value (intercept)	1.07E-17
Se/Sy	1.04

Figure 11 shows the bottom-up fatigue cracking results in comparison with results of new construction calibration from a previous study (Smith and Nair, 2015). Figure 11 shows, in general, that there is some improvement needed for bottom-up fatigue calibration. As mentioned previously, Level 2 and Level 3 severity level data from the PMS were used for bottom-up fatigue calibration. Bias in the data may also be due to this assumption. With a greater AC thickness (>10 in), some of the cracks may be top-down cracking. In rehabilitation projects, some of the cracks may also be reflective cracking. Should VDOT move to Pavement ME Design V2.6, it might be a good idea to repeat the calibration exercise for new construction using the data from the previous project and adding additional sites from the past 10 years. Such a study should also include some forensic investigation to identify top-down vs. bottom-up fatigue cracking. Further, AASHTO provides a calibration assistance tool (which is compatible with V2.6) to help agencies conduct local calibrations of the pavement ME performance models. The tool is being developed in accordance with the 11-step procedure given in AASHTO's *Local Calibration Guide* (AASHTO, 2010) and offers the advantage of quick calibration using the latest versions of Pavement ME Design (V2.5 and V2.6).

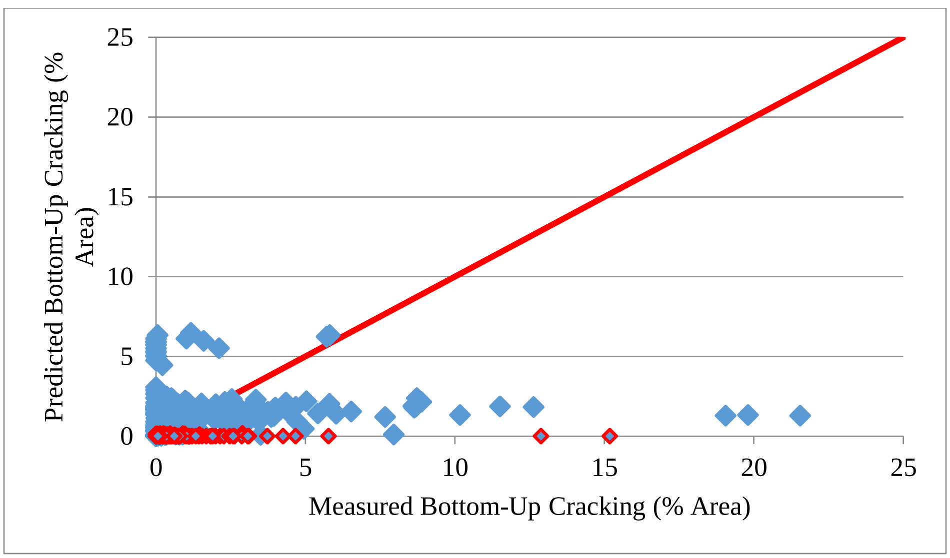

Figure 11. Comparison of Measured vs. Predicted Data. Data in blue are for new construction and are from a previous study (Smith and Nair, 2015); data in red are from the current study.

It should also be noted that V2.6 was globally recalibrated, and new calibration coefficients are shown in Table 15 along with VDOT values (from V.2.2.6). VDOT would be required to calibrate these coefficients when moving to V2.6, so it is important to understand the impact of these coefficients in the calibration. A decrease in K_{f1}* β_{f1} will reduce the fatigue life of mixtures and hence cracking prediction will increase.

The fatigue equation used in Pavement ME Design is shown in Equation 2. However, it should be noted that a few other coefficients also changed. Further, it is anticipated that changes in calibration will be required by adjusting β_{f1}, C_1 and C_2 coefficients. A base mixture project is underway at VTRC in which it is planned to conduct beam fatigue testing to develop K_f factors that can be used to compare the K-factor coefficients of V2.6.

$$N_{f-HMA} = k_{f1}(C)(C_H)\beta_{f1}(1/\varepsilon_t)^{k_{f2}\beta_{f2}}(1/E_{HMA})^{k_{f3}\beta_{f3}} \quad \text{[Eq. 2]}$$

The fracture mechanics–based cracking model was developed for top-down cracking under NCHRP Project 1-52 and added to V2.6 of Pavement ME Design (AASHTO, 2020). Top-down cracking can be considered another criterion if VDOT moves to V2.6.

Figure 12 shows measured and predicted IRI. A good comparison was not obtained. As mentioned previously, because of a limited range of before and after IRI data for VDOT pavements (Table 8), IRI cannot be used as a design criterion.

Figure 13 shows a comparison of predicted bottom-up cracking + reflective cracking and measured bottom-up cracking (Level 2 and 3 severity), which shows that this model (bottom-up + reflective) needs further assessment and calibration. Tables 16 and 17 give comparisons of bottom-up + reflective prediction and bottom-up cracking and total cracking from the PMS for mill and fill and straight overlay application, respectively. It can be seen from the tables that the software predicts very high early reflective cracking compared to measured values. Further calibration of this model will be required if V2.6 is adopted.

Table 15. Bottom-Up Alligator/Fatigue Cracking Coefficient Comparison Between V2.6 and VDOT's Current Version (V2.2.6) With Local Calibration

Fatigue Coefficient	VDOT Current Values	V2.6
K_{f1}, intercept	0.007566	3.75
K_{f3}, E exponent	1.281	1.46
K_{f2}, Strain exponent	3.95	2.87
β_{f1}	42.87 (from local calibration)	Thickness Dependent
β_{f2}	1.0	0.88
β_{f3}	1.0	1.38
C_1	0.319	1.31
C_2	0.319	Thickness Dependent

C_2 is AC thickness dependent in V2.6. Less than 5 in: C_2 = 2.1585; 5 to 12 in: C_2 = 0.867+ 0.2583 (h_{AC}); greater than 12 in: C_2 = 3.9666.
β_{f1} is AC thickness dependent in V2.6. Less than 5 in: β_{f1}= 0.02054: 5 to 12 in: β_{f1}= 5.014 $(h_{AC})^{-3.416}$; greater than 12 in: β_{f1}= 0.001032.

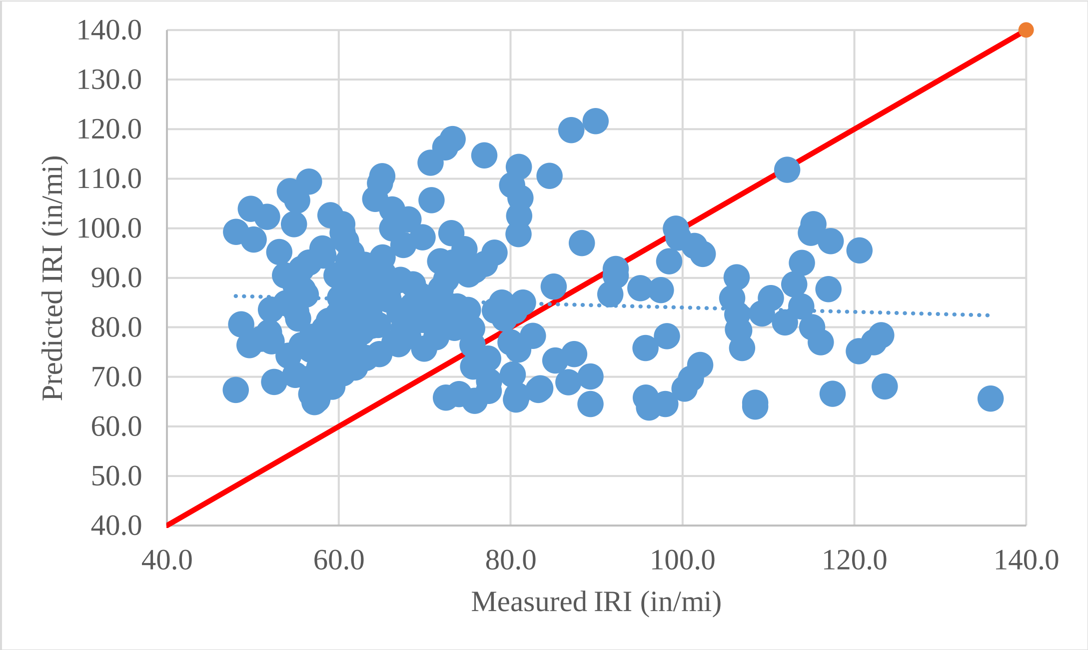

Figure 12. Measured and Predicted IRI Comparison. IRI = International Roughness Index.

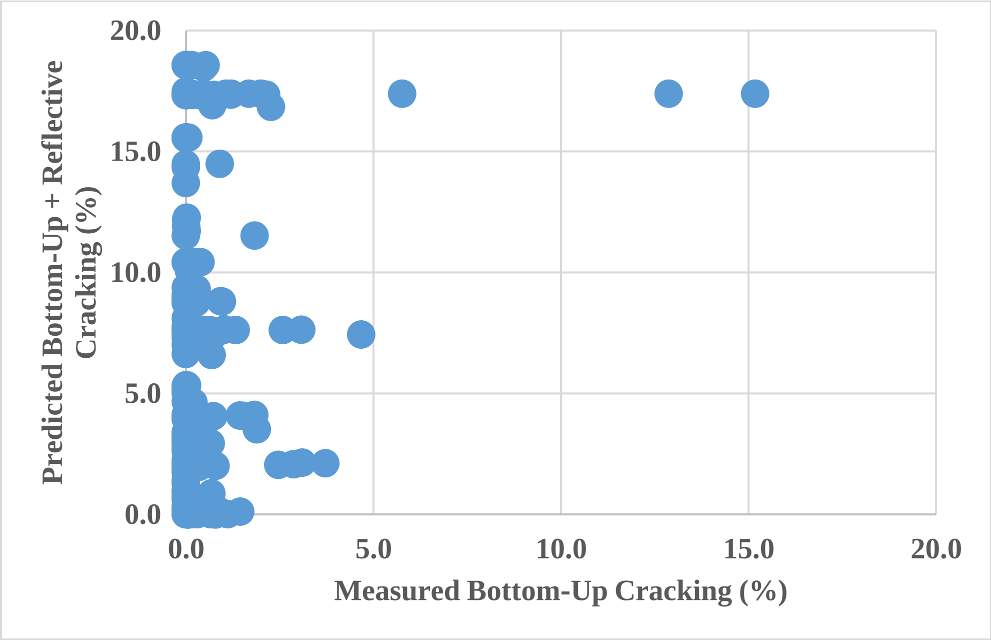

Figure 13. Comparison of Bottom-Up + Reflective Prediction vs. Measured Bottom-Up Cracking

Table 16. Comparison of Bottom-Up + Reflective Prediction vs. Measured Cracking (Mill and Fill)

Project ID	Pavement Age (years)	AC Total Fatigue Cracking: Bottom-Up + Reflective (% lane area)	AC Bottom-Up Fatigue Cracking (% lane area)	PMS Fatigue Sev-II and III, (% lane area)	PMS Fatigue Sev-I, II, and III (% lane area)
Br-I	1	10.41	0.00	0.04	0.09
	2	10.42	0.00	0.00	0.00
	3	10.43	0.00	0.00	0.01
	4	10.43	0.00	0.00	0.00
	5	10.44	0.00	0.00	0.00
	6	10.44	0.00	0.04	0.70
	7	10.45	0.00	0.20	3.17
	8	10.46	0.00	0.11	2.56
	9	10.47	0.00	0.39	3.87

AC = asphalt concrete; PMS = Pavement Management System.

Table 17. Comparison of Bottom-Up + Reflective Prediction vs. Measured Cracking (Straight Overlay)

Project ID	Pavement Age (years)	AC Total Fatigue Cracking: Bottom-Up + Reflective (% lane area)	AC Bottom-Up Fatigue Cracking (% lane area)	PMS Fatigue Sev-II and III (% lane area)	PMS Fatigue Sev-I, II, and III (% lane area)
Br-II	1	17.33	0.00	0.74	0.83
	2	17.33	0.00	0.34	0.41
	3	17.34	0.00	0.00	0.00
	4	17.34	0.00	0.00	0.00
	5	17.34	0.00	0.00	0.36
	6	17.34	0.00	0.02	1.16
	7	17.35	0.00	0.35	5.54
	8	17.35	0.00	0.06	3.71
	9	17.35	0.00	1.20	7.60
	10	17.36	0.00	2.14	8.21

AC = asphalt concrete; PMS = Pavement Management System.

In general, for AC over AC overlay rehabilitation design, V2.2.6 has an important limitation in design criteria and may require some changes in threshold criteria (e.g., for rutting), as well as more emphasis on pavement field investigation (Level 1 inputs for existing pavements). Further sensitivity analysis and an interim approach (e.g., shadow designs using AASHTO 1993 design) may also be necessary for V2.2.6 rehabilitation implementation with current coefficients. This sensitivity analysis was outside the scope of this study.

AC Over CRCP

Figure 14 shows a comparison of measured and predicted rutting values for AC over CRCP. Considerable bias was observed when 8 to 9 years of measured and predicted values were compared. Most of the SMs used were SMA. Figures 15 and 16 show that one-half of the SMA mixtures showed good performance (<0.15 in) in terms of rutting and one-half showed higher rutting (>0.2 in). As mentioned previously, rutting calibration requires adjusting both β_1 and β_3 values to match the measured and predicted values.

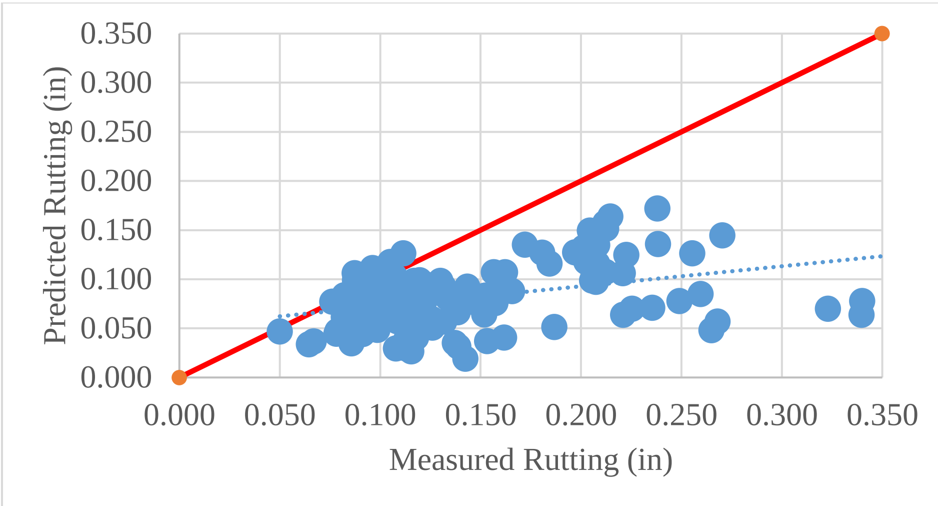

Figure 14. Measured vs. Predicted Rutting Comparison for AC Over CRCP Projects. AC = asphalt concrete; CRCP = continuously reinforced concrete pavement.

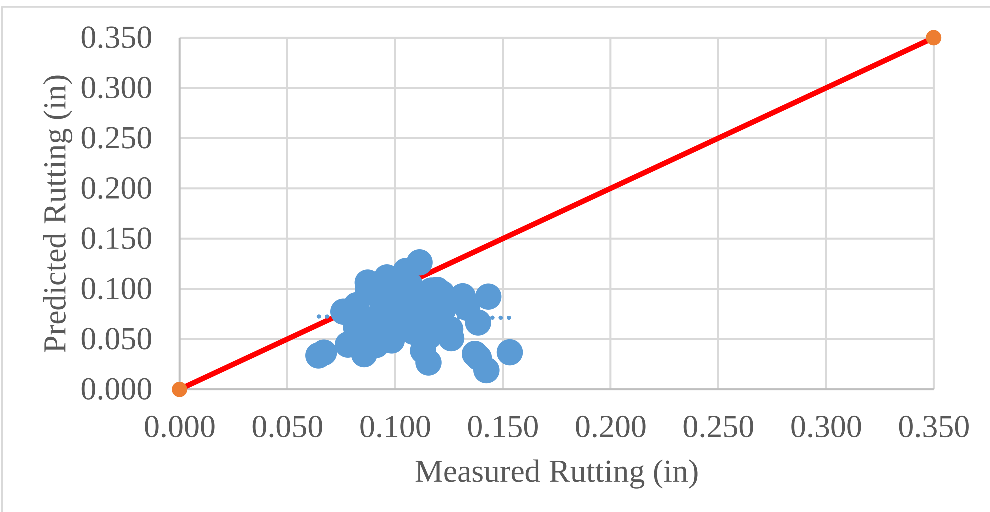

Figure 15. Measured vs. Predicted Rutting Comparison for SMA Over CRCP Projects. Measured rutting <0.2 in. SMA = stone matrix asphalt; CRCP = continuously reinforced concrete pavement.

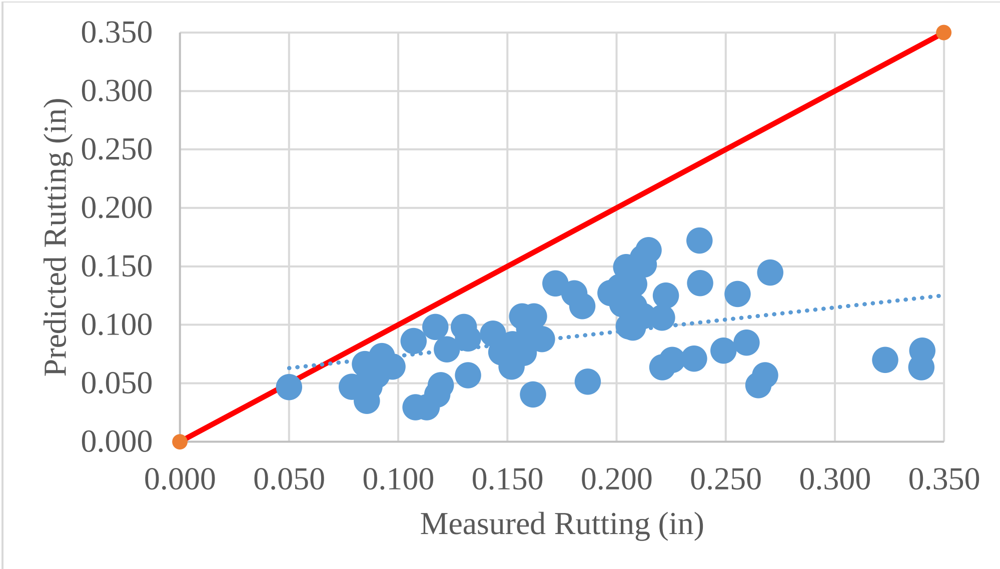

Figure 16. Measured vs. Predicted Rutting Comparison for SMA Over CRCP Projects. Measured rutting > 0.2 in. SMA = stone matrix asphalt; CRCP = continuously reinforced concrete pavement.

Figures 17 through 19 show comparisons of measured and predicted distress for bottom-up cracking, reflective + transverse cracking, and IRI. Except for IRI, all the other distresses showed good correlation with minimum bias. Other than for rutting, Pavement ME Design V2.2.6 with current coefficients can be used to predict bottom-up cracking and reflective + transverse cracking.

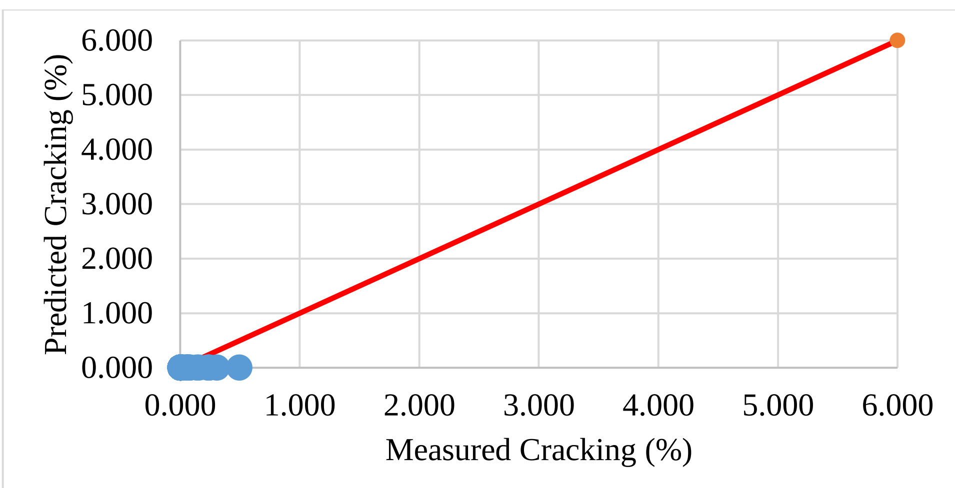

Figure 17. Comparison of Measured and Predicted Bottom-Up Cracking

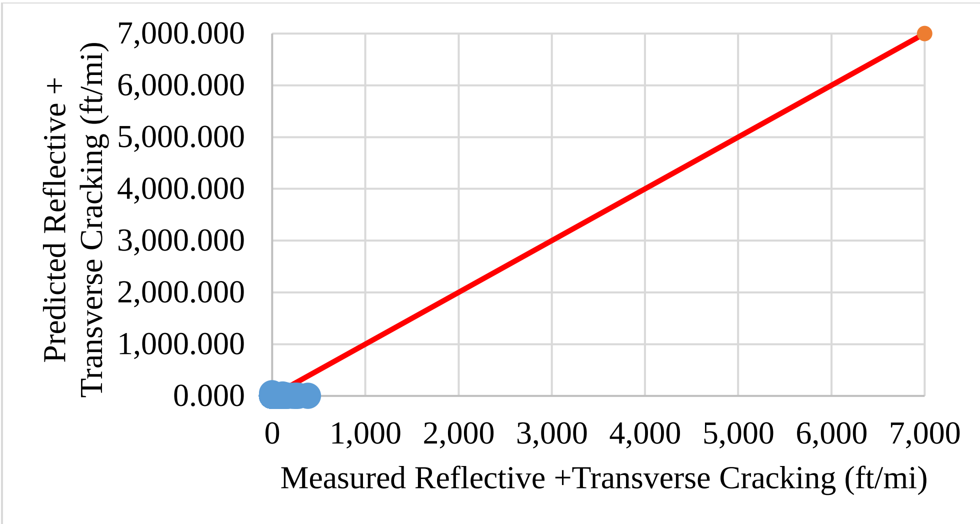

Figure 18. Comparison of Measured and Predicted Reflective + Transverse Bottom-Up Cracking

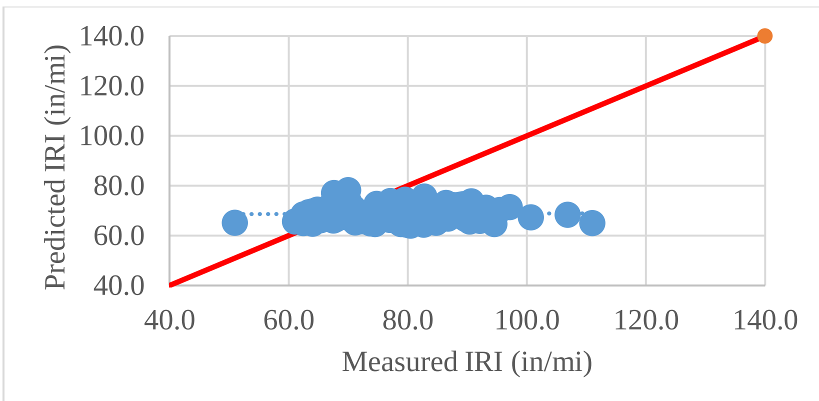

Figure 19. Comparison of Measured and Predicted IRI. IRI = International Roughness Index.

AC Over JPCP

Figure 20 shows a comparison of measured and predicted rutting values for AC over JPCP (majority of the pavements were old JRCP). Considerable bias was observed when 8 to 9 years of measured and predicted values were compared. SMs used included SMA and dense-graded SM E mixtures. Figures 21 and 22 show that the comparison of SMA and SM E mixtures and some of the SMA mixtures showed higher rutting (>0.2 in). Rutting calibration requires adjusting both β_1 and β_3 values for AC over JPCP to match the measured and predicted values. Figures 23 through 25 show a comparison of measured and predicted distress for bottom-up cracking, reflective + transverse cracking, and IRI. Bottom-up cracking showed a good correlation with minimum bias. The IRI model did not show a good correlation. As shown in Figure 18, reflective + transverse cracking showed bias, which may have been attributable to a difference in LTE. It should be noted that 50% LTE was used for all projects since this value was not available.

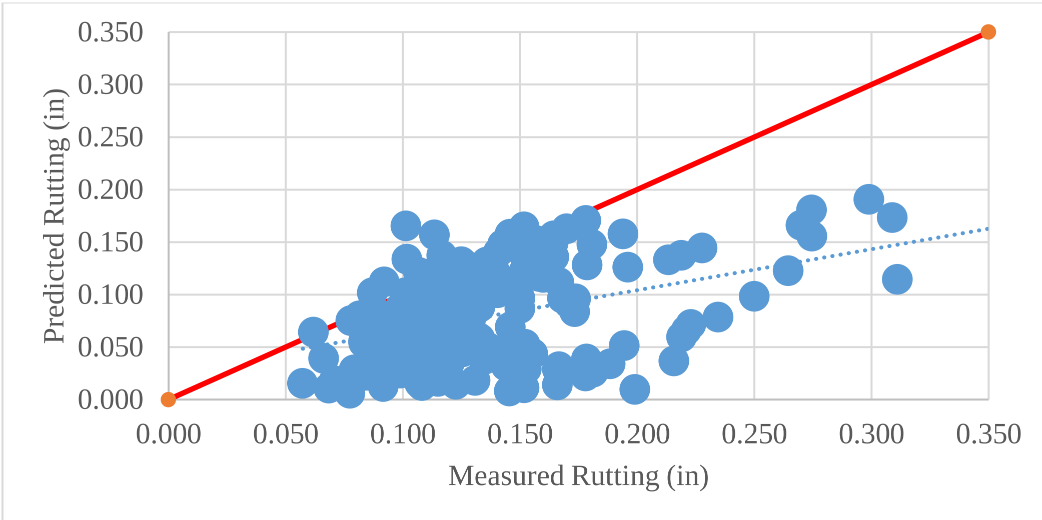

Figure 20. Measured vs. Predicted Rutting Comparison for AC Over JCP Projects. AC = asphalt concrete; JCP = jointed concrete pavements.

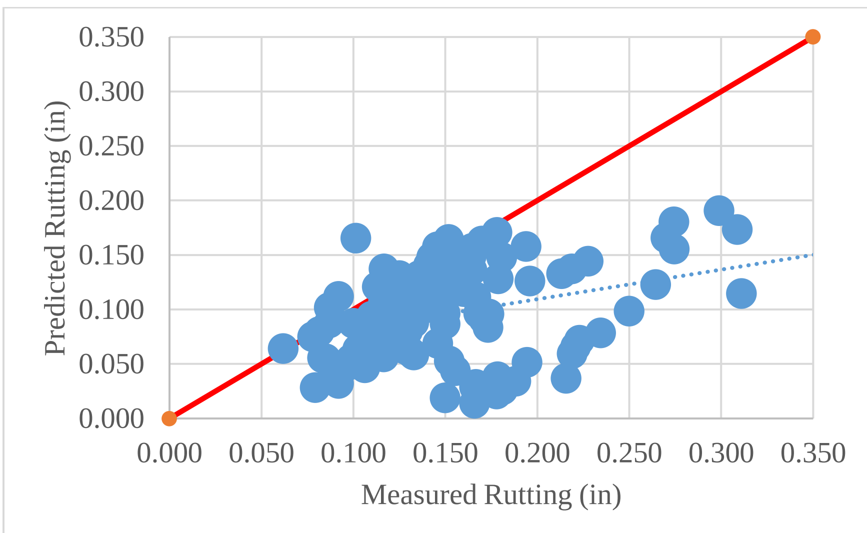

Figure 21. Comparison of Measured vs. Predicted Rutting for SMA Over JCP Projects. SMA = stone matrix asphalt; JCP = jointed concrete pavements.

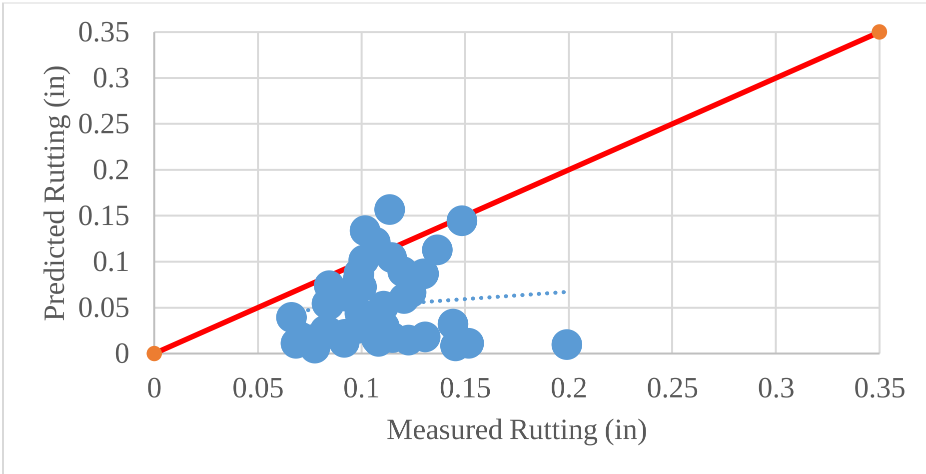

Figure 22. Comparison of Measured vs. Predicted Rutting for SM E Over JCP Projects. SM E = polymer modified surface mixture; JCP = jointed concrete pavements.

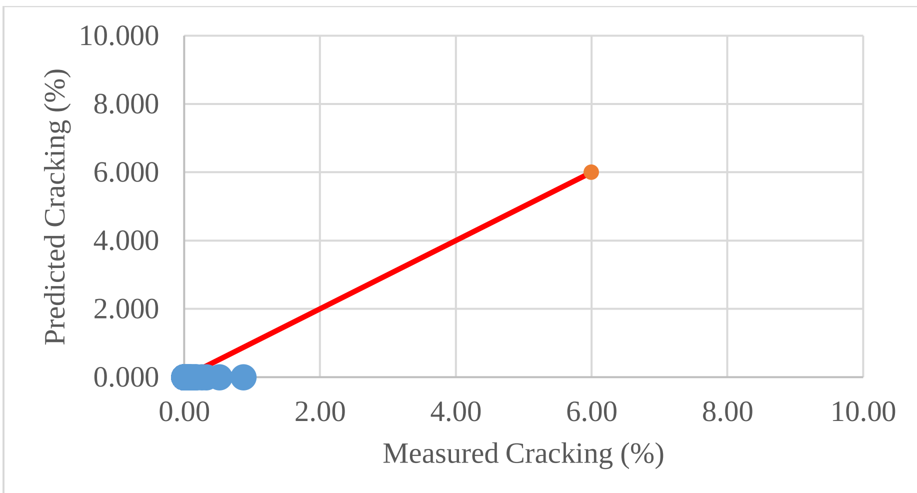

Figure 23. Comparison of Measured and Predicted Bottom-Up Cracking

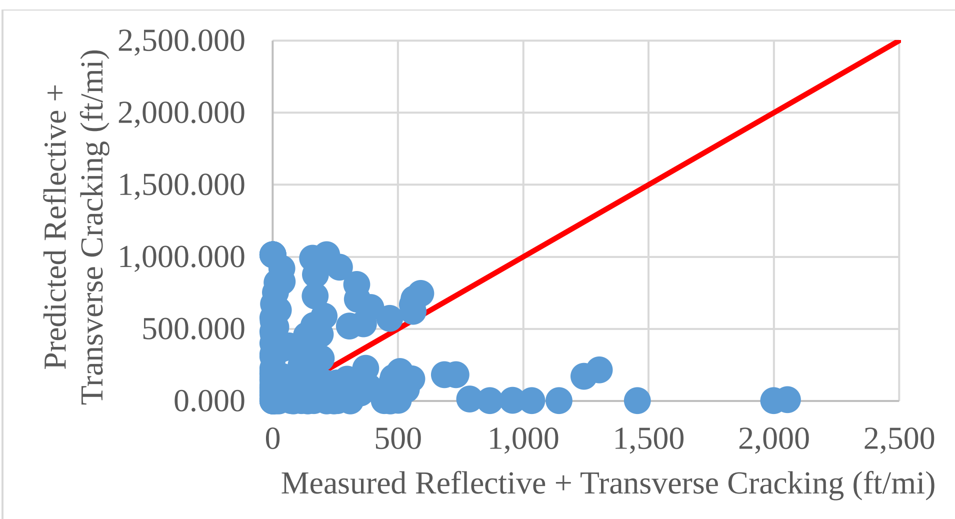

Figure 24. Comparison of Measured and Predicted Reflective + Transverse Bottom-Up Cracking

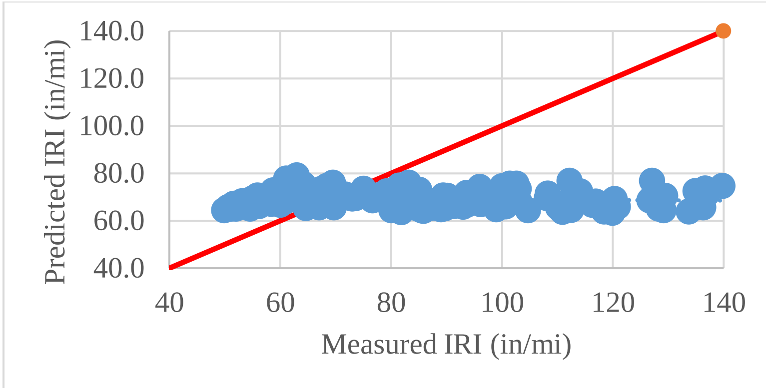

Figure 25. Comparison of Measured and Predicted IRI. IRI = International Roughness Index.

Some of the outlier data that showed very higher cracking compared to others were removed from the analysis. However, most measured values were less than the AASHTO pavement ME default recommended criterion of 2,500 ft/mi. It is recommended that this model be calibrated with only V2.6. Further, it is recommended that reflection cracking mitigation for asphalt overlay over JCP be considered outside Pavement ME Design by means such as binder modification (high polymer binder, ground tire rubber, etc.) and asphalt mixture modification (use of fibers), use of paving fabrics, saw and sealing of the HMA overlay, or use of in-place recycling techniques. Researchers at VTRC have completed several studies in this area and are working further to document comparisons of different methods. Other than rutting, V2.2.6 with current coefficients can be used to predict bottom-up cracking and reflective + transverse cracking.

Need for Separate Threshold Values for New and Rehabilitation Design

It is recommended that current threshold criteria be continued for AC over AC (0.26 in for rutting at 15 years and 6% cracking for 15 years); AC over CRCP; and AC over JPCP sections. It should be noted that Pavement ME Design uses 50% reliability for bottom-up fatigue cracking for AC over AC rehabilitation design, a setting that is fixed in the software. The software default of 2,500 ft/mi can work for transverse + reflective cracking criteria for AC over CRCP and AC over JPCP. Further VDOT internal discussion with pavement experts is needed to refine the threshold criteria.

Modeling Approaches for Composite Pavement or Multiple Overlays of PCC Pavements (AC Over Existing CRCP, JPCP, and JRCP)

Several existing composite pavements in this study were modeled as AC over CRCP and AC over JPCP. Some guidelines are provided in the AASHTO ME Design FY21 Webinar Series (AASHTO, 2021). Figure 26 shows an example of two AC layers over JPCP. The bottom AC layer can be considered an existing layer and field-measured properties (in-place air voids and binder content) can be entered for that layer.

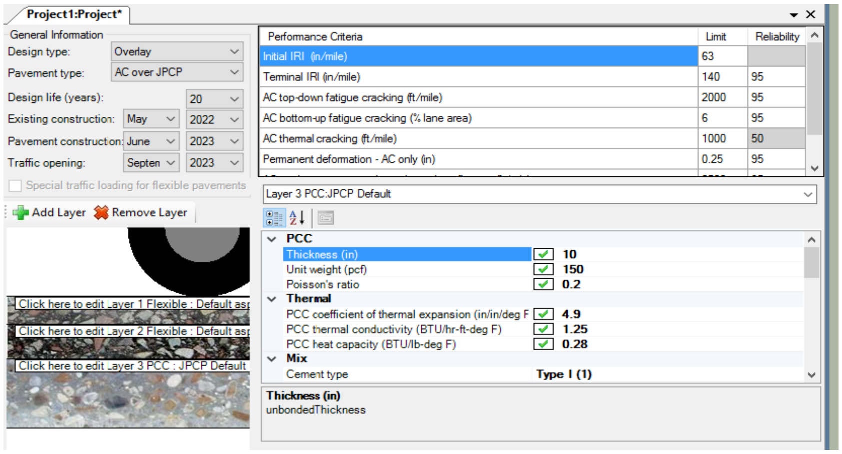

Figure 26. Pavement ME Design Simulation Showing 2 AC Layers Over JCP. AC = asphalt concrete; JCP = jointed concrete pavements/

Summary of Findings

- The first step in the pavement rehabilitation process involves assessing the overall condition of the existing pavement. The condition of the existing pavement has a major effect on the development of damage in the new AC layers.

- The pavement structural evaluation for determining the condition of the existing pavement layers can include visual distress surveys, coring, deflection tests, and other field and laboratory tests.

- Pavement ME Design allows the designer to use different input levels (Levels 1, 2, and 3) for existing AC layers based on the importance of the project and available resources.

- For existing AC layers, rehabilitation input Level 1 analysis requires backcalculated layer moduli values for each existing pavement layer determined using FWD testing. The backcalculated modulus from the deflection basin is used to calculate the damage in the existing layers. Previous research showed the importance of using Level 1 inputs for existing pavement damage determination.

- For existing AC layers, Level 2 inputs require measured distress data for fatigue cracking (%), transverse cracking (ft/mi), and rutting in the existing pavement layers. The fatigue cracking (%) defines the level of damage for the existing layers.

- Information on many of the factors related to the existing pavement condition (e.g., fatigue cracking, transverse cracking, and rutting) can be obtained from the PMS.

- Results from an example project clearly showed that Level 1 input (backcalculated modulus of AC) predicted more distress compared to Level 2. This was due to more damage calculation of the asphalt layer when backcalculated modulus was used compared to damage calculated based on fatigue cracking (Level 2 input).

- Layer interface friction is an input parameter for Pavement ME Design, and cores and visual surveys can be used to determine if debonding exists along the project.

- The thickness of the AC layer from cores and from GPR testing was comparable. GPR analysis showed a potential indication of delamination within the AC layers, and this was confirmed with coring.

- For AC over AC rehabilitation projects, rutting prediction using VDOT current calibration values (for new design) showed a bias.

- Some SMA mixtures showed higher rutting for AC over CRCP and AC over JPCP.

- For AC over JPCP, bottom-up cracking showed a good correlation, with minimum bias. The IRI model did not show a good correlation. Reflective + transverse cracking showed bias, which may have been attributable to a difference in LTE.

CONCLUSIONS

- *An assessment of the overall condition of the existing pavement is an important part of rehabilitation design as it identifies damage (moisture damage, layer debonding, etc.) and helps determine the depth of any milling that may be needed. The pavement structural evaluation for determining the condition of the existing pavement layers can include visual distress surveys, coring, deflection tests, and other field and laboratory tests. At a minimum, coring is required to assess the damage in existing pavement. GPR can also be used as an important tool in forensic investigation of asphalt pavements.*

- *Level 1 and Level 2 inputs for existing pavement can predict damage in the existing layers differently. An included case study showed that Level 1 input (backcalculated modulus of AC) predicted more distress than Level 2 input.*

- *IRI cannot be used as a design criterion since VDOT pavements have such a limited range of before- and after-rehabilitation IRI values.*

- *Pavement ME Design models for bottom-up + reflective cracking need further assessment. The software predicts very high early reflective cracking compared to measured values. Further calibration of this model will be required if V2.6 is adopted.*

- *In general, for AC over AC overlay rehabilitation implementation, the current version of Pavement ME Design (V2.2.6) has limited design criteria and requires calibration for rutting distress or changes in threshold values and reliability level, as well as more emphasis on pavement field investigation (i.e., Level 1 inputs for existing pavements). Further sensitivity analysis and an interim approach (e.g., shadow designs using AASHTO 93 design method) may also be necessary for V2.2.6 rehabilitation implementation with current coefficients.*

- *With rutting as an exception, V2.2.6 with current coefficients can be used for bottom-up cracking and reflective + transverse cracking prediction for AC over CRCP.*

- *With rutting as an exception, V2.2.6 with current coefficients can be used for bottom-up cracking and reflective + transverse cracking prediction for AC over JPCP.*

- *Implementation of V2.2.6 with the current calibration coefficient for AC over AC projects presents some challenges and will require a few additional steps. Adoption of V2.6, which incorporates additional design criteria (e.g., top-down cracking), may be a preferred option for both new and rehabilitation design.*

RECOMMENDATIONS

1. *VDOT's Materials Division and Maintenance Division should consider promoting detailed forensic evaluation as part of rehabilitation design for restorative maintenance projects.*

2. *VDOT's Materials Division should consider implementation of the current Pavement ME Design V2.2.6 for AC over AC rehabilitation projects only after a detailed sensitivity analysis with regard to various distresses using current calibration coefficients.*

3. *VDOT's Materials Division should consider implementation of the current V2.2.6 for AC over CRCP and AC over JPCP sections.* However, a detailed sensitivity analysis and evaluation of threshold criteria and/or local calibration for rutting distress is still needed before implementation.

4. *VDOT's Materials Division should consider adopting V2.6 of Pavement ME Design for new and rehabilitation design.* Calibration/validation will still be needed for V2.6 before adoption.

IMPLEMENTATION AND BENEFITS

Researchers and the technical review panel (listed in the Acknowledgments) for the project collaborate to craft a plan to implement the study recommendations and to determine the benefits of doing so. This 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 Recommendation 1, VDOT's Materials Division and Maintenance Division will promote the need for detailed pavement evaluation. The topic will be highlighted in VDOT's pavement forums and district materials engineer meetings through the spring of 2022.

Regarding Recommendations 2 and 3, VDOT'S Materials Division will lead a detailed sensitivity analysis to inform further its decision on whether to move forward with V2.2.6 for rehabilitation design. A decision is anticipated by December 2022. The following steps should be considered for the sensitivity analysis/implementation:

1. *Conduct additional comparisons of the impacts on design of using Level 1 versus Level 2 inputs.* Since FWD testing is not performed on all rehabilitation projects (and thus Level 1 inputs may not be feasible), VDOT should continue to explore characterization of the existing pavement using Level 2 inputs. Sensitivity analyses of different fatigue cracking percentages (as Level 2 input) on damage prediction of existing pavement and, further, their impact on final design thickness should be conducted. A separate task comparing Level 1 and 2 inputs for a few additional rehabilitation projects will further help in developing guidelines for future use. Guidelines from an earlier study (Ayyala et al., 2018) gave a detailed framework for this study.

2. *Revisit prediction of rutting distresses.* As noted previously, PMS data do not show rutting as a big concern for Virginia pavements. To avoid an unnecessary increase in AC thickness using Pavement ME Design for rehabilitation design, careful consideration should be given to rutting prediction either using a lower reliability level or adjusting the threshold criteria when using current calibration coefficients. Further, sensitivity analyses of current rutting calibration coefficients on AC rehabilitation thickness should be conducted for different truck traffic levels. As mentioned previously, it is recommended that both $k_1* \beta_1$ and $k_3* \beta_3$ be calibrated for new design first and then the same coefficients be applied to rehabilitation design.

3. *Accommodate impact of layer debonding.* Sensitivity analysis is required for determining the impact of layer debonding (no bond or a low interface friction**)** on rutting and bottom-up cracking prediction for AC over AC pavements (with different existing AC thickness and traffic levels).

4. *Prepare a Pavement ME Design user manual for rehabilitation design*. Appendix A along with information from AASHTO's *MEPDG Manual of Practice* (AASHTO, 2020) can be used for this purpose.

5. *Develop training for pavement design staff.* This training should cover extracting data from the PMS for Level 1 and 2 inputs and analyzing FWD data with the BcT tool.

6. *Compensate for limited design criteria.* In general, for AC over AC overlay rehabilitation using V2.2.6, design criteria are limited to rutting and bottom-up fatigue cracking. A sensitivity analysis that determines the impact of limited design criteria on thickness design is therefore needed.

Regarding Recommendation 4, VDOT's Materials Division will deliberate internally on the warrants for adopting V.2.6 for new and rehabilitation design. A decision is anticipated by December 2022.

Benefits

Regarding Recommendation 1, assessing the overall condition of the existing pavement is an important step in rehabilitation design. From this study using GPR and coring, it was found that several pavements had underlying issues such as debonding, moisture damage, etc. A detailed evaluation of the existing pavement will promote use of more appropriate rehabilitation techniques, which can include recycling techniques such as full-depth reclamation, cold-in-place recycling, etc. Further, every year, VDOT conducts restorative maintenance of more than 150 lane-miles of existing pavement and the use of the correct rehabilitation technique based on a pavement investigation will extend service life.

Regarding Recommendations 2 and 3, implementation of rehabilitation design with VDOT's current version (V2.2.6) needed several additional steps for successful implementation.

Regarding Recommendation 4, VDOT currently uses V2.2.6 of Pavement ME Design. Multiple updates have been made to the software, and the latest version available is V2.6. Updates for V2.6 include (1) integration of AASHTO's *MEDPG Manual of Practice;* (2) globally recalibrated flexible and semi-rigid performance models for both new and rehabilitated pavements; (3) a maintenance strategy tool that allows the user to incorporate a single future preventive maintenance treatment; (4) inclusion of the top-down asphalt pavement cracking model; and (5) a semi-automated calibration tool, which will allow users to calibrate the Pavement ME Design models. Further, several additional distress criteria can be used for the successful implementation of rehabilitation design.

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APPENDIX A
Inputs for Pavement ME Design

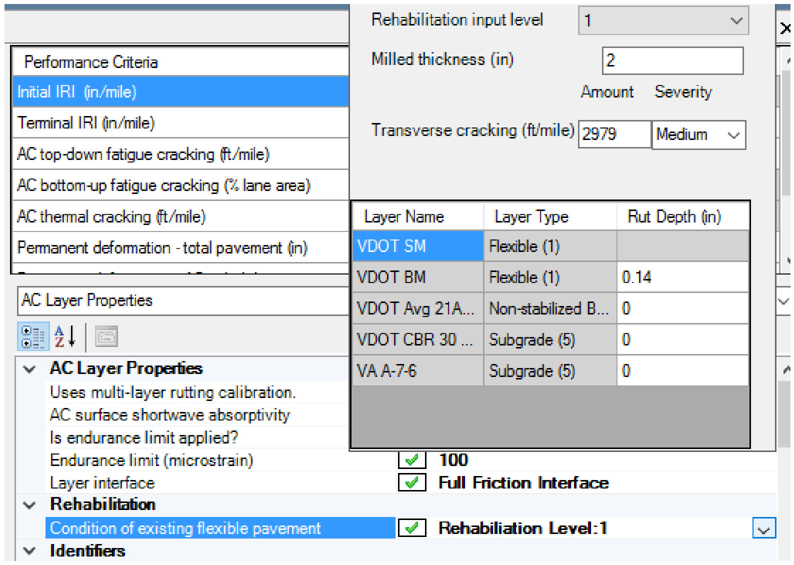


Figure A1. Level 1 Rehabilitation Input in Pavement ME Design

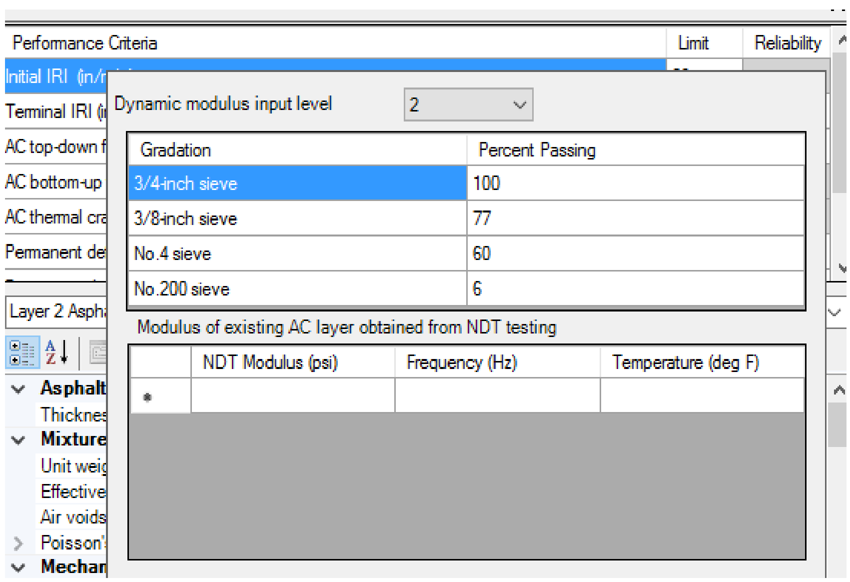


Figure A2. Level 1 Rehabilitation Input in Pavement ME Design (Backcalculated Modulus)

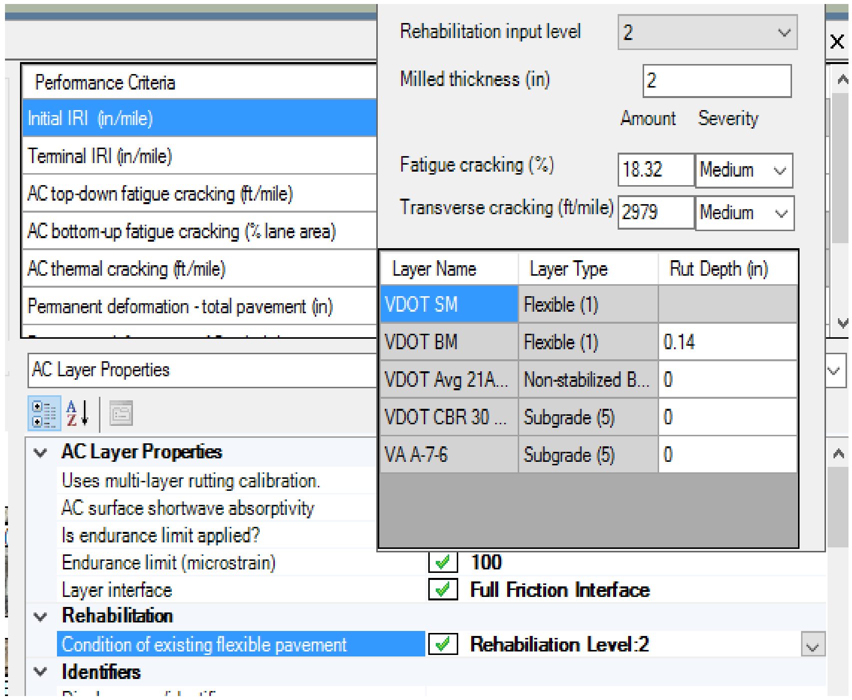


Figure A3. Level 2 Rehabilitation Input in Pavement ME Design

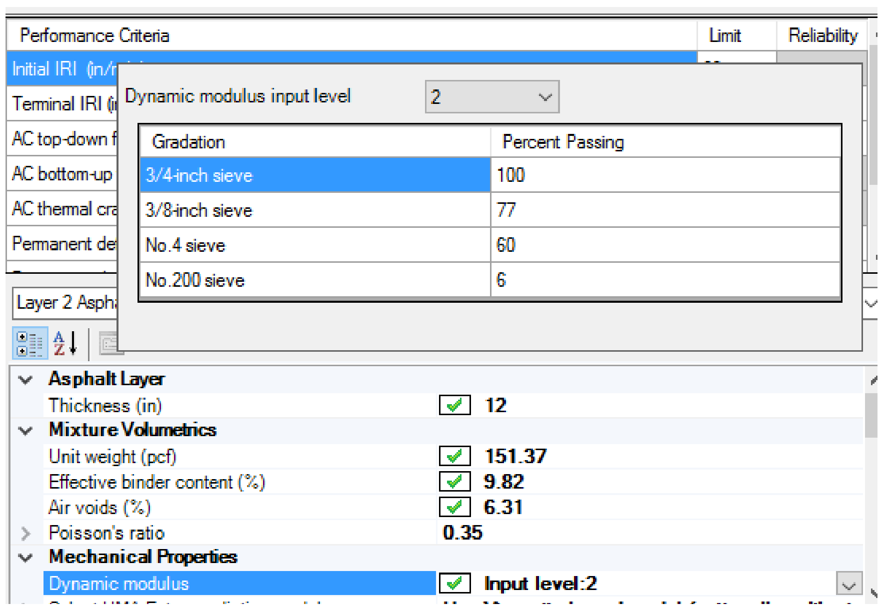


Figure A4. Rehabilitation Input for Existing Pavement in Pavement ME Design

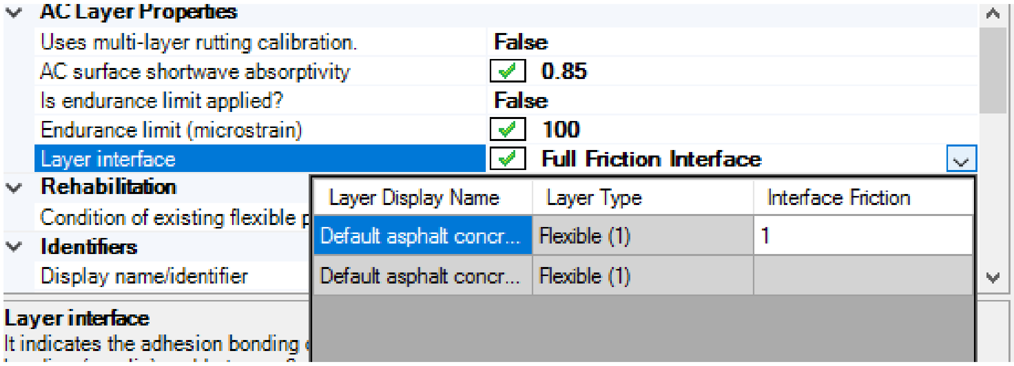


Figure A5. Example of Entering Interface Friction in Pavement ME Design

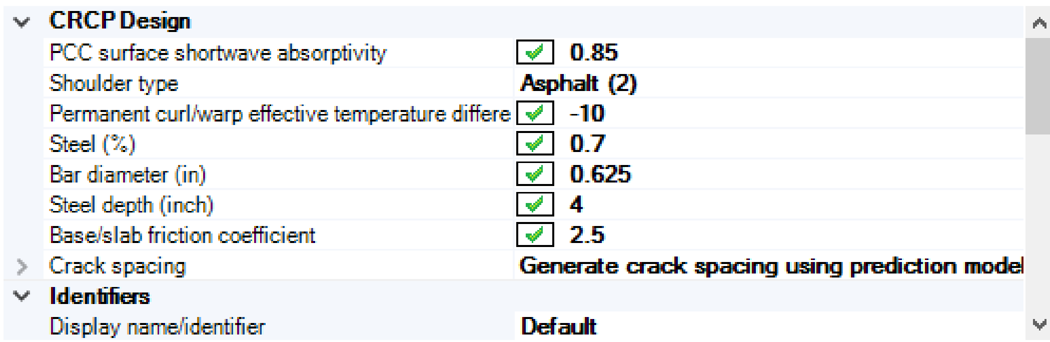


Figure A6. CRCP Inputs in Pavement ME Design. CRCP = continuously reinforced concrete pavement.

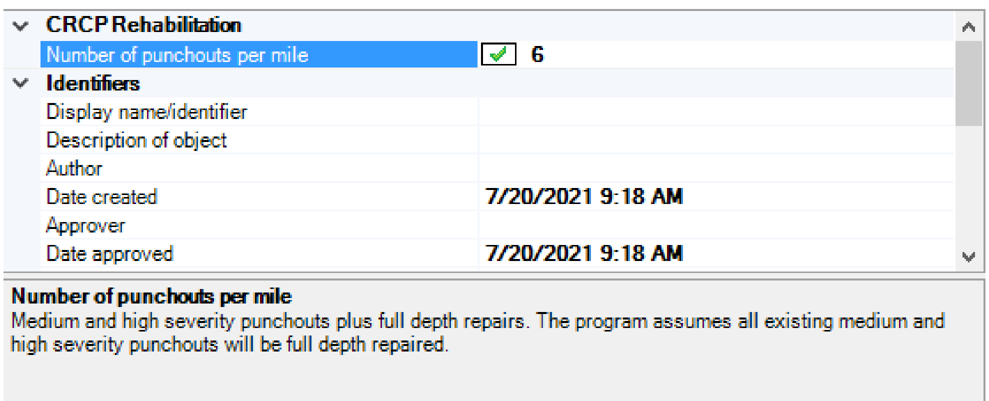


Figure A7. CRCP Rehabilitation Inputs in Pavement ME Design. CRCP = continuously reinforced concrete pavement.

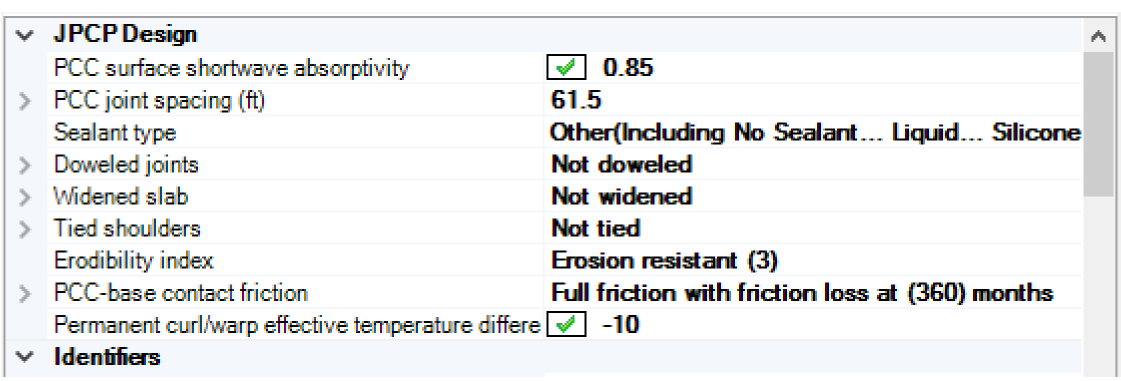


Figure A8. JCP Inputs in Pavement ME Design. JCP = jointed concrete pavement.

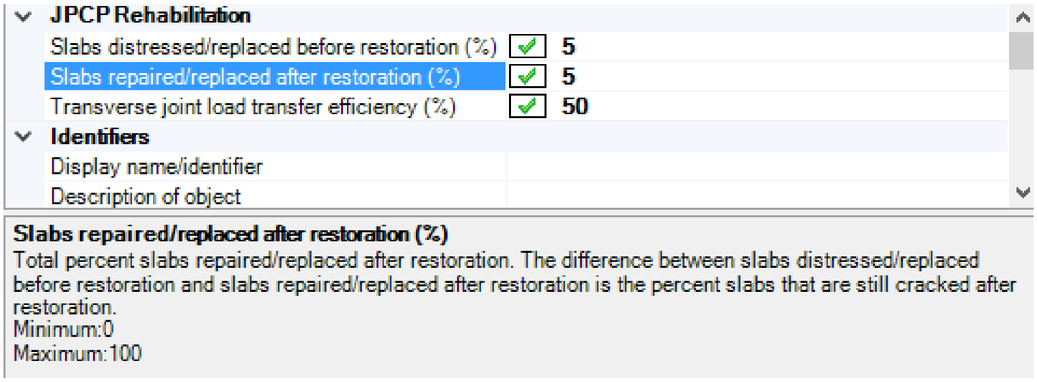


Figure A9. JCP Rehabilitation Inputs in Pavement ME Design. JCP = jointed concrete pavement.

APPENDIX B
Details of Projects Used for Pavement ME Calibration

Table B1. AC Over AC sites

Sl No.	Name	Last Rehab Year	SM	IM	BM	Milling	AC Total	21a/b, Base	Construction Year	AADT Year	Two-Way AADT	GR
1	Br-I	2010	2 in SM12.5E	-	-	2 in	12	6	1987	2010	9660	1.31
2	Br-II	2010	2 in SM12.5E	-	-	2 in	12	8	1978	2010	11200	0.7
3	Cul-I	2010	SMA9.5 1.5 in	IM19.0D 2 in	BM25.0D 3 in	4 in	12	6	1979	2010	4000	0.70
4	Fred-II	2008	1.6 in SM 12.5D	-	-	SOL	8	6	1974	2008	1100	1.3
5	HR-I	2007	2 in SM 12.5D	-	-	SOL	10.5	10	1991	2007	5740	0.4
6	Lynch-I	2010	2 in 12.5D	-	-	2 in	9	6	1966	2010	56	1.0
7	Nova-I	2011	1 in SM9.5A	2 in IM19.0D	-	2 in	10	10	1973	2009	1740	1.8
8	Nova-II	2011	2 in SMA9.5	2 in IM19.0D	-	2 in	10	10	1973	2011	1740	2.4
9	Nova-III	2007	1.5 in SMA9.5D	2 in IM 19.0D	-	3.5	10.8	6	1979	2007	3600	1.41
10	Rich-X	2011	1.5 in SM 9.5D	2 in IM19.0D	-	2 in	9	7	1973	2011	2200	0
11	Rich-V	2009	1.5 in SMA12.5	2 in SMA19.0	-	1.5 in	13.0	9.0	1970	2009	3080	2.5
12	Rich-VII	2009	2 in SMA12.5	2 in SMA19.0	-	4 in	12	6	1994	2009	6600	2.7
13	Rich-IX	2009	1 in SM12.5A	2 in IM19.0D	-	0 in	11.33	7	1967	2009	36	1.9
14	Rich-III	2011	2 in SMA12.5	2 in SMA19.1	-	2	9	6.27	1989	2011	7440	2.9
15	Rich-IV	2010	2 in 12.5D	2 in 19.0D	-	3 in	10	6	1977	2010	9600	2.2
16	Rich-VI	2009	2 in SM12.5E	2 in IM-19.0D	-	4 in	11	6	1979	2009	16640	0.7
17	Rich-II	2010	2 in SMA12.5	2 in SMA19.0	-	2	11	6.35	1966	2010	7440	3.0
18	Rich-XI	2011	1.5 in SM12.D	4 in IM 19.0D	-	4	11	7.1	1997	2011	870	0.6
19	Rich-XVIII	2011	2 in SM 12.5D	2 in 19.0D	-	2	13	7	1977	2011	8640	2.4
20	Rich-XIX	2011	2 in SMA12.5	2 in SMA19.0	-	2	13	7	1977	2011	9540	2.4
21	STN-VI	2008	2 in SMA12.5	2 in SMA-19.0	-	4	11.5	6	1968	2008	12000	1.9
22	STN-I	2010	1.5 in SM12.5D	-	-	1.5 in	9.5	6	1995	2010	1680	2.81
23	STN-II	2010	1.5 in SM12.5D	-	-	1.5 in	9.5	4	1995	2010	1680	2.8
24	STN-III	2010	1.5 in SM12.5A	-	-	SOL	10.6		1972	2010	132	0
25	STN-XIV	2008	1.5 in SM 12.5A	-	-	SOL	10.1		1970	2008	650	0
26	Salem-III	2012	1.5 in SM12.5E	-	-	1.5 in	12	6	1970	2012	4860	1.77

AC = asphalt concrete; SM = surface mix; IM = intermediate mix; BM = base mix; AADT = annual average daily traffic; GR = growth rate; SMA = stone matrix asphalt; - = not used.

Table B2. AC Over CRCP Sites

Sl No.	Name	Last Rehab Year	SM	IM	Milling	Total AC Thickness, in	CRCP	21A/B	Soil Type	Construction Year	AADT Year	Two-Way AADT	Growth Rate
1	Cul-I	2010	1.5 in SMA9.5	-	1.5	4.5	8	6	VA-A-4	1966	2010	4290	1.85
2	HR-I	2011	1.5 in SMA	2 in SMA19.0	-	3.5	8	6	VA-A-4	1970	2010	6080	3.10
3	HR-II	2011	1.5 in SMA	2 in SMA19.0	-	3.5	8	6	VA-A-4	1973	2010	6240	2.09
4	HR-IV	2012	2 in SMA12.5	3 in SMA19.0	-	5	8	6	VA-A-6	1989	2012	1960	7.00
5	HR-V	2012	2 in SMA12.5	3 in SMA19.0	-	5	8	6	VA-A-6	1989	2012	1960	7.00
6	RICH-I	2010	1.5 in SMA9.5	2 in SMA19.0	-	3.5	8	6	VA-A-6	1970	2010	3120	1.59
7	Rich-II	2010	1.5 in SMA9.5	2 in SMA19.0	-	3.5	8	6	VA-A-6	1970	2010	3360	1.28
8	RICH-III	2010	1.5 in SMA9.5	2 in SMA19.0	-	3.5	8	6	VA-A-7	1990	2010	8640	1.72
9	RICH-VI	2011	2 in SM 12.5	-	-	2	8	6	VA-A-7-6	1990	2010	2550	5.10
10	RICH-XIII	2010	1.5 in SMA9.5	2 in SMA19.0	-	3.5	8	6	VA-A-7-6	1968	2010	4200	1.24
11	I-64EB ALBEMARLE	2008	2 in SMA	-	-	6	8	6	VA-A-6	1970	2008	2040	2.14
12	I-64 Louisa_5.22-7.06	2015	2 in SMA	-	2	5	8	6	VA-A-6	1978	2015	4320	2.74
13	I-64 New Kent 17-20	2015	2 in SMA	IM19.0 3 in	4.5	6	8	6	VA-A-7-6	1972	2015	5220	1.00
14	I-64 New Kent 2.5-4.1	2015	1.5 in SMA	IM19.0 3 in	4.5	4.5	8	6	A1-b	1990	2016	4400	3.00

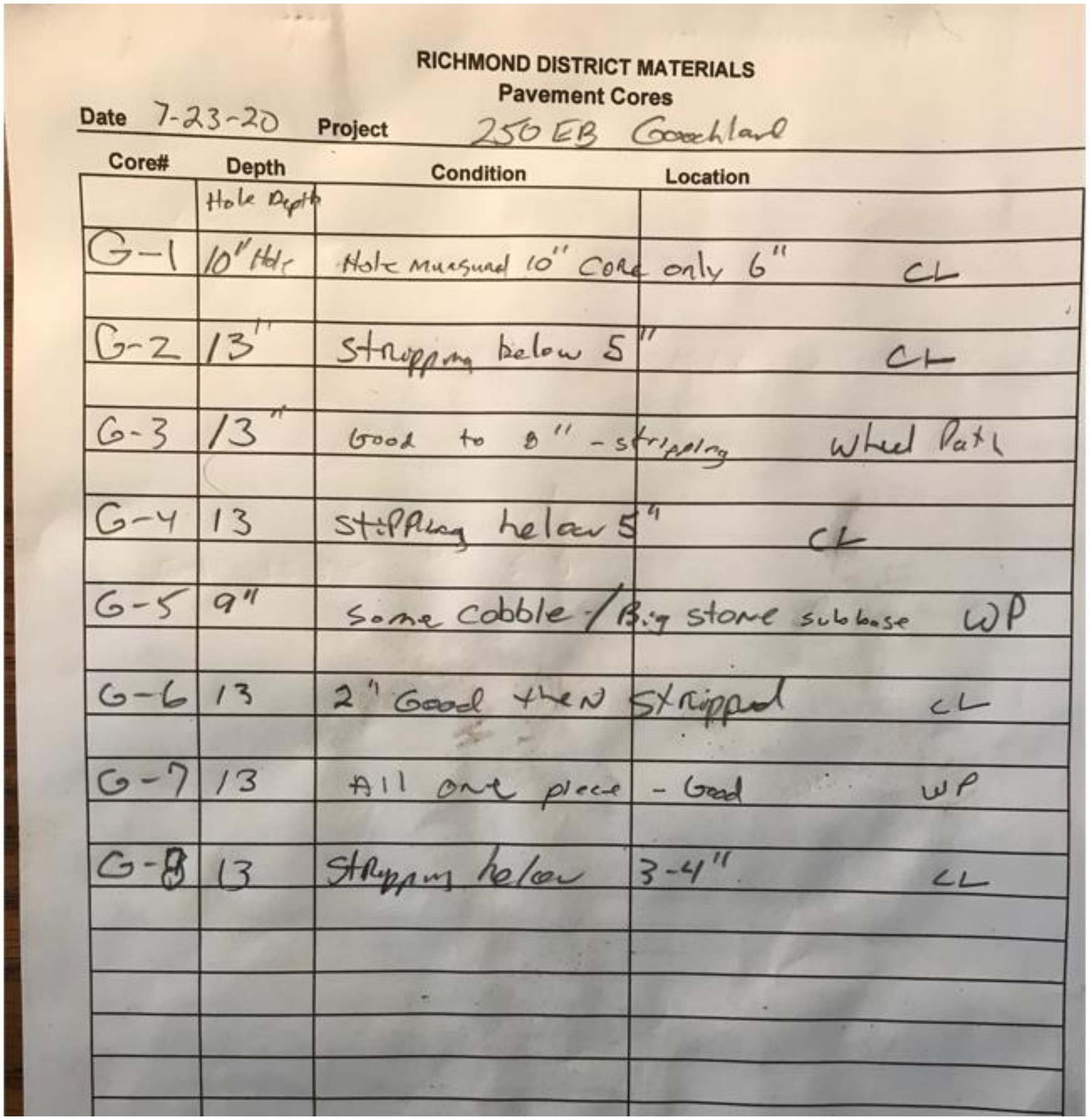
AC = asphalt concrete; CRCP = continuously reinforced concrete pavement; SM = surface mix; IM = intermediate mix; AADT = average annual daily traffic; SMA = stone matrix asphalt; - = not used.

Table B3. AC Over JPCP/JRCP Sites

Sl No.	No.	Pavement Type	Last Rehab Year	SM	IM	Milling	Total AC Thickness, in	JRCP	21 A/ B	Final Subgrade Soil Type Selected	Construction Year	AADT Year	Two-Way AADT	GR
1	Salem-I	BOJ	2009	2 in SMA12.5	3 in SMA 19.0	4.5	5	9	6	VA A-7-6	1965	2009	12230	0.64
2	Salem-II	BOJ	2009	2 in SMA12.5	3 in SMA 19.0	4.5	5	9	6	VAA-7-6	1965	2009	13932	0.58
3	Rich-VII	BOJ	2005	1.5 in SM12.5D	1.4 in S-5	-	3.5	8	6	VA A-4	1989	2005	576	0.50
4	Rich-I	BOJ	2009	1.5 in SMA12.5	4 in IM19.0D (modified)	-	5.5	9	6	VA A-6	1986	2009	6500	2.31
5	Rich-II	BOJ	2008	2 in SM12.5E	-	2	3.5	8	6	VAA-7-6	1980	2008	80	0.45
6	Nova-I	BOJ	2009	2 in SM12.5E	2 in SM12.5E	2	4.5	9	6	VA A-4	1967	2009	17460	0.00
7	Fred-IV	BOJ	2009	2 in SMA12.5	-	2	7.5	9	10	VA A-5	1976	2009	8640	2.00
8	Fred-I New	BOJ	2010	2 in SMA12.5	-	2 in	6	9		A-2-4	1987	2008	13600	0.00
9	Fred-II	BOJ	2009	2 in SMA12.5	-	2 in	8	9	6	A-2-4	1986	2008	16500	0.00
10	Fred-III	BOJ	2008	2 in SMA12.5	-	2	8	9	6	VA A-6	1987	2008	16500	0.00
11	HR-II	BOJ	2010	2 in SMA12.5	-	2	5.5	9	6	A-2-4	1986	2010	3360	2.23
12	Rich -I-CIR	BOJ	2011	2 in SMA-12.5	3 in SMA-19.0	5	9	9	6	VA A-7-6	1969	2011	588	1.53
13	Rich -II-CIR	BOJ	2011	2 in SM 12.5E	2 in IM19.0A	5	9	9	6	VA A-7-6	1969	2011	360	2.60
14	RICH-IX	BOJ	2007	2 in SMA	2 in SMA19.0	-	4	8	6	VA A-4	1966	2010	7200	0.25
15	I-85 Dinwiddie	BOJ	2015	1.5 in SMA	2 in SMA19.0	-	5.5	9	6	VA A-7-6	1968	2015	5040	0.00
16	I-95 Prince George	BOJ	2015	2 in SM 12.5E	-	2	3	9	6	VA A-4	1965	2015	13200	1.00
17	I-495 Fairfax	BOJ	2015	2 in SM 12.5E	2 in SM 12.5E	2	5		6	VA A-4	1976	2015	4500	1.00
18	I-395 Fairfax	BOJ	2015	2 in SM 12.5E	-	2	4.5	8	6	VA A-4	1967	2015	5700	0.00
19	I-95 Spotsyl-vania	BOJ	2015	1.5 in SMA 12.5	2 in SMA 19.0	4	6.7	9	6	VA A-4	1987	2015	15600	1.00

AC = asphalt concrete; JPCP = jointed concrete pavement; JRCP = jointed reinforced concrete pavement; SM = surface mix; IM = intermediate mix; AADT = average annual daily traffic; BOJ = AC over jointed concrete pavement; SMA = stone matrix asphalt; GR = growth rate; - = not used.

APPENDIX C
Coring Details



RICHMOND DISTRICT MATERIALS
Pavement Cores

Date 7-23-20 Project 250 EB Goochland

Core#	Depth	Condition	Location
	Hole Depth		
G-1	10" Hole	Hole measured 10" Core only 6"	CL
G-2	13"	Stripping below 5"	CL
G-3	13"	Good to 8" - stripping	Wheel Path
G-4	13	Stipping below 5"	CL
G-5	9"	Some cobble / Big stone subbase	WP
G-6	13	2" Good then Stripped	CL
G-7	13	All one piece - Good	WP
G-8	13	Stripping below 3-4"	CL

Figure C1. Coring Log for Rte. 250

Figure C2. Core Photographs From Rte. 250