Innovations in Mixture and Structural Design of Cold Recycled Pavements

by

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A dissertation submitted to the Graduate Faculty of
Auburn University
in partial fulfillment of the
requirements for the Degree of
Doctor of Philosophy

Auburn, Alabama
August 8th, 2026

Keywords: Pavement, Asphalt, Cold Recycling, RAP, Rejuvenator, MEPDG

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Abstract

Cold recycled (CR) pavements, the combination of reclaimed asphalt pavement (RAP) and an asphalt stabilizing agent (foamed or emulsified asphalt binder) at ambient temperatures, can provide a sustainable method to rehabilitate, maintain, and construct roadways. The design, testing, and applications of CR pavements have advanced as the knowledge surrounding the topic grows; however, many of the specifications and design methods being implemented are based upon Hot Mix Asphalt (HMA)/Warm Mix Asphalt (WMA) mixture properties and characteristics. If CR pavement is to be used as an alternative to an HMA base or potentially as a surface course, assessment of the mixture design, laboratory testing, and structural design methods is necessary. This dissertation aims to fill this knowledge gap by evaluating existing structural design and laboratory design processes and assessing the shortcomings of the existing processes.

To assess existing structural design methods, the American Association of State Highway and Transportation Officials (AASHTO) AASHTOWare Pavement Mechanistic-Empirical (ME) Design software was used. Structural and mixture properties of 13 flexible pavement sections containing a CR base or intermediate layer were input into the software, and the predicted distresses produced by the bottom up and total rutting distress models were compared to the measured distresses. Results indicated that the global coefficients, all of which were established based on HMA/WMA data, were not sufficient in accurately predicting pavement distresses. Model coefficient calibrations were conducted for the bottom up cracking and total rutting models. The bottom up cracking model coefficient calibrations indicated that the recommended method of measured cracking aggregation led

to distress overprediction, which may result in an over or under designed pavement that does not perform as intended. Per the AASHTO Local Calibration Guide (AASHTO, 2010), recommended aggregation methods state that all cracking should be considered bottom up unless forensics indicate otherwise. The total rutting model coefficient calibrations showed that in pavements with 5 inches of total asphalt thickness and a CR layer placed directly on a granular base, subgrade rutting dominated the predicted rutting model with the subgrade rutting alone being over 4 times the maximum measured rut depth.

CR mixture advancements were also evaluated through the mixture design and field placement of a CR mixture as a surface course. Mixture design phases explored rejuvenator and emulsion contents impact on mixture performance, resulting in an optimum design for field placement. The placement of the rejuvenated CR mixture as a surface course was like that of any other CR mixture. Evaluations of laboratory testing methods and performance criteria concluded that CR mixture performance may not be accurately captured using existing methods and criteria developed around HMA/WMA. Discussion of laboratory performance tests for CR mixtures highlighted the limitations based on current methods and recommended additional testing for CR mixtures intended for surface course applications.

The results from this dissertation aided in identifying the limitations of the existing processes surrounding laboratory performance testing and ME structural design via AASHTOWare Pavement ME Design (PMED) software and provided recommendations for considerations to be made with implementation of the findings and future research.

Artificial Intelligence (AI) Use Disclosure Statement

In the preparation of this dissertation, no Artificial Intelligence (AI) tools were used.

Digital Accessibility Disclosure Statement

In the preparation of this dissertation, the digital accessibility checker within Microsoft Word was used to ensure this document complies with federal requirements. The author acknowledges full responsibility for the intellectual content of this work and has made a good faith effort to comply with digital accessibility requirements in publishing, wherein the nature of the content does not significantly change in order to do so. Furthermore, all content has been reviewed and revised to meet these requirements prior to final publication.

Acknowledgements

The time that passed during my doctoral program has encompassed some of the most trying and most rewarding times of my life. I have been challenged in ways I did not know were possible, fostering strength, persistence, and growth in my personal life, work, and education. Through the late nights and long days of writing, I have been supported by my community. This dissertation would not have been possible without them.

I would like to express gratitude to my advisor, Dr. Benjamin Bowers, for providing me with guidance, research opportunities, and encouragement. The completion of the degree would not have been possible without his continued support and investment towards my learning throughout my academic career and into my professional career.

I would like to thank Dr. Brian Diefenderfer, Dr. Fan Yin, Dr. Carolina Rodezno, and Dr. Suri Gatiganti for their participation on my committee, providing constructive feedback that encouraged growth in my learning and my research.

Many people have provided me with continuous support during my education, both in and out of the lab. First, I want to thank my parents, Rod Turochy and Kathy Gregory. They have provided me with invaluable love, guidance, and unwavering support throughout my journey as a doctoral student and the challenges that came along with it. I would not have been able to successfully complete this degree without them.

Next, my sisters Caroline and Rebecca. They have supported me and provided reprieve from the ebbs and flows of this program. We had the privilege of living together during college and getting to support each other while all being in different stages of our academic careers at Auburn. I am proud to be their older sister.

I would like to thank Sophie May for being my plus one to many conferences and asphalt events (to the point where people ask where she is if she's not there with me), for listening to my constant commentary on pavements, and for tolerating me when I easily get distracted by paving equipment on every road trip.

I would also like to thank my brother-in-law, Jacob, for continuously checking up on my research and writing progress and pushing me to continue while our dissertation and thesis writing overlapped and beyond.

This research would not have been possible without the support of my friends and colleagues at NCAT. The skills I have gained from their guidance during my time in the lab and at the Test Track provided me with the tools to begin my professional career before I had even finished my time at Auburn. I would also like to thank all the graduate students, engineers, and scientists whose research contributed historical knowledge and data on cold recycled pavements that allowed the research presented in this dissertation to be possible.

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Abbreviations

AASHTO	American Association of State and Highway Transportation Officials
AMPT	Asphalt Mixture Performance Tester
ARRA	Asphalt Recycling and Reclaiming Association
ASTM	American Society for Testing and Materials
CCPR	Cold Central Plant Recycling
CIR	Cold In-Place Recycling
CR	Cold Recycled
CT_{Index}	Cracking Tolerance Index
HMA	Hot Mix Asphalt
HT-IDT	High Temperature Indirect Tension Test
IDEAL-CT	Indirect Tensile Asphalt Cracking Test
ITS	Indirect Tensile Strength
LVDTs	Linear Variable Differential Transformers
MS	Marshall Stability
MSR	Marshall Stability Ratio
ME	Mechanistic-Empirical
MEPDG	Mechanistic-Empirical Pavement Design Guide
NCAT	National Center for Asphalt Technology
NCHRP	National Cooperative Highway Research Program
OFC	Optimum Fluid Content
PMED	Pavement Mechanistic-Empirical Design

PMLC	Plant Mixed Laboratory Compacted
RAP	Reclaimed Asphalt Pavement
SSD	Saturated Surface Dry
WMA	Warm Mix Asphalt

Chapter One: Research Background, Objectives, and Organization of Dissertation

1.1 Research Background

Cold recycling (CR) is a process in which an asphalt pavement can be reconstructed, rehabilitated, or maintained using nearly 100% recycled materials that are processed and placed at ambient temperatures. Cold recycling of pavements is typically conducted in one of three ways: Cold Central Plant Recycling (CCPR), Cold In-Place Recycling (CIR), or Full Depth Reclamation (FDR). For this dissertation, only CCPR and CIR are discussed. Both methods (CCPR and CIR) use RAP in combination with an asphalt stabilizing agent (foamed asphalt or asphalt emulsion) to create a stabilized material that can then be placed and compacted as a pavement layer using conventional HMA/WMA equipment.

CR pavements are becoming increasingly applied and documented in both high- and low-volume applications (Diefenderfer et al., 2014; Bowers et al., 2024; Vargas-Nordcbeck and Timm, 2013) and can provide a sustainable alternative to HMA/WMA (Al-Qadi and Ozer, 2020). However, as the applications of CR pavements continue to advance, the need to thoroughly understand the structural design, mixture design, and performance testing of these mixtures becomes more pertinent. Many of the structural design methods and laboratory testing procedures and performance criteria being used to develop and characterize CR mixtures are based upon HMA/WMA mixture properties and characteristics. If CR pavement is to be applied as an alternative to a HMA base or potentially as a surface course, assessment of the mixture design, laboratory testing, and structural design methods is necessary.

The two structural design methods, AASHTO 1993 and Mechanistic-Empirical (ME) design, do not consider any different properties or have differing design methods for CR versus those used for HMA/WMA. In the case of both design methodologies, CR pavements were not considered in the original materials design, testing, or equation development. For the AASHTO 1993 method, structural layer coefficients are used to characterize pavement layers. In many cases the structural coefficient falls between a granular base and HMA (Gallegos and Johnson, 2019; Dave et al., 2019; Chesner et al., 2011; Stroup-Gardiner, 2011; Diefenderfer and Apeagyei, 2014; Diefenderfer et al., 2016). Many highway agencies are transitioning to using the methods outlined in the AASHTO Mechanistic-Empirical Design Guide (MEPDG) and using the AASHTOWare Pavement ME Design (PMED) software. This software provides pavement design and performance prediction tools using transfer functions that have globally calibrated coefficients. Despite the variety of pavement structure types and locational diversity contributing to the global calibrations, structures including one or more CR layers are not considered. The new web-based version of this software provides a template for CR layers; however, the default values are based on data and properties from traditional HMA. Previous research (Schwartz et al., 2017) showed that the stiffness properties of CR mixtures differ from those found in typical HMA. Considering this, it is important to evaluate the globally calibrated model with inputs from pavement structures containing CR layers.

When evaluating the procedures and criteria for the mixture design and laboratory testing of CR mixtures, many still operate based on HMA/WMA properties and characteristics, with some studies using suggested testing criteria specific to CR mixtures (Schwartz et al., 2017, Turochy and Bowers, 2024). Regarding mixture design, AASHTO has established procedures for CR mixtures using foamed asphalt and asphalt emulsion

(AASHTO PP 86 and AASHTO PP 94); however, these specifications do not consider the use of rejuvenators and how they may impact performance. The use of rejuvenators has been shown to increase the density of a CR mixture and shows a potential connection with increased field performance (Turochy and Bowers, 2024), and their incorporation could support the use of CR mixture in applications beyond a base or intermediate layer to a surface course. Applying a CR mixture as a surface course subjects it to different weather conditions and direct traffic loading that a base or intermediate layer does not receive. This prompts the need for additional laboratory performance testing and evaluation of the current HMA/WMA testing methods and criteria for use with CR mixtures.

1.2 Objectives

This research will investigate innovations in CR through assessing structural design and pavement performance determination through ME design, evaluating mixture design, laboratory performance, and construction processes for CR surface course mixtures, and discussing considerations for the structural and mixture design of CR surface courses. The objectives for this dissertation are:

1. **Determine the suitability of existing global coefficients in the AASHTOWare Pavement ME Design software pavement distress prediction models for use in structural design of flexible pavements containing a CR base or intermediate layer.** This study will determine the distress prediction accuracy of version 3.0 of the PMED software global calibration coefficients for pavements containing CR base or intermediate layers. The bottom up cracking and total rutting distress models will be evaluated using the global coefficients, and any necessary coefficient calibrations will be conducted to best predict pavement distresses. Model sensitivity to select mixture properties will also be evaluated.

2. **Assess the mixture and structural design and evaluation process of rejuvenated cold recycled mixtures for surface course applications**. This study will evaluate the mixture design process using existing CR design specifications and determine the necessary considerations and modifications needed to develop a rejuvenated CR mixture design for use as a surface course. The influence of rejuvenator content on mixture performance will also be evaluated and the plant produced mixture placement and laboratory testing will be documented. Structural design for pavements with a CR surface course using the previously calibrated model coefficients will be tested and documented.

1.3 Organization of Dissertation

This dissertation is organized into five chapters:

- Chapter One: *Introduction* - Provides background on CR pavements, dissertation objectives, and the scope of the research projects.
- Chapter Two: *Literature Review* - Establishes the history of CR pavements, the types of CR pavements, mixture and structural design processes, and performance testing used in the context of this dissertation. The asphalt stabilizing agents and rejuvenating agents that have been used in CR pavements and their role within a CR mixture were detailed, followed by the steps that are included within typical CR mixture design processes. Performance testing in relation to this research and CR pavements was also covered. The history of structural pavement design was provided, including both the empirical and mechanistic empirical methods. This was followed by an analysis of the history of CR pavement structural design and advancements surrounding it. The calibration process for the PMED software

pertaining to this dissertation was also detailed to provide context for the methods used in followed chapters.

- Chapter Three: *Calibration of Pavement Distress Prediction Model Coefficients for Cold Recycled Pavements* – This chapter evaluates the accuracy of the global coefficients within version 3.0 of the PMED software, a sensitivity analysis of the model to select mixture specific inputs, and the calibration process of the bottom up cracking and total rutting distress prediction models to best match the predicted distresses to measured distresses.
- Chapter Four: *Rejuvenated Cold Recycled Mixture Design for Surface Course Applications* – This chapter details the mixture design, placement, laboratory testing, and field performance of a rejuvenated CR mixture as a surface course. The impact of rejuvenator content on mixture performance will also be assessed and recommendations for future work using CR mixtures as a surface course will be provided based upon the findings of this research.
- Chapter Five: *Conclusions and Recommendations* – Summarizes the conclusions and provides recommendation based on the research presented in chapters three and four.

Chapter Two: Literature Review

2.1 Cold Recycling

CR pavements can provide a more sustainable option for roadway rehabilitation, maintenance, and construction compared to typical mill-and-fill and full depth rehabilitation with HMA or WMA mixtures. For CR pavements, an asphalt stabilizing agent (foamed asphalt or asphalt emulsion) and RAP millings, either directly from the roadway or from a stockpile, are combined to create a new pavement layer. In some cases, an active filler (e.g. cement) is used as well. By using nearly 100% recycled materials, the CR process reduces the need for virgin material which in turn reduces the material cost. The mixture is compacted at ambient temperatures and often times is mixed on site using materials on site, requiring minimal need for hauling trucks to transport materials and thus reducing emissions (Pakes et al. 2018, Stroup-Gardiner 2011).

This sustainable alternative to hot or warm mix asphalt (HMA or WMA) is typically conducted in one of two ways: CCPR or CIR. Both processes produce nearly identical pavements once placed, but the methods to achieve the final resulting mixture differ. CCPR uses a stationary or mobile plant to mix reclaimed asphalt pavement (RAP) with a stabilizing agent and/or additive (if used). With CCPR, the RAP can either be sourced from a stockpile, millings from the construction site or other location, or a combination of both. Once processed through the plant, the mixture is then transported to the construction site where it can be placed and compacted in a process using conventional HMA or WMA equipment in one or more lifts (Asphalt Recycling and Reclaiming Association (ARRA), 2015). CIR, unlike CCPR, conducts the milling, mixing, and placement of the CR mixture in situ. The CIR process is conducted in a train-like fashion with multiple pieces of equipment. Typically, a milling machine, cement distributor, cold recycler, water truck, paver, and various compaction rollers all work in tandem. Although the production and placement of CCPR and CIR follow different processes, the mixture design development process and asphalt stabilizing agents used are nearly identical.

2.2 Asphalt Stabilizing Agents and Rejuvenating Agents

2.2.1 Foamed Asphalt

One of the two asphalt stabilizing agents used to stabilize CR mixtures and bind the particles together is foamed asphalt. Foamed asphalt is formed when hot asphalt binder is combined with water and pressurized air, resulting in steam bubbles encapsulated by the asphalt binder. Figure 1 shows the formation of these bubbles.

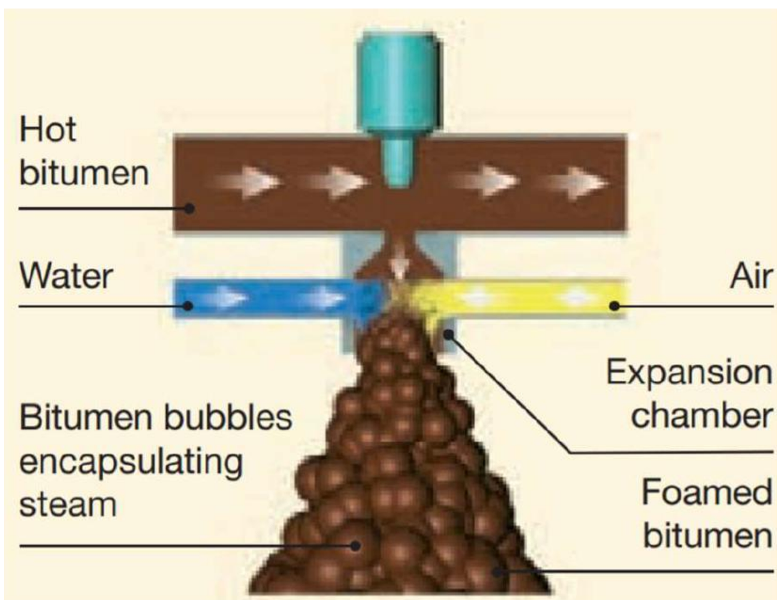


Figure 1. Formation of Foamed Asphalt (Wirtgen 2012)

These asphalt encapsulated steam bubbles are sprayed at their maximum volume into RAP that is actively being mixed for optimum coverage. Upon contact with the RAP, the bubbles burst and the asphalt binder creates spot-welds between RAP particles (Figure 2). These spot-weld bind the mixture together.

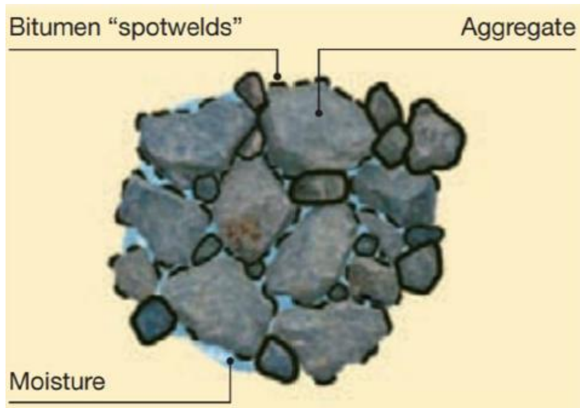


Figure 2. Mix Matrix Bound by Spot-Welds (Wirtgen 2012)

For an asphalt binder to be considered acceptable for use in a CR design, it must meet the expansion ratio and half-life requirements. The expansion ratio of the binder is defined as the volumetric ratio of the unfoamed binder to the foamed binder and is an indication of how well the binder will disperse across the mixture. The half-life of an asphalt binder is the time between the maximum expansion of the foamed binder to when it reaches half of that volume, in seconds. Both temperature and water content are two influences over the binder foaming properties, both of which are altered to determine the optimum binder foaming conditions. The foamed binder must meet the minimum requirements of an expansion ratio of 8:1 and a half-life of 4 seconds (Wirtgen, 2012). Typically, three temperatures at three water contents per temperature are trialed and the combination that produces the optimum expansion ratio and half-life is selected for use in mixture design.

2.2.2 Asphalt Emulsion

Asphalt emulsion, the blend of an asphalt binder, water, and an emulsifying agent, is the second of the two asphalt stabilizing agents used in CR mixture design. When an emulsion is added to a CR mixture, a "breaking" or curing occurs. This breaking process occurs when the excess water in the emulsion begins to evaporate, causing the asphalt residue left to coalesce and adhere to the RAP aggregate. Unlike the spot-welds formed by foamed asphalt binder, asphalt emulsion nearly entirely coats the RAP aggregate, binding the mixture together.

An asphalt emulsion can either be categorized as anionic (negatively charged) or cationic (positively charged). The type and charged of emulsion used is selected on a project-by-project basis based on the aggregate used, temperature, and breaking rate desired. For example, typically, colder weather and higher moisture contents can delay breaking. This can restrict the ability to use some emulsions (Stroup-Gardiner, 2011). Engineered emulsions, however, are a specialized emulsion that can be designed to accommodate potential issues like cooler temperatures.

2.2.3 Rejuvenators

A rejuvenator is a mixture additive designed to fully or partially restore the properties of an oxidized asphalt binder, like exists in RAP. In asphalt mixtures with high recycled binder ratios, such as CR pavements, use of a rejuvenator can help to restore binder properties and/or increase binder accessibility. This in turn can add flexibility to the mixture, decrease stiffness, and increase cracking resistance (Sias et al., 2022). For CR mixtures specifically, rejuvenators help to soften the RAP and increase adhesion between RAP particles. Rejuvenating agents are typically oil based and can be added to a binder, an emulsion, or to the pavement mixture directly.

2.3 Mixture Design

2.3.1 Determination of RAP Optimum Fluid Content

The optimum fluid content (OFC) is essential for determining the amount of fluid in a CR mixture that correlates with the maximum achievable density. Achieving the OFC in a CR mixture helps to reduce compaction effort in mixture containing foamed asphalt, and aids in asphalt emulsion coating RAP particles (ARRA, 2015). Too much fluid, however, can lead to difficulties achieving density and potential flushing of binder and any other mixture additives. Two primary methods are used for the OFC determination: AASHTO T180 *Standard Method of Test for Moisture-Density Relations of Soil Using a 4.54-kg (10-lb) Rammer and a 457-mm (18-in.) Drop* and California Test 313 Section I *Optimum Moisture Content and Maximum Density Determination*. Both methods establish an OFC at the water content in which the sample reaches maximum density, often shown in the form of a moisture-density curve.

2.3.2 Recycling, Stabilizing, and Rejuvenating Agents

Selecting an appropriate asphalt stabilizing agent is a key component in the performance of a CR pavement, with the two options being foamed asphalt and asphalt emulsion (Wang et al., 2018). As mentioned previously, for foamed asphalt binder, selection is based on its expansion ratio and half-life. These properties are indicative of the binder's ability to be well dispersed and create spot-welds between RAP particles. Another key component in the development of spot-welds is the percentage of fines (material passing the No. 200 sieve) in the mixture. Typically, a mixture containing foamed asphalt should contain 5 and 20% fines for optimum performance (ARRA, 2015). Up to 1% cement (by weight of dry RAP) is frequently added in addition to the foamed asphalt binder as it provides additional fines and has been shown to increase strength (Wirtgen, 2012). Unlike foamed asphalt, asphalt emulsions are selected based on their ability to fully coat the aggregate and best fit the construction conditions. They can typically be used for RAP having a lower fines content. In the mixture design development process, three or four selected asphalt stabilizing agent contents are tested to determine the optimum dosage for achieving the desired strength requirements. For mixtures containing foamed asphalt, the typical range is 1.5% to 3.0% and for emulsion it is 0.5% to 4.0%, both by weight of dry RAP (ARRA 2015). Rejuvenators are typically selected based on their compatibility with the recycling agent used and often dosage rates are recommended by the manufacturer. However, little research has been done on the selection of rejuvenating agents for CR mixtures (Bowers et al., 2019; Turochy and Bowers, 2024; Wu et al., 2022).

2.3.3 Specimen Mixing, Compacting, and Curing

Typical mixture design processes use a using a laboratory sized pugmill for mixing, such as the Wirtgen WLM-30. For this equipment, it is recommended that 25-30 kg (55-66 lb) batches be used for optimum results (Wirtgen, 2012). Mixing at this batch size helps to promote thorough mixing of all additives with a homogenous result. After mixing, the CR mixture test specimens are compacted to 30 gyrations using a Superpave gyratory compactor or to 75 blows using a Marshall hammer (ARRA, 2015). The specimens, regardless of asphalt stabilizing agent type, are then subjected to a curing period. This curing period promotes evaporation of excess moisture within the specimen and gives adequate time for the recycling agent to set, leading to increased strengths versus specimens prior to curing. The curing process is considered complete when there is no moisture left in the sample to evaporate, as indicated by no mass change in the specimen when checked periodically. CR mixtures containing asphalt emulsion are cured for 48 hours at 60°C, per AASHTO PP 86. CR mixtures design with foamed asphalt are cured for 72 hours at 40°C per AASHTO PP 94.

Cured CR specimens are then subjected to strength and moisture susceptibility testing to determine the optimum design. There are two commonly used test methods: Indirect Tensile Strength (ITS) and Marshall Stability (MS) testing. For both testing methods, a set of dry specimens and a set of conditioned specimens at each asphalt stabilizing agent content are tested. For ITS testing, dry specimens are tested at the ambient temperature of a temperature-controlled laboratory. For MS testing, the dry specimens are conditioned at 40°C for two hours prior to testing. Both ITS and MS testing methods subject specimens to a constant loading rate, however the loading distribution varies. For ITS testing, specimens are subjected to compressive loading on the top and bottom with two steel loading strips coming in contact with the sample. MS applies the load uniformly around the specimen through two half-circle shaped steel loading frames. It is recommended that dry strengths meet a minimum of 45 psi for ITS testing and 1250lb for MS testing (ARRA, 2015). Both test methods also evaluate moisture susceptibility, which is done through either a tensile strength ratio (TSR) or Marshall stability ratio (MSR). The ratio is the average conditioned specimen strength/stability divided by the average dry specimen strength/stability. For all CR mixtures, a minimum TSR/MSR of 0.7 is specified.

2.4 Performance Testing

2.4.1 Dynamic Modulus Testing

Dynamic modulus testing is used to evaluate pavement mixture stiffness under repetitive traffic loading through the use of various testing temperatures and frequencies. The dynamic modulus of a mixture is represented by the stress-strain ratio of the tested specimen and can be analyzed at multiple temperatures and loading frequencies (Witczak et al., 2002). The data is collected via an Asphalt Mixture Performance Tester (AMPT) and tested per AASHTO T378 *Standard Method of Test for Determining the Dynamic Modulus and Flow Number for Asphalt Mixtures Using the Asphalt Mixture Performance Tester*. While dynamic modulus testing has been conducted on CR mixtures previously (Schwartz et al., 2017; Diefenderfer et al., 2015), little research has been conducted testing rejuvenated CR mixtures (Bowers et al., 2019; Turochy and Bowers, 2024). When evaluating the dynamic modulus master curves produced by various CR mixtures (foamed asphalt and emulsified asphalt stabilized) versus HMA/WMA, the CR mixtures tend to have similar modulus values at lower reduced frequencies. As frequencies increase, however, the CR mixtures maintain a more gradual increase in modulus while HMA/WMA sees a steeper incline (Bowers et al., 2019). The introduction of a rejuvenator to a CR mixture has shown to impact the modulus values, leading to a steeper incline in modulus with a curve that falls between that of a typical CR mixture and typical HMA (Bowers et al., 2019; Turochy and Bowers, 2024).

2.4.1 IDEAL-CT

IDEAL-CT testing and the CT_{Index} aim to capture the cracking resistance of an asphalt-based mixture. This test was developed at the Texas A&M Transportation Institute by Zhou et al. (2019) as a simple and economical way to test mixtures using four inputs: specimen air voids, specimen height, testing temperature, and loading rate. The specimen properties and performance data are collected and analyzed, and a cracking tolerance index (CT_{Index}) is produced. When evaluating IDEAL-CT testing conducted on CR mixtures containing and asphalt emulsion and cement, an increase in cement content led to a more brittle mixture and a lower CT_{Index} (Dong and Charmot, 2019; Diefenderfer et al. 2019). The cracking analysis conducted on CR mixtures using IDEAL-CT by Diefenderfer et al. (2019) found that for a CR pavement mixture containing a slow setting emulsion, an increase in cement from 0% to 1% lead to a decreased CT_{Index} by 32%. Dong and Charmot (2019) also concluded that an increase in emulsion led to a higher CT_{Index}. Due to the presence of aged binder in RAP, incorporating it into asphalt mixtures tends to decrease the long-term fatigue and cracking resistance (Alae et al., 2022). Considering that CR mixtures contain nearly 100% RAP, it is important to evaluate the mixtures via tests like IDEAL-CT to gauge cracking resistance. It is also of interest considering the adoption of this test for HMA mixture design in the Balanced Mix Design (BMD) framework and the potential interaction between HMA and rejuvenated CR mixture design processes in the future. This test, however, is developed around HMA/WMA properties and should be evaluated to determine if the existing methods and criteria capture the cracking resistance of CR mixtures.

2.4.2 HT-IDT

High temperature indirect tension testing (HT-IDT) quantifies the rutting resistance of a mixture. For this test, specimens compacted to 150 mm in diameter and 62 mm in height are conditioned for two hours prior to testing in a 50^{o}C oven. The specimens are then subjected to indirect tensile strength testing at a loading rate of 50mm/min. The specimen is placed on the jig and with loading applied on the top and bottom with two steel loading strips coming in contact with the sample. A study by Yin et al. (2020) showed HT-IDT strengths ranging from 21.3 to 43.6 psi for HMA. There are currently no accepted specifications or performance testing requirements specific to CR mixtures, little to no research has been documented on the use of HT-IDT testing on these mixtures (Turochy and Bowers, 2024). However, it has been documented that increasing RAP quantities in pavement mixtures can lead to an increase in rutting resistance (Ali et al., 2017). It has also been found that the addition of rejuvenators can reduce this resistance (Kim et al., 2019).

For CR mixtures containing a rejuvenator, it is important to monitor performance via tests such as HT-IDT to ensure adequate rutting resistance. The HT-IDT is also under consideration for HMA BMD implementation and was discussed in this dissertation.

2.5 Asphalt Pavement Structural Design

2.5.1 AASHTO 1993 Method

One of the initial methods for structural pavement design was developed by AASHTO. This design method, known as the AASHTO 1993 Method after its development, was based upon data collected from the AASHO Road Test (1958-1960) and focused on determining pavement thickness based on inputs such as traffic loading, subgrade responses, and the desired pavement life. A design equation was developed from these inputs that result in a predicted total number of Equivalent Single Axle Loads (ESALs) that a pavement design can withstand. This introduction of ESALs provided advancements in design as it was able to take a variety of traffic loadings and compile it into a single standard load measurement. This method, however, may not accurately represent the impact of different axle configurations and loading and could lead to inaccurate damage predictions as a result. Another shortcoming of the method was its limited testing set. Applications of the method beyond the pavement structures and types used in the AASHO Road Test required extrapolation from the original data, which could not guarantee designs that fully captured the performance of more modern materials. Research to establish AASHTO 1993 Method structural layer coefficients for CR pavements are discussed in further detail in Section 2.6. These limitations in the AASHTO 1993 method posed a need for a more comprehensive design method.

2.5.2 AASHTO Mechanistic-Empirical Design Method

While the AASHTO 1993 method offers a straightforward, empirical approach to structural pavement design, the ME design approach allows for more mixture-specific properties, performance testing results, environmental conditions, and mechanical responses to traffic loading to be incorporated. This allows for a more accurate prediction of pavement distresses, which is in turn used to select layer thicknesses and materials that meet the pavement performance requirements. The ME design method uses transfer functions that take traffic data, climate data, layer thickness, and material inputs into consideration to predict the pavement performance. These inputs are considered across three different levels. These inputs and their parameters for flexible pavement design can be found in the MEPDG (AASHTO, 2010) and are shown in Figure 3 below.

Inputs	Input parameters
Traffic	AADTT (Annual Average Daily Truck Traffic), axle configuration, vehicle class distribution, monthly traffic adjustment, axles per truck, etc.
Climate	Temperature, precipitation, freeze-thaw cycles, etc.
Layer Properties	Layer thicknesses, base and subgrade resilient modulus, AC dynamic modulus, volumetric properties, gradation, unit weight, Poisson's ratio, etc.
Calibration coefficients	Default values unless mixture- or project-specific calibration is required for performance prediction.

Figure 3. Input Levels and Parameters for AASHTOWare Pavement ME Design Software (AASHTO, 2020)

Level 1 inputs provide the highest accuracy but are the most resource-intensive, as they provide project-specific and directly measured inputs. Level 2 inputs are estimated inputs based on correlations or regional data. These provide both moderate accuracy and moderate effort. Level 3 inputs are the default or assumed values based on regional or national averages. These inputs are the least accurate, but most cost- and time-effective.

The Level 3 (default) values within the software are based upon a global calibration conducted using a set of flexible pavement sections with a variety of input parameters intended to represent a wide range of circumstances. This process, however, did not consider any recycled sections, as is further expanded upon in Section 2.6. Therefore, all default values are based upon HMA.

The MEPDG pavement design method is becoming increasingly implemented. In a 2023 report by the Federal Highway Administration (FHWA), it was listed that eight states have implemented the ME design method as the primary design method, with nine other states using ME design methods in conjunction with other methods (Figure 4). Twenty-seven states indicated they were planning on implementing the design method. The six states remaining have no plans to implement the software with some states such as California and Minnesota having developed their own software for pavement structural design.

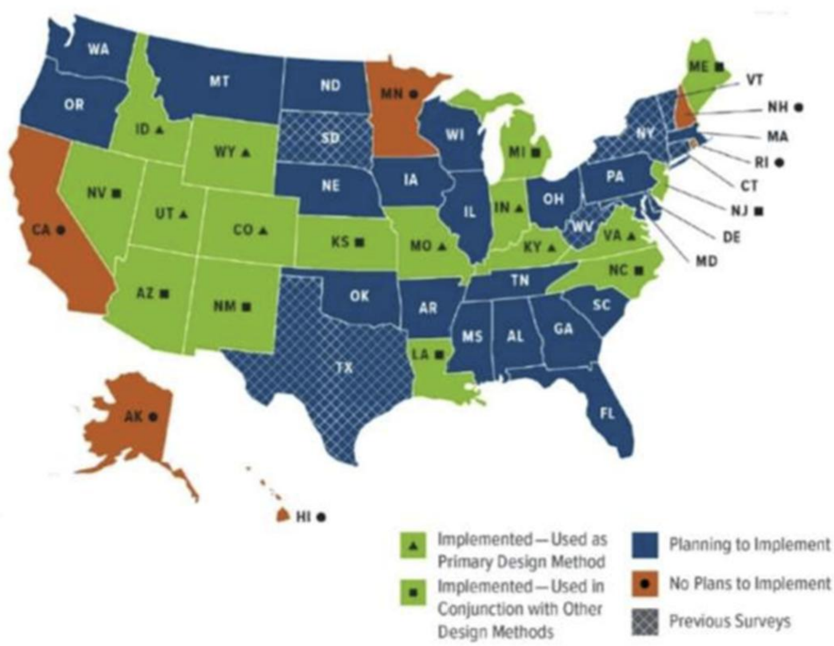


Figure 4. Status of ME Design Methods Implementation (FHWA, 2023)

Many of the states implementing the ME design method have conducted their own local calibration process. This local calibration process utilizes roadway sections from a state (or region) and inputs the traffic, climate, and layer property information into the software. Predicted pavement distresses are then produced using the globally calibrated coefficients within the transfer functions and are compared to the measured distresses from the selected roadway sections. The calibration coefficients within the transfer functions are then altered to best match the predicted and measured distresses, and the final set of coefficients are considered “locally calibrated”.

Development of Mixture-Specific k-Coefficients for Predicted Rutting Model

The rutting transfer function used to determine predicted total rutting within the ME design method uses many inputs, including mixture-specific k-coefficients: K_1, K_2, and K_3. These coefficients are determined from repeated loading permanent deformation (RLPD) testing. When plotted, the data produced from an RLPD testing produces a curve that describes the accumulation of plastic axial strain over increasing loading cycles. This curve can be divided into three stages, as shown in Figure 5.

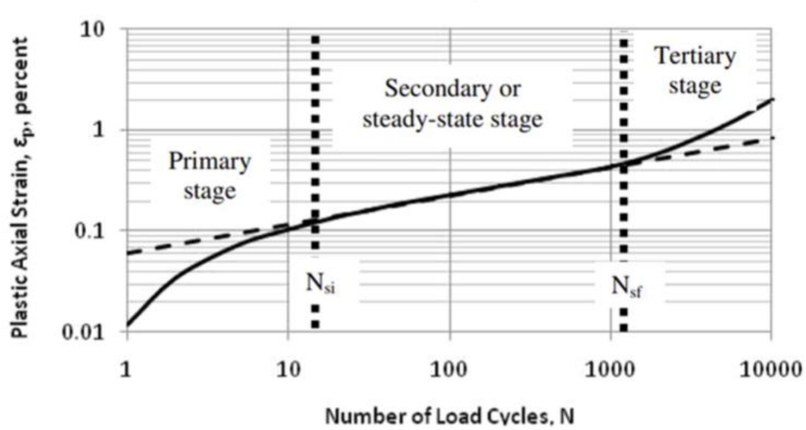


Figure 5. Example Plastic Strain Accumulation Progression Curve from Unconfined RLPD Test (Von Quintus et al., 2012)

The primary stage is the portion of the plastic axial strain curve where the slope is negative. The secondary, or "steady state" stage, is the middle part of the curve with a nearly constant slope, and the tertiary stage is the end of the curve where accumulated strain rapidly increases with loading. The secondary stage of the curve is what is used to determine the three k-coefficients. The K_1 coefficient is the intercept of the secondary zone, with a lower intercept value indicating a lower predicted rut depth. The K_2 coefficient is the temperature exponent and is assumed to be not dependent on time. Lower K_2 values indicate mixtures that the plastic strain is less sensitive to temperature, resulting in lower predicted rut depths. Lastly, the K_3 coefficient is the slope of the secondary zone, with a lower slope value indicating lower rut depths and growth rates.

2.6 History of Cold Recycled Pavement Structural Design

Cold recycling has been implemented across the United States for many decades, with continuous evolution of equipment and improvement to design and construction practices over the years. As with any pavement structure, establishing characteristics for each layer (HMA, CR pavement, etc.) is important in creating a durable road designed to last its anticipated life span. The ways of designing pavements containing CR layers can be placed into two primary methods. These are the utilization of the Mechanistic-Empirical Pavement Design software and calculating layer coefficients via the AASHTO 1993 method. Despite these being the two major methods of pavement design across the United States, only a small portion of states regularly publish their current methods of design for CR pavements.

The publicly available literature noted that many states are actively transitioning to using the PMED software for their pavement design process, with others already actively implementing it. The PMED software takes traffic, climate, material, and layer inputs and uses the ME design method to produce predicted pavement distresses to aid in the pavement structural design selection. Two states that are currently using the PMED software for CR designs are Indiana Department of Transportation (INDOT) and Virginia Department of Transportation (VDOT). For the state of Indiana, CCPR is modeled as a flexible layer with Level 1 inputs and includes dynamic modulus values collected from accelerated pavement testing. In the pavement designs, INDOT (Jones and Cosenza, 2023) noted that to achieve the same structural performance, replacing a HMA base layer with a CR mixture required a thickness increase of the layer by approximately 25%. As for VDOT, the current software implementation treats CCPR and CIR layers as flexible base layers, with input parameters such as air voids and dynamic modulus being modeled with VDOT specific default values for HMA defaults. No detail from either DOT was publicly available on if unique considerations that differ from conventional HMA/WMA are taken.

While the transition to PMED software is becoming more frequent, some states still conduct their CR pavement designs under the 1993 AASHTO design method. A few of these states are New Mexico, Wisconsin, and New Hampshire. Much like the states who are actively using the PMED software for pavement structure design, the states using the 1993 AASHTO design method rarely provided details on the development of their structural coefficients for CR layers on public interfaces, but rather just reported the structural number. New Mexico DOT reported using a structural layer coefficient of 0.30 to 0.35 assigned to all CR layers, which was developed following AASHTO thickness design procedures. No detail on the development of the coefficient or the data used to do so was provided (Gallegos and Johnson, 2019). Wisconsin DOT listed cold in-place recycling explicitly within their provided data and provided the same structural number range as New

Mexico DOT of 0.30 to 0.35. New Hampshire DOT, unlike the two previously mentioned states, published the process followed in the development of their structural layer coefficients (Dave et al., 2019). For CR layers, a structural coefficient of 0.40 is used at 50% reliability, and 0.28 at 95% reliability. The determination process included four CCPR mixtures that were initially subjected to resilient modulus, semi-circular bend, and direct tension cyclic fatigue testing. Three methods were then used to determine layer coefficients: back calculation based on resilient modulus testing, field distress data analysis, and mixture property base predictive modeling for layer coefficients. Ultimately a combination of laboratory performance results with field distress data was used to develop the structural layer coefficient for CR mixtures. This same development process, along with some additional lab testing information, was used to determine structural coefficients for HMA layers as well.

As indicated by the status of ME design methods implementation map show in Figure 4 above, five states are not planning to implement the design method. Two of these states, Minnesota and California, have developed their own respective pavement design software. Much like previous versions of the PMED software, the design software developed by Minnesota (MnPave) treats CIR and CCPR as a stabilized aggregate base layer. Chapter 4 of the Minnesota Department of Transportation (MnDOT) Pavement Design Manual (MnDOT, 2019) details the recommended decision-making process for use of CR for reconstruction and new construction of HMA pavements. No material inputs unique to CR were listed. California implements the ME design principles but in their own software, CalME. CalME provides more detailed differentiation between types of CR than any other documented pavement design software. It treats CR containing foamed asphalt or emulsified asphalt as an “asphaltic bound material”. The characterization of these CR layers follows the same methods as HMA, using beams and cores through laboratory testing. (University of California Pavement Research Center, 2023). The CalME Help document lists CR treated with cement separately as a “non-asphaltic bound material” and categorizes it with cement treated full depth reclamation, and other cement treated bases. No details were provided on the characterization of this material. The level in which CR pavements have been included in the calibration of the CalME software is more extensive than other documented pavement design software; however, no documentation was provided as to whether the thresholds and/or testing methods of the material differed from typical HMA.

2.7 Calibration of the Flexible Pavement Distress Prediction Model Coefficients

Each distress within the flexible pavement type has its’ own equation or set of equations that take traffic, climate, layer thickness, and material property data and output distress predictions for use in pavement structural design determination. The calibration process for the PMED software involves the adjustment of calibration coefficients within each distress transfer function. The goal of these adjustments is to best match the predicted performance with the measured field performance of the set of pavement sections selected for the calibration process.

2.7.1 Bottom Up Cracking

For the bottom up cracking distress prediction, a single transfer equation is used (Equation 1).

$$\mathrm{FC}_{Bottom} = \left(\frac{1}{60}\right)\left\{\frac{C_4}{1+e[\mathrm{C_1 C_1}^{*}+\mathrm{C_2 C_2}^{*}\log(DI_{Bottom}*100]}\right\} \quad (1)$$

Where:

FC_{Bottom} = area of cracking that initiates at the bottom of the AC layers (% of lane area)

DI_{Bottom} = cumulative damage index at the bottom of the AC layers

C_1, C_2, C_4 = transfer function regression constants

$C_1^* = -2C_2^*$

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

Within the PMED software, all three transfer function regression coefficients can be adjusted. The C_1 coefficient pertains to the initial cracking, whereas the C_2 coefficient pertains to the slope and progression of the cracking. C4 is a scaling constant and is not recommended to be adjusted in the calibration process (AASHTO, 2010).

To determine the cumulative damage index at the bottom of the AC layers (DI_{Bottom}) Equation 2 is used.

$$\mathrm{DI}_{Bottom} = \sum \left(\frac{n_i}{N_f}\right) \tag{2}$$

Where:

n_i= actual applied load repetition

N_f = allowable number of load repetitions

To determine the allowable number of load repetitions, Equation 3 is used.

$$\mathrm{N}_f = 0.00432 * \mathrm{C} * \beta_{f1} * k_1 \left(\frac{1}{\varepsilon_1}\right)^{k_2\beta_{f2}} \left(\frac{1}{E}\right)^{k_3\beta_{f3}} \tag{3}$$

Where:

N_f = allowable number of axle load applications

ε_1 = tensile strain at critical location, in/in

E = dynamic modulus of the material, psi

k_1, k_2, k_3 = Global field calibration parameters

β_{f1}, β_{f2}, β_{f3} = local or mixture specific field calibration constants

C = 10^M

$$M = 4.84\left(\frac{V_{be}}{V_a+V_{be}} - 0.69\right) \quad (4)$$

Where:

V_{be} = effective asphalt content by volume (%)

V_a = percent air voids in the mixture

Inputs into the equation for cumulative damage index at the bottom of the AC layers (DI_{Bottom}) were not altered in this research and were therefore left at the globally calibrated default value.

2.7.2 Total Rutting

For the total rutting distress prediction, two transfer equations are used. First, the total rutting in the AC layers is defined (Equation 5).

$$\Delta_{p(AC)} = \beta_{1r} k_z \varepsilon_{r(AC)} 10^{k_{1r}} n^{k_{3r}\beta_{3r}} T^{k_{2r}\beta_{2r}} \quad (5)$$

Where:

$\Delta_{p(AC)}$ = area of cracking that initiates at the bottom of the AC layers (% of lane area)

$\varepsilon_{r(AC)}$ = resilient or elastic strain calculated by the structural response model at the mid-depth of each AC sublayer, in/in

n = number of axle load applications

T = mix or pavement temperature, °F

k_z = depth confinement factor

k_{1r}, k_{2r}, k_{3r} = laboratory-derived coefficients for dense-graded neat AC mixtures

β_{1r}, β_{2r}, β_{3r} = local or mixture calibration or field-shift constants

Next, the total rutting in the unbound pavement sublayers and subgrade are determined using Equation 6.

$$\Delta_{p(soil)} = \beta_{s1} k_{s1} \varepsilon_v h_{soil} \left(\frac{\varepsilon_o}{\varepsilon_r}\right) e^{-\left(\frac{\rho}{n}\right)^{\beta}} \quad (6)$$

Where:

$\Delta_{p(soil)}$ = permanent or plastic deformation for the layer/sublayer, in

n = number of axle load applications

ε_o = intercept determined from laboratory repeated load permanent deformation tests, in/in

ε_r = resilient strain imposed in laboratory testing to obtain material properties ε_o, β, and ρ, in/in

ε_v = average vertical resilient or elastic strain in the layer/sublayer and calculated by the structural response model, in/in

h_{soil} = thickness of the unbound layer/sublayer, in

k_{s1} = material or soil dependent model coefficient

β_{s1} = calibration constant for the rutting in the unbound layers (granular base and subgrade)

Coefficients β_{1r}, β_{2r}, β_{3r}, and β_{s1} can all be adjusted in the PMED software. Adjustment of β_{1r} is associated with the overall rutting, β_{2r} with the impact of climate and temperature on the predicted rutting, and β_{3r} with the influence of traffic loading on the predicted rutting. Each individual AC layer has a β_{1r}, β_{2r}, and β_{3r} coefficient associated with it that can be adjusted. Both the granular base and the subgrade layers have a β_{s1} coefficient that can be adjusted independently of each other.

Chapter Three: Calibration of Pavement Distress Prediction Model Coefficients for Cold Recycled Pavements

3.1 Introduction and Objectives

CR pavements are becoming increasingly studied and more frequently used by state agencies and industry as a technique for new pavements, pavement rehabilitation, and pavement preservation. With this in mind, understanding the structural design process using historical pavement performance data in addition to mixture specific mechanistic structural responses is becoming particularly relevant. A CR layer design template is available within the flexible pavement design module in the current AASHTOWare Pavement ME Design software (version 3). However, the existing default values within the software used for distress prediction and optimized design are based on HMA values. Typically, material properties for HMA are not like those found for CR mixtures and using them in design may lead to pavement structure that overperforms or underperforms the predictions. This presents an opportunity to evaluate the globally calibrated model coefficients and their ability to accurately predict pavement distresses for flexible pavements containing a CR base or intermediate layer.

The objectives of this study are as follows:

1. Evaluate the existing global calibration coefficients in the PMED software version 3.0 for their suitability in accurately predicting pavement distresses for flexible pavements containing a CR base or intermediate layer and determine what, if any, calibrations are needed.
2. Assess model sensitivity to select mixture specific properties.
3. Optimize the calibration coefficients based upon the previous evaluation of the model with global coefficients to accurately predict pavement performance.

3.2 Methodology

3.2.1 Section Selection and Data Compilation

For this effort, flexible pavements containing a CR layer were selected to determine their suitability for use in calibration coefficient optimization. For a section to be considered for use in the model calibration, it was required to meet a certain set of criteria based on the inputs requested in the software. For any calibration effort, performance and structural data are required inputs (AASHTO, 2010), and are thus required for this study. In addition to these inputs, sections must have been placed a minimum of 5 years prior to this calibration process and have dynamic modulus data on the CR layer to be considered. A minimum of 5 years was selected to provide adequate time for oxidation, weathering, and any potential distresses to begin to occur and be documented. Dynamic modulus data was selected as a requirement as it is one of the input material properties that has a larger influence over the distress prediction and has a strong correlation to field performance (Yu and Shen, 2012). Once the sections meeting the previously listed criteria were identified, the associated pavement layer information, laboratory mixture performance data, and distress data were compiled from previous publications and other resources for use in the calibration process. Table 1 details the material data inputs considered in this research. Mixture-specific data was used when available. Otherwise, the default values were used.

Table 1. Selected Mixture-Specific Material Inputs for Model Calibration

Mixture Volumetrics	Air Voids (%)
	Effective Binder Content by Volume (%)
	% Asphalt Content by Weight of Mix (%)
	Unit Weight (pcf)
Mechanical Properties	Dynamic Modulus
	Asphalt Binder G* Data
	Indirect Tensile Strength (psi)
Mix-Specific k-Coefficients	K1, K2, K3

For pavement distress data, total rut depth (in) and fatigue cracking (% lane area) measurements were used in the calibration process, as total rutting (in) and bottom up cracking were the selected distress prediction models for calibration. These two distress prediction models were selected based on recommendations from the AASHTO Local Calibration Guide (2010). International Roughness Index (IRI) measurements were not considered as influential factors for new construction as they are often associated with construction practices and/or mixture inconsistencies with the original design. If IRI predicted performance requirements are not met within a new pavement design, adjustments to the layer thicknesses would be made as both cracking and rutting models are inputs into the IRI model. This adjustment of layer thickness does not correlate with the measures taken in the field to improve IRI measurements, as layer thickness does not impact the IRI of newly constructed pavements.

Thermal cracking and reflective cracking were also not considered, as forensics were not available to identify the origins of the measured cracking for the sections used in this calibration process. All cracking was assumed to be bottom up cracking per recommendations in the AASHTO Local Calibration Guide (2010) that suggest combining cracking types if the location where cracking initiated is not known and thus adjusting the bottom up cracking model to fit the data. It should be noted that this recommendation is based upon the knowledge of how HMA/WMA pavements function and how typical distresses form, as these pavement types are what was used in the global calibration effort and in the development of default values. Little research has been published on the failure mechanisms related to cracking for CR pavements; however, the existing published research indicates that while bottom up cracking can be present (Jones et al., 2024; Ahmed et al., 2022), categorizing all cracking as such may not accurately capture the entirety of the deterioration seen in CR layers. Discussions with state agencies and industry members on the limited amount of data on how distresses form and what distresses typically present specifically in flexible pavements containing a CR layer indicate no documented difference from typical flexible pavements without a CR layer. Therefore, the processes suggested in the local calibration guide were followed in this study.

3.2.2 Evaluation of Global Calibration Coefficients

After the sections were selected and the associated data was compiled, the data were input into the PMED software to generate predicted pavement performance using global coefficients (Appendix C). The predicted distress values were compared to measured distress values collected from sections to assess the accuracy of the global coefficients within the pavement distress models. Based on these comparisons, the bias (Equation 7) and standard error of the estimate (SEE) (Equation 8) were evaluated.

These terms are defined as the systematic offset between predicted and observed values and the variability between the predicted and measured values, respectively (AASHTO, 2026). These statistics are recommended for assessment as per the local calibration guide as they provide indicators of the accuracy of the predicted distress data.

$$Bias = \text{measured data} - \text{predicted data} \tag{7}$$

$$SEE = \sqrt{\sum \frac{(y-\hat{y})^2}{n-2}} \tag{8}$$

Where:

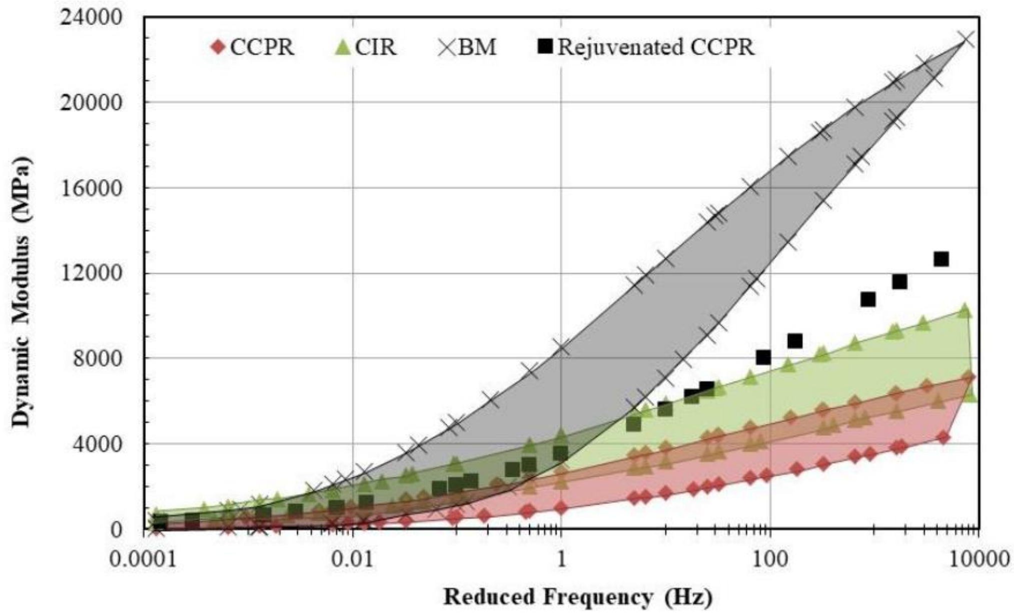
y = actual measured value

$\hat{y}$ = predicted value

n = total sample size

3.2.3 Assessment of Model Sensitivity to Mixture Specific Inputs

When compiling existing data on the selected sections for use in the calibration coefficient optimization, many site-specific, laboratory measured Level 1 data were available. However, for some material properties, data was unavailable and Level 3 (default) inputs were used. The default material properties within the PMED software are based on data collected in the global calibration process. As mentioned previously, this process did not include any CR sections, and the default values were based entirely upon HMA/WMA properties. One of these cases in which this was true was the mix-specific k-coefficients. Of the 13 sections, five did not have mix-specific k-coefficients and would therefore use the default values. For those five sections, a sensitivity analysis was conducted to evaluate the influence of the mix-specific k-coefficients by comparing the predicted distress results using the global coefficients versus k-coefficients from similar mixtures (e.g. foamed asphalt CCPR mixture used k-coefficients from another foamed asphalt CCPR mixture). Based on other mixture-specific data, CIR and CCPR materials tend to behave more similarly to each other as compared to CR and HMA/WMA. For example, E* testing of 15 CIR mixtures and 3 CCPR mixtures (Figure 6) shows more similar results to each other than a typical HMA base mixture (BM) (Bowers et al. 2019), thus presenting a hypothesis that substituting similar mixture properties versus using the default values based upon HMA/WMA data may be more suitable for better predicting mixture performance.



**BM=Base Mix*

Figure 6. Dynamic Modulus Master Curve Showing Rejuvenated Cold Recycled Mixture Data Falling Between Typical CR and HMA (Bowers et al., 2019)

3.2.4 Optimization of Calibration Coefficients

Once the distresses using the global coefficients were produced and the data was assessed, an optimization process was conducted. This process resulted in the calibration of distress model coefficients to best fit the predicted distresses to the measured distresses from the selected sections. When evaluating the fit of the distress models and determining the need for coefficient optimization, two primary parameters are considered: bias and SEE. The bias is the average of the individual residual errors, the difference between the measured and predicted distresses for each section. SEE is the standard error of the estimate. During the optimization process, the goal is to minimize residual error and SEE.

Typically, when calibrating the transfer function coefficients, the Calibration Assistance Tool (CAT) (AASHTO, 2026) is used. This tool produces bias and SEE values and allows for efficient modification of coefficients to optimize the model. For this study, the minimum amount of pavement sections (30 sections for bottom up cracking and 21 sections for rutting) needed to use the CAT was not met. Therefore, the calibrations and calculations were done manually using Microsoft Excel. Selected calibration coefficient iterations were input into the PMED software to produce predicted distress values. These values were then input into Microsoft Excel and compared to measured distresses and statistics were calculated to evaluate the accuracy of the model and to determine the need for further calibration coefficient refinement. Another part of the recommended procedure within the local calibration guide is to use 80% of the sections selected for the calibration effort in the optimization process and reserve the remaining 20% for the validation of locally calibrated coefficients. However, since the minimum number of sections recommended for this process was not met, a "partial jackknife approach" was used. This approach is a (n-1) method, meaning a single random section was removed from the data set and used in the validation process after the calibration coefficients were optimized using the remaining (n-1) sections. The order in which coefficients were optimized and the intended impact they would have on the bias and SEE can be found in the AASHTO Local Calibration Guide (2010).

3.3 Results and Discussion

3.3.1 Section Selection, Section Background, and Data Compilation

After evaluating known pavements containing a CR layer, 13 were selected that met these criteria. At the time of this study, no documentation was found pertaining to the calibration of the PMED software for flexible pavements using CR. Table 2 below lists the sections selected for use in calibration. Table 3 lists the pavement layer thicknesses for each section. The number of measured data points varied per section. In the case of test track related sections, distress measurements were taken approximately once per month.

Other sections had a single collected measurement per distress model. The CR techniques represented include CIR and CCPR, and mixtures containing foamed asphalt with cement and various emulsion types with and without a rejuvenator. 11 of the 13 sections followed traditional mixture design methods for their respective asphalt stabilizing agents, and the remaining two contained a rejuvenator. CR mixture design using a rejuvenator is not incorporated into typical CR specifications and has no standard inclusion method at the time of this research. However, research using rejuvenators in CR mixtures is expanding and thus was included in this study.

Table 2. Selected Section Details for Model Calibration

Section Location	Section ID	Material Type	Construction Year
NCAT Test Track	N4	CCPR (Foamed Asphalt)	2012*
NCAT Test Track Off-Ramp	R6	CCPR (Foamed Asphalt)	2021
NCAT Test Track Off-Ramp	R7	CCPR (Engineered Emulsion)	2021
NCAT Test Track Off-Ramp	R8	CCPR (Emulsion + Rejuvenator)	2021
NCAT Test Track Off-Ramp	R9	CCPR (CR Rejuvenator)	2021
US 280	40	CCPR (Foamed Asphalt)	2015
US 280	41	CCPR (Engineered Emulsion)	2015
US 280	43	CIR (Engineered Emulsion)	2015
US 280	44	CIR (Foamed Asphalt)	2015
70th Street (MnROAD)	7003E	CIR (Foamed Asphalt)	2019
70th Street (MnROAD)	7004E	CIR (Engineered Emulsion)	2019
70th Street (MnROAD)	7005E	CCPR (Engineered Emulsion)	2019
70th Street (MnROAD)	7007E	CCPR (Foamed Asphalt)	2019

**section was reconstructed in 2024, therefore distress data from 2012-2024 was used*

Table 3. Layer Thicknesses by Section

Section	Layer Thickness (in)	Layer Type
N4	1.62	Surface HMA
	1.97	Intermediate HMA
	4.58	CCPR
	6.41	Aggregate Base
	Infinite	Subgrade
R6, R7, R8, and R9	1.00	HMA Thinlay
	4.00	CCPR
	6.00	Aggregate Base
	Infinite	Subgrade
70th Street - 7003E, 7004E, 7005E, 7007E	1.00	HMA Thinlay
	3.00	CIR/CCPR
	1.00	HMA Base
	6.00	Aggregate Base
	Infinite	Subgrade
US 280 - Section 40	1.00	HMA Thinlay
	4.00	CCPR
	3.40	HMA Base
	11.10	Aggregate Base
	Infinite	Subgrade
US 280 - Section 41	1.00	HMA Thinlay
	4.00	CCPR
	8.70	HMA Base
	4.40	Aggregate Base
	Infinite	Subgrade
US 280 - Section 43	1.00	HMA Thinlay
	4.00	CIR
	12.50	HMA Base
	Infinite	Subgrade
US 280 - Section 44	1.00	HMA Thinlay
	4.00	CIR
	7.80	HMA Base
	Infinite	Subgrade

Section N4, sponsored by VDOT, was placed on the National Center for Asphalt Technology (NCAT) Test Track in 2012 as a part of a three-section experiment exploring cold recycling as a layer within the pavement structure in a high traffic volume, heavy loading scenario. The structural composition, mixture information, laboratory testing data, and pavement performance data used in this dissertation is summarized in a publication currently under review for publication in the Transportation Research Record (TRR).

Sections R6-R9 on the NCAT Test Track off-ramp are a part of a cold recycling study in which each section consisted of a different CR mixture placed on top of an aggregate base, with a 1-inch HMA thinlay (an HMA mixture with a nominal maximum aggregate size of 4.75mm) placed on top. All sections have been in place since 2021 and were being subjected to loading in their second test cycle at the time this research occurred. The distress data to date has not been published; however, the structural composition, mixture information, laboratory testing data, and mixture specific properties can be found in a TRR publication (Turochy and Bowers, 2024) and within a master's thesis (Vangala, 2025).

The four sections on US 280 are a part of a larger pavement preservation study conducted by NCAT. The structural information, mixture information, laboratory information, and performance data pertaining to this dissertation have been published in NCAT Report 24-02 "Findings from the Pavement Preservation Group (PG) Study" (Vargas-Nordcbeck et al. 2024) and within Gu et al. (2019)

The four sections placed on 70th Street were a part of a larger CR and FDR study at the MnROAD testing facility in partnership with NCAT. The structural information, mixture properties, and laboratory testing data associated with this dissertation was published in Allain et al. (2022). The performance data was published within the previously mentioned NCAT Report 24-02.

The traffic data for the NCAT Test Track sections was documented in a master's thesis (Guo, 2013). The traffic inputs used for both US 280 and 70th Street were collected from the Alabama Department of Transportation website (ALDOT, 2026) and the Minnesota Department of Transportation website (MNDOT, 2026), respectively.

It should be noted that during the initial site selection process, structures containing both a CR layer and full depth reclamation (FDR) were considered; however, with the version of the software used in this dissertation, adding an FDR layer switches the model from flexible to semi-rigid. Including sections with FDR would require a second complete model evaluation and calibration with an increased amount of material inputs. The inputs, like with CR layers, are largely based on average data from non-recycled materials. Adequate data on FDR specifically for these inputs is not readily available for this research, thus the sections containing CR and FDR were not selected for use in calibration.

3.3.2 Evaluation of Global Calibration Coefficients

Once the associated data with the calibration sites was input into the software and the predicted distresses using the global calibration factors were output, a comparison between the predicted and measured distresses was conducted. This comparison was done across two different distresses: bottom up cracking and total rutting. These two distress prediction models were selected based upon recommendations from the AASHTO Local Calibration Guide (AASHTO, 2010). For the calibration process, predicted distresses at 50% reliability were used. Bias and SEE were used to evaluate the model. The ratings (e.g. Very Good, Good, etc.) associated with the SEE statistics reported for each distress model evaluated are based upon a rating system produced by Applied Research Associates (ARA) as guidance for interpretation of the SEE as a part of the local calibration process. These ratings are anticipated to be included in future PMED software publications. Table 4 details these ratings.

Table 4. Guidance for Rating SEE Values

Range of Statistical Value	Rating or Assessment
< 0.35	Very Good
0.35 to 0.55	Good
0.56 to 0.75	Fair
0.76 to 1.0	Poor
> 1.0	Very Poor

The AASHTO Local Calibration Guide (2010) provides a table of recommended flexible pavement calibration coefficients to be adjusted to best optimize the model. These recommendations were followed during this calibration and are shown in Appendix B.

For this research, only calibration coefficients within the transfer functions were optimized and alterations to the transfer functions were not explored. This is based on the typical calibration process (AASHTO, 2010). As mentioned previously the default values found within the software are based on HMA, and the transfer functions themselves are no different. The origins of the transfer functions come from empirical data collected from pavement sections, none of which contained CR materials. The use of these transfer functions to determine CR mixture performance may not adequately predict performance. Additional research into the suitability of these transfer functions for CR pavements would be beneficial.

Bottom Up Cracking

The initial evaluation of the bottom up cracking model showed a positive bias and a very poor SEE. The predicted pavement distress model using the global coefficients overpredicted bottom up cracking, as shown by the series of data points trending on the left side of the line of equality towards the y-axis in Figure 7. The poor SEE shows poor accuracy of the prediction model when evaluated along the linear regression line. The visual overprediction of the data combined with the poor SEE indicates a need for calibration coefficient optimization.

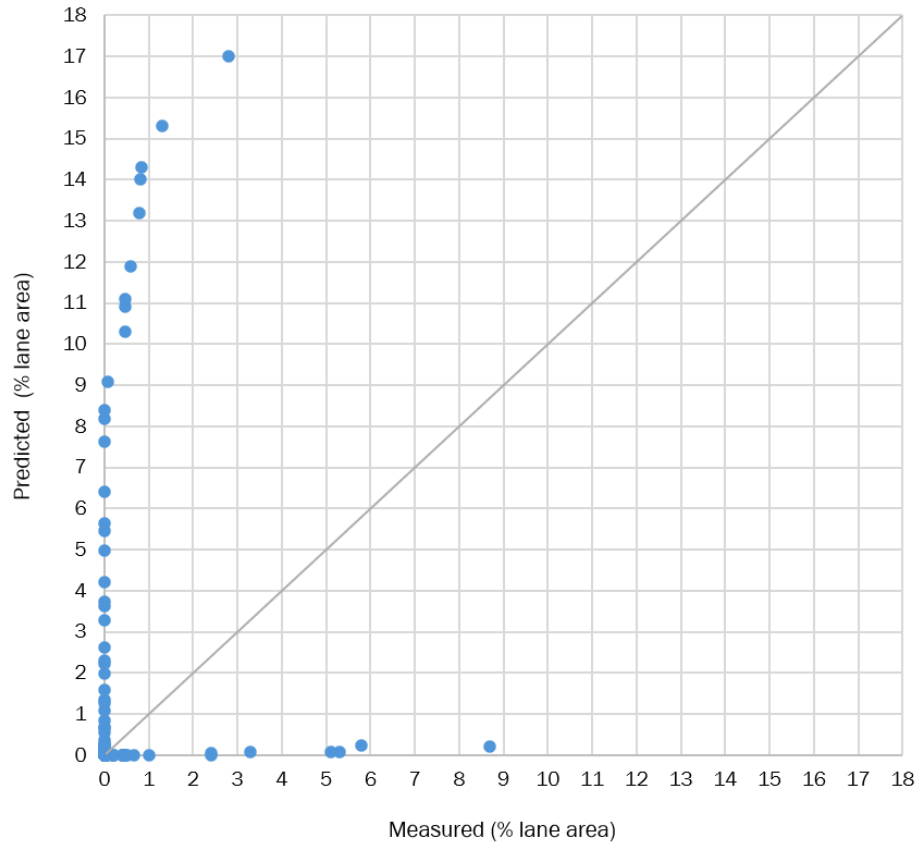


Figure 7. Evaluation of Bottom Up Cracking Model Using Global Coefficients

Total Rutting

The initial evaluation of the total rutting model showed a negative bias and a very good SEE. The predicted pavement distress model using the global coefficients showed overprediction in rutting in almost all data points and is seen in the data trending on left side of the line of equality towards the y-axis in Figure 8. The very good SEE indicates accuracy of the prediction model when evaluated along a linear regression line. Despite the very good SEE, the visual overprediction of the rutting data indicates a need for calibration coefficient optimization.

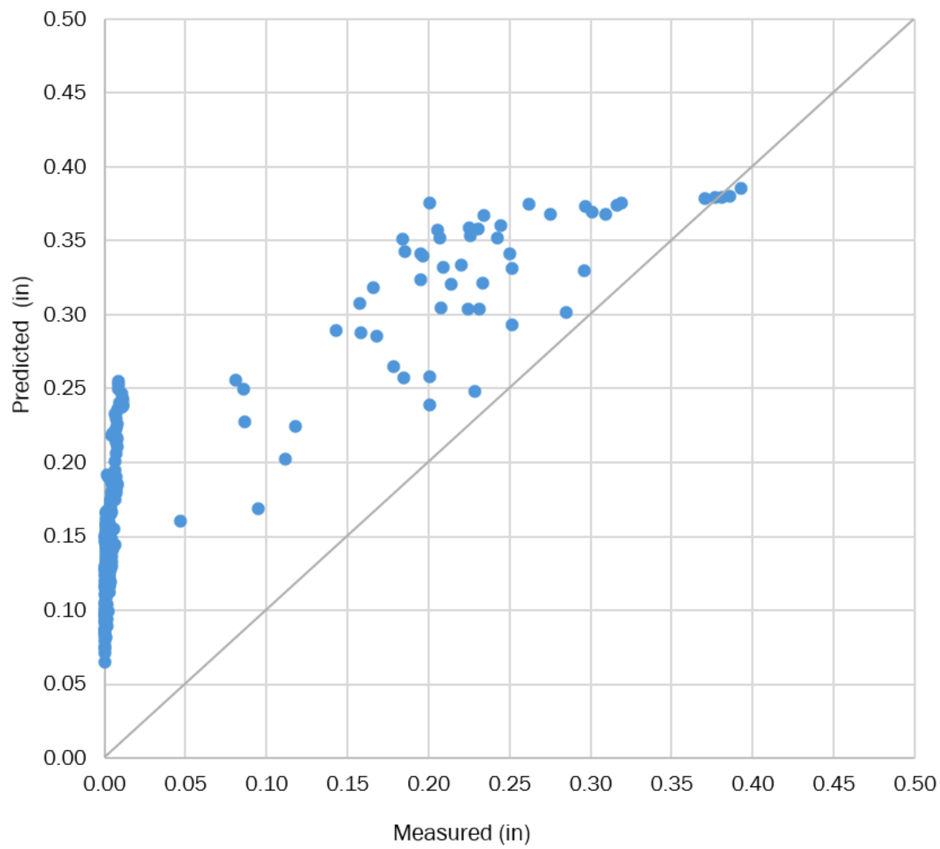


Figure 8. Evaluation of Total Rutting Model Using Global Coefficients

3.3.3 Assessment of Model Sensitivity to Mixture Specific Inputs

Mixture-Specific K-Coefficients

For five of the 13 sections selected for the calibration process, mixture-specific k-coefficients were not available for the CR layer. To determine the sensitivity of the model to these coefficients, data from the five sections were input into the software first with the global coefficients, then with coefficients for similar mixtures. As these mixture-specific k-coefficients are inputs into the rutting model transfer functions, only the impact on the total rutting prediction was evaluated.

Within the sensitivity analysis, three different sets of mixture-specific k-values were evaluated to determine their influence on the resulting predicted total rutting. The first set of coefficients were the global (default) values within the software, which were developed based on HMA/WMA data. The second and third sets were coefficients from the CR sections on the NCAT Test Track off-ramp (Vangala, 2025) and from the CR sections on US 280 (Gu et al., 2019), respectively. The off-ramp coefficients determined by Vangala (2025) were calculated based on temperature shifted data. RLPD testing was conducted at a single temperature and accumulated plastic strains at other temperatures were estimated using a temperature shift procedure, which relied on shift factors derived from dynamic modulus data. All k-coefficients were derived from CR mixture testing. For the k-coefficients calculated for US 280 sections, HMA and CR mixtures were assumed to have comparable thermal characteristics, which set the K2 coefficient at the same value as the global K2 at the time the research was published. The K1 and K3 coefficients were based on measured strain collected from RLPD testing of the CR materials from each section. The coefficients for each section were selected based on mixture stabilizing agent type (e.g. coefficients from a mixture containing foamed asphalt were used for other foamed asphalt mixtures). Table 5 shows the k-coefficient sets tested.

Table 5. K-Coefficient Sets Tested

Section Basis	Mixture Type	K1	K2	K3
Global	Emulsion	-2.45	3.01	0.22
	Foamed Asphalt			
US 280	Emulsion	-3.38	1.56	0.47
	Foamed Asphalt	-3.59	1.56	0.47
Test Track	Emulsion	-2.05	1.22	0.36
	Foamed Asphalt	-3.54	1.78	0.37

Once the predicted total rutting performance was produced for each section, holding all other inputs constant and only varying the k-coefficients, the results were evaluated both practically and statistically. In the case of all three sets of coefficients, total rutting was overpredicted. Use of k-coefficients from the CR mixtures on the Test Track off-ramp produced results closest to the measured total rutting for each section; however, all three sets of coefficients significantly overpredicted rutting for the 70th Street sections. Within each section, the predicted total rutting between differing k-coefficients was within 0.125 inches. The resulting predicted rutting for each k-coefficient set are shown in Table 6 below. Figure 9 displays the results of the k-coefficient analysis in an additional format.

Table 6. Results from K-Coefficient Sensitivity Analysis

Section	K-Coefficient	Predicted Total Rutting (in)
N4	Global Value	0.5000
	Test Track Similar Mixture Type	0.3853
	US 280 Similar Mixture Type	0.4955
	Measured Distress	0.3930
70-CIR-E	Global Value	0.3403
	Test Track Similar Mixture Type	0.3118
	US 280 Similar Mixture Type	0.3433
	Measured Distress	0.2008
70-CIR-F	Global Value	0.3458
	Test Track Similar Mixture Type	0.2247
	US 280 Similar Mixture Type	0.3491
	Measured Distress	0.1181
70-CCPR-E	Global Value	0.3692
	Test Track Similar Mixture Type	0.2481
	US 280 Similar Mixture Type	0.3201
	Measured Distress	0.2283
70-CCPR-F	Global Value	0.3780
	Test Track Similar Mixture Type	0.2991
	US 280 Similar Mixture Type	0.3134
	Measured Distress	0.0866

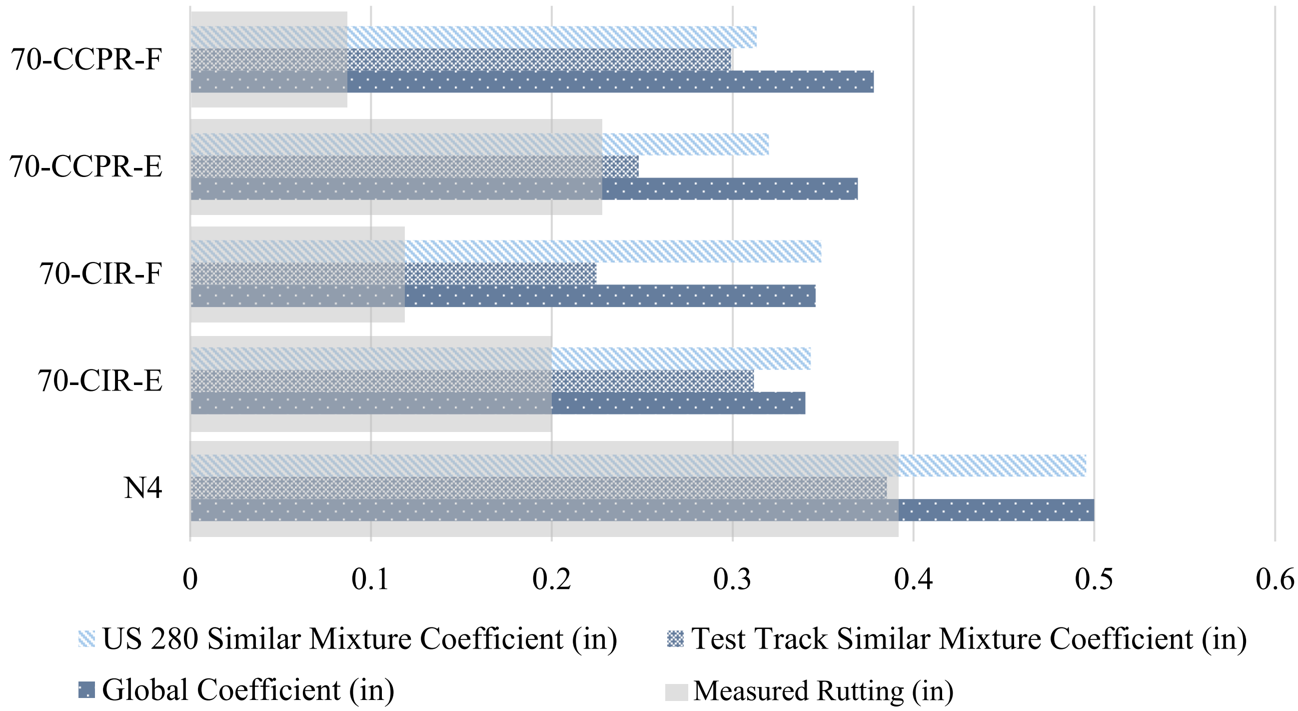


Figure 9. Results from K-Coefficient Sensitivity Analysis

A t-Test assuming unequal variances (Welch's t-Test) was conducted between each predicted total rutting data set to determine if there was any statistical significance in the results. It was determined that the rutting results produced by each k-coefficient set were statistically similar, with the exception of the global versus test track results comparison. At $\alpha = 0.05$, a p-value greater than 0.05 indicates statistically similar results. Table 7 shows the results of the statistical analysis. Based on these results, the global rutting coefficients were used for the sections that did not have mixture-specific k-coefficients. It is not common practice for mixture-specific k-coefficients to be determined for CR mixtures, which was another contributing factor in using the global coefficients.

Table 7. Results from Predicted Total Rutting Statistical Analysis

	p-value
Global vs. US 280	0.054
Global vs. Test Track	0.018
US 280 vs. Test Track	0.076

Air Voids

Infrequently, depending on the specification followed (e.g., AASHTO PP 86), air voids are reported as a by-product or requirement of the design and/or QA/QC process for CR mixtures. Due to the limited amount of air void content data available for CR mixtures, the distress model calibration was conducted using the default 7% air voids for the CR layers. However, for two of the 13 CR mixtures, air voids were recorded in the mixture design process. An analysis was performed on the two sections comparing the bottom up cracking distress predictions using the default 7% air voids versus the mixture specific air voids. In both cases, the predicted cracking was greater using the mixture specific air voids versus the global coefficients. For the first of the two sections tested this worsened the bias, and for the second it improved it. In the case of the second section, more cracking was measured than the former section and the increase in predicted cracking resulted in the slight improvement of the bias (Table 8). Ultimately, it is recommended that a larger data set be analyzed to determine the influence of air voids in a CR base or intermediate layer on the overall bottom up cracking model predictions.

Table 8. Results from Air Void Sensitivity Analysis

Bottom Up Cracking Model	Air Voids (%)	Bias (% lane area)
Section 1	Global	0.001
	Mixture Specific	0.049
Section 2	Global	-4.041
	Mixture Specific	-4.006

Historically, air voids have been used as an indicator of mixture performance for HMA/WMA and therefore carry a larger weight in the distress prediction model within the software. For CR mixtures, measurements such as density and ITS strength are more often tied to mixture acceptance and performance versus air voids (Turochy and Bowers, 2024). The current transfer functions used in the PMED software are developed around data from HMA/WMA performance and likely do not fully capture the properties most indicative of CR mixture performance. This poses potential for future CR pavement structural design advancements; however, operating with the current models, the calibration process was designed around the existing defaults for practicality considering the currently collected data on CR mixtures.

Effective Binder Content by Volume and Percentage Asphalt Content by Weight of Mixture

While many Level 1 material inputs were obtained for the calibration process, many defaults including the previously mentioned k-coefficients and air voids were used. Two other inputs that have been known to have a larger influence on mixture performance and distress prediction for HMA/WMA are effective binder content by volume and percentage asphalt content by weight of mix. These properties are typically required to be recorded as a part of the mixture design and QA/QC process for HMA/WMA, however, are not for CR mixture design due to difficulty in measurement for CR materials and were not collected for any of the sections used in this calibration process. Mixture design procedures vary by agency/industry/company, but there is no existing documentation of effective binder content by volume or percentage asphalt content by weight of mix being reported for CR mixtures; therefore, default values were used in the calibration process.

3.3.4 Optimization of Bottom Up Cracking Distress Prediction Model Calibration Coefficients

The initial evaluation of the bottom up cracking model using global coefficients showed a positive bias, indicating an overprediction of cracking versus measured distresses. This bias, however, was minimal. The SEE for the predicted distress using the global coefficients was considered very poor and therefore was addressed first. For the optimization of the calibration coefficients for the bottom up cracking model, only coefficients within the transfer function were adjusted. The coefficients associated with the damage accumulation equation were not altered, nor was the transfer function itself. This calibration method was followed based upon AASHTO recommendations (2010); however, these recommendations are made based on the assumption that all flexible layers (HMA/WMA and CR) have the same damage behavior.

Per the AASHTO Local Calibration Guide (2010), the C1 coefficient was adjusted. The C1 coefficient pertains to the initial cracking phase of the distress. The first iteration resulted in a reduction of the SEE, producing a “poor” rating. This iteration also resulted in a reduction in the bias. Two more iterations were conducted to further refine the C1 coefficient, which ultimately left the SEE remaining “poor”. Considering the C1 coefficient adjustments did not make notable improvements to the SEE, the data itself was evaluated. Figure 10 shows the distribution of the measured versus predicted distresses after the optimization iterations of the C1 coefficient.

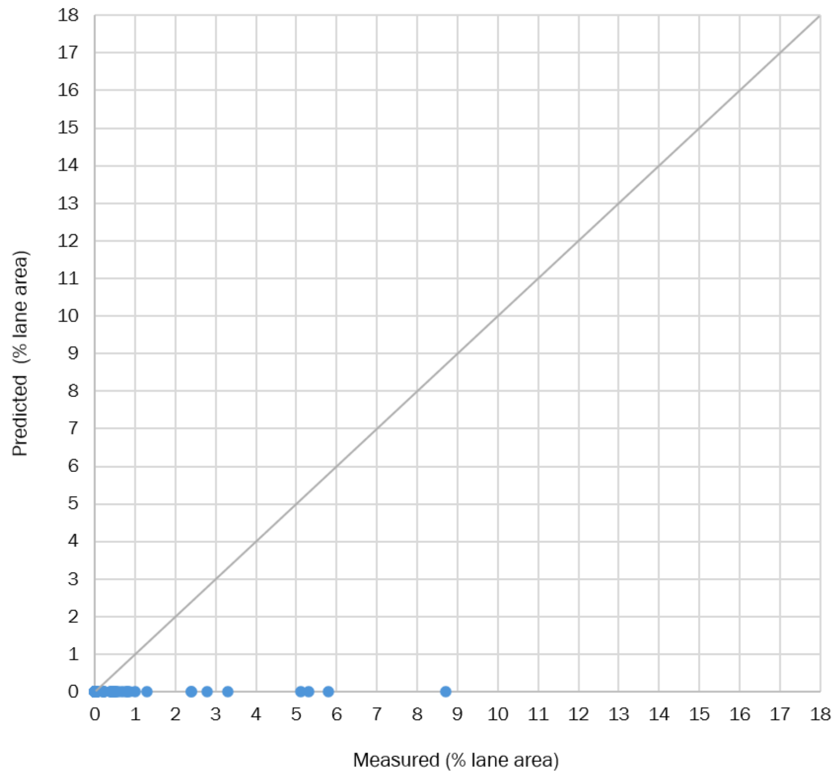


Figure 10. Bottom Up Cracking Model After C1 Coefficient Optimization

The plot shown above indicates significant improvements upon the initial model using the global coefficients, with all on the overpredicted data that was along the y-axis now adjusted so that it is clustered at the base of the line of equality. The data points that were along the x-axis in the initial model verification, however, remained there after the C1 coefficient optimization. Upon further investigation, the sections associated with those data points were the US 280 and 70th Street sections. The plot with the removal of the US 280 and 70th Street sections is shown in Figure 11, and the change in bias and SEE between the two models is shown in Table 9.

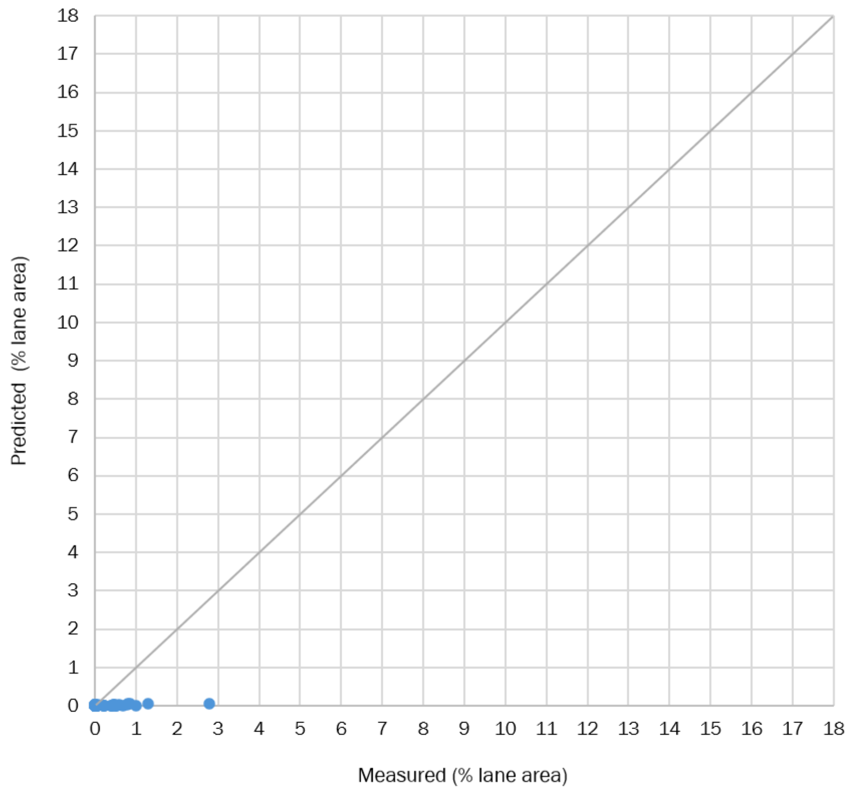


Figure 11. Optimized Bottom Up Cracking Model with US 280 & 70th Street Sections Removed

Table 9. Optimized Bottom Up Cracking Model Statistics with & without US 280 and 70th Street

Statistic	With US 280 and 70th Street Sections	Without US 280 and 70th Street Sections
Bias	0.19	0.06
SEE	0.90	0.24

Removal of the US 280 and 70th Street sections showed significant improvement in the distress prediction model statistics and the measured versus predicted distress plots. These sections, in comparison to the others, showed a much larger amount of cracking. As mentioned previously, it is recommended in the AASHTO Local Calibration Guide to combine all cracking identified and treat it as bottom up cracking, unless it is documented as another form of cracking (e.g. transverse cracking). The cracking recorded on the US 280 and 70th

Street sections was based upon surface observations and no forensics were conducted; therefore, all the cracking was considered bottom up cracking even though this characterization might not accurately depict the type of cracking occurring in the sections. To further explore this, images of the US 280 sections were analyzed visually to determine if any other form of cracking was evident from the surface observations. In all sections, evidence of transverse cracking was noted, but an exact percentage contribution was not determined. The amount of transverse cracking also varied per section. An example of transverse cracking seen on an image of a US 280 section is shown in Figure 12 below.

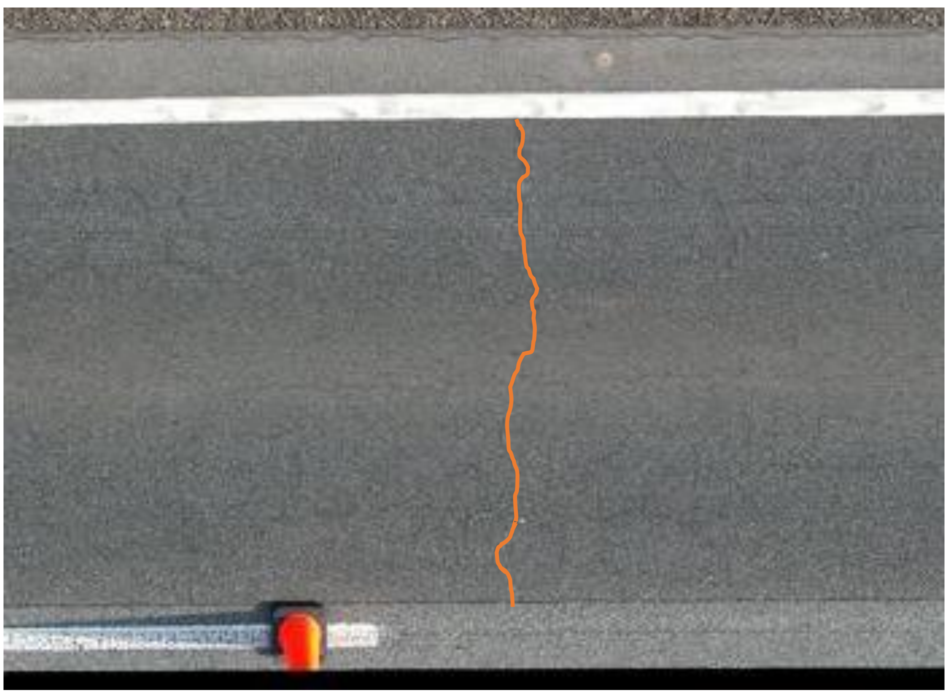


Figure 12. Example of Transverse Cracking on a US 280 Section

An additional commonality between the sections is found within the structure itself. All sections on US 280 and 70th Street contained a layer of existing HMA below the CR and HMA surface course layers. The existing HMA was milled to the appropriate depth for reconstruction but not removed entirely. The distress condition of that layer is not known, and forensics have not been conducted to determine the depth of the measured cracking on these sections. However, the age and potential for distresses to exist in that layer at the time the CR layer and HMA surface course were placed should be considered in the overall elevated cracking distress measurements versus the other sections used in this calibration process. Another consideration into the distribution of the measured cracking across prediction models is the ability to select the layer in which a specified portion of the cracking originates from. The cracking models within the software do not currently allow for specific selection of the layer in which cracking originates. This lack of specificity can pose a limitation if a pavement structure contains a variety of mixture types with varying performances in the field.

Ultimately, after the C1 refinements were conducted, the SEE could not be reduced further due to the increased cracking on the US 280 and 70th Street section. No further calibration coefficient adjustments to the bottom up cracking distress model were conducted. Figure 13 shows the predicted versus measured distress data after the coefficient optimization, including the US 280 and 70th Street sites. Table 10 details the bottom up cracking model coefficient optimization iterations and their resulting impact on bias and SEE.

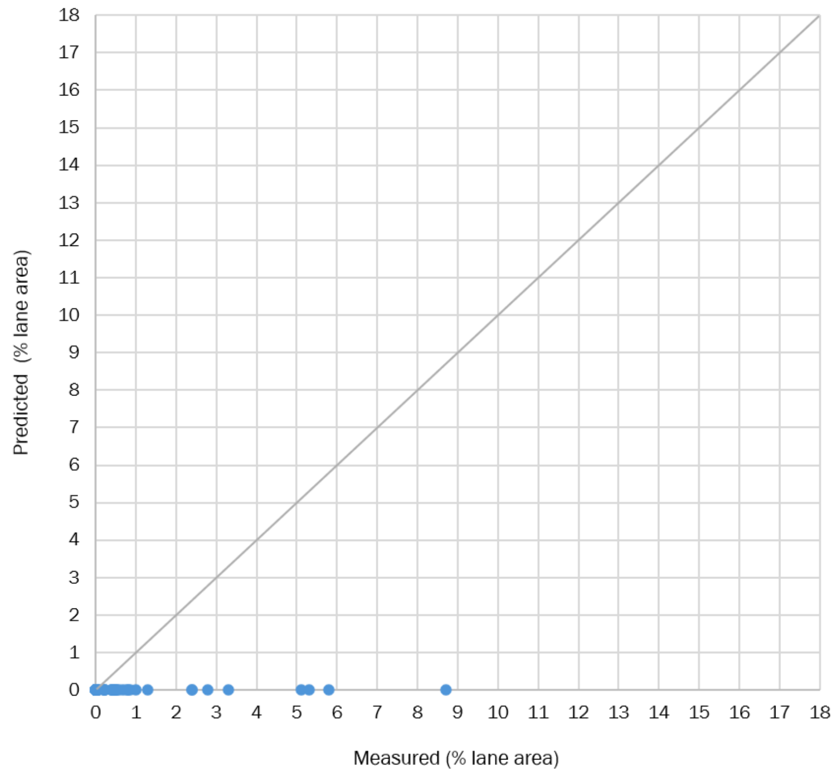


Figure 13. Measured vs Predicted Bottom Up Cracking After the Optimization Process

Table 10. Bottom Up Cracking Model Coefficient Optimization Iterations

Optimization Iteration #	Adjusted Coefficient	Adjustment	Accepted Coefficient	Bias (% of area)	SEE (% of area)
Verification	-	-	-	0.66	2.85
1	C1	2.0	-	-0.16	0.88
2	C1	3.0	3	0.19	0.90
3	C1	2.5	-	0.19	0.90

Once the coefficient optimization was complete, the recommended coefficients were used to determine the predicted performance of the section that was removed from the initial data set for use as the validation site, per the partial jackknife method. The results from this validation process produced minimal bias, but larger than both the global and optimization values for the larger set. The partial jackknife method uses a randomly selected section for validation, and the randomly selected section from the given data set showed a larger % cracking versus many other sections. This resulted in a larger bias value that does not necessarily accurately represent the total prediction capabilities of the model. The randomly selected section was part of the set of sections from US 280, and the underlying distresses prior to treatment are a probable contributing factor to the measured cracking value (i.e., the section would have benefited from a deeper treatment than CR such as FDR, though the selected treatment aligned with the original pavement preservation study goals). Model statistics from the evaluation, optimization, and validation stages are shown in Table 11 below. Table 12 details the final model optimization results with the recommended calibrated coefficients for the total rutting model.

Table 11. Bottom Up Cracking Model Validation Results

Bottom Up Cracking Model Statistics	**Evaluation**		**Optimization**		**Validation**	
Test Statistic	Value	Rating	Value	Rating	Value	Rating
Bias	0.66	-	0.19	-	-2.80	-
SEE	2.85	Very Poor	0.90	Poor	-	-

Table 12. Bottom Up Cracking Model Optimization Results

Bottom Up Cracking Model Coefficient	**Global Value**	**Optimized Value**
C1	1.31	3.0
C2: < 5 in	2.1585	2.1585
C2: 5 in <= h_{AC} <= 12 in	(0.867 + 0.2583 * hac) * 1 + 0	(0.867 + 0.2583 * hac) * 1 + 0
C2: > 12 in	3.9666	3.9666

3.3.5 Optimization of Total Rutting Distress Prediction Model Calibration Coefficients

The total rutting predictions produced with the use of global coefficients resulted in a negative bias and a very good SEE. While these values are within acceptable ranges, the plotted data showed an overprediction in the rutting and optimization iterations were then conducted to further improve the model. Considering the "very good" SEE, the coefficients associated with eliminating bias were adjusted. First, the βr1 coefficient was adjusted. This coefficient relates to the impact that the asphalt layer has on the total rutting prediction. Reduction of this coefficient improved both the bias and SEE. The βs1 coefficients were adjusted for both the granular base and subgrade to further optimize the model. The final selected coefficient offered the bias value closest to zero and a very good SEE. When evaluating the plotted measured versus predicted distress data, there was an evident cluster of data that indicated an overprediction of rutting. Figure 14 shows the predicted versus measured distress data after the coefficient optimization, with the overpredicted data points.

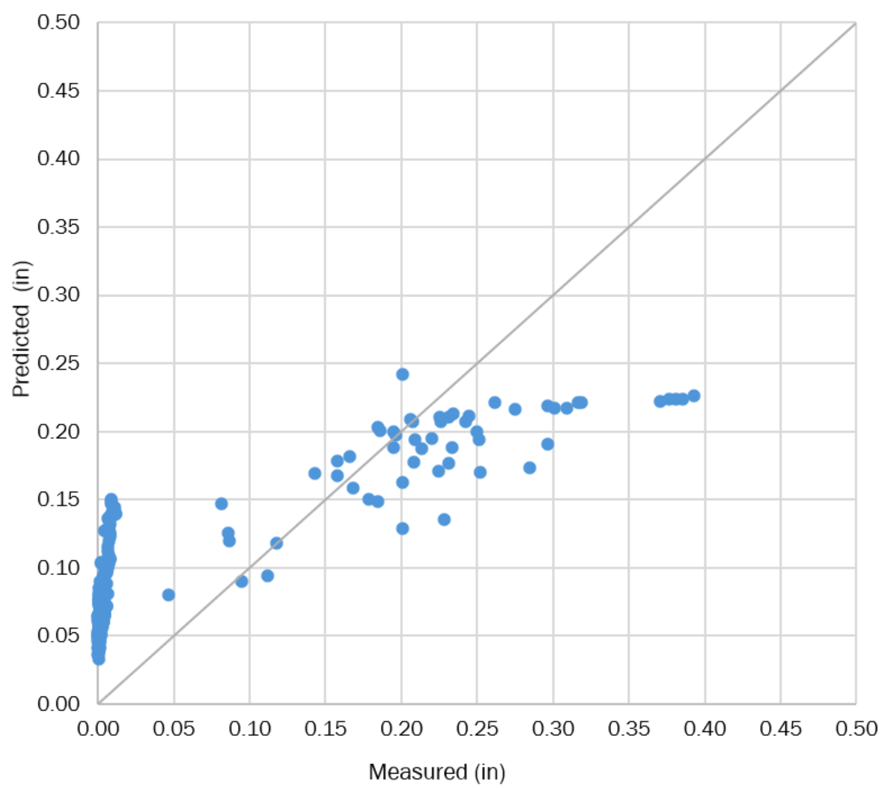


Figure 14. Measured vs Predicted Total Rutting After the Optimization Process

Upon further investigation, the sections associated with those data points were identified as the off-ramp sections. The plot with the removal of the off-ramp sections is shown in Figure 15, and the change in bias and SEE between the two models is shown in Table 13.

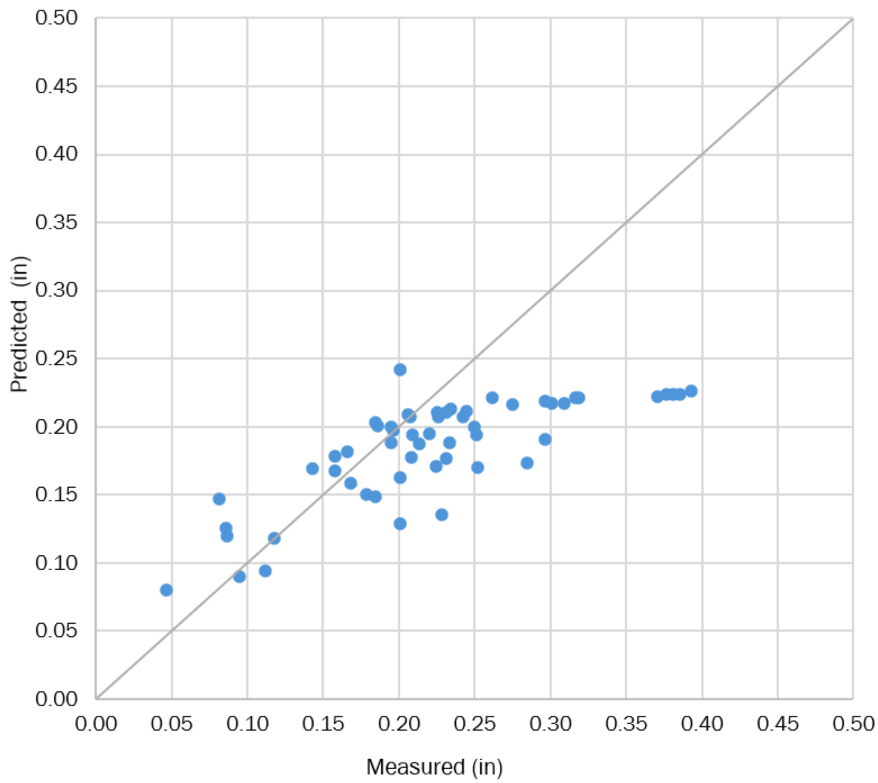


Figure 15. Optimized Total Rutting Model with the Off-Ramp Sections Removed

Table 13. Optimized Total Rutting Model Statistics with & without the Off-Ramp Sections

Statistic	With Off-Ramp Sections	Without Off-Ramp Sections
Bias	-0.05	0.011
SEE	0.08	0.03

Removal of the off-ramp showed improvement in the distress prediction model statistics and the measured versus predicted distress plots. These sections, in comparison to the others, showed a much smaller amount of measured rutting. The larger bias in the off-ramp data versus the other sections used in the calibration process was explored to investigate whether any trends in the data were present. In this effort, the off-ramp sections were subjected to a separate calibration process. In this process, the same coefficients were altered as were in the calibration for all 13 sections ($\beta r1$ and $\beta s1$). Greater reductions in these values were evaluated, all resulting in a consistent over prediction of rutting. The rutting in the AC layers (HMA surface course and CR layer), the granular base rutting, and the subgrade rutting were then each assessed individually. Considering the SEE was categorized as “very good” using the global coefficients, bias and the plotted data were used to assess the impact of the optimization iterations. First, the $\beta r1$ coefficient was reduced to 0.1 from the global value of 0.4. This reduced the bias from -0.148 to -0.132 but left little impact on the plotted data. Next, the $\beta s1$ coefficient was reduced from the global value of 1 to 0.5, this led to a similar magnitude in reduction as $\beta r1$ by reducing the bias from -0.132 to -0.118. The third iteration adjusted the $\beta s1$ value associated with the subgrade rutting from 1 to 0.5 to match the reduction from the optimization of the granular base. This led to the most significant reduction in bias, going from -0.118 to -0.067. This greater reduction in total rutting bias caused by the reduction in the subgrade $\beta s1$ coefficient indicated that the subgrade rutting was the largest contribution to the predicted total rutting.

Building upon these results, the total rutting distress prediction model was investigated for each section. As anticipated based on the previous findings, the subgrade rutting made up much of the predicted total rutting. Figure 16 below shows the predicted rutting distress chart from one of the off-ramp sections.

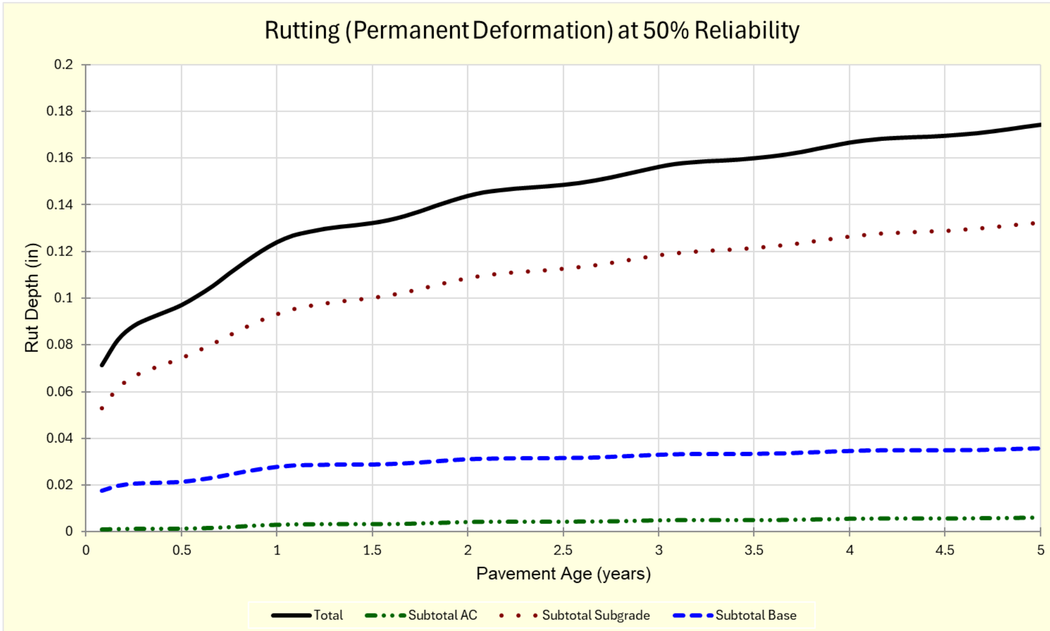


Figure 16. Total Rutting Model Chart for an Off-Ramp Section Using Global Coefficients

From this chart, it can be inferred that the AC layer (HMA surface and CR base) contributes the least to the total rutting, followed by the granular base, with the largest contribution being from the subgrade. While forensics have yet to be conducted on the off-ramp sections, the maximum measured rutting across all four sections is less than 0.012 inches, meaning the predicted subgrade rutting is over 4 times the total rutting measured maximum as indicated by the dotted line in Figure 16. A fourth optimization was then conducted, reducing the subgrade rutting $\beta s1$ coefficient to 0.01. This resulted in a bias of 0.021. Table 14 shows the optimization iterations conducted on the off-ramp and their resulting model statistics. Figure 17 shows the progression of the off-ramp predicted rutting data from optimization iterations 3 and 4.

Table 14. Total Rutting Model Coefficient Optimization Iterations for Off-Ramp Sections

Optimization Iteration #	Adjusted Coefficient	Adjustment	Bias (in)	SEE (in)
Verification	-	-	-0.148	0.153
1	Br1	0.1	-0.132	0.134
2	Bs1 - granular base only	0.5	-0.118	0.120
3	Bs1 - granular base and subgrade	0.5	-0.067	0.068
4	Bs1 – subgrade only	0.01	0.021	0.018

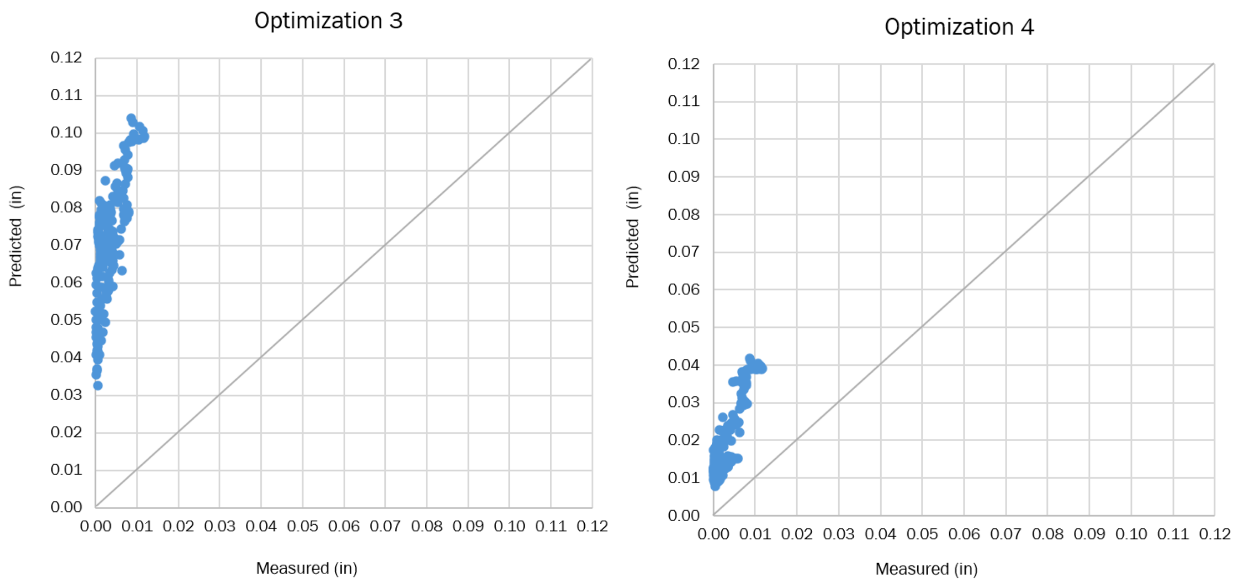


Figure 17. Predicted Total Rutting for Optimization 3 and 4 for the Off-Ramp Sections

It is hypothesized that the subgrade rutting being the greatest contributor to the total rutting was due to a combination of the CR layer being placed directly onto the granular base and the total AC layer thickness being on the thinner end of the sections used in the calibration process (5 inches total). N4 is the only other section in which the CR layer is placed directly on a granular base; however, the optimized total rutting model did not result in an overprediction of rutting. The total AC thickness of N4 is greater than that of the off-ramp, totaling over 8 inches. It is likely that this greater AC thickness results in the subgrade rutting not overwhelming the prediction as it does on the off-ramp. The sections on 70th Street all had total AC thicknesses of 5 inches, like the off-ramp, but contained an HMA layer between the CR layer and the granular base. The total rutting was not overpredicted for these sections as seen for the off-ramp sections.

For the total rutting model using all 13 sections, the refinements conducted resulted in a very good SEE and improved negative bias versus the global coefficients. Figure 18 shows the predicted versus measured distress data after the coefficient optimization, including the off-ramp sections. Table 15 details the total rutting model coefficient optimization iterations and their resulting impact on bias and SEE.

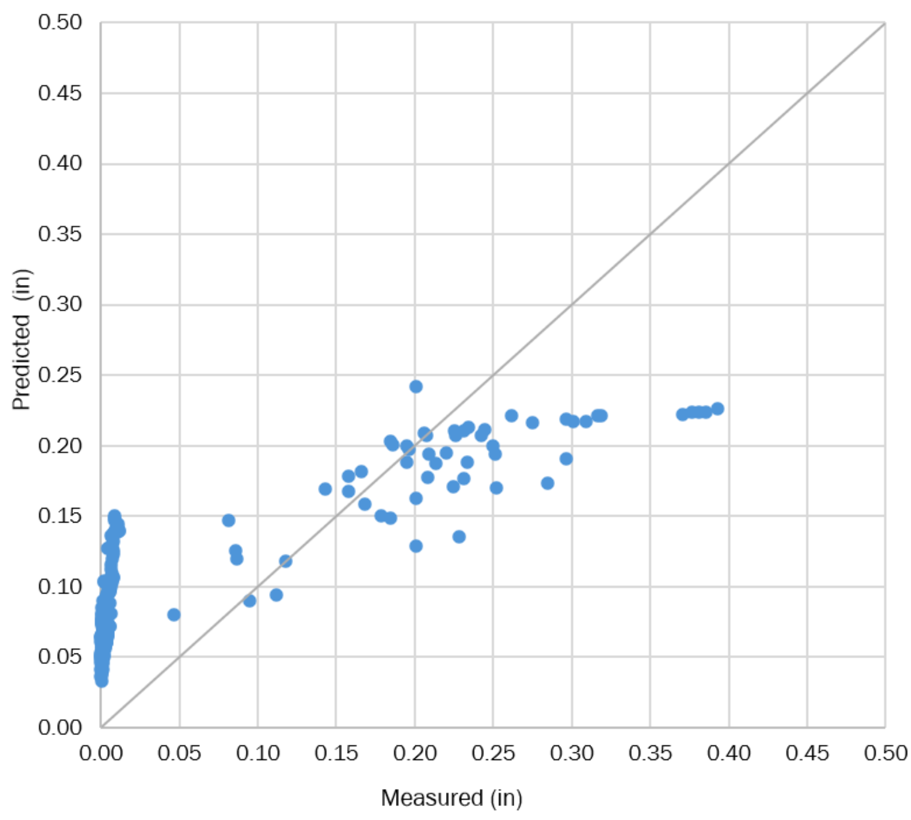


Figure 18. Measured vs Predicted Total Rutting After the Optimization Process

Table 15. Total Rutting Model Coefficient Optimization Iterations

Optimization Iteration #	Adjusted Coefficient	Adjustment	Accepted Coefficient	Bias (in)	SEE (in)
Verification	-	-	-	-0.137	0.15
1	βr1	0.3	0.3	-0.126	0.14
2	βs1 – Granular Base and Subgrade	0.5	0.5	-0.053	0.08

Once the coefficient optimization was complete, the validation process (per the partial jackknife method) was completed by using the calibrated coefficients in determining the predicted performance of the section that was removed from the initial data set. The results from this validation process produced a low bias value comparable to the optimization data set results. The model statistics from the evaluation, optimization, and validation stages are shown in Table 16 below. Table 17 details the final model optimization results with the recommended calibrated coefficients for the total rutting model.

*Table 16. Total Rutting Model Validation Result*s

Total Rutting Model Statistics	Evaluation		Optimization		Validation	
Test Statistic	Value	Rating	Value	Rating	Value	Rating
Bias	-0.14	-	-0.05	-	-0.04	-
SEE	0.15	Very Good	0.08	Very Good	-	-

Table 17. Total Rutting Model Optimization Results

Total Rutting Model Coefficient	Global Value	Optimized Value
βr1	0.4	0.3
βr2	0.52	0.52
βr3	1.36	1.36
βs1: Granular Base	1	0.5
βs1: Subgrade	1	0.5

3.4 Findings

Findings for this chapter include:

- The global coefficients within version 3.0 of the PMED software do not accurately predict the pavement distress progression seen in flexible pavements containing a CR base or intermediate layer.
- Use of similar mixture-specific k-coefficients associated with the rutting model in place of the global coefficients provided statistically similar rutting results with the exception of the global versus test track k-coefficients.
- Use of mixture specific air voids had little and variable impact on the predicted bottom up cracking for the two sections that were evaluated.
- An optimized total rutting prediction model that met the statistical testing thresholds was achievable through modifications within the asphalt layer rutting and subgrade layer rutting coefficients.
- In sections containing a total AC layer of 5 inches or less with the CR layer placed directly on a granular base, subgrade rutting dominated the total rutting prediction model leading to an overprediction of rutting over 4 times the maximum measured rutting.
- An optimized bottom up cracking model through coefficients within the bottom up cracking transfer function that met the statistical testing was not achievable. This was due to a subset of the sections having increased cracking predictions versus the remaining sections.

- The total reported measured cracking for the sections on US 280 and 70th Street was categorized entirely as bottom up cracking as no forensic analysis was conducted to determine the source. Crack map analysis indicated that a portion of the measured cracking is likely transverse and not entirely bottom up cracking. This likely biased the prediction model.
- The lack of opportunity to adjust the originating layer of cracking presents a limitation in the software.
- An optimized bottom up cracking model through coefficients within the bottom up cracking transfer function that met the statistical testing thresholds was achievable when the subset of sections with greater measured cracking was omitted.

3.5 Conclusions

As the use of CR pavements within research, state agencies, and industry continues to increase, the need for a better understanding of the structural design process for pavements containing a CR layer becomes more pressing. Using both laboratory and field performance data can help to better predict the performance and expected time to of a pavement structure. The existing default values within the software used for distress prediction, however, are based on HMA values. This presented an opportunity to compile performance data from existing pavement structures containing a CR layer and evaluate the software's ability to predict the pavement performance of a structure containing a CR intermediate or base layer. The objective of this study included evaluating the existing global calibration coefficients in the PMED software version 3.0 for their suitability in accurately predicting pavement distresses for flexible pavements containing a CR base or intermediate layer, assessing the model sensitivity to select mixture specific properties, and optimizing the model to accurately predict pavement distresses and performance.

Thirteen flexible pavement sections including a CR base or intermediate layer that met the selection criteria were used in the model optimization effort. Initial evaluation of the predicted distresses using global coefficients versus the measured distresses from the sections used in the calibration process showed inaccurate distress predictions. It was determined that the global coefficients used in version 3.0 of the PMED software proved inadequate for use in distress prediction and structural design of flexible pavements containing a CR layer, and further refinement of the coefficients was needed.

During the optimization process, the sensitivity of the rutting model to mixture-specific k-coefficients related was evaluated to determine whether global coefficients were sufficient or if k-coefficients from similar CR mixtures produced significantly more accurate predicted rutting. Results showed differences in rutting between k-coefficient sets; however, none were statistically different from each other or the predicted rutting using the global coefficients. An analysis was also conducted on the impact of air voids for the mixture where the data was available, with similar conclusions drawn. Emphasis was placed on the global values being derived from HMA/WMA properties and that they may not always be applicable for CR pavements, despite the minimal impact the evaluated inputs had on resulting distresses. Emphasis was also placed on the origins of the transfer functions, the lack of consideration of CR materials, and the need for further investigation of the suitability of these functions and their inputs.

The optimization of the bottom up cracking model resulted in a positive bias, with a poor SEE. The reasoning for the poor statistical test results were investigated by evaluating the measured versus predicted distress plots. It was determined that the US 280 and 70th

Street sections exhibited larger amounts of measured cracking versus the other sections and were thus biasing the model. A detailed analysis of the sections highlighted all measured cracking being categorized as bottom up cracking as a shortcoming of the current recommended calibration method. Crack mapping of US 280 showed that a portion of the measured cracking is likely transverse cracking and not bottom up; however, forensic analysis has not been conducted to verify the source of any measured cracking. Ultimately, the optimization process including all 13 sections produced a bottom up cracking model with a bias of 0.19 and a poor SEE of 0.90. It was noted that removal of the US 280 and 70th Street sections resulted in a model with a bias of 0.06 and a very good SEE of 0.24.

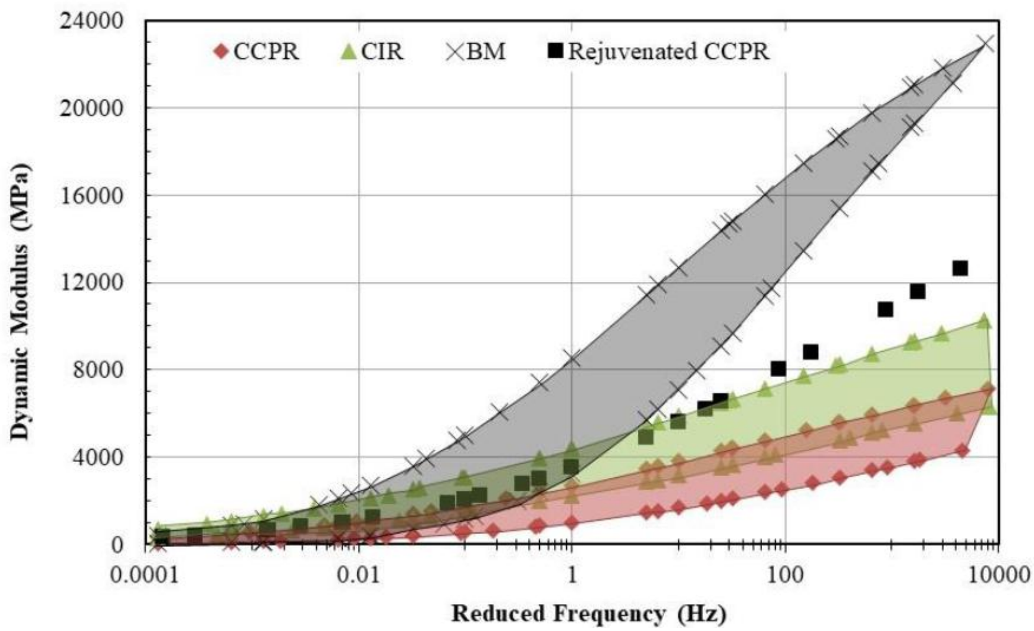
Optimization of the total rutting model resulted in a negative bias and very good SEE; however, upon inspection of the plotted measured versus predicted rutting a subset of the data showed significant overprediction. This subset was identified as the four off-ramp sections. A separate calibration was conducted on these sections to determine if any data trend existed. From this calibration, subgrade rutting was identified as the largest contributor of overprediction. It was hypothesized that this was due to two factors: the total AC layer being only 5 inches thick and the CR layer being placed directly on the granular base. The final optimization of the rutting model, including the off-ramp sections, resulted in a bias of -0.053 and a very good SEE of 0.07.

Chapter Four: Rejuvenated Cold Recycled Mixture Design for Surface Course Applications

4.1 Introduction and Objectives

Historically, research on and applications of CR have focused on use of the mixture as a base course with the intention of an HMA or WMA surface being placed on top (Diefenderfer et al. 2019, Xiao et al. 2018, Gu et al. 2019). This research has helped to refine the design and placement of CR mixtures, creating a more consistent design process and helping to establish ties between lab testing data and field performance. While the use of CR is becoming more prevalent, its applications as a surface course do not have routine use or documentation.

Pushing the limits of the current applications of CR, through its’ use as a surface course, could provide a more sustainable alternative to rehabilitating and maintaining roads. However, for a CR mixture to be used as such, the mixture design process and additives must be evaluated from the perspective of it receiving direct traffic loading and weathering versus being protected between other pavement layers within the structure. Research on CR mixtures (Bowers et al. 2019; Turochy and Bowers 2024) points toward potential improvements in mixture properties with the addition of a rejuvenator. In the research conducted by Turochy and Bowers (2024), the addition of a rejuvenator to a CR mixture led to increased density in the lab and in the field. The increased lab densities also correlated with increased laboratory specimen strengths and lower moisture susceptibility. Dynamic modulus testing results from Bowers et al. 2019, Schwartz et al. 2017, and Turochy and Bowers 2024 indicated that the addition of a rejuvenator shifted the master curve toward values typically seen in the HMA surface course, and falling between that and a typical non-rejuvenated CR mixture (Figure 19).



**BM = Base Mixture*

Figure 19. Dynamic Modulus Master Curve Showing Rejuvenated Cold Recycled Mixture Data Falling Between Typical CR and HMA (Bowers et al. 2019)

For rejuvenated CR mixtures, as the reduced frequency increases, the dynamic modulus increases more rapidly than a non-rejuvenated CR mixture. This trend is more similar to a typical HMA or WMA. These conclusions are important considerations when designing a CR mixture for use as a surface course versus a base course. The larger difference between the initial dynamic modulus across the reduced frequency values for rejuvenated CR mixtures versus non rejuvenated is indicative of a more temperature and frequency dependent mixture. Typically, base courses are designed to be less temperature and frequency dependent as they experience more uniform stress as the load-bearing layer, with a larger nominal maximum aggregate size and lower relative binder content creating a mixture matrix intended to be supported by the aggregate structure at heavier loading. Surface courses, on the other hand, tend to experience more variable strains and are designed to remain flexible to temperature and loading changes. This shift in dynamic modulus values is indicative of rejuvenated CR mixtures being more temperature and frequency dependent, showing potential for use as surface course.

The objectives of this study are as follows:

1. Evaluate the impact of rejuvenator content on mixture laboratory performance and develop a mixture design based upon the findings.
2. Construct a test section on the NCAT Test Track off-ramp using the developed mixture design as a surface course, conduct laboratory testing on plant mixed material, and evaluate the field performance to date.
3. Assess the accuracy of the pavement distress prediction model developed for flexible pavements containing a CR mixture as a base or intermediate layer for use with CR mixtures as the surface course.
4. Discuss considerations for designing and testing of CR mixtures for use as a surface course based upon previous research.

4.2 Methodology

4.2.1 Material Collection and Preparation

Prior to the mixture design phase of this research, materials required for the designs were collected. Materials collected include RAP, emulsion, and rejuvenator. The emulsions and rejuvenator used in this study were shipped to NCAT in plastic 5- and 1-gallon buckets, respectfully. The RAP used in this design was collected in 50-gallon plastic drums from the RAP stockpile selected for use in test track construction and transported to NCAT. The stockpiles were sampled following ASTM D75 to ensure a homogenous sample that was representative of the overall stockpile was collected. The RAP stockpile used for this research was stored outside, uncovered, and was exposed to all weather conditions. At the time of collection, the material contained moisture and was dried prior to material characterization. Being that RAP contains asphalt binder; it cannot be dried in an oven at temperatures used for aggregate drying as it risks altering the existing binder properties and material gradation. To thoroughly dry the materials, each barrel was emptied individually onto a smooth, dry concrete floor in an ambient temperature warehouse. The RAP was stirred frequently to accelerate the process and ensure all portions dried evenly and fans were used to promote airflow through the material. The RAP was considered dry once it passed a hydroscopic moisture test, resulting in a moisture content of less than 0.5%.

4.2.2 Material Characterization

For the emulsions and rejuvenators used in this study, characterization was conducted at their respective manufacturers' facilities, and the information was provided with the material shipped to NCAT for the design and construction phases. For the RAP, a "black rock" gradation was conducted following AASHTO T27 Standard Method of Test for Sieve Analysis of Fine and Coarse Aggregates. A washed gradation was also run to better assess the fine aggregate content passing the No. 200 sieve per AASHTO T11 Standard Method of Test for Materials Finer than 75-μm (No. 200) Sieve in Mineral Aggregates by Washing. Mutliple samples were reduced from the previously dried RAP using a mechanical splitter following AASHTO R76 Standard Practice for Reducing Samples of Aggregate to Testing Size for use in black-rock and washed gradations. The RAP binder content was also determined following AASHTO T308 Standard Method of Test for Determining the Asphalt Binder Content of Asphalt Mixtures by the Ignition Method.

In addition to the gradations, the OFC of the RAP must be determined prior to mixture design. The OFC was determined following California Test 313-I. For this process, the RAP material was passed through the ¾ inch sieve, using only the passing portion for OFC determination. Eight separate 1,150-gram samples were then weighed out. Fluid contents tested ranged from 3% to 6% in 0.5% increments by weight of dry RAP. Each RAP sample containing a specified fluid content was funneled into a 4 inch mold, the material was leveled, rodded 10 to 15 times with a 5/8 in rod in a circular motion, then compacted in a Superpave gyratory compactor to 30 gyrations. The compacted sample was then extracted, and the mass, height, and diameter of the specimen were recorded. A bulk density for each specimen was determined, and the sample was dried in an oven at low temperatures to verify the moisture content. The data was then plotted to evaluate the moisture-density relationship, and the optimum fluid content was established from the curve.

4.2.3 Rejuvenator Content Mixture Performance Impact Assessment

Prior to the selection of a final mixture design for placement on the NCAT Test Track off-ramp, the influence of rejuvenator content on the MS strengths and moisture susceptibility of a mixture was evaluated. A single rejuvenator was selected by the research sponsor for this study, and the contents selected for evaluation were based on manufacturers' recommendations. The emulsion used was also selected by the research sponsor. To evaluate the rejuvenator impact on strength performance, a previously developed mixture design was used as a base mixture, and the rejuvenator content was varied. Once the optimum rejuvenator dosage was selected, the emulsion type was varied while the rejuvenator percentage was held constant. As typically seen with CR designs containing emulsions, design panels testing four different emulsion contents were set up for each unique emulsion type and rejuvenator content combination, resulting in a total of twelve mixture designs to be evaluated.

4.2.4 Specimen Fabrication and Strength and Moisture Susceptibility Testing

All mixture designs developed in the rejuvenator content optimization and for the final mixture design were mixed, compacted, cured, conditioned, and tested following the same procedure. Each mixture was prepared at NCAT using the Wirtgen WLM-30 laboratory scale pugmill. Per Wirtgen's recommendations for optimum mixing conditions, the material was mixed in batches of 25-30 kg (55-66 lb) and 60-second mixing increments were used between each material added (i.e., RAP, water, etc.). The specimens were then compacted, cured, and tested per AASHTO PP 86. Their dry and conditioned strengths were determined via MS testing as indicators of mixture laboratory performance and for mixture design acceptance. The final mixture design was selected based on the mixtures ability to meet the strength requirements while also achieving the most economically viable mix design (the least amount of stabilizing and recycling agents needed to meet the requirements).

Prior to the mixing process for each design, the appropriate amount of emulsion (by weight of dry RAP) was weighed and then dosed with the selected percentage of bio-based rejuvenator (by weight of RAP binder). There were no manufacturer recommendations for the blending of the bio-based rejuvenator and emulsion; therefore, the two were blended by hand using a glass stirring rod in 10 seconds increments (minimum of 30 seconds) until visually homogenous. To begin the mixing process, a 25-30 kg (55-66 lb.) RAP sample was added into the pugmill, followed by the water required to reach the mixture optimum fluid content, and then the emulsion and rejuvenator blend, with a 60-second mixing period between each addition. The CR mixture was then inspected by hand to verify cohesion. A palm-sized sample of the loose mixture was squeezed and evaluated based on whether it falls apart, and if the mixture stays bound together, it indicative of good binder coating and material cohesion. Figure 20 shows the squeezed loose material.

Figure 20. Cohesion Check of the CR Mixture (Turochy, 2023)

Each CR mixture was then placed in a plastic tub and sealed with a lid to maintain the moisture content prior to compaction. All specimens were compacted to 63.5 +/- 3 mm (2.5 +/- 0.12 in) in a 100mm (3.9 in) mold in a Superpave Gyratory Compactor to 30 gyrations. The loose weight of the material was varied so that the specified height could be achieved at 30 gyrations. Once the appropriate amount of loose material had been identified, a minimum of eight specimens were compacted for strength and moisture susceptibility testing. The specimens were cured in a 60°C oven for 48 hours. Two appropriately sized samples of the loose RAP material were also collected and cured for use in determining the theoretical maximum specific gravity (G_{mm}) of the mixture.

After the 48-hour curing period, the specimens were removed from the oven, their weights and heights were recorded to evaluate their dry densities. They then rested for 24 hours at room temperature before further testing. The two samples of cured loose material were loosely broken apart and tested to determine the G_{mm}, which is calculated following Equation 9.

$$G_{mm} = \frac{W_S}{(W_S + W_{B,W} + W_{B,S})} \tag{9}$$

Where:

G_{mm} = maximum specific gravity of the mixture

W_S = sample weight, grams

$W_{B,W}$ = bowl weight in water, grams

$W_{B,S}$ = bowl and sample weight in water, grams

Once the G_{mm} was calculated, the specimens were then subjected to moisture conditioning by applying a vacuum of 26 mm of Hg partial pressure until the specimens were saturated within 55-75%. Once saturated, the specimens were transferred to a 25^{o}C water bath for 23 hours, followed by 1 hour in a 40^{o}C bath. After the 24-hour period, the conditioned specimens were removed from the water bath, dried to saturated surface dry (SSD) conditions, then tested via MS Testing per AASHTO PP 86 using the 850D Pine Test Press. Figure 21 shows a prepared specimen in the MS jig on the Pine Test Press.

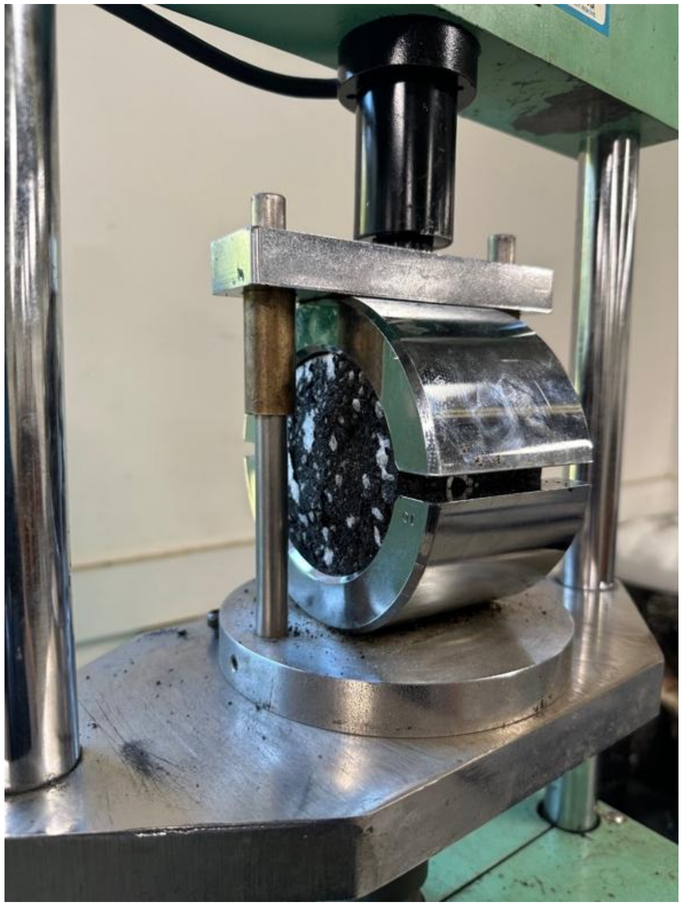


Figure 21. CR Mixture Specimen in the Marshall Stability Jig

For dry specimen conditioning, the specimens were placed in a 40°C oven two hours prior to MS testing. The data collected from testing produced a maximum stability per specimen in pounds. The MSR was calculated following Equation 10.

$$\text{MSR} = \frac{S_{t,c}}{S_{t,d}} \quad (10)$$

Where:

MSR = MS ratio

$S_{t,c}$ = average conditioned MS, psi

$S_{t,d}$ = average dry MS, psi

Per AASHTO PP 86, for a CR mixture containing emulsified asphalts to be considered passing, the dry strength must be equal to or greater than 1250 lb (567 kg), and the MSR of the mixture must be equal to or greater than 0.7. All mixture designs produced during the rejuvenator content evaluation, along with the final mixture design, followed this procedure.

4.2.5 Plant Production and Placement

Once a mixture design was selected, it was produced in a portable plant at the NCAT Test Track for placement on the off-ramp using the Pugmill Systems Portable Pugmill (Figure 22).

Figure 22. Pugmill Systems Portable Pugmill (Turochy, 2023)

After production, the mixture was loaded into a standard dump truck and immediately transported to the off-ramp for placement. At the off-ramp, the mixture was transferred from the dump trunk directly into the hopper of a typical asphalt paver and placed in a similar manner to a HMA mixture. The CR mixture was placed in a single, 5-inch thick lift and compacted until refusal. Wet density of the mat was tested with a nuclear density gauge.

4.2.6 Plant Mixed Lab Compacted Specimen Production and Testing

To create Plant Mixed Laboratory Compacted (PMLC) specimens, loose material samples of each CR mixture were collected on-site approximately halfway through production. A homogenous sample representative of the entire mixture was collected directly from the hopper during placement. The mixture sample was placed in a plastic tub and sealed with a lid to maintain the moisture content until laboratory compaction. Each tub of material was immediately transported to the laboratory for specimen fabrication.

4.2.7 Specimen Compaction, Curing, and Testing

Once the mixture samples collected during construction were transported to the lab, two specimen sizes were compacted. The first specimens compacted were compacted to 63.5 +/- 3 mm (2.5 +/- 0.12 in) in height in a 100 mm (3.9 in) diameter mold to 30 gyrations and cured for 48 hours at 60^{o}C per AASHTO PP 86. The samples were then subjected to MS testing, an identical procedure to the one followed during mixture design. Each specimen had their heights and weights recorded promptly after being removed from the oven to calculate dry densities. A total of eight specimens were compacted per mixture design. The second set of specimens were compacted to testing size for dynamic modulus testing, a height of 150 +/- 1.5mm (5.9 in) in a 100 mm (3.9 in) diameter mold. These specimens were compacted to field density instead of specified gyrations. It should be noted that specimens for dynamic modulus testing are typically compacted as a large specimen size, and then cores are taken from this specimen to reduce the impact that the density gradient has on the specimen. Due to previous experience with certain CR mixtures being unable to be cored due to material shearing during the process, the specimens used in this research were compacted to testing size and coring was not conducted.

4.2.8 - Dynamic Modulus Testing

Dynamic modulus testing was performed on PMLC specimens compacted to field density that were 150 mm (5.9 in) tall and 100 mm (3.9 in) in diameter specimens, with three replicates total. Once all specimens were fully cured, dried, and ready for testing, their dry weights were recorded, and heights and diameters were measured. Studs were placed on each specimen using the IPC Global Gauge Point Fixing Jig shown in Figure 23.

Figure 23. Dynamic Modulus Specimen on Gauge Point Fixing Jig (Turochy, 2023)

The dynamic modulus testing was performed using an IPC Global AMPT PRO. Based on the research completed in National Cooperative Highway Research Program (NCHRP) project 09-51 (Schwartz et al. 2017), three testing temperatures of 4.4^{o}C, 21.1^{o}C, and 37.8^{o}C and six testing frequencies of 25, 10, 5, 1, 0.5, and 0.1 Hz were used. All specimens were tested from lowest to the highest temperature with decreasing frequency to reduce the potential for damage to the specimen. Each testing temperature required a different conditioning time as follows: overnight for 4.4^{o}C, two hours for 21.1^{o}C, and three hours for 37.8^{o}C. Prior to loading the specimen into the AMPT chamber, a linear variable differential transformer (LVDT) target clamp was placed on each stud. At the highest testing temperature, pairs of springs were placed between pairs of target clamps. Once the specimen was loaded, the LVDTs were placed between each pair of target clamps, Teflon sheets were placed between the specimen and the top and bottom loading platens to reduce friction, then the chamber was closed. Figure 24 shows a loaded specimen with LVDTs placed.

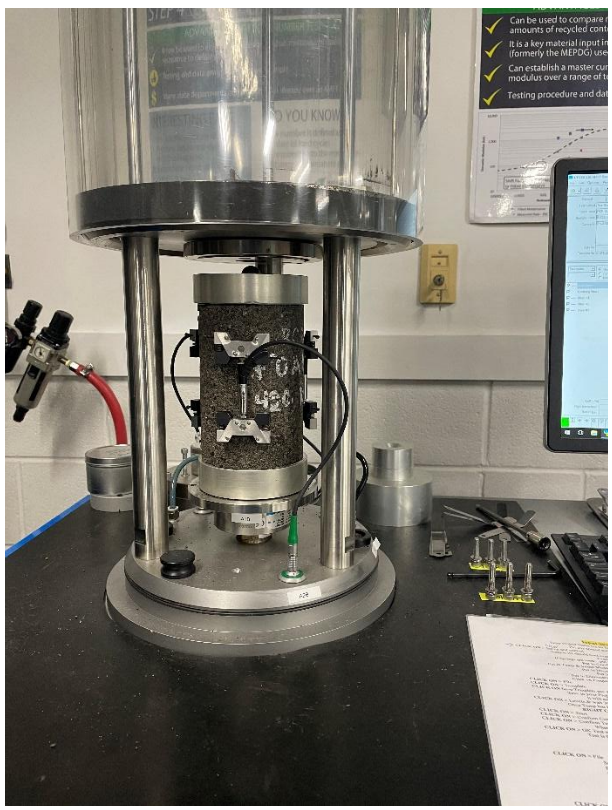


Figure 24. Specimen Loaded into AMPT Pro Chamber with LVDTs Placed (Turochy, 2023)

The specimen was left in the chamber for 30 minutes at the conditioning temperature before starting the dynamic modulus testing. Once the data was collected, it was required to meet a set of data quality characteristics shown in Table 18.

Table 18. Data Quality Characteristics Used for Dynamic Modulus Data

Data Quality Characteristic	Tolerance Value
Load Standard Error	<10%
Deformation Standard Error	<10%
Deformation Uniformity	<30%
Phase Uniformity	$<3^{o}$

4.2.9 – Distress Predictions from AASHTOWare Pavement ME Design Model Calibrated for Use with Flexible Pavements with a CR Base or Intermediate Mixture

In the previous chapter, the flexible pavement model calibration coefficients were optimized to best match the predicted distresses with the measured distress of the sections used in the calibration. The sections used in the calibration process all contained a CR mixture as a base or intermediate layer, meaning none had a CR surface mixture. The structural and mixture information from the mixture design developed and placed on the off-ramp in this study was input into the PMED software calibrated model and the predicted distresses output were compared to the measured distresses that have been collected from the off-ramp section from construction up until the time this research was conducted. The results were then evaluated to determine whether the previously calibrated model could accurately determine pavement distresses for a structure with a CR surface course. All traffic and climate inputs used in this model evaluation were identical to those used in Chapter 3 for the off-ramp sections.

4.3 Results and Discussion

4.3.1 RAP Gradation and Optimum Fluid Content

Four black rock gradations and four washed gradations were collected from the RAP source used and the data was plotted on a 0.45 power chart (Figure 25). Obtaining a washed gradation helped provide a more accurate estimate of material passing the No. 200 sieve that may not be shown in a black rock gradation. The washed gradation showed an increase of approximately 1.6% in material passing the No. 200 sieve. Table 19 shows the RAP percentage passing each sieve prior to and after washing. The results from the OFC determination are shown in Figure 26. The RAP binder content for this source was recorded as 5.27%

Table 19. RAP Gradations

	Black Rock RAP	Washed RAP
Sieve Size	**% Passing**	**% Passing**
2"	100.00%	100.00%
1 1/2"	100.00%	100.00%
1"	100.00%	100.00%
3/4"	99.97%	99.99%
1/2"	97.51%	97.46%
3/8"	89.98%	89.91%
#4	65.83%	65.63%
#8	45.58%	48.13%
#16	31.89%	31.75%
#30	20.82%	19.72%
#50	11.20%	9.83%
#100	4.65%	4.96%
#200	1.63%	3.21%

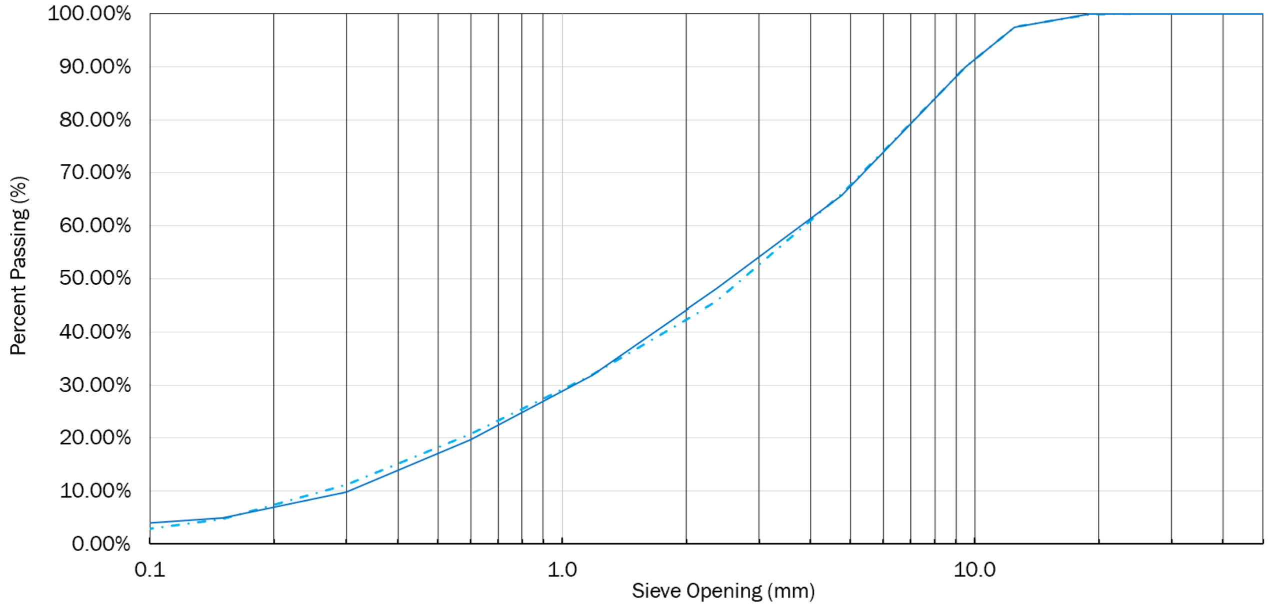


Figure 25. Black Rock and Washed Gradations

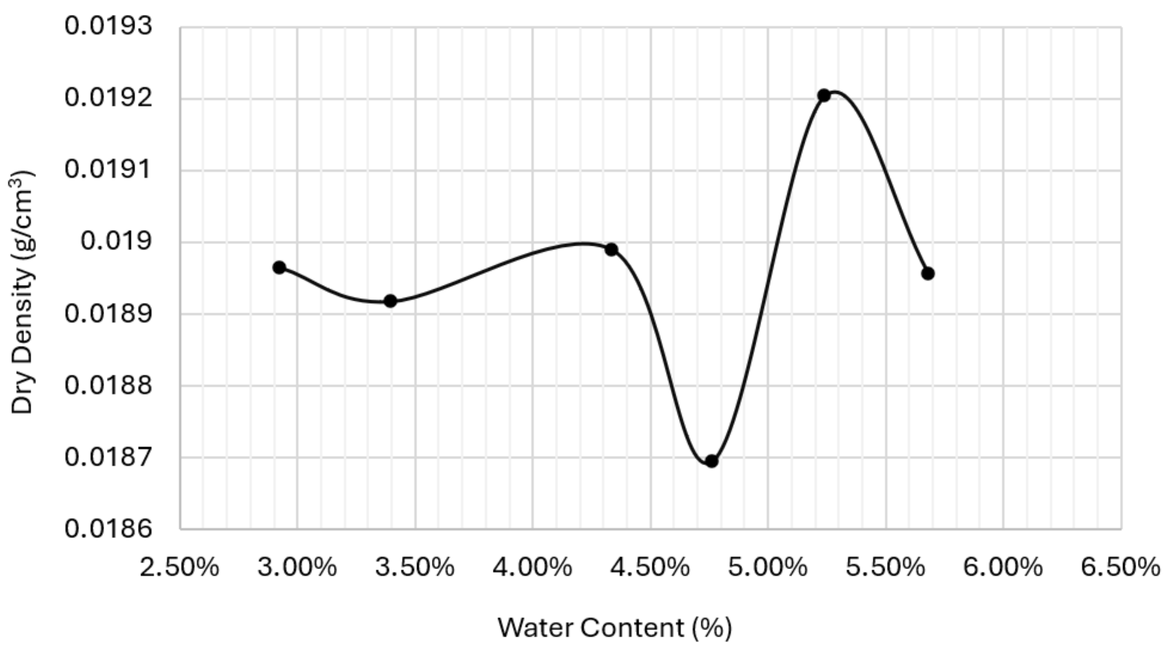


Figure 26. OFC Determination Results

Based on the visual observations made during the specimen preparation, data analysis, and prior knowledge of RAP in CR mixtures, 5% was selected as the OFC for the RAP used in this study. The moisture-density curve does not necessarily exhibit a distinct peak as is often seen with moisture-density curves for soils. Moisture-density curves produced by the modified Proctor method (AASHTO T180-D) are similar, or even less distinct than those exhibited by the gyratory method (California Test 313-I). One hypothesis as to why RAP often presents a curve with a less distinct peak than soil is that soils tend to have more consistency among sources and homogeneity within the material versus RAP, in which gradation, aggregate properties, and binder proportions vary more between sources or even within the same pile. While all efforts are made to maintain homogeneity, variability still exists. Gradation and aggregate properties may, additionally, impact the ability of the material to compact. Another hypothesis explores the liquid limit of the material. In one study (Lee and Suedkamp, 2009), soils with liquid limits below 30 or above 70 produced irregularly shaped moisture-density curve with no or multiple peaks. Tests typically run on soils, such as the liquid limit, were not run on the RAP used in this research. A study was conducted where the liquid limit was identified over multiple RAP sources, and all had liquid limits below 30 (Locander, 2009). The RAP used in this study may have a low liquid limit, which could be a contributing factor toward the irregularly shaped moisture-density curve.

4.3.2 Rejuvenator Content Optimization

Prior to developing a CR mixture design to be placed on the NCAT Test Track off-ramp as a surface course, the impact of rejuvenator content on mixture performance was investigated. The rejuvenator used in this study was selected by the research sponsor. The rejuvenator contents selected were based upon manufacturer recommendations; these rejuvenator contents are 0%, 3.5% and 7%. For the rejuvenator content evaluation, a previously developed mixture design was used as the base. The previously developed mixture design followed the design procedures outlined in Turochy (2023) for an emulsion CR mixture with a rejuvenator. This mixture design used a RAP source that had the same OFC as the RAP used in this research and contained an emulsion and the same rejuvenator. For each rejuvenator content, 8 specimens were mixed, compacted, cured, tested per AASHTO PP 86, resulting in dry and conditioned MS strengths. Dry strength and the ratio between the dry and conditioned strengths were used as acceptance points for passing or failing a mixture design and provided an analysis point for the impact the rejuvenator had on mixture performance. Table 20 shows the values associated with each rejuvenator content.

Table 20. Results from Rejuvenator Optimization

Rejuvenator Content (%)	**Density (pcf)**	**Conditioned Strength (lb)**	**Dry Strength (lb) min** 1250	**MSR** min 0.7
0%	124.35	1975.00	1373.70	1.438
3.50%	124.69	1489.18	1614.65	0.922
7%	124.52	1086.83	1289.77	0.843

All three rejuvenator contents tested resulting in passing designs and nearly identical densities. The 7% rejuvenator content mixture had the lowest dry and wet strength values and the lowest MSR, both indicating it was not the optimum dosage. While the 0% rejuvenator content mixture produced the highest MSR, it did not have the highest dry strength. Ultimately, the 3.5% dosage was selected since it had the highest dry strength and a passing MSR.

In addition to a practical difference being noted between dry strengths and the MSR values for each rejuvenator content, a paired t-Test with an alpha of 0.05 was conducted between the dry strengths and then the conditioned strengths to determine if there was a statistical difference between rejuvenator contents. Table 21 shows the results from the paired t-Test conducted on dry strengths and Table 22 on the conditioned strengths.

Table 21. Paired t-Test Results for Dry Strengths Across Rejuvenator Contents

Dry Strength t-Test	
Rejuvenator Content (%)	**p-value**
0 vs. 3.5	0.0058
3.5 vs. 7	0.0025
7 vs. 0	0.2620

Table 22. Paired t-Test Results for Conditioned Strengths Across Rejuvenator Contents

Conditioned Strength t-Test	
Rejuvenator Content (%)	**p-value**
0 vs. 3.5	0.0015
3.5 vs. 7	0.0053
7 vs. 0	0.0001

In all comparison cases, except the dry strength t-Test between 0% and 7% rejuvenator, the p-value was less than 0.05. This indicates that there is a statistically significant difference between each rejuvenator content for those dry and conditioned strengths. For the dry strength t-Test between 0% rejuvenator and 7%, the p-value was greater than 0.05, indicating those data sets were statistically similar.

4.3.3 Specimen Strength and Moisture Susceptibility Evaluation

Once the optimum rejuvenator content was selected, the mixture design process was conducted. All procedures and mixture design acceptance criteria followed AASHTO PP 86. Four contents of a cationic emulsion containing 3% Styrene-Butadiene-Styrene (SBS) were tested with the previously selected rejuvenator content to determine the design most suitable for construction. The MS strength and MSR results from each emulsion content tested are shown in Table 23.

Table 23. Summary of Strength Testing Results from Mixture Design

Emulsion Content (%)	Density (pcf)	Conditioned Strength (lb)	Dry Strength (lb) min 1250	MSR min 0.7
2.5	122.81	1514.67	1555.64	0.97
3.0	123.49	1324.66	1568.70	0.84
3.5	124.24	1467.30	1582.44	0.93
4.0	124.21	1507.04	1456.39	1.03

All mixture design exhibited strengths well above the minimum requirement, along with passing MSR values. Both 3.0% and 3.5% emulsion content designs had the highest dry strengths; however, the 3.5% emulsion content design showed a higher MSR at 0.93. Therefore, the mixture design using 3.5% emulsion content was chosen as the final mixture design for construction.

4.3.4 Production and Placement of Mixture Designs

The RAP stockpile used in production was stored outdoors and uncovered. Due to its being exposed to the elements and recent rainfall, it contained a moisture content above 0%. Adjustments were made during production to account for the excess fluid. Samples of the RAP stockpile were collected immediately before production, and mixture samples were collected mid-production to record moisture contents. The mixture production was like that of any other CR mixture, and placement was similar to that of an HMA/WMA. Figure 27 shows the material being loaded into the hopper from the dump truck. Figure 28 shows the material immediately after paving and prior to compaction.

Figure 27. Transfer of Material into the Hopper

Figure 28. Material Immediately After Placement

The CR layer was compacted until refusal for density; however, during the initial compaction, emulsion began flushing out of the surface. It is hypothesized that the existing fluid content in the RAP, in combination with the emulsion and rejuvenator, increased the total fluid content beyond optimum. At the first sign of flushing, all rollers were immediately removed from the mat, and the section was allowed to dry further before additional compaction. Compaction occurred towards the end of the day, and a secondary compaction was applied the next morning. During the secondary compaction, no issues regarding flushing occurred and the mat was compacted. It should be noted that secondary compaction was conducted without turning on the mist on the roller to help ensure additional water was not being added to the mat. Density was checked after the secondary compaction and resulted in a higher density than was recorded during mixture design. Table 24 shows the recorded field density collected via Nuclear Density Gauge.

Table 24. Wet Field Density and Percentage of Lab Density

Field Wet Density (pcf)	% Density
126.0	101.42

CR mixtures require an extended period to cure in place before cores can be taken and maintain their integrity. Due to this and the test section being shorter than the standard, cores to determine dry densities were not taken to not compromise the test section.

4.3.5 Plant Mixed Lab Compacted Specimen Performance Testing

The mixture collected during production was fabricated into specimens in the laboratory. The PMLC specimen strength and moisture susceptibility testing results are summarized in Table 25.

Table 25. Summary of Strength and Stability Test Results for PMLC Specimen

Density (pcf)	**Conditioned Strength (lb)**	**Dry Strength (lb)** min 1250	**MSR** min 0.7
128.77	1218.62	1227.25	1.01

In the mixture design phase, all specimens passed the dry strength and MSR requirements. The mixture produced in the field that was transported to the lab for compaction failed the minimum dry strength requirement by a small margin but passed the MSR requirement. This strength drop versus mixture design may be due to the excess moisture in the mixture; however, the dry strength was not significantly below the target dry strength.

4.3.6 Dynamic Modulus Testing

The results collected from dynamic modulus testing of the specimens compacted from plant produced materials were analyzed in Excel. The master curve was developed by plotting the dynamic modulus values against the values calculated for reduced frequency. A larger measured reduced frequency correlates with a lower assumed pavement temperature and higher assumed traffic loading, with smaller reduced frequencies assuming the opposite. The traffic loading applied to the specimen is in terms of vehicle speed, with faster speeds corresponding to higher frequencies and lower speeds corresponding to lower frequencies. Figure 29 shows the master curve for the CR surface course mixture and Figure 30 shows the CR surface course mixture plotted with a foamed asphalt CR mixture and a 19 mm nominal maximum aggregate size HMA base mixture with 4% 67-22 asphalt binder (by weight of dry aggregate).

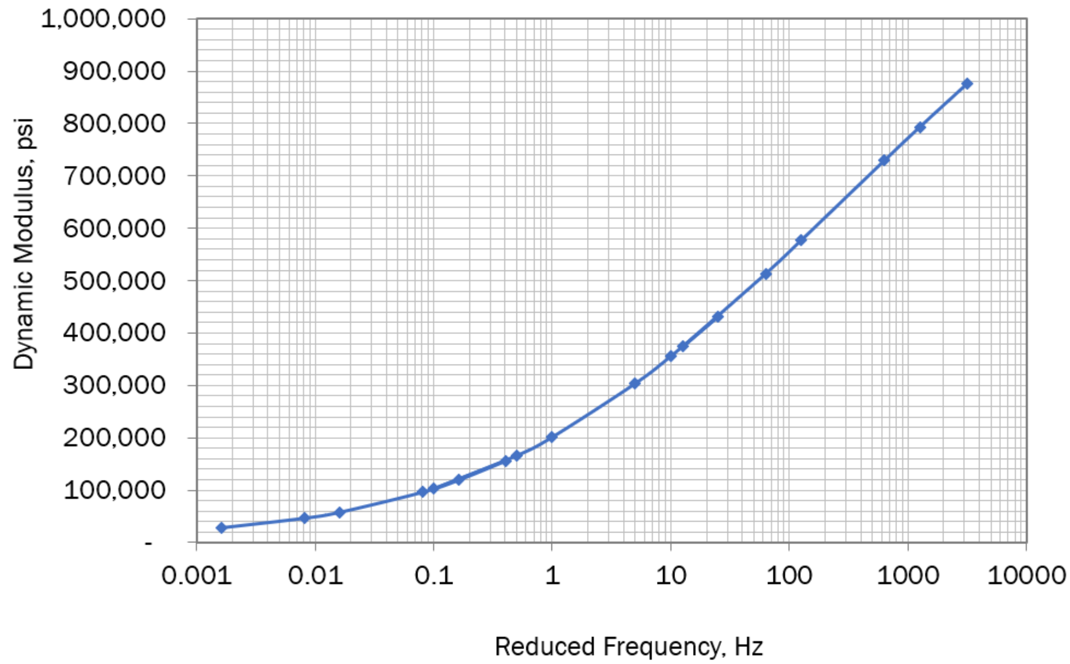


Figure 29. Plot of CR Mixture Master Curve

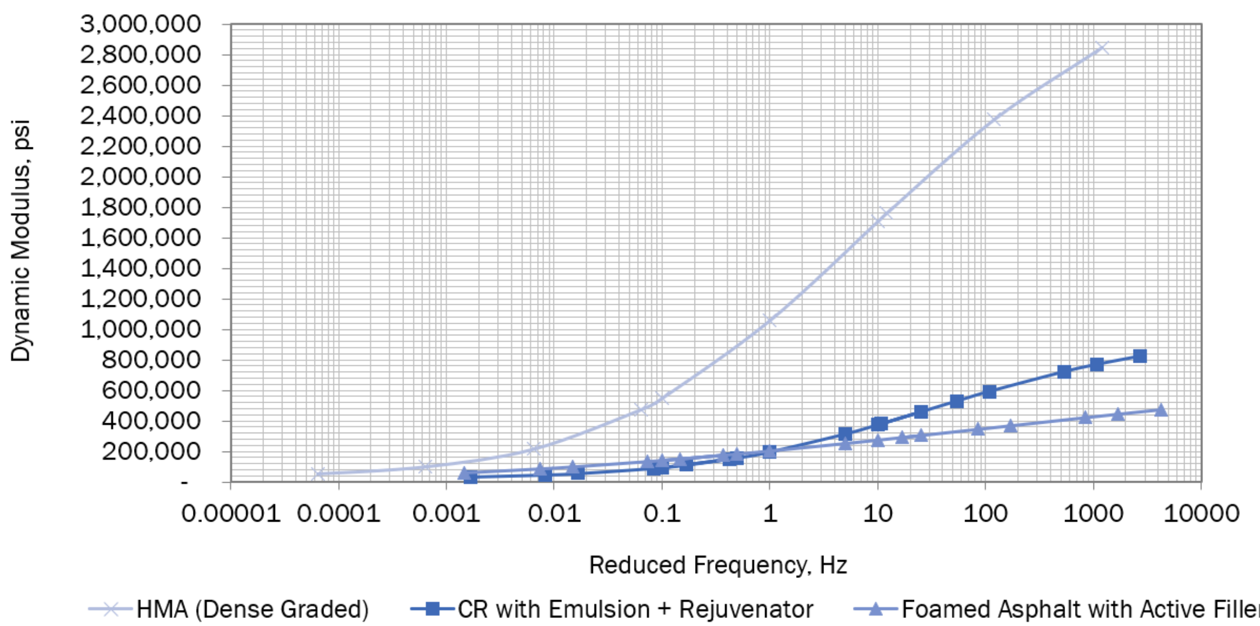


Figure 30. Plot of CR Mixture Master Curve with other Mixtures

When comparing the CR surface course mixture from this study with a foamed asphalt mixture, the dynamic modulus values tend to be similar at lower frequencies, with a distinct change occurring as frequency increases. The mixtures containing emulsion and rejuvenators had higher moduli than that of the mixture containing foamed asphalt at values greater than approximately 1 Hz. The stiffness of the CR mixture with emulsion and rejuvenator tends to vary in stiffness more with respect to loading frequency versus a foamed asphalt mixture. It is also indicative of the mixture being more temperature and loading dependent.

4.3.7 Field Performance

Since construction, the off-ramp section with the CR surface course has received approximately 140,000 ESALs over the course of 19 months. The section will receive approximately 200,000 ESALs by the end of the test cycle and a full distress analysis will be conducted at that time. Cracking (% lane area), rutting (mm), and IRI (in/mi) have been collected monthly. Figures 31-33 show the preliminary data collected thus far. Refinement of the data will be conducted at the end of the test cycle. The test section began to crack visually during the first winter months: however, the cracks remained tight and seemed to heal during the warmer, summer months. Cracking has remained relatively constant at 0.9% of the lane since it appeared. Photos showing initial cracking are provided in Figure 34.

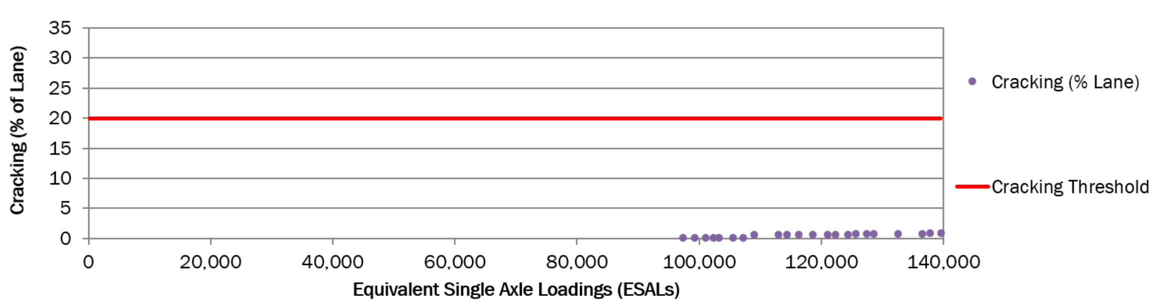


Figure 31. Preliminary Measured Cracking on Off-Ramp Section with CR Surface Course

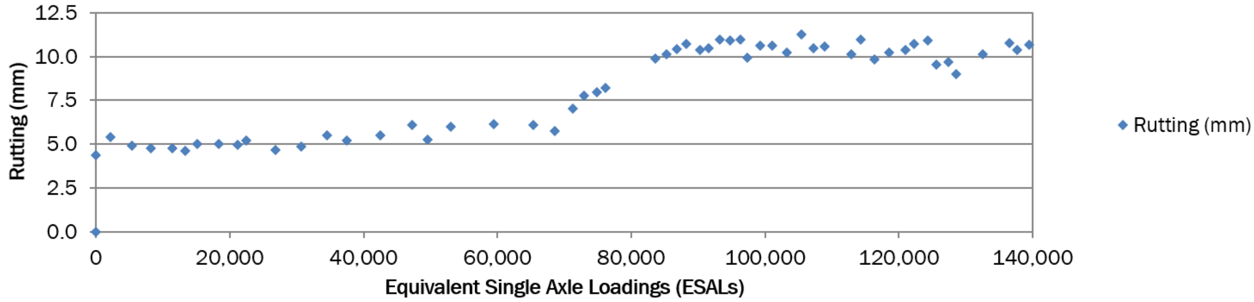


Figure 32. Preliminary Measured Rutting on Off-Ramp Section with CR Surface Course

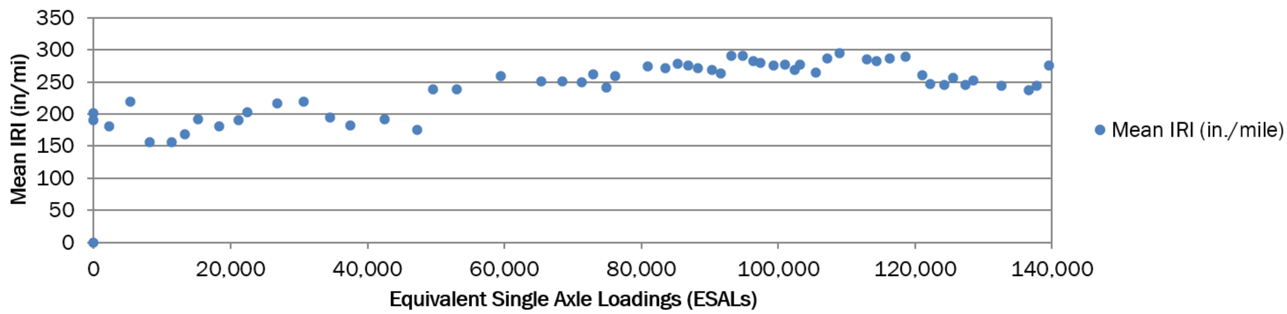


Figure 33. Preliminary Measured IRI on Off-Ramp Section with CR Surface Course

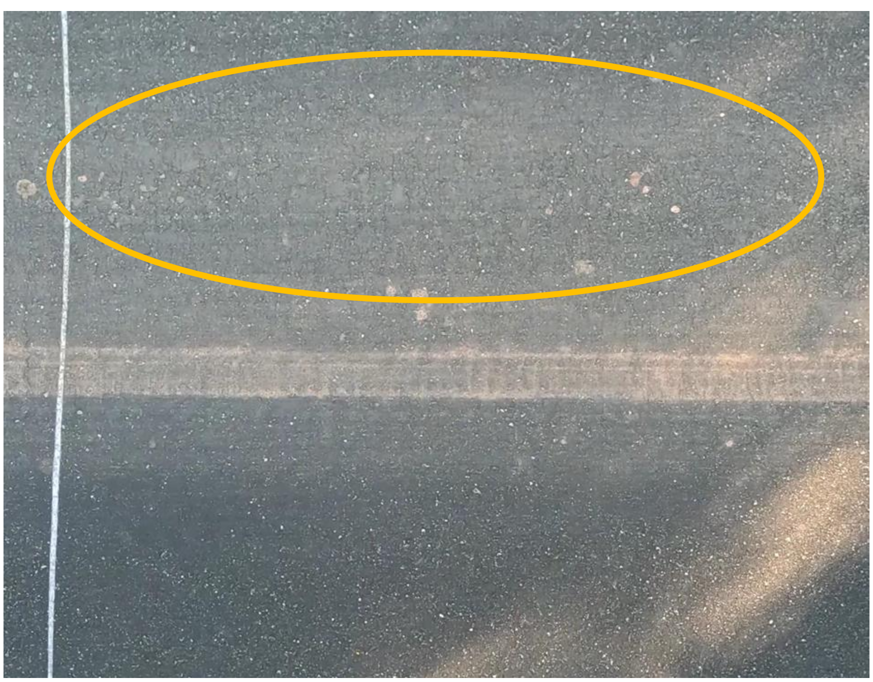


Figure 34. Initial Cracking on Off-Ramp Section with CR Surface Course

4.3.8 Pavement Distress Prediction using a Model Calibrated for a Pavement Structure Containing a CR Mixture as a Base or Intermediate Layer

In the previous chapter, the calibration coefficients in the pavement distress prediction models for flexible pavements with a CR layer were optimized to best match predicted and measured performance. For all sections used in the calibration process, however, the CR layer was a base or intermediate layer and not a surface course. The structural and mixture information for the CR mixture used as a surface course was input into the PMED software and the coefficients were adjusted to match the previously optimized model. Predicted distresses were output and compared to the measured distresses collected on CR surface course section from the beginning of the test cycle to the most recent distress data collection date at the time of this study (approximately 19 months). The measured and predicted bottom up cracking and total rutting using the global and optimized coefficients are shown in Table 26. The bias statistics between the global and optimized model predictions are shown in Table 27.

Table 26. Predicted versus Measured Distresses for CR Surface Mixture

Coefficient Set	Bottom Up Cracking (% lane area)	Total Rutting (in)
Global	1.448	0.4064
Optimized	1.42E-09	0.2140
Measured Distress	0.946	0.2502

Table 27. Bias Statistics between Global and Optimized Model

	Bias	
Coefficient Set	**Bottom Up Cracking (% lane area)**	**Total Rutting (in)**
Global	0.5017	0.1562
Optimized	-0.9465	-0.0362

The optimized model coefficients resulted in an underprediction of the bottom up cracking and the total rutting distress predictions, indicating the calibrated model does not accurately predict the distresses for a structure containing a CR surface course as it does a structure with a CR base or intermediate layer and therefore would require additional calibration efforts. At the time of this study, only fatigue-like cracking is presenting on the surface of the pavement. Forensics have not been conducted to verify the depth of the cracking. In the design software, the surface course of a flexible pavement allows for additional mixture specific inputs that non-surface course does not have (e.g. percentage asphalt content by weight of mix, mixture specific creep compliance, mixture specific Indirect Tensile Strength testing results, etc.). This data is not often collected on CR mixtures and default values were used. As previously stated, the default values in the software are based on HMA/WMA mixture characteristics and may not accurately depict what would be seen in a CR mixture. These inputs and the influence they have over distress prediction should be taken into consideration in future research on the topic.

4.3.9 Considerations for Design and Testing of CR Mixtures for Surface Course Applications

In this study the mixture design process and acceptance criteria were the same as would be followed for a CR base mixture, as a design process and acceptance criteria unique to CR surface courses does not exist in the current literature. This presents a knowledge gap to be addressed; however, before recommending a set of criteria and laboratory testing, the distress mechanisms of the pavement and the sensitivity of a CR mixture to traditional HMA/WMA testing must be investigated. The distress data from the off-ramp section constructed in this study will help develop a better understanding of the pavement distress mechanisms in the field. At this time of this dissertation, the distress data collection is partially complete. Conclusions on the failure mechanisms and what laboratory tests may be beneficial in predicting field performance for this type of mixture will be made following forensics at the end of the test cycle.

Research conducted on similar mixtures during the time of this study provided insight into quantifying CR mixture laboratory performance using testing methods and acceptance criteria developed around HMA/WMA characteristics and highlighted shortcomings in the current methods. The mixture performance testing centered around tests typically associated with balanced mix design: IDEAL-CT and HT-IDT. No testing criteria or thresholds typically exist specifically for CR mixtures for these tests. When investigating IDEAL-CT testing results on three CR mixtures, distinct differences in the calculated CT_{Index} value were noted (Table 28).

Table 28. CT_{Index} for various CR Mixtures

Mixture	CT_{Index}	Standard Deviation
Foamed Asphalt with Active Filler	19.1	5.6
Emulsion + Rejuvenator	117.9	14.1
Engineered Emulsion	81.8	24.9

The CT_{Index} of the CR mixture containing a rejuvenator was notably higher than both mixtures without rejuvenator. When evaluating IDEAL-CT results on these CR mixtures, the foamed asphalt had a lower CT_{Index} versus the emulsion mixtures. This is to be expected as the foamed asphalt mixture tested contained cement and is held together by spot welds versus a more thorough coating of asphalt that an emulsion provides. The values produced by the emulsion mixtures, however, were considered high versus the foamed asphalt and many HMA/WMA mixtures. For example, the minimum CT_{Index} for HMA in the state of Virginia is 70, and the rejuvenated CR mixture tested far exceeds that minimum. Mixture designs using emulsion (sometimes containing cement) tested at NCAT and other research facilities (Dong and Charmont, 2019) produced CT_{Index} values over 100 and pushing up to 400. Initially values like these may appear to indicate that the mixture is highly cracking resistant, but these values may be inflated to the point where they are not representative of the true cracking performance of the mixture.

When evaluating index values produced using the developed equation, one of the primary assumptions made about the characteristics of the mixture being tested is that the air voids are typically targeted at 7% +/- 0.5. With CR mixtures, air voids values are often much higher, often falling between 10% and 20%. While the equation itself does not factor in air voids, the development of it was based on mixtures that were all within the targeted range, thus producing more consistent data curves for analysis. The dense graded HMA specimens that were tested for the development of the CT_{Index} equation led to the conclusion that the inflection point of the post-peak slope was, on average, at the point where the load was reduced to 75% of its peak (Zhou, 2019). Therefore, the researchers concluded that the inflection point would be calculated at 75% of the peak load for all CT_{Index} values. While this may be applicable to dense graded mixtures, when the data curves produced by CR mixtures tested at the NCAT lab were evaluated, the post-peak inflection point did not often land at the 75% mark.

Mixtures containing emulsion and rejuvenator presented a longer strength curve with a lower peak versus the mixture containing foamed asphalt (Figure 35). This shift in the curve shape alters the inflection point and may not accurately capture the cracking performance of a rejuvenated CR mixture. It is hypothesized that, as is, the fundamental inputs into the equation to determine the CT_{Index} may not be applicable for all CR mixtures.

Further research should be conducted on typical CR mixtures to determine their loading curves and whether having a set inflection point is applicable or if the inflection point determination method should be different between mixture types or by mixture characteristics.

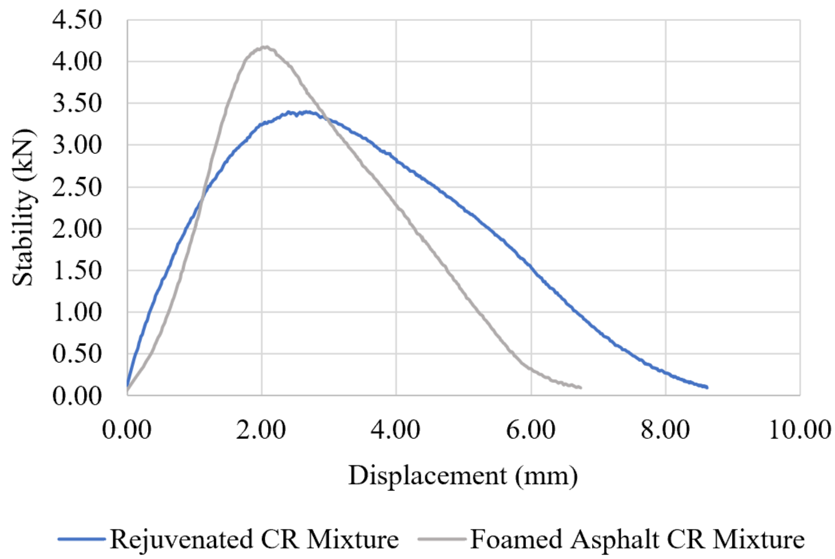


Figure 35. CR Mixture Strength Curve Comparison (Turochy, 2023)

HT-IDT testing on those same mixtures indicated low resistance to rutting, with the rejuvenated mixture having the lowest strength (Table 29). The rutting resistance of these CR mixtures followed the reverse of the results found in the IDEAL-CT testing. This also follows the general Balanced Mix Design logic that a mixture must balance cracking susceptibility and rutting susceptibility. The HT-IDT strengths, however, are far below most standards (e.g. Alabama Department of Transportation uses a 20-psi threshold (National Asphalt Pavement Association, 2022)) and do not reflect rutting results seen in the materials in the field, indicating that this test and/or acceptance thresholds may not be adequate for capturing the performance of CR mixtures.

Table 29. HT-IDT Strength for various CR Mixtures

Mixture	HT-IDT Strength (psi)	Standard Deviation
Foamed Asphalt with Active Filler	9.39	0.67
Emulsion + Rejuvenator	5.89	0.33
Engineered Emulsion	6.67	1.13

While simple cracking and rutting tests help to quantify mixture performance, testing that explores deeper into asphalt pavement fatigue mechanisms is essential to understanding CR mixtures. One example of fatigue testing is cyclic fatigue, which could capture CR mixture performance. However, it is important to consider the testing conditions used as they are often selected based on HMA/WMA characteristics. Typical testing conditions used for HMA/WMA may not fully capture a CR mixture's behavior where it is expected to be most relevant in service. Cyclic fatigue testing for CR mixtures was explored, aiming to investigate methods for testing. Conditioning temperature of the specimen was one of the key testing parameters evaluated. For HMA/WMA, the temperature is commonly selected based upon binder grade. In the case of many CR mixtures, a binder grade is not known or is far higher than is typically seen in HMA/WMA, thus warranting a need for investigation into the establishment of appropriate testing temperatures. Another testing parameter to be considered is testing frequency. Ten Hz is often the selected frequency for cyclic fatigue testing of HMA/WMA, balancing a realistic loading scenario while also being able to capture quality data and the viscoelastic behavior of the mixture. It is unknown whether or not this frequency accurately captures the characteristics of CR mixtures. Testing should be conducted to evaluate the collected fatigue data versus measured field distress data and potentially other laboratory tests that capture associated data.

Based on the data collected and analyzed in this study it is recommended that prior to establishment of CR mixture criteria for surface course applications that additional performance tests surrounding cracking, rutting, and other mixture performance characteristics be explored. Suitability of existing BMD-type cracking tests for CR mixtures should be investigated. For rutting, testing of mixtures using the Hamburg Wheel Tracking Test (HWTT) may be beneficial as it not only provides rutting susceptibility data, but insight into moisture susceptibility as well. Results from previous research also point towards potential for application of BMD concepts for CR mixtures, with a need for balancing cracking and rutting performance.

Exploration into additional testing that quantifies durability, raveling resistance, and surface properties for CR mixtures intended for surface course applications is essential. CR mixtures often have lower binder content than HMA/WMA surfaces, reducing the mastic adhering the mix matrix together. While the HWTT can provide some insight into the surface deterioration potential of mixtures, including testing such as Cantabro testing (abrasion loss/raveling) could provide additional quantification of the mixtures resistance to raveling. Tests such as the Circular Texture Meter for measuring macrotexture of the surface also provides data on essential properties of a surface mixture that should be captured to predict field performance.

4.4 Findings

Findings for this chapter include:

- Rejuvenator content does impact mixture performance. Analysis within this study showed that adjusting the rejuvenator content within the mixture influenced the strength values and an optimum content was identifiable.

- Using a polymer modified emulsion in combination with a rejuvenator resulted in a final mixture design what passed dry strength and MSR requirements
- The placement of the rejuvenated CR mixture as a surface course was identical to that of a traditional CR mixture.
- Samples taken during production for laboratory compaction, curing, and testing showed slightly reduced strengths versus the data collected during mixture design. This was likely due to excess moisture in the RAP during construction.
- Dynamic modulus testing indicated that the combination of a polymer modified emulsion and a rejuvenator produced higher moduli value than a typical foamed asphalt CR mixture.
- The predicted distress models calibrated for flexible pavements containing a CR base or intermediate layer did not accurately predict the pavement distress progression seen in flexible pavements containing a CR base or intermediate layer.
- Laboratory performance testing of CR mixtures follows the general Balanced Mix Design logic that a mixture must balance the cracking and rutting performance.

4.5 Conclusions

To develop a CR mixture design for surface course applications, it is important to understand how each element within the mixture influences the overall laboratory performance testing result. Previous research (Turochy and Bowers 2024) notes that the addition of rejuvenators to CR mixtures can lead to increased density values, potentially resulting in improved mixture performance. While these additions can prove to be beneficial, they also introduce new materials to the mixture design process. This requires additional analysis, evaluating the impact on the designs laboratory performance, field production and placement, and field performance. Additional testing should also be investigated, as the documented use of CR mixtures as a surface course is limited, leaving a gap in knowledge. To better understand the role rejuvenator plays in the laboratory performance of a CR mixture design, three different rejuvenator contents were tested. The resulting strength and moisture susceptibility data for each mixture design was analyzed, and no statistical differences were noted between the rejuvenator contents (with the exception of 0% versus 7%). There were, however, numerical differences in the performance data that determined the selection of the optimum rejuvenator content. This process resulted in an optimum rejuvenator content of 3.5% (by weight of RAP binder). Once the optimum rejuvenator dosage was selected, a standard mixture design panel testing four different 3% SBS modified emulsion contents was developed and the final mixture design for production was selected based on the strength and moisture susceptibility testing results.

Once the appropriate design was selected, it was produced in a mobile CR plant and placed on the NCAT Test Track off-ramp during the 2024 construction season. During this process, the mixture production was monitored to ensure the design was followed and its visual appearance matched that of the lab produced mixture. No differences between the placement of the CR surface course mixture versus a typical CR base mixture were noted. Samples taken during production for laboratory compaction, curing, and testing showed slightly reduced strengths versus the data collected during mixture design; however, the density of the compacted mat and plant mixed laboratory compacted specimens was higher than recorded in the mixture design phase. Dynamic modulus testing of plant mixed, lab compacted specimens produced a curve that falls between a typical foamed asphalt mixture and a typical HMA.

The pavement structure and mixture properties were used with the PMED software and the calibration coefficients developed in the previous chapter for flexible pavement structures containing a CR base or intermediate layer to determine the calibrated coefficient's ability to predict pavement distresses. The measured distresses to date of the off-ramp section (19 months at the time of this study) was compared to the predicted distresses produced by both global coefficients and calibrated coefficients. Results showed that the global coefficients better predicted pavement distresses than the calibrated coefficients, indicating a need for recalibration when considering a CR mixture as a surface course instead of a base or intermediate layer.

A discussion on laboratory performance testing conducted on CR mixtures to date resulted in recommendations for future testing and potential needs for modifications to existing testing methods, thresholds, and testing criteria. Cracking test discussions concluded that IDEAL-CT testing likely does not provide a thorough assessment of CR mixtures containing emulsion and/or rejuvenators, as mixtures exhibited a low peak and long tail in the load-displacement curve that was not fully captured. During data analysis, different strength curves with inflection points that were not at the 75% peak load mark were observed for CR emulsion and rejuvenated mixtures versus a foamed asphalt mixture, which should be considered in future testing. Comparison against results from other cracking tests was also recommended. Results from HT-IDT showed values below typical HMA/WMA specifications, indicating a need for other testing to capture rutting resistance and/or a modification in specification thresholds. Additional testing centering around fatigue, raveling, and surface characteristics were discussed and the shortcomings of the testing and thresholds were highlighted.

Future research pertaining to use of CR mixtures as a surface course should validate laboratory testing methods and criteria and determine if the standards developed around HMA/WMA are suitable for use with CR mixtures. It is recommended that multiple cracking and rutting tests, including IDEAL-CT and HT-IDT, be conducted on a single mixture design to better quantify the existing specifications and testing methods ability to capture CR mixture performance. Balanced Mixture Design concepts show promise in being applicable to CR mixtures; however, further testing must be conducted before testing and performance criteria are established.

Chapter Five: Conclusions and Recommendations

5.1 Conclusions

This goal of this dissertation was to evaluate innovations in CR mixture design, mechanistic-empirical structural design, and applications of CR pavements. The objectives of this study were (1) to determine the suitability of existing global coefficients in the AASHTOWare Pavement ME Design software pavement distress prediction models for use in structural design of flexible pavements containing a CR base or intermediate layer and (2) to assess the mixture and structural design and evaluation process of rejuvenated CR mixtures for surface course applications. A summary of the conclusions from each chapter are as follows:

Chapter Three, Calibration of Pavement Distress Prediction Model Coefficients for Cold Recycled Pavements:

- The global coefficients used in version 3.0 of the AASHTOWare Pavement ME Design software for flexible pavement distress prediction models were inadequate for accurately predicting distresses in flexible pavements containing a CR layer.
- Based on results from a Welch's t-Test, the total rutting prediction was statistically similar between all mixture-specific k-coefficients with the exception of the global versus test track based coefficients.
- The current recommended method of categorizing all cracking of unknown origin as bottom up cracking produced a distress prediction model with poor statistics, a shortcoming of the current recommended calibration method.
- Based on the discussions within this work, the existing transfer functions may not accurately capture distress formation of CR pavements and may not consider mixture

properties indicative of CR pavement performance as they are based upon empirical HMA data.

- On pavement structures with 5 inches of total AC thickness with the CR layer being placed directly on a granular base, subgrade rutting was overpredicted up to 4 times past the total measured rutting.
- The current PMED software lacks typical CR material inputs and appropriate transfer functions that predict the distress mechanisms seen within CR pavements.

Chapter Four, Rejuvenated Cold Recycled Mixture Design for Surface Course Applications:

- Use of a polymer modified emulsion and rejuvenator produced a CR mixture that passed all mixture design criteria and could be constructed in the same fashion as other typical CR mixtures.
- The AASHTOWare Pavement ME Design software and the calibration coefficients developed in the previous chapter for flexible pavement structures containing a CR base or intermediate layer did not accurately predict pavement distress for a pavement structure with a CR surface course.
- Results from HT-IDT showed values below typical HMA/WMA specifications, indicating a need for other testing to capture rutting resistance and/or a modification in specification thresholds.

5.2 Recommendations

With the mixture design, laboratory testing, structural design, and applications of CR mixtures continuing to advance, one of the key topics highlighted throughout this dissertation was the history behind the established methods. The currently established laboratory performance testing methods and performance criteria across all discussed testing types are established based on HMA/WMA characteristics and may not accurately represent the performance of a CR mixture. The establishment of methods based on HMA/WMA continues past laboratory testing and into the mechanistic-empirical design distress prediction models. From global (default) values to the mixture properties considered in the distress prediction equations, HMA/WMA was the basis for their selection. This dissertation helped in identifying the limitations of the existing processes surrounding laboratory performance testing and mechanistic-empirical structural design via AASHTOWare Pavement ME Design software. The identification of limitations is the beginning of a process of establishing performance criteria and structural pavement design methods with CR mixtures and their characteristics as a basis. With this in mind, the following recommendations can be made:

- The bottom up cracking model should be assessed to allow for cracking propagation to originate from a particular layer versus the current option of ranges of total AC thickness to help better model the distress predictions.
- Transfer functions that consider material property inputs indicative of CR mixture performance as well as typical distress mechanisms should be developed.
- CR pavements as a base, intermediate, and surface course should be further investigated via field forensics and laboratory testing to better understand their distress mechanisms.
- As per the discussion in section 4.3.9 of this dissertation, laboratory testing criteria and methods for performance testing that accurately captures CR mixture performance should be established if current testing methods and criteria

established based on HMA/WMA in proven inadequate. This could contribute to both mixture design acceptance and to the mechanistic-empirical pavement distress prediction equations. Further, multiple cracking and rutting tests through a BMD framework should be conducted on CR mixtures to better assess the existing specifications and testing methods and their ability to capture CR mixture performance.

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Appendix A: PMED Transfer Functions for Flexible Pavements from Output Files

AC Rutting

$$\frac{\varepsilon_p}{\varepsilon_r} = k_z \beta_{r1} 10^{k_1} T^{k_2 \beta_{r2}} N^{k_3 \beta_{r3}}$$

$$k_z = (C_1 + C_2 * depth) * 0.328196^{depth}$$

$$C_1 = -0.1039 * H_\alpha^2 + 2.4868 * H_\alpha - 17.342$$

$$C_2 = 0.0172 * H_\alpha^2 - 1.7331 * H_\alpha + 27.428$$

Where:

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

$\varepsilon_p = plastic\ strain({}^{in}/_{in})$

$\varepsilon_r = resilient\ strain({}^{in}/_{in})$

$T = layer\ temperature(^\circ F)$

$N = number\ of\ load\ repetitions$

Subgrade rutting

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

$\delta_a = permanent\ deformation\ for\ the\ layer$

$N = number\ of\ repetitions$

$\varepsilon_v = average\ vertical\ strain (in/in)$

$\varepsilon_0, \beta, \rho = material\ properties$

$\varepsilon_r = resilient\ strain (in/in)$

Fatigue cracking

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

$$C = 10^M$$

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

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

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

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

Table 6-1. Recommendation for the Flexible Pavement Transfer Function Calibration Parameters to Be Adjusted for Eliminating Bias and Reducing the Standard Error

Distress		Eliminate Bias	Reduce Standard Error
Total Rutting	Unbound Materials and HMA Layers	k_{r1}, β_{s1}, or β_{r1}	kr_2, kr_3, and β_{r2}, β_{r3}
Load-Related Cracking	Alligator Cracking	C_2 or k_{f1}	k_{f2}, k_{f3}, and C_1
	Longitudinal Cracking	C_2 or k_{f1}	k_{f2}, k_{f3}, and C_1
	Semi-Rigid Pavements	C_2 or β_{c1}	C_1, C_2, C_4
Non-Load-Related Cracking	Transverse Cracking	β_{f3}	β_{f3}
IRI		C_4	C_1, C_2, C_3

Appendix C: Global Coefficients for Bottom Up Cracking and Total Rutting

Bottom Up Cracking

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

Total Rutting

Flexible Total Rutting Model Coefficient	PMED 3.0 Global Value
βr1	0.4
βr2	0.52
βr3	1.36
βs1: Granular Base	1
βs1: Subgrade	1