Support for Balanced Asphalt Mixture Design Specification Development in Missouri



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16. Abstract

The primary objective of the study conducted herein was to assist MoDOT in the development of a balanced mix design asphalt specification in Missouri. At the onset of the study, a detailed climatic study was conducted. Next, asphalt performance testing was carried out across a range of temperatures to establish property-temperature relationships. Eleven mixtures were tested using the Disk-Shaped Compact Tension Test (DCT) across a range of low temperatures, while the Hamburg Wheel Tracking Test (HWTT) was conducted across a range of high temperatures. Mixture testing was carried out on both plant-produced mixtures and field cores. Two additional cracking tests were investigated in this portion of the study: the Illinois Flexibility Index Test, or I-FIT, and the IDEAL cracking test. Field performance data from MoDOT's pavement management database was extracted to assist with the development of specification thresholds for the cracking and rutting tests investigated. For BMD specification development, a novel approach was taken herein whereby recommended specification thresholds were introduced for all three cracking tests currently under investigation in Missouri and across the Midwest (DC(T), I-FIT and IDEAL). While the current study focused on mainline, high-type mixes, preliminary recommendations for shoulder mixes and other low traffic paving applications are provided, which may be of interest in larger urban areas and for municipal projects where closer control of asphalt performance via testing and materials investment is of concern. Stone-Matrix Asphalt mixes were not considered in this study. The recommended tests and thresholds provided herein should be viewed as preliminary and can be adjusted based on practical considerations and stakeholder discussions in a consensus process. Further validation of the proposed BMD tests and limits via long term field monitoring or testing on a controlled test road or test track facility is highly recommended. Additional work to apply the recommended BMD to quality control specifications and performance-related quality assurance specifications in a practical manner is also recommended.

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SUPPORT FOR BALANCED ASPHALT MIXTURE DESIGN SPECIFICATION DEVELOPMENT IN MISSOURI

Final Report
July 2020

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ABSTRACT

The primary objective of the study conducted herein was to assist MoDOT in the development of a balanced mix design asphalt specification in Missouri. At the onset of the study, a detailed climatic study was conducted. This research task produced data that can be used in the future to tailor the performance-based specification developed herein for the various climatic regions that exist in Missouri. Next, asphalt performance testing was carried out across a range of temperatures to establish property-temperature relationships. Eleven mixtures were tested using the Disk-Shaped Compact Tension Test (DCT) across a range of low temperatures, while the Hamburg Wheel Tracking Test (HWTT) was conducted across a range of high temperatures. Mixture testing was carried out on both plant-produced mixtures and field cores. Two additional cracking tests were investigated in this portion of the study: the Illinois Flexibility Index Test, or I-FIT, and the IDEAL cracking test. Field performance data from MoDOT's pavement management database was extracted to assist with the development of specification thresholds for the cracking and rutting tests investigated.

For BMD specification development, a novel approach was taken herein whereby recommended specification thresholds were introduced for all three cracking tests currently under investigation in Missouri and across the Midwest (DC(T), I-FIT and IDEAL). While the current study focused on mainline, high-type mixes, preliminary recommendations for shoulder mixes and other low traffic paving applications are provided, which may be of interest in larger urban areas and for municipal projects where closer control of asphalt performance via testing and materials investment is of concern. To this end, the recently developed Illinois Tollway asphalt mix design specification was used to establish initial specification thresholds for non-surface and non-mainline (shoulder) paving lifts in Missouri. Stone-Matrix Asphalt mixes were not considered in this study.

The recommended tests and thresholds provided herein should be viewed as preliminary and can be adjusted based on practical considerations and stakeholder discussions in a consensus process. Further validation of the proposed BMD tests and limits via long term field monitoring or testing on a controlled test road or test track facility is highly recommended. Additional work to apply the recommended BMD to quality control specifications and performance-related quality assurance specifications in a practical manner is also recommended. Attention to sample procurement and reheating should be included, as well as testing standardization, repeatability, and balancing of specification risk between the contractor and the owner agency.

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EXECUTIVE SUMMARY

Thermal cracking and rutting are deterioration modes that significantly affect asphalt pavement longevity and ride quality in the United States (US). Thermal cracking is a prevalent problem in colder areas, whereas rutting is a pavement damage form that is more common in warmer regions of the country, and in areas of heavy traffic volume. Cracking under cyclic loading can occur in virtually any climate in the US - in the form of fatigue cracking in the case of cyclic traffic loading, and in the form of block cracking in the case of diurnal, or cyclic environmental conditions, particularly after asphalt aging has occurred. Reflective cracking can be driven by cyclic traffic and/or environmental loading (and in most cases, probably both). In general, asphalt mixes should be designed in a manner that tailors their physical (or 'performance') characteristics to the environmental conditions and vehicle loadings expected during their service lives. Thus, mixture designs that include performance testing can more reliably produce desired field behavior.

Over twenty years have passed since the Superpave mix design system was developed. Three levels were envisioned; however, the required performance tests were never fully implemented, and the resulting design method remains to be based primarily on mixture volumetrics. More recently, a number of DOT's have recognized the need to finally supplement the Superpave design method with simple performance tests to produce more reliable designs, particularly for modern, heterogeneous recycled asphalt mixtures. The Federal Highway Administration (FHWA) became involved in this movement by initiating a Balanced Mix Design (BMD) (or Performance-Engineered Mix Design (PEMD)) Expert Task Group (ETG) to promote the standardized use of performance testing in the design of asphalt mixtures. This has helped to increase the pace of research aimed at implementing performance-based mixture design across the US. The specific performance tests selected, along with the testing thresholds being adopted, tend to vary on a regional basis.

The main motivation of the study conducted herein was to assist MoDOT in the movement towards balanced mix design in Missouri. At the onset of the study, a detailed climatic study was conducted. This research task produced data that can be used in the future to tailor the performance-based specification developed herein for the various climatic regions that exist in Missouri. Pavement temperature maps were developed for Michigan (MI), Illinois (IL), Indiana (IN), Wisconsin (WI) and Missouri (MO) to provide testing temperatures using the Long-Term Pavement Performance (LTTPBind) climate database. Maps are presented in both dot-style and contour-style formats. Pavement temperatures were estimated at two reliability levels and at different depths from the pavement surface.

Next, asphalt performance testing was carried out across a range of temperatures to establish property-temperature relationships. Eleven mixtures were tested using the Disk-Shaped Compact Tension Test (DCT) across a range of low temperatures, while the Hamburg Wheel Tracking Test (HWTT) was conducted across a range of high temperatures. Mixture testing was carried out on both plant-produced mixtures and field cores. Two additional cracking tests were investigated in this portion of the study: the Illinois Flexibility Index Test, or I-FIT, and the IDEAL cracking test. Field performance data from MoDOT's pavement management database was extracted to assist with the development of specification thresholds for the cracking and rutting tests investigated.

For BMD specification development, a novel approach was taken herein whereby recommended specification thresholds were introduced for all three cracking tests currently under investigation in Missouri and across the Midwest (DC(T), I-FIT and IDEAL). While the current study focused on mainline, high-type mixes, preliminary recommendations for shoulder mixes and other low traffic paving applications are provided, which may be of interest in larger urban areas and for municipal projects where closer control of asphalt performance via testing and materials investment is of concern. To this end, the recently developed Illinois Tollway asphalt mix design specification was used to establish initial specification thresholds for non-surface and non-mainline (shoulder) paving lifts in Missouri. Stone-Matrix Asphalt (SMA) mixtures were not considered in this study.

Based on the work conducted herein, the following conclusions were drawn:

- Missouri is ready for preliminary roll out of BMD. University and MoDOT research, along with the comfort level demonstrated by Missouri contractors in discussions such as those held at MAPA-MoDOT joint meetings, all point to the fact that Missouri is ready to begin implementing BMD, starting with high-type, mainline mixes.
- All three cracking tests appear to be suitable for use in Missouri BMD, i.e., the DC(T), I-FIT and IDEAL. These tests have all been shown to relate laboratory testing results to field cracking performance, with the DC(T) designed to more closely control low temperature and block cracking, and the I-FIT and IDEAL tests designed to control top-down fatigue and reflective cracking. The DC(T) appears to be more inclusive of the use of higher amounts of recycled materials (RAP, RAS, GTR), and more repeatable, while the I-FIT and IDEAL tests appear to be more contractor friendly and appropriate for QC and QA as well as offering simplicity in the design stage.
- The Missouri climate differs fairly significantly from the North-West to South-East corners of the state, which in turn affects pavement temperature at the surface, and at depth, ranging from 4 to 12 degrees Celsius depending on a number of variables. The cracking and rutting tests investigated were found to be sensitive to test temperature. Thus, the BMD recommended herein should be considered as a starting point, calibrated to average conditions in the state.
- Using a reliability-based approach for rounding up minimally acceptable specification thresholds based on field observations and testing of core samples, it was possible to develop a rational set of BMD recommendations for high, medium, and low criticality mixtures, based on design traffic. The thresholds developed are in reasonable agreement with values currently under consideration by MoDOT.

Based on the work conducted herein, the following recommendations are made:

- The recommended tests and thresholds provided herein should be viewed as preliminary and can be adjusted based on practical considerations and stakeholder discussions in a consensus process.
- Further validation of the proposed BMD tests and limits via long term field monitoring or testing on a controlled test road or test track facility is highly recommended.
- Future studies should be conducted to establish specification thresholds for SMA mixes.

- Tailoring of the specification to varied climate zones in Missouri can be easily accomplished using the results provided herein, once the BMD begins to gain acceptance and goes through fine-tuning iterations after initial rollout.
- Additional testing of shoulder and low volume mixes, and identification of field sections with evidenced rutting and/or stripping should be pursued, to determine if the recommended Hamburg thresholds can be relaxed from the conservative starting points recommended herein.
- Additional work to apply the recommended BMD to quality control specifications and performance-related quality assurance specifications in a practical manner is also recommended. Attention to sample procurement and reheating should be included, as well as testing standardization, repeatability, and balancing of specification risk between the contractor and the owner agency.

Chapter 1

1. INTRODUCTION

1.1. Background

According to the Federal Highway Administration (FHWA), there are a total of 4.1 million miles of public roads in the US, of which about 2.7 million miles are paved (FHWA, 2018). Reportedly, 94% of those 2.7 million miles are surfaced with asphalt (NAPA, 2020). The asphalt market is not only limited to highway infrastructure- between 85 and 90% of all runways at the nation's 3,364 commercial airports are surfaced with asphalt, and over 90% of parking areas are surfaced with asphalt pavement, among other uses. The US has around 3,500 asphalt plants producing a total of 400 million tons of asphalt paving materials annually (NAPA, 2020). Clearly, significant asphalt pavement infrastructure exists around the nation, demanding significant federal, state, and local spending each year and supporting hundreds of thousands of workers in the transportation industry.

Asphalt concrete (AC), which is one of the principal materials used in highway and airfield pavements, is a complex, viscoelastic material. Hot-mix asphalt (HMA) is highly influenced by temperature and loading rate, and asphalt mixtures must be properly designed to resist various modes of deterioration (distresses) over time that are driven by environmental effects along with traffic loading. Among the most common distresses associated with asphalt pavements related to temperature variation and traffic loading are thermal cracking (also known as low-temperature cracking) and rutting, respectively. Many other forms of cracking distress also exist, along with stripping and raveling.

Thermal cracks are considered the main cause of pavement deterioration in northern, mid-continental and even desert climates in the US. Thermal cracks can be initiated by a single low temperature event or by multiple warming and cooling cycles. Once initiated, these cracks can propagate further due to traffic loading, freeze-thaw cycles and other environmental factors. In a simplified sense, thermal cracking occurs when the temperature at the pavement surface decreases to produce thermally-induced tensile stresses that exceed the local tensile strength of the mixture. Thermal cracks, as shown in Figure 1-1, are transverse in direction (run perpendicular to the roadway centerline) and are often spaced in a relatively uniform fashion. The cracks are thought to initiate at the top of the pavement surface and propagate downward through the layer. When thermal cracks have propagated and widened over time, water enters into the pavement and progresses to the layers below, causing damage and degradation to the entire pavement structure. Secondary distresses such as raveled cracks, cupped or tented cracks, stripping and potholing can progress from a thermally cracked pavement. Thermal cracking is also sometimes accompanied by block cracking, since the mechanisms driving these two distress modes appear to be related (Wang and Buttlar, 2019). One of the popular standardized performance tests used to evaluate thermal cracking is the Disk Shaped Compact Test, or DC(T), which has been successfully implemented in characterizing fracture behavior of asphalt mixtures at low temperatures (Wagoner et al., 2005a). A portion of the research conducted herein addresses the testing of asphalt mixtures at low temperatures.

Figure 1-1. Thermal cracking (Delgadillo, 2014)

Under high temperatures and heavy loads, asphalt layers in a pavement structure may become prone to rutting. Rutting in an asphalt layer is characterized by depressions in the mixture, usually occurring along the wheel paths. This type of deterioration is one of the main distresses in asphalt pavements, especially in high summer temperatures and/or under heavy truck loading. An example of rutting in the asphalt layer is illustrated in Figure 1-2. A major complication of rutting is that significant depressions in the pavement tend to collect rain water, which can result in a hydroplaning and loss of control in vehicles traveling at moderate or high speeds. A widely used performance test to evaluate the rutting potential of asphalt mixtures is the Hamburg Wheel Tracking Test (HWTT). The HWTT procedure is introduced in Chapter 3.

Figure 1-2. Rutting damage (Delgadillo, 2014)

Thermal and block cracking, rutting, and other forms of pavement distress such as fatigue, reflective cracking and moisture damage can limit asphalt pavement longevity unless they are adequately addressed in material design, production, and construction. This report focuses on using modern asphalt mixture performance tests during the mixture design stage to optimize or "balance" mixture performance. Similar performance tests and their associated specification thresholds can be used for the purposes of evaluating asphalt mixture production and construction, as part of quality control and assurance specifications.

1.2. Research Motivation

Performance-based mixture design or *Balanced Mix Design* (BMD) is evolving in earnest in the U.S asphalt industry. Starting with the development of the Superpave program 20 years ago, the development and implementation of new performance tests to supplement volumetric mix design fell short of practical expectations and has yet to be fully implemented (West et al., 2018). To address this shortcoming, researchers and agencies such as the Federal Highway Administration (FHWA) have joined together to developed and promote Balanced Mix Design (BMD) or Performance-Engineering Mix Design (PEMD) to advance the use of performance testing in asphalt mixture design, control and acceptance. As a result, state and local agencies are beginning to experiment with these modern performance-based specifications in their mixture design requirements, and in some cases, in their quality assurance specifications. In the literature review section of this report the BMD/PEMD concept is explained in more detail in Section 2.1.

One of the motivating factors behind the research conducted herein is the need to take steps towards the widespread implementation of BMD in Missouri. In this research, differing types of asphalt mixtures in the lab and field have been evaluated across multiple temperatures to evaluate cracking and rutting performance links between the lab and the field. In addition to recommending standard testing temperatures for the state of Missouri during early specification rollout, the development of a database of performance testing results at different temperatures was deemed necessary for future tailoring of mix designs ranging from the colder North-West corner, to the hotter South-East corner of the state of Missouri ('boot heel').

In the past, asphalt mixture characterization has mainly been correlated with the binder used, that is to say, the performance of a mixture in the field is often thought to be related to the Performance-Grade (PG) of the binder. For instance, if a plan grade is set as PG64-22 for a mixture, it is implied that the mix will perform across an in-service pavement temperature range of 64°C to -22°C. However, with modern ingredients in asphalt binders and mixtures such as polymers, warm-mix additives, Polyphosphoric Acid (PPA) modification, ground tire rubber (GTR), fibers, reclaimed asphalt pavement (RAP), recycled asphalt shingles (RAS), etc., actual asphalt mixture performance has become largely disconnected from the plan PG binder grade used. A central feature in BMD is to provide a method for designing and controlling a mixture after all of its modern components have been added, mixed and age-conditioned in a manner that simulates full-scale plant and laydown operations, and field aging.

1.3. Research

The work conducted herein was carried out under MoDOT research project number TR201811, "Support for Balanced Asphalt Mixture Design Specification Development in Missouri." Six

main tasks were conducted in this study, including:

1. **Literature Review**: A review of the literature was conducted with a focus on new literature in performance testing and specifications. This serves to complement a recently completed comprehensive literature review conducted by the PI on performance testing and balanced mix design for the Illinois Tollway (Buttlar et al., 2020). Results were used to guide recommendations for the purposes of experimental design and specification framework assembly in this study.

2. **Experimental Design, Sampling, and Specimen Fabrication**: In consultation with MoDOT and MAPA, a test matrix of laboratory and field projects/mixtures was created. The goal was to focus on high critical-type Superpave mixes (MoDOT Sec. 403 "B" and "C" mixes), but did not include SMA mixes. Although not tested or validated herein, the findings allowed extrapolation of results to make preliminary recommendations for lower criticality mixes, such as "E" and "F" type mixes. A combination of laboratory compacted and field cored mixtures were included.

3. **Performance Testing of Lab and Field Mixtures**: Selected mixture performance tests were carried out in an effort to link mix performance test results to observed field performance (rutting, cracking, moisture damage, and overall pavement condition and ride quality). The mixture tests conducted include:

- High temperature Hamburg wheel tracking testing (submerged)
- Intermediate temperature flexibility index testing (I-FIT FI)
- Intermediate temperature IDEAL test (IDEAL CT)
- Low temperature fracture testing (DC(T) fracture energy).

The above suite of testing required approximately 12 cores to be taken per site. In addition to the cores, as-produced mix samples available from other projects were tested. A project shadowing process was used, where mixture performance test limits were not imposed on the contractor ahead of time. Rather, results from performance tests and suggestions for more balanced mix designs were shared with the contractor after construction in a collaborative fashion. This database was expanded by incorporating data from mixes tested under a recently completed study for the Illinois Tollway. Field performance data was obtained from MoDOT's online portal, which contains digital images, distress categorization and quantification, and pavement section ratings using the PASER system, along with international roughness index (IRI) and rut depth.

In order to properly assess core test results versus field performance data, it was necessary to predict how mixture performance properties would shift as a function of aging. For instance, it was necessary to shift core test values backwards in age to estimate properties in the as-produced condition (year zero, short-term plant aging only). This allowed the research team to set performance test limits at the selected short-term lab aging level, while still being able to control distress (rutting, cracking, stripping) at a desired level of reliability and at a chosen serviceability deterioration rate (which in turn controls the longevity of the designed and constructed asphalt lift).

4. **Establishment of Draft Specifications**:

Researchers worked in consultation with MoDOT and MAPA to develop a draft specification for the inclusion of mix performance testing as part of mix design using lab and field data, aging results, and field performance data. Reliability was considered, where mixes with higher criticality (higher traffic) were specified to meet more stringent performance thresholds. Different criteria were established for surface vs. non-surface course mixes, and for mainline vs. shoulder mixes. These recommendations were assembled with the goal of providing MoDOT with a systematically-designed, performance-related BMD specification that could be rolled out in stages to eventually capture the full complement of MoDOT mixtures encountered in new and rehabilitative asphalt construction, including Superpave mixes, stone-mastic mixtures (SMAs) for high traffic surface mix applications, and economical shoulder and non-surface course asphalt mixtures. A follow-up study is recommended to develop specific performance test thresholds for SMA mixes.

5. **Technology transfer**

In addition to the project quarterly and final reports, PowerPoint style presentation materials were assembled in preparation for a webinar event. The webinar will allow the project findings to be transferred to agency and industry personnel in both live and recorded formats. Once the final report is approved, the webinar can be hosted as part of the new Missouri Center for Transportation Innovation (MCTI) webinar series.

Chapter 2

2. LITERATURE REVIEW

The first part of this chapter presents a literature review centered on the origin of the Balanced Mix Design, or BMD, 'movement' in the US. The second section contains a review of the effect of testing temperature on cracking and rutting performance tests. Next, a review of climate databases and environmental effects models is conducted, along with a brief review of detailed climatic modeling study conducted by Ms. Loreto Urra-Contreras as part of her recent Master's thesis at the University of Missouri-Columbia (UMC). Finally, a review of a recently completed BMD assembled by the UMC for the Illinois Tollway and its implications on the current work is presented.

2.1. Balanced Mix Design

Twenty years has passed since the Superpave mix design system was developed (West et al., 2018). Three design levels were envisioned that included mixture performance testing as the highest level of input in the mixture design process. However, performance tests (level III) were never fully implemented and the design system thus remains heavily focused on volumetric design, aggregate screening, and performance-based binder requirements. In a sense, this is a step backwards from the Marshall and Hveem mix design methods, which required multiple tests to be conducted on the asphalt mixture. Over the years with the introduction of new material advances in asphalt mixtures such as polymer modification, RAP, RAS, GTR, warm-mix technology etc., many paving agencies have recognized the need to expand Superpave design to include mixture performance tests. Such an effort would address distresses and failures that commonly affect pavement infrastructure, particularly in the case of pavement cracking, which appears to be on the rise in the US. The rise in cracking-related distresses is presumably a result of increased use of RAP and RAS, increased traffic intensity, and changes in binder supply, refining techniques, additives such as PPA, along with increased laboratory compaction effort leading to lower optimum asphalt content (and 'drier' mixes).

In 2015, an Expert Group Task Force (ETG) on mixture design commissioned by the Federal Highway Administration (FHWA) initiated a Balanced Mix Design (BMD) Task Force. The BMD task force sought to identify two or more performance tests addressing pavement cracking and rutting to be added to the Superpave asphalt mixture design system in order to more reliably attain desired pavement performance targets. Three approaches to BMD, each progressively more reliant on mixture performance tests, were identified by the group. These are:

1. Volumetric Design with Performance Verification;
2. Performance-Modified Volumetric Design, and;
3. Performance Design.

A schematic summary of these three approaches is provided in Figure 2-1. Approach 1 is the most common method being considered by highway agencies at the present time. It starts with traditional volumetric design in order to determine the design asphalt content and to check volumetric parameters such as voids in the mineral aggregate (VMA), voids filled with asphalt (VFA), a measure of dust-to-binder content, etc. Next, a suite of mixture performance tests is performed. If the performance criteria are not met, mix proportions are altered in an iterative

process until the mixture satisfies both volumetric and performance test criteria. Approach 2 begins in a similar manner as Approach 1. However, results from the performance test are used to adjust either the binder or mixture components/proportions until performance criteria are met. In this approach, the final design focuses on meeting mixture performance test criteria, while the design itself may not be required to meet all traditional Superpave volumetric criteria. This departure from Superpave volumetrics restrictions allow the designers to build more cracking resistance into the mixtures by regressing (or lowering) the air void target at the design compaction level from the standard level of 4% in Superpave to 3.5% or perhaps 3.0%. The third approach involves adjusting the components and proportions of the binder, aggregates and other additives based on meeting mixture performance thresholds, first-and-foremost, with few if any requirements on mixture volumetrics during the design iterations. Practically speaking, it is likely that most agencies using Approach 3 will still set minimum requirements for asphalt binder and aggregate properties for the foreseeable future and that mixture volumetrics will continue to be considered as valuable for the purposes of quality control and quality assurance.

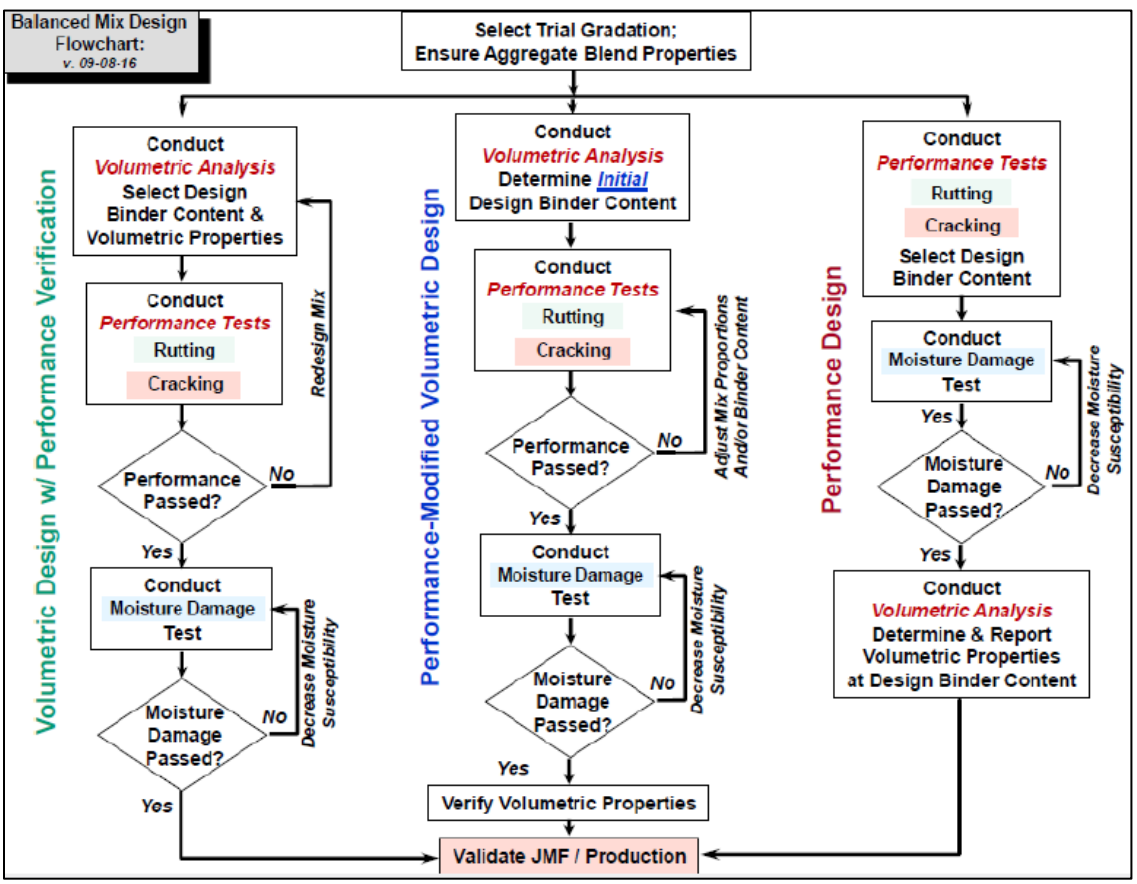


Figure 2-1. Balanced mix design approaches (From: FHWA BMD task Force 2016 West et al., 2018)

According to a survey conducted by the FHWA, a number of highway agencies have begun to either explore or adopt the BMD approach (West et al., 2018). Figure 2-2 illustrates recent trends in the adoption of BMD across the US. The most promising mixture performance tests identified in the literature for use in BMD are the HWTT, APA and the flow number test for rutting control, while the SCB, IDEAL, DC(T) and IDT creep and strength tests are being considered by a number of agencies to control cracking (West et al., 2018).

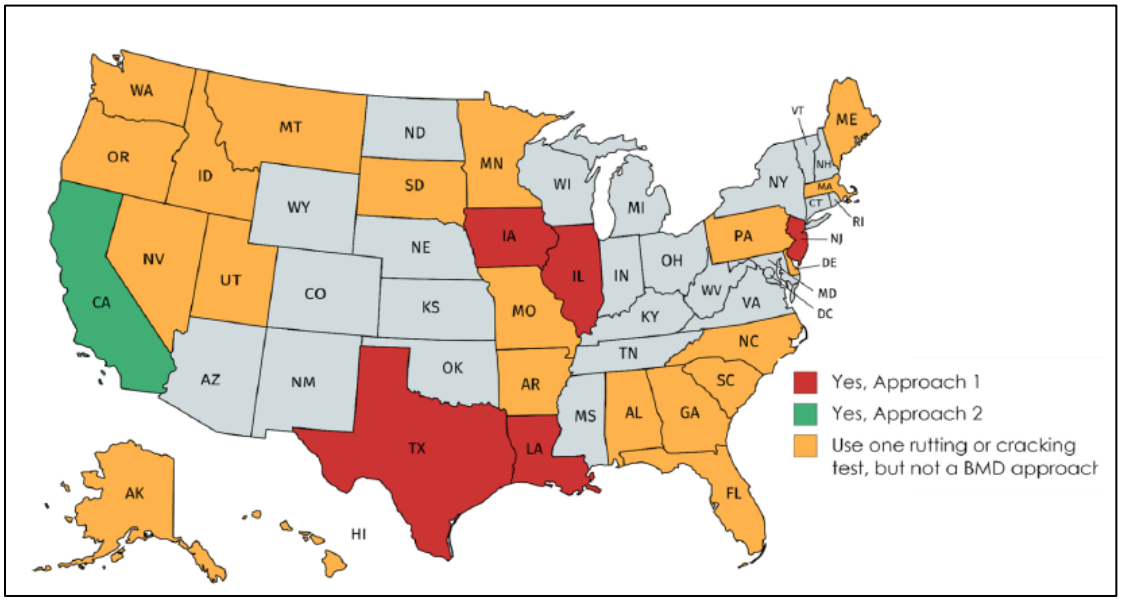


Figure 2-2. US map of current use of BMD approaches (From: West et al., 2018)

2.2. Effect of Testing Temperature on Performance

This section explores the effect of test temperature on low, intermediate, and high temperature performance tests as described in the literature. The Disk-Shaped Compact Tension DC(T) fracture test was developed to characterize thermal cracking resistance of asphalt mixtures at low temperatures. A suitable geometry was developed using ASTM E399, the standard for determining the stress intensity factor for metals under tension using various specimen geometries, as a starting point. The DC(T) was adapted for asphalt concrete, due to its ability to produce highly repeatable results and based on its applicability for testing field cores (Wagoner et al., 2005a). According to the most recent standard (ASTM, 2013), the test is recommended to be performed at a temperature 10ºC above the low temperature PG (Performance Grade) of the virgin binder for a virgin mix (no recycling). For mixes containing recycled content (RAP, RAS), sometimes the plan binder grade for the mix differs from the actual PG binder used in the mix, as a softer base binder is often required to counterbalance the stiffening effect of the recycled asphalt from RAP or RAS. In this case, the DC(T) test temperature is set relative to the plan PG grade for the mix, which is linked to the low temperature environmental characteristics present in the geographical location of the project.

The DC(T) has been extensively used to determine low-temperature fracture properties and possesses the capability to examine the temperature at which the mix transitions from brittle

to ductile behavior as the test temperature is increased from a low to an intermediate range (Behnia et al., 2014; Dave et al., 2013). Further, it has been shown to have excellent correlation to transverse cracking observed on field sections (Buttlar et al., 2018).

Researchers have conducted DC(T) testing across a range of low and intermediate temperatures and have noted the effects on fracture energy in the literature over the past 15 years. Wagoner et al. (2005b) reported fracture energies at 0ºC, -12ºC and -20ºC, concluding that fracture energy increases as the temperature increases. Similar conclusions were stated by Li et al. (2008) for DC(T) testing at 0ºC, -12ºC, and -24ºC. The authors reported that the temperature had a significant effect on the fracture energy, which increased as the temperature increased (Li et al., 2008).

Another popular geometry used for characterizing cracking potential in asphalt mixtures is the semi-circular bend specimen. The Semi-Circular Bend (SCB) test involves a half-disc with a notch on the bottom planar side and is loaded in a three-point bending configuration. Early work in the US focused on low temperature SCB testing, while testing at room temperature has been more prevalent in the past decade. Li et al. (2008) conducted a sensitivity analysis of temperature on SCB low temperature fracture energy and found similar results as the DC(T), i.e., fracture energy increased with warmer temperatures. Marasteanu et al. (2007, 2012) also reported similar results in DC(T) and SCB at low temperatures in a national-level study of low temperature cracking of asphalt mixtures (Marasteanu et al. 2007; Marasteanu et al., 2012).

Al-Qadi et al. (2015) conducted SCB fracture energy tests at temperatures ranging from -30ºC to 30ºC and found that the fracture energy of asphalt specimens increased until room temperature, or 25ºC was approached (Al-Qadi et al., 2015). In addition, Al-Qadi et al. (2015) used a digital image correlation to evaluate strains and damage at different temperatures reporting that at low temperature the strains are highly localized, leading to a brittle failure. As the temperature rises, the asphalt mixture specimen undergoes a transition, similar to the findings by Li et al. (2008) (Al-Qadi et al., 2015). Zegeye et al. (2012) showed similar results for the SCB and DC(T) while characterizing the low temperature fracture properties of Polyphosphoric Acid (PPA) modified asphalt specimens (Zegeye et al., 2012).

Hakimzadeh performed the three-point bending notched beam test at four different testing temperatures (-12ºC, 0ºC, 12ºC and 25ºC). The results showed that as testing temperature increases, the interface fracture energy (FE) increases until reaching a peak around 12ºC, followed by a decrease in FE. The same trend was seen at different loading rates (Hakimzadeh, 2015). The load-CMOD curve for a loading rate of 0.1 mm/min is illustrated in Figure 2-3. Hakimzadeh found similar results using The Single-Edge Notched Disk (SEND) fracture test for asphalt concrete. Hakimzadeh performed the test at -12ºC ,0ºC and 12ºC, concluding that increasing testing temperature resulted in increased FE and decreased peak load (Hakimzadeh, 2016). The taller, blue curve represents a strong-yet-brittle material state, while the flatter red curve on the other extreme represents a weak-yet-ductile state.

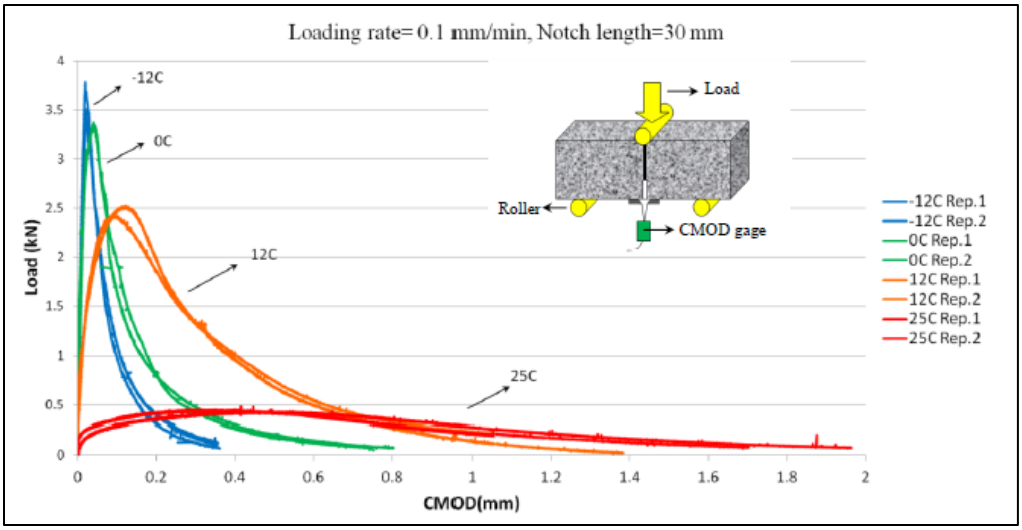


Figure 2-3. Load-CMOD curves at different testing temperature using three-point bending notched beam test (From: Hakimzadeh, 2015)

A high temperature test that has gained considerable popularity in evaluating both rutting and moisture susceptibility in asphalt mixtures is the Hamburg Wheel Tracking Test (HWTT). The HWTT was originally developed in Germany and introduced in the United States in the 1990s. The test is currently specified under AASHTO T-324. The HWTT procedure uses repetitive loading on samples submerged in water (instead of air-controlled temperature) and measures the rut depth induced in the asphalt mixture with increasing load cycles. Traditionally, the test has been performed at 50ºC, and many agencies have adopted this test temperature for all types of mixtures, such as Iowa DOT, Kansas DOT, Illinois DOT, Illinois Tollway, etc. However, other agencies select the HWTT test temperature according to the PG binder used, such us Colorado and Utah (Larrain, 2015). The specification for HWTT test in the city of Hamburg in Germany is more conservative than that used by most DOTs in the US. For instance, a maximum rut depth of 4 mm after 20,000 wheel passes was defined as a passing in the initial research carried out in Germany. During test adoption in the US, a number of test variations were pursued, leading to a wide variation in testing parameters and specification thresholds in the US (Table 2.1).

Table 2-1. HWTT test condition and limits used by some DOTs (From: Larrain, 2015)

Department of transportation	PG Grade	Number of wheel passes	Test Temperature (°C)	Maximum rut depth		Comments
				mm	in	
California	PG58-XX	10,000	50	12.7	0.5	WMA technology tested at 50 °C
	PG64-XX		55			
	PG70 and Higher		60			
Colorado	PG58-XX	10,000	46	4	0.16	Test and limits are used for research only
	PG64-XX		50			
	PG70-XX		55			
	PG76-XX		60			
Illinois	PG58-XX	5,000	50	12.5	0.5	N/A
	PG64-XX	7,500				
	PG70-XX	15,000				
	PG76-XX	20,000				
Iowa	PG58-XX	20,000	50	N/A	N/A	SIP evaluated only for moisture sensitivity purposes
	PG64-XX					
	PG70-XX					
Kansas	N/A	10,000	50	12.5	0.5	N/A
Louisiana	PG70-22 (Level 1)	20,000	50	10	0.39	N/A
	PG76-22 (Level 2)			6	0.24	
Montana	PG58-28	Plant mix: 10,000 Mix desing: 15,000	44	13	0.5	Specified temperature is 14 °C below the the avarage 7 day maximum pavement temperature design
	PG64-XX		50			
	PG70-28		56			
Oklahoma	PG64-XX	10,000	50	12.5	0.5	N/A
	PG70-XX	15,000				
	PG76-XX	20,000				
Texas	PG64-XX	10,000	50	12.5	0.5	N/A
	PG70-XX	15,000				
	PG76-XX	20,000				
Utah	PG58-XX	20,000	46	10	0.4	N/A
	PG64-XX		50			
	PG70-XX		54			

Level 1: Low traffic, Average Daily Traffic (ADT) < 7000 Level 2: High traffic, (ADT) > 7000

Perhaps unsurprisingly, several studies have shown a strong effect of the HWTT testing temperature on the induced rutting depths measured per cycle. Sel et al. (2012) statistically analyzed an 840-point database of mixture results generated from the HWTT. Out of the 840 points, 133 were performed at 50ºC and 707 at 40ºC. Different PG grades were used (PG 64-22, PG 70-22 and PG 76-22). The study showed that testing at 40ºC resulted in a higher number of passes than samples tested at 50ºC to reach a given rutting depth. Ninety percent of the of mixtures tested at 40ºC passed a criteria of 12.5 mm maximum rut depth, whereas 65% passed the same threshold when tested at 50ºC. The results indicate the importance of testing temperature when setting specification thresholds (Sel et al., 2012).

A similar study by Swiertz et al. (2017) in Wisconsin included a total of nine mixtures

using either PG 58-28 or PG 58-34 binder and three aggregate sources (Waukesha, Cisler and Wimmie-Cisler). Testing was carried out at 40ºC, 45ºC and 50ºC. Regardless of the mixture aggregate-binder combination, the mixtures exceeded 12.5 mm of rutting at 50ºC. However, rutting is generally not a problem with these mixtures in the field, and that a lower Hamburg test temperature may be appropriate for colder climates such as those present in northern climates such as Wisconsin. However, researchers concluded that a HWTT temperature of 40ºC might be too low, particularly with respect to the HWTT's ability to effectively evaluate stripping (Swiertz et al., 2017).

Walubita et al. (2016) stated that frequent rutting failures have been observed in the field in Texas for mixtures that passed the 12.5 mm criteria at 50ºC. Investigators hypothesized that climate change may be causing regions to experience elevated summer temperatures for longer periods of time, possibly in combination with higher moisture, leading to very severe rutting/stripping conditions. The failure in controlling rutting and stripping in some cases was attributed to the lower testing temperature compared to the actual pavement temperature in the field. Experimental tests were carried out at temperatures of 50ºC, 60ºC and 70ºC. The results showed that rutting depth increased significantly in mixes tested at 70ºC (but passing 50ºC); therefore, it was recommended to perform the HWTT test at higher temperatures in Texas and similar warm climate, up to perhaps 60ºC (Walubita et al., 2016).

Javilla et al. (2017) studied the effect of stress level on HWTT. Mixture tests were performed at 30ºC, 40ºC, 50ºC, 60ºC and 70ºC using three sequences of three differing stress levels (0.5-0.7-0.9MPa; 0.9-0.7-0.5MPa, and; 0.7-0.9-0.5MPa). The results demonstrated that regardless of the applied stress sequence, the rut damage became exponentially more heavily accumulated at temperatures above 50ºC and temperatures higher than 60ºC. In particular, the stress level of 0.9MPa led to major rutting (Javilla et al., 2017).

Another test to evaluate rutting potential on asphalt mixtures is the Asphalt Pavement Analyzer (APA), shown in Figure 2-4. The APA evolved from the Georgia Loaded Wheel Tester (LWT) in the mid 1980's. It is currently being adopted by various DOT agencies including Alaska, Alabama, Arkansas, Georgia, Idaho, North Carolina, New Jersey and Oregon. The APA is conducted according to AASHTO T 340-10, and a test temperature is selected based on the high temperature binder grade. The test consists of a loaded wheel that applies repetitive load applications across the sample for 8000 cycles, using a 100lb (445N) load and a 100 psi (690 kPa) hose pressure. Test samples are in the form of a beam (100mm x 300 mm x 75 mm thick) or cylinder typically compacted in the Superpave Gyratory Compactor (SGC) with a diameter of 150 mm and a thickness of 75 mm. The test can be conducted under air-dry or immersed conditions. Wang et al. (2017) performed the test at temperatures of 40ºC, 50ºC and 60ºC under dry conditions. The results showed that the rut depth at 50ºC was 2-3 times larger when compared to rut depth at 40°C. Further, rut depth at 60°C was 1.5 times larger than rut depth observed at 50°C and 3-5 times larger than rut depth observed at 40°C. Wang et al. (2017) concluded that temperature appeared to significantly affect the rutting potential of asphalt mixtures (Wang et al., 2017).

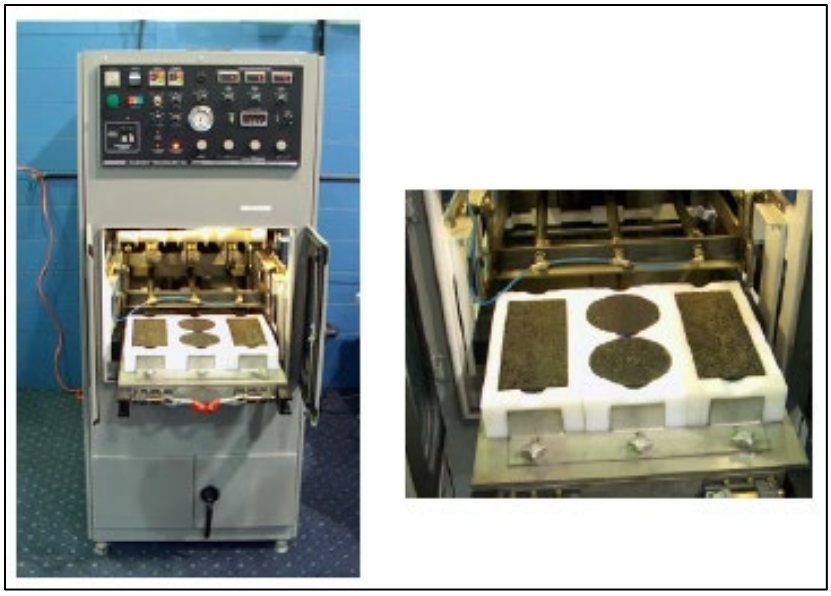


Figure 2-4. Asphalt Pavement Analyzer (APA)

2.2.1. Analysis of Climatic Data

A key aspect of the recent Master's thesis by Urra-Contreras (2019) was the development of pavement temperature maps, which can be used to select a tailored test temperature according to the geographic location where a given mixture would be placed in Missouri (Urra-Contreras, 2019). The climate database used in this research was extracted from the LTPPBind V3.1 software which emanates from two sources: the Canadian Daily Climate data and Surface land daily–Cooperative summary of the day, or TD-3200 database. The Canadian database contains daily temperature, precipitation and snow cover data for 7,627 locations throughout Canada. Information is available up to 1996. The second source corresponds to the climate data collected by the National Climatic Data Center (NCDC) and made publicly available online.

The NCDC database is a compilation of daily observations initially obtained from state universities, state cooperatives, and the US National Weather Service. The information corresponds to 23,000 stations across the US for various years. The period of record and number of stations varies among the states. Most states began collecting data during 1948, but some began in 1946. Prior to 1948, most of these data were collected through cooperative agreements with state universities and other state organizations. Many years of records were digitized with some records dating back as far as the 1850s.

The LTPPBind V3.1 (Figure 2-5) software can be downloaded from the following website. (https://infopave.fhwa.dot.gov/Page/Index/LTPP_BIND). How to extract the climate data from these databased and how to process it to calculate pavement temperatures is explained in Chapter 3 of the thesis by Urra-Contreras (Urra-Contreras, 2019).

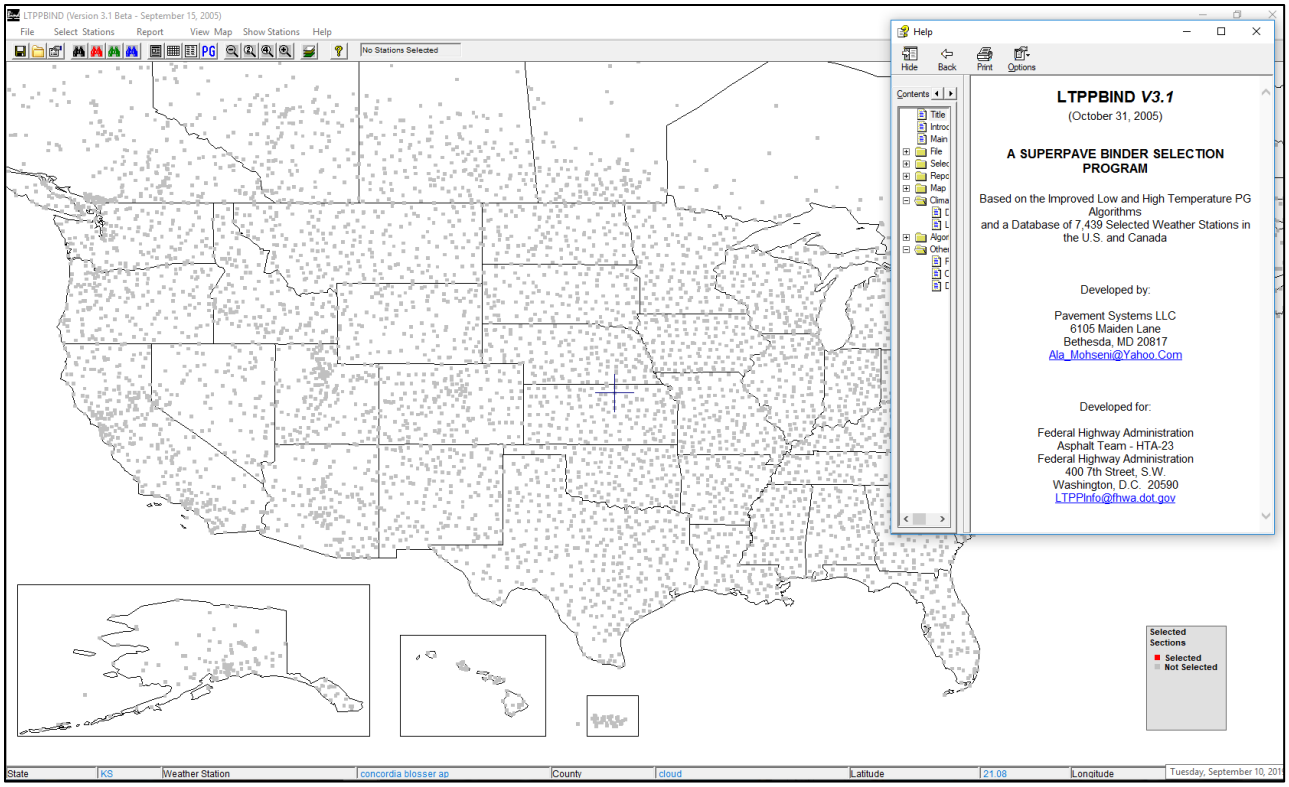


Figure 2-5. LTTPBINDV3.1

2.2.2. Historical Cold Event in January 2019

An interesting event related to pavement temperature that occurred during the time the research reported herein was conducted involved a severe cold event that affected the midwestern US in January of 2019. The event was caused by a polar vortex emanating from the Arctic and it resulted in the coldest temperatures seen in the Midwest U.S in over 20 years. The state of Missouri was also affected by this and some of the northern locations reported record low temperatures.

Asphalt pavements are significantly affected by temperature changes. Under low temperatures, asphalt mixtures could become brittle and crack as a consequence of thermal stresses exceeding the local tensile strength of the mixture. Several pavement temperature models relate air temperature to pavement temperature, such as the Strategic Highway Research Program (SHRP) low-temperature model and the LTPP low temperature model, while other models directly assume the surface temperature to be equal to the air temperature (Mohseni et al., 1998; Watson et al., 2001). Therefore, in order to visualize how this event affected pavements, using data from the National Weather Service Forecast, air temperatures for the coldest day in January 2019 were obtained from 92 weather stations in Missouri. Figure 2-6 shows a contour map created with the air temperatures observed for different locations in Missouri. Air temperatures dipped to -29ºC in areas near Princeton and Kirksville in northern Missouri.

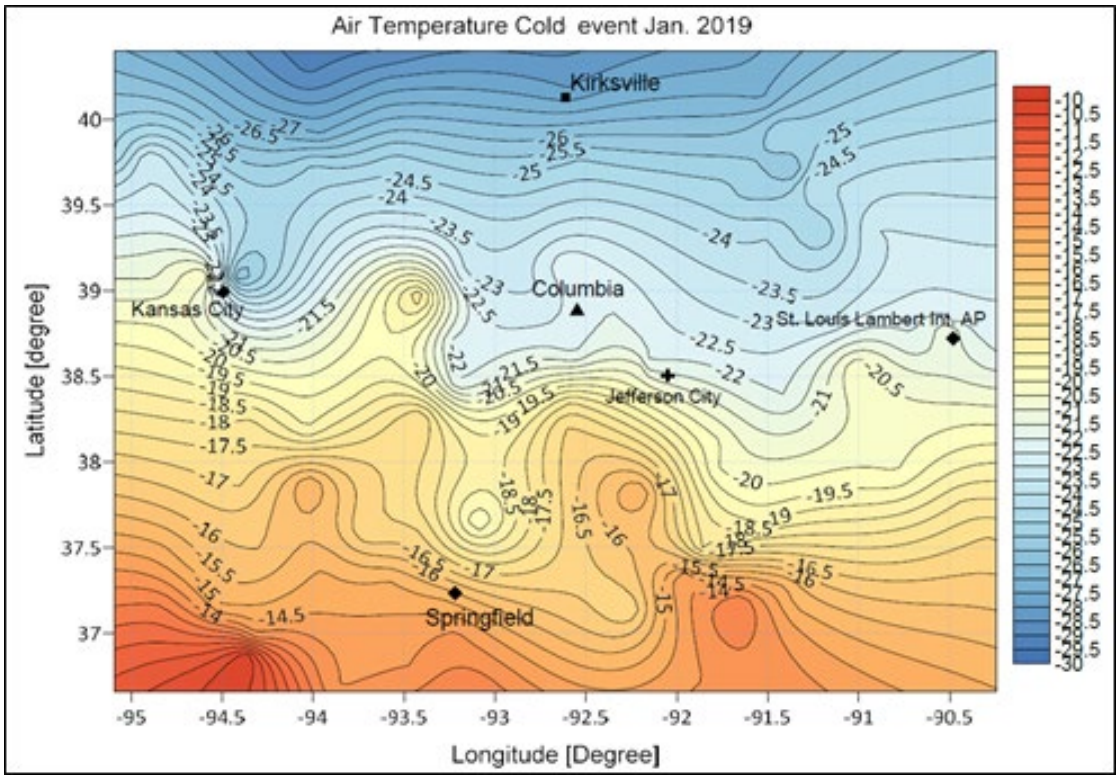


Figure 2-6. Air temperature map for the cold event January 2019 in Missouri

A similar analysis was performed for the area of Chicago IL, with air temperatures from 55 stations. The air temperature reached a low of -37ºC northwest of Chicago. The air temperatures are shown in Figure 2-7.

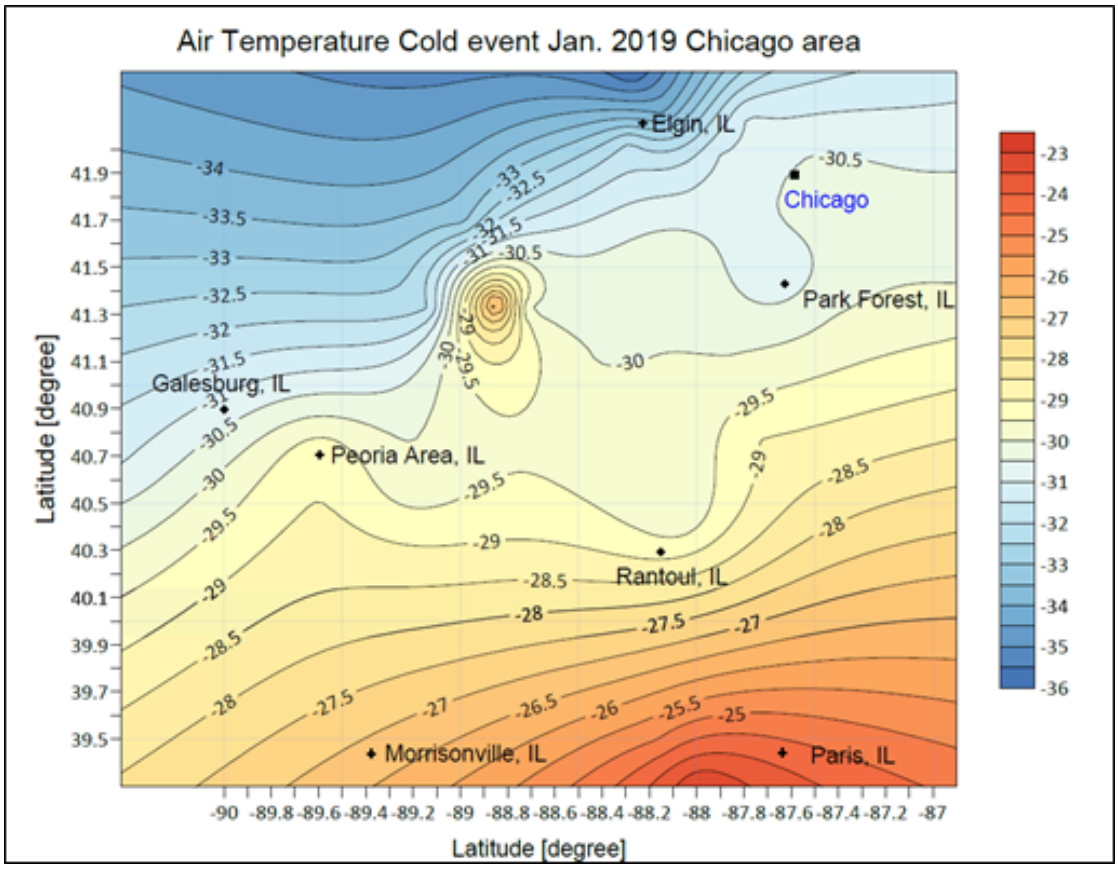


Figure 2-7. Air temperature map for the cold event January 2019 Chicago area

Figure 2-8 presents a 'dot map' showing the required 98% low temperature reliability PG grade for regions spanning across Missouri. Figure 2-9 presents a similar map for the required 98% high temperature reliability PG grade for regions spanning across Missouri.

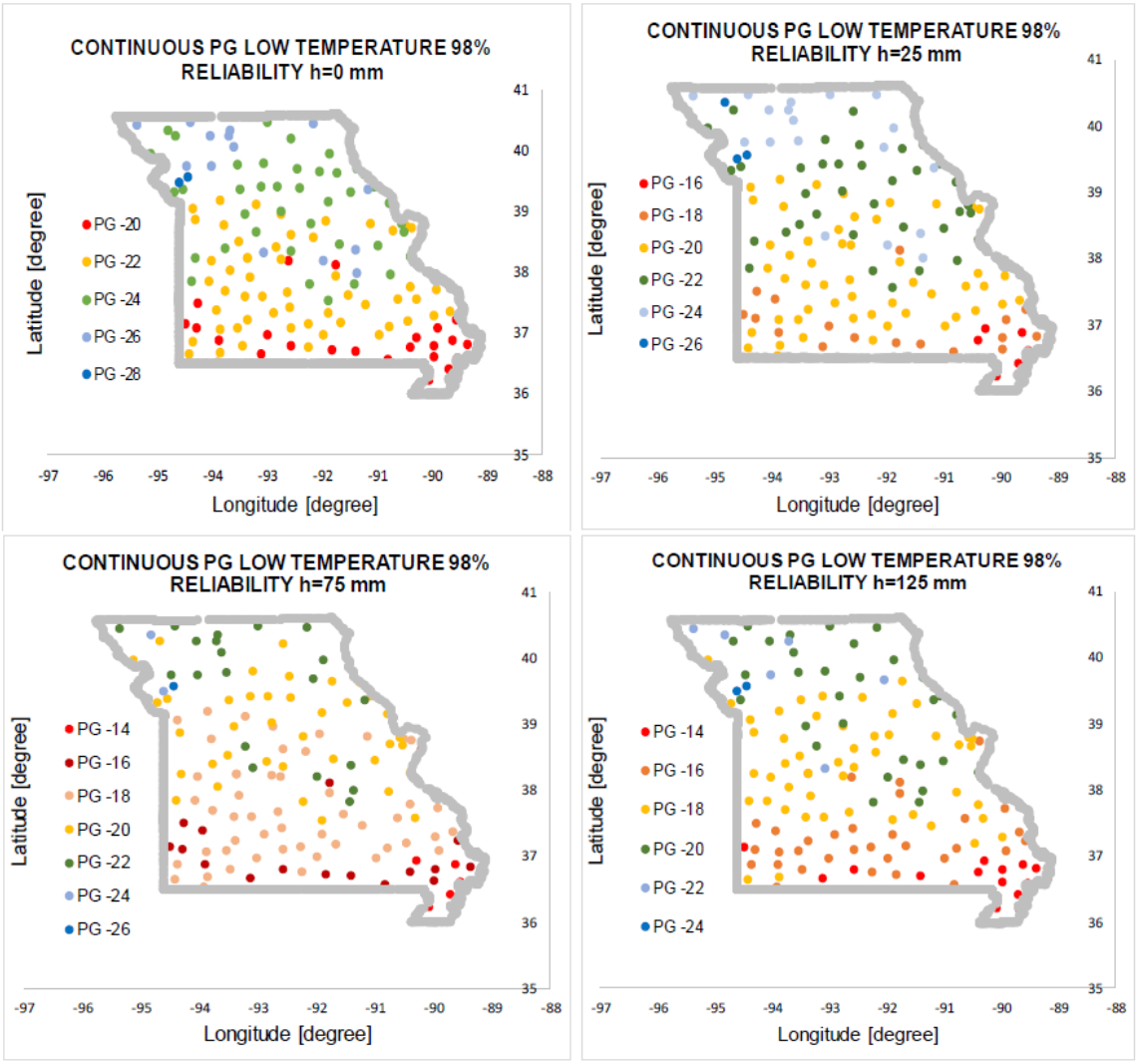


Figure 2-8. Example of 'dot maps' with a continuous low temperature PG at surface and depth of 25mm, 75mm and 125mm with 98% reliability for Missouri

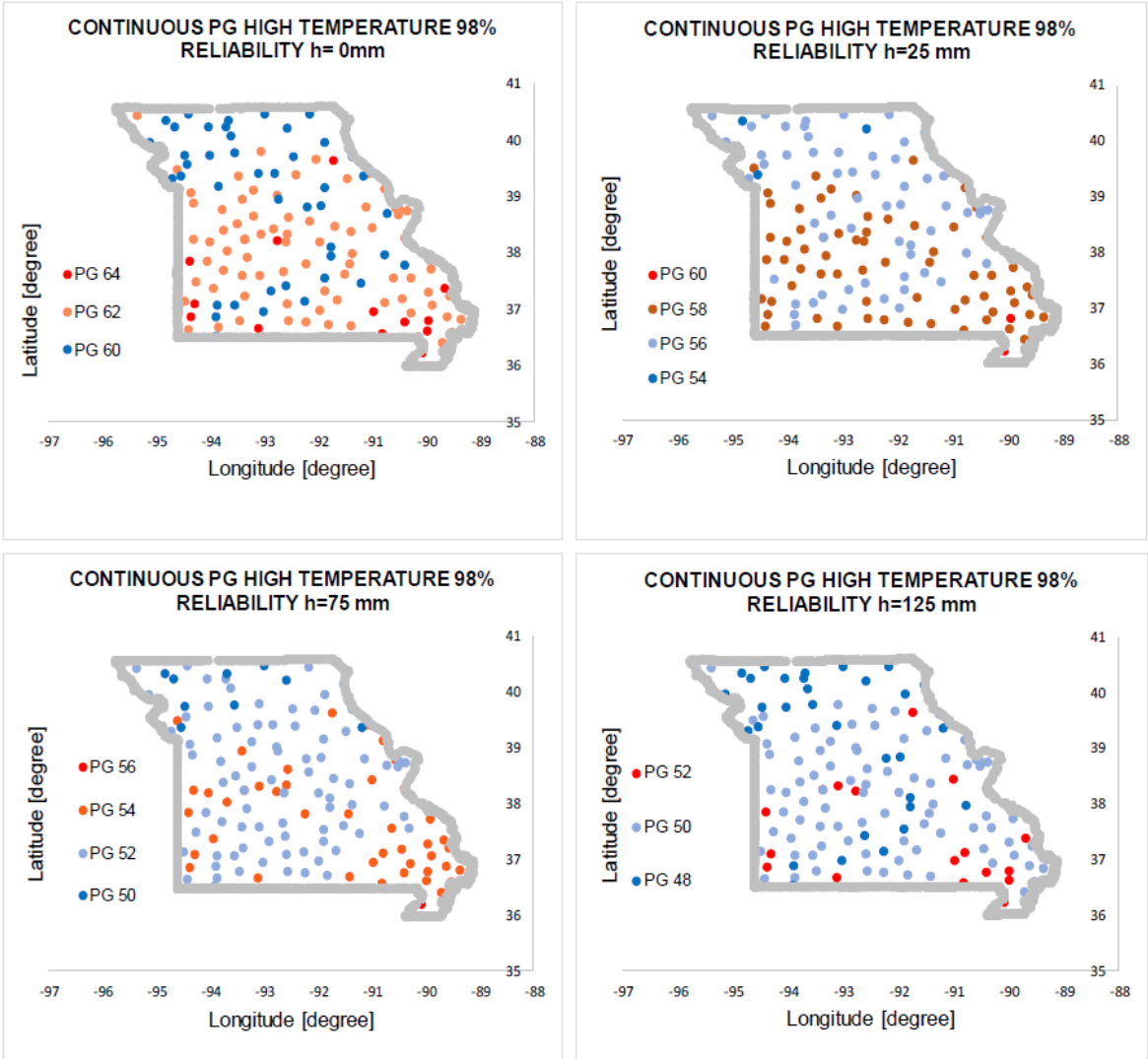


Figure 2-9. Dot maps for a continuous high temperature PG at surface, depth of 25mm, 75mm and 125mm with 98% reliability for Missouri

Although tailored asphalt mixture BMD specifications for various locations across Missouri is beyond the scope of this study, the results obtained (as reported by Urra-Contreras (2019)), led to the following interesting findings that can be used to fine-tune specifications for various paving applications,

- Low pavement surface temperature can vary by as much as 8°C across Missouri, with a low temperature PG grade of PGXX-28 suggested for the North-West corner of the state.
- Low pavement temperature design requirements can be relaxed for non-surface pavement lifts, with up to 6°C difference reported for lifts placed at pavement depths of 125mm (about 5 inches).
- High pavement surface temperature can vary by as much as 4°C across Missouri, with a high temperature PG grade of PG64-XX suggested for the St. Louis area

extending down into the boot heel for low traffic, which would then lead to even higher PG binder requirements considering traffic intensity and grade bumping.

- High pavement temperature design requirements can be relaxed for non-surface pavement lifts, with up to 12°C difference reported for lifts placed at pavement depths of 125mm (about 5 inches). Or, if temperature-wheel pass relationships can be developed for Missouri, then wheel pass requirements can be relaxed for lower paving lifts in a given geographical location.

2.3. Recent BMD Specification Developed for the Illinois Tollway

Researchers at UMC recently developed a BMD specification for the Illinois Tollway (Buttlar et al., 2020). Some of the salient features and findings of this research project and comprehensive report include:

- Three cracking tests were evaluated: the DC(T), I-FIT, and IDEAL CT. The DC(T) was selected as the cracking test to be used along with the Hamburg rutting test in the Tollway's BMD specification. This was due to a number of factors including the familiarity of local researchers and contractors with the DC(T) test, the higher repeatability of the DC(T) test, and the ability of DC(T) test to correctly distinguish between mix types and mix ingredients, and its excellent correlation to field performance.
- The Hamburg stripping inflection point (SIP) parameter was used as a first-order control on asphalt mixture stripping. For SMA mixtures, mixtures with less than 4 mm of rutting at the prescribed number of wheel passes are not subjected to the SIP criteria (and are declared as non-stripping mixes). For mixtures failing to meet SIP requirements, the traditional tensile strength ratio test (AASHTPO T-283) is allowed to be used as a second referee test.
- Cracking and rutting test criteria were linked to desired field performance levels, based on the testing and evaluation of good and poor performing field sections. A reliability-based design system was also developed, setting higher reliability levels for more critically important mixture categories (such as surface SMAs in high traffic areas), and lower reliability levels for lower paving courses ('binder' course mixes) and shoulder mixes (Figure 2-10). The variabilities associated with the testing of both lab and field cores was included in the process of setting desired testing thresholds.

The resulting Illinois Tollway cracking, rutting and stripping test recommendations are provided in Table 2-2 and Table 2-3.

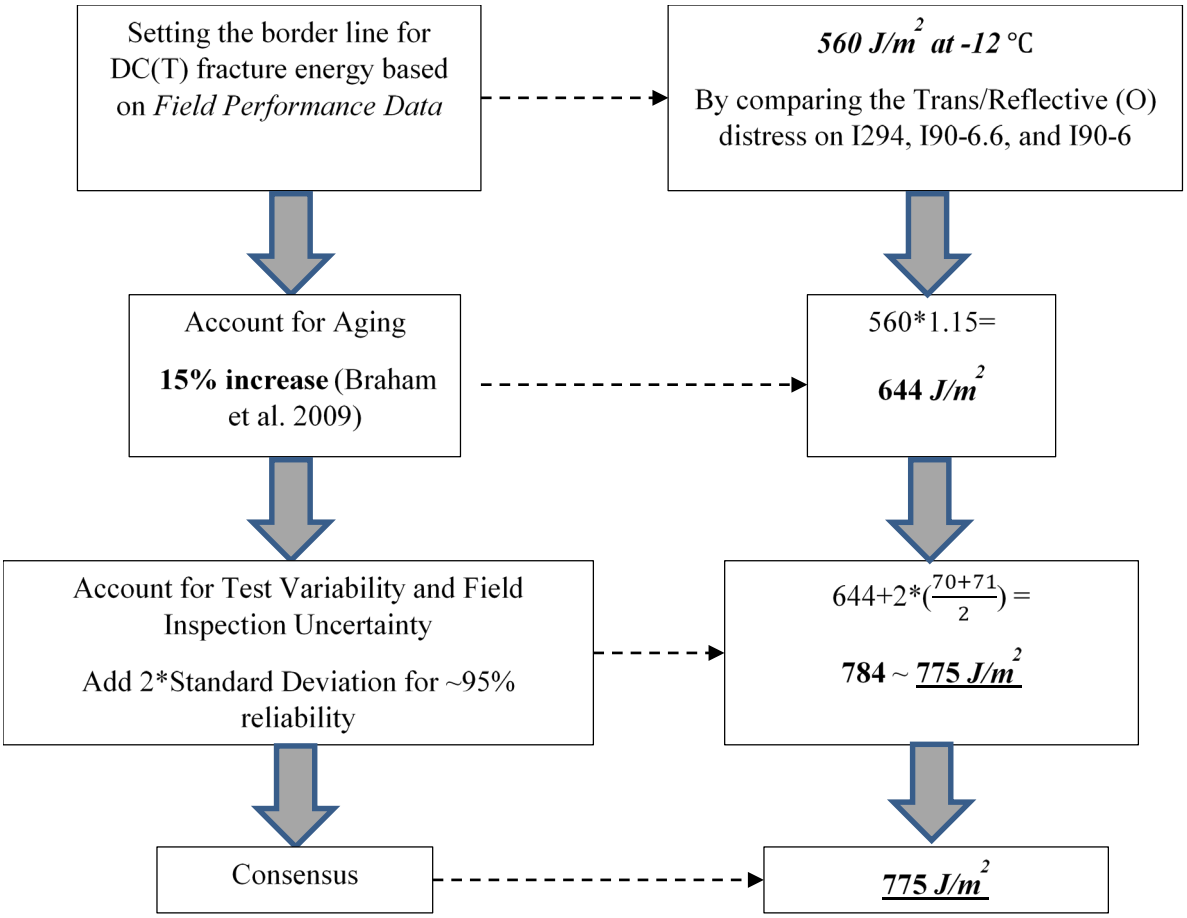


Figure 2-10. Example of Process Used in Establishing BMD Cracking Performance Thresholds for the Illinois Tollway (Buttlar et al., 2020)

Table 2-2. Example BMD cracking test thresholds for various mix types for the Illinois Tollway (Buttlar et al., 2020)

Mix. Type	Category	Existing	Recommended
SMA	**Friction Surface**	750 J/m^2	775 J/m^2
	Surface	700 J/m^2	700 J/m^2
	Binder	650 J/m^2	650 J/m^2
	Unmodified	500 J/m^2	500 J/m^2
Dense graded	**IL 4.75**	450 J/m^2	450 J/m^2
	Mainline Binder (N_{design}>50)	N/A	425 J/m^2
	Mainline Binder (N_{design}=50)	N/A	450 J/m^2
	Shoulder Surface ($N_{design}\leq$70)	N/A	450 J/m^2
	Shoulder Binders	N/A	425 J/m^2

Table 2-3. Example BMD rutting test thresholds for various mix types for the Illinois Tollway (Buttlar et al., 2020)

Mix. Type	Category	Existing		Recommended	
		No. of Passes	Max. Rut Depth	**No. of Passes**	**Max. Rut Depth**
SMA	**Friction Surface**	20,000	6.0 mm	20,000	6.0 mm
	Surface	20,000	6.0 mm	20,000	6.0 mm
	Binder	N/A	N/A	20,000	6.0 mm
	Unmodified	15,000	9.0 mm	15,000	12.5 mm[1]
Dense graded	**IL 4.75**	15,000	9.0 mm	15,000	12.5 mm
	Mainline Binder (N_{design}>50)	N/A	N/A	15,000	12.5 mm
	Mainline Binder (N_{design}=50)	N/A	N/A	10,000	12.5 mm
	Shoulder Surface ($N_{design}\leq$70)	15,000	12.5 mm	10,000	12.5 mm
	Shoulder Binders	N/A	N/A	7,500	12.5 mm

Chapter 3

3. MATERIALS AND PERFORMANCE TESTS

3.1. Material Collection

In this portion of the study, eleven mixtures were tested at various temperatures using the DC(T) and HWTT tests. Additional cracking test data using the IFIT and IDEAL CT tests are included later in this report. Most of mixtures were sampled in Missouri with the exception of two lab mixtures from Oklahoma (referred to as OK_70P and OK_70P5). Some of the mixes were collected during the MoDOT project "Performance Characteristics of Modern Recycled Asphalt Mixes in Missouri, including those containing GTR, RAP, and RAS" (Buttlar et al. 2019). The mixtures were Superpave dense-graded for a medium traffic volume and were used to pave sections of the highways US63, MO13, and US54 in the year 2016. Mixtures were stored in five-gallon steel pails and then brought to the lab for compaction and testing.

Another type of mixture included in this study is a fiber-modified mixture sampled at a production plant in the St. Louis Missouri area in March 2019. The plant mixture was a BP-2 Marshall design mix with 35 blows and a base binder PG58-28. In addition, aggregates and binder were collected from the plant to design a control mix and a fiber-reinforced mixture at the MAPIL Lab (Figure 3-1). In addition, GTR-modified mixtures were included in this study. A MoDOT Superpave lab mixture with a 10% dry GTR and a mixture without GTR (control mix) was developed at the MAPIL Lab. Aggregates and binder were collected from a portable plant near Osage Beach during the 2016 paving season. Another GTR modified-mixture was provided by the Oklahoma DOT in July 2019. The mixture was designed using Superpave and was modified with 5% dry GTR. A control mix with the same gradation and the same base binder (PG 70-28) was also provided by the Oklahoma DOT.

Figure 3-1. Asphalt Mixing Plant in Missouri

3.2. Sample Preparation and Compaction

3.2.1. Mixture Details

Error! Reference source not found. presents the mixtures tested in this report along with the section ID nomenclature established. The gradation of the mixtures tested is presented in Figure 3-2. The first group of plant mixtures studied were from Highways US63_1, MO13_1, US54_1 and US54_6 in Missouri. Some details regarding these sections follow:

- US63_1 is a section on the southbound lanes of US route 63 in Randolph County in Moberly, MO. This mixture has an NMAS of 1/2" and 35.2% asphalt binder replacement (ABR) from reclaimed asphalt pavement (RAP). The mixture asphalt content (AC) is 5.1%, comprised of virgin binder grade PG58-28.
- MO13_1 is a section in Henry County on the northbound lanes of Missouri 13, about 11 miles south of Clinton, MO. The NMAS is 3/8" and contains an ABR level of 16.6% from RAP, along with 5.7% AC of virgin binder grade PG64-22H (AASHTO M302).
- US54_1 is part of a federal research study SPS-10 (an FHWA Special Pavement Section) in the westbound lanes of US route 54 in Miller County near Eldon, MO. The mixture NMAS is ½", and has 33% ABR from RAP, along with 5.2% AC comprised of virgin binder grade PG58-28. The mix contains 3.5% rejuvenator by weight of the mix.
- US54_6 is a mainline mix placed alongside the SPS-10 test sections on US route 54 in Camden and Miller Counties, near Eldon, MO. This mixture has 30.7% ABR from RAS, an asphalt content of 5.1%, and a virgin binder grade of PG58-28.

Table 3-1. Mixture nomenclature used in this portion of study

	Mixture Details	**Mix nomenclature**
A Level 1 Missouri mixtures	MO 13 NB Superpave plant mix	MO13_1
	US 54 NB Superpave plant mix	US54_6
	US 54 SB Superpave plant mix	US54_1
	US 63 SB Superpave plant mix	US63_1
B Control and GTR-modified mixtures	MODOT Superpave lab mix with no dry GTR	TRO_L
	MODOT Superpave lab mix with 10% dry-process GTR	TRO_L10R
	Oklahoma Superpave plant mix, control (no GTR)	OK_70P
	Oklahoma Superpave plant mix with 5% dry-process GTR	OK_70P5
C Control and fiber-reinforced mixtures	Plant mix with aramid fibers	FI_PM
	Control mix (no fibers)	FI_CM
	Lab mix with aramid fibers	FI_FM

The second group of mixes studied involved both a control mixture and rubber-modified mixture (Figure 3-3) from Missouri and Oklahoma. Some details regarding these mixes include:

- MODOT Superpave plant mix TRO_L10R is a 10% GTR-modified (by wt. of mix), dense-graded mixture, with 12.5 mm NMAS, 4.5% AC, and 25% RAP. The binder used was a PG 58-28. The label TRO_L designates the control mixture, which did not contain GTR but utilized the same binder, asphalt content and aggregate skeleton.
- Oklahoma Superpave plant mixture OK_70P5 is a 5% GTR-modified mixture (by wt. of mix), with 12.5mm NMAS, 4.7% AC, and PG 70-28 binder. Also studied was OK_70P, which is the control mix, with similar composition except the exclusion of GTR.

The last group of mixes represents a fiber-modified demonstration project mixture. Fibers were added at a rate of 4.2 oz per ton of mix. An illustration of the aramid fiber used is presented in Figure 3-4. Manufacturer details are not included herein, following guidelines agreed upon by the contractor and supplier for this early demonstration project.

- FI_PM is a low-volume (MoDOT BP-2) plant mix, designed as a 35-blow Marshall mix in Franklin County, MO, with natural sand, RAP and RAS. It has a virgin binder content of 4.4%, using PG58-28 binder. The plan binder grade was PG64-22 with a total design asphalt content of 5.4%. The mix ABR level was 27% (18% by RAP, 9% by RAS).
- FI_FM is a lab mixture utilizing aramid fibers. This mix has 4.4% AC, and a virgin binder grade of PG58-28. This mixture was accompanied by control mixture FI_CM, which has the same gradation and binder but does not contain fibers.

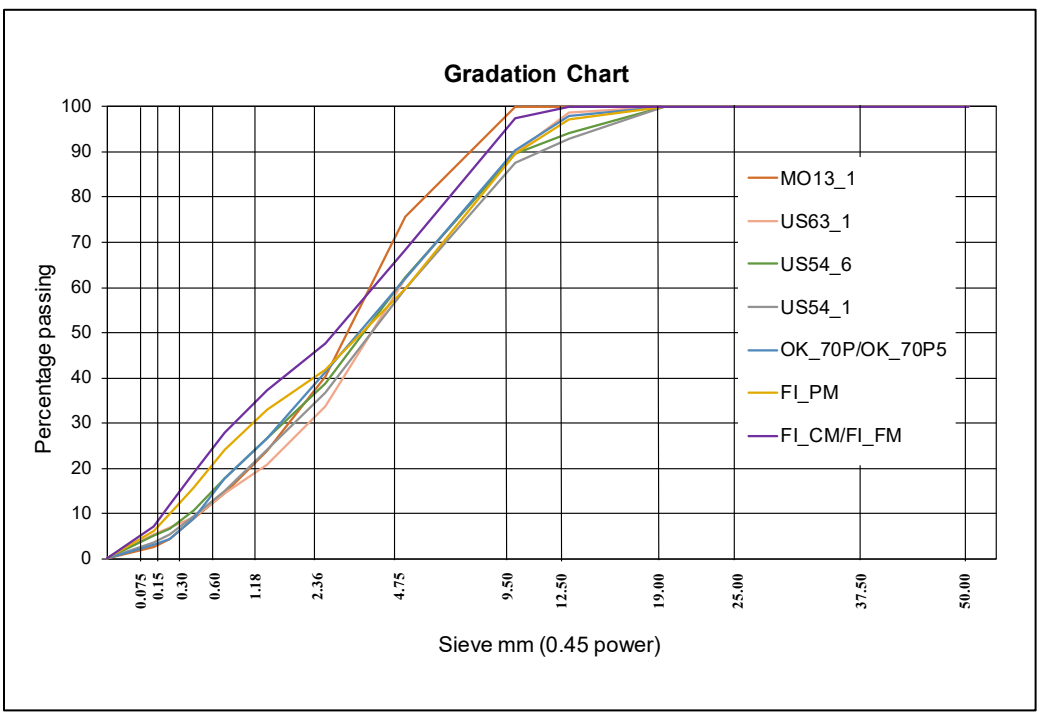


Figure 3-2. Gradation chart of the lab and plant mixtures

Figure 3-3. Ground tire rubber

Figure 3-4. Uncoated polymer (aramid) fibers

3.2.2. Mixing and Compaction

All samples were fabricated and tested at the MAPIL Lab at UMC. Plant mixtures were brought to the lab in 5-gallon steel pails and reheated until the mix was workable. Next, the material was reduced to a gyratory sample following the quartering method described in AASHTO R47 (AASHTO, 2008). Compaction temperatures were set according to the Job Mix Formula (JMF) supplied by the contractor.

Lab mixes were produced from the collected aggregates and binders following the contractor-supplied JMF. Prior to mixing, aggregates were dried overnight and then batched. The mixing process was carried out in a bucket-style lab mixer (Figure 3-5), and mixtures were then short-term aged in the oven for 2 hours before compaction, at the compaction temperature.

For rubber-modified mixtures, GTR was incorporated into the binder before mixing with aggregates (Figure 3-3). Following manufacture recommendations, binder and GTR were heated to 170°C then blended in a high-shear mixer at 3500 rpm for 30 minutes. The GTR-modified binder was then added to aggregates heated to 190°C, and bucket-mixed. For the fiber-reinforced mixes, prior to lab mixing, fiber strands were thoroughly separated (or 'fluffed') following manufacturer recommendations. After fluffing, the fibers were gradually added to the mixture during bucket mixing. The mixing operation for fiber mixes required about 4 minutes.

The plant and lab mixtures specimens were compacted in a Superpave Gyratory Compactor (SGC) and then fabricated into DC(T) and HWTT performance test specimens.

Figure 3-5. Bucket lab mixer (left), gyratory compactor (right)

3.2.3. Sample Size

In this research, DC(T) and HWTT specimens were fabricated from both plant- and lab-produced mixtures. To perform the HWTT test, a 62 mm height cylindrical sample with 150mm diameter was produced in a gyratory compactor according AASHTO T 324. For the DC(T), a gyratory specimen of 150 mm diameter and 140 mm height was produced and, according to ASTM D7313, subsequently cut into two 50 mm thick slices. Two loading holes of 25 mm diameter and a notch of approximately 62 mm in length were fabricated into DC(T) specimens. Figure 3-6 illustrates the sample fabrication process used for DC(T) specimens. A minimum of three replicates were used in DC(T) testing, while a minimum of four replicates were used for HWTT testing.

Figure 3-6. DC(T) fabrication process

3.3. Performance Tests

3.3.1. Disk-shaped Compact Tension Test DC(T)

The Disk-Shaped Compacted DC(T) test, specified in ASTM D7313, was developed to characterize the fracture behavior of asphalt concrete mixtures at low temperatures. DC(T) was originally developed by Wagoner et al.(2005a). The DC(T) test has been extensively used for determining low-temperature fracture properties for use in cracking specifications and for modeling cohesive crack behavior (Arnold et al., 2014; Rath et al., 2019a; Buttlar et al., 2019a). Typically, the test temperature for DC(T) fracture testing is 10°C warmer than the low temperature plan PG grade for the mixture. In this portion of the study, the test was performed over a range of test temperatures between -3°C and -24°C.

The DC(T) test procedure includes conditioning of the fabricated specimen at the selected test temperature in a temperature-controlled chamber for a minimum of two hours. After the conditioning period, the specimens are suspended on loading pins in the DC(T) machine. The test applies a tensile force on the loading holes of the specimens at a constant Crack Mouth Opening Displacement (CMOD) rate, which is controlled by a CMOD clip-on gage mounted at the crack mouth. The CMOD rate specified is 0.017 mm/s (1 mm/min). The test is completed when a crack has propagated and the post-peak load level reduces to 0.1 kN. Figure 3-7 illustrates a DC(T) specimen, and

Figure 3-8 shows a DC(T) test setup. The fracture energy (G_f) in this test can be obtained by computing the area under the load-CMOD curve and dividing it by the area of the fracture face.

A typical curve from the DC(T) test is shown in Figure 3-8. Calculation of the fracture energy is quite straightforward, as follows:

$$G_f = \frac{W_f}{L \times B} \tag{6}$$

Where:

G_f = fracture energy;
W_f = work of fracture;
B = specimen thickness, and;
L = ligament length.

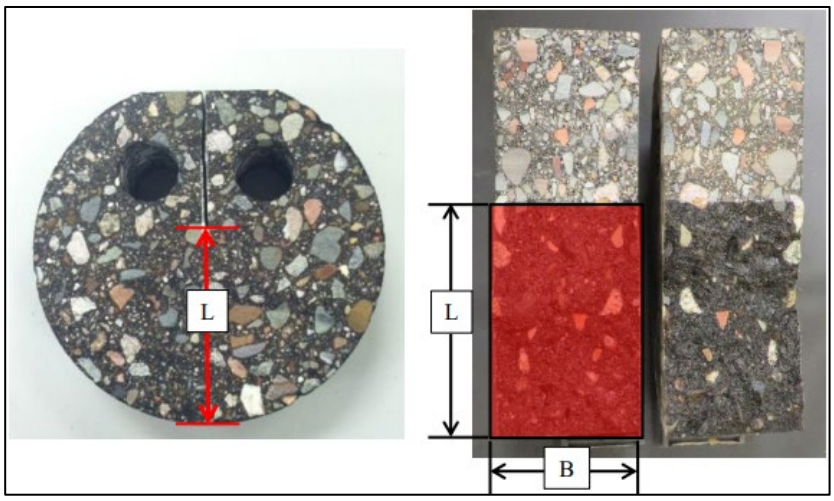


Figure 3-7. DC(T) specimen

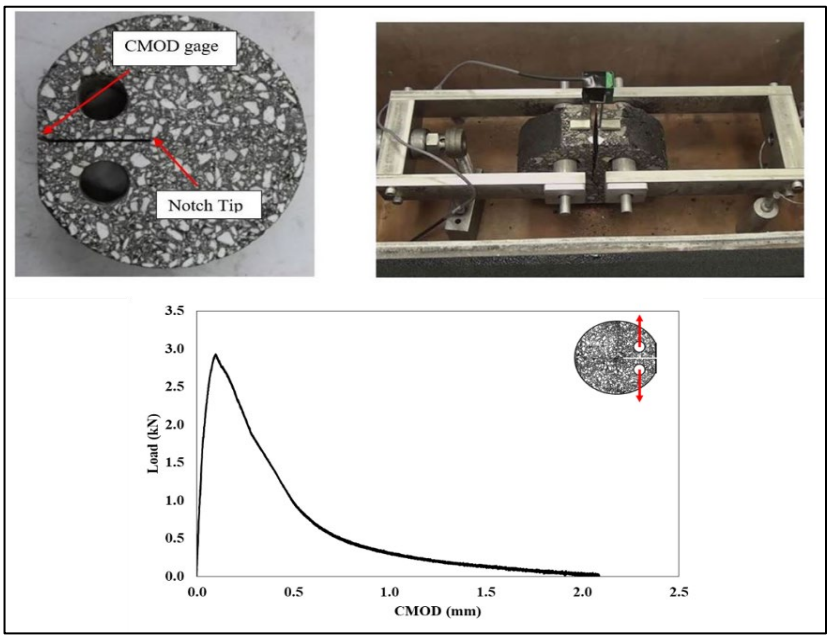


Figure 3-8. DC(T) specimen, loading apparatus, and typical load-CMOD curve (after Rath et al., 2019b)

Recommended values for fracture energy according to a National Low-Temperature Cracking Pooled Fund Study are presented in Table 3-2, where testing was performed at a temperature equal to 10 degrees Celsius warmer than the design (or plan) Superpave Performance-Graded Binder Low Temperature grade, or PGLT.

Table 3-2. Early low-temperature cracking specification based on DC(T) fracture energy at test temperature of PGLT +10°C (Marasteanu et al. 2012)

Test	Traffic Level	Minimum Fracture Energy
DC(T)	Low Medium High	400 J/m^2 460 J/m^2 690 J/m^2

3.3.2. Hamburg Wheel Tracking Test (HWTT)

The Hamburg Wheel Tracking test was first developed in Hamburg, Germany and brought to the United States in the early 1990's. Since then it has been adopted by agencies and widely used as a standard laboratory test to evaluate moisture susceptibility and rutting resistance in asphalt mixtures. The test is conducted in accordance to AASHTO T324 and consists of a loaded steel wheel (71.7 Kg) passing over the asphalt mixture samples at a rate of 52 passes per minute. During the test, the specimens are submerged in water, typically at 50°C. In this report, the temperatures used vary between 40°C and 64°C.

Once the HWTT test has been completed (typically 20,000 wheel passes or when 20 mm of rutting is reached), a curve showing the accumulated rut depth versus the number of passes is constructed. Figure 3-9 shows samples after a test. A Cooper Hamburg device was used in this study.

Figure 3-9. Deformed Hamburg specimen after wheel track testing

A typical curve from the Hamburg test is shown in Figure 3-100. First, a short consolidation stage occurs, where the steel wheel densifies the mixture leading to an air void decrease in the wheel path area. This phase typically occurs within the first 1,000 wheel passes of the test. After this, a creep phase starts to develop, where the deformation is predominantly plastic, and the mixture may begin to exhibit the onset of moisture damage if prone to such behavior. Following this stage, a tertiary deformation phase will sometimes ensue. This occurs in mixtures that are either prone to severe plastic deformation (shear flow) or stripping, or both. The degree of moisture susceptibility can be evaluated during this phase by calculating the so-called Stripping Inflection Point (SIP). The SIP graphically represents the intersection between the fitted line of a 'stripping' curve and a 'creep' curve.

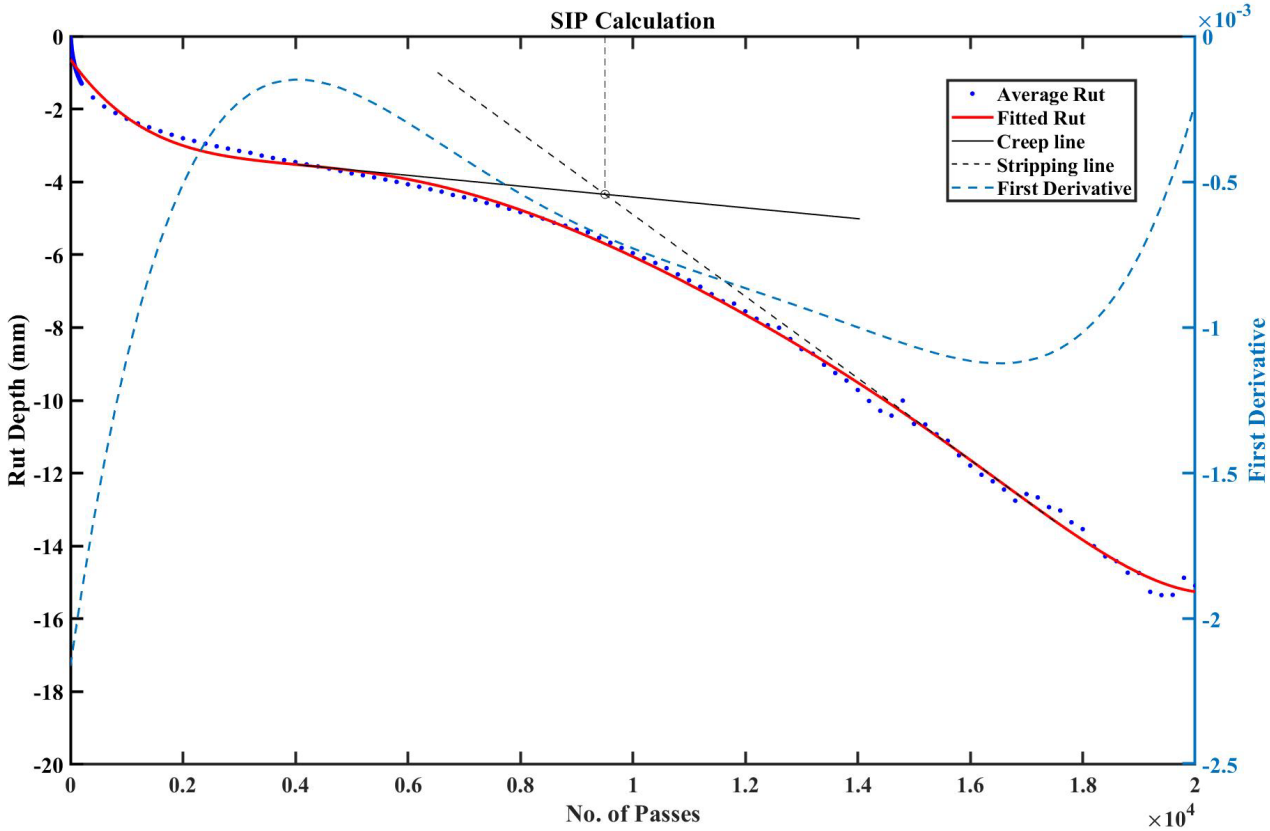


Figure 3-10. Typical curve from Hamburg Wheel Track Test

3.3.3. Illinois Flexibility Index Test

The flexibility index (FI) is an empirical index parameter that is computed as the total fracture energy divided by the absolute value of the slope of the post-peak softening curve. The FI has been proposed as a means to identify brittle mixtures that are prone to premature cracking, and was specifically developed to be sensitive to recycled material content (AASHTO TP124-16). The FI parameter is calculated as follows.

$$FI = \frac{G_f}{|m|}(0.01) \qquad (7)$$

where G_f is computed in a similar manner as to the DC(T) test, and m represents the slope of the post-peak softening curve. There are numerous ways to estimate the slope of a curve resulting from a material test, which poses an inherent challenge from the perspective of test standardization in the development of tests such as the I-FIT. At present, to address this source of variability, the slope parameter is typically determined using a sophisticated software program available from the Illinois Center for Transportation (visit https://ict.illinois.edu/2016/07/01/i-fit-software-now-available-on-ict-website/). To fabricate samples, a notch is cut along the axis of symmetry of a semi-circular bend specimen to a depth of 15±1 mm. Test specimens are then conditioned in the environmental chamber at 25ºC for 2 hrs. ± 10 min. After a contact load of 0.1 kN is reached, the test is carried out at a rate of 50 mm/min load line displacement (LLD). The

test is considered to be complete when the post-peak load drops below 0.1 kN, similar to the DC(T) test termination limit. A sampling rate of 40 points per second was used to collect the data during the test. An executable file named “SCB TestQuip LLC. V2.0.0rc4” was then used to analyze the collected load-displacement data and calculate the FI parameter.

Chapter 4

4. TESTING RESULTS AT MULTIPLE TEMPERATURES

4.1. Testing Temperature

The aforementioned mixtures (Chapter 3) were tested at multiple temperatures, for future use in tailoring BMD across the state of Missouri, and also for the purpose of creating more realistic (relaxed) BMD requirements for lower pavement lifts where less extreme pavement temperatures exist. Typically, HWTT tests are performed at 50ºC. A range between 40ºC and 64ºC was selected with an increment of approximately of 6°C. The DC(T) was performed at temperatures between -24ºC and -3ºC, depending on material availability. For the HWTT, the test was carried out in two wheel passes for each temperature, and for the DC(T) three replicates were considered, except in a few cases two replicates were tested when material availability was limited. Table 4-1 presents the test temperatures used in this report.

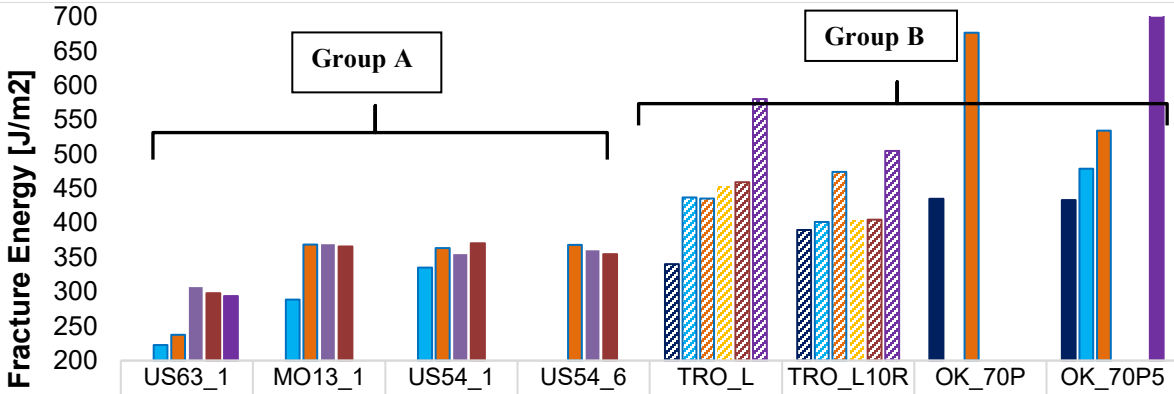
Table 4-1. Experimental program for the HWTT and DC(T) temperature study

Group	Mixtures	Hamburg wheel test temperature ºC	DC(T) fracture test temperature ºC
A Level 1 Missouri mixtures	US63_1	40,46,52,58,64	-3,-6,-9,-12,-18
	MO13_1	46,52,58	-6,-9,-12,-18
	US54_1	46,52,58	-6,-9,-12,-18
	US54_6	52,58	-6,-9,-12
B Control and rubber modified mixtures	TRO_L	40,46,50,58	-3,-6,-9,-12,-18, -24
	TRO_L10R	40,46,50,58,64	-3,-6,-9,-12,-18, -24
	OK_70P	46,50,58,64	-12,-24
	OK_70P5	40,46,50,58,64	-12,-18, -24
C Control and fiber reinforced mixtures	FI_PM	40,46,50,58	-3,-6,-9,-12,-18
	FI_CM	40,50,55	-12,-18,-24
	FI_FM	50, 58	-12, -18

4.2. Results and Discussions

4.2.1. DC(T) Fracture Energy

The results of the fracture energy for each mixture from Groups A & B are presented in Figure 4-1. The coefficient of variation (COV) of the results are presented in Appendix B of the Master's thesis by Urra-Contreras (2019). Load-CMOD curves for each mixture are presented in Figure 4-2 through 4-4.



	US63_1	MO13_1	US54_1	US54_6	TRO_L	TRO_L10R	OK_70P	OK_70P5
-24ºC					340.3	390.0	435.4	433.3
-18ºC	222.6	288.6	335.3		437.3	401.7		479.3
-12ºC	237.3	368.6	363.8	368.5	435.8	474.5	676.9	534.4
-9ºC	306.7	369.5	354.9	360.6	452.7	403.7		
-6ºC	297.8	365.8	370.5	354.9	459.5	405.0		
-3ºC	294.0				580.5	505.0		893.2

**Recommended low-temperature cracking specification for loose mixture according to Pooled Funded Study–Phase II (Marasteanu et al. 2012)*

Figure 4-1. Fracture energy from DC(T) testing

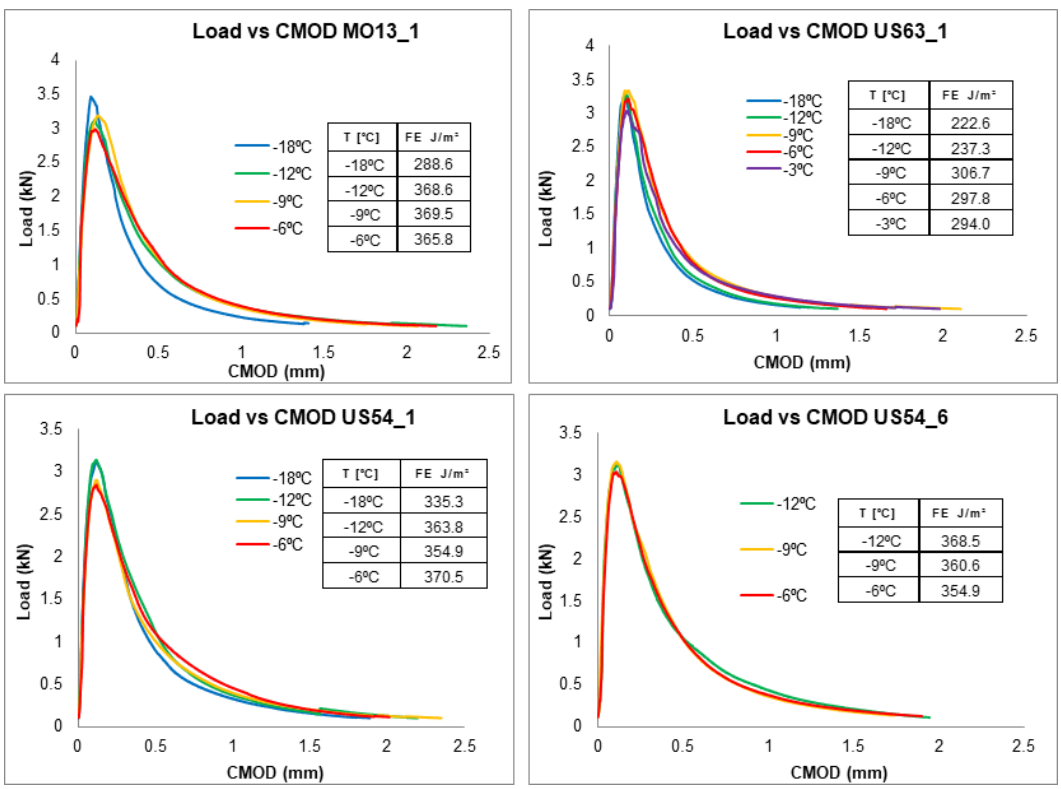


Figure 4-2. DC(T) Load-CMOD curves for Group A Missouri mixtures

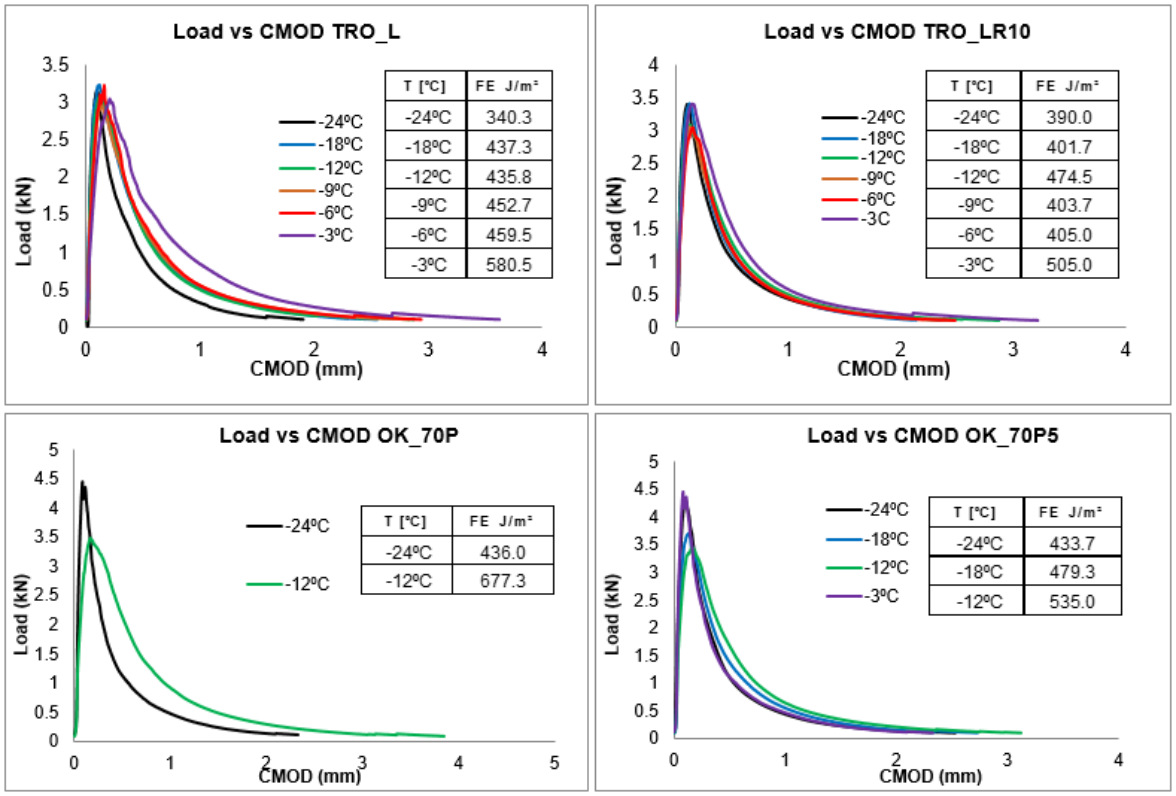


Figure 4-3. DC(T) Load-CMOD curves for rubber-modified mixtures (right) and control mixtures (left)

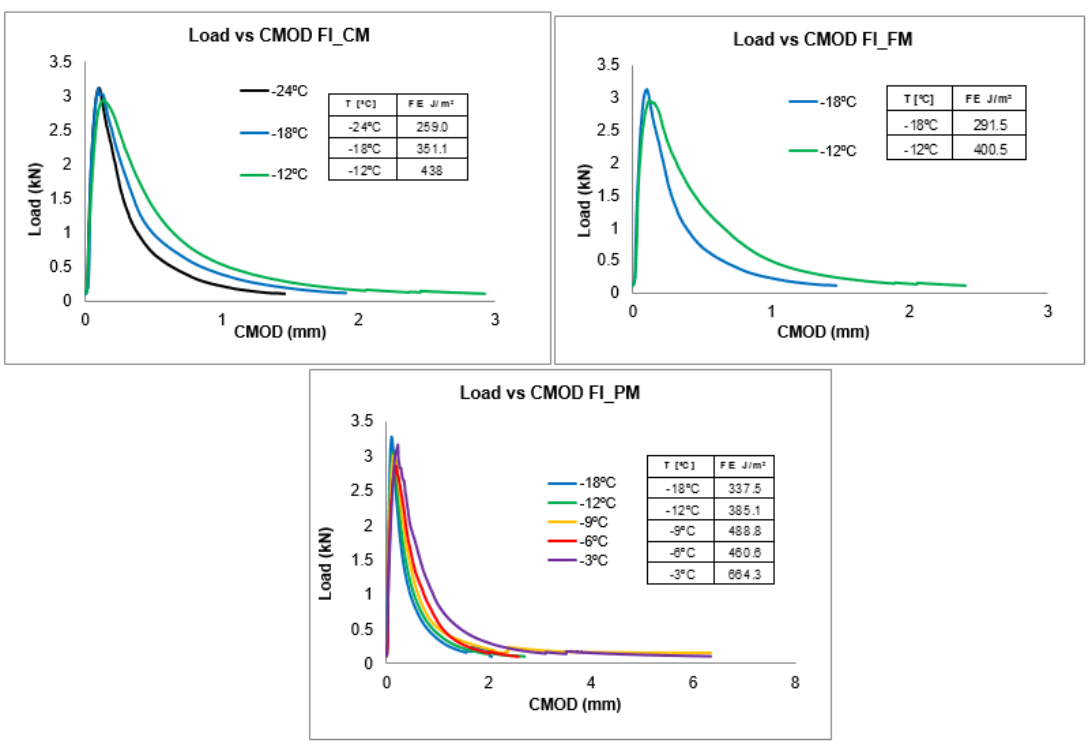


Figure 4-4. DC(T) Load-CMOD curves for fiber-reinforced mixtures and control mixture

4.2.2. Discussion of DC(T) Results

As shown in Figure 4-1, it is clear that fracture energy results are highly influenced by test temperature, reinforcing the importance of test temperature selection in accounting for field conditions. Researchers have observed that a variation of 25 J/m^2 in DC(T) fracture energy is high enough to show a difference in cracking performance between the mixtures (Hoplin, 2016). For the first group (group A) presented in Figure 4-2, all fracture energy results are below the original recommended threshold for low traffic (400 J/m^2), indicating a strong potential for thermal and/or block cracking. Major differences were observed in mixture MO13_1 with an increase of fracture energy in 77 J/m^2 between -18ºC and -6ºC and in mixture US63_1 with an increase of 84.1 J/m^2 between -18ºC and -9ºC. In general, this group shows a stronger influence due to colder test temperatures (between -18ºC and -9ºC), whereas at warmer test temperatures such as -9ºC, -6ºC and -3ºC, the fracture energy does not show relevant changes, tending to be more stable in this range of temperatures.

For rubber-modified mixtures and their respective control mixtures (group B) presented in Figure 4-3, almost all mixes meet the 400 J/m^2 criteria (except for TRO_L at -24ºC), while some cases reached 460 J/m^2, which is appropriate for a moderate traffic level. For control and rubber-modified mixtures TRO_L and TRO_L10R, a large increase of the fracture energy between -24ºC, -12ºC and -3ºC was observed. For the temperatures compared, fracture energies increased by up to 145 J/m^2 (in the case of TRO_L, comparing results at -12ºC and -3ºC).

For the Oklahoma mixes, there was a noticeable increase of fracture energy of about 55% for the control mix that was tested at two temperatures (-24ºC and -12ºC). A similar trend was observed in the Oklahoma rubber-modified mixtures (OK_70P5) with an increase in the fracture energy observed at warmer test temperatures. Mixture OK_70P5 (with 5% GTR) at -3ºC exhibited high cracking resistance, meeting the high traffic level criteria of 690 J/m^2.

For the last group (group C) presented in Figure 4-4, the fiber-reinforced mixtures, DC(T) results follow a similar trend of a higher fracture energy with increased temperature. The main differences were shown in FI_PM with an increase of fracture energy from 337.5 J/m^2 to 664.3 J/m^2 (97% increase) between -18ºC and -3ºC. For fiber modified lab mixtures (FI_CM and FI_FM) the same trend was observed. An increase up to 80% is seen between -24ºC and -12ºC for FI_CM, and an increase of 109 J/m^2 (37%) in fracture energy is observed between -18ºC and -12ºC in mixture FI_FM.

The peak load in the Load-CMOD curves and magnitude of the post-peak slope generally increased as the temperature decreased, indicating a more brittle behavior in the mixture at lower test temperatures, which resulted in the lower fracture energy values. This trend was noticed in the mixtures tested in this project as shown in Figure 4-2, Figure 4-3 and Figure 4-4.

4.2.3. DC(T) Results for Field Cores

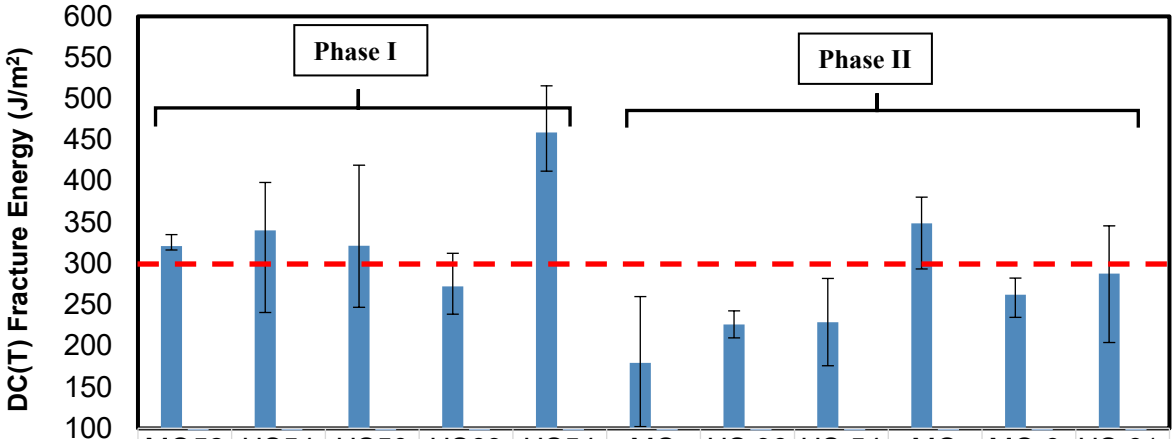
DC(T) samples were fabricated using the top lift of the collected field cores (see Table 4-2). The thickness of the top lift was at least 50 mm (2 inches) for most of the sections, which made it possible to cut the DC(T) samples into 50 mm slices. Three replicates were tested for each section, and the average of DC(T) fracture energies was calculated. Figure 4-5 shows the fracture energies tested at -12 °C. The error bars shown for each section covers the range of fracture energy obtained from testing the replicates. Also, mixture type and year of overlay are indicated for each section. The table attached to the figure provides the amount of recycled materials used in each mixture, including the ABR by RAP and RAS and total ABR. The tested mixtures are divided into two categories, phase I and phase II. The threshold of 300 J/m^2 was considered for these sections. Five sections showed fracture energies of higher than 300 J/m^2. The validity of this threshold will be discussed further in the next chapter by comparing the results with field performance data.

Table 4-2. Field sections with significant time in service

	Section #	**Constr. Year**	**Virgin Binder Grade**	**Asphalt Content (%)**	**ABR (%)**	**ABR by RAP (%)**	**ABR by RAS (%)**
Phase I	MO52_1	2010	PG64-22	4.8	33.5	0	33.5
	US 54_8	2006	PG70-22	5.6	8.6	8.6	0
	US50_1	2011	PG64-22	5.0	24.6	24.6	0
	US63_2	2008	PG64-22	5.6	29.9	19.9	10
	US54_7	2003	PG64-22	6.2	0	0	0
Phase II	MO 151	2010	PG64-22	4.7	30.6	15.9	14.7
	US 36 E	2011	PG64-22	5.1	24.7	24.7	0
	US 54 E	2010	PG70-22	5.7	11.8	11.8	0
	MO 94	2005	PG64-22	5.6	0	0	0
	MO 6 W	2015	PG58-28	5.9	29.6	29.6	0
	US 61 N	2013	PG64-22H	5.3	29.6	29.6	0

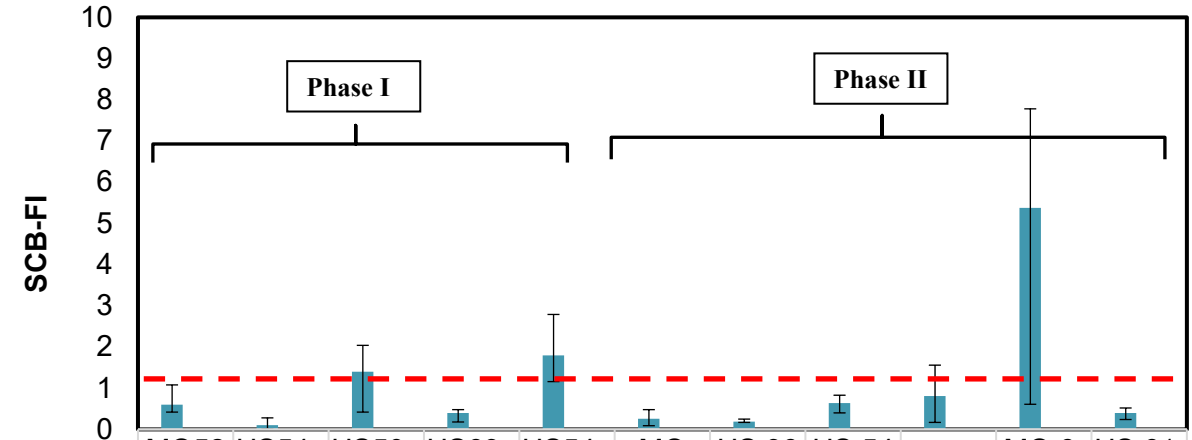
4.2.4. I-FIT Results for Field Cores

I-FIT testing was performed on the sliced top lift of the field cores. Four replicates were prepared and tested and the average of the FI parameter obtained for the twelve sections are presented in Figure 4-6. Similar to the previous figure, the error bars show the range of the FIs calculated for the four replicates. Among the eleven sections, MO 6 W, showed the highest FI. This section is only four years old, as compared to the older field sections studied. The results indicated that the FI is highly sensitive to aging (compare the high FI of the MO6 section to the other, older sections tested).



	MO52_1	US54_8	US50_1	US63_2	US54_7	MO 151	US 36 E	US 54 E	MO 94	MO 6 W	US 61 N
DC(T) FE	321.2	340.2	321.5	272.4	459.3	179.5	226.0	229.0	348.7	262.3	287.7
COV [%]	3.1	20.8	22.5	11.2	9.3	44.0	7.3	32.7	13.8	9.4	25.8

Figure 4-5. DC(T) FE and coefficient of variability for field sections



	MO52_1	US54_8	US50_1	US63_2	US54_7	MO 151	US 36 E	US 54 E	MO 94	MO 6 W	US 61 N
DC(T) FE	0.6	0.1	1.4	0.4	1.8	0.3	0.2	0.6	0.8	5.4	0.4
COV [%]	45.5	99.6	45.0	27.7	33.5	76.8	21.1	33.8	91.9	61.5	29.2

Figure 4-6. I-FIT SCB FI and coefficient of variability for field sections

Table 4-3 lists the Coefficient of Variation (COV) for the two cracking performance tests for the field sections tested. The average DC(T) COV for the field sections was 19.5%, but still within the recommended maximum COV range of 20% for fracture testing. On the other hand, the I-FIT test had an average COV of 52.2% for the field section evaluations. Higher COV values make it more difficult to draw inferences regarding statistical differences between means of sample populations to be compared (for instance, between testing parties), and create more uncertainly when comparing mean values against specification requirements. It may be advisable to use a trimmed mean approach, such as that proposed in Illinois, to reduce variability in the I-FIT test, or consider the use of the IDEAL test, which has been found to be more repeatable.

Table 4-3. Test averages and coefficient of variability for field sections

Section #	DC(T) FE (J/m^2)	COV(%) (DC(T))	FI	COV(%) (FI)
MO 52_1	321	3.8	0.6	51.4
US 54_8	340	25.5	0.14	80.7
US 50_1	322	27.6	1.4	51.6
US 63_2	272	13.7	0.36	35.4
US 54_7	459	11.4	1.85	37.6
MO151	179.5	44	0.3	80.2
US36 E	226	7.3	0.2	21.1
US54 E	229	32	0.6	33.8
MO94	348.7	13.8	0.8	91.9
MO6	262.3	9.4	5.4	61.5
US61	287.7	25.8	0.4	29.2
Average	N/A	19.5	N/A	52.2

Clearly, more test sections are needed to further evaluate performance tests and specification limits needed to establish a balanced mix design approach for mixes containing a high degree of recycled materials. That notwithstanding, the results obtained herein show the strong possibility of arriving at a specification in the near future that will allow contractor innovation in designing modern, heterogeneous recycled mixtures.

4.2.5. Hamburg Testing Results for Field Cores

HWTT results are presented in terms of number of wheel passes vs. rutting depth in Figure 4-7, Figure 4-8, and Figure 4-9. A threshold of 12.5 mm for rutting depth is adopted to discuss the results. Also, Figure 4-10 shows the Hamburg testing results for the tested field cores.

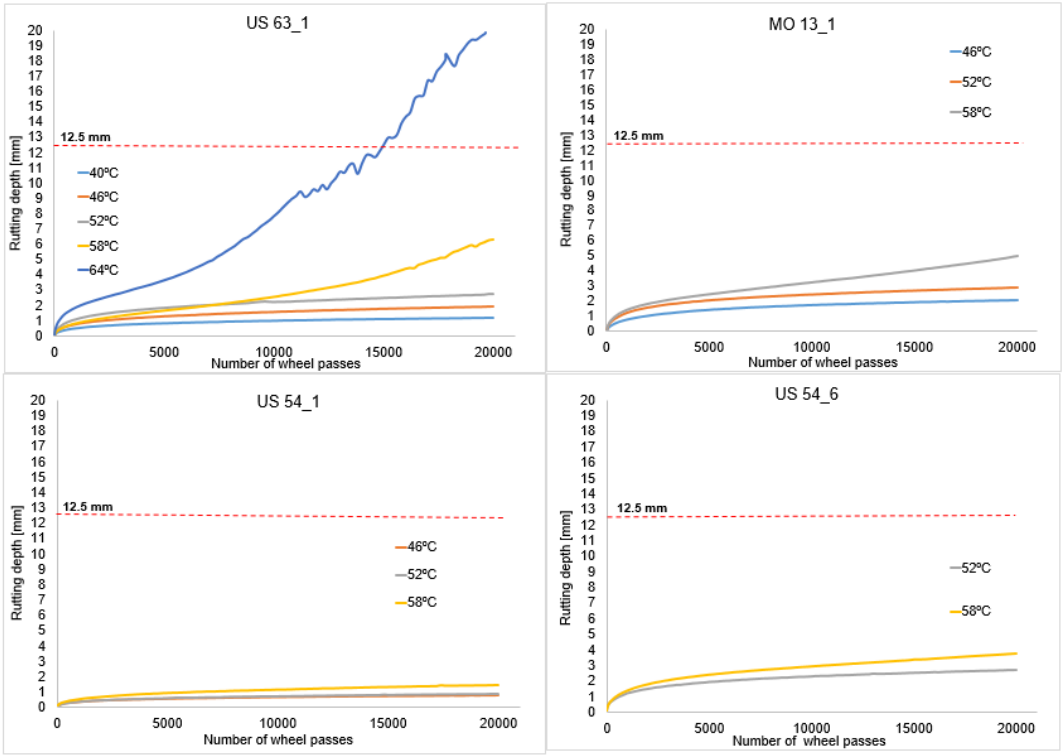


Figure 4-7. Rutting depth vs. wheel passes for level 1 Missouri mixtures

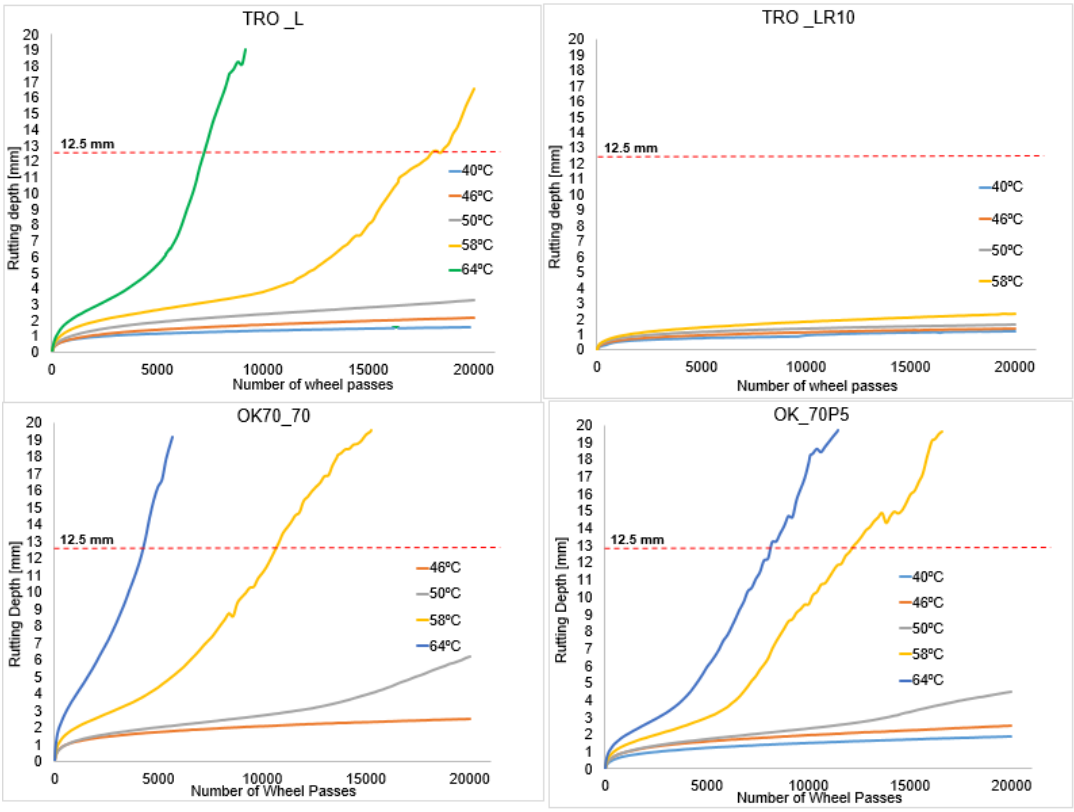


Figure 4-8. Rutting depth vs. wheel passes for rubber-modified and control mixtures

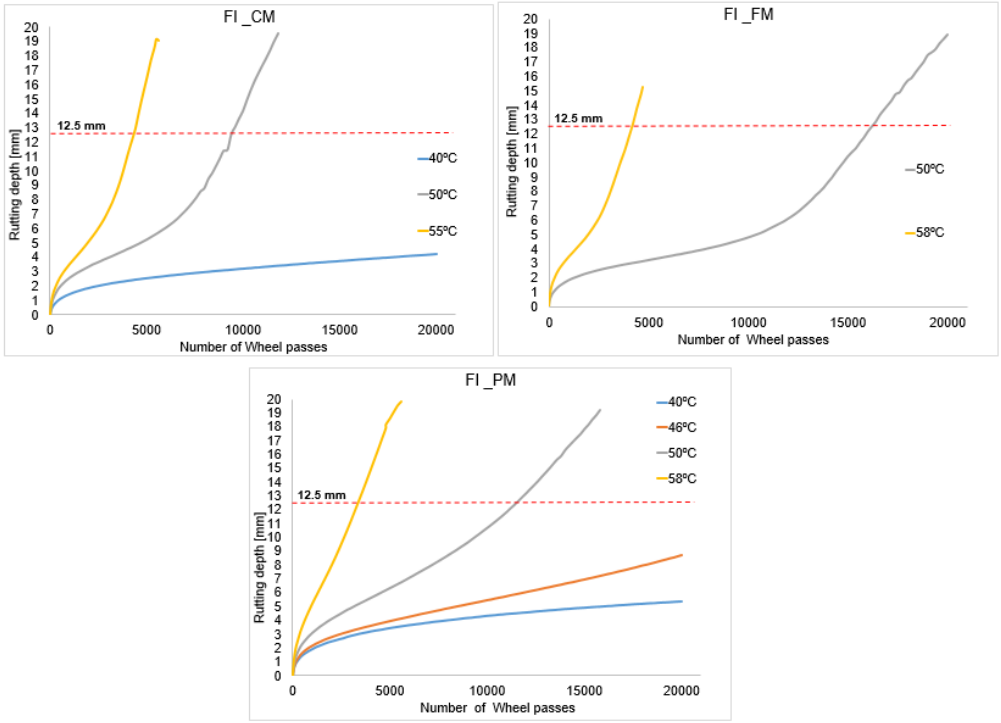


Figure 4-9. Rutting depth vs. wheel passes for fiber-modified and control mixtures

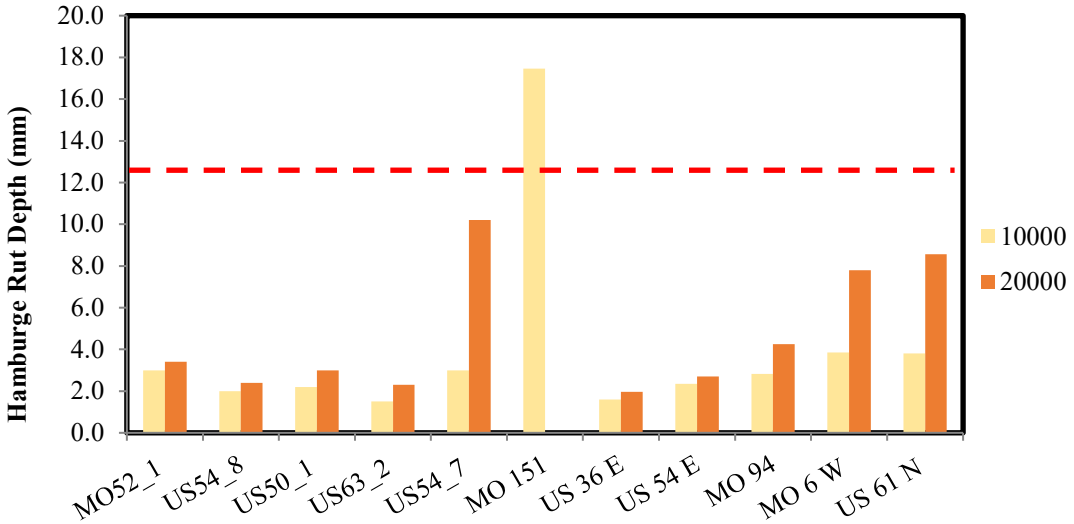


Figure 4-10. Hamburg testing results at required number of passes at 50°C

4.2.6. Discussion of Hamburg Testing Results

For the first group presented in Figure 4-7, mixtures are characterized by a low rutting depth that can be explained by the presence of a high content of recycled material in the mixes. According to researchers, high content of RAP and RAS increase the stiffness of the mixture thereby decreasing rutting magnitude (Buttlar et al., 2019; Rath et al., 2019a; Al-Qadi et al., 2012, Wang et al. 2020). Another factor could stem from the fact that the plant samples were

stored, then reheated prior to compaction, causing extra aging in the mixture and consequently lower deformation. A clear sensitivity to test temperature on the rutting depth is observed in Figure 4-7. Rut depth increases as wheel passes increases and as the temperature increases. Higher deformations were observed on mixture US63_1 where the threshold of 12.5 mm was exceeded when the mixture was tested at 64°C.

For the rubber-modified and control mixture in Figure 4-8 the same trend was observed. Higher rutting depths were measured at test temperatures of 58°C and 64°C. As was explained in Figure 3-10, at those temperatures a stripping slope is evident, suggesting a stripping potential exists in these mixtures at higher temperatures. Furthermore, the threshold of 12.5mm was reached in most of the cases at 58°C and 64°C. In the last group of fiber-modified and control mixture, higher deformation was observed at lower temperatures, reaching 12.5mm at 50°C (Figure 4-8). A strong influence of test temperature was observed in these mixtures. In some cases, the test finished before 5,000 wheel passes were reached.

There is a clear trend that as test temperature increases, the average deformation increases, which follows expectations. Similar behavior has been reported in other studies (Romero et al. 2008; Javilla et al., 2017). For all mixtures investigated, a clear relationship was observed between rut depth and test temperature, especially above 50°C.

4.3. Performance-space Diagram

Figure 4-11 to Figure 4-16 present a useful x-y plotting technique known as the 'performance space diagram,' or more specifically in this case, the Hamburg-DC(T) diagram (Buttlar et al., 2016; Rath et al., 2019a; Jahangiri et al., 2019). This plot allows the simultaneous evaluation of rutting and cracking behavior. Some useful trends that can often be observed when viewing data in this form are:

- The best overall performing mixtures will appear in the upper-right corner of the diagram (low rutting depth, high fracture energy). These can be considered as high 'total energy' mixtures; i.e., rut and crack (or damage) resistant. These are high toughness mixtures, and the best candidates for surfacing materials especially in demanding climates and for high traffic volumes.
- Mix variables that increase net total energy in the mix and thus 'move' mixtures in the direction of the upper-right corner of the plot include:
    - Higher quality binder (low temperature susceptibility, higher Useful Temperature Interval, or UTI), degree of polymer modification;
    - Higher quality aggregate (stronger, more angular, better bond with asphalt), and;
    - The presence of crack interceptors or rut mitigators, such as fibers, rubber particles, and even RAS (but only if properly used).
- Other salient features of the plot include:
    - Binders with different grades but similar UTI tend to move a mixture along a 'binder tradeoff axis', or roughly speaking, diagonal lines moving in the upwards-left or downwards-right directions, for stiffening and softening, respectively;

- Pure stiffening elements, such as RAP, tend to move points upwards and to the left;
- Pure softening elements, such as rejuvenators, tend to move points downwards and to the right;
- Binders with higher UTI, where the grade bump is on the high temperature grade, tend to move points mainly upwards, but also slightly to the right due to the benefits of polymer in intercepting cracks, and;
- Binders with higher UTI, where the grade bump is on the low temperature grade, tend to move points mainly to the right, but also slightly upwards, again, due to the benefits of polymer in intercepting cracks.
- Data points that appear in the undesirable middle-to-lower-left portion of the plot are sometimes those that contain RAP and insufficient binder bumping, and possibly poor bond, where the RAP tended to cause lower DC(T) values, and the nature of the RAP-virgin material combination led to a moisture-susceptible mix with high Hamburg rut depth value.

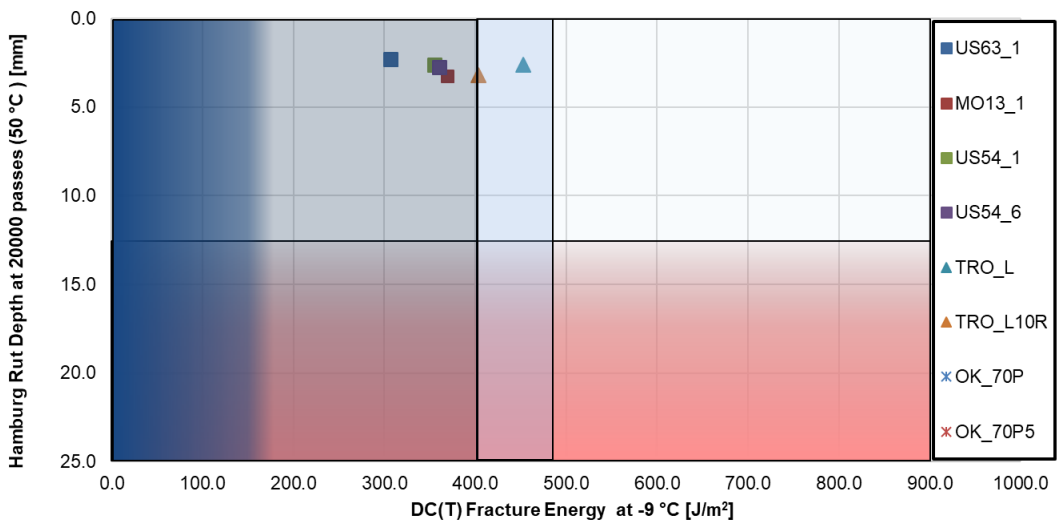


Figure 4-11. Hamburg-DC(T) performance-space diagram for plant and lab mixtures; DC(T) FE results at -9 °C

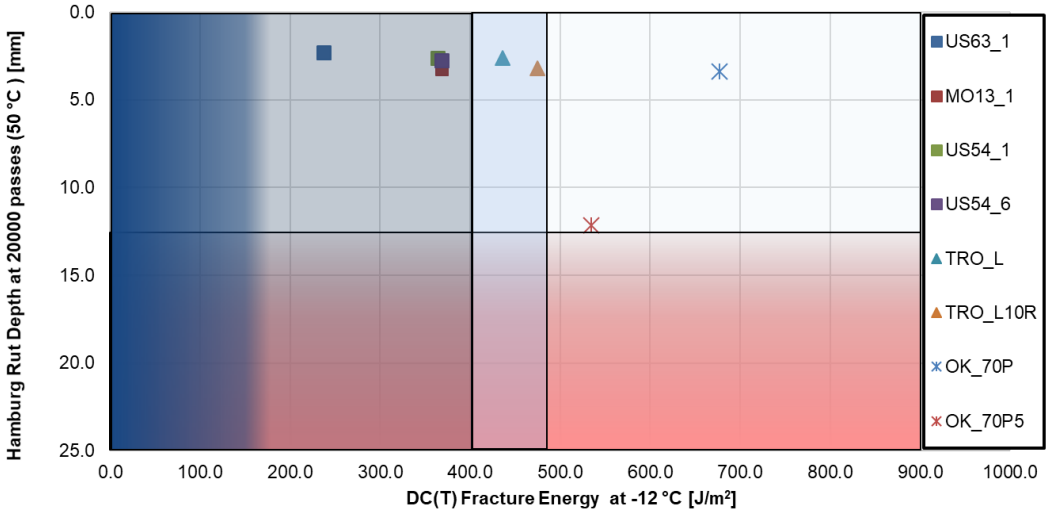


Figure 4-12. Hamburg-DC(T) performance-space diagram for plant and lab mixtures; DC(T) FE results at -12 °C

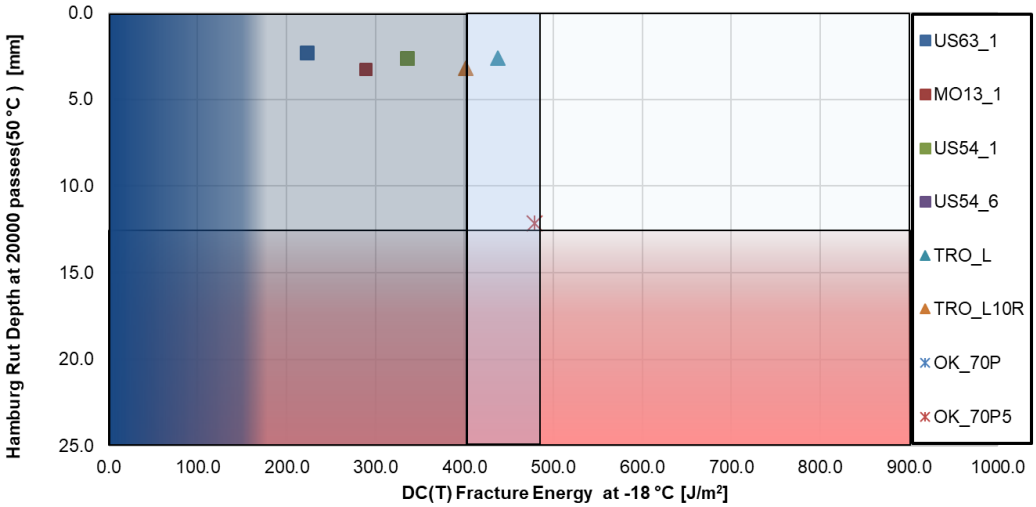


Figure 4-13. Hamburg-DC(T) performance-space diagram for plant and lab mixtures; DC(T) FE results at -18 °C

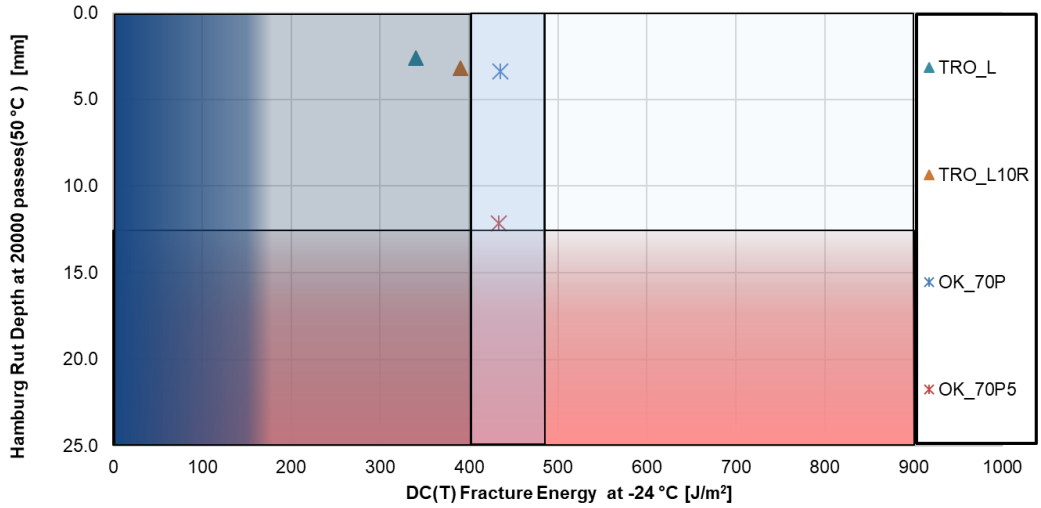


Figure 4-14. Hamburg-DC(T) performance-space diagram for plant and lab mixtures; DC(T) FE results at -24°C

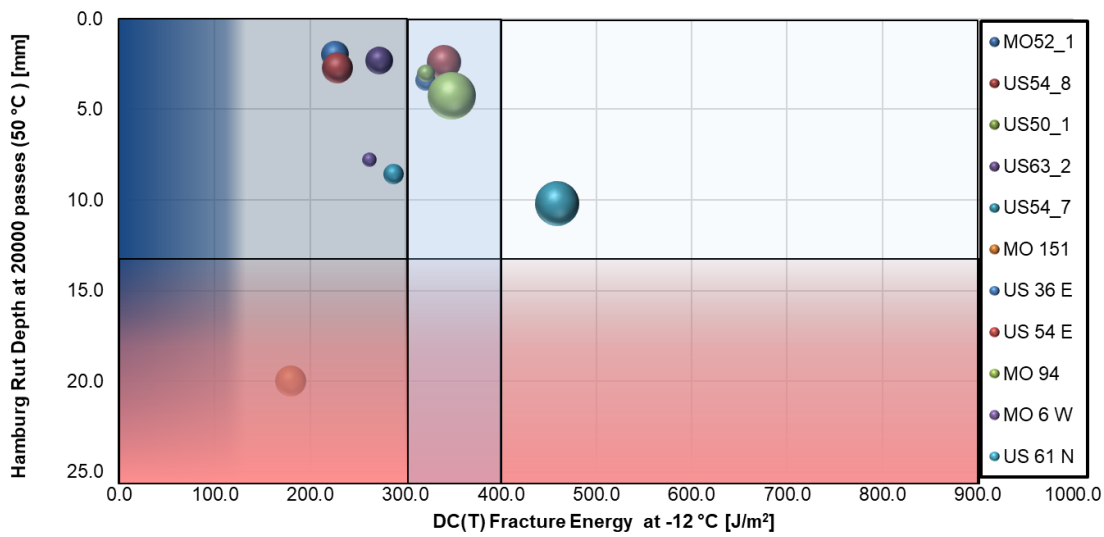


Figure 4-15. Hamburg-DC(T) performance-space diagram for field core mixtures

- The US54_7 mixture showed good performance in terms of fracture energy. However, the rutting is close to the allowable threshold, which might result from the stripping potential of this section after 13 years in service.
- MO151 showed the poorest performance in terms of both rutting and cracking. Among all the sections, only this section was prone to rutting.
- The MO6 asphalt surface had the shortest service life section among all the asphalt overlays investigated. However, this section is highly prone to cracking according to the

low DC(T) fracture energy value. This illustrates the importance of low temperature fracture energy in overlay cracking performance.

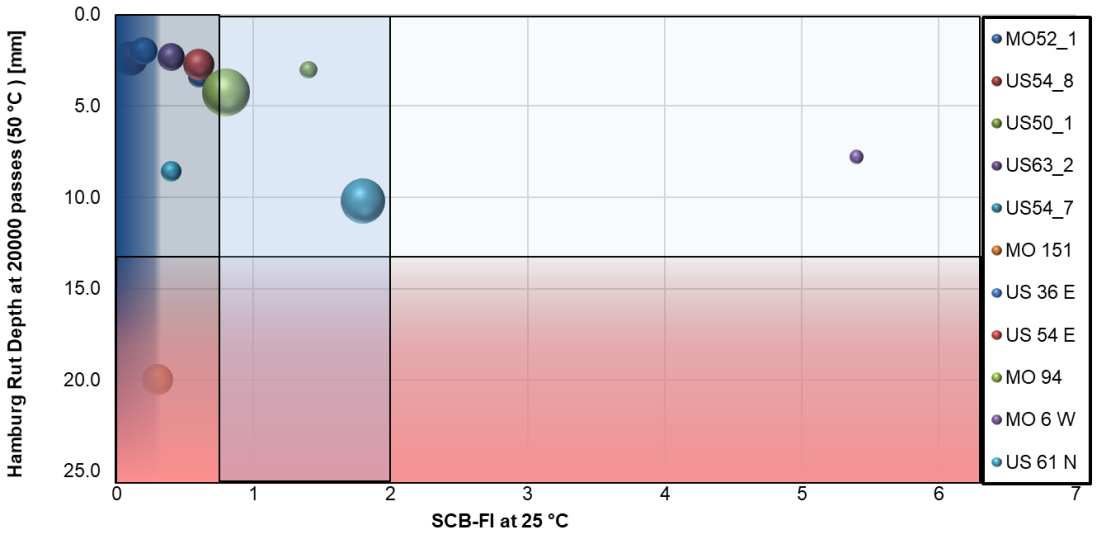


Figure 4-16. Hamburg-SCB (I-FIT) performance-space diagram for field core mixtures

Examining the Hamburg-SCB plot (Figure 4-16), it was observed that:

- The US54_7 mixture showed good performance in terms of flexibility index. However, the rutting is close to acceptable thresholds. This might result from testing of field cores from the 13-year old section, where microdamage accumulated slowly over time could increase moisture sensitivity in the HWTT.
- MO151 showed the poorest performance in both rutting and cracking. Among all the sections, only this section was prone to rutting.
- MO6 was the youngest section among all the sections. Although the DC(T) fracture energy was not high for this section, the flexibility index is 5.4. This section is about four years old, and thus has not experienced a sufficient level of aging to diminish the flexibility index score significantly.

In order to compare the scores obtained from two different cracking tests, the DC(T) and FI results are presented in Figure 4-17. Generally, a higher FI score is obtained as the DC(T) fracture energy increases, and indeed a roughly linear correlation was observed for the investigated field sections. The only section that did not follow this trend was MO6. This section has experienced the lowest service life so far (4 years) and recorded a relatively high FI of 5.4. As the I-FIT test is well known to be very sensitive to aging, it is expected that a drop in FI will occur as the section ages. It is thus expected that the data point for MO6 on the performance space plot will move more towards the unity line as time progresses.

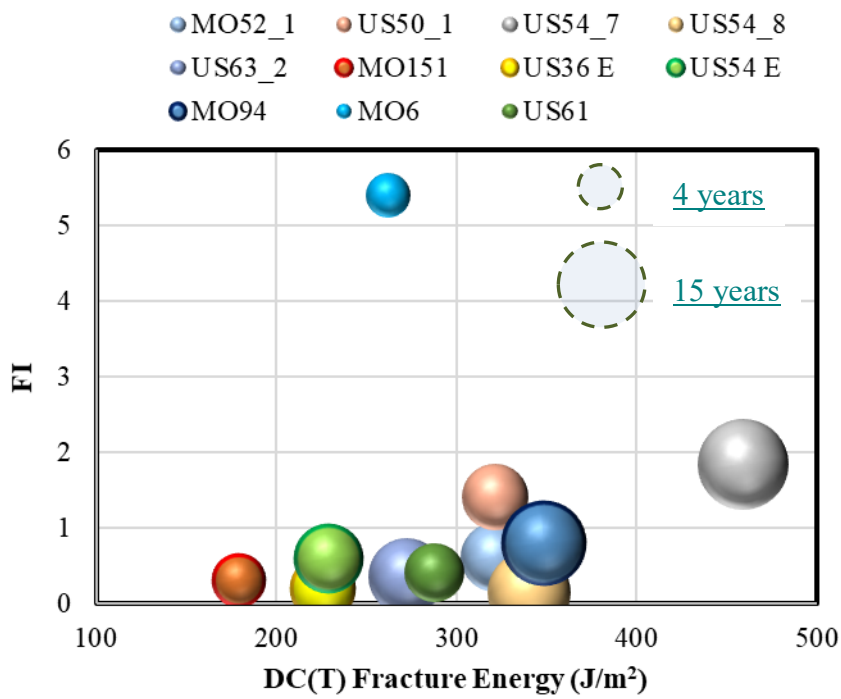


Figure 4-17. Comparison of DC(T) and FI test results for Missouri field sections

Chapter 5

5. FIELD PERFORMANCE DATA AND ANALYSIS

5.1. Overview

Field monitoring is a potential means to identify the most reliable cracking performance test (Majidifard et al., 2020a; Majidifard et al., 2020b). This part of the study evaluates the field performance of eleven sections located in Missouri. Details regarding the geographical location of the investigated sections are provided in Figure 5-1. Field performance data for the 11 oldest sections were collected from the MoDOT online PASER rating system, including ARAN video logs, which were used to delineate thermal cracks from block cracks. A series of pavement condition images are shown in Figure 5-2 for each section investigated in this study. Pavement Surface Evaluation and Rating (PASER) scores were extracted from the MoDOT portal.

To remove the dependency of pavement condition on aging, a deterioration rate was computed based on the average decline in PASER rating over the service life of the overlay. Table 5-1 summarizes the details of the sections, along with their PASER values and performance test results obtained from the extracted field cores for the respective sections. The field sections investigated cover a wide range of recycling levels and also include sections aged 5- 16 years in the field (Table 5-1). Table 5-2 presents the latest field performance measures available for these sections as of Fall 2019.

Table 5-1. Field sections with significant time in service

	Section #	Constr. Year	Virgin Binder Grade	Asphalt Content (%)	ABR (%)	ABR by RAP (%)	ABR by RAS (%)
Phase I	MO52_1	2010	PG64-22	4.8	33.5	0	33.5
	US 54_8	2006	PG70-22	5.6	8.6	8.6	0
	US50_1	2011	PG64-22	5.0	24.6	24.6	0
	US63_2	2008	PG64-22	5.6	29.9	19.9	10
	US54_7	2003	PG64-22	6.2	0	0	0
Phase II	MO 151	2010	PG64-22	4.7	30.6	15.9	14.7
	US 36 E	2011	PG64-22	5.1	24.7	24.7	0
	US 54 E	2010	PG70-22	5.7	11.8	11.8	0
	MO 94	2005	PG64-22	5.6	0	0	0
	MO 6 W	2015	PG58-28	5.9	29.6	29.6	0
	US 61 N	2013	PG64-22H	5.3	29.6	29.6	0

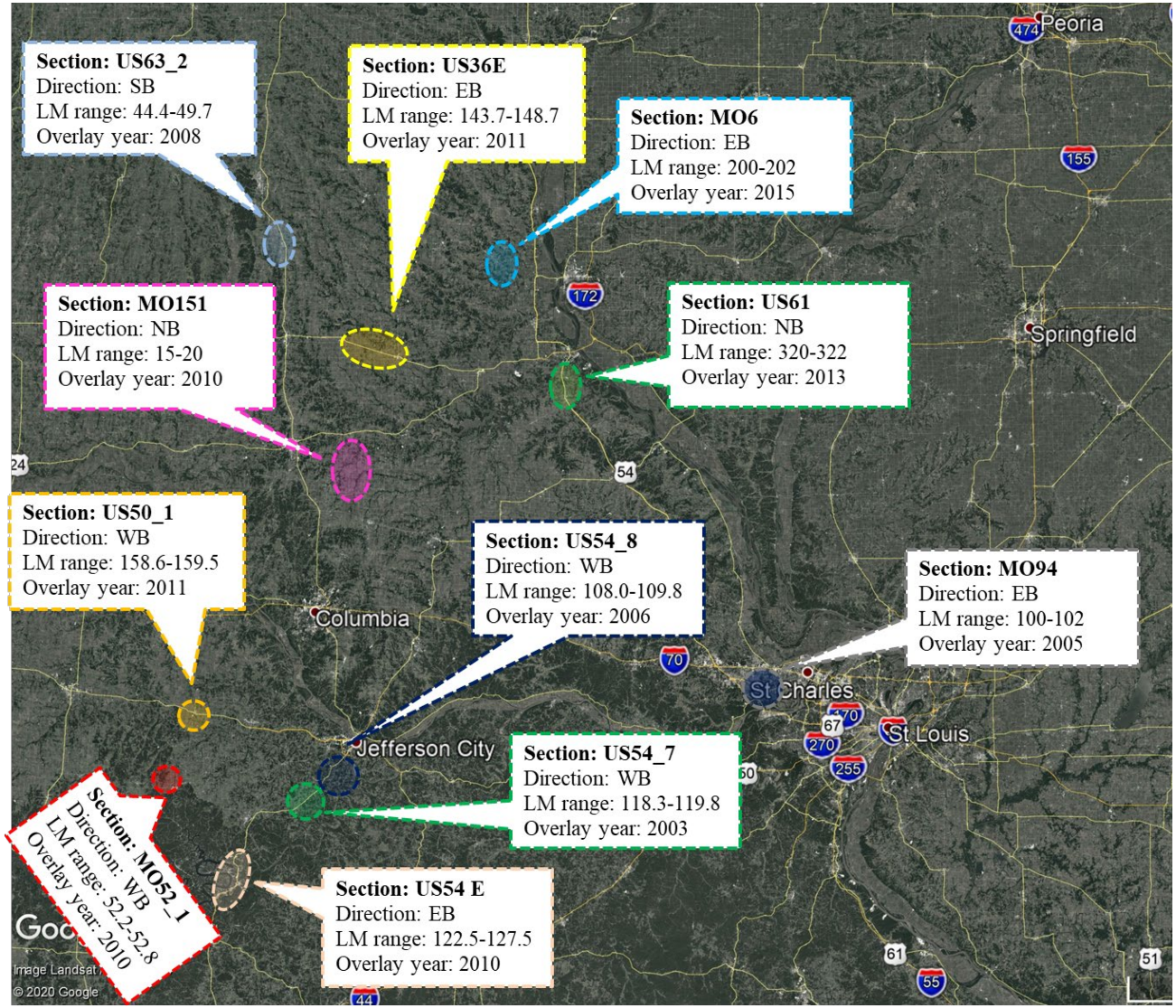


Figure 5-1. Location and details of the investigated field sections in Missouri (LM=Log Mile, NB=North Bound, WB=West Bound, EB= East Bound)

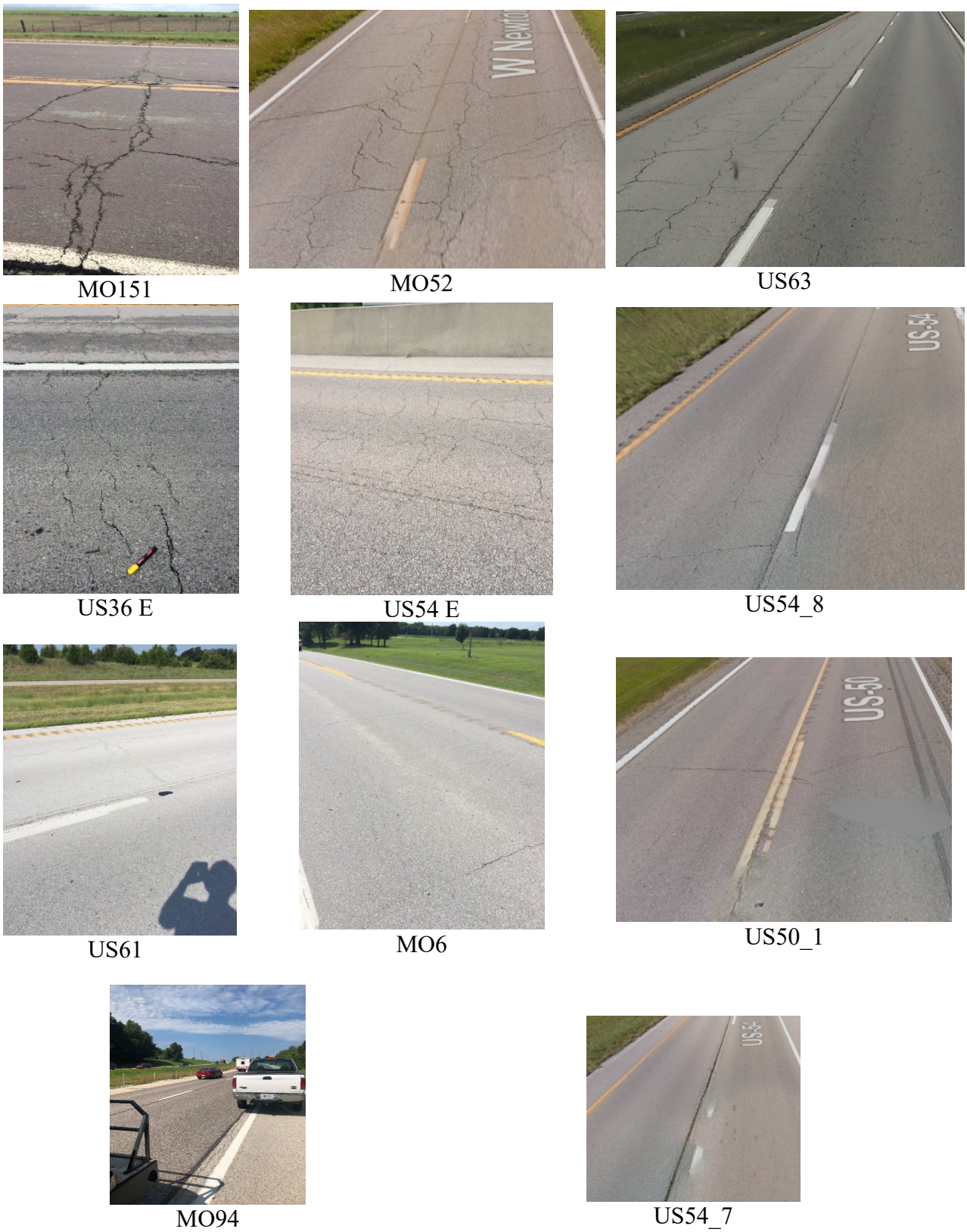


Figure 5-2. Images depicting typical pavement conditions in the eleven selected field sections arranged from highest (MO_151) to lowest (US54_7) deterioration rate

Table 5-2. Performance measures obtained from MoDOTs PASER portal in 2019

Section #	Constr. Year	IRI (in/mi)	PASER	ARAN Rut Depth (mm)	Brief summary distresses observed in ARAN images
MO52_1	2010	91	4.0	7.1	Mainly joint reflective cracking
US 54_8	2006	64	5.5	2.0	Block cracking developing
US50_1	2011	61	6.5	2.2	Very little distress
US63_2	2008	78	4.5	1.5	Dense block & thermal cracking
US54_7	2003	53	7.5	3.0	Very little distress after 14 years
MO 151	2010	52	6.4	4.7	Dense block & thermal cracking
US 36 E	2011	90	5	2.5	Dense block & thermal cracking
US 54 E	2010	88	5	2.5	Block cracking developing
MO 94	2005	88	7.8	0.9	Very little distress after 14 years
MO 6 W	2015	52	8.7	2.5	Very little distress
US 61 N	2013	60	8	2.5	Very little distress

Table 5-3 summarizes the 11 field sections with respect to overlay year, PASER rating, deterioration rate, ARAN rut depth, DC(T) fracture energy, FI, and HWWT rut depth results. A deterioration rate was computed based on the average decline in PASER rating over the service life of the overlay. The US 54_7 section was the best performer, with a very low deterioration rate (still very little cracking after 15 years in service). This section contained no recycling and also possessed a relatively high binder content for a 12.5 NMAS mix (6.2%). The worst 2 performers were MO52_1 and US63_2, which had the highest recycling rates (34 and 30% ABR, respectively), but without any bumping/softening of the virgin binder grade.

Table 5-3. Field section details vs. average deterioration rate

	Route	Overlay Year	PASER (2019)	Det rate*	DC(T) FE	FI (SCB)	HWTT Rut (mm) @10,000	ARAN Rut (mm)
Phase I	MO52_1	2010	4	0.83	321.2	0.6	2.8	7.1
	US54_8	2006	5.5	0.45	340.2	0.1	1.9	2
	US50_1	2011	6.5	0.35	321.5	1.4	2.1	2.2
	US63_2	2008	4.5	0.63	272.4	0.4	1.7	1.5
	US54_7	2003	7.5	0.14	459.3	1.8	3	3
Phase II	MO151	2014	4	0.90	179.5	0.3	17.5	4.7
	US36 E	2011	5	0.50	226	0.2	1.6	2.5
	US54 E	2010	5	0.50	229	0.6	2.3	2.5
	MO94	2005	7.8	0.16	348.7	0.8	2.8	0.9
	MO6	2016	8.6	0.43	262	5.4	3.9	2.5
	US61	2013	7.9	0.40	287.7	0.4	3.8	2.5

*Average deterioration rate in terms of change in PASER rating per year.

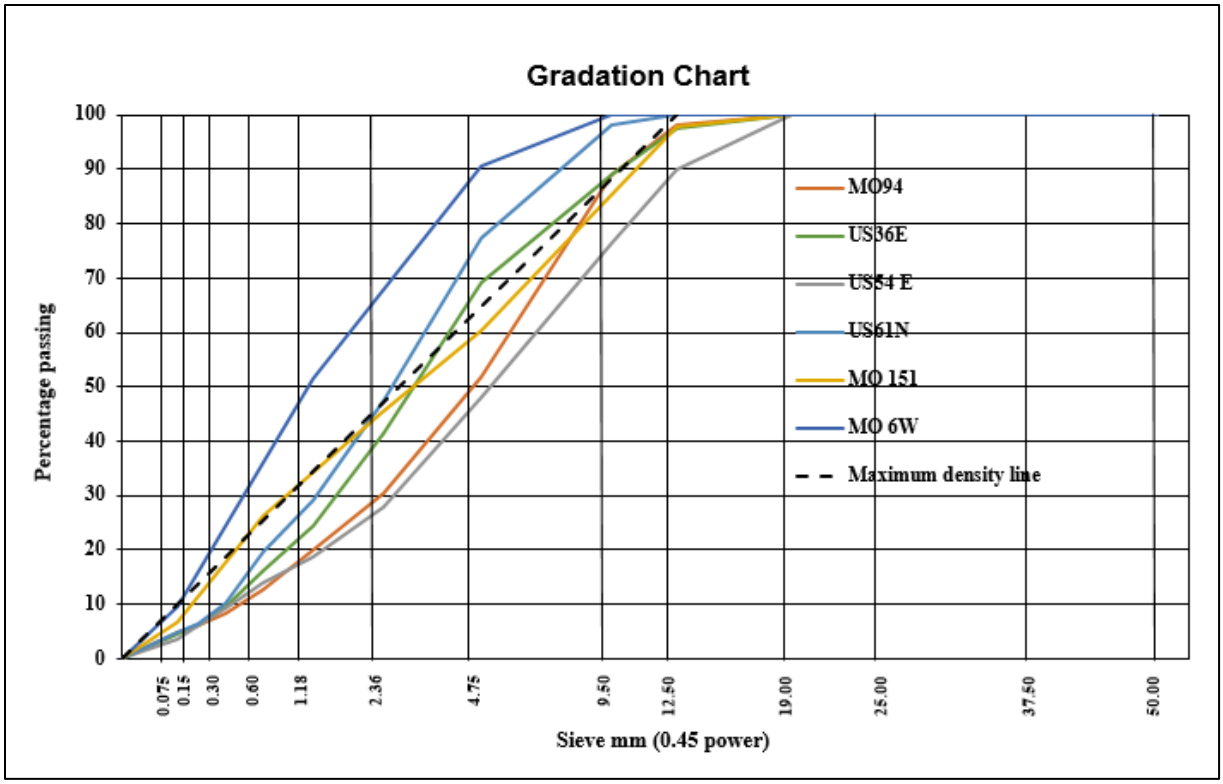


Figure 5-3. Gradation curve of field core mixtures

5.2. Field Performance Measures

Pavements, like all other infrastructure assets, deteriorate over time and thus require routine maintenance activities to be conducted by transportation agencies in order to maintain acceptable levels of serviceability. The first step towards planning a pavement maintenance activity is to measure and track pavement condition. In Missouri, this is achieved by implementing systematic Pavement Condition Surveys (PCSs).

PCSs refer to activities that quantify the pavement serviceability and its physical condition. They are mainly comprised of three aspects, namely, data collection, condition rating, and quality management. The data collection, which is mostly semi-automated or automated at this point in most states, provides a measure of the distresses prevalent in an existing pavement section. The data might also include other details about the pavement construction, such as length and width of the section, location of underlying structures, and details of last conducted preservation or maintenance activity. The condition rating is usually index- or scale-based to help quantify the condition of a pavement section. Various systems for condition rating exist, and the adoption of a system depends on available resources and local familiarity with the rating system. Maintenance decisions can then be made based on current or predicted road condition to ensure that a minimum level of pavement serviceability is retained.

In this section, the field performance data obtained from MoDOTs PCS database are presented, following data processing and analysis by the research team. The data set consists of the pavement condition rating system (PASER), international roughness index (IRI), and rut depth collected from the mainline sections. The objective of the field performance data analysis is to

establish a link between the field performance and laboratory testing results. The link will ultimately be used to determine performance test thresholds and to calibrate the specification.

Pavement roughness is generally defined as an expression of irregularities in the pavement surface that adversely affect the ride quality of a vehicle (and thus the user). Roughness is an important pavement characteristic because it affects not only ride quality but also user delay costs, fuel consumption, and maintenance costs. Pavement ride can alternatively be characterized in terms of "smoothness." That notwithstanding, IRI has become the standardized measure of the reaction of a vehicle to roadway profile. Generally, higher IRI values represent rougher roads and vice versa. The most commonly used units for IRI are inches per mile (in/mile), meters per kilometer (m/km), or millimeters per meter (mm/m). Figure 5-4 shows the IRI values for the eleven field sections investigated as a function of service life.

The time of construction for the most recent asphalt overlay was taken as year zero for each section, and thus the condition of the pre-existing pavement is represented by 'negative' year labels on the plots. Although fluctuations in the PASER and IRI data create noise in the data, the overall deterioration rate and overlay performance can be reasonably assessed in this manner. These plots further demonstrate the relatively good performance of US54_7 and MO 94, and relatively poor performance of MO52_1, US63_2 and MO 151. Identical rankings in terms of IRI and PASER deterioration rates are noted.

5.2.1. International Roughness Index

Figure 5-4 presents the IRI values during the service of the studied sections. Generally, increasing IRI with time as the pavement distress types develop and the road surface roughens under traffic loads and as a result of environmental effects. However, minor surface treatments such as crack sealing result in a drop in IRI values, as apparent in the MO151 IRI data. MO6 is the youngest overlay section, with only four years of service life to date. The two oldest sections were MO94 and US54_7, which have experienced 15 and 14 years of service life, respectively. IRI values during the first several years of service for these sections were not available. Despite the long time in service, US54_7 possesses the lowest IRI among the sections studied. Conversely, MO52_1 yielded the highest IRI after only 7 years. The rate of IRI loss versus time will be presented later in this report as a means to investigate correlations between laboratory and field performance.

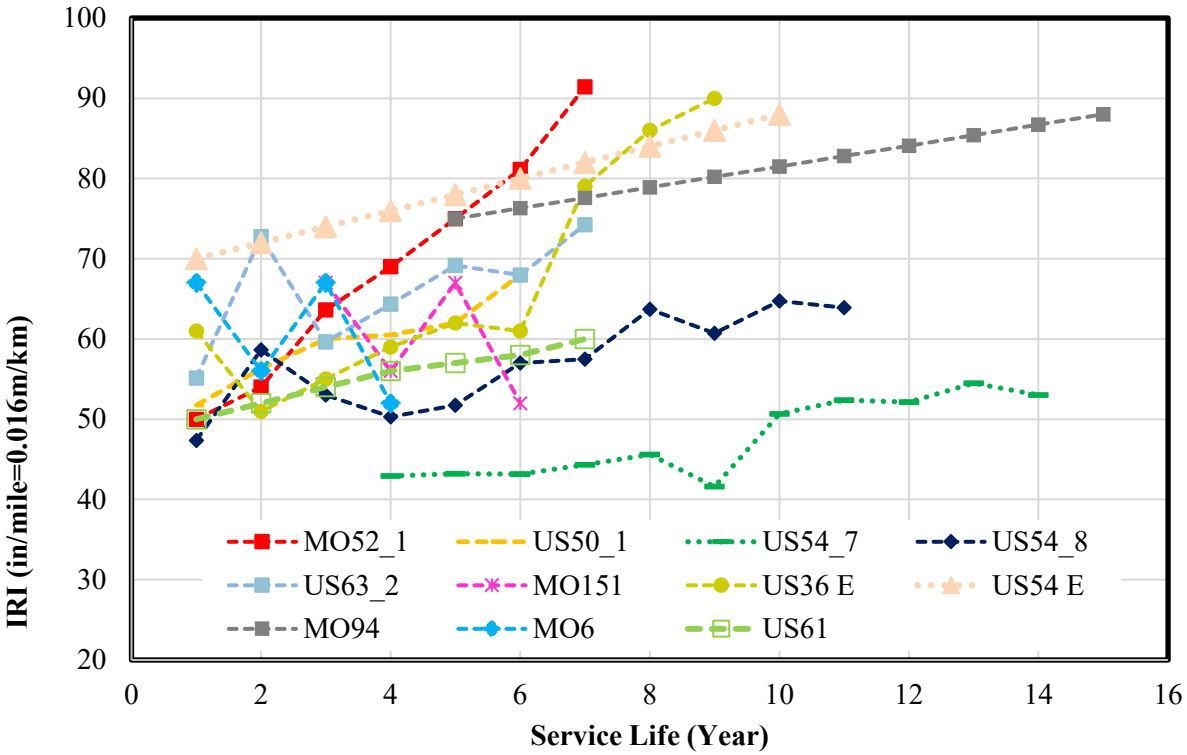


Figure 5-4. IRI as a function of service life

A higher rate of IRI loss suggests higher susceptibility to cracking, rutting, and moisture damage for a given overlay mixture. In order to investigate the relation between the cracking resistance and pavement deterioration rate, Figure 5-5 plots the rate of IRI increase (deterioration rate) vs. the DC(T) fracture energy. The IRI deterioration rate is calculated by dividing the difference between the IRI values at the beginning of service life and that of 2019 by the age of the section. A bubble-style plot is used in this figure to indicate the differing ages of the sections. The two relatively new overlay sections (MO151 and MO6) appear at the bottom of the plots. Although the DC(T) fracture energy recorded in these two sections was not high (180 and 262 J/m^2, respectively), their deterioration rate was negligible as they did develop significant roughness, especially at the beginning of their service lives. However, IRI for these sections is expected to increase with age. A specific trend could not be observed between the IRI deterioration rates as a function of DC(T) fracture energy. As seen in Figure 5-6 an inverse function [y=a/(x+b)] is used to fit the data already presented in Figure 5-5. In this function, y is the IRI deterioration rate, x is the DC(T) fracture energy, and a and b are model coefficients. DC(T) fracture energy is known as a promising tool to evaluate the low temperature and block cracking potential of the pavement. The unclear correlation between the IRI deterioration rate, as a field performance measure, and the DC(T) could be attributed to the sensitivity of the IRI to other distress types such as rutting, shoving, fatigue cracking, and underlying pavement condition. After removing the outliers including MO52_1 (with an extremely high IRI Det. Rate), MO151, and MO6 (with zero IRI Det. Rate), an R^2 of 0.42 is calculated for this correlation.

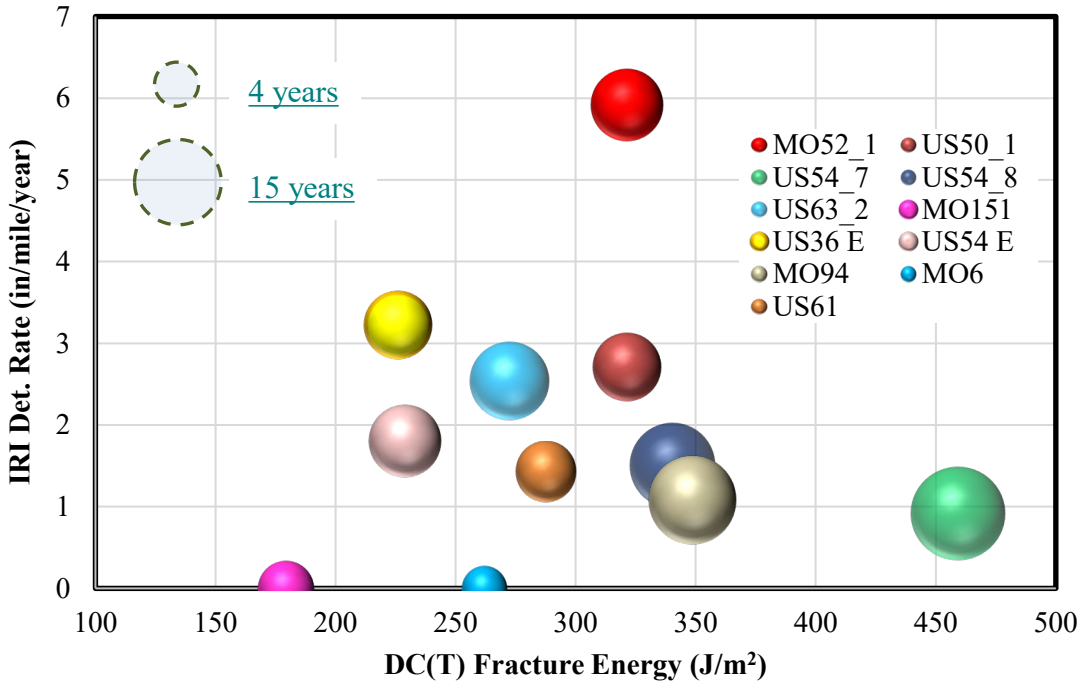


Figure 5-5. IRI deterioration rate as a function of DC(T) fracture energy

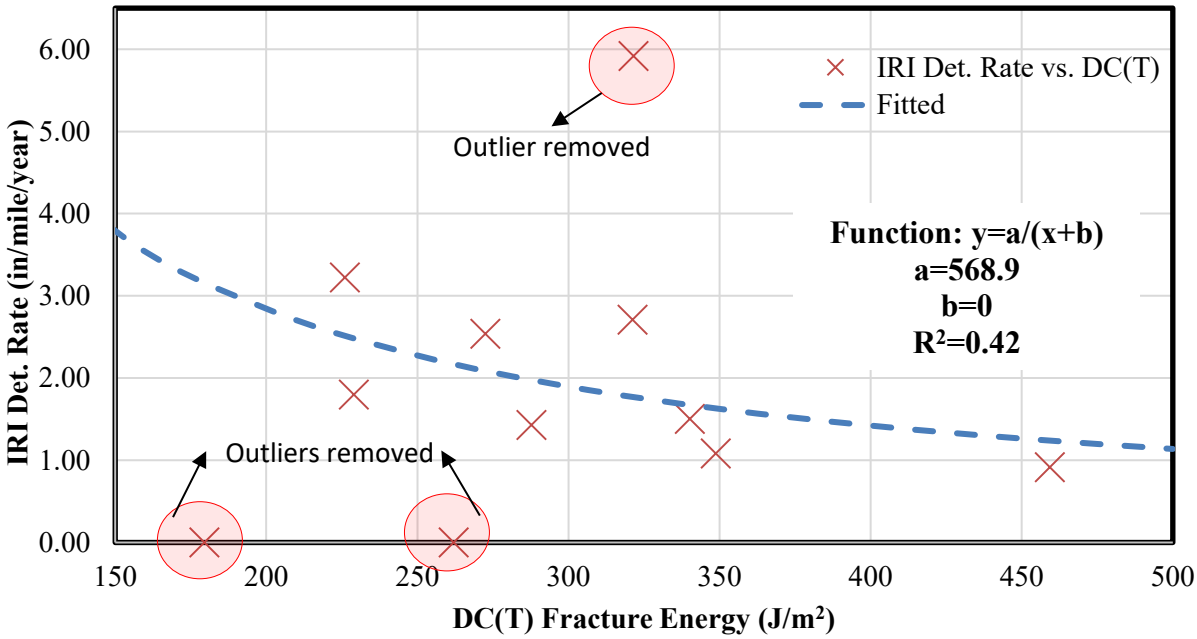


Figure 5-6. Correlation between IRI deterioration rate and DC(T) fracture energy

Another cracking test which was performed in this project is the SCB I-FIT. The IRI deterioration rate is used again to investigate its correlation to the FI parameter. As shown in Figure 5-7, the IRI deterioration rate decreases as the FI increases. Although this could be generally observed from the plot, there are exceptions such as the MO151 section, having a FI score of only 0.3, but possessing an IRI deterioration rate of 0 in/mile/year in the first six years of service life. Similar to the DC(T), the correlation of FI with IRI deterioration rate is investigated in Figure 5-8. Applying the same function previously used for DC(T) in Figure 5-6, the R^2 was 0.11, after removing the same outliers. Although IRI is a useful overall indicator of field performance, especially from the perspective of the traveling motorist, it was decided to next evaluate the correlation between cracking test scores to PASER ratings.

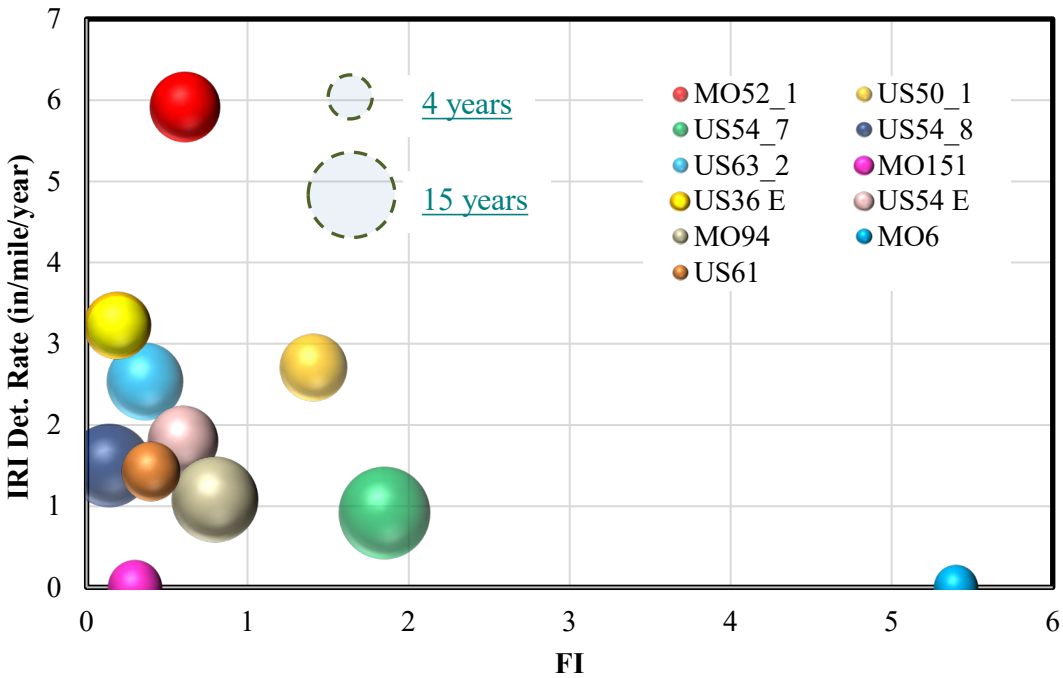


Figure 5-7. IRI deterioration rate as a function of FI

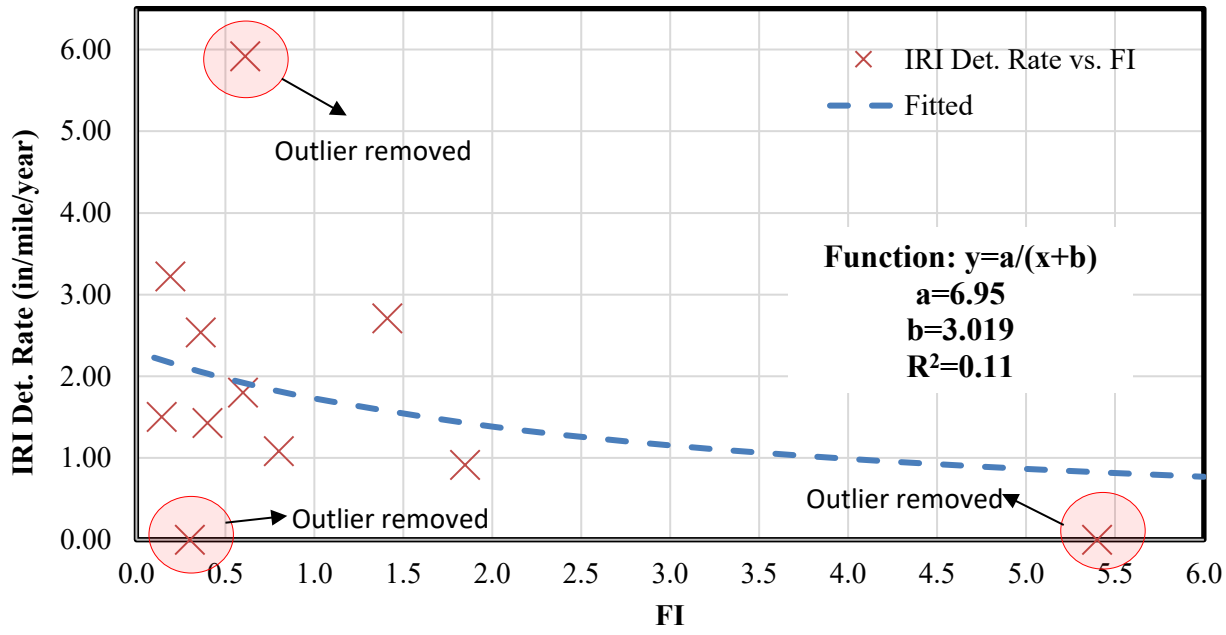


Figure 5-8. Correlation between IRI deterioration rate and FI

5.2.2. PASER

The field performance of the studied sections was also investigated using PASER ratings. Figure 5-9 presents the change in the PASER score vs time. As expected, ratings of all sections generally decreased with the age of the section. Similar to the IRI, the PASER for MO52_1 and MO151 drops with the fastest rate amongst all sections. It appears that MO94 benefited from a maintenance strategy such that its PASER followed an increasing trend after the 10th year of service life. MO52_1 and US54_7 have the highest PASER scores despite their high number of years in service.

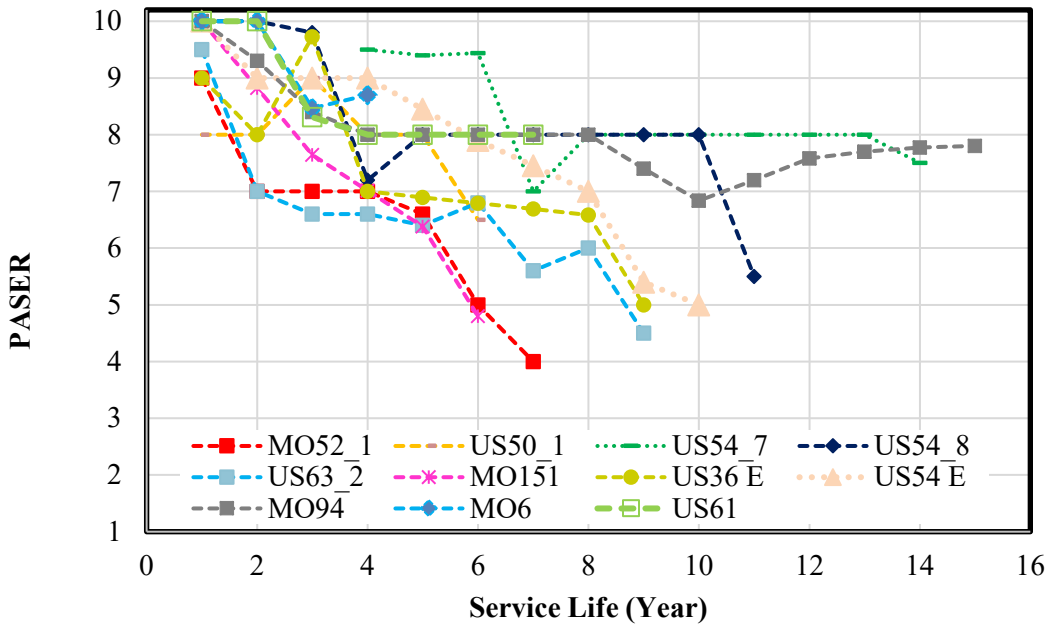


Figure 5-9. PASER as a function of service life

The correlation between PASER deterioration rate and DC(T) fracture energy is shown in Figure 5-10. Clearly, the PASER deterioration rate has a better correlation with DC(T) fracture energy than IRI deterioration rate. Section MO151 has the highest deterioration rate (≈ 0.9 per year) and accordingly recorded the lowest DC(T) fracture energy of 180 J/m^2. In addition, the US54_7 section with the highest fracture energy of 460 J/m^2 yielded a very low PASER deterioration rate of 0.19 per year. A higher R^2 value of 0.51 was calculated after fitting the previously implemented function to the data (Figure 5-11). The higher R^2 seems to indicate that low temperature and block cracking addressed by DC(T) fracture energy plays a more significant role in PASER as compared to IRI. For example, block cracking is easy to detect from pavement images and leads to lower PASER ratings. However, unless highly severe, block cracking may not adversely affect ride quality. Given the better correlation observed between PASER (as a field performance measure) and laboratory testing results (DC(T) fracture energy), the PASER deterioration rate appears to be a better metric for establishing minimum or baseline thresholds for cracking performance tests, as described in Chapter 6.

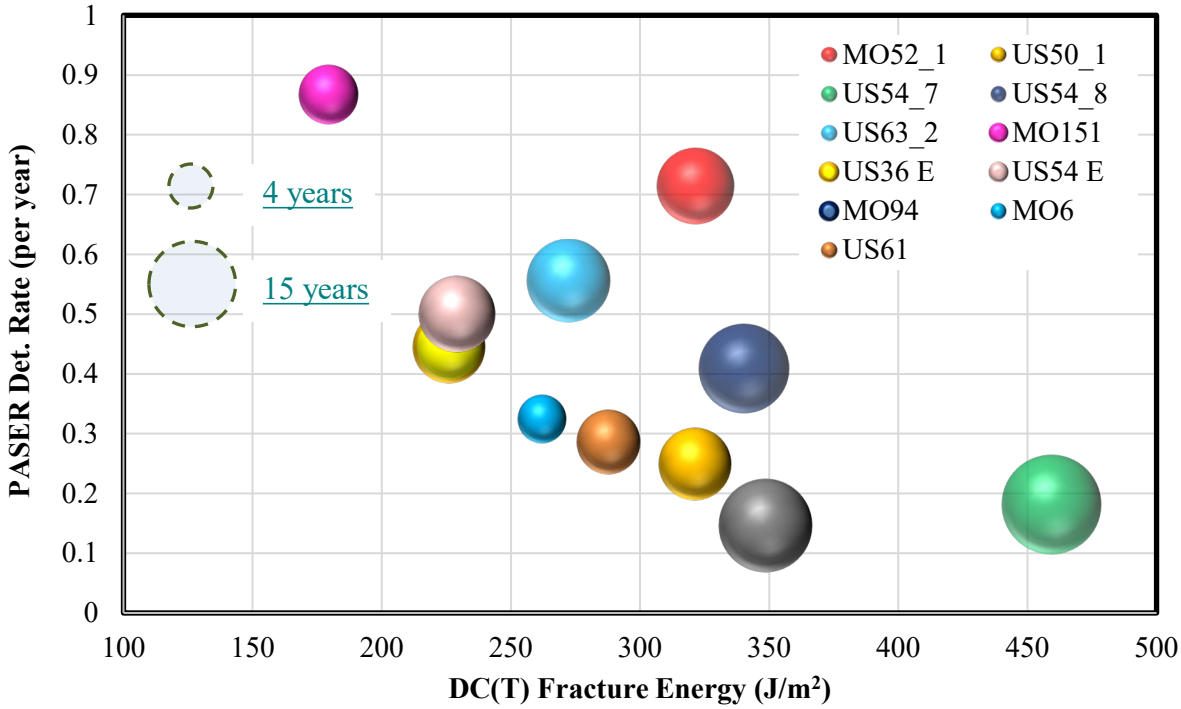


Figure 5-10. PASER deterioration rate as a function of DC(T) fracture energy

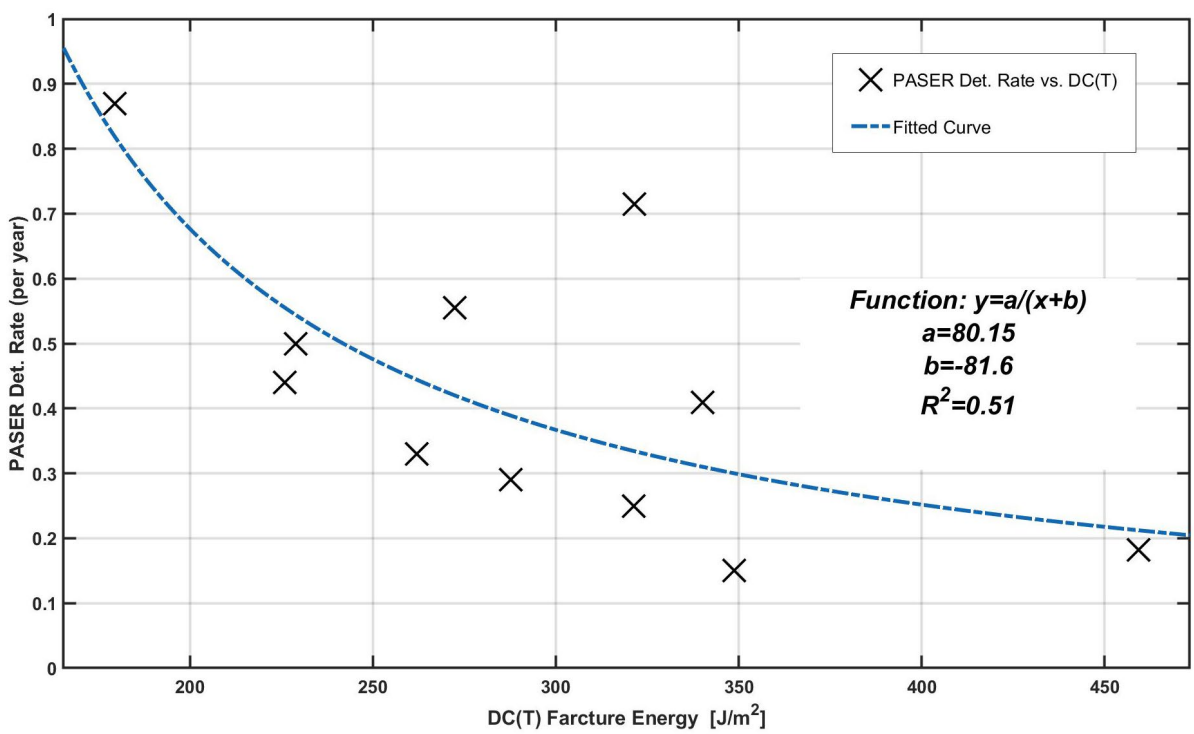


Figure 5-11. Correlation between PASER deterioration rate and DC(T) fracture energy

Figure 5-12 shows the PASER deterioration rate as a function of FI. As expected, the higher FIs resulted in a lower deterioration rate. It is worth mentioning that except for MO6, all sections yielded FI scores lower than 2.0. As I-FIT is very dependent on mixture aging, the size of the bubbles should be taken into consideration when the results are interpreted. The MO6 section (the blue bubble on the bottom right side of the plot) recorded the highest FI, meaning that the cracking potential is expected to be the lowest based on the I-FIT cracking assessment. However, this section is the youngest among the studied sections and the FI might considerably decrease as the section further ages. US54_7, MO94, and US 50_1 were the next best performing sections in terms of the FI parameter. The PASER deterioration rates for these sections were relatively low. The correlation between the PASER deterioration rate and FI is investigated in Figure 5-13. With an R^2 of 0.20, the FI was found to have a better correlation with PASER deterioration rate as compared to the IRI deterioration rate. This is consistent with the findings observed for the DC(T).

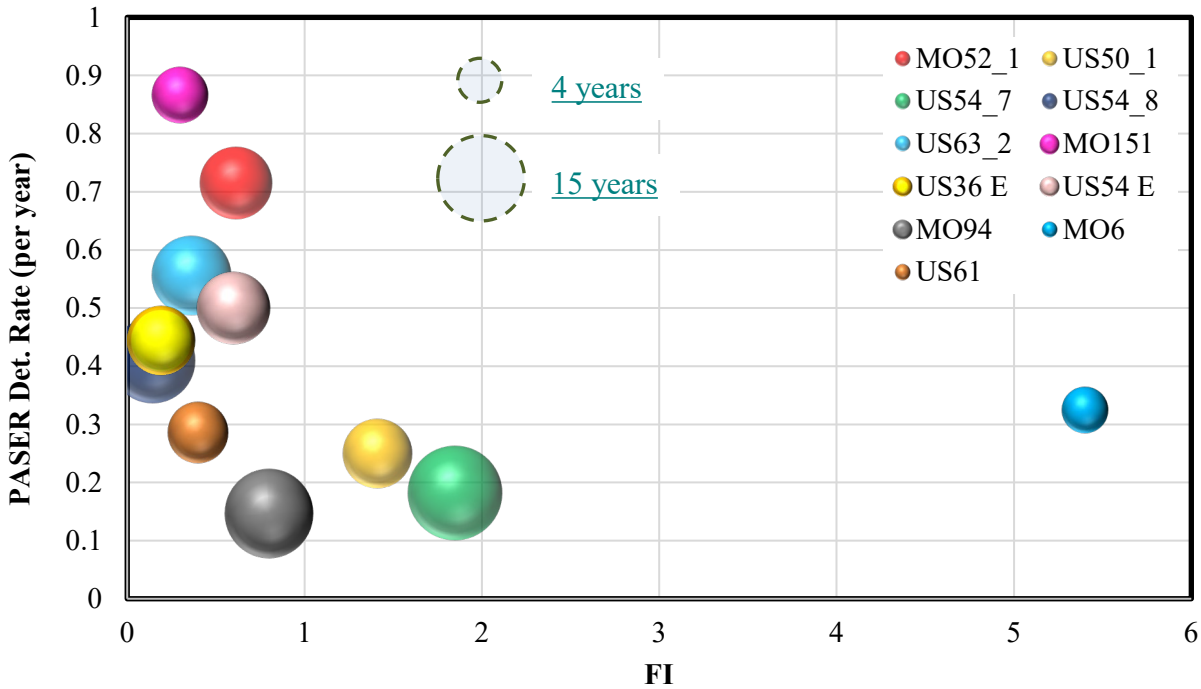


Figure 5-12. PASER deterioration rate as a function of FI

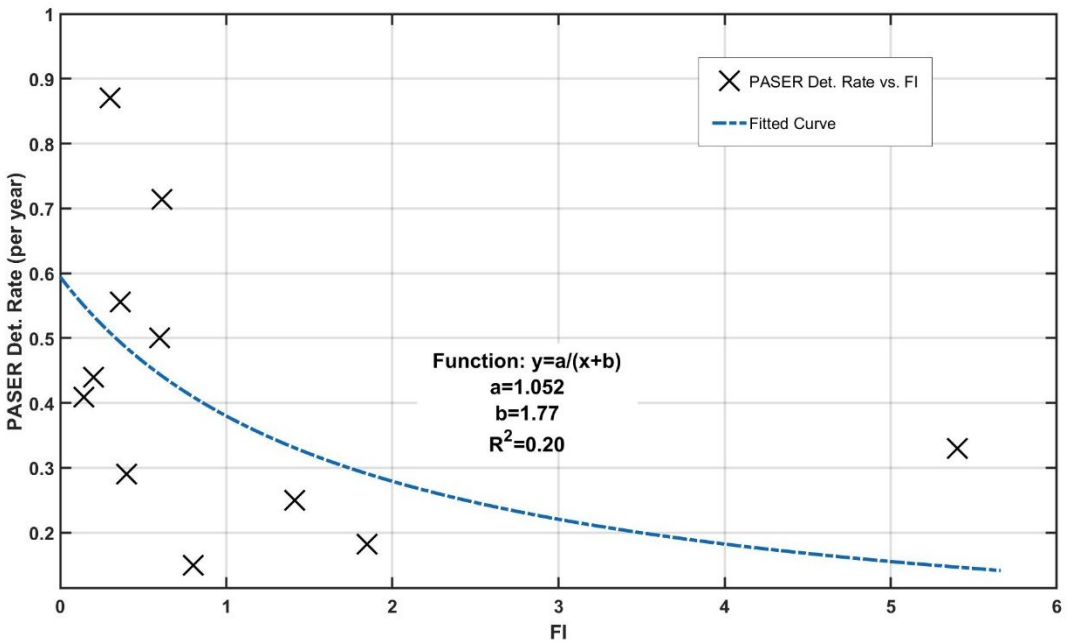


Figure 5-13. Correlation between PASER deterioration rate and FI

5.2.3. Rut Depth

Figure 5-14 presents the rut depth measured in situ for the six sections investigated in 2019. Rut depth values were generally lower than 0.2 in (5 mm) during the service life of these sections. Figure 5-15 plots the measured rut depth against the Hamburg rut depth at 10,000 wheel passes (medium traffic level) at 50 °C. MO151 was the only section exhibiting rutting and/or stripping in the HWTT (17.5 mm of rut depth after 10,000 passes). Note that only one Hamburg testing result was available for this section, due to limited availability of field cores. The notable discrepancy between the performance test and field performance might be attributable to the condition of the tested sample, possibly due to damage present in the field cores. The other 5 sections had both recorded Hamburg rut depths and measured rut depths in the field of less than 5 mm. In these cases, the Hamburg test appeared to properly assess rut resistance.

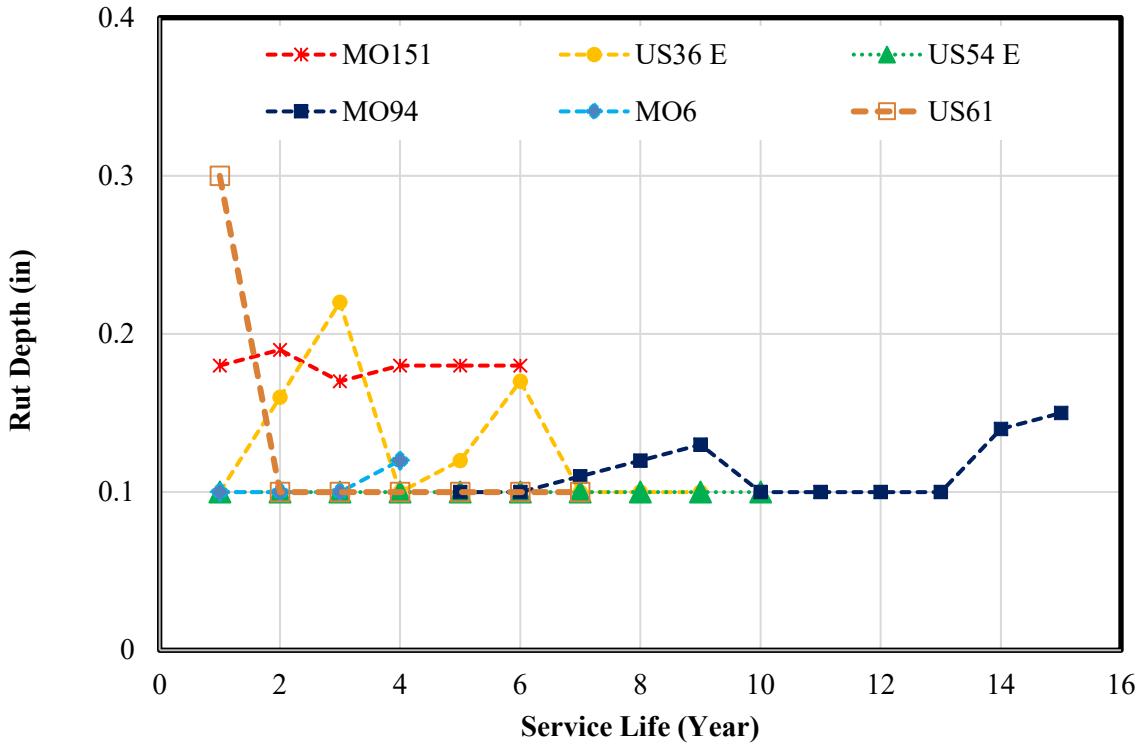


Figure 5-14. Rut depth as a function of service life

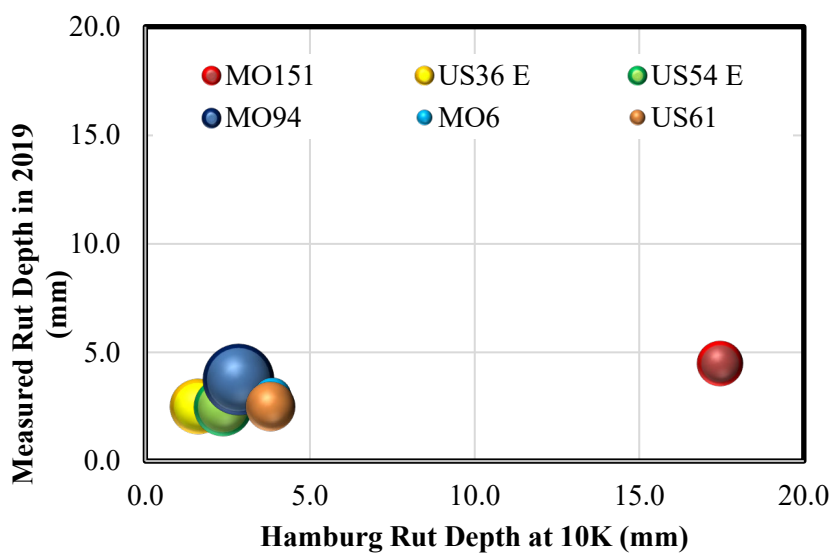


Figure 5-15. Relation between field and Hamburg rutting

Similar to the DC(T) and I-FIT tests, the IDEAL-CT test can be conducted to evaluate the cracking resistance of asphalt mixtures. The IDEAL-CT test benefits from a simple test sample fabrication protocol and does not involve notching or coring. Therefore, the gyratory compacted specimen at an appropriate height (62 mm) can be directly used for conducting the test without further sample fabrication process. The IDEAL-CT parameter obtained from this test has been

found to have a good correlation with FI parameter from I-FIT testing. Figure 5-16 shows I-FIT and IDEAL CT test results from 36 different mixtures, including both SMA and dense-graded mixes. With an R^2 of 0.87, the IDEAL-CT and FI tests are considered highly correlated in the log-log domain according to a simple power law as shown on Figure 5-16. The equation shown can be used to predict the IDEAL-CT score from a known FI score. The equation can be easily inverted in order to convert from FI to CT score.

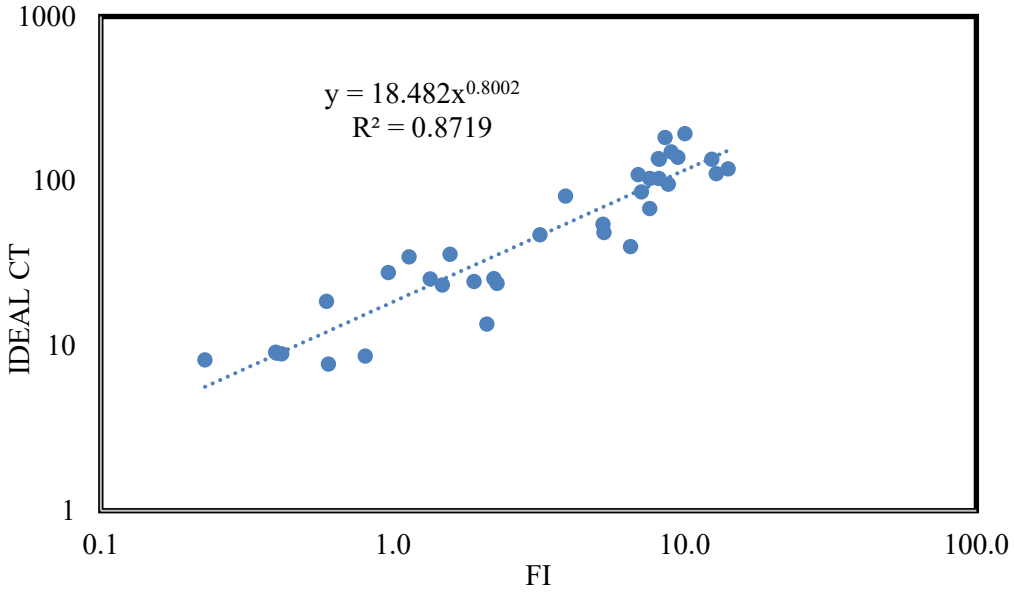


Figure 5-16. Correlation between FI and IDEAL-CT

Chapter 6

6. DEVELOPING INITIAL RECOMMENDATIONS IN SUPPORT OF IMPLEMENTING BALANCED MIX DESIGN IN MISSOURI

6.1. Brief Summary of Approach

As described in the literature review, a comprehensive BMD was recently developed by researchers at the University of Missouri-Columbia for the Illinois Tollway. A similar approach was taken herein, with the following approach:

- All three cracking tests investigated are assumed to be of interest to MoDOT and to Missouri contractors in terms of controlling the specific distress types and distress levels, and to maximize the types and amounts of recycled materials that can be used in asphalt mix designs. Test cost and practicality are also key considerations, which may necessitate the use of all three tests, at least initially, as BMD is rolled out in Missouri.
- A correlation between I-FIT FI and IDEAL CT was used to generate CT test thresholds based on field calibrated FI values, due to the lack of availability of field and lab CT data.
- Very few rutting and stripping failures have been observed in Missouri. Therefore, results from the calibrated Illinois Tollway specification were used to produce conservative rut depth and stripping inflection thresholds for Missouri to be used until further lab and field validation data is available.

The way in which various mixtures for mainline vs. shoulder mixes, and surface vs. non-surface mixtures are categorized in Missouri vs. Illinois differ significantly. Therefore, an approach was taken in an attempt to tailor the techniques developed for the Tollway for Missouri, and to avoid unnecessary proliferation of specification categories. To this end, the following processes and definitions were introduced:

- **High Criticality (HC)** mixes are defined as "B" mixes in Missouri highway specification, section 403, which are used for mainline paving on facilities with over 30 million design ESALs. Note: this study considers only non-SMA, Superpave "B" Mixtures. Additional research is needed to develop balanced mix design thresholds for SMA mixes in Missouri.
- **Medium Criticality (MC)** mixes are defined as "C" mixes in Missouri highway specification, section 403, which are used for mainline paving on facilities with between 3 and 30 million design ESALs.
- **Low Criticality (LC)** mixes are defined as "E" and "F" mixes in Missouri highway specification, section 403, which are used for mainline paving on facilities with fewer than 300,000 design ESALs. More specifically, "E" mixes are used for traffic levels between 300,000 and 3,000,000 ESALs, whereas "F" mixtures are used for traffic levels below 300,000 ESALs. We note that agencies have yet to widely adopt balanced mix design for lower traffic volume facilities. However, there appears to be a growing interest in extending the durability of low traffic volume asphalt overlays, and thus, recommended values are provided herein.

- **Surface mixtures (SM)** are self-explanatory, but the term **"Non-surface mixtures" (NSM)** was introduced to describe mixtures placed as lifts below the surface ("base" or "binder course" mixes).

6.2. Calibration of Cracking Test Specifications

Following the recommendation to include the DC(T), FI, and IDEAL-CT as the cracking tests in a prototype Missouri balanced mix design specification, Figure 6-1 presents the approach developed by the research team to calibrate the specification for various mix types. In the first box, a borderline threshold to serve as a baseline cracking test score was chosen for different mix types based on field performance observations and testing results obtained on field cores. In this study, the PASER deterioration rate was selected as the performance measure to establish baseline cracking test scores. As shown in Chapter 5, PASER deterioration rate had a higher correlation with test results as compared to IRI. PASER deterioration rates of 0.25, 0.3, and 0.4 decrease/year were selected for the high, medium, and low criticality levels, respectively. Figure 6-2 and Figure 6-3 depict the borderline values obtained for the DC(T) and FI, respectively, based on the proposed PASER deterioration rates for different project criticality levels.

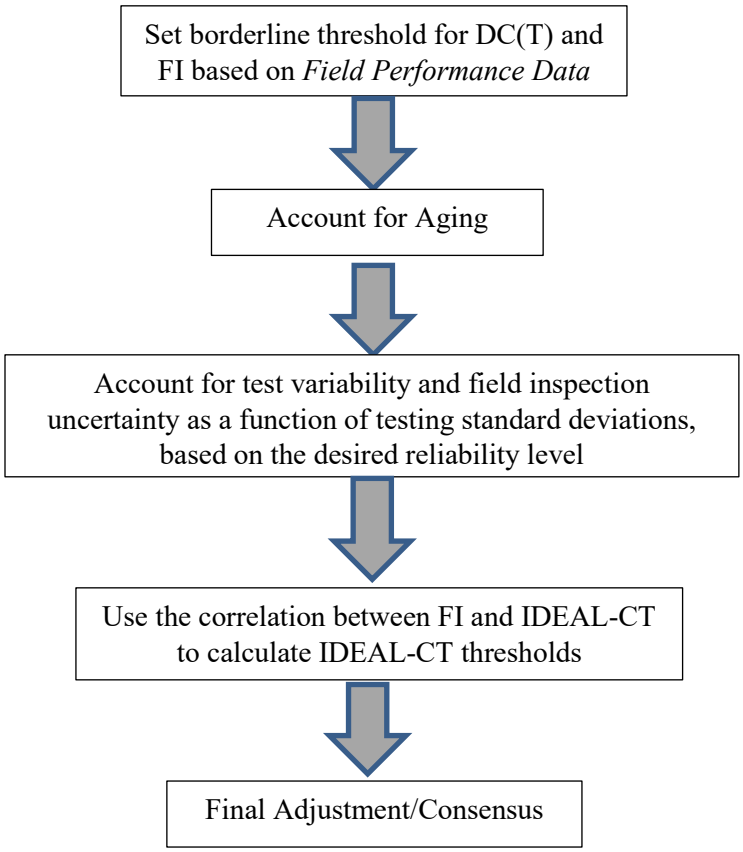


Figure 6-1. Steps in cracking test specification development

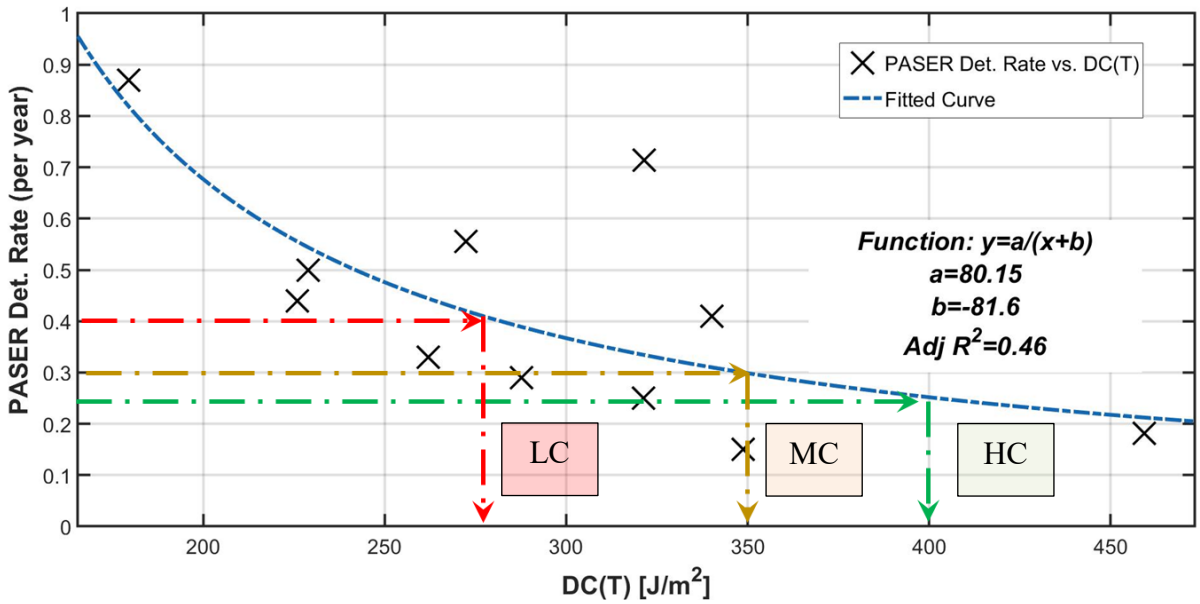


Figure 6-2. Setting baseline DC(T) scores based on PASER Det. Rate for various project criticality levels (HC = high criticality; MC = medium criticality; LC = low criticality)

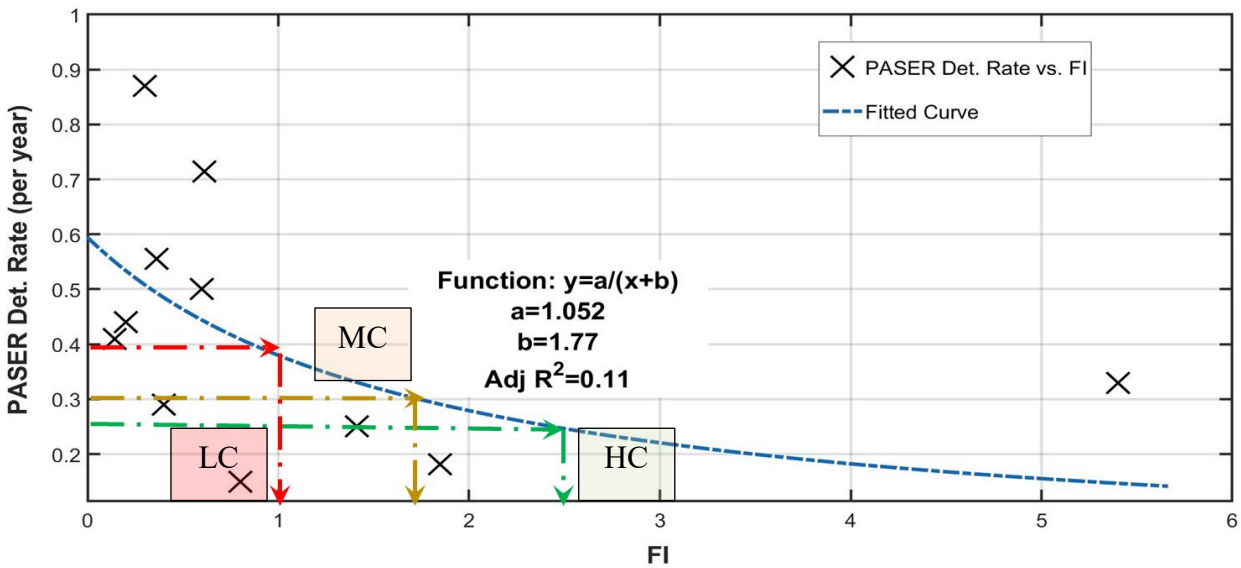


Figure 6-3. Setting baseline FI scores based on PASER Det. Rate for various project criticality levels (HC = high criticality; MC = medium criticality; LC = low criticality)

The core samples tested and used to define borderline test values have, in most cases, experienced long-term aging in the field. However, in order to circumvent the impracticalities associated with requiring long-term aging of mixtures in the lab before mix design, the effect of aging can be calibrated into the specification limits for tests carried out on short-term aged, lab-produced specimens. In the second box shown on the flowchart in Figure 6-1, standard deviations associated with experimental testing were used to account for test variability as a means to instill a specified degree of reliability into the thresholds. The reliability approach

developed was also intended to cover the uncertainties associated with field performance data collection and evaluation, and variabilities associated with test scores. To consider the IDEAL-CT as the third cracking test in the spec, the correlation between this test and I-FIT was implemented and the corresponding values for IDEAL-CT for different categories were determined. Finally, comments and recommendations from experts serving on the project were used, as shown in the last box (consensus step), to capture practical limitations which, for instance, can help avoid high bid prices for certain mix types based on limitations in locally available materials with respect to reaching certain DC(T) thresholds for certain mix types. Consensus adjustment can also create more uniform and logical spreads between thresholds assigned to various mix types in the specification.

Table 6-1 provides the parameters used for different mix categories (criticalities) that were in turn used to develop the specification thresholds for the 3 cracking tests. As the criticality of the mixture decreases, a higher deterioration rate is selected, leading to a more relaxed requirement for these mix types. The effect of aging (second box in Figure 6-1) is accounted for by raising the baseline value by a set percentage, based on prior research results, for each test and mix category. For example, for the HC-Surface category, 15% of the magnitude of the borderline test value (15%*400 J/m^2=60 J/m^2) is added to the baseline value to account for the average effect of long-term field aging on DC(T) fracture energy. It should be noted that a higher percent 'bump' is used for surface mixtures for both DC(T) and I-FIT testing, as mixes in this category will be exposed to the highest degree of sunlight, oxygen and other environmental effects influencing aging. Also, a higher reliability level associated with test repeatability and variability in field performance data is selected for higher project criticality categories. The reliability levels of 95, 87, and 68% correspond to the addition of 2, 1.5, and 1 test standard deviation(s), respectively, to the score as indicated in the third box of Figure 6-1.

Figure 6-4 to Figure 6-10 illustrate the process and values used to arrive at recommended cracking test thresholds for the DC(T) and I-FIT tests. Note that for more highly critical paving applications, lower PASER deterioration rates and higher standard deviation multipliers were used, which lead to higher required performance test scores and higher design reliability. A COV of 10 and 20% was selected for DC(T) fracture energy variance for plant mixtures and field cores, respectively. The average of these two COVs was then used as the standard deviation in the upward bump in required cracking index scores to ensure an added level of reliability. Based upon repeatability measures for the I-FIT, as established in previous chapters, 15 and 25% COV levels were used for plant mixtures and field cores, respectively. In order to reduce the performance requirements in non-surface course layers, and as described in Buttlar et al., 2020, one must account for the fact that the thermal stress level induced by the environment is lower in the lower lifts as compared to the surface lift. Assuming a temperature difference of 3 °C between the surface lift and bottom lifts, a 30% drop in the stress level is estimated. Therefore, 70% of the surface lift baseline cracking test score is taken as the baseline value for the non-surface mixtures. Lower aging adjustment factors were also used, as shown in Table 6-1.

Table 6-1. Summary of parameters used to calibrate cracking test specification

Parameter	High Criticality (HC)	Medium Criticality (MC)	Low Criticality (LC)
PASER Det. Rate	0.25 /year	0.3 /year	0.4 /year
Reliability	95%	87%	68%
DC(T) baseline value	400 J/m^2	350 J/m^2	280 J/m^2
Aging effect on DC(T), Surface	15% for all categories		
Aging effect on DC(T), Non-surface	10% for all categories		
FI baseline score	2.5	1.6	0.8
Aging effect on FI, Surface	400% for all categories		
Aging effect on FI, Non-surface	300% for all categories		

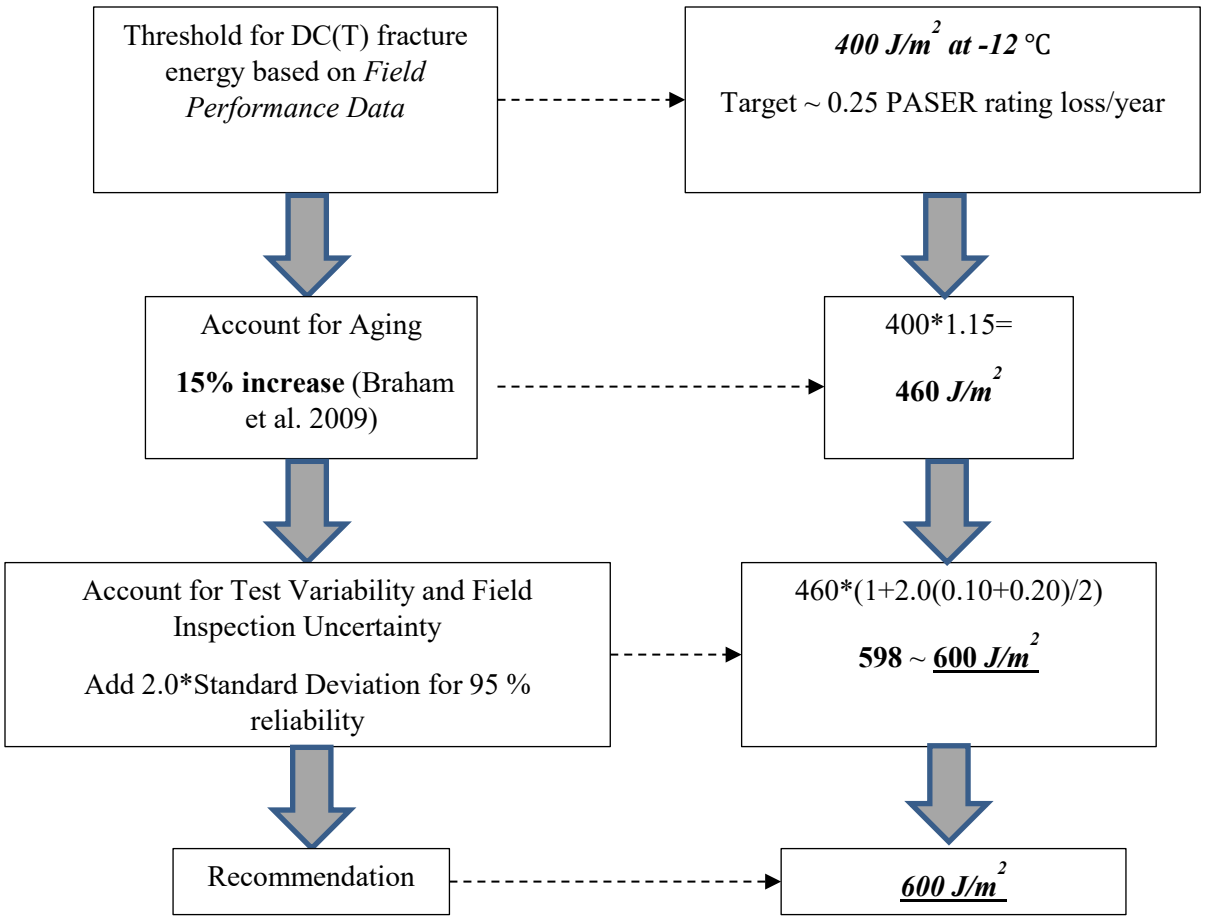


Figure 6-4. Flowchart to calibrate DC(T) threshold for High Cracking Criticality (HC) applications of Surface Lift (SL) mainline mixtures

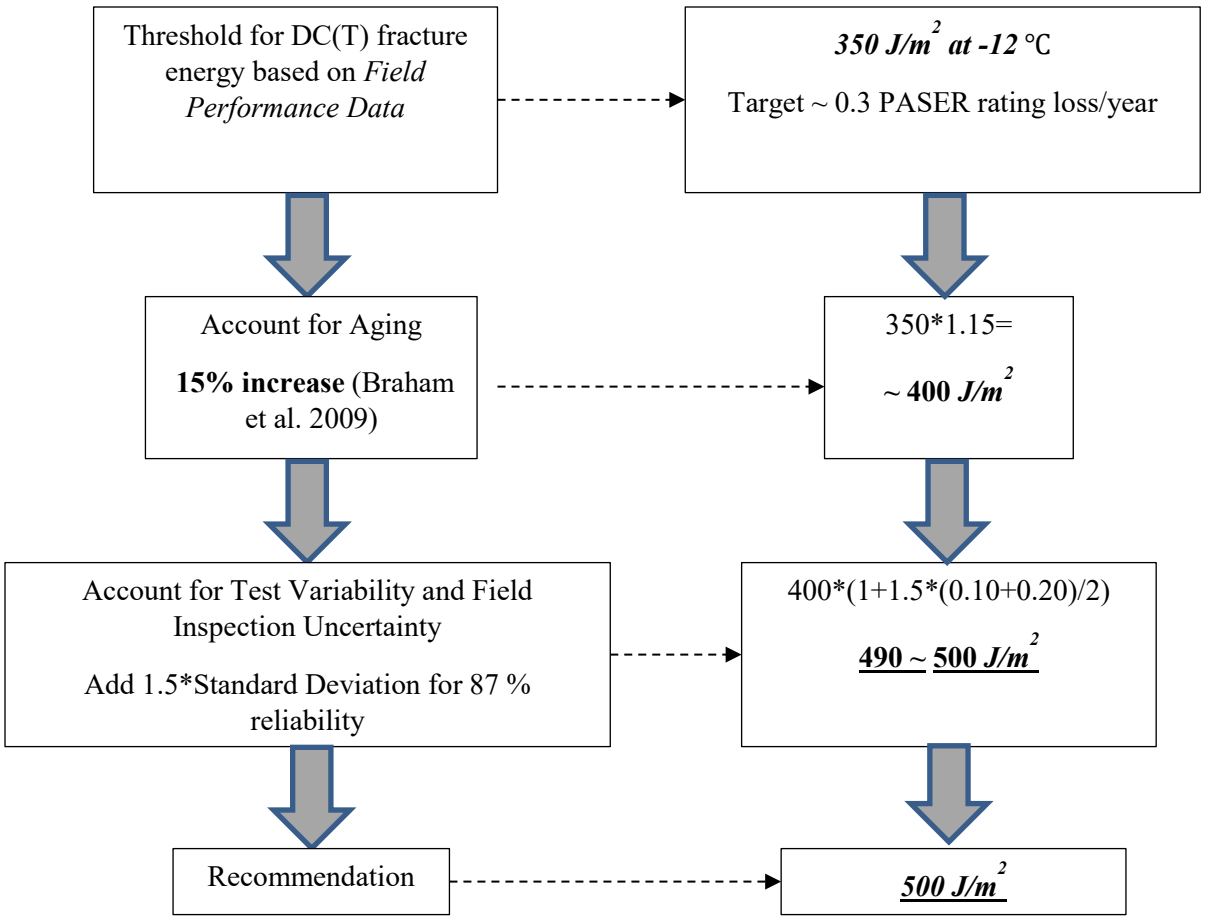


Figure 6-5. Flowchart to calibrate DC(T) threshold for Medium Cracking Criticality (MC) applications of Surface Lift (SL) mainline mixtures

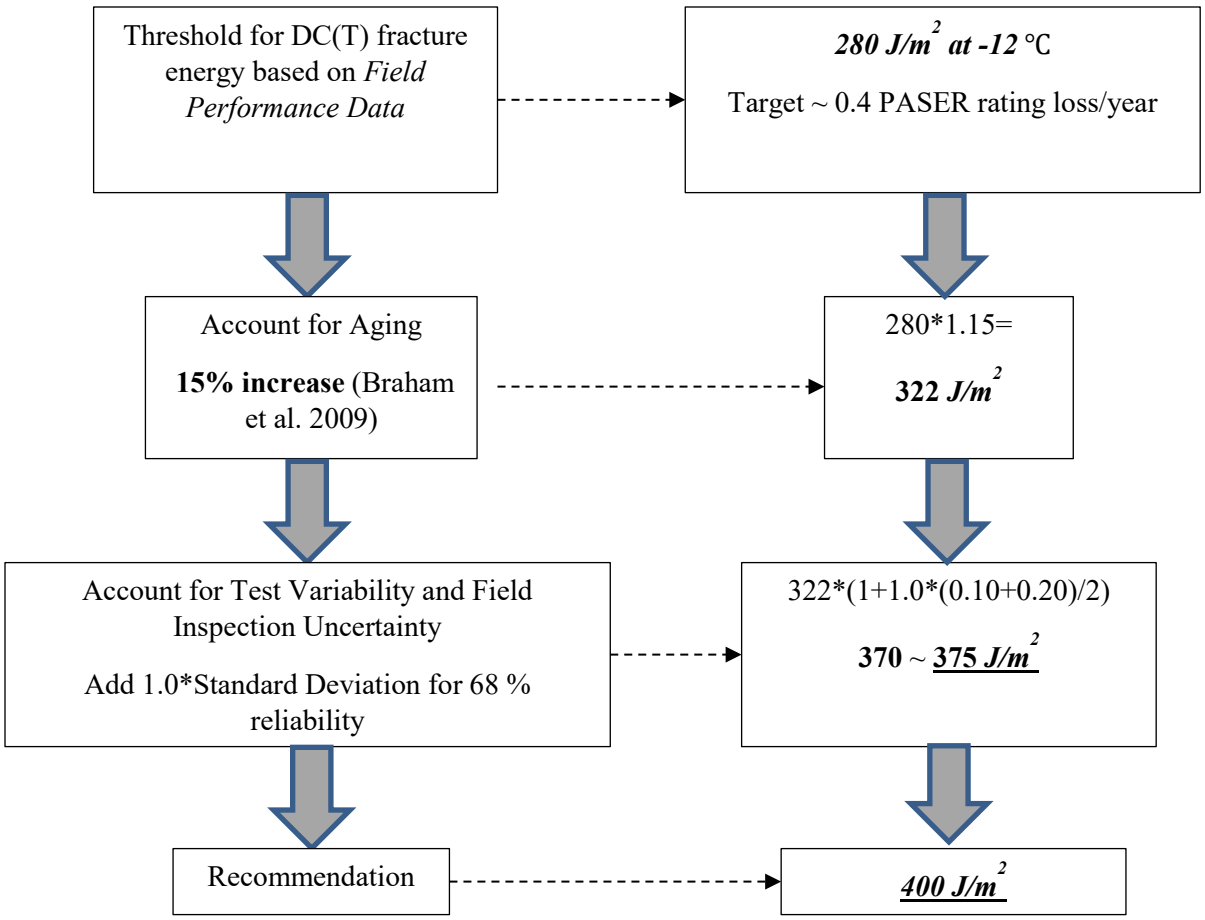


Figure 6-6. Flowchart to calibrate DC(T) threshold for Low Cracking Criticality (LC) applications of Surface Lift (SL) mainline mixtures

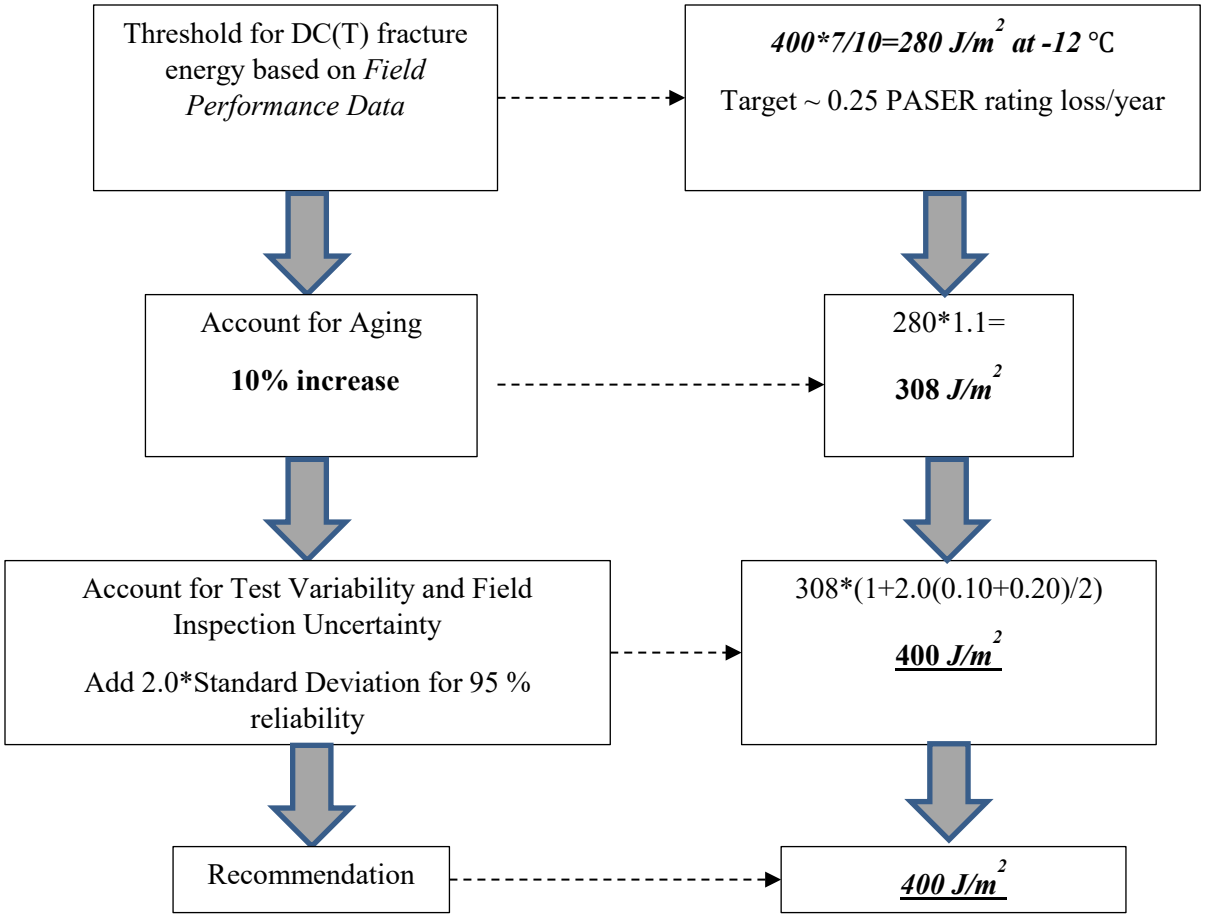


Figure 6-7. Flowchart to calibrate DC(T) threshold for High Cracking Criticality (HC) applications of Non-Surface Lift (NSL) mainline mixtures

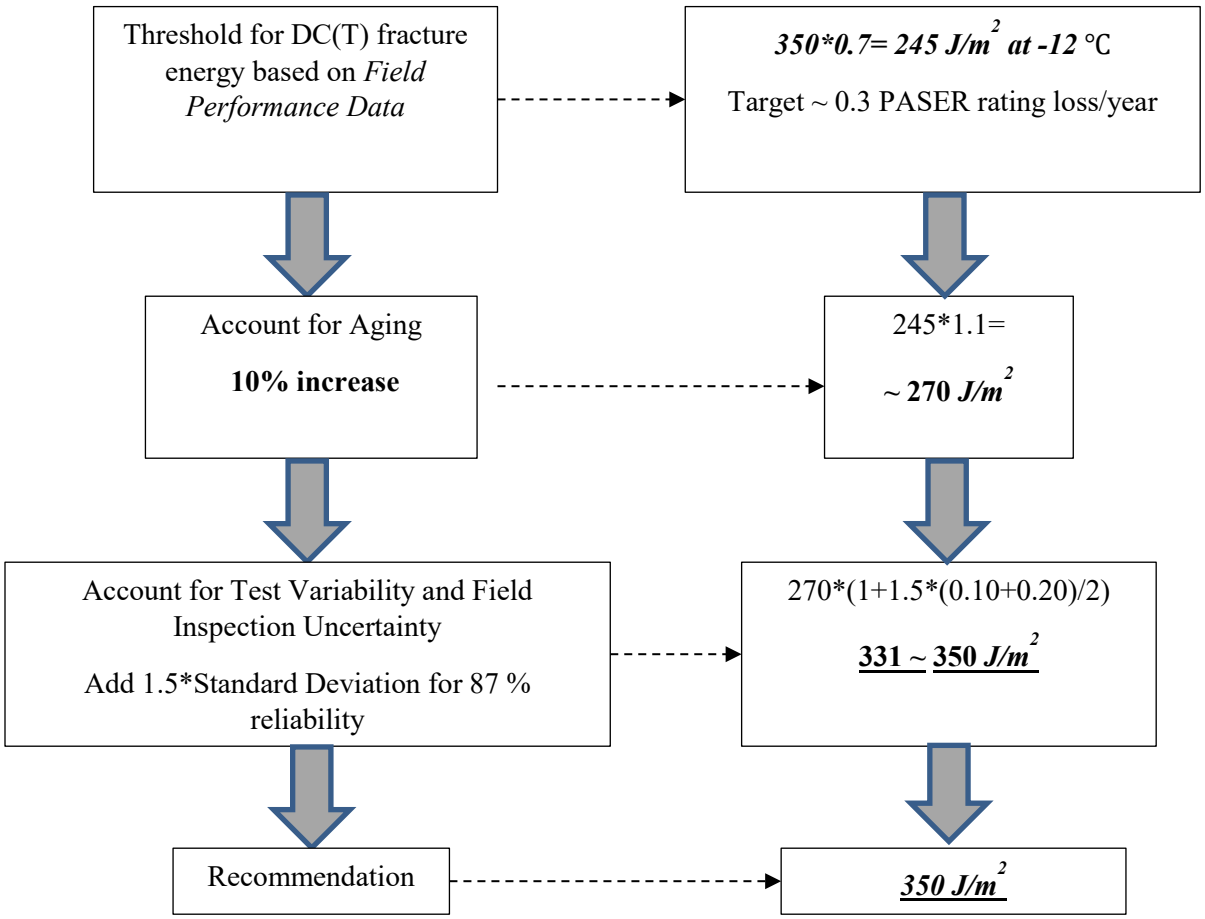


Figure 6-8. Flowchart to calibrate DC(T) threshold for Medium Cracking Criticality (MC) applications of Non-Surface Lift (NSL) mainline mixtures

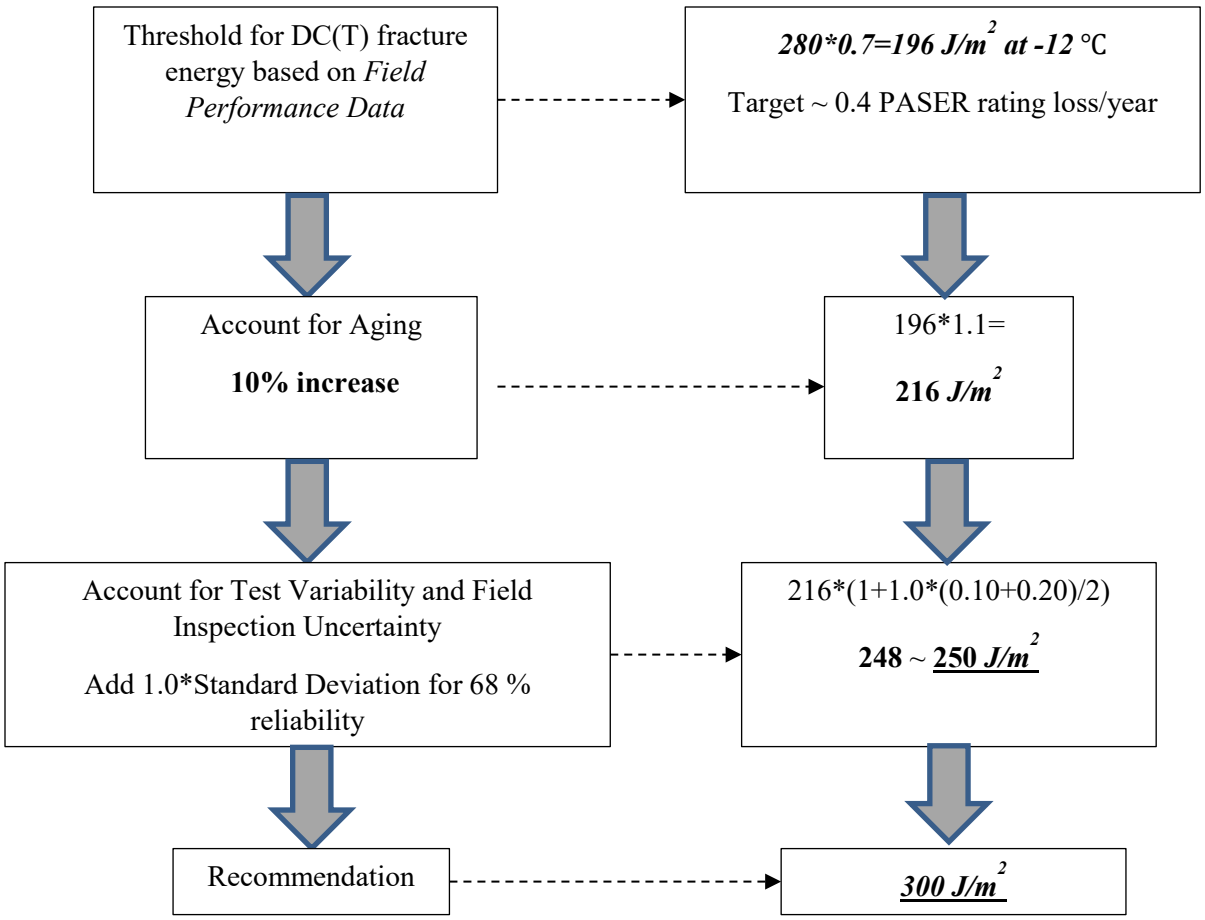


Figure 6-9. Flowchart to calibrate DC(T) threshold for Low Cracking Criticality (LC) applications of Non-Surface Lift (NSL) mainline mixtures

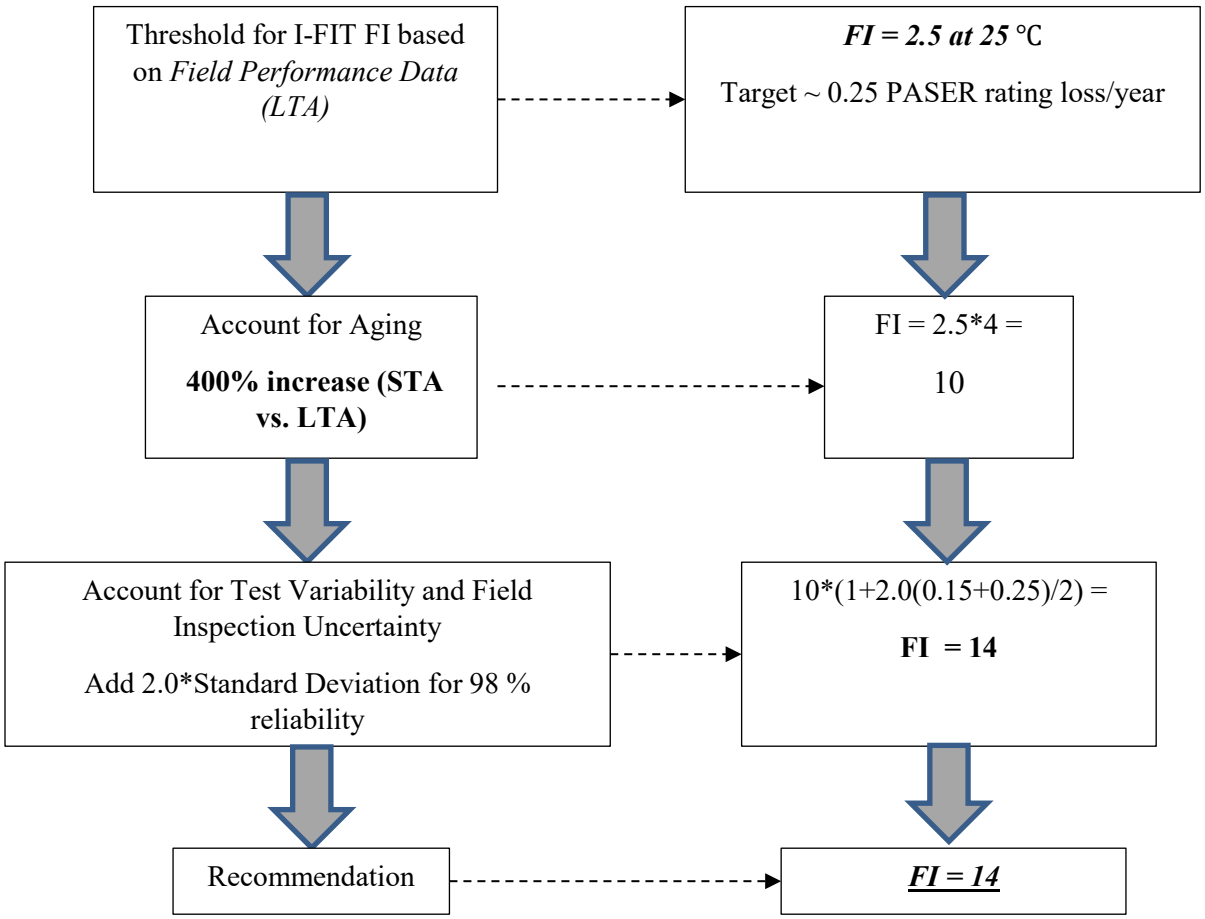


Figure 6-10. Flowchart to calibrate FI threshold for High Cracking Criticality (HC) applications of Surface Lift (SL) mainline mixtures

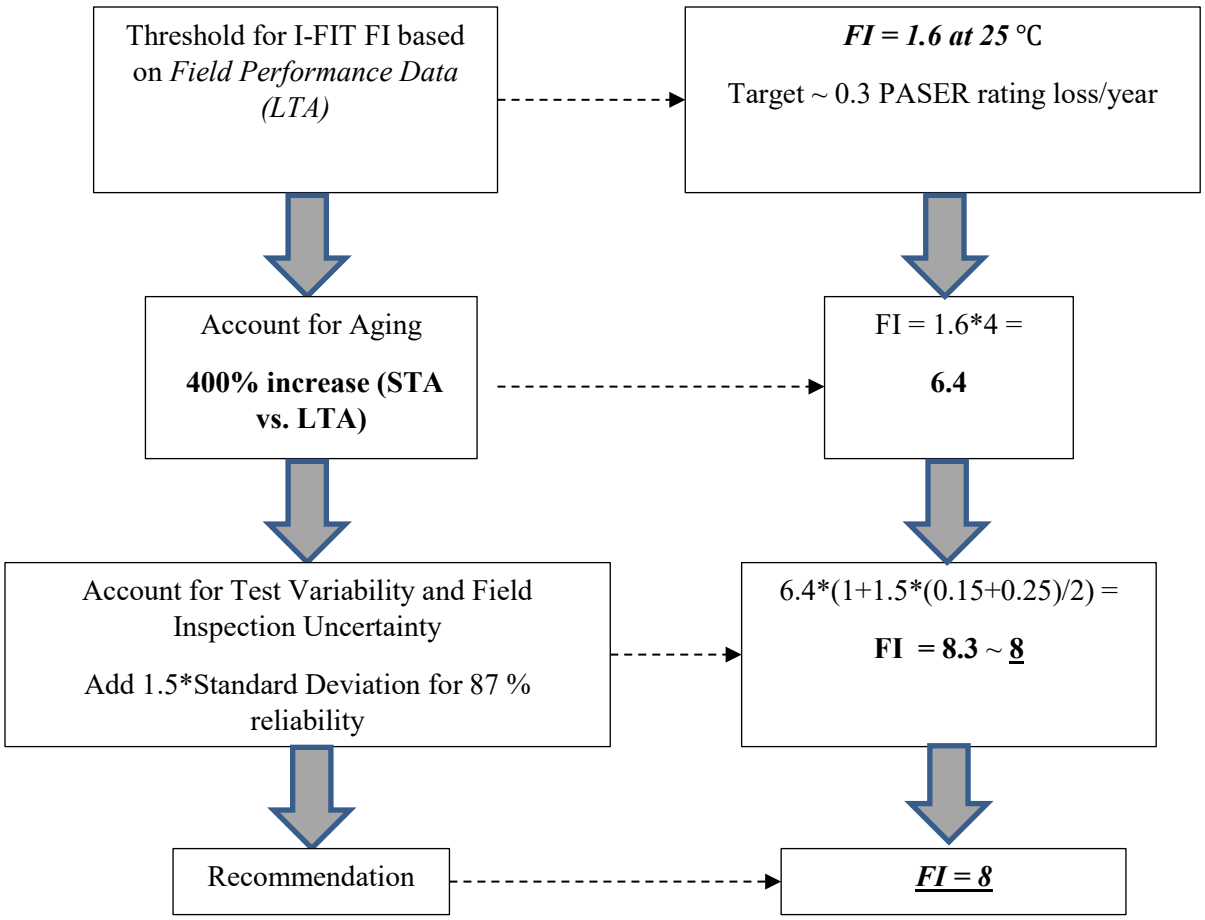


Figure 6-11. Flowchart to calibrate FI threshold for Medium Cracking Criticality (MC) applications of Surface Lift (SL) mainline mixtures

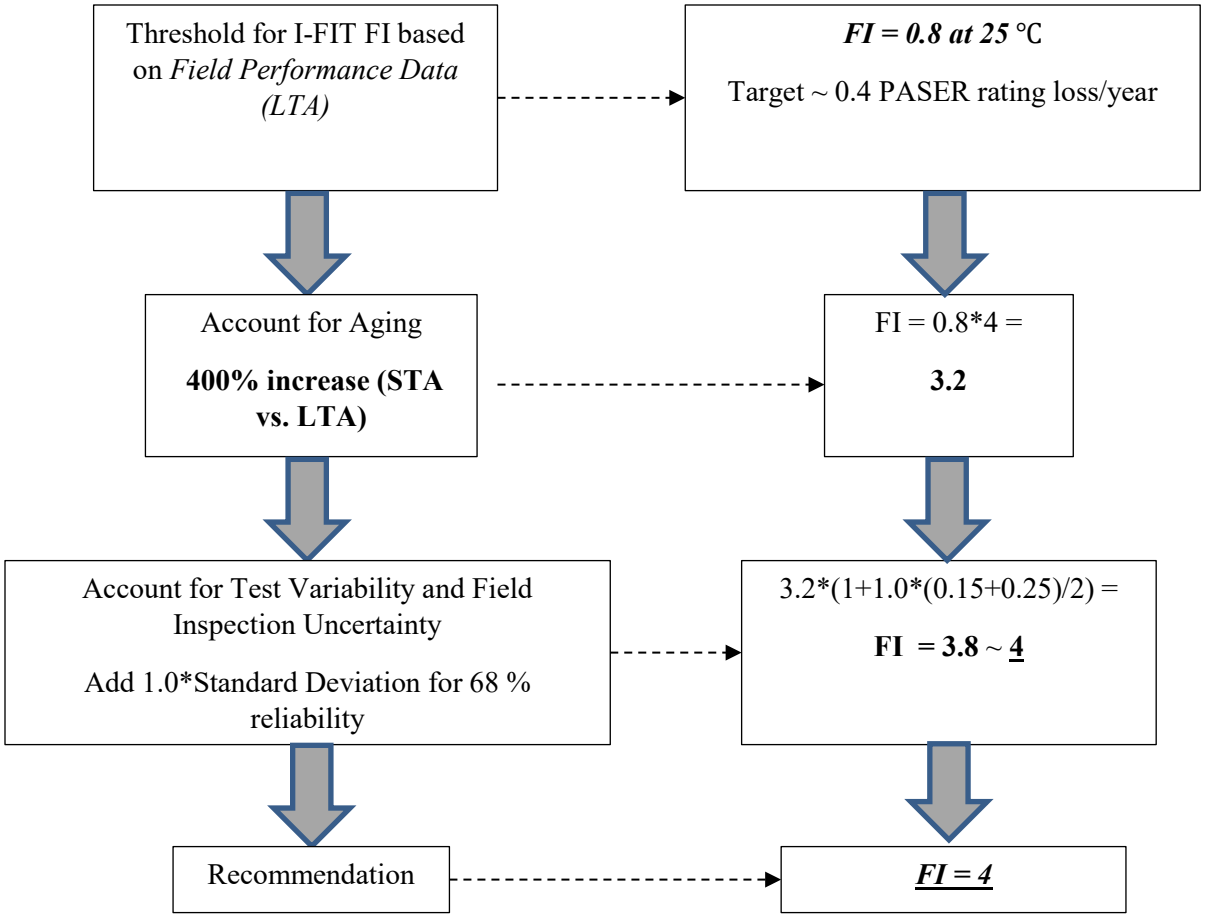


Figure 6-12. Flowchart to calibrate FI threshold for Low Cracking Criticality (LC) applications of Surface Lift (SL) mainline mixtures

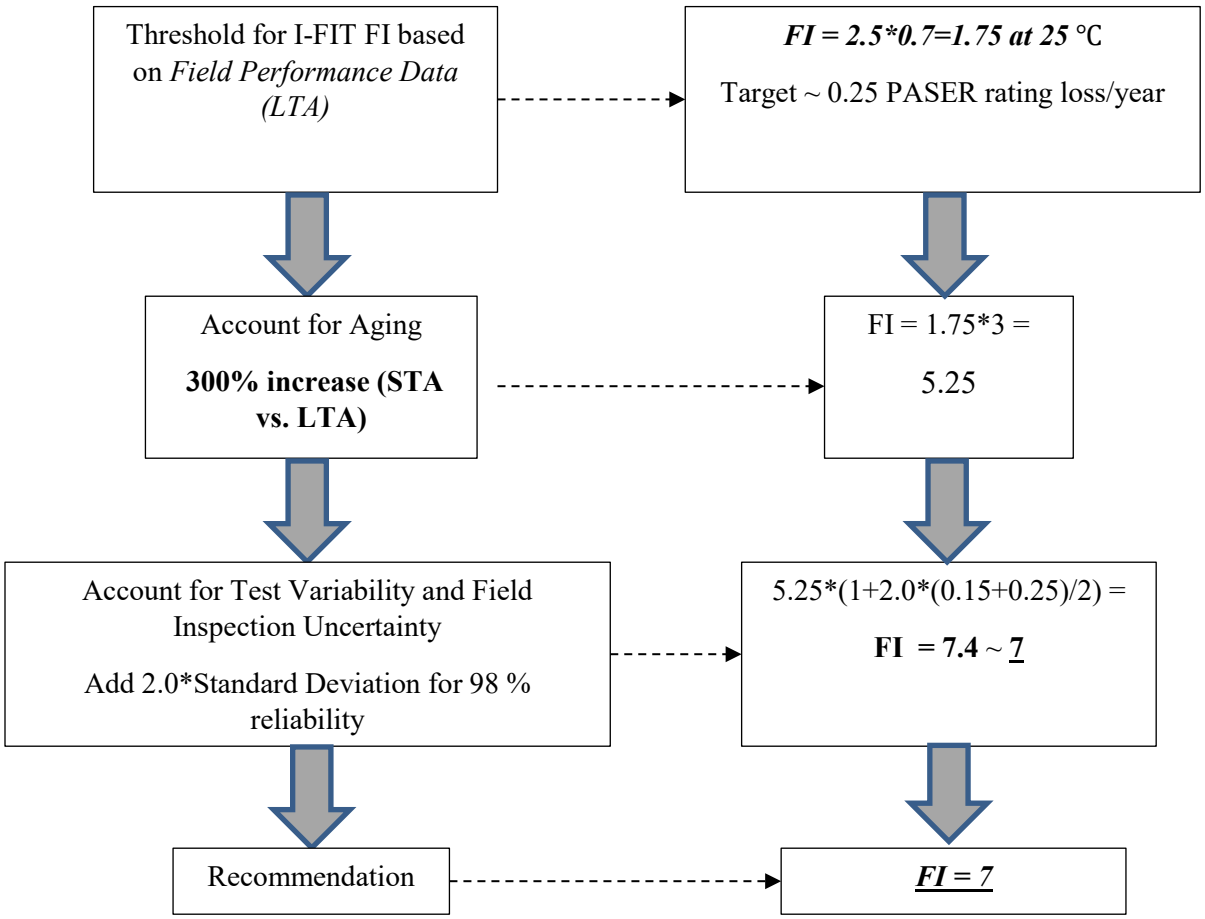


Figure 6-13. Flowchart to Calibrate FI threshold for High Cracking Criticality (HC) applications of Non-Surface Lift (NSL) mainline mixtures

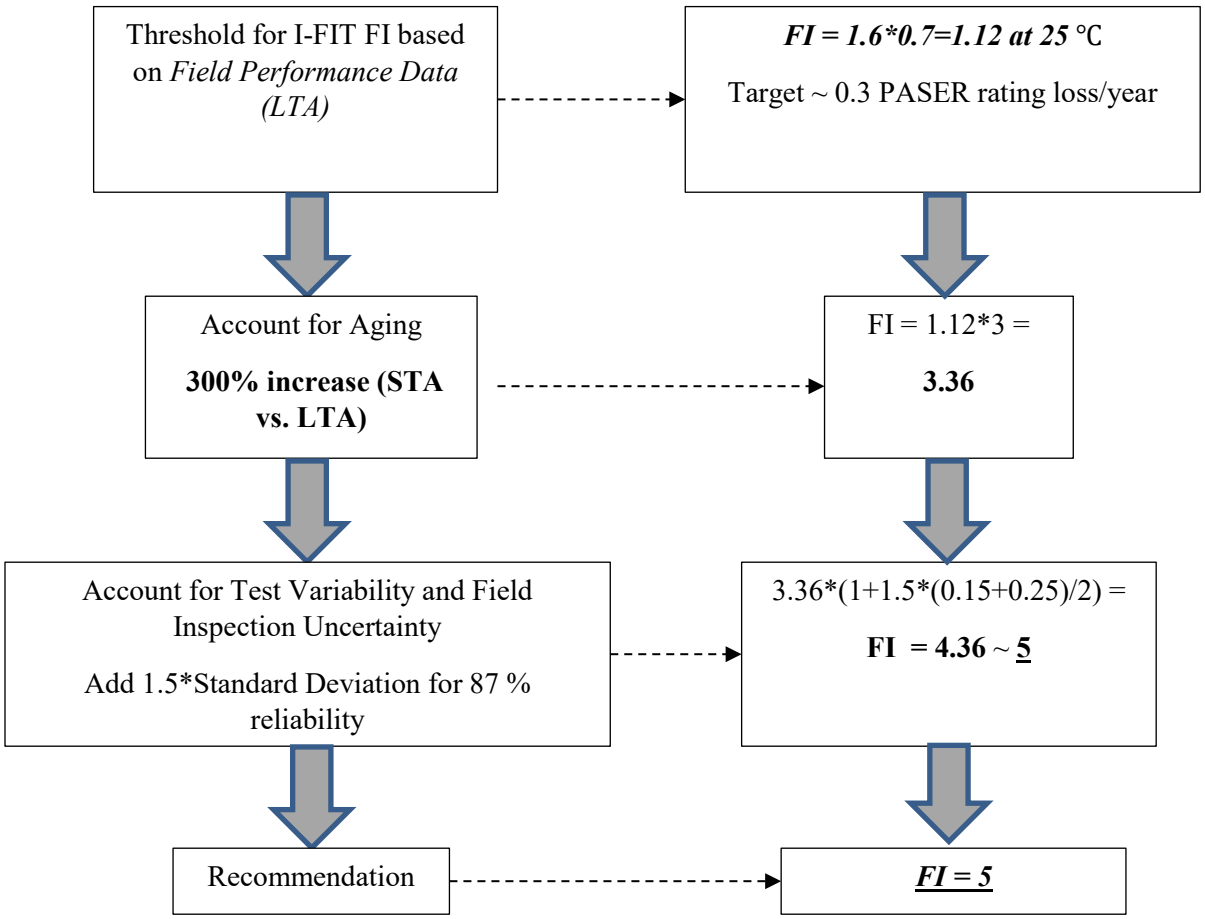


Figure 6-14. Flowchart to calibrate FI threshold for Medium Cracking Criticality (MC) applications of Non-Surface Lift (NSL) mainline mixtures

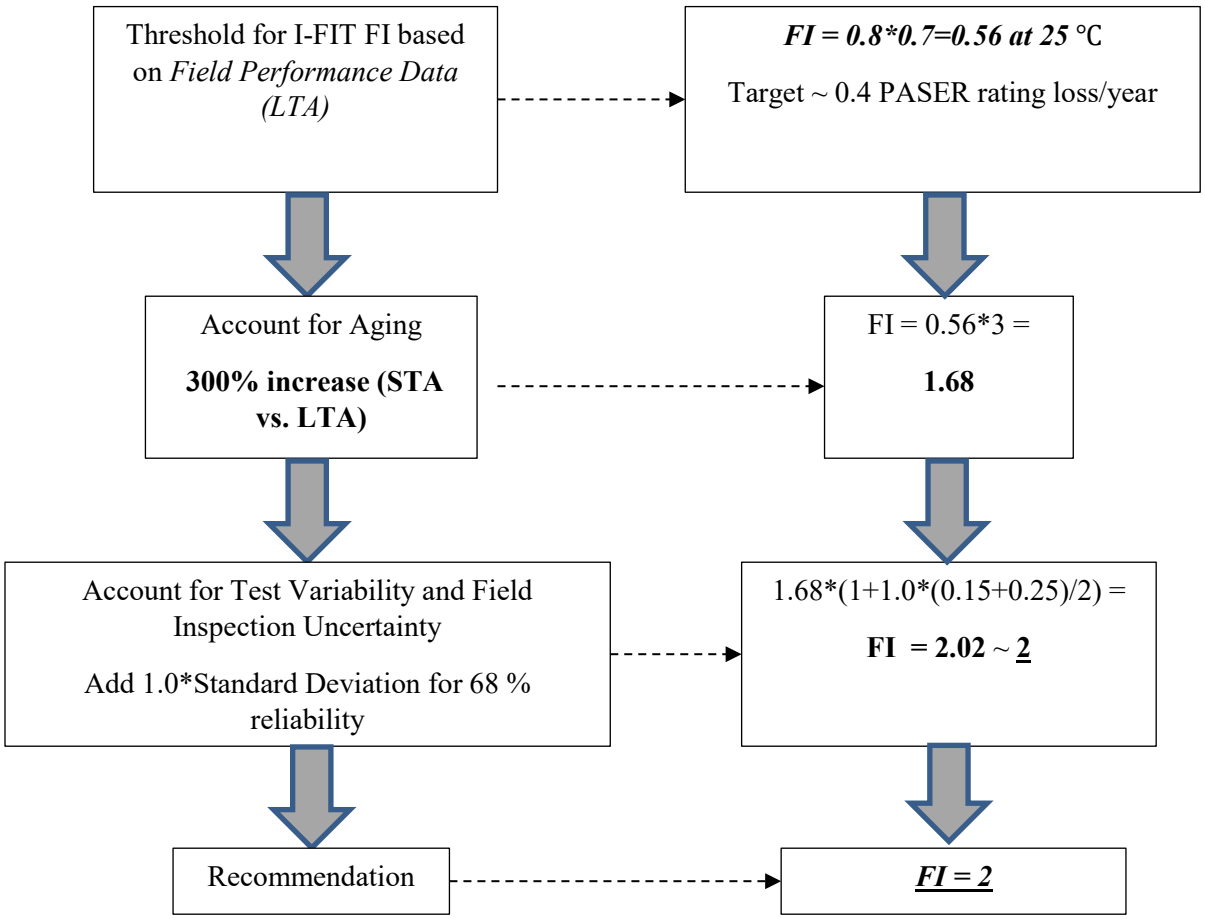


Figure 6-15. Flowchart to calibrate FI threshold for Low Cracking Criticality (LC) applications of Non-Surface Lift (NSL) mainline mixtures

6.3. Performance-based Specification Thresholds for Cracking and Rutting Tests

Table 6-2 and Table 6-3 present a summary of the recommended cracking test thresholds for mainline and shoulder mixes, respectively. Note that the IDEAL CT thresholds were obtained by applying the FI thresholds to the interconversion formula shown on Figure 5-16. Shoulder mix categories were then developed based on observations at the Illinois Tollway. Thus, they should be viewed as provisional and requiring further lab testing and field validation. In general, lower scores are required, as traffic intensities are very low on shoulders and ride quality is not as critical.

As mentioned earlier, Hamburg rut depth requirements and SIP thresholds were preliminarily developed, using test results from Missouri mixes, Missouri measured rut depths, along with field experience gathered on the Illinois tollway (Table 6-4 and Table 6-5). These recommendations are viewed as preliminary, and may be unnecessary at the current time for certain less critical mix categories, especially in shoulder mixes used in rural, low traffic areas.

Table 6-2. Recommended cracking test thresholds at -12 ℃ for mainline, non-SMA mixtures

Category	Position in Pavement	Criticality	DC(T) [J/m^2]	FI	IDEAL-CT
a	SL	H	600	14	150
b	SL	M	500	8	100
c	SL	L	400	4	55
d	NSL	H	450	7	100
e	NSL	M	350	5	70
f	NSL	L	300	2	35

Table 6-3. Recommended cracking test thresholds at -12 ℃ for shoulder mixtures (also non-SMA)

Category	Position in Pavement	Criticality	DC(T) [J/m^2]	FI	IDEAL-CT
a	SL	H	500	8	100
b	SL	M	400	4	55
c	SL	L	325	3	45
d	NSL	H	350	5	70
e	NSL	M	300	2	35
f	NSL	L	250	1	20

Table 6-4. Recommended rutting and stripping test thresholds at 50 ℃ for mainline, non-SMA mixtures

Category	Position in Pavement	Criticality	No. of Passes	Rut Depth (mm)	SIP
a	SL	H	20,000	6	15,000
b	SL	M	20,000	12.5	15,000
c	SL	L	15,000	12.5	10,000
d	NSL	H	15,000	12.5	10,000
e	NSL	M	10,000	12.5	7,500
f	NSL	L	7,500	12.5	5,000

Table 6-5. Recommended rutting and stripping test thresholds at 50 ℃ for shoulder mixtures (also non-SMA)

Category	Position in Pavement	Criticality	No. of Passes	Rut Depth (mm)	SIP
a	SL	H	15,000	12.5	10,000
b	SL	M	15,000	12.5	10,000
c	SL	L	10,000	12.5	7,500
d	NSL	H	10,000	12.5	7,500
e	NSL	M	7,500	12.5	5,000
f	NSL	L	5,000	12.5	N.R

Chapter 7

7. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

This project was conducted to assist MoDOT in the preliminary development and implementation of balanced mix design (BMD) for asphalt mixtures used in Missouri. A comprehensive lab and field testing program was carried out in support of this objective. Three cracking tests were evaluated, along with the Hamburg wheel tracking test for the control of rutting and stripping. A novel approach was taken whereby recommended specification thresholds were introduced for all three cracking tests currently under investigation in Missouri and across the Midwest, namely the DC(T), I-FIT and IDEAL tests. While the current study focused on mainline, high criticality mixes, preliminary recommendations for shoulder mixes and other low traffic paving applications are provided and may be of interest in larger urban areas and for municipal projects where closer control of asphalt performance via testing and materials investment is desired. SMA mixes were not considered in this study; however, the approach taken herein could be applied to SMAs in a straightforward manner once sufficient field and lab data is collected and analyzed.

7.1. Conclusions

Based on the work conducted herein, the following conclusions were drawn:

- Missouri is ready for preliminary roll out of BMD. University and MoDOT research, along with the comfort level demonstrated by Missouri contractors in discussions such as those held at MAPA-MoDOT joint meetings, all point to the fact that Missouri is ready to begin implementing BMD, starting with high-type, mainline mixes.
- All three cracking tests appear to be suitable for use in Missouri BMD, i.e., the DC(T), I-FIT and IDEAL. These tests have all been shown to relate laboratory testing results to field cracking performance, with the DC(T) designed to more closely control low temperature and block cracking, and the I-FIT and IDEAL tests designed to control top-down fatigue and reflective cracking. The DC(T) appears to be more inclusive of the use of higher amounts of recycled materials (RAP, RAS, GTR), and more repeatable, while the I-FIT and IDEAL tests appear to be more contractor friendly and appropriate for QC and QA as well as the design stage.
- The Missouri climate differs fairly significantly from the North-West to South-East corners of the state, which in turn affects pavement temperature at the surface, and at depth, ranging from 4 to 12 degrees Celsius depending on a number of variables. All cracking and rutting tests investigated are also fairly sensitive to test temperature. Thus, the BMD recommended herein should be considered as a starting point, calibrated to average conditions in the state.
- Using a reliability-based approach for rounding up minimally acceptable specification thresholds based on field observations and testing of core samples, it was possible to

develop a rational set of BMD recommendations for high, medium, and low criticality mixtures, based on design traffic. The thresholds developed are in reasonable agreement with values currently under consideration by MoDOT.

7.2. Recommendations

Based on the work conducted herein, the following recommendations are made:

- The recommended tests and thresholds provided herein should be viewed as preliminary, and can be adjusted based on practical considerations and stakeholder discussions in a consensus process.
- While the work conducted herein focused on mainline mixes for medium and high-traffic facilities, using Superpave dense-graded, the results can be easily extended and validated for SMAs and for low-volume roadways and shoulder mixes. This may be of interest in urban areas where rapid traffic growth is expected, or for municipal projects where an added degree of performance assurance is a desired value proposition to be used along with low-volume mix designs, such as MoDOT's bituminous pavement (BP) mixes or other municipal mixes used in the state of Missouri.
- Future studies should be conducted to establish specification thresholds for SMA mixes.
- Further validation of the proposed BMD tests and limits via long term field monitoring or testing on a controlled test road or test track facility is recommended. Advanced modeling is recommended to arrive at more accurate non-surface lift performance requirements, which may lead to mixture cost savings.
- Tailoring of the specification to varied climate zones in Missouri can be easily accomplished using the results provided herein, once the BMD begins to gain acceptance and goes through fine-tuning iterations after initial rollout.
- Additional testing of shoulder and low volume mixes, and identification of field sections with evidenced rutting and/or stripping should be pursued, to determine if the recommended Hamburg thresholds can be relaxed from the conservative starting points recommended herein.
- Additional work to apply the recommended BMD to quality control specifications and performance-related quality assurance specifications in a practical manner is also recommended. Attention to sample procurement and reheating should be included, as well as testing standardization, repeatability, and balancing of specification risk between parties.

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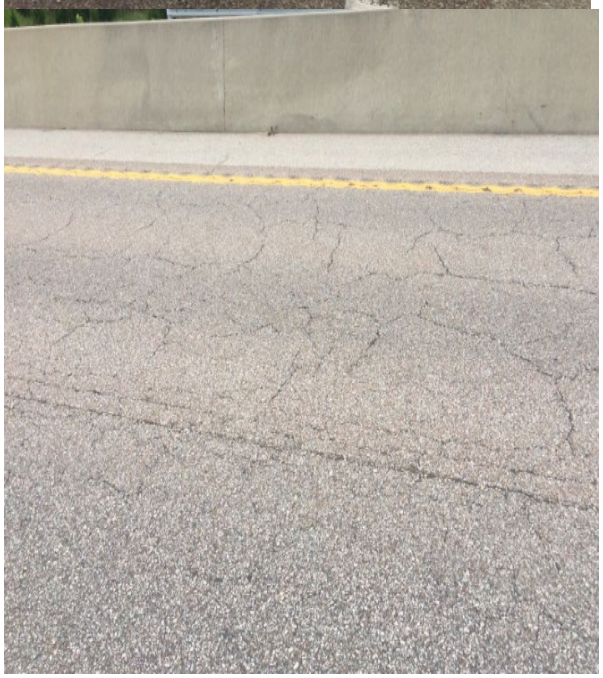
APPENDIX A

Field coring MO 151S
Coring date: Tuesday, July 16, 2019
Number of cores taken: 12
Location: Madison, Monroe County, MO

Field coring US 36E
Coring date: Tuesday, July 16, 2019
Number of cores taken: 12
Location: Shelbina, Shelby County, MO

Field coring US 54E
Coring date: Monday, July 15, 2019
Number of cores taken: 12
Location: Osage Beach, Camden County, MO



Field coring MO 94

Coring date: Thursday, July 18, 2019
Number of cores taken: 12
Location: St. Charles, St. Charles County, MO

Field coring MO 6W
Coring date: Wednesday, July 24, 2019
Number of cores taken: 12
Location: Lewis/Marion County, MO

Field coring US 61N
Coring date: Wednesday, July 24, 2019
Number of cores taken: 12
Location: Pike County, MO