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Nebraska Balanced Mix Design - Phase I

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16. Abstract

Balanced mix design (BMD) is an alternative concept for designing asphalt mixtures that mainly focuses on performance of mixtures rather than only volumetric analysis. Using this concept, it would be possible to account for the incorporation of recycled asphalt mixtures, warm technology, polymers, rejuvenators, and other foreign additives, as well as external effective factors on the mix design such as environmental effects. This project sought to investigate performance-based methodologies for the asphalt mix design by taking a step to develop a preliminary Nebraska BMD framework. With that, selection of appropriate performance tests, finding a functional laboratory aging protocol, and defining performance test criteria were the main long-term goals developed in this phase of study. To this end, three main types of distresses were taken into consideration (rutting, fatigue cracking, and moisture susceptibility), and a set of performance tests including well-established tests (Hamburg Wheel Track (HWT), Illinois Flexibility Index Test (I-FIT), Tensile Strength Ration (TSR)) and surrogate tests (IDEAL-RT, HT-IDT, G-stability, IDEAL-CT) were selected to capture these distresses on two types of high-performance commonly used asphalt mixtures in Nebraska (SLX and SPR). For the fatigue cracking analysis, long-term aging conditioning was conducted using two common aging protocols (NCHRP 09-54 and NCAT). Three types of data were utilized in this study including laboratory performance test results for the lab-compacted and field core specimens, as well as field data based on pavement surface condition monitoring. The validity of the surrogate performance tests was accomplished not only by correlating the field core results with field condition data, but also with correlating every individual surrogate test result to that of a well-established test. Further, the sensitivity, practicality, cost-effectiveness, and variability of different tests were assessed using statistical analysis and review efforts. In terms of rutting and fatigue cracking, IDEAL-RT and IDEAL-CT tests showed the highest correlation to well-established tests as well as significant sensitivity and accuracy in terms of results. For the moisture damage resistance tests, no strong correlation was found between well-established and surrogate tests, except for the G-stability test that showed some potential to be considered for future studies. In terms of long-term aging methods, the NCAT protocol was found to be more severe than NCHRP 09-54, however, selecting an appropriate long-term aging protocol for the Nebraska BMD will be done after long-term data analysis in the next phases of this study. Finally, an initial understanding of each test's pass/fail criteria was achieved based on the test result values obtained from historically acceptable asphalt mix design in the state.

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Chapter 1 Introduction

Roadways are one of the most critical parts of today's transportation system. They provide improved mobility for people, goods, and services. Asphalt-concrete (AC) mixtures were created and developed to fulfill the need to provide long-lasting pavements. The main goal when designing or improving an AC mixture is to find an economic blend of binder and aggregates resulting in a mix with sufficient stability and flexibility to withstand deformation and cracking under traffic loading [1].

The history of asphalt concrete mixture design began in the mid-1920s, when Charles Hubbard and Frederick Field introduced a method called the Hubbard Field (HF) Method of Design. In this method, the volumetric analysis (air void), as well as stability (compression test) were utilized to design an asphalt concrete. In 1927, the Hveem mix design method was developed with the philosophy that an asphalt binder was needed to satisfy aggregate absorption and at the same time have a minimum film thickness on the surface of aggregates [2]. Both stability and durability were considered in this method by applying the Hveem stabilometer test and swell test, respectively. As the stability was the primary focus of Hveem's mix design system, the Hveem pavements generally suffered from fatigue cracking due to lower asphalt contents [3].

The first widely adopted asphalt concrete mix design system was the Marshall method, officially introduced in 1943. This method was a refined version of the HF method with a similar approach, but different practice. In fact, Marshall standardized the compaction energy applied by using only one sized compactor hammer. The key components of this method were to determine the asphalt binder content based on stability and flow, while the air voids (V_a), voids filled with asphalt (VFA), and void in the mineral aggregates (VMA) were also taken into consideration. The shortcomings in the Marshall method were related to the sample compaction process, and aggregate and binder content selection. In contrast to field compaction that applied roller compactors, the Marshall method used a drop hammer and resulted in broken flat aggregates. Further, the climate and region-specific mixture design and its significant influence on mixture performance was not considered in the Marshall method. Finally, the Marshall method put less emphasis on aggregate gradation design, resulting in premature rutting and raveling of asphalt pavements [3].

In 1993, the Superpave method was introduced by the Strategic Highway Research Program (SHRP). The main objective of the Superpave method was to develop a performance-based mix design procedure. The Superpave mix design was developed in three levels (Level 1, Level 2, Level 3) with increased execution complexity as levels increased [4]. The current mix design practice (Level 1) was applied for traffic-based material design and introduced a load advanced asphalt binder selection to suit different climates. Further, some new procedures for mix design analysis and testing such as the Superpave Gyratory Compaction (SGC) method were developed by the Superpave mix design. The level 1 Superpave mix design procedure mostly involved proportioning the asphalt binder and aggregates by considering empirical properties of aggregates and volumetric properties such as VMA, VFA, V_a, and specific gravity. A new system for asphalt binder classification, Performance Grading (PG), was introduced by SHRP in the early 1990s. Level 2 and level 3 incorporated performance-based specification where several procedures were developed to predict and evaluate mixture performance; however, these levels were never implemented in any state Departments of Transportation (DOTs).

The main concern about the Superpave level 1 procedure and its commonly used asphalt mixture properties was this procedure cannot imitate the long-term performance of asphalt mixtures, according to most state DOTs' and asphalt contractors' perspective. In fact, the shortcoming of this method stemmed from its limitations in terms of accuracy and heavy reliance on volumetric characteristics of asphalt mixtures [5]. For instance, although some requirements were defined to set volumetric properties during the design process, their calculation was heavily dependent on measuring aggregate specific gravity, which affects the accuracy. There were many reports observing considerable issues of accuracy and variability during the measurement of specific gravity [6]. Furthermore, the incorporation of Reclaimed Asphalt Pavement (RAP) [7]–[9], along with some additive modifications such as adding rejuvenating agents [9]–[11], antioxidants [12], polymers [13], and fibers [14] could have further increased the complexity and inaccuracy of this method. To be more specific, using these new technologies could have highly affected the asphalt mixture performance while volumetric mixtures design methods were not capable of capturing all these effects. Consequently, two mixtures with almost identical volumetric characteristics may have shown a completely different performance in rutting or cracking properties [15].

To address all these problems, a performance-based asphalt concrete mixture design was proposed that attempted to evaluate the performance of asphalt mixtures at the time of design. This new method, called Balanced Mix Design (BMD), aimed to provide a balance between the cracking and rutting performances of asphalt mixtures. These two properties (i.e., rutting and fracture) often required opposing-characteristic form mixtures such that higher binder contents providing soft mixtures led to improved cracking resistance, while stiff mixtures with lower values of binder content resulted in better rutting resistance [16]. Hence, the BMD procedure involved varying the composition of asphalt concrete mixtures with/without considering volumetric characteristics, yielding a balance between these two core characteristics [17]. To include any mixture performance test in the BMD procedure, criteria for the test result must have been established based on a strong relationship to field performance and specific mixtures used in a particular state.

There were several potential performance tests, however, BMD was mostly focused on simple index tests to assess rutting, cracking, and moisture damage resistance of AC mixtures. There were also four BMD approaches already specified by AASHTO PP 105-20 (details in Section 2.1) to design asphalt mix: A) volumetric design with performance verification, B) volumetric design with performance optimization, C) performance-modified volumetric design, and D) performance design. All BMD approaches stipulated designers check and meet performance-based properties of the end product rather than only relying on volumetric properties. This end-product testing procedure not only allowed designers to be more innovative in terms of material selection, but also provided agencies with a more reliable way of accepting mixtures for a specific pavement section. As a result, it was highly important for all state DOTs to have an understanding about the best performance-related tests in terms of reliability, repeatability, and simplicity; as well as the most suitable approach to be used for the BMD implementation.

The Nebraska Department of Transportation (NDOT) had also started to investigate the feasibility of BMD implementation in the state's asphalt mix design. In this regard, two test methods, Semi-circular Bending (SCB) and G-stability tests, have been investigated for the appropriate testing conditions that can provide repeatable results [5]. The critical testing conditions and testing configurations have been explored and the optimum values that can aid repeatable results and practical implementation were found out. Also, the newly developed G-stability test results were validated by finding a good correlation between its test results to that of the established flow number (FN) test. As NDOT worked toward a BMD specification for mix design approval and a final mixture acceptance on paving projects, further information was needed regarding what protocol should be applied for implementing BMD on regular mixture design in the state (highly recycled mixtures with major binder modifications through using recycling agents and antioxidants). Also, it was necessary to find the appropriate performance-related tests to address rutting, cracking, and moisture damage resistance considering repeatability and simplicity of the results. Therefore, this research project was mainly focused on the development of BMD for the possible implementation in Nebraska.

1.1 Research Objectives

This study aims to examine the feasibility of implementing BMD in Nebraska by focusing on:

- Selecting appropriate performance tests that can address multiple modes of distress including rutting, mid-temperature (fatigue) cracking, and moisture induced damages,
- Feasibility evaluation of applying surrogate performance tests in lieu of well-established ones for the BMD in Nebraska, and
- Field evaluation on pavement sections to establish initial values for pass/fail criteria.

To achieve these objectives, a comprehensive experimental plan is developed to be performed on highly recycled asphalt mixtures with major binder modifications using recycling agents and antioxidants. The field evaluation is conducted by monitoring pavement surface conditions, as well as laboratory tests on field core samples from different projects. At the end of this project, it will be possible to specify the main performance test methods required for BMD implementation in Nebraska, as well as determine some initial criteria, and this study will pave the way for future research to design the final BMD framework.

1.2 Research Methodology

In this research project, the high-, and mid-temperature BMD performance tests performed in other states are considered and some of them are utilized on various Nebraska mixtures collected from field projects. In addition, moisture performance tests are also included in the Nebraska BMD. The Illinois Flexibility Index Test (I-FIT) and Indirect Tensile Asphalt Cracking Test (IDEAL-CT) are conducted to evaluate cracking resistance, while the Hamburg Wheel Tracking Test (HWTT), Gyratory Stability (G-stability) test, High Temperature Indirect Tensile (HT-IDT) test, and Indirect Tensile Asphalt Rutting (IDEAL-RT) test are performed to assess rutting resistance. Further, the Tensile Strength Ratio (TSR), HWTT, G-Stability, and HT-IDT tests are performed to estimate the moisture damage resistance of the mixtures. To have further insight on fatigue cracking resistance methods, Long-Term Aging (LTA) protocols are also considered while analyzing different asphalt mixture types. Two common LTA protocols are applied during sample fabrication and the results are used for fatigue cracking evaluation purposes and to compare the accuracy of different aging protocols.

Field evaluation is also conducted on all pavement sections by monitoring pavement surface conditions during their service lives. Field core samples are obtained from different projects during specific intervals and the same tests are conducted to find out the possible relationship between different types of data. Field data collection will continue annually as a long-term process, while the results will be used to establish initial pass/fail thresholds for future quality assurance and acceptance purposes. The plant-produced asphalt concrete mixtures applied in this study are the typical asphalt mixtures utilized in Nebraska on low-traffic volume roads, medium to high-traffic roads, and high-traffic roads such as the interstate. These asphalt mixtures contain recycled materials, recycling agents, and WMA additives, as such, the effects of these recycled materials and modifiers on pavement performance can be fully addressed in the Nebraska BMD, differing from the current Superpave volumetric mix design. The research method adopted in this study is depicted in Figure 1.1.

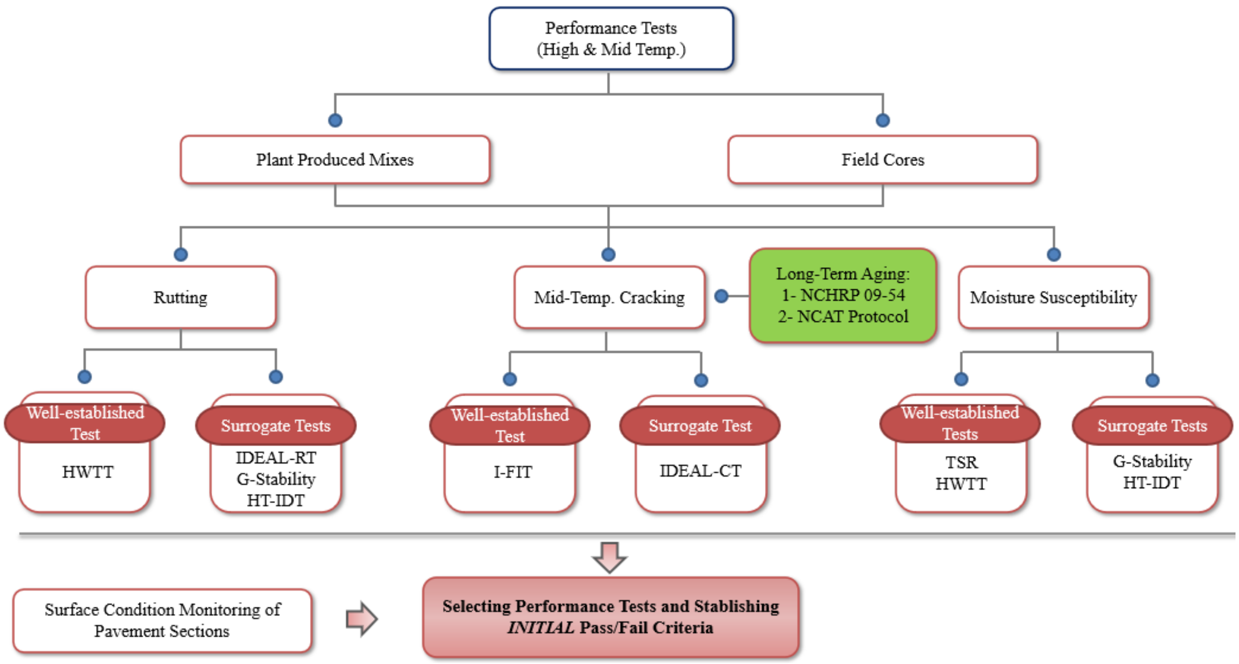


Figure 1.1 Experimental plan adopted in this study

1.3 Organization of the Report

This report is categorized into six chapters. Chapter 1 presents an introduction about the asphalt mix design history and motivations behind developing the BMD concept; it also mentions the current knowledge gap in the field, as well as objectives of this study. Subsequently, Chapter 2 presents a comprehensive literature review including but not limited to different BMD approaches and the current state of BMD application in the USA. Chapter 3 represents the exact methodology of this study by focusing on the materials, experimental plan, field evaluation, and protocol establishment. In Chapter 4 the laboratory test results and discussions are presented, while Chapter 5 is dedicated to statistical analysis and comparisons. Finally, Chapter 6 summarizes findings and conclusions from this research project.

Chapter 2 Literature Review

2.1 BMD Definition and Different Approaches

In September 2015, the Federal Highway Administration (FHWA) expert task group on mixtures and construction formed a BMD task force consisting of researchers, pavement engineers, and practitioners. The task force defined BMD as "asphalt mix design using performance tests on appropriately conditioned specimens that address multiple modes of distress taking into consideration mix aging, traffic, climate, and location within the pavement structure." Based on the BMD definition, beyond just relying on volumetric properties, an appropriate selection of performance tests was also utilized to achieve the optimal balance between rutting and cracking resistance. AASHTO PP 105-20 described four BMD approaches differing mainly on how strict they meet existing volumetric criteria and their innovative potential to meet the performance criteria. Each BMD approach is discussed in detail as follows.

2.1.1 Approach A: Volumetric Design with Performance Verification (VDPV)

The Volumetric Design with Performance Verification (VDPV) approach is the most commonly used approach and starts with determining the Optimum Binder Content (OBC) in a way that the existing volumetric requirements are met. Accordingly, one of the current volumetric mix design methods such as Superpave or Marshall are applied to define the OBC, although an existing agency-approved mix design method can also work. The yielded mix design at the OBC is then tested for different performance factors (e.g., rutting, cracking) using appropriate test methods. If the initial mix design failed to pass the performance requirements, the entire process must be repeated. This redesigning process can be fulfilled by using different materials (e.g., aggregates, asphalt binders, etc.) or different mix proportions, and the adjustments should continue until a point that all the volumetric and performance criteria are satisfied. The next step is evaluating the designed mixture in terms of moisture damage resistance using the appropriate test method. If the designed mix met the moisture test criterion, the Job Mix Formula (JMF) can be established; otherwise, some treatments may be required to reduce the moisture sensitivity of the designed mixture. It should be noted that using some anti-stripping agents for moisture resistant improvements can make it necessary to repeat some performance tests on the modified mixture. Specifically, some liquid anti-strip additives can increase the rutting potential of AC mixtures by softening the asphalt binder [6].

Consequently, the VDPV approach keeps the existing volumetric requirements, while additional performance requirements are enforced to meet the AC mixture design. This approach, as the most conservative BMD approach with the lowest innovation potential, at the time of reporting is being implemented by state DOTs in Illinois, Texas, Louisiana, New Jersey, and Wisconsin. The schematic of VDPV approach is depicted in Figure 2.1.

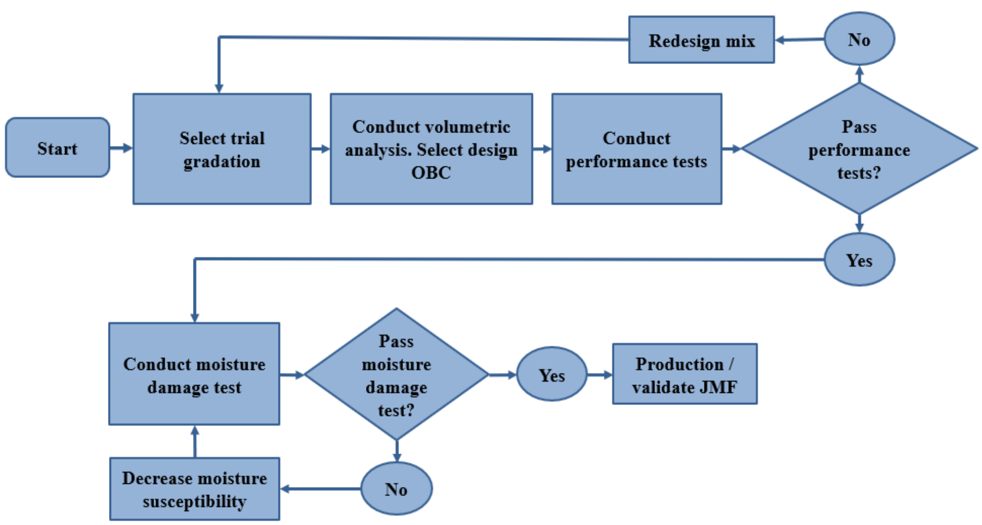


Figure 2.1 Schematic of Approach A: Volumetric Design with Performance Verification (VDPV)

2.1.2 Approach B: Volumetric Design with Performance Optimization (VDPO):

As an optimal version of Approach A, the Volumetric Design with Performance Optimization (VDPO) approach also uses volumetric specification (whether Superpave, Marshall, Hveem, etc. or alternatively an agency-approved mix design method) to define the preliminary OBC such that the volumetric requirements be satisfied. Subsequently, the mix designed with preliminary OBC along with two or more mixes produced with additional binder contents (± 0.3 to 0.5% of the OBC) are evaluated thorough rutting and cracking performance tests. Thus, the binder content that meets both performance criteria is selected as the final OBC. In cases where no binder content passes the criteria, the mixture is redesigned considering alternative components or proportional values for aggregates, asphalt binders, additives, or recycling materials. As the final OBC is achieved, the moisture damage assessment should be performed to define the JMF for production. Similar to approach A, any modification using anti-strip agents needs further assessment for performance verifications. In the VDPO approach the existing volumetric requirements are retained for the preliminary OBC selection, however, moderate changes in asphalt binder content are allowed for performance criteria satisfaction. Compared to Approach A, this approach appears to be more flexible for AC mixture design, however, it is still highly conservative with limited innovation potential. Figure 2.2 represents the schematic of Approach B:

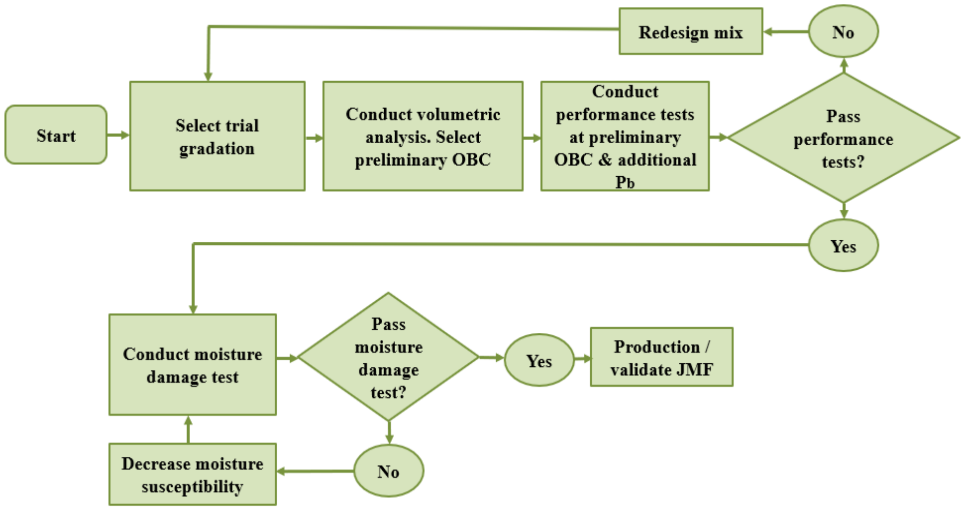


Figure 2.2 Schematic of Approach B: Volumetric Design with Performance Optimization (VDPO)

2.1.3 Approach C: Performance-Modified Volumetric Mix Design (PMVD)

In the Performance-Modified Volumetric Mix Design (PMVD) approach, the initial aggregate structure and binder content are determined by applying a volumetric mix design process (i.e., Superpave, Marshall, and Hveem) or an alternative agency-approved mix design method. Subsequently, performance tests are conducted to decide the appropriate adjustment of the mix proportions such that both rutting and cracking criteria be satisfied. Afterward, the mix design is evaluated in terms of moisture susceptibility using appropriate test methods. If any modification was applied to improve the moisture damage resistance, additional performance analysis is required on the modified mix for performance verification purposes. As the moisture criterion is passed, before establishing the production JMF, volumetric properties are measured again to verify the compliance with relaxed volumetric requirements. In the PMVD approach, the volumetric mix design requirements are not strictly enforced, and some of them may be removed provided the performance criteria are still satisfied. Compared to Approaches A and B, Approach C is less conservative with a higher degree of innovation potential for the mix design. A schematic of Approach C is indicated in Figure 2.3.

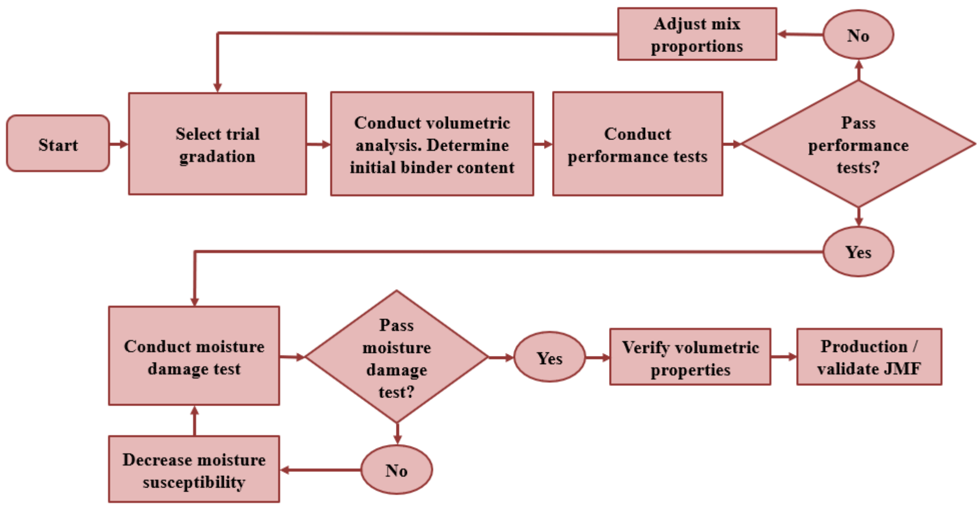


Figure 2.3 Schematic of Approach C: Performance-Modified Volumetric Mix Design (PMVD)

2.1.4 Approach D: Performance Design (PD)

In the Performance Design (PD) approach, existing agency-approved or trial mix designs are selected and evaluated for three or more binder contents (intervals of ± 0.3 to 0.5%) with various performance tests. The OBC is defined as the binder content that can satisfy both rutting and cracking criteria, whether in the first try or after adjusting mix components or proportions. Afterward, the moisture damage resistance of the mix design is assessed through appropriate test methods. The JMF can be established provided the moisture test criterion is passed, otherwise, anti-strip agents need to be added to a level where the moisture criterion is met, and re-evaluation of the modified mix design is necessary to ensure the performance requirements are all satisfied. In this approach the volumetric mix design is entirely skipped while criteria for rutting and cracking performance are added to the mix design. However, some volumetric properties (e.g., air void, void in mineral aggregates, etc.) are suggested to be measured at the end, so they can be utilized as a guideline for future mix design. The PD approach is considered the least conservative approach and has the highest degree of innovation potential. Figure 2.4 shows the schematic of Approach D.

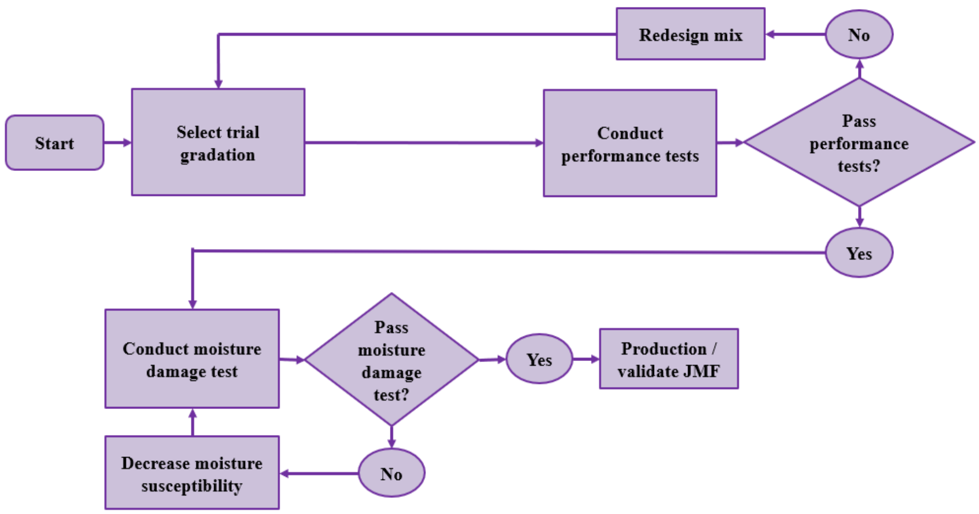


Figure 2.4 Schematic of Approach D: Performance Design (PD)

2.2 The Current Practice of BMD

The application of BMD is growing rapidly among various states by developing provisional specifications, drafts, or standard specifications. Considering the information derived from a survey of state highway agencies (SHAs) and asphalt pavement industry in May 2020 [18], 11 states have already identified a BMD specification as indicated in Figure 2.5. Illinois, Louisiana, New Jersey, Texas, and Vermont have chosen Approach A where asphalt mixture design is based on meeting both volumetric and performance requirements. Approach B has not been selected formally by any state, while California, Missouri, and Oklahoma currently apply Approach C by relaxing some volumetric criteria and meeting performance requirements. Approach D, performance design, is currently under investigation by Alabama and Tennessee, while Virginia allows the application of both Approach A and Approach D in the state.

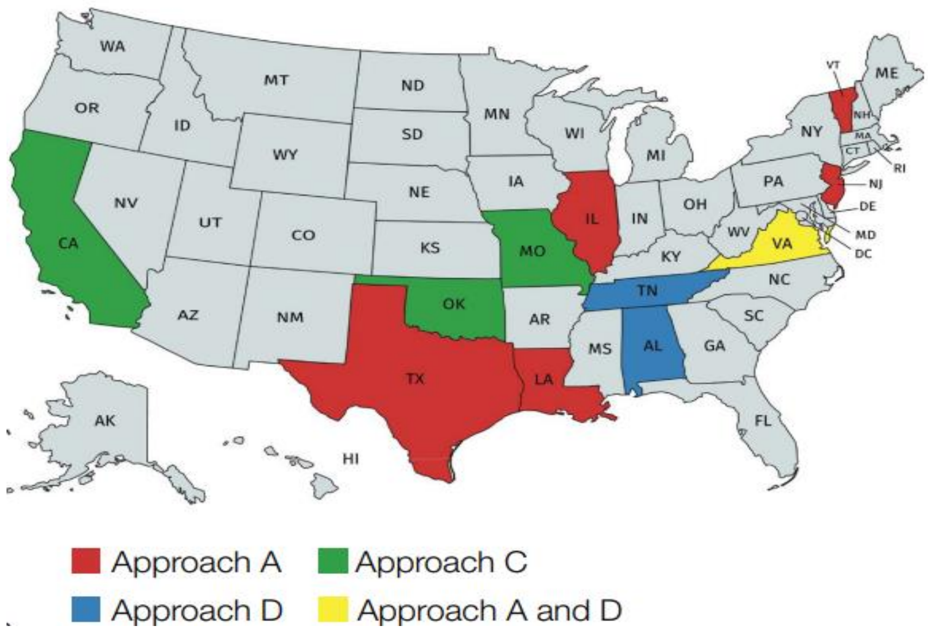


Figure 2.5 the U.S. current practice of BMD in different states [18]

Some additional information including applicable mixture type, specified performance tests, and performance criteria in different states are provided in Table 2.1.

Table 2.1 Summary of the current practice of BMD (Some parts from [18])

BMD Approach	State	Applicable mixture type	Distress	Test	Criteria
Approach A	Illinois	High ESAL mixture	Rutting	HWTT	<12.5 mm at 20000 passes (PG 76-xx)
			Cracking	I-FIT	FI>8.0 for HMA
	Louisiana	Wearing and binder course mixtures	Rutting	HWTT	<10 mm at 20000 passes
			Cracking	SCB-Jc	Jc > 0.5 KJ/m2
	New Jersey	Specialty mixtures	Rutting	APA	<4 mm at 8000 cycles
			Cracking	OT, BBF	Cycles to failure ≥ 600
	Texas	Surface mixtures	Rutting	HWTT	<12.5 mm at 20000 passes (PG 76-xx)
			Cracking	OT, IDEAL-CT	CFE > 1.0 in.-lb/in.2 CPR < 0.45
	Vermont	Superpave Type IVS mixtures	Rutting	HWTT	<10 mm at 20000 passes
			Cracking	I-FIT	FI≥10
Approach A&D	Virginia	Surface mixtures	Rutting	APA	<8 mm at 8000 cycles
			Cracking	Cantabro, IDEAL-CT	Cantabro mass loss < 7.5% $CT_{index} \geq 70.0$
Approach C	California	Long-life pavement mixtures	Rutting	FN, HWTT	Cycles at 3%strain ≥941 <12.5 mm at 20000 passes
			Cracking	BBF, I-FIT	FI≥3.0
	Missouri	Mainline pavement mixtures	Rutting	HWTT	<12.5 mm at 20000 passes (PG 64v-22)
			Cracking	I-FIT, IDEAL-CT	2.0<FI<8.0 32< CT_{index}< 97
	Oklahoma	Superpave mixtures	Rutting	HWTT	<12.5 mm at 20000 passes (PG 76-xx)
			Cracking	IDEAL-CT	$CT_{index} \geq 80.0$
Approach D	Alabama	Superpave mixtures	Rutting	HT-IDT	>20psi
			Cracking	IDEAL-CT	≥83 for 1<ESALs<10 ≥110 for 10<ESALs<30
	Tennessee	All mixtures	Rutting	HWTT	<12.5 mm at 20000 passes
			Cracking	IDEAL-CT	$CT_{index} \geq 100$

In addition to the states with a specified BMD approach, there are a number of states with ongoing research focused on embedding performance evaluations in asphalt concrete mixture design. To complete the BMD implementation, Arkansas DOT conducted a study to develop a cracking test along with the APA rutting method. In this regard, the I-FIT and IDEAL-CT tests were performed on both laboratory and field mixtures (NMAS of 9.5 and/or 12.5 mm). As the CT_{index} and FI values indicated similar patterns for the mixtures, the IDEAL-CT, as a simple test method, with a minimum CT_{index} of 50 was recommended to assess cracking damage resistance of Arkansas AC mixture design [19]. In a similar study conducted by Oklahoma DOT in 2019, the cracking resistance of typical mixtures was evaluated for the possible application in the BMD procedure. Accordingly, I-FIT and IDEAL-CT tests were performed on the existing mixtures while a weak correlation was found between FI and CT_{index}. To be more specific, it was mostly reported that a CT_{index} value of 80 is equivalent to an FI value of 8, however, the results of this study recommended CT_{index} values higher than 100. Although, the IDEAL-CT test was proposed to be used in Oklahoma's BMD due to its simplicity, it was also suggested that all the applicable mixtures in the ODOT projects be tested for the possible establishment of CT_{index} criteria [20].

In a project conducted by Utah Department of Transportation (UDOT), the cracking potential of asphalt mixtures was under investigation through the IDEAL-CT test. The project objective was to find the possible correlation between IDEAL-CT and I-FIT test results considering field performance, repeatability, and simplicity to finally recommend the most appropriate test for adoption by state agencies. As of this report, the project had not yet concluded [6]. In a pilot study conducted by Wisconsin DOT (WisDOT), an asphalt concrete mixture design considering performance tests was evaluated for mixtures with 25% or more recycled materials [21]. Accordingly, HWTT was required to assess moisture and rutting resistance, while DCT and

SCB-J_c tests were required to assess low-temperature and fatigue cracking resistances, respectively. In another research project sponsored by WisDOT, the impact of increasing asphalt content using a regressed air void approach was evaluated on the results derived from HWTT, DCT, and I-FIT tests. Test results on AC mixtures with different amounts of RAP and RAS designed for various traffic levels showed an improvement in cracking resistance without compromising the rutting resistance of AC mixtures [22].

2.3 Asphalt Mixture Performance Tests

Performance tests, as the key components in BMD implementation, are utilized to evaluate rutting resistance, cracking resistance, and moisture susceptibility of asphalt concrete mixtures. Selecting an appropriate test involves considering various factors such as availability of equipment and test standards, simplicity of the test, repeatability, accuracy, and variability. Accordingly, several tests have been developed toward performance assessment of asphalt concrete mixtures. A comprehensive list of performance tests considered by different state highway agencies is presented in Table 2.2.

Table 2.2 Commonly used asphalt mixture performance tests [6]

Mixture property	Laboratory Test	Test Standard	Test Parameter(s)
Rutting Resistance	Asphalt Pavement Analyzer (APA)	AASHTO T340	Rut Depth
	Flow Number (FN)	AASHTO T378	Flow Number
	Hamburg Wheel Tracking Test (HWTT)	AASHTO T324	Rut Depth
	AMPT Stress Sweep Test (SSR)	AASHTO TP 134	Rutting Shift Model, Index Parameter RSI
	High Temperature Indirect Tension (HT-IDT)	ALDOT 458	Indirect Tension Strength
	Rapid Shear Rutting Test (IDEAL-RT)	N/A	Rutting Tolerance Index (RT_{Index})

Table 2.2 Commonly used asphalt mixture performance tests (Continue) [6]

Mixture property	Laboratory Test	Test Standard	Test Parameter(s)
Cracking Resistance/Durability	AMPT Cyclic Fatigue Test	AASHTO TP 107 AASHTO TP 133	Damage Characteristic Curve & Fatigue Model, Index Parameter (S_{app})
	Disk-Shaped Compact Tension (DCT) Test	ASTM D7313	Fracture Energy
	Flexural Bending Beam Fatigue (BBF) Test	AASHTO T 321 ASTM D8273	Cycles to Failure Fatigue Equation
	Illinois Flexibility Index Test (I-FIT)	AASHTO TP 124	Flexibility Index (FI)
	Indirect Tensile Creep & Strength Test	AASHTO T 322	Creep Compliance & Tensile Strength
	Indirect Tensile Cracking Test (IDEAL-CT)	ASTM D 8225	Cracking Tolerance Index (CT_{Index})
	Indirect Tensile Energy Ratio (ER) Test	N/A	Dissipated Creep Strain Energy & Energy Ratio
	Intermediate-Temperature Semi-Circular Bend (SCB-LA)	LaDOTD TR 330 ASTM D 8044	Strain Energy Release Rate
	Low-Temperature Semi-Circular Bend Test	AASHTO TP 105	Fracture Energy
	Texas Overlay (OT) Test	TxDOT Tex-248-F NJDOT B-10	Cycles to Failure & Fracture Properties
	Thermal Stress Restrained Specimen Test (TSRST)	BS EN12697-4	Fracture Temperature & Fracture Strength
	Cantabro Abrasion Loss	AASHTO TP 108	Mass Loss
Moisture Resistance	Hamburg Wheel Tracking Test (HWTT)	AASHTO T 324	Rut Depth & Stripping Inflection Point
	Moisture Induced Stress Tester (MIST)	ASTM D7870	Changes in G_{mb} and Visual Observation of Stripping
	Tensile Strength Ratio (TSR)	AASHTO T 283	Tensile Strength Ratio & Wet IDT Strength

Chapter 3 Site Location, Materials, and Test Methods

This chapter is mostly focused on describing different materials used in this study plan, as well as describing the applied performance test methods and procedures. Further, site locations and reference points for collecting loose asphalt mixtures, field cores, and field condition monitoring are provided in detail.

3.1 Site Locations and Pavement Sections

There were four pavement locations with different asphalt mixture specifications considered as test sections in this research study. Loose asphalt mixtures were collected from each section at the time of paving, to be used for experimental evaluation. Two commonly used high-quality asphalt mixture types in Nebraska, SPR and SLX, were selected for the scope of this research. Table 3.1 shows the information associated with each pavement project. As can be seen in Table 3.1, this study aimed to evaluate a diverse selection of projects. For instance, in terms of binder source and grade, the projects were selected in a way that include three different binder sources and two different performance grades. More to this point, the RAP content in the selected projects ranged from 25 to 45%, while the total binder content was from 5.20 to 6.30%. Among all four projects, three of the selected were applied to the top-lift layer, and the remaining one was utilized in the second layer of pavement.

Table 3.1 Information associated with different pavement projects

Type	SPR (Gresham)	SPR (Tekamah)	SLX (I-Bridge)	SLX (Crofton)
Binder Source/Grade	Flint Hills/ PG 58H-34	Monarch Oil/ PG 58H-34	Flint Hills/ PG 58V-34	Jebro/ PG 58H-34
Executed Layer/Thickness	2^{nd} layer/7 inches	Top-lift layer/2 inches	Top-lift layer/Less than 2 inches	Top-lift layer/ 2 inches
RAP (%)	45%	45%	25%	35%
WMA Additive	AD-here®	Delta S®	Delta S®	Evotherm®
Total Binder	5.20%	5.35%	5.30%	6.30%

Apart from loose asphalt mixtures, filed core specimens were also collected from two out of four pavement projects in this research study (Tekamah and Crofton). The Gresham project was not used since the second layer was under assessment, and the I-bridge project was also not considered because the executed surface layer thickness was lower than the minimum required thickness for coring purposes. The field core specimens were supposed to be collected every six months starting from pavement installation time. However, due to the logistic issues, the first round of coring right after construction was skipped, while 6-month and 12-month core specimens were gathered and analyzed. This process will continue as an annual procedure and the data will be utilized in the next phases of Nebraska balanced mix design projects.

To take a further step in analyzing different performance tests, field condition monitoring of all four sections was fulfilled during this research project with the purpose of gathering data from the pavement surface during its service life. The data derived from field condition monitoring includes the IRI index, rutting depth, temperature cracking index, and fatigue cracking index of each individual project. The collected data might be useful in validating the results derived from the field core assessment. Figure 3.1 shows the site location of different selected projects in this study.

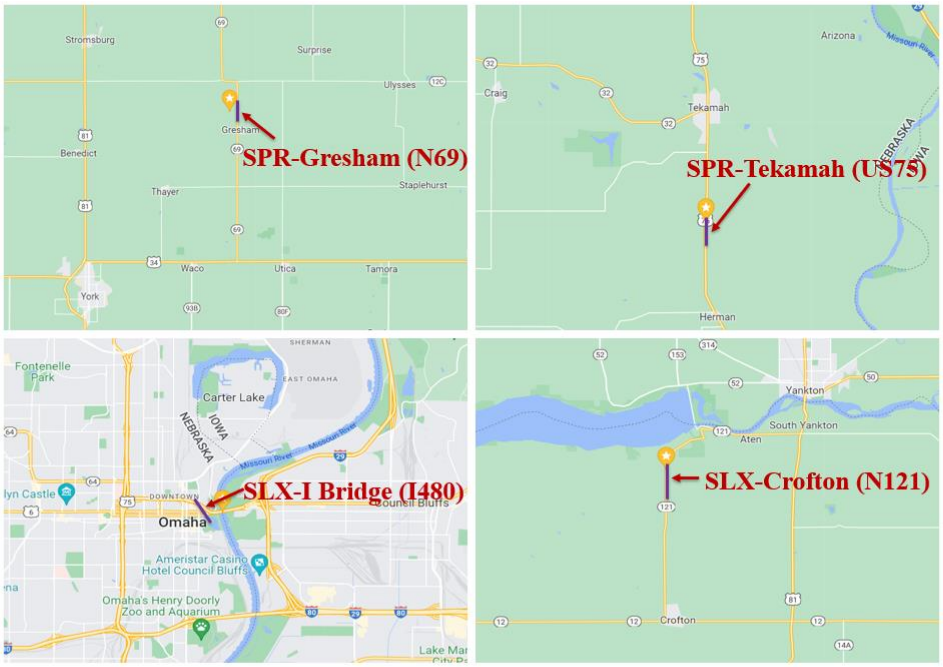


Figure 3.1 Site location of selected projects in this study

3.2 Materials

3.2.1 Warm Mix Asphalt (WMA) Additives

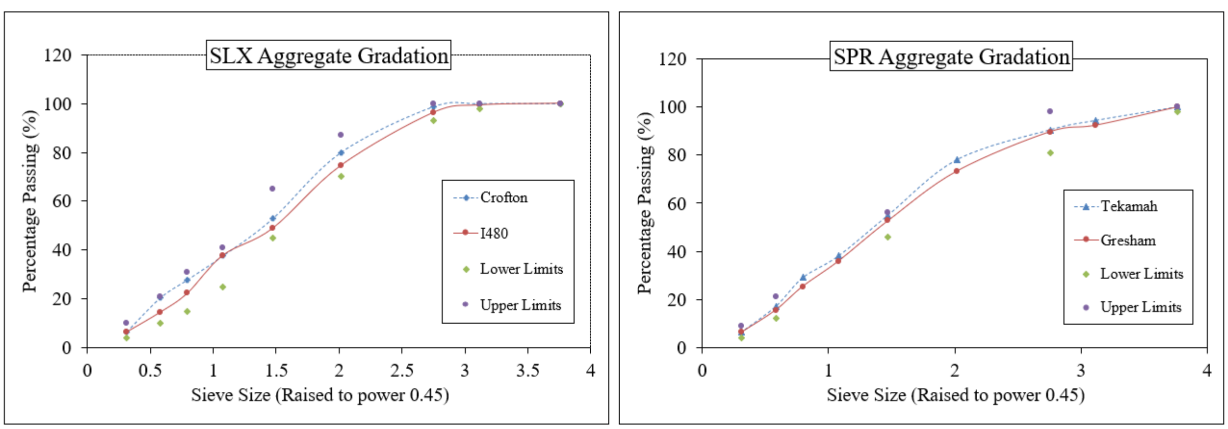
Evotherm® is one of the WMA additives used in the SLX Crofton project (0.7%w) in this study. This warm additive is capable of reducing the viscosity of asphalt binder. The product was owned by the Ingevity company and was used as a cost-effective WMA additive.

Delta S®, categorized and claimed as a true rejuvenator and additive-based WMA technology, can return the binder part of a recycled asphalt to its original functionality by reversing the natural oxidation process. Further, mixing and compaction temperatures can reduce in the presence of this warm additive. This product was developed and owned by Collaborative Aggregates and was utilized in the asphalt mixture design of two projects in this study, SLX-I480 Missouri, and SPR Tekamah (0.7% w).

AD-here® ULTRA warm mix additive was formulated to enable the asphalt mixture to be produced and compacted at lower temperatures, while protecting it from moisture induced damages. As the lower viscosity and production temperatures were achieved by using this product, the cooler ambient temperatures and longer distances were also applicable for pavement implementation. The product was developed and owned by ArrMaz and utilized in 0.7%/w in the SPR Gresham project.

3.2.2 Aggregates (RAP and Virgin Materials)

This study used two main types of asphalt mixtures, SPR and SLX, for the state of Nebraska. The SLX mixtures were prepared with 35% and 25% RAP in two different projects. The blend of virgin aggregates for the SLX Crofton project (with 35% RAP) was composed of 20% Qtz chips, 7% Qtz Man Sand, 33% crushed gravel, and 5% gravel; while the SLX I bridge project (with 25% RAP) was produced with 22% limestone, 29% limestone Man-Sand, 19% limestone screenings, and 5% 47B gravel. Figure 3.2a indicates the gradation of aggregates (virgin and RAP blended) along with the higher and lower limits set by the Nebraska DOT for SLX mixtures in this study.



(a) (b)

Figure 3.2 Aggregate gradation of asphalt mixtures in this study (a) SLX and (b) SPR type

The SPR asphalt mixtures in this study both contained 45% RAP material. The SPR-N69-Gresham had virgin aggregates with a gradation of 12% 47B Rock, 38% 3A crushed gravel, and 5% 2A gravel; while the SPR-US75-Takamah aggregates had a gradation of 10% limestone gravel, 26% limestone man sand, 12% asphalt stone, and 7% 47B gravel. Figure 3.2b indicates the gradation of aggregates (virgin and RAP blended) along with the higher and lower limits set by the Nebraska DOT for the SPR asphalt mixtures in this study.

3.2.3 Asphalt Binder

Two types of asphalt binders, PG 58H-34 and PG 58V-34, from three different sources were used in this research study. A total binder content of 5.20, 5.35, 5.30, and 6.30% were applied for the SPR-N69, SPR-US75, SLX-I480, and SLX-N121 types of asphalt mixtures, respectively. Table 3.2 demonstrates the related information about virgin and RAP binder contents in this study.

Table 3.2 The virgin and RAP binders in the different types of asphalt mixtures

Description	**SPR-N-69 (Gresham)**	**SPR-US-75 (Tekamah)**	**SLX-I-480 (I-Bridge)**	**SLX-N-121 (Crofton)**
Binder Source/grade	Flint Hills/58H-34	Monarch Oil/58H-34	Flint Hills/58V-34	Jebro/PG 58H-34
Total Binder	5.20%	5.35%	5.30%	6.30%
Virgin Binder	2.90%	3.10%	4.25%	4.07%
Binder from RAP	2.30%	2.25%	1.05%	2.23%

<u>3.3 Laboratory-compacted and Field Core Specimens</u>

In this study, plant-mixed laboratory compacted (PMLC), and plant-mixed field compacted (PMFC) specimens were obtained from NDOT-approved high quality asphalt mixtures. Accordingly, the plant produced SPR and SLX mixtures were collected from each project during construction and the PMLC specimens were compacted and prepared using these loose asphalt mixtures. The loose mixtures were reheated for two hours at 135°C to be workable enough for the compaction process. Further, to simulate long-term aging, NCHRP 09-54 and NCAT protocols were applied on the loose asphalt mixtures, as will be described in Section 3.4.8. Applying a target air void of 7%, the Superpave Gyratory Compactor (SGC) was utilized to prepare the PMLC specimens with a diameter of 150 mm and various thicknesses based on the specific test methods (Figure 3.3). Table 3.3 indicates summarized information and acronyms utilized for different types of PMLC specimens in this study.

Figure 3.3 Part of PMLC specimens utilized in this study

Table 3.3 Summarized information of PMLC and PMFC specimens used in this study

Specimen ID	Project	Compaction Method	Specimen ID	Project	Compaction Method
LT	Tekamah	Laboratory	LTA-NCHRP	Tekamah	Laboratory (LTA by NCHRP)
LG	Gresham	Laboratory	LGA-NCHRP	Gresham	Laboratory (LTA by NCHRP)
LC	Crofton	Laboratory	LCA-NCHRP	Crofton	Laboratory (LTA by NCHRP)
LI	I-bridge	Laboratory	LIA-NCHRP	I-bridge	Laboratory (LTA by NCHRP)
FT6	Tekamah	Field (Cored after 6 months)	LTA-NCAT	Tekamah	Laboratory (LTA by NCAT)
FC6	Crofton	Field (Cored after 6 months)	LGA-NCAT	Gresham	Laboratory (LTA by NCAT)
FT12	Tekamah	Field (Cored after 12 months)	LCA-NCAT	Crofton	Laboratory (LTA by NCAT)
FC12	Crofton	Field (Cored after 12 months)	LIA-NCAT	I-bridge	Laboratory (LTA by NCAT)

In terms of field evaluation, several PMFC (field cores) specimens were collected from two pavement sections, six months and one year after construction. Among a total of four different projects, one SPR and one SLX types (Tekamah and Crofton) were selected for the coring process. The field core specimens, with a diameter of 150 mm and thickness of 75 mm were collected while the specific lanes were closed during the process. As the top lift layer in Nebraska paving have a normal thickness ranging from 1.5 to 2 inches (38 to 51 mm), the core specimens were trimmed in a way that the maximum final thickness was obtained for each specimen (Figure 3.4). A total of 36 cores were taken from each pavement section (72 in total): 18 cores were taken six months after construction, and 18 cores were taken one year after the construction process. Due to logistic issues, the core specimens were not collected right after construction. Table 3.3 shows summarized information and acronyms utilized for different types of PMFC specimens in this study.

Figure 3.4 An example of PMFC specimens before and after trimming

3.4 Test Methods

3.4.1 Hamburg Wheel Tracking (HWT) Test

The HWTT was developed in Germany [23] as a procedure to test the rutting and moisture susceptibility of asphalt concrete mixtures. The method tested specimens (typically two discs from SGC) loaded by a steel wheel until the point that failure rutting depth was achieved. According to AASHTO T324-14 and Tex-242-F, the test temperature was based upon the applicable specification (over a range of 25 to 70°C) while the loaded wheel of 705 N shut off when 20,000 passes occurred or when the maximum impression depth was achieved. HWTT showed a significant correlation to the field performance data, however, the complexity of the test limited its practical and repeatable application.

In this study, the HWT test was conducted on both PMLC and PMFC (field cores) specimens following the AASHTO T324-14 standard [24]. In terms of laboratory compacted mixtures, the specimens with 62 ± 1 mm thickness and 7 ± 0.5% air void compacted by SGC were prepared and the necessary cuts were performed based on the standard. For the field core specimens with a thickness of 50 mm, the mix capping compound was used on top of cores to achieve the 62 ± 1 mm total core height. Figure 3.5 shows the HWT test setup along with sample preparation for both PMLC and PMFC specimens.

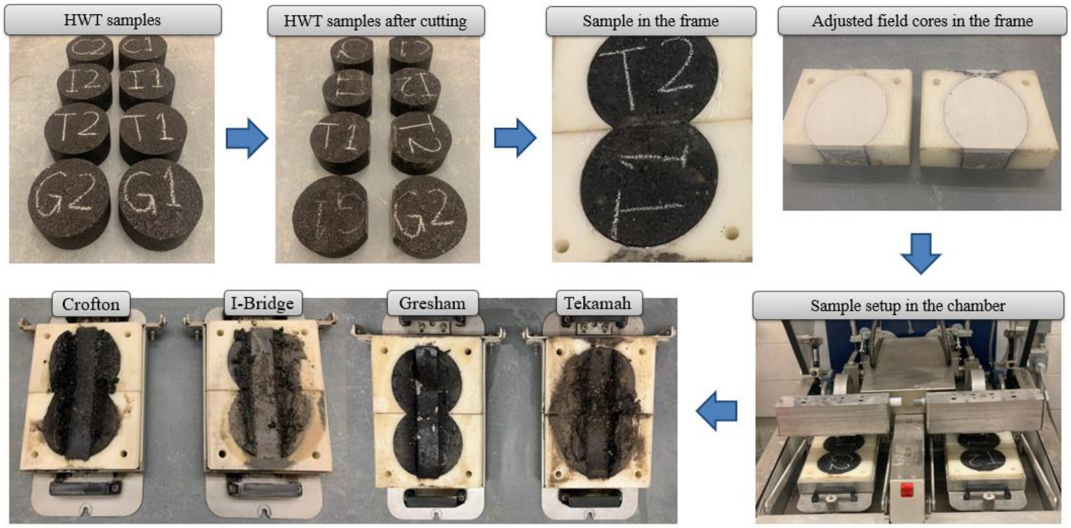


Figure 3.5 HWT test setup and sample preparation for PMLC and PMFC specimens

To analyze the results derived from the HWT test, 20,000 passes was selected as the termination point to further record the total rutting depth associated with this number of load cycles. Rutting depth is defined as the direct measurement of deformation depth over different points on the surface of the specimen. With that, the rutting parameter was obtained from the total rutting depth measurement at a certain number of load cycles. The lower the total rutting depth, the higher the permanent deformation resistance of the specimen. To analyze moisture damage resistance, the Stripping Inflection Point (SIP) was defined as the number of passes derived from the intersection point of the secondary (Creep) and tertiary (stripping) phases. A higher SIP represented higher moisture damage resistance of specimens. Figure 3.6 indicates a typical HWT test output.

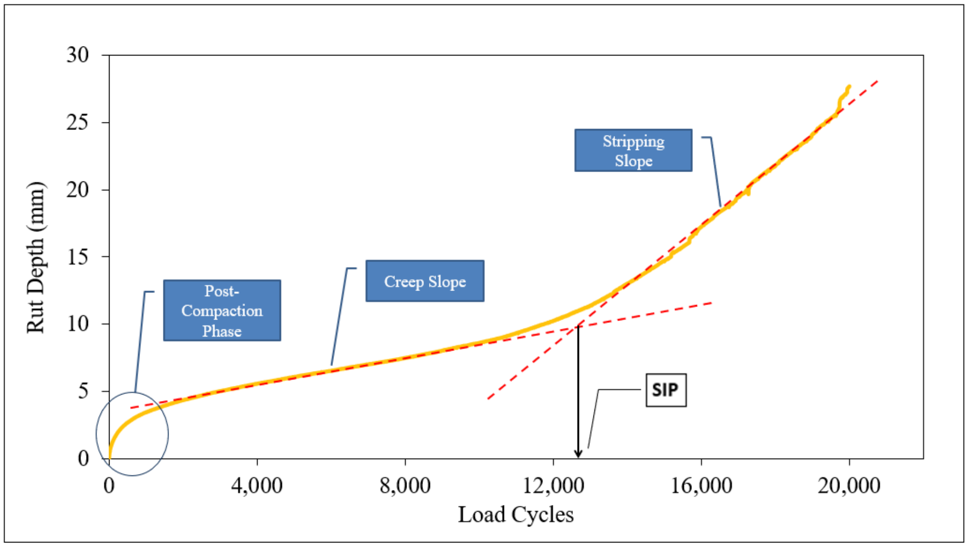


Figure 3.6 typical HWT test output with creep slope, stripping slope, and SIP determination

To further analyze the results derived from the HWT test a new approach was also taken into consideration [25]. Although the HWT test proved able to discriminate asphalt mixture performance in terms of rutting and moisture damage resistance, the current parameters derived from this test were not always precisely characterizing these properties. More to this point, rutting depth at a certain number of load cycles may have been specified based on the interacting effects of both loading and stripping, particularly, in the case of mixtures that are more susceptible to moisture damages. Additionally, the bias that came from fitting two straight lines to distinguish the creep phase and stripping phase can significantly affect the moisture susceptibility analysis [25]. However, in this new analysis method, a polynomial curve was fitted on typical rut depth versus load cycle plot (Figure 3.7a). The first part of this curve (negative curvature) was mostly related to the mixture stiffening due to repeated wheel loads that resulted in viscoplastic deformation and an increased rut depth, while the second part (positive curvature) was mostly about mixture softening (increased rutting) due the stripping of the asphalt binder from the aggregates. The inflection point, where the curvature turns from positive to negative, was referred to as stripping number (SN), and the number of load cycles to this point (LC_{SN}) was used as a parameter to quantify moisture susceptibility.

To quantify mixture rutting resistance, firstly, the specimen's viscoplastic strain was calculated as the ratio of rut depth over the specimen thickness. Accordingly, viscoplastic strain versus load cycle curve was plotted up to the LC_{SN} point (Figure 3.7b). Fitting the Tseng-Lytton model to this part of graph, the viscoplastic strain curve was projected into the stripping phase. Finally, the parameter viscoplastic strain increment ($\Delta\varepsilon^{vp}$) calculated as the slope of this projected curve at a certain number of load cycles (i.e., 10000) was utilized to quantify rutting resistance of asphalt mixtures by only considering the creep phase of HWT test results.

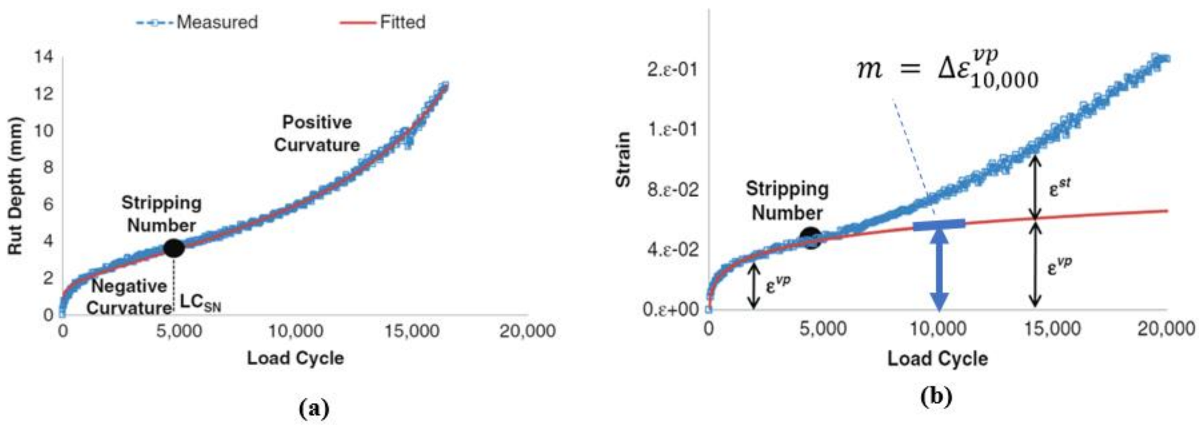


Figure 3.7 HWTT analysis (a) stripping number determination (b) Projected viscoplastic strain

3.4.2 Indirect Tensile Asphalt Rutting Test (IDEAL-RT)

The IDEAL-RT procedure was recently developed by Texas A&M University [26] with the goal of simplifying rutting evaluation of asphalt concrete mixtures through a rapid test; particularly, during the QA/AC phases. This test was conducted at 50 °C with a loading rate of 50 ± 2.0

mm/min, while a newly developed bottom fixture was used to apply shear stress. As reported by developers, a good correlation was found between the test results and the field performance of asphalt mixtures which makes it a repeatable test in addition to its simplicity and efficiency [26]. Also, this cost-effective test was workable for both laboratory-molded and field cored specimens with no need for sophisticated training.

In this study, cylindrical laboratory specimens were prepared with a diameter of 150 ± 2 mm and a height of 62 ± 1 mm. The load and Load-Line Displacement (LLD) were recorded during the test to calculate the maximum shear strength. The filed core specimens followed the same procedure, only the specimen height was 50 mm, which satisfied the required thickness ranging from 38 to 95 mm for the field cores [26]. The shear strength of an asphalt mixture was a performance indicator of the rutting resistance, while the higher shear strength value showed better rutting resistance and smaller rutting depth in the field. Figure 3.8 indicates the IDEAL-RT test setup used in this research study. Equation (3.1 was used to calculate the shear strength value:

$$\tau_f = 0.356 \times \frac{P_{max}}{t \times w} \tag{3.1}$$

Where, τ_f is the shear strength (Pa), P_{max} is the maximum load (N), t is the specimen thickness (m), and w is the width of the upper loading strip (=0.0191 m).

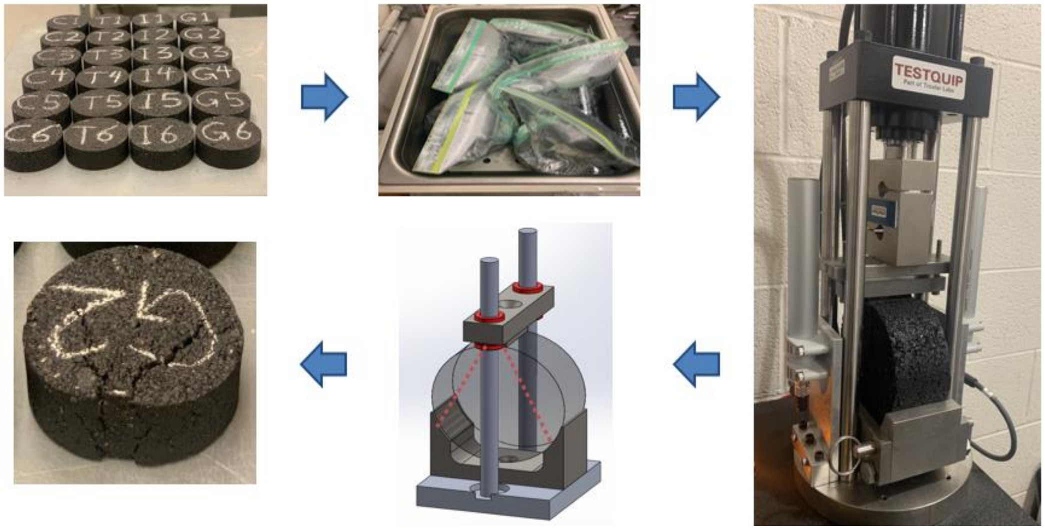


Figure 3.8 IDEAL-RT test setup and sample conditioning

3.4.3 High-Temperature Indirect Tensile (HT-IDT) Test

Recently developed by the Alabama Department of Transportation [27], the HT-IDT test was suitable for evaluating the rutting resistance of asphalt mixtures via an indirect tensile method on a Marshall press. A vertical load with a displacement rate of 50 mm/min was applied to the test fixture and the maximum load was recorded. The samples were conditioned in a forced draft oven at the test temperature of 50 ± 1 °C (122 ± 2 °F) for two hours prior to testing. Afterward, the HT-IDT strength was calculated through Equation (3.2:

$$HT - IDT\ strength\ = \frac{2\ *\ P_{max}}{\pi\ \times\ D\ \times\ H} \tag{3.2}$$

Where HT-IDT strength is the rutting resistance parameter (MPa), P_{max} is the maximum load (KN), D is the average diameter (mm), and H is the height (mm).

In this study, the HT-IDT test was utilized to evaluate not only rutting resistance but also moisture induced damage of asphalt concrete mixtures. One set of plant-produced asphalt concrete mixtures was provided for the freeze-thaw cycle prior to the HT-IDT test, and the HT-IDT strength ratio of conditioned samples over unconditioned ones was considered as a parameter for moisture susceptibility. As the equipment needed for this test was available in most of the laboratories, and at the same time, a quick test procedure was followed by a simple data analysis, this test was a good candidate for the BMD QC/QA phases. Figure 3.9 indicates the HT-IDT test setup in this research study.

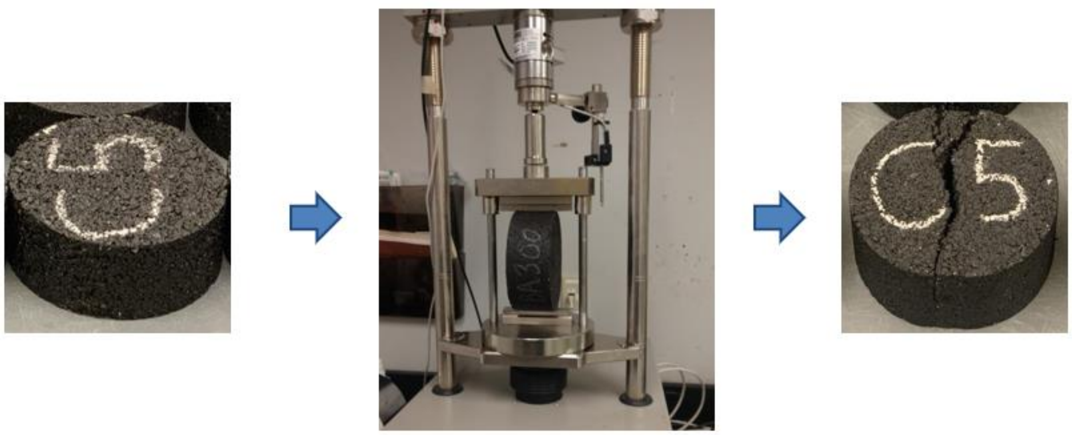


Figure 3.9 HT-IDT test setup and sample geometry before and after the test running

3.4.4 Gyratory Stability (G-stability) Test

The G-stability test was developed and introduced by NDOT and the University of Nebraska-Lincoln [5] to evaluate the rutting resistance of asphalt concrete mixtures. In this test, a disc-shaped specimen with a diameter of 150 mm was loaded using the Marshall stability test fixture, while the force and displacement were recorded over the testing time. The "Stability" and "Flow" were defined as the peak load and the displacement corresponding to the peak load, respectively. The peak load was an indicator of the rutting resistance of asphalt mixtures. Furthermore, there was potential for the moisture susceptibility of asphalt concrete mixtures to be analyzed through the G-stability test by easily introducing the specimens to a freeze-thaw cycle before running the test and measuring the stability ratio of conditioned over unconditioned specimens.

Considering simple monotonic displacement-controlled loading equipment needed to fulfill the G-Stability test, as well as a straightforward data analyzing process, this test is a good candidate for the QC/QA aspect. Further, the testing fixture (Marshall stability fixture) is widely available in almost every asphalt laboratory. Figure 3.10 shows the test set-up for the G-stability test as well as sample geometries before and after running the test.

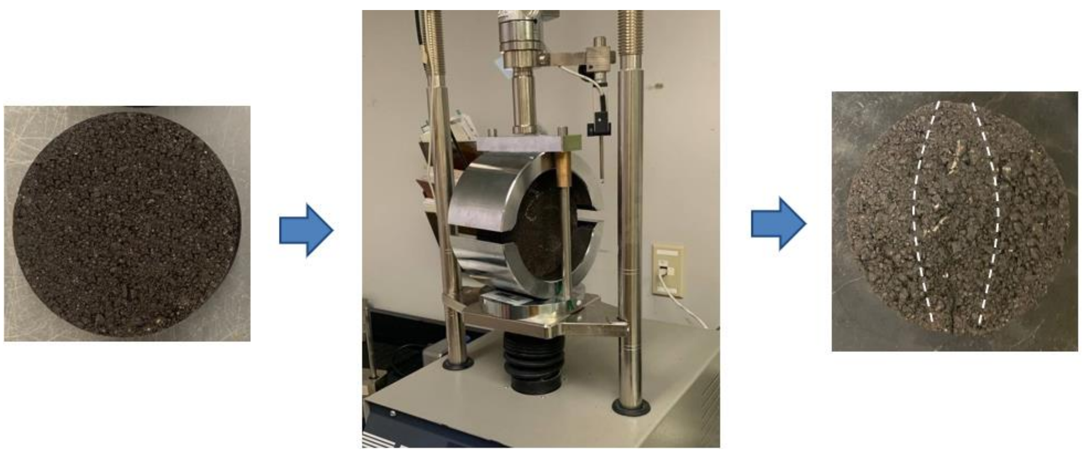


Figure 3.10 G-stability test setup and sample geometry before and after the test running

3.4.5 Semi-Circular Bending Illinois Flexibility Index Test (SCB-IFIT)

The SCB test was first introduced as a simple method to evaluate the fracture resistance of rock materials [28] and it is now utilized in the asphalt community to characterize the cracking resistance of asphaltic materials (ASTM D8044 16) (AASHTO TP 124-20). A semi-circular shaped specimen with a single edged notch is loaded through a cyclic/static loading while the geometry leads to crack propagation throughout the specimen because of the tension inducement at the bottom of the sample. High repeatability, reproducibility, and consistency of specimen production and testing were the main reasons that made this test method a promising candidate to evaluate asphalt concrete mixtures in terms of fracture properties. Various indicators including fracture energy, cracking resistance index, and flexibility index can be employed to interpret the results of different SCB test protocols [29].

In this study, the Semi-Circular Bending Illinois Flexibility Index Test (SCB-IFIT) was conducted following AASHTO TP124-20 standard [30]. Accordingly, the cut and notched semi-circular specimens (150 mm diameter and 50 mm thickness) positioned in the test fixture were loaded along the vertical radius (rate of 50 mm/min) at a testing temperature of 25 ± 0.5 °C. The load and load line displacement (LLD) were recorded through the process so the fracture energy (G_f) and post peak slope (m) could be determined to further develop the Flexibility Index (FI) as a cracking resistance indicator for asphalt concrete mixtures. In terms of field evaluation, it is allowed to use core specimens with thicknesses ranging from 25 to 50 ± 1 mm, so core specimens with a height of 38 mm were utilized during field evaluation while a thickness correction factor was applied on the test results. To determine the post peak slope (m), a tangential curve was drawn at the inflection point while its slope showed the m value. The Flexibility Index (FI) can be calculated through Equation (3.3 (AASHTO TP 124-20):

$$FI = \frac{G_f}{|\, m \,|} \times A \tag{3.3}$$

Where G_f is the fracture energy (J/m^2), $|\, m \,|$ is the absolute value of post-peak load slope (KN/mm), and A is a unit conversion and scaling factor equal to 0.01.

The FI proved to be affected by variations in the thickness of test specimens. Investigating the relationship between FI and specimen thickness indicated an opposite linear trend between these two parameters, which can be explained by the effect of specimen thickness on the post peak slope value. As such, Equation (3.4 was recommended as a simple correction method to turn the FI values of specimens with different thicknesses into the standard 50-mm thick specimens. Figure 3.11 presents the I-FIT test setup along with sample preparation and conditioning steps.

$$FI_{50} = FI_t \times \frac{t}{50} \tag{3.4}$$

Where FI_{50} is the corrected index value of the 50-mm reference thickness, FI_t is the measured FI value of specimens with different thicknesses, and t is the average specimen thickness (mm).

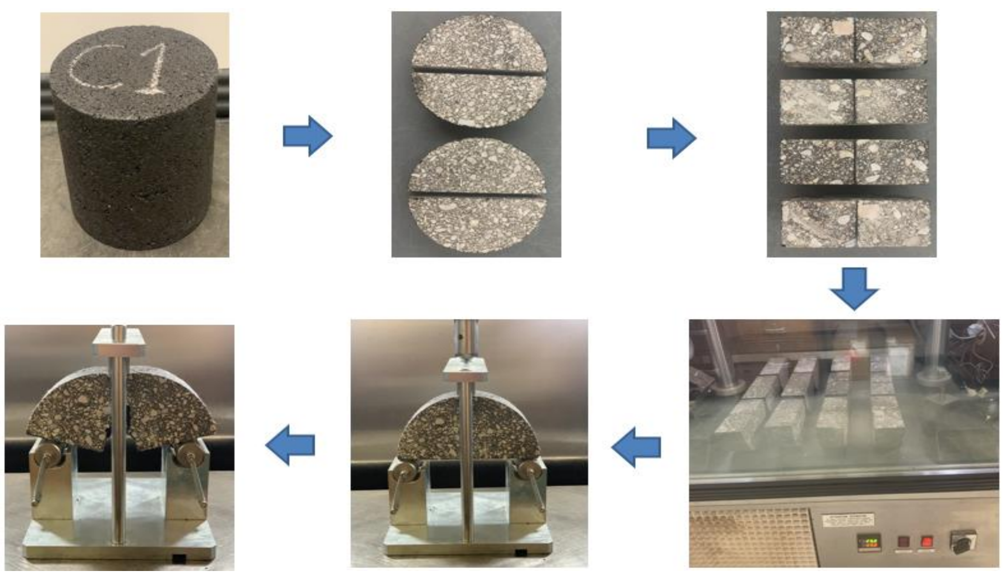


Figure 3.11 I-FIT test setup along with sample preparation and conditioning steps

3.4.6 Indirect Tensile Asphalt Cracking (IDEAL-CT) Test

The IDEAL-CT was an indirect tension test developed by Zhou, Im [31] as a practical and easy method utilized to evaluate fracture resistance of asphalt concrete mixtures. The process included fabricating cylindrical specimens with 150 mm diameter, 62 mm height, and 7 ± 0.5 percent air void, without necessitating cutting, notching, drilling, or gluing. A load point displacement (LPD) mechanism with a rate of 50 mm/min was applied on specimens at room temperature, while the cracking tolerance index (CT_{Index}), as a performance related cracking parameter, was derived as the main parameter in this method. During field evaluation, different thicknesses (38, 50, 62, 75 mm) of core specimens were allowed to be used, while a correction factor needed to be applied in the formula. The CT_{Index} value, considering different thickness of specimens, was derived from Equation (3.5. The rationale behind the selection of the critical point as the point where the load decreased to 75% of the peak load (post-peak load (PPL75)) is explained elsewhere [31].

$$CT_{Index} = \frac{t}{62} \times \frac{G_f}{m_{75}} \times \frac{l_{75}}{D} \tag{3.5}$$

Where G_f is the fracture energy (J/m^2), m_{75} is the "modulus" parameter (interval slope of load-displacement curve between 65% and 85% of the peak load), and l_{75} is a "strain" tolerance parameter when the load reduced to 75% of the peak load.

IDEAL-CT was considered a simple, practical, and efficient test method (test completion within 1 minute) which was performed using regular indirect tensile strength test equipment. Further, a good sensitivity was observed between the test results and the mixture component and volumetric properties [31]. As a result, the IDEAL-CT test is a good candidate for QA/QC-related purposes. Figure 3.12 indicates the IDEAL-CT test setup, sample conditioning, and sample geometry before and after running the test.

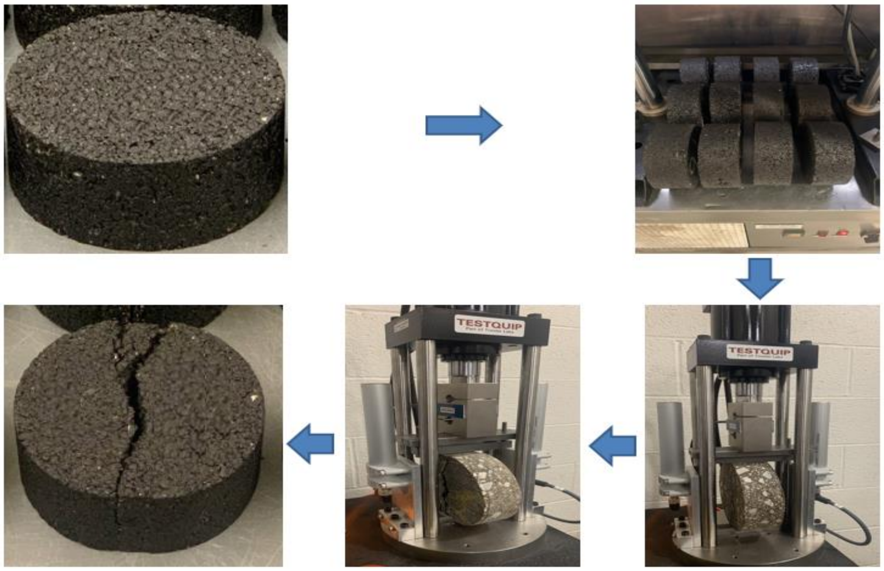


Figure 3.12 IDEAL-CT test setup, sample conditioning, and sample geometry before and after running the test

3.4.7 Tensile Strength Ratio (TSR) Test

The TSR was a typical test for evaluating moisture damage resistance of asphalt concrete mixtures and was conducted following AASHTO T283 standard [32]. Accordingly, the Indirect Tensile Strength (ITS) ratio of conditioned asphalt concrete mixtures over unconditioned ones was considered as the main criterion in this study. Based on the standard, the asphalt specimens with a diameter of 150 mm and a thickness of 95 ± 5 mm were used while the air-void was designed to be 7 percent. The specimens were embedded inside the water bath with a temperature of 25 ± 0.5 °C for two hours prior to placing between two bearing plates in the testing machine. The loading rate was 50 mm/min, and the maximum compressive strength was recorded during the testing process for further calculating the tensile strength as per Equation (3.6.

$$S_t = \frac{2000P}{\pi \times t \times D} \tag{3.6}$$

Where S_t is the tensile strength (kPa), P is the maximum load (N), t is the specimen thickness (mm), and D is the specimen diameter (mm).

3.4.8 Short-term Aging (STA) and Long-term Aging (LTA) Protocols

With the knowledge of aging's effects on the mechanical performance of asphalt mixtures, several laboratory aging protocols have been developed over the past few decades to account for the aged asphalt mixtures during mix design and production. To simulate Short-term Aging (STA) (during production phase), AASHTO R30 recommends two hours of conditioning at compaction temperature (for design), or four hours at 135 °C for mechanical testing. However, because this study used loose asphalt mixtures collected directly from the project sites, a short-term aging procedure was not required. As a result, only two hours of conditioning at the compaction temperature was applied before the specimens' fabrication.

There are two common approaches to simulate Long-Term Aging (LTA): aging compacted specimens and aging loose mixture before compaction. The latter method was selected for this project. The former approach (i.e., AASHTO R30 protocol of 5 days conditioning compacted mixture at 85 °C) suffers from lack of severity to simulate long-term aging and correlating the actual field aging of asphalt concrete mixtures [33], [34]. Two common protocols, the National Cooperative Highway Research Program (NCHRP) 09-54 and the National Center for Asphalt Technology (NCAT) protocols, have been utilized in this research study. With respect to the NCHRP 09-54 protocol, the loose mixture was aged at 95 °C for a period of 3 days to match 8 years of field aging at 20 mm below the pavement surface in the state of Nebraska [35]. Although this protocol proved to be more realistic to simulate long-term aging in the field, there were some operational challenges due to the time-consuming nature of this method. Particularly, with respect to balanced mix design, there was an increased interest in adopting performance testing in the mix design and production phases of asphalt mixtures which required implementing a fast and field-validated long-term aging procedure that could be accomplished overnight.

As a result, another long-term aging protocol developed by NCAT was considered in this research study [36]. This protocol applied 8 hours of conditioning at 135 °C on the loose asphalt mixtures to simulate long-term aging phenomena. In this study, the conditioned specimens derived from these two aging protocols were used in fatigue cracking test analyses and results were compared to each other. As time passes, with more field data derived from actual pavement surfaces, it would be possible to have a better understanding about the accuracy and applicability of each long-term aging procedure.

Chapter 4 Laboratory Test Results and Discussion

In this study, two different tests (I-FIT and IDEAL-CT) were used to evaluate mid-temperature cracking resistance, four tests (HWTT, IDEAL-RT, HT_IDT, and G-stability) were applied to evaluate rutting resistance, and three tests (TSR, HWTT, HT-IDT) were utilized to inspect the moisture damage resistance of asphalt mixtures. Furthermore, all tests were conducted on the Plant Mixed Lab Compacted (PMLC), and Plant Mixed Field Compacted (PMFC) specimens. The long-term aging process following NCHRP 09-54 and NCAT protocols were also considered in this plan for the mid-temperature cracking tests. In this chapter, the performance test results are analyzed for each testing protocol, considering both categories of asphalt specimens, PMLC and PMFC.

4.1 Mid-temperature (Fatigue) Cracking Resistance

4.1.1 Illinois Flexibility Index Test (I-FIT) Results

Running I-FIT test, Equation (3.3 was utilized to calculate the Flexibility Index (FI) values. To perform this test, short-term aged and long-term aged PMLC specimens were prepared following procedures discussed in Section 3.4.8. The PMLC specimens were fabricated with the standard thickness of 50 mm, while the PMFC specimens had non-standard thicknesses and the FI values were adjusted using Equation (3.4.

Figure 4.1 shows the results of the I-FIT test in terms of FI values for the short-term aged and long-term aged PMLC specimens. According to Figure 4.1, in terms of short-term aged specimens, LT and LG containing 45% RAP had FI values of 12.3 and 26, respectively. This demonstrated good fatigue cracking resistance, but a lower resistance compared to the two other specimens. The results were reasonable, as the higher content of RAP in LT and LG made the asphalt mixture more brittle and less resistant to fatigue cracking due to the presence of an aged stiff binder in the RAP material and limited blending of the binder around the RAP [37], [38]. Comparing two other specimens (with lower RAP contents) showed FI values of 33.6 and 42.4 for the LI and LC, respectively. This indicated superior performance of LC in terms of fatigue cracking resistance over all types of mixtures. At first glance, one expects LI to have a superior cracking resistance because of lower RAP content present in its mix design (10% less RAP compared to LC). In practice, the higher flexibility index value resulted in the LC having a higher asphalt content present in its mix design (6.3% for LC compared to 5.3% for LI) [39]. This higher asphalt content value was effective in compensating for the negative effect of a higher RAP content, as the final amount of added virgin binder was almost the same for both types of mixtures (about 4% of PG58V-34). To further validate these results, field investigation is a necessary step.

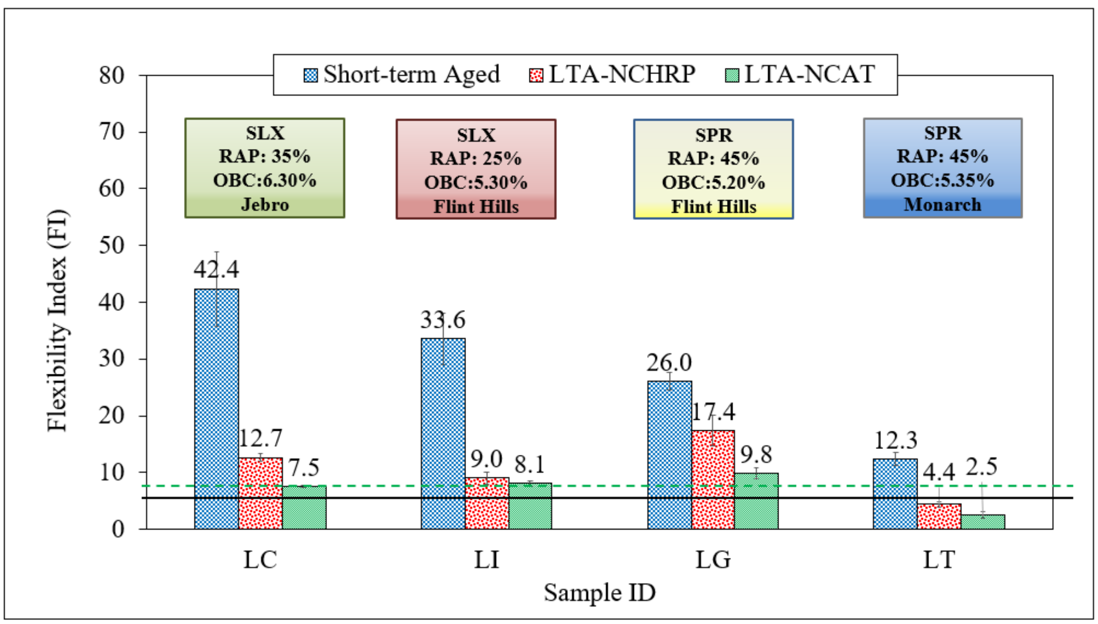


Figure 4.1 Flexibility Index values for the PMLC specimens (Short-term aged and long-term aged specimens)

The FI values for LTA specimens are shown in Figure 4.1. As can be seen, all mixtures faced a reduction in FI values after long-term aging conditioning. This reduction rate was different among types of mixtures and long-term aging protocols. For instance, with respect to the NCHRP 09-54 protocol, the LI mixture with only 25% RAP showed 73% reduction in the FI value, while for the LG mixture with 45% RAP this reduction was found to be 33%. The reason might be related to the fact that RAP mixtures include already aged binders that do not age as aggressively under laboratory conditioning [40]. As a result, compared to other specimens, the LG mixture had the best performance in terms of fatigue cracking after the long-term aging conditioning (NCHRP 09-54 protocol). In the case of LT with 45% RAP, this reduction rate was still 64% which was comparable to the mixtures with lower RAP contents. As LT had the lowest cracking performance before long-term aging, this high rate of reduction after long-term aging might be because of a binder source, as well as properties of RAP materials utilized in the mix design. Compared to the NCHRP 09-54 aging protocol, the NCAT procedure resulted in a higher rate of reduction in FI values for all types of specimens. It can be an indication of the higher level of aging yielded by the NCAT protocol compared to the NCHRP 09-54 [41]. Interestingly, in the case of NCAT aged specimens, LG and LT have the highest and lowest FI values, respectively. This is a similar trend to the NCHRP 09-54 long-term aged specimens. To further investigate and compare NHCRP and NCAT aging protocols, long-term field data are required.

To have a better understanding about fatigue cracking performance of asphalt mixtures, the values needed to be compared to an established threshold. As Nebraska state criteria for fatigue cracking resistance was still under development, another state with similar climatic conditions had been selected as a reference for the threshold values. Accordingly, Illinois defined its fatigue cracking criteria as FI > 8 for the STA specimens (green dash line in Figure 4.1), and FI > 5 for the LTA ones (black straight line in Figure 4.1) [42]. With that, only long-term aged LT (both aging protocols) showed unacceptable fatigue cracking resistance among all projects.

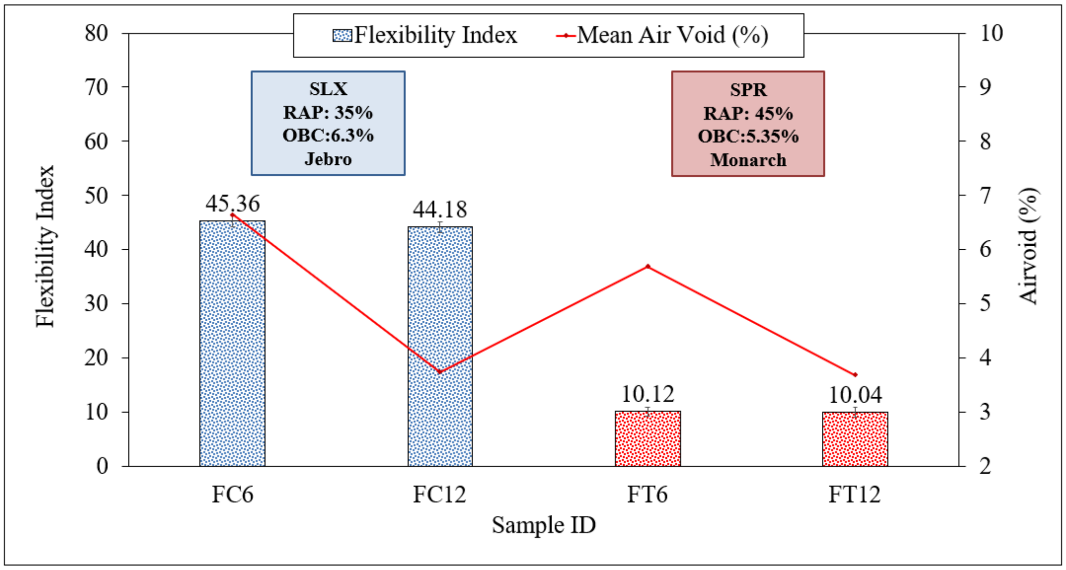


Figure 4.2 Flexibility index values of PMFC specimens for Tekamah and Crofton Projects

To further evaluate the I-FIT test for possible implementation in the Nebraska BMD, field investigation was carried out on the same asphalt mixtures. To this end, PMFC specimens (field cores) were extracted and analyzed for two out of four projects in this study. Unfortunately, due to technical issues, the PMFC specimens were not analyzed immediately after construction of each project, however, the analysis was fulfilled on the six-month- and one-year-old core specimens. The same evaluation process will continue in the future as an annual investigation. Figure 4.2 indicates the FI values of six-month and one-year PMFC specimens. For the FT specimens (45% RAP and 5.35% binder content), the FI values were 10.12 and 10.04 after six months and one year, respectively. In the case of FC specimens (35% RAP and 6.30% binder content), the FI values were 45.36 and 44.18, respectively. The lower values of the flexibility index for FT specimens compared to FC ones followed the same results derived from lab compacted specimens.

Comparing six-month and one-year PMFC results indicated an insignificant difference in terms of mid-temperature cracking performance, while the air-void values reduced 2-3% in the period of six months to one year. In fact, the I-FIT results imply that either long-term aging effects will not appear during the first year of pavement service life in Nebraska, or the FI parameter did not capture this effect during the first year of service life. To better understand this observation, the PMFC specimens (field cores) are going to be continuously evaluated in the future. Meanwhile, the pavement condition data derived from surface monitoring after six months of service life, indicated only 0.2% fatigue cracking in the Tekamah (T) project. The other three projects did not show any fatigue damage during the first six months of service life. This observation was compatible with the low cracking performance of the Tekamah (T) project in both PMLC and PMFC specimens.

4.1.2 Indirect Tensile Asphalt Cracking (IDEAL-CT) Test Results

Running the IDEAL-CT test, Equation (3.5 was utilized to calculate CT_{Index} values. To properly address the mid-temperature cracking resistance, similar to the I-FIT test, short-term aged and long-term aged specimens were utilized in the IDEAL-CT test protocol. The PMLC specimens were fabricated in the standard thickness of 62 mm, while a lower thickness of PMFC (field cores) specimens was addressed by the correction factor provided in Equation (3.5. The CT_{Index} values for the short and long-term aged PMLC specimens are presented in Figure 4.3.

As indicated in Figure 4.3, in case of short-term aged specimens, LT and LG with 45% RAP in the mix design had lower cracking resistance compared to the two other specimens. CT_{Index} values of 42 and 85 were reported for LT and LG, respectively, which followed the same trend as FI in the I-FIT test. However, LC and LI showed CT_{Index} values of 110 and 156, respectively. To be more specific, compared to LC (35% RAP and 6.30% binder), LI (25% RAP and 5.30% binder) had higher fatigue cracking resistance, which was not compatible with the I-FIT test results. However, it was still possible to justify this observation (higher fatigue cracking resistance of LI compared to LC), as the LI has 10% less RAP in the structure compared to the LC mixture type. Overall, to better validate the results derived from fatigue cracking tests, real pavement condition monitoring, as well as field data, should be analyzed in a long-term process during the pavement service life.

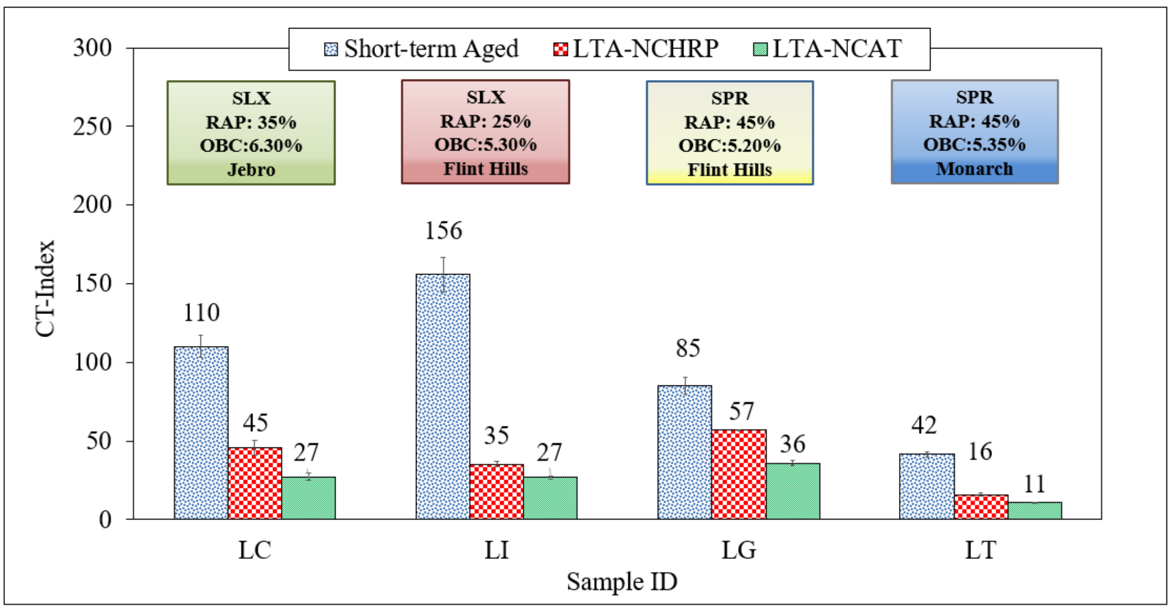


Figure 4.3 The CT_{Index} values for the PMLC specimens (Short-term aged and long-term aged specimens)

Running the IDEAL-CT test on the long-term conditioned specimens caused a significant reduction in the CT_{Index} values. The rate of reduction was different among various types of mixtures, as well as different aging protocols. However, following the NCHRP 09-54 protocol, the highest reduction (77%) was observed in the LI specimen with the lowest amount of RAP in its mix design, as an expected result. Considering the NCAT protocol for long-term aging, the

CT_{Index} reduction rate of all specimens was even higher than the NCHRP 09-54 protocol. Interestingly, the LG specimen with 45% RAP has the lowest CT_{Index} reduction rate, yielding the best fatigue cracking performance after long-term aging in both aging protocols. These results were generally compatible with the I-FIT test results with respect to the cracking performance after long-term aging conditioning. Based on literature review, different states have their own pass/fail criteria for CT_{Index} values, for instance, the minimum values range from 32 in Missouri to 100 in Texas [43]. This indicates that each state should come up with their own threshold values.

To further evaluate the IDEAL-CT test, the CT_{Index} values were measured for the PMFC specimens collected six months and one year after pavement installation. As depicted in Figure 4.4, the CT_{Index} values were 267 and 134 for the FC6 and FC12 specimens, respectively, which showed a significant drop in the cracking parameter after six months of pavement service life. The same happened in the case of FT6 and FT12 with CT_{Index} values of 124 and 60, respectively. Taking a closer look showed an air void reduction of 1.5% and 2.8% in a period of six months for the FC12 and FT12 with respect to the FC6 and FT6. This air void content reduction due to traffic loads was reported to be effective on decreasing the cracking resistance of high-RAP asphalt mixtures [44]–[46]. Moreover, it was expected for one-year core specimens to be more prone to crack as they were exposed to traffic load and environmental conditions for a longer period. Accordingly, the IDEAL-CT test showed a good sensitivity to the mixture volumetric properties by capturing these changes. Overall, pavement performance condition data after six months of service life showed 0.2% fatigue crack happening in the Tekamah (T) project, while other projects did not show any fatigue crack during this period. This makes sense, as the lowest fatigue cracking performance was also captured in the specimens obtained from this project.

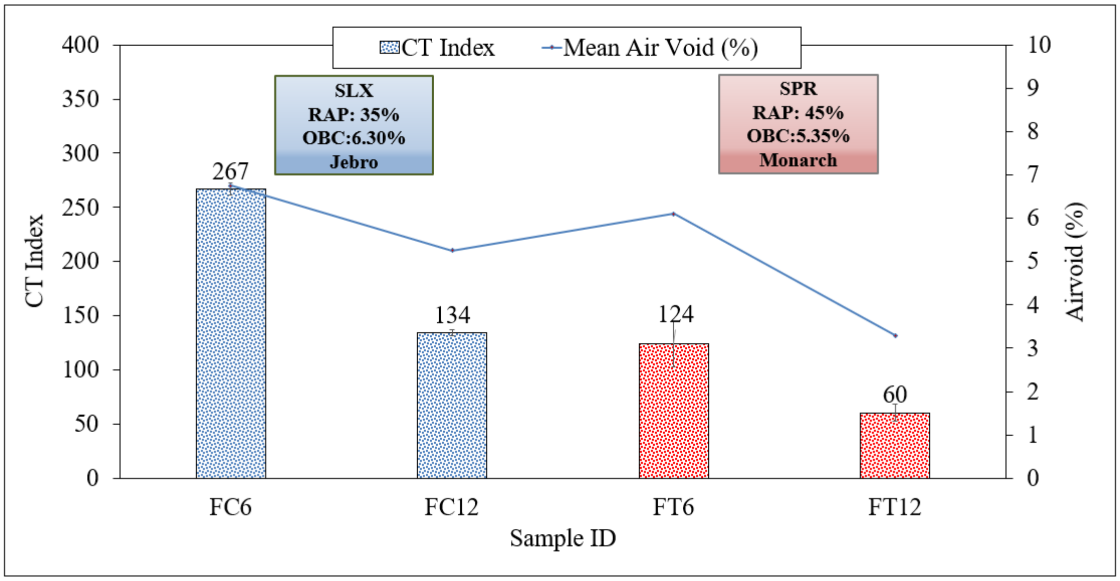


Figure 4.4 The CT_{Index} values of PMFC specimens for Tekamah and Crofton Projects

4.2 Permanent Deformation (Rutting) Resistance

4.2.1 Hamburg Wheel Track (HWT) Test Results

The rut depth vs. load cycles graphs were obtained from running the HWT test and are shown in Figure 4.5. Three phases are happening in this graph: the post-compaction phase, creep phase, and stripping phase as they were recognized in LC, LI, and LT specimens. However, in the case of LG, the third phase did not happen until 20,000 load cycles (termination point) which indicated a good performance of LG in terms of moisture damage resistance. Different practices have been used to analyze the results for the HWT test [47], [48], in which, the most common and traditional one was to find out the total rut depth at a certain number of load cycles. The lower the value of total rut depth, the higher the resistance of asphalt mixture against rutting distresses. Different criteria have been developed to specify the number of load cycles representing the rutting resistance of asphalt mixtures. As Nebraska's HWT test criteria is still under development, the Illinois criteria has been considered in this study based on the similarities in climate condition and binder grade utilization in both states. Accordingly, Illinois evaluated the total rut depth at 5,000 load cycles for PG 58-xx (or lower), and total rut depth at 7,500 load cycles for PG 64-xx binder types [49]. With that, these parameters have been selected for finding the total rut depth of different asphalt mixtures and for comparison.

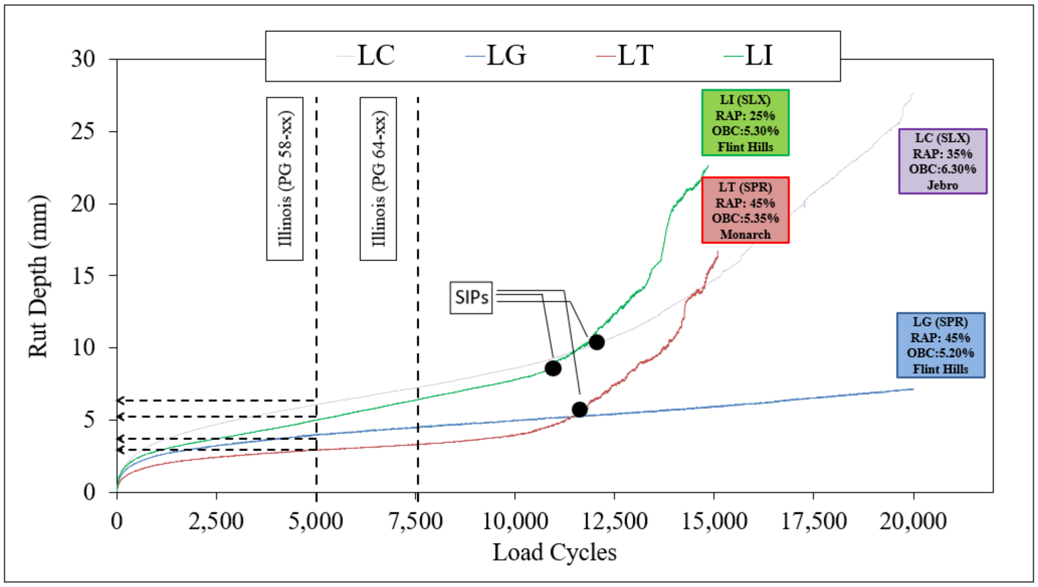


Figure 4.5 Total rut depth versus load cycles results for the HWT test

Figure 4.6 indicates the total rut depth values at 5,000 load cycles for different PMLC and PMFC specimens. Based on Figure 4.6a, LT had the lowest rut depth at 5,000 passes with 2.93 mm, followed by LG with 3.99 mm. This superior rutting performance might have been related to the presence of 45% RAP in the mix design of LT and LG. The RAP materials can improve the rutting resistance of asphalt mixtures due to the stiffening effect of aged binders present in their structure [50]. LI and LC had total rutting depths of 5.01 and 6.09 mm at 5,000 passes, respectively, which indicates they had better rutting performance compared to LT and LG. LC had 10% more RAP in its structure than LI, however, checking the JMF of both specimens showed a higher optimum binder content of LC compared to LI (6.3% for LC, and 5.3% for LI). This higher binder content appeared to be effective in reducing rutting resistance of the LC compared to the LI one.

Analyzing total rutting depth at 5,000 passes for PMFC specimens (Figure 4.6b) indicated lower rut depth values for FT specimens compared to FC ones, which was compatible with the results derived from lab compacted specimens. Further, comparing 12-month with 6-month cores showed a reduction in the rut depth for both types of specimens. More to this point, in case of FC specimens, FC6 had a total rut depth of 6.89 mm while the value dropped to 5.85 mm for the FC12 specimen. A similar drop happened in the case of FT, as FT6 and FT12 had a total rutting depth of 3.79 and 2.86 mm, respectively. It was an expected result, as oxidation continued to stiffen the asphalt mixture during the service life of the pavement [51].

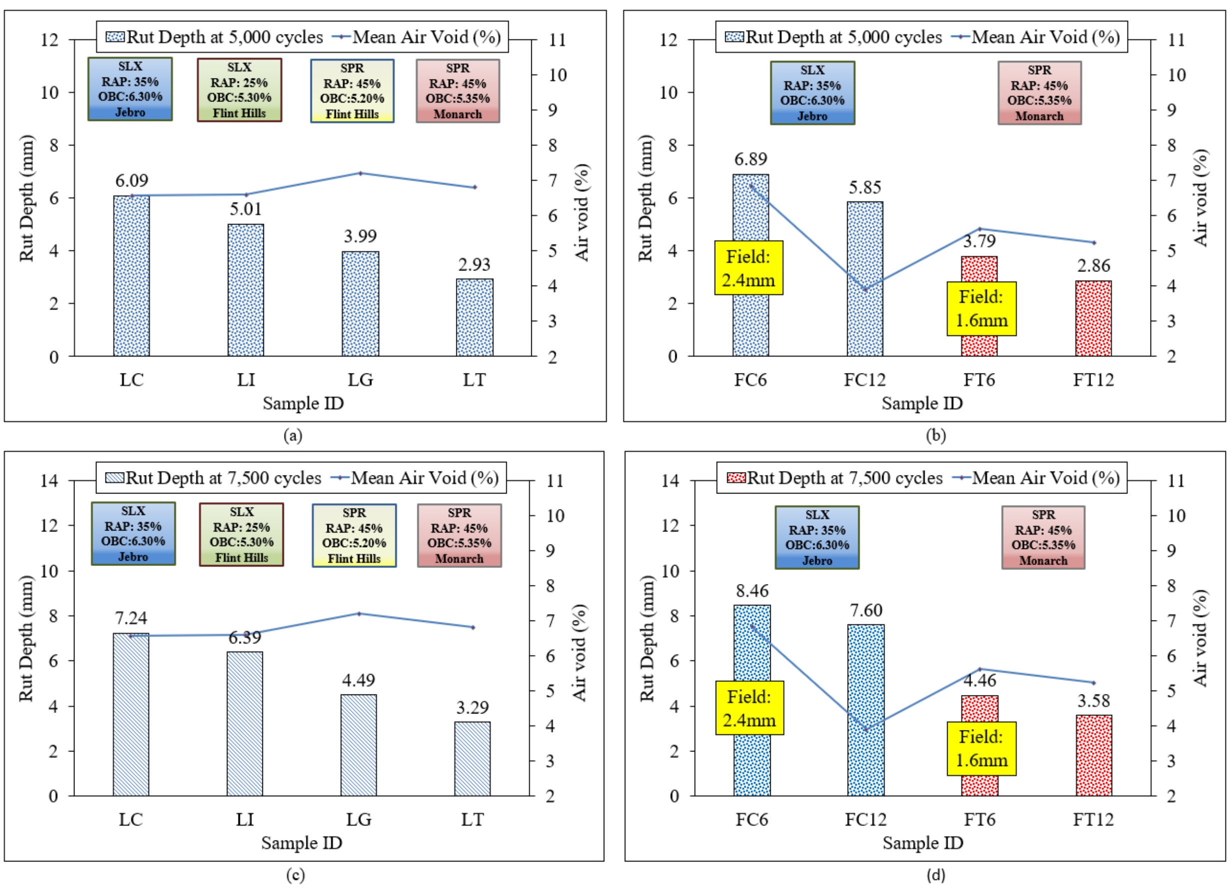


Figure 4.6 Total rut depth at (a) 5,000 passes for the PMLC, (b) 5,000 passes for PMFC, (c) 7,500 passes for PMLC, and (d) 7,500 passes for PMFC specimens

Analyzing total rutting depth after 7,500 passes (Figure 4.6c and d) also showed the same trends for both PMLC and PMFC specimens. Based on the results, all mixtures had total rutting depth values of less than 12.5 mm, which indicated that these common types of mixtures in Nebraska can meet the Illinois rutting criteria (<12.5 mm at 7,500 passes). Another interesting point from Figure 4.6 is that when comparing PMLC and PMFC specimens, lab-compacted results were very similar to 12-month field core results. For instance, after 7,500 passes, LC and FC12 specimens had rut depth values of 7.24 and 7.60 mm, respectively. Further, for LT and FT12 specimens, rut depth values were 3.29 and 3.58, respectively. This indicated that, in terms of rutting performance, lab-compacted specimens closely resembled 12-months field cores. However, to make a solid conclusion about this observation, more data is required (Phase 2 of this study).

The data derived from pavement condition monitoring are also shown in yellow boxes in Figure 4.6. This data was collected from pavement surfaces after six months of service life at the same reference points where loose mixtures and field cores were collected. Comparing six-month pavement condition data with six-month field core results for the two available projects indicated that a higher rutting depth in the actual pavement surface (2.4 mm) was attributed to the FC6 specimen that had lower rutting resistance. Further, in the case of the FT6 specimen with higher rutting resistance, a rutting depth of 1.6 mm was reported based on pavement condition monitoring data. Overall, pavement condition data supported the results derived from field core samples up to the time of reporting. The field data collection is going on annually to further validate the results derived from the HWT test.

As previously discussed in Section 3.4.1, a new approach was also utilized to analyze the HWT test results [25]. To this end, a polynomial curve was fitted on the typical rut depth vs. load cycle plot for each test specimen (Figure 4.7). Afterward, the point where negative curvature turned into positive was determined and selected as the LC_{SN} point. To quantify rutting resistance, the viscoplastic strain vs. load cycle curve was plotted up to the LC_{SN} point and the curve was projected into the stripping phase using the Tseng-Lytton model. With that, the slope of this projected curve was measured at 10,000 load cycles; this number was reported as the viscoplastic strain increment ($\Delta\varepsilon^{vp}_{10,000}$) to quantify the rutting resistance of asphalt mixtures. Figure 4.8 shows the viscoplastic strain slope (VP slope) at 10,000 load cycles for different specimens.

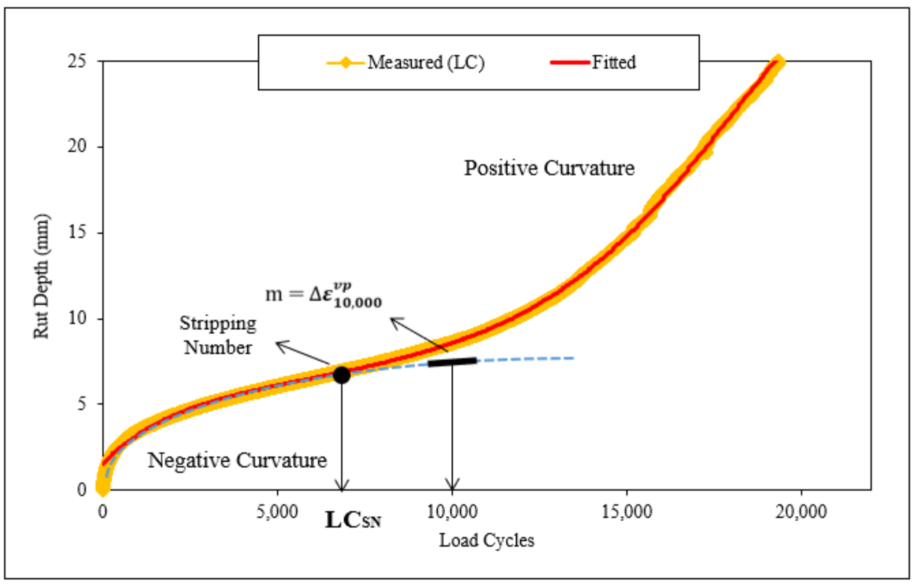


Figure 4.7 Determination of viscoplastic strain slope and load cycles to stripping number for LC mixture

As was previously mentioned, the lower the $\Delta\varepsilon^{vp}_{10,000}$ value, the higher the rutting resistance of specimens. As can be seen in Figure 4.8a, in terms of PMLC specimens, the lowest VP slope (highest rutting resistance) was associated with the LT specimen followed by LG with values of 1.49 and 2.62, respectively. On the other hand, the highest VP slope was observed in the LC specimen with a value of 4.83, followed by LI with a VP slope of 2.74. Generally, the results obtained from this new analysis method were in total agreement with the traditional method which was based on rutting depth at a certain number of passes. As a result, the new method appeared to be capable of analyzing the HWT test results by separating the effects of rutting and moisture damage. This can be beneficial, specifically, in the case of mixtures with low moisture damage resistance, in which the stripping inflection point shows up at a lower number of load cycles.

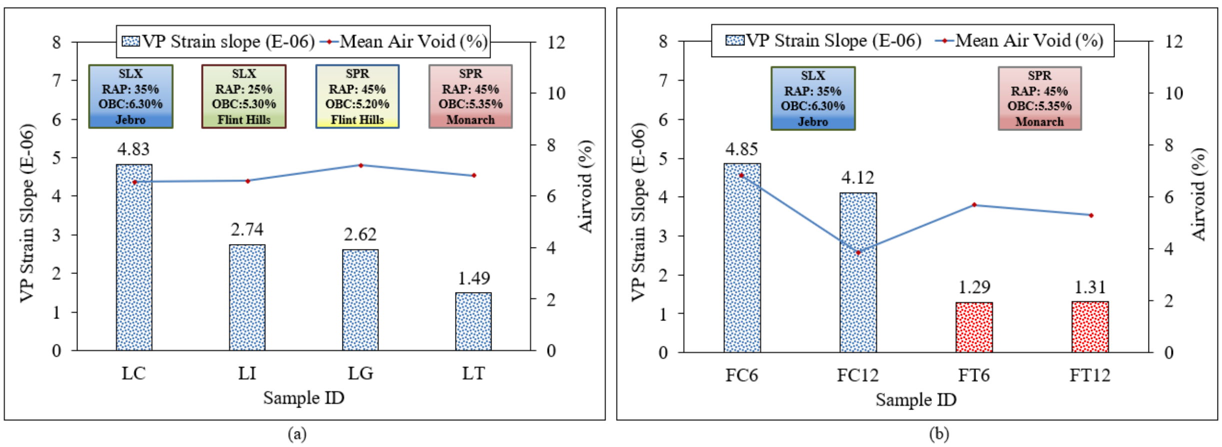


Figure 4.8 Viscoplastic strain slope in the HWT test for (a) PMLC and (b) PMFC specimens

In addition, results derived from the HWT test and fatigue cracking tests were expected to show an opposite trend, as generally these properties require opposite characteristics from asphalt mixtures. In the case of HWT and I-FIT tests, this opposite trend was observed, as LT had the best rutting resistance and had the worst cracking resistance, while LC with the worst rutting performance indicated the best performance in terms of fatigue cracking. In the case of LG and LI, a similar opposite trend is observed, as LG had better rutting resistance compared to LI, but was also more susceptible to fatigue cracking distress. Regarding the HWT and IDEAL-CT tests, generally, the same trend was observed, except for the LI specimen, which was ranked first in terms of fatigue cracking resistance among all specimens, while it did not have the worst performance in terms of rutting (LC showed worse rutting resistance compared to LI).

4.2.2 Rapid Shear Rutting (IDEAL-RT) Test Results

The shear strength values derived from the IDEAL-RT test were calculated using Equation (3.1. Figure 4.9 shows the shear strength results for PMLC and PMFC specimens. The higher shear strength values were associated with better resistance against rutting distress [52]. As can be seen in Figure 4.9a, LT and LG specimens had shear strength values of 1.31 and 0.95 MPa, respectively, which showed more rutting resistance compared with the two other specimens. This can be attributed to the higher RAP content (45%) available in their mix design. LC had the highest binder content among all specimens (6.3%), and the lowest shear strength value, as expected.

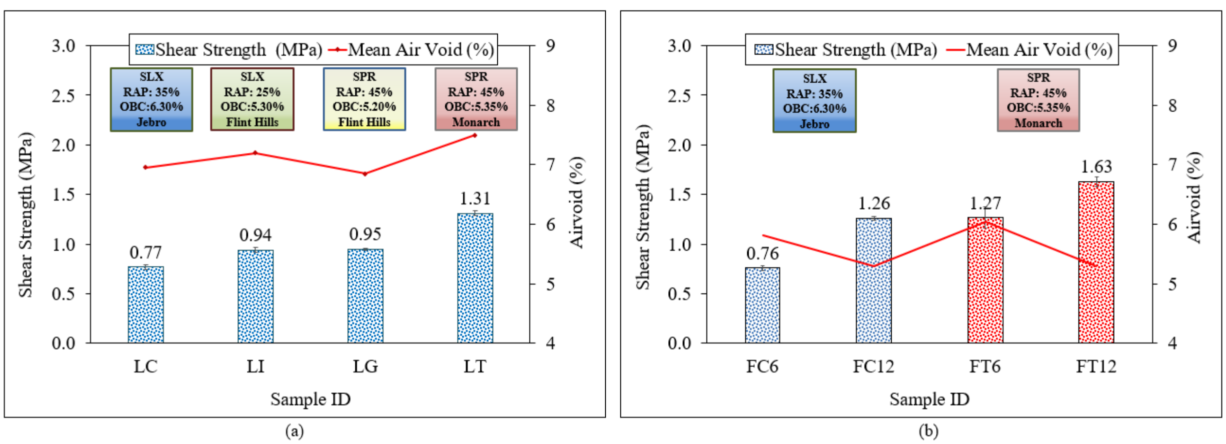


Figure 4.9 Shear strength values derived from IDEAL-RT test for (a) PMLC and (b) PMFC specimens

In terms of PMFC specimens, the results show an enhanced rutting resistance in both cases after 12 months of service life with six-month cores. Accordingly, FC6 and FC12 had shear strength values of 0.76 and 1.26 MPa, respectively; while FT6 and FT12 had shear strength values of 1.27 and 1.63 MPa, respectively. Overall, the results derived from the IDEAL-RT test correlated with the HWT test results. The statistical analysis and comparisons will be explored in Chapter 5 of the report.

4.2.3 High-Temperature Indirect Tensile Strength (HT-IDT) Test Results

Running the HT-IDT test, Equation (3.2 was applied to calculate the indirect tensile strength of specimens at a high temperature of 50 °C. The results for PMLC and PMFC specimens are depicted in Figure 4.10. Considering Figure 4.10a, LT and LC had the highest and lowest rutting resistance, respectively. The LI specimen had an IDT value in-between that of LT and LC totaling 0.17 MPa. This observation agreed with the HWT test result, however, in the case of LG the results were contradictory between these two test methods. More to this point, LG, with 45% RAP in the structure, was found to have an IDT value of 0.12 MPa, which was lower than the LI specimen. As the field cores are not collected for the Gresham project (G), it was impossible to compare the pavement condition data with field core results and make a solid conclusion about it.

Considering field core samples (Figure 4.10b), FC12 and FT12 showed some improvement in rutting resistance after six months (Compared with FC6 and FT6), however, the rate of improvement was much more noticeable for FT12. To be more specific, FT6 and FT12 had IDT values of 0.220 and 0.359, respectively, which shows 63% improvement in the IDT value after six months of service life. However, in the case of FC6 and FC12, the IDT values were 0.139 and 0.141, respectively, which showed less than 2% improvement. The statistical and comparison analyses are shown in Chapter 5.

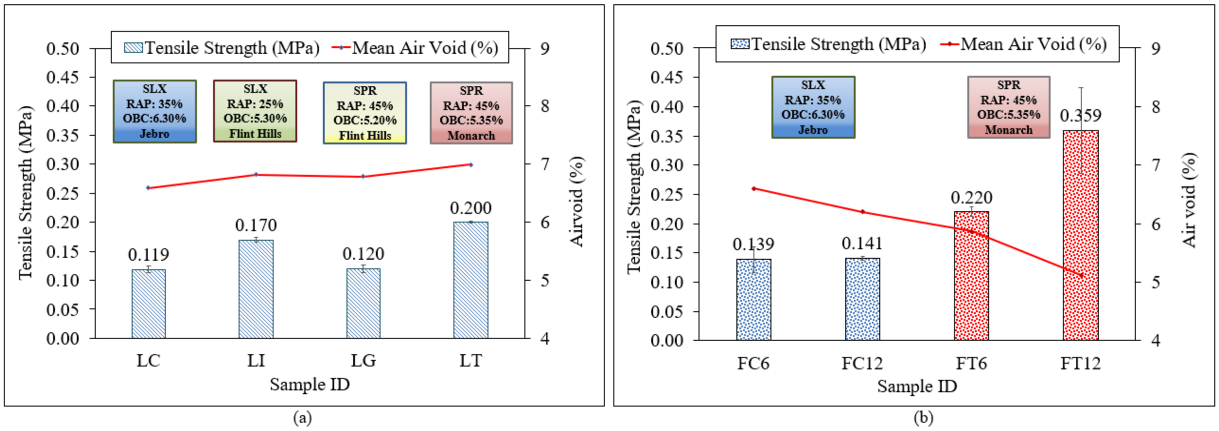
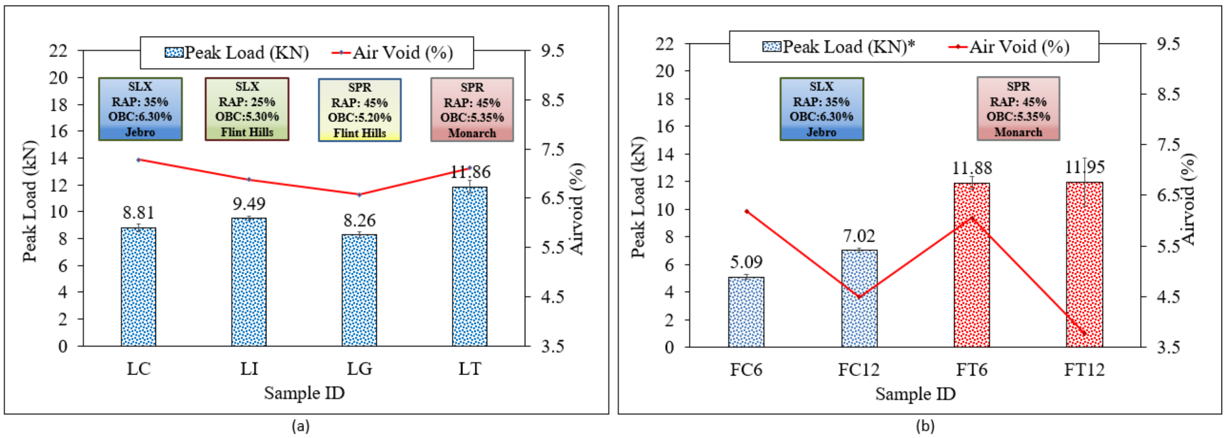


Figure 4.10 Indirect tensile strength values derived from HT-IDT test for (a) PMLC and (b) PMFC specimens

4.2.4 G-stability Test Results

During the G-stability test, peak load values were extracted for each specimen to compare the rutting resistance property. Figure 4.11 indicates the G-stability test results for both categories of PMLC and PMFC specimens. Before going through analysis, it should be noted that the G-stability test was developed for cylindrical specimens with a geometry of 150 × 50 mm. In this study, some of the PMFC specimens (field cores) had other geometries, in which, in some cases the thickness of samples was less than 50 mm. As the correction factor for specimens with other geometries is still under development, the results for field cores were reported without any correction factor. So, these results were not used for comparison purposes.

Based on Figure 4.11a, LT had the highest peak load value among all specimens, followed by LI, LC, and LG, respectively. LT had the best performance with a peak load value of 11.86 kN, while LG had the lowest performance in this test with a peak value of 8.26 kN. However, based on the standard error of mean showed on the graphs, there were no differences between LC and LG peak load values, and they can be ranked the same. To have better insight on the accuracy of this test, correction factors should be developed for field core specimens. Further statistical analyses are covered in Chapter 5 of this report.



*The peak load values reported for core samples are not corrected (The thickness of samples are less than 50 mm). The correction factor is under development.

Figure 4.11 Peak load values derived from G-stability test for (a) PMLC and (b) PMFC specimens

4.3 Moisture Damage Resistance Test Results

In BMD development in Nebraska, moisture damage resistance was also considered as a main distress that should be addressed at the time of design and production. To this end, four different performance tests have been considered to further evaluate moisture sensitivity of specimens. Two of these tests, Hamburg Wheel Tracking (HWT) and Tensile Strength Ratio (TSR), are common tests for assessing moisture damage resistance of asphalt concrete mixtures, while two surrogate tests called G-stability and High-Temperature Indirect Tensile Strength (HT-IDT) were also considered as potentially practical tests for analyzing the moisture damage resistance of asphalt concrete mixtures.

Figure 4.12 shows the moisture damage test results for all types of PMLC specimens. Considering the TSR test, LC had the highest TSR with a value of 94%, followed by LG with a

TSR value of 89%. Meanwhile, LT and LI were jointly standing in the third rank as the TSR value was 83% in both cases. With respect to the HWT test as another common method to analyze moisture damage resistance of asphalt mixtures, the SIP was reported as a measure of moisture sensitivity. Based on Figure 4.12, LG had superior moisture damage resistance, as the SIP did not appear until 20,000 passes. LC, LT, and LI had SIP values at 13,200, 10,950, and 10,460 passes, respectively. Based on the results, the TSR and HWT test trends were similar, except for LG and LC were switched for the first and second rank according to moisture damage resistance.

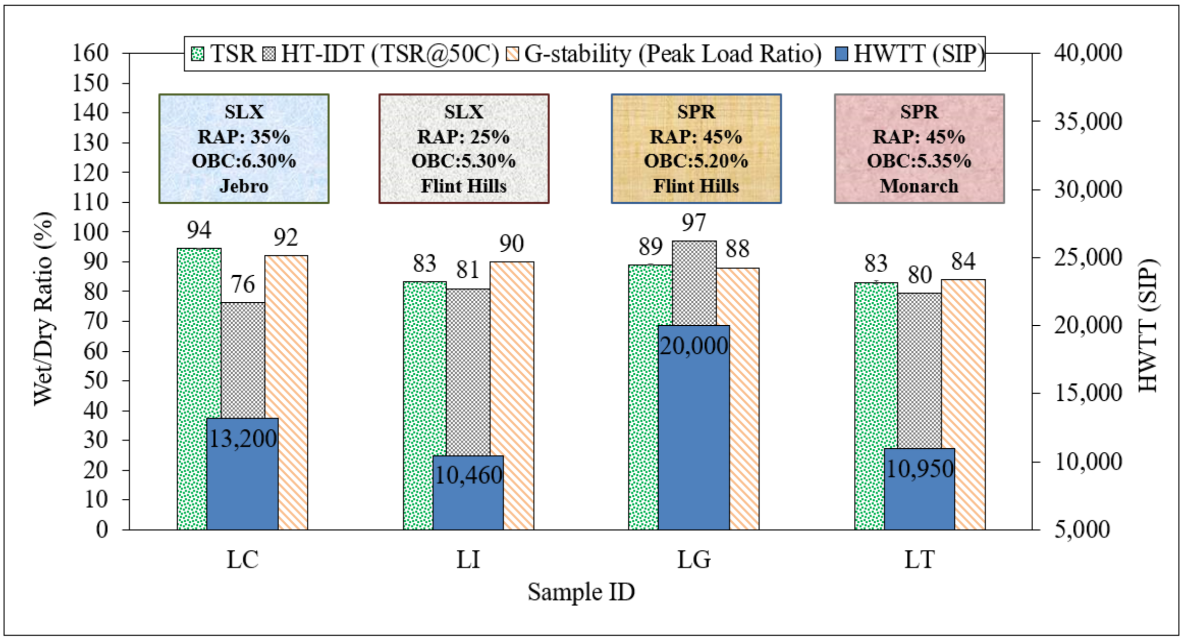


Figure 4.12 Moisture damage test results for PMLC specimens

With respect to the G-stability test results, the peak load ratio of conditioned over unconditioned specimens led to a similar trend with TSR test, except for LI specimen. With that, LC and LT had the highest and lowest moisture damage resistance in both tests, respectively. In the case of HT-IDT test, the specimens' rankings were different from well-established tests. For instance, LC had the best performance based on the TSR test, and the worst performance according to the HT-IDT test. The reason for this diversity of results might be due to the huge differences in influential factors associated with various tests in the study. To be more specific, the TSR test was fulfilled at 25 °C, while the other three tests were done at higher temperatures. The mechanisms were also different, for instance, the HWT test specimens were submerged in water, while in the other three tests dry and wet specimens were utilized. These factors can significantly affect moisture damage results. Overall, based on the limited amount of data derived from this study, G-stability test shows some correlations with the TSR test in predicting moisture damage resistance of AC mixtures. However, more investigation is required, particularly regarding the loading mechanism, to draw a solid conclusion.

Chapter 5 Statistical Analysis and Tests Comparisons

In this chapter, statistical analyses are provided for each performance tests method. The sensitivity of each test was assessed by applying Tukey's HSD method, while possible correlation between different tests was investigated through bivariate correlation analysis. Further, practicality, variability, complexity, and availability of different tests methods will be characterized in this chapter. Having these statistical analyses and practical information, along with the performance data derived from each test method in the previous chapter, makes it possible to select the most appropriate performance test for each type of distress in the Nebraska BMD framework.

5.1 Mid-temperature (Fatigue) Cracking Tests Comparison

The Tukey's honestly significant difference (HSD) test was conducted to determine if the results from I-FIT and IDEAL-CT tests showed a difference in terms of specimen types. The Tukey's HSD test was a practical tool for identifying differences in asphalt mixtures performances and was utilized by many researchers in the field [44], [53]. Due to the left skew of the responses, a log transformation was used to meet the normality assumptions. In Tukey's HSD if some group of mixtures show the same grouping letter in a specific test parameter, the mean values of that parameter were not statistically different. The results for Tukey's test for the flexibility index and CT_{Index} values are shown in Table 5.1.

With respect to Table 5.1, LG and LT were statistically different as they had different grouping letters, while there was no statistically significant difference between LC and LI. Considering the FI and CT_{Index} values (Figure 4.1 and Figure 4.3), LC and LI both had an acceptable performance in terms of fatigue cracking. Based on FI results, LC performed better than LI, however, CT_{Index} values showed higher cracking resistance of LI compared to LC. As lower RAP content and higher binder content were two influential factors in improving cracking resistance of asphalt mixtures, it was possible to justify the results from both tests. In fact, compared to LI, LC had 10% more RAP and 1% more binder content. As a result, the higher binder content in LC can improve its cracking resistance, while at the same time, lower RAP content in LI can enhance the cracking resistance of this specimen. Overall, the Tukey's HSD test results verified that LC and LI were not statistically different in terms of FI and CT_{Index} values.

Considering FI values for the field core specimens, FC6 and FC12 shared the same group letter (a). Further, the same group (cd) was also shared by FT6 and FT12, which indicated there was no evidence of difference between FI values of 6-month and 12-month core specimens. However, in terms of CT_{Index} values, FC6 and FC12 had different groups (a and bc), and FT6 and FT12 had different groups (bc and de). This observation, in turn, showed potentially a higher sensitivity to the field aging effects in IDEAL-CT, compared to the I-FIT test. However, to verify this result, long-term field data are required during years of service life. With that, it would be possible to compare the field core test results and pavement condition data during service life and make a solid conclusion about the sensitivity of each cracking test.

Table 5.1 Tukey's HSD grouping for Flexibility index and CT_{Index}

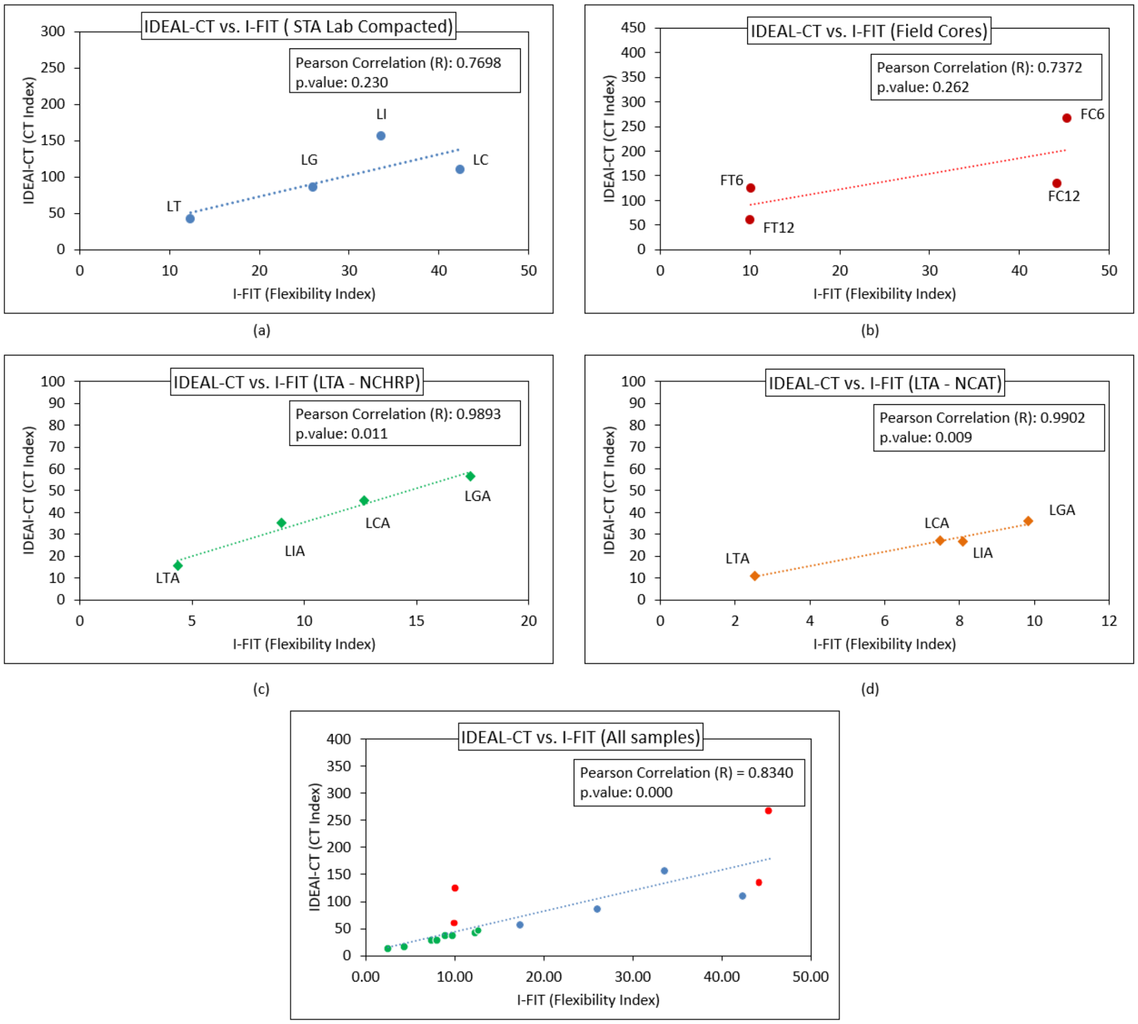
Flexibility Index (I-FIT test)			CT_{Index} (IDEAL-CT Test)		
Specimen ID	Response	Groups	Specimen ID	Response	Groups
LC	42.4	a	LC	109.2	bc
LI	33.6	a	LI	154.8	ab
LG	26.1	ab	LG	84.9	cd
LT	12.3	cd	LT	41.4	e
FC6	45.3	a	FC6	266.4	a
FC12	44.2	a	FC12	134.2	bc
FT6	10.1	cd	FT6	117.7	bc
FT12	10.0	cd	FT12	58.6	de
LCA (NCHRP)	12.6	cd	LCA (NCHRP)	44.9	e
LIA (NCHRP)	9.0	d	LIA (NCHRP)	34.4	e
LGA (NCHRP)	17.4	bc	LGA (NCHRP)	56.7	de
LTA (NCHRP)	4.3	e	LTA (NCHRP)	15.7	f

To further investigate the relationship between IDEAL-CT and I-FIT tests, a correlation effort was fulfilled to define the possible compatibility between these two test methods. To this end, the FI and CT_{Index} values of lab-compacted and field core specimens were compared to each other using bivariate (Pearson) correlation analysis. The coefficient obtained from this method varied between -1 and +1, in which, the exact values of +1 and -1 indicated the perfect direct or inverse relationship between parameters, respectively. This method had been widely used by many researchers in the asphalt mixture realm to identify the relationship between different parameters or test methods [5], [54]. Figure 5.1 shows the Pearson's correlation between FI and CT_{Index} mean values for different types of specimens, while Table 5.2 represents the ranking of LTA specimens based on different tests for fatigue cracking resistance.

Table 5.2 Ranking of LTA specimens' fatigue cracking resistance

Rank (Based on LTA conditioning)	IDEAL-CT	I-FIT
1 (best)	Gresham (G)	Gresham (G)
2	Crofton (C)	Crofton (C)
3	I-Bridge (I)	I-Bridge (I)
4	Tekamah (T)	Tekamah (T)

As can be seen in Figure 5.1a, there was a correlation between FI and CT_{Index} for the STA lab-compacted specimens with a Pearson correlation value (R) of 0.7698. With respect to the field cores (Figure 5.1b), the value of Pearson's correlation was equal to 0.7372 showing a direct relationship between FI and CT_{Index} results. However, to have better insight about fatigue cracking performance of asphalt mixtures, long-term aged specimens should be taken into consideration. It was an established argument that long-term aging was a major factor controlling the fatigue performance of asphalt mixtures [55]. With that, Figure 5.1c and d indicate the correlation between FI and CT_{Index} values of the LTA lab-compacted specimens. Accordingly, there was evidence of correlation between FI and CT_{Index} values for the LTA specimens (NCHRP 09-54 protocol) with a Pearson correlation of 0.9893 (p. value = 0.011). Also, in the case of LTA specimens following the NCAT protocol, a Pearson correlation of 0.9902 was achieved (p. value = 0.009) which showed a strong direct relationship between I-FIT and IDEAL-CT test results after a long-term aging processing. The same ranking of specimens' cracking performance after long-term aging (Table 5.2) further proved the relationship between these two cracking test methods. Finally, the combination of all specimens with 16 data points shows a Pearson correlation of 0.8340 (p. value < 0.001), which stablished a strong correlation between two tests with a 95% confidence level (Figure 5.1e).



(e)

Figure 5.1 Relationship between FI and CT_{Index} for (a) STA lab-compacted, (b) field cores, (c) LTA-NCHRP 09-54, (d) LTA-NCAT, (e) combination of all specimens

With a correlation established between I-FIT and IDEAL-CT, a model was developed to predict CT_{Index} using the FI values. As noted in Figure 5.1, CT_{Index} appears to have increased in variability as the FI increased. As a result, a log transformation was utilized on the CT_{Index} to develop the model. Equation (5.1 represents a general model that combined all types of specimens together (R^2 = 0.7039):

$$\ln(CT_{Index}) = 2.711 + 0.071 \times (FI) \tag{5.1}$$

Where CT_{Index} is derived from IDEAL-CT test for all types of specimens, and FI is the flexibility index value derived from I-FIT test.

Table 5.3 Model summary for fatigue cracking tests

Term	Estimate	Std. Error	Statistic	p. value
Intercept	2.711	0.144	23.574	0.000
Flexibility Index	0.071	0.005	7.319	0.000

The model summary can be found in Table 5.3. Overall, the results obtained from statistical analysis, particularly correlations between two fatigue cracking test methods, indicated that IDEAL-CT and I-FIT tests showed a strong relationship to each other. Specifically, the results after long-term aging, which was the critical condition for fatigue cracking performance of asphalt mixtures, verified the direct relationship between these two test methods.

5.2 Rutting Test Methods Comparison

In terms of rutting resistance, four different test methods including HWT, IDEAL-RT, HT-IDT, and G-stability were conducted on asphalt concrete specimens. Since there was only one observation for each type of specimen in the HWT test, the data were insufficient to make comparisons between different specimen IDs. This was because statistical models need replications of individual units to quantify uncertainty. As a result, for the sensitivity analysis, comparisons were only made between different types of specimens (PMLC and PMFC). As the HWT test was considered a well-stablished method in this project and the results from other tests were compared to it, only specimens from the first replicate groups were considered for all test methods.

Due to the small sample size with the exclusion of observations without the HWT measurement, the Wilcoxon Rank Sum (WRS) test was used as an alternative to a t-test. The WRS test was a non-parametric test that was suitable in situations where the data was skewed or there was a small sample size. The boxplots of PMLC and PMFC specimens for the different rutting test methods are shown in Figure 5.2. Red dots represent individual data points for each type of specimen and test method. As can be seen in Figure 5.2, only the G-stability test had evidence of a difference between lab-compacted and field core specimens. The p. value of the WRS test for the difference between specimen types was equal to 0.043 in the G-stability test. However, in the case of HWT, IDEAL-RT, and HT-IDT the p. values were equal to 0.773, 0.248, and 0.772, respectively. It is important to note that due to the small sample size (4 per sample type), the data had higher variability that limited the ability to detect smaller differences between groups.

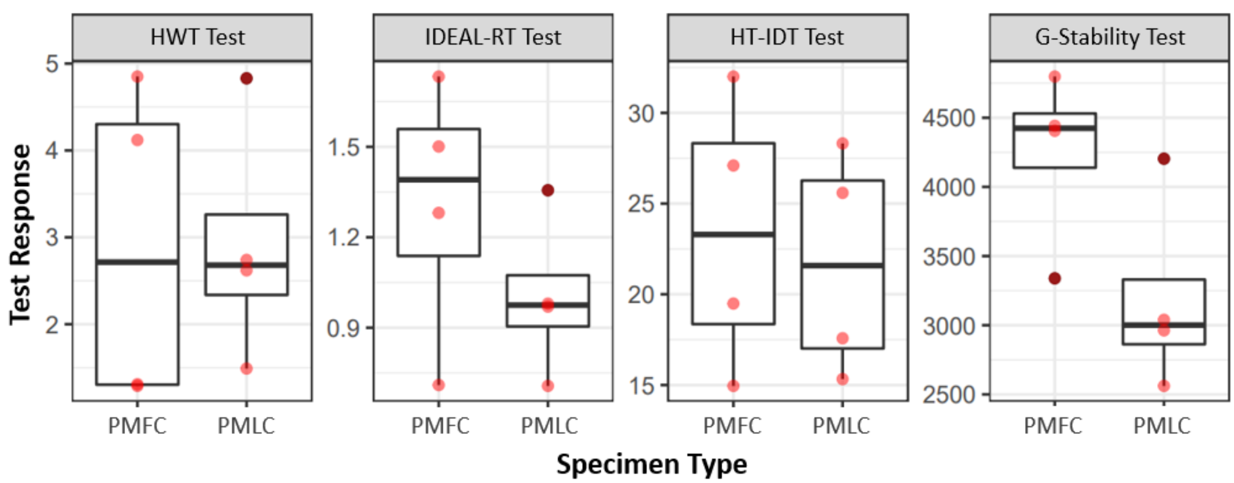


Figure 5.2 The boxplots derived from Wilcoxon Rank Sum test for PMLC and PMFC specimens in different rutting tests

To further investigate the relationship between the HWT test and other surrogate tests, a correlation effort was carried out to find the possible compatibility between different tests. To this end, using bivariate (Pearson) correlation analysis, HWT test results were compared to the IDEAL-RT, HT-IDT, and G-stability test results for both PMLC and PMFC specimens. Considering Illinois criteria, a rutting depth at 7,500 passes was selected as the main parameter obtained from the HWT test. This was because the main binders used in this study were PG 58H-34 and PG 58V-34 which were comparable to the PG 64-xx used in the Illinois criteria selection. Table 5.4 shows the ranking of PMLC specimens based on different test methods.

Table 5.4 The ranking of PMLC specimens based on HWT, IDEAL-RT, HT-IDT, and G-stability tests

Rank	**HWT Test (Rut @ 7,500 passes)**	**IDEAL-RT (Shear Strength (MPa))**	**HT-IDT (Tensile Strength (MPa))**	**G-stability (Peak Load (KN))**
1st (Best)	Tekamah (T)	Tekamah (T)	Tekamah (T)	Tekamah (T)
2nd	Gresham (G)	Gresham (G)	I-Bridge (I)	I-Bridge (I)
3rd	I-Bridge (I)	I-Bridge (I)	Gresham (G)	Crofton (C)
4th	Crofton (C)	Crofton (C)	Crofton (C)	Gresham (G)

Considering Table 5.4, in the case of HWT and IDEAL-RT tests, a similar trend with the same ranking was achieved for different specimens' rutting performances. More than that, based on Figure 5.3a and b, there was evidence of a correlation between rut depth and shear strength for PMLC and PMFC specimens with Pearson correlation values (R) of -0.8939 and -0.8398, respectively. With respect to all specimens combined (Figure 5.3c), the Pearson correlation value of -0.6885 also indicated a strong inverse relationship between rut depth and shear strength in the HWT and IDEAL-RT tests, while the p. value was less than 0.05 (95% confidence level).

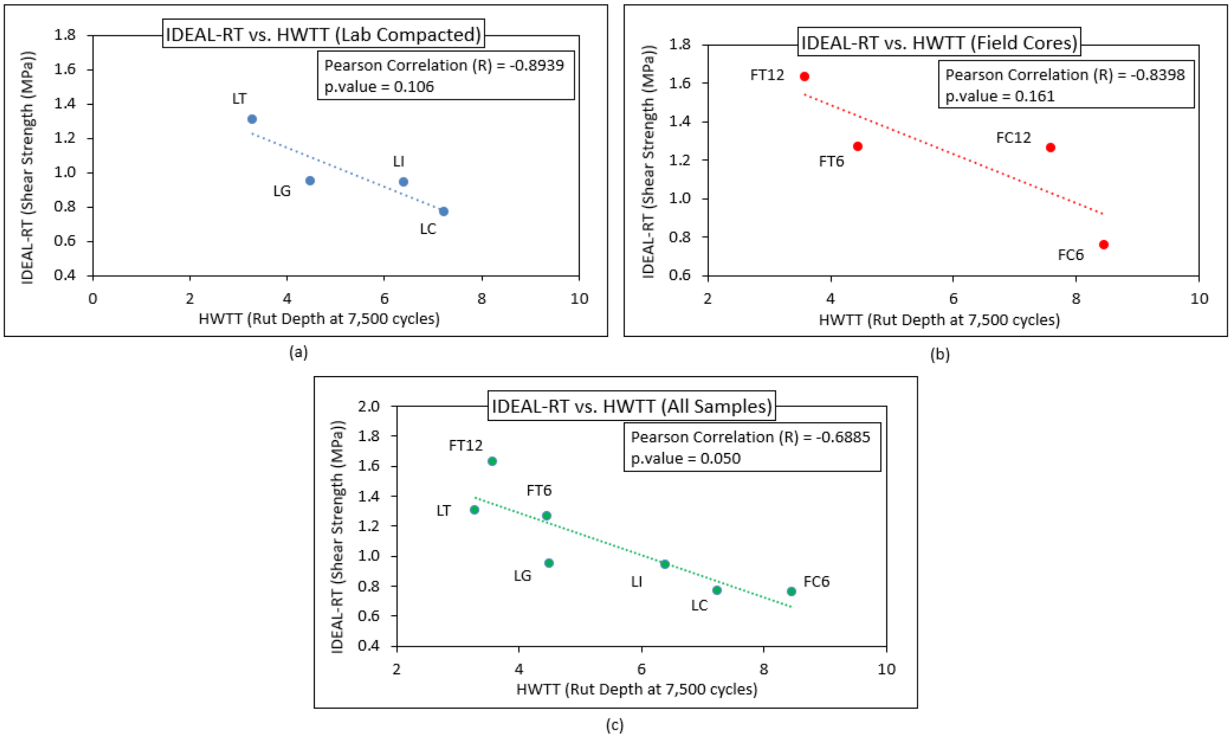


Figure 5.3 Relationship between HWT and IDEAL-RT tests for (a) PMLC, (b) PMFC, (c) all specimens

When comparing HWT and HT-IDT, LT and LC showed the best and worst performance, respectively, in both tests (Table 5.4). However, the LG and LI rankings were switched in the HT-IDT test compared to the HWT test. With respect to Figure 5.4a and b, the evidence of correlation between rutting depth and tensile strength had a Pearson correlation (R) of -0.5402 for PMLC specimen, and a correlation of -0.8910 for PMFC specimen. In Figure 5.4b, similar results for the tensile strength values of FC6 and FC12, significantly improved the correlation value. At the same time, in all other rutting performance tests, there was more difference between the results for FC6 and FC12 rutting parameters. Rut depth obtained from pavement condition data after 6 months and 12 months of service life could be helpful, however, due to the logistic issues, the data were available only after 6 months of service life. Overall, all the correlations between HWT and HT-IDT tests (Figure 5.4a, b, c) had p. values of higher than 0.05.

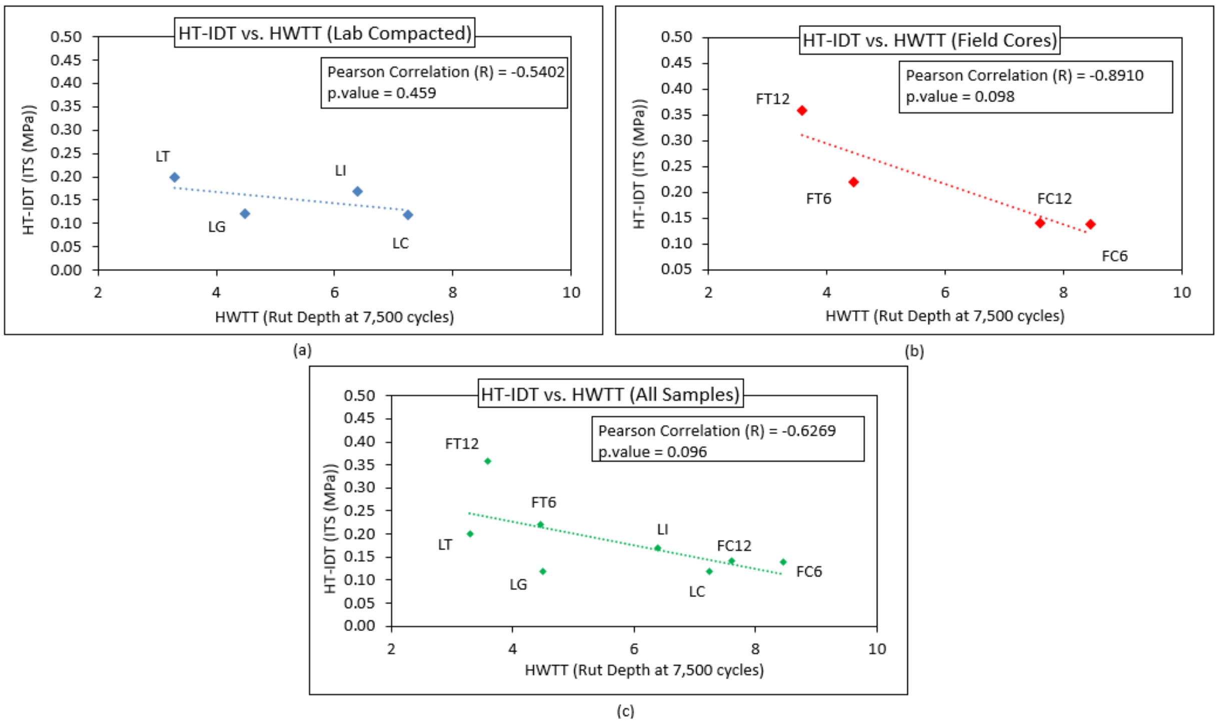


Figure 5.4 Relationship between HWT and HT-IDT tests for (a) PMLC, (b) PMFC, (c) all specimens

Taking into account HWT and G-stability tests, Figure 5.5a indicates the relationship between parameters derived from these tests for the PMLC specimens. Accordingly, there was evidence of correlation between rutting depth and peak load value with a Pearson correlation (R) of -0.5996, while, in the case of PMFC specimen, it was not possible to consider the relationship, as the values were reported without considering correction factors. Further, with respect to Table 5.4, HWT and G-stability both showed LT as the best specimen in terms of rutting resistance. However, the rank of LI, LC, and LG were switched in G-stability compared to the HWT test method. Overall, as the correction factor for the G-stability test is still under development, it was impossible to make a solid conclusion about the relationship between this test and the HWT test at the time of reporting.

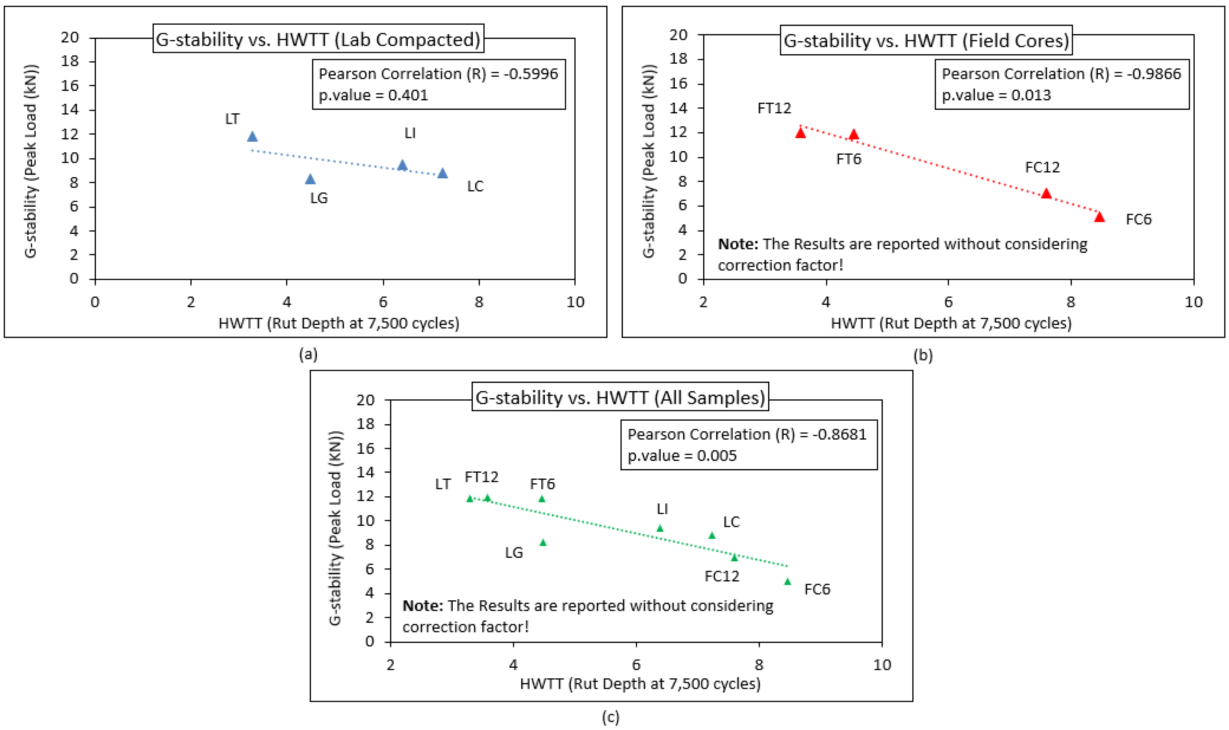


Figure 5.5 Relationship between HWT and G-stability tests for (a) PMLC, (b) PMFC, (c) all specimens

5.3 Moisture Susceptibility Tests Comparison

To investigate moisture damage resistance of asphalt mixtures, TSR and HWT were considered as two established tests, along with HT-IDT and G-stability as two potentially surrogate performance tests. In the case of HWT test, SIP was extracted from the test results, while for the other three tests, different parameters were obtained for the conditioned (wet) and unconditioned (dry) specimens, and the ratio of the wet over dry was reported as an index for moisture damage resistance. As the comparison of interest was the ratio of average values for each testing type and condition, a sensitivity analysis needed to have multiple replicates of the ratio of averages for each specimen type. As a result, the current data did not allow for the comparison of ratios for average wet to average dry responses.

Table 5.5 The ranking of PMLC specimens for moisture damage resistance based on TSR, HWT, HT-IDT, and G-stability tests

Rank	TSR Test (Wet/Dry Ratio)	HT-IDT Test (Wet/Dry Ratio)	HWT Test (SIP)	G-stability Test (Wet/Dry Ratio)
1^{st} (Best)	Crofton (C)	Gresham (G)	Gresham (G)	Crofton (C)
2^{nd}	Gresham (G)	I-Bridge (I)	Crofton (C)	I-Bridge (I)
3^{rd}	Tekamah (T)	Tekamah (T)	Tekamah (T)	Gresham (G)
4^{th}	I-Bridge (I)	Crofton (C)	I-Bridge (I)	Tekamah (T)

The performance ranking of different specimens based on each testing method is provided in Table 5.5. Different rankings were obtained for each specimen type in terms of moisture damage resistance. For instance, LG had the best moisture damage resistance based on HT-IDT and HWT test results, while LC proved to be the best according to TSR and G-stability tests. Using the average wet/dry specimens' ratio as the response, bivariate correlation analysis was applied to find out the possible relationship between different test methods. Based on Pearson's correlation analysis (Table 5.6), there was no evidence of a correlation between the TSR test and the two other surrogate tests (G-stability and HT-IDT). In comparison between the HWT test and the two surrogate tests, there was also no evidence to support a correlation between results (p. values were reported to be high). However, considering Table 5.5, TSR and G-stability tests showed similar ranking for Crofton, Gresham, and Tekamah projects. Based on the limited amount of data derived from this study, G-stability might have some correlation with the TSR test, and it can be considered for future studies in analyzing moisture resistance performance tests.

Table 5.6 Pearson's correlation between different moisture damage resistance test methods

TSR Correlation	**Pearson's correlation (R)**	**p. value**	**HWT Correlation**	**Pearson's correlation (R)**	**p. value**
HT-IDT Test	-0.011	0.981	HT-IDT Test	0.672	0.128
G-stability Test	0.652	0.348	G-stability Test	0.127	0.942

Chapter 6 Research Conclusion and Future Works

This study was focused on developing a balanced mix design for the possible implementation in the state of Nebraska. As the first technical step to having a framework for the BMD is having appropriate performance tests, several testing methods including well-established tests and surrogate tests have been investigated in this research plan. To this goal, three main types of asphalt pavement distresses including rutting, fatigue cracking, and moisture damage were considered in the plan and several performance tests were utilized to characterize these distresses on common asphalt mixtures in Nebraska. The research conclusion is provided in the following section.

6.1 Research Conclusion

Effective factors for selecting appropriate performance tests include, but are not limited to, accuracy, sensitivity, variability, practicality, required time, cost-effectiveness, and field validity. Results were provided for every individual test in Chapter 4, and the statistical analysis for comparing different test methods were provided in Chapter 5. Table 6.1 and Table 6.2 present a summary of different performance tests evaluated for the Nebraska BMD obtained from these results and analyses.

Table 6.1 Summary of rutting performance tests characteristics

Test	Test Method	Cost	Testing time (without SGC)	Data analysis complexity	Test variability	Field validity	Practicality	Possibility for other distress
Hamburg Wheel-Tracking Test (HWTT)	AASHTO T 324-19	$45,000-70,000 (Including saw machine)	8-9 h (Including cutting and conditioning)	Fair	10-30% COV	More data required!	Good for Mix Design Fair for QA/QC	Moisture damage
High Temp. Indirect Tension (HT-IDT)	ALDOT 458	$8,000-10,000 (Including water bath)	1 min per sample + 2 h conditioning	Simple	Less than 10% COV		Good for Mix Design Good for QA/QC	Moisture damage
Rapid Shear Rutting Test (IDEAL-RT)	Draft ASTM (WK 71466)	$8,000-10,000 (Including water bath)	1 min per sample + 2 h conditioning	Simple	Less than 7% COV		Good for Mix Design Good for QA/QC	-
Gyratory stability (G-stability)	NDOT-SPR-P1(19) M080 Report	$8,000-10,000 (Including water bath)	1 min per sample + 2 h conditioning	Simple	Less than 7% COV		Good for Mix Design Good for QA/QC	Moisture damage

Three surrogate performance tests were evaluated for rutting resistance, and the results were compared with field performance and one well-established rutting test. Considering Table 6.1, IDEAL-RT, HT-IDT, and G-stability tests are much more cost-effective in comparison with the HWT test. The equipment required for running each of these tests is available in most asphalt laboratories around the state, and it is possible to make them functional with minimum adjustments to the bottom fixture of the test apparatus. With respect to time, all these surrogate tests can be fulfilled within a few minutes after two hours of sample conditioning. On the other hand, the required time for specimen fabrication and the testing procedure for the HWT test can exceed eight hours, which makes it difficult, if not impossible, to consider this test for the production phase and QC/QA purposes in a BMD framework. Accordingly, NCHRP 20-07 reported time as one of the main concerns of the state transportation agencies for BMD implementation, specifically, in the QC/QA phases [56].

With respect to test variability, HT-IDT results in this study show a Coefficient of Variance (COV) of less than 10 percent, while for the IDEAL-RT and G-stability tests (only PMLC specimens were considered for G-stability test) the COV was less than 7 percent. The COV of the HWT test was reported from other studies to range from 10 to 30 percent [43]. This higher test variability, along with more complexity in terms of data analysis, make this test more challenging for possible implementation in the BMD of Nebraska. At the same time, considering the relationship between HWT and other surrogate tests, IDEAL-RT had the highest evidence of correlation followed by HT-IDT and G-stability tests, respectively. Overall, IDEAL-RT, HT-IDT, and G-stability are more practical tests for both design and production phases in the BMD framework.

Table 6.2 Summary of mid-temperature (fatigue) cracking performance tests characteristics

Test	Test Method	Cost	Testing time (without SGC)	Data analysis complexity	Test variability	Field validity	Practicality	Possibility for other distress
Illinois Flexibility Index Test (I-FIT)	AASHTO T 124-20	$12,000-18,000 (Including cut & saw machines)	1 min + 4 hours cutting and conditioning	Simple	Less than 25% COV	More data required!	Good for Mix Design Fair for QA/QC	Moisture damage
Indirect Tensile Asphalt Cracking Test (IDEAL-CT)	ASTM D8225-19	$6,000-8,000	1 min per sample + 2 h conditioning	Simple	Less than 10% COV		Good for Mix Design Good for QA/QC	Moisture damage

Table 6.2 summarizes the fatigue cracking resistance characteristics associated with I-FIT and IDEAL-CT tests. As can be seen, IDEAL-CT test equipment costs around $7,000 while the I-

FIT test costs increase because of the need to use saw machines in sample preparation. Additionally, this cutting and notching procedure required in the I-FIT test not only raises the testing time to almost twice the IDEAL-CT test, but it also has an increased variability in test results. Specifically, the COV of the I-FIT test is reported to be around 25% while the IDEAL-CT has a COV of less than 10% obtained in this study.

The complexity of the data analysis is simplified in both tests if the apparatus is equipped with data analysis software. However, both tests have a fair amount of data analysis complexity for hand calculations. Accounting for the relationship between two tests, evidence of a strong correlation was obtained, specifically in the case of long-term aged specimens. With respect to all these parameters, IDEAL-CT is a “good” test for both design and production phases in the BMD, while the I-FIT test is categorized as “good” for design, and “fair” for the production and QC/QA phases.

With respect to moisture damage resistance tests, no strong evidence of correlation was observed between surrogate tests and established ones, however, G-stability and TSR tests showed similar rankings for the performance of three out of four projects As it was impossible to have sensitivity analysis using statistical methods, and no field data was available to check the field validity, it was impossible to make a conclusion about appropriate surrogate test for the moisture susceptibility assessment, at this stage of the study. Among established tests, TSR is the one that is currently used in Nebraska with a criterion that is widely accepted by state DOTs. Overall, the major findings of this study can be summarized as follows:

- Considering rutting performance tests, HWT as a well-established and valid test, is also costly and time-consuming which makes it difficult, if not impossible, to apply this test at the production phases in the BMD framework. As such, it is recommended that NDOT switch from HWT to a more practical test or continue considering HWT and one surrogate test for the rutting characterization of asphalt concretes in the BMD of Nebraska.

- Among different surrogate tests evaluated for the rutting characterization, the IDEAL-RT test results are highly correlated with the established test, followed by the HT-IDT and G-stability tests. As a result, based on limited data derived from this study, the IDEAL-RT test is a good candidate for the BMD framework in Nebraska; however, long-term field data, along with a more extensive scope of materials for future research are required to make a solid conclusion.
- Considering fatigue cracking performance tests, I-FIT as a well-established test was found to have high variability in terms of results which can be due to the complex procedure of sample fabrication in this test. Plus, considering costs and required time associated with this test, it is recommended that NDOT switch from I-FIT to IDEAL-CT as a well-correlated surrogate test, or alternatively, consider both tests for the BMD framework in Nebraska. However, to make a solid conclusion, long-term field data are required.
- Considering moisture damage resistance tests, except for G-stability and TSR tests, no strong relationship was found between tests. Future studies can consider G-stability, as well as other surrogate tests and different conditioning protocols for the moisture damage characterization of asphalt mixtures.
- For the long-term aging conditioning protocols, NCAT protocol was found to be more severe than NCHRP 09-54. However, to have better insight about field validity of each protocol, long-term data are required along with rheological and chemical characterization of the aged binders.

- In terms of defining initial pass/fail criteria, with respect to the data obtained from this study, there is now an understanding about acceptable values for each parameter based on the performance history of different types of asphalt mixtures. However, to find out trusted threshold values, more field data during the service life of different pavement sections are required.

6.2 Future Works

This study was phase one of a multi-phase study focusing on different approaches of defining a framework for the Nebraska balanced mix design. In this phase, performance tests were investigated to address various types of distresses in asphalt mixtures. The primary future task from Phase 1 is to continue collecting and analyzing field performance data. This data collection includes field core sampling and surface condition monitoring as an annual process. The obtained data not only are a valuable source for the state DOT, but also will be used to define the pass/fail criteria for each performance test method. The long-term data can be beneficial for the cracking tests validation and criteria, while new projects will be selected in the upcoming phases of this study for further assessment of performance tests and their criteria.

Collecting field data as well as running cracking tests on the field conditioned core samples will continue in the future to evaluate long-term aging protocols. Comparing the results from laboratory aging protocols to field data can provide a better insight into aging mechanisms. Furthermore, rheological and chemical characterization of the long-term aged binders is another step to understand the aging mechanism of asphaltic concrete mixtures. At the end of this three-phase project, a BMD framework will be defined for Nebraska including required performance tests, a long-term aging protocol, a performance test diagram, and an appropriate BMD approach.

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