Final Report

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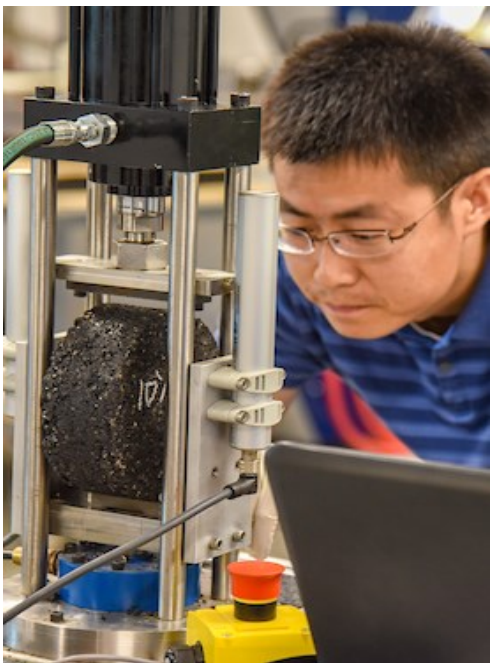

BE719

DEVELOPMENT OF A LABORATORY TESTING PROTOCOL TO EVALUATE ALTERNATIVE MATERIALS FOR USE IN MODIFYING ASPHALT BINDERS AND ALTERNATIVE MATERIALS FOR USE IN MODIFYING ASPHALT MIXTURES

Final Report

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METRIC CONVERSION CHART

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
	NOTE: volumes greater than 1000 L shall be shown in m^3			
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	$candela/m^2$	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	$candela/m^2$	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.
(Revised March 2003)

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16. Abstract

The objectives of this project were to develop separate approval protocols for alternative asphalt binder additives and alternative asphalt mixture additives. To develop these protocols, PG 76-22 (PMA) binder and its Superpave 12.5-mm mix with granite (i.e., control binder and mixture) and two alternatively modified asphalt PG 76-22 binders and their corresponding mixtures (one produced with a reactive terpolymer and the other produced with a bio-rejuvenator) were subjected to several rutting and cracking tests to develop the first protocol. The control mixture, the terpolymer- and bio-rejuvenator-modified mixtures, and two additional alternatively modified mixtures (produced by mixing granite aggregates, control binder with the mix of aramid fibers and Sasobit wax or the mix of aramid and polyolefin fibers) were subjected to various rutting and cracking tests to develop the second protocol. Test results showed that the FDOT-mandated Superpave performance tests of asphalt binders, the Hamburg wheel-track and Ideal cracking tests of asphalt mixtures were able to show whether alternatively modified binders and mixtures would perform equivalent to or better than the control binder or mixture. Results obtained by utilizing these protocols on one additional alternatively modified asphalt binder and two alternatively modified asphalt mixtures seconded these conclusions.

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EXECUTIVE SUMMARY

Several different types of asphalt binder and asphalt mixture additives have been introduced over the years to address the increasing demand of modified asphalt mixtures. The Florida Department of Transportation (FDOT) maintains approved product lists that specify the types and the dosages of additives or modifiers that are allowed in the asphalt pavements. However, comprehensive protocols are needed to maintain and update such lists. Such protocols would invite innovation among producers, provide alternative ways to construct pavements during the shortages of typical modifiers, reduce the cost of modified mixtures due to competition, and even increase pavement life by using better performing modified mixtures. The overall objective of this project was to develop two such protocols: (1) one protocol to properly assess a new asphalt binder additive to determine if its performance is equal to or better than FDOT's currently approved styrene butadiene styrene (SBS) polymer, and (2) another protocol to properly assess a new asphalt mixture additive to determine if its performance is equal to or better than FDOT's currently approved SBS-modified (PG 76-22 PMA) mixture.

To achieve the objective, Texas A&M Transportation Institute (TTI) researchers reviewed several previous studies on the performance of asphalt binders and asphalt mixtures produced with common and alternative additives. The review showed that several products have been proposed as alternative asphalt binder or mixture additives with varying degrees of success in delivering results as claimed or expected. The review also revealed that multiple test methods have been proposed to evaluate the same type of performance of asphalt binders and asphalt mixtures. Based on this review and in consultation with FDOT, the researchers selected several laboratory tests for evaluating asphalt binders and mixtures prepared with common and alternative additives. A full suite of binder tests was used to evaluate the control binder and the alternatively modified binders. Furthermore, PG 76-22 (PMA) binder and its Superpave 12.5-mm mix with granite (i.e., control binder and mixture, respectively) and two alternatively modified asphalt PG 76-22 binders and their corresponding mixtures (one produced with a reactive ethylene terpolymer and the other produced with a bio-rejuvenator) were subjected to several rutting and cracking tests (i.e., to develop the first protocol for asphalt binder additives). The rutting tests included the asphalt pavement analyzer, the Hamburg wheel-track, and the Ideal rutting tests while the cracking tests included the Ideal cracking, the flexibility index, and the Texas overlay tests. The control mixture, the

terpolymer- and bio-rejuvenator-modified mixtures, and two additional alternatively modified mixtures (produced by mixing granite aggregates, control binder with the mix of aramid fibers and Sasobit wax or the mix of aramid and polyolefin fibers) were subjected to the same rutting and cracking tests as mentioned above to develop the second protocol (i.e., the protocol for asphalt mixture additives). Test results showed that the FDOT-mandated Superpave performance tests of asphalt binders (i.e., the dynamic shear rheometer, and the bending beam rheometer tests and their variants), the Hamburg wheel-track tests, and the Ideal cracking tests of asphalt mixtures were able to show if alternatively modified binders and mixtures would perform equivalent to or better than the control binder or mixture. Results obtained by utilizing these protocols on one additional alternatively modified asphalt binder and two alternatively modified asphalt mixtures seconded these conclusions. Based on the results of these additional tests, the researchers finalized the alternative asphalt binder additive protocol and the alternative asphalt mixture additive protocol.

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LIST OF ACRONYMS AND ABBREVIATIONS

A
AAB Alternative Asphalt Binder
AAM Alternative Asphalt Mixture
AASHTO American Association of State Highway Transportation Officials
ABFE Asphalt Binder Fracture Energy
AMA Alternatively Modified Asphalt
APA Asphalt Pavement Analyzer
APL Approved Product List
APT Accelerated Pavement Testing
ASTM American Society for Testing and Materials
B
BBR Bending Beam Rheometer
BIO Bio-Rejuvenator
C
CA Carbonyl Area
CMA Cold Mix Asphalt
CMOD Crack Mouth Opening Displacement
COV Coefficient of Variation
CRI Cracking Resistance Index
CRM Crumb Rubber Modifier
CRTB Crumb Rubber Terminal Blend
CTOD Crack Tip Opening Displacement
D
DCSE Dissipated Creep Strain Energy
DENT Double-Edge Notched Tension
DGA Dense Graded Asphalt
DOT Department of Transportation
DSC Differential Scanning Calorimetry
DSR Dynamic Shear Rheometer
E
EA Epoxy Asphalt
EBA Ethylene Butyl Acrylate
EBBR Extended BBR
EVA Ethylene Vinyl Acetate
F
FC Friction Course
FDM Fracture Damage Mechanics
FDOT Florida Department of Transportation
FHWA Federal Highway Administration
FI Flexibility Index
FM Florida Method
FMA Fiber-Modified Asphalt
FN Flow Number
FREI Fracture Resistance Energy Index
FST Frequency Sweep Test
FTIR Fourier Transform Infrared

G
GMA Glycidyl Methyl Acrylate
GR Glover-Rowe
H
HDPE High-Density Polyethylene
HMA Hot-Mix Asphalt
HP High Polymer
HSMA High SBS-Modified Asphalt
HVS Heavy Vehicle Simulator
HWT Hamburg Wheel-Track
I
IDEAL-CT Ideal Cracking Tolerance Test
IDEAL-RT Ideal Rutting Tolerance Test
IDT Indirect Tension
IFIT Illinois Flexibility Index Test
IRLPD Incremental Repeated Load Permanent Deformation
ISTEA Intermodal Surface Transportation Efficiency Act
ITS Indirect Tensile Strength
J
JMF Job Mix Formula
L
LAS Linear Amplitude Sweep
LLD Load-Line Displacement
LTLG Low Temperature Limiting Grade
LVE Linear Viscoelastic
M
MDI Methylene Diphenyl Diisocyanate
MMT Montmorillonite
MSCR Multiple Stress Creep Recovery
MTO Ministry of Transportation, Ontario
N
NCAT National Center for Asphalt Technology
NCHRP National Cooperative Highway Research Program
NJDOT New Jersey Department of Transportation
NMAS Nominal Maximum Aggregate Size
O
OB Original Binder
OGFC Open Graded Friction Course
ON Overnight
OT Overlay Tester
P
PAV Pressure Aging Vessel
PC Polychloroprene
PG Performance Grade
PE Polyethylene
PEG Polyethylene Glycol
PENG Penetration Grade

PGH High Temperature Performance Grade
PGL Low Temperature Performance Grade
PLAS Pure Linear Amplitude Sweep
PMA Polymer Modified Asphalt
PP Polypropylene
PPA Polyphosphoric Acid
PPG Polypropylene Glycol
PS Polybutadiene

Q

QC Quality Control
QA Quality Acceptance

R

RAP Reclaimed Asphalt Pavements
RAS Recycled Asphalt Shingles
REOB Refined Engine Oil Bottom
RET Reactive Ethylene Terpolymer
RFB Repeated Flexural Bending
RLPD Repeated Load Permanent Deformation
RTFO Rolling Thin Film Oven

S

SB Styrene Butadiene
SBS Styrene Butadiene Styrene
SCB Semi-Circular Bending
SCB-FI Semi-Circular Bending-Flexibility Index
SD Same Day
SEBS Styrene Ethylene Butylene Styrene
SEM Scanning Electron Microscopy
SENB Single-Edge Notched Bending
SEPS Styrene Ethylene Propylene Styrene
SGC Superpave Gyratory Compaction
SHRP Strategic Highway Research Program
SIP Stripping Inflection Point
SIS Styrene-Isoprene-Styrene
SMA Stone Mastic Asphalt
SST Simple Shear Test
STOA Short-Term Oven Aging

T

TDI Toluene diisocyanate
TXDOT Texas Department of Transportation
TTI Texas A&M Transportation Institute

V

VA Vinyl Acetate
VECD Viscoelastic Continuum Damage
VEFD Viscoelastic Fracture Damage
VFA Voids Filled with Asphalt
VG Viscosity Grade
VMA Voids in Mineral Aggregates
VTAE Vacuum Tower Asphalt Extenders

1 INTRODUCTION

1.1 Background

Different types of asphalt binder additives and asphalt mixture additives have been introduced over the years to address the increasing demand of modified asphalt mixtures. Several factors have played a key role in this increase in their demand and available modification options (Bdour et al., 2015; Corun et al., 2016; Daly, 2017; Hand, 2018; Roque et al., 2006). Many state Departments of Transportation (DOTs) maintain approved product lists that specify the types and the dosages of additives or modifiers that are allowed in their pavements. However, comprehensive protocols are needed to maintain and update such lists. Such protocols would invite innovation among producers, provide alternative ways to construct pavements during the shortages of typical modifiers, reduce the cost of modified mixtures due to competition, and even increase pavement life by using better performing modified mixtures.

This report presents protocols developed for evaluating and approving alternative modifiers and additives that can be used to modify asphalt binders and asphalt mixtures in line with the two objectives of the Florida Department of Transportation (FDOT) project BE719:

- Determine a laboratory testing protocol to properly assess new asphalt binder additives to determine if their performance is equal to or better than FDOT's currently approved additives. SBS polymer will be considered the baseline.
- Determine a laboratory testing protocol to properly assess new asphalt mixture additives to determine if their performance is equal to or better than FDOT's currently approved SBS modified (PG 76-22) mixture.

To achieve these objectives, Texas A&M Transportation Institute (TTI) researchers conducted several tasks as outlined in this chapter and described in depth in ensuing chapters.

1.1.1 Literature Review

At first, a comprehensive literature review was conducted on the use of modified asphalt binders and modified asphalt mixtures to select the most effective tests to be included in the experimental part of this study. To this end, the researchers reviewed several previous studies, including the ones conducted by FDOT, that evaluated the performance of asphalt binders and asphalt mixtures produced with common and alternative additives. The review

showed that several different types of products have been proposed as asphalt binder and mixture additives with varying degrees of success in delivering results as claimed/expected. The review also revealed that multiple test methods have been proposed to evaluate the same type of performance of asphalt binders and asphalt mixtures, and that most tests lag in one or more essential features of any laboratory test such as simplicity, and sensitivity to mix design variables including the effect of additives, practicality, repeatability, reproducibility, cost effectiveness, and field correlation.

Based on this review, eight different asphalt binder tests—two rutting, five cracking, and one chemical test—and six different asphalt mixture tests—three cracking and three rutting tests—were selected for use in this study (see in Figure 1-1).

1.1.2 Protocol Development

Two protocols—the AAB and the AAM additive approval protocols—were developed as a result of this study. The AAB additive approval protocol was developed to evaluate if the AMA binders and their mixtures [i.e., asphalt binders/mixtures modified with AAB additives] would perform equivalently to or better than the FDOT's standard PG 76-22 polymer-modified asphalt (PMA) binders and their mixtures [i.e., styrene-butadiene (SB)- or styrene-butadiene-styrene (SBS)-modified PG 76-22 (PMA) binders/mixtures]. To develop this protocol, two different PG 76-22 AMA binders were produced—the first by modifying the unmodified PG 67-22 binder with the Elvaloy Reactive Ethylene Terpolymer (RET) 4170 with the N200 Poly Phosphoric Acid (PPA) and the second by modifying the High Polymer (HP) binder with the Sylvaroad RP1000 bio-rejuvenator (BIO). Then, a series of tests were conducted on the three PG 76-22 binders [i.e., PG 76-22 (PMA), PG 76-22 (RET), and PG 76-22 (BIO) binders] as well as their Type SP-12.5 mixtures with granite aggregates to determine the most effective tests and parameters that would help in making approval decisions for the AAB additives (see Chapter 3).

Similarly, the AAM additive approval protocol was developed to evaluate if asphalt mixtures modified with the AAM additives such as polymer fibers, polyethylene terephthalate (PET) plastics, etc. would perform equivalently to or better than the FDOT's standard modified mixtures [i.e., asphalt mixtures produced with PG 76-22 (PMA) binders]. To develop this protocol, two SP-12.5 fiber-modified asphalt (FMA) mixtures with granite aggregates—the first by modifying PG 76-22 (PMA) mixture with the ACE XP fibers and

the second by modifying the PMA mixture with the FORTA-FI fibers—were first produced and subjected to three rutting and three cracking tests (see Chapter 3).

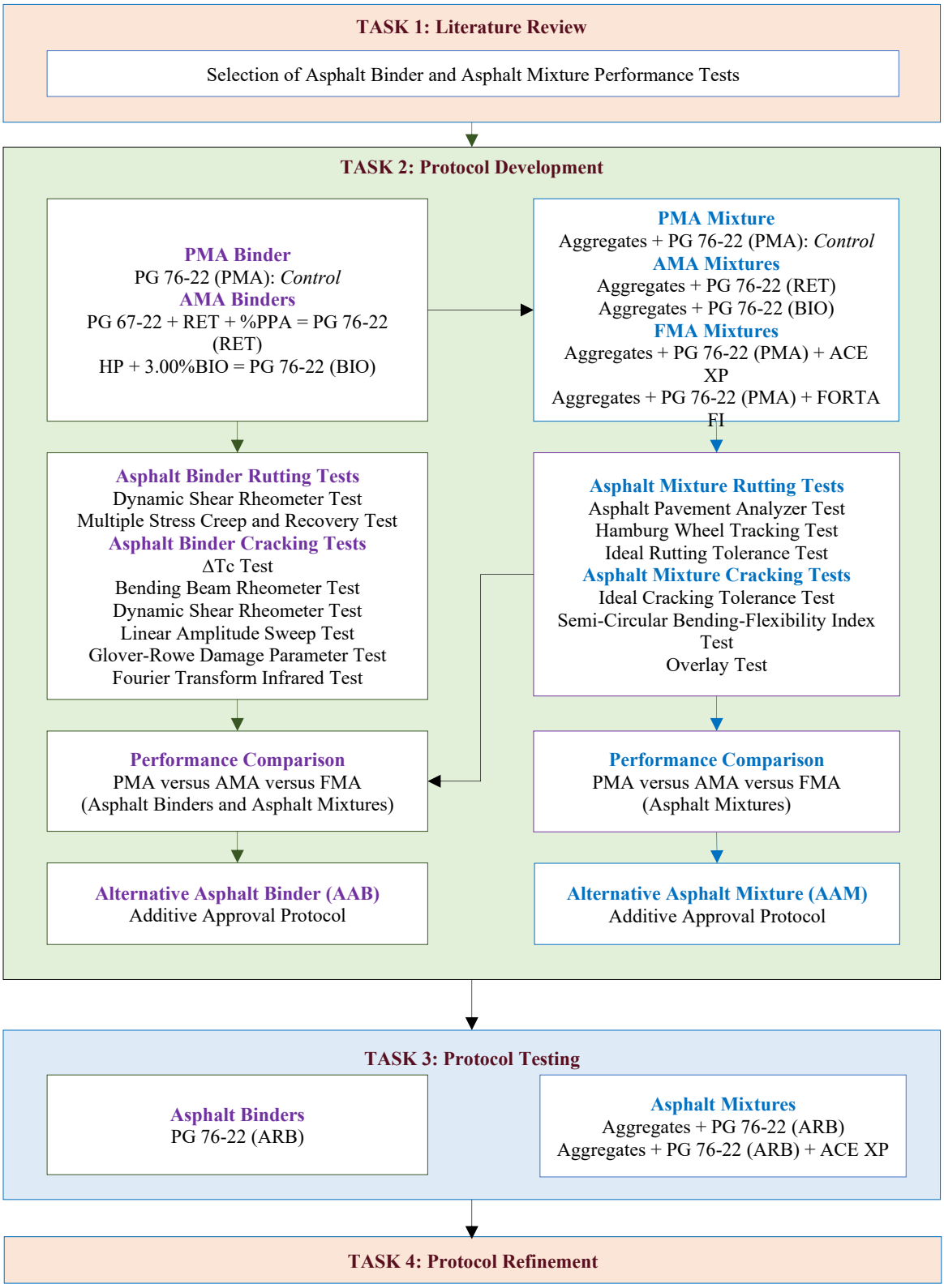


Figure 1-1. Approved Work Plan

1.1.3 Protocol Testing

To test the effectiveness of these protocols and refine them as needed, two different Type SP-12.5 AMA mixtures with granite aggregates were produced—the first with ground tire rubber (GTR)-modified PG 76-22 asphalt rubber binder (ARB) and the second with PG 76-22 (ARB) binder and the ACE XP fibers (see Chapter 4).

The AAB additive approval protocol was then used to evaluate if PG 76-22 (ARB) binder/mixture would perform equivalently to or better than PG 76-22 (PMA) binder/mixture. The protocol showed that, compared to PG 76-22 (PMA) binder, PG 76-22 (ARB) binder had a higher rutting resistance but a lower cracking resistance, and therefore would disapprove the use of PG 76-22 (ARB) binder as an equivalent or a better substitute of PG 76-22 (PMA) binder. The protocol also showed that, compared to the PMA mixture, the ARB mixture had a better rutting performance but neither an equivalent nor a better cracking performance, and therefore would disapprove the use of the ARB mixture as an equivalent or a better substitute of the PMA mixture. Based on both binder and mixture tests, the protocol would disapprove the use of this AMA binder/mixture as an equivalent or better substitute of the PMA binder/mixture.

The AAM additive approval protocol was then used to evaluate if the AMA mixture produced with PG 76-22 (ARB) binder and the ACE XP fibers would perform equivalently to or better than the PMA mixture. The protocol showed that, compared to the PMA mixture, the ARB + ACE XP mixture had a higher rutting resistance and an equivalent cracking performance, and therefore would approve the use of the ARB + ACE XP mixture as an equivalent substitute of the PMA mixture. However, the only contribution of the ACE XP fibers in the mixture was elevating the cracking resistance of the ARB mixture and making it equivalent to the PMA mixture without improving its rutting resistance at all.

1.1.4 Protocol Refinement

Based on the FDOT's feedback on the original version of the drafts and the results obtained by testing the use of the AAAB and the AAM additive approval protocols on one additional AMA binder and two additional AMA mixtures, several refinements were made on the AAB and the AAM additive approval protocols. The conclusions and limitations of this study are presented in Chapter 5, and the final versions of these protocols are presented in Chapters 6 and 7.

1.2 Report Organization

This report includes seven major chapters:

- Chapter 1 (this chapter) introduces the objectives, individual tasks, and overall scope of this study.
- Chapter 2 summarizes the results obtained by conducting a comprehensive search of literature on the use, evaluation, and approval of standard and AMA binders and mixtures.
- Chapter 3 summarizes the results obtained by conducting laboratory tests on standard and AMA binders and mixtures to develop the AAB and the AAM additive approval protocols.
- Chapter 4 summarizes the results obtained by testing the use of these protocols on one additional AMA binder and two additional AMA mixtures and the refinement made based on these results.
- Chapter 5 presents the conclusions obtained from the literature review and protocol development, testing and refinement, the limitations of this study, and recommendations for future steps.
- Chapter 6 presents the refined version of the AAB additive approval protocols.
- Chapter 7 presents the refined version of the AAM additive approval protocols.

2 LITERATURE REVIEW

Different types of asphalt binder additives and asphalt mixture additives have been introduced over the years to address the increasing demand of modified binders and modified mixtures. Several factors have played a key role in this increase in their demand and available modification options (Bdour et al., 2015; Corun et al., 2016; Daly, 2017; Hand, 2018; Roque et al., 2006), such as the following:

- The demand for better-performing roads to deal with incessant increases in traffic volumes, truck size, truck weight, and tire pressure that led to the use of polymers in asphalt binders.
- The need to meet Superpave binder grade.
- The demand for higher-quality asphalt binders to deal with construction of thinner pavement layers.
- The improvement in the capacity of the petroleum industry to extract lighter fuels that leaves behind low-quality asphaltene-rich asphalt binders and leads to the use of re-refined engine oil bottoms (REOBs)/vacuum tower asphalt extenders (VTAEs).
- Environmental and economic pressures and incentives to use abundantly available recyclable waste materials and industrial by-products such as GTR, reclaimed asphalt pavements (RAPs), recycled roof shingles (RAS), tear-off asphalt shingles, manufacturing waste asphalt shingles, fly ash, slag, etc.
- The willingness of state DOTs to pay a higher cost for long-lasting pavements.
- Legislature mandates such as the Federal Intermodal Surface Transportation Efficiency Act of 1991 and the State of Florida's Solid Waste Management Act of 1988 that led to the use of GTR in asphalt binders.
- The short supply of asphalt binders due to oil crises such as the one in the 1970s that led to the use of first-generation asphalt binder additives, such as extenders like sulfur, fillers like carbon black and mineral fillers, and hydrocarbon compounds like recycling agents and rejuvenators.
- The price hike of asphalt binders due to the shortage of additives, such as the one in 2008 that led to the greater use of alternative asphalt binder additives such as REOB/VTAE, biomass, fatty acids, waste oils, vegetable oils, etc.

State DOTs usually maintain specifications and lists of approved products that specify the types and the dosages of additives that are allowed in their pavements. FDOT currently allows the use of SBS and SB polymers and GTR in asphalt binder modification and the use of cellulose and mineral fibers in asphalt mixture modification (FDOT, 2016a). FDOT also allows limited use of PPA, REOB/VTAE, and silicone in asphalt binders provided asphalt binders meet the designated grade and asphalt mixtures meet all relevant volumetric and performance requirements. FDOT specifies the type, dosage, properties, and blending process for asphalt binder/mixture additives in its specifications and mix design sheets and maintains the Approved Products List (APL).

However, the past events have proven that approval of a limited number of additives can be problematic sometimes, especially during shortages of either these additives or the asphalt binders produced with them. If history is any indication, shortages of asphalt binder additives can happen any time due to issues that are beyond the control of state DOTs and pavement contractors. For example, the shortage of SB di-block copolymer and SBS linear and radial copolymers in 2008 led to the shortage of PMA binders (Berkley et al., 2009). Similarly, even when the additives are not in short supply, the asphalt binders modified with these additives might not be always readily available, especially when there is low demand of such binders. For example, when there was low demand of GTR-modified asphalt binders in Florida, several suppliers either stopped producing these binders anymore or produced them only after receiving a purchase order, thereby creating a short supply of PG 76-22 (ARB). To deal with such shortages, the asphalt pavement industry considered three primary solutions:

- Using unmodified asphalt binders: One of the solutions considered by the Federal Highway Administration (FHWA) to deal with the shortage of SB- and SBS-modified asphalt binders in 2008 was to replace high grade PG 76-22 PMA binders with the same or lower grade unmodified asphalt binders (FHWA, 2008). However, pavements constructed with unmodified asphalt binders do not perform equivalently to pavements constructed with modified asphalt binders under the same level of traffic load.
- Using alternatively modified additives: To deal with the shortage of PG 76-22 (ARB) asphalt binders due to inadequate production of these binders in Florida, FDOT issued a memorandum that allowed the use of SBS-modified PG 76-22

(PMA) asphalt binders in their place on sections that belonged to the critical path of the schedule (FDOT, 2015). Similarly, during the supply shortage of SBS-modified asphalt binders in 2008, FHWA recommended using asphalt binders modified with RET, Ethylene Vinyl Acetate (EVA), or crumb rubber modifier (CRM) in place of SB- or SBS-modified asphalt binders—a position seconded by the asphalt industry (Hirt, 2017).

- Using hybrid asphalt binders: Studies conducted by FDOT showed that asphalt binders modified with SBS can be replaced with asphalt binders modified together with SBS and GTR; these modified binders are also called hybrid asphalt binders (Roque et al., 2009; The Balmoral Group, 2008).

Such shortages can cause an increase in the price of asphalt binders, delay in construction, and potentially compromise asphalt binder quality and, consequently, asphalt pavement life as well—each of which can add direct and indirect costs to the state DOTs during construction or over the life of pavement. Therefore, state DOTs should not be limited in their choice of alternatives. However, not all products can be blindly approved because the use of some alternatives can cause stability issues during construction and long-term performance after construction. Therefore, well-established protocols are required to verify that alternative additives are equivalent to or better than commonly used additives. If the alternatively modified asphalt binders/mixtures indeed perform equivalently to or better than the commonly used modified asphalt binders/mixtures, their use does not incur additional cost to the public, or the performance outweighs the added cost, their use not only helps during shortages but also can lower the cost of asphalt binders and asphalt mixtures due to competition, require less frequent and less costly maintenance, and make the pavement last longer. Therefore, through this project, FDOT intended to develop laboratory protocols that can properly assess

- if the performance of asphalt binders/mixtures modified with alternative additives is equal to or better than the performance of currently approved SBS-modified asphalt binders/mixtures (i.e., PG 76-22 PMA), and
- if the performance of asphalt mixtures modified with alternative asphalt mixture additives is equal to or better than the performance of asphalt mixtures produced with currently approved SBS-modified asphalt binders (i.e., PG 76-22 PMA)

To accomplish the objectives of this project, TTI researchers reviewed several previous studies, including the ones conducted by FDOT, that evaluated the performance of asphalt binders and asphalt mixtures produced with common and alternative additives. This review is composed of five major sections:

- The first, the second, and the third sections introduce different types of additives used to modify asphalt binder and asphalt mixtures.
- The fourth and the fifth sections provide a summary of the different types of tests used to evaluate the effect of additives on the performance of asphalt binders and mixtures under most common types of pavement distresses. These sections cover origins, features, benefits, and limitations.
- The sixth section provides the concluding remarks.

2.1 Asphalt Binder Additives

Several different types of asphalt binder additives have been introduced over the years. The following sections briefly introduce the four most used asphalt binder additives (i.e., polymers, GTR, PPA, and REOB/VTAE) and offer highlights from some of the studies conducted on these additives.

2.1.1 Polymers

Polymers are essentially macromolecules made by linking many (poly) smaller molecules (monomers) of hydrocarbon compounds with one another to form long chains or clusters, of which the sequence, composition, and branching of each decide the overall properties (Daly, 2017; Roque et al., 2006). In the asphalt industry, polymers made of two or three species of monomers, called copolymers and terpolymers, are usually used. For example, SBS copolymer is made of styrene and butadiene monomers while RET is made of polyethylene, ester (ethyl/methyl/butyl acrylate), and glycidyl methacrylate monomers. Polymers are generally used with crosslinking agents such as sulfur and PPA to form crosslinks (Gama et al., 2018). Such agents prevent asphalt binders from turning into gel, thereby making them workable during production and construction (Jasso et al., 2015). For example, elemental sulfur used with SBS copolymer to modify asphalt forms crosslinks of styrene molecules with butadiene via double bonds and crosslinks of polymers with asphalt binders via sulfide or polysulfide bonds, thereby producing the asphalt binders as desired.

PMA binders have been used in asphalt pavements since the mid-1960s. Though initially used to enhance rutting resistance of asphalt binders—a ubiquitous problem at that time—the use of PMA in asphalt pavements has drastically increased. Asphalt binders modified with polymers are proven to have better adhesion and cohesion, higher stiffness and viscosity, and lower temperature susceptibility; as such, their rutting and fatigue cracking performance in the field are superior (Arámbula-Mercado et al., 2016; Kanitpong and Bahia, 2005; Kök and Çolak, 2011). Depending upon the predominant mechanical properties, polymers can be categorized as elastomers and plastomers. In addition, depending upon how polymers interact with asphalt binders, polymers can be also classified as either non-reactive or reactive.

2.1.1.1 Non-reactive Polymers

Non-reactive polymers do not react with any components of asphalt binders. These polymers usually develop crosslinks within asphalt binders and enhance their overall properties. Most of the polymers that have been used to modify asphalt binders fall into this category. Depending upon the mechanical properties of polymers used in modification, non-reactive polymers can be further classified as elastomers and plastomers (Hansen et al., 2000; Zhu et al., 2014).

2.1.1.1.1 Elastomers

As their name suggests, elastomeric polymers are predominantly elastic in nature. Elastomers form long chains or clusters with asphalt binders and improve their stiffness and elastic recovery properties, thereby making asphalt binders more resistant to rutting and cracking (Nuñez et al., 2014; Xiao et al., 2014). SBS copolymer, SB copolymer, Styrene-Isoprene-Styrene (SIS) copolymer, Styrene-Ethylene Butylene-Styrene (SEBS) copolymer, polyisoprene (PI) polymer, SB rubber (SBR) latex, and polychloroprene (PC) latex are some examples of nonreactive elastomers. As the list suggests, most of the polymers used in asphalt binder modification are based on relatively rigid polystyrene and relatively soft and rubber polybutadiene (PS) polymers.

The SBS copolymer is the most commonly used polymer in asphalt binder modification as illustrated by Figure 2-1 (Daly, 2017; Polacco et al., 2015). Compared to other polymers, SBS is relatively more dispersive (Polacco et al., 2005) and less expensive to produce (Chen et al., 2002; Zhu et al., 2014). Structurally, in the SBS copolymer, polystyrene blocks with

high glassy modulus ≈ 90°C serve as the bridge between polybutadiene blocks with glassy modulus ≈ −90°C (Laukkanen et al., 2016); polystyrene end-block induces strength while polybutadiene mid-blocks induce elasticity like rubber (Airey, 2003).

Based on whether these links form a linearly connected chain-like structure or a radially connected cluster-like structure, two types of SBS can be formed—linear SBS that have relatively low molecular weight and radial SBS, or r-SBS, that have relatively high molecular weight. Linear SBS copolymers usually have higher viscosities and therefore also need higher temperatures than radial SBS copolymers (Daly, 2017).

To modify asphalt binders with SBS, asphalt binders are first heated to above 160°C, and SBS particles are then blended in using a high shear mixer at that temperature (Hansen et al., 2000). At high temperature, SBS becomes soft and forms crosslinks with various components of asphalt binders, thereby forming a complex three-dimensional intertwined structure. The typical dosage of SBS in asphalt binders ranges from 2 to 5 percent by weight of asphalt binders (Gama et al., 2018). The exact dosage depends on the minimum dosage set by the state DOT or the one required that meets specific grade. SBS-modified asphalt binders are less expensive to produce than asphalt binders modified with other polymers. SBS usually enhances the rutting and cracking performance of asphalt mixtures by increasing elasticity, stiffness, and thermal stability of asphalt binders; however, its effect on moisture susceptibility is still being studied (Tarefder and Zaman, 2010).

SBR latex is the second most used polymer in asphalt binder modification (see Figure 2-1). SBR latex is produced by crosslinking styrene and butadiene monomers in the presence of sulfur through vulcanization (Daly, 2017). The end properties of SBR latex directly depend on the complexity of crosslinks. To produce SBR-modified asphalt binders, SBR is first introduced into asphalt binders as a latex emulsion and then flashed with water (Hansen et al., 2000). The end properties of SBR-modified asphalt binders depend not only on the properties of SBR but also on the steps and timing of each step used to produce them.

2.1.1.1.2 Plastomers

As their name suggests, plastomeric polymers are prevalently plastic in nature. Polyolefin-based polymers such as EVA copolymer, ethylene butyl acrylate (EBA) copolymer, polyethylene (PE) and polypropylene (PP) polyolefins are common examples of plastomers (Daly, 2017; Zhu et al., 2014). Typically, plastomer-modified asphalt binders are

less expensive and more stable than elastomer-modified asphalt binders (Gama et al., 2018; Polacco et al., 2015).

PE and PP polyolefins (except the atactic PP) are extremely nonpolar and crystalline in nature. As such, these polymers are almost completely immiscible with asphalt binders (Polacco et al., 2015).

EVA, a copolymer of PE and vinyl acetate (VA), is the most common plastomer used in asphalt binder modification (Saboo and Kumar, 2016). The PE blocks serve as the link between the VA blocks. Because the VA blocks are amorphous in nature, they disrupt the crystalline nature of the PE blocks, resulting in semi-crystalline EVA copolymers. Thus, the end properties of EVA drastically depend on the concentration of VA. Plastomers are usually used in asphalt binders to increase the high temperature grade of asphalt binders and make asphalt mixtures more resistant to rutting (Nuñez et al., 2014; Pérez-Lepe et al., 2006; Xiao et al., 2014).

Despite similar benefits, plastomers have not been used as extensively as elastomers primarily because of two reasons (Pérez-Lepe et al., 2006): (a) plastomer modified asphalt binders have two separate phases of asphalt binder and polymer that are mostly incompatible with each other and therefore undergo phase separation at a high temperature—a nuisance in terms of thermal stability during construction—and (b) plastomer modified asphalt binders usually have higher stiffness that can make asphalt binders brittle and susceptible to oxidation or aging—another nuisance in terms of cracking during service.

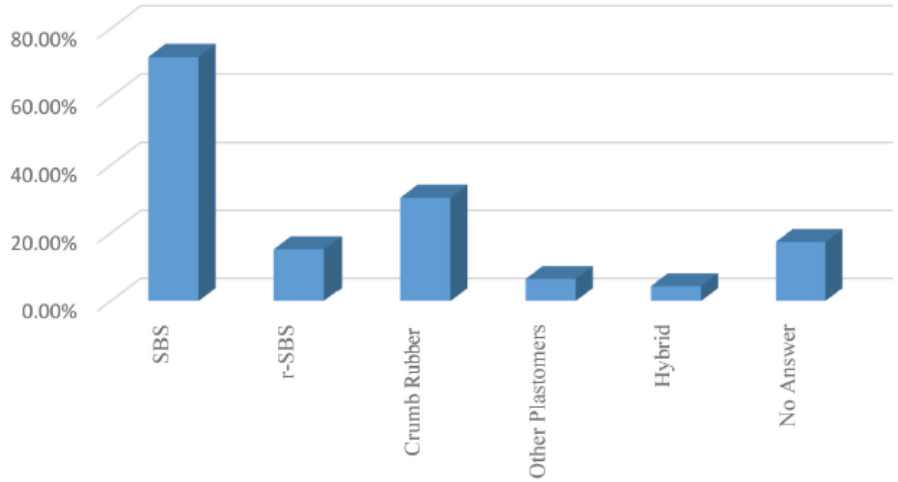


Figure 2-1: Use of the PMA Binders in the United States (Daly, 2017)

Researchers have investigated the possibilities, benefits, and limitations of using several non-reactive polymers. For example, Button et al. (1996) modified asphalt binders obtained from four field sections with different additives (SBR acetate, SBS block copolymer, SBS vulcanized with asphalt, manganese organic complex, and carbon black) and evaluated their properties to identify (a) the tests that better discriminated asphalt binder cracking resistance and (b) the properties that better correlated with field cracking. They also conducted the dynamic shear rheometer (DSR), the bending beam rheometer (BBR), the direct tension (DT), the permeation chromatography, and the Fourier-transform infrared (FTIR) tests on unaged, accelerated short-term aged and accelerated long-term aged asphalt binder specimens. Next, they also conducted the indirect tension (IDT) and the resilient modulus tests on mixture specimens of these asphalt binders. They observed inconsistent correlations of cracking with any of the asphalt binder parameters, thereby implying the inability of selected test methods to discriminate among the additives in terms of their effects. They observed that, in general, PMA binders were more resistant to fatigue cracking.

Similarly, Panda and Mazumdar (1999) modified a penetration grade asphalt binder with two grades of EVA copolymer using different modification schemes and found that the dosage and grade of EVA governed the blending temperature and decreased the penetration and temperature susceptibility while they increased the softening point and asphalt binder retention. Marshall stability tests, IDT tests, and stripping tests conducted on their asphalt mixtures revealed that EVA increased Marshall stability and tensile strength and reduced stripping, thus showcasing the positive effect of EVA.

Likewise, Airey (2002) studied modified asphalt binders with EVA copolymers and examined their rheological, thermal, and morphological properties using DSR, differential scanning calorimetry (DSC), and fluorescent microscopy techniques. The study reported that the asphalt binder source, the polymer concentration, and their compatibility heavily influenced the increase in stiffness (complex modulus) and elasticity (storage modulus), and the degree of modification.

Similarly, Kim et al. (2003) subjected asphalt mixtures containing 6.1 percent unmodified and SBS-modified and 7.2 percent unmodified and 7.2 percent SBS-modified asphalt binders to tensile strength tests, longer-term creep tests, and repeated-load fracture and healing tests in IDT mode to evaluate their cracking and healing properties. They

determined that the residual dissipated energy obtained from the tensile strength tests and the crack initiation time obtained from the longer-term creep tests both could discern the effect of SBS modification on asphalt mixture performance. They also determined that asphalt mixtures containing SBS-modified asphalt binders had lower m-values (reduced rate of microdamage accumulation) than their counterparts. Moreover, the researchers recommended exploring the effect of SBS on cracking resistance by studying the change in creep and fracture behavior.

Similarly, Gonzalez et al. (2004) modified asphalt binders with both virgin and recycled EVA copolymers and conducted several rheological, chemical, optical and thermal tests such as DSR, FTIR, DSC, thermogravimetric analysis and so forth. Test results revealed that both virgin and recycled EVA made asphalt binders rheologically more resistant to cracking at a low temperature and rutting at a high temperature and that EVA can make them thermally stable and resistant to phase separation only up to a certain dosage level. Moreover, they also suggested that the presence of carbon fillers made recycled EVA more effective than virgin EVA.

Likewise, Punith and Veeraragavan (2007) modified a penetration grade asphalt binder with five different dosages of PE (0, 2.5, 5.0, 7.5, and 10.0 percent by total weight of asphalt binder) and prepared asphalt mixtures using those dosages. The researchers ran the Hamburg wheel-track (HWT) tests, the resilient modulus tests, the IDT tests, and the unconfined dynamic creep tests on each of these asphalt mixtures. Test results revealed that PE-modified asphalt mixtures had higher resilient modulus (stiffness) and showed relatively better resistance to cracking. Test results also revealed that the asphalt mixtures modified with 5 percent PE showed better temperature susceptibility and rutting resistance.

Recently, Saboo and Kumar (2016) modified a viscosity grade (VG) asphalt binder (VG10) with the EVA copolymer at seven rates (1, 2, 3, 4, 5, 6, and 7 percent by weight of total asphalt binder) at four different temperatures (160˚, 170˚, 180˚, and 190˚C), four different mixing times (20, 30, 40, and 60 minutes) and five shear rates (300, 600, 900, 1200, and 1500 rpm). Penetration and softening point tests conducted on each of these blends, 80 in total, highlighted an increasing trend of stiffness (i.e., with a lower penetration and a higher softening point) with an increase in EVA content. Storage stability and fluorescence microscopy tests revealed 2 percent and 3 percent, respectively, as the limit for the optimum

amount of EVA. Overall, researchers concluded that, when compared to the rate of mixing and the duration of blending, temperature plays the most important role in asphalt binder modification with EVA.

Most recently, Saboo et al. (2018) compared the modification effect of SBS and EVA copolymers on fatigue cracking and rutting properties of asphalt binders using four different VG asphalt binders (VG 10, VG 30, PMB(S) = VG 10 + 5 percent SBS and PMB (E) = VG 10 + 3 percent EVA). The researchers subjected these asphalt binders to the linear amplitude sweep (LAS) tests at three different intermediate level temperatures (10, 20, and 30°C) and to the multiple stress creep and recovery (MSCR) tests at four stress levels (0.1, 3.2, 5.0, and 10 kPa) and at three different high temperatures (40, 50, and 60°C). Researchers determined the non-recoverable creep compliance (Jnr) and the percent recovery ($\%Rec$) from MSCR tests and cracking resistance parameters (A and α) from LAS tests. They gave different priorities to each of these parameters and formulated eight different cases for two different fictitious locations and ranked the asphalt binders in terms of their rutting and cracking performance using a new parameter called global total ranking value. Based on the results of this value, the researcher found that SBS- and EVA-modified asphalt binders had higher resistance to fatigue cracking and rutting than unmodified asphalt binders. They also determined that applied strain and temperature affected the ranking between SBS- and EVA-modified asphalt binders. More relevant to this project, their method better discriminated asphalt binders based on overall performance.

2.1.1.2 Reactive Polymers

Reactive polymers chemically interact with different components of asphalt binders and change their properties. These polymers are produced by treating non-reactive polymers, mainly plastomers with certain chemicals. The main purpose of this treatment is to add chemical groups that can change specific functions or properties of asphalt binders such as storage stability, strong adhesion with aggregates and resistance to rutting. This process of adding specific functional groups is therefore also known as functionalization (Zhu et al., 2014).

2.1.1.2.1 RETs

The terpolymers produced by functionalizing PE and ester (ethyl, methyl, or butyl acrylate) polymers with glycidyl methacrylate (GMA) are one of the most studied reactive

polymers for use in asphalt binders. GMA functional groups make polymers able to react with carboxylic, hydroxyl or amine groups usually present in the asphaltene phase of asphalt binders, and form strong chemical bonds (Polacco et al., 2015). Researchers have investigated the possibility of using non-reactive terpolymers as a substitute of commonly used additives, such as GTR, SBS, and EVA.

For example, Selvavathi et al. (2002) studied the addition of RET and GTR to a penetration grade asphalt binder and found that both additives stiffened asphalt binders (decreased the penetration depth and increased the softening point).

Similarly, Polacco et al. (2004) studied such possibility by using different base asphalt binders modified with non-reactive (SBS or EVA) or reactive (RET) polymers. Researchers reported that the viscosity curves of some SBS- or EVA-modified asphalt binders exhibited Newtonian behavior at low shear rates followed by two distinct shear-thinning phenomena at certain temperatures while others exhibited small shear-thickening followed by shear-thinning phenomena. However, in the case of RET-modified asphalt binders, the researchers did not notice any such phenomena and attributed this behavior to the temporary nature of the physical polymer network in them.

Likewise, Keyf (2015) modified asphalt binders together with 1.0 percent RET, 0.5–2.5 percent EVA, and 1.0 percent SBS polymers and evaluated their penetration depth, penetration index, softening point, ductility and elastic recovery properties. Test results showed that modification decreased the values of penetration and ductility while increasing the values of the penetration index, softening point and elastic recovery. They also compared the penetration index, softening point and elastic recovery of asphalt binders modified with 2.5 percent EVA, 1.0 percent RET and 1.0 percent SBS and those of eight different types of standard PMA binders. Test results showed the alternatively modified asphalt binders satisfied the requirements of two standard PMA asphalt binders.

2.1.1.2.2 Reactive Copolymers

Copolymers produced by functionalizing polyethylene glycol (PEG) and polypropylene glycol (PPG) with the 4,4-methylene diphenyl diisocyanate (MDI) or with 2,4-toluene diisocyanate (TDI) are other types of reactive polymers studied for use in asphalt binders. Common examples of these polymers include MDI-PEG, MDI-PPG, TDI-PPG, and TDI-PPG (Abroumand et al., 2018; Carrera et al., 2010c; Cuadri et al., 2014, 2015; Fang et al.,

2016; Izquierdo et al., 2011, 2012, 2013, 2014; Martín-Alfonso et al., 2008; Partal and Martínez-Boza, 2011; Singh et al., 2003, 2006; Xia et al., 2016; Yu et al., 2018; Zhang et al., 2017).

Researchers have also investigated the possibility of using isocyanate-functionalized copolymers in place of SBS copolymers. Their studies have reported positive effects of these copolymers on storage stability (Carrera et al., 2010a; Carrera et al., 2010b, 2010c; Navarro et al., 2009, 2007) and rutting resistance (Carrera et al., 2010c; Cuadri et al., 2015; Izquierdo et al., 2014; Partal and Martínez-Boza, 2011; Xia et al., 2016; Yu et al., 2018; Zhang et al., 2017). However, studies are divided concerning their resistance to low temperature cracking (Abroumand et al., 2018; Bazmara et al., 2018; Izquierdo et al., 2014; Navarro et al., 2007). Furthermore, the effect of these polymers on the aging potential of these asphalt binders has not been fully investigated (Navarro et al., 2009; Yu et al., 2018). Similarly, most claims regarding performance and stability have not been validated using asphalt mixtures and pavement sections.

2.1.1.2.3 Epoxy Asphalt

Epoxy asphalt (EA) is composed of two parts: specialized asphalt binders and epoxy resins. The two parts are first mixed with each other at a high temperature during construction, and before the two parts react with each other completely, the blend is introduced to the aggregates. Sometimes, EA is diluted by blending it together with standard asphalt binders (Herrington, 2013). The reaction continues at an ambient temperature during and after compaction for a few weeks. During this process, crosslinks continue to develop between resin and asphalt binders, resulting in a continuous gain in asphalt mixture stiffness (Polacco et al., 2015). EA was first studied as a reactive asphalt binder additive in the 1960s when the United States Corps of Engineers conducted laboratory and field evaluations of two asphalt binders modified with epoxy resins and a “flexibilizing” additive (Burns, 1964). The corps found that EA passed tests for all criteria (curing, thermosetting, swelling and leaching) but one (tensile strength). The corps also noticed that EA-modified tack coat had a stronger bond at the interface of overlay pavements. Based on these illustrated benefits, the product has been used in bridge decks all over the world and recently in high traffic friction courses in Europe.

Researchers evaluated the use of seven different types of EA (15–41 percent epoxy resin added to the virgin asphalt binders by weight) on local pavement surfaces in Europe (International Transport Forum, 2008). Asphalt binder test results showed that EA outperformed conventional asphalt binders in aging and fatigue cracking/fracture resistance. Based on mixture tests, they also estimated that pavements constructed with EA could last longer even under heavy traffic. The study highlighted the importance of proper aggregate selection due to the reaction of some aggregates with the EA binders and of close supervision of the mixture production, placement, and compaction practices because of the temperature on the epoxy curing behavior.

Other researchers (Herrington, 2013) monitored the performance of four pavement sections constructed with open-graded friction course (OGFC) mixtures containing the same asphalt binder content but with four different concentrations (or dilution levels) of EA by total weight of binder:

- 0 percent EA + 100 percent standard binder,
- 25 percent EA + 75 percent standard binder,
- 50 percent EA + 50 percent standard binder, and
- 100 percent EA + 100 percent standard binder

Researchers concluded that pavement performance improved with an increase in the dosage of EA or a decrease in the dilution of EA with standard binder.

Alabaster et al. (2015) conducted viscosity tests on unmodified and EA-modified binders and Cantabro tests on unmodified and EA-modified OGFC mixtures at different aging conditions. Viscosity tests of asphalt binders revealed that 80 days of aging was equivalent to approximately 12 years of field aging. Cantabro tests revealed that EA made the OGFC asphalt mixtures more durable. Fitting mass loss as a function of aging, the researchers found that EA-modified OGFC would have a service life that is 12 times longer compared to unmodified OGFC. They also estimated that unmodified OGFC would be cheaper initially but 2.4 times costlier than 25 percent EA-modified GFC over its life.

In summary, these studies showed that EA had equivalent or better properties than unmodified asphalt binders and that EA would make asphalt mixtures better or equivalent to control asphalt mixtures in terms of texture, skid resistance, cracking performance, rutting performance, and overall cost, therefore showing promising prospects for EA-modified

binders—a focus of an ongoing FDOT project (FDOT, 2018a). However, reactive polymers can make stronger chemical bonds with asphalt binders and form their own separate network, which might lead to phase separation at a high temperature (Daly, 2017). Also, excessive functionalization coupled with the presence of poly-functional asphaltene aggregates or molecules can increase the risk of gel formation, thereby speeding phase separation (Polacco et al., 2015).

2.1.2 GTR

GTR, which is also referred to as CRM, refers to finely pulverized particles of tire rubbers (FHWA, 2014). The first use of GTR-modified asphalt binders dates to the 1960s, when the Arizona Department of Transportation used GTR-modified asphalt materials to patch asphalt pavement surfaces (Heitzman, 1992). However, the use of GTR-modified asphalt binders increased drastically after the Intermodal Surface Transportation Efficiency Act (ISTEA) Section 1038(d) of 1991 specified the minimum amount of GTR state DOTs ought to use in asphalt pavements each year starting in 1994 (FHWA, 2014). Since standards and a deployment plan were not properly formulated before the enactment of this law, the requirement was later lifted (Ghabchi et al., 2016; Heitzman, 1992). However, many state DOTs saw the potential of lowering skidding, hydroplaning, noise, aging, and rutting while also benefiting the environment—with the use of GTR-modified asphalt binders and continued researching and implementing the outcomes of these research activities. GTR has today evolved as one of the major additives used to modify asphalt binders used in asphalt pavements (see Figure 2-2).

Figure 2-2: Use of the ARBs in the United States (Blumenthal, 2013)

2.1.3 PPA

PPA is an inorganic, acidic liquid obtained by thermally condensing orthophosphoric (i.e., monophosphoric) acid or by hydration of phosphorus pentoxide (Baumgardner, 2009). PPA has been used in the asphalt industry as an asphalt binder modifier, as a crosslinking additive in reactive polymer modification of asphalt binders and as a catalytic agent in the air-blowing oxidation process (Maurer and D'Angelo, 2012; PFA, 2015). PPA is being increasingly used as an alternative asphalt binder modifier because PPA can increase the high temperature grade of asphalt binders with minimal effect on the low temperature grade, thereby widening the useful temperature interval, UTI (Corun et al., 2016). When PPA is mixed with asphalt, it reacts with asphaltenes (Orange et al., 2004); increases the viscosity, softening point, and stiffness; and decreases the phase angle, thereby making asphalt binders more elastic in the process (Corun et al., 2016).

Research has shown that asphalt binders modified with even a small amount of PPA perform equivalently to asphalt binders modified with a higher amount of polymers (Baldino et al., 2013; Jasso et al., 2015; Nuñez et al., 2014). For example, Nuñez et al. (2014) evaluated the rutting and fatigue cracking performance of asphalt binders modified alone or together with PE and PPA. Based on MSCR and LAS tests, the researchers concluded that asphalt binders modified with PPA alone had a better rutting and fatigue cracking performance than asphalt binders modified with polymer with or without PPA.

PPA is also used as a crosslinking additive in the modification of asphalt binders with SBS copolymer and ethylene terpolymer. Research has shown that asphalt binders modified with polymer in the presence of PPA perform better than asphalt binders modified with polymer alone. For example, D'Angelo (2010) showed that asphalt binders modified with linear or radial SBS and PPA together exhibited higher values of UTI, Jnr, $\%Rec$ and elastic recovery (i.e., performed better than asphalt binders modified with SBS alone). Similarly, research has shown that asphalt binders modified with RET and PPA together stabilize much faster than asphalt binders modified with RET alone (PFA, 2015; Prejean and Babcock, 2007). Another study showed that asphalt mixtures produced with asphalt binders modified with SBS and PPA are equivalent to asphalt mixtures modified with SBS alone in terms of rutting and fatigue cracking performance, reinforcing the benefits of using PPA together with SBS (Bennert and Martin, 2012).

Typically, PPA is used at 0.2–1.5 percent by total weight of asphalt binder. The exact dosage depends on the source of the asphalt binders, compatibility with aggregates and other additives used in mix design, desired level of asphalt binder grade, specifications, and regulations on the use of PPA-modified asphalt binders. State DOTs are reluctant to use PPA because some of the PPA-modified asphalt binders are reportedly not compatible with alkaline additives, such as lime, liquid anti-stripping agents, or warm mix additives (Corun et al., 2016).

Another study (Al-Qadi et al. 2014) highlighted the importance of the source of aggregates as another factor to consider before using PPA-modified asphalt binders citing two previous studies that showed an unintended reaction between PPA with acidic aggregates such as granite (Buncher, 2010; TFHRC, 2010). A recent study (Arnold, 2014) re-highlighted the incompatibility of PPA with some liquid anti-stripping agents and the negative effects of such combinations on mixture properties—for example, moisture damage. Consequently, a number of state DOTs in the United States have restricted excess or any use of PPA-modified asphalt binders in their contracts as shown in Figure 2-3 (Maurer and D'Angelo, 2012).

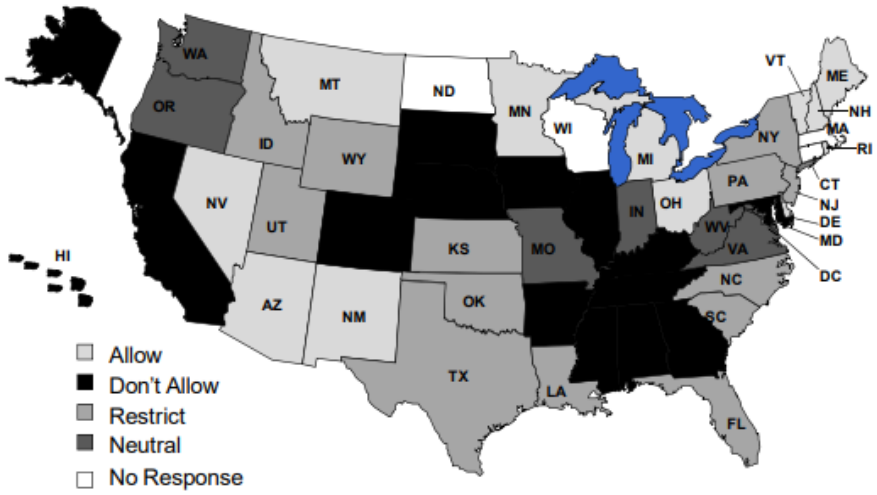


Figure 2-3: Use of the PPA-Modified Binders in the United States (Maurer and D'Angelo, 2012)

2.1.4 REOB/VTAE

The asphalt pavement industry defines REOB/VTAE as the residual distillation product obtained from a vacuum tower in a re-refinery of used lubricating oil (Asphalt Institute, 2016). Because of the use of vacuum tower distillation, REOB/VTAE usually is less volatile and more viscous than waste engine oils — the materials often confused with REOB/VTAE. REOB/VTAE has been used in asphalt binder modification for over 3 decades (Herrington, 1992; Herrington et al., 1993; Mooney, 2015; Wielinski et al., 2015). FHWA recently found that asphalt binder samples obtained from 16 out of 31 state DOTs contained REOB, — in some cases, more than 20 percent REOB/VTAE (Arnold and Shastry, 2015). The asphalt pavement industry annually consumes over 160,000 tons of REOB/VTAE (Asphalt Institute, 2016). REOB/VTAE are increasingly being used in asphalt binders to compensate for the decrease in the total amount of maltenes in asphalt binders due to the over-extraction of light weight fuels from raw petroleum products.

After several state DOTs recently noticed that pavements constructed with REOB/VTAE-modified asphalt binders accumulated more distresses and failed much faster than pavements constructed with REOB/VTAE-free asphalt binders, a review of previous literature revealed arguments both for and against their use (Asphalt Institute, 2016). Some studies cited REOB/VTAE for negative effects (Ahearn, 2015; Hesp et al., 2009a; Johnson and Hesp, 2014; Reinke et al., 2015; Rubab et al., 2011; Soleimani et al., 2009; Wright et al.,

2011), but other researchers disagreed citing more positive or indifferent effects (D'Angelo et al., 2013; Golalipour and Bahia, 2014; Herrington, 1992; Wielinski et al., 2015). The new consensus is that the excessive use of REOB/VTAE can be self-defeating. As a precaution, some state DOTs (Colorado, Connecticut, Illinois, Maine, Massachusetts, Michigan, New Hampshire, New York, Rhode Island, and Vermont DOTs) have completely banned the use of REOB/VTAE-modified asphalt binders in their projects while some state DOTs (South Carolina and Georgia DOTs) have established a maximum allowable dosage for REOB/VTAE (Bennert et al., 2016). Texas DOT (TXDOT) has recently started not allowing REOB/VTAE-modified binders in its projects.

2.2 Asphalt Mixture Additives

Several different types of asphalt mixture additives have been introduced over the years. The additives include products used for lowering mixing and compaction temperatures, reducing stripping of asphalt binders, and enhancing under-construction stability and long-term performance of asphalt mixtures (Hirt, 2017). Since the objective of this project is to develop protocols for evaluating asphalt mixture additives that can outperform or match the most commonly used additives in terms of their effect on asphalt mixture performance, this review mainly focused on fibrous additives that have been used to enhance stability, performance, and durability of asphalt mixtures in the field.

Fibers have been used to modify asphalt mixtures since the 1900s (Kietzman, 1960). However, the use of fibers in asphalt pavements began to intensify in the 1950s when asbestos fibers were found to increase tensile strength, compressive strength and the Marshall stability of hot mix asphalt (HMA) while also increasing their ability to maintain integrity against weathering for an indefinite period. Though the use of asbestos in asphalt pavements was discontinued due to environmental and health concerns, other types of fibers are still being used to stabilize or reinforce asphalt mixtures.

According to a recent survey (McDaniel, 2015), most state DOTs in the United States allow and, in some cases, require the use of fibers in asphalt mixtures. The survey revealed that 11 state DOTs use fibers in gap-graded (i.e., stone mastic asphalt, SMA) and open-graded (e.g., friction courses, FC) asphalt mixtures, eight state DOTs including FDOT use fibers in porous OGFC, 12 state DOTs use fibers in gap-graded asphalt (i.e., SMA) mixtures only, and only one state DOT (i.e., Idaho DOT) allows their use in the dense-graded asphalt

(DGA) hot mix overlays (see Figure 2-4). The survey also revealed that some state DOTs use fibers in cold mix asphalt (CMA) mixtures (one state DOT: Connecticut DOT), curb asphalt mixtures (two state DOTs: New Hampshire DOT and Pennsylvania DOT), and thin hot mix overlays and stabilized asphalt mixtures (one state DOT: Virginia DOT). The survey also revealed that Florida annually uses approximately 500,000 tons of fiber-modified OGFC, with the fibers used mainly to prevent draindown of asphalt binders during construction (McDaniel, 2015). As of 2015, at least four state DOTs used fibers to modify asphalt mixtures for enhancing rutting and cracking performance, one state DOT used fibers to enhance rutting performance and only one state DOT used fibers to enhance cracking performance and reduce asphalt binder draindown (McDaniel, 2015). Literature shows that fibers have been used in asphalt mixtures:

- To stabilize asphalt mixtures by preventing draindown of asphalt binders, usually in open- or gap-graded asphalt mixtures (Hansen et al., 2000).
- To reinforce asphalt mixtures by enhancing their tensile properties (McDaniel, 2015).
- To do both.

The following paragraphs describe common types of fibers that have been either allowed or studied for use as asphalt mixture additives.

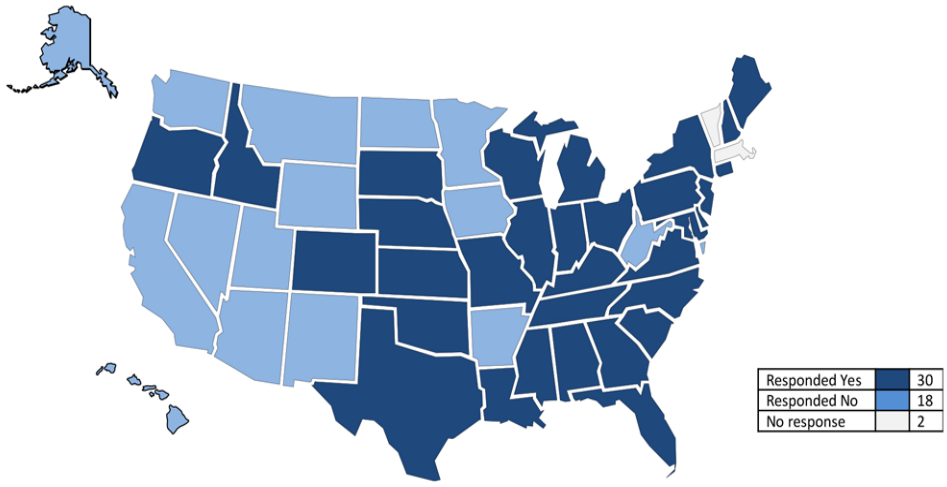


Figure 2-4: Use of the FMA Mixtures in the United States (McDaniel, 2015)

2.2.1 Cellulose Fibers

Cellulose fibers are the most widely used fibers in asphalt mixtures in the United States; only a few state DOTs (Connecticut, Idaho, New York, and New Hampshire) do not allow or specify their use in asphalt mixtures (McDaniel, 2015). These fibers are typically prepared from plants or their products such as newspapers. Because of their organic origin, these fibers are more absorptive than other fibers, and consequently asphalt mixtures modified with these fibers can be more susceptible to moisture-related problems (Cooley et al., 2000). The absorptive nature of cellulose fibers also increases the required amount of asphalt binder, thereby making their asphalt mixtures more durable but also making them more expensive. Studies have shown that these fibers not only can reduce the draindown of asphalt binders in asphalt mixtures during construction but also enhance their mechanical properties (Chowdhury et al., 2006). As such, state DOTs apply different criteria to ensure quality and prevent excess use of these additives. For example, FDOT requires that cellulose fibers used in its contracts satisfy the requirements not only for average length, gradation, ash content and acidity (or pH value) but also for oil absorption and moisture content (FDOT, 2016a).

2.2.2 Mineral Fibers

Mineral fibers are the second most used stabilizing fibers in asphalt mixture in the United States (McDaniel, 2015). These fibers are typically made of minerals such as asbestos, basalt, diabase, slag, steel slag, etc. The properties of asphalt mixtures modified with these fibers can vary drastically depending upon the rawness of the mineral and the method used to process the fibers. Generally, mineral fibers are used at the rate of 0.3–0.4 percent by total weight of mixture (Chowdhury et al., 2006). After asbestos was recognized as a hazardous material, other fibers, such as basalt fibers, received attention because, in addition to not being hazardous, alternative mineral fibers are relatively less expensive and reportedly can enhance the thermal, chemical, mechanical and hydrophobic properties of asphalt pavements (Celauro and Praticò, 2018; Fiore et al., 2015). To control the quality of mineral fibers and prevent unintended consequences, state DOTs set different criteria for these fibers as well. For example, FDOT requires that mineral fibers used in its contracts be made of virgin basalt, diabase or slag and satisfy the requirements for average length, average thickness and shot content and be treated with a cationic sizing agent (FDOT,

2016a). The cationic agents enhance the disbursement of the fiber and increase adhesion of the fiber surface to the asphalt binder.

2.2.3 Polymeric Fibers

Polymeric fibers are the most used reinforcing fibers in asphalt mixtures in the United States (McDaniel, 2015). Eight state DOTs allowed or specified their use in asphalt mixtures, while one state DOT (i.e., Georgia DOT) used them in some test sections on an experimental basis. As their name suggests, these polymers are prepared from different types of polymers: PET and polybutylene terephthalate (PBT) polyesters (Button and Hunter, 1984; Wu et al., 2007); aromatic polyamides or aramids (Button and Hunter, 1984); polyamide nylon (Lee et al., 2005); PP and PE polyolefins (Button and Hunter, 1984; Huang and White, 1996; Jiang and McDaniel, 1993); or their combinations (Button and Hunter, 1984; Kaloush et al., 2008, 2010; Muftah et al., 2017; Noorvand et al., 2018).

Polyester fibers can maintain their mechanical strength at temperatures as high as 482°F (250°C). A typical dosage rate for polyester fiber varies from 0.15 percent to 0.38 percent of the total mix and increases the design asphalt content by approximately 0.1 to 0.2 percent (Chowdhury et al., 2006). Polyolefin fibers are used at 0.3 percent by total weight of asphalt mixture. They are used in asphalt mixtures that need temperatures over 300°F (150°C) for mixing or compaction because of their relatively low melting point (Chowdhury et al., 2006). Aromatic polyaramids (aramids) fibers are synthetic fibers that are highly resistant to heat and have strong tensile strength. Therefore, these fibers are used to enhance rutting resistance. Polymeric fibers are sometimes used in conjunction with each other such as aramid and polyolefin fibers (Muftah et al., 2017) and aramid and PP fibers (Kaloush et al., 2008). The combined use of fibers is reportedly more effective in lessening draindown and improving the mechanical properties of asphalt mixtures.

2.2.4 Other Fibers

Glass fibers: More commonly known as fiberglass, these fibers have a relatively high tensile strength compared to other fibers. Glass fibers are primarily used to manufacture grids that are placed between two layers of pavements for reinforcing interlayer bonding (Brown, et al., 2001b), but they are also being increasingly considered as an alternative asphalt mixture additive. Except for some experimental purposes, these fibers have not been widely used or allowed by state DOTs in the United States.

Recycled/waste fibers: These fibers refer to fibers that are obtained from waste materials such as ceramics, wire wools, recycled tires, etc. Researchers are increasingly studying the prospect of recycled fibers as alternative additives for use in asphalt mixtures due to their low cost, abundant availability, and environmental and sustainability advantages. However, state DOTs have not allowed their use in asphalt pavements, except in experimental sections, because of limited supporting research.

The ensuing paragraphs provide brief synopses of some studies since 1980s that have investigated the use of fibers as an asphalt mixture additive:

Button and Hunter (1984) modified asphalt mixtures with 10 different types of fibers and conducted resilient modulus tests, wet and dry IDT tests, freeze-thaw moisture treatment, flexural fatigue cracking tests, overlay tests and direct compression tests. The fibers in this study included PP, polyester, aramid, fiber glass, asbestos, a combination of PP and aramid, and a fiber product consisting of cellulose, starch, and ash. Test results showed that fibers did not influence temperature susceptibility, tensile strength and rutting resistance much but increased resistance to moisture damage and cracking.

Jiang and McDaniel (1993) analyzed rutting and reflective cracking of overlays constructed with and without PP fiber-modified asphalt mixtures for 7 years. The researchers found that both methods did not reduce or delay reflection cracking; however, they noticed that a fiber-modified overlay had relatively less frequency of cracking. The researchers also observed that both construction methods reduced rutting over the years. More noteworthy, the fibers improved rutting resistance in both control and crack and seated sections.

Serfass and Samanos (1996) conducted a three-part study involving asphalt mixtures modified with and without asbestos, rock wool, glass wool, and cellulose fibers. The first part of the study involved loading the test sections constructed with these asphalt mixtures with a 13 metric-ton axle load for 1.1 million times. Based on the level of volumetric properties retained by the asphalt mixtures after such loading, they concluded that fiber-modified asphalt mixtures would provide better drainage and would be less susceptible to moisture than their counterparts. The second part of the study involved loading the overlays constructed with these asphalt mixtures for 2.0 million times. Based on the level of macrostructure maintained after such loading, the researchers concluded that fiber-modifier asphalt mixtures provide better resistance to fatigue cracking than unmodified asphalt

mixtures. The third part of this study involved loading the overlays constructed with fiber-modified asphalt mixtures for 1.2 million times. Based on the intensity of fatigue cracking and rutting-related signs, the researchers concluded that fiber-modified asphalt mixtures performed better than their counterparts.

Huang and White (1996) conducted flexural fatigue cracking tests and complex modulus tests on slab and core samples of PP fiber-modified asphalt mixtures obtained from a series of overlay test sections and layers. Complex modulus test results revealed that fiber significantly changed the viscoelastic properties of asphalt mixtures, but an analysis of variance (ANOVA) of these results showed an inconsistent effect of fiber content on the dynamic modulus and phase angle, possibly highlighting the fact that such tests might not be good candidate tests to determine the fiber's effect. Beam fatigue cracking tests revealed that fiber-modified overlays may have up to twice the life of unmodified overlays. Researchers also conducted the extraction of fibers through three different methods. Test results showed a discrepancy between the applied and designed fiber content. Though none of these methods were standard, the fact that all three resulted in similar conclusions underscored the need for better technology to introduce fiber into the mixture.

Chowdhury et al. (2006) modified three types of limestone asphalt mixtures with two types of recycled tire fibers and cellulose fiber, and evaluated whether recycled tire fibers can effectively replace cellulose fibers. The mix used in this study included one SMA mix of PG 76-22TR asphalt binders, one PFC mix of PG 76-22S asphalt binders, and one coarse-mix high-asphalt binder mix of PG 64-22 asphalt binders. The two fibers differed in the length, the thickness, the source, and the dosage (0.3 percent for shorter, 1.0 percent for longer). Researchers conducted the draindown tests, the IDT tests, the dynamic modulus tests, the overlay tests and the HWT tests and determined that (a) asphalt mixtures modified with recycled tire fibers, though not in all cases, outperformed cellulose fiber-modified asphalt mixtures, and (b) recycled tire fiber provided better resistance to draindown. Consequently, researchers concluded that recycled tire fibers worked well as alternative asphalt mixture modifiers.

Wu et al. (2007) prepared modified asphalt mixtures using 0.3 percent cellulose, 0.3 percent polyester and 0.4 percent mineral fibers by total weight of mixture and evaluated their dynamic modulus. Their study revealed that fiber-modified asphalt mixtures were stiffer

(i.e., had a higher dynamic modulus) at any given temperature and frequency, more elastic (i.e., had lower phase angles) at lower temperatures, but more viscous (i.e., higher phase angles) at higher temperatures than unmodified asphalt mixtures. Test results showed that cellulose, polyester and mineral fibers always enhanced rutting performance in lower to higher ranking order (i.e., rutting resistance parameters were respectively 1.12, 1.14, and 1.24 times that of the control mixture).

Kaloush et al. (2008, 2010) obtained one control and two fiber-modified HMA asphalt mixtures from test sections in Tempe, Arizona, and compared their engineering properties. Each mixture was prepared with PG 70-22 asphalt binders and a mixture of aramid fibers and PP fibers. The modified HMA contained 0.454 kg (1 lb) or 0.908 kg (2 lb) of fibers per 1 ton of asphalt mixture. Researchers subjected SGC specimens of these asphalt mixtures to triaxial shear strength tests, repeated load permanent deformation (RLPD) tests, dynamic modulus tests, bending beam fatigue cracking tests, flexural strength tests, IDT tests and C* integral tests. Researchers also subjected unmodified and PP-modified asphalt binders to consistency tests (penetration tests, ring and ball tests and Brookfield viscosity tests) and found that modified asphalt binders possessed similar viscosities at lower temperatures but higher viscosities at higher temperatures and did not create any handling issues. Test results revealed that fiber-modified asphalt mixtures exhibited higher cohesion, more susceptibility to permanent deformation, higher indirect tensile strength (ITS) and better resistance to crack propagation by similar stiffness to control mixtures. Test results also highlighted that the dosage of fiber directly impacted the degree of these effects.

Liu et al. (2010) modified porous asphalt mixtures with three types of steel fibers or steel wools with different lengths and diameters. Researchers found that electrical conductivity is higher in asphalt mixtures modified with fibers that have larger lengths and smaller diameters (i.e., more slender fibers) than asphalt mixtures modified with fibers that have smaller lengths and larger diameters. They also determined 10 percent as the ideal steel wool content to obtain optimum electrical conductivity, induction heating speed and resistance to raveling.

Morova (2013) modified HMA asphalt mixtures with basalt fibers at dosage rates by weight of mixture and conducted Marshal stability tests on the samples. The test results showed that the stability value increased with the change of basalt fiber content from 0.25 to 0.5 percent and decreased with the change of basalt fiber content from 0.5 to 2 percent. These

results suggest that there is an optimum basalt fiber content that can be used to modify asphalt mixtures with desired stability. However, the study also revealed through ultrasound velocity tests that the gap between aggregates increased with increasing dosages of basalt, which should be considered when designing asphalt mixtures.

Guo et al. (2015) prepared HMA mixtures using diatomite-modified asphalt binders and glass fibers at different dosages (max 0.3 percent by weight of asphalt mixture), and conducted the wheel tracking tests on their roller-compacted specimens and IDT tests on the Marshall-compacted specimens to evaluate their rutting, low temperature cracking, fatigue cracking and stiffness properties. Researchers found that the use of glass fiber improved rutting performance, low temperature cracking performance (or tensile strength), and fatigue cracking performance and reduced the overall stiffness of asphalt mixtures more than the use of diatomite.

Bdour et al. (2015) modified HMA with 0.25 and 0.5 percent wire wool — abrasives used for polishing wood or metal objects, cleaning household cookware, cleaning windows, and sanding surfaces — and compared their properties with unmodified asphalt mixtures. They conducted Marshall stability tests, ITS tests, dynamic creep tests, fatigue cracking tests, and rutting tests for the comparison. Marshall stability tests showed that modified and unmodified asphalt mixtures did not significantly differ from each other, negating the need for a stability test that could discriminate different additives. Other test results demonstrated that asphalt mixtures modified with wire wool fibers had higher tensile strength and had longer fatigue cracking life but lower rutting resistance than control asphalt mixtures. Researchers attributed the loss in rutting resistance to the higher than ideal content of wool fibers in the asphalt mixtures, highlighting the possibility of obtaining the optimum wool fiber content.

Fakhri and Hosseni (2017) prepared warm mix asphalt (WMA) mixtures using 0.3 percent glass fiber and 0-50 percent RAP by the total weight of mixture and conducted the wheel track tests on their roller-compacted specimens to determine their rutting and moisture susceptibility. Researchers found that glass fibers improved rutting resistance (longer final loading cycles, improved secondary inflection point and higher dynamic stability) but could not improve moisture susceptibility.

Muftah et al. (2017) conducted a case study for the Idaho DOT to determine the effects of fibers on pavement distresses. FDOT had replaced existing pavement sections with fiber-modified asphalt mixtures at three sections and with unmodified (control) asphalt mixtures at one section. Polyolefin and aramid fibers, wax-treated aramid fibers and Type E glass fibers were used in these sections at fixed rates. For this study, the researchers collected asphalt mixtures from each of these sections and conducted dynamic modulus tests, flow number (FN) tests, HWT tests, semi-circular bend (SCB) tests, low temperature IDT tests, and creep compliance tests on each of them. Researchers conducted these tests on asphalt mixtures modified with three additional dosages of polyolefin and aramid fibers. Based on test results, the researchers concluded that asphalt mixtures needed a minimum of 0.3 percent glass fiber content to achieve significant rutting resistance and that fibers did not add tensile strength before crack ignition but helped asphalt mixtures resist crack propagation at all but the low temperature domain. The study also highlighted the importance of well-distributed fibers and greater bonding between the fibers and the matrix for mitigating distresses. Researchers reported that fibers started to pull out from one side of a crack during SCB tests even though the fibers did not break themselves. Researchers attributed the occurrence of cracking without the breakage of fibers to the lack of bonding between the fibers and the asphalt matrix, and therefore recommended using adhesion promotors that can keep the fibers intact in mixtures and better help resist cracking.

Shanbara et al. (2018) prepared CMA samples using three natural fibers—hemp, jute and coir and one synthetic glass fiber. Researchers conducted scanning electron microscopy tests on fibers to determine the properties of fibers and conducted IDT tests, wheel tracking tests and SCB tests, tensile strength ratio (TSR) tests for capturing stiffness, rutting resistance, cracking resistance, and water sensitivity of asphalt mixtures, respectively. The researchers observed improved higher stiffness, tensile stiffness, cracking resistance, rutting resistance and reduced water sensitivity tests in fiber-modified CMA. They hypothesized that these improvements were due to interlocking provided by the fibers, which they found to have rough surfaces.

Celauro and Praticò (2018) modified asphalt mixtures with 3–5 mm long basalt fibers at the rate of 0.3 percent by total weight of asphalt mixture. The wheel slab tests conducted on their slabs revealed that basalt fibers reduced rut depths, which the researcher attributed to

relatively thicker asphalt binder film and a stiffer mixture. Surface texture and laser profilometer tests revealed that basalt-modified asphalt mixtures complied with skid resistance and smoothness.

Lee et al. (2005) investigated the influence of nylon fibers on the fatigue cracking resistance of asphalt mixtures. Researchers embedded the single threads of fibers with different lengths into asphalt binders filled in disposable paper cups and conducted the single fiber pull-out tests with a sewing machine using a 5 lb-load and 300 mm/min loading rate. The tests showed the minimum embedded length of selected nylon should be 9.2 mm. Researchers then subjected asphalt mixtures modified with 0.25, 0.50 and 1.0 percent nylon with 6 mm and 12 mm length to ITS tests. Test results revealed that asphalt mixtures modified with 1.0 percent nylon of average length of 12 mm resulted in the highest fracture energy (85 percent higher than unmodified asphalt mixtures), showcasing the potential of nylon as an alternative fracture-resisting additive of asphalt mixtures.

Noorvand et al. (2018) obtained one unmodified and two fiber-modified asphalt mixtures from a plant and used the dynamic modulus tests, RLPD tests and the uniaxial fatigue cracking tests to determine their stiffness, rutting and cracking properties. A mixture of aramid and polyolefin fibers was used to produce one of the two asphalt mixtures (referred to as an FA mixture) while aramid fibers coated with a thin membrane of wax were used to produce the second modified mixture. This part of the study revealed that fiber improved fatigue cracking and rutting resistance of asphalt mixtures significantly but did not affect stiffness. Additionally, the researchers measured the dispersion, distribution, and surface morphology of fibers in modified asphalt mixtures. The researchers used fibers extracted from asphalt mixtures to determine their dispersion state ratio, the failure surface of tested specimens to determine their distribution and the scanning electron microscopy (SEM) images to determine their morphology. This part of the study revealed that aramid fibers were similarly distributed but better dispersed with more fibrillation compared to wax-coated aramid fibers, thereby highlighting the benefits from the combined use of some of the fibers.

Arabani and Shabani (2019) modified asphalt binders with 1, 3, and 5 percent ceramic fibers and conducted penetration, softening point and ductility, rotational viscometer, DSR and BBR tests. Researchers found that ceramic fiber modified asphalt binders exhibited a lower cracking resistance at a low temperature but a higher cracking resistance at

intermediate temperatures and a higher rutting resistance at high temperatures. Researchers conducted wheel track tests and IDT fatigue cracking tests on asphalt mixtures produced with these asphalt binders and confirmed these properties.

2.3 Approved Additives in Florida

FDOT currently allows the use of two grades of SBS/SB-modified asphalt binders — PG 76-22 (PMA) and HP binders and one type of GTR-modified asphalt binder — PG 76-22 (ARB). FDOT approved the use of these binders after investigating the effect of SBS/SB polymer and GTR on asphalt binder properties, and asphalt mixture performance and then validating the results of these studies through the heavy vehicle simulator (HVS) tests at its accelerated pavement testing (APT) facility, at the National Center for Asphalt Technology (NCAT) test track, and in several local pavements. The ensuing sections provide brief synopses of each of these studies.

2.3.1 Polymers

FDOT (2016a) currently requires all PMA binders having a high temperature designation higher than PG 67 to be produced with a SBS or SB elastomeric polymer modifier and meet all requirements of Section 916 of FDOT's standard specification. FDOT maintains APL for PG 76-22 (PMA), PG 82-22 (PMA) and HP. FDOT has conducted several in-house and sponsored studies to evaluate the suitability of PMA binders in its pavements.

For example, through a FDOT-sponsored study, Tia et al. (2002) evaluated the rutting performance of two identical test sections, both constructed with the same fine-graded SP12.5 mix design and same aggregate types but with different asphalt binder types in its APT facility — one with unmodified PG 67-22 and the other with SBS-modified PG 76-22 asphalt binders (prepared by mixing PG 67-22 asphalt binder with SBS). They conducted HVS testing to replicate rutting. They obtained core samples from test sections after being tested by the HVS and used them to measure change in height and density, resilient modulus at 5°C and 25°C and ITS at 25°C and asphalt pavement analyzer (APA) at 64°C. They also recovered asphalt binders from these cores and measured asphalt binder viscosity at 60°C. Additionally, the researchers also conducted gyratory compaction and APA tests on plant-collected mixture samples. The HVS test results showed that the two-lift SBS-modified sections showed less rut depth than the two-lift unmodified section. The one-lift SBS-modified over the one-lift unmodified section had about the same rut depth as the two-lift

unmodified section at 50°C. APA results mostly reflected the HVS results. Based on a change in thickness and density of the cores, researchers inferred that the SBS-modified PG 76-22 section was compacted primarily due to densification alone while the unmodified PG 67-22 section was compacted by densification and experienced shear flow (shoving). They also determined that the SBS-modified PG 76-22 mixture had an almost similar resilient modulus but about a 10 percent higher ITS than the unmodified PG 67-22 mixture. They also determined that the SBS-modified PG 76-22 asphalt binder was significantly more (2–3 times) viscous than the unmodified PG 67-22 asphalt binders. They mentioned this difference in viscosity as one of the main reasons behind the vast difference in rutting in corresponding sections.

FDOT (Moseley et al., 2003) evaluated the rutting resistance of asphalt mixtures containing four different types of asphalt binders — unmodified PG 67-22, polymer-modified PG 76-22, 5 percent GTR-modified binder (ARB-5) and 12 percent GTR-modified binder (ARB-12) — by conducting APA tests on the asphalt mixtures at 64°C. FDOT used a fine-graded SP 9.5 mix, a fine-graded SP 12.5 mix, and a coarse-graded SP 12.5 mix for each of these asphalt binders, totaling 12 different types of asphalt mixtures. One additional mix for ARB-12 containing 0.5 percent more asphalt binder was also used in this research. The test results showed that PG 76-22 had the least rut depth in each mix type, followed by ARB-12, ARB-12 with 0.5 percent extra asphalt binder content, ARB-5 and unmodified PG 67-22 asphalt binders. Based on these results, the use of PG 76-22 PMA was recommended on heavy traffic pavements. Test results also showed that the coarse-graded SP 12.5 mix had the least rut depth followed by the fine-graded SP 12.5 mix, and then by the fine graded SP 9.5 mix. Based on this result, the use of a coarse-graded mix was recommended on heavy traffic pavements.

FDOT (Gokhale and Sholar, 2007) evaluated the rutting performance of asphalt mixtures made of SBS-modified PG 76-22 asphalt binders and unmodified PG 67-22 asphalt binders using the APA. FDOT determined that mixtures containing SBS-modified PG 76-22 asphalt binders had about 1 mm less rut depth than the mixtures containing unmodified PG 67-22 asphalt binder, highlighting the benefit of using modified asphalt binders over unmodified asphalt binders. FDOT also determined that void content and sand content

showed a negative impact while aggregate type and the change in height and density showed no impact of SBS on rutting resistance.

FDOT commissioned a study in which the researchers (Roque et al., 2006) developed guidelines for the use of modified asphalt binders based on laboratory tests, HVS experiments and field performance evaluation. For HVS tests, FDOT constructed test sections with two different asphalt binders: unmodified PG 67-22 asphalt binders and SBS-modified PG 76-22 asphalt binders. The number of lifts was varied to determine the effects. HVS test results showed the rutting performance of the two-lift-SBS-modified section was significantly better than the rutting performance of the two-lift unmodified section but statistically equivalent to the one-lift-SBS-modified over the one-lift unmodified section. Researchers also evaluated the properties—namely resilient modulus, ITS, and creep behavior—of HMA and OGFC asphalt mixtures produced with unmodified and GTR-modified asphalt binders and HMA asphalt mixtures produced with unmodified and SBS-modified asphalt binders. Additionally, they also evaluated stiffness, fracture and rutting-related properties of unmodified and GTR-modified asphalt binders. Based on HVS tests, the researchers recommended using GTR in FC to reduce draindown. SBS manifested a positive effect on rutting performance, as illustrated by the APA and gyratory shear tests, and on cracking performance, as illustrated by the reduced creep rate. SBS did not show an effect on stiffness or dissipated cumulative strain energy, highlighting the inability of some of the fracture parameters to capture the effect of SBS.

FDOT (Kim et al., 2009) also studied the effect of RAP on the rutting and cracking resistance of asphalt mixtures composed of SBS-modified asphalt binders. For capturing the effects of SBS and RAP on mixture performance, FDOT prepared asphalt mixtures containing SBS-modified asphalt binders and 0 (control mixture), 15, and 25 percent RAP and subjected compacted samples to APA tests at 64°C and IDT tests at 10°C. FDOT also prepared asphalt binder blends of SBS-modified PG 76-22 asphalt binder and 0 (control), 10, 16.8, 24.4, and 100 percent RAP-extracted asphalt binders and subjected their unaged samples to rotational viscosity tests at 135°C, their rolling-thin film oven (RTFO)-aged samples to DSR tests at 76°C and to MSCR tests at 76°C or 82°C, and their pressure-aging vessel (PAV)-aged samples to DSR tests at 25°C and 10°C, respectively. Asphalt mixture test results showed that asphalt mixtures containing SBS-modified asphalt binders and RAP

had equivalent rutting and cracking performance compared to asphalt mixtures containing SBS-modified asphalt binders alone. Asphalt binder test results showed that, instead of the DSR-obtained rutting resistance parameter ($G^*/Sin\delta$), the MSCR-obtained $\%Rec$ could better discriminate SBS-modified asphalt binders from others.

Ping and Xiao (2009) evaluated the effect of SBS modification, among others (aggregate gradation and type), on fracture properties of asphalt mixtures using 12 different asphalt mixture samples. These asphalt mixtures included those produced by blending granite and limestone aggregates with PG 67-22 asphalt binders modified with 0 (control mix), 3, 4.5 and 6 percent SBS. Resilient modulus, creep compliance and tensile strength tests were performed on each of these asphalt mixtures and a statistical analysis was conducted. It was determined that SBS-modified asphalt binders enhanced both fracture and rutting properties of asphalt binders and asphalt mixtures. At high temperatures, stiffness was maintained while creep compliance decreased with an increase in SBS content, which made asphalt mixtures stiffer and more resistant to rutting, thus demonstrating a positive effect of SBS on asphalt mixture rutting performance. At intermediate-to-low temperatures, an increase in SBS content decreased the stiffness value while increasing the creep compliance and failure strain values, which implied that SBS made asphalt mixtures more ductile, thus demonstrating a positive effect of SBS on asphalt mixture thermal cracking performance. For this study, SBS content can be optimized somewhere between 3 and 6 percent by total weight of asphalt binder.

FDOT (Greene et al., 2014a) constructed two sets of three pavement sections in its APT facility, each set containing one section with unmodified PG 67-22 asphalt binder, one with SBS-modified PG 76-22 asphalt binder and one with SBS-modified PG 82-22 asphalt binder and conducted the HVS tests. The SBS dosage in PG 67-22, PG 76-22 and PG 82-22 asphalt binders was 0, about 2–3 and 6 percent, respectively (Kwon et al., 2018). FDOT performed DSR tests on original samples at their upper temperatures, MSCR tests of RTFO-aged samples at 64°C and the fracture energy tests on RTFO plus PAV-aged samples at 10°C to determine the properties of each asphalt binder used in the study. Similarly, FDOT cored samples from PG 76-22 and PG 82-22 mix sections immediately after construction and used them to conduct IDT tests to determine asphalt mixture cracking properties. FDOT determined that PG 67-22, PG 76-22, and PG 82-22 mixtures showed an increasing order of

rutting performance, which FDOT attributed to an increasing order of $G^*/sin\delta$, and asphalt binder fracture energy and $\%Rec$, coupled with a decreasing order of Jnr. FDOT predicted the life of the fatigue cracking sections using the tensile strains observed at the bottom of the top layers and determined that the PG 82-22 section would last about 7 and 20 times longer than the PG 76-22 and PG 67-22 sections with respect to fatigue cracking at 20°C. The researchers attributed the potential longer life of the PG 82-22 section to a lower rate of damage accumulation (lower creep rate) and higher resistance to fracture (higher value of energy ratio). This study provided supporting data for the implementation of modified asphalt binders in Florida (Kwon et al., 2018).

After a newly formulated high SBS-modified asphalt (HSMA) binder came to market, FDOT again constructed three pavement sections in its APT facility, one with standard SBS-modified asphalt binder (control section) and two with HSMA binders and conducted the HVS tests (Kwon et al., 2018). The control section and one of the two HSMA sections were designed with 5.1 percent asphalt binder content while the second HSMA section was designed with 0.5 percent extra (i.e., 5.1 + 0.5 = 5.6 percent) asphalt binder content. The SBS dosage was about 2 to 3 percent in standard SBS-modified PG 76-22 (PMA) asphalt binders but almost twice that amount in HSMA asphalt binders (Kwon et al., 2018). FDOT subjected the unaged samples of each asphalt binder to DSR tests at a high PG temperature and ring and ball softening point tests, the RTFO-aged samples of SBS-modified asphalt binders at 67°C and that of HSMA binders at 67°C and 76°C and their PAV-aged samples to DSR tests at an intermediate temperature. FDOT also conducted IDT tests on asphalt mixtures sampled by coring corresponding sections immediately after their construction. FDOT determined that both types of HSMA sections performed significantly better than the SMA section in terms of rutting. Also, the HSMA section with 0.5 percent higher asphalt binder content showed lower rutting than the standard HSMA section. FDOT attributed better rutting performance of HSMA sections to higher $G^*/Sin\delta$, softening point and $\%Rec$ values and lower J_{nr} values. Similarly, FDOT determined that the HSMA content sections would have a fatigue cracking life that is about 2.3 to 3.5 times longer than the control section at 20°C. FDOT attributed this increase in the fatigue cracking life of HSMA sections to lower values of asphalt binder fatigue cracking resistance parameter ($G^*.Sin\delta$) and higher values of initial ITS.

2.3.2 GTR

FDOT's Standard Specifications for Road and Bridge Construction (2016a) allows GTR-modified PG 76-22 ARB under certain circumstances. Sections 916-2.1 and 916-2.1.1 specify the requirements for any asphalt binder to be approved as PG 76-22 (ARB) while Section 919 provides the requirement for a product to be approved as GTR. FDOT also maintains separate APL for GTR and GTR-modified PG 76-22 (ARB) binder.

FDOT (Page et al., 1992) started investigating the feasibility of GTR in asphalt pavements in the late 1980s following the mandate of the Solid Waste Management Act of 1988. As part of this effort, researchers (West et al., 1996) investigated the properties of tire rubber particles produced by grinding GTR at ambient (normal) and cryogenic (low) temperatures. Researchers found significant differences in the gradation, surface area and specific gravities of the two rubber types. They also compared the properties of asphalt binders modified with the same dosage (i.e., 12 percent) of ambiently and cryogenically produced GTR in terms of their viscosity at 135, 149 and 163°C (275, 300 and 325°F), asphalt binder separation after 48 hours of asphalt binder heating at 163°C (325°F) and draindown after 1 hour of asphalt binder heating at 143°C (290°F). FDOT's study revealed that asphalt binders modified with cryogenically produced GTR had relatively more separation and draindown issues than ambiently produced GTR. This was primarily attributed to the greater surface area of the ambient ground rubber particles compared to the cryogenically ground rubber particles.

From this study, FDOT determined that the best use of GTR-modified asphalt binders would be in OGFC and asphalt membrane interlayers (Choubane et al., 1999; Ruth, 1992). FDOT cited various reasons to use GTR-modified asphalt binders in OGFC. For example, GTR could interfere with rejuvenation of asphalt binders available in reclaimed materials used in DGA mixtures. Since the OGFC mixtures did not contain reclaimed materials, they deemed such interference as non-existent in this type of asphalt mixtures. The OGFC mixtures are more prone to raveling than the DGA mixtures because of higher in-place air void contents and relatively lower layer thickness. GTR improves the retention and reduces the draindown of asphalt binders from such asphalt mixtures.

Two new specifications were developed — one related to GTR alone and the other related to GTR-modified asphalt binders (Ruth, 1992). The first specification, "Ground Tire

Rubber for Use in Rubber Modified Asphalt Binders," set requirements for physical and chemical specifications, packaging, identification, and certification of GTR itself. The second specification, "Rubber Modified Asphalt Binders," set production, testing and certification requirements and fixed the dosages of GTR in ARB asphalt binders used in friction courses at 5 percent for DGA mixtures, 12 percent for open-graded mixtures and 20 percent for asphalt rubber membrane interlayers by total weight of asphalt binders, resulting in three types of GTR-modified asphalt binders: ARB-5, ARB-12, and ARB-20.

To evaluate the effectiveness of GTR-modified asphalt binders, FDOT constructed three demonstration projects, one with a DGA mixture and two with OGFC mixtures (Choubane et al., 1999; Ruth, 1992). Each of these projects contained a series of test sections with different types and contents of GTR. FDOT monitored rideability, skid resistance, rutting, cracking, and patching of each of these sections for 10 years after their construction. FDOT made several observations from this multi-year study (Choubane et al., 1999):

- GTR improved rutting resistance of DGA mixtures.
- GTR did not show any positive or negative effect on skid resistance.
- The method by which GTR was introduced to the mixture influenced its impact on cracking resistance. When GTR was introduced to asphalt binder and then to aggregates (i.e., through the wet process), only 1–6 percent of the pavement surface showed cracking. But, when GTR was introduced to asphalt binder and aggregates together (i.e., through the dry process), 30 percent of the pavement surface showed cracking. This result highlighted the benefit of using wet-processed GTR-modified binders over dry-processed GTR-modified binders.
- Pavements constructed with unmodified binders and dry-processed GTR-modified binders resulted in similar cracking resistance, thus emphasizing the minimal advantage of the use of dry processed GTR-modified binders.
- GTR optimum content should be within 10-15 percent by total weight of asphalt binder for OGFC mixture types.

Based on these results, FDOT revised all relevant OGFC and asphalt membrane interlayer specifications and started allowing the use of GTR-modified asphalt binders that are produced through the wet process (i.e., asphalt binders and GTR are blended first and then mixed with aggregates).

FDOT commissioned yet another study to evaluate the possibility of using hybrid asphalt binders in asphalt pavements (Roque et al., 2009). The study involved evaluating the properties of three commercially available hybrid asphalt binders, two GTR-modified asphalt binders (ARB-5 and ARB-12), one SBS-modified binder (PG 76-22 PMA), and one PG 67-22 asphalt binder using multiple stress creep recovery, elastic recovery, force ductility and DSR tests after PAV at 100°C and 110°C, in addition to the tests required by FDOT. The cracking resistance parameters obtained from a new direct tension test of asphalt binders showed that hybrid asphalt binders were equivalent to PG 76-22 (PMA) but better than PG 67-22, ARB-5, and ARB-12 asphalt binders, highlighting the potential to replace each of these asphalt binders. The IDT tests conducted on the OGFC asphalt mixtures of these asphalt binders showed that hybrid asphalt binders could possibly replace PG 76-22 PMA and ARB-12 asphalt binders only. However, based on those conclusions, the study also cautioned that the draindown potential of hybrid asphalt binders produced with finer rubber be checked.

To address unpredictable performance and separation issues of GTR-modified and hybrid asphalt binders, FDOT commissioned another study to determine the best method to produce GTR-modified asphalt binders that met the performance criteria of PG 76-22 (PMA) asphalt binders (Greene et al., 2015, 2014b). This study evaluated the performance of GTR-modified, SBS-modified, and hybrid asphalt binders and developed a new specification for GTR-modified asphalt binders, i.e., PG 76-22 (ARB). The new PG 76-22 (ARB) specification required them to be produced with a minimum 7.0 percent GTR and have a maximum of 15°F difference in the softening point tested at 163 ± 5°C. Asphalt binder tests revealed that not all GTR-modified asphalt binders passed the asphalt binder's separation criteria for PG 76-22 (ARB) asphalt binders. These results showed that the new specification can identify asphalt binders that have the same PG but might have different handling issues. HVS tests, FN tests, and Superpave IDT tests revealed that asphalt mixtures prepared with GTR-modified, SBS-modified, and hybrid asphalt binders had similar rutting and cracking resistance. These results showed that asphalt binders that satisfy FDOT's new specification also have comparable performance with PG 76-22 (PMA) asphalt binders.

Based on these studies, FDOT replaced ARB-5 and ARB-12 asphalt binders with PG 76-22 (ARB) asphalt binder.

2.3.3 Fibers

FDOT specifically started requiring the use of stabilizing fibers in the new version of the OGFC friction course, called FC-5, in 2000 (Cunagin et al., 2014; McDaniel, 2015). FDOT developed this new friction course based on FDOT's experience with previous friction courses (such as, the FC-2 mix that contained ARB-12 binder without fibers) and the Georgia DOT's experience with D-modified OGFC. The FC-5 asphalt mixture usually results in relatively higher optimum binder content compared to the DGA mixtures, which the fibers help to retain, thereby enhancing durability. FDOT's current specification (FDOT, 2016a) requires either 0.3 percent cellulose or 0.4 percent mineral fibers by total weight of mixture. The specification requires cellulose fibers to satisfy several requirements such as average length (max. 6.35 mm or 0.25 inch), gradation, ash content (18 ± 5 percent), pH (7.5 ± 1.0), oil absorption (5 ± 1.0 percent times fiber weight), and moisture content (max. 5 percent). Similarly, the specification requires mineral fibers (made of virgin basalt, diabase, or slag) to satisfy similar requirements for average length (max. 6.35 mm or 0.25 inch), average thickness (max. 0.0051 mm or 0.0002 inch) and shot content (90<Passing No. 60<100; 65<Passing No. 230<100) and be treated with a cationic sizing agent to enhance the disbursement of the fiber and increase adhesion of the fiber surface to the asphalt binder. The specification currently does not specify the use of reinforcing fibers such as aramid fibers, either in the FC-5 or other types of mixtures.

Table 2-1 and Figure 2-2 present the list of different types of additives used to modify asphalt binders/mixtures. The tables also provide FDOT's current practice regarding their use in its contracts.

Table 2-1. Common Asphalt Binder Additives (Button et al., 1996; Daly, 2017; Hirt, 2017; Ping and Xiao, 2009; Zhu et al., 2014)

Additives	FDOT Specification/ Practice
Elastomeric Polymers • Styrene-Butadiene: SB • Styrene-Butadiene-Styrene: SBS • Styrene-Isoprene-Styrene: SIS • Styrene-Ethylene-Butylene-Styrene: SEBS • Styrene-Ethylene-Propylene-Styrene: SEPS • Styrene-Butadiene Rubber: SBR ***Plastomeric Polymers*** • Polyethylene: PE • Polypropylene: PP • Ethylene-Vinyl Acetate: EVA • Ethylene-Butyl Acrylate: EBA ***Reactive Polymers*** • Terpolymers: Plastomer- Ester-GMA • Copolymers: Plastomer-Isocyanate • Epoxy Asphalt: EA	• Requires the use of SBS and SB elastomeric copolymers in PMA binders that have higher than PG 67 grade (FDOT Specification Section 916-2.1.4). • Maintains APLs for PMA binders HP asphalt binder (FDOT, 2019a), PG 76-22 (PMA) asphalt binder (FDOT, 2019b) and PG 82-22 (PMA) asphalt binder (FDOT, 2019c).
GTR	• Requires the use of minimum 7% GTR in rubber-modified PG 76-22 (ARB) asphalt binder (Section 916-2.1.1). • Maintains APLs for not only the PG 76-22 (ARB) asphalt binder (FDOT, 2019d) but also for GTR itself (FDOT, 2019e).
PPA	• Allows the use of less than 0.75% PPA as a modifier in PG 76-22 (PMA) and PG 76-22 (ARB) asphalt binders by weight of asphalt binders (Section 916-2.1.5) • Prohibits its use in HP asphalt binders.
REOB/VTAE	• Started allowing the use of maximum 8% REOB/VTAE as a modifier in asphalt binders by weight of asphalt binders (Section 916-2.1.9). • Follows Asphalt Institute Document IS-235 for defining REOB/VTAE.

Table 2-1. Common Asphalt Binder Additives (Button et al., 1996; Daly, 2017; Hirt, 2017; Ping and Xiao, 2009; Zhu et al., 2014) (Continued)

Additives	FDOT Specification/ Practice
Extenders: • Sulfur • Lignin **Oxidants:** • Manganese Salts **Antioxidants** • Carbon • Calcium Salt • Lead Compounds **Organic Materials:** • Trinidad lake asphalt • Gilsonite **Miscellaneous:** • Silicone • Montmorillonite: MMT • Deicing calcium chloride granules	• Allows the use of Silicone in all PG asphalt binders used in HMA at the rate of 25 cubic centimeters of silicone per 5,000 gallons of PG asphalt binders (section 916-2.1).

Table 2-2. Common Asphalt Mixture Additives (Button et al., 1996; Daly, 2017; Hirt, 2017; Ping and Xiao, 2009; Zhu et al., 2014)

Additives	FDOT Specification/ Practice
Fibers: • Cellulose Fibers • Mineral Fibers • Polymer Fibers • Glass Fibers • Recycled Fibers	• Requires the use of 0.4% mineral fibers or 0.3% cellulose fibers in FC-5 mixtures by weight of total mixture. (section 337-3.2.1.5) • Requires mineral fibers meet physical (e.g., diameter, length and shot) requirements and come from virgin basalt, diabase, or slag and treated with a cationic sizing agent (to enhance the disbursement of the fiber and to increase adhesion of the fiber surface to the bitumen) (Section *337-2.7.1)* • Requires cellulose fibers meet several physical and chemical requirements (e.g., length, sieve, acidity, ash content, moisture content, absorption, etc.) (Section *337-2.7.2)*
Anti-stripping Agents • Hydrated Lime • Liquid Anti-Stripping Agents	• Requires liquid anti-stripping agents pass FM 1-T 283 requirements (Section 916-4.1) • Maintains APL of Anti-stripping Agents (FDOT, 2019f) • Requires hydrated lime in granite-based mixtures and liquid anti-stripping agents in limestone-based mixtures.
Recycling Agents: • Rejuvenators • Aromatic Extracts • Fatty Acids • *Waste Oils* • Tall Oils • Bio Oils	• Prohibits the use of waste oils in the production of any PG asphalt binder (Section 916-2.1.8) • Defines waste oils as recycled oil products that have not been processed through a vacuum tower and have an initial boiling point of 385°C (725°F) or lower when tested in accordance with ASTM D6352-15
Recycled Materials: • Reclaimed Asphalt Pavements • Recycled Roof Shingles • Tear-Off Asphalt Shingles • Manufacture Waste Asphalt Shingles • Recycled Tires, Glass **Fillers:** • Carbon Black, Mineral Fillers, Fly Ash, Crusher Fines, Portland Cement **Warm Mix Additives**	

2.4 Common Asphalt Binder Performance Tests

Several attempts have been made to develop tests that can determine the performance of asphalt binders with varying degree of success in correctly predicting pavement performance. These attempts have led to the development and refinement of several American Association of State Highway and Transportation Officials (AASHTO) and American Society for Testing and Materials International (ASTM) laboratory test protocols (AASHTO, 2014a, 2018a).

The ensuing sections provide brief synopses of the geneses and features of the most common tests used to evaluate the resistance of asphalt binders against permanent deformation (or rutting) at a high temperature and cracking at intermediate and low temperatures (fatigue cracking and thermal cracking). The sections also provide synopses of studies that exposed strengths or weaknesses in discriminating the effect of additives on the performance. These additives include products that are added to asphalt binders through a wet process, as well as products that are added to aggregates and asphalt binders together through a dry process.

2.4.1 Asphalt Binder Rutting Tests

Several different tests have been developed to evaluate rutting resistance of asphalt binders at a high temperature. These include:

- High Temperature PG Test
- Repeated Creep and Recovery Test
- Zero Shear Viscosity Test
- MSCR Test

High temperature performance grade (PGH) tests are one of the most used rutting tests for distinguishing the high temperature performance of asphalt binders. The test measures the complex shear stiffness (G^*) and phase angle (δ) values of asphalt binders by conducting oscillatory tests at 10 rad/sec at different temperatures (AASHTO, 2016a). The minimum of the temperatures at which $G^*/Sin\delta$ is equal to 1.0 kPa for unaged asphalt binders and 2.2 kPa for RTFO-aged asphalt binders is referred to as PGH (AASHTO, 2017a). This concept hypothesizes that rutting is more prevalent in asphalt binders that have lower G^* and higher δ (or are softer and more viscous) than in asphalt binders that have higher $G^*$and lower δ (or

are stiffer and more elastic). The parameter has been used with varying degrees of success to determine the rutting resistance and performance grade (*PG*) of asphalt binders, including those modified with different types of additives (see Figure 2-5).

However, several studies (D'Angelo and Dongre, 2002; Dongre and D'Angelo, 2003; Stuart et al., 1999) reported that the correlation between $G^*/Sin\delta$ and the performance of asphalt binders in the field does not always hold true, especially with regard to field performance of asphalt mixtures containing modified asphalt binders. As such, researchers made several efforts to develop alternative asphalt binder tests, such as repeated creep and recovery tests.

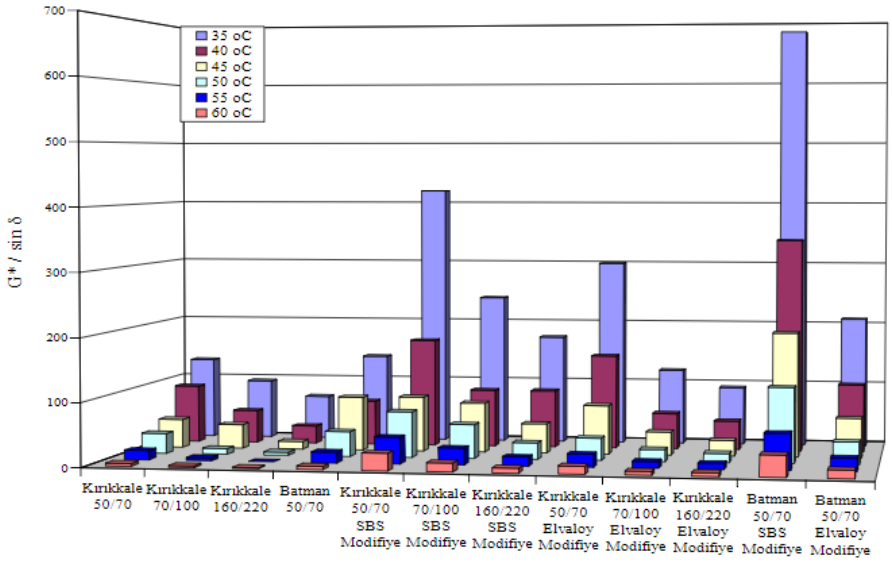


Figure 2-5: Sensitivity of *G/sinδ to Asphalt Binder Additives (Güngör and Sağlik, 2012)**

One of those efforts was made by Bahia et al. (2001), who recommended characterizing rutting resistance of asphalt binders through a repeated number of creep and recovery tests in the shear mode at a high temperature. This repeated creep and recovery test involved first running a creep test of the asphalt binder at a shear stress of 300 kPa for 0.10 seconds and then allowing the binder to recover for 0.90 seconds without applying any load. This cycle was repeated 100 times. However, the use of only one stress level for the creep test made the test unable to capture the influence of different stress levels on asphalt binder properties. D'Angelo and Dongre (2009; 2006) therefore recommended running repeated creep and recovery tests at two different stress levels instead of one stress level. Since multiple stress levels were recommended, the test aptly took its namesake, the MSCR test. This modified

test can measure not only the recoverable and non-recoverable shear strains at a given shear stress level but also the Jnr and the $\%Rec$ parameters.

The standardized version of this test (AASHTO, 2014a) involves first running 10 steps of 1-second long creeps at 0.1 kPa followed by a 9-second recovery and then running another 10 steps of 1-second long creeps at 3.2 kPa followed by a 9-second long recovery on each specimen and finally determining the average J_{nr} and $\%Rec$ and their percent difference in Jnr from the two stress levels, $\%J_{nr,diff}$. The test was used to develop the MSCR-based performance grading specification for asphalt binders (AASHTO, 2014b). The new asphalt binder grading system (also called PG Plus or PG+) measures Jnr and/or $\%Rec$ of asphalt binders at a temperature indicated by local climate and uses these values to decide if the binder is suitable for Extreme High (E), Very High (V), High (H) and Standard (S). For example, if the highest pavement temperature at a selected location is 64°C, the new system might grade asphalt binders as PG 64-22S, PG 64-22H, PG 64-22V or PG 64-22E, even when the conventional PG is PG 76-22 or something else.

Kim et al. (2009) used the test to study the rutting resistance of binders combining SBS-modified binder with recycled asphalt binders of varying percentages (0–25 percent). MSCR tests of RTFO-aged asphalt binders containing RAP binder showed that, instead of DSR-obtained $G^*/Sin\delta$, MSCR-obtained $\%Rec$ could better discriminate SBS-modified asphalt binders from others. FDOT currently specifies the MSCR test as a required routine test to determine and verify the high temperature PG of asphalt binders (FDOT, 2016a).

Nuñez et al. (2014) used this test to evaluate rutting resistance of asphalt binders modified alone or together with PE and PPA. Test results showed that asphalt binders modified with PPA alone had higher rutting resistance than asphalt binders modified with polymer PE alone or PE and PPA together.

Zhou et al. (2014) conducted round-robin tests on five unmodified and modified asphalt binders to determine the repeatability and reproducibility of MSCR tests and their correlation with rutting performance of asphalt mixtures. In terms of repeatability and reproducibility, they found high variability with $Rec_{0.1kPa}$ and Jnr_{diff} and therefore recommended exercising some caution to grading asphalt binders based on this parameter. In terms of ranking, the researchers reported that compared to $G^*/Sin\delta$-based ranking, MSCR-based ranking better represented the rutting performance of asphalt mixtures in the field, especially

in ranking unmodified and modified asphalt binders with the same high PG such as PG 64-22, PG 64-28 and PG 64-34 asphalt binders (see Figure 2-6).

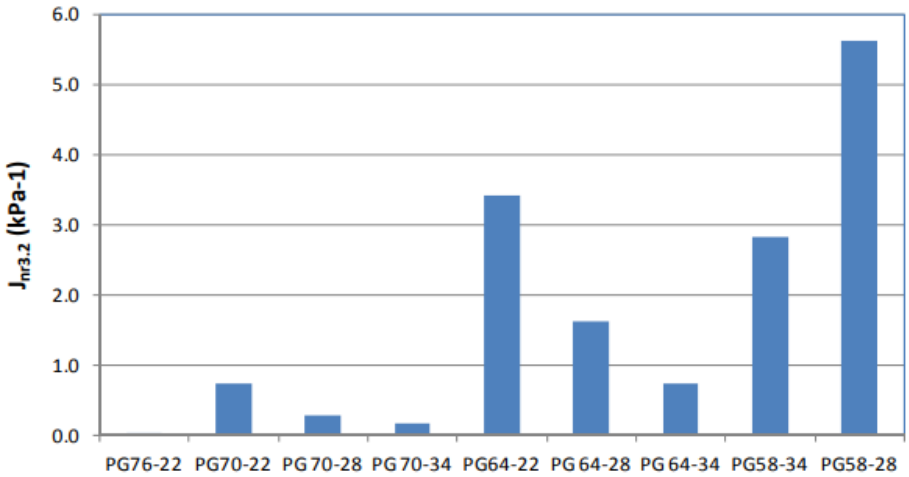


Figure 2-6: Sensitivity of Jnr_3.2 kPa to Asphalt Binder Grade (Zhou et al., 2014)

2.4.2 Asphalt Binder Fatigue Cracking Tests

As with rutting, several different tests have been developed to evaluate cracking resistance of asphalt binders at a high temperature. These tests use mainly two theories to describe the evolution of cracking and to derive associated parameters:

- Continuum Damage Mechanics
  - Intermediate Temperature PG Test
  - Time Sweep Test
  - MSCR Test
  - LAS Test
  - Glover-Rowe (GR) Parameter Test
- Fracture Damage Mechanics
  - Single-Edge Notched Bending (SENB) Test
  - Double-Edge Notched Tension (DENT) Test
  - Pure Linear Amplitude Sweep (PLAS) Test
  - Asphalt Binder Fracture Energy (ABFE) Test

In terms of test equipment, most of these tests load samples either in shear or tension mode. For example, time sweep test, multiple stress creep recovery test, LAS test, GR parameter tests and PLAS test are conducted in the shear mode while the rest are conducted in the tension mode.

2.4.2.1 Intermediate Temperature PG Test

The intermediate temperature PG, PG_{IT} (AASHTO, 2017a), a test that measures $G^*.Sin\delta$ value at intermediate temperatures, has been the conventional choice to evaluate the cracking resistance of asphalt binders. However, several researchers (Andriescu et al., 2004; Bahia et al., 2002, 2001; Deacon et al., 1997; Tsai and Monismith, 2005; Zhou et al., 2014) have reported that the $G^*.Sin\delta$ parameter does not have a strong correlation with the cracking performance of asphalt mixtures, especially those mixtures prepared with modified asphalt binders. Consequently, repeated attempts have been made to develop test methods that can discriminate asphalt binders accurately based on their fracture/fatigue cracking properties.

2.4.2.2 Time Sweep Test

The time sweep test is an ideal fatigue cracking test theoretically since the test involves repeated application of a constant shear strain (or constant shear stress) at a constant frequency and an intermediate temperature (Bahia et al., 2002, 2001; Deacon et al., 1997). However, Anderson et al. (2001) deemed this test unsuitable for characterizing fatigue cracking behavior of asphalt binders citing issues such as edge fracture effects and unstable flow of asphalt binders that make the concept of continuum damage invalid in these tests. Additionally, this test is time consuming, making it impractical as a regular laboratory test. As such, the test is largely no longer being used to characterize fatigue cracking resistance of asphalt binders.

2.4.2.3 LAS Sweep Test

The LAS test is a two-step DSR test (Hintz et al., 2011; Hintz and Bahia, 2013a). The first step of the LAS test is a linear viscoelastic (LVE) test that involves the application of a very small shear strain over a range of frequencies at an intermediate temperature without any damage and the determination of the LVE properties such as the LVE complex modulus (G^*_{LVE}) and the LVE slope (m). The second step of the LAS test is a shear strain amplitude sweep test that involves increasing the amount of strain amplitude, the construction of a characteristic pseudo stiffness (C) versus damage (S) curve and the estimation of fatigue cracking life at selected shear strain levels. The LAS test has also been incorporated in a provisional standard: AASHTO TP 101-12 (2018a). Therefore, state DOTs and researchers

are increasingly using the LAS test to compare asphalt binders with different grades, modifications, and aging conditions. However, despite being a simple and fast test, the fatigue cracking life estimated from LAS tests does not always correlate well with the cracking performance of asphalt mixtures and asphalt pavements. TTI researchers (Zhou et al., 2017c) observed this issue mostly with aged and modified asphalt binders.

2.4.2.4 PLAS Test

Zhou et al. (2017c) developed an alternative test, called the PLAS test, and a new parameter, called fracture resistance energy index (FREI), to characterize the cracking resistance of asphalt binders. FREI is a parameter that combines the effect of shear stiffness ($G_{0.5\tau_{max}}$) and shear strain ($\gamma_{0.5\tau_{max}}$) corresponding to a half-peak load and with shear fracture energy ($J_{f-\tau max}$) corresponding to a full-peak load:

$$FREI = \frac{J_{f-\tau max}}{G_{0.5\tau_{max}}} . \left(\gamma_{0.5\tau_{max}}\right)^2$$

PLAS and LAS tests have essentially the same steps and generate the same raw data; however, the PLAS test hypothesizes viscoelastic fracture damage (VEFD) mechanics while the LAS hypothesizes the VECD mechanics as the core phenomenon of damage evolution in asphalt binders. Thus, researchers believe that the FREI parameter is more representative of fatigue cracking resistance than the LAS-derived parameters (see Figure 2-7).

TTI researchers (Arámbula-Mercado et al., 2019) recently used this test to compare the fatigue cracking life of PG 76-22 PMA and PG 76-22 HP asphalt binders in another FDOT-supported project. They found that compared to LAS-estimated fatigue cracking life, PLAS-calculated FREI better discriminated modified versus unmodified, unaged versus aged, and original versus recycled/rejuvenated asphalt binders. Because PLAS is a newly minted test, the effectiveness and sensitivity of this test is still being vetted by several other researchers.

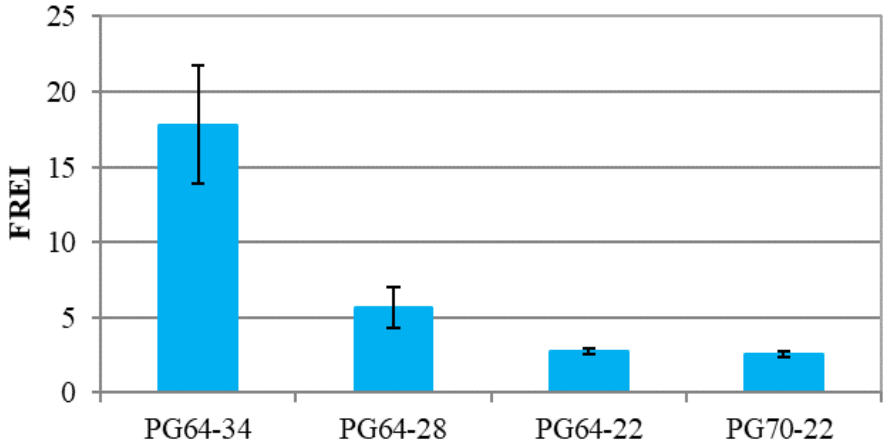


Figure 2-7: Sensitivity of FREI to Asphalt Binder Grade (Zhou et al., 2017c)

2.4.2.5 MSCR Test

The MSCR is a multiple-step, high temperature test of asphalt binders. However, the MSCR-measured parameter, $\%Rec$, has also been studied as a potential indicator of resistance of asphalt binders to fatigue cracking. For example, Kim et al. (2009) used the test to study the effect of recycled asphalt binders (0–25 percent) on the cracking resistance of SBS-modified asphalt binders. MSCR tests of PAV-aged asphalt binders at 25°C and 10°C showed that MSCR-obtained $\%Rec$ increased slightly with more RAP binder contrary to a similar performance of asphalt mixtures in IDT loading. Consequently, they did not consider this parameter a dependable asphalt binder cracking indicator.

Similarly, Mogawer et al. (2011) used Texas OT-derived fatigue cracking life, MSCR-derived %Rec. and the elastic recovery to rank PG 64-28 (control), PPA-modified PG 64-28 (PG 64-28 + PPA), SBS-modified PG 64-34, SBS-modified PG 76-22, GTR modified PG 88-16 (PG 64-22 + GTR), latex modified (PG 64-28 + latex), and their asphalt mixtures. They determined the rankings for both the asphalt binder tests and mixture tests were not consistent enough to specifically indicate whether the asphalt binder elasticity tests, including MSCR, truly represent the fatigue cracking behavior of the mixture.

Likewise, Zhou et al. (2014) later ranked modified and unmodified asphalt binders based on $\%Rec.$, the elastic recovery and the Texas overlay fatigue cracking life parameters and found a very weak correlation of %Rec. with other parameters. Zhang et al. (2016) seconded this assessment after finding MSCR parameters to be weakly correlated to elastic recovery of asphalt binders and fracture properties of asphalt mixtures. This inconsistent

correlation of MSCR parameters with other more representative fatigue cracking resistance parameters stems mainly from a fundamental difference in the loading mode (shear in MSCR tests versus tension in elastic recovery tests) and testing temperature (10°C in elastic recovery tests versus 58°C or above in MSCR tests).

Based on these studies, the consensus is that the MSCR test cannot be used to determine the cracking resistance of asphalt binders.

2.4.2.6 GR Test

The GR damage parameter is a cracking and aging parameter determined by conducting the DSR test on asphalt binder samples at a frequency of 0.005 rad/sec, a strain of 0.10 percent and a temperature of 15°C (or at a temperature dictated by local climate). Several efforts were made to develop and define this parameter. Glover et al. (2005) was the first to determine that asphalt binders with greater than 3.0 kPa/sec value for the parameter $G'/(\eta'/G')$ and less than 3.0 cm value for ductility usually showed surface cracking. Rowe (2011) defined this parameter as a function of G^* and δ, making it possible to be presented in their black space diagram.

$$\frac{G'}{\left(\frac{\acute{\eta}}{G'}\right)} = G^* \left(\frac{\cos^2\delta}{\sin\delta}\right) \omega$$

Researchers later proposed two separate values of the G-R parameter – 180 kPa to represent the onset of microdamage and light raveling and 600 kPa to represent the appearance of visible cracking on the surface (Anderson et al., 2011; King et al., 2012; Rowe and Baumgardner, 2011). To better characterize cracking resistance of asphalt binders, researchers usually run these tests on asphalt binders aged in a PAV for different durations.

For example, King et al. (2012) aged asphalt binders obtained from Western Canada, the Southeast Gulf, and West Texas for 0, 20, 40 and 80 hours after RTFO aging and then conducted G-R tests on each of them. Test results showed that a sole increase in stiffness (as represented by log G*) did not guarantee significant damage in the asphalt binder zone despite showing the greatest change after 40 hours of aging in a PAV. Based on test results, they concluded that the G-R parameter, when plotted in G*- δ black space diagram, can distinguish asphalt binders with different proneness to aging and aging-related cracking.

Similarly, TTI researchers (Arámbula-Mercado et al., 2019) conducted G-R parameter tests on SBS modified PG 76-22 (PMA) and HP modified PG 76-22 (HP) asphalt binders after aging their RTFO-aged samples again in the PAV for 0, 20, 40, 60 and 80 hours. The two asphalt binders exhibited distinctly different aging and cracking behavior (see Figure 2-8). Compared to PMA asphalt binders, HP asphalt binders exhibited a smaller change in modulus and even a smaller change in phase angle with aging and did not meet the 600-kPa line, which represents the condition of visible damage after 80 hours of aging. This effect can be attributed to the higher content of thermoplastic polymers, such as SBS in HP.

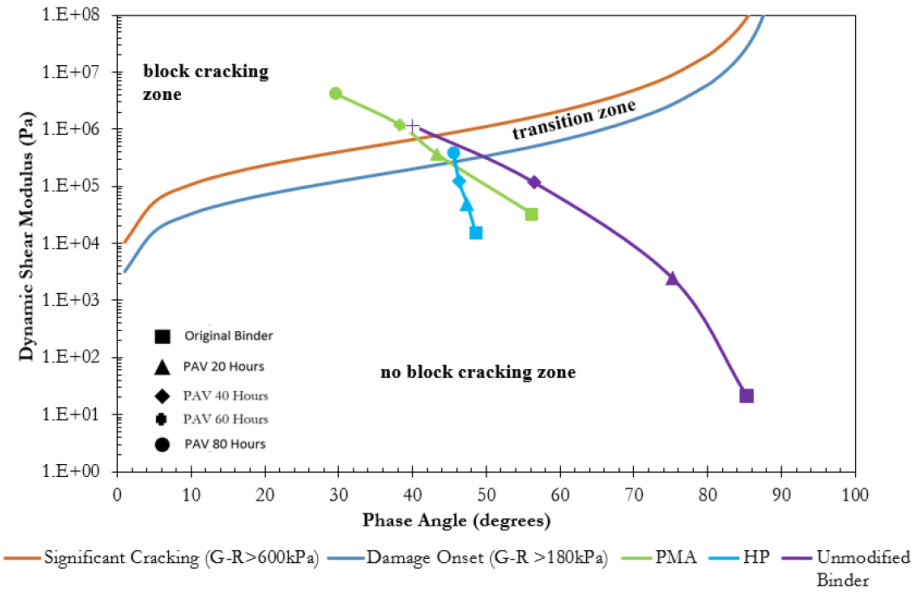


Figure 2-8: Sensitivity of G-R Parameter to Asphalt Binder Additives (Arámbula-Mercado et al., 2019)

Note: WC = Western Canada, GSE = Southeast Gulf and WTX = West Texas

2.4.2.7 DENT Test

The DENT test involves pulling specially configured asphalt binder samples, with one notch at each edge of the sample, from both ends in tensile mode and extracting the essential work of fracture (w_e), the plastic work of fracture (w_p), and the approximate crack-tip opening displacement (CTOD) from the raw load versus displacement data.

Several previous researchers (Edwards and Hesp, 2006; Hesp et al., 2009b; Wright et al., 2011) verified the correlation of parameters obtained from DENT tests, such as approximate CTOD, with the cracking performance of asphalt mixtures and pavements (see Figure 2-9). Based on these correlations, the researchers recommended using asphalt binder

DENT tests to rank asphalt mixtures for low temperature cracking performance instead of using labor- and cost-intensive asphalt mixture DENT tests. More specifically, they recommended using DENT tests to distinguish better-performing from poor-performing asphalt binders. The Ministry of Transportation, Ontario (MTO) has standardized the test as MTO LS-299 (2006), which later got adopted as AASHTO TP113-15 (2015a). Like other fracture-based tests, there is a high chance of variability in this test, mostly coming from sample preparation and instrumentation issues.

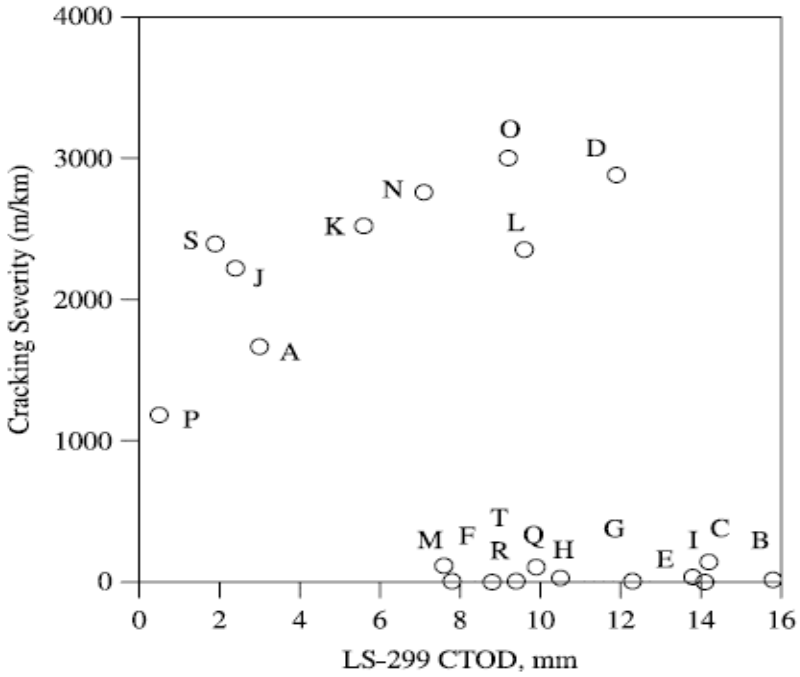


Figure 2-9: Sensitivity of CTOD to Asphalt Binder Additives (Hesp et al., 2009b)

[Note: Each letter and dot combination denote a separate pavement section.]

2.4.3 Asphalt Binder Thermal Cracking Tests

Three types of tests have been primarily used to evaluate the resistance of asphalt binders to non-load induced (thermal) cracking at a low temperature:

- BBR test
- Extended BBR (EBBR) Test
- SENB Test

2.4.3.1 BBR Test

The BBR test (AASHTO, 2016b) is used to determine the low temperature PG (PG_{LT}) of asphalt binders (AASHTO, 2017a). The temperature at which creep stiffness, S is equal to

300 MPa, T_{cs} or the temperature at which creep slope, m is equal to 0.300, T_{cm} or the maximum of these two temperatures, PG_{LT} (AASHTO, 2017a) can be used to determine the cracking performance of asphalt binders at a low temperature. These tests are conducted on PAV-aged asphalt binders.

Researchers are also increasingly using the difference of critical temperatures based on creep stiffness and creep slope, $\Delta T_c = T_{cs} - T_{cm}$ as an indicator of asphalt binder quality and low temperature (thermal) cracking ever since a previous study (Anderson et al., 2011) reported a good correlation of ΔT_c with ductility — a parameter long proven to have good correlation with age-induced surface raveling and cracking (Kandhal, 1977).

Several other researchers (Bennert et al., 2016; Li et al., 2016) have reported that pavements are more likely to show thermal cracking at low temperatures if $\Delta T_c < 0$ or $T_{cs} < T_{cm}$. Because the ΔT_c value can be directly obtained from BBR tests and used to reliably evaluate and compare the quality of asphalt binders, this test parameter has been gaining more attention over conventional asphalt binder quality tests such as the force ductility tests (AASHTO, 2013a) and elastic recovery tests (AASHTO, 2013b).

Recently, TTI researchers (Karki and Zhou, 2018) studied the effect of seven sources of different asphalt binders, five different asphalt binder grades, four different reclaimed asphalt binders, four different dosages of additives (such as one aromatic extract, twelve bio-rejuvenators, eight different fatty acids, one PPA and six different REOB/VTAE) on low temperature PG and ΔTc parameters. Sample test results showed that ΔTc was sensitive to several of these parameters (see Figure 2-10):

- Increasing the use of bio-rejuvenators, aromatic extracts and fatty acids made ΔTc less negative or more positive, signifying the enhancement of asphalt binder quality (increased ductility, increased durability, and possibly higher cracking resistance).
- Increasing the use of REOB/VTAE and aging made ΔTc more negative or less positive, signifying the degradation of asphalt binder quality (reduced ductility, reduced durability, and possibly lower cracking resistance).

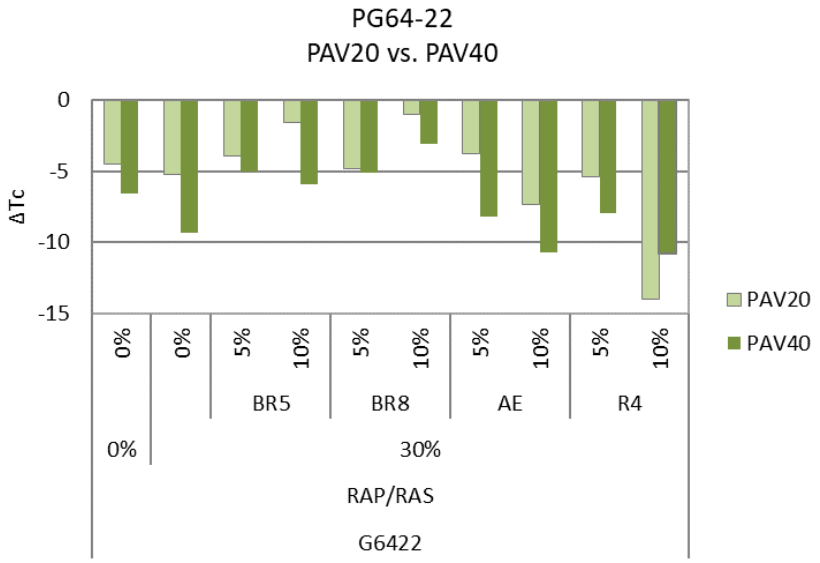


Figure 2-10: Sensitivity of ΔTc to Asphalt Binder Additives (Karki and Zhou, 2018) [Note: AE = Aromatic Extract; BR5, BR8 = Bio-Rejuvenator No. 5 and No. 8; R4 = REOB/VTAE No. 4]

2.4.3.2 EBBR Test

The EBBR test involves conducting BBR tests at PG_{LT}+10°C and PG_{LT}+16°C after preconditioning the beam samples of asphalt binders at PG_{LT}+10°C and PG_{LT}+20°C for 1, 24 and 72 hours to determine parameters such as grade loss and low temperature limiting grade (LTLG) after 72 hours (OMOT, 2011). Researchers (Hesp et al., 2009a, 2007) have reported that, compared to regular BBR-extracted parameters, EBBR-extracted parameters better represent reversible physical hardening at extremely low temperatures and therefore are better correlated to premature low-temperature cracking of asphalt pavements at such temperatures.

2.4.3.3 SENB Test

The SENB test involves loading the asphalt binder beam samples with a notch on one edge in a three-point configuration and determining fracture energies by analyzing force versus displacement data (Iliuta et al., 2004). From this test, parameters such as brittle fracture energy and critical low temperature based on this energy are determined. Researchers reported that SENB-estimated critical low temperatures were always higher than BBR-estimated critical low temperatures and were more correlated to the subnormal temperature at which pavements failed by cracking (Iliuta et al., 2004). As with other

fracture-based tests, this test also has high variability, owing to the complicated sample preparation and instrumentation steps.

2.5 Common Asphalt Mixture Performance Tests

As with asphalt binders, different types of AASHTO and ASTM laboratory test protocols have been developed to characterize asphalt mixtures, often more than one type of test to capture the same type of property, such as the SCB tests, the IDT tests, the disc-shaped compact tension (DCT) tests for characterizing cracking properties and different types of wheel tracking tests to characterize rutting properties.

The ensuing sections summarize the tests used to characterize (a) rutting, (b) cracking, (c) moisture sensitivity, and (d) durability of unmodified and modified asphalt mixtures in the laboratory. The synopses also include the summary of some studies that have discussed the strengths or the weaknesses of these tests in distinguishing the effects of asphalt binder and asphalt mixture additives.

2.5.1 Asphalt Mixture Rutting Tests

Different types of laboratory tests and parameters have been developed to evaluate the rutting resistance of asphalt mixtures over the last 100 years, starting with the Hubbard-Field stability test and Hveem stabilometer tests from the 1920s (Huber, 2013) and culminating with the just recently developed ideal rutting tolerance (IDEAL-RT) test (Zhou et al., 2019):

- Tests Standardized/Developed Before Superpave
    - Hubbard-Field Stability Test
    - Hveem Stability Test
    - Marshall Stability Test
- Tests Standardized/Developed After Superpave
    - Simple Shear Test (SST)
    - Flow Number Test
    - Incremental RLPD (iRLPD) Test
    - APA Test
    - HWT Test
    - IDEAL-RT Test

2.5.1.1 Stability Tests

State DOTs used to evaluate the rutting resistance of asphalt mixtures mainly using stability-based tests, such as the Hubbard-Field stability, Hveem stabilometer or Marshall stability tests for several decades. These tests primarily used a bulk stability parameter instead of a shear parameter as a measure of rutting. After determining that the stability parameters did not have a strong correlation to rutting performance of asphalt mixtures in the field, many state DOTs stopped specifying these tests in their mix designs. Despite recent efforts to relate the stability factor (S-factor) obtained from these tests to the stiffness of asphalt mixtures (Chua and Tenison, 2003), as of 2018, only two state DOTs still specify the Hveem Stabilometer test in their mix design process (West et al., 2018a).

2.5.1.2 Simple Shear Tests

The Strategic Highway Research Program (SHRP), a national effort to modernize asphalt pavement design and analysis, developed a new test called SST as an answer to these stability-based tests (Monismith and Tayebali, 1994). The SST test involves loading cylindrical samples of asphalt mixtures with cycles of shear load and determining permanent shear strain and associated stiffness. Though the SST test rightfully recognized shear as the driving force behind the rutting problem, the parameter did not correlate well to the rutting performance of asphalt mixtures in the field, required relatively expensive instrumentation and involved complex sample preparation steps. As such, most state DOTs have not accepted the SST test as a routine mixture test.

2.5.1.3 FN Tests

The flow number (FN) test was developed as one of the simple performance tests (SPTs) of the National Cooperative Highway Research Program (NCHRP) 9-19 (Witczak et al., 2002). The FN test involves repeatedly loading the cylindrical specimens of asphalt mixtures with a haversine load for 0.1 sec followed by 0.9 sec-long rest periods at a given temperature until failure. NCHRP 9-19 recommended using a stress level between 69–207 kPa without confinement and 483–966 kPa with confinement. The load versus displacement data obtained from this test are used to determine the total number of cycles required to reach the minimum rate of change in the axial strain (i.e., the tertiary flow condition) and corresponding accumulated permanent strain, $\varepsilon_p(FN)$ and time, $t(FN)$. NCHRP 9-19

showed that FN correlated well with the rutting resistance of asphalt mixtures used in test sections at the MnROAD, the WesTrack, and the FHWA Pavement Testing Facility. Based on this and many other studies, the test was included in the HMA mix design manual (NCHRP 9-33, 2011). Several studies, including some by FDOT, have used this test to compare rutting resistance of unmodified and modified asphalt mixtures.

For example, Willis et al. (2014) used the FN, APA and HWT tests to evaluate the resistance of OMA and ARB mixtures. Test results showed that PMA mixtures had lower mean and higher coefficient of variation (COV) values for FN than ARB mixtures. Statistically, they determined that the latter would perform better than the former in terms of permanent deformation. However, both wheel tracking tests showed that these asphalt mixtures were equivalent to each other (HWT test: 1.4 mm versus 1.3 mm and APA: 2.52 versus 2.39 mm).

Similarly, Green et al. (2015, 2014b) used this test along with the HWT test to evaluate the rutting resistance of asphalt mixtures containing SBS-modified, GTR-modified and hybrid (SBS+GTR-modified) asphalt binders. The study showed that FN ranked all three asphalt mixtures similarly.

Recently, Muftah et al. (2017) used the FN and HWT tests together to evaluate the rutting resistance of asphalt mixtures modified with polyolefin and aramid fibers (FM1), wax-treated aramid fibers (FM2) and Type E glass fibers (FM3**)**. Test results showed that fiber-modified mixtures had higher FN values than the control mix (CM), which meant the fibers increased their resistance to rutting. Statistically, analysis of the test results showed the fiber-modified and control asphalt mixtures were equivalent to each other (see Figure 2-11).

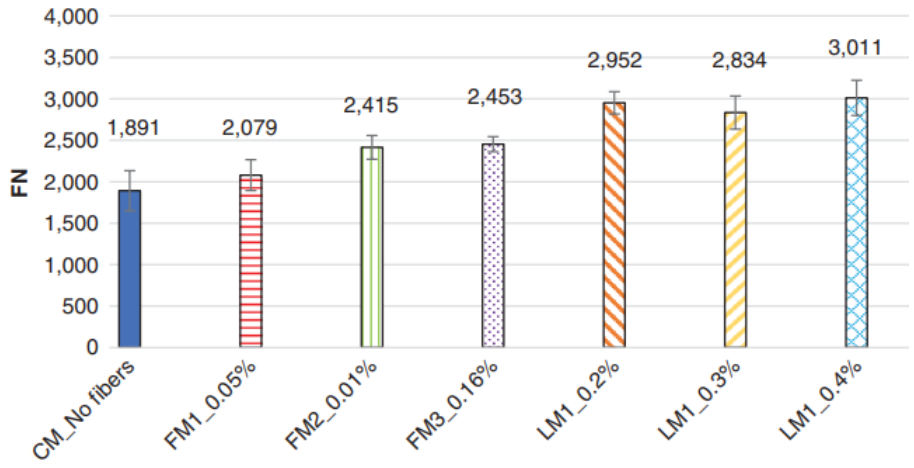


Figure 2-11: Sensitivity of FN to the Use of Fiber (Muftah et al., 2017)

[Note: CM = Control Mix; FM1 = Polyolefin and Aramid Fiber Mix, FM2 = Wax-Treated Aramid Fiber Mix; FM3 = Type E Glass Fibers]

Despite this test being included in NCHRP's manual for HMA design, researchers have identified various limitations to this test (Azari and Mohseni, 2013): (a) the lack of criteria that can be reliably used for discriminating asphalt mixtures, (b) the inability of some asphalt mixtures to show tertiary flow, (c) the variation in testing time to reach the tertiary flow, (d) the use of only one stress level and only one temperature, and (e) not being a pure shear test due to the use of primary and secondary stages of deformation.

2.5.1.4 Wheel-Track Tests

The 1970s and 1990s also saw the development of more empirical wheel-track tests such as the HWT tests and the Georgia Loaded-Wheel test—the precursor of the APA (Lai, 1986; Williams and Prowell, 1999). These instruments simulated permanent deformation under the moving tracks of heavily loaded wheels in a small scale. State DOTs have been mainly using the HWT and the APA tests for determining rutting resistance of asphalt mixtures. According to a recent survey, out of 24 state DOTs in the United States that specify rutting tests in their mix design specifications, 11 state DOTs specify the APA, 10 state DOTs specify the HWT test, and only one state DOT (i.e., Delaware DOT) specifies the FN test, while two state DOTs still specify the almost-absolute Hveem Stabilometer tests (West et al., 2018a).

2.5.1.4.1 APA Test

The APA test involves application of continuous passes of heavily loaded wheels on 75-mm thick, cylindrical specimens at a certain high temperature (usually PG) through pressurized rubber hoses until the total number of wheel passes reaches a DOT-specified number (usually 8,000 cycles) or until the accumulated rut depth reaches a certain value (usually, 12.5 mm or 0.50 inch). Several researchers (Brown et al., 1998; Brown et al., 2001a; Kandhal and Cooley, 2003) have used the APA and the HWT tests to evaluate the rutting performance of modified and unmodified asphalt.

For example, Moseley et al. (2003) used the APA test to evaluate the rutting resistance of mixtures produced with unmodified PG 67-22, polymer-modified PG 76-22, GTR-modified ARB-5 and GTR-modified ARB-12 at 64°C. Test results showed that, though all mixtures passed the rutting criteria, PMA mixture ranked the best, at first followed by the ARB mixtures and then by the unmodified binders in terms of their rutting potential. Test results also showed that increasing the GTR content from 5 percent to 12 percent made mixtures less resistant to rutting (see Figure 2-12).

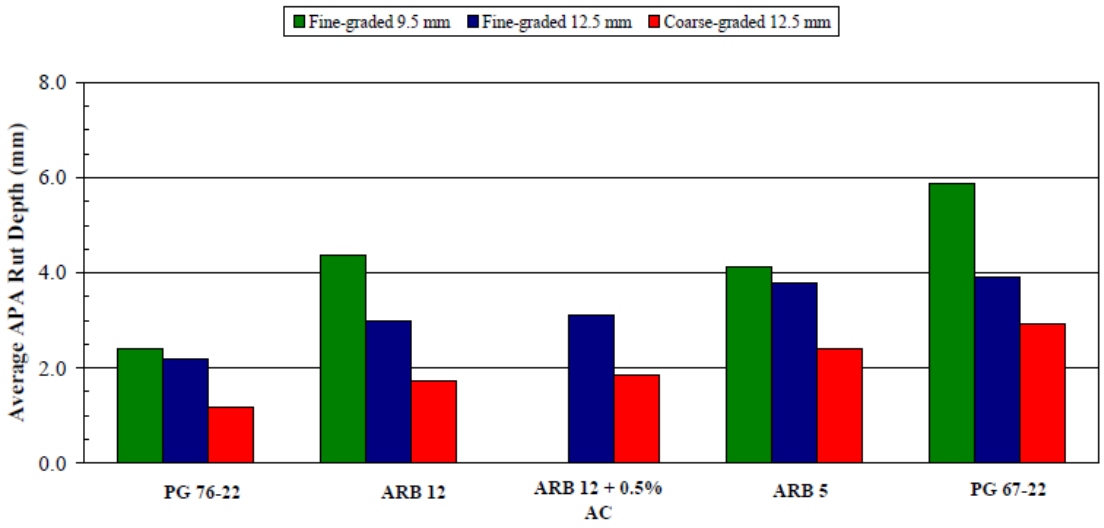


Figure 2-12: Sensitivity of Rut Depth to Asphalt Binder Additives (Moseley et al., 2003)

2.5.1.4.2 HWT Test

The HWT test involves application of continuous passes of heavily loaded wheels directly on 62-mm thick cylindrical specimens at a certain high temperature (usually 50°C) until the number of wheel passes reach a DOT-specified value (usually 20,000 wheel passes)

or until the accumulated rut depth reaches a certain value (usually, 12.5 mm or 0.50 in.), whichever comes first.

Putnam and Amirkhanian (2004) prepared asphalt mixtures with cellulose, polyester, scrap tire, and waste nylon carpet fibers and conducted APA tests to determine their rutting resistance. Researchers did not observe any significant difference in rutting resistance of these asphalt mixtures, which meant that waste fibers, polyester fibers and cellulose fibers were equivalent in terms of their effect on rutting performance.

Likewise, TTI researchers (Zhou et al., 2014) used the HWT tests to evaluate the rutting resistance of asphalt mixtures produced with unmodified and PMA binders, PG 64-22, PG 64-28, PG 64-34, PG 70-22 and PG 76-22 asphalt binders (see Figure 2-13).

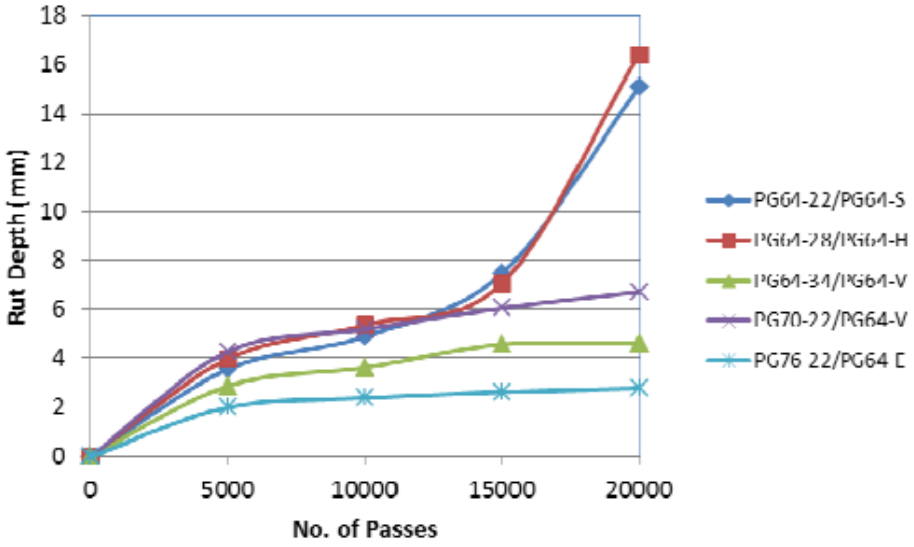


Figure 2-13: Sensitivity of Rut Depth to Asphalt Binder Grade (Zhou et al., 2014)

Manosalvas-Paredes et al. (2016) used the HWT tests as one of the main tests to determine the possibility of avoiding the use of cellulose fiber in SMA asphalt mixtures by replacing PMA binders that contain a moderate amount of SBS with hybrid asphalt binders that contain a relatively higher amount of GTR (8.9 percent) and a lower amount of SBS content. They found that asphalt mixtures containing hybrid asphalt binders and no cellulose fibers were equivalent to asphalt mixtures containing both SBS-modified asphalt binders and cellulose fibers (see Figure 2-14). Researchers concluded that the use of rubber obtained from the end-of-life tires in asphalt binder modification could be an alternative to the use of cellulose fiber in asphalt mixture modification.

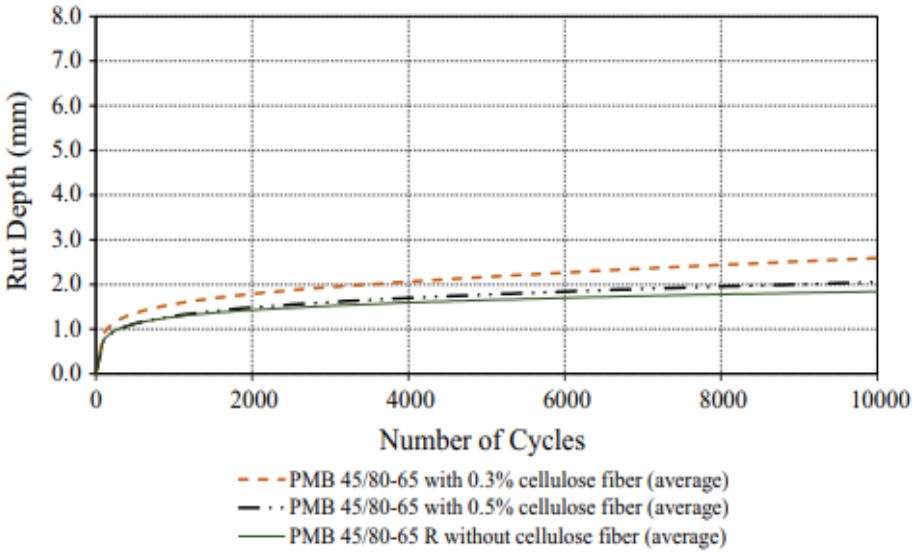


Figure 2-14: Sensitivity of Rut Depth to the Use of Fibers (Manosalvas-Paredes et al., 2016)

2.5.1.5 RLPD Tests

Azari and Mohseni (2013) proposed a new test, called the iRLPD test, as an alternative to the FN test and other previous tests. Unlike the FN test, the iRLPD test (AASHTO, 2015b) involves loading cylindrical specimens of asphalt mixtures to several increments of repeated load cycles at multiple stress levels and determining the permanent or minimum strain rate (MSR) at the end of each test increment, culminating with the construction of the master curve for MSR versus the Product of Temperature and Pressure (TP).

Azari and Mohseni (2013) used iRLPD tests to evaluate the rutting resistance of nine different asphalt mixtures selected by the FHWA Expert Task Group to appraise several existing rutting determination protocols. The researchers compared results of iRLPD test results to three different FN tests conducted with the same confining stress but different deviatoric stresses. Test results showed the strain rates obtained from the relatively slower FN tests coincided on the MSR master curves to results obtained from the comparatively faster iRLPD tests. Based on this comparison, the researchers highlighted several advantages of the iRLPD test over the FN test: (a) shorter duration of testing time, (b) creation of more informative master curve, (c) straightforward calculation of MSR, (d) use of shear mode of loading, (e) well-defined input variables, (f) criteria for discrimination of asphalt mixtures, and (g) wider application.

2.5.1.6 IDEAL-RT Tests

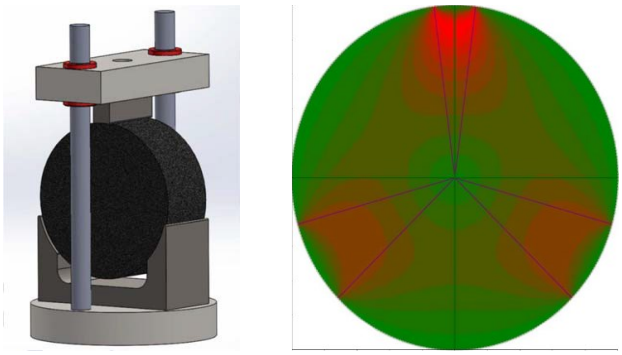
TTI researchers (Zhou et al., 2019) just recently developed yet another test, called the IDEAL-RT, for evaluating the rutting resistance of asphalt mixtures. The new test involves the application of a load on cylindrical specimens (same as the standard HWT test sample without cutting and coring: 150 mm in diameter and 62 mm in thickness) at a rate of 50 mm/min (2 inch/min) at a high temperature (usually, the temperature used for HWT or APA tests) and the determination of a rutting resistance parameter through the relationship of shear strength, τ and peak contact stress, p (see Figure 2-15-a):

$$\tau = 0.356 \, . p$$

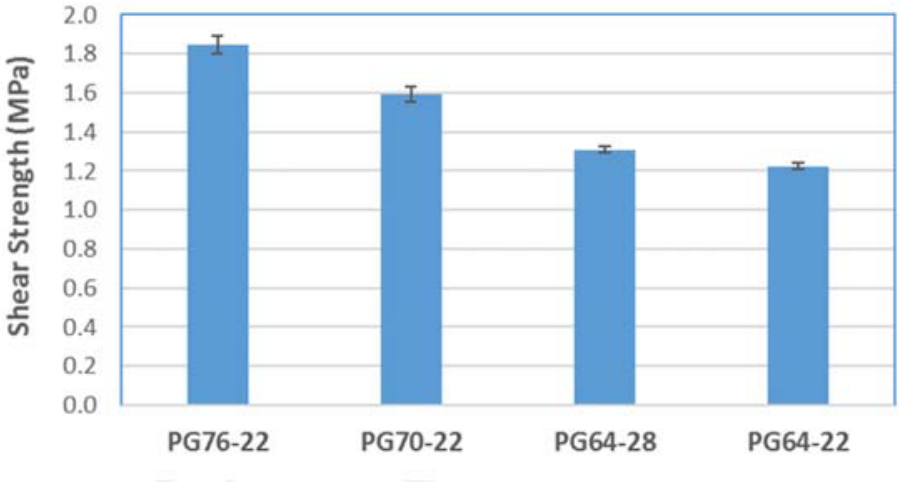
The larger the shear strength, the better the rutting resistance of asphalt mixtures. Through a rigorous experimental design, the researchers showed that IDEAL-RT has the following advantages over other conventional tests:

- Simple: It uses 62 mm-thick x 150 mm-diameter cylindrical samples without coring, cutting, and gluing.
- Practical: It requires a minimum amount of training.
- Economic: It requires the least amount of modification of common test equipment.
- Efficient: It completes within 1 minute.
- Sensitive: IDEAL-RT parameter (or rutting resistance), τ increases with higher grade, lower asphalt binder content, higher RAP/RAS content, more angular aggregates, more severe aging, and lower air void content (see Figure 2-15-b).
- Repeatable: It has less than 10 percent coefficient of variation among three replicates.
- Truly representative: It uses shear mode of damage as the driving force of rutting.
- Correlation with conventional tests: It correlates well with HWT and RLPD tests.
- Field validation: IDEAL-RT has very good correlations with rutting data collected from WesTrack, MnROAD, and in-service Texas test sections (see Figure 2-15-c).

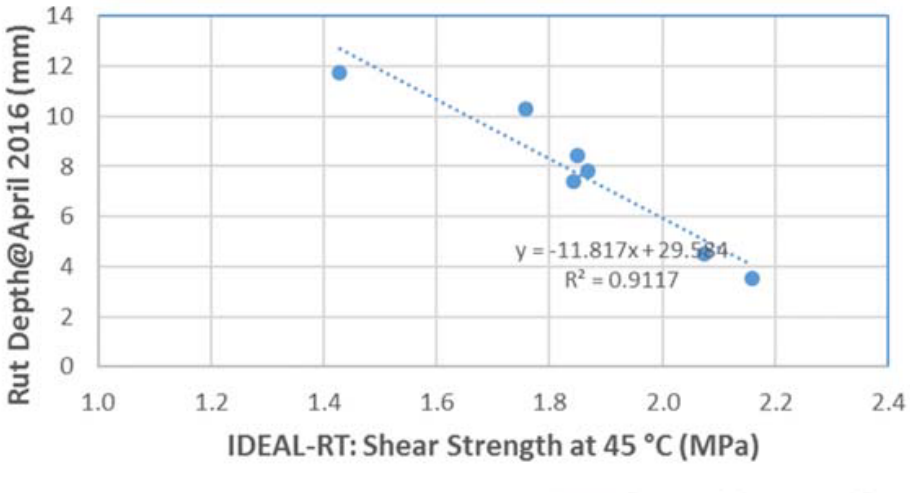
Therefore, researchers deemed this test implementable during the mix design, mix production, and field placement for quality control (QC) and quality acceptance (QA) purposes.



(a) Test Set-Up Features and Shear Stress Distribution



(b) Sensitivity to Asphalt Binder Grade



(c) Validation with the Minnesota DOT Performance

Figure 2-15: IDEAL-RT Test: Features, Sensitivity and Field Validation (Zhou et al., 2019)

2.5.2 Asphalt Mixture Cracking Tests

Cracking is one of the major distresses that cause early failure of asphalt pavements. Over the years, many efforts have been made to understand the mechanics behind asphalt pavement cracking, to determine material properties that describe this mechanism, and to devise test methods that can measure these properties, all for the sole purpose of mitigating cracking in asphalt pavements. Because asphalt pavements exhibit a wide variety of cracking such as bottom-up cracking, top-down cracking and block cracking due to various factors, researchers have taken several different approaches to understand, characterize, measure, and mitigate these various types of cracking.

The literature review shows that these approaches differ from each other in many aspects such as working principle (continuum damage mechanics versus fracture damage), geometry (cylindrical, semi-circular, beam), notch (none, single, double), temperature (intermediate and low) and others. However, based on the literature review, these tests fall into two groups—repeatedly or monotonically loaded cracking tests.

2.5.2.1 Repeatedly Loaded Cracking Tests

These tests apply repeated cycles of load on asphalt mixture specimens at constant frequency and stress or strain (force or displacement) amplitude to induce cracking and use the continuum damage mechanics or empirical relationships to determine parameters that represent their resistance to cracking. Among the very first efforts that used this approach were the WASHO Road Tests and the AASHTO Road Tests, including the work of Hveem (1955). These tests led to the development of the classical fatigue cracking relationship that presents fatigue cracking life, N_f of asphalt mixtures as a function of tensile strain, ε_t and asphalt mixture stiffness, S_{mix} (Monismith et al., 1961):

$$N_f = C.\left(\frac{1}{\varepsilon_t}\right)^{\alpha}.(S_{mix})^{\beta}$$

These tests also stand out because they established (a) the relationship between the thickness of the asphalt concrete layer, the role of deformations of bound layer and the deterioration of asphalt pavements, and (b) the need for the development of laboratory tests that can describe this connection (Underwood and Braham, 2019). This early work laid the foundation of several other tests that use repeated cycles of loading, such as the following tests:

- Repeated Flexural Bending Test
- Direct Tension Fatigue Cracking Test
- Overlay Test
- Modified Overlay Test

2.5.2.1.1 RFB Test

The repeated flexural bending (RFB) test is one of the earliest tests developed for evaluating cracking resistance of asphalt mixtures. The most current version of the RFB test (AASHTO, 2017b) involves repeatedly applying a sinusoidal load at two locations of asphalt mixture beam specimens repeatedly at the two ends and determining the number of load cycles needed to fail them, N_f. The test defines failure as the condition when stiffness drops to 50 percent of its initial value (usually measured at 50th load cycle) or the product of flexural stiffness and load cycle ($S \times n$) when it reaches its peak value. The test has been used to compare the fatigue cracking life of different types of asphalt mixtures and to determine the effects of modification of asphalt binders/asphalt mixtures on their performance.

For example, Souliman and Kaloush (2011) compared the fatigue cracking life of reference asphalt mixtures (unmodified) to the fatigue cracking life of asphalt mixtures containing ARB and PMA binders. They found that ARB mixtures had a life expectancy that is 27 times greater than PMA mixtures which in turn had a life expectancy that is 91 times greater than reference asphalt mixtures, thereby highlighting the ability of this test to differentiate additives based on mix performance.

Similarly, Mateos and Harvey (2019) evaluated the fatigue cracking life of reference asphalt mixtures (unmodified) and aramid fiber-modified asphalt mixtures. They found that fiber-modified asphalt mixtures had a significantly higher value of load cycles at failure than the unmodified asphalt mixtures, especially at high strain levels, highlighting the positive effect of fiber in cracking resistance and the ability of this test to discern such effects in the right conditions. However, despite being based on simple theory, the requirements of a larger sample and a longer testing time make these tests impractical. Thus, this test is not usually designated as a routine asphalt mixture design, QC and QA tests (Kim et al., 2012).

2.5.2.1.2 Direct Tension Fatigue Cracking Test

The direct tension fatigue cracking test (AASHTO, 2018b) involves applying strain-controlled tensile load on cylindrical specimens of asphalt mixtures repeatedly at a constant fixed frequency (10 Hz) and at asphalt binder intermediate temperature, constructing a pseudo-stiffness (C) versus damage (S) curve based on viscoelastic continuum damage (VECD) mechanics, culminating with the estimation of their fatigue cracking lives at selected strain levels using a model that fits the C versus S curve. This model is primarily based on Schapery's work potential theory that describes time rate of damage evolution (dS/dt) in asphalt material as the α-power function of work potential or dissipated pseudo-strain energy, W^R:

$$\frac{dS}{dt} = \left(-\frac{\partial W^R}{\partial S}\right)^{\alpha}$$

FHWA refers to the direct tension fatigue cracking test as an intermediary approach between the classical fatigue cracking tests, such as flexural beam fatigue cracking tests that usually require longer testing time, and the index-based tests, such as IDT tests, that usually require shorter testing time (as discussed in ensuing sections). Researchers have used this test to evaluate and compare the fatigue cracking resistance of asphalt mixtures containing SBS and other alternative additives.

In their study, Underwood et al. (2012) used this test to estimate the fatigue cracking life of asphalt mixtures produced with asphalt binders that had been modified with SBS copolymer, RET and crumb rubber terminal blend (CRTB). Comparing the results of these tests and their performance in FHWA's ALF sections revealed that estimated fatigue cracking life strongly correlated with the fatigue life of mixtures in the field when the results of RET-modified asphalt mixtures were excluded (as warranted by non-uniform density problems related to construction). Nevertheless, the ranking of CRTB and SBS asphalt mixtures were well correlated.

Xie et al. (2015) used this test to compare fatigue cracking performance of four different types of porous European mixes – containing wet-processed SBS-modified asphalt binders (control), dry-processed GTR-modified asphalt binders, dry-processed GTR-modified asphalt binders and the terminal GTR-modified asphalt binders. Test results showed that dry- and wet-processed rubberized asphalt binders exhibited similar resistance to fatigue

cracking but weaker resistance than asphalt binders and SBS-modified asphalt binders (see Figure 2-16). The researchers determined that this ranking held strong despite a change in temperature (0, 10, and 20°C) and loading mode (stress versus strain)—a characteristic strength of VECD-based models (Karki et al., 2016).

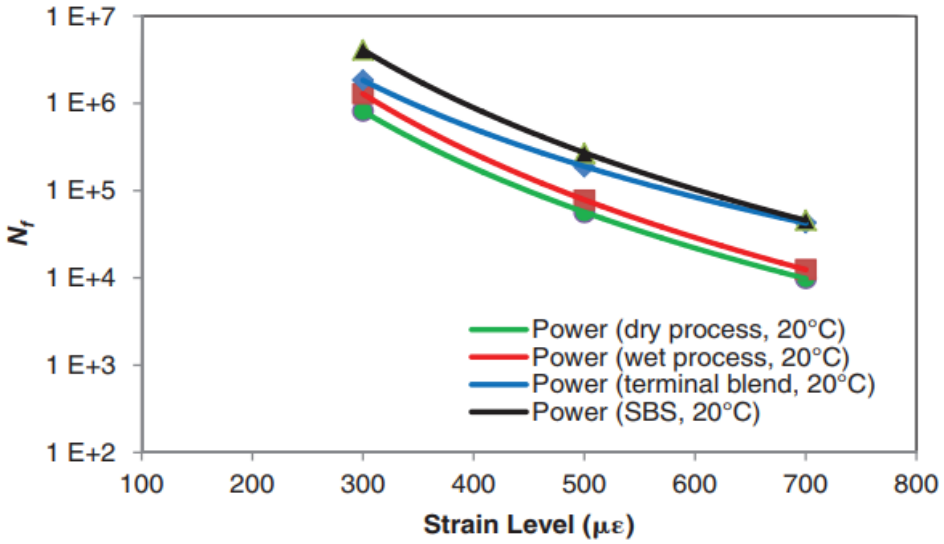


Figure 2-16: Sensitivity of Fatigue Life to Asphalt Binder Additives (Xie et al., 2015)

The main advantage of this test over other alternative tests is the inherent nature of the VECD model itself, due to which C versus S is independent of loading frequency, loading strain/strain level, and temperature (Underwood et al., 2012; Xie et al., 2015). However, the test requires cutting, lengthy sample preparation and testing time and estimates fatigue cracking based on a relatively complex formula instead of direct measurement.

2.5.2.1.3 Overlay Test

Zhou and Scullion (2005) developed an overlay test to evaluate the cracking resistance of asphalt-over-asphalt overlays by refining asphalt-over-concrete overlay tests developed in the 1970s (Germann and Lytton, 1979). The TXDOT-standardized version of this test (TxDOT, 2019) involves gluing one half of a rectangular asphalt mixture specimen to a moveable plate and the other half to a stationary metal plate and applying horizontal movements on the moveable metal plate for 5 seconds followed by 5-second rest periods repeatedly at 25°C until the load reaches 93 percent of the initial peak load or 1000 cycles, whichever comes first. State DOTs have adopted this test as a routine test and set criteria to prevent cracking in asphalt concrete overlays based on test results of local materials. For

example, New Jersey DOT (NJDOT) requires overlay samples produced with PG 64-22 or high RAP content to not fail before 150 load cycles and those produced with PMA binders to not fail before 175 load cycles.

Button and Hunter (1984) used the precursor of the Texas overlay test to evaluate the cracking resistance of asphalt mixtures modified with PP, polyester, aramid, fiber glass, asbestos, a combination of PP and aramid, and a fiber product consisting of cellulose, starch, and ash. Test results showed that fiber-modified asphalt mixtures exhibited less intense cracking over a wider area than control asphalt mixtures, hinting at the ability of fibers to spread the load.

Karki et al. (2019) used this test to evaluate the cracking performance of asphalt mixtures produced with 10 PG 58-XX asphalt binders. The researchers produced these asphalt binders by blending unmodified PG 58-28 asphalt binder with REOB/VTAE (R1 and R4), PPA and RAP-extracted binder together or separately at different proportions. Test results showed that asphalt mixtures performed very differently in terms of cracking resistance (see Figure 2-17). The fact that the same PG delivered different cracking resistances in asphalt mixtures is noteworthy. However, these binders were not used in the field and therefore the ranking was not fully corroborated.

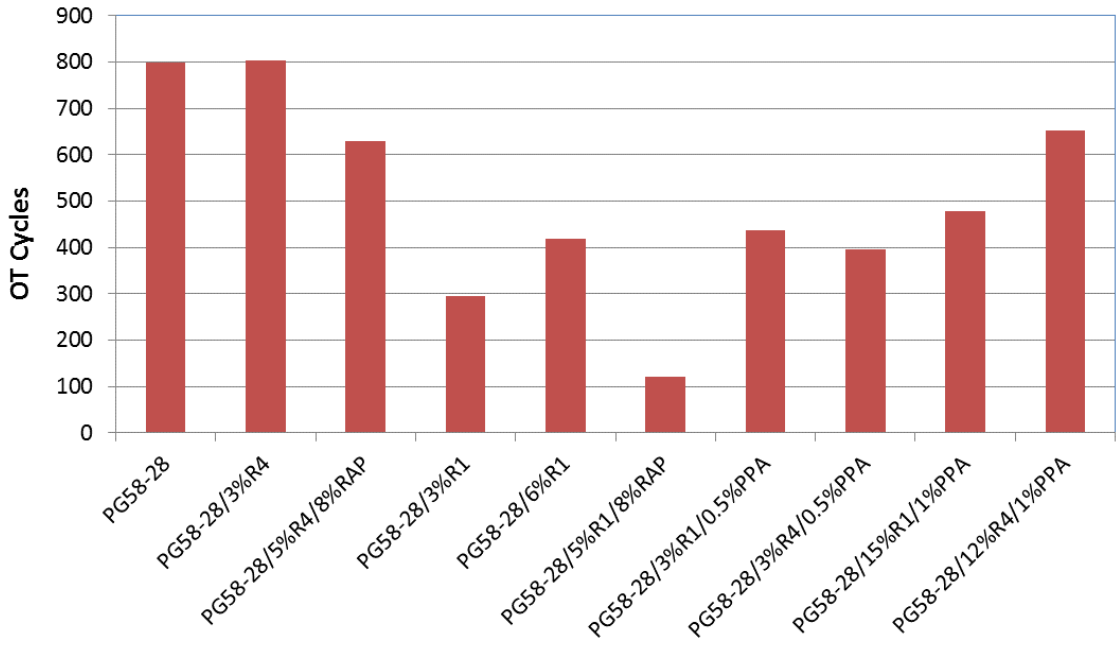


Figure 2-17: Sensitivity of OT Cycles to Asphalt Binder Additives (Karki et al., 2019)

Despite its well-reported simplicity and effectiveness, the test has several limitations such as high variability (30–0 percent COV), the need to select three close results out of five minimum tests to deal with this COV, and the requirement of separate conditioning and testing equipment.

2.5.2.1.4 Modified Overlay Test

To address its high variability, some researchers (Ma, 2014) recently applied three modifications to this test: (a) a reduction in the total duration of the loading and unloading cycle from 10 seconds to 1 second, (b) a reduction in maximum displacement from 0.635 mm to 0.381 mm, and (c) the use of the peak value of "maximum load × load cycle" instead of 93 percent initial peak load as the failure criteria. The researchers reported that these modifications reduced COV value from 30–50 percent to 20–30 percent, which is still high.

2.5.2.2 Monotonically Loaded Cracking Tests

These tests apply load on asphalt mixture specimens at a constant rate of displacement or strain to induce cracking and use the fracture damage mechanics to determine parameters that represent their resistance to cracking. Some of the early work that used this approach include the bending tests of beams (Little and Mahboub, 1985) and the IDT tests of cylindrical samples of asphalt mixtures with a notch at the center of the bottom surface (Abdulshafi and Majidzadra, 1985). These early works provided the foundation for most cracking tests that have been developed since then to evaluate the cracking resistance of asphalt mixtures. Based on loading mode and specimen geometry, these tests can be divided into two groups:

- Flexural Tests of Notched Semi-Circular Beams
  - SCB Test
  - SCB-Flexibility Index (SCB-FI) Test
  - Low Temperature SCB Test
- IDT Tests of Full Cylindrical Specimens
  - Creep Compliance and Tensile Strength Test
  - Energy Ratio Test
  - IDEAL-CT Test

2.5.2.2.1 SCB Tests

Mull et al. (2002) were the among the first researchers who recommended running the bending tests on notched semi-circular beams (SCBs) instead of running such tests on notched beams (Little and Mahboub, 1985) or running IDT tests on notched cylinders (Abdulshafi and Majidzadra, 1985) to evaluate the cracking performance of asphalt mixtures. These studies mainly characterized the fracture behavior of asphalt mixtures at a low temperature using the nonlinear elastic energy release rate — the J-integral, J_c (Rice, 1968). Theoretically, materials that have higher resistance to cracking need a larger amount of energy to initiate microcracks and propagate them, and therefore have a higher J_c value. Mull et al. (2002) applied a vertical load on SCB with three different notch depths at a constant rate in a simply supported three-point beam configuration. They determined fracture energy (U) per unit thickness (b) for each notch depth and determined the value of J_c integral from the slope of the U/b versus notch depth (a) curves:

$$J_c = -\frac{1}{b}\left(\frac{dU}{da}\right)$$

Mull et al. (2002) used this test to compare the fracture resistance of asphalt mixtures produced with three types of asphalt binders — air-blown, unmodified PG 70-28 asphalt binders (control), plain crumb-rubber-modified PG 70-22 asphalt (CRA) and chemically modified crumb rubber PG 76-28 asphalt (CMCRA) binders. Test results showed that the unmodified mix, the plain crumb rubber-modified mix and the chemically modified crumb rubber mix had an increasing order of J_c value (0.54, 0.65 and 1.23 kJ/m2, respectively), highlighting the positive effect of crumb rubber on cracking resistance and elevating this effect even more by chemically modifying the crumb rubber (see Figure 2-18). More importantly, these results displayed the ability of the SCB test and that of the J_c parameter to differentiate the effect of different types of asphalt binders/mixture additives on the cracking resistance of asphalt mixtures.

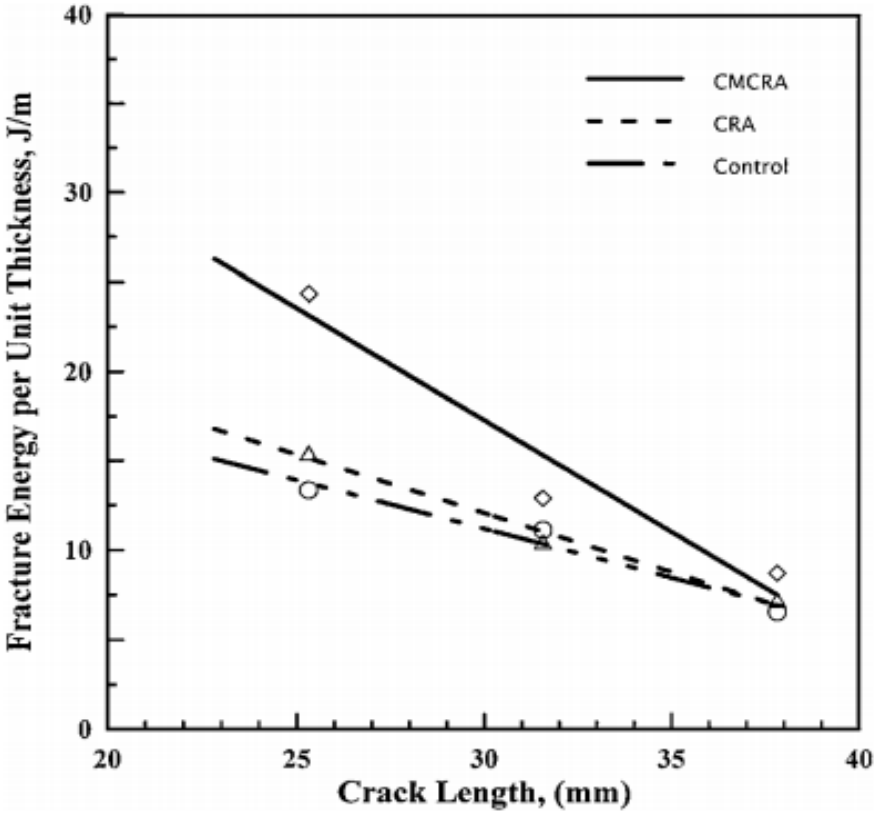


Figure 2-18: Sensitivity of Fracture Property to Asphalt Binder Additives (Mull et al., 2002)

[Note: Control = Air-Blown, Unmodified PG 70-28 Asphalt; CRA = Plain Crumb-Rubber-Modified PG 70-22 Asphalt; CMCRA = Chemically Modified Crumb Rubber PG 76-28 Asphalt]

Since its introduction, several researchers have used the SCB test to evaluate the cracking resistance of asphalt mixtures with different modifications, resulting in one standard for low temperatures (AASHTO, 2015c) and two standards for intermediate temperatures (AASHTO, 2018c; ASTM, 2016a).

The current version of the low temperature SCB test (AASHTO, 2015c) involves fabricating SCB specimens with a notch ($a = 15 \pm 0.5$ mm; width ≤ 1.5 mm) at the center of the flat side of the specimens and loading them at a constant crack mouth opening displacement (CMOD) rate of 0.0005 mm/min and at a local climate-dictated low temperature until failure. Unlike the intermediate temperature SCB test, the low temperature SCB test uses a linear variable differential transformer to measure the CMOD rate and a very

slow rate of loading. The analysis part of this test involves extracting fracture energy (G_f) from the load versus displacement (P versus u) curve and fracture toughness (K_{IC}).

The current version of the intermediate temperature SCB test (ASTM, 2016a) involves fabricating SCB specimens with a notch (a = 25 ± 1, 32 ± 1 and 38 ± 1 mm, width ≤ 3.5 mm) at the center of the flat side of the specimens and loading them at a constant load-line displacement rate of 50 mm/min (2 inch/min) at a local climate-dictated intermediate temperature until failure. The analysis part of this test involves calculating the fracture energy for each notch depth following loading (Mull et al., 2002). The intermediate temperature SCB test has been used to evaluate the cracking resistance of both unmodified and modified asphalt mixtures.

For example, Mohammad et al. (2004) used this test to evaluate the cracking resistance of asphalt mixtures produced with different combinations of virgin and recycled SBS PMA binders. Test results showed that the J_c value increased with an increase in recycled PMA binder content (0.55 $kJ/m2$ at 0 percent and 0.65 $kJ/m2$ at 60 percent).

Similarly, Wu et al. (2005) used this test to evaluate the cracking resistance of 13 different asphalt mixtures designed with different asphalt binder grades (unmodified PG 64-22 and PG 70-22, PG 70-22 PMA, and PG 76-22 PMA), aggregate gradations [25.0- and 19.5-mm nominal maximum aggregate size or NMAS] and compaction levels (75, 97, and 125 gyrations). Test results and statistical analysis showed that the J_c value obtained from multiple notch depths was sensitive to mix design variables, asphalt mixtures with larger NMAS and stiffer, PMA binders more resistant to cracking.

Likewise, Cooper et al. (2016) also used this test to evaluate the cracking resistance of 51 asphalt mixtures produced with different types of asphalt binders and to determine if they complied with the minimum value of J_c (0.5 kJ/m2) set by Louisiana DOT. Test results showed that significantly more PMA mixtures (91 percent of PG 76-22 mixtures and 68 percent of PG 76-22 mixtures) passed this criterion than did the unmodified and GTR-modified asphalt mixtures (38 percent of PG 64-22 mixtures and 20 percent of PG 82-22 mixtures). Based on test results, they recommended using two different minimum criteria for asphalt mixtures prepared based on the high temperature G of asphalt binders (0.6 kJ/m2 for PG 76 and higher while 0.5 kJ/m2 for others).

Recently, Mohammad and Cao (2017) used this test to study the effect of modifying asphalt mixtures with RAP and showed that the J_c value decreased with an increase in RAP percentage (see Figure 2-19). This study proved that RAP had a negative effect on cracking resistance, which was in line with field experience.

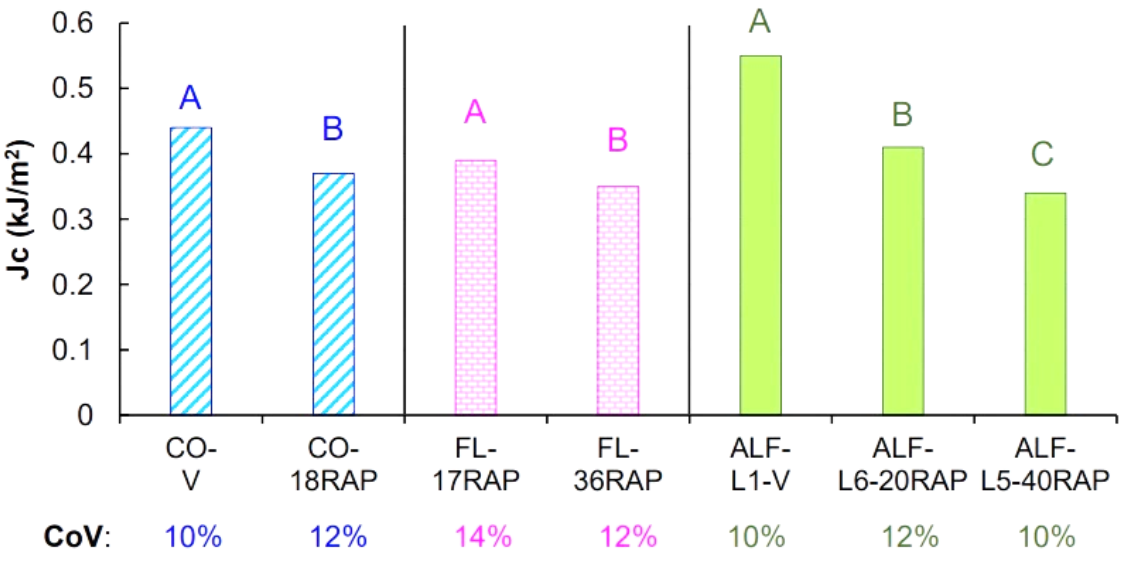


Figure 2-19: Sensitivity of J-Integral to the Use of Recycled Materials (Mohammad and Cao, 2017)

These studies revealed various advantages of the SCB test over other tests (Mull et al., 2002; Wu et al., 2005):

- Possibility of using gyratory-compacted or field-cored samples.
- Possibility of obtaining more than one specimen from a single core/compacted sample.
- Reduction in variability caused by heterogeneity from different cores.
- Minimal deformation due to the self-weight of the specimen.

However, West et al. (2018b) recently reported the following issues with this test:

- Lack of a measure of variability for the J_c value, making it impossible to conduct a statistical comparison.
- Higher number of specimens than other tests due to the use of three notch depths.
- Longer time for sample preparation and testing time.
- Higher cost of equipment than the IDEAL-CT test as discussed later.
- Poor correlation with other tests and field performance.

- High coefficient of variation.

2.5.2.2.2 SCB-FI Test

Al-Qadi et al. (2015) noticed an inconsistent correlation of fracture energy with field performance. They instead observed that asphalt mixtures that performed well in terms of cracking exhibited less steep post-peak slopes and significantly more ductile failure than asphalt mixtures that performed poorly in the field. They also noticed that the post-peak slope of the load versus displacement curve was more sensitive to material properties and loading conditions than was the fracture energy. Thus, the researchers proposed a different version of the SCB test, which has been recently standardized as AASHTO TP124-16 (2018c). The new test uses only one notch depth instead of three notch depths and yields a new index for fracture resistance, which is called the flexibility index (FI). The analysis part of this test involves calculating the work of fracture from load (P) versus displacement (u) data until failure (W_f), calculating fracture energy (G_f) by dividing the W_f value by the ligament area (A = specimen thickness x ligament length) and finally calculating the FI value by dividing the G_f value by the absolute value of the post-peak slope at the inflection point (m) and multiplying by a unit conversion or scaling factor (C):

$$FI = \frac{G_f}{|m|}$$

where,

$$G_f = \frac{W_f}{A}$$

where,

$$W_f = \int P \times du$$

$$A = t \times (r - a)$$

where,

t = thickness

a = notch depth

r = radius

In their study, Al-Qadi et al. (2015) evaluated the FI value of asphalt mixtures produced with 6.0 percent PG 70-22 SBS, PG 64-22, PG 58-28 or PG 52-34 and 0–40 percent RAP and/or 0–7 percent RAS by weight of mixture. Test results showed that the FI captured the

effect of the asphalt binder grade (positive), field aging (negative) and RAP/RAS use (negative) on the cracking resistance of asphalt mixtures. Test results also exhibited a good correlation between the FI and the field performance (see Figure 2-20). Several recent studies have since used the FI to distinguish differences in asphalt mixture properties.

Ozer et al. (2016) determined that asphalt mixtures with excellent cracking resistance had a FI value of 10.0 or above while the ones with poor cracking resistance had a FI value of 6.0 or below. Al-Qadi et al. (2017) later found the minimum value of 8.0 for the FI to distinguish good-performing from poor-performing asphalt mixtures.

Kaseer et al. (2018) used this test to evaluate the cracking resistance of field-collected asphalt mixtures obtained from three states in the United States. The asphalt mixtures contained different combinations of PG 58-22, PG 58-22, PG 64-22 or PG 70-22 unmodified asphalt binders, RAP and RAS, and recycling agents. In addition to the FI, they also calculated a new parameter called the cracking resistance index (CRI) obtained by dividing G_f with peak load, P_{max}. They showed the FI and CRI both were sensitive to asphalt binder grade, aging condition, RAP/RAS contents and rejuvenating agent dosages.

Batioja-Alvarez (2019) used this test to evaluate the cracking resistance of plant-mixed, laboratory compacted and plant-mixed, field compacted specimens of asphalt mixtures obtained from Indiana. The asphalt mixtures contained different combinations of PG 64-22 and PG 70-22 unmodified asphalt binders and 0–32 percent RAP. From the SCB tests, they determined the FI and the CRI. They showed that both FI and CRI parameters were sensitive to asphalt binder grade, aging condition, RAP/RAS contents and rejuvenating agent dosages.

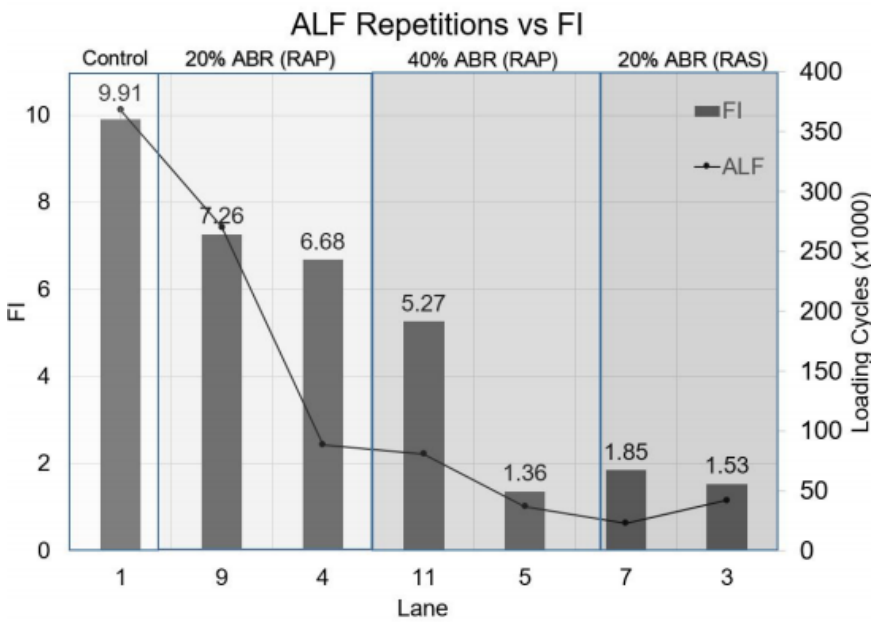


Figure 2-20: Sensitivity of FI to Change in Recycled Material Content (Al-Qadi et al., 2015)

Despite these attested sensitivities of the FI parameter, the test itself still has some issues:

- Difficulty in obtaining the FI parameter of some asphalt mixtures that have high RAP/RAS contents (Barry, 2016; Zhou et al., 2017a) because of a lack of enough reliable data to calculate post-peak slope at the inflection point due to their brittle failure.
- Difficulty in obtaining the FI parameter for some of the asphalt mixtures that have been aged for an extended period of time (Kaseer et al., 2018) because these asphalt mixtures undergo brittle failure, resulting in inadequate data to calculate the post-peak slope at inflection point.
- High variability when the load-displacement curves are not smooth and, as such, do not have well-defined inflection points (Kaseer et al., 2018).
- Insensitive to some mix design parameters such as asphalt binder content (Kaseer et al., 2018).
- Need of correction factors for air void content and specimen thickness, especially for field samples (Kaseer et al., 2018).
- High COV due to variability in post-peak slopes (Batioja-Alvarez et al., 2019).

2.5.2.2.3 IDT Test

The IDT test was developed as part of the creep compliance and tensile strength tests — one of the outcomes of SHRP — to measure the resistance of asphalt mixtures to low temperature cracking (Buttlar and Roque, 1994, 1992). The IDT part of creep compliance and tensile strength tests (AASHTO, 2007) primarily involves loading cylindrical specimens with a vertically moving ram on one diametrical end at a certain rate (12.5 mm/min or 0.5 inch/min) while being supported at the other diametrical end, thereby exerting tensile strain on the specimens indirectly at a right angle to the line of the load application. The load and displacement curve obtained from this test is used to extract several different parameters such as peak load, strain at peak load, strain at failure and ITS. The test has been modified several times for use as both a simple performance test in mix design and a materials characterization test in structural design (Christensen and Bonaquist, 2004). Due to its simplicity, this test has been used as the foundation of several other tests such as the energy ratio test developed in the 2000s (Roque et al., 2004) and the most recently developed ideal cracking tolerance (IDEAL-CT) test (Zhou et al., 2017a).

2.5.2.2.4 Energy Ratio Test

Roque et al. (2004) derived a new parameter, called energy ratio (ER), to distinguish asphalt mixtures that have different resistance to top-down cracking (surface-initiated longitudinal wheel path cracking). The researchers recommended determining the total amount of energy required to completely fail, also called dissipated creep strain energy at failure, $DCSE_f$ by running IDT tests on cylindrical specimens of asphalt mixtures at 10°C. They also recommended determining the minimum amount of DCSE required to produce 2-inch-deep cracks, $DCSE_{min}$ by running creep compliance and resilient modulus tests. They defined the ratio of these two energies as the ER parameter — an index that represents the resistance of asphalt mixtures to cracking:

$$ER = \frac{DCSE_f}{DSCE_{min.}}$$

In their study, Roque et al. (2004) conducted these tests on asphalt mixtures obtained from several field test sections and compared their $DSCE_{min.}$ and ER values. Test results showed that asphalt mixtures with a record of early cracking had a lower value of ER than asphalt mixtures with a record of low or no cracking. Based on these results, the researchers

recommended using the minimum value of 1.0 for ER and 0.75 $kJ/m3$ for $DSCE_{min}$ as thresholds for differentiating well-performing asphalt mixtures from poorly performing asphalt mixtures.

Willis et al. (2014) also used this test to evaluate the cracking resistance of rubber and PMA mixtures. Test results showed that ARB mixtures had higher ER (6.68) than PMA mixtures (5.42). Though both asphalt mixtures passed the minimum criteria of ER recommended for a traffic volume of 1,000,000 ESALS per year or less, the results showed that the ER could differentiate two asphalt mixtures from each other.

Similarly, Yan et al. (2017) used this test to evaluate the cracking resistance of DGA mixtures (structural FC) produced with SBS-modified PG 76-22 PMA, two different sources of virgin aggregates, and three different dosages and two different sources of RAP following the FDOT's FC12.5 design. Test results showed that the ER value decreased differently with an increase in RAP content (20, 30, and 40 percent by total weight of mixture) depending upon the source of aggregates. They also reported that each of seven asphalt mixtures (one control, six RAP-modified) passed the minimum criteria of $DSCE_{min.}$ and ER, signifying the effectiveness of the use of PMA asphalt binders.

Recently, Moore and Timm (2019) used this test to evaluate the effect of RAP, RAS, GTR, and high polymer modified asphalt (HPMA) on the cracking resistance of asphalt mixtures. Researchers compared the rankings determined from the ER and field performance. Test results revealed contradictory correlations, such as the GTR mix had the lowest ER and the 5 percent RAS mix had the highest ER. They cautioned that continuous monitoring of the full-scale sections was still warranted to refute or validate the use of ER in such cases.

2.5.2.2.5 IDEAL-CT Test

TTI researchers (Zhou et al., 2017b) recently developed a new cracking test called the IDEAL-CT test for determining the cracking resistance of asphalt mixtures. TTI researchers developed this method as the key product of the NCHRP-IDEA 20-30/IDEA 195 Project. The test has been recently standardized as ASTM D8225 (2019). The test involves loading the 38–75 mm thick SGC specimens with IDT at a constant displacement rate of 50 mm/min (2 inch/min) diametrically. The analysis step of this test involves determining the fracture energy (G_f) from the total area under the load (P) versus displacement (l or u) curve, the absolute slope of this curve at 75 percent post-peak load (m_{75}), the post-peak displacement at

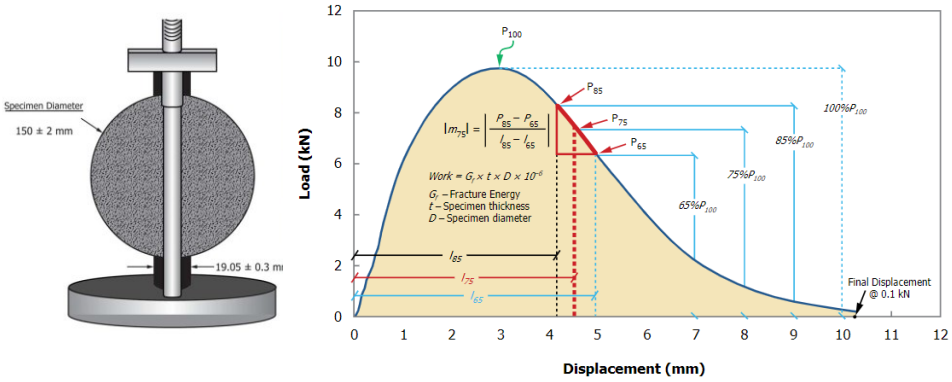
75 percent peak load (u_{75}), and an index for cracking called the cracking test index (CT_{index}) with correction applied for thickness (t) (see Figure 2-21(a)):

$$CT_{Index} = \frac{t}{62} \times \frac{G_f}{|m_{65}|} \times \left(\frac{u_{75}}{D}\right) x10^6$$

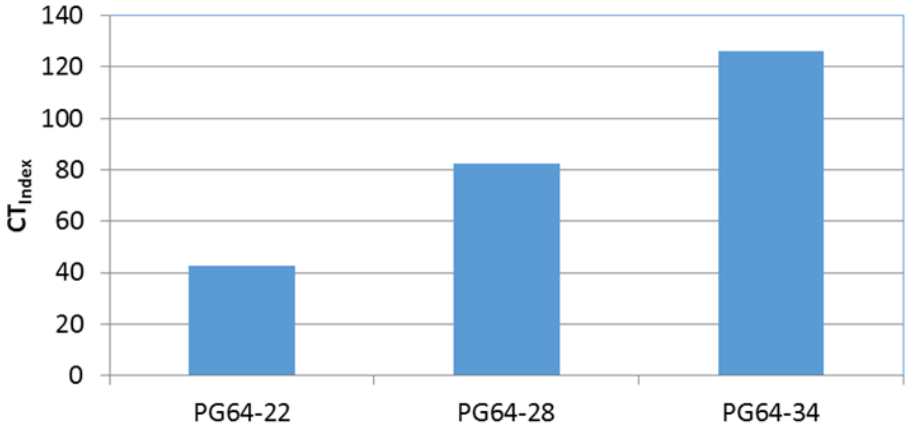
Ideally, asphalt mixtures with a higher CT_{index} are more tolerant to cracking and perform better than their counterparts. The researchers conducted these tests using 17 different asphalt mixtures and determined that the test offered the following attributes:

- Simple: no instrumentation, cutting, gluing, drilling, or notching.
- Repeatable/reproducible: COV < 20 percent.
- Efficient: test completion within minutes.
- Less expensive: existing or inexpensive equipment or parts.
- Practical: a minimum amount of training needed for routine operation.
- Sensitive: sensitive to asphalt mix characteristics (see Figure 2-21(b)).
- Representative: Good correlation with field test sections (see Figure 2-21(c)).

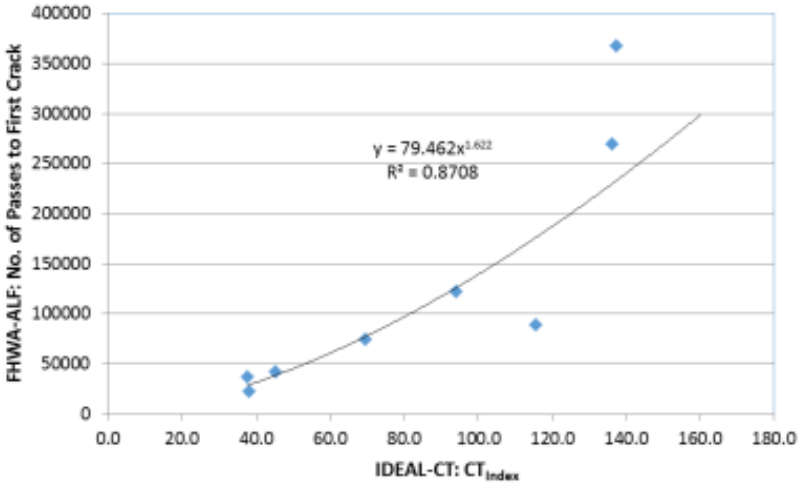
West et al. (2018b) recently reported that the IDEAL-CT and the I-FIT tests ranked asphalt mixtures similarly, owing to the use of post-peak analysis used in both methods. They also reported that the COV for seven different reheated plant-produced mixtures were very comparative. Based on the Pearson correlation analysis, they determined that IDEAL-CT results were closely correlated to Texas overlay and I-FIT test results. Furthermore, the researchers found that sample preparation was faster in the IDEAL-CT than in the other five cracking tests because there was no need of cutting and notching the samples in IDEAL-CT. They also found that IDEAL-CT and I-FIT were equivalent to each other but better than the other four cracking tests in terms of equipment cost and testing time.



(a) Test Set-Up Features and Parameter Definitions



(b) Sensitivity to Asphalt Binder Grade



(c) Validation with FHWA-ALF Performance

Figure 2-21: IDEAL-CT Test: Features, Sensitivity and Field Validation (Zhou et al., 2017b)

2.5.3 Asphalt Mixture Moisture Susceptibility Tests

Moisture damage is one of the major contributors of premature distresses in asphalt pavements (Epps et al., 2003; Little et al., 2006). Researchers have made several efforts to better understand the mechanism of stripping or the loss of asphalt binder film from the surface of aggregates due to moisture (McGennis et al., 1984; Kennedy and Anagnos, 1984; Putnam and Amirkhanian, 2006), to devise tests and criteria that can accurately discriminate moisture-susceptible asphalt mixtures from moisture-insusceptible asphalt mixtures (Al-Swailmi and Terrel, 1992; Aschenbrenner et al., 1995; Emery and Seddik, 1997; Lottman, 1982; McGennis et al., 1984; Newcomb et al., 2015; Tunnicliff and Root, 1984) and to identify solutions to mitigate or reduce stripping, such as the use of hydrated lime and liquid anti-stripping agents (Aschenbrenner, 2003; Sebaaly et al., 2007).

These studies revealed that moisture damage or stripping depends not only on factors that cannot usually be controlled such as the environment and traffic but also on factors that can be controlled such as the aggregate type, mix design, and production, including asphalt binder grade and modification. These studies also developed several test methods that essentially measure and compare the values of certain parameters of asphalt mixtures with water (e.g., ITS in dry or unconditioned and wet or conditioned states and wet-to-dry IDT strength ratio, TSR from modified Lottman tests):

- Compacted Mixture Tests
    - IDT Tests
        - Tunnicliff-Root Test (Tunnicliff and Root, 1984)
        - Lottman Test (Lottman, 1982)
        - Modified Lottman Test (AASHTO, 2014c)
    - Compression Tests
        - Immersion Compression Test (AASHTO, 2002)
        - Triaxial Test (Al-Swailmi and Terrel, 1992)
    - Marshall Stability Tests
    - Immersion Marshall Stability Test (Ministry of Transportation (MTO), 2017)
    - Wheel Tracking Tests
        - HWT Tests (Aschenbrenner et al., 1995)

    - Others
        - Texas Freeze-Thaw Pedestal Test (Kennedy et al., 1984)
        - Asphalt Absorption & Desorption Test (Emery and Seddik, 1997)
- Loose Mixture Tests
    - Boiling Water Test (ASTM, 2012)
    - Coatability Index Tests (Newcomb et al., 2015)

State DOTs have adopted different types of tests based on simplicity, practicality, efficiency, repeatability, sensitivity, and their correlation to actual performance in local pavement sections. A survey of 55 transportation agencies in the United States and Canada (Aschenbrenner, 2003) revealed that the overwhelming majority of the state DOTs use the IDT tests (modified Lottman tests) to conduct moisture susceptibility tests at the time of the survey. Some state DOTs also use the boiling tests as the screening tests in conjunction with other mechanical tests.

Based on the literature review, most recent studies have used or recommended using the Lottman-modified IDT and the ITS value obtained from this test to discriminate whether asphalt mixtures prepared with selected materials would satisfy moisture susceptibility criteria.

Saboo and Kumar (2016) conducted these tests on four bituminous concrete (BC), four dense bituminous macadam (DBM) and two SMA mixtures prepared with unmodified (VG10 and VG30), EVA- and SBS-polymer-modified [PMB(E) and PMB(S)] asphalt binders. Test results showed that TSR and ITS both ranked PMA mixtures higher than unmodified asphalt mixtures. The results also showed that TSR and ITS both also ranked asphalt mixtures prepared with EVA-modified asphalt binders [PMB(E)] higher than asphalt mixtures prepared with SBS-modified asphalt binders [PMB(S)].

Othman et al. (2017) conducted IDT tests to evaluate moisture sensitivity and cracking resistance of asphalt mixtures containing unmodified asphalt binder (Pen. 60/70), polymer-modified PG 76 asphalt binder, hydrated lime-modified (HL) asphalt binder, 2 and 4 percent Portland cement-modified (CEMENT) asphalt binders, 2 and 4 percent nano-clay-modified (NANO CLAY) asphalt binders. Test results showed that all but two (control and hydrated lime modified) asphalt mixtures met the 80 percent lower limit for the TSR value of the Superpave specifications (see Figure 2-22). The results also showed that TSR ranked PG 76

PMA, NC-modified (4 and 2 percent NC) and PC-modified (2 and 4 percent CEMENT) in an ascending order in terms of their ability to resist moisture damage.

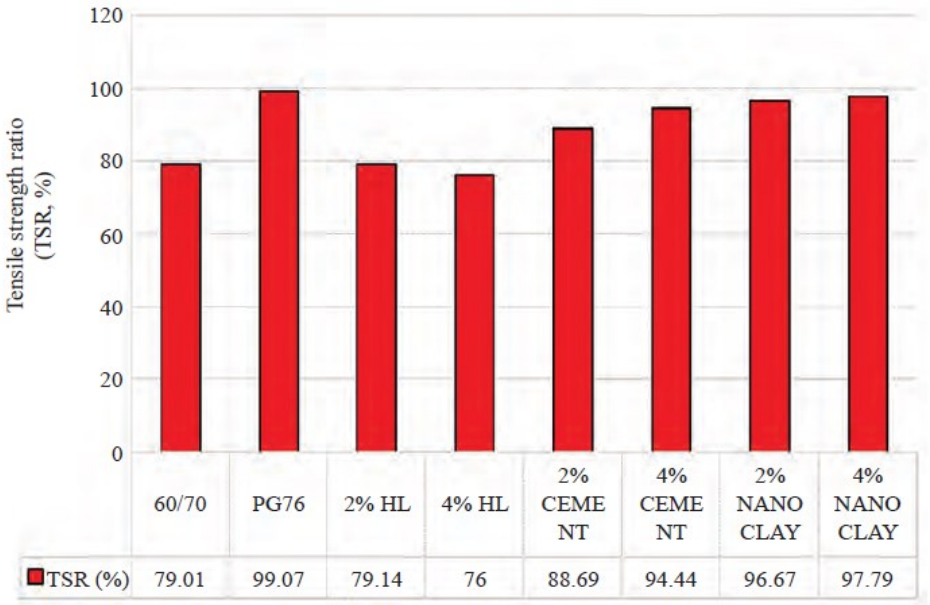


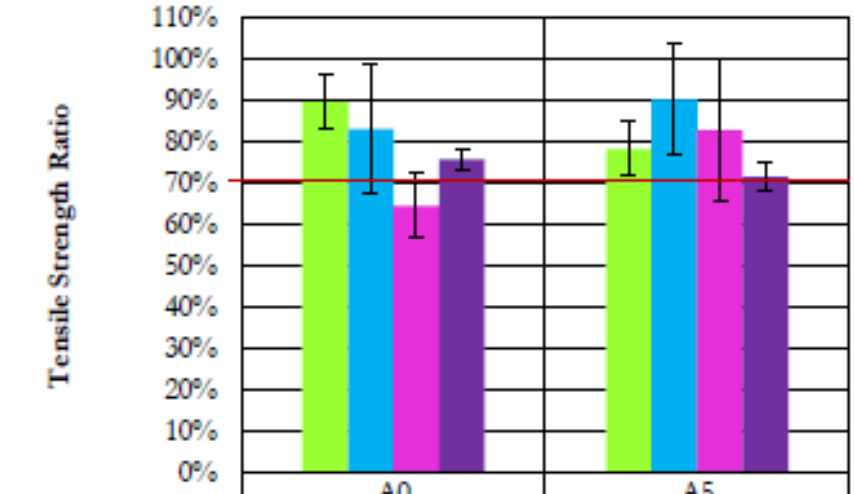
Figure 2-22: Sensitivity of TSR to Asphalt Binder Additives (Othman et al., 2017)
[Note: HL = Hydrated Lime; CEMENT = Portland Cement; NANO CLAY = Nano Clay]

Manosalvas-Paredes et al. (2016) compared the moisture susceptibility of one GTR-modified asphalt mixture and two fiber-modified asphalt mixtures by conducting IDT tests before and after moisture conditioning. Asphalt binders used to prepare fiber-modified asphalt mixtures were wet-modified with SBS while asphalt binders used to prepare GTR-modified asphalt mixtures were wet-modified with GTR and SBS. Test results showed that GTR-modified, 0.3 percent fiber-modified and 0.5 percent fiber modified asphalt mixtures resulted in a TSR value of 96.9, 95.5, and 95.8 percent, respectively. The similar TSR values simply highlight the fact that TSR can determine if a new additive or a combination of different types of additives would provide equivalent resistance to moisture damage to the commonly used additive.

TTI researchers (Arámbula-Mercado et al., 2016) used IDT tests to evaluate the moisture susceptibility of six different OGFC asphalt mixtures that were prepared with different combinations of granite or limestone aggregates (with or without screening) and SBS- or GTR-modified PG 76-22 asphalt binders. The researchers conditioned one set of their samples using a conventional method and the second set using a moisture induced stress tester with a few modifications. Test results revealed that all but one mixture (limestone +

PMA asphalt binders) satisfied the FDOT-set TSR ≥ 70 percent criterion. The results also revealed that all but one asphalt mixture (limestone with screening + PMA) ranked better than or equivalent to ARB asphalt mixtures, which is consistent with field observations of mixtures.

TTI researchers (Arámbula-Mercado et al., 2019) later used FDOT's own version of AASHTO T283 (FDOT, 2018b) to compare moisture susceptibility of four different types of OGFC asphalt mixtures. The asphalt mixture samples were prepared by mixing granite or limestone aggregates with SBS modified or HP modified PG 76-22 PMA or HP asphalt binders, oven aging the loose asphalt mixtures for different durations (standard 2 hours and 5 days), and then compacting these asphalt mixtures. The study revealed that all but one mixture (2-hour short-term aged mixture of limestone with PG 76-22 PMA asphalt binders) satisfied the FDOT-set TSR ≥ 0.70 criterion (see Figure 2-23). However, the study also revealed that TSR could not rank PMA and HP asphalt mixtures and asphalt binders consistently the same in each aging condition.



	A0	A5
Granite PMA	89.77%	78.44%
Granite HP	83%	90%
Limestone PMA	65%	83%
Limestone HP	76%	72%

Figure 2-23: Sensitivity of TSR to the Change in Polymer Content (Arámbula-Mercado et al., 2019)

[Note: A0 = 2-Hour Aging; A5 = 5-Day Aging]

Besides the IDT test, several researchers have also used HWT test parameters such as stripping inflection point (SIP), stripping slope (SS), stripping life (SF), etc. to compare moisture susceptibility of asphalt mixtures.

For example, Izzo and Tahmoressi (1999) were among the first researchers to prove the ability of the HWT test to distinguish moisture-susceptible asphalt mixtures from moisture-insusceptible asphalt mixtures. The researchers illustrated this capacity by successfully ranking asphalt mixtures containing anti-stripping agents over asphalt mixtures containing no anti-stripping agents in the same order as the TSR test ranking.

Similarly, Epps Martin et al. (2014) used this test to assess the moisture susceptibility of warm asphalt mixtures (WMAs). The researchers also developed minimum thresholds for SIP and SS for use in the QA process. Based on the success of this test in discriminating moisture-susceptible from moisture-insusceptible WMA, they recommended using the HWT test in lieu of AASHTO T-283.

Likewise, Dave et al. (2018) used the TTI approach to compared HWT test results of asphalt mixtures produced with anti-stripping agents from asphalt mixtures produced without anti-stripping agents and found a clear distinction between these two groups (see Figure 2-24). Based on the comparative study of results obtained from IDT, dynamic modulus, DCT, SCB, HWT, and ultrasonic pulse velocity tests, they recommended adopting the HWT test as a routine moisture susceptibility test and ultrasonic pulse velocity as a screening test.

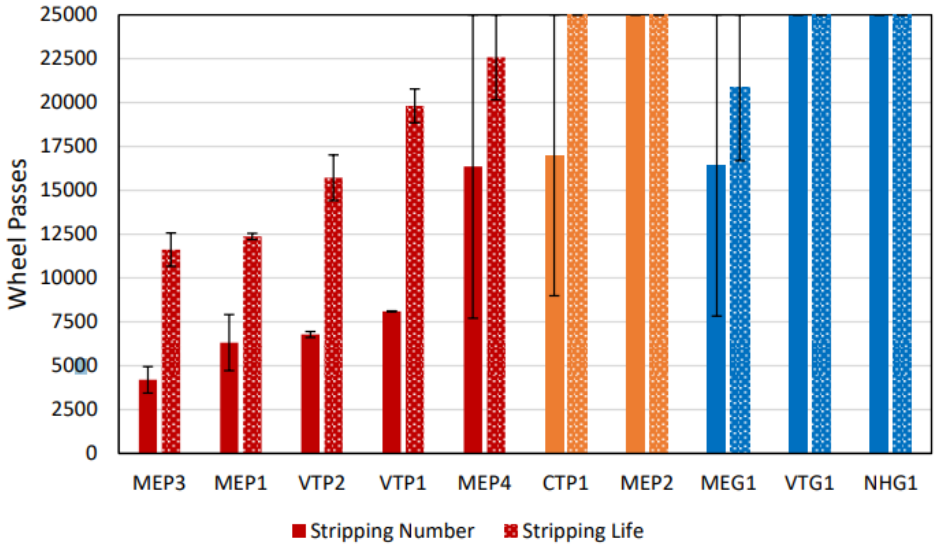


Figure 2-24: Sensitivity of TSR to the Use of Anti-Stripping Agents (Dave et al., 2018)

2.5.4 Asphalt Mixture Durability Tests

Cantabro mass loss tests are used to evaluate the durability of asphalt mixtures. The SGC specimens are placed in an enclosed drum without steel spheres and then the drum is rotated for 300 revolutions. The mass lost due to abrasion, or simply mass or abrasion loss, is determined from the difference in specimen mass before and after the test (AASHTO, 2018d). Ideally, asphalt mixtures with higher mass loss are less durable than the others. The test has been used for evaluating the durability (or raveling resistance) of friction course mixtures (Arámbula-Mercado et al., 2019, 2016; Cooley et al., 2009; Tsai et al., 2012) as well as the cracking performance of the DGA mixtures (Baumgardner et al., 2012; Doyle and Howard, 2016).

Tsai et al. (2012) designed OGFC asphalt mixtures using two different types of aggregates with three different types of asphalt binders (one PG 76-22 PMA, one GTR-modified PG 64-28 and one unmodified PG 64-10) and conducted draindown tests, Cantabro tests and HWT tests to determine their compliance with the maximum 0.3 percent draindown, maximum 30 percent mass loss and maximum 12.5 mm rut depth requirements. The researchers found that each PMA asphalt mixture satisfied all but the mass loss requirement and so did most of the GTR asphalt mixtures. However, unmodified asphalt mixtures satisfied only the draindown requirement. Though the study did not rank asphalt mixtures containing the same percentage of asphalt binders, the study highlighted the ability of HWT and Cantabro tests to distinguish unmodified asphalt mixtures from PMA or ARB asphalt mixtures — a fact very relevant to this project.

Arámbula-Mercado et al. (2016) evaluated which tests would accurately rank the performance of four limestone and two granite asphalt mixtures prepared with different gradations and asphalt binder types (PG 76-22 PMA and PG 76-22 ARB) in coherence with field performance. The limestone asphalt mixtures used in this study contained 0.5 percent liquid anti-stripping agents by total weight of asphalt binder, while the granite mixture had 1.0 percent hydrated lime by total weight of aggregates; each mixture also contained 0.4 percent mineral fiber by total weight of asphalt mixture. After running performance tests, such as the Cantabro, ITS, and HWT tests, the researchers found that asphalt mixtures ranked very differently for raveling, stripping and rutting. However, they found a good correlation between mass loss and field observation, and therefore concluded it to be the best indicator of

OGFC field performance. Additionally, the researchers conducted G-R parameter tests and found that this parameter ranked PMA and ARB binders consistent with asphalt mixture and field observation.

Arámbula-Mercado et al. (2019) compared the performance and durability of FC-5 asphalt mixtures fabricated with two different aggregate types (granite and limestone: GA and L) and two different modified asphalt binders — a standard PG 76-22 (PMA) asphalt binder and a HP-modified asphalt binder. They performed fracture tests and Cantabro tests on these samples as well as linear viscoelastic, surface free energy, fatigue cracking, and creep and recovery tests on the asphalt binder samples at different aging conditions (2-hr-oven-aged: $A0$ versus 5-day-oven-aged: $A5$. Asphalt binder test results showed that HP asphalt binders outperformed PMA asphalt binders in fatigue cracking and creep recovery (see Figure 2-25). Cantabro and fracture tests both ranked HP asphalt mixtures better than PMA asphalt mixtures, which they also corroborated with finite element modeling of raveling.

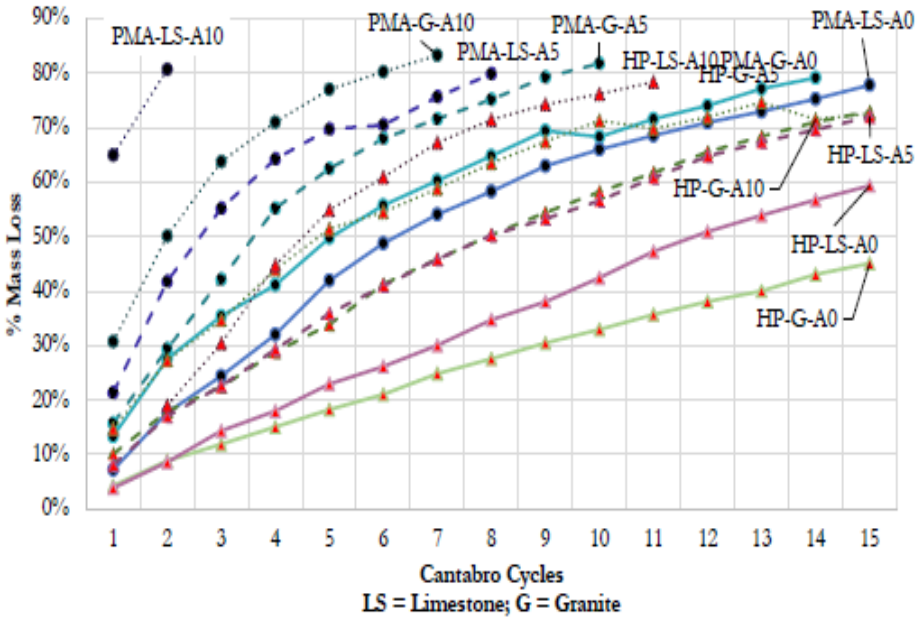


Figure 2-25: Sensitivity of Mass Loss to the Change in Polymer Content (Arámbula-Mercado et al., 2019)

[Note: A0 = 2-Hour Aging; A5 = 5-Day Aging; LS =Limestone; G = Granite]

Doyle and Howard (2011) evaluated the possibility of using Cantabro tests to evaluate the durability of DGA mixtures containing high RAP contents. For this study, the researchers

prepared DGA mixtures with different aggregate types, aggregate gradations, asphalt binder contents and high RAP contents. Two of these asphalt mixtures did not contain any RAP. Seven of them contained PG 67-22 asphalt binder, while one contained PG 76-22 asphalt binder. Based on test results, they determined mass loss to be a promising measure of raveling of DGA mixtures containing high RAP contents.

Baumgardner et al. (2012) later evaluated the effect of adding GTR additives to asphalt binders on the durability of DGA using Cantabro tests. For this study, the researchers used SBS- and GTR-modified asphalt mixtures. In preparing GTR-modified asphalt mixtures, they used two different methods: one involved adding additives to the asphalt binders first (wet process), and the other involved directly adding additives to the mixture (dry process). For preparing SBS-modified asphalt mixtures, they used the wet-processed SBS-modified asphalt binders. The Cantabro test result showed that SBS-modified PG 76-22 and wet-processed GTR-modified asphalt mixtures lost less mass than did the dry-processed GTR-modified asphalt mixtures, thereby highlighting the effectiveness of wet-processing asphalt binders.

Doyle and Howard (2016) again evaluated whether the Cantabro test can be used to evaluate the durability of DGA asphalt mixtures. For this study, they evaluated its sensitivity to mix design parameters, asphalt mixture oven aging, and RAP content in WMA along with its specimen-to-specimen variability. Statistical analysis of the test results revealed that mass loss was sensitive to asphalt binder grade (increased with lower PG), aggregate gradation, aggregate type, and aging (increased with longer aging and RAP content).

Cox et al. (2017) evaluated the effectiveness of the Cantabro test in assessing the durability of nine DGA mixtures (i.e., A1 to A9) by analyzing 1,200 Cantabro test data. The study found that the test was sensitive to various factors, such as the following:

- RAP content: Mass loss increased with higher RAP content.
- Aging: Mass loss increased with higher aging.
- Aggregate type: Mass loss changed with change in aggregate type.
- Asphalt binder grade: Mass loss increased with an increase in high temperature in unmodified asphalt binders until polymer was used.
- Additives: The test successfully discriminated asphalt mixtures containing GTR-modified asphalt binders from SBS-modified asphalt binders and hybrid (GTR+SBS-modified) asphalt binders (see Figure 2-26).

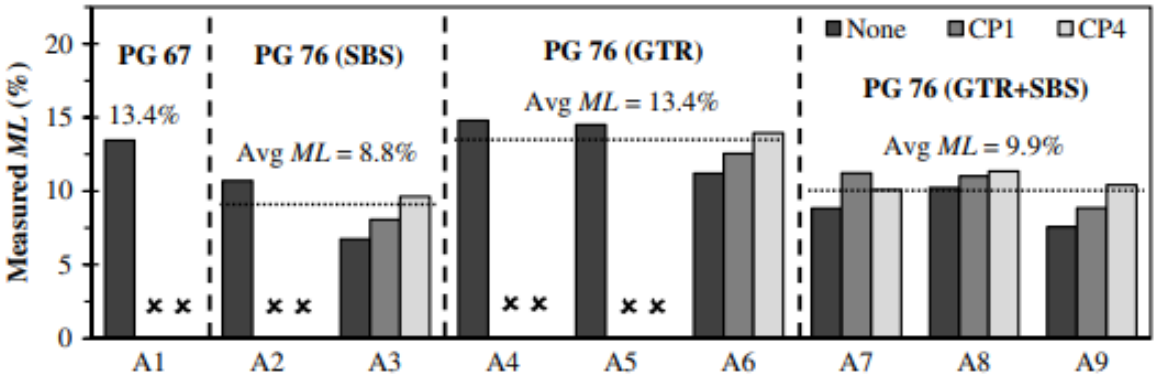


Figure 2-26: Sensitivity of Mass Loss to Asphalt Binder Additives (Cox et al., 2017)

Table 2-3 presents different types of tests used to determine the rutting and cracking performance of asphalt mixtures. The table also presents the standards/references, input parameters (i.e., conditioning and loading), measured parameters and literature-referred limitations.

Table 2-3. Common Asphalt Mixture Performance Tests (Al-Qadi et al., 2015; West et al., 2018a, 2018b; Zhou et al., 2019, 2017b)

Test	Standard	Configuration	Conditioning	Loading	Parameters	Limitations
Cracking Performance						
Flexural Bending Beam Fatigue Cracking	AASHTO T321 (2017b)		Int. Temp.: 20 ± 0.5°C	10 Hz	Bottom-Up Cracking: Measured failure cycle: N_f	Preparation: Cutting Instrument: Separate, Expensive Testing time: Long (in hrs.) Variability: High; Self-weight errors
Direct Tension Fatigue Cracking	AASHTO TP107 (2018b)		Int. Temp.: Max. of (21°C, $\frac{PG_{HT}+PG_{LT}}{2} - 3$)	10 Hz	Bottom-Up Cracking: $C\ x\ S$ Curve Predicted failure cycle: N_f	Preparation: Cutting, Gluing, Curing Instrument: Expensive, Separate Testing time: Long (in hrs.) Variability: High; Eccentricity errors
Texas Overlay	TXDOT Tex-248-F (2019)		Int. Temp.	0.1 Hz	Reflective Cracking: Load cycles to 93% initial load: N_f	Preparation: Cutting, Gluing, Curing Instrument: Expensive, Separate Testing time: Variable (in min to hrs.) Variability: High; 30-50% COV
Modified Texas Overlay	N/A		Int. Temp.	1 Hz	Reflective Cracking: Load cycles to peak value of normalized load x cycle: N_f	Preparation: Cutting, Gluing, Curing Instrument: Expensive, Separate Testing time: Variable (in min to hrs.) Variability: High; 20-30% COV
DCT	ASTM D7313 (2013)		Low Temp. $PG_{LT}+10$	1.0 mm/ min	Thermal/Reflective Cracking: Fracture energy: G_f	Preparation: Cutting, Notching, Boring Instrument: Expensive, Separate Testing time: Short (in min) Variability: High; Breakage near holes

Table 2-3. Common Asphalt Mixture Performance Tests (Al-Qadi et al., 2015; West et al., 2018a, 2018b; Zhou et al., 2019, 2017b), Continued

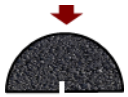
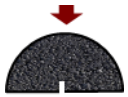
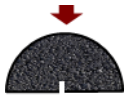
Test	Standard	Configuration	Conditioning	Loading	Parameters	Limitations
Cracking Performance						
Low Temperature SCB	AASHTO TP105 (2015c)		Low Temp.: PG_{LT}-2 & PG_{LT}+10	0.0005 mm/ min	Thermal/Reflective Cracking: Fracture energy: G_f Fracture toughness: K_{IC}	Preparation: Cutting, Notching Instrument: Inexpensive, Adaptable Testing time: Short (in min) Variability: High
SCB	ASTM D8044 (2016a)		Int. Temp. $\frac{PG_{HT}+PG_{LT}}{2}+4$	50 mm/ min	Bottom-Up/ Top-Down/ Reflective Cracking: Critical strain energy release rate J_c	Preparation: Cutting, Notching Instrument: Inexpensive, Adaptable Testing time: Short (in min) Variability: High
SCB-FI	AASHTO TP124 (2018c)		Int. Temp. 25 ±0.5°C	50 mm/ min	Bottom-Up/ Top-Down/ Reflective Cracking: Flexibility Index: FI Cracking resistance index: CRI	Preparation: Cutting, Notching Instrument: Inexpensive, Adaptable Testing time: Short (in min) Variability: High; 10-20% COV; Depends on post-peak data quality
Creep Compliance and Tensile Strength	AASHTO T322 (2007)		Low. Temp.: -30, -20, -10°C for PG ≤ -34 -20, -10, 0°C for PG = -22, -28 -10, 0, 10°C for PG ≥ -16	12.5 mm/ min	Thermal Cracking: Tensile creep compliance Indirect tensile strength	Preparation: Simple; Cutting Instrument: Inexpensive, Adaptable Testing time: Short (in min) Variability: Depends
Energy Ratio	N/A		Int. Temp.	50 mm/ min	Top-Down Cracking: Minimum dissipated creep strain energy: $DSCE\_min$ Energy ratio: ER	Preparation: Simple; Cutting Instrument: Inexpensive, Adaptable Testing time: Short (in min) Variability: Depends

Table 2-3. Common Asphalt Mixture Performance Tests (Al-Qadi et al., 2015; West et al., 2018a, 2018b; Zhou et al., 2019, 2017b), Continued

Test	Standard	Configuration	Conditioning	Loading	Parameters	Limitations
Cracking Performance						
IDEAL-CT	ASTM D8225 (2019)		Int. Temp.: 25°C or $\frac{PG_{HT} + PG_{LT}}{2} + 4$	50 mm/ min	Bottom-Up/ Top-Down/ Reflective Cracking: Cracking test index: CT_{index}	Preparation: Simple; No cutting Instrument: Inexpensive, Adaptable Testing time: Short (in min) Variability: Low, COV<20%
Rutting Performance						
Superpave Shear	AASHTO T320 (2016c)		High Temp.	Varies	Freq. Sweep Test: G^*, δ Simple Shear Test: γ_{max}, $\%Recovery$ Reheated Shear Test: γ_p	Preparation: Complex Instrument: Expensive Testing time: Moderate (in min to hrs.) Variability: Depends
Flow Number	AASHTO T378 (2017c)		High Temp.	Load: 0.1 sec, Rest: 0.9 sec.	Flow number: FN	Not a Shear Test Sample Prep.: Complex Instrument: Expensive Testing time: Depends on tertiary flow Variability: Depends on tertiary flow
RLPD	AASHTO TP116-15 (2015b)		High Temp	Repeated load cycles at multiple stress levels	Master curve of Minimum strain rate and Product of temperature & pressure (MSR **x** TP)	Preparation: Complex Instrument: Expensive Testing time: Depends Variability: Moderate
APA	AASHTO T340 (2010)		High Temp.	60 cycles/ min (max. 8,000 cycles or max. 4.5 mm); Dry	Rut depth No. of wheel passes	Preparation: No cutting Instrument: Expensive, Separate Testing time: Depends Variability: Moderate

Table 2-3. Common Asphalt Mixture Performance Tests (Al-Qadi et al., 2015; West et al., 2018a, 2018b; Zhou et al., 2019, 2017b), Continued

Test	Standard	Configuration	Conditioning	Loading	Parameters	Limitations
Cracking Performance						
HWT	AASHTO T324 (2017d)	 Cutting	High Temp.	52 passes/ min (max. 20,000 passes or max. 12.5 mm); Submerged	Rut depth No. of wheel passes	Preparation: Cutting; Submergence Instrument: Expensive, Separate Testing time: Depends Variability: Moderate
IDEAL-RT	N/A		High Temp.	50 mmm/ sec	Rutting test Index: τ	Preparation: No cutting Instrument: Inexpensive Testing time: Short (in min) Variability: Low, COV <10%
Moisture Susceptibility						
TSR	FM 1-T283 (2018b)		Int. Temp.	50 mm/ min	Tensile strength ratio: TSR Dry indirect strength: ITS_{dry} Wet indirect strength: ITS_{wet}	Preparation: Cutting; Submergence Instrument: Separate, Expensive Testing time: Long (hrs.) Variability: Depends
HWT	AASHTO T324 (2017d)		High Temp.	52 passes/ min (max. 20000 passes or max. 12.5 mm); Submerged	Stripping inflection point Stripping slope Stripping number Stripping life	Preparation: Cutting; Submergence Instrument: Separate, Expensive Testing time: Long (hrs.) Variability: Moderate
Durability						
Cantabro Abrasion Mass Loss	AASHTO TP108 (2018d)		Int. Temp.	30-33 cycles/min (300 cycles)	Percent mass loss	Preparation: No Cutting Instrument: Separate, Inexpensive Testing time: Short (min) Variability: Moderate

2.6 Summary

A comprehensive search was conducted on the previous studies, including the ones conducted by FDOT, that evaluated the performance of asphalt binders and asphalt mixtures produced with common and alternative additives. The review showed that several different types of products have been proposed as asphalt binder/mixture additives with varying degrees of success in delivering results as claimed/expected. The review also revealed that multiple test methods have been proposed to evaluate the same type of performance of asphalt binders and asphalt mixtures, and that most of these tests lag in one or more essential features of any laboratory test such as simplicity, and sensitivity to mix design variables including the effect of additives, practicality, repeatability, reproducibility, cost effectiveness and field correlation (see Table 2-3 for the most common asphalt mixture performance tests, associated standards, inputs and outputs, and their benefits and limitations). Based on this review, the team made several observations relevant to this project:

- Asphalt Binder Additives:
    - Asphalt binders are mostly modified with non-reactive polymers, GTR with or without the aid of PPA to modify high temperature PG and with REOB/VTAE, extenders, aromatic extracts, fatty acids, etc. to meet low temperature performance.
    - Reactive polymers (e.g., terpolymers and copolymers) are not used in pavements except in laboratory-based research projects. Though they show promise in terms of enhancing PG, these additives involve more than one step and more than one type of material for their production. As such, their reactivity depends on several factors:
        - Molecular weight and structure of the base polymer.
        - Reactivity of functionalization chemicals.
        - Ratio of the polymer and the functionalization chemical.
        - Blending temperature.
        - Shear speed.
        - Curing condition and time before their use in asphalt binders.

- Excessive use of any additive is self-defeating; it is especially true in the case of reactive polymers because these polymers can form a separate gel-like phase if they are not properly blended and cured.

- Asphalt Mixture Additives:
  - Asphalt mixtures are mostly modified with cellulose and mineral fibers. Polymeric fibers have been allowed or experimented with by only a few state DOTs. The main purpose of fibers is to stabilize asphalt mixtures during construction and reinforce their long-term performance. Most of the asphalt mixtures that use fibers are either gap- or open-graded (e.g., SMA, OGFC). Polymeric fibers can stabilize and reinforce asphalt mixtures at the same time.
- Asphalt Binder Performance Tests:
  - Superpave asphalt binder rutting and cracking resistance parameters cannot clearly determine the effect of additives. Some of the parameters obtained from the new rutting tests (e.g., Jnr and $\%Rec.$ obtained from MSCR test) and cracking tests (e.g., GR from Glover-Rowe and FREI obtained from PLAS) along with the parameters obtained from traditional tests ΔTc values from BBR tests show potential in capturing the effect of additives on rutting and cracking resistance. Tests such as the SENB and the DENT tests have too much variability, and tests such as the EBBR tests are too harsh.
- Asphalt Mixture Performance Tests:
  - Wheel tracking tests such as APA and HWT tests can show whether materials are rutting resistant, but these tests are long, and costly while relatively less sensitive to asphalt binders/mixture modification.
  - Repeatedly loaded cracking tests either need a longer testing time (e.g., bending beam fatigue cracking test, direct tensile fatigue cracking test), knowledge of complex formulation (e.g., direct tension fatigue cracking) or separate expensive instruments (e.g., beam fatigue overlay tester) or they have high variability (e.g., overlay tests).

- Monotonically loaded cracking tests require shorter testing time and less expensive instruments but longer preparation time and have high variability (e.g., SCB, DCT tests, etc.).
- Among all rutting and cracking tests, the APA test and the two newly developed IDEAL-RT and IDEAL-CT tests are the only tests that do not need any cutting. They are reportedly sensitive to mix design variables, including the use of additives.

3 PROTOCOL DEVELOPMENT

Two separate protocols, each with a different objective, were developed in this project:

- The first protocol was developed to evaluate if the alternatively modified asphalt (AMA) binders/mixtures (i.e., asphalt binders/mixtures modified with alternative asphalt binder (AAB) additives, such as reactive terpolymers and bio-rejuvenators) would perform equivalent to or better than FDOT's standard PMA binders/mixtures [i.e., SB- or SBS-modified PG 76-22 (PMA) binders/mixtures], and
- The second protocol was developed to evaluate if the fiber-modified asphalt (FMA) (i.e., asphalt mixtures modified with alternative asphalt mixture (AAM) additives, such as fibers) would perform equivalent to FDOT's standard PMA mixtures (i.e., asphalt mixtures produced with PG 76-22 (PMA) binders].

3.1 Materials

3.1.1 Asphalt Binders

Three different types of Superpave PG binders were used to develop these protocols. These binders were obtained from the sources listed in FDOT's APL:

- PG 67-22 binder obtained from one of the sources listed in its own APL (see *https://fdotwp1.dot.state.fl.us/ApprovedProductList/ProductTypes/Index/173*)
- HP binder obtained from one of the sources listed in its own APL (see *https://fdotwp1.dot.state.fl.us/ApprovedProductList/ProductTypes/Index/90*)
- PG 76-22 (PMA) binder obtained from one of the sources listed in its own APL (see https://fdotwp1.dot.state.fl.us/ApprovedProductList/ProductTypes/Index/87)

Among them, PG 67-22 binder is an unmodified asphalt binder while PG 76-22 (PMA) and HP binders are commercially available, polymer modified asphalt binders containing two different percentages of SB/SBS copolymers. The suppliers, the types (i.e., linear or branched), and the proportions of the base asphalt binders and the SB/SBS copolymers used to produce PG 76-22 (PMA) and HP binders were not available because of their proprietary nature.

The asphalt binders were first subjected to Superpave PG verification tests. After verifying their grades, PG 67-22 and HP binders were then used to produce two different PG 76-22 (AMA) binders and their corresponding mixtures. PG 76-22 (PMA) binder was also

used to produce one "control" asphalt mixture (without any fibers) and two different FMA mixtures (each with fibers obtained from a different source), thereby resulting in a total of five asphalt mixtures.

3.1.2 Aggregates

The job-mix formula (JMF) of fine-graded Type SP-12.5 mixture and the aggregates required to produce asphalt mixtures in this study were obtained from FDOT. Section 334 of the FDOT's Specification (FDOT, 2020) identifies Type SP-12.5 mixture as a Superpave asphalt mixture that has a NMAS of 12.5 mm and an aggregate gradation as defined in Table 3 of AASHTO M 323-12 (AASHTO, 2015d). Based on the JMF, the combined batch had a bulk specific gravity (Gsb) value of 2.741 and contained the following proportions of aggregate sub-types:

- 30 percent S1A granite stones (C47; #78 Stone) with a Gsb value of 2.775
- 16 percent S1B granite stones (C53: #89 Stone) with a Gsb value of 2.764
- 47 percent granite screenings (F22; W-10 Screenings) with a Gsb value of 2.730
- 7 percent local sand (334-LS: Archer Sand) with a Gsb value of 2.630

The granite subtypes—C46, C53 and F22—were obtained from the Junction City Mining, Georgia while the local sand—334-LS—was obtained from V.E. Whitehurst & Sons, Florida. FDOT's Approved Aggregate Production Facility List identifies this source of granite aggregates as GA553 (see *https://mac.fdot.gov/smoreports*). Except washed sieve analysis, no other tests were conducted on these aggregates.

3.1.3 AAB Additives

Two types of the AAB additives were obtained for use in this part of the study. Neither additive is listed in FDOT's APLs. These additives differed from each other in their compositions, sources, and functions.

3.1.3.1 Elvaloy RET 4170

This is a RET-based additive obtained from Dow Chemical for use in this project following FDOT's recommendation. According to its supplier, this additive chemically reacts with the asphalt binder components and creates linkages between them with the help of a crosslinking agent and changes their overall properties (see *https://www.dow.com/en-us/pdp.elvaloy-4170-copolymer.1891815z.html*). In this study, this additive was used with the

Innovalt N200 PPA—a crosslinking agent—to modify the unmodified PG 67-22 binder into a PG 76-22 equivalent binder.

3.1.3.2 Sylvaroad RP1000

This is a liquid bio-rejuvenator (BIO) directly obtained from Kraton Chemical for use in this project following FDOT's recommendation. According to its supplier, this additive helps mobilize the chemically aged (i.e., oxidized) asphalt binder present in RAP or RAS (see *https://kraton.com/products/sylvaroad.php*). In this study, this additive was used to modify HP binder to a PG 76-22 equivalent binder. Unlike with the RET, no crosslinking agent was used in conjunction with the BIO.

3.1.4 AAM Additives

Two types of the AAM additives were obtained for use in this part of the project. These additives differed in their compositions, sources, blending steps, recommended dosages, and application methods.

3.1.4.1 ACE XP

These were the mixture of 50 percent aromatic polyamide (aramid) polymeric fibers and 50 percent Sasobit Wax (see *https://surface-tech.com/asphalt-ace-xp)* directly obtained from Surface Tech following FDOT's recommendation. These ACE XP fibers obtained for use in this project were 38 mm (1.5 inches) in length. Its manufacturer-recommended dosage is 4.2 ounces of this fiber mix per 1.0 ton of asphalt mix. In each dosage, 50 percent is the aramid fiber and 50 percent is the wax by weight of total fiber mix, i.e., 4.2 ounces (119.068 g) of ACE XP fibers refers to 2.1 ounces (59.534 g) of the aramid fibers mixed with 2.1 ounces (59.534 g) of the wax in each 1.0 ton (907.185 kg) of the asphalt mixture. In this study, these fibers were added into the asphalt mixture continuously while mixing the aggregates and the asphalt binder together as recommended by the supplier.

3.1.4.2 FORTA-FI

These were the mixture of the aromatic polyamide (aramid) and the polyolefin fibers (see *http://www.forta-fi.com/products/forta-fi*) directly obtained from FORTA following FDOT's recommendation. The FORTA-FI fibers obtained for use in this product were shorter than the ACE XP fibers. The manufacturer-recommended dosage of this product is 1.0 pound (453.592 g) of this fiber mix per each 1.0 ton (907.185 kg) of asphalt mixture. In

this study, these fibers were added into the asphalt mixtures in several steps starting from right before mixing the aggregates and the asphalt binder as recommended by the supplier.

3.2 Asphalt Binder Performance Evaluation

Three Superpave PG asphalt binders obtained from their FDOT-approved sources and two AAB additives obtained from the FDOT-recommended sources were used to produce two PG 76-22 (AMA) binders and develop the protocol for evaluating AAB additives in line with the first objective of this study:

- AAB additives:
  - RET
  - BIO
- Superpave PG asphalt binders:
  - PG 67-22 binder
  - HP binder
  - PG 76-22 (PMA) binder: "control" binder
- AMA binders
  - RET-modified PG 76-22 asphalt binder
  - BIO-modified PG 76-22 asphalt binder

3.2.1 Asphalt Binder PG Verification

The three Superpave PG asphalt binders—PG 67-22, PG 76-22 (PMA), and HP binders—were first subjected to PG verification tests. To this end, each binder was subjected to four different thermorheological tests at three different aging levels according to Section 916 of the FDOT's current Standard Specifications for Road and Bridge Construction (FDOT, 2020). The current specification (FDOT, 2020) grades asphalt binders modified with higher than normal dosages of SB or SBS copolymers as HP binder. The HP binder was also graded using the previous version of FDOT's specification (FDOT, 2016a), which includes PG 82-22 (PMA) binder in its list of Superpave PG binders.

Three different aging conditions were included in PG verification tests: (1) unaged, original binder (OB) samples, (2) RTFO-aging of 35.0 ± 0.5 grams of unaged samples in an RTFO at a temperature of 163.0 ±1.0 °F (325.4 ± 1.8°F) and a normal atmospheric pressure for 85.0 minutes following AASHTO T 240-13 (AASHTO, 2017e), and (3) PAV-aging of

50.0 ± 0.5 grams of RTFO-aged samples in a PAV at a temperature of 100.0 ± 1.0°C (212.0 ± 1.8°F) and a compressed air pressure of 2.1 ± 0.1 MPa (304.6 ± 14.5 psi) for 20.0 hours following AASHTO R 28-12 (AASHTO, 2012). Four different thermorheological tests were conducted on these samples (one on the OB samples, one on the RTFO-aged samples and two on the PAV-aged samples) as described in the following sections.

3.2.1.1 DSR Tests of Unaged Binder Samples

The OB samples were subjected to the DSR tests at a loading frequency of 10.0 rad/sec at grade temperatures, using two parallel plate specimens that measured 25.0 mm in diameter and 1.0 mm in gap thickness, by following AASHTO T 315-12 (AASHTO, 2016a). From these tests, the corresponding values of complex shear modulus (G^*), phase angle (δ), and $G^*/\mathrm{Sin}\delta$ at each test temperature were obtained.

The $G^*/\mathrm{Sin}\delta$ and the δ values measured at the FDOT-specified high temperature PG [e.g., 67.0°C for PG 67-22 and 76.0°C for PG 76-22 (PMA) binders] were used to verify if the binders met the corresponding PGH designations as specified in Section 916 of the Specification (FDOT, 2020). Likewise, the $G^*/\mathrm{Sin}\delta$ and the δ values of the HP binder measured at 82.0°C was used to check if the HP binder could be also graded as PG 82-22 (PMA) according to the previous version of FDOT's Specification (FDOT, 2016a).

Additionally, the $G^*/\mathrm{Sin}\delta$ and the δ values measured at a minimum of two different grade temperatures [e.g., 64°C, 67°C, 70°C, and 76°C for PG 67-22 binder; 76.0°C and 82.0°C for PG 76-22 (PMA) binder; and 82°C, 88°C, and 94°C for HP binder] were used to calculate their true grades at $G^*/\mathrm{Sin}\delta$ value of 1.0 kPa based on AASHTO M 320 (AASHTO, 2017a).

3.2.1.2 MSCR Tests of RTFO-Aged Binder Samples

The RTFO-aged asphalt binder samples were subjected to repeated stress creep and recovery cycles—first at a shear stress of 0.1 kPa and then at a shear stress of 3.2 kPa—using parallel plate specimens that measured 25.0 mm in diameter and 1.0 mm in gap thickness following AASHTO T 350-14 (AASHTO, 2014a). In this study, the first two binders [i.e., unmodified PG 67-22 and PG 76-22 (PMA) binders] were tested only at 67.0°C. The third binder [i.e., HP binder] was tested at 76.0°C following FDOT's current Specification (FDOT, 2020) for grading purposes and also at 67.0°C following the previous version of FDOT's

Specification (FDOT, 2016a) for comparison purposes. From these tests, the measured values of several parameters were obtained: the Jnr values at shear stresses of 0.1 kPa and 3.2 kPa ($Jnr_{0.1}$ and $Jnr_{3.2}$, respectively), the percent differences between $Jnr_{0.1}$ and $Jnr_{3.2}$ (Jnr_{diff}), the $\%Rec$ values at shear stresses of 0.1 kPa and 3.2 kPa ($Rec_{0.1}$ and $Rec_{3.2}$, respectively) and the minimum required percent recoveries for modified binders at a shear stress of 3.2 kPa ($Rec_{3.2} \geq 29.37 \times Jnr_{3.2}^{-0.2633}$).

The $Jnr_{3.2}$, the $\%Rec_{3.2}$, and the Jnr_{diff} values measured at selected test temperatures were used to verify if the asphalt binders met their MSCR-based grade (i.e., "*S*" for PG 67-22 and "*V*" for others) according to Section 916 of the Specification (FDOT, 2016, 2020), and if not, to determine their true MSCR-based grades according to AASHTO M 332-14 (AASHTO, 2014b).

3.2.1.3 BBR Tests of PAV-Aged Binder Samples

The 20-hour PAV-aged samples were subjected to the BBR tests at grade temperatures + 10.0°C, using a minimum of two beam specimens that measured 127.0 mm in length, 12.7 mm in width and 6.25 mm in height, following AASHTO T 313-12 (AASHTO, 2016b). From these tests, the corresponding values of creep stiffness (S) and creep slope (m) measured after 60 seconds of creep loading at selected test temperatures were obtained.

The S and the m values measured at the FDOT-specified low temperature PG (PGL), i.e., PGL + 10°C [e.g., -12°C for PG 76-22 (PMA) binders] were used to verify if asphalt binders met the corresponding PGL designations as specified in Section 916 of the Specification (FDOT, 2020). The S and the m values measured at PGL + 10.0°C and PGL + 4.0°C [e.g., -12.0°C and -18.0°C for PG 67-22 and PG 76-22 (PMA) binders] were used to determine their S- and m-based critical PGL values (T_{cs} and T_{cm}, respectively), and their true PGL = max (T_{cs}, T_{cm}) values following AASHTO M 320 (AASHTO, 2017a), and to calculate their $\Delta Tc = (T_{cs} - T_{cm})$ values following ASTM D7643-16 (ASTM, 2016b).

3.2.1.4 *DSR Tests of PAV-Aged Binder Samples*

The 20-hour PAV-aged samples were also subjected to the DSR tests at a loading frequency of 10.0 rad/sec at an intermediate temperature of 26.5°C, using a minimum two parallel plate specimens that measured 8.0 mm in diameter and 2.0 mm in thickness,

following AASHTO T 315-12 (AASHTO, 2016a). From these tests, the G^*, the δ and the fatigue $G^*.Sin\delta$ values at 26.5°C were obtained.

The $G^*.Sin\delta$ values at 26.5°C were then used to verify whether asphalt binders met the requirement (i.e., $G^*.Sin\delta \leq 5000$ kPa for PG 67 or higher and $G^*.Sin\delta \leq 6000$ kPa for PG 76 and higher) specified for corresponding binder grade in Section 916 of the Specification (FDOT, 2020).

Table 3-1 presents the average and the standard deviation results of asphalt binder PG verification tests. The results show that the first and the second binders respectively met the requirements of PG 67-22 and SBS-modified PG 76-22 (PMA) binders as specified in the current Specification (FDOT, 2020). The results also show that the third binder met the requirements of not only the HP binder as specified in the current specification (FDOT, 2020) but also PG 82-22 binder as specified in the previous specification (FDOT, 2016a). In terms of their true grades, these binders were graded as 68.4-23.1 "*S*", 78.9-24.1 "*E*", and 90.1-22.3 "*E*".

Table 3-1. Control and Base Binder PG Verification Results

Test	Type	Temp.	Criteria	PG 67-22	PG 76-22 (PMA)	HP
OB						
DSR: Two replicates	All Binders	Grade	$G^*/Sin\delta \geq 1.0$ kPa	1.18 ± 0.02	1.29 ± 0.00	1.89± 0.01
	Modified Binders	Grade	$\delta \leq 75$ deg.	87.3 ± 0.1	69.8 ± 0.1	62.1 ± 0.1
	PG 82-22 (PMA)	Grade	$\delta \leq 65$ deg.			
	True Grade			68.4 ± 0.1	78.9 ± 0.0	90.1 ± 0.0
RTFO-Aged Residue						
MSCR: Two replicates	Unmodified	67.0°C	$Jnr_{3.2} \leq 4.5$ kPa^{-1}	2.1 ± 0.5		
			$Grade \geq$ S	S		
			Jnr_{diff} (no requirement)	15.5 ± 1.5		
			$\%Rec_{3.2}$ (no requirement)	1.5 ± 0.2		
	Modified Binders	67.0°C	$Jnr_{3.2} \leq 1.00$ kPa^{-1}		0.3 ± 0.0	
			$Grade \geq$ V		E	
			$Jnr_{diff} \leq 75\%$ (not applicable if $Jnr_{3.2} \leq 0.50$)		61.3 ± 0.6	
			$Rec_{3.2} \geq 39.9\%$		67.8 ± 2.3	
	HP Binder	76.0°C	$Jnr_{3.2} \leq 0.10$ kPa^{-1}			0.2 ± 0.00
			Grade $\geq$ V			E
			Jnr_{diff} (not applicable if $Jnr_{3.2} \leq 0.50$)			-430.4 ± 7.0
			$Rec_{3.2} \geq 90\%$			80.8 ± 1.2
	PG 82-22 (PMA) (FDOT, 2016a)	67.0°C	$Jnr_{3.2} \leq 0.50$ kPa^{-1}			0.04 ± 0.0
			$Grade \geq$ E			E
			Jnr_{diff} (not applicable if $Jnr_{3.2} \leq 0.50$)			402.3 ± 305.9
			$Rec_{3.2} \geq 66.7\%$			90.7 ± 2.9
PAV-Aged Residue						
BBR: Two replicates	All Binders	-12.0°C	m @ 60 sec ≥ 0.300	0.311 ± 0.001	0.323 ± 0.003	0.303 ± 0.003
			S @ 60 sec ≤ 300 MPa	190.5 ± 12.0	182.0 ± 26.9	212.0 ± 2.0
			T_{cm} *in* °C (no requirement)	-23.1 ± 0.1	-24.1 ± 0.2	-22.3 ± 0.3
			T_{cs} *in* °C (no requirement)	-26.1 ± 0.4	-26.7 ± 0.6	-25.0 ± 0.3
			$\Delta Tc \geq$ -5°C	-3.0 ± 0.5	-2.6 ± 0.7	-2.6 ± 0.5
	True Grade			-23.1 ± 0.1	-24.1 ± 0.2	-22.3 ± 0.3
DSR: Two replicates	PG 67-22	26.5°C	$G^*Sin\delta \leq 5000$ kPa	4106 ± 170		
	Others		$G^*Sin\delta \leq 6000$ kPa	-	2657 ± 114	4174 ± 953

Table 3-1. Control and Base Binder PG Verification Results, Continued

Test	Type	Temp.	Criteria	PG 67-22	PG 76-22 (PMA)	HP
Grade:			Current Specification (FDOT, 2020)	PG 67-22	PG 76-22 (PMA)	HP
			Previous Specification (FDOT, 2016a)			PG 82-22 (PMA)
Notes:	G^*= Complex shear modulus; δ = Phase angle; PGH = High temperature performance grade; $Jnr_{3.2}$ = Non-recoverable compliance at a shear stress of 3.2 kPa; Jnr_{Diff} = Difference of non-recoverable compliances at the shear stresses of 3.2 kPa and 0.1 kPa; $\%Rec_{3.2}$ = Percent recovery at a shear stress of 3.2 kPa; m = Creep slope or rate of stress relaxation; S = Creep stiffness; ΔTc = Difference of critical low temperatures corresponding to S-value 300 MPa and m-value of 0.300; PGL = Low temperature performance grade					

3.2.2 Asphalt Binder Modification

As discussed above, one SBS-modified PG 76-22 (PMA) binder, one RET-modified PG 76-22 binder, and one BIO-modified PG 76-22 asphalt binder were used to develop the protocol for evaluating the AAB additives in line with the first objective of this study. Among them, PG 76-22 (PMA) binder was commercially produced by blending a base asphalt binder with a SB or SBS copolymer by the supplier itself while the RET-modified PG 76-22 binder was produced by blending unmodified PG 67-22 with an optimum dosage of RET and PPA, and the BIO-modified PG 76-22 binder was produced at TTI's laboratory by blending the HP binder with an optimum dosage of RET.

To determine the optimum dosages of the RET/PPA and the BIO additives required to produce the AMA binders that met PG 76-22, the base asphalt binders were first blended with two different trial dosages of the AAB additives, resulting in four trial blends in total (two per each additive type). Then, the correlations of high and low temperature properties of trial blends with applied dosages of the AAB additives were utilized to determine the optimum dosages of these additives required to produce the AMA binders that met the requirements of PG 76-22 (PMA) binder.

For example, to determine the optimum dosage of RET required to produce the AMA binder that met PG 76-22 requirements, the unmodified PG 67-22 base asphalt binder was first blended with 1.50 percent and 2.00 percent RET by weight of the base asphalt binder at 163.0 ± 5.0°C (325.4 ± 9.0°F) for 1.5 hours, and then with 0.30 percent and 0.40 percent

PPA by weight of the resultant blends, respectively at 163.0 ± 5.0°C (325.4 ± 9.0°F) for an additional 1.0 hour, resulting in two RET-modified trial blends:

- 99.70% (98.50% PG 67-22 + 1.50% RET) + 0.30% PPA = 100%
- 99.60% (98.00% PG 67-22 + 2.00% RET) + 0.40% PPA = 100%

Similarly, to determine the optimum dosage of the BIO additive required to produce another AMA that met PG 76-22, HP binder was blended with 2.00 percent and 4.00 percent BIO by weight of the binder for a total of 2.5 hours at 163.0 ± 5.0°C (325.4 ± 9.0°F), resulting in two BIO-modified trial blends:

- 98.00% HP + 2.00% BIO = 100%
- 96.00% HP + 4.00% BIO = 100%

The laboratory modification of the two base asphalt binders, each with two trial and one optimum dosage of the AAB additives, involved several steps. To exemplify, the steps taken to produce 500.0 grams of the AMA binders using 2.00 percent AAB additive are listed below:

1. Enough amount of base asphalt binder (i.e., PG 67-22 for RET/PPA and HP for BIO) was heated at 163.0 ± 5.0°C (325.4 ± 9.0°F).
2. 98.0 percent of 500.0 grams (i.e., 490.0 grams) of the base asphalt binder was poured into a separate can.
3. The can with 490.0 grams of the base asphalt binder was heated at 163.0 ± 5.0°C (325.4 ± 9.0°F).
4. 2.00 percent of 500.0 grams (i.e., 10.0 grams) of the selected AAB additive (either RET or BIO) was slowly added into the heated base asphalt binder (see Figure 3-1) while continuously blending them together with a shear mixer, resulting in 500.0 grams of the blend in total. The shear speed was adjusted to create a vortex in the middle of the can to ensure homogeneity.
5. The blend was continuously stirred at 163.0 ± 5.0°C (325.4 ± 9.0°F) for 1.5 hours with the shear mixer.

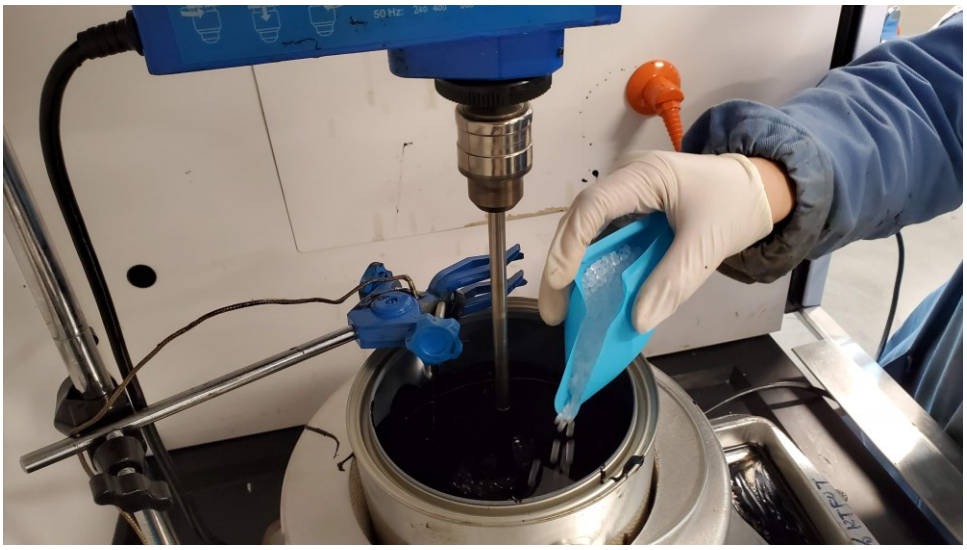
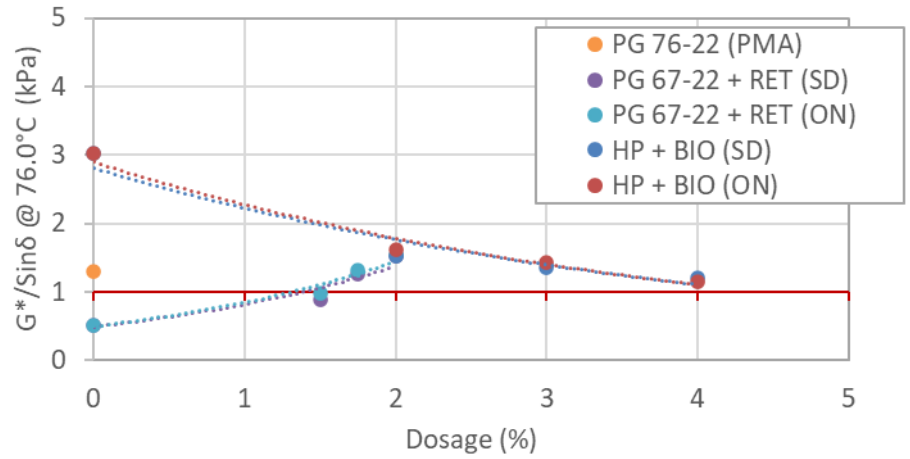


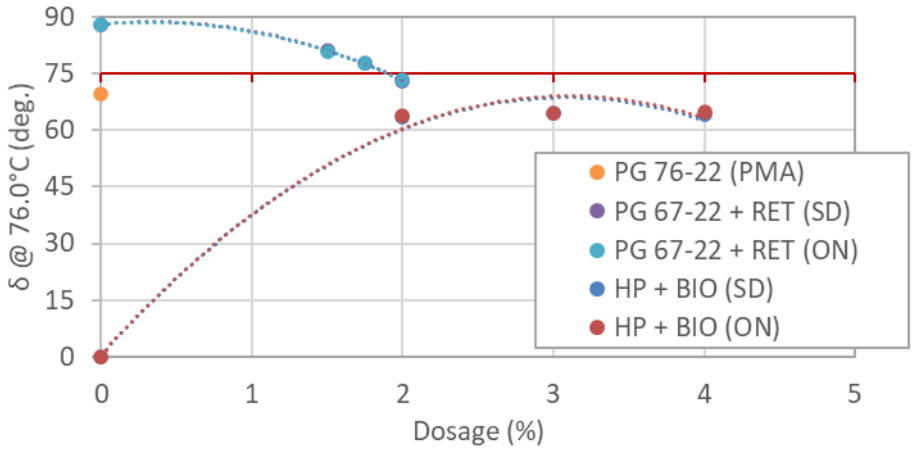
Figure 3-1. Binder Modification

6. 2.0 grams of PPA was added to 500.0 grams of the blend of PG 67-22 binder and the RET additive. This step was skipped when the HP binder was blended with the BIO additive.
7. The blending was continued for another 1.0 hour at 163.0 ± 5.0°C (325.4 ± 9.0°F), resulting in 2.5 hours of blending irrespective of the type, number and dosage of additives.
8. Parallel plate specimens were sampled from the blend in the can and subjected to DSR tests to determine PGH and the corresponding properties of the "same-day" ("*SD*") sampled blends.
9. The can of the blend was capped with a lid and cured at 163.0 ± 5.0°C (325.4 ± 9.0°F) in an oven undisturbed for almost 18.0 hours over one night to let the reactions complete.
10. Parallel plate specimens were sampled from the blend in the can and subjected to DSR tests to determine corresponding properties of the over-a-night ("*ON*") sampled blends.

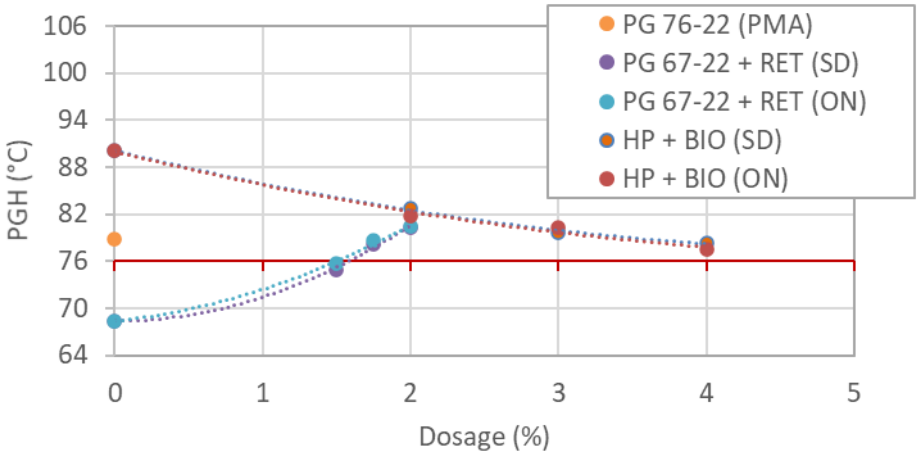
Figure 3-2 presents the results of DSR tests of the *SD* and the *ON* samples. The results show that these samples had minimal difference in their $G^*/Sin\delta$, δ and PGH values, which confirmed that overnight curing was adequate to complete the reactions between the base asphalt binder and the AAB additive.



(a)



(b)



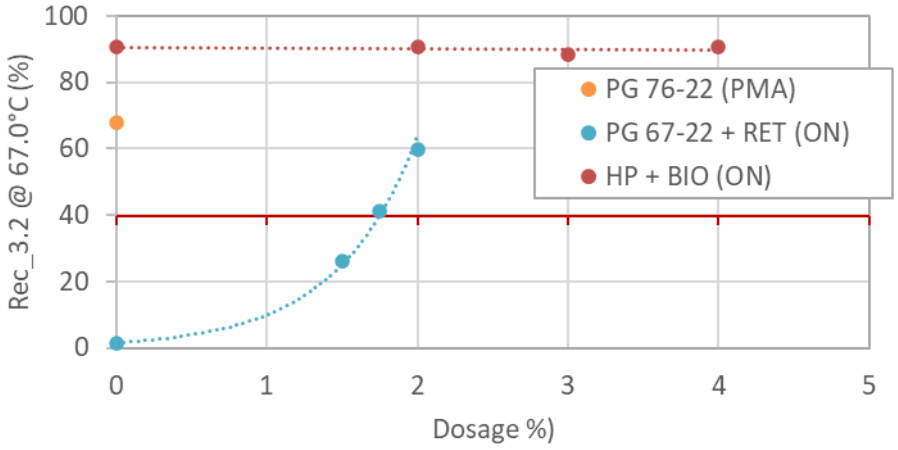
(c)

Figure 3-2. DSR Tests: (a) *G*/Sinδ* at 76.0°C versus Dosage; (b) *δ* at 76.0°C versus Dosage; (c) *PGH* versus Dosage

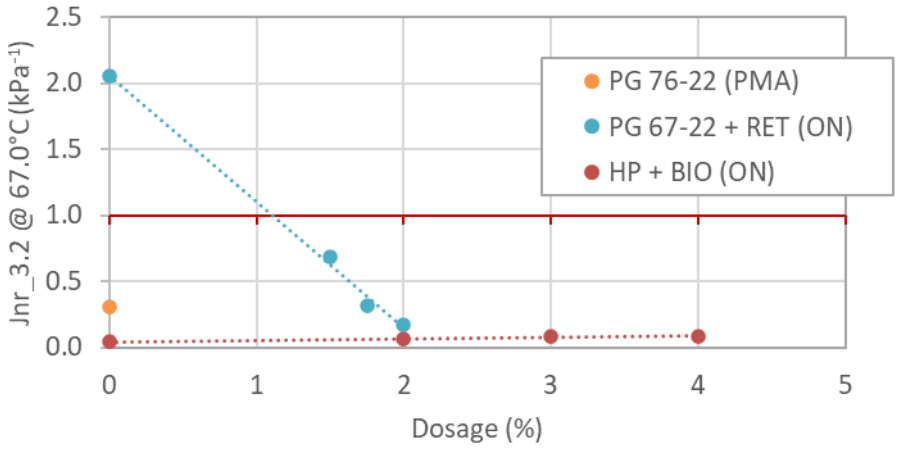
Figure 3-2, Figure 3-3, and Figure 3-4 show that the blend of PG 67-22 binder, 1.50 percent RET and 0.30 percent PPA met all but two requirements (minimum G^*/Sinδ value of 1.0 kPa at the grade temperature of 76.0°C and minimum $\%Rec_{3.2}$ value at 67.0°C) of PG 76-22 (PMA) binder and could not be graded as such. In contrast, the figures show that the blend of PG 67-22 binder, 2.00 percent RET and 0.40 percent PPA satisfied each requirement of PG 76-22 (PMA) binder and could be graded as PG 76-22.

These figures also show that the true grade changed from 68.4-23.1 (PG 67-22) to 75.8-22.3 (PG 67-22) with the use of 1.50 percent RET and 0.30 percent PPA and to 80.5-23.1 (PG 76-22) with the use of 2.00 percent RET and 0.40 percent PPA, which revealed that an increase in the RET and the PPA dosages would result in an increase in the PGH value (i.e., stiffening, or hardening effect) but would have no effect on the PGL value. Based on these correlations, PGH governed both minimum and maximum RET dosages required to modify PG 67-22 binder into PG 76-22 equivalent binder at 1.52 percent and 2.16 percent, respectively. As such, 1.75% RET and 0.35% PPA were selected to modify PG 67-22 binder to PG 76-22 equivalent binder.

Figure 3-2, Figure 3-3, and Figure 3-4 also show that the blends of HP binder with 2.00% and 4.00% BIO satisfied all requirements of PG 76-22 (PMA), meaning both could be graded as such. These figures also show that true grade changed from 90.1-22.3 (PG 88-22) to 83.3-25.5 (PG 82-22) with the use of 2.00% and to 78.3-28.8 (PG 76-28) with the use of 4.00% BIO, respectively, which revealed that an increase in BIO dosage resulted in a decrease in both PGH and PGL values (i.e., softening effect). Based on these correlations, PGH and PGL governed both maximum and minimum dosages of BIO required to convert HP to PG 76-22 binder at 2.36% and 3.51%, respectively. Therefore, 3.00% BIO was selected to modify HP binder to PG 76-22 equivalent binder.

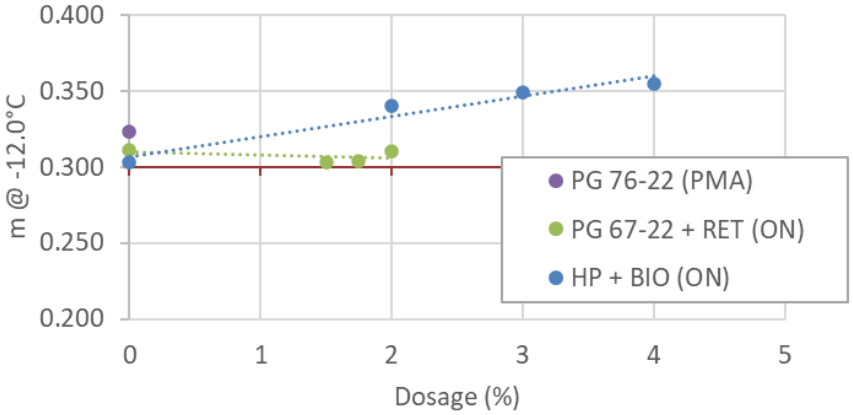


(a)



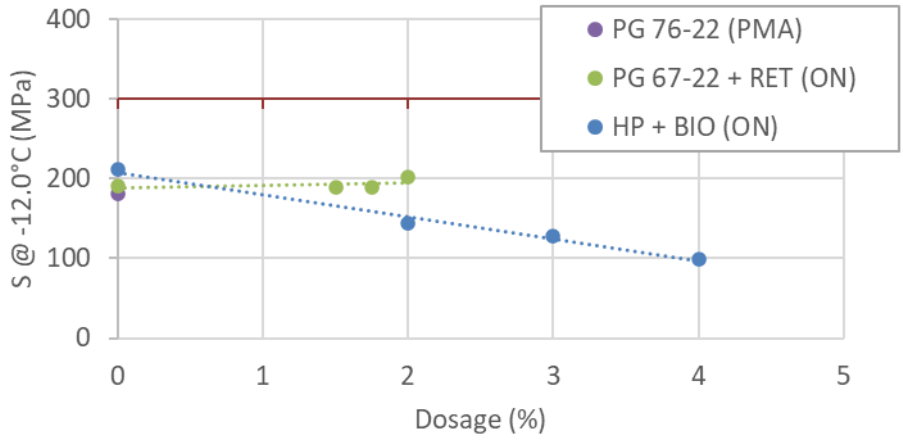
(b)

Figure 3-3. MSCR Tests: %$Rec_{3.2}$ versus Dosage; (b) $Jnr_{3.2}$ versus Dosage at 67.0°C

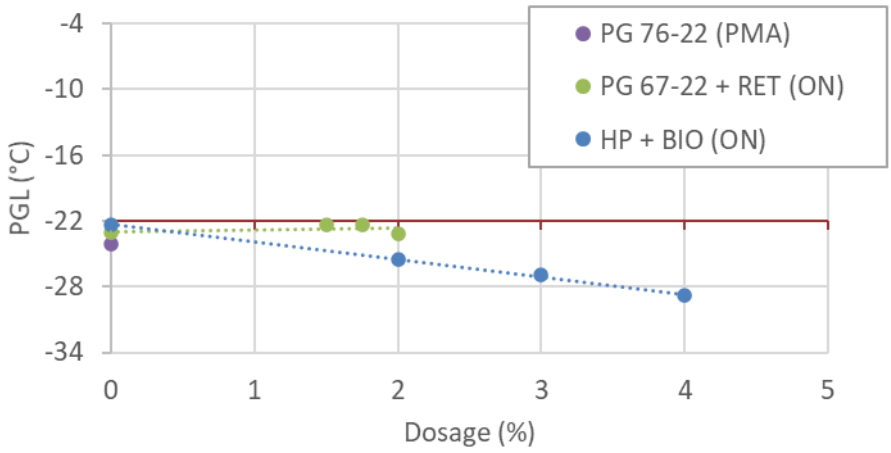


(a)

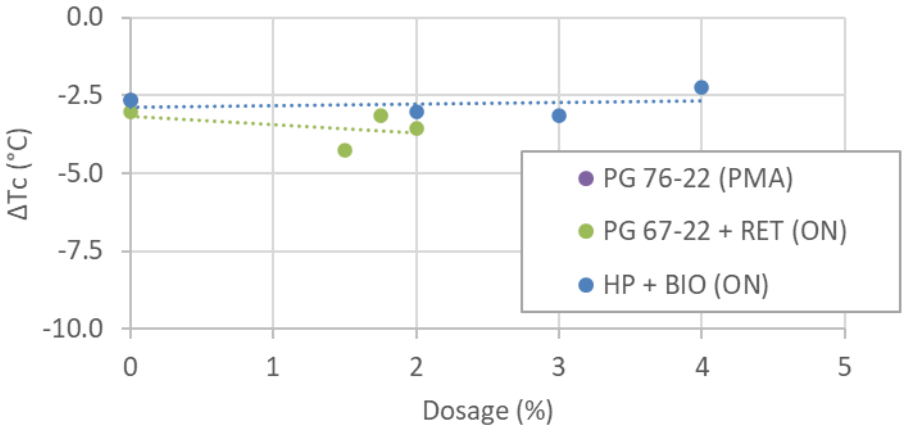
Figure 3-4. BBR Tests: (a) *m* at -12.0°C versus Dosage; (b) *S* at -12.0°C versus Dosage; (c) *PGL* versus Dosage; (d) ΔTc versus Dosage



(b)



(c)



(d)

Figure 3-4. BBR Tests: (a) *m* at -12.0°C versus Dosage; (b) *S* at -12.0°C versus Dosage; (c) *PGL* versus Dosage; (d) Δ*Tc* versus Dosage, Continued

Table 3-2 presents the results of the PG verification tests conducted on the blends of PG 67-22 binder modified with 1.75% RET and 0.35% PPA and on the blends of HP binder and 3.00% BIO by weight of HP binder. The table clearly shows that, in terms of their true grade, these blends satisfied each requirement of PG 76-22 (PMA). As such, these binders are referred to as PG 76-22 (RET) and PG 76-22 (BIO) binders in this document.

Table 3-2. Control and AMA Binder PG Verification Results

Test	Temp. (°C)	PG 76-22 (PMA) Criteria	PG 76-22 (PMA)	PG 67-22 + 1.75%RET + 0.35%PPA	HP + 3.00%BIO
DSR	76.0	$G^*/Sin\delta \geq 1.0$ kPa	1.29 ± 0.00	1.31 ± 0.00	1.44 ± 0.16
		$\delta \leq 75$ deg.	69.8 ± 0.1	77.6 ± 0.0	64.4 ± 0.9
	PGH		79.8 ± 0.0	78.6 ± 0.0	80.3 ± 1.4
MSCR	67.0	$Jnr_{3.2} \leq 1.0$ kPa	0.3 ± 0.0	0.3 ± 0.1	0.1 ± 0.0
		$Grade \geq$ V	E	E	E
		$Jnr_{Diff} \leq 75\%$ (No applicable if $Jnr_{3.2} \leq 0.5$)	61.3 ± 0.6	39.0 ± 19.8	86.0 ± 39.1
		$\%Rec_{3.2} \geq 29.37 \times Jnr_{3.2}^{0.2633}$	67.8 ± 2.3 > 39.9	41.3 ± 1.4 > 40.2	88.2 ± 1.4 > 56.4
BBR	-12.0	m @ 60 sec ≥ 0.300	0.323 ± 0.003	0.304 ± 0.003	0.349 ± 0.004
		S @ 60 sec ≤ 300 MPa	182.0 ± 2.6	189.0 ± 28.3	128.3 ± 5.5
		$\Delta Tc \geq$ -5°C	-2.6 ± 0.7	-3.2 ± 0.2	-3.2 ± 0.5
	PGL		-24.1 ± 0.2	-22.4 ± 0.2	-26.9 ± 0.1
DSR	26.5	$G^*.Sin\delta \leq 6000$ MPa	2657 ± 114	4823 ± 173	2133 ± 614
True Grade			78.9-24.1	78.6-22.4	80.3-26.9
PG 76-22?			Yes	Yes	Yes

PG 76-22 (PMA), PG 76-22 (RET), and PG 76-22 (BIO) binders were then subjected to several different asphalt binder performance tests to evaluate whether PG 76-22 (RET) and PG 76-22 (BIO) binders would perform at least equivalent to PG 76-22 (PMA) binder. The tests of PG 67-22 and HP were also included in this evaluation to study the type of effect each AAB additive had on the properties of the base asphalt binders. Table 3-3 presents the selected types of asphalt binder tests including the geometrical dimensions, the loading and aging conditions, and the associated parameters. As seen in this table, these tests include:

- Four basic Superpave PG specification tests (FDOT, 2020): (a) the DSR tests at PGH, (b) the MSCR tests at 67.0°C, (c) the DSR tests at 26.5°C and (d) the BBR tests at PGL, and

- Four additional tests: (a) the ΔTc tests at PGL, (b) the LAS tests at 15.0°C, (c) the GR damage parameter tests at 15.0°C, and (d) the FTIR spectroscopy tests at normal room temperature.

Table 3-3. Asphalt Binder Test Plan: Base, PMA, and AMA Binders

Test	Geometry	Loading	Aging	Parameters
Rutting				
DSR	D = 25.0 mm H = 1.0 mm	ω = 10 rad/sec. T = Grade	OB RTFO	Rutting resistance parameter: G^*/Sinδ High Temp. PG: PGH
MSCR	D = 25.0 mm H = 1.0 mm	τ = 0.1, 3.2 kPa t_c = 0.1 sec t_r = 0.9 sec T = 67°C	RTFO	Non-recoverable compliance: Jnr Percent recovery: $\%Rec$
Cracking				
BBR	L = 127.0 mm B = 12.7 mm H = 6.25 mm	P = 980 mN T = Grade	RTFO + PAV20	Creep slope: m Creep stiffness: S Critical temp: T_c Low temp. PG: PGL
ΔTc	L = 127.0 mm B = 12.7.0 mm H = 6.25 mm	P = 980 mN T = Grade	RTFO + PAV20 RTFO + PAV40 RTFO + PAV60	$\Delta Tc = T_{cs} - T_{cm}$
DSR	D = 8.0 mm H = 2.0 mm	ω = 10 rad/sec T = 26.5°C	RTFO	Cracking resistance parameter: $G^*Sin\delta$
LAS	D = 8.0 mm H = 2.0 mm	f = 10 Hz γ = 0.1-30% T = 15°C	RTFO	Failure cycles: N_f Fracture resistance energy index: $FREI$
GR	D = 8.0 mm H = 2.0 mm	ω = 0.005 rad/sec γ = 0.1% T = 15°C	RTFO RTFO + PAV20 RTFO + PAV40 RTFO + PAV60 RTFO + PAV80	Damage Parameter: GR $PAV_{GR=180\ kPa}$ $PAV_{GR=600\ kPa}$
FTIR	H = Thin Film	T = Normal	RTFO RTFO + PAV20 RTFO + PAV40 RTFO + PAV60 RTFO + PAV80	Carbonyl area: CA
Notes:	D = diameter; L = length; B = breath; H = gap thickness (or height); ω = angular frequency; f = linear frequency; T = temperature; τ = shear stress; t_c = creep time; t_r = recovery time; P = creep load; G^* = complex modulus; δ = phase angle; PAVx, where x = 20, 40, 60 or 80 hours of PAV aging; $PAV_{GR=180\ or\ 600\ kPa}$ = critical PAV hours at damage onset and severe damage			

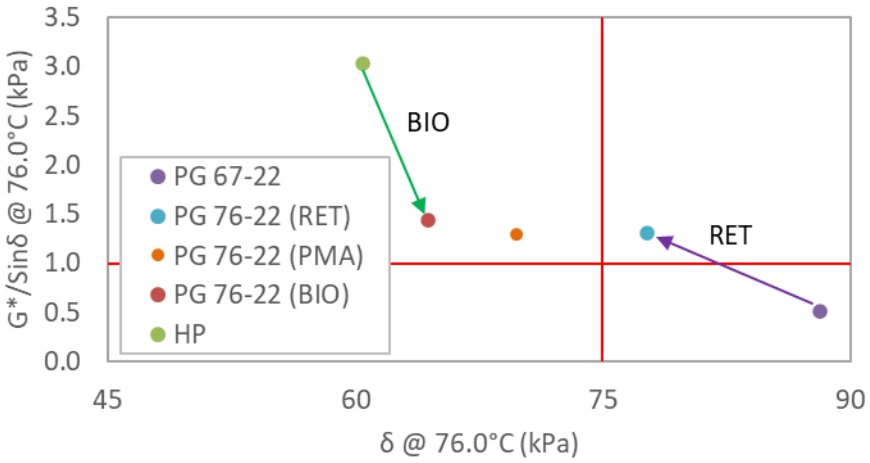
3.2.3 Asphalt Binder Rutting Tests

3.2.3.1 DSR Tests

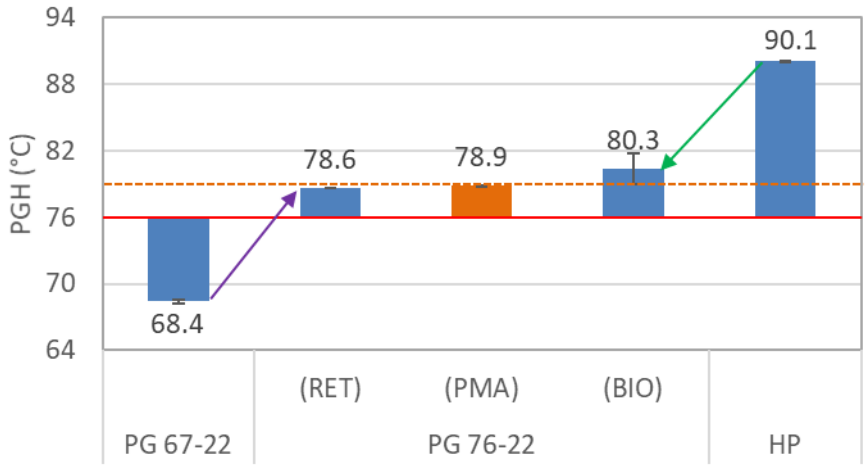
Figure 3-5(a) and Figure 3-5(b) present $G^*/Sin\delta$ versus the δ values of the control, the two base asphalt binders, and the two AMA binders at 76.0°C and the bar chart of their PGH values, respectively. The $G^*/Sin\delta$ and the δ values of PG 67-22, PG 76-22 (PMA), PG 76-22 (RET) and PG 76-22 (BIO) binders at 76.0°C were directly measured using the DSR tests at that temperature. The $G^*/Sin\delta$ and the δ values of the HP binder at 76.0°C were estimated by using the trend lines of directly measured $G^*/Sin\delta$ and δ values of the HP binder at three other temperatures (i.e., 82.0°C, 88.0°C and 94.0°C). The figures show that, at 76.0°C, PG 76-22 (RET) binder had a significantly lower δ value (significantly less ductility) but significantly higher $G^*/Sin\delta$ and PGH values (significantly higher rutting resistance or tolerance) compared to PG 67-22 binder while it had a slightly higher δ value (slightly more ductility) but equivalent $G^*/Sin\delta$ and PGH values (equivalent rutting resistance) compared to PG 76-22 (PMA) binder.

In contrast, PG 76-22 (BIO) binder had a slightly higher δ value (slightly more ductility) but significantly lower $G^*/Sin\delta$ and PGH values (significantly lower rutting resistance) than HP binder while it had a slightly lower δ value (slightly less ductility) but slightly higher $G^*/Sin\delta$ and PGH values (slightly higher rutting resistance) compared to PG 76-22 (PMA) binder.

Based on these results, both AMA binders would have at least equivalent rutting resistance despite having slightly different phase angle (brittleness or ductility) when compared to PG 76-22 (PMA) binder.



(a)



(b)

Figure 3-5. DSR Tests: (a) *G*/Sinδ* versus *δ* at 76.0°C; (b) *PGH*

3.2.3.2 MSCR Tests

Figure 3-6 presents the $Rec_{3.2}$ versus the $Jnr_{3.2}$ diagram and the $Rec_{3.2} = 26.93\, Jnr_{3.2}^{-0.2633}$ comparison line. The figure shows that PG 76-22 (RET) binder had significantly higher $\%Rec_{3.2}$ and lower $Jnr_{3.2}$ values (i.e., significantly more rutting resistance, and completely different MSCR-based grade i.e., "*E*" grade versus "*S*" grade) compared to PG 67-22 binder but lower $\%Rec_{3.2}$ and equivalent $Jnr_{3.2}$ values (i.e., lower overall rutting resistance despite the equivalent MSCR-based grade, i.e., both "*E*" grades) compared to PG 76-22 (PMA) binder.

The figure also shows that PG 76-22 (BIO) binder had slightly lower $\%Rec_{3.2}$ and slightly higher $Jnr_{3.2}$ values (i.e., almost equivalent rutting resistance and the same MSCR-based grade, i.e., both "*E*" grades) compared to PG 82-22 binder but higher $\%Rec_{3.2}$ and lower $Jnr_{3.2}$ values (higher rutting resistance despite the equivalent MSCR-based grade, i.e., both "*E*" grades) compared to PG 76-22 (PMA) binder.

Based on these results, when compared to PG 76-22 (PMA) binder, despite having the same PG and MSCR-based grade (i.e., PG 76-22 "*E*"), PG 76-22 (RET) binder had equivalent rutting resistance based on $Jnr_{3.2}$ but lower rutting resistance based on $Rec_{3.2}$ value while PG 76-22 (BIO) binder always had better rutting resistance based on both parameters.

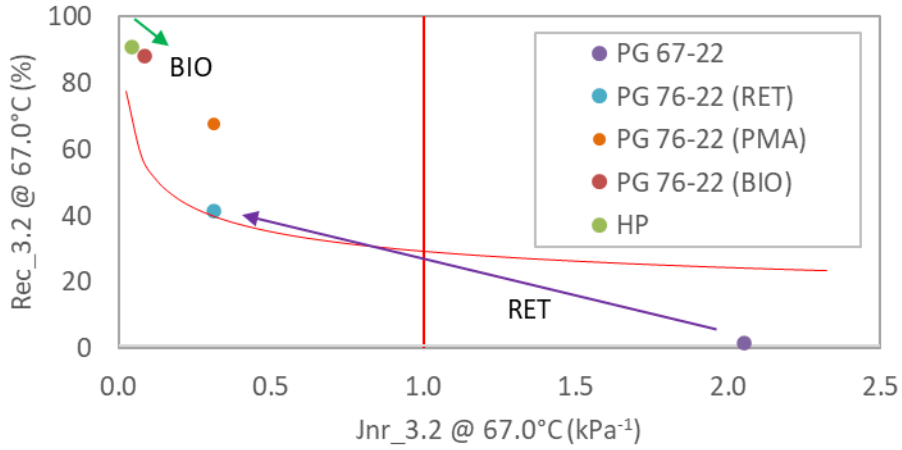
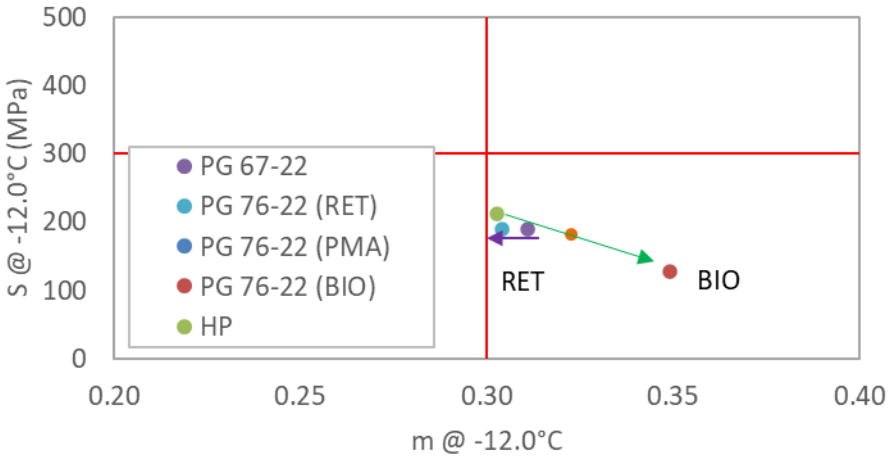


Figure 3-6. MSCR Tests: *Rec3.2* versus *Jnr3.2* at 67.0°C

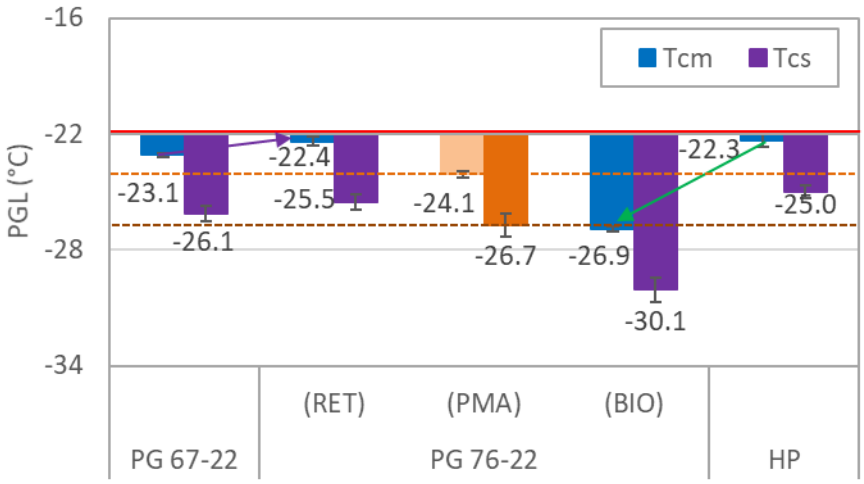
3.2.4 Asphalt Binder Cracking Tests

3.2.4.1 BBR Tests

Figure 3-7(a) and Figure 3-7(b) present the black space diagram of S versus m values of PAV20-aged samples at 76.0°C and the bar charts of T_{cs} and T_{cm} values of PAV20-aged samples. The figures show that PG 67-22, 76-22 (RET), PG 76-22 (PMA) and HP binders had equivalent S, m, T_{cm}, and T_{cs} values (i.e., similar low temperature cracking resistance) compared to each other while higher S, lower m, higher T_{cm}, and higher T_{cs} values (i.e., lower temperature cracking resistance) compared to PG 76-22 (BIO) binder. Based on these results, both AMA binders would perform at least equivalent to PG 76-22 (PMA) binder at lower temperatures.



(a)



(b)

Figure 3-7. BBR Tests: (a) *S* versus *m* at -12.0°C; (b) *PGL*

3.2.4.2 ΔTc Tests

Figure 3-8 presents the bar chart of the ΔTc values of their PAV20-, PAV40- and PAV60-aged samples. The figure shows that ΔTc of each PAV20-aged asphalt binder sample was slightly lower than the damage onset criterion of -2.5°C and higher than the severe damage criterion of -5°C (Anderson et al., 2011). These values suggest that each binder would perform almost equivalent to each other at least until the PAV20-equivalent life cycle.

Figure 3-8 also shows that the ΔTc value decreased (i.e., became more negative) with an increase in the aging level, overcoming -5.0°C in both PAV40 and PAV60 aging levels in

each case, which was not a welcome outcome. The figure shows that ΔTc at PAV40 (i.e., ΔTc_{PAV40}) value ranked PG 67-22 binder, HP, PG 76-22 (PMA), PG 76-22 (RET) and PG 76-22 (BIO) from the best to the worst (i.e., -5.1°C, -5.2°C, -6.0°C, -6.0°C, and -7.3°C, respectively). And, the ΔTc value at PAV60 (i.e., ΔTc_{PAV60}) ranked PG 76-22 (PMA), HP, PG 67-22, PG 76-22 (BIO) and PG 76-22 (RET) from the best to the worst (based on -6.6°C, -7.4°C, -8.0°C, -9.3°C, and -9.8°C, respectively) with PG 76-22 (PMA) as the best and PG 76-22 (RET) as the worst (i.e., -6.6°C versus -9.8°C).

Based on these results, the effect of aging on ΔTc values of PG 76-22 (RET) and PG 76-22 (BIO) binders would be equivalent until PAV20 but more severe afterward (i.e., from PAV40 until PAV60) when compared to PG 76-22 (PMA) binder.

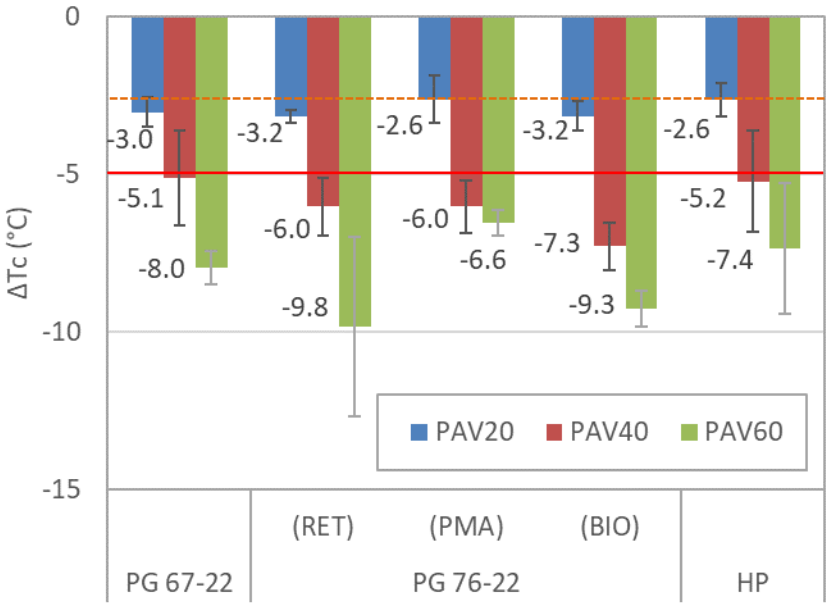


Figure 3-8. ΔTc Tests: PAV20, PAV40, and PAV60

3.2.4.3 DSR Tests

Figure 3-9 presents the $G^*.Sin\delta$ values of PAV20-aged asphalt binders at 26.5°C and 10.0 rad/sec. The figure shows that PG 76-22 (RET) binder had a higher $G^*.Sin\delta$ value (i.e., stiffer with a lower fatigue cracking resistance) than PG 67-22 base asphalt binder but a lower $G^*.Sin\delta$ value (i.e., softer with a higher fatigue cracking resistance) than PG 76-22 (PMA) binder. The figure also shows that PG 76-22 (BIO) binder had a lower $G^*.Sin\delta$ value (i.e., softer with a higher fatigue cracking resistance) than HP binder but higher

$G^*.Sin\delta$ value (i.e., stiffer with lower fatigue cracking resistance) than PG 76-22 (PMA) binder. In addition, the figure shows that the $G^*.Sin\delta$ value of PG 76-22 (RET) binder almost surpassed its maximum allowable value i.e., 4,945 kPa measured versus 5000 kPa maximum allowed, which was not that surprising since PG 67-22 binder itself had a $G^*.Sin\delta$ value of 3,986 kPa.

Based on these results, despite having the same grade, PG 76-22 (RET) binder would have less while PG 76-22 (BIO) binder would have more resistance to fatigue cracking than PG 76-22 (PMA) binder.

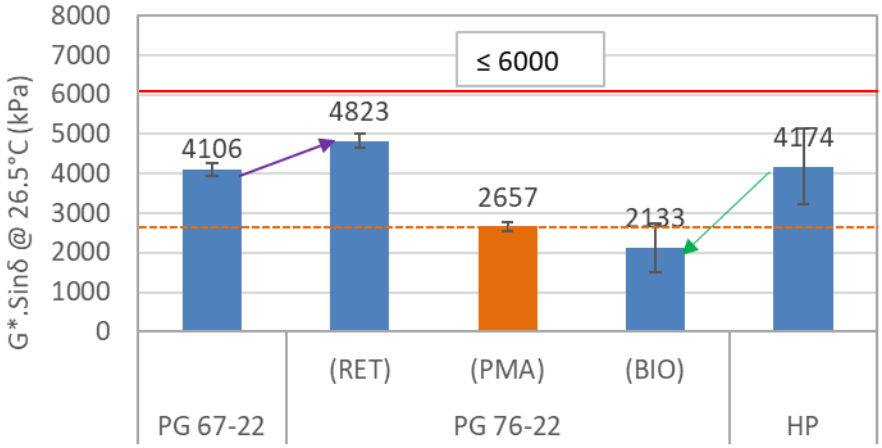


Figure 3-9. DSR Tests: $G^*\sin\delta$ at 26.5°C

3.2.4.4 LAS Tests

The LAS test is a two-step damage-inducing test of asphalt binder according to AASHTO TP 101-12 (AASHTO, 2018a). The first step is a frequency sweep test (FST) step that runs from 0.1 to 100.0 radian per sec at a predetermined temperature and LVE shear strain of 0.1% without causing any damage to the specimen. The second step is an actual LAS step that runs from 0.2% to 30.0% shear strain at 10.0 Hz at the FST temperature and causes incremental damage. In this study, the LAS tests were conducted on the RTFO-aged samples of each binder at 15.0°C (chosen to be the same as the GR test temperature) on a DSR using parallel plate specimens that measured 8.0 mm in diameter and 2.0 mm in thickness. The data obtained from these tests were analyzed using two different approaches: (a) VECD mechanics-based approach (Hintz and Bahia, 2013b), and (b) fracture damage mechanics (FDM)-based approach (Zhou et al., 2017c).

In the VECD mechanics-based approach, the first step of this test was used to obtain the LVE slope parameter, m and corresponding inverse parameter, α (Hintz and Bahia, 2013b):

$$\alpha = \frac{1}{m}$$

$$\log G' = m \times \log \omega + c$$

where,

α = damage rate parameter

G'= storage shear modulus (MPa)

ω = loading frequency (rad/sec)

m = slope obtained from FST step

c = intercept obtained from FST step

Similarly, the second step of this test was used to obtain the characteristic (pseudo stiffness, C versus continuum damage, S curves and estimate the number of cycles to fatigue failure (N_f) at predetermined loading conditions (Hintz and Bahia, 2013b):

$$C(t) = \frac{|G^*|(t)}{|G*|_{initial}}$$

$$S(t) = S_0 + \sum_{i=0}^{N} \left[\pi \times (\gamma_0)^2 \times (C_{i-1} - C_i)^{\left(\frac{\alpha}{1+\alpha}\right)} \times (t_i - t_{i-1})^{\left(\frac{1}{1+\alpha}\right)}\right]$$

where,

$C(t)$ = pseudo stiffness at testing time, t

t = testing time (sec)

$|G^*|$ = complex shear modulus at testing time, t (MPa)

S = accumulated damage

S_0 = initial damage = 0.0

The C versus S curves were fitted and then used to estimate corresponding number of cycles to fatigue failure (N_f) at the predetermined loading conditions (Hintz and Bahia, 2013b):

$$C(t) = C_0 - C_1(S)^{C_2}$$

$$N_f = A \times (\gamma_{max})^{-B}$$

$$A = \frac{f \times (S_f)^k}{\pi \times C_1 \times C_2}$$

$$k = 1.0 + (1 - C_2) \times \alpha$$

$$S_f = \left(\frac{C_0 - C_{\tau_{max}}}{C_1}\right)^{\frac{1}{C_2}}$$

$$f = \frac{\omega}{2 \times \pi}$$

$$B = 2 \times \alpha$$

where,

C_0 = 1.0

C_1, C_2= characteristic curve-fit coefficients

N_f = number of cycles at failure

S_f = damage at failure

f = loading frequency = 10 Hz

τ_{max} = maximum shear stress experienced by the asphalt binder during LAS step (kPa)

γ_{max} = maximum shear strain expected to be experienced by the asphalt binder for a given pavement structure (%)

In the FDM-based approach, the second step was used to obtain the fracture resistance energy index, $FREI$ (Zhou et al., 2017c):

$$FREI = \frac{J_{f,\tau max}}{G_{0.5\tau_{max}}} \times \left(\gamma_{0.5\tau_{max}}\right)^2$$

where,

$J_{f,\tau max}$ = shear fracture energy calculated till maximum shear stress

$G_{0.5\tau_{max}}$ = shear modulus at the point of half of the maximum shear stress

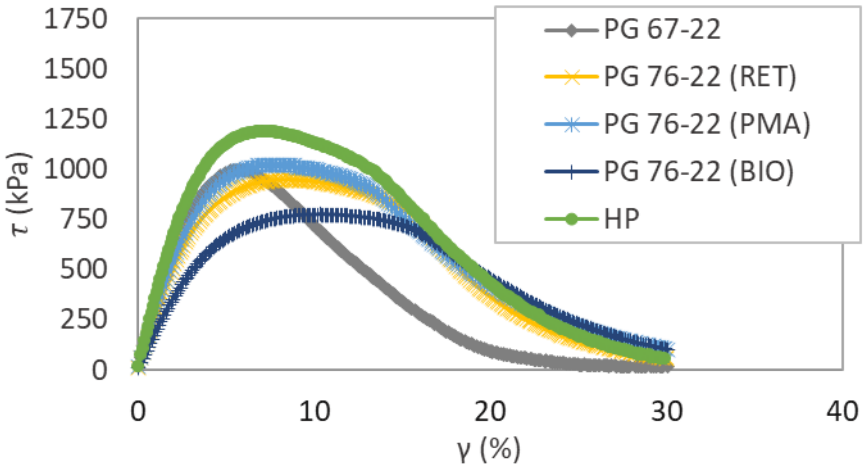
$\gamma_{0.5\tau_{max}}$ = shear strain at the point of half of the maximum shear stress

τ_{max} = maximum shear stress

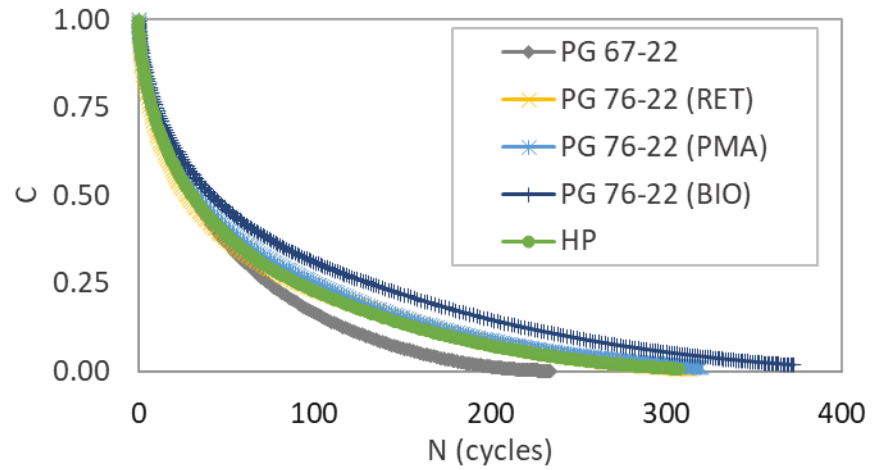
Generally, asphalt binders that have higher predicted N_f values or measured $FREI$ values perform better in terms of fatigue performance.

Figure 3-10(a), Figure 3-10(b), Figure 3-10(c), and Figure 3-10(d) present the τ versus γ curves, the characteristic C versus S curves, the VECD-estimated failure cycles at 2.5% and 5.0% shear strain, and their FREI parameters, respectively. The figures show that both

PG 76-22 (RET) and PG 76-22 (BIO) binders had higher N_f and FREI values compared to PG 67-22, 76-22 (PMA), and HP binders, and therefore would resist fatigue cracking more effectively than these three binders. The figures also show that, in terms of both N_f and FREI values, PG 76-22 (BIO) was the best followed by PG 76-22 (RET), PG 76-22 (PMA), HP, and PG 67-22 (worst).

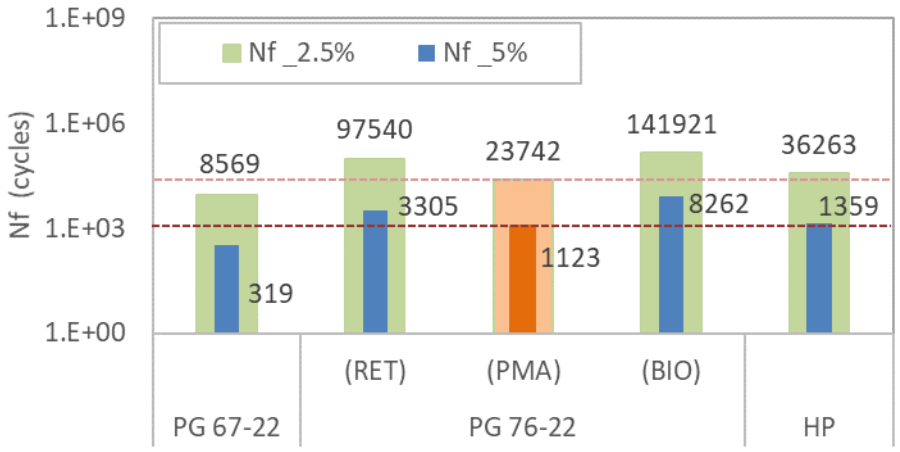


(a)

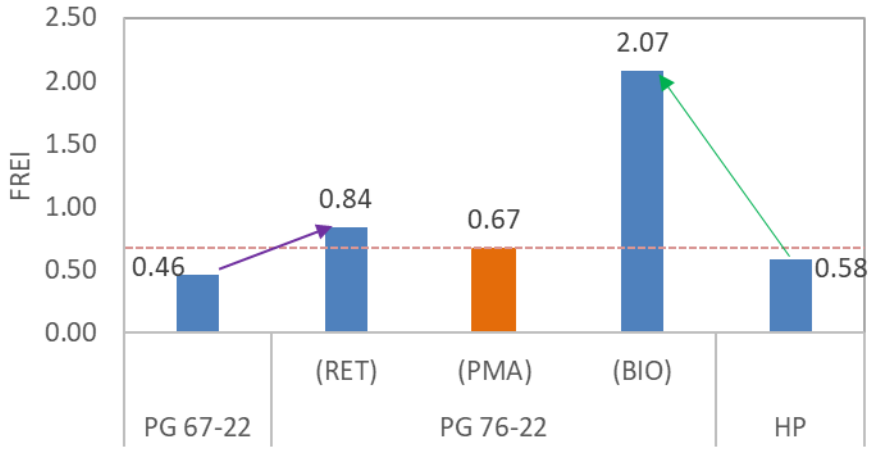


(b)

Figure 3-10. LAS Tests: (a) τ versus γ at 15.0°C; (b) *C* versus *S* at 15.0°C; (c) N_f at 15.0°C; (d) *FREI* at 15.0°C



(c)



(d)

Figure 3-10. LAS Tests: (a) τ versus γ at 15.0°C; (b) *C* versus *S* at 15.0°C; (c) N_f at 15.0°C; (d) *FREI* at 15.0°C, Continued

3.2.4.5 GR Tests

The GR damage parameter tests were conducted on the parallel plate specimens (8.0 mm in diameter and 2.0 mm in thickness) of asphalt binders at six different aging levels (OB, RTFO or PAV0, PAV20, PAV40, PAV60, and PAV80) using a DSR. For these tests, a temperature of 15.0°C, a shear strain amplitude of 0.1%, and a loading frequency of 0.005 rad/sec were used.

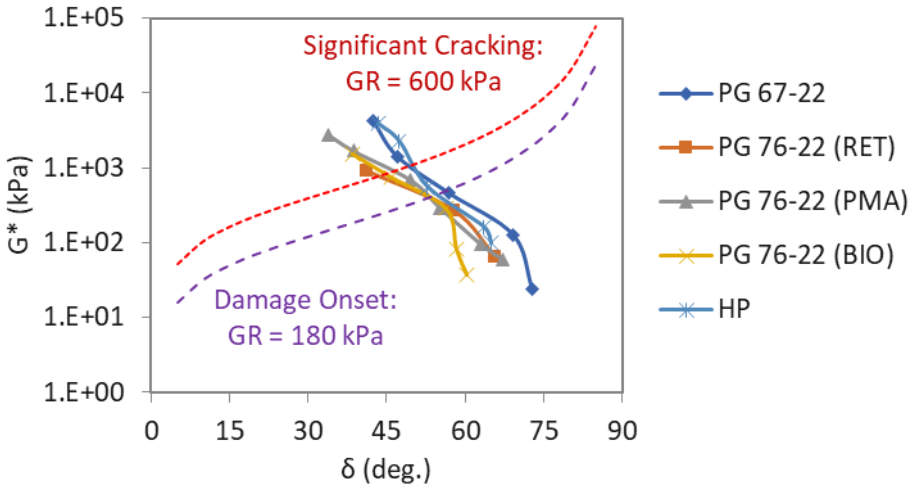
From these tests, G^* and δ were obtained at each aging level, which were then used to calculate corresponding values of the GR parameter using:

$$GR = \frac{G'}{\left(\frac{\eta'}{G'}\right)} = G^* \times \frac{Cos^2\delta}{Sin\delta}$$

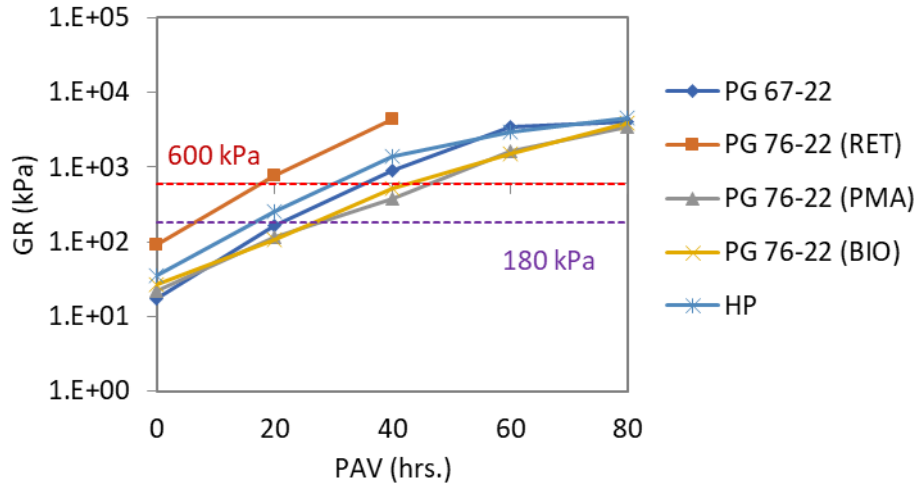
The GR damage parameter serves as an indicator of brittle failure proneness of asphalt binders (Glover et al., 2005; King et al., 2012). Generally, G^* increases and δ decreases with more aging, thereby increasing the GR values and making asphalt binders more prone to brittle failure by cracking.

Figure 3-11(a) and Figure 3-11(b) present respectively (1) the black space diagram of the G^* versus the δ values obtained from the OB to the PAV80 samples and (2) the bar chart of the GR versus PAV (aging) durations obtained from RTFO (or PAV0) to PAV80 samples. The figures show that, at a given aging level, PG 76-22 (RET) binder had generally a higher G^*, a lower δ, and a higher GR (i.e., higher brittleness) while PG 76-22 (BIO) binder had generally a lower G^*, a higher δ, and lower GR values (i.e., lower brittleness) compared to corresponding base asphalt binders, which highlights opposite (i.e., negative versus positive) effects of the RET and the BIO additives on the GR values. The figures also show that, at a given aging level, PG 76-22 (RET) binder had generally higher G^*, lower δ, and higher GR values (i.e., higher brittleness) while PG 76-22 (BIO) binder had generally equivalent G^*, δ, and GR values (i.e., equivalent brittleness) compared to PG 76-22 (PMA) binder.

Figure 3-11(a) and Figure 3-11(b) also present the curves corresponding to the GR values of 180.0 kPa and 600.0 kPa. These values refer to the DSR-based criteria developed for damage onset and severe damage based on 3.0 cm and 5.0 cm of ductility of unmodified asphalt binders at 15.0°C and 1.0 cm/min, respectively (Glover et al., 2005). Even though neither of these two criteria have been validated for modified asphalt binders and no new GR criteria have been proposed for such binders (Newcomb et al., 2018), GR values of each binder and these two criteria were used to evaluate the type (positive or negative) of effect of the AAB additive on overall cracking properties and to determine critical PAV hours corresponding to these criteria.



(a)



(b)

Figure 3-11. GR Tests: (a) G^* versus δ at 15.0°C; (b) GR at 15.0°C versus PAV

Figure 3-12 presents the critical PAV hours at which the GR curves of asphalt binders intersect the damage onset (i.e., GR = 180.0 kPa) and severe damage (i.e., GR = 600.0 kPa) curves, respectively. The figure shows that the GR curves of PG 76-22 (RET) binder intersected these curves at much lower PAV hours than PG 67-22 binder, revealing an increase in aging rate with the use of RET and PPA (not a welcome outcome). In contrast, the figure also shows that the GR curves of PG 76-22 (BIO) binder intersected these curves at much higher PAV hours than HP binder, underscoring a decrease in aging rate with the use of BIO (a welcome outcome). The figure also shows that, compared to PG 76-22 (PMA) binder, the GR curves of PG 76-22 (RET) and PG 76-22 (BIO) binders intersected these

curves at much lower PAV hours. This figure shows that both PAV corresponding to damage onset and severe damage (i.e., $PAV_{GR=180\,kPa}$ and $PAV_{GR=600\,kPa}$, respectively) ranked PG 76-22 (PMA), PG 76-22 (BIO), PG 67-22, HP, and PG 76-22 (RET) binders from best to the worst in terms of aging rate, revealing the potential use of this parameter.

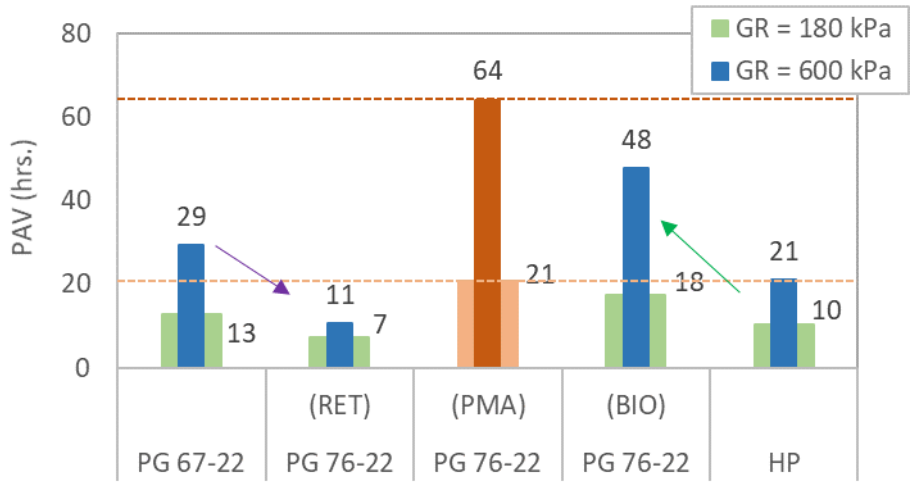


Figure 3-12. GR Tests: *PAV* at Damage Onset at 15.0°C for 180 kPa and 600 kPa Conditions

Based on the correlation of the field and the laboratory data, Anderson et al. (2011) previously suggested using either the GR or the ΔTc to evaluate asphalt pavement cracking resistance. However, the GR values of 180.0 and 600.0 kPa for damage onset and severe damage were originally developed for unmodified binders; therefore, whether the same criteria can be used for modified binders is debatable. Similarly, the ΔTc values showed significant difference between asphalt binders only at higher PAV aging levels, which makes it more promising but less practical in terms of the amount of binders needed to run BBR tests at PAV60.

3.2.4.6 FTIR Tests

The FTIR test is a spectroscopy test used to determine the chemical composition of selected material by measuring the amount of infrared light absorbed as it passes through or emitted as it reflects from the test material. In terms of asphalt binder chemistry, aging is attributed to the increase in the concentration of the carbonyl functional group (C=O) at the benzylic position of an aromatic molecule (Newcomb et al., 2018). This group absorbs

infrared light with frequencies from 1650 cm^{-1} to 1820 cm^{-1} with a peak near 1700 cm^{-1} (Glover et al., 2005).

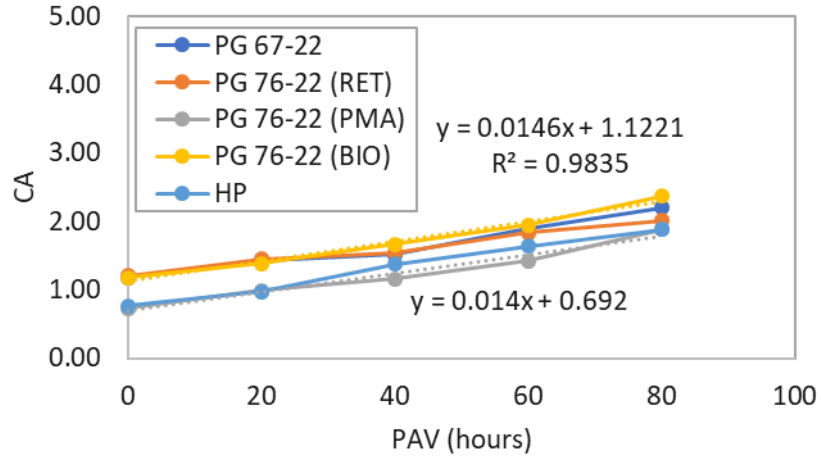
In this study, FTIR tests were conducted on thin films of OB, RTFO- (or PAV0-), PAV20-, PAV40-, PAV60-, and PAV80-aged asphalt binder samples. Then, the areas under absorbance versus frequency spectra that corresponded to carbonyl functional group, called carbonyl area (CA), were used to determine their aging susceptibility.

Figure 3-13(a) presents the absolute CA values of each binder. The results show that, as expected, the CA value increased with each aging level irrespective of asphalt binder type. The figure also shows that, at each aging level, the absolute CA values were lower in PG 76-22 (PMA) and HP binders than in PG 67-22, PG 76-22 (RET) and PG 76-22 (BIO) binders, highlighting differences in SBS-only modified binders versus their counterparts. The relatively higher absolute CA values mirror the relatively lower $PAV_{GR=180kPa}$ and the $PAV_{GR=600\ kPa}$ values in PG 67-22, PG 76-22 (RET) and PG 76-22 (BIO) binders compared to corresponding values in PG 76-22 (PMA) binder.

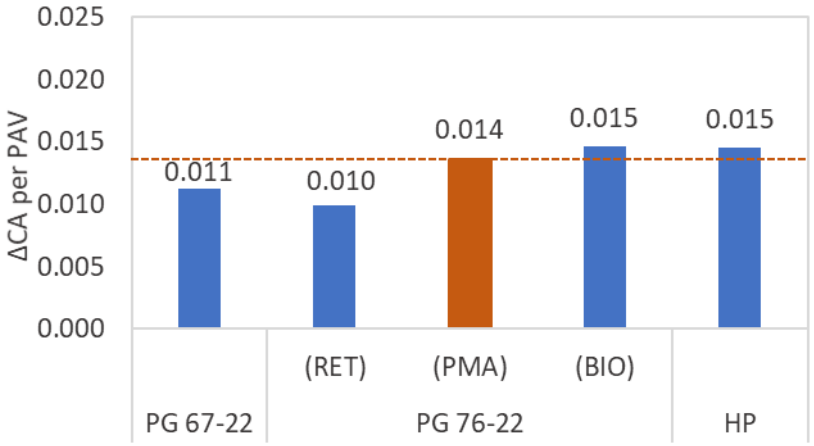
Figure 3-13(b) presents the rate of change (an increase, to be precise) in CA values of each binder. In terms of aging rate (defined in terms of the ΔCA per PAV), the figure shows that HP and PG 76-22 (BIO) binders aged at almost the same rate, which suggest that BIO had minimal effect on aging rate. The figure also shows that unmodified PG 67-22 and PG 76-22 (RET) binders aged at almost the same rate, which suggests that RET also had minimal effect on aging rate. Based on these effects, it can be concluded that the AAB additives did not affect the aging rate of the base asphalt binders. Figure 3-13(b) also shows that PG 67-22 and PG 76-22 (RET) binders aged slightly slower than PG 76-22 (PMA) binder (which is generally a good outcome) while HP, PG 76-22 (BIO) and PG 76-22 (PMA) binder aged at almost the same rate compared to each other, most likely due to the insignificant effect of BIO in the SBS-modified HP binder (which is not a bad outcome).

The PAV_{GR} and the $\Delta CA/hr$ values for the PG 67-22 and PG 76-22 (RET) binders contrast each other when compared to the PG 76-22 (PMA) binder. The researchers believe the $PAV_{GR=180\ or\ 600\ kPa}$ properties better capture the effect of aging on rheology than $\Delta CA/hr$ test. This can be attributed to the fact that FTIR is purely a chemical test while GR is a mechanical test measuring binder rheology at different aging conditions. Since exact

criteria have not been developed yet for both, the results can be used for relative comparison only.



(a)



(b)

Figure 3-13. FTIR Tests: (a) *CA* versus *PAV*; (b) Δ*CA*/*PAV*

3.3 Asphalt Mixture Performance Evaluation

Five different types of Type SP-12.5 asphalt mixtures were selected to develop the protocol for evaluating the AAB and the AAM additives in line with the second objective of this study. These mixtures include the asphalt mixture produced with PG 76-22 (PMA) binder without any AAB and AAM additives (i.e., the control asphalt mixture), two asphalt mixtures produced with the AMA binders without any AAM additives, and two asphalt mixtures produced with PG 76-22 (PMA) binder and the AAM additives together without AAB additives:

- Control Asphalt Mixture:
  - Aggregates + PG 76-22 (PMA) = PMA mixture
- AMA Mixtures:
  - Aggregates + PG 76-22 (RET) = RET mixture
  - Aggregates + PG 76-22 (BIO) = BIO mixture
- FMA Mixtures:
  - Aggregates + PG 76-22 (PMA) + ACE XP = ACE XP mixture
  - Aggregates + PG 76-22 (PMA) + FORTA-FI = FORTA-FI mixture

PG 76-22 (PMA), PG 76-22 (RET) and PG 76-22 (BIO) and their corresponding mixtures [i.e., the PMA, the RET, and the BIO mixtures] were used to develop the protocol for evaluating and approving the AAB additives in line with the first objective of this study. Similarly, the PMA, the AMA and the FMA mixtures (the PMA, the RET, the BIO, the ACE XP, and the FORTA-FI mixtures) were used to develop the protocol for evaluating and approving the AAM additives in line with the second objective of this study.

3.3.1 Asphalt Mix Gradation Verification

As the first step of mix design verification, the granite aggregates and local sand obtained for use in this study were subjected to washed sieve analysis for mix gradation verification. The test results showed that the two gradations were very similar to each other, which suggested no need of adjustment to meet the design gradation (see Table 3-4).

Table 3-4. Control Mixture Gradation Verification Results

Sieve No.	Sieve Size (mm)	JMF	Washed Sieve Analysis Gradation	Difference
¾”	19.0	100.0	100	+0.0
½”	12.5	100.0	100	+0.0
3/8”	9.5	88.0	87.4	-0.6
#4	4.75	64.0	62.7	-1.3
#8	2.36	43.0	42.9	-0.1
#16	1.19	31.0	31.1	+0.1
#30	0.60	21.0	21.5	+0.5
#50	0.30	16.0	16.3	+0.3
#100	0.15	8.0	8.0	+0.0
#200	0.075	4.2	4.6	+0.4

3.3.2 Asphalt Mix Design Verification

Next, the mix design verification was then conducted on the asphalt mixture produced with the granite aggregates and PG 76-22 (PMA) binder without any AAB and AAM additives (i.e., the PMA mixture) through a series of steps:

1. Aggregates were batched following the JMF and pre-heated in an oven at the JMF-specified mixing temperature of 154.0°C (309.2°F) for a minimum of 6.0 hours.
2. The asphalt binder (usually, in a quarter can) and the mixing-and-compaction tools (such as a bucket, a paddle, a scoop, a spatula, a blade and compaction molds) were preheated in the oven at the JMF-specified mixing temperature of 154.0°C (309.2°F) for at least 2.0 hours prior to mixing.
3. A crater was made at the center of the heated aggregates to pour the binder.
4. The heated aggregates were mixed with the JMF-specified percentage of pre-heated asphalt binder (i.e., P_b = 5.1% by weight of total mixture) using the bucket and paddle mixer within 3 minutes from the time asphalt binder was added.
5. The just-prepared loose mixture was short-term oven aged (STOA) at the JMF-specified compaction temperature of 154.0°C (309.2°F) for 2.0 hours following AASHTO R 30 (AASHTO, 2014d). The mixture was stirred at least once during this aging process.
6. The STOA mixture was compacted with a Superpave gyratory compactor (SGC), until the JMF-specified design number of gyrations (i.e., N_{design}= 75) was reached, to obtain two cylindrical samples following AASHTO T 312 (AASHTO, 2015d).
7. The remaining part of the STOA mixture was cooled to room temperature and, after separating the conglomerate particles adequately, was used to obtain theoretical maximum specific gravity of the mixture (Gmm) at 25.0 ± 1.0°C (77.0 ± 1.8°F) following FM 1-T 209 (FDOT, 2017).
8. The two SGC samples were allowed to cool to room temperature overnight in open air and their bulk specific gravity (Gmb) at 25.0 ± 1.0°C (77.0 ± 2.0 °F) was obtained following FM 1-T 166 (FDOT, 2016b).
9. The relative density ($\%Gmm$), air void content (Va), voids in mineral aggregates (VMA), voids filled with asphalt (VFA), dust-to-effective binder ratio ($P_{0.075}/P_{be}$) were calculated following AASHTO M 323 (AASHTO, 2017f).

Table 3-5 presents the volumetric properties of the PMA mixture at 75 gyrations. The results show that the volumetric properties were very close to the FDOT-provided values in the JMF. Since these properties satisfied all requirements of Type SP-12.5 specified in Section 334-3.2.2 of the FDOT's current Specification (FDOT, 2020), no adjustment in asphalt binder content was performed.

Table 3-5. Control Mixture Volumetric Design Verification Results

Parameter		Criteria	FDOT	TTI
Gmm	Average	-	2.576	2.574
	Range	≤ 0.013	-	0.008
	Standard Deviation	≤ 0.00449	-	0.00333
Gmb	Average		2.473	2.468
	Range	≤ 0.011	-	0.003
Va		4.0 ± 0.5	4.0	4.1
VMA		≥14.0	14.4	14.6
VFA		65-75	72.0	71.8
$\frac{P_{0.075}}{P_{be}}$		0.6-1.2	1.0	1.0
Notes:	Gmm = theoretical maximum specific gravity of asphalt mixture; Gmb = bulk specific gravity of asphalt mixture; Va = air void content (%); VMA = voids in mineral aggregates (%); VFA = voids filled with asphalt (%); $P_{0.075}$ = percentage of aggregates passing no. 200 or 0.075-mm sieve (%); P_{be}= effective asphalt binder content (%)			

3.3.3 Asphalt Mixture Modification

Next, asphalt mixtures were produced for fabricating SGC samples for performance testing. To produce the BIO and the RET mixtures, the same steps used to produce the PMA mixture as mentioned in Section 3.3.1 were followed except that these mixtures were produced with PG 76-22 (RET) and PG 76-22 (BIO) binders, respectively. But, to produce the ACE XP and the FORTA-FI mixtures, several additional steps were followed (see Table 3-6).

Table 3-6. Control, AMA, and FMA Mixture Production[1]

Mixture	Control and AMA Mixtures	ACE XP Mixture	FORTA-FI Mixture
Additive Weighing	The AMA binders already incorporated the required amount of the RET and the BIO additives.	(a) The weight of fibers and wax estimated for the whole project was sent to the supplier. (b) Two separate bags—one with fibers and one with wax required for the whole project—were obtained from the supplier. (c) Fibers required for the individual batch were weighed on a precision scale (at the rate of 2.1 ounces of aramid fibers and 2.1 ounces of wax i.e., 4.2 ounces of the fiber mix per each ton of total mixture or 0.66 grams of the fiber mix per 10,000 grams of total mixture); their individual strands were separated and kept covered in a pan to prevent their loss before mixing. (d) Wax was weighed on a precision scale at the same rate of fibers (i.e., 2.1 ounces each per 1.0 ton of total mixture or 0.66 grams per 10,000 grams of total mixture).	(a) The weights of each individual mixture were sent to the supplier as required by them. (b) Two separate bags of fibers—Bag #1 and Bag # 2 as required for individual batch—were obtained from the supplier. (c) Bag #1 was separated into two equal parts at the laboratory, herein referred to as Bag #1A and Bag #1B. (d) The individual strands of the fibers in each bag were then separated and kept covered on a pan to prevent their loss before mixing.

[1] The steps used to produce ACE XP and FORTA-FI were shared by their suppliers.

Table 3-6. Control, AMA, and FMA Mixture Production (Continued)

Mixture	Control and AMA Mixtures	ACE XP Mixture	FORTA-FI Mixture
Aggregate Heating	(a) Aggregates were batched following the JMF. (b) Aggregates were heated at 154.0°C (309.2°F) for a minimum of 6.0 hours as recommended by the supplier of the fibers.	(a) Aggregates were batched following the JMF. (b) Aggregates were then heated at 154.0°C (309.2°F) for a minimum of 6.0 hours as recommended by the supplier of the fibers.	(a) Aggregates were batched following the JMF. (b). Aggregates were then separated into three equal weights in separate pans (Pan #1, Pan #2 and Pan #3). (c) Aggregate pans were heated at 6.0°C or 10.8°F higher than mixing temperature (i.e., 160.0°C or 320.0°F) for a minimum of 6.0 hours as recommended by the supplier of the fibers.
Binder Heating	Binder, mixing tools (such as a bucket mixer, a paddle, spatula, a blade, etc.) and compaction tools (such as a scoop, SGC molds, etc.) were heated in an oven at 154°C (309.2°F) for a minimum of 2.0 hours.		

Table 3-6. Control, AMA, and FMA Mixture Production (Continued)

Mixture	Control and AMA Mixtures	ACE XP Mixture	FORTA-FI Mixture
Mixing	(a) Aggregates were poured into the bucket in a circular motion. (b) A crater was made at the center of the aggregates. (c) Heated binder was poured into the crater. (d) Mixing was immediately started. (e) Mixing continued for about 3.0 minutes (a minimum of 1.5 minutes).	(a) Aggregates were poured into the bucket in a circular motion. (b) Wax was immediately added directly to the aggregates. (c) A crater was made at the center of the aggregates. (d) Heated binder was poured into the crater. (e) Mixing was immediately started. (f) Mixing continued for about 3.0 minutes (a minimum of 1.5 minutes) while manually adding the separated stands of aramid fibers.	(a) Aggregates from Pan #1 were poured into the bucket in a circular motion. (b) Bag #1A fibers were poured into the bucket in a circular motion. (c) Pan #2 aggregates were poured into the bucket in a circular motion. (d) Bag #1B fibers were poured into the bucket in a circular motion. (e) Pan #3 aggregates were poured into the bucket in a circular motion. (5) A crater was made at the center of the aggregates. (g) Heated binder was poured into the crater. (h) Bag #2 fibers were poured into the binder. (i) Mixing was immediately started. (j) Mixing continued for about 3 minutes (a minimum of 1.5 minutes).
STOA Aging	The mixture including the portion scraped from the side of the bucket was transferred to a pan and kept in the oven at 154.0ºC (309.2°F) for 2.0 hours following AASHTO R 30 (AASHTO, 2014d). The mixture was stirred midway during the STOA process.		
***Gmm* Test**	The STOA-aged mixture was cooled down and was subjected to Gmm test at 25.0 ± 1.0°C (77.0 ± 1.8°F) by following FM 1-T209 (FDOT, 2017).		

Table 3-6. Control, AMA, and FMA Mixture Production (Continued)

Mixture	Control and AMA Mixtures	ACE XP Mixture	FORTA-FI Mixture
SGC Compaction	The STOA-aged mixture was compacted at 154ºC (309.2°F) to SGC samples in a controlled gyration for mix design verification purpose or in a controlled height mode for performance testing by following AASHTO T 312 (AASHTO, 2015d).		
***Gmb* Test**	The compacted SGC samples were allowed to cool down and then subjected to Gmb test at 25.0 ± 1.0°C (77.0 ± 1.8°F) by following FM 1-T 166 (FDOT, 2016b) before or after cutting them as required by the selected test method.		
Notes:	The steps needed to produce PG 76-22 (RET) and PG 76-2 (BIO) binders are already provided in Section 0. The steps needed to produce the PMA mixture containing PG 76-22 (PMA) are already provided in Section 3.3.2. The steps needed to produce the ACE XP or the FORTA-FI mixtures were exactly the same as recommended by their suppliers.		

In this study, each asphalt mixture was compacted to a relative density of 93% and certain heights (as dictated by selected test type) and subjected to three rutting and three cracking tests.

Table 3-7 presents these tests and associated specimen geometries, test temperatures, loading conditions and parameters. At least three trial masses of the asphalt mixtures were compacted to design height (different for each test type) and the relationship obtained between the trial masses and the measured air void contents in trial SGC specimens were used to calculate the mass required to fabricate test specimens.

Table 3-7. Asphalt Mixture Test Plan: PMA, AMA, and FMA Mixtures

Test	Geometry	Condition	Loading	Parameters
Rutting				
Asphalt Pavement Analyzer: APA	D = 150 mm H = 75 ± 1 mm Va = 7.0 ± 0.5%	T = 64 ± 1.0°C	N_{max} = 8,000 cycles d_{max} = 4.5 mm p_w = 445 ± 22 N p_h = 690 ± 35 kPa f = 60 cycles/minute	$d_{8,000}$ $N_{4.5}$
Hamburg Wheel-Track: HWT	D = 150 mm H = 62 ± 1 mm Va 7.0 ± 1.0%	T = 50 ± 1.0°C (under water)	N_{max} = 20,000 passes d_{max} = 12.5 mm p_w = 705 ± 4.5 N f = 52 passes/minute	$d_{20,000}$ $N_{12.5}$ SIP
Ideal Rutting Tolerance: IDEAL-RT	D = 150 mm H = 62 ± 1 mm Va = 7.0 ± 0.5%	T = 50 ± 1.0°C	LLD = 50 ± 2 mmm/sec	τ_f RT_{index}
Cracking				
Ideal Cracking Tolerance: IDEAL-CT	D = 150 mm H 62 ± 1 mm Va = 7.0 ± 0.5%	T = 25 ± 0.5°C	LLD = 50 ± 2 mm/min	CT_{index}
Semi-Circular Bending-Flexibility Index: SCB-FI	D = 150 mm H = 50 ± 1 mm a = 15 ± 1 mm b = 1.5 ± 0.5 mm Va = 7.0 ± 0.5%	T = 25 ± 0.5°C	LLD = 50 ± 2 mm/min	FI
Overlay Tester: OT	D = 150 mm H = 38.1 ± 0.5 mm Va = 7.0 ± 0.5%	T = 25 ± 0.5°C	N = 1000 cycles f = 1 cycle/sec (1 Hz) Δ = 0.6 mm	$N_{f,93\%}$ G_c b
Notes:	D = diameter; H = specimen height; Va = air void content; a = notch depth; b = notch width; T = temperature; N_{max} = maximum number of cycles or passes; d_{max} = maximum allowed rut depth; p_w = wheel load; p_h = hose pressure; f = frequency; LLD = load-line displacement rate; Δ = displacement amplitude; $d_{8,000}$ = APA rut depth at 8,000 cycles; $d_{20,000}$ = HWT rut depth at 20,000 cycles; SIP = stripping inflection point; τ_f = shear strength; RT_{index} = rutting tolerance index; CT_{index} = cracking tolerance index; FI = flexibility index; $N_{f,93\%}$ = cycles at 93% reduction of peak load; G_c = critical failure energy, b = crack progression rate			

3.3.4 Asphalt Mixture Rutting Tests

Each mixture (PMA, two AMA, and two FMA mixtures) was subjected to the APA, the HWT and the IDEAL-RT tests as detailed in this section.

3.3.4.1 APA Tests

3.3.4.1.1 Description

For these tests, four SGC specimens, each with a diameter of 150.0 mm (5.91 in.), a target compaction height of 75.0 ± 1.0 mm (2.95 ± 0.04 in.) and a target air void content of 7.0 ± 0.5 percent (93.0 ± 0.5%Gmm), were fabricated for each mixture using AASHTO T 312 (AASHTO, 2015d). The SGC specimens were then subjected to the Gmb tests using FM 1-T 166 (FDOT, 2016b).

The specimens that met the target air void content were mounted in two separate high-density polyethylene (HDPE) molds, each containing slots or holes for two SGC specimens, and conditioned at 64.0 ± 0.5°C (147.2 ± 0.9°F) for a minimum of 6.0 hours.

The specimens were then subjected to a steel wheel load of 445 ± 22.0 N (100.0 ± 4.9 lbf) and a rubber hose pressure of 690.0 ± 35 kPa (100.1 ± 5.1 psi) at a rate of 60 cycles/minute for a minimum of 8,000 cycles (i.e., N_{max} = 8,000) and maximum 14.0-mm rut depth (i.e., d_{max} = 14.0 mm) following AASHTO T 340 (AASHTO, 2010). In this study, a rut depth of 14.0 mm instead of FDOT's maximum acceptable rut depth value of 4.5 mm was used to let the test run for a minimum of 8,000 cycles. The data acquisition system of the APA was used to automatically measure individual and average values of rut depths along each wheel path.

3.3.4.1.2 Test Results

Figure 3-14 presents the rut depth (d) versus wheel cycle (N) profiles obtained from the APA tests of four specimens of each mixture at 64.0°C. The figure shows that the APA tests stopped only after reaching 8,000 cycles and registering rut depths below the maximum acceptance value of 4.5 mm (based on correspondence with FDOT). Therefore, all these five mixtures passed the APA tests. The relatively low values of rut depth could be attributed to the use of a higher grade (i.e., PG 76-22) binder in each mixture.

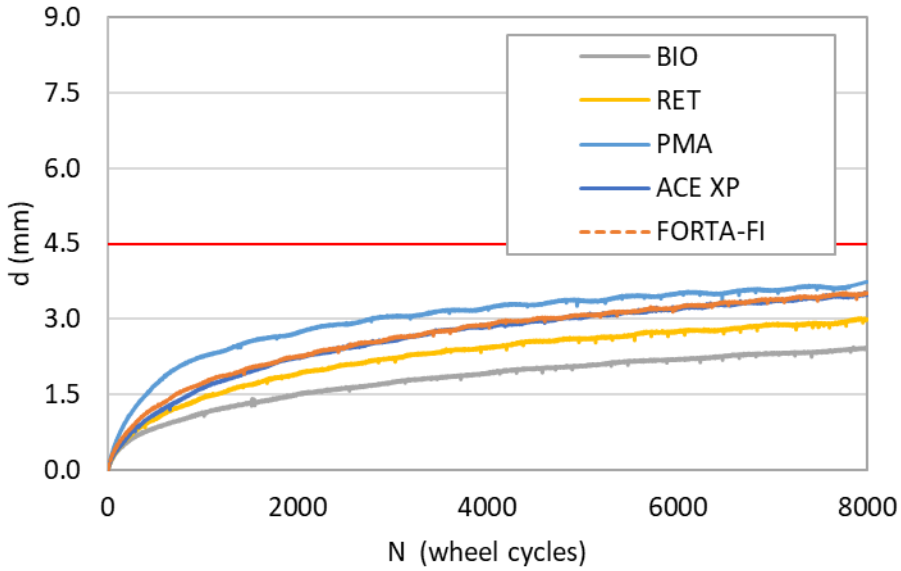


Figure 3-14. APA Tests: *d* versus *N* at 60 cycles/min at 64.0°C

Figure 3-15 presents the average and the standard deviation values of rut depths obtained from the APA tests of each mixture at 8,000 cycles and 64.0°C (i.e., $d_{8,000}$). The figure shows that the BIO mixture registered the lowest and the RET mixture registered the next lowest value of rut depth at 8,000 cycles compared to the PMA mixture (i.e., 2.41 mm and 2.97 mm respectively versus 3.63 mm). This was true even when the ACE XP mixture (with a rut depth of 3.50 mm) and the FORTA-FI mixture (with a rut depth of 3.52 mm) were also considered. These results suggest that the BIO mixture would perform the best while the RET mixture would perform next best compared to the PMA mixture. The best rutting resistance in the BIO mixture could be because of its highest rutting resistance parameter (therefore, true PGH value) and due to the polymeric network of SB/SBS copolymers still in PG 76-22 (BIO) binder [i.e., the polymeric network carried over from HP binder]. No tests were done to validate this hypothesis though.

Figure 3-15 also shows that the control, the ACE XP and the FORTA-FI mixtures had APA rut depths of 3.63 mm, 3.50 mm and 3.52 mm, respectively. These values suggest that the ACE XP and the FORTA-FI mixtures would perform equivalent to each other and only slightly superior to the PMA mixture in rutting.

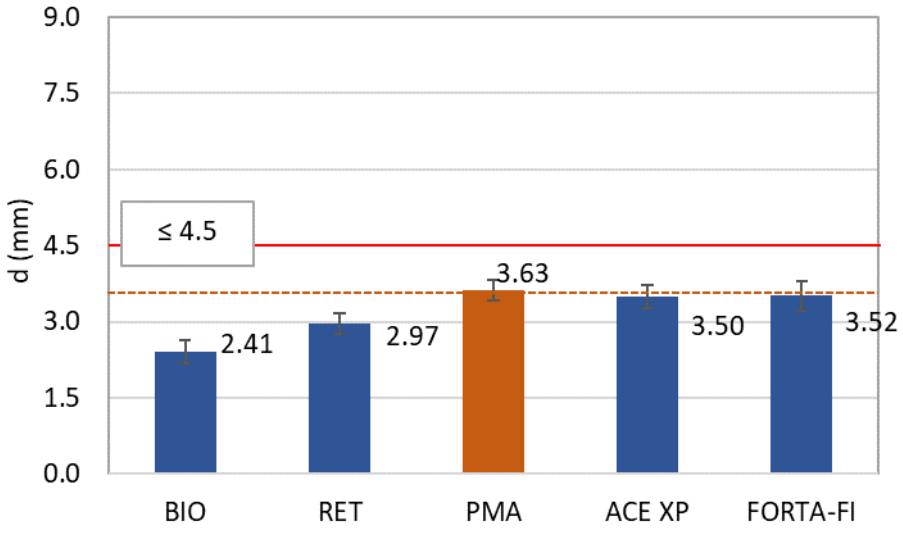


Figure 3-15. APA Tests: *d* at 8,000 cycles and 64.0°C

To summarize, the APA test results suggest that both AMA mixtures (the BIO and the RET mixtures) would perform generally better than the FMA mixtures (ACE XP and the FORTA-FI mixtures), and each of these mixtures would perform at least equivalent to the PMA mixture. If the rut depth of the PMA mixture (i.e., 3.63 mm in this case) was considered the maximum allowable rut depth, both AAB and AAM additives would be approved.

3.3.4.2 HWT Tests

3.3.4.2.1 Description

For these tests, SGC specimens, each with a diameter of 150.0 mm (5.91 in.) and a target compaction height of 62.0 ± 1.0 mm (2.44 ± 0.04 in.), were fabricated for each mixture using AASHTO T 312 (AASHTO, 2015d). Each SGC specimen was then cut along secant lines (or chords) perpendicular to compaction direction so that two specimens could be mounted together in the HDPE molds without any space between the cut edges. These specimens were then subjected to Gmb tests using FM 1-T 166 (FDOT, 2016b).

Four specimens that met the target air void content of 7.0 ± 1.0 percent (93.0 ± 1.0%Gmm) were selected for HWT tests and mounted on two HDPE molds, each containing two test specimens of the same mixture. Both molds were then fully submerged in water and

conditioned at 50.0 ± 0.5°C (122.2 ± 0.9°F) for 45.0 minutes after the water had reached this temperature.

The specimens were then subjected to wheel loads of 705.0 ± 4.5 N (158.5 ± 1.0 lb) at the rate of 52 passes/minute for a minimum of 20,000 passes (i.e., N_{max} = 20,000) or maximum 12.5-mm rut depth (i.e., d_{max} = 12.5) by following AASHTO T 324 (AASHTO, 2017d). The data acquisition system of the HWT instrument was used to measure the average values of rut depths along the left and right wheel paths automatically. Many state DOTs in the United States use a maximum rut depth of 12.5 mm after 20,000 passes as the acceptance criterion for mixtures with PG 76-22. Besides rut depth, it was originally planned to determine the values of stripping inflection points, SIP, from these tests; however, none of the tests exhibited any such inflection point.

3.3.4.2.2 Test Results

Figure 3-16 presents the rut depth (d) versus wheel pass (N) profiles obtained from the HWT tests of four specimens of each mixture. The figure shows that the HWT tests stopped after reaching 20,000 passes, registering rut depths far below the maximum acceptance value of 12.5 mm. Therefore, all these five mixtures passed the HWT acceptance criterion. The figure also verifies the absence of stripping inflection points (i.e., SIP), which are required for evaluating moisture damage susceptibility. Relatively low rut depths and absence of SIPs could be possibly attributed to the use of a higher-grade binder (i.e., PG 76-22) in each mixture.

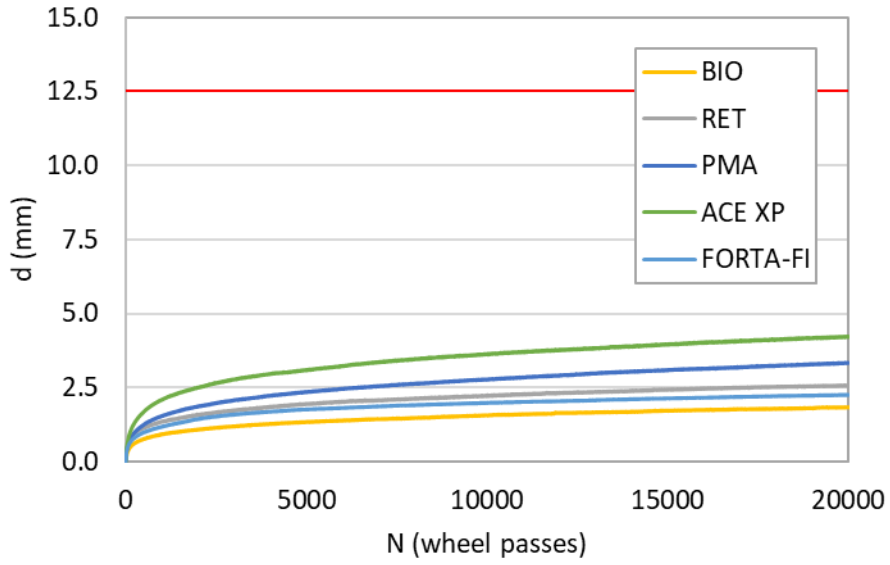


Figure 3-16. HWT Tests: *d* versus *N* at 52 passes/min at 50.0°C

Figure 3-17 presents the average and the standard deviation values of rut depths obtained from the HWT tests of each mixture at 20,000 passes and 50.0°C (i.e., $d_{20,000}$). The figure shows that the BIO mixture registered the lowest and the RET mixture registered the next lowest rut depths compared to the PMA mixture (i.e., 1.85 mm and 2.57 mm, respectively versus 3.34 mm), which means that the BIO mixture would perform the best followed by the RET mixture among these three (1 x PMA + 2 x AMA) mixtures in rutting, reiterating the same conclusion drawn based on the APA test results.

Figure 3-17 also shows that the FORTA-FI mixture registered a lower rut depth while the ACE XP mixture registered a higher rut depth compared to the rut depth value of the PMA mixture (i.e., 2.26 mm and 4.21 mm versus 3.34 mm), which suggests that the FORTA-FI mixture would perform the best among these three (1 x PMA + 2 x FMA) mixtures in rutting. However, the average rut depth values of the ACE XP and the PMA mixture differ from each other by less than 1.0 mm, which suggests that the ACE XP mixture might not be that much inferior compared to other mixtures.

Figure 3-17 also shows that, among the five mixtures, the ACE XP mixture had the highest rut depth and would therefore perform the worst while the BIO mixture had the lowest rut depth and therefore would perform the best ($d_{20,000}$= 1.85 mm).

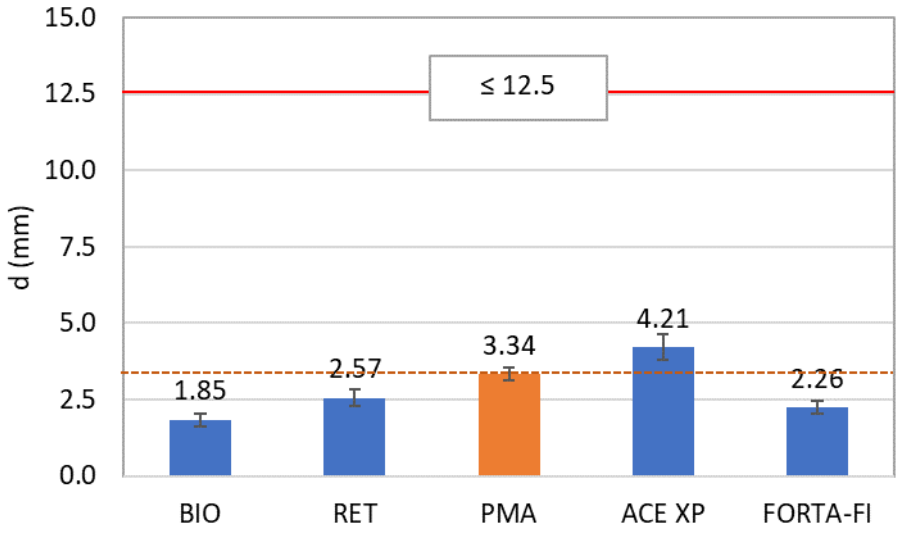


Figure 3-17. HWT Tests: *d* at 20,000 passes and 50.0°C

To summarize, the AMA and the FMA mixtures showed at least equivalent rutting resistance to the PMA mixture in all but one case. The absence of SIP in each d versus N curve also suggested that the HWT test could not reveal anything about stripping potential of the mixtures regardless of modification techniques, most likely because of the use of PG 76-22 binder in them. If the rut depth of the PMA mixture (i.e., 3.34 mm in this case) was considered the maximum allowable value, both AAB additives (i.e., RET and BIO) and only one AAM additive (i.e., FORTA-FI) would be approved.

3.3.4.3 IDEAL-RT Tests

3.3.4.3.1 Description

For these tests, a minimum of three SGC specimens, each with a diameter of 150.0 mm (5.91 in.), a target compaction height of 62.0 ± 1.0 mm (2.44 ± 0.04 in.) and a target air void content of 7.0 ± 0.5 percent (93.0 ± 0.5%Gmm), were fabricated in accordance with AASHTO T 312 (AASHTO, 2015d). The SGC specimens were then subjected to Gmb tests using FM 1-T 166 (FDOT, 2016b).

Each SGC specimen that met the target air void content was conditioned in an environmental chamber at a temperature equivalent to the HWT test temperature i.e., 50 ± 0.5°C (122.2 ± 0.9°F) for 2.5 hours ± 5.0 min before testing by following ASTM Work Item No. 71466 (ASTM, 2020). Within 2.0 minutes of removing the specimen from the

environmental chamber, each test specimen was tested under a shear fixture (see Figure 3-18). The loading strip at the top of the shear fixture had a width of 19.05 ± 0.3 mm (0.75 in ± 0.01) and a length as equal to or greater than specimen thickness (i.e., 62.0 ± 1.0 mm or 2.44 ± 0.04 in.).

The specimens were finally subjected to a shearing load at the LLD rate of 50.0 ± 2.0 mm/min (1.99 ± 0.08 in./min) at a temperature equivalent to the HWT test temperature i.e., 50.0 ± 0.5°C (122.2 ± 0.9°F) until the load dropped below 20% the peak load. The load (*P*) versus LLD data recorded during each test was then used to determine the shear strength (τ_f) and the rutting tolerance index (RT_{index}) (Zhou et al., 2020):

$$\tau_f = 0.356 \times p$$

$$RT_{Index} = 66.18 \times \frac{\tau_f}{1.0\ MP_a}$$

where,

$$p = \frac{P_{peak}}{A}$$

$$A = b \times h$$

where,

τ_f = shear strength (MPa)

RT_{index} = rutting tolerance index

p = peak shear stress (MPa)

P_{peak} = peak load (kN)

A = area of loading strip (mm^2)

b = width of upper loading strip (= 19.01 mm) (see Figure 3-18).

h = length of loading strip = specimen height (mm) (see Figure 3-18).

The larger the RT_{index}, the better the rutting resistance. Recently, Zhou et al. (2020) recommended minimum RT_{index} = 75.0 (equivalent τ_f = 1.13 MPa) for PG 76-22 asphalt mixtures.

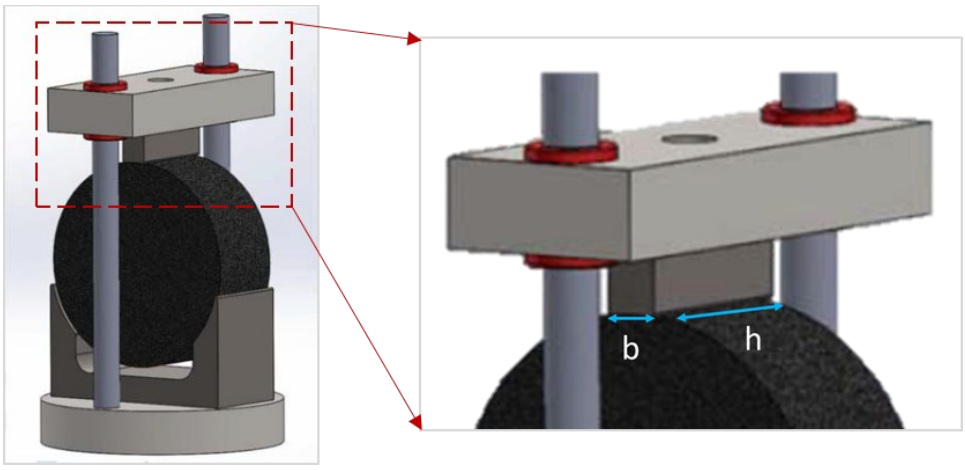
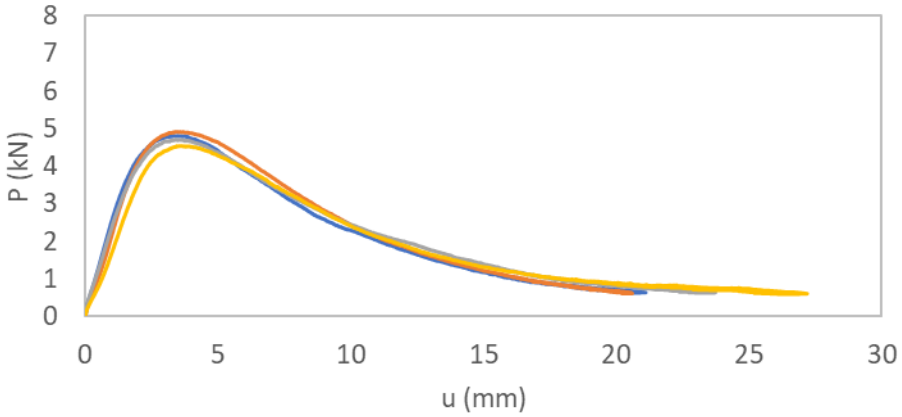


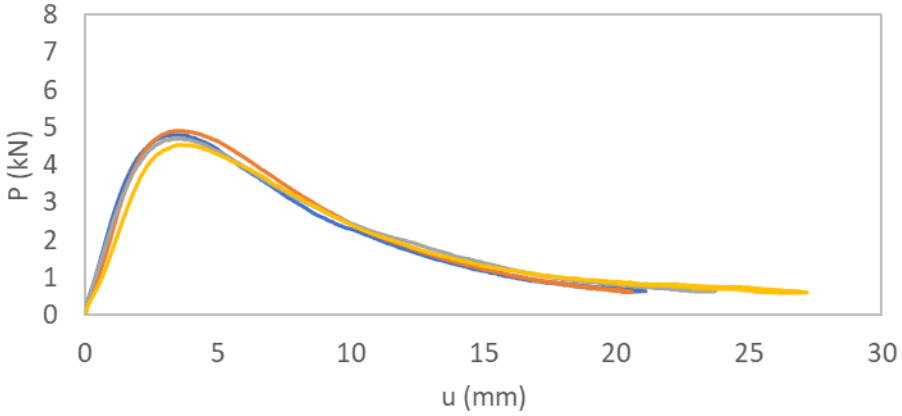
Figure 3-18. IDEAL-RT Tests: Shear Fixture

3.3.4.3.2 Test Results

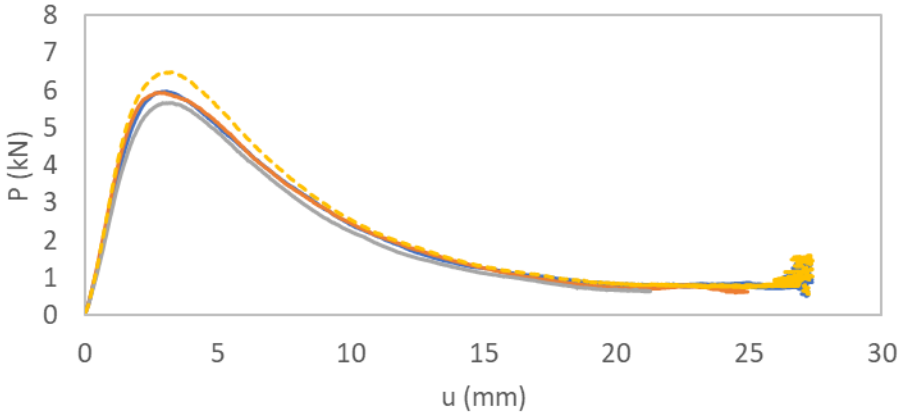
Figure 3-19 presents the load versus LLD (i.e., P versus u) profiles obtained from the IDEAL-RT tests at 50.0°C. The figure shows that rutting tests were quite repeatable, and each of these tests took less than a minute to complete (divide max. 30.0 mm by 50.0 mm/minute).



(a)

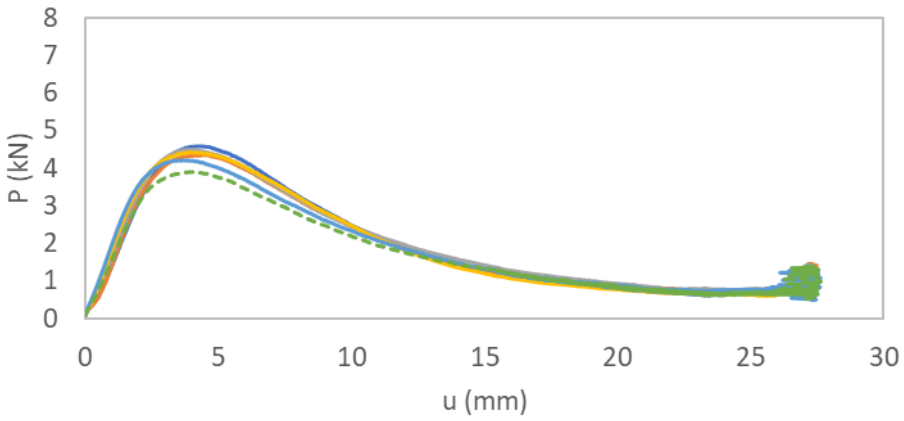


(b)

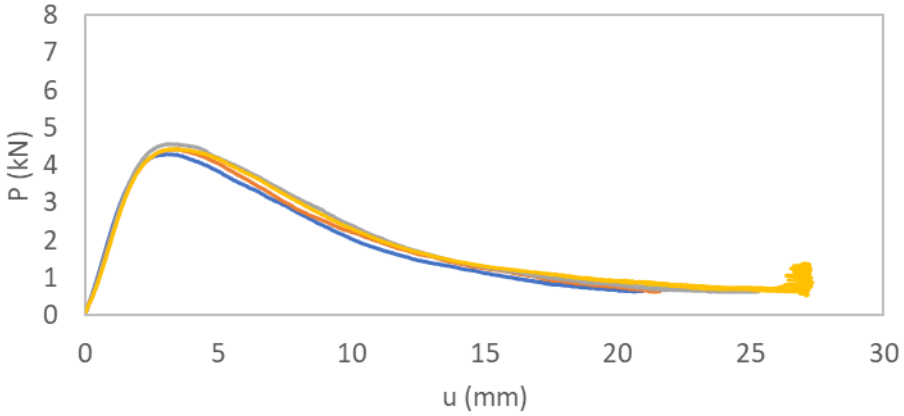


(c)

Figure 3-19. IDEAL-RT Tests: *P* versus *u* at 50.0 mm/min at 50.0°C: (a) PMA; (b) BIO; (c) RET; (d) ACE XP; (e) FORTA-FI



(d)



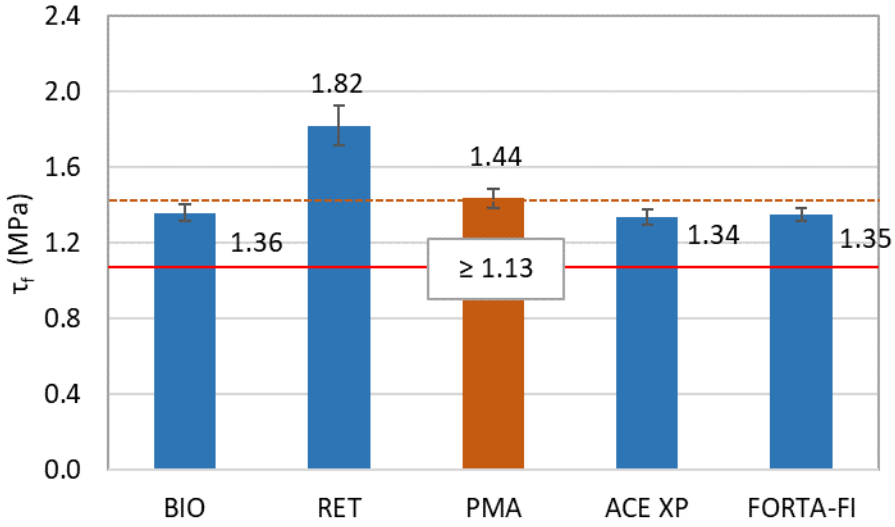
(e)

Figure 3-19. IDEAL-RT Tests: *P* versus *u* at 50.0 mm/min at 50.0°C: (a) PMA; (b) BIO; (c) RET; (d) ACE XP; (e) FORTA-FI, Continued

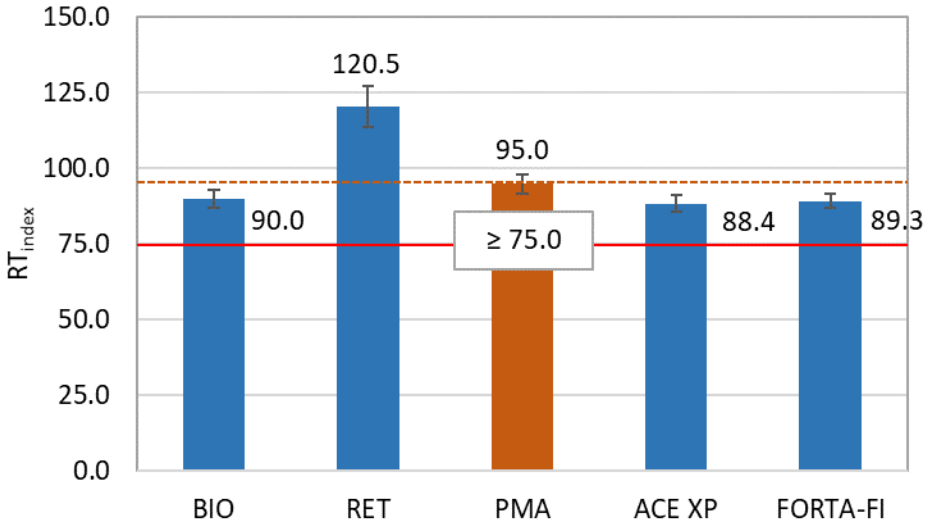
Figure 3-20(a) and Figure 3-20(b) present the average and the standard deviation values of the τ_f and the RT_{index} values obtained from the IDEAL-RT tests of a minimum of four specimens from each mixture (four control, BIO, RET, and FORTA-FI and five ACE XP mixture specimens) at 50.0°C. The figures show that all five mixtures had higher τ_f and RT_{index} values than the minimum acceptable criteria of 1.13 and 75.0, respectively, for the mixtures with PG 76-22 binder. Thus, all five mixtures had very good rutting resistance, which is consistent with the findings from both APA and HWT tests.

Figure 3-20(a) and Figure 3-20(b) also show that each of the BIO, the ACE XP and the FORTA-FI mixtures had a relatively lower RT_{index} value than the PMA mixture, while the

RET mixture had the highest RT_{index} value compared to the PMA mixture. In terms of RT_{index} value, the PMA mixture ranked better than the ACE XP, the FORTA-FI and the BIO mixtures, while the RET mixture ranked better than the PMA mixture. These rankings based on RT_{index} values are different from those based on the APA and the HWT rut depth values, although all three tests show that all five mixtures have very good rutting resistance. One of the main reasons for this discrepancy may be because of the use of repeated loading in the APA and the HWT tests versus the use of monotonic loading in the IDEAL-RT tests. Further validation with field data is needed for the IDEAL-RT test.



(a)



(b)

Figure 3-20. IDEAL-RT Tests: (a) τ_f at 50.0°C; (b) RT_{index} at 50.0°C

3.3.5 Asphalt Mixture Cracking Tests

Each mixture (PMA, two AMA, and two FMA mixtures) was subjected to the IDEAL-CT, the SCB-FI and the OT tests as detailed in this section.

3.3.5.1 IDEAL-CT Tests

3.3.5.1.1 Description

For these tests, a minimum of four SGC specimens, each with a diameter of 150.0 mm (5.91 in.), a target compaction height of 62.0 ± 1.0 mm (2.44 ± 0.04 in.) and a target air void content of 7.0 ± 0.5 percent (93.0 ± 0.5%Gmm), were fabricated for each mixture using AASHTO T 312 (AASHTO, 2015d). The SGC specimens were then subjected to Gmb tests using FM 1-T 166 (FDOT, 2016b).

Each SGC specimen that met the target air void content was conditioned in an environmental chamber at 25.0 ± 0.5°C (77.0 ± 0.9°F) for 2.5 hours ± 5.0 min before testing. Within 2.0 minutes of removing the specimen from the environmental chamber, each SGC test specimen was secured vertically on two loading strips, one at the top and one at the bottom. Each loading strip had a width of 19.05 ± 0.3 mm (0.75 ± 0.01 in.) and a length equivalent to the test specimen thickness (i.e., 62.0 ± 1.0 mm or 2.44 ± 0.04 in. in length).

The test specimens were finally subjected to IDT tests at the LLD rate of 50.0 ± 2.0 mm/min (1.99 ± 0.08 in/min) at 25.0 ± 0.5°C (77.0 ± 0.9°F) until the load dropped below 0.1 kN (22.5 lb) by following ASTM D8225 (ASTM, 2019). The P versus u data recorded during each test was then used to determine the cracking tolerance (CT) index of the test material (ASTM, 2019) [see Figure 3-21]:

$$CT_{index} = \left(\frac{u_{75}}{D}\right) \times \left(\frac{G_f}{|m_{75}|}\right) \times 10^6$$

where,

$$G_f = \left(\frac{W_f}{A}\right) \times 10^6$$

$$W_f = \int_0^{u_{final}} P(u).du$$

$$A = D \times t$$

where,

CT_{index} = cracking tolerance index

G_f = fracture energy = energy required to create a unit surface area of a crack (J/m^2)

m_{75} = a "modulus" parameter = the slope at 75% the peak load after the peak (N/mm)

u_{75}/D = a "strain tolerance" parameter

u_{75} = displacement recorded at 75% the peak load after the peak (mm)

W_f = work of fracture = work done to create a unit surface area of a crack (J or Nm)

P = load (kN)

u = LLD (mm)

u_{final} = displacement recorded at the 0.1 kN cut-off load (mm)

A = effective area (mm^2)

D = diameter of test specimen (mm)

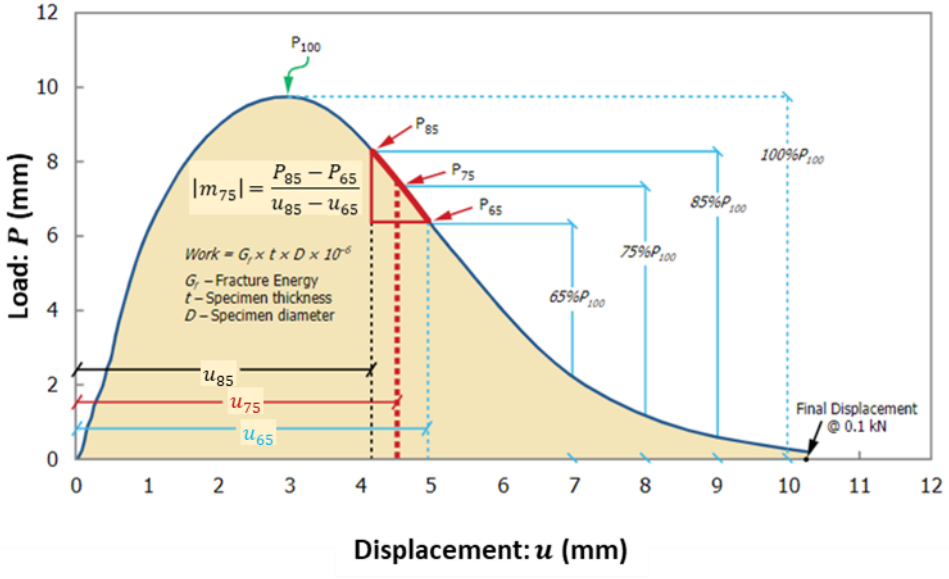
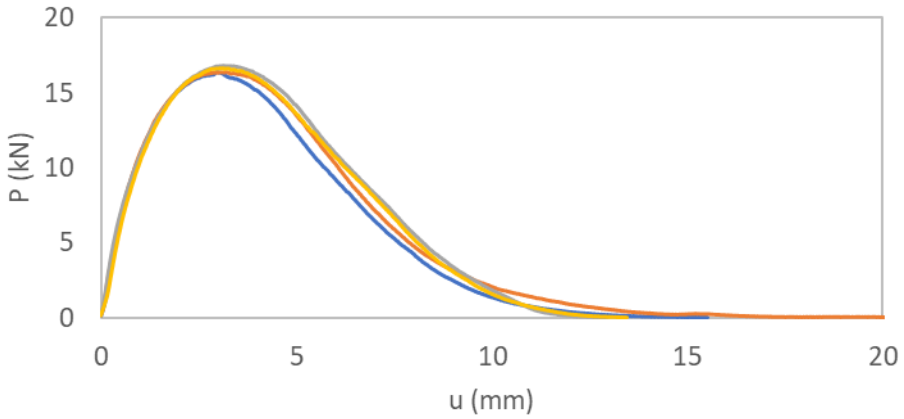


Figure 3-21. IDEAL-CT Tests: Parameter Definitions

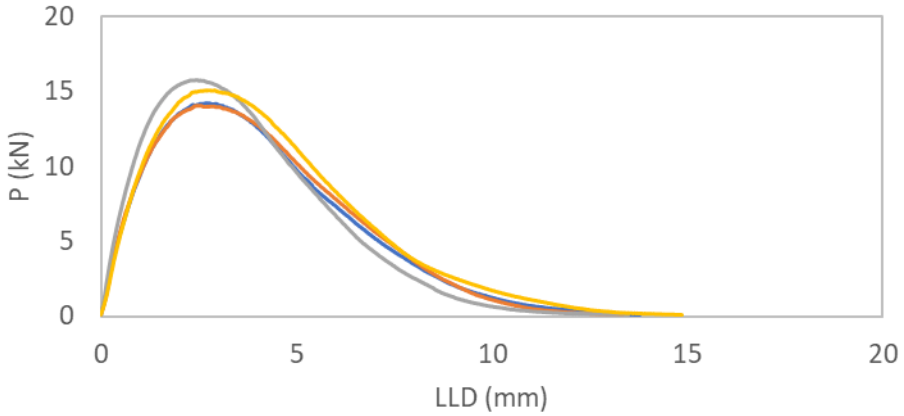
Theoretically, asphalt mixtures that are more brittle (i.e., more prone to cracking) have higher $|m_{75}|$ and lower u_{75} values and therefore lower CT_{index} values than their counterparts. Recently, Zhou et al. (2020) recommended minimum CT_{index}= 90.0 for the DGA mixtures.

3.3.5.1.2 Test Results

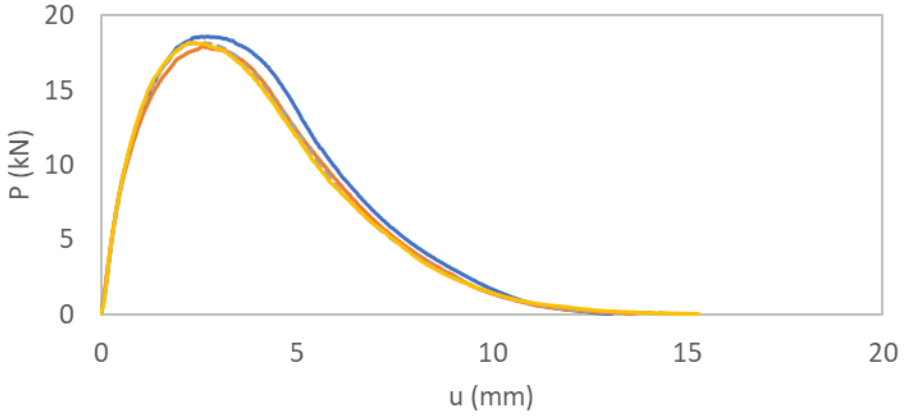
Figure 3-22 presents the load versus LLD (i.e., P versus u) profiles obtained from the IDEAL-CT tests of four specimens from each mixture. The figure shows that the IDEAL-CT tests were quite repeatable, and each test took less than a minute to complete (divide max. 20.0 mm by 50.0 mm/minute).



(a)

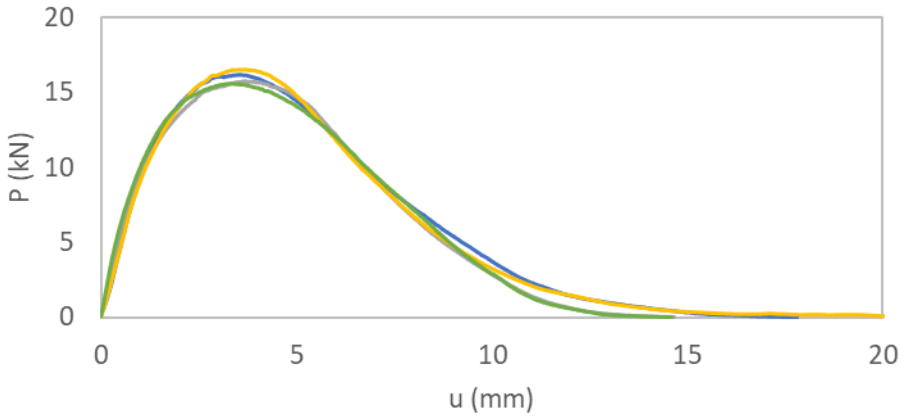


(b)

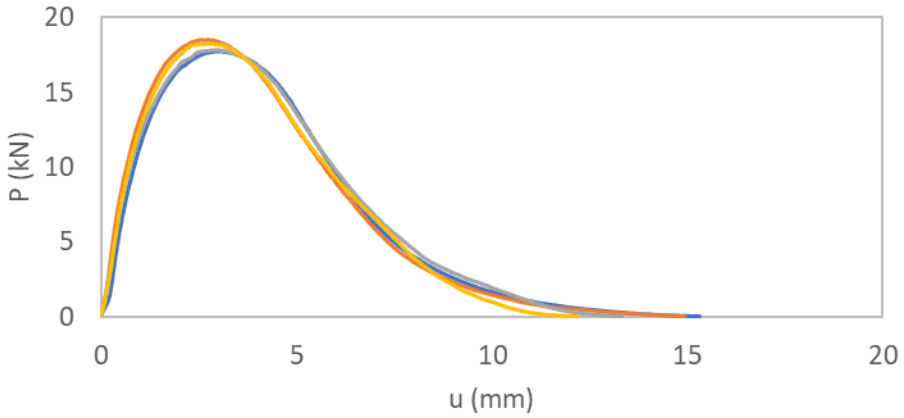


(c)

Figure 3-22. IDEAL-CT Tests: P versus *u* at 50.0 mm/min and 25.0°C: (a) PMA; (b) BIO; (c) RET; (d) ACE XP; (e) FORTA-F



(d)



(e)

Figure 3-22. IDEAL-CT Tests: P versus *u* at 50.0 mm/min and 25.0°C: (a) PMA; (b) BIO; (c) RET; (d) ACE XP; (e) FORTA-F, Continued

Figure 3-23 presents the average and the standard deviation values of CT_{index} obtained from a minimum of four specimens from each mixture. The figure shows that the CT_{index} values of all five mixtures were larger than the minimum acceptance value of 90.0, although the use of alternative modifiers or additives reduced cracking resistance in some cases. Furthermore, the figure shows that the BIO and the RET mixtures had equivalent CT_{index} values, both of which were lower than the CT_{index} value of the PMA mixture. Based on this index, the BIO and the RET mixtures would perform equivalent to each other but worse than the PMA mixture. The fact that the BIO and the RET mixtures showed higher rutting resistance in the APA and the HWT tests (i.e., lower rut depth values) but lower cracking resistance in the IDEAL-CT tests (i.e., lower CT_{index} values) compared to the

PMA mixture aligns with the observation that has been made over the years, i.e., an inverse ranking of mixtures in terms of rutting and cracking (i.e., the higher rutting resistance, the lower the cracking resistance).

Figure 3-23 also shows that, compared to the PMA mixture, the ACE XP mixture had a higher CT_{index} value but the FORTA-FI mixture had a lower CT_{index} value, which suggests that the ACE XP mixture would perform better than the PMA mixture and significantly better than the FORTA-FI mixture. This observation is consistent with the HWT test results, wherein the use of the ACE XP fibers led to a higher rut depth value and the use of FORTA-FI fibers led to a lower rut depth value compared to the PMA mixture.

Figure 3-23 also shows that the BIO, the RET and the FORTA-FI mixtures would perform worse than the PMA mixture because their CT_{index} were smaller than 123.1, while the ACE XP mixture would perform better than the PMA mixture because its CT_{index} was greater than 123.1. The figure also shows that, when standard deviations are considered, BIO, the RET and the FORTA-FI mixtures would perform equivalent to each other.

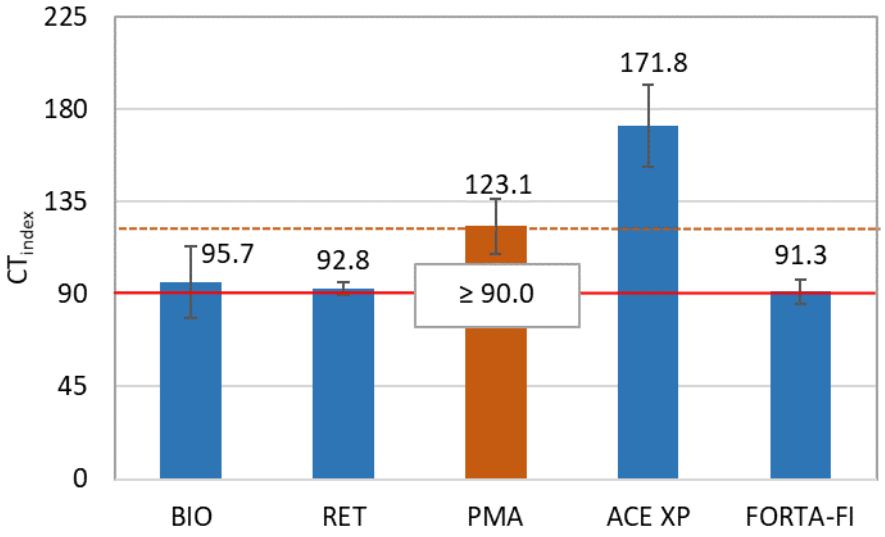


Figure 3-23. IDEAL-CT Tests: CT_{index} at 25.0°C

3.3.5.2 SCB-FI Tests

3.3.5.2.1 Description

For these tests, a minimum of four SGC specimens, each with a diameter of 150.0 mm (5.91 in.) and a target compaction height of 160.0 ± 1.0 mm (6.30 ± 0.04 in.), were fabricated

for each mixture using AASHTO T 312 (AASHTO, 2015d). Each SGC specimen was cut thrice perpendicular to the compaction direction (top, bottom and center) to obtain two identical cylindrical discs, each measuring 50.0 ± 0.5 mm (1.97 ± 0.02 in.) in thickness, from the center of the compacted specimen. The discs were then subjected to Gmb tests using FM 1-T 166 (FDOT, 2016b). Each cylindrical disc that met the target air void content of 7.0 ± 1.0 percent (93 ± 1.0%Gmm) was then cut into two identical semi-circular halves. Finally, a notch with a depth of 15.0 ± 1.0 mm (0.60 ± 0.04 in) and a width of 1.5 ± 0.5 mm (0.06 ±0.02 in) was sawn at the center of the flat edge (i.e., opposite to the curved edge) of each SCB specimen, resulting in four identical single-notched SCB test specimens from each SGC sample. Each single-notched semi-circular specimen was then conditioned in an environmental chamber at 25.0 ± 0.5°C (77.0 ± 0.9°F) for 2.5 hours ± 5.0 min before testing.

Within 2.0 minutes of removing the specimen from the environmental chamber, each specimen was positioned between two rollers as a simply supported beam with the notched side facing downwards. The specimen was finally subjected to a contact load of 0.1 ± 0.01 kN (22.5 ± 2.25 lb) at the rate of 0.05 kN/sec (11. 24 lb/sec) followed by a (damage-inducing) vertical load at its center at the LLD rate of 50 ± 1 mm/min (1.99 ± 0.08 in/min) until the load drops below 0.1 kN (22.5 lb). The load and LLD data recorded during each test was analyzed using software developed by the original developers of this test—the Illinois Center for Transportation researchers—to obtain the FI of the test material (see Figure 3-24):

$$FI = \frac{G_f}{|m|} \times k$$

where,

$$G_f = \frac{W_f}{A_{lig}} \times 10^6$$

$$W_f = \int_0^{u_{final}} P(u).du$$

$$A_{lig} = l_{lig} \times t$$

$$l_{lig} = \frac{D}{2} - a$$

where,

FI = flexibility index

G_f = fracture energy = energy required to create a unit surface area of a crack (J/m2)

m = post-peak slope measured at the first inflection point of load-LLD curve (KN/mm)

k = unit conversion factor = 0.01

W_f = work of fracture = work done to create a unit area of a crack (J or Nm)

P = load (kN)

u = LLD (mm)

u_{final} = LLD recorded at the 0.1 kN cut-off load (mm)

A_{lig} = area of ligament (mm^2)

l_{lig} = length of ligament (mm)

t = specimen thickness (mm)

D = diameter (mm)

a = notch length (mm)

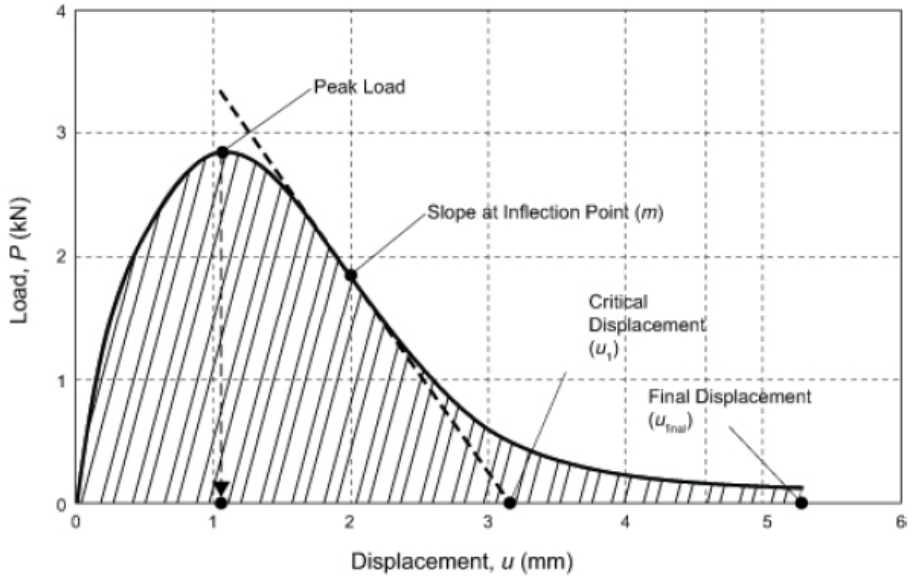


Figure 3-24. SCB-FI Tests: Parameter Definitions

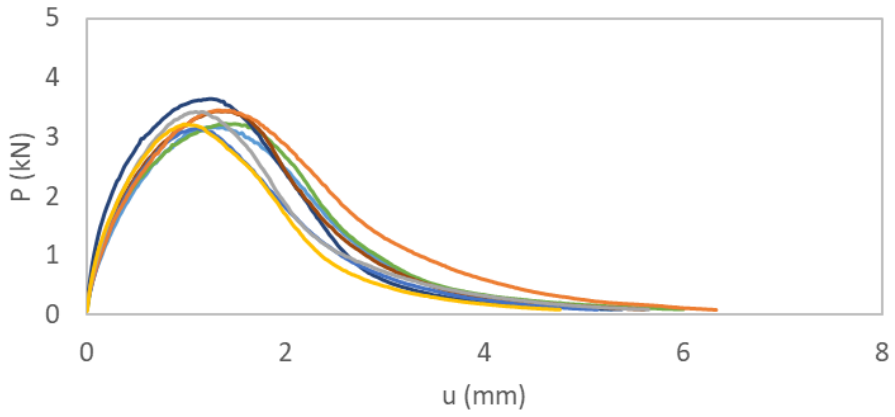
Several recent studies have suggested using the minimum FI value of 8.0 to 10.0 (Al-Qadi et al., 2017; Ozer et al., 2016) to distinguish asphalt mixtures with better cracking resistance from their counterparts. In general, higher FI means better cracking resistance and vice versa.

In this study, a minimum of one to a maximum of two SGC samples were fabricated for each mixture, resulting in a minimum of four to a maximum of eight SCB specimens. Only

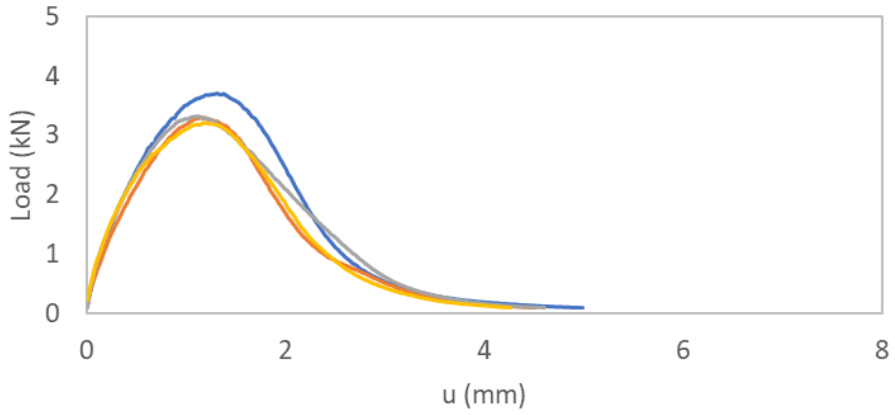
those specimens that met the requirements of specimen geometry (specimen thickness, notch depth, notch width) and volumetric properties (air void content) were used for testing and analysis.

3.3.5.2.2 Test Results

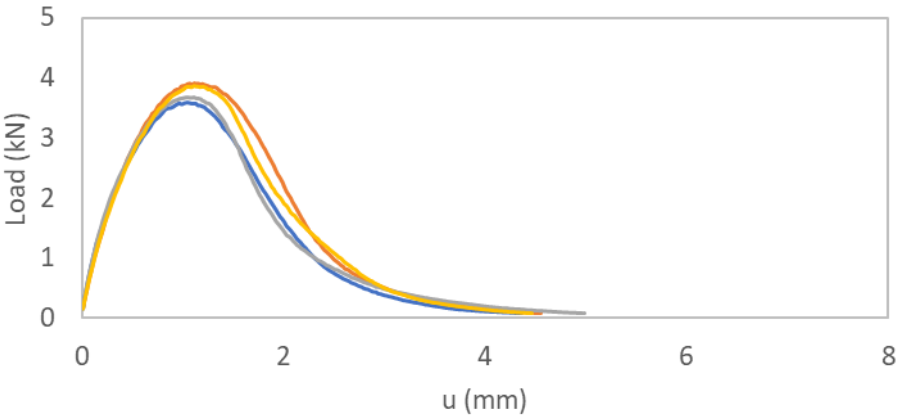
Figure 3-25 presents the load versus LLD profiles (i.e., P versus u) obtained from the SCB-FI tests at 25.0°C. The results also imply that the tests were completed within a few seconds (divide 8.0 mm by 50.0 mm/minute). More importantly, the figures show that there is much more variability in these tests than in the IDEAL-CT tests despite having the same loading rate and test temperature. The higher variation in the SCB-FI tests can be attributed to five different cutting and four different notching steps involved in fabricating four test specimens per SGC sample.



(a)

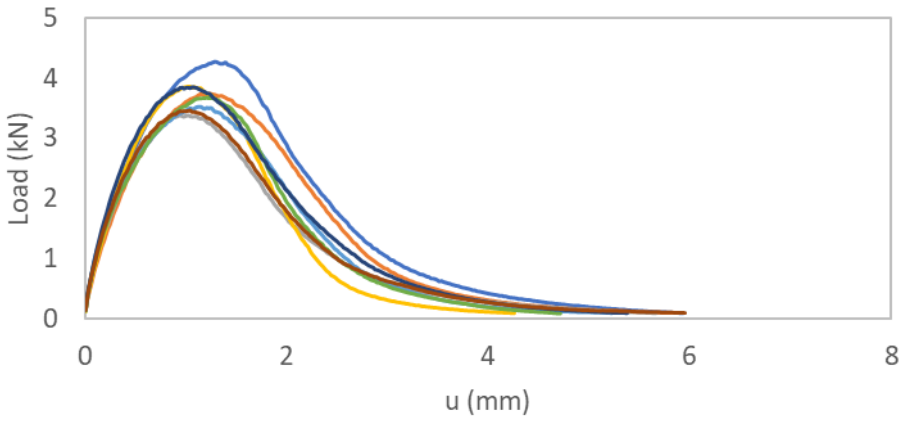


(b)

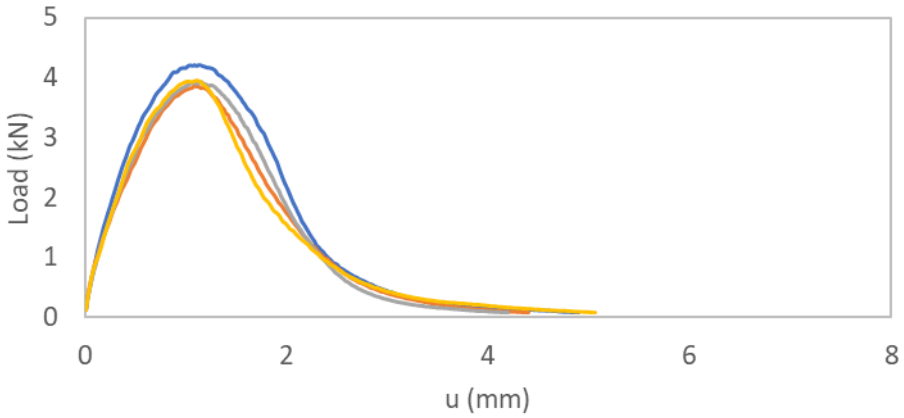


(c)

Figure 3-25. SCB-FI Tests: *P* versus *u* at 50.0 mm/min at 25.0°C: (a) PMA; (b) BIO; (c) RET; (d) ACE XP; (e) FORTA-FI



(d)



(e)

Figure 3-25. SCB-FI Tests: *P* versus *u* at 50.0 mm/min at 25.0°C: (a) PMA; (b) BIO; (c) RET; (d) ACE XP; (e) FORTA-FI, Continued

Figure 3-26 presents the average and the standard deviation values of FI obtained from the SCB-FI tests of a minimum of four specimens from each mixture (eight control, four BIO and RET, seven ACE XP, and six FORTA-FI mixture specimens). The figure shows that the BIO and the RET mixtures had almost equivalent FI values compared to each other but lower FI values compared to the PMA mixture, which suggests that the BIO and the RET mixtures would perform equivalent to each other but not as good as the PMA mixture. This conclusion seconds the same conclusion drawn from the IDEAL-CT tests.

Figure 3-26 also shows that the ACE XP mixture had almost equivalent FI values to the PMA mixture, while the FORTA-FI mixture had slightly lower FI values compared to the

PMA mixture, meaning the ACE XP mixture would perform as good as the PMA mixture while the FORTA-FI mixture would perform only slightly inferior to the PMA mixture. These results show that modified mixtures, irrespective of the use of AAB or AAM additive, did not help improve cracking resistance drastically, most likely because of the use of the already stiff (PG 76-22) binder.

Based on average values and sample-to-sample variations shown in this figure, the PMA mixture would perform the best followed very closely by the ACE XP mixture, then by FORTA-FI and lastly by the BIO and the RET mixtures in terms of their cracking resistance.

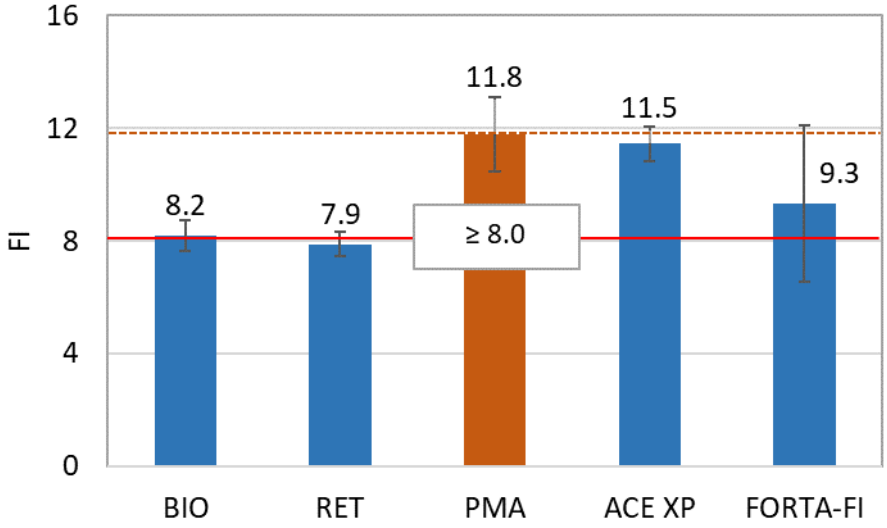


Figure 3-26. SCB-FI Tests: *FI* at 25.0°C

3.3.5.3 OT Tests

3.3.5.3.1 Description

For these tests, a minimum of four SGC samples, each with a diameter of 150.0 mm (5.91 in.) and a target compaction height of 62.0 ± 1.0 mm (2.44 ± 0.04 in.), were fabricated for each mixture using AASHTO T 312 (AASHTO, 2015d). Each SGC specimen was then cut perpendicular to the top surface (i.e., parallel to compaction direction) to obtain a disc that measured 76.0 ± 0.5 mm (3.0 ± 0.02 in.) in width. Each disc was then trimmed at the top and bottom surfaces (i.e., perpendicular to the compaction direction) to obtain a test specimen that measured 38.1 ± 0.5 mm (1.5 ± 0.02 in.) in thickness, resulting in one test

specimen per SGC sample. The specimens were then subjected to Gmb tests using FM 1-T 166 (FDOT, 2016b).

Each specimen that met the target air void content of 7.0 ± 1.0 percent (93.0 ± 1.0%Gmm) was then glued to two base plates using 16.0 grams of two-part epoxy (i.e., 8.0 grams of epoxy to glue each half of the specimen to one base plate); a gap of 4.2 mm (0.16 in.) was maintained between the two base plates using a spacer bar. A 2.27-kg (or a 5.0-lb) weight was placed on the top of the test assembly for at least 24.0 hours to cure the glue and to ensure good contact between the test specimen and base plates.

Each test assembly was then conditioned at 25.0 ± 0.5°C (77.0 ± 0.9°F) in an environmental chamber without any load on its top for 2.5 hours ± 5.0 min before testing. For testing, each assembly was positioned on the test instrument such that one base plate was fastened to a horizontally sliding block and the other base plate was fastened to a fixed block. The assembly was finally subjected to repeated direct tension loads with triangular waveforms at a constant displacement amplitude of 0.6 mm (0.025 in.) at a frequency of 0.1 Hz (i.e., a loading period of 10 seconds) and a temperature of 25.0 ± 0.5°C (77.0 ± 0.9°F) until the maximum load drops more than 93.0% or until the number of cycles reaches 1,000 in total. The load versus displacement (i.e., P versus u) data recorded during the first cycle (which typically had the maximum peak load) were analyzed to determine the value of critical fracture energy of the test material (see Figure 3-27):

$$G_c = \frac{W_c}{A_c}$$

$$A_c = b \times t$$

$$W_c = \int P(u).du$$

where,

G_c = critical fracture energy = energy required to create a unit surface area of a crack (kN/mm^2)

W_c = critical work of fracture (kN×mm)

A_c = cracked area (mm^2)

P = load (kN)

u = displacement (mm)

b = width of test specimen (mm)

t = thickness of test specimen = (mm)

Also, the peak load obtained from each cycle, until the number of cycles at 93% reduction, was fitted with a power function to determine the crack progression rate:

$$P_N = a \times N^b$$

where,

P_N = crack driving force (i.e., peak load at given cycle)

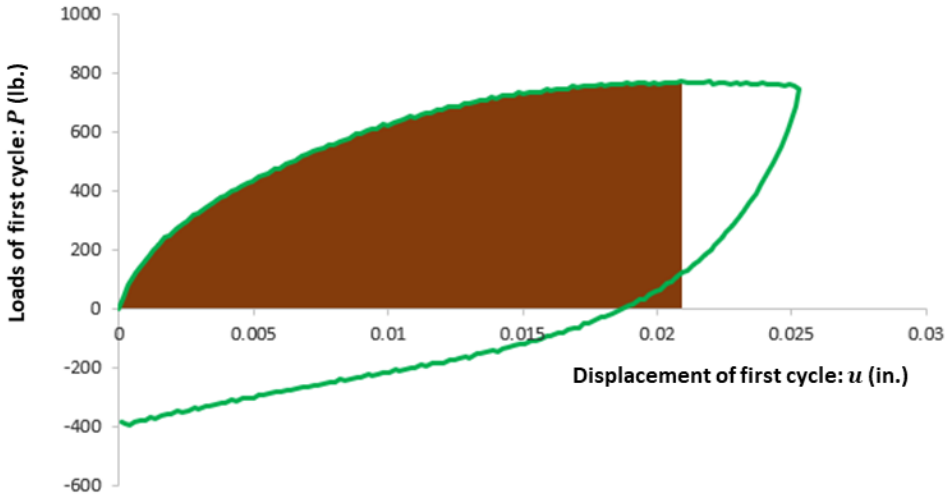
N = corresponding number of cycles

a = 1.0

b = crack progression rate

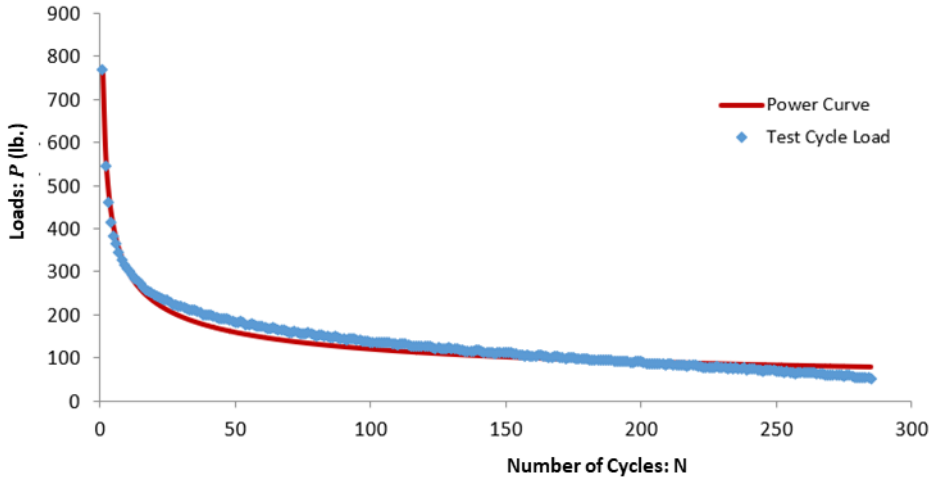
$N_{f,93\%}$ = Number of cycles at 93 percent reduction of peak load

Both TXDOT and NJDOT are using this test to evaluate the cracking resistance of asphalt mixtures and have some preliminary criteria. For example, NJDOT requires overlay samples produced with the PMA binder to not fail before 175 load cycles.



(a)

Figure 3-27. OT Tests: (a) *P* versus *u* at 25°C; (b) P_{peak} versus *N* at 25.0°C



(b)

Figure 3-27. OT Tests: (a) *P* versus *u* at 25°C; (b) P_{peak} versus *N* at 25.0°C, Continued

3.3.5.3.2 Test Results

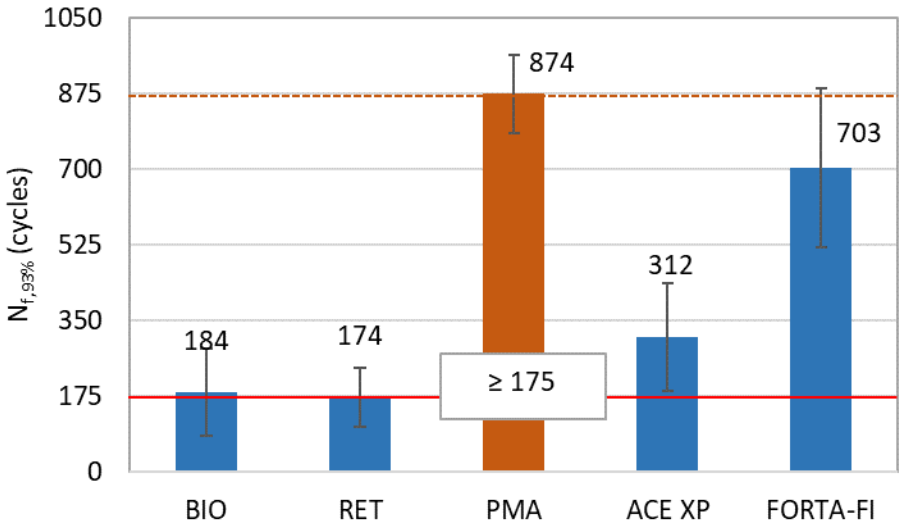
Figure 3-28(a) presents the average and the standard deviation values of $N_{f,93\%}$ obtained from the OT tests of a minimum of three specimens from each mixture (three control, four BIO and RET, and six ACE XP and FORTA-FI mixture specimens) at 25.0°C. The figures show that the PMA mixture had the highest $N_{f,93\%}$ values and the BIO and RET mixtures had the lowest $N_{f,93\%}$ values. All but one mixture (i.e., RET) met the minimum value of 175 cycles set by NJDOT when specimen-to-specimen variations were considered. Since the test method recommends this $N_{f,93\%}$ for informational purposes only, this parameter was not selected for in-depth mixture-to-mixture evaluation in this study.

Figure 3-28(b) presents the average and the standard deviation values of G_c obtained from the OT tests of the mixtures at 25.0°C. The figure shows that RET, the ACE XP, and the FORTA-FI mixtures had almost equivalent G_c values but the BIO mixture had lower G_c values compared to the PMA mixture, which suggests that the RET, the ACE XP, and the FORTA-FI mixtures would need almost equivalent amounts of energy to the PMA mixture, but the BIO mixture would need less energy to create one more unit area of fracture surface compared to the PMA mixture.

Figure 3-28(c) presents the average and the standard deviation values of b obtained from the OT tests of the mixtures at 25.0°C. The figure shows that the BIO and the RET mixtures had equivalent b values compared to each other but slightly higher b values

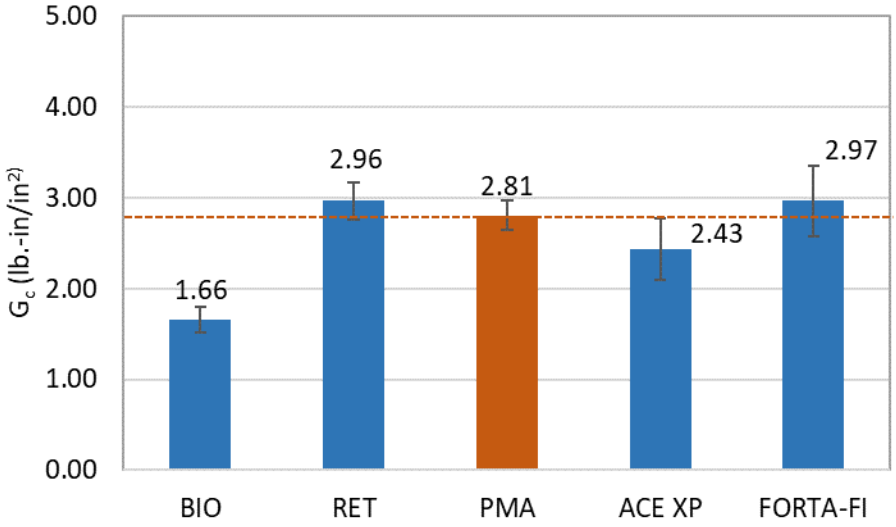
compared to the PMA mixture, which suggests that cracks would propagate much faster in the BIO and the RET mixtures than in the PMA mixture as expected (i.e., the higher the rutting resistance, the lower the cracking resistance). The figure also shows that the ACE XP and the FORTA-FI mixtures had almost equivalent b values compared to each other and the PMA mixture, which suggests that cracks would propagate much in the ACE XP and the FORTA-FI mixtures in the same rate as they would do in the PMA mixture.

Based on average values and sample-to-sample variations shown in these figures, the PMA mixture would perform the best followed very closely by the FORTA-FI and the ACE XP mixtures and afar by the BIO and the RET mixtures in terms of their cracking resistance.

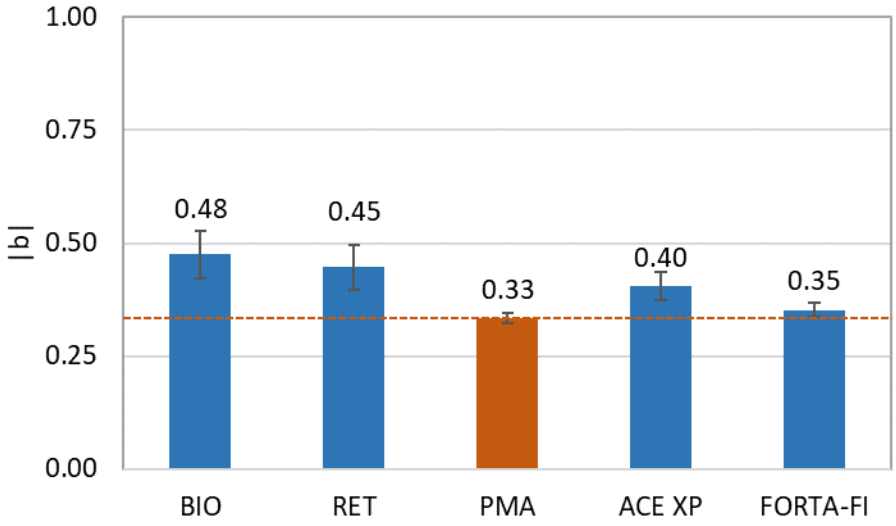


(a)

Figure 3-28. OT Tests: (a) $N_{f,93\%}$ at 25.0°C; (b) G_c at 25.0°C; (c) $|b|$ at 25.0°C



(b)



(c)

Figure 3-28. OT Tests: (a) *Nf*$_{93\%}$ at 25.0°C; (b) *Gc* at 25.0°C; (c) |*b*| at 25.0°C, Continued.

3.4 Summary

3.4.1 AAB Additive Evaluation

Figure 3-29(a) and Figure 3-30(a) present the normalized values of the rutting and cracking resistance parameters of PG 76-22 (RET) and PG 76-22 (BIO) binders with respect to the rutting and cracking resistance parameters of PG 76-22 (PMA) binder. Generally,

asphalt binders that are more resistant to rutting have relatively higher values of $G^*/Sin\delta$, PGH, and $\%Rec_{3.2}$ and relatively lower values of δ and $Jnr_{3.2}$. Similarly, asphalt binders that are more resistant to cracking have relatively higher values of m, ΔTc_{PAV20} (less negative value) ΔTc_{PAV40}, ΔTc_{PAV60}, $N_{f,2.5\%}$, $N_{f,5.0\%}$, $FREI$, $PAV_{GR=180\,kPa}$, and $PAV_{GR=600\,kPa}$ values and relatively lower values of S, PGL, and $G^*.Sin\delta$. Based on the normalized values of these parameters, the following observations were made:

- Compared to PG 76-22 (PMA) binder, PG 76-22 (RET) binder was equivalent in terms of two rutting resistance parameters, superior in terms of two cracking resistance parameters (i.e., N_f and $FREI$ parameters) but inferior in terms of all remaining (most) rutting and cracking resistance parameters.
- Compared to PG 76-22 (PMA) binder, PG 76-22 (BIO) binder was superior in terms of all rutting resistance parameters but inferior in terms of five cracking resistance parameters (ΔTc_{PAV20}, ΔTc_{PAV40}, ΔTc_{PAV60}, $PAV_{GR=180\,kPa}$, and $PAV_{GR=600\,kPa}$).

However, asphalt binder parameters alone do not guarantee whether the corresponding mixtures would perform equivalent to, better than or worse than the PMA mixture. Therefore, the rutting and cracking resistance parameters of the PMA and the AMA mixtures were also determined to better evaluate the performance of these mixtures.

Figure 3-29(b) and Figure 3-30(b) respectively present the normalized values of the rutting and cracking resistance parameters of the RET and the BIO mixtures (including the ACE XP (also the FORTA-FI mixtures) with respect to the rutting and cracking resistance parameters of the PMA mixture. Generally, asphalt mixtures that have better rutting resistance have relatively lower values of the APA and the HWT rut depths, but relatively higher values of RT_{index} that signifies better rutting resistance. Similarly, asphalt mixtures that have better cracking resistance have relatively higher values of CT_{index} and FI, but relatively lower values of $|b|$. Based on the absolute values of these parameters, the following observations were made:

- Compared to the rutting resistance of the PMA mixture, the RET mixture was superior in terms of each rutting test parameter while the BIO mixture was inferior in terms of the APA and the HWT test parameters (i.e., d_{APA} and d_{HWT}) but

superior in terms of the IDEAL-RT test parameter (i.e., RT_{index}), although all three mixtures had very good rutting resistance. The relatively superior rankings of the AMA mixtures in terms of their APA and HWT test parameters (i.e., d_{APA} and d_{HWT}) mirror the better ranking of corresponding AMA binders in terms of their high temperature PG and MSCR test parameters (i.e., $G^*/Sin\delta$, PGH, $\%Rec_{3.2}$ and $Jnr_{3.2}$). Since Section 916 of FDOT's current specification already includes these tests, parameters and criteria, the new protocols included only PG verification and reporting but no change in specification.

- Compared to the cracking resistance of the PMA mixture, both AMA mixtures were inferior in terms of each cracking resistance parameter (i.e., CT_{index}, FI and $|b|$). The relatively inferior ranking of the AMA mixtures (specially, the BIO mixture) in terms of their IDEAL-CT, SCB-FI and OT test parameters (i.e., CT_{index}, FI, and $|b|$ parameters) mirror the inferior rankings of corresponding AMA binders in terms of their low temperature and intermediate temperature cracking and aging test parameters (ΔTc_{PAV20}, ΔTc_{PAV60}, $PAV_{GR=180\,kPa}$, and $PAV_{GR=600\,kPa}$)—that is, asphalt mixture cracking resistance had better correlation with ΔTc_{PAV} and PAV_{GR} than their counterparts.

Among these parameters, ΔTc_{PAV20} is already included in Section 916 of FDOT's current specification and in other state DOT specifications. Similarly, the GR parameter is not included in Section 916 of FDOT's current specification or in other state DOT specifications. Because of the lack of well-verified criteria for GR parameters, it was decided not to include the GR test in the new protocol. In other words, no additional asphalt binder tests were included in the new protocol (see Chapter 6)

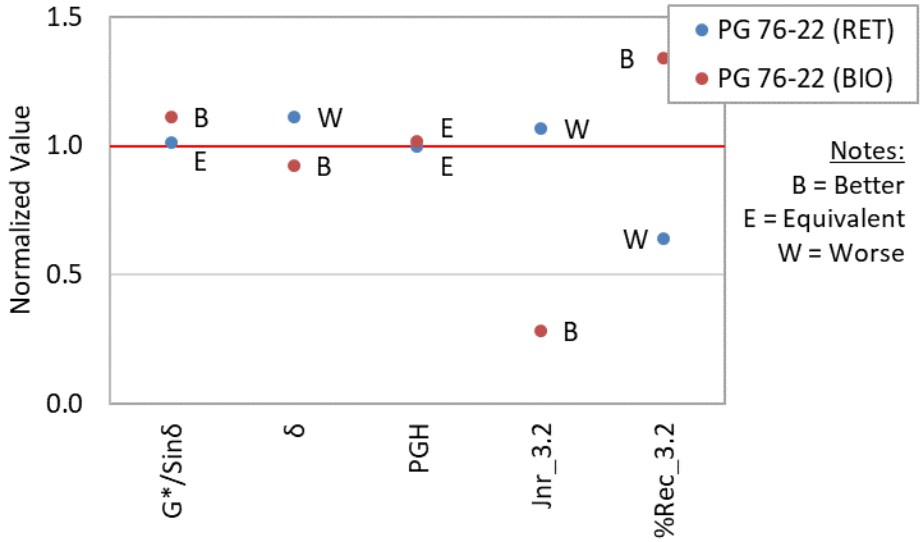
3.4.2 AAM Additive Evaluation

Based on the normalized values of rutting and cracking resistance parameters of the PMA and both FMA mixtures, the following observations were made:

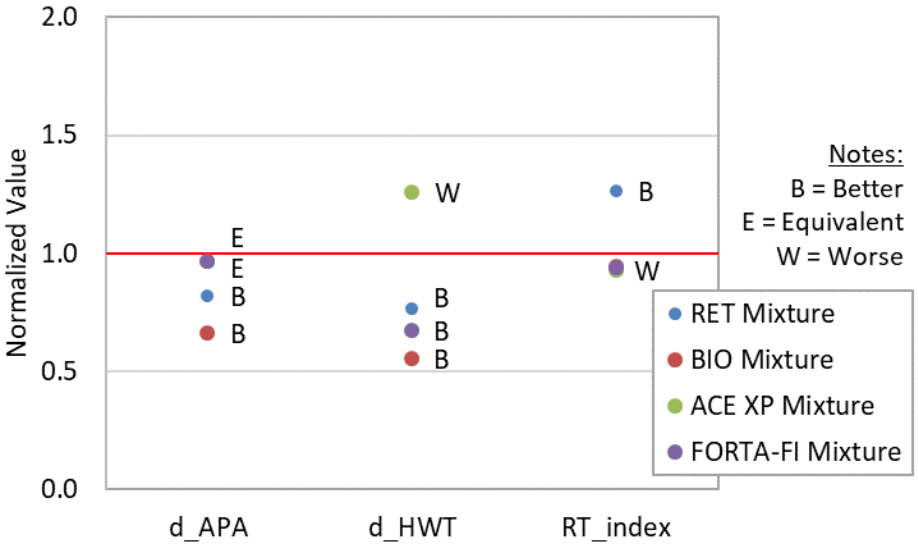
- Compared to the rutting resistance of the PMA mixture, the ACE XP mixture was superior in terms of the APA test parameter (i.e., d_{APA}) but inferior in terms of the HWT and the IDEAL-RT test parameters (i.e., d_{HWT} and RT_{index}) while the FORTA-FI mixture was superior in terms of the APA and the HWT test parameters

(i.e., d_{APA} and d_{HWT}) but slightly inferior in terms of the IDEAL-RT test parameter (i.e., RT_{index}).

- Compared to the cracking resistance of the PMA mixture, the ACE XP mixture was superior in terms of the IDEAL-CT test parameter (i.e., CT_{index}), equivalent in terms of the SCB-FI test parameter (i.e., FI), and slightly inferior in terms of the OT test parameter (i.e., $|b|$) while the FORTA-FI mixture was inferior in terms of the IDEAL-CT and the SCB-FI test parameters (i.e., CT_{index} and FI) but almost equivalent in terms of the OT test parameter (i.e., $|b|$).

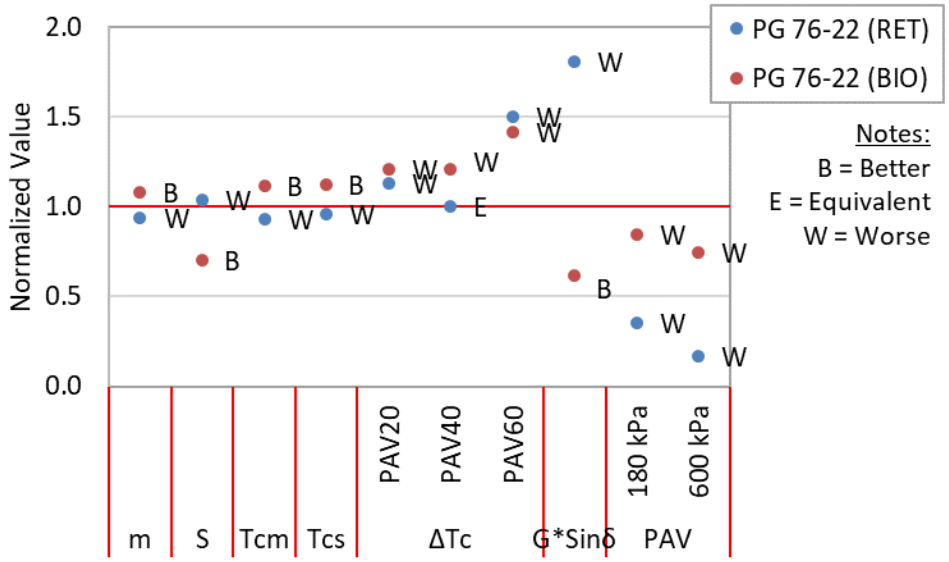


(a)

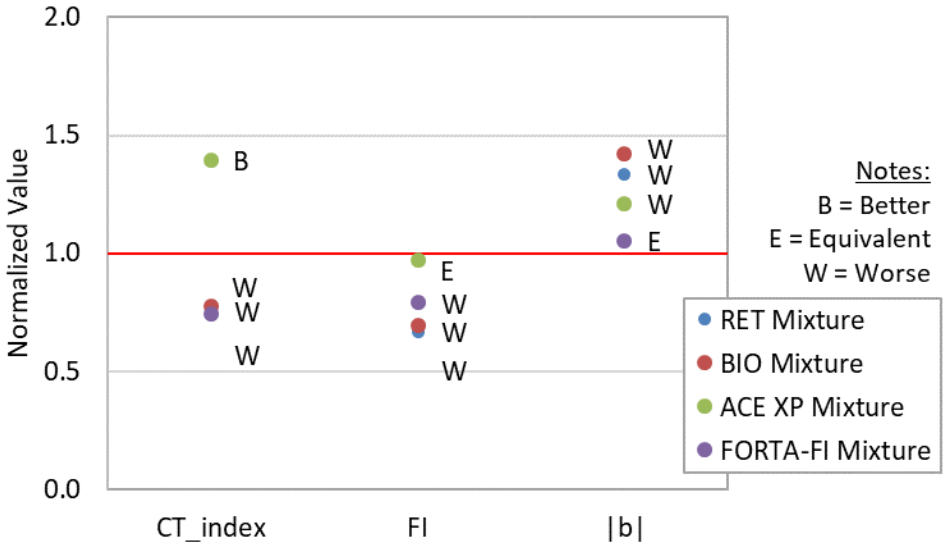


(b)

Figure 3-29. Normalized Rutting Resistance: (a) Asphalt Binders; (b) Asphalt Mixtures



(a)



(b)

Figure 3-30. Normalized Cracking Resistance: (a) Asphalt Binders; (b) Asphalt Mixtures

3.4.3 Protocol Development

In terms of rutting resistance, the APA and the HWT tests revealed an almost similar ranking of mixtures unlike the IDEAL-RT test. Despite having several advantages (such as requiring no cutting and being more efficient and mechanistic) over the APA and the HWT tests, the IDEAL-RT test currently does not have widely verified pass/fail criteria, and

therefore was not chosen as one of the tests for evaluating and approving the AAB/AAM additives in this study. Among the remaining two tests, the HWT test was chosen for this purpose because, compared to the APA test, the HWT test (a) is faster in terms of conditioning and testing time, (b) requires less materials, (c) has well established acceptance criteria, and (d) is accepted as a routine rutting test by many state DOTs, and (e) showed its capability to distinguish mixture (see Figure 3-17). The only drawback of the HWT test is the need of cutting the test specimens at one edge.

In terms of cracking resistance, the SCB-FI, the OT and the IDEAL-CT tests revealed an almost similar ranking of mixtures (see the ranking based on the FI, the CT_{index} and the $|b|$ values). Among them, the IDEAL-CT test was chosen as one of the two tests for evaluating and approving the AAB/AAM additives because this test (a) does not require cutting and notching unlike the other two tests, (b) already has well-verified pass/fail criteria unlike the OT test, and (c) is far more repeatable (than the SCB-FI and the OT tests).

Chapter 6 presents the protocols developed using these tests for evaluating if the AMA binders [i.e., asphalt binders modified with the AAB additives] would perform at least equivalent to the PMA binder and the AMA mixtures [i.e., asphalt mixtures produced with the AMA binders] would perform at least equivalent to the PMA mixture in line with the first objective of this study.

Similarly, Chapter 7 presents the protocol developed using these tests for evaluating if the FMA mixtures [i.e., asphalt mixtures modified with the AAM additives] would perform equivalent to the PMA mixture in line with the second objective of this study. Both protocols use the previously verified criteria for approving new AAM additives.

Both protocols were written in the format of Florida Method of Test for Laboratory Testing the Effectiveness of Anti-Strip Additives, FM 5-508 (FDOT, 2018c). As shown in Figure 3-31, the approval protocols mainly involve three major steps in terms of mixtures:

1. the preparation of SP-12.5 mix of PG 76-22 (PMA) binder (i.e., PMA mixture), SP-12.5 mix of PG 76-22 (AMA) binder (i.e., AMA mixture) and SP-12.5 mix of PG 76-22 (PMA) binder and AAM additive (i.e., FMA mixture).
2. the evaluation of rutting resistance and cracking resistance of the PMA, AMA and FMA mixture, and

3. the approval decision based on whether the AMA binder/mixture has at least equivalent rutting and cracking resistance compared to the PMA binder/mixture.

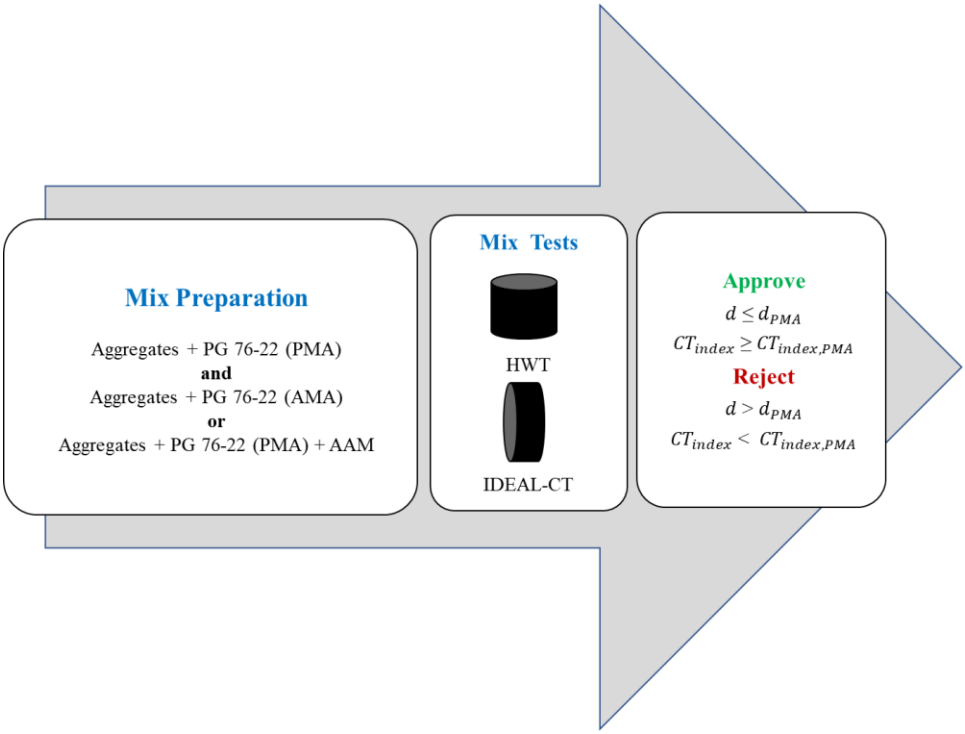


Figure 3-31. Major Steps of the Two Protocols

4 PROTOCOL TESTING

This chapter presents (a) the results obtained by utilizing the first protocol to evaluate if the GTR-modified PG 76-22 (ARB) binder/mixture would perform equivalent to or better than PG 76-22 (PMA) binder/mixture, and (b) the results obtained by utilizing the second protocol to evaluate if the mixture produced with the GTR-modified PG 76-22 (ARB) binder and the ACE XP fibers together would perform equivalent to or better than PG 76-22 (PMA) mixture. The binder used in this part of the study was obtained from one of the sources listed in its APL (see https://fdotwp1.dot.state.fl.us/ApprovedProductList/ProductTypes/Index/91).

4.1 Asphalt Binder Performance Evaluation

PG 76-22 (PMA) and PG 76-22 (ARB) binders were used to test if the GTR-modified PG 76-22 (ARB) binder would perform equivalent to or better than SBS-modified PG 76-22 (PMA) binder:

- Control Binder: PG 76-22 (PMA)
- AMA Binder: PG 76-22 (ARB)

As documented in Chapter 2, GTR is a widely used AAB additive. Based on Section 916-2.1.1 of FDOT's Standard Specification for Road and Bridge Construction (FDOT, 2020), PG 76-22 (ARB) binders shall contain a minimum of 7.0 percent GTR by weight of asphalt binder but the exact percentage of GTR in this binder was not available. Since this binder was obtained in an already-modified state from the FDOT-approved supplier, asphalt binder modification steps of the AAB additive approval protocol (see Section 6.7) were skipped in this study.

4.1.1 Required Asphalt Binder Tests

PG 76-22 (ARB) binder was directly subjected to four different asphalt binder performance tests—the same tests that were previously conducted on PG 76-22 (PMA) binder and were included in the AAB additive approval protocol:

- DSR tests of the OB samples at 10 rad/sec at 76°C (i.e., the grade temperature) to verify its PG, and also at 82°C to determine its true high temperature grade, using two parallel plate specimens [25.0 mm in diameter and 2.0 mm in gap thickness for PG 76-22 (ARB) binder; 25.0 mm in diameter and 1.0 mm in gap thickness for PG

76-22 (PMA) binder] at each temperature following AASHTO T 315-12 (AASHTO, 2016a).

- MSCR tests of the RTFO-aged residue at 67°C—first at a shear stress of 0.10 kPa and then at a shear stress of 3.20 kPa—using two parallel plate specimens [25.0 mm in diameter and 2.0 mm in gap thickness for PG 76-22 (ARB) binder; 25.0 mm in diameter and 1.0 mm in gap thickness for PG 76-22 (PMA) binder] following AASHTO T 350-14 (AASHTO, 2014a).
- BBR tests of the PAV-aged residue at -12.0°C (i.e., the grade temperature + 10°C) to verify its PG, and also at -18°C to determine its true low temperature grade, using two beam specimens (127.0 mm in length, 12.7 mm in width, and 6.25 mm in height for each type of binder) at each temperature following AASHTO T 313-12 (AASHTO, 2016b).
- DSR tests of PAV-aged residue at 10 rad/sec and 26.5°C (i.e., an intermediate temperature) using two parallel plate specimens (8.0 mm in diameter and 2.0 mm in gap thickness for each type of binder) following AASHTO T 315-12 (AASHTO, 2016a).

Table 4-1 presents the PG verification results obtained by testing two specimens of PG 76-22 (ARB) binder and the standard PG 76-22 (PMA) binder at each test temperature. The results show that, based on average values of measured parameters, PG 76-22 (PMA) binder satisfied all the criteria while PG 76-22 (ARB) binder satisfied all but one criterion (i.e., ΔTc = -5.1°C < -5.0°C) specified for these binders in Section 916 of the FDOT's Specification (FDOT, 2020).

Table 4-1. PMA and AMA Binder PG Test Results

Test	Temp. (°C)	Criteria	PG 76-22 (PMA)	PG 76-22 (ARB)
Unaged OB				
DSR: Two replicates	76.0	$G^*/Sin\delta \geq 1.0$ kPa	1.29 ± 0.00	3.44 ± 0.38
		$\delta \leq 75$ deg.	69.8 ± 0.1	66.8 ± 0.5
	$PGH \geq 76$°C		78.9 ± 0.0	90.6 ± 1.3
RTFO-Aged Residue				
MSCR: Two replicates	67.0	$Jnr_{3.2} \leq 1.0$ kPa	0.3 ± 0.0	0.1 ± 0.0
		$Grade \geq$ V	V	V
		$Jnr_{Diff} \leq 75\%$ (No applicable if $Jnr_{3.2} \leq 0.5$)	61.3 ± 0.6 (not applicable)	42.2 ± 0.1 (not applicable)
		$\%Rec_{3.2} \geq 29.37 \times Jnr_{3.2}^{-0.2633}$	67.8 ± 2.3 > 39.9	71.3 ± 0.1 > 53.5
PAV-Aged Residue				
BBR: Two replicates	-12.0	m @ 60 sec ≥ 0.300	0.323 ± 0.003	0.308 ± 0.005
		S @ 60 sec ≤ 300 MPa	182.0 ± 26.9	151.0 ± 25.5
		$\Delta Tc \geq$ -5°C	-2.6 ± 0.7	-5.1 ± 0.9*
	$PGL \leq$ -22°C		-24.1 ± 0.2	-22.9 ± 0.4
DSR: Two replicates	26.5	$G^*.Sin\delta \leq 6{,}000$ MPa	2657 ± 114	2961 ± 49#
Notes:	G^*= Complex shear modulus; δ = Phase angle; PGH = High temperature PG; $Jnr_{3.2}$ = Non-recoverable compliance at a shear stress of 3.2 kPa; Jnr_{Diff} = Difference of non-recoverable compliances at the shear stresses of 3.2 kPa and 0.1 kPa; $\%Rec_{3.2}$ = Percent recovery at a shear stress of 3.2 kPa; m = Creep slope or rate of stress relaxation; S = Creep stiffness; ΔTc = Difference of critical low temperatures corresponding to S-value 300 MPa and m-value of 0.300; PGL = Low temperature PG * represents failing test result in terms of average values; # three replicates			

Various observations were made based on these results:

- $\left(\frac{G^*}{Sin\delta}\right)_{76°C,AMA} > \left(\frac{G^*}{Sin\delta}\right)_{76°C,PMA}$ suggests that PG 76-22 (ARB) binder is much more resistant to rutting in comparison to PG 76-22 (PMA) binder.
- $(\delta)_{76°C,AMA} < (\delta)_{76°C,PMA}$ suggests that PG 76-22 (ARB) binder is slightly more elastic in comparison to PG 76-22 (PMA) binder, which is understandable because of the use of rubber in PG 76-22 (ARB) binder.
- $PGH_{AMA} > PGH_{PMA}$ suggests that PG 76-22 (ARB) binder has the same high temperature PG [i.e., 76°C for both binders based on FDOT's Specification (FDOT, 2020); 88°C for the AMA binder versus 76°C for the PMA binder based on

AASHTO M 320 (AASHTO, 2017a)] but has a much greater true high temperature grade in comparison to PG 76-22 (PMA) binder (i.e., 90.6°C versus 78.9°C).

- $Jnr_{3.2,AMA} < Jnr_{3.2,PMA}$ suggests that PG 76-22 (ARB) binder is more resistant to rutting in comparison to PG 76-22 (PMA) binder.
- $Jnr_{3.2,\text{AMA or PMA}} < 0.50$ suggests that the comparison of $Jnr_{diff} \leq 75\%$ criterion was not applicable to both binders.
- $Rec_{3.2,AMA} > Rec_{3.2,PMA}$ suggests that PG 76-22 (ARB) binder recovers deformation slightly more effectively in comparison to PG 76-22 (PMA) binder, although both binders recover it very well (i.e., $Rec_{3.2} \geq 29.37 \times Jnr_{3.2}^{-0.2633}$ in both binders).
- $m_{-12°C,AMA} < m_{-12°C,PMA}$ suggests that PG 76-22 (ARB) binder has a lower creep slope (i.e., a lower rate of stress relaxation) at a low temperature compared to PG 76-22 (PMA) binder.
- $S_{-12°C,AMA} < S_{-12°C,PMA}$ suggests that PG 76-22 (ARB) binder is softer (i.e., more ductile) than PG 76-22 (PMA) binder at a low temperature, which is understandable because of the use of rubber in PG 76-22 (ARB) binder.
- $PGL_{AMA} > PGL_{PMA}$ suggests that PG 76-22 (ARB) binder has the same low temperature PG (i.e., -22) but a slightly higher true low temperature grade (i.e., -22.9°C versus -24.1°C; making the PG 76-22 (ARB) binder slightly more susceptible to low temperature cracking) compared to PG 76-22 (PMA) binder.
- $\Delta Tc_{AMA} < \Delta Tc_{PMA}$ suggests that PG 76-22 (ARB) binder is more susceptible to low temperature cracking compared to PG 76-22 (PMA) binder.
- $(G^*.Sin\delta)_{AMA} > (G^*.Sin\delta)_{PMA}$ suggests that PG 76-22 (ARB) binder is slightly more susceptible to intermediate temperature cracking (such as fatigue damage) compared to PG 76-22 (PMA) binder.

To summarize, after considering sample-to-sample variation, the $G^*/Sin\delta$, the PGH, the $Jnr_{3.2}$, and the $Rec_{3.2}$ results show that PG 76-22 (ARB) binder has equivalent, if not better, rutting resistance compared to PG 76-22 (PMA) binder. Similarly, the $G^*.Sin\delta$ and the PGL results show that PG 76-22 (ARB) binder has slightly lower cracking resistance at intermediate and low temperatures compared to PG 76-22 (PMA) binder. However, the ΔTc

results show that PG 76-22 (ARB) binder does not have equivalent or better cracking resistance at a low temperature compared to PG 76-22 (PMA) binder. This result highlights the strength of the ΔTc parameter in capturing asphalt binder quality more effectively than the low temperature PG alone. Based on these documented lower cracking resistance (primarily, because $\Delta Tc_{AMA} < \Delta Tc_{PMA}$) results, PG 76-22 (ARB) binder and the type and proportion of GTR used in this binder would not be approved as a better or equivalent substitute of PG 76-22 (PMA) binder.

4.1.2 Additional Asphalt Binder Tests

In addition to the above four tests, PG 76-22 (PMA) and PG 76-22 (ARB) binders were also subjected to three additional tests —the ΔTc tests, the GR damage parameter tests, and the FTIR tests—to verify if the parameters included in the current AAB additive protocol and the parameters obtained from these additional tests help reach the same conclusions.

4.1.2.1 ΔTc Tests

In these tests, the beam specimens of once, twice, or thrice PAV-aged asphalt binder samples (i.e., referred to as PAV20-, PAV40-, and PAV60-aged samples, respectively) were subjected to the BBR tests following AASHTO T 315-12 (AASHTO, 2016a). Two specimens were used at each temperature to run these tests. From these tests, the differences between S- and m-based critical low temperature values (i.e., $\Delta Tc = T_{cs} - T_{cm}$) were determined following ASTM D7643-16 (ASTM, 2016b).

Figure 4-1 presents the ΔTc values of PAV20-, PAV40- and PAV60-aged PG 76-22 (PMA) and PG 76-22 (ARB) binders. The figure shows that the ΔTc value of each binder became more negative with increasing level of aging, which suggests that both binders became more brittle and prone to cracking with increasing level of aging. The figure also shows that PG 76-22 (ARB) binder had generally more negative ΔTc value than PG 76-22 (PMA) binder at each aging level. The results strongly suggest that PG 76-22 (ARB) binder used in this study would not perform equivalent to or better than PG 76-22 (PMA) binder in terms of low temperature cracking performance and that the ΔTc value at PAV20 alone is sufficient to reveal this fact for this comparison.

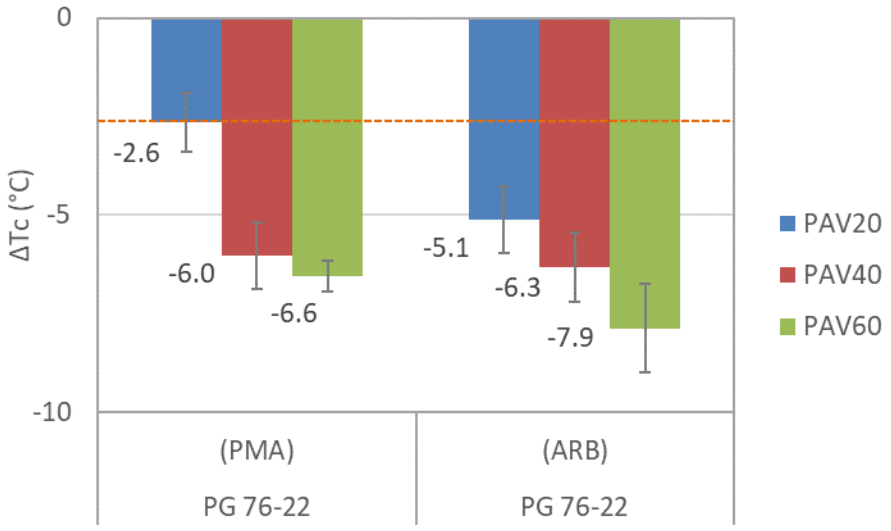


Figure 4-1. ΔTc Tests: PAV20, PAV40, and PAV60

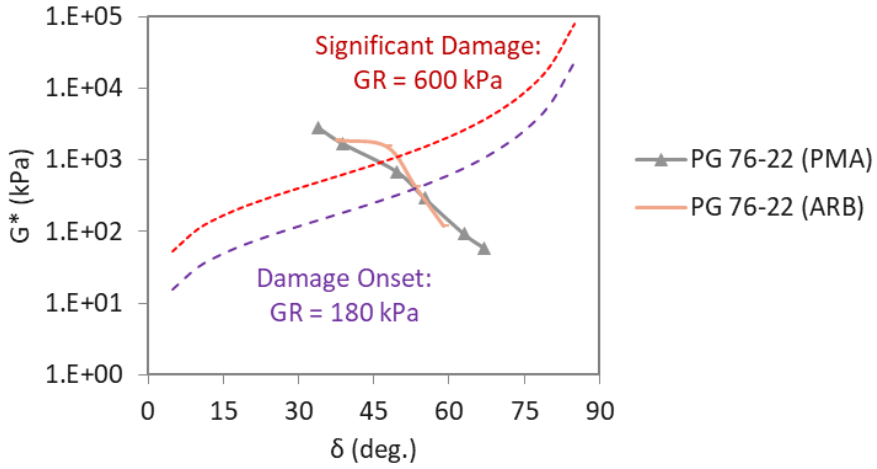
4.1.2.2 GR Tests

In these tests, the parallel plate specimens (8.0 mm in diameter and 2.0 mm in thickness) of OB, RTFO- (or PAV0-), PAV20-, PAV40-, PAV60-, and PAV80-aged asphalt binder samples were subjected to a shear strain amplitude of 0.10% on the DSR at a loading frequency of 0.005 rad/sec and a temperature of 15.0°C. From these tests, the G^*, δ, and GR values were obtained at each aging level and used to determine the PAV hours at GR values of 180.0 kPa and GR = 600.0 kPa, i.e., the damage (cracking) onset and the severe damage conditions of asphalt binders, respectively (Glover et al., 2005; King et al., 2012).

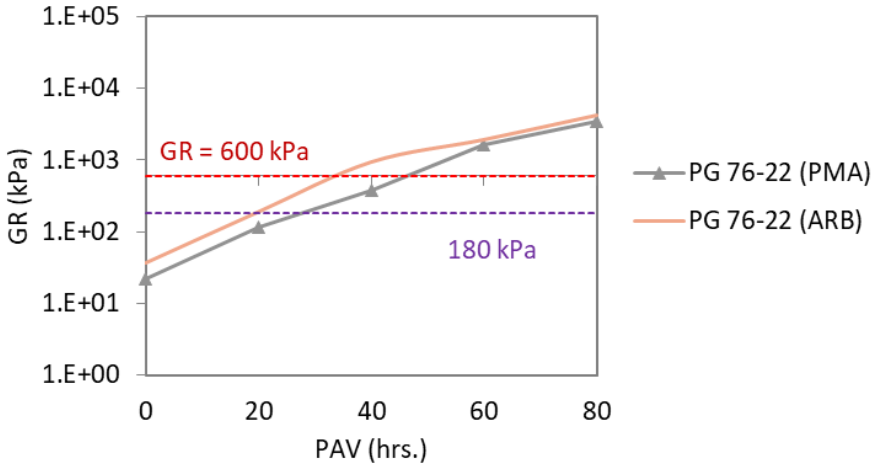
Figure 4-2 (a) presents the G^* versus the δ values of PG 76-22 (ARB) and PG 76-22 (PMA) binders at different aging levels and Figure 4-2 (b) presents their GR values as a function of the PAV aging hours. These figures clearly show that both binders reached the damage onset and the severe damage conditions at different aging levels. Figure 4-2 (c) shows that PG 76-22 (ARB) binder reached these conditions at a lower aging level (i.e., sooner) than PG 76-22 (PMA) binder. These results reinforced the conclusion that discounted PG 76-22 (ARB) binder as an equivalent or better substitute of PG 76-22 (PMA) binder in terms of cracking resistance.

It must be noted that the GR test was not included in the AAB approval protocol developed in this study because (a) the damage onset and the severe damage criteria of 180.0

and 600.0 kPa for GR were actually developed for unmodified binders only (Glover et al., 2005), (b) the effectiveness of the same or different set of criteria for modified asphalt binders has not been widely validated (Newcomb et al., 2018), (c) the sampling, aging, and testing steps of the GR tests have not been standardized, (d) the standardized GR (either AASHTO or ASTM) test method is not currently accepted, and above all, (e) the ΔTc test was already able to discriminate asphalt binders with different cracking resistances in this study.

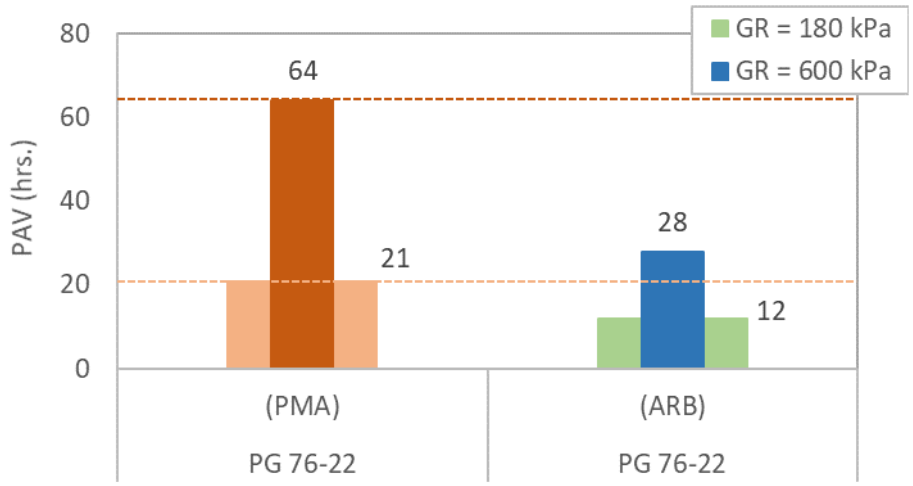


(a)



(b)

Figure 4-2. GR Tests: (a) G^* versus δ; (b); GR versus PAV; (c) Critical PAV



(c)

Figure 4-2. GR Tests: (a) G^* versus δ; (b); *GR* versus *PAV*; (c) Critical *PAV*, Continued

4.1.2.3 FTIR Tests

In these tests, thin films of RTFO- (or PAV0-), PAV20-, PAV40-, PAV60-, and PAV80-aged asphalt binder samples were subjected to the FTIR radiation and the absorption spectra and, after being obtained, were used to calculate the corresponding areas between the wavelengths of 1650 cm^{-1} to 1820 cm^{-1}. Since the absorption of infrared ray in this range of wavelength is due to the carbonyl compounds, this area is commonly referred to as CA. A minimum of two specimens were used to run these tests at each aging level in this study.

Figure 4-3(a) presents the CA values of PG 76-22 (ARB) and PG 76-22 (PMA) binders at five different aging levels. The figure clearly shows that these binders had equivalent CA values at each aging level and equivalent rates of change in the CA values per PAV hour (i.e., aging rates). However, the equivalent CA values and the equivalent rates of change in the CA values per PAV hour did not result in equivalent rheological properties as documented in the previous sections. These results therefore support the exclusion of the FTIR test—a purely chemical test—in the AAB additive approval protocol—a purely mechanical test protocol.

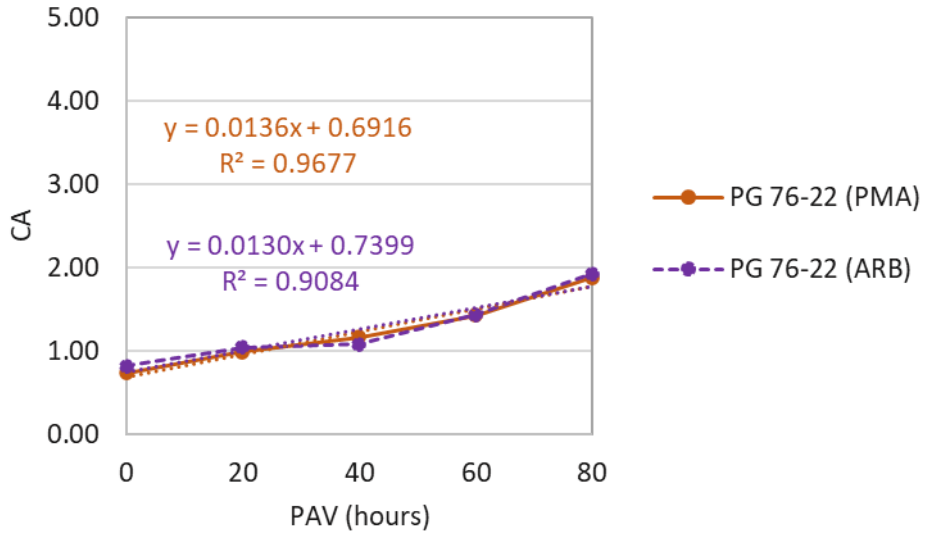


Figure 4-3. FTIR Tests: *CA* versus *PAV*

4.2 Asphalt Mixture Performance Evaluation

Three different types of SP-12.5 mixtures with granite aggregates were used in Task 3 to test the effectiveness of the AAB and the AAM additive approval protocols:

- Control Mixture: Aggregates + PG 76-22 (PMA) = PMA Mixture
- AMA Mixture: Aggregates + PG 76-22 (ARB) = ARB Mixture
- FMA Mixture: Aggregates + PG 76-22 (ARB) + ACE XP = ARB + ACE XP Mixture

Among them, the PMA and the ARB binders/mixtures were used to test the AAB additive approval protocol while the PMA and the ARB + ACE XP mixtures were used to test the AAM additive approval protocol. A series of steps were followed to produce the ARB (see Section 6.9) and the ARB + ACE XP mixtures (see Section 7.7) in this task. The major steps involved:

- Heating pre-batched aggregate samples in an oven at the mixing temperature of 154.0°C (309.2°F) for a minimum of 6.0 hours,
- Heating PG 76-22 (ARB) binder in an oven at the mixing temperature of 154.0°C (309.2°F) for a minimum of 2.0 hours,
- Producing the ARB mixture by mixing the heated aggregates and 5.1% heated binder with each other for about 3.0 minutes without any fibers,

- Producing the ARB + ACE XP mixture by (a) adding the Sasobit wax on the heated aggregates in the bucket, (b) mixing aggregates/Sasobit wax and 5.1% asphalt binder for about 3.0 minutes while adding the separated strands of the ACE XP fibers into the mixture,
- Aging the mixtures in an oven for 2.0 hours at the compaction temperature of 154°C (309.2°F) and stirring them at least once during this period,
- Determining the Gmm values of the loose mixtures,
- Compacting the mixtures into cylindrical specimens with a diameter of 150.0 mm until 75 gyrations to prepare the mix design specimens or until a height of 62.0 ± 1.0 mm using a SGC to prepare the HWT and the IDEAL-CT test specimens,
- Cutting the SGC specimens prepared for the HWT tests along the secant lines on one edge,
- Measuring the Gmb values of mix design (uncut), the HWT (i.e., one-edge cut) and the IDEAL-CT (uncut) specimens, and
- Confirming the volumetric properties (air void contents) of the specimens before running the HWT and the IDEAL-CT tests.

Table 4-2 presents the volumetric properties of the mix design and performance test specimens of the PMA, the ARB, and the ARB + ACE XP mixtures. The results show that each mixture satisfied all requirements of SP-12.5 specified in Section 334-3.2.2 of the FDOT's current Standard Specification for Road and Bridge Construction (FDOT, 2020). The table also shows that the volumetric properties of the PMA, the ARB, and the ARB + ACE XP mixtures did not differ much from each other. As such, the same weights used to prepare the HWT and the IDEAL-CT specimens of the PMA mixture were used to prepare the corresponding specimens of the ARB and the ARB + ACE XP mixtures.

Table 4-2. PMA, AMA, and FMA Volumetric Properties

Specimens	Parameter		Criteria	PMA	ARB	ARB + ACE XP
Mix Design (N = 75 gyrations)	Gmm	Average	-	2.574	2.563	2.566
		Range	≤ 0.013	0.008	0.001	0.001
		Std. Dev.	≤ 0.00449	0.00373	0.00087	0.00039
	Gmb	Average	-	2.468	2.470	2.470
		Range	0.011	0.003	0.002	0.003
		Std. Dev.	-	0.00208	0.00207	0.00138
	Va	N/A	4.0 ± 0.5	4.1	3.6	3.8
	VMA	N/A	≥14.0	14.6	14.5	14.5
	VFA	N/A	65-75	71.8	75.0	74.0
	$\frac{P_{0.075}}{P_{be}}$	N/A	0.6-1.2	1.0	0.9	0.9
HWT (h = 62 mm)	Va	Average	7.0 ± 1.0	7.4	6.9	7.2
IDEAL-CT (h = 62 mm)	Va	Average	7.0 ± 0.5	7.3	6.7	7.4
Notes:	N = Number of Gyrations; h = Compacted Height; Gmm = Theoretical maximum specific gravity of asphalt mixture; Gmb = Bulk specific gravity of asphalt mixture; Va = Air void content (%); VMA = Voids in mineral aggregates (%); VFA = Voids filled with asphalt (%); $P_{0.075}$ = Percentage of aggregates passing no. 200 or 0.075-mm sieve (%); P_{be}= Effective asphalt binder content (%); N/A = Not available or not applicable					

4.2.1 Required Asphalt Mixture Rutting Tests: HWT Tests

Four SGC specimens, each with a diameter of 150.0 mm (5.91 in.), a compaction height of 62.0 ± 1.0 mm (2.44 ± 0.04 in.) and an air void content of 7.0 ± 0.5 percent (93.0 ± 0.5%Gmm) after cutting along secants on one edge, were subjected to the HWT tests at 50.0 ± 0.5°C (122.2 ± 0.9°F) following AASHTO T 324 (AASHTO, 2017b).

Figure 4-4 presents the rut depth versus the wheel pass (d versus N) profiles obtained from the HWT tests of the PMA, the ARB, and the ARB + ACE XP mixtures. The figure shows that the HWT tests stopped after reaching 20,000 passes and registering rut depths below the maximum acceptance value of 12.5 mm (0.50 inch).

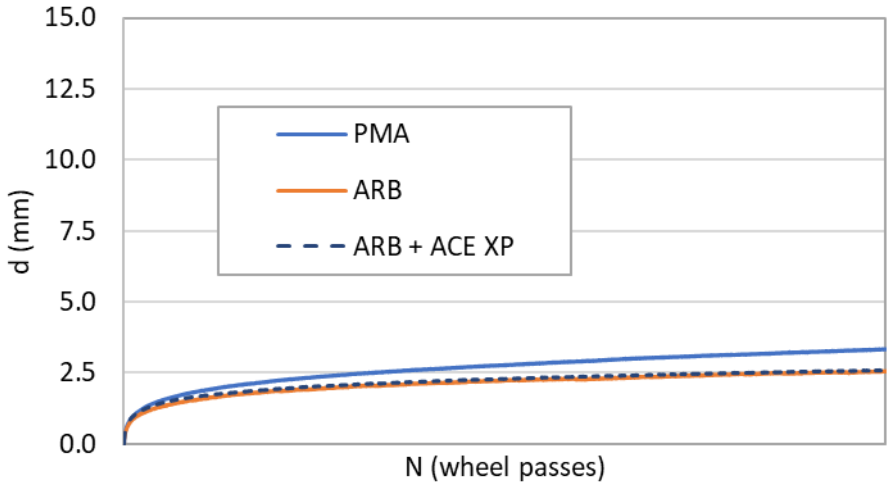


Figure 4-4. HWT Tests: *d* versus *N* at 52 passes/min at 50.0°C

Figure 4-5 presents the average and the standard deviation values of rut depths obtained from the HWT tests of each mixture at 20,000 passes and 50.0°C (i.e., d_{20000}). The figure shows that the ARB mixture had a lower d_{20000} value compared to the PMA mixture [i.e., $d_{20000,ARB}$ (2.56 ± 0.25 mm) < $d_{20000,PMA}$ (3.34 ± 0.21 mm)], which means that the ARB mixture would perform better than the PMA mixture in terms of rutting resistance (see Section 6.12).

Figure 4-5 also shows that the ARB + ACE XP mixture also had a lower d_{20000} value compared to the PMA mixture [i.e., $d_{20000,ARB+ACE\ XP}$ (2.60 ± 0.30 mm) < $d_{20000,PMA}$ (3.34 ± 0.21 mm)], which means that the ARB + ACE XP mixture (i.e., the FMA mixture used in this task) also would perform better than the PMA mixture in terms of rutting resistance (see Section 7.10).

The figure also reveals that the ARB and the ARB + ACE XP mixture had equivalent d_{20000} values compared to each other [i.e., $d_{20000,ARB}$ (2.56 ± 0.25 mm) ≈ $d_{20000,ARB+ACE\ XP}$ (2.60 ± 0.30 mm)], which means that the ARB and the ARB + ACE XP mixtures (i.e., the AMA and the FMA mixtures used in this task) would perform equivalent to each other in terms of rutting resistance. In other words, the addition of the ACE XP fibers did not help improve the rutting resistance of the ARB mixture at all.

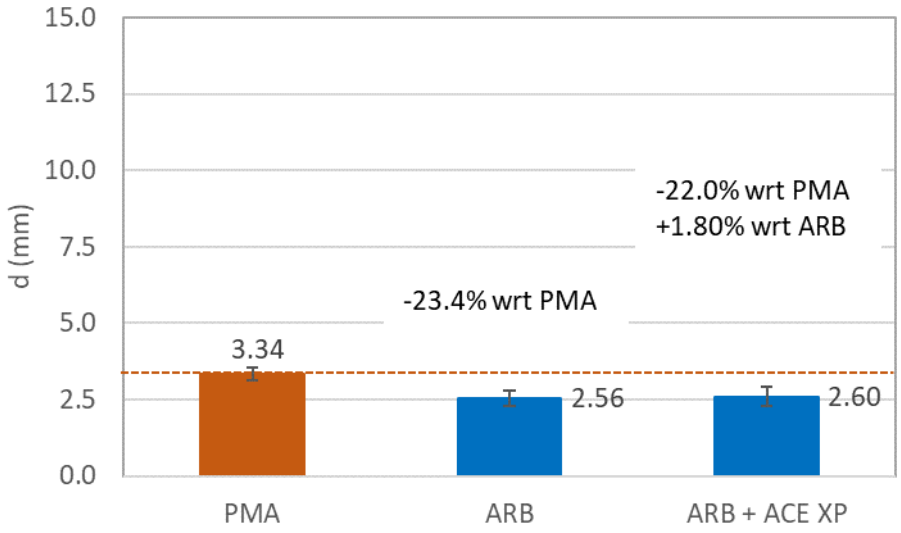


Figure 4-5. HWT Tests: *d* at 20,000 passes and 50.0°C

4.2.2 Additional Asphalt Mixture Rutting Tests: APA Tests

In addition to running the HWT tests included in the AAB and the AAM additive approval protocols, the APA tests were also conducted on each of these three mixtures to re-verify their ranking based on their rutting performance.

For these tests, four additional SGC specimens, each with a diameter of 150.0 mm (5.91 in.), a compaction height of 75.0 ± 1.0 mm (2.95 ± 0.04 in.) and an air void content of 7.0 ± 0.5 percent (93.0 ± 0.5%Gmm), were prepared. These tests were conducted at 64.0 ± 0.5°C (147.2 ± 0.9°F) using the same loading level, the loading rate and the stop criteria as used in Task 2 following AASHTO T 340 (AASHTO, 2010). The average air void contents in the APA samples of the PMA, the ARB, and the ARB + ACE XP mixtures were 7.3, 6.7 and 7.4 percent, respectively.

Figure 4-6 presents the rut depth versus the number of cycle (d versus N) profiles obtained from the APA tests of the PMA, the ARB, and the ARB + ACE XP mixtures. The figure shows that the APA tests stopped after reaching 8,000 cycles, registering rut depths below the FDOT-allowed maximum rut depth value of 4.5 mm (0.18 inch).

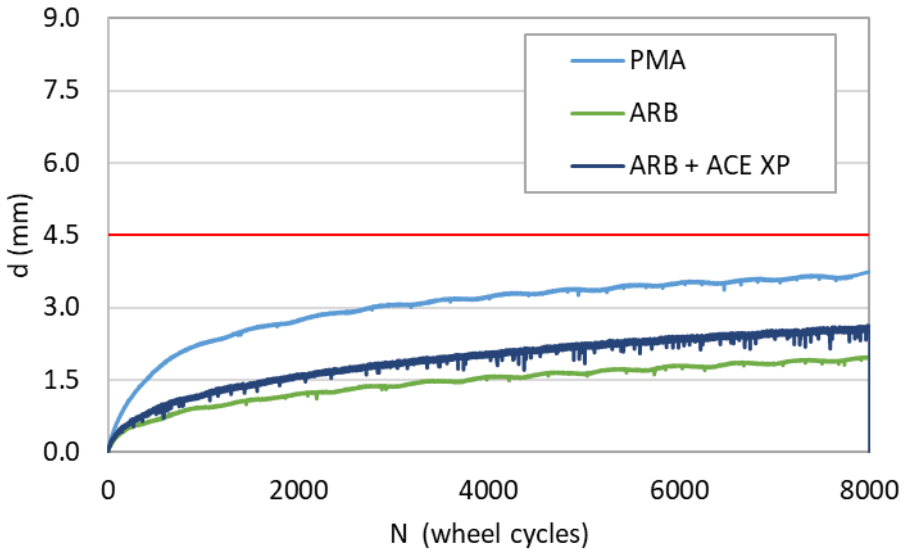


Figure 4-6. APA Tests: *d* versus *N* at 60 cycles/min at 64.0°C

Figure 4-7 presents the average and the standard deviation values of rut depths obtained from the APA tests of each mixture at 8,000 cycles and 64.0°C (i.e., d_{8000}). The figure shows that the ARB mixture had a lower $d_{8,000}$ value compared to the PMA mixture [i.e., $d_{8000,ARB}$ (1.96 ± 0.27 mm) < $d_{8000,PMA}$ (3.63 ± 0.20 mm)], which means that the ARB mixture would perform better than the PMA mixture in terms of rutting resistance.

Figure 4-7 also shows that the ARB + ACE XP mixture also had a lower $d_{8,000}$ value compared to the PMA mixture [i.e., $d_{8000,ARB+ACE\ XP}$ (2.51 ± 0.29 mm) < $d_{8000,PMA}$ (3.63 ± 0.20 mm)], which means that the ARB + ACE XP mixture (i.e., the FMA mixture used in this task) also would perform better than the PMA mixture in terms of rutting resistance. The figure also reveals that the addition of the ACE XP fibers did not help improve the rutting performance of the ARB mixture.

Since the APA test results essentially ranked these mixtures in the same order as did the HWT test results (i.e., ARB mixture > ARB mixture > ARB + ACE+XP mixture) in terms of rutting performance, these results reinforce the inclusion of only one rutting test (i.e., HWT test) instead of two rutting tests (both APA and HWT tests).

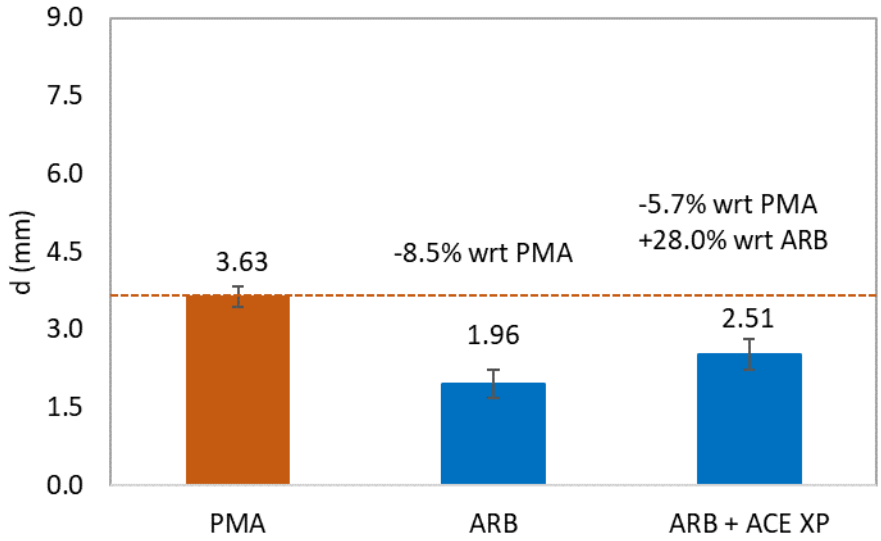
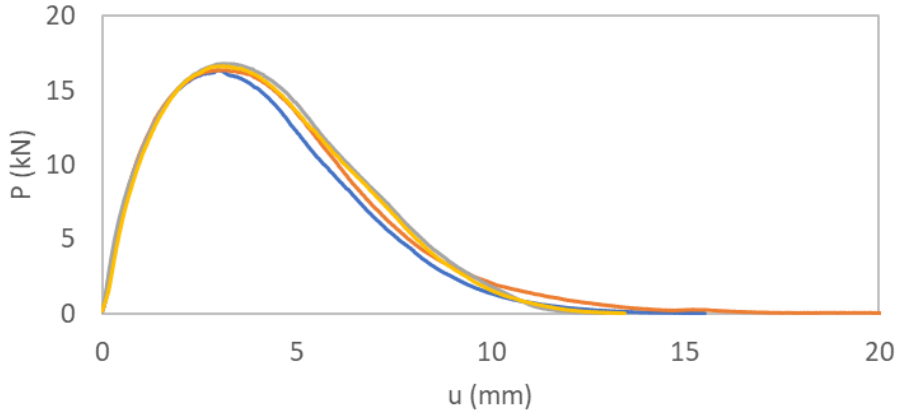


Figure 4-7. APA Tests: *d* at 8,000 cycles and 64.0°C

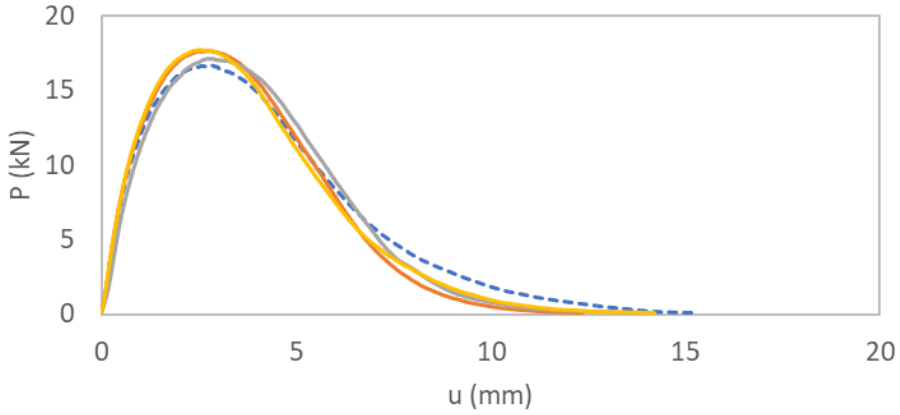
4.2.3 Required Asphalt Mixture Cracking Tests: IDEAL-CT Tests

A minimum of four SGC specimens, each with a diameter of 150.0 mm (5.91 in.), a compaction height of 62.0 ± 1.0 mm (2.44 ± 0.04 in.) and an air void content of 7.0 ± 0.5 percent (93.0 ± 0.5%Gmm) without cutting, were subjected to the IDEAL-CT tests at the LLD rate of 50 mm/sec at 25.0 ± 0.5°C (77.0 ± 0.9°F) following ASTM D8225 (ASTM, 2019).

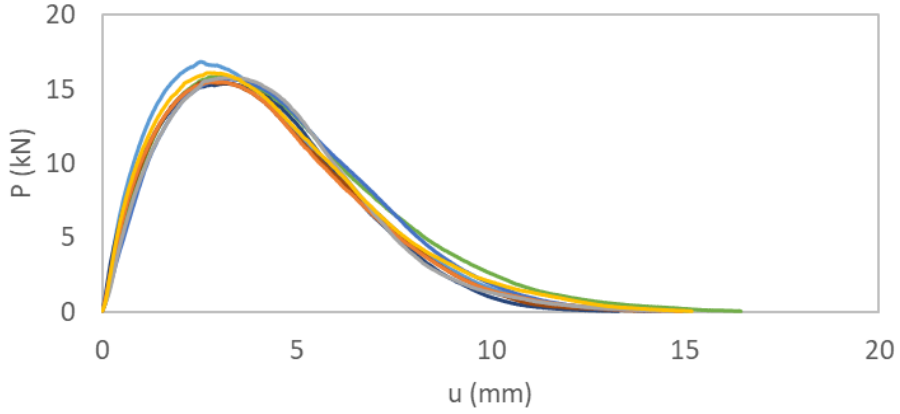
Figure 4-8 presents the load versus LLD (i.e., P versus u) profiles obtained from the IDEAL-CT tests of a minimum of four specimens from each mixture. A total of eight specimens of the ARB + ACE XP mixture was prepared and tested. Only the test results obtained from the specimens that met 7.0 ± 0.5 percent air void content criterion (93.0 ± 0.5%Gmm) were used for the final analysis. Two out of eight specimens had 7.6 percent air void content, and therefore, only the results obtained from the other six specimens were included in the final analysis.



(a)



(b)



(c)

Figure 4-8. IDEAL-CT Tests: *P* versus *u* profiles at 50.0 mm/min and 25.0°C: (a) PMA; (b) ARB; (c) ARB + ACE XP

Figure 4-9 presents the average and the standard deviation values of cracking tolerance index (CT_{index}) obtained from a minimum of four specimens from each mixture (four PMA, four ARB and six ARB + ACE XP specimens). The figure shows that the ARB mixture had a lower CT_{index} value compared to the PMA mixture [i.e., $CT_{index,ARB}$ (85.0 ± 10.2) < $CT_{index,PMA}$ (123.1 ± 13.6)], which suggests that the use of GTR-modified PG 76-22 (ARB) binder would result in an inferior mixture compared to the PMA mixture in terms of intermediate temperature cracking resistance (see Section 6.12).

Figure 4-9 also shows that the ARB + ACE XP mixture had an equivalent CT_{index} value compared to the PMA mixture [i.e., $CT_{index,ARB+ACE\ XP}$ (128.7 ± 11.0) > $CT_{index,PMA}$ (123.1 ± 13.6)], which suggests that the use of PG 76-22 (ARB) binder and the ACE XP fibers together would result in an equivalent mixture compared to the PMA mixture in terms of intermediate temperature cracking resistance (see Section 7.10). The figure also reveals that the ARB + ACE XP mixture had a significantly higher CT_{index} value compared to the ARB mixture [i.e., $CT_{index,ARB+ACE\ XP}$ (128.7 ± 11.0) > $CT_{index,ARB}$ (85.0 ± 10.2)], which suggests that the use of PG 76-22 (ARB) binder and the ACE XP fibers together would result in a significantly superior mixture compared to the ARB mixture in terms of intermediate temperature cracking resistance.

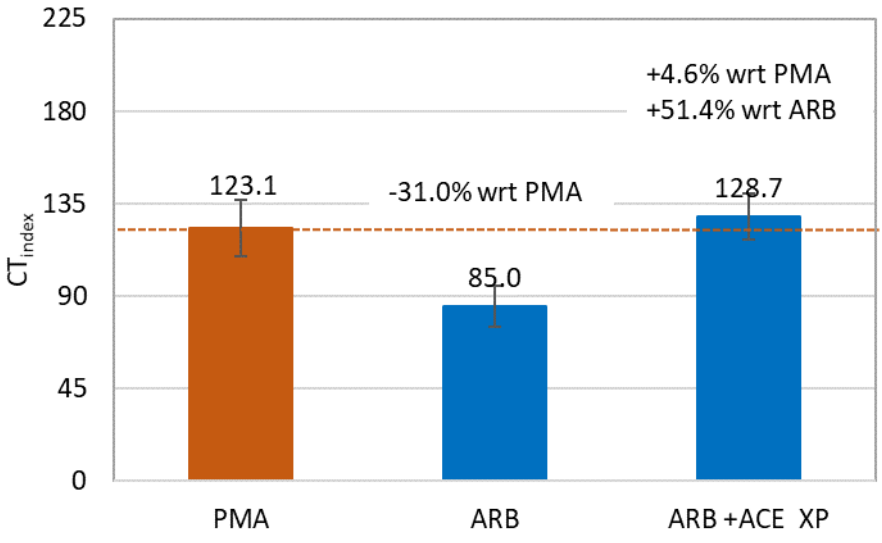


Figure 4-9. IDEAL-CT Tests: CT_{index} at 25.0°C

4.3 Summary

The AAB additive approval protocol showed that the GTR-modified PG 76-22 (ARB) binder (i.e., the AMA binder used in this task) had a slightly lower δ value (i.e., a slightly higher elasticity), a slightly higher $G^*.Sin\delta$ value (i.e., a slightly lower cracking resistance at an intermediate temperature) and a much lower ΔTc value (i.e., a lower cracking resistance at a low temperature) than the PMA binder. The AAB additive approval protocol also showed the PG 76-22 (ARB) binder had a higher rutting resistance than PG 76-22 (PMA) binder. Because of the lower resistance of PG 76-22 (ARB) binder to cracking, the protocol would not approve this binder as an equivalent or a better substitute of PG 76-22 (PMA) binder. The conclusion was also supported by the ΔTc and the GR tests conducted on several aging levels.

The AAB additive approval protocol also showed that the ARB mixture (i.e., the AMA mixture used in this task) exhibited a lower rut depth value (i.e., a higher rutting resistance) and a lower CT_{index} value (i.e., a lower cracking resistance) than the PMA mixture in terms of their mean values. When sample-to-sample variations of these parameters were considered, the results showed that the GTR-modified PG 76-22 (ARB) mixture would be a better substitute of the PMA mixture in terms of rutting performance but neither an equivalent nor a better substitute of the SBS-modified PG 76-22 (PMA) mixture in terms of intermediate temperature cracking performance. Since the ARB mixture was not even equivalent to the PMA mixture in terms of rutting resistance parameter, the protocol would disapprove the use of this AMA binder/mixture as an equivalent or better substitute of the PMA binder/mixture (see Section 6.12).

The AAM additive approval protocol showed that the ARB + ACE XP mixture [i.e., the FMA mixture produced with the GTR-modified PG 76-22 (ARB) binder and the ACE XP fibers in this task] had a lower rut depth value (i.e., a higher rutting resistance) and only a slightly higher CT_{index} value (i.e., only a slightly higher cracking resistance) than the PMA mixture in terms of mean values of the measured parameters. When sample-to-sample variations of these parameters were also considered, the results showed that the mixture produced using the GTR-modified PG 76-22 (ARB) binder and the ACE XP fibers together would be a better substitute of the SBS-modified PMA mixture in terms of rutting

performance and an equivalent substitute of the SBS-modified PMA mixture in terms of cracking performance. Since the ARB + ACE XP mixture was at least equivalent to the PMA mixture in terms of both rutting and cracking performance, the protocol would approve the use of this FMA mixture only as an equivalent substitute of the PMA mixture (see Section 7.10).

It must be noted that the only contribution of the ACE XP fibers was elevating the cracking resistance of the ARB mixture and making it equivalent to the PMA mixture without improving its rutting resistance at all. Since this protocol would approve the ARB + ACE XP mixture only as an equivalent substitute of the PMA mixture based on their mechanical properties, the cost-benefit analysis of the ARB + ACE XP mixture use must be conducted.

These results verified the effectiveness of both the AAB and the AAM approval protocols.

5 Conclusions, Limitations, and Recommendations

5.1 Conclusions

TTI researchers conducted several tasks for developing the protocols that can evaluate and approve AAB and AAM additives. From each task, several conclusions were drawn as summarized below.

5.1.1 Literature Review

- *Asphalt Binder Additives*: Asphalt binders are mostly modified with non-reactive polymers, GTR with or without the aid of PPA to modify high temperature PG and with REOB/VTAE, extenders, aromatic extracts, fatty acids, etc. to meet low temperature performance. Reactive polymers (e.g., terpolymers and copolymers) are used in pavements less extensively than in laboratory-based research projects. Excessive use of any additive is self-defeating, which is especially true in the case of reactive polymers because these polymers can form a separate gel-like phase if they are not property blended and cured.
- *Asphalt Mixture Additives*: Asphalt mixtures are mostly modified with cellulose and mineral fibers. Polymeric fibers have been allowed or experimented with by only a few state DOTs. Most of the asphalt mixtures that use fibers are either gap- or open-graded (e.g., SMA, OGFC). Polymeric fibers can stabilize and reinforce asphalt mixtures at the same time.
- *Asphalt Binder Performance Tests*: Superpave asphalt binder rutting and cracking resistance parameters cannot clearly determine the effect of additives. Some of the parameters obtained from the new rutting tests (e.g., Jnr and $\%Rec$. obtained from MSCR test) and cracking tests (e.g., GR from Glover-Rowe and FREI obtained from PLAS) along with the parameters obtained from traditional tests (ΔTc values from BBR tests) show potential in capturing the effect of additives on rutting and cracking resistance.
- *Asphalt Mixture Performance Tests*: The most often used rutting tests are APA and HWT tests, but these tests are long, and costly while relatively less sensitive to asphalt binders/mixture modification. These two tests (i.e., the APA and the HWT tests) and a more recent test (i.e., Ideal rutting test) were included in the

experimental design of this study. Many cracking tests have been developed in the literature, but each test has its pros and cons. After considering the practicality and other DOTs' experiences, three cracking tests: SCB-FI, OT, and IDEAL-CT were included in this study to evaluate asphalt mix performance in the laboratory.

5.1.2 Protocol Development

- *Rutting Resistance*: The APA and the HWT tests revealed an almost similar ranking of mixtures unlike the IDEAL-RT test. Despite having several advantages (such as requiring no cutting and being more efficient and mechanistic) over the APA and the HWT tests, the IDEAL-RT test currently does not have widely verified pass/fail criteria, and therefore was not chosen as one of the tests for evaluating and approving the AAB/AAM additives in this study. Among the remaining two tests, the HWT test was chosen for this purpose because, compared to the APA test, the HWT test (a) is faster in terms of conditioning and testing time, (b) requires less materials, (c) has well established acceptance criteria, (d) is accepted as a routine rutting test by many state DOTs, and (e) showed its capability to distinguish mixtures (see Figure 3-17). The only drawback of the HWT test is the need of cutting the test specimens at one edge.
- *Cracking Resistance*: The SCB-FI, the OT and the IDEAL-CT tests revealed an almost similar ranking of mixtures (see the ranking based on the FI, the CT_{index} and the $|b|$ values). Among them, the IDEAL-CT test was chosen as one of the two tests for evaluating and approving the AAB/AAM additives because this test (a) does not require cutting and notching unlike the other two tests, (b) already has well-verified pass/fail criteria unlike the OT test, and (c) is far more repeatable (than the SCB-FI and the OT tests).
- *Protocols for Approving the AAB and the AAM Additives*: With the selected rutting and cracking tests, the approval protocols mainly involve three major steps in terms of mixtures:
  1. The preparation of SP-12.5 mix of PG 76-22 (PMA) binder (i.e., PMA mixture), SP-12.5 mix of PG 76-22 (AMA) binder (i.e., AMA mixture) and SP-12.5 mix of PG 76-22 (PMA) binder and AAM additive (i.e., FMA mixture).

2. The evaluation of rutting resistance and cracking resistance of the PMA, the AMA and the FMA mixtures, and
3. The approval decision based on whether the AMA binder/mixture has at least equivalent rutting and cracking resistance compared to the PMA binder/mixture.

5.1.3 Protocol Testing

- The AAB additive approval protocol showed that the GTR-modified PG 76-22 (ARB) binder (i.e., the AMA binder used in this task) had a slightly lower δ value (i.e., a slightly higher elasticity), a slightly higher $G^*.Sin\delta$ value (i.e., a slightly lower cracking resistance at an intermediate temperature) and a much lower ΔTc value (i.e., a lower cracking resistance at a low temperature) than the PMA binder. The AAB additive approval protocol also showed the PG 76-22 (ARB) binder had a higher rutting resistance than PG 76-22 (PMA) binder due to larger $G^*/Sin\delta$, smaller $Jnr_{3.2}$, and larger % recovery. Because of the lower resistance of PG 76-22 (ARB) binder to cracking, the protocol would not approve this binder as an equivalent or a better substitute of PG 76-22 (PMA) binder. The conclusion was also supported by the ΔTc and the GR tests conducted on several aging levels.
- The AAB additive approval protocol also showed that the ARB mixture (i.e., the AMA mixture used in this task) exhibited a lower rut depth value (i.e., a higher rutting resistance) and a lower CT_{index} value (i.e., a lower cracking resistance) than the PMA mixture in terms of their mean values. When sample-to-sample variations of these parameters are considered, the results show that the GTR-modified PG 76-22 (ARB) mixture would be a better substitute of the PMA mixture in terms of rutting performance but neither an equivalent nor a better substitute of the SBS-modified PG 76-22 (PMA) mixture in terms of intermediate temperature cracking performance. Since the ARB mixture was not even equivalent to the PMA mixture in terms of cracking resistance, the protocol would disapprove the use of this AMA binder/mixture as an equivalent or better substitute of the PMA binder/mixture (see Section 6.12).

- The AAM additive approval protocol showed that the ARB + ACE XP mixture [i.e., the FMA mixture produced with the GTR-modified PG 76-22 (ARB) binder and the ACE XP fibers in this task] had a lower rut depth value (i.e., a higher rutting resistance) and only a slightly higher CT_{index} value (i.e., only a slightly higher cracking resistance) than the PMA mixture in terms of mean values of the measured parameters. When sample-to-sample variations of these parameters are also considered, the results show that the mixture produced using the GTR-modified PG 76-22 (ARB) binder and the ACE XP fibers together would be a better substitute of the SBS-modified PMA mixture in terms of rutting performance and an equivalent substitute of the SBS-modified PMA mixture in terms of cracking performance. Since the ARB + ACE XP mixture was at least equivalent to the PMA mixture in terms of both rutting and cracking performance, the protocol would approve the use of this FMA mixture only as an equivalent substitute of the PMA mixture (see Section 7.10).

5.2 Limitations

The conclusions presented in the preceding section were drawn based on the results obtained from the literature review and laboratory tests conducted for developing AAB and AAM additive evaluation and approval. There were a few limitations that may need further research and field validation. These limitations include:

- Use of only one type of aggregate (i.e., granite): The results and conclusions (i.e., the parameter values and the rankings) obtained from the mixtures of granite should not be used to evaluate the rutting and cracking resistances of mixtures produced with other aggregate types (such as limestone, gravel, etc.) even if the same additives are being used.
- Use of only one type of mix design (i.e., Type SP-12.5 mix): The results and conclusions (i.e., the parameter values and the rankings) obtained from Type SP-12.5 mix should not be directly used to evaluate the rutting and cracking resistances of other mix types (such as Type SP-9.5 mix) even if the same additives are being used.

- Use of only two types of distresses: This study focused only on rutting and cracking resistances but not on durability and moisture damage. Therefore, the protocols developed in this study should be used to judge superiority, equivalency, or inferiority of AMA mixtures over PMA mixture only with regards to rutting and cracking resistances.
- Absence of other additives: The mix design used in this study did not include any hydrated lime or liquid antistrip agents. Therefore, the impact of interaction between such additives with the AAB and AAM additive mix performance remains unknown at this stage. Therefore, since all mixtures are actually produced in the field with an anti-stripping agent, testing the mixtures in the laboratory with an anti-stripping agent should be considered in the future.
- Comparison is only with PMA mixture: As was not the scope of this study, TTI researchers did not compare the performance of one type of FMA mixture with the other. TTI researchers only compared the performance of a given FMA mixture type with the performance of PMA mixture because the two fibers used in this study had different composition, length and blending procedure.
- Laboratory versus field blending of AAM additives: This study did not investigate whether the lab-produced FMA mixtures would be equivalent to the field-produced FMA mixtures in terms of their blending of the fibers homogeneously and their performance.

5.3 Recommendations

- Based on the conclusions presented above, implementation is recommended of the approval protocols for evaluating and approving AAB and AAM additives using Type SP-12.5 mixture of granite aggregates.
- Further laboratory and field studies are needed to verify the recommended additive approval protocols. These studies should include more aggregate types (limestone), more mix design types (Type SP-9.5), more distress types (moisture susceptibility and durability), and more additive inclusions (presence of anti-strip agents) at the minimum.

6 ALTERNATIVE ASPHALT BINDER ADDITIVE APPROVAL PROTOCOL

6.1 Scope

This method covers the laboratory production of the Superpave gyratory compacted (SGC) specimens of asphalt mixtures containing PG 76-22 (PMA) and AMA binders, and laboratory measurement of their rutting and cracking resistances. The results of these tests will be used for evaluating rutting and cracking resistance of the PMA and the AMA mixtures, and for approving AAB additives. The values stated in the International System of Units (SI) of units are to be regarded as the standard.

6.2 Referenced Documents

AASHTO Standards

- AASHTO R 28 Standard Practice for Accelerated Aging of Asphalt Binder Using A Pressurized Aging Vessel
- AASHTO R 30 Standard Practice for Mixture Conditioning of Hot-Mix Asphalt (HMA)
- AASHTO T 240 Standard Method of Test for Effect of Heat and Air on a Moving Film of Asphalt
- AASHTO T 312 Standard Method for Preparing and Determining the Density of Hot-Mix Asphalt (HMA) by Means of the Superpave Gyratory Compactor
- AASHTO T 313 Standard Method of Test for Determining the Flexural Creep Stiffness of Asphalt Binder Using the Bending Beam Rheometer (BBR)
- AASHTO T 315 Standard Method of Test for Determining the Rheological Properties of Asphalt Binder Using a Dynamic Shear Rheometer (DSR)
- AASHTO T 324 Standard Method of Test for Hamburg Wheel-Track Testing of Compacted Asphalt Mixtures
- AASHTO T 350 Standard Method of Test for Multiple Stress Creep Recovery (MSCR) Test of Asphalt Binder Using a Dynamic Shear Rheometer (DSR)
- AASHTO M 332 Standard Specification for Performance-Graded Asphalt Binder Using Multiple Stress Creep Recovery (MSCR) Test

ASTM International Standards

- ASTM D7643 Standard Practice for Determining the Continuous Grading Temperatures and Continuous Grades for PG Graded Asphalt Binders
- ASTM D8225 Standard Test Method for Determination of Cracking Tolerance Index of Asphalt Mixture Using the Indirect Tensile Cracking Test at Intermediate Temperature

Florida Method of Tests

- FM 1-T 166 Bulk Specific Gravity of Compacted Bituminous Mixtures
- FM 1-T 209 Maximum Specific Gravity of Bituminous Paving Mixtures

6.3 Significance and Use

This method will be used for evaluating the effects of the AAB additives on rutting and cracking resistance of a PMA mixture and for approving the AAB additives.

6.4 Summary of Method

Pre-batched aggregates, an AAB additive, crosslinking agent (if applicable), and the protocol to modify one of the Department-approved Superpave asphalt binders to a PG 76-22 asphalt binder using the AAB additive are obtained from the supplier of the AAB additive.

The AMA binder is produced by blending the supplier-identified Department-approved Superpave asphalt binder with the supplier-provided AAB additive and the supplier-provided crosslinking agent, if applicable, at the supplier-recommended dosages following the supplier-provided blending protocol. PG 76-22 (PMA) and the AMA binder are then subjected to the tests specified in Section 916 of the Department's Specifications for Road and Bridge Construction.

The PMA mixture is then prepared by mixing the aggregates with PG 76-22 (PMA) binder while the AMA mixture is prepared by mixing the aggregates with the AMA binder that meets PG 76-22 requirements, i.e., [PG 76-22 (AMA) binder].

The SGC specimens of the PMA and the AMA mixtures are then subjected to the Hamburg Wheel-Track (HWT) and the Ideal Cracking Tolerance (IDEAL-CT) tests to measure their rutting and cracking resistance parameters. A minimum of two test specimens from each binder type and a minimum of four test specimens from each mixture are used in

each test (see Figure 6-1). Approval of the AAB additive is contingent upon whether the AMA mixture satisfies the criteria set for both rutting and cracking resistance parameters.

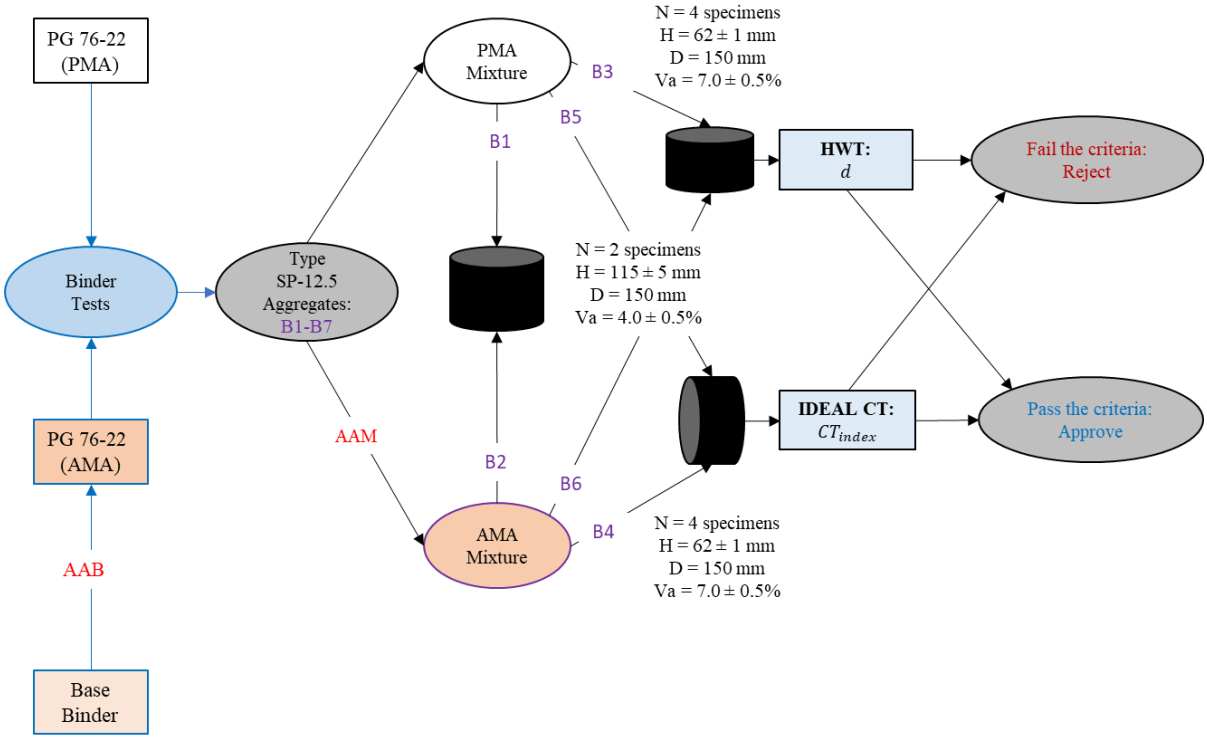


Figure 6-1. AAB Additive Approval Protocol Illustration

6.5 Test Apparatus

For Asphalt Binders

- Shear mixer, heating plate, oven, and accessories for binder modification.
- Rolling thin film oven (RTFO) for AASHTO T 240 tests.
- Pressure aging vessel (PAV) for AASHTO R 28 tests.
- Dynamic shear rheometer (DSR) for AASHTO T 315 and AASHTO T 350 tests.
- Bending beam rheometer (BBR) for AASHTO T 313 and ASTM D7643 tests.

For Asphalt Mixtures

- Bucket and paddle mixer, blades, scoops, and metal pans.
- Thermostatic, forced-draft oven that can maintain any desired temperature from room temperature to at least 165.0 ± 5.0°C (329 ± 9.0°F) in accordance with AASHTO R 30.
- Superpave gyratory compactor for AASHTO T 312.

- Equipment, scales, and water bath for determining the theoretical maximum specific gravity (Gmm) of asphalt mixtures in accordance with FM 1-T 209.
- Scales and water bath for determining bulk specific gravity of asphalt mixtures (Gmb) in accordance with FM 1-T 166.
- Environmental chamber for conditioning asphalt mixture samples at 25.0 ± 1.0°C (77.0 ± 1.8°F) and 50.0 ± 1.0°C (122.0 ± 1.8°F).
- Loading, conditioning, and data acquisition system in accordance with AASHTO T 324.
- Loading and data acquisition system in accordance with ASTM D8225.

6.6 Test Materials

- Submit a minimum of seven pre-batched aggregate specimens of Type SP-12.5 mix that is representative of a mix design currently approved for use in the State of Florida by the Department. Each of the first two batches (referred to as B1 and B2 in Figure 6-1) shall be enough to prepare a minimum of two mix design specimens (150.0 mm in diameter, 115.0 ± 5.0 mm in height, and 4.0 ± 0.5 percent in air void content at the design number of gyrations) and a minimum of two Gmm test specimens and labeled accordingly. Each of the second two batches (referred to as B3 and B4 in Figure 6-1) must be enough to prepare at least four HWT test specimens (150.0 mm in diameter, 62.0 ± 1.0 mm in height, and 7.0 ± 0.5 in air void content) and labeled accordingly. Each of the third two batches (referred to as B5 and B6 in Figure 6-1) must be enough to prepare at least four IDEAL-CT test specimens (150.0 mm in diameter, 62.0 ± 1.0 mm in height, and 7.0 ± 0.5 percent in air void content). The seventh batch (referred to as B7 in Figure 6-1) must weigh the highest of the first six batches and shall be used only if needed during the use of this protocol.
- Submit enough of the AAB additive and crosslinking agent, if applicable, with the appropriate label and recommended required dosages to produce PG 76-22 binder.
- Submit the approved product list number of the Department-approved Superpave asphalt binder required to produce PG 76-22 binder.
- Submit the temperature and the duration required for blending.

6.7 Asphalt Binder Sample Preparation

- Heat enough of the suppler-recommended, Department-approved Superpave (base) asphalt binder at the supplier-recommended temperature.
- Add the AAB additive to the base asphalt binder at the supplier-recommended dosage slowly.
- Blend the base asphalt binder with the AAB additive at the supplier-recommended temperature for a minimum of 2.5 hours for a quarter can or for the supplier-recommended duration, whichever is longer. Adjust the shear speed to create a vortex at the middle of the can to ensure better blending.
- If a crosslinking agent is required, add it to the blend at the supplier-recommended dosage and time, and continue blending for a minimum of 1.0 hour or the supplier-recommended duration, whichever is longer.
- Put a lid on the can of new blend and keep it cured in an oven undisturbed for a minimum of 18.0 hours over one night or for the supplier-recommended duration, whichever is longer, to let the reactions complete.
- Age representative samples of the PMA and the AMA binders in a RTFO according to AASHTO T 240 and PAV according to AASHTO R 28.

6.8 Asphalt Binder Performance Grade Test

- Run the DSR tests on the unaged asphalt binder samples at the high-temperature grade temperature (i.e., 76.0°C) in accordance with AASHTO T 315 and Section 916 of the Department's Specifications for Road and Bridge Construction.
- Run the MSCR tests on the RTFO-aged asphalt binder samples at 67.0°C according to AASHTO T 350 and grade in accordance with AASHTO M 332 and Section 916 of the Department's Specifications for Road and Bridge Construction.
- Run the BBR tests on the 20-hour PAV-aged asphalt binder samples at the low-temperature grade temperature (i.e., -22.0°C) in accordance with AASHTO T 313 and Section 916 of the Department's Specifications for Road and Bridge Construction.

- Run the DSR tests of the 20-hour PAV-aged asphalt binder samples at the intermediate temperature of 26.5°C in accordance with AASHTO T 315 and Section 916 of the Department's Specifications for Road and Bridge Construction.
- Check if the AMA binder passes PG 76-22 designation in accordance with Section 916 of the Department's Specifications for Road and Bridge Construction. If it passes, this binder is referred to as PG 76-22 (AMA) in this protocol hereafter. If it fails, reject the AAB additive.
- Report for each binder that passes the PG tests:
  - Properties of the unaged sample:
    - Rutting resistance parameter: $G^*/Sin\delta$
    - Phase angle: δ
    - High temperature PG at $G^*/Sin\delta$ value of 1.0 kPa: PGH
  - Properties of the RTFO-aged samples:
    - Non-recoverable creep compliance at a shear stress of 3.2 kPa: $Jnr_{3.2}$
    - Percent recovery at a shear stress of 3.2 kPa: $Rec_{3.2}$
    - Percent difference of the non-recoverable creep compliances at shear stress of 0.1 kPa and 3.2 kPa: $\%Jnr_{diff}$
  - Properties of the PAV-aged sample:
    - Creep slope: m
    - Creep stiffness: S
    - Critical low temperature at m-value of 0.300: T_m
    - Critical low temperature at S-value of $300\ MPa$: T_S
    - Low Temperature PG: PGL = maximum (T_S, T_m)
    - Critical low temperature difference: $\Delta Tc = T_S - T_m$
    - Cracking resistance parameter: $G^*.Sin\delta$

6.9 Asphalt Mixture Sample Preparation

- Preheat selected asphalt binder type [PG 76-22 (PMA) or PG 76-22 (AMA) binder] in an oven at the mixing temperature of 154.0°C (309.2°F) for a minimum of 2.0 hours.

- Preheat pre-batched aggregates in an oven at the mixing temperature of 154.0°C (309.2°F) for a minimum of 6.0 hours.
- Mix aggregates with selected asphalt binder for about 3.0 minutes to produce the PMA or the AMA mixtures.
- Age the mixture for two hours at the compaction temperature of 154.0°C (309.2°F) according to AASHTO R 30. Stir the mixture at approximately the 1.0-hour mark for uniform aging.
- Determine Gmm in accordance with FM 1-T 209.
- Compact the mix in a cylindrical sample mold with an inside diameter of 150.0 mm (until the design number of gyrations to prepare the mix design specimens or until a height of 62.0 ± 1.0 mm to prepare the HWT and the IDEAL-CT test specimens).
- Cut the SGC specimens prepared for the HWT test along the secant lines (or chords) perpendicular to compaction direction so that two specimens could be mounted together on the HWT molds without any space between the cut edges according to AASHTO T 324.
- Do not cut the SGC specimens prepared for the IDEAL-CT test.
- Determine Gmb of the HWT (i.e., one-edge cut) and the IDEAL-CT (uncut) test specimens in accordance with FM 1-T 166.
- Determine the air void content (Va) in each specimen:

$$Va\ (\%) = \frac{Gmm - Gmb}{Gmm} \times 100\%$$

- Confirm the SGC specimens prepared for mix design verification have 4.0 ± 0.5 percent air void content (96.0 ± 0.5%Gmm) at N_{design} and the test specimens prepared for the HWT and the IDEAL-CT tests have 7.0 ± 0.5 percent air void (93.0 ± 0.5%Gmm) according to AASHTO T 312.

6.10 Asphalt Mixture Rutting Test

- Select the one-edge cut specimens that measure 62.0 ± 1.0 mm in height, 150.0 mm in diameter, and 7.0 ± 0.5 percent in air void content for the HWT test.
- Place the specimens in two molds, each containing two specimens, side-by-side in the HWT test instrument along the two wheel paths, and condition them at a

temperature of 50.0 ± 0.5°C (122.0 ± 0.9°F) under water for 45 minutes after the water has reached this temperature following AASHTO T 340.

- Run the HWT test by subjecting the specimens to wheel loads of 705.0 ± 4.5 N (158.5 ± 1.0 lb) at the rate of 52.0 passes/minute for either a minimum of 20,000 passes or a maximum of 12.5-mm rut depth, whichever comes first, following AASHTO T 340.
- Use the data acquisition system of the HWT test instrument to record the rut depth (d) versus wheel pass (N) data along each wheel path.
- Determine the rut depth at 20,000 passes ($d_{20,000}$) or the total number of wheel-passes to achieve a 12.5-mm rut depth ($N_{12.5\,mm}$), whichever comes first.
- Report the average and the standard deviation values of:
  - Gmm,
  - Gmb,
  - Va,
  - $d_{20,000}$, and
  - $N_{12.5}$.

6.11 Asphalt Mixture Cracking Test

- Select the uncut specimens that measure 62.0 ± 1.0 mm in height, 150.0 mm in diameter, and 7.0 ± 0.5 percent in air void content for the IDEAL-CT test.
- Place them in an environmental chamber and condition at a temperature of 25.0 ± 0.5°C (77.0 ± 1.0°F) without water for 2.5 hours ± 5.0 min before testing.
- Remove each specimen from the environmental chamber, one at a time, and within 2.0 minutes, correctly position the specimens on the IDEAL-CT test loading frame.
- Run the IDEAL-CT test immediately by subjecting the test specimens to the load-line displacement (LLD) rate of 50.0 ± 2.0 mm/min until the load drops below 0.1 kN following ASTM D8225.
- Use the data acquisition system of the IDEAL-CT test instrument to extract the load versus LLD (i.e., P versus u) data and determine the cracking tolerance (CT) index:

$$CT_{index} = \left(\frac{u_{75}}{D}\right) \times \left(\frac{G_f}{|m_{75}|}\right) \times 10^6$$

where,

$$G_f = \left(\frac{W_f}{A}\right) \times 10^6$$

$$W_f = \int_0^{u_{final}} P(u).du$$

$$A = D \times t$$

where,

CT_{index} = cracking tolerance index

G_f = fracture energy = energy required to create a unit area of a crack (J/m2)

m_{75} = the slope at 75% the peak load after the peak (N/mm)

u_{75}/D = a strain tolerance parameter

u_{75} = LLD recorded at 75% the peak load after the peak (mm)

W_f = work of fracture = work done to create a unit area of a crack (J or Nm)

P = load (kN)

u = LLD (mm)

u_{final} = LLD recorded at the 0.1 kN cut-off load (mm)

A = effective area (mm^2)

D = diameter of test specimen (mm)

- Report the average and the standard deviation values for each mixture:
  - Gmm,
  - Gmb,
  - Va, and
  - CT_{index}.

6.12 Approval Decisions

- Approve the AMA binder (or the corresponding AAB additive) if the parameters obtained from the AMA and the PMA binders satisfy each of these criteria:
  - In the unaged condition:
    - $PGH_{AMA} \geq PGH_{PMA}$

- In the RTFO-aged condition:
  - $Jnr_{3.2,AMA} \leq Jnr_{3.2,PMA}$
  - $(Grade)_{AMA} \geq (Grade)_{PMA}$
  - $Rec_{3.2,AMA} \geq Rec_{3.2,PMA}$

- In the PAV-aged condition:
  - $PGL_{AMA} \leq PGL_{PMA}$
  - $\Delta Tc_{AMA} \geq \Delta Tc_{PMA}$
  - $(G^*.Sin\delta)_{AMA} \leq (G^*.Sin\delta)_{PMA}$

- Approve the AMA mixture (or the corresponding AAB additive) if the values of the parameters obtained from the AMA and the PMA mixtures satisfy each of these criteria:
  - $d_{20,000;AMA} \leq d_{20,000;PMA}$ or $N_{12.5,AMA} \geq N_{12.5,PMA}$, and
  - $(CT_{index})_{AMA} \geq (CT_{index})_{PMA}$

7 ALTERNATIVE ASPHALT MIXTURE ADDITIVE APPROVAL PROTOCOL

7.1 Scope

This method covers the laboratory production of the Superpave gyratory compacted (SGC) specimens of the polymer-modified asphalt (PMA) and the alternatively modified asphalt (AMA) (such as fiber-modified) mixtures, and the laboratory measurement of their rutting and cracking resistances. The results of these tests will be used for evaluating the effects of the alternative asphalt mixture (AAM) additives on rutting and cracking resistance of the PMA mixture and for approving the AAM additives. The values stated in the International System (SI) of units are to be regarded as the standard.

7.2 Referenced Documents

AASHTO Standards

- AASHTO R 30 Standard Practice for Mixture Conditioning of Hot-Mix Asphalt (HMA)
- AASHTO T 312 Standard Method for Preparing and Determining the Density of Hot-Mix Asphalt (HMA) by Means of the Superpave Gyratory Compactor
- AASHTO T 324 Standard Method of Test for Hamburg Wheel-Track Testing of Compacted Asphalt Mixtures

ASTM Standards

- ASTM D8225 Standard Test Method for Determination of Cracking Tolerance Index of Asphalt Mixture Using the Indirect Tensile Cracking Test at Intermediate Temperature

Florida Method of Tests

- FM 1-T 166 Bulk Specific Gravity of Compacted Bituminous Mixtures
- FM 1-T 209 Maximum Specific Gravity of Bituminous Paving Mixtures

7.3 Significance and Use

- This method will be used for evaluating the effects of the AAM additives on the rutting and cracking resistance of the PMA mixture and for approving the AAM additives.

7.4 Summary of Method

Pre-batched aggregate specimens, pre-packaged AAM additive, and the step-by-step protocol for modifying an asphalt mixture with an AAM additive are obtained from the supplier of the AAM additive.

The PMA mixture is prepared by mixing the aggregates with PG 76-22 (PMA) binder alone while the AMA mixture is prepared by mixing the aggregates with either PG 76-22 (PMA) or PG 76-22 (AMA) binder [i.e., AMA binder with PG 76-22) and the AAM additive together.

The SGC specimens of the PMA and the AMA test specimens are then subjected to the Hamburg Wheel-Track (HWT) and the Ideal Cracking Tolerance (IDEAL-CT) tests to measure their rutting and cracking resistance parameters. A minimum of four test specimens are used from each mixture in each test (see Figure 7-1). Approval of the AAM additive is contingent upon whether the AMA mixture satisfies the criteria set for both rutting and cracking resistance parameters.

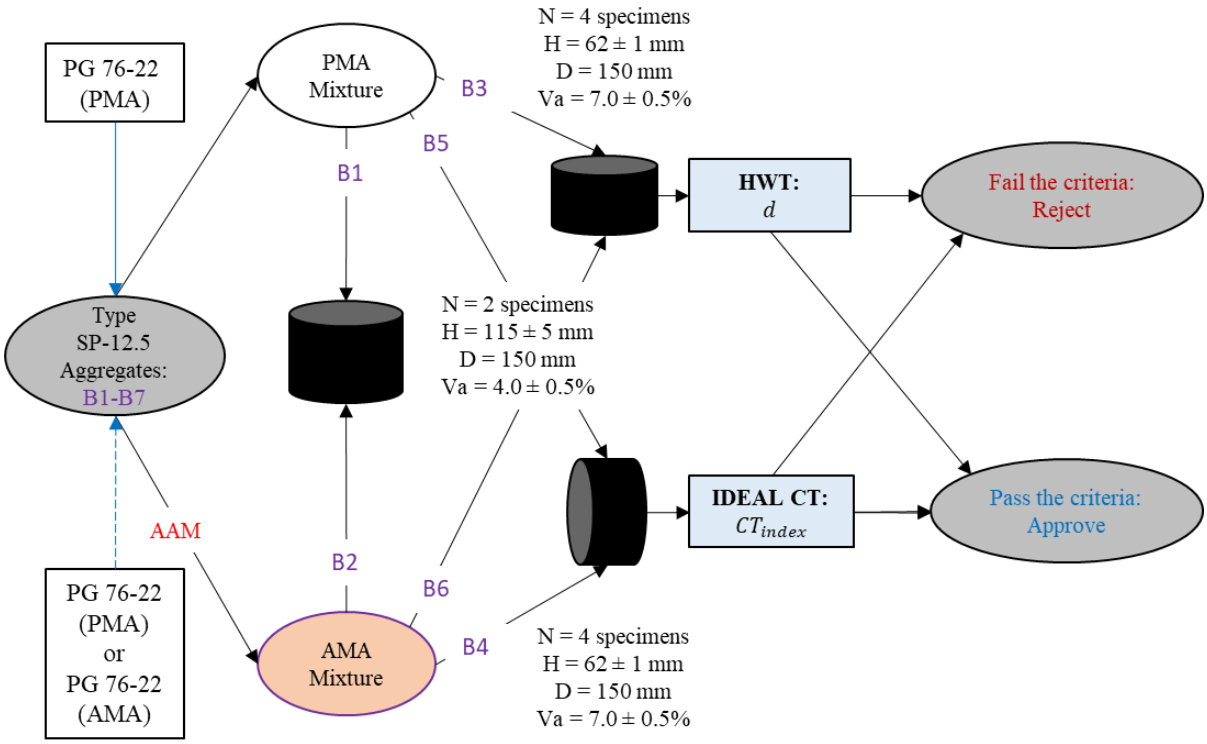


Figure 7-1. AAM Additive Approval Protocol Illustration

7.5 Test Apparatus

- Bucket and paddle mixer, blades, scoops, and metal pans.
- Thermostatic, forced-draft oven that can maintain any desired temperature from room temperature to at least 165.0 ± 5.0°C (329 ± 9.0°F) in accordance with AASHTO R 30.
- Superpave gyratory compactor for AASHTO T 312.
- Equipment, scales, and water bath for determining the theoretical maximum specific gravity (Gmm) of asphalt mixtures in accordance with FM 1-T 209.
- Scales and water bath for determining bulk specific gravity of asphalt mixtures (Gmb) in accordance with FM 1-T 166.
- Environmental chamber for conditioning asphalt mixture samples at 25.0 ± 1.0°C (77.0 ± 1.8°F) and 50.0 ± 1.0°C (122.0 ± 1.8°F).
- Loading, conditioning, and data acquisition system in accordance with AASHTO T 324 test.
- Loading and data acquisition system in accordance with ASTM D8225 test.

7.6 Test Materials

- Submit a minimum of seven pre-batched aggregate specimens of Type SP-12.5 mix that is representative of a mix design currently approved for use in the State of Florida by the Department. Each of the first two batches (referred to as B1 and B2 in Figure 7-1) shall be enough to prepare a minimum of two mix design specimens (150.0 mm in diameter, 115.0 ± 5.0 mm in height, and 4.0 ± 0.5 percent in air void content at the design number of gyrations) and a minimum of two Gmm test specimens and labeled accordingly. Each of the second two batches (referred to as B3 and B4 in Figure 7-1) must be enough to prepare at least four HWT test specimens each (150.0 mm in diameter, 62.0 ± 1.0 mm in height, and 7.0 ± 0.5 percent in air void content) and labeled accordingly. Each of the third two batches (referred to as B5 and B6 in Figure 7-1) must be enough to prepare at least four IDEAL-CT test specimens each (150.0 mm in diameter, 62.0 ± 1.0 mm in height, and 7.0 ± 0.5 percent in air void content). The seventh batch (referred to as B7 in

Figure 7-1) must weigh the highest of the first six batches and shall be used only if needed during the use of this protocol.

- Submit a pre-weighed package of the AAM additive with the appropriate label.
- Submit the protocol to add the AAM additive into the mixture.

7.7 Asphalt Mixture Sample Preparation

- Preheat PG 76-22 (PMA) or PG 76-22 (AMA) binder in an oven at the mixing temperature of 154.0°C (309.2°F) for a minimum of 2.0 hours.
- Preheat pre-batched aggregates in an oven at the mixing temperature of 154.0°C (309.2°F) for a minimum of 6.0 hours. (Note: To produce the AMA mixture, use a slightly higher temperature and a different oven to preheat the aggregates if recommended by the supplier.)
- Mix aggregates with asphalt binder alone for about 3.0 minutes to produce the PMA mixture. [To produce the AMA mixture, follow the supplier-provided protocol to add the pre-packaged weight of the AAM additive into the mix.]
- Age the mixture for two hours at the compaction temperature of 154°C (309.2°F) according to AASHTO R 30. Stir the mixture at approximately the 1.0-hour mark for uniform aging.
- Determine Gmm according to FM 1-T 209.
- Compact the mix in cylindrical sample molds with an inside diameter of 150.0 mm (until the design number of gyrations to prepare the mix design specimens or until a height of 62.0 ± 1.0 mm to prepare the HWT and the IDEAL-CT test specimens).
- Cut the SGC specimens prepared for the HWT test along the secant lines (or chords) perpendicular to compaction direction so that two specimens could be mounted together on the HWT molds without any space between the cut edges according to AASHTO T 324.
- Do not cut the SGC specimens prepared for the IDEAL-CT test.
- Determine Gmb of the HWT (i.e., one-edge cut) and the IDEAL-CT (uncut) test specimens by following FM 1-T 166.
- Determine the air void content (Va) in each specimen:

$$Va\ (\%) = \frac{Gmm - Gmb}{Gmm} \times 100\%$$

- Confirm the SGC specimens prepared for mix design verification have 4.0 ± 0.5 percent air void content (96.0 ± 0.5%Gmm) at N_{design} and the test specimens prepared for the HWT and the IDEAL-CT tests have 7.0 ± 0.5 percent air void content (93.0 ± 0.5%Gmm) according to AASHTO T 312.

7.8 Asphalt Mixture Rutting Test

- Select the one-edge cut specimens that measure 62.0 ± 1.0 mm in height, 150.0 mm in diameter, and 7.0 ± 0.5 percent in air void content for the HWT test.
- Place the specimens in two molds, each containing two specimens, side-by-side in the HWT test instrument along the two wheel paths, and condition them at a temperature of 50.0 ± 0.5°C (122.0 ± 0.9°F) under the water for 45 minutes after the water has reached this temperature following AASHTO T 340.
- Run the HWT test by subjecting the specimens to wheel loads of 705.0 ± 4.5 N (158.5 ± 1.0 lb) at a rate of 52.0 passes/minute for either a minimum of 20,000 passes or a maximum of 12.5-mm rut depth, whichever comes first, following AASHTO T 340.
- Use the data acquisition system of the HWT test instrument to record the rut depth (d) versus wheel pass (N) data along each wheel path.
- Determine the rut depth at 20,000 passes ($d_{20,000}$) or the total number of wheel passes to achieve a 12.5-mm rut depth ($N_{12.5}$), whichever comes first.
- Report the average and the standard deviation values of:
  - Gmm,
  - Gmb,
  - Va,
  - $d_{20,000}$, and
  - $N_{12.5}$.

7.9 Asphalt Mixture Cracking Test

- Select the uncut specimens that measure 62.0 ± 1.0 mm in height, 150.0 mm in diameter, and 7.0 ± 0.5 percent in air void content for the IDEAL-CT test.

- Place them in an environmental chamber and condition at a temperature of 25.0 ± 0.5°C (77.0 ± 1.0°F) without water for 2.5 hours ± 5.0 min before testing.
- Remove each specimen from the environmental chamber, one at a time, and within 2.0 minutes, correctly position the specimens on the IDEAL-CT test loading frame.
- Run the IDEAL-CT test immediately by subjecting test specimens to the load-line displacement (LLD) rate of 50.0 ± 2.0 mm/min until the load drops below 0.1 kN following ASTM D8225.
- Use the data acquisition system of the IDEAL-CT test instrument to extract the load versus LLD (i.e., P versus u) data and determine the cracking tolerance (CT) index:

$$CT_{index} = \left(\frac{u_{75}}{D}\right) \times \left(\frac{G_f}{|m_{75}|}\right) \times 10^6$$

where,

$$G_f = \left(\frac{W_f}{A}\right) \times 10^6$$

$$W_f = \int_0^{u_{final}} P(u).du$$

$$A = D \times t$$

where,

CT_{index} = cracking tolerance index

G_f = fracture energy = energy required to create a unit area of a crack (J/m2)

m_{75} = the slope at 75% peak load after the peak (N/mm)

u_{75}/D = a strain tolerance parameter

u_{75} = LLD recorded at 75% peak load after the peak (mm)

W_f = work of fracture = work done to create a unit area of a crack (J or Nm)

P = load (kN)

u = LLD (mm)

u_{final} = LLD recorded at the 0.1 kN cut-off load (mm)

A = effective area (mm^2)

D = diameter of test specimen (mm)

- Report the average and the standard deviation values for each mixture:
  - Gmm,
  - Gmb,
  - Va, and
  - CT_{index}.

7.10 Approval Decisions

- Approve the AMA mixture (or the corresponding AAM additive) if the values of the parameters obtained from the AMA and the PMA mixtures satisfy each of these criteria:
  - $d_{20,000;AMA} \leq d_{20,000;PMA}$ or $N_{12.5,AMA} \geq N_{12.5,PMA}$, and
  - $(CT_{index})_{AMA} \geq (CT_{index})_{PMA}$

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