May 2023

Report No. 23-045

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Optimizing of MassDOT's High Performance Asphalt Overlay (HPOL) Mixtures

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U.S. Department of Transportation
Federal Highway Administration

Technical Report Document Page

<table>
<tr><td>1. Report No.
23-044</td><td>2. Government Accession No.
n/a</td><td colspan="2">3. Recipient's Catalog No.
n/a</td></tr>
<tr><td colspan="2" rowspan="2">4. Title and Subtitle
Optimizing of MassDOT's High Performance Asphalt Overlay (HPOL) Mixtures</td><td colspan="2">5. Report Date
September 2023</td></tr>
<tr><td colspan="2">6. Performing Organization Code
n/a</td></tr>
<tr><td colspan="2">7. Author(s)
Walaa S. Mogawer, Alexander J. Austerman, Kevin D. Stuart and Ibrahim M. Abdalfattah</td><td colspan="2">8. Performing Organization Report No.
23-044</td></tr>
<tr><td colspan="2" rowspan="2">9. Performing Organization Name and Address
University of Massachusetts Dartmouth
285 Old Westport Road
North Dartmouth, MA 02747</td><td colspan="2">10. Work Unit No. (TRAIS)
n/a</td></tr>
<tr><td colspan="2">11. Contract or Grant No.</td></tr>
<tr><td colspan="2" rowspan="2">12. Sponsoring Agency Name and Address
Massachusetts Department of Transportation
Office of Transportation Planning
Ten Park Plaza, Suite 4150, Boston, MA 02116</td><td colspan="2">13. Type of Report and Period Covered
Final Report - September 2023
[July 2021 – September 2023]</td></tr>
<tr><td colspan="2">14. Sponsoring Agency Code
n/a</td></tr>
<tr><td colspan="4">15. Supplementary Notes
Project Champion – Edmund Naras, MassDOT & Mark Brum, MassDOT</td></tr>
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The FHWA Targeted Overlay Pavement Solutions (TOPS) program presents innovative overlay solutions for asphalt and concrete pavements. The objective of this study was to conduct a comparative evaluation of three TOPS types for asphalt pavements (Asphalt Rubber Gap-Graded, Stone Matrix Asphalt, and High-Performance thin overlay) in terms of performance, how they extend pavement service life, and whether they can be used interchangeably. Asphalt binders from two sources were used in each of these three TOPS types, providing six TOPS mixtures. All TOPS passed the mixture performance tests where the test had a MassDOT specified or a preliminary criterion, and the test results indicated that all three TOPS types can be used interchangeably. All six TOPS mixtures met MassDOT's pilot specification criteria for rutting, moisture damage, and intermediate-temperature cracking. They also passed the established mixture performance tests for reflective cracking and raveling. AASHTOWare PMED predictive models for bottom-up fatigue cracking indicated that 2 in. of a TOPS mixture placed on an existing in-service pavement can extend pavement service life between 11.2 and 14 years, depending on the type of TOPS and source of binder. The maximum difference in service life between the six TOPS was slightly less than 3 years, although most of them were within 2 years of each other. None of the three TOPS types could be chosen as the best TOPS.</td></tr>
<tr><td colspan="2">17. Key Word
Targeted Overlay Pavement Solutions, TOPS, Asphalt Rubber Gap-Graded, ARGG, Stone Matrix Asphalt, SMA, High-Performance Thin Overlay, HPTO, Performance, Service Life, Cost</td><td colspan="2">18. Distribution Statement
unrestricted</td></tr>
<tr><td>19. Security Classif. (of this report)
unclassified</td><td>20. Security Classif. (of this page)
unclassified</td><td>21. No. of Pages
63</td><td>22. Price
n/a</td></tr>
</table>

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Optimizing of MassDOT's High Performance Asphalt Overlay (HPOL) Mixtures

Final Report

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September 2023

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Acknowledgments

Prepared in cooperation with the Massachusetts Department of Transportation, Office of Transportation Planning, and the United States Department of Transportation, Federal Highway Administration.

The Project Team would like to acknowledge the efforts of Edmund Naras (MassDOT Pavement Management Engineer) and Mark J. Brum (MassDOT Materials Quality Systems Engineer).

Disclaimer

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Executive Summary

This study of Optimizing of MassDOT's High Performance Asphalt Overlay (HPOL) Mixtures was undertaken as part of the Massachusetts Department of Transportation Research Program. This program is funded with Federal Highway Administration (FHWA) State Planning and Research (SPR) funds. Through this program, applied research is conducted on topics of importance to the Commonwealth of Massachusetts transportation agencies.

The FHWA Targeted Overlay Pavement Solutions (TOPS) program presents innovative overlay solutions for both asphalt and concrete pavements. It encourages transportation agencies to consider using underutilized types of overlays that have proven effective in enhancing the performance of high-priority or high-maintenance pavements such as interstate pavements, intersections, bus lanes, ramps, and curves. For asphalt pavements, the FHWA has presented the following eight TOPS: asphalt rubber gap-graded (ARGG), crack attenuating mix (CAM), enhanced friction overlay, high-performance thin overlay (HPTO), open-graded friction course (OGFC), stone matrix asphalt (SMA), ultra-thin bonded wearing course (UTBWC), and highly modified asphalt (HiMA).

The Massachusetts Department of Transportation (MassDOT) wanted to proactively follow the recommendation and lead of the FHWA by using more of these types of overlay solutions. This study was conducted to investigate some of the overlays from the FHWA TOPS list of eight in terms of their performances, their potential to extend pavement service life, cost, and whether they can be used interchangeably. MassDOT selected three TOPS (ARGG, SMA, and HPTO) from the FHWA list of eight for this study. The ARGG and HPTO overlays were selected because MassDOT had some previous experience using them. SMA was selected because it was noted by the FHWA that several State Transportation Agencies (STAs) have adopted SMA due to its increased service life and performance.

The ARGG TOPS is a gap-graded asphalt mixture that typically has a maximum aggregate size of 3/8 or 1/2 in. The binder used in an ARGG mixture is typically a rubber-modified asphalt binder (AR) incorporating approximately 20% ground tire rubber. ARGG has been reported to be very durable with resistance to reflective cracking, thermal cracking, and rutting, while also exhibiting good friction resistance.

The SMA TOPS is a gap-graded asphalt mixture that provides a durable wearing course. It consists of a stable stone-on-stone coarse aggregate skeleton and a rich asphalt binder content along with a stabilizing agent such as fibers and/or asphalt modifiers. SMA's stone-on-stone coarse aggregate skeleton can reduce rutting while the rich asphalt binder content can reduce cracking.

The HPTO TOPS is a fine-graded polymer-modified asphalt mixture. It is generally placed at a thickness of 1 in. HPTO mixtures have been reported to improve cracking and rutting resistance and extend pavement life. Previous attempts at using HPTO in Massachusetts had been called a high-performance surface course (HPSC) or simply high performance (HP). For clarity, the acronym HP will be used in this study to be consistent with MassDOT nomenclature.

The objective of this study was to conduct a comparative evaluation of three MassDOT selected TOPS types for asphalt pavements, namely ARGG, SMA, and HP in terms of performance, how they extend pavement service life, cost, and whether they can be used interchangeably. Asphalt binders from two sources were used in each of these three TOP types, thus, providing a total of six TOPS mixtures.

All TOPS passed the mixture performance tests where the test had a MassDOT specified or preliminary criterion, and the results indicated that the three TOPS types can be used interchangeably. More specifically, all TOPS met MassDOT's pilot specification criteria for rutting, moisture damage, and intermediate-temperature cracking. Tests for reflective cracking and raveling indicated that all TOPS passed the criterion established by other STAs.

Statistical analysis indicated that both TOPS type and binder source had significant effects on the mixture performance tests except for the overlay test critical fracture energy (CFE) results where the binder source did not have a significant effect.

AASHTOWare PMED predictive models for bottom-up fatigue cracking indicated that 2 in. of a TOPS placed on an existing in-service pavement can extend pavement service life between 11.2 and 14 years depending on the type of TOPS and source of binder. Thus, the maximum difference between the six TOPS mixtures was slightly less than 3 years, although most of them were within 2 years of each other. Furthermore, none of the three TOPS types could be chosen as the best.

All TOPS passed the mixture performance tests where there was a MassDOT specified or preliminary criterion, while the AASHTOWare PMED predictive models did not show that any of the three TOPS types would perform better or worse than the others based on bottom-up fatigue and thermal cracking. Overall, the data did not indicate one of the three TOPS was superior to the others. Finally, Life Cycle Cost Analysis (LCCA) indicated there was no significant difference in the net present value costs for the three TOPS.

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List of Acronyms

Acronym	Expansion
AADT	Average annual daily truck traffic
AASHTO	American Association of State Highway and Transportation Officials
ABCD	Asphalt binder cracking device
AMPT	Asphalt mixture performance tester
ANOVA	Analysis of variance
AR	Asphalt rubber
ARGG	Asphalt rubber gap-graded
BBR	Bending beam rheometer
BMD	Balanced mixture design
CAI	Center for accelerating innovation
CAM	Christensen-Anderson Model
CFE	Critical fracture energy
CPR	Crack progression rate
CTOD	Critical crack tip opening displacement
DENT	Double-edge-notched tension
DOT	Department of transportation
DSR	Dynamic shear rheometer
E*	Dynamic modulus
EBBR	Extended bending beam rheometer
EDC	Everyday counts
FHWA	Federal Highway Administration
FI	Flexibility index
FIT	Flexibility index test
HiMA	Highly modified asphalt
HMA	Hot mix asphalt
HP	High performance
HPOL	High performance asphalt overlay
HPSC	High performance surface course
HPTO	High performance thin overlay
HWTT	Hamburg wheel tracking test
IDEAL-CT	Indirect tensile asphalt cracking test
J_{nr}	Non-recoverable creep compliance
LCA	Life-cycle assessments
LCCA	Life cycle cost analysis
LLD	Load line displacement
LTLG	Low-temperature limiting grade
MassDOT	Massachusetts Department of Transportation
MSCR	Multiple stress creep recovery
NMAS	Nominal maximum aggregate size
NPV	Net present value
OGFC	Open-graded friction course
OT	Overlay test
PAV	Pressurized aging vessel
PMED	Pavement mechanistic-empirical design

Acronym	Expansion
PG	Performance grade
PMA	Polymer-modified asphalt
R-Value	Rheological index
RAP	Reclaimed asphalt pavement
RTFO	Rolling thin-film oven
SCB	Semicircular bend geometry
SGC	Superpave gyratory compactor
SIP	Stripping inflection point
SMA	Stone matrix asphalt
SPR	State planning and research
STA	State transportation agency
TOM	Thin overlay mixture
TOPS	Targeted overlay pavement solutions
TSRST	Thermal stress restrained specimen test
UTBWC	Ultra-thin bonded wearing course
VMA	Voids in mineral aggregate

1.0 Introduction and Objective

This study entitled "Optimizing of MassDOT's High Performance Asphalt Overlay (HPOL) Mixtures" was undertaken as part of the Massachusetts Department of Transportation (MassDOT) Research Program. This program is funded with Federal Highway Administration (FHWA) State Planning and Research (SPR) funds. Through this program, applied research is conducted on topics of importance to the Commonwealth of Massachusetts transportation agencies.

1.1 Introduction

Targeted Overlay Pavement Solutions (TOPS) are described by the FHWA as "Solutions for integrating innovative overlay procedures into practices that can improve performance, lessen traffic impacts, and reduce the cost of pavement ownership" (*1*). These TOPS are part of the FHWA Everyday Counts (EDC) program, which falls under the FHWA Center for Accelerating Innovation (CAI) (*2*). In 2012, the FHWA established the CAI to identify and prioritize innovations for a variety of highway transportation needs (*2*). The EDC program was established to identify and rapidly deploy underutilized innovations that have been proven effective by the transportation agencies using them (*3*). Every two years, the FHWA collaborates with American Association of State Highway and Transportation Officials (AASHTO), State Transportation Agencies (STAs), local governments, tribes, private industry, and other stakeholders to identify a new collection of proven innovations that merit rapid deployment (*3*). The innovations promoted through EDC are selected with respect to their ability to save transportation agencies time, money, and resources (*3*). In the EDC sixth round (EDC-6) for 2021–2022, seven new innovations were identified, with one of those innovations being TOPS (*1*).

The FHWA states, "Approximately half of all infrastructure dollars are invested in pavements, and more than half of that investment is in overlays" (*1*). Because of this large investment being placed into overlays, it is integral that their designs be optimized in terms of performance, traffic impacts, motorist safety, and costs. The FHWA TOPS presents innovative overlays for both asphalt pavements and concrete pavements. These overlay solutions can be integrated into current STA practices as an alternative to pavement reconstruction to help extend pavement life, increase load-carrying capacities, increase user safety, enhance mobility and satisfaction, and reduce the cost of pavement ownership (*4*). The TOPS program encourages transportation agencies to consider using underutilized types of overlays that have proven effective in enhancing the performance of high-priority or high-maintenance pavements such as interstate pavements, intersections, bus lanes, ramps, and curves (*4*). Therefore, TOPS are specialized and inherently different from overlays that are commonly used within the industry. For asphalt pavements, the FHWA has presented the following eight TOPS (*5*) with thorough descriptions in the accompanying references: Asphalt Rubber Gap-Graded (ARGG) (*6*), Crack Attenuating Mix (*7*), Enhanced Friction Overlay (*8*), High-Performance Thin Overlay (HPTO) (*9,10*), Open-Graded Friction Course (OGFC) (*11*), Stone

Matrix Asphalt (SMA) (*12*), Ultra-Thin Bonded Wearing Course (UTBWC) (*13*), and Highly Modified Asphalt (HiMA) (*14*).

For this study, MassDOT wanted to proactively follow the recommendation and lead of the FHWA by using more of these types of overlay solutions. Prior to the EDC program, MassDOT had some experience using ARGG, HPTO, and OGFC. Moving forward, MassDOT wanted to further expand its selection of strategies so they had more available options. By exploring other overlay alternatives, MassDOT gains more flexibility in specifying an overlay so that existing field, production, or construction limitations may be most appropriately addressed and overcome.

The purpose of this study was to investigate, for MassDOT, some of the overlays from the FHWA TOPS list of eight in terms of their performance, their potential to extend pavement service life, and whether they can be used interchangeably. Three TOPS were selected by MassDOT from the available eight, which were ARGG, SMA, and HPTO. The ARGG and HPTO overlays were selected because MassDOT had some previous experience using them. SMA was selected because it was noted by the FHWA that several STAs have adopted SMA due to its increased service life and performance. Maryland, Alabama, and Utah STAs each used over 1 million tons of SMA during a 5-year period (*1*). This positive experience by other STAs was a driving force for its selection for this study. Besides understanding their performance and pavement service life implications, MassDOT wanted to assess if these three TOPS could be used interchangeably. If they can, this would give them greater flexibility in their specifications by potentially allowing contractors to select and bid their preferred overlay option based on their available materials, expertise, and capabilities. This theoretically has the potential to reduce overall construction costs because more options might allow for greater competition and lower bids.

For the three selected TOPS (ARGG, SMA, and HPTO), a pilot specification was developed by MassDOT prior to commencing this study. ARGG TOPS is a gap-graded asphalt mixture that typically has a maximum aggregate size of 3/8 or 1/2 in. (*6*). The binder used in an ARGG mixture is typically a rubber-modified asphalt binder (AR) incorporating approximately 20% ground tire rubber (*6*). Because this mixture has a greater volume of binder due to the rubber particles, the aggregate gradation is gap-graded so that this greater volume of binder can be incorporated while maintaining stone-on-stone contact. ARGG has been reported to be durable with resistance to reflective cracking, thermal cracking, and rutting while also exhibiting good friction resistance (*6*). An ARGG mixture is typically placed at 1.25 to 2.25 in. in thickness (*6*).

SMA TOPS is a gap-graded asphalt mixture that was first developed in Germany in the 1960s to provide a durable wearing course that is resistant to damage from studded tires for heavily traveled pavements (*12*). It consists of a stable stone-on-stone coarse aggregate skeleton and a rich asphalt binder content along with a stabilizing agent such as fibers and/or asphalt modifiers. SMA’s stone-on-stone coarse aggregate skeleton can reduce rutting while the rich asphalt binder content can reduce cracking (*15*). SMA mixtures are typically placed where heavy traffic or high stress is anticipated (*12*).

HPTO TOPS is a fine-graded polymer-modified asphalt mixture. It is generally placed at a thickness of 1 in. (*9*). HPTO mixtures have been reported to improve cracking and rutting resistances and extend pavement life (*9*). Texas has a version of HPTO referred to as Thin Overlay Mixture (TOM) (*10*). The previous attempts at using HPTO in Massachusetts had been called a High Performance Surface Course (HPSC) or simply High Performance (HP). For clarity, the acronym HP will be used in this study to be consistent with MassDOT nomenclature.

1.2 Objective

The objective of this study was to conduct a comparative evaluation of three MassDOT selected TOPS—namely, ARGG, SMA, and HP—in terms of performance, how they extend pavement service life, and whether they can be used interchangeably. To accomplish this objective, the following tasks were undertaken:

1. Conduct a mixture design for an ARGG, SMA, and HP. Each design was developed using two asphalt binders that had the same PG but were obtained from two different sources.

2. Measure the rheological and performance characteristics of the asphalt binders.

3. Measure the performance characteristics of the asphalt mixtures.

4. Determine the impact of binder source on asphalt binder and mixture performance.

5. Using predictive models, determine how the three selected TOPS extend pavement service life.

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2.0 Methodology

2.1 Scope of Work

The study presented herein focused on evaluating the performances of the following three EDC-6 asphalt TOPS selected by MassDOT: ARGG, SMA, and HP. First, a mixture design for each of them was performed in accordance with the pilot specifications developed by MassDOT. Each mixture was developed using the specified asphalt binder by MassDOT that was obtained from two different sources within the state. The ARGG mixtures were developed using two different AR binders, while the SMA and HP mixtures were developed using two different polymer-modified asphalts (PMA) having the same performance grade (PG). This was done in an effort to capture any differences in mixture performance due to binder source. Next, each binder was thoroughly characterized and tested. The PG was determined and then each binder was subjected to a suite of tests to determine their resistances to (1) low-temperature, intermediate-temperature, and non-load-associated cracking; and (2) rutting. Performance testing of the mixtures was then evaluated in the laboratory in terms of each TOPS's resistance to (1) low-temperature, intermediate-temperature, non-load-associated, and reflective cracking; (2) rutting; and (3) raveling.

Statistical analysis of all collected data was completed to determine if the three TOPS performed similarly and could potentially be used interchangeably. A statistical analysis was also utilized to determine if changes in the binder source significantly impacted performance.

Finally, the laboratory collected data and AASHTOWare Pavement Mechanistic-Empirical Design (PMED) software predictive models were used to predict the potential added pavement service life when using each of the three selected TOPS.

2.2 Experimental Plan

To address the objectives of this study, an experimental plan was developed as shown in Figure 2.1

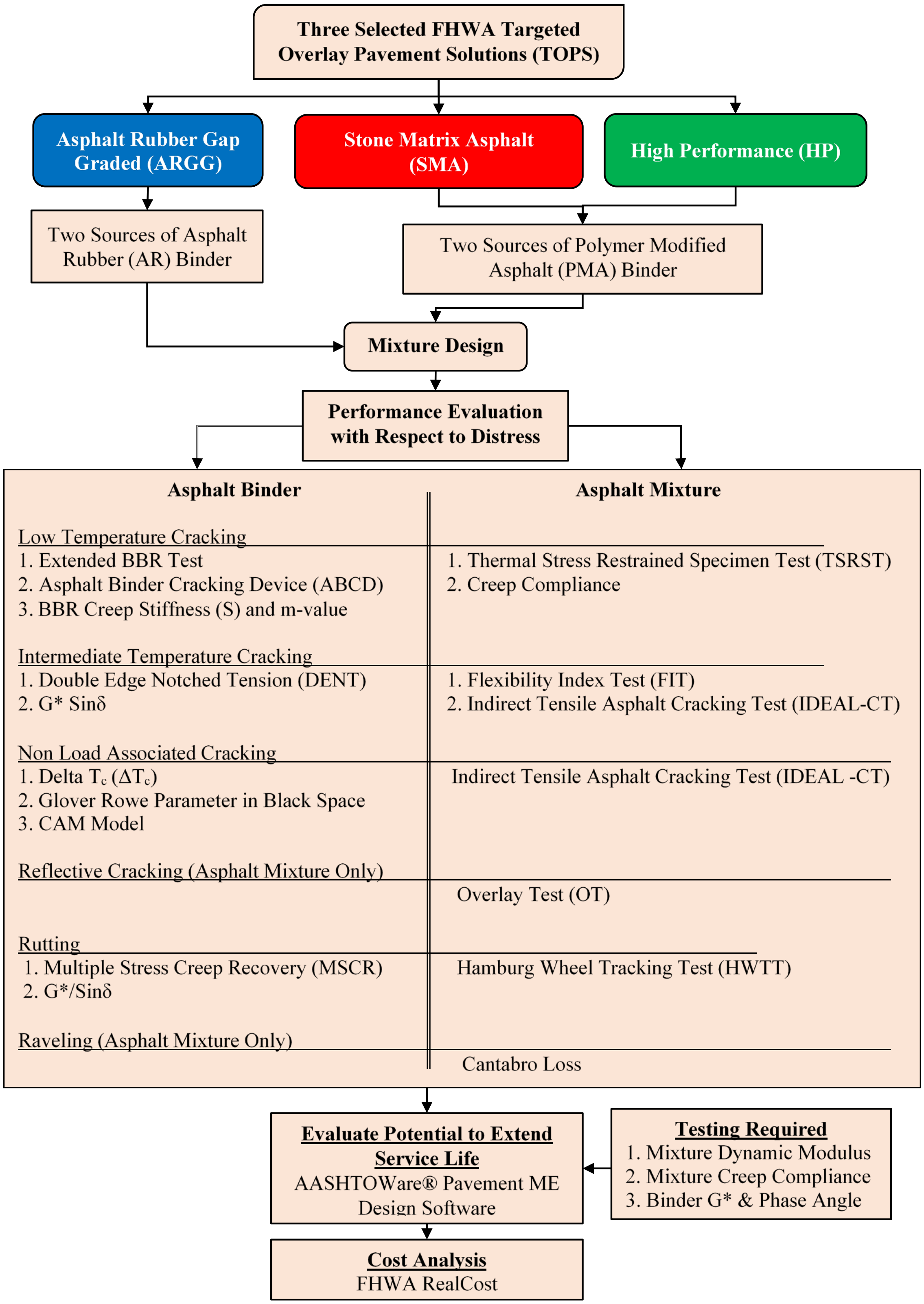

Figure 2.1: Experimental plan

3.0 Materials

3.1 Asphalt Binders

Two AR binders, denoted as AR1 and AR2, and two PMA binders, denoted as PMA1 and PMA2, being used in Massachusetts were obtained for this study. The base binder used to fabricate these AR and PMA binders were also obtained. Two sources of the AR and PMA were sampled because research studies have shown that asphalt binders that have the same PG, but obtained from different suppliers/refineries, could have significantly different performance characteristics (*16*). Hence, the performances of the resultant asphalt mixture might change significantly if the source of asphalt binder is changed during the mix design and/or the production process, which is why investigating the influence of binder source on the performance of each TOPS was included in this study. The continuous grade and PG of the obtained binders was determined in accordance with AASHTO M 332 "Standard Specification for Performance-Graded Asphalt Binder Using Multiple Stress Creep Recovery (MSCR) Test" (*17*). The results are shown in Table 3.1, with the base binder results being shown for reference. MassDOT's pilot specification for the ARGG required the use of an AR binder with a minimum grade of PG 64E-28. For the SMA and HP, MassDOT's pilot specification required a PMA binder with a minimum grade of PG 76E-34 and minimum average percent recovery of 65% at 3.2 kPa as measured using AASHTO T 350 "Standard Method of Test for Multiple Stress Creep Recovery (MSCR) Test of Asphalt Binder Using a Dynamic Shear Rheometer (DSR)" (*17*). Table 3.1 shows that the AR and PMA binders met MassDOT's pilot specification requirements. The type and the dosages of the modifiers used to prepare the AR and PMA binders were not provided by the binder suppliers.

Table 3.1: Asphalt Binder Grading Results

Grade	Base Binders			ARGG Binders		PMA Binders	
	Used for AR1 and PMA1	Used for PMA2	Used for AR2	AR1	AR2	PMA1	PMA2
Continuous grade	61.0-30	55.9-34.3	65.4-30.3	87.4-28.2	87.4 -33.3	89.5-36.0	77.7-34.2
Performance grade	PG 58S-28	PG 52H-34	PG 64S-28	PG 64E-28	PG 64E-28	PG 76E-34	PG 76E-34
MSCR, average recovery at 3.2 kPa (%)	1.3	2.4	2.7	18.9	15.8	93.4	85.1

3.2 Aggregates and Reclaimed Asphalt Pavement

Aggregates were obtained from a local contractor for use in the mixture design and performance testing. The aggregate stockpiles used in this study were 12.5 mm crushed stone, 9.5 mm crushed stone, stone sand, and stone dust. These stockpiles were sieved to individual size fractions to facilitate batching by individual size fractions to maintain the aggregate gradations accurately and precisely.

MassDOT's pilot specifications allow a maximum of 10% reclaimed asphalt pavement (RAP) in the ARGG, SMA, and HP. RAP was obtained from the same source as the aggregates. Each mixture was developed to incorporate 10% RAP.

3.3 Stabilizing Fiber

For the SMA mixture only, a cellulose stabilizing fiber was required. This fiber was added at a rate of 0.3% by mass of mixture, which is the minimum dosage outlined in AASHTO M 325 "Standard Specification for Stone Matrix Asphalt (SMA)" (*17*).

4.0 Mixture Designs

MassDOT's pilot specifications for ARGG and HP required performing a volumetric mixture design in accordance with AASHTO M 323 "Superpave Volumetric Mix Design" and AASHTO R 35 "Superpave Volumetric Design for Hot Mix Asphalt" (*17*). MassDOT's pilot specification for SMA required performing a mixture design in accordance with AASHTO M 325 "Standard Specification for Stone Matrix Asphalt (SMA)" and R 46 "Standard Practice for Designing Stone Matrix Asphalt (SMA)" (*17*). All three TOPS were developed with nominal maximum aggregate size (NMAS) of 12.5 mm. Additionally, each was developed with two sources of binder.

The mixture volumetric properties and pilot specification requirements for each TOPS mixture is provided in Table 4.1.

Table 4.1: Mixture Design Volumetrics and MassDOT Specification Requirements

Mixture property	ARGG			SMA			HP		
	Design		**Spec**	**Design**		**Spec**	**Design**		**Spec**
Binder	AR1	AR2	—	PMA1	PMA2	—	PMA1	PMA2	—
Design gyrations	100		100	100		100	75		75
Percent binder (P_b), %	7.8		7.6% min	6.2		6.0% min	6.4		6.4% min
Air voids (V_a), %	4.2	3.5	3–5%	4.1	3.9	4.0%	3.6	3.1	3.5%
Voids in mineral aggregate (VMA), %	18.9	18.5	18–23%	17.1	17.0	17% min	17.4	17.1	17% min
Dust to binder ratio	n/a	n/a	n/a	n/a	n/a	n/a	0.5	0.5	0.3–1.0
Draindown, %	0.1	0.1	0.3% max	0.1	0.1	0.3% max	n/a	n/a	0.3% max

Note: n/a = not applicable; Spec = specifications; min = minimum; and max = maximum.

In addition to volumetric requirements, MassDOT's pilot specifications outline some performance testing and the corresponding pass/fail criteria shown in Table 4.2. The performance of each TOPS mixture with respect to these tests and other tests will be discussed later.

Table 4.2: MassDOT's Pilot Specifications Performance Testing Requirements

Distress	Test	Test Method Specification	Criteria
Rutting	Hamburg wheel track test (HWTT) maximum rut depth	AASHTO T 324 at 45°C	< 0.5 in. after 20,000 passes
Moisture damage	HWTT stripping inflection point	AASHTO T 324 at 45°C	> 15,000 passes after 20,000 passes
Cracking	Semicircular bend, flexibility index (FI)	AASHTO T 393 at 25°C	≥ 20*
Low-temperature cracking	Thermal stress restrained specimen test (TSRST)	EN 12697-46	SMA only: for information only

*FI shall be an average of four specimens with no individual specimen with an FI < 15.

5.0 Performance Testing and Results by Distress

Performance testing was conducted on the asphalt binders (AR1, AR2, PMA1, and PMA2) and each TOPS mixture (ARGG, SMA, and HP) relative to specific distresses shown in Figure 2.1. The description of the test methods are outlined in the subsequent sections.

The asphalt binders were aged in accordance with the specific test specification prior to testing. The exact aging performed is noted in the subsequent sections, but generally the binder would be tested either unaged, after rolling thin-film oven (RTFO) aging in accordance with AASHTO T 240 “Standard Method of Test for Effect of Heat and Air on a Moving Film of Asphalt Binder (Rolling Thin-Film Oven Test),” or after RTFO and pressure aging vessel (PAV) aging in accordance with AASHTO R 28 “Standard Practice for Accelerated Aging of Asphalt Binder Using a Pressurized Aging Vessel (PAV)” (*17*).

Mixtures were short-term aged (conditioned) in a loose state prior to specimen compaction in accordance with AASHTO R 30 “Standard Practice for Laboratory Conditioning of Asphalt Mixtures” (*17*). This specification states that after mixing, the mixture should be conditioned for 4 h at 135°C. This is followed by bringing the mixture to the compaction temperature and then compacting it immediately in the Superpave Gyratory Compactor (SGC). This short-term aging method was used for the all the mixture performance tests.

5.1 Low-Temperature (Thermal) Cracking

5.1.1 Asphalt Binder: Extended Bending Beam Rheometer (EBBR)

EBBR testing of the asphalt binders was conducted on RTFO + PAV aged binder residues in accordance with AASHTO TP 122 “Provisional Standard Method of Test for Determination of Performance Grade of Physically Aged Asphalt Binder Using Extended Bending Beam Rheometer (BBR) Method” (*17*). This test is an extension of the conventional BBR test and requires testing the binder at two conditioning temperatures (10°C and 20°C above the low PG grade temperature) and three conditioning times (1, 24, and 72 h). From the data collected for each conditioning time and temperature, a limiting temperature can be determined. By comparing all the limiting temperatures, the warmest one obtained is the low-temperature limiting grade (LTLG). Moreover, the difference between the limiting grade at a conditioning time of 1 hour and the LTLG is used to calculate the grade loss. These parameters are meant to characterize the binder after going through the physical aging process. The results of the EBBR tests are shown in Table 5.1. Colder LTLG are considered more desirable for resistance to low-temperature cracking. The two PMA binders provided colder LTLG than the two AR binders. Hence, based on this test, it would be expected that TOPS prepared using PMA rather than AR would perform better in terms of resistance to low-temperature cracking.

Table 5.1: Asphalt Binder Test Results for Low-Temperature (Thermal) Cracking

Binder Testing	ARGG Binders		PMA Binders	
	AR1	AR2	PMA1	PMA2
EBBR Test on RTFO + PAV Residue (AASHTO TP 122)				
Average Low-Temperature Limiting Grade (LTLG), °C(°C)	−20.3	−23.9	−31.6	−31.0
Average Grade Loss (°C)	−7.3	−6.3	−4.3	−3.4
ABCD Test on RTFO + PAV Residue (AASHTO T 387)				
Average ABCD Crack Temperature (°C)	−43.3	−44.4	−43.4	−38.9
BBR Test on RTFO + PAV Residue (AASHTO T 313 and M 320)				
Test Temperature 1 (°C)	−18	−18	−24	−24
Test Temperature 2 (°C)	−24	−24	−30	−30
Average Creep Stiffness, S in MPa at 60 s Temperature 1 (max. 300 MPa)	144	106	196	292
Average Creep Stiffness, S in MPa at 60 s Temperature 2 (max. 300 MPa)	287	216	402	585
Average m-value at 60s Temperature 1 (min. 0.300)	0.301	0.330	0.316	0.310
Average m-value at 60s Temperature 2 (min. 0.300)	0.267	0.296	0.267	0.257
Pass/Fail Temperature based on Creep Stiffness, S = 300 MPa (°C)	−34.4	−36.7	−37.6	−34.2
Pass/Fail Temperature based on m-value = 0.300 (°C)	−28.2	−33.3	−36.0	−35.1
BBR Test Passing Low-Temperature (°C)	−28.2	−33.3	−36.0	−34.2

5.1.2 Asphalt Binder: Asphalt Binder Cracking Device (ABCD)

ABCD testing of asphalt binders was conducted on RTFO + PAV aged binder residues in accordance with AASHTO T 387 “Standard Method of Test for Determining the Cracking Temperature of Asphalt Binder Using the Asphalt Binder Cracking Device (ABCD)” (*17*). In this test, an asphalt binder is poured around a specialized ring containing embedded strain gauges. These specimens are placed in an environmental chamber and the temperature is cooled at a rate of −20°C/hour. As a specimen cools, strain accumulates in it until a point where a strain jump occurs. This is the point at which the binder specimen has cracked and the magnitude of strain jump and the temperature at which it occurs are determined, with the latter called the ABCD cracking temperature. A minimum of four specimens were tested for each binder. Colder ABCD temperatures generally indicate better performance in terms of low-temperature cracking. The average results of the ABCD tests are shown in Table 5.1. AR1, AR2, and PMA1 had thermal cracking temperatures within 1°C, whereas PMA2 had a thermal cracking temperature that was approximate 4°C warmer than the other binders. Hence, based on this test, it would be expected that the two ARGG mixtures prepared using the two AR binders would perform similarly in terms of resistance to low-temperature cracking, and the two SMA and two HP mixtures prepared using the two PMA would perform differently in terms of resistance to low-temperature cracking.

5.1.3 Asphalt Binder: BBR Creep Stiffness (S) and Slope (m-value)

The standardized Superpave asphalt binder performance test related to low-temperature cracking resistance was conducted on RTFO + PAV aged binder residues in accordance with AASHTO T 313 “Standard Method of Test for Determining the Flexural Creep Stiffness of Asphalt Binder Using the Bending Beam Rheometer (BBR)” and AASHTO M 320 “Standard Specification for Performance-Graded Asphalt Binder” (*17*). These tests are part of a suite of tests required to determine the PG of a binder. In this test, a thin beam of asphalt binder is molded and conditioned at a low temperature. After 1 hour of conditioning, the beam is placed in a simply supported beam configuration and then constantly loaded using 980 ± 50 mN at its midpoint for 240 s. During the duration of the test, the midpoint deflection is measured. From the loading and deflection data, the flexural creep stiffness (S) and the absolute value of the slope of the logarithm of the stiffness curve versus the logarithm of time (m-value) are determined. AASHTO M 320 provides guidance that the maximum creep stiffness should be 300 MPa and the m-value should be a minimum of 0.300, both at a test time of 60 s. These are industry accepted standards.

The results of the BBR tests for each binder are shown in Table 5.1. A minimum of two beams were tested at each temperature for each binder. The warmest BBR passing low temperature is presented, which is the low-temperature continuous PG of the binder, and is used to determine the low-temperature PG. Binders exhibiting colder temperatures in the BBR test would be considered to have better resistance to low-temperature cracking. The two PMA binders provided colder temperatures than the two AR binders. Hence, based on this test it would be expected that TOPS prepared using PMA rather than AR would perform better in terms of resistance to low-temperature cracking, which was also indicated by the EBBR results.

5.1.4 Asphalt Mixture: Thermal Stress Restrained Specimen Test (TSRST)

Mixture TSRST testing was conducted in accordance with AASHTO TP10-93 “Standard Test Method for Thermal Stress Restrained Specimen Tensile Strength.” In this test, a mixture specimen is restrained from contracting while cooled at a rate of −10°C/hour. As the temperature drops, the thermal stress begins to accumulate since the specimen cannot contract to relieve the stress. Eventually, the thermal stress exceeds the materials capability and cracks. The temperature at which it cracks is the TSRST low cracking temperature.

Cylindrical TSRST test specimens of 38 mm diameter by 160 mm tall were cored and cut from larger SGC compacted specimens with dimension of 150 mm diameter by 180 mm tall. The air voids of the final test specimens were 7.0 ± 1.0%. A minimum of three replicates were prepared and tested for each mixture. The results of the TSRST testing are shown in Figure 5.1. Colder TSRST temperatures generally indicate better performance in terms of low-temperature cracking. The error bars in Figure 5.1 represents the standard deviation of the results for each mixture. Mixtures with overlapping error bars are considered not to have significantly different resistances to thermal cracking. The HP with PMA2 was significantly different from the HP using PMA1. MassDOT's pilot specifications presented in Table 4.2 required this test only for the SMA because they did not have experience using SMA mixtures and desired to collect as much information as possible.

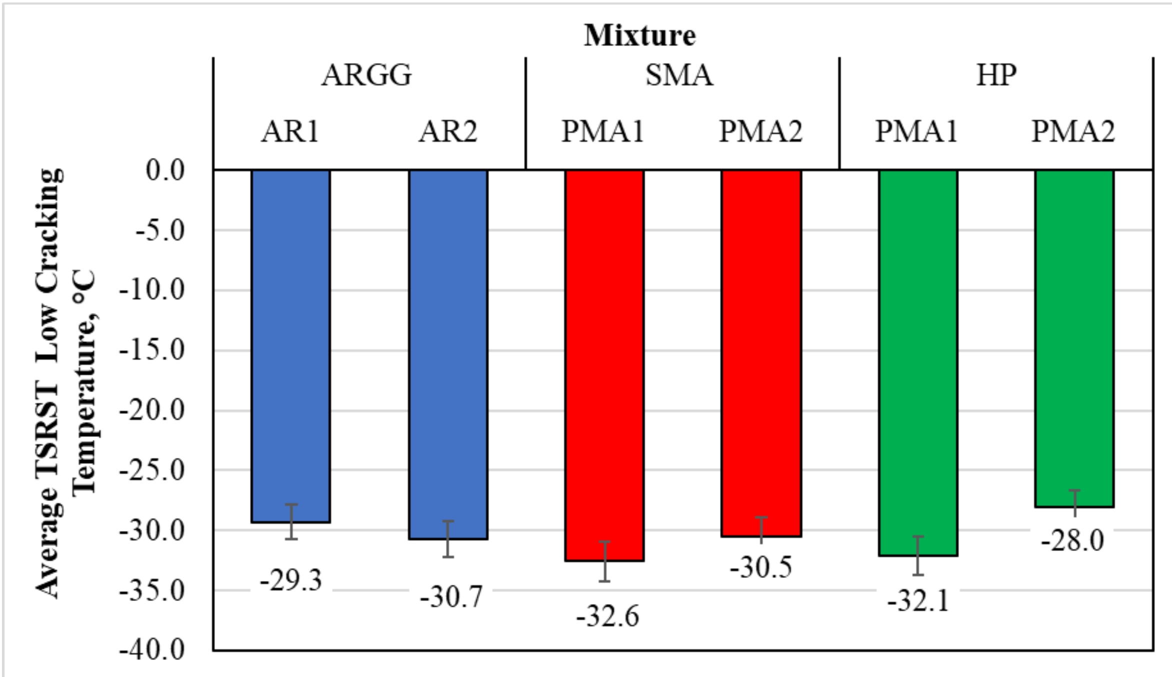

Figure 5.1: Asphalt mixture TSRST test results

5.1.5 Asphalt Mixture: Creep Compliance

Mixture creep compliance testing was conducted in accordance with AASHTO T 322 "Standard Method of Test for Determining the Creep Compliance and Strength of Hot Mix Asphalt (HMA) Using the Indirect Tensile Test Device" (*17*). This test was included in the study as a second test to measure the low-temperature (thermal) cracking characteristics of the selected TOPS mixtures and also to construct creep compliance mixture master curves, which would be utilized later to evaluate the effects of the selected TOPS on extending the life of an in-service asphalt pavement.

Using this test, the creep compliance, Poisson's ratio, and tensile strength, which are required to conduct a thermal cracking analysis of a mixture, are collected. Testing is completed at three cold temperatures. First, creep compliance and Poisson's ratio are measured by loading the vertical diametrical axis of a specimen for 100 s (1,000 s for complete analysis) with a load sufficient enough to produce a horizontal deformation in a specific range. The horizontal and vertical deformations are recorded during the duration of the test. Next, the data needed to determine tensile strength is measured on the same specimen. At the middle temperature of the three test temperatures, the specimen is loaded at a rate of 12.5 mm/min and the peak load recorded. From the peak load, the tensile strength can be calculated.

Cylindrical test specimens of 150 mm diameter by 50 mm tall were cut from larger SGC compacted specimens with dimension of 150 mm diameter by 100 mm tall. The air voids of the final test specimens were 7.0 ± 1.0%. A minimum of three replicates were prepared and tested for each mixture. Testing was performed at temperatures of 0°C, −10°C, and −20°C per the recommendations provided in AASHTO T 322.

The collected data was then analyzed using an Excel spreadsheet called "LTSTRESS" developed by the Northeast Center for Excellence in Pavement Technology based on an analysis methodology developed by Don Christensen (*18*). The spreadsheet calculates the creep compliance as function of time, develops a master curve of creep compliance versus time, and estimates a critical pavement temperature, which is the temperature at which the surface tensile stress reaches the tensile strength of the mixture. The critical pavement cracking temperatures are shown in Figure 5.2. Colder temperatures generally indicate better performance in terms of low-temperature cracking. Changing the binder source changed the critical pavement temperature only by 1°C.

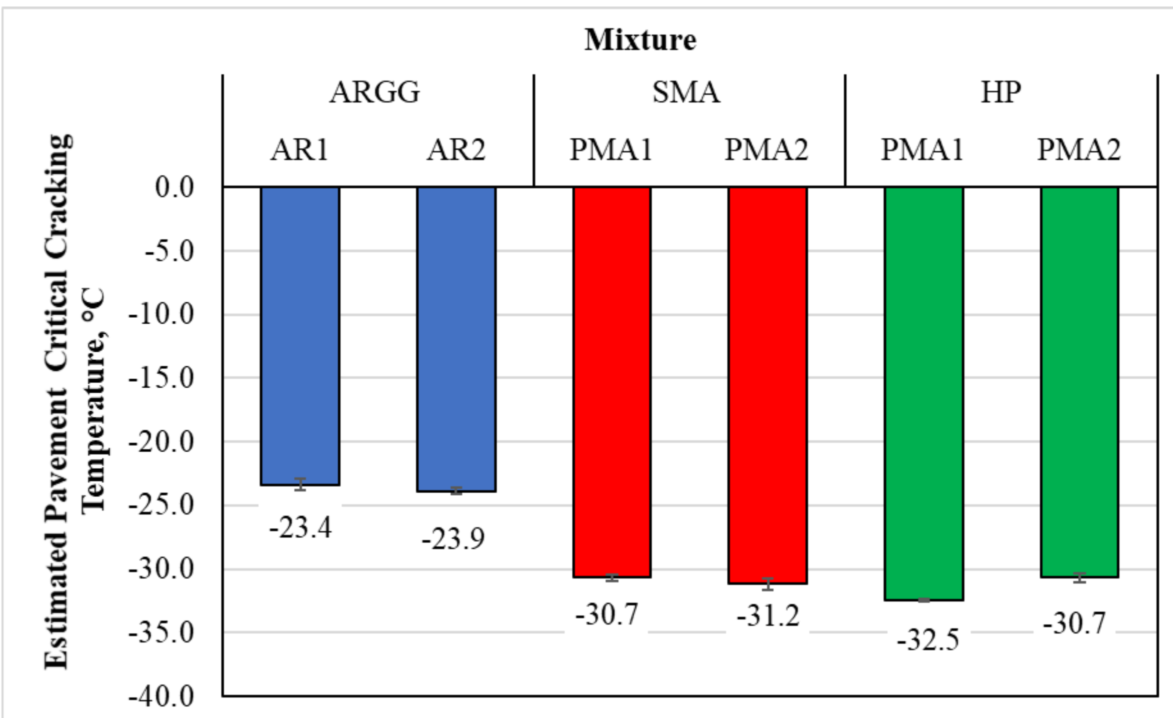

Figure 5.2: Asphalt mixture estimated pavement critical cracking temperature from creep compliance data analysis

5.1.6 Low-Temperature (Thermal) Cracking: Conclusions

MassDOT specifies a PG 64-28 for its asphalt mixture surface layers. Hence, according to the TSRST, the three TOPS can be used interchangeably within the state because regardless of the binder source, each of them provided a low-temperature cracking of −28°C or colder. According to the estimated pavement critical cracking temperatures from the creep compliance analysis, the SMA and the HP can be used interchangeably. It is noted that the creep compliance analysis provided a significantly lower thermal cracking temperature for the ARGG. This might be attributed to the fact that the low-temperature PG for the AR binders was −28°C and for the PMA binders it was −34°C. Regarding the asphalt binder tests, the ABCD did not agree with the EBBR and BBR creep stiffness and m-value.

5.2 Intermediate-Temperature Cracking

5.2.1 Asphalt Binder: Double-Edge-Notched Tension (DENT)

DENT testing of asphalt binders was conducted on RTFO + PAV aged binder residues in accordance with AASHTO TP 113 “Standard Method of Test for Determination of Asphalt Binder Resistance to Ductile Failure Using Double-Edge-Notched Tension (DENT) Test” (*17*). The DENT test utilizes a ductility bath with specialized end pieces having three different ligament lengths of 5, 10, and 15 mm. The test is performed similarly to the elastic recovery test except the displacement rate is 100 ± 2.5 mm/min. A minimum of two specimens were tested for each ligament length at a test temperature of 25°C for each binder.

This test evaluates an asphalt binder’s resistance to ductile failure. This is done through the calculation of the critical crack tip opening displacement (CTOD, mm) from the test data. CTOD is the ultimate elongation for a zero-ligament length and represents the strain tolerance in the vicinity of a crack. Higher CTODs are considered to indicate better fatigue cracking resistance. The method to calculate CTOD is summarized in AASHTO TP 113 (*17*). The results of the DENT test are shown in Table 5.2. The two PMA binders with a PG 76E-34 had a greater CTOD than the two AR binders with a PG 64E-28, indicating that the TOPS designed with these PMA binders might have better resistances to fatigue cracking at intermediate temperatures.

Table 5.2: Asphalt Binder Test Results for Intermediate-Temperature Cracking

Binder Testing	ARGG Binders		PMA Binders	
	AR1	AR2	PMA1	PMA2
DENT Test on RTFO + PAV Residue (AASHTO TP 113)				
Critical Crack Tip Opening Displacement (CTOD), mm	29.9	20.5	40.5	40.5
DSR Test on RTFO + PAV Residue (AASHTO T 315 and M 320)				
Test Temperature (°C)	22	22	25	25
Average G* Sinδ (kPa)	2,210	1,645	968	1,070
Pass/Fail Criterion = G* Sinδ kPa max. 6,000 kPa	Pass	Pass	Pass	Pass

5.2.2 Asphalt Binder: G* Sinδ

The standardized Superpave asphalt binder performance test related to intermediate-temperature cracking resistance was conducted on RTFO + PAV aged binder residues in accordance with AASHTO T 315 "Standard Method of Test for Determining the Rheological Properties of Asphalt Binder Using a Dynamic Shear Rheometer (DSR)" and M 320 "Standard Specification for Performance-Graded Asphalt Binder" (*17*). This test is part of a suite of tests required to determine the PG of a binder. For intermediate-temperature cracking, the parameter of interest is G* Sinδ measured in the DSR.

For the binders in this study, a temperature sweep was performed to determine G* Sinδ at various intermediate temperatures. As indicated in Table 5.2, all binders met the AASHTO M320 requirement of a maximum of 6,000 kPa, indicating acceptable intermediate-temperature cracking performance.

5.2.3 Asphalt Mixture: Flexibility Index Test (FIT)

Mixture FIT testing was conducted in accordance with AASHTO T 393 "Standard Method of Test for Determining the Fracture Potential of Asphalt Mixtures Using the Illinois Flexibility Index Test (I-FIT)" (*17*). This test is used to measure the intermediate-temperature cracking resistances of semicircular bend geometry (SCB) shaped asphalt specimens. During the test, a load is applied along the vertical radius of the SCB specimen at a rate of 50 mm/min while concurrently the load and load line displacement (LLD) are measured and recorded until the specimen fails. From the plot of load versus LLD, parameters including the Flexibility Index (FI) can be determined. The FI is a parameter intended to characterize the cracking resistance of an asphalt mixture.

SCB test specimens of 150 mm base diameter by 50 mm thick were cut from the middle of larger SGC compacted specimens with dimension of 150 mm diameter by 150 mm tall. The air voids of the final cut and notched test specimens were 7.0 ± 1.0%. A minimum of four replicates were prepared and tested for each mixture at a test temperature of 25°C. The results of the FI testing are shown in Figure 5.3. All mixtures passed the criteria listed in MassDOT's pilot specification presented in Table 4.2, which was an average >20 with no individual specimen with an FI < 15. Larger FI generally indicate better performance in terms of intermediate-temperature cracking. The HP prepared using PMA1 binder illustrated significantly higher FI compared to the same HP prepare using the PMA2 binder because the standard deviation bars for these two mixtures did not overlap. However, this test showed that the SMA prepared using the same two PMAs had no significant difference in FI. Similarly, ARGG prepared using the two AR binders had no significant difference in FI.

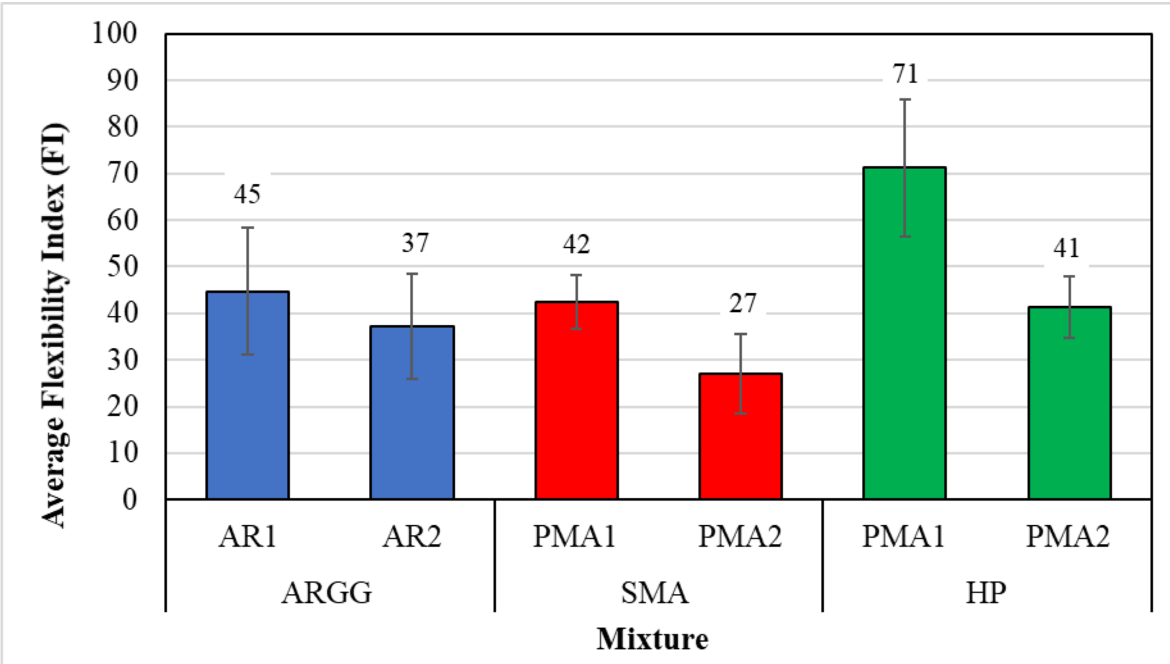

Figure 5.3: Asphalt mixture FIT results

5.2.4 Asphalt Mixture: Indirect Tensile Asphalt Cracking Test (IDEAL-CT)

Mixture IDEAL-CT testing was conducted in accordance with ASTM D 8225 "Standard Test Method for Determination of Cracking Tolerance Index of Asphalt Mixture Using the Indirect Tensile Cracking Test at Intermediate Temperature." (*19*) In this test, a load is applied along the vertical diametral axis of a cylindrical mixture specimen at a constant LLD rate of 50 ±2.0 mm/min. The load and the LLD are measured throughout the duration of the test and are then used to calculate a cracking index known as the cracking index tolerance, or CT_{Index}. A detailed description of how the CT_{Index} is calculated is provided in ASTM D 8225. (*19*)

IDEAL-CT test specimens were compacted in the SGC to 150 mm diameter by 62 mm tall. This test requires no cutting or gluing of the specimens. The air voids of test specimens were 7.0 ± 1.0%, and a minimum of four replicates were prepared and tested for each mixture at a test temperature of 25°C. The results of the IDEAL-CT testing are shown in Figure 5.4. Larger CT_{Index} values generally indicate better performance in terms of intermediate-temperature cracking. The HP prepared using the PMA1 binder performed better than the other mixtures, which was in agreement with FI. Even so, because this is a pass/fail test, the acceptability of a mixture will depend on the STA's criterion for the test, which is currently being researched.

This test was added to this study because MassDOT has selected the IDEAL-CT to be the cracking test in its balanced mixture design (BMD) protocol.

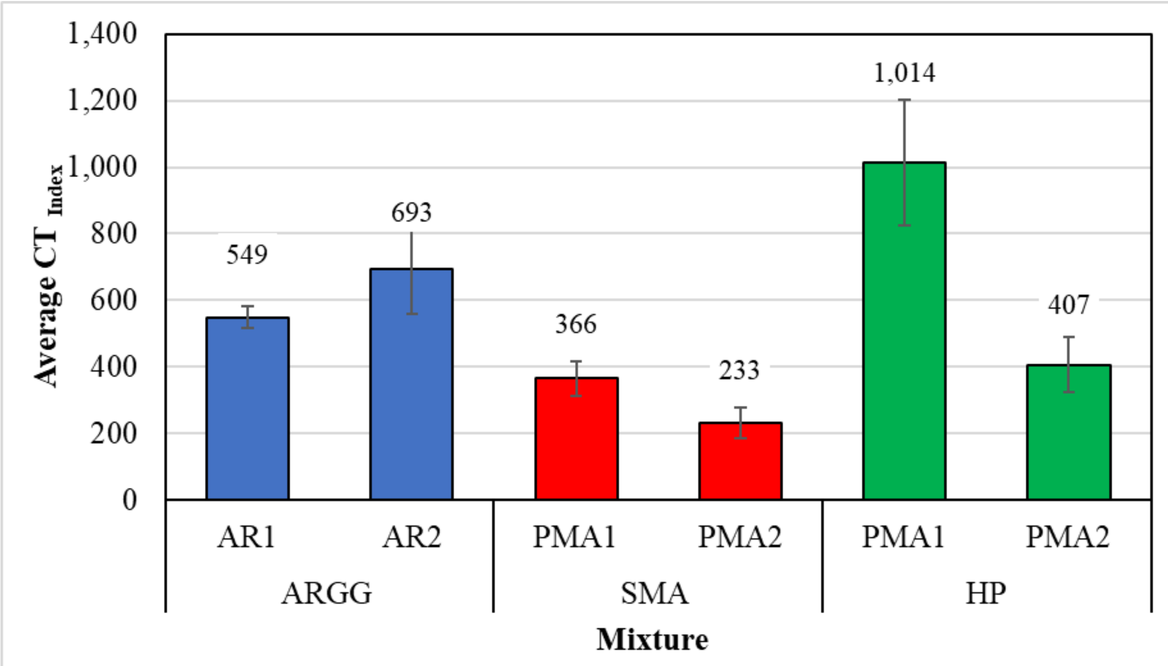

Figure 5.4: Asphalt mixture IDEAL-CT results

5.2.5 Intermediate-Temperature Cracking: Conclusions

All mixtures passed the criteria for the FI listed in MassDOT's pilot specification, which was an average >20 with no individual specimen with an FI < 15. Accordingly, MassDOT can use the three TOPS interchangeably to address intermediate-temperature cracking. The IDEAL-CT was performed because it is the test that the MassDOT is planning to include in its BMD protocol. The preliminary criterion for their surface mixtures is $CT_{index} \geq 90$. Using this criterion again shows that MassDOT can use the three TOPS interchangeably to address intermediate-temperature cracking.

5.3 Non-Load Associated Cracking

5.3.1 Asphalt Binder: Delta T_c

Delta T_c (ΔT_c) is a binder parameter that is used to measure a binder's loss of stress relaxation due to aging, which increases the risk of non-load-associated cracking (*20,21*). It is calculated using BBR data from testing RTFO + PAV aged binder residues and is the mathematical difference between the two critical low temperatures measured in the BBR ($\Delta T_c = T_{c,s} - T_{c,m}$). The two temperatures correspond to a creep stiffness (S) of 300 MPa and an m-value of 0.300. A minimum ΔT_c of −5.0°C has been suggested as a preliminary pass/fail criterion (*20*); therefore, binders with a ΔT_c of −5.0°C or colder (more negative) are considered unacceptable.

As shown in Table 5.3, AR1 had a ΔT_c of −6.2°C, which did not meet the recommended criterion, whereas binder AR2 had ΔT_c of −3.4°C, which met the recommended criterion. Although the two binder sources had the same PG and similar continuous PGs, they had different relaxation properties. PMA1 and PMA2 both passed the recommended criterion. Passing the criteria indicates there should not be an issue with non-load-associated cracking for these binders. For informational purposes, the ΔT_c for the base binders used to fabricate AR1 and PMA1, AR2, and PMA2 were −0.4, 0, and +1.0, respectively.

Table 5.3: Asphalt Binder Test Results for Non-Load-Associated Cracking

Binder Testing	ARGG Binders		PMA Binders	
	AR1	**AR2**	**PMA1**	**PMA2**
BBR Test on RTFO + PAV Residue (AASHTO T 313)				
Delta T_c (°C) ($\Delta T_c = T_{c,s} - T_{c,m}$)	−6.2	−3.4	−1.6	+0.9
ΔT_c Warmer than −5°C = Pass	Fail	Pass	Pass	Pass

5.3.2 Asphalt Binder: Glover Rowe Parameter in Black Space & CAM Model

Block cracking is a non-load-associated distress of asphalt pavement that is a function of the aging experienced by the asphalt binder in the field. The Superpave PG specification does not have a parameter that accounts for the non-load-associated cracking characteristics of asphalt binders. Three types of analysis have been used to evaluate the non-load-associated cracking of an asphalt binders. First, the parameter Delta T_c (ΔT_c), which describes the loss of relaxation of a binder due to aging (*20*), can be calculated as noted in the previous section. Second, the complex shear modulus (G*) and phase angle (δ) can be measured and used to generate a rheological plot commonly referred to as Black Space Diagram (*22*). Third, rheological data can be used to evaluate binders using is the Christensen-Anderson Model (CAM). (*23*)

To evaluate the binders using the Black Space diagram and CAM model, data was collected using a temperature-sweep frequency test using the DSR. Testing was performed to measure G^* and the corresponding δ at a temperature range from 10°C to 58°C at 12°C intervals with test frequencies from 0.1 to 100 rad/s. The test was performed on binders aged first in the RTFO and then in the PAV for 0 and 20 h. The intent was to also PAV age the binders at 40 and 60 h; however, after 20 h aging in the PAV, some of the binders could not be tested because they were too stiff to pour. For analysis of the collected DSR data, the RHEA software was used (*24*). RHEA is a tool used for rheology analysis of materials with focus on the asphalt binders.

Binder master curves were constructed at a reference temperature of 15°C using the CAM model. The crossover frequency (ω_c) and the rheological index (R-value), which are indicators for overall hardness and rheological type, respectively (*25*), were calculated and plotted in ω_c – R-value space as shown in Figure 5.5. Typically, ω_c decreases and the R-value increases with increased aging. An asphalt binder with a higher ω_c and lower R-value indicates more resistance to cracking. The binders in this study followed this trend after aging, but data at longer aging periods are needed to draw any conclusions about the performance of the binders.

Unfortunately the materials did not flow enough to perform testing after longer aging periods. This suggests this type of analysis may not be appropriate for the AR and PMA binders in this study.

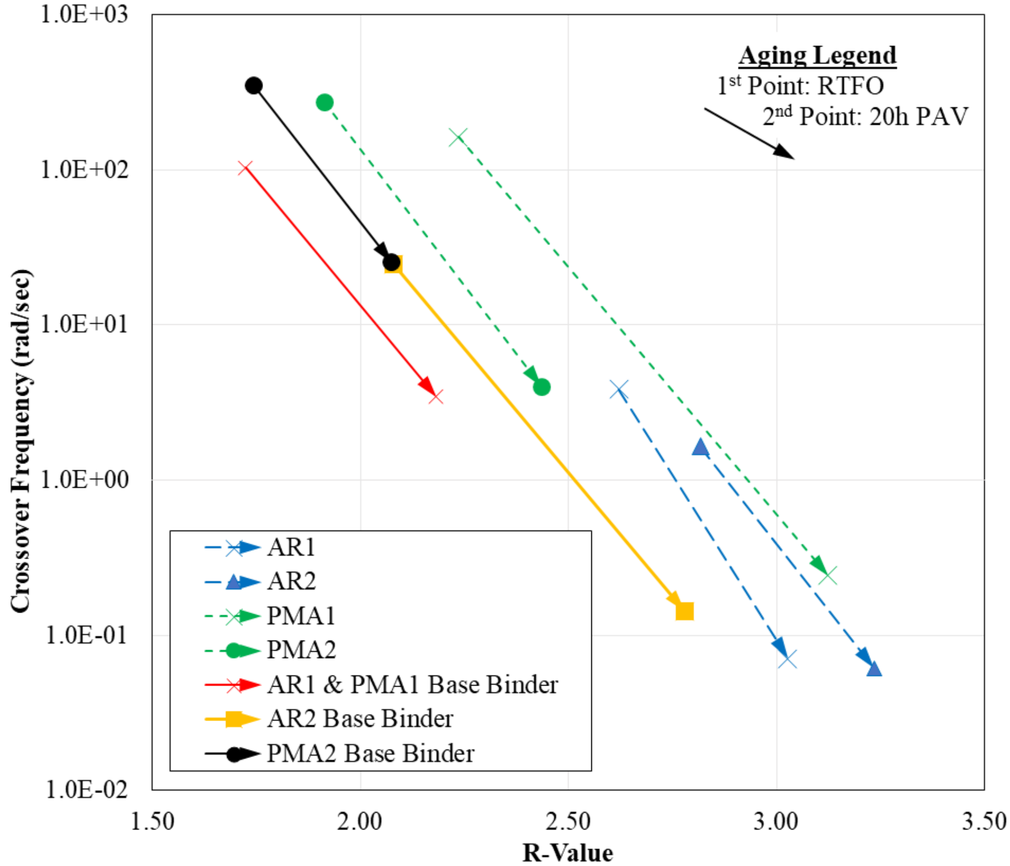

Figure 5.5: Rheological index (R-value) plotted in ω_c – R-value space

The Glover-Rowe parameter [$G^*(\cos\delta)^2/\sin\delta$] was determined at 0.005 rad/s and plotted on the Back Space diagram (*25*) as shown in Figure 5.6. This parameter indicates the block cracking resistance of an asphalt binder relative to known thresholds. The proposed thresholds for binders after RTFO and 20 h of PAV aging are 180 kPa for the onset of block cracking and 600 kPa for significant cracking (*25*). None of the binders tested in this study crossed the onset of cracking threshold, which would indicate that non-load-associated cracking is not an issue for these binders.

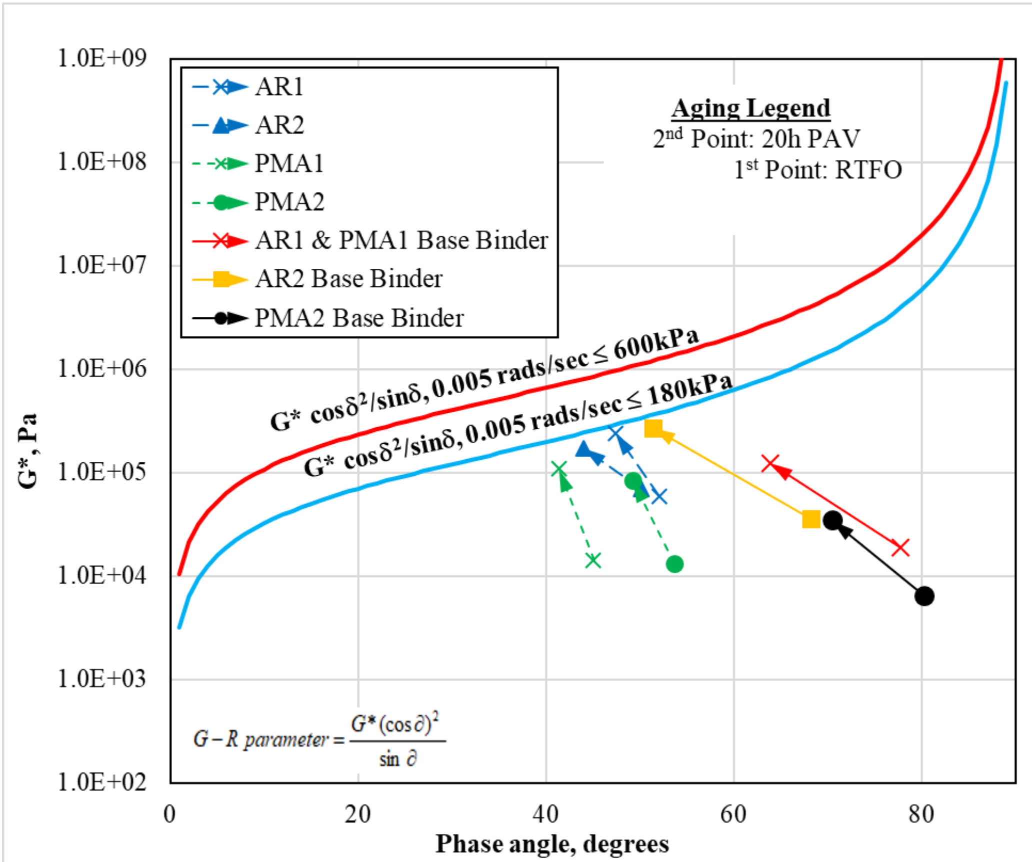

Figure 5.6: Asphalt binder Glover-Rowe parameter in black space

5.3.3 Asphalt Mixture: Indirect Tensile Asphalt Cracking Test (IDEAL-CT)

The details of this test and the corresponding results have been previously presented in the intermediate-cracking temperature distress section of this paper. The test has also proven reliable for evaluating an asphalt mixture's susceptibility to non-load-associated cracking (*26*). Hence, the same data in Figure 5.4 was used in this section. The HP prepared using the PMA1 binder performed better than the other mixtures. Because this test is a pass/fail test, the acceptability of a mixture will depend on the STA's criterion for the test, which is currently being researched.

5.3.4 Non-Load-Associated Cracking: Conclusions

The IDEAL-CT test was used to evaluate the non-load-associated cracking of the mixtures. MassDOT has a preliminary criterion for its surface mixtures: $CT_{index} \geq 90$. Accordingly, they can use any of the three TOPS to address non-load-associated cracking even though AR1 failed the suggested criterion for Delta T_c.

5.4 Reflective Cracking (Asphalt Mixtures Only)

5.4.1 Asphalt Mixture: Overlay Test (Tex-248-F)

Mixture reflective cracking was evaluated in accordance with Texas DOT Designation Tex-248-F "Overlay Test" (*27*). In this overlay test (OT), an asphalt specimen is glued onto two separate blocks. One block is fixed during the test, and the other is cyclically moved horizontally in tension using a triangular waveform with a constant displacement of 0.025 in. (0.06 cm). One cycle of block movement is from the initial position to the maximum displacement and then back again, which takes 10 s. The peak load required to move the specimen to the constant displacement is recorded each cycle, and the test terminates when the load is reduced 93% from the peak load at cycle number one. Additionally, the maximum number of cycles for this test is typically set to 1,200. A larger number of cycles to failure generally indicates better reflective cracking resistance.

Besides determining a number of cycles to failure (N_f), two other parameters can be determined from the collected data: critical fracture energy (CFE) and crack progression rate (CPR). The method for their determination is outlined in Tex-248-F (*27*). CFE gives an indication of the cracking response of the specimen during the first load cycle (crack initiation phase) as shown in Figure 5.7 (*27*). CFE is the area under the load displacement curve up to the peak load during the first cycle. A high CFE is desirable because it indicates that more energy is required to initiate a crack. The CPR (Figure 5.8) (*27*) is the exponent of the normalized load reduction curve obtained while the crack propagates under the cyclic loading. CPR is used to characterize the flexibility of the specimen because it is indicative of a mixture's resistance to crack propagation. A low CPR is desirable because it indicates more mixture flexibility.

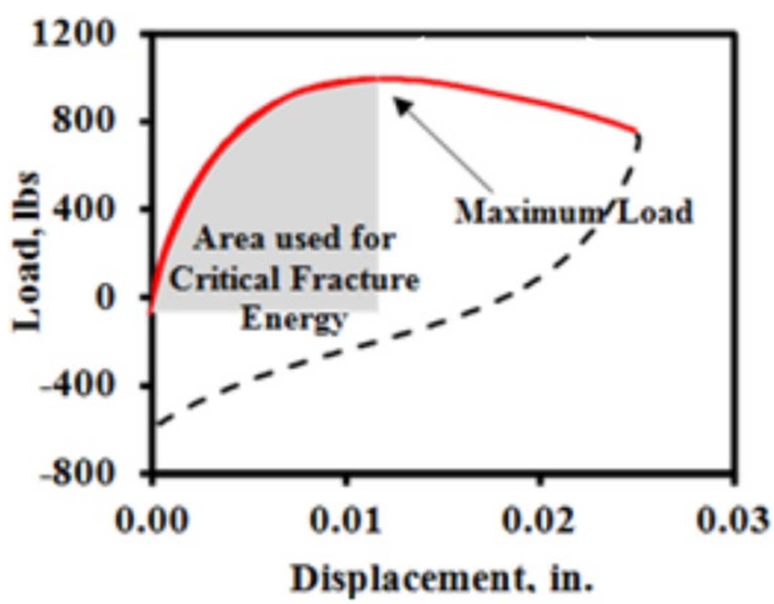

Figure 5.7: Overlay Test CFE determined from first test cycle data (*27*)

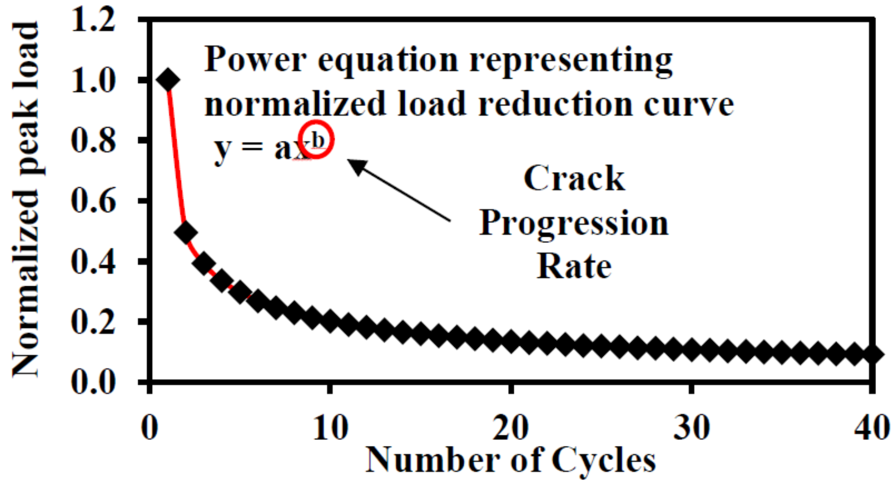

Figure 5.8 Overlay Test CPR determined from all cycle data (*27*)

OT test specimens were cut from the middle of larger SGC compacted specimens with dimensions of 150 mm diameter by 115 mm tall. The exact final specimen dimensions, cutting procedure, and gluing are outlined in Tex-248-F (*27*). The air voids of the final cut test specimens were 7.0 ± 1.0%. A minimum of five replicates were prepared and tested for each mixture at a test temperature of 25°C.

The average N_f for all mixtures tested exceed the 1,200-cycle test termination setting, meaning that the mixtures would fail somewhere beyond 1,200 cycles. The results of the CFE and CPR from the OT testing are shown in Figures 5.9 and 5.10, respectively. The N_f, CFE, and CPR are all indices for which an agency must establish pass/fail criteria, because a conclusion cannot be drawn without them. Nevertheless, the N_f results indicate that the three TOPS types are quality mixtures in resisting reflective cracking. Figure 5.9 shows that the CFE parameter was able to capture the effect of binder source for one type of the TOPS. PMA2 provided a HP with significantly higher value than the HP with PMA1.

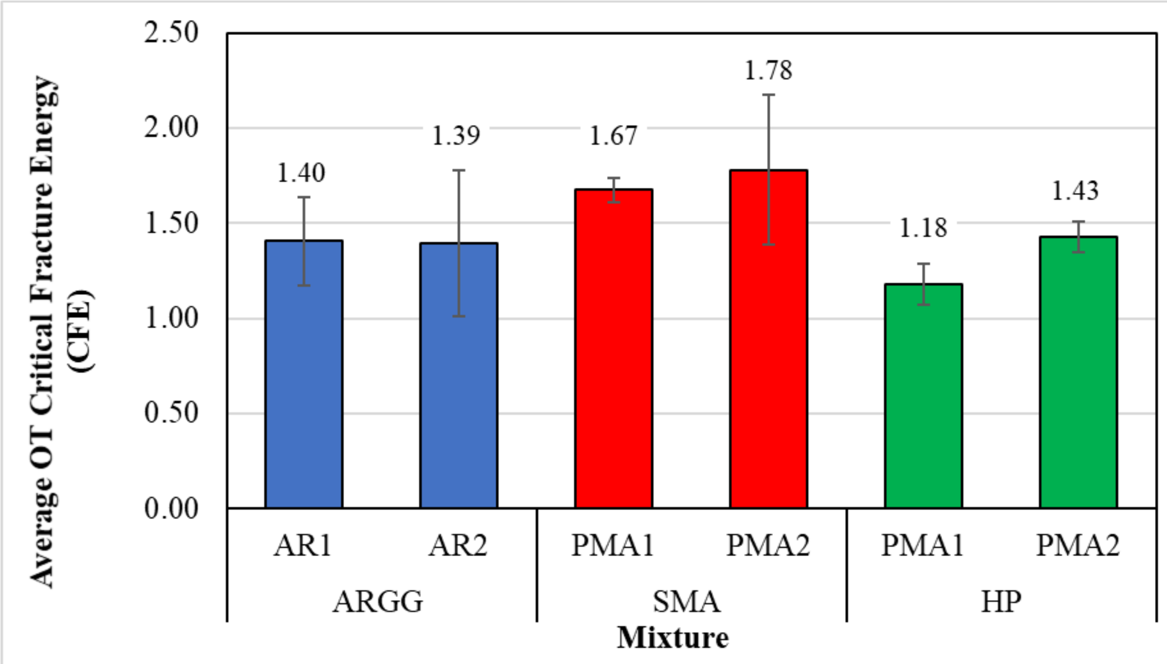

Figure 5.9 Asphalt mixture OT CFE results

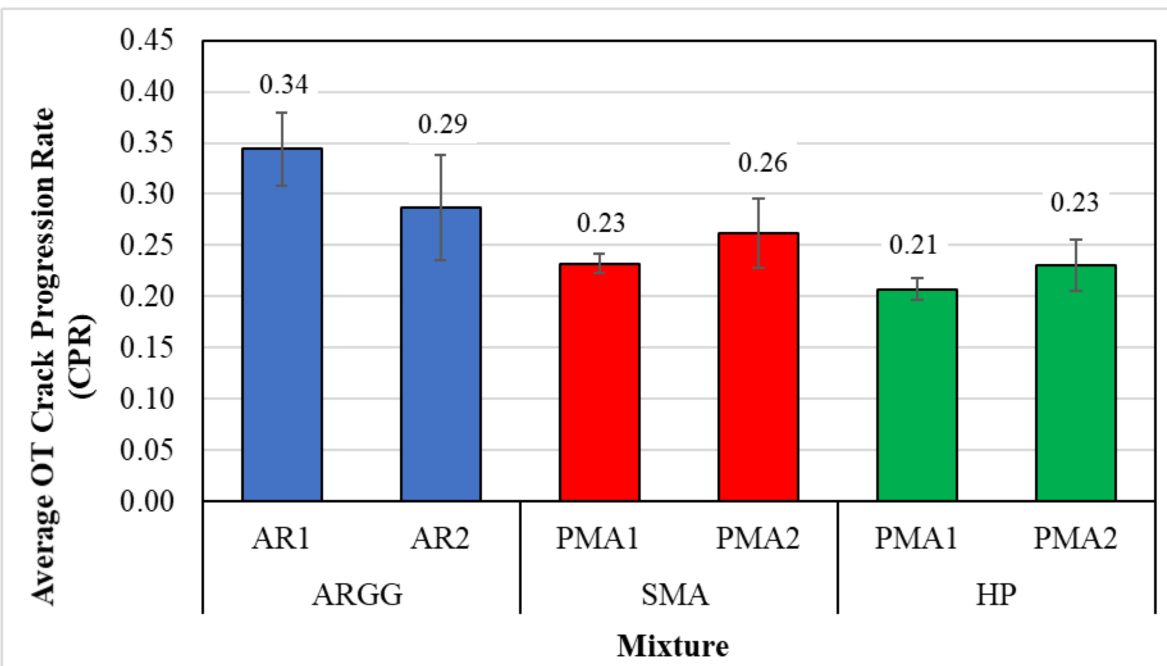

Figure 5.10: Asphalt mixture OT CPR results

5.5 Rutting

5.5.1 Asphalt Binder: Multiple Stress Creep Recovery (MSCR)

The asphalt binder performance test related to rutting resistance was conducted on RTFO aged binder residues in accordance with AASHTO T 350 "Standard Method of Test for Multiple Stress Creep Recovery (MSCR) Test of Asphalt Binder Using a Dynamic Shear Rheometer (DSR)" and M 332 "Standard Specification for Performance-Graded Asphalt Binder Using Multiple Stress Creep Recovery (MSCR) Test" (*17*). This test is part of a suite of tests required to determine the PG of a binder in accordance with AASHTO M 332. For rutting, the parameters of interest are nonrecoverable creep compliance (J_{nr}) and J_{nr} difference measured in the DSR.

The MSCR was performed at the specified temperature to determine J_{nr} at 3.2 (kPa^{-1}) and J_{nr} difference. In accordance with AASHTO M 332, these values are then used to assign a traffic designation for the binder related to the appropriate traffic loading in which it can be suitably used [Standard (S), Heavy (H), Very Heavy (V), and Extremely Heavy (E)]. As indicated in Table 5.4, all binders yielded the MSCR designation E for extremely heavy traffic. This indicated the best possible rutting resistance.

Table 5.4: Asphalt Binder Test Results for Rutting

Binder Testing	ARGG Binders		PMA Binders	
	AR1	AR2	PMA1	PMA2
DSR Test on RTFO Residue (AASHTO T 315, T 350, and M 332)				
MSCR Testing Temp. (°C) AASHTO M332	64	64	76	76
MSCR, Average J_{nr} at 3.2 (kPa^{-1}), AASHTO M 332	0.18	0.22	0.07	0.26
MSCR Traffic Designation (J_{nr} max. 0.5 kPa^{-1} = E)	E	E	E	E
MSCR, Average J_{nr} Difference, AASHTO M 332	360.6%	530.8%	93.5%	96.7%
DSR Test on RTFO Residue (AASHTO T 315 and M 320)				
Test Temperature (°C)	64	64	76	76
Average G*/ Sinδ kPa at 10 rad/s	13.8	14.7	4.5	2.5
Pass/Fail Criterion = G*/ Sinδ kPa min. 2.20 kPa	Pass	Pass	Pass	Pass

Note: E = extremely heavy traffic.

5.5.2 Asphalt Binder: G*/Sinδ

The standardized Superpave asphalt binder performance test related to rutting resistance was conducted on RTFO aged binder residues in accordance with AASHTO T 315 "Standard Method of Test for Determining the Rheological Properties of Asphalt Binder Using a Dynamic Shear Rheometer (DSR)" and M 320 "Standard Specification for Performance-Graded Asphalt Binder" (*17*). This test is part of a suite of tests required to determine the PG of a binder. For rutting, the parameter of interest is G*/Sinδ measured in the DSR at high temperatures.

For the binders in this study, a temperature sweep was performed to determine G*/Sinδ at various high temperatures for binder RTFO residue. As indicated in Table 5.4, all binders met the AASHTO M320 requirement of a minimum of 2.20 kPa.

5.5.3 Asphalt Mixture: Hamburg Wheel-Track Test (HWTT)

Mixture rutting was evaluated in accordance with AASHTO T 324 "Standard Method of Test for Hamburg Wheel-Track Testing of Compacted Asphalt Mixtures" (*17*). The method is used to determine premature failure of asphalt mixtures due to "weakness in the aggregate structure, inadequate binder stiffness, or moisture damage" (*17*). In this test, asphalt mixture specimens are submerged in heated water and then loaded by a 703N steel reciprocating rolling wheel for 20,000 passes or until excessive rutting is observed. During the test, the rut depths of each specimen are recorded with respect to each wheel pass. From this data the stripping inflection point (SIP) can also be determined, which provides an indication of the onset of moisture damage. Prior to the SIP, or in the absence of a SIP, the rut depth provides an indication of rutting performance. AASHTO T 324 does not provide pass/fail criteria for this test; however, MassDOT requires that mixtures have a rut depth less than 12.5 mm after 20,000 passes combined with no SIP before 15,000 passes when tested at 45ºC.

HWTT test specimens for each mixture were compacted in the SGC to 150 mm diameter by 62 mm tall. They were then trimmed to fit into the HWTT molds. The air voids of test specimens were 7.0 ± 1.0%, and a minimum of two replicates were prepared and tested for each mixture at a test temperature of 45°C. The results are shown in Figure 5.11. All TOPS exhibited only minor rut depths and passed MassDOT's specification criterion. No SIP was observed in any of the data, indicating that moisture damage was not an issue with these mixtures.

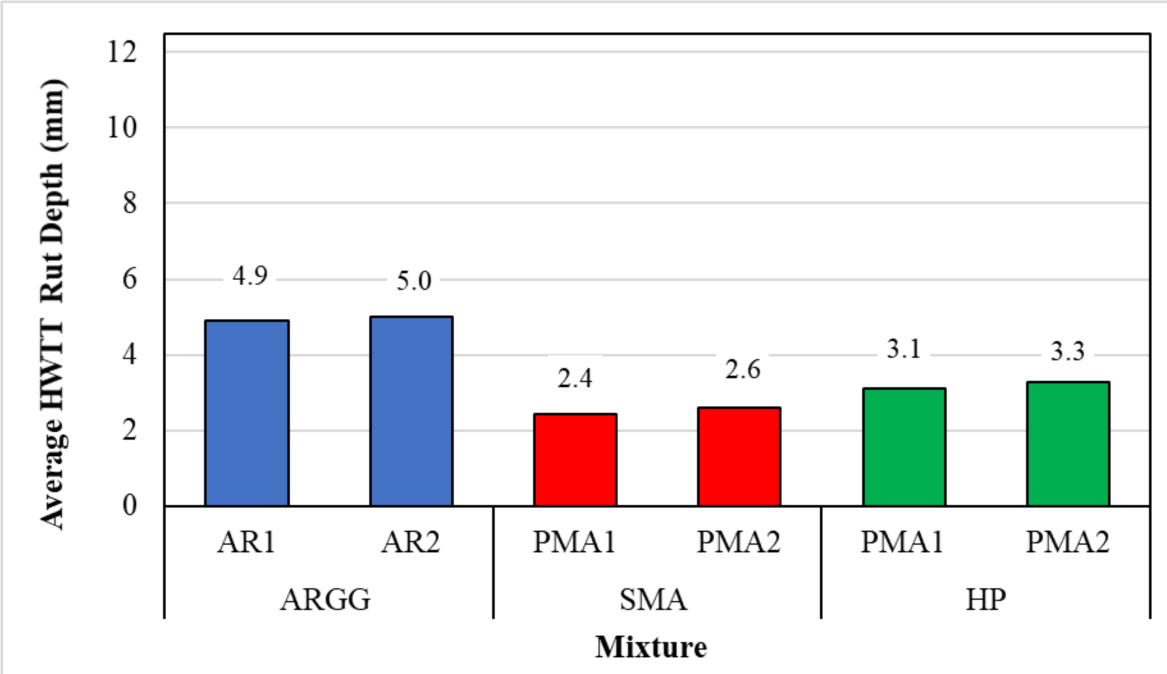

Figure 5.11: Asphalt mixture HWTT results

5.5.4 Rutting: Conclusions

All mixtures passed the MassDOT for rutting. Accordingly, MassDOT can use the three TOPS interchangeably to address rutting.

5.6 Raveling (Asphalt Mixtures Only)

5.6.1 Asphalt Mixture: Cantabro Loss

Mixture raveling was evaluated in accordance with AASHTO TP 108 "Standard Method of Test for Determining the Abrasion Loss of Asphalt Mixture Specimens" (*17*). In this test an asphalt mixture specimen is placed in the drum of a Los Angeles Abrasion machine with no steel spheres and subjected to 300 revolutions of rotation. The percentage of material lost during this test is then calculated from the original specimen mass. Lower percent losses indicate better resistances to raveling.

Cantabro test specimens for each mixture were compacted in the SGC to 150 mm diameter by 115 mm tall. The air voids of test specimens were 7.0 ± 1.0%, and a minimum of three replicates were prepared and tested for each mixture at a test temperature of 25°C. The results of the Cantabro testing are shown in Figure 5.12**.** There are no standardized pass/fail criteria for the Cantabro test, but Virginia DOT is utilizing a ≤7.5% loss based on extensive testing of surface mixtures for developing their BMD protocol (*28*). All mixtures tested in this study would pass this criterion, and hence MassDOT can use the three TOPS interchangeably to address raveling.

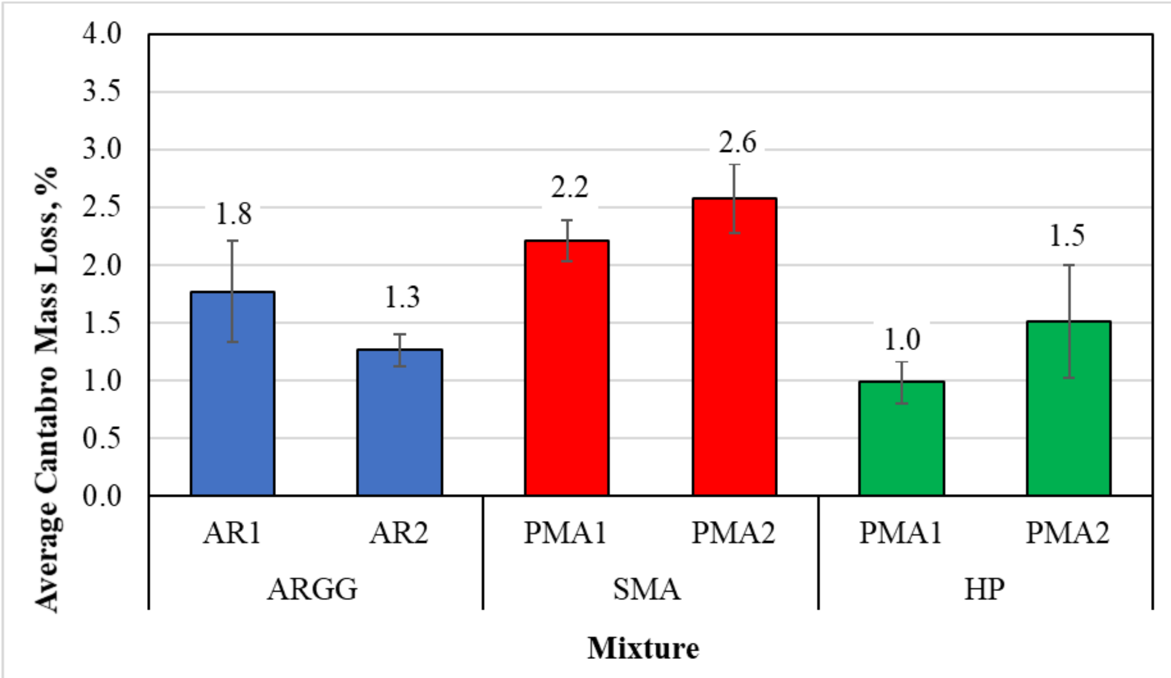

Figure 5.12: Asphalt mixture Cantabro results

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6.0 Statistical Analysis of the Mixture Testing Results

A two-way analysis of variance (ANOVA) was performed using the IBM SPSS Statistics software to check the effect of the two independent variables—namely, TOPS type and binder source and any interactions between them—on the dependent variables of TSRST low cracking temperature, creep compliance critical cracking temperature, intermediate-temperature cracking FI and CT_{Index}, reflective cracking CFE and CPR, and Cantabro percent abrasion loss. Table 6.1 shows the results. If the significant value is less than 0.05, then the independent variable had a significant effect on the dependent variable, or there was an interaction between the two independent variables.

Table 6.1: Results of ANOVA analysis

Test	**Variables**		**Interaction of Variables**
	TOPS Type	**Binder Source**	**TOPS Type Binder Source**
Low-Temperature Cracking: TSRST Low Cracking Temperature			
Significant Value	0.034	0.017	0.323
S or NS	S	S	NS
Low-Temperature Cracking: Creep Compliance Critical Cracking Temperature			
Significant Value	<0.001	<0.001	<0.001
S or NS	S	S	S
Intermediate-Temperature Cracking: FI			
Significant Value	<0.001	<0.001	0.145
S or NS	S	S	S
Intermediate-Temperature Cracking: CT_{Index}			
Significant Value	<0.001	<0.001	<0.001
S or NS	S	S	S
Reflective Cracking: Overlay Test CFE			
Significant Value	0.007	0.340	0.562
S or NS	S	NS	NS
Reflective Cracking: Overlay Test CPR			
Significant Value	<0.001	0.018	0.021
S or NS	S	S	S
Raveling: Cantabro Percent Abrasion			
Significant Value	<0.001	0.028	0.657
S or NS	S	S	NS

Note: ANOVA analysis results are significant (S) when the significant value <0.05; otherwise, not significant (NS).

Based on the output, TOPS type and the binder source had a significant effect on all dependent variables except for the overlay test CFE, where the binder source did not have a significant effect. Additionally, the interactions between the two independent variables did not have significant effect on the low-temperature cracking temperature of the mixtures measured using the TSRST, the Overlay CFE, and the percent abrasion loss. While there were significant effects, all TOPS passed the mixture performance tests where there was a MassDOT specified or preliminary criterion.

Field performance or using performance predictive models can aid in distinguishing the effect of the three TOPS on extending the service life of an existing asphalt pavement. Since field performance data was not available, predictive models incorporating the fundamental material characteristics of the mixtures were used to evaluate the TOPS on increasing the service life of an asphalt pavement.

7.0 Numerical Ranking of the Mixture Testing Results

Table 7.1 summarizes the lab testing of the three types of TOPS. Each cell in the table represents the average of the two binders from the two different sources. A numerical ranking of the different mixture performances is also listed in the table. Although the data and analysis in Section 6.0 indicated that the three TOPS can be used interchangeably, the numerical ranking is provided to illustrate which TOPS type among the three might perform, in term of a specific distress, better than the other two.

Table 7.1: Numerical Ranking of the Mixture Testing Results

Mixture	Average TSRST Low Cracking Temperature (°C)	RANKING TSRST Low Cracking Temperature
ARGG	−30.0	3
SMA	−31.6	1
HP	−30.1	2
Mixture	**Average Creep Compliance Estimated Pavement Critical Temperature (°C)**	**RANKING Creep Compliance Estimated Pavement Critical Temperature**
ARGG	−23.7	3
SMA	−31.0	2
HP	−31.6	1
Mixture	**Average Flexibility Index**	**RANKING Flexibility Index**
ARGG	41.0	2
SMA	34.5	3
HP	56.0	1
Mixture	**Average CT_{Index}**	**RANKING CT_{Index}**
ARGG	621.0	2
SMA	299.5	3
HP	710.5	1
Mixture	**Average OT Cycles to Failure**	**RANKING OT Cycles to Failure**
ARGG	>1,200	1* (tie)
SMA	>1,200	1* (tie)
HP	>1,200	1* (tie)
Mixture	**Average HWTT Rut Depth**	**RANKING Rut Depth**
ARGG	5.0	3
SMA	2.5	1
HP	3.2	2
Mixture	**Average Cantabro Loss (%)**	**RANKING Cantabro Loss**
ARGG	1.6	2
SMA	2.4	3
HP	1.3	1

8.0 Evaluation of TOPS Service Life Extension

The potential to extend the service life of an in-service asphalt pavement when using any of the three TOPS as a rehabilitation strategy was evaluated using the AASHTOWare PMED version 2.6.2 software. The AASHTOWare PMED software can predict pavement distresses over time by considering the interaction of traffic, climate, materials, and pavement structure. The focus in this study was on predicting fatigue and thermal cracking.

AASHTOWare PMED includes three hierarchical input levels defined as Levels 1, 2, and 3. Level 1 provides the most accurate predications because its inputs are project-specific measured data, Level 2 represents the intermediate level of accuracy because it uses estimated regional inputs that are not project specific, and Level 3 represents the lowest level of accuracy because it uses default values of inputs. In this study, the fatigue cracking and thermal cracking of the three TOPS types were predicted using AASHTOWare PMED with Level 1 inputs for the asphalt mixture layers. The same design traffic, climate, reliability, and unbound layer material properties were applied to a typical asphalt pavement cross section in Massachusetts and used for all AASHTOWare PMED simulations. Thus, all inputs were kept constant except the material characterizations of the three TOPS types (six different mixtures), which included the properties of the asphalt binders, volumetric properties and unit weight of the mixtures, and the dynamic moduli and the creep compliance of the mixtures. The same properties were measured for the typically used in-service pavement onto which each of these TOPS types would be applied as a rehabilitation strategy. Because there was insufficient materials to measure the creep compliance of the original surface layer mixture of the in-service pavement, Level 3 default values were used for it.

8.1 Asphalt Binder Characterization Needed for AASHTOWare PMED

The binder properties needed for the software analysis were the complex shear moduli (G*) and phase angles (δ) at multiple testing temperatures and a frequency of 10 rad/s (1.59 Hz) after short-term aging the binders using the RTFO. These properties were measured for the AR and PMA binders, each obtained from the two sources. The RTFO-aged binders were tested in the DSR at 52°C, 58°C, 64°C, 70°C, and 76°C (125.6°F, 136.4°F, 147.2°F, 158.0°F, and 168.8°F).

8.2 Asphalt Mixture: Dynamic Modulus E*

The dynamic modulus (|E*|) is a major input to predict asphalt pavement performance using the AASHTOWare PMED. The |E*| of the six mixtures (three TOPS types by two binder sources) was performed in accordance with AASHTO T 378 "Determining the Dynamic Modulus and Flow Number for Asphalt Mixtures Using the Asphalt Mixture Performance Tester (AMPT)" (*17*) using four test temperatures of 4.4°C, 21.1°C, 37.8°C, and 54.4°C and six loading frequencies of 0.1, 0.5, 1, 5, 10, and 25 Hz at each temperature. Therefore, each specimen was tested at 24 combinations of temperature and load frequency. The measured |E*| for three replicate specimens of each mixture were loaded into an analyses tool called MasterSolver workbook, developed in NCHRP Project 9-29, to construct an |E*| master curve and to calculate |E*| at the temperatures and loading frequencies required to perform an AASHTOWare PMED analysis. Figure 8.1 shows the master curves for the six TOPS mixtures at a reference temperature of 21.1°C. The master curve of the typical in-service pavement used in the analysis is also presented.

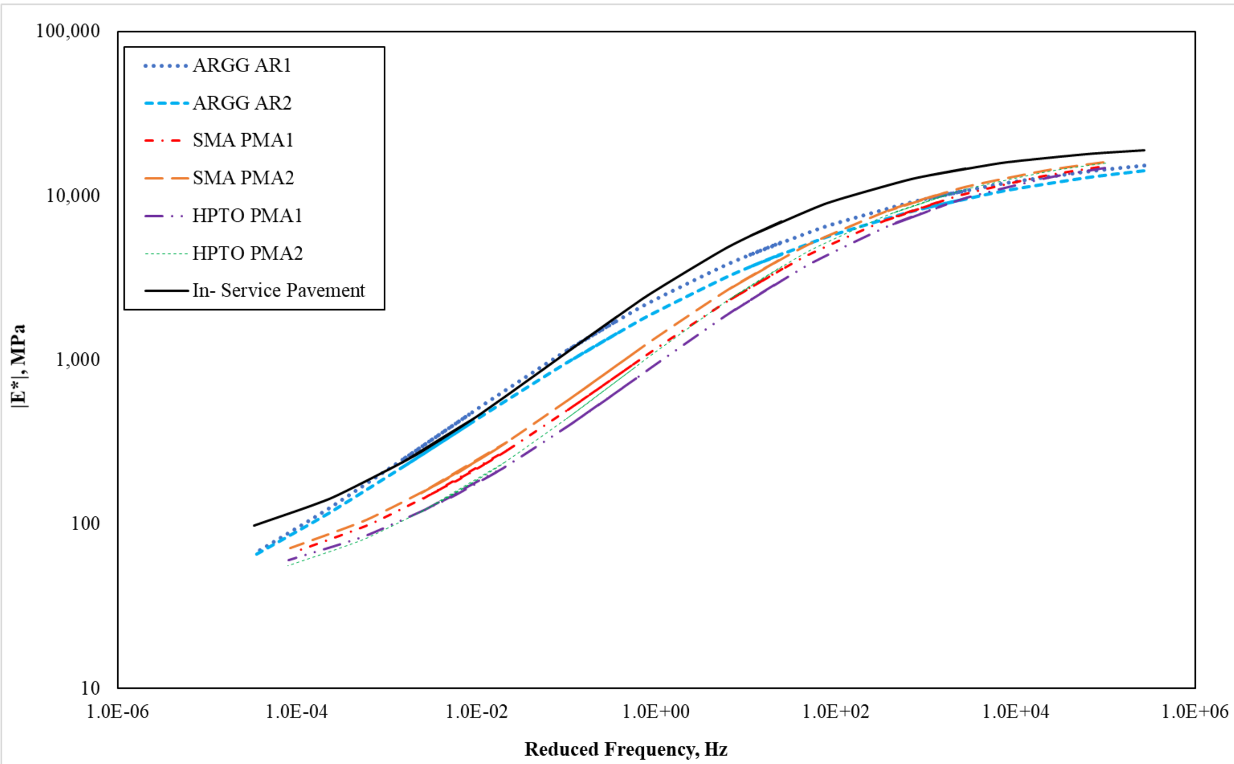

Figure 8.1: Master curves for six TOPS mixtures

8.3 Asphalt Mixture: Creep Compliance

As discussed in the low-temperature cracking section of this paper, creep compliance testing was conducted. An Excel workbook called "LTSTRESS" was used to develop the master curve of creep compliance versus time for the six mixtures as shown in Figure 8.2.

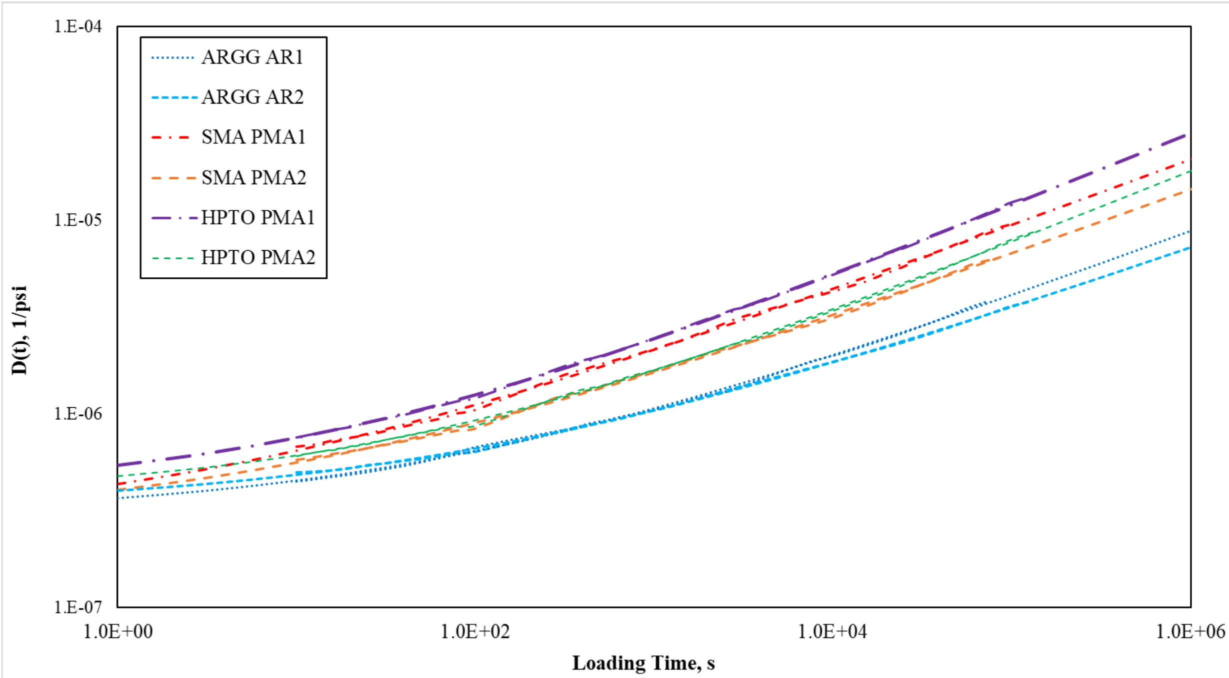

Figure 8.2: Creep compliance master curves for six TOPS mixtures

8.4 Prediction of Bottom-Up Fatigue Cracking and Thermal Cracking Using AASHTOWare PMED

To allow for a consistent comparison of the three types of TOPS, a common set of pavement structure, traffic, and climate inputs were used for all predictions. Analyses were conducted using a flexible pavement structure composed of a 9-in. asphalt pavement surface layer; a 15-in. crushed stone base layer with a resilient modulus of 22 ksi and a prepared A-2-4 subgrade soil with a modulus of 7 ksi. An initial two-way average annual daily truck traffic (AADTT) of 3,000 with a 3% annual linear growth factor, 50% trucks in the design direction, 80% trucks in design lane, 90% reliability, and a 20-year design life were used. These selections resulted in about 11.3 million trucks in the design lane over the design period. AASHTOWare PMED incorporates the laboratory measured G* and δ of the asphalt binders, volumetric properties and unit weight, and the |E*| master curve to predict fatigue cracking. In addition, it incorporates the creep compliance master curve and tensile strength to predict thermal cracking.

MassDOT has its own distress identification manual (*29*), which defines the extent of fatigue cracking as "light" when less than 15% of the pavement surface is cracked, thus indicating that this percentage of cracked area is acceptable. Also, the manual defines the extent of thermal cracking as "light" when approximately one transverse crack is observed every 1,500 ft/mile.

The AASHTOWare PMED analysis determined that the typical in-service pavement exhibited fatigue cracking in 15% of the pavement surface after 16 years of service. The analysis also showed that the in-service pavement would exhibit 216.3 ft/mile total predicted thermal cracking through its entire design period. Based on conventional pavement rehabilitation/preservation practices used in the Northeast region of the United States, the top 2 in. (51 mm) of the surface layer was removed when the predicted bottom-up fatigue cracking reached 15% of the lane area. These top 2 in. were replaced in the analysis with 2 in. of one of the six TOPS mixtures (three TOPS by two binder sources). When the fatigue cracking failure threshold was reached, the simulation was continued using the AADTT at the time of failure with a 3% annual traffic growth rate until the minimum 20-year design life was reached.

Figure 8.3 illustrates the AASHTOWare PMED predicted extension in the service life of the existing pavement when the TOPS were applied after 15% area cracked was initially reached. The TOPS extended the service life of the pavement between 11.2 and 14 years depending on the type of TOPS and source of binder. Thus, the maximum difference between the TOPS was slightly less than 3 years, although most of them were within 2 years of each other. Using Level 1 input properties for thermal cracking, the three types of TOPS exhibited thermal cracking substantially lower than the MassDOT definition of "light" thermal cracking.

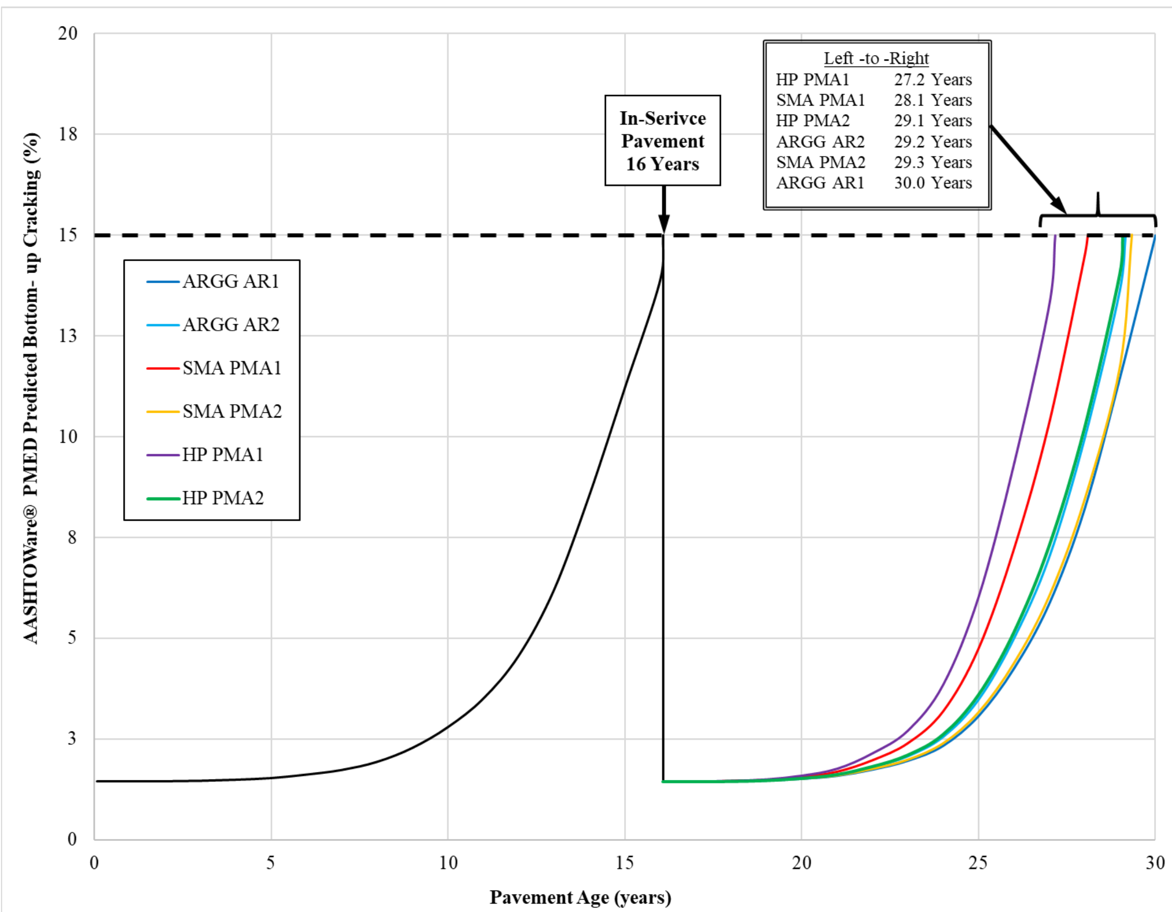

Figure 8.3: Predicted bottom-up fatigue cracking service life extension

9.0 Cost Analysis

The AASHTOWare PMED predictions shown in Figure 8.3 were used to perform LCCA using the FHWA RealCost. A 27-year analysis period was selected for the three TOPS using the two different asphalt binder sources (six alternatives).

A unit length of 1 mile (1.6 km) for a four-lane two-way highway was assumed for the analysis. For each alternative, the deterministic net present value (NPV) was determined using a discount rate of 4%. The NPV is a common economic indicator that is used to evaluate the return on investment (ROI) over a period of time among actions taken at different times by converting all costs during the analysis period to current year dollars using the discount rate; consequently, it allows fair comparison among different alternatives. The LCCA of the six alternatives were evaluated and are summarized in Figure 9.1 in terms of the NPV for agency, user, and total costs for all alternatives. The LCCA analysis indicated there was no significant difference in the NPV of the three TOPS, nevertheless, the binder source impacted the NPV.

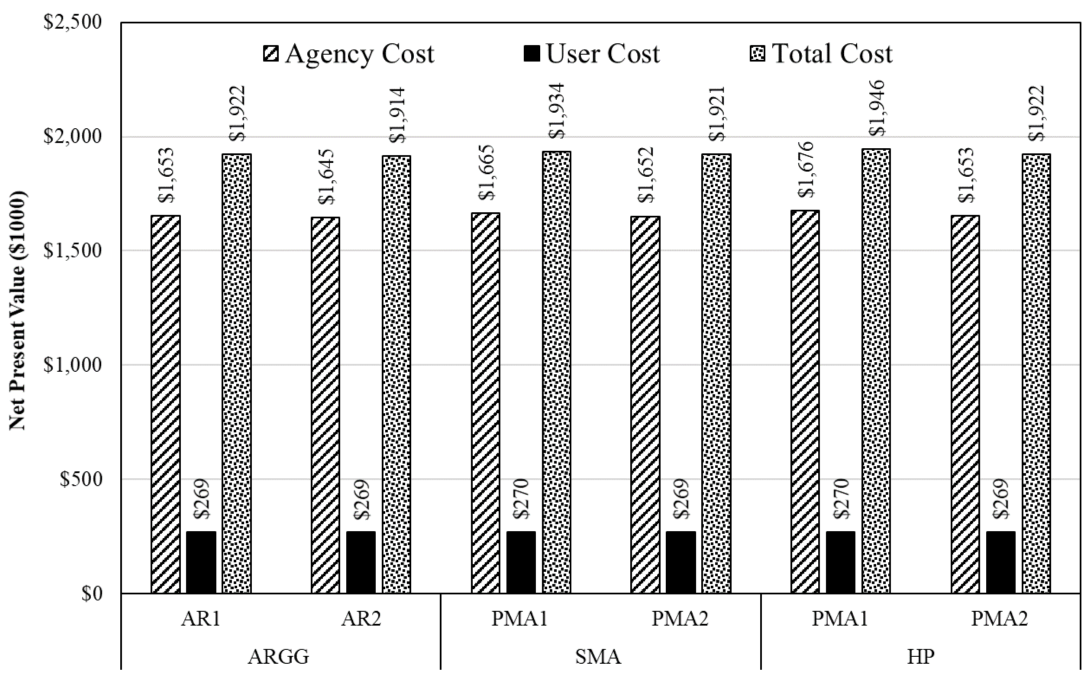

Figure 9.1: Life cycle cost analysis

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10.0 Conclusions

The objective of this study was to conduct a comparative evaluation of three MassDOT selected TOPS—namely, ARGG, SMA, and HP—in terms of performance, how they extend pavement service life, and whether they can be used interchangeably.

- Mixture designs for an ARGG, SMA, and HP were developed using two different asphalt binders that had the same PG but were obtained from two different sources. Two AR binders were used to develop the ARGG, and two PMA binders were used to develop both the SMA and HP (i.e., three TOPS types; six TOPS mixtures). The asphalt binders were tested to determine their resistance to low-temperature cracking, intermediate-temperature cracking, non-load-associated cracking, and rutting. The mixtures were tested to determine their resistance to low-temperature cracking, intermediate-temperature cracking, non-load-associated cracking, reflective cracking, rutting, and raveling.

- All TOPS passed the mixture performance tests where the test had a MassDOT specified or preliminary criterion, and the results indicated that the three TOPS types can be used interchangeably.

- More specifically, all TOPS met the MassDOT's pilot specification criteria for rutting, moisture damage, and intermediate-temperature cracking. Tests for reflective cracking and raveling indicated that all TOPS passed the criterion established by other STAs.

- AASHTOWare PMED predictive models for bottom-up fatigue cracking indicated that 2 in. of a TOPS placed on an existing in-service pavement can extend pavement service life between 11.2 and 14 years, depending on the type of TOPS and source of binder. Thus, the maximum difference between the six TOPS mixtures was slightly less than 3 years, although most of them were within 2 years of each other. Furthermore, none of the three TOPS types could be chosen as the best.

- AASHTOWare PMED predictive models for thermal cracking showed that all TOPS exhibited thermal cracking substantially lower than the MassDOT's definition of "light" thermal cracking.

- Statistical analysis indicated that both TOPS type and binder source had significant effects on the mixture performance tests except for the overlay test CFE results where the binder source did not have a significant effect. Although there were significant effects, as just presented, all TOPS passed the mixture performance tests where there was a MassDOT specified or preliminary criterion, and the AASHTOWare PMED predictive models did not show that any of the three TOPS types would perform better or worse than the others based on bottom-up fatigue and thermal cracking. Overall, the data did not indicate one of the three TOPS as superior to the others.

- The LCCA analysis indicated there was no significant difference in the NPV of the three TOPS, nevertheless, the binder source impacted the NPV.

11.0 Recommendations

The following recommendations are made:

- To confirm the overall outcome of this research, which is that the three TOPS can be used interchangeably, it is recommended that MassDOT select at least one pavement site to construct these three TOPS adjacent to each other and observe their performances over time. It is important that these experiments have the same underlying layers, traffic, and climate.
- Perform life-cycle assessments (LCA) to quantify the environmental impacts of each of the three TOPS.

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