Recommendations for Extending Asphalt Pavement Surface Life within Washington State

WA-RD 860.1

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October 2016

Washington State Department of Transportation

Office of Research & Library Services

WSDOT Research Report

17-05-0191

Research Report
WA-RD 860.1

Research Project T 1462-02

Optimizing Asphalt Pavement Performance

RECOMMENDATIONS FOR EXTENDING ASPHALT PAVEMENT SURFACE LIFE WITHIN WASHINGTON STATE

FINAL REPORT

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October 2016

1. REPORT NO. WA-RD 860.1	2. GOVERNMENT ACCESSION NO.	3. RECIPIENT'S CATALOG NO.
4. TITLE AND SUBTITLE **RECOMMENDATIONS FOR EXTENDING ASPHALT PAVEMENT SURFACE LIFE WITHIN WASHINGTON STATE**		5. REPORT DATE October 2016 6. PERFORMING ORGANIZATION CODE
7. AUTHOR(S) Haifang Wen, Stephen T. Muench, Skyler L. Chaney, Kevin Littleton, Tim Rydholm		8. PERFORMING ORGANIZATION REPORT NO.
9. PERFORMING ORGANIZATION NAME AND ADDRESS Washington State University Department of Civil and Environmental Engineering Pullman, WA 99164-2910		10. WORK UNIT NO. 11. CONTRACT OR GRANT NO. Agreement T1462, Task 02
12. SPONSORING AGENCY NAME AND ADDRESS Washington State Department of Transportation Transportation Building, MS: 47372 Olympia, WA 98504-7372 Project Manager: Lu Saechao, 360.705.7260		13. TYPE OF REPORT AND PERIOD COVERED Research Report 14. SPONSORING AGENCY CODE
15. SUPPLEMENTARY NOTES This study was conducted in cooperation with the U.S. Department of Transportation and the Pacific Northwest Transportation Consortium.		

16. ABSTRACT

This study identifies and evaluates hot mix asphalt (HMA) mix design and construction techniques with potential for improving WSDOT pavement surface life. WSDOT pavement failure mechanisms are found to be predominantly cracking. Rutting may reach a failure threshold first in areas with high traffic or studded tire use. A literature review, survey of state DOT practices, case study, WSPMS (Washington State Pavement Management System) data analysis, and limited laboratory testing identified 17 construction and mix design techniques with promise. Of these 17 techniques the use of stone matrix asphalt (SMA) mixtures for high-traffic interstate routes, 3/8-inch nominal maximum aggregate size mixes for medium/low traffic routes and mountain passes, non-Superpave aggregate gradation, reduced N-design gyration levels for Superpave mix design, and warm mix asphalt (WMA) to aid compaction are highly recommended for further investigation and implementation.

Other techniques recommended for further investigation and possible implementation are: adoption of a cracking performance test for Superpave mix design; applying a BST within a year of paving to reduce pavement surface aging; specifying the use of Pave IR for mountain pass jobs and cold weather paving; and using steel slag aggregate in situations that call for improved friction and resistance to studded tire wear (i.e., at mountain passes).

17. KEY WORDS Asphalt pavement; Cracking, Rutting, Climate, Studded tire wear		18. DISTRIBUTION STATEMENT No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22616	
19. SECURITY CLASSIF. (of this report) Unclassified	20. SECURITY CLASSIF. (of this page) Unclassified	21. NO. OF PAGES	22. PRICE

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EXECUTIVE SUMMARY

Hot mix asphalt (HMA) pavement performance in Washington State varies across the state's three broad climatic zones:

- **Western Washington**. West of the Cascade mountain range, the climate is classified as mild marine with warm, humid summers and cool, wet winters. The average surface life of WSDOT pavements in Western Washington is 16.7 years.
- **Eastern Washington**. East of the Cascades range is classified as continental with hot, dry summers and cold winters. The average surface life of WSDOT pavements in Eastern Washington is 10.9 years.
- **Mountain pass areas**. Mountain passes are associated harsh, cold, wet winters that greatly impact pavement life. The average surface life of WSDOT pavements in mountain pass regions is as low as 5 years, which may be due to a combination of issues ranging from mixture choice, structural design, construction practices, and climate impacts.

Study Purpose and Scope

This study identifies and evaluates HMA design and construction techniques with potential for improving WSDOT pavement surface life in Eastern Washington and mountain pass areas. Since this study was not intended to be an intensive laboratory investigation, evaluation of these techniques relies on corroborating multiple data sources rather than a compilation of statistically defensible experiments. Data sources used in this study are (in order of least-to-most impactful): literature review, DOT survey, the Washington State Pavement Management System (WSPMS), laboratory tests, case study, and cost analysis.

Key Findings

Predominant WSDOT HMA pavement failure mechanisms. WSDOT pavements tend to fail by cracking. From a statewide perspective, the WSPMS indicates that cracking first reaches critical thresholds requiring resurfacing 86% of the time, compared to only 13% for rutting. However, rutting plays a more substantial role as traffic levels increase. This is especially true in Eastern Washington, where studded tire use can more than double the rate of rutting.

Construction techniques to improve pavement surface life. Six techniques were identified and investigated (Table 1).

Table 1. Construction Techniques to Increase Pavement Surface Life Identified in the Literature Review and WSDOT's Experience with Them

Technique	Specified	Standard Practice	Allowed	Experimental	Not Done
Avoid late season paving	x				
Increase longitudinal joint density	x	x			
Mitigate temperature differentials: density profile	x				
Mitigate temperature differentials: Pave-IR				x	
Use intelligent compaction				x	
WMA as a compaction aid			x		

Additionally, a mountain pass paving case study was documented in order to better understand the impacts of some of these techniques and others required by *WSDOT Pavement Policy* (2015) for mountain pass paving. Significant observations from this case study are:

- The contractor had issues paving the 3/8-inch NMAS HMA largely because of plant production problems, long haul times (1.5 hours), and low mix temperature at laydown (about 220°F).
- WSDOT's Hamburg wheel tracking test requirement for the project caused the contractor to adjust their proposed mix design once.
- The straight PG asphalt used as a tack coat demonstrated pickup problems when placed on the old concrete pavement.
- Longitudinal joints were successfully constructed using the notched wedge joint and the cut back method.
- The Pave IR system was present on site but was not actively used.

Mix design techniques to improve pavement surface life. Eleven techniques were identified and investigated (Table 2).

Table 2. Mix Design Techniques to Increase Pavement Surface Life Identified in the Literature Review and WSDOT's Experience with Them

Technique	Specified	Standard Practice	Allowed	Experimental	Not Done
Non-Superpave aggregate gradation					x
Reduce N_{design}					x
3/8-inch NMAS			x		
Polymer modified asphalt	x				
Rubberized asphalt				x	
Lime addition				x	
SMA				x	
Steel slag aggregate			x		
Performance tests (rutting, stripping)	x				
Performance tests (cracking)					x
BST applied within one year of paving				x	

Recommendations

In general, WSDOT should focus on techniques to improve pavement cracking resistance. For all but high-volume pavements, this generally involves techniques to increase asphalt binder content, reduce surface aging, or additive use. For high-traffic and mountain pass pavements, improved rutting, raveling and studded tire wear resistance should be priorities, which generally involve specialty mix designs and additive use. Table 3 shows specific recommendations for the 17 techniques investigated.

Table 3. Recommendations for Techniques to Increase Pavement Surface Life

Technique	Recommendation	Priority
Construction		
Avoid late season paving	Continue current spec.	-
Increase longitudinal joint density	Continue current spec.	-
Mitigate temperature differentials: density profile	Continue current spec.	-
Mitigate temperature differentials: Pave-IR	Special provision	Low
Use intelligent compaction	No further investigation	-
WMA as a compaction aid	Special provision	High
Mix Design		
Non-Superpave aggregate gradation	Research project	High
Reduce N_{design}	Research project	High
3/8-inch NMAS	Implement policy	High
Polymer modified asphalt	Continue current spec.	-
Rubberized asphalt	No further investigation	-
Lime addition	No further investigation	-
SMA	Implement policy	High
Steel slag aggregate	Test sections	Low
Performance tests (rutting, stripping)	Continue current spec.	-
Performance tests (cracking)	Research project	Medium
BST applied within one year of paving	Test sections	Medium

It may not be necessary to implement all these techniques. The recommended approach is to begin implementing them with the most impactful first ("high" priority in

Table 3), and implement others as needed.

1 INTRODUCTION

1.1 Background

Asphalt pavements in Washington State exhibit different performance lives based on climate zones. The climate west of the Cascade Mountains is generally mild with wet winters, while the climate east of the Cascades is drier, hotter and sunnier with more extreme temperatures, which often drop below freezing during winter. Studded tires are more widely used east of the Cascades during winter time, which causes additional damage (stud wear in the wheelpaths) to the asphalt pavements, although the lower studded tire use rate in Western Washington is somewhat offset by higher overall traffic levels (Cotter and Muench 2010, Angerinos et al. 1999). The climate within the Cascade Range is generally mild in summer but more severe in winter with frequent snow and freezing conditions. The average surface life of pavements west of the Cascade Mountain rage (16.7 years) is significantly longer than those east of the Cascades (10.9 years); both of which are much longer than mountain pass areas (5 to 7 years) (WSDOT, 2012). These differences in asphalt pavement surface lives are likely due to a combination of factors including environment, mix design, construction, pavement structure, and related policies. This disparity in surface life highlights an opportunity in Eastern Washington and mountain pass regions to substantially improve pavement surface life. It may require changes in WSDOT pavement policy, mix design, and/or construction methods. This research project is an initial effort to identify the most promising options for improving pavement surface life.

1.2 Objective and Scope

The research objective is to identify and evaluate HMA design and construction practices, and identify those with potential for improving WSDOT pavement surface life in

Eastern Washington and mountain pass areas. This research was determined to be a data analysis project with limited laboratory investigation. The bulk of analysis is done using data (obtained from a literature review, survey, and WSPMS) with limited laboratory analysis of field samples (meaning laboratory analysis of only a few field locations with only a few replicates of each). Therefore, the evaluation approach this project uses relies on a “preponderance of evidence” (multiple data sources that show similar results or trends) rather than statistically defensible experiments. Data sources relied on for this approach are (in order of least-to-most impactful): literature review (evidence from others that a method has, will, or should work), DOT survey (to see what others are doing), WSPMS data search (WSDOT data that provides a performance history of a method), laboratory tests (experimental evidence used to corroborate what the literature review and WSPMS data suggest), and cost analysis (to assist in determining viability of some methods).

The intended outcome of this research is to catalog and prioritize viable options to extend WSDOT asphalt pavement surface life in Eastern Washington and Mountain Pass areas. Some options, however, can provide benefit across all climate regions. It is expected that WSDOT may select from this prioritized catalog the methods it considers most promising (and consistent with strategic direction) for future field evaluation and ultimate implementation.

1.3 Report Organization

This report first uses the WSPMS to identify predominant pavement failure mechanisms by broadly defined climate zone (Western Washington, mountain pass, Eastern Washington). This information is used to focus the research on techniques that would best address how pavements currently fail in these zones. Then the literature is reviewed for construction and mix design techniques that have potential to improve pavement surface life. These items are then corroborated with a survey of state DOTs, and categorized by their use within WSDOT (e.g., specified, standard practice, allowed, experimental, not used). For techniques that WSDOT has tried, a WSPMS data analysis and limited laboratory testing are used to corroborate findings in the literature review. This report consists of six chapters and three appendices.

- Chapter 1: Introduction. Provides project background and objectives.
- Chapter 2: Failure Mechanism Data Analysis. Identifies predominant WSDOT pavement failure mechanisms.
- Chapter 3: Literature Review, Survey, and Case Study. Literature review, state agency survey results, and a mountain pass construction case study.
- Chapter 4: WSPMS Analysis and Laboratory Test Results. Data analysis and laboratory tests done on specific WSDOT pavement sections that employ some of the identified techniques identified in Chapter 3.
- Chapter 5: Cost Analysis. Some techniques identified in Chapter 3 are analyzed for cost using price information from the literature and personal correspondence.
- Chapter 6: Conclusions and Recommendations.
- Appendix A: Complete state agency survey results.
- Appendix B: Details laboratory tests performed for this study.
- Appendix C: Mix designs of projects chosen for laboratory testing.

2 FAILURE MECHANISM DATA ANALYSIS

An analysis of data in the Washington State Pavement Management System (WSPMS) is used to identify predominant pavement failure mechanisms by climate zone (Western Washington vs. Eastern Washington).

2.1 Method

In general, failure mechanism is analyzed by obtaining WSPMS data related to Superpave paving contracts and analyzing the year paved, WSDOT Region, and pavement deterioration (as described by cracking, rutting, and roughness indexes) for each contract.

2.1.1 Climate Zones

For this study, climate zones are defined as follows:

- Eastern Washington: WSDOT Eastern, North Central, and South Central Regions
- Western Washington: WSDOT Northwest, Olympic, and Southwest Regions
- Mountain Pass: see Table 4.

Figure 1. Washington map showing WSDOT regions.

Table 4. WSDOT Pavements Defined to be in "Mountain Pass Areas" for this Study

Mountain Pass	Highway	State Route Milepost Limits	Regions
Blewett	SR 97	158 to 172	North Central
Satus	SR 97	21 to 31	Southwest – South Central
Snoqualmie	I-90	59 to 68	Northwest – South Central
Stevens	SR 2	57 to 78	Northwest – North Central
White	SR 12	140 to 160	Southwest – South Central

2.1.2 WSPMS Pavement Deterioration Data

Pavement deterioration data are based on WSPMS Survey Units, which are standard units of 0.1 mile of roadway length. Each paving contract contains multiple Survey Units, thus the condition of a pavement section described by a paving contract is a summary of the condition data from the individual Survey Units that together span the length of the contract. Three condition indexes are analyzed for each Survey Unit:

- **Pavement structural condition (PSC)**. PSC is a score given based on a combination of observed crack types (longitudinal, transverse, and alligator) and severity (low, medium and high). A PSC of 50 corresponds to approximately 10% high severity alligator cracking in the wheelpaths.
- **Pavement rutting condition (PRC)**. PRC is a score given based on rut depth. A PRC of 50 corresponds to a rut depth of 0.5 inches.
- **Pavement profile condition (PPC)**. PPC is a score given based on roughness. A PPC of 50 corresponds to an IRI value of 220 inches/mile.

2.1.3 WSPMS Pavement Deterioration Modeling

For each index, WSPMS fits a regression model to annual index data in order to predict when a pavement will reach a threshold indicating that the pavement should be rehabilitated (called the “Due Year” in that the section is due for resurfacing). Each Survey Unit is assigned an overall Due Year based on the minimum Due Year for each of the three indexes (PSC, PRC and PPC).

Only a small percentage of the asphalt pavements analyzed in this study (2% based on Survey Units) were considered due because of roughness (PPC). Typically, these sections are related to short abnormalities (e.g., isolated areas such as traffic rumble strips, dips due to unstable slopes, etc.) as opposed to issues with the deterioration of the pavement itself. Therefore, sections identified by roughness as the primary failure mechanism in WSPMS are excluded in this analysis.

2.1.4 Determining Pavement Surface Life

Because some pavements are kept in service for several years after reaching a designated rehabilitation threshold, for this study the predicted time to reach the rehabilitation threshold was considered the most consistent indication of pavement surface life (rather than the total time between overlays or construction activities, which has been used in other studies). However, use of this predicted time also introduces a confounding variable: in essence, the analysis treats the prediction as truth. To partially account for this, the analysis filters the WSPMS database to include only sections that were considered Superpave projects through 2007. Projects paved after 2007 would have most of their regression model based on a default model rather than actual condition information.

2.2 Population Description

Table 5 gives a summary of the 2,523.69 miles of Superpave projects analyzed using the previously described method.

Table 5. Miles of Superpave HMA Paving in the WSPMS Analysis by WSDOT Region and Climate Zone

WSDOT Region	Climate Zone			Totals
	Western	Mountain	Eastern	
Olympic	509.74			509.74
Southwest	363.58	1.75		365.33
Northwest	482.78			482.78
North Central		24.29	308.20	332.49
South Central		26.03	433.93	459.96
Eastern			373.39	373.39
All Regions	1,356.10	52.07	1,115.52	2,523.69

2.3 Predicted Failure Mechanisms

Figure 2 shows the distribution of failure mechanisms across the entirety of Washington, as well as in Eastern Washington, Western Washington, and over Washington mountain pass areas. In general, PSC reaches the WSPMS index failure threshold before PRC for most pavements across the State of Washington, indicating that cracking is the predominant failure mechanism. This does not mean that rutting does not exist, but simply that cracking reaches its index threshold value for rehabilitation before rutting does. Instances of rutting were somewhat higher in Eastern Washington than other areas, which may be an indicator of studded tire wear (Cotter and Muench, 2010) rather than mixture deformation. This is consistent with results from the state agency survey in Appendix A that indicates cracking is a primary failure mechanism in states with harsh climates similar to that of Eastern Washington. The states of Alaska, California, Utah, Colorado, Nevada, Nebraska, Oklahoma, Missouri, South Dakota, and Wisconsin indicated that top pavement failure modes include thermal and fatigue cracking.

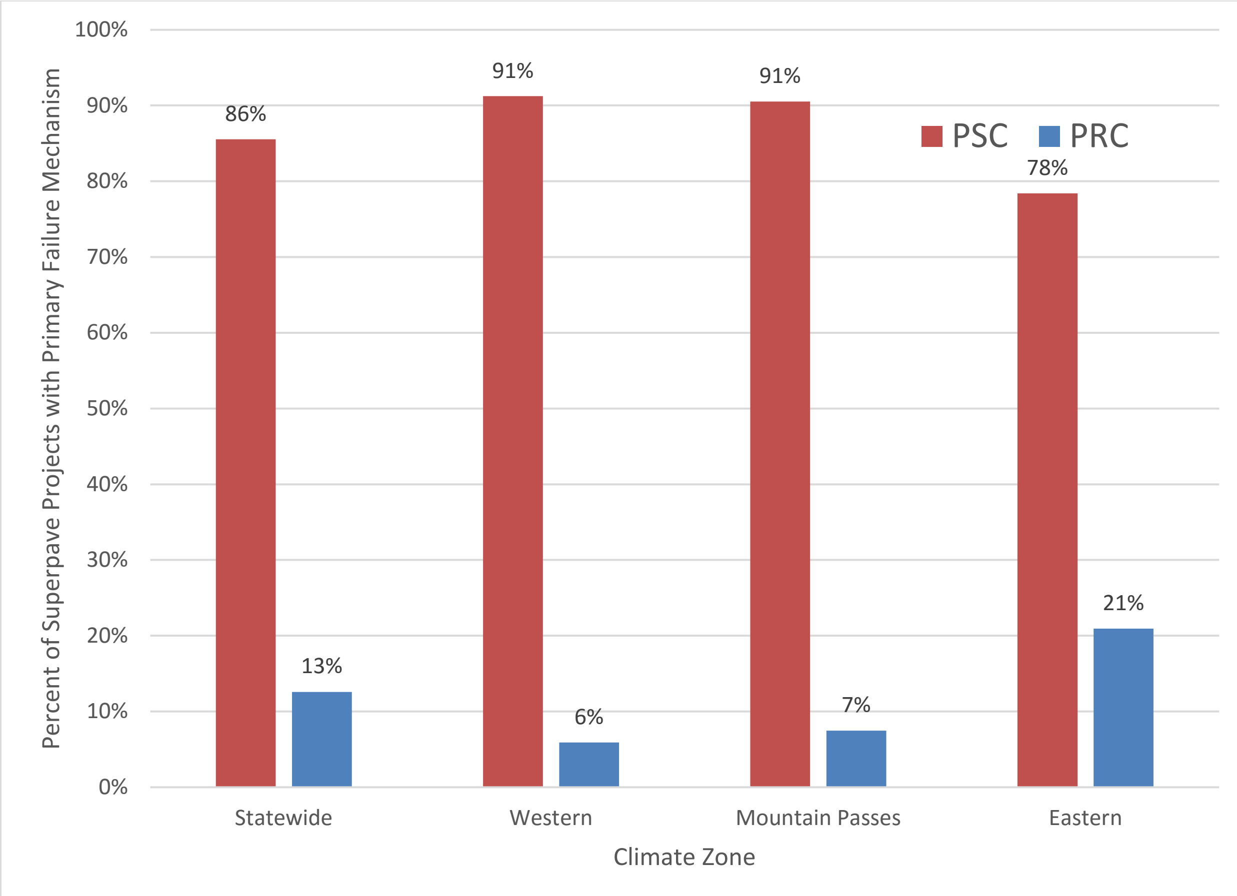

Figure 2. Primary failure mechanism by climate zone for Superpave projects completed through 2007. Most projects reach a cracking index rehabilitation threshold first indicating that cracking is the predominant failure mode.
PSC = Pavement Structural Condition; pavement fails due to cracking first.
PRC = Pavement Rutting Condition; pavement fails due to rutting first.

2.4 Predicted Failure Mechanisms by Traffic and ESAL Level

Table 6 and

show failure mode summaries for WSDOT Superpave projects completed through 2007 broken down into three AADT levels (Table 6) and three ESAL levels (

Table 7). These tables exhibit some general trends:

- **The time to reach a cracking rehabilitation threshold is relatively independent of traffic and loading**. This is somewhat expected. This means that environment (or something else consistent between all traffic levels in a climate zone) is the driver for cracking. If it were bottom-up cracking, which would indicate inadequate pavement structure, PSC average time to threshold would decrease with increasing traffic. If it were top-down cracking, loads could certainly contribute to failure, but environment (specifically its contribution to asphalt binder aging on the pavement surface) would be a significant contributor. The exception to this logic are mountain pass pavements with high traffic, which show a markedly shorter time to cracking failure.
- **The higher the traffic and loading levels, the more significant rutting becomes**. The fraction of paving projects with a WSPMS-generated rutting model (indicating a non-zero rutting progression over time) increases with increasing traffic and loading levels. This could be indicative of (1) mix design issues, or (2) studded tire wear. Since a majority of paving projects have essentially zero rutting it may be more likely that studded tire wear tends to drive rutting issues.
- **Western Washington pavement performance appears to be better than Eastern Washington and Mountain Passes**. This is consistent with WSDOT observations in the past.

As expected, as traffic and ESAL levels increase, the overall rutting life tends to decrease. Sections in Eastern Washington are more susceptible to this pattern, likely because of increased studded tire wear. Therefore, sections with high vehicle and freight traffic must find solutions that maximize both rutting and cracking resistance, while sections with lower traffic may be able to better focus on cracking resistance alone.

Table 6. Failure Mechanisms of WSDOT Superpave Paving Projects Completed Through 2007 by Traffic (AADT) Levels

Climate Zone	**Traffic Level[a]**	**Miles[b]**	**Fraction with a rutting model in WSPMS[c]**	**PSC Average Time to Threshold (yrs)**
Western	Low	455.32	4.2%	17.6
	Medium	399.41	16.1%	17.1
	High	459.03	50.1%	17.0
	All	1313.76	23.8%	17.2
Mountain	Low	28.9	5.6%	13.2
	Medium	18.12	20.9%	15.3
	High	6.76	45.3%	8.0
	All	53.78	15.8%	13.2
Eastern	Low	250.21	7.7%	14.5
	Medium	666.81	44.8%	13.9
	High	128.45	61.5%	14.0
	All	1045.47	37.9%	14.0

Notes:

a. Traffic levels are: Low (< 5,000 AADT), Medium (5,000-20,000 AADT), High is (> 20,000 AADT)
b. Centerline miles of pavement within the stated climate zone and traffic level category.
c. Represents the fraction of paving projects in which WSPMS had recorded enough change in rutting data over time to allow for the calculation of a rutting deterioration model. This is an indicator of the prevalence of significant (non-zero) rutting progression.
d. The average time (in years) for the modeled PSC index to reach the threshold for rehabilitation (PSC = 50).

Table 7. Failure Mechanisms of WSDOT Superpave Paving Projects Completed Through 2007 by Traffic (AADT) Levels Low is < 3 M, Medium is 3 to 10 M and High is > 10 M.

Climate Zone	Loading Level[a]	Miles[b]	Fraction with a rutting model in WSPMS[c]	PSC Average Time to Threshold (yrs)
Western	Low	1000.82	12.5%	17.4
	Medium	213.86	55.5%	16.9
	High	99.08	70.1%	16.7
	All	1313.76	23.8%	17.2
Mountain	Low	46.55	11.6%	14.0
	Medium	0.47	0.00%	17.0
	High	6.76	45.3%	8.0
	All	53.78	15.8%	13.2
Eastern	Low	631.15	30.4%	14.5
	Medium	366.7	44.2%	13.3
	High	47.62	89.1%	13.9
	All	1045.47	37.9%	14.0

Notes:

a. Loading levels are: Low (< 3 million 15-year ESALs), Medium (3-10 million 15-year ESALs), High is (> 10 million 15-year ESALs)
b. Centerline miles of pavement within the stated climate zone and traffic level category.
c. Represents the fraction of paving projects in which WSPMS had recorded enough change in rutting data over time to allow for the calculation of a rutting deterioration model. This is an indicator of the prevalence of significant (non-zero) rutting progression.
d. The average time (in years) for the modeled PSC index to reach the threshold for rehabilitation (PSC = 50).

3 LITERATURE REVIEW, SURVEY, AND CASE STUDY

A literature review was performed to determine possible construction and mix design techniques that might be used to increase pavement surface life in Eastern Washington and in mountain pass areas. The intention is to obtain a broad range of potential techniques that can then be reduced in number to a list of viable techniques based on WSDOT current practices and potential for use by WSDOT. For each technique discussed, a brief overview of the technique and potential benefits are included followed by relevant information from a survey of state DOTs (complete results in Appendix A), and a review of WSDOT experience with that technique.

A final section presents a case study of a mountain pass paving job that employed many of the required techniques listed in Section 9.2.6 of the WSDOT Pavement Policy (2015) document. The intention of this case study is to observe how WSDOT requirements are being carried out in the field.

3.1 Construction Practices

Most construction-related pavement surface performance issues identified are associated with compaction; either low density or non-uniform compaction. Construction practices that could potentially affect asphalt pavement surface life are described in the following sections.

3.1.1 Avoid Late Season Paving

Paving in cold weather can result in faster mix cool down and result in less time available for compaction. In some instances, cool down can be quick enough such that the pavement lift reaches cessation temperature before it is adequately compacted. Decker (2006) presents a concise review of the risks associated with cold weather paving. Because of these risks, many organizations contain specifications that identify standard dates when asphalt paving is not allowed, as well as specific temperature restrictions.

Survey information: none.

WSDOT experience: This idea is well-established in WSDOT and is included in the 2016 *WSDOT Standard Specifications for Roads, Bridges and Municipalities* (Amended 4 April 2016). Section 5-04.3(1) discusses weather limitations both seasonally, and based on current temperature (e.g., Figure 3). In the past, anecdotal evidence suggests that paving does occur outside the bounds of Section 5-04.3(1) and has contributed to early surface failure (e.g., Willoughby, 2000; Russell et al., 2010).

5-04.3(1) Weather Limitations

Do not place HMA for wearing course on any Traveled Way beginning October 1st through March 31st of the following year, without written concurrence from the Engineer.

Do not place HMA on any wet surface, or when the average surface temperatures are less than those specified in Table 5, or when weather conditions otherwise prevent the proper handling or finishing of the HMA.

Table 5
Minimum Surface Temperature for Paving

Compacted Thickness (Feet)	Wearing Course	Other Courses
Less than 0.10	55°F	45°F
0.10 to 0.20	45°F	35°F
More than 0.20	35°F	35°F

Figure 3. 2016 *WSDOT Standard Specifications for Roads, Bridges and Municipalities* (Amended 4 April 2016) Section 5-04.3(1) discussing weather limitations.

3.1.2 Increase Longitudinal Joint Density

Problems related to longitudinal joint construction, especially low density along the joint, can contribute to reduced asphalt pavement life. According to the Texas Department of Transportation (TxDOT, n.d.), poorly compacted longitudinal joints result in increased cracking and raveling along the joint. This allows water to enter and weaken the subgrade or flexible base, resulting in the deterioration of the pavement structure.

Construction of the notched wedge joint in hot mix asphalt is believed to improve long term performance of longitudinal joints and is gaining popularity in Texas due to the better compaction of longitudinal joints and the reduced slope of the drop-off that overnight traffic will drive over during paving (TxDOT, n.d.). A study by Williams (2011) in Arkansas on two highways, US 167 and US 65, evaluated eight longitudinal joint construction techniques. Results indicate that use of a joint heater, joint stabilizer, and notched wedge methods are the most successful techniques to achieve density and resist permeability and infiltration. Joint adhesives reduced the permeability in the finite area of application, but not for the area surrounding the joint; therefore, the joint stabilizer is recommended instead. A study by NCAT (Kandhal et al. 2002) recommends rubberized joint adhesives or notched wedge joints and advises rolling a confined lane on the hot side 6 inches (150 mm) from the joint with the cold lane for the first roller pass. Kandhal et al. (2002) also state that joint density should be determined using cores, as nuclear density gauges may give erroneous results on joints.

Survey information: Many states indicate specified techniques for longitudinal joint construction. Techniques mentioned in the survey as having success include use of longitudinal joint heaters, paving in echelon, specifications for joint densities, use of notched wedge joints, coring for density at the joints, use of joint adhesive or heavy tack coat on longitudinal joints, and paving and trimming six inches. Additionally, Illinois DOT indicated that Stone Matrix Asphalt (SMA) pavements resist moving from the roller and compactor, leaving a straight edge.

WSDOT Experience: This idea is well-established in WSDOT and is included in the 2016 *WSDOT Standard Specifications for Roads, Bridges and Municipalities* (Amended 4 April 2016). Section 5-04.3(12)B specifies longitudinal joint location at a lane or edge line, and the use of a notched wedge joint in the wearing surface of all new HMA.

3.1.3 Mitigate Construction-Related Temperature Differentials

Construction-related temperature differentials can reduce pavement life (Willoughby, et al., 2001). General procedures to mitigate construction-related temperature differentials are:

- **Thermal imaging**. Use a hand-held infrared thermal camera to identify isolated areas in the mat with significantly lower temperatures (often defined as 10-25°F lower) than the surrounding mat.
- **Pave-IR**. An infrared temperature monitoring system mounted on the back of an asphalt paver capable of capturing a continuous record of mat surface temperature. Sebesta and Scullion (2012) document how TxDOT implemented Pave-IR as a standard test method (Tex-244-F) and integrated it into the HMA specification. The current *2014 TxDOT Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges* allow paving anytime the roadway is dry and the surface temperature is at least 32°F if the contractor is using a “thermal imaging system” (essentially, Pave-IR) as defined by test procedure Tex-244-F.
- **Density profile**. Taking multiple densities along a skewed profile line (often about 50 ft.) behind the paver. The goal is to identify isolated areas of significantly lower density than the rest of the mat, which would likely indicate resultant areas of low density caused by construction-related temperature differentials.

Survey information: none.

WSDOT Experience: WSDOT has been a leader in identifying, analyzing, and implementing procedures to address this phenomenon. The *2016 WSDOT Standard Specifications for Roads, Bridges and Municipalities* Section 5-04.3(10)B identifies the standard WSDOT procedure to evaluate HMA for low cyclic density. WSDOT has also used Pave-IR experimentally but has no formal specification for its use. On one particular project, observed as part of this study, the Pave-IR was used, observed to be turned on, but was not functioning because it could not obtain a GPS signal to locate its thermal readings. While the contractor had the Pave-IR turned on, it was effectively not used.

3.1.4 Adjust minimum density requirements

Adjustments to minimum density requirements may affect pavement surface life. Currently, WSDOT has other ongoing efforts to evaluate their density specification, which are more detailed than this. Therefore, this study will not evaluate density requirements.

Survey information: Most agencies specify a minimum density of 92% or higher. A few agencies, such as Delaware and Utah, specify 93% or higher and North Carolina uses 95%. It is recommended that the current 91% minimum density in WSDOT's specification be increased to 92% or higher.

WSDOT Experience: Currently, WSDOT uses a statistical evaluation for materials acceptance (Section 1-06.2(2) of the *2016 WSDOT Standard Specifications for Roads, Bridges and Municipalities* (Amended 4 April 2016)) for HMA quantities of 4,000 tons or more, and a non-statistical acceptance plan for < 4,000 tons. For each, the lower specification limit is 91% of theoretical maximum density.

3.1.5 Use Intelligent Compaction to Achieve Consistent Compaction

Intelligent Compaction (IC) is a new development for rollers that uses global positioning satellite (GPS) technology to monitor compaction while operators are rolling (Van Hampton, 2009). This can help prevent over-compacting and under-compacting by providing operators with feedback while they are rolling and by automatically changing drum frequency and amplitude. IC is used by some agencies. The Minnesota Department of Transportation (MnDOT) reports better uniformity, performance, and longevity, and has increased the amount of information available to both the contractor and agency (Johnson, 2012).

Survey information: none.

WSDOT Experience: WSDOT has conducted an IC demonstration on the SR 539, Lynden-Aldergrove Port of Entry Improvements project (25-28 August 2014). This demonstration project investigated two different IC rollers (Hamm HD+ 140, Caterpillar CB54XW) and compared intelligent compaction measurements with nuclear gauge densities, core densities, falling weight deflectometer (FWD) asphalt layer modulus calculations, and lightweight deflectometer (LWD) asphalt layer modulus calculations. In general, IC is able to provide compaction estimates, mat temperature, and number of passes in map or frequency format. Simple linear regression analysis of the data indicated a generally poor correlation of IC compaction estimates with core and nuclear gauge densities. Poor correlation was generally explained by measurement property differences: IC measures a form of layer modulus (stiffness), while nuclear gauge and cores measure density.

3.1.6 Warm Mix Asphalt (WMA) as a Compaction Aid

In addition to environmental benefits, Warm mix asphalt (WMA) technologies can aid in compaction by reducing the viscosity of asphalt concrete during the laydown process and, therefore, allowing better compaction at lower temperatures. Despite this difference during construction, most have concluded WMA to be equivalent to HMA in terms of long-term field performance if designed and constructed correctly (e.g., West, et al., 2010; Bonaquist, 2011; Wen, 2013). Preliminary conclusions of NCHRP Project 9-49A, a comparison of long-term field performance of WMA and HMA (Wen, 2013), support this view. Issues with moisture susceptibility do not seem to be significant so long as mixtures are properly evaluated for moisture susceptibility and proper precautions are taken (Epps et al., 2014; Bonaquist, 2011). Specific benefits related to asphalt pavement surface life are compaction-related (Kristjansdottir et al., 2007):

- Reduce compaction risks associated with cold weather.
- Lower the risk of poor compaction when working with stiff mixtures.

One specific WMA study is directly applicable to mountain pass pavement performance. In 2011 Aschenbrener et al. (2011) reported that after three years of evaluation WMA performance on Interstate 70 in mountain terrain areas (elevations in the range of 8,800 to 11,100 feet) was equivalent to that of traditional HMA.

In addition, contractors will use the WMA method, but heat it to over 300°F to ensure workability, resulting in a “hot warm mix” that is especially beneficial when paving in cold weather or in mountain pass areas with a long haul (Guy Anderson, personal communication, July 21st, 2014).

Survey information: none.

WSDOT Experience: WMA is specified in the 2016 *WSDOT Standard Specifications for Roads, Bridges and Municipalities* (Amended 4 April 2016). Section 5-04.2(2)B allows the contractor to use WMA processes for producing HMA after Engineer's approval, which is a simple description of the WMA process used along with the manufacturer's recommended temperature limits. For 2014, Hansen and Copeland (2015) estimate WMA to constitute about 14% of the total estimated WSDOT consumption of asphalt concrete, with 87% of that total coming from plant foaming processes and 25% of companies/branches reporting using WMA.

3.2 Mix Design

The materials of which a pavement is constructed have direct consequences on the pavement performance. The following materials-related aspects have been reviewed for their impact on the surface life for HMA pavements.

3.2.1 Non-Superpave Aggregate Gradation

Factors to consider in the design of the asphalt pavement wearing course include the mix design, pavement thickness, and structure design (Hicks et al. 2012). Fromm and Corkill (1971) found that hard volcanic or synthetic stones and coarser mixes with higher percentages of stone resist wear better than softer sedimentary stones, and higher asphalt content gives better wear resistance. However, the selection of aggregate source is economically controlled by the geography. In regards to mix design, AASHTO (1997) advises that aggregate in gap-graded mixtures segregate more than in dense-graded mixtures. To reduce segregation, gradations with two to four percentage points above the maximum density curve for fine mixes, and two to four points below the curve for coarse mixes are recommended, creating a bowed curve, as shown in

Figure 4 Gradations that make an “S” curve, as shown in Figure 5, tend to have segregation problems.

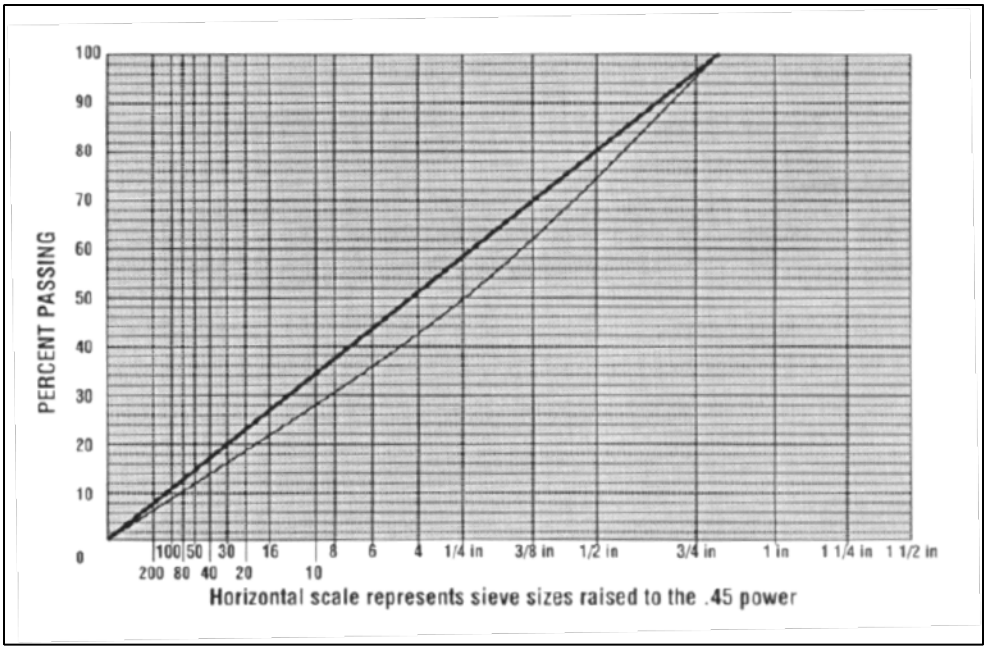

Figure 4. Bowed Gradation Curve (AASHTO, 1997)

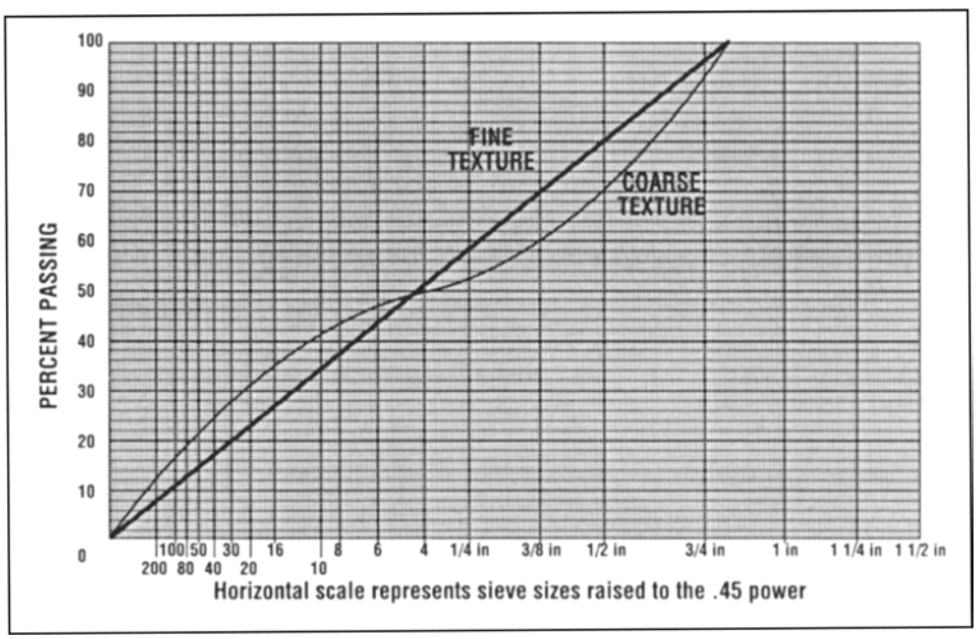

Figure 5. “S” Gradation Curve (AASHTO, 1997)

Survey information: Gradations used in Georgia, Nebraska and Oklahoma tend to be finer than many other states.

WSDOT Experience: The 2016 *WSDOT Standard Specifications for Roads, Bridges and Municipalities* (Amended 4 April 2016) only specify standard Superpave gradation bands from AASHTO M323.

3.2.2 Increased Asphalt Content

In *NCHRP Report 567* Christensen and Bonaquist (2006) state,

> "It appears that current HMA mixtures tend to be somewhat leaner (lower in asphalt binder content) compared with mixtures designed and placed prior to the implementation of Superpave; this may be a contributing factor to the observed frequency of raveling and surface cracking in Superpave mixtures. Because the Superpave system has encouraged the use of coarse aggregate gradations—below the maximum density gradation—they also contain relatively few fines, which, in combination with relatively high in-place air voids, can result in mixtures with high permeability and less resistance to age hardening."

Essentially, Superpave mix design adoption has (perhaps unintentionally) led to lower asphalt contents, and coarse aggregate gradations with few fines that have contributed to more raveling and surface cracking. The implications for WSDOT may be less severe since WSDOT's legacy Hveem mix design method tends to result in mix designs a bit closer to typical Superpave asphalt contents than the more common legacy Marshall method.

Increasing the asphalt binder content can be done in multiple ways, including (1) decreasing the mix design compactive effort (i.e., the number of design gyrations in Superpave mix design), or (2) reducing the nominal maximum aggregate size (NMAS).

Reduce N_{design}

Multiple studies (Li and Gibson, 2011; Mohammad and Shamsi, 2007) have suggested that the current Superpave recommended number of design gyrations (N_{design}) are higher than necessary and additional gyrations do not benefit the mixture but can actually result in over-compaction and damage to the aggregate skeleton.

In *NCHRP Report 573* Prowell and Brown (2007) found that laboratory densities obtained using AASHTO M323 gyration levels were 1.5 percent more than ultimate in-place density of pavements evaluated in their study. Therefore, they concluded laboratory compaction effort was too high and recommend reducing N_{des} levels (and providing separate criteria for ≥ PG 76-XX binders or mixes not placed within 4 inches of the surface) as shown in Table 8. They also question the utility of $N_{initial}$ and recommend eliminating $N_{maximum}$ since neither seems to indicate rutting potential. WSDOT SOP 732 Volumetric Design for Hot-Mix Asphalt (HMA) uses the AASHTO M323 values (which are based on 15-year ESALs).

Table 8. Recommended N_{design} Levels Compared to Existing AASHTO M323 Levels (from Prowell and Brown, 2007)

20-Year Design Traffic, (millions of ESALs)	AASHTO M323	N_{des} for binders < PG 76-XX	N_{des} for binders ≥ PG 76-XX or mixes placed > 4 inches (100 mm) from surface
< 0.3	50	50	NA
0.3 to < 3	75	65	50
3 to < 30	100	-	-
3 to < 10	-	80	65
10 to < 30	-	80	65
≥ 30	125	100	80

Prowell and Brown (2007) concluded that reduced N_{design} should improve in-place density and, accompanied with a 0.5 percent increase in VMA, increase the optimum asphalt content.

Survey information: Seven of eight agencies that indicated they modify their Superpave design procedure mentioned increasing asphalt content, generally by decreasing the design gyrations. The Ohio DOT recommends fewer gyrations for more binder content, and North Carolina DOT reports they have decreased gyrations and increased liquid asphalt content to reduce cracking. Illinois DOT stated they are working with the University of Illinois to quantify the value of polymers and high asphalt content.

WSDOT Experience: WSDOT SOP 723 uses the current AASHTO M323 gyration levels.

3/8-inch Nominal Maximum Aggregate Size (NMAS)

A smaller NMAS (3/8-inch) can also be used to indirectly increase mixture design asphalt content because the higher surface-to-volume ratio of the smaller aggregate sizes requires relatively more asphalt to coat these surfaces. Additionally, smaller NMAS is appealing because of (1) decreased permeability (Newcomb, 2009), (2) reduced segregation issues, and (3) better workability during construction leading to better compaction.

According to Adam Hand of Granite Construction (personal communication, April 23rd, 2014), 3/8-inch mixes are used in California as a "bonded wearing course" with a specification for film thickness, percent material passing the No. 200 sieve (P200) in the range of 5%-8%, and a low Los Angeles (L.A.) Abrasion specification of 25-30 maximum. For these bonded wearing course pavements, it is essential to have enough mastic in the mix; otherwise, the pavements will ravel quickly. The 3/8-inch bonded wearing course has shown to be successful on a section of US 395 from Bridgeport to Bishop, which is a mountain pass of elevation 8,000 feet.

Additionally, according to Adam Hand, I-65 from Indianapolis to Chicago is predominantly 3/8” asphalt pavement and has performed well.

Survey information: none.

WSDOT Experience: WSDOT has constructed multiple pavements using 3/8-inch NMAS HMA; at least two on I-90 in the Snoqualmie Pass region. WSDOT Pavement Policy (2015) recommends considering HMA Class 3/8-inch for the wearing course for mountain pass regions.

3.2.3 Polymer Modified Asphalt

Modifying the asphalt binder with polymers is one of the most extensive and successfully used methods in cold regions. When added to an asphalt binder, some polymers have shown to expand a pavement’s ideal temperature range to increase resistance to cracking in cold temperatures and resistance to rutting in warm temperatures, as shown by Mix I, compared to Mix III in Figure 6. This is one of several ways contractors can increase the PG grade of Superpave mixes.

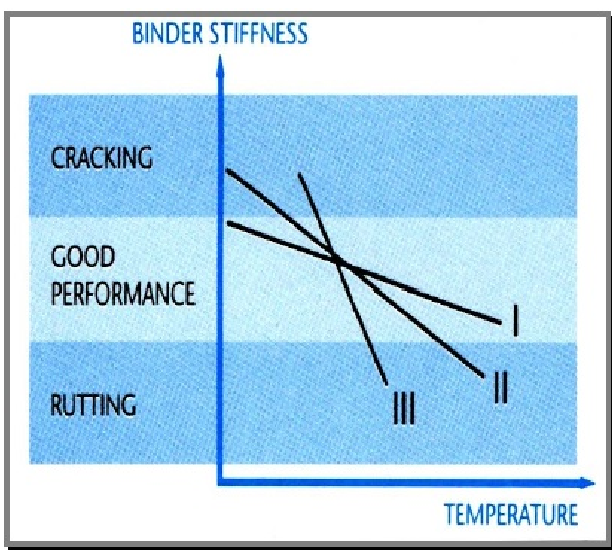

Figure 6. Polymer expansion of the ideal temperature range (IDOT, 2005)

Polymer modified asphalts (PMAs) make asphalt concrete more viscous and increases adhesion between the aggregate and the binder (Deb, 2012). This also increases resistance to rutting under heavy and slow moving truck loads. A survey of state agencies showed that polymer modified asphalts have shown to extend a pavements life by up to 60% and significantly reduce maintenance costs by reducing the effects of thermal cracking (Glanzman, 2005; Peterson and Anderson, 1998; Von Quintus and Mallela, 2005). PMAs have been successfully used in regions that experience extreme temperature variations. The use of polymer modified dense graded asphalt works very well, according to the Maryland Department of Transportation (Gloria Burke, personal communication, June 23, 2014). Though there may be environmental concerns with the effects of using polymers in asphalt mixes, they do not produce any more emissions than HMA in the mixing plant (Bethard and Zubeck, 2002).

There are many different types of polymers that can be used to improve properties of asphalt concrete. Elastomers are some of the most widely used polymers, such as styrene-butadiene rubber (SBR), styrene-butadiene-styrene (SBS), and styrene-isoprene-styrene (SIS) (Lewandowski, 2004). Plastomers such as polyethylene (PE) and ethylene-vinyl-acetate (EVA) have been shown to strongly resist rutting and improve the layer coefficients of modified asphalt binders as much as 75-85% (Qi et al. 1995). However, EVA tends to be more brittle and does not perform as well in cold temperatures (Stroup-Gardiner and Newcomb, 1995). Sulfur-extended asphalt modifiers (SEAMs) can reduce effects of rutting and thermal cracking and improve overall strength of asphalt mixtures (Chuanfeng and Yazhi, 2011). However, asphalt binders respond differently to different polymer additives. While increasing the polymer concentration in a binder will generally decrease the accumulated strain, the same polymer will not necessarily give the same result when used in different binders (Chen et al. 2002; MTE, 2001). The polymer dosage rate is generally 3 percent to 5 percent polymer by weight asphalt, depending on which polymer is used (Deb, 2012).

A performance study of 28 projects in Colorado found that using polymer modified asphalt mixtures extended pavement life by 2 to 10 years and reduced fatigue cracking, rutting, and transverse cracking (Von Quintus and Mallela, 2005). In Utah, a study of various combinations of PMA and HMA indicated PMAs reduced thermal cracking, delayed reflective cracking, and helped resist rutting in an area of I-70 exposed to many freeze-thaw cycles (Anderson, 2002). Field surveys in Alaska indicated use of PMAs decreased thermal cracking, and the most effective modifiers were SBS, SBR, crumb rubber modifiers (CRM), and ULTRAPAVE (Raad et al. 1997). In an investigation of the performance of multiple PMAs the Dalles-California Highway (Zhou et al. 1993), transverse cracking distressed all of the sections, though the polymer modified sections showed less loss of aggregate than the HMA control sections. In Kentucky, SBR modified pavements outperformed unmodified HMA in thermal cracking (Lewandowski, 2004).

Polymer modification often leads to an extension of performance grade. Increasing the PG grade of an asphalt binder is an important factor in improving asphalt pavement durability (Christensen and Bonaquist, 2005) as well as resistance to stripping and rutting (Gogula et al. 2003). A mix with PG grade increase from PG 58-28 to PG 70-28 was shown to have improved resistance to bottom-up cracking and top-down fatigue cracking, as well as rutting resistance, including resistance to studded tire wear (Wen and Bhusal, 2015). From the state agency survey, multiple states including Colorado, Georgia, and Minnesota indicated the importance of proper PG binder selection for extending pavement life in harsh climates. It should be noted that simple increase of PG grade does not necessarily leads to a good performance. For example, the use of Polyphosphoric Acid (PPA) to extend the PG grade has been controversial (FHWA 2012).

Survey information: Alaska, Nebraska, and Nevada use polymer-modified asphalt as a method to increase the life of asphalt pavements in their states.

WSDOT Experience: The 2016 *WSDOT Standard Specifications for Roads, Bridges and Municipalities* (Amended 4 April 2016) Section 9-02.1 contain an elastic recovery requirement (AASHTO T 301) residue for PG 64-28, PG 70-22, PG 70-28, and PG 76-28 binders. This specification is a generalized means to require a polymer-modified binder.

3.2.4 Rubberized Asphalt

Crumb-rubber modified asphalt, or rubberized asphalt, has shown to perform well in cold regions by providing resistance to wear caused by studded tires, and generally does well in resisting tensile and thermal cracking (Takallou et al. 1987). Addition of crumb rubber makes the binder thicker, which improves the ability of the pavement to resist aging due to oxidization, and the durability can improve by use of carbon black (Papagiannakis and Lougheed, 1995). Rubber-modified asphalt is generally more expensive due to the increased asphalt content and rubber content; however, this can be offset by the longer life and better performance (Takallou et al. 1987).

According to Adam Hand (personal communication, April 23, 2014), rubberized asphalt is used throughout California. The California Department of Transportation (Caltrans) has specifications for wet process rubberized asphalt (crumb rubber is blended with the asphalt binder before being added to the aggregate), terminal blend dry process rubberized asphalt (crumb rubber is used as part of the fine aggregate), and an “M Specification” that allows for either method as long as certain criteria are met. Cities and counties in California also use rubberized asphalt, following Caltrans specifications in Northern California and the slightly different Greenbook Committee specifications in Southern California. Caltrans uses rubberized asphalt with gap-graded and open graded asphalt mixes. The wet process binder requires an open graded mix to allow room in the aggregate matrix for the rubber particles which are not fully dissolved. The rubber content of wet process binders is approximately 18% to 20% by weight, and is used when reflective cracking is the primary concern. It has lasted up to 7-10 years on pavement sections with extensive cracking, whereas conventional HMA would likely only last 2-4 years. The terminal blend behaves similarly to an SBS polymer modified binder, and does not produce the same results as the wet process, specifically in regards to cracking resistance. The terminal blend employs approximately 10% to 20% rubber by weight. Caltrans does not allow recycled asphalt pavement (RAP) with rubberized asphalt, as it may contain additional rubber that would interfere with the mix design. This has resulted in decreased use of RAP in California, as Caltrans has increased the rubber content requirement over the years. According to Dave Jones (personal communication, 2014), there is a concern about low-temperature paving of rubberized pavements in California.

Rubberized asphalt pavements have been implemented to reduce the rutting problem in Alaska; this includes projects in Fairbanks (Saboundjian and Raad, 1997) and Anchorage (Bingham et al. 2010). Saboundjian and Raad reported the rubberized sections were comparable to the HMA control sections for fatigue cracking and outperformed the HMA in resisting transverse cracking. Bingham et al. reported reduced rutting for the rubberized sections, and the dry process performed best to reduce rutting from studded tire wear. Bingham et al. noted that some of the rubberized sections constructed in the 1980’s in Anchorage were still in service in 2010.

Survey information: none.

WSDOT Experience: WSDOT has had two periods of experience with rubberized asphalt pavements. The first, stretching from the late 1970s to early 1990s, involved a number of experimental projects using various forms of rubber asphalt in chip seals, stress absorbing interlayers, and open-graded friction courses (OGFCs). In general, rubber asphalt was found to be more expensive and be at greater risk of early failure (Anderson and Jackson, 1992). The second, was in the 2006-2009 time period when it paved experimental “quieter pavement” sections that, amongst other mixtures, use a rubberized asphalt binder for an OGFC. While initially such pavements produced appreciably less tire-pavement noise, within 6 months they were as loud as traditional HMA control sections and deteriorated quickly due to studded tire wear and snow chain usage (Anderson et al.; 2008, Anderson et al., 2012; Anderson, et al., 2013).

3.2.5 Lime Addition

An asphalt pavement modification that has shown to be successful in harsh climates is the addition of lime to hot mix asphalt pavements. According to Berger and Huege (2006), whether used alone or in addition to polymer modifiers, lime has proven to decrease the effects of moisture damage and increase a pavement’s resistance to rutting, fatigue cracking, aging, and oxidation. Hydrated lime helps resist stripping caused by moisture by strengthening the bond between the aggregate and the asphalt binder. The National Lime Association (2006) details the process by which lime causes a reaction with the bitumen and calcium hydroxide, which prevents reactions with the environment that can cause oxidation and premature aging later on. Lime can also increase a binder PG grade to make it more durable to high temperatures without getting too stiff in low temperatures. In the mountainous regions of France, lime is used as part of a "mountain mix," that is believed to outperform liquid antistripping additives in resisting moisture damage and oxidation, and provide better adherence (Collet, 2012; Didier Carré, Personal Communication, May 5th, 2013). Kennedy and Anagnos (1984) stated it was better to add hydrated lime slurry to the asphalt mixture than to add dry hydrated lime. However, both are effective treatments to improve the moisture damage resistance and freeze-thaw durability. Huang et al. (2005) determined that mixing the hydrated lime with the asphalt directly resisted moisture damage better than adding the hydrated lime to the aggregate before mixing. Based on the agency survey, Nevada Department of Transportation also specifies the use of lime in the mix.

Survey information: none.

WSDOT Experience: WSDOT tends to use a liquid anti-strip additive instead of lime. WSDOT has limited experience with lime as an antistrip: WSDOT specified the use of lime on one recent project detailed in Section 3.3 of this report.

3.2.6 Stone Matrix Asphalt

Gap-graded asphalt mixtures can provide increased resistance to permanent deformation based on their stone matrix skeleton (providing stone-on-stone contact) as well as fatigue cracking due to their increased binder content. Widely used in northern and central Europe for over 25 years, stone matrix asphalt (SMA), or stone mastic asphalt, provides stone-on-stone contact and high asphalt content that increase durability and resistance to rutting (Michael et al. 2003) as well as improved wet weather performance and noise reduction (Root, 2009). Fibrous materials and polymers may be used in SMA to increase resistance to permanent deformation; however, Al-Hadidy and Tan (2010) found that the use of fibrous material gives the best overall structural stability.

SMA has shown to perform well in hot climates (Asi, 2006) but is also used in states with extreme temperature fluctuations. In Norway, SMA is used in conjunction with polymers and has proven to perform well against rutting in the extreme Nordic climate (Bjørn Ove Lerfald, personal communication, October 31, 2013). It is noted that in that region, rutting due to studded tire wear is predominant (and, in fact, the original reason for SMA mixtures), when compared to rutting due to plastic deformation. SMA is also frequently used with polymers in Ontario, Canada (Brown, 2007). SMA mixes have been used in Illinois, combined with steel slag to increase the strength of the mix (National Slag Association, Publication 203-1). SMA mixtures with a 3/8-Inch NMAS are often used as thin overlays for pavement rehabilitation. In South Dakota and Wyoming, SMA mixtures with a polymer modified asphalt binder and 3/8-Inch aggregate are used (Root, 2009).

In the U.S., most of the literature on SMA performance is found in Maryland and Georgia, where it has proven to perform well against rutting and roughness for periods exceeding 10 years. According to the Maryland Department of Transportation (Gloria Burke, personal communication, June 23, 2014), SMA is widely used in Maryland on high volume roads and can last between 12-15 years, which outperforms the HMA.

Survey information: 15% of agencies that responded said SMA is used as wearing course in climates similar to Eastern Washington. Colorado DOT indicated an average life of 18 years for SMA compared to 12 years for HMA, Minnesota 15 years for SMA compared to 10-16 for HMA, Missouri 20-25 years for SMA compared to 15-20 for HMA, and Utah 16-20 years for SMA compared to 10-16 for HMA. Maryland indicated an average of 13 years for SMA compared to 16 years for HMA statewide, but noted that SMA pavements are used in areas which have much more traffic. Illinois indicated that SMA pavements resist moving from the roller and compactor, leaving a straight edge, making longitudinal joint construction more effective than conventional HMA.

WSDOT Experience: WSDOT paved four ½-inch NMAS SMA projects from 1999-2004 totaling about 48,000 tons (Myers, 2007). Two performed well, while one had mix design and construction issues causing sections to be replaced (SR 524 64th Ave. W to I-5 in Lynnwood in 1999), and one (I-90 Ritzville to Tokio paved in 2000) experienced severe flushing and was replaced the following year. The other two SMA jobs (I-90 SR 21 to Ritzville in 2001, and I-90 Dodson Rd. to Moses Lake in 2004) remain in place and in good condition.

3.2.7 Steel Slag Aggregate

Replacement of fine or coarse aggregate with steel slag aggregate (SSA) in asphalt mixtures has shown to strengthen a mix by improving the indirect tensile strength, resilient modulus, creep modulus, and resistance to rutting and stripping (Ahmedzade and Sengoz, 2009; Asi et al. 2007). According to the National Slag Association (n.d.), SSA is much harder than aggregates such as limestone. It is used in both hot mix asphalt and stone matrix asphalt to give more friction and shear strength due to the better aggregate interlock and high coarse and fine aggregate angularity. This makes the mixture strong, cohesive, durable, and provides resistance to abrasive wear as well as moisture damage (Ahmedzade and Sengoz, 2009). Wen and Bhusal (2014) recommend

SSA for use in the Northwest region of the United States for its durability and resistance to studded tire wear.

In 1997, Illinois Department of Transportation (IDOT) constructed a steel slag SMA mix (Table 9) at the intersection of Margaret and Williams Streets in Thornton, Illinois, a roadway that has carried nearly 16 million ESALs of heavy truck traffic as of 2013 (Murphy, 2013). An evaluation of the pavement after 16 years showed that the pavement had basically needed no maintenance and was continuing to perform well; it has been called “the world’s strongest intersection” (Murphy, 2013). According to Ross Bentsen (personal communication, July 3, 2014), IDOT has used steel slag extensively as a “friction aggregate” for high traffic surface mixes. Steel slag aggregate has also been used on the Illinois Tollway and has shown a comparable life to HMA performance, though its performance hasn’t been thoroughly tracked. As long as the mixture was designed and constructed effectively, the aggregate type has not mattered for pavement life. However, the availability of steel slag is decreasing due to the decreasing steel production in Northwest Indiana.

Table 9. SMA Steel Slag Mix Design (National Slag Association, n.d.)

IDOT Mix Design

GRADATION	CM 11 Steel Slag	CM 13 Steel Slag	FA 20 Dolomite	Mineral Filler	FIBER Slag	BLEND	SPECIFICATION
1”	100	100	100	100	100	100	100
¾”	100	100	100	100	100	100	100
½”	39	100	100	100	100	84.8	85 – 100
3/8”	9.0	78.4	100	100	100	64.1	26 -78
#4	4.0	21.0	99.5	100	100	27.7	20 -28
#8	3.0	6.9	84.0	100	100	17.8	16 -24
#16	3.0	7.8	52.4	100	100	14.5	
#30	3.0	4.4	29.2	100	100	12.7	12 – 18
#50	3.0	4.1	15.7	100	100	11.7	12 -15
#100	3.0	3.4	10.0	99	100	10.8	
#200	1.5	3.0	7.1	83.9	100	8.9	8 – 12

In 1994, Oregon Department of Transportation (ODOT) constructed a section of hot mix asphalt with 30% steel slag on U.S. Highway 30 (Lower Columbia River Highway), an area with moderate climatic conditions (Hunt and Boyle, 2000). When analyzed in 2000, it showed no noticeable difference from the conventional mix in rutting performance or skid resistance. It was noted that there was a 15% reduction in coverage due to the increase in weight.

In Sweden, steel slag has been used as a surface course aggregate in test sections of multiple mixtures, including SMA and various grades of hot mix asphalt (Göransson and Jacobson, 2013). From 2005 to 2012, the test sections were subjected to a high truck volume and intense Swedish winters, during which there is generally a high use of studded tires. When evaluated, the SSA sections did not show any stone loss, which is a common problem with studded tires plucking out the aggregate, and had good friction.

Survey information: none.

WSDOT Experience: Recycled Steel Furnace Slag is a Standard Specifications item: 9-03.21(1)D. WSDOT investigated the use of slag aggregate as a result of a legislative mandate in 2015 (WSDOT Construction Division Pavement Office, 2015). Recommendations from this report were (1) testing to determine the expansive nature of slag aggregate and its long-term strength under repeated loads, (2) treat slag aggregate projects as experimental and compare their performance to a control HMA section, and (3) do not use slag aggregate as an unbound base material due to potentially high pH of some steel slag aggregates. Current availability of steel slag aggregate in Washington State comes entirely from Nucor Steel in Seattle and is about 80,000 tons/year. Availability varies based on steel output.

3.2.8 Performance Tests

Mix design performance tests such as the Hamburg Wheel Track Test (HWTT) for rutting and moisture susceptibility can differentiate performance of mixes when compared to the volumetric-based specifications that do not include a performance test. Currently, WSDOT has specified rutting index based on HWTT test results. In addition, elastic recovery is also included in the specification of asphalt binders. However, what is lacking is a performance-based specification for cracking. The thresholds of these tests need to be established based on local climate and materials. The multiple stress creep and recovery (MSCR) test evaluates rutting performance, verifies the use of polymers, and eliminates the need for the Elastic Recovery (ER) test (FHWA, 2011).

Results from the state agency survey indicate that several states, including Alaska, California, Colorado, Georgia, Oklahoma, and Utah require the HWTT or Asphalt Pavement Analyzer (APA) for mixes. Alaska, California, Colorado, Kentucky, Missouri, Nebraska, and Tennessee also indicated specification of the ER test for modified binders. Delaware indicated use of the Indirect Tensile (IDT) test, and Oklahoma indicated use of the MSCR test.

Survey information: none.

WSDOT Experience: WSDOT uses the Hamburg Wheel-Track Test (AASHTO T 324) as a performance test for rutting and stripping. Currently, no performance test for cracking is used.

3.2.9 Bituminous Surface Treatment (BST) Applied within One Year of Paving

A bituminous surface treatment (BST), or chip seal, can be used to cover an asphalt pavement immediately after paving. According to Rolt (2001), a surface treatment such as a BST overlay can reduce the risk of top-down cracking due to age hardening of the top 0.1 inches of asphalt cement of the wearing course. Essentially a “sunscreen” layer to prevent aging of surface asphalt cement from sun exposure. Of course, BST have also been shown to reduce longitudinal, transverse, and fatigue cracking, as well as effectively seal and protect centerline joints (Galehouse et al. 2005).

WAIt is the common practice of the Montana Department of transportation to place a BST overlay immediately following HMA paving (Dan Hill, personal communication, November 20, 2013), however this may be more to address chronic raveling issues rather than to reduce surface asphalt age hardening: it has been noted in a study by Von Quintus and Moulthrop (2007) that applying a BST overlay after paving hot mix asphalt in Montana has decreased the amount of raveling due to stripping, compared to adjacent states by over 30 percent. Additionally, in areas where BST overlays were placed, the amounts of transverse, longitudinal, and fatigue cracking were much less compared to other asphalt pavement sections. Von Quintus and Moulthrop estimated that HMA pavements constructed with an initial BST overlay as a preservation strategy experienced a service life extension of over five years. The BST overlay practice immediately after construction has shown mixed performance over Montana mountain Passes (Dan Hill, personal communication, November 27, 2013). The traffic volume in these areas is generally not as high as the mountain passes in Washington State, with a high of 12,000 ADT and an average daily traffic (ADT) of 3,000 to 6,000. Montana DOT uses two types of BST grades, Grade 4A and Grade 2A, also known as Type I and Type II. Type II has less material passing the No. 4 and No. 2 sieves, and may also contain larger chips, as shown in Table 10. Type II tends to be more durable in harsh environments and performs better on mountain passes and is recommended if chip seals are to be implemented in mountain passes in Washington State.

Table 10. BST Gradation (Montana DOT, 2006)

PERCENTAGE BY WEIGHT PASSING SQUARE MESH SIEVES					
Sieve Size	**Grade 1A**	**Grade 2A**	**Grade 3A**	**Grade 4A**	**Grade 5A**
5/8 inch (16.0 mm)	100				
1/2 inch (12.5 mm)		100	100		
3/8 inch (9.5 mm)	33-55	40-100	95-100	100	100
No. 4 (4.75 mm)	0-15	0-8	0-30	0-15	9-50
No. 8 (2.36 mm)	0-5	-	0-15	-	2-20
No. 200 (0.75 mm)	0-2	0-1	0-2	0-2	2-5

Survey information: none.

WSDOT Experience: WSDOT has had some experimental experience with applying BSTs to newly overlayed pavements.

3.3 Mountain Pass Paving Case Study

Historically, the Washington State Department of Transportation (WSDOT) has experienced several early-life failures and generally shorter surface life associated with mountain pass HMA pavements. In an effort to improve mountain pass pavement surface performance WSDOT has, over time, implemented several new construction, testing, and materials requirements (WSDOT, 2015).

This section describes paving Contract 8443 as a case study to examine the use and impact of the following requirements:

- 3/8-inch HMA
- Hamburg wheel tracking device
- Straight PG tack coat
- Longitudinal joints (notched wedge and sawcut)
- Lime as an anti-strip additive
- Pave IR

3.3.1 Contract Description

Contract 8443, MP 65.54 to Easton Hill EB & WB – Paving, was a $5.4 million resurfacing project located on I-90 from MP 65.54 to MP 67.43 near Easton Hill on the East side of Snoqualmie Pass. Notice to proceed was given on 17 May 2013 with HMA paving starting on 8 July 2013. This rehabilitation project included cracking and seating the existing 8-inch thick portland cement concrete pavement (PCCP), for the westbound lanes only, followed by a full width overlay of 0.20 ft HMA Cl. 3/8-inch PG 64-28. The eastbound lanes were not cracked and seated because the inside lane concrete slabs ran under the centerline barrier is some areas. The HMA plant was located in Ellensburg, WA, about 40 miles away, and all paving was done at night (Hicks, 2015). RAP was used on the project but not in the mix design (Hicks, 2015), which is typical for WSDOT mix designs.

3.3.2 Construction Process

This section describes how each of the evaluated techniques occurred in the field.

3/8-inch NMAS HMA

Using a 3/8-inch NMAS HMA should provide the benefits listed in Section 0, which can reduce the risk of poor construction due to harsh environmental conditions present on Snoqualmie Pass (e.g., low temperatures, longer haul distances, etc.). It may also reduce raveling issues associated with winter conditions and snow plows. For a variety of reasons, likely not due to the 3/8-inch NMAS, the mix paved during observation of this contract was problematic and may contribute to lesser pavement performance.

Four Superpave mix designs were approved for use by the contractor on this job; two N_{design} = 75 gyration mixes for the lower lifts (used in areas where full-depth paving was required) and two N_{design} = 100 gyration mixes for the top/overlay lift. One of each of these mixes contained lime added at 0.6% (the intention was to add at 1.0% but a plant calibration issue resulted in the lower lime content).

This project was the first time the contractor's Ellensburg HMA plant had produced a 3/8-inch NMAS mix and the contractor had trouble keeping the fine aggregate fractions within JMF limits. WSDOT gave the contractor the opportunity to place a section of the N_{design} = 100 gyration mix as an underlying HMA layer prior to the surface course test section so they could verify the mix could be properly produced and placed, but this opportunity was not used. Ultimately, the project required three test sections before paving was allowed to continue. In order to pass the third test section the contractor removed all RAP from the mix, which reduced the fines to within JMF tolerances. Subsequently, RAP was included in the production mix. Compacting the mix also proved somewhat problematic: during the observed test section the contractor noticed that the first roller pass only resulted in 88-89% of theoretical maximum density at a temperature of 220°F, but would more readily compact at temperatures of 170-200°F.

Density continued to increase after the third roller pass even if it was several hundred yards behind the paver. Overall, the composite pay factor (CPF) for the test strips ranged from 0.77 to 1.01. CPFs for the job were mostly between 1.01 and 1.05 (12 of 15 CPFs) with three low CPFs of 0.75, 0.88 (removed and replaced), and 0.86.

Hamburg Wheel Test

The Hamburg Wheel Tracking Device (HWTD) was specified for this job with requirements of less than 10 mm rut depth after 20,000 passes along with no stripping inflection point after 15,000 passes. For this job to contractor sent their mix design out-of-state to their testing facility for the HWTD test. Their first job mix formula (JMF) failed the HWTD test causing them to adjust their JMF, which then passed the HWTD test.

Straight Performance Grade Asphalt Tack Coat

A straight PG binder asphalt tack coat (no emulsion) was specified for this contract (Figure 7). The contractor used a sub-contractor to place the tack coat. Tack coat placed on the existing concrete pavement suffered from excessive pickup in the wheelpaths as noted by direct researcher observation and WSDOT Inspector Daily Reports (Hicks, 2015). Several methods were used to ensure the tack coat was being placed on a clean surface (i.e., multiple sweeper passes along with individual workers walking the milled area behind the sweeper with air compressors to remove as much of the fine material as possible) but it could be seen that the majority of the tack in the wheel paths was being picked up by the trucks on the night observed. When placed on the newly laid asphalt the majority of the tack remained in place.

Figure 7. Straight PG grade tack coat placement.

Longitudinal Joints (notched wedge and sawcut)

For this contract both a notch wedge joint (westbound lanes, Figure 8) and a saw cut joint (eastbound lanes, Figure 9) were used with performance grade asphalt binder applied to all joints. Specifically, notched-wedge joints were used for all lifts on the WB lanes and all but the top lift of the EB lanes. The top lift for the eastbound was specified as a cut back joint. The contractor chose to place an extra 6-inch width and then use a milling machine to remove that extra width. Both types of wearing course joints were coated with a joint adhesive (Figure 10 and Figure 11), which was done after the sweeper had removed all debris.

Figure 8. Notched wedge joint.

Figure 9. Cut joint (done using a milling machine).

Figure 10. Joint adhesive applicator.

Figure 11. Joint adhesive application on milled joint.

Lime as an Anti-Strip Additive

One of each of these N_{design} = 75 and N_{design} = 100 gyration mixes contained lime added at 0.6%. The original intention was to add lime at 1.0% but a plant calibration issue, discovered only after job completion, resulted in the lower lime content. In general, the addition of lime was preferred by both the contractor and WSDOT during testing based on Mix Design Report C08443. One byproduct of lime use in place of a liquid anti-strip (WSDOT's usual method) was a lower asphalt binder content. After the project, the contractor was required by WSDOT to hire an independent expert to evaluate the impacts associated with lime addition at 0.6% instead of the required 1.0% (Hicks, 2015). This report concluded that rock loss and raveling were due primarily to surface abrasion from studded tire wear and not from moisture damage caused by low lime addition rates (Hicks, 2015).

Pave IR System

This contract specified the use of a Pave IR system (Figure 12). The intention was to collect Pave IR data for informational purposes, therefore, only its use was specified while data collection requirements or data use was not. As a result, the Pave IR system did not seem to be actively used. Specific issues were:

- The Pave IR equipment initially provided was faulty, which eliminated time the contractor had planned for training on it.
- Issues with the GPS occurred on the first night the replacement equipment was used. A MOBA representative was able to come to sight the second night of using the Pave IR and resolved the GPS issue along with answering some questions the crews had. The repair was short lived as the issue reappeared shortly thereafter.

Figure 12. Pave IR installed.

Other Pave IR notes on use:

- The Pave IR did not have wireless capability, which would have allowed both the contractor and WSDOT to view the results more closely. Despite this, a few comparisons were done by jobsite personnel to see if any cold spots were being detected.
- According to jobsite personnel, the Pave IR appeared to not detect areas with a >25°F temperature differential that could be seen with an infrared thermometer.
- The large range of temperatures in the legend (Figure 13) made identifying problem areas difficult. However, there were not many problem areas because the contractor used a Roadtec Shuttlebuggy, which reduces construction-related temperature differentials (Willoughby, et al., 2000).

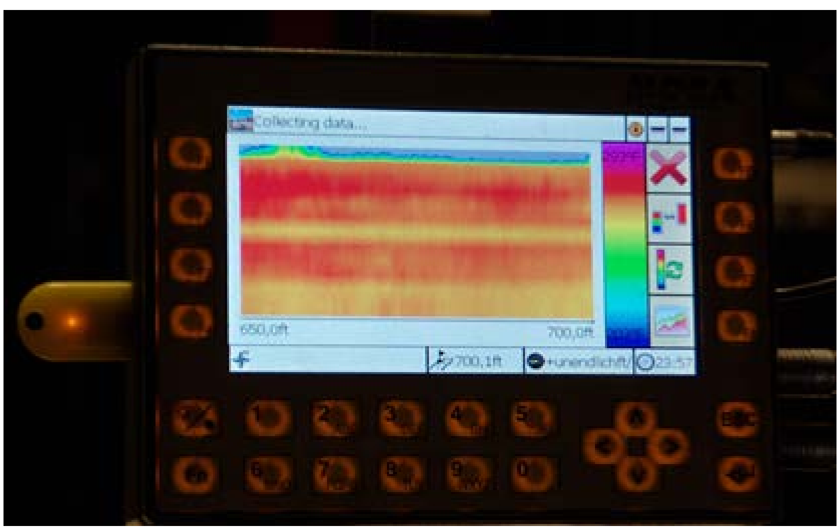

Figure 13. Pave IR display screen.

Other Project Observations

Several other items were observed that may impact pavement quality. For night work (the majority of paving on this contract) on average mix did not arrive on site until after midnight and then throughout the night large delays would occur as the crew awaited truck arrivals. It was not uncommon to see four or more trucks in the queue at a time. The long travel time from plant to site, an average of 1.5 hours, combined with other delays caused mix temperature to be close to 200°F at placement. The method for dumping the belly dump trucks was to drop the entire windrow at once and then send the truck back to the plant rather than keeping the mix in the truck until needed.

3.4 Literature Review, Survey, and Case Study Summary

This literature review provides a brief review of major construction and mix design techniques that show potential for increasing pavement surface life. Table 11 summarizes the WSDOT experience with each of these techniques.

Table 11. WSDOT Experience with Techniques to Increase Pavement Surface Life Identified in the Literature Review

Technique	Specified	Standard Practice	Allowed	Experimental	Not Done
Construction					
Avoid late season paving	x				
Increase longitudinal joint density	x	x			
Mitigate temperature differentials: density profile	x				
Mitigate temperature differentials: Pave-IR				x	
Use intelligent compaction				x	
WMA as a compaction aid			x		
Mix Design					
Non-Superpave aggregate gradation					x
Reduce N_{design}					x
3/8-inch NMAS			x		
Polymer modified asphalt	x				
Rubberized asphalt				x	
Lime addition				x	
SMA				x	
Steel slag aggregate					x
Performance tests (rutting, stripping)	x				
Performance tests (cracking)					x
BST applied within one year of paving				x	

Of the 16 identified techniques in the literature review those identified as "allowed", "experimental," and "not done" may warrant further consideration.

- **Pave-IR**. Done experimentally on several paving projects, however no formal evaluation of the method or attempt to develop a specification for its use has been done. Therefore, its impact is unknown and warrants further investigation. Based on the case study experience, Pave IR will not have an impact until its use is properly specified.

- **Intelligent compaction**. Used experimentally on one project. Not formally evaluated. Further investigation may be warranted, however its direct impact on pavement surface life will be difficult to determine.
- **WMA as a compaction aid**. WSDOT, like most DOTs, uses a permissive WMA specification. In mountain pass regions, where compaction can be particularly difficult due to cold temperatures, it may be beneficial to require several compaction aids to lessen the risk of a poorly compacted pavement.
- **Non-Superpave aggregate gradation**. Superpave gradation bands are notoriously broad and substantial evidence exists that more tightly controlled gradation bands may reduce the risk of pavement failure due to aggregate structure. Notably, prior to Superpave mix design adoption with the 2004 Standard Specifications, WSDOT had such gradation bands (i.e., Class A, B, D, E, F, and G).
- **Reduce N_{design}.** There appears to be a growing consensus in the U.S. that AASHTO M 323 gyration levels are too high, which is somewhat confirmed by survey responses. Prowell and Brown (2007) present compelling arguments for their recommended lower gyration levels and corresponding 0.5% increase in VMA. These lower gyration levels warrant consideration by WSDOT.
- **3/8-inch NMAS**. WSDOT has some experience using 3/8-inch mixes and initial results are good, although no long-term studies have yet to be done. Given that 3/8-inch NMAS mixes are less prone to segregation, are likely to have higher asphalt content, are easier to compact, and can be more workable such mixes warrant consideration by WSDOT. Long-term performance associated with the case study

contract may not be representative of 3/8-inch NMAS mix long-term performance because of the construction issues encountered.

- **Rubberized asphalt**. WSDOT has twice undertaken research efforts to evaluate rubberized asphalt. Both times, results have been poor. It is not likely that WSDOT has the appetite to revisit rubberized asphalt in the near future.
- **Lime addition**. WSDOT does not use lime as an anti-strip agent and the evidence for lime's influence on pavement surface life outside of its anti-strip qualities is minimal. However, lime used in the case study seemed to be positively received, however it was added at a lower-than-specified rate. Further consideration of lime addition should be a low priority for WSDOT.
- **SMA**. There is substantial evidence in the U.S. that properly constructed SMA pavements outperform HMA equivalents. The two SMA pavements properly constructed by WSDOT are still in place. SMA performance will be further analyzed in Section 4.3 and should be considered by WSDOT.
- **Steel slag aggregate**. It is likely that steel slag aggregate will perform adequately and even offer some benefits in HMA. Given the limited supply in Washington (about 80,000 tons/year, which is enough for about 100 lane-miles of a 2-inch overlay), slag aggregate should only be considered for specialty use (e.g., high-friction requirement, combat studded tire wear), however other considerations (extended haul distances, limited sources) may restrict use. WSDOT has proposed a paving project in 2016 that uses between 20 and 25 percent slag.
- **Performance tests (cracking)**. WSDOT does not currently have a cracking performance test. Consideration to adopting one should be given.

- **BST applied within one year of paving**. Evidence from Montana suggests improved performance, and WSDOT has some pavement where a BST was applied within one year of paving. These sections are further investigated in Section 4.4.

Based on the previous review, the following recommendations are made for WSDOT consideration:

- Incorporate Pave-IR and WMA into specifications for mountain pass paving.
- Return to more tightly controlled aggregate gradation bands similar to legacy Class A/B specifications.
- Reduce N_{design}, which (along with a recommended increase in VMA) will result in higher asphalt content and potentially extend pavement surface life for those pavements suffering from cracking failure.
- Use 3/8-inch NMAS mixes in a wider range of projects. Their potential to reduce compaction and segregation risk, ease of workability, higher asphalt content, and lower permeability for a given density are attractive benefits.
- Investigate the performance of the two remaining SMA mixes on I-90.
- Consider adoption of a cracking performance test.
- Investigate the properties and performance of existing WSDOT pavements that have employed a BST within one year of paving.

The next sections of this report provide further insight into specific techniques that WSDOT has tried experimentally and are able to be analyzed using WSPMS data and laboratory testing.

4 WSPMS ANALYSIS AND LABORATORY TEST RESULTS

A number of techniques summarized in Section 3.4 are either already WSDOT standard practice, or have already been used experimentally by WSDOT. These techniques are:

- 3/8-inch NMAS
- Rubberized asphalt
- SMA
- BST applied within one year of paving

Projects associated with each of these techniques are identified, evaluated using WSPMS data, and, where an associated control section could be identified, evaluated using a limited suite of laboratory tests. The purpose of these analyses is to corroborate actual WSDOT data and first-hand laboratory tests with expected performance based on the literature review.

4.1 3/8-inch NMAS Projects

Four 3/8-inch NMAS contracts, paved between 2009 and 2014, were identified in WSPMS. One, C8611 was placed concurrently with a ½-inch mixture, which warrants comparative laboratory testing, and one, C7763, had enough data to warrant WSPMS data evaluation.

4.1.1 Contract 8611: I-90 Barker Road to Idaho State Line

Contract 8611 was paved in 2014 and is located on I-90 from Barker Road to Idaho State Line in the Eastern Region. The Eastbound right lane from MP 297.956 to MP 298.335 is 3/8-inch PG 70-28 HMA and the rest of the project is ½-inch PG 70-28 HMA. Since the performance data is limited due to recent construction, a performance data review in WSPMS was not performed. The presence of an adjacent ½-inch NMAS section provides the opportunity for comparative laboratory tests. Tests conducted for this project include:

- Studded tire wear resistance (Figure 14). This is a Washington State University developed test to measure resistance to stud wear. There is no established benchmark for success or failure so results are evaluated relative to one another.
- IDT (indirect tension test) fatigue and thermal cracking test (Kim and Wen, 2002). An indirect tension test that produces three measures:
    - IDT strength (Figure 15). The peak stress experienced by the sample.
    - Fracture work density (Figure 16 and Figure 17). Calculated value that correlates well with bottom-up fatigue cracking and thermal cracking (Wen, 2013).
    - Horizontal failure strain (Figure 18). A calculated value that correlates well with top-down cracking (Wen and Bhusal, 2014).
- Creep compliance (Figure 19). AASHTO T 322. A diametral creep test that can be correlated to mix rutting potential.
- IDT Dynamic modulus |E*| test (Figure 20). This viscoelastic IDT test relates mixture modulus to time and rate of loading. Test results correlate reasonably well with in-service pavement rutting.
- Hamburg Wheel-Tracking Test. AASHTO T 324. This test repeatedly rolls a steel wheel across a HMA sample immersed in water. A reasonable indicator of rutting and stripping potential.
- Asphalt content. AASHTO T 164. This is a solvent-based method of separating the asphalt binder form aggregate and gives an estimate of sample binder content.
- Gradation. AASHTO T 27. Standard gradation test.

Appendix B describes test details and statistical methods to determine significance as well as full test results. Appendix C shows mix designs. Only select test results are discussed here.

Asphalt binder test results indicate the ½-inch asphalt mixture contained 4.9% binder and the 3/8-inch asphalt mixture contained 5.4% binder. This likely influenced test results as much or more than differences in NMAS.

Even given the differences in NMAS and binder content, the performances of the 3/8-inch and ½-inch mixes are similar. Relatively, compared to the 1/2-inch mix, the 3/8-inch mix has similar studded tire resistance, equivalent strength, and equivalent top-down cracking resistance. The 3/8-inch mix showed slightly better bottom-up fatigue and thermal cracking resistance, as indicated by results of the fracture work density at intermediate and low temperatures (Figure 16 and Figure 17). The two mixes have approximately the same stiffness, as indicated by results of creep compliance and dynamic modulus (Figure 19 and Figure 20).

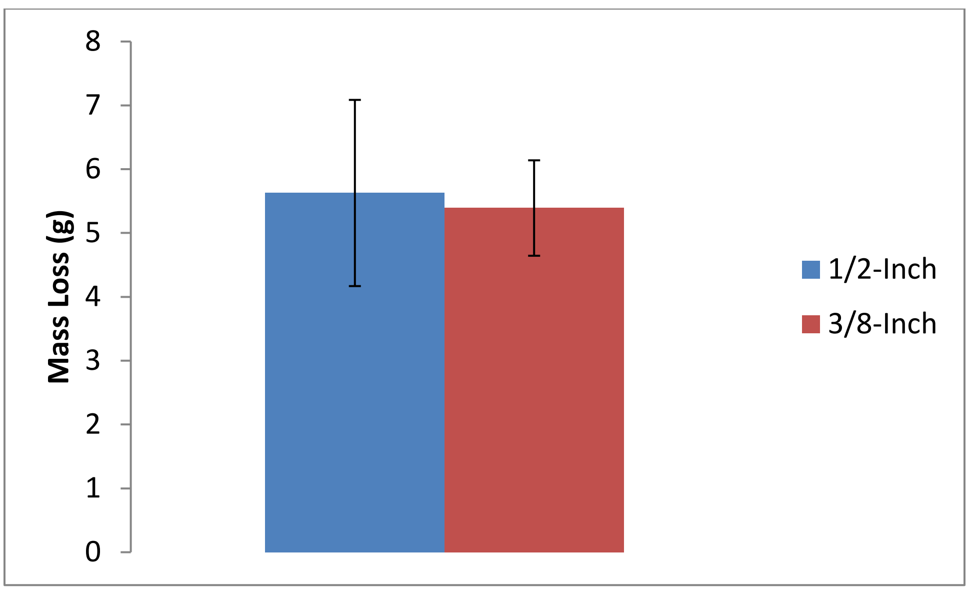

Figure 14. Contract 8611 studded tire wear test result comparison.

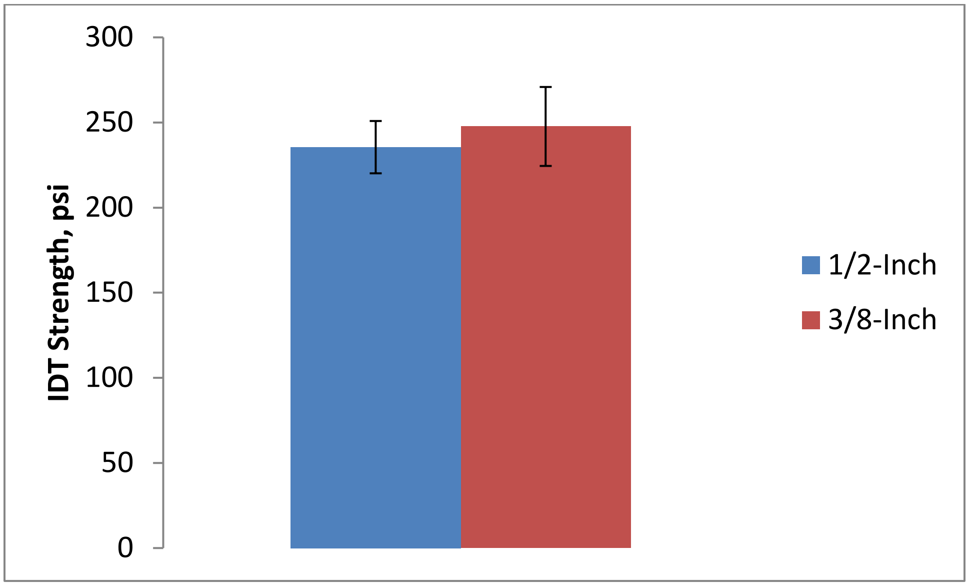

Figure 15. Contract 8611 IDT strength results comparison.

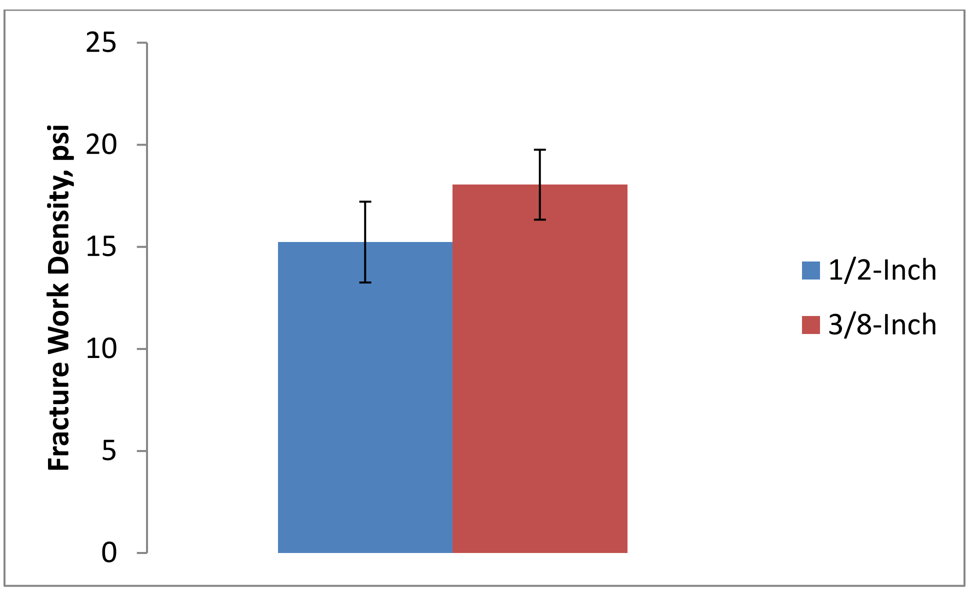

Figure 16. Contract 8611 fracture work density at intermediate temperature for fatigue results comparison.

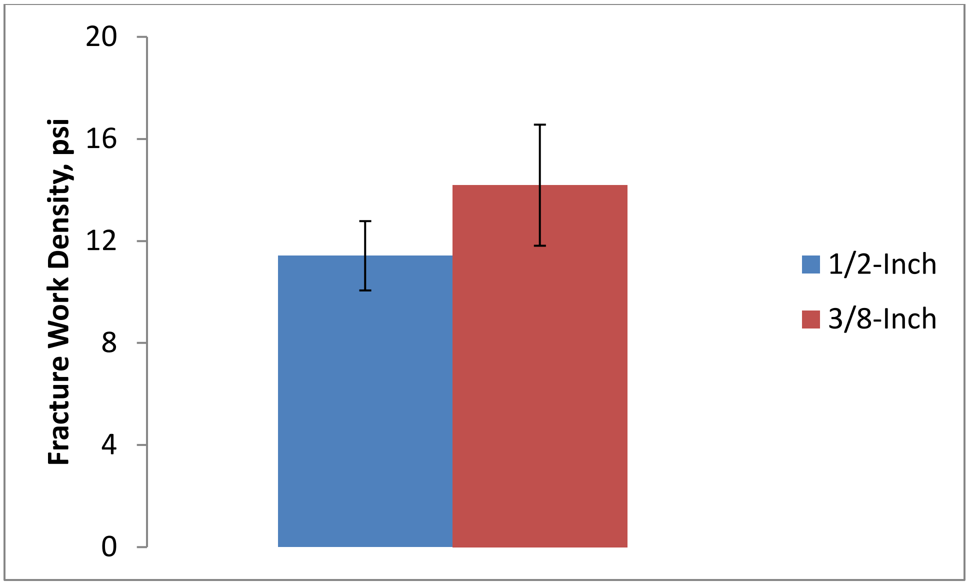

Figure 17. Contract 8611 fracture work density at low temperature for thermal cracking results comparison.

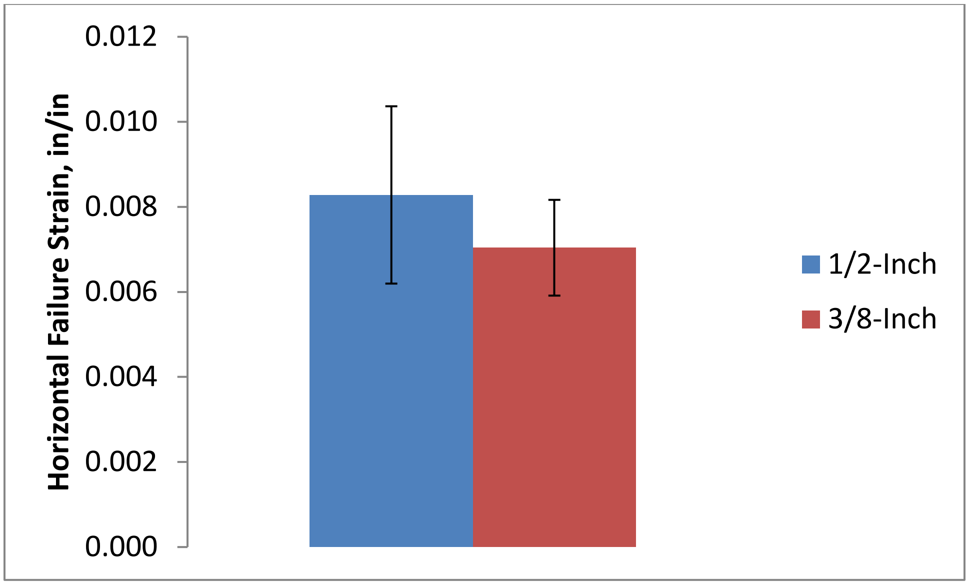

Figure 18. Contract 8611 horizontal failure strain results comparison.

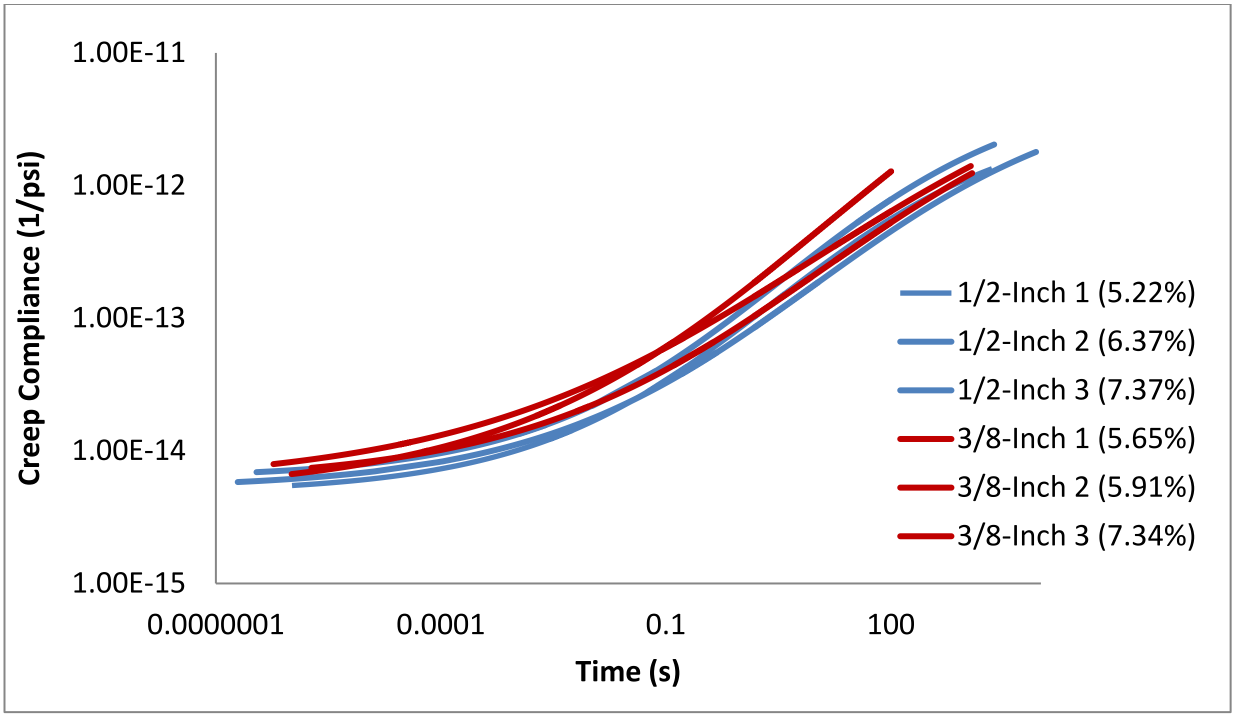

Figure 19. Contract 8611 creep compliance master curves with field core air voids (in percent of theoretical maximum density).

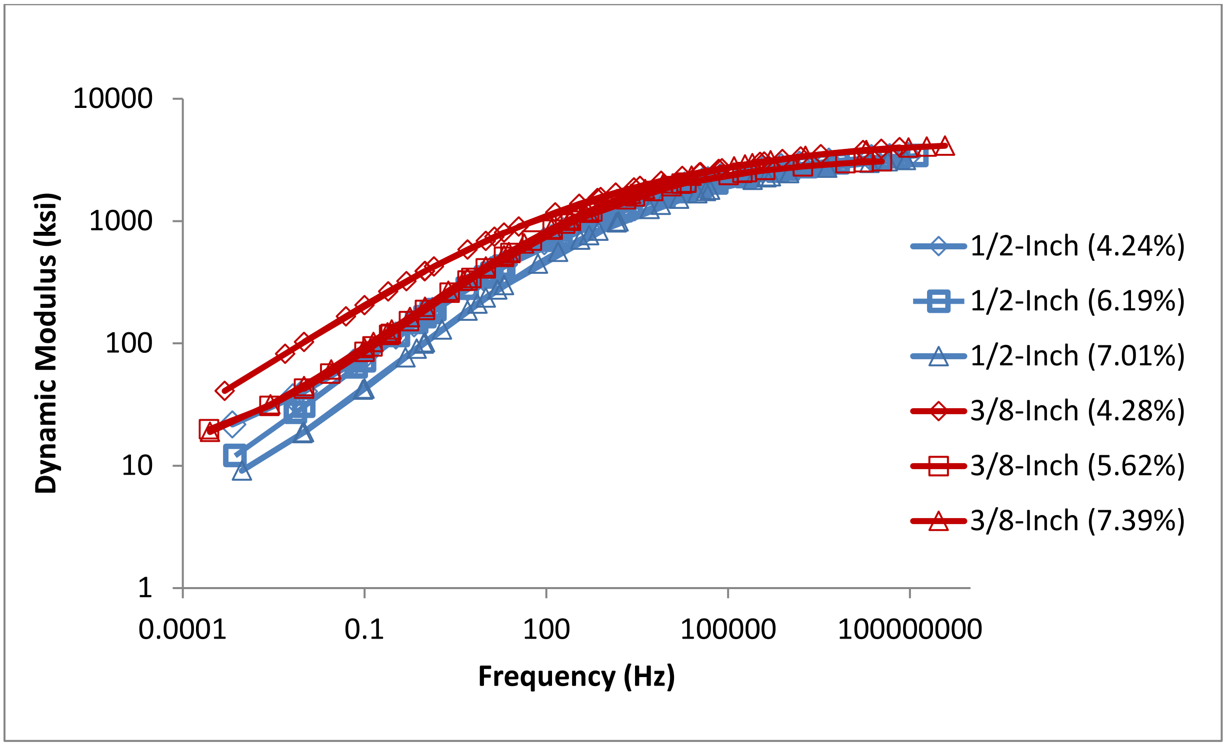

Figure 20. Contract 8611 dynamic modulus master curves with field core air voids (in percent of theoretical maximum density).

4.1.2 Contract 8447: SR 21 1.1 Miles North of Rin Con Creek Road to Canada

Contract 8447 was constructed in 2013 and is located on SR 21 in the Eastern Region, 1.1 miles north of Rin Con Creek Road to the Canadian border. Both lanes were paved from MP 183.80 to MP 191.34. This project consists of 0.15 feet of 3/8-inch PG 64-28 HMA overlay, with crack sealing over two miles of the existing roadway (MP 185.00 to MP 186.01, and MP 187.00 to 188.00). No performance data is available due to recent construction.

4.1.3 Contract 8443: I-90 MP 65.54 to Easton Hill EB & WB

Contract 8443 was constructed in 2013 and is located on I-90 in the South Central Region from MP 65.54 to MP 67.34 in the EB and WB lanes. The project was an overlay of concrete that was cracked and seated and the asphalt mixture was 3/8-inch PG 64-28 HMA. No performance data is available due to recent construction.

4.1.4 Contract 7763: US 2 JCT SR 211 to Newport

Contract 7763 is located on US 2 at approximately MP 321.77 to 333.89 in the Eastern Region. It was paved in 2009 and has an AADT of approximately 4,800. The pavement is 3/8-inch PG 64-28 HMA and seems to be performing well. Figure 21 shows WSPMS performance data for both the EB and WB lanes. This section was crack sealed in 2014, mostly due to cracks at the construction joints between lanes and at the shoulder joints.

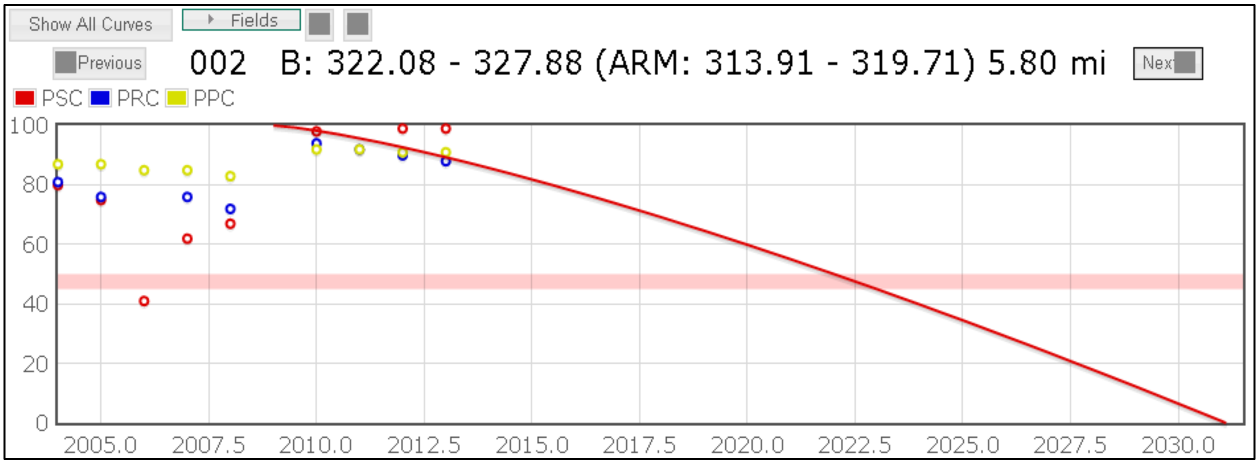

Figure 21. Contract 7763 WSPMS plotted condition ratings and performance curve.

4.2 Rubberized Asphalt Projects

4.2.1 Contract 4250: I-5 Nisqually River to Gravelly Lake I/C

Contract 4250 on I-5 from Nisqually River to Gravelly Lake I/C is a dense-graded HMA with rubberized asphalt built in 1994. The southbound Class A PBA-6GR rubberized section from MP 118.77 to 120.77 was reconstructed in 2015 after being in place for nearly 20 years. Although the mix is not a Superpave project, it gives evidence that rubberized pavements can perform well in Washington State if constructed properly. The WSPMS performance curves of Contract 4250 are shown in Figure 24.

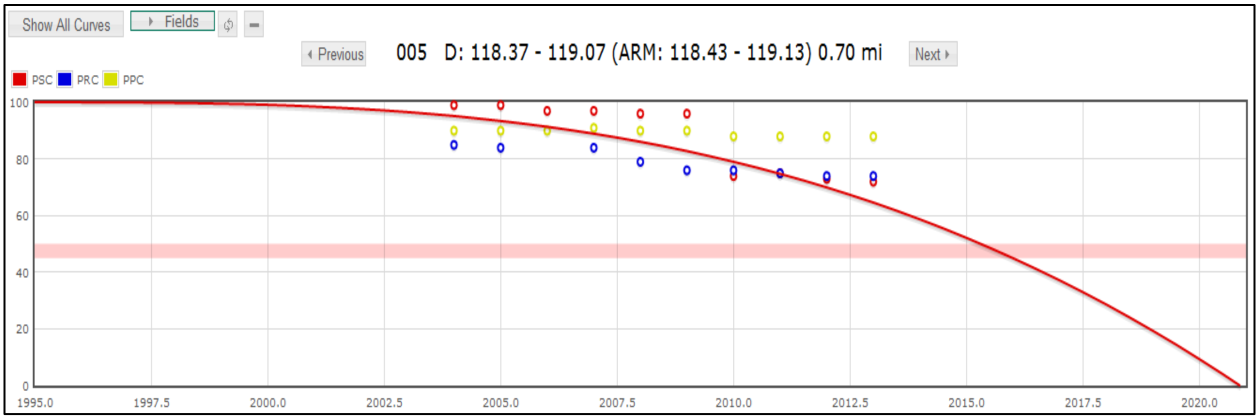

Figure 22. Contract C4250 WSPMS plotted condition ratings and performance curve.

4.3 Stone Matrix Asphalt Projects

The two remaining WSDOT SMA projects that are performing well are evaluated more closely with WSPMS data. Although both SMA contracts have associated HMA sections with which to directly compare, only one contract, 6151, was cored and subjected to laboratory testing.

4.3.1 Contract 6151: I-90 SR 21 Vic. to Ritzville

Contract 6151 on I-90 from SR 21 to Ritzville is located between MP 208.16 to 218.6 in the Eastern Region. It was paved in 2001 and has an AADT of approximately 38,300. The project was constructed with a section of ½-inch PG 76-28 SMA in the right westbound (WB) lane from MP 211.541 to 214.225. The left WB lane and the rest of Contract 6151 consists of ½-inch PG 64-28 HMA. Rutting and cracking performance from WSPMS of these pavement sections are detailed in Table 12. Note that MP locations listed in WSPMS do not exactly match with the MP locations in the field, possibly due to changes made during construction from the original project plans. The MP locations listed in this study for Contract 6151 are from locations recorded by field inspection. From WSPMS, the performance of the SMA in the WB lane from MP 212.93 to

213.43 is shown in Figure 23, and the performance of the HMA in the WB lane from MP 214.05 to 215.23 is shown in Figure 24. Discussions with WSDOT noted that the WSPMS performance curve for the SMA section fits the data poorly for unknown reasons. For this study, the performance evaluation of the SMA section is based on individual data points, instead of the performance curve. Based on a rough estimate, the SMA section may last upwards of 20 years compared to a 14-year WSPMS predicted life for the HMA section (both well above the WSDOT reported 11-year average surface life for Eastern Washington).

Table 12. Contract 6151 Comparison of WSPMS Condition Data for SMA and HMA Sections

Section	Cracking (PSC)	Rutting (PRC)	Rut Depth, in.
HMA	74	85	0.28
SMA	80	88	0.23

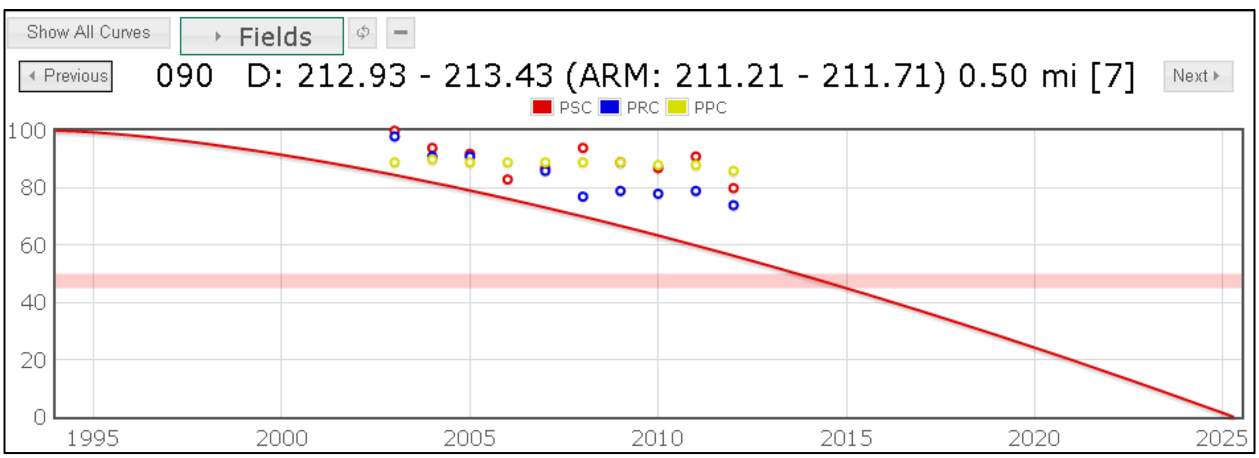

Figure 23. Contract 6151 SMA WSPMS plotted condition ratings and performance curve.

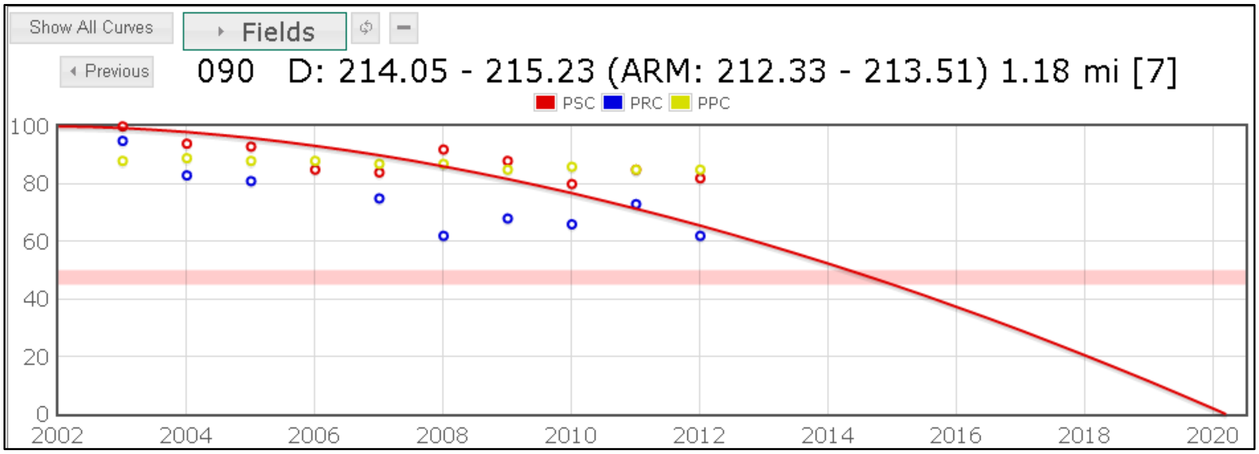

Figure 24. Contract 6151 HMA WSPMS plotted condition ratings and performance curve.

Cores were taken from Contract 6151 to compare the laboratory performance of the ½-inch PG 76-28 SMA and ½-inch PG 64-28 HMA. Parameters evaluated for this project are:

- Studded tire wear resistance (Figure 25). This is a Washington State University developed test to measure resistance to stud wear. There is no established benchmark for success or failure so results are evaluated relative to one another.
- IDT (indirect tension test) fatigue and thermal cracking test (Kim and Wen, 2002). An indirect tension test that produces three measures:
  - IDT strength (Figure 26). The peak stress experienced by the sample.
  - Fracture work density (Figure 27 and Figure 28). Calculated values that correlate well with bottom-up fatigue cracking and thermal cracking (Wen, 2013).
  - Horizontal failure strain (Figure 29). A calculated value that correlates well with top-down cracking (Wen and Bhusal, 2014).
- Asphalt content. AASHTO T 164. This is a solvent-based method of separating the asphalt binder form aggregate and gives an estimate of sample binder content.

- PG binder grading. AASHTO PP 6. Grades the PG binder as if it were short-term aged (i.e., not aged in the RTFO).
- Gradation. AASHTO T 27. Standard gradation test.

Appendix B describes test details and statistical methods to determine significance as well as full test results. Appendix C shows mix designs. Only select test results are discussed here.

The laboratory performance of the SMA versus HMA samples appear consistent with field performance as measured by WSPMS data. Results indicate the SMA and HMA samples are no different in studded tire wear resistance (Figure 25). IDT strength (Figure 26) shows HMA greater than SMA although this measurement does not readily correlate to any field performance. The SMA samples show significantly superior performance over the comparable HMA section for top-down, bottom-up, and thermal cracking resistance, as indicated by results of horizontal failure strain and fracture work density at intermediate and low temperatures shown in Figure 27 through Figure 29.

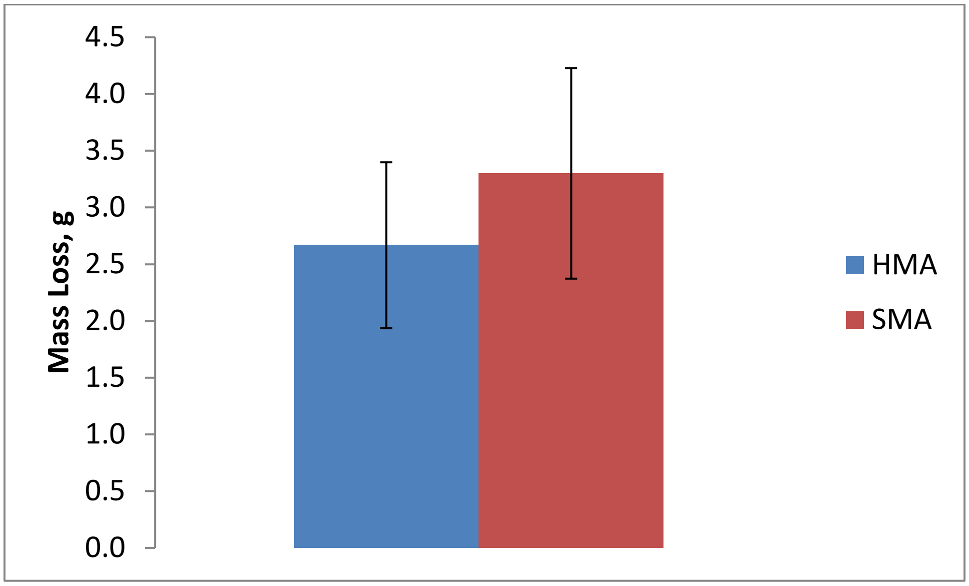

Figure 25. Contract 6151 studded tire wear test result comparison.

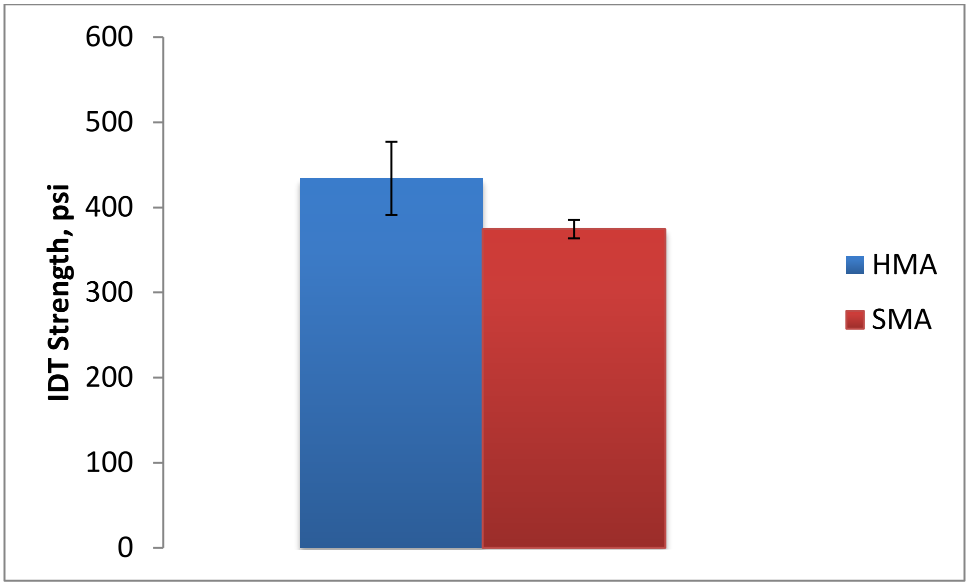

Figure 26. Contract 6151 IDT strength results comparison.

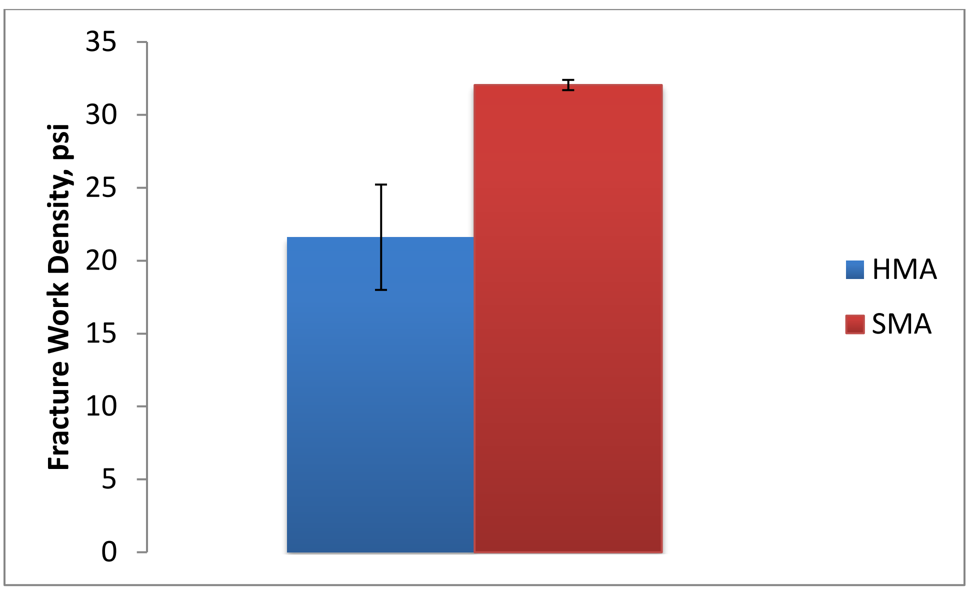

Figure 27. Contract 6151 fracture work density at intermediate temperature for fatigue results comparison.

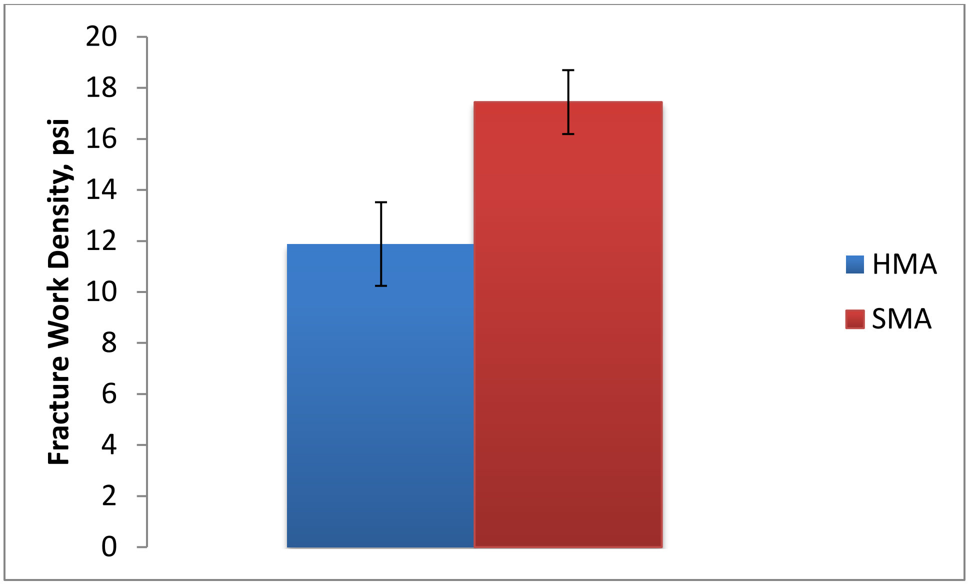

Figure 28. Contract 6151 fracture work density for thermal cracking results comparison.

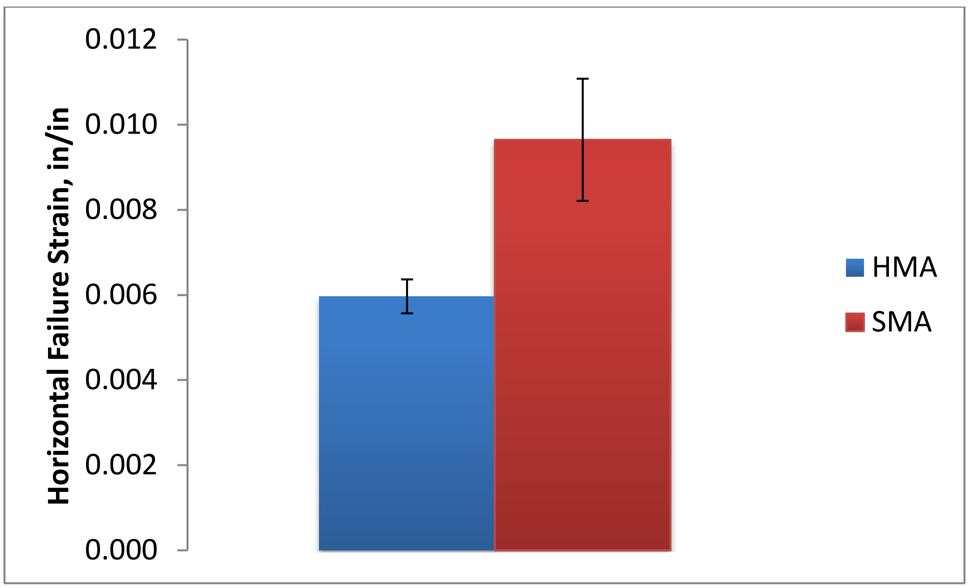

Figure 29. Contract 6151 horizontal failure strain results comparison.

4.3.2 Contract 6687: I-90 Dodson Road to Moses Lake (West of Moses Lake)

In the North Central Region, Contract 6687 was paved in 2004 from Dodson Road to Moses Lake, MP 164.15 to MP 181.77, and has an AADT of approximately 9,700. The EB lane is ½-

inch PG 76-28 SMA and the WB lane is ½-inch PG 64-28 HMA. According to the WSDOT North Central Region Materials Engineer, the SMA in the eastbound lane is outperforming the standard PG 64-28 HMA mix in the westbound lane, although the cost per ton of SMA was 57% more than the HMA (Bob Romine, personal communication, April 15, 2014). This high cost may be attributed to the fact that this was a small experimental section and was only the third SMA pavement to be constructed in Washington State. The WSPMS performance curves show visibly superior rutting performance for the SMA (Figure 30 and Figure 31).

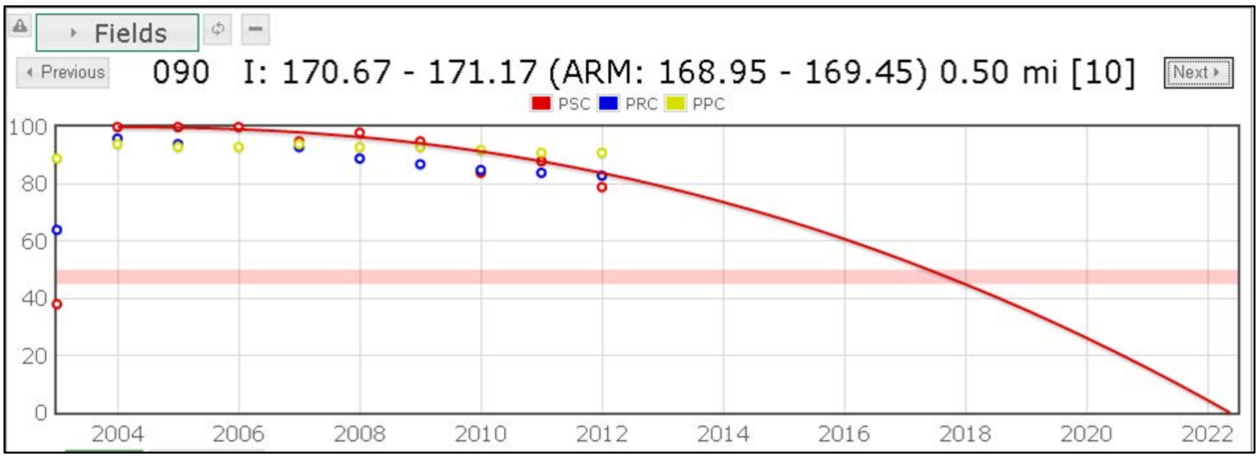

Figure 30. Contract 6687 SMA WSPMS plotted condition ratings and performance curve.

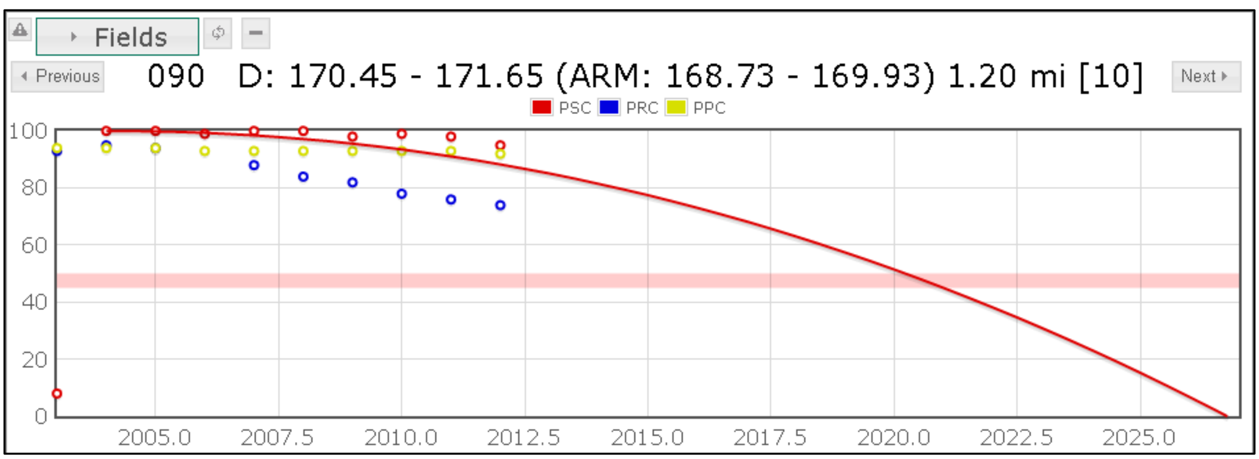

Figure 31. Contract 6687 HMA WSPMS plotted condition ratings and performance curve.

4.4 BST Applied within One Year of Paving Projects

The following sections describe two WSDOT HMA pavements that were overlaid with a BST within one year of construction. Both contracts contained HMA sections with BST overlays and HMA sections without BST overlays that were paved at the same time. Results indicate that applying a BST overlay seems to protect the underlying HMA from oxidation, which reduces binder aging in the underlying HMA. After the experiments were conducted, it was also found that in almost all cases, fractured aggregates were present indicating that the mixes may have been over-compacted.

Sample preparation. Since these experiments must identify binder aging at the surface of the HMA, core samples were cut into 1-inch thick layers and tests were conducted on each layer. For Contract 7109 the core was thick enough to cut into two 1-inch layers, and for Contract 8262 the core was thick enough to cut into three 1-inch layers.

For each of the two contracts discussed in this section, the following tests were done:

- IDT (indirect tension test) fatigue and thermal cracking test (Kim and Wen, 2002). This was performed on the top layer of each core. An indirect tension test that produces three measures:
    - IDT strength. The peak stress experienced by the sample.
    - Fracture work density. Calculated values that correlate well with bottom-up fatigue cracking and thermal cracking (Wen, 2013).
    - Horizontal failure strain. A calculated value that correlates well with top-down cracking (Wen and Bhusal, 2014).

- Creep compliance. AASHTO T 322. A diametral creep test that can be correlated to mix rutting potential. This was performed on the top layer of each core.
- IDT Dynamic modulus |E*| test. This viscoelastic IDT test relates mixture modulus to time and rate of loading. Test results correlate reasonably well with in-service pavement rutting. This was performed on the top layer of each core.
- Bulk specific gravity. AASHTO T 166. This provides density and air voids of field compacted samples.
- Asphalt binder PG grade. AASHTO M 320. When compared to the mix design PG binder grade, this provides an indication of binder oxidation (aging) over time. A binder sample was first recovered from the core sample using the AASHTO T 164 solvent-based method.

Appendix B describes test details and statistical methods to determine significance as well as full test results. Appendix C shows mix designs. Only select test results are discussed here.

4.4.1 Contract 7109: SR 20 et al 2006 Eastern Region Chip Seal

Contract 7109, performed in 2006, is located on SR 20 from MP 404.41 to 422.92 and is essentially a BST application that includes substantial HMA pre-level. The HMA for pre-level was 3/8-inch PG 64-28 and the BST layer was Class D CRS-2P that was fog sealed on completion. This contract was chosen for laboratory analysis because there is an adjacent SR 20 section that was paved with no BST overlay at roughly the same time that can be used for comparison.

The performance curve from WSPMS for a section of Contract 7109 is shown in Figure 32. It can be seen that this section of pavement performed well after seven years in service.

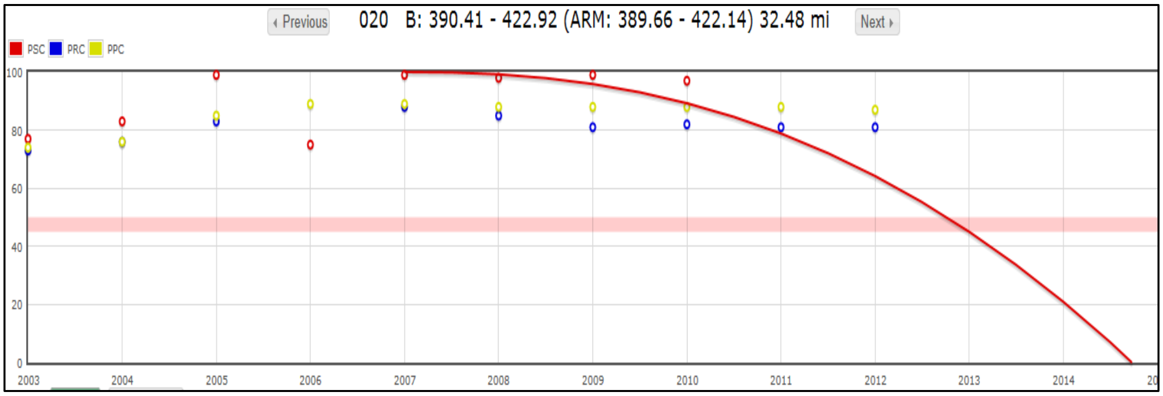

Figure 32. Contract 7109 WSPMS plotted condition ratings and performance curve.

The section of HMA without a BST application used for comparison is located on an approach that may carry different traffic, however the assumption of similar traffic is likely reasonable.

In general, test results indicate that the BST effectively protected the underlying HMA layer from oxidation and reduced the binder aging at the top of the HMA layer. Specifically, the IDT strength (Figure 33), dynamic modulus (Figure 36), and creep compliance (Figure 37) show the BST-protected HMA sample to be softer, and undergo more strain and creep as a result. The horizontal failure strain (Figure 35) indicates that the HMA sample protected by the BST has greater resistance to top-down fatigue cracking than the HMA sample without the BST. As might be expected, the fracture work density calculations (Figure 34) did not show much of a difference since bottom-up cracking would not be expected to be impacted by less HMA oxidation in the top layer. Based on the PG grading (Figure 38), the HMA without BST has significantly aged, when compared to HMA with the BST.

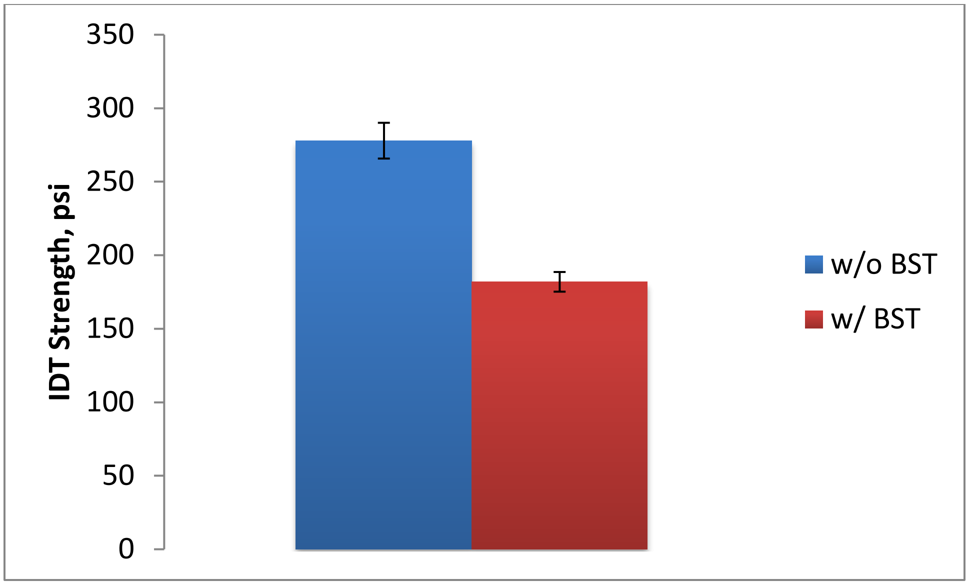

Figure 33. Contract 7109 IDT strength results comparison.

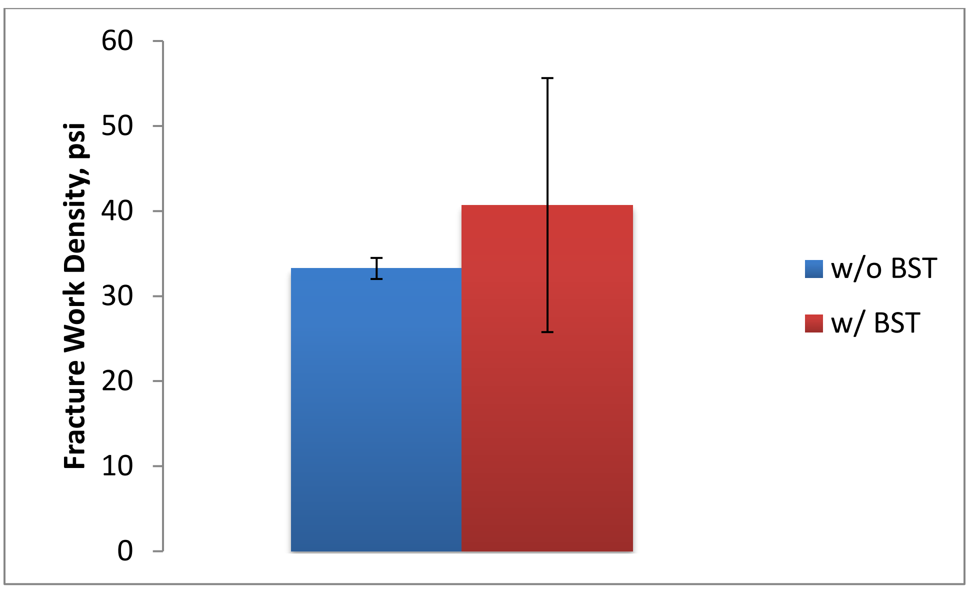

Figure 34. Contract 7109 fracture work density at intermediate temperature for fatigue results comparison.

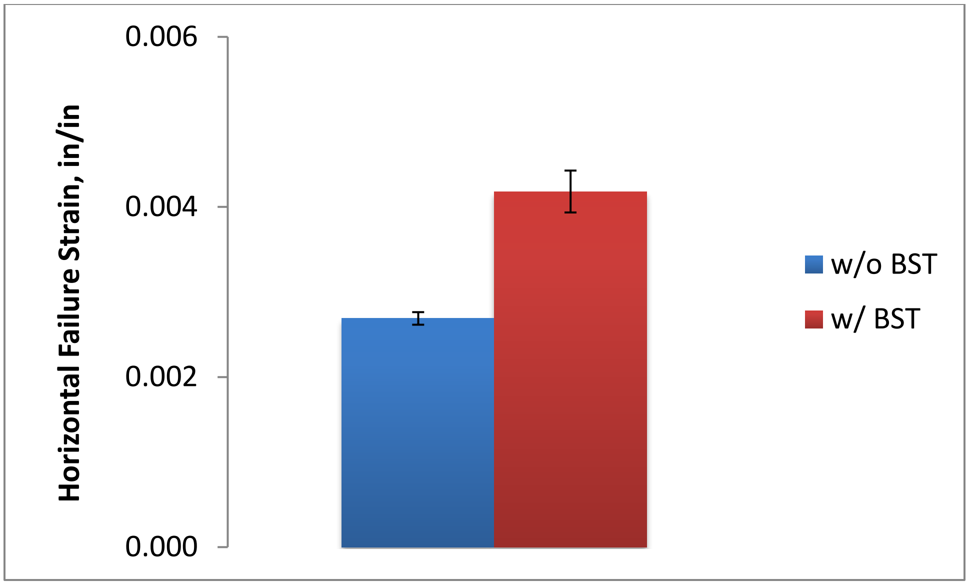

Figure 35. Contract 7109 horizontal failure strain results comparison.

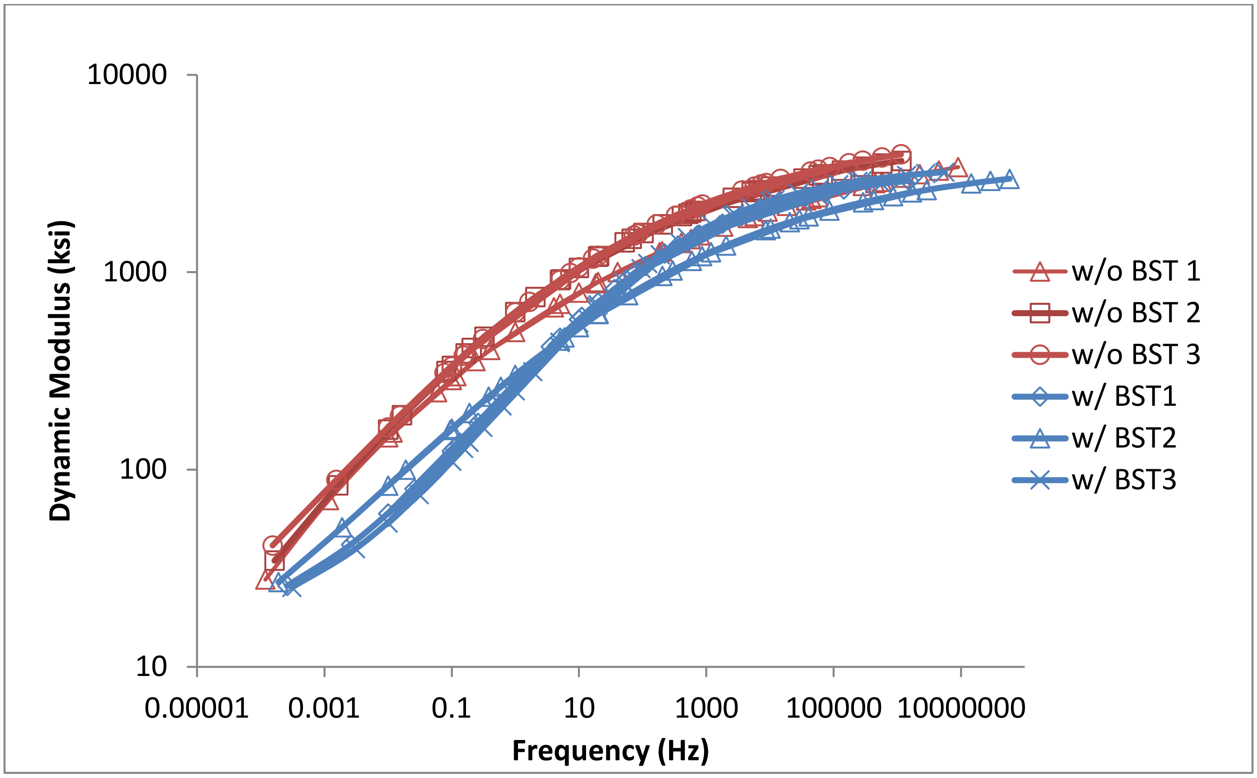

Figure 36. Contract 7109 dynamic modulus master curves.

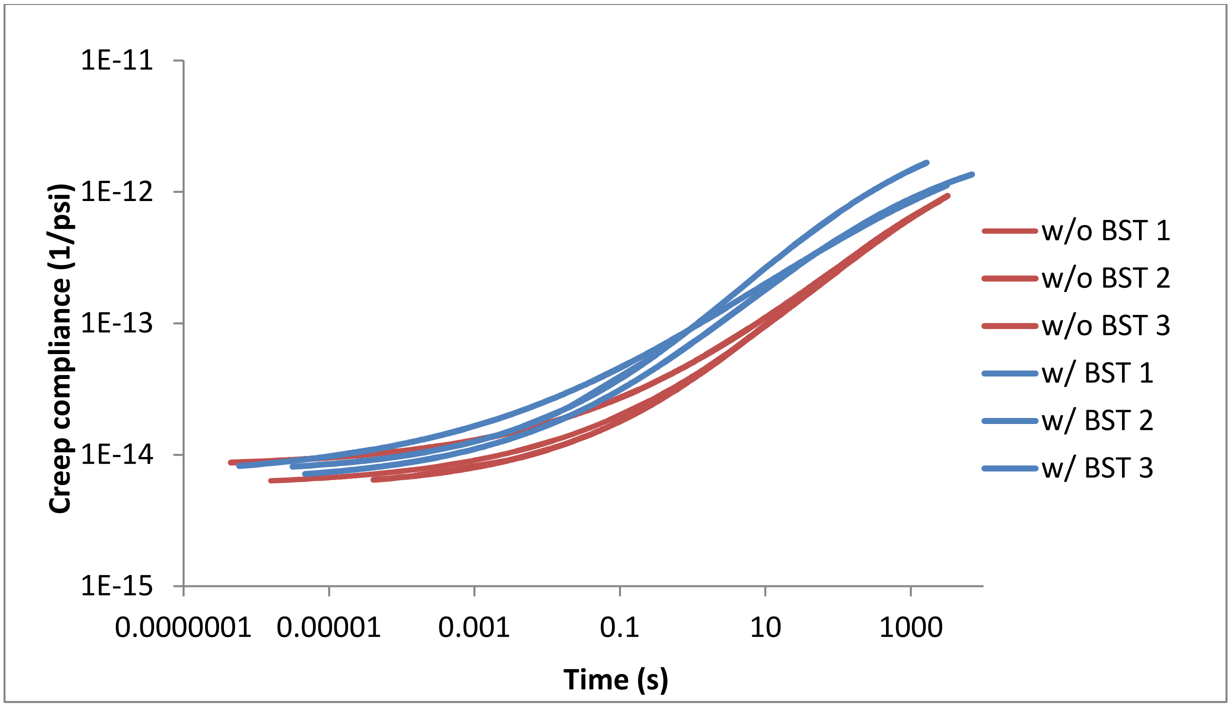

Figure 37. Contract 7109 creep compliance master curves.

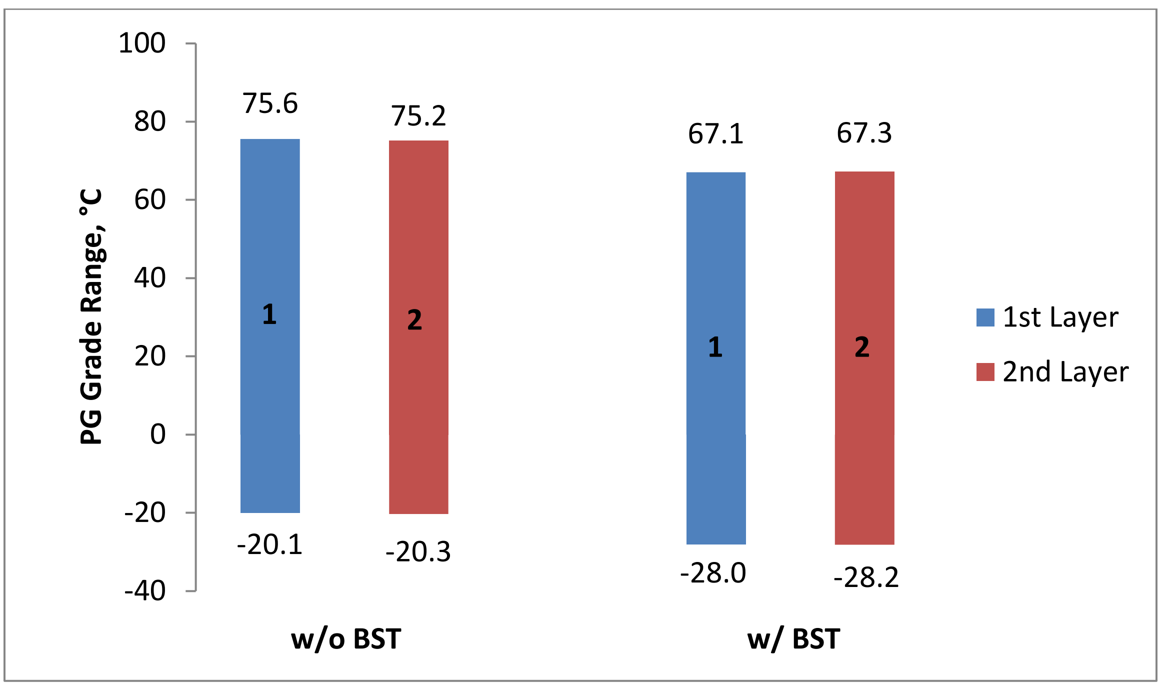

Figure 38. Contract 7109 range of PG grades for each 1-inch layer of the sample (layer 1 is on top, layer 2is on the bottom).

4.4.2 Contract 8262: SR 278 Eastern Region Chip Seal 2012

The portion of Contract 8262 located on SR 278 from MP 0.00 to MP 5.50 included HMA paving with a BST overlay in 2012. The project included 0.15 ft. depth grind and inlay of 3/8-inch PG 64-28 HMA and the BST was CRS-2P. No performance data exists due to the recent construction of this project.

For Contract 8262, the effects of the BST overlay are not as pronounced as Contract 7109 because it had less time to age since construction (Figure 39 through Figure 43). The PG grades of the different layers (Figure 44) show a slightly higher grade for the HMA without BST protection, perhaps indicating some aging.

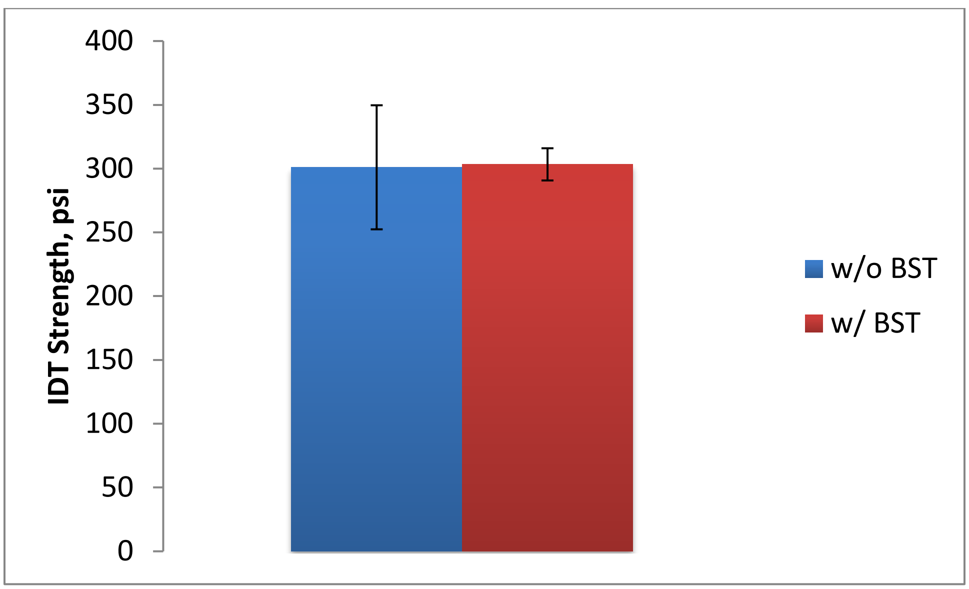

Figure 39. Contract 8262 IDT strength results comparison.

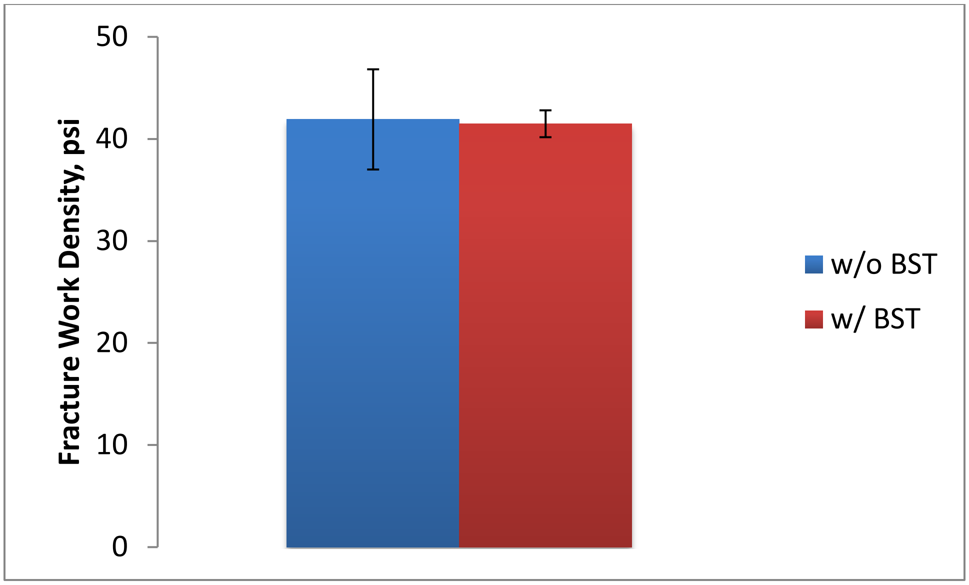

Figure 40. Contract 8262fracture work density at intermediate temperature for fatigue results comparison.

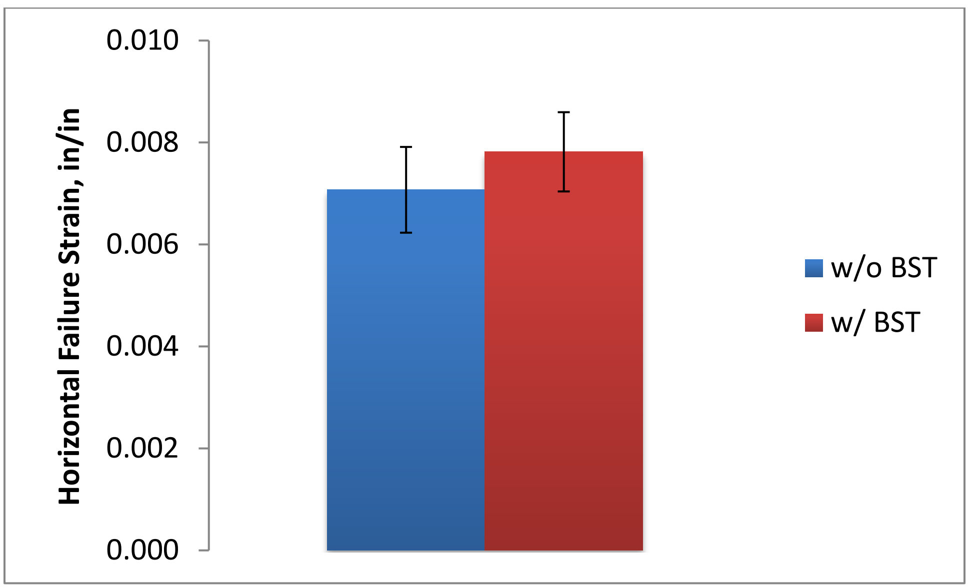

Figure 41. Contract 8262 horizontal failure strain results comparison.

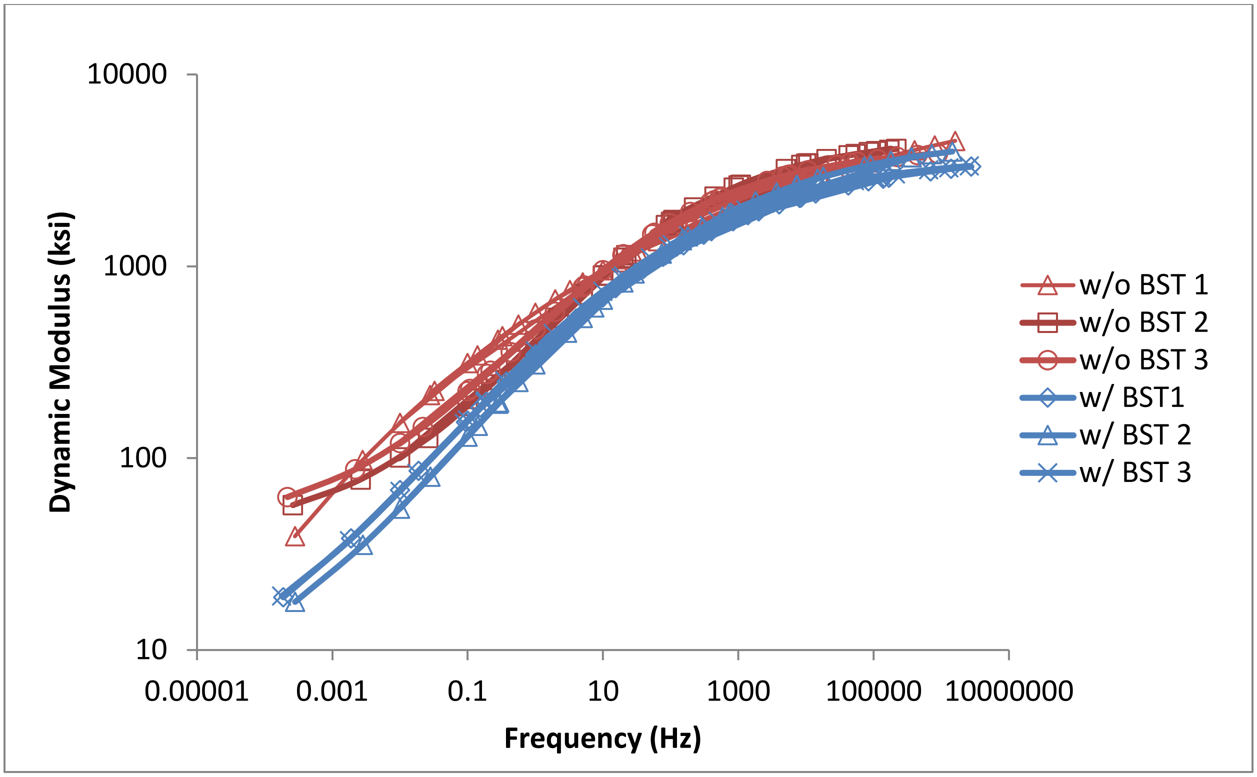

Figure 42. Contract 8262 dynamic modulus master curves.

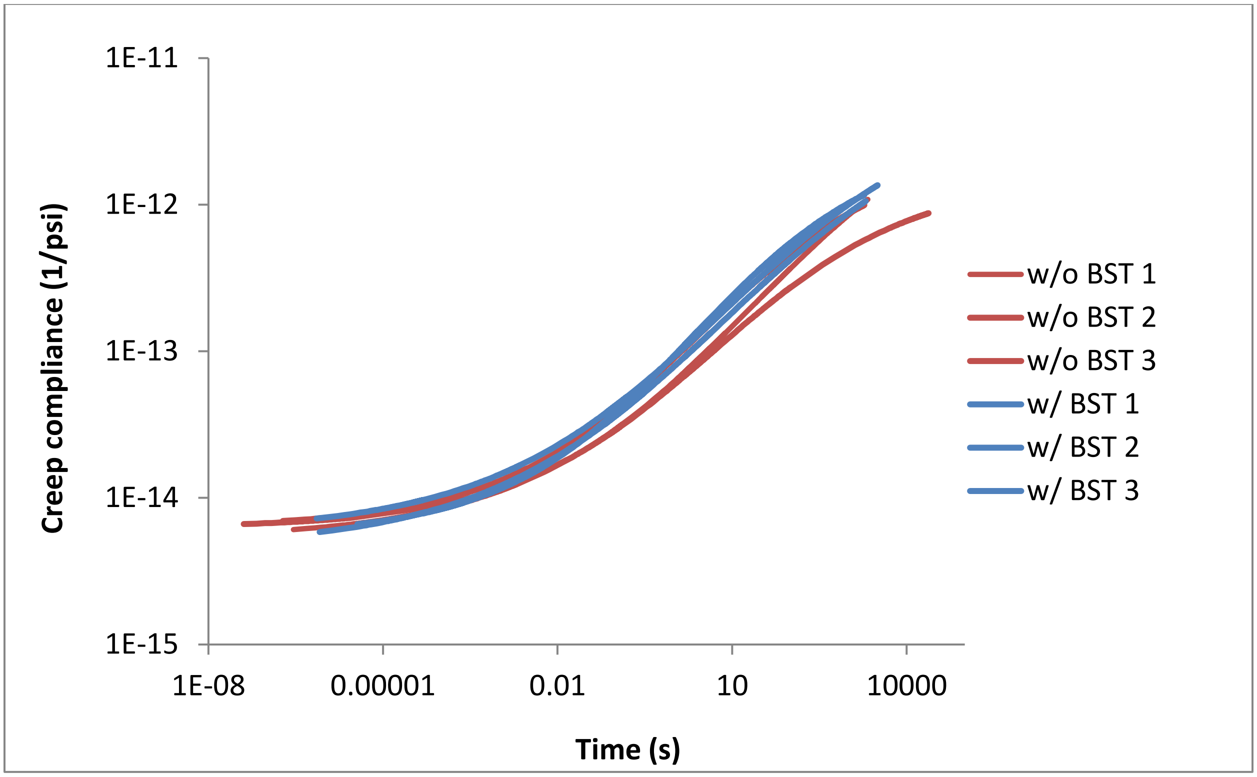

Figure 43. Contract 8262 creep compliance master curves.

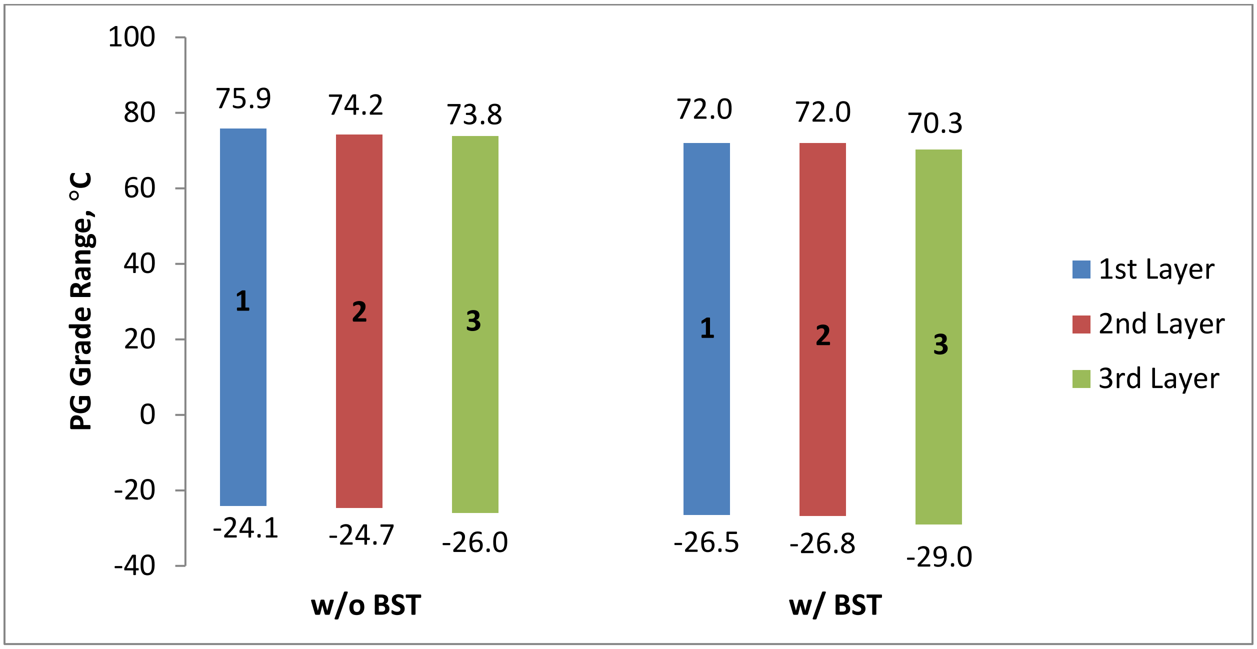

Figure 44. Contract 8262 range of PG grades for each 1-inch layer of the sample (layer 1 is on top, layer 3 is on the bottom).

4.5 WSPMS and Laboratory Test Results Summary

WSPMS data review and limited laboratory testing provides the following broad conclusions:

- While initial reports of 3/8-inch NMAS mixes have been generally positive and the potential benefits from the literature are compelling, their performance history is short so no conclusions about long-term performance can be made. While laboratory testing indicated better performance in cracking resistance, the significantly higher asphalt content in the 3/8-inch mixture, which may or may not be due solely to mix design, likely impacted those results.
- WSDOT Rubberized asphalt projects have generally had limited success, but one section has stood out as an excellent performer. The 20-year life of Contract 4250 may be an anomaly, but it does show that it is possible for asphalt rubber sections to perform well.

Given the much larger history of asphalt rubber failure in Washington, further investigation of asphalt rubber should be given a low priority.

- Both SMA projects are outperforming the average surface life in Eastern Washington (11 years) and may very well reach 20 years. Laboratory testing is consistent with field performance showing significantly superior performance for crack resistance, studded tire wear resistance, and mix deformation resistance.
- BSTs applied within one year of paving show promise but do not have enough performance history to draw strong conclusions. Laboratory tests confirm that the intended function of the BST, as a layer that protects the underlying HMA from oxidation, is happening in the field.

5 COST

This chapter summarizes information regarding the cost of polymer modified asphalt, rubberized asphalt, and SMA. Results are presented as a required increase in life for an overlay that uses a certain material as compared to a traditional HMA pavement overlay that does not. In this way the reader can visualize what increase in performance the material must provide in order for its increased initial cost to be made up for in extended life.

5.1 Method

- Determine the cost of asphalt mixtures that use polymer modification, rubberized asphalt, and SMA as well as the per-ton price of HMA. This cost information comes from the literature, the study survey, and interviews with industry professionals.
- Determine the fraction of a typical overlay project cost in Eastern Washington that is attributed to HMA material alone. This was done using a brief review of five projects since in 2013 and 2014 (Table 13). This was found to be approximately 55%.
- Determine a "mixture cost ratio".

$$Mixture\ cost\ ratio = \frac{New\ material\ cost\ per\ ton}{HMA\ cost\ per\ ton}$$

- Determine a "project cost ratio".

$$Project\ cost\ ratio = 0.45 + 0.55(mixture\ cost\ ratio)$$

- Determine the project life necessary to offset the higher cost of the project using the new material.

$$Life\ to\ break\ even = Project\ cost\ ratio(HMA\ project\ life)$$

- Determine the required increase in pavement life

$$Required\ increase\ in\ pavement\ life = Life\ to\ break\ even - HMA\ project\ life$$

Table 13. Historical Eastern Region HMA Pavement Project Cost

Year	Project	PG	Asphalt Cost/ton	Tons HMA	HMA Cost	Total Project Cost	HMA % Cost of Project
2014	8611	70-28	$57	22,950	$1,308,150	$2,450,965	54%
		70-22	$70	350	$24,500		
2014	8557	64-28	$63	16,600	$1,045,800	$1,959,214	53%
2013	8540	70-28	$66.50	49,400	$3,285,100	$5,510,044	60%
2013	8539	70-28	$69	8,102	$559,038	$1,098,212	51%
2013	8538	70-28	$63	57,200	$3,603,600	$6,401,072	56%
Average							**55%**

Table 14 shows the results of this analysis.

Table 14. Cost Analysis of Different Asphalt Mix Types based on 55% of Total Project Cost

	HMA	Polymer modified binder	Rubberized asphalt	SMA
Mix cost/ton	$63.90	$68.90	$73.80	$90
Mixture Cost Ratio	1.00	1.1	1.2	1.4
Project Cost Ratio	1.00	1.0	1.1	1.2
Life to break even (years)	11.0	11.5	11.9	13.4
Added life needed to break even (years)	-	0.5	0.9	2.4

The following sections are brief discussions of data used and results for each analyzed material.

5.2 Polymer Modified Asphalt

Polymer modifiers generally add to the cost of asphalt. At the current price of oil (2015), a PG 76-XX would cost approximately $100 per ton more than a PG 70-XX (personal communication with asphalt supplier, 2014). This translates to an increase of approximately $5 per ton of asphalt mix. Given that the average asphalt pavement life in Eastern Washington is approximately 11 years, a polymer modified asphalt pavement would need to last approximately 11.5 years (in other words, an additional 6 months) in order to break even on cost in this scenario.

5.3 Rubberized Asphalt

According to Roschen (2014), rubberized asphalt pavements cost $95.40 per ton compared to $80.55 per ton for conventional HMA, a difference of 15.5%. In California in 2011, the cost was reported to be approximately 20%-25% higher than HMA (Cheng and Hicks, 2012). Given that the average asphalt pavement life in Eastern Washington is approximately 11 years, a rubberized asphalt pavement would need to last approximately 11.9 years (in other words, an additional 11 months) in order to break even on cost.

According to Adam Hand (personal communication, April 24th, 2014), Caltrans allows calculated HMA pavement overlay thickness to be halved if cracking is found to be the controlling distress and the overlay is to use rubberized asphalt (this is based on testing done by the University of California Pavement Research Center that shows superior crack resistance of rubberized asphalt overlays used by Caltrans). There is also a cost incentive from the California Department of Resources Recycling and Recovery (CalRecycle) for using rubberized asphalt.

The combination of half overlay thickness and cost incentives results in the total project cost being approximately equal to using standard HMA.

5.4 Stone Matrix Asphalt

The cost of SMA in Maryland and Georgia (two states that use SMA extensively and, thus, can provide a better price-point for a stabilized market) is about $90 per ton, whereas the cost of hot mix asphalt is generally between $60 and $80 per ton resulting in a SMA price premium of about 12-50%. Results from the survey indicate an average of $97 per ton for SMA, ranging from $89 to $116 per ton, and an average of $73 per ton for HMA, ranging from $60 to $86 per ton resulting in an average price premium of about 30%.

According to the Georgia Department of Transportation (Georgene Geary, personal communication, June 23, 2014), SMA is used on high volume roads above 25,000 ADT, but is overlain with open graded friction course HMA for drainage and safety concerns. A brief review of recent pavement project costs in Eastern Washington reveals that asphalt pavements make up approximately 55% of the total project cost of a typical WSDOT 0.15 ft. overlay (Table 13).

As the average life of HMA pavements in Eastern Washington is approximately 11 years, an SMA pavement would need to last approximately 13.4 years (in other words, 29 more months) in order to break even on cost.

5.5 Cost Analysis Summary

All three mix design techniques (polymer-modified asphalt binder, rubberized asphalt, and SMA) are initially more expensive than HMA using an unmodified PG grade asphalt binder. The added life needed to make up for this higher initial expense is, in all three cases, quite reasonable

(0.5 years for polymer-modified asphalt binder, 0.9 years for rubberized asphalt, and 2.4 years for SMA). Based on the literature review and state DOT survey, these added life values should be expected for properly constructed pavements.

6 CONCLUSIONS AND RECOMMENDATIONS

This study identifies and evaluates HMA design and construction techniques with potential for improving WSDOT pavement surface life in Eastern Washington and mountain pass areas. Since this study was not intended to be an intensive laboratory investigation, evaluation of these techniques relies on corroborating multiple data sources rather than a compilation of statistically defensible experiments. Data sources used in this study are: literature review, DOT survey, WSPMS data search, laboratory tests, case study, and cost analysis.

6.1 Summary of Findings

6.1.1 Failure Mechanism

Cracking is the predominant failure mechanism for WSDOT Superpave pavements completed prior to and including 2007. More specifically, WSPMS indicates that cracking first reaches critical thresholds requiring resurfacing 86% of the time, compared to only 13% for rutting. Rutting is more likely to be non-zero as traffic level increases. This is especially true in Eastern Washington, where studded tire use can more than double the rate of rutting. While not specifically analyzed, this observation is likely generalizable to all WSDOT HMA pavements.

Construction Techniques

Overall, six construction techniques were identified in the literature (Table 15). These construction practices have general applicability, but are most applicable to mountain pass paving.

Table 15. WSDOT Experience with Construction Techniques to Increase Pavement Surface Life Identified in the Literature Review

Technique	Specified	Standard Practice	Allowed	Experimental	Not Done
Avoid late season paving	x				
Increase longitudinal joint density	x	x			
Mitigate temperature differentials: density profile	x				
Mitigate temperature differentials: Pave-IR				x	
Use intelligent compaction				x	
WMA as a compaction aid			x		

Of those construction practices not already specified, WSDOT should consider (1) a Pave IR specification, (2) how it wishes to incorporate intelligent compaction into specifications, and (3) specifying WMA for mountain pass projects (as a compaction aid). The case study identified several WSDOT-required mountain pass paving elements (WSDOT, 2015) that were implemented with varying impacts. For the described case study, the 3/8-inch NMAS HMA had production and laydown issues, the HWTD test caused the contractor to adjust the JMF, the straight PG tack coat suffered from excessive wheelpath pickup when placed on existing concrete, both longitudinal joint techniques were successfully built (notched wedge, cut back), and Pave IR was ineffective because there were no specific guidance or requirements for its use. Ultimately, while additional specification requirements are intended to induce project actions that can contribute to pavement life, construction issues still need to be worked out so that they can be consistently implemented as intended.

Mix Design

Overall, 11 mix design techniques were identified in the literature (Table 16), of which four (3/8-inch NMAS, rubberized asphalt, SMA, and BST applied within one year of paving) were investigated further with WSPMS data and limited laboratory testing.

Table 16. WSDOT Experience with Mix Design Techniques to Increase Pavement Surface Life Identified in the Literature Review

Technique	Specified	Standard Practice	Allowed	Experimental	Not Done
Non-Superpave aggregate gradation					x
Reduce N_{design}					x
3/8-inch NMAS			x		
Polymer modified asphalt	x				
Rubberized asphalt				x	
Lime addition				x	
SMA				x	
Steel slag aggregate			x		
Performance tests (rutting, stripping)	x				
Performance tests (cracking)					x
BST applied within one year of paving				x	

Four techniques that WSDOT does not do are identified. Reducing N_{design} has the strongest evidence in favor with several national studies recommending such actions, and several more state DOTs indicating that they have done so. Furthermore, since WDOT pavements fail predominantly by cracking, the potential for higher asphalt content with a reduced N_{design} (along with a corresponding slight increase in VMA as recommended by Prowell and Brown (2007)) may increase surface life by delaying cracking failure. Another promising technique is a reversion to more tightly controlled aggregate gradation bands. While national literature does not say much about this, and the wholesale adoption of Superpave has turned many agencies towards broad aggregate gradation specification bands, WSDOT has substantial experience with its legacy Class A, B, D, E, F, and G mixtures to determine if such a reversion is worth considering. Steel slag aggregate shows promise in some specialty applications, and those applications (high friction, reduced studded tire wear) should be considered by WSDOT. However, steel slag aggregate will not likely be in abundance and transport of it to appropriate projects for its use may be expensive so wholesale adoption is not warranted. Finally, since a majority of WSDOT pavements fail by cracking, a cracking performance test should be considered.

6.1.2 WSPMS Analysis and Laboratory Test Results

Five mix design techniques are identified that WSDOT has done experimentally. WSPMS data analysis and limited laboratory testing on 3/8-inch NMAS, rubberized asphalt, SMA, and BST applied within one year of paving reveal the following about these WSDOT experimental sections:

- **3/8-inch NMAS**. While initial performance is anecdotally positive, performance history is too short to draw significant conclusions. It may be that the higher asphalt content associated with 3/8-inch NMAS mixes can improve resistance to cracking.
- **Rubberized asphalt**. While most WSDOT rubberized asphalt projects have not performed well, one has. The preponderance of evidence suggests WSDOT should not pursue rubberized asphalt as a promising technique.
- **SMA**. Both analyzed projects are performing well after 12 and 15 years in-place. Laboratory testing is consistent with observed superior field performance.
- **BSTs applied within one year of paving**. Observed sections show early promise but lack enough performance history to draw significant conclusions. Laboratory tests confirm that the intended function of the BST (to protect the HMA from oxidation) is working.

While lime has been used as an anti-stripping agent on at least one WSDOT project (see Section 3.3), it was not investigated in WSPMS or the laboratory.

6.2 Recommended Overall WSDOT Strategy

In general, WSDOT should focus on techniques to improve pavement cracking resistance. For all but high-volume pavements, this generally involves techniques to increase asphalt binder content, reduce surface aging, and additive use. For high-traffic and mountain pass pavements, improved rutting and studded tire wear resistance should also be priorities, which generally involve specialty mix designs and additive use. Table 17 shows specific recommendations for the 17 techniques investigated. Importantly, it may not be necessary to implement all these techniques. The recommended approach is to begin implementing them with the most impactful first ("high" priority in Table 17), and only implement others as needed. For instance, it may be that using 3/8-inch NMAS as a standard mix will address compaction issues and raise typical asphalt binder, which would make a reduction in N_{design} redundant.

Table 17. Recommendations for Techniques to Increase Pavement Surface Life

Technique	Recommendation	Priority
Construction		
Avoid late season paving	Continue current spec.	-
Increase longitudinal joint density	Continue current spec.	-
Mitigate temperature differentials: density profile	Continue current spec.	-
Mitigate temperature differentials: Pave-IR	Special provision	Low
Use intelligent compaction	No further investigation	-
WMA as a compaction aid	Special provision	High
Mix Design		
Non-Superpave aggregate gradation	Research project	High
Reduce N_{design}	Research project	High
3/8-inch NMAS	Implement policy	High
Polymer modified asphalt	Continue current spec.	-
Rubberized asphalt	No further investigation	-
Lime addition	No further investigation	-
SMA	Implement policy	High
Steel slag aggregate	Test sections	Low
Performance tests (rutting, stripping)	Continue current spec.	-
Performance tests (cracking)	Research project	Medium
BST applied within one year of paving	Test sections	Medium

Recommendation Notes:

- Implement policy: Adopt a policy that requires the technique's use under specific conditions.
- Continue current spec.: Already specified by WSDOT, recommend continuing the current specification as-is.
- Special provision: Not specified by WSDOT; write a special provision and test its use on several projects.
- Test sections: Investigate the technique by constructing test sections and monitoring performance.
- Research project: Conduct research to determine if the technique is worth adopting and, if deemed worthy, the best way to adopt the technique.
- No further investigation: The technique has only a tenuous relationship with pavement surface life, or past performance has been poor.

Priority Notes:

- High: Implement as soon as possible. The technique is well-established in the literature, other state DOTs use it with success, WSDOT has positive experience.
- Medium: Implement when ready. The technique shows promise, but others have more evidence of impact.
- Low: The technique shows some promise, but its impacts are likely to be minimal or have not been adequately documented in the literature.

6.3 Specific Implementation Recommendations

Specifics associated with Table 17 are (in order of priority):

- **High priority**. A preponderance of evidence (literature, survey, WSPMS data, laboratory experiments, WSDOT experience) suggests these techniques have a high probability of substantial impact.
    - **SMA**. Implement a paving policy that requires SMA to be the first option to consider for pavement type for Interstate routes with traffic > 20,000 AADT.
    - **3/8-inch NMAS HMA mixtures**. Implement a policy that requires 3/8-inch NMAS HMA mixtures to be considered first for low to medium traffic pavements (< 20,000 AADT), and mountain pass pavements.
    - **Non-Superpave aggregate gradation**. Research the merits of returning to legacy WSDOT aggregate gradations. It is hypothesized that these more stringent specifications (when compared to Superpave) may reduce instances of poor aggregate gradation leading to poor pavement performance.
    - **Reduce N_{design}.** Research lower N_{design} values that are appropriate for WSDOT. New values should be introduced by test section and provisionally at first.
    - **WMA as a compaction aid**. Specify the use of WMA as a compaction aid for mountain pass paving jobs and cold weather paving. This reduces the risk of poor compaction for little to no cost.
- **Medium priority**. Substantial evidence (literature, survey, WSPMS data, laboratory experiments, WSDOT experience) suggests these techniques have impact, but the nature and magnitude of that impact are less certain than “high” priority techniques.

- **Performance tests (cracking)**. Research an appropriate cracking tests for use in WSDOT mix design. Since WSDOT pavements fail predominantly by cracking, the mix design process should have a basic test for cracking.
- **BST applied within one year of paving**. Build and monitor test sections for low-traffic pavements subject to high solar radiation (i.e., Eastern Washington).

- **Low priority**. Some evidence (literature, survey, WSPMS data, laboratory experiments, WSDOT experience) suggests these techniques have impact, but their impact is thought to be minimal or confined to specialty situations.
  - **Pave IR**. Specify the use of Pave IR for mountain pass paving jobs and cold weather paving.
  - **Steel slag aggregate**. Build a test section using steel slag aggregate in an area of high studded tire wear on a mountain pass.

6.4 Recommendations for Future Research

Based on the conclusions in this section the following are recommended future studies:

- Determine appropriate N_{design} values for WSDOT.
- Quantify the advantages and disadvantages of returning to legacy aggregate gradations.
- Determine an appropriate cracking test for use with mix design.
- Document a successful mountain pass paving project using *WSDOT Pavement Policy* requirements and monitor long-term performance.
- Continue to monitor WSDOT projects that use the techniques described in this report (Table 18)

Table 18. Summary of Projects to Monitor

Project Type	Contract	Location	Material	Milepost	Year Built
PMA	7455	US 2 Creston to Rocklyn Road	1/2-inch PG 70-22 SBS	243.099 to 245.45	2008
			1/2-inch PG 64-28 HMA	230.07 to 243.099	
Rubber	4250	I-5 Nisqually River to Gravelly Lake I/C	Class A PBA-6GR	SB lane 118.77 to 120.77	1994
3/8-inch	8611	I-90 Barker Road to Idaho State Line	3/8-inch PG 70-28 HMA	297.956 to 298.335	2014
			1/2-inch PG 70-28 HMA	rest of project	
3/8-inch	8447	SR 21 1.1 Miles North of Rin Con Creek Road to Canada	3/8-inch PG 64-28 HMA	183.80 to 191.34	2013
3/8-inch	8443	MP 65.54 to Easton Hill EB & WB	3/8-inch PG 64-28	65.54 to 67.34	2013
3/8-inch	7763	US 2 JCT SR 211 to Newport	3/8-inch PG 64-28 HMA	321.77 to 333.89	2009
SMA	6151	I-90 SR 21 Vic. to Ritzville	1/2-inch PG 76-28 SMA	right WB lane 211.541 to 214.225	2001
			1/2-inch PG 64-28 HMA	left WB lane and rest of contract	
SMA	6687	I-90 Dodson Road to Moses Lake (West of Moses Lake)	1/2-inch PG 76-28 SMA	EB lane 164.15 to 181.77	2004
			1/2-inch PG 64-28 HMA	WB lane 164.15 to 181.77	
BST Overlay	7109	SR 20 et al 2006 Eastern Region Chip Seal	Class D CRS-2P BST	SR 20 404.41 to 422.92	2006
			3/8-inch PG 64-28 HMA (pre-level)		
BST Overlay	8262	US 2 et al Eastern Region Chip Seal 2012	CRS-2P BST	SR 278 0.00 to 5.50	2012
			3/8-inch PG 64-28 HMA		

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APPENDIX A: SURVEY OF STATE AGENCIES

The following survey was distributed to state agencies in the form of an online SurveyMonkey link in the summer of 2014. Responses are included.

Survey on the Use of Successful Asphalt Pavement Methods for Climate Zones Similar to Eastern Washington and Washington State Mountain Pass Areas

This survey is intended to collect feedback on successful HMA pavement construction practices, preservation methods, and/or material selection for climate zones similar to Washington's mountain pass areas and east of the Cascades. The purpose of the research is to determine if changes can be made east of the mountains and in mountain passes to improve pavement performance in those areas. The main concern is the performance of HMA rehabilitation/preservation treatments consisting of inlays and overlays using ½ inch Superpave HMA which has been used in Washington. Currently, Washington State Department of Transportation (WSDOT) uses the same mix design procedures (except for binder PG selection) and construction methods for HMA pavements throughout the state. We appreciate your timely response on this survey.

The climate of Washington *west of the Cascade Mountains* is mild with light to moderate rainfall 150 to 200 days each year. Temperatures range from 75° to 90°F in summer and 25° to 45°F in the winter. Hot Mix Asphalt (HMA) pavements in this area perform well with an average service life of 16.9 years. However, HMA in Washington's *mountain pass areas* are subject to mild summers with extreme winter events that include frequent snow flurries and freezing conditions, including many freeze/thaw cycles. Temperatures range from as low as -15°F with an average of 15° to 35°F in winter, to as high as 105°F with an average of 45° to 85°F in summer. Snowfall over the Cascades ranges from 50 inches to as much as 400 inches and HMA pavements have an average service life as low as 5 years. The climate *east of the Cascades* is drier and sunnier with more extreme temperatures which often drop below freezing during the winter. Temperatures can exceed 100°F in the summer and drop to as low as -10° in winter. It rains from 70 to 120 days each year and the average HMA pavement life is 11 years. Additionally, studded tires are widely used over the mountain pass areas and in Eastern Washington.

1) Does your state have regions that have substantially different climates?

   a. Yes

   b. No

Yes [11 of 27 responses to this question]

No [16 of 27 responses]

State	Different Climates
Alaska	Yes
Arkansas	No
California	Yes
Colorado	Yes
Connecticut	No
Delaware	No
Florida	No
Georgia	Yes
Illinois	Yes
Kentucky	No
Maryland	Yes
Michigan	Yes
Minnesota	No
Missouri	No
Nebraska	No
Nebraska	No
Nevada	Yes
North Carolina	No
Ohio	No
Oklahoma	No
Oregon	Yes
South Carolina	No
South Dakota	No
Tennessee	No
Utah	Yes
Washington, D.C.	No
Wisconsin	No
Wyoming	Yes

2) What has your agency found to be the average surface life of HMA pavements for the following climatic conditions within your state?

   a. Dry and sunny with more extreme which often drop below freezing during winter (temperatures ranging from -10° to 100°F)

      __________ years (min) to __________ years (max)

[17 Agencies Responded]

State	Average Pavement Life (Years)
Alaska	7-15
California	10
Colorado	12-16
Delaware	8-15
Georgia	10-14
Kentucky	10-20
Minnesota	10-16
Missouri	15-20
Nebraska	6-15
Nevada	10-18
North Carolina	11.4
Ohio	10-13
Oklahoma	10-15
South Carolina	8-15
South Dakota	12-20
Tennessee	9-12
Utah	12

   b. Mild in summer with severe winters with frequent snow and freezing conditions (mountain passes) (temperatures ranging from -15° to 85°F)

      __________ years (min) to __________ years (max)

[12 Agencies Responded]

State	Average Pavement Life (Years)
Alaska	7-15
California	12.5
Colorado	8-12
Connecticut	11
Illinois	8-18
Maryland	17
Minnesota	10-16
Nevada	10-12
Ohio	8-11
South Carolina	8-15
Utah	10
Wisconsin	18

3) What are the top failure modes of HMA pavements in climates similar to *east of the Cascades* (warm, dry, 90°F+ summers and cold winters with periods of freezing weather) and Washington's *mountain passes* (snow zone with freezing and thawing temperatures, snow removal, studded tire and chain use, inadequate pavement structure, heavy deicing and anti-icing chemical use)? Failure modes would include: fatigue cracking, thermal cracking, rutting, etc.

[21 Agencies Responded]

State	Top Failure Modes
Alaska	Rutting from studded tires, Thermal cracking, Fatigue cracking
California	Fatigue, Thermal Cracking, Rutting
Colorado	Warm Climate: Thermal cracking, Mountain passes: fatigue cracking
Connecticut	Raveling (wet freeze climate), cracking (reflective cracking in overlays), polishing (when pavements last a long time, 15 years +)
Delaware	Fatigue, Structural failures, Environmental cracking
Georgia	Thermal cracking, Raveling, Fatigue cracking
Illinois	in our focus to eliminate rutting we now have cracking, raveling and potholing, No studs/chains allowed
Kentucky	Age related thermal cracking, Reflective cracking, Joint deterioration
Maryland	Weathering and raveling
Minnesota	Thermal/reflective cracking, Deterioration at cracks, Stripping
Missouri	Fatigue cracking, Thermal cracking
Nebraska	Thermal cracking, Fatigue cracking due to stripped HMA and/or subgrade failure/ freeze-thaw, Rutting
Nevada	Longitudinal cracking, Raveling/Stripping, Thermal Cracking
North Carolina	Cracking, Block cracking from oxidation, not from cold temperatures. Mild rutting
Ohio	Oxidative distress such as raveling, potholes, delamination, etc., Limited rutting, Cracking
Oklahoma	Physical/Chemical damage - Plow truck damage and deicing agent Stripping - Freeze/thaw cycles and extreme swings in temperature Thermal cracking
South Carolina	Fatigue, reflective, block cracking underneath, rutting
South Dakota	Fatigue Cracking, Thermal Cracking, Block Cracking
Tennessee	Fatigue cracking, Delamination, Premature longitudinal joint failure
Utah	Thermal cracking, rutting, fatigue cracking, poor construction
Wisconsin	Fatigue cracking, thermal cracking

4) Does your state have regions with climates similar to *east of the Cascades* (warm, dry, 90°F+ summers and cold winters with periods of freezing weather)? If yes, please answer the following:

    a. What procedure do you use to design your HMA mixes? (Superpave, etc.)

    ____

    ____

[15 Agencies Responded]

State	Mix Design Method
Alaska	Marshall (Type II-A, Type II-B mixes) mainly in rural areas; Superpave in specific urban areas
California	Hveem historically, (Superpave last two years)
Colorado	Superpave
Delaware	Superpave
Georgia	Superpave dense-graded mixtures, Marshall for open-graded SMA mixtures
Kentucky	Superpave
Michigan	Superpave
Minnesota	Superpave
Missouri	Major Routes - Superpave & Minor Route - Superpave or Marshall
Nebraska	Superpave
Nevada	Hveem
North Carolina	Superpave
Oklahoma	Superpave
Tennessee	Tennessee Marshall Specification
Utah	Superpave

b. If you use Superpave, is there any modification to the procedure?

[13 Agencies Responded]

State	Modification to Mix Design
Alaska	N-design=75gyrations; AggFracture(2-face)= 98%min; Flat&Elong=8%max(1:5); NordicAbrasion=8.0%max; mix has to pass APA test.
California	Added Hamburg and AASHTO T-283
Colorado	No
Delaware	Increased VMA 1/2% more than recommended minimum in R35
Georgia	Age mixtures for only 2 hours and gyrate at 65 gyrations
Kentucky	No
Michigan	No
Minnesota	No
Missouri	No
Nebraska	state specific # of gyrations
North Carolina	We have decreased gyrations and increased the liquid asphalt to reduce cracking
Oklahoma	Yes. http://www.odot.org/c_manuals/specprov2009/oe_sp_2009-708-26.pdf
Utah	No

c. If you use Superpave, do your mixes tend to be coarse graded, fine graded, or both?

____

____

[13 Agencies Responded]

State	Mix
Alaska	Maybe both, typically ~ 50% passing #4.
California	Both
Colorado	Both - depends on traffic volumes
Delaware	Follow the recommended lift thickness based upon the nominal aggregate size, 3x
Georgia	fine
Kentucky	Both
Michigan	Both
Minnesota	Both
Missouri	Coarse
Nebraska	1/2-inch gradation band which typically leads to finer gradations than surrounding states
North Carolina	Initially coarse, then contractors moved to the mid-range for workability
Oklahoma	Fine
Utah	both

d. What type of mix(es) do you typically use for the wearing course (½ inch, SMA, etc)?

[15 Agencies Responded]

State	Wearing Course
Alaska	Type II mix: 100% passing 3/4in; 75-90% passing 1/2in; Superpave: 65-90% passing 1/2in, with CoarseAgg. NordicAbrasion of 8.0%(max)
California	3/4-inch HMA and 5/8-inch RHMA-G (Rubberized HMA - Gap Graded)
Colorado	Typically 1/2-inch SMA's in the Metro Areas where traffic/truck volumes are higher
Delaware	9.5mm, 12.5mm and more 4.75mm (for 3/4-inch thin overlays)
Georgia	1-1/2-inch 9.5mm SP, 1-12/-inch 12.5mm SP, on interstates we use 3/4-inch or 1-1/4-inch of 12.5mm open graded mix
Kentucky	0.38 inch Superpave Surface
Michigan	3/8-inch
Minnesota	Has been 3/4-inch, moving to 1/2-inch or using Nova Chip
Missouri	Interstate - 3/8 & 1/2 inch SMA, Major and Minor Rte - 3/8 & 1/2 inch Superpave or Marshall
Nebraska	1/2-inch
Nevada	3/4-inch thick Open-Graded mix
North Carolina	1.5-inch S9.5C
Oklahoma	1/2-inch NMS
Tennessee	Either 1/2-inch NMAS dense-grade or 1/2-inch NMAS OGFC
Utah	SMA, 1/2-inch chip seal

e. What is your minimum density requirement?

[16 Agencies Responded]

State	Minimum Density Requirement
Alaska	92%
California	91%
Colorado	92% of theoretical maximum specific gravity
Delaware	93%, and below 88% is remove and replace
Georgia	maximum 7% in-place air voids
Kentucky	92%-96.5%
Michigan	92%
Minnesota	92% of Gmm for 4% design void mixes and 93% for 3% design void mixes
Missouri	94% for SMA 92% for all others
Nebraska	92.5% based off max density
Nevada	90% single, 92% average
North Carolina	95%
Oklahoma	88.1%. See Table 411:2 in http://www.odot.org/c_manuals/specbook/oe_ss_2009.pdf
Tennessee	92%
Utah	93.5

f. Do you use the Hamburg Wheel Tracking Test (HWTT), Indirect Tensile Strength (IDT), or Elastic Recovery (ER)?

____

____

[15 Agencies Responded]

State	Method
Alaska	Asphalt Pavement Analyzer (APA) is used for mixes; ER is sometimes used for PMA binder.
California	Hamburg on mix, ER on binder
Colorado	HWTT and ER
Delaware	IDT
Georgia	HWTT and APA for rut resistance testing
Kentucky	ER
Michigan	No
Minnesota	No
Missouri	ER for major routes - min 65%
Nebraska	ER
Nevada	No
North Carolina	No
Oklahoma	HWTT, AASHTO T 283 (TSR), and MSCR Recovery - http://www.odot.org/c_manuals/specprov2009/oe_sp_2009-708-28.pdf
Tennessee	We do specify T301 Elastic Recovery for modified binders
Utah	HWTT

g. Please describe any other procedures to extend the pavement life in this climatic zone.

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[8 Agencies Responded]

State	Methods
Alaska	Use Min 0.3% liquid antistrip agent in binder; polymer-modified binder; WMA (chemical, organic, not foamed) as compaction aid; MTV; IC; longitudinal joint-heater; echelon paving if feasible; maybe avoid RAP in wearing course.
Colorado	Selecting the proper binder for the climate
Georgia	Proper binder selection, perform crack filling/sealing, strip sealing, and looking at fog sealing
Minnesota	PG binder selection, TSR
Nebraska	Highly polymerized binders (64-34). -34 is used in part due to high RAP contents (ave. 40%)
Nevada	PMA
Oklahoma	We don't use it but Steel Slag or other hard aggregate types would be good for studded tires.
Utah	Seal every 8-10 years with microsurface or chip seal

5) Does your state have regions with climate similar to Washington *mountain passes* (snow zone with freezing and thawing temperatures, snow removal, studded tire and chain use, heavy deicing and anti-icing chemical use)? If yes, please answer the following:

    a. What procedure do you use to design your HMA mixes? (Superpave, etc.)

    ____

    ____

[11 Agencies Responded]

State	Mix Design
Alaska	Marshall (Type II-A, Type II-B mixes) mainly in rural areas; Superpave in specific urban areas.
California	Hveem historically, (Superpave last two years)
Colorado	Superpave
Illinois	Superpave, HWTT, Modified T183 (min and max strengths)
Maryland	Superpave
Michigan	Superpave
Minnesota	Superpave
Nevada	Hveem
Ohio	Superpave for high traffic, Marshall for lower traffic
Utah	Superpave
Wisconsin	Superpave

b. If you use Superpave, is there any modification to the procedure?

[9 Agencies Responded]

State	Modifications to Mix Design
Alaska	N-design=75gyrations; AggFracture(2-face)= 98%min; Flat&Elong = 8%max(1:5); NordicAbrasion=8.0%max; mix has to pass APA test.
California	Added Hamburg and AASHTO T-283
Colorado	No
Maryland	No
Michigan	No
Minnesota	No
Ohio	Fewer gyrations for more binder content
Utah	No
Wisconsin	No

c. If you use Superpave, do your mixes tend to be coarse graded, fine graded, or both?

[10 Agencies Responded]

State	Mix
Alaska	Maybe both, typically ~ 50% passing #4.
California	Both
Colorado	Both - depends on traffic volumes
Illinois	Fine
Maryland	Coarse
Michigan	Both
Minnesota	Both
Ohio	Middle to fine
Utah	Both
Wisconsin	Fine

d. What type of mix(es) do you typically use for the wearing course (½ inch, SMA, etc)?

State	Wearing Course
Alaska	Type II mix: 100% passing 3/4in; 75-90% passing 1/2in; Superpave: 65-90% passing 1/2in, with Coarse agg. Nordic abrasion of 8.0 %(max)
California	3/4-inch HMA and 5/8-inch RHMA-G (Rubberized HMA - Gap Graded)
Colorado	1/2-inch
Illinois	We have abandoned 1/2-inch and are focusing on 9.5mm, looking to use more SMAs
Maryland	SMA for interstate, 1.5-inch 9.5mm dense or 2-inch 12.5mm dense for others
Michigan	3/8-inch
Minnesota	Has been 3/4-inch, moving to 1/2-inch or using Nova Chip
Nevada	3/4-inch thick open-graded mix
Ohio	12.5mm for high traffic, 9.5 or similar for low traffic
Utah	SMA, 1/2-inch, chip seal
Wisconsin	SMA and 12.5mm

e. What is your minimum density requirement?

[11 Agencies Responded]

State	Minimum Density Requirement
Alaska	92%
California	91%
Colorado	92% of theoretical maximum specific gravity
Illinois	Surface 92, binder 91. Would like to increase but have pushback from industry
Maryland	92% for full pay, 88% for acceptance
Michigan	92%
Minnesota	92% of Gmm for 4% design void mixes and 93% for 3% design void mixes
Nevada	90% single, 92% average
Ohio	93 for low traffic, 94 for high traffic
Utah	93.5%
Wisconsin	91.5%

f. Do you use the Hamburg Wheel Tracking Test (HWTT), Indirect Tensile Strength (IDT), or Elastic Recovery (ER)?

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[10 Agencies Responded]

State	Method
Alaska	Asphalt Pavement Analyzer (APA) is used for mixes; ER is sometimes used for PMA binder.
California	Hamburg on mix, ER on binder
Colorado	HWTT and ER
Illinois	Hamburg and ER, trying to develop thermal/fatigue test
Michigan	No
Minnesota	No
Nevada	No
Ohio	ER
Utah	HWTT
Wisconsin	No

6) If SMA is used for the wearing course:

   a. What are the pavement lives of SMA and HMA layers, respectively?

[13 Agencies Responded]

State	SMA Life	SMA Cost
Alaska		
California		
Colorado	18 years	$90/ton
Delaware		$90/ton
Georgia	SMA not left as wearing coarse	
Illinois	SMAs have been mainly used in higher traffic locations. Due to their good performance their use is being explored for lower traffic locations (higher initial cost but longer life)	
Maryland	13 (much more traffic)	$95/ton
Minnesota	15	$100/ton
Missouri	20-25	$88.65/ton (PG 76-22)
Nevada	No	No
Oklahoma		$116/ton
Utah	16-20	$10-15 more per ton $100/ton
Wisconsin		Highly variable

b. What are the costs of SMA and HMA in your state, respectively?

[10 Agencies Responded]

State	HMA Life	HMA Cost
Alaska		$80-100/ton; Neat PG 52-28 = $600/ton
California	15?	$102 for HMA; $110 for RHMA (averaged over last 4 yrs)
Colorado	12 years	$70/ton
Delaware		$70-80/ton
Illinois	Prices have varied widely due to recent materials changes, acceptance methods and program size. When SMA aggregates are available locally the mix may be only a few $$ more per ton.	
Maryland	16 statewide	$80/ton
Minnesota	10-16	$60/ton
Missouri	15-20	$69.34/ton (PG 76-22)
Oklahoma		$86 http://www.odot.org/contracts/avgprices/index.php
Utah	10-16	$70

7) What does your state do when constructing HMA longitudinal joints to maximize their performance in climates similar to *Eastern Washington* (warm, dry, 90°F+ summers and cold winters with periods of freezing weather) and *mountain passes* (snow zone with freezing and thawing temperatures, snow removal, studded tire and chain use, heavy deicing and anti-icing chemical use)?

[18 Agencies Responded]

State	Longitudinal Joint Construction Technique
Alaska	Use Min 0.3% liquid antistrip agent in binder; polymer-modified binder; WMA (chemical, organic, not foamed) as compaction aid; MTV; IC; longitudinal joint-heater; echelon paving if feasible; maybe avoid RAP in wearing course.
California	Nothing special
Colorado	Long. Joint spec. with a target density of 92% of theoretical maximum specific gravity +/- 4%
Connecticut	Use notched-wedge joint since 2008 (has significantly improved the pavement longevity at the joints at least through 2014) - measure density of joint on the hot side and on the mat via cores
Delaware	No variations in joint construction
Georgia	tack the vertical face of longitudinal joint and stagger each subsequent layer 12 inches
Illinois	Longitudinal joints continue to be an issue. Recently pave and trim 6 inches has been tried with success. also, the introduction of much "heavier" tack coats seems to help "confine" the edge aiding compaction. SMAs usually resist moving from the roller and compact leaving a straight edge
Maryland	Overlap existing pavement 1 to 1.5 inches
Michigan	We have an incentive special provision for density at the longitudinal construction joints
Minnesota	Use joint adhesive, fog on LJ, longitudinal joint density requirement
Missouri	MoDOT has a density requirement. Within 6 inches of the unconfined joint in the travelway the density shall not be less than 2% of the specified density. If min. density is 92% the unconfined joint can't be lower than 90%. Confined joint in the travelway shall have the same density as the mainline.
Nebraska	We are in our first year of using a joint density specification and promoting construction of a notched wedge for improved density
Nevada	MTV, cold in-place recycling
North Carolina	Set up rolling patterns and pay attention during laydown
Ohio	Joint cores on high traffic, nothing on low traffic
Oklahoma	Longitudinal Joint Density - http://www.odot.org/materials/pdfs-ohdl/ohdl14.pdf, http://www.odot.org/c_manuals/specprov2009/oe_sp_2009-411-12.pdf
Tennessee	We specify longitudinal joint density on select projects and a spray coat of bituminous material (tack) is required to be placed on the joint face prior to the 2nd pass on all projects.
Utah	We are looking at this, nothing right now Nothing special, this is a problem for us as well

8) Please describe any other construction practices, preservation methods, or material selection that your agency has found to be successful for HMA pavements in climate zones similar to Washington's mountain pass areas and Eastern Washington.

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[9 Agencies Responded]

State	Other Methods
Illinois	We are trying to quantify the value of polymer, high asphalt content, adhesion agents, etc. with an ongoing research project with the University of Illinois
Michigan	Placement requirements based on surface temperature of pavement or base being overlaid. Use of warm mix asphalt. Aggressive preventive maintenance program. Modified binders.
Minnesota	IR thermal imaging, IC rollers
Missouri	Ensure the correct amount of tack is applied (min. 0.05 gallons per square yard) and is applied uniformly. Conduct QC and QA TSR tests on field produce mix.
Nevada	MTV, Lime treatment
North Carolina	We struggle to get uniform quality tack coats. Definitely a work in progress.
Ohio	Fine grading with better tack and more thickness of lift
Utah	Chip seal and microsurfacing
Wisconsin	Proper and effective maintenance

APPENDIX B: LABORATORY ANALYSIS OF FIELD CORES

B.1 Preparation of Samples

All field cores were collected by WSDOT from the center of the outside lane for each project. The field cores were 4 inches in diameter and varied in height, depending on the depth of the core. The bottom ends of the cores were sawn to a produce a flat surface. Cores taken directly after construction were treated as being short term aged in production and placement and were aged in an oven at compaction temperature for five days before conducting performance tests.

B.2 Description of Experiments

The following sections describe the various laboratory experiments used for this study.

B.2.1 Studded Tire Wear Tests

Studded tires are commonly used to improve traction on snowy roads in areas of the United States that experience heavy snowfall. While providing increased traction, studded tires cause significant and costly damage to the roadway surface. Transportation agencies in states that experience this problem are in need of the development of studded tire wear resistant asphalt mixtures.

The studded tire wear simulator/tester developed at Washington State University is shown in Figure B.1. The wear simulator consists of a modified drill press with two free-rolling rubber tires with studs. The tires are contacted with the asphalt sample surface at 100 psi and torque is applied to the wheels at a speed of 140 revolutions per minute (RPM). Friction causes the two wheels to roll and the asphalt sample is worn in a similar way to conditions in the field.

The resulting wear on a field core is shown in Figure B.2. The studded tire wear simulation tests were performed at a temperature of 69.8°F.

Figure B.145 Studded Tire Wear Simulator with Gyratory Sample

Figure B.2 Field Core Sample after Studded Tire Simulation

The wearing resistance of the asphalt mixtures was measured by the sample mass loss after two minutes in the studded tire wear simulator. The mass loss was calculated as the difference in the specimen mass before and after the studded tire wearing. A lower amount of mass loss indicates greater resistance to studded tire wear.

B.2.2 Preparation of Samples for Indirect Tensile (IDT) Test Machine

After the studded tire wear simulation, the gyratory samples were cut to a height of 1.5 inches and cored to a diameter of 4 inches, with a target air void content of 4% (±0.5%). For the field cores, a thin layer was cut off the top lift to produce a smooth surface. The bottom end of the field core was cut to produce a specimen height of 1.5 inches. Linear variable differential transformers (LVDTs) were mounted to the smooth surfaces of the gyratory samples and field cores to measure deformation during the IDT tests.

B.2.3 IDT Machine and Setup

A servo-hydraulic Geotechnical Consulting Testing System (GCTS) with an environmental chamber was used to test the field cores and gyratory compacted specimens. The setup consists of four linear variable differential transformers (LVDTs) that are mounted to each sample, with two in the front and two in the back, as shown in Figure B.3. The distance between the mounts, known as the gauge length, is two inches. The sample with mounted LVDTs is placed in the loading apparatus and is only contacted vertically, on the top and bottom. Plates with curved loading strips are guided by four steel columns to apply a uniform load along the vertical plane. When a load is applied to the sample, the LVDTs measure the horizontal and vertical deformations, which are used to determine various parameters such as dynamic modulus and creep compliance values. The IDT test setup is shown in Figure B.4.

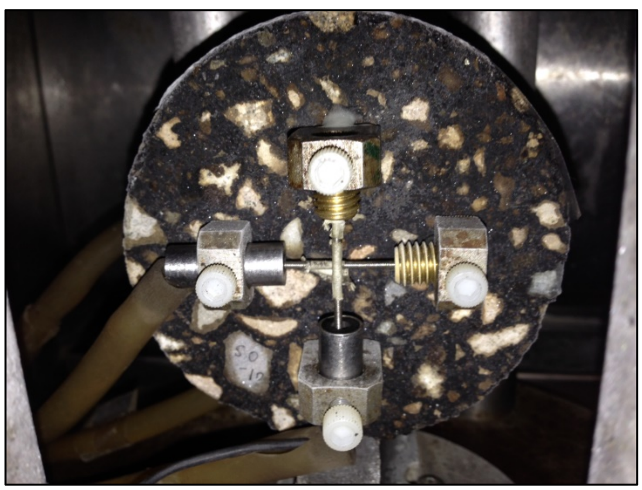

Figure B.3 Asphalt Mixture Sample Mounted with LVDTs

Figure B.4 IDT Test Machine Setup

For each project, at least three samples were used for the dynamic modulus and creep compliance tests. Because the dynamic modulus and creep compliance tests are non-destructive, the same samples could be used for the fatigue tests. Three other samples were used for the low temperature tests for thermal cracking properties. A minimum of six cores were taken from each location to ensure three samples could be used for each test. When more than six cores were available, the air void levels of the cores chosen for each group of tests (i.e., fatigue and thermal), include representative low, medium, and high levels within the range of all the available cores, with a target average air void that was representative of the average of all the available cores.

B.2.4 Dynamic Modulus Test

The dynamic modulus, |E*|, is regarded as a good indicator of the stiffness of asphalt mixtures. The test is performed by applying a cyclic loading to the sample in order to produce approximately 100 microstrain and avoid damaging the sample. The tests were performed at six temperatures (-4, 14, 32, 50, 68, 86 °F) and six loading frequencies (20, 10, 5, 1, 0.1, 0.01 Hz) at each temperature. The test progresses with temperatures increasing from low to high, and with frequencies decreasing from high to low. The purpose of this order is to minimize the deformation of the sample throughout the test, as the most strain will occur at the highest temperature and lowest frequency.

The dynamic modulus was calculated following procedures outlined by Kim and Wen (2002). Due to the non-uniform distribution of strain throughout each gauge length, the deformation recorded by the vertical and horizontal LVDTs must be converted to strain in the center vertical plane of the specimen where the maximum tensile stress/strain or fracture occurs. To do this, the average deformations measured by the vertical and horizontal strain gauges are multiplied by constant values dependent on the strain gauge length and specimen diameter. First, Poisson's ratio is obtained using Equation B.1.

$$v = -\frac{\alpha_1 U(t)+V(t)}{\alpha_2 U(t)+\alpha_3 V(t)} \tag{B.1}$$

where:

v = Poisson's ratio

$\alpha_1, \alpha_2, \alpha_3$ = constants dependent on strain gauge length and sample geometry. For this study: 4.58, 1.316, and 3.341, respectively.

$U(t)$ = average horizontal deformation, in.

$V(t)$ = average vertical deformation, in.

t = time, sec.

Once Poisson's ratio is obtained, the strain at the center of the sample is calculated using Equation B.2

$$\varepsilon_{x=0} = U(t)\frac{\gamma_1+\gamma_2 v}{\gamma 3+\gamma_4 v} \tag{B.2}$$

where:

$\varepsilon_{x=0}$ = strain at the center of the sample, in/in

$\gamma_1, \gamma_2, \gamma_3, \gamma_4$ = constants related to strain gauge length and specimen geometry. For this study: 12.4, 37.7, 0.471, and 1.57, respectively.

The tensile stress along the vertical plane was calculated using Equation B.3.

$$\sigma_{x=0} = \frac{2P}{\pi t D} \tag{B.3}$$

where:

$\sigma_{x=0}$ = tensile stress at the center of the sample, psi

P = applied load, lbs.

t = sample height, in.

D = sample diameter, in.

The last ten cycles of stress amplitudes and center strain amplitudes were averaged for each test. The dynamic modulus values were then calculated by dividing the stress amplitudes by the strain amplitudes, as shown in Equation B.4. The dynamic modulus values were determined for each combination of six temperatures and six loading frequencies, resulting in a total of 36 dynamic modulus values for each sample.

$$|E^*| = \frac{\sigma_0}{\varepsilon_0} \tag{B.4}$$

where:

$|E^*|$ = dynamic modulus, psi

σ_0 = average of last ten load amplitudes, psi

ε_0 = average of last ten amplitudes of strain at the center of the sample, in/in

The principal of time-temperature superposition was used to shift the dynamic modulus values along the frequency axis to develop master curves for a wide range of frequencies. The master curves were constructed by fitting a sigmoidal function to the calculated dynamic modulus values using non-linear least squares regression methods. The sigmoidal function used to construct the dynamic modulus master curves is given in Equation B.5.

$$Log|E^*| = a + \frac{b}{1+e^{c-d\left(Log(F)+Log(a_T)\right)}} \quad \text{(B.5)}$$

where:

a, b, c, d = regressed model constants

F = frequency, Hz

a_T = shift factor for each temperature

B.2.5 Creep Compliance Test

Creep compliance is regarded as a good indicator of the softness of asphalt mixtures. The test is performed by applying a constant load for 100s to the sample. The tests were performed at six temperatures (-4, 14, 32, 50, 68, 86-°F), progressing from low to high temperatures. The tests are done in this order to minimize the deformation of the sample throughout the test, as the most deformation will occur at the highest temperature.

The creep compliance was calculated following the procedure outlined by Wen and Kim (2002) and given in Equation B.6.

$$D(t) = -\frac{d}{P} * [\beta_1 U(t) + \beta_2 V(t)] \quad \text{(B.6)}$$

where:

D(t) = creep compliance, 1/psi

t = time, s

d = sample thickness, in.

P = applied load, lb.

U(t) = average horizontal deformation, in.

V(t) = average vertical deformation, in.

β_1, β_2 = constants related to strain gauge length and specimen geometry. For this study: 0.4032 and 1.024, respectively.

The principal of time-temperature superposition was used to shift the creep compliance values along the time axis to develop master curves for a wide range of time. The master curves were constructed by fitting a sigmoidal function to the calculated creep compliance values using non-linear least squares regression methods. The sigmoidal function used to construct the creep compliance master curves is given in Equation B.7.

$$Log|D(t)| = a + \frac{b}{1+exp^{d+eLog(t)}} \tag{B.7}$$

where:

D(t) = creep compliance as a function of time, 1/psi

t = time, s

a, b, d, e = regressed model constants

B.2.6 IDT Fatigue and Thermal Cracking Test

The asphalt mixture fatigue and thermal cracking properties were evaluated using IDT monotonic fracture energy tests and by following procedures outlined by Kim and Wen (2002). Tests were performed on samples of 1.5 inch thickness and 3.9 inch diameter. The fracture tests are performed to calculate peak IDT strength at failure, fracture work density, and horizontal failure strain. These parameters are summarized in Table B.1.

Table B.1 Fatigue and Thermal Cracking Test for Asphalt Mixtures

Test	Fatigue	Thermal Cracking
Temperature, °F	68	14*
Loading Rate, in/min	2	0.1
Mechanical Parameters	IDT strength, fracture work density, horizontal failure strain	IDT strength, fracture work density

**Note: The temperature used for thermal cracking varies with the low temperature PG grade of the asphalt binder (AASHTO T 322). For this project, 14°F was selected.*

B.2.6.1 IDT Strength

IDT strength is the peak stress the sample experiences during the fracture test. IDT strength is displayed graphically in Figure B.5 and is calculated using Equation B8.

$$IDT\ Strength = \frac{2P}{\Pi DT} \tag{B.8}$$

where:

P = peak load, lb.

D = specimen diameter, in.

t = specimen thickness, in.

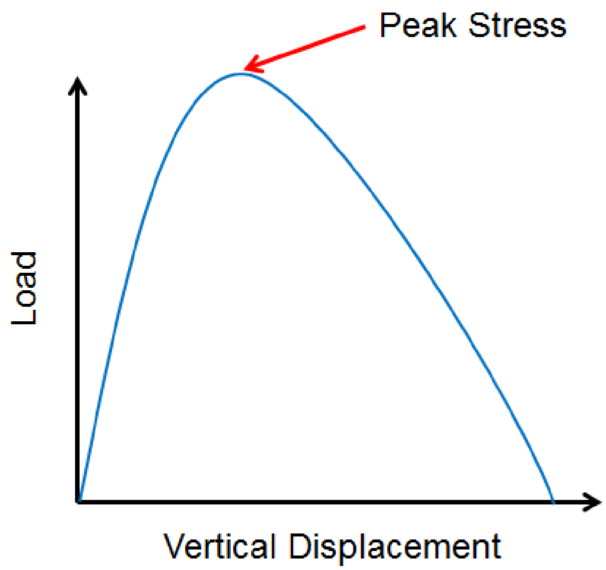

Figure B.546 IDT Strength

B.2.6.2 Fracture Work Density

The fracture work density was obtained from the IDT fatigue and thermal cracking test results. Fracture work density is defined as the area under the loading curve versus the vertical displacement, as shown in Figure B.6, per unit volume. According to Wen (2013), fracture work density correlates well with bottom-up fatigue cracking.

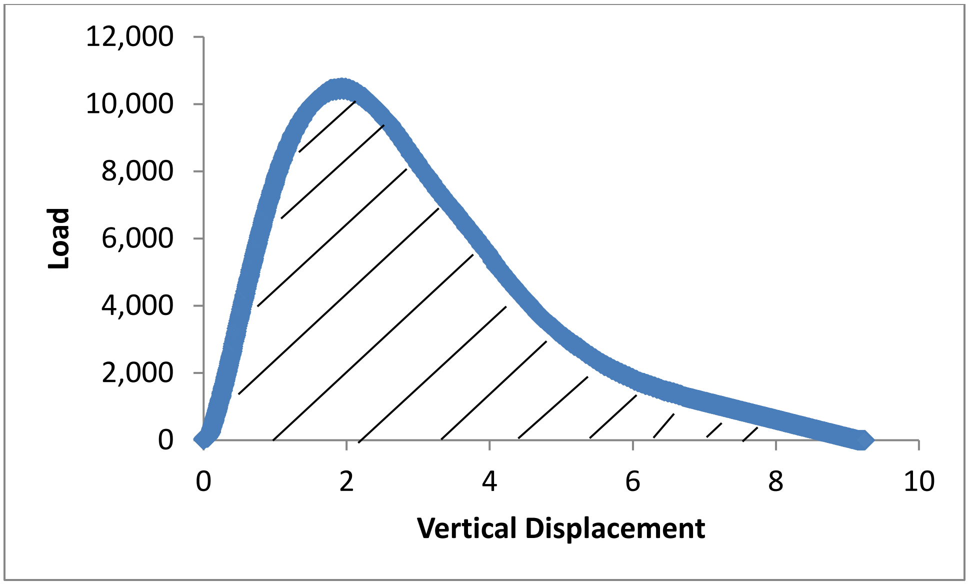

Figure B.6 IDT Fracture Work

B.2.6.3 Horizontal Failure Strain

The horizontal failure strain is the strain along the horizontal axis at failure. Horizontal failure strain at intermediate temperatures has shown to correlate well with top-down cracking when performed at intermediate temperatures (Wen and Bhusal, 2014) and was calculated from the fatigue test results.

B.2.7 Hamburg Wheel-Tracking Test

The Hamburg Wheel Tracking Test is a laboratory test to measure rutting and moisture damage of asphalt mixtures by repeatedly rolling a steel wheel across a 6 inch diameter specimen surface while it is immersed in water at 122°F (WSDOT, 2012). According to Hurley and Prowell (2006), a mix is considered good if it does not meet the stripping inflection point (Figure B.7) by 10,000 passes. WSDOT performs the HWTT for 20,000 repetitions and the rut depth is measured in millimeters. The HWT tests were performed in accordance with AASHTO T 324 by WSDOT.

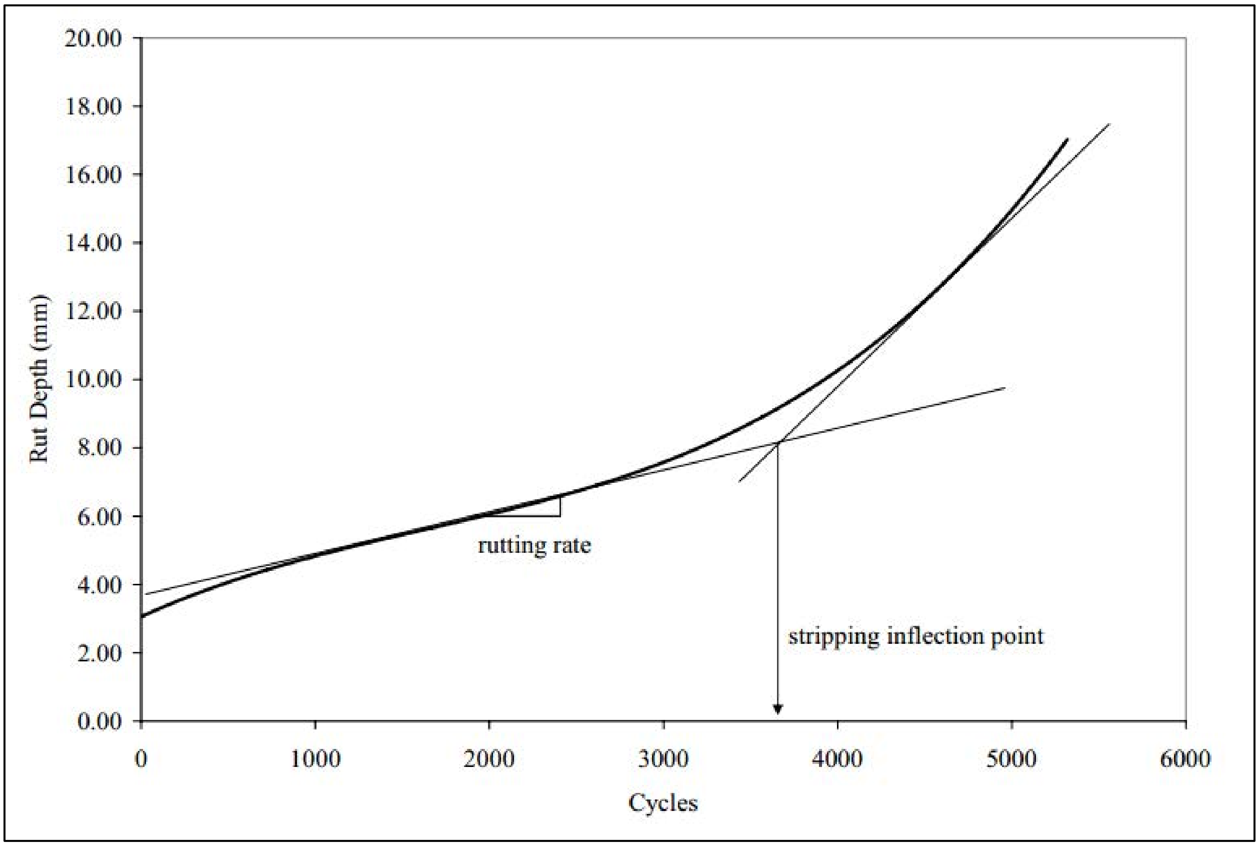

Figure B.7 Hamburg Test Results (Hurley and Prowell, 2006)

B.2.8 Asphalt Binder Tests

B.2.8.1 Binder Extraction and Recovery

The asphalt binders were extracted from field cores according to AASHTO T 164: Standard Method of Test for Quantitative Extraction of Asphalt Binder from Hot-Mix Asphalt (HMA) (AASHTO, 2014). The samples were first heated in a conventional oven to 230°F until they could be broken apart and separated, then allowed to cool at room temperature. Approximately 17 ounces of combination 85% Toulene and 15% Ethanol by volume was placed in a Houghton centrifuge extractor with approximately 1 pound of loose mix. The loose mix and chemical was left for 15 minutes to allow the binder to dissolve before turning on the centrifuge and increasing extraction speed up to 3,600 RPM. It generally took several extractions to ensure that most of the binder had been extracted. The binders were recovered from the chemical solution according to AASHTO T 170. The solution was heated to its boiling point and distilled until the chemical was separated.

B.2.8.2 Binder PG Grading

Binder PG grading was performed according to AASHTO PP6 and the PG grade was calculated based on the high and low temperature test results. The recovered binder was treated as being short-term aged in the field; therefore, the recovered binder was not aged in the rolling thin-film oven (RTFO). When evaluating treatment samples, a variation in high or low PG grade by ≥ 6 degrees was considered to be a significant difference.

B.2.9 Statistical Tests for Significance

Tests of statistical significance such as the t-test are inadequate for interpreting data when only three replicates are used for each sample. When evaluating performance parameters from results of laboratory experiments which involved ≤ 3 replicates, the effect size method (Cohen, 1992) was used to determine whether a statistical difference existed among mixtures. The effect size is calculated using Equation B.8. For this study, an effect size of 1.6 was used to determine significant differences between treatment and control groups. When more than three replicates were available for testing, a two-tailed t-test was used for an analysis of variance (ANOVA) and a significance level (p-value) of less than 0.05 indicated statistical significance.

$$d = \frac{|\overline{x_t} - \overline{x_c}|}{\sqrt{\frac{(n_t - 1){s_t}^2 + (n_c - 1){s_c}^2}{n_t + n_c}}} \tag{B.9}$$

where:

d	= effect size
$\overline{x_t}$	= mean of treatment group
$\overline{x_c}$	= mean of control group
n_t	= number of samples in treatment control group
n_c	= number of samples in control group
s_t	= standard deviation of treatment control group
s_c	= standard deviation of control group

APPENDIX B1: 3/8-inch VS. 1/2-inch NMAS PROJECT

B1.1 Project Description

In order to determine the effect of using a smaller nominal maximum aggregate size, a 3/8-inch NMAS asphalt mixture was compared to a conventional 1/2-inch NMAS asphalt mixture. A test section of 3/8-inch NMAS HMA was constructed on I-90 next to a 1/2-inch NMAS control section. Contract 8611 was paved in 2014 and is located on I-90 from Barker Road to Idaho State Line in the Eastern Region. The Eastbound right lane from MP 297.956 to MP 298.335 was constructed as a 3/8-inch PG 70-28 HMA and the rest of the project was paved with 1/2-inch PG 70-28 HMA. The 1/2-inch asphalt mixture contains 4.9 percent binder, and the 3/8-inch asphalt mixture contains 5.4 percent binder. The gradations of the two mixes are shown in Figure B1.1 and further details of the mix designs are provided in Table B1.1. The averaged air void percentages of the pavement cores are shown in Table B1.2; the 1/2-inch and 3/8-inch mixes had average air void contents of 6.3% and 6.1%, respectively. Field cores of the 1/2-inch and 3/8-inch mixtures were taken by WSDOT. It is noted that both mixes were warm mix asphalt by foaming and were heated to over 300°F, as shown in Figure B1.2. The contractor on site stated that it was common practice to heat WMA to over 300°F to ensure workability, resulting in a “hot warm mix” that is said to aid workability when paving at night and during lower temperatures (Guy Anderson, personal communication, July 21st, 2014).

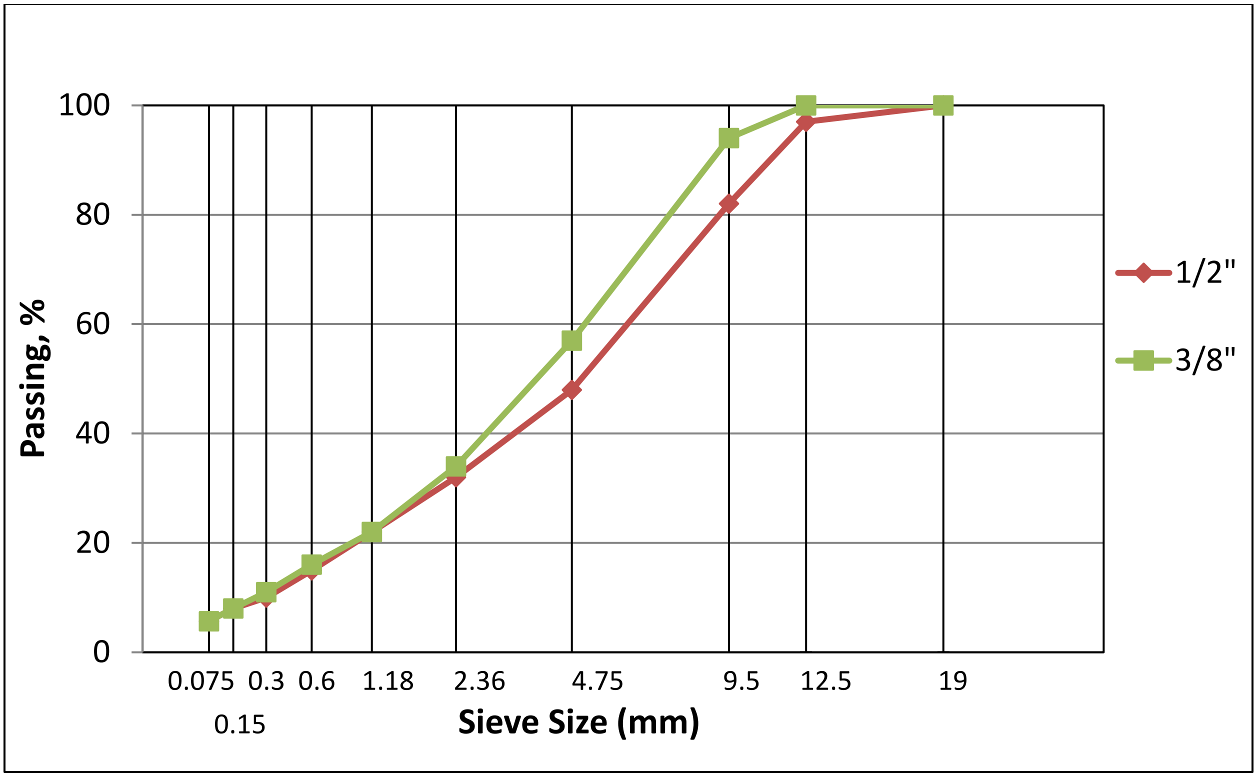

Figure B1.1 Gradations of 1/2-inch and 3/8-inch Mixes

Table B1.1 Volumetrics of 1/2-inch and 3/8-inch Mixtures

Volumetric	1/2-inch NMAS	3/8-inch NMAS
PG Grade	70-28	70-28
P_b (%)	4.9	5.4
N_{design}	100	100
% G_{mm} @ $N_{initial}$	85.9	83.9
% V_a @ N_{design}	4.3	5.8
% VMA @ N_{design}	13.8	15.8
% VFA @ N_{design}	69	64
% G_{mm} @ N_{max}	97.1	96.2
Dust to Asphalt Ratio (D/A)	1.4	1.3
P_{be}	4.1	4.4
G_{mm}	2.483	2.466
G_{mb}	2.376	2.324
G_b	1.031	1.031
G_{se}	2.677	2.679
Hamburg Wheel-Tracking Test (mm)	2.5	3.0

Table B1.2 Air Voids of Field Cores

Pavement Section	Air Voids (%)	Standard Deviation

1/2-inch	6.3	0.97
3/8-inch	6.1	1.44

Figure B1.2 WMA over 300°F in Paver Screed

B1.2 Parameters Evaluated

Tests parameters evaluated for this project include studded tire wear resistance, dynamic modulus |E*|, creep compliance, intermediate and low temperature IDT strength, fracture work density, horizontal failure strain, Hamburg Wheel-Tracking Test, and asphalt content.

B1.3 Results and Discussion

B1.3.1 Studded Tire Wear Simulator

The numerical results of the studded tire wear simulator are shown in Table B1.3, and the results are displayed graphically in Figure B1.3. The error bars represent standard error. No statistically significant difference was found between the 3/8-inch and 1/2-inch field core samples.

Table B1.3 Studded Tire Wear Mass Loss

Mean Mass Loss (g)		p-value
1/2-inch	3/8-inch	
5.6	5.4	0.59

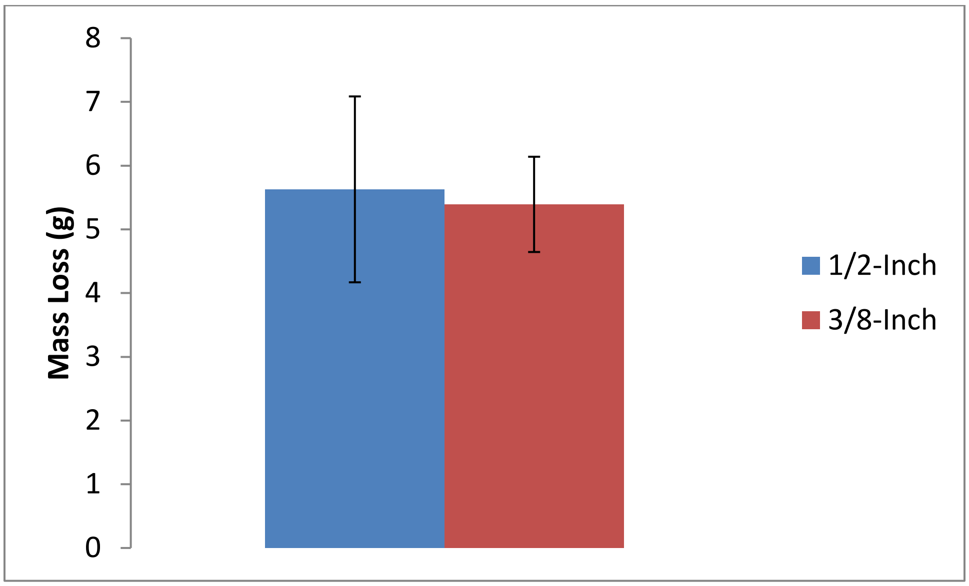

Figure B1.3 Studded Tire Wear

B1.3.2 Dynamic Modulus

The dynamic modulus master curves for the field cores are shown in Figure B1.4. The effect sizes of dynamic modulus at low, intermediate, and high levels of temperature-frequency combinations are shown in Table B1.4. The effect sizes indicate the two mixes have equal stiffness at high and low temperature levels, indicating they have similar stiffness. The 3/8-inch mix was statistically significantly stiffer at the intermediate temperature level. However, it is believed that this significance may have been caused by one outlying 3/8-inch mix sample (the 4.28% air void sample) being stiffer than the others.

Table B1.4 Dynamic Modulus Effect Sizes

Temperature-Frequency Level	Mean Dynamic Modulus (ksi)		Effect Size
	1/2-inch	3/8-inch	
Low	3,727	4,466	0.9
Intermediate	142	203	*1.8*
High	28	32	1.3

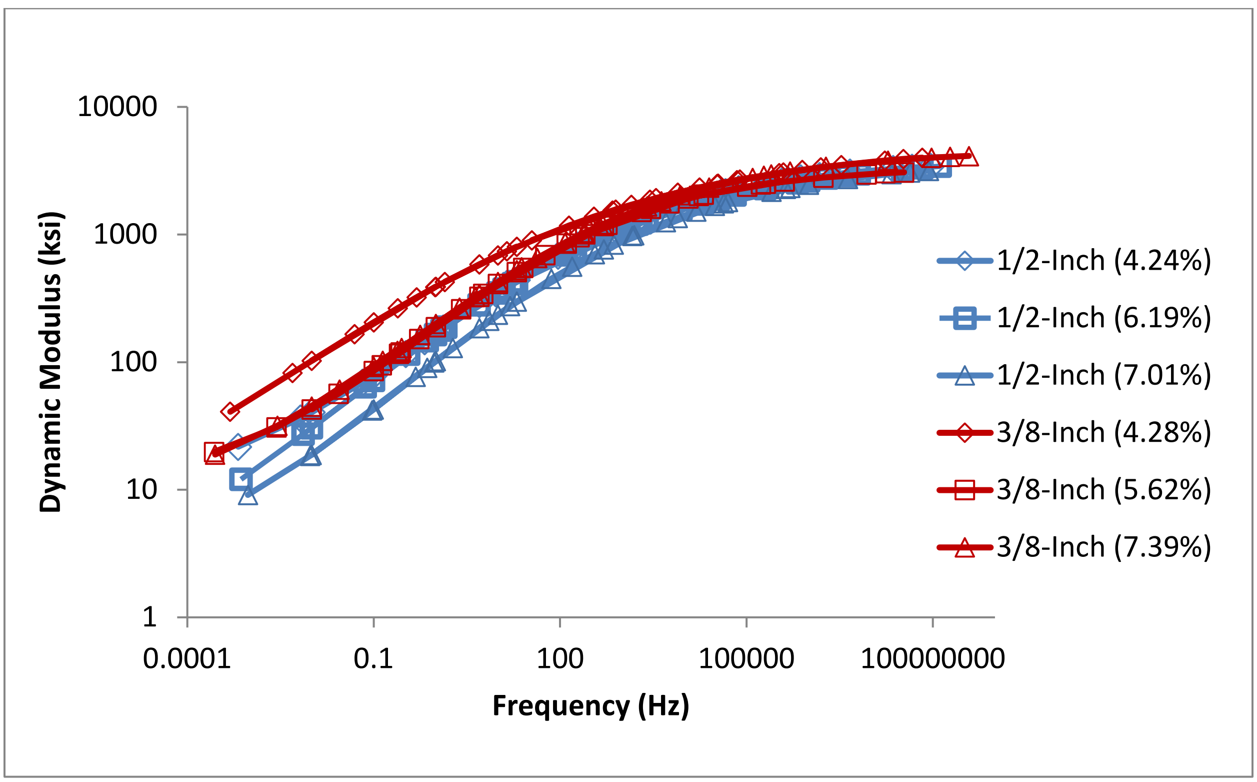

Figure B1.4 Dynamic Modulus Master Curves with Field Core Air Void Percentage

B1.3.3 Creep Compliance

Results from creep compliance tests are shown in Figure B1.5. The effect sizes at low, intermediate, and high time-temperature combination levels are shown in Table B1.5. The effect sizes indicate that the 3/8-inch mix is softer than the 1/2-inch mix at low and high time-temperature levels, probably due to the relatively higher asphalt content in the 3/8-inch mix.

Table B1.5 Creep Compliance Effect Sizes

Time-Temperature Level	Mean Creep Compliance (1/psi)		Effect Size
	1/2-inch	3/8-inch	
Low	5.76E-15	7.34E-15	*3.1*
Intermediate	2.31E-13	3.16E-13	1.3
High	1.75E-12	1.32E-12	*2.2*

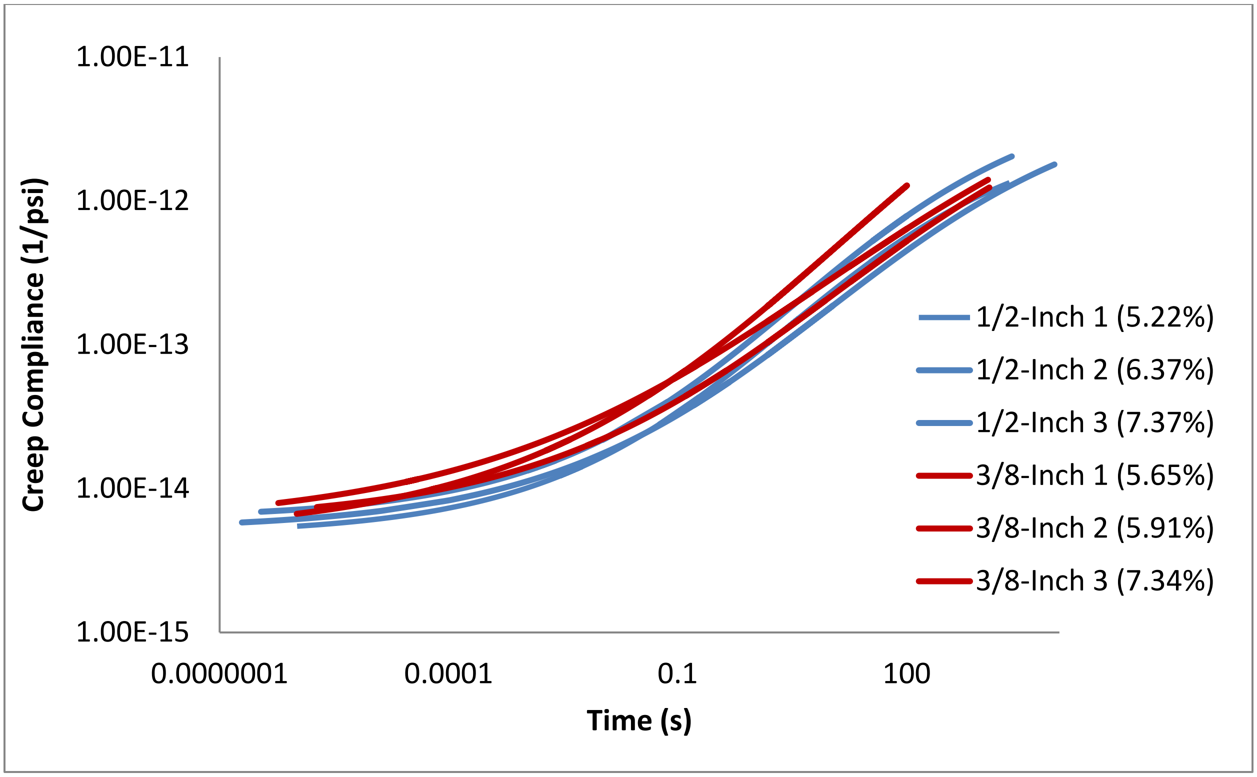

Figure B1.5 Creep Compliance Master Curves with Field Core Air Void Percentage

B1.3.4 IDT Fatigue Test

The fractured field core samples after the IDT fatigue test are shown in Figure B1.6. From visual inspection, the asphalt seems to have had good coating of the aggregate. The results for IDT strength, fracture work density, and horizontal failure strain of the 1/2-inch and 3/8-inch samples are shown in Figures B1.7 through B1.9. Results of the IDT fatigue tests are summarized in Table B1.6. The effect sizes for IDT strength and horizontal failure strain indicate no statistically significant difference in strength or top-down cracking resistance for the field cores. The results for fracture work density indicate the 3/8-inch mix is statistically significantly more resistant to bottom-up cracking.

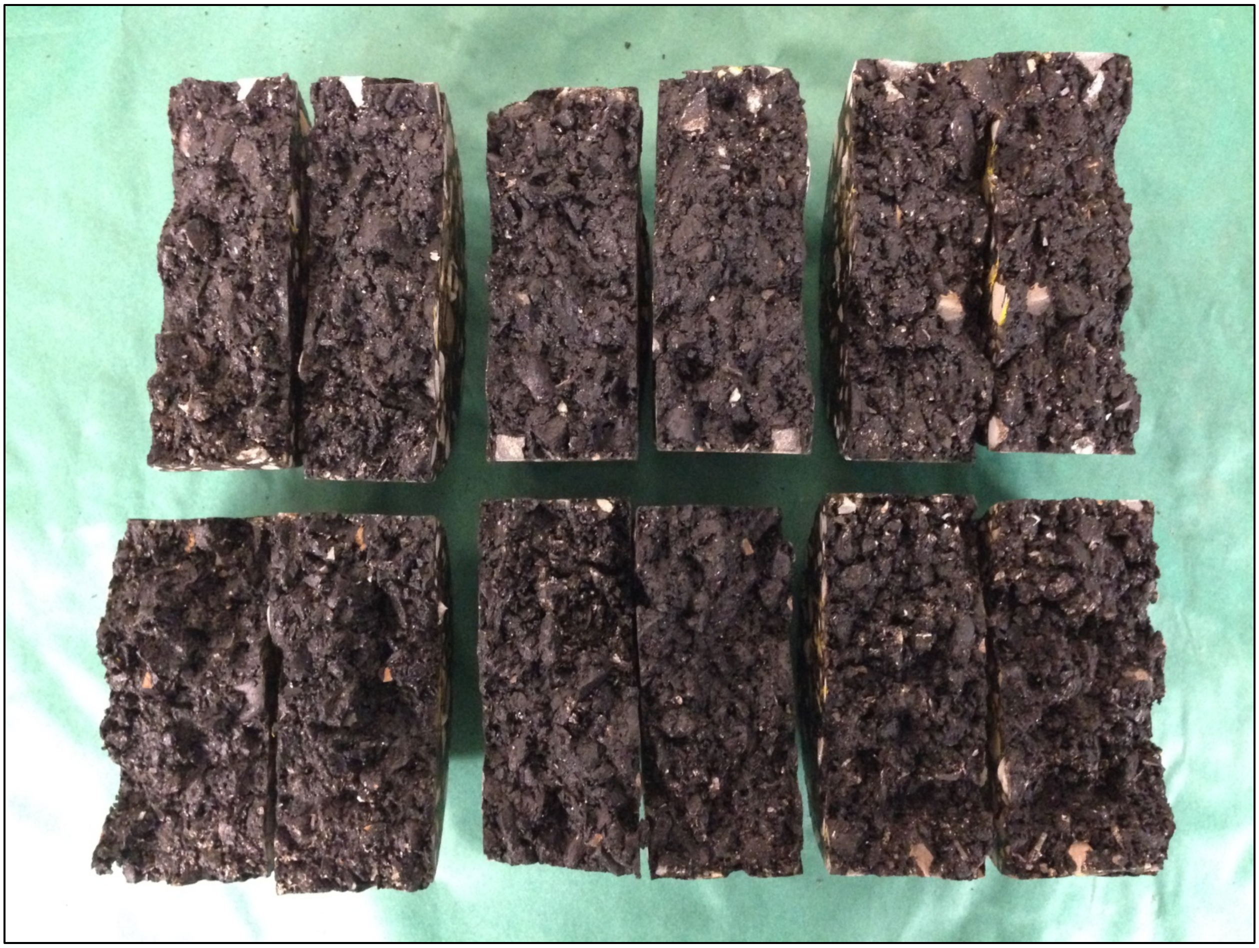

Figure B1.6 1/2-inch (Top Row) and 3/8-inch (Bottom Row) Mixes After IDT Fatigue Tests

Table B1.6 IDT Fatigue Test Results Summary

Parameter	Unit	Mean Value		Effect Size
		1/2-inch	3/8-inch	
IDT Strength	psi	236	248	0.8
Fracture Work Density	psi	15.2	18.0	*1.9*
Horizontal Failure Strain	in/in	0.008	0.007	0.9

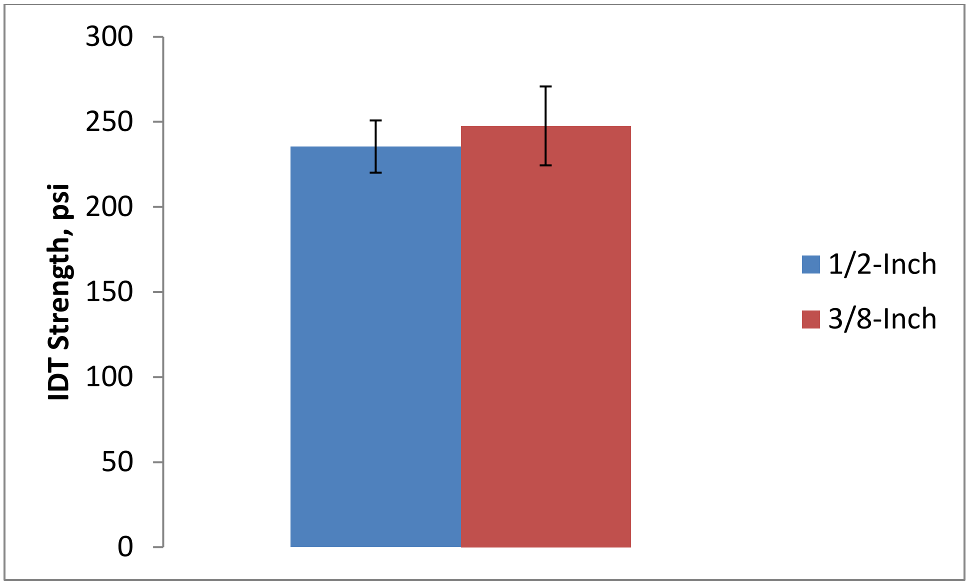

Figure B1.7 IDT Strength

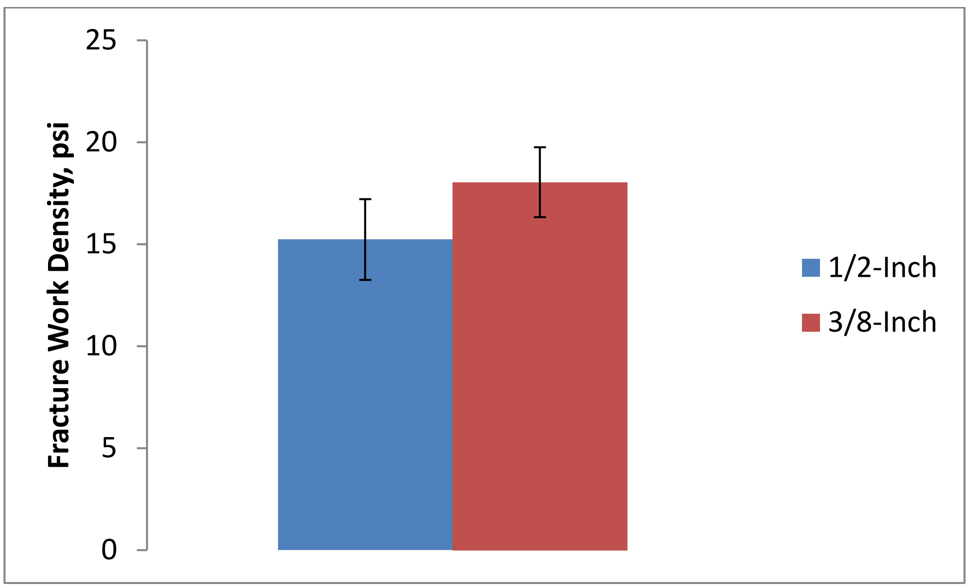

Figure B1.8 Fracture Work Density

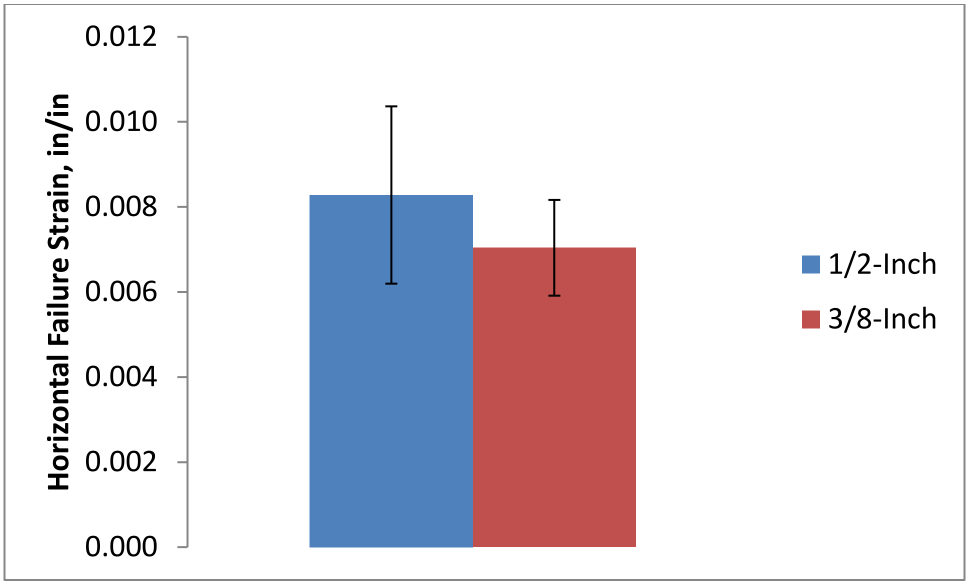

Figure B1.9 Horizontal Failure Strain

B1.3.5 IDT Thermal Cracking Test

The fractured field core samples after the IDT thermal cracking test are shown in Figure B1.10. Results of IDT thermal cracking tests are shown in Figures B1.11 through B1.12. Results of the IDT thermal cracking tests are summarized in Table B1.7. The 3/8-inch mix has statistically significantly higher IDT strength than the 1/2-inch mix and based on fracture work density, is more resistant to thermal cracking than the 1/2-inch mix. It is noted that broken aggregates are pronounced at the fracture plane.

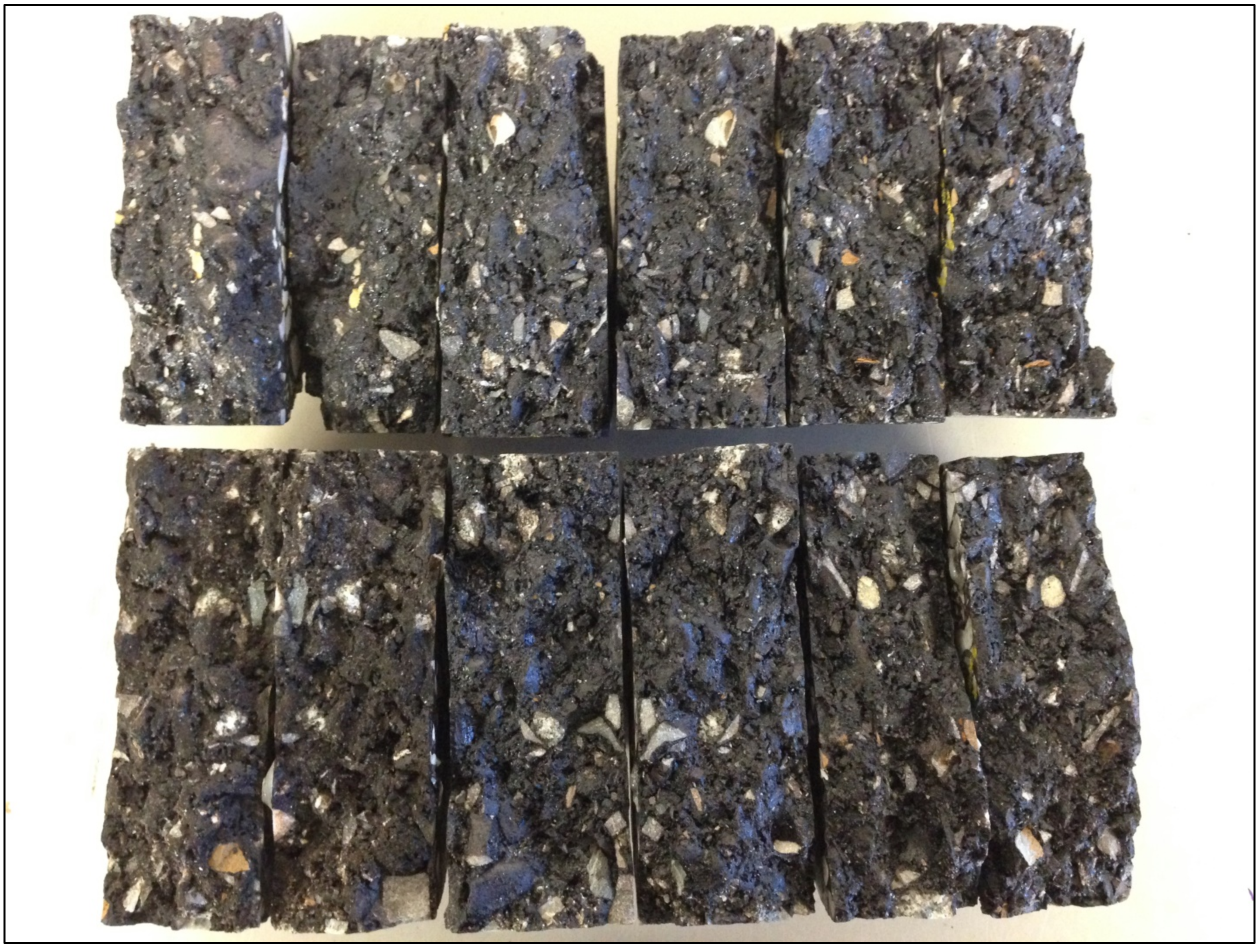

Figure B1.10 1/2-inch (Top Row) and 3/8-inch (Bottom Row) Mixes after IDT Thermal Cracking Tests

Table B1.7 IDT Thermal Cracking Test Results Summary

Parameter	Unit	Mean Value		Effect Size
		1/2-inch	3/8-inch	
IDT Strength	psi	578	672	*2.0*
Fracture Work Density	psi	15.2	18.0	*1.9*

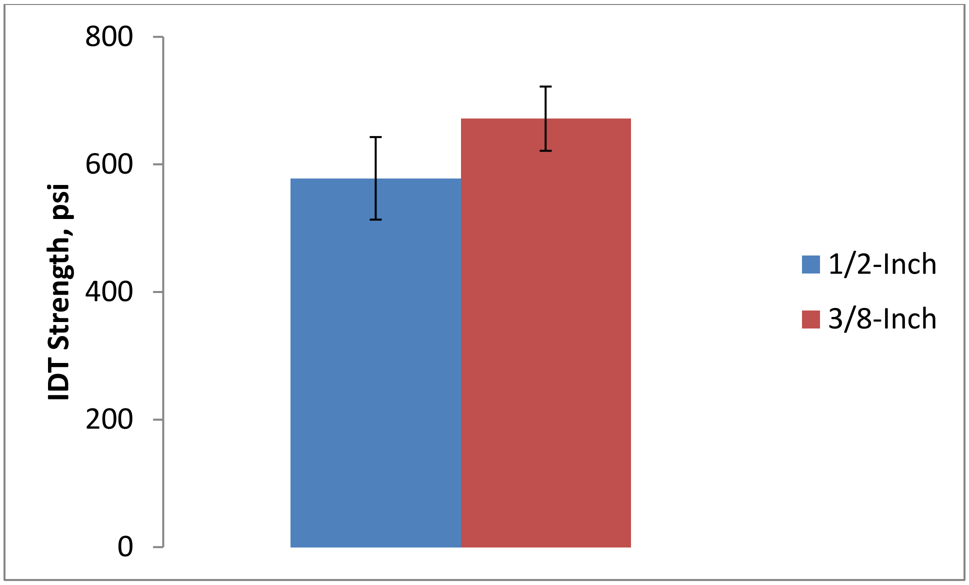

Figure B1.11 IDT Strength from Thermal Cracking Test

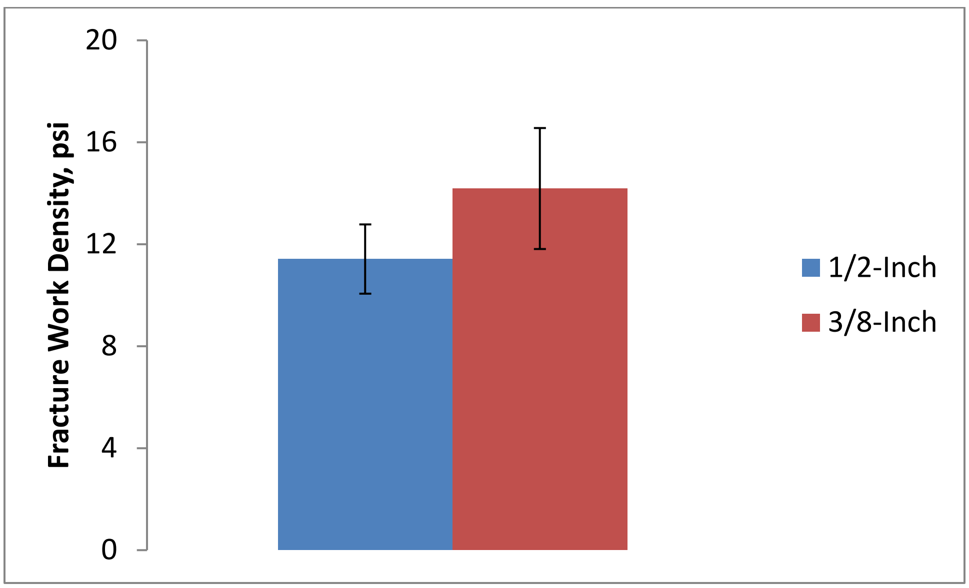

Figure B1.12 Fracture Work Density from Thermal Cracking Test

B1.3.6 Hamburg Wheel-Tracking Test

Results for the Hamburg Wheel-Tracking Test for the 1/2-inch and 3/8-inch mixes are shown in Figure B1.13. The 3/8-inch had nearly 1 mm more rutting than the 1/2-inch. However, neither mix was near the stripping inflection point at 20,000 passes, indicating that both are good mixes. No antistrip additives were used in the HWTT samples, which is consistent with the mixtures placed on Contract 8611.

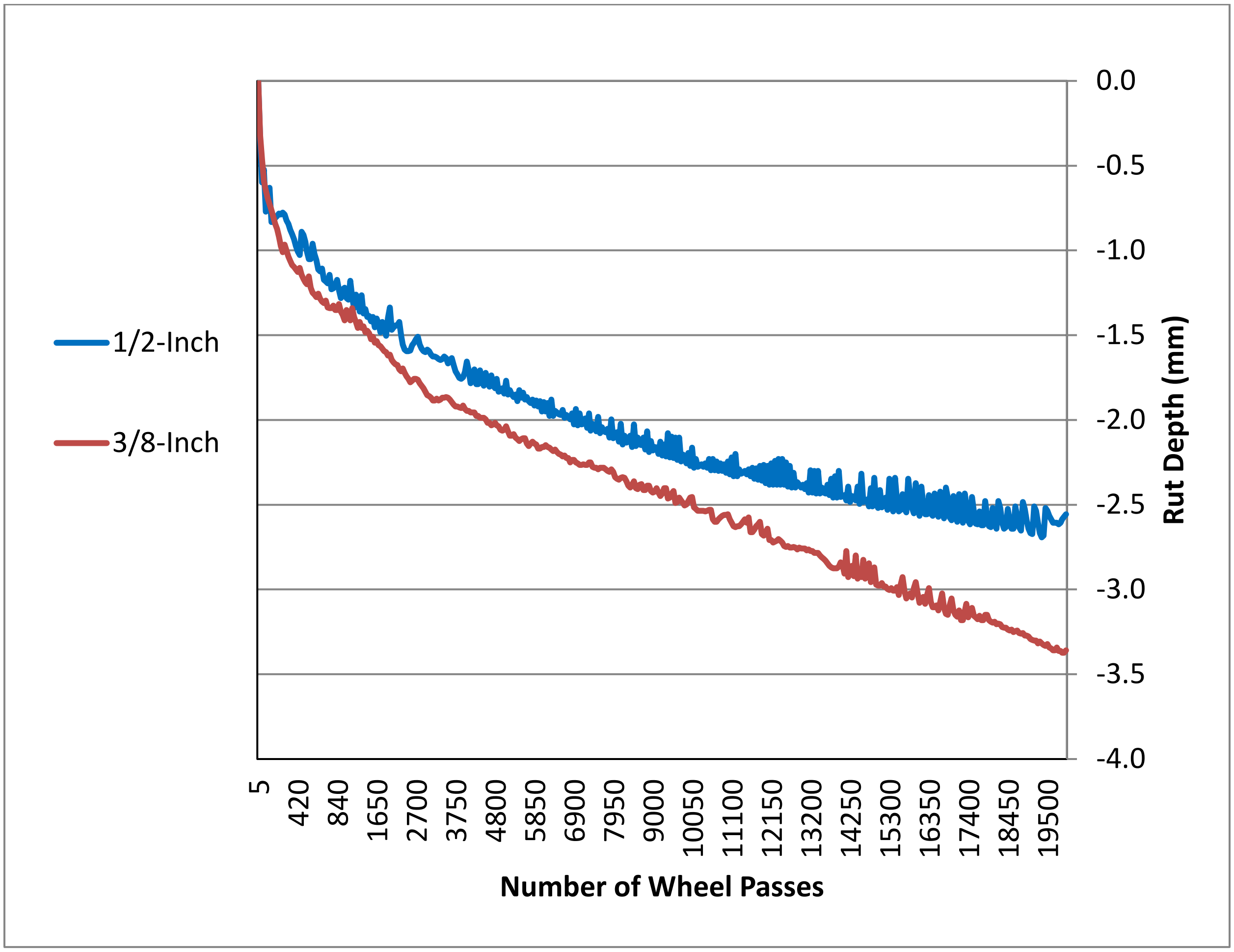

Figure B1.13 HWTT Results

B1.3.7 Asphalt Content

The asphalt content of the field cores and from the job mix formulas for the 1/2-inch and 3/8-inch sections are given in Table B1.8. The asphalt contents for the field cores were approximately 1/2% higher than those of the job mix formulas for both the 1/2-inch and 3/8-inch mixes. It is hypothesized that the contractor used more asphalt than the mix design for workability and compaction purposes.

Table B1.8 Asphalt Content

Mix	Pb (%)	
	1/2-inch Field Cores	3/8-inch Field Cores
Actual	5.49	5.99
JMF	4.9	5.4

B1.4 Conclusions

Overall, the performances of the 3/8-inch and 1/2-inch mixes are similar. Relatively, compared to the 1/2-inch mix, the 3/8-inch mix has similar studded tire resistance, equivalent strength, and equivalent top-down cracking resistance. The 3/8-inch mix showed better bottom-up fatigue and thermal cracking resistance. The two mixes have approximately the same stiffness.

APPENDIX B2: SMA AND INCREASED ASPHALT CONTENT PROJECT

B2.1 Project Description

Contract 6151 is located on I-90 from MP 208.16 to 218.6 in the Eastern Region. It was paved in 2001 and has an AADT of approximately 38,300. The right WB lane from MP 211.53 to 213.85 consists of 1/2-inch PG 76-28 SMA. This SMA section has performed remarkably well for over 13 years and is showing no need of rehabilitation. The left WB lane and the rest of the project consists of 1/2-inch PG 64-28 HMA.

Figure B2.1 shows WSPMS performance curves of the SMA in the WB lane from MP 212.93 to 213.43. The performance curves of the HMA from MP 214.05 to 215.23 are shown in Figure B2.2. It is noted that the MP locations recorded in WSPMS may not precisely reflect the actual MP locations in the field. It is also noted that the WSPMS performance curves are not likely calibrated for the SMA section and may not accurately reflect the field performance of the SMA section. Instead, the individual data points should be used to judge the performance.

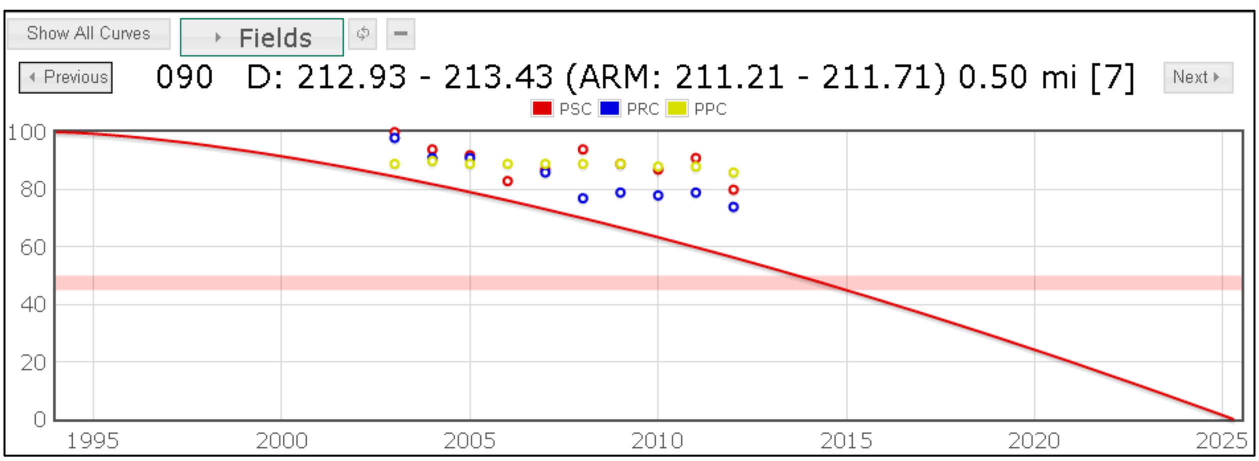

Figure B2.1 C6151 SMA Performance Curves

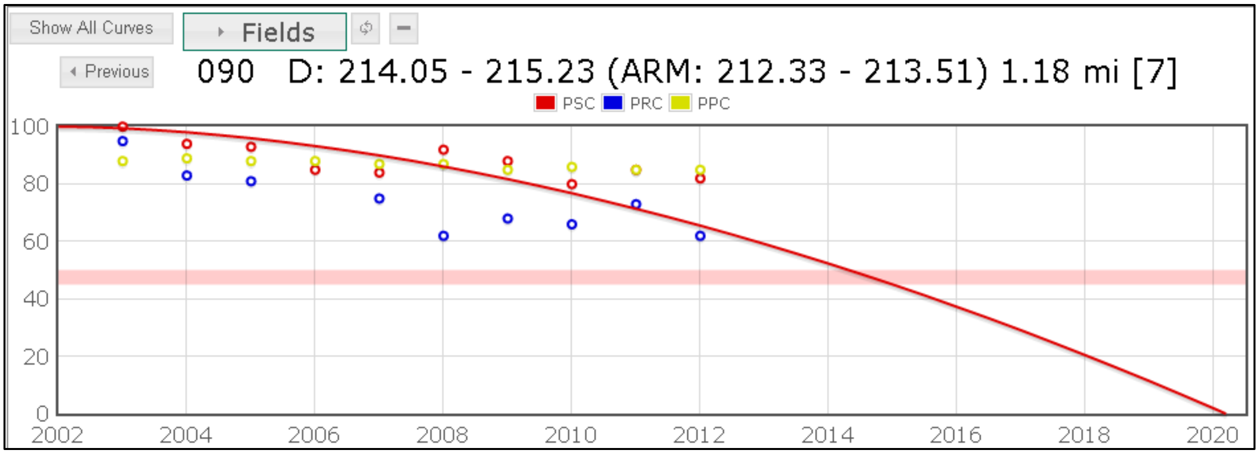

Figure B2.2 C6151 Performance Curves

Field cores were collected from I-90 by WSDOT in late 2014 and early 2015 in order to compare the performance of the PG 76-28 SMA and PG 64-28 HMA. The cores were taken from the center of the outside lane, outside the wheel path. The performance data from WSPMS is shown in Table B2.1 and indicates the SMA section is performing better than the HMA section. A field inspection verified that the SMA is visibly outperforming the HMA.

Table B2.1 Performance Data from WSPMS (2013 data)

Pavement Section	Rutting (in)	Rutting (PRC)	Cracking (PSC)
HMA	0.280	74	86
SMA	0.227	80	90

B2.2 Parameters Evaluated

Parameters evaluated for this project include studded tire wear resistance, intermediate and low temperature IDT strength, fracture work density, horizontal failure strain, asphalt content, binder PG gradation, and aggregate gradation.

B2.3 Results and Discussion

B2.3.1 Studded Tire Wear Simulator

The results of the studded tire simulator for the HMA and SMA sections are shown in Figure B2.3. The error bars represent standard error. For the statistical analysis of the results of the studded tire wear simulation, 11 replicates were available for testing for the 1/2-inch mix and 12 replicates were available for the 3/8-inch mix. Therefore, enough samples were available to perform a two-tailed t-test for analysis of variance. A significance level (p-value) of less than 0.05 indicated statistical significance. A summary of the results and statistical analysis are shown in Table B2.2. There was found to be no statistically significant difference between the two pavements for resistance to studded tire wear.

Table B2.2 Studded Tire Wear

Pavement Section	Mean Mass Loss (g)	Standard Deviation	p-value
HMA	2.7	1.46	0.73
SMA	3.3	0.75	

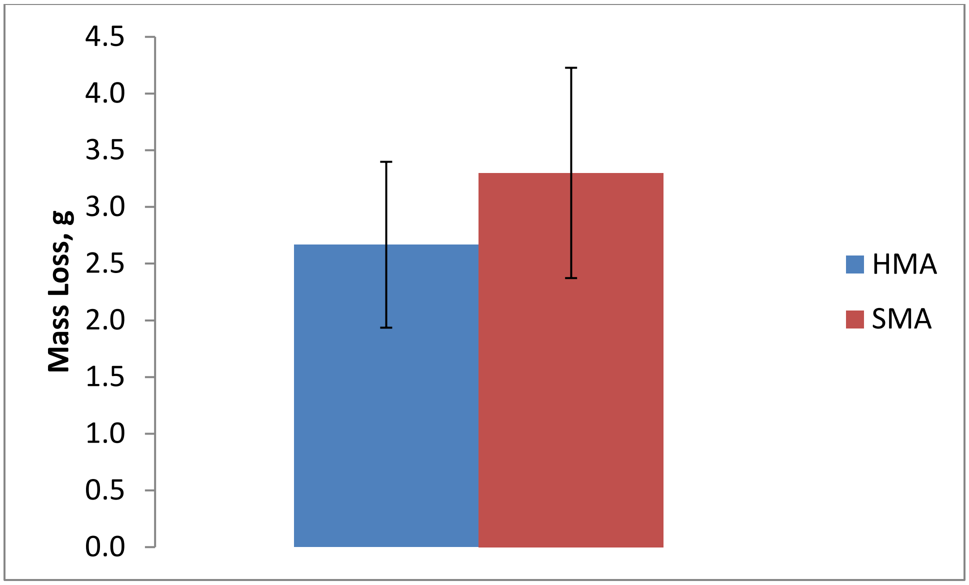

Figure B2.3 Studded Tire Wear

B2.3.2 IDT Fatigue Test

The fractured HMA and SMA samples after the IDT fatigue test are shown in Figures B2.4 and B2.5. From visual inspection, the samples often failed by the aggregate fracturing. Because the cores are taken between the wheel path, the fractured aggregates indicate that the pavements were over-compacted during construction, causing the aggregate to break. Graphical results for IDT strength, fracture work density, and horizontal failure strain of the HMA and SMA sections are shown in Figures B2.6 through B2.8. Results of the IDT fatigue tests are summarized in Table B2.3. Based on the fracture work density and the horizontal failure strain, the SMA is better than the HMA for bottom-up and top-down cracking resistance, respectively.

Figure B2.4 HMA Samples after IDT Fatigue Test

Figure B2.5 SMA Samples after IDT Fatigue Tests

Table B2.3 IDT Fatigue Test Results Summary

Parameter	Unit	Mean Values		Effect Size
		HMA	SMA	
IDT Strength	psi	434.0	374.4	*2.3*
Fracture Work Density	psi	21.6	32.0	*5.0*
Horizontal Failure Strain	in/in	0.0060	0.0096	*4.3*

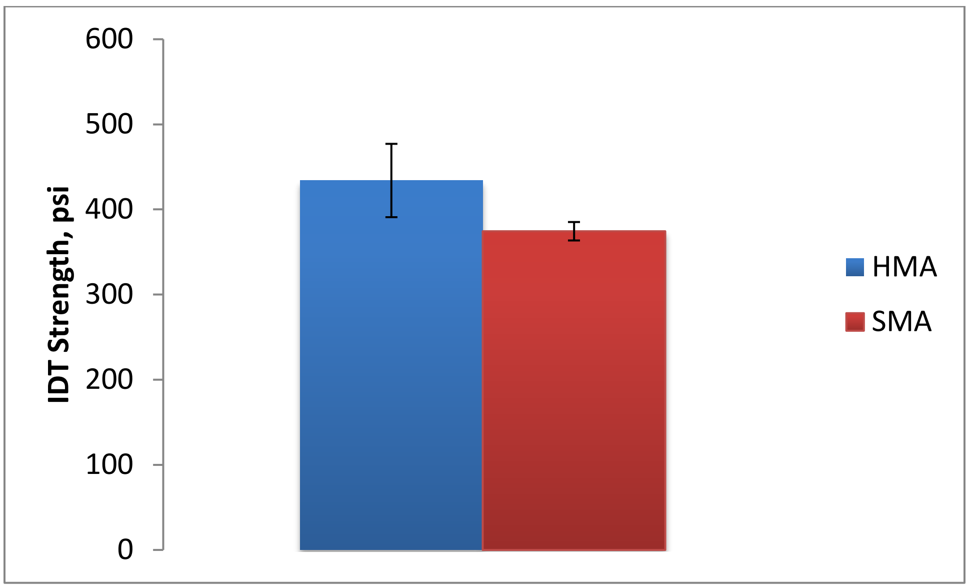

Figure B2.6 IDT Strength

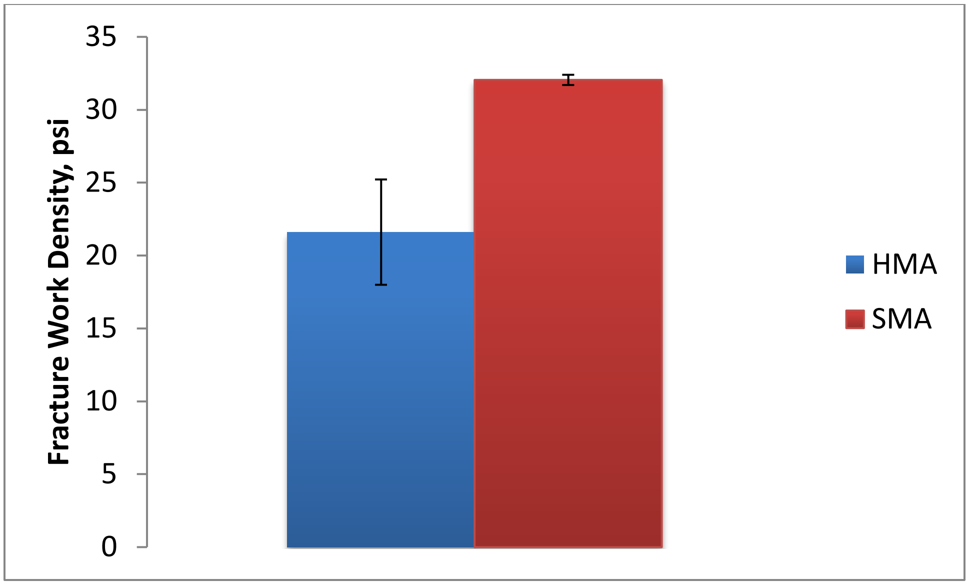

Figure B2.7 Fracture Work Density

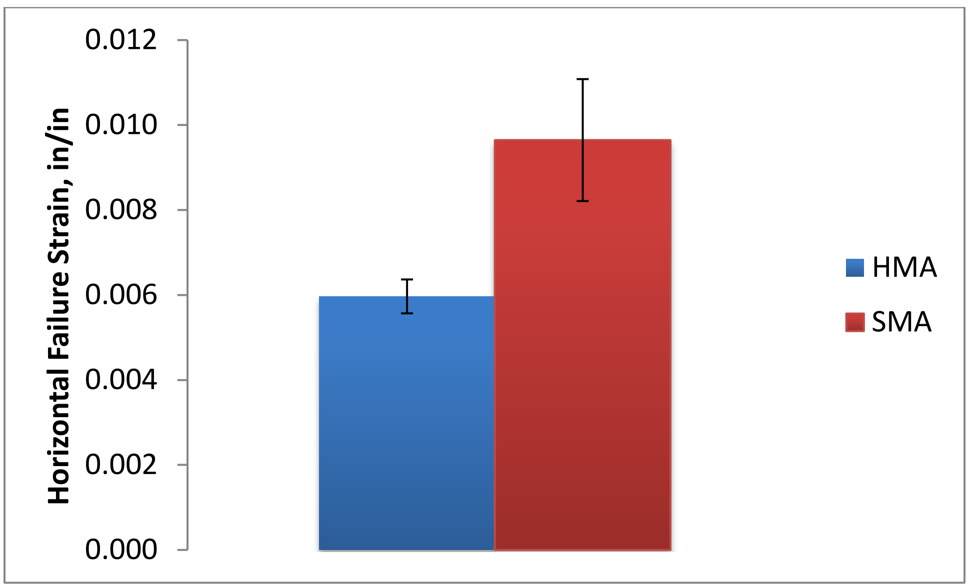

Figure B2.8 Horizontal Failure Strain

B2.3.3 IDT Thermal Cracking Test

The fractured HMA and SMA samples after the IDT thermal cracking test are shown in Figures B2.9 and B2.10. Results of IDT thermal cracking tests are shown in Figures B2.11 and B2.12. Results of the IDT thermal cracking tests are summarized in Table B2.4. There was no statistically significant difference between the HMA and SMA for IDT strength. Based on results of the IDT test performed at a low temperature, the SMA mix performed better than the HMA mix for thermal cracking resistance.

Figure B2.9 HMA Samples after IDT Thermal Cracking Tests

Figure B2.10 SMA Samples after IDT Thermal Cracking Tests

Table B2.4 IDT Thermal Cracking Test Results Summary

Parameter	Unit	Mean Values		Effect Size
		HMA	SMA	
IDT Strength	psi	647.6	637.8	0.3
Fracture Work Density	psi	11.9	17.4	*4.7*

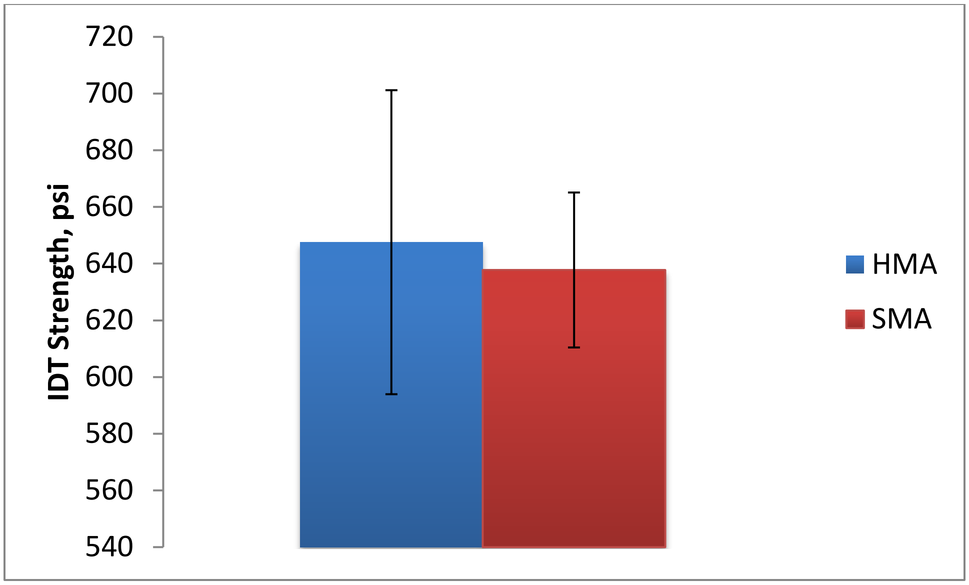

Figure B2.1147 IDT Strength from IDT Thermal Cracking Test

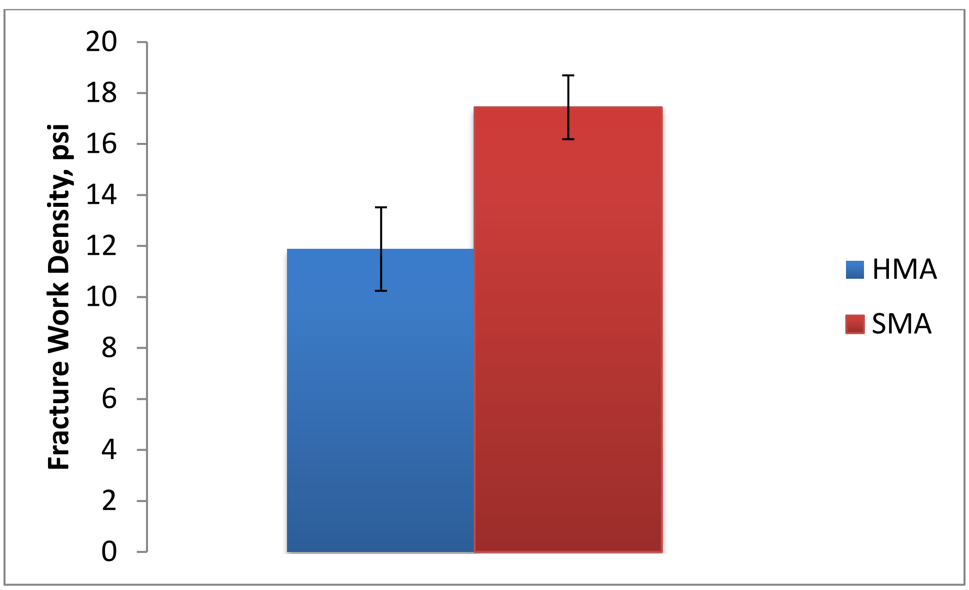

Figure B2.12 Fracture Work Density from IDT Thermal Cracking Test

B2.3.4 Asphalt Content

The asphalt content of the field cores of the HMA and SMA sections are given in Table B2.5. Both the HMA and SMA mixtures met the specification for asphalt content.

Table B2.5 Asphalt Content

Mix	Pb (%)	
	HMA	SMA
Field Core	5.6	6.8
JMF	5.44	6.8

B2.3.5 Binder PG Grading

After extracting the asphalt binder from the field cores, the true binder PG grades were determined for the HMA and SMA sections. The original PG grades of the SMA and HMA mixes were PG 76-28 and PG 64-28, respectively. Aging in the field resulted in an increase in PG grades for both mixes. The binder grades are shown in Figure B2.13 and are summarized in Table B2.6.

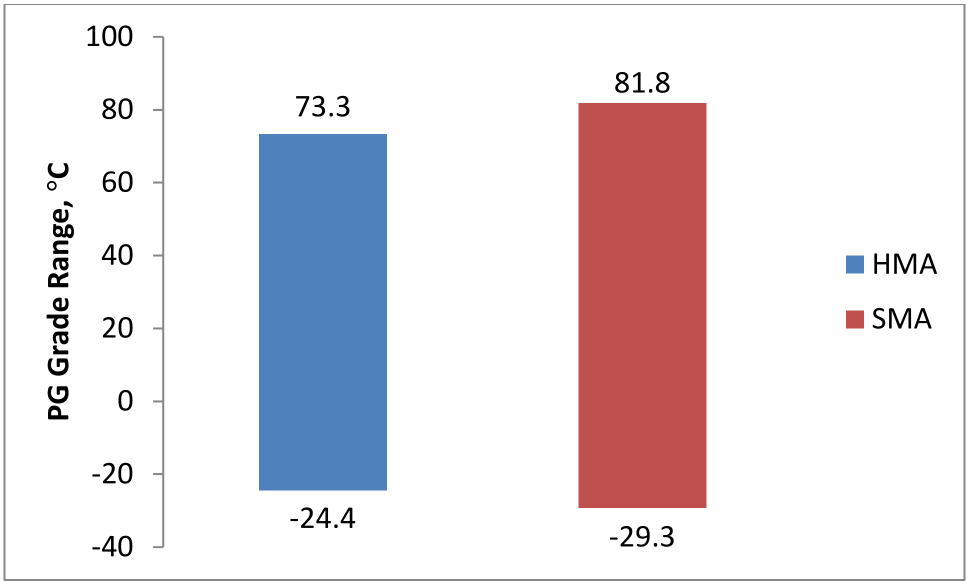

Figure B2.13 PG Grade Ranges

Table B2.6 Binder PG Grades

Pavement Section	Temperature Level	Design PG Grade (°C)	True PG Grade (°C)
HMA	High	64	73.3
	Low	-28	-24.4
SMA	High	76	81.8
	Low	-28	-29.3

B2.3.6 Gradation

A sieve analysis of the aggregate was performed after the binder was extracted. The gradations of the two mixtures are shown in Figure B2.14. It is noted that the fiber from the SMA mixture was removed from the aggregate prior to performing the sieve analysis.

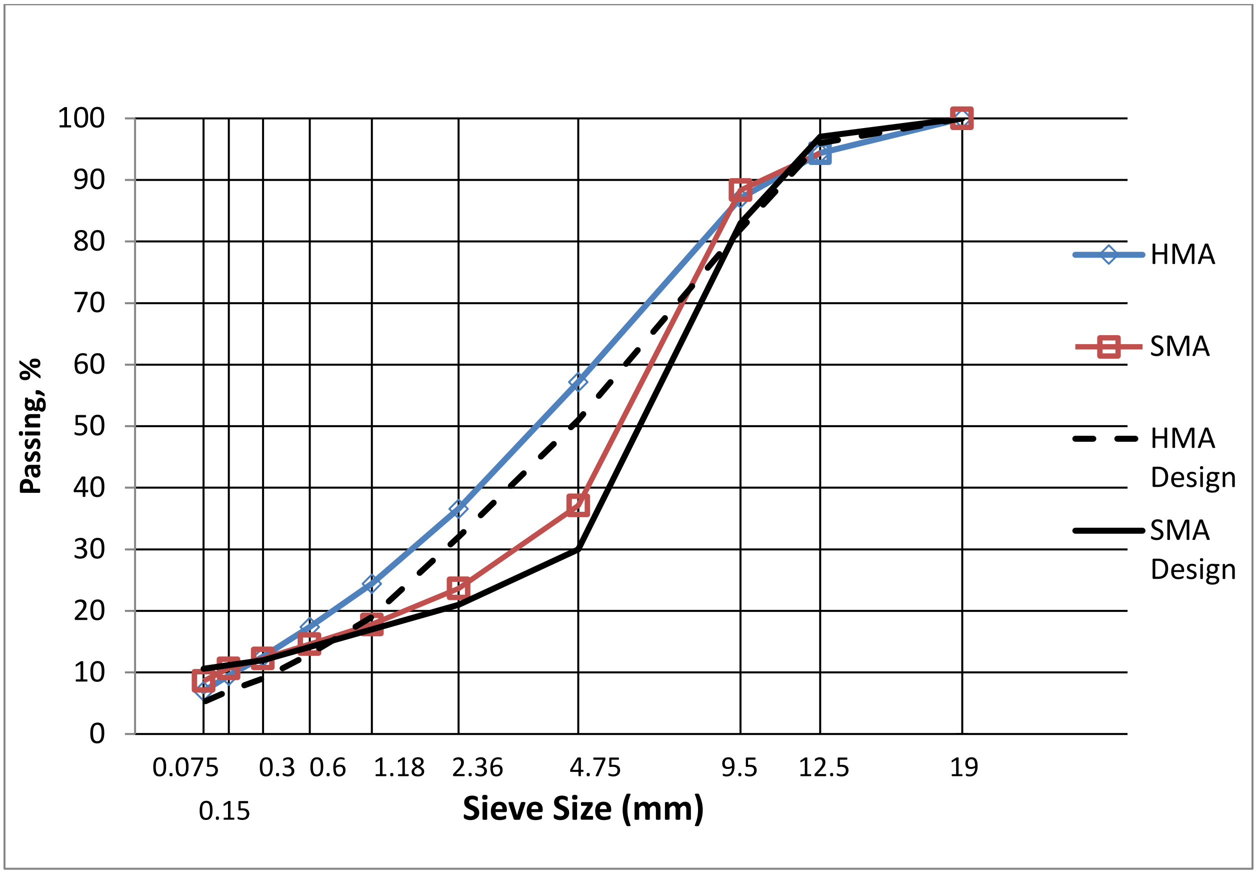

Figure B2.14 Gradations of Field Cores

B2.4 Conclusions

Laboratory testing results indicate the SMA section has superior performance over the HMA section for top-down, bottom-up, and thermal cracking resistance. This performance is consistent with the field performance for Contract 6151. The SMA has performed remarkably well for over 13 years; it is visibly out-performing the adjacent HMA section and may last as long as 20 years.

APPENDIX B3: BST OVERLAY PROJECT

B3.1 Project Description

The effects of BST on the underlying HMA were evaluated for two projects in this study: Contract 7109 on SR 20, which was paved in 2006, and Contract 8262 on SR 278, which was paved in 2012.

Contract 7109 is located on SR 20 from MP 404.41 to MP 422.92 and included a pre-level and BST overlay in 2006. The HMA for pre-level was 3/8-inch PG 64-28 and was paved 0.1 feet thick. The BST layer was Class D CRS-2P. The surface was fog sealed. At approximately MP 417.17, an HMA approach was not chip sealed and can serve as a control. It is noted that WSPMS does not indicate an underlying HMA layer for the Contract 7109. The performance curves from WSPMS for a section of Contract 7109 is shown in Figure B3.1.

Contract 8262 on SR 278 from MP 0.00 to 5.50 was paved and overlaid with BST in 2012. The pavement was ACP Class 3/8-inch PG 64-28 and was a grind and inlay of 0.15 ft. depth. The BST had CRS-2P asphalt emulsion. No performance data from WSPMS was available yet for Contract 8262, due to its recent construction.

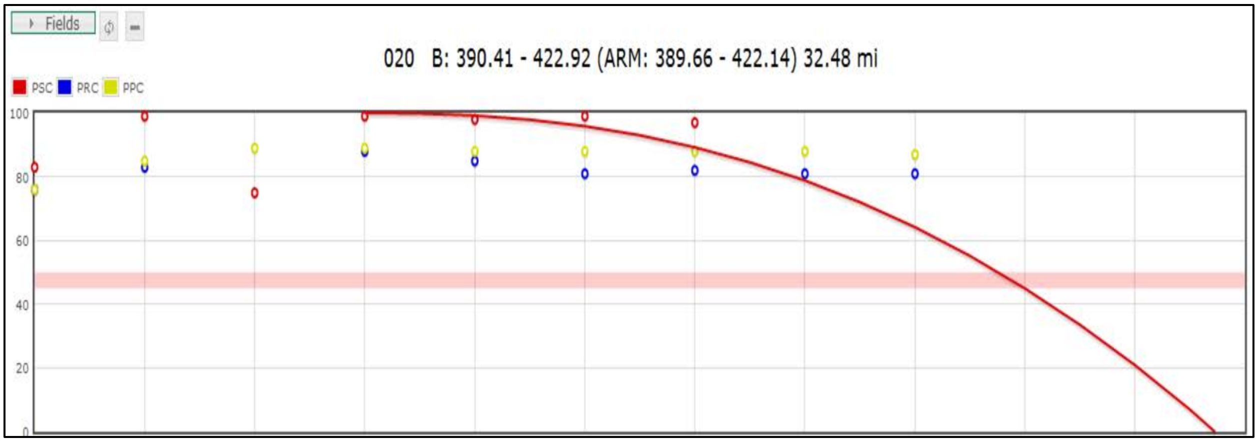

Figure B3.1 C7109 (SR 20) Performance Curves

Field cores were collected from SR 20 and SR 278 in August, 2014. The cores were taken from the center of the outside lane of the westbound lane. For both the SR 20 and SR 278 projects, three cores were taken from the section with HMA overlaid with BST, and three were taken from the HMA section that was not overlaid with BST. When preparing the HMA with BST core samples to be tested, the entire BST layer was removed, so that only the HMA portion of the core was tested. It is noted that the SR 20 project was approximately 6 years older than the SR 278 project; therefore, the effects of the chip seal overlay were likely to be more pronounced for the SR 20 project. The averaged air void percentages of the pavement cores for the two projects are shown in Table B3.1. The SR 20 HMA without BST has higher air void level than the HMA with BST. This is likely due to the fact that the approach from which the cores were taken for the HMA without BST was further away from the wheel path than the cores of the HMA with BST and, therefore, received less densification from traffic.

Table B3.1 Air Voids of Field Cores

Project	Pavement Section	Air Voids (%)
SR 20	HMA w/o BST	5.6
	HMA w/ BST	3.9
SR 278	HMA w/o BST	4.0
	HMA w/ BST	3.9

B3.2 Parameters Evaluated

For the SR 20 and SR 278 projects, the following parameters were evaluated: dynamic modulus |E*|, creep compliance, IDT strength at intermediate temperatures, fracture work density, horizontal failure strain, and binder PG grading.

B3.3 Results and Discussion

B3.3.1 Dynamic Modulus

The dynamic modulus master curves of the HMA with BST and the HMA without BST from SR 20 and SR 278 are shown in Figure B3.2 and Figure B3.3, respectively. A summary of dynamic modulus values at low, intermediate, and high levels of temperature-frequency combinations are shown in Table B3.2. The effect sizes indicate that the HMA with BST is significantly softer than the HMA without BST at all temperature-frequency levels for both the SR 20 and SR 278 projects.

Table B3.2 Dynamic Modulus Effect Sizes

Project	Temperature-Frequency Level	Mean Dynamic Modulus (ksi)		Effect Size
		HMA w/o BST	HMA w/ BST	
SR 20	Low	3,567	3,142	1.9
	Intermediate	607	296	7.3
	High	80	44	3.7
SR 278	Low	4,426	3,845	2.3
	Intermediate	538	351	4.5
	High	56	19	5.5

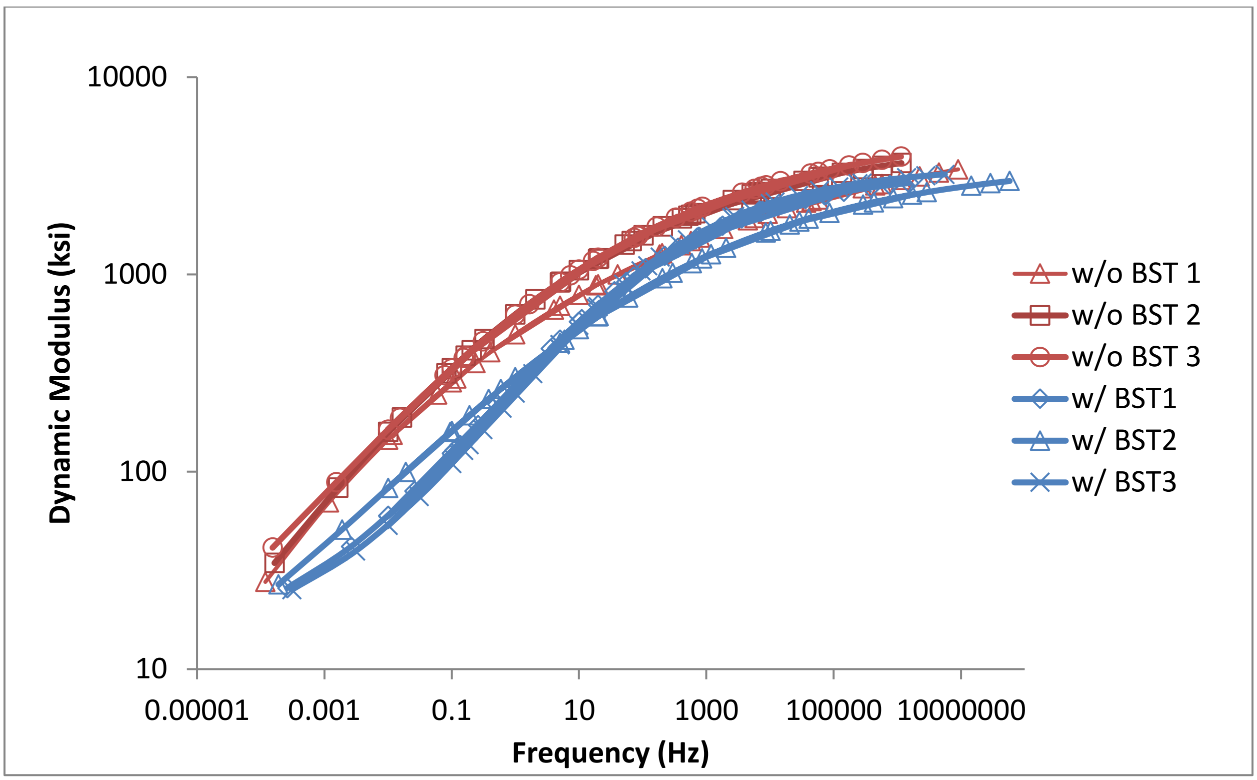

Figure B3.2 SR 20 Dynamic Modulus Master Curves

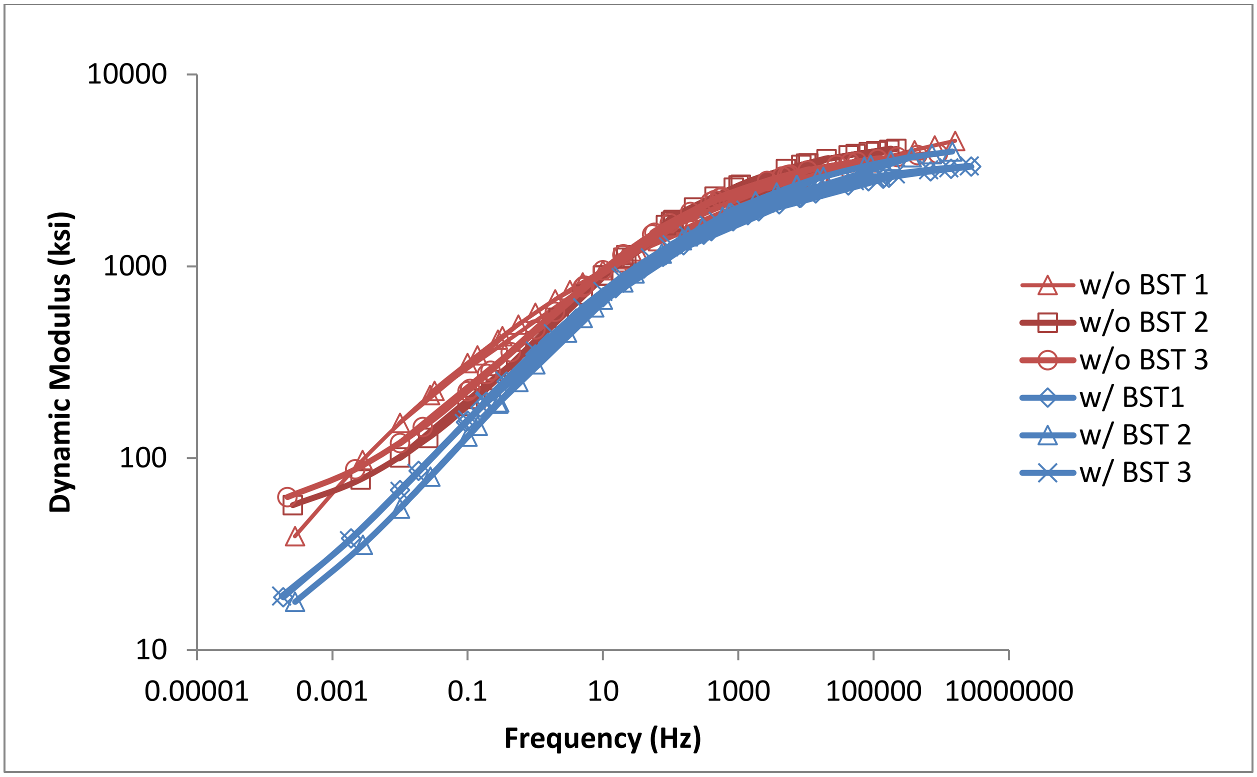

Figure B3.3 SR 278 Dynamic Modulus Master Curves

B3.3.2 Creep Compliance

The creep compliance master curves of the HMA with BST and the HMA without BST from SR 20 and SR 278 are shown in Figure B3.4 and Figure B3.5, respectively. A summary of the creep compliance values at low, intermediate, and high time-temperature combination levels are shown in Table B3.3. The effect sizes indicate the HMA with BST is significantly softer than the HMA without BST at intermediate and high time-temperature combination levels for both the SR 20 and SR 278 projects.

Table B3.3 Creep Compliance Effect Sizes

Project	Time-Temperature Level	Mean Creep Compliance (1/psi)		Effect Size
		HMA w/o BST	HMA w/ BST	
SR 20	Low	6.66E-15	7.30E-15	0.8
	Intermediate	1.02E-13	2.20E-13	5.5
	High	9.07E-13	1.43E-12	3.4
SR 278	Low	5.92E-15	6.37E-15	1.1
	Intermediate	1.64E-13	2.18E-13	1.9
	High	9.92E-13	1.22E-12	2.5

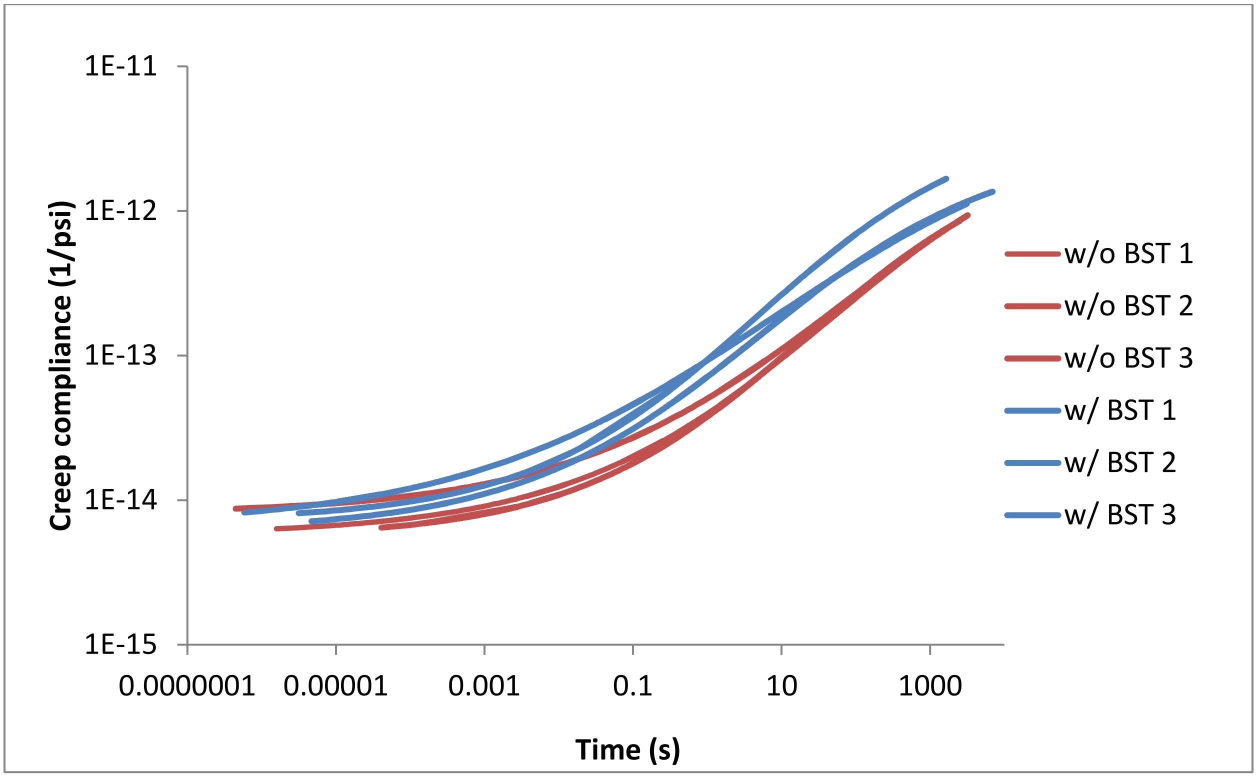

Figure B3.4 SR 20 Creep Compliance Master Curves

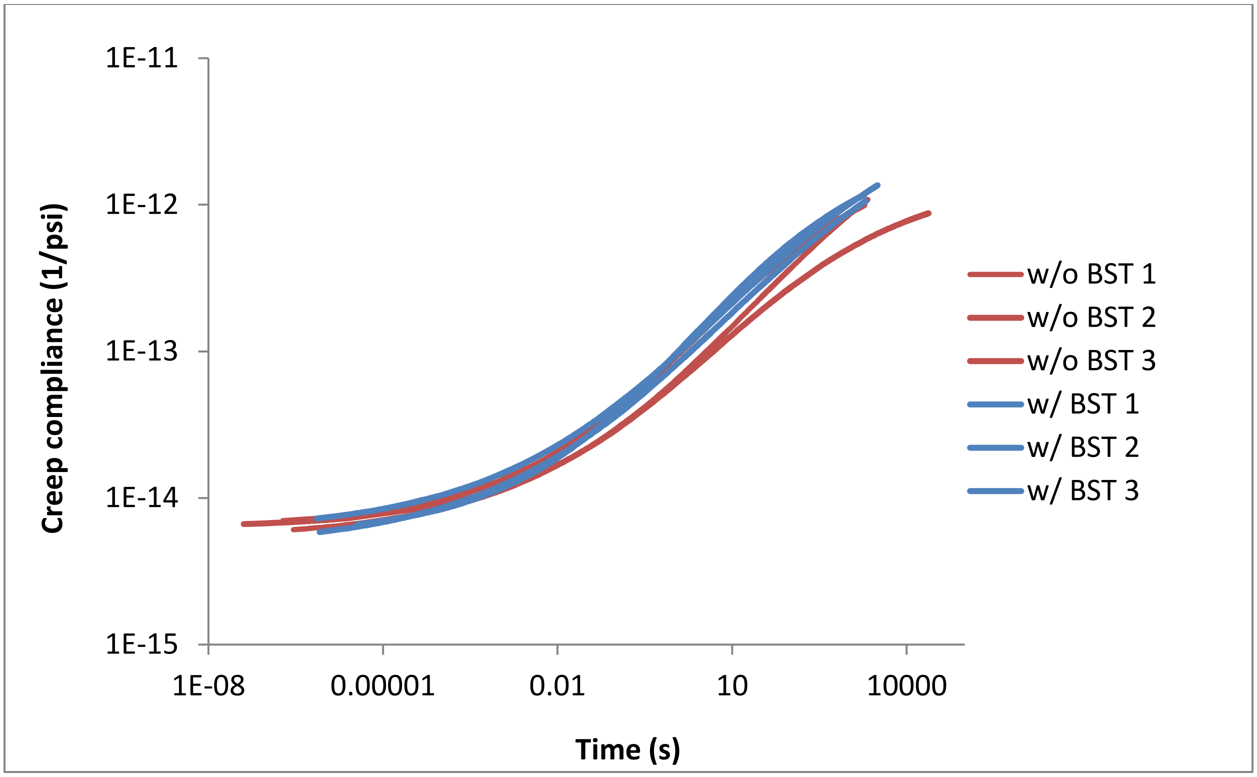

Figure B3.5 SR 278 Creep Compliance Master Curves

B3.3.3 IDT Fatigue Test

Graphical results of parameters evaluated from the IDT fatigue tests are shown in Figures B3.6 through B3.11. Results of the IDT fatigue tests are summarized in Table B3.4. The effect sizes indicate the HMA without BST is significantly stronger and more resistant to top-down cracking than the HMA with BST from the SR 20 project, but there is no significant difference in strength or top-down cracking resistance between the HMA with BST and HMA without BST from the SR 278 project, likely due to the recent construction of SR 278. The BST overlay did not significantly affect bottom-up cracking resistance for either project.

Table B3.4 IDT Fatigue Test Results Summary

Project	Parameter	Unit	Mean Value		Effect Size
			HMA w/o BST	HMA w/ BST	
SR 20	IDT Strength	psi	278	182	*12*
	Fracture Work Density	psi	33.3	40.7	0.9
	Horizontal Failure Strain	in/in	0.0027	0.0042	*10*
SR 278	IDT Strength	psi	301	303	0.1
	Fracture Work Density	psi	41.6	41.1	0.2
	Horizontal Failure Strain	in/in	0.0071	0.0078	1.1

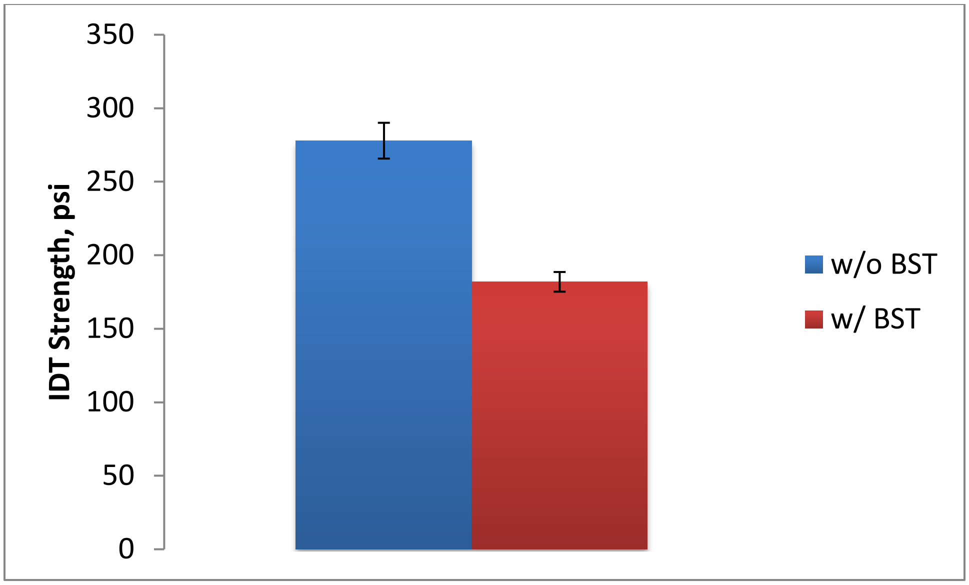

Figure B3.6 SR 20 IDT Strength

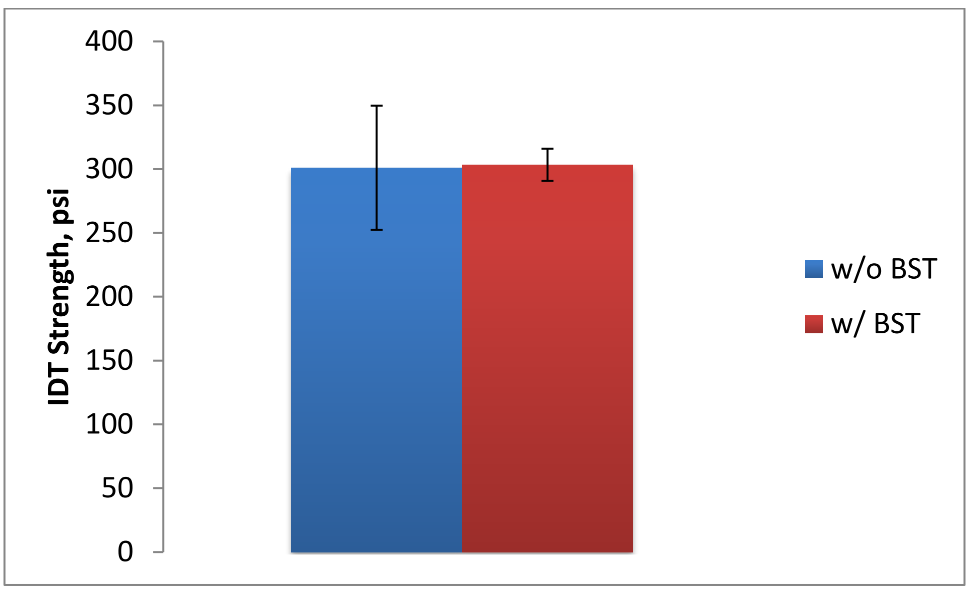

Figure B3.7 SR 278 IDT Strength

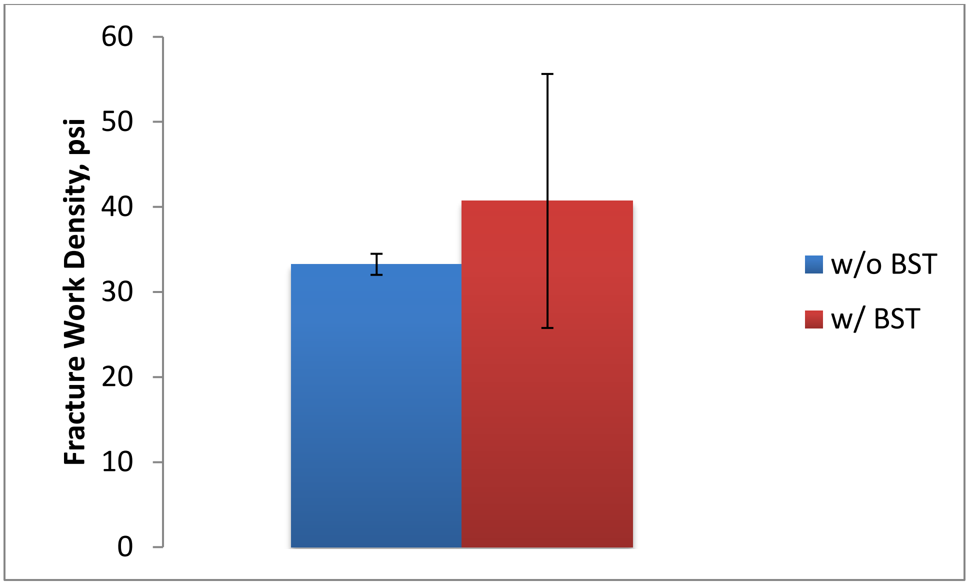

Figure B3.8 SR 20 Fracture Work Density

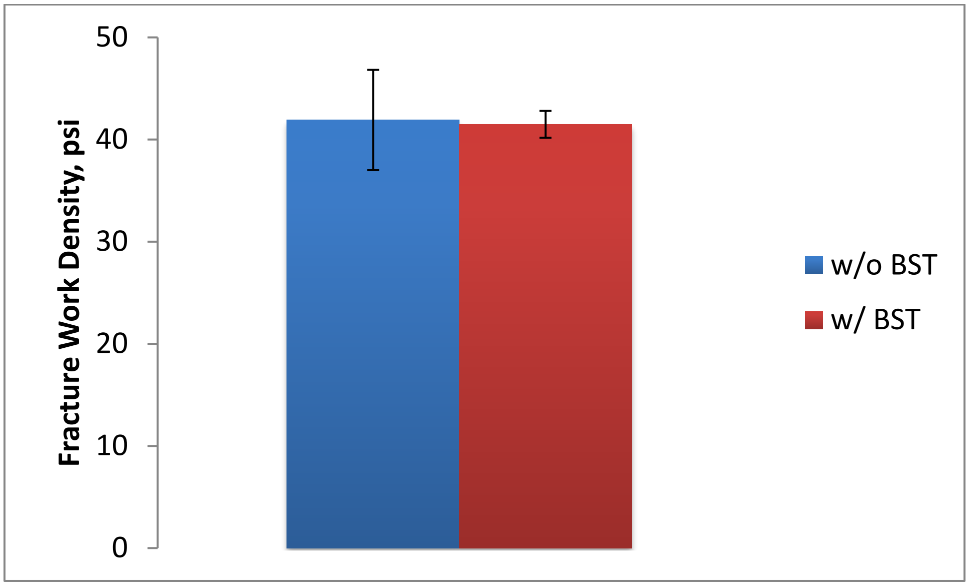

Figure B3.9 SR 278 Fracture Work Density

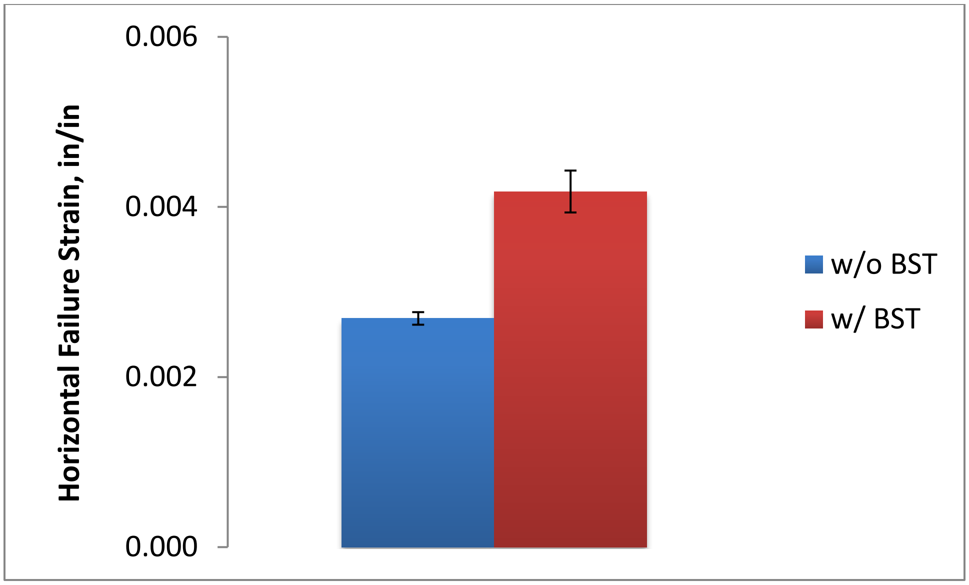

Figure B3.10 SR 20 Horizontal Failure Strain

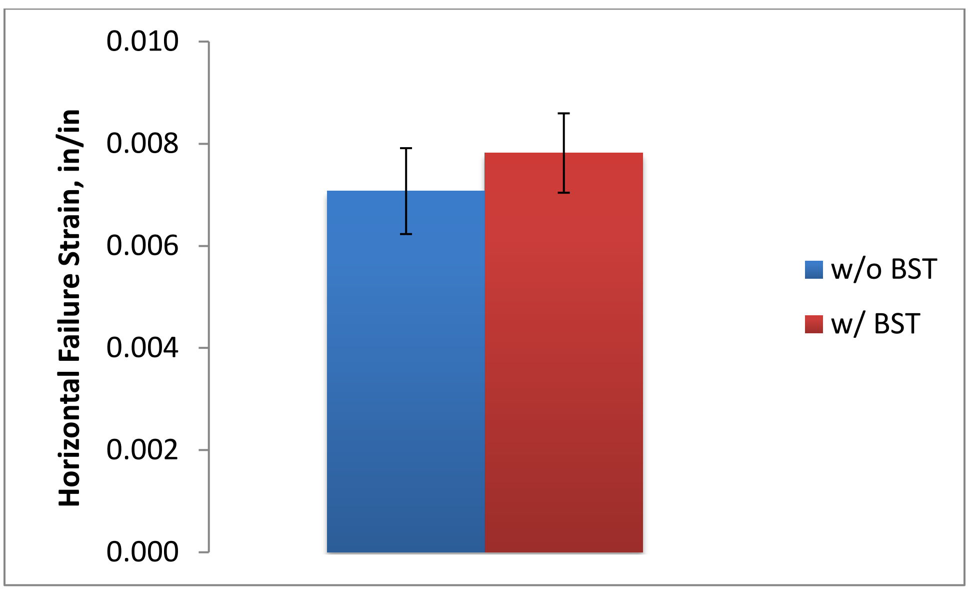

Figure B3.48 SR 278 Horizontal Failure Strain

B3.3.4 Binder PG Grading

For this project, the PG grades were determined at increasing depths at increments of 1 inch, as shown. PG grades at three layer depths were evaluated for SR 278, as shown in Figure B3.12, but only two layers were available to be tested for SR 20 due to the depth of the cores. The results of high and low PG grades of the layers of HMA with BST and HMA without BST from SR 20 and SR 278 are shown in Figures B3.13 and B3.14, respectively. A summary of the PG grade results is shown in Table B3.5. For the SR 20 project, the differences in PG grades between the HMA with BST and the HMA without BST were ≥ 6 degrees, indicating that the BST overlay had significantly reduced the aging of the binder in the underlying HMA. For the SR 278 project, the differences in PG grades between the HMA with BST and the HMA without BST were not ≥ 6 degrees, probably due to the recent construction, but the BST overlay did slightly reduce the aging of the underlying asphalt.

Table B3.5 Binder PG Grades

Project	Temperature Level	Field Core Layer	True PG Grade (°C) HMA w/o BST	True PG Grade (°C) HMA w/ BST	PG Grade Difference (°C)
SR 20	High	1	75.6	67.1	*8.5*
		2	75.2	67.3	*7.9*
	Low	1	-20.1	-28.0	*7.9*
		2	-20.3	-28.2	*7.9*
SR 278	High	1	75.9	72.0	3.9
		2	74.2	72.0	2.2
		3	73.8	70.3	3.5
	Low	1	-24.1	-26.5	2.4
		2	-24.7	-26.8	2.1
		3	-26.0	-29.0	3.0

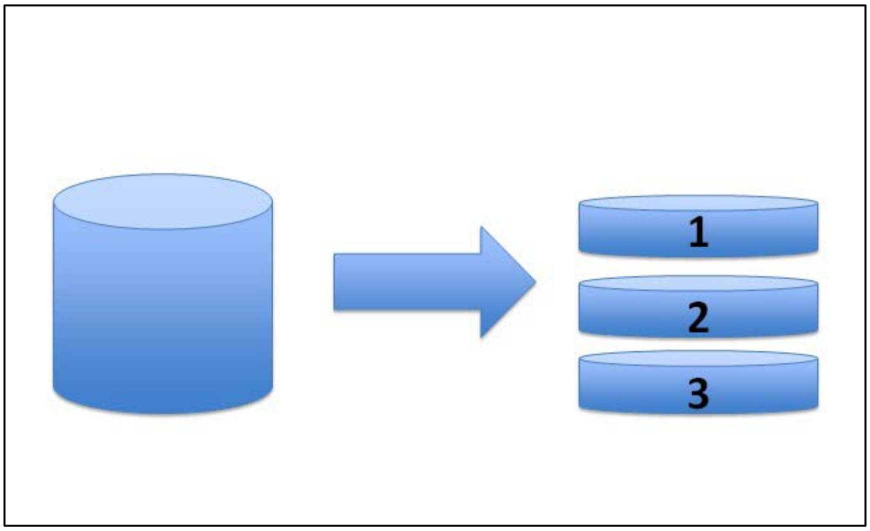

Figure B3.12 Field Core Layers for PG Grade Determination

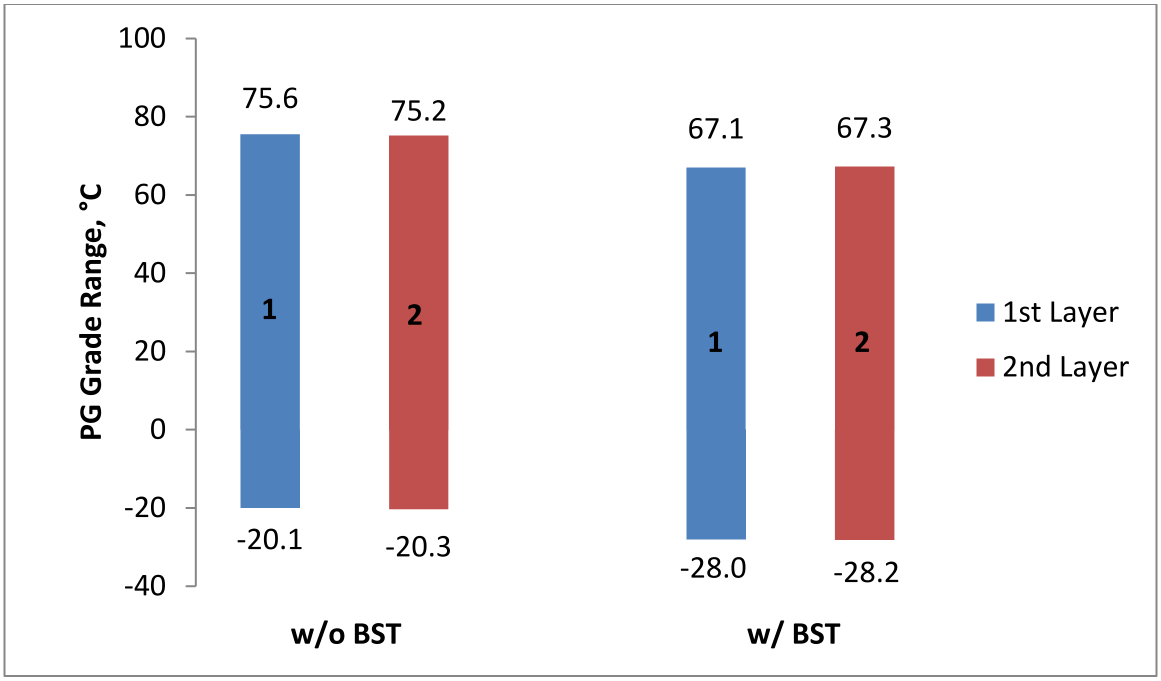

Figure B3.13 SR 20 High and Low PG Grades

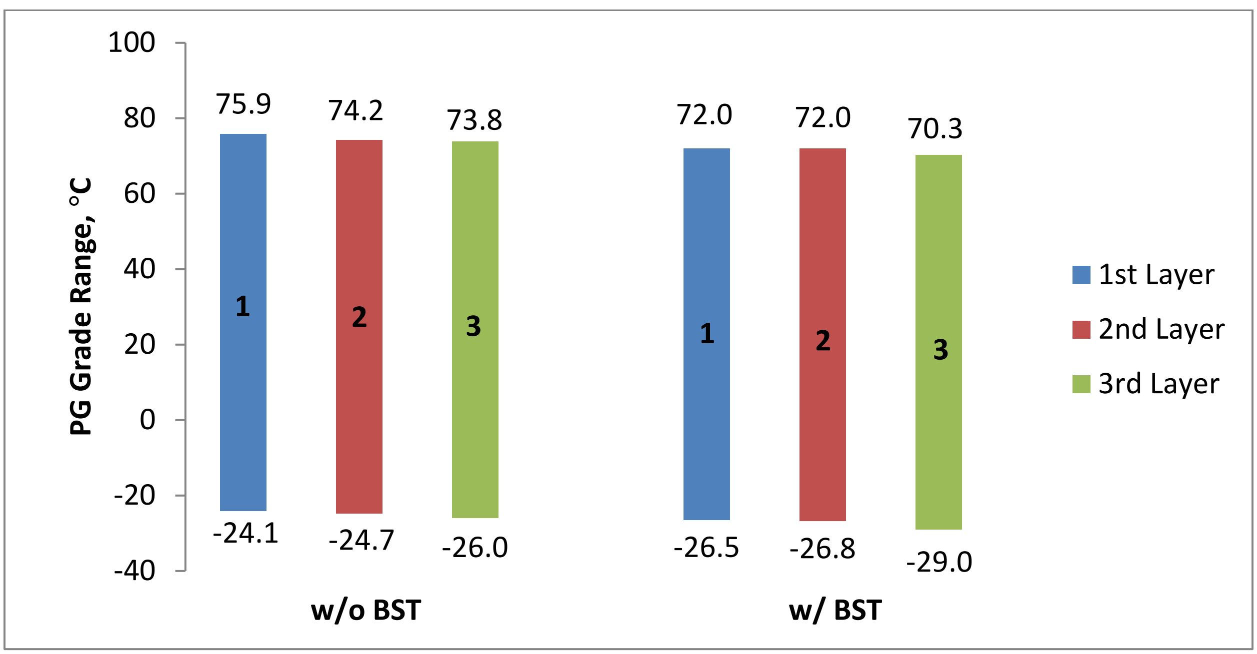

Figure B3.14 SR 278 High and Low PG Grades

B3.4 Conclusions

The results of the dynamic modulus and creep compliance tests indicate that the BST overlay kept the underlying HMA softer than the HMA that was exposed to oxidation without a BST overlay. The IDT fatigue test results indicate that HMA with BST has greater resistance to top-down fatigue cracking than the HMA without BST. Applying a BST overlay effectively protected the underlying HMA from oxidation and reduced the aging of the binder in the underlying HMA.

APPENDIX C: MIX DESIGNS OF PROJECTS USED IN LABORATORY ANALYSIS

The mix designs of the asphalt pavements analyzed in this study are included in the following pages.

APPENDIX C1: 3/8-inch VS. 1/2-inch NMAS PROJECT MIX DESIGNS

The mix designs for the 3/8-inch and 1/2-inch sections of Contract 8611 on I-90 are shown below.

Washington State Department of Transportation - Materials Laboratory
PO Box 47365 Olympia WA 98504 / 1655 S. 2nd Ave. Tumwater WA 98512
BITUMINOUS MATERIALS SECTION MIX DESIGN EVALUATION REPORT

MATERIAL :	HMA Class 3/8" - 9-03.8 - 2014	WORK ORDER NO :	008611
DATE SAMPLED :	06/04/2014	SAMPLE ID :	00000116df0
DATE RECV'D :	06/12/2014	MIX ID NO :	MD140057
SR NO :	090	CONTRACTOR :	Inland Asphalt
SECTION :	Barker Rd to Idaho State Line Paving		
PROJECT ENGINEER :	Larson, Larry	ORG CODE :	464304

CONTRACTOR'S MIX DESIGN TEST DATA

					Specification
Pb		4.9	5.4	5.9	
% Gmm @ Ninitial	8	84.2	84.9	86.9	≤ 89.0
% Va @ Ndesign	100	5.1	4.2	1.7	Approximate 4.0
% VMA @ Ndesign	100	15.4	15.3	14.4	≥ 15.0
% VFA @ Ndesign	100	67	73	88	73 - 76
% Gmm @ Nmax	160		97.1		≤ 98.0
Dust to Asphalt Ratio (D/A)		1.2	1.2	1.0	0.6 - 1.6
Pbe		4.6	4.8	5.5	
Gmm		2.462	2.457	2.432	
Gmb		2.338	2.354	2.391	
Gb		1.031	1.031	1.031	
Gse		2.652	2.668	2.659	
Hamburg Wheel-Test (mm)					≤ 10.0
Stripping Inflection Point					None @ 15,000
Indirect Tensile Strength (psi)					≤ 175

STATE MATERIALS LABORATORY VERIFICATION TEST DATA

					Specification
Pb		4.9	5.4	5.9	
% Gmm @ Ninitial	8	82.9	83.9	85.0	≤ 89.0
% Va @ Ndesign	100	7.2	5.8	4.2	Approximate 4.0
% VMA @ Ndesign	100	16.0	15.8	15.4	≥ 15.0
% VFA @ Ndesign	100	55	64	73	73 - 76
% Gmm @ Nmax	160		96.2		≤ 98.0
Dust to Asphalt Ratio (D/A)		1.5	1.3	1.2	0.6 - 1.6
Pbe		3.9	4.4	4.9	
Gmm		2.486	2.466	2.451	
Gmb		2.308	2.324	2.348	
Gb		1.031	1.031	1.031	
Gse		2.681	2.679	2.682	
Hamburg Wheel-Test (mm)			3.0		≤ 10.0
Stripping inflection Point			Pass		None @ 15,000
Indirect Tensile Strength (psi)			114		≤ 175

STATE MATERIALS LABORATORY RECOMMENDATIONS

Asphalt Binder Supplier	**WSA**	
Asphalt Binder Grade	PG 70-28	
Percent Binder (Pb) (By Wt. Total Mix)	5.4	
% Anti-Strip (By Wt. of Asphalt Binder) / Type	0.0	
Sample Wt. (grams)	4600	(Informational Only)
Ignition Calibration Factor	0.29	(Informational Only)
Optimum Mixing Temperature (°F)	321	
Compaction Temperature (°F)	298	

Washington State Department of Transportation - Materials Laboratory
PO Box 47365 Olympia WA 98504 / 1655 S. 2nd Ave. Tumwater / WA 98512
BITUMINOUS MATERIALS SECTION MIX DESIGN VERIFICATION REPORT

MATERIAL: HMA Class 3/8" - 9-03.8 - 2014 WORK ORDER NO : 008611

SAMPLE ID : 00000116df0 MIX ID NO : MD140057

CONTRACTOR'S DESIGN AGGREGATE STRUCTURE AND AGGREGATE TEST DATA

Material:	3/8"-0	Combined	Spec	Tolerance
Source:	C173			
Ratio:	100.0%			
1/2 in	100.0	100	99 - 100	99 - 100
3/8 in	94.0	94	90 - 100	90 - 100
No. 4	57.0	57	90 Max	51 - 63
No. 8	34.0	34	32 - 67	32 - 40
No. 16	22.0	22		
No. 30	16.0	16		
No. 50	11.0	11		
No. 100	8.0	8		
No. 200	5.7	5.7	2.0 - 7.0	3.7 - 7.0

Gsb Coarse	2.642		
Gsb Fine	2.618		
Gsb Blend	2.628	2.628	
Sand Equivalent (SE)		81	45 Min
% Uncompacted Voids		45	44 Min
% Fracture	100	100	90 Min Double Face Fracture

STATE MATERIALS LABORATORY AGGREGATE TEST DATA

Gsb Coarse	2.615		
Gsb Fine	2.606	2.606	
Gsb Blend	2.610	**2.610**	
Sand Equivalent (SE)	78	78	45 Min
% Uncompacted Voids		49	44 Min
% Fracture	100	100	90 Min Double Face Fracture

COMMENTS

Remarks:

Result Code:

Billing Code

T177 - 1

T185 - 18

T194 - 2

T19B - 3

Kurt R. Williams, P.E.
State Materials Engineer
Joseph R. DeVol
Assistant Construction Materials Engineer
Date : 7/3/2014
Phone : (360) 709-5421

Washington State Department of Transportation - Materials Laboratory
PO Box 47365 Olympia WA 98504 / 1655 S. 2nd Ave. Tumwater WA 98512
BITUMINOUS MATERIALS SECTION MIX DESIGN EVALUATION REPORT

MATERIAL : HMA Class 1/2" - 9-03.8 - 2014
DATE SAMPLED : 05/27/2014
DATE RECV'D : 06/02/2014
SR NO : 90
SECTION : Barker Rd to Idaho State Line Paving
PROJECT ENGINEER : Larson, Larry

WORK ORDER NO : 008611
SAMPLE ID : 00000116ad4
MIX ID NO : MD140053
CONTRACTOR : Inland Asphalt
ORG CODE : 464304

CONTRACTOR'S MIX DESIGN TEST DATA

VALID FOR 2014

					Specification
Pb		4.4	4.9	5.4	
% Gmm @ Ninitial	8	84.2	85.5	87.2	≤ 89.0
% Va @ Ndesign	100	6.0	3.9	2.8	Approximate 4.0
% VMA @ Ndesign	100	14.6	14.0	13.9	≥ 14.0
% VFA @ Ndesign	100	59	72	80	65 - 75
% Gmm @ Nmax	160		97.0		≤ 98.0
Dust to Asphalt Ratio (D/A)		1.5	1.3	1.2	0.6 - 1.6
Pbe		3.8	4.4	4.7	
Gmm		2.503	2.481	2.469	
Gmb		2.353	2.384	2.398	
Gb		1.031	1.031	1.031	
Gse		2.679	2.674	2.682	
Hamburg Wheel-Test (mm)					≤ 10.0
Stripping Inflection Point					None @ 15,000
Indirect Tensile Strength (psi)					≤ 175

STATE MATERIALS LABORATORY VERIFICATION TEST DATA

CONTRACT 008611 ONLY

					Specification
Pb		4.4	4.9	5.4	
% Gmm @ Ninitial	8	84.1	85.9	87.2	≤ 89.0
% Va @ Ndesign	100	6.7	4.3	2.7	Approximate 4.0
% VMA @ Ndesign	100	14.6	13.8	13.3	≥ 14.0
% VFA @ Ndesign	100	55	69	80	65 - 75
% Gmm @ Nmax	160		97.1		≤ 98.0
Dust to Asphalt Ratio (D/A)		1.7	1.4	1.3	0.6 - 1.6
Pbe		3.5	4.1	4.6	
Gmm		2.509	2.483	2.469	
Gmb		2.342	2.376	2.403	
Gb		1.031	1.031	1.031	
Gse		2.687	2.677	2.682	
Hamburg Wheel-Test (mm)			2.5		≤ 10.0
Stripping inflection Point			Pass		None @ 15,000
Indirect Tensile Strength (psi)			114		≤ 175

STATISTICAL

STATE MATERIALS LABORATORY RECOMMENDATIONS

Asphalt Binder Supplier	WSA	
Asphalt Binder Grade	PG 70-28	
Percent Binder (Pb) (By Wt. Total Mix)	4.9	
% Anti-Strip (By Wt. of Asphalt Binder) / Type	0.00	
Sample Wt. (grams)	4700	(Informational Only)
Ignition Calibration Factor	0.28	(Informational Only)
Optimum Mixing Temperature (°F)	321	
Compaction Temperature (°F)	298	

Washington State Department of Transportation - Materials Laboratory
PO Box 47365 Olympia WA 98504 / 1655 S. 2nd Ave. Tumwater / WA 98512
BITUMINOUS MATERIALS SECTION MIX DESIGN VERIFICATION REPORT

MATERIAL: HMA Class 1/2" - 9-03.8 - 2014 WORK ORDER NO : 008611
SAMPLE ID : 00000116ad4 MIX ID NO : MD140053

CONTRACTOR'S DESIGN AGGREGATE STRUCTURE AND AGGREGATE TEST DATA

Material:	5/8" Chip	1/2"-#4	3/8"-0	#8-0	Combined	Spec	Tolerance
Source:	C173	C173	C173	C120			
Ratio:	12.0%	8.0%	75.0%	5.0%			
3/4 in	100.0	100.0	100.0	100.0	100	99 - 100	99 - 100
1/2 in	77.0	100.0	100.0	100.0	97	90 - 100	91 - 100
3/8 in	17.0	57.0	94.0	100.0	82	90 Max	76 - 88
No. 4	1.0	4.0	57.0	100.0	48		43 - 53
No. 8	1.0	2.0	36.0	99.0	32	28 - 58	28 - 36
No. 16	1.0	1.0	24.0	80.0	22		
No. 30	1.0	1.0	17.0	42.0	15		
No. 50	1.0	1.0	12.0	13.0	10		
No. 100	1.0	1.0	10.0	6.0	8		
No. 200	0.7	1.0	7.1	4.0	5.7	2.0 - 7.0	3.7 - 7.0

Gsb Coarse	2.669	2.666	2.642			
Gsb Fine			2.618	2.610		
Gsb Blend	2.669	2.666	2.628	2.610	2.635	
Sand Equivalent (SE)					81	45 Min
% Uncompacted Voids					47	44 Min
% Fracture	99				99	

STATE MATERIALS LABORATORY AGGREGATE TEST DATA

Gsb Coarse	2.665	2.670	2.615			
Gsb Fine			2.606	2.596	2.605	
Gsb Blend	2.665	2.670	2.610	2.596	**2.620**	
Sand Equivalent (SE)			78	85	79	45 Min
% Uncompacted Voids					47	44 Min
% Fracture	96	100	100		99	90 Min Double Face Fracture

COMMENTS

Remarks:

Result Code:

Billing Code

T177 - 1
T185 - 18
T194 - 2
T19B - 3

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State Materials Engineer
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Date : 6/23/2014
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8 APPENDIX C2: SMA AND INCREASED ASPHALT PROJECT MIX DESIGN

The mix designs for the SMA and the HMA sections of Contract 6151 on I-90 are shown below.

Washington State Department of Transportation - Materials Laboratory
PO Box 47365 Olympia / 1655 S 2nd Ave. Tumwater / WA 98504
BITUMINOUS SECTION TEST REPORT

TEST OF: A.C.P. JOB MIX DESIGN CLASS STONE MATRIX 1/2"
DATE SAMPLED:
DATE RECVD HQS: 8/8/01
SR NO: 90
SECTION: SR21 I/C TO RITZVILLE

WORK ORDER NO: 006151
LAB ID NO: 0000340767
TRANSMITTAL NO: 194671
MIX ID NO: G10069

CONTRACTOR'S PROPOSAL

Mat'l	5/8"- 3/8"	3/8" - 0	3/8"-#4	FILLER	COMBINED
Source:	AD-137	AD-137	AD-137		
Ratio:	15%	35%	42%	8%	
1"	100.0	100.0	100.0	100.0	100
3/4"	100.0	100.0	100.0	100.0	100
1/2"	78.1	100.0	100.0	100.0	97
3/8"	19.0	96.1	90.6	100.0	83
#4	2.2	58.3	2.0	100.0	30
#8	1.8	34.6	1.7	100.0	21
#50	1.7	10.1	1.4	100.0	12
#200	1.5	5.6	1.0	100.0	10.6

LABORATORY ANALYSIS / SPECIFICATIONS

				SPECIFICATIONS
ASPH % BY TOTAL WT OF MIX:	5.8	6.3	6.8	≥ 6.0
% VOIDS @ Ndes 100	5.2	4.5	3.8	4.0%
% VMA @ Ndes: 100	16.4	16.7	17.1	≥ 17.0%
VCAmix			40%	< 42%
DRAINDOWN @ PRODUCTION TEMP.			0.0%	0.3%MAXIMUM
Gmm - MAX S. G. FROM RICE	2.532	2.520	2.503	
Gmb - BULK S. G. OF MIX	2.401	2.406	2.409	
Gsb - OF AGGREGATE BLEND			2.706	
Gsb - OF FINE AGGREGATE			2.631	
Gb - SPECIFIC GRAVITY OF BINDER			1.030	

LOTTMAN STRIPPING EVALUATION

	0%	1/4%	1/2%	3/4%	1%
Visual Appearance:					
% Retained Strength:					

RECOMMENDATIONS

SUPPLIER	IDAHO
GRADE	PG76-28
% ASPHALT (BY TOTAL MIX)	6.8
% ANTI STRIP (BY WT ASPH)	0,5 %
IGNITION CALIBRATION FACTOR	0.63 (INFORMATIONAL ONLY)
MIX ID NUMBER	G10069
MIXING TEMPERATURE	340°F
COMPACTION TEMPERATURE	295°F

Headquarters:
Construction Engineer------ X T178- 1
Materials File------------------X T166-
General File--------------------X T172-
Bituminous Section-----------X T175-
Region: T152-
Administrator----------------46--X T153-
Materials Eng.--------------46--X
PE: -----G. OLSON-----------X (2)

REMARKS: VERIFY MIXING AND COMPACTION TEMPERATURES PRIOR TO PRODUCTION. PE WILL ADD 0.5% A/S. RICE VALUE OF 2.503 = 155.8 LBS/FT³. 0.3% STABILIZER TO BE ADDED TO MIX

THOMAS E. BAKER, P.E.
Materials Engineer
By: Dennis M. Duffy P.E. [______]
(360) 709-5420
Date: ___/___/___

NOTE* THIS IS A CORRECTED MIX DESIGN REPORT AS OF 10/18/01. A SINGNED COPY WILL FOLLOW ON EITHER 10/19/01 OR 10/22/01.

Post-it™ brand fax transmittal memo 7671 | # of pages ▸ 10H
To G. Gibson | From D. Ratcliff
Co. WSDOT | Co. WSDOT
Dept. | Phone #
Fax # | Fax #

Washington State Department of Transportation - Materials Laboratory
PO Box 47365 Olympia / 1655 S 2nd Ave. Tumwater / WA 98504
BITUMINOUS SECTION TEST REPORT

TEST OF: A.C.P. JOB MIX DESIGN CLASS SUPERPAVE 1/2"
DATE SAMPLED: 8/2/01
DATE RECVD HQS: 8/8/01
SR NO: 90
SECTION: SR21 VIC. TO RITZVILLE

WORK ORDER NO: 006151
LAB ID NO: 0000340770
TRANSMITTAL NO: 194674
MIX ID NO: G10051

CONTRACTOR'S PROPOSAL

Mat'l	5/8"-3/8"	3/8"-0	BLEND SAND	RAP	COMBINED
Source:	AD-137	AD-137	FN-65	I-90	
Ratio:	18%	65%	2%	15%	
1"	100.0	100.0	100.0	100.0	100
3/4"	100.0	100.0	100.0	100.0	100
1/2"	78.1	100.0	100.0	99.2	96
3/8"	19.0	96.1	100.0	94.6	82
#4	2.2	58.3	99.0	70.2	51
#8	1.8	34.6	91.5	46.4	32
#16	1.8	20.1	61.0	31.0	19
#30	1.8	13.6	18.5	21.8	13
#50	1.7	10.1	4.0	15.0	9
#100	1.7	7.4	1.5	11.2	7
#200	1.5	5.6	1.2	8.4	5.2

LABORATORY ANALYSIS / SPECIFICATIONS

				SPECIFICATIONS
ASPH % BY TOTAL WT OF MIX:	5.2	5.6	5.7	
%Gmm @ Ninit: 8	84.9	85.9	86.1	≤ 89.0%
% VOIDS @ Ndes:100	5.5	4.4	4.1	4.5%
% VMA @ Ndes: 100	14.7	14.5	14.5	≥ 14.0%
% VFA @ Ndes: 100	63	70	72	65 - 75
DUST / ASPHALT RATIO	1.3	1.2	1.2	0.6 - 1.6
Pbe- PERCENT BINDER EFFECTIVE	3.9	4.3	4.4	
Gmm - MAX S. G. FROM RICE	2.591	2.577	2.573	
Gmb - BULK S. G. OF MIX	2.450	2.465	2.469	
Gsb - OF AGGREGATE BLEND		2.723		
Gsb - OF FINE AGGREGATE		2.687		
Gb - SPECIFIC GRAVITY OF BINDER		1.035		

LOTTMAN STRIPPING EVALUATION

	0%	1/4%	1/2%	3/4%	1%
Visual Appearance:	NONE	NONE	NONE	NONE	NONE
% Retained Strength:	75	84	88	86	93

RECOMMENDATIONS

SUPPLIER	IDAHO
GRADE	PG64-28
% ASPHALT (BY TOTAL MIX)	5.6
% ANTI STRIP (BY WT ASPH)	0.25%
IGNITION CALIBRATION FACTOR	0.93 (INFORMATIONAL ONLY)
MIX ID NUMBER	G10051
MIXING TEMPERATURE	325F
COMPACTION TEMPERATURE	293F

Headquarters:
Construction Engineer------ X
Materials File-----------------X
General File-------------------X
Bituminous Section----------X
Region:
Administrator------------46--X
Materials Eng.-----------46--X
PE: -----G. OLSON----------X (2)

T178- 1
T166-
T172-
T175-
T152- 1
T153- 1

REMARKS: VERIFY MIXING AND COMPACTION TEMPERATURE PRIOR TO PRODUCTION
RICE VALUE OF2.577 = 160.4 LBS./FT^3

THOMAS E. BAKER, P.E.
Materials Engineer
By: Dennis M. Duffy P.E.
(360) 709-5420
Date: 9/10/2001

VALID FOR THE YEAR 2001

Lab ID No. 0000340769
Lab Number G -10050
Trans. No. 194673
Bid. Item No.
Org. No. 464310
F.A. No. IM-0904(109)

Date Sampled:
Sampled By: **INLAND**
Date Recvd HQ: **08/08/2001**
S.R. No.: **90**
Section: **SR 21 VIC. TO RITZVILLE**
Contractor: **INLAND ASPHALT COMPANY**

Material: **3/8"-0 MIN AGG FOR SUPERPAVE**

Pit No.: **AD-137**

Sample Loc.: **QS-AD-137**
Test Loc.:

By:

Fracture: (Test Method WSDOT #103)

Sieve Size	Single Face	Double Face
3/8 in. (%)	**100**	**99**
No. 4 (%)	**100**	**100**

Coarse Aggregate: (AASHTO T-85)

Bulk Specific Gravity (SSD)	2.822
Bulk Specific Gravity	2.770
Apparent Specific Gravity	2.921
Absorption (%)	1.86

Fine Aggregate: (AASHTO T-84)

Bulk Specific Gravity (SSD)
Bulk Specific Gravity
Apparent Specific Gravity
Absorption (%)

Asphalt Content-Recycle Mat.:
(per WSDOT Std. Specs. 9-03.11)

Sand Equivalent: (AASHTO T-176) 81

Distribution:

Materials File		X
Region Administrator	46	X
Project Engineer:		
G. OLSON		X(2)

Result: **INFORMATIONAL**
Remarks:
ACP MIX DESIGN PREPARATION

THOMAS E. BAKER, P.E.
MATERIALS ENGINEER

Kurt R. Williams, P.E. By:
Date: 09/05/2001
Phone: (360)709-5446

T43B-	T43N-**29.0**	T44R-
T43C-**1.0**	T44B-**1.0**	T44T-
T43L-	T44C-	T44U-
T43M-	T44Q-1.0	

aggtests.dfr 03/14/01

9 APPENDIX C3: BST OVERLAY PROJECT MIX DESIGN

The mix design for Contract 8262 (SR 278) is shown below. The mix design for Contract 7109 (SR 20) was unavailable.

Washington State Department of Transportation - Materials Laboratory
PO Box 47365 Olympia / 1655 2nd Ave. Tumwater / WA 98504
BITUMINOUS SECTION MIX DESIGN VERIFICATION REPORT

HMA CLASS:	3/8"	WORK ORDER NO:	8262
DATE SAMPLED:	6/7/2012	LAB ID NO:	0000010F793
DATE RECV'D:	6/8/2012	TRANSMITTAL NO:	10F793
SR NO:		MIX ID NO:	MD120045
SECTION:	EASTERN REGION CHIP SEAL	CONTRACTOR:	CWA

VALID FOR 2012

CONTRACTOR'S MIX DESIGN TEST DATA

						Specifications
Pb		5.5	5.9	6.5		
% Gmm @ Nini:	6	87.7	87.9	89.3	≤	89.0
% Va @ Ndes:	50	4.9	3.9	2.0	Approximate	4.0
% VMA @ Ndes:	50	16.3	16.1	15.9	≥	15.0
% VFA @ Ndes:	50	70	76	87		70-80
% Gmm @ Nmax:	75		97.8		≤	98.0
D/A		1.3	1.2	1.0		0.6 - 1.6
Pbe		4.7	5.0	5.7		
Gmm		2.634	2.624	2.596		
Gmb		2.505	2.521	2.543		
Gb		1.035	1.035	1.035		
Gse		2.894	2.904	2.900		

CONTRACT 8262 ONLY

STATE MATERIALS LABORATORY VERIFICATION TEST DATA

						Specifications
Pb		5.1	5.6	6.1		
% Gmm @ Nini:	7	86.0	87.6	88.5	≤	89.0
% Va @ Ndes:	75	5.1	3.0	1.95	Approximate	4.0
% VMA @ Ndes:	75	15.7	15.0	15.1	≥	15.0
% VFA @ Ndes:	75	68	80	87		70-80
% Gmm @ Nmax:	115		98.2		≤	98.0
D/A		1.4	1.3	1.1		0.6 - 1.6
Pbe		4.4	4.9	5.3		
Gmm		2.650	2.627	2.612		
Gmb		2.579	2.550	2.562		
Gb		1.035	1.035	1.035		
Gse		2.893	2.891	2.899		

STRIPPING EVALUATION

% Anti-Strip:	0.0%	0.25%	0.50%	0.75%	1.0%
Visual Appearance:	NONE	NONE	NONE	NONE	NONE
% Retained Strength:	101	102	102	97	100

STATISTICAL

STATE MATERIALS LABORATORY RECOMMENDATIONS

		Remarks:
Asphalt Binder Supplier	**IDAHO**	
Asphalt Binder Grade	PG64-28	
Percent Binder (Pb) (By Wt. Total Mix)	5.6	
% Anti-Strip (By Wt. Asphalt Binder)	0.00%	
Type of Anti-Strip		
Mix ID Number	MD120045	
Sample Wt. (grams)	5075	(Informational Only)
Sample Height @ Ndes	115.0	(Informational Only)
Ignition Calibration Factor	0.46	(Informational Only)
Optimum Mixing Temperature	321°F	
Compaction Temperature	290°F	
Rice Density (lbs/ft^3)	163.5	

Washington State Department of Transportation - Materials Laboratory
PO Box 47365 Olympia / 1655 2nd Ave. Tumwater / WA 98504
BITUMINOUS SECTION MIX DESIGN VERIFICATION REPORT

TEST OF: AGGREGATE PROPERTIES FOR HMA CLASS: 3/8" WORK ORDER NO: 8262
LAB ID NO: 0000010F793 MIX ID NO: MD120045

CONTRACTOR'S DESIGN AGGREGATE STRUCTURE AND AGGREGATE TEST DATA

				Combined	Specifications	Tolerance
Material:	1/2"-#4	3/8"-0	SAND			
Source:	C68	C68	GT154			
Ratio:	20%	70%	10%			
1 1/2" square						
1" square						
3/4" square						
1/2" square	100.0	100.0	100.0	100	100	99-100
3/8" square	80.0	100.0	100.0	96	90 - 100	90-100
U.S. No. 4	3.0	85.0	100.0	70	MAX 90	65-75
U.S. No. 8	1.0	45.0	100.0	42	32 - 67	38-46
U.S. No. 16	1.0	30.0	100.0	31		
U.S. No. 30	1.0	18.0	100.0	23		
U.S. No. 50	1.0	12.0	95.0	18		
U.S. No. 100	1.0	10.0	30.0	10		
U.S. No. 200	1.0	8.0	2.0	6.0	2.0 - 7.0	4.0-7.0
Gsb Coarse	2.905	2.829				
Gsb Fine		2.811	2.717			
Gsb Blend	2.905	2.823	2.717	2.828		
Sand Equivalent				78	45 MIN.	
Uncompacted Voids (FAA)				49	44% MIN.	
Course Agg Frac						
U.S. No. 4				100	≥ 90% Single	Face Fracture

STATE MATERIALS LABORATORY AGGREGATE TEST DATA

				Combined	Specifications	Tolerance
Gsb Coarse	2.864	2.853				
Gsb Fine		2.839	2.708	2.819		
Gsb Blend	2.864	2.841	2.708	**2.832**		
Sand Equivalent		93	72	89	45 MIN.	
Uncompacted Voids (FAA)				48	44% MIN.	
Course Agg Frac						
U.S. No. 4	100	100		100	≥ 90% Single	Face Fracture

COMMENTS

Remarks:
WSDOT testing and anti-strip evaluation performed at 75 Ndesign gyrations. The Pb to intersect approximately 4.0% Va at 75 gyrations is 5.3%.

Environmental & Engineering Programs:		T152 -		THOMAS E. BAKER P.E.
Construction Engineer------------------------------	X	T153 -		Materials Engineer
Accounting Section---------------------------------	X	T166 -		By: Joseph R. DeVol
General File--	X	T177 -	1	Bituminous Materials Engineer
Bituminous Materials Section----------------------	X	T185 -	18	(360) 709-5421
Region: EASTERN		T194-	1	Date: 7/19/2012
Construction Office--46 ---------------------------	X			
Materials Engineer---46 ---------------------------	X			
P.E.: C. SIMONSON	X(2)			