OPTIMIZING ASPHALT PAVEMENT PERFORMANCE FOR CLIMATE ZONES WITHIN WASHINGTON STATE

FINAL PROJECT REPORT

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16. Abstract

Asphalt pavement performance in Washington State varies greatly across the different climatic zones found within the state. The average surface life of pavements west of the Cascades is 16.7 years, compared to 10.9 years for pavements east of the Cascades and as low as 5 years in mountain pass areas. Currently, Washington State Department of Transportation (WSDOT) standards specify only two standard PG binders with the standard one or two grade bump to account for the climate conditions. This study evaluates potential material and construction practices to improve the longevity of pavements in the harsh climates of Eastern Washington and mountain pass areas. Promising strategies for improving pavement performance are determined from a literature review, survey of state agencies, and interviews of industry professionals. Performance history within the Washington State Pavement Management System (WSPMS) is analyzed to determine the causes of pavement failure within these climatic zones and verify the promising methods/technologies from the literature review, survey of state agencies, and interviews of industry professionals to determine if these methods/technologies have been tried in the state. Performance tests are conducted on field cores and extracted binders from some in-service pavements in Washington to quantify the effects of some of the methods/technologies. Recommendations for strategies to improve pavement performance are made for project-specific factors of traffic volume and historical failure modes.

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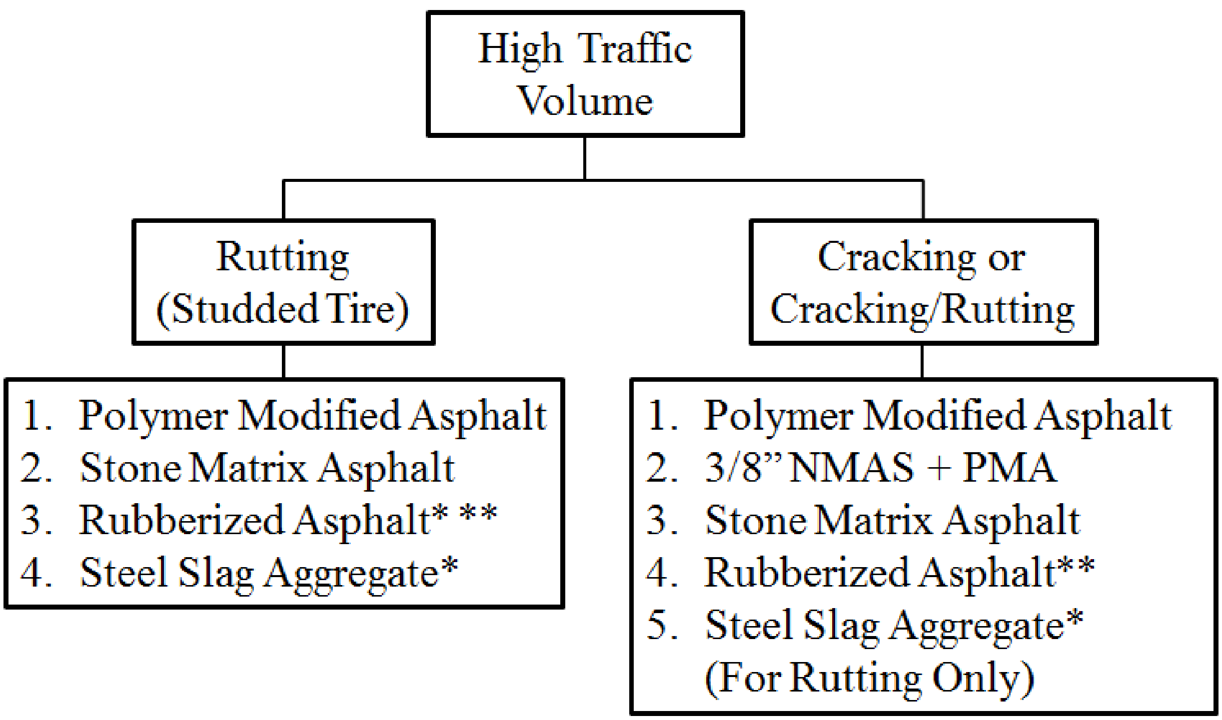
EXECUTIVE SUMMARY

Asphalt pavement performance in Washington State varies greatly across the different climatic zones found within the state. There are three general climatic categories within Washington State. West of the Cascade mountain range the climate is classified as mild marine with warm humid summers and cool wet winters. East of the Cascade range is classified as continental with hot dry summers and cold winters. There are also mountain areas with associated harsh wet winters controlling pavement life. The average surface life of pavements west of the Cascades is 16.7 years, compared to 10.9 years for pavements east of the Cascades and as low as 5 years in mountain pass areas. Currently, Washington State Department of Transportation (WSDOT) standards specify the same HMA classes and only two standard PG binders with the standard one or two grade bump to account for the climate conditions. This study evaluates potential material and construction practices to improve the longevity of pavements in the harsh climates of Eastern Washington and mountain pass areas. Promising strategies for improving pavement performance are determined from a literature review, survey of state agencies, and interviews of industry professionals. Performance history within the Washington State Pavement Management System (WSPMS) is analyzed to determine the causes of pavement failure within these climatic zones and verify the promising methods/technologies from the literature review, survey of state agencies, and interviews of industry professionals to determine if these methods/technologies have been tried in the state. Performance tests are conducted on field cores and extracted binders from some in-service pavements in Washington to quantify the effects of some of the methods/technologies. Recommendations for strategies to

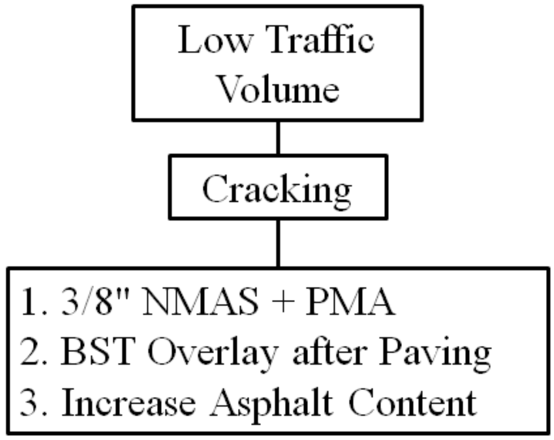
improve pavement performance are made for project-specific factors of traffic volume and historical failure modes.

Recommendations for improving performance of high traffic volume and low traffic volume asphalt pavements in Eastern Washington, and asphalt pavements in Washington's mountain areas are summarized in the flowcharts below. For high and low traffic volume pavements in Eastern Washington, the specific recommendations are listed in order of priority. For example, if a high traffic volume pavement section historically has failed due to rutting from studded tire wear, the first recommendation would be to use polymer modified asphalt in the surface course. For asphalt pavements in Washington mountain areas, recommendations are not prioritized by traffic volume or historical distress factor.

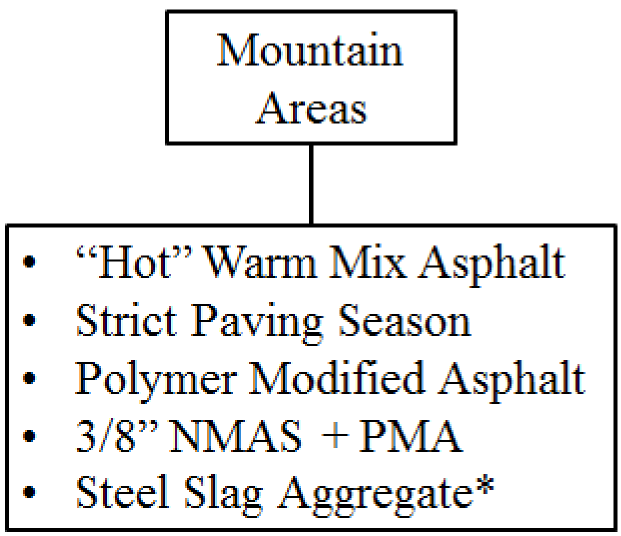


*It is recommended that test sections be constructed before widespread implementation
**Dry process

High Traffic Volume Flowchart



Low Traffic Volume Flowchart



*It is recommended that test sections be constructed before widespread implementation

Mountain Area Flowchart

It is recommended that the sites that are included in this study, such as the 3/8” mix, high PG mix, BST immediately following the paving, etc. be monitored over time. In addition, performance-based specifications can account for various loading and environmental conditions, which cannot be realized by "recipe" specifications, and are recommended to be implemented. Specifically, cracking performance tests for mix designs to complement the current Hamburg Wheel Tracking (HWT) test for rutting is recommended to be considered. Multiple stress creep and recovery (MSCR) is also recommended to be included in the specification for asphalt binder to receive polymer modification. In addition, the issue of fractured aggregates by the overcompaction during construction should be investigated and mitigated.

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CHAPTER 1: INTRODUCTION

1.1 Background

Asphalt pavements in Washington State have shown to have great differences in performance, depending on the climatic zones. The climate west of the Cascade Mountains is generally mild with wet winters, while the climate east of the Cascades is drier and sunnier with more extreme temperatures which often drop below freezing during winter. Studded tires are widely used in this area during winter time, creating additional damage (rutting and abrasion) to the asphalt pavements, although the lower studded tire use rate in Western Washington is somewhat offset by higher overall traffic levels. The climate within the Cascade Range is generally mild in summer but much more severe in winter with frequent snow and freezing conditions. As reported by WSDOT, the average surface life of pavements west of the Cascades (16.7 years) is significantly longer than those east of the Cascades (10.9 years) or in mountain pass areas (as low as 5 years) (WSDOT, 2012). These differences in asphalt pavement surface lives is likely due to a combination of factors; weather and studded tire wear being the most prominent.

1.2 Objective

The objective of this study is to identify mix design and construction practices that will improve asphalt pavement performance in Eastern Washington and Washington mountain pass areas.

1.3 Organization of Report

This study consists of five chapters and three appendices. Chapter 1 contains the project background and objectives. Chapter 2 contains the literature review and supporting information from the state agency survey results. Chapter 3 details the results of the WSPMS analysis and results of laboratory tests performed in this study. Chapter 4 contains cost information determined from the literature, survey, and interviews with industry professionals. Conclusions and recommendations from this study are presented in Chapter 5. Appendix A provides the complete state agency survey results. Appendix B details the laboratory tests performed in this study, with results of each test for each project. Finally, the mix designs of projects chosen for laboratory testing are provided in Appendix C.

1.4 Identification of Failure Mechanism of Pavements in Washington

An analysis of data in the Washington State Pavement Management System was used to identify failure mechanisms and well-performing and poor-performing asphalt pavements across the State of Washington. An analysis of historical performance data in the Washington State Pavement Management System provided a wealth of information that could not otherwise be gathered through laboratory experiments or field investigations alone. From WSPMS, the lives of individual pavement contracts and sections and the controlling distress factor (rutting, cracking, or roughness) at time of failure are available in terms of Performance Periods. A Performance Period is a segment of roadway indicating the length of time a construction activity lasted or is projected to last. In this study, primarily projects with asphalt pavements designed using the Superpave mix design method were analyzed.

In WSPMS, pavement distresses are measured with respect to cracking, rutting, and roughness using the following three metrics (WAPA, 2010):

- Pavement Structural Condition (PSC): A measure of pavement cracking from 100 (new pavement) to zero (total cracking failure).
- Pavement Rutting Condition (PRC) A measure of pavement rutting from 100 (no rutting) to zero (0.70 inches of rutting).
- Pavement Profile Condition (PPC): A measure of pavement roughness using International Roughness Index (IRI) from 100 (perfectly smooth ~~new~~ pavement) to zero (IRI=380 inches/mile).

When a pavement reaches a designated level of PSC, PRC, or PSC, the pavement is considered to have reached the end of its service life. Due to the fact that some pavements are kept in service for several years after reaching a designated failure threshold, for this study the time to reach the failure threshold was considered a more reliable indication of the length of pavement life than the total time between activities. Superpave HMA projects that have reached a failure threshold in Washington levels are shown by contract with annual average daily traffic (AADT) levels in Figure 1.1. Pavements that exceeded the average 11 year life in Eastern Washington are shown by contract with annual average daily traffic levels in Figure 1.2.

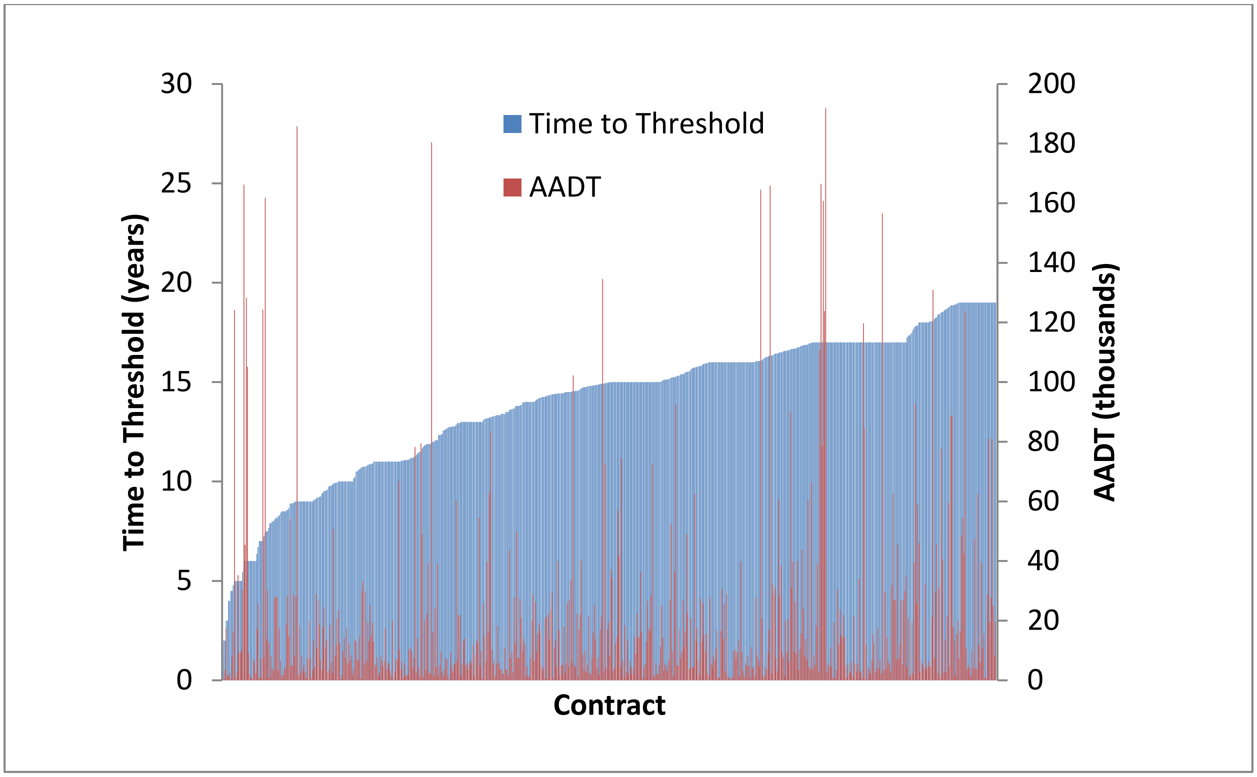


Figure 1.1 Superpave HMA Projects in Washington State

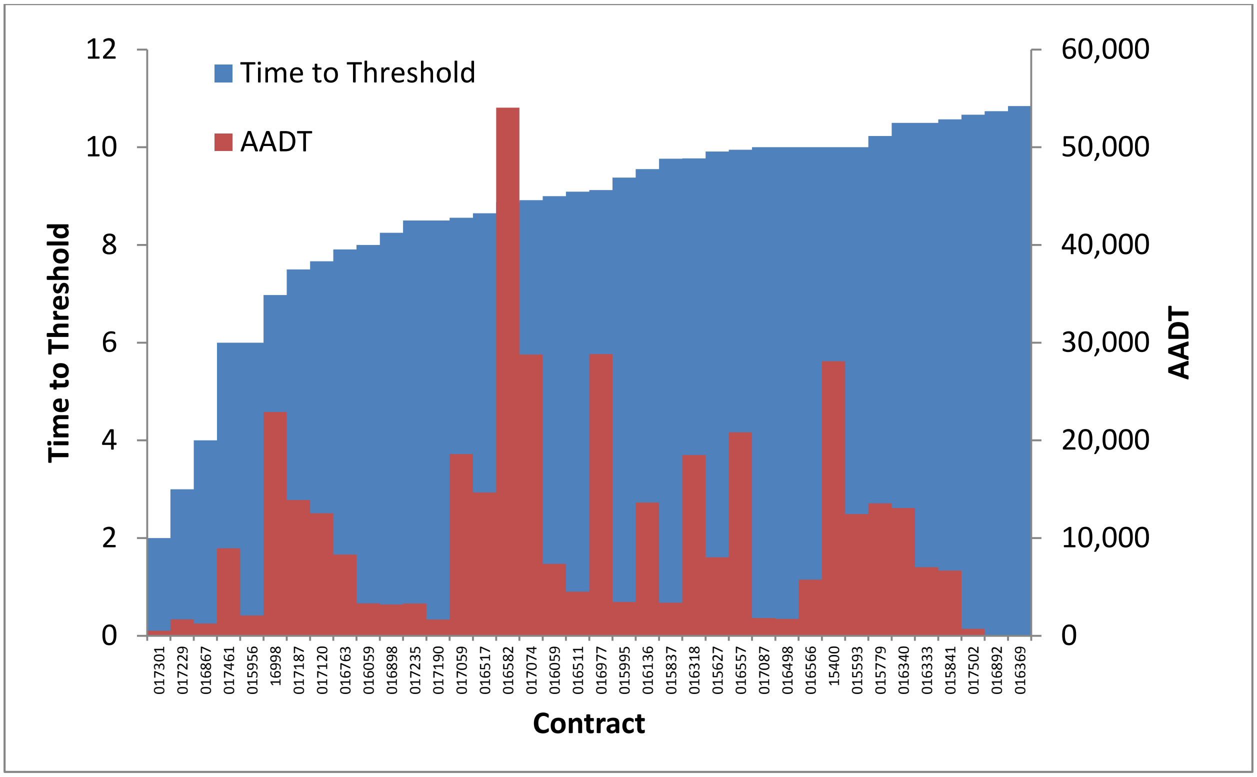


Figure 1.2 Eastern Washington Pavements

PSC was found to reach the failure threshold before PRC or PPC for most pavements across the State of Washington, indicating that cracking is the primary failure mechanism. Although WSPMS indicated some pavements failing by roughness, WSDOT does not use roughness as a trigger for rehabilitation/reconstruction; therefore only pavement failures by cracking and rutting were considered as primary failure mechanisms in this study. The distribution of failure mechanisms across the entirety of Washington, as well as in Eastern Washington, Western Washington, and over Washington mountain pass areas is shown in Figure 1.3. For this study, Eastern Washington is defined by the WSDOT Eastern, North Central, and South Central Regions, and Western Washington includes the Northwest, Olympic, and Southwest Regions (Figure 1.4). Highways in mountain pass areas are defined for this study in Table 1.1.

Detailed summaries of failure modes for Superpave HMA projects that have reached either the rutting failure threshold or cracking failure threshold are categorized by three AADT levels and three ESAL levels in Tables 1.2 and 1.3, respectively. While the total number of projects is relatively small, the pattern shows that cracking was the predominant failure mode for pavements with AADT under 20,000 or with equivalent single axle loads (ESALs) under 10 million. Rutting was the predominant failure mode for pavements with AADT over 20,000 or ESALs over 10 million. These findings are in line with the results based on the agency survey (Appendix A) and most agencies reported that cracking is the dominant failure mechanism. The average total asphalt layer depth for all pavements on low and high volume roadways that failed by cracking was 0.55 feet, and 0.42 feet for pavements that failed by rutting.

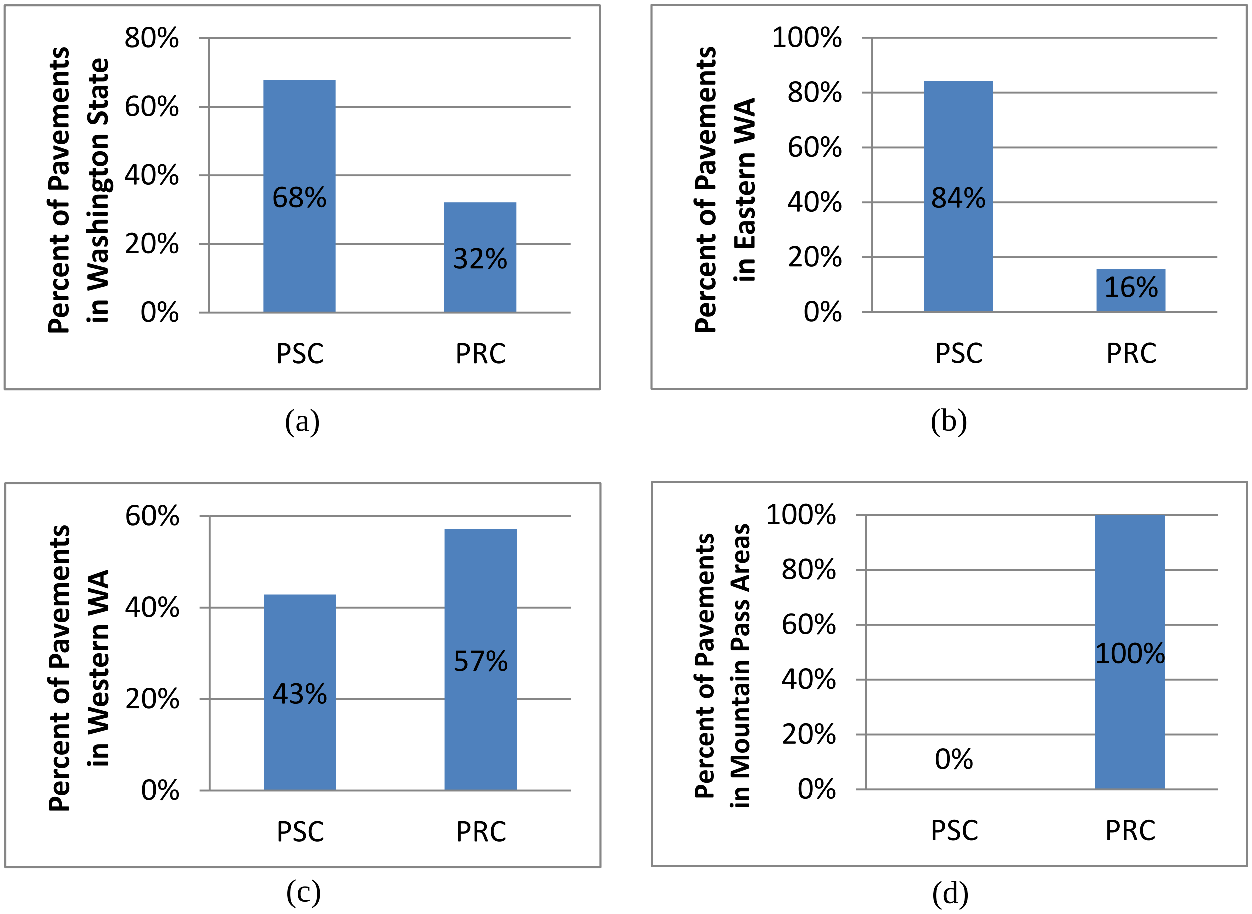


Figure 1.3 Failure Mechanisms of Pavements (a) Across Washington (b) in Eastern Washington (c) in Western Washington (d) in Washington Mountain Passes Areas

Figure 1.4 Eastern and Western Washington

Table 1.1 Mountain Pass Areas

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

Table 1.2 Failure Mechanisms of Pavements by AADT

Region	Low AADT (0-5,000)		Medium AADT (5,000-20,000)		High AADT (20,000 +)	
	Rutting	Cracking	Rutting	Cracking	Rutting	Cracking
Eastern	0	0	1	3	1	0
North Central	1	5	1	5	1	0
South Central	0	1	0	3	0	0
Northwest	0	1	1	1	2	1
Olympic	0	3	0	1	0	0
Southwest	1	0	0	0	0	0
TOTAL	**2**	**10**	**3**	**13**	**4**	**1**
Percent	**6%**	**30%**	**9%**	**39%**	**12%**	**3%**

Table 1.3 Failure Mechanisms of Pavements by ESALs

Region	Low ESALs 0-3 Million		Medium ESALs 3-10 Million		High ESALs 10+ Million	
	Rutting	**Cracking**	**Rutting**	**Cracking**	**Rutting**	**Cracking**
Eastern	0	2	0	1	2	0
North Central	0	5	3	4	0	1
South Central	0	1	0	2	0	1
Northwest	2	3	0	0	1	0
Olympic	0	4	0	0	0	0
Southwest	1	0	0	0	0	0
TOTAL	**3**	**15**	**3**	**7**	**3**	**2**
Percent	**9%**	**45%**	**9%**	**21%**	**9%**	**6%**

CHAPTER 2: LITERATURE REVIEW AND SURVEY

2.1 LITERATURE REVIEW AND STATE AGENCY SURVEY

A iterature review was performed and a survey of state agencies was conducted to determine possible measures to increase the life span of pavements in Eastern Washington and in Washington mountain pass areas. The literature review was conducted to evaluate the impact of mix design and construction practices on asphalt pavement performance within different climate zones worldwide. A review was conducted of mix types that have shown to perform well in climates similar to Eastern Washington and mountain pass areas and under studded tire wear. The survey was distributed among state highway agencies to gather information for best pavement design, construction, and preservation practices in harsh environmental climates. This was done in order to gain a better understanding of what is the state-of-the-practice for maximizing pavement life in harsh climates. Results from the survey are referenced throughout the literature review. The complete survey and responses are located in Appendix A.

2.1.1 CONSTRUCTION PRACTICES

As expected, most of the construction-related problems are associated with compaction, either low density or non-uniform compaction. Based on the literature, areas or technologies that could potentially affect the service life of asphalt pavement are described in the following sections. It is noted that many of the following construction practices have been or are being considered by WSDOT. Nonetheless, these practices are summarized herein to provide a summary of promising methods to improve asphalt pavement construction practices. As shown later in this report, this study is focused on the aspect of mix design and material selection.

2.1.1.1 Avoid Late Season Paving

Paving is usually one of the last portions of a project and can be pushed back into later fall months due to construction delays and other factors. A forensic investigation of I-90 by Washington State Department of Transportation showed that as ambient temperatures drop, it may be difficult to achieve proper densification by traffic, resulting in moisture entering the pavement and causing premature aging. Therefore, it is recommended that pavement construction be finished as early as possible to allow for proper compaction by traffic to achieve a more impermeable pavement (Russell et al. 2010). When paving in cold temperatures, the use of insulating tarps on trucks may keep the asphalt mix warm during the haul to the paving site. Material transfer devices may also be used to re-combine cooler mix exposed to air with the rest of the mix if the windrows are overlapped, and it is suggested that the paver keep moving to minimize cooling of the mix prior to compaction (Willoughby, 2000).

2.1.1.2 Increase Density along the Longitudinal Joint

Problems related to longitudinal joint construction, especially low density along the joint, have greatly affected asphalt pavement life. According to the Texas Department of Transportation Construction (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 dropoff that overnight traffic will drive over during paving (TxDOT, n.d.). According to Pavement Interactive (2009), to improve joint quality, use rubberized joint adhesives or notched wedge joints. When rolling a confined lane, roll the longitudinal joint 6 inches (150 mm) from the joint for the first roller pass. The joint density should be determined using cores, as nuclear density gauges may give erroneous results on joints. 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) evaluated the performance of eight different longitudinal joint construction techniques after a performance period of six years on SR 441 in Lancaster County, Pennsylvania. Kandhal et al. recommended the use of the rubberized joint adhesive or notched wedge joint for best longitudinal joint performance.

2.1.1.3 Mitigate Thermal Segregation

Thermal segregation is another cause of premature failure in asphalt pavements. According to AASHTO (1997), segregation causes non-uniform mixes that exhibit poor performance and low durability, resulting in a lower life expectancy. While thermal segregation does not always indicate issues with density, it is believed to be associated with aggregate segregation and should be minimized as much as possible when constructing asphalt pavements (Bode, 2012). It is WSDOT practice to use thermal imaging to find low temperature areas and take cores to catch areas of low density, resulting in a penalty to the contractor.

According to Larson (2010), TxDOT is offering incentives to contractors to use Pave-IR, an infrared temperature monitoring system, during cold temperature paving. Contractors are now allowed to pave during temperatures as low as 32ºF instead of the usual 50°F or 60°F, as long as they can use the Pave-IR system to prove there is no thermal segregation. This can extend the paving season and provide contractors with live feedback and the opportunity for corrective action while paving.

2.1.1.4 Use Intelligent Compaction to Achieve Consistent Compaction

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

2.1.1.5 Use Technologies to Facilitate Compaction

Russel et al. (2010) state that late season and cold temperature paving cause problems obtaining traffic densification. According to Goh and You (2009), warm mix asphalt (WMA) mixes are appealing for use in cold region pavements because of the smaller difference between production temperature and ambient temperature during construction, and it allows for a longer construction hauling distance when paving during cold weather without compromising the pavement performance. According to Robjent and Dosh (2009), some benefits for using WMA are that emissions are reduced during construction compared to HMA, and WMA has a comparable moisture susceptibility to HMA.

A comparison of long-term field performance of WMA and HMA was performed by the National Cooperatuive Highway Research Program (NCHRP) 09-49A at the Washington Center for Asphalt Technology (WCAT) (Wen, 2013). The field survey was conducted for 23 in-service pavements with 5+ years in the field throughout the United States. The survey included representative pavements in wet-freeze, dry-freeze, dry-no freeze, and wet-no freeze climatic zones. Preliminary conclusion of the study was that WMA is comparable to or better than HMA in transverse cracking.

A study in Norway (Veiteknisk Institutt, 2011) was conducted that analyzed the performance of warm mix asphalt with several additives. Cecabase RT and Rediset WMX were added to reduce the production temperature and increase the adhesion to offset the reduced adhesion from the lower production temperature and presence of water from foaming. Rediset WMX also improves resistance to permanent deformation. Sasobit® wax was added to lower the production temperature and resist permanent deformation, but not promote adhesion. The asphalt binder was also modified with WAM-foam, Green Asphalt and LMK foam to lower production temperature. Analysis of the pavement performance shows that the WMA was comparable to the HMA control section in roughness, resistance to deformation by Hamburg Wheel-Tracking Test, and initial field rutting.

Schiebel (2011) reports that Colorado Department of Transportation (CDOT) is experimenting with the use of warm mix asphalt combined with Advera®, Sasobit®, and Evotherm® on Interstate 70 in mountain terrain areas of elevations in the range of 8,800 to 11,100 feet. According to Aschenbrener et al. (2011), this area has extreme winter conditions with heavy tire chain usage while carrying vehicle loads of nearly five million ESALs over a ten year period and has shown to be comparable to HMA control sections in regard to rutting, cracking, and raveling.

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).

2.1.2 MATERIALS

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.

2.1.2.1 Aggregate and its 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 segregates 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 2.1. Gradations that make an “S” curve, as shown in Figure 2.2, tend to have segregation problems.

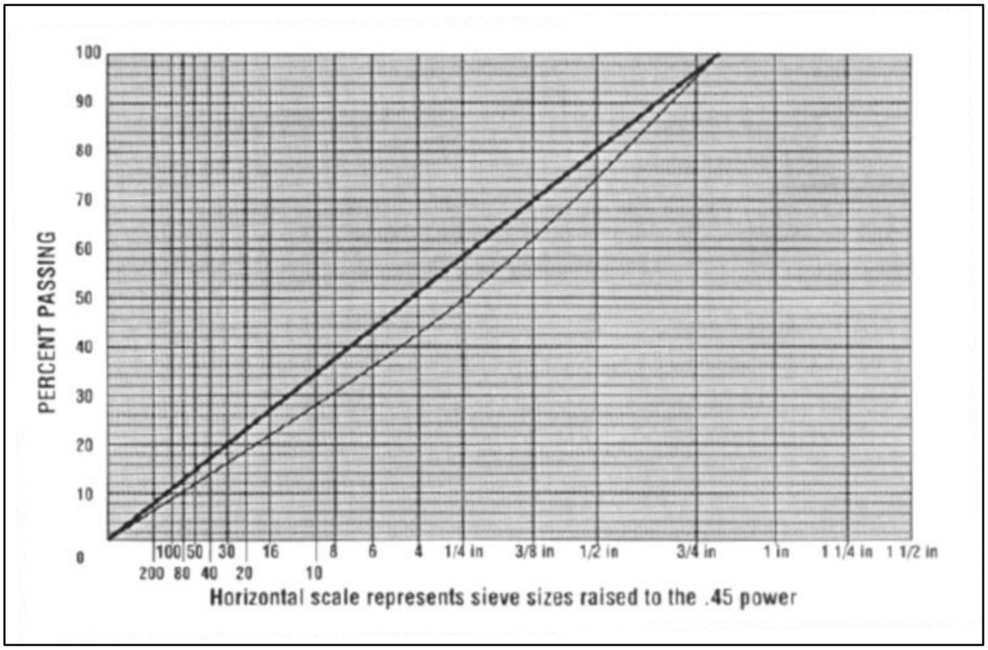


Figure 2.1 Bowed Gradation Curve (AASHTO, 1997)

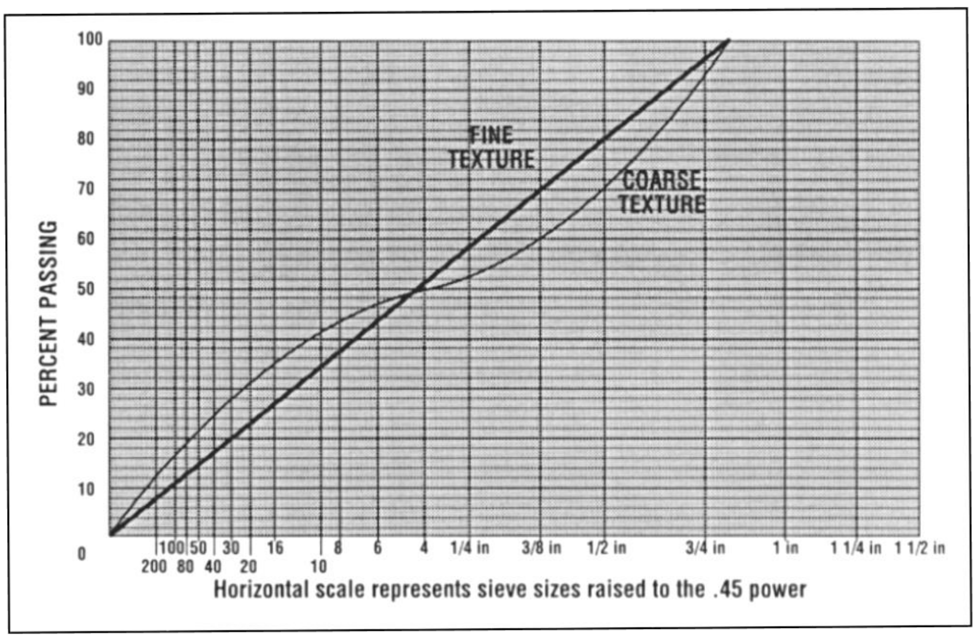


Figure 2.2 "S" Gradation Curve (AASHTO, 1997)

2.1.2.2 Increased Asphalt Content

The literature indicates that increasing the asphalt binder content in hot mix asphalt (HMA) mix designs can increase wear resistance (FHWA, 2011; Fromm and Corkill, 1971), as well as fatigue cracking. Increasing the asphalt binder content can be done in multiple ways, including reducing the minimum nominal aggregate size (NMAS) or using gap-graded mixtures such as stone matrix asphalt (AASHTO, 1997; FHWA, 2011), or decreasing the number of design gyrations (Ayyala et al, 2014). According to NCHRP Report 567, Christensen and Bonaquist (2006) state that increasing the minimum VMA will improve fatigue and rut resistance and will also decrease the permeability of the mixture. Based on the agency survey (Appendix A), Delaware Department of Transportation also increased VMA by 0.5% to improve the performance of mixes. Decreasing the design air voids to 3.0%-4.0% while decreasing the target air voids in the field will also improve the rut and fatigue resistance.

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

From the survey of state agencies, seven of eight agencies that modify their Superpave design procedure mentioned increasing asphalt content, generally by decreasing the design gyrations. Dave Powers of the Ohio Department of Transportation (ODOT) recommends fewer gyrations for more binder content, and Judith Corley-Lay from North Carolina Department of Transportation reports they have decreased gyrations and increased liquid asphalt content to reduce cracking. Studies by Ayyala et al. (2014) and Khosla and Ayyala (2013) were performed to optimize North Carolina surface mixture performance for fatigue cracking and rutting in high volume surface mixes (traffic levels of 3-30 million and greater than 30 million equivalent single axle loads (ESALs)). The goal was to increase asphalt content to improve fatigue resistance without compromising durability in regards to plastic deformation. It was found that 85 was a practical N_{des} value for both fatigue cracking and rutting performance for high volume roads. Prowell and Brown (2007) recommended reducing N_{des} levels and providing separate criteria for PG 76-XX binders, as shown in Table 2.1.

Table 2.1 Recommended N_{design} Levels (Prowell and Brown, 2007)

20-Year Design Traffic, ESALs	N_{des} for binders < PG 76-XX	N_{des} for binders ≥ PG 76-XX or mixes placed > 4 inches (100 mm) from surface
< 300,000	50	NA
300,000 to 3,000,000	65	50
3,000,000 to 10,000,000	80	65
10,000,000 to 30,000,000	80	65
> 30,000,000	100	80

2.1.2.3 3/8" Nominal Maximum Aggregate Size

The literature indicates that reducing the aggregate size will increase the asphalt binder content in HMA mixtures (FHWA, 2011). This can increase wear resistance (Fromm and Corkill, 1971), specifically in regards to fatigue cracking. The small nominal maximum aggregate size (NMAS) of a 3/8" aggregate mix is appealing for pavements in harsh climates due to the decreased permeability (Newcomb, 2009). A 3/8" NMAS mix is commonly used in Stone Matrix Asphalt (SMA) mixes as thin overlays for pavement rehabilitation. In South Dakota and Wyoming, it is common practice to employ mixes similar to SMA with a polymer modified asphalt binder and 3/8" aggregate (Root, 2009).

According to Adam Hand (personal communication, April 23rd, 2014), 3/8” 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” 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.

2.1.2.4 Polymer Modified Asphalt

Modifying the asphalt binder with polymers is one of the most extensively 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 2.3. This is one of several ways contractors can increase the PG grade of Superpave mixes.

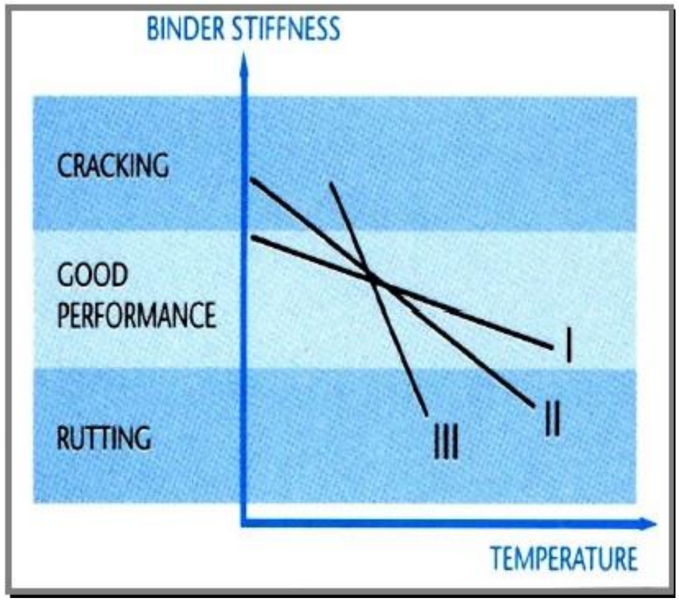


Figure 2.3 Polymer expansion of the ideal temperature range (IDOT, 2005)

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 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).

2.1.2.4.1 High PG Grade

Polymer modification often leads to an increase 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, top-down fatigue cracking, as well as rutting resistance, including studded tire wear resistance (Wen and Bhusal, 2015).

2.1.2.5 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. This includes Northern California which has a climate similar to Western Washington. 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%-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%-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.

2.1.2.6 Addition of Lime

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 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 providing 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.

2.1.2.7 Stone Matrix Asphalt

According to NCHRP Report 673 by the Federal Highway Administration (2011), gap-graded asphalt mixtures provide increased resistance to permanent deformation 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, 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 Yi-qiu (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). 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 a national survey of state agencies, 15% of agencies that responded said SMA is used as wearing course in climates similar to Eastern Washington (Figure 2.4).

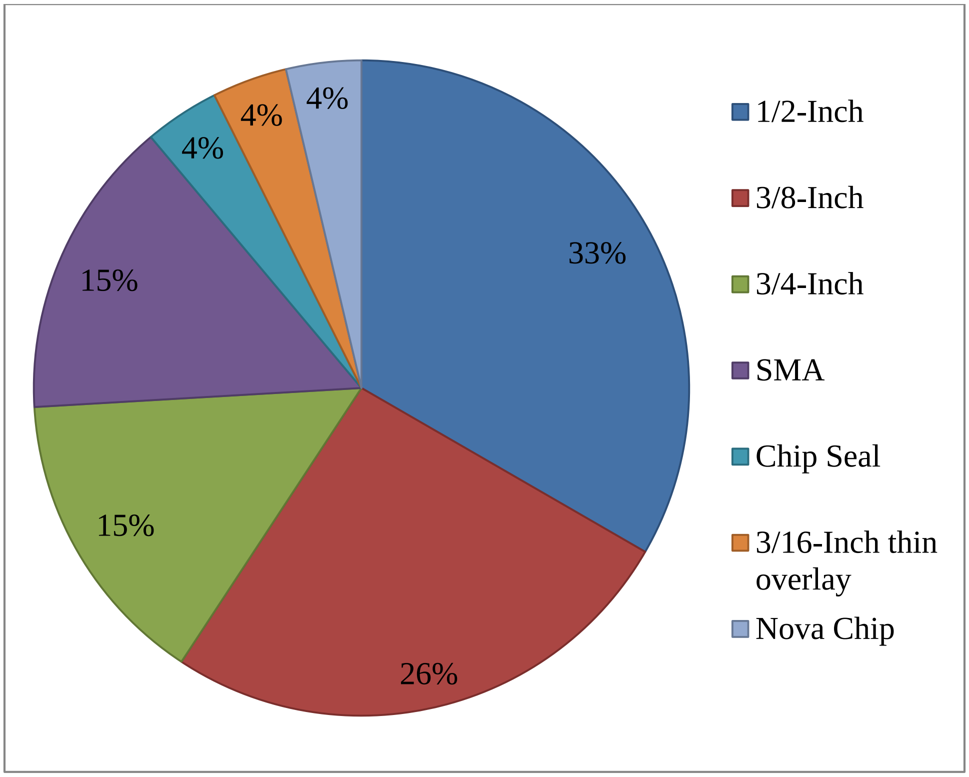


Figure 2.4 Asphalt Mix used as Wearing Course from Survey

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, though it is generally designed for 20 years.

Stone Matrix asphalt seems promising for further use in Eastern Washington, however, there are cases in which performance and construction issues were encountered. According to Myers (2007), the SMA project on SR 524 from 64th Avenue West to I-5 in Lynwood, WA was constructed in 1999 and experienced mix design and construction problems and had to be partially replaced. The next SMA project, I-90 from Ritzville to Tokio, was constructed in 2000 and was replaced the following year due to inadequate control over the mix production. Based on literature, it can be stated that SMA would be a good mix, provided that the mix design and construction are executed properly.

2.1.2.8 Steel Slag Aggregate (SSA)

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 and 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, and durable, and great in resistance to overall 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 2.2) 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 in the Illinois Tollway and has shown a comparable life to HMA performance, though its performance hasn’t been strictly 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 2.2 SMA Steel Slag Mix Design (National Slag Association, n.d.)

IDOT Mix Design

GADATION	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.

2.1.2.9 Performance-based Tests

Performance-based specifications such as the Hamburg Wheel Track Test (HWTT) for rutting and moisture susceptibility, can differentiate performance of mixes, when compared to the volumetrics-based “recipe” specifications. 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.

2.1.3 PRESERVATION

Pavement preservation methods may extend pavement life between major rehabilitations or overlays. The following preservation method has been reviewed for its impact on the surface life of HMA pavements.

2.1.3.1 BST Overlay

A bituminous surface treatment (BST), or chip seal, is often used to restore surface conditions of a deteriorated pavement, but can also be used to cover an asphalt pavement immediately after paving. BST overlays are generally used on low volume roads and are not meant to carry a large loading on a pavement structure; their purpose as a preservation technique is to provide a protective layer that reduces exposure of the underlying asphalt layer to sunlight and prevents the pavement from experiencing oxidation (Kuennen, 2005). According to Rolt (2001), a surface dressing such as a BST overlay can reduce the risk of top-down cracking due to hardening of the top 3 mm of bitumen of the wearing course. BST overlays have also shown to reduce longitudinal, transverse, and fatigue cracking, as well as effectively seal and protect centerline joints (Galehouse et al. 2005).

It 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). 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 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 to have 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 2.3. 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 2.3 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

CHAPTER 3: WSPMS ANALYSIS AND TEST RESULTS

3.1 Projects in Washington State

A number of promising methods/technologies that are mentioned in Chapter 2 have been at least experimentally used if not fully implemented by WSDOT. This section discusses performance to date for those methods/technologies actually constructed by WSDOT. The effectiveness of these methods/technologies on pavement performance was reviewed to verify the findings from the literature review and state agency survey. It is noted that not all methods/technologies can be verified through WSPMS. For instance, it was found to be difficult to locate projects that use lime as anti-stripping agent.

3.1.1 Polymer Modified Projects

3.1.1.1 Contract 7455: US 2 Creston to Rocklyn Road

Contract 7455 on US 2 in the Eastern Region of Washington State was paved in 2008 and has an average daily traffic (ADT) of approximately 2,700. The project consists of 1/2" PG 70-22 SBS polymer modified asphalt between mileposts 243.099-245.45 and 1/2” PG 64-28 HMA between mileposts 230.07-243.099. Performance data from WSPMS is shown in Figure 3.1. The SBS section seems to begin outperforming the HMA, pending further monitoring.

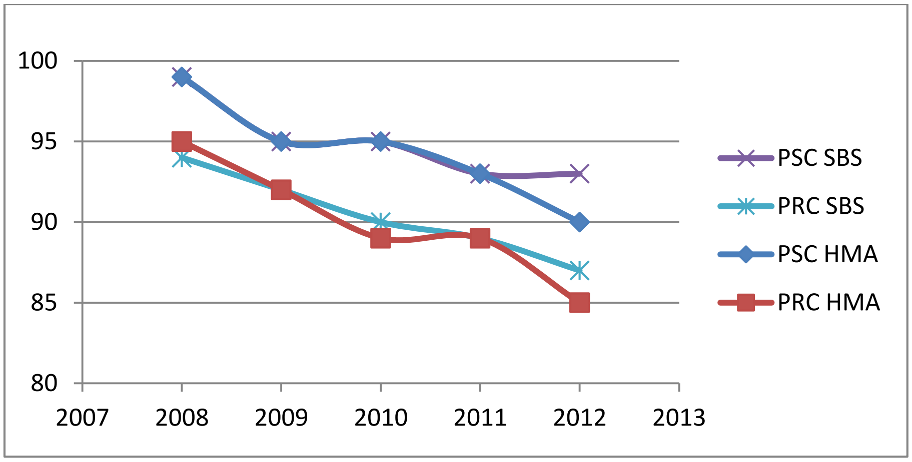


Figure 3.1 C7455 Performance Data

According to Anderson et al. (2008), four experimental rubberized pavement projects constructed between 1992 and 1997. Two of these projects were open graded and two were dense graded. One of the open graded projects with Modified Class D rubberized mix was over asphalted and failed by excessive rutting. Part of the other project with open graded Modified Class D rubberized mix was milled and overlaid after 12 years, which exceeded the average life of HMA pavements in that region. One of the projects with open graded Class A PBA-6GR was performing well until reconstructed to build the SR 520 Floating Bridge. Contract 4250 on I-5 from Nisqually River to Gravelly Lake I/C is the other dense graded section and it is still in place. The mix is also Class A PBA-6GR and although it 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 3.2.

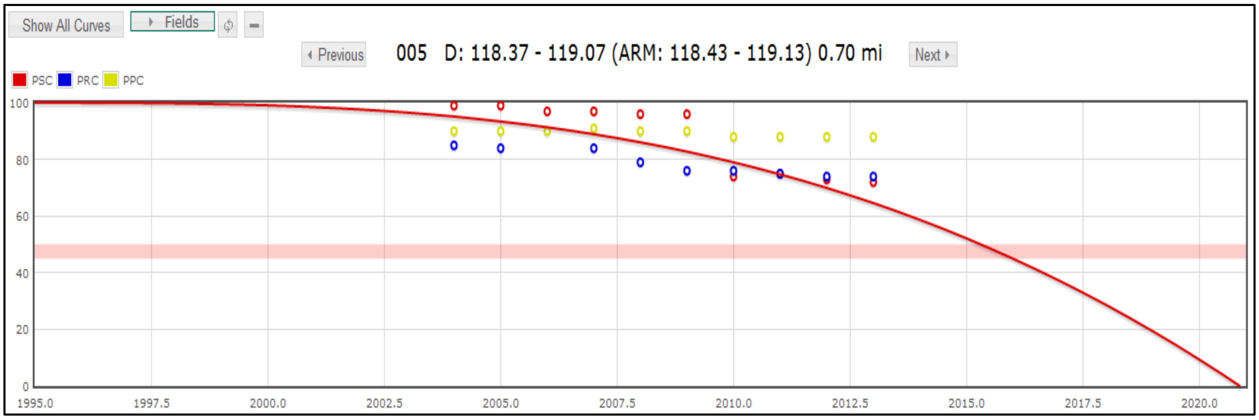


Figure 3.2 C4250 Performance Curves

3.1.2 3/8” NMAS Projects

Mixtures with 3/8” NMAS have only recently been implemented as wearing course in Washington State on primarily low volume roads; therefore performance data is limited.

3.1.2.1 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” HMA PG 70-28 and the rest of the project is ½” HMA PG 70-28.

Since the performance data is limited due to recent construction, performance tests were conducted on field cores taken from this project to determine the benefit of using a 3/8” NMAS asphalt mixture. 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. Overall, the performances of the 3/8” and 1/2” mixes are similar. Relatively, compared to the 1/2” mix, the 3/8” mix has similar studded tire resistance, equivalent strength, and equivalent top-down cracking resistance, as indicated by results of the studded tire wear test, IDT strength, and horizontal failure strain shown in Figures 3.3 through 3.5. The 3/8" 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 shown in Figures 3.6 and 3.7. The two mixes have approximately the same stiffness, as indicated by results of creep compliance and dynamic modulus shown in Figures 3.8 and 3.9. Details of this study and additional test results are provided in Appendix B1. The mix designs of the 3/8" and 1/2" HMA pavements are included in Appendix C1.

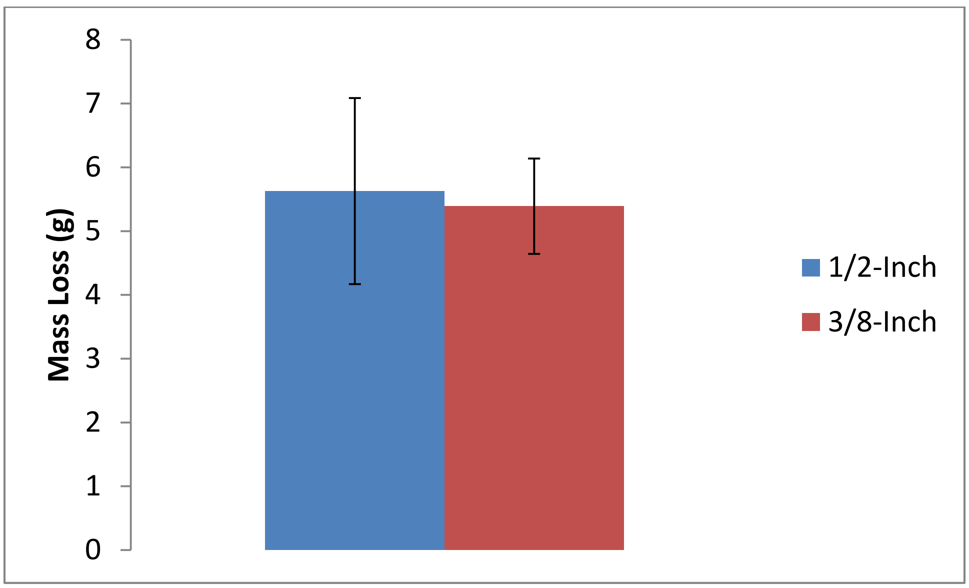


Figure 3.3 Studded Tire Wear

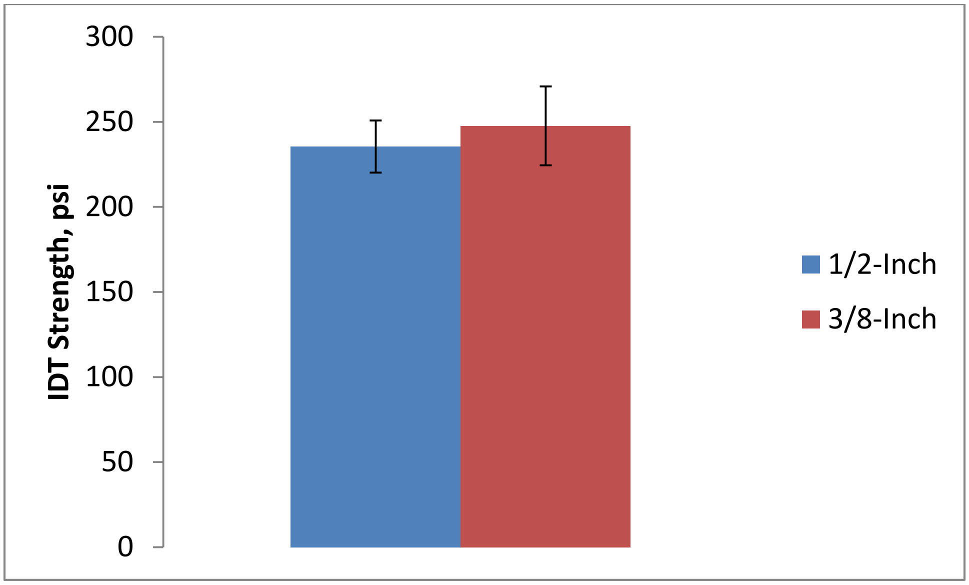


Figure 3.4 IDT Strength

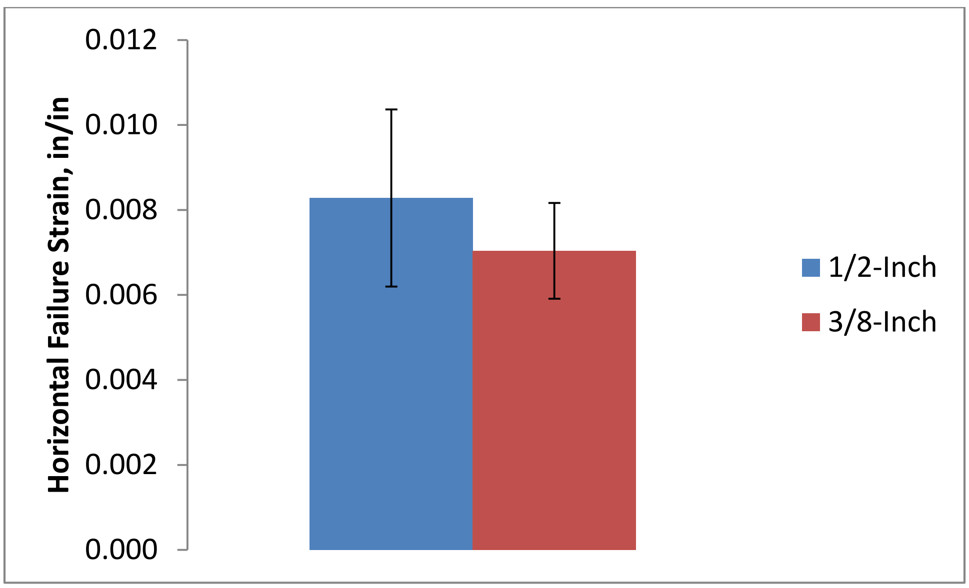


Figure 3.5 Horizontal Failure Strain

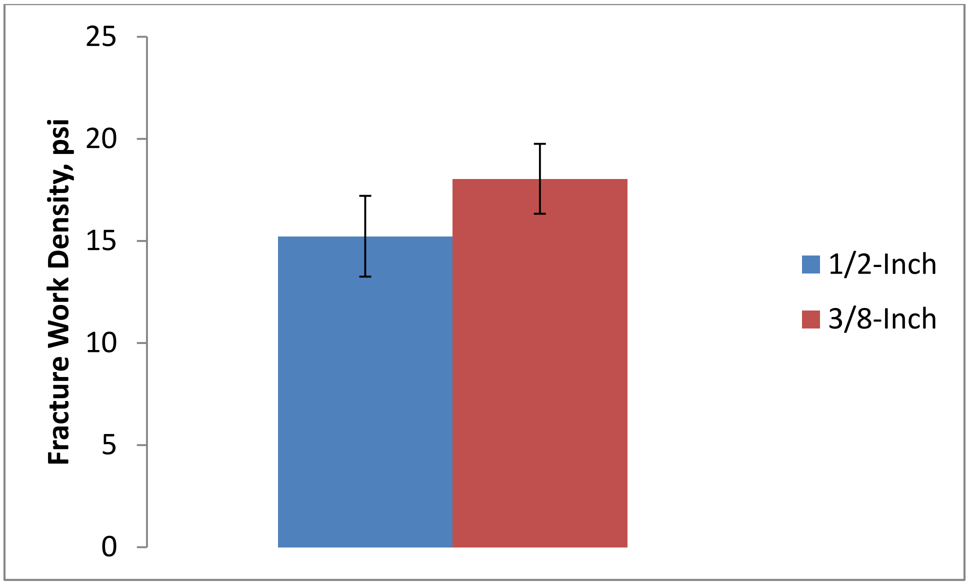


Figure 3.6 Fracture Work Density

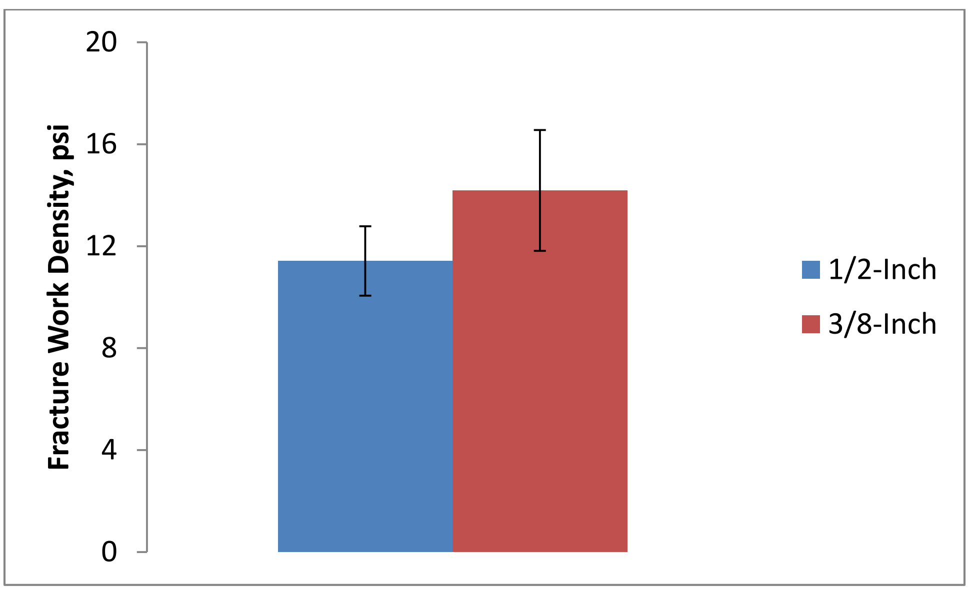


Figure 3.7 Fracture Work Density from Thermal Cracking Test

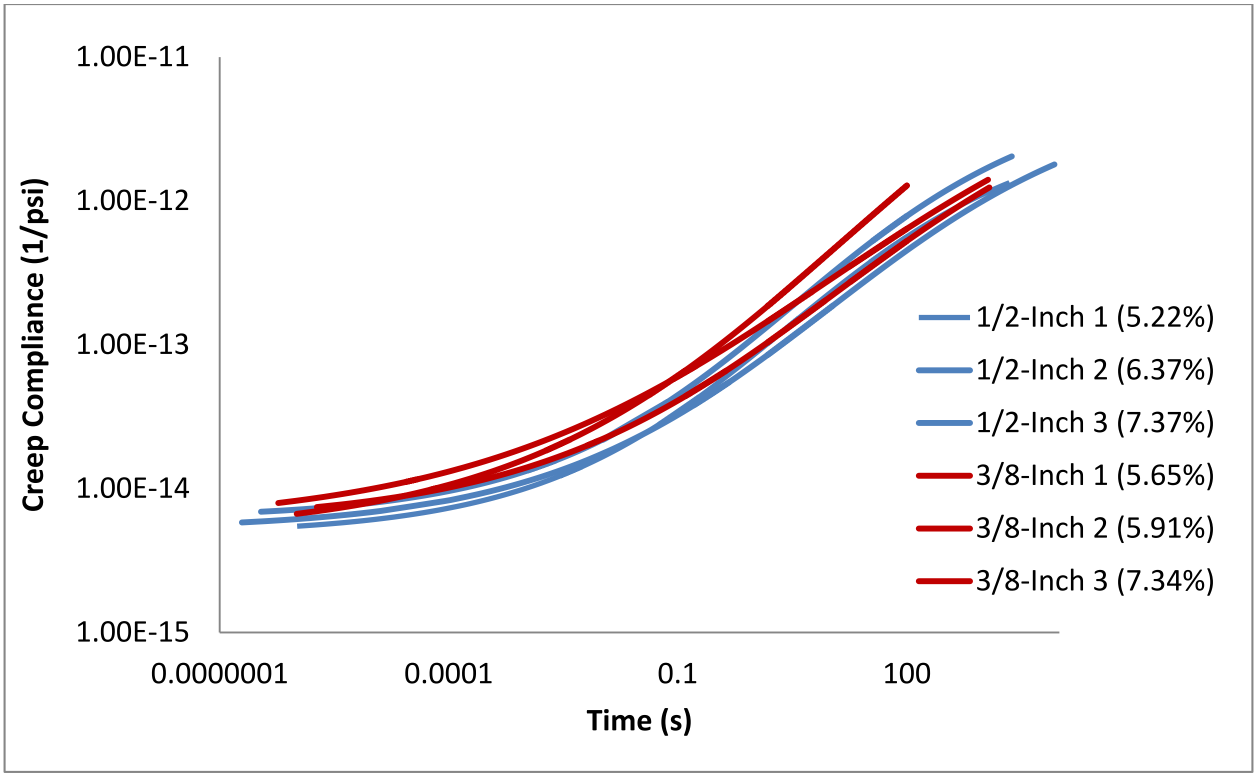


Figure 3.8 Creep Compliance Master Curves

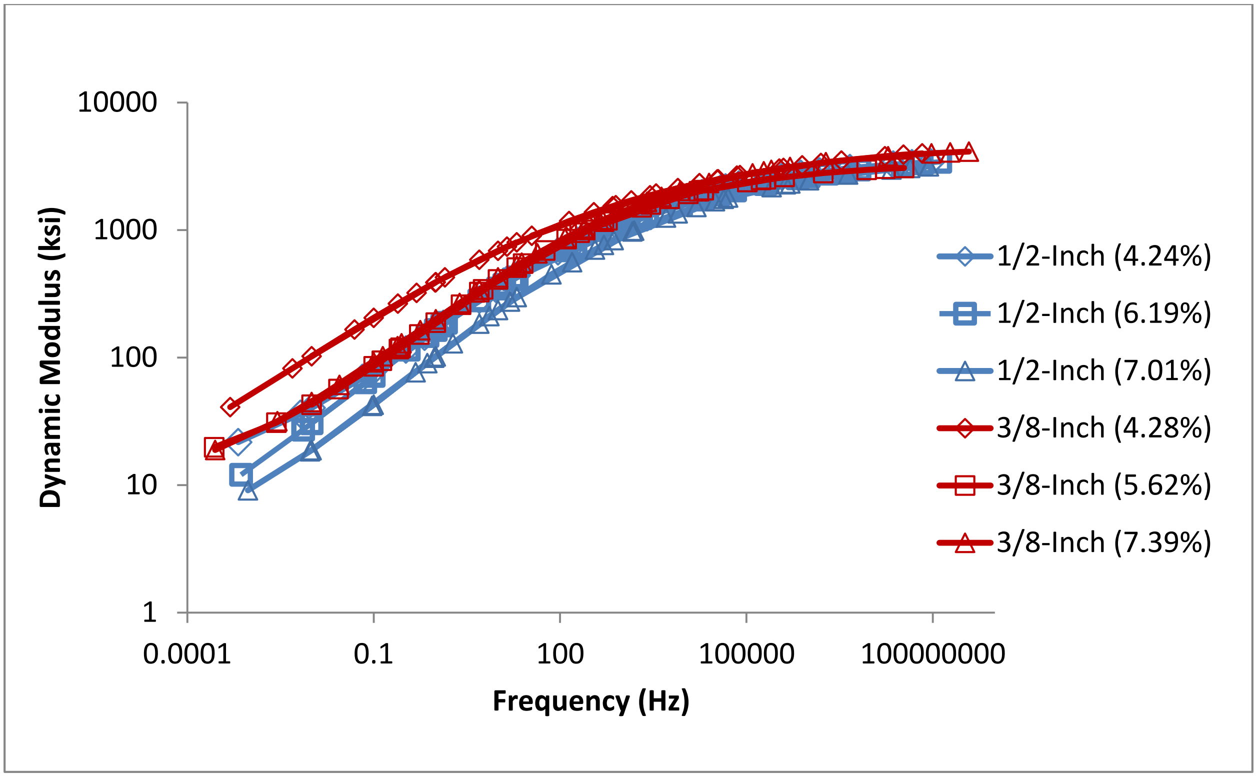


Figure 3.9 Dynamic Modulus Master Curves

3.1.2.2 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" HMA PG 64-28 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.

3.1.2.3 Contract 8443: 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” HMA PG 64-28. Again, there is no performance data available on this project.

3.1.2.4 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” PG 64-28 HMA and seems to be performing well. Figure 3.10 shows the WSPMS performance data for both the EB and WB lanes of Contract 7763. This section is still doing very well and was crack sealed in 2014, mostly due to cracks at the construction joints between lanes and at the shoulder joints.

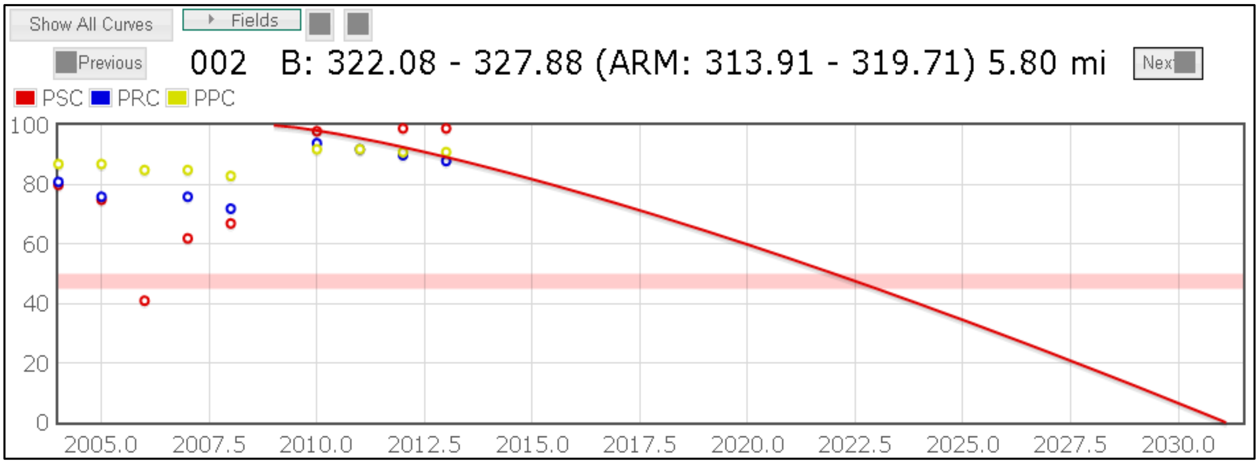


Figure 3.10 C7763 Performance Curves

These in-service 3/8” projects (Contracts 8611, 8447, and 7763) should be monitored in high and low traffic volume areas for long-term performance in resistance to rutting by plastic deformation, rutting caused by studded tire wear, and the combination of fatigue cracking and rutting.

3.1.3 Stone Matrix Asphalt Projects

Myers (2007) reports between 1999 and 2004, four SMA projects were constructed in Washington State. The projects used varying grades of asphalt binders with 1/2-Inch nominal maximum aggregate size SMA mixtures. The projects were as follows: SR from 524 64th Avenue West to I-5 in Lynnwood (1999), I-90 from Ritzville to Tokio (east of Ritzville) (2000), I-90 from SR 21 to Ritzville (west of Ritzville) (2001), and I-90 from Dodson Road to Moses Lake (2004). The Lynwood project had mix design and construction problems and some sections had to be replaced. The Ritzville to Tokio project experienced severe flushing and raveling and was replaced with HMA within one year.

3.1.3.1 Contract 6151: I-90 from SR 21 to Ritzville

Contract 6151 on I-90 from SR 21 to Ritzville 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 project was constructed with a section of 1/2" 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" PG 64-28 HMA. Rutting and cracking performance from WSPMS of these pavement sections are detailed in Table 3.1. It is noted that the mileposts listed in WSPMS do not exactly match with the milepost locations in the field, possibly due to changes made during construction from the original project plans. The mileposts 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 3.11, and the performance of the HMA in the WB lane from MP 214.05 to 215.23 is shown in Figure 3.12. It is noted that the WSPMS performance curves are not calibrated for the SMA section and should not be used as a prediction of pavement life for the SMA section. The performance evaluation was based on individual data points, instead of the performance curve, in this study.

Table 3.1 C6151 SMA and HMA Sections

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

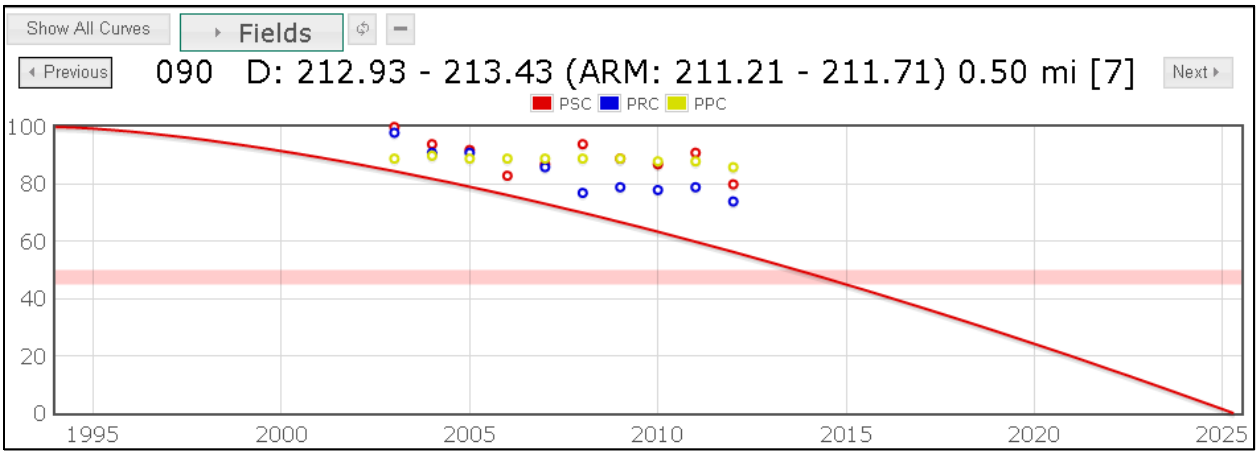


Figure 3.11 C6151 SMA Performance Curves

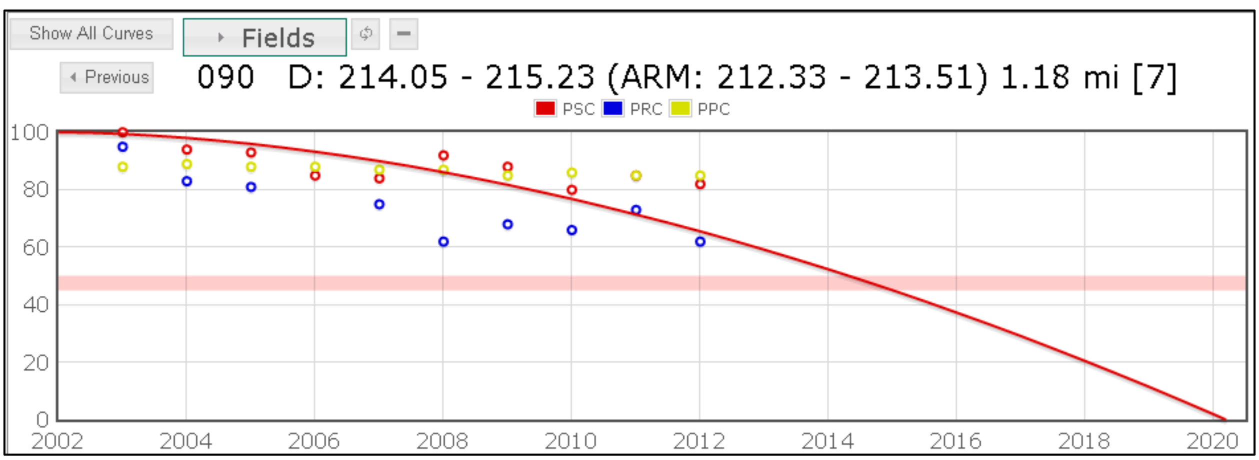


Figure 3.12 C6151 HMA Performance Curves

The SMA in Contract 6151 has performed extraordinarily well for Eastern Washington and cores were taken to compare the laboratory performance of the 1/2" PG 76-28 SMA and 1/2" PG 64-28 HMA. 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. Results of the laboratory tests indicate the SMA section has significantly superior performance over the 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 Figures 3.13 through 3.15. The laboratory performance for the SMA is consistent with its field performance. The SMA has visibly out-performed the adjacent HMA section in the field for 13 years, and may last as long as 20 years. Details of this study and additional test results are provided in Appendix B2. Mix designs of the SMA and HMA pavements from this contract are located in Appendix C2.

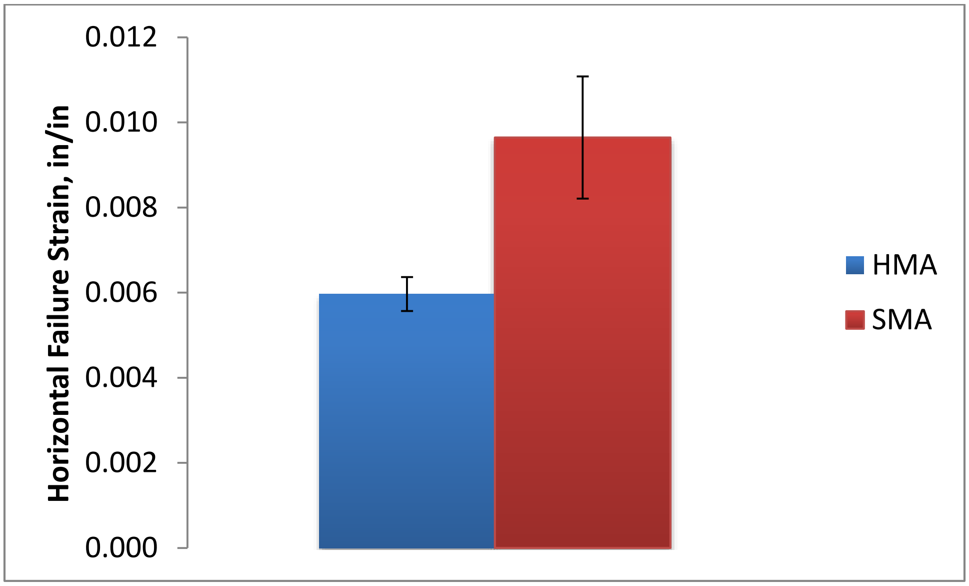


Figure 3.13 Horizontal Failure Strain

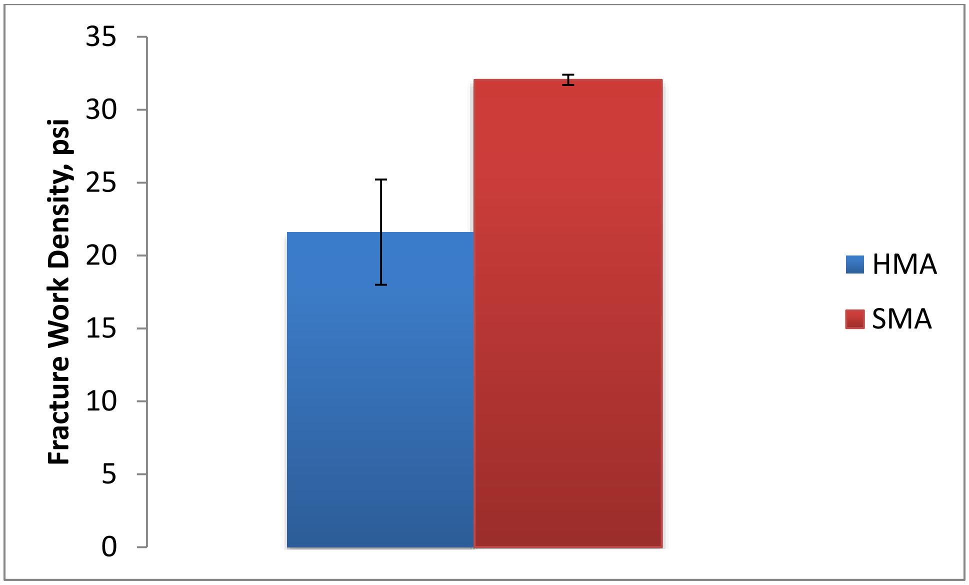


Figure 3.14 Fracture Work Density

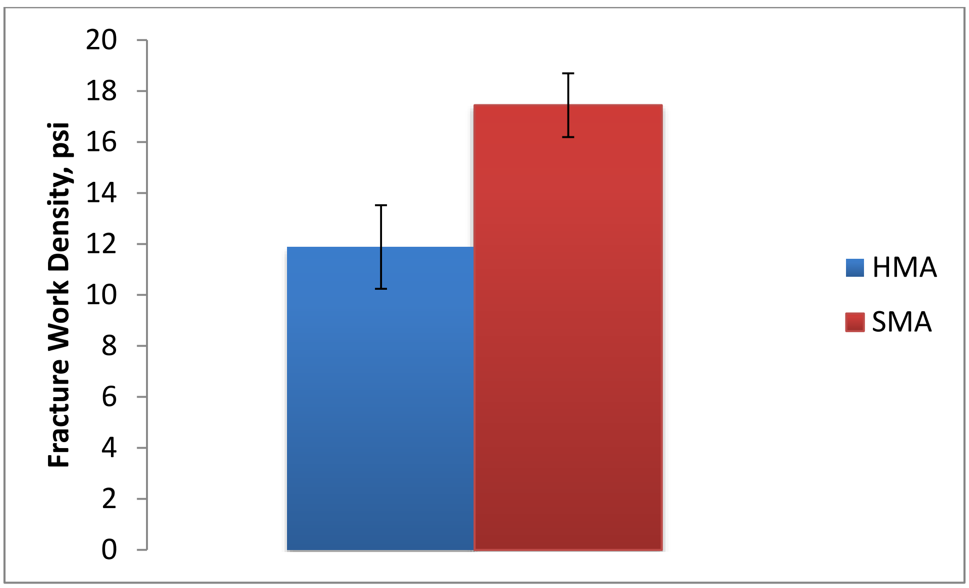


Figure 3.15 Fracture Work Density from IDT Thermal Cracking Test

3.1.3.2 Contract 6687: I-90 from Dodson Road to Moses Lake

In the North Central Region, Contract 6687 was paved in 2004 from Dodson Road to Moses Lake and has an AADT of approximately 9,700. The EB lane is ½" PG 76-28 SMA and the WB lane is ½" PG 64-38 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 was 57% more (Bob Romine, personal communication, April 15, 2014). The WSPMS performance curves show visibly superior rutting performance for the SMA (Figures 3.16 and 3.17).

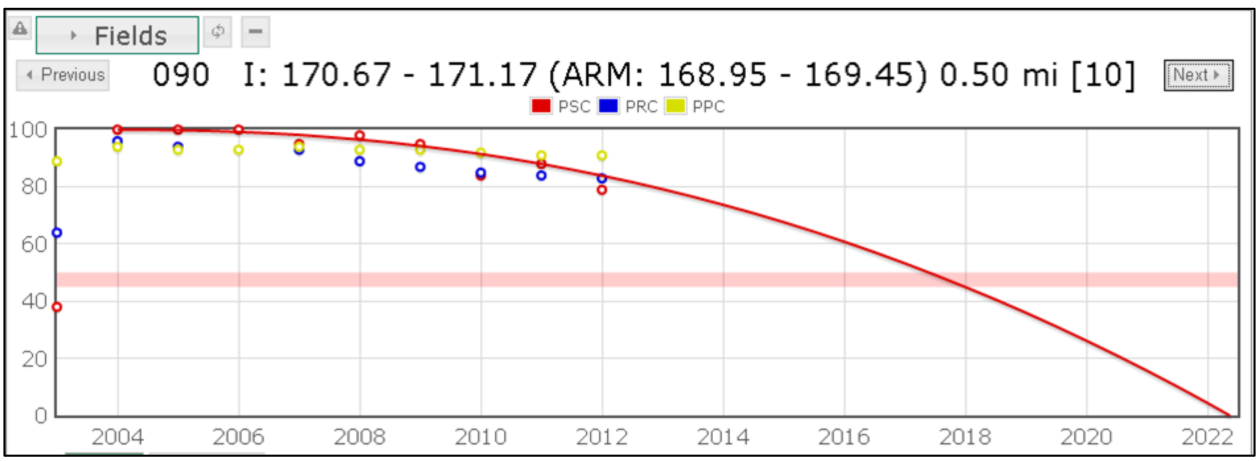


Figure 3.16 C6687 SMA Performance Curves

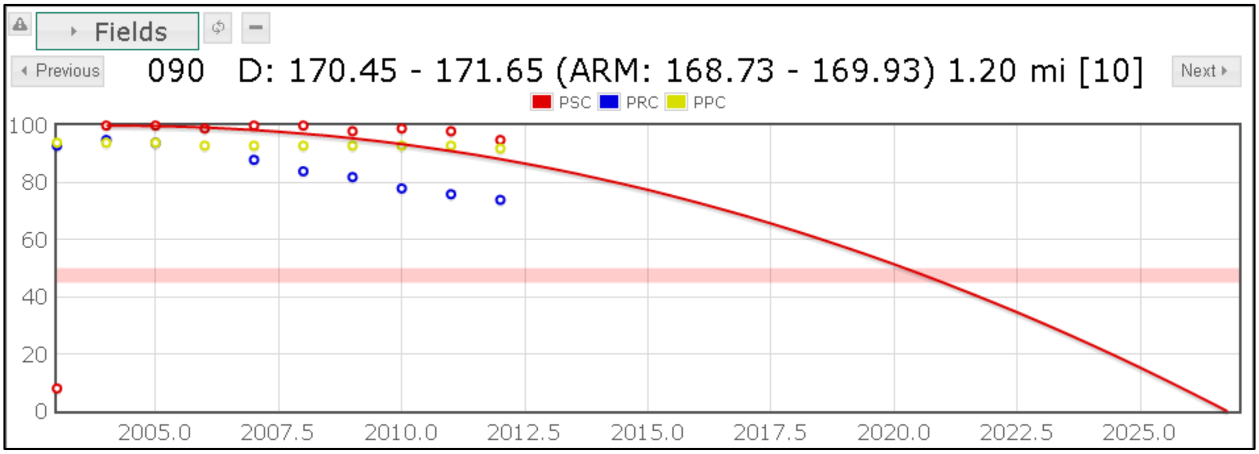


Figure 3.17 C6687 HMA Performance Curves

3.1.4 BST Overlay Projects

The following sections describe HMA pavements that were overlaid with BST within one year of construction. A study was performed to evaluate the effects of applying BST overlays to new HMA pavements. For both the SR 20 and SR 278 projects, there exist HMA sections with BST overlays and HMA sections without BST overlays that were paved at the same time. These sections of HMA without BST were used as control sections. Performance tests were conducted on field cores and binders extracted from the HMA. The parameters evaluated include dynamic modulus |E*|, creep compliance, IDT strength at intermediate temperatures, fracture work density, horizontal failure strain, and binder PG grading. Results of the test indicated that applying a BST overlay effectively protected the underlying HMA from oxidation and reduced the aging of the binder in the underlying HMA. It was also found after the experiments that in almost all cases, fractured aggregates are pronounced. Since these cores are taken in the middle of the traffic lane, this finding indicates that the mixes have been over compacted. It might also be related to the selection of the aggregate skeleton. The fractured aggregate weakens the integrity of mixes and should be avoided. The use of finer gradations or more asphalt mastics may alleviate this problem.

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

In Washington State, Contract 7109 is located on SR 20 and included a pre-level and BST overlay in 2006. The HMA for pre-level was 3/8" PG 64-28 and the BST layer was Class D CRS-2P. The surface was fog sealed. It is noted that WSPMS does not indicate an underlying HMA layer for the Contract 7109. The performance curves from WSPMS of a section of Contract 7109 are shown in Figure 3.18. It can be seen that this section of pavement performs well after seven years in service.

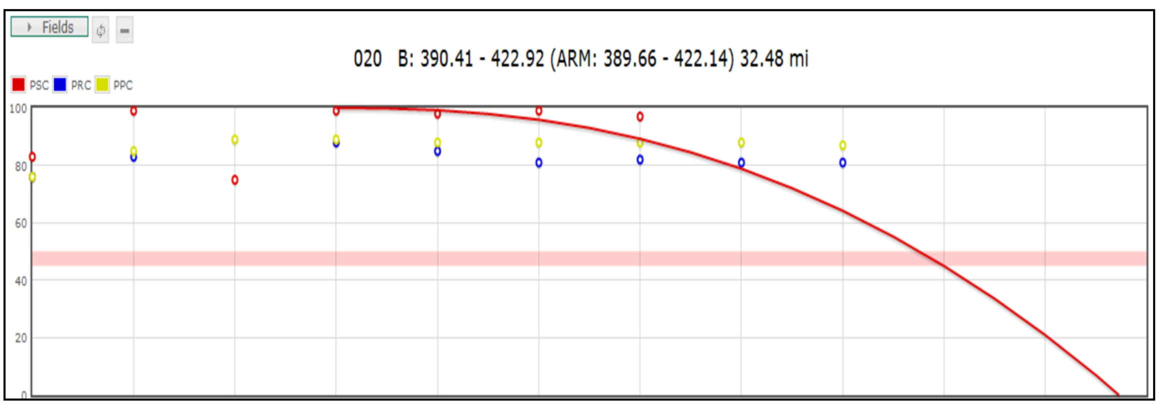


Figure 3.18 Contract 7109 Performance Curves

The section of HMA without BST for this project is located on an approach which may carry different volume of traffic, which may make the comparison of field performance difficult. Therefore, field cores were taken to evaluate the effects of BST on pavement performance. For the SR 20 project, the dynamic modulus and creep compliance test results indicate that the BST overlay kept the underlying HMA softer than the HMA that was exposed to oxidation without a BST overlay. This is indicated by the results of the dynamic modulus, creep compliance, and

IDT strength tests shown in Figures 3.19 through 3.21. The IDT fatigue test results indicate that HMA with BST has greater resistance to top-down fatigue cracking than the HMA without BST, as shown by the results of horizontal failure strain shown in Figure 3.22. Applying a BST overlay effectively protected the underlying HMA from oxidation and reduced the aging of the binder in the underlying HMA. The PG grades of the HMA with a BST overlay and the HMA without a BST overlay are shown in Figure 3.23. Details of this study are provided in Appendix B3. The mix design of the HMA used in Contract 8262 on SR 20 is located in Appendix C3.

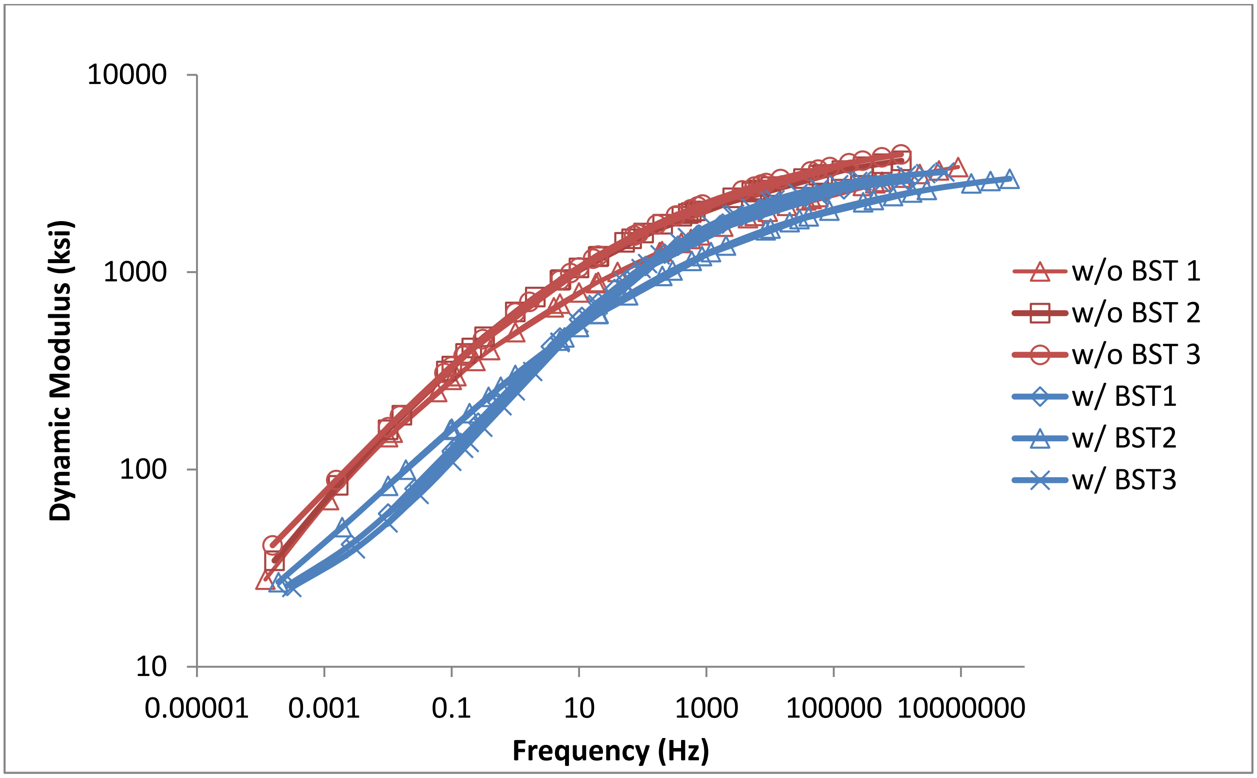


Figure 3.19 SR 20 Dynamic Modulus Master Curves

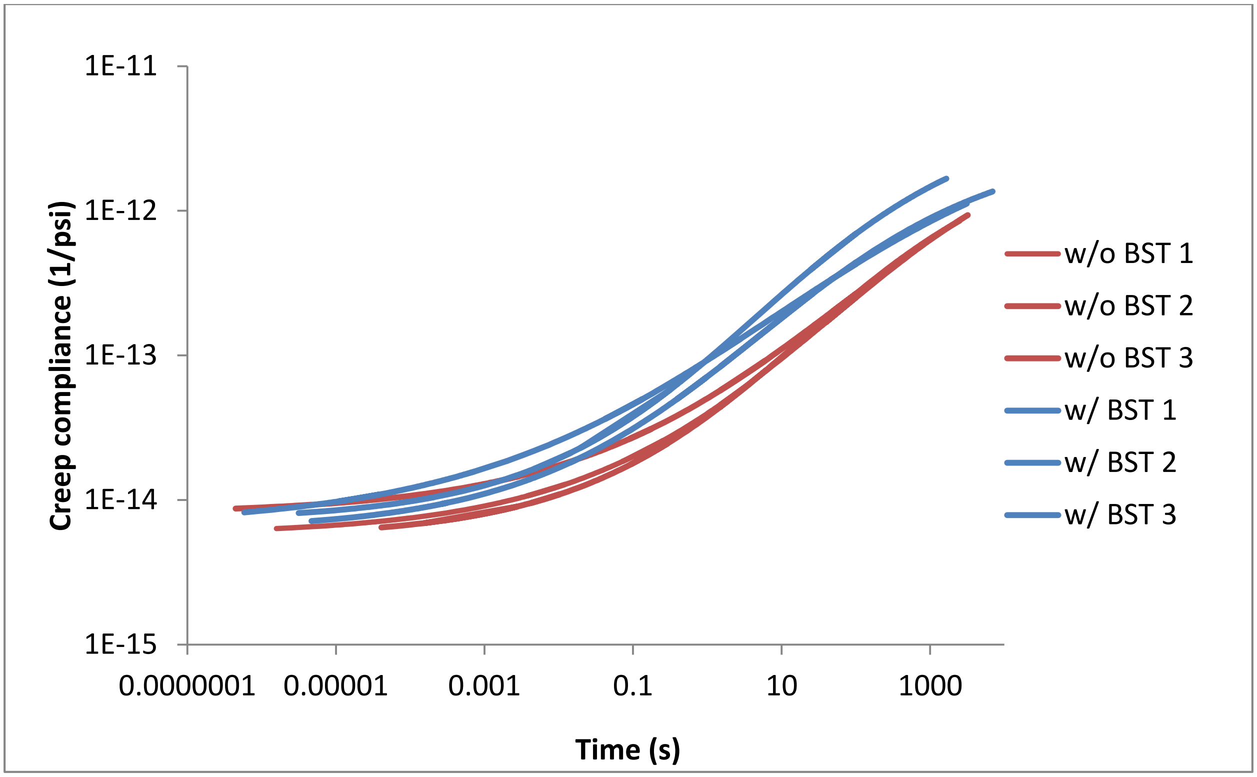


Figure 3.20 SR 20 Creep Compliance Master Curves

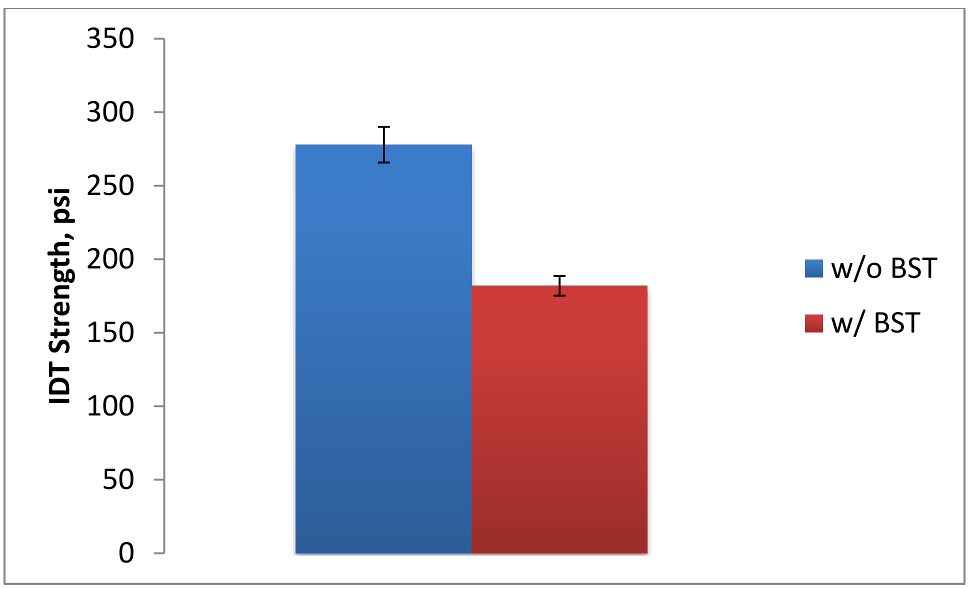


Figure 3.21 SR 20 IDT Strength

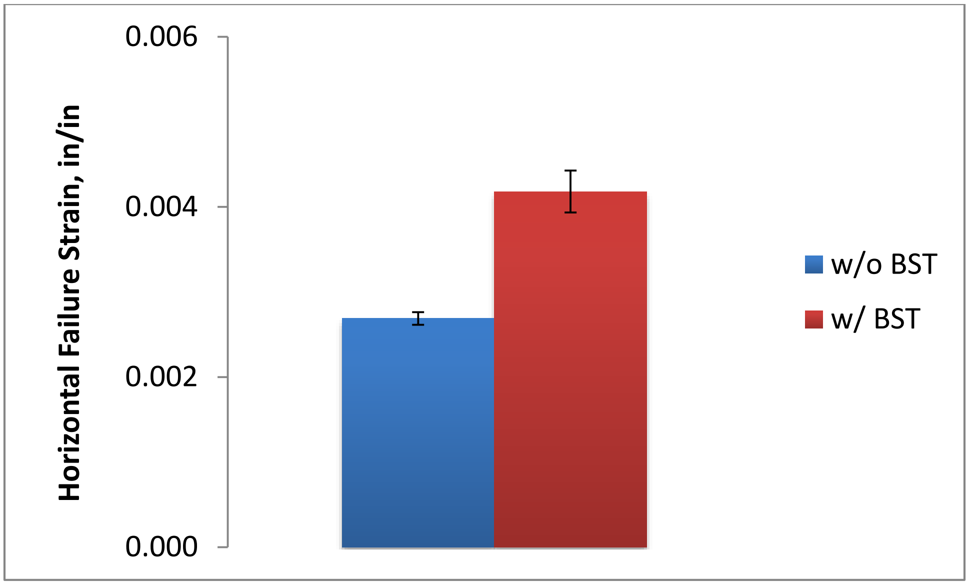


Figure 3.22 SR 20 Horizontal Failure Strain

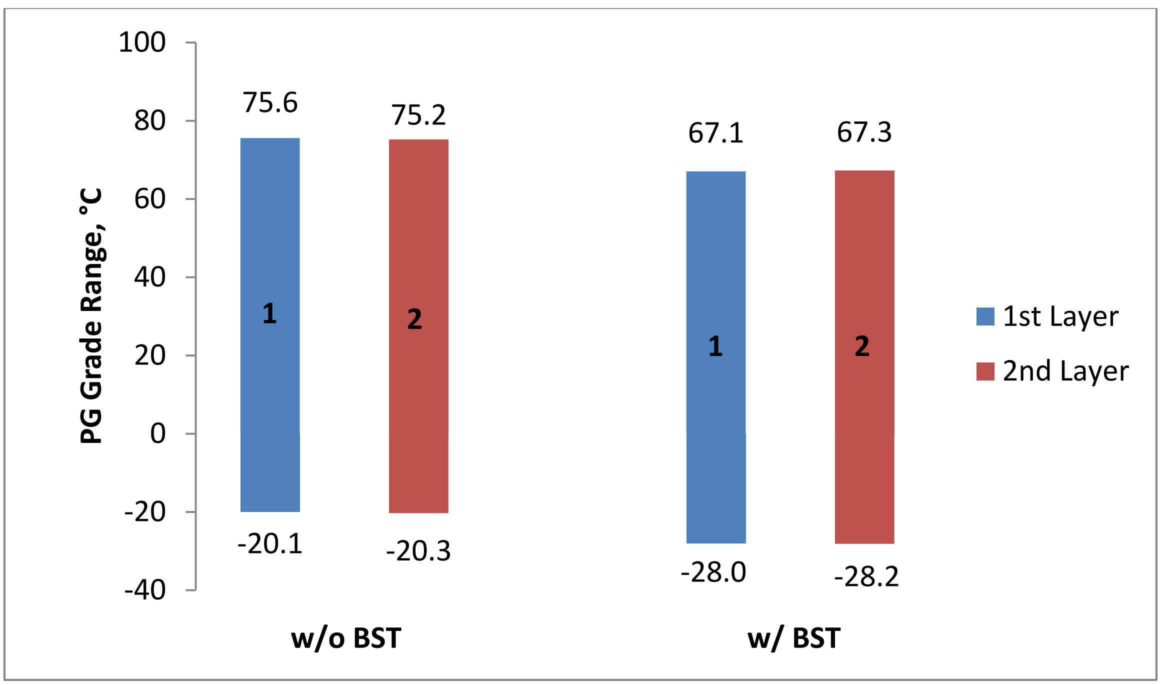


Figure 3.23 SR 20 High and Low PG Grades

3.1.4.2 Contract 8262: US 2 et al Eastern Region Chip Seal 2012

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

For the laboratory evaluation of the SR 278 project, the effects of the BST overlay are not as pronounced as the SR 20 project due to the shorter age since construction. The dynamic modulus and creep compliance test results indicate that the BST overlay kept the underlying HMA softer than the HMA that was exposed to oxidation without a BST overlay, as indicated by the results of the dynamic modulus and creep compliance tests shown in Figures 3.24 and 3.25. Applying a BST overlay effectively protected the underlying HMA from oxidation and reduced the aging of the binder in the underlying HMA, as shown in the results of the PG grades of the asphalt in the HMA with the BST overlay and the HMA without the BST overlay shown in Figure 3.26. Details of this study and additional test results are provided in Appendix B3. The mix design of the HMA used in Contract 8262 is located in Appendix C3.

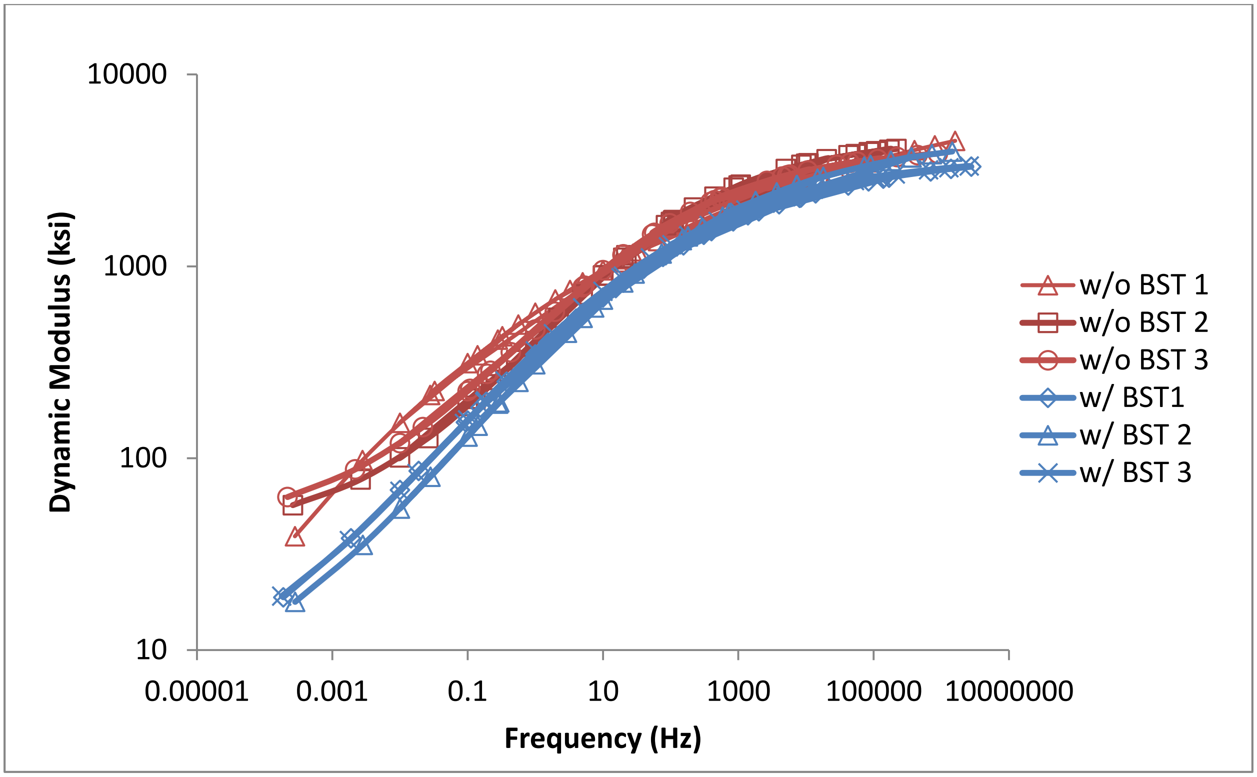


Figure 3.24 SR 278 Dynamic Modulus Master Curves

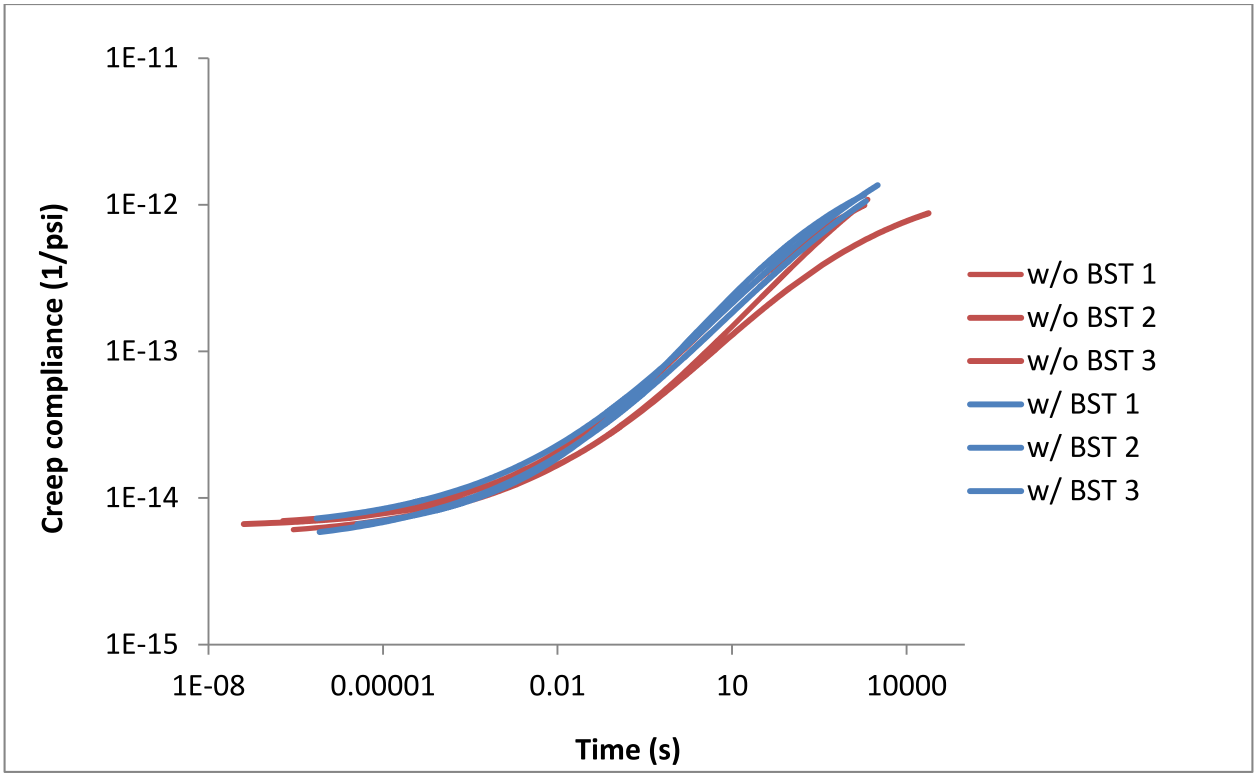


Figure 3.25 SR 278 Creep Compliance Master Curves

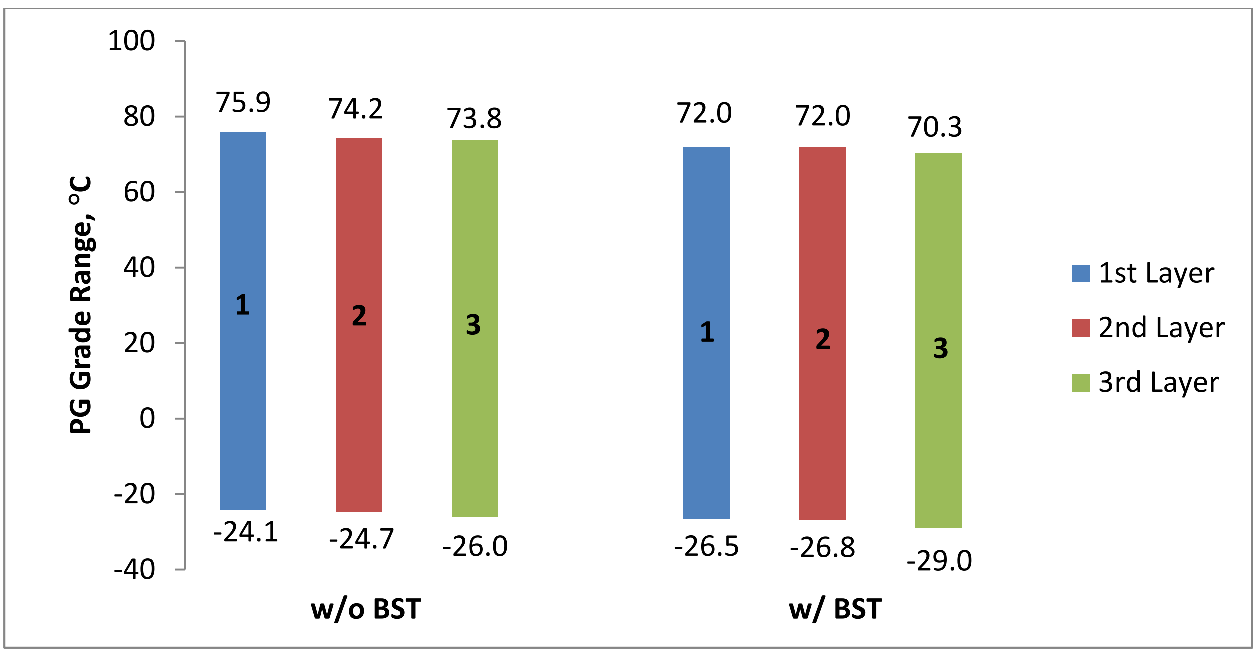


Figure 3.26 SR 278 High and Low PG Grades

CHAPTER 4: COST

Information regarding the cost of materials with potential for further implementation in Eastern Washington was gathered from the literature, survey, and interviews with industry professionals. This information is summarized in this chapter.

4.1 Polymer Modified Asphalt

Polymer modifiers are generally used to increase a binder PG grade from a PG 70 to a PG 76. At the current price of oil, a PG 76 would cost approximately $100 per ton more than a PG 70 (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 11½ years in order to break even on cost, as shown in Table 4.2 at the end of this chapter.

4.2 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 12 years in order to break even on cost, as shown in Table 4.2 at the end of this chapter. According to Adam Hand (personal communication, April 24th, 2014), Caltrans allows the overlay thickness to be halved if cracking is found to be the controlling distress. 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.

4.3 Stone Matrix Asphalt

The cost of SMA in Maryland is about $90 per ton, whereas the cost of hot mix asphalt is generally between $60 and $80 per ton. 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. In Georgia, SMA costs about $90 per ton compared to $60 to $80 per ton for HMA. A review of the cost of recent pavement projects reveals that asphalt pavements make up approximately 55% of the total project cost. As the average life of HMA pavements in Eastern Washington is approximately 11 years, an SMA pavement would need to last approximately 13 years in order to break even on cost, as shown in Table 4.2 at the end of this chapter. 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, as shown in Figure 4.1.

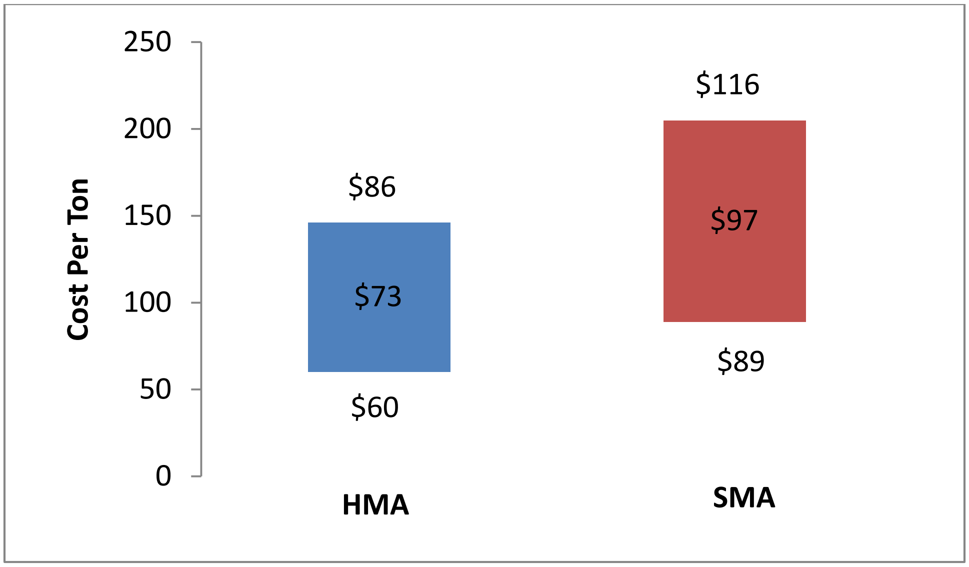


Figure 4.1 HMA and SMA Cost Range

4.4 Cost Analysis

A brief analysis of the prices of several recent paving projects in the WSDOT Eastern Region indicates that HMA comprises approximately 55% of the total project cost, as shown in Table 4.1. An analysis of the life required to break even on cost compared to the average 11 year life of HMA pavements in Eastern Washington was performed for SMA, PMA, and rubberized asphalt, and is shown in Table 4.2. The cost per ton of each material was representative of prices gathered from the literature, survey of state agencies, and interviews of industry professionals. With asphalt pavement comprising 55% of the total project cost, the required life to break even on cost for SMA, PMA, and rubberized asphalt is 13.5, 11.5, and 11.9 years, respectively. With asphalt pavement comprising 100% of the total project cost, the required life to break even on cost for SMA, PMA, and rubberized asphalt is 15.5, 11.9, and 12.7 years, respectively, as shown in Table 4.3.

Table 4.1 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 4.2 Cost Analysis at 55% of Total Project Cost

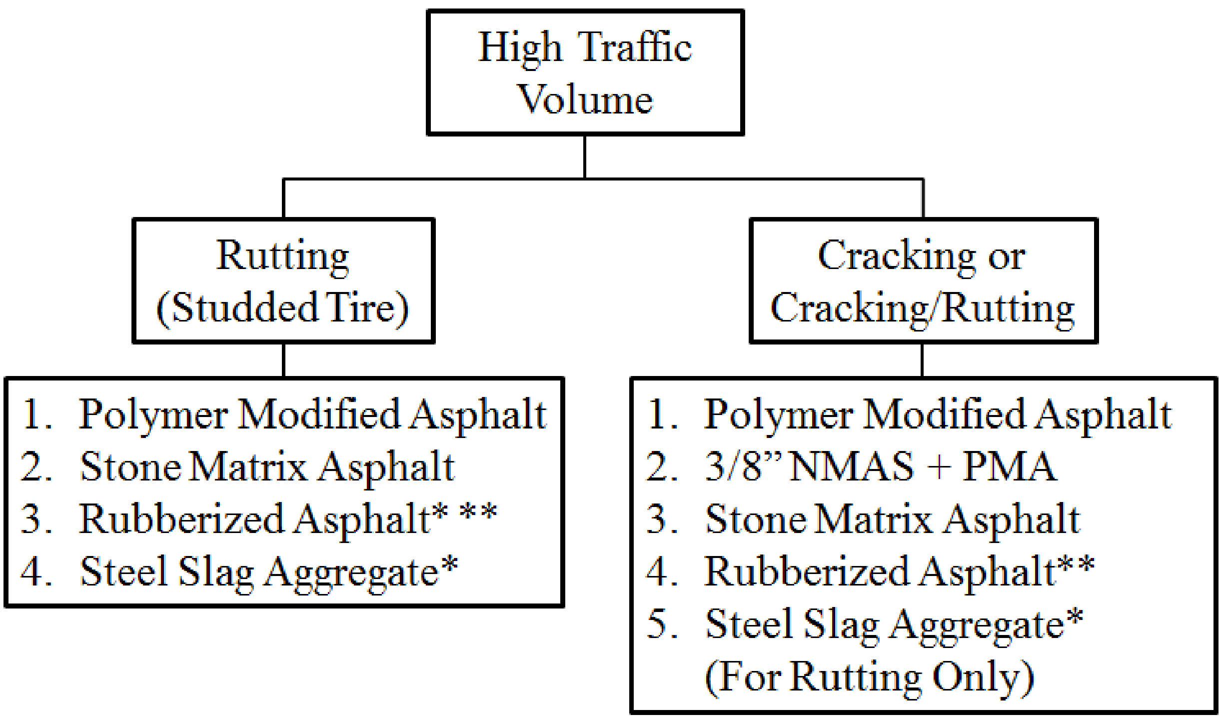
	HMA	SMA	PMA/High PG	Rubberized
Cost/ton	$63.90	$90	$68.90	$73.80
Asphalt Cost Ratio	1	1.4	1.1	1.2
Project Cost Ratio	1	1.2	1	1.1
Life to Breakeven	11	13.5	11.5	11.9

Table 4.3 Cost Analysis at 100% of Total Project Cost

	HMA	SMA	PMA/High PG	Rubberized
Cost/ton	$63.90	$90	$68.90	$73.80
Asphalt Cost Ratio	1	1.4	1.1	1.2
Project Cost Ratio	1	1.4	1.1	1.2
Life to Breakeven	11	15.5	11.9	12.7

CHAPTER 5: CONCLUSIONS AND RECOMMENDATIONS

Recommendations for improving asphalt pavement performance in Eastern Washington are summarized in the flowcharts in Figures 5.1 and 5.2 for high traffic volume and low traffic volume, respectively. The specific recommendations are listed in order of priority. For example, if a high traffic volume pavement section historically has failed due to rutting from studded tire wear, the first recommendation would be to implement polymer modified asphalt. Recommendations for pavements in mountain areas are shown in Figure 5.3 and are not prioritized by traffic volume or historical distress factor.



*It is recommended that test sections be constructed before widespread implementation
**Dry process

Figure 5.1 High Traffic Volume Flowchart

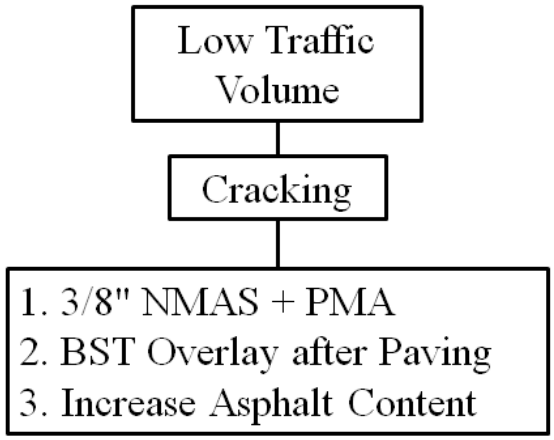
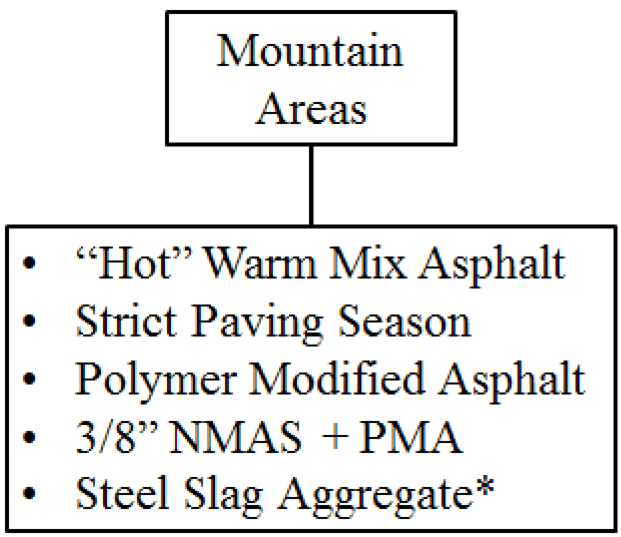


Figure 5.2 Low Traffic Volume Flowchart



*It is recommended that test sections be constructed before widespread implementation

Figure 5.3 Mountain Area Flowchart

It is recommended that the sites that are included in this study, such as the 3/8" mix, high PG mix, BST immediately following the paving, etc. be monitored over time. In addition, performance-based specifications can account for various loading and environmental conditions, which cannot be realized by "recipe" specifications, and are recommended to be implemented.

Specifically, cracking performance tests for mix designs to complement the current HWT test for rutting is recommended to be considered. Multiple stress creep and recovery (MSCR) is also recommended to be included in the specification for asphalt binder to receive polymer modification. In addition, the issue of fractured aggregates by the overcompaction during construction should be investigated and mitigated.

REFERENCES

American Association of State Highway and Transportation Officials. (2007). *Segregation Causes and Cures for Hot Mix Asphalt.* National Asphalt Pavement Association. American Association of State Highway and Transportation Officials: pp. 1-6.

American Association of State Highway and Transportation Officials. (2014). *Standard Specifications for Transportation Materials and Methods of Sampling and Testing, 34th Edition and AASHTO Provisional Standards, 2014 Edition.* American Association of State Highway and Transportation Officials.

Ahmedzade, P. and Sengoz, B. (2009). Evaluation of steel slag coarse aggregate in hot mix asphalt. *Journal of Hazardous Materials* (165): pp. 300-305.

Al-Hadidy, A.I. and Yi-qiu, T. (2010). Comparative Performance of the SMAC Made with the SBS- and ST-Modified Binders. *Journal of Materials in Civil Engineering*, 22(6): pp. 580-587.

Anderson, H. (2002). *PMA Field Performance on UDOT Projects.* International Denter for Aggregates Research (ICAR) 2002 10th Annual Symposium Research Papers.

Anderson, K.W. (1997). *Rubber-Asphalt Open-Gaded Friction Course I-5 Columbia River Bridge to 39th Street*. Final Report WA86-10. Washington State Department of Transportation.

Anderson, K.W., Pierce, L.M., and Uhlmeyer, J.S. (2008). *A Brief History of the Performance of Rubberized Pavements In Washington State.* Special Report No. WA-RD 693.1

Anderson, K.W., Uhlmeyer, J.S., Sexton, T., Russell, M., and Weston, J. (2013). *Evaluation of Long-Term Pavement Performance and Noise Characteristics of Open-Graded Friction Courses – Project 3: Final Report.* Washington State Department of Transportation. WA-RD 749.2.

Aschenbrener, T., Schiebel, B., and West, R. (2011). *Three-Year Evaluation of the Colorado Department of Transportation's Warm-Mix Asphalt Experimental Feature on I-70 in Silverthorne, Colorado.* NCAT Report 11-02.

Asi, I.M. (2006). *Laboratory comparison study for the use of stone matrix asphalt in hot weather climates.* Construction and Building Materials, 20(10): pp. 982-989. DOI:10.1016/j.conbuildmat.2005.06.011

Asi, I.M., Qasrawi, H.Y., and Shalabi, F.I. (2007). Use of steel slag aggregate in asphalt concrete mixes*.* *Canadian Journal of Civil Engineering* (34): pp. 902-911. DOI:10.1139/L07-025

Ayyala, D., Qarouach, S., Khosla, N.P., and Tayebali, A.A. (2014). *An Investigation of Ndesign Values for Superpave Surface Mixtures.* AAPT 2014 Annual Meeting, Atlanta, GA, March 16-19, 2014.

Berger, E., and Huege, F. (2006). *Achieving longer lasting road surfaces with the use of Hydrated Lime: A Multi-functional Additive for Hot Mix Asphalt.* Submitted to the International Lime Association, 11th International Lime Congress, Prague, CZ, May 17-18, 2006.

Bethard, T. and Zubeck, H. (2002). *Polymer-Modified Asphalt Emissions from Alaskan Hot Plants – A Questionnaire Study.* Cold Regions Engineering, ASCE 2002: pp. 336-347. DOI:10.1061/40621(254)28

Bingham, N., Saboundjian, S., and Brunette, B. (2010). *Use of Rubber-Modified Hot-Mix Asphalt to Reduce Studded Tire Wear and Plastic Deformation.* Alaska Department of Transportation Report Number FHWA-AK-RD-10-03

Bode, T. (2012). *An Analysis of the Impacts of Temperature Segregation on Hot Mix Asphalt.* University of Nebraska Masters Thesis.. Construction Systems – Dissertations & Theses. Paper 10. http://digitalcommons.unl.edu/constructiondiss/10

Brown, A.S. (2007). *Polymer Modified Asphalt The Canadian Experience*. Ontario Hot Mix Producers Association. http://amap.ctcandassociates.com/wp/wp-content/uploads/05-Brown-2007-02-12-AMAP-Boston-Polymer-Modified-Asphalt-v1.pdf

Chen, J., Liao, M., and Shiah, M. (2002). Asphalt Modified by Styrene-Butadiene-Styrene Triblock Copolymer: Morphology and Model. *Journal of Materials in Civil Engineering*, 14(3): pp. 224-229.

Cheng, D. and Hicks, R.G. (2012). *Life Cycle Cost Comparison of Rubberized and Conventional HMA in California.* California Department of Recources Recycling and Recovery. May, 2012.

Christensen, D.W. and Bonaquist, R.F. (2005). *VMA: One Key to Mixture Performance.* Submitted to the South Central Superpave Center for Publication in the National Superpave Newsletter. February, 2005.

Christensen, D.W. and Bonaquist, R.F. (2006). *Volumetric Requirements for Superpave Mix Design.* NCHRP Report 567. TRB, 2006.

Chuanfeng, Z. and Yazhi, X. (2011). *Using Seam Asphalt Mixtures in Surface Course in Cold Areas*. ICTE 2011: pp. 1481-1486. DOI:10.1061/41184(419)245

Clyne, T., Johnson, E.. McGraw, J., and Reinke, G. (2012). *Field Investigation of Polyphosphoric Acid-Modified Binders at MnROAD.* Transportation Research Circular No. E-C160, January 2012. Polyphosphoric Acid Modification of Asphalt Binders, *A Workshop*, April 7-8, 2009: pp. 115-130.

Cohen, J. (1992). *A Power Primer*. Psychological Bulletin, Vol. 112, No. 1, pp: 155-159.

Collet, A. (2012). Hydrated Lime in Hot Mix Asphalt. Epsilon Ingenierie.

Deb, S. (2012). *Polymer Modified Asphalt – A Solution to Many Asphalt Problems.* The Masterbuilder, November, 2012, pp: 194-198

Federal Highway Administration. (2011). *A Manual for Design of Hot Mix Asphalt with Commentary.* NCHRP Report 673. Transportation Research Board.

Fromm, H.J. and Corkill, J.T. (1971). *An evaluation of surface course mixes designed to resist studded tire wear.* Downsview, Ontario, Research and Transportation Systems Branch, Ontario Department of Highways.

Galehouse, L., King, H., Leach, D., Moulthrop, J., and Ballou, B. (2005). *Preventive Maintenance Treatment Performance at 14 Years:* pp. 19

Glanzman, T. (2005). *Quantifying the Benefits of Polymer Modified Asphalt.* Asphalt, Volume 20, Issue 1, Spring, Asphalt Institute: pp. 18-20.

Gogula, A., Hossain, M., Boyer, J., and Romanoschi, S.(2003). *Effect of PG Binder Grade and Source on Performance of Superpave Mixtures under Hamburg Wheel Tester.* Proceedings of the 2003 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2003. Iowa State University.

Goh. S. and You, Z. (2009). *Warm Mix Asphalt using Sasobit® in Cold Region*. Cold Regions Engineering 2009. ASCE 2009: pp. 288-298. DOI:10.1061/41072(359)29

Göransson, N. and Jacobson, T. (2013). *Steel Slag in Asphalt Paving Field Studies 205-2012*. VTI Note 19, 2013.

Hicks, G., Cheng, D., Zubeck, H., Liu, J., and Mullin, A. (2012). *Develop Guidelines for Pavement Preservation Treatments and for Building a Pavement Preservation Platform for Alaska.* INE/AUTC 12.07, FHWA Report No. FHWA-AK-RD-12-14.

Huang, S., Robertson, R.E., Branthaver, J.F., and Petersen, J.C. (2005). Impact of Lime Modification of Asphalt and Freeze-Thaw Cycling on the Asphalt-Aggregate Interaction and Moisture Resistance to Moisture Damage. *Journal of Materials in Civil Engineering, ASCE, November/December 2005*, (17): pp. 711-718. DOI:10.1061/(ASCE)0899-1561(2005)17:6(711)

Hunt, L. and Boyle, G.E. (2000). *Steel Slag in Hot Mix Asphalt Concrete.* State Research Project #511, Report No. OR-RD-00-09, April 2000.

Hurley, G.C. and Prowell, B.D. (2006). *Evaluation of Evotherm® for Use in Warm Mix Asphalt Mixes.* NCAT Report No. 06-02, National Center for Asphalt Technology, Auburn, Alabama.

Illinois Department of Transportation. (2005). *Pavement Technology Advisory: Polymer Modified Hot Mix Asphalt.* Design, Construction and Materials PTA-D5, Illinois Department of Transportation, Bureau of Materials and Physical Research.

Johnson, G. (2012). *Intelligent Compaction and Pave-IR in Minnesota.* PowerPoint presentation. 2012 NCAUPG Technical Conference, February 16, 2012.

Kandhal, P.S., Ramirez, T.L., and Ingram, P.M. (2002). *Evaluation of Eight Longitudinal Joint Construction Techniques for Asphalt Pavements in Pennsylvania.* National Center for Asphalt Technology at Auburn University. February, 2002.

Kennedy, T. W. and Anagnos, J.N. (1984). *A Field Evaluation of Techniques for Treating Asphalt Mixtures With Lime.* Research Study 3-9-79-253. Report No. FHWA/TX-85//47+253-6, November, 1984.

Kim, Y.R. and Wen, H. (2002). Fracture Energy from Indirect Tensile Test. *Journal of the Association of Asphalt Paving Technologists*, Vol. 71, pp. 779-793.

Khosla, P.N. and Ayyala, D. (2013). *A Performance-based Evaluation of Superpave Design Gyrations for High Traffic Surface Mixes.* 2nd Conference of Transportation Research Group of India. 104: pp. 109-118. DOI: 10.1016/j.sbspro.2013.11.103

Kuennen, T. (2005). *Making high-volume roads last longer.* Better Roads, 75(4), April, 2005: pp. 50-64.

Larson, M. (2010). *An Incentive To Take Asphalt's Temperature.* Engineering News-Record. 18 August 2010. Web. 24 November, 2013.

Lewandowski, L. (2004). *Historical Performance of Polymer Modified Asphalt Pavements: Part I.* Goodyear Chemical, June, 2004.

Li, X. and Gibson, N. (2011). Mechanistic Characterization of Aggregate Packing to Assess Gyration Levels During HMA Mix Design. *Journal of the Association of Asphalt Paving Technologists*, 80: pp. 33-64.

Liu, J., and Li, P. (2012). Low Temperature Performance of Sasobit®-Modified Warm-Mix Asphalt. *Journal of Materials in Civil Engineering*, 24(1): pp. 57-63.

McPherson, E.G. and Muchnick, J. (2005). Effects of Street Tree Shake on Asphalt Concrete Pavement Performance. *Journal of Arboriculture*, 31(6): November 2005P pp. 303-310.

Montana Department of Transportation. (2006). *Standard Specifications for Road and Bridge Construction.* Section 701 Aggregates. Montana Department of Transportation. 2006.

Michael, L., Burke, G., and Schwartz, C.W. (2003). *Performance of Stone Matrix Asphalt Pavements in Maryland.* Asphalt paving technology, 72: pp. 287-314.

Miller, J.S. and Bellinger, W.Y. (2003). *Distress Identification Manual for the Long-Term Pavement Performance Program (Fourth Revised Edition).* FHWA Report No. FHWA-RD-03-031.

Mills, B., Tighe, S., Andrey, J., Huen, K., and Parm, S. (2006). *Climate Change and the Performance of Pavement Infrastructure in Southern Canada: Context and Case Study*. IEEE EIC Climate Change Technology: pp. 1-9.

Mohammad, L.N. and Shamsi, K.A. (2007). *A Look at the Bailey Method and Locking Point Concept in Superpave Mixture Design*. Transportation Research Circular, Number E-C124, December 2007: pp. 12-32.

Mathy Technology & Engineering Services, Inc. (2001). *Study of Binder & Mix Properties of 5 Mixtures Using Weld County, Colorado Airport Aggregate.* Dupont Elvaloy Research Report.

Murphy, T.R. (2013). *Stone Matrix Asphalt and Perpetual Pavements.* PowerPoint presentation, Idaho Asphalt Conference, October 24, 2013, Moscow, Idaho.

Myers, N. (2007). *Stone Matrix Asphalt – The Washington Experience*. (Master's thesis). University of Washington.

National Slag Association (NSA). (n.d.). *Steel Furnace Slag SMA Mix Proves to be "The World's Strongest Intersection"* NSA Document 203-1, http://www.nationalslag.org/sites/nationalslag/files/documents/nsa_203-1_worlds_strongest_intersection.pdf

Newcomb, D.E. (2009). *Thin Asphalt Overlays for Pavement Preservation.* National Asphalt Pavement Association Information Series 135, July, 2009.

National Lime Association. (2006). *Hydrated Lime – A Solution for High Performance Hot Mix Asphalt.* Lime: The Versatile Chemical Fact Sheet. National Lime Association, November, 2006: pp. 1-4.

Papagiannakis, A.T. and Lougheed, T.J. (1995). *A Review of Crumb-Rubber Modified Asphalt Concrete Technology.* Research Report for Research Project T9902-09 "Rubber Asphalt Study."

Pavement Interactive. (2009). *Longitudinal Joint Construction*. 2 April, 2009. Web. 24 November 2013. http://www.pavementinteractive.org/article/longitudinal-joint-construction/

Peterson, C. and Anderson, H. (1998). *Interstate 70 Polymerized Asphalt Pavement Evaluation, unpublished Report, Utah Department of Transportation, Materials Division, February 1998.*

Prowell, B. and Brown, R. (2007). *Superpave Mix Design: Verifying Gyration Levels in the Ndesign Table.* NCHRP Report 573. Transportation Research Board. National Research Council, Washington D.C., 2007.

Qi, X., Sebaaly, P.E., Epps, J.A. (1995). Evaluation of Polymer-Modified Asphalt Concrete Mixtures. *Journal of Materials in Civil Engineering*, 7(2): pp. 117-124.

Raad, L., Saboundjian, S., Sebaaly, P., Epps, J., Camilli, B. and Bush, D. (1997). *Low Temperature Cracking of Modified AC Mixes in Alaska.* Report No. INE/TRC 97.05, AKDOT&PF No. SPR-95-14.

Robjent, L., and Dosh, W. (2009). *Warm-Mix Asphalt for Rural Country Roads.* Cold Regions Engineering 2009. ASCE 2009: pp. 438-454.

Rolt, J. (2001). *Top-down Cracking: Myth or Reality*. The World Bank Regional Seminar on Innovative Road Rehabilitation and Recycling Technologies. Amman, Jordan, 24-26 October, 2000.

Root, R.E. (2009). *Investigation of the use of Open-Graded Friction Courses in Wisconsin.* Wisconsin Highway Research Program, WHRP 09-01, SPR# 0092-07-01.

Roschen, T. (2014). *2014 Tire Conference.* CalRecycle Rubberized Asphalt Concrete (RAC) Engineering and Technical Assistance.

Russell, M., Uhlmeyer, J., Anderson, K., Weston, J., De Vol, J, and Baker, T. (2010). *Interstate 90 East of Snoqualmie Pass Hot Mix Asphalt Pavement Forensic Evaluation.* Environmental and Engineering Programs, Materials Laboratory – Pavements Division.

Saboundjian, S. and Raad, L. (1997). *Performance of Rubberized Asphalt Mixes in Alaska.* TRB 1997 Annual Meeting.

Schiebel, B. (2011). *CDOT WMA Project and Specifications Summary*. PowerPoint presentation. 2011 Rocky Mountain Asphalt Conference and Equipment Show.

Stroup-Gardiner, M. and Newcomb, D.E. (1995). *Polymer Literature Review*. Minnesota Department of Transportation Report Number MN/RC-95-27, September, 1995.

Takallou, H.B., Hicks, R.G., Esch, D.C. (1987). *Use of Rubber-Modified Asphalt in Cold Regions*. Prepared for Workshop on Paving in Cold Areas, Ottawa, Ontario, July 20-24, 1987.

Texas Department of Transportation. (n.d.). *Use of Tapered Longitudinal Joints such as the Notched Wedge Joint.* Technical Advisory. Texas Department of Transportation.

Van Hampton, T. (2009). *Intelligent Compaction Is on a Roll.* Engineering News-Record. 08 July, 2009. Web. 24 November, 2013.

Veiteknisk Institutt. (2011). *Norwegian WMA Project - Low Temperature Asphalt 2011 Main Report.*

Von Quintus, H. and Mallela, J. (2005). *Reducing Flexible Pavement Distress In Colorado Through the use of PMA Mixtures.* Final Report No. 16729.1/1. Colorado Asphalt Pavment Association.

Von Quintus, H. and Moulthrop, J.S. (2007). *Mechanistic-Empirical Pavement Design Guide Flexible Pavement Performance Prediction Models For Montana: Volume I Executive Research Summary.* Report No. FHWA/MT-07-008/8158-1.

Washington Asphalt Pavement Association. (2010). *Condition Rating Systems*. WAPA 2010. http://www.asphaltwa.com/2010/10/01/pavement-evaluation-condition-rating-systems/

Wen, H. (2013). *Comparison of Long-Term Field Performance between HMA and WMA Pavement.* PowerPoint presentation. October 31, 2013.

Wen, H. (2013). Use of fracture work density obtained from indirect tensile testing for the mix design and development of a fatigue model. *International Journal of Pavement Engineering,* 14.6 (2013): 561-68. DOI: 10.1080/10298436.2012.729061

Wen, H. and Bhusal, S. (2014). Towards Development of Asphaltic Materials to Resist Studded Tire Wear. *Transportation Research Record: Journal of Transportation Research Board,* No. 2446. Transportation Research Board of the National Academies, Washington D.C., 2015. pp. 78-88

Wen, H. and Bhusal, S. (2014). *Development of a Phenomenological Top-down Cracking Initiation Model for Mechanistic-Empirical Pavement Design.* Paper accepted for publication by Transportation Research Records, 2014.

Wen, H. and Kim, R.Y. (2002). *Simple Performance Test for Fatigue Cracking and Validation with WesTrack Mixtures.* Transportation Research Record 1789, Paper No. 02-2924, Transportation Research Board.

Williams, S.G. (2011). *HMA Longitudinal Joint Evaluation and Construction.* TRC-0901 Final Report. February, 2011. http://www.pavetechinc.com/downloads/TRC0801FinalReport.pdf

Willoughby, K. (2000). *SR 90 Hyak Vicinity to Ellensburg – Phase 2 MP 55.50 to MP 67.40. Contract 005306 Post Construction Report.* Washington State Department of Transporation.

Washington State Department of Transportation. (2012). *Average Pavement Life in Washington State* (2012) Part of the WSDOT Pavement Notebook. www.wsdot.wa.gov/Business/MaterialsLab/Pavements/PavementNotebook.htm.

Washington State Department of Transportation. (2012). *Estimate of Annual Studded Tire Damage to Asphalt Pavements.* Technical Brief. Washington State Department of Transportation – Engineering and Regional Operations, Construction Division, State Materials Laboratory.

Washington State Department of Transportation. (2012) *Hamburg Wheel Tracking Device Update*. Engineering and Regional Operations Construction Division, State Materials Laboratory. April, 2012.

Zhou, H., Nodes, S., and Nichols, J. (1993). *Field Test of Polymer Modified Asphalt Concrete: Murphy Road to Lava Butte Section.* Final Report, FHWA Experimental Project No. 3, Report No. FHWA-OR-RD-96-05.

APPENDICES

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.

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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, Thermal cracking, Fatigue cracking
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" 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)?

____

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[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" HMA and 5/8" RHMA-G (Rubberized HMA - Gap Graded)
Colorado	Typically 1/2" SMA's in the Metro Areas where traffic/truck volumes are higher
Delaware	9.5mm, 12.5mm and more 4.75mm (for 3/4" thin overlays)
Georgia	1-1/2" 9.5mm SP, 1-12/" 12.5mm SP, on interstates we use 3/4" or 1-1/4" of 12.5mm open graded mix
Kentucky	0.38 inch Superpave Surface
Michigan	3/8"
Minnesota	Has been 3/4", moving to 1/2" 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"
Nevada	3/4" thick Open-Graded mix
North Carolina	1.5" S9.5C
Oklahoma	1/2" NMS
Tennessee	Either 1/2" NMAS dense-grade or 1/2" NMAS OGFC
Utah	SMA, 1/2" 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)?

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[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)?

____

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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" HMA and 5/8" RHMA-G (Rubberized HMA - Gap Graded)
Colorado	1/2"
Illinois	We have abandoned 1/2" and are focusing on 9.5mm, looking to use more SMAs
Maryland	SMA for interstate, 1.5" 9.5mm dense or 2" 12.5mm dense for others
Michigan	3/8"
Minnesota	Has been 3/4", moving to 1/2" or using Nova Chip
Nevada	3/4" thick open-graded mix
Ohio	12.5mm for high traffic, 9.5 or similar for low traffic
Utah	SMA, 1/2", 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)?

____

____

[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"
Illinois	Longitudinal joints continue to be an issue. Recently pave and trim 6" 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"
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.15 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.

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

where:

v	= Poisson's ratio
α_1, α_2, α_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

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

where:

$\varepsilon_{x=0}$	= strain at the center of the sample, in/in
γ_1, γ_2, γ_3, γ_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.

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

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.

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

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.

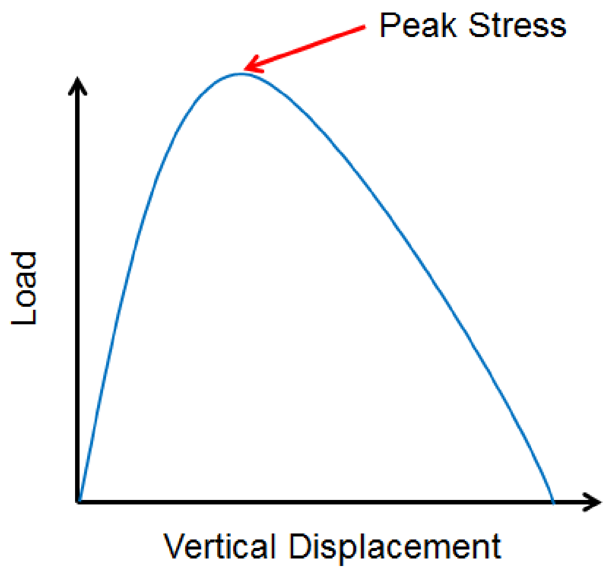

Figure B.56 IDT Strength

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.

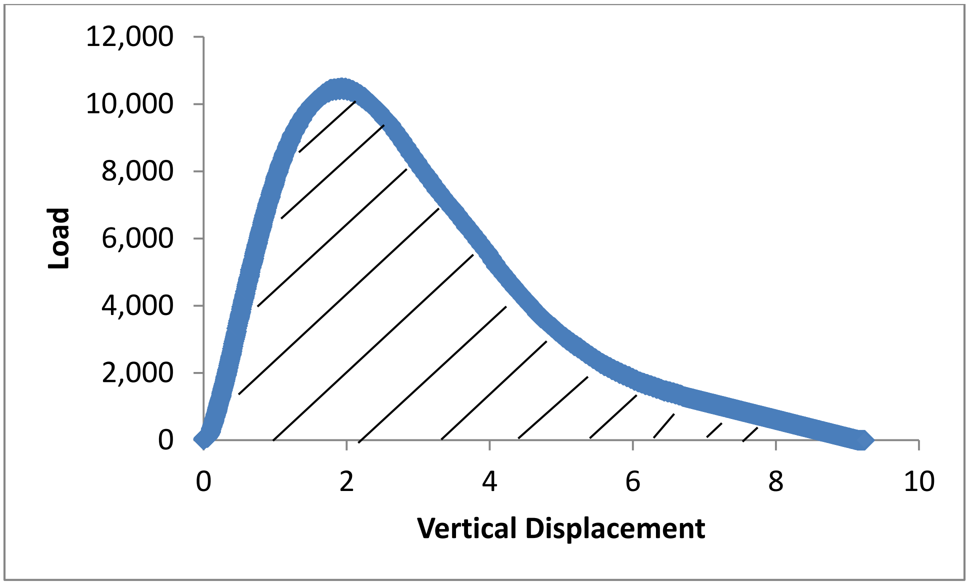

Figure B.6 IDT Fracture Work

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.

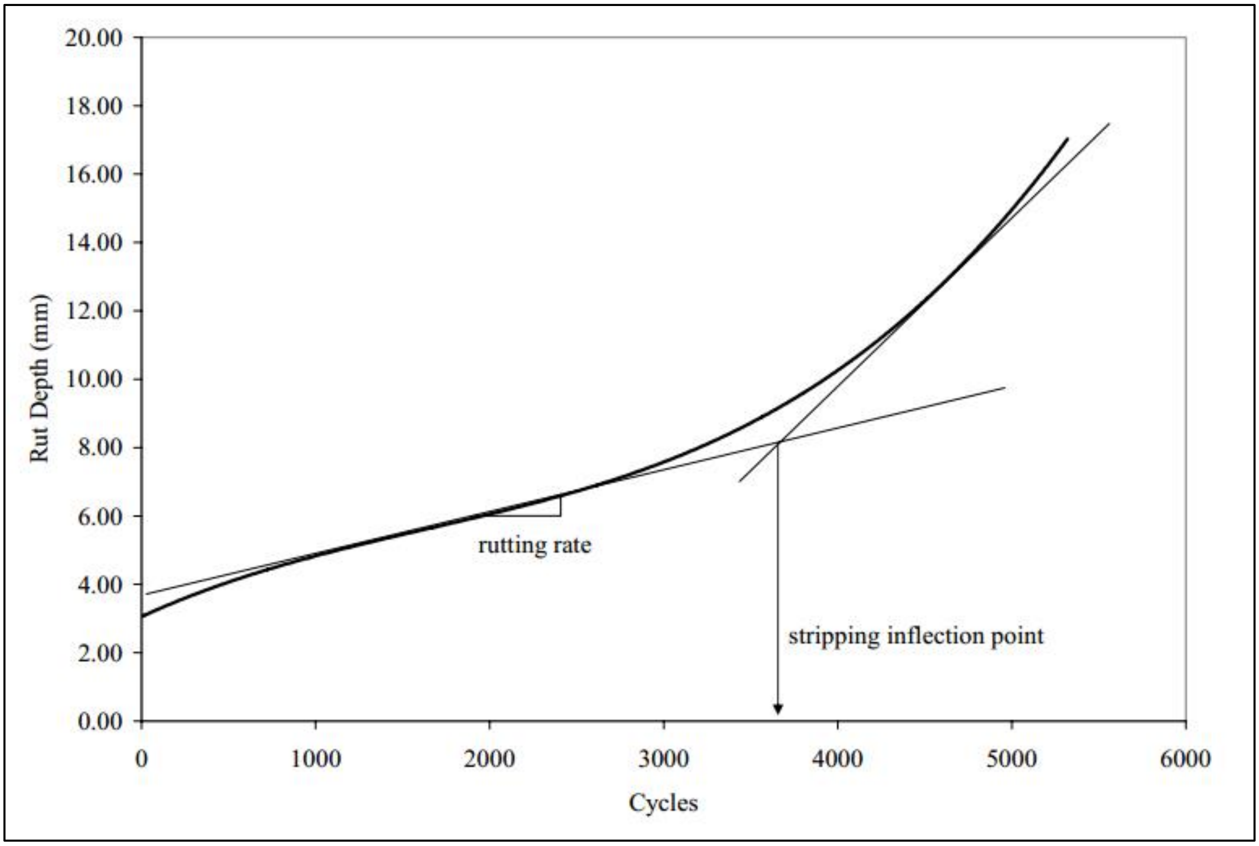

Figure B.7 Hamburg Test Results (Hurley and Prowell, 2006)

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-filmed 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” VS. 1/2” NMAS PROJECT

B1.1 Project Description

In order to determine the effect of using a smaller nominal maximum aggregate size, a 3/8” NMAS asphalt mixture was compared to a conventional 1/2” NMAS asphalt mixture. A test section of 3/8” NMAS HMA was constructed on I-90 next to a 1/2” 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” PG 70-28 HMA and the rest of the project was paved with 1/2” PG 70-28 HMA. The 1/2” asphalt mixture contains 4.9 percent binder, and the 3/8” 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” and 3/8” mixes had average air void contents of 6.3% and 6.1%, respectively. Field cores of the 1/2" and 3/8” 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).

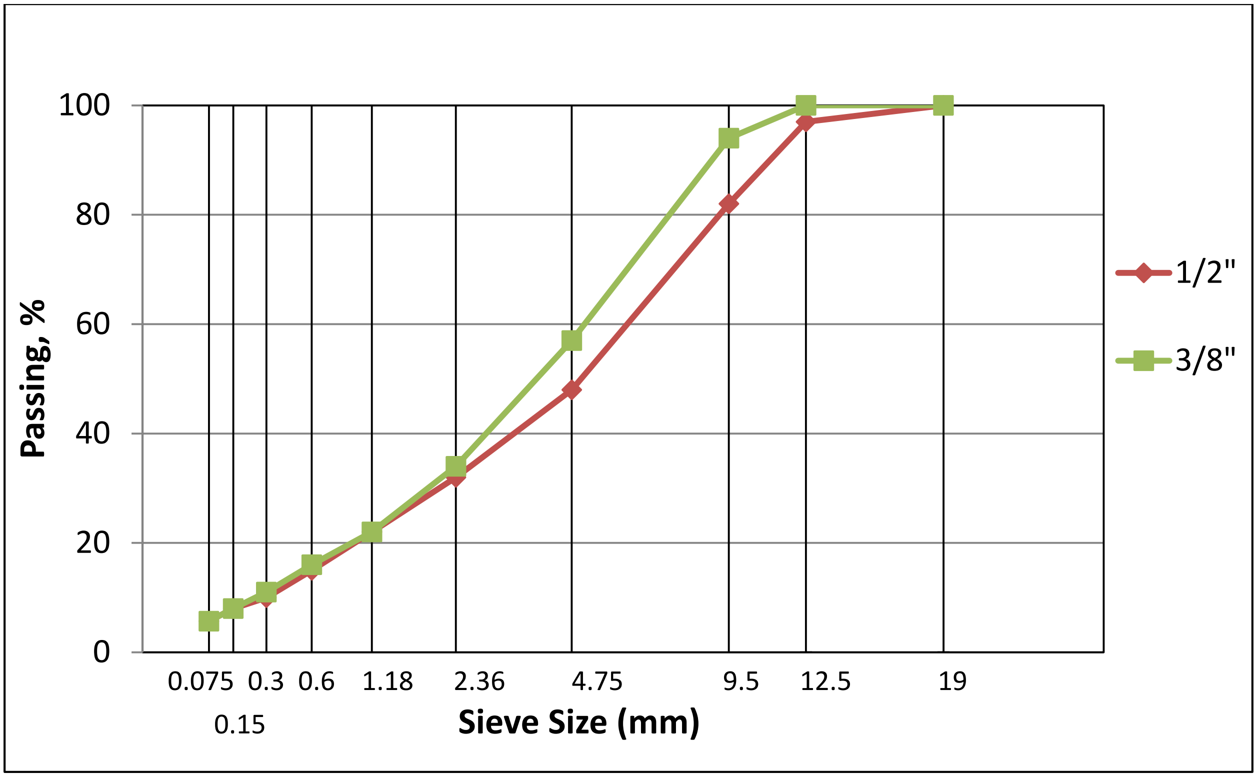

Figure B1.1 Gradations of 1/2" and 3/8" Mixes

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

Volumetric	1/2” NMAS	3/8” NMAS
PG Grade	70-28	70-28
P_b (%)	4.9	5.4
% G_{mm} @ Ninitial	85.9	83.9
% V_a @ Ndesign	4.3	5.8
% VMA @ Ndesign	13.8	15.8
% VFA @ Ndesign	69	64
% G_{mm} @ Nmax	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"	6.3	0.97
3/8”	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” and 1/2” field core samples.

Table B1.3 Studded Tire Wear Mass Loss

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

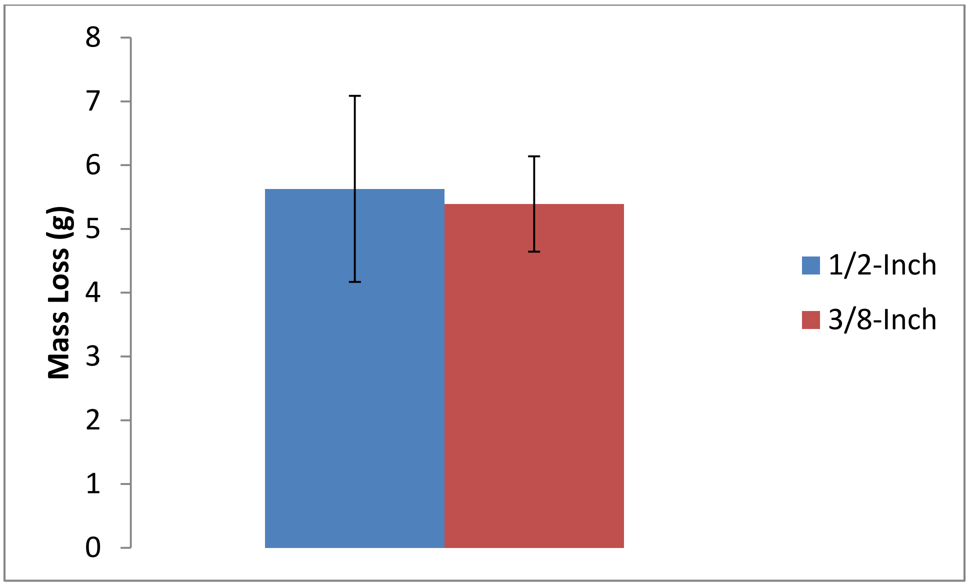

Figure B1.3 Studded Tire Wear

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.

Table B1.4 Dynamic Modulus Effect Sizes

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

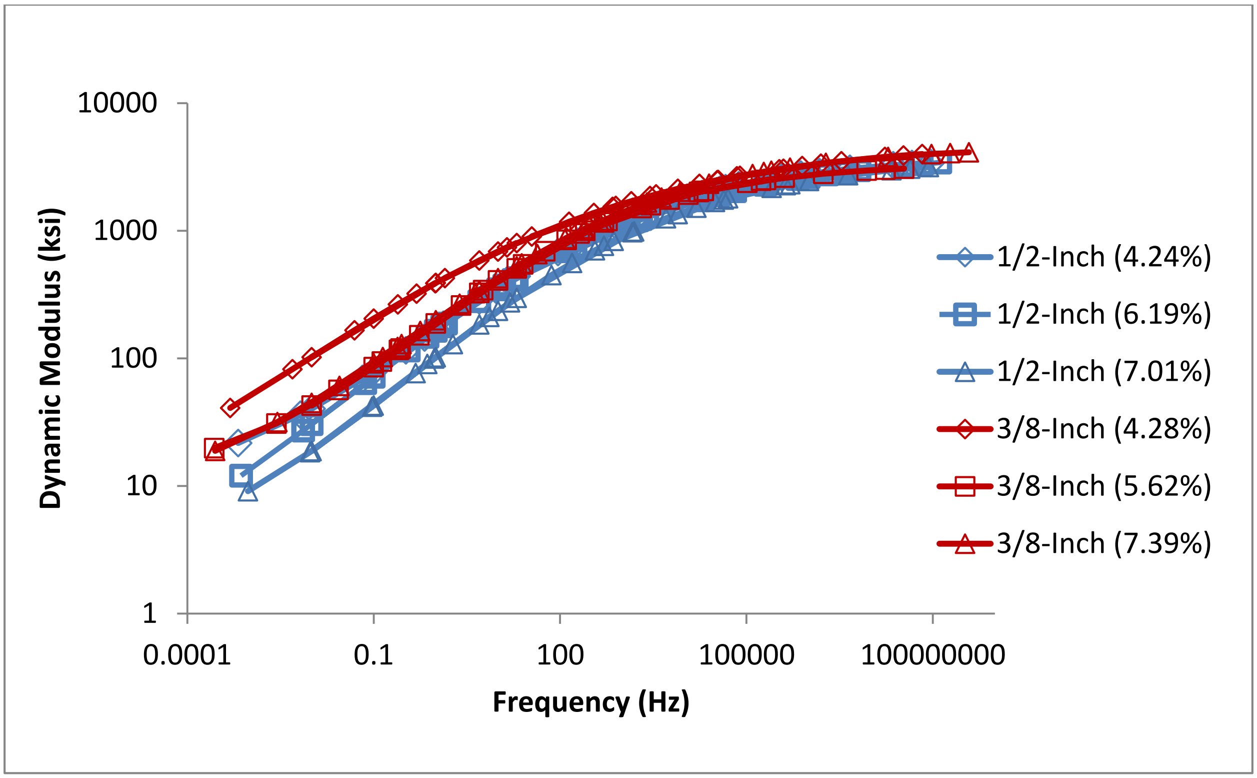

Figure B1.4 Dynamic Modulus Master Curves

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" mix is softer than the 1/2" mix at low and high time-temperature levels, probably due to the relatively higher asphalt content in the 3/8" mix.

Table B1.5 Creep Compliance Effect Sizes

Time-Temperature Level	Mean Creep Compliance (1/psi)		Effect Size
	1/2"	3/8"	
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*

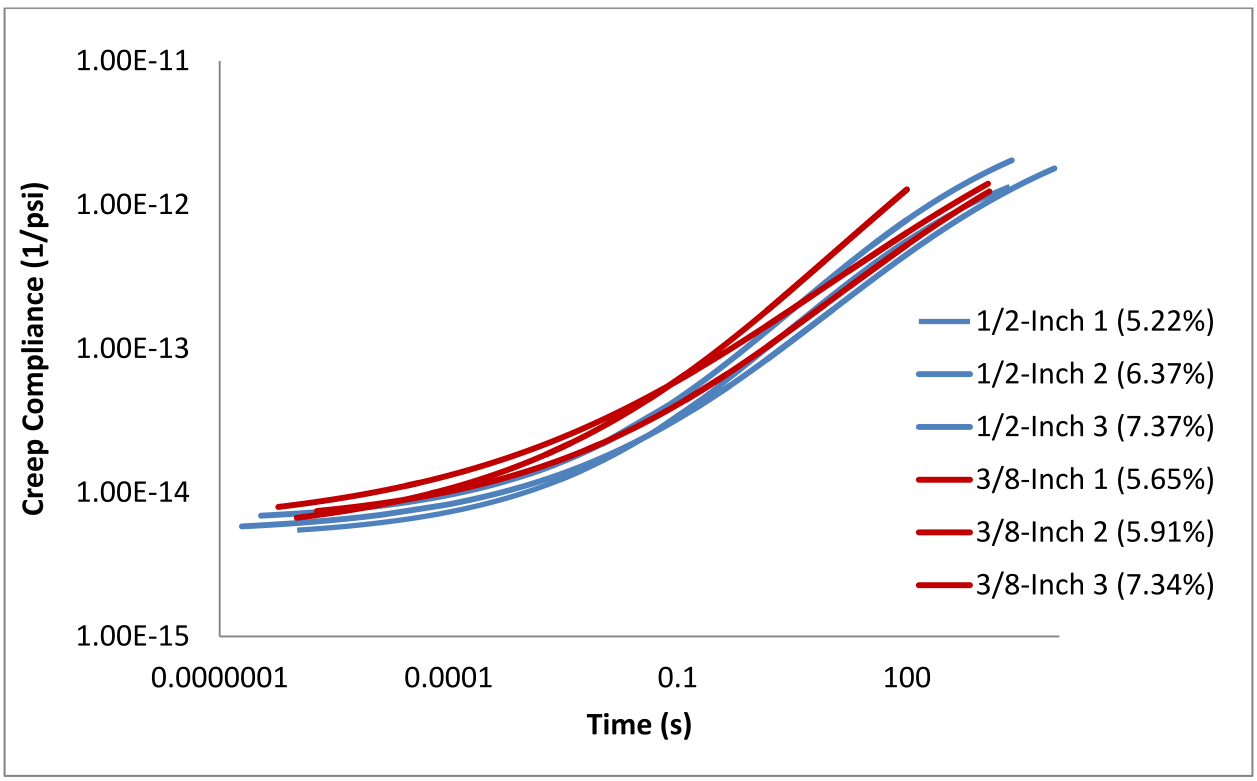

Figure B1.5 Creep Compliance Master Curves

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” and 3/8” 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” mix is statistically significantly more resistant to bottom-up cracking.

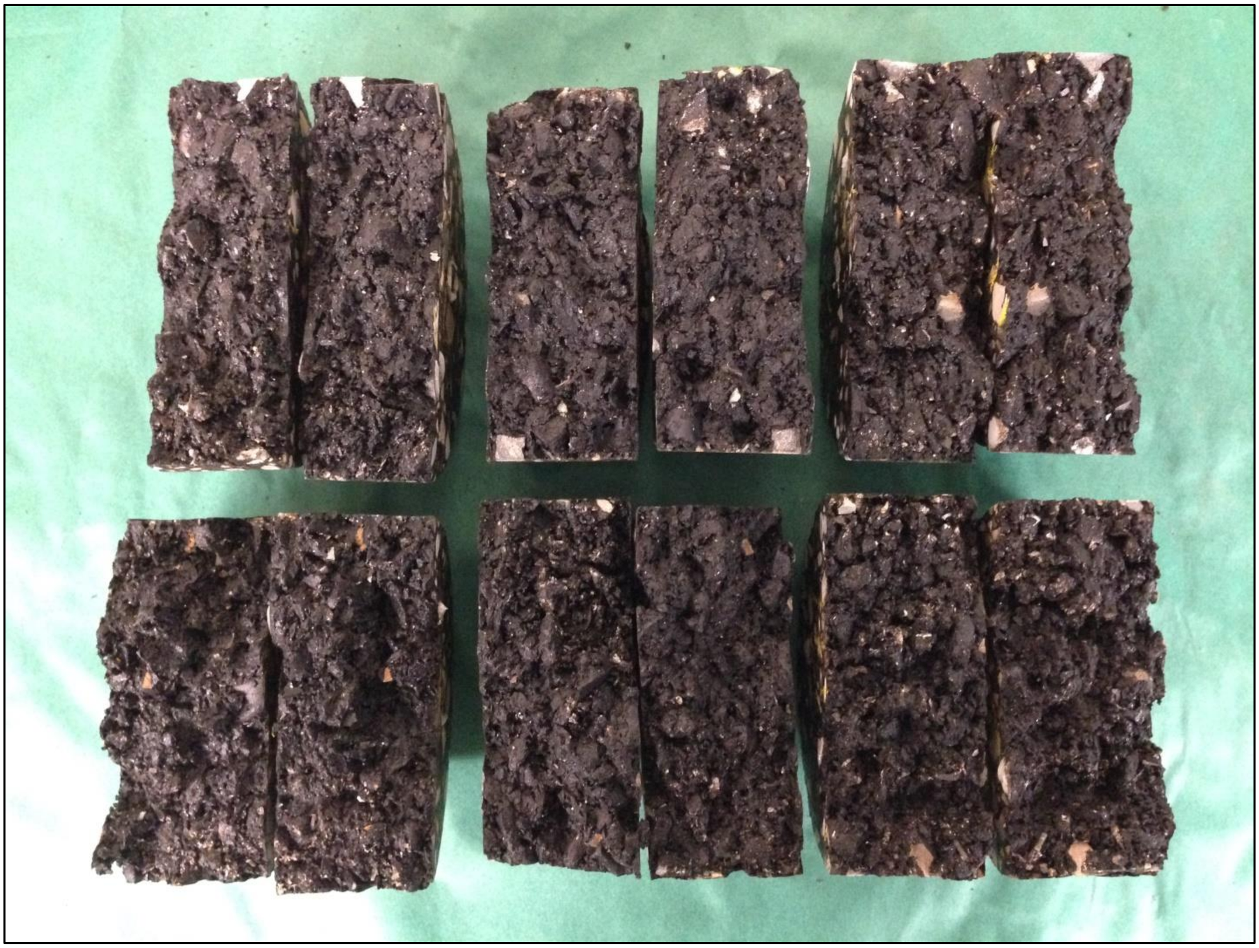

Figure B1.6 1/2" (Top Row) and 3/8" (Bottom Row) Mixes After IDT Fatigue Tests

Table B1.6 IDT Fatigue Test Results Summary

Parameter	Unit	Mean Value		Effect Size
		1/2”	3/8”	
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

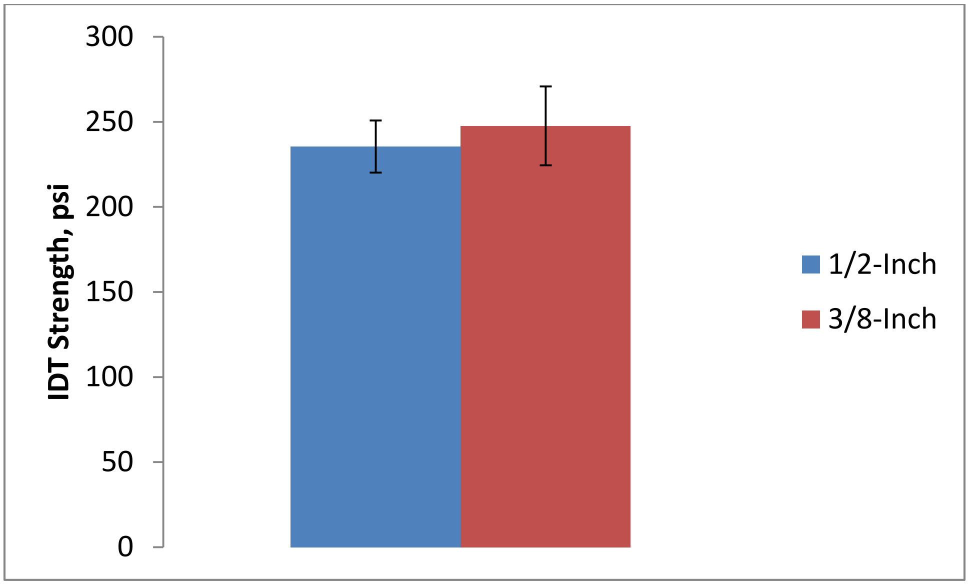

Figure B1.7 IDT Strength

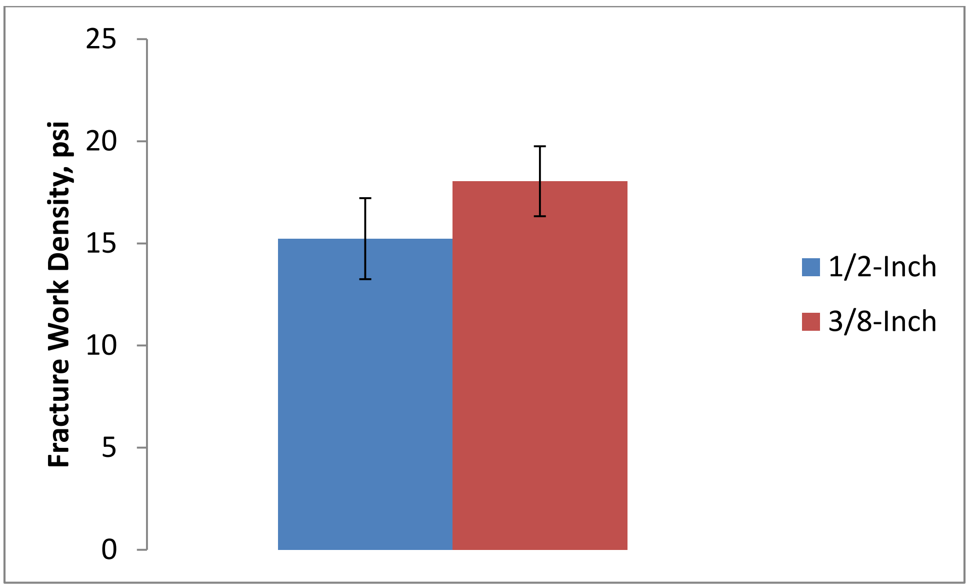

Figure B1.8 Fracture Work Density

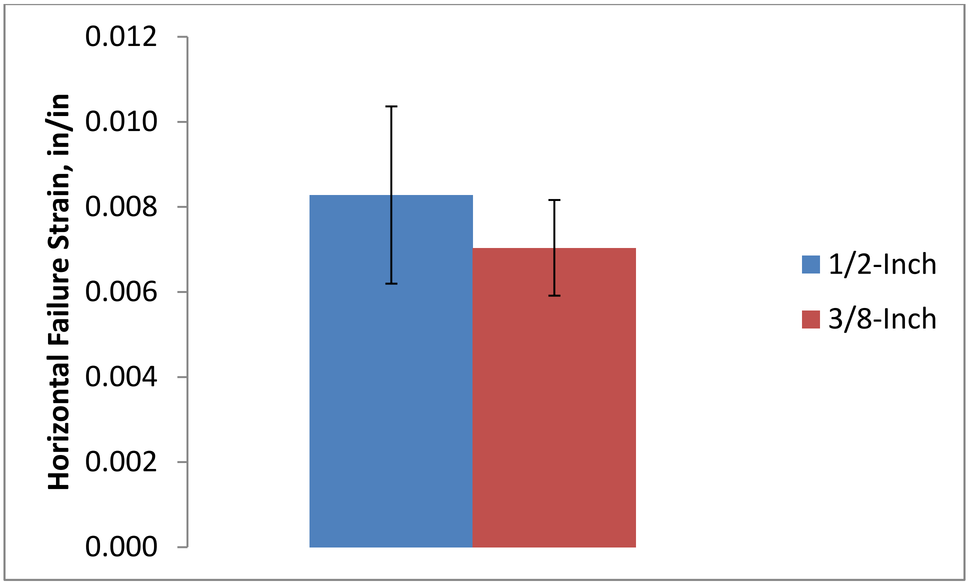

Figure B1.9 Horizontal Failure Strain

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" mix has statistically significantly higher IDT strength than the 1/2" mix and based on fracture work density, is more resistant to thermal cracking than the 1/2" mix. It is noted that broken aggregates are pronounced at the fracture plane.

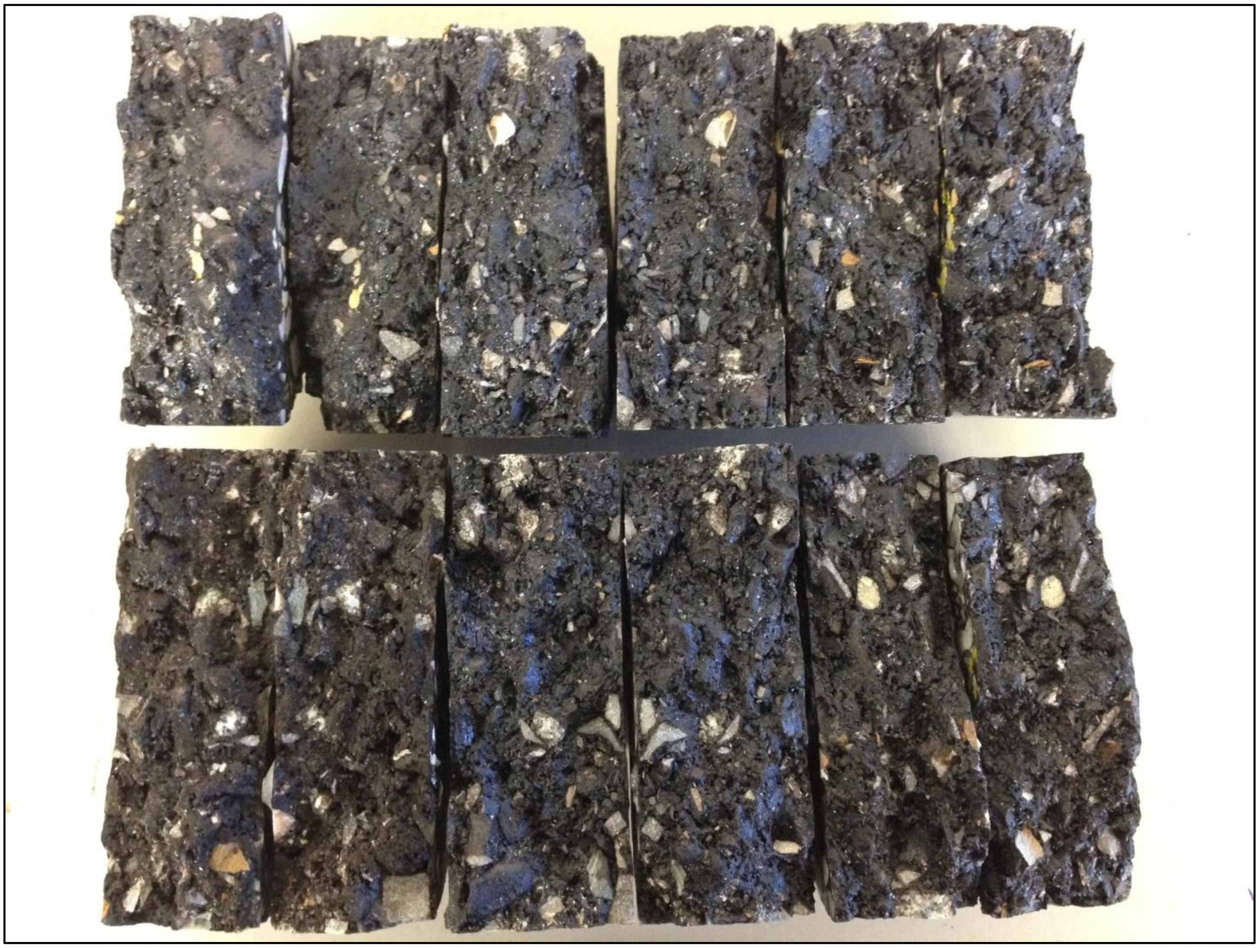

Figure B1.10 1/2" (Top Row) and 3/8" (Bottom Row) Mixes after IDT Thermal Cracking Tests

Table B1.7 IDT Thermal Cracking Test Results Summary

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

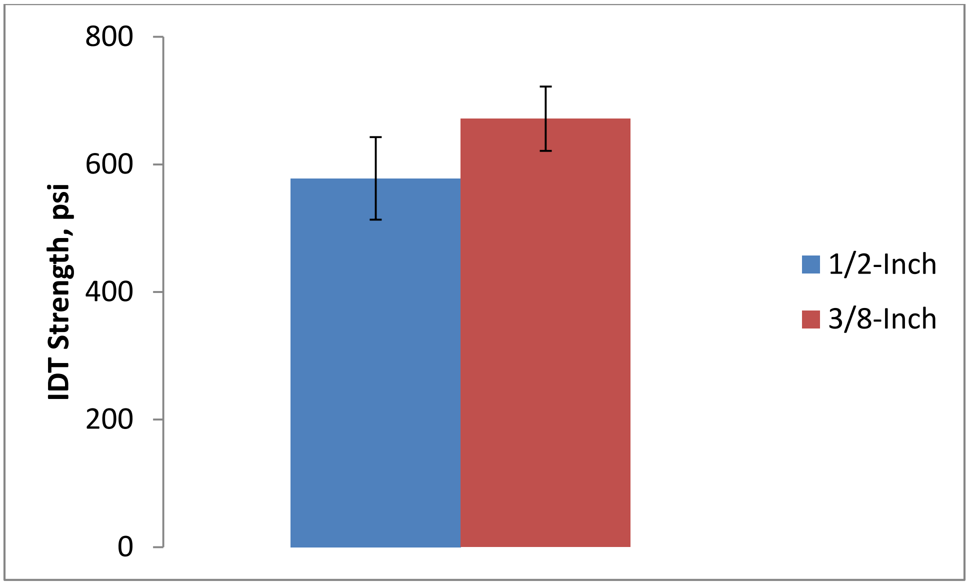

Figure B1.11 IDT Strength from Thermal Cracking Test

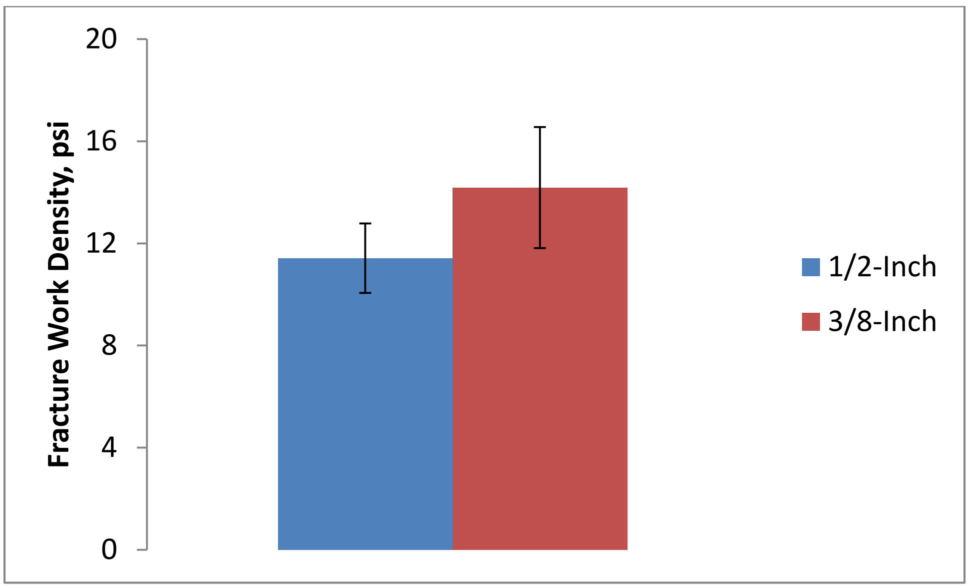

Figure B1.12 Fracture Work Density from Thermal Cracking Test

B1.3.6 Hamburg Wheel-Tracking Test

Results for the Hamburg Wheel-Tracking Test for the 1/2" and 3/8" mixes are shown in Figure B1.13. The 3/8" had nearly 1 mm more rutting than the 1/2". 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.

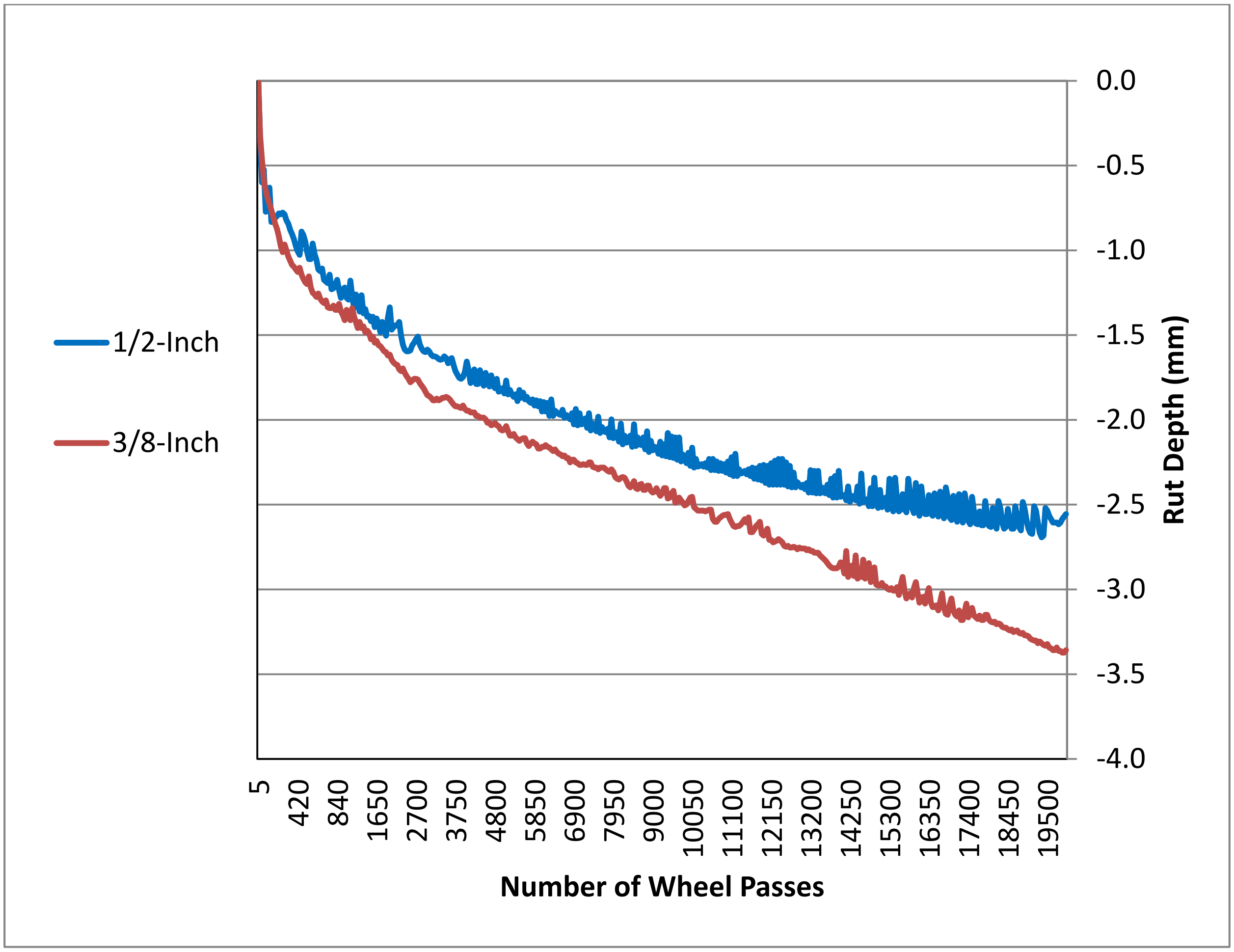

Figure B1.13 HWTT Results

B1.3.7 Asphalt Content

The asphalt content of the field cores and from the job mix formulas for the 1/2" and 3/8" 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" and 3/8" mixes.

Table B1.8 Asphalt Content

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

B1.4 Conclusions

Overall, the performances of the 3/8" and 1/2" mixes are similar. Relatively, compared to the 1/2" mix, the 3/8" mix has similar studded tire resistance, equivalent strength, and equivalent top-down cracking resistance. The 3/8" 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” 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” 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 mileposts recorded in WSPMS may not precisely reflect the actual mileposts 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.

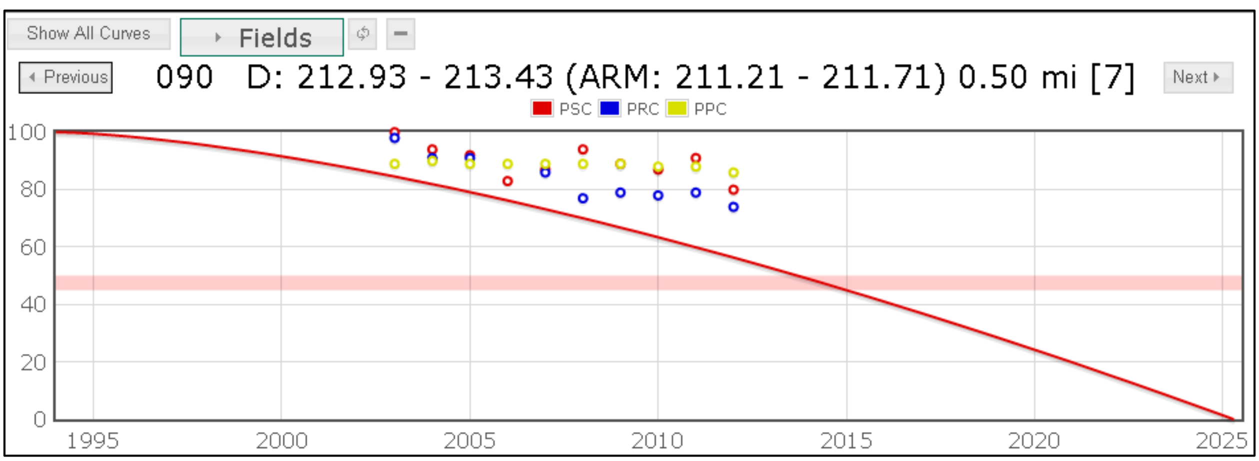

Figure B2.1 C6151 SMA Performance Curves

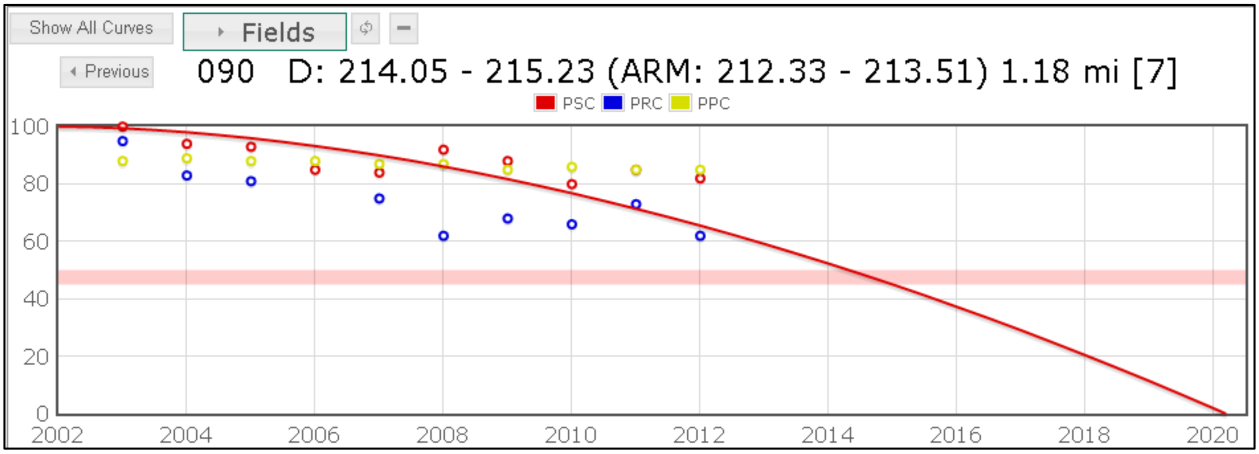

Figure B2.2 C6151 Performance Curves

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

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” mix and 12 replicates were available for the 3/8” 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	

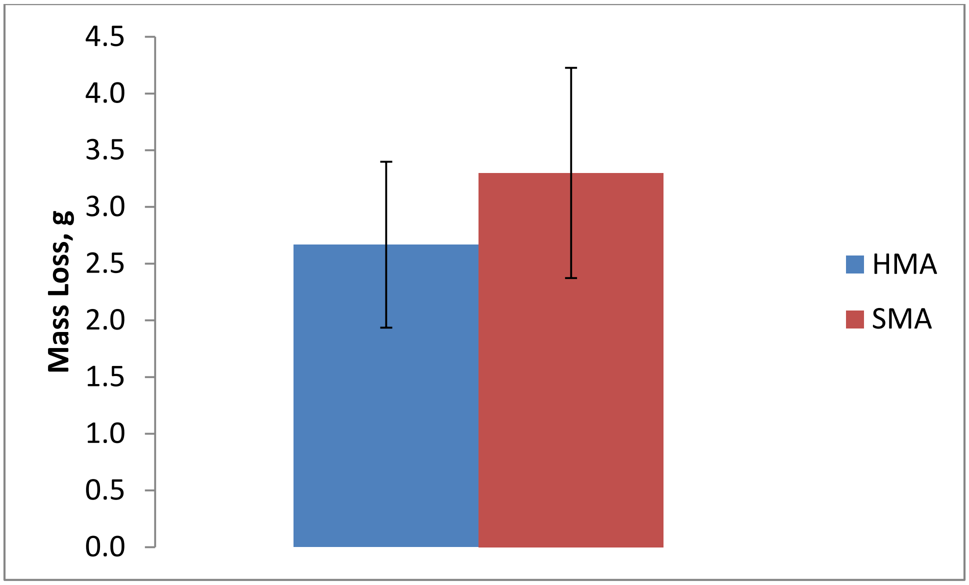

Figure B2.3 Studded Tire Wear

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

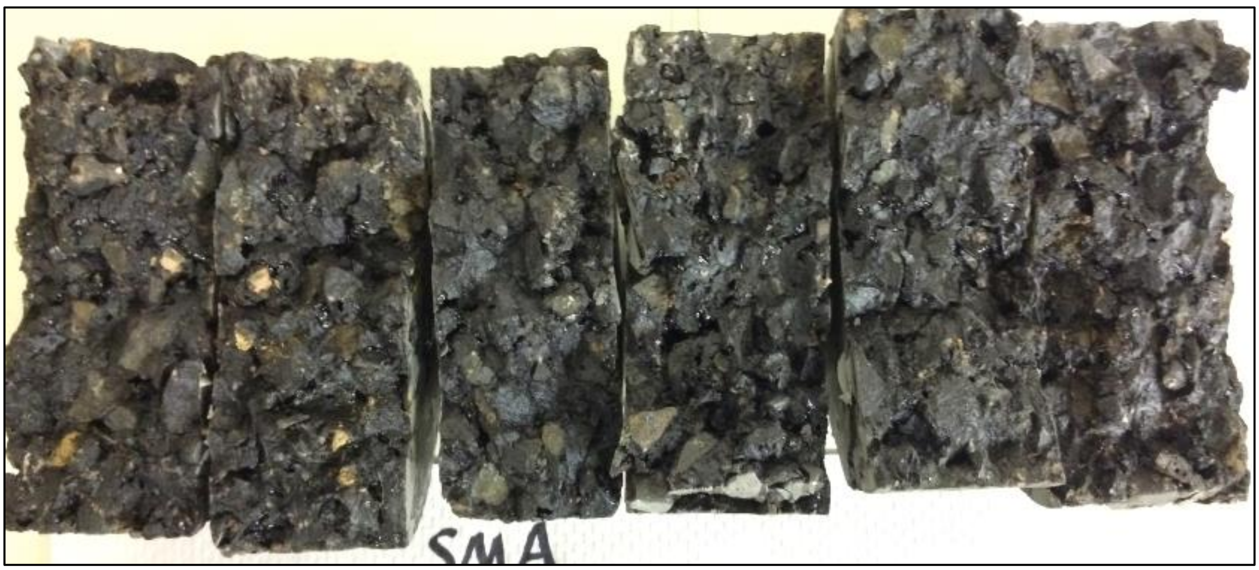

Figure B2.5 SMA Samples after IDT Fatigue Tests

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*

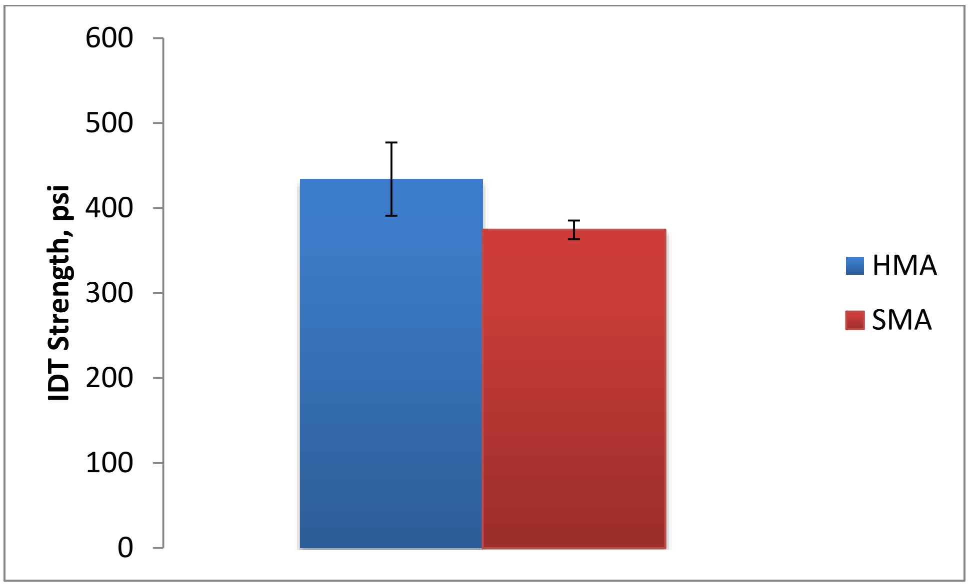

Figure B2.6 IDT Strength

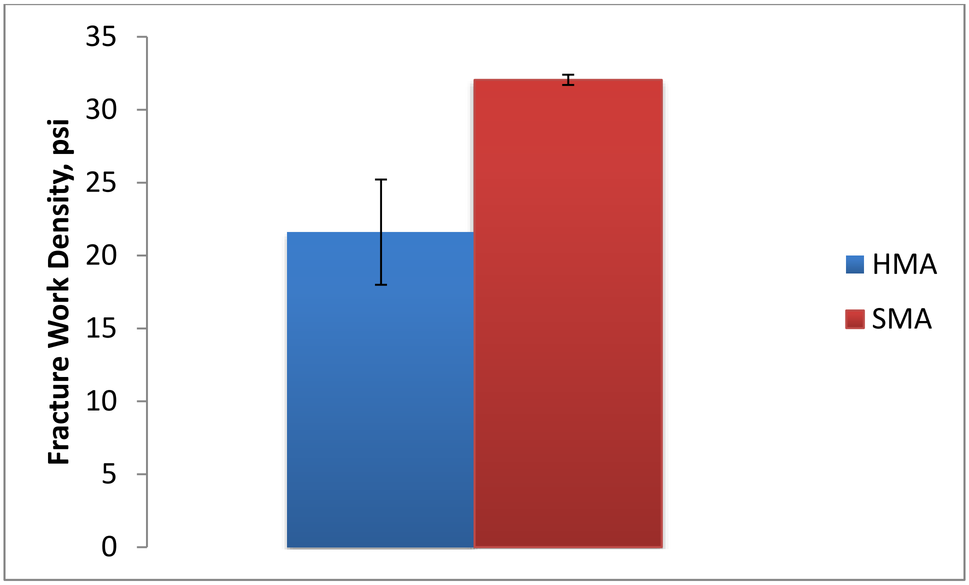

Figure B2.7 Fracture Work Density

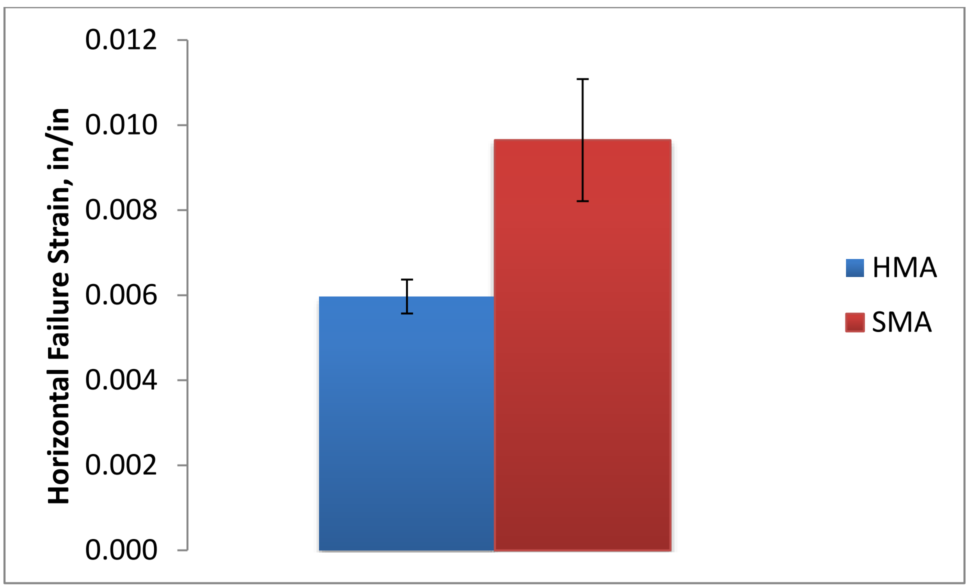

Figure B2.8 Horizontal Failure Strain

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*

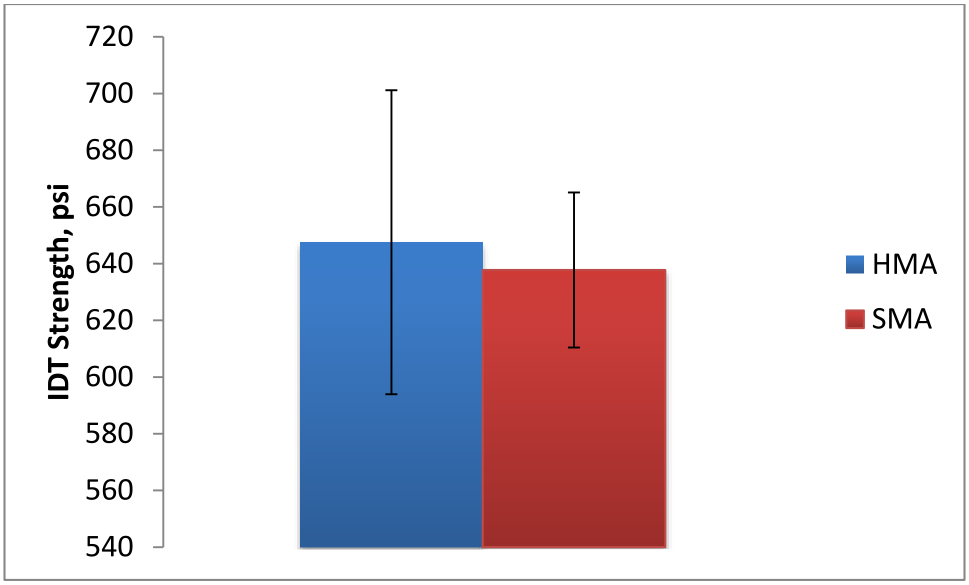

Figure B2.117 IDT Strength from IDT Thermal Cracking Test

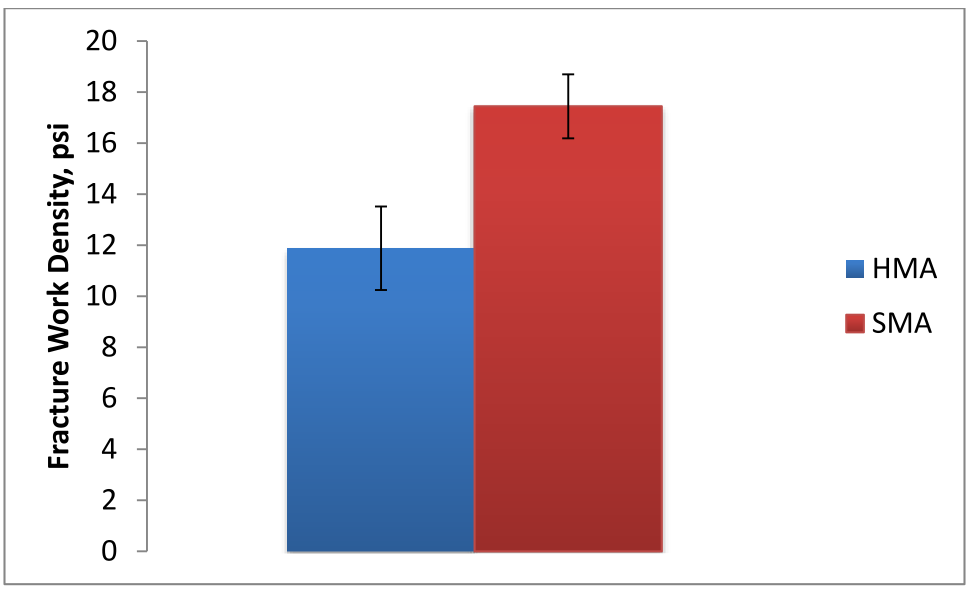

Figure B2.12 Fracture Work Density from IDT Thermal Cracking Test

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.

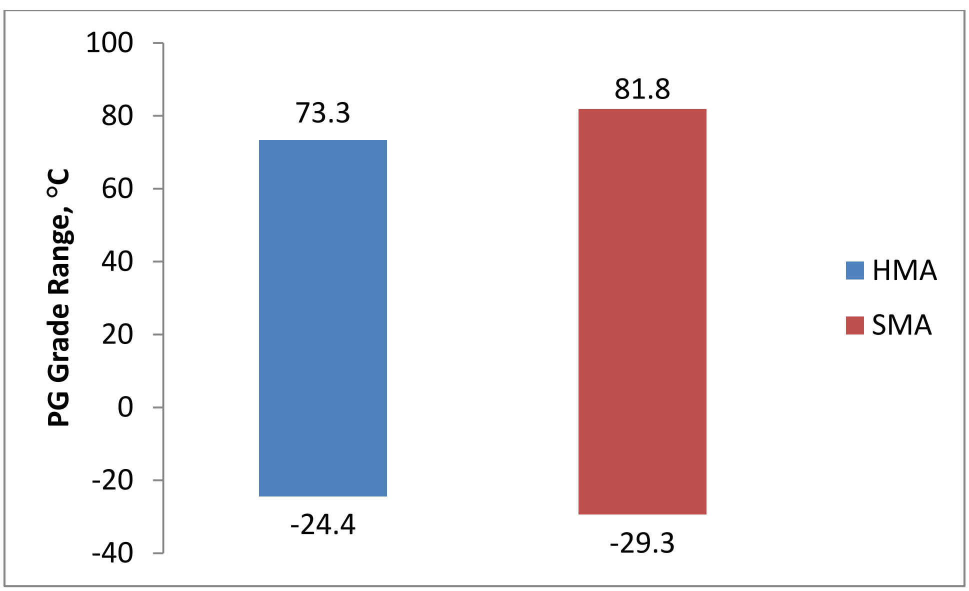

Figure B2.13 PG Grade Ranges

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.

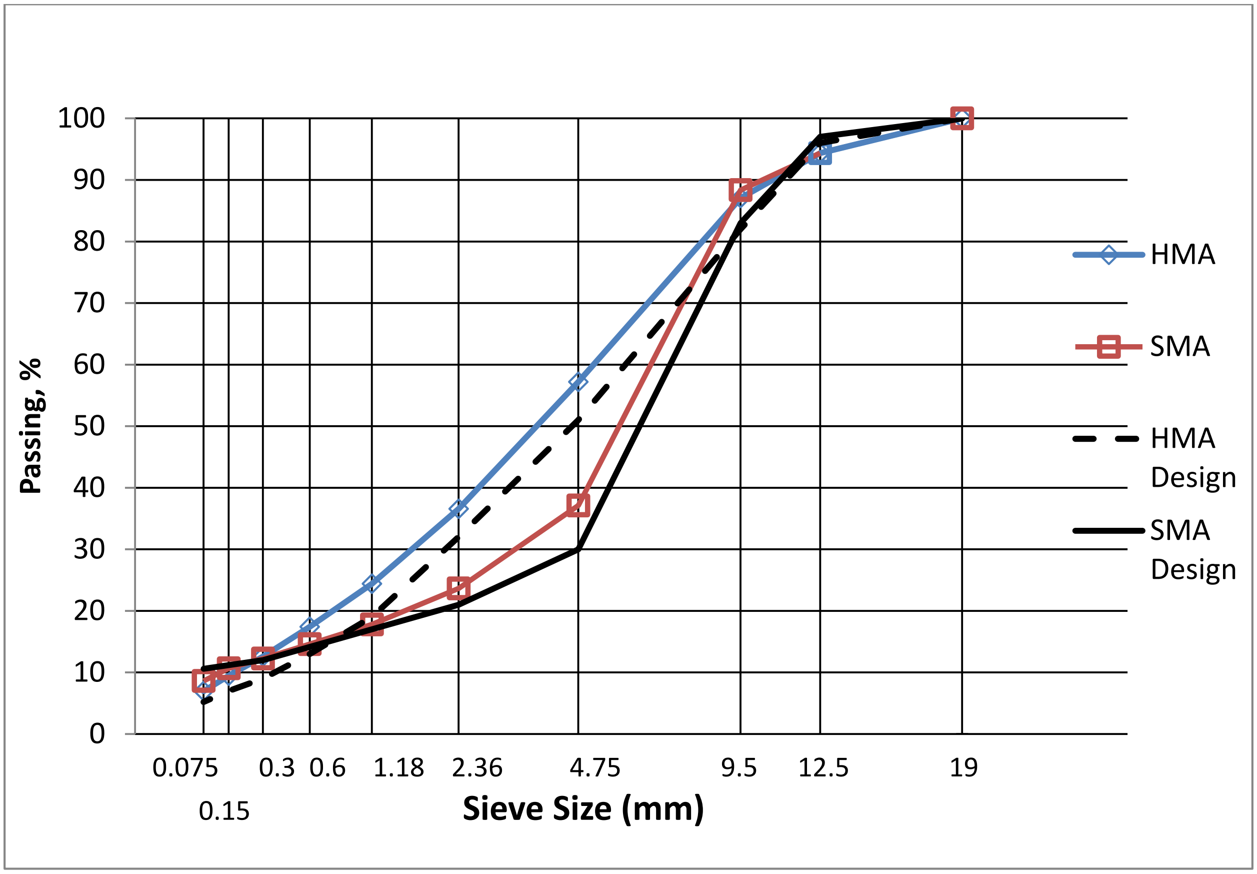

Figure B2.14 Gradations of Field Cores

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 and included a pre-level and BST overlay in 2006. The HMA for pre-level was 3/8” PG 64-28. The pre-seal layer was BST Class D CRS-2P and 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 was paved and overlaid with BST in 2012. The pavement was ACP Class 3/8” PG 64-28 and was a grind and inlay of 0.15 ft. depth. No performance data from WSPMS was available yet for Contract 8262, due to its recent construction.

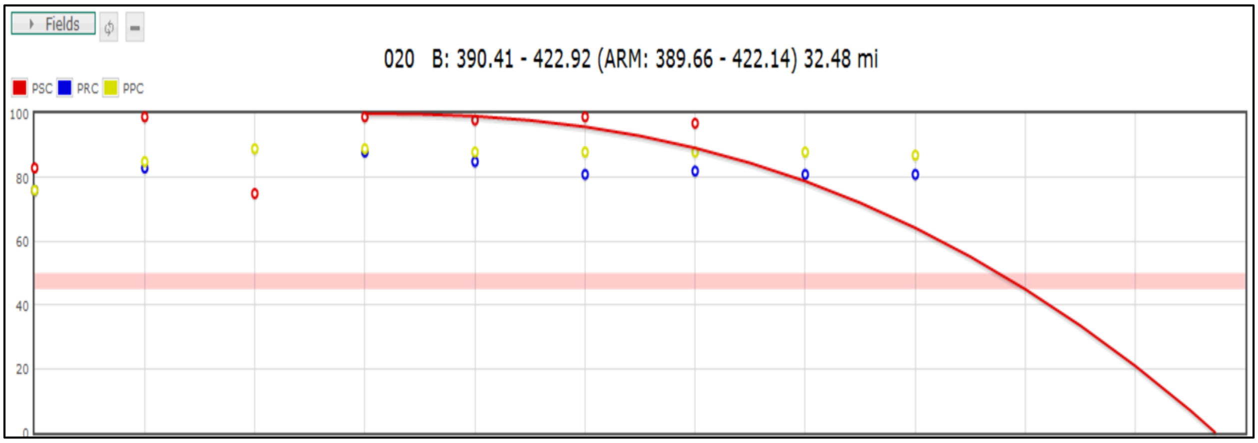

Figure B3.1 C7109 (SR 20) Performance Curves

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 with BST has higher air void level than the HMA without BST. It is hypothesized that the reduced aging in HMA with BST facilitated the consolidation by the 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*

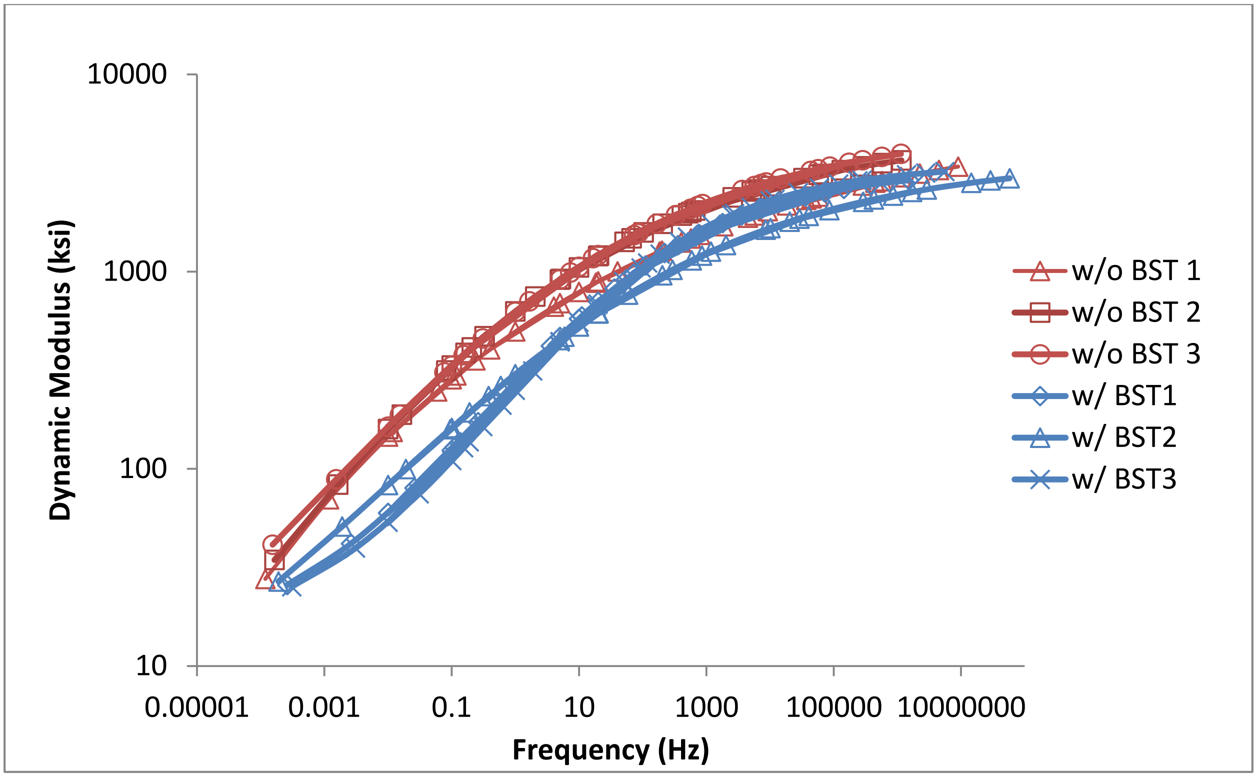

Figure B3.2 SR 20 Dynamic Modulus Master Curves

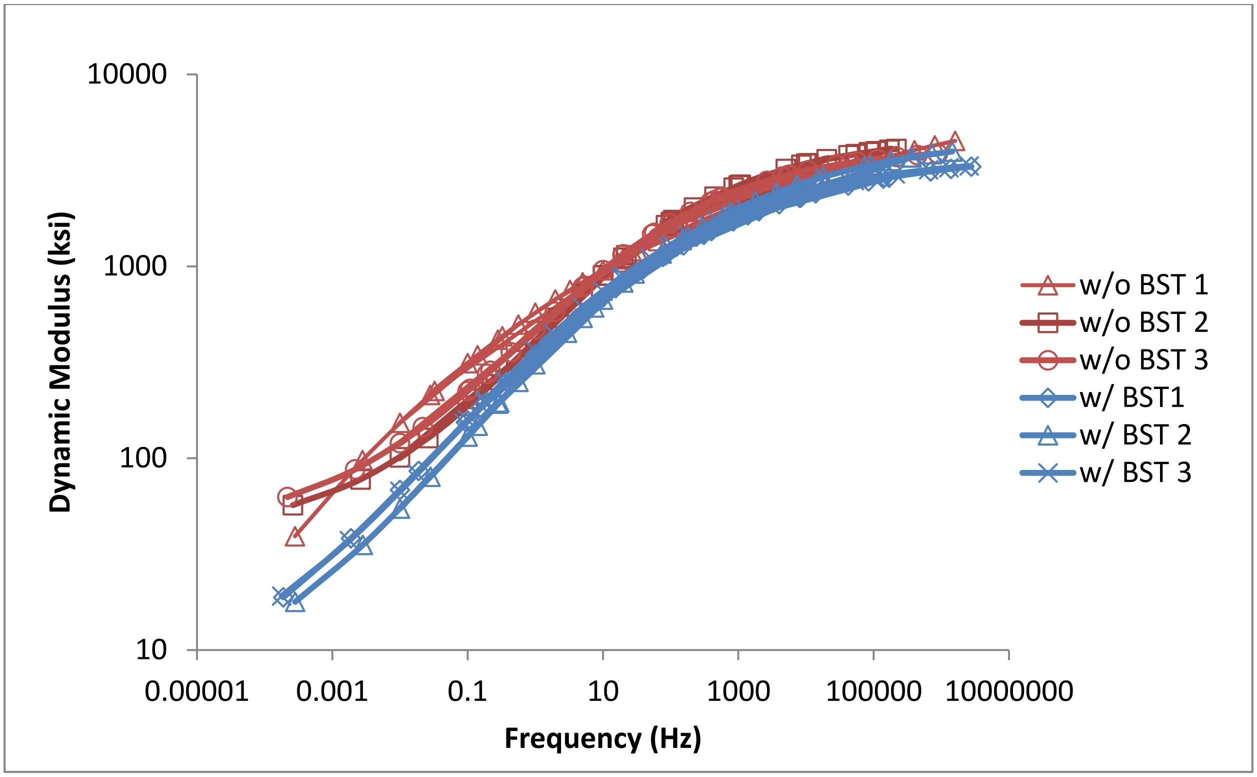

Figure B3.3 SR 278 Dynamic Modulus Master Curves

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*

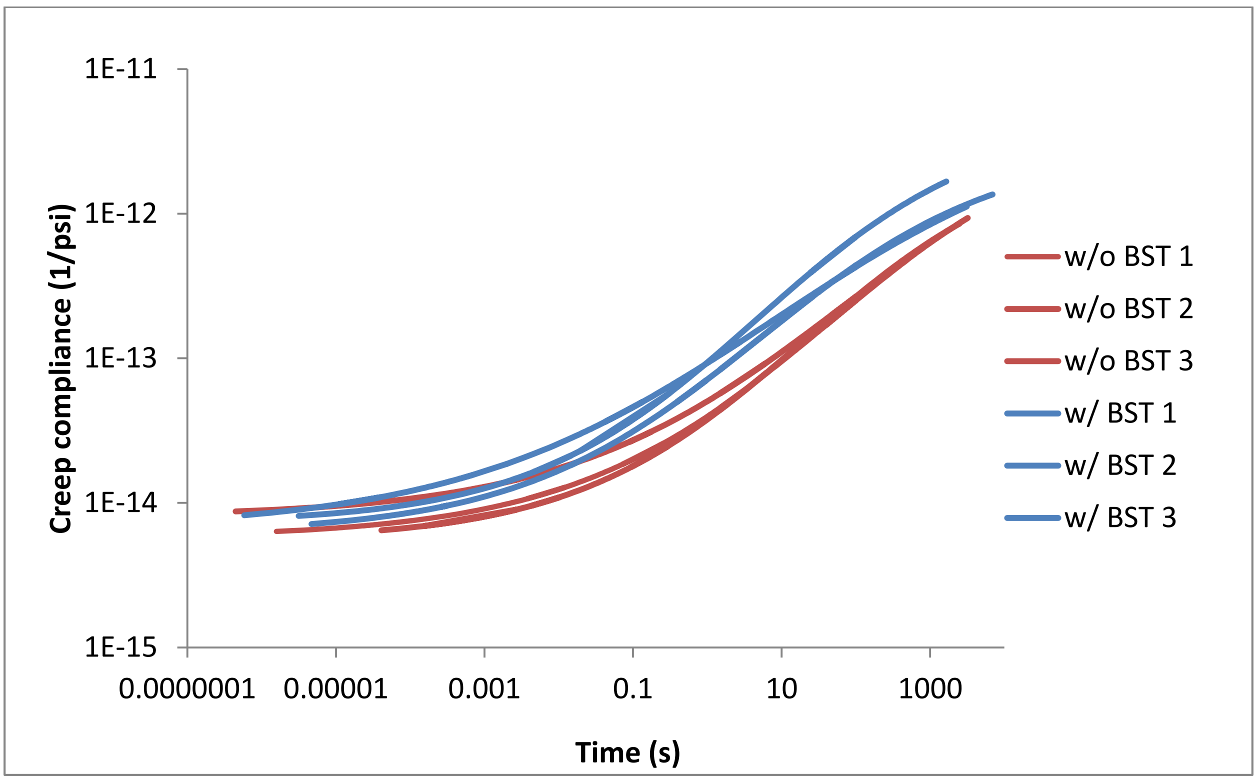

Figure B3.4 SR 20 Creep Compliance Master Curves

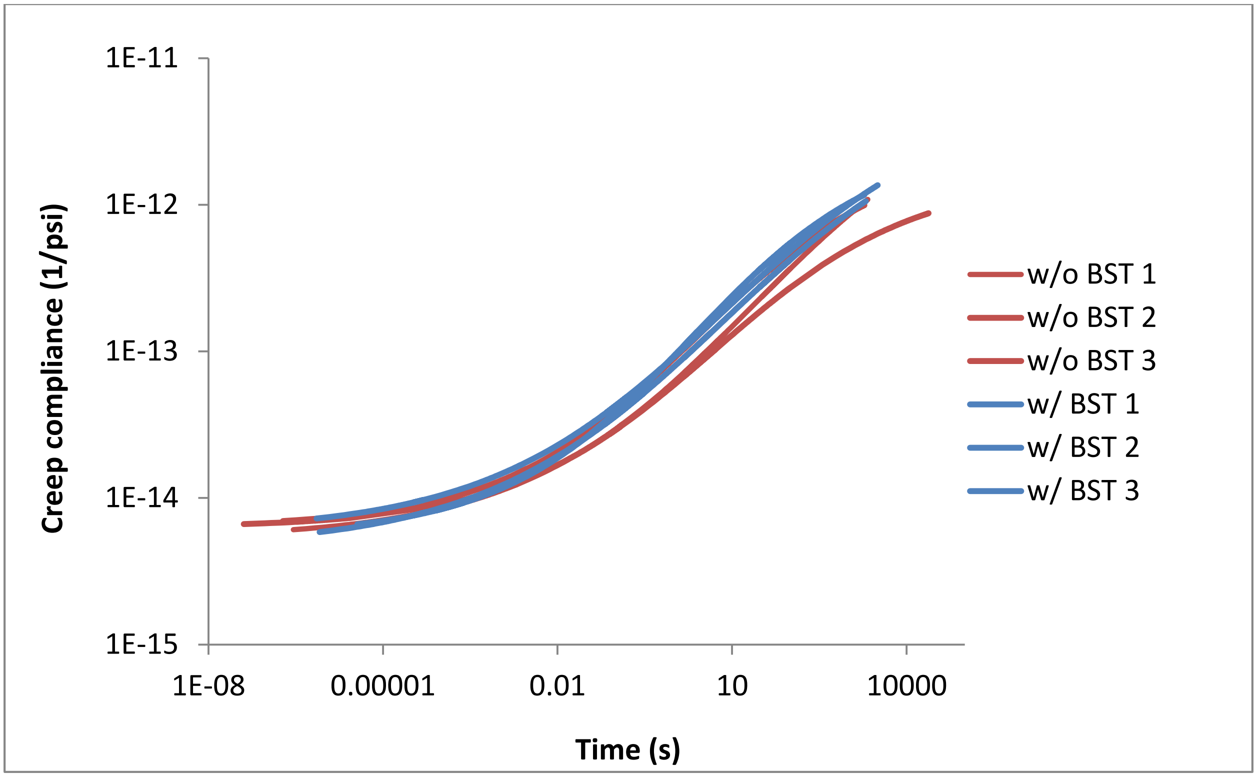

Figure B3.5 SR 278 Creep Compliance Master Curves

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

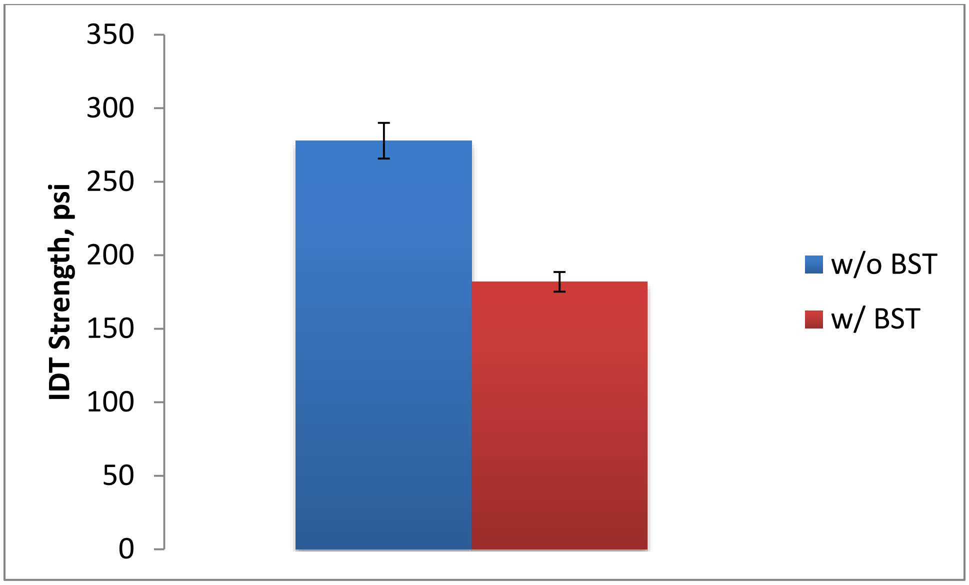

Figure B3.6 SR 20 IDT Strength

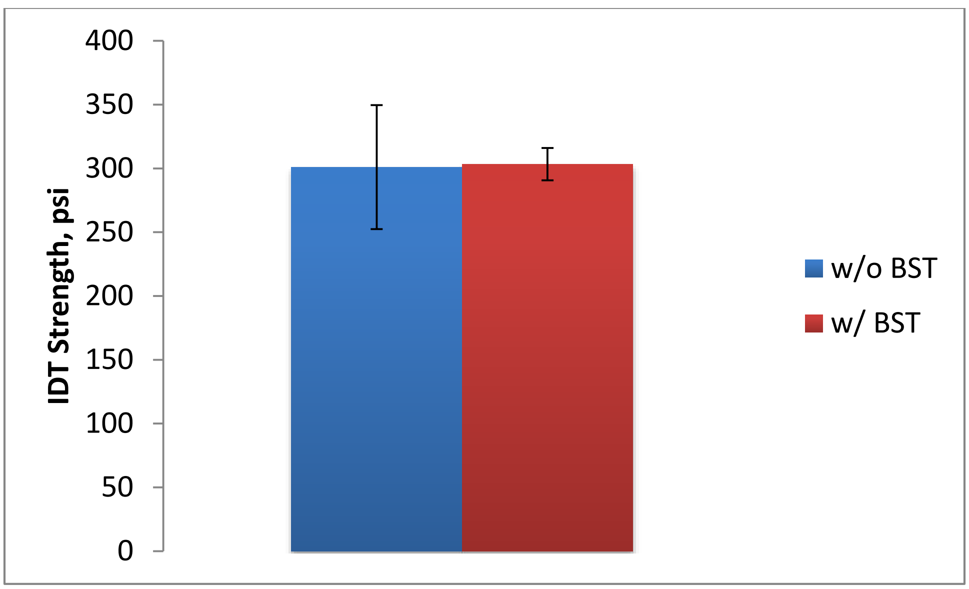

Figure B3.7 SR 278 IDT Strength

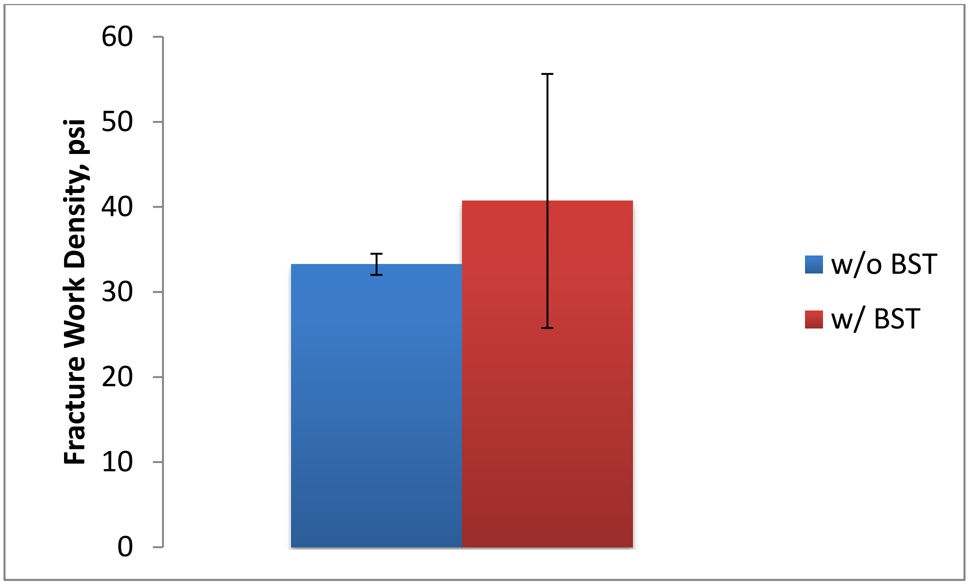

Figure B3.8 SR 20 Fracture Work Density

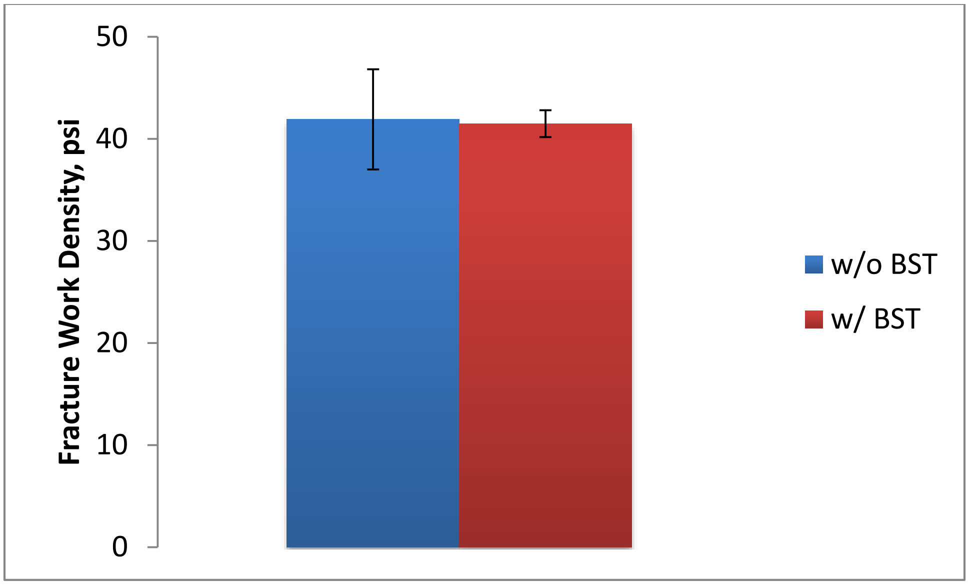

Figure B3.9 SR 278 Fracture Work Density

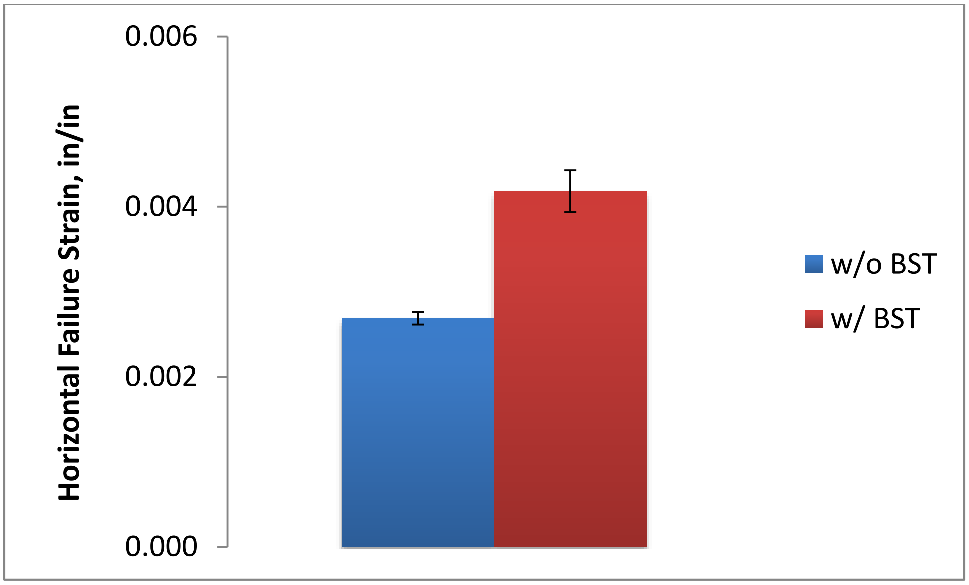

Figure B3.10 SR 20 Horizontal Failure Strain

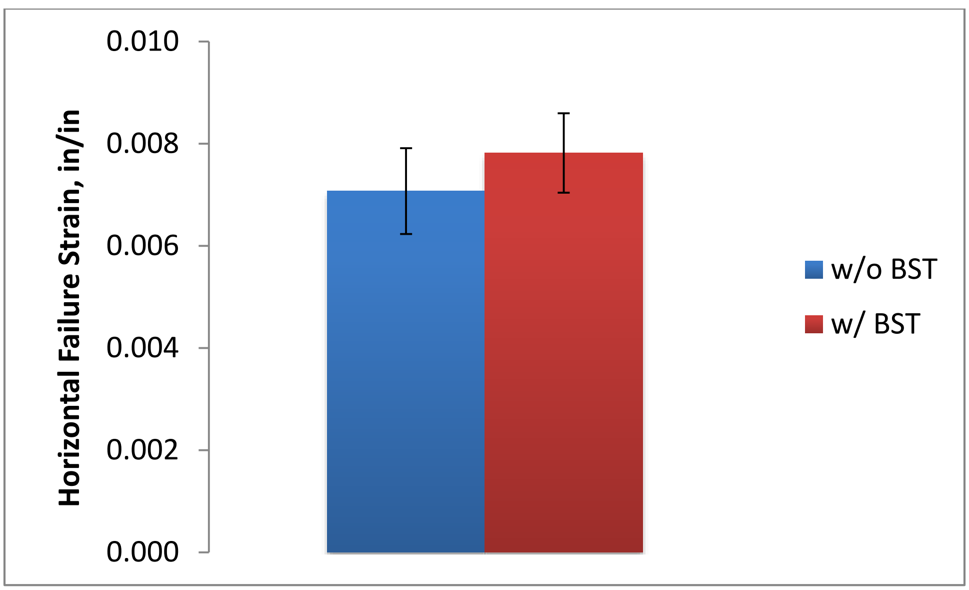

Figure B3.8 SR 278 Horizontal Failure Strain

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 reduced the aging of the underlying asphalt.

Table B3.5 Binder PG Grades

Project	Temperature Level	Field Core Layer	True PG Grade (°C)		PG Grade Difference (°C)
			HMA w/o BST	HMA w/ BST	
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

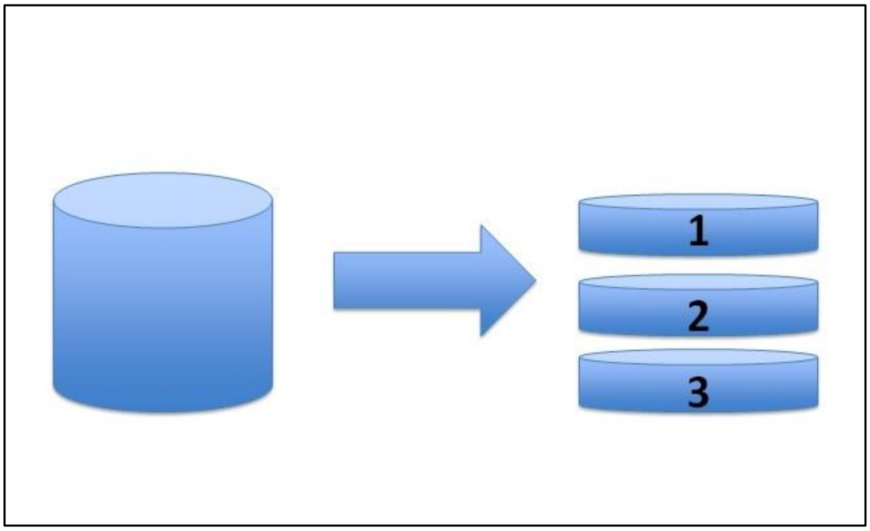

Figure B3.12 Field Core Layers for PG Grade Determination

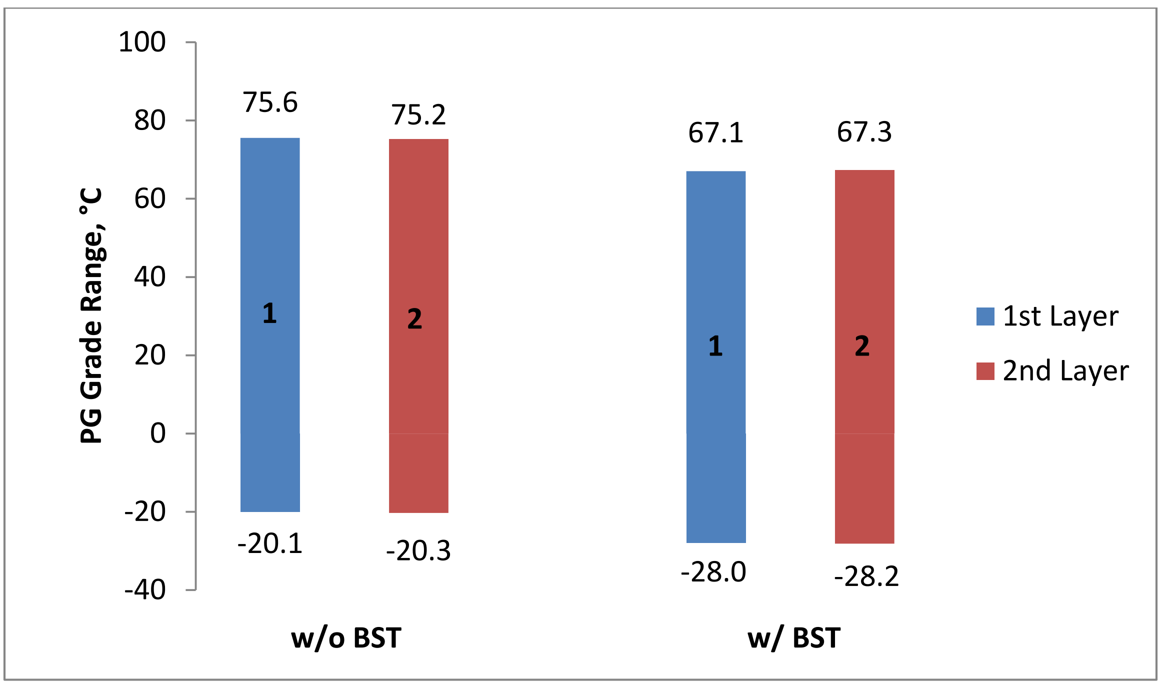

Figure B3.13 SR 20 High and Low PG Grades

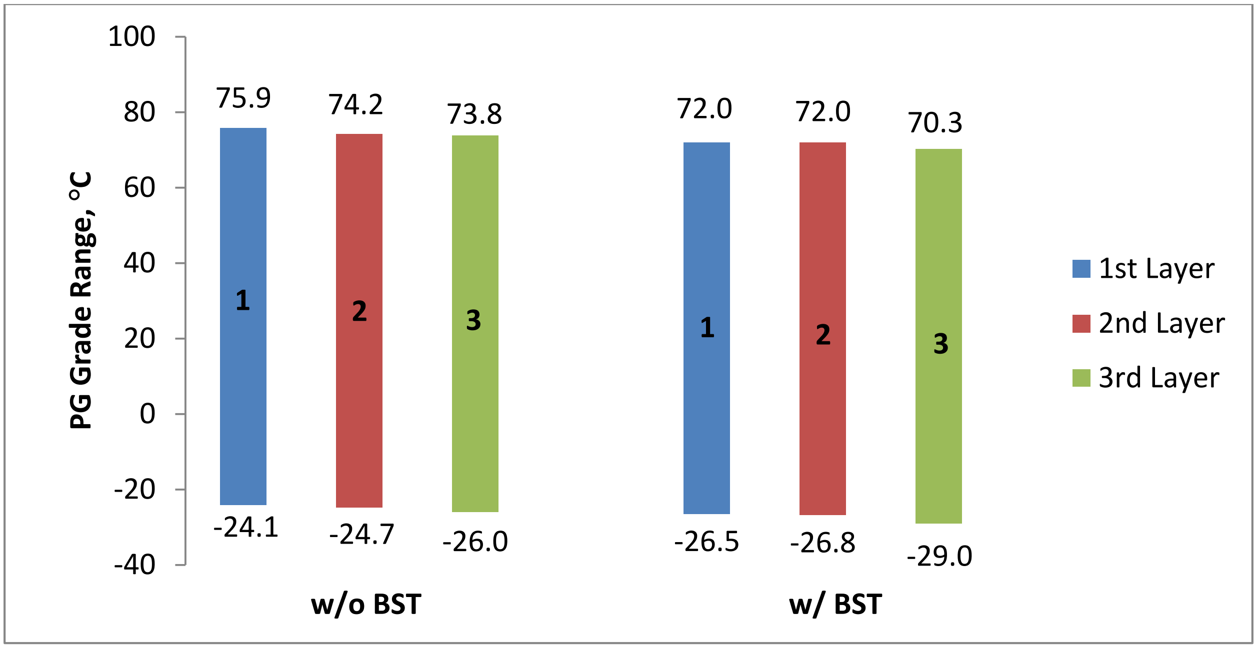

Figure B3.14 SR 278 High and Low PG Grades

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” VS. 1/2” NMAS PROJECT MIX DESIGNS

The mix designs for the 3/8” and 1/2” 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

VALID FOR 2014

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

CONTRACT 008611 ONLY

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

NON-STATISTICAL

COMMENTS

Remarks:

Result Code:

Billing Code

T177 - 1

T185 - 18

T194 - 2

T198 - 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

Kurt R. Williams, P.E.
State Materials Engineer
Joseph R. DeVol
Assistant Construction Materials Engineer
Date : 6/23/2014
Phone : (360) 709-5421

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

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

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-
T153-

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.

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: I94674
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

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

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		

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

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)			