DESIGN AND EVALUATION OF 4.75mm MIXTURE FOR THIN ASPHALT OVERLAY IN WASHINGTON STATE

By

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Abstract

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Thin overlays with 4.75mm nominal maximum aggregate size (NMAS) mixtures are an alternative pavement preventive maintenance strategy that are becoming more popular in recent years mainly because of their cost effectiveness, ability to be placed in thin lifts, and for using screening stockpiles. Washington State is considering introducing this preventive maintenance strategy after an appropriate evaluation of all aspects of this treatment. The objective of this study was to develop mix designs and evaluate the potential of using 4.75mm NMAS thin overlays for WSDOT considering local conditions. A thorough literature review and agency survey was conducted to determine how 4.75mm NMAS thin overlay could be designed, constructed, and applied appropriately. Four mix designs were developed for high traffic volume roads using two binders (PG70-28 and PG76-28) and two gradations (coarse and medium). The design was based on packing concept to help achieve good aggregate interlock and satisfactory volumetric properties. The cracking, rutting, and moisture resistance of the mixtures were evaluated using laboratory indirect tensile test and Hamburg wheel-tracking test. It was found the PG70-28 coarse graded mixture had the best overall performance results with all mixtures showing good results. Using a life cycle cost analysis (LCCA), the 4.75mm thin overlay was compared to traditional 12.5mm overlays and chip seals for cost effectiveness in Washington State. According to historical

data and estimation, it was suggested that 4.75mm thin overlays be cost effective when compared to traditional overlay but not chip seals. 4.75mm thin overlays could be a viable pavement preservation strategy in Washington State. Based on the findings from this study, a draft special provision was also created to assist WSDOT in implementing this mix type into their preventive maintenance program.

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CHAPTER 1: INTRODUCTION

1.1 BACKGROUND

In this ever evolving world, budget constraints are making it harder to maintain the highway infrastructure that is vital to economic prosperity and transportation in the United States. Sound pavement preventive maintenance programs can reduce costs while improving the quality of the pavement network. As shown in Figure 1-1, different from traditional approaches which wait until deficiencies are evident or until reconstruction or major rehabilitation becomes a must, the preventive maintenance treatments are usually applied early on very good or good pavement conditions and when the pavement is still structurally sound to maximize cost effectiveness and return the pavement back to its original service level. The use of 4.75mm nominal maximum aggregate size (NMAS) thin overlays is one of the preventive maintenance treatment strategy which has become more attractive recently.

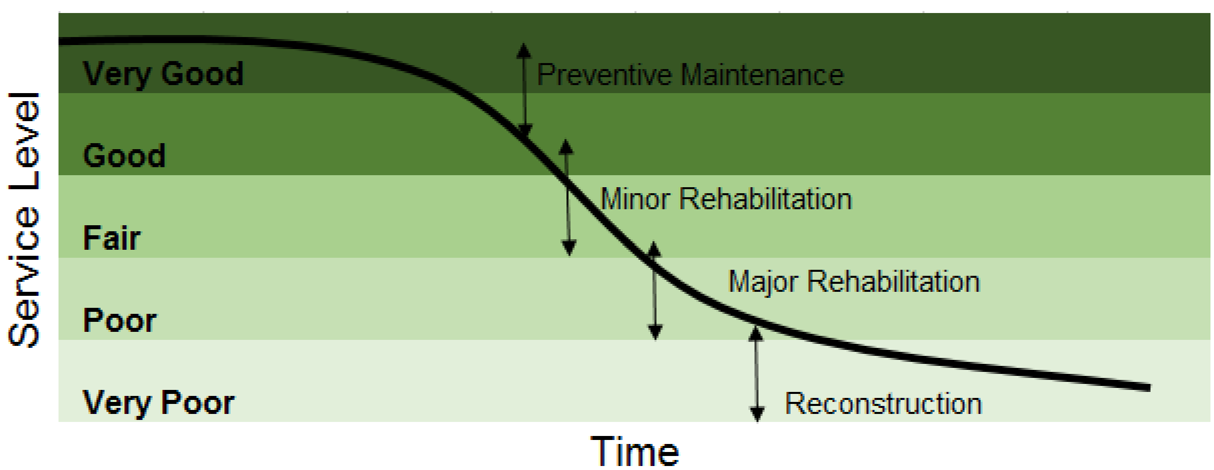


Figure 1-1: Service Level Change with Time and Associated Treatment Types

A 4.75mm NMAS thin overlay is a non-structural layer which is normally applied to provide functional improvements that enhance smoothness, friction, and the profile of the road while adding little to no structural capacity. These layers are normally a thin lift of 0.75 inches to

1 inch and can be referred to as thin or ultra-thin overlays. No major structural distress should exist prior to applying 4.75mm NMAS thin overlay. Being able to be placed in thin lifts and use screening stockpiles, Wolters and Thomas (2010) describe this treatment as one of the most cost-effective and versatile pavement preservation options available. With the difference in aggregate gradation there are three types of thin overlays that can be used (Morian, 2011), dense-graded, open-graded, and gap-graded (SMA). Dense-graded overlays are the most popularly used and will be the main focus for this research.

For WSDOT, climate is a special concern that provides several challenges to consider. The climatic condition challenges of cold weather regions such as Washington include a shorter construction season, frozen water problems (snow, ice, freeze-thaw conditions), and studded tire wear (Zubeck and Liu, 2012). These factors must be considered when evaluating 4.75mm mixtures for Washington State.

In 2002, the National Center for Asphalt Technology (NCAT) conducted a study to develop a Superpave mix design specification for 4.75mm NMAS mixtures (NCAT, 2011). This study recommended similar design procedure but different volumetric criteria such as dust proportion ratio and Void in Mineral Aggregate (VMA) from conventional larger NMAS mixtures. The 1.18mm sieve is suggested to be the middle control sieve rather than the 2.36mm sieve used for larger NMAS mixture. However, because the design of such finer aggregate mixtures are highly dependent on the local material source (mostly the availability of aggregate screening materials), very different gradations have been adopted by a number of agencies showing reasonably good performance (Zaniewaki and Diaz, 2004 and Cooley Jr. et al, 2002). For the same reason, it is important to develop 4.75mm NMAS mixtures using Washington materials. The mixtures should still have satisfactory volumetric properties and engineering performance. A new mix design method based on packing concept was developed by Shen and Yu (2011) for 12.5mm NMAS mixtures. By evaluating and controlling the particle packing characteristics, this method helps quickly determine aggregate gradations that satisfy volumetric properties and further estimate the design asphalt content at the early stage of the mix design. It also provides opportunities of optimizing the aggregate gradations to achieve good aggregate interlocking. It is possible this new mix design method can be extended to smaller NMAS mixtures such as 4.75mm mix to generate mix designs using Washington local materials.

Life cycle cost analysis (LCCA) is defined by AASHTO (1986) as a technique founded on economic analysis principles which enables the evaluation of overall long-term economic efficiency between competing alternative investments. It is used as an important factor when determining the viability of a pavement treatment method. When determining the overall cost of pavement management activities there are multiple factors to consider. Some of these factors include material costs, construction costs, maintenance costs, and design costs. This overall cost is then used along with service life and a discount rate to compute the life cycle cost of certain pavement treatments. In the preventive maintenance program, it is important to determine the life cycle cost for a specific treatment types and compare it with other treatment types that address similar pavement distresses/conditions. With respect to a new treatment type that has not been adopted by the local agency, an estimation of the LCCA based on historical information would also be beneficial to help determine its application potential and strategy.

1.2 PROBLEM STATEMENT AND RESEARCH OBJECTIVES

Roadways in Washington State are generally performing well in recent years, but with a continued reduction in funding the need for cost effective preventive maintenance strategies have increased (WSDOT, 2010). By 2010, Washington’s Roadway Preservation budget had been reduced over 0.58 billion dollars over the last 10 years (WSDOT, 2010). Thin overlays using 4.75mm NMAS mixtures are thus considered by WSDOT as a possible alternative after its engineering performance and cost effectiveness are thoroughly evaluated based on Washington climate, traffic and other conditions. Concerns on such thin overlay applications including friction, reflective cracking and rutting should be evaluated carefully so that wise decisions of using 4.75mm NMAS thin overlay on appropriate pavement projects can be made.

The objective of this study was to develop mix designs and evaluate the use of 4.75mm NMAS thin overlays for WSDOT considering local conditions. Specifically, it will (1) review and summarize the general performance and application procedures of the thin overlay; (2) design and evaluate 4.75mm NMAS mixtures in the laboratory for high traffic volume road applications; and (3) estimate the life cycle cost of the 4.75mm NMAS thin overlay application based on historical data. To achieve the objectives, a comprehensive literature review and agency survey were conducted to provide a general recommendation on the design, construction, application procedures, and overall performance evaluation of the 4.75mm NMAS thin overlay with respect to other typical surface treatment methods. Because the 4.75mm NMAS mix is new to WSDOT, no mix design (particularly aggregate gradation) is available using local materials. A new packing based method was used in this study to determine aggregate gradations that can satisfy volumetric properties and improve particle interlocking. The developed mixtures were further evaluated for rutting, moisture susceptibility, and cracking potential using laboratory Hamburg Wheel Tracking test and Indirect Tensile test. A life cycle cost analysis was conducted to estimate the cost effectiveness comparing to 0.15' HMA inlay and chip seal, two preventive maintenance strategies typically used by WSDOT. Finally, based on the findings from this study, a draft special provision for 4.75mm NMAS thin overlay design and application was proposed for WSDOT.

1.3 ORGANIZATION OF THESIS

This study describes an overview of 4.75mm NMAS thin overlays and their possible effectiveness in Washington State. Chapter 1 is a basic introduction to 4.75mm thin overlays and the research to be conducted in this thesis. In Chapter 2, a thorough literature review is conducted to summarize the existing 4.75mm mix experience. A project selection criteria will also be developed in this section. Chapter 3 summarizes the survey results of State DOT's and Transportation agencies in the United States and Canada on their experience in 4.75mm NMAS thin overlay application. Chapter 4 demonstrates several trial mix designs for 4.75mm NMAS mixes and their laboratory performance. In Chapter 5 a life cycle cost analysis is developed. In Chapter 6, a preliminary special provision on mix design and special construction practice of 4.75mm thin overlay is proposed based on information gathered in the literature review and survey. These results can be used as basis for future field project application by WSDOT. Finally, conclusions and recommended future studies are summarized in Chapter 7.

CHAPTER 2: LITERATURE REVIEW

This literature review on the 4.75mm NMAS thin overlay mainly focuses on four aspects: performance, mix design, test section practice, and construction. Emphasis is given to the unique aspects of this special application that is different from the conventional HMA overlay with larger nominal maximal sieve size.

2.1 PERFORMANCE OF THIN OVERLAY PAVEMENT

There are many factors that affect the performance of a thin overlay pavement. In this section roughness, rutting, noise level, cracking, raveling, stripping, friction, delamination, service life, and life cycle costs are presented. The main distress/failure modes of thin overlays from reports include reflective cracking, rutting, fatigue cracking, raveling, stripping, and delamination. Often these are addressed with proper binder and mix selection along with proper construction practices. All of these performance factors and problem solutions are covered later in this section. Finally selection criteria will be developed to aid project selection for 4.75mm NMAS thin overlays.

2.1.1 Roughness

One of the reasons a thin overlay may be chosen over another surface treatment is because of its smoothness (ride quality). Peshkin and Hoerner (2005) stated that according to a 1998 study “Thin HMA overlays performed well, improved ride quality, reduced rutting, and reduced the severity of reflective cracking." Hall et al. (2002) stated that thin overlays not only had a significant effect on initial roughness but long term roughness as well. There are two main factors that affect the smoothness of a new thin overlay which include the condition of the existing pavement and the amount of surface preparation done prior to the application of an overlay. According to NAPA

(2009) there is a general improvement in ride quality between 40% and 60% when a thin overlay is applied. Labi et al. (2005) reported that with a thin overlay there is an 18 to 36% decrease in International Roughness Index (IRI). Ride quality of a pavement is measured by the International Roughness Index (IRI) in in/mile or m/km and a decreased IRI results in better ride quality (Chou and Pulugurta, 2008).

In a study conducted in Ohio, Chou and Pulugurta (2008) found their initial IRI decrease to be 31% and 45% on priority (4 lane divided highways) and general (2 lane undivided highways) systems respectively. The increase in ride quality from various sources ranged between 18% and 60%. Figure 2-1 and 2-2 show how ride conditions significantly increase after application of a thin overlay. The IRI decreases from 98 to 68 on the priority system and from 140 to 78 on the general system. For both systems it takes approximately 16 years for thin overlay to reach the same average IRI as the prior flexible pavement. These thin overlays in Ohio had an average expected service life of 12 years. This study shows that the improvements to smoothness from a thin overlay can be very substantial. Though there is a high variance in results, all studies show that there is an increase in ride quality with the application of thin overlays.

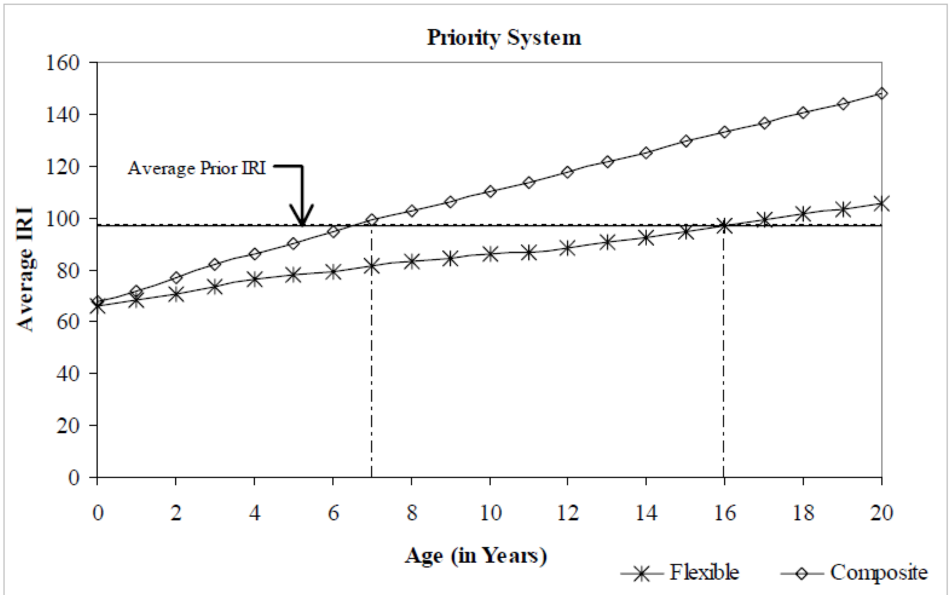


Figure 2-1: Average Ride Quality Deterioration Trend of Thin Overlay/Priority System (Chou and Pulugurta, 2008)

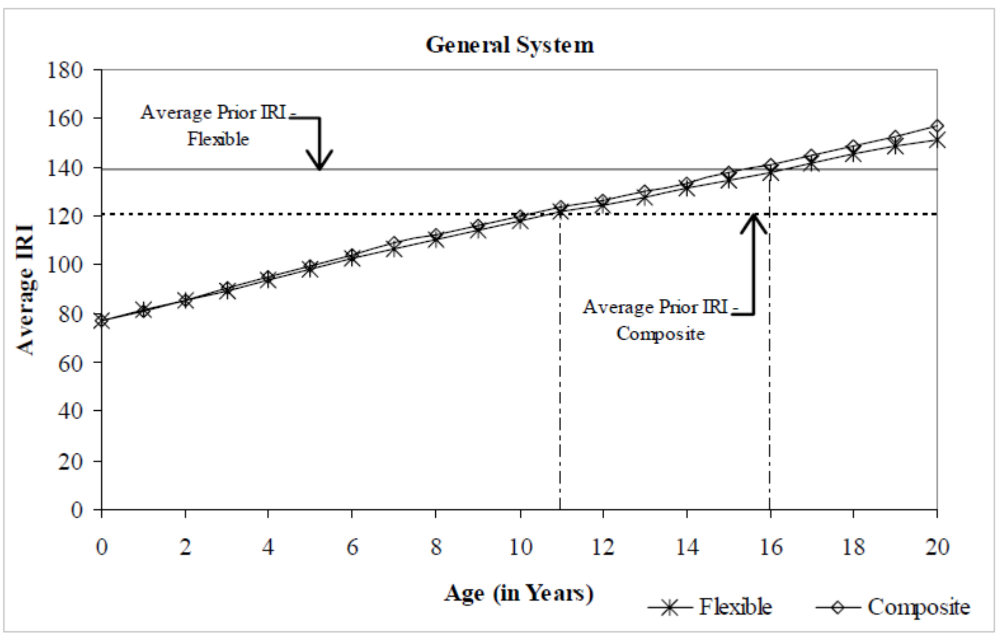


Figure 2-2: Average Ride Quality Deterioration Trend of Thin Overlay/General System (Chou and Pulugurta, 2008)

Morian (2011) summarized the possible maintenance treatments for varying distress types, which is shown in Table 2-1. As can be seen, thin overlays are the only option to address stability related roughness. Thin overlay, milling and overlay, micro-surfacing, and cape seal are the only treatments to address nonstability related roughness. Milling and overlay is a corrective maintenance treatment and should not be compared the same as the other preventive maintenance treatments. This is because corrective maintenance is used to correct failed pavement while preventive maintenance is used to prevent failures from occurring in pavement. According to Table 2-1, chip seal, sand seal, fog seal, and slurry seal do not address roughness or rutting which means they will have minimal to no effect on the ride quality. Table 2-2 shows the same result for roughness and rut correcting. From both of these tables it can be determined that the only preventive treatments that may improve ride quality are micro-surfacing, cape seal, and thin overlay.

Table 2-1: Possible maintenance treatments for various distress types (Morian, 2011)

Pavement Distress		Thin Overlay	Milling & Overlay	Micro-surfacing	Chip Seal	Sand Seal	Fog Seal	Slurry Seal	Cape Seal
Roughness	Nonstability Related	X	X	X					X
	Stability Related	X							
Rutting		X	X	X					
Cracking		X	X	X	X	X	X	X	X
Flushing/Bleeding			X	X	X				
Raveling & Wear				X	X	X	X	X	X

Table 2-2: Possible Preventive Maintenance Treatments for Various Distress Types (Hicks et al., 2000)

Pavement Distress	Crack Sealing	Fog Seal	Microsurfacing	Slurry Seal	Cape Seal	Chip Seal	Thin HMA Overlay	Mill or Grind[a]
Roughness								
Nonstability Related			X		X		X	X
Stability Related							X	
Rutting			X				X	X
Fatigue Cracking[b]		X	X	X	X	X	X	
Longitudinal and Transverse Cracking	X		X	X	X	X	X	
Bleeding			X			X		X
Raveling		X	X	X	X	X		

Key: X = appropriate strategy
[a]This is a corrective maintenance technique
[b]For low severity only; preventive maintenance is not applicable for medium to high severity fatigue cracking

From the LTTP SPS-3 study, a statistical analysis was conducted to compare average IRI values for different surface treatments. As can be seen in Figure 2-3, a thin overlay had the lowest weighted average IRI compared to slurry seal, cape seal, chip seal, and the control section which was not treated. From this study it was determined there are many factors that can affect roughness and make an option superior. In freezing conditions, high traffic, and poor pavement conditions, thin overlay outperformed the other treatments. There was no significant difference when there was no freeze, low traffic, precipitation, or good prior pavement condition. This analysis shows that thin overlays can be the best option in areas with poor conditions and hold up better to larger traffic volumes even though it is recommended they be used on low volume good quality roads. Shiarzi et al. (2010) stated thin overlays outperform other treatments in most design conditions with respect to rutting and in some cases with respect to roughness.

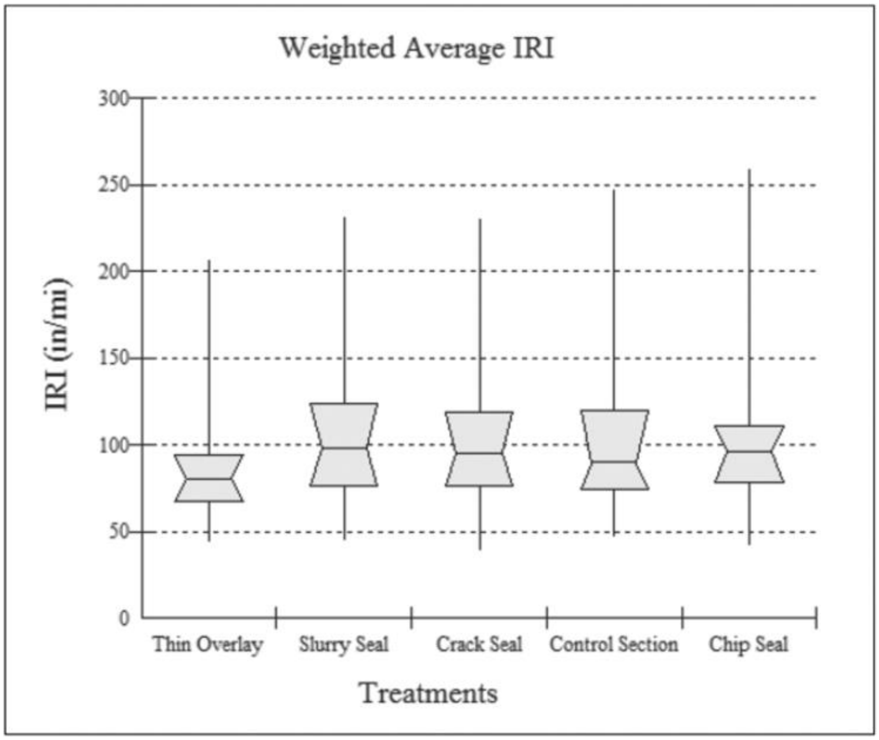


Figure 2-3: Weighted Average IRI (Shirazi et al., 2010)

Milling the previous pavement before applying a thin overlay is another option that can improve ride quality significantly. This option is recommended when roughness and cracking are present on the current pavement surface (NAPA, 2009). Milling the surface of the existing pavement can remove cracks, ruts, and other surface distresses to provide an initial level surface for placement of the thin overlay. It can provide material for recycling, avoid edge of pavement drop offs, and keep bridge clearances the same. Ride quality can only be improved to a certain point beyond the initial pavements level. This means the better the initial pavement is the better the smoothness will be once the thin overlay is applied.

2.1.2 Rutting

Rutting is defined as a distortion of the pavement surface in the wheelpath, resulting from lack of shear strength in one or more pavement layers (Hicks et al., 2000). It can also be caused by repetitive traffic producing a depression in the surface. The main factors of rutting potential in thin overlay mixes are dust content, air voids, aggregate type, and binder content. Most preventive surface treatments do not address rutting problems of the existing pavement. Hall et al. (2003) and Morian (2011) found thin overlay treatments were able to achieve significant immediate reduction in rutting. NAPA (2009) claims a 5% to 55% immediate decrease in rut depth with the application of a thin HMA overlay. In Figure 2-4 the weighted average of the 81 SPS-3 sites showed average rutting of a thin overlay was greatly lower than other treatments (Shirazi et al., 2010).

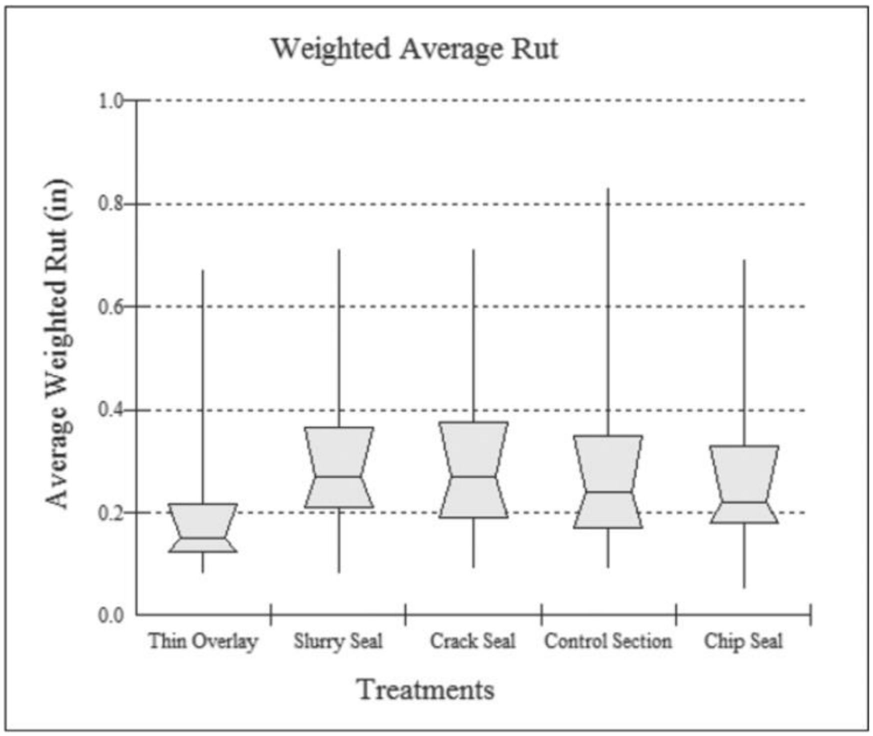


Figure 2-4: Weighted average rutting (Shirazi et al., 2010)

According to a comparison study at the NCAT test track (Powell and Buchanan 2012), after 20 million ESALs of traffic were applied the 4.75mm mix had 6mm ruts, the 9.5mm mix had 4mm ruts, and the 12.5mm mix had 4mm ruts. When measured in the lab by the asphalt pavement analyzer (APA) the rut depth after 8000 cycles of 4.75mm, 9.5mm, and 12.5mm NMAS mixes averaged 2.2, 3.4, and 3.4mm respectively. All results fell under the 4.5 to 5mm threshold commonly used to screen mixes that are suspected to exhibit poor rutting performance in the field.

So from this analysis 4.75mm NMAS mix was comparable to coarser mixes in the field and in the lab the mix actually performed better.

Williams (2006) used three separate aggregate sources (limestone, sandstone, and syenite) to develop a 4.75mm NMAS mix. From each of these aggregate 6 mix designs were created with 2 air void levels (4.5 and 6.0%) and 3 compaction levels (N_{des} = 50, 75, and 100). These mixes were evaluated for the best characteristics with respect to rutting, stripping, and permeability. Natural sand use was also evaluated in this study. 100 and 75 gyration mixes exhibited similar rutting depth while a 50 gyration mix exhibited larger rut depths in this study. Single source screening stockpiles were determined to have the ability to make rut resistant mixes (Williams, 2006). Table 2-3 compares the rutting performance of 4.75mm NMAS mix with the control 12.5mm NMAS mix for different aggregate sources using two wheel-tracking devices, ERSA and RAWT. ERSA is the Evaluator of Rutting and Stripping in Asphalt and RAWT is the Rotary Asphalt Wheel Tester. In general, 4.75mm mixes exhibited rutting resistance similar to or greater than that of 12.5mm mixes. If the density is too high after compaction the mat is also more prone to rutting.

Table 2-3: Summary of Rutting Test Data (Williams, 2006)

Rutting Test Data for 4.75mm and 12.5mm Mixtures									
Mix Design		Rutting Response							
		ERSA						RAWT	
Aggregate Source	NMAS	Avg Rut20k	Avg Rut10k	Avg Rut5k	Avg RSlope	Avg SIP	Avg SSlope	Final Rut	Rut per Cycle
LS	4.75	18.25	18.25	18.25	235	1000	163	12.56	0.00187
	12.5	17.10	16.14	8.93	1006.6	4175.0	339.5	14.00	0.00104
SS	4.75	13.60	3.95	2.45	3108.5	13150.0	965.5	6.62	0.00025
	12.5	16.01	12.18	5.44	1964.4	5031.3	434.4	11.61	0.00051
SY	4.75	5.35	2.70	2.20	8679.0	DNS[1]	8679.0	16.05	0.00224
	12.5	13.95	8.30	3.66	2353.7	6887.5	883.4	12.03	0.00070

2.1.3 Traffic Noise

Traffic noise has become an issue that state agencies are increasingly noticing. Noise is defined by Hanson et al. (2004) as the generation of sounds that are unwanted. Traffic noise can also be considered as environmental pollution because it lowers the standard of living where it occurs. Sound walls or noise barriers can be used to mitigate noise in sensitive areas and have been used since the 1970's. Improved pavement mixes and surface treatments can be an alternative or aid to reducing noise pollution.

Characteristics of the aggregate used can have an effect on traffic noise generation. Macrotexture is a characteristic of the longitudinal road profile that influences the interaction between vehicle tires and the road surface. Generally macrotexture is a large factor in pavement-tire noise generation because of its interaction between road surface and vehicle tires. The coarser the macrotexture of the surface, the noisier the traffic passing over the pavement will be (NAPA, 2009). Macrotexture values increase with larger NMAS aggregate size so the greater the NMAS the greater the noise level produced is generally true as can be seen in Figure 2-5. Also noise level was observed to be affected by NMAS rather than gradation according to Al-Qadi (2011). Small reductions in decibels can help noise levels, because every 3 dB decrease in noise would be equivalent to reducing the noise generated by traffic in half.

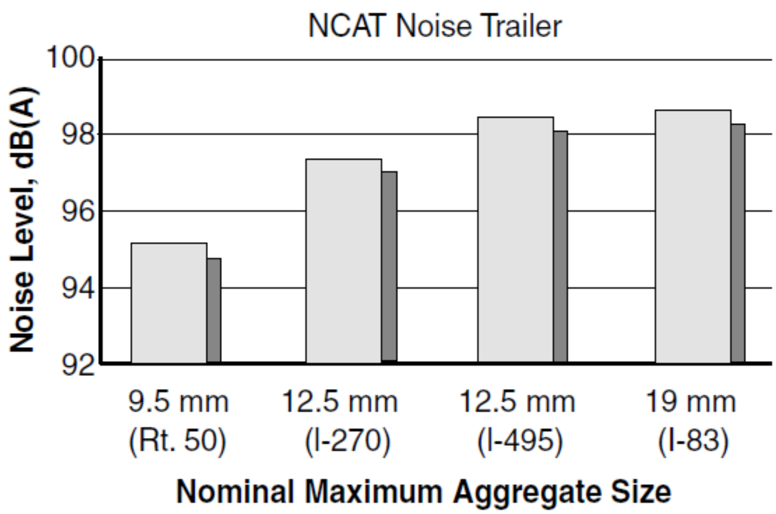


Figure 2-5: Relationship between NMAS and Tire-Pavement Noise Level (NAPA, 2009)

Different surface treatments have different effect on surface characteristics including noise, as summarized in Table 2-4. As can be seen, slurry seals, micro-surfacing, ultrathin friction course, and thin overlays all give major improvements to noise levels. Maher et al. (2005) reported that chip seals can increase noise by 2dB, while ultrathin friction course can reduce noise by 1.4 to 2.1 dB. Morian (2011) found double chip seals generate less tire noise than single chip seal but still do not have a major effect on noise. Li et al. (2012) performed a noise level test on micro-surfaced and 4.75mm overlaid roadways with a passenger car. Micro-surfacing was louder than the 4.75mm overlay on the roadside by 1.9dB, while in the vehicle micro-surfacing was only 1.4dB louder. Noise differences between 4.75mm overlay and micro-surfacing were not perceptible by human ears on the roadside or in a vehicle but thin overlays did have better noise level performance according to instrument measurements. It can be seen from these results that thin overlays have a very good potential to decrease the noise levels of an existing pavement surface.

Table 2-4: Primary benefits of different maintenance treatments (Peshkin et al., 2004)

Treatment	Roughness	Friction	Noise	Life Extension	Moisture Reduction
Bituminous-Surfaced Pavements					
Crack Sealing				X	✓
Fog Seals				X	✓
Scrub Seals				✓	✓
Slurry Seals	✓	✓	✓	✓	X
Microsurfacing	✓	✓	✓	✓	X
Chip Seals	✓	✓		✓	X
Ultrathin Friction Course	✓	✓	✓	✓	✓
Thin Overlays	✓	✓	✓	✓	✓
PCC Pavements					
Joint and Crack Sealing				X	✓
Diamond Grinding	✓	✓	✓	✓	

✓ = Major effect
x = Minor effect

2.1.4 Cracking

To be most cost effective, preventive maintenance needs to be applied in the early stages of cracking. Harvey (2009) states waiting until the later stages of cracking can lead to 14 percent higher life cycle costs compared to applying treatment in the early stages. Preventive maintenance treatments are not applicable for medium/high severity fatigue cracking. This severity fatigue cracking should be treated with corrective maintenance, not preventive. Thin overlays can correct longitudinal cracking out of the wheelpath and transverse cracking according to NAPA (2009) as well as block cracking. Maryland has used thin HMA overlays with 4.75mm NMAS mixes showing excellent resistance to cracking (Williams, 2006).

Thin overlays and chip seals were the best performers against existing fatigue cracking in the SPS-3 experiment as seen in Figure 2-6. Qi and Gibson (2011) found un-aged 4.75mm NMAS overlays performed much better for top-down fatigue cracking prevention than untreated existing pavement but once aged show little improvement. Hall et al. (2002) explains that with pavements ranging from 2 to 11 years in age, some control section had more than 4 times more fatigue cracking than thin overlays at the same sites. This shows that thin overlays can have a dramatic effect on resisting fatigue cracking even though they are not intended to be used on medium/high severities.

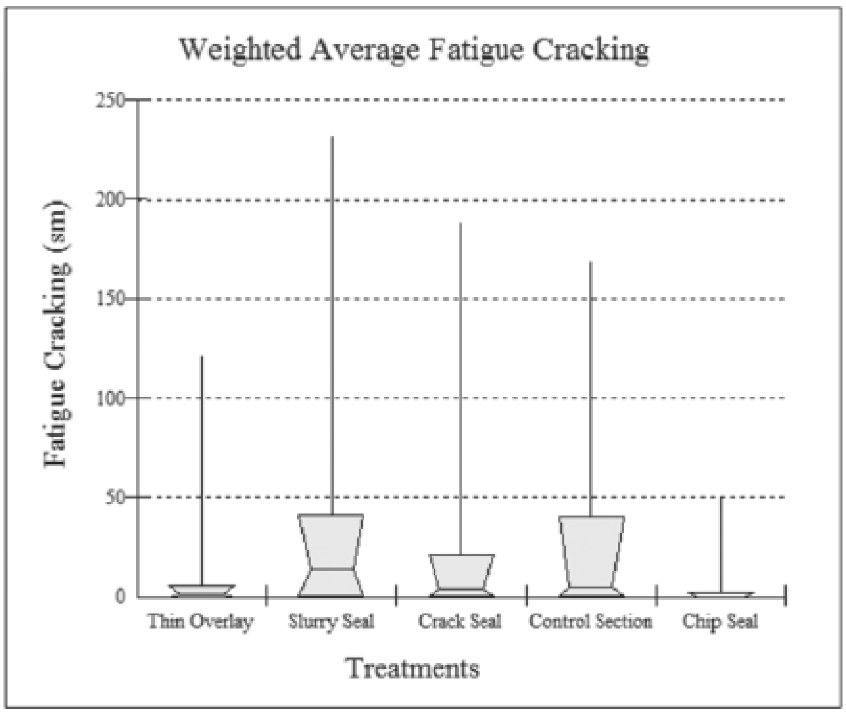


Figure 2-6: Weighted Average Fatigue Cracking (Shirazi et al., 2010)

A 4.75mm mix was developed by Virginia DOT and placed as a thin treatment on existing accelerated test sections. Half the loaded wheelpath was paved with and without treatment to see the rutting and cracking susceptibility of the 4.75mm thin treatment. An un-aged 4.75mm NMAS inlay first cracked at 425,000 passes. This was slightly lower than the neighboring control subsection which cracked at 500,000 passes at an earlier date but much higher than the aged control subsection which cracked at 50,000 passes. This shows how un-aged 4.75mm overlays have the ability to delay top down cracking of pavement. Once the overlay has aged it has nearly identical performance to that of the aged pavement without an overlay and therefore preventive maintenance should be considered again at that point.

A 4.75mm mixture's ability to sustain cracking resistance is a function of both asphalt content and dust content. Therefore, criteria for a 4.75 mm mix should include a minimum V_{be} and a maximum dust-to-binder ratio to assure good durability (NCAT, 2011). Studies recommend milling the surface to the depth of the cracking to remove the effects of the cracks prior to placement.

2.1.5 Raveling

Raveling occurs when the aggregate of the mix is not adhering to the binder. It is caused by the dislodging of aggregate particles and loss of binder and is a sign of surface aging. If significant raveling occurs it can expose the underlying binder and cause lower skid friction values. Raveling can also cause noise problems, roughness, and/or spray and splash. According to Powell and Buchanan (2012) the performance of 4.75mm NMAS thin overlays was slightly better than 9.5mm NMAS mixes in terms of raveling. 4.75mm had less change in macro texture indicating less raveling and better durability.

Almost all surface treatments address some severity of raveling because there is some material being added to the top of the raveled existing pavement covering the problem and keeping it from growing. Thin overlays are suitable for correcting raveling as long as placed on structurally sound pavement (Raush, 2006). Fog seal is a cheap surface treatment that is usually applied to address minor surface raveling. Milling can also be performed before surface treatment application to eliminate the cause of the distress (Kuennen, 2010). Table 2-5 summarizes literature recommendations when different surface treatments can be used on different raveling severity levels.

Table 2-5: Severity Level Surface Treatments Can be used

	Fog Seal	Chip Seal	Double Chip Seal	Slurry Seal	Micro-surfacing	Thin Overlay	Cape Seal
Low severity	X				X	X	X
Medium severity	X	X			X	X	X
High severity		X	X	X	X	X	

Raveling can occur because of a number of factors including hardening of the binder, moisture damage, low binder content, and low compaction (Caltrans, 2007). The amount of binder content in the mix has an effect on raveling potential because when it is too low it can leave aggregate particles thinly coated. This reduces the level of adhesion and makes the overlay more susceptible to raveling (Williams, 2006). Increased permeability of a mix to air or water can lead to higher degrees of raveling. Higher permeability leads to aging of the mix which is why higher degrees of raveling occur. Polymer modified asphalts can improve the mixture's resistance to raveling. If compaction is carried out at the proper temperature to ensure proper compaction, raveling potential is reduced.

2.1.6 Stripping

Stripping is when the asphalt binder de-bonds with the aggregate which typically begins at the bottom of the HMA layer unlike raveling. Moisture intrusion is a main cause of stripping but modern advances have reduced its prevalence (Wood et al., 2009). Caltrans (2007) reports stripping as one of the various distresses of dense graded thin overlays. Some agencies will add anti-stripping or anti-aging agents into the mix to enhance the adhesion of the binder therefore enhancing durability and reducing stripping. West et al. (2011) stated 4.75mm mixtures may be resistant to moisture intrusion with up to 9% air voids and therefore resistant to stripping. Williams (2006) also found that 4.75mm NMAS mix exhibited stripping resistance similar and sometimes greater than 12.5mm NMAS mixes.

2.1.7 Friction

Few studies have been conducted on the friction of 4.75mm mixes and results are limited. There are a few guidelines to help ensure good friction results from multiple studies. Fine aggregate angularity is an important property to ensure a high degree of internal friction for fine aggregate and aids in rutting resistance. Also using skid resistant aggregate and a gradation falling below the line of maximum packing on the .45 power gradation chart helps ensure friction improvement with appropriate micro and macro texture (NAPA, 2009).

West et al. (2011) constructed four projects to research the 4.75mm mix criteria created by NCAT. Initial friction results were obtained from the circular track meter (CTM) and the dynamic friction tester (DFT). The CTM is used to measure the macro texture of the 4.75mm HMA surface after compaction, and the Mean Profile Depth (MPD) is used to quantify the surface characteristics of the pavement. The Missouri test resulted in a MPD of 0.17 to 0.22 mm from the CTM test. These results are normal for fine graded dense HMA with a small NMAS. No DFT results were obtained from this test site. The virgin Tennessee mix resulted in DFT_{20} of 0.25 - 0.35 and a MPD of 0.16 - 0.33mm. The 15% RAP mix from Tennessee resulted in a DFT_{20} of 0.28 - 0.33 and a MPD of 0.19 - 0.33mm. The Minnesota test resulted in a DFT_{20} of 0.34 - 0.49 and a MPD of 0.13 - 0.18mm. High asphalt binder film on the surface creates lower friction values initially after application, but once the film is worn by traffic, friction characteristics improve. The level of surface texture (MPD) is normal for fine-graded HMA with small NMAS aggregates.

Testing by INDOT was conducted on four different 4.75mm overlay road sections throughout the state of Indiana. To determine the friction number (FN) the locked wheel friction testing was performed with both standard smooth and ribbed tires. Standard DF-tester and CTM tests were performed as well. DFT_{20} results from the friction coefficient at 20 km/h and should decrease when the test speed could increase. CTM is used to measure mean profile depth (MPD). Results from these tests are shown in Table 2-6 through Table 2-8.

Hard polish resistant aggregate is the key to the friction performance of a 4.75mm mix. Surface texture of the layer will not have a major effect on friction according to West et al. (2011). Initial DFT values also do not correlate directly to the maximum friction resistance because the thin asphalt film negatively affects friction until it is worn by traffic. This usually happens in a few weeks to months of traffic and then the friction values rely on the polish resistance of the aggregate. Tough and high angular fine aggregates can provide good friction in dry conditions and in wet weather at slow speeds. According to West et al. (2011), 4.75mm mixtures should not be used on heavy traffic, high speed roadways because of friction concerns.

Table 2-6: Summaries of Test Results on I-465 (Li et al., 2012)

Test Section	MPD (mm)	Friction		
		DFT_{20}	FN (smooth tire)	FN (rib tire)
4.75mm HMA on I-465	0.24	0.43	16.7	44.4

Table 2-7: Summaries of Surface Characteristics Test Results on US-27 and SR-227 (Li et al., 2012)

Test Results	US-27		SR-227	
	SB	NB	SB	NB
MPD (mm), 18 months	0.24	0.30	0.18	0.20
DFT_{20}, 18 months	0.25	0.27	0.30	0.27
FN (smooth tire), 18 month	19.7	28.6	20.1	19.8

Table 2-8: Summaries of Surface Characteristics Test Results on SR-29 (Li et al., 2012)

Test	Location	
	SB	NB
FN (smooth tire), fresh surface	32.9	36.6
FN (smooth tire), 6 months	21.6	27.6
MPD (mm), 6 months	0.21 (Scanner)	0.22 (CTM)
DFT_{20}, 6 months		0.23

Li et al. (2012) compared the frictional characteristics of various pavement surfaces and summarized the results in Table 2-9. As shown, the 4.75mm mixes demonstrated the smallest texture depth. The surface friction of this mix is much less than other mixes with larger NMAS and gap or open graded gradations. Overall, poor surface friction may be a serious problem with 4.75mm NMAS mixes.

Table 2-9: Frictional Characteristics of Various Pavement Surfaces (Li et al., 2012)

Surface Type	Friction	MPD (mm)
4.75-mm HMA on I-465, 36 months	16.7 (smooth tire)	0.24 (CTM)
4.75-mm HMA on US-27, 18 months	19.7/28.6 (smooth tire)	0.24/0.30 (CTM)
4.75-mm HMA on SR-227, 18 months	20.1/19.8 (smooth tire)	0.18/0.20 (CTM)
4.75-mm HMA on SR-29, 6 months	21.6/27.6 (smooth tire)	0.21/0.22 (CTM)
4.75-mm HMA, new (47)	62.5 (rib tire)	0.21 (CTM)
4.75-mm HMA, 15 months (47)	63.8 (rib tire)	0.26 (CTM)
12.5-mm HMA, new (47)	65.5 (rib tire)	0.39 (CTM)
12.5-mm HMA, 15 months (47)	45.9 (rib tire)	0.39 (CTM)
4.75-mm HMA, new, MODOT (51)	–	0.17–0.22 (CTM)
4.75-mm HMA, new, TNDOT (virgin mix) (51)	0.25–0.35 (DFT_{20})	0.16–0.33 (CTM)
4.75-mm HMA, new, TNDOT (15% RAP) (51)	0.28–0.33 (DFT_{20})	0.19–0.33 (CTM)
4.75-mm HMA, new, MNDOT (51)	0.34–0.49 (DFT_{20})	0.13–0.18 (CTM)
9.5-mm HMA (52)	42.5 (smooth tire)	0.59 (CTM)
9.5-mm Dense-Graded HMA (53)	–	0.53 (CTM)
12.5-mm SMA (53)	–	1.07 (CTM)
19.0-mm SMA (53)	–	1.11 (CTM)
12.5-mm OGFC (53)	–	2.31 (CTM)
Dense-graded HMA to OGFC (54)	32.5 (smooth tire)	1.63 (sand patch)
	27.4 (smooth tire)	0.53 (sand patch)
	41.4 (smooth tire)	1.57 (sand patch)
	33.4 (smooth tire)	2.19 (sand patch)
	37.8 (smooth tire)	2.97 (sand patch)
9.5-mm HMA (55)	61.9/64.3 (BPN)	0.62/0.78 (sand patch)
Polymer modified 9.5-mm HMA (55)	56.6/65.6 (BPN)	0.76/0.82 (sand patch)
Rubber modified 9.5-mm HMA (55)	60.6/64.5 (BPN)	0.78/0.73 (sand patch)
Polymer modified 12.5-mm SMA (55)	58.4/67.6 (BPN)	0.89/0.94 (sand patch)
12.5-mm SMA with fibers (55)	60.5/67.8 (BPN)	0.89/1.07 (sand patch)
19.0-mm HMA (55)	60.9/62.9 (BPN)	0.73/0.82 (sand patch)
9.5-mm PFC, new (56)	0.51 (DFT_{20})	1.37 (CTM)
9.5-mm PFC, 36 months (56)	0.52 (DFT_{20})	1.37 (CTM)
9.5-mm PFC, 60 months (56)	0.42 (DFT_{20})	1.48 (CTM)
9.5-mm SMA, newly constructed (56)	0.37 (DFT_{20})	1.17 (CTM)
9.5-mm SMA, 36 months (56)	0.61 (DFT_{20})	1.03 (CTM)
9.5-mm SMA, 60 months (56)	0.69 (DFT_{20})	0.93 (CTM)
9.5-mm HMA, newly constructed (56)	0.52 (DFT_{20})	0.30 (CTM)
9.5-mm HMA, 36 months (56)	0.39 (DFT_{20})	0.55 (CTM)
9.5-mm HMA, 60 months (56)	0.41 (DFT_{20})	0.63 (CTM)

2.1.8 Delamination

Delamination is when a proper bond is not formed between an overlay and the existing pavement, so de-bonding occurs in the form of a slippage failure. Thin HMA overlays usually have an excellent bond with the existing surface meaning delamination is not a problem (Peshkin and Hoerner, 2005). But many sources also report delamination as a possible concern with thin overlays. During application delamination can be caused by improper tack coat application, compaction results not being adequate, or the existing surface being improperly cleaned (Caltrans, 2007). These problems can be avoided by proper tack coat application and compaction strategies, as well as making sure the surface is substantially free of debris. Temperature of the mix or existing surface can also cause delamination problems. Construction crews should always make sure temperatures are adequate for paving which includes some quality control aspects. Also because thin overlays cool so much faster than traditional HMA, proper rolling strategies become more important to avoid delamination (Caltrans, 2007). If proper construction strategies take place, delamination should not be a significant problem.

2.1.9 Service Life

Service life is the number of years from initial construction of a surface treatment to its replacement. Service life of any surface treatment varies greatly between research reports. The differences in service life can be attributed to different specifications, materials, thickness, traffic loading, underlying pavement condition, surface preparation, etc. Local agencies use different asphalt grades and aggregates to construct surface treatments depending on what is available in the area. Using higher quality asphalt and aggregates can be more expensive but can give a higher service life as well. In this section, a comparison of the service life between thin overlay and other surface treatments are conducted and a summary of the average service life for each type of surface treatment is provided in the end of this section.

2.1.9.1 Thin Overlay

The expected service life of a thin overlay is longer than most other surface treatments. There have been many different studies conducted and from these studies the thicknesses of thin overlays ranged from 0.75 to 2.0 in. Outside the United States thin overlays have been used in many different countries. Thin HMA overlays have been used in Australia and the UK as 0.5 in NMAS mixes with a thickness of 0.8 to 1.6 in and reported service life is from 10 to 15 years

(Walubita and Scullion, 2008). Denmark utilizes thin HMA overlays for surfacing and waterproofing steel and concrete bridges with service life expectancy of 10 to 15 years (Walubita and Scullion, 2008). Germany has also used thin overlay but used SMA mixes with service lives of up to 18 years (Walubita and Scullion, 2008). New Zealand also uses SMA overlays from 0.5 to 1.2 in thick with expected service lives of at least 15 years (Walubita and Scullion, 2008).

In the United States, averages for service life of thin overlays are lower than abroad. Von Quintus et al. (2001) conducted survival analysis of SPS-3 sites in the Southern LTTP region and found the median survival time to be 7 years. From Table 2-10 it can be seen that the average service life of thin HMA overlays is over 8 years. A MnDOT report and national survey reported functional life of thin overlays to be 16-18 years depending on original pavement conditions. Table 2-11 shows reported thin overlay service life by different states and countries. ODOT (2001)'s Pavement Preventive Maintenance Guideline estimates that “pavements that are structurally sound, due to a recent minor or major rehabilitation, and are treated with a thin HMA overlay are expected to last 8 to 12 years”. As can be seen from all the reports there are many different time frames accepted for service life of thin overlays. From all the information gathered, 10 years was determined to be the approximate average service life for thin overlays.

Table 2-10: Performance Summaries of Thin Overlays (NAPA, 2009)

Location	Performance (years)	Reference
Ohio	16	Chou et al., 2008
Ontario	8	Uzarowski et al., 2005
Illinois	7 - 10	Reed, 1994
New York	5 - 8	New York Construction Materials Association, undated
Indiana	9 - 11	Labi and Sinha, 2003
Austria	>10	Litzka et al., 1994
Georgia	10	Hines, 2009

Table 2-11: Thin HMA Overlay Treatment Life as Reported by Various Sources (Cuelho et al., 2006)

Reference	Treatment Life (years)	Notes
Geoffroy, 1996	> 6	according to NCHRP
Geoffroy, 1996	8	according to New York State DOT
Geoffroy, 1996	8 to 11	according to FHWA
Hicks et al., 2000	7 to 10	average life in Ohio
Hicks et al., 2000	2, 7, 12	min, average, max (respectively)
Johnson, 2000	5 to 8	average reported value in Minnesota
Ohio DOT, 2001	8 to 12	expected life in Ohio
Peshkin et al., 2004	7 to 10	when placed for preventive maintenance
Wade et al., 2001	10 to 12	interstates with OGFC in Florida

2.1.9.2 Fog Seal

A fog seal is a light application of diluted slow-setting asphalt emulsion to the surface of an oxidized pavement surface (Attoh-Okine and Park, 2007). It is applied when there are minor surface defects and restores flexibility in the pavement surface. Hicks et al. (2000) suggests rutting should be less than 3/8in and cracking should be minimal for application. According to Li, et al. (2012) the maximum typical life of fog seal is 24 months without the effects of traffic. From Table 2-12 it is seen that a fog seal has an average service life from 2 years.

Table 2-12: Fog Seal Treatment Life as Reported by Various Sources (Cuelho et al., 2006)

Reference	Treatment Life (years)	Notes
Bolander, 2005	2 to 4	for ADT < 100
Bolander, 2005	1 to 3	for ADT 100 to 500
Hicks et al., 2000	2, 3, 4	min, average, max (respectively)
Hicks et al., 2000	1 to 2	average life in Ohio
Peshkin et al., 2004	1 to 2	generally reported range
Wade et al., 2001	1 to 2	generally reported range

2.1.9.3 Slurry Seal

A slurry seal is a cold-mix combination of slow-setting asphalt emulsion, fine aggregate, mineral filler, and water (Hicks et al., 2000). According to a 2008 NAPA survey slurry seals last 3.25 years. Respondents to a survey conducted by Geoffroy (1996) indicated 5 to 6 years of service life as the most repeated selection by the 13 respondents. The treatment life of slurry seal from many different sources is shown in Table 2-13.

Table 2-13: Slurry Seal Treatment Life as Reported by Various Sources (Cuelho et al., 2006)

Reference	Treatment Life (years)	Notes
Bolander, 2005	5 to 10	for ADT < 100
Bolander, 2005	5 to 8	for ADT 100 to 500
Geoffroy, 1996	1 to 6	according to NCHRP
Geoffroy, 1996	3 to 5	according to FHWA
Geoffroy, 1996	3 to 6	according to US Corps of Engineers
Hicks et al., 2000	2 to 5	average life according to Ohio DOT
Hicks et al., 2000	3, 5, 7	min, average, max (respectively)
Hicks et al., 2000	3 to 4	life expectancy from Caltrans
Maher et al., 2005	3 to 8	expected treatment life

2.1.9.4 Chip Seal

Chip Sealing (also called seal coating) is an application of asphalt followed by a layer of aggregate rolled over the asphalt layer (Gransberg and James, 2005). A double chip seal is when another chip seal is placed immediately on top of the previous chip seal. According to a 2008 NAPA survey the average single chip seal lasts 4.08 years. A survey by Geoffroy (1996) indicated the typical life of a single chip seal is 5 to 6 years. A double chip seal can be expected to last from 5 to 10 years. These results are shown in Tables 2-14 and 2-15.

Table 2-14: Single Chip Seal Treatment Life as Reported by Various Sources (Cuelho et al., 2006)

Reference	Treatment Life (years)	Notes
Bolander, 2005	3 to 6	for ADT 100 to 500
Bolander, 2005	4 to 12	for ADT < 100
Geoffroy, 1996	4	median life in Oregon
Geoffroy, 1996	4	average life in Indiana
Geoffroy, 1996	4 to 7	according to FHWA
Geoffroy, 1996	1 to 6	according to NCHRP
Gransberg & James, 2005	5.76	US average based on a survey
Gransberg & James, 2005	5.33	Canada average based on a survey
Gransberg & James, 2005	10	Australia average based on a survey
Gransberg & James, 2005	7	New Zealand average based on a survey
Gransberg & James, 2005	12	South Africa average based on a survey
Gransberg & James, 2005	10	United Kingdom average based on a survey
Hicks et al., 2000	3 to 5	average life in Ohio
Johnson, 2000	3 to 6	expected service life

Table 2-15: Double Chip Seal Treatment Life as Reported by Various Sources (Cuelho et al., 2006)

Reference	Treatment Life (years)	Notes
Bolander, 2005	5 to 15	for ADT < 100
Bolander, 2005	5 to 7	for ADT 100 to 500
Hicks et al., 2000	4 to 8	average life in Ohio
Johnson, 2000	7 to 10	depending on type and amount of traffic
Maher et al., 2005	4 to 8	average life expectancy

2.1.9.5 Cape Seal

A cape seal is a combination of a slurry and chip seal. The slurry seal is applied to the already placed chip seal to enhance its performance and reduce chip losses (Cuelho et al., 2006). The treatment life of a cape seal can range from 6 to 15 years as is shown in Table 2-16 and has an average life of 9 years.

Table 2-16: Cape Seal Treatment Life as Reported by Various Sources (Cuelho et al., 2006)

Reference	Treatment Life (years)	Notes
Bolander, 2005	6 to 8	for ADT 100 to 500
Bolander, 2005	8 to 15	for ADT < 100
Maher et al., 2005	7 to 15	typical serviceable life

2.1.9.6 Scrub Seal

A scrub seal is a variation of chip seal where a polymer-modified asphalt emulsion is sprayed on the pavement and broom scrubbed (Cuelho et al., 2006). This sweeping process fills cracks in the chip seal. The treatment life of scrub seal ranges from 1 to 6 years as can be seen in Table 2-17 and has an average life of 4 years.

Table 2-17: Scrub Seal Treatment Life as Reported by Various Sources (Cuelho et al., 2006)

Reference	Treatment Life (years)	Notes
Bolander, 2005	2 to 8	for ADT < 100
Bolander, 2005	2 to 6	for ADT 100 to 500
Maher et al., 2005	2 to 6	for ADT < 1,500
Peshkin et al., 2004	1 to 3	generally reported range
Wade et al., 2001	3 to 4	typically reported range

2.1.9.7 Sand Seal

Sand seal is similar to chip seal in that a layer of asphalt emulsion is covered by clean sand or fine aggregate. This is mainly done to seal the pavement surface and improve surface characteristics. An overall performance life of 3 to 4 years can be expected with sand seal (Morian, 2011).

2.1.9.8 Crack Seal

Crack sealing is a widely used preventive maintenance treatment that is applied to keep water out of cracks in the pavement structure. Sealing cracks can extend the service life of pavements by 2 to 5 years (Wood et al., 2009). A survey by Geoffroy (1996) reported the most repeated result was 2 to 4 years of service life. From Table 2-18, the average service life is approximately 3 years.

Table 2-18: Crack Treatment Life as Reported by Various Sources (Cuelho et al., 2006)

Reference	Treatment Life (years)	Notes
Geoffroy, 1996	2.2	Crack seal in Indiana
Geoffroy, 1996	2 to 5	Route and seal in Ontario
Geoffroy, 1996	2	Crack fill in New York
Geoffroy, 1996	2 to 5	Route and seal in New York
Johnson, 2000	7 to 10	Average reported value in Minnesota, performed on new pavement

2.1.9.9 Micro-surfacing

Micro-surfacing is a modified slurry seal and is a mixture of polymer-modified emulsion, mineral aggregate, mineral filler, water, and other additives spread onto a pavement surface (Cuelho et al., 2006). The fine aggregate of the mixture allows thin application and is generally not compacted. A 2008 NAPA survey showed micro-surfacing to have a service life of 4.67 years.

From Table 2-19 it can be seen this procedure has a general life of 4 to 7 years and an average of 6.5 years.

Table 2-19: Micro-surfacing Treatment Life as Reported by Various Sources (Cuelho et al., 2006)

Reference	Treatment Life (years)	Notes
Geoffroy, 1996	4 to 6	according to NCHRP
Geoffroy, 1996	5 to 7	according to FHWA
Johnson, 2000	7	for high volume roads
Johnson, 2000	>7	for low volume roads
Labi et al., 2006	5	based on roughness
Labi et al., 2006	7	based on pavement condition rating
Labi et al., 2006	24	based on rutting
Peshkin et al., 2004	4 to 7	from review of literature
Smith & Beatty, 1999	7 to 10	suggested expected life
Wade et al., 2001	4 to 7	generally reported range

2.1.9.10 Ultrathin Friction Course

An ultrathin friction course is hot-mix asphalt with gap-graded aggregate placed on a polymer-modified asphalt emulsion coat (Cuelho et al., 2006). The thickness ranges from 0.375 to 0.75 in. This treatment can also be referred to as NovaChip® which was the first ultrathin friction course. Being a relatively new technology, the service life is not entirely known. Based on Table 2-20 it can be reasonably expected to last at least 7 years.

Table 2-20: Ultrathin Friction Course Treatment Life as Reported by Various Sources (Cuelho et al., 2006)

Reference	Treatment Life (years)	Notes
Gilbert et al., 2004	8 to 12	expected life in South Africa
Maher et al., 2005	10 to 12	typical serviceable life
Peshkin et al., 2004	7 to 10	generally reported range

2.1.9.11 Summary

Below are a few tables from various sources comparing the service lives of different surface treatments. Table 2-21 shows state DOT's responses to a survey where service life of various surface treatments was to be determined. Table 2-22 shows how service life is affected by the existing pavements PCI. As shown when the initial PCI is better every type of treatment last longer than when the initial PCI is lower.

Table 2-21: Summary of state DOT treatment life reported in survey (Morian, 2011)

Treatment Type	State Highway Agency												
	WV	VA	OH	NH	MI	MD	MN	IN	NC	TX	CT	KY	NY
Thin Overlay	8	10	10	7	3-6	5-10	8-12	9	10	10-12	NR	7	8-10
Micro-surfacing	8	6	8	6	3-6	NR	12	8	7	5-7	NR	8	8-10
Crack Sealing	4	N/A	2	5	1-3	NR	5	3	3	2-3	NR	4	2-5
Chip Seal	5	6	7	5	3-6	NR	10	4	7	7-8	4	NR	4-6
NovaChip®	NR	10	NR	10	3-6	NR	15	8	8	8-10	10	NR	8-10
Fog Seal	NR	5	NR	NR	NR	NR	3	2	NR	NR	NR	NR	NR
Cape Seal	NR	NR	NR	NR	NR	NR	NR	NR	8	NR	NR	NR	NR
Sand Seal	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR	NR
Slurry Seal	NR	6	NR	NR	NR	NR	NR	NR	3	NR	NR	NR	4-6

Note: NR: No Response

Table 2-22: Service Life of Various Treatments under different PCI's (Morian, 2011)

Treatment	Good condition PCI=80	Fair condition PCI=60	Poor condition PCI=40
Fog seal	3 - 5	1 - 3	1 - 2
Chip seal	7 - 10	3 - 5	1 - 3
Slurry seal	7 - 10	3 - 5	1 - 3
Microsurfacing	8 - 12	5 - 7	2 - 4
Thin HMA	8 - 12	5 - 7	2 - 4

From the information gathered, Table 2-23 was created showing minimum, average, and maximum service life of each surface treatment mentioned earlier. Thin overlay was determined to have the longest average service life but also had the highest range from maximum to minimum. Papers reported a large variance in thin overlay performance which shows more localized studies should be conducted to find the service life in that area.

Table 2-23: Service Life of Pavement Rehabilitation Treatments

Treatment Type	Researched Service Life (Years)		
	Minimum	Average	Maximum
Thin Overlay	2	10	16
Micro-surfacing	3	6.5	10
Slurry Seal	1	5	10
Chip Seal (single)	1	5.5	12
Chip Seal (double)	4	7	15
Fog Seal	1	2	4
Ultrathin Friction Course (NovaChip)	4	8.5	12
Crack Seal	1	3	10
Cape Seal	6	9	15
Sand Seal	2	3	5
Scrub Seal	1	4	8

2.1.10 Life Cycle Costs

Life cycle costs are the costs through the whole life of the pavement from construction to replacement. There are multiple costs that are used to determine the overall cost of pavement management activities. This overall cost is then used along with service life and inflation and interest to compute the life cycle cost of certain treatments. Costs of materials varied greatly in research because of the year they were found and because different states material costs can vary. It is best to find the costs of material in your area than showing a list of outdated non-relevant cost information.

In 2008 the National Asphalt Pavement Association (NAPA) conducted a survey of state asphalt associations on the cost and effectiveness of several pavement preservation treatments. Thin overlays had a larger initial cost than the other treatments but because the expected life is so much higher it becomes the most cost efficient option as can be seen in Table 2-24.

Table 2-24: 2008 NAPA Survey of State Asphalt Associations (Newcomb, 2009)

Treatment	Expected Life, yrs	Range	Cost, $\$/yd^2$	Range	Annual Cost, $/lane-mile
Chip Seal	4.08	2.5 – 5	2.06	0.50 – 4.25	3,554.51
Slurry Seal	3.25	2 – 4	1.78	1.00 – 2.20	3,855.75
Micro-surfacing	4.67	4 – 6	3.31	2.30 – 6.75	4,989.81
Thin Surfacing	10.69	7 – 14	4.52	2.40 – 6.75	2,976.69

Michigan DOT (MDOT) has been applying thin overlays on their roadways for many years as preventive maintenance. They have low, medium, and high volume classification mixes for the different roadways that the overlays are built on. Low has less than 380 two way truck ADT, medium has 380 – 3400, and high has greater than 3400. In Michigan as of October 2008, the initial costs of ultra-thin overlays were comparable to that of double chip seals and micro-surfacing. Since the overlays last much longer their annual cost per mile becomes far less than the other alternatives as can be seen in Table 2-25.

Table 2-25: Prevention Maintenance Treatments Cost Comparison (Huddleston, 2009)

Treatment	$/yd^2	Cost/mile (24' wide)	Life extension range (years)	APAM Life extension range average (years)	Cost/mile per year
Double Chip Seal	2.40	33,791	3-5	4	8,448
Micro-surface	2.44	34,354	3-5	4	8,589
Ultra-thin low	2.20	30,975	5-9	7	4,425
Ultra-thin med	2.55	35,903	5-9	7	5,129
Ultra-thin high	2.83	39,845	5-9	7	5,692
Single course overlay (1.5")	4.12	58,078	5-10	7.5	7,743
Mill and Fill (1.5")	5.15	72,509	5-10	7.5	9,668

The Minnesota Asphalt Pavement Association (MAPA) researched the life cycle costs of several different surface treatments in 2010. They used the national state average costs for 5 different types of surface treatments. One interesting note is they consider thin HMA overlay to have a 15 year service life. One of their sources is the NAPA IS-135 document where the only state that reported performance greater than 11 years was Ohio at 16 years. The other source is a MnDOT document that states "Average of 12 to 16 years, but highly dependent on condition of existing pavement." (Wood et al., 2009). There is documentation that it may last 15 years but this number is at the end of the service life range and is not an average. From this analysis they determine fog seal to be the cheapest option but it has a different functionality than the other surface treatments and wouldn't be used to resurface anything but minor irregularities in a surface. A thin overlay is the second most cost effective option and is approximately $2000 dollars cheaper for each mile a year as can be seen in Table 2-26.

Table 2-26: Minnesota Asphalt Pavement Association (Wolters and Thomas, 2010)

Treatment	Initial Cost/yd^2	Years Life	Cost/Service Life/yd^2	Life Cycle Cost per Mile
Thin Seal (Fog)	0.25	2	0.12	1,690
Chip Seal	1.30	4	0.32	4,506
Slurry Seal	1.40	4	0.35	4,928
Micro-surfacing	1.95	4	0.49	6,899
Thin HMA Overlay (1")	2.70	15	0.18	2,534

From these studies it can be seen that thin overlays are a cheap and long lasting alternative in preventive maintenance of pavement. In these three studies they were the lowest costing treatment besides fog seal and should be considered because of their cost effectiveness.

2.1.11 Project Selection Criteria

When selecting a pavement surface to apply a thin HMA overlay to, at least two aspects should be considered:

Existing pavement condition. The existing pavement should have a sound structure because a thin overlay is a preventive maintenance treatment that does not address

failures. Cracks should be confined to the surface layer and rutting should be caused by the pavement layer and not the underlying base layer (NAPA, 2009). There should not be high amounts of load related distress and no more than 10% medium or 2% high severity fatigue cracking present. Medium severity wheel track cracking should be repaired to full depth before an overlay is placed because this cracking has a high potential to reflect through the new surface. Deteriorated cracks and localized failures should also be repaired because they have a high potential of reflecting as well. Wade et al. (2001) suggests rutting should be limited to 1.0 in and potholes should be repaired to full depth and rut filling should have taken place in the past if rutting has been a problem in the area. If previous patches exist and are in good condition they should not pose a problem to the new surface (Wood et al., 2009). There should be no more than 20% medium to high severity patching either (Wade et al., 2001). Crack sealing should not have been performed in the previous year because it can cause problems during construction that result in bumps.

Pavement condition rating (PCR) can be used to determine when a thin overlay should be applied to an existing pavement surface. On a priority system (4 lane divided highways) the PCR score should be 70 to 90 while on the general system (2 lane undivided highways) the PCR score should be 65 to 80. These values can be used as a guideline to determine when or if to apply a thin overlay if existing pavements are graded on the PCR scale (Chou and Pulugurta 2008).

Thin overlays are also applied to correct functional problems. These include roughness, skid resistance, and noise generation. If the existing surface was constructed with polished aggregate or has bled it may also be a candidate of thin overlay for friction improvement (NAPA, 2009). The amount of needed friction improvement will depend on road classification, speed limit, geometric considerations, and the presence of cross traffic. Friction improvement can be accomplished with a thin overlay by using a skid-resistant aggregate and a gradation that falls below the line of maximum packing on the 0.45 power gradation chart. This will ensure the appropriate micro- and macro- texture. Roughness is affected by the cracking and rutting in the existing layer and can be improved appreciably by milling the existing surface. Areas of ponding or poor subsurface drainage need to be identified and corrected before a thin overlay is applied (NAPA, 2009).

Milling should be performed before application of a thin overlay if certain features exist. If high severity raveling or bleeding is present then the surface should be milled. Milling is also recommended where there is severe cracking present, to correct the surface profile, or when curbs are present (Wood et al., 2009). If rutting is evident it may either be milled or a leveling course may be used instead. Surface preparations for certain distress types and severities can be seen in

Table 2-27. Milling can also help maintain drainage features such as curbs and storm-water inlets or drains, and will help edge of pavement drop offs, loss of bridge clearance, and manhole adjustments due to buildup of pavement overlays (NAPA, 2009).

Table 2-27: Suggested Approaches to Surface Preparations Prior to Thin Overlay

Distress Type	Recommended Investigation	Extent	Severity	Surface Preparation Prior to Overlay
Raveling	Visual Observation	Up to 100% of Pavement Area	Any	Clean and Tack
Longitudinal Cracking (non-wheelpath)	Visual Observation and Coring	Crack Depth Confined to Surface Layer	Low/Medium	Mill to Crack Depth or Fill Crack, Clean, and Tack
Longitudinal Cracking (wheelpath)	Visual Observation and Coring	Crack Depth Confined to Surface Layer	Low	Mill to Crack Depth or Fill Crack, Clean, and Tack
Transverse	Visual Observation and Coring	Crack Depth Confined to Surface Layer	Low/Medium	Mill surface, Clean, Fill Exposed Cracks, Clean, and Tack
Alligator of Fatigue Cracking	Visual Observation and Coring	Crack Depth Confined to Surface Layer	Low	Mill to Crack Depth, Clean, Fill, and Tack
Rutting or Shoving	Visual Observation and Transverse Trench or Coring	Rutting Confined to Surface Layer	Low/Medium	Mill to Depth of Surface Layer, Clean, and Tack

Traffic level. There is no consensus on the traffic level where 4.75mm thin overlays should be applied. Some agencies report that thin overlays can be used on all traffic levels (Wood et al., 2009) and some like NAPA report 4.75mm thin overlays should only be used on low volume roadways. Low volume roadways seem to be the most prevalent suggestion though. Low volume is also defined differently by varying agencies. ODOT (2001) state low volume traffic is less than 2500 ADT. However, low is defined by Hicks et al. (2000) as less than 1000 ADT while Li et al.

(2012) reports low volume to be less than 2000 ADT. There also have been limits on annual ESALs proposed. Chou et al. (2008) stated that annual ESALs above 200,000 decrease the performance of thin overlays. Zaniewski and Diaz (2004) suggest medium volume roads being less than 3 million ESALs for 20 year design periods, which equates to 150,000 annual ESALs. They define low as 0.3 million ESALs over a 20 year period which equates to 15,000 annual ESALs.

2.1.12 Concerns

The following section notes the concerns with 4.75mm thin overlays found in the literature review.

2.1.12.1 Reflective Cracking

Reflection cracking is noted as being one of the possible failure modes of 4.75mm thin overlays. If longitudinal cracking is too high severity in the existing surface, it has a high probability of propagating through the new surface. Johnson (2000) stated deteriorated cracks and localized pavement failures can quickly reflect through the new overlay to cause major distress to the new surface. Some adjustments to the mix or preparations can be used to help slow reflective cracking but cannot usually prevent it. The thickness of the overlay has not shown any effect on reflective cracking. Using modified binders can help address reflective cracking so this should be an option if reflective cracking is expected (Caltrans, 2007). Also milling the surface is another option to remove the cracks and the possibility of them reflecting to the surface. If the cracks are the full depth of the existing pavement layer or too deep to mill there is a high possibility they will reflect to the surface. The best way to avoid reflective cracking is to place the overlay while the existing pavement layer is in good condition with little cracking. Placing before high levels of cracking occur is always the best option but is not always an option.

2.1.12.2 Winter Damage

Winter damage to thin overlays is a concern that agencies have with 4.75mm NMAS mixes but little research has been conducted on the topic. Thin surfaces are often susceptible to snowplow damage as the plow blade rides on the surface removing aggregate. This is a large problem with chip sealed surfaces because aggregate particles may not be securely bonded to the surface. Also rutting can cause the blade to ride on the high surfaces giving greater chance for damage. Jahren et al. (2003) states thin overlays should not have a problem with snowplow damage because of the small aggregate gives less of a surface for the blade to grab and rip out.

Another source of winter damage comes from rutting produced by studded tires. Studded tires are used in the winter to reduce snow and ice related accidents. WSDOT (2010) states that all pavement types are effected by studded tires. The studded tires wear down the pavement at a much greater rate than normal pavement tire interaction. These studs abrade the pavement surface and may prevent certain pavement preservation treatments in areas of extensive studded tire usage (Zubeck and Liu, 2012). According to Zubeck and Liu (2012) crack sealing, patching, and thin overlay are most commonly used in areas of heavy studded tire usage.

2.1.12.3 Friction

Friction is the main concern when dealing with 4.75mm NMAS mix type. In theory, the small sized aggregates give the surface less macro texture and less friction between tire and roadway surface but few studies have been conducted to confirm this. Four test sections conducted by NCAT gave normal friction results for small NMAS aggregates according to West et al.(2011). INDOT also conducted a study on four test sections which gave poor friction results when compared to larger NMAS mixes. Friction results were shown to be good after initial construction but reduce 20 to 50% after 12 months. The key to good performance in respect to friction is using hard polish resistant aggregates. Friction is a major concern with 4.75mm NMAS mixes and more research on the subject needs to be completed to see the effects of the small NMAS on friction.

2.2 MIX DESIGN

In the past, state DOT's have been skeptical about using small aggregate size mixes because of the increased rutting susceptibility due to the small NMAS. NCAT has shown fine mixes have no more rutting potential than coarse mixes. In 2002, 4.75mm NMAS designation and criteria were added to the AASHTO Superpave specifications. These criteria were mostly based on experience, limited laboratory research, and engineering judgment (Rahman et al., 2010). This fact made it a priority for additional research to be conducted to refine the mix design and base it upon more performance results. The majority of research on 4.75mm mix design has been conducted by the National Center for Asphalt Technology (NCAT). However state agencies and other entities have also conducted some research and all this research combined have further refined the mix design of 4.75mm NMAS mixes. There are choices that the implementing agency will have to decide upon and these results can be used as guidelines to making those choices.

2.2.1 Aggregate

There are many types of aggregate that can be used to effectively create a 4.75mm NMAS mix. From research it can be seen there are natural and synthetic materials that can be blended to make a proper mix. Natural materials used in several studies included granite, limestone, sandstone, syenite, dolomite, crushed stone, crushed gravel, and natural sand. Synthetic material used include taconite tailings, blast furnace slag (BFS), and steel furnace slag (SF). Taconite tailings are a by-product from taconite mining in Minnesota and mostly end up in landfills around mines. Eshan (2011) reported these tailings performed well as the main source of aggregate in a Minnesota study. Aggregate choice will depend on what is available in the area of construction.

Only fine aggregate stockpiles can be selected to blend 4.75mm NMAS gradations. Fine aggregate angularity (FAA) and natural sand content need to be controlled in the mix to ensure a high degree of fine aggregate internal friction and helps prevent severe rutting (Zaniewski and Diaz, 2004). Cooley Jr. et al. (2002) reports FAA should be 40 or greater for less than 0.3 million design ESALs and 43 and above for 0.3 to 3 million design ESALs. This helps ensure rounded particles do not make up the entire aggregate blend. Cooley Jr. et al. (2002) notes these are suggestions from this study for 4.75mm NMAS mixes but no specific FAA requirements were conducted.

An increase in natural sand can cause performance problems in a 4.75mm NMAS mix. Cooley Jr. et al. (2002) suggests the use of natural sand should be limited to 15 - 20% for high volume roadways and 20 - 25% for low/medium volume roads. This helps control the detrimental rutting effects that natural sand can cause in excess. There is also evidence that natural sand content above 15% can adversely affect moisture and rutting susceptibility as well as permeability so this should be considered (Cooley Jr. et al., 2002). Zaniewski and Diaz (2004) found that over 10% natural sand resulted in increased rutting and over 20% natural sand resulted in pronounced rutting potential. Though the amount of natural sand varies between reports it is agreed upon that too much natural sand can cause problems in the mix.

2.2.2 Gradation

Gradation of an aggregate is one of the most influential properties that determine the performance of a mix. Zaniewaki and Diaz (2004) suggest 4.75mm mixes should be controlled at the 1.18mm sieve with 30 to 54% passing and the 0.075mm sieve with 6 to 12% passing. The control point on the 0.075mm sieve is the dust content of the mix. The suggested control sieve by Cooley Jr. et al. (2002) for 4.75mm mixes is also the 1.18mm sieve. From the study done by Cooley

Jr. et al. (2002) the limits of 30% to 54% passing the 1.18mm sieve seemed reasonable. Limestone had better rutting results at 54% passing while granite mix had better rutting results at 30% passing. The gradation requirements from different agencies and states that have implemented 4.75mm mixes are shown in Table 2-28. Figure 2-7 shows gradation curves for 4.75mm mixes of several states.

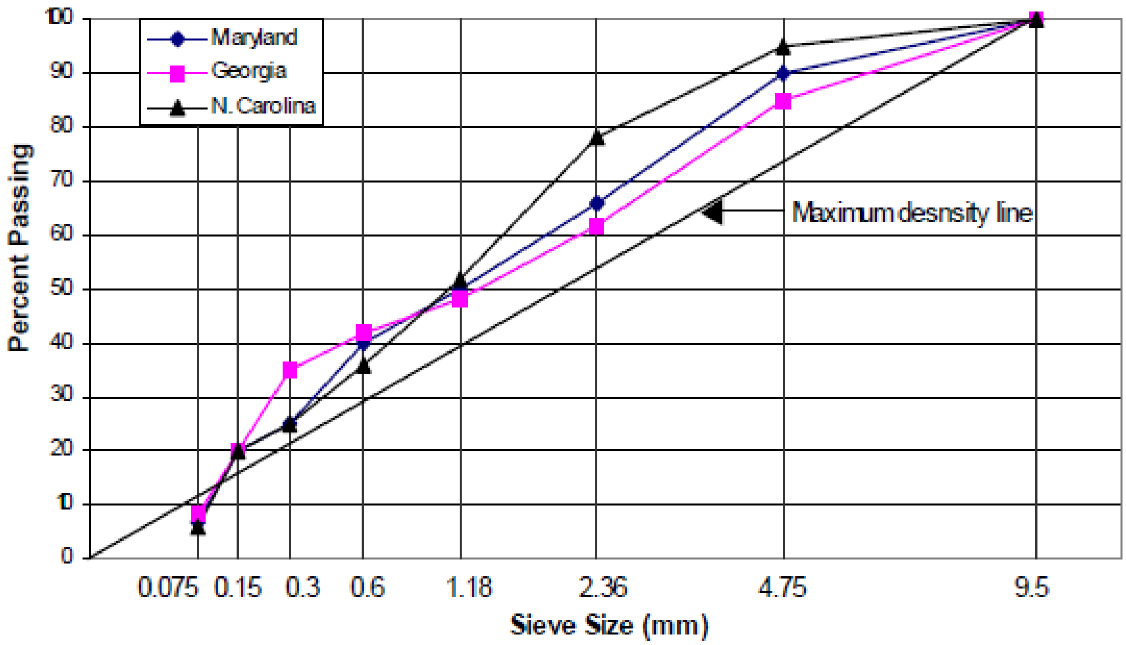


Figure 2-7: Typical gradation curves for 4.75mm mixes (Zaniewski and Diaz, 2004)

Table 2-28: Gradation Requirements of Several Agencies and States

% Passing	AASHTO Original	NCAT Suggested	Georgia	Maryland	North Carolina	West Virginia	New Jersey
9.5mm Sieve	95 - 100	95 - 100	90 - 100	100	100	100	100
4.75mm Sieve	90 - 100	90 - 100	75 - 95	80 - 100	90 - 100	90 - 100	65 - 85
2.36mm Sieve			60 - 65	36 - 76	65 - 90	< 90	33 - 55
1.18mm Sieve	30 - 60	30 - 55				40 - 65	20 - 35
0.6mm Sieve							15 - 30
0.3mm Sieve			20 - 50				10 - 20
0.15mm Sieve							5 - 15
0.075mm Sieve	6 - 12	6 - 13	4 - 12	2 - 12	4 - 8	3 - 11	5 - 8

2.2.3 Dust Content

Dust content is the percent of aggregate passing the 0.075mm sieve and has a considerable effect on VMA and rutting. In general, as the percent dust content increases, VMA decreases. According to Williams (2006) for every 3% increase in dust content, optimum binder content decreased by an average of 0.5%. 6% dust content had higher film thickness results by about 2 to 3 micrometers than a higher dust content of 12%. When dust contents decreased, rut depths increased as can be seen in Figure 2-8. This is due to the fact that lower dust content mixes have higher design binder contents. The original specification set by AASHTO was 6 to 12% dust content. Other agencies have specifications with a requirement as low as 2%, but 4% and 6% are more common as a lower threshold.

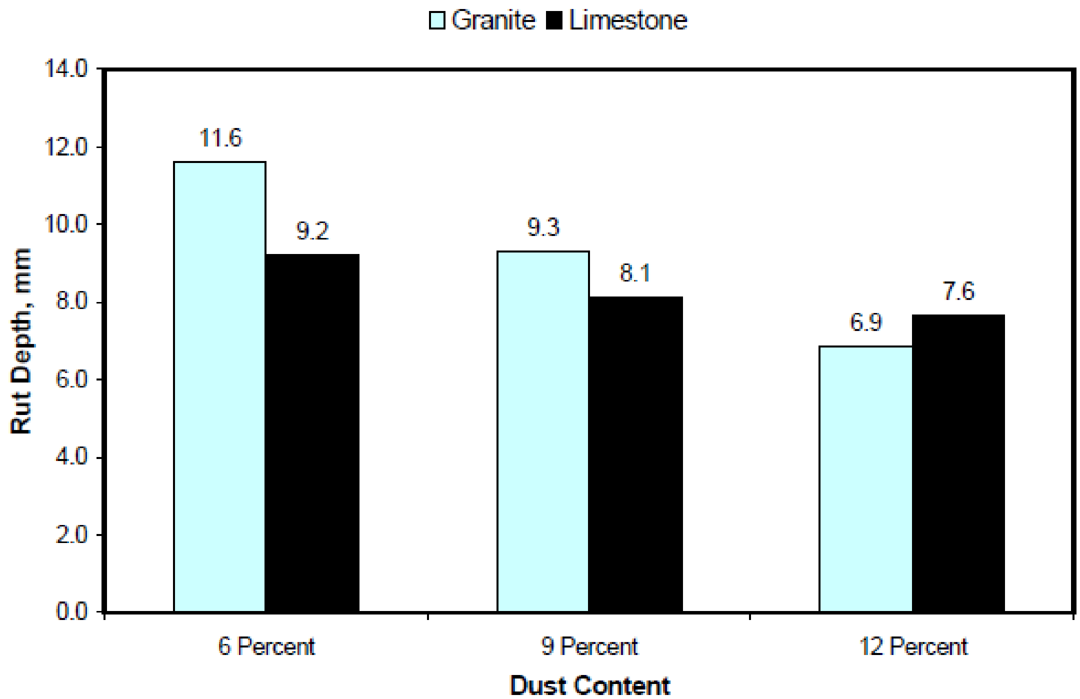


Figure 2-8: Interaction between Aggregate Type and Dust Content (Cooley Jr. et al., 2002)

2.2.4 Asphalt Binder

Asphalt binder is used to bond the aggregate structure of the mix together and is considered the "glue" of the aggregate structure. During compaction it acts as a lubricant aiding in consolidation and reducing space between aggregate particles (Raush, 2006).

The appropriate binder content must be selected to reach a balance of acceptable performance with respect to multiple failure modes. Zaniewski and Diaz (2004) in their study of 4.75mm mix design for West Virginia found the optimum binder content for 4% air voids ranged from 5.0 - 6.8% and 5% air voids ranged from 4.8 - 6.3%. Increasing the air voids by 1% results in a decrease in optimum asphalt content of 0.38% on average. Increases in dust content by 4% will decrease the optimum binder content by about 0.7%. Fine aggregate mixes required asphalt content of 5.9%. Table 2-29 shows the typical binder content ranges from various sources.

Table 2-29: Typical Binder Content Range for 4.75mm Mix from Various Sources

	Arkansas	Georgia	Maryland	Tennessee
% Binder Content	4.5 - 7.5	4 - 7	5 - 8	7 - 11

Laboratory tests were performed by NCAT to find a cheaper asphalt mix that could be used and still perform well. The reason to find a cheaper alternative was because the asphalt content of the 4.75mm mix was 1% higher than the more standard 12.5mm mix in NCAT test track results (Powell and Buchanan, 2012). This is a disadvantage because the higher cost per ton of the mix may discourage usage. Several cost reduction technologies shown in Table 2-30 were then tested in an APA test using a load of 445N under a 689KPa hose pressure. The samples were subject to 8000 load cycles at 64C (Powell and Buchanan, 2012).

Table 2-30: Potential Cost Reduction Technologies Included in Laboratory APA Study (Powell and Buchanan, 2012).

Treatment	Brief Description of Technology
iBind	Phosphate waste product filler with fiber properties
Wool Fibers	Sometimes used to stabilize intersection mixes
Thiopave	Sulfur replacement warm-mix asphalt package
TLA Pellets	Natural mined asphalt binder from Trinidad Lake
50% Fine RAP	Fractionated RAP from 2009 Pavement Test Track
5% RAS	Industrial waste from roofing shingle production

The mixes that produced lower rut depths than the PG76-22 control mix were the 50% Fine RAP, Thiopave, and 5% RAS mixes. The 50% fine RAP and Thiopave with 7% binder showed similar rutting depths as the control PG70-22 mix with 6% binder (Powell and Buchanan, 2012). These mixes showed good rutting performance with lower costing materials and with higher binder contents and may be able to offset durability and fatigue concerns.

2.2.5 Dust-to-Effective Binder Ratio (D:B ratio)

D:B ratio is the aggregate fines to effective asphalt content ratio. Fines or dust content is determined using the percent of aggregate passing the 0.075mm sieve. D:B ratio is used to ensure there is sufficient asphalt to coat the mineral filler in a mix, and is a major contributor to the cohesion of the mix (Zaniewski and Diaz 2004, Williams 2006). Fines stiffen the binder and affect the rutting potential of the mix. From the original AASHTO guidelines the D:B ratio was 0.6 to 1.2. They also noted that if the gradation passes below the restricted zone the D:B ratio could be 0.8 to 1.6. Based on Maryland and Georgia reports, the upper limit should be a D:B ratio of 2.2 for 4.75mm mixes. According to a later study it was suggested the lower limit be raised from 0.9 to 1.0 and the upper limit stay at 2.0 (NCAT, 2011). Table 2-31 shows what different sources report as a proper ranges for D:B ratio.

Table 2-31: D:B Ratio Range from Various Sources

	AASHTO	NCAT	Arkansas	(Williams, 2006)	West Virginia
D:B Ratio	0.9 - 2.0	1.0 - 2.0	0.6 - 1.4	0.9 - 2.2	0.6 - 1.2

2.2.6 Design Air Voids

Design air voids are defined as the total volume of voids between the coated aggregate particles throughout the compacted paving mixture. It is expressed as a percent of the bulk volume of the compacted paving mixture (Cooley Jr. et al., 2002). In general, 4.75mm mixtures are most stable with air voids between 3 and 8% (Williams, 2006). If air voids are too low it indicates premature densification which could increase instability and shear deformation in the mix. If air voids are too high it makes the mix more permeable which can cause oxidization, stripping, and raveling. The original AASHTO specification for 4.75mm mixes calls for 4% air voids alone which provides the desired characteristics. Other sources like Rahman and Romanoschi (2011) and West et al. (2011) recommend using 6% air voids for 4.75mm mixes. In general, a range of design air voids between 4 and 6% could be used for 4.75mm mixes depending on different applications. Table 2-32 shows designated air voids from various sources.

Higher air voids (around 6%) are more suitable for low/medium volume roads while lower air voids (around 4%) are suitable for any traffic level. 6% air voids mix is not suggested for higher traffic levels according to Williams (2006). Tests have been performed to determine the performance of mixes at different design air voids. Williams (2006) examined 4.5% air voids against 6% air voids and Cooley Jr. et al. (2002) tested 4% against 6% air voids. For stripping and rutting, 100 gyration mixes performed better for 4.5% than 6% air void mixes as expected. Rutting performance is better under 6% air voids but is not affected much by the number of gyrations. Better rutting results with higher air voids can be attributed to a reduced binder content of the mix. 6% air voids are more sensitive to compaction levels than 4.5% air voids, but this mix type is still essentially impermeable.

Different design air voids also affect VMA results. When 4.5% air voids were used, VMA percentages were in the low to middle portion of their acceptable range. While with 6% air voids, VMA was close to the maximum allowed value. If selecting 4% air voids it most likely means the maximum VMA limit need to be set to prevent excessive binder content in the mix. Mixes designed with 4% air voids had binder content of 6% while 6% air voids had a binder content of 5.3%. As can be seen there is approximately 0.4% decrease in binder content for every 1% increase in air voids during this test (Cooley Jr. et al., 2002). Average film thickness was 6.16 microns for 4% air voids while it was 5.3 microns at 6% air voids.

Table 2-32: Air Void Percentage from Various Sources

	Maryland	Georgia	Indiana	Superpave	Screenings
% Air Voids	4	4 - 7	4	4	4 - 6

2.2.7 Voids in the Mineral Aggregate (VMA)

VMA is the portion of the volume in the compacted asphalt mixture that is not occupied by aggregate or absorbed binder (Zaniewski and Diaz, 2004). This means that it is the volume of unabsorbed binder plus air voids and is expressed as a percent of the total volume of the mix.

AASHTO has a minimum VMA requirement of 16% for 4.75mm mixes which is the same for Superpave criteria. Williams (2006) determined the critical VMA value from the relationship with dust content to be 16% which matches AASHTO and Superpave criteria. Zaniewski and Diaz (2004) suggest mixes designed 75 gyrations and above should have a maximum VMA of 18% to avoid excessive optimum binder. Zaniewski and Diaz (2004) states no maximum VMA criteria should be used for 50 gyration mixes. If air voids are at 4% on a low volume road, a range of 16 - 18% for VMA may be used because it is believed the higher values can be tolerated by low volume roads (Williams, 2006). Figure 2-9 shows how VMA changes rutting depth. VMA criteria from various sources are displayed in Table 2-33.

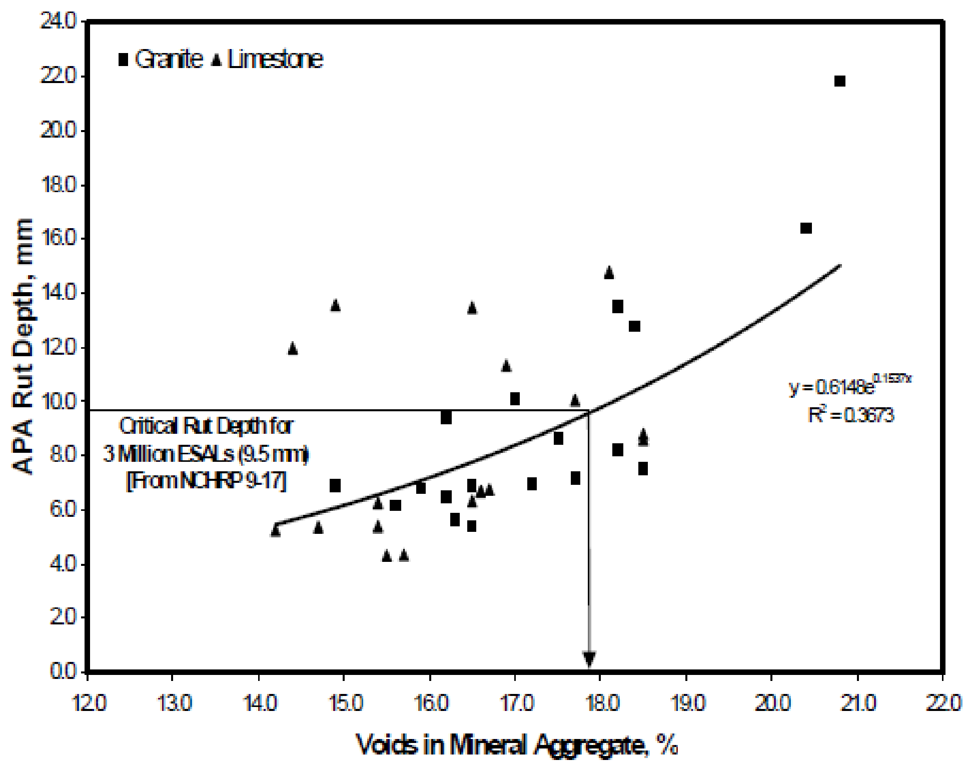


Figure 2-9: Relationship between APA Rut Depths and Voids in Mineral Aggregate (Cooley Jr. et al., 2002)

Table 2-33: VMA Criteria from Various Sources

	AASHTO	Arkansas	Indiana	North Carolina
% VMA min	16	15	17	20

2.2.8 Film Thickness

Proper film thickness is necessary to provide durability and limit permeability. Coating that are too thin can allow air and water intrusion and may not provide a cohesive mix. If VMA increases past the point of minimum, binder content is higher than optimum which leads to higher asphalt binder films. As film thickness increases the aggregate particles are forced apart and VMA increases. Film thickness is related to VMA and is the thickness of binder coating on individual aggregate particles (Williams, 2006).

Film thickness is difficult to measure but is calculated by dividing the effective volume of asphalt binder by estimated surface area of the aggregate particles. Zaniewski and Diaz (2004) suggested based on their study an increase in dust content of 4% decreased film thickness by 2.63 microns on average. An alternative to using VMA to control the effective asphalt content is to use asphalt film thickness as the controlling parameter. Zaniewski and Diaz (2004) recommended minimum film thickness be 8 microns.

2.2.9 Voids Filled with Asphalt (VFA)

VFA relates to VMA and air voids, and represents the percent of VMA that is occupied by the effective binder content (Zaniewski and Diaz, 2004). Some sources give a specification for VFA and some do not because if VMA and air voids are both restricted there is no need to design VFA. Zaniewski and Diaz (2004) suggest if VFA is used the range should be from 75% to 78% for 4.75mm mixes of 75 gyrations and above. 6% air void mixes can have below 70% VFA, so an air void content should be chosen and corresponding VFA range is used. A maximum VFA of 80% for 50 gyration mixes is also reasonable. Zaniewski and Diaz (2004) found that for 4.75mm NMAS mixes an increase in air voids by 1% had an average reduction of 6.2% of VFA and every 4% increase in dust content reduced VFA by 2.4% on average. Cooley Jr. et al. (2002) showed the relationship between VFA and rutting depths for the 4.75mm mix, as shown in Figure 2-10. The rut depths were measured using the APA test. It was found that in general higher VFA will lead to higher rut depth. The suggested VFA range is for mixes with a range of 4 - 6% air voids based on many research studies. VFA percentages from various sources are shown in Table 2-34.

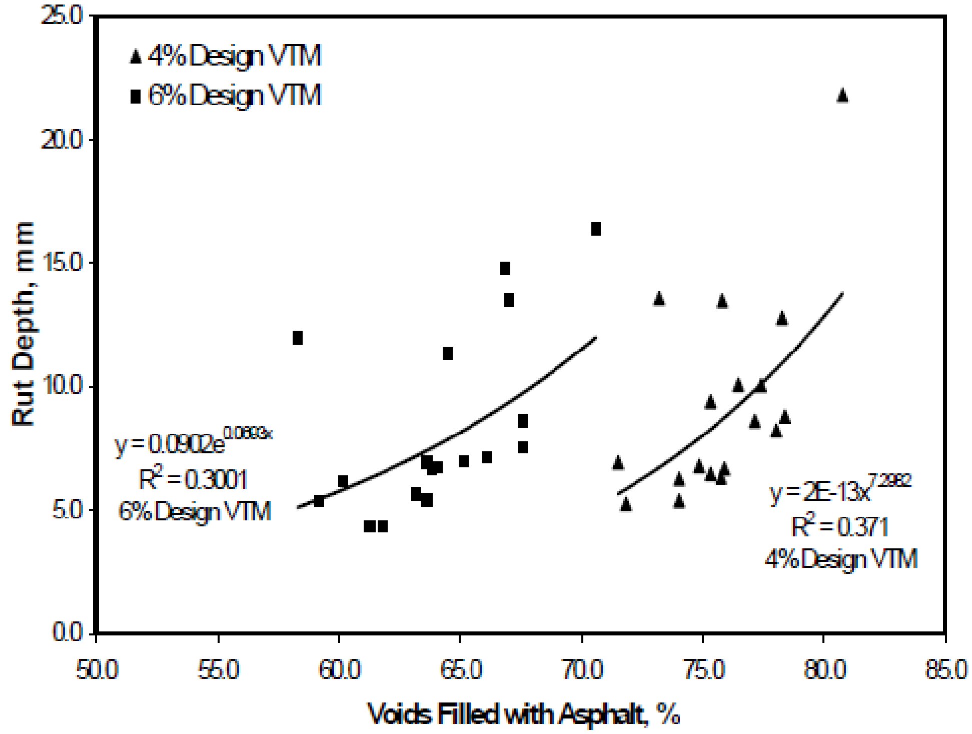


Figure 2-10: Relationship between APA Rut Depths and VFA (By Design Air Void Content) (Cooley Jr. et al., 2002)

Table 2-34: VFA Percentage from Various Sources

	AASHTO	Georgia	Maryland	(Williams, 2006)
% VFA	75 - 78	67 - 80	67 - 80	75 - 80

2.2.10 Volume of Effective Binder

V_{be} is the volume of effective binder content and is found by subtracting the design air voids from the VMA range. This value has been suggested to be used as a requirement rather than VMA and VFA by NCAT (2011) when using a range of design air voids. Figures 2-11 and 2-12 show a range of V_{be} versus rutting rate with varying fine aggregate angularity (FAA) and percent of natural sand (Raush et al. 2006). As can be seen high natural sand content makes the curve much steeper and therefore more susceptible to rutting. When V_{be} is greater than 14%, mixes with FAA less than 45 showed much higher rutting susceptibility than mixes with FAA greater than 45. Mixes designed with less than 13.5% V_{be} have better rutting resistance than those with greater than

13.5% V_{be}. NCAT (2011) suggests for medium/high volume roads a maximum value of 13.5% V_{be} is recommended while on low volume roads a maximum V_{be} of 15% is recommended. These recommendations can be seen in Table 2-35.

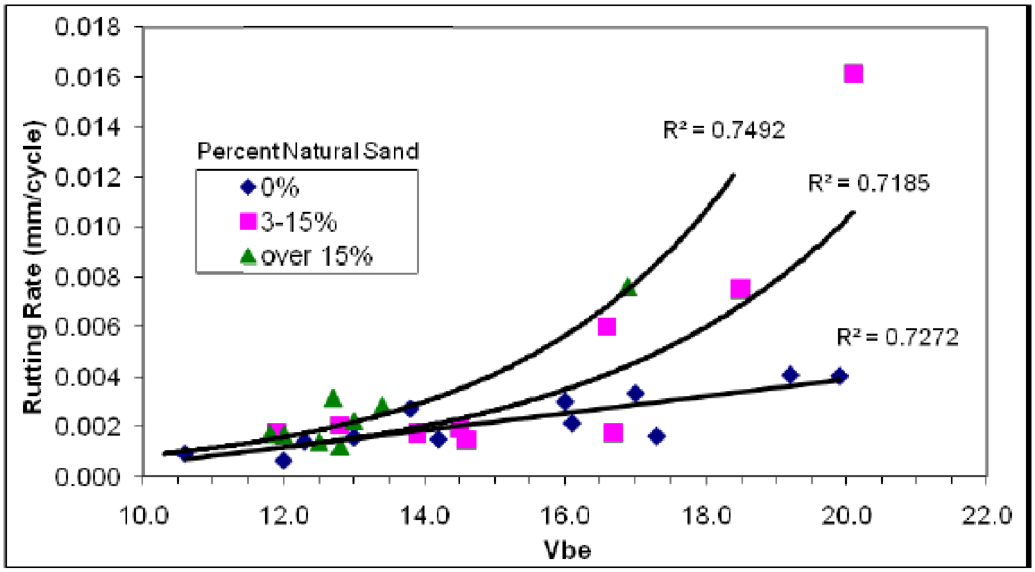


Figure 2-11: V_{be} Versus Rutting Rate for all Mixtures, Sorted by Percent Natural Sand (Raush, 2006)

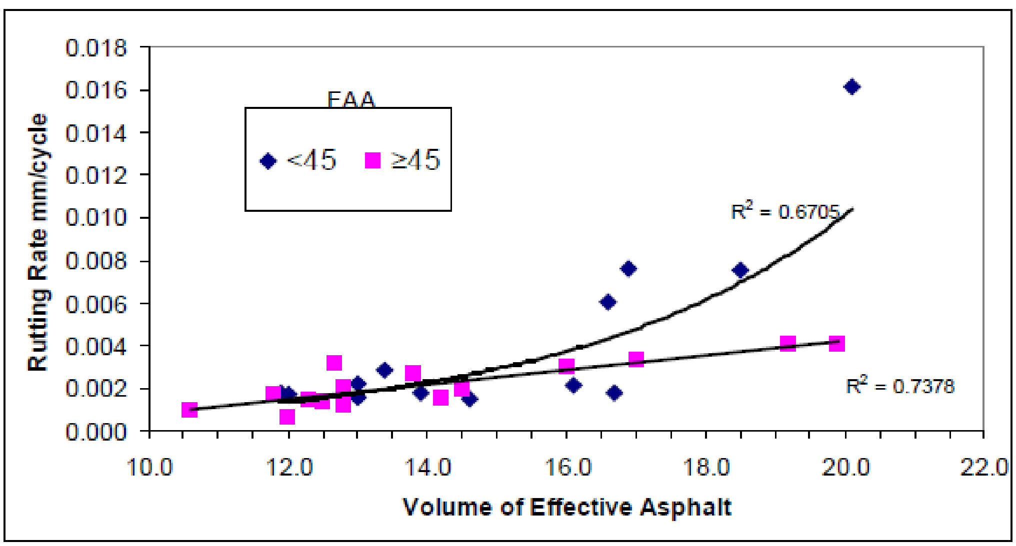


Figure 2-12: V_{be} versus Rutting Rate for All Mixtures, Sorted by FAA (Raush, 2006)

Table 2-35: Proposed Design Criteria for 4.75mm NMAS Superpave-designed mixtures (NCAT, 2011)

Design ESAL Range (Millions)	N_{des}	Minimum FAA	Minimum Sand Equivalent	Minimum V_{be}	Maximum V_{be}	$\%G_{mm}@N_{ini}$	Dust-to-Binder Ratio
<0.3	50	40	40	12.0	15.0	$\leq$91.5	1.0 to 2.0
0.3 to $\leq$3.0	75	45	40	11.5	13.5	$\leq$90.5	1.0 to 2.0
3.0 to $\leq$30	100	45	45	11.5	13.5	$\leq$89.0	1.0 to 2.0

2.2.11 Warm Mix Asphalt (WMA) and Reclaimed Asphalt Pavement (RAP) Usage

Warm mix asphalt (WMA) is produced at a temperature 30 to 100°F cooler than normal HMA (Newcomb, 2009). This temperature is reduced using techniques such as foaming, adding chemical additives, and adding organic additives. Warm mix asphalt can be more workable and compactable than regular HMA at lower temperatures. Warm mix can increase haul distances, allow paving in slightly cooler temperatures, achieve density at lower temperatures, extend the paving season, and give the ability to pave over crack seal while minimizing bumps (NAPA, 2009). Warm mix design also gives the ability to add more recycled material into the mix. The reduced production temperature for WMA decreases emissions and fuel consumption, making it a more environmentally friendly paving material compared with conventional HMA mix. Warm mix asphalt can be especially beneficial in the production and construction of thin-lift asphalt mixtures (NAPA, 2009). As NMAS decreases plant temperatures are generally higher and in this instance, warm mix can reduce plant temperatures while maintaining quality (NAPA, 2009). This in turn helps warm mix improve the already excellent environmental record of the asphalt industry. The potential cooling of the mat prior to attaining density is a problem with thin HMA applications. This makes WMA a very good match when using thin lifts because it gives an edge in extending the window for compaction (Kuennen, 2010). When the mix starts out cooler it takes longer for the material temperature to drop a comparable amount allowing the additional compaction time

(NAPA, 2009). Warm mix asphalt practice will become even more economical as it goes main stream. Prices continue to decline for the value-added material and a lot of suppliers are looking for ways to improve their product and lower costs. This makes it even more advantageous to look at the possibility of using warm mix for 4.75mm NMAS mixes (Kuennen, 2010).

Reclaimed asphalt pavement (RAP) and recycled asphalt shingles (RAS) can be used to replace a portion of new material in 4.75mm NMAS mixtures. According to Kuennen (2010) the RAP needs to be sized, crushed, and screened in a plant as a conventional virgin aggregate. The RAP included in the mix should be sand sized which will have higher asphalt content. This residual asphalt will help mitigate the higher asphalt content of the mix. RAP also helps prevent rutting, scuffing, and gives more stability to the mix (Newcomb, 2009). With the application of WMA, RAP percentage may be able to be increased to 50%. Mogawer et al. (2008) found high percentages of RAP (up to 50%) can be successfully used in thin lift applications and still meet gradation specifications and volumetric properties. A 30% RAP mix was tested in the field showing no problems with lay down, compaction, or workability. Two years after application this pavement had no signs of distress. Several 4.75mm mixes with different percentages of RAP material were designed and tested (designs are shown in Table 2-36). With RAP's contents of both aggregate and aged binder it can decrease the amount of virgin binder needed and could therefore lower costs. Workability is shown to decrease with the addition of higher RAP contents. With a higher percentage of RAP, increased WMA additive may be needed. It is thought that using softer PG binder and incorporating WMA technology will alleviate stiffness and workability issues for mixes containing high percentages of RAP.

Table 2-36: Superpave 4.75mm Mixtures JMF and Volumetric Properties (Mogawer et al., 2008)

	Percent of Aggregate Stockpile for JMF				
	0% RAP	15% RAP	30% RAP	50% RAP	Specification
9.5mm Aggregate	4.8%	7.5%	7.0%	7.0%	-
Manufactured Sand	88.4%	64.0%	57.5%	41.5%	-
Natural Sand	2.8%	11.5%	4.0%	1.5%	-
RAP	0%	15.0%	30.0%	50.0%	-
Fines	4.0%	2.0%	1.5%	0%	-
	JMF Percent Passing				
Sieve Size	0% RAP	15% RAP	30% RAP	50% RAP	Specification
12.5 mm	100.0	100.0	100.0	100.0	100.0
9.5 mm	99.2	98.9	99.0	99.0	95-100
4.75 mm	91.9	90.2	90.8	91.2	90-100
2.36 mm	66.3	67.8	67.5	68.5	-
1.18 mm	42.3	46.5	45.7	47.3	30-60
600 μm	27.8	31.7	31.5	33.2	-
300 μm	17.5	19.1	20.0	21.5	-
150 μm	10.3	10.7	12.0	13.3	-
75 μm	7.3	6.9	8.0	8.6	6-12
	Volumetric Properties PG64-28 w/1.5% Sasobit®				
	0% RAP	15% RAP	30% RAP	50% RAP	Specification
% Virgin Binder Added	7.0	5.8	4.3	2.3	-
% Extracted Binder	7.0	7.2	6.8	6.4	-
% Air Voids	4.6	4.0	4.0	4.1	4.0%
%VMA	19.5	19.0	17.7	16.5	16-18%
%VFA	76.3	78.7	77.4	75.0	65-78%
Dust-to-Binder Ratio	1.15	1.07	1.36	1.63	0.9-2.0
	Volumetric Properties PG52-33 w/1.5% Sasobit®				
	0% RAP	15% RAP	30% RAP	50% RAP	Specification
% Virgin Binder Added	6.8	5.5	4.0	2.2	-
% Extracted Binder	6.7	6.6	6.4	6.2	-
% Air Voids	4.0	3.7	3.2	4.5	4.0%
%VMA	18.8	16.5	16.0	16.8	16-18%
%VFA	78.7	77.8	80.2	73.3	65-78%
Dust-to-Binder Ratio	1.17	1.27	1.59	1.65	0.9-2.0
	Volumetric Properties PG52-33w/1.5% Sasobit ®+1.5% Latex				
	0% RAP	15% RAP	30% RAP	50%RAP	Specification
% Virgin Binder Added	-	-	3.7	-	-
% Extracted Binder	-	-	6.3	-	-
% Air Voids	-	-	4.4	-	4.0%
%VMA	-	-	17.5	-	16-18%
%VFA	-	-	74.9	-	65-78%
Dust-to-Binder Ratio	-	-	1.55	-	0.9-2.0

- Not Applicable, VMA = Voids in Mineral Aggregate, VFA = Voids Filled with Asphalt

2.2.12 Screening Material Usage

Since the implementation of Superpave mix design, coarse materials have been used in HMA mixes. These coarse graded mixtures were used because they were less susceptible to rutting. Other agencies have started to use stone-matrix asphalt (SMA) mixes which are dependent on stone to stone contact. This led to large stockpiles of screenings (manufactured fine aggregate) because screenings were used less frequently in Superpave HMA mixes. “The implementation of 4.75mm NMAS Superpave mix will reduce the accumulated screening stockpiles and hence, provide a use for materials that could become a “by-product” of the HMA industry" (Rahman et al., 2010). This can help mitigate environmental issues due to disposal or stockpiling problems.

The properties of these aggregates are critical to pavement performance, but are specific to each state or stockpile. Critical values of these properties are typically established by local agencies because they are so source specific. This means there is no national set standard for these properties which include toughness, soundness, and deleterious materials (Williams, 2006). 4.75mm NMAS mixes should use at least three aggregate stockpiles so the blend can be controlled well during plant production. Cooley Jr. et al. (2002) stated Maryland uses a 4.75mm NMAS mix that generally contains 65% manufactured screenings and 35% natural sand which has received excellent rutting and cracking resistance.

Based on the conclusions from Raush (2006), mixes only using screening stockpiles with the correct Superpave gradation should follow 4.75mm NMAS Superpave mix design. When the gradation does not meet the requirements for 4.75mm Superpave mixes, it should be designed using the criteria in Table 2-37.

Table 2-37: Criteria to Use if Superpave Gradation Not Met (Raush, 2006)

Property	Criteria
Design Air Void Content, %	4 to 6
Effective Volume of Binder, %	12 min.
Voids Filled with Asphalt, %	67-80

2.2.13 Summary

Mix design of 4.75mm NMAS is subjective to materials and previous experience in the area to determine which range of values to choose from. Aggregate can be natural or synthetic material and mostly depends on what is available in the area. FAA and natural sand play an important role in rutting control. Gradation is one of the most influential properties on the performance of a mix and should fall within the control points selected. Dust content is the material passing the 0.075mm sieve and has a large effect on VMA and rutting. PG binder selection should follow the guidelines explained earlier as well as experience. Effective binder of the mix is the total binder added minus the binder absorbed and is used to determine the dust to binder ratio. The D:B ratio helps ensure proper coating of the mineral filler of the mix.

Design air voids have a significant impact on the mix and when changed almost every category has different value ranges. Most reports had air voids between 4 - 6% and it was determined high volume roads should use 4 - 4.5% and low/medium volume roads can use 6% because there is less rutting potential due to reduced traffic. 6% air voids also helps reduce binder content which in turn helps reduce high costs of binder. Meeting a minimum VMA provides good mix durability and a maximum limit of VMA may be needed for mixes designed above 75 gyrations. If air voids and VMA are controlled, VFA is implied and not necessarily needed. Film thickness has been identified as an alternative to using VMA and VFA to control the mix. Also V_{be} could also be used as a controlling parameter instead of the previous parameters. Table 2-38 shows the suggested values from previous sections combined into the suggested design criteria for varying 4.75mm mixes.

WMA has the potential to be used in a 4.75mm NMAS mix. It can reduce plant temperatures which are already increased by the small NMAS therefore reducing costs. WMA can also increase time for compaction which is a definite benefit since thin lifts cool much quicker than thicker lifts. WMA gives the ability to add more RAP to the mix also potentially reducing costs. Screening materials can be used in 4.75mm NMAS mixes which are a by-product of traditional HMA mixes. There is potential to create stable mixes from one stockpile but blending 2-3 stockpiles is much more common and usually generates better results.

Table 2-38: Proposed design criteria for 4.75mm NMAS Superpave-designed mixtures (NCAT, 2011)

Design ESALs (Millions)	N_{des}	Minimum FAA	Minimum Sand Equivalent	Minimum V_{be}	Maximum V_{be}	D:B ratio
<0.3	50	40	40	12.0	15.0	1.0 - 2.0
0.3 - 3.0	75	45	40	11.5	13.5	1.0 - 2.0
>3.0	100	45	45	11.5	13.5	1.0 - 2.0

Gradation	
Sieve Size (mm)	% Passing
12.5	100
9.5	95 - 100
4.75	90 - 100
1.18	30 - 55
0.075	6 - 13

2.3 TEST SECTIONS

There have been several field tests on the adequacy of using 4.75mm NMAS mix thin overlay as a preventive maintenance strategy on pavement. National Center for Asphalt Technology (NCAT) was one of the first to test this mix type and has done the majority of the testing on this subject. Several DOTs have also conducted tests to refine and test the original mix design specifications for various reasons. The following summarizes several tests conducted by these agencies.

2.3.1 NCAT Test Track

Powell and Buchanan (2012) reported NCAT conducted a 4.75mm NMAS thin overlay study at the experimental NCAT test track facility near Auburn University. This facility, a 2.8 kilometer pavement test track, is used by many governmental agencies to research ways to extend flexible pavement life, and is managed by NCAT. A .8 inch thick 4.75mm NMAS test section of 60 meters was built into this test track in 2003. A 1.0 inch thick 9.5mm and 1.7 inch thick 12.5mm NMAS overlay section were also built so the 4.75mm NMAS mix could be compared to these larger aggregate mixes. The purpose of this experiment was to see if mixes made from screening stockpiles could compare favorably to conventional 12.5mm and 9.5mm mixes. Every mix was placed on a perpetual foundation to ensure performance differences were because of the quality of the experimental surfaces. Between 2003 and 2008 twenty million ESALs were applied to all 3 pavements and another ten million ESALs were applied to the 4.75mm and 9.5mm surfaces as part of a 2009 test track study.

All three pavements (4.75, 9.5, and 12.5mm) had comparable rutting with resulting ruts not exceeding 6mm. After 30 million ESALs had been applied to the 4.75mm and 9.5mm NMAS sections, the rutting averaged 6mm and 4mm respectively. After 20 million ESALs the 12.5mm NMAS mix had ruts of approximately 4mm. Laboratory tests were also conducted via the Asphalt Pavement Analyzer (APA). Samples were heated to 64^{o}C and loaded with a hose pressure of 689 kPa under a 534 N load. After 8000 cycles the ruts in the 4.75mm, 9.5mm, and 12.5mm NMAS mixes averaged 2.2, 3.4, and 3.4mm respectively. All of the results fell under the 4.5-5mm threshold for pavements expected to result in poor rutting performance.

At the end of 20 million ESALs being applied, there was no cracking present on any of the three sections. At 30 million ESALs there were slight longitudinal top-down cracking in the 9.5mm NMAS surface but the 4.75mm NMAS surface still showed no signs of cracking. The roughness of the three at 20 million ESALs was very comparable. Since all were built on perpetual foundations there was not an expected difference in roughness performance. For macro texture performance, the 12.5mm was most durable while 9.5mm was the least durable mixes (e.g. more raveling). While the 4.75mm mix was not better than the 12.5mm mix, it was more stable than the 9.5mm surface. “This is seen as a positive finding because the 9.5mm NMAS mix is commonly used as a surface mix on high volume roads in the southeastern United States.” (Powell and Buchanan, 2012). In other words, the macrotexture performance of the 4.75mm mix is promising because it showed better macrotexture than the 9.5mm mix.

2.3.2 NCAT Pooled-Fund Study

West et al. (2011) documented NCAT mix design and testing of four test sections. Two of the four SHRP climate zones were studied which included wet-freeze and wet-no freeze. The field validations were to examine the following issues: in-place densities after compaction, appropriate spread rates and lift thicknesses, workability of the mixture during construction, variability in mixture volumetric and aggregate properties during production and construction, friction of in-place mixtures, stability of the mixture during compaction, and permeability of in-place mixtures. The production of the mixes was performed independently of NCAT by a contractor but NCAT was present to take samples of the materials. The following summarizes the four test sites procedures and results.

2.3.2.1 Alabama (2006)

This project was constructed in Alabama near Auburn and the climate zone of this project was wet-no freeze. The traffic level was 4700 AADT and was estimated to have 0.3 to 3.0 million ESALs. A 4.75mm NMAS mix was placed as a 0.75 inch lift and the mix was produced at a drum plant and paved with conventional paving equipment. Table 2-39 shows the mix design used for this project. No initial friction tests took place during placement. NCAT collected loose materials to take back for testing and results are summarized in Table 2-40.

Table 2-39: Alabama Validation Project 4.75mm Mix Design Summary (West et al., 2011)

Mix Type	Proposed AASHTO Criteria	Alabama 424 (surface mixture)
Mix Size	4.75 mm NMAS	3/8-inch maximum aggregate size (4.75 mm NMAS)
Binder Type		PG 67-22
Binder Content		6.8%, P_{be} 6.53%
Aggregate Blend		19% granite (#89 VMC Columbus, GA) 30% granite (M10 VMC Columbus, GA) 30% limestone (#8910 OCM Opelika) 20% man-sand (MM Pinkston Shorter) 1% baghouse fines
Target Gradation	30-55% passing 1.18 mm Sieve 6-13% passing 0.075 mm Sieve	47% passing 1.18 mm Sieve 6.0% passing 0.075 mm Sieve
Aggregate Properties	FAA 45(min) SE 40(min) Nat. Sand 15(max) if FAA<45	FAA = 46 Not reported N/A
Air Voids	4.0-6.0% (N_{des}=75 gyrations) 90.5 max (%G_{mm} @ N_{ini})	V_a=3.3% at N_{des} = 65 gyrations N_{ini} = 89% of G_{mm} at 7 gyrations
Volumetric Properties	V_{be} 12.0 to 15.0 VMA 16.0 min (note 1) VFA 65-78 (note 1) D:B ratio 1.0-2.0	V_{be} = 14.7 VMA = 18.0 VFA 81.8 D:B ratio = 0.92
Moisture Susceptibility		TSR = 0.85 with no anti-strip treatment

Note-1: current AASHTO criteria

Table 2-40: NCAT Field Sampling and Testing for the Alabama Project (West et al., 2011)

Test [no. of samples / no. of replicates]	Mix Design Target	Production QC
Mixture V_a - Lab (%G_{mm}@N_{des})	3.3%	2.2 - 3.4%
G_{mm}	2.467	2.444 - 2.482
Binder Content - by Ignition Method (Pb)	6.8%	6.7 - 7.1%
Gradation - washed from ignition samples	47% pass 1.18 6.0% pass 0.075	50.7 - 55.3 8.3 - 11.0
V_{be} VMA VFA D:B ratio	14.7 18.0 81.8 0.92	14.4 - 16.2 17.8 - 18.7 80.7 - 88.1 1.24 - 1.83
Moisture Susceptibility (TSR)	0.85	0.80
Rut Testing - by MVT		13.0 mm
Lab Permeability from Field Cores (cm/sec)		90×10^{-5}
In-place V_a - From Cores (note-1)		11.7 avg. 9.5 - 13.2
Surface Friction - by DFT and CTM		Note-2

Note-1: Cores were taken at 200-ft intervals from Station 157+50 to 175+50
Note-2: DFT and CTM equipment were not available at the time of construction

2.3.2.2 Missouri (2007)

This project was conducted in Missouri near Kennett which is a wet-freeze climate zone. The traffic level is 2500 AADT with less than 5% trucks designed at 0.3 million ESALs. It was a 4.75mm NMAS mix placed at a 0.75 inch thickness. The mix was produced at a drum plant and paved with conventional paving equipment. The mix design for this project is summarized in Table 2-41 and field test and laboratory tests are summarized in Table 2-42.

Table 2-41: Missouri Validation Project 4.75mm Mix Design Summary (West et al., 2011)

Mix Type	Proposed AASHTO Criteria	Missouri BP-3 Plant Mix Bituminous
Mix Size	4.75 mm NMAS	4.75 mm NMAS
Binder Type		PG 64-22
Binder Content		6.4%, P_{be}=5.4%
Aggregate Blend		55% dolomite (LD Williamsville #1) 25% man-sand (MSGV BS&G Dexter) 20% nat-sand (NS1 BS&G Dexter, MO)
Target Gradation	30-55% passing 1.18 mm Sieve 6-13% passing 0.075 mm Sieve	48% passing 1.18 mm Sieve 7.6% passing 0.075 mm Sieve
Aggregate Properties	FAA 40(min) SE 40(min) Nat. Sand 15(max) if FAA<45	FAA = 45 Not reported N/A
Air Voids	4.0-6.0% (N_{des}=50 gyrations) 91.5 max (%G_{mm} @ N_{ini})	V_a=4.0% at N_{des} = 50 gyrations Not Reported
Volumetric Properties	V_{be} 12.0 to 15.0 VMA 16.0 min (note 1) VFA 70-80 (note 1) D:B ratio 1.0-2.0	V_{be} = 12.2 VMA = 16.3 VFA 75.2 D:B ratio = 1.4
Moisture Susceptibility		Not tested, generally not required for mixtures on low volume roads

Note-1: current AASHTO criteria

Table 2-42: NCAT Field Sampling and Testing for the Missouri Validation Project (West et al., 2011)

Test [no. of samples / no. of replicates]	Mix Design Target	Production QC
Mixture V_a - Lab (%G_{mm}@N_{des})	4.0%	3.6 - 4.9%
G_{mm}	2.456	2.453 - 2.460
Binder Content - by Ignition Method (Pb)	6.4%	6.8 - 7.4%
Gradation - washed from ignition samples	48% pass 1.18 7.6% pass 0.075	49 - 58 11.8 - 12.3
V_{be} VMA VFA D:B ratio	12.2 16.3 75.2 1.4	12.5 - 13.3 16.6 - 17.7 74.3 - 76.5 2.1 - 2.2
Moisture Susceptibility (TSR)	Not tested	0.66 - 0.74
Rut Testing - by APA (note-2)		6.7 mm
Lab Permeability from Field Cores (cm/sec)		40×10^{-5}
In-place V_a - From Cores		10.1 avg. 9.2 - 11.9
Surface Friction - by DFT and CTM		MPD 0.17 - 0.22 mm

Note-1: The DFT was not available for this project

2.3.2.3 Tennessee (2007)

This project was constructed in Robertson County, Tennessee and the climate in this location is wet-no freeze. The traffic level was 1620 AADT with 18% trucks and designed at 0.3 to 3 million ESALs. The primary distress in the existing pavement was transverse cracking with 10 to 40 foot spacing. The overlay was placed with a 4.75mm NMAS mix at 0.75 inches thick. Two different mixes were used for overlaying; a mix comprised of virgin mix only in the eastbound lanes and another mix with 15% RAP in the westbound lanes. The mix was produced at a batch plant, and 5 passes of a steel roller was determined to be an appropriate rolling technique for proper compaction. The mix design for this project is summarized in Table 2-43 and 2-44 while field test and laboratory tests are summarized in Table 2-45 and 2-46.

Table 2-43: Tennessee Validation Project 4.75mm Virgin Mix Design Summary (West et al., 2011)

Mix Type	Proposed AASHTO Criteria	ACS-HM (surface mixture)
Mix Size	4.75 mm NMAS	4.75 mm NMAS
Binder Type		PG 64-22
Binder Content		6.8%
Aggregate Blend	Nat. Sand=15% max. if FAA<45	75% screenings (#10-hard Aggr USA) 10% screenings(#10-soft Aggr USA) 15% natural-sand (Ingram Mtls)
Target Gradation	30-55% passing 1.18 mm Sieve 6-13% passing 0.075 mm Sieve	58% passing 1.18 mm Sieve 12.1% passing 0.075 mm Sieve
Aggregate Properties	FAA 45(min) SE 40(min)	Not Reported
Air Voids	4.0-6.0% (N_{des}=75 gyrations) 90.5 max (%G_{mm} @ N_{ini})	V_a=4.0% at 75-blow Marshall
Volumetric Properties	V_{be} 12.0 to 15.0% VMA 16.0 min (note 1) VFA 65-78 (note 1) D:B ratio 1.0-2.0	V_{be} = 15.1 VMA = 19.1 VFA 79.0 D:B ratio = 1.8
Moisture Susceptibility		Not tested, not required based on asphalt binder content

Note 1: current AASHTO criteria

Table 2-44: NCAT Field Sampling and Testing for the Tennessee Validation Project (Virgin Mix) (West et al., 2011)

Test [no. of samples / no. of replicates]	Mix Design Target	Production QC (note-1)
Mixture V_a - Lab ($\%G_{mm}@N_{des}$)	4.0%	4.6 - 5.9%
G_{mm}	2.389	2.398 - 2.407
Binder Content - by Ignition Method (Pb)	6.8%	7.5 - 7.7%
Gradation - washed from ignition samples	58% pass 1.18 12.1% pass 0.075	50 - 51 11.7 - 13.4
V_{be} VMA VFA D:B ratio	15.1 19.1 79.0 1.8	14.9 - 15.3 19.9 - 20.5 72.8 - 75.8 1.8 - 1.9
Moisture Susceptibility (TSR)	Not tested	0.68 - 0.75
Rut Testing - by APA (note-2)		4.5 mm
Lab Permeability from Field Cores (cm/sec)		160×10^{-5}
In-place V_a - From Cores (note-1)		11.9 avg. 7.5 - 14.2
Surface Friction - by DFT and CTM (note-3)		DFT_{20} 0.25 - 0.35 MPD 0.16 - 0.33 mm

Note-1: NCAT lab density results based on N_{des} at 125 gyrations to match 4% V_a
Note-2: Tested at design V_a and at 7% voids
Note-3: Three replicates for DFT and two replicates for CTM

Table 2-45: Tennessee Validation Project 4.75mm RAP Mix Design Summary (West et al., 2011)

Mix Type	Proposed AASHTO Criteria	ACS-HM (surface mixture with RAP)
Mix Size	4.75 mm NMAS	4.75 mm NMAS
Binder Type		PG 64-22
Binder Content		6.8%
Aggregate Blend	Nat. Sand=15% max. if FAA<45	60% screenings (#10-hard Aggr USA) 10% screenings(#10-soft Aggr USA) 15% natural-sand (Ingram Mtls) 15% RAP (pass 5/16 Lojac)
Target Gradation	30-55% passing 1.18 mm Sieve 6-13% passing 0.075 mm Sieve	56% passing 1.18 mm Sieve 12.1% passing 0.075 mm Sieve
Aggregate Properties	FAA 45(min) SE 40(min)	Not Reported
Air Voids	4.0-6.0% (N_{des}=75 gyrations) 90.5 max (%G_{mm} @ N_{ini})	V_a=4.0% at 75-blow Marshall
Volumetric Properties	V_{be} 12.0 to 15.0% VMA 16.0 min (note 1) VFA 65-78 (note 1) D:B ratio 1.0-2.0	V_{be} = 15.0 VMA = 19.0 VFA 79.0 D:B ratio = 1.8
Moisture Susceptibility		Not tested, not required based on asphalt binder content

Note-1: current AASHTO criteria

Table 2-46: NCAT Field Sampling and Testing for the Tennessee Validation Project (15% RAP Mix) (West et al., 2011)

Test [no. of samples / no. of replicates]	Mix Design Target 15% RAP	Production QC (note-1)
Mixture V_a - Lab (%G_{mm}@N_{des})	4.0%	3.5 - 4.5%
G_{mm}	2.380	2.393 - 2.411
Binder Content - by Ignition Method (Pb)	6.8%	7.2 - 7.3%
Gradation - washed from ignition samples	56% pass 1.18 12.1% pass 0.075	52 - 54 13.2 - 14.1
V_{be} VMA VFA D:B ratio	15.0 19.0 79.0 1.8	14.3 - 15.0 18.4 - 19.0 77.7 - 79.7 2.0 - 2.2
Moisture Susceptibility (TSR)	Not tested	0.67 - 0.79
Rut Testing - by APA (note-2)		3.3 mm
Lab Permeability from Field Cores (cm/sec)		140×10^{-5}
In-place V_a - From Cores (note-4)		11.7 avg. 10.7 - 12.7
Surface Friction - by DFT and CTM (note-3)		DFT_{20} 0.28 - 0.33 MPD 0.19 - 0.33 mm

Note-1: NCAT lab density results based on N_{des} at 125 gyrations to match 4% V_a
Note-2: Tested at design V_a and at 7% voids
Note-3: Three replicates for DFT and two replicates for CTM
Note-4: One replicate measured Va=20.1 and was not included in the analysis

2.3.2.4 Minnesota (2008)

This project was constructed on I-94 in Minnesota in the wet-freeze climate zone. The typical ESALs were 600,000 annually so the design ESALs were from 3 to 30 million. A single 2 in lift was placed on top of a joint-doweled PCC with 15 feet spacing. The mix was produced at a drum plant and paved under tight experimental quality control. The mix design for this project is summarized in Table 2-47 and field and laboratory tests are summarized in Table 2-48.

Table 2-47: Minnesota Validation Project 4.75mm Mix Design Summary (West et al., 2011)

Mix Type	Proposed AASHTO Criteria	MnDOT SPWEB440F Special
Mix Size	4.75 mm NMAS	4.75 mm NMAS
Binder Type		PG 64-34 (polymer modified)
Binder Content		7.4%, Pbe=6.9
Aggregate Blend		55% Taconite tailings (Mintac) 10% Taconite tailings (Ispat) 35% Man-sand (Loken)
Target Gradation	30-55% passing 1.18 mm Sieve 6-13% passing 0.075 mm Sieve	51% passing 1.18 mm Sieve 7.7% passing 0.075 mm Sieve
Aggregate Properties	FAA 45(min) SE 40(min)	FAA = 47 SE = 83 N/A
Air Voids	4.0-6.0% (N_{des}=75 gyrations) 90.5 max (%G_{mm} @ N_{ini})	V_a=3.9% at Ndes=75 gyrations Not reported
Volumetric Properties	V_{be} 12.0 to 15.0% VMA 16.0 min (note 1) VFA 65-78 (note 1) D:B ratio 1.0-2.0	V_{be} = 16.4 VMA = 20.3 VFA 80.8 D:B ratio = 1.1
Moisture Susceptibility		TSR=0.82 @ Va = 9.0%

Note-1: current AASHTO criteria

Table 2-48: NCAT Field Sampling and Testing for the Minnesota Validation Project (West et al., 2011)

Test [no. of samples / no. of replicates]	Mix Design Target	Production QC
Mixture V_a - Lab (%G_{mm}@N_{des})	3.9%	2.9 - 3.9%
G_{mm}	2.551	2.532 - 2.546
Binder Content - by Ignition Method (Pb)	7.4%	8.8 - 9.1%
Gradation - washed from ignition samples	51% pass 1.18 7.7% pass 0.075	54 - 60 8.5 - 9.9
V_{be} VMA VFA D:B ratio	16.4 20.3 80.8 1.1	17.9 - 18.5 21.0 - 22.1 82.7 - 85.4 1.1 - 1.3
Moisture Susceptibility (TSR)	0.82 (9%)	0.68 - 0.82
Rut Testing - by APA (note-1)		5.3 mm
Lab Permeability from Field Cores (cm/sec)		5 x 10^{-5}
In-place V_a - From Cores		6.6 avg. 4.9 - 8.0
Surface Friction - by DFT and CTM (note-2)		DFT_{20} 0.34 - 0.49 MPD 0.13 - 0.18 mm

Note-1: Two replicates at design Va and 2 replicates at 7% Va
Note-2: Five tests randomly spaced in each lane

2.3.2.5 Summary

Table 2-49: Summary of Mix Designs for Validation Projects (West et al., 2011)

Mix Design Property	Alabama	Missouri	Tennessee	Minnesota
Mix Level	65 gyrations	50 gyrations	75-blow	75 gyrations
Design Traffic (estimate)	1M ESAL	0.3M ESAL	1M ESAL	12M ESAL
Binder Content (% of mix)	6.8	6.4	6.8	7.4
Effective Binder Content	6.5	5.4	6.8	6.9
1.18 mm Target (% passing)	47	48	58	51
0.075 mm Target (% passing)	6.0	7.6	12.1	7.7
V_a (%)	3.3	4.0	4.0	3.9
VMA (%)	18.0	16.3	18.0	20.3
V_{be} (%)	14.7	12.2	15.1	16.4
VFA (%)	81.8	75.2	79.0	80.8
D:B ratio	0.9	1.4	1.8	1.1
FT (microns)	9.3	7.1	6.5	8.5
TSR	0.85	Not measured	Not measured	0.82

Two of the four field projects did not use AASHTO mix design standards for compaction. Alabama used 65 gyrations for N_{des} but had only 3.3% V_a. Tennessee used the Marshall 75-blow mix design but the NCAT laboratory had to apply 125 gyrations to achieve 4% V_a. It should be noted that the overlay thickness of the Minnesota project is much higher than the other three projects, which may contribute to the better compactability and in-place density. All four of these mixtures were fine graded mixtures but the Alabama mixture did not comply with NMAS criteria. All four mixtures were designed and produced near the upper control point on the 1.18mm sieve. The very fine mixtures are a common characteristic of 4.75mm mixes. During production gradations generally were even finer than designed. Gradations of the mixes can be seen in Figure 2-13.

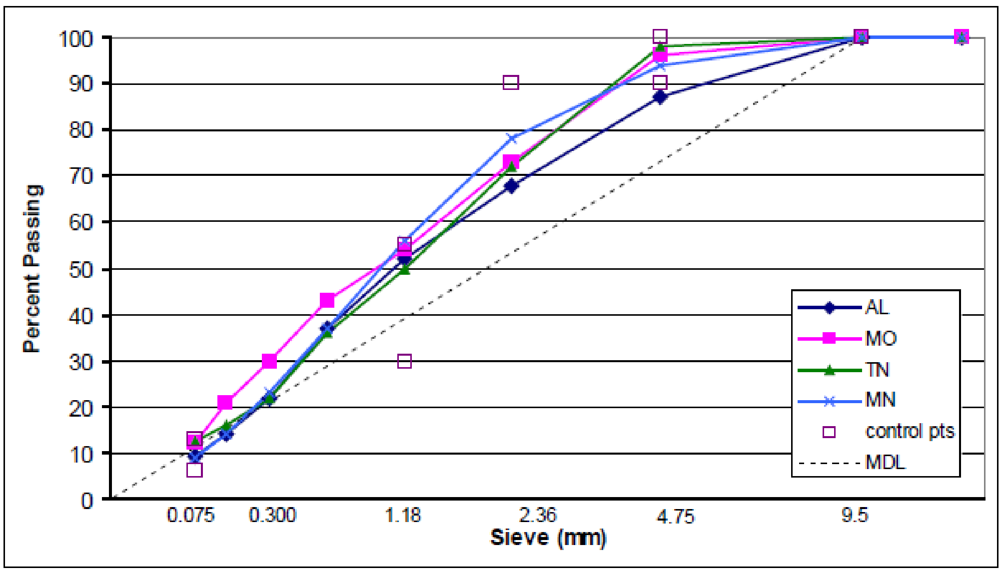


Figure 2-13: Average Plant-Production Gradation for Field Validation Projects (West et al., 2011)

In every mix but the Alabama mix, the asphalt contents had to be increased substantially over the mix design targets even with the high dust contents. The laboratory phase recommended using V_{be} in place of VMA and VFA. The Missouri mix was designed and produced within the V_{be} range while the three others were produced at or above the maximum recommended V_{be}. Despite the high V_{be} results the rutting results of these mixes were good. VMA and VFA are not recommended for continued use in 4.75mm mixtures. Table 2-50 gives a summary of field properties of plant-produced mixes. West et al. (2011) also summarized the mix design criteria for the four field projects, as shown in Table 2-51.

Since 4.75mm mixes are typically placed in thin lifts, field in-place densities or V_a are not usually measured. A set rolling pattern is typically adopted by the contractor instead of relying on the in-place densities. The expected in-place V_a is 6 to 8% for most HMA but several V_a values for 4.75mm mixes were as high as 13 and 14%. Permeability tests showed even with a high V_a the pavement is still essentially impermeable.

Table 2-50: Summary of Plant-Produced Mixes for Validation Projects (West et al., 2011)

Field Property (average, range)	Alabama	Missouri	Tennessee (virgin mix)	Minnesota
Mix Level	65 gyrations	50 gyrations	125 gyrations (note1)	75 gyrations
Binder Content (% of mix)	6.9, 6.7-7.1	7.2, 6.8-7.4	7.6, 7.5-7.7	9.0, 8.8-9.1
1.18 mm Sieve (% passing)	52, 51-55	54, 48-58	50, 50-51	56, 54-60
0.075 mm Sieve (% passing)	9.2, 8.3-11	12.1, 11.4-12.5	12.7, 11.7-13.4	9.0, 8.7-9.9
V_a (lab compacted)	2.8, 1.9-3.7	4.2, 4.1-4.5	5.1, 4.6-5.9	3.6, 2.9-3.9
VMA (%)	18.3, 17.8-18.7	17.2, 16.6-17.7	20.3, 19.9-20.5	21.6, 21.0-22.1
V_{be} (%)	15.6, 14.4-16.2	13.0, 12.5-13.3	15.1, 14.9-15.3	18.2, 17.9-18.5
VFA (%)	85.1, 80.6-88.1	75.3, 74.3-76.5	74.5, 72.8-75.8	84.2, 82.7-85.4
In-place V_a (%)	11.7, 9.5-13.2	10.1, 9.2-11.9	11.9, 7.5-14.2	6.6, 4.9-8.0
D:B ratio	1.4, 1.2-1.8	2.1, 2.0-2.2	1.9, 1.8-1.9	1.2, 1.1-1.3
FT (microns)	7.5	5.0	6.7	8.6
TSR	0.80	0.70, 0.66-0.74	0.71, 0.68-0.75	0.76, 0.68-0.82
APA (mm)	13	6.7 (Va=4%)	4.0 (Va=7%)	6.1 (Va=7%)
Permeability (cm/sec)	90×10^{-5}	40×10^{-5}	140×10^{-5}	$<10\times10^{-5}$
DFT_{20}	Not measured	Not measured	0.25-0.35	0.34-0.49
CTM (mm)	Not measured	0.17-0.22	0.16-0.33	0.13-0.18

Note-1 Field mix compacted to 125 gyrations required to match 75-blow Marshall mix design at 4.0% V_a.

Table 2-51: Mix Design Criteria Validation Summary (West et al., 2011)

Mix criteria	Traffic Level (ESALs)	Current AASHTO	Prelim. Recomm. Criteria	Alabama	Missouri	Tennessee	Minnesota
N_{des} Gyrations	<0.3M <3.0M >3.0M	50 75 75	No change	65 OK	50 OK	125 (note 1) <3.0M HIGH 75-blow non-standard	75 OK
FAA (note-3)	<0.3M <3.0M >3.0M		40 45 45	Design OK	Design OK	Design not reported	Design OK
Natural Sand (if FAA<45)	<0.3M <3.0M >3.0M		15 max 15 max	Design n/a	Design n/a	Design OK	Design n/a
SE (note-3)	<0.3M <3.0M >3.0M	40 40 45	No change	Design Not reported	Design Not reported	Design Not reported	Design OK
Gradation Control 1.18 mm	<0.3M <3.0M >3.0M	30-60	30–55	Design OK Plant OK	Design OK Plant OK	Design HIGH Plant OK	Design OK Plant HIGH
Gradation Control 0.075 mm		6–12	6–13	Design OK Plant OK (note 2)	Design OK Plant OK (note 2)	Design OK Plant OK (note 2)	Design OK Plant OK (note 2)
V_a (%G_{mm}@N_{des})		4.0	4.0–6.0	Design LO Plant LOW	Design OK Plant OK	Design OK Plant OK	Design OK Plant LOW
VMA	<0.3M <3.0M >3.0M	16 min 16 min 16 min		Design OK Plant HIGH	Design OK Plant OK	Design OK Plant HIGH	Design HI Plant HIGH
V_{be}	<0.3M <3.0M >3.0M		12.0-15.0 12.0–15.0 11.5–13.5	Design OK Plant HIGH	Design OK Plant OK	Design OK Plant OK	Design HI Plant HIGH
%G_{mm} @ N_{ini} (note-3)	<0.3M <3.0M >3.0M	91.5 max 90.5 max 89.0 max	No change	Design OK	Design Not reported	Design n/a	Design Not reported
D:B Ratio	<0.3M <3.0M	0.9–2.0 0.9–2.0	1.0–2.0 1.0–2.0	Design LO Plant OK	Design OK Plant HIGH	Design OK Plant OK	Design OK Plant HIGH
	>3.0M	0.9–2.0	1.5–2.0				

Note-1 Applied 125 gyrations to match 75-blow Marshall mix design volumetrics.
Note-2 All production mixture increased the mineral fines over the mix design.
Note-3 These mix design criteria were not examined as part of field validation.

2.3.3 Maryland and Georgia

Maryland has used thin HMA overlays as part of their preventive maintenance program. The gradation could fit either 9.5mm or 4.75mm NMAS in the Superpave system, and generally contains 65% manufactured screenings and 35% natural sand. Asphalt content ranged within the range of 5.0 - 8.0% with optimum air void content of 4% and a typical lift thickness ranged from 0.75 to 1.0 in. This mix showed excellent resistance to rutting and cracking (Williams, 2006). Georgia has also successfully used a thin lift type of mix for over 30 years for leveling and for paving low-volume roads (Cooley Jr. et al., 2002). These mixes are mostly made from screenings and a small quantity of 2.36mm sized stone. 60 - 65% of the aggregate passed the 2.36mm sieve and there was an 8 percent dust content. The mix was designed with an N_{des} of 50 gyrations with a target air void range of 4 - 7% and placed in thin lifts of 1.0 inches thick. Good performance was shown from this mix in the field. Both mix criteria are shown in Table 2-52 and both have performed well.

Table 2-52: Georgia/Maryland Design Specifications for 4.75mm Mixtures (Cooley Jr. et al., 2002)

Gradation Requirements		
	Georgia	Maryland
% Passing 12.5mm Sieve	100	-
% Passing 9.5mm Sieve	90 - 100	100
% Passing 4.75mm Sieve	75 - 95	80 - 100
% Passing 2.36mm Sieve	36 - 76	60 - 65
% Passing 0.30mm Sieve	20 - 50	-
% Passing 0.075mm Sieve	4 - 12	2 - 12
Design Requirements		
Asphalt Content (%)	6 - 7.5	5 - 8
Optimum Air Voids (%)	4 - 7	4
Voids Filled with Asphalt (VFA)	50 - 80	-

2.3.4 Indiana

INDOT placed an experimental 4.75mm HMA pavement on a high volume road in 2006. This test section exhibited poor surface friction so INDOT initiated research to study friction performance of 4.75mm overlays. Four test sections were placed to test ultrathin overlays using 4.75mm HMA mixes (Li et al., 2012). The first was placed on I-465 in 2006 in a 0.75 inch lift on a milled surface. The AADT was over 100,000 and truck traffic was approximately 20%. In 2009 two additional test sections on US-27 and SR-227 were constructed both with 0.75 inch thickness. US-27 has an AADT of 7,735 with about 10% truck traffic and SR-227 section had an AADT of 1964 with about 4% truck traffic. The last section was constructed in 2010 on SR-29 and had an AADT of 5552 with about 22% truck traffic. Material selection and mix designs of these test sections are summarized in Table 2-53 (Li et al., 2012). Locked wheel trailer tests and circular track meter (CTM) tests were conducted to evaluate friction characteristics of these surfaces.

Table 2-53: Summaries of Materials, Gradations and Mixes for Experimental Pavements (Li et al., 2012)

(a) Mix Materials

Test Section	Aggregate Components	Binder	RAP
I-465	BFS sand (39%), #24 dolomite sand (39%), #24 sand (20%), and Baghouse fines (2%)	PG 70-22	0%
US-27/SR-227	#12 dolomite (34%), #24 dolomite sand (34%), dolomite sand (30%), and Baghouse fines (2%)	PG 64-22	0%
SR-29	#12 dolomite (27%), #24 QA sand (48%), QA fine sand (23%), and Baghouse fines (2%)	PG 64-22	0%

(b) Aggregate Gradations

Test Section	% Passing through Sieve Sizes (sieve size unit: mm)							
	9.5	4.75	2.36	1.18	0.60	0.30	0.15	0.075
I-465	100	96.1	78.6	52.3	34.8	17.5	9.6	6.3
US-27/SR-227	100	90.1	67.2	47.8	32.7	22.6	14.7	7.8
SR-29	100	90.0	68.4	42.4	25.4	15.8	11.0	6.7

(c) Aggregate and Mix Volumetric Properties

Test Section	FAA	SE	AC		V_a (%)	VMA (%)	VFA (%)	DBR
			Total	Eff.				
I-465	45.9	92.1	7.8	5.9	4	17.7	76.9	1.0
US-27/SR-227	47.0	80.0	6.9	5.2	4	16.0	75.0	1.5
SR-29	47.1	88.3	6.8	5.7	4	17.3	76.9	1.2

Initially the I-465 pavement test section demonstrated poor friction performance in terms of friction number and texture depth. The US-27 and SR-227 had good surface friction when first constructed, but surface friction dramatically decreased after 12 months. Dramatic reduction in surface friction was observed after 6 months with the SR-29 roadway. After the I-465 pavement was constructed, the mixes on US-27, SR-227, and SR-29 had more coarse aggregate and replaced natural sand with dolomite sand. This changed the surface friction characteristics but this change was minimal. So freshly constructed lifts can give good friction numbers above 30 but after 12 months in service this friction number can drop 35 - 48%. The typical friction number is around 20 and can be lower with high traffic. The typical MPD ranged from 0.2 - 0.25mm after 12-18 months of service. Table 2-54 shows the summary of the results of these test sections over time (Li et al., 2012).

Table 2-54: Summary of Test Results on all 4 Test Sections (Li et al., 2012)

Test Section	Service Life	MPD (mm)	DFT_{20}	FN (smooth tire)	FN (rib tire)
I-465	36 months	0.24	0.43	16.7	44.4
US-27	18 months	0.24 - 0.30	0.25 - 0.27	19.7 - 28.6	-
SR-227	18 months	0.18 - 0.20	0.27 - 0.30	19.8 - 20.1	-
SR-29	6 months	0.21 - 0.22	0.23	21.6 - 27.6	-

2.3.5 Virginia Accelerated Testing

In Virginia, a 4.75mm mix was developed and tested as a thin lift on existing accelerated pavement test sections of traditional HMA (Li et al., 2012). The accelerated load facilities (ALFs) were used in this experiment to determine pavement performance under conditions where axle loading and pavement temperature can be controlled. Half of the loaded wheelpath was paved with 4.75mm NMAS overlay and half was an existing 12.5mm NMAS control mix. The previous pavement was milled to a depth of 28mm +/- 4mm and the 4.75mm mix was placed in 25mm lifts. Accelerated aging was used to test performance later in the pavements life as well. Radiant heaters were used to heat the pavement continuously at 74°C for 8 weeks before loading to simulate aging. Four combinations of 12.5mm or 4.75mm mix and aged or unaged were used for testing. These four combinations were placed together to compare full scale cracking and rutting performance of 4.75mm thin overlays. Mix design volumetrics and gradations for the 4.75mm mix are shown in Table 2-55 and Table 2-56. Laboratory tests were also run on loose mix gathered while paving.

Table 2-55: Gradation of the mix design; job mix formula and production (Li et al., 2012)

Sieves #	Bealton sand	#10	RAP	Nat. Sand	Bag House	Mix Design	Gradation Check
¾"(19mm)	100	100	100	100	100	100	100
½"(12.5mm)	100	100	99.8	100	100	100	99.7
3/8"(9.5mm)	100	100	95	100	100	99.1	97.0
#4 (4.75mm)	96	96	67	98	100	92.3	87.6
#8 (2.36mm)	62	66	50	86	100	68.7	60.1
#16(1.18mm)	38	45	39	66	100	45.7	43.1
#30(0.60mm)	26	33	29	36	100	31.9	31.0
#50(0.30mm)	17	24	21	12	100	21.6	21.4
#100(0.15mm)	10	18	14	5	98	14.7	15.1
#200(.075mm)	5.2	12.4	9.3	2.5	95	10.3	10.4
Blend %	26	44	20	10	1	-	-

Table 2-56: Volumetric properties of the mix design; job mix formula and production (Li et al., 2012)

Specification Criteria Ndesign = 50 gyrations			Job Mix Formula	Produced FHWA Gmm = 2.595 Contractor's Gmm = 2.584	
Volumetrics		Virginia DOT		FHWA extracted aggregate G_{SB} = 2.813	Contractor's aggregate G_{SB} = 2.789
VTM	Design	5%	4.4%	-	
	Production	3% - 6%	-	4.21% - 3.98%	
VFA	Design	70% - 75%	74%	-	
	Production	70% - 80%	-	75.1% - 76.2%	74.0% -75.2%
VMA		16.5% minimum	16.9%	16.9% - 16.7%	16.2 %– 16.0%
V_{be}		-	-	14.96%	14.86%
Dust to Binder based on effective asphalt		1 – 2	1.98 10.3÷5.2	1.99 10.4÷5.2	2.11 10.4÷4.93

For the field testing results were found for the mixes previously mentioned. The unaged 4.75mm NMAS inlay first cracked at 425,000 passes which is slightly lower than the control 12.5mm which cracked at 500,000 passes. But the aged 12.5mm section cracked at only 50,000 passes. This shows that an unaged 4.75mm NMAS mix has the ability to delay top-down cracking when placed on an aged pavement. Once aged, the 4.75mm mix has almost identical properties as the aged 12.5mm mix. This indicates the mix becomes brittle and provides little to no benefit at that point and should be replaced.

From these results several conclusions are revealed to the performance of 4.75mm NMAS mix. The NCAT recommendations for 4.75mm NMAS Superpave criteria seem to be sound and valid. There is not a large concern about this mix because the material properties do not induce compressive stresses that contribute to rutting. Full scale rutting and fatigue loading indicated no concerns with the 4.75mm mix. The relatively low stiffness and thin application are advantageous to the mix resulting in mostly compressive stresses and leading to better or equal rutting results as the larger 12.5mm NMAS layer. Thin 4.75mm NMAS overlays have the ability to significantly delay top-down cracking when used as a preservation treatment.

2.3.6 New Jersey Test Sections

New Jersey has two test section they paved with 4.75mm NMAS HPTO in 2008. HPTO is high performance thin overlay and is meant for use on high volume roadways. It typically uses polymer modified binder for better performance.

2.3.6.1 I-295

This project was a 5.3 mile test section of 4.75mm NMAS thin overlay paved in 2008. The roadway had a 35 million ESAL rating with the previous resurfacing 8 years old. The existing structures IRI was 90 in/mi and rut depth was .4 inches. For this project the surface was milled at a 1 inch depth and paved with 1 inch of HPTO. The gradation and other properties are shown in Table 2-57 and 2-58 respectively.

Table 2-57: Gradation and Percent Asphalt of I-295 Project

Sieve Size	Average Percent Passing
3/8” (9.5 mm)	100
#4 (4.75 mm)	85
# 8 (2.36 mm)	47
#16 (1.18 mm)	31
#30 (600 µm)	22
#50 (300 µm)	17
#100 (150 µm)	12
#200 (75 µm)	8.1
Percent Asphalt	8.07

Table 2-58: Properties of I-295 Project

Parameter	Average Result
Density @ Ndes	97.4 %
Dust/Binder	1.2
VMA	20.1 %
Rut Testing (APA)	3.5 mm

The projects contractor was noted as running the paver and rollers too fast. They also used a vibratory rollers and there was some “chatter” in the finished pavement. The finished pavement has an average air voids of 6.12% and the skid number was 32 which is less than the 43 recommended. After 3 years the skid resistance was up to 44, the IRI was 87 in/mi, and the rut depths were 0.2 inches.

2.3.6.2 I-287

This project was a 5.3 mile test section of 4.75mm NMAS thin overlay paved in 2008. The roadway had a 44 million ESAL rating with the previous resurfacing 8 years old. The existing structures IRI was 71 in/mi and rut depth was .2 inches. For this project the surface was milled at a 1 inch depth and paved with 1 inch of HPTO. The gradation and other properties are shown in Table 2-59 and 2-60 respectively.

Table 2-59: Gradation and Percent Asphalt of I-287 Project

Sieve Size	Average Percent Passing
3/8” (9.5 mm)	100
#4 (4.75 mm)	83
# 8 (2.36 mm)	42
#16 (1.18 mm)	30
#30 (600 μm)	21
#50 (300 μm)	14
#100 (150 μm)	9
#200 (75 μm)	5.5
Percent Asphalt	7.1

Table 2-60: Properties of I-295 Project

Parameter	Average Result
Density @ Ndes	96.9 %
Dust/Binder	1.0
VMA	19.0 %
Rut Testing (APA)	3.2 mm

The finished pavement had an average air voids of 5.8%. After 3 years the skid resistance was 51, the IRI was 87 in/mi, and the rut depths were 0.1 inches.

2.4 CONSTRUCTION

Construction of thin HMA overlays utilizes both conventional manufacturing facilities and construction equipment (Li et al., 2012). This gives every pavement contractor the ability to construct without specialized equipment or too much extra training. With a service life averaging 10 years, the 4.75mm thin overlay induces less traffic delays than other surface treatments that may have to be repeated more frequently. Also with 4.75mm overlays, there are no loose stones after initial construction and very little dust generated during construction (NAPA, 2009). Thin HMA overlays are easy to construct compared to other surface treatments and are a viable option for pavement preservation. Also warm mix asphalt has the potential to be a beneficial alternative to traditional HMA mixes for thin lifts. In general, the construction procedure of the 4.75mm thin overlay is similar to the conventional HMA overlay. Therefore, only the difference in the 4.75mm thin overlay construction is emphasized below.

2.4.1 Production

Stockpiles are an important part of the production of 4.75mm NMAS asphalt mixes. Small NMAS asphalt mixtures are taken from one or two stockpiles generally because of the small amount of coarse aggregate content. When multiple stockpiles are used it is usually to blend natural and manufactured sand. 4.75mm NMAS asphalt mixes can use traditional continuous drum type hot mix plants or a batch plant. With small NMAS mixtures the asphalt plant generally runs slower than with larger stone mixtures (NAPA, 2009). This is for many reasons which include the fine aggregate having more surface area to coat which requires more asphalt, generally fine aggregate has higher moisture content which requires longer drying time, and a thicker aggregate veil is used in the drying or production drum (NAPA, 2009). If RAP is added to the mixture it should never exceed the NMAS of the mixture and should act as sand size particles in a 4.75mm mixture (Kuennen, 2010). Asphalt rubber is not usually used in dense graded thin layers because it can be more difficult to compact and gives less resistance to reflective cracking (Caltrans, 2007).

2.4.2 Application

Milling can be done to help improve the initial conditions by leveling the surface and removing defects before a thin overlay is applied. This creates a rough surface which has a greater degree of shear resistance and is less likely to shove and de-bond (NAPA, 2009). If milling is not conducted, high severity cracks should be patched or sealed and all surface deformities should be filled as necessary (Caltrans, 2007).

A 4.75mm thin overlay can be paved with conventional paving equipment. While paving, the paver should be continuously moving to avoid an uneven surface where starts and stops occur.

Thin lifts require less HMA per foot of road length which can result in higher paver speeds. Delivery of material from the plant must then keep up with this demand. If it cannot the paver should start and stop as rapidly as possible to minimize mat roughness created from this action (NAPA, 2009). Also a material transfer device should be used to help with this problem and give more leeway in truck delays. Thin lifts are also applied at a higher temperature if WMA is not used to help offset how much more rapidly they cool. Table 2-61 shows recommended minimum application temperature for various stages of construction from (Caltrans, 2007). A 1.0 inch mat cools twice as fast as a 1.5 inch mat from 300° to 175° which leaves less time for proper compaction.

Table 2-61: Recommended Application Temperatures (Caltrans, 2007)

Binder Type	Minimum Air Temperature, °F	Minimum Surface Temperature, °F	Minimum Breakdown Rolling Temperature, °F	Minimum Finishing Temperature, °F
Conventional (Unmodified)	55	60	250	150
PG-PM	50	55	240	140

**These are minimum temperatures. It is recommended that spreading and compacting be performed at temperatures above these minimums, but not to exceed 325ºF.*

The thickness of thin overlays vary between different agencies. Morian (2011) stated that a thin overlay must have a thickness of 0.75 to 1.0 inch or less. Kansas DOT uses a 0.75 inch thick overlay in its preventive maintenance program. Ohio DOT and MAPA (Minnesota Asphalt Pavement Association) define a thin overlay thickness as less than 2.0 inches and ODOT (2001) states that this limit is required to be considered preventive maintenance. Michigan DOT considers lifts placed 1.0 to 1.25 inches thick to be thin overlays and 0.6 to 0.8 inch to be ultra-thin overlays (Huddleston, 2009). Johnson (2000) and Wade et al. (2001) report thickness of thin overlays as typically ranging from 0.75 to 1.5 inches and Montana DOT considers this range to be preventive maintenance. NAPA and INDOT state thin overlays are less than 1.5 inches thick. There is a wide range to what is considered to be a thin overlay. It can be seen that no value is above 2.0 inches meaning this could be considered the upper limit.

Compaction is used in thin overlays to increase the stability of the mat while sealing the voids of the material making it impermeable as possible (NAPA, 2009). Compaction can have a problem keeping up with the higher paver speeds. Thin lifts also cool quicker than traditional thick lifts which means there can be as little as 3 to 5 minutes to compact using HMA. This is a situation where warm mix asphalt could be beneficial to give extended compaction window (Kuennen, 2010, NAPA 2009). Proper thickness of the mat is also an important part of construction.

Lower in-place density is acceptable with this overlay because permeability is lower with smaller NMAS mixes. Vibrating rollers should not be used on thin lifts because they may cause roughness and tearing of the mat (NAPA, 2009). Pneumatic rollers may often result in HMA pickup especially where modified asphalt binders are used (Walubita and Scullion, 2008). Therefore mat density is best met using static steel wheel compactors with many specifications calling for this type only. It is recommended by Li et al. (2012) that large rollers (27,000 lb.) are necessary for construction rather than 15,000lb and 8,000lb rollers. The number of rollers used is specified by the placement rate and can be seen in Table 2-62.

Table 2-62: Number of Rollers Required based on Placement Rate (MDOT, 2005)

Average Laydown Rate, square yards per hour	Number of Rollers Required	
	Compaction Rollers	Finish Rollers
Less than 800	1	1*
801 – 2000	1	1
2001 – 5500	2	1
5501 – 7200	3	1

*The compaction roller may be used as the finish roller also.

One technique being developed in Japan is heating and scratching of cracked existing asphalt pavement to prevent decrease in pavement temperature before application. A road heater is used to heat and scratch the existing asphalt surface which is thought not to significantly increase the cost of construction while giving a great benefit (Kanai et al., 2012). The heating and scratching technique is confirmed to enhance bonding, cracking, and rutting by laboratory results. From observation results after 5 years of performance the overlay was found to be very durable (Kanai et al., 2012). This technique may be used to keep temperature higher longer to allow longer time for compaction. With very thin lifts, the final mat's density of the 4.75mm overlay is difficult to check and often is unnecessary. Instead density should be accomplished by specifying a set rolling pattern (NAPA, 2009).

2.5 SUMMARY OF LITERATURE REVIEW

4.75mm NMAS thin overlay is a viable alternative to other preventive maintenance treatments. In terms of pavement performance, thin overlays are the only preventive maintenance option found to address stability related roughness. The thin overlay should be placed on a lightly cracked surface where a tack coat is applied and/or is milled. The service life of 10 years for thin overlays is in general higher than other preventive maintenance treatments. Even though thin overlays can have a higher upfront cost, because its service life is longer, literature shows its life cycle cost ends up being lower than most treatments. Both Superpave specifications and local design practices have been used for the design of 4.75mm NMAS mix and the results were found to be acceptable. Although some researchers suggest the 4.75mm NMAS thin overlays only be applied on low volume roadways, there is evidence that it can be used on any traffic volume. The use of screening materials and the combination of WMA and RAP can reduce costs of the 4.75mm NMAS mix. The major concerns with this mix include reflective cracking, friction, and heat loss before density is achieved.

CHAPTER 3: SURVEY RESULTS

3.1 INTRODUCTION

In September 2012, an online questionnaire was distributed throughout the United States and Canadian provinces to learn about the use of 4.75mm thin overlays in these areas. This questionnaire was focused on how 4.75mm thin overlays worked for the local agencies and whether it should be an alternative considered by Washington State Department of Transportation. The survey was first developed in Microsoft Word and the questions were then transferred to surveymonkey.com for distribution. There were 38 respondents to the survey with a map of how each state responded was shown in Figure 3-1. The detailed survey questionnaire and responses can be seen in their entirety in Appendix A.

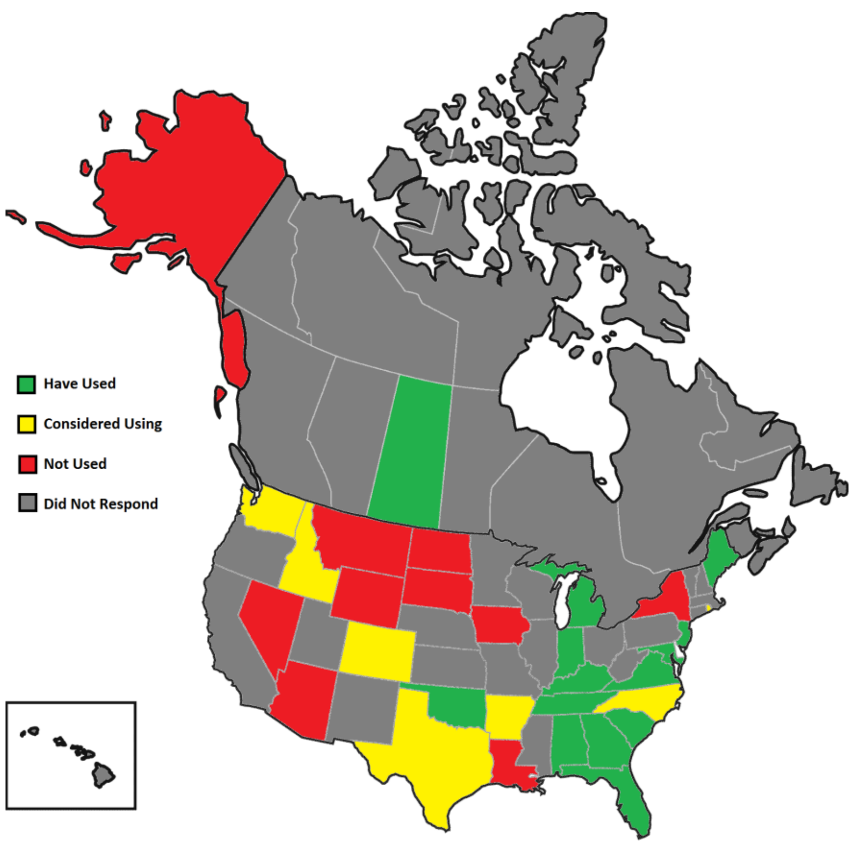


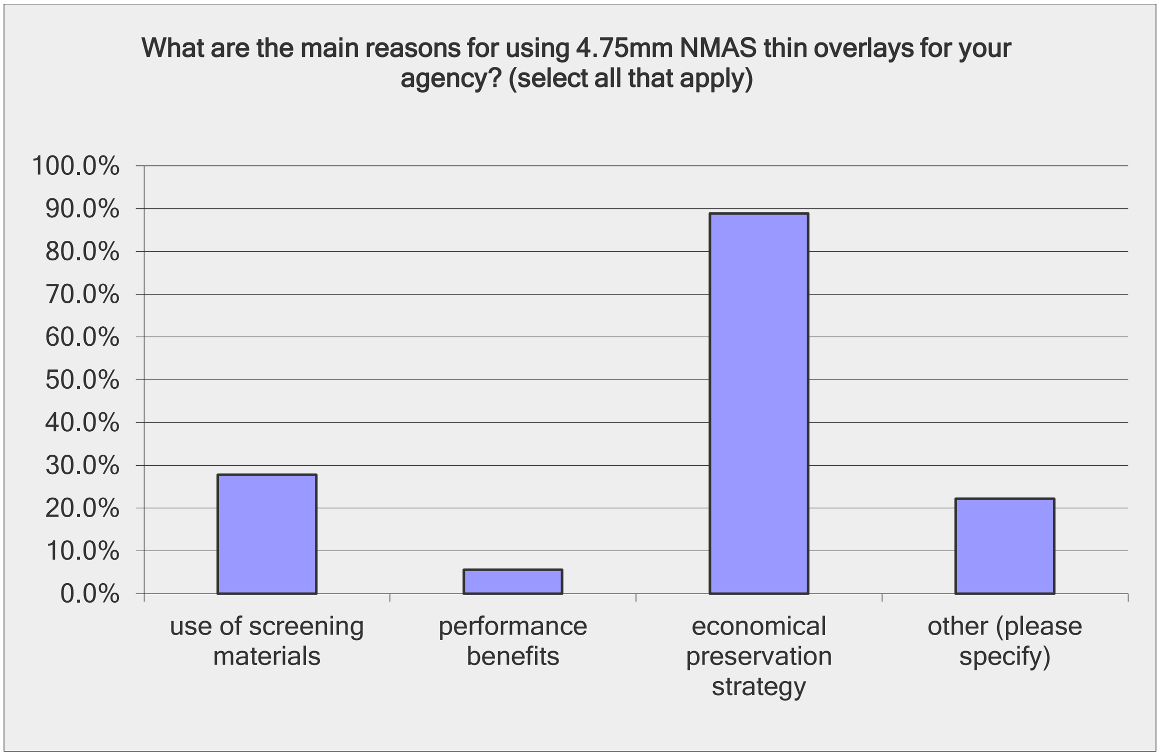
Figure 3-1: Map of Questionnaire Respondents

3.2 SUMMARY

In total 38 responses were received of which 16 agencies had actually used 4.75mm NMAS thin overlays. Most of these states that had used 4.75mm NMAS thin overlay located in the Southeastern part of the United States with one Canadian Provence. The following three sections summarize the survey results.

3.2.1 Usage of 4.75mm thin overlay

There were many reasons for using 4.75mm NMAS thin overlays for different states, and the distribution percentile is shown in Figure 3-2. The most popular reason was an economical preservation strategy which was selected by 88.9% of respondents. All agencies reported less than 100 lane miles had been paved with 4.75mm NMAS thin overlay annually. Some agencies paved as little as 5 to 10 lane miles annually and the maximum reported was 100. The maximum thickness of an overlay reported was 1.125 inches with the main answer being 0.75 - 1 inches thick. Typically agencies would apply 4.75mm NMAS thin overlays when the existing pavement condition was good to fair.



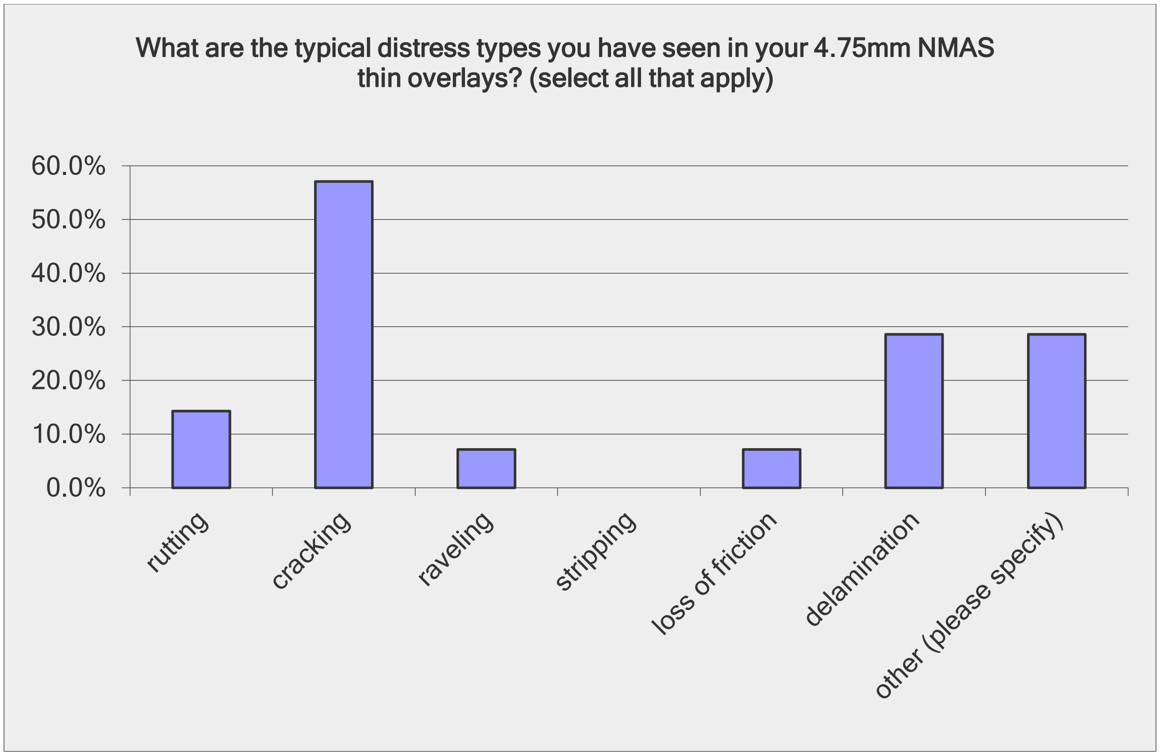
Note: "other" includes: rutting and patching, experimental, and leveling.

Figure 3-2: Main reasons of using 4.75mm NMAS thin overlay

3.2.2 Performance of 4.75mm NMAS thin overlays.

The 4.75mm NMAS thin overlays are reported to be applied to various traffic levels. The ADT levels reported ranged from <1000 ADT to all levels of traffic. The average service life varied greatly as well from 4 - 7 years to up to 15 years, but the average seemed to be about 8 - 10 years. Most agencies (76.9%) reported good performance for the 4.75mm thin overlays except one agency reported poor performance. The different distresses and frequency that are seen in 4.75mm NMAS thin overlays are shown in Figure 3-3, which included rutting, cracking, raveling, stripping, loss of friction, delamination. The two main distresses seen were cracking and delamination and were reported by 57.1% and 28.6% of agencies respectively. Many agencies reported negligible rutting while the highest reported rut depth was 1/4 of an inch. Rutting of 4.75mm NMAS thin overlays compared to typical HMA overlays results are shown in Figure 3-4, which is in general similar or better than typical HMA overlays.

The friction levels were reported mainly the same as typical overlays with 16.7% being reported worse but acceptable. Friction was reported in the literature review as a main concern so these results are helpful. The survey also noted reflective cracking as a concern for 71.4% of agencies ranging mostly from moderate to severe. This result confirms that reflective cracking is a potential problem as the literature review noted. Thermal cracking and stripping were mainly reported as not a concern. Among the states that responded to the survey, eight states permitted the usage of studded tires (Canada, Maine, New Jersey, Oklahoma, Indiana, Kentucky, Tennessee, and Virginia). All of them reported that studded tires do no more damage to 4.75mm thin overlay than larger NMAS mixture conventional overlays. Almost half of reporting agencies used warm mix with good results. These results show the performance of 4.75mm NMAS thin overlays by these agencies is mainly good.



Note: "other" includes: reflective cracking, too early to identify, and only with poor project selection.

Figure 3-3: Typical distresses seen in the 4.75mm NMAS thin overlay

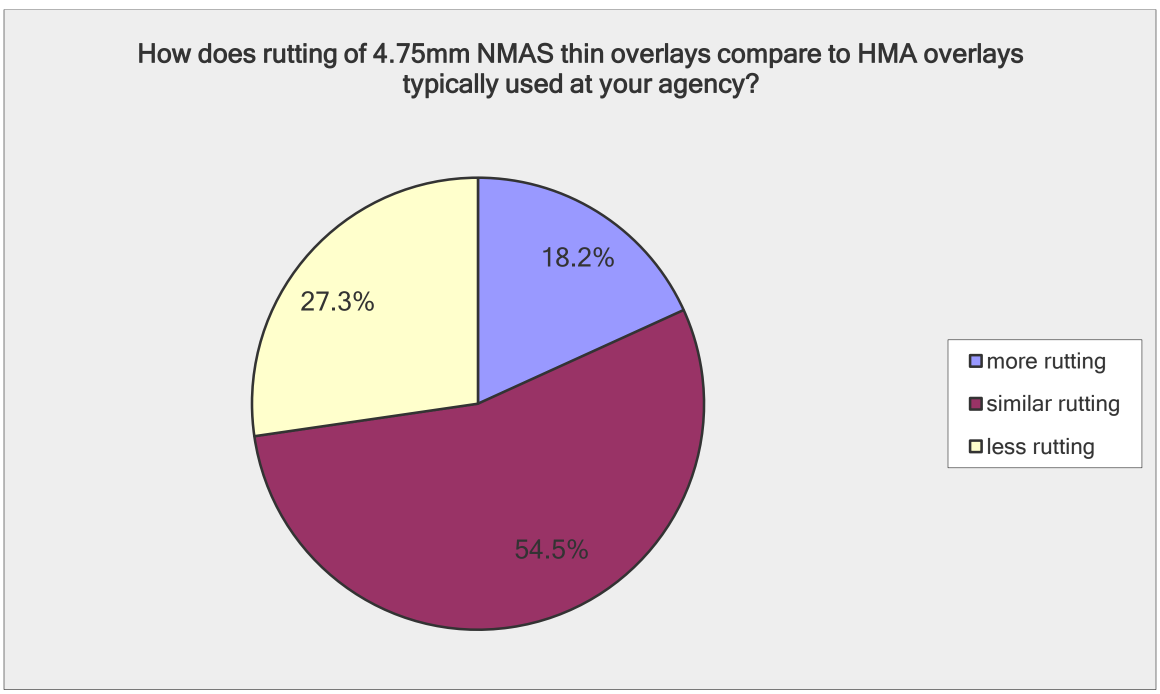
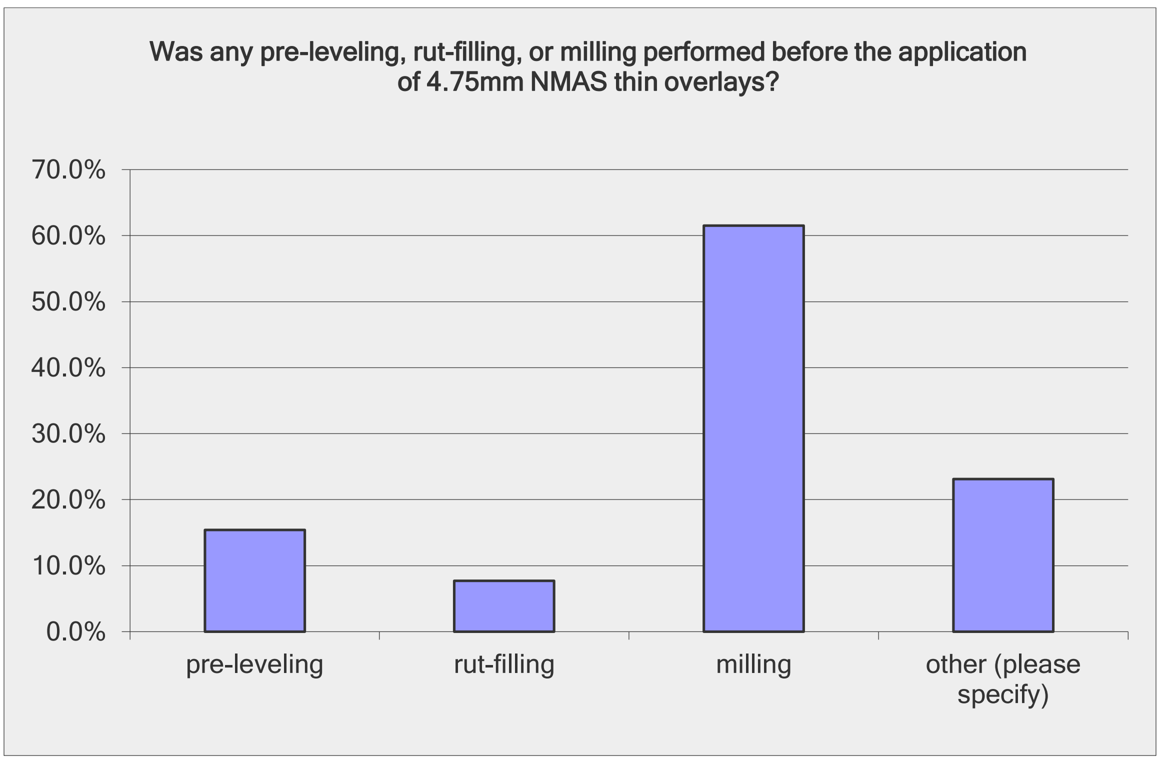


Figure 3-4: Rutting performance of 4.75mm NMAS thin overlay compared to typical HMA overlay

3.2.3 Mix design and construction of 4.75mm NMAS thin overlays

Only 35.7% of agencies use the NCAT specifications while the other 64.3% use their own local method. 73.3% of agencies used RAP in their mixes. The amount of RAP ranged from 10 - 40% with the majority using 10 - 20%. As seen in Figure 3-5, several different types of pavement preparation is done before the application of 4.75mm NMAS thin overlays. The majority (61.5%) milled before application while other types such as thin shim were also used. Most agencies reported density was either not measured or measured by coring because it is laid in thin lifts. No agency reported using vibrating rollers instead using static steel rollers. They also reported either the contractor deciding the rolling pattern or 2 - 4 passes with a steel roller.



Note: “other” responses include: thin shim, no, and some on existing layer

Figure 3-5: Preparation methods for existing pavement before overlay

3.3 ADDITIONAL INFORMATION

After the survey was completed, 13 agencies were contacted further for more information on their mix design and project selection criteria. Seven responded to this request but none of the responses provided additional information on project selection criteria. Responses are summarized in the following sections.

3.3.1 New York DOT

New York DOT uses a 6.3mm mix and not a 4.75mm NMAS mix so all information further will be based on their 6.3mm NMAS mix. 6.3 mm has been used in NY since 2004. It was developed as a HMA alternative to both microsurfacing and Paver Placed Surface Treatment (Novachip).

When a project in NY City called for preventive treatment and neither microsurfacing nor Novachip contractors wanted to do work there, it made necessary to develop an alternative. Knowing its first use was to be on Grand Central Parkway in NYC which is very heavily trafficked roadway, it was decided to use polymer modified asphalt to make sure the mixture did not rut or shove. Two years later, the mixture looked very good. It was decided then that this mix shall be used as an alternative and guidance was provided to the designers for use as a preventive maintenance treatment. The benefits to this mixture as provided by NYDOT include:

1. Reduces the overall cost of the project by 33-50% because it can be placed at minimum of ¾ inch to 1 inch compared to 9.5 mm and 12.5 mm mixes which are 1 ½ inches minimum.
2. The pavement seems to perform much better by reducing moisture intrusion, raveling, and rutting.

3. The ridability on these pavements is much better than some of their normal HMA pavements.

The mixture is designed using Superpave system. The design air void is 4% at 75 gyrations and the PG binder used is either 64-22 or 76-22 polymer modified meeting the Elastic Recovery requirements of 60+.

3.3.2 Michigan DOT

Michigan DOT is currently transitioning to the Superpave Design but its existing specification follows Marshall Design. Table 3-1, 3-2, and 3-3 summarize the design parameters used by Michigan DOT for ultra-thin overlay mix.

Table 3-1: HMA Ultra-Thin Overlay Mixture Requirements

Parameter	Low Volume Comm. ADT <380	Medium Volume Comm. ADT 380 - 3400	High Volume Comm. ADT >3400
Marshall Air Voids %	4.5	4.5	5.0
VMA % (min.) based on Gsb	15.5	15.5	15.5
Fines/Binder % Max.	1.2	1.4	1.4
Flow (0.01 in.)	8-16	8-16	8-16
Stability Min. (lbs.)	1200		

Table 3-2: HMA Ultra-Thin Overlay Aggregate Gradation

Sieve Size	Total Passing Percent by Weight
1/2 inch	100
3/8 inch	99-100
No. 4	75-95
No. 8	55-75
No. 30	25-45
No. 200	3-8

Table 3-3: HMA Ultra-Thin Overlay Aggregate Physical Requirements

Parameter	Low Volume Comm. ADT <380	Medium Volume Comm. ADT 380 - 3400	High Volume Comm. ADT >3400
Percent Crush (min.)	50%	75%	95%
Angularity Index (MTM 118) (min.)	2.5	3.0	4.0
L.A. abrasion loss (max.)	40	35	35
Aggregate Wear Index	(a)	(a)	(a)
a. AWI of 220 is required for projects with less than or equal to 2000 ADT, projects with ADT greater than 2000 the minimum AWI requirement is 260.			

3.3.3 Georgia DOT

Georgia DOT set local specifications on mix design and application of 4.75mm NMAS thin overlays for their state. The design number of gyrations was 50. Table 3-4 provides the recommended design parameters and table 3-5 shows the layer thickness and spread rate control.

Table 3-4: Design for 4.75mm NMAS mix

ASPHALTIC CONCRETE – 4.75 mm Mix		
Sieve Size	**Mixture Control Tolerance**	**Design Gradation Limits, % passing**
1/2 in	±0.0	100*
3/8 in	±5.6	90-100
No. 4	±5.7	75-95
No. 8	±4.6	60-65
No. 50	±3.8	20-50
No. 200	±2.0	4-12
Range for % AC	±0.4	6.00 - 7.50
Design optimum air voids (%)		4.0 – 7.0
% Aggregate voids filled with AC		60 - 80
Minimum Film Thickness (microns)**		>6.00

* Mixture control tolerance is not applicable to this sieve for this mix
** 4.75mm Mixtures approved prior January 31, 2012 may be adjusted to meet Minimum Film Thickness requirement by mixture adjustments made by the State Bituminous Construction Engineer.

Table 3-5: Layer Thickness and Spread Rate

Base Year Two Way ADT	**MIX TYPE**	**LAYER THICKNESS AND/OR SPREAD RATE Customary, (Metric)**			**REMARKS**
		(Minimum)	**USE**	(Maximum)	
<800 800 to 1000	4.75 mm	¾”. 85 lbs./yd^2, (19mm, 45 kg/m^2)	**7/8”. 90 lbs./yd^2, (22mm, 50 kg/m^2)**	1-1/8”. 125 lbs./yd^2, (28mm, 70 kg/m^2)	For State and Off-system Routes with low truck traffic volume (<100 trucks per day)

3.3.4 South Carolina DOT

SCDOT has 2 types of 4.75mm designs, PMTLSC and Type E. Both of these designs use the VMA criteria from Table 3-6. Other specifications are detailed in Tables 3-7 and 3-8. PMTLSC is used as an alternative to mirosurfacing and ST-E is normally used for cross slope corrections, leveling, or as a corrective measure for limited segregation to seal out moisture and prevent raveling.

Table 3-6: VMA requirements for Surface and Intermediate Course

Nominal Max. Aggregate Size	**Minimum, %**
¾”	13.5
½”	14.5
3/8”	15.5
No. 4	17.5

Table 3-7: PMTLSC Mix Information

Required Job Mix Criteria		
Sieve Designation		**% By Weight Passing**
½”	(12.5 mm)	100
3/8”	(9.5 mm)	98-100
No. 4	(4.75 mm)	70-98
No. 8	(2.36 mm)	50-70
No. 30	(0.60 mm)	25-42
No. 100	(0.150 mm)	6-20
No. 200	(0.075 mm)	2-10
Binder Content (%)		5.5 – 7.0
Gyratory Stability (95 +/-5mm)		2500 lbs. min. (50 gyrations)

Table 3-8: Type E Mix Information

Designation	**Type E**
System Application	Seal Course
3/8”	100
No. 4	90-100
No. 8	70-100
No. 30	36-70
No. 100	4-28
No. 200	2-10
Gyrations	50
Binder Limits, %	6.0 – 7.0
LA Abrasion (B), max%	60.0
Absorption, max. %	1.5

3.3.5 Tennessee DOT

Tennessee uses Marshall Mix design for 4.75mm NMAS mixes. A maximum of 15% of both natural sand and RAP can be used in the mix. Other design parameters are shown in Table 3-9.

Table 3-9: Composition by Percent Weight

Sieve Size	Percent Passing (% weight)
½ in.	100
3/8 in.	100
#4	89 - 94
#8	53 - 77
#30	23 - 42
#50	-
#100	9 - 18
#200	6 - 14
Asphalt Cement	5.7 – 7.5%
Design Air Voids	4.0% ± 0.3%
Production Air Voids	3 – 5.5%
Stability	2,000 lbs.
Dust/Asphalt	1.0 – 2.0
VMA	16 min.

3.3.6 Indiana DOT

Indiana has altered NCAT specifications for 4.75mm NMAS mix for use in their state. They chose to use 5% air voids because it was in the middle of the 4-6% design air void limit set by NCAT. They did not want to allow a range for their specifications. Indiana decided on 3 – 8% passing the No. 200 sieve rather than the NCAT recommended 6 – 13%. They used the lower range because of concerns by Indiana DOT and the local HMA industry about the high level of fines replacing the asphalt binder in the mix design. The lower limit of the dust-to-binder ratio was lowered from 1.0 to 0.8 because of the change in the No. 200 sieve limits. The minimum VMA was set at 16% for Indiana. Finally the largest concern Indiana had was the 4.75mm mixtures friction performance. Based on projects Indiana had conducted in the past they determined increasing macrotexture would be the key to improving wet friction performance. Therefore a minimum fineness modulus on the gradation of 3.30 was implemented.

Table 3-10: Gradation of 4.75mm Mixture

Dense Graded Control Points (Percent Passing)	
Sieve Size	**4.75 mm**
12.5 mm	100
9.5 mm	95 - 100
4.75 mm	90 - 100
1.18 mm	30 - 60
0.075 mm	6 - 12

Table 3-11: VFA Criteria vs. ESAL Level

VOIDS FILLED WITH ASPHALT, VFA, CRITERIA @ N_{des}	
ESAL	**VFA, %**
< 300,000	70 - 80
300,000 to < 3,000,000	65 - 78
3,000,000 to < 10,000,000	75 - 78
10,000,000 to < 30,000,000	75 - 78
≥ 30,000,000	75 - 78

3.3.7 Saskatchewan MOH&I

The mix design of 4.75mm NMAS mixes for this Canadian province has the following aggregate specifications. 100% passing the 4.75mm sieve with 100% manufactured crushed fines and a ratio of 80% fines and 20% sands for the mixture. The mix is normally laid at a thickness of 20mm.

Some typical scenarios when 4.75mm NMAS thin overlay is used include:

- Areas of poor ride, segregated surfaces, transverse, and longitudinal cracking.
- Areas where transverse cracking occurs about every 30ft because of the cold weather.
- Usually it is not used in areas of rutting but is used in areas of severe fatigue.

Good performance was noted throughout its use no matter the conditions. It was also noted to be a good alternative to microsurfacing and has a service life from 5 to 8 years.

CHAPTER 4: MIX DESIGN AND LABORATORY PROPERTY EVALUATION

4.1 MIX DESIGN

High performance 4.75mm NMAS thin overlay mixtures were developed in the laboratory. It could be used by WSDOT as a potentially cost effective pavement preservation strategy for high traffic volume roads. Laboratory testing was also conducted to evaluate the properties of the mixtures and compare the results with typical 12.5mm NMAS mixtures for conventional HMA overlay.

In this process first materials were obtained (asphalt and aggregates) for use in creating four different 4.75mm mix designs. Second the mix designs were created using the packing method of mix design. Third the engineering properties of these four mixtures were determined and compared with each other and typical 12.5mm NMAS mixtures for conventional HMA overlay.

4.1.1 Materials

The materials used in this experiment consisted of aggregates and asphalt binder. The aggregates were donated by Poe Asphalt in Pullman, WA. The asphalt binders were donated by Idaho Asphalt Supply in Idaho Falls, ID.

4.1.1.1 Aggregates

The aggregates obtained were from a single 1/4”- pile at Poe Asphalt. The aggregates were shoveled from the pile and brought back to the WCAT laboratory to be dried. Sieve analysis was conducted on the dried aggregate giving the gradation shown in Table 4-1. Other aggregate properties were also tested and the results were shown in Table 4-2.

Table 4-1: Aggregate Gradation

Gradation	% Passing
1/2"	100
3/8"	100
4	85.8
8	58.0
16	38.9
30	28.1
50	20.8
100	15.6
200	11.1

Table 4-2: Aggregate Properties

Testing Procedure	Coarse	Medium	Specifications
Bulk SG (Gsb)	2.759	2.759	
App. SG (Gsa)	2.968	2.968	
Absorption	2.5%	2.5%	
Surface Area	38.36	37.38	Min
Sand Equivalency	78	78	45
Flat and Elongated	99	99	90
Uncompacted Voids	47.8	47.8	45

4.1.1.2 Asphalt

Targeting on high traffic volume roads, the asphalt used for this study were PG70-28 and PG76-28 binders. All asphalt properties were given by Idaho Asphalt Supply and shown in Table 4-3.

Table 4-3: Asphalt Binder Properties

	Specific Gravity	Mixing Temp. Range	Compaction Temp Range
PG76-28	1.0322	164-171	145-154
PG70-28	1.0324	159-166	141-150

4.1.2 Background of Mix Design Method Based on Packing Theory

In this study, a new mix design method based on the packing theory was used to develop a mixture with good aggregate interlocking (Shen and Yu, 2011). Packing can be defined as the arrangement of the particles which fit together to fill voids. By developing a balanced gradation, the aggregate interlock can be realized and the stability of the mix can be improved (Shen and Yu, 2011). Following the procedure described by Shen and Yu (2011), new sets of packing parameters fv, which is defined as the percent of voids change by volume due to the addition of unit aggregate, were developed for coarse and medium graded 4.75mm NMAS mixtures using computational Discrete Element Method (DEM). Detailed explanation of the DEM approach for fv values determination has beyond the scope of this thesis and will be presented elsewhere. These fv values can be used to predict the VMA of a certain design gradation, therefore, to evaluate the suitableness of the gradation. While the overall packing theory method was developed by Shen and Yu (2011), several adjustments were made so this mix design method could be used for 4.75mm NMAS mixtures. These include a new P_{dc} equation, gradation P_{dc} cutoffs, and DEM fv values.

4.1.3 Mix Design Process

The concept of the new mix design method consists of two major steps:

1. **Selecting gradation based on VMA.** The VMA of asphalt mixtures based on the aggregate gradation is predicted based on packing theory.
2. **Estimating design asphalt content.** The effective asphalt content can be calculated and the target optimum asphalt content can be estimated as well. Gyratory specimens are made to verify the asphalt content.

Details of the new mix design method based on the packing concept is shown below. It should be noted that this new design method produces mixtures that satisfies Superpave volumetric specification in an easy way. At the same time, it considers particle interlocking for a strong mix.

4.1.3.1 Gradation Type Definition

As a first step for a systematic gradation design, more scientific definitions of gradation types and shapes are needed since different gradations can behave quite differently in terms of particle packing, volumetric properties, and field performance. Conceptually, coarse and fine graded mixtures are usually categorized depending on whether the gradation curve is passing below or above the maximum density line. A new method was developed by Shen and Yu (2011) which categorizes the different gradation types based on their packing characteristics and was proved to be able to be related to the aggregate contact performance. $P_d(d)$, the percent of aggregates (size d) deviating from the maximum density line, can be obtained from Equation 4-1.

$$P_d(d)=P(d)-P_{Dens.} \tag{4-1}$$

Where

P(d). : Percent of aggregates passing sieve size d for a specific gradation (%);

$P_{Dens.}$: Percent of aggregates passing the maximum density line (%), which can be calculated from Equation 4-2;

$$P_{Dens.}=\left(\frac{d}{D_{\max}}\right)^{0.45}\times 100\% \tag{4-2}$$

Where

d: sieve size (mm);

D_{max}: maximum sieve size for that gradation (mm).

A critical deviation value, P_{dc}, classifies different gradation types. The P_{dc} is the sum of the deviations of three medium sieve sizes, the sieves that play important roles in determining a gradation curve shape. These sieves (2.36mm, 1.18mm, and 0.6mm) have the largest difference between different mixtures and are shown outlined in Figure 4-1. For a gradation with a NMAS of 4.75mm, the P_{dc} can be calculated using Equation 4-3.

$$P_{dc} = P_d(2.36)+P_d(1.18)+P_d(0.6) \tag{4-3}$$

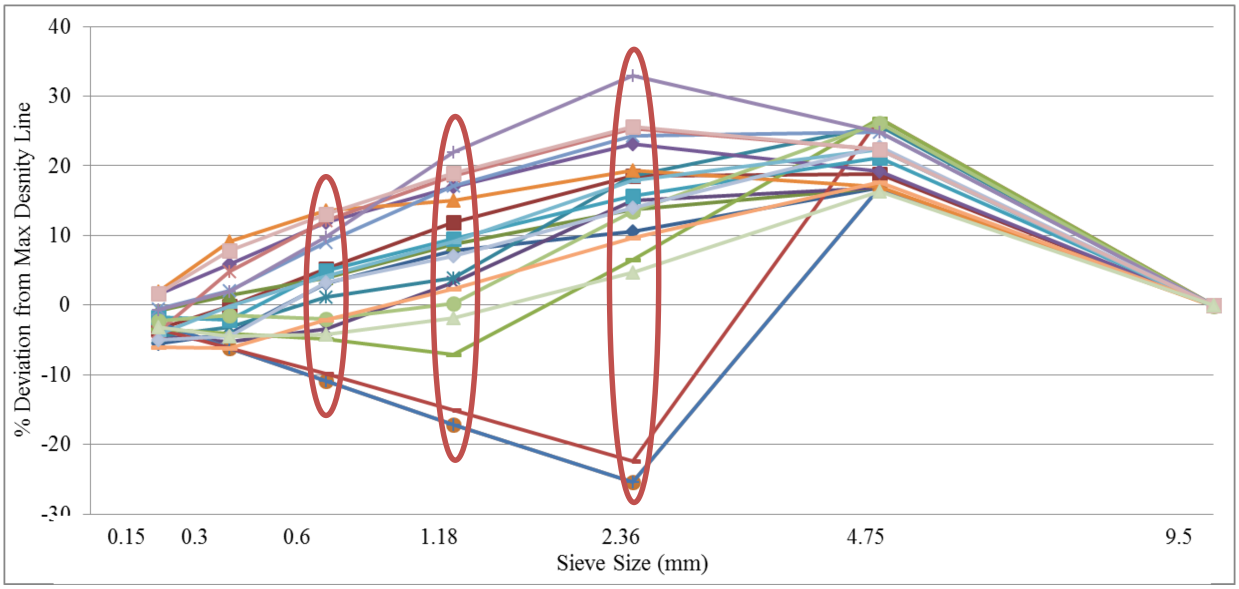


Figure 4-1: Percent Deviation from Max Density Line of a large number of 4.75mm Mix Designs with sieves used outlined in red

Table 4-4 lists the recommended ranges for P_{dc} to categorize three different gradation types, coarse-graded, medium-graded, and fine-graded gradations. The breaking points were determined by grouping gradations, as can be seen in Figures 4-2, 4-3, and 4-4, and reviewing the Pdc values to determine the groupings.

Table 4-4: P_{dc} criteria for different gradation types

P_{dc}	Gradation type
$P_{dc} \leq 0$	coarse-graded
$0 < P_{dc} \leq 30$	medium-graded
$P_{dc} > 30$	fine-graded

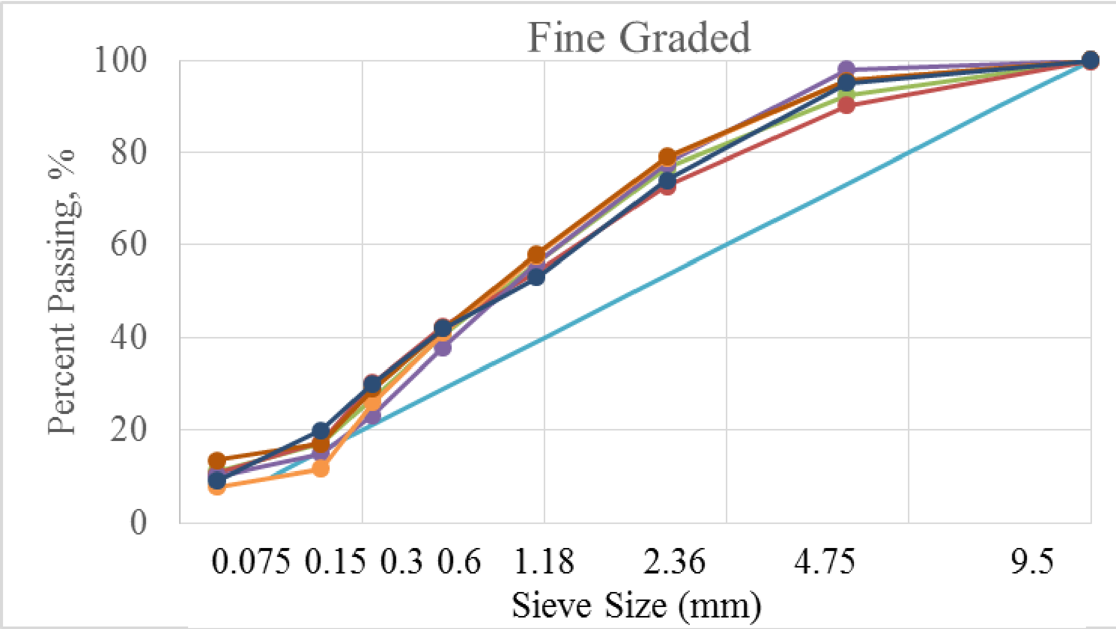


Figure 4-2: Grouped Fine Graded Mixtures

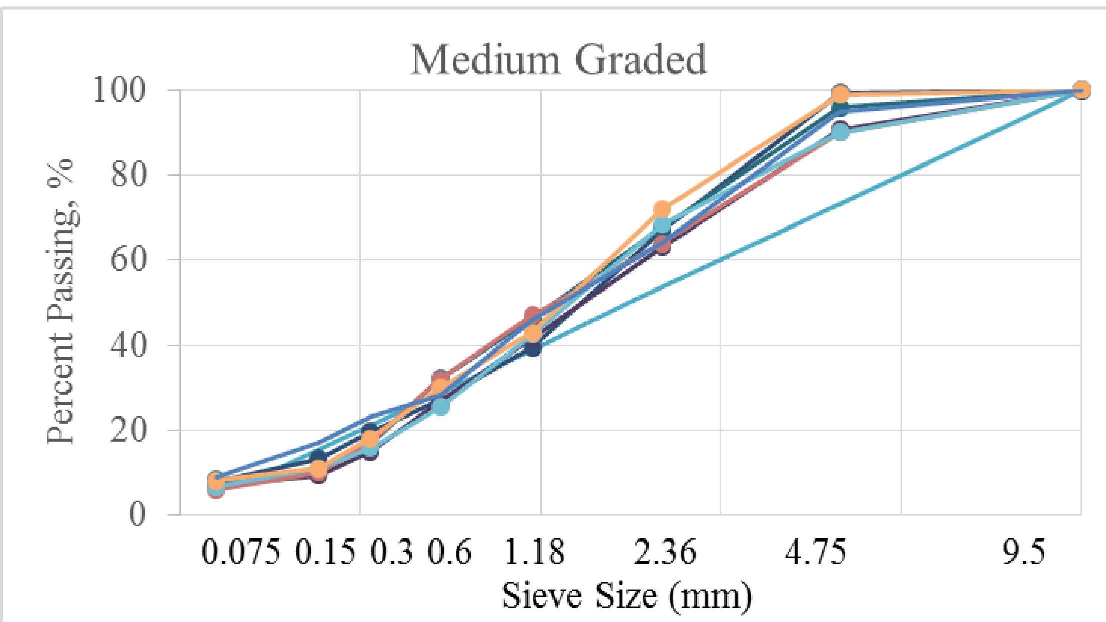


Figure 4-3: Grouped Medium Graded Mixtures

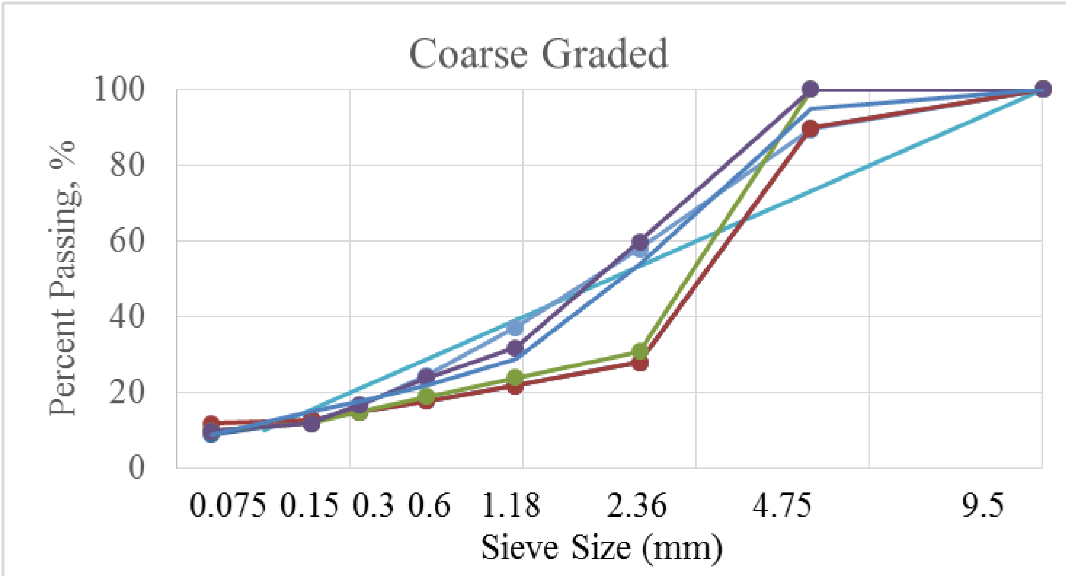


Figure 4-4: Grouped Coarse Graded Mixtures

4.1.3.2 VMA Prediction

A gradation weighing factor, f_v, is developed by Shen and Yu (2011) to link the gradation information directly to the VMA. The prediction of VMA is an iteration process starting from an aggregate structure with the largest uniform size aggregates (i.e. NMAS). When the aggregates one size smaller were added into the structure based on the target gradation, the percent of voids change of the aggregate structure due to the newly added aggregates can be determined by Equation 4-4.

$$f_v = \frac{V_{v2} - V_{v1}}{V_{a2}} \tag{4-4}$$

Where,

V_{v1}: Total air void volume of the initial aggregate structure

V_{v2}: Total air void volume of the new aggregate structure after adding smaller size aggregates

V_{a2}: The added aggregate volume determined according to aggregate gradation.

The newly added aggregates typically have two effects, either enlarge the structures by creating more voids, or fill the voids created by the original aggregates without changing the total volume of the structure. If all added aggregates contribute to filling the voids created by the aggregate, the final porosity will be reduced and the f_v will be constant -1. In real gradations, it is typical that part of the added aggregates serves as creating voids while others serve as filling the voids. The actual f_v values will thus be in between these two extreme cases, i.e., between $p_1/(1-p_1)$ and -1.

Following the same procedure, smaller aggregates can be further added into the structure of upper sieve size aggregates according to the target gradation, and the corresponding *fv* values for each added sieve sizes can be determined. Shen and Yu (2011) suggested two ways to determine the *fv* values, either by data regression based on existing designs, or from the packing simulation using the DEM modeling. The regression method takes into account the realistic particle morphological properties (shape, angularity, etc.) while the DEM modeling assumes spherical particles but with calibrated model parameters. The regression method requires a large number of mix designs with known VMA values while the DEM modeling method is more useful when only limited number of known designs are available. Shen and Yu (2011) found both methods produce similar VMA prediction quality. Because there is not a large enough database to determine the *fv* values of the 4.75mm NMAS mix through a regression method, this study used the DEM approach to calculate the *fv* values. The recommended f_v results for coarse-graded and medium-graded gradations of 4.75mm NMAS are shown in Table 4-5.

Table 4-5: f_v factors for different graded mixes and sieve sizes

	Coarse-grade	Medium-graded
Sieve size (*mm*)	DEM Simulation	DEM Simulation
9.5	0	0
4.75	0.582	0.59
2.36	0.462	0.475
1.18	0.322	0.402
0.6	0.158	0.203
0.3	-0.09	-0.09
0.15	-0.366	-0.266
0.075	-0.518	-0.47
Pan	-0.518	-0.47

Once all f_v values for each sieve size are determined, Equation 5-5 will be used to predict the VMA (or porosity) of the asphalt mixtures.

$$p = \frac{\sum_{i=1}^{n} f_{vi} V_{ai}}{\sum_{i=1}^{n} (1 + f_{vi}) V_{ai}} \tag{4-5}$$

Where

f_{vi} = the f_v value for ith sieve size of the gradation

V_{ai} = the percentage by volume of aggregate retained in the ith sieve size

p = the porosity or VMA of the aggregate structure

4.1.3.3 Gradation Selection

The method of predicting VMA can be used for selecting design gradations. Given any trial gradations, their target VMA can be determined in the excel spread sheet using the procedure described above. Initial adjustment on gradation could have been made based on the VMA criteria. It is possible the gradations need to be adjusted again to satisfy performance requirement and volumetric requirement, which will be described in later sections of this paper.

4.1.3.4 Estimating Design Asphalt Content

Based on the selected aggregate gradation and the predicted VMA, the design asphalt content corresponding to 4% air voids can be estimated. The effective asphalt content (P_{be}) is calculated first using Equation 4-6.

$$P_{be}=(VMA-V_a)* G_b/G_{sb} \tag{4-6}$$

Where,

VMA = voids in mineral aggregate

V_a = percent air voids

G_b = specific gravity of binder

G_{sb} = specific gravity of aggregates

For a given type of aggregate and asphalt binder, the absorption rate of asphalt binder should be relative consistent and can be determined from experimentation. Therefore, the design asphalt content (P_b) required producing a mix with known VMA and a design air void of 4% can be estimated based on Equation 4-7.

$$P_b = (P_{be} +(P_{ba}/100))/(1+(P_{ba}/100)) \tag{4-7}$$

Where,

P_{ba} = the asphalt absorption rate by weight of total aggregates, and

This method can also be used to optimize asphalt content by adjusting the gradations. As indicated above, the optimum asphalt content is mainly determined by the VMA for a given type of aggregate and asphalt binder. By adjusting the proportions of aggregates and the way of aggregate packing, a designer will be able to minimize the asphalt binder content for cost and other consideration while still maintain the necessary volumetric properties. To verify the design asphalt content, it is recommended to prepare 3 gyratory specimens using the selected aggregate gradation and the design asphalt content and determine the volumetric properties of the specimens.

4.1.4 Final Expected Volumetrics

Using the design procedure described above four mix designs were created with two gradations (medium graded and coarse graded) and two binder types (PG70-28 and PG76-28). The target VMA was 17%, 1% above the minimum criteria given by Superpave. The gradations of the mix designs are shown in Table 4-6 and Figure 4-5.

Table 4-6: Gradations

Medium	Coarse
100	100
100	100
90	90
58	62
36	42
24	31
18	20
13	13
8	6

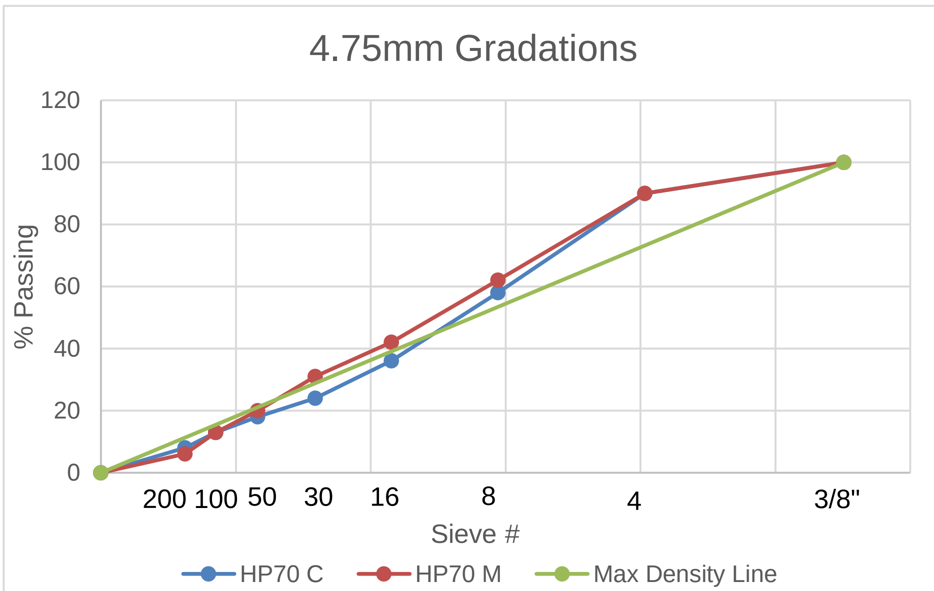


Figure 4-5: Proposed Design Gradations

From the gradations in Figure 4-5 complete volumetrics were determined for these 4.75mm mixtures using the procedure described above and are shown in Table 4-7. All of the estimated volumetrics pass Superpave specifications (NCAT, 2011).

Table 4-7: Medium and Coarse Estimated Volumetrics

	Coarse	Medium
Va	4.0%	4.0%
VMA	17%	17%
Gsb	2.759	2.759
Gb	1.034	1.034
SA	38.36	37.38
Pbe	5.54	5.54
Pba	1.70	1.90
Pb	7.13	7.31
Film Thick	7.04	7.22
Vbe	0.130	0.130
DP	1.44	1.08
VFA	76.47	76.47

Laboratory gyratory specimens were prepared to verify the estimated mix designs. Consequently, asphalt content was slightly reduced to 6.75% to ensure the designed mix satisfy all the volumetric criteria. The volumetrics of the verification specimens for all mixtures are shown in Table 4-8 and 4-9.

Table 4-8: Mix Design Results for High Performance PG 70-28 Mixture

Blend	PG70-28 Coarse Graded			PG70-28 Medium Graded			Superpave Specification
	Spec 1	Spec 2	Avg	Spec 1	Spec 2	Avg	
Asphalt Content	6.75%			6.75%			
Gmm	2.577			2.589			
Gmb @Ndes	2.482	2.484	2.483	2.476	2.475	2.476	
%Gmm @Ndes	96.3%	96.4%	96.3%	95.6%	95.6%	95.6%	96%
Air Voids @Ndes	3.7%	3.6%	3.7%	4.4%	4.4%	4.4%	4%
VMA @Ndes	16.1	16.1	16.1	16.3	16.4	16.3	16 Min.
VFA @Ndes	77.1	77.4	77.3	73.3	73.0	73.2	65 - 78
Vbe	12.4	12.4	12.4	12.0	11.9	11.9	11.5 - 13.5
%Gmm @Nini	85.4%	85.8%	85.6%	86.1%	85.1%	85.6%	< 89
Effective Asphalt	5.17%			4.98%			
Film Thickness	6.56			6.49			> 6
D:B Ratio	1.55			1.20			0.9 - 2.0

Table 4-9: Mix Design Results for High Performance PG 76-28 Mixture

Blend	PG76-28 Coarse Graded			PG76-28 Medium Graded			Superpave Specifications
	Spec 1	Spec 2	Avg	Spec 1	Spec 2	Avg	
Asphalt Content	6.75%			6.75%			
Gmm	2.575			2.579			
Gmb @Ndes	2.481	2.484	2.482	2.482	2.483	2.482	
%Gmm @Ndes	96.4%	96.5%	96.4%	96.2%	96.3%	96.3%	96%
Air Voids @Ndes	3.6%	3.5%	3.6%	3.8%	3.7%	3.7%	4%
VMA @Ndes	16.1	16.1	16.1	16.1	16.1	16.1	16 Min.
VFA @Ndes	77.5	78.0	77.7	76.6	76.9	76.7	65 - 78
Vbe	12.5	12.5	12.5	12.3	12.4	12.4	11.5 - 13.5
%Gmm @Nini	86.9%	86.8%	86.8%	85.9%	87.4%	86.6%	< 89
Effective Asphalt	5.20%			5.14%			
Film Thickness	6.61			6.69			> 6
D:B Ratio	1.54			1.17			0.9 - 2.0

4.1.5 Summary

In summary four mix designs were created that pass Superpave specifications (NCAT, 2011) with the same amount of added asphalt binder. This will allow for easy comparison without the contributing factor of asphalt content. The next step is to create samples for performance testing with these four mix designs and determine the viability of each.

4.2 PERFORMANCE TESTING

Two critical performance tests were conducted to compare the four different mix designs. The first test was to find the moisture susceptibility and rutting resistance using Hamburg Wheel-Track Testing. This test was conducted by WSDOT because of the lack of equipment available in the WCAT laboratory at WSU. Second, cracking potential was determined using Indirect Tensile (IDT) fracture tests. Low temperature (-10C) and room temperature (20C) were used in IDT testing to help determine cracking potential in different situations. At low temperature thermal cracking potential can be determined while at room temperature fatigue cracking potential is determined.

These results are compared between these 4.75mm mixtures as well as control 12.5mm mixtures. The procedures and results of the tests for all four mixtures are presented as well as the sample preparation and testing equipment used.

4.2.1 Sample Preparation

The samples were prepared using the Superpave Gyratory Compactor (SGC) in the WCAT laboratory at WSU shown in Figure 4-7. For Hamburg wheel track testing, four samples of each mix design were created at a height of 62mm with a diameter of 150mm. These samples were then sent to WSDOT for testing. For IDT testing six samples were created for each mix design. These samples were cored with a diameter of 102mm and cut at a height of 38.1mm samples for IDT testing. In the end a total of four Hamburg and six IDT samples were created for each of the four mix designs and one of each sample is shown in Figure 4-6. A list of samples made is shown in Table 4-10.

Figure 4-7: Superpave Gyratory Compactor

Figure 4-6: Hamburg Sample (left), IDT sample (right)

Table 4-10: List of Samples Made

	High Performance Mix			
	Low IDT	Med IDT	Hamburg	
PG76 Coarse	3	3	4	
PG76 Medium	3	3	4	
PG70 Coarse	3	3	4	
PG70 Medium	3	3	4	Subtotal
Total	12	12	16	40

4.2.2 Equipment

Below is a summary of the testing equipment used for performance results.

4.2.2.1 Hamburg Wheel-tracking Testing

The Hamburg Wheel-Tracking Machine is an electronically powered machine capable of moving a 203.2-mm (8-in) diameter, 47-mm (1.85 in) wide steel wheel over a test specimen. The load on the wheel is 705 ± 4.5 N (158 ± 1.0 lb.). The wheel shall reciprocate over the specimen, with the position varying sinusoidally over time. The wheel shall make 50 passes across the specimen per minute. The maximum speed of the wheel shall be approximately 0.305 m/s (1 ft/s) and will be reached at the midpoint of the specimen. The test takes approximately 6.5 hours.

Figure 4-8: Hamburg wheel-tracking device

4.2.2.2 Indirect Tensile Strength (IDT) Testing

The machine used for indirect tensile strength was an MTS hydraulic powered system with a Geotechnical Consulting Testing Systems (GCTS) environmental chamber, servo valve controlled computer and software. This machine was used to find fatigue and thermal cracking performance of HMA mixtures. The performance characteristics measured included fracture energy, fracture work density, and IDT strength. The devices environmental chamber can control the temperature by raising or lowering depending on the test. The specimen is placed in the loading frame with a capacity load cell of 44,000N with this load being applied until the specimen fails. A computer records the data and the software puts it into a viewable format for analysis.

Figure 4-9: IDT testing device

4.2.2.3 CoreLok®

The CoreLok® is a system for sealing asphalt samples so that the sample densities may be measured by water displacement methods. Samples are automatically sealed in specially designed puncture resistant polymer bags. Densities measured with the CoreLok® system are highly reproducible and accurate. The results are not dependent on material type or sample shape. The GravitySuite™ PC software package calculates and manages your data for ease of operation (InstroTek, Inc., 2012). In this study the CoreLok® system was used to measure the porosity of the samples but not the density.

Figure 4-10: CoreLok® Machine

4.2.3 Porosity

Porosity measures the percentage of water permeable voids in a compacted HMA sample. Porosity describes the interconnectivity of voids within the sample and gives an accurate estimate of the samples permeability (InstroTek, Inc., 2012). The permeability of 4.75mm mixtures is desired because the less permeable the mixture the less susceptible to moisture damage. Using the CoreLok® porosity was found using the following steps. First the sample and an empty vacuum bag are weighed and their masses are recorded. Next the sample is placed into the bag and the air is removed from the bag using the CoreLok® machine. Third the bag plus sample are weighed underwater and finally the bag is cut and the mass is taken again. Using these masses the porosity can be calculated using Equation 4-8 in the GravitySuite™ software. The results of porosity and air voids for the mix design samples are shown in Table 4-11.

$$\% \text{ Porosity}=(p2-p1/p2)*100 \tag{4-8}$$

Where,
p1 = the CoreLok® vacuum sealed density of compacted sample
p2 = Density of the vacuum sealed sample after opening under water

Table 4-11: Porosity and Air Voids of Mix Design Samples

Sample ID	Porosity %	Air Voids %
HP70C3	1.49	3.7
HP70C4	1.91	3.6
HP70M4	2.47	4.4
HP70M5	1.66	4.4
HP76C1	2.58	3.6
HP76C3	1.53	3.5
HP76M1	2.18	3.8
HP76M3	2.16	3.7

As can be seen from Table 4-11, the porosity of all samples is less than their respective air voids. With smaller aggregate particles, the 4.75mm NMAS mixtures appear to be less permeable even with a higher air void content. This made it a desirable surface mix to reduce moisture damage.

4.2.4 Hamburg Wheel Tracking Testing

Hamburg Wheel Tracking test was used to evaluate the rutting and moisture damage potential of the designed mixtures. The testing procedure follows WSDOT FOP for AASHTO T 324. In this test, gyratory compacted HMA specimens were repetitively loaded using a reciprocating steel wheel. The specimens are submerged in a temperature-controlled water bath of 49°C ± 1.0°C. The deformation of the specimen caused by the wheel loading is measured at 20 and 50 pass intervals. These results are loaded into an excel file where the data can be analyzed and a plot can be created.

The data received from Hamburg wheel-track testing included height data increasing by 20 and 50 passes from 0 to 20,000 passes. Sensor 1-11 height data is given at each of the passes reported. For this analysis the lowest value at each reported pass is found. A plot of the number of wheel passes vs. rut depth in millimeters was then created using the lowest rut depth of all 11 sensors and can be seen in Figure 4-11.

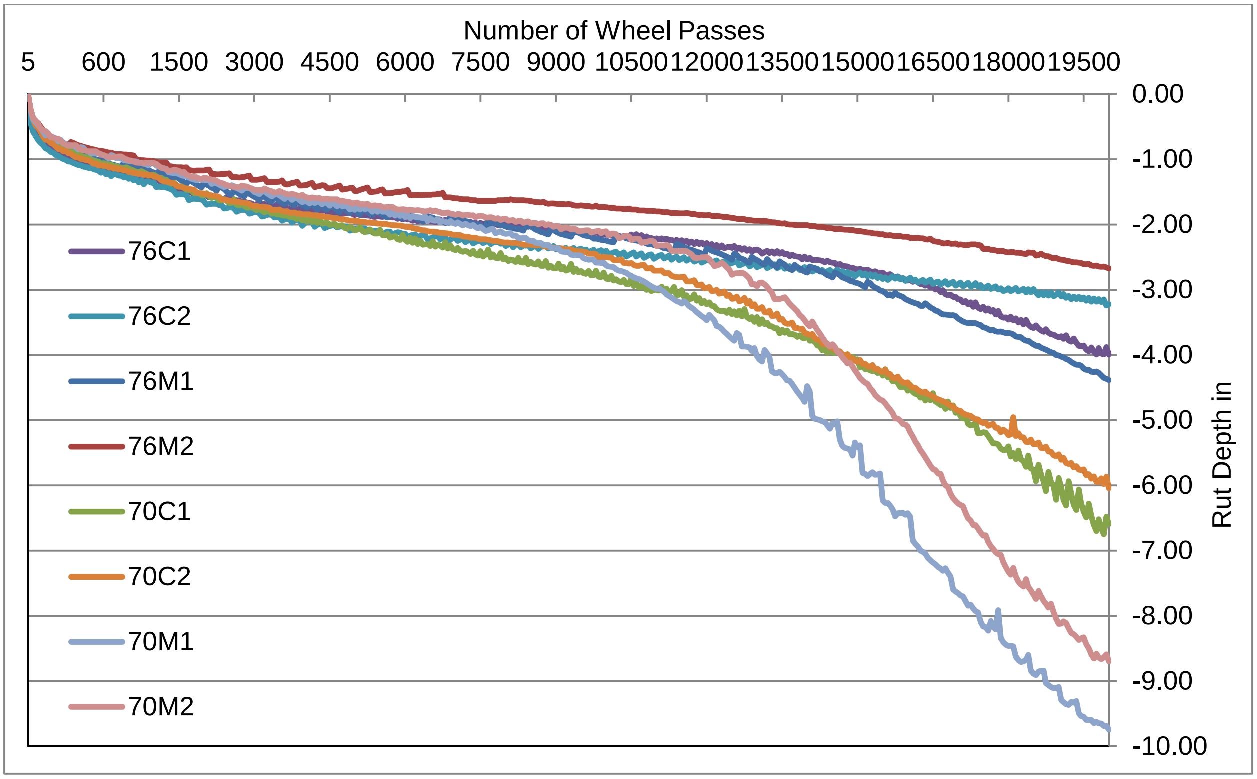


Figure 4-11: Hamburg Wheel Track Results

Stripping inflection point (SIP) is defined as the transition point between the creep slope and the stripping slope. After this point moisture damage starts to dominate performance (Yildirim, 2001). In this study, the SIP is determined using the following practical method. First the slope and intercept of the first segment of the curve before transition point is determined, defined as m_1 and b_1 respectively as shown in Figure 4-12 as slope 1. Second, the slope and intercept of the second segment of the curve after transition point is determined, defined as m_2 and b_2 in Figure 4-12 as slope 2. Using Equation 4-9, the slope and intercepts from both lines are used to find the number of passes where the SIP occurs.

$$SIP = (b_2-b_1)/(m_1-m_2) \tag{4-9}$$

Where,

SIP = stripping inflection point

b_1 = intercept line one

b_2 = intercept line two

m_1 = slope line one

m_2 = slope line two

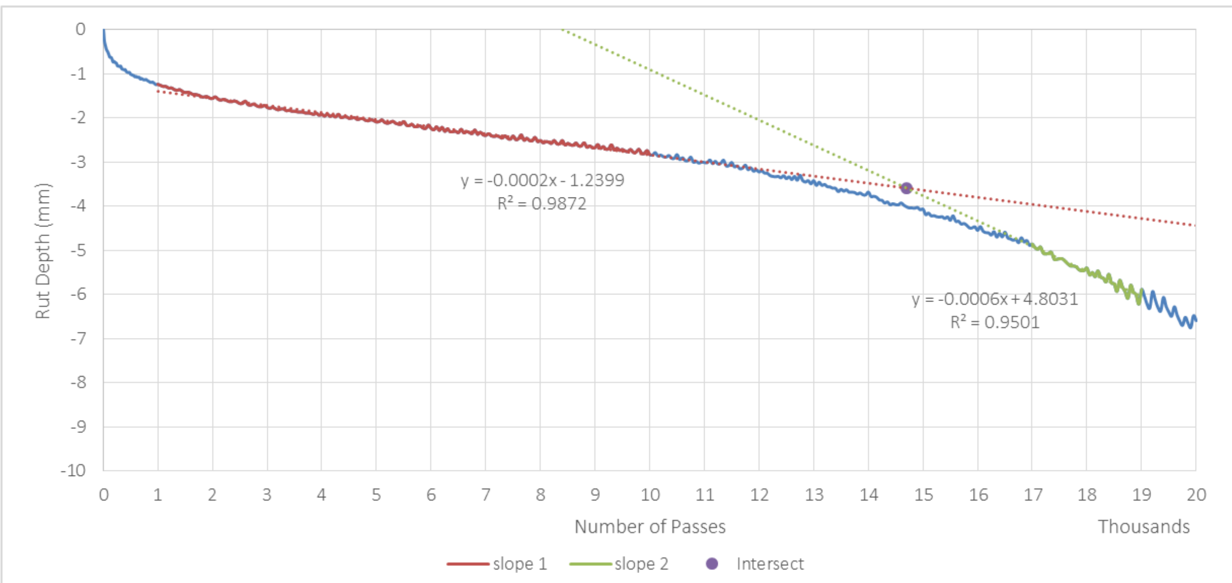


Figure 4-12: A Schematic of Stripping Inflection Point Diagram from Hamburg Test Result

4.2.4.1 Hamburg Test Results

Results from Hamburg wheel tracking test are shown in this section. The rut depths were found at the end of 20,000 passes and are shown in Table 4-12. The stripping inflection points were found at the intersection of the creep and stripping slopes and are shown in Table 4-13.

Table 4-12: Hamburg Rut Depths at 20,000 Passes

Rut Depth (mm)	70C	70M	76C	76M
Test 1	6.59	9.74	3.99	4.38
Test 2	6.05	8.70	3.22	2.67
Average	6.32	9.22	3.61	3.53

Table 4-13: Hamburg Stripping Inflection Point

SIP	70C	70M	76C	76M
Test 1	14702	11634	14962	14271
Test 2	10604	13360	none	none
Average	12653	12497	14962	14271

4.2.4.2 Discussion

Comparing four 4.75mm mix designs both PG76 mixtures are more rut resistant than PG70 mixtures. The maximum allowable rut depth according to 2014 WSDOT standards is 10mm at 15,000 passes. Note this standard is for 12.5mm mixture which could be different for 4.75mm mixture depending on future evaluations. All 4.75mm NMAS mixtures met the requirements for Hamburg wheel-track testing even at 20,000 passes with the largest rut depth being 9.22mm by the medium graded PG70-28 mixture.

The 2014 WSDOT standard for SIP is there should be no SIP before 15,000 passes. It can be seen that all mixtures fails this criteria. The largest SIP occurs at 14,962 passes just below the minimum by the 76C mixture while the lowest occurs by the 70M mixture at 12,497 passes. This indicates that some amount of anti-stripping agent may need to be added to these mixtures to allow for passing of this requirement.

For PG76-28 binder mixtures, there was no clear difference in rut depth and SIP between medium and coarse graded gradations. The PG70-28 binder mixtures on the other hand had almost 3mm deeper rut depth in medium graded mix than that in coarse graded mix. When evaluating the effect of binders, the PG70-28 binder mixtures had much higher rut depths in general than that of the PG76-28 binder mixtures. Although the PG76-28 binder mixtures showed more rut resistance the PG70-28 mixtures also pass WSDOT standards and can be considered for use.

The rutting resistances of the developed 4.75mm NMAS mixtures were also compared with the conventional 12.5mm NMAS mixtures that were used by WSDOT in overlay projects in 2010 and 2011. These 12.5mm mixtures were not created for this study and therefore have different mix designs, materials, and other factors that can lead to an inaccurate comparison. Therefore they should be viewed as a general comparison only. Figure 4-13 shows the averages and standard deviation (shown in error bars) for the four mixtures from this study and two comparison mix types.

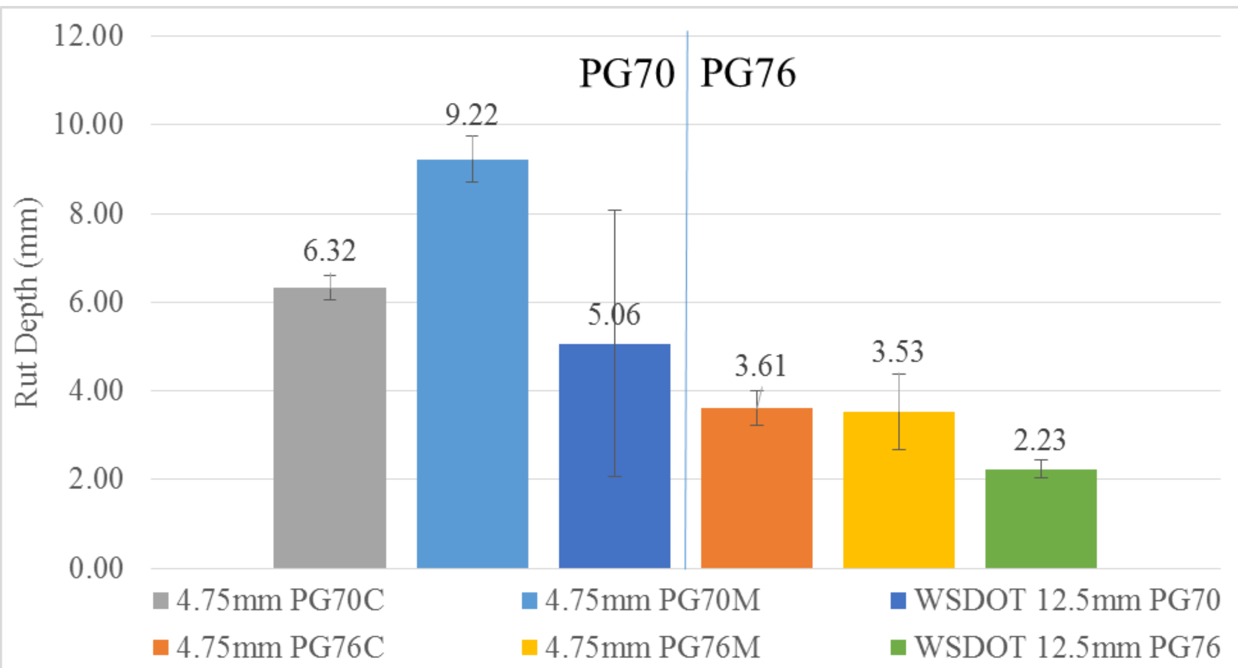


Figure 4-13: Rut Depth of Mix Designs with Error Bars

As can be seen from Figure 4-13, the 4.75mm mixtures in general have higher rut depth than the 12.5mm mixtures. However, all mixtures met the WSDOT specification and should perform well with respect to rutting performance.

4.2.5 IDT Testing

The indirect tensile testing involves the vertical loading of a cored specimen along its vertical diameter. A 44,000 N capacity load cell is used for testing. The specimen was placed in a loading frame consisting of a plate on the top and bottom guided by four steel bars. The bars keep the load applied in the vertical direction only. The samples are loaded at a rate of 52mm/min for fatigue samples and 2.54mm/min for thermal samples.

For measurement of deformations in each sample, four linear variable differential transformers (LVDT) were attached to mounts on the specimen. The LVDT's were mounted at the center of the specimen with two LVDT's placed in the vertical direction and two in the horizontal direction. From these LVDT deformation measurements, strain in the center of the specimen is calculated. The temperature is controlled in an environmental chamber at 20C for fatigue tests and -10C for thermal tests. The results are loaded into an excel file where the data can be analyzed.

The fracture energy and fracture work required to split the specimen were calculated for both fatigue and thermal tests. Fracture energy, also known as strain energy, is calculated as the area under the stress-strain curve but only until maximum stress (Figure 4-14). Fracture work density is determined as the area under the entire load-displacement curve (Figure 4-15). Both fracture energy and fracture work density evaluate not only the strength but also the ductility of the material. According to Kim and Wen (2002), the fatigue fracture energy results were found to correlate well to fatigue cracking resistance. Zborowski (2007) found the thermal fracture energy to be good indicator of resistance to thermal cracking.

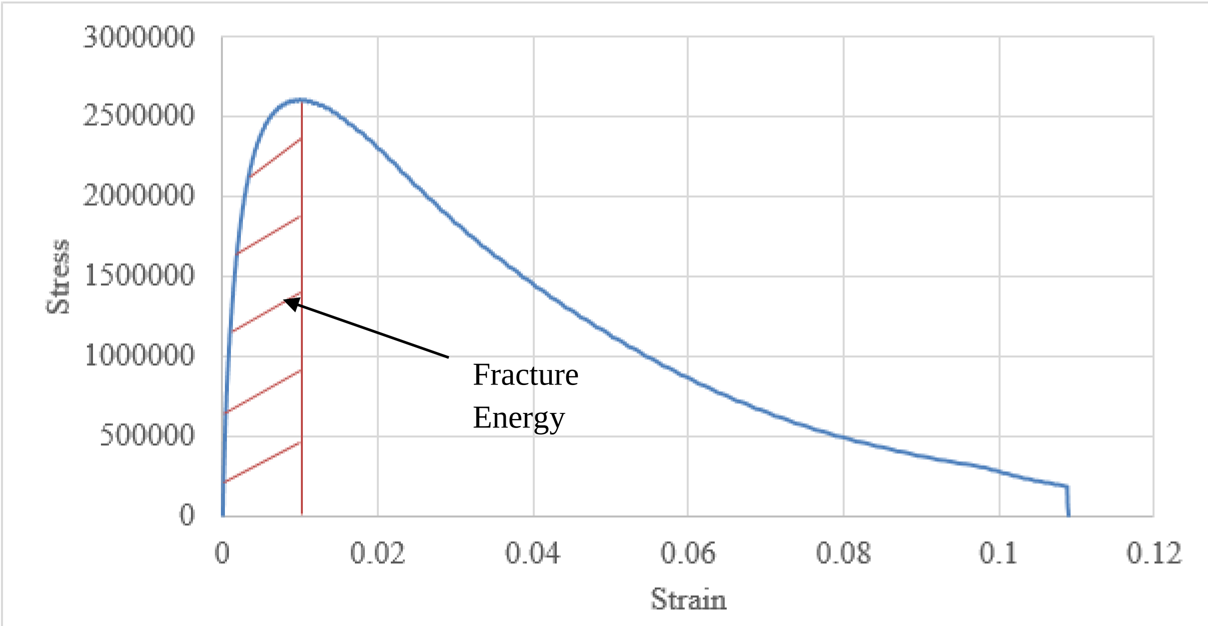


Figure 4-14: Stress vs. Strain Diagram for Determining Fracture Energy

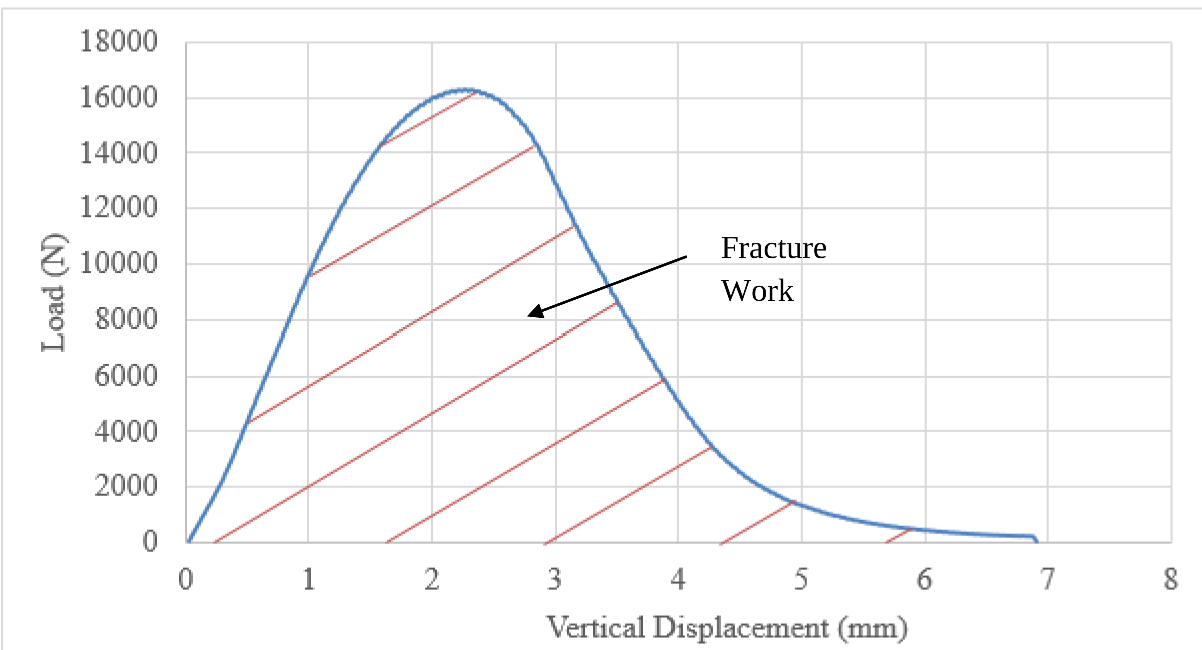


Figure 4-15: Load vs. Frame Displacement for Determining Fracture Work

4.2.5.1 IDT Test Analysis Technique

The radial sensors displacements are first plotted against time to check the validity of the sensor data. There should be a steady decrease in the vertical sensors (4&5) and a steady increase in the horizontal sensors (7&9). An example of this diagram is shown in Figure 4-16. If a sensors shows that it did not obtain data or was inaccurate that sensor will be removed from the data analysis.

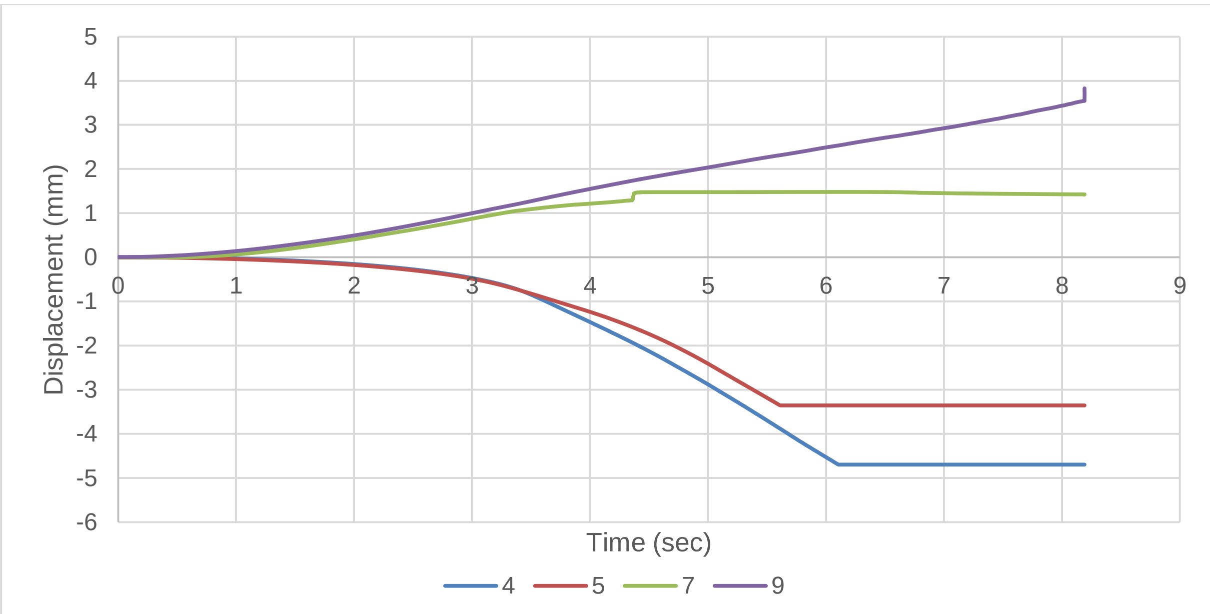


Figure 4-16: Sensor Displacements

At this time several computations need to be made to complete calculations for fracture energy, fracture work density, and IDT strength. First the two vertical and horizontal sensors are averaged to give average vertical and horizontal displacement. Next Poisson's ratio is found using the displacements and several constants shown in Equation 4-10. Strain and stress are calculated next using Equations 4-11 and 4-12. The maximum stress is found from all the raw data which is the IDT strength.

$$\nu = -(a_1 * U(t)+V(t))/(a_2 * U(t)+a_3 * V(t)) \quad (4\text{-}10)$$

Where,

ν = Poisson ratio

a1, a2, and a3 = constants

U(t) = average horizontal displacement (mm)

V(t) = average vertical displacement (mm)

t = time (s)

$$\varepsilon = U(t) * (\gamma_1+\gamma_1 * \nu)/(\gamma_1+\gamma_1 * \nu) \quad (4\text{-}11)$$

Where,

ε = strain

$\gamma_1, \gamma_2, \gamma_3, \gamma_4$ = constants

$$\sigma = (2 * P)/(\pi * t * r) \quad (4\text{-}12)$$

Where,

σ = stress

P = applied load (N)

t = thickness of specimen (m)

r = radius of sample (m)

Equation 4-13 is used to determine the fracture energy. Equation 4-14 is used to determine the fracture work density of the sample which divides the total area under the load-displacement curve by the volume of the sample.

$$FE = \sum_{i=1}^{n}\sum_{j=1}^{n}\left|(\sigma_i + \sigma_j) * (\varepsilon_i + \varepsilon_j) + 0.5\right| \tag{4-13}$$

Where,

FE = fracture energy (Pa)

σ_i = Stress at t_i

σ_j = Stress at t_j

ε_i = Stress at t_i

ε_j = Stress at t_j

$$FWD = \left[\sum_{i=1}^{n}\sum_{j=1}^{n}\left|(P_i + P_j) * (\delta_i + \delta_j) + 0.5\right|\right]/V \tag{4-14}$$

Where,

FWD = fracture work density

P_i = Load at t_i

P_j = Load at t_j

δ_i = Frame LVDT displacement at t_i (mm)

δ_j = Frame LVDT displacement at t_j (mm)

V = volume of the sample tested

4.2.5.2 IDT Fatigue Results

Results from IDT fatigue testing are shown in this section. Three replicates of each mix design were tested and the averages were reported. The IDT strength (Pa), Fracture Work Density (Pa), and Fracture Energy (Pa) are shown in Table 4-14 for all four mix designs. It can be seen the

HP70 C mixture has both the highest IDT strength and fracture work density while the HP76 C has the highest fracture energy.

Table 4-14: IDT Fatigue Results

	IDT Strength (Pa)	Fracture Work (Pa)	Fracture Energy (Pa)
HP70 C	3155475	167110	20488
HP70 M	2904463	146615	20383
HP76 C	2469364	141041	22846
HP76 M	2532960	142121	17893

Comparison of the fatigue fracture energy results for 4.75mm and typical 12.5mm mixtures that were recently tested at WCAT laboratory are shown in Figure 4-17. For 4.75mm mixtures, the PG70 binder mixtures have similar fatigue fracture energy while the PG76 coarse graded mix has higher fatigue fracture energy than the medium graded mix. When comparing the different NMAS mixtures the PG70 4.75mm mixtures are both larger than the 12.5mm mix by a large amount. The PG76 coarse mix is within standard deviation of the 12.5mm mixture while the medium graded is much lower. Note these 12.5mm mixtures were not specifically created for this study and were only used for general comparison. Different aggregate types and binder sources could also contribute to their difference in addition to the mix design differences. In addition, because the low temperature PG grade of the control 12.5mm mixtures (-22) are different from the one used for 4.75mm mixtures (-28), no comparison was made for low temperature IDT properties.

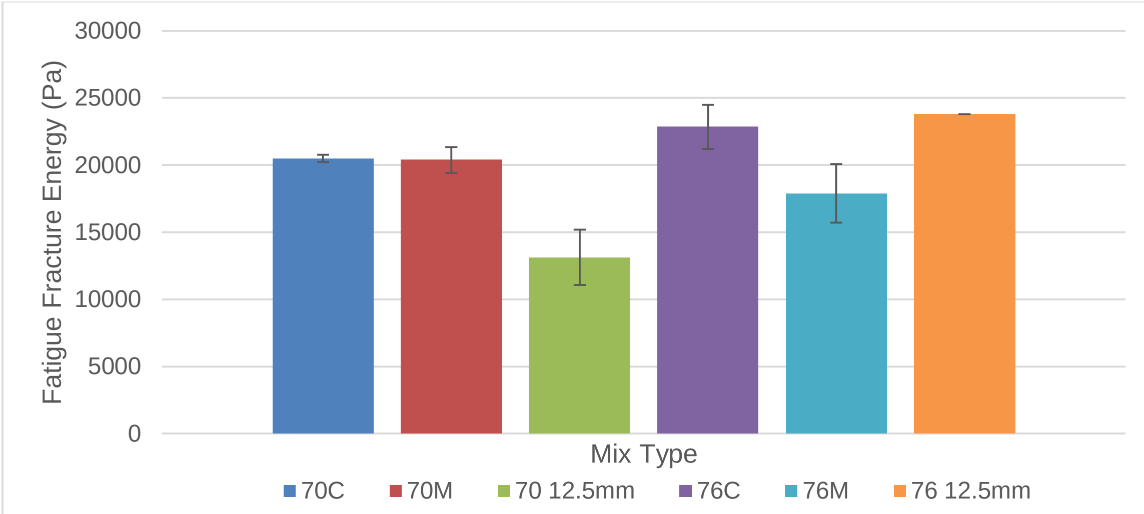


Figure 4-17: Fatigue Fracture Energy Comparison

Fracture work density results for 4.75mm and 12.5mm mixtures are shown in Figure 4-18. For PG70 mixtures, both coarse and medium graded 4.75mm mixes have higher fracture work than the 12.5mm control mixes, indicating their good fatigue resistance. For PG76 mixtures, both coarse and medium graded 4.75mm mixes have similar but lower fatigue fracture work than the control 12.5mm mixes.

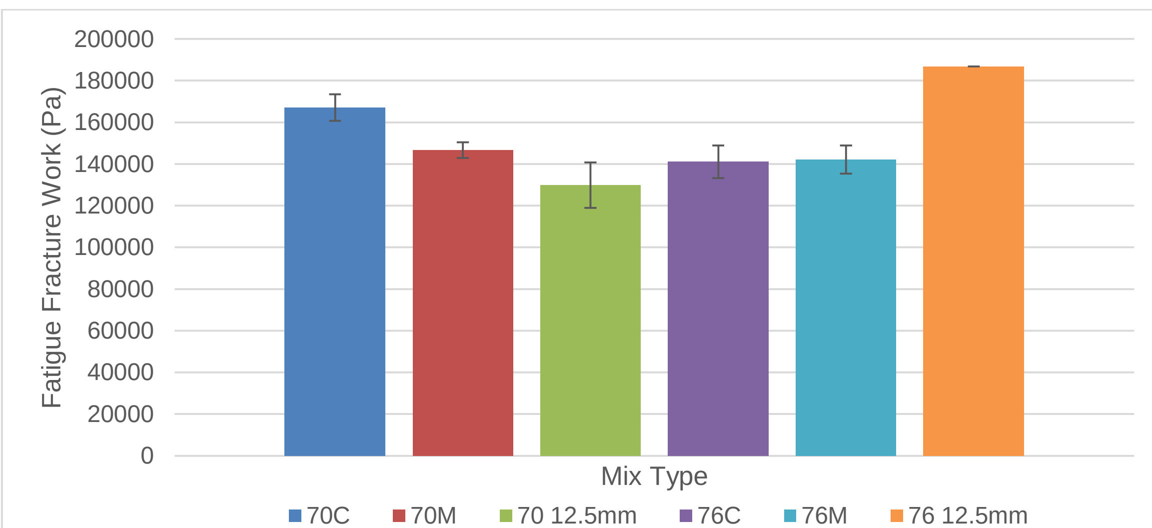


Figure 4-18: Fatigue Fracture Work Density Comparison

IDT strength results for 4.75mm and 12.5mm mixtures are shown in Figure 4-19. The IDT strength results shown are similar to that of the fracture work density results. The PG70 coarse mix is larger than the medium mix while the PG76 mixtures are very similar. The coarse PG70 mix has a very large standard deviation. When comparing the different NMAS mixtures the PG70 4.75mm mixtures are both larger than the 12.5mm mix, the coarse more than the medium. Both PG76 mixtures are well below the 12.5mm mixture.

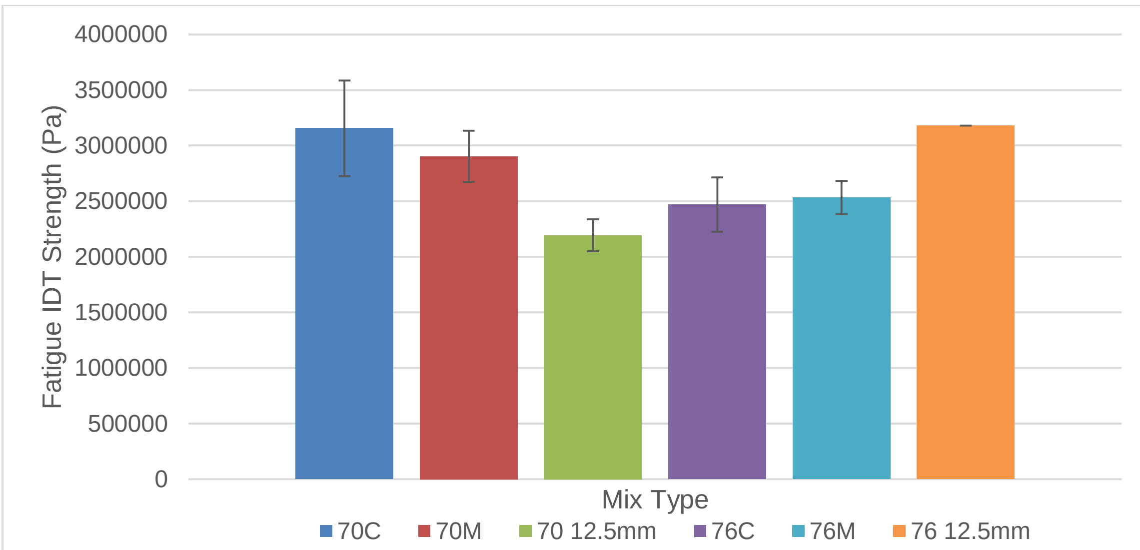


Figure 4-19: Fatigue IDT Strength Comparison

4.2.5.3 IDT Thermal Results

Results from IDT thermal testing are shown in this section. Three replicates of each mix design were tested and the averages were reported. The IDT strength (Pa), Fracture Work Density (Pa), and Fracture Energy (Pa) are shown in Table 4-15 for all four mix designs. It can be seen the HP70 C mixture has both the highest IDT strength and fracture work density while the HP76 M has the highest fracture energy.

Table 4-15: IDT Thermal Results

	IDT Strength (Pa)	Fracture Work (Pa)	Fracture Energy (Pa)
HP70 C	6452877	1155934	60411
HP70 M	6272862	1006279	56097
HP76 C	6023367	1131883	59048
HP76 M	6252781	989314	68419

Fracture energy results for the four 4.75mm mixtures in low temperature are shown in Figure 4-20. No 12.5mm results will be compared in this section for lack of same low binder type results. The PG70 binder mixtures show the coarse gradation higher than the medium while the PG76 medium mix is higher than the coarse gradation. Overall the results are very similar with the medium graded PG76 mixture standing out with the highest results.

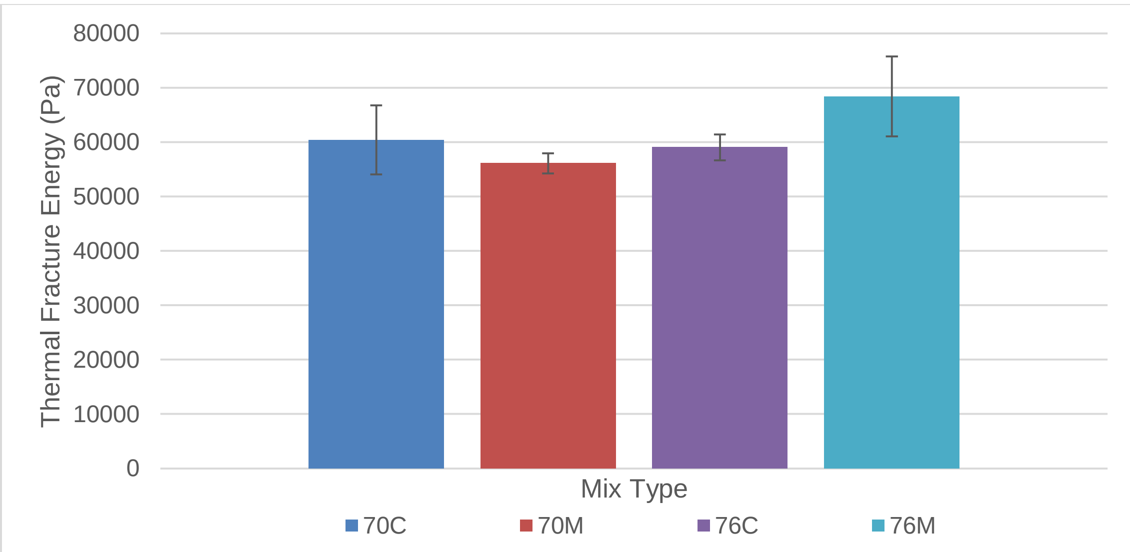


Figure 4-20: Thermal Fracture Energy Comparison

Fracture work density results for the four 4.75mm in low temperature are shown in Figure 4-21. Both coarse mixtures are larger than their medium mix counterpart varying from the fracture energy results. The PG70 coarse graded mixture has the highest average overall followed closely by the PG76 coarse graded mixture.

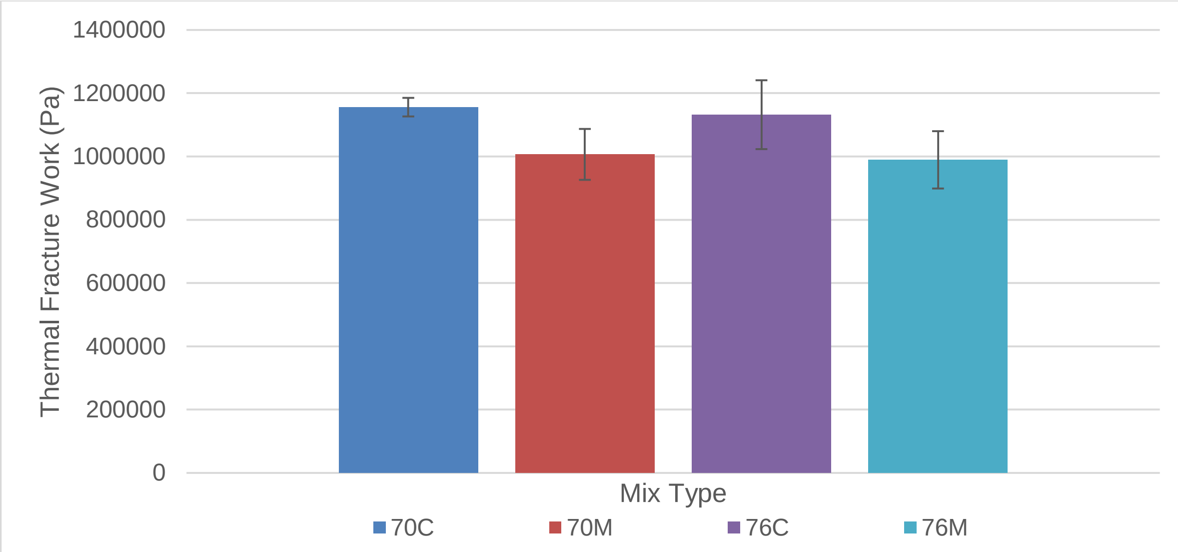


Figure 4-21: Thermal Fracture Work Comparison

IDT strength results for the four 4.75mm in low temperature are shown in Figure 4-22. The coarse graded PG70 mixture has the largest average by far but is well within a standard deviation of both medium graded mixtures. The PG76 coarse mixture shows a very low result from this test.

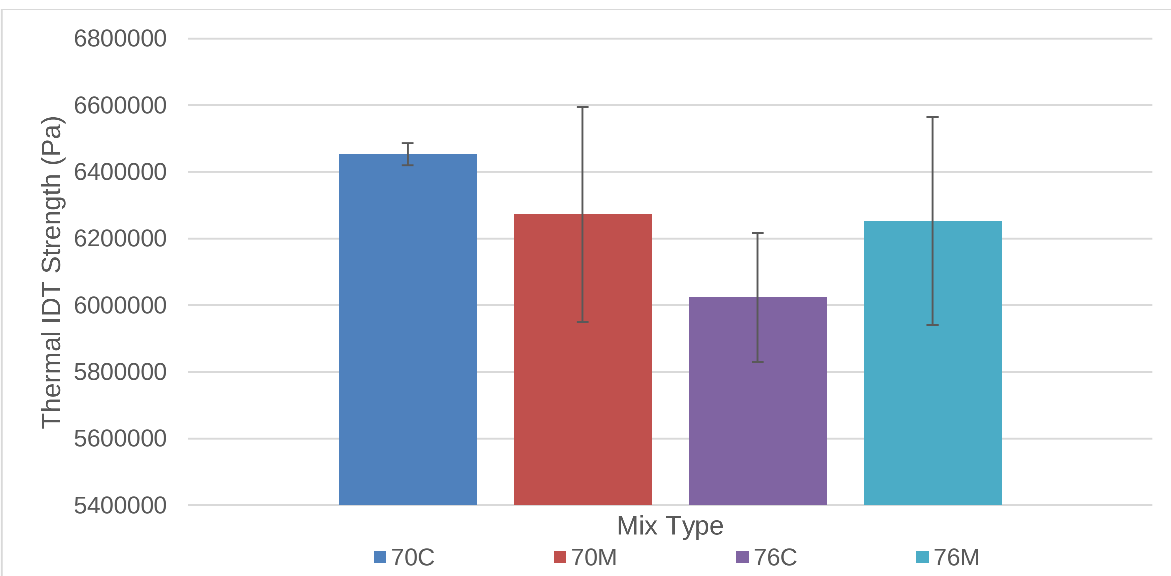


Figure 4-22: Thermal IDT Strength Comparison

4.2.6 Summary

In conducting laboratory testing many insights were gained into the validity of using a 4.75mm mixture. Among the developed four mix designs with different binder and gradation types, the coarse and medium graded PG76 mixtures perform the best in rutting and moisture behaviors as determined from the Hamburg Wheel-track test which is comparable to 12.5mm mixtures. Although the coarse and medium graded PG70 mixtures had relatively larger rutting depth compared to the 12.5mm mixture both were still valid in terms of WSDOT standards and therefore can still be considered. The coarse PG70 mix did show a significant improvement over the medium PG70 mix showing it would be the better choice when using a PG70 binder.

Based on IDT fracture test at room temperature, it was found that the cracking resistance of 4.75mm mixtures were comparable to conventional 12.5mm mixtures and not significantly less crack resistant. The coarser 4.75mm mixtures had better cracking resistance using fracture work density. Fracture work density is used because it has been shown to be a better indicator of field fatigue performance than fracture energy (Wen, 2012). As summarized from the literature review and survey, cracking resistance is one of the largest concerns with this small NMAS mixture. In addition, the main reason for using this mixture is for its cost effectiveness. From these two main points it can then be concluded while all mixtures seem to be viable choices, the coarse-graded PG70-28 mixture is the best overall. It is presumably more cost effective than the PG76-28 binder with the higher grade being expected to be higher in cost. It is also expected to have the best cracking resistance with reasonable rutting results. It fits both main criteria and therefore seems to be the best option as a high performance thin overlay mix.

CHAPTER 5: LIFE CYCLE COST ANALYSIS

Life cycle cost analysis (LCCA) is defined by AASHTO (1986) as a technique founded on economic analysis principles which enables the evaluation of overall long-term economic efficiency between competing alternative investments. It consequently has important applications in pavement design and management. When determining the overall cost of pavement management activities there are multiple factors to consider. Some of these factors include material costs, construction costs, maintenance costs, and design costs. This overall cost is then used along with service life and an interest rate to compute the life cycle cost of certain pavement treatments.

A life cycle cost analysis was conducted for WSDOT to compare 4.75mm NMAS thin overlays to chip seal and 0.15' HMA inlay, two treatments that are typically used by WSDOT as pavement preservation strategies. In this paper, two scenarios of 4.75mm thin overlay application were considered for the LCCA analysis:

1. 4.75mm thin overlay is applied to existing pavement with no milling to remove the existing wearing course. Such application is comparable to a chip seal treatment assuming fair to good existing pavement condition.
2. 4.75mm thin overlay is used as an inlay. Only the main traffic lane is milled and a 4.75mm thin overlay is applied to match the elevation of the existing shoulder. This application is comparable to the typical 0.15' HMA inlay. Minor rutting or cracking distresses to the depth of milling thickness in existing pavements can be expected in this application.

Washington State is separated into six regions including Olympic, Northwest, Southwest, North Central, South Central, and Eastern as shown in Figure 5-1. For the purposes of this life cycle cost analysis these regions were consolidated into two large areas, Eastside and Westside, mainly due to their similarities in project costs and surface treatment methods. Pavement performance is much different in the Eastside than the Westside. The Cascade Mountains split the state with the Westside receiving more rain and having a milder climate. The Eastside has more localized climate conditions with the areas being warmer and dryer while others being colder and wetter. Also areas of significant studded tire usage, mainly on the Eastside, can effect pavement life. These results in generally longer pavement life on the Westside than the Eastside. The Eastside includes North Central, South Central, and Eastern regions while the Westside includes Olympic, Northwest, and Southwest regions.

Figure 5-1: Washington State region separation.

5.1 GENERAL PROCEDURE FOR LIFE CYCLE COST ANALYSIS

The calculation of life cycle cost for a specific treatment generally have two steps: (1) determine the total cost of the treatment per lane-mile; and (2) determine the equivalent uniform annualized costs (EUAC) of the treatment. The EUAC is the cost per year of owning an asset over its entire lifespan. In construction of pavements the EUAC is practically used to determine the life cycle cost of a specific paving option. It is calculated by annualizing the total initial construction cost over the service life of the pavement considering a special discount rate.

To find the total construction cost per lane-mile for each treatment, WSDOT provided project bid tabulations for chip seals and 0.15’ HMA inlay projects in 2012 from each region. From the bid tabulations, weighted average costs for each treatment type and category were determined for the generalized Eastside and Westside of the state, respectively, to find the final cost per lane mile in each region. For 0.15’ HMA inlay costs included preparation, grading/repair, surfacing, paving, erosion control, traffic items, and other. For chip seals costs included preparation, grading, drainage, surfacing, liquid asphalt, BST (aggregate surfacing), HMA, erosion control, traffic, and other. The weighted average cost per lane mile for each region can be calculated using Equation 5-1.

$$W = \sum_{i=1}^{n} \left(\frac{C_i * PL_i}{TL}\right) \quad (5\text{-}1)$$

Where,

W = Weighted average cost in the region

n = Number of projects in the region

C_i = Projects i’s total cost per lane mile

PL_i = Projects i’s lane-miles

TL = Total lane-miles of all projects in the region

The final weighted average cost for chip seals in the Westside were \$61,421 and in the Eastside were \$33,741. The final weighted average cost for 0.15’ HMA inlays in the Westside were \$150,107 and in the Eastside were \$129,325. The weighted average value for each of these treatments categories is shown in Table 5-1 and 5-2. As seen, these costs differed greatly with the cost of Westside usually higher than Eastside. For both 0.15' HMA Inlay and chip seals, the costs of preparation, erosion control, and engineering have account for the main differences between east and west side. The determined weighted average costs from the existing projects will be used to calculate the equivalent uniform annualized costs (EUAC).

Table 5-1: 0.15' HMA Inlay Cost Tabulation per lane-mile

Treatment	**Eastside**	**Westside**	**Difference**
Preparation	$ 4,926	$ 8,609	54%
Grading/Repair	$ 16,236	$ 17,031	5%
Surfacing	$ 63	$ 62	1%
Paving	$ 72,933	$ 65,662	10%
Erosion Control	$ 107	$ 633	142%
Traffic	$ 17,650	$ 24,964	34%
Other Items	$ 280	$ 972	111%
Engineering	$ 17,131	$ 32,173	61%
Total	**$ 129,325**	**$150,107**	**15%**

Table 5-2: Chip Seal Initial Cost Tabulation per lane-mile

Treatment	**Eastside**	**Westside**	**Difference**
Preparation (Incl. Mobilization)	$ 2,410	$ 4,211	54%
Grading	$ 355	$ 2,244	145%
Drainage	$ -	$ -	0%
Surfacing	$ 168	$ 182	8%
Liquid Asphalt	$ 13,642	$ 17,569	25%
Bituminous Surface Treatment	$ 4,505	$ 8,362	60%
Hot Mix Asphalt	$ 5,887	$ 7,188	20%
Erosion Control and Planting	$ 29	$ 229	155%
Traffic	$ 4,008	$ 8,841	75%
Other Items	$ 98	$ 1,006	165%
Engineering	$ 2,641	$ 11,589	126%
Total	**$ 33,741**	**$ 61,421**	**58%**

Creating the 4.75mm NMAS thin overlay estimations of costs per lane-mile was more complicated because WSDOT does not have historical cost data for this type of overlay. Therefore the cost estimation was based on the 0.15' HMA Inlay values but with some modifications as listed below.

1. Paving Cost: The differences of paving cost between 4.75mm mixes and 0.15' HMA inlay include two main parts. The first is the material unit costs. Based on practical experience and consulting WSDOT engineers, an average unit cost of $90/ton was used for 4.75mm mixes and compared to conventional 12.5mm mixes being $67.62/ton, a 28% unit cost increase was applied. This increase considers the increase of higher binder content due to finer aggregate gradation and the mix being a special mix not used regularly by contractors in the area. The second factor is the material usage per lane mile. Since the layer thickness is decreasing from 2 inches to 0.75 inches a reduction factor of 0.375 can be used. Multiplying the material unit cost of each project by these differences gives the final paving costs.
2. Milling Cost: Milling cost counted for another major cost differences among treatments. For a 4.75mm inlay the milling will only be 0.75" meaning a reduction in milling costs. Associated with the milling thickness reduction, other costs will be reduced such as cost of transporting materials and labor work. For this reason a reduction factor based solely on thickness could not be used but instead a reduction factor of 40% was used as determined with the help of WSDOT engineers. For 4.75mm overlay on the other hand milling cost were completely eliminated because no milling is to occur.

After modification to all projects the total costs and weighted averages were calculated the same way as the previous treatments. Table 5-3 shows the Eastside and Westside itemized weighted average costs for 4.75mm inlay and overlay. As seen the main differences between

Westside and Eastside include preparation, traffic, and engineering. For all treatments the total cost per lane-mile including engineering costs are shown in Figure 5-2.

Table 5-3: 4.75mm Inlay and Overlay Cost Tabulation per lane-mile

	4.75mm Inlay			**4.75mm Overlay**		
	Eastside	**Westside**	**Diff**	**Eastside**	**Westside**	**Diff**
Preparation	$ 4,926	$ 8,609	54%	$ 4,926	$ 8,609	54%
Grading/Repair	$ 12,652	$ 12,563	1%	$ 7,276	$ 5,861	22%
Surfacing	$ 63	$ 62	1%	$ 63	$ 62	1%
Paving	$ 38,245	$ 35,760	7%	$ 38,245	$ 35,760	7%
Erosion Control	$ 107	$ 633	142%	$ 107	$ 633	142%
Traffic	$ 17,650	$ 24,964	34%	$ 17,650	$ 24,964	34%
Other Items	$ 280	$ 972	111%	$ 280	$ 972	111%
Engineering	$ 17,131	$ 32,173	61%	$ 17,131	$ 32,173	61%
Total	**$ 91,053**	**$ 115,738**	**24%**	**$ 85,677**	**$109,036**	**24%**

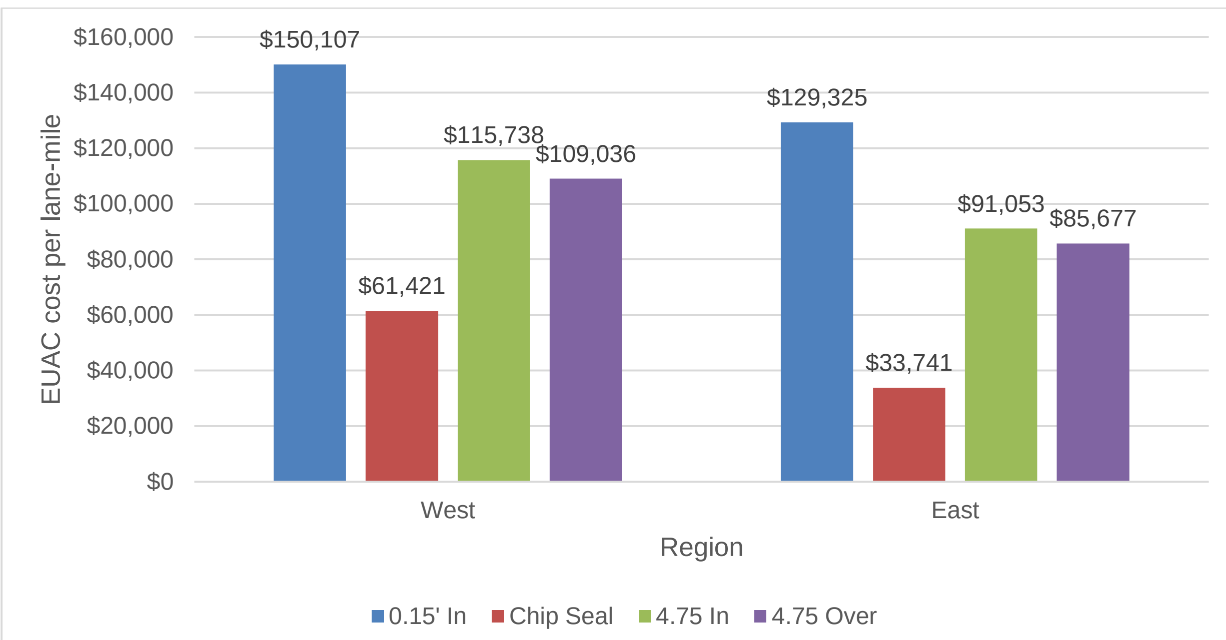


Figure 5-2: Total Cost per lane-mile (including engineering and taxes)

The service life for each treatment was then determined so EUAC could be calculated. The average 0.15' HMA inlay service life was given by WSDOT based on historical data, which are 11 years in the East and 17 years in the West. The average service life of chip seal was determined from the literature review and WSDOT, as 6 years for the Eastside and 8 years for the Westside. Based on literature review results (8-12 years' service life) and referencing the different HMA overlay/inlay service life for the Westside and Eastside, it was determined that 4.75mm overlay may have longer life on the Westside (12 years) than in the Eastside (8 years). These service lives are shown in Table 5-4.

Table 5-4: Service Life

Service Life	Western	Eastern
0.15' HMA Inlay	17	11
Chip Seal	8	6
4.75mm In	12	8
4.75mm Over	12	8

Finally the EUAC was determined for each treatment. The equivalent uniform annualized costs were determined by annualizing the initial cost over the service life of the treatment using a discount rate of 4 percent as shown in Equation 5-2. Figure 5-3 summarizes the EUAC for each treatment for both the Eastside and Westside.

$$EUAC = (0.04 * 1.04^{n}) / ((1.04^{n}) - 1) * W \qquad (5\text{-}2)$$

Where,

EUAC = equivalent uniform annualized cost

n = years of service life

W = weighted average cost

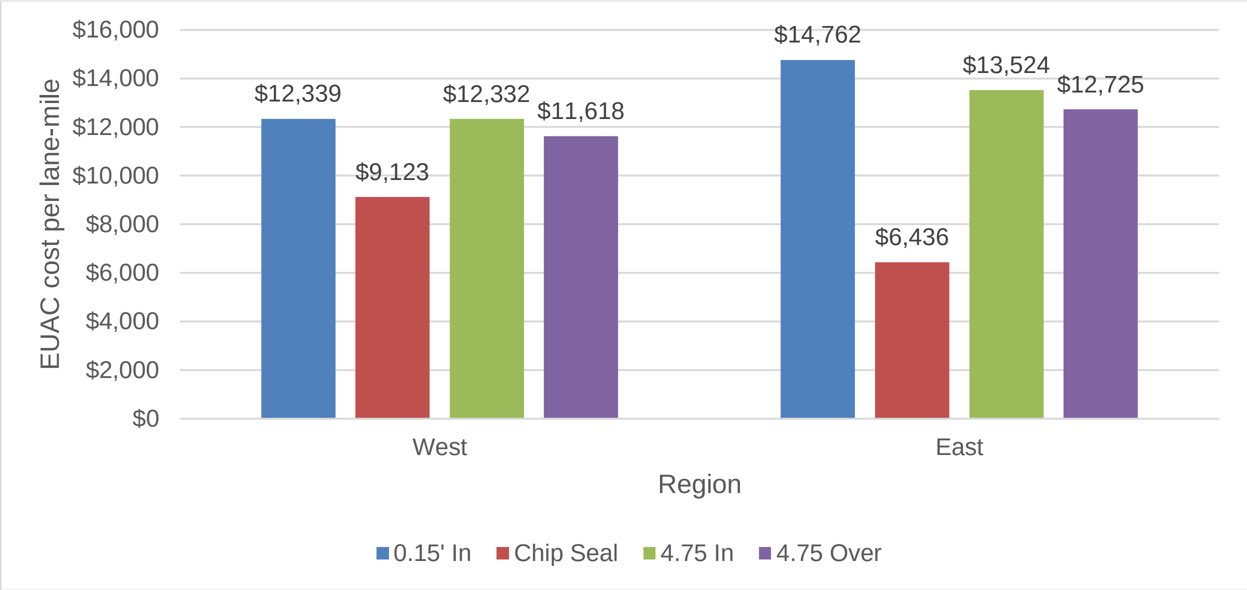


Figure 5-3: EUAC per lane-mile (including engineering and taxes)

5.2 DISCUSSION

Based on the presented life cycle cost analysis, it can be seen that 4.75mm NMAS thin overlays can be cost effective in the right situation. On the Eastside 4.75mm NMAS inlays showed a much lower life cycle costs than conventional 0.15' HMA inlays. There is potential for 4.75mm inlays to replace 0.15' HMA inlays in certain situations. However, because chip seal has been used very economically in the Eastside, 4.75mm overlay may not be competitive in terms of life cycle cost.

In the Westside of the state, both 4.75mm thin inlay and overlay had similar life cycle costs to that of the 0.15' HMA inlay. However chip seal was again a much more attractive maintenance treatment than a 4.75mm HMA overlay. In general, 4.75mm thin overlay pavement preservation strategy will be less cost-effective than the chip seal when both strategies can properly address the existing pavement conditions. Additional research will be needed either to reduce material and construction costs or extend the service life of 4.75mm thin overlay in order to make it a more cost attractive alternative to chip seal in both regions.

5.3 SENSITIVITY ANALYSIS

A sensitivity analysis was also run on the annualized costs of each 4.75mm treatment to determine the service life needed to be a viable alternative to the corresponding treatment. Using Equation 4-2, EUAC is fixed using 0.15’ HMA inlay and chip seals EUAC in each region. Then using the respective 4.75mm mixture as an input we can find the service life needed (n). 4.75mm inlays are compared to 0.15’ HMA inlay and 4.75mm overlay is compared to chip seal. As seen in Table 5-5, for 4.75mm inlay to be cost competitive to conventional 0.15’ HMA inlay, a minimum service life of 12 years are needed if used in the Westside but 7.2 years if used in the Eastside of the Washington State. Though they are very different both values are within what can possibly be expected from 4.75mm thin overlays. For 4.75mm overlays to be cost competitive to chip seal surface treatment, a minimum service life of 16.2 years are needed if used in the Westside but 19.3 years if used in the Eastside. Such high service life requirement for 4.75mm NMAS thin overlay will be very hard to achieve according to literature data. Therefore, 4.75mm NMAS thin overlays may not be a viable alternative to a chip seal.

Table 5-5: Estimated Service Lives for 4.75mm Thin Overlay to be Cost Competitive

Treatment to be compared with 4.75 mm NMAS thin overlay	Needed 4.75 mm thin overlay life	
	Westside (years)	Eastside (years)
0.15’ HMA inlay	12.0	7.2
Chip Seal	16.2	19.3

5.4 SUMMARY FINDINGS

4.75mm NMAS thin overlays provide many benefits as a pavement preservation strategy. As concluded from the literature summary and agency survey results, the economic benefits have become the most important reason of its increasing applications throughout the country. Thin overlays have longer service lives than most other preventive maintenance treatments. Though they have a higher initial cost than some treatments, with the long service life the annualized cost of a thin overlay is typically lower. It also gives many performance benefits including a smooth riding surface, low permeability, and life extension.

Based on the LCCA analysis for Washington State, it is suggested that the 4.75mm NMAS thin overlay is a promising and cost effective preventive maintenance treatment when compared to a 0.15' HMA inlay. In the Westside 4.75mm inlay needs to last 12 years to be cost effective while on the Eastside 4.75mm inlay needs to last 7.2 years to be cost effective. Both of these service lives are below or in the range of service lives expected from the literature review. 4.75mm mixtures can be used as a cost effective alternative to 0.15' HMA inlay but not cost effectively as an alternative for a chip seal.

It is worth noting that the provided LCCA analysis is mainly based on historical construction data in Washington and is used to provide cost estimation for the future application. Construction of test strips is recommended to validate the cost estimations presented in this paper and should provide ideas to help improve the cost effectiveness of 4.75mm thin overlay for Washington State. Future studies are also recommended to improve the design and application method of the thin overlay so that to extend its service life and reduce its overall life cycle cost.

CHAPTER 6: DRAFT SPECIAL PROVISION

A draft Special Provision for 4.75mm NMAS thin overlay design and construction is provided herein based on the findings from literature, agency survey, and past experience for WSDOT conventional HMA overlay. It is strongly recommended that test sections be constructed to gain experience for WSDOT and the details of the provision can be revised.

6.1 PREVENTIVE MAINTENANCE 4.75mm NMAS THIN OVERLAY

6.1.1 Description

The work shall consist of producing and placing a 4.75mm NMAS thin overlay by milling and sweeping the previous surface followed by placing a tack coat and placing the new surface course to the thickness specified on the plans. All work shall be in accordance with WSDOT standard specifications for HMA with the following additions.

6.1.2 Materials

6.1.2.1 Aggregate

Mineral aggregate shall be according to section 9-03.8 except for the following exceptions.

(a) The single percentage of aggregate passing each required sieve shall be within the following limits:

Sieve Size	Mix Design (% passing)		Field Tolerance
	Min	Max	
3/8 inch	95	100	± 6%
No. 4	90	100	± 5%
No. 8			± 4%
No. 16	30	55	± 4%
No. 30			± 3%
No. 50			± 3%
No. 100			± 3%
No. 200	6	13	± 2%

(b) The fine aggregate angularity value shall be as follows:

Traffic Level	FAA Requirement
< 0.3 million ESALs	≥ 40
> 0.3 million ESALs	≥ 43

(c) The natural sand in the mixture shall be limited to 10 – 15% of the aggregate blend.

6.1.2.2 Asphalt Binder

Polymer modified binders should be used if thin overlay is placed on roadways with high traffic levels.

6.1.3 Mix Design Criteria

The following design criteria should conform to the limits presented.

Design ESALs (Millions)	N_{des}	% Air Voids	VMA	VFA	Min. Film Thickness (microns)	D:B Ratio	V_{be}
<0.3	50	4 - 6	16 - 18	75 - 80	6	1.0 – 2.0	12.0 – 15.0
0.3 - 3.0	75	4 - 6	16 - 18	75 - 80	6	1.0 – 2.0	11.5 – 13.5
>3.0	100	4 - 6	16 - 18	75 - 80	6	1.0 – 2.0	11.5 – 13.5

6.1.4 Construction

Mixing should take place at a batch or continuous drum plant. Place and compact the preventive maintenance 4.75mm NMAS thin overlay in a manner to provide the desired in-place compaction, and to produce a smooth riding surface.

6.1.4.1 Surface Preparation

Proper surface preparation should be followed prior to placement of the 4.75mm NMAS thin overlay. Proper preparation includes items such as milling, and a clean broomed/dry surface.

6.1.4.2 Tack Coat

Apply approved emulsified asphalt to the surface on which the HMA thin overlay will be placed after surface preparations have been completed.

6.1.4.3 Temperature

The temperature of the surface shall be at least 55°F for paving to be conducted.

6.1.4.4 Spreading, Finishing, and Compaction

Spread the thin overlay to ensure a minimum lift thickness of 3/4 inch. Hauling equipment and paver should be of a type normally used for the transport and placement of dense grade asphalt hot mix. Vibratory and pneumatic tired rolling should not be used rather using static rollers.

6.1.4.5 Potential Applications

4.75mm NMAS thin overlays should only be used when the distresses are in the limits shown in the table below.

	Rutting	Cracking				Raveling
		Longitudinal (wheelpath)	Longitudinal (out of wheelpath)	Transverse	Fatigue	
Low	X	X	X	X	X	X
Medium	X		X	X		X
High						X

CHAPTER 7: CONCLUSIONS AND FUTURE WORK

7.1 SUMMARY AND CONCLUSIONS

4.75mm thin overlays offer a viable alternative in preventive maintenance according to the literature review. Thin overlays should be placed on lightly cracked surfaces where a tack coat is applied and/or the existing pavement is milled. The service life of thin overlays is generally higher than other preventive maintenance treatments lasting 8 – 12 years. Thin overlays are suggested by literature to be cost effective than other surface treatments even with a higher upfront cost because of its longer service life. Research suggests that thin overlays can be applied to any traffic level including high volume roadways. The use of screening materials and RAP in 4.75mm thin overlay mixtures help reduce costs. The major concerns found included reflective cracking, friction, and heat loss before density is achieved.

A survey was taken of government agencies throughout the United States and Canada. The main reason of using thin overlay was its economic benefits (88.9% of respondents), followed by its usage of screening material (28% of respondents). 4.75mm NMAS thin overlays were reported to be applied at traffic levels ranging from less than 1,000 ADT to all levels of traffic. Most agencies (76.9%) reported good performance for 4.75mm thin overlays and only one agency reported poor performance. The two main distresses were cracking and delamination and were reported by 57.1% and 28.6% of agencies respectively. Many agencies reported negligible rutting while the highest reported rut depth was 1/4 inch. 81.8% agencies indicated that their 4.75mm NMAS thin overlays had similar or better rutting results compared to typical HMA overlays. Although friction was reported in the literature as one of the main performance concerns for thin overlays, the survey results indicated that for most states the friction levels were the same as typical overlays with 16.7% reporting worse but acceptable friction. Studded tire damage was found to have no worse effect on 4.75mm thin overlays than conventional mixtures. The survey also noted reflective cracking as a concern for 71.4% of agencies ranging mostly from moderate to severe. This survey results confirmed the literature review findings that reflective cracking was a potential problem. Thermal cracking and stripping were mainly reported as not a concern. Almost half of reporting agencies used warm mix with good results. These results confirm the literature review findings and give a detailed look at what experiences agencies have with 4.75mm thin overlays.

In this study, four high performance 4.75mm NMAS thin overlay mixtures were developed for high traffic volume roads. They were coarse graded and medium graded PG70-28 mixtures and PG76-28 mixtures. The mix design was based on packing theory to consider aggregate interlock and followed the volumetric criteria recommended by Superpave specification. These mixtures were evaluated for rutting and moisture resistance using Hamburg Wheel Tracking test and cracking resistance using IDT test at different temperatures. It was found that the developed coarse graded PG70-28 gave the most consistent IDT testing results while the PG76-28 mixtures had the best rutting results. In general the coarse mixtures showed the best results throughout performance testing. The PG70-28 coarse graded mixture is the most viable choice because it has the best crack resistance as well as being more cost effective than PG76-28 mixes.

Based on historical data and WSDOT practical experience, the presented life cycle cost analysis suggested that 4.75mm NMAS thin overlays can be cost effective in the right situation. On the Eastside 4.75mm NMAS inlays showed a much lower life cycle costs than conventional 0.15’ HMA inlays. There is potential for 4.75mm inlays to replace 0.15’ HMA inlays in certain situations. However, because chip seal has been used very economically in the Eastside, 4.75mm overlay may not be competitive in terms of life cycle cost. On the Westside of the state, both

4.75mm thin inlay and overlay had similar costs to that of the 0.15' HMA inlay. However chip seal was again a much more attractive maintenance treatment than a 4.75mm HMA overlay. In general, 4.75mm thin overlay pavement preservation strategy will be less cost-effective than the chip seal when both strategies can properly address the existing pavement conditions. In the sensitivity analysis the Westside 4.75mm inlay needs to last 12 years to be cost effective while on the Eastside 4.75mm inlay needs to last 7.2 years to be cost effective. Both of these service lives are below or in the range of service lives expected from the literature review. 4.75mm mixtures can be used as a cost effective alternative to 0.15' HMA inlay but not cost effectively as an alternative for a chip seal.

A draft special provision for the design and construction of 4.75mm NMAS thin overlays was developed for WSDOT to aid in the construction of test sections. Aggregate properties, mix design criteria, and construction practices were suggested. Also potential applications were suggested by noting limits on distresses.

7.2 RECOMMENDED FUTURE WORK

Some further work is recommended to fully conclude this mix as a viable preventive maintenance alternative for WSDOT. A test strip needs to be constructed to help determine the actual paving costs and verify the life cycle cost effectiveness of the 4.75mm NMAS thin overlay. The test strip will also help identify potential construction and performance concerns under Washington climate and traffic conditions and provide suggestions on mix design improvement. Particularly, the effect of studded tires on this special mix should be evaluated carefully based on field performance. After the test strip a reevaluation of the draft special provision should be conducted to establish the recommended practice and specifications for the 4.75mm NMAS thin overlay to be used as a cost effective pavement preventive maintenance option for WSDOT.

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APPENDIX A

Survey Question 1

Has your agency used or considered using 4.75mm NMAS thin overlays as a preventive maintenance technique?

Table A-1 presents agency responses and includes summary statistics with the answers.

Table A-1: Summary of survey question 1

Yes	Considering	No
16/38 (42.1%)	8/38 (21.1%)	14/38 (36.8%)

Survey Question 2

What are the main reasons for using 4.75mm NMAS thin overlays for your agency? (select all that apply)

Table A-2 presents agency responses and includes summary statistics with the answers. Individual agency comments associated with this question are also included below.

Table A-2: Summary of survey question 2

Use of screening materials	Performance benefits	Economical preservation strategy	Other (please specify)
5/18 (27.8%)	1/18 (5.6%)	16/18 (88.9%)	4/18 (22.2%)

Agency Comments:

- use for patching and rut filling
- Experimental Usage
- mix is nonpermeable and in Alabama has no density requirement
- Leveling to help eliminate raveling and as a seal course

Survey Question 3

How many lane-miles annually does your agency pave with 4.75mm NMAS thin overlays?

Below are individual agency comments associated with this question.

Agency Comments:

- Only have done 4 or 5 lane miles to date approximately
- 25-35

- 5-10 miles
- less than 100
- Varies annually. Last 3 years have been 210 (2010), 79 (2011) and 24 (2012) miles annually.
- 150
- About 10 (or less)
- approximately 50-100 lane miles
- We have only done 5 projects for about 40 lane miles.
- Very limited use on a few local off system rural roads
- In 2008, we paved 2 projects - combined about 30 lane miles. We have one project this year with about 10 lane miles of paving.
- 100

Survey Question 4

What is the typical thickness (inches) used by your agency when paving 4.75mm NMAS thin overlays?

Below are individual agency comments associated with this question.

Agency Comments:
- less than 1 inch
- 15 mm
- Approximately a 1/4 inch, I believe it is specked at 25lbs/sy
- from 0.8 to 1.125 inches
- 3/4"
- 0.75
- 0.75
- 0.75
- 83#/syd
- 3/4"
- 15-20 mm
- 0.75"
- 90 pounds per square yard
- 3/4 to 1 inch
- 3/4"

Survey Question 5

Have you done a life cycle cost analysis for 4.75mm NMAS thin overlays at your agency?

Table A-3 presents agency responses and includes summary statistics with the answers.

Table A-3: Summary of survey question 5

Yes	No
1/17 (5.9%)	16/17 (94.1%)

Survey Question 6

At what condition does your agency apply 4.75mm NMAS thin overlays to an existing pavement?

Table A-4 presents agency responses and includes summary statistics with the answers.

Table A-4: Summary of survey question 6

Very good	Good	Fair	Poor	Very poor
1/16 (6.3%)	8/16 (50%)	12/16 (75%)	3/16 (18.8%)	0/16 (0%)

Survey Question 7

What ADT levels dictate the use of 4.75mm NMAS overlays and what is the average service life (years) of these overlays in your agency?

Below are individual agency comments associated with this question. The first is ADT level answers and the second is service life

Agency Comments:

- Non-NHS: <5000 AADT
- N/A
- ESAL C/D
- None explicitly. Typically less than 5000.
- Various.
- 1000
- <3 Million ESALS in 20 Years (Levels 1 or 2)
- all roads
- TL C or less (less than 10 million ESALs over 20 years)
- no specific ADT level
- Whatever gives less than 3 million ESALs over 20 years
- <1000
- all levels of ADT
- < 20000

Agency Comments:

- 5-7 years
- N/A
- unknown, mix still in service
- 6 to 15 years depending on conditions
- New program. Can't say yet.
- 8 years
- 8
- varies
- Not known. Just constructed five projects less than one year ago.
- 4-7 years
- 8 - 12 years
- undetermined - oldest pavements are 4 years old and still performing well
- 8-10 years

Survey Question 8

What is the typical overlay thickness (inches) and mix type used by your agency?

Below are individual agency comments associated with this question. The first is thickness (in), the second is gradation, and the third is NMAS (mm).

Agency Comments:

- 1.5 to 2.0
- 5/8" - 1-1/4"
- 1.25
- 1.5 to 2.0 inches
- 1.5" and 2.0"
- 1.25
- 2
- 2
- 1.5 or 2"
- Avg. 1.25"
- 40-80 mm
- 1.5
- 1.25 - 1.5
- 2 inches
- 1.5"

Agency Comments:

- Dense graded. Stone matrix Asphalt on Interstate

- Dense
- Dense
- dense
- Dense
- Dense, open.
- Dense
- Dense
- superpave mixes
- Dense
- dense graded
- Dense
- dense
- dense graded Superpave
- dense

Agency Comments:

- 9.5 and 12.5
- 9.5 mm
- 9.5
- 9.0 mm
- 9.5mm and 12.5 mm
- 12.5
- 12.5
- 12.5
- superpave mixes
- Either 9.5 or 12.5
- top size 12.5 mm , NMAS 9.0 mm
- 9.5
- 9.5 mm and 12.5 mm
- 12.5
- 9.5 mm

Survey Question 9

What is the typical service life (years) of a typical HMA overlay for your agency?

Below are individual agency comments associated with this question.

Agency Comments:

- 10 to 12 years
- 7 - 10 years

- 13 years
- 8 to 10 years
- 15
- 10 to 12 years
- 14
- 12
- 15
- varies
- 15 years before considered deficient. 17-18 years before resurfaced.
- 15 years
- 12
- 10/14/2012
- 15 years
- 12 years

Survey Question 10

What was the overall performance of 4.75mm NMAS thin overlays in your area?

Table A-5 presents agency responses and includes summary statistics with the answers.

Table A-5: Summary of survey question 10

Very good	Good	Fair	Poor	Very poor
0/0 (0.0%)	10/13 (76.9%)	2/13 (15.4%)	1/13 (7.7%)	0/0 (0.0%)

Survey Question 11

What are the typical distress types you have seen in your 4.75mm NMAS thin overlays? (select all that apply)

Table A-6 presents agency responses and includes summary statistics with the answers. Individual agency comments associated with this question are also included below.

Table A-6: Summary of survey question 11

Rutting	Cracking	Raveling	Stripping	Loss of friction	Delamination	Other (please specify)
2/14 (14.3%)	8/14 (57.1%)	1/14 (7.1%)	0/0 (0.0%)	1/14 (7.1%)	4/14 (28.6%)	4/14 (28.6%)

Agency Comments to Other:

- reflective cracking
- reflective cracking
- Too early to identify.
- distresses typically only occur with poor project selection

Survey Question 12

What are the average rut depths (inches) in 4.75mm NMAS thin overlays at your agency?

Below are individual agency comments associated with this question.

Agency Comments:

- 0.25"
- N/A
- less than 0.5 inches
- 5mm
- VERY low to 0.
- .14
- 0.15
- n/a
- Too early to identify.
- none , they seldom rut
- negligible
- Not typically used in higher traffic, no real documentation of rutting
- 0.2 after 4 years
- 0

Survey Question 13

How does rutting of 4.75mm NMAS thin overlays compare to HMA overlays typically used at your agency?

Table A-7 presents agency responses and includes summary statistics with the answers.

Table A-7: Summary of survey question 13

More rutting	Similar rutting	Less rutting
2/11 (18.2%)	6/11 (54.5%)	3/11 (27.3%)

Survey Question 14

How do the friction levels of 4.75mm NMAS thin overlays compare to traditional HMA pavements in your agency? Are the friction levels of the 4.75mm NMAS thin overlays acceptable?

Table A-8 presents agency responses and includes summary statistics with the answers.

Table A-8: Summary of survey question 14

Better	Same	Worse (acceptable)	Worse (unacceptable)
0/12 (0.0%)	10/12 (83.3%)	2/12 (16.7%)	0/12 (0.0%)

Survey Question 15

Is reflective cracking a concern in 4.75mm NMAS thin overlay pavements (compare to typical HMA overlay)? If yes, to what extent?

Table A-9 presents agency responses and includes summary statistics with the answers. Individual agency comments associated with this question are also included below.

Table A-9: Summary of survey question 15

Yes	No
10/14 (71.4%)	4/14 (28.6%)

Agency Comments (severe, moderate, or minor):

- moderate
- severe, without milling
- Moderate
- moderate
- Severe
- minor
- It is a concern but the amount is not known at this time.
- moderate
- Again, 4.75 mm is not typically used where reflective cracking is an issue
- moderate

Survey Question 16

Is thermal cracking a concern in 4.75mm NMAS thin overlay pavements (compare to typical HMA overlay)? If yes, to what extent?

Table A-10 presents agency responses and includes summary statistics with the answers. Individual agency comments associated with this question are also included below.

Table A-10: Summary of survey question 16

Yes	No
3/13 (23.1%)	10/13 (76.9%)

Agency Comments (severe, moderate, or minor):

- moderate
- severe
- minor

Survey Question 17

Is raveling or stripping a concern in 4.75mm NMAS thin overlay pavements (compare to typical HMA overlay)? If yes, to what extent?

Table A-11 presents agency responses and includes summary statistics with the answers. Individual agency comments associated with this question are also included below.

Table A-11: Summary of survey question 17

Yes	No
4/13 (30.8%)	9/13 (69.2%)

Agency Comments:

- Moderate
- moderate
- Unknown.

Survey Question 18

Have studded tires and snowplows caused more damage to 4.75mm NMAS thin overlays than traditional HMA? What kind of damage?

Table A-12 presents agency responses and includes summary statistics with the answers.

Table A-12: Summary of survey question 18

Yes	No
0/11 (0.0%)	11/11 (100%)

Survey Question 19

Does your agency have specific project selection criteria used to determine when to apply 4.75mm NMAS thin overlays?

Table A-13 presents agency responses and includes summary statistics with the answers.

Table A-13: Summary of survey question 19

Yes	No
9/14 (64.3%)	5/14 (35.7%)

Survey Question 20

Have you used or considered using warm mix asphalt to pave 4.75mm NMAS thin overlays? If used what was the overall performance of the 4.75mm NMAS thin overlays with WMA?

Table A-14 presents agency responses and includes summary statistics with the answers. Individual agency comments associated with this question are also included below.

Table A-14: Summary of survey question 20

Used	Considered	Not used
6/13 (46.2%)	1/13 (7.7%)	6/13 (46.2%)

Agency Comments (very good, good, fair, poor, or very poor):

- good
- good performance
- Good
- Fair
- good
- used as a leveling course with very good results
- good

Survey Question 21

Have you done any studies to compare the performance of 4.75mm NMAS thin overlay with other preservation/maintenance strategies? If so can you specify which preservation/maintenance strategies?

Table A-15 presents agency responses and includes summary statistics with the answers. Individual agency comments associated with this question are also included below.

Table A-15: Summary of survey question 21

Yes	No
9/14 (64.3%)	5/14 (35.7%)

Agency Comments:

- we will
- Bidding head to head with micro surfacing - contractor's option, research project underway but no data yet
- In the process of constructing a pavement preservation test section.
- Study currently underway. No conclusions yet.
- Micro surface

Survey Question 22

Do you follow NCAT specifications or use your own design method for designing 4.75mm NMAS mix?

Table A-16 presents agency responses and includes summary statistics with the answers.

Table A-16: Summary of survey question 22

NCAT specification	Local method
5/14 (35.7%)	9/14 (64.3%)

Survey Question 23

Have you used RAP in 4.75mm NMAS mixes? If so in what % of the mix?

Table A-17 presents agency responses and includes summary statistics with the answers. Individual agency comments associated with this question are also included below.

Table A-17: Summary of survey question 23

Yes	No
11/15 (73.3%)	4/15 (26.7%)

Agency Comments:

- Not sure. Only a couple projects
- 10%

- 20% max
- 20
- Up to 30% of the AGED BINDER not the mix
- 15
- 15%
- not sure
- Specifications allow it up to 20%.
- Have used up to 25 %
- 40%

Survey Question 24

Was any pre-leveling, rut-filling, or milling performed before the application of 4.75mm NMAS thin overlays?

Table A-18 presents agency responses and includes summary statistics with the answers. Individual agency comments associated with this question are also included below.

Table A-18: Summary of survey question 24

Pre-leveling	Rut-filling	Milling	Other (please specify)
2/13 (15.4%)	1/13 (7.7%)	8/13 (61.5%)	3/13 (23.1%)

Agency Comments:
- some on existing layer
- no
- in some cases a thin shim was used

Survey Question 25

What was the compaction strategy used for the 4.75mm NMAS mix thin overlay to achieve desired density of the mat? How was the density measured by your agency?

Below are individual agency comments associated with this question.

Agency Comments:
- Method specification - no density testing due to layer thickness.
- Mix is compacted to the satisfaction of the project engineer
- Static for thin layer
  Gauge and cores

- 2 to 3 passes until mix is seated. No gauge or cores due to depth of mix.
- No density measurement. Minimum 2 static steel wheel rollers.
- Not measured since less than 1.5".
- When the 4.75 mm mix is placed less than 1 inch, here is an excerpt from our 504.03.12 Thin Lifts and Wedge/Level Courses, updated June, 2011:

  "Construct a 400 to 500 ft control strip on the first day of paving to determine optimum pavement density.

  Using an asphalt density gauge in accordance with the manufacturer's recommendation, take readings from the control strip in 5 random locations to determine roller patterns and the number of passes needed to obtain optimum density. Optimum density is defined as when the average density does not change by more than 1.0 percent between successive roller passes and the percent density is between 90.0 and 97.0.

  Core the five random gauge reading locations to verify the gauge calibration and to determine the percent pavement density. The cores will be tested by the contractor's QC laboratory and results will be verified by the Office of Materials Technology. The QA cores will be saved by the contractor and made available to the Administration for retesting until the end of the project or as otherwise determined.

  On the first day of paving, the target optimum density will be determined using the density gauge readings from the control strip; verified by the core results. The lot average density from the five control strip cores will be used as the target optimum density.

  Take a minimum of 10 QC/QA gauge readings daily from random locations per day's paving per mix or two per 500 tons of paving per mix; whichever yields the higher frequency of locations. A density lot is defined as a day's paving per mix. A sublot shall not exceed 500 tons. A paving day shall begin with a new lot and sublots.

  For the remainder of the project, any lot average 2.0 percent or more below optimum and below 92 percent shall require a new control strip to be constructed, tested and approved before paving continues.

  Take a minimum of 2 QA cores daily when production is in excess of 500 tons per location, or when successive days of less than 500 tons production totals 1000 tons or greater. If the average of the two density gauge readings and the average of the two respective QA core densities are within 3.0 lb per cubic foot, the Administration will accept all the daily density gauge readings. If they do not compare within 3.0 lb per cubic foot, a new control strip will be run and the density gauge recalibrated.

  Wedge/Level courses placed at variable thicknesses shall be tested and accepted in accordance with this Thin Lift specification. Incentives are not applicable."
- not sure

- Density was not measured. Used a "standard rolling pattern."
- 3 to 4 passes with steel roller(s)
- Steel wheel roller in static mode to seat the mix.
- Do not require compaction at 90 lbs per square yard.
- Density measured using cores. Compaction strategy determined by Contractor based on test strip results.
- Vibratory rollers were not allowed; oscillatory rollers are typically used. Density was originally measured by cores, but the cores proved to be too thin to safely cut. Density is now not measured but roller passes are counted.

Survey Question 26

Is there any additional information or comments you would like to add regarding the use of 4.75mm NMAS mixes?

Below are individual agency comments associated with this question.

Agency Comments:

- We find 4.75mm overlays to be an effective alternative compared to chip seals/micro surfacing.
- N/A
- APA rut depths were bad
- Binder content is higher than dense graded mixtures so additional binder cost needs to be considered when figuring any savings.
- Fairly cost effective. Districts at MDSHA which have used thin lifts, and particularly 4.75 mm mixes, are now typically steering away from it due to early cracking and delamination problems, often opting for a thin 1" 9.5 mm lift instead. However, note that we were not using polymer modified binders with the 4.75 mixes, would have helped.
- I'm answering these questions for our ultra-thin hma mix which has 75-95% passing the no. 4 sieve
- Too early to say in Florida.
- considered to be a very successful preservation treatment
- I have a presentation on the NJDOT use of 4.75 mm mix that I gave at the Mid-Atlantic QAW last year that I can provide.
- The performance of the 4.75 mixture itself has been very good in our state, the only major problems we have had are friction related. We have made some specification modifications to try to address these friction issues, and are currently monitoring results. We have had good success in urban areas where friction is not a concern.

Survey Question 27

What are the reservations or concerns, if any, to using 4.75mm NMAS mix for your agency? If your agency is not using 4.75mm NMAS mixes, indicate reasons for not using.

Below are individual agency comments associated with this question.

Agency Comments:

- Not enough experience
- cost, use for rut filling and maintenance patching
- At this point the cost is not significantly less than a traditional 1.25 inch HMA overlay.
- We do not have a specification in place at this time. Some contractors have used in for city and county jobs with success, but it has not been used by the state so far.
- currently use this size mix
- premature rutting
- This size is relatively new to AASHTO M 323, Table 3 and we have just started allowing it as a design option. As ITD does more and more thin preservation overlays, the greater the opportunity we will have to use it. The main concern is the high asphalt content that goes with a small NMAS. We are looking for results from other states to see how it is working.
- Research currently evaluating the use of 4.75 mm NMAS in Texas
- Cost, binder
- We don't have an abundance of crusher fines since we do not have quarries, natural fines only do not give is the rut resistance we are looking for.
- No need. Seems to add cost. Current thin lift specification allows placement of 3/4" of materials using -1/2" material.
- Proper project selection is key. Thin lifts should not be placed in areas with moderate cracking.
- WSDOT does not have any experience with these mixes (that is why we are sponsoring this research). Some questions we need answered include the potential for delamination of a thin lift and the potential for rutting.
- I did not think about so thin overlay.
- Performance and LCCA concerns.
- Less skid resistance, higher potential for plane slippage, delamination, and raveling.
- 3/8"' NMAS mixes have worked well for thin overlays and leveling layers. Do not have quarries with a surplus of fines. Question the cost of 4.75 mixes when having to manufacture aggregate and the high asphalt contents.
- Early cracking. Fast cooling. Delamination. Small reduction in friction.
- We have not yet implemented.
- Some concern with performance on high volume roadways. In the process of converting from a marshall mix to a gyratory mix.
- NCDOT has developed a new 4.75mm NMAS mix for use for pavement preservation. This new specification has been used on only one trial project so far in 2012. Unfamiliarity with this new mix type is our only reservation at this time.

- NYSDOT has a specification for a 6.3 NMAS mix that is working very well. It requires polymer modified binder, straight emulsion tack coat and is placed 3/4 to 1 inch thick. Contact Zoeb Zavery at Zoeb.Zavery@dot.ny.gov if you would like additional information re: our 6.3 mix.
- Main concern is raveling.
- We place rubberized open graded friction course as a wearing course on both AC and PCCP.
- None
- No reservations at this time. Using 9.5 mm NMAS for thin overlays. Conducting research investigating the use of 4.75 mm NMAS mixes.
- No reservations or concerns
- Given current economic conditions, many of our roads now need more that just a thin overlay.
- Finding the right candidate projects is the biggest concern.
- The only concern is friction performance.
- Typically "mill & pave" strategy is used, with conventional aggregate NMAS, larger than 4.75mm
- May rut, no structural integrity or stability
- MDT doesn't have any concerns. It's just not something we have pursued using at this point.
- nonstructural mix

  other options more cost effective