Development of Special Provision for Mix Design of Foamed WMA Containing RAP

Rouzbeh Ghabchi, Ph.D., A.M. ASCE
Musharraf Zaman, Ph.D., P.E., F. ASCE
Amir Arshadi, Ph.D., A.M. ASCE
Ashiqur Rahman
Manik Barman, Ph.D., A.M. ASCE

SPTC15.1-31-F

Southern Plains Transportation Center
201 Stephenson Parkway, Suite 4200
The University of Oklahoma
Norman, Oklahoma 73019

DISCLAIMER

The contents of this report reflect the views of the authors, who are responsible for the facts and accuracy of the information presented herein. This document is disseminated under the sponsorship of the Department of Transportation University Transportation Centers Program, in the interest of information exchange. The U.S. Government assumes no liability for the contents thereof

1. REPORT NO. SPTC15.1-31	2. GOVERNMENT ACCESSION NO.	3. RECIPIENTS CATALOG NO.
4. TITLE AND SUBTITLE Development of Special Provision for Mix Design of Foamed WMA Containing RAP		5. REPORT DATE November 30, 2019 6. PERFORMING ORGANIZATION CODE
7. AUTHOR(S) Rouzbeh Ghabchi, Musharraf Zaman, Amir Arshadi, Ashiqur Rahman, and Manik Barman		8. PERFORMING ORGANIZATION REPORT
9. PERFORMING ORGANIZATION NAME AND ADDRESS: The University of Oklahoma 660 Parrington Oval, Norman, OK 73019		10. WORK UNIT NO. 11. CONTRACT OR GRANT NO.
12. SPONSORING AGENCY NAME AND ADDRESS Southern Plains Transportation Center 201 Stephenson Pkwy, Suite 4200 The University of Oklahoma Norman, OK 73019		13. TYPE OF REPORT AND PERIOD COVERED Final July 2016 – November 2019 14. SPONSORING AGENCY CODE

15. SUPPLEMENTARY NOTES
The research team consisted of members from the School of Civil Engineering at the University of Oklahoma. Matching funds for this project were provided by the Silver Star Construction Co. Inc.

16. ABSTRACT
Volumetric properties of an HMA mix and a foamed WMA were investigated in this study to examine if there is a need to develop special provisions for designing foamed WMA mixes containing RAP. To this end, foamed WMA mixes were produced in the laboratory using an asphalt foamer and volumetric properties were determined. Performance tests, namely Hamburg wheel tracking test (HWT) to evaluate rutting resistance, semi-circular bend (SCB) test to evaluate fatigue resistance, and tensile strength ratio (TSR) test to evaluate stripping resistance were conducted. For each mix type, the amount of RAP was varied within the range of ODOT's interest (25% for S3 and 5% for S4 mixes). It was found that the volumetric properties of an HMA mix and a foamed WMA mix designed in the lab were statistically identical. As a result, the current practice of using the HMA mix design procedure for designing a foamed WMA mix for temperatures considered in this project was found to be acceptable. Consequently, no modifications to this design procedure were recommended. Performance-wise, however, foamed WMA was found to exhibit higher cracking resistance and lower rutting resistance compared to HMA. Because of the presence of moisture from partially dried aggregates at lower mixing and compaction temperatures and use of water in the foaming process, WMA exhibited a lower resistance to moisture-induced damage than HMA. A technology transfer workshop was organized to share the outcomes of this study with ODOT, Oklahoma Asphalt Pavement Association (OAPA) and other stakeholders.

17. KEY WORDS Warm Mix Asphalt, Foam based WMA, RAP		18. DISTRIBUTION STATEMENT No restrictions. This publication is available at www.sptc.org and from the NTIS.	
19. SECURITY CLASSIF. (OF THIS REPORT) Unclassified	20. SECURITY CLASSIF. (OF THIS PAGE) Unclassified	21. NO. OF PAGES 82	22. PRICE

SI* (MODERN METRIC) CONVERSION FACTORS				
APPROXIMATE CONVERSIONS TO SI UNITS				
SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
	NOTE: volumes greater than 1000 L shall be shown in m^3			
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa
APPROXIMATE CONVERSIONS FROM SI UNITS				
SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

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

ACKNOWLEDGMENTS

The authors would like to acknowledge the financial support provided by the Southern Plains Transportation Center (SPTC) for this project. Also, the matching support provided by the Oklahoma Department of Transportation through SPTC is gratefully acknowledged. Assistance received from Mr. Kenneth Hobson in technical aspects of this project and material collection is highly appreciated. Moreover, help of Silver Star Construction Company and Haskell Lemon Construction Company in providing materials and asphalt mixes is gratefully acknowledged. Finally, the research team is thankful to Mr. Michael F. Schmitz, Mr. Syed Ashik Ali, Ms. Shivani Rani, Mr. Nayan Kumar Naidu Raguluri, Mr. Vijay Kanth Reddy Sriram, and Mr. Sai Srikanth Kola for their help in the laboratory.

Development of Special Provision for Mix Design of Foamed WMA Containing RAP

Final Report

November 2019

Rouzbeh Ghabchi, Ph.D., A.M. ASCE
Musharraf Zaman, Ph.D., P.E., F. ASCE
Amir Arshadi, Ph.D., A.M. ASCE
Ashiqur Rahman
Manik Barman, Ph.D., A.M. ASCE

Southern Plains Transportation Center
OU Gallogly College of Engineering
201 Stephenson Pkwy, Suite 4200
Norman, OK 73019

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

EXECUTIVE SUMMARY

For more than a decade, as a part of efforts toward establishing a sustainable and eco-friendly construction practice, the pavement industry has been using reclaimed asphalt pavement (RAP) and different warm mix asphalt (WMA) technologies in production of asphalt mixes and construction of flexible pavements. Among the available WMA technologies, the plant foaming technique using water injection method (called foamed WMA in this report) has been found to be economical by producers because of low mixing and compaction temperature. Economic benefits are also contributed by using no chemical additives to reduce mixing and compaction temperatures. Although the use of RAP and foamed WMA is rapidly increasing in Oklahoma and other states in Region 6, a widely accepted guideline/specification for the design of foamed WMA containing RAP is not available. In the absence of such guidelines/specifications, the paving industry is designing the foamed WMA mixes by using the specifications (AASHTO R 35) originally developed for the design of hot mix asphalt (HMA), which is produced at temperatures approximately 28°C higher than those used for the production of WMA.

In this study, the mix design volumetrics and laboratory performances, namely rutting, cracking and moisture-induced damage potential of foamed WMA containing RAP, were evaluated and compared with their hot mix asphalt (HMA) counterparts. One coarse (S3) mix having a nominal maximum aggregate size (NMAS) of 19 mm and one fine (S4) mix (NMAS = 12.5 mm), containing 25% and 5% RAP, respectively, were used for evaluation. It was found that the foaming process increased the coating ability of the binder, which in turn, lowered the mixing and compaction temperatures for foamed WMA. Therefore, both HMA and foamed WMA exhibited similar mix design volumetrics for the mixing and compaction temperatures considered in this study. In spite of foamed WMA mixes exhibiting similar volumetric properties as compared to those of HMA, their laboratory performance was found to be significantly different. The foamed WMA was found to exhibit a lower stiffness compared to HMA (Rahman, 2019). An increase in RAP content was found to increase the stiffness of asphalt mixes due to incorporation of aged binder from RAP. A stiffer asphalt mix is expected to exhibit lower cracking resistance and higher rutting resistance. Therefore, foamed WMA was found to exhibit higher cracking resistance in SCB tests compared to HMA. The rutting performance of foamed WMA,

however, was found to be of concern as they exhibited lower resistance compared to their HMA counterparts. Coarser mixes exhibited higher rutting resistance compared to finer mixes due to higher RAP content. The foamed WMA exhibited higher moisture-induced damage potential compared to HMA. The presence of moisture from partially dried aggregates at lower WMA mixing and compaction temperatures and use of water in the foaming process were possible reasons for the reduction in moisture-induced damage resistance for foamed WMA. Additionally, a test section was constructed in this project using the foamed WMA mix with the help of the industry partner (Silver Star Construction Co.). Performance tests (SCB, TSR and HWT) were conducted on cores extracted from the constructed pavement sections as well as on the laboratory-prepared samples from loose mixes collected from the test site. Overall, it was concluded that although the volumetrics of foamed WMA and HMA mixes were statistically identical, foamed WMA containing RAP can exhibit mixed performance particularly when the RAP content is higher than certain level (Rahman, 2019). A technology transfer workshop was organized to share the outcomes of this study with ODOT, Oklahoma Asphalt Pavement Association (OAPA) and other stakeholders.

CHAPTER 1 - INTRODUCTION

Construction of sustainable and environment-friendly transportation infrastructures results in saving of natural resources, conserving environment, and reducing energy consumption. For more than a decade, as a part of efforts toward establishing a sustainable and eco-friendly construction practice, the pavement industry has been using reclaimed asphalt pavement (RAP) and different warm mix asphalt (WMA) technologies in production of asphalt mixes and construction of flexible pavements. In recent years, with increasing asphalt binder cost and scarcity of high-quality aggregates, the demand for using RAP in asphalt mixes has been increasing steadily. Due to its economic and environmental benefits, increased amounts of RAP are being used in WMA and hot-mix asphalt (HMA). The WMA technologies allow a reduction in mixing and placement temperatures, leading to major savings in fuel cost, reduction in emission, and better workability at a lower temperature. Therefore, many Departments of Transportation (DOTs) and other agencies are interested in evaluating the performance of WMA mixes and developing pertinent special provisions and specifications. According to NCHRP, WMA mixes are generally produced using additives such as chemicals, waxes, and synthetic zeolites, as well as plant foaming (NCHRP, 2012b). Among the available WMA technologies, the plant foaming technique using water injection method (called foamed WMA in this report) is found by producers to be most economical. Although the use of RAP and foamed WMA is increasing rapidly in Oklahoma and other states in Region 6, a widely accepted guideline/specification for the design of foamed WMA containing RAP is not available yet. In the absence of such guidelines/specifications, the paving industry is designing the foamed WMA mixes by using the specifications originally developed for the HMA mix design (AASHTO R 35), which is produced at temperatures approximately 28°C higher than those used for the production of WMA.

Due to the differences in mixing and placement temperatures and presence of water in foamed WMA as well as differences in aggregate coating quality and binder film thickness, using HMA mix design for a foamed WMA without making adjustments in the job mix formula (JMF) may or may not be a realistic approach. Consequently, the compactability, moisture-induced damage potential, rutting resistance and fatigue resistance of WMA mixes can be significantly different than those of their HMA counterparts (NCHRP, 2012b). Therefore, it is important to investigate the appropriateness of using the mix design developed for the design of HMA in designing foamed WMA mixes containing RAP. The present study was undertaken to address this need.

In the present study, the differences in major volumetric properties between an HMA mix and a foamed WMA were compared to evaluate if there is a need for the development of a special provision for designing foamed WMA mixes containing RAP. To this end, foamed WMA was produced in the laboratory using an asphalt foamer. WMA specimens were compacted in a Superpave® gyratory compactor (SGC) for determination of volumetric properties. Also, performance tests (rutting, cracking, and moisture-induced damage) were conducted on foamed WMA and HMA specimens and the results compared. The performance tests were focused on the following: Hamburg wheel tracking test (HWT) (OHD L-55) to evaluate rutting resistance, semi-circular bend (SCB) test (AASHTO TP 105-13) to evaluate fatigue cracking resistance, and tensile strength ratio (TSR) test (AASHTO T283) to evaluate moisture-induced damage potential. For each mix type, the amount of RAP varied within the range of ODOT's interest. These performance data as well as the volumetric properties were used to analyze the differences between the HMA and foamed WMA. Additionally, a test section was constructed in this project using the foamed WMA mix with the help of the industry partner (Silver Star Construction Co.). Performance tests (SCB, TSR and HWT) were conducted on cores extracted from the constructed pavement sections as well as on the laboratory-prepared samples from loose mixes collected from the test site. The test results were used to evaluate the performance of the foamed WMA designed using the current specification. A technology transfer workshop was organized to share the outcomes of this study with ODOT, Oklahoma Asphalt Pavement Association (OAPA) and other stakeholders. The results and findings are presented in this report.

CHAPTER 2 – LITERATURE REVIEW

2.1. General

Construction of sustainable and environment-friendly transportation infrastructure results in saving of natural resources, conserving the environment, and reducing energy consumption. For more than a decade, as a part of efforts toward establishing a sustainable and eco-friendly construction practice, the asphalt paving industry has been using RAP and various WMA technologies in the production of asphalt mixes and the construction of flexible pavements (Kim and Lee, 2006; Kim et al., 2007; Kasozi et al., 2012; Zhao et al., 2013; Guo et al.; Dong et al., 2017). Warm mix asphalt, which was originally developed in Europe, is used to produce asphalt mixes at lower mixing and compaction temperatures with workability, strength, and durability equivalent to or better than those of traditional HMA (D'Angelo, et al., 2008; You, et al., 2011). There are several other benefits of using WMA which include extended paving window season, a shorter turnover time to traffic, improved working conditions due to lower odour, fume, and emission levels, enhanced compactability, reduced oxidative hardening of binders, and reduced cracking in pavements (Hurley et al., 2006; Gandhi et al., 2009; Rubio et al., 2012). The most commonly used WMA technologies in the U.S. can be classified into two groups: (1) process-driven technologies, such as plant foaming like Double Barrel Green® and Low Energy Asphalt; and (2) chemical or organic additives such as Evotherm®, Rediset WMX, REVIX™, and Sasobit®, among others (Chowdhury, et al., 2008). The additive-based WMA processes are commonly classified into two groups based on the additive types: (i) organic additive-based process (e.g., Fischer-Tropsch synthesis wax, fatty acid amides, and Montan wax); and (ii) chemical additive-based process (usually emulsification agents or polymers) (Rubio et al., 2012). The first experience of WMA technology application on a test track in the US was the Evotherm® test sections at the National Center for Asphalt Technology (NCAT) Pavement Test Track at Auburn University in September 2005 (Prowell, et al., 2007). Many asphalt technologists, design engineers, and agency personnel were quick to recognize that by reducing production and construction temperatures in the WMA technologies, the asphalt binder was less oxidized, which, in turn, resulted in asphalt mixes with a higher cracking resistance compared to that of the traditional HMA mixes (Prowell, et al., 2007; Williams, 2010). Based on the National

Asphalt Pavement Association (NAPA) data, more than 105 million tons of WMA were produced in 2013 (Hansen, 2015). This is a 23 percent increase from 2012 and more than 533 percent increase in the use of WMA since 2009. On an average, the use of WMA results in approximately 20% savings in energy, which is equivalent to saving $1.9 billion in energy costs in 2013, alone. According to NCHRP, use of WMA reduces CO_2 emissions by about 20% (Hansen, 2015). Details of benefits and potential concerns of WMA can be found in the following references: (NCHRP (2012a,b); NCHRP (2013); AASHTO (2013); and ODOT (2013).

2.2. Types of WMA Technologies

2.2.1. Organic Additive-Based WMA

Organic additives augment waxes to the mix. When the temperature exceeds the melting point of a wax, a reduction in the viscosity of the binder is observed (Zaumanis, 2010; Kheradmand et al., 2014). As the mix cools down, these additives transform into microscopically small and uniformly dispersed solid particles, which work in the same manner as fiber-reinforced materials by increasing the stiffness of the binder. Silva et al. (2010) suggested that the type of wax must be selected cautiously to avoid possible temperature-related issues. More specifically, problems may arise if the wax has a lower melting point than the mixing temperature. Waxes used in this technology consist of high molecular hydrocarbon chains with a melting point of 80 to 120 °C and are able to modify the characteristics of the binder (Rubio et al., 2012). The length of the carbon chain (C45 or more) controls the temperature at which the wax melts (Bueche, 2009). In practice, two to four percent wax is added to a mix based on the total mass of the binder. As noted below, the wax-based organic WMA technology can be classified into three categories depending on the wax type, namely Fischer-Tropsch wax, fatty acidamide, and Montan wax (Rubio et al., 2012).

2.2.2. Chemical Additive-Based WMA

A chemical additive-based WMA involves the use of chemical additives in the asphalt binder to reduce mixing and compaction temperatures. These additives are mixed with the asphalt binder before mixing with aggregates. Based on the circumstances, different types of chemical additives can be used. They generally include a combination of surfactants, polymers, emulsifying agents, and adhesion promoting additives (i.e., antistripping agents) to improve workability, coating, and compaction (Rubio et al., 2012). For example, by using chemical additive REVIX®, a reduction in production temperature of up to 30°C can be achieved. Another commonly used additive, Evotherm®, can reduce the production temperature up to 85°C (Rubio et al., 2012; Kheradmand et al., 2014).

2.2.3. Foamed WMA

Foamed WMA involves adding a small amount of water at high temperature by either injecting it into the binder or directly introducing it into the mixing compartment (Larsen, 2001). The addition of water generates a large volume of foam, which temporarily increases the volume of the binder and reduces its viscosity. This process improves the workability and coatability of the binder (Rubio et al., 2012). Butz et al. (2001) tested this method for soft grade and medium grade binders. Water is usually injected at a rate of approximately one to two percent by binder's weight (NCHRP, 2012b). The optimum water content is determined based on two major factors: maximum expansion ratio and half-life. The term half-life is defined as the time in seconds it takes for foam to become half of the maximum volume of the foamed asphalt. Generally, at a temperature above 150°C a good foaming of asphalt binder can be achieved. Yongjoo and Lee (2006) found that, although increasing foaming temperature and water increases the expansion ratio, it reduces the half-life, which is not desirable for the foamed asphalt. At a temperature of 170°C, an air pressure of 400 kPa, and a water pressure of 500 kPa, a water content of 1.3% is found as the optimum foaming water content (Yongjoo and Lee, 2006). Among the WMA technologies, plant foaming technique (foamed WMA), which uses water for foaming, has gained the most attention in Oklahoma as well as other states in Region 6 (NCHRP,2013). This is mainly because production of foamed WMA using water injection is cost-effective as there is no need for changing the mixing process or using chemical additives. Other plant foaming technologies have also been introduced to asphalt paving industry. A summary of these technologies is presented in Table 1 (NCHRP, 2012b).

Table 2.1 Summary of popular plant foaming technologies in the U.S. (NCHRP, 2012b)

Plant Foaming Technology	Company
Accu-Shear Dual Warm Mix Additive System	Stansteel
AquaFoam	Reliable Asphalt Products
Adesco/Madsen Static Inline Vortex Mixer	Adesco/Madsen
AQUABLACK	Maxam Equipment Company, Inc.
Double Barrel Green	Astec, Inc.
Meeker Warm Mix Asphalt	Meeter Equipment
Terex Warm Mix Asphalt	Terex Roadbuilding
Ultrafoam GX	Gencor Industries, IUnc.

2.3. Reclaimed Asphalt Pavement (RAP)

Among various recycled materials, RAP is the most widely used recycled material by the asphalt industry (Sengoz and Oylumluoglu, 2013). Utilization of RAP in asphalt mixes is beneficial because it reduces the use of virgin aggregates and the overall need for virgin asphalt binder in asphalt mixes. Also, it preserves natural resources and the environment. The RAP is generated when asphalt pavements are milled for reconstruction or resurfacing (FHWA, 1997; ICT, 2007). Before the use of RAP in new mixes, economic issues, energy usage, and environmental factors along with the technical issues should be considered. For example, Dinis-Almeida et al. (2012) suggested various recycling techniques based on the binder type, production location, and construction temperature.

2.4. Laboratory and Field Performance of WMA Containing RAP

Although the volumetric properties of WMA mixes can be similar to those of HMA mixes, the compactability, rutting resistance, fatigue resistance and moisture-induced damage potential of WMA mixes can be significantly different than those of an HMA mix (NCHRP, 2012b). The results from this study support this view.

Based on previous studies summarized by Rahman (2019), performance evaluation of additive-based and foaming-based WMA mixes shows that WMA mixes produced by using chemical additives perform similar to conventional HMA in terms of rutting. However, the foaming-based WMA mixes were found to have a higher rutting potential compared to the HMA (You, et al., 2011). Haggag et al. (2011) evaluated the performance of three WMA mixes produced using Advera®, Evotherm®, and Sasobit® and concluded that there was no significant difference between dynamic modulus and fatigue characteristics of WMA and HMA mixes (Haggag, et al., 2011). In a similar laboratory study, it was shown that the overall performance of WMA mixes produced with Advera®, Sasobit®, and Evotherm® was comparable to that of the traditional HMA (Aschenbrener, et al., 2011). Comparisons of mixes placed in three different sections of the NCAT Test Track indicated that WMA mixes produced using Evotherm® had in-place densities equal to or better than those of the HMA mixes. Also, the rutting performance of WMA mixes was similar to that of HMA mixes (Prowell, et al., 2007). A laboratory study conducted on two different asphalt mixes produced by the addition of Rediset WMX® and Sasobit® to asphalt binders indicated that using these additives the compaction temperature can be reduced by 40ºC, without any significant changes in stiffness and permanent deformations compared to a reference HMA mix (Zaumanis, 2010). A field study was performed in Washington to examine different WMA technologies including Sasobit® and three water foaming technologies, namely Gencor® Green Machine Ultrafoam GX®, Aquablack™ and water injection (Bower, et al., 2016). The field data showed that WMA mixes exhibited rutting, roughness, and cracking resistance comparable with their corresponding control HMA mix (Bower, et al., 2016). Dong et al. (2017) studied the high temperature performance and microstructure of foamed WMA and HMA prepared with various amounts of RAP binder contents (0, 20, 40, 60, and 80% by total weight of binder). According to this study, the increment of RAP binder content increased the mix stiffness for both foamed WMA and HMA; however, a change in the stiffness of HMA was relatively higher than that of foamed WMA. It was concluded that with the addition of RAP binder the high temperature performance of HMA was better, but workability was less relative to foamed WMA (Dong, et al., 2017).

Volumetric properties, such as air voids, optimum asphalt binder content and injected water, used in mix design are among the important parameters that affect the quality of the foamed asphalt mix containing RAP. Yongjoo et al. (2007) indicated that the main factor in determining the optimum asphalt binder content for foamed asphalt for cold in-place recycling application was stiffness of the RAP materials not the amount of the residual asphalt content present in the RAP materials (Yongjoo, et al., 2007). In a study conducted by Yu et al. (2013), various rheological properties of asphalt binder blends in a mix were considered in determining the optimum water content of the foamed warm mix asphalt. Different amounts of water were added to foam non-modified and styrene–butadiene–styrene (SBS) tri block copolymer-modified asphalt. The test results indicated that the optimum water content of the unmodified and SBS modified binders were 1% and 3%, respectively (Yu, et al., 2013). In another study, Yin et al. (2014) examined the workability and coatability properties of foamed WMA for various foaming water contents (1%, 2% and 3%) and sources (N, O and Y) with respect to control HMA samples. In that study, the maximum shear stress in the Superpave® Gyratory Compaction (SGC) was considered as the mix workability parameter and a factor based on aggregate absorption was suggested as coatability index. For WMA mixes produced by using a PG 64-22 asphalt binder from two different sources, the maximum workability and coatability were observed at 1% foaming water content. Although WMA mixes were produced at about 17°C lower than that of HMA, the mix containing 1% foaming water content showed a better workability and coatability compared to those observed for a similar HMA mix. Also, the mixes produced using a PG 64-22 asphalt binder from two different sources showed better workability and coatability as compared to those containing a PG70-22 binder from two other sources. This is due to the existence of polymer-modified binders with high viscosity in the PG 70-22 (Yin, et al., 2014). In another study it was found that a reduction in production temperature resulted in the foamed WMA to become more susceptible to rutting and moisture-induced damage (Ali, et al., 2013). Therefore, the maximum temperature reduction of 17ºC from the HMA production temperature was recommended for the foamed WMA. Secondly, it was shown that increasing the foaming water content to 2.6% of the weight of asphalt binder had no negative impact on the rutting and moisture-induced damage performance of the foamed WMA. Lastly, it was suggested that the aggregates used in the foamed WMA may need to dry for a longer period of time compared to those used in an HMA mix because of the lower production temperature in WMA mixes.

In the last few decades, RAP and reclaimed asphalt shingles (RAS), to a lesser extent, have been used in the U.S. since they significantly reduce the cost of the mix used in construction in addition to their benefits for the environment. However, utilization of these materials presents a concern about the performance of the mix, specifically fatigue performance due to highly aged RAP/RAS binder incorporated in the mix. Mogawer et al. (2015) evaluated the effect of long-term aging on fatigue performance of high-RAP mixes modified with rejuvenators. They concluded that long-term aging did not have any significant effect on fatigue characteristics of the high-RAP mixes with or without rejuvenators. Also, it was shown that fatigue performance comparable to control mix with no RAP can be achieved for high-RAP mixes by application of rejuvenators. In a recent study, the effect of Evotherm® on WMA mixes was studied by incorporating RAP contents ranging from 0 to 70% (Dai, et al., 2016). It was shown that although rutting resistance of WMA with 0% RAP is similar to that of control HMA, WMA mixes showed poorer fatigue performance than HMA mixes. The addition of RAP stiffened the mixes and improved the rutting resistance significantly, while reducing their fatigue resistance. It was also indicated that utilization of Evotherm® greatly improved the resistance of WMA mixes containing RAP against moisture-induced damage. In another study, Sabouri et al. (2015) investigated the fatigue performance of twelve plant-produced mixes with RAP contents ranging from 0 to 40% by total weight of the mix. I was concluded that, in general, utilization of RAP in the asphalt mix increased the mix stiffness which resulted in a reduction in its fatigue resistance. It was also reported that lowering the virgin binder PG grade improved the fatigue resistance.

In general, there are at least four approaches to improve the cracking resistance and durability of RAP/RAS mixes and balance their performance: (i) reducing RAP/RAS amount; (ii) lowering design air voids; (iii) utilization of Rejuvenating Agent; and (iv) using soft virgin binders (Zhou, et al., 2013). Performance of WMA mixes produced using different technologies and containing high percentages of RAP was evaluated by Zhao et al. (2013). The test results showed that the rutting resistance of the WMA samples was lower than that of the HMA regardless of the WMA technology, RAP content, and structural layer. The rutting resistance of both WMA and HMA was found to increase with the addition of RAP. The use of RAP, however, had a more favourable effect on rutting resistance of HMA mixes than that for the corresponding WMA mixes. The dissipated creep strain energy (DCSEf) and beam fatigue test data indicated that, when high percentages of RAP were incorporated in a mix, WMA mixes performed equal to HMA in terms of cracking and fatigue resistance (Zhao et al., 2013). The results of both tensile strength ratio (TSR) and resilient modulus (M_R) tests showed that the moisture-induced damage in base layer remained a concern in foamed WMA containing RAP. These test results suggested that the RAP content should be no more than 30% (Zhao et al., 2013). Shu et al. (2012) evaluated the moisture-induced damage potential of foamed WMA containing a high percentage of RAP. It was found that water as the foaming agent and a lower production temperature increased the possibility of existence of moisture in new mix, which may lead to moisture-induced damage in foamed WMA mixes. The test results indicated that the ITS and M_R values of asphalt mixes increased with the addition of RAP content. The increase in ITS and M_R values was mainly attributed to a stiffer and more brittle asphalt mix as a result of incorporation of RAP. Also, the addition of RAP was found to improve the resistance of the mix to moisture-induced damage. Furthermore, it was found that the MIST conditioning resulted in a larger reduction in resilient modulus, whereas freeze-thaw cycles caused a larger reduction in ITS. The plant-produced WMA was found to have a moisture-induced damage potential similar to its HMA counterpart. The test results indicated that the resistance of asphalt mixes to moisture-induced damage can be characterized based on resilient modulus and ITS results (Shu, et al., 2012).

2.5. Selection of Laboratory Performance Tests

Based on the literature review conducted in this project, HWT, TSR, and SCB tests were selected and used to evaluate rutting, moisture-induced damage potential and fatigue characteristics of asphalt mixes, respectively. A summary these tests is presented in this section.

2.5.1. Hamburg Wheel Tracking (HWT) Test

To evaluate the rutting potential of asphalt mixes, many transportation agencies have been using loaded wheel testers (LWT). Different types of LWTs are currently being used for accelerated evaluation of rutting potential of asphalt mixes, which include Georgia Loaded Wheel Tester, Hamburg Wheel Tracking (HWT) device, Asphalt Pavement Analyzer (APA), Laboratoire Central des Ponts et Chausées (French) Wheel Tracker, and Purdue University Laboratory Wheel Tracking Device (PURWheel) (Miller et al., 1995; Choubane et al., 2000; Cooley et al., 2000; Corte, 2001; Kandhal and Cooley, 2002).

Lu and Harvey (2006) evaluated the effectiveness of HWT tests to determine the moisture sensitivity of asphalt mixes and predict field performance by using laboratory test data. From that study, it was found that the HWT test results failed to correlate with the actual field performance of the asphalt mixes in some cases. However, the HWT device showed better predictions of field performance for mixes containing polymer-modified binders than mixes containing non-modified binders. In case of binders with no polymers, the field rutting was overestimated.

The rut depths obtained from the HWT tests were compared with the rut depths from the Mechanistic-Empirical Pavement Design Guide (MEPDG) by Grebenschikov and Prozzi (2011). The results of two types of asphalt mixes from a previous project in Texas were used for this purpose. Each mix was prepared using five binder contents and three aggregate gradations, namely fine, target and coarse. It was found that both the MEPDG and the HWT tests ranked the mixes in the same order.

Walubita et al. (2012) evaluated three laboratory tests, namely dynamic modulus (DM), repeated load permanent deformation (RLPD), and HWT, for characterizing permanent deformations or rutting of HMA mixes relative to the field performance under both conventional traffic loading and accelerated pavement testing (APT). It was observed that all three test methods provided consistent results in terms of rutting behavior. Also, the Superpave® mixes generally were found to exhibit higher moduli values with a greater resistance to rutting than the conventional mixes. The HWT test was found to exhibit the best repeatability and the lowest variability in results compared to the DM and RLPD tests. It was suggested that the HWT test be used for the assessment of rutting and stripping performance of HMA (Walubita et al., 2012).

Sel et al. (2014) evaluated the effect of test temperature on rut depths of asphalt mixes obtained from HWT tests. Statistical analyses of the collected data showed that the binder grade was an influential factor on HWT-based rut performance. Asphalt mixes containing binders with a higher PG grade were found to exhibit a higher resistance to rutting than those containing binders with a lower PG grade. Significant differences in performance were observed when the samples were tested at 40°C and 50°C, indicating a high sensitivity of rutting to temperature.

Tsai et al. (2016) suggested some important provisions to attain consistent results from the HWT test. From the 2-D micromechanical finite element (MMFE) model, it was suggested that the specimens should be glued together during HWT testing to ensure full bonding of cylindrical specimens. Otherwise, there is a chance of localized failures around the joint. From the MMFE analysis, it was also found that segments less than 120 mm wide can result in lower rut depths due to shape effect. Tsai et al. (2016) also suggested that rutting in slab specimens occurred at a faster rate than that in glued cylindrical specimens. Therefore, an agency may not allow cylindrical and slab specimens simultaneously in a given project. Lastly, it was suggested that the Weibull three stage curve fitting be used to interpret the HWT rutting evolution curve. The stripping initiation point (SIP) can be determined more effectively using this method than the traditional SIP suggested in AASHTO T 324 (AASHTO, 2016).

An image processing software, image processing and analysis system 2 ($IPAS^2$), was used by Chaturabong, and Bahia (2017) to identify the mechanism(s) responsible for permanent deformations or rutting in asphalt mixes in a dry HWT test. An increase in contact/proximity zones between aggregates was observed during the initial creep stage due to load application. In the secondary creep stage, the aggregate structure was found to begin dilating due to the deformation along the loading directions and shifting to the sides. At this stage, the aggregate structure was still in a stable condition and no significant reduction in proximity zone was observed. In the tertiary stage, however, the aggregate structure was observed to dilate completely. The failure in the mix in the dry HWT test was mainly attributed to localized deformation in the mix skeleton showing failure criteria similar to that observed in the confined and unconfined flow number (FN) test (AASHTO, 2017).

2.5.2. Fatigue Performance Tests

Fatigue cracking is one of the most common distresses in asphalt pavements caused by thermal gradients and traffic loading (Colombier, 1997; Baek J. 2010; Moreno, & Rubio, 2013).

To reduce fatigue cracking in asphalt mixes, mastic type (composed of the asphalt, filler and fine aggregate fraction) should be selected cautiously in the design phase. The cracking process in asphalt mixes shows a similar trend to that in concrete (Topcu & Bilir, 2010), which usually starts in the mastic and propagates through the mix (Jenq and Perng, 1991; Kim and Little, 2005; Dave et al., 2007).

Shu et al. (2008) evaluated the impact of RAP content on the fatigue resistance of HMA mixes. In their study, the Superpave® indirect tensile (IDT) strength test and beam fatigue tests were conducted to evaluate the fatigue resistance of HMA mixes containing 0%, 10%, 20%, and 30% of RAP. It was observed that the fatigue performance of a flexible pavement is directly related to the cracking resistance of the asphalt mixes. The IDT test results indicated that incorporation of RAP increases the stiffness of HMA. However, the fatigue life of a mix may be compromised. Also, it was observed that the inclusion of RAP in a mix reduced the creep strain energy threshold and energy ratio. Hence, increasing the RAP amount from 0% to 30% was found to reduce the fatigue life. This was mainly attributed to an increase in the brittleness of the HMA mixes due to incorporation of RAP.

Kim et al. (2012) suggested a fatigue test based on the measurement of the critical strain energy release rate (J_c) in semi-circular beam (SCB) samples with different notch depths. Three different notch depths, namely 25.4, 31.8, and 38 mm, were used to increase the measurement accuracy by developing a linear regression correlation for the strain energies versus notch depths for each mix. Three semi-circular specimens were prepared for each notch depth. The specimens were loaded monotonically at a rate of 0.5 mm/min and tested until failure. A good correlation was observed between fatigue performance of asphalt pavements in the field with Jc values. Asphalt mixes with polymer-modified asphalt binders exhibited greater fracture resistance than those containing non-modified asphalt binders. Moreover, results of that study indicated that a reduction in production temperature for WMA techniques did not adversely affect the fracture resistance.

The effect of nature of coarse aggregate on the fatigue-cracking behavior of asphalt mixes was discussed by Moreno and Rubio (2013). In their study, the University of Granada–Fatigue Cracking Asphalt Test (UGR–FACT) was used to assess the cracking behavior of two asphalt mixes (Ophite and Limestone) at different frequencies, load amplitudes, and test temperatures. It was reported that UGR–FACT method can be used as an effective tool to evaluate the cracking resistance of asphalt mixes. The results of that study indicated that both mixes followed a similar trend of cracking resistance with a change in the test conditions. The damage observed in each cycle was greater for mixes containing the limestone aggregates than those with Ophite.

Huang et al. (2013) used a notched semi-circular bending fatigue test to evaluate the fracture resistance of asphalt mixes by using the finite element method. In their study, the fracture mechanics approach was used to characterize the fatigue damage in asphalt pavements. The fatigue life was defined as the number of load applications required to propagate a dominant flaw in the mix. From the finite element analysis (FEA) it was observed that, due to stress concentrations, the crack in the notched specimen does not initiate at the center of the cut but close to one of the corners of the notch. In their study, an asymmetric mesh was considered because the sample was not symmetric, once the crack initiated at either of the corners of the notch. The results also indicated that the asphalt binder plays an important role in the fracture performance of a mix. With the addition of suitable additives in the asphalt binder, significant improvements in the fracture resistance of the asphalt mixes were observed.

Ozer et al. (2016) proposed Illinois SCB test to evaluate fracture potential of asphalt mixes. It was suggested that the SCB test be performed at a temperature of 25ºC temperature and a loading rate of 50 mm/min due to reasonable repeatability of results under these conditions. In their study, an increase in fracture energy was observed with an increase in temperature and loading rate. A qualitative relation between the SCB test and Texas overlay test was observed. Ozer et al. (2016) proposed an index, called flexibility index (FI), to characterize fatigue cracking. According to these researchers, the FI depends on the total fracture energy and post peak slope of the load-deformation curve. The FI values for laboratory-produced mixes varied between two to ten. A higher FI value indicates a ductile material and vice versa. According to Ozer et al. (2016), asphalt mixes with FI values greater than 6.7 can be classified as best performing, while mixes with FI values less than 2 can be considered poor performing. Mixes with in between FI values are expected to exhibit intermediate performance.

A survey conducted by Barman et al. (2018) revealed that many DOTs do not perform fatigue tests for screening of asphalt mixes during the design phase due to lack of trained personnel, unavailability of proper equipment and consensus about the most suitable fatigue test. That study also suggested that indirect tensile test (IDT) was the most common fatigue test for many DOTs. Barman et al. (2018) proposed a new parameter called fatigue index (f_i), based on the IDT test data, for screening of mixes. The test results indicated that f_i can differentiate the fatigue resistance of different asphalt mixes in an effective way. Moreover, finer mixes with modified binders showed better cracking resistance compared to coarser mixes with unmodified binders.

2.6. Summary

While the foamed WMA and HMA mixes have significant differences in their mechanical properties and performance aspects (e.g., rutting, moisture-induced damage potential), no formal mix design procedure is available for foamed WMA (with or without any RAP). The current state of practice for designing foamed WMA is to design an HMA in the laboratory according to the AASHTO R 35 standard and substitute it with the corresponding WMA with no changes to the mix design parameters (NCHRP, 2012b). Also, a majority of the asphalt mix design laboratories in Oklahoma do not own a laboratory foamer. It is worth nothing that foamed WMA mixes containing RAP is widely used in paving projects in Oklahoma as well as other Region 6 states. The literature review presented above indicates a need for a study to indicate if a different a mix design procedure is needed for foamed WMA containing RAP. In this project the research team focused on determining the volumetric differences between a HMA mix and the corresponding foamed WMA mix in the lab. The results were used to investigate the need for a special provision for designing foamed WMA. Also, performance of foamed WMA containing RAP was evaluated and compared with its HMA counterpart. Furthermore, a life cycle cost analysis (LCCA) was performed to highlight the long-term and short-term financial benefits of using foamed WMA for construction of pavements.

CHAPTER 3 - MATERIALS AND METHODS

3.1. General

This chapter provides information about material selection and collection process, mix designs and their volumetric properties. Performance tests conducted on asphalt mixes for evaluating their rutting, fatigue cracking and moisture-induced damage potential are also discussed. In addition, the techniques used for producing foamed binder and foamed WMA are discussed herein.

3.2. Materials

Two types of asphalt mixes with nominal maximum aggregate size (NMAS) of 12.5 (S4) and 19.0 mm (S3) were selected and evaluated in this project. These mixes are typically used in Oklahoma as intermediate and surface course mixes, respectively. The S3 and S4 mixes contained 25% and 5% RAP, respectively (Table 3.1). A PG 64-22 asphalt binder was used as the virgin binder in the mix designs. As shown in Table 3.1, for each aggregate gradation type, one hot mix asphalt (HMA) and one foamed warm mix asphalt (WMA) were produced. Thus, a total of four mixes was studied.

On September 6, 2016 the research team collected the required materials including aggregates, RAP, asphalt binder, and mix design sheets for these mixes from Silver Star Construction Co., Moore, OK.

Table 3.1 Gradation and binder properties of asphalt mixes

Source	Mix ID	Mix Type	NMAS (mm)	Binder Type	RAP Content (%)
Silver Star Co.	HMA-S3	HMA	19.0	PG 64-22	25%
Silver Star Co.	WMA-S3	WMA	19.0	PG 64-22	25%
Silver Star Co.	HMA-S4	HMA	12.5	PG 64-22	5%
Silver Star Co.	WMA-S4	WMA	12.5	PG 64-22	5%

3.2.1. Aggregates and RAP

Approximately 1,000 kg aggregates and 270 kg RAP were collected (Figure 3.1) from the asphalt plant of Silver Star Construction Co. in Moore, Oklahoma. The collected materials were transported and stored in the University of Oklahoma Broce Civil

Engineering Materials Laboratory. Table 3.2 and Table 3.3 present the collected materials and percentages of different aggregates used for each of the S3 and S4 mixes. The S3 mix contained 10% of 1″ rocks, 27% of 5/8″ chips, 12% of screening, 15% of manufactured sand and 11% fine sand. The S4 mix contained 35% of 5/8″ chips, 10% of screening, 15% of manufactured sand and 14% fine sand. The gradation curves for both mixes are shown in Figure 3.2.

Figure 3.1 Collection of Materials from Silver Star Construction Co. (a) Aggregates Used for Laboratory Produced Asphalt mixes; (b) Collected Aggregates and Asphalt Binders

Table 3.2 Aggregate stockpiles, sources and amounts used in S3 mixes (HMA S3 and WMA S3)

Aggregate Type	Supplier/ Pit #	% Used in the Mix
1" Rock	Hanson 5008	10
5/8" Chips	Hanson 5008	27
3/16" Screenings	Hanson 5008	12
Manufactured Sand	Martin-Marietta (Davis, OK) P/S # m002285005	15
Sand	General Materials Inc. (Oklahoma City, OK) P/S # m009215515	11
Fine RAP	Contractor / Project Site P/S # Contractor	25

Table 3.3 Aggregate stockpiles, sources and amounts used in S4 mixes (HMA S4 and WMA S4)

Aggregate Type	Supplier/ Pit #	% Used in the Mix
5/8" Chips	Hanson 5008	35
Manufactured Sand	Hanson 5005	36
3/16" Screenings	Hanson 5008	10
Sand (Unlisted Source)	General Materials Inc., 63rd St. (Oklahoma City, OK)	14
Fine RAP	Contractor / Project Site P/S # Contractor	5

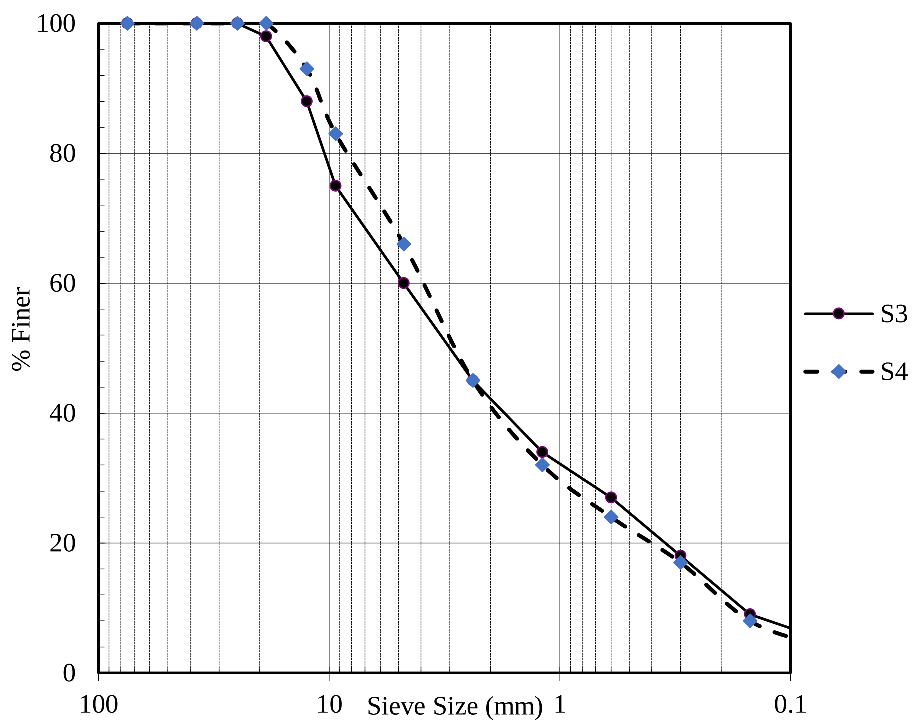

Figure 3.2 Combined Aggregate Gradation Curves (S3 Mixes and S4 Mixes)

3.2.2. Asphalt Binder

A PG 64-22 OK asphalt binder was used as the virgin binder in producing the asphalt mixes. The HMA mixes were produced directly by using this binder. However, for the production of WMA mixes, the collected binder was foamed using a laboratory foamer, namely AccuFoamer™ from InstroTek® (Figure 3.3). Figure 3.4 presents a schematic diagram of the foaming mechanism used in Accufoamer™. As shown in Figure 3.4, the foamer has two tanks: one for asphalt binder and another for foaming water. Both tanks have separate airlines to precisely maintain pressure. Once the desired temperatures and pressures are reached, the valves are opened to produce foamed binder (Figure 3.4). The foamed binder is produced by injecting pressurized water into preheated asphalt binder (Jenkins, 2000; Van et al., 2007). The injected water vaporizes and produces steam, which increases the volume of binder and reduces the binder's viscosity (Van et al., 2007; Zaumanis, 2010). In this study, foamed binder was produced at 135°C, which is a typical temperature used in the local asphalt plants.

Figure 3.3 Laboratory Foamer (AccuFoamer™ from InstroTek®, Inc.)

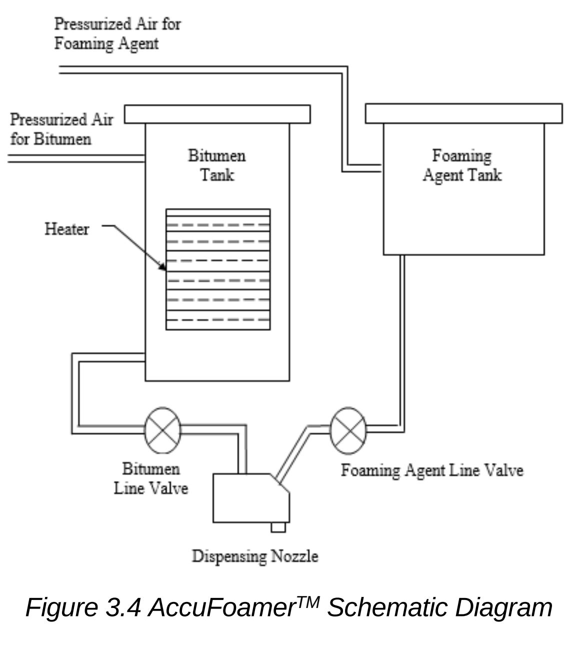

Figure 3.4 AccuFoamer™ Schematic Diagram

3.2.3. Asphalt Mixes

Mix design sheets for the foamed WMA S3 and foamed WMA S4 were collected from a local asphalt plant, Silver Star Construction Co., and are shown in Figure 3.5 and Figure 3.6, respectively. The mix design sheets for the WMA S3 and the WMA S4 mixes were initially developed as HMA S3 and S4 mixes, respectively based on the AASHTO R 35 method, considering light traffic condition (AASHTO, 2013). This method targets the volumetric properties of compacted samples at four percent air voids at HMA mixing and compaction temperatures (AASHTO, 2013). The mixing and compaction temperatures used for HMA mixes were 163°C and 149°C, respectively. Similar mix design methods were followed for foamed WMA mixes except using foamed binder and lower production temperatures. The mixing and compaction temperatures were lowered to 135°C and 127°C, respectively, for foamed WMA mixes. These are typical temperatures used for producing foamed WMA mixes in local asphalt plants. As noted earlier, a total of four asphalt mixes was produced in the laboratory. The characteristics of these mixes are summarized in Table 3.4.

Table 3.4 Superpave® volumetric mix design parameters

Mix Type	Mixing/Compaction Temperatures (°C)	NMAS (mm)	Binder Type	Foamed Binder	RAP Content (%)	Binder Replacement (%)	Total AC (%)
HMA S3	163/149	19	PG 64-22	No	25	31.1	4.5
WMA S3	135/127	19	PG 64-22	Yes	25	31.1	4.5
HMA S4	163/149	12	PG 64-22	No	5	4.1	4.9
WMA S4	135/127	12	PG 64-22	Yes	5	4.1	4.9

3.2.3.1. Asphalt Mix Preparation

The HMA S3 and WMA S3 were prepared using S3 aggregate gradation and HMA S4 and WMA S4 were prepared using S4 aggregate gradation. The HMA S3 and HMA S4 mixes were prepared using the HMA design procedure at higher production temperatures (Table 3.4) without foamed binder and are considered as control mixes. Based on the four percent air voids requirements, the optimum binder contents for the HMA S3 and HMA S4 mixes were found as 4.5% and 4.9% by weight, respectively (Table 3.4). The amount of binder replaced by RAP for the HMA S3 and HMA S4 mixes were found to be 31.1% and 4.1%, respectively. The WMA S3 and WMA S4 mixes were prepared with foamed binder in the laboratory using a procedure that simulates the conditions used in the local WMA plant, in terms of mixing (135°C) and compaction (127°C) temperatures. The aggregate and RAP were also dried at a lower temperature (135°C) for two hours. Controlling the aggregate temperature was found to be a critical factor. As reported by Jenkins (2000), the mixing temperature is mainly controlled by the temperature of the aggregate, not the temperature of the foamed binder. If the temperature difference between aggregates and foamed WMA is higher than 70°C, dispersion of foam becomes very fast due to rapid heat transfer (Jenkins, 2000). Table 3.5 presents the test matrix of asphalt mixes and the number of samples evaluated in each test, including performance tests (rutting, cracking and moisture-induced damage potentials).

Table 3.5 Test matrix for asphalt mixes

Mix Type	NMAS (mm)	Volumetrics Check	HWT Test	Louisiana SCB Test	TSR Test
HMA S3	19	6	2	3	6
WMA S3	19	6	2	3	6
HMA S4	12	6	2	3	6
WMA S4	12	6	2	3	6

Version: V03.40

Oklahoma Department of Transportation Mix Design Report

Asphalt Concrete, Type S3 (PG 64-22 OK) Mat'l. Code: asco009
(Material Full Name and Material Code)

Binder - Recycled ID: B2
(Design Type and Design Type ID)

PMI-Silver Star P/S # m00565
(Producer/Supplier Name and Producer/Supplier Code)

WS3pv0261480100
(Mix ID)

PMI-Silver Star (Moore, OK) - 400TPH PLANT ID # m00565-01
(Plant Name and Plant ID)

Aggregate	Producer/Supplier	% USED
1" Rock	Hanson Aggregates, WRP Inc (Davis, OK) P/S # m001985008	10
5/8" Chips	Hanson Aggregates, WRP Inc (Davis, OK) P/S # m001985008	27
3/16" Scrns.	Hanson Aggregates, WRP Inc (Davis, OK) P/S # m001985008	12
Man. Sand	Martin-Marietta (Davis, OK) P/S # m002285005	15
Sand	General Materials Inc (Oklahoma City, OK) P/S # m009215515	11
Fine R.A.P.	Contractor / Project Site P/S # Contractor	25

Warm Mix Asphalt (WMA) Technology: MAXAM (Foaming Process) qual028 Maxam Equipment m00802
(Product Name, Material Code, Producer/Supplier Name, Producer/Supplier Code)

Asphalt Cement: Asphaltic Cement Type PG 64-22 OK, acem003, Valero (Ardmore, OK), m00352
(Material Full Name, Material Code, Producer/Supplier Name, Producer/Supplier Code)

Requires Form 93-E0 signed by the Department for production use. -Oklahoma D.O.T. Materials-

Producer/Supplier:	*Hanson Aggregates, WRP Inc (Davis, OK) P/S # m001985008*	*Hanson Aggregates, WRP Inc (Davis, OK) P/S # m001985008*	*Hanson Aggregates, WRP Inc (Davis, OK) P/S # m001985008*	*Martin-Marietta (Davis, OK) P/S # m002285005*	*General Materials Inc (Oklahoma City, OK) P/S # m009215515*	*Contractor / Project Site P/S # Contractor*							
Sieve Size	**1" Rock**	**5/8" Chips**	**3/16" Scrns.**	**Man. Sand**	**Sand**	**Fine R.A.P.**			**Comb. Agg.**	***JMF***	***Min.***	***Max.***	***% Tol. (±)***
1 in (25 mm)	100	100	100	100	100	100			100	**100**	100	100	0
3/4 in (19 mm)	76	100	100	100	100	100			98	**98**	91	100	7
1/2 in (12.5 mm)	31	81	100	100	100	99			88	**88**	81	95	7
3/8 in (9.5 mm)	9	46	100	100	100	96			75	**75**	68	82	7
#4 (4.75 mm)	2	8	100	96	100	81			60	**60**	53	67	7
#8 (2.36 mm)	2	2	77	59	99	61			45	**45**	40	50	5
#16 (1.18 mm)	2	2	49	32	98	47			34	**34**	30	38	4
#30 (.600 mm)	2	2	32	18	91	37			27	**27**	23	31	4
#50 (.300 mm)	2	2	22	11	54	28			18	**18**	14	22	4
#100 (.150 mm)	2	2	15	6	10	17			9	**9**	6	12	3
#200 (.075 mm)	1.3	1.5	11.1	3.9	1.4	10.6			5.3	**5.3**	3.3	7.3	2
AC Content %						5.5			4.4	**4.4**	4.0	4.8	0.4

Warm Mix Asphalt (WMA) Additive % 2.0

	°F (°C)	Required
Mix temperature @ discharge from mixer:	275 (135)	± 20 °F (± 10 °C)
Optimum roadway compaction temperature:	260 (127)	
Laboratory mixing temperature:	325 (163)	
Laboratory compaction temperature:	300 (149)	

Tests on Asphalt Cement	Found
Specific Gravity @ 77 °F	1.0100

Requires Form 93-E0 signed by the Department for production use. -Oklahoma D.O.T. Materials-

Tests on Compressed Mixtures (@ Design AC)

	# Gyr.	% Density of Gmm	% Density Required
Nini	6	90.2	85.5 - 91.5
Ndes	50		96.0

Tests on Aggregates		Required		Units
Durability Index	80	40	min.	%
F.A.A. %U		N/A		%
Flat and Elongated	0	10	max.	%
Fractured Faces	100/100	85/80	min.	%
Insoluble Residue		N/A		%
LA Abrasion	24	40	max.	%
Micro-Deval	9.7	N/A		%
Permeability	0.2	12.5	max.	10^{-5}cm/s
Sand Equivalent	79	40	min.	%
Pba	0.42			
IOC	0.50			%
Gse	2.716			
Gsb	2.686			
Specimen Weight	4940			g

Tests on Compressed Mixtures

%AC	Gmb	Gmm	% Density of Gmm	% Density Required Design / Field	% VMA	% VMA Required Design / Field	% VFA	% VFA Required
4.3	2.426	2.532	95.8		13.6		69.1	
4.8	2.447	2.512	97.4	96.0 / 94.5 - 97.4	13.3	13.5 / 13.0	80.5	70 - 75
5.3	2.468	2.493	99.0		13.0		92.3	

Dust Prop.	Dust Prop. Req.
1.4	
1.2	0.6 - 1.6
1.1	

ITS (PSI) 95 N/A min.

TSR 0.83 0.80 / 0.75 min. (Design / Field)

Compacted Wt. (lbs/sy/1" thick) = 111.2 @ 4.4 % Asphalt Cement
3.0 % New Asphalt Cement

Hamburg Rut Test Depth (mm) 3.95 12.50 max. @ 10,000 cycles

MEETS SPECIFICATION REQUIREMENTS PER SPECIAL PROVISION 708-26(a-f) 09

Comments:

Last Modified By: Schratwieser, Edward P. eschratw
(User Name and User ID)

Date: 8/31/2016
(mm/dd/yyyy)

Figure 3.2 Mix Design for S3 Mix

Version: V03.40

Oklahoma Department of Transportation Mix Design Report

Asphalt Concrete, Type S4 (PG 64-22 OK) Mat'l. Code: asco012
(Material Full Name and Material Code)

Insoluble - Recycled ID: I2
(Design Type and Design Type ID)

PMI-Silver Star P/S # m00565
(Producer/Supplier Name and Producer/Supplier Code)

WS4pv0261600700
(Mix ID)

PMI-Silver Star (Moore, OK) - 400TPH PLANT ID # m00565-01
(Plant Name and Plant ID)

Aggregate	Producer/Supplier	% USED
5/8" Chips	Hanson Aggregates, WRP Inc (Davis, OK) P/S # m001985008	35
Man. Sand	Martin-Marietta (Davis, OK) P/S # m002285005	36
3/16" Scrns.	Hanson Aggregates, WRP Inc (Davis, OK) P/S # m001985008	10
Sand (Unlisted Source)	General Materials Inc., 63rd St.(Oklahoma City, OK)	14
Fine R.A.P.	Contractor / Project Site P/S # Contractor	5

Warm Mix Asphalt (WMA) Technology: MAXAM (Foaming Process) qual028 Maxam Equipment m00802
(Product Name, Material Code, Producer/Supplier Name, Producer/Supplier Code)

Asphalt Cement: Asphaltic Cement Type PG 64-22 OK, acem003, Trumbull Asphalt Co (OKC, OK), m00354
(Material Full Name, Material Code, Producer/Supplier Name, Producer/Supplier Code)

Requires Form 93-E0 signed by the Department for production use. -Oklahoma D.O.T. Materials-

Producer/Supplier:	Hanson Aggregates, WRP Inc (Davis, OK) P/S # m001985008	Martin-Marietta (Davis, OK) P/S # m002285005	Hanson Aggregates, WRP Inc (Davis, OK) P/S # m001985008	General Materials Inc., 63rd St.(Oklahoma City, OK)	Contractor / Project Site P/S # Contractor				
Sieve Size	5/8" Chips	Man. Sand	3/16" Scrns.	Sand (Unlisted Source)	Fine R.A.P.	Comb. Agg.	JMF	Min.	Max.	% Tol. (±)
3/4 in (19 mm)	100	100	100	100	100	100	**100**	100	100	0
1/2 in (12.5 mm)	80	100	100	100	93	93	**93**	86	100	7
3/8 in (9.5 mm)	55	100	100	100	81	83	**83**	76	90	7
#4 (4.75 mm)	13	94	100	100	58	65	**65**	58	72	7
#8 (2.36 mm)	3	54	81	99	41	45	**45**	40	50	5
#16 (1.18 mm)	3	29	57	97	31	32	**32**	28	36	4
#30 (.600 mm)	2	16	40	91	25	24	**24**	20	28	4
#50 (.300 mm)	2	10	29	66	19	17	**17**	13	21	4
#100 (.150 mm)	2	6	20	17	12	8	**8**	5	11	3
#200 (.075 mm)	1.9	3.9	14.9	1.1	7.7	4.1	**4.1**	2.1	6.1	2
AC Content %					4.7	4.9	**4.9**	4.5	5.3	0.4

Warm Mix Asphalt (WMA) Additive % 2.0

	°F (°C)	Required
Mix temperature @ discharge from mixer:	275 (135)	± 20 °F (± 10 °C)
Optimum roadway compaction temperature:	260 (127)	
Laboratory mixing temperature:	325 (163)	
Laboratory compaction temperature:	300 (149)	

Tests on Asphalt Cement	Found
Specific Gravity @ 77 °F	1.0100

Requires Form 93-E0 signed by the Department for production use. -Oklahoma D.O.T. Materials-

Tests on Compressed Mixtures (@ Design AC)

	# Gyr.	% Density of Gmm	% Density Required
Nini	6	89.0	85.5 - 91.5
Ndes	50		96.0

Tests on Aggregates		Required		Units
Durability Index	74	40	min.	%
F.A.A. %U		N/A		%
Flat and Elongated	0	10	max.	%
Fractured Faces	100/100	85/80	min.	%
Insoluble Residue	63.5	30	min.	%
LA Abrasion	27	40	max.	%
Micro-Deval	12.1	N/A		%
Permeability	3.1	12.5	max.	10^{-5}cm/s
Sand Equivalent	80	40	min.	%
Pba	0.46			
IOC	0.50			%
Gse	2.701			
Gsb	2.668			
Specimen Weight	4870			g

Tests on Compressed Mixtures

%AC	Gmb	Gmm	% Density of Gmm	% Density Required Design / Field	% VMA	% VMA Required Design / Field	% VFA	% VFA Required
4.8	2.395	2.500	95.8		14.5		71.0	
5.3	2.412	2.481	97.2	96.0 / 94.5 - 97.4	14.4	14.5 / 14.0	80.6	72 - 77
5.8	2.426	2.462	98.5		14.3		89.5	

Dust Prop.	Dust Prop. Req.
0.9	0.6 - 1.6
0.8	
0.8	

ITS (PSI) 123.1 N/A min.
TSR 0.93 0.80 / 0.75 min. (Design / Field)
Compacted Wt. (lbs/sy/1" thick) = 109.8 @ 4.9 % Asphalt Cement
4.7 % New Asphalt Cement

Hamburg Rut Test Depth (mm) 2.06 12.50 max. @ 10,000 cycles

MEETS SPECIFICATION REQUIREMENTS PER SPECIAL PROVISION 708-26(a-f) 09

Comments:

Last Modified By: Schratwieser, Edward P. eschratw
(User Name and User ID)

Date: 6/17/2016
(mm/dd/yyyy)

Figure 3.3 Mix Design for S4 Mix

3.2.3.2. Mix Design Volumetrics

The AASHTO R 35 design method is based on the volumetric properties i.e., air voids (AV), voids in the mineral aggregate (VMA), and voids filled with asphalt (VFA) of the asphalt mixes (AASHTO, 2013). The aggregate gradations of S3 and S4 mixes were maintained in a way to satisfy the AASHTO limits for VMA and VFA (AASHTO, 2013). A summary of the volumetric properties of control HMA S3 mix and control HMA S4 mix are shown in the Table 3.6 and Table 3.7, respectively. For calculating the percent air voids of asphalt mixes, theoretical maximum specific gravity (G_{mm}) and bulk specific gravity (G_{mb}) of asphalt mixes were determined. The G_{mm} of loose asphalt mixes was determined by the Rice density test in accordance to AASHTO T 209 (AASHTO, 2012). The bulk specific gravity (G_{mb}) values of compacted asphalt samples were determined by AASHTO T 166 method (AASHTO, 2010). The term G_{mb} is defined as the ratio of the mass of a unit volume permeable material to the same volume gas free distilled water in the air at 25°C. About 4,800 g of loose asphalt mixes were used to prepare compacted samples for G_{mb} testing. In the Superpave® Gyratory Compactor (SGC), 50 gyrations were used to compact the asphalt samples considering light traffic condition to obtain a final height of 115 ± 5 mm. The percent air voids were calculated using Equation 3.1.

$$\%\ \text{Air Voids} = \frac{G_{mm}-G_{mb}}{G_{mm}} * 100\% \qquad (3.1)$$

Table 3.6 Summary of volumetric properties of HMA S3 mix

Volumetric Properties	Values	Required
Maximum Specific Gravity, G_{mm}	2.499	
Virgin Binder Type	PG 64-22	
Total Binder Content (%)	4.5	
Virgin Binder Content (%)	3.1	
Voids in the Mineral Aggregate, VMA (%)	13.5	Minimum 13.0
Voids Filled with Asphalt, VFA (%)	71.4	70-75
Absorbed Binder, P_{ba} (%)	0.42	

Table 3.7 Summary of volumetric properties of HMA S4 mix

Volumetric Properties	Values	Required
Maximum Specific Gravity, G_{mm}	2.496	
Virgin Binder Type	PG 64-22	
Total Binder Content (%)	4.9	
Virgin Binder Content (%)	4.7	
Voids in the Mineral Aggregate, VMA (%)	14.5	Minimum 14.0
Voids Filled with Asphalt, VFA (%)	73.0	72-77
Absorbed Binder, P_{ba} (%)	0.46	

3.2.3.3. Statistical Analysis

The average percent air voids for volumetric samples of WMA S3 and WMA S4 mixes were compared with those of HMA S3 and HMA S4 mixes, respectively. Two-tail t-tests were conducted to identify the statistical difference between WMA mixes and their HMA counterparts with respect to average percent air voids at 95% confidence level. Based on the statistical analyses, adjustments in the optimum binder content were suggested for the WMA mixes, when significant statistical differences were observed.

3.3. Sample Preparation

Cylindrical samples are required for SCB, IDT and HWT testing. The asphalt specimens for all performance tests were prepared in the laboratory using a SGC. The target air voids were 7.0±0.5% based on the densities typically obtained in the field. After mixing, bulk asphalt mixes were short-term aged in accordance with AASHTO R 30 in order to simulate the conditioning of plant-produced mix (AASHTO, 2002). The SGC was operated at the height mode during compaction to obtain the desired air voids under a specific height. After compaction, volumetric tests were conducted to check the actual air voids in accordance with AASHTO T 166 (AASHTO, 2010). Compacted samples were then sawed to the sizes required for conducting the aforementioned tests (SCB, IDT and HWT). At least 6 samples were prepared for each test, considerably higher than the required numbers to account for samples discarded due to any imperfections. The test samples having air voids within the target air voids tolerance (7.0%±0.5%) were selected *and used in testing.*

Figure 3.7, Figure 3.8, and Figure 4.9 show photographic views of samples immediately after compaction and the test-ready samples prepared for the ITS, SCB, and HWT tests, respectively.

Figure 3.7 ITS Sample preparation

Figure 3.8 SCB Sample Preparation

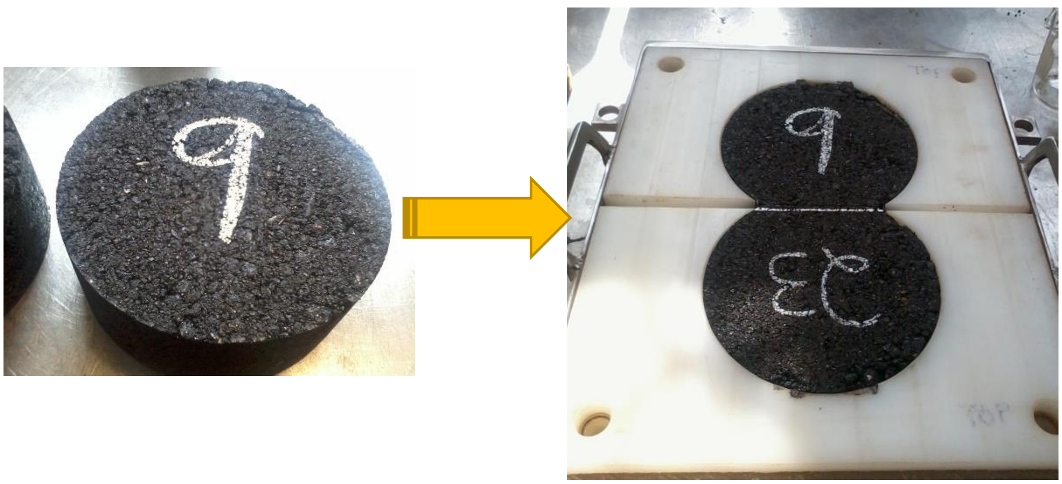

Figure 4.9 HWT Sample Preparation

3.4. Laboratory Testing of Asphalt Mixes

3.4.1. Semi-Circular Bend (SCB) Test (Louisiana Method)

Several researchers have reported that the SCB test can be used for the evaluation of fatigue cracking of asphalt mixes (Al-Qadi et al.2015, Pirmohammad and Ayatollahi, 2015; Tang, S., 2014; Aragao and Kim, 2012; Biligiri et al., 2012a; Biligiri et al., 2012b; Kim et al., 2012; Huang et al., 2011; Mohammad et al., 2011; Liu, 2011; Hassan et al., 2010; Perez-Jimenez et al., 2010; Li et al., 2010a; Li et al., 2010b; Huang et al., 2009; Tarefder et al., 2009; Wu et al. ,2005; Lie et al., 2004). This test can be conducted on laboratory-compacted samples as well as on field cores. SCB test was conducted by applying a monotonically increasing load on a semi-circular sample until failure as per AASHTO TP 105 (AASHTO, 2015) standard test method (Figure 3.10, Figure 3.11, and

Figure 3.12). According to Biligiri et al. (2012), SCB test on pavement cores can be used to estimate the residual or remaining life of flexible pavements. The SCB test method for asphalt mixes is relatively new and currently being investigated by several DOTs to verify its feasibility for screening asphalt mixes with regard to fatigue life. While a standard test method, AASHTO TP-105 is available for conducting the SCB test, this standard is not uniformly followed by different state DOTs (AASHTO, 2015). Specifically, Illinois (Ozer, 2016) and Louisiana (Kim et al., 2012) have come up with their own SCB test method and data analysis procedures.

In this study, the Louisiana SCB tests were conducted as per ASTM D 8044 (ASTM, 2016). This method characterizes the cracking resistance of asphalt mixes at an intermediate temperature (25°C in this study). These tests were conducted on half-disk-shaped specimens having a diameter of 150 mm and a thickness of 50 mm. At first, samples having a 150 mm diameter and 120 mm height were prepared using SGC. Then each sample was saw cut to produce four half circle specimens with a diameter of 150 mm and a thickness of 50 mm. The Louisiana SCB tests were conducted on specimens with three different notch depths, namely 25.4, 31.8, and 38.0 mm. For each notch depth, three replicate specimens were prepared to check the repeatability of test results. As shown in Figure 3.10, the specimens were loaded monotonically at a rate of 0.5 mm/min using a three-point flexural apparatus (Kim et al., 2012). The fracture resistance was analyzed based on an elasto-plastic fracture mechanics concept of critical strain energy release rate (Mohammad et al., 2011; Wu et al., 2005). In order to determine the fatigue resistance of an asphalt mix, critical strain energy release rate, J-integral (J_c), was calculated from the SCB test data. The method for computing the J_c is illustrated in Figure . To determine J_c, the strain energy at failure (*U*) is calculated from the load-vertical deformation curve. The area under the load-vertical deformation curve until the peak load (shaded portion in the Figure) is equivalent to *U*. Then, J_c is computed using the specimen thickness and rate of change of *U* with notch dept (d*U*/d*a*), which is the slope of the *U* versus notch depth curve in Figure . Mathematically, J_c can be represented by Equation 3.2. The higher the J_c value, the higher the fatigue resistance of an asphalt mix (Kim et al., 2012).

$$J_c = \frac{-1}{b}\left(\frac{dU}{da}\right) \tag{3.2}$$

where, J_c = critical strain energy release rate (kJ/m^2), b = specimen thickness (mm), a = notch depth (mm), U = strain energy at failure or peak load (kN-mm).

Figure 3.10 Semi-Circular Bend (SCB) test (a) Setting the test; (b) Test in Progress

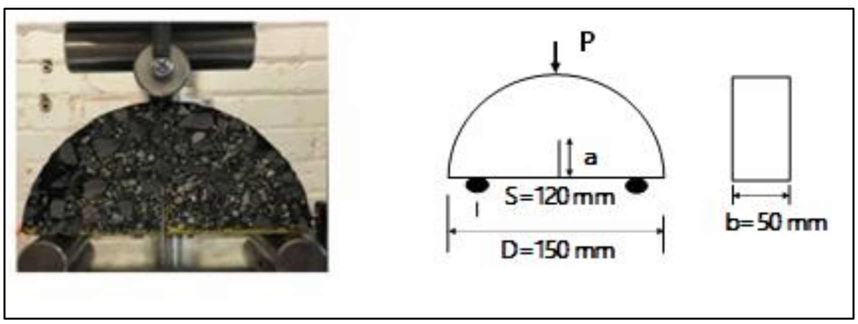

Figure 3.11 Test Setup and Dimensions of a SCB Sample

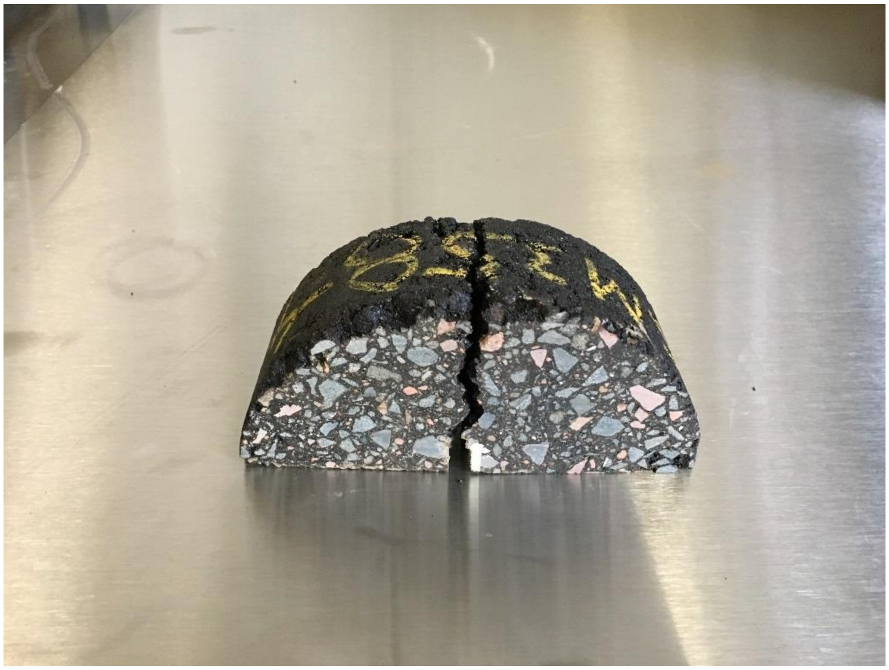

Figure 3.12 SCB Sample After Testing

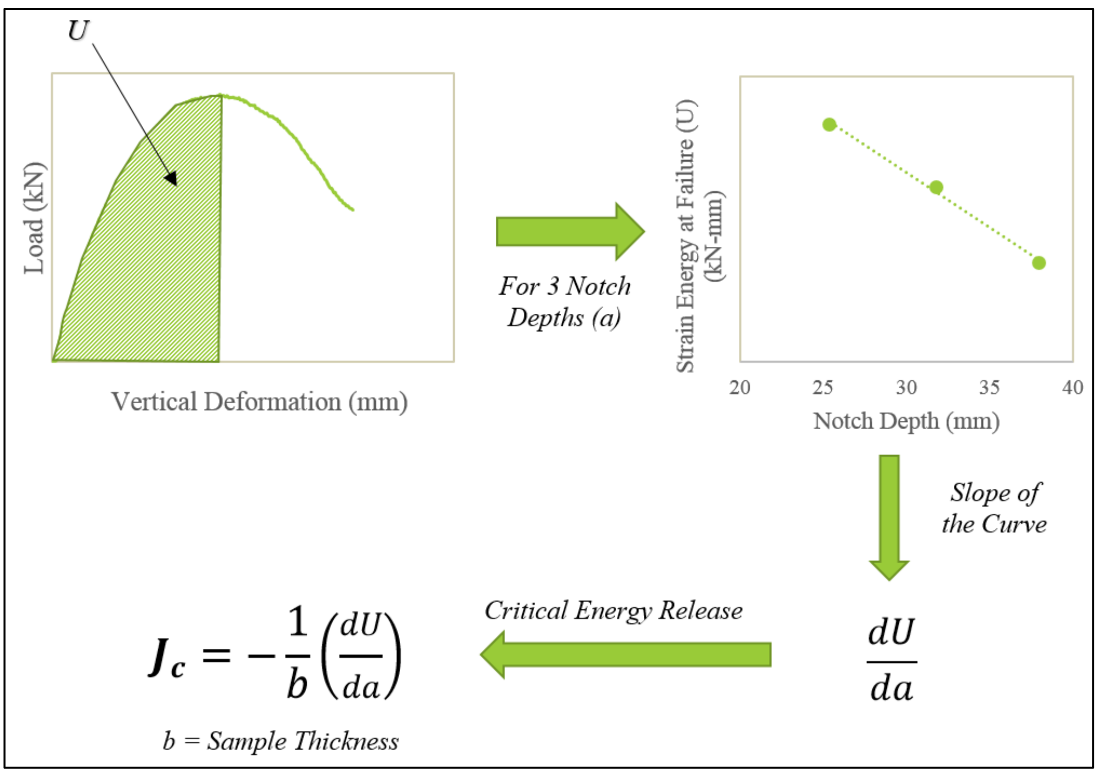

Figure 3.13 Computation of J_c using SCB Test Results

The SCB test results can also be analyzed to determine the flexibility index (FI) (Al-Qadi, 2015). The concept of flexibility index facilitates characterization of asphalt mixes based on their post-crack performance. "*The FI describes the fundamental fracture* *processes consistent with the size of the crack tip process zone* (Al-Qadi, 2015)." Different parameters involved in the calculation of the *FI* are given in Figure 3.14. Theoretically, *FI* can be expressed by Equation 3.3.

$$FI = A \times (\text{Fracture Energy/slope at inflection}) \quad (3.3)$$

where, A = calibration coefficient for unit conversions and age shifting for lab versus plant versus field materials.

One of ehe main advantages of SCB testing is its simplicity in performing the test. Also, different notch depths can be introduced easily, and the crack propagation can be evaluated directly (Wu et al., 2005). The potential weakness of the current SCB test protocol is that only monotonic load is applied to the sample. However, the fatigue failure in pavement occurs due to cyclic loading. Therefore, application of cyclic loading to conduct SCB testing would be a better representation of the fatigue failure mechanisms in the field.

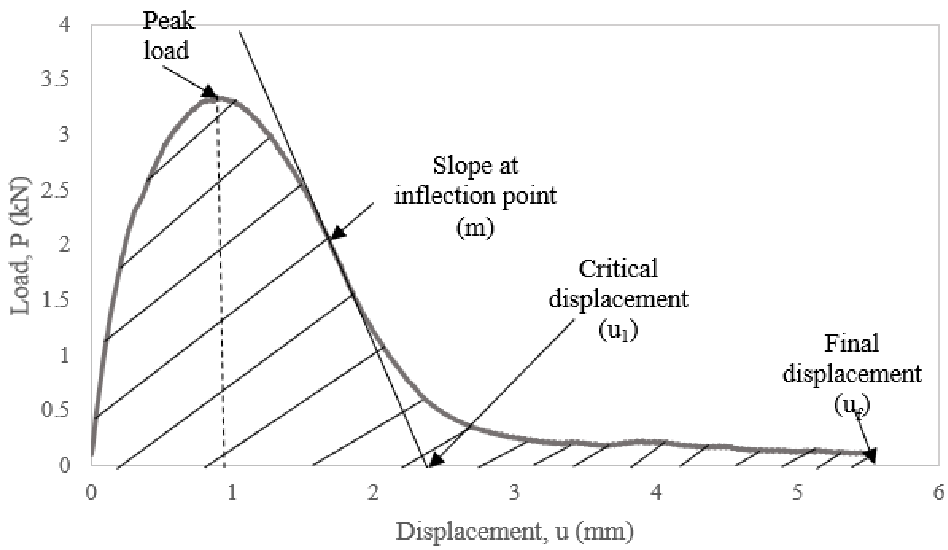

Figure 3.14 Illinois-SCB Test illustrating the Parameters Derived from the Load-Displacement curve (After Al-Qadi, 2015)

3.4.2. Indirect Tensile Strength Ratio (TSR) Test

Several researchers have proposed TSR test as a potential indicator of moisture-induced damage (Solaimanian et al., 2003; Kim et al., 2012; Shu et al., 2012). Tensile strength ratio (TSR) is the strength ratio of conditioned sample and dry sample. In this study, samples were conditioned in accordance with the AASHTO T 283 method to evaluate their TSR values (AASHTO, 2014). At least three replicate cylindrical samples having 150 mm diameter and 115± 5 mm height were prepared in the laboratory using a SGC. The compacted samples with an air void of 7.0 ± 0.5% were selected for conducting the TSR tests. To determine the moisture-induced damage potential of asphalt mixes, the indirect tensile strength (ITS) test was conducted on both dry and conditioned asphalt samples (Figure 3.15 and Figure 3.16). For simulating the moisture-induced damage in the laboratory on the compacted specimens, samples were conditioned by vacuum saturating them in water (70-80% saturation) followed by a freezing cycle (-18°C for 16 hours) and a thawing cycle (60°C water bath for 24 hours). The ITS values were calculated using Equation 3.4 (AASHTO, 2014).

$$S_t = \frac{2000 * P}{\Pi * t * D} \tag{3.4}$$

where,

S_t = Indirect tensile strength (kPa);

P = Maximum load (N);

t = Sample height immediately before test (mm);

D = Sample diameter (mm).

To calculate the TSR value, Equation 3.5 was used.

$$TSR = \frac{ITS\ of\ conditioned\ Specimens\ (ITS_{wet})}{ITS\ of\ Dry\ Specimens\ (ITS_{dry})} \tag{3.5}$$

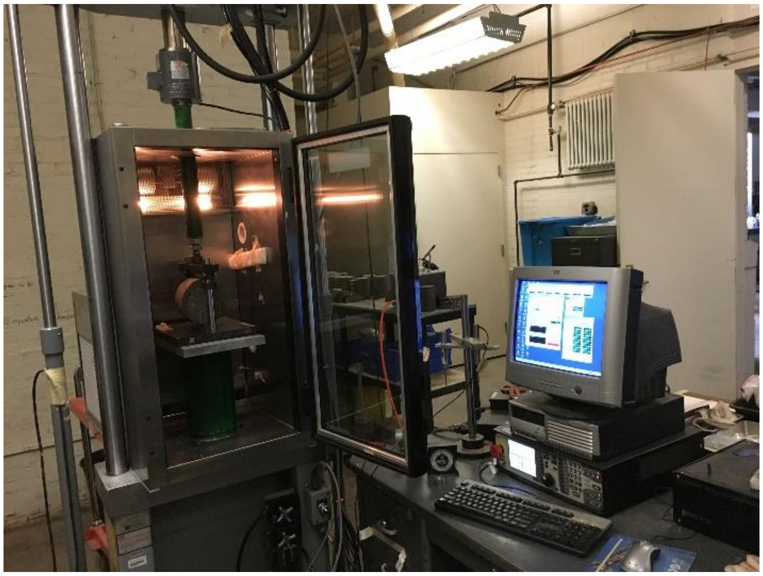

Figure 3.15 Indirect Tensile Strength (ITS) Test in Progress

Figure 3.16 ITS Sample After Testing

3.4.3. Hamburg Wheel Tracking Test

Rutting and moisture-induced damage potential of asphalt mixes were evaluated using Hamburg wheel tracking (HWT) tests in accordance with the AASHTO T 324 standard test method (AASHTO, 2016). At least four replicate specimens having 150 mm diameter and 60 mm height and 7.0% ± 0.5% air voids were compacted using a Superpave® Gyratory Compactor (SGC). Then each set of samples, consisting of two specimens, were saw-cut from the side to match the mould dimensions of the HWT device (Figure 3.9). Volumetric analyses, in accordance with AASHTO T 166 (AASHTO, 2013), were performed after compaction in order to ensure achieving the target air voids. The prepared specimens were submerged in a temperature-controlled water bath at 50°C and were repetitively loaded in a HWT device using a reciprocating steel wheel having a wheel load of 705 N and wheel pass frequency of 52 passes/minute (Figure 3.17).

Figure 3.17 HWT test in progress

The average linear speed of the moving wheel was approximately 1.1 km/h, and the wheel traveling approximately 230 mm before reversing the direction of movement. The test was automatically terminated after reaching a maximum rut depth of 20 mm or 20,000-wheel passes, whichever reached first. Deformations were measured along the length of the wheel path at 11 equally spaced points. The rut depths at the three mid-points (5^{th,} 6th, and 7th points) of the sample were considered in the analysis. For each mix, two sets (four in total) of HWT samples were tested to check the repeatability of the results. The noise in the rut depth readings were observed due to movement of HWT steel wheel on the rutted sample. Lu and Harvey (2006) proposed Equations 3.6, 3.7 and 3.8 to calculate the moving averages in eliminating the noise in HWT test results.

$$\overline{d_t}=0.40d_t + 0.25d_{t+1} + 0.15d_{t+2} + 0.10d_{t+3} + 0.10d_{t+4} \quad (1 \le t \le 5) \tag{3.6}$$

$$\overline{d_t}=0.05d_{t-5} + 0.05d_{t-4} + 0.075d_{t-3} + 0.075d_{t-2} + 0.15d_{t-1} + 0.20\, d_t + 0.15d_{t+1} + 0.075d_{t+2} + 0.075d_{t+3} + 0.05d_{t+4} + 0.05d_{t+5} \quad (5 < t < 19{,}995) \tag{3.7}$$

$$\overline{d_t}=0.40d_t + 0.25d_{t-1} + 0.15d_{t-2} + 0.10d_{t-3} + 0.10d_{t-4} \quad (19{,}995 \le t \le 20{,}000) \tag{3.8}$$

where:

d = rut depth;

t = number of wheel passes.

After analyzing the HWT test results, post-compaction deformation, creep slope, stripping slope, and stripping inflection point (SIP) were determined manually, as shown in Figure 3.18. The post-compaction deformation indicates the initial densification of asphalt pavement due to traffic. Yildirim and Kennedy (2002) suggested the rut depth at 1,000-wheel passes as the post-compaction point. The primary deformation under repeated loading is presented by this zone. After post-compaction zone, the rut depth increased almost linearly with number of wheel-passes up to a certain point. The secondary deformation under repeated loading is represented by this zone. The slope of this secondary zone is commonly known as creep slope. After secondary deformation, a rapid increase in rut depth was observed with increasing wheel-passes. This rapid deformation of HWT sample is attributed to tertiary deformation. The slope of the tertiary zone is commonly known as stripping slope. The stripping slope measures the moisture-induced damage potential of asphalt mixes. A steeper slope indicates a higher possibility of moisture damage of asphalt pavements. The intersection between the stripping slope and creep slope indicates the SIP.

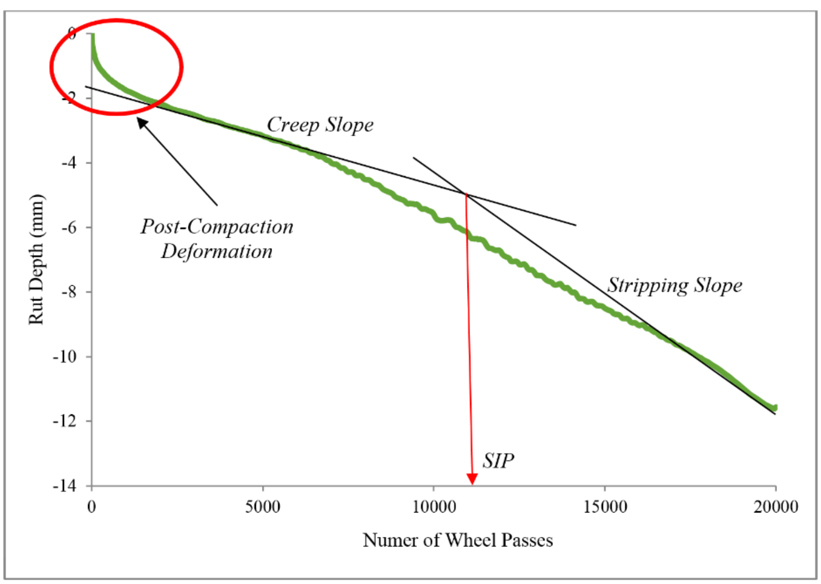

Figure 3.18 Typical Hamburg wheel tracking (HWT) test output

Figure 3.19 HWT Samples After Testing: (a) Severe moisture-induced damage; and (b) Minimum Moisture-Induced Damage

3.5. Research Approach

As discussed earlier, four mixes (2 WMA and 2 HMA), each having two types of gradations (S3 with NMAS = 19 mm and S4 with NMAS = 12.5 mm), were designed and evaluated in this project. Based on the experience gained from the previous studies conducted by the research team and the ODOT practice, the S3 and S4 mixes were designed with 25% RAP and 5% RAP, respectively. A PG 64-22 asphalt binder was used as the virgin binder in all mix designs. Materials for this project including aggregates, asphalt binders and RAP were provided by the project's industry partner (Silver Star Construction Co.). To keep the aggregate structure of all asphalt mixes consistent, the same type of aggregate (i.e., limestone) was used in all mix designs. The project's industry partner also provided the mix design sheets for the selected S3 (Figure 3.2) and S4 mixes (Figure 3.2). These designs were based on the AASHTO R35 mix design procedure and served as a reference for batching the HMA and WMA mixes in the lab. The selected mixes (S3 and S4) were produced in the laboratory using a procedure that simulates the conditions prevailing in a typical HMA and WMA plant, in terms of mixing temperature and aging (Table 3.4). The selected binder (PG 64-22) was used to produce foamed binder using a laboratory foamer (Figure). Temperature used in the foaming was comparable with that used in an asphalt plant. The foamed binder was used in producing these mixes. The mixing temperature was comparable to the temperature in a WMA plant and adjusted as necessary to achieve desired workability. The number of gyrations was kept the same as that used currently in the mix design according to the AASHTO R 35 method. This set of mix is called Set A-1 in this report. A second set of mix (called Set B) was produced in the lab based on the design sheets provided by our industry partner (i.e., based on the AASHTO R 35 method). No foamed binder was used for this case. SGC-compacted samples were prepared for both sets. Also, volumetric properties were determined for both sets and compared, as noted below. Based on the volumetric parameters determined for each set, two different scenarios were considered.

Scenario I- If the volumetric properties of the WMA samples for Set A-1 were found considerably different (e.g. difference in % air voids > 1%) from those for Set B, then it was an indicator that the AASHTO R 35 procedure could not be directly used for the design of the foamed WMA. The binder content would then be adjusted and a new set of samples produced (Set A-2), following the aforementioned approach in order to determine the appropriate level of adjustment in binder content. At least three different binder contents were tried (i.e., Set A-1, Set A-2 and Set A-3). Statistical analyses were performed on the volumetric properties of all of these samples to determine the adjustments in the binder content required for designing the foamed WMA, but following the AASTHO R 35 procedure.

Scenario II- If the difference in the volumetric properties between the two sets (i.e., Set A-1 and Set B) was found insignificant (e.g., difference in % air voids < 1%), then it was concluded that it would be appropriate to use the AASHTO R 35 procedure directly to design the foamed WMA without any adjustments. The aforementioned procedure used for the design of foamed WMAs is summarize in Figure .

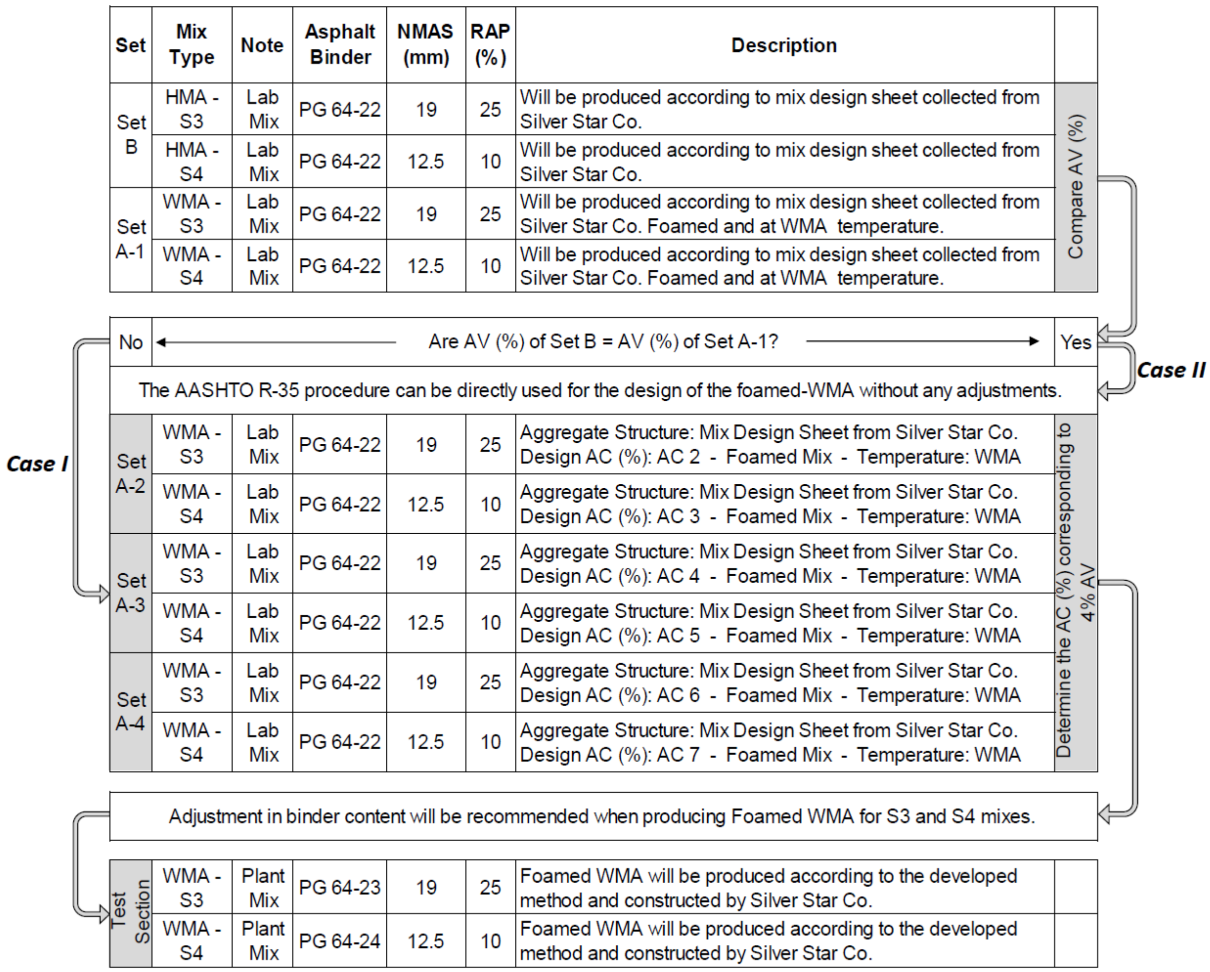

Figure 3.20 Procedure for Development of Foamed WMA Mix Design

3.6. Life Cycle Cost Analysis (LCCA)

A life cycle cost analysis (LCCA) was conducted in this project. The LCCA was conducted for a pavement section constructed using two alternatives, only if the volumetric test data showed that two different procedures were needed for the design of HMA and foamed WMA. The following alternatives were considered: (i) HMA asphalt mix designed using the current practice (i.e., AASHTO R35); and (ii) foamed WMA mix designed based on the adjusted procedure described above. Based on the differences in performance between these two types of mixes, different components of the life cycle cost analysis namely, performance period, maintenance and rehabilitation costs, user cost and salvage value were determined for the two alternatives. The LCCA for the two alternatives, if necessary, was performed for the same analysis period (e.g., 35 years).

CHAPTER 4 - RESULTS AND DISCUSSION

4.1. General

Volumetric properties as well as the results of performance tests conducted on both foamed WMA and HMA mixes containing RAP are presented and discussed in this chapter. As noted previously, performance tests included the following: rutting, cracking and moisture-induced damage potential.

4.2. Volumetric Properties of Foamed WMA and HMA Containing RAP

The volumetric properties of two control HMA mixes (HMA S3 and HMA S4) and two corresponding foamed WMA mixes (WMA S3 and WMA S4) were determined in the laboratory. The differences in volumetric properties of foamed WMA and control HMA samples were analyzed using statistical tools.

As noted in Chapter 3, typical mixing and compaction temperatures for foamed WMA mixes used in local asphalt plants are 135°C and 127°C, respectively. Therefore, volumetric properties of foamed WMA mixes (WMA S3 and WMA S4) produced at these temperatures were compared with their HMA counterparts (WMA S3 and WMA S4). The control HMA mixes were mixed and compacted at 163°C and 149°C, respectively. The same aggregate gradation (NMAS = 19.0 mm) and RAP content (25%) were used for both HMA S3 (control HMA) and WMA S3 (foamed WMA) mixes so that their volumetric properties can be compared. Similarly, the volumetric properties of both S4 mixes (HMA and foamed WMA) were compared. Both of these mixes were prepared with the same aggregate gradation (NMAS = 12.5) and RAP content (5%). The RAP contents used in these mixes are consistent with the ODOT recommendation (ODOT, 2013).

The volumetric properties of asphalt mixes are generally dictated by the percent air voids at the desired number of gyration (Huang et al., 2005; Zhao et al., 2012). As noted in Chapter 3, the percent air voids of the volumetric samples were calculated based on the theoretical maximum specific gravity (G_{mm}) of asphalt mixes and the bulk specific gravity (G_{mb}) of compacted asphalt samples. The average G_{mm} value for the HMA S3 mix was found to be 2.499. The same G_{mm} value (2.499) was observed for the WMA S3 mix. The WMA S3 mix, however, was mixed at 135°C and compacted at 127°C. These results were expected as similar aggregate gradation was used for both cases. Similar findings were reported by Hurley and Prowell (2005, 2006).

Figure 4.1 and Table 4.1 present the percent air voids for samples compacted using HMA S3 and WMA S3 mixes. For both mixes, six volumetric samples were compacted to check the repeatability of air voids. According to Table 4.1 and Figure 4.1, the average percent air voids were 4.2% for both mixes. Also, the standard deviations of percent air voids for HMA S3 and WMA S3 mixes were found as 0.1% and 0.2%, respectively. The statistical difference in the percent air voids between HMA S3 and WMA S3 samples was analyzed using a two-tail t-test. A summary of the t-test results is presented in Table 4.2. The p-value obtained from the statistical test was 0.58, which is greater than 0.05. Therefore, it is evident that at 95% confidence level, statistically the difference in percent air voids between the HMA S3 and WMA S3 samples is insignificant. According to Bonaquist (2011), the percent absorbed binder is one of the key parameters in volumetric properties of WMA mixes. The term “absorbed binder” is defined as the percent of total binder absorbed by permeable pores (on surface and near surface) of aggregates in asphalt mixes (Brown et al., 2009). Similar volumetric properties for both HMA and WMA mixes were reported when the absorbed binder content was less than one percent (Bonaquist, 2011). For S3 mixes (both HMA S3 and WMA S3) used in the present study, the percent absorbed binder was 0.42%, which is significantly lower than 1.00% (Table 3.6). Therefore, similar percent air voids were expected for both HMA S3 and WMA S3 mixes. Also, an increase in coating ability of binder due to foaming process was expected to counteract the effect of lowering the production temperatures of WMA mixes (Jones et al., 2010; Bonaquist, 2011). Therefore, compaction effort required for both HMA (HMA S3) and foamed WMA (WMA S3) samples was expected to be similar. A number of previous studies also reported similar findings (Hurley and Prowell, 2006; Wielinsk et al., 2009; Jones et al., 2010; WSDOT, 2012; Xiao et al., 2012; Malladi, 2015).

While using RAP in WMA mixes, the blending of aged binder from RAP and new binder might be hindered by the lower production temperature of WMA mixes than HMA mixes (Bonaquist, 2011). It was reported by Bonaquist (2011) that the compaction temperature of WMA mixes greater than the high-temperature PG of the RAP binder was required to ensure proper blending between the aged and new binders. From DSR tests conducted on the extracted RAP binder, the high-temperature PG of RAP was found to be 94.1°C, which was significantly lower than mixing and compaction temperatures used in this study. As the volumetric samples of WMA S3 were compacted at 127°C, which was higher than the high-temperature PG of RAP binder (94.1°C), proper blending of aged and new binder was expected for the foamed WMA mixes. Thus, insignificant differences observed in percent air voids between the HMA S3 and WMA S3 were justified. From these results it was concluded that using volumetric properties as a measure, the foamed WMA S3 mixes containing 25% RAP might be designed using the AASHTO R 35 method (AASHTO, 2013), when mixed at 135°C and compacted at 127°C.

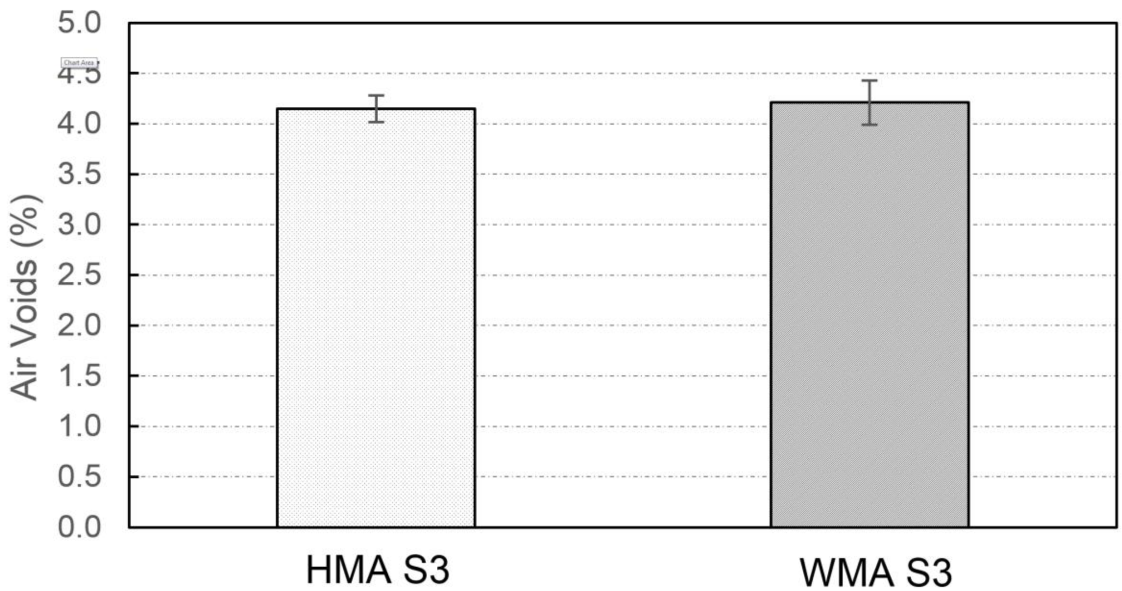

Figure 4.1 Percent air voids for HMA S3 and WMA S3 mixes

Table 4.1 Percent Air Voids for HMA S3 and WMA S3 Mixes

Mix Type	Weight in Air, W_d (g)	Saturated Surface Dry Weigh, W_{ssd} (g)	Weight in Water, W_{water} (g)	Theoretical Maximum Specific Gravity, G_{mm}	Air Voids (%)	Average Air Voids (%)	Standard Deviation (%)
HMA S3	4801.8	4814.1	2810.8	2.499	4.1	4.2	0.1
	4802.2	4813.4	2811.5		4.0		
	4800.2	4815.3	2809.4		4.2		
	4800.3	4815.7	2809.5		4.3		

Mix Type	Weight in Air, W_d (g)	Saturated Surface Dry Weigh, W_{ssd} (g)	Weight in Water, W_{water} (g)	Theoretical Maximum Specific Gravity, G_{mm}	Air Voids (%)	Average Air Voids (%)	Standard Deviation (%)
	4802.8	4815.9	2813.7		4.0		
	4801.2	4815.8	2808.1		4.3		
WMA S3	4800.8	4817.2	2819.2	2.499	3.9	4.2	0.2
	4803.6	4819.0	2810.9		4.3		
	4800.4	4818.2	2814.6		4.1		
	4817.5	4833.2	2815.2		4.5		
	4798.3	4819.4	2811.3		4.4		
	4801.0	4819.3	2815.0		4.2		

Table 4.2 t-Test results at 95% confidence interval

Parameter	HMA S3	WMA S3
Mean	4.2	4.2
t Stat.	-0.573	
P(T ≤ t) two-tail*	0.58	

* *P>0.05 = insignificant difference*

A similar trend in variations in percent air voids was also observed for the S4 mixes (HMA S4 and WMA S4) containing 5% RAP (Table 4.3 and Figure 4.2). The G_{mm} values for both S4 mixes (HMA S4 and WMA S4) were found to be 2.496. According to Table 4.3, the average percent air voids for both HMA S4 and WMA S4 mixes were found to be 4.5%. For both mixes, three volumetric samples were compacted to check repeatability of the measured air voids. Also, the standard deviations of percent air voids of HMA S4 and WMA S4 mixes were found to be 0.1% and 0.2%, respectively. Table 4.4 presents the t-test results for differences in percent air voids between the HMA S4 and WMA S4 mixes. According to Table 4.4, the p-value obtained from the t-test was 0.98, which was much higher than 0.05. Comparatively, for S4 mixes the p-value (0.98) was higher than that for S3 mixes (0.58). This could be attributed to the lower amount of RAP used in S4 mixes, as the variability in the mix properties generally increases with an increase in RAP content (Jones, 2008). Also, the percent absorbed binder both HMA S4 and WMA S4 was 0.46%, which was significantly lower than 1.00% (Table 3.7). Moreover, the compaction temperature (127°C) for the WMA S4 mix was greater than the high-temperature PG of extracted RAP binder (94.1°C). Therefore, similar volumetric properties were expected for both HMA S4 and WMA S4 mixes. Similar findings were reported by other researchers (Hurley and Prowell, 2006; Wielinsk et al., 2009; Jones et al., 2010; Bonaquist, 2011; WSDOT, 2012; Xiao et al., 2012; Malladi, 2015).

Table 4.3 Percent air voids for HMA S4 and WMA S4 mixes

Mix Type	Weight in Air, W_d (g)	Saturated Surface Dry Weigh, W_{ssd} (g)	Weight in Water, W_{water} (g)	Theoretical Maximum Specific Gravity, G_{mm}	Air Voids (%)	Average Air Voids (%)	Standard Deviation (%)
HMA S4	4796.1	4808.4	2797.6	2.496	4.4	4.5	0.1
	4799.0	4810.6	2794.0		4.7		
	4797.4	4810.3	2799.5		4.4		
WMA S4	4797.1	4804.3	2795.0	2.496	4.4	4.5	0.2
	4799.0	4810.3	2793.0		4.7		
	4797.2	4811.2	2799.5		4.5		

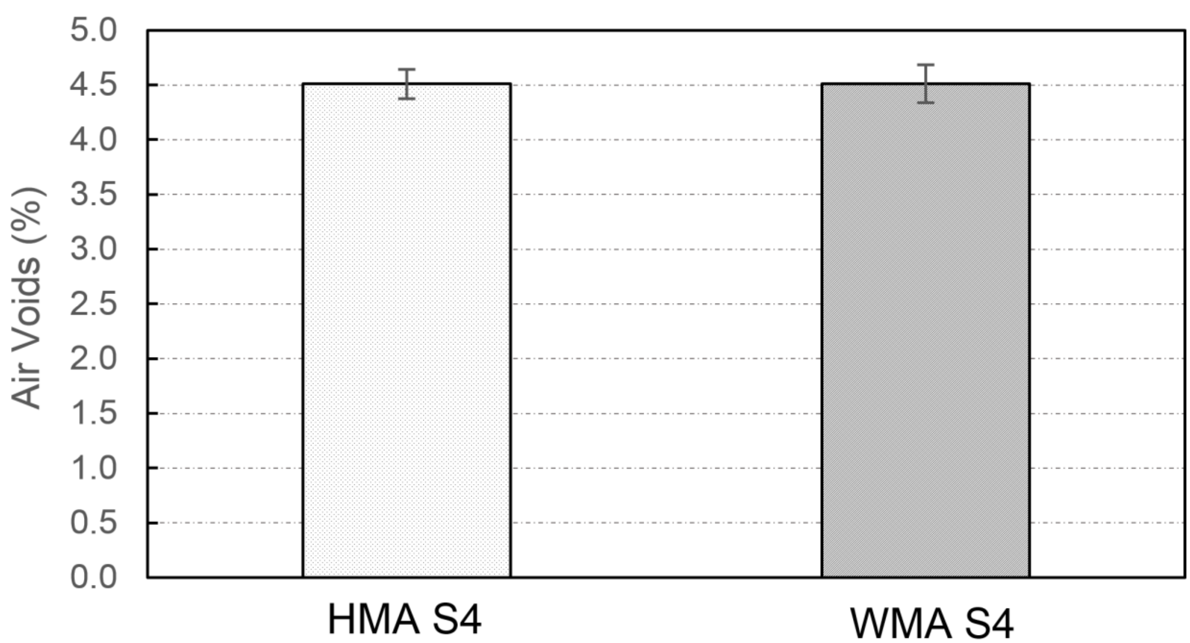

Figure 4.2 Percent Air Voids for HMA S4 and WMA S4 Mixes

Table 4.4 t-Test results at 95% confidence interval

Parameter	HMA S4	WMA S4
Mean	4.5	4.5
t Stat	-0.03	
P(T ≤ t) two-tail*	0.98	

* *P>0.05 = insignificant difference*

In conclusion, the foaming process used in WMA increased the coating ability of the binder. This increased coating ability of binder counteracted the effects of lower production temperature of WMA mixes. Thus, the control HMA and foamed WMA mixes produced at temperatures generally used in local asphalt plants showed statistically identical volumetric properties. Also, the results presented above supported the findings of the previous studies. It was confirmed that while incorporating RAP in WMA mixes, the compaction temperature should be greater than the high temperature grade of the extracted RAP binder. This ensures adequate blending of active binder from RAP with the new binder. Furthermore, the results presented above supported the view that the temperature difference between foamed binder and aggregates should be kept as low as possible during mixing so as to avoid rapid collapse of foams when they come in contact with aggregates.

4.3. Performance Tests Conducted on Asphalt Mixes

In spite of similar volumetric properties of HMA mixes and foamed WMA mixes, their performance can be significantly different. A reduction in mixing and compaction temperature would cause reduced aging of the asphalt binder. Also, the level of drying of aggregates in WMA mixes is expected to be different than that in HMA mixes. Accordingly, laboratory performance tests were conducted on samples prepared using foamed WMA mixes and HMA mixes, and the results were compared. The following performance tests were used for this purpose: rutting, cracking and moisture-induced damage potential. The results are presented in this section.

4.3.1. Cracking Resistance

Fatigue cracking is one of the common distresses observed in the asphalt pavements. Repeated traffic loading is believed to the primary cause of this distress (Colombier, 1997; Baek J., 2010; Moreno and Rubio, 2013). As discussed in Chapter 2, different state DOTs currently use different test methods to characterize fatigue resistance of asphalt mixes. In the present study, Louisiana SCB tests were used to evaluate fatigue performance. These tests were conducted in accordance with ASTM D 8044 method (ASTM, 2013). Accordingly, samples with three different notch depths (25.4 mm, 31.8 mm and 38.0 mm) were prepared and tested under a loading rate of 0.5 mm/min.

Figure 4.3 presents the variation of average strain energy at failure with respect to different notch depths for both HMA S3 and WMA S3 mixes. The repeatability of the test results was checked wusing coefficient of variance (COV) of average strain energy. For each notch depth at least three specimens were tested. The COV of average strain energy was found to be less than 15% for each set of samples (Table 4.5), which is within the expected range (Kim et al., 2012; Khan, 2016). From Figure 4.3 it is evident that the average strain energy decreased with an increase in notch depth for both mixes. This is due to a reduction in the effective loading area with increasing in notch depths (Kim et al., 2012; Khan, 2016; Saeidi and Aghayan, 2016). For all notch depths, the foamed WMA S3 samples exhibited a higher average strain energy compared to the HMA S3 samples. For example, at 25.4 mm notch depth, the average strain energy at failure for the HMA S3 and WMA S3 specimens were found to be 0.33 J and 0.50 J, respectively. As noted by Kim et al. (2012), samples with a higher average strain energy might not exhibit a higher cracking resistance (Kim et al., 2012). Instead, the strain energy release rate (Jc) is found to be a more representative parameter for characterizing cracking resistance of asphalt mixes. Figure 4.4 presents the Jc values of HMA S3 and WMA S3 mixes. From Figure 4.4, a higher Jc value was also observed for the WMA S3 samples compared to that of the HMA S3 samples. Thus, the foamed WMA S3 mix containing 25% RAP was found to exhibit a higher cracking resistance than the HMA mix. As noted earlier, a lower production temperature for WMA mixes was expected to produce softer mixes (lower stiffness) due to less aging (Hurley and Prowell, 2006; Alhasan et al., 2014; Malladi, 2015). Consequently, a lower stiffness of the WMA mixes was expected to have a higher cracking resistance than the HMA mixes (Lee et al., 2009; Hill et al., 2012b; Kim et al., 2012; Zhao et al., 2013; Dong et al., 2017; Valdes-Vidal et al., 2018). A Jc value of 0.5 to 0.60 kJ/m2 was recommended to ensure sufficient cracking resistance of asphalt mixes (ASTM, 2013). However, both HMA S3 and WMA S3 were found to have lower Jc values than the minimum requirements specified in ASTM D 8044 (Figure 4.4). The WMA S3 samples failed to satisfy the minimum requirement only by a small margin of 0.03 kJ/m^2.

Table 4.5 SCB Tests' coefficients of variation (%) for U values for HMA S3 and WMA S3

	Notch Depth (mm)		
	25.4	31.8	38.0
Mix Type	COV (%) for Measured U		
HMA S3	9	4	3
WMA S3	13	14	12

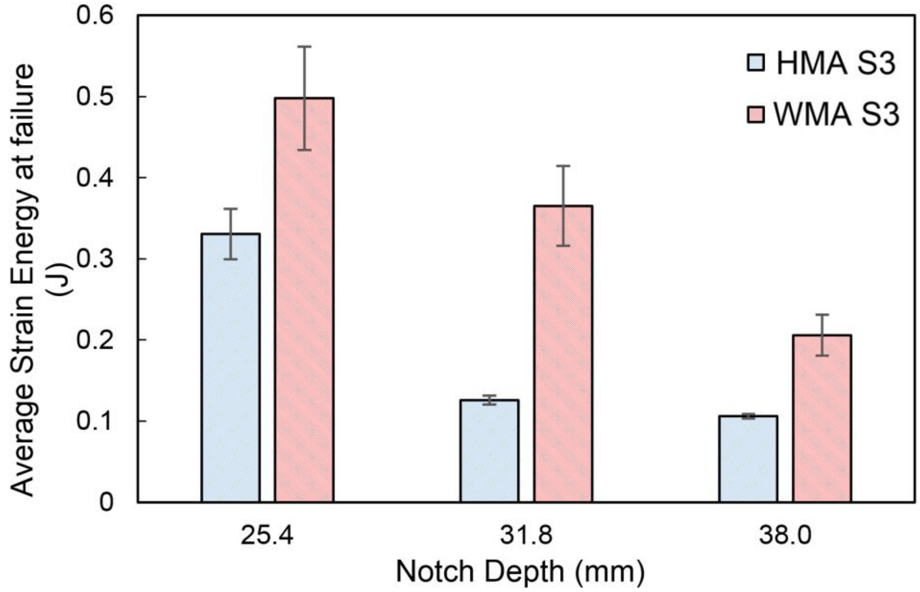

Figure 4.3 Average strain energy at failure (U) for HMA S3 and WMA S3 mixes

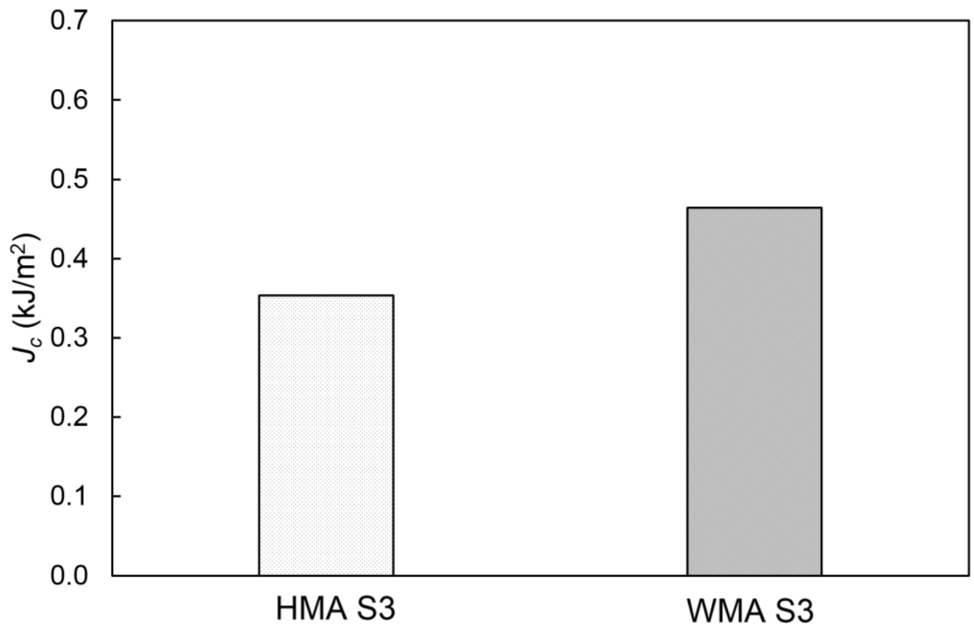

Figure 4.4 J_c Values for HMA S3 and WMA S3 mixes

Figure 4.5 presents the variations in average strain energy with notch depths for HMA S4 and WMA S4 mixes. From Figure 4.5 it was observed that the average strain energy decreased with an increase in notch depths for both mixes due to the reduction in effective loading areas (Kim et al., 2012; Khan, 2016; Saeidi and Aghayan, 2016). The variability of the measured strain energies for both mixes (HMA S4 and WMA S4) with notch depths are presented in Table 4.6. From Table 4.6 it was observed that the values of the COVs calculated for the measured average strain energies were less than 15% for all notch depths, which is an indicator of acceptable repeatability. The strain energy release rate (J_c) values for HMA S4 and WMA S4 mixes are presented in Figure 4.6. From Figure 4.6 it can be observed that the J_c value measured for WMA S4 (0.60 kJ/m^2) was found to be considerably (54%) higher than that for HMA S4 mix (0.39 kJ/m^2). Also, the measured J_c value for WMA S4 samples met the minimum requirement (>0.5) recommended by ASTM D 8044 (ASTM, 2013). However, the minimum J_c value requirement was not met by the HMA S4 samples. Therefore, foamed WMA S4 mix, which contained 5% RAP, showed a relatively higher fatigue resistance compared to its HMA counterpart. Several other researchers also reported similar findings in terms of the fatigue resistance of WMA compared to that of HMA mixes (Kim et al., 2012; Zhao et al., 2013; Yu et al., 2016; Dong et al., 2017).

From the comparison of J_c values of the HMA S3 with HMA S4 mixes with that of the WMA S3 with WMA S4 mixes it is evident that S4 mixes had higher J_c values compared to S3 mixes. This can be due to an increase in brittleness of asphalt mixes due to incorporation of a high amount of RAP in S3 mixes. The amount of RAP content for S3 mixes (25%) was much higher compared to that of S4 mixes (5%). Based on previous studies, the cracking resistance of asphalt mixes is expected to reduce with aging due to oxidation or interaction between oxygen and asphalt binder (Shu et al., 2008; Gue et al., 2014; Lu and Saleh, 2016; Saeidi and Aghayan, 2016). Therefore, incorporation of a high percentage of RAP might lower the cracking resistance of asphalt mixes. However, McDaniel et al. (2000), Huang et al. (2004) and Ghabchi et al. (2016) reported that incorporation of RAP up to a certain limit has a positive effect on the cracking resistance of asphalt mixes. Furthermore, the finer mixes (S4 mixes) was expected to show a higher fatigue resistance than coarser mixes (S3 mixes) due to a higher binder content and differences in crack propagation mechanism (Barman et al., 2018). In case of a coarser mix, crack generally propagates within the mastic (composed of the asphalt, filler and fine aggregates fraction) resulting in a lower fracture energy. However, for finer mixes crack propagates through the aggregate as shown in Figure 4.7. As a result, S4 mixes showed a higher cracking resistance compared to S3 mixes.

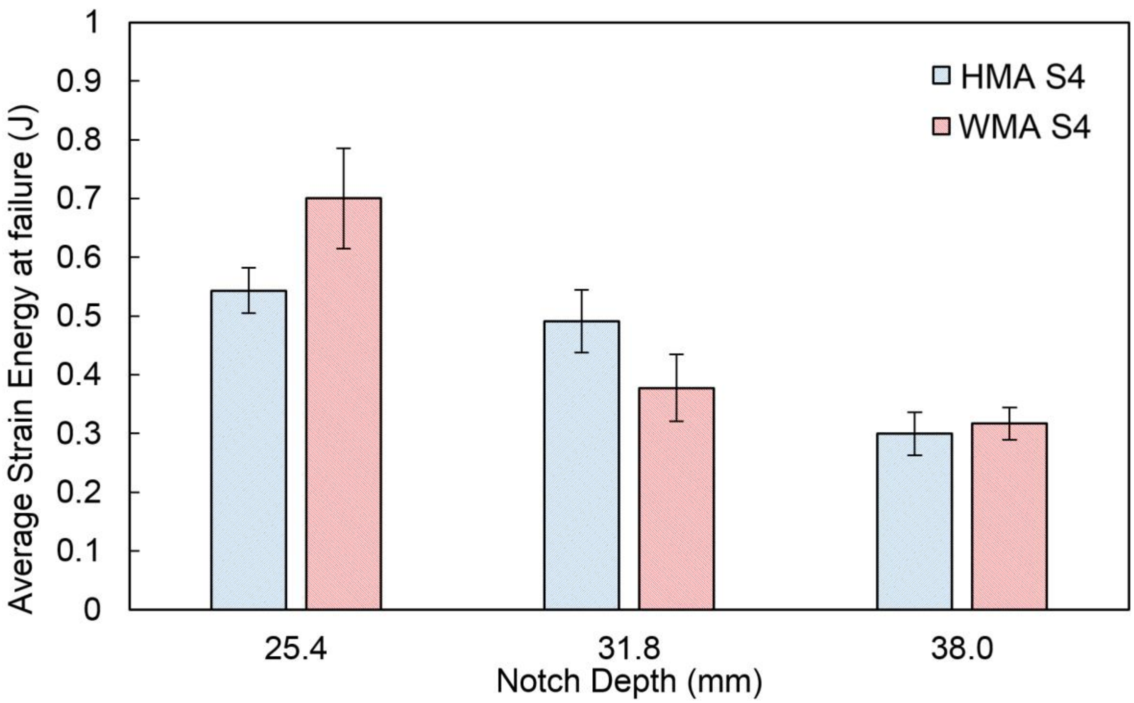

Figure 4.5 Average strain energy at failure (U) for HMA S4 and WMA S4 mixes

Table 4.6 SCB Tests' coefficients of variation (%)for U values for HMA S4 and WMA S4 mixes

Mix Type	Notch Depth (mm)		
	25.4	31.8	38.0
	COV (%) for Measured U		
HMA S4	7	11	12
WMA S4	12	15	9

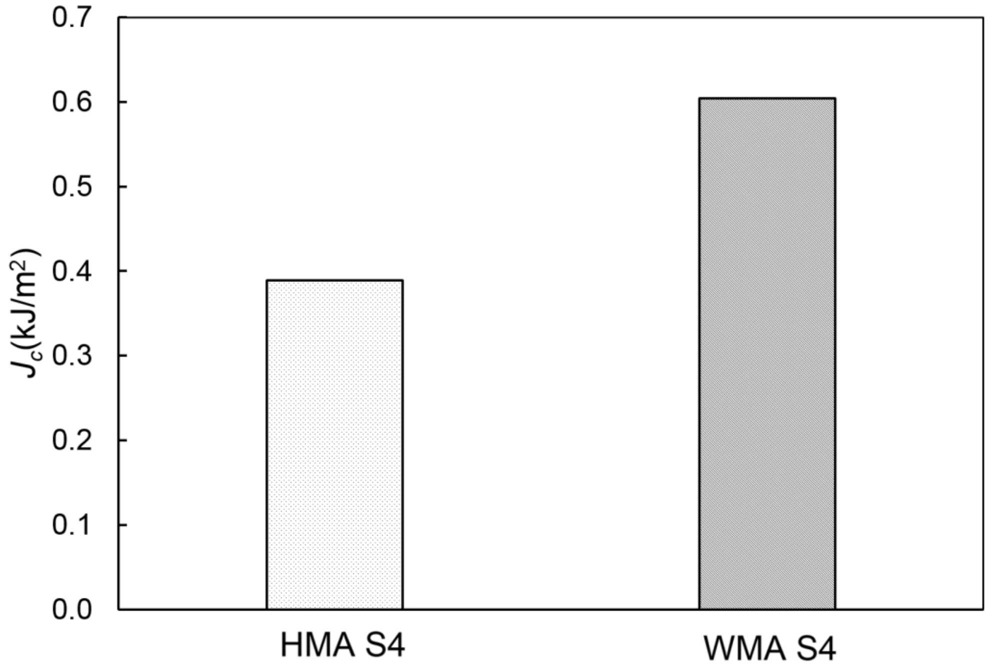

Figure 4.6 J_c Values for HMA S4 and WMA S4 mixes

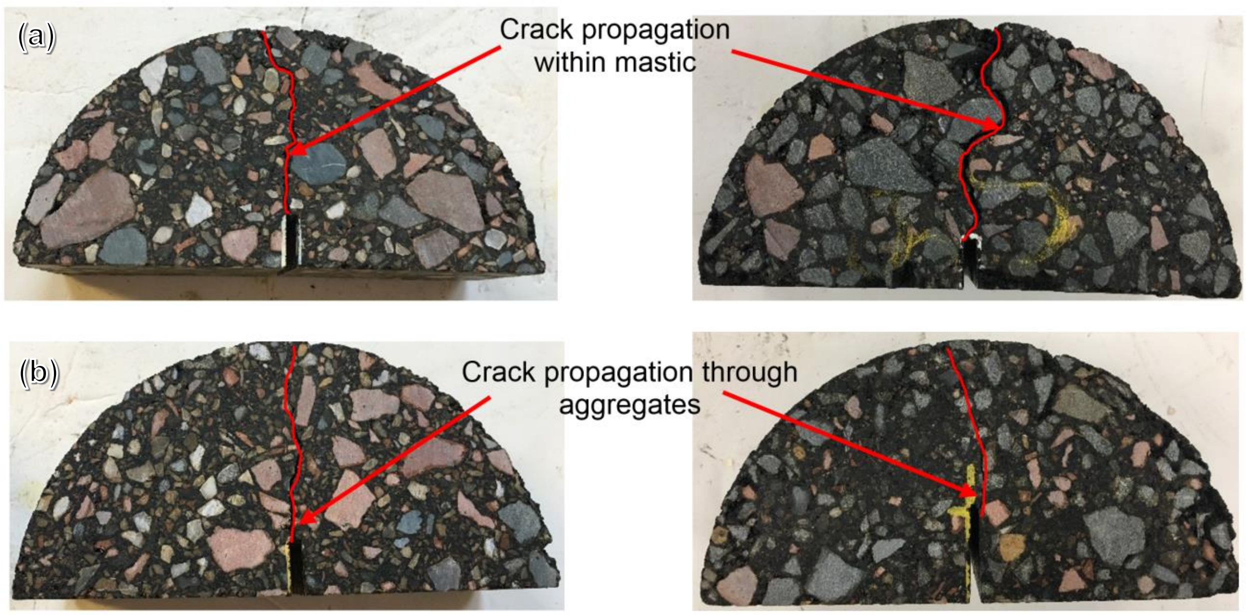

Figure 4.7 Crack propagation patters observed for (a) S3; and (b) S4 mixes

4.3.2. Rutting Performance

The permanent deformation or rutting is an important factor affecting the serviceability and longevity of an asphalt pavement. During the hot summer months, rutting becomes more crucial as binder partially loses its stiffness. Then, the moving traffic load is mainly supported by the aggregate structure (Brown et al., 2009). Different researchers have followed different approaches to identify the rutting potential of asphalt mixes (Miller et al., 1995; Cooley et al., 2000; Kandhal and Cooley, 2002). In this study, Hamburg wheel tracking (HWT) test was considered for evaluation of rutting resistance of the WMA and HMA mixes.

The HWT test was conducted according to AASHTO T 324 (AASHTO, 2014) at 50°C temperature. Figure 4.7 and Figure 4.8 present the HWT test results obtained for the S3 (HMA S3 and WMA S3) and S4 mixes (HMA S4 and WMA S4), respectively. From Figure 4.7 and Figure 4.8 it is evident that for both S3 and S4 mixes, HMA mixes showed better rutting resistance than WMA mixes. The test data indicated that for a similar numbers of wheel passes, higher rut depths were observed in WMA specimens than HMA specimens. This difference was more pronounced at higher number of passes. The rut depths at 1,000, 5,000, 10,000, 15,000, and 20,000 wheel passes for all mixes are presented in Table 4.7. In all cases HMA mixes showed lower rut depths compared to WMA mixes. A possible explanation of this might be the lower production temperatures for WMA mixes. Lower production temperatures are expected to cause less binder aging and softer mixes (Hurley and Prowell, 2006; Alhasan et al., 2014). Similar observations were made by Hill (2011), Bonaquist (2011), Ali et al. (2013), Mo et al. (2012), Zhao et al. (2013), and Yu et al. (2016). However, Jones (2004), Prowell et al. (2007), and Wielinski et al. (2009) observed an insignificant difference in rutting performance beteen HMA and WMA mixes. Also, both HMA and WMA mixes satisfied the ODOT requirement of less than 12.5 mm rut depth at 10,000-wheel passes for PG 64-22 grade binder (ODOT, 2011).

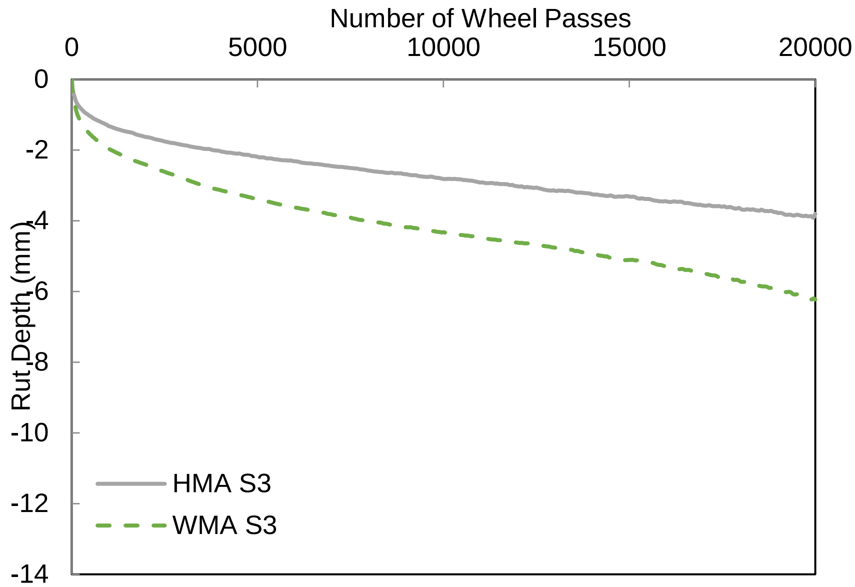

Figure 4.7 Comparison of HWT graphs for WMA and HMA S3 mixes (25% RAP)

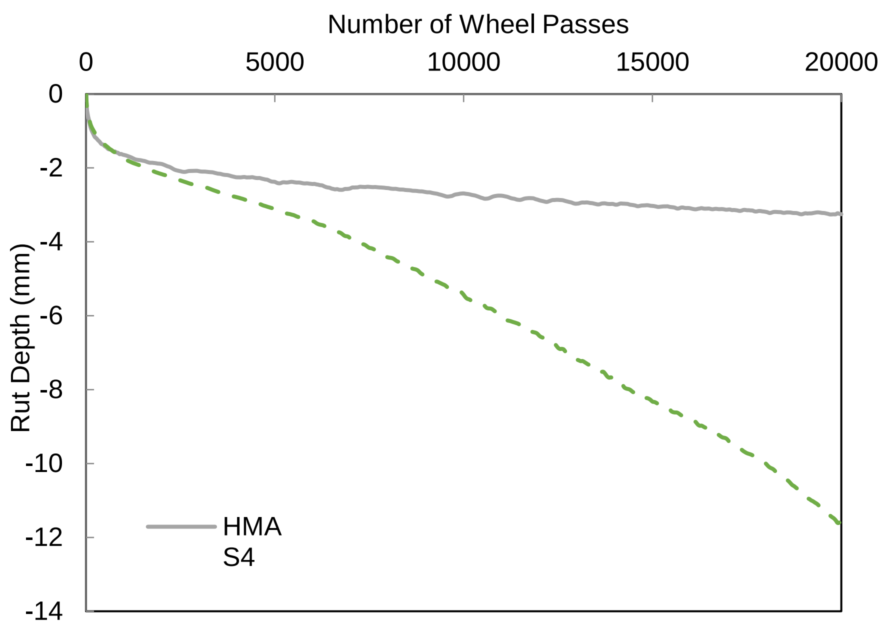

Figure 4.8 Comparison of HWT graphs for S4 mixes (5% RAP)

Table 4.7 Rut depths for foamed WMA and HMA mixes at different number of wheel passes

Mix Type	Average Air Voids (%)	RAP Content (%)	Number of Wheel Passes Rut Depth (mm)				
			1,000	5,000	10,000	15,000	20,000
HMA S3	6.8	25	1.1	1.7	2.2	2.7	3.5
WMA S3	6.8	25	2.0	3.4	4.3	5.1	6.2
HMA S4	6.9	5	1.7	2.4	2.7	3.0	3.3
WMA S4	7.0	5	1.8	3.1	5.4	8.3	11.2

Several parameters, namely post-compaction deformation, creep slope, stripping slope, and stripping inflection point (SIP) were determined from the HWT test results and are presented in Table 4.8. The post-compaction deformation indicates the initial densification of asphalt pavement due to traffic loads. Yildirim and Kennedy (2002) suggested the rut depth at 1,000 wheel passes as the post-compaction point. From Table 4.8 it is evident that WMA mixes showed a higher post-compaction deformation compared to HMA mixes due to lower stiffness. This is mainly attributed to lower production and compaction temperatures for WMA mixes. Distinct inflection points for both WMA S3 and WMA S4 mixes were observed at approximately 17,100 and 12,000 wheel passes, respectively. These points indicate the possibility of moisture-induced damage in the WMA samples. Consequently, after those points, the rate of rutting increased rapidly due to the intrusion of moisture in the specimen and stripping. The resistance to deformation reduced from 6,667 passes/mm to 2,400 passes/mm after the SIP for the WMA S3 mix. A similar trend was observed for the WMA S4 mix. However, no inflection point was observed in HMA mixes (both HMA S3 and HMA S4). Therefore, HMA mixes were found to be less prone to moisture-induced damage.

Table 4.8 A summary of HWT test parameters for HMA and WMA mixes

Mix Type	Average Air Voids (%)	HWT Parameters						
		Post Compaction (mm)	Creep Slope (mm /passes)	Inverse Creep Slope (passes /mm)	Stripping Inflection Point (passes)	Stripping Slope (mm /passes)	Inverse Stripping Slope (passes /mm)
HMA S3	6.8	1.1	0.000100	10,000	N/A	N/A	N/A
WMA S3	6.8	2.0	0.000150	6,667	17,100	0.000417	2,400
HMA S4	6.9	1.6	0.000067	15,000	N/A	N/A	N/A
WMA S4	7.0	1.8	0.000339	2,946	12,000	0.000756	1,324

Also, from Table 4.8 it is evident that the inverse creep slope for the WMA S3 mix was 6,667 passes/mm, which was significantly lower than that observed for the HMA S3 mix (10,000 passes/ mm). This indicates that the HMA S3 mix had a resistance to creep deformation twice as high as that of the WMA S3 mix. Furthermore, the creep slope for the HMA S4 mix (15,000 passes/mm) was found to be approximately five times higher than that of the WMA S4 mix. Therefore, it may be concluded that the foamed WMA (both WMA S3 and WMA S4) are more susceptible to rutting compared to their HMA counterparts containing an identical amount of RAP (both HMA S3 and HMA S4). Similar observations were also made by Wielinski et al. (2009), Hill (2011), Ali et al. (2013), and Zhao et al. (2013).

From the results presented above it can be observed that an increase in RAP content and a coarse aggregate structure improved the rutting resistance of asphalt mixes due to aged and stiffer binder. From Table 4.7 and Figure 4.8 it is evident that, in general, lower rut depths were observed for S3 mixes containing 25% RAP compared to S4 mixes containing 5% RAP and a finer aggregate structure. Similar findings were also reported by other researchers (Shu et al., 2008; Hong et al., 2010; Zhao et al., 2013; Guo et al., 2014; and Dong et al., 2017). However, Daniel and Lachance (2005), Shah et al. (2007) and Apeagyei et al. (2011) did not observe any significant difference in rutting resistance of mixes containing RAP and those without any RAP.

Overall, it was observed that the rutting performance of asphalt mixes mainly depends on three major parameters, namely mix type (HMA vs. WMA), gradation, and RAP content. From the HWT test data, rut depth at 10,000 wheel passes and inverse creep slope (passes/mm) were considered in ranking the asphalt mixes. A maximum allowable rut depth of 12.5 mm at 10,000 wheel passes is specified by the ODOT's current special provision for mixes containing a PG 64-22 binder (ODOT, 2011). It is known that the post-compaction rut depth mainly represents the initial level of compaction of asphalt mixes. Therefore, this parameter was not used in ranking. Table 4.9 presents the ranking of asphalt mixes based on their resistance to rutting.

Table 4.9 Ranking of asphalt mixes based on their resistance to rutting

Mix Type	HWT Parameters			
	Rut Depth at 10,000 Wheel Passes (mm)	Rank	Inverse Creep Slope (passes /mm)	Rank
HMA S3	2.2	1	10,000	2
WMA S3	4.3	3	6,667	3
HMA S4	2.7	2	15,000	1
WMA S4	5.4	4	2,946	4

From Table 4.9 it is evident that the ranking of asphalt mixes in terms of resistance to rutting using rut depth at 10,000 wheel passes and inverse creep slope is very similar. The difference between the observed rut depths measured at 10,000 wheel passes for the HMA S3 and HMA S4 mixes is only 0.5 mm, which is relatively small. Overall, the HMA mixes (HMA S3 and HMA S4) exhibited a significantly better rutting performance compared to the WMA mixes. Among the WMA mixes, the WMA S3 mix exhibited better rutting resistance compared to the WMA S4 mix. While, the laboratory test results show that the WMA mixes have a lower resistance to rutting than their HMA counterparts, a field study conducted by Wielinski et al. (2009) suggested that both HMA and WMA can perform equally in Southern California climate and subjected to heavy traffic loads. Also, Sargand et al. (2011) observed a similar International Roughness Index (IRI) after 46 months of service life for both WMA and HMA pavements.

4.3.3 Moisture-Induced Damage

The stripping or moisture-induced damage is defined as the loss of adhesion between aggregates and binder or cohesion within the binder in asphalt mixes usually due to moisture (Brown et al., 2009). The foamed WMA mixes might be more susceptible to moisture due to reduced mixing temperature and incorporation of water in the foaming process (Hurley and Prowell, 2005, 2006; Prowell et al., 2007; Wasiuddin et al., 2007; Ali et al., 2013; Xu et al., 2017). Several test methods are available to evaluate moisture-induced damage of asphalt mixes. Among these methods, SIP from the HWT test and TSR from the indirect tensile strength test are most commonly used in ranking asphalt mixes (Kim YR et al., 2012; Abuawad et al. 2015). In this study, SIP and TSR were used as an indicator of moisture-induced damage potential of asphalt mixes.

4.3.3.1. Stripping Inflection Point (SIP)

The SIP parameter is defined as the number of wheel load cycles in the HWT test at which an abrupt increase in rut is observed due to stripping (Brown et al., 2009). From Figures 4.7 and 4.8, no distinct SIP was observed for HMA mixes (both HMA S3 and HMA S4). However, SIPs were found for WMA S3 and WMA S4 mixes at 17,100 and 12,000 wheel passes, respectively. Therefore, WMA mixes were found to be more sensitive to moisture-induced damage than HMA mixes. This was attributed to partial aggregate drying at lower WMA production temperature and incorporation of water in the foaming process (Hurley and Prowell, 2005; Hurley and Prowell, 2006; Prowell et al., 2007; Wasiuddin et al.; 2007; Ali et al., 2013; Xu et al., 2017). However, a number of researchers did not report any major difference between moisture-induced damage potential of foamed WMA and HMA mixes (Punith et al., 2012; Xiao et al. 2012; Hailesilassie et al., 2015).

Based on the SIP values for the WMA S3 mix were found at a higher number of wheel passes (17,100) than the WMA S4 mix (12,000). Thus, a higher resistance of the WMA S3 mix to moisture-induced damage was observed compared to the WMA S4 mix. Also, from Table 4.8 it is evident that the WMA S3 mix has a higher inverse stripping slope (2,400) compared to that of the WMA S4 mix (1,324), which indicates a higher resistance to stripping due to a higher RAP content. Therefore, the addition RAP to an asphalt mix and introducing more aged binder into the mix was found to have a positive effect on its resistance to moisture-induced damage. Ghabchi et al. (2016), Zhao et al. (2012), Shu et al. (2012), and Hill et al., (2012a) have also reported an increase in resistance to moisture-induced damage with an increase in RAP content.

4.3.3.2. Tensile Strength Ratio (TSR) Method

The TSR test conducted in accordance with AASHTO T 283 standard method (AASHTO, 2014), commonly known as freeze-thaw method, was used to screen asphalt mixes for their moisture-induced damage potential. Figures 4.9 and 4.10 present the indirect tensile strength (ITS) values of dry and freeze-thaw conditioned samples and their TSR values for the HMA and WMA mixes. From Figure 4.9 it is evident that the average ITS_{Dry} value the for WMA S3 mix (2596.4 kPa) was 9% higher than that for the HMA S3 mix (2380.8 kPa) in dry condition. Similarly, the average ITS_{Wet} value for the WMA S3 mix (2054.4 kPa) was 2% higher than that for the HMA S3 mix (2008.9 kPa). A lower degree of aging for binder used in WMA may be responsible for the increase in tensile strength of WMA mixes (Hurley and Prowell, 2006; Alhasan et al., 2014). However, the TSR value the for HMA S3 mix (0.84) was higher than that of the WMA S3 mix (0.79). Also, both mixes satisfied the ODOT’s requirement for minimum TSR (0.8). Kavussi and Hashemian (2011), Ali et al. (2013) and Sebaaly et al. (2015) have reported similar TSR values for foamed WMA and HMA fixes. Several other studies have reported no major difference between foamed WMA and HMA mixes with respect to moisture0induced damage (Punith et al., 2012; Xiao et al. 2012; Hailesilassie et al., 2015). However, Hurley and Prowell (2005), Prowell et al., (2007), Wasiuddin et al. (2007), Ali et al. (2013), and Xu et al. (2017) have reported a higher moisture susceptibility for WMA mixes that HMA mixes due to lower production temperature of WMA mixes.

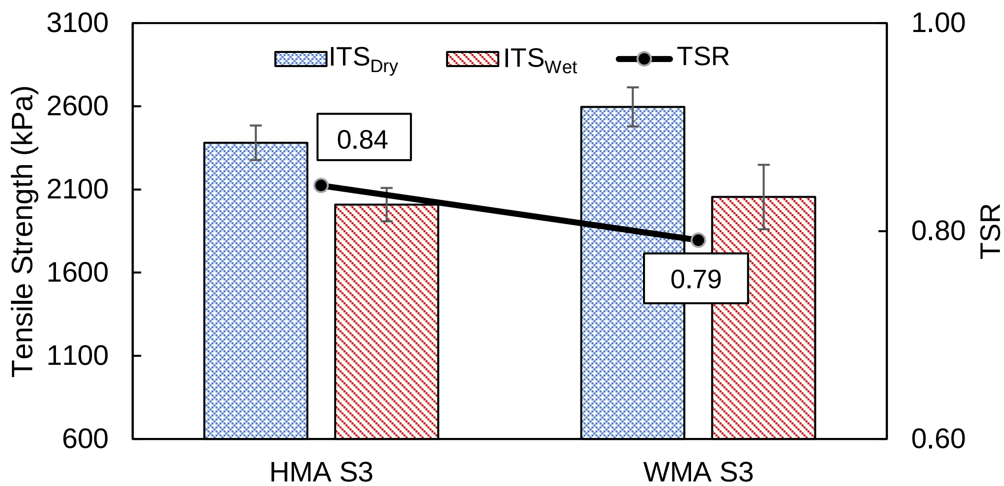

Figure 4.9 ITS_{Dry}, ITS_{wet} and TSR values measured for HMA S3 and WMA S3 mixes

From Figure 4.10 it is evident that the measured ITS_{Dry} and ITS_{Wet} values for WMA S4 mix were 2,206 and 1,681 kPa, respectively. Comparatively, the measured ITS_{Dry} and ITS_{Wet} vales for the HMA S4 mix were 2,367 and 1,897 kPa, respectively. Therefore, it was evident that the HMA S4 mix had higher ITS values in both dry and moisture-conditioned states compared to the WMA S4 mix. Also, both S4 mixes (WMA and HMA) were found to have a lower tensile strength compared to their S3 counterparts. It was attributed to a lower amount of aged RAP binder in the mix. McDaniel et al. (2000), Huang et al. (2004) and Ghabchi et al. (2016) have also reported that incorporation of RAP up to a certain level has a positive effect on the tensile strength of asphalt mixes. Also, the TSR value for the WMA S4 mix (0.76) was slightly lower than that of the HMA S4 mix (0.80). Several studies have reported similar TSR values for both foamed WMA and HMA mixes (Kavussi and Hashemian, 2011; Ali et al., 2013; Sebaaly et al., 2015). Additionally, the WMA S4 mix failed to satisfy the ODOT's minimum requirement for TSR (0.8), while the HMA S4 mix satisfied this requirement. Also, Hurley and Prowell (2005), Prowell et al., (2007), Wasiuddin et al. (2007), Ali et al. (2013), and Xu et al. (2017) have observed lower resistance to moisture damage of foamed WMA mixes due to partially dried aggregates and incorporation of water in the foaming process.

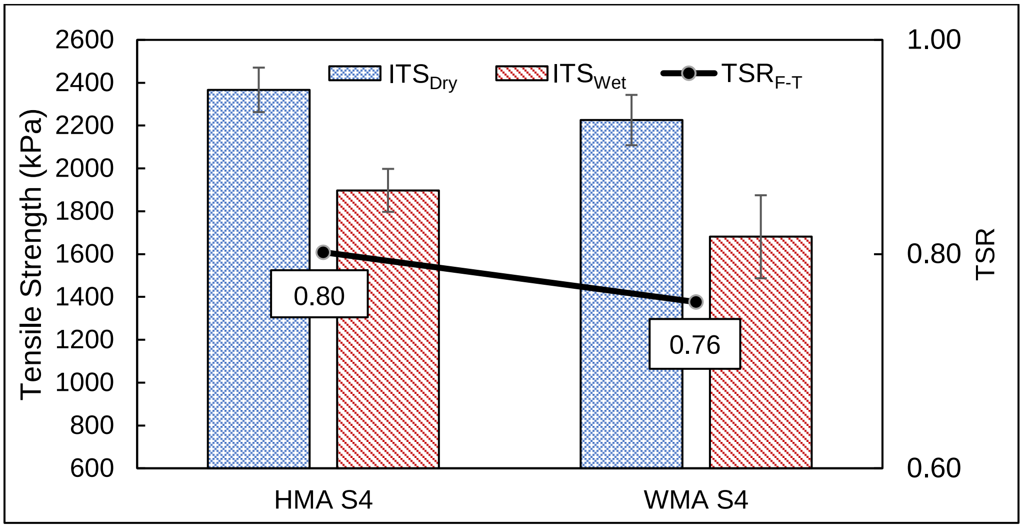

Figure 4.10 ITS_{Dry}, ITS_{wet} and TSR values measured for HMA S4 and WMA S4 mixes

Asphalt mixes used in this study were ranked based on their resistance to moisture-induced damage using different parameters obtained from the HWT and TSR test results (Table 4.10). From Table 4.10 it was observed that both HWT and TSR values ranked the mixes in the same order based on their resistance to moisture-induced damage. The HMA mixes did not exhibit any SIP in the HWT test and therefore, the same rank (1 and/or 2) was assigned for both HMA S3 and HMA S4 mixes. Additionally, the WMA S3 (SIP = 17,100) and WMA S4 (SIP = 12,000) were ranked 3rd and 4th in the list, based on their SIP values. Acording to TSR values, the HMA S3 mix exhibited the highest resistance to moisture-induced damage followed by the HMA S4, WMA S3, and WMA S4 mixes. Therefore, the foamed-WMA technology used herein produced more moisture-susceptible mixes due to a lower production temperature and water injection. Ali et al. (2013) suggested a longer drying period for aggregates in case of WMA to allow the entrapped water to evaporate. The moist aggregates increased the potential of moisture-induced damage for foamed WMA due to inadequate aggregate coating in presence of water (Ali et al., 2013). Moreover, an increase in RAP content was found to be beneficial for the resistance of mixes to moisture-induced due to a stronger bond between the aged binder and aggregates.

Table 4.10 Ranking of asphalt mixes based on their resistance to moisture-induced damage

Mix Type	HWT Test	TSR Test
	SIP	TSR
HMA S3	1 and/or 2	1
WMA S3	3	3
HMA S4	1 and/or 2	2
WMA S4	4	4

4.4. Summary

It was found that following the HMA mix design process in designing foamed WMA mixes containing RAP results in the same volumetric properties statistically for both types of mixes. Thus, no changes in the current mix design practice for foamed WMA mixes in Oklahoma are needed when using similar mixing (135^{o}C) and compaction (127^{o}C) temperatures and RAP contents as those used in the present study. However, SCB, HWT, and TSR tests conducted on the WMA and HMA mixes revealed that despite similar volumetric properties their fatigue, rutting, and moisture-induced damage performances can be quite different. Overall, WMA mixes containing RAP showed lower resistance to rutting and moisture-induced damage and a higher resistance to fatigue cracking compared to HMA mixes. Therefore, it is recommended that both volumetric properties and performance-based mix design practices be used for foamed WMA mixes to avoid premature distresses and to ensure longer pavement life.

CHAPTER 5 - COST AND ENVIRONMENTAL BENEFITS OF USING WMA

Life Cycle Cost Analysis (LCCA) is an effective tool to determine the most cost-effective construction method and material amongst two or more alternatives. According to FHWA (1998), the LCCA technique is built on the well-founded principles of economic analysis over long-term economic efficiency between competing alternative investment options. It does not address equity issues. It incorporates initial and discounted future agency, user, and other relevant costs over the life of alternative investments. It attempts to identify the best value for investment expenditures. One of the initial objectives of the current study was to conduct a life cycle cost analysis.

As indicated in Chapter 1, the WMA is currently designed using the HMA mix design procedure. In the current study, efforts were made to identify if a separate design procedure is needed for the foam-based WMA. Accordingly, volumetric parameters were evaluated and used to evaluate the effectiveness of the foamed-WMA mixes (both S3 and S4) produced using the HMA-based design practice. As discussed previously, based on the volumetric properties, it was found that a separate mix design may not be needed for foamed-WMA containing RAP. Thus, it was decided that a comprehensive life cycle cost analysis was not necessary. Instead, this chapter is focused on the relative benefits of foamed-WMA relative to HMA, which include cost. An example of saving due to reduced fuel cost is included.

Economic benefits of WMA, particularly foamed-WMA, are well-established. Several previous studies have outlined the benefits of WMA, including reduced emission of greenhouse gases, increased usage of RAP, improved working condition, and reduced cost (Anderson et al., 2008). The fuel saving has been reported in the range of 10-35%, depending on the temperature and humidity, among other factors.

A longer paving season is another benefit of using WMA. In several European case studies, WMA was used to pave at temperatures as low as -3^{o}C (D'Angelo, et al., 2008). The NCHRP Project 9-47 suggested a better workability of WMA mixes compared to HMA mixes (Anderson et al., 2008). Also, significant reductions in hazardous gases such as, sulfur dioxide (SO_2), nitrogen oxide (NO_x), and carbon dioxide (CO_2) can be achieved by foamed-WMA.

As noted in the NCHRP Report 779 (West et al., 2014), a WMA mix can yield an energy saving of 1,100 BTU/ºF/ton, but it depends on the type of technology used. For water injection-based foaming technology, the mixing temperature can be reduced by 25ºF. For additive-based WMA, however, the mixing temperature can be reduced by 50ºF on an average. Based on these data, the typical energy savings can be in the range of 27,500 to 55,000 BTU per ton of asphalt. A summary of the estimated cost savings and potential benefits of WMA, taken from NCHRP Report 779, are provided in Table 5.1 (West et al., 2014).

In an NCAT study, Willis (2014) provided a life cycle assessment (LCA) of several sustainable asphalt mixtures, including foamed-WMA. Based on that study, the use of WMA technology can result in reduction in the energy cost by 12-17% and CO_2 emission by 6-9%, compared to the HMA.

Figure 5.1 shows a typical breakdown of HMA production cost (https://www.cement.org/docs/default-source/about-pca-pdfs/paving_cost_comparisons_flash.pdf; accessed Nov. 9, 2019). Based on this figure, the fuel cost accounts for about 15% of the overall HMA cost. Based on an estimated diesel cost of $3.94/gal (2018 data) and an estimated 20% saving in energy, $1.58/ton can be saved using foamed-WMA. This amounts to a saving of $2,400 per lane mile (12-ft wide and 4-inch lift). Considering environmental benefits, the actual savings using foamed-WMA are expected to be much more.

Table 5.1 Benefits of using WMA as reported in NCHRP Report 779 (2014)

WMA Type	**Water-Injection Foaming**	**Additive**
Typical technology cost ($/ton)	($0.08)	($2.50)
Assumed temperature reduction	25°F	50°F
Typical energy savings ($/ton)		
RFO	$0.39	$0.79
Natural gas	$0.16	$0.31
Typical incentive/disincentive spec. savings ($/ton)		
Density improvement	0 to $1.13	0 to $1.13
Smoothness	?	?
Possible savings from eliminated antistripping agent		
Liquid ASA	0	0 to $0.75*
Hydrated lime	0	0 to $1.50*

* Applicable only to WMA additives with antistripping capabilities

Table 5.2 Savings in energy cost in producing WMA mixes (https://www.cement.org/docs/default-source/about-pca-pdfs/paving_cost_comparisons_flash.pdf)

Price Inputs			Fuel Cost Savings			Asphalt Bid Price Savings		
Year	*Diesel Price ($/gal)*	*HMA Bid Price ($/ton)*	*Chemical 1.0 gal/ton*	*Organic 0.7 gal/ton*	*Water-Based 0.4 gal/ton*	*Chemical*	*Organic*	*Water-Based*
2014	$3.48	$46.02	$3.48	$2.44	$1.39	7.6%	5.3%	3.0%
2015	$3.56	$45.89	$3.56	$2.49	$1.42	7.8%	5.4%	3.1%
2016	$3.68	$48.19	$3.68	$2.57	$1.47	7.6%	5.3%	3.1%
2017	$3.81	$51.01	$3.81	$2.67	$1.52	7.5%	5.2%	3.0%
2018	$3.94	$54.28	$3.94	$2.76	$1.58	7.3%	5.1%	2.9%
2019	$4.08	$57.80	$4.08	$2.86	$1.63	7.1%	4.9%	2.8%
2020	$4.20	$61.61	$4.20	$2.94	$1.68	6.8%	4.8%	2.7%

Source: EIA, PCA

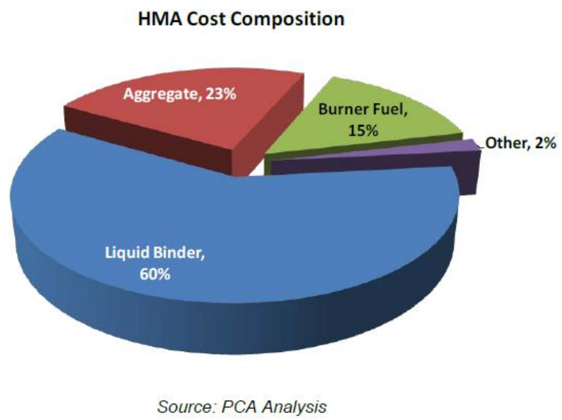

Figure 5.1 HMA cost composition (https://www.cement.org/docs/default-source/about-pca-pdfs/paving_cost_comparisons_flash.pdf)

CHAPTER 6 - TECHNOLOGY TRANSFER WORKSHOP

On December 1, 2017 the research team organized a technology transfer workshop at the ODOT headquarter to allow broader participation by ODOT employees, OAPA members, and other stakeholders. Approximately 30 participants from ODOT, OAPA, industry and other stakeholders attended this event (Figure 6.1). The findings of the study were presented in the workshop, followed by discussions and questions and answers.

Figure 6.1. Technology transfer workshop at ODOT headquarter (December 1, 2017)

CHAPTER 7 – CONCLUSIONS

In this study, the volumetric properties of two control HMA mixes (HMA S3 and HMA S4) and two corresponding foamed WMA mixes (WMA S3 and WMA S4) were determined in the laboratory. The S3 mixes (NMAS = 19.0 mm) and S4 mixes (NMAS = 12.5 m) each contained 25 and 5% RAP, respectively. A light weight traffic condition (ESAL < 0.3M) was considered in designing all asphalt mixes (AASHTO, 2013). The mixing and compaction temperatures of 135°C and 127°C, respectively, were used for the foamed WMA mixes. For the HMA mixes, higher mixing (163°C) and compaction(149°C) temperatures were used. Differences in the volumetric properties of foamed WMA mixes were compared to their HMA counterparts using statistical analyses. Also, the fatigue, rutting, and moisture-induced damage resistance of the foamed WMA and HMA mixes containing RAP were evaluated and compared by conducting SCB, HWT, and TSR tests, respectively. From the test results presented in the preceding chapters, the following conclusions were drawn:

1. From a statistical point of view, identical G_{mm} values were observed for both HMA and WMA mixes having the same gradations.
2. When designing foamed WMA mixes using the HMA mix design procedure, the differences between the percent air voids of HMA and WMA mixes (both S3 and S4 mixes) were insignificant, at 95% confidence level. This finding, in combination with the fact that both HMA and foamed WMA had similar G_{mm} values, suggests that the volumetric properties of a HMA mix and a foamed WMA mix designed in the laboratory using AASHTO R 35 are statistically identical. Thus, the current practice of using the HMA mix design procedure for designing foamed WMA mix for the temperatures specified in this project was found to be acceptable. Therefore, no modifications to this procedure (AASHTO R 35) were recommended.
3. A follow-up study by Rahman (2019) has shown that volumetric properties of foamed WMA and HMA mixes can be different when using lower temperatures (than those used in this study) and higher RAP contents, while keeping the number of gyrations (traffic level) unchanged.
4. Using the current practice for design of foamed WMA mixes was found to overestimate the resistance of both S3 and S4 mixes to moisture-induced damage.

While HMA mixes met the minimum TSR requirement (0.8), the WMA mixes failed this requirement by a narrow margin. Also, notable SIPs observed for foamed WMA mixes obtained from HWT tests confirmed this finding. No SIPs were observed for HMA mixes.

5. Using the current design practice, the foamed WMA mixes were found to overestimate the resistance of both S3 and S4 mixes relative to rutting. From HWT tests it was found that foamed WMA mixes exhibited a higher rutting propensity compared to their HMA counterparts.
6. Using the current practice for design of foamed WMA mixes was found to underestimate the resistance of both S3 and S4 mixes relative to fatigue cracking. Foamed WMA mixes (both S3 and S4) were found to exhibit higher J_c values compared to those of HMA mixes. This indicates a higher resistance of foamed WMA to fatigue cracking at intermediate temperature.
7. Although a detailed LCCA was not conducted in this study for reasons included in Conclusion 2, potential benefits of foamed WMA are summarized in this report. Considering saving in fuel cost alone, it was evident that foamed WMA can result in significant financial benefits. Consideration of environmental benefits would make a much stronger case for using foamed WMA in paving projects in Oklahoma and elsewhere in the country. Both short term and long term performance should, however, be evaluated carefully to ensure desired pavement performance.

REFERECES

AASHTO R-35 (2013). “Standard Practice for Superpave Volumetric Design for AsphaltMixtures,” Transportation Research Board, National Research Council, DC.

AASHTO. (2011). "Standard Specifications for Transportation Materials and Methods of Samplingand Testing," American Association of State Highway and Transportation Officials (AASHTO),Washington, DC.

Anderson, R., M., Baumgardner, G., May, R., Reinke, G., Engineering Properties, Emissions, and Field Performance of Warm Mix Asphalt Technologies, NCHRP Project 9-47, Transportation Research Board, Washington, DC, 2014.

Board (TRB).

D’Angelo, J., Harm, E., Bartoszek, J., Baumgarderdner, G., Corrigan, M., Cowsert, J., Harman, T., Jamshidi, M., Jones, W., Newcomb, D., Prowell, B., Sines, R., and Yeaton, B. Warm-mix asphalt: European Practice, Report No. FHWA-PL-08-007, Federal Highway Administration, Washington, DC, 2008.

D’Angelo, J., Harm, E., Bartoszek, J., Baumgardner, G., Corrigan, M., Cowsert, J., Harman, T.,Jamshidi, M., Jones, W., Newcomb, D., Prowell, B., Sines, R., and Yeaton, B. (2008). “Warm-MixAsphalt: European Practice,” Report FHWA-PL-08-007, American Trade Initiatives, Alexandria,VA.

Diefenderfer, S., and Hearon, A. (2008). “Laboratory evaluation of a warm asphalt technology for use in Virginia” Final Report No. FHWA/VTRC 09-R11.

Ghabchi, R., Singh, D., and Zaman, M. (2015a). “Laboratory evaluation of stiffness, low temperaturecracking, rutting, moisture damage, and fatigue performance of WMA mixes,”Journal of Road Materials and Pavement Design, (available online, ahead-of-print), pp. 1-24.

Ghabchi, R., Singh, D., Zaman, M., and Hossain, Z., (2014a). “Laboratory Characterization ofAsphalt Mixes Containing RAP and RAS,” International Journal of Pavement Engineering (available online, vDOI:10.1080/10298436.2015.1022778).

Ghabchi, R., Singh, D., Zaman, M., and Hossain, Z., (2014b). “Micro-Structural Analysis of Moisture-Induced Damage Potential of Asphalt Mixes Containing RAP,” ASTM, Journal of Testing and Evaluation (available online).

Ghabchi, R., Singh, D., Zaman, M., and Tian, Q. (2013a). “Mechanistic Evaluation of the Effect of WMA Additives on Wettability and Moisture Susceptibility Properties of Asphalt Mixes,” ASTM, Journal of Testing and Evaluation, Vol. 41, No. 6, pp. 933-942.

Ghabchi, R., Singh, D., Zaman, M., and Tian, Q. (2013b). “A Laboratory Study of Warm Mix Asphalt for Moisture Damage Potential Using Surface Free Energy Method,” Proceedings of the2013 Airfield & Highway Pavement Conference: Sustainable and Efficient Pavements, Transportation & Development Institute of ASCE, Los Angeles, California, June 9-12, 2013, pp.54-63.

Ghabchi, R., Zaman, M., Barman, Singh, D. V., and Boeck, D. (2015b). “Fatigue Performance of Asphalt Pavements Containing RAS and RAP,” Final Report, Submitted to: Oklahoma Department of Transportation, January 2015, The University of Oklahoma, Norman, OK, 2015 (pp. 119).

Ghabchi, R., Zaman, M., Bulut, R., Koc, M., and Singh, D. (2013c) “WMA Pavements inOklahoma: Moisture Damage and Performance Issues” Research Report OTCREOS10.1-06-F, Oklahoma Transportation Center (OkTC), Midwest City, OK.

Goh, S. W., and You, Z. (2011). “Evaluation of Warm Mix Asphalt Produced at Various Temperatures through Dynamic Modulus Testing and Four Point Beam Fatigue Testing.” In Proceedings of the GeoHunan international conference–pavements and materials: recent advances in design, testing, and construction. Hunan, China: Geotechnical Special Publication, Vol. 212, pp. 123-30.

Hossain, Z., and Zaman, M. (2013b) “Behavior of Selected Warm Mix Asphalt Additive Modified Binders and Prediction of Dynamic Modulus of the Mixes,” ASTM Journal of Testing and Evaluation, 41(1).

Hossain, Z., Ghosh, D., Zaman, M., and Hobson, K. (2014). "Implementation of the Multiple Stress Creep Recovery (MSCR) Test Method to Characterize Polymer-Modified Asphalt Binders for Oklahoma Conditions," Under Review, in the Journal of Testing and Evaluation.

Hossain, Z., Lewis, S., Zaman, M., Buddhala, A., and O'Rear, E. (2013a) "Evaluation for Warm Mix Additive- Modified Asphalt Binders Using Spectroscopy Techniques Journal of Materials in Civil Engineering, 25(2), 149–159.

Hossain, Z., Solanki, P., Zaman, M., Laguros, M., and Lewis, S. (2012a). "Test Methods For Use of Recycled Asphalt Pavement In Asphalt Mixes," Final Report, March 2012, ODOT SPR No. 2223, Oklahoma Department of Transportation, Oklahoma City, 149 pages.

Hossain, Z., Solanki, P., Zaman, M., Lewis, S., and Hobson, K. (2012b) "Influence of Recovery Processes on Properties of Binders and Aggregates Recovered from Recycled Asphalt Pavement," in the Journal of ASTM International (JAI), Version, Vol. 9, Issue 3, pp. 1-18.

Hossain, Z., Zaman, M., and Doiron, C. (2012c). "Evaluation of Resilient Response of Unbound Aggregates Toward Implementation of the Mechanistic-Empirical Pavement Design in Oklahoma," in the 91st Annual Meeting DVD-ROM Compendium Paper, Transportation Research

Hot-Mix Asphalt Paving Handbook 2000. AC 150/5370-14A, USACE, 2000.

Issa, R. I., Zaman, M., and Miller, G. A. (2000a). "Mix design using asphalt millings," Item 2148, ORA 125-6106, Summary Report, Summary Rep. to ODOT, Aug. 2000 (36 pages).

Issa, R. I., Zaman, M., and Miller, G. A. (2000b). "Pavement Rehabilitation Using Asphalt Millings," Project Report submitted to Koch Materials Co, January. 2000 (109 pages).

Kasozi, A. M., Hajj, E. Y., Sebaaly, P. E., & Elkins, J. C. (2012). "Evaluation of foamed warm-mix asphalt incorporating recycled asphalt pavement for volumetric and mechanical properties." International Journal of Pavement Research and Technology, 5(2), 75.

Khoury, N. N., Teredesai, R., Zaman, M., and Khoury, C. N. (2007). "Laboratory and Field Assessment of Pile run Chat from the Tar Creek Superfund Site Stabilized with Class C Fly Ash and Cement Kiln Dust," International Journal of Geotechnical Engineering, Vol. 1, Issue 1, pp. 71- 80.

Kvasnak, A. N., and West, R. C. (2009). "Case Study of Warm-Mix Asphalt Moisture Susceptibility in Birmingham, Alabama," Transportation Research Board 88th Annual Meeting. No. 09-3703.

NCHRP Report-691 (2012b), "Mix Design Practices for Warm Mix Asphalt", Advanced Asphalt Technologies, LLC, Report prepared for Transportation Research Board of the National Academics, DC.

NCHRP Report-714 (2012a), "Special Mixture Design Considerations and Methods for Warm Mix Asphalt: A Supplement to NCHRP Report 673: A Manual for Design of Hot Mix Asphalt with Commentary", Advanced Asphalt Technologies, LLC, Report prepared for Transportation Research Board of the National Academics, DC.

NCHRP Report-714 (2013), "QC/QA Testing differences between Hot Mix Asphalt (HMA) and Warm Mix Asphalt (WMA)", Final Report ~ FHWA-OK-13-02, Report prepared for Oklahoma Department of Transportation, OK.

ODOT (2013). "Special Provision for Warm Mix Asphalt Material Requirements," Oklahoma Department of Transportation, 708-22(a) 99, OK.

Selvamohan, S., Zaman, M., and Laguros, J. (2007). "A Laboratory Evaluation of Unmodified and Polymer Modified Performance-Grade Binders with Anti-Stripping Additives," Journal of Engineering, Computing and Architecture, SJI, Vol. 1, Issue 2, pp. 1-11.

Singh, D.V., Mai, A.T., Beainy, A.F., Commuri, S., and Zaman, M.M. (2011a). "In-Situ Assessment of Stiffness during Construction of an HMA Pavement," International Journal of Pavement Research and Technology (IJPRT), Vol.4, Issue: 3, pp. 131-139.

Singh, D.V., Zaman, M., and Commuri, S. (2011b). "Evaluation of Predictive Models for Estimating Dynamic Modulus of Hot-Mix Asphalt in Oklahoma," Transportation Research Record: Journal of the Transportation Research Board (TRB), Vol. 2210, pp.57-72.

Singh, D.V., Zaman, M., and Commuri, S. (2011c). "Evaluation of Measured and Estimated Dynamic Moduli for Selected Asphalt Mixes," Journal of ASTM International (JAI), Vol. 8, No.9, 19 pages, DOI: 10.1520/JAI103535.

Singh, D.V., Zaman, M., and Commuri, S. (2012a). "Evaluation of Dynamic Modulus of Modified and Unmodified Asphalt Mixes for Different Input Levels of the MEPDG," International Journal of Pavement Research and Technology (IJPRT), Vol.5, Issue: 1, pp. 1-11.

Singh, D.V., Zaman, M., and Commuri, S. (2012b). "Comparison of Shape Parameters for Selected Coarse Aggregates in Oklahoma," ASTM Journal of Testing and Evaluation (February 2012)(To be appeared in Volume: 40, Issue: 3, May 2012)

Singh, D.V., Zaman, M., and Commuri, S. (2012c). "Effects of Production and Sample Preparation Methods on Aggregate Shape Parameters," International Journal of Pavement Engineering, Available online at: http://www.tandfonline.com/doi/abs/10.1080/10298436.2011.635792

Solanki, P., Khoury, N., & Zaman, M. M. (2009a). "Engineering properties and moisture susceptibility of silty clay stabilized with lime, class C fly ash, and cement kiln dust," Journal of Materials in Civil Engineering, Vol. 21, Issue: 12, pp. 749-757.

Solanki, P., Zaman, M., and Ebrahimi, A. (2009b). "Regression and Artificial Neural Network Modeling of Resilient Modulus of Subgrade Soils for Pavement Design Applications" In Intelligent and Soft Computing in Infrastructure Systems Engineering, Springer Berlin Heidelberg, pp. 269-304.

Tarefder, R. A., White, L., and Zaman, M. (2005b). "Neural network model for asphalt concrete permeability," Journal of Materials in Civil Engineering, Vol. 17, Issue: 1, pp. 19-27.

Tarefder, R. A., Zaman, M. M., and Hobson, K. (2002). "A Laboratory Assessment of Binders' Contribution to Rutting Susceptibility," International Journal of Pavement Engineering, Vol. 1, No. 2, pp. 36-47.

Tarefder, R. A., Zaman, M. M., and Hobson, K. (2003a). "A Laboratory and Statistical Evaluation of Factors Affecting Rutting," International Journal of Pavement Engineering, IJPE, Vol. 4, No. 1, pp. 59-68.

Tarefder, R. A., Zaman, M. M., and Patton, D. (2003b). "Formulation of Mix Design and Asphaltic Encapsulation of Hydrocarbon Contaminated Soil," ASCE J. of Materials in Civil Eng., Vol. 15, No. 2, pp. 166-173.

Tarefder, R. A., Zaman, M. M., and Ting, C. F. (2005a). "Testing of RAP materials for overlay design standards," Proceedings of the Construction Research Congress, Broadening Perspectives, April 5-7, 2005: pp. 1089-1098.

Tarefder, R. A., Zaman, M., Sisson, R., and Ting, C. F. (2006). "Field and laboratory properties of lime-treated cold In-place recycled asphalt pavements," ASTM Journal of Testing and Evaluation, Vol. 34, Issue :1, pp. 31-41.

Walls III, J., and Smith, M., R. Life Cycle Cost Analysis in Pavement Design- Interim Technical Bulletin, FHWA report FHWA-SA-98-079 Federal Highway Administration (FHWA), Washington, DC, 1998.

Wasiuddin, N. M., (2007) "Effect of Additives on Surface Free Energy Characteristics of Aggregate and Binders in Hot Mix Asphalt," Ph.D. Dissertation, University of Oklahoma, Norman, OK.

Wasiuddin, N. M., Zaman, M. M., and Commuri, S. (2009b). "Calibration of Non-Nuclear PQI Gauges and Field Comparison of PQI and Nuclear Gauge Densities," International Journal of Pavement Research and Technology, Vol. 2, Issue: 5, pp. 181-187.

Wasiuddin, N. M., Zaman, M. M., and O'Rear, E. A. (2008). "Effect of Sasobit® and Aspha-Min® on wettability and adhesion between asphalt binders and aggregates," Transportation Research Record, No. 2051, Journal of the Transportation Research Board, Washington, D.C., pp. 80–89.

Wasiuddin, N., and Zaman, M., Nairn, R., and Kolothody, N. (2006). “Characterization of Chat- Asphalt for Road Base Applications,” Journal of Solid Waste Technology and Management, Volume 32, Number 3, pp. 125-137.

Wasiuddin, N., Zaman, M., Nairn, R., Miller, G., Navaratnarajah, S., and Teredesai, R. (2009a). “Maximum Chat Utilization in Asphalt Paving at Tar Creek Superfund Site-Test Road,” Final Report, Submitted to ODOT, Oklahoma Department of Transportation, OK.

West, R., Rodezno, C., Julian, G., Prowell, B., Obsorn, L. V., Field Performance of Warm Mix Asphalt Technologies, NCHRP Report 779, Transportation Research Board, Washington, DC, 2014.

Willis, R., J., Life-Cycle Assessment of 2012 NCAT Pavement Test Track Green Group Mixtures, NCAT Report 14-02, National Center for Asphalt Technology, Auburn, IL, 2014.

Zaman, M., Hossain, N., Singh, D., Alam M. R., Shazzad S.M., and Tim, D. (2015). "Field Performance Monitoring and Modeling of Instrumented Pavement on I-35 In Mcclain County (Phase 2)." Final Report, Submitted to: Oklahoma Department of Transportation, April 2015, The University of Oklahoma, Norman, OK, 2015 (pp. 64).