CONSTRUCTING HIGH PERFORMANCE ASPHALT PAVEMENTS BY IMPROVING IN-PLACE PAVEMENT DENSITY

Final Report

PROJECT SPR 826

Oregon Department of Transportation

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by

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16. Abstract: Increasing the density of asphalt concrete materials is expected to result in significant economic and environmental benefits by increasing asphalt mix strength and reducing cracking and rutting. In addition, reduced air-void content is expected to reduce permeability and moisture-induced pavement damage. Reduced permeability is also expected to reduce asphalt mix aging in the field and mitigate top-down cracking, which is currently the most critical distress type in Oregon. Improved cracking performance is expected to result in reduced life cycle costs, increased pavement condition ratings, and reduced roughness for the Oregon roadway network.

This research study provided information and guidelines for ODOT to implement in asphalt mixture design to achieve high density and better compactibility during construction. Current Contractor Mix Design guidelines (suggested limits for filler contents, gyration levels, etc.) are expected to be modified based on the findings of this research study. Pilot sections with high densities (95-96%) were suggested to be constructed by using the asphalt mixes designed by following the findings of this research study. Long-term performance of pilot sections should be monitored by automated pavement condition surveys (performed every two years by ODOT) to further evaluate the impact of high density on rutting and cracking resistance. Using the results of this study, cost and performance benefits of using high-density mixes were quantified by life-cycle cost analysis (LCCA). Environmental impact of using warm-mix asphalt and increased recycled asphalt pavement (RAP) content strategies on compactibility and cracking resistance was also quantified and evaluated by using pavement life-cycle assessment (LCA).

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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	millimeters squared	mm^2
ft^2	square feet	0.093	meters squared	m^2
yd^2	square yards	0.836	meters squared	m^2
ac	Acres	0.405	hectares	ha
mi^2	square miles	2.59	kilometers squared	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	ml
gal	Gallons	3.785	liters	L
ft^3	cubic feet	0.028	meters cubed	m^3
yd^3	cubic yards	0.765	meters cubed	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	Mg
		TEMPERATURE (exact)		
°F	Fahrenheit	(F-32)/1.8	Celsius	°C

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	millimeters squared	0.0016	square inches	in^2
m^2	meters squared	10.764	square feet	ft^2
m^2	meters squared	1.196	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	kilometers squared	0.386	square miles	mi^2
		VOLUME		
ml	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	meters cubed	35.315	cubic feet	ft^3
m^3	meters cubed	1.308	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams	1.102	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celsius	1.8C+32	Fahrenheit	°F

*SI is the symbol for the International System of Measurement

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

1.0 INTRODUCTION

Asphalt pavements constitute more than ninety percent of the nation's roadway network while 95% of Oregon Highway network is composed of asphalt surfaced pavements (asphalt concrete and chip seals). Besides serving the key role of providing smooth and durable all-weather surface that benefits range of users, they are also the lifelines of the nation and the world and contribute tremendously towards economic and social development. Asphalt pavements are easy to maintain and have a rapid construction process. They also provide smooth and quiet ride, and have the highest level of durability at a reasonable cost. However, overall cost of construction, maintenance, and preservation of asphalt pavements can be high due to the massive size of the U.S. roadway network. Based on 2008 data, the total annual cost of roadway construction, maintenance, and rehabilitation in the U.S. was $182.1 billion (FHWA 2010). Considering their widespread use and importance, it becomes imperative to integrate the concept of sustainability (consideration of economical, societal, and environmental factors in the decision-making process) into pavements.

According to the 2020 ODOT Pavement Condition Report, current ODOT pavement program is underfunded, which is expected to result in a decline in pavement conditions in Oregon within the next 4 years. An estimated $220 million a year funding level is needed to repair pavements that are in poor condition, while providing timely preventive preservation and maintenance on roads in fair-or-better condition. However, pavement program funding levels after 2021 are planned to be around $107 million (expected 21-24 annual STIP funding) per year according to the report (almost half of the needed funding level). For this reason, implementing innovative and sustainable asphalt mixture design strategies to improve long-term pavement performance in Oregon is critical.

In recent years, there have been several advancements towards improving and enhancing the sustainability of asphalt pavements. Sustainability principles encompass the methods to reduce negative impacts associated with pavement construction and material production activities. The impact of roadway roughness on vehicle operating costs (in terms of vehicle maintenance and fuel use) and the environment is also another factor that should be considered while selecting the most sustainable pavement strategies (Harvey et al. 2016; Sogol et al. 2017). Some of the sustainable practices associated with pavement engineering are the increasing use of recycled materials and modified binders. The use of Reclaimed Asphalt Pavement (RAP) and Reclaimed Asphalt Shingles (RAS) in asphalt mixtures as a substitute for virgin material (asphalt binder and aggregate) directly brings down the upfront cost of a project (Hansen and Copeland 2015; Willis et al. 2013) and allows agencies to pave a larger roadway network to improve overall network-level pavement roughness. Recycling does not only create positive economic impacts but also has environmental benefits, which together help in achieving some of the sustainability goals.

The use of recycled materials in asphalt pavements have proven to be quite beneficial in reducing the overall cost of pavement construction. However, it is not the exclusive method to achieve the sustainability targets. The required investment on building pavement infrastructure can also be reduced by improving the long-term performance. One simple and comparatively low-cost way of improving the pavement performance is by achieving higher levels of asphalt density during construction. The pavement's durability can be improved, its service life can be extended and the frequency of rehabilitation and reconstruction can be reduced by achieving higher levels of asphalt concrete density during construction.

Several recent research studies (Fisher et al. 2010; Tran et al. 2016; Coleri et al. 2018; Sreedhar and Coleri 2018) showed that increasing asphalt concrete pavement density by modifying mix design methods, using compaction aids in asphalt mixes, and following better construction practices can lead to significant performance improvements and cost savings. Results of a study conducted by Tran et al. (2016) indicated that the long-term fatigue cracking and rutting performance of asphalt pavements are improved by 33.8% and 66.3%, respectively, by achieving just 1% reduction in air voids.

In Oregon, asphalt concrete fatigue cracking is one of the major distress modes. ODOT's Pavement Management System has shown that asphalt mixes placed in the last 20 years have had a tendency to develop premature cracking after 6 to 8 years of service before reaching the structural design life of 15 years. ODOT research project SPR 785 (Coleri et al. 2017) showed that a 2% reduction in air-void content (increasing density by 2% during construction) increases the cracking resistance of asphalt mixes by 1.5 to 2 times. For this reason, producing asphalt mixtures that are easy to compact and achieving higher densities during construction can potentially create a significant improvement in the cracking resistance of asphalt mixtures. Since the impact of high density on reducing asphalt aging (results in top-down cracking which is the major distress mode in Oregon) and moisture sensitivity (which is also a critical factor controlling pavement performance in Oregon) was not investigated in SPR785, improving asphalt compaction and increasing in-place density during construction are expected to result in a cracking performance improvement significantly higher than the benefits reported in the published SPR 785 ODOT research report.

Since the effect of increased density in asphalt mixes is highly significant, it is expected that the cost of achieving this higher in-place density can be considerably less than the cost savings made on operation and maintenance from the prolonged service life of the pavements. This result makes the idea of having asphalt mixes with improved compactibility highly cost effective. It should be noted that the emphasis on adequate compaction is not new. For instance, as early as in 1977, the New Jersey Turnpike construction engineers described compaction as the most critical process in construction which can ensure long-term serviceability of asphalt pavements (Hughes 1989).

However, suggesting an increase in density without providing guidelines on how to achieve them can result in a negative impact on asphalt mix durability. For instance, increasing mix density by using excessive amounts of fillers and asphalt binder can result in long-term durability issues. Thus, current mix design procedures and mix compaction processes should be improved to produce high density and high-performance asphalt mixes during construction without creating a detriment to the overall performance of the pavement. This study aims to provide practical modifications to mix design procedures for achieving higher in-place density (compactibility) and thereby increasing the long-term performance of asphalt mixtures.

1.1 KEY OBJECTIVES OF THIS STUDY

The main objectives of this study are to:

- develop recommendations for the current mix design procedures to increase density (compactibility) and to improve long-term performance of asphalt mixes;
- quantify the impact of increasing density by using additives (polymer modification, warm mix technologies, etc.), increasing compaction temperatures, and increasing binder and filler content on cracking and rutting performance; and
- quantify the impact of all developed suggestions for mix design procedures and guidelines on constructibility, density, cost, and environmental impact of asphalt pavements.

2.0 LITERATURE REVIEW

2.1 IMPACT OF IN-PLACE DENSITY ON LONG-TERM PAVEMENT PERFORMANCE

Pavement performance greatly depends upon the in-situ asphalt mixture density. Density and air void content (or percent air voids) are interrelated terms and sometimes used interchangeably. Air void content is the ratio of the volume of air voids to the total volume of an asphalt mix. It is expressed in percentage. When a loose asphalt mix is compacted, it gets densified up to a certain percentage of its theoretical maximum density (TMD) or G_{mm}. The compacted specimen has a matrix of asphalt coated aggregates packed together with air trapped between them. Thus, difference between the bulk density of the compacted specimen (G_{mb}) and TMD of the mix can be attributed to these air voids. This can be expressed using the following Equation **Error! Reference source not found.**:

$$\boldsymbol{Air\ voids\ (percent) = \frac{G_{mm} - G_{mb}}{G_{mm}}}$$

(2-1)

Where:

G_{mm} = Theoretical maximum specific gravity of the mixture; and

G_{mb} = Bulk specific gravity of the mixture.

The conventional method of determining the bulk specific gravity of an asphalt mix is by measuring the mass of a compacted sample in dry, submerged and surface saturated dry (SSD) condition. For actual pavement sections, the bulk density is determined either indirectly by the same procedure on cores obtained from the pavement section or directly by nondestructive testing. These standard procedures have been discussed in detail in the subsequent section. Theoretical maximum specific gravity, also known as the Rice density, is measured by using water displacement method on the loose mix. It becomes necessary to precisely measure bulk and theoretical maximum specific gravity because in-place density of the asphalt layer i.e. the density after compaction on site has been linked directly with the pavement longevity. With similar reasoning, the air void content has been correlated with the pavement performance and durability. Some of these studies are given in the following paragraphs.

In collaboration with the Kentucky Transportation Center at the University of Kentucky and Kentucky Transportation Cabinet, Asphalt Institute investigated the impact of asphalt pavement density on the pavement durability. It was concluded that the pavements, which do not achieve ninety two percent of G_{mm} after compaction, are more vulnerable to premature pavement distresses. The most common form of distresses are cracking, rutting, premature oxidation aging, structure weakening, stripping and raveling (Fisher et al. 2010). Theoretically, lower density implies higher amount of air voids present in the compacted asphalt concrete layer which weakens the asphalt microstructure. Filling those voids with asphalt binder is expected to result in higher density and increased rutting and cracking resistance. In addition to this improved strength and ductility, increasing density also reduces the permeability of the asphalt layer and moisture damage, which is a major issue in Oregon due to frequent rain events. Reduced permeability also reduces the penetration of air into the asphalt layer and reduces asphalt aging. This reduced aging also lowers the likelihood of having top-down cracking issues, which is the major distress mode in Oregon (Coleri et al. 2017; Coleri et al. 2018).

Past studies have shown that reduced in-place density has detrimental effects on the fatigue life of asphalt pavements (See Table 2.1). Three asphalt mixtures, each with different grading and binder content were selected in a study by Epps and Monismith (1969). The mixes used were British Standard 594 grading, California Fine Grading, and California Coarse Grading with 7.9%, 6% and 6% binder content respectively. These were subjected to controlled-stress fatigue tests. The test results indicated that high air void content mixes exhibit shorter fatigue lives. On average, 1% reduction in air voids created a 32.7% improvement in fatigue life (Seeds et al. 2002) (Table 2.1).

Harvey and Tsai (1996) used bending beam fatigue test (BBF) to determine the impact of density and binder content on fatigue cracking resistance. Asphalt mixture with three different levels of air void and five different binder content were tested. Results of this study showed that fatigue life increases with decreasing air void content and increasing binder content. Reduction in air void content from 8 to 5 percent led to a 100 to 200 percent increase in fatigue life. Likewise, when binder content was increased by 0.5 percent, the fatigue life increased by 10 to 20 percent (Table 2.1).

Epps et al. (2002) also examined the effect of binder content, aggregate gradation, and density on asphalt mixtures' fatigue lives. It was observed for all cases that increased density improves fatigue cracking resistance. In addition, increasing the density of asphalt mixtures with finer gradations results in more significant improvement in fatigue life when compared to the improvement for the mixes with coarser gradations (See Table 2.1).

Table 2.1. Summary of Results on Effect of HMA Fatigue Performance due to Air Void Content (Seeds et al. 2002)

Study	Lab/Field Experiment	Mix Type	Increase in Fatigue Life for 1% Decrease in Air Voids
UCB (Epps and Monismith, 1969)	Lab	British Standard	20.6%
		California Fine	43.8%
		California Coarse	33.8%
UCB (Harvey and Tsai, 1996)	Lab	California Dense-Graded	15.1%
WesTrack (Epps et al., 2002)	Lab	Fine	13.5%
		Fine-Plus	13.3%
		Coarse	9.0%
	Field	Fine/ Fine-Plus	21.3%
		Coarse	8.2%

Using these results from three research studies, Seeds et al. (2002) concluded that 1% reduction in air void content results in about a 20% improvement in fatigue cracking resistance on average. Knowing that reduced air void content also generally improves rutting resistance of asphalt mixtures, improving in-place density can create significant improvements in pavement longevity.

William and Shaidur (2015) investigated the causes of early cracking on the State of Oregon highways system. The results of their density evaluations indicated that all six of the sections with top-down cracking showed higher variability in density compared to the four non-cracked pavement sections. The results formed the basis of a recommendation to reduce the design air voids from 4 percent to 3 percent. It was indicated that this reduction in design air void content would effectively increase the design binder content of mixes by approximately 0.25% and it would also be beneficial in lowering in-situ air voids. Thus, the study confirms that in-place density greatly affects the fatigue cracking performance

Coleri et al. (2017) conducted tests on the asphalt mixtures used in Oregon to evaluate the fatigue cracking performance and select the most effective cracking experiment for Oregon. Results of this study showed that air void content significantly affects the flexibility index [FI, a parameter from the semi-circular bend (SCB) test showing the fatigue cracking performance of the asphalt mix]. Figure 2.1 shows the result of SCB tests conducted on three mixes (M1: Mix 1-PG70-22ER-Fine gradation, M2: Mix 2-PG70-22ER-Coarse gradation, M3: Mix 3-PG70-22-Coarse gradation) produced in the laboratory with two binder contents (5.3% and 6%) and two air-void contents (5% and 7%). FI was the parameter used to evaluate the fatigue cracking resistance of the test samples and it was found that a 2% reduction in air void content increased the FI by 1.5 to 2 times, which points out the importance of producing HMA with lower air voids. Results of flow number (FN, a test used to evaluate rutting resistance of asphalt mixtures) tests also showed that asphalt mixtures with higher densities always provided higher rutting resistance (Figure 2.2). In other words, improving density improves both rutting and cracking resistance (Sreedhar and Coleri 2018). This result points out the importance of achieving high in-place density during construction to improve the condition of the pavement network in Oregon.

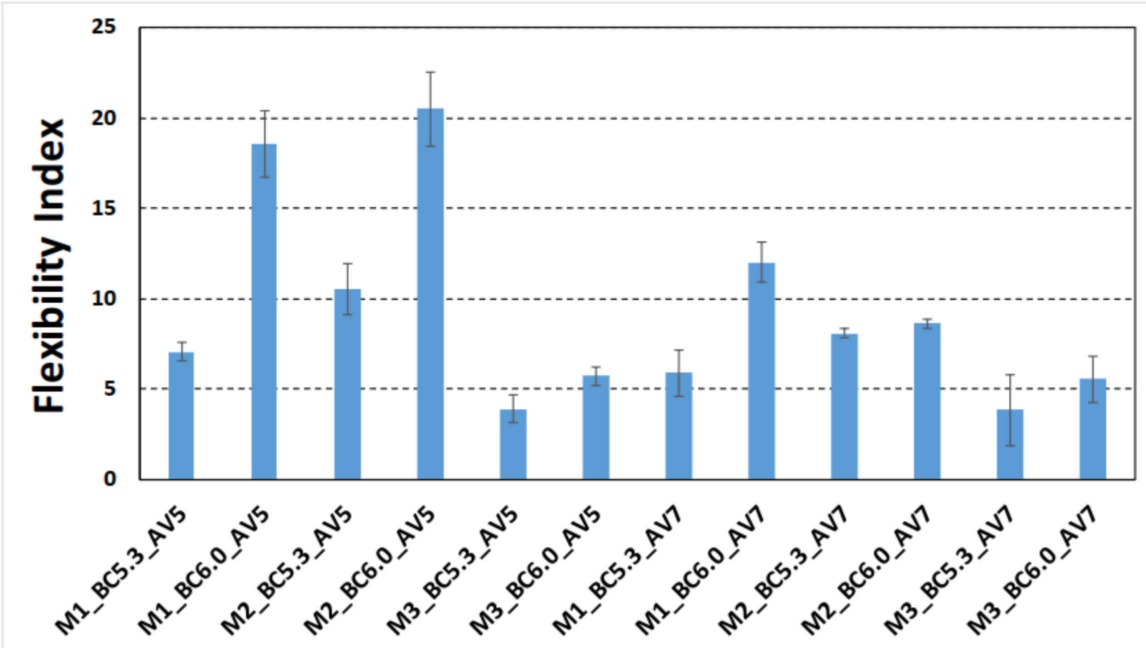

Figure 2.1. FI for the mixtures with different binder contents (BC), and air void contents (AV) (Coleri et. al., 2017)

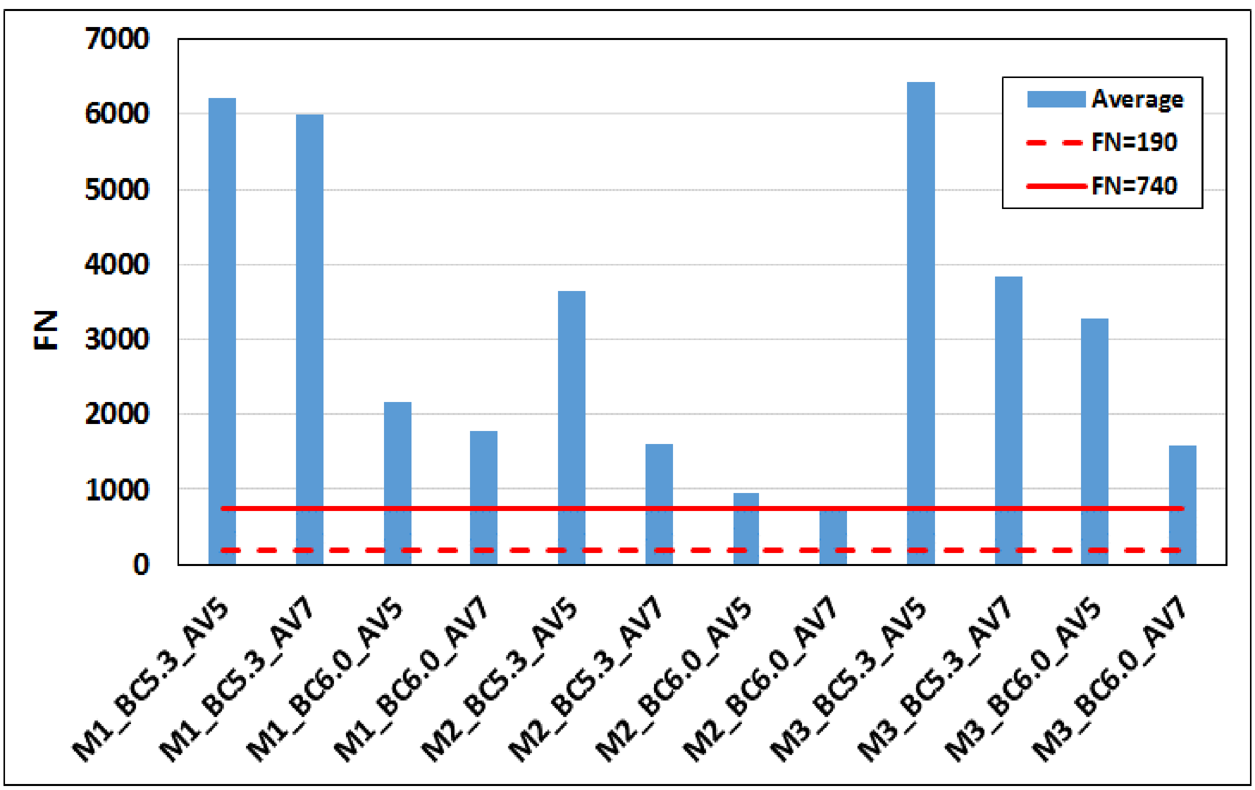

Figure 2.2. Flow number for mixes with different binder contents (5.3% and 6%), air void contents (5% and 7%) (Coleri et. al., 2017).

Other studies have also made similar conclusions that the pavement's in-situ density affects its rutting performance significantly. By conducting Flow Number (FN) test on five different mixtures with three replicates of each and doing a regression analysis on the data obtained, Blankenship and Zeinali (2018) observed that the asphalt mixture density is highly correlated to the FN. Higher density improves the FN which means rutting resistance of pavements enhances with increased in-place density. Previously, Blankenship and Anderson (2010) had also shown that the FN increases by 34% if the density is increased by 1.5%. Fisher et al. (2010) also arrived at a similar conclusion in their Asphalt Institute study. Measured dynamic modulus and FN values from their study have been plotted against air voids in Figure 2.3. It can be concluded that that the FN (rutting resistance) decreases with increasing air void content.

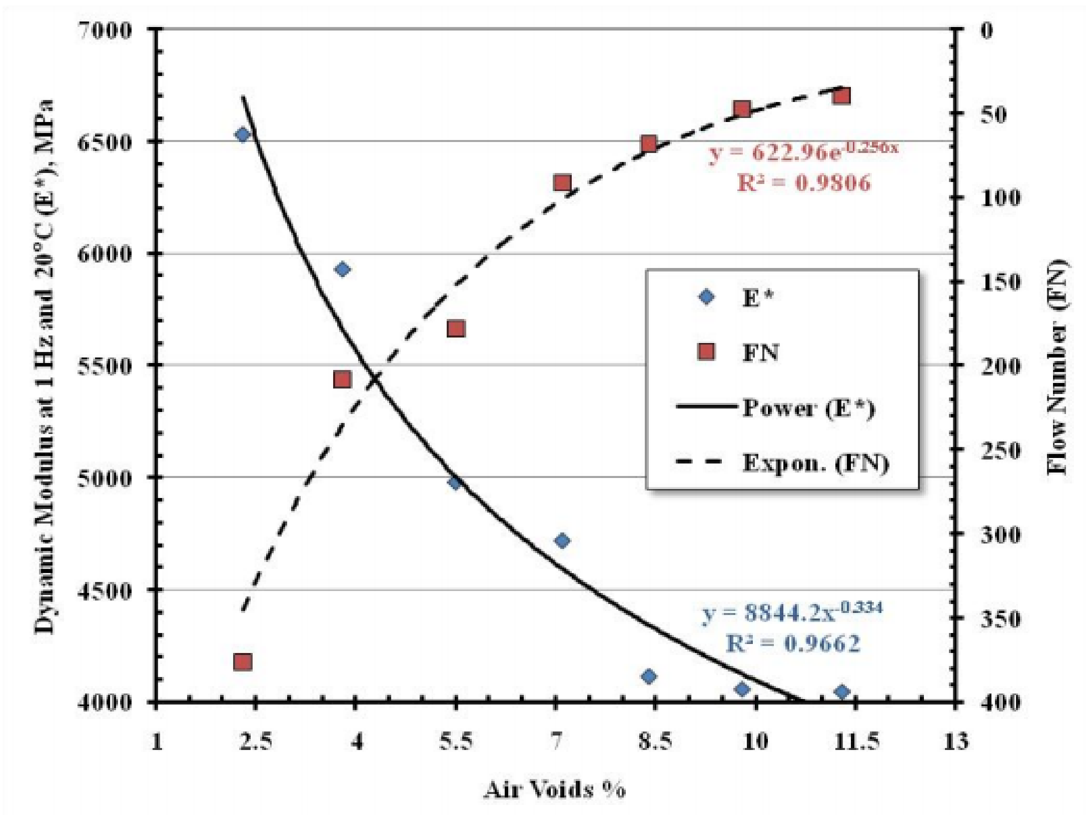

Figure 2.3. Dynamic modulus and flow number as a function of air voids in HMA mixture (Fisher et al. 2010)

Tam et al. (1989) and Lindel et al. (1989) indicated that air void content is also related to the rate of aging. Thus, increasing density will be beneficial in reducing age-hardening (Zhao 2011). Moreover, based on previous studies, it has also been observed that one percent increase in air-void content above the target reduces HMA stiffness by five percent (Seeds et al. 2002). Thus, air voids have been found to affect the stiffness as well as the rate of aging of the asphalt mixes.

The mechanism behind the pavement deterioration due to higher air void content and low density is based on the void structure of the asphalt mixes. Inadequate compaction of the asphalt layer during construction results in a higher air void content. If the asphalt layer has an interconnected void structure after compaction, the potential for deterioration of asphalt pavement layer increases due to increased infiltration of air and water into the asphalt layer. Infiltration of air and water causes reduction in ductility due to excessive aging and stripping (Coleri et al. 2013), respectively. High air void content also reduces the stiffness of the mixture which can lead to permanent deformation.

Although majority of the studies in the literature showed an improvement in both rutting and cracking performance due to increase in-place density, excessively low in-place air voids (less than 3%) have also been shown to be connected with distresses such as flushing/ bleeding and shoving/rutting. Brown and Cross (1989) compared prematurely rutted pavements with the pavements that had shown no sign of rutting after more than ten years of service. Huber and Herman (1987) had also conducted a similar study. The conclusion in both studies was that low (less than 3%) air void content can create excessive shear related deformation (change in shape of the material due to aggregate movement) due to inadequate level of densification allowed in the mix. In other words, due to extremely low air-void content, the energy coming from vehicular loads cannot be dissipated by creating a reduction in the air-void content of the asphalt layer during the secondary compaction phase. However, according to McDaniel and Levenberg (2013), low air void may not always be an issue as there are some other factors which determine its impact such as asphalt mix properties, volume of traffic, climatic conditions etc.

2.2 STRATEGIES TO INCREASE IN-PLACE DENSITY

In-place density has a considerable impact on asphalt pavement performance. Moreover, higher in-place density has proven to increase the pavement service life and reduce life cycle cost of the asphalt pavements. Several research studies have contributed towards developing methods to help achieve a higher in-place density. In this section, possible strategies to increase in-place density are discussed based on the findings from the literature.

2.2.1 Increased Asphalt Binder Content

HMA, in the simplest form, is combination of asphalt binder, aggregate of different sizes and air voids. A simple but effective tool to represent the volumetric relationship between the three components is the phase diagram as shown in Figure 2.4.

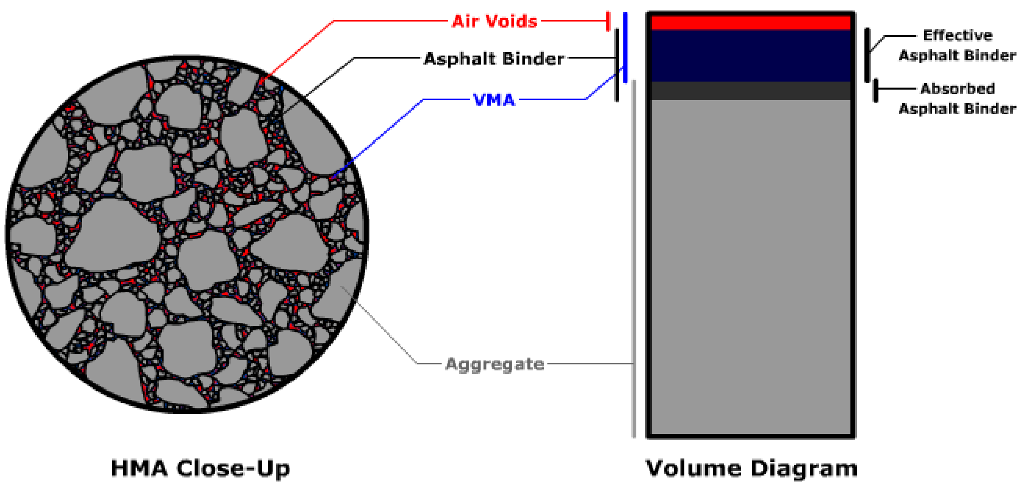

Figure 2.4. Volumetric relationship of HMA components (*Source: www.pavementinteractive.org*)

VMA or voids in the mineral aggregate is the volume of air void space between the aggregates. These spaces are occupied by air and asphalt binder. A properly designed and compacted asphalt mix should have sufficient air voids as it directly affects the permeability. Asphalt binder provides a coating on these aggregates and makes the mix more compactible leading to reduction in air voids by increasing the in-place density. Mixes with low binder content have inadequate lubrication and are harder to compact. If there is high binder content, the proportion of air void decreases in VMA as the binder occupies most of the space as thicker films coating the aggregates. Thus, binder content directly affects the air void content present in the pavement after compaction. Consequently, increasing the binder content is a reasonable method to achieve higher mix density. However, excessive binder content may lead to over-compaction resulting in insufficient air voids in the pavement. In such cases, asphalt is pushed on to or bleed out of the pavement surface under the traffic load over time (Finn and Epps 1980). This phenomenon is termed as bleeding. Moreover, high binder content results in thicker binder film around the aggregates reducing the aggregate to aggregate friction of the mix. This lowers the stability (resistance to permanent deformation under traffic loads) of the pavement. Figure 2.5 and Figure 2.6 show the distresses in pavement resulting due to excessive binder content.

Figure 2.5. Shoving of asphalt pavement due to excessive binder content and low air void in the mix (McDaniel and Levenberg 2013)

Figure 2.6. Rutting in the pavement due to high binder content and low air void in the asphalt mix (McDaniel and Levenberg 2013)

Binder content in the asphalt mix design is dictated by the target air-void content in the compacted sample. The optimum binder content in Superpave method is determined using a four-step process. Several trial blends with different asphalt content are prepared. Compaction of these trial mixes is then carried out in the Superpave Gyratory Compactor (SGC). The number of gyrations for compaction is selected based on ODOT asphalt mix design specification. Then, the volumetric properties (including air-void content) of the laboratory compacted mixes are determined. A curve between air void content and asphalt binder content is plotted. Finally, the binder content required to achieve the target air-void content is selected as the design/optimum binder content of the mix.

Aschenbrener et al. (2017) looked into the concerns of Superpave mix design producing too dry (low asphalt content) asphalt mixes leading to durability issues. Noting some example cases using Superpave 5, Level 2 and Level 3 mix designs, the report puts a strong argument in favor of incorporating performance testing into current mix design procedures to reduce distresses such as cracking, rutting and moisture damage. Hekmatfar et al. (2015) also showed that an increase in in-place density was achievable with optimum binder content chosen at 5 percent air voids, rather than the currently specified 4 percent, and using 50 gyrations as the compactive effort for asphalt specimen preparation. It was concluded that if the mixtures are designed in the laboratory to have the same density as the target density in the field, better compaction and higher density would be easier to achieve during construction. In other words, different from Superpave 4 that uses 4% air void content as the target to determine the optimum binder content, ***Superpave 5 design method*** suggests designing the asphalt mixtures for a target 5% air void content. In this process, the objective during construction is also going to be reaching 95% density.

Fisher et al. (2010) showed that increasing asphalt content in a mixture increases the potential to achieve higher field densities. In this study, 0.5 and 1 percent extra asphalt binder was added to the mixture design as a what-if scenario. Test specimens were compacted to 7 ± 0.5 percent air voids in lab and then the fatigue cracking, dynamic modulus, and flow number (rutting) tests were conducted with the prepared samples. In addition, the repeated shear at constant height (RSCH) and asphalt pavement analyzer (APA) tests were also included. The study concluded that this increased binder content should produce a better performing mix. The impact of the varying asphalt content on fatigue life, rutting resistance and gyratory density based on the additional tests (APA and RSCH) is shown in Figure 2.7.

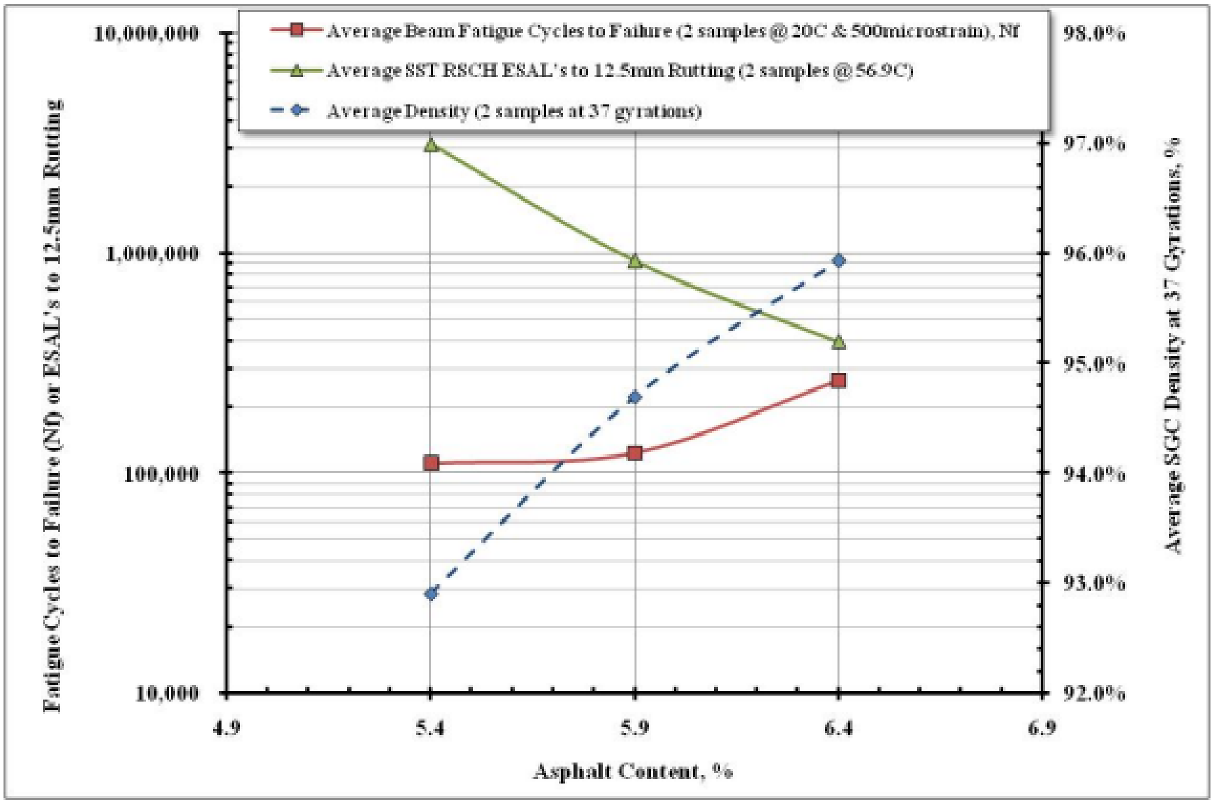

Figure 2.7. Effect of asphalt content on fatigue, rutting and gyratory density (Fisher et al. 2010)

Higher binder content tends to reduce air-void content of the mix (for equal compaction effort), as expected. However, increased binder content tends to considerably increase the rutting potential and construction cost (Blankenship and Zeinali 2018). Therefore, it was concluded that increasing binder content is not always a suitable density improvement measure. However, this study only evaluated the construction costs while the performance benefits of this strategy and corresponding impact on the overall life-cycle costs were not investigated.

Development of premature longitudinal cracks on a newly rehabilitated pavement section in Colorado caught attention of researchers. The rehabilitation work consisted of milling and filling of 3 inches of existing pavement with a new asphalt mix. Several such occurrences over time called for a deeper investigation of pavements all across Colorado. Results of the investigation suggested that 18 of the 28 sites which were evaluated, showed signs of top-down cracking resulting from segregation (Figure 2.8). Harmelink et al. (2008) showed that increasing the binder content puts a check on segregation and consequently on top-down cracking incidences. This study recommended decreasing the number of design gyrations (N_{des}) to accommodate the increased binder content in the mix.

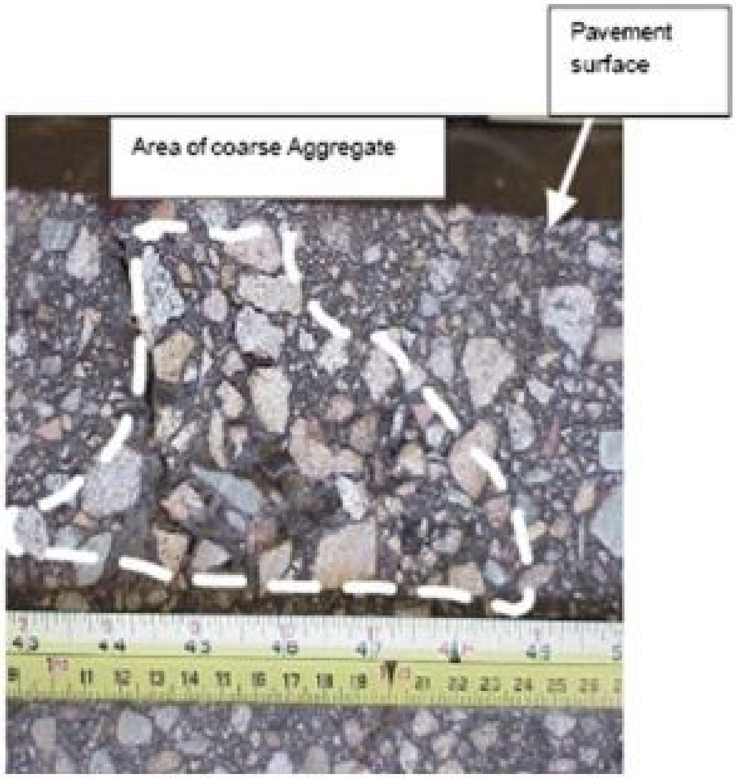

Figure 2.8. Segregation and cracking (Harmelink et al. 2008).

2.2.2 Binder Performance Grade

Before the advent of Superpave design method, Hveem and Marshall Design methods were in use. In these design methods, asphalt binders were characterized according to their physical properties. Penetration grading and viscosity grading were two such grading systems. Penetration grading was determined by the depth penetrated in binder by a standard needle when 100 g load is applied on the needle for 5 seconds. This was a simple grading system. However, this system relied on just one standard temperature of 77ºF for characterization of the binder. The test equipment for the penetration grading system is shown in Figure 2.9.

Another grading system to measure penetration of asphalt binder is viscosity grading (symbolized as AC grades). This system measures asphalt binder's viscosity at two different temperatures, 140ºF and 275ºF. The drawback of this grading system is that it does not test the rheology of asphalt binder at low temperatures.

Figure 2.9. Penetration test apparatus used for penetration grading (*Source: www.pavementinteractive.org*)

Performance grading is the concept which takes into account the physical and environmental conditions of the location while selecting the properties of asphalt binder (Kim 2009). Superpave performance grading system involves performance tests which the binder is required to pass at particular temperatures. These test temperatures are based on the extreme temperatures recorded at the pavement location. In addition, binder grade is also dependent upon the duration of loading and the volume of traffic. Since all of the above factors vary from place to place, the performance graded (PG) binder is also location specific. Figure 2.10 shows the summary table of the performance grade specifications.

Performance Grades

		PG 46	PG 52	PG 58	PG 64	PG 70	PG 76	PG 82
Max. Design Temp.		PG 46	PG 52	PG 58	PG 64	PG 70	PG 76	PG 82
Min. Design Temp.		-34 -40 -46	-10 -16 -22 -28 -34 -40 -46	-16 -22 -28 -34 -40	-10 -16 -22 -28 -34 -40	-10 -16 -22 -28 -34 -40	-10 -16 -22 -28 -34	-10 -16 -22 -28 -34
Original								
≥230 °C	**Flash Point**							
≤ 3 Pa-s @ 135 °C	**Rotational Viscosity**							
≥ 1.00 kPa	**DSR G*/sin δ** (Dynamic Shear Rheometer)	46	52	58	64	70	76	82
(Rolling Thin Film Oven) RTFO, Mass Change ≤ 1.00%								
≥ 2.20 kPa	**DSR G*/sin δ** (Dynamic Shear Rheometer)	46	52	58	64	70	76	82
(Pressure Aging Vessel) PAV								
20 hours, 2.10 MPa		90	90	100	100	100(110)	100(110)	100(110)
≤ 5000 kPa	**DSR G*sin δ** (Dynamic Shear Rheometer) Intermediate Temp. = [(Max. + Min.)/2] + 4	10 7 4	25 22 19 16 13 10 7	25 22 19 16 13	31 28 25 22 19 16	34 31 28 25 22 19	37 34 31 28 25	40 37 34 31 28
S ≤ 300 MPa m ≥ 0.300	**BBR S (creep stiffness) & m-value** (Bending Beam Rheometer)	-24 -30 -36	0 -6 -12 -18 -24 -30 -36	-6 -12 -18 -24 -30	0 -6 -12 -18 -24 -30	0 -6 -12 -18 -24 -30	0 -6 -12 -18 -24	0 -6 -12 -18 -24
If BBR m-value ≥ 0.300 and creep stiffness is between 300 and 600, the Direct Tension failure strain requirement can be used in lieu of the creep stiffness requirement.								
ε_f ≥ 1.00%	**DTT** (Direct Tension Tester)	-24 -30 -36	0 -6 -12 -18 -24 -30 -36	-6 -12 -18 -24 -30	0 -6 -12 -18 -24 -30	0 -6 -12 -18 -24 -30	0 -6 -12 -18 -24	0 -6 -12 -18 -24

Figure 2.10. Performance graded asphalt binder specification (Asphalt Institute)

In the summary table above:

- The PG grade designations are shown in the top two rows. The ‘max. design temp.’ row denotes the average 7-day maximum temperature and the ‘min. design temp.’ row shows the range of minimum pavement temperature as sub-columns below each maximum pavement design temperature. The maximum and minimum pavement temperature recorded at a location are thus used to designate the PG grade of binder. For example, PG 70 in the first row denotes that the 7-day maximum pavement temperature of a location is between 64^0C and 76^0C. The minimum pavement design temperature is then selected from the sub-columns under PG 70 cell. For the same example, if the minimum temperature is selected as -22ºC then the binder is designated as PG 70-22.

- The PG classification lays down the required temperatures at which the different performance tests are carried out. The binder is then subjected to these tests which address pavement distresses (three in particular) such as permanent deformation, fatigue cracking and thermal cracking.

- The above-mentioned tests are run on unaged binder, short-term aged binder and long-term aged binder. The short-term aging is simulated by the rolling thin film oven (RTFO) and the long-term aging is simulated by the pressure aging vessel (PAV).

The effect of binder grade on the compactability of asphalt mix can be understood by the concept of viscosity. Viscosity is defined as resistance to flow and it measures the internal friction of a fluid. Therefore, it has a role to play during mixing and compaction of asphalt mix. If the binder has high viscosity, the resulting mix does not sufficiently rearrange under the applied compactive effort to achieve the target density. On the contrary, the mix becomes unstable if the viscosity of the binder is too low. The viscosity of binder is a function of temperature. Accordingly, viscosity-temperature chart is used to determine the mixing and compaction temperature of the asphalt mix (Hensley and Palmer 1998).

Schmitt et al. (2009) studied the effect of temperature and pressure on the density gain of asphalt mixes. Loose mix samples were collected during a construction season in Wisconsin. These samples were subjected to compaction using the Superpave Gyratory Compactor at different temperature and pressure. The factors responsible for densification of mixes in field as well as in lab were recorded. The factors included mat temperature, type of roller, whether the vibratory setting was on or off, number of passes, PG binder grade etc. Notably, lab compaction results did not show any significant effect of PG binder on density gain. However, this study did not include PG70-series or other modified binders.

Contrary to the findings of Schmitt et al. (2009), Decker (2006) had considered the binder grade an important factor which contributes to the compactibility of the mix. According to his study, softer binder grades have been used as a technique that facilitates mixing and compaction at a lower temperature i.e. in cold weather.

2.2.3 Increased Placement Temperatures

As pointed out earlier, viscosity of the asphalt binder varies with temperature. Further, viscosity affects the compaction of the mix. Hence, it follows that temperature directly influences the compaction of HMA mix. If the temperature is low, the asphalt binder has higher viscosity, which in turn results in lower densification of the mix. With cooling of the mix, the binder stiffens further, rendering secondary compaction (compaction under traffic loading over longer period of time) ineffective. This temperature is called cessation temperature. At higher temperature, due to low viscosity, the mix may undergo shoving or lateral displacement when rolling load is applied for compaction.

If the placement temperature is high enough, there will be more time available for compaction as the mat would require longer time to cool down. The time available for compaction can thus be extended by increasing the initial mat temperature (Roberts et al. 1996). Fisher et al. (2010) conducted field tests on eight pilot projects to determine the effects of paving temperatures and environmental conditions on laboratory and in-situ pavement density. To measure the core densities, 7 points were chosen from 10 random sections of each project. The seven testing points are represented by solid bubbles in Figure 2.11.

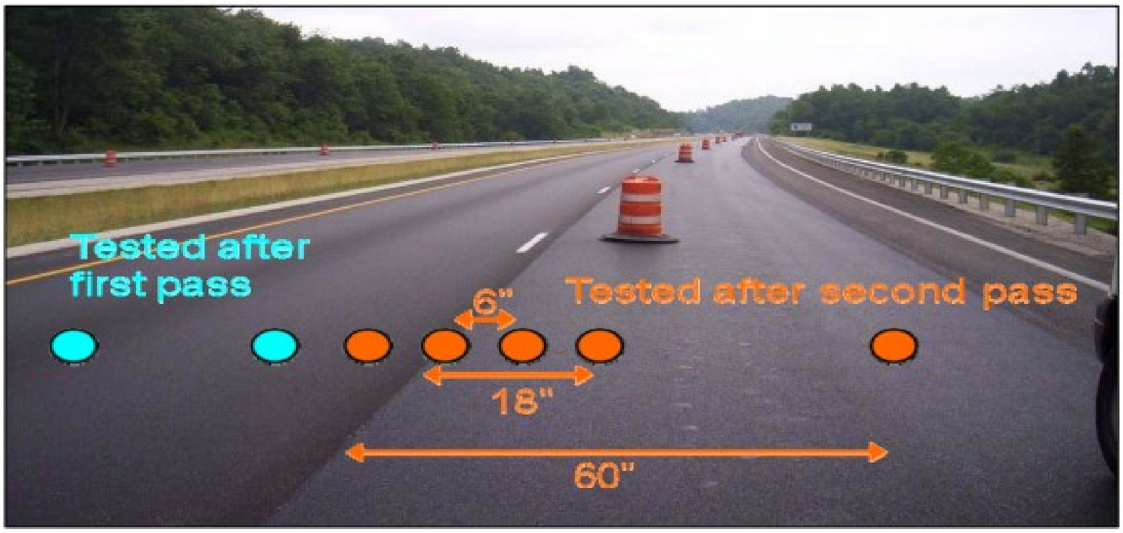

Figure 2.11. Schematic of the surface testing (Fisher et al., 2010)

Test results (core density measurements) were used to find a relationship between the pavement temperatures before compaction and the in-situ pavement density. Figure 2.12 shows the relationship between the two parameters as a percentage of TMD. The data bubbles show the density corresponding to a given range of temperature. The spread of the bubble covers the data points within a temperature range on x-axis. The relationship suggests that the higher pavement density could be achieved if the compaction was carried out at higher mat temperatures.

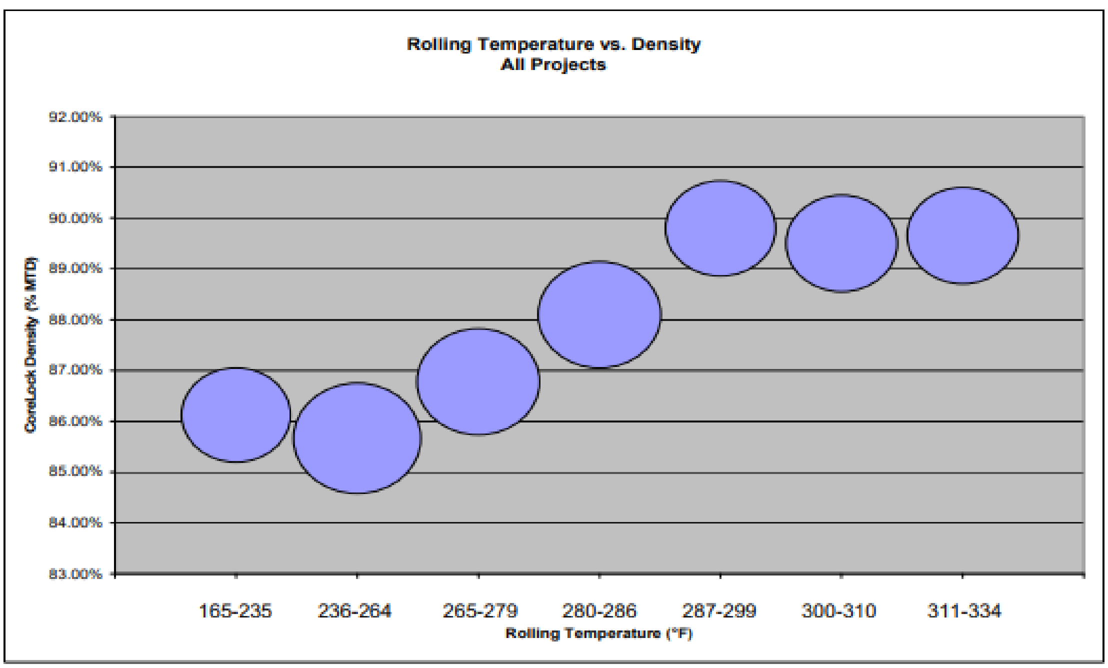

Figure 2.12. Relationship between pavement rolling temperature and in-situ density measured by using the CoreLok device (Fisher et al. 2010). Temperature conversion: [C=(F-32)/1.8]

Saedi (2012) in a study also found that the density of HMA increased with increase in compaction temperature. However, this increasing trend was up to 145ºC only. According to Hughes (1989), a mix that has a higher temperature, can be compacted easily, usually, than the same asphalt mix placed at a lower temperature. Huner et al. (2000) also reported that the compactibility of an asphalt mix increases as the temperature increases. Result of their study are illustrated in Figure 2.13. Effect of temperature on the compactibility of asphalt mixes (Huner et al 2000) Temperature conversion: [C=(F-32)/1.8]

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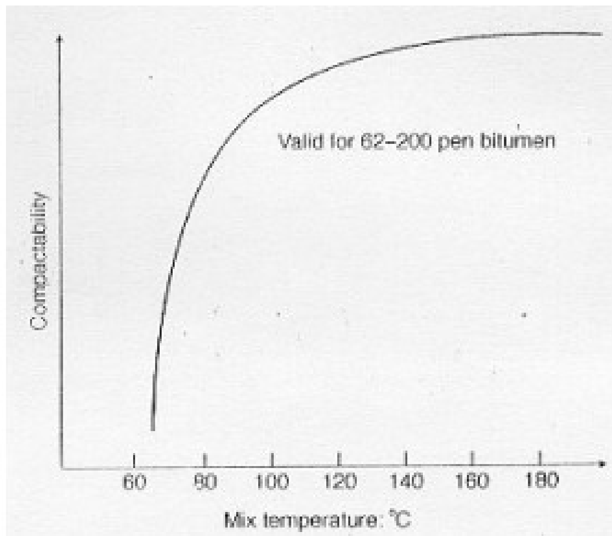


Figure 2.13. Effect of temperature on the compactibility of asphalt mixes (Huner et al 2000) Temperature conversion: [C=(F-32)/1.8]

As per Superpave mix design, the mixing and compaction temperature is determined by the range of temperatures corresponding to 0.17 ± 0.02 and 0.28 ± 0.03 Pa·s viscosities. The temperature-viscosity curve is established is accordance with ASTM D2493 standard. As per the standard, Rotational Viscometer (RV) test is required to be carried out on the unmodified binder sample. However, for modified binders, the mixing and compaction temperatures determined using this approach are higher than expected. The increased compaction temperature has been reported to adversely affect the modifier-binder bond leading to thermal dissociation, and in some cases higher temperature also become a safety issue for workers due to the excessive fumes (Tang and Haddock 2006).

The time available to compact (TAC) is a significant factor in achieving the target density. TAC is generally controlled by mix temperature, base temperature, mix properties, and mat thickness (Decker 2006). Higher placement temperature increases the TAC which results in smoother compaction. Figure 2.14 illustrates the cut-off/ cessation temperature concept. When placement temperature of HMA is much higher than the cut-off temperature, there is a greater possibility of achieving target density.

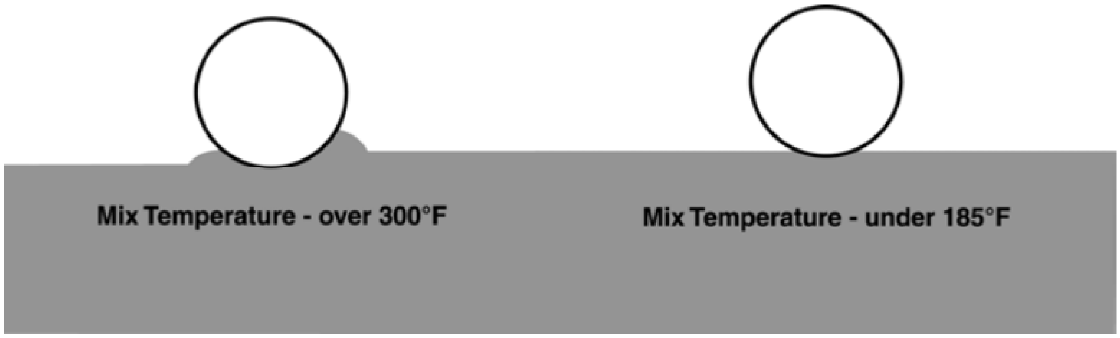


Figure 2.14. High temperature increases the possibility of achieving higher density (Decker 2006) Temperature conversion: [C=(F-32)/1.8]

2.2.4 Modified Gradation and Volumetrics for the Asphalt Mixture

Aggregates constitute 80-85% of the HMA mixture by volume and approximately 95% of the mixture by weight. Strategic Highway Research Program (SHRP) convened an expert panel in 1990's to determine the aggregate properties that are most important for pavement performance. One of the properties included by the panel was gradation. Gradation plays role in determining several HMA properties such as fatigue resistance, stiffness, durability, permeability etc. (Roberts et al. 1996). Most of the studies in the literature suggest that performance of a mix design is dependent upon the balance between volumetric of the mixture and the amount of constituent materials like binder, aggregate and additives, if any.

The gradation, the maximum aggregate size, and the amount of coarse and fine aggregates, play important roles in achieving high density levels during construction (Hughes 1989). Having a uniform distribution of all different size aggregates leads to higher compactibility during construction due to better packing, which reduces the VMA in the aggregate structure. This produces a denser HMA. However, some void space is desirable to accommodate the asphalt binder. Equation 2-2 determines the gradation with maximum density:

$$\boldsymbol{P} = \left(\frac{\boldsymbol{d}}{\boldsymbol{D}}\right)^{\boldsymbol{n}}$$

(2-2)

Where:

P = percent passing through the sieve

d = aggregate size under consideration

D = maximum size of particle/ aggregate

N = factor which adjusts the gradation curve

Based on the Equation 2-2, FHWA came up with a 0.45 power chart as shown in Figure 2.15. This graph uses n = 0.45 in the above Equation 2-2 and determines maximum density line. The gradations can be adjusted accordingly based on the maximum density line. Gradation curve lying farther from the 0.45 power maximum density line has higher VMA resulting in reduced compactibility. Conversely, an aggregate mix with gradation closer to the diagonal straight line representing the maximum density, will achieve better packing. However, increasing the dust content by getting the gradation curve closer to maximum density line can reduce the fatigue cracking resistance of the asphalt mixtures (Coleri et al. 2017). Increased dust content (percent passing No. 200 sieve) will also increase moisture susceptibility of the asphalt mixture. For these reasons, dust-to-binder ratios set by specifications for different mix designs need to be followed to avoid excessive amount of dust in the asphalt mixture.

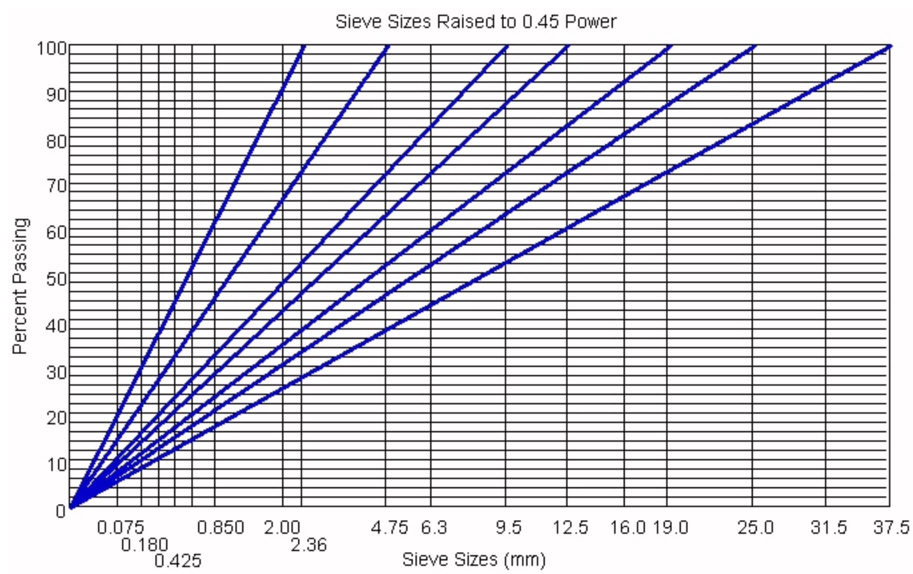


Figure 2.15. 0.45 Power chart showing maximum density lines for different aggregate sizes (*Source: www.pavementinteractive.org*)

The volumetric and the in-situ properties of the HMA mix is greatly influenced by the aggregate gradation (NAPA, 1997). Changes in the gradation directly alter the VMA and air void content of the mix. The aggregate interlocking and aggregate rearrangement under compactive loads are highly affected by the variations in gradation. Using the data from the WesTrack project, Pellinen et al. (2004) evaluated different fatigue models and reported a strong correlation between cracking resistance of asphalt mixtures and volumetric properties. In particular, VFA (voids filled with asphalt) was claimed to be best correlated to the fatigue cracking.

Using unconfined dynamic modulus and triaxial shear strength tests, Pellinen (2003) determined that lowering VMA from 23% to 11% while keeping the effective binder content (Vbeff) constant at 7.5% could enhance the stiffness. The scale of increase in stiffness of the mix, as per the study, is expected to be similar to what would be achieved by bumping the binder grade from PG 58 to PG 76. Moreover, the study also concluded that mechanical properties of the mixture are highly correlated with VMA and VFA.

2.2.5 Increased Compactive Effort

When external forces are applied on a hot mix asphalt mixture, the aggregate structure gets rearranged. This process reduces the volume of air in the mix and is called compaction. Since the air void content and the density of the mix are interrelated, any reduction in the air voids percentage results in improvement of density (Roberts et al. 2006). Several studies have found that the ability of pavement to resist distresses is mostly governed by the compaction process. (Scherocman 1984; Geller 1984; Brown 1984; Bell et. al. 1984; Hughes 1984; Hughes 1989).

The compaction of the HMA pavement is achieved by following process:

1. *Application of self-weight of compaction equipment on loose mix to compress the material into a dense mass:* The duration of interaction between the material and the roller determines the magnitude of compression. The longer this duration, the higher is the compression achieved. Moreover, increase in roller's weight also increases the intensity of compression. However, excessive weights can fracture the aggregates altering the gradation and making compaction harder.

2. *By introducing shear stresses through the compressed material and the material underneath the surface-roller interaction point:* Lower roller speed increases the shear stresses and results in higher compactive effort. Higher shear stresses reduce the air-void content of the asphalt mixture.

The total forces and stresses created by the compactor constitute its 'compactive effort'. According to Beainy et al. (2014), HMA pavement density is not a linear function of the compactive effort. The reason can be attributed to the changing orientation of aggregate skeleton. The change in density is rather random.

Compaction of HMA layer can be achieved through the application of vibratory, dead weight, oscillatory, and pneumatic tire compactors or their combinations. Increasing the roller passes (additional passes) are a general way of increasing the in-place density of asphalt pavement. It also helps in achieving uniform compaction and increased overall density of the pavement. There are three major types of rollers used for field compaction – static wheel roller, the pneumatic roller, and the vibratory roller. However, there have been many recent developments such as oscillatory rollers and vibratory pneumatic tire rollers which can improve the in-place density.

Static wheel rollers are the most basic compaction equipment which are self-propelled and use steel drum to compress the HMA layer under the contact area. Figure 2.16 displays a typical compaction curve of number of roller-passes versus density. The two factors influencing this curve are the type of steel wheel roller and the effective rate of compaction.

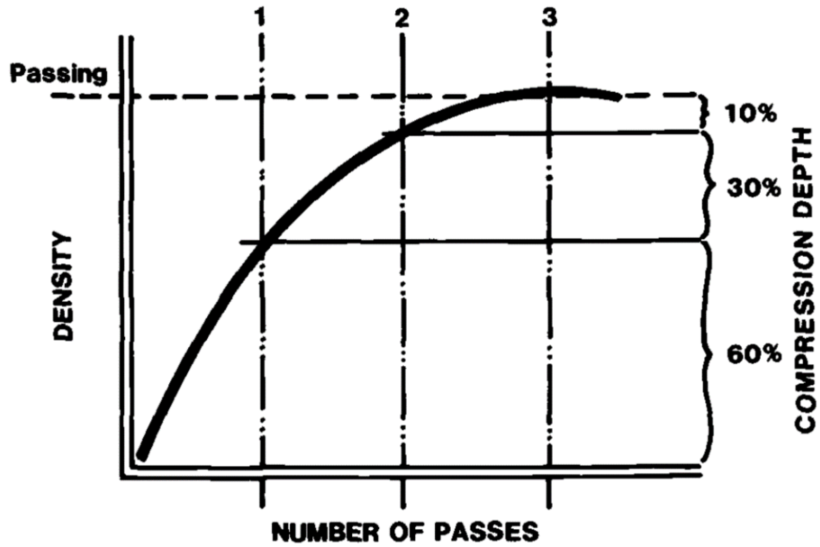


Figure 2.16. Compaction curve for steel wheel roller (Geller, 1984)

Geller (1984) mentioned two reasons why static wheel rollers are not as productive as vibratory rollers:

1. Static wheel rollers require more passes to compact the same pavement width, and
2. Lesser compaction rate in static mode.

Pneumatic tire rollers have been claimed to hold an advantage over the steel wheel rollers according to several past studies (Geller, 1984; Hughes 1989). However, the claims that the construction cracks due to rolling action of steel wheel rollers can be eliminated by pneumatic roller has been discarded (Abd l Halim et al. 1993). One certain advantage of pneumatic tire rollers is that the travel speed for such rollers are higher and hence the speed of achieving compaction is higher. Figure 2.17 shows a typical pneumatic tire roller. Pneumatic tire rollers can also be more effective for compacting soft asphalt mixtures.

Figure 2.17. Pneumatic tire Roller (*Source: www.pavementinteractive.org*)

Vibratory rollers use a combined static (similar to static wheel roller) and dynamic force for producing the compactive effort. Vibratory roller manufacturing has made advances over the past 15 years and have brought in the high frequency rollers into the picture. This technology ensures higher rolling speeds which reduces the delay between paving operation and the compaction, even for patterns with several passes (Tran et al., 2016). Starry (2006) and Scherocman (2006) have marked the importance of staying in proper temperature zone for rolling making it crucial for the rollers to tightly follow the paver. The capability of the vibratory rollers to implement this technique makes it an effective compaction equipment. However, excessive temperatures can result in shoving of softer asphalt mixtures.

With the advent of the Intelligent Compaction (IC) system, it is possible to keep a track of compaction in real time and make necessary adjustments. The vibratory rollers can be mapped with Global Positioning System (GPS). This makes it possible to monitor the steps involved in the compaction process. The sophisticated devices such as compaction meters and accelerometers measure and monitor the compactive effort. The IC display serves as an efficient tool to keep the operator aware of the exact progress of compaction process in the form of necessary information such as no. of passes, speed of roller and percentage coverage.

Attempts have been made to explore the possibility of IC density measurements as a substitute to the nuclear density gauge (NDG) tests which is presently the in-situ quality control/quality assurance test. However, the density obtained from IC system does not correlate well with that from NDG or laboratory core density measurement (Minchin et al. 2001; Maupin 2007; Chang et al. 2011; Chang et al. 2014). On the positive side, IC systems continue to enhance the efficiency of the compaction process.

2.2.6 The Use of Warm-Mix Additives

Warm mix asphalt (WMA) is an asphalt mix which is mixed and compacted at temperatures lower than the conventional hot mix asphalt. The mixing temperatures are generally reduced by using different mixing techniques and additives. The mixing temperatures of WMA have been reported to range from 100ºC to 140ºC compared to that of 150ºC-180ºC for conventional hot mix asphalt (Mo et al. 2012). The type of warm mix technique and additive can result in a significant temperature reduction of 20–40°C (D'Angelo et al. 2008). This reduction in temperature can be utilized to increase the TAC. WMA is also particularly useful in facilitating compaction in cold weather paving operations.

WMA technologies comprise the use of asphalt foaming processes, surfactants, non-foaming additives and combination of the three (Prowell et al. 2012). A brief description of these technologies and additives are given below:

- Asphalt foaming technologies - involve injection and dispersion of water in hot asphalt. This is done either via foaming nozzles or by using wet aggregates. Sometimes materials such as zeolites are also used. The water evaporates into steam during construction expanding the volume of binder and enhancing the coating of aggregates and overall compaction.

- Organic additives – these are waxes that turn the binder less viscous resulting in improved compaction. The selected wax should melt at temperature higher than the expected pavement temperature throughout its service period.

- Surfactants or chemical additives – help binder in enhanced lubrication of aggregates through different mechanisms.

The effect of warm-mix additives on compaction and achieving in-place target density has been studied by several researchers in past. Estakhri et al. (2009) studied the effectiveness of WMA during a resurfacing operation of a pavement section in Texas. The two mixes used were the HMA mixture (acting as control mix) and an Evotherm P25® WMA mix. Keeping other factors such as gradation, binder grade, thickness and rolling pattern same for both, the mixes were placed at two different temperatures. The control mix was placed at about 100^{o}F higher temperature than the warm mix. Subsequent to the placement and compaction of the two mixes over a course of three nights, the in-place densities of the pavement sections were measured. The HMA mixture was found to achieve 0.6% higher in-place density than the WMA mixture on an average, which is not significant compared to the difference in their placement temperatures.

In 2006, NCAT evaluated the performance of three different types of warm mix additives in Ohio. The additives used in the study were Asphamin® zeolite, Sasobit®, and Evotherm P25®. The comparison between conventional HMA and WMA was based on a control section that was paved as a part of the project. The mixture placed used 15 percent RAP and PG 70-22 asphalt binder modified with styrene-butadiene-styrene (SBS). Table 2.2 shows the details of the mixture constituents and amount of WMA added.

Table 2.2. Mixture Constituents and Amount of WMA Additives Mixed (Hurley et al., 2009b)

Aggregate Type	Coarse Aggregate	Fine Aggregate	RAP	Asphalt Binder
Size	No. 8	Sand	Crushed	
Aggregate Type	Limestone	Natural	Limestone/ Natural	PG 70-22 SBS Modified
Percent of Mixture	53	32	15	Virgin: 5.3% Total: 6.1%
WMA				
WMA Type	Aspha-min®	Evotherm P25®	Sasobit®	
Amount Added	0.3% by weight of total mix	5.3% asphalt binder with additive by weight of total mix	1.5% by weight of total binder	

According to this study, the difference in compaction temperature for the pavement sections with WMA mix was 30ºF to 60ºF lower and that in air voids was 0.7 to 1.2 percent lower than the HMA mix sections. (Hurley et al. 2009a)

Similar studies were carried out in Colorado and Connecticut. In Colorado, the performance of WMA mix was at par with the HMA mix (Aschenbrener et al. 2011). In Connecticut, except for the mix containing both Sasobit and SBS polymer, all the mixes used in the study easily achieved the target density. The issue with the combination of Sasobit and SBS polymer was corrected by increasing the production temperature (Zinke et al. 2014).

Anderson et al. (2014) also evaluated the performance of pavements constructed using WMA in comparison to the conventional HMA pavements placed on the same project. The study was sponsored by the Washington DOT. For the evaluation purpose, a section of an existing pavement was removed up to 3 inches of depth, and was resurfaced by HMA as control section and WMA with Sasobit as test section. The density results turned out to be similar for the HMA and WMA as shown in Figure 2.18. The report concluded that the WMA mix requires lower temperatures for compaction to achieve density similar to HMA mix. Moreover, the number of WMA sections which failed the density tests was significantly lower. Out of 95 density tests on the HMA, six (6.3 percent) failed to reach the 91.0 percent minimum specified density. Only one out of 55 (1.8 percent) density tests on the WMA was below 91.0 percent. The compactability of the WMA was significantly improved due to the use of 0.3% higher binder content for WMA (Anderson et al., 2014).

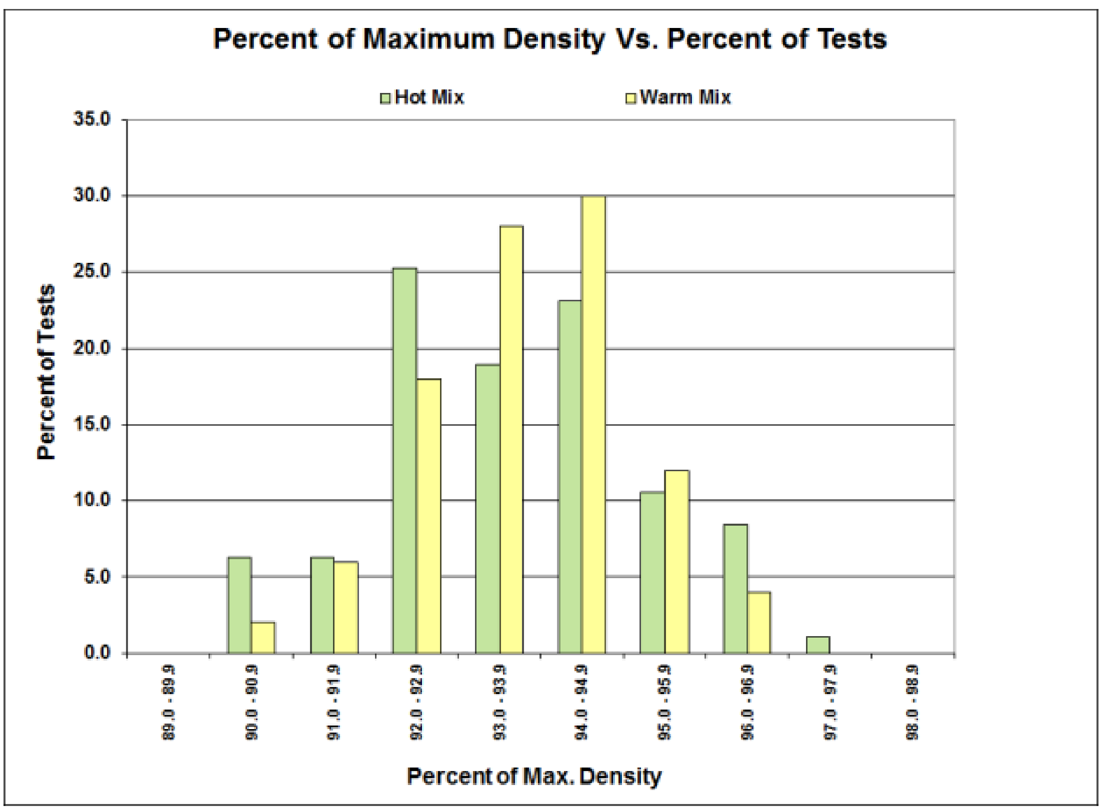


Figure 2.18. Compaction test results (Anderson et al. 2014)

Some researchers have also tried to study the behavior of the mix with both WMA additives and RAS and investigated how WMA affects the compaction of RAS mix. Bennert et al. (2017) included two different WMA additives to study their possible impact on RAS asphalt mixtures. For this purpose, the study utilized PG 64-22 asphalt binder premixed with Evotherm P25® and SonneWarmix WMA additives. RAS types used for the study were Post Manufacturer Waste (PMW) and Post-Consumer Waste (PCW). The proportions in which these RAS were mixed with the binder in terms of percentage of the binder's overall weight were 7.5% and 15%. The basis for selecting these percentages was the specifications set by the state agencies with some experiences in RAS mixtures. The study concluded that the WMA additives lower the otherwise high mixing and compaction temperature required by RAS asphalt mixtures.

Based on the findings of these studies, it can be concluded that WMA can help in achieving target in-place densities at lower compaction temperatures. This can be a useful technique in colder regions as well as situations where hauling time is expected to be longer (Tran et al., 2016).

2.2.7 The Use of Fiber-Reinforcement

Fibers as a reinforcement in asphalt mixtures has been gaining more and more popularity as a measure to improve pavement performance. Fibers, both synthetic (glass, polymer, carbon) and natural (flax, hemp, coir, jute), are known to serve as additives in the asphalt mixture. Previous researchers have investigated the reinforcing effects of fibers in asphalt mixtures. Fibers have been found to stabilize asphalt mixtures by preventing asphalt draindown during transportation and paving, and improve moisture susceptibility, rutting resistance and cracking performance of asphalt mixtures.

The volumetric properties of some fiber-reinforced asphalt mixtures are different from the conventional asphalt mixtures. Chen et al. (2009) investigated the volumetric properties of fiber-reinforced asphalt mixtures for designing a more reliable mixture. In this study, four different fiber types were used. These are shown in Figure 2.19. Based on the tests performed to measure the volumetric and mechanical properties of asphalt mixtures, Chen et al. (2009) concluded that adding fibers into mixture reduces specific gravity of mixture due to fiber's low density compared to asphalt and aggregate.

BoniFiber@ Fiber   Dolanit@AS fiber   Lignin fiber   Asbestos Fiber

Figure 2.19. Fibers used for study (Chen et al. 2009)

Studies have shown that addition of fibers increases the air voids in the HMA mixtures. Xu et al. (2010) recommended greater compactive effort or higher compaction temperature to reduce the air voids of fiber reinforced asphalt concrete (FRAC) so that the fiber's reinforcing effects can be optimized. In the same study, they concluded that FRAC is not suitable under water freezing-thaw effect due to the increased air voids which leads to formation of more ice at low temperatures. Peltonen (1991) also concluded that more compactive effort may be required by FRAC to achieve the density same as unmodified HMA mixture.

Air void content has also been observed to increase in dense FRAC due to the phenomenon of clustering. Clustering is the agglomeration of fibers due to poor dispersion in the HMA matrix. Gracia et al. (2013) observed that the air void content in dense HMA increases with increase in percentage of clusters formed by the fibers in the mixture. In this study, they also found that both the percent of clusters and air void content are dependent on the percentage of fibers. Higher the volume of fibers, higher are the air void content and percentage of clusters in the mixture. Although the higher percentage of steel wool fibers used in the study was found to improve the particle loss resistance of dense asphalt concrete, the accompanied increase in air void reduced the positive effects.

2.2.8 Longitudinal Joints

A longitudinal joint is the junction between two adjacently paved HMA lanes. The longitudinal joints are often a major source of distresses in asphalt pavements. The service life of the pavements can be significantly increased if the issue of longitudinal joints is properly addressed. To achieve better longitudinal joints, it is necessary to improve their compaction.

Three primary factors determine the joint density: i) density of the unconfined edge of the first paved lane; ii) compaction of the mix placed in the joint; and iii) compaction level achieved on the second paved lane (Brown, 2006). The free edge of the paved lane should be correctly compacted. This can be ensured by rolling the drum of the compactor on the lane with 150 mm of the drum wheel projecting out of the unsupported edge (Scherocman, 2006) as shown in Figure 2.20.

Figure 2.20. Compaction of unsupported free edge by steel wheel roller (Benson and Scherocman, 2006)

There should be sufficient overlap between the two lanes for a better longitudinal joint. This overlap is shown in Figure 2.21. In Figure 2.21, the overlap is lying in the range of 25 mm to 40 mm. Such an overlap eliminates the need of raking or moving off the extra mix which otherwise results in low density of joint (Benson and Scherocman 2006).

Figure 2.21. Proper overlap between the two lanes (Benson and Scherocman 2006).

The final point to be taken care of is the location of the roller during the compaction of the longitudinal joints between the two paved lanes. The rollers should avoid operating on the first lane i.e. the cold HMA mat side (Scherocman 2006). The wheel should be placed on the lane which was paved later i.e. the hot lane. However, it should be again ensured that there is an overlap of 150 mm of the drum on the cold lane (Benson and Scherocman 2006). Figure 2.22 shows this rolling pattern.

Figure 2.22. Rolling pattern of hot side of the longitudinal joint (Benson and Scherocman 2006)

Illinois DOT implemented two innovative bituminous joint sealants, J-band and QuickSeam, on four projects in 2003 as an experiment. The expectation was to lower the permeability of the joint by reducing the interconnected air voids since low permeability at the joint would result in better performance of the joint. The joints were embedded with the sealants before the HMA placement process. It was postulated that the placement of surface layer would soften the sealant material and coupled with the vibratory compaction, the material would migrate up into the surface through roughly three-fourths the matt thickness (Winkelman 2004). The J-band material was found to be easier to apply with minimum installation problems as compared to the QuickSeam. However, mixed results were found from the four projects. The sealants on two projects did not show any improvement with respect to permeability, but were able to reduce permeability on the other two projects.

2.3 METHODS USED FOR ASPHALT CONCRETE DENSITY MEASUREMENTS

The volumetric mix design and analysis forms the basis of the current Superpave mix design method. The trial asphalt mixtures with known aggregate gradation and different binder contents are compacted to reach a certain volume with a specific compactive effort. This is followed by determination of the volumetric properties of the compacted samples and the optimum asphalt binder content to achieve the target air void content. Since it is difficult to measure the volumetric properties directly, specific gravities of material are used to convert the measured weight to volume. Bulk specific gravity (G_{mb}) helps in determination of air void content, VMA and VFA. Therefore, precise measurement of G_{mb} is the key to successful mix design. AASHTO T166-12 is the specification followed to measure G_{mb}. Theoretical maximum density (TMD, G_{mm}) helps in determining the design number of gyrations. The procedure for determining TMD of HMA mix is standardized as ASTM D2041 (Rice 1956) and AASHTO T209-12. Recommendations have also been made to improve this procedure (Kandhal and Khatri 1992). Several researchers have developed and evaluated different methods to achieve more accurate laboratory and in-place density measurements (Crouch et al. 2003; Dep and Troxler , 2002; Praticò and Moro 2011; Preisset al. 1972; Prowell and Dudley 2002; Sargand et al. 2005; King and Kabir, 2009; Zaniewski and Yan, 2013). The following sections present the most common methods and procedures based on previous standards and studies.

2.3.1 Laboratory Methods

There are a number of laboratory methods available for determining bulk specific gravity. Table 2.3 shows some of these laboratory density measurement methods along with their respective standards. Each test has a different way of determining the volume of the HMA specimen which may result in variation in the G_{mb} values.

Table 2.3. Laboratory Methods for Determination of Density (Praticò and Moro 2011)

S.No.	Method	Standard
1	Saturated Surface Dry (SSD)	AASHTO T 166 - ASTM D 2726
2	Paraffin and parafilm	AASHTO T 275-A - ASTM D 1188
3	CoreLok (Vacuum Sealing Method)	ASTM D 6752 - AASHTO T-331
4	Dimensional	AASHTO T 269

2.3.1.1 Saturated Surface Dry (SSD)

SSD method is the commonly used water-displacement method which helps in measurement of G_{mb} of a compacted HMA sample. It is governed by Archimedes' Principle. SSD refers to the condition of HMA sample when the air voids present inside as well as on its surface are completely filled with water. This method is a three-step process. In the first step, the air-dry weight of the specimen is recorded. This is followed by placing the specimen into the water bath for 4 minutes (±1 minute) onto the submerged scale set-up. The submerged weight is noted from the scale after 4 minutes. As a last step, the specimen is removed from the water bath, the surface is blotted off with a damp towel and the mass is recorded. This final step has to be quick. Figure 2.23 shows the test set-up for this method.

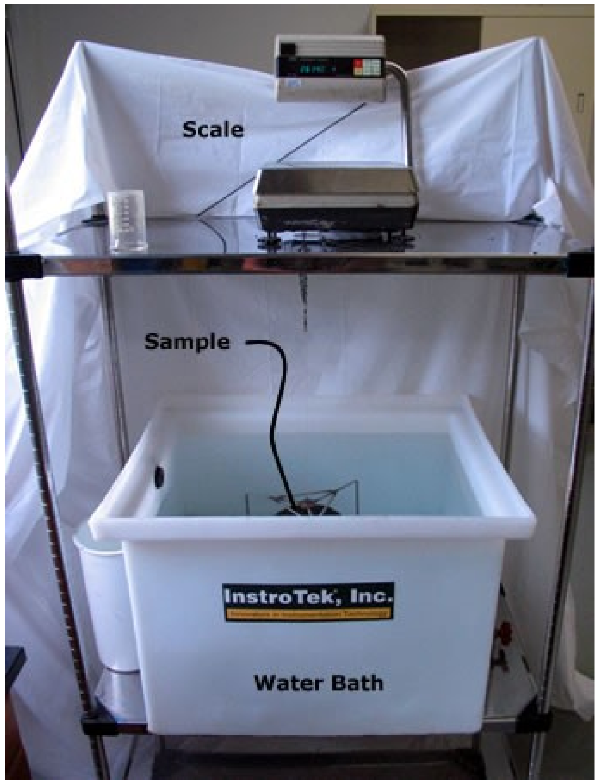


Figure 2.23. Saturated surface dry (SSD) test (*Source: www.pavementinteractive.org*)

Following Equation 2-3 is used to determine the G_{mb} of the specimen:

$$G_{mb} = \frac{A}{B - C}$$

(2-3)

Where:

A = air dried weight (grams) of the specimen in air,

B = saturated surface dry weight (grams),

C = submerged weight (grams),

Percent water absorbed by the specimen by volume is given by Equation 2-4:

$$\textit{Volume of water absorbed}\ (\%) = \frac{B - A}{B - C}$$

(2-4)

Although SSD method is the most popular method owing to its simplicity, researchers have shown that it produces erroneous results for coarse graded Superpave and Stone Matrix Asphalt

(SMA) (Buchanan and White, 2005; Cooley et al. 2002) with air void contents higher than 8%. This error has its source in the air voids which are interconnected. During the final step of this method, water from these voids can get undesirably removed. This additional loss of water results in a lower SSD weight and higher G_{mb} values than the actual. Therefore, for samples absorbing more than two percent of water by volume, AASHTO T 275 (Paraffin) or AASHTO T 331 (CoreLok) should be used for determining the bulk specific gravity. Figure 2.24 and Figure 2.25 show the differences between air void distributions for coarse and fine graded asphalt mixtures and the reason for the need to use alternative methods for determining density for coarse graded mixtures.

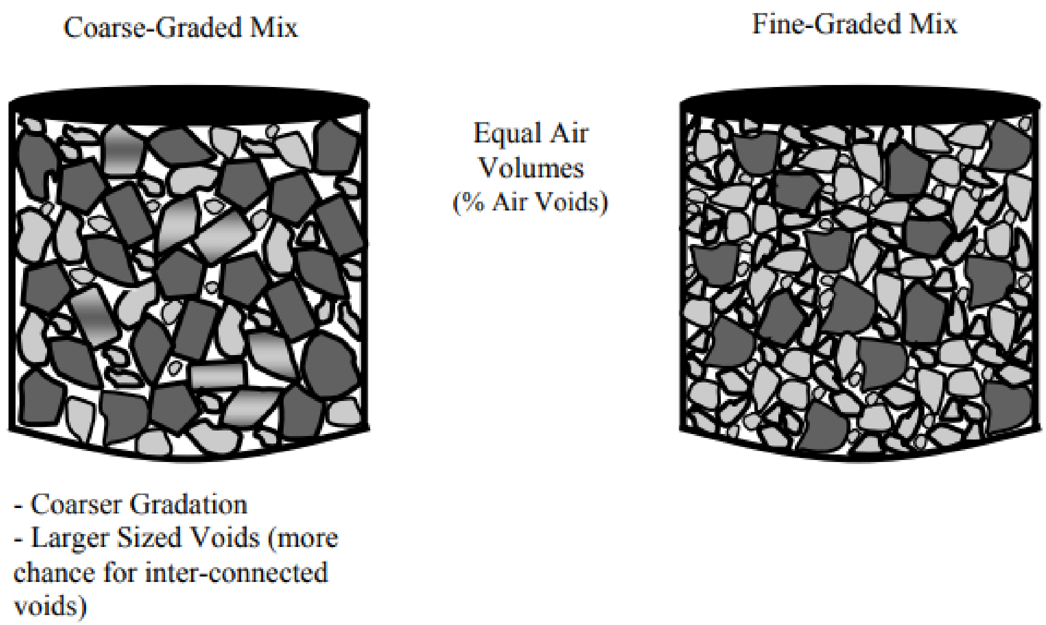


Figure 2.24. Internal void structure for coarse-graded and fine-graded mix (Zaniewski and Yan 2013)

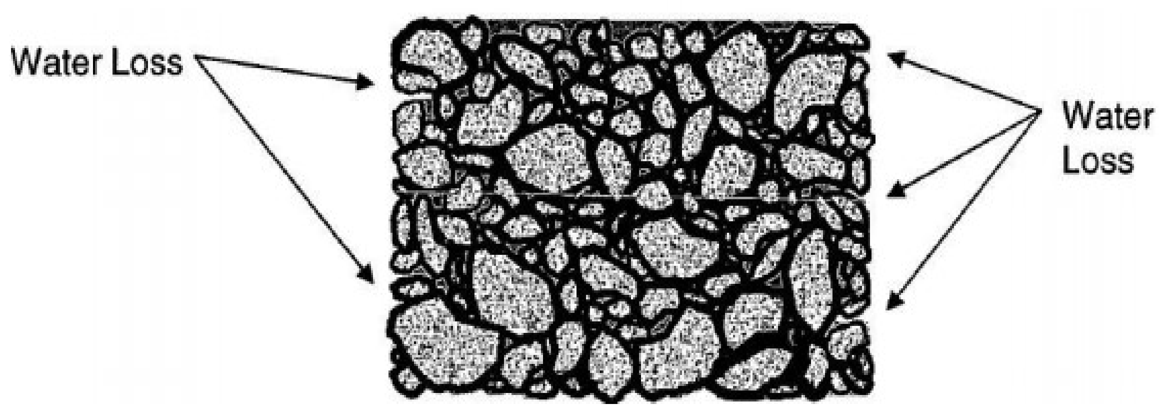


Figure 2.25. Potential water loss from compacted asphalt mix in SSD condition (Zaniewski and Yan 2013)

2.3.1.2 Paraffin and Parafilm Method

The measurement procedures for the paraffin and parafilm methods are available in AASHTO T 275 or ASTM D 1188 specifications. Paraffin method is similar to SSD method but instead of water, melted paraffin wax is used to seal the specimen's surface air voids. Figure 2.26 shows the paraffin covered HMA specimen. The wax, when set, makes it impossible for the water to seep out and the mass of oven-dried specimen is recorded before and after sealing the voids with the wax.

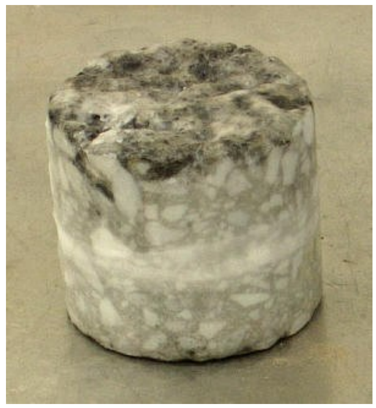


Figure 2.26. Paraffin-covered HMA specimen (*Source: www.pavementinteractive.org*)

Bulk specific gravity for this method is calculated as per the Equation 2-5 below:

$$G_{mb} = \frac{A}{\left(D - E - \left(\frac{D - A}{F}\right)\right)} \tag{2-5}$$

Where:

A= air-dried mass of the specimen (grams),

D= paraffin coated mass of the specimen in air (grams),

E= paraffin coated mass of the specimen in water (grams), and

F= paraffin's specific gravity at 25±1°C (77±1.8°F).

In the parafilm method, the specimen in wrapped in a thin paraffin film. The wrapped specimen is weighed both in air and when submerged in water. G_{mb} calculation is similar to the paraffin method. HMA specimen wrapped in paraffin film is shown in Figure 2.27.

Figure 2.27. HMA specimen covered with Parafilm (Schroer 2012)

The parafilm method has been argued to be more precise than the paraffin method. The latter records higher G_{mb} value because of the paraffin seeping into pore structure of the specimen (Schroer 2012). However, other studies have put forward the disadvantages of the parafilm method in terms of practicality (Buchanan 2000), unreliable results in case of higher air voids in the specimen (Praticò et al. 2007) and lack of specification for 150 mm diameter specimens (Bhattacharjee and Mallick 2002).

2.3.1.3 Vacuum Sealing (CoreLok) Method

Vacuum Sealing method (VSM) involves a vacuum chamber to cover the specimen with a plastic bag and this is what differentiates it from the parafilm method. The plastic bag used to cover the specimen is resilient and puncture resistant which prevents water from entering into the sample (Figure 2.28). Seal can be easily removed after the weight measurement and the specimen can be used for testing. Agencies have been using the InstroTek Inc. manufactured CoreLok vacuum-sealing device (Cooley et al. 2002) as shown in Figure 2.29 and the same has been incorporated in AASHTO T331 and ASTM D 6752 specifications.

Summary of test procedure:

Air-dry weight of HMA sample (either a laboratory compacted cylinder or a field core) is recorded. The sample is wrapped in plastic bag and then placed in the vacuum chamber. Selection of Program #1 brings the pressure the pressure inside the chamber to 760 mm Hg. Once the specimen is completely shrink-wrapped in the bag, the door of the chamber automatically opens. The specimen wrapped in the bag is then placed in the water bath and the submerged weight is recorded. While keeping the sample submerged, the plastic bag is cut open and water is allowed to freely enter and exit through the cut portion. The reading on the scale, once it stabilizes, is recorded. G_{mb} is given by the following Equation 2-6:

$$\boldsymbol{G_{mb}} = \frac{\boldsymbol{A}}{(\boldsymbol{A}+\boldsymbol{B}-\boldsymbol{C})-(\frac{\boldsymbol{B}}{\boldsymbol{D}})} \tag{2-6}$$

Where:

A= the sample weight (grams),

B= the plastic bag weight (grams),

C= the wrapped sample weight in water (grams), and

D= the plastic bag density.

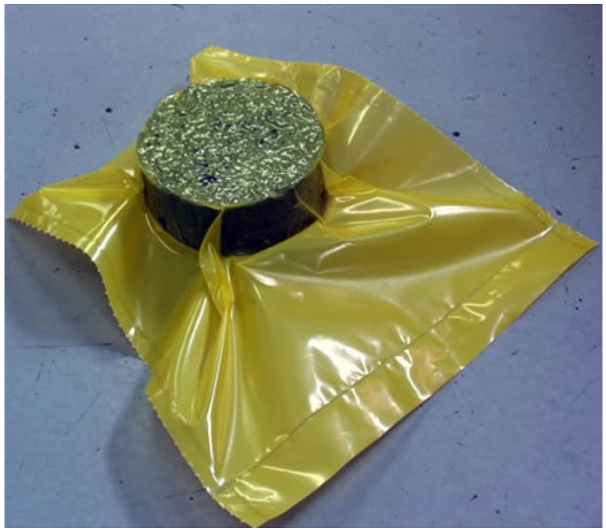


Figure 2.28. CoreLok sealed specimen (*Source:* www.pavementinteractive.org)

Figure 2.29. CoreLok vacuum chamber (*Source:* www.pavementinteractive.org)

CoreLok procedure has been recommended for more accurate use such as determination of G_{mb} for coarse-graded mixes (Buchanan and White 2005; Williams et al. 2007). Cooley et al. (2002) concluded that this procedure works better with higher air voids sample. Based on the results of the precision and accuracy study, Crouch et al. (2003) recommended the Instrotek Corelok System as the most widely applicable method out of the four most different methods (SSD, dimensional analysis, parafilm and VSM) for determining G_{mb} of compacted HMA mixtures.

The CoreLok samples showed lesser standard deviation in the test results compared to SSD and dimensional method (Hallet al. 2001). However, CoreLok method has been criticized for being less practical because experience with CoreLok test procedure produces variation in the test results (Cooley et al. 2002).

Cooley et al. (2002) determined that some bags can be easily punctured during Corelok testing in a number of laboratories. Another source of inaccuracy is the 'bridging effect' (Sholar et al. 2005). The vacuum sealing of the plastic bag over an irregular sample surface tends to bridge over the voids on the surface leaving air pockets, which alter the result. Williams (2007) in the study of the G_{mb} measurements of 25.0 mm and 37.5 mm coarse-graded Superpave mixes argued against the elimination of the traditional (SSD) test method. Brown et al. (2004) in the NCHRP 531 report recommended the absorption limit for the displacement test method to be reduced to 1 percent as opposed to 2 percent specified in AASHTO T166. According to the same NCHRP report, a correction factor for vacuum-sealing and water-displacement methods could provide comparable results even at low air void level. This correction factor can be determined measuring air voids of two test samples, both compacted to a standard air void content (design air void) and calibrating the difference.

2.3.1.4 Dimensional Method

This is the simplest method which involved direct measurement of the geometry of the sample. However, this method makes an unrealistic assumption of perfectly smooth surface of the sample. On a positive side, this method does not make use of the SSD condition and hence avoids the associated problems.

2.3.2 Methods Used for In-place Density Measurement

In-place density is an important factor affecting the long-term performance of HMA pavements.

Harvey and Tsai (1996) observed that the pavement degradation and deterioration can be greatly affected by the void space which can possibly be filled with ice, water or air. The in-situ density of HMA can be accepted to be fundamental parameter for pavement quality evaluation.

Traditionally, in-place density or HMA void ratio in the field were assessed from core samples obtained through drilling/cutting the pavement after compaction. The density measurements from this method (core method) were assumed to be highly accurate. However, the process was time-consuming, destructive, and expensive. As a better and non-destructive alternative, nuclear technology was later developed for density measurements. This was a much-needed improvement as this could be done in less than five minutes and provided reasonably accurate and real-time information aiding to the better quality control. The disadvantages associated with nuclear methods were the accuracy and cost. In addition, nuclear measurement devices contain radioactive materials which require special training and permits to use.

Since last two decades, the focus of research has been shifting from nuclear to non-nuclear method of density measurement. These methods are almost as quick as nuclear methods and have an additional benefit of being safer and more practical.

2.3.2.1 Nuclear Density Gauge

The California Division of Highways did pioneering study on nuclear gauges as early as in 1954. During that time, nuclear gauges were mostly used in the measurement of soil density. It was then gradually implemented for asphalt and PCC pavement density measurement. The element used for this purpose is a radioactive gamma-emitting isotope such as radiocesium (Cesium 137 or ${}^{137}_{55}Cs$). The principle involved in this process is the measurement of the amount of the particle reflected back (backscattering) by the test surface (asphalt pavement) when it is exposed to the emitted photons.

Emission of a cloud of photons in the form of gamma rays on the pavement surface triggers the interaction of the photons with the electrons present in the pavement. There is loss of energy due to this collision (Padlo et al. 2005) which can be explained by Compton scattering. Although there are two modes of operation available for nuclear density gauges (direct transmission and backscatter), backscatter model is generally used for density measurements of asphalt pavement. Figure 2.30 shows a nuclear density gauge and the backscatter mode of operation is shown in Figure 2.31.

Figure 2.30. Nuclear density gauge (Schwartz et al. 2014)

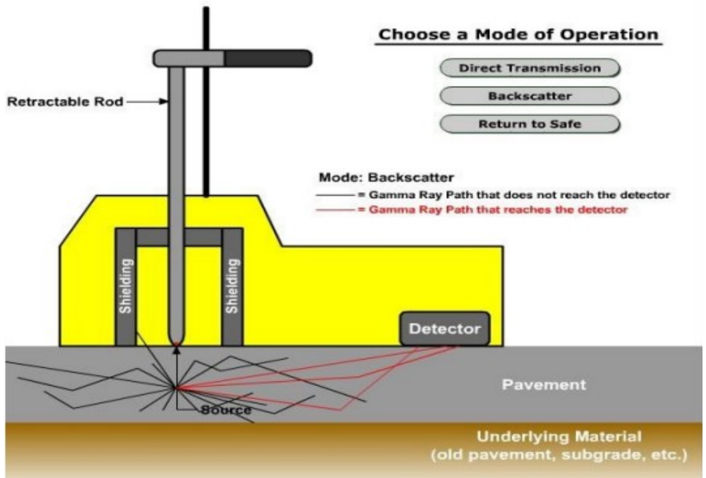


Figure 2.31. Nuclear density gauge backscatter mode of operation (*Source:* www.pavementinteractive.org)

The procedure for performing nuclear density test on HMA pavements is outlined in ASTM D 2950. The trigger of the retractable rod is pushed down to the asphalt surface in backscatter mode until a click sound is heard. As explained earlier, the emitted photons get scattered in different directions in the pavement and collide with the electrons. A fraction of the total emitted radiations is reflected back and counted by the detector. The amount of bounced back radiations is linked with the density of the pavement. The readings obtained are converted to density by calibration with laboratory testing results obtained from pavement cores (Mitchell 1984). This calibration process is also commonly referred to as "core correlation" and should not be confused with the gauge calibration process conducted by using hot substrate and standard blocks over a range of densities. Gauge calibration process is described in Section 2.4.1. This "core correlation" process is accepted as the best practice but is not always employed. Figure 2.32 shows the nuclear density gauge in operation.

Figure 2.32. Nuclear density gauge in operation (*Source:* www.pavementinteractive.org)

Zha (2002) observed that the calibration of the NDG is bound to change over a period of time due to use on uneven surface, unsophisticated handling of the equipment, isotope's degradation or other such factors. California Department of Transportation recommended an interval of 15 months between two calibrations based on a study (Zha 2002).

Despite several advantages, NDG test methods has some drawbacks as well. The results obtained from nuclear methods have been found to have more variability than the core density method. Cost of equipment and its maintenance is much higher than the traditional laboratory methods (Smith and Diefenderfer, 2008). Variation in results can be minimized by gauge calibration from direct methods such as CoreLok, parafilm, and SSD. However, the calibrations will be required to be repeated for different test strips along different pavement sections because of the change in structural properties.

2.3.2.2 Non-Nuclear Density Gauge

Non-nuclear gauges use electromagnetic field to measure density of asphalt pavement (Williams 2008). An electric current is passed through the pavement and is forced to go around an isolation ring. The emerging current thus reaches the receiver. Analogous to the count of radiations received by detector in NDG, the value of recorded dielectric constant in the non-nuclear density gauge is correlated to the pavement density. The measurement of dielectric constant throws light on the property of medium through which the current passes in terms of the resistance offered to the flow of current. Figure 2.33 shows the schematic of the process.

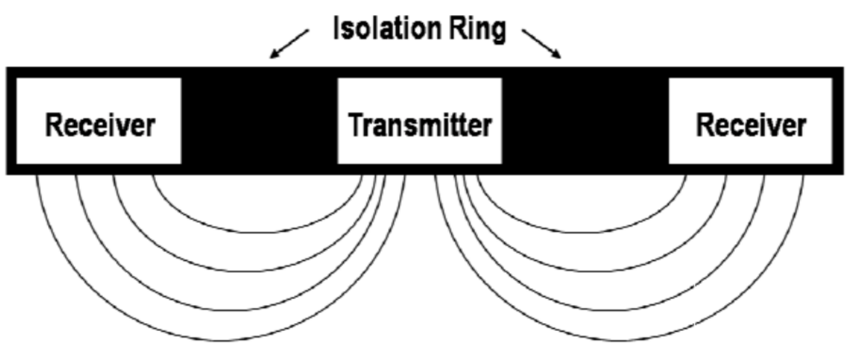


Figure 2.33. Non-nuclear gauge process (Williams 2008)

The density of pavement is therefore, a linear function of its dielectric constant (Henault 2001). Pavement Quality Indicator (PQI) and the PaveTracker (shown in Figure 2.34 and Figure 2.35, respectively) are commonly available non-nuclear density gauges.

Researchers have investigated these gauges for their accuracy under different conditions (Sargand et al. 2005). The study by Smith and Diefenderfer (2008) rated non-nuclear density gauge a better testing method than the nuclear density gauge for dense-graded pavement. However, the readings need to be corrected for moisture present in the HMA layer. Unlike devices that use a radioactive source, these devices do not have licensing procedure and meticulous training. Moreover, these devices have no specific storage or transportation issues.

Although PQI 300 & PQI 301 have been found acceptable for quality control studies (Hurley et al. 2004; Prowell and Dudley 2002; Rao et al. 2007, Romero 2002), the use of these devices without calibration for every construction section is not recommended. A study by Kvasnak et al. (2007) revealed that several factors related to the mixture properties and project-specific factors affect electromagnetic gauge readings. The study concluded that these devices can serve as QA test only when they are calibrated for every project.

Figure 2.34. Pavement quality indicator (Williams 2008)

Figure 2.35. PaveTracker (Williams 2008)

2.3.3 Research Studies Focusing on the Development of More Practical Methods for Density Measurement

2.3.3.1 Permeability Test

Permeability has been found to affect the HMA density (Brown et al. 1989; Kumar and Goetz 1977; Mallick et al. 1999; Zube 1962). Zaniewski and Yan (2012) found a correlation between air void content and permeability of the HMA layers based on power model. According to the test results on four mixes with different sizes (37.5 mm, 19 mm, 12.5 mm and 9.5 mm), the study concluded that the HMA samples with air voids less than six percent are not permeable. It was also observed that the samples with six to seven percent air voids are permeable and their permeability increases with air voids beyond eight percent. Cooley et al. (2001) had observed similar trend for 9.5 mm and 12.5 mm mixes. As shown in Figure 2.36, permeability values for 9.5 mm and 12.5 mm HMA mixes are very low (almost zero) at air void contents lower than 5.75% and as the air void contents increase beyond 6%, permeability starts drastically increasing.

However, high air void content does not necessary translate into high permeability. Factors such as interconnection of voids (Choubane and Musselman 1998) and size of air void (Hudson and Davis 1965) also play important roles in determining the permeability of compacted asphalt mixtures.

It has been widely accepted that permeability is an important characteristic and investigation of this property can help evaluate the performance of asphalt pavements. However, presently there is no standard test to measure the permeability of compacted HMA samples.

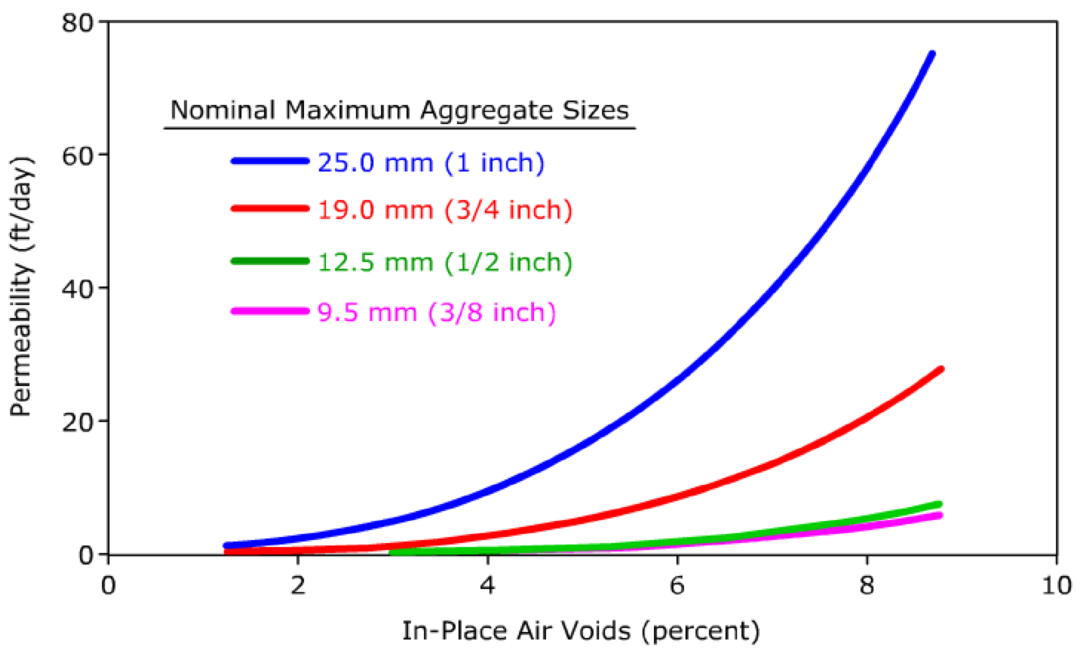


Figure 2.36. Nominal maximum aggregate size vs. field permeability (Cooley et al. 2002)

2.3.3.2 Ground Penetrating Radar (GPR)

GPR is a tool for measuring thicknesses of the pavement layer nondestructively. The possible use of GPR for in-place asphalt layer density measurement has been investigated by different research studies. GPR incorporates the use of ultrasonic pulses which interact with the pavement surface in the form of stress waves. The reflected waves are detected by a receiver which produces information about the structure (Morey 1998). Similar to the PQI equipment, GPR implements electromagnetic radiation and relies on the measurement of dielectric constant to determine the pavement layer properties (Saarenketo and Scullion 2000). GPR has been successfully utilized in pavement engineering for layer thickness measurement and has its own advantages and limitations.

GPR technology is practical and easy to use. The design of the GPR equipment is based on the target material. As technology advances, GPR also improves and evolves as a system with intelligent sensing which can be utilized to measure different pavement properties. GPR can also be implemented as comprehensive tool for pavement management (Benedetto and Pensa 2007).

The primary components of a GPR system are shown in Figure 2.37.

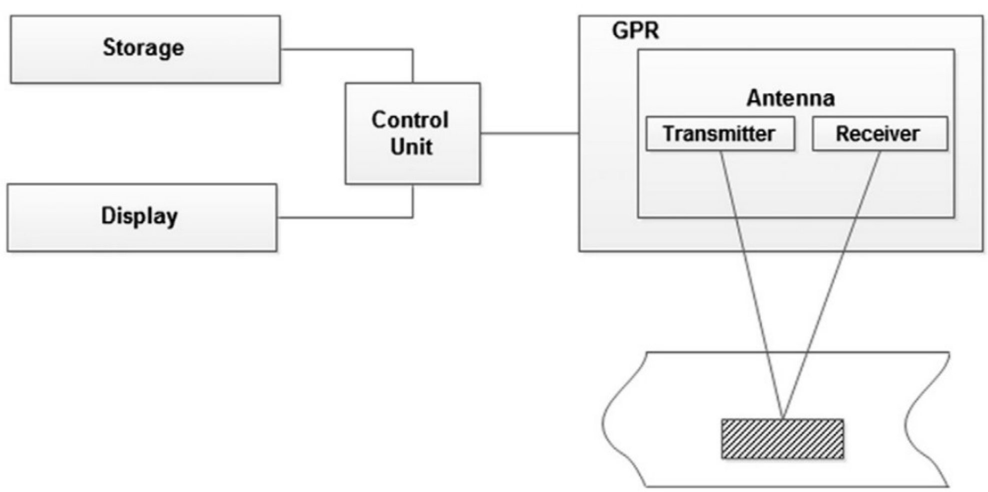


Figure 2.37. GPR system and its main components (Plati and Loizos 2013)

Electromagnetic pulses are emitted through a radar, which moves on or slightly above the ground. These pulses collide with the ground and the receiver device then collects the reflected signals. The antenna unit transmits and receives the signals and the control unit sends the collected data to storage and display units.

Al-Qadi et al. (2010) observed that the density measurements obtained from the GPR are more accurate than those from nuclear density gauge. However, there are no standardized data correction and processing procedures (required frequency of antenna, measurement parameters for different pavement materials etc.) developed to deal with precision and accuracy issues that might originate from the variable nature of asphalt pavement layers (Plati and Loizos 2013).

Vibration-Based Onboard Asphalt Density Measuring System (ODMS)

Patented by Penn State University, the Onboard Density Measuring System (ODMS) is a computer controlled accelerometer based asphalt layer density measurement system. System can make real-time density measurements during construction based on the accelerations at the roller. ODMS also presents the contractor and the owner with a record of density readings for the entire project length. It is a major improvement over the nuclear density gauge and other methods since it can collect the density data during on-going compaction process and keeps the operator aware of any variation in recorded densities. This can help in eliminating low density segment from a specific location(Minchin and Thomas 2003). Figure 2.38 shows the schematic of ODMS system.

ODMS keeps measuring the in-place pavement density as the compaction process progresses. The accelerometer response is recorded and the data is combined with other stiffness related parameters of the asphalt layer (such as temperature, unbound layer properties, etc.). The pavement density is then predicted using multiple regression equations (Minchin and Thomas 2003).

The drawback in the use of this system during compaction is that it is not effective beyond the top layer of the pavement. In addition, the equations developed for the conversion of measured acceleration to density may not work for different asphalt mixes and/or pavement types. For instance, if the conversion equations were developed by using the data from a flexible pavement, the effectiveness of predictions for an asphalt layer on a concrete pavement (composite pavement) would be questionable. For these reasons, ODMS type systems are not widely accepted for quality control during construction.

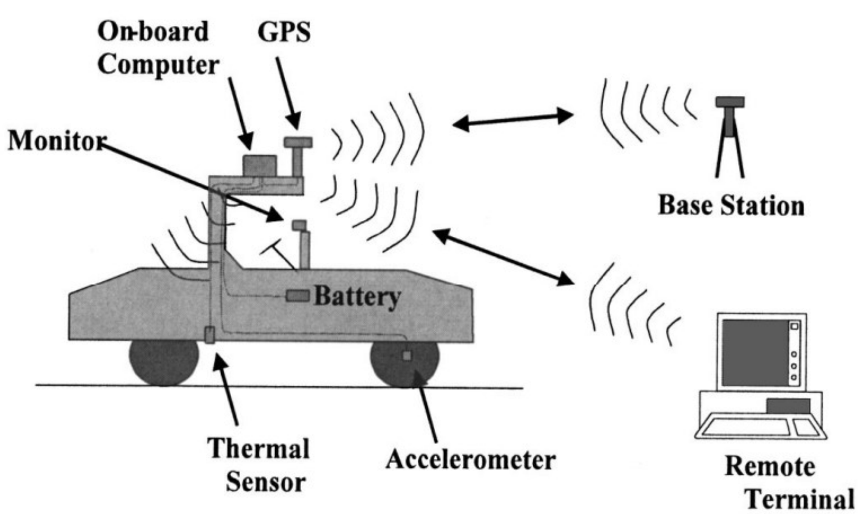


Figure 2.38. Schematic of ODMS system (Minchin and Thomas 2003)

2.4 CURRENT QC/QA PROCEDURES FOR COMPACTION

The quality control/quality assurance (QC/QA) of HMA pavement construction commonly utilizes the acceptance criteria based on the statistical analysis of results (Russell et al. 1998). The statistical acceptance criterion is a decision-making tool for acceptance or rejection of work done by a contractor. Statistical specifications are also laid out to set a threshold of acceptance. The quality of work done by the contractor is gauged by a statistical measure of 'percent within limits' (PWL). Moreover, the final payment made to the contractor is also based on the PWL method. The contractor is given a bonus for providing quality of work higher than the threshold limit, and is penalized for poor quality determined by PWL statistical method. The final payment is made through Pay Factors (PF) which incorporates the bonuses and penalties. Another method that used QC/QA specifications is composite pay factors (Russell et al. 2001). When a specification has more than one quality parameters, a weighted average of pay factors is calculated as composite pay factor to determine the payment. The in-place density (air-voids) holds the highest weight in these specifications. In Oregon, density constitutes 44% of the PF while aggregate gradation and asphalt content constitute 28% and 28% of the PF, respectively.

In Oregon, ODOT mandates calibration of all NDG every year using the procedure provided in ODOT TM 304. Moreover, the nuclear gauges are often correlated to the pavement core densities based on request or requirement (ODOT 2018). Additional core correlation of nuclear gauge readings may be requested by both the project engineer and the contractor. The core correlations are performed according to AASHTO 355 and ODOT TM 327.

2.4.1 Calibration of Nuclear Gauge

Nuclear gauge calibration is carried out in accordance with ODOT TM 304. The procedure provided in the Manual of Field Test Procedures is briefly summarized below (Scholz and Darra 2010):

1. The gauge and all components are conditioned at room temperature for at least 4 hours. It is to be ensured that all components are at room temperature (60ºF -75ºF).

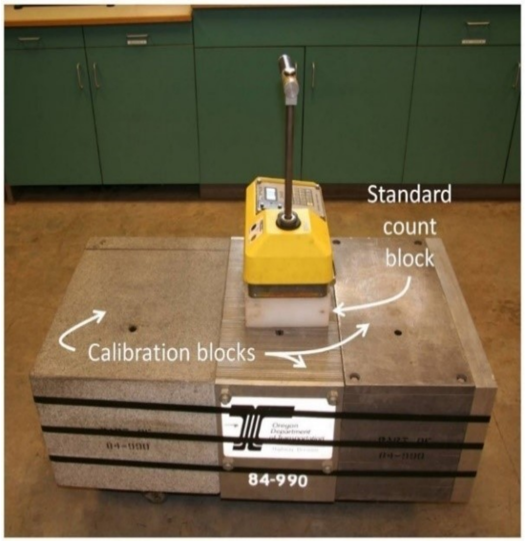
2. The gauge is switched on and left to warm up for no less than 10 minutes.

3. Five readings are taken by placing the gauge on a standard count block. The position of gauge is shown in Figure 2.39a. If the readings show variances beyond the tolerance limit, the gauge is required to be repaired.

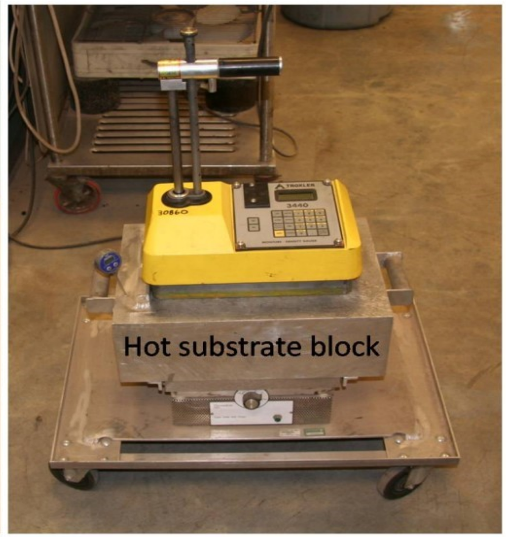
4. If the readings in step 3 are within tolerance limit, the gauges are set to take one-minute counts. These gauges are then subjected to hot substrate test. The gauge conditioned at room temperature is allowed to take four successive initial readings (at one-minute count mode) on an aluminum block heated to 185ºF serving as a hot bed as shown in Figure 2.39b. Four final readings are recorded after a gap of 10 minutes. If the difference between averages of initial and final readings are less 1.0 lb. /ft^3 then the gauge is concluded to have passed the test.

5. After hot substrate test, gauges' calibration are verified against the three standard blocks categorized as low, medium and high-density blocks as shown in Figure 2.39c.

Gauges that pass the density readings check are then checked for moisture density readings. This is followed by the calibration of the gauges that pass these tests. The readings are taken against calibration blocks in both backscatter and direct transmission mode. These readings when processed in a computer program, provide calibration constants for the gauge. The gauge, with the calibration constants fed into it, is rechecked with the two tests subsequent to the hot substrate test.



a



b

c

Figure 2.39. Steps involved in nuclear gauge calibration (a) Position of gauge on standard count block (b) Hot substrate test (c) Calibration blocks of varying densities (Scholz and Darra 2010)

2.4.2 Correlation of Nuclear Gauges to Pavement Core Densities

As per ODOT specifications (ODOT, 2021), the correlation of nuclear gauge readings and the density of cores removed from the roadway is to be performed when it is required by contract or requested by either the agency or the contractor. In Oregon, core correlation is performed according to AASHTO T 355 and ODOT TM 327. The procedure is summarized as follows:

1. Ten core locations are identified on the site at random.

2. The gauge is placed on the test site parallel to the direction of roller passes as shown in Figure 2.40 and its position is marked. First reading is recorded in this direction and the second reading in the direction perpendicular to it, again marking the position of the gauge. The average of the densities in two directions is reported as the density of the pavement. However, if the difference between the two readings is more than 2.5 lb/ft^3, the process is repeated.

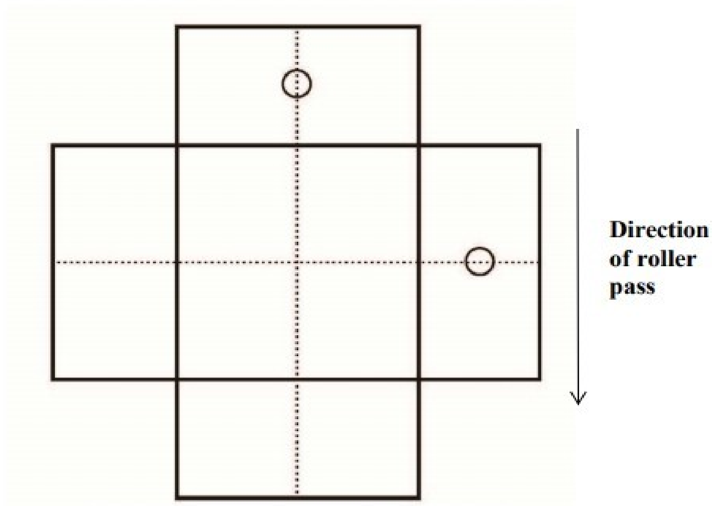


Figure 2.40. Position of nuclear gauge alignment (AASHTO T 355)

3. For correlation, samples are cored from each test site from the center of the perpendicular markings of the gauge as shown in (Figure 2.41).

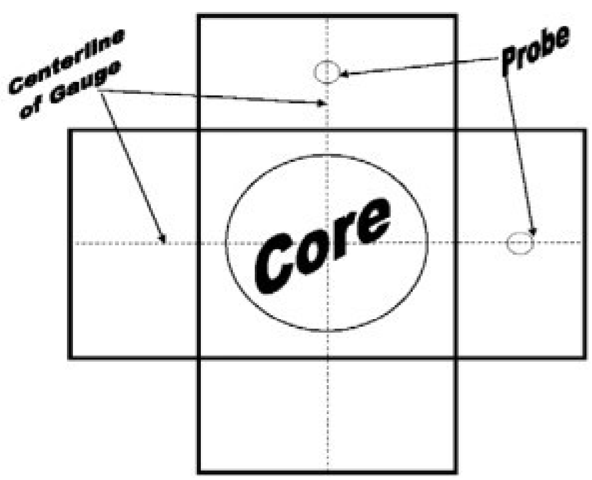


Figure 2.41. Core location as per ODOT TM 327

4. G_{mb} of these cored samples are obtained by standard AASHTO T 166 procedure.

5. The average of the ratios of the bulk densities of the cores to the density obtained from NDG is used to calculate a correlation factor.

2.4.3 Quality Control Tests

The responsibility of quality control (QC) generally lies with the contractor. For QC tests, both sampling and the testing is provided by the contractor. The testing operations are observed and performed by the contractor's QC Certified Technician i.e. Certified Asphalt Technician I (CAT I). ODOT requires the contractor to conduct nuclear gauge measurements by following the procedure outlined in the previous section at five different locations per sublot (one sublot equals 1,000 tons of HMAC) (Scholz and Darra 2010). ODOT is currently in the process of moving to 200-ton density sublots rather than the average of five shots.

2.5 INTELLIGENT COMPACTION

2.5.1 Intelligent Compaction Methods and Technologies

Intelligent compaction (IC) is a recent compaction technology brought to the U.S. in the last 30 years and is currently being implemented in Oregon. IC systems are vibratory rollers equipped with thermal imaging systems, acceleration measurement sensors, a global positioning system (GPS), and on-board computers that display various IC measurements. These IC measurements are called IC measurement values (ICMV) and the data includes roller passes and locations, asphalt surface temperature distributions, roller vibration frequencies/amplitudes, and speeds. IC uses the roller wheel acceleration measurements in back-calculation algorithms to determine the density of material during compaction. However, predicted density measurements were determined to be inaccurate and unreliable to be used in QC/QA procedures.

2.5.1.1 Thermal Imaging Systems

Thermal imaging systems are complex infrared temperature cameras, which measure light (or heat) from a given object. These complex infrared cameras have many variables that measure emissivity (the ratio of the energy radiated from a material's surface to that radiated from an ideal emitter), reflected temperature (the temperature of surrounding objects), the distance between the object and the camera, the relative humidity and the atmospheric temperature (Plati et al. 2014). All these variables are used to help determine the temperature of an object. Thermal imaging systems are currently being used to monitor surface temperatures of pavements during the paving and compaction of asphalt layers. An IC roller is equipped with one or two infrared (IR) temperature sensors near the end of the drum on rollers. Thermal imaging systems are usually mounted on pavers. Thermal imaging systems provide a better understanding of the full surface temperature profile in real-time during the compaction process.

Plati et al. (2014) investigated how past research studies have expressed the importance of temperature segregation happening when hot-mix asphalt (HMA) is being placed and compacted. Also, Plati et al. (2014) provided a technique called Infrared Thermography (IRT) to capture surface temperatures. IRT can be defined as the science of the acquisition and analysis of data from non-contact thermal imaging systems and allows continuous real-time data collection. The ThermoVision A325G camera used by Plati et al. (2013) had an accuracy of ±2 °C. It should be noted this camera was not attached directly to the roller, but on top of a van which trailed behind the process. The temperature profile could have been different if the camera was on the roller drum since one factor in thermal imaging systems is the distance between the object and IR camera along with emissivity, reflected temperature (the temperature of surrounding objects), the relative humidity and the atmospheric temperature. Pave-IR system is another thermal imaging system used on pavers, which uses infrared temperature bars and accompanying Pave-IR software. Pave-IR can be considered to provide a significant improvement over older methods of thermal profiling that used infrared cameras or spot radiometers (Sebesta et al. 2006). Pave-IR systems today (2021) use scanning thermal camera rather than infrared temperature bars. This technology has the same accuracy as the ThermoVision camera when checked by using temperature guns and thermocouples for verification of temperature readings. TxDOT fully implemented the Pave-IR system as a QC/QA test for dense-graded asphalt mixtures to determine temperature segregation (Sebesta and Scullion 2012).

2.5.1.2 IC Density Measurement Systems

Intelligent compaction (IC) is capturing real-time uniform compaction of an entire pavement structure during the compaction process, making it an ideal tool for quality control (QC). IC density measurement systems use accelerations responses coming from the sensors (accelerometers) installed on the rollers to back calculate asphalt layer density. Measurements are first calibrated by using the density measurements from other methods, such as nuclear density gauge or CoreLok, to improve accuracy and precision. After calibration, roller accelerations are used to back calculate real-time pavement densities. Figure 2.42 shows that the IC roller is unable to distinguish between individual layers in the pavement structure. The accelerations measured by the IC roller sensors are affected by multiple layers within a given HMA pavement structure. For this reason, isolating the asphalt layer response from the overall acceleration is generally not possible. Therefore, predicted asphalt layer densities from the IC system are not used for QC/QA.

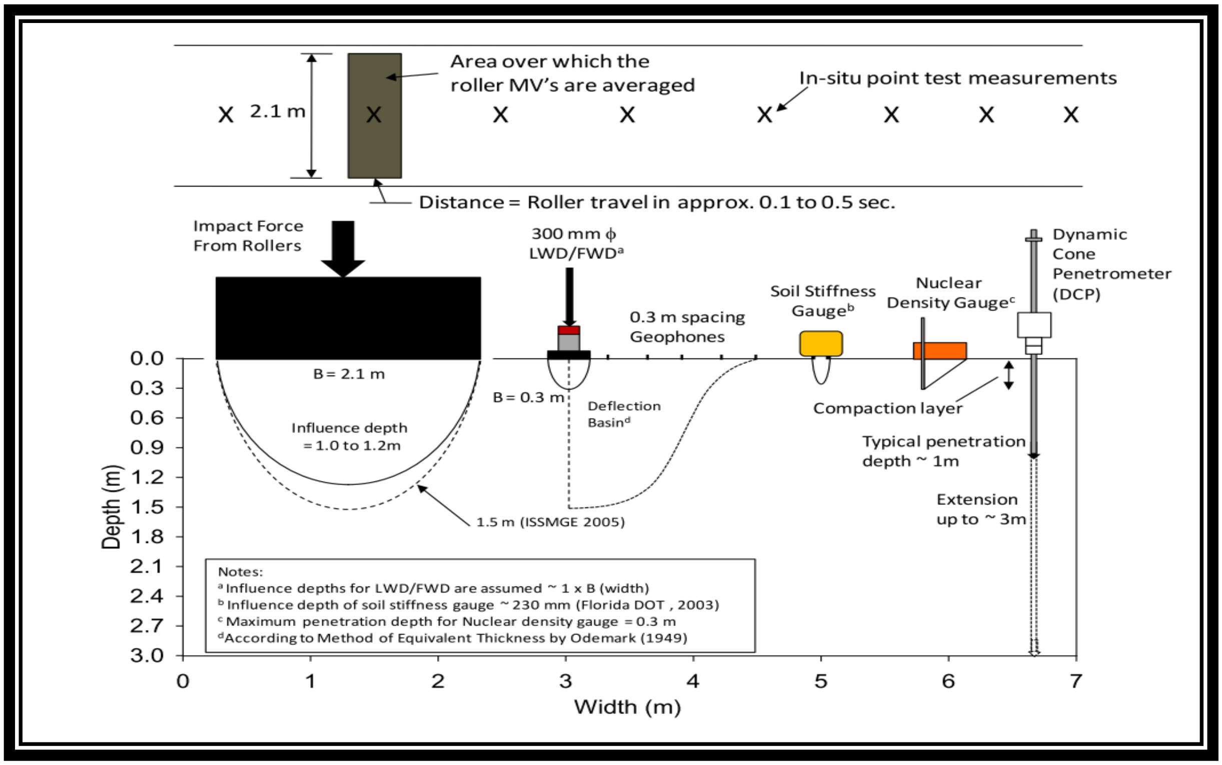


Figure 2.42. Influence depth of various measurements (Cheng et al. 2014)

The NDG is the most commonly used density measurement test for QC/QA after compaction in Oregon and most other state DOTs. NDG measurements are performed on top of the HMA layer after the IC roller has compacted the material (not real-time). For this reason, NDG measurements after compaction slows down the construction and reduces the practicality of using intelligent compaction as a tool for QC/ QA. The FHWA (Chang et al. 2014) used an IC-based Multivariate Nonlinear Panel Density Model calibrated with pass-by-pass NDG measurements and core density data from a test strip of a specific project to predict asphalt layer density using the IC roller accelerations. This model can produce predicted density values along with other existing IC measurements for enhanced QC during construction. The model used the ICMV, roller passes, roller vibration frequency/amplitude, HMA surface and base temperatures, and roller speeds as input parameters to predict asphalt layer densities. Results showed that the model's predicted densities were not correlated reasonably well with core densities although using a panel model helped capture a family of compaction curves in spatial and temporal domains.

Another IC based density measurement technology developed by Commuri et al. (2014) was able to achieve relatively higher correlations between the Intelligent Asphalt Compaction Analyzer (IACA) and the Nonnuclear Density Gauge (NNDG) measurements (Commuri et al. 2014). The IACA system predict asphalt layer densities by using a neural network model that reads the whole frequency spectrum of roller vibrations and categorize them into distinct levels. However, this technology is not commercially available and still needs to be further vetted.

Chang et al. (2011) indicated that different ICMVs are related to the stiffness of the material and not the density. Each IC roller has their ICMV or model for determining stiffness during the compaction process. These ICMVs and in-situ density test data are then used with different statistical methods or algorithms to correlate IC outputs with density to help improve QC/QA practices as discussed previously. In the same study, a relatively low correlation between ICMV and NDG measurements was achieved since IC density predictions are controlled by the stiffness of the entire pavement structure and the underlying support while NDG measures the actual density of the top 6" of HMA layers.

Chang et al.(2014) also concluded that there is not a good correlation between the final IC predicted densities and the core densities. Thus, the final IC predicted density data are not recommended to replace cores for acceptance. There are many likely causes of this, including differences in measurement depths as well as the change in drum rebounds when asphalt temperatures drop below a certain threshold which can cause the measurement depth of ICMV to extend beyond the compacted layer.

2.5.2 Benefits of Intelligent Compaction

Although previous research studies showed a significant lack of correlation between final ICMV and NDG as well as asphalt core densities, IC systems can still provide important benefits for state agencies. Some main benefits of IC currently are:

- To avoid over/under compaction, the IC roller's on-board display with color-coded imaging is used to ensure that the correct number of roller passes are applied for compaction. By integrating the color-coded images with the Pavement Management Systems (PMS), the relationship between roller passes and long-term performance can also be determined. Based on the correlation, roller patterns can be optimized to improve pavement longevity;
- By integrating IC collected asphalt heat maps with the PMS systems, the impact of asphalt temperature and thermal segregation on long-term pavement performance can be determined; and
- Developed automated IC systems can improve compaction coverage during night paving.

A life-cycle cost analysis (LCCA) was performed by Savan et al. (2014) to determine the cost and performance benefits of IC. The study only considered construction and roadway life cost cycles, using hypothetical overlay thicknesses ranging from 5 to 10 centimeters. By using IC, a 54% decrease in life cycle costs was achieved. Increased uniform compaction resulted in a savings of $15,385 per year for a 1.6 lane-kilometer roadway section.

2.6 NUMERICAL MODELS USED TO EVALUATE ASPHALT COMPACTION

Asphalt mixture compaction is an important and final step in controlling the quality of constructed pavement layer. The performance of asphalt pavement greatly depends on the compaction. Under compaction of asphalt mixture reduces the strength of the pavement due to higher air voids and increases the susceptibility to moisture. Over compaction restricts the asphalt concrete from expanding and contracting under heavy truck loads and therefore, increases the probability of rutting.

Many laboratory compaction methods and devices have been developed to adequately simulate field compaction. These laboratory compaction systems aim to produce specimens in the laboratory that are similar to the ones taken from field sections. Superpave Gyratory Compactor (SGC), Linear Kneading Compactor, Marshall Impact hammer, vibratory kneading compactor, and the mobile steel wheel simulator are such laboratory compaction devices.

Although the laboratory compaction methods and systems are effective in simulating field compaction, the movement of aggregates and the changes in internal microstructure of asphalt mixtures during the compaction process may not be effectively visualized and analyzed using the above-mentioned methods.

Numerical models, on the other hand, provide alternative tools for evaluating asphalt mixture compaction. When compared to laboratory methods, numerical simulation methods have an edge of cost, time and flexibility. Some researchers (Koneru et al. 2008; Wang et al. 2007; Zheng et al. 2008) have used the finite element method (FEM) to simulate the asphalt compaction. Koneru et al. (2008) used a framework based on thermodynamic laws to develop a model to study compaction. The Superpave gyratory compactor compaction process was simulated using the ABAQUS software (a FEM software). Ter Huerne et al. (2008) simulated the HMA compaction process under roller compactor conditions utilizing FEM with DiekA code. The differences between reality and the simulated situation for this condition is illustrated in Figure 2.43.

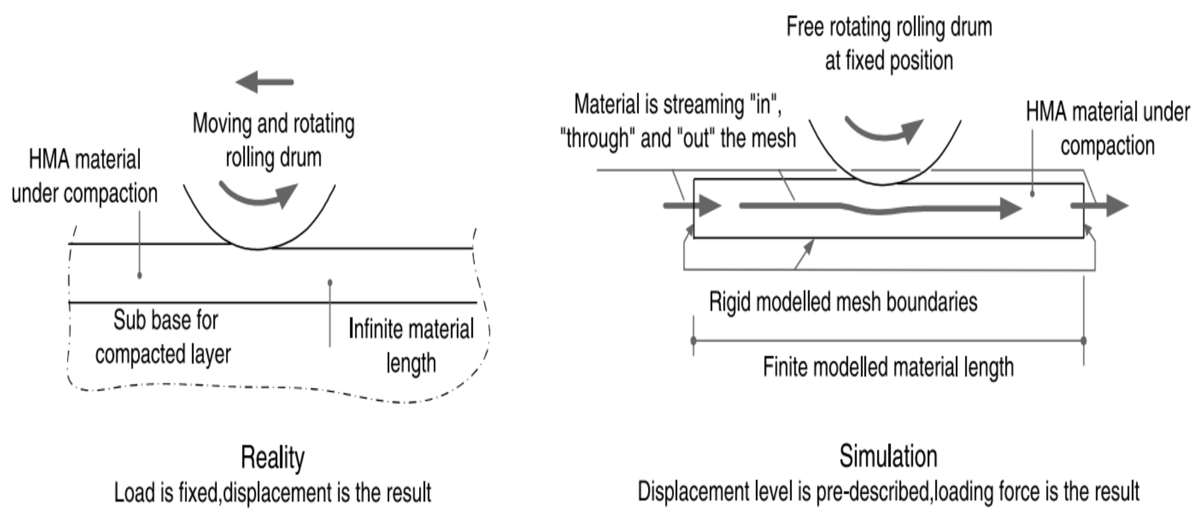


Figure 2.43. The similarities and differences between real rolling process on HMA and its simulation by using FEM (Ter Huerne et al. 2008)

The FEM simulations, however, assume HMA mixture as continuum media and therefore ignore the rearrangement of aggregate particles that occur when the mixture is still hot. Thus, the FEM simulations do not entirely replicate the real scenario and therefore, have limited use (Chen et al. 2014).

Discrete Element Method (DEM) is another numerical simulation technique used by the researchers to understand the interaction among different constituents of mixed materials. Unlike FEM, DEM is a discontinuum analysis method and recognizes contacts and joints between discrete bodies present within a system. It can be used to model the behavior of a composite system and interfaces therein. For this reason, DEM is a more effective tool than FEM to simulate asphalt mixture compaction (Chen 2011).

Wang et al. (2007) used the 3-Dimensional Particle Flow Code (PFC3D) to investigate the mechanisms by which specific factors influence the compaction of pavement materials. The properties investigated were particle shape effect, particle contact property effect and temperature effect. The observations made with the help of DEM simulation were that the mixtures with aggregates that have more elongated and flat shapes, aggregates with high surface textures and coarser aggregates are more difficult to compact. It was also observed that the temperature drop in mixture during transportation and hauling operation makes the compaction more difficult.

Chen (2011) modified an open source DEM code (YADE) and used it with C++ programming language for DEM simulation of asphalt mixture compaction. The objective was to study the effect of compaction factors on the distributions of air void in asphalt-aggregate matrix. Virtual digital specimens were analyzed under different compaction conditions to evaluate the impact of all these factors on compaction (see Figure 2.44). Simulation results were found to be correlated with the data obtained from experiments and results from past research studies. This paved a way for implementation of DEM simulation in asphalt mix design by providing a more simplified method with less laboratory specimen preparation effort. However, accuracy and precision of DEM predicted mixture properties and compaction methods should be confirmed by conducting comprehensive laboratory research studies.

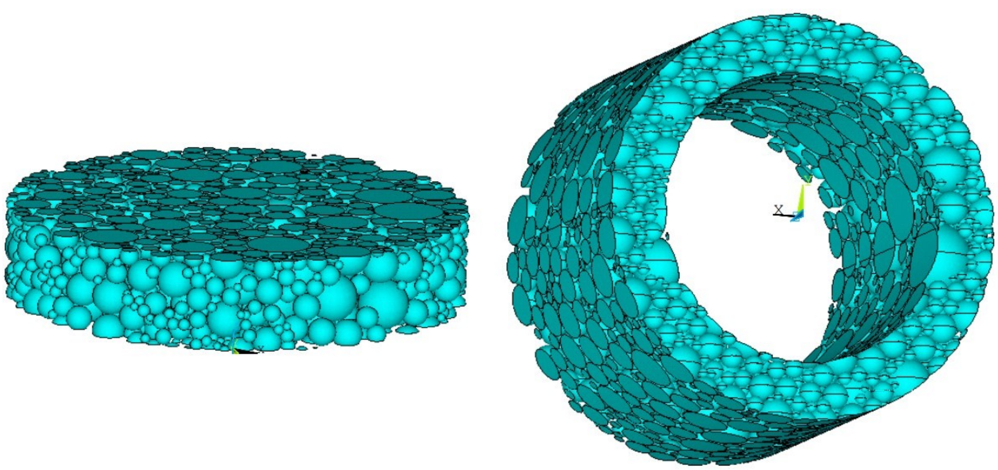


Figure 2.44. DEM digital asphalt mixture specimen (Chen 2011)

2.7 SUMMARY

A review of the literature indicated that the pavement service life can be extended by producing asphalt mixes which are more compactible and are more likely to result in higher in-place density during construction. Results of previous research studies clearly indicate that increased in-place air voids (reduced density) has a detrimental effect on fatigue and rutting performance of asphalt pavements. Several recent research studies (Fisher et al. 2010; Tran et al. 2016) showed that increasing asphalt concrete pavement density by modifying mix design methods, using compaction aids in asphalt mixes, and following better construction practices can lead to significant performance improvements and cost savings.

Fisher et al. (2010) showed that increasing asphalt content in a mixture increases the potential to achieve higher field densities. Higher binder content tends to reduce air-void content of the mix (for equal compaction effort), as expected. However, increased binder content tends to considerably increase the rutting potential and construction cost (Blankenship and Zeinali 2018). Hekmatfar et al. (2015) showed that an increase in in-place density was achievable with optimum binder content chosen at 5 percent air voids, rather than the currently specified 4 percent, and using 50 gyrations as the compactive effort for asphalt specimen preparation. This new approach, popularly called as “Superpave5 design method”, needs further investigation.

Based on the results of previous studies, it can be concluded that density of the asphalt pavement is directly correlated with the pavement performance. For this reason, producing a mix which can achieve higher in-place densities without significantly higher compactive effort can prove to be highly cost-effective. This comprehensive literature review suggests many tools, techniques and strategies to improve in-place density. Some of these strategies such as Superpave5 mix design method, warm mix asphalt, increased binder content, using softer binder, etc., have been evaluated and compared in this study. Recommendations for the best practices that can be adopted in asphalt materials production and pavement construction in Oregon have also been provided in this report.

3.0 IMPACT OF DENSITY ON CRACKING AND RUTTING PERFORMANCE OF ASPHALT MIXTURES – A DETAILED LOOK AT THE SUPERPAVE5 MIX DESIGN

3.1 INTRODUCTION

The current Oregon Department of Transportation (ODOT) standard specification for asphalt mix design calls for a design air-void content of 4 percent for all of their roadway paving mixes except the Level 1 mixes (which targets 3.5% air-void content) that are commonly used for constructing bike paths and sidewalks (for Superpave Gyratory Compacted [SGC] specimens). In Oregon, the asphalt mixture is required to be compacted to a minimum of 92 percent of moving average maximum density (MAMD) in the field (Al-Khayat et al. 2020; ODOT 2021). Moreover, the density of the asphalt mixture exceeding 95 percent of MAMD are required to be reviewed in terms of mix volumetrics and compactive effort to ensure that the higher density is not an indication of a mix with excessive voids-filled with asphalt (VFA) values that may result in high permanent deformation during the service life (ODOT 2021). Considering an average density level, the density levels needed for compliance of this criteria comes to about 93 percent which translates into 7 percent of air void content. These criteria may lead to an uneven distribution of pavement density in field with some sections having density below 92 percent. Such low-density locations along the construction section can undergo cracking due to increased permeability and aging.

During the past decade, some researchers have suggested changing the criteria of the design air void content of laboratory-compacted specimens from 4 percent to 5 percent (Huber et al. 2016; Hekmafter et al. 2015; Motoya et al. 2018). This new mix design method is therefore called as Superpave5. The Superpave5 mix is designed by lowering the design number of gyrations and increasing the target field density to 95 percent. Researchers supporting the Superpave5 expect that the mixtures prepared with this mix design method can easily be compacted to an average of 95 percent density during construction without needing additional compactive effort. It was also proposed that increasing pavement density by following this mix design method can result in improved pavement fatigue life. However, the trial mixtures used for designing Superpave5 mixtures by these researchers generally did not contain any RAP or RAS materials. In Oregon, essentially all asphalt mixtures used in pavement construction have RAP. Huber et al. (2016) found that in terms of rutting resistance, Superpave5 mixes perform same as or better than the conventional asphalt mixes designed with 100 gyrations and 7 percent in-place target air void content level (which were designed to reach 4% air voids after 100 gyrations in the Superpave4 mix design method). Hekmafter et al. (2015) made a similar conclusion about the rutting resistance of the redesigned Superpave5 mixes obtained from FN tests. However, in the same study, the Hamburg test results indicated an opposite trend. Moreover, the cracking resistances of the two mixtures obtained from SCB testing by Hekmafter et al. (2015) were almost the same despite the improved density of the Superpave5 mix. In addition, in almost all laboratory trials, higher densities were achieved by making the aggregate gradation finer and increasing the dust-to-binder ratios. Thus, the impact of increased dust content on fatigue cracking resistance must be investigated before adapting Superpave5 as a mix design strategy in Oregon.

In this part of the study, materials from three different construction projects in Oregon were sampled and asphalt mixtures were produced in the laboratory using the standard Superpave mix design method that is currently being followed in Oregon (called as Superpave4 or SP4 in this report). These mixtures were also re-designed in the laboratory using the Superpave5 mix design method. Cracking and rutting performance of the mixtures prepared by using two different mix design methods were evaluated and compared. Effectiveness of Superpave5 mix design method in Oregon to improve long-term pavement performance was also determined.

3.2 OBJECTIVES

This part of the study has the following objectives:

- Re-designing three conventional Superpave mixes used in Oregon by following the Superpave5 mix design method;
- Determining the cracking and rutting performance of asphalt mixtures designed by using the Superpave4 and Superpave5 mix design method; and
- Comparing the cracking and rutting performance of Superpave5 mixes with the mixes designed by following the current standard method (Superpave4).
- Compactibility of Superpave5 mixes and the mixes design by following the current mix design method were also compared in Chapters 4.0 and 5.0 in this research report.

3.3 MATERIALS

This section provides information about the materials used in the study. Three typical ODOT mixes from three different construction projects were selected. Materials such as virgin aggregates, virgin binders, and RAP and RAS materials were sampled from the asphalt plants of each project. Figure 3.1 shows the RAP material sampled from one of the plants.

Figure 3.1. RAP material sampled from the plant

All three mixes differed in binder source, binder grade and content, aggregate source and gradation, RAP source and gradation, RAP and RAS content, and amount of lime. All three mixes were dense graded with 12.5 mm (1/2" inch) nominal maximum aggregate size (NMAS). Project 1 (Pr1) mix was comprised of 30% of RAP content and PG 64-22 grade virgin asphalt binder. Project 2 (Pr2) mix consisted of 20% RAP and PG70-22ER (polymer modified binder) grade virgin asphalt binder. Project 3(Pr3) mix was produced from a combination of 20% RAP, 1% RAS, and 1% lime with PG 70-28ER grade virgin asphalt binder. Pr2 and Pr3 mixes were designed as Level 4 ODOT mixes which are generally used for constructing highway sections exposed to high traffic volumes. Pr1 mix was a Level 3 ODOT mix which is used for construction at relatively lower traffic locations. In this study, it was assumed that all the RAP binder was completely blended with the virgin binder (100 % blending). The mix design variables of all three mixes are presented in Table 3.1.

Table 3.1. Details of all the Mixtures used in the Study.

Project ID	Highway ID	Mix Design Level	Nominal maximum aggregate size (NMAS)	Binder Grade	Binder Content of the Plant Mix (%)	RAP (%)	RAS (%)	Lime (%)
Pr1	HWY34	Level 3	12.5 mm	PG 64-22	5.7	30	-	-
Pr2	I5	Level 4	12.5 mm	PG 70-22ER	5.9	20	-	-
Pr3	US97	Level 4	12.5 mm	PG 70-28ER	5.1	20	1	1

3.4 RESEARCH APPROACH

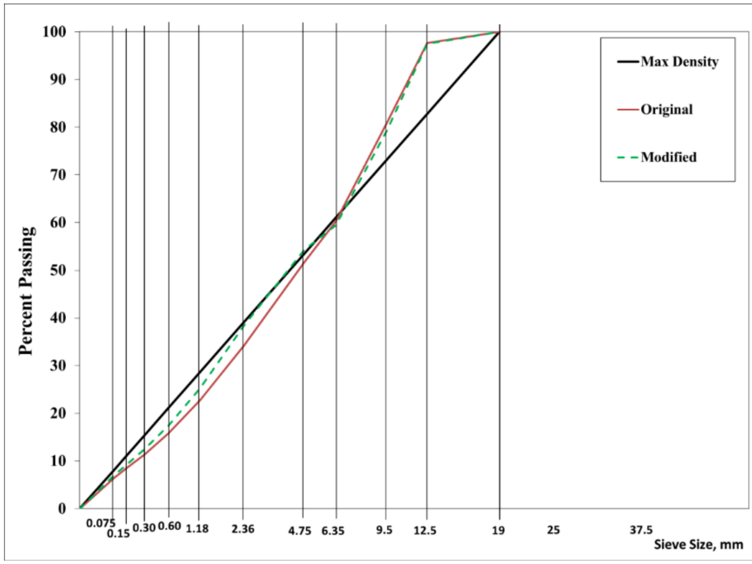
The main purpose of this study was to compare the effectiveness of the current Superpave mix design method (called as Superpave4 or SP4 in this report) with the new Superpave5 mix design method that is expected to produce more compactible (higher density) asphalt mixtures (also called as SP5 in this report). To achieve this objective, three typical mixes used in three different ODOT construction projects were reproduced in the laboratory with mix design variables, such as gradation and binder content, identical to the respective projects. These mixes followed the current ODOT mix design method (SP4) in which the optimum binder content is selected to provide a 6inch diameter asphalt specimen with 4% air voids for a gyration level given in the ODOT specifications. Asphalt laboratory test samples were produced by using this design binder content and compacting them to 7±0.5% air voids. On the other hand, Superpave5 mixes were designed for a 5% air-void content level (instead of the 4% for SP4) for a reduced number of gyrations. Laboratory test samples were compacted to 5±1% air voids. The allowed air-void content error for the SP5 mixes was selected to be ±1% (rather than the ±0.5% followed for all SP4 mixes) since for some SP5 mix types, it was not possible to achieve air-void content values less than 5.5%. For producing Superpave5 specimens, although the same aggregate and binder types were used, the mix design involved reduced design gyration levels and different gradations (finer gradation with more dust), and resulting in different volumetric properties than the SP4 asphalt mixtures.

3.4.1 Modified Gradation and Design Gyration Level

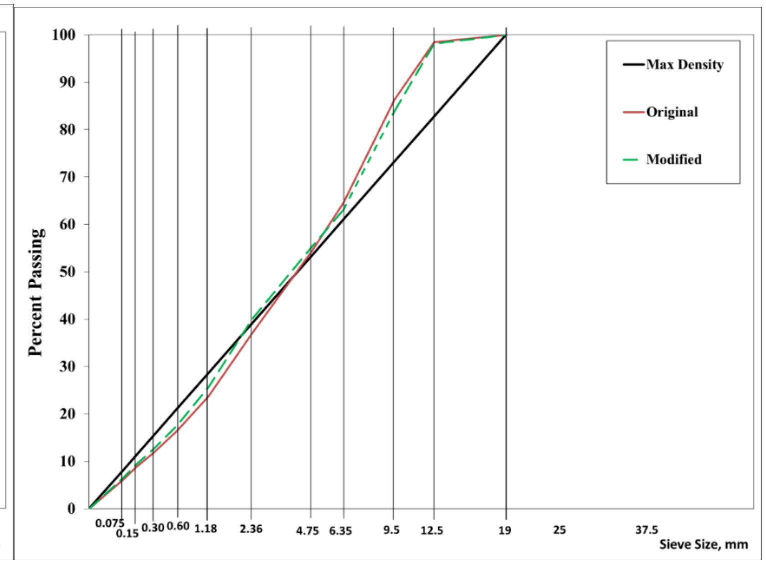
The Superpave5 mix design differs from the current mix design (SP4) in terms of the target final density of the mix (5% air-void content rather than the 4% currently used in SP4). To be able to produce a denser mix, the gradations of the SP4 mixes were modified by changing the proportions of coarse, medium and fine aggregates in each mix such that the gradation curve shifts closer to the maximum density line as shown in Figure 3.2. It is important to note that only the proportions of the virgin aggregates were changed to achieve the SP5 design gradation while the percentages of RAP, RAS, and additive(s) were not changed in the mix. The final modified gradations for the SP5 designs were slightly different from the SP4 gradations. SP5 gradations had more fine aggregates than the SP4 mixes with higher dust in the mixture. This finer gradation component is expected to make the compaction easier by fillings voids between larger aggregates with relatively finer ones.

In addition to the change in gradation, design gyration levels of these SP5 mixes were also reduced depending on the corresponding traffic levels of the three projects investigated in this study. In general, an approximate reduction of 30 gyration levels result in an increase in air voids by 1 percent (Huber et al. 2016; Hekmafter et al. 2015). Therefore, for the SP5 mix designs to achieve a 5% air-void content, design gyration levels of 50 and 70 were selected for the Level 3 and Level 4 mix designs, respectively. These gyration levels were selected based on the suggestions provided by Huber et al. (2016). As per ODOT specifications, standard Level 3 and Level 4 mixes are designed with 80 gyrations and 100 gyrations (to achieve a 4% air-void content), respectively. It is important to note that gyration levels for the SP5 mix design were later changed to 30 and 50 for medium and high traffic locations by INDOT, respectively. The impact of those updated gyration levels on mixture volumetrics and performance should be evaluated in a future research study.

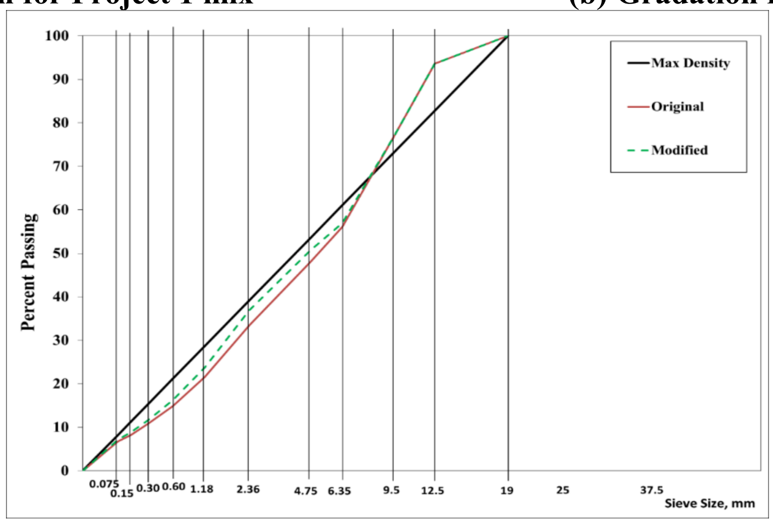
The lower design gyration level, in conjunction with the relatively tighter aggregate configuration to achieve compaction with less effort, was expected to result in design binder content similar to the SP4 mixes. Moreover, reduction in the design compactive effort (in terms of number of gyrations) was used as a method to attain 1% higher design air voids (5% instead of 4%) and also to make the mix more compactible in field. It was purported that designing the mix for a density level that will be targeted during construction will increase the possibility of achieving denser asphalt mixtures (Huber et al. 2016; Hekmafter et al. 2015). The 4% air-void content followed in the SP4 mix design method comes from the air-void content level that the constructed asphalt mixtures are expected to achieve after several years of compaction through vehicular loading (called as secondary compaction). According to recent research studies (Hekmafter et al. 2015), most of the time, asphalt mixtures do not reach the theorized 4% air-void level in the field. For this reason, the target construction air-void content level (not the air-void level expected after secondary compaction) should be used for designing asphalt mixtures (which is the 5% air-void content in SP5 design)



(a) Gradation for Project 1 mix



(b) Gradation for Project 2 mix



(c)Gradation for Project 3 mix

Figure 3.2 Original and modified gradations of all the mixes used in the study

3.4.2 Superpave5 Laboratory Mix Design

Once the target gradations were finalized, four trial binder contents for each mixture were selected for mix design. For each binder content, G_{mm} samples were mixed in triplicate according to AASHTO T 312-12 and their respective G_{mm} values were determined as per AASHTO T 209-12 procedures. Subsequently, four replicate mix design samples were prepared for each binder content and compacted in the Superpave Gyratory Compactor (SGC) using the lower design number of gyrations selected for the SP5 mix design. After compaction, the air-void content for each sample was determined according to AASHTO T 166. The binder content corresponding to the 5 percent air voids was selected as the optimum binder content (OBC) for each mix. The mix design information for both SP4 and SP5 mixes are summarized in Table 3.2. Current ODOT standard (Section 00745.13) for the VMA, VFA, and dust-to-binder ratio parameters are given in parenthesis and italics under the actual measured values. It can be observed that the effective binder content and VFA values for the SP5 mix designs are significantly lower than the SP4 designs. This is a result of having a reduced need for asphalt binder due to the finer gradation and higher dust content for the SP5 mixes filling the voids in the asphalt concrete microstructure. However, it can be observed that all VFA values are still within the ODOT limits for Level 3 and 4 mixtures (between 65 and 75). Dust-to binder ratio for SP5 mixtures are also significantly higher for the SP5 mix designs. In the balanced mix design research project SPR 801 conducted for ODOT, Coleri et al. (2020) determined that dust-to-binder ratio is a statistically significant parameter controlling the cracking resistance of asphalt mixtures used in Oregon. This result suggests that higher dust-to-binder ratio for the SP5 designs may result in lower cracking resistance. Cracking and rutting tests were conducted in this part of the study to determine the impact of using SP5 design on long-term performance.

Table 3.2. Volumetric Properties of all the Mixes

	Project 1		Project 2		Project 3	
Binder PG	PG 64-22		PG70-22ER		PG 70-28ER	
NMAS	1/2" Dense		1/2" Dense		1/2" Dense	
Level	3		4		4	
RAP (%)	30		20		20 (with 1% RAS & 1% lime)	
Mix type	SP4	SP5	SP4	SP5	SP4	SP5
Design air void content (V_a)	4	5	4	5	4	5
Design No. of gyrations (N_d)	80	50	100	70	100	70
Target compacted air void content (AV)	7	5	7	5	7	5
Binder content (P_b)	5.7	6.0	5.9	5.8	5.1	4.7
% Eff. Binder (P_{be})	4.73	4.19	4.58	4.05	4.44	4.22
VMA	14.8 *(14-16)*	14.6 *(14-16)*	14.4 *(14-16)*	14.2 *(14-16)*	14.7 *(14-16)*	15.1 *(14-16)*
VFA	73 *(65-75)*	66 *(65-75)*	72 *(65-75)*	65 *(65-75)*	73 *(65-75)*	67 *(65-75)*
Dust-to-binder ratio (P_{200}/P_{be})	1.4 *(0.8-1.6)*	1.7 *(0.8-1.6)*	1.4 *(0.8-1.6)*	1.6 *(0.8-1.6)*	1.5 *(0.8-1.6)*	1.7 *(0.8-1.6)*

Note: Current ODOT standard (Section 00745.13) for the VMA, VFA, and dust-to-binder ratio parameters are given in parenthesis and italics under the actual measured values.

3.5 SAMPLE PREPARATION AND TEST METHODS

3.5.1 Preparation of Test Specimens

For sample preparation, aggregates and RAP were batched to meet the final gradation and 7% ± 0.5% air content for the standard SP4 mix and the modified gradation and 5% ± 1% air content for the SP5 mix that was re-designed in the lab. The allowed air-void content error for the SP5 mixes was selected to be ±1% (rather than the ±0.5% followed for all SP4 mixes) since for some SP5 mix types, it was not possible to achieve air-void content values less than 5.5%. Then, the batched samples were mixed and compacted by following the AASHTO T 312-12 (2012) specification. Before mixing, aggregates were kept in the oven at 10°C higher than the mixing temperature, RAP materials were kept at 110°C, and binder was kept at the mixing temperature for 2 hours.

After mixing, the AASHTO R 30 (2010) recommends conditioning the prepared loose mixtures for 4 hours at 135°C to simulate short-term aging (STA). The goal of short-term aging is to simulate the aging and binder absorption that occurs during the production and silo storage phases. However, based on the suggestions from the NCHRP 815 (Newcomb et al. 2015) and the findings from the SPR801 ODOT research project (Coleri et al. 2020), a short-term conditioning period of 2 hours at 135°C was adopted. The long-term aging protocol developed for ODOT in Sreedhar & Coleri (2020) was followed for conditioning asphalt mixtures for the SCB cracking tests. Based on the results and recommendations from Sreedhar & Coleri (2020), short-term aged loose mixtures were further aged at 95°C for 24 hours to simulate long-term aging. The conditioning was carried out in a forced draft oven and mixtures were stirred at regular intervals to ensure uniform aging. After LTA conditioning, mixtures were further kept in the oven at compaction temperature for 2 more hours prior to compaction. The mixing and compaction temperatures were obtained from viscosity versus temperature plots for the binder provided by the plants. Cylindrical samples were compacted using a Superpave Gyratory Compactor (SGC) in accordance with the AASHTO T312-12 specification. Asphalt mixtures used for HWTT sample production were only short-term aged (no long-term aging) since rutting generally occurs early in the design life. Asphalt mixtures for only SCB samples were long-term aged to simulate the impact of aging (oxidation and volatilization of different components in the asphalt binder) on long-term cracking resistance.

3.5.2 Test Methods

3.5.2.1 Semi-Circular Bend (SCB) Test

In a previous research study performed at Oregon State University (Coleri et al. 2017), semi-circular bend (SCB) test was selected as the most effective cracking experiment to characterize asphalt mixtures used in Oregon (Sreedhar et al. 2018). Therefore, SCB tests were conducted in this study to determine the cracking resistance of asphalt mixtures and to determine a suitable threshold for the test's output parameter (flexibility index) to be used as an acceptance criterion in the proposed balanced asphalt mixture design process. Test method for evaluating the cracking performance of asphalt concrete at intermediate temperatures developed by Ozer et al. (2016) was followed with few modifications. A displacement rate of 0.5 mm/min was used instead of 50 mm/min (Sreedhar et al. 2018, Coleri et al. 2017).

130 mm tall samples were compacted in the laboratory according to AASHTO T 312-12. Two samples with the thickness of 57 ± 2 mm were sawn from each gyratory compacted sample using a high-accuracy saw. Then, cylindrical samples (cores) were cut into two identical halves using a special jig. Tests were conducted at 25°C with a displacement rate of 0.5 mm/min. Samples were kept in the chamber at the testing temperature for conditioning the day before being tested. Flat side of the semi-circular samples was placed on two rollers (See Figure 3.3). A vertical load with constant displacement rate is applied to the samples and the applied load is measured via a load cell. Test stops when the load drops below 0.5 kN. Flexibility index (FI) is the testing parameter obtained from this test and used for cracking resistance evaluation.

Flexibility Index (FI) is the ratio of the fracture energy (G_f) to the slope of the line (m) at the post-peak inflection point of the load-displacement curve (see Figure 3.4). FI correlates with ductility. Lower FI values show that the asphalt mixtures are more brittle with the higher crack growth rate.

$$\boldsymbol{FI = A \times \frac{G_f}{abs(m)}}$$

(3-1)

Where:

A is a unit conversion and scaling coefficient taken as 0.01.

Figure 3.3. SCB test up

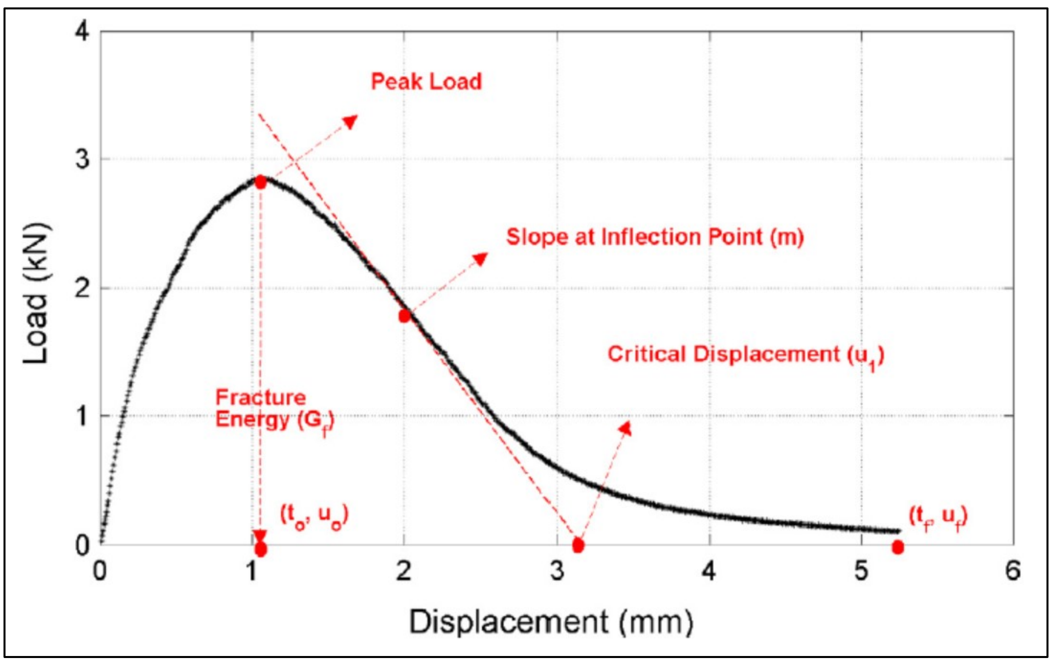


Figure 3.4. Load-displacement curve and slope at the inflection point (m) (Ozer et al. 2016)

3.5.2.2 Hamburg Wheel-Tracking Test (HWTT)

The Hamburg Wheel-Tracking Test (HWTT) system was developed to measure rutting and moisture damage (stripping) susceptibility of an asphalt concrete sample. A typical HWTT system (from the OSU Asphalt Materials Performance Laboratory) has been shown in Figure 3.5.

Figure 3.5. Hamburg wheel tracking device (HWTT) at the OSU Asphalt Materials Performance Lab.

The HWTT follows the AASHTO T 324 standard. According to the specification, either a slab or a cylindrical specimen can be tested. Tests are conducted by immersing the asphalt concrete sample in a hot water bath (at 40°C or 50°C) and rolling a steel wheel across the surface of the sample to simulate vehicular loading (See Figure 3.5). Approximately 20,000 wheel passes are commonly used to evaluate the rutting and stripping resistance of a sample. The test provides information related to the total rut depth, post-compaction, creep slope, stripping inflection point and stripping slope of the asphalt concrete sample (Yildirim et al. 2007; Tsai et al. 2016). Figure 3.6 shows a typical plot of rut depth vs load cycles.

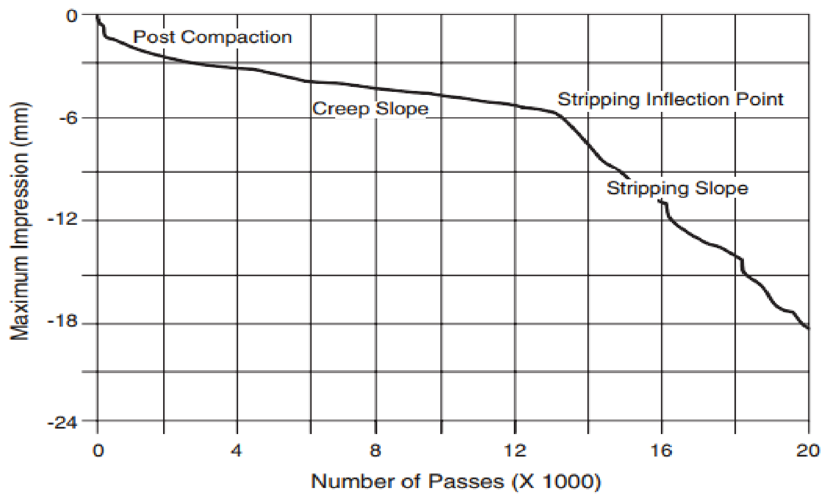


Figure 3.6. Typical HWTT test results obtained from HWTT (Yildirim et al. 2007)

In this study, measured rut depth after 20,000 wheel passes is used for rutting performance evaluation. Cylindrical specimens were used for testing. In this study, selected test temperature for HWTT was 50°C.

3.6 EXPERIMENTAL PLAN

In this study, specimens produced by two different mix design methods (SP4 and SP5) were evaluated for cracking and rutting performance. SP5 mix design method was used to prepare HWTT and SCB test specimens with 5±1% air voids. These were compared with the HWTT and SCB specimens prepared according to the standard SP4 mix design method with 7±0.5% air voids (AASHTO M323). All testing of the prepared samples was carried out at the OSU Asphalt Materials Performance Laboratory following relevant AASHTO test specifications. Table 3.3 shows the experimental plan followed in this study.

Table 3.3. Experimental Plan for Performance Evaluation of the Mix Designs

Tests	Mix Design Type	Projects	Replicate cores	Total no. of test results*
HWTT	SP5 SP4	Pr1, Pr2, Pr3	6	18
SCB	SP5 SP4	Pr1, Pr2, Pr3	4	24

*2 cores of HWTT = 1 test result, 1 SGC core of SCB = 4 specimens and 4 replicate test results

3.7 RESULTS AND DISCUSSIONS

3.7.1 Student's t-test

The air void contents of the all the samples were measured using the standard saturated surface dry (SSD) test method according to AASHTO T 166 specifications. The measured air void content of the samples prepared from two different mix design methods were statistically analyzed to check whether the difference in the air voids between the samples was statistically significant. Student's t-test was conducted on the air void content dataset from two different sample sets, i.e. SP4 and SP5, for each project separately. The results of the test are presented in Table 3.4.

Table 3.4. Student's T-test Results

Project	SP4 Mean AVC	SP5 Mean AVC	t-stat	p-value
Project 1	6.95	5.63	9.18	<0.05
Project 2	7.23	5.89	-8.18	<0.05
Project 3	6.96	5.58	-7.21	<0.05

The p-values in Table 3.4 are less than 0.05 for all the projects at significance level of 5 percent. The results indicate that although there is some variability in SP4 and SP5 specimens' air-void content (SP5 having air void content values generally higher than the target 5%), the difference between the air-void content of these two design methods is statistically significant for all Projects (SP5 samples being denser).

3.7.2 SCB Test Results

Figure 3.7 shows the SCB test results for both mix types. For all the cases, the SP5 mixes have significantly lower flexibility index (FI) than the standard SP4 mixture. The average FI of both Project 2 and Project 3 SP4 type mix was approximately 35% higher than the SP4 Project 1 mix. Both Project 2 and Project 3 had polymer modified asphalt binder which is proven to create mixtures with higher durability and fatigue life. The SP5 mixes were re-designed from the standard mixes to have higher density and were expected to have higher cracking resistances than the SP4 mixes. However, the results shown in Figure 3.7 indicated that, for all three projects, SP4 mixes (current standard in Oregon) had significantly higher cracking resistances than the SP5 mixes although the SP5 SCB test specimens had 2% higher density (5% target air-void content) than the SP4 SCB specimens (7% target air-void content). In addition, using the SP5 mix design method completely removed the benefits of using polymer modified binders to improve cracking resistance (all three projects designed with SP5 had almost equal average FI values although Projects 2 and 3 had polymer-modified binders).

To determine and understand the reasons behind lower cracking resistance for the SP5 mixes, it is imperative to compare other factors that govern the cracking resistance of the asphalt mixes. From Table 3.2, it can be observed that the effective binder content (P_{be}) of all the SP5 mixes were significantly lower than the SP4 mixes. Since the finer gradation with small sized aggregates fill the void space in the asphalt mixture, the binder content needed to achieve the 5% air-void content during the mix design was reduced. In other words, SP5 mix design method tends to fill the void space with fine aggregate particles rather than using the asphalt binder to achieve the required density level. Reduced effective binder content reduced the flexibility of the asphalt mixture and resulted in lower cracking resistance. Higher dust-to-binder ratios for all SP5 mix designs also point out the issue of having excessive amount of dust and not enough binder to achieve the desired high cracking resistance. These results suggested that increasing the density of the mix is not always a guaranteed strategy to improve pavement durability. It is possible to improve compactibility by changing the gradation while higher fine aggregate content can result in lower cracking resistance.

These results also point out the importance of adapting a balanced mix design method in Oregon. Almost all the SP5 mixes are meeting the volumetric requirements specified by ODOT (with the exception of Projects 1 and 3 dust-to-binder ratios while they are just slightly over the upper 1.6 threshold) while the cracking resistance of SP5 mixes are significantly lower than the SP4 mixes. For this reason, conducting cracking and rutting tests during the mix design process and adapting the balanced mix design approach is expected to result in mixes with higher durability.

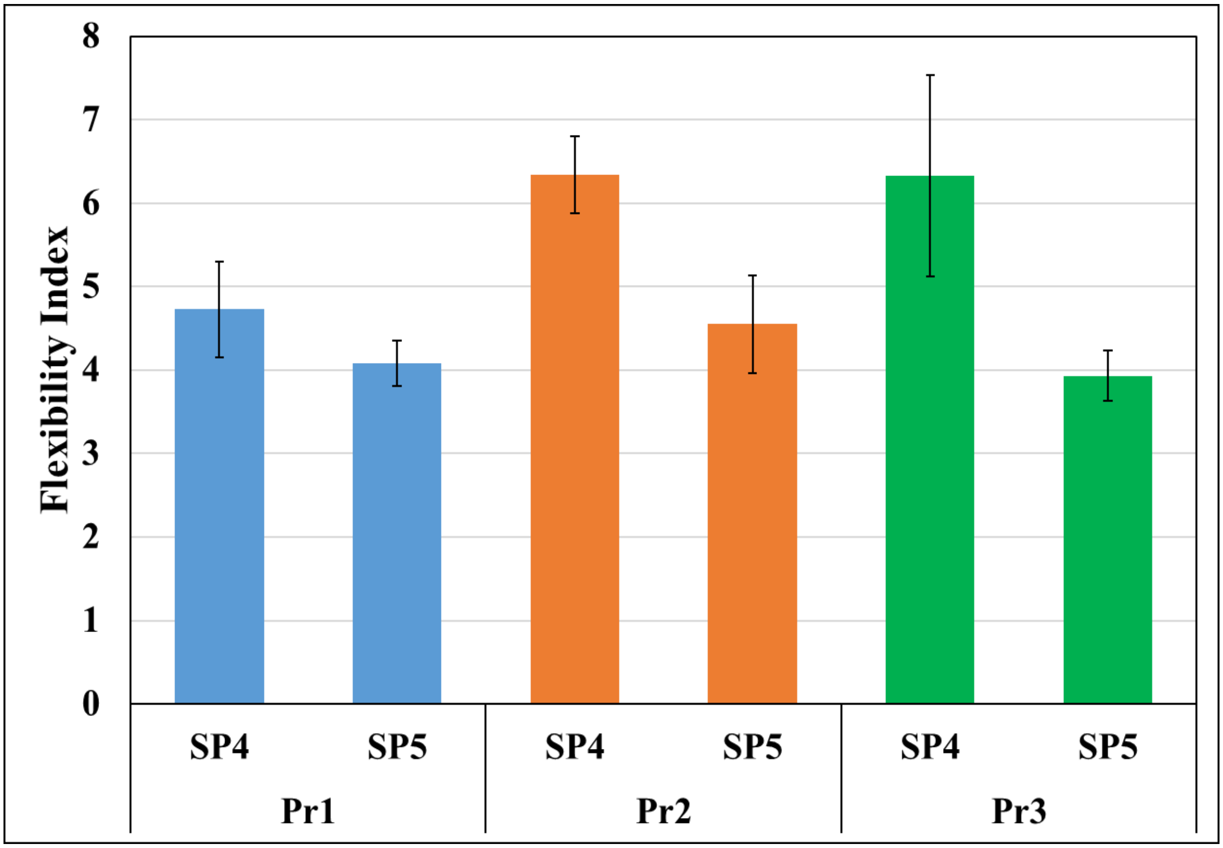


Figure 3.7. SCB test results for both the standard SP4 (SP4) and the re-designed Supepave5 (SP5) mix types (length of the error bar is equal to one standard deviation).

3.7.3 HWTT Test Results

Test samples prepared by using the two different mix design methods were tested in the HWTT test system to determine the mixture rutting performance. The test was carried out according to the AASHTO T 324 standard at 50ºC. The maximum allowable rut depth and the number of passes were input as 12.5 mm and 20,000 passes, respectively, into the test system. Figure 3.8 presents the Hamburg test results for the SP4 and SP5 type mixtures for all the projects. The results indicated that for all the cases, the SP5 mix showed lower rut depth (better rutting resistance) than the corresponding SP4 mix. However, the average rut depths of all the mixtures were lower than 3.0 mm, which is significantly lower than the maximum rut depth allowed by several agencies in the U.S. (which is 12.5mm). This result suggested that both SP4 and SP5 mixes are on the “dry” side and need more asphalt binder to improve their cracking resistance while still meeting the 12.5mm maximum rut depth requirement.

Project 1 mix had the highest average rut depth among all the projects. This was expected because Project 1 binder was unmodified and had a softer PG 64-22 binder compared to the other two projects which both had modified binders. Thus, using polymer modified binders improved both the cracking and rutting resistance of Project 2 and 3 mixtures. Project 3 SP4 mix was found to be more susceptible to permanent deformation than the Project 2 SP4 mix while the difference was not significant.

The observed trend of higher rut resistance in the denser SP5 type mix in Figure 3.8 is expected. In general, asphalt mixtures with lower air-void content experience lower permanent deformation. It is also important to note that the significantly lower effective binder content of the SP5 mixes might be another reason for having higher rut resistance. Since fatigue cracking is the major distress mode in Oregon and the measured HWTT rut depths for both the SP4 and SP5 mixes are significantly lower than the 12.5mm upper limit, adapting the SP5 mix design in Oregon is not expected to create any performance benefits (although it can improve the compactibility of the mix and result in higher asphalt layer density).

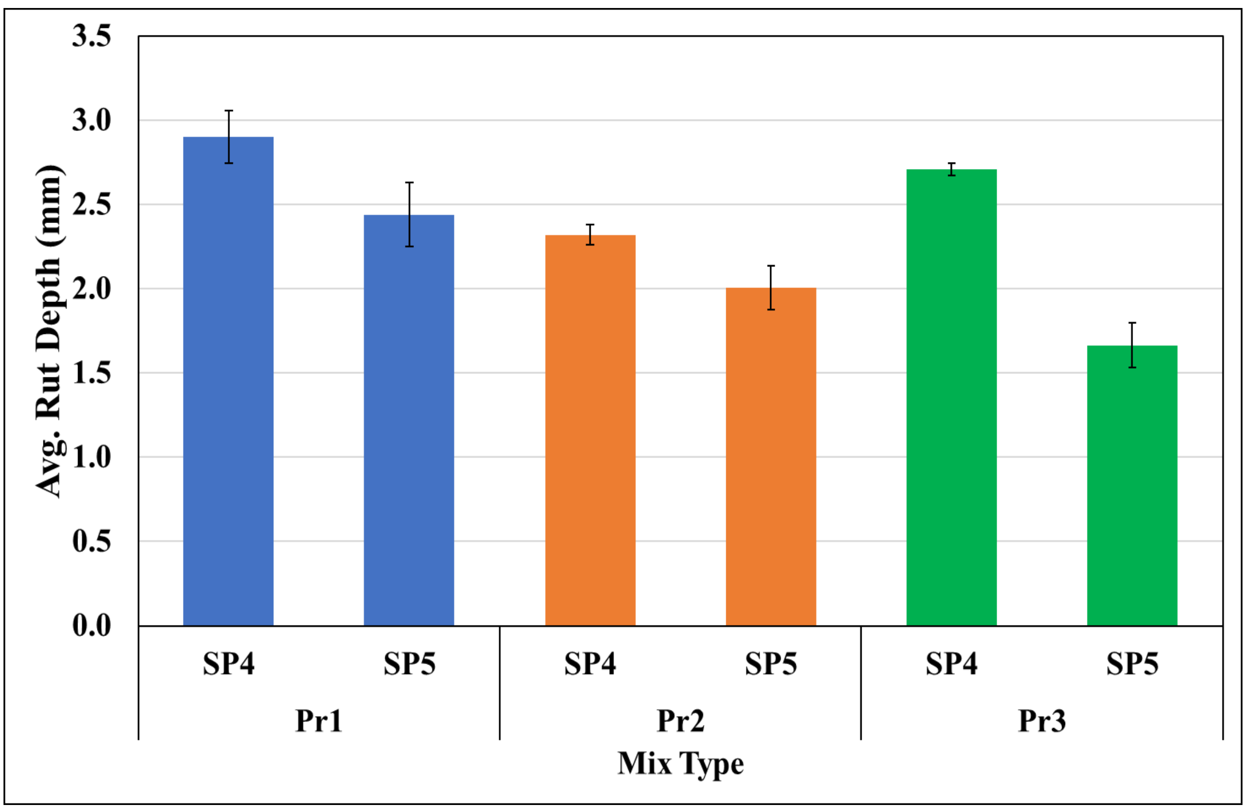


Figure 3.8. HWTT test results (average rut depth at 20,000 repetitions at 50°C temperature) for both the standard SP4 (SP4) and the re-designed Supepave5 (SP5) mix types (length of the error bar is equal to one standard deviation).

3.8 CONCLUSIONS

The major conclusions derived from this part of the study are as follows:

Conclusions based on statistical analysis of air void contents are as follows:

1. The difference between the average air void content of the samples prepared by following the standard mix design (SP4) method and the modified mix design (SP5) was statistically significant. SP5 mixes had significantly lower air-void content than the SP4 mixes.

2. SP5 mix design method is capable of producing mixes that are more compactible. It might be possible to achieve about 1 to 2% higher density during construction by using the SP5 mix design method.

Conclusions based on the SCB testing are listed below:

1. For asphalt mixtures from all the Projects (Project 1, Project 2 and Project 3), SP5 mixes had significantly lower flexibility index than the standard SP4 mixes. The SP5 mixes were re-designed from the standard mix to have higher density and were expected to have higher cracking resistance than the SP4 mixes. However, for all three projects, SP4 mixes (current standard in Oregon) had significantly higher cracking resistance than the SP5 mixes although the SP5 SCB test specimens had 2% higher density (5% target air-void content) than the SP4 SCB specimens (7% target air-void content).

2. SP5 mix design method tends to fill the void space with fine aggregate particles rather than using the asphalt binder to achieve the required density level. Reduced effective binder content reduced the flexibility of the asphalt mixture and resulted in lower cracking resistance.

3. For the SP4 mixes, polymer modified binder used in Project 2 and Project 3 mixes resulted in higher rutting resistance and cracking resistance than the unmodified Project 1 mix.

4. Using the SP5 mix design method completely removed the benefits of using polymer modified binders to improve cracking resistance (all three projects designed with SP5 had almost equal average FI values although Projects 2 and 3 had polymer-modified binders).

5. Higher dust-to-binder ratios for all SP5 mix designs point out the issue of having excessive amount of dust and not enough binder to achieve the desired high cracking resistance. These results suggested that increasing the density and compactibility of the mix is not always a guaranteed strategy to improve pavement durability. It is possible to improve compactibility by changing the gradation while higher fine aggregate content may result in lower cracking resistance. However, it may still be possible to achieve high cracking resistance from asphalt mixes with high dust

content by carefully analyzing the dust-to-binder ratio and overall measured cracking resistance of the mix.

6. While the cracking resistance of the SP5 mixes are significantly lower than the SP4 mixes, almost all the SP5 mixes are meeting the volumetric requirements specified by ODOT (with the exception of Projects 1 and 3 dust-to-binder ratios while they are just slightly over the upper 1.6 threshold). This result suggested that conducting cracking and rutting tests during the mix design process and adapting the balanced mix design approach (performance testing at the mix design stage) is expected to result in mixes with higher durability. Volumetrics alone may not provide the most durable asphalt mixture.

Conclusions based on HWTT testing are given below:

1. For all the cases, the SP5 mix showed lower rut depth (better rutting resistance) than the corresponding SP4 mix. However, the average rut depths of all the mixtures were lower than 3.0 mm which is significantly lower than the maximum allowed rut depth according to AASHTO T 324 (which is 12.5mm). This result suggested that both SP4 and SP5 mixes are on the “dry” side and need more asphalt binder to improve their cracking resistance while still meeting the 12.5mm maximum rut depth requirement.

2. The observed trend of higher rut resistance in the denser SP5 type mix in Figure 3.8 is expected. In general, asphalt mixtures with lower air-void content experience lower permanent deformation. It is also important to note that the significantly lower effective binder content of the SP5 mixes might be another reason for having higher rut resistance. Since fatigue cracking is the major distress mode in Oregon and the measured HWTT rut depths for both the SP4 and SP5 mixes are significantly lower than the 12.5mm upper limit, adapting the SP5 mix design in Oregon is not expected to create any cracking performance benefits (although it can improve the compactibility of the mix and result in higher asphalt layer density).

4.0 IMPACT OF INCREASED COMPACTION TEMPERATURE AND THE USE OF WARM-MIX ADDITIVES ON THE PERFORMANCE OF ASPHALT MIXTURES

4.1 INTRODUCTION

Better compaction and increased density have emerged as two of the most important factors governing the asphalt behavior with regard to pavement longevity, durability and overall performance. In Chapter 2.0, several strategies and techniques to improve density and compactibility of asphalt mixtures were discussed. Two of the potential strategies to improve compactibility and increase density during construction were using warm-mix asphalt (WMA) additives and increasing the asphalt mixing and compaction temperatures. In this part of the study, the impact of using Evotherm P25®, a chemical WMA additive, and the effect of high mixing and compaction temperatures on asphalt mixture performance were investigated. In Chapter 5.0, the impact of using WMA additives and higher compaction temperatures on the compactibility and performance of asphalt mixtures are quantified by using specimens prepared by a roller compactor (instead of the SGC).

Over the past few years, warm mix additives have gained a reputation of being a compaction aid (Prowell et al. 2007; Hurley and Prowell 2005; Tran et al. 2016). Agencies and contractors have already started using different warm mix additives in the construction projects all over the world and also in the United States. In Oregon, ODOT standard specification also allows the use of warm mix asphalt (WMA) concrete at the discretion of the supervising engineer (ODOT 2021). However, the effect of warm-mix additive on performance and compactibility of the asphalt mixture has not been quantified in Oregon in a research study yet. For this purpose, this study evaluates the use of Evotherm P25® as a chemical warm-mix additive and quantifies its impact on the cracking and rutting performance of asphalt concrete materials. However, this study is not entirely focusing on the use of warm mix additives and is only expected to provide a starting point for ODOT in understanding the potential impact of using WMA technologies in Oregon.

The efficiency of compaction is often directly related to the temperature of the mix which in-turn governs the time available for compaction (TAC). There is a common misconception that heating the binder and the mix to high temperatures results in better compaction. This notion has been challenged by many studies (Delgadillo and Bahia 2008). Increasing the mixing and compaction temperatures of asphalt mixtures may result in excessive aging of the asphalt binder. Excessively aged binder may have lower cracking resistance and lower compactibility. In this part of the study, the effect of high mixing and compaction temperatures on the performance of HMA mixtures was determined.

4.2 OBJECTIVES

The main objectives of this part of the study were to:

1. Determine the effect of using WMA chemical additive (Evotherm P25®) on the compactibility and long-term performance of HMA mixes; and
2. Determine the effect of mixing and compaction temperatures on the compactibility and long-term performance of HMA mixes.

4.3 MATERIALS AND METHODS

4.3.1 Experimental Design

4.3.1.1 Experimental plan to quantify the impact of WMA on density and performance of asphalt mixtures

To investigate the impact of using WMA on cracking and rutting resistance of asphalt mixtures, materials from the same three construction projects used in Chapter 3.0 were also used in this part of the study. WMA mixes were prepared by adding 0.5% Evotherm P25® by weight of total binder in the mix while keeping all other components such as binder content, gradation and target air voids exactly the same as the mixes in the previous chapter. In order to observe the effect of the inclusion of warm mix additive in the asphalt mixture, the two different sets of HMA mixes (SP4 and SP5) used in the previous study were considered as the control mixes. The impact of using WMA (instead of the HMA) on the performance of the asphalt mixture was quantified and compared against that of the control HMA mixes.

In addition to cracking and rutting tests, the number of gyrations required to reach the target density for each WMA test specimen were also recorded and compared to that of the equivalent HMA test specimen. The recorded number of gyrations was used as a measure of the compactibility of the mix or the effort required to compact the mixes to the desired level of density. A detailed experimental plan for cracking and rutting tests is shown below in Table 4.1. Two different gradations, one original (SP4) and the other modified (SP5), were used in this section along with the associated binder content as designed in Chapter 3.0. The gradation and binder content (BC) combination of the mixes prepared by conventional mix design was called SP4-$SP4_{BC}$ and the other combination produced by using the Superpave5 mix design method has been referred to as SP5-$SP5_{BC}$.

Table 4.1. Experimental Design for Rutting and Cracking Tests with the WMA Mixes

Tests	Mix type	Gradation-BC	Projects	No. of cores for each mix type	Total Cores	Total no. of test results*
HWTT	WMA at WMA compaction and mixing temperature	SP4-$SP4_{BC}$ SP5-$SP5_{BC}$	Pr1, Pr2, Pr3	6	36	18
SCB		SP4-$SP4_{BC}$ SP5-$SP5_{BC}$	Pr1, Pr2, Pr3	1	6	24

*2 cores of HWTT = 1 test result, 1 SGC core of SCB = 4 test results

4.3.1.2 Experimental plan to determine the impact of higher mixing and compaction temperatures on the compactibility and performance of asphalt mixtures

Two out of three projects used in the previous study were included in this part of the experimental plan. Since Project 1 and Project 2 mixes vary in terms of the type of binder (PG64-22 and PG70-22ER), RAP content (30% and 20%), and design traffic levels (Level 3 and Level 4), those two projects were selected to study the impact of increased mixing and compaction temperatures on the density and performance of the HMA mixtures. In order to capture the effect of temperature on cracking and rutting resistance of the mixes, test specimens were prepared by using two different mix design methods (SP4 and SP5) as described in the previous chapter. The gradation, binder content and other components were kept identical to the mixes used in Chapter 3.0. The only change introduced in this part of the study was that the mixing and compaction temperatures of the HMA were both increased by an average of 20^{o}C.

Table 4.2 shows the experimental plan that was developed for this part of the study and the results were compared against the control mixes from Chapter 3.0.

Table 4.2. Experimental Design for Rutting and Cracking Tests Conducted with HMA Mixes Mixed and Compacted at Increased Temperatures (+20^{o}C)

Tests	Mix type	Gradation-BC	Projects	No. of cores for each mix type	Total Cores	Total no. of test results*
HWTT	HMA at increased temperature	SP4-SP4$_{BC}$ SP5-SP5$_{BC}$	Pr1, Pr2	6	24	12
SCB		SP4-SP4$_{BC}$ SP5-SP5$_{BC}$	Pr1, Pr2	1	4	16

*2 cores of HWTT = 1 test result, 1 SGC core of SCB = 4 test results

4.3.2 Materials

This section provides information about all the materials used in this study. The asphalt mixtures prepared from the raw materials sampled from three different construction projects in Oregon consisted of virgin aggregates, RAP materials, virgin binders, RAS materials, and lime as described in Chapter 3.3. All the materials were provided by local producers in Oregon.

In addition to all materials mentioned above, a chemical warm mix additive (Evotherm P25®) was also used in this study.

4.3.2.1 Warm Mix Additive

The chemical warm mix additive Evotherm P25® was used to prepare the WMA mixes. There are other warm mix additives available in the market such as organic and foaming technology as discussed in Section 2.2.6. However, a chemical additive was preferred in this study because it was shown by several research studies to provide the highest performance benefits (see Section 2.2.6). Evotherm P25® was provided by Albina Asphalt in Oregon. Chemical additives, in general, do not reduce the viscosity of the asphalt binder. Instead, they lower the friction between the aggregate and binder phases of the mix. This provides an opportunity to reduce the mixing and compaction temperatures while reaching the same density levels as HMA. The decisions about the dosage and compaction and mixing temperatures of the WMA mixes were made based on the recommendations of the producer, which is discussed in more detail in the following section.

4.3.3 Sample Preparation

4.3.3.1 WMA mixing and compaction temperatures

As discussed in the previous section, one major advantage of using WMA is that the mixing and compaction temperature can be significantly reduced. In this study, both the mixing and the compaction temperature of the WMA mixes were reduced by 35ºC (95ºF) for both unmodified and modified binder based on the manufacturer's recommendation (see Table 4.4) and selected dosage of WMA additive.

Table 4.3. Recommended Mixing and Compaction Temperature Ranges for WMA Mixes with Evotherm P25® (Ingevity, 2019)

	Unmodified Asphalt	Polymer Modified Asphalt
Dosage rate (by wt% total asphalt binder)	0.25-0.50	0.30-0.75
Mixing temperature range	>104ºC	>118 ºC
Initial (breakdown) compaction range	>99ºC	>104 ºC
Finish rolling compaction range	>60ºC	>66 ºC

Based on the chart shown in Table 4.3, the mixing and compaction temperatures of the WMA mixes were selected. Table 4.4 presents the temperatures used for the control and WMA mixes.

Table 4.4. Mixing and Compaction Temperatures of HMA (control) and WMA Mixes

Project	Grade	HMA (control)		WMA	
		Mixing Temp	Compaction Temp	Mixing Temp	Compaction Temp
1	PG64-22	159ºC	147ºC	124ºC	112ºC
2	PG70-22ER	173ºC	160ºC	138ºC	125ºC
3	PG70-28ER	181ºC	167ºC	146ºC	132ºC

4.3.3.2 Evotherm P25® dosage and mixing procedure

The dosage of the WMA additive was selected based on the material and type of additive used as recommended by the manufacturer as shown in Table 4.5 below:

Table 4.5. Recommended Typical Starting Dosage of WMA Additives (Ingevity, 2019)

Material Used	Typical Starting Dosage (% of total binder content)	Evotherm 3G
Virgin Mix	0.4	M1
Polymer Modified Asphalt (PMA)	0.5	M15
RAP 10% or less	0.4	M17
RAP more than 10%	0.4	M18
RAP 10% or less/ PMA	0.5	J1
RAP more than 10%/ PMA	0.5	J12
RAP/ RAS	0.4	U3
RAP/ RAS/ PMA	0.5	P25®

As discussed before, the selected WMA additive, Evotherm P25®, was to be blended with both unmodified and polymer modified asphalt (PMA) binders. The resulting Evotherm P25® emulsion was incorporated into the mixes with a combination of virgin aggregates, RAP, and RAS depending on the project. Considering the type of additive and material used from the chart provided in Table 4.5, 0.5% dosage was identified as the optimum dosage for all the mixes. Selection of only one dosage value for all the mixes also ensured consistency.

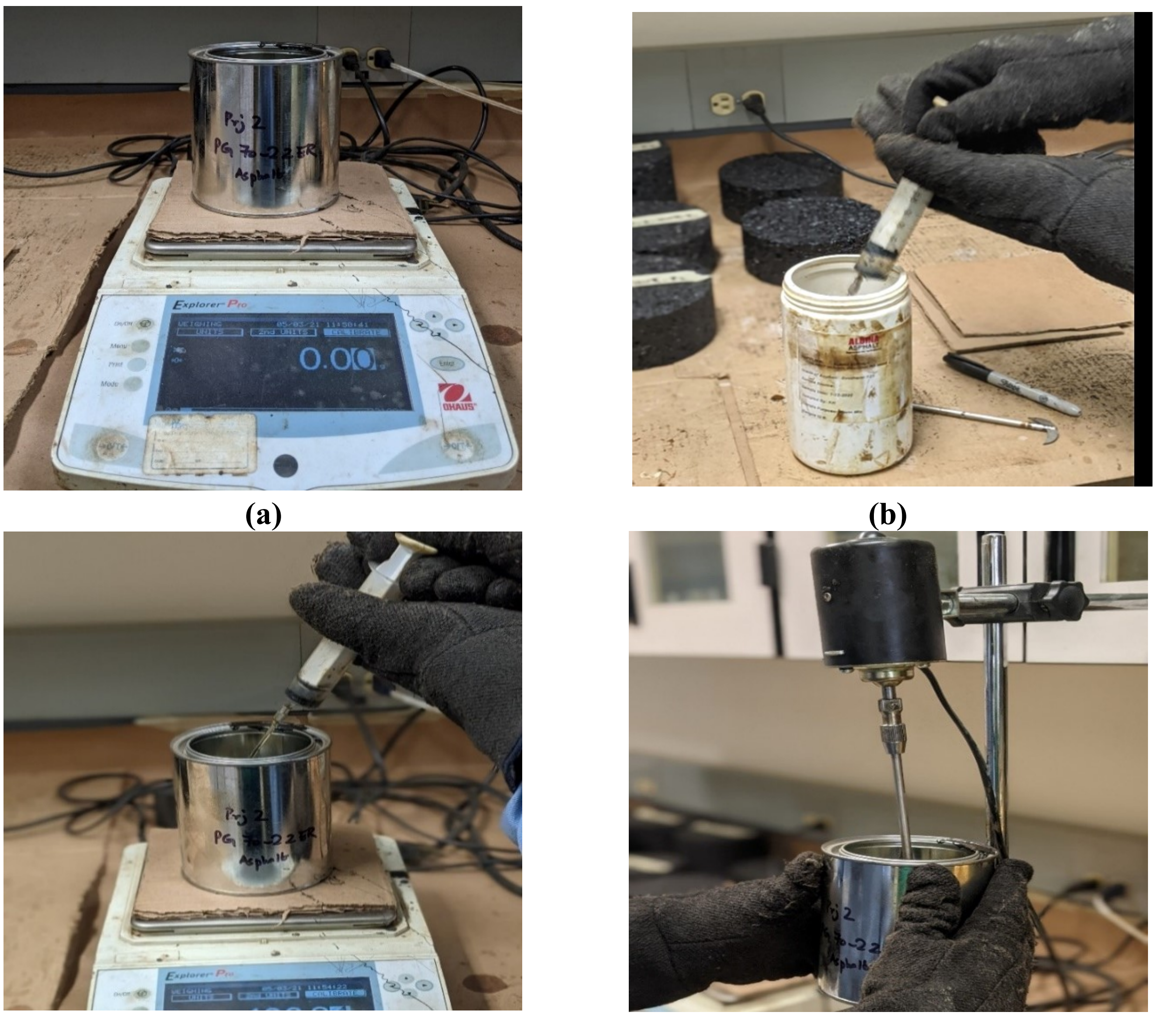
Since the selected dosage was based on the amount of total binder content of the mix, it needed to be adjusted for the mixes with RAP and RAS because the total binder content is the sum of the virgin binder content and the binder content blended from the RAP/ RAS materials. The adjusted Evotherm P25® dosage was calculated by using Equation 4-1 below:

$$\%\ \boldsymbol{Adjusted\ Evotherm\ dosage} = \frac{(\%\ \boldsymbol{Target\ Evotherm\ dosage}) \times (\%\ \boldsymbol{Total\ binder})}{(\%\ \boldsymbol{Total\ binder}\ -\ \%\ \boldsymbol{Binder\ from\ RAP})} \tag{4-1}$$

4.3.3.3 WMA test sample preparation

For the WMA test sample preparation, aggregates and RAP (also RAS and lime for Project 3) were batched to meet the final gradation and air content depending on the mix design method used. Virgin aggregates were kept in the oven at the lower WMA mixing temperature and the RAP materials were kept at 110ºC for 2 hours before mixing. Moreover, the asphalt binder was also heated to the lower mixing temperature (given in Table 4.4) for 1.5 hours prior to the addition of the warm mix additive. After 1.5 hours, the measured amount of additive was immediately injected into the hot binder using a syringe (see Figure 4.1b and c). This step was followed by placing the binder on top of a hot plate and mixing the binder and the injected additive using a shear mixer for approximately 5 minutes (Figure 4.1d). This step was needed to be quickly completed because the binder was already at a low temperature level and tended to cool down rapidly. Once the binder and the additive were completely mixed, the binder was placed back into the oven at the designated WMA mixing temperature for another 30 minutes. The step by step process is shown in Figure 4.1. Immediately before mixing the binder and the aggregates, the temperature of the binder was checked using a thermometer. This was done to ensure that the binder had achieved the required mixing temperature prior to mixing.

After mixing, the prepared loose mixtures were conditioned for 2 hours at 135°C. After STA conditioning, the loose mixtures prepared for SCB test sample production were conditioned for an additional 24 hours at 95°C to simulate long-term aging (Coleri et al. 2020; Sreedhar and Coleri 2020). After conditioning, mixtures were further kept in the oven at the WMA compaction temperature for 2 more hours prior to compaction.



(a) (b)

(c) (d)

Figure 4.1. Mixing process of the chemical WMA additive with the asphalt binder (a) Container with heated binder placed and zeroed on scale (b) Additive filled into a syringe (c) Exact amount of additive injected into the binder (d) Binder and the injected additive mixed using a shear mixer

4.3.3.4 HMA sample preparation with increased temperature

As discussed in the experimental plan 4.3.1.2, the mixing and compaction temperatures of the HMA mixtures used in the previous study were decided to be increased by 20 degree Celsius for this part of the study. However, this 20ºC bump in mixing temperature was not possible for the Project 2 mix because of the presence of the modified binder in the mix. Since at a temperature higher than 182ºC, the polymers in the modified binder might get damaged, the mixing temperature of the Project 2 mix was restricted to 182 ºC, instead of 193 ºC.

Standard sample preparation method discussed in Chapter 3.0 was adopted with only change that the mixes were mixed and compacted at the higher temperatures listed in Table 4.6. The STA conditioning for all the test specimens and the LTA conditioning of the SCB test specimens were done at the standard temperatures and durations as given in Chapter 3.0.

Table 4.6. Increased Mixing and Compaction Temperatures for each Project.

Project	Binder Grade	Original mixing temp.	Original compaction temp.	Increased Mixing Temp.	Increased Compaction Temp.
1	PG64-22	159	147	179	167
2	PG70-22ER	173	160	182	180

4.3.4 Test methods

To evaluate cracking and rutting performance of the asphalt mixture samples prepared in the study, SCB and HWTT test methods discussed in Section 3.5.2 were followed.

4.4 RESULTS AND DISCUSSIONS

4.4.1 SCB Test Results

4.4.1.1 Warm mix asphalt

Figure 4.2 presents the SCB test results for all three projects as well as for SP4 and SP5 mix design methods. The test results for cracking performance of the control mixes were obtained from the previous section (mixes from Section 3.0) and have been shown here for comparison with the cracking resistance of the WMA mixes. SP4 mixes had the gradation, binder content and other volumetric properties identical to the mixes used in the respective construction projects. These mixes were prepared using the traditional mix design method with target 7%±0.5% air void content. SP5 mixes, on the other hand, were prepared with the modified gradation (Figure 3.2), lower number of gyrations (Table 3.2), and redesigned optimum binder content to achieve as-compacted target air void content of 5%±1%. Thus, SP5 mixes were denser than the SP4 mixes. The allowed air-void content error for the SP5 mixes was selected to be ±1% (rather than the ±0.5% followed for all SP4 mixes) since for some SP5 mix types, it was not possible to achieve air-void content values less than 5.5%.

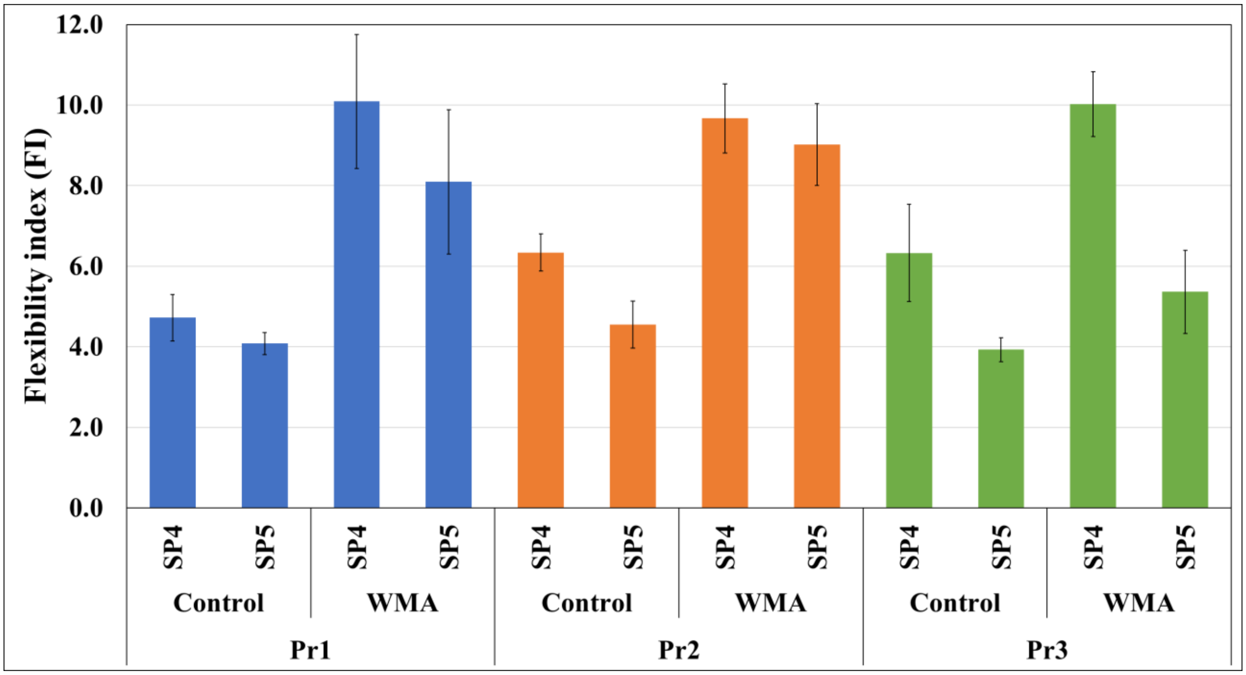


Figure 4.2. FI test results and comparison of durability of HMA (control) and WMA mixes prepared with two different design methods, SP4 and SP5 (length of the error bar is equal to one standard deviation).

It can be observed from Figure 4.2 that all WMA mixes provided significantly higher (1.5 to 2 times higher than the HMA levels in some cases) FI values than the corresponding HMA/control mixes. It should be noted that the WMA mixes were mixed and compacted at temperatures 35ºC lower than the HMA mixes. Thus, addition of Evotherm P25® by 0.5% of the total weight of binder resulted in a considerable improvement in the cracking resistance without increasing the binder content and at lower compaction temperatures that maybe more comfortable for construction workers.

From Figure 4.2, it can also be noted that the FI values of the SP5 type mixes for all WMA mixes of the three projects were lower than the respective SP4 type mixes. Similar trends were also observed in the previous study (Section 3.0) and the reasons were speculated to be lower effective binder content and higher dust-to-binder ratio in the SP5 variant of the conventional mix. Although the SP5 mixes were designed at and compacted to 5%±1% air voids, the expected improvement in durability due to higher density was eliminated due to the significant changes in other volumetric properties, such as effective binder content and dust-to-binder ratio.

Another important observation drawn from Figure 4.2 is that all the WMA mixes except the Project 3 SP5 mix reached the FI thresholds recommended by ODOT Research Project SPR 801 (Coleri et al. 2020). Coleri et al. (2020) stated that an FI threshold of 8 for Level 4 mixes and FI threshold of 6 for Level 3 mixes are expected to provide high in-situ cracking resistance in Oregon. The low cracking resistance of the Project 3 SP5 mix was expected because of lower binder content (4.7%), lower effective binder content (4.22%), and higher dust-to-binder ratio.

4.4.1.2 HMA with increased mixing and compaction temperatures

Figure 4.3 compares the SCB test results for HMA samples prepared at the standard mixing and compaction temperatures and the samples mixed and compacted at higher temperatures. It can be observed that the high mixing and compaction temperatures significantly affected the flexibility index (FI). Higher mixing and compaction temperatures significantly lowered the cracking resistance of the mixes. The Project 1 mix with unmodified binder showed a reduction in FI value by approximately 1 point for both SP4 and SP 5 mixes compared to the control mix. For the Project 2 mix with modified binder, the reduction in average FI values was measured to be 1.5 to 2 points. This higher loss of cracking resistance in Project 2 mixes due to increased mixing and compaction temperatures compared to the Project 1 mixes is most likely to be a result of the damaged polymers in the modified binder at high temperatures. The results suggested that asphalt mixtures start to exhibit more brittle behavior when the mixing and compaction temperatures were increased by 20ºC. Based on the test results, it can be concluded that increasing the production temperatures of the HMA mix to achieve higher density in field is not an effective strategy as it sacrifices the durability of the mix.

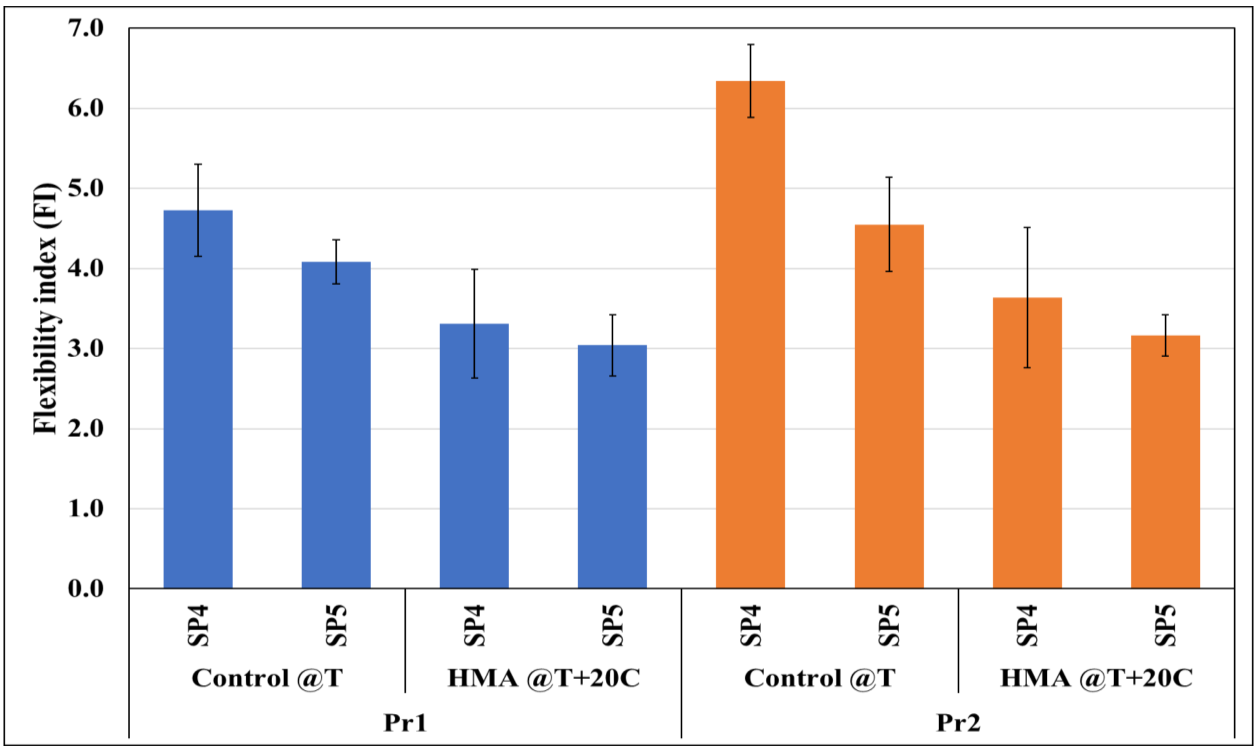


Figure 4.3. The impact of increased mixing and compaction temperatures on cracking resistance (length of the error bar is equal to one standard deviation).

4.4.2 HWTT Test Results

4.4.2.1 Warm mix asphalt

HWTT tests were conducted on the WMA test samples and the results were compared with the control test samples. The results of the HWTT tests are presented in Figure 4.4.

Average rut depth accumulated on the test specimens after 20,000 wheel passes was used as a measure to quantify the rutting resistance of the asphalt mixture.

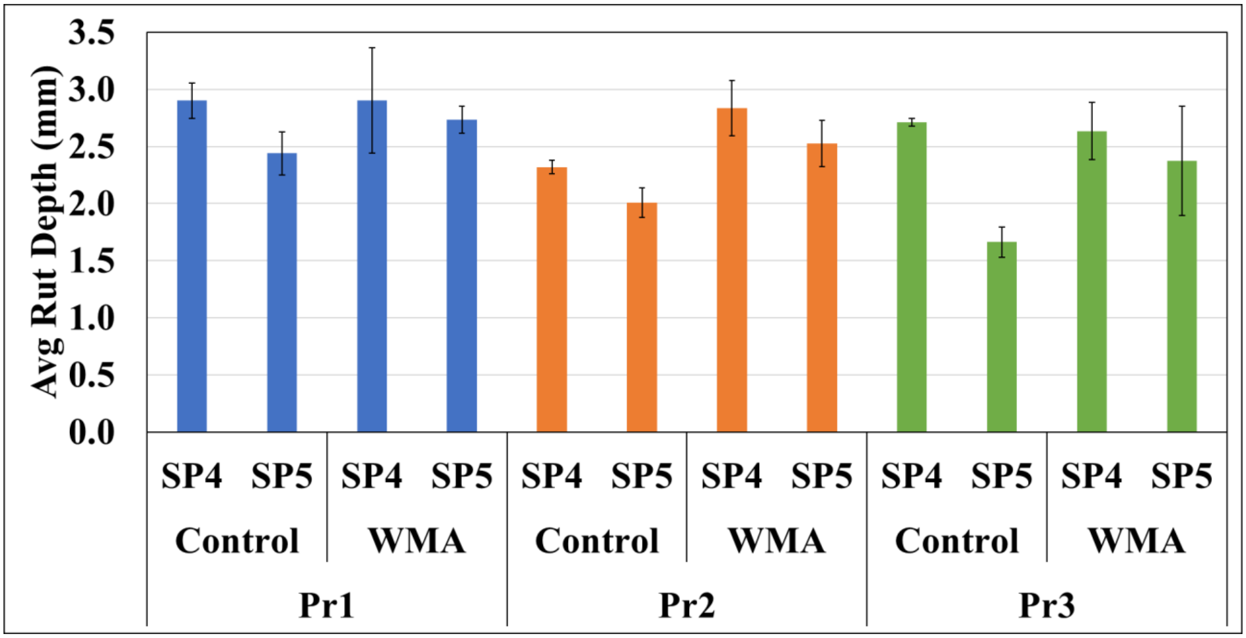


Figure 4.4. HWTT test results for the HMA (control) and WMA mixes prepared with two different design methods, SP4 and SP5 (length of the error bar is equal to one standard deviation).

It can be observed from Figure 4.4 that the average rut depth of all the WMA mixes are greater than the control mixes. This can either be related to the softening effect of the Evotherm P25® additive or due to the low mixing and compaction temperatures used for the WMA mixes resulting in less aging or stiffening of the mix, or both. To ascertain the main reasons behind this behavior, WMA samples were also prepared at the standard HMA mixing and compaction temperatures separately (see Chapter 5.0). It should be noted that compared to the control HMA mix, WMA mixes did not perform significantly lower. In fact, the average rut depths of WMA mixes varied only in the range of approximately 0.5 mm from that of the control mixes. In addition, it should be noted that HWTT rut depths for all the mixtures are significantly lower than the 12.5mm rut depth threshold followed by several state DOTs and other agencies. Therefore, based on the results, it can be concluded that the use of Evotherm P25® WMA chemical additive in the asphalt mixtures has the potential of reducing the production temperatures by about 35ºC without significantly reducing the rut resistance of the mix. SP5 variant of all the WMA mixes showed lower rut depth and higher rut resistance than the standard SP4 type mix. This behavior of SP5 type mix can be attributed to the combination of lower binder content (except for Project 1) and lower air void content of the SP5 test samples.

According to Coleri et al. (2020), the HWTT rut depth thresholds for the state of Oregon were selected as 2.5 mm for Level 4 mixtures and 3.0 mm for Level 3 mixtures. From the results shown in Figure 4.4, it can be seen that Project 1 mixes agree with the rut depth threshold of Level 3 mixes and Project 2 and Project 3 mixes are also almost in compliance with the Level 4 thresholds. The average rut depths of all the mixes are below 3 mm which can be categorized as excellent performance against permanent deformation. However, it should be noted that all the measured rut depth values were significantly lower than the maximum allowed rut depth according to the AASHTO T 324 (which is 12.5mm). This result suggested that both SP4 and SP5 mixes with and without WMA additives are on the “dry” side and need more asphalt binder to further improve their cracking resistance while still meeting the 12.5mm maximum rut depth requirement.

4.4.2.2 HMA with increased mixing and compaction temperatures

Figure 4.5 shows the HWTT test results of the HMA mixes mixed and compacted at temperatures 20°C higher than the standard temperature. The results show that the average rut depth of all mixes prepared at higher mixing and compaction temperatures are significantly lower than the mixes prepared at lower temperatures. The reduction in rut depth is more than 0.5 mm for both projects. When exposed to higher temperatures, asphalt mixes undergo an accelerated aging process. This faster aging process leads to the stiffening of the mix. Stiffer mixes have lower rutting potential than the softer mixes and this phenomenon was clearly observed in the test results. Similar to the trend observed for the WMA mixes, the SP5 mixes had higher rut resistance than the SP4 mixes owing to the 2% lower air-void content (higher density) and also lower binder content. Moreover, Project 2 mixes showed higher rut resistance than the Project 1 mixes. This behavior is most likely due to the use of polymer modified asphalt in the Project 2 mixes (PG 70-22ER), which is stiffer than the Project 1 binder (PG 64-22).

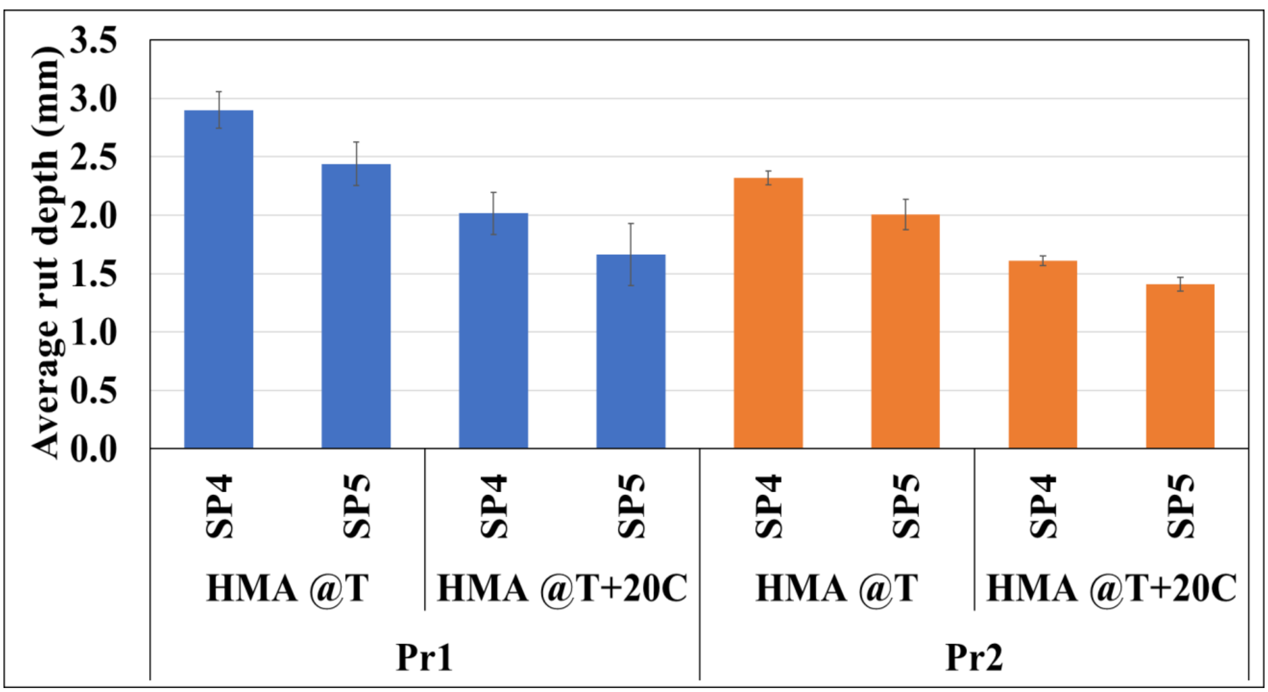


Figure 4.5. Comparison of the rut resistance of HMA (control) mixes mixed and compacted at the standard temperatures and HMA mixes mixed and compacted at higher (+20°C) temperatures (length of the error bar is equal to one standard deviation).

4.5 CONCLUSIONS

The major conclusions derived from this part of study are as follows:

Semi-Circular Bend (SCB) Test:

1. All WMA mixes provided significantly higher (1.5 to 2 times higher than the HMA levels in some cases) FI values than the corresponding HMA/control mixes. Addition of Evotherm P25® by 0.5% of the total weight of binder resulted in a considerable improvement in the cracking resistance without increasing the binder content and at lower compaction temperatures that maybe more comfortable for construction workers.

2. The FI values of the SP5 type mixes for all WMA mixes of the three projects were lower than the respective SP4 type mixes. Similar trends were also observed in the previous study (Section 3.0) and the reasons were speculated to be lower effective binder content and higher dust-to-binder ratio in the SP5 variant of the conventional mix. Although the SP5 mixes were designed at and compacted to 5%±1% air voids, the expected improvement in durability due to higher density was eliminated due to the significant changes in other volumetric properties, such as effective binder content and dust-to-binder ratio.

3. All the WMA mixes except the Project 3 SP5 mix reached the FI thresholds recommended by the ODOT Research project SPR 801 (Coleri et al. 2020). The low cracking resistance of the Project 3 SP5 mix was expected because of lower binder content (4.7%), lower effective binder content (4.22%), and higher dust-to-binder ratio.

4. Increasing the production temperatures of the HMA mix to achieve higher density in field is not an effective strategy as it sacrifices the durability of the mix. The results suggested that the asphalt mixtures start to exhibit more brittle behavior (due to excessive aging of the asphalt binder) when the mixing and compaction temperatures were increased by 20°C.

5. The loss of cracking resistance due to increased mixing and compaction temperatures was more prominent in the Project 2 mixtures than the Project 1 mixtures. This may be indicating that in addition to excessive aging of the binder, the polymers present in the Project 2 modified binder were also getting damaged at high mixing temperatures.

Hamburg Wheel-Tracking Test (HWTT):

1. The use of Evotherm P25® WMA chemical additive in the asphalt mixtures has the potential of reducing the production temperatures by about 35°C without significantly reducing the rut resistance of the mix.

2. SP5 variant of all the WMA mixes showed lower rut depth and higher rut resistance than the standard SP4 type mix. This behavior of SP5 type mix can be attributed to

the combination of lower binder content (except for Project 1) and lower air void content of the SP5 test samples.

3. Project 1 mixes agree with the rut depth threshold of Level 3 mixes suggested by the ODOT Research project SPR 801 (Coleri et al. 2020). Project 2 and Project 3 mixes are also almost in compliance with the Level 4 thresholds. The average rut depths of all the mixes are below 3 mm which can be categorized as excellent performance against permanent deformation. However, it should be noted that all the measured rut depth values were significantly lower than the maximum allowed rut depth according to the AASHTO T 324 (which is 12.5mm). This result suggested that both SP4 and SP5 mixes with and without WMA additives are on the “dry” side and need more asphalt binder to further improve their cracking resistance while still meeting the 12.5mm maximum rut depth requirement.

4. The average rut depth of all mixes prepared at higher mixing and compaction temperatures are significantly lower than the mixes prepared at lower/standard temperatures. This is expected to be a result of the excessive binder aging during the mix production stage.

5.0 EVALUATION OF PERFORMANCE AND COMPACTABILITY OF ASPHALT MIXTURES USING A HYDRAULIC ROLLER COMPACTOR

5.1 INTRODUCTION

Volumetric asphalt mix design process provides a basis to estimate the asphalt mix behavior in the field by helping with the prediction of constructability and performance of asphalt surfaced pavements. Among all the variables associated with asphalt mix design evaluated in the laboratory, compaction is one of the most significant factors affecting the performance of the mix. Compaction is the process of reduction in volume (densification) of asphalt mixtures during construction by moving the aggregates to their lowest energy positions via a compactive effort provided by a compactor (generally a roller compactor in Oregon). Achieving high construction densities during construction has immense importance to construct asphalt surfaced pavements with high long-term performance. Therefore, to be able to prepare realistic laboratory specimens for volumetric and performance evaluations, laboratory compaction should accurately simulate the field conditions.

Superpave Gyratory Compactor developed (SGC) by the Strategic Highway Research Program (SHRP) has emerged as the most popular compaction method because of its simplicity, portability and better correlation with the field compaction than the kneading mechanism-based compaction methods (Swiertz et al. 2010). However, according to several research studies (Harvey and Monismith 1993; Airey and Collop 2014; Harvey et al. 2014), roller compactors are capable of more accurately simulating field compaction while excessive compactive effort applied by the SGC creates an asphalt mixture with an unrealistic microstructure that does not resemble the microstructure of specimens sampled from the field. Since the compaction process with a laboratory roller compactor is almost identical to field compaction, it offers a more accurate understanding of the compactibility and performance of asphalt mixtures. Although the SGC can still be used for volumetric asphalt mix design and preparation of mixes for cracking and rutting performance testing, evaluation of compactibility via an SGC is not expected to provide any conclusive results. For this reason, in this part of the study, a laboratory hydraulic roller compactor (with simulated roller vibration) was used to quantify the compactibility of asphalt mixtures with different properties. Specimens prepared to quantify the compactibility were also used to conduct HWTT and SCB tests to determine the impact of different mix production and construction variables on long-term deformation and fatigue cracking resistance of asphalt mixtures.

In this part of the study, all the samples were prepared with a laboratory hydraulic roller compactor. Two different mix design types (SP4 and SP5) from Section 3.6 with Project 1 and Project 2 materials (see Table 3.1) were reproduced in the laboratory using a hydraulic roller compactor.

5.2 OBJECTIVES

The major objectives of this part of the study are to:

- Conduct a compactibility evaluation via the laboratory hydraulic roller compactor based on the number of passes to reach a specified density level;
- Determine the cracking and rutting resistance of all roller compacted specimens;
- Determine the impact of using WMA on the compactibility and performance of asphalt mixtures;
- Quantify the impact of using the standard HMA mixing and compaction temperatures for the preparation of WMA mixes on compactibility and performance; and
- Examine the effect of increasing the design binder content by 0.5% on the compactibility, performance, and cost of HMA mixes.

5.3 MATERIALS AND METHODS

5.3.1 Experimental Design

5.3.1.1 Experimental plan to compare the cracking and rutting resistance of roller compacted HMA and WMA mixes

This part of the study was designed to determine the cracking and rutting resistance of HMA and WMA mixes compacted by using a laboratory Hydraulic Roller Compactor (HRC). The HMA mix was treated as the control mix for the purpose of comparison throughout this study. Two mix types, SP4 and SP5, which differed in gradation, mix design method and binder content were prepared by selecting two out of three construction projects used in Chapter 3.0. The two projects used in this portion of study were Project 1 and Project 2 (Table 3.1). The details and volumetrics of the two construction project mixes and the mix types are presented in Table 3.1 and Table 3.2, respectively. Since the same mixes and mix types were used in this part of the study, comparison could also be drawn between the two compaction methods – SGC versus HRC.

HWTT and SCB tests were selected as the performance tests for rutting and cracking, respectively. The experimental plan followed in this part of the study is given in Table 5.1. The WMA samples were prepared at the low recommended WMA temperatures while the HMA mixes were mixed and compacted at the designated (higher) temperatures.

Table 5.1. Experimental Plan for Cracking and Rutting Tests on HMA and WMA Mixes

Tests	Mix type and preparation temp.	Gradation-Binder Content (BC)	Projects	Block Replicates	Test samples from each block[a]	Total no. of test results[b]
HWTT	WMA at low-WMA temp. HMA at high-HMA temp.	$SP4\text{-}SP4_{BC}$ $SP5\text{-}SP5_{BC}$	Pr1, Pr2	3	2	24
SCB	WMA at low-WMA temp. HMA at high-HMA temp.	$SP4\text{-}SP4_{BC}$ $SP5\text{-}SP5_{BC}$	Pr1, Pr2	1	4	32

[a] 2 core samples extracted from each block. 1 SCB core yields 2 test replicates.
[b]2 cores of HWTT = 1 test result

5.3.1.2 Experimental plan to determine the impact of high mixing and compaction temperatures on the WMA mix and higher binder content on the HMA mix

To investigate the impact of higher binder content on the roller compacted HMA mix performance as well as to observe the effect of raising the mixing and compaction temperature of the WMA mix to the level of the standard HMA mix on the performance of the roller compacted WMA mix, Project 2 was chosen to produce test samples using both SP4 and SP5 mix design methods. The binder content of the same Project 2 HMA mix used in the previous section was bumped up by 0.5% by weight to prepare the higher binder content HMA mixes. Tests conducted on each HMA and WMA samples included both HWTT and SCB tests for measuring the rutting and cracking resistances, respectively. The experimental plan to achieve the above-mentioned objective is shown below in Table 5.2.

Table 5.2. Experimental Plan for Performance Tests on HMA Mix with Increased Binder Content and WMA Mix Samples Prepared at the Standard (higher) HMA Temperature.

Tests	Mix type and preparation temp.	Gradation-Binder Content (BC)	Projects	Block Replicates	Test samples from each block[a]	Total no. of test results[b]
HWTT	HMA+0.5%AC WMA at high-HMA temp.	$SP4\text{-}SP4_{BC}$ $SP5\text{-}SP5_{BC}$	Pr2	3	2	12
SCB	HMA+0.5%AC WMA at high-HMA temp.	$SP4\text{-}SP4_{BC}$ $SP5\text{-}SP5_{BC}$	Pr2	1	4	16

[a] 2 core samples extracted from each block. 1 SCB core yields 2 test replicates.
[b]2 cores of HWTT = 1 test result

5.3.2 Materials

The information about virgin binders, virgin aggregates, RAP material, and WMA additive used in this study has been provided below. All the materials discussed in this section were obtained from local sources.

5.3.2.1 Aggregates and Recycled Asphalt Pavement (RAP)

The same projects and mix types that were used in Chapter 3.0 were reproduced in this part of the study. The only difference was the compaction method which was changed from gyratory compaction method to roller slab compaction. The virgin aggregates of Project 1 were obtained from the Knife River Plant in Corvallis, Oregon and that of Project 2 were donated by the River Bend Sand and Gravel in Salem, Oregon. The aggregates from both plants were taken from three different gradation stockpiles, namely coarse (1/2” to #4), medium (#4 to #8) and fine (#8 to zero). Aggregates from these three stockpiles were collected and stored in different barrels, labeled and stored at OSU (Figure 5.1).

(a) **(b)**

Figure 5.1. Storage of virgin aggregates

To determine the gradation of each stockpiled aggregate, wet-sieve and dry-sieve analyses were performed on multiple samples of each stockpile following AASHTO T 27-14 (AASHTO 2014). RAP materials were also provided by the same source. AASHTO T 308-10 (AASHTO 2010) was followed for binder extraction and RAP binder content measurements. The quantity of binder in Project 1 and Project 2 RAP materials were determined as 5.80% and 6.00%, respectively. AASHTO T 30-10 was followed to determine the gradations of extracted RAP aggregates of each project. The RAP material was stored in a temperature-controlled environment to prevent aging of the coated binder on the RAP aggregates.

5.3.2.2 Binders

Knife River Plant in Corvallis, Oregon provided the asphalt binder for Project 1 while the binder for Project 2 was sampled from Oregon Mainline Paving in Salem, Oregon. Project 1 binder grade was PG 64-22 and Project 2 binder grade was PG 70-22ER. Temperature curves, mixing temperatures and compaction temperatures were provided by the producer. Laboratory mixing and compaction temperatures were estimated by using the viscosity-temperature lines following the procedure described in AASHTO T 316-11 (AASHTO 2011).

5.3.2.3 Warm mix additive

Evotherm P25® was used as the chemical warm mix additive in this study and it was provided by Albina Asphalt, Oregon.

5.3.2.4 Target gradations

Two different mix design methods were used to prepare test samples. The original gradation curves obtained from the production sheets provided by the plants were reproduced in the lab and were called as SP4 type mix. The gradation curves of the two projects were then modified and the mixes were redesigned with lower number of gyrations and design air void content of 5%. These mixes were called SP5 type mix. The test samples of SP5 mix type were compacted to 5%±1% instead of the conventional 7%±0.5%. The allowed air-void content error for the SP5 mixes was selected to be ±1% (rather than the ±0.5% followed for all SP4 mixes) since for some SP5 mix types, it was not possible to achieve air-void content values less than 5.5%. The original and the modified gradations of Project 1 and Project 2 mixes are given in Figure 3.2 (a) and (b).

5.3.3 Sample preparation

5.3.3.1 Batching and mixing

For sample preparation, aggregates and RAP were batched to meet the final gradation and target air-void content depending on the mix type. Then, batched samples were mixed using the AASHTO T 312-12 procedure. Before mixing, for the HMA mix, aggregates were kept in the oven at 10ºC higher than the mixing temperature while the RAP materials were kept at 110ºC. Asphalt binder was kept at the mixing temperature for 2 hours. After mixing, prepared loose mixtures were kept in the oven for 2 hours at 135ºC to simulate short-term aging (Newcomb et al. 2015). The goal of short-term aging is to simulate the aging and binder absorption that occurs during mixing and storage phases of the production process. For SCB test samples, the loose mixtures were further aged at 95°C for 24 hours to simulate long-term aging (LTA) (Sreedhar and Coleri 2020). The conditioning was carried out in a forced draft oven and mixtures were stirred at regular intervals to ensure uniform aging. After LTA conditioning, mixtures were further kept in the oven at compaction temperature for 2 more hours prior to compaction. The mixing and compaction temperatures of the HMA mixes were obtained from the viscosity versus temperature plots for the binder provided by the plants.

For the WMA mix, a procedure similar to the one described in the previous paragraph was followed for batching. The batched samples and binder were kept in the oven at a temperature lower than the designated temperature for the first part of the study. The WMA mixing and compaction temperatures, admixture dosage, and the mixing procedure of Evotherm P25® with the binder have been discussed in detail in Sections 4.3.3.1, 4.3.3.2, and 4.3.3.3. For the second part of the study, the mixing and compaction temperatures of WMA mixes were kept identical to the HMA mix.

Asphalt mixtures used for HWTT sample production were only short-term aged (no long-term aging) since rutting generally occurs early in the design life. Asphalt mixtures for only SCB samples were long-term aged to simulate the impact of aging (oxidation and volatilization of different components in the asphalt binder) on long-term cracking resistance.

5.3.3.2 Compaction by roller compactor

The loose asphalt mixtures were compacted in custom 260 mm x 400 mm x 60 mm (10.2 in x 15.7 in x 2.36 in) compaction molds using a Hydraulic Roller Compactor (HRC). Figure 5.2 shows the entire test sample preparation process of the roller compacted samples from compaction to coring. Prior to compaction, the pre-weighed pans of loose asphalt mixtures were placed in the oven at compaction temperature for 2 hours along with the compaction mold. The mold was removed from the oven and a thin layer of grease was applied to all interior surfaces of the mold to prevent the asphalt from sticking to the mold. The heated asphalt mixture was then loaded into the mold and spread evenly throughout the mold (Figure 5.2a). The loaded mold was then placed into the roller compactor and secured (Figure 5.2b).

Parchment paper was placed between the roller surface and the asphalt mixture to avoid any asphalt material sticking to the roller. Mold dimensions were entered into the software controlling the roller compactor. A target number of passes was selected to facilitate full compaction of the mixture to the target air void content. The compaction was performed by applying pressure to the asphalt using an adjustable dial on the roller compactor until the sample was compacted to the specified height. The target number of passes was selected as 25 (significantly higher than the required number passes to achieve compaction) and the maximum pressure that was applied to each sample was 1,000 psi. Although the target number of passes was fixed at 25, the samples were expected to be compacted up to the desired air void level and the input specimen thickness in fewer number of passes. This observation was based on a number of trials conducted in the laboratory before starting the compaction for the actual test specimens. Thus, the actual total number of passes required to achieve the desired compaction was recorded as the one after which the pressure dial dipped from 1,000 psi to 0 psi and remained at 0 until the end of 25 passes. This actual total number of passes served as the measure of compactibility (number of passes required to achieve the required density level) of the different types of mixes.

Once compacted, the mold was removed from the roller compactor and allowed to cool until the internal temperature of the sample fell below the softening point of the asphalt binder used in the mixture. This was done to ensure that the sample would not unravel when removed from the mold. The mold was disassembled after the sample had cooled sufficiently (below 40 °C (104 °F)). Prepared block samples were placed aside for cutting and coring.

Block samples were allowed to rest at room temperature for two weeks prior to coring. The direction of compaction was marked on each block sample prior to coring to allow for consistency in HWTT and SCB testing. Block samples were cored on a stationary core drill using a six-inch (152.4 mm) core drill bit. In order to fit the blocks into the core drill jig, the block samples were cut in half along the midpoint of the sample (Figure 5.2c). Six-inch diameter cores were then removed from the block samples using a stationary core drill (Figure 5.2d and e). Core samples were then allowed to dry at room temperature for at least three days prior to testing.

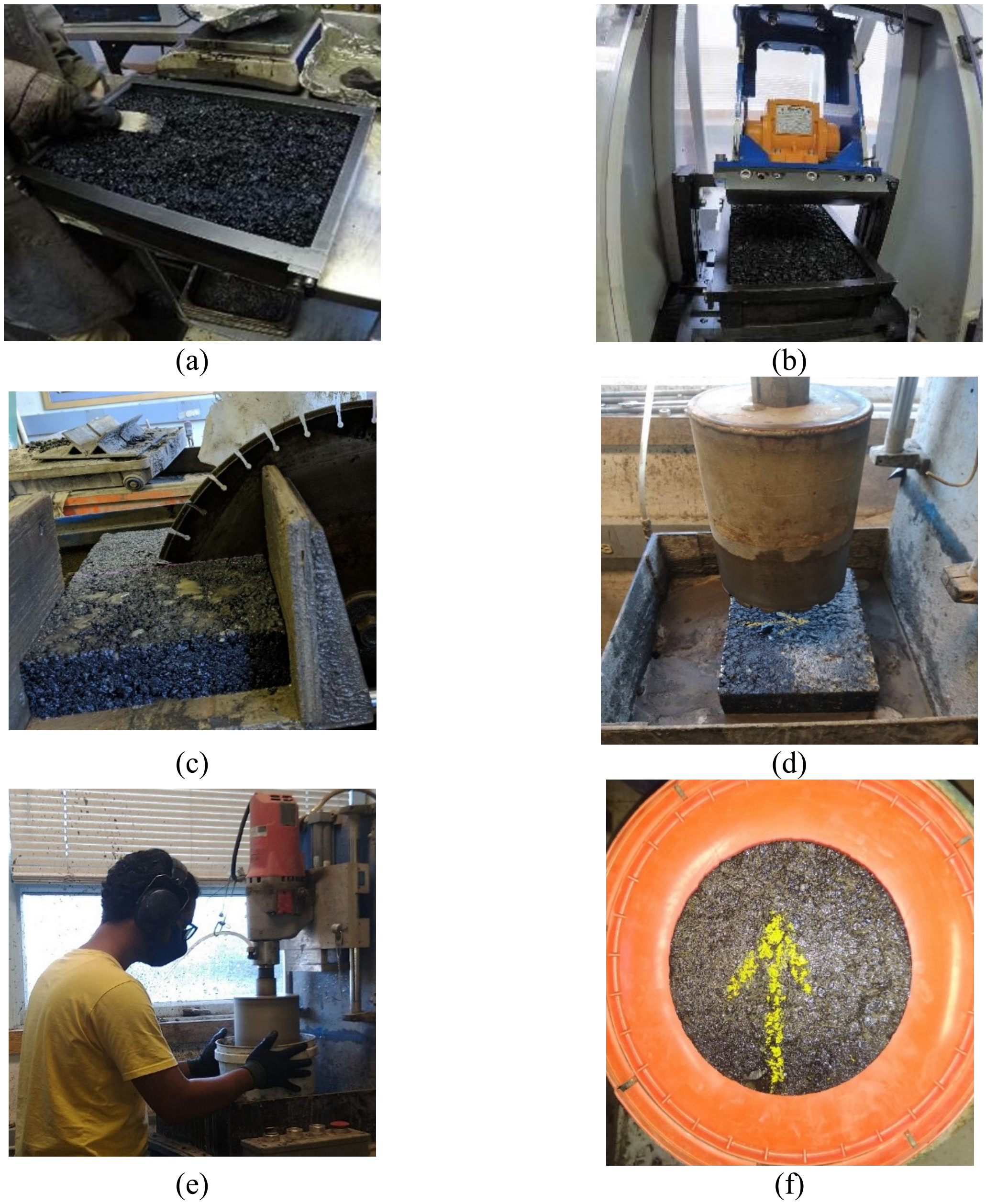


Figure 5.2. Sample preparation process of the roller compacted samples

For the HWTT test, the core samples were further trimmed in a high-precision saw to be able to fit them into the test mold. This process is illustrated in Figure 5.3. For the SCB tests, the circular core samples were cut into two identical halves and a notch was introduced in the middle of the test sample. The sample thickness of all the test samples was 60 mm.

(a) **(b)**

Figure 5.3. HWTT test sample preparation and the mold

5.3.4 Test Methods

To quantify the cracking and rutting resistance of asphalt mixtures evaluated in this study, SCB and HWTT test methods were followed. Both methods are discussed in detail in Section 3.5.2.

5.4 RESULTS AND DISCUSSIONS

5.4.1 SCB Test Results

5.4.1.1 Control – Hot Mix Asphalt

SCB test results for the roller compacted asphalt mixtures are presented in Figure 5.4. FI was calculated and used to evaluate the cracking performance of all asphalt mixtures. The process followed to calculate the FI from laboratory SCB test results is described in Section 3.5.2.1.

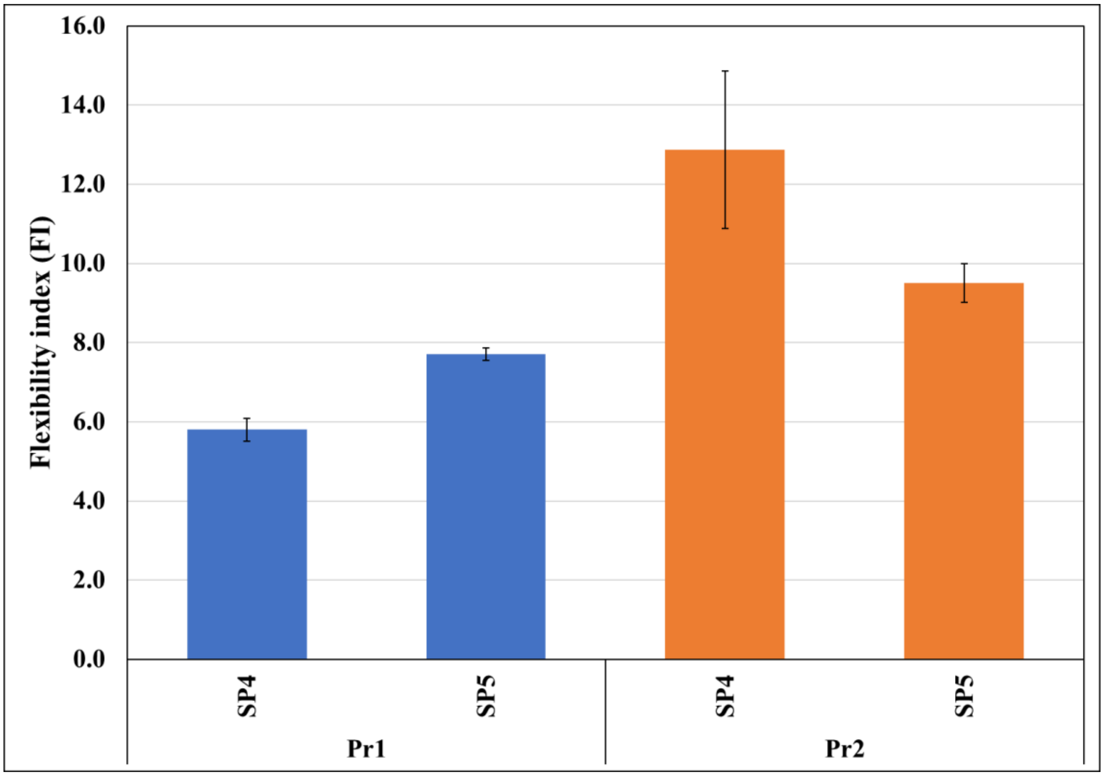


Figure 5.4. Cracking performance of HMA mixes prepared with two different mix design methods, SP4 and SP5 - HRC compaction (length of the error bar is equal to one standard deviation).

It can be observed from Figure 5.4 that for Project 2, the standard SP4 type mix showed higher flexibility index (FI) than the SP5 type mix. The results for Project 2 agree with the results obtained from the SGC test samples in Section 4.4.1. The roller compacted samples, however, yielded higher flexibility indices as compared to the gyratory compacted samples. This is also in agreement with the findings of Coleri et al. (2017). Thus, switching the compaction method from gyratory compactor to roller compactor increased the measured cracking resistance of the mixes but did not alter the performance rankings of the evaluated mix types.

However, for the Project 1 mixes, the SP5 type mix showed higher cracking resistance than the conventional SP4 type mix. This change in the previously observed ranking of cracking performance for SP4 and SP5 type mixtures can be explained by the higher density of the SP5 type mixes. Although the high fine aggregate and dust content of the SP5 mix is reducing the overall cracking performance, increased density of the mix, for this particular case, is dominating and resulting in higher cracking resistance for the SP5 mix. It is important to note that this altered trend in the performance of the SP4 and SP5 type mixes was only observed in the Project 1 HMA mix. For all the subsequent parts of the study and different asphalt mixtures (WMA, HMA with increased binder content, and WMA at standard temperature), the cracking performance ranking of the SP4 and SP5 type mixes remained consistent with the ranking observed in SGC test results (SP4 cracking performance being higher than SP5). It should also be noted that Project 1 SP4 and SP5 mixes were reproduced in the laboratory to validate the trend observed in Figure 5.4 and almost identical results were obtained in the second trial.

5.4.1.2 Warm Mix Asphalt Prepared at Low-WMA Mixing and Compaction Temperatures

Results from the SCB tests conducted with the WMA test specimens prepared at lower mixing and compaction temperatures (at recommended WMA temperatures) are presented in Figure 5.5. It can be observed that the WMA samples prepared with the conventional mix design method (SP4) had FI values that are higher than the samples prepared with the SP5 mix design method. Project 2 mixes were observed to have significantly higher flexibility indices than the Project 1 mixes. Although there are several variables governing this difference in response between the two projects, the primary reason can be attributed to the use of modified binder (PG 70-22ER) in the Project 2 mixtures. By comparing the results in this section to the flexibility indices of the roller compacted HMA control mixtures shown in Figure 5.4, it can be concluded that the WMA samples mixed and compacted at temperatures 35ºC lower than the HMA samples resulted in approximately 100% (almost 2 times higher flexibility for WMA) improvement in the cracking resistance. Similar increase in the flexibility indices for the WMA mixes was also observed in the SGC test samples (see Section 4.4.1.1 and Figure 4.2). However, the percentage differences in FI values of the roller compacted HMA and WMA samples were significantly higher than the gyratory compacted samples.

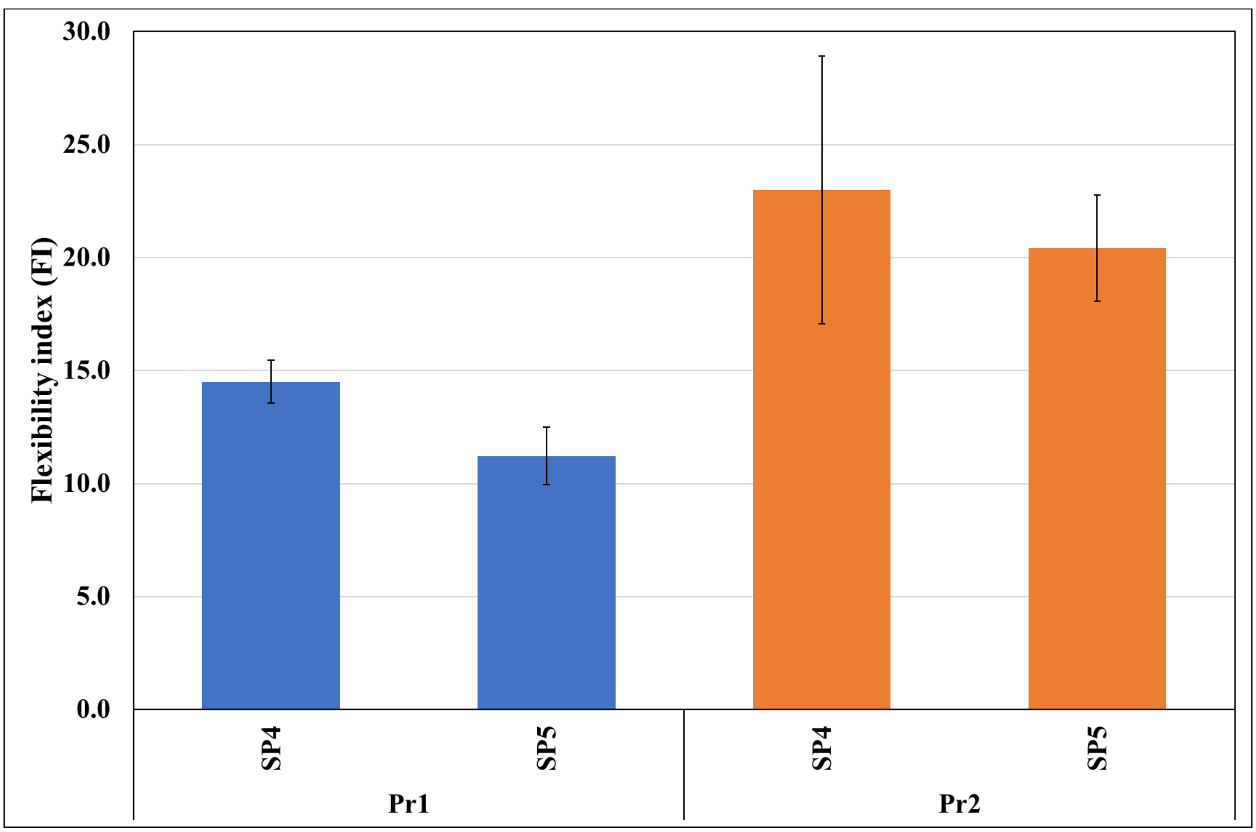


Figure 5.5. SCB test results of WMA specimens mixed and compacted at low-WMA temperatures (length of the error bar is equal to one standard deviation)

5.4.1.3 Warm Mix Asphalt Prepared at High-HMA Mixing and Compaction Temperatures

After observing the positive effects of using WMA mixtures mixed and compacted at low temperatures (at the recommended WMA temperatures) on the cracking resistance of the asphalt mixtures, Project 2 WMA mixtures with the same properties were again mixed and compacted at the standard (higher than the WMA) HMA mixing and compaction temperatures. From Table 4.3, it can be seen that the mixing and compaction temperatures for Project 2 HMA mixtures were 173ºC and 160ºC, respectively. Thus, Project 2 WMA SCB slab specimens were reproduced in the laboratory by mixing the binder (blended with the warm mix additive) and aggregates at 173ºC instead of the recommended 138ºC temperature and conditioning the loose mixture for 2 hours at 160ºC instead of the recommended 125ºC temperature prior to compaction in the roller compacter. The SCB cores were obtained from the roller compacted block samples and were cut into identical halves before introducing a notch.

The SCB test results for the roller compacted WMA samples prepared at higher HMA mixing and compaction temperatures are presented in Figure 5.6. The results of this part of the study were compared with the results obtained from the HMA control mixtures which is shown as HMA (Control) in Figure 5.6. The SP5 type mix had lower flexibility indices than the SP4 type. This conclusion was again consistent with the trend observed so far in this study.

It was also observed that the cracking resistance of the WMA mix mixed and compacted at higher HMA temperatures was almost identical to the cracking resistance of the control HMA mixtures. Most importantly, the cracking resistance of the WMA mix prepared at the standard HMA mixing temperatures was significantly lower than the same mixtures mixed and compacted at lower (recommended WMA) temperatures as shown in Figure 5.5. This loss of cracking resistance in the WMA mix was due to the increased mixing and compaction temperatures. Evotherm P25® is a volatile chemical and based on the results of the SCB tests, it can be concluded that high temperatures nullify the positive effect of this warm mix additive on the cracking resistance of the mix. In addition, the major benefit of WMA admixtures is the ability to achieve required densities in the field without exposing the asphalt binder to higher mixing and compaction temperatures. Increased mixing and compaction temperatures for the WMA mixture might be aging the binder and resulting in significantly lower cracking resistance. For this reason, preparation of WMA mixtures at high temperatures (close to the HMA mixing and compaction temperatures) is not recommended. Moreover, mixing and compacting the WMA at the standard HMA mix removes all the environmental benefits of WMA coming from the reduced energy use during the mix production stage. The impact of mix preparation temperatures on compactibility was also investigated in this study and results were presented in Section 5.4.4.

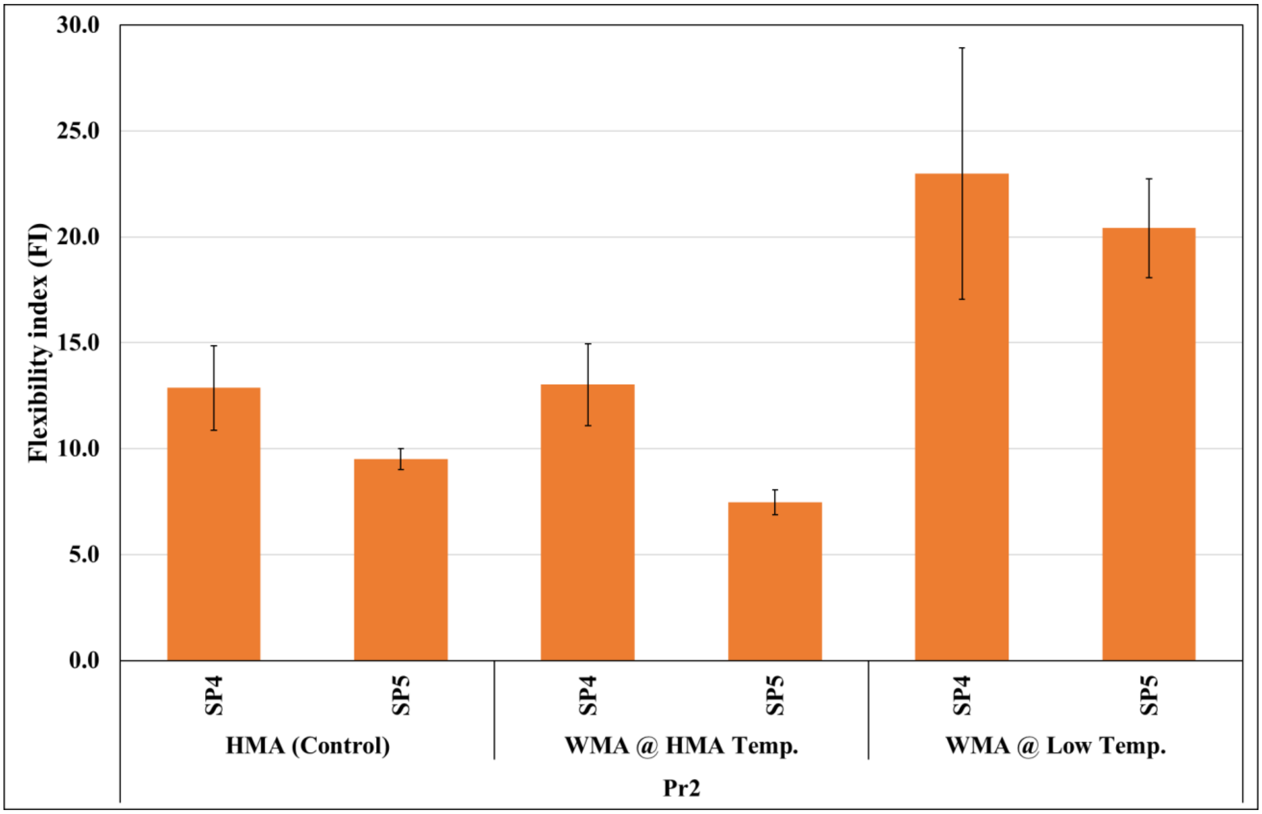


Figure 5.6. SCB test results of WMA mixtures mixed and compacted at high-HMA mixing temperatures (length of the error bar is equal to one standard deviation)

5.4.1.4 Hot Mix Asphalt with Higher Binder Content (+0.5% BC)

In this part of the study, Project 2 HMA mixtures identical to the control mixtures were prepared by only changing the binder content of the mix. The binder contents (BC) of the control SP4 and SP5 type HMA mixtures were increased by 0.5% by weight. The new binder contents of the Project 2 mixes were 6.4% and 6.3% for SP4 and SP5 type mixes, respectively. The SCB test results are outlined in Figure 5.7. From the results, it can be observed that the average FI values for both SP4 and SP5 type mixes are significantly higher than the respective average FI of the control mixtures. This increase in the cracking resistance of the asphalt mixtures was expected due to higher binder content of the mix compared to the control mix.

Comparing the SCB test results of high binder content HMA mix with the WMA mix prepared at low temperatures yielded an interesting finding shown in Figure 5.8. The two mixes had same gradation, aggregate type, binder grade and percent RAP but one mix had Evotherm P25® blended into the binder and was mixed and compacted at significantly lower temperatures than the other mix. Moreover, the other mix had 0.5% higher binder content by weight than the Evotherm P25® mix (WMA). Figure 5.8 indicates that WMA samples show cracking resistance comparable to the HMA samples with 0.5% higher binder content. Although, for the SP4 mix design, HMA mix with higher binder content (BC) had approximately 25% higher average flexibility index than the WMA-low temperature mix, for SP5 mix design the indices were almost identical. Thus, WMA presents a promising alternative for improving the cracking resistance of asphalt mixture in Oregon without increasing the binder content. There are also additional advantages of using WMA mixes and lowering the plant mixing temperatures. The use of WMA additives has the potential of reducing the overall cost of the mix, making the process more environmentally friendly by reducing the GreenHouse Gas (GHG) emissions, and also making the construction process more comfortable for the construction workers.

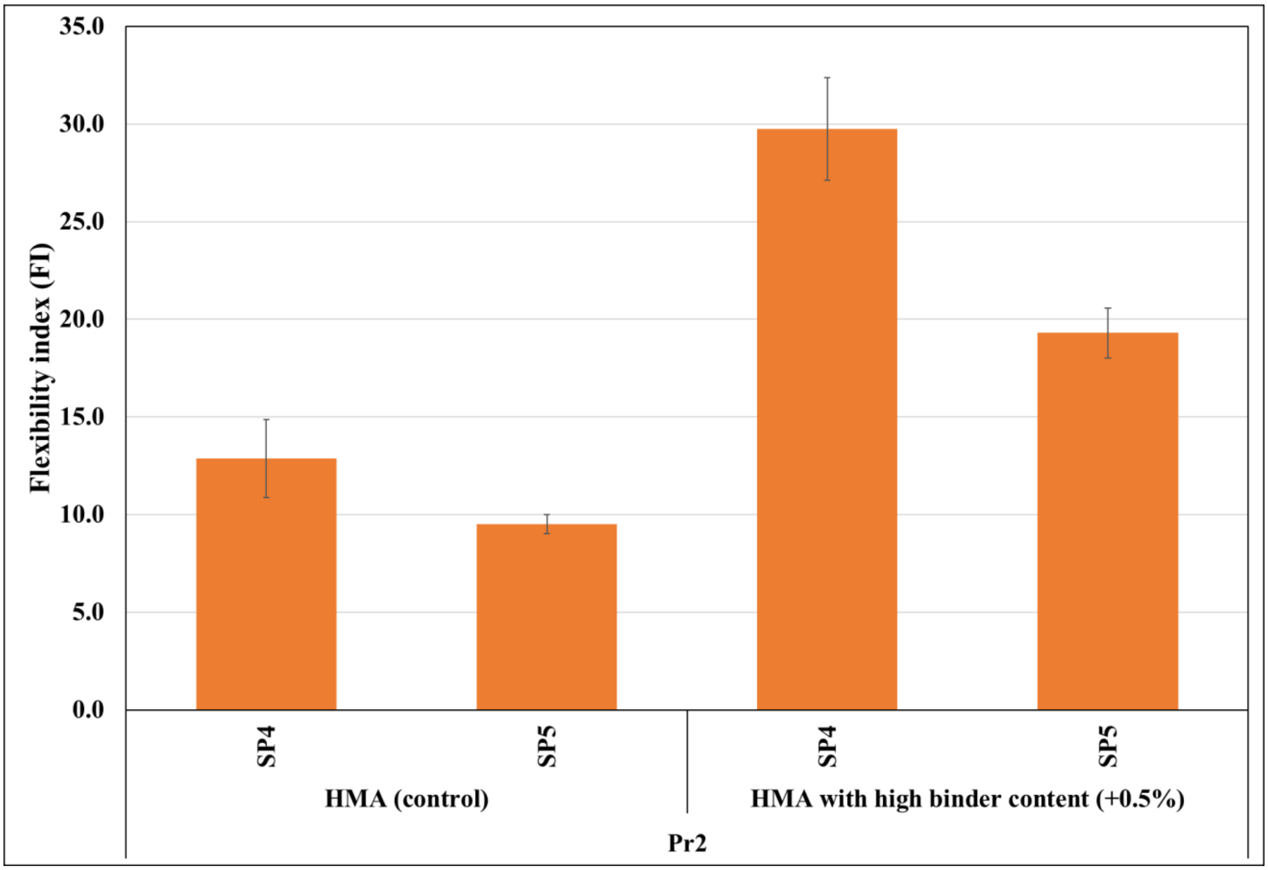


Figure 5.7. The impact of increasing binder content on cracking resistance (length of the error bar is equal to one standard deviation)

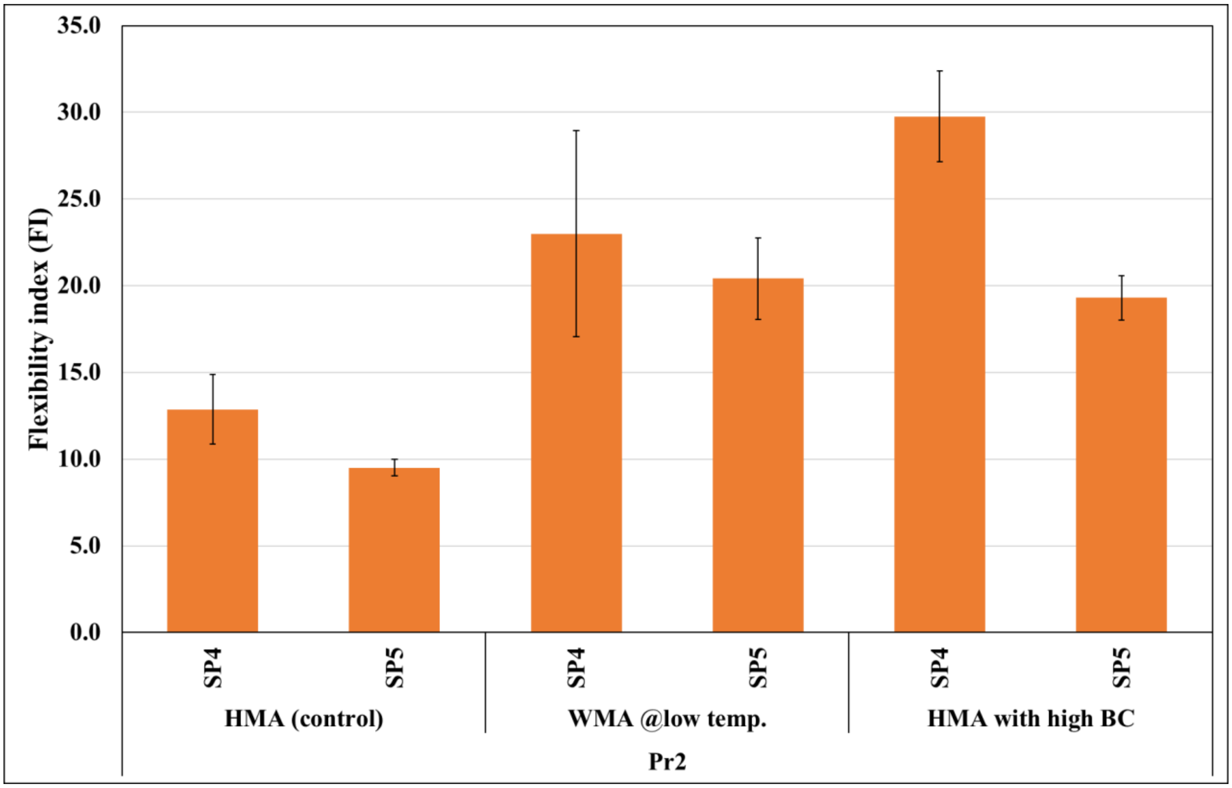


Figure 5.8. Cracking resistance of HMA mixtures with increased binder content compared with the WMA mixes produced at lower WMA mixing temperatures (length of the error bar is equal to one standard deviation)

5.4.2 HWTT Test Results

5.4.2.1 Control – Hot Mix Asphalt

Roller compacted Project 1 and Project 2 block specimens were cut, cored and trimmed to produce the HWTT test samples. These samples were tested in the Hamburg wheel-tracking device (HWTD). The orientation of the twin-samples in the testing mold was in the direction of compaction marked on the cores as shown in Figure 5.3(b). Thus, the direction of the wheel passes aligned with the direction of compaction.

Test results for rutting obtained from the HWTD are presented in Figure 5.9. Respective SP4 and SP5 type mixes of both Project 1 and Project 2 showed similar average rut depths contrary to the same specimens produced by using the SGC (see Figure 3.8). The effect of modified binder on rutting performance of Project 2 mix is also not as evident as the SCB test results. In other words, the rutting resistance of the roller compacted specimens are less affected by the binder type as per the HWTT results. However, this result needs to be verified by testing identical mixes with different binder types. In this study, the results might be getting affected by the presence of confounding variables such as aggregate type, binder content and other volumetric properties.

The comparison of the HWTT test results based on compaction methods suggested that measured rut depths for the SGC specimens were also significantly lower than the roller compactor prepared specimens. The cores obtained from the slab specimens exhibit higher average rut depths in the range of 4mm to 5 mm against the range of 2 mm to 3 mm for the SGC laboratory cores. This conclusion is a result of the aggressive-high energy compaction created by the SGC. Although the laboratory roller compactor is known to provide a better simulation of field compaction by directly simulating the actual field roller compaction process, these results should be compared and validated with the HWTT test results of field cores in a future research study in order to further confirm this conclusion.

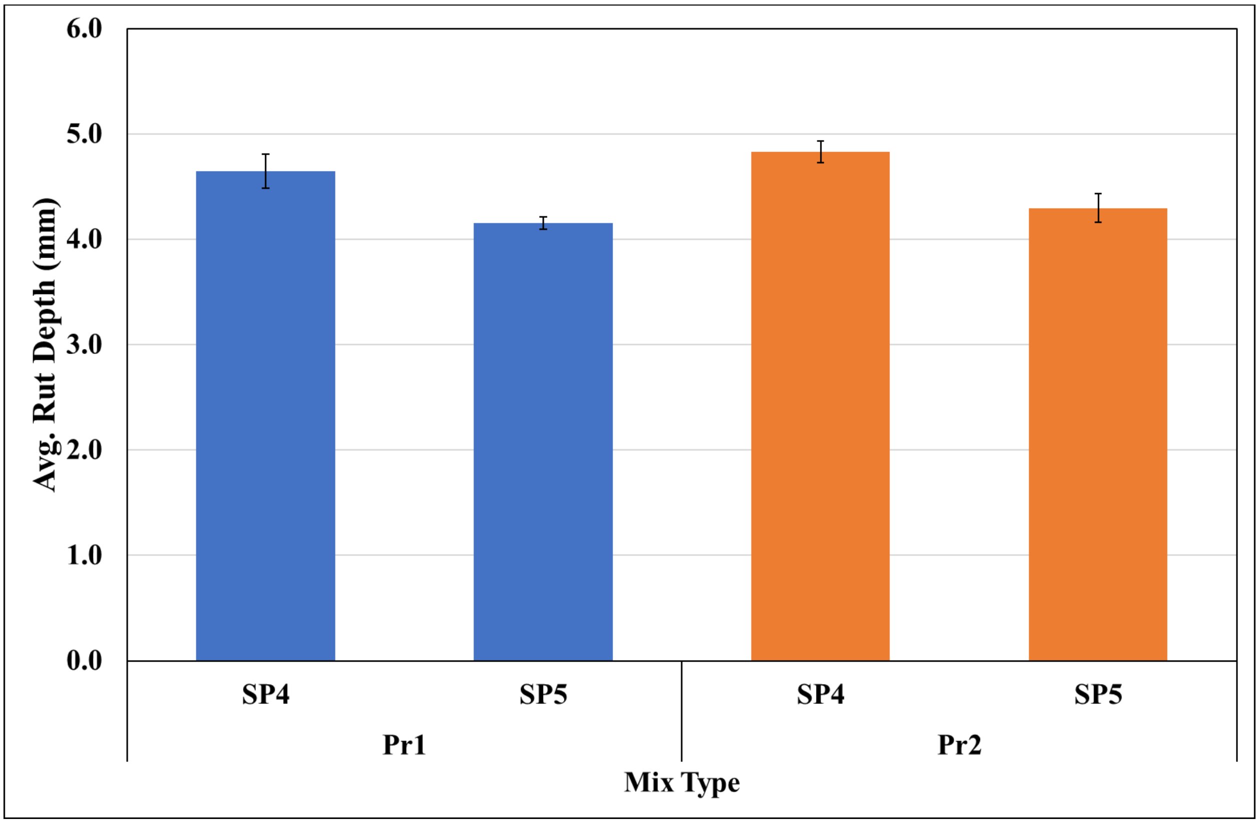


Figure 5.9. Rutting resistance of roller compacted HMA cores prepared with two different design methods, SP4 and SP5 (length of the error bar is equal to one standard deviation).

5.4.2.2 Warm Mix Asphalt Prepared at Low-WMA Mixing and Compaction Temperatures

The results of the Hamburg wheel tracking tests conducted with the roller compacted WMA specimens are presented in Figure 5.10. Similar to the results of the SGC samples, the average rut depths of the roller compacted WMA samples were higher than the HMA samples. It was suspected that either the chemical warm mix asphalt additive made the mix softer or the lower mixing or compaction temperature resulted in less aging/stiffening of the asphalt binder in the mixes (or both). However, as noted in the case of SGC samples in Section 4.4.2.1 (see Figure 4.4), rutting resistance of the WMA samples was not significantly lower than the HMA samples also for the roller compacted samples. The maximum difference in the average rut depths between the WMA and HMA samples was about 1.97 mm and the overall average rut depth values for the WMA mixes were less than 7 mm. These rut depths are higher than the recommended rut depth thresholds of 3 mm (for Level 3 mix) and 2.5 mm (for Level 4 mix) by Coleri et al. (2020). However, the recommendation in Coleri et al. (2020) was based on the SGC laboratory samples (not the roller compacted specimens). In addition, measured rut depths for the WMA mixtures are still significantly lower than the 12.5mm threshold specified by several agencies.

SP5 mixes were also observed to have higher rutting resistances than the SP4 mixes. The main reason behind this behavior is that the SP5 mixtures have denser gradation, lower effective binder content, and lower air void content and this is consistent with the results from the SGC samples (discussed in Section 4.4.2.1).

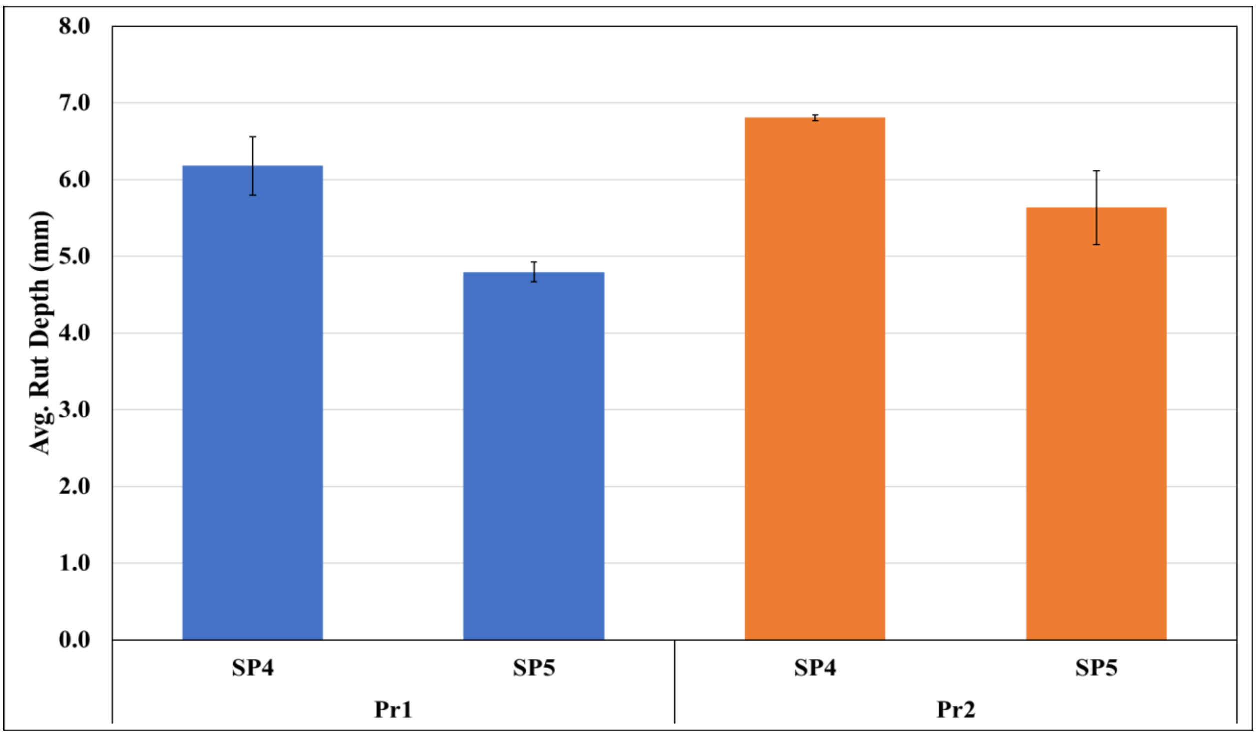


Figure 5.10. HWTT test results for the roller compacted WMA specimens mixed and compacted at low (WMA recommended) temperatures

5.4.2.3 Warm Mix Asphalt Prepared at High-HMA Mixing and Compaction Temperatures

To gain an understanding of the impact of mixing and compaction temperatures on the roller compacted WMA mixtures, Project 2 HWTT samples were reproduced by mixing and compacting at the standard (higher than the WMA) HMA mixing and compaction temperatures given in the production sheets. In Figure 5.11, the average rut depths of the SP4 and SP5 WMA (with low and high mixing temperatures) test samples of Project 2 were compared with the control HMA mix test samples. Results showed that the rutting resistance of the WMA test samples prepared at the HMA temperatures is almost identical to the control mix samples. Increasing the mixing and compaction temperatures of the WMA mix creates a positive impact on the rutting resistance. However, when compared with the SCB test results (see Section 5.4.1.3), the improvement in the rutting resistance is not as significant as the improvement in cracking resistance of WMA test samples prepared at low temperatures (see Figure 5.6). As discussed earlier, mixing, placement and compaction of mixtures with chemical warm mix asphalt additives without lowering the operating temperatures fail to create the added benefits of fuel cost savings, more comfortable paving conditions for the workers, and positive environmental impact. The significant improvement in durability and fatigue life are also other advantages.

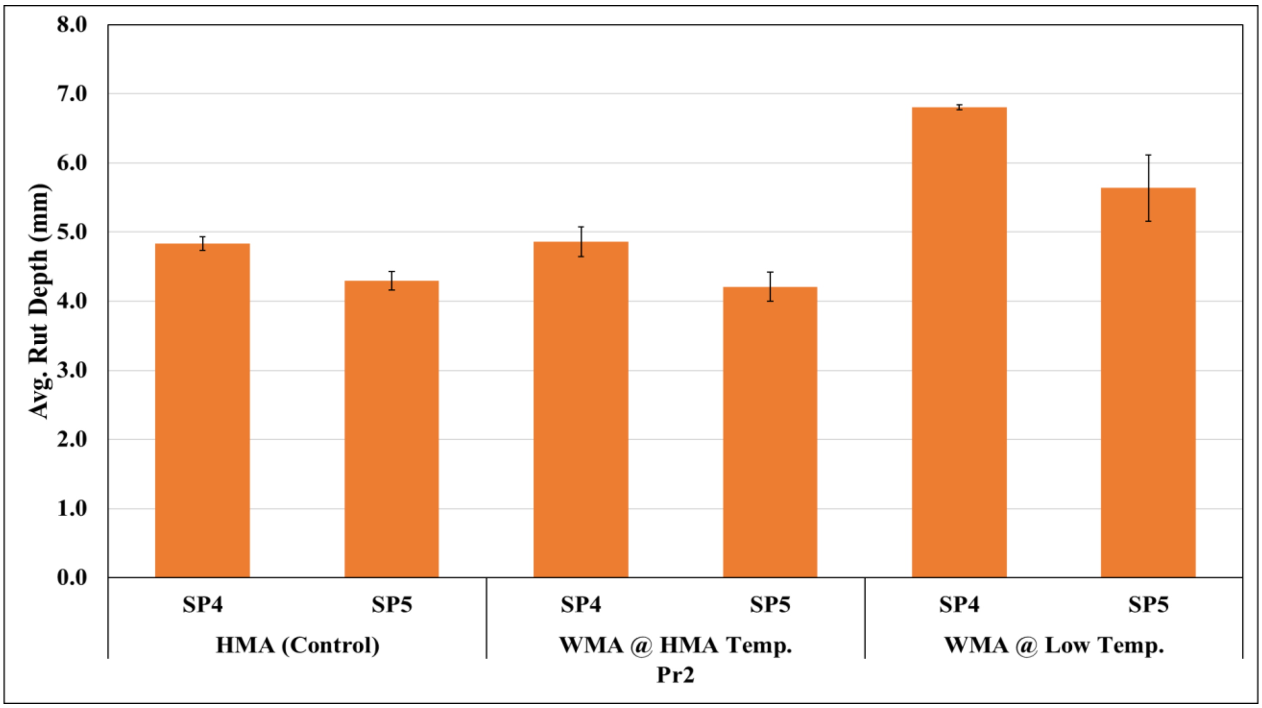


Figure 5.11. Rutting resistance of WMA mixtures mixed and compacted at standard HMA and WMA mixing temperatures compared with the control mixture test results (length of the error bar is equal to one standard deviation)

5.4.2.4 Hot Mix Asphalt with Higher Binder Content (+0.5% BC)

Figure 5.12 presents the results of HWTT tests conducted with the roller compacted Project 2 specimens to determine the impact of high binder content on the rutting resistance of the HMA mixes. The only difference between the control HMA mix and the other mix is that the latter has 0.5% higher binder content by weight. It can be observed from the results that increasing the binder content increases the average rut depths for both mix types (SP4 and SP5), as expected. The average rut depths of the Project 2 SP4 and SP5 type mixes with high binder content were 1.38mm and 0.58mm higher than the corresponding SP4 and SP5 mixes of the control mix. These results also indicated that the increase in the rutting potential of the SP4 mix due to the increased binder content is higher than the SP5 type mix. This can again be due to the combined effect of denser gradation and lower air voids in the SP5 type mix. In other words, SP5 type mix could not be further densified and sheared under the HWTD loading as easily as the SP4 type mix even with the help of the increased binder content. Moreover, increasing the design binder content by 0.5% still did not get the average measured rut depth closer to the 12.5mm threshold currently being followed by several state DOTs. On the other hand, increasing the binder content created a significant improvement in the cracking performance of the mix (see Figure 5.7).

The results of the HWTT tests carried out with the high binder content HMA mixes were also compared with the test results of the WMA samples prepared at low mixing temperatures. Figure 5.13 presents this comparison. It can be observed that the HMA mixes with 0.5% higher binder content performed slightly better in terms of rutting resistance than the WMA mixes with 0.5% less binder. However, the differences in the average rut depths of the two mixtures (0.60 mm and 0.76 mm for SP4 and SP5 type mix, respectively) are not significant. This result is also consistent with the SCB test results (Figure 5.8). One major factor that should be noted while comparing the performance of the two mixes is that the WMA mix achieved the cracking and rutting resistance comparable to the HMA mix with 0.5% higher binder content through a mixing process that requires about 35ºC lower mixing and compaction temperatures than the conventional HMA mixes.

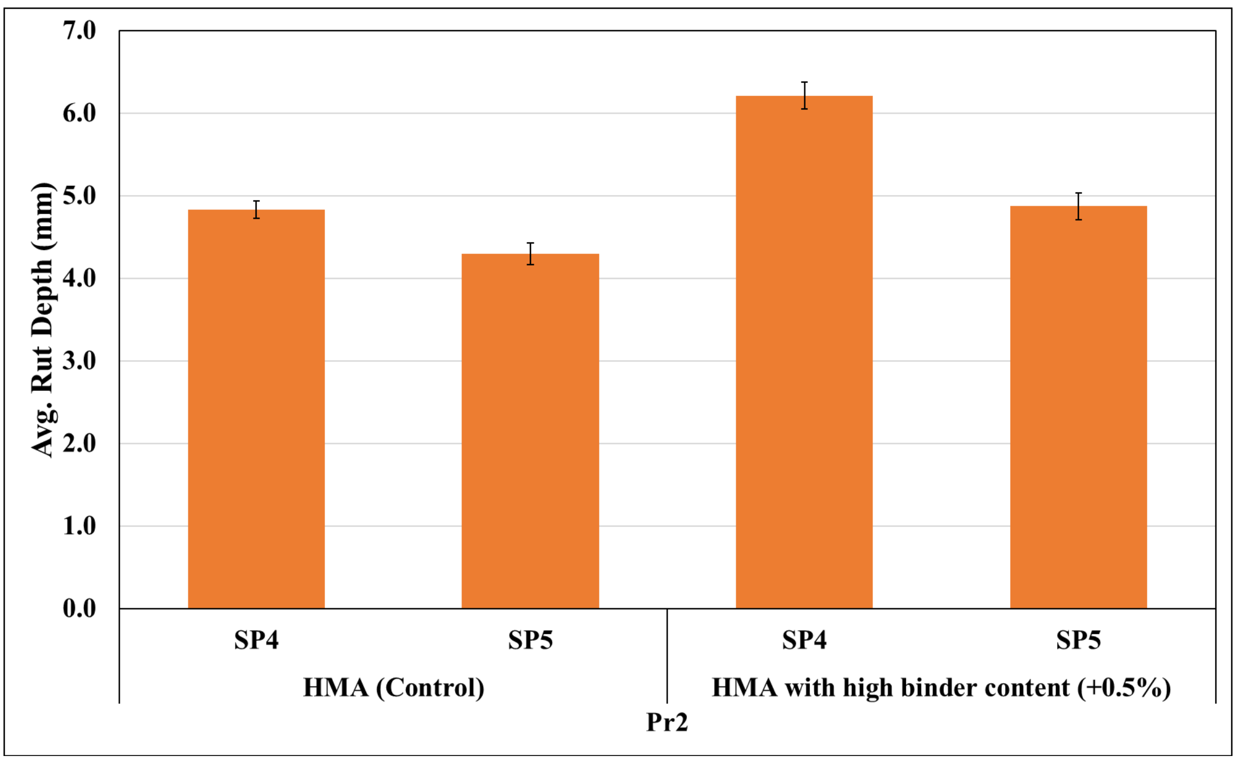


Figure 5.12. Rutting resistance of HMA mixtures with increased binder content compared with the control HMA mixtures (length of the error bar is equal to one standard deviation)

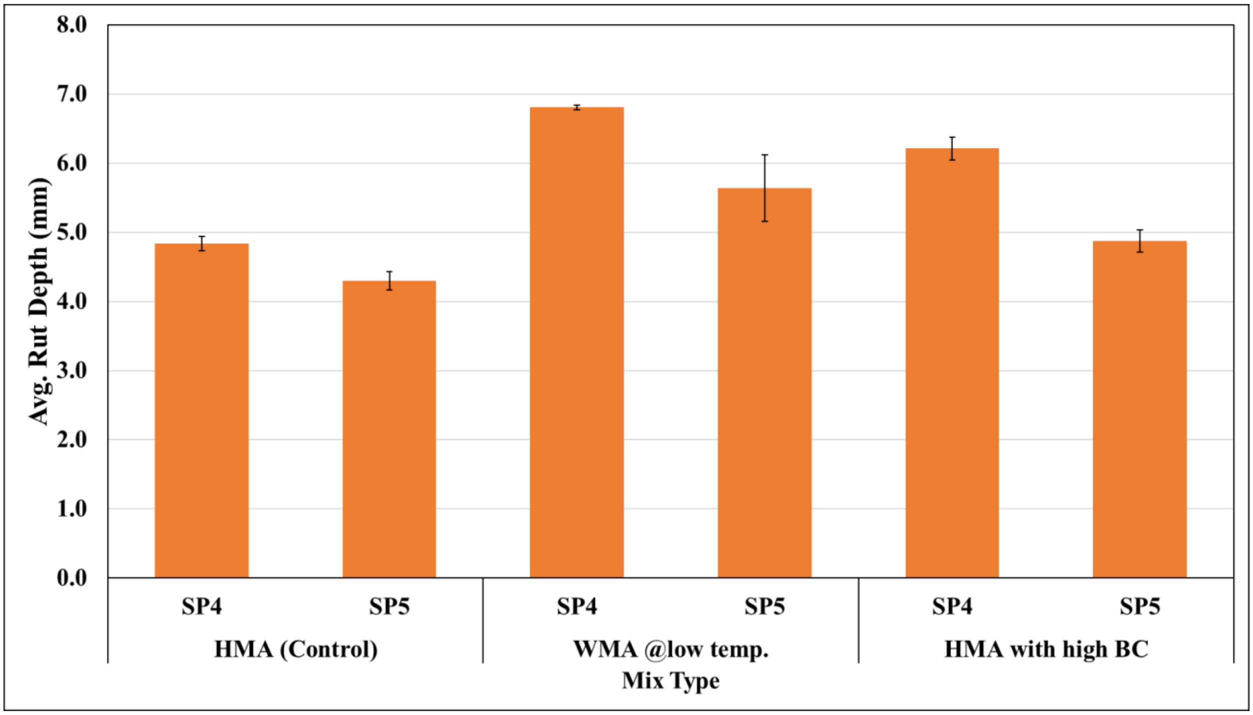


Figure 5.13. Rutting resistance of HMA mixtures with increased binder content compared with the WMA mixes produced at lower WMA mixing temperatures (length of the error bar is equal to one standard deviation)

5.4.3 Cost Comparison

It is evident from the SCB and HWTT test results that the HMA mix with 0.5% higher binder content and WMA mix at low mixing and compaction temperatures show similar cracking and rutting resistance. Although WMA has many other environmental and economic benefits, it is important to compare the cost involved in adopting the two strategies. For this purpose, a cost calculation tool developed by Coleri et al (2018) was used and only the upfront costs were calculated and compared for the two mix strategies. Only the SP4 mix design type was taken into consideration because of the better performance potential. The assumptions, selected costs of materials, and the details of the cost calculation tool are discussed in Section 6.6.4.

Figure 5.14 shows the result based on the output of the cost calculation tool for a one-mile roadway section with a single 12 ft wide lane and 2 inches of compacted asphalt concrete layer thickness. It can be seen from Figure 5.14 that WMA mix amounts to less upfront cost than the HMA mix with 0.5% higher binder content when the burner fuel cost is factored into the cost calculation process (*the process followed for the calculation of the burner fuel costs are summarized in Section 6.6.4*). However, the HMA mix with 0.5% higher binder content results in slightly lower cost than the WMA mix when the plant burner costs were not considered in the calculations. It should be noted that these calculations were just for the upfront raw material costs and life-cycle cost (LCCA) analysis were not performed in this section. More detailed environmental and life-cycle cost analysis for WMA and high RAP mixes were conducted in Section 6.0.

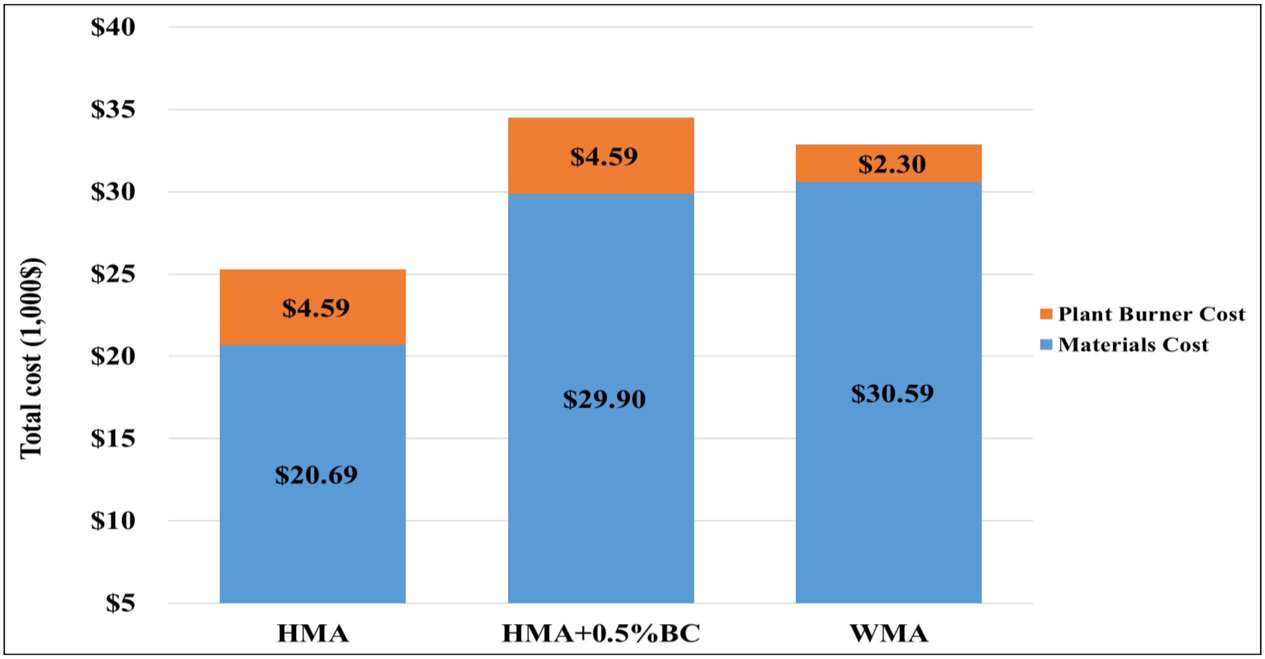


Figure 5.14. Cost comparison for the standard HMA mix, HMA mix with 0.5% higher binder content, and WMA mix at low mixing temperature by only considering raw material and burner fuel costs (costs calculated for a one-mile roadway section with a single 12 ft. wide lane and 2 inches of compacted asphalt concrete layer thickness)

5.4.4 Compactibility Results

The average number of roller compactor passes required to achieve the desired air voids were recorded and results were summarized in Figure 5.15. As discussed in Section 5.3.3.2, the maximum number of passes and the specimen thickness were input to the roller compactor as 25 passes and 60 mm, respectively. Each test sample was compacted by gradually increasing the pressure to 1,000 psi within the first three passes and was then left to be compacted at that pressure until the dial stopped moving indicating that the compaction head had stopped applying pressure. The number of passes to reach this stage (number of passes to reach 60mm specimen height) was recorded and used as a measure of compactibility for different asphalt mixes evaluated in this study. It should also be noted that the compaction was performed with the vibratory rolling option to simulate on-site vibratory roller compactors.

In Figure 5.15, different types of asphalt mixtures that were used in this study are shown on the horizontal axis. These mixtures have been divided into two sections based on the project type (Project 1 and Project 2) and further into two subcategories of mix design type (SP4 and SP5). Average number of passes required to compact each mix category are shown on the primary vertical axis. The four numbers on every “average No. passes” point are the replicate measurements for the number of passes required to compact the samples in Figure 5.15. For SP4 mix design type, the target as-compacted air void content was 7%±0.5% and for SP5 mix design type it was 5%±1%.

The average number of passes to compact the SP4 type mixes was lower than the SP5 type mixes for all the cases except Project 1 HMA mix (control). This was due to the 2% density difference between the SP4 and SP5 mixtures (SP5 having an average 95% density while SP4 was 93%). When the WMA mix was compared with the HMA mix for both projects, the WMA mix was found to be more compactable (lower number of passes required to compact the blocks). The effect of Evotherm P25® on the compactibility of the asphalt mixes was not captured well by the gyratory compactor and some anomalies were seen in the number of gyrations required by the samples to achieve desired specimen thickness and air voids. In the roller compactor, the warm mix additive emerged as a compaction aid for both projects and also for the SP4 and SP5 mix designs. In other words, the asphalt mixtures with Evotherm P25® were compacted to the desired thickness and density in lesser number of passes than the same asphalt mixtures without any additive. This conclusion presents yet another advantage of using WMA mix over the conventional HMA mix other than the other performance, cost, and environmental benefits that were already discussed.

In Project 2, two variations of WMA mixes were used to prepare asphalt blocks. In the first case, the WMA test samples were produced at temperatures 35ºC lower than the standard HMA mixing temperatures (called as WMA@Low WMA Temp). In the second case, the mixing and compaction temperatures were kept identical to the conventional HMA mix (called as WMA@High HMA Temp). By comparing the average number of roller passes for these two cases, it can be observed that for the SP4 mix design, both were almost equally compactable. However, for the SP5 mix design, the WMA test samples prepared at higher HMA mixing temperatures took a greater number of passes to compact than the samples produced at recommended-lower mixing temperatures. The increased effort required to compact the WMA mix at higher HMA mixing temperatures can be attributed to the aging or stiffening of the asphalt binder. Moreover, it is also suspected that Evotherm P25® becomes less effective at mixing temperatures higher than the recommended temperatures due to its volatility.

Project 2 mix with high binder content showed the highest compactibility among all the mixtures as shown in Figure 5.15. Roller compactor system was able to clearly capture the impact of increased binder content on the compactibility of the mixture. Compactibility of the HMA mixture with 0.5% higher binder content was also significantly higher than the compactibility of the WMA mixtures. The reason behind the reduction in the average number of passes required for compaction of the mixes with high binder content is the reduced friction between the aggregates and within the asphalt mixture microstructure. Improved compactibility is expected to result in asphalt concrete layers with higher densities in the field. The improved density is directly correlated with the improved cracking resistance of the mix as discussed in Section 5.4.1.4. Thus, increasing the binder content is an effective strategy (although it increases the upfront cost of paving) to improve compactibility, density and durability of the asphalt mixtures. However, the use of chemical warm mix additives in the asphalt mixtures also provided similar results and can be considered as another cost-effective strategy similar to the increased binder content strategy. Environmental benefits of WMA strategies should also be considered before developing any long-term paving strategies. However, these strategies should be evaluated in a field trial (pilot sections with and without these strategies) to determine the actual field performance of WMA and HMA with increased binder content mixes.

Chapter 6.0 presents the research conducted in this study to quantify the environmental, performance, and economic benefits of using WMA in Oregon.

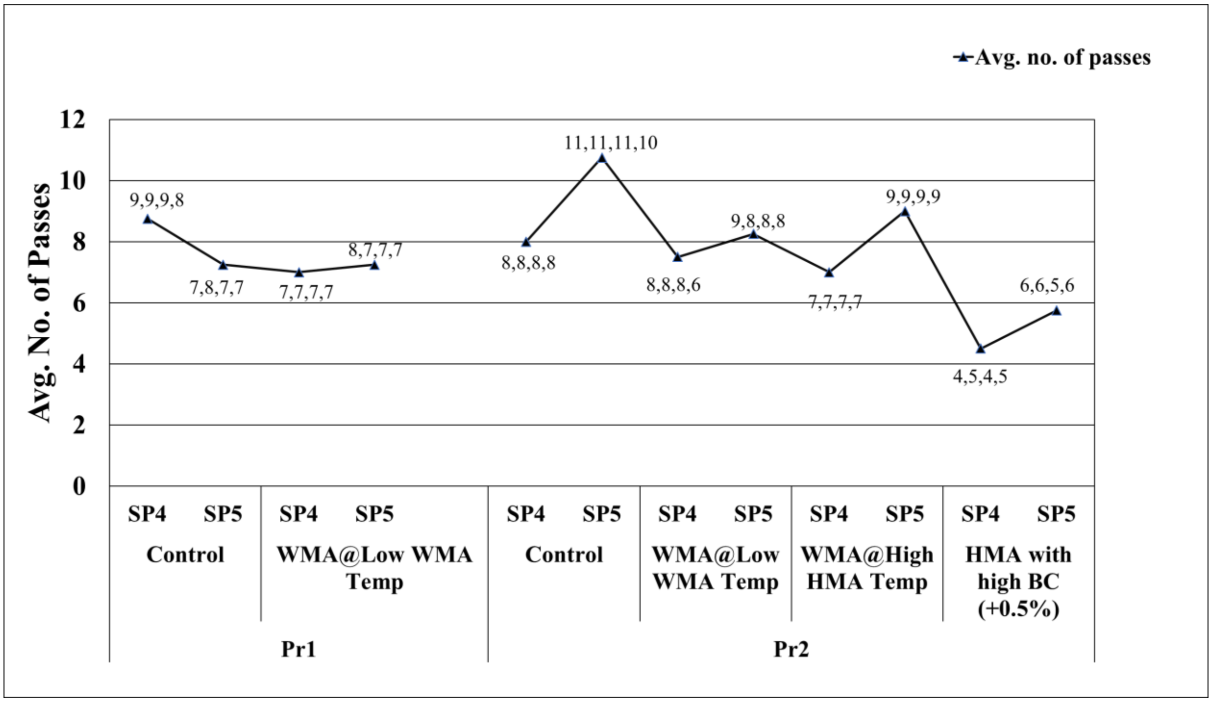


Figure 5.15. Average number of passes as the measure of compactibility for the test specimens produced from different mixes and mix design methods (the four numbers on every "average No. passes" point are the replicate measurements for the number of passes required to compact the samples)

5.5 CONCLUSIONS

In this study, all the test samples were cored from the roller compacted slabs in the laboratory. Two different mixes designed by following two different methods (SP4 and SP5) described in Section 3.6 with Project 1 and Project 2 materials (Table 3.1) were reproduced in the laboratory. The study was divided into four parts: 1) HMA test samples were prepared and tested as the control mix and the WMA test samples were prepared at lower temperatures (WMA@Low WMA Temp), 2) Project 2 mix was used to reproduce WMA mix test samples at higher mixing and compaction temperatures (WMA@High HMA Temp), and 3) HMA test samples with increased binder content (+0.5%) were reproduced using the Project 2 mix. The samples were tested for cracking and rutting resistance. Compactibility of the mixes was also quantified based on the number of passes required to compact the specimen in a hydraulic roller compactor. The results were also compared with the corresponding gyratory compacted test samples.

The conclusions derived from this part of the study are as follows:

Semi-Circular Bend (SCB) Test:

1. For all the mixes, the roller compacted samples yielded higher flexibility indices as compared to the gyratory compacted samples.

2. Switching the compaction method from gyratory compactor to roller compactor increased the measured cracking resistance of the mixes but did not change the ranking of the mix types based on performance. All test samples prepared by the standard-SP4 mix design method showed higher cracking resistance than the test samples prepared by using the Superpave5 (SP5) mix design method.

3. WMA samples mixed and compacted at temperatures 35ºC lower than the HMA samples resulted in approximately 100% higher cracking resistance than the control HMA mix without warm mix additives.

4. Increasing the mixing and compaction temperatures of the WMA test samples to the level of standard HMA mixes significantly lowered the cracking resistance. High mixing and compaction temperatures eliminate the positive impact of Evotherm P25® additive on the mix.

5. Increasing the binder content of HMA control mix by 0.5% by weight considerably improved its cracking resistance.

6. WMA samples showed cracking resistance comparable to the HMA samples with 0.5% higher binder content. The use of WMA additives has the potential of reducing the overall cost of the mix, making the process more environmentally friendly by reducing the Green House Gas (GHG) emissions, and also making the construction process more comfortable for the construction workers.

7. WMA can be considered as a promising alternative for improving the cracking resistance of asphalt mixtures in Oregon without increasing the binder content.

Hamburg Wheel-Tracking Test (HWTT):

1. For all the mixes, the roller compacted samples exhibited higher average rut depths than the gyratory compacted samples.

2. All test samples prepared by following the standard SP4 mix design method showed higher average rut depths than the test samples prepared by using the SP5 mix design method. This is expected to be a result of the lower air void and binder content of the SP5 mixes.

3. Increasing the mixing and compaction temperatures of the WMA mix creates a positive impact on the rutting resistance. However, when compared with the SCB test results (see Section 5.4.1.3), the improvement in the rutting resistance is not as significant as the improvement in cracking resistance of WMA test samples prepared at low temperatures (see Figure 5.6). The significant improvement in durability and fatigue life are also other advantages. As discussed earlier, mixing, placement and compaction of mixtures with chemical warm mix asphalt additives without lowering the operating temperatures fail to create the added benefits of fuel cost savings, more comfortable paving conditions for the workers, and positive environmental impact.

4. The HMA mixes with 0.5% higher binder content performed slightly better in terms of rutting resistance than the WMA mixes with 0.5% less binder. However, the differences in the average rut depths of the two mixtures (0.60 mm and 0.76 mm for SP4 and SP5 type mix, respectively) are not significant. This result is also consistent with the SCB test results (Figure 5.8). One major factor that should be noted while comparing the performance of the two mixes is that the WMA mix achieved the cracking and rutting resistance comparable to the HMA mix with 0.5% higher binder content through a mixing process that requires about 35°C lower mixing and compaction temperatures than the conventional HMA mixes.

5. WMA mix amounts to less upfront cost than the HMA mix with 0.5% higher binder content when the burner fuel cost is factored into the cost calculation process. However, the HMA mix with 0.5% higher binder content results in slightly lower cost than the WMA mix when the plant burner costs were not considered in the calculations. It should be noted that these calculations were just for the upfront material costs and life-cycle costs (LCCA) analysis were not performed in this section. More detailed environmental and life-cycle cost analysis for WMA and high RAP mixes were conducted in Section 6.0.

Compactibility evaluations:

1. The average number of passes to compact the SP4 type mixes was lower than the SP5 type mixes for all the cases except Project 1 HMA mix (control). This was due to the 2% density difference between the SP4 and SP5 mixtures (SP5 having an average 95% density while SP4 was 93%).

2. When the WMA mix was compared with the HMA mix for both projects, the WMA mix was found to be more compactable (lower number of passes required to compact the blocks).

3. For the SP5 mix design, the WMA test samples prepared at higher HMA mixing temperatures took a greater number of passes to compact than the samples produced at recommended-lower mixing temperatures. The increased effort required to compact the WMA mix at higher HMA mixing temperatures can be attributed to the aging or stiffening of the asphalt binder. Moreover, it is also suspected that Evotherm P25® becomes less effective at mixing temperatures higher than the recommended temperatures due to its volatility.

4. Project 2 mix with high binder content showed the highest compactibility among all the mixtures. The reason behind the reduction in the average number of passes required for compaction of the mixes with high binder content is the reduced friction between the aggregates and within the asphalt mixture microstructure. Improved compactibility is expected to result in asphalt concrete layers with higher densities in the field.

6.0 SELECTION OF DURABLE, ENVIRONMENTALLY FRIENDLY, AND COST-EFFECTIVE ASPHALT MIXTURES FOR OREGON – EFFECTS OF DENSITY, WMA, AND HIGH RAP

6.1 INTRODUCTION

In Oregon, fatigue cracking is the major distress mode for asphalt concrete pavement structures. It is one of the main reasons for large road maintenance and rehabilitation expenditures, as well as reduced user comfort and increased fuel consumption due to high road roughness. The resistance of the pavement to this distress mechanism is dependent upon the ductility of the asphalt pavement mixture. According to the literature, aging of asphalt binder associated with the oxidation of the binder is a major factor controlling the fatigue performance of asphalt mixtures. Increasing the asphalt binder content, using modified binders, and/or using softer binder grades were proved to improve fatigue cracking resistance (Coleri et al. 2017, Coleri et al. 2018). Coleri et al. (2017) showed that binder content of the asphalt mixtures produced with the current volumetric design method can be increased without having rutting failures. The low binder content suggested by the current volumetric design methods results in early fatigue cracking and moisture damage. Increasing density (compactibility) and flexibility by using higher binder contents and/or different types of additives were also recommended to be viable options to improve longevity of Oregon roadway network. To address these issues, Coleri et al. (2020) developed a robust performance-based asphalt mix design method to be able to recommend these strategies for performance improvement. In this study, balanced mix design procedures developed by Coleri et al. (2020) in the SPR801 ODOT research project were followed to design three asphalt mixtures for Oregon roads with high traffic levels (Level 4 mixtures).

The main objectives of this part of the study are to:

- Design three asphalt mixtures with different constituents to determine the most effective strategies for Oregon;
- Evaluate the trial mixes for cracking and rutting performances;
- Determine design binder content range for each mix using the balanced asphalt mix design method developed for Oregon by incorporating performance tests for rutting and cracking into the current volumetric design process (Coleri et al. 2020);
- Determine the cost and environmental impact of all three mixtures by performing life cycle cost and environmental impact analysis; and
- Recommend the "best" asphalt mixture for the given conditions by considering the cost-effectiveness, sustainability and the long-term performance of the mixes.

6.2 RESEARCH APPROACH

The Federal Highway Administration (FHWA) formed an Expert Task Group to develop a Balanced Mix Design (BMD) process (West et al. 2018). The group defines BMD as *"asphalt mix design using performance tests on appropriately conditioned specimens that address multiple modes of distress taking into consideration mix aging, traffic, climate and location within the pavement structure"*. Figure 6.1 illustrates the difference between conventional volumetric mix design and proposed balanced mix design process. In volumetric mix design, an optimum binder content required to achieve 4% air-void content by applying a predetermined compactive effort (number of gyrations in a Superpave Gyratory Compactor) is determined. However, performance properties of asphalt mixtures are not accounted for in the design process. On the other hand, in a balanced mix design process, performance properties of asphalt mixtures are evaluated in addition to volumetric properties. In the example presented in Figure 6.1, the binder content determined by the volumetric process is 5.7%. This binder percentage satisfies the rutting criteria for asphalt mixtures. However, this binder content does not satisfy the cracking performance requirements (flexibility index of 8 from the IFIT test). On the other hand, the balanced mix design approach yields a binder content ranging between 6.2% and 6.7%. Within this range, both cracking and rutting criteria are met.

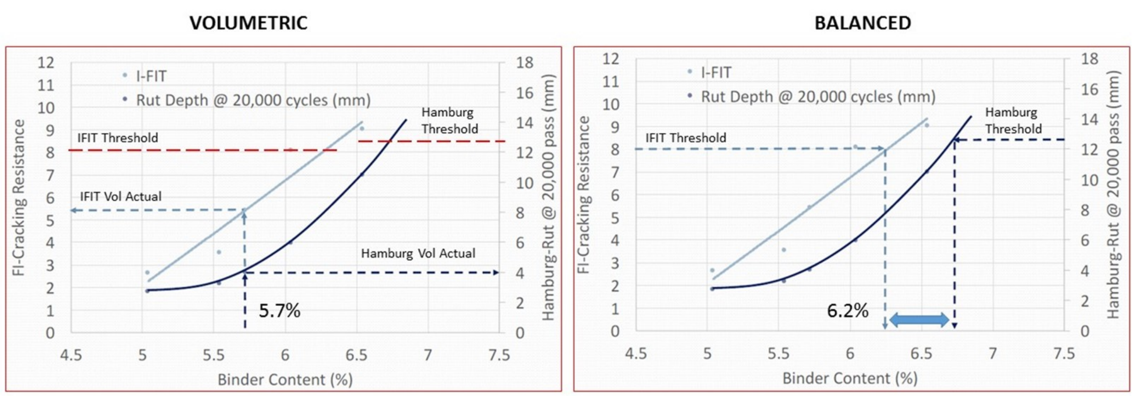


Figure 6.1. Volumetric mix design vs balanced mix design example. (West et al. 2018)

The FHWA group also determined three potential approaches to implement BMD (West et al. 2018), which are briefly described in the SPR801 ODOT research report (Coleri et al. 2020). The BMD approach suggested by SPR801 is summarized below:

Volumetric Design with Performance Verification: This is the most commonly used approach researched and employed by different agencies. In this approach, the mixture is designed based on Superpave specifications. Then, performance tests are conducted to validate whether the mix meets the performance requirements. The mixture should satisfy both volumetric and performance testing criteria. If the mixture does not meet the requirements, the entire mix design process is repeated. The adjustments to the mixture can be made through aggregate source, aggregate gradation, binder source, binder grade, and or additives. This approach is currently being implemented by state department of transportations (DOTs) in Illinois, Texas, Louisiana, New Jersey, and Wisconsin. The process is illustrated in Figure 6.2.

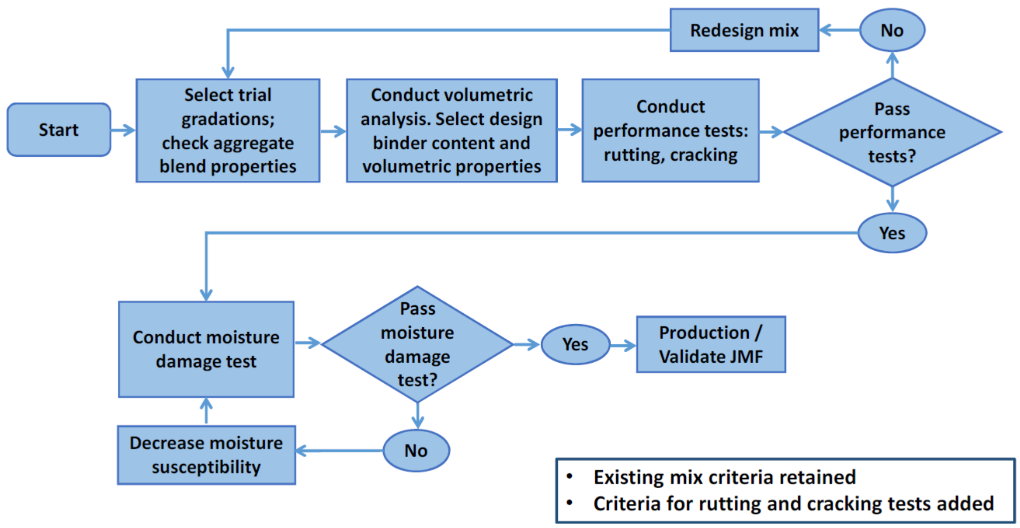


Figure 6.2. Approach 1 - Volumetric design with performance verification. (West et al. 2018)

6.3 BALANCED MIX DESIGN PROCESS RECOMMENDED FOR ODOT – DESIGN APPROACH

The BMD approach proposed by Oregon State University (OSU) in the SPR 801 research project (Coleri et al. 2020) is using volumetric design plus performance testing (as described in the previous section). The motivation behind implementing this approach was to: i) address the performance issues related to the use of higher contents of RAP, ii) increasing binder contents to improve long-term cracking performance; and iii) quantifying the impact of using recently developed additive technologies (warm-mix, fibers, polymer modified binders, etc.) on long-term pavement performance. In the proposed process, binder content is determined by using the Superpave volumetric mixture design process after selecting a suitable aggregate gradation and binder grade.

SCB tests were conducted at 25ºC with a displacement rate of 0.5 mm/min (AASHTO TP 105-13; Coleri et al. 2017). The Flexibility Index (FI) (Ozer et al. 2016) is used to evaluate the cracking performance after long-term conditioning (24 hours of loose mixture aging at 95 ± 2°C, based on the aging protocol that was also developed in the SPR 801 research project), while HWTT is used to evaluate the rutting resistance after only short-term conditioning (two hours of loose mix aging at 132 ± 3°C). HWTT was conducted at 50ºC and the total rut depth (RD) accumulated after 20,000 repetitions was used for rutting performance evaluation. For balanced mix design in Oregon, Coleri et al. (2020) recommended an FI threshold of 6 for Level 3 (for medium ESAL roadway sections) mixes, while the threshold for Level 4 (for high ESAL roadway sections) mixes was selected as 8. A HWTT RD threshold of 3mm was recommended for Level 3 mixes while the threshold for Level 4 was selected as 2.5mm. All designs in this part of the report were developed for Level 4 mixes in Oregon (See ODOT 2019-Table 23). For this reason, an FI threshold of 8 and an RD threshold of 2.5mm were used for balanced mix design (Coleri et al. 2020). However, it should be noted that the 2.5mm rut depth at the 20,000th

repetition in a HWTT tests conducted at 50^{o}C is low. A common rut depth threshold used by many agencies in the U.S. is 12.5mm. If the balanced mix design process is implemented in Oregon in a shadow specification, these thresholds (especially the rut depth thresholds) should be modified within the first years based on the laboratory measured rutting and cracking performances and the actual field performances. In the BMD approach suggested for Oregon in SPR 801, different requirements for binder content adjustments, change in binder source, or reduction in quantities of recycled materials are generally made to achieve the desired mixture performance.

6.4 MATERIALS AND SAMPLE FABRICATION

This section provides information about the materials used in this study (including virgin binders, virgin aggregates and RAP materials). The materials were sampled from the Knife River-Coffee Lake plant in Sherwood, Oregon. In this study, laboratory mixed-laboratory compacted (LMLC) samples were used for testing and evaluation. LMLC is defined as follows:

- *Laboratory Mixed-Laboratory Compacted (LMLC) samples:* Aggregates, virgin binders and RAP material used to produce asphalt mixtures for field construction were sampled from the asphalt plant. These materials were used to produce LMLC samples at the Asphalt Materials Performance Laboratory at Oregon State University.

Three different asphalt mixtures were used in this study (called as Mix1, Mix2, and Mix3). These mixes varied in gradation, amount of RAP content, and presence of additives. Mix1 was further divided into two mixes Mix1_AV5 and Mix1_AV7, differing by the compacted air void contents of the test samples (5% and 7%, respectively) to quantify the impact of density on performance (both designed by the SP4 mix design method with 65 gyrations to compact samples to 96% density via SGC). Mix 2 had 45% RAP content and Mix 3 was identical to Mix 1 except that in Mix3, Evotherm P25® was used as a warm-mix additive. Both Mix2 and Mix3 were compacted to 93 percent theoretical maximum density (±0.5%) in a gyratory compactor to produce test samples with conventional 7% air-void content. In this study, BMD samples were produced with 7% air-void content since 93% density during construction is the expected average density for contractors in Oregon.

Figure 6.3 shows the gradation curves used for the production of the three mixtures. Once the target gradation was finalized, three trial binder contents were selected for mix design. For each binder content, G_{mm} samples were mixed in triplicate according to AASHTO T 312-12 and their respective G_{mm} values were determined as per AASHTO T 209-12 procedures. Subsequently, three replicate mix design samples were prepared for each binder content and compacted in the gyratory compactor by fixing the number of gyrations to 65. It should be noted that the number of gyrations for Level 4 (mixtures for the highest traffic volume) mixes in Oregon is 100. A lower gyration level (65) was selected in this part of the study to determine the impact of reduced gyrations on the mixture performance.

The air-void content for each sample was also determined throughout this study. The binder content corresponding to the target design air void was selected as the optimum binder content (OBC) for each mix. The volumetric and the other mix design variables of the three mixes used in this study are summarized in Table 6.1. The asphalt mixture with 45% RAP has a lower binder content than the other two mixes based on the results of the volumetric mix design. This is expected to be a result of the fine gradation with higher dust content used to prepare those mixes (Figure 6.3). It should be noted that Mix 3 with the WMA additive was not volumetrically designed. The binder content suggested by the volumetric design for Mix 1 was used to prepare asphalt mix test samples for Mix 3. The reason was to clearly identify the impact of using WMA additives on cracking and rutting resistance.

Table 6.1. Mix Design and Volumetric Properties for the Three Trial Mixes

ID [a]	Binder Grade	RAP [b] (%)	AC_{RAP} (%)	AC [c] (%)	P_{be} [d] (%)	P_{200}/P_{be} [e] Ratio	Addi.[f]
Mix1_AV5	PG 70-22ER	30		5.6	4.60	1.4	1% Li[h]
Mix1_AV7		30		5.6	4.60	1.4	1% Li
Mix2		45	5.02	5.3	4.35	1.7	1% Li
Mix3		30		5.6	4.60	1.4	1% Li, 0.68% Evm[i]

ID[a] All mixtures had dense gradation and aggregates with a nominal maximum aggregate size of 12.5mm;
[b] RAP = Reclaimed asphalt pavement added by weight;
[c] AC = Total asphalt content by weight from volumetric design for 65 gyrations;
[d] P_{be} = Effective asphalt content present by weight in the total mix;
[e] P_{200}/P_{be} = Dust to binder ratio in the mix (0.8-1.6 is the range);
[f] Addi. = Additive; [h] Li = Lime; [i] Evm = Evotherm P25® warm mix additive.

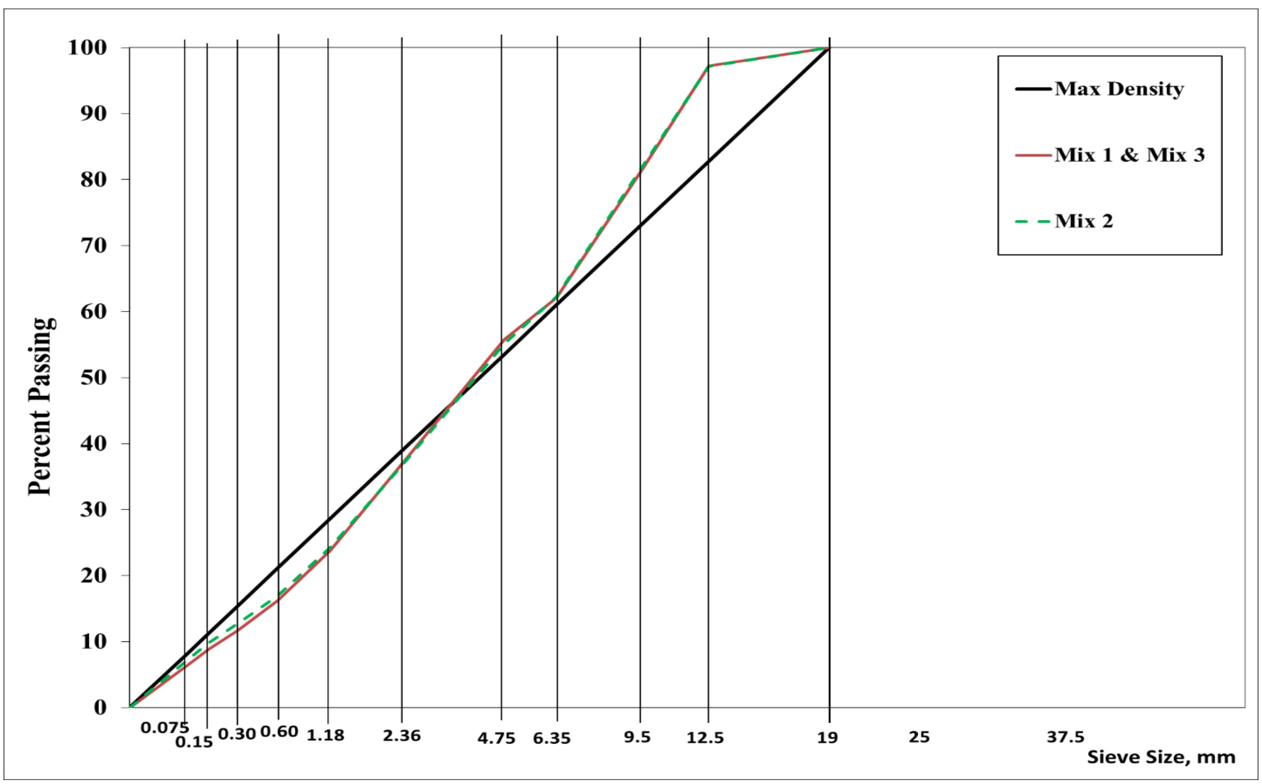


Figure 6.3. Gradation curves for all three mixes on a 0.45 power chart.

6.4.1 Preparation of LMLC Specimens

For sample preparation, aggregates and RAP were batched to meet the final gradation and the 7% ± 0.5% air content for all the mixes (except Mix1_AV5 for which the target air content was 5%± 0.5% to determine the impact of density on performance). Then, batched samples were mixed and compacted by following the AASHTO T 312-12 (2012) specification. Before mixing, aggregates were kept in the oven at 10°C higher than the mixing temperature, RAP materials were kept at 110°C, and binder was kept at the mixing temperature for 2 hours. After mixing, the AASHTO R 30 (2010) recommends conditioning the prepared loose mixtures for 4 hours at 135°C to simulate short-term aging (STA). The goal of short-term aging is to simulate the aging and binder absorption that occurs during the production and silo storage phases. However, based on the suggestions from the NCHRP 815 (Newcomb et al. 2015), a short-term conditioning period of 2 hours at 135°C was adopted (which is also the short-term aging protocol suggested by Coleri et al (2020) for Oregon).

The long-term aging protocol developed for Oregon in SPR 801 research project (Coleri et al. 2020) was followed for conditioning asphalt mixtures for the SCB cracking tests. Based on the results and recommendations from SPR 801, short-term aged loose mixtures were further aged at 95°C for 24 hours to simulate long-term aging. The conditioning was carried out in a forced draft oven and mixtures were stirred at regular intervals to ensure uniform aging. After LTA conditioning, mixtures were further kept in the oven at compaction temperature for 2 more hours prior to compaction. The mixing and compaction temperatures were obtained from viscosity versus temperature plots for the binder provided by the plant. Cylindrical samples were compacted using a Superpave Gyratory Compactor (SGC) in accordance with the AASHTO T312-12 specification. Asphalt mixtures used for HWTT sample production were only short-term aged (no long-term aging) since rutting generally occurs early in the design life. Asphalt mixtures for only SCB samples were long-term aged to simulate the impact of aging (oxidation and volatilization of different components in the asphalt binder) on long-term cracking resistance.

For warm mix asphalt sample preparation, aggregates and RAP were batched following the same guidelines as the hot mix asphalt. Before mixing, binder and the warm mix additive Evotherm P25® were mixed using a counter top stationary mixer (see Figure 4.1). Calculated Evotherm P25® dosages were 0.66%, 0.68%, and 0.71% by weight of total binder for asphalt mixtures with 6.1%, 5.6%, and 5.1% total binder contents, respectively. The chemical additive dosage was calculated according to Equation 4-1 considering the total binder in the mix (virgin binder and binder derived from RAP) and starting from a target Evotherm P25® dosage (in this case it was considered 0.5% by weight of total mix).

For warm mix asphalt, the mixing temperature was 140°C. After mixing, the prepared loose mixtures were conditioned for 2 hours at 135°C. After STA conditioning, the loose mixtures prepared for SCB test sample production were conditioned for an additional 24 hours at 95°C to simulate long-term aging. After conditioning, mixtures were further kept in the oven at a compaction temperature of 126°C for 2 more hours prior to compaction.

6.4.2 Test Methods

To evaluate cracking and rutting performance of the asphalt mixture samples prepared in the study, SCB and HWTT test methods were followed, respectively. Both test methods are discussed in Section 3.5.2.

6.5 EXPERIMENTAL DESIGN

This study was performed to evaluate three different mixes for their cracking and rutting performance and volumetrics. Hamburg Wheel-Tracking Test (HWTT) was selected as the performance test for rutting. SCB test was used to quantify the cracking performance of the asphalt mixtures. General experimental plan followed in this study is given in Table 6.2. A total of 96 laboratory experiments were conducted for the balanced mix design portion of this study. Several additional samples were also prepared for the G_{mm} measurement and volumetric design stages.

Table 6.2. Experimental Plan for Balanced Mix Design.

Specimen Type [a]	Mix ID [b]	Test	Temperature (°C)	Asphalt Content (%)	Replicates	Total
LMLC	Mix1_AV5, Mix1_AV7, Mix3	SCB	25.0	OBC [c], - 0.5%, + 0.5%	4	36
		HWTT	50.0		4	36
	Mix2	SCB	25.0	OBC [c], + 0.5%, + 1%	4	12
		HWTT	50.0		4	12

a LMLC = Laboratory mixed and laboratory compacted;
b Mix1_AV5 – Mix3/ = LMLC samples from three trial mixes as described in Table 6.1.
c OBC = Optimum binder content obtained from volumetric mix design.

6.6 RESULTS AND ANALYSES

The three selected mixes (see Table 6.1) were mixed and compacted to produce test specimens. Target test specimen air-void content was 7%. Binder contents from volumetric design are given in Table 6.1. For Mix1 and Mix3, three different asphalt contents (AC) were used for balanced mix design: AC_{design} from volumetric mix design, AC_{design}-0.5%, AC_{design}+0.5%. For Mix2, AC_{design}-0.5% was too low and could result in a very dry mix (due to high RAP content) and hence the three asphalt contents considered were: AC_{design} from volumetric mix design, AC_{design}+0.5%, and AC_{design}+1%. Four replicate tests were conducted for SCB tests while four replicate tests (four core samples with two rut depth measurements) were conducted for HWTT.

6.6.1 SCB Test Results

Figure 6.4 presents the results of tests for cracking (SCB) performance. FI was calculated and used to evaluate the cracking performance of all asphalt mixtures. The horizontal black line in Figure 6.4 is the FI thresholds selected in this study for Level 4 ($FI_{threshold}$=8) mixtures [determined by Coleri et al. (2020)].

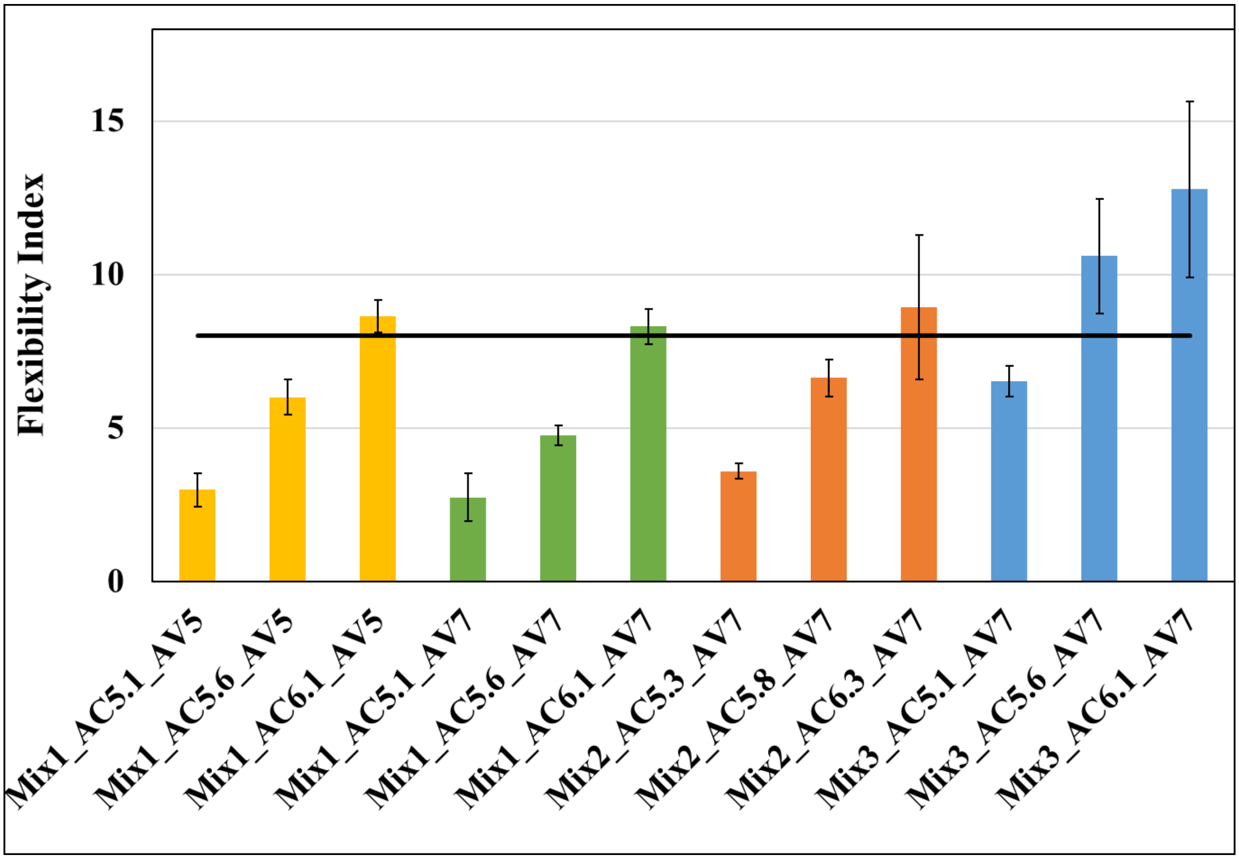


Figure 6.4. FI test results for all mixtures (length of the error bar is equal to one standard deviation).

It can be observed from Figure 6.4 that increasing binder content increases Flexibility Index (FI) for all cases, as expected. FI is able to capture the impact of increased binder content on cracking resistance. It should be noted that all the three mixes were Level 4 mixtures (designed with 65 gyrations).

From Figure 6.4, it can be observed that the average FI values of Mix 3 were significantly higher than that of the other mixes. In Figure 6.4, the first bar for Mix2 and the second bar of the other mixes show the FI value for the LMLC samples prepared at the volumetric design binder content. It can be observed that Mix3 has cracking resistances significantly higher than all other mixtures. Higher cracking resistance for the Mix3 is likely to be a result of the use of a warm mix additive. It is important to mention that the mixtures with warm mix additive are showing better cracking resistance than other corresponding mixes with same or higher binder contents. The FI value for Mix1 with 5% air-void was slightly higher than the same mix with 7% air void. Thus, density of the mix appears to have an effect on the cracking resistance. High RAP mix (Mix2) has better cracking resistance than the low RAP mix (Mix1) but this can be explained by the higher binder content of Mix2 specimens. BMD suggested optimum binder contents (calculated and presented in Section 6.6.3) for 30% and 45% RAP cases should be checked to determine the impact of increased RAP percentage on performance and design binder content.

6.6.2 HWTT Test Results

Figure 6.5 presents the results of HWTT tests conducted to determine the rutting performance of asphalt mixtures. Average surface rut depth after 20,000 wheel passes was used to evaluate the rutting performance of all asphalt mixtures. A mixture with higher rut depth is expected to show lower rutting resistance. The horizontal black line in Figure 6.5 is the HWTT rut depth threshold used in this study for BMD ($RD_{threshold}$=2.5mm for Level 4 mixes determined by Coleri et al. (2020)).

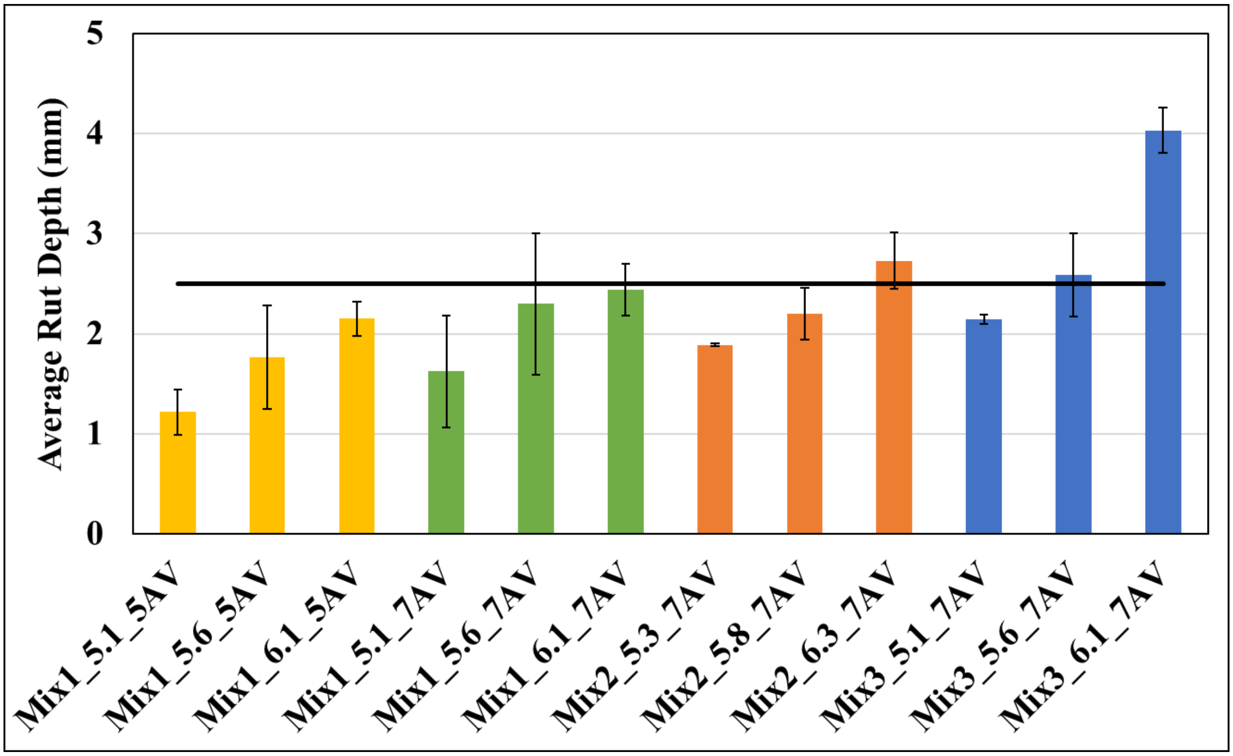


Figure 6.5. HWTT test results for all mixtures (length of the error bar is equal to one standard deviation).

It can be observed from Figure 6.5 that increasing binder content increases rut depth for all the cases, which is expected. In addition, it can be observed that Mix1_AV5 has the best rutting resistance among all the mixes. Samples for only this mixture were compacted at 5% air-void. Higher density (2% higher than 7% air-void samples) resulted in an improved rutting resistance. It is important to note that 2% increase in density resulted in significant improvements in both rutting and cracking performance. Although not simulated in this study, increased density is also expected to reduce long-term aging and moisture susceptibility of the asphalt mixtures due to reduced permeability. It is possible that Mix3 with warm-mix additives can have better "compactibility" due to lower viscosity of the modified asphalt binder. Improved compactibility will result in higher density values with associated long-term performance benefits.

In this study, four replicate asphalt cores were produced for HWTT testing. Since two cores were attached edge-to-edge to run the experiment, a total of two rut depth values were collected from the test system for each case. Increasing replicate test results from two to three is recommended in this study to minimize the impact of high-test results' variability on average measured rut depth. In addition, since HWTT experiments were conducted under water, test results are also affected by the moisture susceptibility of the asphalt mixture in addition to rut resistance. Combined effect of moisture and rut resistance reflected in the test results might be increasing the variability of the test.

Mix3 is showing the highest rut-depth among all three mixes as the warm mix additive and reduced aging during the mix preparation due to the lower mixing temperature is making the mix softer. High RAP mix is showing higher rut depth than the low RAP mix (Mix1) but it should be noted that the high RAP mix also has higher binder content (0.2% more binder for every case).

The average rut depths of all the mixtures were lower than 3.0 mm (except the WMA mix with highest binder content) which is significantly lower than the maximum rut depth allowed by several agencies in the U.S. (which is 12.5mm). This result suggested that all mixes used in this part of the study are on the "dry" side and need more asphalt binder to improve their cracking resistance while still meeting the 12.5mm maximum rut depth requirement.

6.6.3 Balanced Mix Design

Balanced mix design approach helps in determining the binder content range that satisfies both cracking and rutting performance criteria. Minimum binder content is the lowest asphalt binder percentage allowed in the mix to satisfy the FI threshold of 8 for Level 4 mixtures and FI of 6 for Level 3 mixtures in Oregon. Maximum asphalt content is the highest percentage that satisfies the rutting criteria, rut depth of 2.5mm for Level 4 mixtures and 3mm for Level 3 mixtures in Oregon (Coleri et al. 2020). Figure 6.6(a)-(d) depict balanced mix design charts for all the mixes used in this study. Based on the volumetric mix design, Mix1 and Mix3 have an asphalt content of 5.6% and Mix2 has an asphalt content of 5.3%.

From Figure 6.6(a), it can be observed that Mix1 does not meet the cracking and rutting criteria at the design asphalt content. However, with the balanced mix design approach, the minimum asphalt binder content required is about 6% (see Figure 6.6(a)). This increased binder content is expected to significantly increase the cost of the Mix1_AV5 asphalt mixture while still keeping it in the acceptable region for rutting and cracking performance. However, to ensure a high long-term cracking performance, 6.3% asphalt binder content can also be used for production. However, it should be noted that using 6.3% design asphalt content creates a high risk for rutting since plant produced mixtures are allowed to have ±0.5% variability in production binder content in Oregon. ODOT is currently in the process of changing the binder content variability tolerance from ±0.5% to ±0.35%. This change is expected to reduce the risk of rutting or cracking failures due to production binder content variability. However, for practicality purposes and considering the mix costs, this study recommends to use the lower limit obtained from the balanced mix design approach. Similarly, based on the balanced mix design plots for other three mixes, the required asphalt content for Mix1_AV7, Mix2 and Mix3 are 6.05%, 6.10% and 5.30%, respectively. Although there is no binder content range for Mix 1_AV7 (See Figure 6.6b) that satisfies both the rutting and cracking requirements, the upper limit number that satisfies the rutting requirement is selected as the design binder content for balanced mix design.

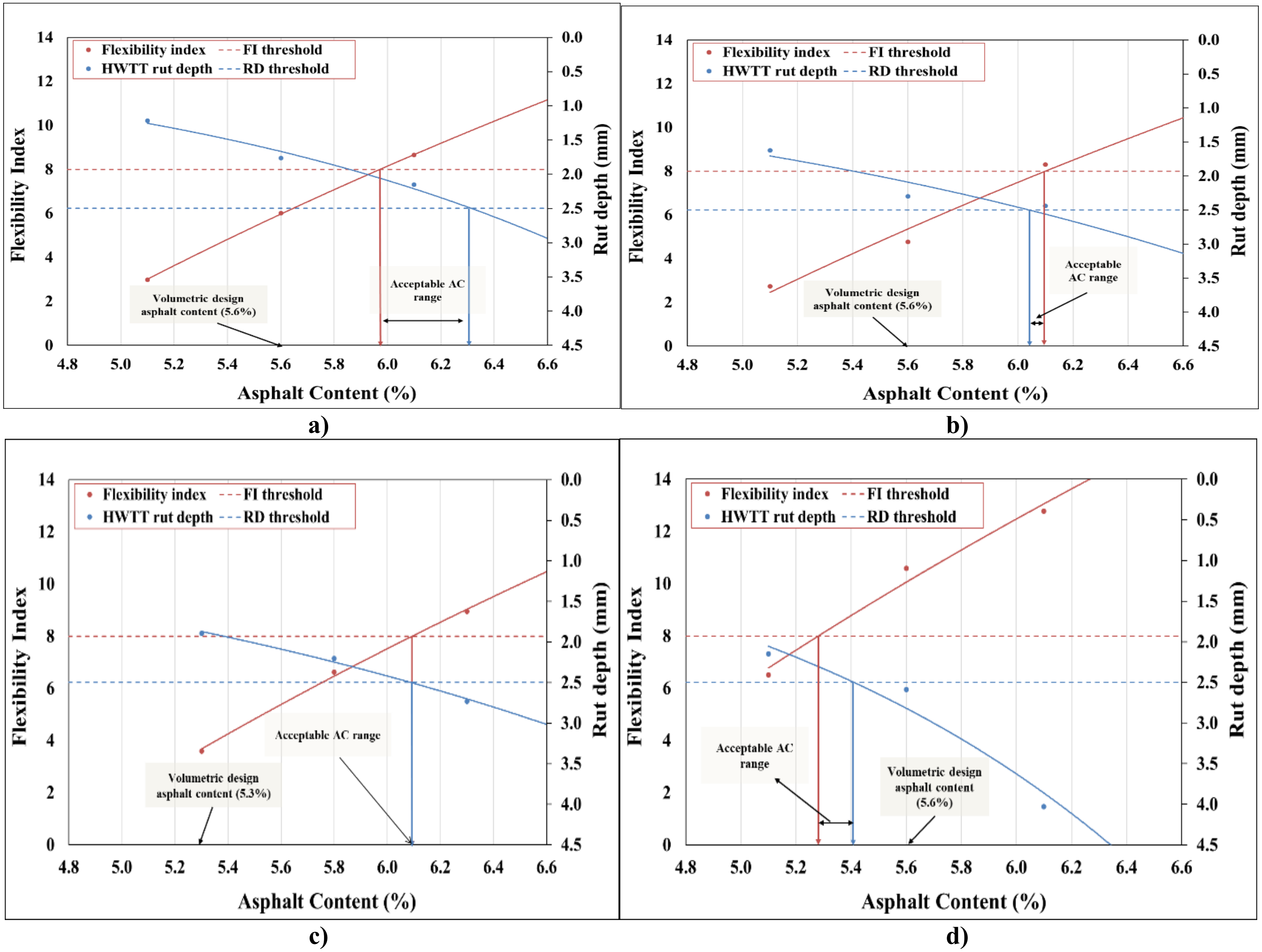


Figure 6.6. Balanced mix design for (a) Mix1_AV5 (b) Mix1_AV7 (c) Mix2 and (d) Mix3.

The asphalt content derived from the above balanced mix design plots and the results of the previously conducted G_{mm} measurements were used to back calculate some of the volumetric properties of the mixes. Results are shown in Table 6.3. It can also be observed by comparing Table 6.1 and Table 6.3 that dust-to-binder ratios (P_{200}/P_{be}) for the mixes designed by balanced mix design are lower than the values for the volumetric mix design (except Mix3 which had a lower binder content than all other mixes due to the presence of WMA additives) although a gyration number (which is 65) significantly smaller than the current standard (which is 100 for Level 4 mixes) was used for the volumetric mix design.

Table 6.3. Volumetric Properties for the Three Mixes Based on BMD Design Binder Content

ID [a]	Binder Grade	RAP [b] (%)	AC_{RAP} (%)	AC [c] (%)	P_{be} [d] (%)	P_{200}/P_{be} [e] Ratio	Addi.[f]
Mix1_AV5	PG 70-22ER	30		6.00	5.12	1.30	1% Li[h]
Mix1_AV7		30		6.05	5.17	1.20	1% Li
Mix2		45	5.02	6.10	5.30	1.40	1% Li
Mix3		30		5.30	4.17	1.50	1% Li, 0.68% Evm[i]

[a] All mixtures had dense gradation and aggregates with a nominal maximum aggregate size of 12.5mm;
[b] RAP = Reclaimed asphalt pavement added by weight;
[c] AC = Design BMD asphalt content added by weight;
[d] P_{be} = Effective asphalt content present by weight in the total mix;
[e] P_{200}/P_{be} = Dust to binder ratio in the mix (0.8-1.6 is the range);
[f] Addi. = Additive; [h] Li = Lime; [i] Evm = Evotherm P25® warm mix additive.

6.6.4 Cost Calculation Tool

The use of RAP in Hot Mix Asphalt (HMA) paving is often considered a cost-saving measure. Although it can make the pavement more susceptible to cracking failure, it is considered a sustainable alternative to asphalt mixtures with all-virgin materials, both in terms of cost and environmental impacts. However, contractors and agencies who are not able to accurately quantify savings brought on by using RAP in HMA mix may be discouraged from using these materials due to the reduction in cracking resistance that they may create. The culmination of these factors yields a necessity for a simple way to analyze different mix design options.

The use of Warm Mix Asphalt (WMA) is also seen as a method of decreasing costs. It is considered to be a sustainable alternative to HMA considering the cost (burner fuel reductions), environment (less CO_2 emissions) and safety (improving the labor conditions for workers). The use of high RAP in WMA can be one of the best solutions for asphalt mixtures.

In this study, a tool created by Coleri et al. (2018) that allows the users to compare the cost of mix design strategies against one another in order to calculate the potential savings they can realize by choosing mix designs with different RAP and RAS contents, as well as different binder types and binder contents was used. This tool is meant to increase incentive for users to use recycled materials in their HMA mixes, thereby increasing the sustainability and cost-effectiveness of asphalt pavement construction. Given the geometry of a pavement section and pertinent material cost data, the contractor and/or agency can evaluate the total estimated cost of implementing a particular mix design strategy for their project.

A screenshot of the tool's input tab is given in Figure 6.7. Figure 6.8 presents the comparisons of all the mixes based on materials and plant burner fuel costs. In order to use the tool, the user must input data from their HMA and WMA mix design, such as target density, binder content and recycled materials content. Input data about the geometry of the pavement section, such as length, lane width, number of lanes and compacted layer thickness, should also be entered. The tool will automatically calculate the volume and weight of asphalt mixture that is anticipated for the target density and pavement section geometry. The user must also input cost data for the materials. The user can input their unit costs for binder, aggregate and recycled materials (RAP). Input fields are shown in orange with blue text and calculated fields are shown in gray with orange text. The total mix cost for the pavement section is shown at the bottom of each mix design spreadsheet in dark gray text. It should be noted that calculated asphalt mixture costs are based on the cost calculations in the spreadsheet by using the raw material costs and do not include any plant operation costs or added profit for the producer. Since 45% RAP is not allowed in Oregon and warm-mix is not commonly used, it was not possible to get exact mixture costs for those alternatives.

The last step is calculating the production burner cost which was not included in the previous calculations. The burner fuel cost can be the key factor in determining whether the HMA or the WMA is the most cost-efficient asphalt mixture. In order to assess the contribution of the production costs, a fuel consumption of 2 gallons of diesel fuel per ton for HMA and 1 gallon of diesel fuel per ton for WMA with chemical additive Evotherm P25® (Sullivan and Moss, 2014) were considered, which means a reduction of 50% burner fuel. Also, a price of $3/gallon diesel fuel for Oregon was used (Statista, 2020). Table 6.4 shows the amount of burner fuel savings for WMA dependent on the additives used.

Table 6.4. Amount of Burner Fuel Savings for WMA (Sullivan and Moss, 2014)

Method	Example Product	Burner Fuel Savings
Chemical Additives	Advera®	1.0 gal/ton (50%)
Organic Additives	Sasobit®	0.7 gal/ton (35%)
Water-Based Foaming	Double-Barrel Green®	0.4 gal/ton (20%)

	A	B	C	D	E	F
1	**RAP & RAS Cost Calculator**					
2	**Mix Design 4**					
3	**Inputs:**					
4	**Product**	**Cost**	**Unit**	**Source**	**Type**	
5	**Binder Type 4**	$ 490.00	ton	ODOT	PG 70-22ER	
6	**RAP**	$ 20.00	ton			
7	**RAS**	$ 40.00	ton			
8	**Aggregate**	$ 13.00	ton			
9						
10	**Segment Property**	**Measure**	**Unit**	**Source**		
11	**Geometry**	Straight	-	Assumption		
12	**Length**	1.0	mi	Assumption		
13	**Lane Width**	12.0	ft	Assumption		
14	**Number of Lanes**	1.0	each	Assumption		
15	**mpacted Layer Thickness**	2.0	in	Assumption		
16						
17	**Mix Property**	**Measure**	**Unit**	**Source**		
18	**Compacted Density**	145.0	lb/ft^3	NAPA website		
19	**Target Binder Content**	6.0%	by weight	Estimate		
20	**RAP Content**	30.0%	by weight	Estimate		
21	**RAS Content**	0.0%	by weight	Estimate		
22	**Aggregate Content**	**64%**	by weight	Calculation		
23	**er Content (RAP material)**	5.0%	by weight	Estimate		
24	**er Content (RAS material)**	0.0%	by weight	Estimate		
25	**Virgin Binder Added**	**4.5%**	by weight	Calculation		
26						
27	**Outputs:**	**Measure**	**Unit**			
28	**Section Volume**	**10560**	ft^3 (all lanes)			
29	**Section Tonneage**	**765.6**	tons (all lanes)			
30	**Mix Cost**	**$ 27,822.36**	**segment**			

Mix1_AV5 | Mix1_AV7 | Mix2_AV7 | Mix3_AV7 | Summary

Figure 6.7: Cost calculation tool input tab

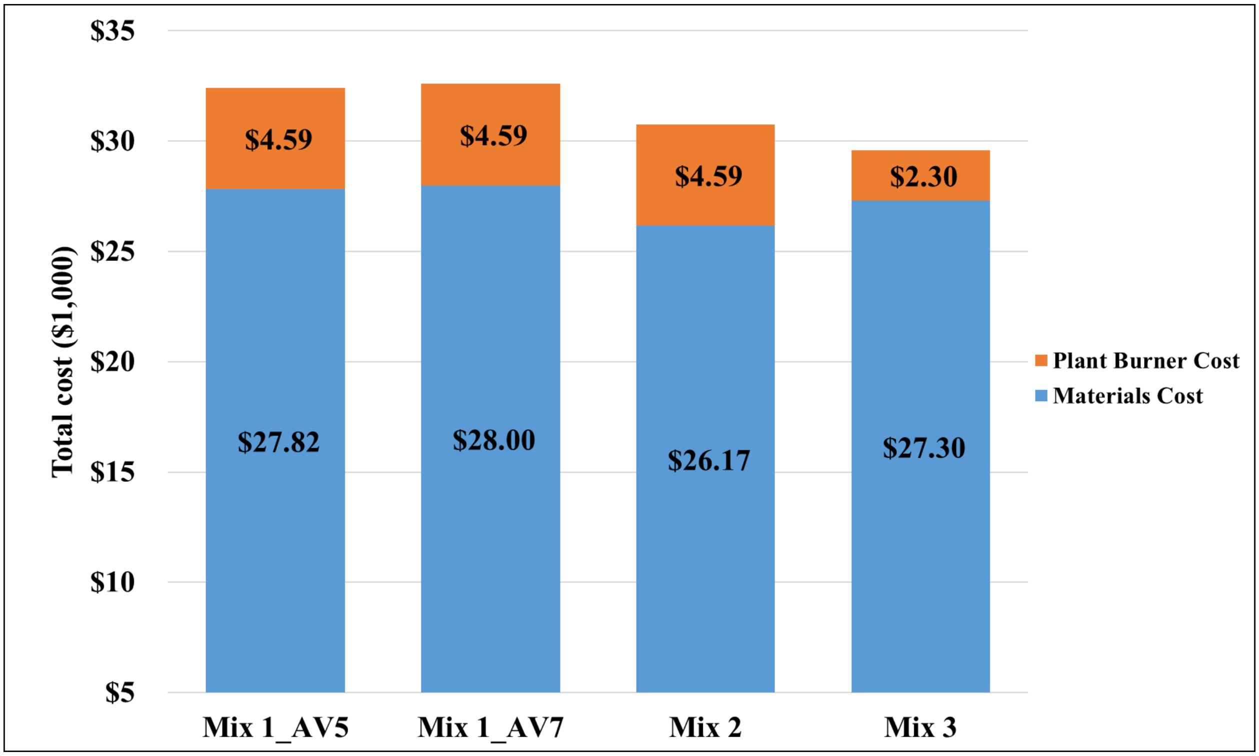


Figure 6.8. Cost comparison for all the mixes based on materials and burner fuel cost (costs calculated for a one-mile roadway section with a single 12 ft. wide lane and 2 inches of compacted asphalt concrete layer thickness)

The tool can compare up to four different mix strategies. This means the user can evaluate differences in total cost for up to four different binder types and/or RAP contents. A summary spreadsheet compares the various mix design options. This sheet shows the cost differences between each individual mix design, as well as maximum and minimum cost options. The lowest and highest cost options are indicated. Considering the production costs (burner fuel usage), the mixes total cost was also calculated (materials + production burner cost). A bar chart shows a side-by-side comparison of each mix design strategy in order to visualize the costs of each option and also it shows a comparison of total cost for all mixes.

In this study, the following costs were used to calculate the total material cost of asphalt mixtures. These are average costs based on previous years' productions:

- RAP: $20/ton
- Aggregate: $13/ton
- PG70-22ER binder: $490/ton
- Evotherm P25®: $70/ton (added to the per ton cost of binder for a 0.7% WMA by weight of binder)

6.6.5 Life-Cycle Cost Analysis (LCCA)

In this study, analyses were first performed by only considering material costs to be able to compare the impact of RAP content, binder content, and additives on life cycle costs. Then, a second set of LCCA was performed after including the plant burner costs to be able to determine the cost impact of using warm-mix.

In this study, each section was assumed to be a single-lane having a width of 12 ft (3.7 m) and a length of 1 mile and material costs were calculated for all mixes based on a 2inch (50.8mm) layer thickness. The cost calculation tool described in Section 6.6.4 was used to calculate the material costs.

Net present value (NPV) of agency costs were determined using a 4 percent interest rate for a 60-year analysis period by using Equation 6-1. Since all mix designs had a 20-year design period, it was assumed that same mixtures will be used every 20 years for the next 60 years. It should be noted that the purpose of LCCA is to be able to compare the cost effectiveness of all mixtures. Calculated NPV values can only be used for comparison and cannot be used for bidding or long-term cost predictions. The diagrams used for LCCA are shown in Figure 6.9.

$$\mathbf{NPV} = \sum_{t=0}^{T} \frac{C_t}{(1+r)^t} \tag{6-1}$$

Where:

C_t = estimated agency costs at year t,

r = interest rate, and

T = number of time periods.

In this study, the NPV was calculated for all the mixes and the equation below describe how the NPV for Mix1_AV5 was calculated (as an example).

$$\mathbf{NPV_{6\%BC}Mix1\_AV5} = \frac{\$27,823}{(1+0.04)^0} + \frac{\$27,823}{(1+0.04)^{20}} + \frac{\$27,823}{(1+0.04)^{40}} = \$46,316 \tag{6-2}$$

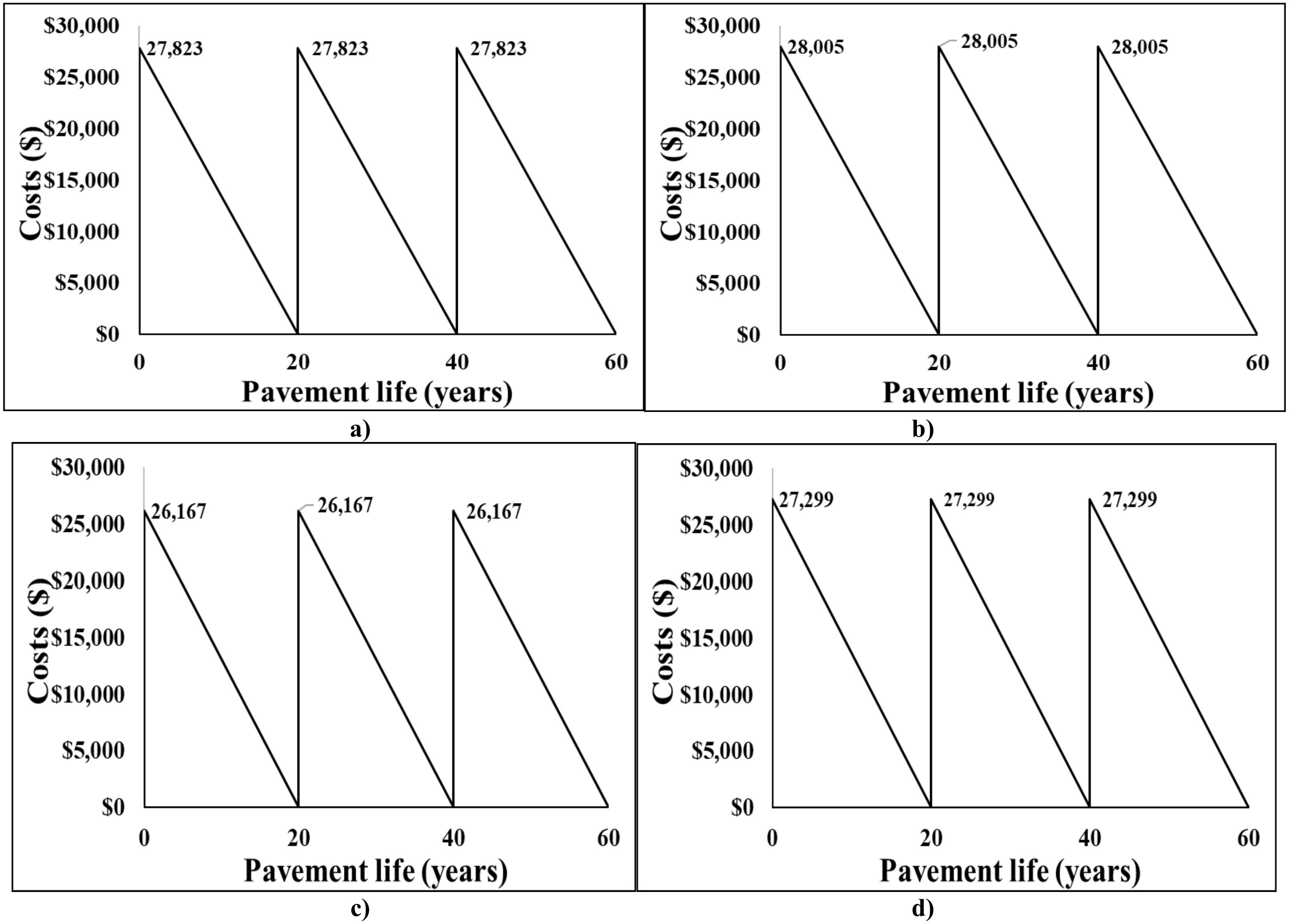


Figure 6.9. Diagrams used for LCCA (a) Mix1_AV5 (b) Mix1_AV7 (c) Mix2 and (d) Mix3.

In Table 6.5, the NPVs without the burner fuel consumption costs (by just considering raw material costs) were summarized for all asphalt mixtures of this study.

Table 6.5. NPVs for all the Mixes – Without Burner Fuel Consumption Cost

S. No.	Mix ID	Initial cost ($)	NPV-1 ($)	NPV-2 ($)	NPV ($)
1.	Mix1_AV5	27,823	12,698	5,795	46,316
2.	Mix1_AV7	28,005	12,781	5,833	46,619
3.	Mix2	26,167	11,942	5,450	43,560
4.	Mix3	27,299	12,459	5,686	45,444

It can be observed from Table 6.5 that the mix with 45% RAP content (Mix 2) has the lowest NPV over the course of 60 years analysis period followed by the warm mix asphalt (Mix 3) and the mix with 30% RAP (Mix 1) when only the raw material costs are considered. However, this ranking altered when the plant burner fuel consumption was incorporated into the life cycle cost analysis as can be seen in Table 6.6. When the burner costs are included in the LCCA, the most cost-effective mix is the warm mix asphalt (Mix 3) considering the reduced production (burner) temperature and consequently less fuel consumption during production.

Table 6.6. NPVs for all the Mixes – With Burner Fuel Consumption Cost

S. No.	Mix ID	Initial cost ($)	NPV-1 ($)	NPV-2 ($)	NPV ($)
1.	Mix1_AV5	32,416	14,794	6,752	53,962
2.	Mix1_AV7	32,599	14,878	6,790	54,267
3.	Mix2	30,761	14,039	6,407	51,207
4.	Mix3	29,597	13,508	6,165	49,269

6.6.6 Environmental Impact

Pavement Life Cycle Assessment (LCA) procedures were used to calculate the environmental impact of each pavement mixture for the material production and construction stages of the pavement life cycle. For a base case, a mixture of 6% binder content and 20% RAP content was selected (Mix F in the plots). This represents the most common pavement design in Oregon. The roadway for all cases had an existing layer thickness of 13.5inches (with three layers of pavement with thicknesses of 2.5 inches, 5.5 inches, and 5.5 inches). The length of roadway was set to 0.62 mile (1 km), with three lanes of 12 feet each, a typical width for roadways in the U.S.

In order to determine the differences in environmental performance, the primary characteristics for each pavement design were entered into the Pavement LCA software. Materials by percentage of total mixture weight were input (binder content, additives, RAP content, etc.) along with the asphalt type (HMA or WMA). All factors for which no data was available, or those factors which were not considered (such as hauling distance) were set to be default and equal between mixes so as not to affect the results. Pavement vehicle interaction (PVI), being a separate option in the software, was excluded entirely since all mixes were designed for 20 years and PVI related vehicle operating costs should be theoretically equal for all analyzed mixtures.

In order to accurately compare different pavement designs, each mixture was assumed to conform to a 60-year lifespan with rehabilitation occurring at every 20th year. For rehabilitation

2 inches of asphalt is milled and removed and then replaced (mill and fill process which is commonly used in Oregon for rehabilitation).

Results were exported from the software and plotted using excel. Results are given in Global Warming Potential (GWP), Acidification Potential (AP), and Eutrophication Potential (EP) for all three mixtures of this study. Mix 1 with 5% air void case was not evaluated since density does not directly change the environmental impact during the material production and construction stages. Units do not represent the chemical composition of the pollution itself, but instead represent the amount of a standard normalizing factor representative of each pollution type (Myhre et al. 2013).

Figure 6.10 displays the results for global warming potential by mix type, in units of kilograms of carbon dioxide. Global warming potential acts as a useful parameter to assess the future impact of an emission on the atmosphere (Myhre et al. 2013).

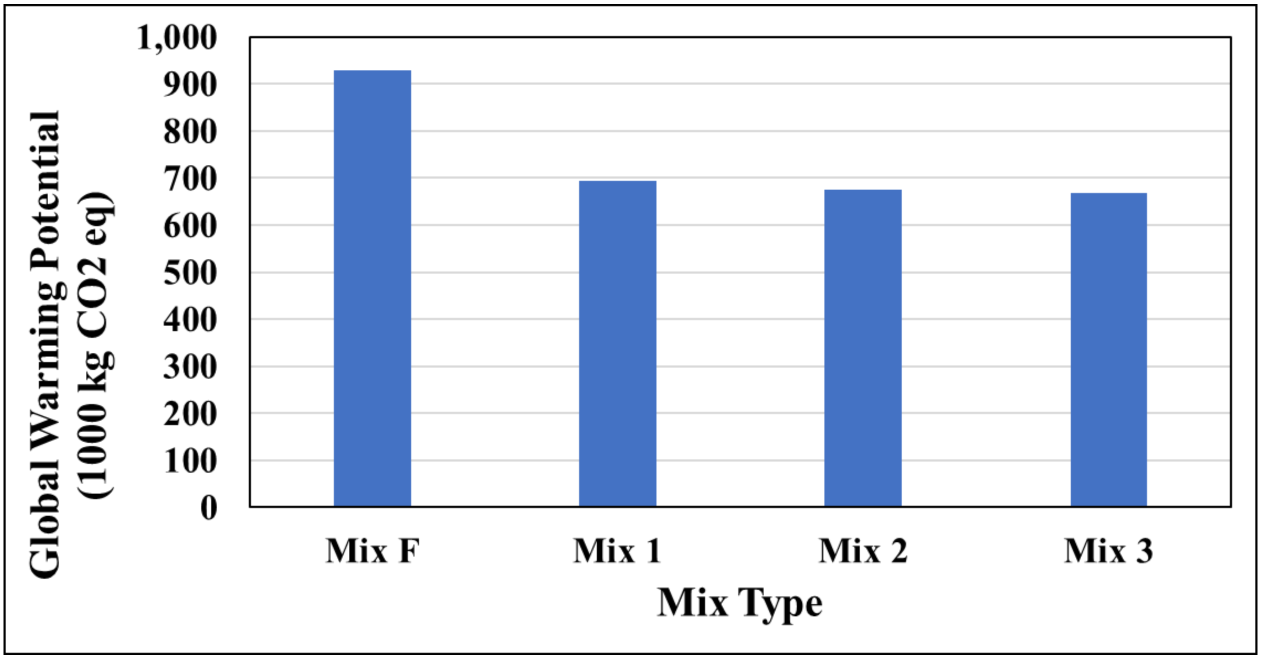


Figure 6.10: Global Warming Potential (GWP) by mix type

Mixtures 1, 2, and 3 each performed nearly equivalently, with mixture 1 exhibiting slightly worse performance and mixture 3 (warm-mix) being the best. All mixtures had significantly lower impact when compared to the typical Oregon asphalt mixture with lower RAP content. This is likely caused by the difference in the production process between HMA and WMA and higher RAP content in the designed mixtures. This result also proves the importance of further increasing the RAP content of asphalt mixtures in Oregon for the environment.

Figure 6.11 displays the acidification potential of each pavement mix. Acidification results from carbon dioxide released into the atmosphere dissolving into ocean waters which increases the concentration of carbonate ions and lowers ocean water pH (Feely et al. 2009).

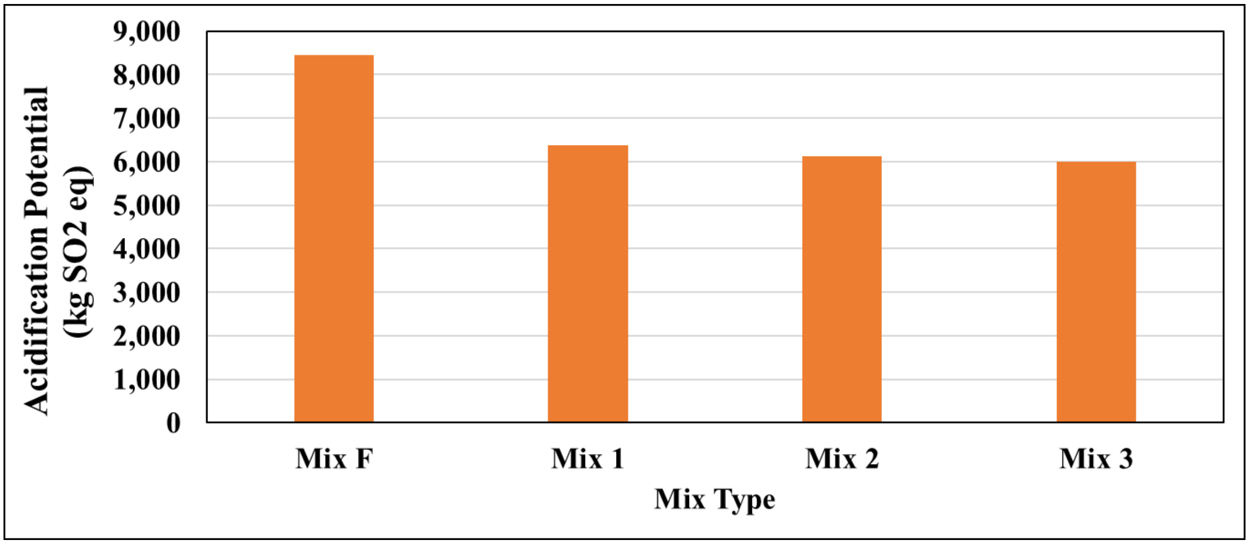


Figure 6.11: Acidification Potential (AP) by mix type

The results for acidification potential are similar to that of global warming potential. Mix F again performed poorly while mixes 1, 2, and 3 performed similarly. Mix 3 (warm-mix) again outperformed both Mixes 1 and 2. This is most likely a result of the WMA production process being significantly less energy intensive as well as the design allowing for a lower binder content and higher RAP.

Figure 6.12 displays the eutrophication potential generated by each mixture measured in kilograms of nitrogen. Eutrophication is a measure of the increased availability of normally population limiting factors for aquatic based photosynthetic organisms (Carpenter et al. 2015). Increased eutrophication can lead to the destabilization of ocean ecosystems.

The results indicate that mixtures 1, 2, and 3 again outperformed the typical pavement design. Mixture 3 (warm-mix) performed the highest of the three design mixtures. The differences between the three design mixtures and the typical mixture are likely explained by the increased RAP content in the three designs as well as the lower energy cost of WMA. Differences between the three designs is likely to be caused by the slight difference in binder content as well as RAP content.

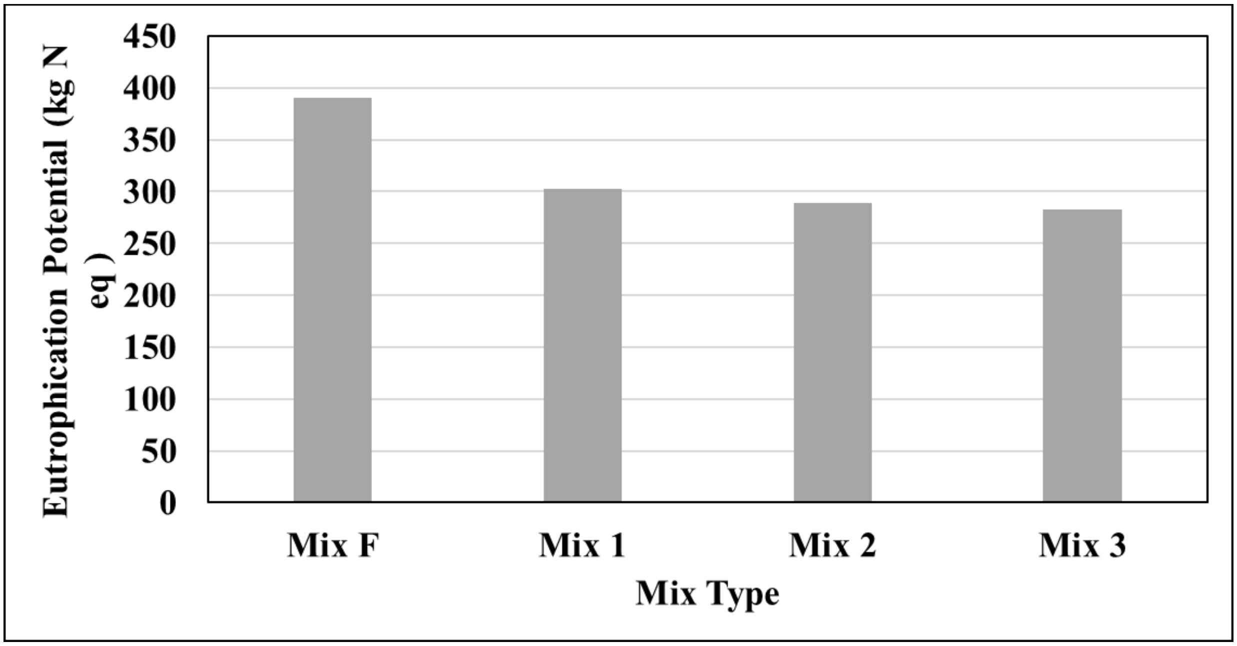


Figure 6.12: Eutrophication Potential (EP) by mix type

6.7 CONCLUSIONS AND RECOMMENDATIONS

In this study, volumetric and balanced mix designs were conducted to determine the optimum asphalt binder content for four different asphalt mixtures. Cost effectiveness and the environmental impact of those asphalt mixtures were also quantified and compared. Based on the quantified cost, performance, and environmental impact values, the mixture with warm-mix additives (Mix 3) was selected as the best asphalt mixture with highest cracking resistance, lowest cost, and lowest environmental impact. Other conclusions derived from this study are as follows:

1. Mix3 has cracking resistances significantly higher than all other mixtures. Higher cracking resistance for the Mix3 is likely to be a result of the use of a warm mix additive. It is important to mention that the mixtures with warm mix additive are showing better cracking resistance than other corresponding mixes with same or higher binder contents.

2. The FI value for Mix1 with 5% air-void was slightly higher than the same mix with 7% air void. Thus, density of the mix appears to have a significant effect on the cracking resistance.

3. High RAP mix (Mix2) has better cracking resistance than the low RAP mix (Mix1) according to SCB test results but this is expected to be a result of the higher binder content of Mix2 specimens. The higher BMD binder content of Mix 2 (when compared to lower RAP mix-Mix 1) suggested that performance of high RAP mixture can be improved by slightly increasing the binder content.

4. Although Mix 2 (45% RAP) had a higher BMD binder content than Mix 1 (30% RAP), it was still more cost effective due to the increased use of recycled asphalt material in the mix.

5. Mix1_AV5 has the best rutting resistance among all the mixes. Samples for only this mixture were compacted at 5% air-voids. Higher density (2% higher than 7% air-void samples) resulted in an improved rutting resistance. It is important to note that 2% increase in density resulted in significant improvements in both rutting and cracking performance. Although not simulated in this study, increased density is also expected to reduce long-term aging and moisture susceptibility of the asphalt mixtures due to reduced permeability.

6. It is possible that Mix 3 with warm-mix additives can have better "compactibility" due to lower viscosity of the modified asphalt binder. Improved compactibility may result in higher density values with associated long-term performance benefits.

7. Based on the balanced mix design plots for other three mixes, the required asphalt content for Mix1_AV7, Mix2 and Mix3 are 6.05%, 6.10% and 5.30%, respectively.

8. The mix with 45% RAP content (Mix 2) has the lowest NPV over the course of 60 years analysis period followed by the warm mix asphalt (Mix 3) and the mix with 30% RAP (Mix 1) when only the raw material costs are considered. However, this ranking altered when the plant burner fuel consumption was incorporated into the life cycle cost analysis. When the burner costs are included in the LCCA, the most cost-effective mix is the warm mix asphalt (Mix 3) considering the reduced production (burner) temperature and consequently less fuel consumption during production.

9. Mix 3 (warm-mix) is also the most environmentally friendly mix with lower expected GWP, EP, and AP values for a 60-year analysis period.

7.0 SUMMARY AND CONCLUSIONS

The effect of increased density in asphalt mixes is highly significant. It is expected that the cost of achieving this higher in-place density can be considerably less than the cost savings made on operation and maintenance from the prolonged service life of the pavements. This result makes the idea of having asphalt mixes with improved compactibility highly cost effective.

However, suggesting an increase in density without providing guidelines on how to achieve them can result in a negative impact on asphalt mix durability. For instance, increasing mix density by using excessive amounts of fillers and asphalt binder can result in long-term durability issues. Thus, current mix design procedures and mix compaction processes should be improved to produce high density and high performance asphalt mixes during construction without creating a detriment to the overall performance of the pavement. This study aimed to provide practical modifications to mix design procedure for achieving higher in-place density (compactibility) and thereby increasing the long-term performance of asphalt mixtures. Some of the strategies that were investigated in this study are Superpave5 design method, warm mix additive, increased temperature, higher binder content, and increased RAP content. Major conclusions derived from the laboratory testing and analytical findings (pavement LCA and LCCA) are discussed in this section.

In this study, for the SP5 mix designs to achieve a 5% air-void content, design gyration levels of 50 and 70 were selected for the Level 3 and Level 4 mix designs, respectively. These gyration levels were selected based on the suggestions provided by Huber et al. (2016). As per ODOT specifications, standard Level 3 and Level 4 mixes are designed with 80 gyrations and 100 gyrations (to achieve a 4% air-void content), respectively. It is important to note that gyration levels for the SP5 mix design were later changed by INDOT to 30 and 50 for medium and high traffic locations, respectively. The impact of those updated gyration levels on mixture volumetrics and performance should be evaluated in a future research study.

7.1 MAJOR CONCLUSIONS

The major conclusions drawn from the results of this study are as follows:

IMPACT OF DENSITY ON CRACKING AND RUTTING PERFORMANCE OF ASPHALT MIXTURES – A DETAILED LOOK AT THE SUPERPAVE5 MIX DESIGN

1. SP5 mix design method is capable of producing mixes that are more compactible. It might be possible to achieve about 1 to 2% higher density during construction by using the SP5 mix design method.

2. For all three projects, SP4 mixes (current standard in Oregon) had significantly higher cracking resistance than the SP5 mixes although the SP5 SCB test specimens had 2% higher density (5% target air-void content) than the SP4 SCB specimens (7% target air-void content). SP5 mix design method tends to fill the void space with fine

aggregate particles rather than using the asphalt binder to achieve the required density level. Reduced effective binder content reduced the flexibility of the asphalt mixture and resulted in lower cracking resistance.

3. For the SP4 mixes, polymer modified binder used in Project 2 and Project 3 mixes resulted in higher rutting resistance and cracking resistance than the unmodified Project 1 mix.

4. Using the SP5 mix design method completely removed the benefits of using polymer modified binders to improve cracking resistance (all three projects designed with SP5 had almost equal average FI values although Projects 2 and 3 had polymer-modified binders).

5. Higher dust-to-binder ratios for all SP5 mix designs point out the issue of having excessive amount of dust and not enough binder to achieve the desired high cracking resistance. These results suggested that increasing the density and compactibility of the mix is not always a guaranteed strategy to improve pavement durability. It is possible to improve compactibility by changing the gradation while higher fine aggregate content can result in lower cracking resistance.

6. While the cracking resistance of the SP5 mixes are significantly lower than the SP4 mixes, almost all the SP5 mixes are meeting the volumetric requirements specified by ODOT (with the exception of Projects 1 and 3 dust-to-binder ratios while they are just slightly over the upper 1.6 threshold). This result suggested that conducting cracking and rutting tests during the mix design process and adapting the balanced mix design approach (performance testing at the mix design stage) is expected to result in mixes with higher durability. Volumetrics alone may not provide the most durable asphalt mixture.

7. For all the cases, the SP5 mix showed lower rut depth (better rutting resistance) than the corresponding SP4 mix. However, the average rut depths of all the mixtures were lower than 3.0 mm which is significantly lower than the maximum allowed rut depth according to AASHTO T 324 (which is 12.5mm). This result suggested that both SP4 and SP5 mixes are on the "dry" side and need more asphalt binder to improve their cracking resistance while still meeting the 12.5mm maximum rut depth requirement.

8. Adapting the SP5 mix design in Oregon is not expected to create any cracking performance benefits although it can improve the compactibility of the mix and result in higher asphalt layer density.

IMPACT OF INCREASED COMPACTION TEMPERATURE AND THE USE OF WARM-MIX ADDITIVES ON THE PERFORMANCE OF ASPHALT MIXTURES

1. All WMA mixes provided significantly higher (1.5 to 2 times higher than the HMA levels in some cases) FI values than the corresponding HMA/control mixes. Addition of Evotherm P25® by 0.5% of the total weight of binder resulted in a considerable improvement in the cracking resistance without increasing the binder content and at

lower compaction temperatures that provide more comfortable working conditions for construction workers.

2. Increasing the production temperatures of the HMA mix to achieve higher density in field is not an effective strategy as it sacrifices the durability of the mix.

3. The use of Evotherm P25® WMA chemical additive in the asphalt mixtures has the potential of reducing the production temperatures by about 35ºC without significantly reducing the rut resistance of the mix.

4. The average rut depth of all mixes prepared at higher mixing and compaction temperatures are significantly lower than the mixes prepared at lower/standard temperatures. This is expected to be a result of the excessive binder aging during the mix production stage.

EVALUATION OF PERFORMANCE AND COMPACTABILITY OF ASPHALT MIXTURES USING A HYDRAULIC ROLLER COMPACTOR

1. For all the mixes, the roller compacted samples yielded higher flexibility indices as compared to the gyratory compacted samples.

2. WMA samples mixed and compacted at temperatures 35ºC lower than the HMA samples resulted in approximately 100% higher cracking resistance than the control HMA mix without warm mix additives.

3. Increasing the mixing and compaction temperatures of the WMA test samples to the level of standard HMA mixes significantly lowered the cracking resistance. High mixing and compaction temperature eliminate the positive impact of Evotherm P25® additive on the mix.

4. Increasing the binder content of HMA control mix by 0.5% by weight considerably improved its cracking resistance.

5. WMA samples showed cracking resistance comparable to the HMA samples with 0.5% higher binder content. WMA can be considered as a promising alternative for improving the cracking resistance of asphalt mixtures in Oregon without increasing the binder content.

6. The HMA mixes with 0.5% higher binder content performed slightly better in terms of rutting resistance than the WMA mixes with 0.5% less binder. However, the differences in the average rut depths of the two mixtures (0.60 mm and 0.76 mm for SP4 and SP5 type mix, respectively) are not significant. This result is also consistent with the SCB test results (Figure 5.8). One major factor that should be noted while comparing the performance of the two mixes is that the WMA mix achieved the cracking and rutting resistance comparable to the HMA mix with 0.5% higher binder content through a mixing process that requires about 35ºC lower mixing and compaction temperatures than the conventional HMA mixes.

7. WMA mix amounts to less upfront cost than the HMA mix with 0.5% higher binder content when the burner fuel cost is factored into the cost calculation process. However, the HMA mix with 0.5% higher binder content results in slightly lower cost than the WMA mix when the plant burner costs were not considered in the calculations.

8. When the WMA mix was compared with the HMA mix for both projects, the WMA mix was found to be more compactable (lower number of passes required to compact the blocks).

9. For the SP5 mix design, the WMA test samples prepared at higher HMA mixing temperatures took a greater number of passes to compact than the samples produced at recommended-lower mixing temperatures. The increased effort required to compact the WMA mix at higher HMA mixing temperatures can be attributed to the aging or stiffening of the asphalt binder. Moreover, it is also suspected that Evotherm P25® becomes less effective at mixing temperatures higher than the recommended temperatures due to its volatility.

10. Project 2 mix with high binder content showed the highest compactability among all the mixtures. The reason behind the reduction in the average number of passes required for compaction of the mixes with high binder content is the reduced friction between the aggregates and within the asphalt mixture microstructure. Improved compactibility is expected to result in asphalt concrete layers with higher densities in the field.

SELECTION OF DURABLE, ENVIRONMENTALLY FRIENDLY, AND COST-EFFECTIVE ASPHALT MIXTURES FOR OREGON – EFFECTS OF DENSITY, WMA, AND HIGH RAP

1. Based on the quantified cost, performance, and environmental impact values, the mixture with warm-mix additives (Mix 3) was selected as the best asphalt mixture with highest cracking resistance, lowest cost, and lowest environmental impact.

2. Mix3 has cracking resistances significantly higher than all other mixtures. Higher cracking resistance for the Mix3 is likely to be a result of the use of a warm mix additive.

3. The FI value for Mix1 with 5% air-void was slightly higher than the same mix with 7% air void. This mix also had the best rutting performance of all three mixtures. Thus, density of the mix appears to have a significant effect on the cracking and rutting resistance. Although not simulated in this study, increased density is also expected to reduce long-term aging and moisture susceptibility of the asphalt mixtures due to reduced permeability.

4. High RAP mix (Mix2) has better cracking resistance than the low RAP mix (Mix1) according to SCB test results but this is expected to be a result of the higher binder content of Mix2 specimens. The higher BMD binder content of Mix 2 (when compared to lower RAP mix-Mix 1) suggested that performance of high RAP mixture can be improved by slightly increasing the binder content.

5. Although Mix 2 (45% RAP) had a higher BMD binder content than Mix 1 (30% RAP), it was still more cost effective due to the increased use of recycled asphalt material in the mix.

6. The mix with 45% RAP content (Mix 2) has the lowest NPV over the course of 60 years analysis period followed by the warm mix asphalt (Mix 3) and the mix with 30% RAP (Mix 1) when only the raw material costs are considered. However, this ranking altered when the plant burner fuel consumption was incorporated into the life cycle cost analysis. When the burner costs are included in the LCCA, the most cost-effective mix is the warm mix asphalt (Mix 3) considering the reduced production (burner) temperature and consequently less fuel consumption during production.

7. Mix 3 (warm-mix) is also the most environmentally friendly mix with lower expected GWP, EP, and AP values for a 60-year analysis period.

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