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16. Abstract
A recent rise in asphalt binder prices has led state agencies and contractors to use higher quantities of Reclaimed Asphalt Pavement (RAP). Besides being economic, sustainable, and environmentally friendly, RAP can be replaced for a portion of aggregates in Hot Mix Asphalt (HMA) where quality aggregates are scarce. In this project, the effect of increasing RAP percentage and using fractionated RAP (FRAP) in HMA mixture on moisture resistance, rutting, and fatigue cracking were evaluated. Mixtures with five different RAP and FRAP contents (20%, 30%, and 40% RAP, and 30% and 40% FRAP) were studied. The Hamburg Wheel Tracking Device (HWTD) Test (TEX-242-F), Kansas Standard Test Method KT-56 or modified Lottman Test, and Dynamic Modulus Test (AASHTO TP: 62-03) were used to predict moisture damage, rutting potential and fatigue cracking resistance of the mixes. HMA specimens were prepared based on Superpave HMA mix design criteria for 12.5 mm (1/2 inch) Nominal Maximum Aggregate Size (NMAS) and compacted using the Superpave gyratory compactor. Results of these tests showed that although mixture performance in the laboratory tests declined as the percentage of RAP increased in the mix, even mixtures with 40% RAP passed the minimum requirements in commonly used tests. When RAP is compared with FRAP, FRAP does not seem to improve performance of the HMA mixtures. This was largely confirmed by the statistical analysis. Mixtures with RAP performed more or less the same as or better than the mixtures with FRAP.

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Abstract

A recent rise in asphalt binder prices has led state agencies and contractors to use higher quantities of Reclaimed Asphalt Pavement (RAP). Besides being economic, sustainable, and environmentally friendly, RAP can be replaced for a portion of aggregates in Hot Mix Asphalt (HMA) where quality aggregates are scarce.

In this project, the effect of increasing RAP percentage and using fractionated RAP (FRAP) in HMA mixture on moisture resistance, rutting, and fatigue cracking were evaluated. Mixtures with five different RAP and FRAP contents (20%, 30%, and 40% RAP, and 30% and 40% FRAP) were studied. The Hamburg Wheel Tracking Device (HWTD) Test (TEX-242-F), Kansas Standard Test Method KT-56 or modified Lottman Test, and Dynamic Modulus Test (AASHTO TP: 62-03) were used to predict moisture damage, rutting potential and fatigue cracking resistance of the mixes. HMA specimens were prepared based on Superpave HMA mix design criteria for 12.5 mm (1/2 inch) Nominal Maximum Aggregate Size (NMAS) and compacted using the Superpave gyratory compactor. Results of these tests showed that although mixture performance in the laboratory tests declined as the percentage of RAP increased in the mix, even mixtures with 40% RAP passed the minimum requirements in commonly used tests. When RAP is compared with FRAP, FRAP does not seem to improve performance of the HMA mixtures. This was largely confirmed by the statistical analysis. Mixtures with RAP performed more or less the same as or better than the mixtures with FRAP.

Chapter 1 Introduction

1.1 Introduction

Construction of Hot Mix Asphalt (HMA) pavements requires large quantities of virgin aggregates and asphalt binder. According to National Asphalt Pavement Association (NAPA), the current annual U.S. production of HMA materials is approximately 500 million tons per year, which includes about 60 million tons of reclaimed material that is reused or recycled directly into pavements. As of 2007, about 90 million tons of Reclaimed Asphalt Pavement (RAP) are reused or recycled into other pavement-related applications every year for a total use of over 100 million tons of RAP each year. This is an increase from 72 million tons of RAP used each year in the early 1990s. There is no doubt that these quantities are required to maintain current infrastructures or reconstruct new pavements, but it definitely is critical to consider their future re-usage. Besides sustainability/environmental-related reasons for using RAP in constructing new pavements, saving resources and disposal costs, the rapidly increasing price of crude oil and lack of quality aggregates at some locations are other prevalent reasons to use RAP in HMA pavements (Zofka et al 2010).

Recycling is beneficial in most cases because of reduced consumption of virgin materials but pavement performance should not be compromised for cost reduction (Mohammad, Cooper, and Elseifi 2011). It has long been accepted that RAP can be a feasible constituent in HMA pavements and if properly designed and constructed, HMA pavements incorporating mixtures with RAP can perform as well as conventional mixtures (Huang, Shu, and Vukosavljevic 2011). The only issue is to what extent RAP should be allowed in different HMA mixes without sacrificing durability for lower initial cost. The average use of RAP across the United States is currently estimated at 12% of the mix, however based on agencies' specifications, there is

potential to use up to 30% RAP in the intermediate and surface layers of pavement (FHWA 2010). However, there are some concerns about long term performance and durability of asphalt pavements containing RAP especially in the major load-carrying and surface layers. Generally, as a result of having some long term-aged asphalt in the mixtures containing RAP, asphalt cement tends to be stiffer. The advantage of having a stiffer mix is it is less susceptible to rutting, and its disadvantage is being less resistance to fatigue and thermal cracking.

Because of the aforementioned concerns, traditionally many state transportation agencies limited the maximum amount of RAP used in surface layers, certain mixture types, and, in some instances, in large or critical projects. The amount of RAP was typically limited to 15% or lower because there were no binder grade changes or additional tests needed for these lower percentages in Superpave mixtures. Additionally, there was no significant economic incentive for using larger percentages of RAP either. However, in 2006 and again in 2008, there were sharp increases in asphalt binder costs. As a result, RAP use spiked as indicated by greater percentage of RAP now being allowed or used (fig. 1.1). In addition, stricter environmental regulations, and an emphasis on “green” technologies (e.g., warm mix asphalt (WMA)) and sustainable pavements, the highway agencies are more open to allowing higher percentages of RAP in HMA pavements. However, there is a lack of guidance on the use of high percentages of RAP (high RAP) in mixtures as well as information on performance of these mixtures.

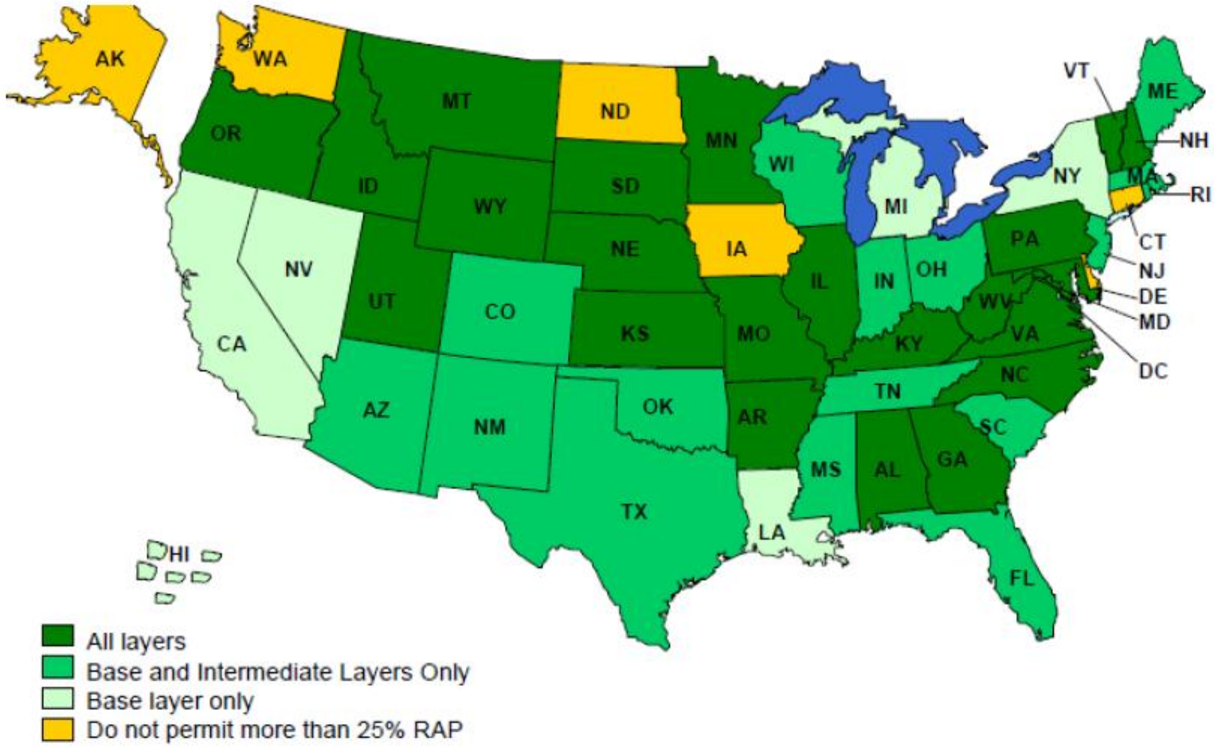

Figure 1.1 States that allow more than 25% of RAP in HMA mixtures (Copeland 2010)

1.2 Problem Statement

There are three main reasons for RAP to be favored over virgin materials: the increasing cost of crude oil and asphalt binder, the scarcity of quality aggregates, and the pressing need to preserve the environment. Many state agencies have also reported significant savings when RAP is used. Considering material and construction costs, it has been estimated that use of RAP provides a savings ranging from 14% to 34% for RAP content varying between 20% and 50%. Because of recent increases in asphalt binder price, contractors are willing to use high percentages of RAP in HMA. The current national guideline, AASHTO M 323, for determining the binder grade adjustment in HMA mixes is shown in table 2.1. The table shows that a softer binder will be required if more than 15% RAP is going to be used in the HMA mix. Softer

binders are more expensive and in the recent past, contractors were not willing to pay extra. However, as the asphalt price is rising, they are opting for higher percentages of RAP in Superpave mixtures.

Table 1.1 Binder selection guidelines for RAP mixtures according to AASHTO M 323

Recommended Virgin Asphalt Binder Grade	RAP Percentage
No change in binder selection	<15
Select virgin binder one grade softer than normal	15-25
Follow recommendations from blending charts	>25

One of the requirements in the Superpave mix design is the control of the gradation of aggregates. Due to segregation in RAP stockpiles and its influences on asphalt and dust content in the final mix, gradation control has been very difficult with RAP, especially when higher percentages of RAP were being added to the mix. The problem with segregated RAP is that the finer fraction will contain a higher asphalt content, because of higher surface area, making the mix air void control very difficult. As mentioned earlier, fractionation is a process in which RAP is separated into at least two sizes, typically a coarse fraction, plus 12.5 or 9.5 mm (1/2 or 3/8 inch), and a fine fraction, minus 12.5 or 9.5 mm, in order to ensure the required consistency in RAP. In the United States, while some states are drafting specifications for fractionated RAP (FRAP), some others allow higher percentages of FRAP in the mix in comparison to RAP. However, no systematic studies have been performed to date comparing these two products.

1.3 Objectives

The objective of this research was to determine the impacts of having higher percentages of RAP and FRAP on mixture performance while meeting the current requirements of the Superpave mix design.

1.4 Report Organization

This report is divided into four chapters, including this introductory chapter (Chapter 1). Chapter 2 provides a literature review. Chapter 3 describes the methodology and laboratory testing. Chapter 4 discusses test results and related analysis. Chapter 5 presents conclusions based on this study and recommendations for further study.

Chapter 2 Literature Review

2.1 Introduction

Reclaimed Asphalt Pavement (RAP) is any removed or reprocessed pavement material that contains aggregates and asphalt cement. RAP is obtained during rehabilitation or reconstruction of existing asphalt pavements, or from utility cuts across the roadways which were necessary to gain access to underground utilities. When RAP is properly crushed and screened, it will consist of high-quality aggregates coated with asphalt cement binder which can be used in a number of highway construction applications. These include its use as an aggregate substitute and asphalt cement supplement in new or recycled asphalt mixes, as granular base or sub-base, as a stabilized base aggregate, or as an embankment or fill material. Use of RAP in asphalt mixes helps reduce costs, conserves asphalt and aggregate resources, and limits the amount of waste material going into landfills (Copeland 2010). Asphalt pavement is generally removed either by milling or by full-depth removal. Milling is typically done in rehabilitation projects where the existing wearing course is removed and then replaced to increase the pavement's service life. RAP produced from milling is ready to be recycled with little or no processing, depending on the amount being used in the mixture. Full-depth removal involves milling the existing HMA pavement structure in several passes, depending on existing depth of the structure, or by ripping and breaking the pavement into large pieces using rippers on a bull dozer. Broken RAP pieces are collected, loaded onto trucks, and usually transported to processing facilities. RAP is processed by crushing and screening, and then is conveyed and stockpiled (Brown et al. 2009, Copeland 2010).

2.2 Benefits of RAP

Use of reclaimed asphalt pavement in hot mix asphalt has the following benefits (Al-Qadi et al. 2007, Copeland 2010):

- Reduction of construction costs;
- Conservation of construction materials like aggregate and binders;
- Preservation of existing pavement geometrics;
- Preservation of environment; and
- Conservation of energy.

Figure 2.1 Milled Reclaimed Asphalt Pavement (Copeland 2010)

Figure 2.2 RAP Stockpiles at an Asphalt Concrete Production Plant (Copeland 2010)

2.3 Challenges of Increased RAP Usage

Currently, the average RAP usage in HMA is estimated at 12% in the United States. Less than half of state departments of transportation use no more than 20% RAP, though specifications in most states would allow up to 30% RAP in HMA. A number of states, including Kansas, have experimented with or routinely use high RAP. However, most do not use high RAP percentages in the intermediate and surface layers of pavements. Despite similarities between producing virgin asphalt mixtures and RAP asphalt mixtures, there are still challenges for maximizing RAP use and routinely using high RAP. According to AASHTO M 323, the current binder selection guidelines for RAP mixtures were formulated based on the assumption that complete blending occurs between the virgin and RAP binders.

It is understood that the amount of blending that occurs between the virgin and RAP binders is somewhere between complete blending and no blending at all; however, there is no direct method available to accurately determine the amount of blending that occurs. Currently,

researchers are performing ongoing studies to develop methods to determine if proper blending has occurred by using mixture properties, such as dynamic modulus, to estimate blended binder properties and to compare estimate blended binder properties to measured binder properties. For high RAP mixtures, blending charts can be used to properly determine the virgin binder grade. They can also be used to optimize the amount of RAP used if the virgin binder grade is known. However, blending charts require expensive, time-consuming binder extraction and recovery procedures that use hazardous solvents, which is followed by testing of the recovered binder. Consequently, many state transportation departments are reluctant to permit RAP content that require this testing. Additionally, many contractors are not equipped to perform binder extractions and recoveries or the subsequent binder tests. In general, state transportation departments are concerned with the consistency of RAP materials and whether mixtures with high RAP are inferior and fail earlier than virgin mixtures. In some instances, state transportation departments place limitations on the amount of RAP that can be used based on previous bad experiences with RAP. According to the 2007 NCDOT survey, the four most common factors preventing the use of additional RAP are (Copeland 2010):

- Specification limitations;
- Lack of processing (i.e., variability of RAP);
- Lack of RAP availability; and
- Past experiences.

In the 2009 NCDOT survey, participants were asked to identify major concerns and obstacles that limit or preclude the use of RAP in HMA (Copeland 2010). The two concerns cited most often regarded the quality of the blended virgin and RAP binder, especially for high RAP mixes and polymer modified binders, and stiffening of the mix from high RAP quantities

and resulting cracking performance. Several states were concerned that the use of RAP with polymer-modified binders may reduce the quality of the polymer-modified virgin binder. Furthermore, high RAP may affect binder properties resulting in an “overly stiff” mix that may experience low-temperature cracking. There was also concern that an overly stiff mix may not be as resilient and may crack prematurely for pavements experiencing high deflections.

The most common barriers among state transportation departments are:

- Quality concerns;
- RAP consistency;
- Binder grade and blending;
- Mix design procedures;
- Volumetric requirements;
- Durability and cracking performance; and
- Use with polymers.

The most common barriers among contractors are:

- State transportation department specifications;
- Control of RAP;
- Dust and moisture content; and
- Increased quality control (QC).

2.4 Characteristics of RAP Materials

As mentioned earlier, RAP can be used as a constituent in new HMA mixtures. During service, the blend of aggregates and asphalt binders of RAP undergoes various physical and rheological changes that must be considered in the HMA design process to ensure that HMA mixtures with RAP perform similarly to HMA mixtures containing only virgin materials. It is

important to know how much asphalt binder is present in the RAP material so that it can be accounted for in the mix design process. It is also important to know some physical properties of the RAP aggregates, such as gradation and angularity. These properties can be determined by one of several methods. The asphalt can be extracted from the RAP using solvent in a centrifuge, vacuum, or reflux extractor, or it can be burned off the aggregates in an ignition oven. When higher RAP contents are used there is a need to test binder properties of the RAP; it is recommended to extract and recover the binder and perform performance grade (PG) testing on the extracted binder. A combined procedure for extraction and recovery is given in AASHTO T 319, Quantitative Extraction and Recovery of Asphalt Binder from Asphalt Mixtures. This method was recommended because it was found to change the recovered binder properties less than other methods. For low RAP contents, 10% to 20%, it is not necessary to do this testing because there is not enough old, hardened RAP binder present to change the total binder properties (McDaniel and Anderson 2001).

Aggregate extracted from RAP, after determining the binder content, is analyzed to determine its gradation and other physical properties. An important property to be determined is bulk specific gravity (G_{sb}) of RAP aggregate. If the source of the RAP is known and original construction records are available, the G_{sb} value of the virgin aggregate from construction records may be used as the G_{sb} value of the RAP aggregate. However, if construction records are not available, effective specific gravity (G_{se}) of the RAP aggregate could be used instead of its bulk specific gravity. G_{se} can be calculated using RAP mixture maximum specific gravity, which can be easily determined by conducting AASHTO T209. For any given aggregate, G_{sb} is always smaller than G_{se}, so substituting G_{se} for G_{sb} of RAP will result in overestimating the combined aggregate bulk specific gravity. The error introduced by the substitution will magnify when

higher percentages of RAP are used. For this reason an alternative approach used is to assume a typical value for asphalt absorption based on experience with mix designs for the specific location and to calculate the G_{sb} of the RAP aggregate from the calculated G_{se} (Copeland 2010).

2.5 RAP Fractionation

Fractionation is the processing and separating of RAP materials into at least two sizes, typically a coarse fraction (+1/2 or +3/8 inches, or +12.5 or +9.5 mm) and a fine fraction (-1/2 or -3/8 inches, or -12.5 or -9.5 mm) (Copeland 2010). According to a survey in 2008 that received responses from 29 states, three states (South Carolina, Texas, and Alabama) had specifications for fractionating RAP, and three other states (Ohio, Wisconsin, and Illinois) were drafting specifications for fractionating RAP. These six states would allow higher amounts of RAP if it has been fractionated. A 2009 survey showed that 10 state transportation departments reported requiring fractionation. Those 10 states were Arizona, Georgia, Illinois, Kansas, North Carolina, Ohio, Texas, Utah, Wisconsin, and Washington, DC. Wisconsin allows an increase of 5% binder replacement for surface mixes if fractionation is used. Some states consider crushing and screening RAP over a single screen as fractionation, which is incorrect. One of the reasons fractionation is required is that it is believed to improve the consistency of RAP. However, data gathered by NCAT in 2008 and 2009 from the contractors across the United States showed that fractionated RAP stockpiles were no more consistent than processed unfractionated RAP stockpiles (Copeland 2010).

2.6 Mix Design Considerations with RAP

Superpave is the most common method of asphalt mixture design used in the U.S. for RAP mixes, including those that contain greater than 20% RAP. The percentage of RAP used in the mix may be selected by determining the contribution of the RAP toward the total mix by

weight, or by determining the contribution of the RAP binder toward the total binder in the mix by weight, while meeting volumetric properties requirements. Due to the stiffening effect of the aged binder in the RAP, the specified binder grade may need to be adjusted. The current national guideline, AASHTO M 323 Standard Specification for Superpave Volumetric Mixture Design, for determining binder grade adjustment in HMA mixes incorporating RAP has three tiers (Copeland 2010). Each tier has a range of percentages that represent the contribution of the RAP toward the total mix by weight. Up to 15% of RAP can be used without changing the virgin binder grade from that selected for the project location and conditions. When RAP content is between 15% and 25%, the high and low temperature grades of the virgin binder are both reduced by one grade to account for the stiffening effect of the aged binder (i.e. a PG 58-28 would be used instead of a PG 64-22). If more than 25% RAP is to be used in the HMA, blending charts are used to determine the appropriate virgin asphalt binder grade. For percentages of RAP greater than 25%, procedures developing a blending chart are provided in the appendix of AASHTO M 323. If a specific virgin asphalt binder grade must be used and the desired blended binder grade and recovered RAP properties are known, the allowable percentage of RAP is determined according to blending chart procedures (Copeland 2010).

The mix design process for mixes incorporating RAP is similar to the mix design containing all virgin materials. Once the RAP has been characterized, it can be combined with virgin aggregates for blend gradation for mix design purposes. To satisfy gradation requirements the selected blend must pass between the control points. Mixture volumetric requirements consist of voids in the mineral aggregate (VMA), voids filled with asphalt (VFA), dust proportion, and densification properties at 4% air voids at the N_{design} level. RAP material generally contains relatively high percentages of material passing a 0.075 mm (No. 200) sieve as result of the

milling and crushing operations. This limits the amount of RAP that can be used in a mix design and meet the volumetric properties. The percentage of asphalt binder in the RAP should also be considered when determining asphalt binder content. Asphalt binder content of the total mix batching includes virgin and reclaimed asphalt binder. The RAP material is to be heated separately at much lower temperatures (about 140 ^{0}F) than that needed for mixing and compaction. Virgin aggregates are heated enough so that when mixed, the resulting mix is within the required mixing temperature range. Heating the RAP at a lower temperature prevents additional hardening of the RAP asphalt binder. The recycled HMA should meet all test procedures and criteria as required for virgin materials (Al-Qadi et al. 2007, Brown et al. 2009).

2.7 High RAP Mix Design

For asphalt mixtures containing high RAP, appropriate grade for the virgin binder must be selected. Often, a softer virgin binder would be required to balance the aged, stiff binder in the RAP materials. A blending chart or blending equation is often used to determine the amount of RAP to be used if the virgin binder grade is known or to select the grade of virgin binder if the percentage of RAP binder is known. Procedures for using a blending chart are provided in the appendix of AASHTO M 323. In this process, RAP is subjected to a solvent extraction and recovery process to recover the RAP binder for testing. After that, physical properties and critical temperatures of the recovered RAP binder are determined. The critical high temperature (Tc(High)) based on the original Dynamic Shear Rheometer (DSR) and rolling thin film oven (RTFO) is determined. The high temperature PG of the recovered RAP binder is the lowest of the original DSR and RTFO DSR critical temperatures. The intermediate critical temperature (Tc(Int)) of the recovered RAP binder is determined by performing intermediate temperature DSR testing on the RTFO-aged recovered RAP binder, as if the RAP binder were aged in a

pressure-aging vessel. The critical low temperature (Tc(S) or Tc(m)) is determined based on bending beam rheometer testing on the RTFO-aged recovered RAP binder, or m-value. The low critical temperature (Tc(Low)) is the higher of the two low critical temperatures, Tc(S) or Tc(m). The low temperature PG of the recovered RAP binder is based on this low critical temperature value.

Once the physical properties and critical temperatures of the recovered RAP binder are known, there are two approaches for blending:

- Blending at a known RAP percentage, and
- Blending with a known virgin binder grade.

2.7.1 Blending at a Known RAP Percentage

When the desired final blended binder grade, the desired percentage of RAP, and the recovered RAP binder properties are known, the required properties of a virgin binder grade can then be obtained at each temperature (high, intermediate, and low) separately, as follows:

$$T_{virgin} = \frac{T_{blend} - (\%RAP \times T_{RAP})}{(1 - \%RAP)} \tag{2.1}$$

where:

Tvirgin = Critical temperature of virgin asphalt binder (high, intermediate, or low).

Tblend = Critical temperature of blended asphalt binder (final desired) (high, intermediate, or low).

%RAP = Percentage of RAP expressed as a decimal.

TRAP = Critical temperature of recovered RAP binder (high, intermediate, or low).

2.7.2 Blending with a Known Virgin Binder Grade

When the final blended binder grade, the virgin asphalt binder grade, and the recovered RAP properties are known, the allowable RAP percentage can be determined as follows:

$$\%RAP = \frac{T_{blend} - T_{virgin}}{T_{RAP} - T_{virgin}} \qquad (2.2)$$

The RAP percentage should be determined at high, intermediate, and low temperatures. The RAP content or range of contents meeting all three temperature requirements should be selected. NAPA, in partnership with AASHTO and FHWA, has published a guide for designing HMA mixtures with high RAP percentages (Copeland 2010). The guide includes information on evaluating RAP material, mix design, plant verification, and quality control (QC).

2.8 Performance of HMA Mixtures with RAP

In Louisiana, performance of five recycled and five conventional asphalt pavements used as control was evaluated over a five-year period. Laboratory and field evaluations conducted examined the pavements for pavement condition, serviceability, and structural analysis. It was observed that after six to nine years of service life, the recycled pavements containing reclaimed asphalt concrete materials, in the range of 20% to 50% by weight of mixture in both binder and wearing course, performed similar to the conventional pavements. No significant difference was reported in terms of pavement condition and serviceability rating (Paul 1995).

Five projects, each consisting of a recycled section and virgin (control) section, were evaluated in the state of Georgia. On each project, virgin and recycled mixtures used the same aggregates and were subjected to the same traffic and environmental conditions during service. In recycled mixtures, a RAP percentage between 10% to 25% was used. The performance

evaluation showed that after one to two and a half years in service, no significant rutting, raveling and fatigue cracking had occurred on any of the test sections. This indicates that both recycled and virgin mixtures performed equally well. Laboratory tests on field cores indicated comparable results for the virgin and recycled sections (Kandhal 1995).

A comprehensive evaluation was done to determine if the tiered approach of the Federal Highway Administration and Superpave RAP specifications are applicable to the materials obtained from Indiana, Michigan, and Missouri. In that study, laboratory mixtures were compared to plant-produced mixtures with the same materials at RAP contents between 15% and 25%. Additional mixtures were designed and tested in the laboratory, with RAP content up to 50%, to determine the effect of recycled materials on mix performance. Results showed that plant-produced mixes were similar in stiffness to laboratory mixtures at the same RAP content for the Michigan and Missouri samples. Mixtures with up to 50% RAP could be designed with Superpave, provided RAP gradation and aggregate quality were sufficient. Linear blending charts were found to be appropriate in most cases. It was observed that increasing RAP content in a mixture increased stiffness and decreased shear strain, indicating increased resistance to rutting. It was concluded that when RAP properties are appropriately accounted for in the material selection and mix design process, Superpave mixtures with RAP can perform very well (McDaniel 2002).

The Virginia Department of Transportation (VDOT) evaluated the effect of increased RAP percentages and relative mixture cost on projects using more than 20% RAP in three VDOT districts. Mix containing less than 20% RAP was also sampled and tested for comparison purposes. Laboratory test results showed no significant difference between higher RAP mixes and control mixes for fatigue, rutting, and moisture susceptibility. No construction problems

were reported for high RAP mixes. The researchers also concluded that slight price adjustments assessed were not due to use of high RAP percentages (Maupin et al. 2008).

Recently, another study investigated short- and long-term performance of RAP mixes and compared them with virgin HMA overlays used on flexible pavements. Data from 18 projects from the long-term pavement performance (LTPP) program in North America were analyzed. Projects ranged in age from eight to 17 years. Distress parameters considered were roughness, rutting, and fatigue cracking. Structural performance of overlaid sections was also evaluated with deflection data. Results of analysis of variance indicated the performance of RAP mixes and virgin HMA were not statistically different. Statistical similarity of deflections showed that RAP overlays can provide structural improvement that is equivalent to virgin HMA overlays (Carvalho et al. 2010).

A study conducted by the Florida Department of Transportation (FDOT) randomly sampled mix designs with more than 30% RAP content (RAP content ranged from 30% to 50%) (Musselman 2009). The projects were constructed from 1991 to 1999. The age of the pavements till rehabilitation was noted. The most common distress on these pavements was cracking. The average life of the virgin mixtures was 11 years. For 30%, 35%, 40%, 45%, and 50% RAP content mixes, average age ranged from 10 to 13 years. There was no significant difference in life between pavements with virgin asphalt mixture and those with 30% RAP. However, there is evidence of degraded performance of mixtures with more than 30% RAP.

2.9 Summary

A large amount of Reclaimed Asphalt Pavement (RAP) is generated each year in the United States. However, RAP is also the most recycled material. Use of RAP in hot-mix asphalt (HMA) has gained renewed interest because of high crude oil prices. Higher proportion of RAP

in HMA is being considered now. However, such mixtures tend to have some mixture design and performance challenges, especially due to variability in the source and material itself. In general, pavements with RAP mixes perform as well as the pavements with virgin mixes, provided RAP quantities are low.

Chapter 3 Methodology

3.1 Materials

According to AASHTO M 323, due to the stiffening effect of the aged binder in RAP, the specified binder grade of the virgin binder needs to be adjusted for asphalt mixes containing more than 15% RAP. The adjustments in this study were made using the blending chart used by the Kansas Department of Transportation (KDOT). In order to use the KDOT blending chart it is required to know the PG asphalt binder grade for the RAP and virgin asphalts. The RAP PG grade was acquired through a set of tests conducted by KDOT and the virgin PG grade was derived based on the climatic conditions and 20-year design traffic of the project. Based on KDOT's blending chart, the low sides of PG limits were -26 and -23 for 20% and 40% RAP, respectively. Consequently, -28 was chosen as the lower limit for the PG binder in this study. The high sides were 73 and 76 for 20% and 40% RAP respectively, which resulted in PG 70 for the binder grade high side. Therefore, PG 70-28 was chosen for all HMA mixes containing 20% to 40% RAP. Figure 3.1 shows the excel sheets used to determine higher and lower limits for the PG grade.

Virgin aggregates were mixed with three different percentages of RAP and selected virgin binder quantity. The combined blend had five different virgin aggregates: coarse crushed limestone (CS-1), fine crushed limestone (CS-1A), manufactured sand (MSD-1), crushed gravel (CG-5), and natural/river sand (SSG). The percentages of RAP added to the mix were 20%, 30%, and 40%. In addition, 30% and 40% FRAP mixes were made and tested to control the effect of RAP consistency on its performance.

Table 3.1 shows the gradation of various aggregates and RAP used in this study and the target values. Table 3.2 shows their percentages in each blend, and table 3.3 shows the

percentage of fine (minus 12.5 mm) and coarse aggregates (plus 12.5 mm) in the mixes containing FRAP. Since the RAP mix had a nominal maximum aggregate size (NMAS) of 9.5 mm (3/8 in.), a higher fraction of fine materials were used.

Figure 3.2 shows the 0.45 Power chart for all five virgin aggregates and RAP used in mix design, and table 3.4 shows the square mesh sieve analysis results for RAP.

KDOT BLENDING CHART CALCULATION FOR LOW SIDE OF THE BINDER

RAP & Virgin Binder Inputs		
Temperatures	**PG_{upper}**	**PG_{lower}**
PG_{RAP}	84	-16
PG_{virgin}	70	-28

* If utilizing FRAP insert total FRAP percent (coarse and fine) in Mix Design

Blending Chart Calculations

%RAP	PG_{blend} =
0.00	-28
5.00	-27
10.00	-27
15.00	-26
20.00	-26
25.00	-25
30.00	-24
35.00	-24
40.00	-23
45.00	-23
50.00	-22
55.00	-21
60.00	-21

KDOT BLENDING CHART CALCULATION FOR HIGH SIDE OF THE BINDER

RAP & Virgin Binder Inputs		
Temperatures	**PG_{upper}**	**PG_{lower}**
PG_{RAP}	84	-16
PG_{virgin}	70	-28

* If utilizing FRAP insert total FRAP percent (coarse and fine) in Mix Design

Blending Chart Calculations

%RAP	PG_{blend} =
0	70
5	71
10	71
15	72
20	73
25	74
30	74
35	75
40	76
45	76
50	77
55	78
60	78

Figure 3.1 KDOT's blending charts for PG grade adjustments

Table 3.1 Aggregate gradation and the target values

Sieve Sizes	% Retained					
	CS-1A	MSD-1	CG-5	SSG	RAP	Target
1.5						
1						
¾					0	0
½	0.00	0.00	0.00		2	0-10
3/8	0.09	0.00	0.32	0.00	6	10 Min
#4	73.85	1.39	5.86	3.67	20	
#8	99.05	43.48	29.07	19.17	36	42-61
#16	99.52	72.33	57.47	42.59	53	
#30	99.64	86.25	75.91	64.95	67	
#50	99.72	94.51	88.06	85.69	80	
#100	99.78	98.12	95.03	97.70	90.1	
#200	100.00	99.88	99.93	99.79	99.79	90-98

Table 3.2 Aggregate percentages in different mixes

RAP (%)	CS-1 (%)	CS-1A (%)	MSD-1 (%)	CG-5 (%)	SSG (%)
20	20	12	12	16	20
30	16	15	13	12	14
40	12	13	13	12	10

Table 3.3 Percentage of fine and coarse aggregates in FRAP

% of FRAP in Mix	% of RAP plus 12.5 mm (1/2 inch)	% of RAP minus 12.5 mm (1/2 inch)
30	9	21
40	12	28

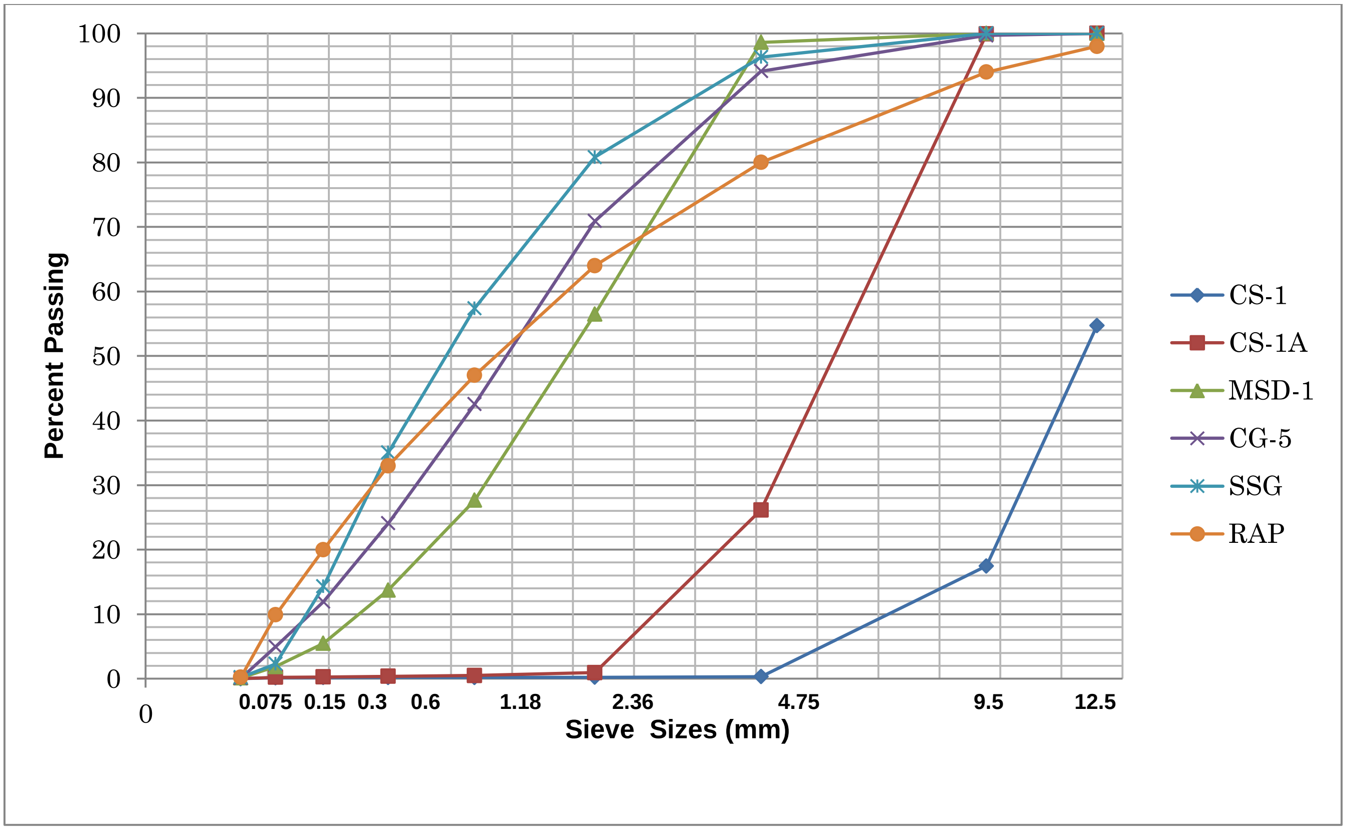

Figure 3.2 0.45 Power chart for the aggregates used in mix design

Table 3.4 Square mesh sieve analysis results for RAP

Sieve Sizes mm	% Retained	Cumulative % retained	% Passing
19	0.00	0.00	100.00
12.5	2.00	2.00	98.00
9.5	4.00	6.00	94.00
4.75	14.00	20.00	80.00
2.36	16.00	36.00	64.00
1.18	17.00	53.00	47.00
0.6	14.00	67.00	33.00
0.3	13.00	80.00	20.00
0.15	7.00	87.00	13.00
0.075	3.00	90.00	10.00

3.2 Laboratory Testing

Superpave mix designs were developed for HMA with 12.5 mm (1/2 inch) Nominal Maximum Aggregate Size (NMAS). Virgin aggregates were blended, heated, and finally mixed with the heated virgin binder and RAP. Binder was heated to the recommended mixing temperature (309 - 320 ^{0}F) based on the virgin PG binder grade and RAP was heated to 122 ^{0}F. To make up for the low temperature of RAP, virgin aggregates were heated to 350 ^{0}F before being mixed with the binder and RAP. All mixes were aged at the recommended compacting temperature (270 - 281 ^{0}F) for two hours before compaction in the Superpave Gyratory Compactor. Bulk specific gravity and unit weight of compacted asphalt mixtures (G_{mb}) and theoretical maximum specific gravity of asphalt mixtures (G_{mm}) were determined based on the AASHTO T-166 (KT-15) and AASHTO T-209 (KT-39) test methods, respectively. Table 3.5 shows the volumetric properties of all five different mixes and KDOT requirements for 12.5 mm Nominal Maximum Aggregate Size (NMAS). All mixes in this study met these requirements. In general, the total asphalt contents for these mixtures were lower than Superpave mixes with all virgin materials. This was due to the fact that most coarse aggregates in Kansas are soft lime stones with high absorption. The use of 20% to 40% RAP and FRAP considerably reduces total asphalt content used for the recycled mixes. This is reinforced by the fact that the mixtures containing 40% RAP and FRAP have the lowest asphalt contents.

Table 3.5 Volumetric properties of five different mixes and KDOT requirements

Mix Design	Total asphalt content (%)	Virgin asphalt added (%)	Asphalt contained in RAP (%)	%Air void @ N_{des}	%VMA	%VFA	Dust to binder ratio	% Gmm @ N_{ini}	% Gmm @ N_{des}
20% RAP	4.7	3.59	1.11	3.9	14.1	71.6	0.6	88.5	96.0
30% RAP	4.8	3.14	1.66	4.0	14.0	71.3	0.6	88.0	96.0
40% RAP	4.3	2.07	2.23	4.0	14.2	71.9	0.7	87.9	96.0
30% FRAP	4.3	2.63	1.67	4.0	14.1	71.6	0.7	87.7	96.0
40% FRAP	4.4	2.13	2.27	4.1	14.3	72.0	0.7	87.8	96.0
KDOT Superpave volumetric mix design requirements for 12.5 mm NMAS				4.0%	Min. 14	65-78	0.6-1.2	Max. 90.5	Max. 98

3.2.1 Hamburg Wheel-Tracking Device

The Hamburg Wheel-Tracking Device (HWTD) is a common tool to assess stripping and rutting susceptibility of HMA mixtures. This device, manufactured by PMW, Inc. of Salina, Kansas (fig. 3.3), was used in this study to see how a higher percentage of RAP and FRAP affects rutting and stripping susceptibility of Superpave mixtures containing RAP/FRAP. The tests were performed following the Tex-242-F test method of the Texas Department of Transportation (TxDOT). The samples were made using the Superpave gyratory compactor following AASHTO T-324 specification. The HWTD can test two specimens simultaneously. The device is operated by rolling a pair of steel wheels across the surface of specimens submerged in a water bath held at 50^{o}C. The wheels have a diameter of 204 mm (8 inches) and width of 47 mm (1.85 inches). The device operates at approximately 50 wheel passes/min and

the load applied by each wheel is approximately 705±22 N (158±5 lbs). Specimens used in this test were compacted to 7±1% air voids using a Superpave gyratory compactor. The specimens were 150 mm (6 inches) in diameter and 62 mm (2.4 inches) in height. Rut depth was measured automatically and continuously at 11 different points along the wheel path of each sample with a linear variable differential transformer (LVDT) with an accuracy of 0.01 mm (0.0004 inch). HWTD automatically ends the test if the preset number of cycles is reached or if the rut depth measured by the LVDTs reaches a value of 20 mm (0.8 inch) for an individual specimen.

Figure 3.3 Hamburg Wheel-Tracking Device (HWTD) Test Setup

Once the test was completed, performance of the HMA was interpreted from the various parameters derived from the typical test output shown in figure 3.4. These parameters are

assumed to describe HMA failure due to weakness in the aggregate structure, inadequate binder stiffness, and/or moisture damage.

The post-compaction consolidation is the deformation in millimeters at 1,000 wheel passes and occurs rapidly during the first few minutes of the test. This test is referred to as the post-compaction consolidation because it is assumed that the wheel is densifying the mixture within the first 1,000 wheel passes.

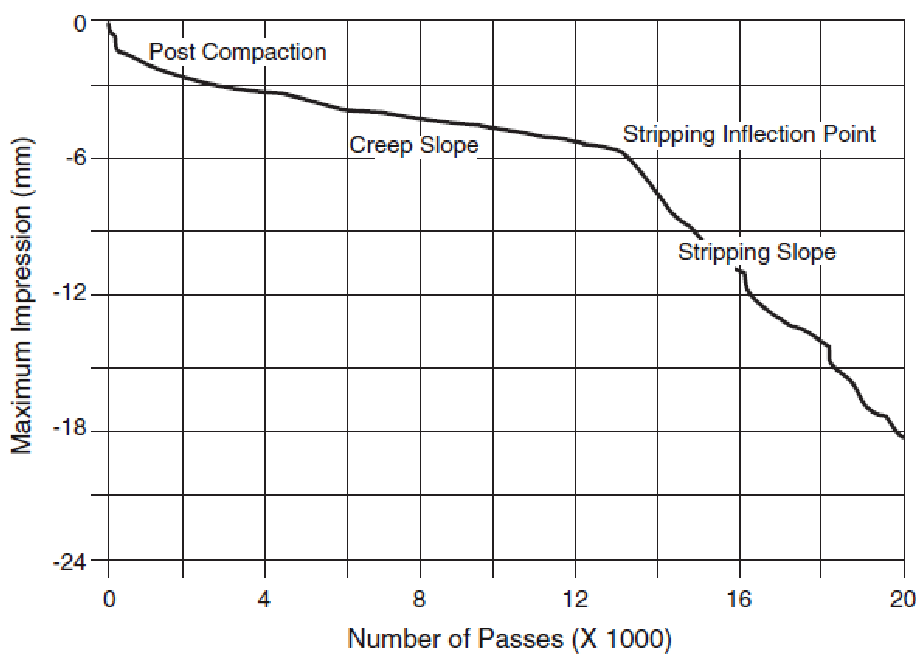

Figure 3.4 Typical Hamburg test curve and its major characteristics

The creep slope is the inverse of the deformation rate within the linear region of the deformation curve after post compaction and prior to stripping (if stripping occurs). The creep slope measures rutting susceptibility. It measures the accumulation of permanent deformation primarily due to a mechanism other than moisture damage.

The stripping slope is the inverse of the deformation rate within the linear portion of the deformation curve, after the stripping began. The stripping inflection point is the number of wheel passes corresponding to the intersection of the creep slope and the stripping slope. The stripping slope measures the accumulation of permanent deformation due to moisture damage. It is used to estimate the relative resistance of the HMA sample to moisture-induced damage. In other words, this is the number of wheel passes at which moisture damage starts to dominate performance. The lower the inverse stripping slope the more severe the moisture damage *(*Yildirim et al 2007).

3.2.2 Moisture Susceptibility Test

The moisture susceptibility test evaluates the effect of saturation and accelerated moisture conditioning on compacted HMA samples utilizing freeze-thaw cycles. Kansas Test Method KT-56, Resistance of Compacted Asphalt Mixtures to Moisture-Induced Damage, commonly known as the modified Lottman test in Kansas, was used to evaluate moisture susceptibility in this study. For this test, specimens should be 150 mm (6 inches) in diameter and 95 mm (3.75 inches) in height. Six specimens were compacted to 7±0.5% air voids using the Superpave gyratory compactor. After compaction and air void determination, the six specimens were subdivided into two subsets of three samples so that the average air void contents of the two subsets were approximately equal. Diameter and thickness of the specimens were measured before further testing. Three specimens were selected as a control set and tested dry (without conditioning). The other subset of three specimens was conditioned by subjecting those to a partial vacuum saturation of 70% to 80% of air voids by placing them in a vacuum container filled with water, so that at least 25 mm (1 inch) of water was covering them. A partial vacuum of 250 mm to 650 mm of Hg was applied to the container for a short time. After the degree of saturation for each

specimen was verified and met the test protocol, the conditioned samples were individually wrapped with a plastic film, and placed and sealed in a zip-lock bag with 10 mL water. Samples were then placed in a freezer for a minimum of 16 hours at -18^{o}C. After freezing, the samples were thawed by placing them in a hot water bath for 24±1 hrs at 60^{o}C. The conditioned samples were then removed from the hot water bath and the saturated surface dry (SSD) mass was recorded, and mass under water was also measured. All conditioned and unconditioned (sealed in plastic wrap) specimens were then placed in a water bath for two hours at 25^{o}C. Final diameter and thickness of conditioned samples was measured after removing them from the water bath before testing. The specimens were tested at a loading rate of 51 mm/minute and peak loads were recorded. The tensile strength was computed using equation 3.1 (Hossain et al. 2010). Figure 3.5 shows the different steps in this test method.

$$S = \frac{2000P}{\Pi t D} \tag{3.1}$$

Where

S = tensile strength (kPa), P = maximum load (N),

t = specimen thickness (mm), and D = specimen diameter (mm).

(a)

(b)

(c)

(d)

Figure 3.5 Modified Lottman Test Steps: (a) Vacuum Saturation, (b) Specimen in Freezer, (c) Specimens in Hot Water Bath, and (d) Specimen in Testing Frame

The tensile strength ratio (TSR) was used to denote HMA resistance to the detrimental effects of moisture. It is defined as the ratio of average tensile strength retained after freeze-thaw conditioning (average tensile strength of conditioned specimens) to average tensile strength of unconditioned samples. The percent tensile strength ratio was computed using equation 3.2.

$$TSR = \frac{S_2}{S_1} \times 100 \qquad (3.2)$$

where

S_1 = average tensile strength of unconditioned subset, and

S_2 = average strength of conditioned subset.

The KDOT and Superpave criterion for acceptable minimum tensile strength ratio is 80% (Hossain et al. 2010).

3.2.3 Dynamic Modulus Test

The HMA resistance to permanent deformation or rutting and fatigue cracking can be characterized using the dynamic modulus and phase angle of HMA. In order to measure the dynamic modulus (E*) and phase angle (δ) a sinusoidal axial compressive load was applied to the cylindrical specimen at a sweep of testing frequencies. The dynamic modulus (E*) was calculated by dividing the peak-to-peak stress by the peak-to-peak strain, as shown in figure 3.6. For mixtures to be rut resistant and exhibit higher stiffness at high temperature, a greater (E*) value and a lower phase angle are desirable (FHWA 2010).

A Dynamic Modulus test was conducted on specimens cored and trimmed to the size of 4 inches in diameter and 6 inches in height from a sample 6 inches in diameter and 11 inches in height. The taller samples were fabricated using the Gyratory Compactor and were compacted to an air void level of 7±1%. The 7±1% is the core air void and was chosen to make the comparison between the HWTD and Dynamic Modulus test results possible (as mentioned earlier, Hamburg plugs are compacted at 7±1% air void).

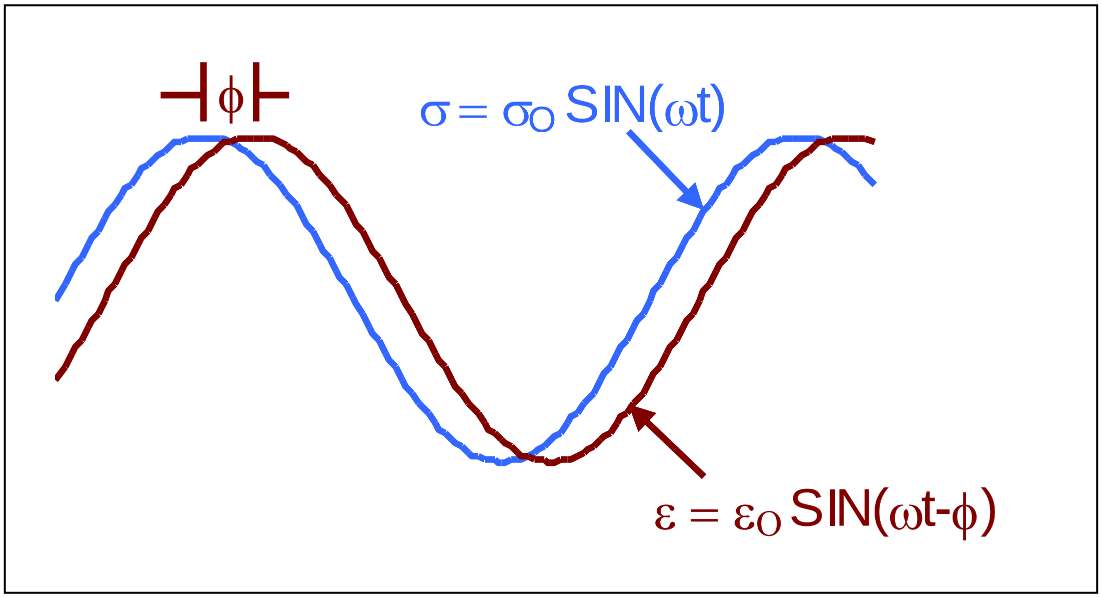

Figure 3.6 Sinusoidal loading in the Dynamic Modulus test

Figure 3.7 shows a tall sample that was fabricated in the Superpave gyratory compactor and a Dynamic Modulus test sample that was cored and trimmed from it. The dynamic load ranges between 10 and 690 kPa (1.5 to 100 psi); the higher load is used for lower test temperatures. The effective test temperature varies and the design frequency ranges between 0.1 Hz and 25 Hz. The dynamic load should be adjusted to obtain axial strains between 50 and 150 micro-strains. Specimen ends were treated to reduce friction. The specimen was then placed in the testing chamber at the desired test temperature, and it was left to stabilize before the sample was tested. The test specimen was first preconditioned with 200 cycles at 25 Hz using the target dynamic load. Then the specimen was loaded using specified temp, frequency and number of cycles. The loading stress and recoverable axial strain were computed for each frequency. The Dynamic Modulus and the phase angle were then calculated.

Figure 3.7 Superpave gyratory compactor sample, and the cored and trimmed sample

In this study, Dynamic Modulus tests were conducted using a Universal Testing Machine (UTM-25) following AASHTO TP: 62-03 (Standard Test Method for Determining Dynamic Modulus of Hot Mix Asphalt Concrete Mixtures). To accomplish the Dynamic Modulus test, three LVDTs were used for axial deformation data collection, providing an estimated limit of accuracy of 13.1%. Figure 3.8 shows specimen setup and LVDT connections.

Some minor modifications in test temperature were made because at the highest temperature (54^oC) glue and the samples started softening and LVDTs could not be kept attached to the samples, whereas at the lowest temperature (-10^oC) UTM and LVDT's started freezing. As the result, in this study, three temperatures (4, 21, and 37^oC) and six loading frequencies (0.1, 0.5, 1, 5, 10, and 25 Hz) were used.

Figure 3.8 Sample set up in UTM machine with attached LVDTs

Chapter 4 Results and Analysis

4.1 Hamburg Wheel-Tracking Device Test Results

The Hamburg Wheel-Tracking test was conducted on three replicate specimens for each mix. Table 6.1 lists the number of passes for each mix at failure. All mixes being tested in this study either failed before 40,000 passes or reached 40,000 passes with rut depth very close to 20 mm. For the second replicates of 20% RAP and 40% FRAP, the rut depth at 40,000 passes was very low (3.5 mm and 12.6 mm, respectively) when compared to other replicates of the same mix. Therefore, those results were not taken into consideration when comparing the results. When one replicate of 30% FRAP samples was being tested, the machine stopped due to a power failure. Thus, the final number of passes could not be obtained. This also happened for a few other instances.

4.1.1 Hamburg Wheel-Tracking Device Test Outputs (Creep Slope, Stripping Slope and Stripping Inflection Point)

To better understand the HWTD performance test results, the test outputs other than number of passes to failure, shown in table 4.1, need to be studied too. Figures 4.2, 4.3, and 4.4 show Creep Slope, Stripping Slope, and Stripping Inflection Points for the mixes in this study, respectively. Figure 4.1 shows that the mix with 20% RAP had the highest number of passes, and then the number of passes decreased as the RAP percentage increased in the mix. Contrary to the RAP, when FRAP was added to the mix, the number of passes at 40% FRAP was considerably higher than 30% FRAP. However, the number of passes with 30% and 40% FRAP were lower than that for the mixture with 20% RAP.

Table 4.1 Number of passes in HWTD test for five different mixes

Mix design	First replicate Number of passes			Second replicate Number of passes			Third replicate Number of passes			Average of three replicates
	Left Wheel	**Right Wheel**	**Average**	**Left Wheel**	**Right Wheel**	**Average**	**Left Wheel**	**Right Wheel**	**Average**	
20% RAP	40,000	40,000	40,000	-			28,871	24,829	26,850	33,425
30% RAP	38,449	32,575	35,512	30,078	23,056	26,567	23,208	24,292	23,750	28,610
40% RAP	20,600	21,200	20,900	31,700	34,167	32,934	23,822	21,800	22,811	25,548
30% FRAP	30,290	27,860	29,075	-	29,275	29,275	24,385	-	24,385	27,578
40% FRAP	39,800	27,762	33,781	-			31,820	28,292	30,056	31,919

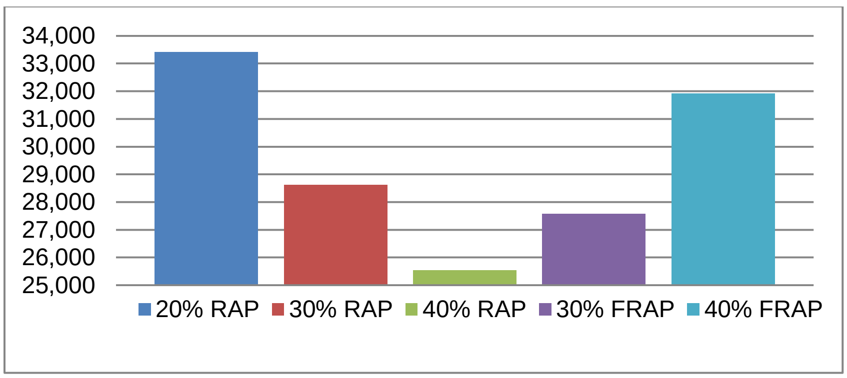

Figure 4.1 Comparison of average number of passes for five different mixes

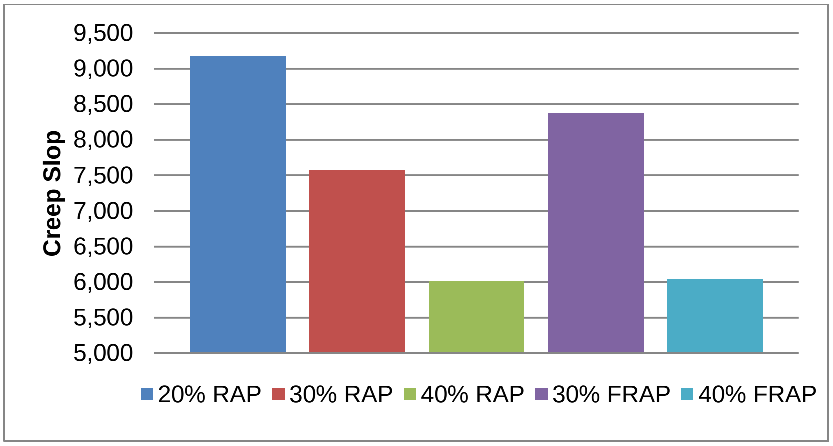

Figure 4.2 Effect of varying RAP percentage on Creep Slope (Passes/mm)

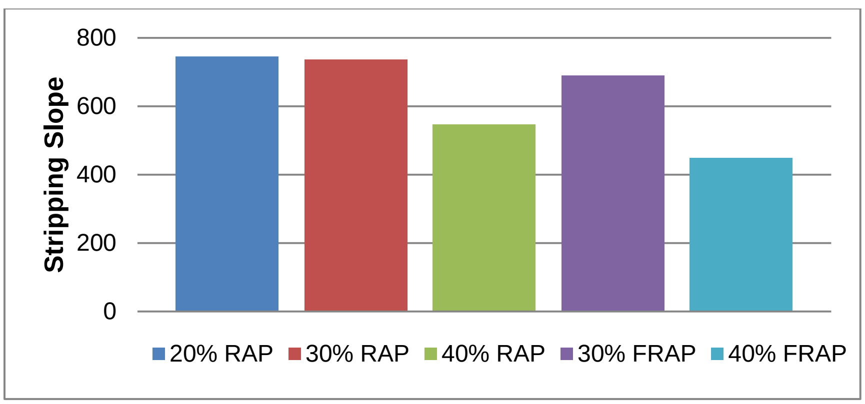

Figure 4.3 Effect of varying RAP percentage on Stripping Slope

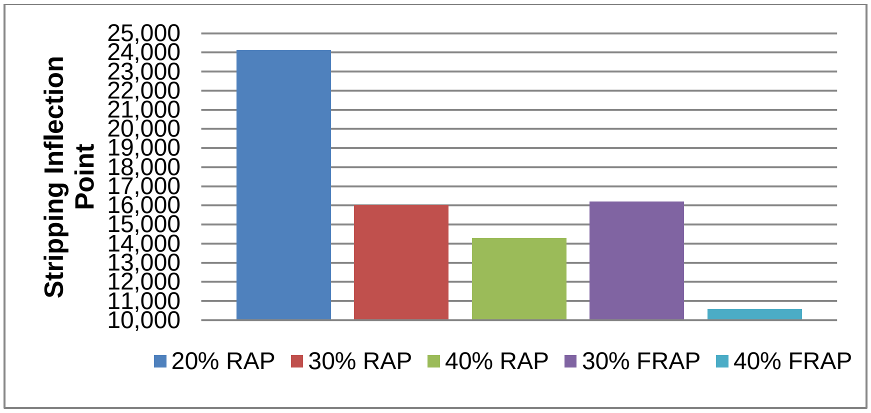

Figure 4.4 Effect of varying RAP percentage on Stripping Inflection Point

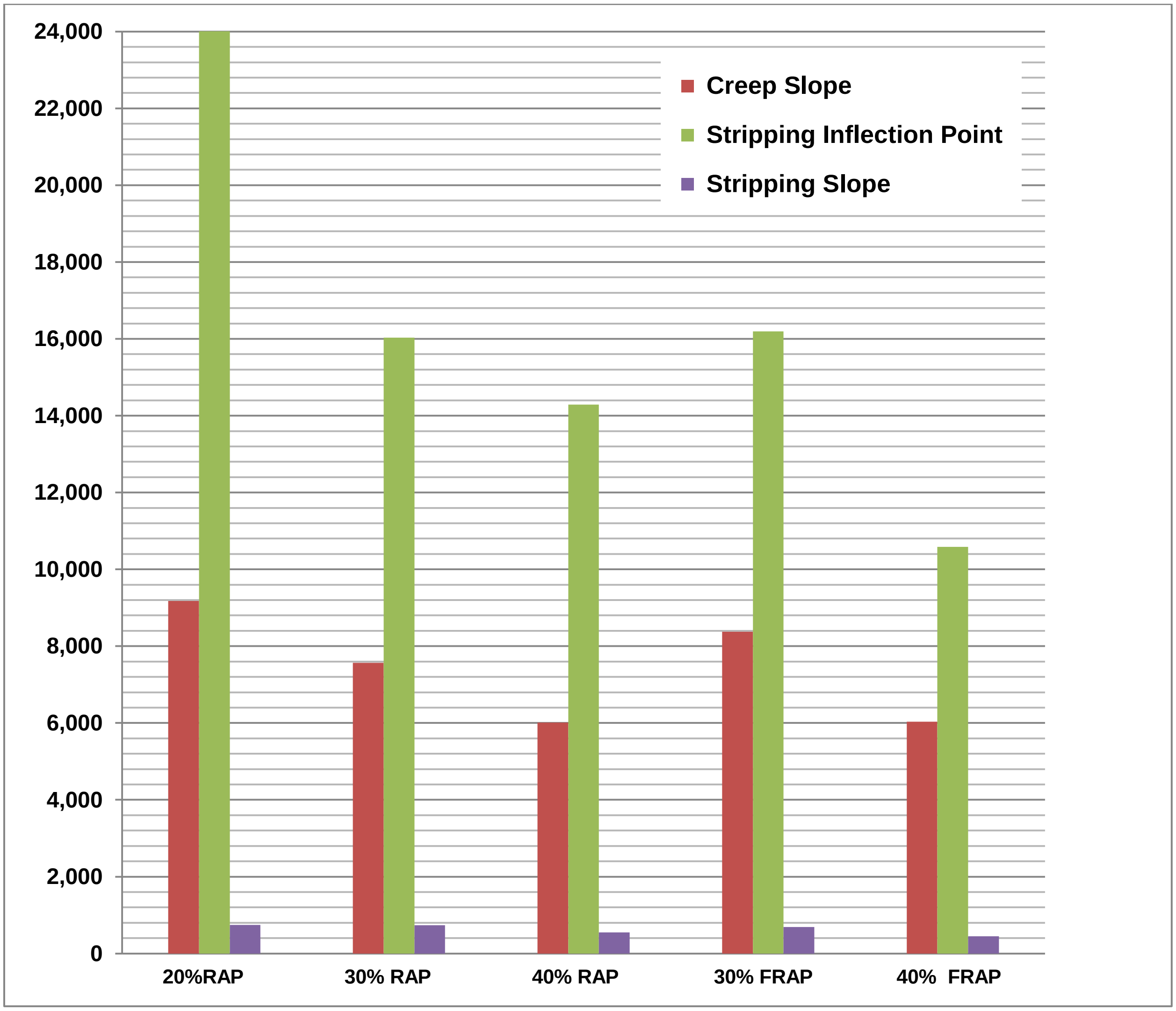

Figure 4.5 Effect of varying RAP percentage on HWTD output parameters

As it can be seen in figures 4.2 through 4.5, 30% FRAP did not show any better performance when compared to 30% RAP. The trend was also similar for 40% RAP and 40% FRAP. Although the number of wheel passes was higher for 40% FRAP when compared to 30% FRAP (fig. 4.2), the creep slope was decreasing with an increasing percentage of FRAP. This may indicate that the mixture with a higher percentage of FRAP is more vulnerable to rutting failure. When 40% FRAP and 40% RAP mixtures were compared, except for number of wheel passes, all other output parameters were comparable, indicating that FRAP may not have improved mixture performance.

Figures 4.6 to 4.9 show the number of wheel passes and HWTD output parameters based on the percentage of virgin binder added to the mix. The results indicate that the amount of virgin binder played a role in the rutting and stripping resistance of the mixture containing RAP or FRAP. The best performance in terms of wheel passes to 20 mm rutting was obtained for the mixture containing the highest amount of virgin binder. For the mixtures with 40% RAP or FRAP, the mixture with FRAP performed better. This phenomenon can be confirmed by looking at the creep (rutting) slope and stripping slope.

Figure 4.7 illustrates the creep slopes of all mixes. It appears that the best rutting resistance was obtained by the mixture with the highest amount of virgin binder. Rutting resistance of the mixtures with 30% RAP and FRAP were somewhat comparable.

Figure 4.8 shows that stripping started earliest for the mixture with 40% FRAP. This probably was due to the presence of a larger fraction of fine materials in this mixture. However, figure 4.9 also indicates that the stripping slope was the lowest for the mixture with 40% FRAP, which had a similar amount of virgin binder as the mixture with 40% RAP. Yet, this mixture had

a higher number of wheel passes to 20-mm rut depth. This phenomenon cannot be explained at this time.

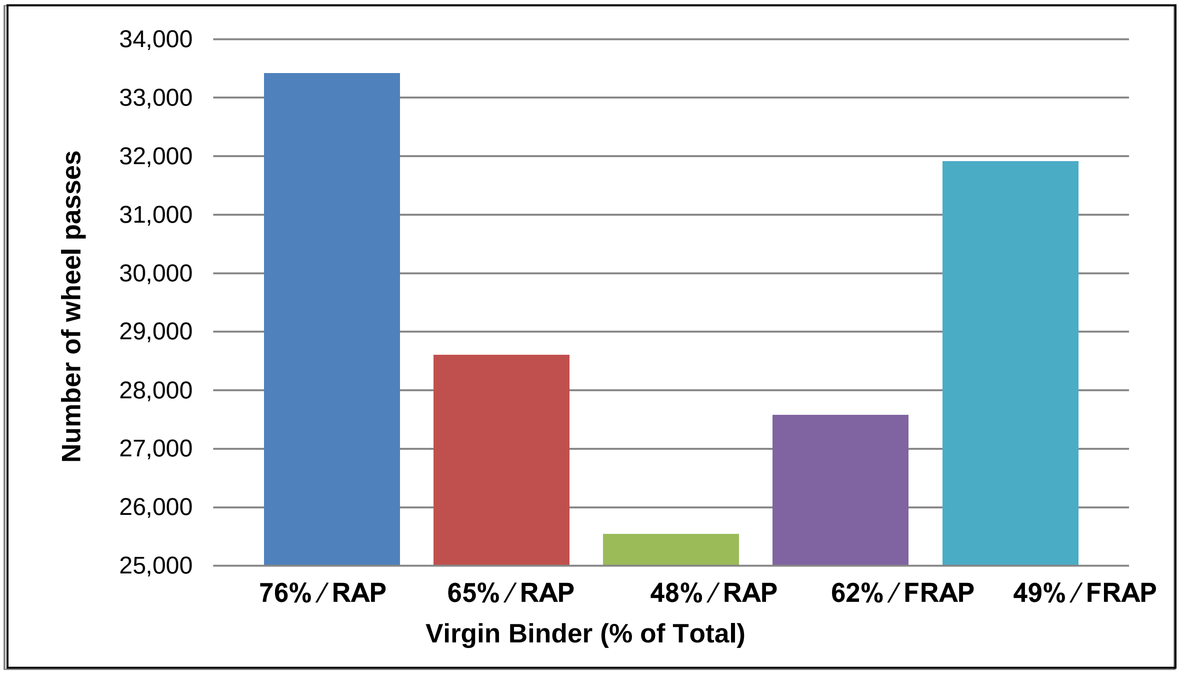

Figure 4.6 Number of wheel passes for five different mixes

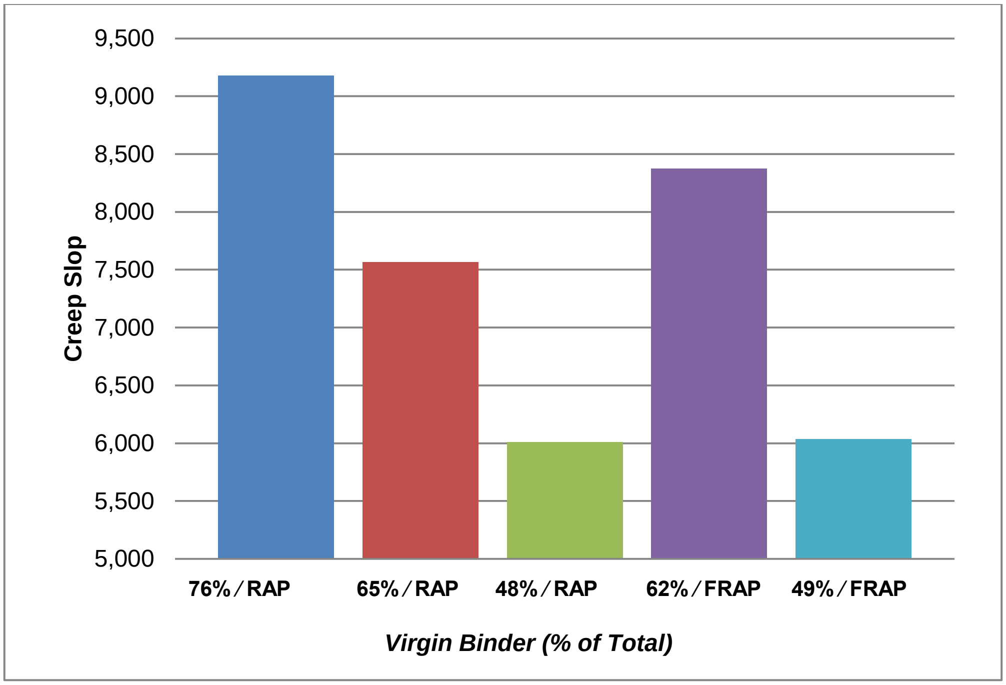

Figure 4.7 Creep Slope (Passes/mm) for five different mixes

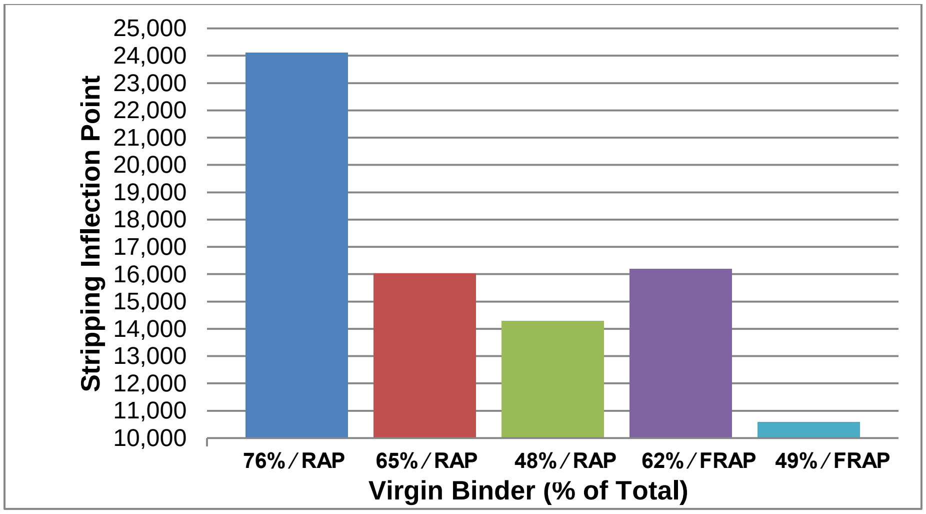

Figure 4.8 Stripping Inflection Point for five different mixes

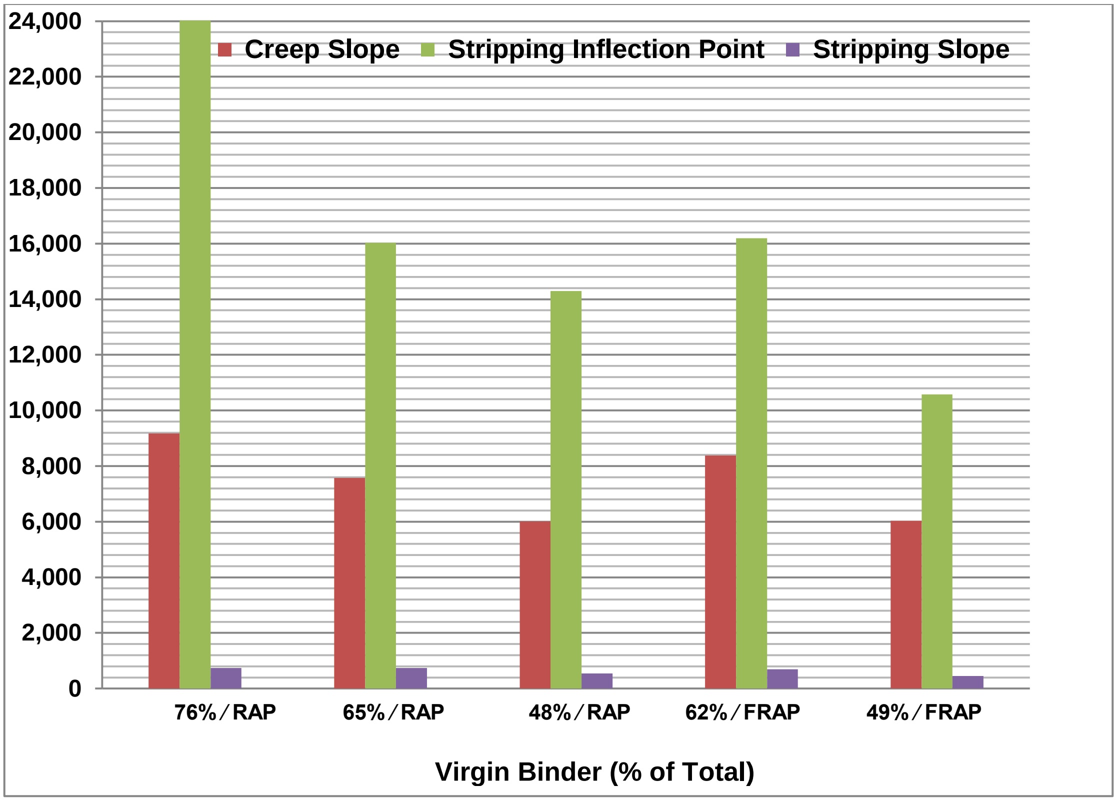

Figure 4.9 HWTD Output Parameters for five different mixes

4.1.2 Statistical Analysis of HWTD Output Data

The Analysis of Variance (ANOVA) was conducted by taking all the parameters in the HWTD test output as response variables and by taking different mixes as "treatments." Statistical analysis software, SAS was used for this purpose. Table 4.2 shows the summary results. The results show that overall the effect of mixture type on the total number of wheel passes was not significant, i.e. the mixture performance in the HWTD test could not be explained only by the mixture type. However, there were significant differences between the number of wheel passes to failure for mixtures with 20% RAP and 40% RAP. Both creep slope and stripping slope were also unaffected by mixture type, but there was a significant difference in creep slopes between 20% and 40% RAP and between 40% RAP and 40% FRAP. However, treatment type did

significantly affect the stripping inflection point, which refers to when stripping started in the HWTD test. The mixture with 20% RAP showed significantly different behavior than other mixtures with RAP and FRAP.

Table 4.2 Treatment vs. Response Variable in ANOVA

Treatment	Response Variable	Significant @ $\alpha = 0.1$	Significant @ $\alpha = 0.05$	Significant Difference between Treatments Ho: $\mu_i = \mu_j$
20% RAP	No. of Wheel Passes	N p value = 0.2844>0.1	N p value = 0.2844>0.05	20% RAP & 40% RAP (p value<0.1)
30% RAP				
40% RAP				
30% FRAP				
40% FRAP				
20% RAP	Creep Slope	N p value = 0.2426>0.1	N p value = 0.2426>0.05	20% RAP & 40% RAP 40% RAP & 40% FRAP (p value<0.1)
30% RAP				
40% RAP				
30% FRAP				
40% FRAP				
20% RAP	Stripping Inflection Point	Y p value = 0.0445<0.1	Y p value = 0.0445<0.05	20% RAP & 30% RAP 20% RAP & 40% RAP 20% RAP & 30% FRAP 20% RAP & 40% FRAP (p value<0.1)
30% RAP				
40% RAP				
30% FRAP				
40% FRAP				
20% RAP	Stripping Slope	N p value = 0.5455>0.1	N p value = 0.5455>0.05	None
30% RAP				
40% RAP				
30% FRAP				
40% FRAP				

4.2 Moisture Susceptibility Test (KT-56) Results

Table 4.3 presents the tensile strength and tensile strength ratios (TSRs) for different percentages of RAP and FRAP in the mix. The Kansas Department of Transportation (KDOT) criterion for acceptable TSR is 80% and above. It means that if the average tensile strength of conditioned plugs is greater than or equal to the 80% of the average tensile strength of un-conditioned plugs, then the set has passed the minimum requirement. The TSR is not the only important parameter in the indirect tensile strength test; it is also of significant importance to compare conditioned and unconditioned sets in each mix design to find out how increasing the RAP percentage and adding FRAP will affect the HMA performance. Table 6.2 and figure 6.6 show how the HMA performance was affected by increasing RAP percentage and by adding FRAP to the mix.

Table 4.3 shows that as the percentage of RAP increased in the mix, the TSR decreased and mixes with FRAP performed worse than the mixes with RAP. The increment of TSR implies that mixes with high RAP will not perform well in freeze-thaw condition. It should be mentioned that although the TSR decreased as the RAP percentage increased, all mixes with RAP passed the KDOT criteria for the KT-56 test. The TSR for 30% RAP and 30% FRAP was exactly the same, and the TSR was slightly lower for 40% FRAP when compared to 40% RAP, and the mixture with 40% FRAP failed to meet the minimum required value (80%).

Table 4.3 Indirect Tensile Strength Results for Conditioned and Unconditioned Plugs

Mixed Design	Sample ID	Conditioned	Unconditioned	% Air Voids @ N des	Tensile Strength (lbs)	Average Tensile Strength (lbs)	Tensile Strength Ratio (%)	Passed	Failed
20% RAP	a			7.17	4,023	3,990	90	√	
	b			7.09	3,930				
	c			7.42	4,018				
	e			7.12	4,431	4,430			
	f			7.22	4,428				
	g			7.26	4,431				
30% RAP	a			6.69	4,199	4,257	86	√	
	b			6.83	3,817				
	c			6.96	4,756				
	e			7.06	4,402	4,964			
	f			6.53	5,259				
	g			6.67	5,231				
40% RAP	a			6.53	4,559	4,425	82	√	
	b			6.52	4,277				
	c			6.85	4,440				
	e			6.56	5,221	5,391			
	f			6.48	5,654				
	g			6.81	5,297				
30% FRAP	a			6.96	3,777	3,963	86	√	
	b			9.87	4,136				
	c			6.54	3,976				
	e			7.04	4,512	4,616			
	f			6.76	4,447				
	g			6.54	4,890				
40% FRAP	a			7.13	4,115	3,872	78		√
	b			6.74	3,772				
	c			6.66	3,730				
	e			6.96	5,105	4,934			
	f			6.86	5,151				
	g			6.78	4,547				

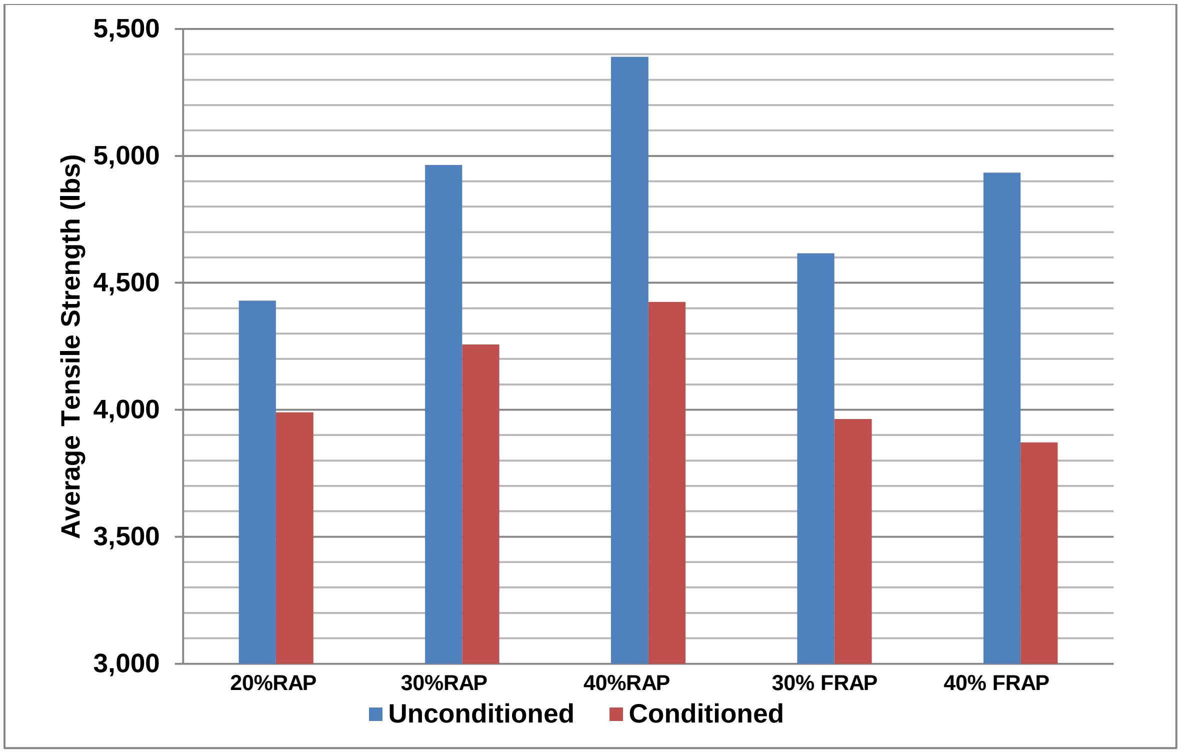

Figure 4.10 Tensile strength results for five different mixes

The indirect tensile strength, however, increased as the RAP percentage increased in the mix, and it was the highest at 40% RAP. When RAP and FRAP are compared, FRAP mixes had slightly lower indirect tensile strength. It seems like adding FRAP to the mix neither helped with TSR nor with the indirect tensile strength.

4.3 Dynamic Modulus Test Results

The Dynamic Modulus test results were automatically recorded with the operation software in the UTM-25 machine. For each mix, three replicates were made and tested. Figure 4.11 shows the typical output of a test on a sample. Tables 4.4 and 4.5 list the dynamic modulus and phase angle results, respectively, at six frequencies and three test temperatures.

Figures 4.12, 4.13, and 4.14 illustrate the average dynamic modulus values. The dynamic modulus and phase angle were affected by both temperature and loading frequency. At low

temperature and high loading frequency, the asphalt mixture was elastic and had a high dynamic modulus. At high temperature and low loading frequency, the asphalt mixture was more viscous and had a low elastic modulus. As can be expected, dynamic modulus values were higher at lower temperature and lower at higher temperature. It was also observed that the dynamic modulus decreased as the loading frequency changed from 25 Hz to 0.1 Hz.

The five different mixes followed almost the same trend for dynamic modulus values at 4^{o}C and 37^{o}C. Figure 4.12 shows that the 20% RAP mixture had the highest dynamic modulus, followed by 30% FRAP and 30% RAP. The mixtures with 40% RAP and 40% FRAP had almost a similar dynamic modulus at 4^{o}C. At 37^{o}C, 40% RAP had slightly higher dynamic modulus values than that for mix with 40% FRAP. This trend in dynamic modulus values of different mixtures is similar to that observed in the HWDT test. RAP and FRAP mixture moduli are very similar while the modulus goes down as the percentage of these materials increases in the mix.

The dynamic moduli trends for RAP and FRAP at 4^{o}C and 37^{o}C were also observed at 21^{o}C with one exception – the mixture with 20% RAP did not show a higher modulus when compared to 30% RAP/FRAP and 40% RAP/FRAP. This is more likely due to the fact that the mix with 20% RAP had the highest amount of PG 70-28 binder. Thus its behavior at the extreme ends of the temperature range is quite different than the other mixtures.

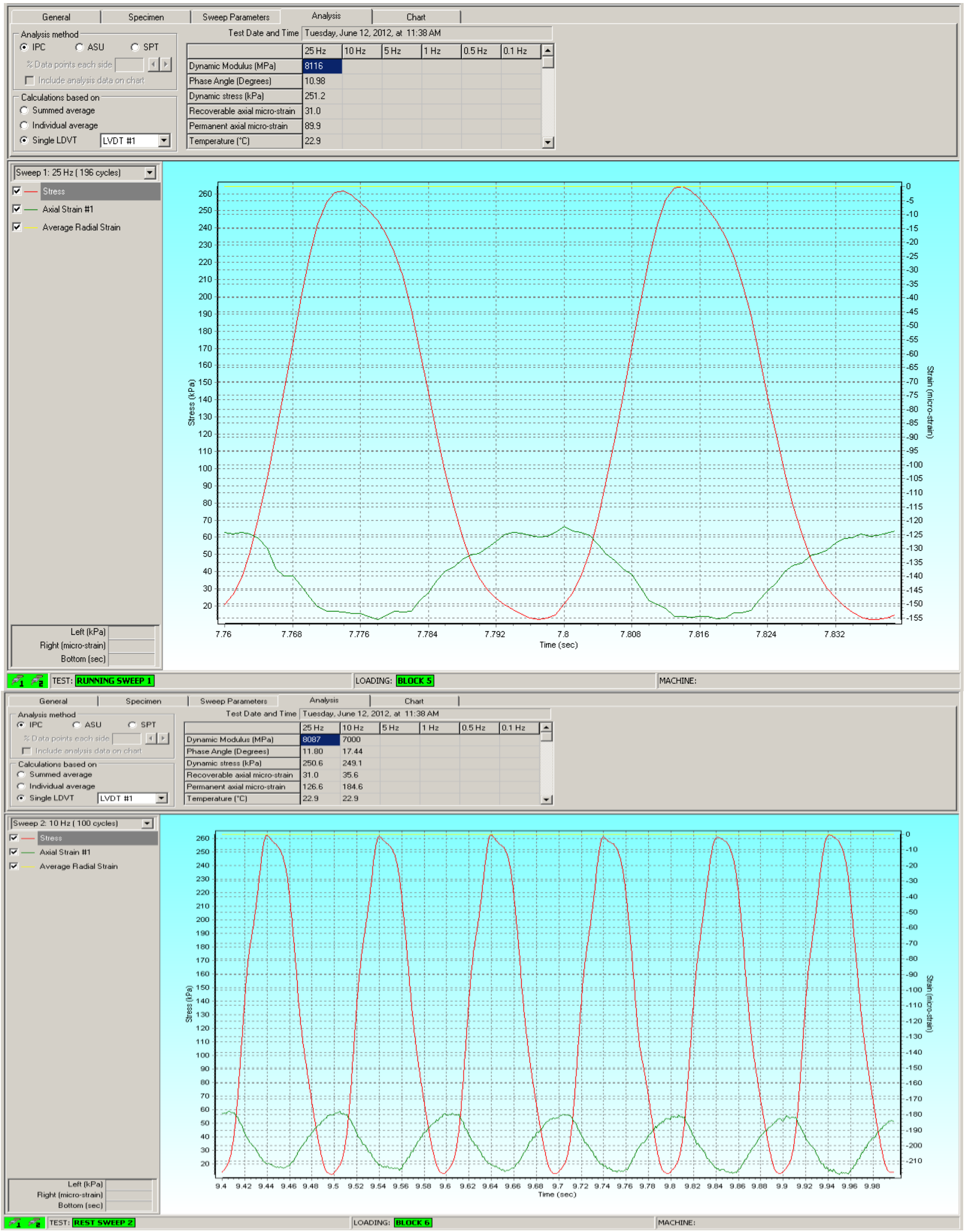

Figure 4.11 Typical output in the Dynamic Modulus Test

Table 4.4 Dynamic Modulus (MPa) results for five different mixes at three different temperatures

Mix Design	Sample ID	4°C						21°C						37°C					
		25 Hz	10 Hz	5 Hz	1 Hz	0.5 Hz	0.1 Hz	25 Hz	10 Hz	5 Hz	1 Hz	0.5 Hz	0.1 Hz	25 Hz	10 Hz	5 Hz	1 Hz	0.5 Hz	0.1 Hz
20% RAP	*S1*	27,740	24,816	23,171	18,935	18,013	15,024	8,074	7,084	6,452	4,945	4,401	3,114	18,524	13,604	10,746	6,274	5,327	3,035
	S2	25,607	21,587	19,295	15,518	14,645	11,354	7,177	6,463	5,874	4,508	4,044	2,872	14,773	10,122	7,754	4,005	3,315	1,790
	S3	17,989	16,897	15,544	12,311	11,457	8,509	14,072	12,999	11,698	8,423	7,553	5,322	9,073	9,746	8,313	5,774	4,869	3,400
	Average	23,779	21,100	19,337	15,588	14,705	11,629	9,774	8,849	8,008	5,959	5,333	3,769	14,123	11,157	8,938	5,351	4,504	2,742
	SD	5,126	3,982	3,814	3,313	3,278	3,266	3,749	3,608	3,209	2,145	1,931	1,350	4,759	2,127	1,591	1,192	1,055	844
	C.V%	0.22	0.19	0.20	0.21	0.22	0.28	0.38	0.41	0.40	0.36	0.36	0.36	0.34	0.19	0.18	0.22	0.23	0.31
30% RAP	*IV-I*	10,199	9,287	9,583	8,297	7,693	6,121	13,564	13,468	13,009	11,168	9,480	8,143	4,159	3,587	3,111	2,124	1,786	1,156
	IV-II	11,431	10,755	10,298	8,940	8,696	7,285	13,764	11,601	10,412	7,731	7,052	4,850	7,354	5,842	5,100	3,644	3,155	2,163
	S2	20,818	19,935	19,631	17,578	16,797	13,038	14,013	13,109	12,078	9,049	8,509	5,879	2,266	2,033	1,786	1,279	1,062	719
	Average	14,149	13,326	13,171	11,605	11,062	8,815	13,780	12,726	11,833	9,316	8,347	6,291	4,593	3,821	3,332	2,349	2,001	1,346
	SD	5,808	5,771	5,606	5,183	4,992	3,704	225	991	1,316	1,734	1,222	1,685	2,572	1,915	1,668	1,198	1,063	741
	C.V%	0.41	0.43	0.43	0.45	0.45	0.42	0.02	0.08	0.11	0.19	0.15	0.27	0.56	0.50	0.50	0.51	0.53	0.55
40% RAP	*S2*	11,015	10,541	10,056	8,730	8,240	6,642	27,598	21,013	18,608	12,342	11,786	7,191	6,099	3,864	2,999	1,769	1,431	865
	S(2-1)	11,223	10,834	10,218	8,546	7,948	6,125	14,723	12,898	10,942	6,546	6,036	3,090	2,937	2,276	1,864	1,180	964	646
	S(2-2)	13,589	12,632	12,055	10,281	9,596	7,649	8,984	7,757	6,989	4,776	4,317	2,650	5,264	4,309	3,612	2,228	1,858	1,194
	Average	11,942	11,336	10,776	9,186	8,595	6,805	17,102	13,889	12,180	7,888	7,380	4,310	4,767	3,483	2,825	1,726	1,418	902
	SD	1,430	1,132	1,110	953	879	775	9,532	6,683	5,908	3,958	3,912	2,504	1,639	1,069	887	525	447	276
	C.V%	0.12	0.10	0.10	0.10	0.10	0.11	0.56	0.48	0.49	0.50	0.53	0.58	0.34	0.31	0.31	0.30	0.32	0.31
30% FRAP	*S1*	21,551	19,611	19,849	18,000	17,240	15,401	14,025	12,682	11,915	10,127	9,195	7,661	2,665	2,353	2,105	1,588	1,346	995
	SII	13,543	12,298	12,028	10,036	9,657	8,090	14,808	12,233	10,871	8,173	7,566	5,145	8,200	6,537	5,709	4,023	3,504	2,353
	SIII	10,600	9,665	9,967	8,397	7,903	6,321	13,144	12,765	12,157	10,072	8,760	7,016	5,801	4,279	3,344	1,880	1,451	853
	Average	15,231	13,858	13,948	12,144	11,600	9,937	13,992	12,560	11,648	9,457	8,507	6,607	5,555	4,390	3,719	2,497	2,100	1,400
	SD	5,667	5,153	5,213	5,137	4,962	4,814	832	286	683	1,113	843	1,307	2,776	2,094	1,831	1,330	1,217	828
	C.V%	0.37	0.37	0.37	0.42	0.43	0.48	0.06	0.02	0.06	0.12	0.10	0.20	0.50	0.48	0.49	0.53	0.58	0.59
40% FRAP	*S1*	10,794	10,287	9,757	8,307	7,869	6,159	13,688	11,065	9,478	6,267	5,791	3,277	4,217	3,184	2,567	1,468	1,178	761
	S2	13,175	12,431	11,766	9,922	9,364	7,409	24,992	19,652	17,523	12,084	11,465	6,992	5,199	4,206	3,465	2,161	1,788	1,101
	S(2-2)	12,314	11,826	11,344	9,479	9,068	7,395	8,087	7,000	6,147	4,299	3,739	2,280	2,683	2,254	2,021	1,446	1,192	827
	Average	12,094	11,515	10,956	9,236	8,767	6,988	15,589	12,572	11,049	7,550	6,998	4,183	4,033	3,215	2,684	1,692	1,386	896
	SD	1,206	1,105	1,059	834	792	718	8,611	6,459	5,849	4,048	4,002	2,483	1,268	976	729	407	348	180
	C.V%	0.10	0.10	0.10	0.09	0.09	0.10	0.55	0.51	0.53	0.54	0.57	0.59	0.31	0.30	0.27	0.24	0.25	0.20

Table 4.5 Phase Angle (degrees) results for five different mixes at three different temperatures

Mix Design	Sample ID	4°C						21°C						37°C					
		25 Hz	10 Hz	5 Hz	1 Hz	0.5 Hz	0.1 Hz	25 Hz	10 Hz	5 Hz	1 Hz	0.5 Hz	0.1 Hz	25 Hz	10 Hz	5 Hz	1 Hz	0.5 Hz	0.1 Hz
20% RAP	*S1*	7.62	10.18	14.88	13.73	15.89	18.51	10.41	13.41	15.33	18.25	21.47	24.57	32.67	40.92	40.72	37.76	41.24	39.35
	S2	26.08	11.15	11.37	11.33	11.96	13.14	7.33	13.37	14.94	18.04	20.91	24.35	68.14	50.52	53.94	50.92	52.61	47.93
	S3	14.54	16.06	22.19	19.79	20.66	21.86	15.04	19.63	24.90	23.31	25.81	27.70	29.28	33.67	36.33	36.90	38.94	39.32
	Average	16.08	12.46	16.15	14.95	16.17	17.84	10.93	15.47	18.39	19.87	22.73	25.54	43.36	41.70	43.66	41.86	44.26	42.20
	SD	9.33	3.15	5.52	4.36	4.36	4.40	3.88	3.60	5.64	2.98	2.68	1.87	21.52	8.45	9.17	7.86	7.32	4.96
	C.V%	0.58	0.25	0.34	0.29	0.27	0.25	0.36	0.23	0.31	0.15	0.12	0.07	0.50	0.20	0.21	0.19	0.17	0.12
30% RAP	*IV-I*	3.76	12.60	10.06	11.72	14.16	17.66	5.47	21.05	23.72	24.98	27.91	26.45	13.68	25.45	26.36	27.07	31.37	31.69
	IV-II	15.96	10.30	10.17	14.06	14.98	16.99	8.22	10.90	13.97	16.60	19.75	21.99	19.59	24.79	26.60	28.94	33.50	35.72
	S2	2.39	3.23	5.07	6.68	8.43	11.02	11.30	12.41	16.74	19.38	21.69	23.43	13.00	18.83	22.15	24.14	28.39	29.12
	Average	7.37	8.71	8.43	10.82	12.52	15.22	8.33	14.79	18.14	20.32	23.12	23.96	15.42	23.02	25.04	26.72	31.09	32.18
	SD	7.47	4.88	2.91	3.77	3.57	3.66	2.92	5.48	5.02	4.27	4.26	2.28	3.62	3.65	2.50	2.42	2.57	3.33
	C.V%	1.01	0.56	0.35	0.35	0.28	0.24	0.35	0.37	0.28	0.21	0.18	0.10	0.23	0.16	0.10	0.09	0.08	0.10
40% RAP	*S2*	0.67	6.37	7.89	10.58	12.24	15.97	22.69	21.73	21.22	23.52	28.20	30.09	25.60	25.80	27.45	28.48	32.30	29.70
	S(2-1)	0.48	5.24	7.42	9.88	12.35	15.87	39.65	21.23	29.13	33.49	38.06	41.48	21.70	27.83	29.55	28.69	31.82	28.48
	S(2-2)	10.15	12.62	14.82	16.88	18.15	20.95	9.74	16.53	17.78	21.69	26.20	29.03	22.07	27.37	27.99	28.68	32.78	30.43
	Average	3.77	8.08	10.04	12.45	14.25	17.60	24.03	19.83	22.71	26.23	30.82	33.53	23.12	27.00	28.33	28.62	32.30	29.54
	SD	5.53	3.98	4.14	3.86	3.38	2.90	15.00	2.87	5.82	6.35	6.35	6.90	2.15	1.06	1.09	0.12	0.48	0.99
	C.V%	1.47	0.49	0.41	0.31	0.24	0.17	0.62	0.14	0.26	0.24	0.21	0.21	0.09	0.04	0.04	0.00	0.01	0.03
30% FRAP	*S1*	8.74	11.90	12.39	13.34	14.70	15.82	11.67	13.13	14.40	16.11	18.15	19.47	10.14	15.36	17.81	20.57	25.61	27.40
	SII	15.90	7.74	10.14	11.56	12.21	14.03	7.70	10.44	14.20	17.67	19.97	23.14	19.73	25.34	26.07	29.39	33.46	35.69
	SIII	3.76	12.5	8.77	12.1	14.07	17.23	4.82	15.85	19.21	21.35	24.54	25.26	30.22	36.45	38.54	38.73	41.10	35.75
	Average	9.47	10.71	10.43	12.33	13.66	15.69	8.06	13.14	15.94	18.38	20.89	22.62	20.03	25.72	27.47	29.56	33.39	32.95
	SD	6.10	2.59	1.83	0.91	1.29	1.60	3.44	2.71	2.84	2.69	3.29	2.93	10.04	10.55	10.44	9.08	7.75	4.80
	C.V%	0.64	0.24	0.18	0.07	0.09	0.10	0.43	0.21	0.18	0.15	0.16	0.13	0.50	0.41	0.38	0.31	0.23	0.15
40% FRAP	*S1*	2.24	6.52	8.72	10.90	12.50	15.98	19.52	23.66	26.19	30.69	34.70	38.39	25.18	32.72	33.20	32.48	36.25	31.65
	S2	10.22	13.36	15.01	16.46	18.43	21.09	21.24	20.60	21.85	24.11	28.46	30.17	20.59	26.11	27.40	28.81	32.68	29.81
	S(2-2)	1.33	6.56	8.12	10.72	12.50	16.50	11.80	17.44	19.36	22.92	27.77	30.03	12.99	21.16	23.34	24.61	30.84	29.97
	Average	4.60	8.81	10.62	12.69	14.48	17.86	17.52	20.57	22.47	25.91	30.31	32.86	19.59	26.66	27.98	28.63	33.26	30.48
	SD	4.89	3.94	3.82	3.26	3.42	2.81	5.03	3.11	3.46	4.19	3.82	4.79	6.16	5.80	4.96	3.94	2.75	1.02
	C.V%	1.06	0.45	0.36	0.26	0.24	0.16	0.29	0.15	0.15	0.16	0.13	0.15	0.31	0.22	0.18	0.14	0.08	0.03

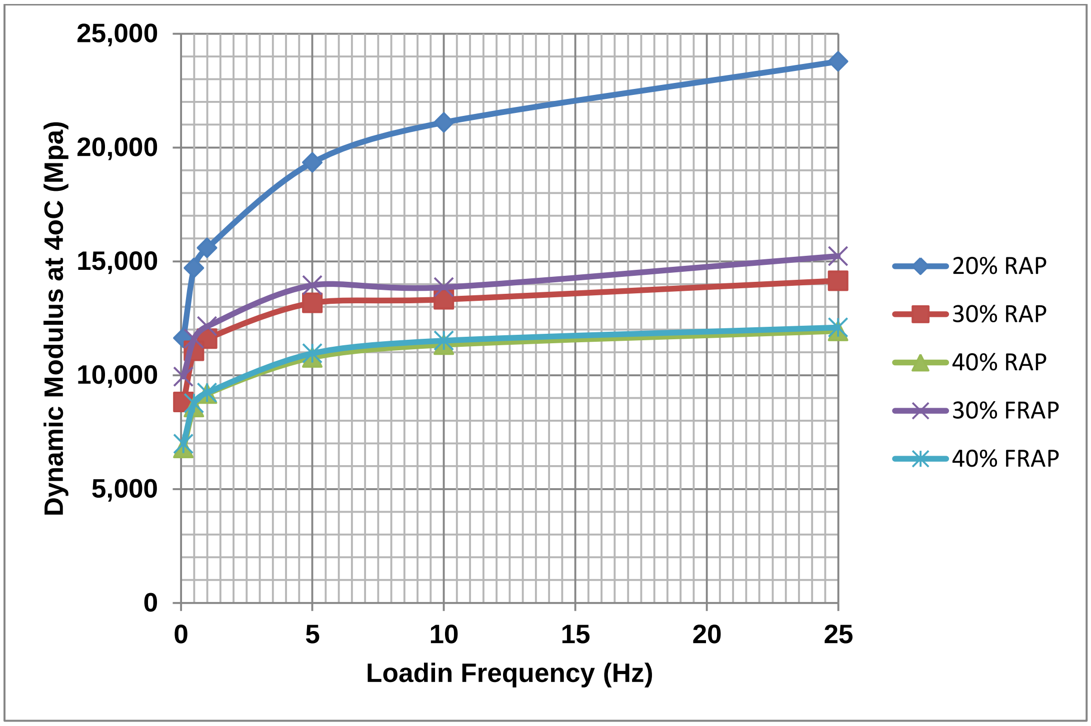

Figure 4.12 Dynamic Modulus results for five different mixes at 4^{o}C.

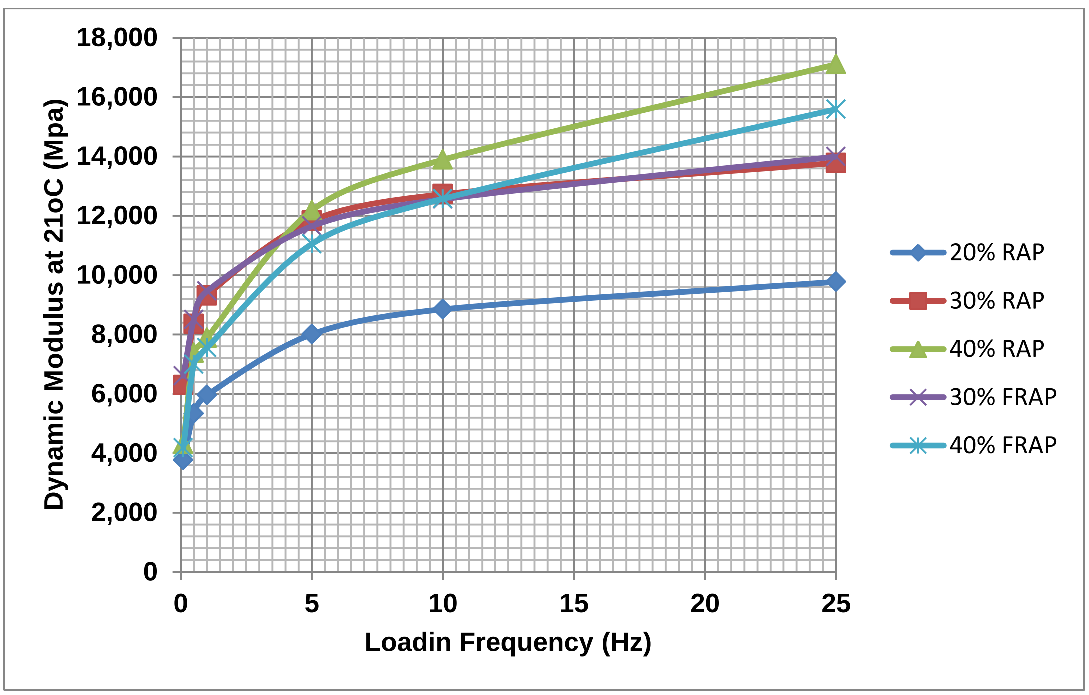

Figure 4.13 Dynamic Modulus results for five different mixes at 21^{o}C.

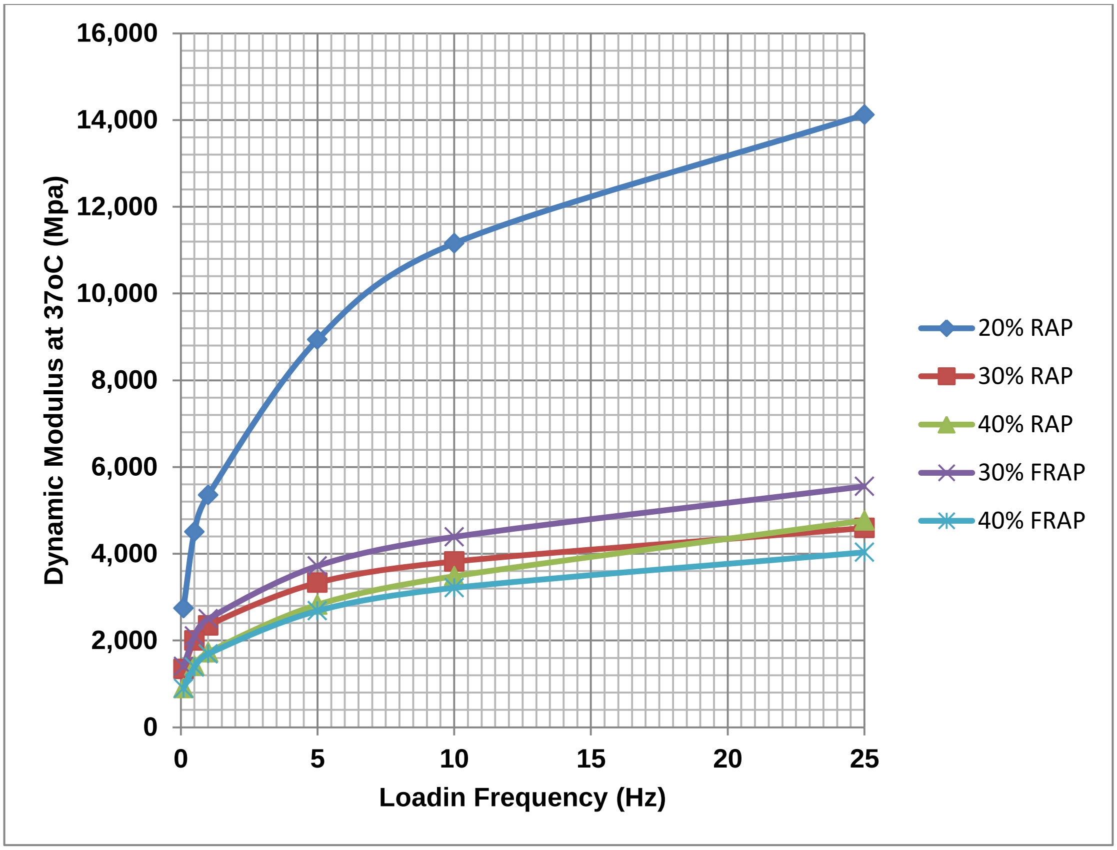

Figure 4.14 Dynamic Modulus results for five different mixes at 37^{o}C

4.4 Possible Effect on Pavement Performance

The possible effect of these RAP and FRAP mixes can be understood by studying the performance models in the newly released Mechanistic-Empirical Pavement Design Guide (MEPDG) performance models for flexible pavements.

4.4.1 Permanent Deformation Models

MEPDG offers models for predicting permanent deformation in each pavement layer. The average vertical resilient strain in each layer/sublayer is computed for each analysis period of the entire design period with a linear elastic analysis program for each axle load configuration (NCHRP 2004). The rutting distress is predicted in absolute terms. The incremental distress is computed for each analysis period and is directly accumulated over the entire design life of the

pavement. The model used for predict rutting of the asphalt mixes is based upon a field calibrated statistical analysis of repeated load permanent deformation test results. The model is:

$$\varepsilon_p / \varepsilon_r = k_1 * 10^{-3.4488} * T^{1.5606} * N^{0.479244} \quad (4.1)$$

$$k_1 = (C_1 + C_2 * depth) * 0.328196^{depth} \quad (4.2)$$

$$C_1 = -0.1039 * h_{ac}^{2} + 2.4868 * h_{ac} - 17.342 \quad (4.3)$$

$$C_2 = 0.0172 * h_{ac}^{2} - 1.7331 * h_{ac} + 27.428 \quad (4.4)$$

where:

ε_0 , β and ρ – Material properties;

ε_r – Resilient strain of the asphalt material as a function of mix properties, temperature and time rate of loading (in/in);

ε_p – Accumulated plastic strain at N repetitions of load (in/in);

T – Temperature (deg F);

N – Number of traffic repetitions; and

h_{ac} – Thickness of the layer/sublayer.

The final calibrated model parameters were derived from the permanent deformation data collected on 88 LTPP sections in 28 states (NCHRP 2004). The model developed above was derived based on observed deformation of in-service pavement structures and hence, is

empirical. However, a desirable feature is that it includes the effect of temperature on the dynamic modulus for the asphalt concrete layers.

Equation 4.1 indicates that accumulated plastic strain due to load repetitions is directly proportional to the resilient strain of the asphalt material that in turn, is a function of mix properties, temperature and time rate of loading (in/in). For a given temperature and rate of loading, higher stiffness or dynamic modulus of asphalt mixture will result in lower resilient strain. Rutting is usually considered critical at higher service temperature of the pavement. Thus dynamic modulus results in figure 4.13 indicate that asphalt mixtures with high RAP or FRAP would result in higher rutting than mixture with 20% RAP. This is most likely due to the higher aged binder content in the high RAP/FRAP mixtures. This observation matches the performance of these mixtures in the HWTD tests. Again, performance of high RAP and FRAP mixtures in the pavements would be fairly identical as far as rutting is concerned.

4.4.2 Load-Associated Cracking Models

Load-associated cracking is one of the most common asphalt concrete pavement distresses. The repeated traffic loads result in repeated tensile stresses in the bound layers. Under these repeated strains, fatigue cracks initiate at locations where the largest tensile strains and stresses develop. These critical locations depend on many factors, such as pavement structural configuration, layer stiffness, and load configuration (area of load distribution, magnitude of stresses at the tire-pavement interface, etc.). After crack initiation at the critical locations, the repeated traffic load effect causes the cracks to propagate through the entire layer. These cracks allow water infiltration, thereby reducing the overall performance of the pavement. Many pavement structural models assume that cracks initiate at the bottom of the asphalt concrete surface layer and then propagate upward. These cracks are named bottom-up fatigue cracks.

MEPDG considers the alligator cracking as bottom-up fatigue cracking (NCHRP 2004). MEPDG also takes another type of fatigue cracking, now known as top-down cracking, which are longitudinal cracks in the wheel path. The cause of top-down cracking is hotly debated but they do seem to exist especially at hot-weather locales.

MEPDG adopted Miner's hypothesis to estimate fatigue damage (NCHRP 2004):

$$D = \sum_{i=1}^{T} \frac{n_i}{N_i} \tag{4.5}$$

where,

D = damage,

T = total number of periods,

n_i = actual traffic for period i, and

N_i = allowable repetitions to failure under conditions prevailing in period i.

The most commonly used model to predict the number of repetitions to fatigue cracking involves both tensile strain and mix stiffness. One well known model proposed by the Asphalt Institute is based on constant stress criterion. The final fatigue model used in MEPDG can be obtained by numerical optimization and other modes of comparison as below:

$$N_f = 0.00432 * k_1^{'} * C(1/\varepsilon_t)^{3.9492}(1/E)^{1.281} \tag{4.6}$$

where:

$C = 10^M$ and $M = 4.84*[V_b / (V_a+V_b) – 0.69]$

V_b = effective binder volumetric content (%), and

V_a = air voids (%).

The parameter $k_1^{'}$ was introduced to account for different asphalt layer thicknesses and is given by below for bottom-up cracking.

$$k_1^{'} = \frac{1}{0.000398 + [0.003602/(1+e^{(11.02-3.49*h_{ac})})]} \tag{4.7}$$

For top-down cracking, it is given by:

$$k_1^{'} = \frac{1}{0.01 + [12.00/(1+e^{(15.676-2.8186*h_{ac})})]} \tag{4.8}$$

Finally, the transfer function to estimate fatigue cracking from fatigue damage is expressed in the equations below for bottom-up and top-down cracking respectively.

Bottom-up cracking:

$$F.C. = \left(\frac{6000}{1+e^{(C_1*C'_1+C_2*C'_2*\log10(D*100))}}\right)*\left(\frac{1}{60}\right) \tag{4.9}$$

Where:

F.C.=bottom-up fatigue cracking, percent lane area,

D= bottom-up fatigue damage,

C1 = 1.0,

C2 = 1.0,

C’1= -2 * C’2, and

$C^{'}_{2}$ = -2.40874-39.748*(1+h_{ac})$^{-2.856}$

Top-down cracking:

$$F.C. = 1000*10.56 / [1+ e^{(7 - 3.5*\log10(100*D))}] \quad (4.10)$$

where,

F.C.= top-down fatigue cracking, ft/mile, and

D= top-down fatigue damage.

The fatigue cracking model for asphalt concrete was calibrated based on data from 82 LTPP sections located in 24 states, using 441 observations for alligator cracking and 408 data points for longitudinal cracking. The bottom-up cracking was calculated as a percentage of lane area while the longitudinal cracking was expressed in terms of linear feet per mile of pavement (NCHRP 2004).

Equation 4.6 indicates that for a given tensile strain and volumetric properties of an asphalt mix, the number of repetitions to fatigue cracking is inversely related to the stiffness of an asphalt mix. Fatigue cracking is considered a normal temperature phenomenon. Figure 4.13 illustrates the dynamic modulus results at 21^{o}C for the five mixes in this study. It is obvious that asphalt mixtures with higher proportions of RAP and FRAP will be more susceptible to fatigue cracking. The higher the RAP/FRAP content the worse would be the performance. Again, the fatigue performance of pavements containing mixtures with RAP and FRAP are expected to be comparable.

Chapter 5 Conclusions and Recommendations

5.1 Conclusions

The objective of this research was to determine the impacts of having higher percentages of RAP and FRAP on mixture performance while meeting the current requirements of Superpave mix design. The following conclusions were drawn based on this study:

1. Superpave mixtures with 20% RAP carried the highest number of wheel passes till 20 mm rut depth in the Hamburg Wheel Tracking Device (HWTD) test. The number of passes decreased as the RAP percentage increased in the mix. When FRAP was added to the mix, the number of passes at 40% FRAP was considerably higher than 30% FRAP. However, the number of passes with 20% and 30% FRAP were lower than that for the mixture with 20% RAP. Besides, other parameters obtained from the HWTD test outputs consistently indicated that mixture with 20% RAP performed the best and there were no discernible differences in performance of 30% and 40% RAP and FRAP mixtures. These observations were largely supported by the statistical analysis of HWTD test outputs. This trend was also confirmed by analyzing the results in terms of virgin binder content. Given the large difference in performance between the mixtures with 20% RAP (76% virgin binder) and those with 30% RAP (62% virgin binder) or 30% FRAP, it can be surmised that minimum virgin binder content for the mixtures with RAP or FRAP should be about 75%. This finding may support the specifications of some state departments of transportation that require a minimum of 70% virgin binder.

2. The Modified Lottman test results indicated that as the percentage of RAP increased in the mix, the Tensile Strength Ratio (TSR) decreased and mixes with FRAP performed worse than the mixes with RAP. The TSRs for 30% RAP and 30% FRAP were exactly the same, and the TSR was slightly lower for 40% FRAP when compared to 40% RAP. The mixture with 40%

FRAP failed to meet the minimum required TSR value (80%). The indirect tensile strength, however, increased as the RAP percentage increased in the mix and it was the highest at 40% RAP. When RAP and FRAP are compared, FRAP mixes have slightly lower indirect tensile strength.

3. The Dynamic Modulus test results show that 20% RAP mixture had the highest dynamic modulus, followed by 30% FRAP and 30% RAP. The mixtures with 40% RAP and 40% FRAP have almost similar dynamic moduli at 4^{o}C. At 37^{o}C, 40% RAP had slightly higher dynamic modulus values than that for mix with 40% FRAP. These results, when combined with the MEPDG permanent deformation models, indicated that asphalt mixtures with high RAP or FRAP would result in higher rutting than mixture with 20% RAP. Again, the fatigue models in MEPDG indicated that asphalt mixtures with a higher proportion of RAP and FRAP will be more susceptible to fatigue cracking. The higher the RAP/FRAP content the worse would be the performance. The fatigue performance of pavements containing mixtures with RAP and FRAP are expected to be comparable.

5.2 Recommendations

1. Only one source of RAP has been studied in this project. Multiple RAP sources should be investigated.

2. Some form of cracking test, such as the Semi-circular Bending Test, Texas overlay test, etc., should be investigated to assess cracking susceptibility of high RAP mixtures.

3. Life of pavements incorporating high RAP mixtures should be assessed using MEPDG or a similar tool in order to assess the long-term performance of these pavements.

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