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Characterization of reclaimed asphalt and performance based evaluation of its use in recycled mixtures

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CHARACTERIZATION OF RECLAIMED ASPHALT AND PERFORMANCE BASED EVALUATION OF ITS USE IN RECYCLED MIXTURES

By

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Reclaimed asphalt pavement (RAP) is a valuable resource that can be recycled into new asphalt mixtures. In recent years, the continued rise of raw material costs has generated considerable interest in increasing RAP usage. Warm mix asphalt (WMA) is a modern development in the asphalt industry that can potentially help increase RAP usage and achieve adequate mixture performance. The purpose of this dissertation is to: 1) develop a method to characterize the absorbed, inert and effective bituminous components in RAP; and 2) evaluate performance of high RAP-WMA mixtures for various pavement applications including airfield surfaces, highway surfaces and highway bases.

A unique approach was taken to characterize RAP properties that coupled a dataset of 568 asphalt mix designs spanning five years of practice and testing 100% RAP with added virgin binder; 394 compacted specimens and 68 loose specimens were tested. A method to predict RAP absorbed asphalt was developed and shown to yield more reasonable results than conventional methods which were shown very likely to give

incorrect absorbed asphalt contents in some conditions. The relative effectiveness of RAP surface asphalt was evaluated and estimates of inert and effective RAP asphalt were made for a variety of temperature, compactive effort, and warm mix additive conditions. Results showed different behaviors between RAP sources and between hot and warm mix temperatures. These results were also observed in volumetrics of high RAP mixtures.

Performance evaluation was based on testing 75 slab specimens and more than 1100 gyratory specimens. Test data indicated a potential for decreased durability as RAP content increases; however 25% RAP highway surface mixtures and 50% RAP base mixtures had similar performance to current practice. Low temperature mixture stiffness testing and thermal cracking analysis indicated slightly increased stiffness with high RAP and 25% RAP highway surface mixtures that had comparable performance to current practice. Dry rut testing indicated high RAP mixtures are rut resistant. Moisture damage testing of high RAP mixtures indicated passing results in tensile strength ratio testing but potential for moisture damage in loaded wheel tracking. Overall, 25% RAP highway surface mixtures are recommended for immediate implementation.

DEDICATION

This dissertation is dedicated to my family for their love and support. Especially to my parents David and Terri Doyle and also especially to Richard Ahlgren and Doris Doyle who were unable to see me complete this degree.

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ξ	Reduced time
β	Christensen-Anderson exponent
λ	Christensen-Anderson critical time (sec)
α_1	Linear coefficient of thermal contraction in the fluid region; also mixture dependent constant in the Arrhenius equation
α_g	Linear coefficient of thermal contraction in the glassy region
α_T	Arrhenius shift factor
AASHTO	American Association of State Highway and Transportation Officials
Abs	Water absorption of aggregate by aggregate mass (%)
AC Ratio	Ratio of specimen asphalt content to mixture design asphalt content
AM	Airfield mixture
APA	Asphalt Pavement Analyzer
APT	Accelerated pavement tester
B_{AC}	Volumetric coefficient of thermal contraction of asphalt binder (1/C)
B_{agg}	Volumetric coefficient of thermal contraction of aggregate (1/C)
BBR	Bending Beam Rheometer
B_{mix}	Linear coefficient of thermal contraction of asphalt mixture
CAM	Christensen-Anderson-Marasteanu
CAS	Christensen-Anderson-Sharrock
C.I.	Confidence interval
CM	Control mixture
COV	Coefficient of variation
CST	Critical stripping temperature
D_{b-s}	Bulk slab density (g/cm^3)
DCSE	Dissipated creep strain energy
DIM	Dual instrument method
DOT	Department of transportation
DSR	Dynamic Shear Rheometer
(d2s)	Difference two-sigma limit of *ASTM C 670*
E^*	Dynamic modulus of asphalt mixture
ESAL	Equivalent single axle load
FHWA	Federal Highway Administration
FOD	Foreign object debris
FWD	Falling weight deflectometer
GLWT	Georgia loaded wheel tester
G-1 to *G-4*	Crushed gravel aggregate source 1 to crushed gravel aggregate source 4
G_b	Specific gravity of asphalt binder
G_{mb}	Bulk specific gravity of a compacted asphalt specimen

G_{mm}	Maximum theoretical specific gravity according to *AASHTO T 209*
GR-1 to GR-3	Crushed gravel airfield gradation 1 to crushed gravel airfield gradation 3
G_{sa}	Apparent specific gravity of aggregate
G_{sb}	Bulk specific gravity of aggregate
G_{se}	Effective specific gravity of aggregate
HLWT	Hamburg loaded wheel tester
HL-1	Hydrated lime source 1
HMA	Hot mix asphalt
IDT	Indirect tensile test
LAC	Linear Asphalt Compactor
L-1 to *L-4*	Limestone aggregate source 1 to limestone aggregate source 4
LS-1 to LS-3	Limestone airfield gradation 1 to limestone airfield gradation 3
$LST_{+4.75}$	Amount of limestone aggregate retained on the 4.75 mm sieve as percentage of total coarse aggregate retained on the 4.75 mm sieve
$LST_{+2.36}$	Amount of limestone aggregate retained on the 2.36 mm sieve as percentage of total coarse aggregate retained on the 2.36 mm sieve
LTPP	Long term pavement performance program
NCAT	National Center for Asphalt Technology
NCHRP	National Cooperative Highway Research Program
MDOT	Mississippi Department of Transportation
MEPDG	Mechanistic-Empirical Pavement Design Guide
ML	Mass loss in Cantabro durability test as percentage of original mass (%)
ML Ratio	Ratio of specimen mass loss to average mass loss of the mix at design asphalt content
MSU	Mississippi State University
MTV	Material transfer vehicle
N_{des}	Number of design gyrations used in the *SGC*
NMAS	Nominal maximum aggregate size
N_p	Number of load applications
OGFC	Open graded friction course
PFC	Porous friction course
P_{200}	Percent passing No. 200 (0.075 mm) sieve
P_{AC}	Total asphalt content on mix mass basis (%)
PAV	Pressure aging vessel
P_b	Design asphalt content on mix mass basis (%)
$P_{b(R)}$	Total RAP asphalt content on mix mass basis (%)
$P_{b(V)}$	Total virgin asphalt content on mix mass basis (%)
$P_{ba(s)}$	Asphalt absorption of aggregate by aggregate mass (%)
$P_{ba(mix)}$	Asphalt absorption of aggregate by mixture mass (%)
$P_{ba(R)}$	Absorbed RAP bitumen on mix mass basis (%)
P_{be}	Total effective binder on mix mass basis (%)
$P_{be(R)}$	Effective RAP binder on mix mass basis (%)
$P_{be(V)}$	Effective virgin binder content on mix mass basis (%)
$P_{bi(R)}$	Ineffective RAP binder on mix mass basis (%)
$P_{bs(R)}$	Total RAP surface asphalt on mix mass basis (%)

PC	Pavement constant
PCI	Pavement condition index
PCR	Pavement condition rating
PG	Performance Grade of asphalt binder
$P_{s(R)}$	RAP aggregate on mix mass basis (%)
R^2	Statistical coefficient of determination
R-1 to *R-5*	RAP source 1 to RAP source 5
R_M	Total rut depth measured manually (mm)
$R_{Adj.}$	Adjusted rut depth accounting for LVDT to manual measurement difference (mm)
R_T	Rut depth measured by PURWheel LVDT (mm)
RAP	Reclaimed asphalt pavement
RM	Recycled mixture
RMS	Root mean square
RTFO	Rolling thin film oven
RVE	Representative volume element
QC	Quality control test specimen
QA	Quality assurance test specimen
S-1 and *S-2*	Coarse sand aggregate source 1 and coarse sand aggregate source 2
SA	Aggregate surface area (m^2/kg)
SAP	Single asymptote method
Sat	Percent saturation of conditioned moisture damage specimen
SETC	Single event thermal cracking
SGC	Superpave Gyratory Compactor
SHRP	Strategic Highway Research Program
SIP	Stripping inflection point
SR	Sand ratio
S	Stiffness modulus
S_{glassy}	glassy modulus (GPa)
S_t	Indirect tensile strength at failure
S_{td}	Dry indirect tensile strength in *TSR* testing
Std. dev.	Standard deviation
t	physical loading time (sec)
T_{ref}	Reference temperature (Kelvin)
TAP	Two asymptote method
TCE	Trichloroethylene
T_{cr}	Pavement single event thermal cracking temperature (C)
T_g	Glass transition temperature (C)
TSR	Tensile strength ratio (%)
USACE	United States Army Corps of Engineers
V_a	Air voids of compacted asphalt mixture (%)
$V_{a(T166)}$	Air voids measured specifically with *AASHTO T 166* (%)
$V_{a(T331)}$	Air voids measured specifically with *AASHTO T 331* (%)
V_{agg}	Volume of aggregate in asphalt mixture (%)
Var.	Variance

V_{Total}	Total volume (i.e. 100%)
VFA	Voids filled with asphalt
VMA	Voids in mineral aggregate
WMA	Warm mix asphalt
a, b	Material constants
n	Number of observations or number of data points
μ	Population mean
σ	Population standard deviation
$z_{\alpha/2}$	Statistical coefficient accounting for variability in the prediction of G_{sb}

CHAPTER 1

INTRODUCTION

1.1 Research Motivation

Reclaimed asphalt pavement (RAP) is asphalt concrete that has been removed from an existing pavement after some un-quantified amount of environmental exposure and traffic. RAP is most commonly obtained by cold milling of pavements as part of maintenance and rehabilitation activities. RAP can be recycled into new mixture by heating and mixing with virgin aggregate and asphalt binder. The recycling process can be conducted at conventional hot mix temperatures or at warm mix temperatures.

There are several reasons to use RAP in new asphalt mixtures, including: 1) cost savings from replacement of virgin materials with lower cost reclaimed material; and 2) conservation of natural resources through reduced demand for virgin binder and aggregate. The more RAP is utilized in a mixture, the greater the potential advantages. On the other hand, there are several potential disadvantages to use of RAP, including: 1) stiffening of the composite binder component of the recycled mixture due to contribution of stiff RAP asphalt; and 2) difficulty meeting gradation requirements in recycled mixtures due to the contribution of RAP aggregate which frequently has high fines contents due to aggregate degradation during service and the reclamation process. In addition, the use of RAP in high quantities (greater than 25% of the asphalt mixture) raises other problems such as accurate determination of RAP aggregate properties and assessment of RAP asphalt absorption.

The use of lower mixing and compaction temperatures in warm mix asphalt (WMA) is an emerging trend in the asphalt pavement industry. Various techniques are used to temporarily modify the properties of asphalt binder and allow mixture temperature reduction; these techniques include wax based binder additives, chemical additives such as surfactants, as well as processes and additives designed to increase volume of asphalt binder with foam produced by steam. Potential advantages of lower production temperatures associated with warm mix include: 1) reduced cost due to lower energy requirements; 2) improvement of long term pavement properties due to reduced binder aging; and 3) reduced emissions. However, there are also some potential disadvantages to WMA such as: 1) incomplete drying of aggregates leading to moisture susceptibility, and 2) increased propensity for permanent deformation early in the pavement service life due to reduced binder aging.

Use of a combination of high RAP contents and WMA has the potential to alleviate some of the individual disadvantages of each component. For example, the reduced binder aging associated with lower production temperatures of warm mix could potentially offset some of the increased stiffness associated with high RAP. Use of warm mix could potentially allow greater percentages of RAP to be utilized than are used in current practice. At the same time, it must be ensured that no new problems present themselves in warm mixtures containing RAP and that performance of high RAP-WMA is adequate for its intended application in a pavement structure.

1.2 Scope and Objectives

For the purposes of this study, high RAP mixtures are defined as 25% or more of the total mixture (Copeland 2011). Warm mix asphalt is defined as asphalt mixed and compacted at temperatures lower than conventional hot mix for the given binder grade [50 F (28 C) or more reduction in temperature (Bonaquist 2011)]. The focus of this dissertation is on characterization of RAP and laboratory properties of high RAP-WMA mixtures to evaluate their suitability for different applications and functions in a pavement structure.

Four primary objectives are addressed in this dissertation. They are:

1. Characterization of RAP to estimate absorbed and effective asphalt components as well as evaluation of 100% RAP mixtures to investigate relative performance characteristics of different RAP sources.
2. Evaluation of high RAP-WMA for airfield surface mixtures. Performance properties evaluated include durability, thermal cracking potential, rutting, and moisture susceptibility.
3. Evaluation of high RAP-WMA for highway surface mixtures. Performance properties evaluated include durability, thermal cracking potential, rutting, and moisture susceptibility.
4. Evaluation of high RAP-WMA for highway base mixtures. Performance properties evaluated include durability, cracking potential, rutting, and moisture susceptibility.

The mixtures investigated in this study would be intended for central plant recycling; this study does not consider in place or cold recycled mixtures, although many of the same considerations apply. Furthermore, this study does not consider the plant production and economic aspects of high RAP-WMA. For example, production of high RAP mixtures may require fractionation of RAP into multiple stockpiles to ensure adequate control of gradation; also, the increased cost per ton associated with warm mix additives could potentially be offset by reduced virgin binder costs from using RAP.

1.3 Organization of Study

This dissertation is organized by chapters. Each chapter begins with an overview section which describes what is contained within the individual chapter, and how it is organized. Chapter one contains an introduction, as well as the scope and objectives of the study. Chapter two is a review of literature as well as a discussion of RAP and warm mix. Chapter three describes the materials, sample preparation and properties of mixtures tested. Chapter four describes test methods, and presents the experimental designs utilized for this study. Chapter five presents data analysis and discussion related to the first objective of this dissertation. Chapter six presents data analysis related to the second objective. Chapter seven presents data analysis related to the third objective. Chapter eight presents data analysis related to the fourth objective. Chapter nine evaluates relative compactability of high RAP-WMA. Chapter ten presents an overall discussion of performance of high RAP-WMA, using data from chapters five to nine. Conclusions and recommendations for future work are presented in chapter eleven.

CHAPTER 2

LITERATURE REVIEW

2.1 Overview of Literature Review

This chapter provides review of literature organized by topic. The topics of interest during literature review included historical use of RAP, properties of RAP, interaction of RAP with virgin binder and aggregates in recycled mixtures, mix design methods for high RAP content, test methods of interest to this study, as well as performance of mixtures with RAP and warm mix in the laboratory and in the field.

Al-Qadi et al. (2007) is a recent literature review related to RAP use. Some sources referenced in Al-Qadi et al. (2007) are included in this document, while others are not referenced since they provide no additional insight into the objectives of the current work. According to the literature review of Al-Qadi et al. (2007), various researchers have investigated the proper methods of utilizing RAP, alongside its corresponding performance characteristics, with widely mixed results providing no clear conclusions. In some studies given parameters have been reported superior, while in other studies given parameters have been reported inferior.

2.2 History of RAP Use

Recycling and reuse of asphalt paving materials has been practiced for many years in the paving industry. Hot recycling of existing pavements has been practiced since at least the 1930’s and 1940’s (Taylor 1975) and probably earlier. However recycling of asphalt pavements was of limited extent in early years due to the relatively low cost of virgin raw materials as well as limited experience and knowledge.

The oil embargo of the early 1970’s and generally rising costs of raw paving materials generated a large interest in recycling of existing pavement. Central plant hot recycling of RAP had been experimented with as early as 1915 (Epps et al. 1980); however the first full description of central plant hot recycling that was identified in literature was Dunning et al. (1975). Effective use of RAP necessitated a much better understanding of its properties and behavior in recycled mixtures. As a result of the desire to understand RAP more completely, the FHWA initiated Demonstration Project No. 39, Hot Recycling of Asphalt Pavement Materials. Reasoning for the demonstration project was stated in the background section:

> *The pressing need to conserve energy and minimize costs in highway construction requires that special effort be made to identify and make the maximum use of procedures that will result in reduced energy usage and minimum cost. Because recycling of asphalt pavements has the potential to be an effective method of conserving energy and materials and reducing costs, it is FHWA's policy that recycled asphalt concrete, defined as asphalt concrete containing salvaged paving materials including the use of suitable reclaimed material from other projects, be allowed for use on all projects. States with insufficient experience to properly evaluate the reuse of these materials should take immediate steps to initiate experimental projects.* (FHWA 1979).

There was no limit placed on use of softening agents, added asphalt grade, or percent of RAP (Epps et al. 1980). Some projects used 100 percent RAP, but it was recognized that batch plants were generally limited to 50 to 70 percent RAP. There were problems of production, emissions, and achieving consistent mixture properties (Epps et al. 1976, Kari et al. 1979, Betenson 1979, Smith 1980, Dunning 1983).

A synthesis of highway practice performed in 1978 addressed multiple facets of recycling, including central hot mix plants (Copas and Pennock 1978). During the same period, White (1977) studied 100% RAP in the laboratory in conjunction with two soft asphalt binders (AC-10-127 pen; AC-5-270 pen) and reported that an addition of 1.75% asphalt content was satisfactory in the laboratory for the conditions encountered, and noted that the approach taken was only one of the possibilities. Viscosity modifiers were used by Dunning et al. (1975). They were selected to create a target final blend viscosity; specimens were compacted by the Marshall method to determine the optimum additional asphalt content. Based on the results, addition of up to 1.5% AR-8000 asphalt binder to recycled pavement was recommended.

Problems observed during the late 1970's to the early 1980's drastically reduced research and implementation of high RAP content mixtures. Many of the problems disappeared with HMA mixtures using lower percentages of RAP, the advent of new equipment (drum mixing plants, milling machines, etc), and industry experience. Into present day, HMA mixes with RAP in the 10 to 25 percent range are routinely used.

The state of knowledge of high RAP content mixtures did not fully develop over the years from the initial wave of research into the present day, possibly due to reduced motivation for high RAP use (e.g. reduction in raw material cost) and the comfort that was developed when using small RAP quantities. Consequently, the approaches taken to evaluate RAP were likely not fully developed and stayed along familiar research paths. Note many of the failures of high RAP content mixes have occurred when unprocessed RAP has been used in HMA plants not equipped to handle the high contents (Bonaquist 2007). White (1977) noted problems of this nature some three decades prior while studying the effect of crushing on mixture voids. After approximately three decades of investigation, a comprehensive understanding of high RAP mixtures is not available. This is significant in the current environment, with high material demand and premium virgin material prices. A recent document written by Brock and Richmond (2007) indicated the amount of recycling will likely increase over the next 20 years.

2.3 RAP Properties

Four key attributes related to RAP are: 1) total asphalt content within RAP; 2) amount of asphalt absorbed into the RAP aggregate; 3) properties of RAP asphalt; and 4) properties of RAP aggregate. Understanding of these parameters is critical to the successful use of high RAP contents. They are discussed in the following sections.

2.3.1 Determination of RAP Asphalt Content

Measurement of RAP asphalt content poses several issues. Ignition methods and solvent extraction both have positive and negative aspects, especially with regards to RAP. A portion of the asphalt materials community has expressed concern that the asphalt content determined via these two methods could be very different. Ignition methods require correction factors for aggregate loss that can be difficult to determine for RAP (Prowell and Hurley 2005). Hurley and Prowell (2005a) indicated a furnace using Tempyrox technology and an internal scale might be able to address the issue for RAP. On the other hand, Huang et al. (2005) reported the same asphalt content (6.8%) from both ignition and extraction procedures.

Peterson et al. (2000) examined several solvent extraction methods in preparation for NCHRP 9-12 and chose the Asphalt Institute TP-2 test method using an n-Propyl Bromide solvent. The authors felt it offered the best combination of safety, accuracy, and repeatability. The state of Oregon uses ignition methods to determine RAP asphalt content and assumes a 0.5% aggregate correction factor, but notes the potential for error in doing so (Thompson 2003). At present, RAP contents are limited to 30%, so increasing this value without properly accounting for binder in the RAP could be detrimental to payments and performance. Thompson (2003) attempted to account for the variability using two forms of equations without success.

Thakur et al. (2011) used the centrifuge extraction method with TCE solvent to recover RAP aggregate and then used the aggregate to generate aggregate correction factors for use in the ignition method. The method was effective in producing corrected asphalt contents that closely matched solvent extracted asphalt contents. However the process is not very practical for day to day determination of RAP asphalt contents.

Kvasnak et al. (2010) conducted a laboratory study of four simulated RAPs; four aggregate types and two asphalt binders were utilized. The simulated RAPs were made by following the *AASHTO R-30* aging protocol (4 hours at 135 C followed by 5 days at 85 C) on loose samples of asphalt mixture. Three methods of determining the RAP asphalt content were examined: 1) centrifuge with TCE solvent; 2) reflux with TCE solvent; and 3) ignition oven. The researchers found that the asphalt contents determined by all three of the methods were consistently lower than the actual asphalt contents of the mixes, with the ignition method generally yielding results closest to the actual value and the centrifuge method always yielding the overall lowest asphalt content results.

Research and experience has shown that the asphalt content of the finer fraction of RAP is higher than the coarser fraction (Khedaywi and White 1995). Zearley (1979 experimented with determining asphalt content on plus 4.75 mm (No 4 sieve) and minus 4.75 mm RAP and found asphalt contents of 3.0% and 6.8% for the coarse and fine fractions respectively. The approximate asphalt film thicknesses were calculated for the coarse and fine aggregate fractions, and were found to be identical.

Al-Qadi et al. (2009) extracted the bitumen for several aggregate size ranges from two RAP samples and found that the larger aggregate particles had lower bitumen content than finer aggregate particles. The fine RAP portions had bitumen content of about 7% for the two RAPs tested, which was higher than the overall average content for the RAP sources of 4.7 and 5.1%. The coarse RAP portions had a bitumen content of about 3 to 4% which was lower than the overall average contents.

2.3.2 Asphalt Absorption

Kandhal and Khatri (1992) conducted a laboratory study to investigate the absorption of asphalt binder by aggregate as part of the Strategic Highway Research Program (SHRP). Eight aggregate sources and four asphalt binders (32 mixture combinations) from the SHRP materials reference library were utilized in the study that encompassed a wide range of material properties. Aggregate types tested included gravel, granite, limestone, sandstone, and basalt; the binder grades were AC-5, AC-10, AC-20, and AC-30. Asphalt absorption was found to be a function of both aggregate and asphalt properties. Data from the study indicated a general relationship between aggregate water absorption and asphalt absorption. An equation was developed relating asphalt ($P_{ba(s)}$) to water (Abs) absorption (Eq. 2.1), where both terms reference aggregate mass. However the relatively low coefficient of determination for the relationship indicated that aggregate water absorption alone does not fully predict asphalt absorption.

$$P_{ba(s)} = 0.277(Abs) + 0.15 \qquad R^2 = 0.55 \qquad n = 96 \qquad \text{(Eq 2.1)}$$

Kandhal and Khatri (1992) further found that aggregate particle shape and texture did not correlate with asphalt absorption. Measurements of aggregate pore diameter indicated that no appreciable asphalt absorption occurred in aggregate pores less than 0.05 micron in diameter though reasonable correlations were developed relating asphalt absorption to the size and quantity of aggregate pores larger than 0.05 micron. In some cases the amount of asphalt absorption of any given aggregate source varied noticeably depending on the asphalt binder source.

Analysis of the component chemistry of the asphalt binders tested provided some evidence that selective absorption of asphalt binder components might be occurring but no definitive conclusions could be drawn with regards to selective absorption. The experimental results indicated that asphalt absorption was dependent not only on the specific aggregate and asphalt binder binders tested, but also on the interaction between the factors. The primary conclusion of the study was that each type and source of asphalt binder must be treated independently (Kandhal and Khatri 1992).

2.3.3 RAP Asphalt Properties

A long standing question about RAP is: what measures can be taken to account for the aged binder? Stiffness of the RAP binder is believed to be a key to producing successful high RAP mixtures; excessive stiffness may cause cracking and compaction problems. During a literature review, Al-Qadi et al. (2007) identified six primary mechanisms associated with age hardening:

1. Oxidation through diffusive reactions between binder and oxygen
2. Volatilization (evaporation) of light binder elements, mostly during construction
3. Polymerization via chemical reaction of molecular components
4. Thixotropy caused by long structure formation within binder
5. Syneresis due to the exudation of thin and oily components
6. Separation via removal of oils, resins, and asphaltenes by absorptive aggregates

The greater the pavement damage where RAP was obtained, the greater the changes in binder properties relative to their original state (Al-Qadi et al. 2007). RAP binder can be softened/rejuvenated using materials including flux oil, lube stock, slurry oil, lubricating oils, extender oils, and other specialty blends of bituminous materials.

Stiffness of RAP bitumen is not necessarily uniform throughout the entire asphalt film thickness. Staged extraction was used in combination with the Abson recovery method in Iowa as early as the 1970's (Zearley 1979). Increased penetration values (softer asphalt properties) were observed for the inner layers of the RAP asphalt film. Staged extraction of the asphalt film coating RAP was utilized by Noureldin and Wood (1987) and demonstrated that the stiffness of RAP bitumen is not uniform throughout the asphalt film coating an aggregate particle. They extracted the bitumen from RAP in four stages using TCE then recovered and tested each stage independently; results are reproduced in Table 2.1. Note that the original asphalt used to create the mix from which the RAP came was an AC-20 (Penetration = 40, Viscosity = 2000 ± 400). Total asphalt content of RAP was 6%. The viscosity of the outermost layer of RAP asphalt was more than 7 times higher than that of the innermost asphalt layer; penetration of the innermost layer was more than double that of the outer layer. Noureldin and Wood (1987) noted that "the amount of hardening that occurred in the old binder was relatively low compared to that in previous recycling projects." Selective absorption of the "light end" fractions of asphalt by aggregate is thought to occur (Noureldin and Wood 1987).

The data of Noureldin and Wood (1987) and Zearly (1979) support a view that a gradient of stiffness exists in the RAP asphalt film. Their data indicate that the outer portion of RAP bitumen film has a higher stiffness than the inner portion. For an individual particle of RAP coated with a film of aged asphalt, the innermost portion of the film is partially absorbed into the pores of the aggregate particle. The stiffness of this innermost layer of the film has been affected somewhat by the aging process undergone by the pavement during its production, construction, and service life, but in general it has been protected from the most detrimental effects of aging by the outer portion of the asphalt film. The outermost portion of the bitumen film will have the highest stiffness due to a greater exposure to detrimental environmental effects. The process of full extraction of RAP asphalt with solvent will effectively mix all the layers together and destroy any variation in stiffness existing in the RAP asphalt film.

Table 2.1 Results for Reclaimed Stage-Extracted RAP (Noureldin and Wood 1987)

TCE Increment (mL)	Binder (% by weight)	Penetration	Viscosity at 140 F (poises)
First (200)	55.5	24	24,000
Second (200)	26.5	33	15,000
Third (300)	11.2	65	2,500
Fourth (700)	6.8	57	3,300

Note: Results are averages of three replications, each conducted on seven 1200 g samples; percentage of asphalt cement to 6 percent by weight of mix; and original asphalt was AC-20.

2.3.4 RAP Aggregate Properties

Prowell and Carter (2000) conducted a laboratory study of aggregate properties when recovered from the ignition oven asphalt content test. Ten asphalt mixtures were tested that utilized nine aggregate sources commonly used in Virginia. The mixtures were used to produce simulated RAP in the laboratory by loose mix short term oven aging. Aggregate samples were recovered with the ignition oven, tested for G_{sb}, and the results compared to measured virgin aggregate G_{sb}. In 8 of the 10 cases for fine aggregate, G_{sb} of extracted aggregates were lower than the known virgin aggregate G_{sb}; the average difference of the cases that were lower was 0.026. For coarse aggregate, 60% of the cases were significantly different from the known virgin aggregate G_{sb}. In all 10 cases for coarse aggregate, G_{sb} of extracted aggregates were lower than the known virgin aggregate G_{sb}; the average difference was 0.039.

Hall and Williams (1999*a*) also studied the effects of ignition oven testing on recovered aggregate properties. Eight mixtures were produced from a range of aggregate types used in Arkansas. Measurements of G_{sb} on recovered aggregate were lower than those for virgin aggregate in all eight cases, the average difference was 0.036. The authors stated that in a number of cases the differences were within the acceptable range of two test results specified by the test method.

McDaniel and Anderson (2001) stated that it can be difficult to accurately measure G_{sb} of extracted RAP aggregate because of potential changes in the aggregate properties or gradation, due to the extraction process. They recommended use of one of two approaches to avoid this difficulty. The first approach was substitution of effective aggregate specific gravity for bulk aggregate specific gravity in volumetric calculations.

While this approach can provide a reasonable approximation in cases where asphalt absorption by the aggregate is low, in many instances that is not the case. The second approach discussed was back-calculation of aggregate G_{sb} by measurement of RAP G_{mm} and use of an assumed value for absorbed asphalt for the RAP.

Newcomb et al. (2007) discussed the difficulty of accurately measuring G_{sb} for RAP aggregate. The authors mentioned that the ignition method could change aggregate properties and that solvent extraction methods did not always remove all of the absorbed asphalt from the aggregate pores. It was recommended to use the back-calculation method for RAP aggregate G_{sb} with measured G_{mm} data and using either known asphalt absorption values from similar aggregates or an assumed value of 1.5%.

A laboratory study of four simulated RAPs conducted by Kvasnak et al. (2010): details of the materials and methods were discussed in section 2.3.1 of this literature review. Aggregate G_{sb} values determined with ignition oven extracted aggregate were found to be generally similar to or lower than the actual aggregate G_{sb} values determined by testing virgin aggregate. Aggregate G_{sb} values determined with solvent extracted aggregate were found to be generally similar to or higher than the actual aggregate G_{sb} values. In 47% of the comparisons (all test methods) the extracted aggregate G_{sb} values were significantly different than the virgin aggregate values; results were dependent on aggregate type. Kvasnak et al. (2010) recommended that the back-calculation approach to estimate aggregate RAP G_{sb} by measurement of RAP G_{mm} be used whenever a reasonable estimate of absorbed asphalt content is available. If an estimate of absorbed asphalt content is not available, measurement of RAP aggregate G_{sb} was recommended as the next best option but that caution should be used when selecting an extraction method for certain aggregate types.

Thakur et al. (2011) examined differences in recovered RAP aggregate properties from the ignition and solvent-centrifuge extraction methods. They observed that G_{sb} of ignition recovered aggregates was lower than solvent-centrifuge recovered aggregates. Their results align with those of Kvasnak et al. (2010).

2.4 Interaction of RAP and Virgin Materials

Broadly speaking there are three theories of how RAP bitumen interacts with virgin materials when recycled into new asphalt mix:

> *There are three conceptual positions that can be taken regarding the bituminous material within RAP: 1) black rock-all bituminous material acts as aggregate; 2) fully blendable-all bituminous material becomes fluid and totally blends with virgin asphalt binder; 3) partially reusable-some bituminous material livens and is reusable in the new mixture with the extent being dependent on several factors including aged binder properties, temperature, aging time, and additives* (Doyle and Howard 2010a).

McDaniel et al. (2000) addressed two main questions in NCHRP 9-12: does RAP binder act as part of the cohesive binder or is it inert (i.e., a "black rock") and, if the RAP binder does blend, how does it affect the composite binder and the mixture? Results of McDaniel et al. (2000) conclusively disproved the black rock theory and "strongly suggest[ed] that actual practice achieves a situation much closer to total blending than to no blending (black rock)." The question of whether RAP acts as a black rock in recycled mixes was addressed in more detail by Soleymani et al. (2000) using NCHRP 9-12 data.

Doyle and Howard (2010a) compacted specimens of 100% RAP with no additional binder at a range of temperatures from 25 to 177 C. If RAP truly behaves as a black rock, the addition of heat to a sample before compaction would not aid compaction or increase density of the compacted specimen. They found that for temperatures below 71 C, compacted RAP behaved as compacted aggregate and specimens were not cohesive, implying that the aged RAP bitumen did not aid compaction and instead acted nearly as a black rock at low compaction temperature. On the other hand, specimens compacted at 71 C and above were cohesive and resembled ordinary compacted asphalt mix. A strong decrease in air voids was observed with increasing compaction temperature indicating that the RAP bitumen has an effect on compaction of RAP aggregate. The black rock theory is not tenable for HMA or WMA since bitumen that has an effect on RAP aggregate will also affect performance in a recycled mixture to some degree, although the specific effects are not fully understood.

The extent of blending has been widely disputed. Some claim mixes have near 100% blending and that it can occur relatively quickly, while others believe little blending occurs. The literature summary of Al-Qadi et al. (2007) is quoted as follows:

> *Research has shown that typical recycling projects have achieved blending of the RAP binder and the virgin binder, but have not been able to predict a-priori what the percentage of the RAP binder that effectively combines with the new binder will be. The blending is somewhere between 0 (black rock) and 100% (complete combining of the two binders)* (Al-Qadi et al. 2007).

This finding led to the statement that before higher RAP percentages can be utilized, methods to determine blending potential and account for relative RAP effectiveness must be developed. If total blending is assumed and no blending occurs the result is a very soft binder with inadequate stiffness and too little asphalt. The reverse is no blending assumed and total blending occurring. The result is a very stiff mixture with excess asphalt. Stephens et al. (2001) notes current design methods assume complete blending and states this does not occur. Complete blending is a fundamental assumption behind the use of blending charts for recycled mix design with elevated levels of RAP. Blending charts have been used when over 25% RAP is included in Superpave mixtures. At intermediate RAP contents of more than 15% and up to 40%, (McDaniel et al. 2000) recommended use of blending charts. However, McDaniel et al. (2000) found that when blending charts were used for recycled mixes with 40% RAP that "some non-linearity begins to appear in the blending equations."

The partial blending theory is that RAP behaves in some manner between the two extremes of no blending and complete blending; it was advanced by McDaniel et al. (2000) as probable for HMA with elevated RAP contents, especially for stiffer grades of virgin binder. It was neither conclusively supported nor disproved for HMA by the data of McDaniel et al. (2000). The concept of partial blending is also supported by the results of Druta et al. (2009) who investigated blending of virgin binder infused with metallic powder and RAP asphalt on the surface of RAP aggregate using X-ray computed tomography; the precise extent of partial blending could not be determined by Druta et al.

A version of the partial blending theory was proposed by Tia et al. (1980) for cold recycled mix made with asphalt emulsion and rejuvenating agents. Tia et al. (1980) stated that RAP asphalts "are usually hardened and have lost most of their original characteristics." Tia et al. (1980) further stated that new binder must "be added to the recycled mixture to replace the "ineffective" portion of the existing" bitumen. Tia et al. (1980) opined that "the combination of the high compactive effort and the shearing action of the gyratory compactor forces the new and old binder to act together." Tia et al. (1980) assumed that 80% of the RAP bitumen was "effective" and used that assumption to calculate percentages of total effective asphalt after virgin binder was added. They found that for cold recycled mixes produced with asphalt emulsions and rejuvenating agents, mix stiffness as measured by the Hveem stabilometer varied with curing time after compaction indicating a time dependence of partial blending.

Doyle and Howard (2010a) proposed a partially-reusable extension of the partial blending theory of RAP behavior for HMA and WMA. They hypothesized that there are three categories of RAP bitumen: "1) binder on the aggregate surface available for blending; 2) bitumen unavailable for blending; and 3) absorbed bitumen." Data from compaction of 100% RAP specimens with additional virgin binder was used to compute a term called AC_{eff}. AC_{eff} was defined as "the ratio of effective RAP surface binder to total RAP bitumen." Their data showed a range for AC_{eff} of 67 to 87% for three RAP sources at a 116 C (240 F) compaction temperature. However a fundamental shortcoming of the AC_{eff} approach outlined in Doyle and Howard (2010a) is that it was based on the assumption that the current RAP bitumen content is adequate to satisfy the current requirements of the RAP aggregate to meet the goals of the recycled mix design. It also failed to provide any way to estimate the amount of inaccessible absorbed RAP bitumen independently of the amount of RAP bitumen unavailable for blending that may potentially exist on the surface of the RAP aggregate.

The extent of partial blending is a function of many variables including temperature, time, and additives (e.g. warm mix additives). Under some combinations of variables the extent of blending likely approaches or perhaps includes total blending; however under other combinations of factors the extent of partial blending is possibly noticeably less than total blending. The factors affecting blending and the potential extent of blending are addressed in the following sections.

2.4.1 Factors Affecting Blending

There are numerous factors that can affect how the RAP truly acts within an HMA mixture. Factors related to production, storage, transportation, and placement can all affect how much blending takes place. The amount of blending can have a significant effect on performance. In order for blending of the new and old asphalt binder to take place, there must first be heat transfer between the new and old asphalt binder. This heat transfer begins in the production stage. The amount of time that the RAP materials are mixed with the virgin materials will affect the amount of blending and depends upon the type and configuration of the HMA production facility.

Several sources have observed that in the laboratory, the amount of time that RAP is heated will affect mix properties. McDaniel et al. (2000) observed that heating times in excess of two hours could change RAP binder properties. Results of Doyle and Howard (2010a) indicated that RAP specimens compacted after four hours of heating time have generally lower air voids than specimens compacted after two hours of heating.

Stephens et al. (2001) investigated the effect of heating time on RAP with 11 mixes containing 15% RAP. Pre-heating time of the RAP before specimen fabrication was varied between zero and 540 minutes; results were then compared to a 12th mix made with extracted RAP aggregate and no RAP asphalt. The addition of RAP with no pre-heating time increased unconfined compression and indirect tensile strengths of the resulting specimens when compared to the same aggregate blend and all virgin binder.

Strength of the specimens remained relatively constant for when RAP had between zero and approximately thirty minutes of pre-heating time. Specimen strengths began to increase for RAP pre-heating times greater than 30 minutes and began to level out at the longest RAP pre-heating times investigated.

Daniel and Lachance (2005) performed laboratory testing on HMA with up to 40% RAP (two RAP sources with extracted binders graded as PG 94-14 and PG 82-22) combined with virgin PG 58-28 binder. The results showed an increase in VMA and VFA due to the RAP. To assess the effect of aging, RAP was aged between 2 to 8 hours and observations indicated there was an optimum heating time to allow softening, break down, and blending of virgin materials. Further research into this issue was recommended to simulate plant operations in the lab for mix design purposes. Carpenter and Wolosick (1980) studied the effects of asphalt modifiers on RAP with time after mixing. They found that time-dependent diffusion of asphalt modifiers through the recycled asphalt caused variations in the resilient modulus with time.

Since the amount of time that mixture spends at high temperature affects the softening of RAP particles and the time-dependent diffusion of asphalts of different viscosities, the use of storage silos could also potentially affect the level of blending that occurs. The longer mixture is stored the more time for the aged RAP asphalt to become heated which increases the potential for blending of the aged and virgin bituminous materials. Once the HMA is produced, it is placed into haul trucks and transported to the paving site. Depending upon the length of haul time, the amount of blending may change. Long haul times will allow for more blending and short haul times will result in less blending. Any blending of binders that does occur is believed to be time dependent.

In addition to these production/construction issues, the properties of the RAP itself will likely affect the amount of blending that occurs. RAP taken from the roadway via cold milling will generally be a graded material. Crushing and processing is sometimes used to produce a consistent RAP material. The resulting gradation of the RAP material will affect the potential for blending. Within the HMA production process, the finer particles contained within the RAP will become heated first and the larger particles will take longer to reach the intended mixing temperature. Research and experience has shown that the asphalt content of the finer fraction of RAP is higher than the coarser fraction [Khedaywi and White (1995), Watson et al. (2008), Al-Qadi et al. (2009)]. Therefore, since more asphalt binder is contained within the fine fraction and these materials will reach temperature quicker, there is more potential for RAP materials containing large fine fractions to blend with virgin materials than RAP materials containing a larger coarse fraction.

Additionally, RAP materials that contain very oxidized and hard binders will require more heat, mixing and time for blending to occur. The asphalt from some sources of RAP is stiffer than others. McDaniel et al. (2000) observed recovered asphalt grades of PG 82-25, PG 82-24, and PG 89-15 for RAP sources from Florida, Connecticut and Arizona respectively. Daniel et al. (2010) determined RAP asphalt grades ranging from PG 76-22 to PG 94-10 for seven RAP sources in New Hampshire; Daniel and Lachance (2005) determined RAP extracted asphalt grades of PG 94-14 and PG 82-22. Li et al. (2008) observed that stiffness of RAP asphalt affected high temperature stiffness of 20 and 40% RAP mixtures more than it affected low temperature stiffness and fracture properties. Daniel et al. (2010) also observed greater affect on high temperature properties than low temperature properties when RAP was included in the mixture.

Properties of the RAP aggregates can also affect the amount of RAP asphalt binder available for blending. Aggregates contained within the RAP that are highly absorptive will have aged asphalt that is absorbed into the aggregate pores. It is highly unlikely that this absorbed RAP asphalt will become blended with the new asphalt binder. Therefore, asphalt content of the RAP alone may not always indicate blending potential. This material is not worthless since it prevents absorption of virgin asphalt by RAP aggregate; however it should not be considered effective binder.

2.4.2 Extent of Blending

The interaction that occurs between new and aged asphalt is a combination of mechanical mixing that can transfer some amount of the RAP bitumen away from RAP aggregate and chemical diffusion between RAP bitumen and virgin binder on the surface of RAP aggregate. When RAP is incorporated into a recycled mix through a RAP collar or other means in an asphalt plant, it receives a brief but relatively intense (but not as much as the virgin aggregate) period of heating which softens the bitumen film. Some, but not all, of the bitumen film may be removed from the RAP aggregate and transferred to virgin aggregate particles by mechanical mixing. The portion of the bitumen film absorbed into the RAP aggregate particles will not be removed.

Mechanical mixing of RAP and virgin materials was studied by Huang et al. (2005) for fine RAP fractions. RAP (10 to 30% of minus 4.75 mm) was combined with plus 4.75 mm virgin aggregate (no binder) and a relatively consistent loss of bitumen from the RAP fraction was noted (11% of the aged bitumen). This amount of lost bitumen determined by Huang et al. (2005) could be viewed as readily available to interact with virgin binder or aggregate. Al-Qadi et al. (2007) noted that purely mechanical mixing of this type will not determine the true level of blending between RAP asphalt and virgin binder due to the process of diffusion between bitumen and binder that will occur during and after the mixing process. The results of Huang et al. (2005) support a view that the majority of RAP bitumen will remain as a film coating RAP aggregate and that interaction of RAP bitumen with virgin aggregate will be trivial. (Shirodkar et al. 2010) also investigated transfer of RAP asphalt from fine RAP to coarse virgin aggregate through mechanical mixing and estimated the range of partial blending between RAP and virgin binder to be 48 to 77%.

A diffusion process occurs at the interface between the virgin binder and the RAP bitumen. Diffusion is the process where asphalts of different viscosities in contact will intermingle without external assistance until viscosity equilibrium is reached (Carpenter and Wolosick 1980). The diffusion of asphalts is time dependent and the rate is driven by the magnitude of the viscosity differential (Carpenter and Wolosick 1980). This supports the observation of McDaniel et al. (2000) that when stiffer grades of virgin binder were used in recycled mixes, the resulting performance appeared to more closely resemble partial blending than complete blending.

Lee et al. (1983) used a dye chemistry technique to evaluate dispersion of recycling agents in recycled mix and found that only localized dispersion occurred after compaction. Staged extraction of asphalt from RAP in recycled mixes was utilized by Huang et al. (2005) to investigate stiffness of different layers in the asphalt film. Huang et al. (2005) mixed 20% RAP with virgin asphalt and aggregate to allow staged extraction of the asphalt film thickness from the RAP particles. The testing indicated two distinct viscosity zones. The outer portion (≈ 40% of the film thickness) appeared to blend with the virgin asphalt, while the inner portion (≈ 60% of the film thickness) retained much of the pure RAP bitumen properties. The viscosity of the inner portion was nearly double the viscosity of the outer portion.

The results of Huang et al. (2005) support a view that the diffusion process between virgin binder and RAP bitumen does not result in complete blending. It logically follows that if a softer grade of virgin binder or a rejuvenating agent is used in the recycled mix that the resulting properties would begin to approach the complete blending case. Ozer et al. (2009) investigated blending in 9.5 mm NMAS HMA with 40% RAP in the laboratory and observed that a double bumped softer virgin binder grade PG 58-28 had better blending than standard binder grade of PG 64-22. On the other hand, Mogawer et al. (2009) found that mixture complex modulus of 30 and 50% RAP WMA (Sasobit®) mixtures (4.75 mm NMAS) was the same whether binder was the regional specified grade PG 64-28 or double bumped PG 52-33.

Kim et al. (2007) investigated blending of RAP asphalt and virgin binder for five RAP sources in mixtures containing 30% RAP. After mixing, samples of coarse RAP aggregate, coarse virgin aggregate and mixture mastic (combination of RAP and virgin materials) were taken. A method to measure large molecular size with a gel-permeation chromatography (GPC) technique without complete binder extraction was utilized. Previous studies cited by Kim et al. indicated that GPC data was highly correlated to asphalt viscosity. Both the raw GPC data and the estimated viscosity data indicated that stiffness of the asphalt coating the coarse RAP aggregates was always higher than stiffness of the asphalt coating the coarse virgin aggregates. Stiffness of the mastic material always fell in between the other values. The asphalt film coating virgin aggregate had properties similar to or slightly stiffer than those measured for completely virgin mixture depending on RAP source.

Actual viscosity of the mixtures was measured on recovered asphalt (completely mixed by extraction process) and compared to the estimated values for RAP and virgin aggregate binder films. In all five cases examined by Kim et al. (2007) the actual viscosity values were greater than the estimated values for the coarse virgin aggregate binder film and less than the estimated values for the RAP coarse aggregate film. The data of Kim et al. (2007) supports a view that aged RAP asphalt primarily remains attached to RAP aggregate but that some amount of chemical diffusion occurs between the RAP asphalt and virgin binder.

For HMA and WMA using standard binder grades (i.e. not rejuvenators or recycling agents), essentially all of the blending between bitumen and binder that will occur is believed to occur while recycled mix is at an elevated temperature and that the magnitude of any long term blending (i.e. at ambient temperature after compaction) will be negligible. Recycling agents have a different formulation and much lower viscosity than standard grades of asphalt; because of this, the process of their diffusion with RAP bitumen will likely occur faster than the diffusion process between RAP bitumen and standard grade virgin binder (i.e. higher viscosity). Heat accelerates the process of diffusion (i.e. diffusion occurs faster at elevated temperatures). If so, then the opposite must also be true (i.e. diffusion slows down or is not as complete at less elevated temperatures). This could have important implications for WMA recycled mixes. The length of time that a mix is held at elevated temperature (i.e. heated storage in silos, transport etc) may also have an effect on the diffusion process.

A regional pooled fund study in the Midwest looked at three RAP sources at contents up to 50% (McDaniel et al. 2002). This study included a comparison of plant produced mixes to a linear blending chart. In two of the three cases, linear blending worked very well. In the third case, however, the mixture was consistently stiffer than expected based on linear blending, perhaps showing the effects of plant production variables. Shah et al. (2007) investigated properties of plant produced mixtures containing 0, 15, 20 and 40% RAP. Results indicated that mixture stiffness properties did not increase as RAP was added nearly as much as was predicted from blending charts and that the standard binder grade (PG 64-22) could be used for mixtures with up to 40% RAP. Stephens et al. (2001) observed PG binder grades of extracted asphalt from a 10% RAP mix were higher for plant produced than for laboratory produced mixture.

Bonaquist (2007) used the modulus of plant produced mix to estimate the effective binder modulus. This value was subsequently compared to extracted binder properties; good overlap of the modulus curves was indicative of good mixing. Bennert and Dongré (2010) proposed a four step procedure to back calculate effective binder stiffness properties of recycled mixtures from mixture dynamic modulus properties.

2.5 High RAP Mix Design Methods

Davidson et al. (1977) outlined the first comprehensive mix design method for RAP with recycling agents. The basic procedure consisted of determining RAP aggregate gradation, asphalt content and properties of the recovered RAP asphalt. A dosage rate for the desired recycling agent that produced an acceptable consistency of the final asphalt blend was then determined experimentally. Nomographs were provided and demonstrated to estimate the amount of recycling agent required. Final mixture properties were then evaluated to ensure adequacy for the desired use of the mixture.

Kallas (1984) proposed modifications to the Hveem and Marshall mix design methods to incorporate RAP with the use of blending charts. The approach did not address the issue of binder blending directly but instead experimentally determined the optimum recycling agent or new asphalt content. Five mix designs were performed with five different RAP sources from five different states using 40 to 52% RAP. High RAP variability was noted as a potential concern. High RAP variability and mixtures containing RAP has been noted by others including Solaimanian and Tahmoressi (1996) who analyzed four field projects in Texas that contained 35 to 50% RAP.

McDaniel et al. (2001) evaluated three very different RAP sources and RAP contents up to 40% in NCHRP 9-12. The guiding principle was that mixes with and without RAP should meet the same requirements. In the end, when the results of the black rock, binder, and mixture studies were considered, a consistent pattern emerged. Low RAP contents had negligible effect, high RAP contents had a significant effect and intermediate RAP contents had mixed results, supporting a tiered system for RAP.

The recommendations of NCHRP 9-12 were adopted by AASHTO. The current Superpave mix design specification (AASHTO 2007) prescribes that up to 15% RAP by weight of mix may be added without changing the virgin binder grade. At RAP contents higher than 15% up to 25%, the virgin binder grade is adjusted one grade softer to account for the stiffening effect of the hardened RAP binder; complete mixing of new and recycled binder is assumed. At RAP contents above 25%, a detailed design is necessary to select the properties of the virgin binder or to determine the amount of RAP that can be used with a given virgin binder.

McDaniel et al. (2001) noted that designing mixtures conforming to Superpave specifications may not be feasible in mixtures with greater than 40% RAP due to the high fines content of many RAP stockpiles. If pavement to be recycled has a high percentage of minus 0.075 mm material, it may be hard to use it since it will have even more minus 0.075 mm material after milling (Roberts et al. 1996).

Two major obstacles in designing high RAP content mixes were identified by Newcomb et al. (2007). The first being stiffness of the aged RAP binder. Use of a softer binder grade to compensate could introduce problems with mixing and diffusion of the binders. The resulting pavement would be vulnerable to damage early in its life before adequate dispersion and diffusion has taken place to reach the target asphalt blend properties. Secondly, use of large RAP percentages can lead to excessive fines due to the often finely crushed nature of RAP from the milling process.

With regard to design, Chehab and Daniel (2006) used the MEPDG software (Level 3) and determined RAP content and binder grade are significant variables. Stiffer binder grade was found to have a significant effect on predicted amounts of thermal cracking and permanent deformation. The effective binder grade, therefore, is significant to agencies desiring to implement the MEPDG approach (most if not all states eventually). Interestingly, increasing binder low PG temperature resulted in more predicted transverse cracking up to a point after which predicted thermal cracking leveled off. Daniel et al. (2009) investigated the effects of RAP mixture variables with MEPDG in more detail and found that the assumed PG for RAP mixtures does not greatly affect results for Level 1 analysis but can have a significant effect on results when using Level 2 or Level 3 analysis.

Recently, the state of Illinois has recognized that 100% contribution from residual RAP asphalt may be inaccurate (Al-Qadi et al. 2007); as of 2007, Illinois HMA mix designs with RAP include a 100% contribution. Many (if not most) other states use similar practices. According to Al-Qadi et al. (2007), the Illinois DOT allowed up to 30% RAP in HMA designed according to Superpave; with up to 50% RAP in shoulders and stabilized sub-bases. Specifying a maximum amount of RAP binder replacement instead of maximum RAP content has been recommended by Daniel et al. (2010).

Current specifications dealing with mix design of HMA with RAP are: *AASHTO M323: Superpave Volumetric Mix Design; ASTM D 3515: Standard Specification for Hot-Mixed, Hot-Laid Bituminous Paving Mixtures*; and *ASTM D 4887: Standard Practice for Preparation of Viscosity Blends for Hot Recycled Bituminous Materials.* These standards rely on blending charts to assess the effect of RAP on the mix design. For relatively low percentages of RAP this approach can be successful. For high percentages of RAP, this approach may not have the ability to capture the performance of the mixture. High RAP content mix designs that adequately account for all parameters are not available.

2.6 Test Methods and Relevant Parameters

This section includes information related to the test methods utilized in this study. Of interest was information related to specific test method parameters, studies that focused on use of RAP utilizing the test methods of interest, as well as associated background information or analysis methods needed for later analysis or discussion in this dissertation. The information is organized by topic in the following subsections.

2.6.1 Cantabro Durability Test

The Cantabro abrasion loss test is often used in design of open-graded friction course (OGFC) mixtures, also referred to as porous friction course (PFC), as a measurement of durability and of the potential for aggregate loss from mixtures (Watson et al. 2003). An upper limit of aggregate loss for un-aged OGFC mixture specimens of 20% has been recommended (Watson et al. 2004). Use of polymer-modified binders was found to noticeably reduce the aggregate loss compared to specimens made with an un-modified binder and the same aggregate type and gradation (Watson et al. 2004).

Celauro et al. (2010) utilized the Cantabro test to evaluate the durability of dense graded mixtures containing RAP; the test was performed according to European standard *EN 12697-17* with 18 C test temperature on un-conditioned Marshall compacted specimens. Three gradations were studied, two surface mixes and one base mix; two asphalt contents were studied for each surface mix and three asphalt contents were studied for the base mix (seven mixtures total). Each of the seven mixtures was tested with 0, 40 and 50% RAP content. For 0% RAP mixes, mass loss ranged from 4 to 7%;

for 40% RAP mixes, mass loss ranged from 5.5 to 10%; and for 50% RAP mixes, mass loss ranged from 8 to 12.5%.

Alvarez et al. (2008) found the Cantabro test to be somewhat sensitive to changes in fundamental binder properties due to aging but stated that test results might be more influenced by aggregate properties of the mixtures tested than by the binder properties. Based on a limited data set, Kraus (2008) provided evidence of a possible relationship between Cantabro aggregate loss for mixtures and results of Dynamic Shear Rheometer (DSR) testing on the polymer-modified binder components of the mixtures; testing was performed on both un-aged and laboratory aged binders and mixtures.

2.6.2 Relevant Low Temperature Asphalt Mixture Properties

Fundamentally the formation of thermal cracks within asphalt pavements is related to the volume contraction undergone by the pavement caused by a temperature decrease. Asphalt mixture expands or contracts in response to temperatures changes; the rate of thermal volumetric change of asphalt mixture is dependent on its binder properties, asphalt content, aggregate type, gradation, VMA (Nam and Bahia 2004), and possibly other factors.

For asphalt mixtures the rate of thermal volumetric change is often assumed to be isotropic (i.e. the same in all three directions) (Vinson et al. 1989). However others (Hills and Brien 1966) have stated that orientation and particle shape of aggregates can result in anisotropy; Jones et al. (1968) also mention that thermal volume change of many rocks and minerals is anisotropic. If isotropic behavior is assumed then a linear thermal coefficient of contraction is defined as simply one third the rate of cubic thermal volumetric change (Jones et al. 1968, Vinson et al. 1989).

Lytton et al. (1993) proposed a relationship to estimate the thermal contraction coefficient of mixture from the binder thermal contraction coefficient and the mixture volumetric properties (Eq. 2.2). Lytton et al. (1993) recommended use of an average value for B_{AC} of 345x10^{-6} (1/C) for most asphalt binders in lieu of testing. Nam and Bahia (2004) provided values for B_{agg} of 5.1x10^{-6} (1/C) for limestone aggregate and 11.3 x10^{-6} (1/C) for gravel aggregate.

$$B_{mix} = \frac{VMA \times B_{AC} + V_{agg} \times B_{agg}}{3 \times V_{Total}} \qquad \text{(Eq 2.2)}$$

where:

B_{mix} = linear coefficient of thermal contraction for asphalt mixture (1/C)

B_{AC} = volumetric coefficient of thermal contraction of asphalt binder (1/C)

B_{agg} = volumetric coefficient of thermal contraction of aggregate (1/C)

VMA = volume of voids in mineral aggregate (%)

V_{agg} = volume of aggregate in asphalt mixture (%)

V_{Total} = total volume (i.e. 100%)

The rate of thermal volumetric change for asphalt mixture is not constant throughout the range of temperatures experienced by asphalt pavements (Nam and Bahia 2004). At temperatures on the order of 25 C and less the linear expansion coefficient of asphalt is typically broken into two regions on either side of a temperature known as the glass transition temperature (T_g). The region above the glass temperature is known as the fluid region and the region below as the glassy region. A marked discontinuity in the rate of thermal volumetric change occurs at T_g; the rate is essentially constant above or below T_g within either the fluid or glassy regions respectively. Typical nomenclature is that the linear coefficient of thermal contraction in the fluid region is termed α_l and within the glassy region is termed α_g (Nam and Bahia 2004).

Work by Nam and Bahia (2004) measured the T_g, α_l, and α_g values of several asphalt mixtures and found that for mixtures T_g was dependent only on T_g of the binder component. Values for α_l and α_g were dependent on not only corresponding binder properties but also on aggregate type, gradation, VFA, and effective asphalt content (Nam and Bahia 2004). They also found that the glass transition temperature was not easily defined as a single value but rather that there was a range of temperatures within which transition occurred. Twenty-four mixtures were investigated by Nam and Bahia (2004) including six asphalt binders and four 12.5 mm NMAS aggregate gradations. The test results for mixtures with PG XX-22 binder are summarized in Table 2.2. The other mixtures contained PG 58-40 binder with a variety of modifications; thermal contraction coefficients for those mixtures were similar to the values shown in Table 2.2 but T_g values were much lower (on the order of -47 to -60 C).

Table 2.2 Summary of Asphalt Mixture Thermal Properties Taken from Nam and Bahia (2004)

Binder Grade and Type	**Aggregate Type and Gradation**	**Thermal Properties** T_g **(C)**	α_g **(10^{-6}/C)**	α_l **(10^{-6}/C)**
PG 82-22 SBS Radial	Limestone Coarse	-24.5	16.2	73.9
	Limestone Fine	-33.0	9.8	66.5
	Gravel Coarse	-33.0	32.0	77.8
	Gravel Fine	-25.2	28.0	96.6
	Average	**-28.9**	**21.5**	**78.7**
PG 82-22 Steam Distilled	Limestone Coarse	-25.4	32.6	85.5
	Limestone Fine	-26.1	16.6	57.9
	Gravel Coarse	-18.9	31.3	75.0
	Gravel Fine	-27.4	25.8	78.3
	Average	**-24.5**	**26.6**	**74.2**
PG 76-22 Ethylene Terpoly	Limestone Coarse	-30.9	3.0	64.6
	Limestone Fine	-39.7	26.3	66.7
	Gravel Coarse	-34.7	28.7	81.7
	Gravel Fine	-32.0	23.8	89.5
	Average	**-34.3**	**20.5**	**75.6**
Average	**Limestone Coarse**	**-26.9**	**17.3**	**74.7**
Average	**Limestone Fine**	**-32.9**	**17.6**	**63.7**
Average	**Gravel Coarse**	**-28.9**	**30.7**	**78.2**
Average	**Gravel Fine**	**-28.2**	**25.9**	**88.1**

2.6.3 Low Temperature Cracking

As the temperature drops, asphalt pavements shrink and longitudinal tensile contraction stresses are developed. If the tensile stress exceeds the mixture tensile strength at the same temperature fracture will occur and a crack will develop. When the temperature drops very quickly or reaches an unusually low value a crack will often develop suddenly; this is often referred to as single event thermal cracking (SETC). The critical temperature at which SETC occurs is denoted T_{cr}.

Bouldin et al. (2000) developed a semi-empirical mechanistic model and analysis method to estimate T_{cr} for pavements using binder test data. Bending beam rheometer

(*BBR*) data and direct tension data were used for the analysis. The cumulative binder tensile stress developed due to thermal contraction was determined numerically and compared to the binder tensile strength at the same temperature; the intersection of stress and strength was considered T_{cr}. A simple damage transfer function was used to determine pavement thermal stress by multiplying the binder thermal stress by a pavement constant (PC). Calibration of the transfer function yielded a best fit value of PC = 24 using observed SETC temperatures from the Lamont test road in Alberta, Canada (seven different binders). Data from a test road in Pennsylvania was used to validate the analysis (three binders). A single cooling rate of 1 C/hr was used for all calculations but it was noted that maximum cooling rates for different geographic regions have been observed to range from 0.5 C/hr to 3 C/hr.

Rowe et al. (2001) evaluated four different numerical fitting techniques for producing master curves of relaxation modulus from *BBR* data. The methods evaluated included: 1) Christensen-Anderson (CA); 2) Christensen-Anderson-Sharrock-Bouldin (CASB); 3) Christensen-Anderson-Sharrock (CAS); and 4) Discrete Spectrum (DS). They found that the CAS method generally provided the best fit of the experimental data used for the evaluation.

Shenoy (2002) demonstrated a variation of the T_{cr} analysis method of Bouldin et al. (2000) that did not require binder direct tension data. The method involved fitting of asymptotes to the thermal stress curve developed during T_{cr} analysis. The first asymptote was fitted to data at the end of the thermal stress curve (i.e. lowest temperatures evaluated) and a second asymptote was fitted to data at the beginning of the thermal stress curve (i.e relatively high temperature). A single asymptote procedure (SAP) calculated the intersection of the first asymptote with the x-axis (i.e. zero thermal stress). A two asymptote procedure (TAP) calculated the intersection temperature of the two asymptotes as T_{cr}. Forty-nine binders were evaluated by Shenoy according to the proposed TAP and SAP methods as well as the dual instrument method (DIM) developed by Bouldin et al. (2000). For 90% of the binders evaluated, the T_{cr} values determined by TAP were within 1 C of the DIM values and within 1.5 C for the SAP. For the other binders the maximum difference between TAP and SAP values and the DIM values were 2 C and 2.8 C respectively. Shenoy noted that selection of a different reference temperature for computation of thermal stress by any of the procedures would change the computed T_{cr} temperature somewhat.

Marasteanu et al. (2004) found that for the standard one hour specimen conditioning time, deviations from the time-temperature superposition assumptions did not greatly affect results with either the DIM or SAP methods of T_{cr} analysis. For nine asphalt binders investigated the DIM results were found to be different from SAP results; the differences were -1.6 to 5.7 C for a PC of 18 and were -3.6 to 3.6 C for PC of 24. They recommended that the asymptote in the SAP method be fitted to data at consistent thermal stresses and not at consistent analysis temperatures for better results.

The T_{cr} thermal stress analysis technique originally developed by Bouldin et al. (2000) and refined by others was formalized into *AASHTO R 49-09: Standard Practice for Determination of Low-Temperature Performance Grade (PG) of Asphalt Binders* (AASHTO 2009). The method requires *BBR* and direct tension test data as well as assumptions of cooling rate, linear thermal contraction coefficient. Relaxation master curves are determined from *BBR* data by the Christensen-Anderson-Marasteanu (CAM) method. A pavement constant (PC) value of 18 is specified. The technique could be extended for analysis of asphalt mixture data with use of appropriate parameters.

Shah et al. (2007) investigated low temperature properties of plant produced mixtures containing, 0, 15, 25 and 40% RAP using indirect tensile creep compliance strength data. Thermal stress analysis was performed with the data and used to estimate T_{cr} for the mixtures. Results indicated that for mixtures with standard virgin binder grade (PG 64-22) the estimated T_{cr} temperatures for mixtures with RAP were 3 to 6 C higher than for a 0% RAP mixture (i.e. less resistance to thermal cracking); interestingly, T_{cr} values were nearly identical for mixtures with 15 and 40% RAP (0.5 C difference). Use of a softer binder grade (PG 58-28) improved performance in 25% RAP mixtures but did not result in much change for 40% RAP mixtures (0.6 C difference).

Daniel et al. (2010) investigated properties of plant produced mixtures containing 0, 15, 20 and 25% RAP. Using the T_{cr} analysis method on recovered binder, they observed that increasing the proportion of RAP asphalt in the mix improved the low temperature properties (i.e. calculated T_{cr} was lower). The results did not correspond to the low temperature grading results using just *BBR* data. The authors mentioned that the extraction and recovery process resulted in complete binder blending and that this was not necessarily representative of the mixture properties.

2.6.4 Bending Beam Rheometer Mixture Testing

Zofka et al. (2005) presented a method to measure low temperature stiffness properties with the Bending Beam Rheometer (*BBR*) on thin beams of asphalt mixture made from gyratory compacted specimens. Mixture beams were prepared from 6 asphalt mixtures that contained 0, 20, or 40% RAP and either PG 58-28 or PG 58-34 virgin binder. Testing of the mixture beams was performed at -18 and -24 C. Measured stiffness of the mixtures increased as RAP was added to the mixtures for both test temperatures. A limited amount of indirect tension testing was also performed on the mixtures and results indicated that mixture stiffness at 60 seconds of loading time as measured by the two methods was fairly similar for both test temperatures.

Zofka et al. (2008) used the *BBR* mixture beam test and the indirect tension test to produce creep compliance curves for twenty asphalt mixtures. The data was then used as input for the thermal cracking module of the Mechanistic-Empirical Pavement Design Guide (MEPDG) to estimate thermal cracking of a pavement over its service life. The two test methods were observed to yield slightly different creep compliance curves, however the authors developed and presented an easy to use shifting function to transform *BBR* results to results by indirect tension testing. Comparison of thermal cracking estimates with the MEPDG using data from both test methods showed that similar data from both test methods for the same mixture resulted in similar predicted thermal cracking performance.

Use of the *BBR* mixture beam test on specimens of 100% RAP mortar (RAP particles smaller than 2.36 mm) with virgin binder was investigated by Ma et al. (2010) and Bautista et al. (2009). Good correlations were observed between mortar stiffness and binder stiffness and a method was developed to estimate low temperature properties of RAP asphalt without extraction and recovery. The method however, is complex and requires testing of several combinations of RAP mortar and new binder. Also the final method developed only uses RAP particles passing the 0.3 mm sieve (#50) and retained on the 0.15 mm sieve (#100); it is not clear if the asphalt on this one particle size of RAP is representative of all the RAP asphalt.

Marasteanu et al. (2009) evaluated use of the *BBR* mixture beam test method. Statistical analysis of dimensions for 660 mixture beams showed that test specimens of appropriate dimensions can be reliably prepared. A sensitivity analysis performed using the Hirsch model showed that with a target mixture air void level of 4%, a range of air voids up to 4% above or below the target value (i.e. air voids ranging from 0 to 8%) would have a very small affect on measured asphalt mixture stiffness at low temperatures. The error would be 2% or less and for test loading times of about ninety seconds and longer, the error would be less than 1%. Three dimensional finite element modeling of *BBR* mixture beam testing was performed with the ABAQUS finite element code; a digital image scanning technique was utilized to base the ABAQUS models on actual mixture beam specimens. Results showed that the distribution of aggregate particles within a mixture beam was very important to measured stiffness. It was possible for distributions of aggregate to occur that produced a cross section in the beam composed entirely of asphalt mastic. For these beams the stiffness values were dramatically lower than for beams with a more uniform aggregate particle distribution.

Velasquez (2009) investigated the sensitivity of determination of asphalt mixture stiffness at low temperatures to varying test specimen dimensions. A combination of experimental data and finite element modeling was utilized. Ten mixtures were tested at three test temperature with a variety of test specimen dimensions. Results showed that as testing temperature is decreased the disparity in binder and aggregate stiffness lessens and the mixture stiffness becomes less reliant on aggregate size and distribution. The consequence of this is that the minimum test specimen dimensions required to ensure that a representative volume element (RVE) of the mixture is obtained become much smaller. Experimental data suggested that the *BBR* mixture beam test method could produce representative measurements of creep stiffness for asphalt with a minimum of three test replicates even when the nominal maximum aggregate size (NMAS) of the mixtures was greater than the smallest dimension of the beam.

Marasteanu and Anderson (2001) discussed how to identify errors in rheological test data for asphalt binders (not asphalt mixtures). For *BBR* test data a quick check of the results was recommended to verify that slope of the m-value parameter decreases as the loading time increases. Data that did not follow this rule of thumb would likely be due to testing error. The concept could be extended to asphalt mixtures.

2.6.5 Relevant Pavement Density Parameters

A department of transportation (DOT) survey questionnaire reported by Linden et al. (1989) with 48 respondents provided the following information. Core samples were used by essentially all responders in some form. Twenty-one respondents used a maximum V_a density criteria (1-10%, 1-9%, 12-8%, 2-7.5%, 5-7%). One agency noted rejection below 8% V_a, while seventeen agencies reported price adjustments and thirteen additional agencies reported price adjustments or removal and replacement below their minimum density limit; a range of other options were also reported with less frequency. The average maximum in place air voids reported by state DOT's was 9.9% with a range of 5 to 15%. Current in-place air voids requirements for Southeastern United States DOTs are summarized in Table 2.3.

Measurement of air voids using different methods poses challenges. Questions arise such as what are the issues between different test methods due to surface texture or water absorption? Another issue that has been speculated to occur is volume reduction in the specimens tested by the Corelok® method as a result of the vacuum pressure applied during removal of air from the specimen. Buchanan (2000) indicated that the Corelok® method provided the most consistent and accurate results of specimen bulk gravity for compacted mixtures with high air void contents.

Table 2.3 In-Place Air Void Specification Summary of the Southeastern U.S.

	Specification State and V_a Requirements								
Surface Layer	**MS**	**AL**	**GA**	**FL**	**SC**	**NC**	**AR**	**LA**	**TX**
Target Air Voids	7.0	6.0	<7.8	<7.0	6.0	<8.0	6.0	<8.0	7.0
Full Pay-High	7.0	9.7	7.8	8.1	7.8	8.0	8.0	89 PWL	8.5
Full Pay-Low	4.0	2.2	3.8	2.0	4.0	---	4.0	---	4.7
Removal Required	9.0	11.2	13.5	9.5	9.4	10.8	9.1	30 PWL	10.0

Notes: All states shown specify bulk gravity of roadway cores be measured by *AASHTO T 166* or an equivalent state test method utilizing submerged specimens. For states that specify a range of target in-place density, the median of the range is reported. Louisiana utilizes a percent within limits (PWL) criteria. References to all nine DOT specifications are provided in the references.

As of July 2010, the Corelok® was not part of MDOT protocol for dense graded mixtures (only used for OGFC design). Typically, a core is cut at the beginning of a project and G_{mb} measured via *AASHTO T 166*. The result is used to adjust nuclear density measurements, which are used for acceptance. Periodically, the correlation between *AASHTO T 166* and the nuclear gage measurement is updated throughout the project. Paraffin coated specimens are required when excessive water absorption occurs during the laboratory test.

Cooley (2003) studied permeability of Mississippi field cores. As part of the study G_{mb} of the cores was measured by the Corelok® method (*AASHTO T 331*) and by the submerged specimen method (*AASHTO T 166*). Twelve different mixtures (two 9.5 mm, five 12.5 mm, and five 19.0 mm NMAS) that included both fine and coarse aggregate gradations were tested for a total of 175 data points with air voids ranging from 3.8 to 12.4% via *T 166* (equivalent of 3.8 to 15.1% via *T 331*). For all twelve mixtures the two measurement methods yielded significantly different results. The author observed that similar results were obtained from both methods when the air voids were less than about 5% but that results from the two methods began to diverge noticeably as the air voids increased above 5%. The cause of the divergence in results between the two methods was stated to be likely due to the large interconnected voids present in specimens with high air void contents. The data indicated that coarse graded mixtures generally had larger differences in results than fine graded mixtures. Cooley (2003) presented a linear relationship between air voids measured according to the two methods (Eq. 2.3). According to the relationship, air voids of 7% and 10% measured by the submerged method were equivalent to air voids of 8.3% and 12.3% respectively as measured by the Corelok® method.

$$V_{a(T331)} = 1.333 V_{a(T166)} - 1.04 \qquad R^2 = 0.87 \qquad n = 175 \qquad \text{(Eq 2.3)}$$

Brown and Cross (1989) measured in place density of five pavements as part of a study of rutting; the pavements had between six and sixteen years of traffic at the time of investigation. Air voids were observed to range from nearly zero to 8%. Badaruddin and

White (1994) reported in place air voids of twenty-three pavements in Indiana that ranged from nearly zero to 13.2%. Lu (2005) reported that mean air voids of sixty-three pavements examined in California were about 7% and ranged from 2 to 14%. Two sets of data were used by Seo et al. (2007) to study air void reduction in service; first data set was from a study to refine gyratory compaction requirements, and the second data set was from cores taken from I-85 after two years of traffic. It was observed that 8 to 11% initial air void levels were around 6 to 8% after two years of service. Prowell and Brown (2007) measured density of forty pavements across the United States with air void levels that ranged from 5.0 to 14.5% immediately after construction. For the same pavements air voids ranged from 1.9 to 11.5% two years after construction. It was observed that about two-thirds of pavement densification due to traffic occurred in the first three months after construction.

Density specifications are often misused, as discussed by Brown (1990), as mixture changes (e.g. increases in fines or asphalt content) can reduce voids when the correct way to reduce voids of a properly designed mix is through compaction. Hughes (1989) recommended using 7% air voids as the mean requirement in conjunction with 1.5% standard deviation within statistically based end result specifications. Linden et al. (1989) used 7% air voids as a baseline and reported that every 1% air void increase resulted in approximately 10% loss in pavement life. Literature review and a DOT survey questionnaire were the data sources used by Linden et al. (1989). Literature review revealed 10 to 30% fatigue life reduction and 4 to 6% penetration reduction per percent increase in air voids. Multiple studies (e.g. Huber and Heiman 1987) report air voids below 3% are a primary indicator of rutting. Brown (1990) reported that in place air voids of 3 to 8% would generally provide good performance of a surface mixture passing through or above the restricted zone.

2.6.6 Loaded Wheel Rut Testing

Sivasubramaniam et al. (2004) compared data from the PURWheel laboratory loaded wheel tester to the Purdue accelerated pavement tester (APT) and to mixture performance on the NCAT test track. All PURWheel testing was performed at 50 C. The authors found relatively weak correlations (R^2 = 0.35) between PURWheel results and mixture performance at the NCAT track when PURWheel testing was performed on slab specimens cut from the APT test sections (air void range 8.0 to 11.6%). Much better correlations (R^2 = 0.69) were found between PURWheel results and NCAT test track results when PURWheel testing was performed on slab specimens cut directly from the NCAT test track (air void range 4.9 to 6.6%). The difference in the correlation was stated to be likely due to differences in air voids and compaction parameters.

Sivasubramaniam et al. (2004) used the power law model given in Eq. 2.4 that was fitted to rut data for analysis; the total rut depth was considered to be the sum of specimen downward deformation in the wheel-path and any specimen uplift relative to the original surface outside of the wheel-path. The model constant *a* was stated to be related to mixture properties and initial air voids. The model constant *b* was stated to depend on test temperature as well as mixture type and to be a good indicator of mixture rutting potential.

$$\text{Total Rut} = a\left(N_p\right)^b \qquad \text{(Eq 2.4)}$$

where:

N_p = number of load applications

a, b = material constants

2.6.7 Moisture Sensitivity

Kiggundu and Roberts (1988b) defined moisture damage as “the progressive functional deterioration of a pavement mixture by loss of the adhesive bond between the asphalt cement and the aggregate surface and/or loss of the cohesive resistance within the asphalt cement principally from the action of water.” Kiggundu and Roberts (1988*a*) stated that the mechanisms of stripping “are likely to be asphalt-aggregate specific, environmentally specific and service conditions specific.”

Moisture damage in asphalt mixtures is generally thought to be due to one of or a combination of two major causes: 1) loss of cohesion within the binder film (i.e. softening of the binder in the presence of moisture); and 2) loss of adhesion between the binder film and the aggregate particles (Hicks 1991). Loss of adhesion in a mixture is visually apparent (e.g. stripping). Loss of cohesion in a mixture is less readily observed visually, but can be measured by loss of strength of the mixture.

Kandhal (1992) identified a number of factors that can lead to premature failure of pavements due to moisture damage. They include: 1) poor pavement drainage; 2) poor compaction; 3) excessive dust coating of aggregate; 4) insufficient drying of aggregate during production; and 5) use of weak aggregates. Hughes (1989) also emphasized the importance of good compaction in the field to reduce the level of permeable voids in producing pavements that are resistant to moisture damage.

Terrel and Al-Swailmi (1993) discussed a hypothesis for moisture damage in asphalt mixture that was developed during the Strategic Highway Research Program (SHRP). Laboratory test data was utilized to support the hypothesis that a range of air voids on the order of 7 to 11% in compacted mixture will produce a void structure and conditions that are favorable to occurrence of moisture damage. The range of air voids where this occurs was termed "pessium" voids (i.e. opposite of optimum). Void levels less than this range result in a relatively impermeable pavement where moisture does not intrude and void levels higher than this range result in a relatively free-draining pavement where moisture cannot remain for long periods of time (i.e. PFC or OGFC).

Cooley et al. (2001) investigated permeability and density of eleven coarse-graded field mixtures and found that for 9.5 and 12.5 mm NMAS mixtures excessive permeability occurred at air void contents greater than approximately 7.7%. For 19.0 mm NMAS mixtures, excessive permeability was observed for air voids greater than about 5.5%. For 25.0 mm NMAS mixtures the critical air void content was about 4.4%. The data of Cooley et al. (2001) supports the hypothesis of Terrel and Al-Swailmi (1993).

2.6.7.1 Tensile Strength Ratio (*TSR*) Testing

Amirkanian and Williams (1993) investigated indirect tensile and resilient modulus strengths of 15% RAP HMA mixtures with both laboratory specimens and field cores. The source pavement for the RAP was known to be moisture damaged before reclamation. Results indicated that the RAP mixture had significantly higher wet and dry tensile strengths and resilient moduli than the 0% RAP control. *TSR* and resilient modulus ratio values were higher than the control, but not significantly.

Castro-Fernandez (1996) performed moisture resistance testing with two different HMA mixtures from Nevada with RAP contents between 0 and 70%. Virgin aggregate type was not specified; PG 64-22 binder was used for the 0% RAP mix. Blends of RAP and very soft binders were selected for mixtures with RAP based on binder blending charts. When lime was included, *TSR* values were acceptable for both mixtures with any amount of RAP. When lime was not included the amount of RAP had a significant effect on *TSR* values. For both mixtures without lime and between 0 and 30% RAP the *TSR* values were below 0.50 (as low as 0.30); when the RAP content was increased to 50% the *TSR* value for one mixture was 0.60 and for the second mixture was greater than 0.80. For both mixtures with 70% RAP the *TSR* values were greater than 0.80. The results of Castro-Fernandez (1996) indicate that inclusion of RAP in moisture susceptible mixes was able to significantly improve the moisture resistance of the two mixes studied.

Zaniewski and Viswanathan (2006) reported on use of the *AASHTO T 283* test method for three mixtures of known good field performance. The 16 hour loose mix aging at 60 C required by the test method was included as part of the specimen preparation method; conditioning by vacuum saturation alone, and by one freeze-thaw cycle in addition to vacuum saturation was evaluated. Results indicated that test method was not sensitive to saturation level or to inclusion of a freeze-thaw cycle as part of the conditioning protocol. The *TSR* results indicated that all three mixtures were moisture sensitive; the authors concluded that "*TSR* is not a reliable indicator of field performance" for the mixtures tested.

Bagampadde et al. (2006) stated that variability in moisture sensitivity test data was primarily due to aggregates and not to binder. Chen et al. (2007) studied the effect of RAP on moisture sensitivity with mixtures produced using blends of virgin binder and recovered RAP asphalt mixed with aggregate. Increasing the amount of recovered RAP asphalt was observed to have a detrimental effect on *TSR* results. It is important to note that the testing of Chen et al. is not representative of actual practice since the bond between the RAP asphalt and aggregate was broken by the extraction and recovery process. Chen et al. (2007) also presented a concept of relative energy loss to analyze indirect tensile strength test results and found the concept was capable of identifying moisture susceptibility.

Al-Qadi et al. (2009) studied HMA mixtures containing 0 to 40% RAP and found that in general *TSR* values improved as RAP was added to the mixture. PG 64-22 was used for the 0 and 20% RAP mixtures and PG 58-28 was used for the 40% RAP mixture; details of the aggregates used were not provided. The authors stated that selective absorption of binder into aggregate for RAP could potentially produce a bond that was resistant to stripping and also that incomplete blending could result in double coating of RAP particle resulting in improved *TSR* values.

2.6.7.2 Loaded Wheel Testing

Aschenbrener (1993) used the HLWT to evaluate moisture damage for twenty pavement mixtures of known field performance ranging from very good to very poor. Testing was performed on slab specimen at 50 C; stripping inflection points (SIPs) were computed from the test data and used for mixture evaluation and comparison. Specific details of the mixtures tested were not provided but the test results were found to be sensitive to aggregate properties such as amount of dust coating, dust to binder ratio, and clay content. The author observed that average SIP for mixtures as determined by the Hamburg test provided an excellent correlation to field performance with respect to moisture damage. Pavements with good field performance had average SIPs generally greater than 10,000 passes and pavements with poor field performance had average SIPs less than 3,000 passes.

Additional test data presented by Aschenbrener (1995) for four aggregates (details not given) and 4 asphalt binder grades (PG 52, PG 58, PG 64, and PG 70) indicated that moisture resistance of the mixes was improved by increasing the binder grade. The amount of short term aging used in the laboratory was also found to affect HLWT results; more short term aging resulted in better performance. Aschenbrener (1995) also found not all binders graded as PG 58-22 provided the same performance; crude oil source and refining process were observed to influence HLWT results.

Pan and White (1999) conducted moisture sensitivity testing of seven mixtures and a variety of anti-strip agents with both the PURWheel loaded wheel tester and *AASHTO T 283 TSR* testing. Results indicated that in general the PURWheel provided a better indication of the stripping potential of a mixture than *TSR* testing. Results of PURWheel testing were able to demonstrate stripping in mixtures at test temperatures ranging from 25 to 60 C.

Izzo and Tahmoressi (1999) evaluated the repeatability of the Hamburg loaded wheel tester (HLWT) and stated that its use with steel wheels provided good repeatability on gravel mixes and poor repeatability for limestone mixes. Use of solid rubber wheels with the HWLT was observed to produce significantly less damage than steel wheels.

Hall and Williams (1999*b*) performed a limited evaluation of the HWLT in comparison to the Evaluator of Rutting and Stripping in Asphalt (ERSA); both field and laboratory compacted specimens of a field produced mixture were tested. Gyratory compacted specimens were observed to have significantly lower rut depths than field compacted specimens.

Cross et al. (2000) performed testing of eight different mixtures with the *APA* at 40 C. Results of wet testing using one of three pre-conditioning procedures was compared to results of standard dry testing. The three pre-conditioning procedures were 1) 2 hour soak at 40 C; 2) specimen saturation and 24 hr conditioning at 40 C; and 3) specimen saturation plus one freeze-thaw cycle followed by 24 hr conditioning at 40 C. The authors found that the 2 hour soak pre-conditioning procedure produced mean rut depth results that were significantly different from the dry *APA* test data. The other two preconditioning procedures were not found to produce statistically significant differences in mean rut depths compared to dry testing. The *APA* results corresponded well with *TSR* results from standard *AASHTO T 283* testing in ranking moisture susceptibility of mixes. Hunter and Ksaibati (2002) performed wet testing of asphalt mixtures with the Georgia loaded wheel tester (GLWT) and found that neither saturation conditioning nor saturation plus one freeze-thaw cycle conditioning prior to testing significantly affected results.

West et al. (2004) evaluated under water testing in the *APA* for moisture sensitivity assessment of asphalt mixtures. Data was obtained for a variety of mixtures, specimen geometries, load application methods, and conditioning protocols. Initially testing was to be performed at 64 C; however that temperature was found to be too severe for steel wheel testing and therefore all testing was performed at 50 C. The results were mixed but some general conclusions were presented by the authors. Pre-conditioning of specimens by vacuum saturation and one freeze-thaw cycle was stated to appear to be able to distinguish mixtures prone to stripping from those that were not. The steel wheel load application method was stated to be more severe than pressurized hose load application method but also appeared to be more variable. The authors further indicated that wet testing of unconditioned specimens was inadequate to cause stripping. Beam specimens were found to generally yield the most meaningful results but cylindrical specimens were stated to be more practical.

Buchanan and Smith (2005) tested 24 Mississippi mixtures with a rotary wheel tester; the test method was found to be a severe performance test for moisture susceptibility. Gravel mixtures were found to exhibit much greater deformation than gravel/limestone mixtures. A normalized rut depth parameter (specimen deformation divided by number of test cycles) was used during analysis.

Lu (2005) performed an extensive evaluation of the HLWT on California pavements. Test parameters were 50 C test temperature and a 30 minute wait period once specimens were placed in the machine for the water bath to reach temperature. Twelve laboratory mixtures were evaluated by testing of slabs specimens (24 by 33 by 7.6 cm) prepared by rolling wheel compaction to between 6 and 8% air voids. Cores were also taken and tested from 57 pavement sections of known moisture performance. Lu (2005) found that the HLWT did not clearly distinguish between mixes with different observed moisture sensitivities. The test method tended to overestimate performance of mixes with conventional binder and to underestimate performance of mixes with polymer-modified binder. The author provided five major recommendations to improve the HLWT: 1) vacuum saturation of specimens prior to testing; 2) pre-conditioning of specimens at the test temperature prior to testing; 3) use of different test temperatures based on binder grade; 4) performing wet and dry tests on mixtures and using a ratio of results for evaluation; and 5) that the HLWT equipment be modified with an air-heating system or environmental chamber to maintain high air temperatures during testing.

Kim and Lutif (2006) performed moisture susceptibility testing on an aggregate blend of limestone and gravel with combinations of mineral fillers and lime treatment (16 mixture combinations) with *TSR*, wet *APA*, and wet HLWT test methods. All mixes were HMA and the binder was PG 64-22. *TSR* testing according to *AASHTO T 283* was performed with six freeze-thaw conditioning cycles. Wet *APA* testing was performed at 64 C after a 16 hour conditioning period on gyratory compacted specimens with a target 4% air void content. Wet HWLT testing was performed at 70 C after a minimum 30 minute temperature equilibrium period on gyratory compacted specimens with target 7% air voids. The three test methods provided consistent rankings of aggregate blends by moisture susceptibility from high to low.

Shiwakoti (2007) compared the *APA* and HLWT for moisture sensitivity assessment for six HMA mixtures with acceptable *TSR* values (>0.80). Five of the six mixtures used PG 64-22 binder and the sixth mixture used PG 70-28 binder; details of aggregate types were not provided but were a range of materials used in Kansas. The wet HLWT was performed at 50 C on gyratory compacted specimens; a minimum 30 minute soak period was specified after the water bath reached temperature before testing commenced. Both wet and dry *APA* testing at 50 and 60 C test temperatures was performed on gyratory compacted specimens vacuum saturated then brought to the test temperature and allowed to soak for at least one hour before testing commenced. Pressurized hose load application was used for all *APA* testing. Four of the six mixes tested exhibited visual evidence of stripping in wet *APA* testing; the *APA* did not indicate any stripping inflection point (SIP) in the test data for any of the mixes. The HLWT correctly showed SIPs for the four mixes which exhibited visual evidence of stripping and did not show SIPs for the two mixtures without visual stripping. For *APA* testing conducted at 60 C the dry test exhibited greater rut depths than the wet test. For *APA* testing conducted at 50 C the opposite trend was observed; the wet test exhibited greater rutting than the dry.

Cooper (2009) used *TSR* and HLWT testing to evaluate a mixture both with and without Sasobit®; the base binder was a polymer modified PG 76-22. The mixture was 75% limestone, 6% sand, and 19% RAP. HMA Compaction temperature was 157 C and compaction temperature for the Sasobit® mix was 143 C. Hamburg testing was performed according to *AASHTO T 324* at 50 C after 90 minutes of conditioning; 320 by 260 by 80 mm slab specimens prepared by kneading compaction were utilized for Hamburg testing. Mixtures with and without Sasobit® both erformed well in *TSR* and HLWT testing and were stated to not exhibit evidence of moisture susceptibility. The mixture containing Sasobit® rutted less than the mixture without, however the difference was hypothesized to be at least partly due to lower air voids of the Sasobit® specimens.

Nielson (2010) evaluated the test temperature used in the HLWT for three asphalt binder grades and two asphalt binder sources (six combinations) on a single blend of limestone aggregate with a known history of stripping. All mixes were HMA and the binders tested were PG 70-28, PG 64-28, and PG 58-28. Slab specimens were prepared by linear kneading compaction and tested at a range of test temperatures from 45 to 60 C. The author defined a critical stripping temperature (CST) below which no stripping would occur and above which stripping would occur; the existence of a stripping inflection point (SIP) in the data was considered evidence of stripping. Based on the data, it was stated that the number of cycles required to induce stripping in the mix was highly variable and independent of the test temperature and binder grade, provided the test temperature was greater than the CST for the mix. A range of CSTs was reported for each PG high temperature grade tested. For PG 58 binder grade the reported range of CSTs was 44 to 49 C. For PG 64 binder grade the reported range of CSTs was 49 to 54 C. For PG 70 binder grade the reported range of CSTs was 54 to 55 C. The author recommended that the best approach would be to select HLWT test temperatures based on anticipated environmental conditions during service, including both geographic location and location within the pavement structure.

Azari (2010) conducted wet HLWT at 50 C and *TSR* testing of two mixtures of general low and high moisture susceptibility and observed that HLWT results were more consistent with observed field performance of the mixtures. The *TSR* results predicted that both mixes were acceptable with regards to moisture susceptibility.

Austermann et al. (2009) performed HLWT on hot and warm mixed 10 and 25% RAP mixtures. Two dosage rates of Sasobit® were investigated, 1.0 and 3.0% by total binder weight. All the WMA mixtures performed worse than the HMA control, however increasing either the Sasobit® dosage rate or the RAP content improved performance.

Mogawer et al. (2011a) performed HLWT at 40 C on hot and warm mixed 9.5 mm NMAS 40% RAP mixtures. The virgin binder grade was PG 52-28 and a wax based additive (not Sasobit®) was used for the warm mix. The RAP mixture provided good performance and no evidence of SIP at either hot or warm temperatures. Performance of the no RAP control mixture decreased noticeably for the WMA compared to the HMA and SIPs were evident for all control mixture testing.

2.7 Field Performance of High RAP

This section contains pertinent information from studies of high RAP mixtures. The focus was on field studies or studies of plant mixed material. The information is organized by topic in the following sections.

2.7.1 Airfields

Shoenberger and Demoss (2005) reported on performance of four recycled military airfield pavements containing 35, 40, 41.5 and 60% RAP after between eight and twelve years of service. Original construction data was evaluated when available as well as pavement condition index (PCI) survey data over the life of the pavement and current mixture properties at the time of evaluation. All of the mixtures were designed using the Marshall method with 75 blows per face; relatively soft asphalt grades were used and the 60% RAP mixture also used a recycling agent. Review of construction records indicated that adequate compaction was obtained. PCI data showed a relatively similar change with time for the recycled mixtures that was not dramatically different than what was observed for non-recycled mixtures at some of the airfields. High severity block cracking was observed in the 60% RAP mixture. The original mix designs had relatively high asphalt contents yet rutting was not observed to be a problem. Most of the observed pavement distresses were environmentally related. Resilient modulus testing of field cores indicated stiffness values within the normally observed range of airfield mixtures.

A recent review of the potential for use of RAP in airfield pavements (Hajj et al. 2010) found that previous use of RAP (less than 20%) in airfield pavements had performed acceptably or that the excessive distresses were not due to use of RAP. A municipal airfield in Illinois was performing well after five years that had used 100% RAP as base course underneath a new HMA overlay. Su et al. (2009) investigated use of 40 and 70% RAP mixes for airfield surfaces in Japan; the mixtures utilized rejuvenating agents and virgin asphalt. Laboratory test results from the Japanese raveling test indicated that good raveling performance could be anticipated for the recycled mixtures. Test sections were placed on an airfield taxiway and their performance monitored for three years. No cracking or raveling was observed during the monitoring period.

2.7.2 Highways

As early as 1975, Utah was experimenting with asphalt recycling (Betenson 1979). An initial trial section yielded good results and a second, larger field trial was conducted with 77%, 80%, and 100% RAP in 1977. The 100% RAP required 1.5% of AC-10 virgin asphalt and 0.5% of a softening agent with the goal of combined asphalt graded as AC-5. Severe problems with emissions requirements were seen during mix production likely due to how RAP was introduced directly into the drum plant. Resilient moduli of cores taken from the pavement containing RAP one year after placement were lower than those for the conventional 100% virgin mix placed at the same time.

In 1977, Arizona produced a recycled asphalt mix with 80% RAP and 20% virgin aggregate in a drum mix plant (McGee and Judd 1978). 2.7% of soft virgin AR 2000 binder (AR 2000 ≈ AC-5 or AC-10) was combined with 50% of an aromatic extender oil and added to the RAP mix. The overall asphalt content of the final mix as determined by extraction was 5.3%. The mix output temperature of from the plant was reduced to around 200 F and 2% moisture was added to the aggregate to meet emissions requirements.

Little and Epps (1980) evaluated 25 field projects constructed between 1974 and 1978 involving levels of RAP of 30 to 100%, with most utilizing 70% RAP or more. Both surface and base courses were included. Cores taken from the pavements and in place FWD testing were used to characterize the recycled pavement performance. An analysis was conducted to determine the appropriate pavement design structural coefficients for these pavement layers as used in the 1972 AASHTO pavement design guide. It was found that "based on the structural coefficient evaluation, recycled materials used as surface courses are comparable to conventional asphalt concrete surfaces." The surface courses containing RAP were found to be slightly stiffer compared to ordinary HMA surface layers. Little and Epps (1980) felt that recycled materials, while stiffer than conventional materials, would perform adequately in relatively thick pavement systems. However, the potential for fatigue cracking of recycled materials in thinner pavement systems was felt to be higher than conventional pavements and would warrant extensive further investigation.

Hossain et al. (1993) reported on the long term field performance of asphalt overlays containing 50% RAP placed on the surface of Interstate-8 in Arizona; all of the recycled overlay test sections experienced approximately 7 million equivalent single axle loads (ESALs) over their 10 year service life. Two rehabilitation strategies were evaluated: 1) mill and replace followed by overlay and 2) simple overlay. Functional pavement performance was evaluated by roughness and skid resistance measurements; results indicated that the "functional performances of recycled and virgin mix overlays were similar" Hossain et al. (1993). Pavement structural performance was evaluated by pavement condition index (PCI) ratings and by rut depth measurements. For the simple overlay rehabilitation strategy, the virgin sections performed better than the recycled sections. For the mill and replace followed by overlay rehabilitation strategy, "the recycled mix outperformed the virgin mix" Hossain et al. (1993). Rutting performance of the recycled and virgin sections was similar. For eastbound lanes, cracking of recycled sections was higher than for virgin sections; for westbound lanes, cracking of recycled and virgin sections was similar. No moisture damage problems were observed in any of the sections but this is not particularly surprising given the arid climate (average annual rainfall for the area was about 6 inches).

Paul (1996) compared pavements containing RAP to virgin mixtures. Pavements were constructed between 1978 and 1982 and were 6 to 9 years old at the time of evaluation. RAP percentages of 20 to 50% were incorporated and there were no significant differences found between the recycled and virgin mixtures. Evaluation was based on structural and serviceability aspects with a pavement condition rating (PCR) score and deflection measurements. Ten locations were sampled per roadway to determine material properties with time (e.g. asphalt content, viscosity, penetration, ductility, and gradation).

Kandhal et al. (1995) studied five projects that each consisted of a recycled section and a control section containing between 10 to 25% RAP. Laboratory and field characterization was performed, and paired *t*-testing indicated no significant differences between the RAP and virgin sections when the pavements had been in service 18 to 27 months. A state of recycling practice conducted by FHWA determined that well controlled and constructed pavements containing RAP had performed well up to 17 years after construction (Sullivan 1996).

Potter and Mercer (1997) reported on six field projects in the United Kingdom containing between 18 and 60% RAP. Both surface and base mixtures were included in the evaluation and the pavements had experienced between three and nine years of traffic. Rutting performance of the pavements was good and visual conditions surveys also indicated good performance of the recycled mixes. Deflection testing (equipment similar to FWD) indicated that structural capacity of the recycled sections was similar to the control sections. Accelerated full scale testing of a 50% RAP base mix indicated performance was as good as the conventional base mix control.

Chen and Daleiden (2005) reported on performance of a 30% RAP HMA test section on the surface of a highway in Texas after ten years of service. The test section performed as well as the virgin mixture control section. Two additional pavement sections that were hot-in place recycled with 75% RAP were also discussed. One of the sections was too brittle, and cracked much sooner than expected; the other section performed reasonably well.

Zaghloul and Holland (2008) looked at historical pavement data in California to investigate the long term performance of pavements with HMA containing 15% RAP compared to no RAP pavements. Data from 131 pavement sections was investigated (RAP and controls) and evaluated for structural adequacy, pavement roughness and pavement distresses. Results suggested that long term pavement performance with RAP mixtures would be comparable to other nearby pavements subject to similar conditions.

Maupin et al. (2009) reported on six projects placed in Virginia with HMA mixtures containing 20 to 30% RAP. Laboratory testing of plant produced mixture indicated no significant difference in performance between moderate RAP content and control mixtures for beam fatigue, *APA* rutting, or *TSR* moisture susceptibility tests.

Vavrik et al. (2008) evaluated nine hot mixed plant produced mixtures containing 15 to 40% RAP. The RAP was fractionated before use. Binder single grade bumping (high temperature PG reduced one grade) and double grade bumping (high PG reduced one grade, and low PG increased one grade) was investigated. Laboratory evaluation of the field produced mix was conducted for fatigue and stiffness (dynamic modulus). Fatigue performance of the high RAP mixtures was better than the Illinois DOT fatigue design criteria in use at the time. Stiffness of all the RAP mixes was higher than the assumed design value in use by the Illinois DOT. Mixtures with single bumped binder and double bumped binder grades had very similar stiffness; it was concluded that double bumping was not necessary.

West et al. (2011) evaluated the long term performance of eighteen projects from the Long Term Pavement Performance (LTPP) program encompassing sixteen states and two Canadian provinces; the projects ranged were eleven to twenty years old. The projects examined performance of 0 and 30% RAP HMA mixtures for overlays; most of the mixtures were placed before the advent of current Superpave mix design specifications. Results indicated that the RAP mixtures had a slightly higher incidence of fatigue, longitudinal, transverse, and block cracking but a slightly lower incidence of raveling. The RAP mixtures were observed to perform better than or equal to virgin mixtures in a majority of cases. The same LTPP data set was evaluated by Carvalho et al. (2010) who looked at FWD deflection measurements as an estimate of structural capacity of the pavements. No statistical differences were found between the RAP and virgin mixtures for any of the eighteen projects indicating that structural capacity was the same for virgin and 30% RAP overlays of the same thickness and comparable materials.

Aguiar-Moya et al. (2011) looked at the LTPP overlay project in Texas with virgin and 30% RAP HMA after seventeen years of service. Similar to West et al. (2011), the authors determined that RAP overlays had a generally improved resistance to rutting but a somewhat higher incidence of all types of cracking distresses compared to virgin overlays. They used the observed pavement performance data to calibrate models that predict pavement service life. They then used the models in conjunction with typical Texas material costs and traffic parameters to estimate life cycle costs of the various rehabilitation alternatives. Their analysis indicated that the potentially reduced lifespan of RAP mixtures due to earlier crack development may negate initial cost savings, especially for thin overlays on the order of 50 mm. It is important to remember that the RAP mixtures examined by West et al.(2011) , Carvalho et al. (2010), and Aguiar-Moya et al. (2011) were almost all designed prior to the advent of Superpave and did not include any warm mix technologies so their properties are not entirely representative of current practice mixes.

2.7.3 Accelerated Loading Facilities

West et al. (2009) reported on performance of 20 and 45% RAP HMA test sections compared to a 0% RAP control section under accelerated full scale loading. Six surface mixtures were produced: 1) 20% RAP with PG 76-22 binder; 2) 20% RAP with PG 67-22 binder; 3) 45% RAP with PG 52-28 binder; 4) 45% RAP with PG 67-22 binder; 5) 45% RAP with PG 76-22 binder and 6) 45% RAP with PG 76-22 binder and 1.5% Sasobit® as compaction aid (no change in mix temperature). The RAP was fractionated into coarse and fine stockpiles before production. It was observed during construction that the mixtures with 45% RAP and polymer modified PG 76-22 binder (mixes 5 and 6) required more effort to reach the target compaction level.

The pavement structure for the test sections was designed as a perpetual pavement with 560 mm of HMA on top of an aggregate base. Each mixture was placed in a 50 mm thick layer on the previously milled surface. Performance of the test sections was monitored weekly for two years during which time traffic of approximately 9.4 million ESALs was applied. Pavement surface macrotexture results indicated that all RAP sections had very good raveling performance. Some low severity longitudinal cracking was observed in the mixture 6 test section that was determined to be reflective from the underlying pavement. Some low severity cracking was also observed in the mixture 1 test section. Field rutting of the mixture 2 test section was higher than the control section; rutting of all other test sections was less than the control section. Over the two years of testing, roughness of the control mixture and mixture 2 test sections increased somewhat. Roughness of the other test sections did not change noticeable over the study period.

Laboratory testing was also performed on the plant produced mixtures by West et al. (2009). *APA* rut testing results indicated a similar ranking of mixture performance to what was observed in the field; mixtures 5 and 6 had the lowest laboratory rut depths and the control mixture had the highest. Dynamic modulus testing indicated the generally expected ranking of stiffness in most cases, with the control mixture being least stiff and the 45% RAP mixtures being stiffest; 20% RAP mixtures fell in between. Strain controlled beam fatigue testing was conducted on laboratory conditioned specimens of each mixtures; results indicated that the 45% RAP mixtures had the lowest number of cycles to failure (failure was defined as 50% reduction in initial stiffness) while the 20% RAP mixtures were only slightly worse performing than the control mixture. Dissipated creep strain energy (DCSE) testing was conducted on field cores to generate an estimate of potential for top down cracking of the mixtures. Results indicated that the control mixture and all RAP mixtures with polymer modified PG 76-22 binder would likely have good resistance to top down cracking. DCSE results indicated that RAP mixtures with PG 67-22 and PG 52-28 binder might be susceptible to top down cracking, but field performance of the corresponding test sections was good.

2.8 Performance Studies of RAP and Warm Mix

This section contains pertinent information from studies of warm mixed RAP; both laboratory and field studies are included. The information presented was restricted to the warm mix techniques investigated in this study. Relevant information about the particular warm mix techniques is also included as applicable. The information is organized by topic in the following sections.

2.8.1 Sasobit®

Sasobit® is an organic hydrocarbon based wax produced by the *Fischer-Tropsch* process (SasolWax 2004). It is manufactured by Sasol Wax GmbH. It has been used in Europe for a number of years and has performed well in service (D'Angelo et al. 2008). Above its melting point of 100 C (212 F) Sasobit® reduces the measured asphalt viscosity which permits reduction of the mix temperature and promotes asphalt mixing and compaction. Below its melting point Sasobit® solidifies into a lattice structure that stiffens the asphalt binder (SasolWax 2004) and (Mallick at al. 2008). The reduction in mix temperature with Sasobit® is thought to reduce binder aging which will help compensate for its stiffening effects (Hurley and Prowell 2005b).

Laboratory investigation of Sasobit's® effects on volumetric criteria, mix stiffness with indirect resilient modulus, rutting potential in the *APA*, and moisture sensitivity with the *TSR* test and the Hamburg wheel tracking device has been performed (Hurley and Prowell 2005b). Three PG binder grades and two different aggregate types (granite and limestone) with similar gradations were used at a range of temperatures. Volumetric criteria were met in mixes with Sasobit® and air voids were generally reduced compared to the control specimens. Results indicated that the potential for rutting was reduced with the use of Sasobit® and the resilient modulus was not significantly affected. Moisture sensitivity was found to be a potential issue with Sasobit® due to incomplete aggregate drying at lower mixing temperature.

A number of field trials with Sasobit® have been constructed in the United States. Hurley and Prowell (2008) reported on two test sections constructed with Sasobit® in Milwaukee and St. Louis that mix properties were identical or improved in comparison to the virgin controls. The exception being a possibly increased susceptibility to moisture damage as indicated by laboratory tests run on the field mixed asphalt. Two trial pavement sections with Sasobit® were placed in late 2006 in Virginia (Diefenderfer et al. 2007). The mixtures used for the sections contained 20% and 10% RAP. 1.5% Sasobit® by total binder weight was added to both mixtures. No significant changes in volumetric properties or rut measurements in the *APA* were seen. One trial section did not meet the *TSR* requirements but it was thought this was likely due to high stockpile moisture conditions and lower mix temperature during production.

Mallick et al. (2007) investigated use of 100% RAP as a base layer by the addition of 2.0% neat PG 64-28 asphalt binder in the laboratory. Sasobit® at 1.0% and 1.5% of total asphalt content was tested in 100% RAP at 125 C and compared to 100% RAP without Sasobit® at 150 C. The resulting mixtures were evaluated for workability, compactibility, resilient modulus, moisture sensitivity, and indirect tensile strength. Workability results indicated that the use of Sasobit® at 125 C either increased the workability (mix was less stiff) or was nearly the same as the 150 C mix without Sasobit®. Resilient modulus was measured and no statistical difference was found between the 150 C RAP mix and the Sasobit® with RAP mixes. Tensile strength was significantly lower for the 1.0% Sasobit® mix compared to the no Sasobit® mix in the dry state and after one freeze-thaw conditioning cycle, but the retained strength values were not statistically different.

Similar laboratory work performed by Mallick et al. (2008) used 75% RAP with Sasobit® and varying grades of additional virgin binder for base courses. The goal was to create mixtures containing 75% RAP with similar performance properties to a control mixture that consisted of 75% extracted RAP aggregate mixed with 25% virgin aggregate and neat PG 64-28 binder at 150 C (the specified mixing temperature for this binder). Tests for air voids, tensile strength, stiffness, and rutting were designated as the comparison criteria. The control mix with PG 64-28 binder had the highest average tensile strength (at -10 C) of any of the mixtures while the mix with PG 42-42 binder and Sasobit® H8 had the lowest. This indicates a reduction in overall mixture stiffness and the potential for low-temperature cracking with the use of a much softer neat asphalt binder. Rut depths were less than 4 mm for all mixes. The seismic moduli stiffness results indicated that the mix produced with PG 42-42 binder and Sasobit® H8 had a significantly lower modulus than mixtures produced with PG 52-28 binder. Similar levels of performance to conventional HMA for 75% RAP mixtures was possible with the use of very soft grades of asphalt binder and Sasobit® H8 warm mix additive. Similar air voids and comparable mixture stiffness was observed in the mixtures as well as an equal or decreased rutting potential. Although mix temperatures were intentionally not greatly reduced, the addition of Sasobit® H8 to mixes containing RAP produced comparable air voids to RAP mixes at standard mix temperature without additive.

Kristjansdottir et al. (2007) presented a case study in Maryland where Sasobit® was used as workability and compaction aid for mixtures with 35 to 45% RAP. Production temperatures were 138 to 166 C (280 to 330 F) and compaction temperatures were 135 to 154 C (275 to 310 F). No adverse affects were reported based on laboratory and field data (primarily construction) and the authors note that long term performance data is needed to make comparative assessments. Prowell and Hurley (2007) summarize thirteen field test sections that incorporate Sasobit®. They contained 0 to 45% RAP (6 with 0% RAP, 5 with 10 to 25% RAP, 1 with 35% RAP, and 1 with 45% RAP). The thirteen projects were a combined 21,300 tons.

2.8.2 Evotherm™

Evotherm™ 3G is a proprietary formula liquid asphalt additive designed to improve coating, mixing, workability and compaction of asphalt mixtures (MeadWestvaco 2011). Laboratory investigation of Evotherm™'s effects on volumetric criteria, mix stiffness with indirect resilient modulus, rutting potential in the APA, and moisture sensitivity with the *TSR* test and the Hamburg wheel tracking device has been performed (Hurley and Prowell 2006). Two PG binder grades and two different aggregate types (granite and limestone) with similar gradations were used at a range of temperatures. Volumetric criteria were met in mixes with Evotherm and air voids were generally reduced compared to the control specimens. Compaction was improved relative to HMA at temperatures down to 88 C. Results indicated that the potential for rutting was significantly reduced with the use of Evotherm when used at hot mix temperatures, but rutting was significantly increased for Evotherm mixtures at lower temperature relative to the HMA control. Resilient modulus was lower than the control mixes in some cases. Moisture sensitivity was found to be a potential issue with Evotherm™ and the limestone aggregate; however a change in Evotherm™ formulation by the manufacturer was able to correct the problem.

Prowell et al. (2007) examined the laboratory and early life field performance of WMA mixtures produced with Evotherm™. The mixing temperature was 115 C and the Evotherm™ mix was successfully compacted at temperatures ranging from 108 C all the way down to 96 C. Laboratory *APA* testing on specimen of plant produced mixture indicated rutting performance comparable to the HMA control mixture while field rutting data indicated nearly identical performance between the WMA and HMA. Laboratory *TSR* moisture susceptibility testing indicated potential problems with the WMA mixtures; however *TSR* testing of cores from the test section did not indicate much difference between HMA and WMA *TSR* results.

Prowell and Hurley (2007) summarize eighteen field test sections that incorporate Evotherm™. They contained 0 to 30% RAP (8 with 0% RAP, 7 with 10 to 25% RAP, 1 with 20% RAP, and 2 with 30% RAP). The eighteen projects were a combined 48,600 tons. Prowell and Hurley also stated that over 100,000 tons of WMA produced with Evotherm™ had been placed as of 2007.

Kvasnak et al. (2009) tested laboratory and plant produced 15% RAP mixture with Evotherm™ and HMA. Three measurements of moisture susceptibility were investigated, *TSR*, HLWT, and absorbed energy ratio (analysis technique using indirect tensile test data from the *TSR* test). Results generally indicated that the WMA might be more susceptible to moisture but most of the data passed the test criteria. The plant produced WMA performed better than the laboratory produced mixture but the plant produced HMA performed slightly worse than the laboratory mixture.

2.8.3 Foam Process

The foamed warm mix asphalt process uses water to produce asphalt foam at the plant; it requires installation of a water injection system to the asphalt input of an asphalt plant. Numerous manufacturers produce suitable water injection systems. A version of the foam warm mix process can be produced with the use of high water content additives such as zeolites. Tao and Mallick (2009) experimented with 100% RAP and zeolite additives in the laboratory and observed that the stiff RAP asphalt appeared to hinder the foaming process somewhat compared to what was observed for virgin binders.

A demonstration project conducted in South Carolina using the *Double Barrel Green System* used 50% RAP (Boggs 2008). The RAP was fractionated into three sizes prior to production. A total of 15,000 tons of warm mixed asphalt containing RAP was placed, approximately half as surface course. Measured field densities were nearly identical between the WMA and HMA control section and were reached at temperatures as low as 88 C (190 F). Rutting tests conducted in the APA on plant produced mix had lower measured rut depths for WMA than the HMA control (2.9 mm for WMA and 4.4 mm for HMA).

A foamed warm mix demonstration project was constructed in Memphis, TN (Nelson 2008). One of the mixes tested was a Mississippi gravel surface specification mix. No difficulties were encountered reaching density at the reduced production and lay down temperatures. Target compaction temperature was 127 C however one truckload of mix was also successfully compacted at 110 C. Another foamed warm mix demonstration project was constructed in Florida (Bistor 2008). The mixture contained

30% RAP and was successfully produced at 124 C. The compacted warm mix had 2% lower air voids than the same mixture produced as conventional hot mix.

Two foamed asphalt mixtures were placed on the surface of a city street in Chattanooga, TN in 2007; one of the mixtures contained 50% RAP and the control mix had no RAP (Hodo et al. 2009). The RAP was fractionated into coarse and fine stockpiles before production. Both mixes were successfully placed; the 50% RAP mixture was compacted at approximately 132 C and the control mixtures was successfully compacted at temperatures as low as 110 C. In place density of the mix was somewhat higher than desired (average 9% air voids for four cores). No distresses were apparent in either mix after one year of traffic. Recovered binder from the 50% RAP mix was a PG 82-16 (continuous grade of 84.3-18.0); virgin binder in the mix was PG 64-22. The RAP had a stiffening effect on binder properties but low temperature properties were felt to still be reasonable. The high temperature PG was increased three grades by the addition of 50% RAP while the low temperature PG was only changed by one grade.

Field mix was sampled and brought back to the laboratory for evaluation of moisture damage with *TSR* and with Hamburg testing as well as rutting evaluation in the *APA*. Results indicated that *TSR* values were marginal (78 for 0% RAP mix and 82 for 50% RAP mix); interestingly the 0% RAP mix had higher dry and wet tensile strengths than the 50% RAP mix. Hamburg rut depths were acceptable but average SIPs for both mixes were less than the desired 10,000 passes (8,900 for 0% RAP and 8,500 for 50% RAP) indicating a potential for moisture damage (Hodo et al. 2009). *APA* rutting results were good with all mixtures rutting less than 4 mm; interestingly the 50% RAP mix rutted slightly more than the 0% RAP (2.4 and 3.9 mm for 0 and 50% RAP respectively).

Middleton and Forfylow (2009) evaluated four plant produced foam mixtures placed in Canada. The four 75 blow Marshall designed mixtures produced were: 1) 0% RAP virgin mixture; 2) 15% RAP mixture; 3) 15% RAP with 5% recycled shingles mixtures; and 4) 50% RAP mixture. No major differences in any of the mixtures were observed in *APA* rut testing. *TSR* testing indicated slightly below passing values for the virgin mix but adequate values for all recycled mixtures; recycled mixtures had higher indirect tensile strengths and *TSR* values than the virgin mix. Resilient modulus testing indicated that the recycled mixtures were somewhat stiffer than the virgin mix as would be expected but did not indicate any problems with the foaming process. Production temperature for all mixes was about 130 to 135 C; this represented an approximate reduction of 24 C from HMA temperatures. A reduction in energy required for plant operation of approximately 24% was observed with the foam warm mix compared to conventional hot mix in the same plant.

Copeland et al. (2010) reported on a field project with 45% fractionated RAP 12.5 mm NMAS mixture in Florida. Both HMA and foamed WMA versions of mixture were produced. A soft recycling agent was used as the virgin binder; it was graded PG 52-28. Binder testing was performed in conjunction with mixture dynamic modulus and flow number testing. Results indicated that the WMA mixture likely achieved only a partial level of blending between RAP asphalt and virgin binder but that the HMA mixture had relatively complete level of blending. The recovered binder properties indicated that the WMA mixture did not experience as much aging during plant production as the HMA.

Abbas and Ali (2011) investigated foamed WMA without RAP in the laboratory. A natural gravel and a crushed limestone were evaluated with two binder grades (neat PG

64-22 and polymer-modified PG 70-22). Moisture susceptibility was evaluated with *TSR* and rutting resistance with the *APA*. A slight increase in moisture susceptibility and an increase in rutting (especially with the neat binder) was observed.

2.9 Summary of Literature Review Findings

2.9.1 Durability

Durability (especially raveling and weathering) is an important aspect of asphalt mixture performance that has not been widely addressed, especially for dense graded mixtures. The increased stiffness associated with high RAP mixtures is potential cause for concern in regards to durability; however a few sources reported adequate durability performance from high RAP mixtures in practice (Su et al. 2009, West et al 2009, West et al. 2011). The Cantabro test has been used by several researchers to assess durability of OGFC and PFC mixtures and is used by some agencies as mix design tool. It has potential to provide a relative assessment of durability for high RAP mixtures.

2.9.2 Non-Load Associated (Thermal) Cracking

Thermal cracking is a distress mode of asphalt pavements that is primarily due to environmental factors; it is most severe in cold climates. High RAP mixtures are thought to be more susceptible to this distress due to increased mixture stiffness at low temperatures. A higher incidence of cracking has been observed for high RAP mixtures by some researchers (Shoenberger and Demoss 2005, West et al. 2011). The *BBR* mixture test is promising for evaluation of high RAP mixture stiffness without need for extraction. Thermal cracking analysis can likely be performed with *BBR* data using the analysis methods of *AASHTO R 49* and Shenoy (2002).

2.9.3 Permanent Deformation (Rutting)

Reduced binder aging associated with lower mixing temperatures of WMA has the potential to result in rutting problems in service soon after construction. Some evidence has been reported in literature for increased rutting with WMA (Hurley and Prowell 2006, Abbas and Ali 2011); however other researchers have reported adequate rutting performance for WMA (Hurley and Prowell 2005b, Prowell et al. 2007). Rutting does not appear to be major issue with high RAP mixtures (Shoenberger and Demoss 2005, Hossain et al. 1993, Potter and Mercer 1997, Aguiar-Moya et al. 2011). Limited information is available concerning use of high RAP in conjunction with WMA but RAP use RAP may offset rutting potential (Mallick et al. 2008, Boggs 2008, Hodo et al. 2009).

2.9.4 Moisture Damage

Damage to asphalt pavements due to moisture is a major cause of pavement distress. The distress is commonly manifested as removal of asphalt binder from the aggregate particles (stripping) and physical disintegration of the asphalt mixture. Numerous researchers have reported that inclusion of RAP in mixtures can improve resistance to moisture damage (Amirkanian and Williams 1993, Castro-Fernandez 1996, Al-Qadi et al. 2009, Maupin et al. 2009). Only one source was found that reported reduced performance from RAP (Chen et al. 2007); however the researcher extracted the RAP aggregate before testing and thus breaking the strong bond between RAP aggregate and asphalt. Numerous researchers have reported potentially increased susceptibility to moisture damage by WMA (Hurley and Prowll 2005b, Hurley and Prowell 2006, Prowell et al. 2007, Diefenderfer et al. 2007, Hurley and Prowell 2008, Kvasnak et al. 2009). A few researchers have investigated high RAP in conjunction with WMA; some have found that that RAP generally compensates for the decrease in performance due to WMA (Austerman et al. 2009, Middleton and Forfylow 2009, Mogawer et al. 2011a); others have found marginal overall performance of high RAP-WMA (Hodo et al. 2009).

Mogawer et al. (2011b) evaluated the effects of varying laboratory short term conditioning times and temperatures on moisture susceptibility of WMA. Four WMA technologies were investigated including water based foaming additive (Advera), two wax based additives (one of them Sasobit®) and Evotherm™. One 9.5 mm NMAS gradation and one base binder (PG 64-22) were used. Nine experimental factor-level combinations were evaluated for each WMA technology, consisting of three conditioning temperatures (standard 146 C, 129 C and 113 C) and three conditioning times (standard 2 hr, 4 hr and 8 hr). The mixing temperature was not specified. Performance was evaluated with HLWT, E^* ratio for all mixtures; an adhesive energy test and a repeated load fracture test were performed with some of the WMA technologies. Results indicated that longer conditioning times had improved performance and that lower conditioning temperatures had decreased performance. None of the WMA mixtures at the lowest conditioning temperature (113 C) passed the HLWT test without anti-strip additives. The results demonstrated that more binder aging results in a better bond with aggregate and that insufficient binder aging could result in moisture susceptibility.

It has been hypothesized that the potential for moisture damage is at a maximum in pavements with about 7 to 11% air voids due to high permeability (Terrel and Al-Swailmi 1993); high permeability has been measured in 9.5 mm and 12.5 mm NMAS mixtures with greater than 7.7% air voids (Cooley et al. 2001). DOTs in the Southeastern U.S. target 6 to 8% air voids during construction but will allow pavements to remain in place with over 9% air voids and as high as 11% in some cases (Table 2.3). High air void levels of 11% or more have been measured in actual pavements by many researchers (Badaruddin and White 1994, Lu 2005, Seo et al. 2007, Prowell and Brown 2007). Conditions ripe for moisture damage are unfortunately common in actual practice. Moisture damage and is especially likely at high temperatures and slow rates of loading (Williams and Breakah 2010). Test parameters for a worst case scenario moisture damage test should attempt to replicate the high air void levels, high temperatures and slow loading rates that are most likely to induce moisture damage.

The *TSR* test is commonly used by DOTs to evaluate the potential for moisture damage of asphalt mixtures (Mogawer et al. 2011b). However, *TSR* results do not always correspond to observed field performance (Zaniewski and Visawanathan 2006); Hamburg loaded wheel moisture damage testing has been stated to better correspond to field performance than *TSR* testing (Azari 2010). PURWheel loaded wheel testing has been stated to give a better indication of moisture damage potential than *TSR* testing (Pan and White 1999). On the other hand, moisture damage testing with the *APA* has not been as successful (Cross et al. 2000, Hunter and Ksaibati 2002, West et al. 2004); the Hamburg test has been shown to provide better results than the *APA* (Shiwakoti 2007).

2.9.5 Load Associated (Fatigue) Cracking

Bottom up fatigue cracking of high RAP (stiff) mixes has enough potential to be problematic in base or binder pavement layers to be given some consideration. Generally speaking, bottom up fatigue cracking is not a big problem for surface mixtures in a properly designed flexible pavement structure. As discussed in the following paragraphs, several researchers have discussed or provided evidence that fatigue cracking in high RAP mixtures may not be as big an issue as first thought, even in base or binder layers.

Huang et al. (2005) summarized two studies where HMA fatigue resistance was improved by including up to 30% RAP. Huang et al. (2005) performed a limited amount of finite-element modeling and based on the results, hypothesized that retention of a stiff layer of RAP bitumen coating RAP aggregate beneath an outer coating of virgin binder due to partial blending would actually reduce stress concentrations and possible improve a mixture's fatigue performance. Reasoning was that the retained high stiffness (for asphalt) RAP bitumen layer at the aggregate surface acted as a buffer between the extremely stiff aggregate (relative to asphalt) particles and soft virgin binder film coating.

Santos et al. (2010) reported better fatigue performance in the laboratory (using a Portuguese beam fatigue test method) for mixtures containing 20, 30 and 40% RAP compared to a virgin HMA control; results were consistent for both laboratory mixed and plant mixed material. Shu et al. (2008) investigated fatigue performance in the laboratory for plant produced mixtures containing 0, 10, 20 and 30% RAP; testing consisted of Superpave IDT and flexural beam fatigue (600 microstrain and 10 Hz). IDT test results and several data analysis methods indicated that fatigue life of the mixtures may be reduced by inclusion of RAP. Beam fatigue results and plateau value of dissipated energy data analysis method indicated that RAP mixtures might have lower fatigue life in some but not all instances. Beam fatigue results and failure defined as 50% reduction initial stiffness data analysis method indicated that 30% RAP mixture might have longer fatigue life than the other mixtures (120,000 cycles to failure compare to 80,000 cycles to failure for other mixtures).

Tabaković et al. (2010) investigated performance of mixtures containing 0, 10, 20 and 30% RAP; testing consisted of indirect tensile fatigue (British test method) and a circular wheel tracking device. The circular wheel tracker allows for determination of crack propagation, permanent deformation, peak strains developed at bottom of the large slab specimen (30 by 30 by 5 cm); test parameters were 20 C, 695 kPa contact pressure and 3 km/hr speed. Optimum design asphalt content of the 20 mm mixture (gradation was typical of those used in Ireland) was determined according to Marshall procedures but the mixtures tested varied slightly from designed optimum asphalt contents to have a set amount of virgin asphalt added to each mixture (0% RAP mix was -0.2% of optimum, 10% RAP was +0.3% of optimum, 20% RAP was +0.3% of optimum, 30% RAP was +05% of optimum).

Indirect tensile results of Tabaković et al. (2010) indicated that 30% RAP performed significantly better than all other mixtures with respect to fatigue. Circular wheel tracker results were as follows. Mixtures with RAP had generally shorter cracks than virgin mixture. Mixtures with 20 and 30% RAP had the least amount of cracking, 0% RAP had the most and 10% RAP fell in between. Mixtures with RAP had better rutting performance than virgin mixture. The 30% RAP mixture had best fatigue performance of all the mixture with respect to measured strains.

Cascione et al. (2011) investigated laboratory properties of plant mixed high RAP mixtures; the evaluation included beam fatigue testing. Four base mixtures (19.0 mm NMAS) were produced that contained 5% post consumer recycled asphalt shingles and 25, 35, or 45% RAP; a 50% RAP mixtures with no shingles was also produced. Two binder mixtures (19.0 mm NMAS) were produced, 5% shingles with 35% RAP and 40% RAP with no shingles. Two surface mixtures (9.5 mm NMAS) were produced, 5% shingles with 20% RAP and 25% RAP with no shingles. The base and binder mixtures were designed to be binder rich and fatigue resistant (2% design V_a for base mixtures and 3% design V_a for binder mixtures). All mixtures were HMA with PG 58-22 virgin binder. Fatigue test results indicated that all mixtures would likely have adequate fatigue performance except the 5% shingles 45% RAP base mixture.

Timm et al. (2011) reported on fatigue performance of foamed WMA and conventional HMA both containing fractionated 50% RAP placed on the NCAT test track. The three virgin control mixtures were HMA, foamed WMA and Evotherm™ WMA. Each test section consisted of 76 mm thick base course (19.0 NMAS), 64 mm thick binder course (19.0 NMAS) and 38 mm thick surface course (9.5 mm NMAS). Strain gages installed at the bottom of the asphalt layer were monitored during application of approximately four million ESALs of traffic. The measured strains were observed to be strongly temperature dependent with higher strains measured at higher temperatures. Measured data were corrected for construction differences in layer thickness and shifted to one of three reference pavement temperatures.

At the 10 C reference temperature none of the mixtures were statistically different. At the 20 C test temperature the foamed 50% RAP WMA had statistically lower strains than the other mixtures. At the 43 C reference temperature the control mixture had the highest strains, the virgin WMA mixtures were statistically the same and lower than the control, and the 50% RAP mixtures (HMA and WMA) had strains statistically lower than all other mixtures. Laboratory beam fatigue testing was also performed with plant produced mixture at 800 and 400 microstrain with 20 C test temperature (note that the laboratory strain levels were higher than the measured field strains at 20 C). Laboratory test results were extrapolated with a fatigue transfer function; results indicated that the high RAP-WMA mixture may have the best performance; however no cracking had yet been observed in the field for any mixture.

Aravind and Das (2007) observed that recycled mixtures had better fatigue performance than virgin mixture at low strain rates but that recycled mixtures had considerably worse fatigue performance than virgin mixture at high strain rates. This information is promising in light of the findings of Timm et al. (2011) that high RAP-WMA mixtures may have lower tensile strains in a given pavement structure than conventional mixture. Much of the beam fatigue testing that is performed is done at high strain rates (unrepresentative of actual pavement strains) due to speed of testing. This potentially may result in poor fatigue performance results for high RAP mixtures but more testing is needed at strain rates representative of actual field conditions.

CHAPTER 3

MATERIALS AND MIXTURES

3.1 Overview of Materials and Mixtures

This chapter provides properties of all materials tested as part of this study. The terminology provided in Section 3.2 is utilized throughout the document. Specimen preparation methods are described in Section 3.3. Properties of all mixtures used in this study are provided in Section 3.4.

3.2 Terminology

Aggregate sources are identified with a single letter followed by a dash and a number; the letter designates what type of aggregate and the number indicates the specific aggregate of that type (e.g. *G-1* refers to gravel aggregate source one). RAP sources are identified with a unique designation beginning with the letter *R* to represent RAP source (e.g. *R-1* refers to RAP source one). For cases where other materials were used, generic terminology has been incorporated (e.g. gravel would refer to a source other than the gravel specifically referred to as *G-1*).

All named aggregate and RAP sources utilized in this study are as follows:

- *G-1* Crushed gravel aggregate source 1
- *G-2* Crushed gravel aggregate source 2
- *G-3* Crushed gravel aggregate source 3

- *G-4* Crushed gravel aggregate source 4
- *L-1* Limestone aggregate source 1
- *L-2* Limestone aggregate source 2
- *L-3* Limestone aggregate source 3
- *L-4* Limestone aggregate source 4
- *S-1* Coarse sand aggregate source 1
- *S-2* Coarse sand aggregate source 2
- *HL-1* Hydrated lime source 1
- *R-1* RAP source 1
- *R-2* RAP source 2
- *R-3* RAP source 3
- *R-4* RAP source 4
- *R-5* RAP source 5

To identify mixtures used in this experimental program, an identification system was set up according to the general format given in Eq. 3.1. The individual components of the identification system are described as follows.

1-2/3-4 (Eq 3.1)

1: The first position in the mixture identification code designates the NMAS of the aggregate gradation. Possible values for this label are:

9.5: 9.5 mm NMAS gradation

12.5: 12.5 mm NMAS gradation

19.0: 19.0 mm NMAS gradation

2: This portion of the label indicates the percentage of RAP aggregate in the mixture as a percentage of the total aggregate. Possible values for this label are:

- 0: 0% RAP
- 15: 15% RAP
- 25: 25% RAP
- 50: 50% RAP
- 75: 75% RAP
- 100: 100% RAP

3: This portion of the label indicates the mixture type. Possible values for this label are:

- AM: Airfield Mixture
- CM: Control Highway Mixture
- RM: Recycled Mixture

4: This portion of the label is a numeric code that indicates the specific mixture of that type. If the number is followed by a lower-case letter, the letter indicates slight changes to the same aggregate blend and mixture composition; the slight changes may include: asphalt binder grade, mixing method (e.g. plant or laboratory mixed), or design traffic level.

For example, *12.5-0/CM-1* refers to a 12.5 mm NMAS gradation with 0% RAP that is the first control mixture. Likewise, *9.5-25/RM-1* refers to a 9.5 mm NMAS gradation with 25% RAP that is the first recycled mixture.

3.3 Materials Tested

3.3.1 RAP

Five RAP sources were utilized in this experimental program. RAP sources *R-1*, *R-2*, and *R-3* were selected to represent the range of possible RAP sources available in Mississippi. RAP source descriptions are provided in the following paragraphs.

R-1 represents a high traffic mix (85 design gyrations); the material was milled from the surface of a 22.5 km stretch of Interstate 55 near Grenada, MS. The material was acquired from a producer's stockpile in fall 2007. The material was originally placed in 1992. In general the material was from a 12.5 mm binder course developed with Marshall Mix Design. Within the section milled, both polymer modified and non polymer modified binders were used, along with varying amounts of sand.

R-2 was selected to represent an intermediate traffic mix commonly used on lower volume roads and state highways. The low volume design would currently be categorized as a medium traffic mix (65 design gyrations). The material was milled from State Highway 25 in Monroe County, the project was 12 km, and the maximum milling depth was 50 mm. The material was obtained from a producer stockpile in fall 2007.

R-3 is representative of a typical Mississippi RAP stockpile where a variety of materials are present. In this particular stockpile nearly all of the material was acquired from MDOT highways. The material was obtained from the stockpile in fall 2007.

R-4 and *R-5* were used for verification testing of the approach developed in Chapter 5 to estimate RAP absorbed asphalt content. *R-4* was obtained from surface milling of U.S. Highway 49 in Madison County in the summer of 2010. *R-5* was obtained from surface milling of U.S. Highway 61 in Warren County in summer 2010.

Asphalt contents for samples of the *R-1*, *R-2* and *R-3* RAP sources were determined at the MSU laboratory according to *ASTM D 2172 Method A* using trichloroethylene as the extraction solvent. Washed gradations (*AASHTO T 30*) were performed on the extracted aggregates from the MSU samples. Samples of the three RAP sources were also sent to the Mississippi department of transportation (MDOT) central materials laboratory to check asphalt contents and aggregate gradations. The ignition procedure (*AASHTO T 308 Method A*) was used to determine asphalt content at MDOT. Washed gradations were performed on solvent extracted aggregate at the MDOT laboratory.

RAP properties are given in Table 3.1 and extracted RAP aggregate gradations are shown in Figure 3.1. Note the high value of sand ratio (*SR*) for *R-3*, this likely indicates a large percentage of natural sand is present; the MDOT specification for sand ratio is 60 or less (MDOT 2006). The washed gradation on extracted aggregate for *R-1* performed at MSU compared favorably to the results obtained by MDOT. However a laboratory error invalidated the MSU gradation results for RAP sources *R-2* and *R-3*; the test was not rerun and the MDOT test results were used instead. For the *R-1* RAP source, the washed gradation on extracted aggregate and the combined aggregate properties are average values of test data from MSU and MDOT. For *R-2* and *R-3* RAP sources, the aggregate gradations and combined properties are the MDOT obtained values only. Asphalt contents determined by MSU and those determined by MDOT were within multi-laboratory precision ranges for all three RAP sources; values reported in Table 3.1

are averages of MSU and MDOT test results. Asphalt content of *R-4* and *R-5* was only determined at MDOT; gradations were not determined.

An attempt was made to extract the effective asphalt from the RAP and leave the majority of the absorbed asphalt for PG grading because it was expected that absorbed asphalt within the RAP aggregate would not contribute to blended properties of the 25 and 50% RAP mixtures. Three washes of trichloroethylene solvent were used with a 45 minute soak period for each wash. Less than all of the RAP surface asphalt was extracted using the three wash procedure. The outer portion of the binder is expected to have aged more than the absorbed asphalt so it is expected that the grade of the recovered asphalt would have been less if all of the asphalt had been extracted. For *R-1*, roughly 3.6% asphalt was extracted from the RAP using the three wash procedure. The total asphalt content was 5.5% so approximately 1.9% asphalt remained with the RAP. Note that low temperature performance grade is a positive value for RAP sources *R-1* and *R-3* indicating very brittle asphalt.

Crushed gravel and crushed limestone are the two primary coarse aggregate types used in Mississippi. To determine their proportions in RAP, a coarse aggregate sorting procedure was developed to estimate the amount of limestone. The procedure consists of visually inspecting and categorizing extracted RAP aggregate. Extracted aggregate washed with water is separated into two fractions: 1) retained on the 4.75 mm sieve; and 2) passing the 4.75 mm sieve and retained on the 2.36 mm sieve. Based on visual inspection the aggregate was categorized as limestone or gravel (Figure 3.2).

Table 3.1 Properties of RAP Materials Tested After Asphalt Extraction

RAP ID		*R-1*[a]	*R-2*	*R-3*	*R-4*	*R-5*
Percent Passing	**25.0 mm**	100	100	100	---	---
	19.0 mm	100	100	100	---	---
	12.5 mm	96.5	99.8	92.2	---	---
	9.5 mm	90.0	98.2	82.1	---	---
	4.75 mm	60.1	73.1	55.5	---	---
	2.36 mm	41.9	52.8	43.7	---	---
	1.18 mm	34.1	40.3	38.5	---	---
	0.60 mm	29.2	33.4	33.2	---	---
	0.30 mm	19.5	22.9	20.6	---	---
	0.15 mm	11.8	13.0	11.4	---	---
	0.075 mm	8.4	9.3	7.3	---	---
G_{sb}		2.483	2.526	2.504	---	---
G_{sa}		2.600	2.597	2.577	---	---
Abs **(%)**		1.8	1.1	1.1	---	---
$LST_{+4.75}$ **(%)**		8.1	28.2	24.6	---	---
$LST_{+2.36}$ **(%)**		8.6	32.9	24.2	---	---
P_{AC} **(%)**[b]		5.5	5.6	5.0	5.6	5.7
Viscosity (Pa•s)[c]		52.9	9.1	26.5	---	---
Continuous PG		117.8+1.71	105.8-3.47	112.6+4.36	---	---

a) Aggregate properties for *R-1* are average values of all valid test results obtained.
b) Avg. value obtained from MSU and MDOT central laboratory. *R-4* and *R-5* are MDOT results only.
c) Tested at MSU according to *AASHTO T 316*, test temperature was 135 C.

For the aggregate retained on the 4.75 mm sieve the percentage by mass of limestone aggregate on a basis of total aggregate retained on the 4.75 mm sieve was determined; the variable $LST_{+4.75}$ is used to denote this value in Table 3.1. For all aggregate retained on the 2.36 mm sieve (including the portion retained on 4.75 mm sieve) the percentage by mass of limestone aggregate on a basis of total aggregate retained on the 2.36 mm sieve was determined; the variable $LST_{+2.36}$ is used to denote this value in Table 3.1.

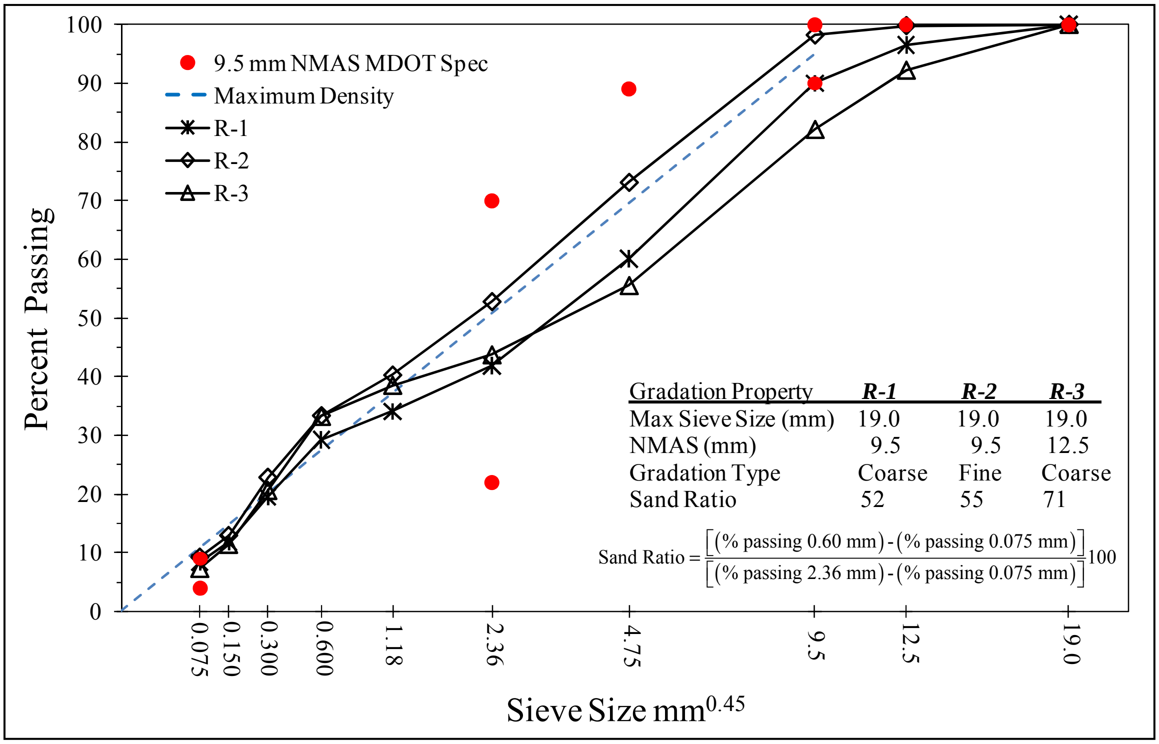
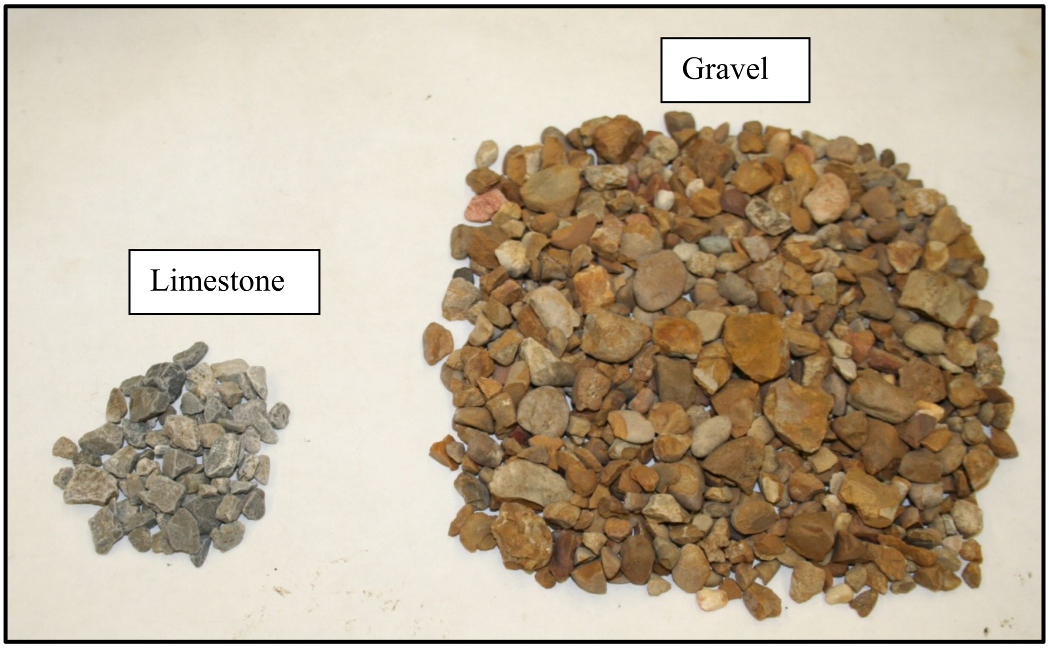
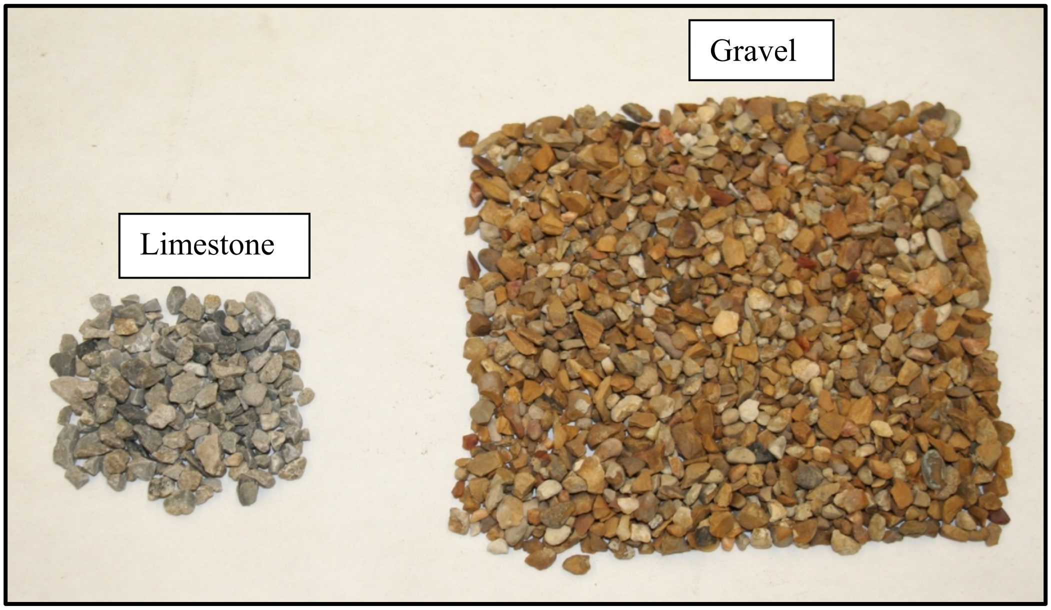


Figure 3.1 9.5 mm NMAS 0.45 Power Gradation Plot of RAP Extracted Aggregate

For example, 1025 g of *R-1* extracted coarse aggregate was retained on the 2.36 mm sieve; of that total, 663g was retained on the 4.75 mm sieve and 362g passed the 4.75 mm sieve and was retained on the 2.36 mm sieve. The aggregate sorting procedure determined that of the material retained on the 4.75 mm sieve, 53.5g was limestone and the remainder was gravel (663g - 53.5g = 609.5g). This results in a $LST_{+4.75}$ value of 8.1% (53.5g / 663g = 8.1%). The aggregate sorting procedure determined that of the 362g of aggregate passing the 4.75 mm sieve and retained on the 2.36 mm sieve, 35g was limestone and the remainder (362g - 35g = 327g) was gravel. This results in a $LST_{+2.36}$ value of 8.6% [(53.5g+35g) / 1025g = 8.6%].



a) Aggregate Retained on 4.75 mm Sieve



b) Aggregate Passing 4.75 mm and Retained on 2.36 mm Sieve

Figure 3.2 RAP Aggregate Sorting Procedure (*R-1* shown)

3.3.2 Virgin Aggregates

Ten virgin aggregates were used for the majority of testing, which were obtained from local producers. Properties of crushed gravel are given in Table 3.2, properties of limestone are given in Table 3.3 and properties of sand and hydrated lime are given in Table 3.4. Other aggregates were tested as part of this experimental program in lesser quantities (e.g. as part of plant produced mixes). Those aggregates were given generic designations. Specific details of those aggregates are not provided; only composite aggregate blend properties are provided.

Table 3.2 Properties of Virgin Crushed Gravel Aggregates Tested

	Aggregate ID	***G-1***	***G-2***	***G-2b***	***G-3***	***G-4***
	Size	< 12.5 mm	< 12.5 mm	< 12.5 mm	< 19.0 mm	< 19.0 mm
Percent Passing	**25.0 mm**	100.0	100.0	100.0	100.0	100.0
	19.0 mm	100.0	100.0	100.0	100.0	100.0
	12.5 mm	100.0	100.0	100.0	77.0	86.2
	9.5 mm	92.0	93.9	93.9	58.0	67.2
	4.75 mm	47.0	50.3	50.3	29.0	34.9
	2.36 mm	26.0	28.0	28.0	16.0	20.3
	1.18 mm	16.0	16.6	16.6	11.0	13.0
	0.60 mm	11.0	10.9	10.9	9.0	8.9
	0.30 mm	8.0	7.8	7.8	7.0	6.4
	0.15 mm	7.0	6.0	6.0	5.0	4.9
	0.075 mm	5.2	4.9	0.2	4.0	4.0
	G_{sb}	2.395	2.380	2.380	2.391	2.397
	G_{sa}	2.625	2.595	2.595	2.611	2.612
	***Abs* (%)**	3.66	3.48	3.48	3.52	3.43

Notes: All crushed gravel aggregates obtained from Scribner Pit in Hamilton, MS. Gravel aggregate *G-2b* was *G-2* aggregate washed over a 0.075 mm sieve to remove dust.

Table 3.3 Properties of Virgin Limestone Aggregates Tested

	Aggregate ID	***L-1***	***L-2***	***L-3***	***L-3b***	***L-4***
	Size	#810	#810	Screenings	Screenings	#7
Percent Passing	**25.0 mm**	100.0	100.0	100.0	100.0	100.0
	19.0 mm	100.0	100.0	100.0	100.0	100.0
	12.5 mm	100.0	100.0	100.0	100.0	96.8
	9.5 mm	100.0	100.0	100.0	100.0	65.9
	4.75 mm	92.0	94.0	97.3	98.2	8.6
	2.36 mm	68.0	74.9	62.8	68.9	1.7
	1.18 mm	53.0	61.2	38.3	42.6	1.1
	0.60 mm	41.0	52.0	25.5	25.3	0.9
	0.30 mm	27.0	44.2	17.6	13.8	0.9
	0.15 mm	19.0	39.1	12.7	6.5	0.8
	0.075 mm	14.8	10.9	10.0	2.1	0.8
	G_{sb}	2.625	2.625	2.666	2.779	2.754
	G_{sa}	2.711	2.711	2.768	2.782	2.789
	***Abs* (%)**	1.21	1.21	1.40	0.03	0.46

Notes: Limestone aggregates *L-1* and *L-2* were obtained from Russellville, AL. Limestone aggregates *L-3* and *L-4* were obtained from a quarry in Calera, AL. Limestone aggregate *L-3b* was *L-3* aggregate washed over a 0.075 mm sieve to remove dust.

Table 3.4 Properties of Virgin Sand Aggregates and Hydrated Lime Tested

	Aggregate ID	***S-1***	***S-2***	***HL-1***
	Size	< 9.5 mm	< 9.5 mm	---
Percent Passing	**25.0 mm**	100.0	100.0	100.0
	19.0 mm	100.0	100.0	100.0
	12.5 mm	100.0	100.0	100.0
	9.5 mm	100.0	100.0	100.0
	4.75 mm	95.0	97.4	100.0
	2.36 mm	82.0	84.6	100.0
	1.18 mm	72.0	74.7	100.0
	0.60 mm	55.0	61.5	100.0
	0.30 mm	21.0	18.3	100.0
	0.15 mm	2.0	2.1	100.0
	0.075 mm	0.5	1.3	100.0
	G_{sb}	2.572	2.538	2.300
	G_{sa}	2.644	2.640	2.300
	***Abs* (%)**	1.06	1.52	0.00

Note: All sand aggregates were obtained from Scribner Pit in Hamilton, MS.

3.3.3 Virgin Binders and Warm Mix Additives

Three virgin binders were used for laboratory prepared mixtures; two PG 67-22, and one PG 76-22. The PG 76-22 and the primary PG 67-22 virgin binder were supplied by Ergon Asphalt and Emulsions, Inc. from Vicksburg, MS. The typical high, intermediate, and low PG temperatures of the primary base binder were 68.7, 23.5, and -24.0 respectively. A secondary PG 67-22 virgin binder was sampled from the asphalt plant that produced one of the control mixtures; it was originally supplied by Hunt Refining Company from Tuscaloosa, AL. The primary PG 67-22 binder was used for production of all laboratory mixed asphalt with the following two exceptions. The secondary PG 67-22 binder source was used for the highway control mixture *9.5-15/CM-4b*. The PG 76-22 (modified with radial SBS polymer) binder was used for highway control mixture *9.5-15/CM-4c* and as a substitute binder for airfield mixtures *12.5-0/AM-1* and *12.5-0/AM-13*.

Virgin binder was heated to a mixing temperature of 154 C for PG 67-22 and not held at the mixing temperature for more than six hours; the number of re-heat cycles was minimized to ensure that properties of the binder were not adversely affected. The mixing temperature for PG 76-22 virgin binder was 188 C; once the mixing temperature was achieved, the binder was mixed for one hour with a high shear mixer before use. The polymer-modified virgin binder was not held at mixing temperature for more than six hours and any remaining binder at the end of the day was discarded.

Three warm mix additives were used in this experimental program: 1) Sasobit®; 2) Evotherm™ 3G; and 3) water for foamed asphalt. For airfield mixtures (Chapter 6) Sasobit® was used at a dosage rate of 1.5% based on total virgin binder weight. For performance tested highway mixtures (Chapters 7 and 8) Sasobit® was used at a dosage rate of 1.0% based on total binder weight (including asphalt contributed from RAP). Similarly, Evotherm™ 3G was used at a dosage rate of 0.5% based on total virgin binder weight for airfield mixtures and 0.5% based on total binder weight for highway mixtures. For highway mixtures, the Sasobit® or Evotherm™ 3G added to compensate for the RAP asphalt was added based on the total extracted asphalt cement content of the RAP. Water added during foaming was 2% of binder mass and was not considered part of the binder mass for calculation of asphalt content; the foaming process is discussed in Section 3.4.1.

Sasobit® was added to binder according to manufacturer recommendations. The binder was heated to 127 C and a paddle mixer was used to mix in the pellets that were slowly added into the binder. If all the pellets are added at once even dispersion might not have occurred. Once added and mixed, the Sasobit® will not settle in the binder. To compensate for the RAP binder in highway mixtures, additional Sasobit® pellets were added immediately before mixing as described in Section 3.4.2.

Evotherm™ 3G was premixed with virgin binder before use according to manufacturer recommendations. Binder was first heated to mixing temperature (154 C) before the liquid Evotherm™ 3G was added and then mixed with a high shear mixer until fully incorporated into the binder (approximately 10 minutes based on manufacturer's recommendations). To compensate for the RAP binder in highway mixtures, the virgin binder was overdosed with Evotherm™ 3G so that the final dosage rate once samples were mixed would be 0.5% by total asphalt cement weight.

3.4 Preparation of Test Specimens

For laboratory asphalt production, samples of aggregate were batched according to aggregate stockpile gradations. RAP was batched according to the aggregate stockpile gradations given in Howard et al. (2009); this resulted in the RAP extracted aggregate gradations given in Table 3.1. For all moisture damage testing (i.e. *TSR* and PURWheel), the virgin aggregate and hydrated lime were mixed with approximately 2% water to ensure adequate coating of the aggregate by the hydrated lime.

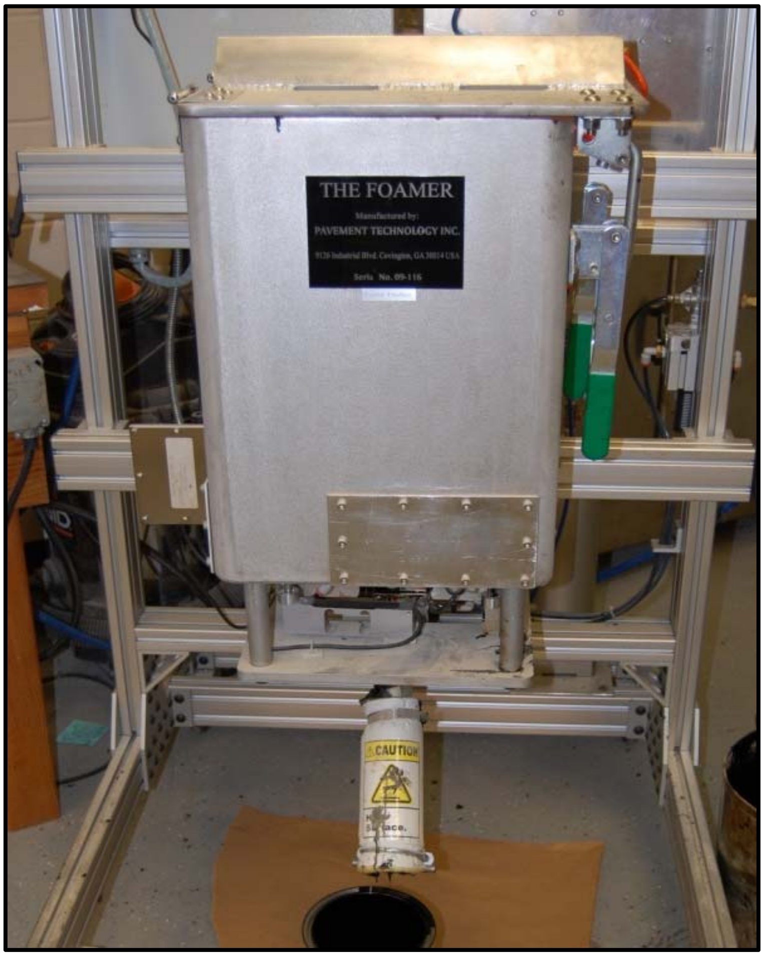
For the recycled mixtures in this experimental program, the percentage of RAP in the mixture was determined on the basis of percentage of extracted RAP aggregate contributed to the total aggregate in the mixture. This approach was simple to use for batching material in the laboratory during mix design and for practical purposes was the same value as percentage of the RAP in the final mixture (e.g. a 50% RAP mixture on an extracted aggregate to total aggregate basis might be 49.7% RAP on a RAP to total mixture basis).

The virgin aggregate and RAP were heated separately and then combined during mixing. Prior to mixing, virgin aggregate was heated for a minimum of 240 minutes in a forced draft oven; typical heating time was overnight. Prior to mixing, RAP was heated for 120 minutes in a forced draft oven at the mixing temperature. After heating, the materials were mixed as described in Section 3.4.2. Total heating time for the RAP was 210 minutes prior to compaction (i.e. 120 minutes heating before mixing plus 90 minutes of heating during short term conditioning of the mixture).

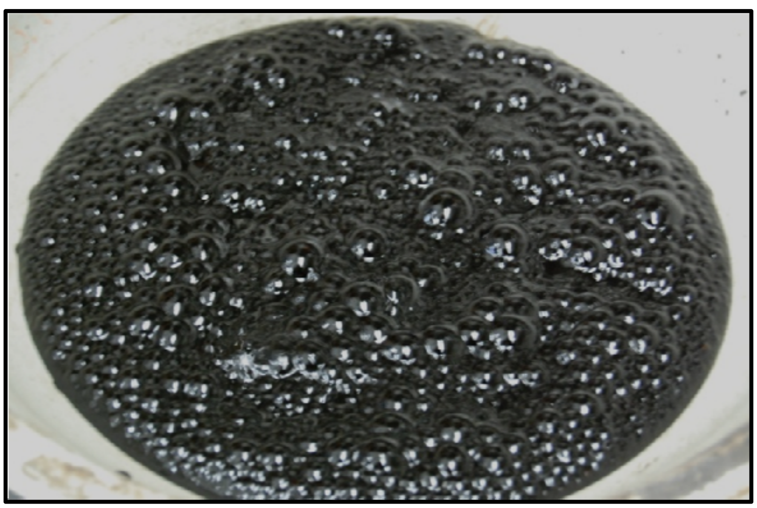
In addition to the laboratory preparation method, two preparation methods for plant produced asphalt mixture were utilized: 1) field sampled and prompt compaction prior to heat loss; and 2) plant sampled that was reheated prior to compaction. The first plant production method involved sampling of plant mixed material at the paving location, either from an asphalt paver or a material transfer vehicle (MTV). The samples were quickly brought to the laboratory in insulated containers and then compacted promptly at the field compaction temperature without any additional heating. For compaction of multiple specimens from one sampling trip, the mix was kept in an oven set to the measured field mix temperature for not more than one hour. The second plant production method consisted of sampling the plant mixed material at the asphalt plant and bringing it back to the laboratory; the mixture was allowed to cool. At a later time, the asphalt mixture was reheated to compaction temperature before compaction.

3.4.1 Laboratory Asphalt Foaming Process

Foamed asphalt was produced with an initial binder temperature of 163 C, which reduced to 120 C during foaming and was added to the heated aggregate at this temperature. The laboratory asphalt foaming device utilized for production of specimens with foamed asphalt is shown in Figure 3.3*a*. It features an automated control system that automatically proportions the water and asphalt binder at an operator-selected ratio. Figure 3.3*b* shows a sample of the foamed asphalt (2% water by binder mass).



a) Laboratory Asphalt Foaming Device



b) Sample of Foamed Asphalt

Figure 3.3 Laboratory Production of Foamed Asphalt

3.4.2 Laboratory Mixing

For airfield control mixtures, the target mixing and compaction temperatures were taken from temperature-viscosity charts for the asphalt binder and were 165 C and 146 C respectively. For airfield recycled mixtures, target mixing temperatures were 130 C; target short term conditioning and compaction temperatures were 116 C. The short term conditioning time for airfield mixtures was 120 min at the compaction temperature.

For highway control mixtures, the target mixing and compaction temperatures were either taken from the appropriate MDOT mix design or from temperature-viscosity charts for the asphalt binder. For highway recycled mixtures, target mixing, short term conditioning and compaction temperatures were the same. The purpose of using the same mixing and compaction temperatures for recycled highway mixtures was to attempt to isolate the contribution of RAP bitumen at a specific temperature to the overall mixture properties. For all laboratory produced highway mixtures, the standard MDOT short term conditioning time of 90 minutes was used at the compaction temperature.

All laboratory mixing of asphalt was performed with a bucket mixer; two capacities of bucket mixer were utilized depending on the size of mixture sample required: 1) conventional 19 L capacity; and 2) large 38 L capacity. The conventional 19 L capacity mixer was used for preparation of all asphalt mix for G_{mm} and *SGC* compacted specimens. The large 38 L capacity mixer was used for preparation of all asphalt mix for compaction in the *LAC*.

The mixing procedure was the same regardless of the mixer was used. A heated mixing bucket was placed on a scale and the pre-heated virgin aggregate and RAP were added to the bucket. A well was created in the center of the hot aggregate and the appropriate amount of virgin binder was weighed into the mixing bucket. When required, Sasobit® was added to compensate for RAP binder by heating it to just below its melting temperature, and placing it into the pool of liquid asphalt formed inside the mixing bucket (Figure 3.4). The bucket was placed in the mixer and the asphalt components were mixed continuously for 60 to 90 seconds. Care was taken to ensure the components were fully blended and the aggregate was coated.

The quantity of asphalt mixture needed for compaction of slab specimens in the *LAC* (≈ 30 kg) could not all be mixed at the same time in the 38 L mixer. The aggregate and RAP for slab specimens was batched in two equal parts (≈ 15 kg) and handled separately during heating and mixing. The first part was mixed according to the procedure described above then placed in a 19 L steel pail for short term conditioning. The mixing bucket was placed back into an oven for about 5 minutes to reheat and then the second batch was mixed using the same procedure. The second part was added to the same 19 L steel pail as the first part of the sample for short term conditioning. During the compaction process care was taken to prevent segregation of the mix by mixing the first and second parts of the asphalt mixture sample in the *LAC* compaction mold.

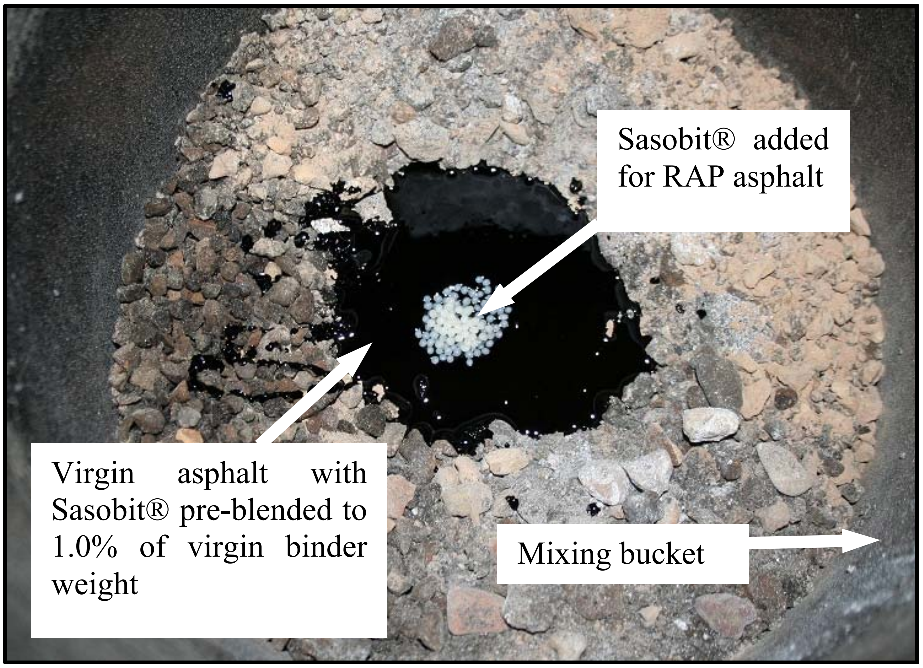


Figure 3.4 Addition of Sasobit® for RAP Mixture

3.4.3 Compaction of Test Specimens

Two types of asphalt compaction equipment were utilized: 1) Superpave Gyratory Compactor (*SGC*); and 2) Linear Asphalt Compactor (*LAC*). The *SGC* was used to compact standard 150 mm and 100 mm diameter cylinder specimens either with a specified compactive effort (i.e. number of gyrations) or to a target height and density. All *SGC* compaction was performed with a Pine Instrument brand compactor that was calibrated to 1.25 ± 0.02° by the external angle method.

Salient features and an overview of operation of the *LAC* in use at MSU is described briefly herein, further details can be found in Doyle and Howard (2011). Figure 3.5 shows the *LAC* and its major components. The *LAC* produces rectangular slabs of asphalt mixture that are 29.3 by 62.4 cm that can be any target thickness between 3.8 and 10.2 cm. For this study, two target slab thickness were utilized: 1) nominal 3.8

cm thickness for skid resistance test specimens; and 2) nominal 7.6 cm thickness for PURWheel test specimens.

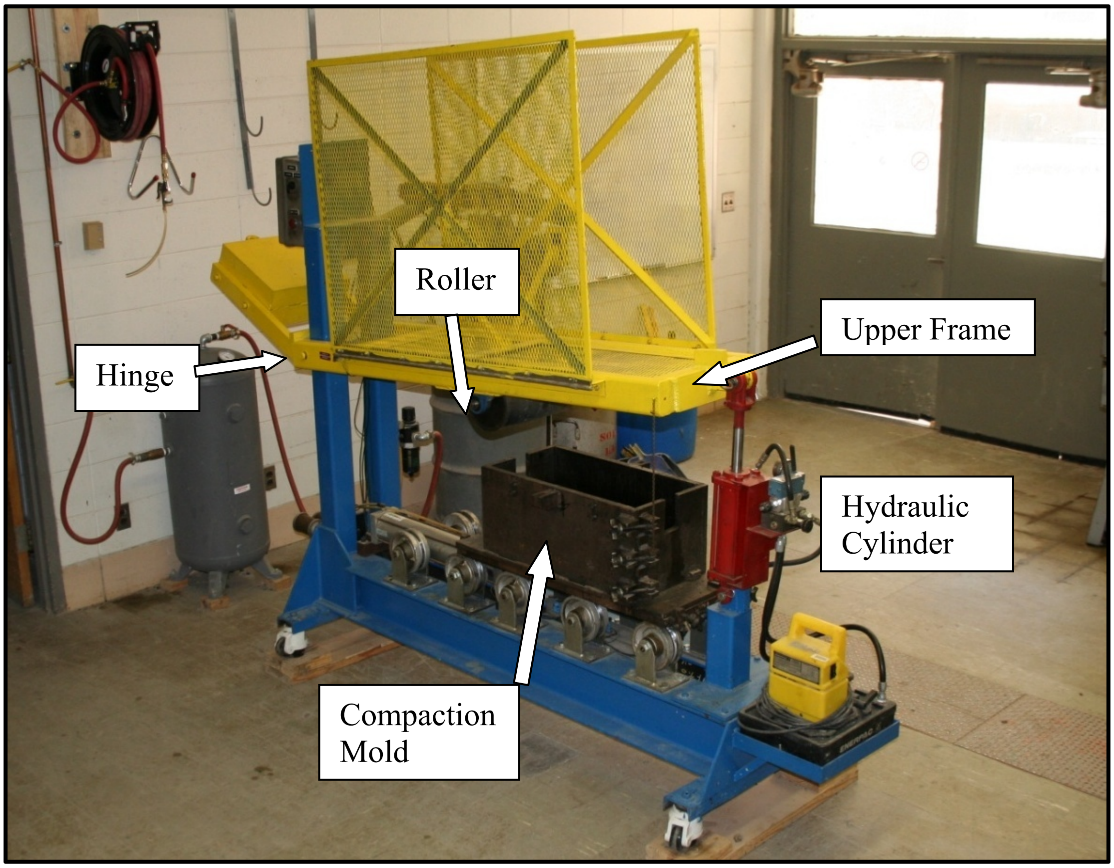


Figure 3.5 Linear Asphalt Compactor (*LAC*)

During the *LAC* compaction process, the compaction mold is moved backward and forward beneath a roller attached to the upper frame (Figure 3.5). Compactive effort is applied by a hydraulic cylinder attached to one end of the upper frame; the hydraulic pressure is regulated to provide a constant downward force on the upper frame of the compactor. The compactive force of the roller is transmitted to the asphalt mixture through a series of vertically aligned steel plates (not shown in Figure 3.5); this results in a kneading action during compaction. The level of compactive effort exerted by the *LAC* can be varied by adjusting the hydraulic system pressure used to operate the hydraulic ram and by varying the number of passes of the compaction mold beneath the roller. For all slabs of nominal 7.6 cm thickness the compactive effort parameters were 18 passes and 2413 kPa hydraulic system pressure.

The general compaction process for slabs produced in the *LAC* is shown in Figure 3.6. At the conclusion of the short term conditioning period, the mixture is loaded into the pre-heated compaction mold as shown in Figure 3.6*a*. The asphalt mixture is spread evenly in the mold while taking care to prevent segregation (Figure 3.6*b*) before a sheet of release paper is placed on top followed by a thin steel sheet; the purpose of the steel sheet is to distribute the weight of the compaction plates and ensure a smooth surface to the final compacted slab. Next, the vertically aligned compaction plates are lowered on top of the loose asphalt mixture. The upper frame of the *LAC* is brought down and pinned to the hydraulic cylinder (Figure 3.6*c*). After compaction is complete, the upper frame is unpinned, the vertical plates are removed, and the detachable portion of the compaction mold is removed to allow removal of the compacted slab. An example final compacted slab is seen in Figure 3.6*d*; the exposed corner of each slab is marked as a reference corner for to identify the slab's orientation during compaction.

a) Loading Mix into Compaction Mold
b) Leveling of Mix in LAC
c) LAC Ready for Compaction
d) Compacted Slab

Figure 3.6 *LAC* Slab Compaction Process

3.4.4 Sawing of Test Specimens

Specimens compacted to 150 mm diameter and nominal 115 mm height with the *SGC* were sawn to produce test specimens for *BBR* mixture testing. Two major steps were performed to produce test specimens: 1) sawing into rectangular blocks; and 2) sawing of rectangular blocks into final test specimens. Test specimens were produced from the interior of *SGC* compacted specimens as illustrated schematically in Figure 3.7.

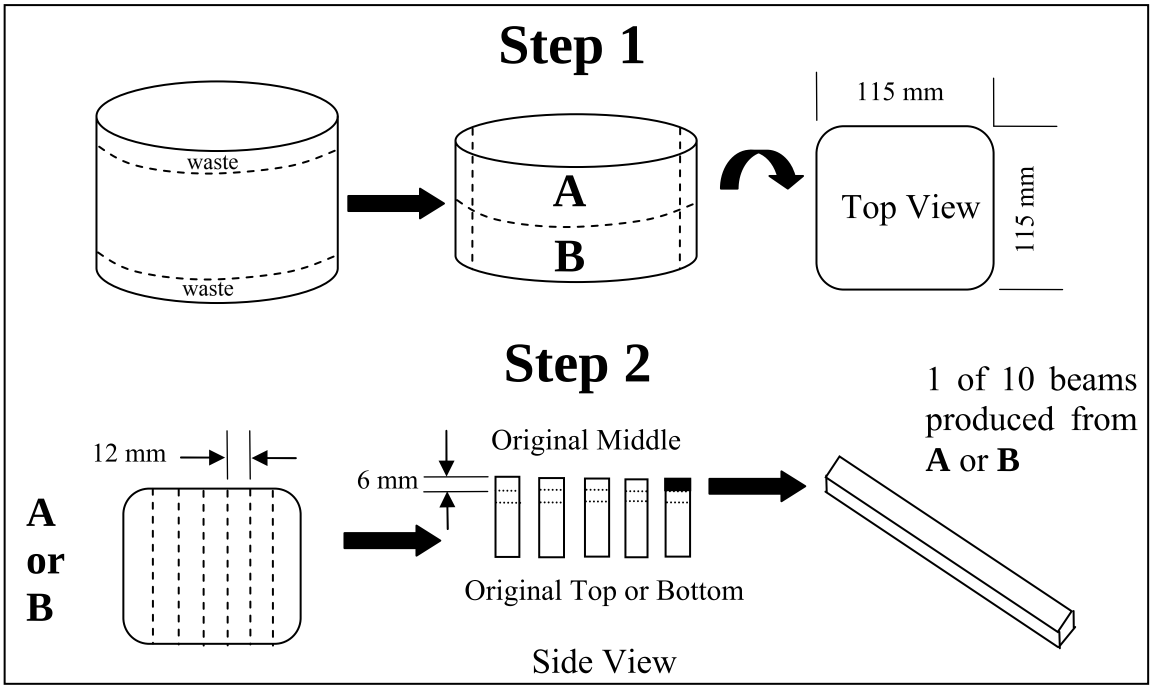
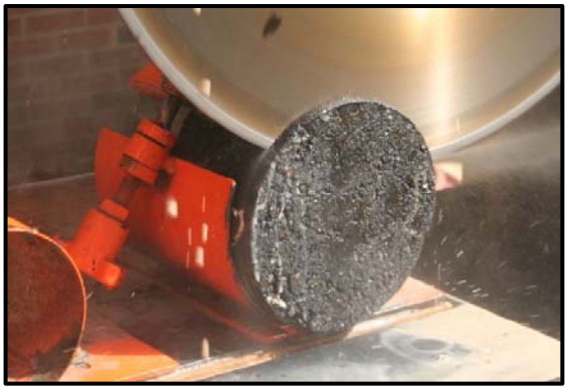


Figure 3.7 Schematic of *BBR* Specimen Preparation Method (Not to Scale)

During step 1, a masonry saw was used to remove horizontal slices 12.5 mm thick (top and bottom) as seen in Figure 3.8*a* and 3.8*b*; these slices were discarded. Four vertical cuts were then made to produce a rectangular block approximately 115 mm square (Figure 3.8*c* to Figure 3.8*e*). The resulting block was then sliced horizontally into two blocks approximately 50 mm thick; the final product is seen in Figure 3.8*f*. The final blocks were marked such that the face that was originally the interior of the compacted specimens was evident.

During step 2, *BBR* mixture beams 6 mm by 12 mm by 115 mm were produced as shown in Figure 3.9. Prepared rectangular blocks from the first sawing step were cut with a Buehler Delta AbrasiMet® precision abrasive saw utilizing a 25 cm diameter 2 mm thick diamond blade. Six vertical cuts were made in the block such as to produce five slices 12 mm wide (Figure 3.9*a* and Figure 3.9*b*).



a) Cutting of 12.5 mm thick slice

b) Top and bottom slices removed

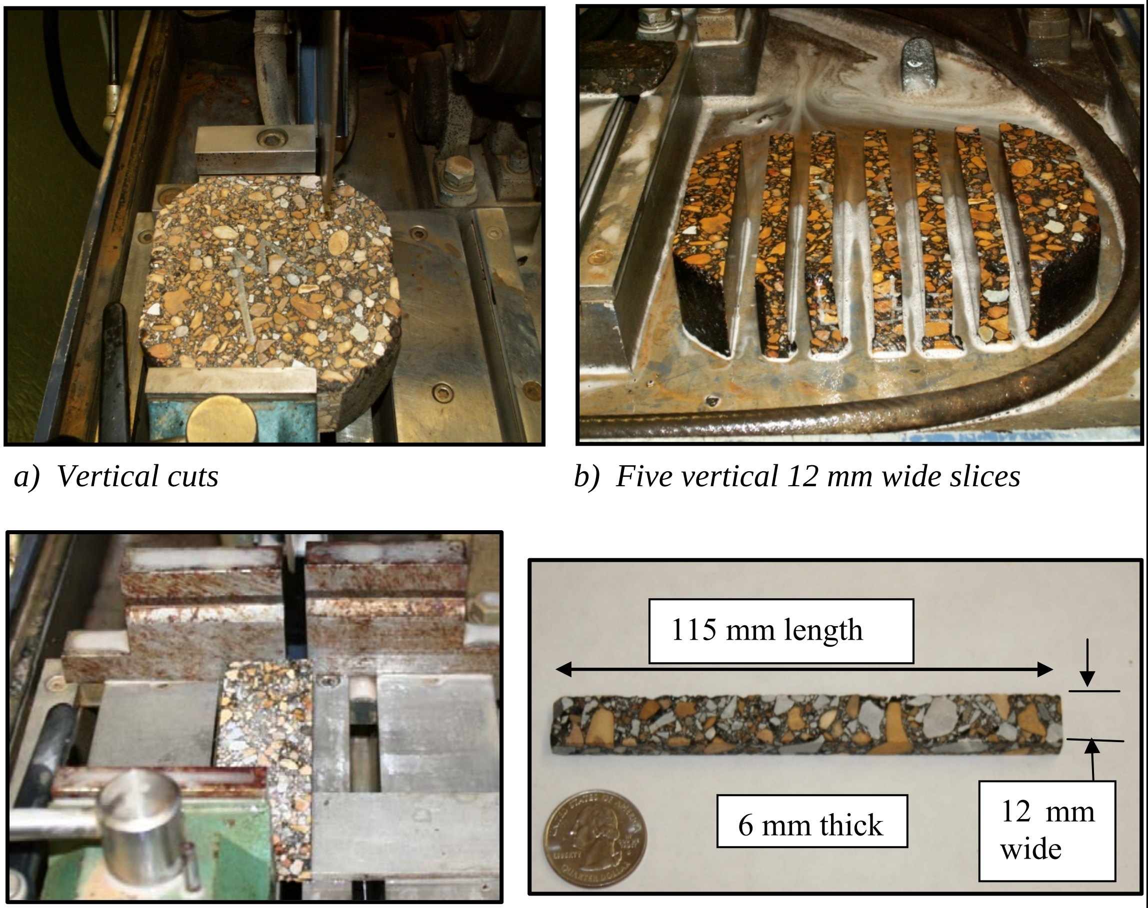
c) Marking for vertical perimeter cuts

d) Cutting vertical slices around perimeter

e) 115-mm x 115-mm rectangular block

f) Final 115 mm x 115 mm x 50 mm blocks

Figure 3.8 Step 1-Rectangular Block Preparation for *BBR* Specimen Production



a) Vertical cuts

b) Five vertical 12 mm wide slices

c) Horizontal cuts

d) Final mixture beam specimen

Figure 3.9 Step 2-Preparation of *BBR* Specimens from Rectangular Blocks

Each of the 12 mm wide vertical slices was then turned on its side and cut to produce 6 mm thick beams (Figure 3.9*c*). Two cuts were taken to produce two mixture beams per vertical slice. A final prepared mixture beam specimen is shown in Figure 3.9*d*. Twenty mixture beams can be cut from one gyratory specimen. The 6 mm thickness of the mixture beam corresponds to a vertical dimension in the original compacted specimen. The 12 mm width of the mixture beam corresponds to a horizontal dimension in the original compacted specimen.

3.5 Mixtures Tested

Four categories of mixtures were evaluated as part of this study: 1) 100% RAP mixtures; 2) airfield surface mixtures; 3) highway surface mixtures; and 4) highway base mixtures. Properties of the mixtures tested are given in the following sections.

3.5.1 100% RAP Mixtures

100% RAP mixtures were designed in the laboratory to 4% air voids according to the conventional Superpave design method for RAP sources *R-1, R-2* and *R-3.* The design compactive effort for 100% RAP mixtures was 65 gyrations. As noted in Section 3.3.1, the sand ratio for the *R-3* RAP aggregate gradation is especially high and would not be permitted as a standalone gradation. This mixture was investigated to determine the effect of testing a mixed RAP source that would not have been functioning in a pavement in the proportions of the stockpile. Properties of the designed 100% RAP mixtures are given in Table 3.5. Gradations of the 100% RAP mixtures are those of the respective RAP aggregates (Table 3.1 and Figure 3.1).

Table 3.5 Properties of 100% RAP Recycled Mixtures with 4% Air Voids

Mixture ID	***9.5-100/RM-1***	***9.5-100/RM-2***	***12.5-100/RM-3***
N_{des}	65	65	65
Virgin Binder Grade	PG 67-22	PG 67-22	PG 67-22
WMA	Sasobit® 1.0%	Sasobit® 1.0%	Sasobit® 1.0%
Mix Temp (C)	116	116	116
Comp Temp (C)	116	116	116
NMAS	9.5 mm	9.5 mm	12.5 mm
RAP (%)	100	100	100
RAP Source	*R-1*	*R-2*	*R-3*
$P_{AC} = P_b$ (%)	7.4	6.8	6.4
$P_{b(R)}$ (%)	5.4	5.6	5.0
$P_{be(V)}$ (%)	2.0	1.2	1.4
G_{mm}	2.317	2.370	2.381
G_{se}	2.574	2.619	2.614

3.5.2 Airfield Surface Mixtures

Airfield surface mixtures were designed in the laboratory to 4% air voids according to Superpave. The design compactive effort was 75 gyrations for all airfield mixtures. Properties of all airfield mixtures are given in the following sections. All gradations had a nominal maximum aggregate size (NMAS) of 12.5 mm. The gradations were designed to in general meet specifications used for airfield surface mixtures. The job mix formula (JMF) requirements of Unified Facilities Guide Specification UFGS-32 12 15 (USACE 2010) were the specifications considered herein. Slight gradation deviations (e.g. for mixtures *12.5-0/AM-13* to *12.5-0/AM-16* the gradation of the 1.18 mm sieve was 3% outside the specification) occurred in a few instances to preserve uniformity between gradations for performance comparisons while using substantial amounts of RAP, but these deviations from the specifications are within acceptable tolerance limits. Properties of airfield surface mixtures are given in the following tables (Tables 3.6 to 3.11) organized by RAP content and primary virgin aggregate type.

Table 3.6 Properties of 12.5 mm NMAS 0% RAP Limestone Virgin Aggregate Airfield Mixtures

Mixture ID		***12.5-0/AM-1***	***12.5-0/AM-2***	***12.5-0/AM-3***	***12.5-0/AM-4***
N_{des}		75	75	75	75
Binder Grade		PG 67-22	PG 67-22	PG 67-22	PG 67-22
WMA		None	Sasobit 1.0%	Evotherm 0.5%	Foam
Mix Temp (C)		165	130	130	130
Comp Temp (C)		146	116	116	116
Percent Passing	**25.0 mm**	100.0	100.0	100.0	100.0
	19.0 mm	100.0	100.0	100.0	100.0
	12.5 mm	99.0	99.0	99.0	99.0
	9.5 mm	89.8	89.8	89.8	89.8
	4.75 mm	70.7	70.7	70.7	70.7
	2.36 mm	44.5	44.5	44.5	44.5
	1.18 mm	27.1	27.1	27.1	27.1
	0.60 mm	17.9	17.9	17.9	17.9
	0.30 mm	12.6	12.6	12.6	12.6
	0.15 mm	9.1	9.1	9.1	9.1
	0.075 mm	7.2	7.2	7.2	7.2
***L-3* Limestone (%)**		70	70	70	70
***L-4* Limestone (%)**		30	30	30	30
RAP (%)		0	0	0	0
RAP Source		0	0	0	0
$P_{b(R)}$ **(%)**		0	0	0	0
G_{sb}		2.692	2.692	2.692	2.692
G_{sa}		2.774	2.774	2.774	2.774
***Abs* (%)**		1.12	1.12	1.12	1.12
$P_{AC} = P_b$ **(%)**		4.9	4.9	4.8	5.0
P_{be} **(%)**		4.1	4.1	3.9	4.3
$P_{ba(s)}$ **(%)**		0.9	0.9	0.9	0.8
G_{mm}		2.545	2.545	2.553	2.537
G_{se}		2.754	2.754	2.759	2.749
VMA		13.9	14.1	14.0	14.0
VFA		69.3	68.5	65.9	72.1
P_{200}/P_{be}		1.8	1.8	1.8	1.7

Table 3.7 Properties of 12.5 mm NMAS 0% RAP Crushed Gravel Virgin Aggregate Airfield Mixtures

	Mixture ID	***12.5-0/AM-13***	***12.5-0/AM-14***	***12.5-0/AM-15***	***12.5-0/AM-16***
	N_{des}	75	75	75	75
	Binder Grade	PG 67-22	PG 67-22	PG 67-22	PG 67-22
	WMA	None	Sasobit 1.0%	Evotherm 0.5%	Foam
	Mix Temp (C)	165	130	130	130
	Comp Temp (C)	146	116	116	116
Percent Passing	**25.0 mm**	100.0	100.0	100.0	100.0
	19.0 mm	100.0	100.0	100.0	100.0
	12.5 mm	94.5	94.5	94.5	94.5
	9.5 mm	84.4	84.4	84.4	84.4
	4.75 mm	52.8	52.8	52.8	52.8
	2.36 mm	35.1	35.1	35.1	35.1
	1.18 mm	25.4	25.4	25.4	25.4
	0.60 mm	19.3	19.3	19.3	19.3
	0.30 mm	12.1	12.1	12.1	12.1
	0.15 mm	8.8	8.8	8.8	8.8
	0.075 mm	5.6	5.6	5.6	5.6
	***G-2* Gravel (%)**	41.0	41.0	41.0	41.0
	***G-4* Gravel (%)**	40.0	40.0	40.0	40.0
	***L-2* Limestone (%)**	8.0	8.0	8.0	8.0
	***S-2* Sand (%)**	10.0	10.0	10.0	10.0
	***HL-1* Lime (%)**	1.0	1.0	1.0	1.0
	RAP (%)	0	0	0	0
	RAP Source	0	0	0	0
	$P_{b(R)}$ **(%)**	0	0	0	0
	G_{sb}	2.419	2.419	2.419	2.419
	G_{sa}	2.612	2.612	2.612	2.612
	***Abs* (%)**	3.05	3.05	3.05	3.05
	$P_{AC} = P_b$ **(%)**	6.6	6.0	6.0	6.5
	P_{be} **(%)**	5.8	5.4	5.4	5.7
	$P_{ba(s)}$ **(%)**	0.9	0.7	0.7	0.8
	G_{mm}	2.262	2.269	2.269	2.261
	G_{se}	2.471	2.458	2.458	2.466
	VMA	16.0	15.9	16.1	16.0
	VFA	76.2	71.1	70.1	75.7
	P_{200}/P_{be}	1.0	1.0	1.0	1.0

Table 3.8 Properties of 12.5 mm NMAS 25% RAP Limestone Virgin Aggregate Airfield Mixtures

Mixture ID		***12.5-0/AM-5***	***12.5-0/AM-6***	***12.5-0/AM-7***	***12.5-0/AM-8***
N_{des}		75	75	75	75
Binder Grade		PG 67-22	PG 67-22	PG 67-22	PG 67-22
WMA		None	Sasobit 1.0%	Evotherm 0.5%	Foam
Mix Temp (C)		165	130	130	130
Comp Temp (C)		146	116	116	116
Percent Passing	**25.0 mm**	100.0	100.0	100.0	100.0
	19.0 mm	100.0	100.0	100.0	100.0
	12.5 mm	98.2	98.2	98.2	98.2
	9.5 mm	87.3	87.3	87.3	87.3
	4.75 mm	61.4	61.4	61.4	61.4
	2.36 mm	39.2	39.2	39.2	39.2
	1.18 mm	26.1	26.1	26.1	26.1
	0.60 mm	18.9	18.9	18.9	18.9
	0.30 mm	13.1	13.1	13.1	13.1
	0.15 mm	8.9	8.9	8.9	8.9
	0.075 mm	6.8	6.8	6.8	6.8
***L-3* Limestone (%)**		45.0	45.0	45.0	45.0
***L-4* Limestone (%)**		30.0	30.0	30.0	30.0
RAP (%)		25.0	25.0	25.0	25.0
RAP Source		*R-1*	*R-1*	*R-1*	*R-1*
$P_{b(R)}$ **(%)**		5.5	5.5	5.5	5.5
G_{sb}		2.643	2.643	2.643	2.643
G_{sa}		2.730	2.730	2.730	2.730
***Abs* (%)**		1.23	1.23	1.23	1.23
$P_{AC} = P_b$ **(%)**		5.3	5.3	5.3	5.3
P_{be} **(%)**		4.4	4.4	4.4	4.4
$P_{ba(s)}$ **(%)**		1.0	1.0	1.0	1.0
G_{mm}		2.495	2.495	2.495	2.495
G_{se}		2.711	2.711	2.711	2.711
VMA		13.6	13.7	13.8	13.7
VFA		75.3	74.8	74.2	74.8
P_{200}/P_{be}		1.5	1.5	1.5	1.5

Table 3.9 Properties of 12.5 mm NMAS 25% RAP Crushed Gravel Virgin Aggregate Airfield Mixtures

Mixture ID	***12.5-0/AM-17***	***12.5-0/AM-18***	***12.5-0/AM-19***	***12.5-0/AM-20***
N_{des}	75	75	75	75
Binder Grade	PG 67-22	PG 67-22	PG 67-22	PG 67-22
WMA	None	Sasobit 1.0%	Evotherm 0.5%	Foam
Mix Temp (C)	165	130	130	130
Comp Temp (C)	146	116	116	116
Percent Passing				
25.0 mm	100.0	100.0	100.0	100.0
19.0 mm	100.0	100.0	100.0	100.0
12.5 mm	95.2	95.2	95.2	95.2
9.5 mm	86.2	86.2	86.2	86.2
4.75 mm	53.9	53.9	53.9	53.9
2.36 mm	35.7	35.7	35.7	35.7
1.18 mm	26.4	26.4	26.4	26.4
0.60 mm	20.8	20.8	20.8	20.8
0.30 mm	14.0	14.0	14.0	14.0
0.15 mm	9.9	9.9	9.9	9.9
0.075 mm	6.7	6.7	6.7	6.7
***G-2* Gravel (%)**	35.0	35.0	35.0	35.0
***G-4* Gravel (%)**	28.0	28.0	28.0	28.0
***L-2* Limestone (%)**	6.0	6.0	6.0	6.0
***S-2* Sand (%)**	5.0	5.0	5.0	5.0
***HL-1* Lime (%)**	1.0	1.0	1.0	1.0
RAP (%)	25.0	25.0	25.0	25.0
RAP Source	*R-1*	*R-1*	*R-1*	*R-1*
$P_{b(R)}$ **(%)**	5.5	5.5	5.5	5.5
G_{sb}	2.430	2.430	2.430	2.430
G_{sa}	2.607	2.607	2.607	2.607
***Abs* (%)**	2.82	2.82	2.82	2.82
$P_{AC} = P_b$ **(%)**	6.6	6.4	6.5	6.3
P_{be} **(%)**	5.5	5.6	5.7	5.3
$P_{ba(s)}$ **(%)**	1.2	0.9	0.9	1.0
G_{mm}	2.286	2.276	2.273	2.286
G_{se}	2.501	2.480	2.480	2.489
VMA	15.7	15.7	16.0	15.6
VFA	74.1	75.6	75.2	72.5
P_{200}/P_{be}	1.2	1.2	1.2	1.3

Table 3.10 Properties of 12.5 mm NMAS 50% RAP Limestone Virgin Aggregate Airfield Mixtures

		12.5-0/AM-9	12.5-0/AM-10	12.5-0/AM-11	12.5-0/AM-12
Mixture ID		*12.5-0/AM-9*	*12.5-0/AM-10*	*12.5-0/AM-11*	*12.5-0/AM-12*
N_{des}		75	75	75	75
Binder Grade		PG 67-22	PG 67-22	PG 67-22	PG 67-22
WMA		None	Sasobit 1.0%	Evotherm 0.5%	Foam
Mix Temp (C)		165	130	130	130
Comp Temp (C)		146	116	116	116
Percent Passing	**25.0 mm**	100.0	100.0	100.0	100.0
	19.0 mm	100.0	100.0	100.0	100.0
	12.5 mm	97.6	97.6	97.6	97.6
	9.5 mm	88.2	88.2	88.2	88.2
	4.75 mm	61.1	61.1	61.1	61.1
	2.36 mm	41.0	41.0	41.0	41.0
	1.18 mm	29.4	29.4	29.4	29.4
	0.60 mm	22.4	22.4	22.4	22.4
	0.30 mm	14.6	14.6	14.6	14.6
	0.15 mm	8.9	8.9	8.9	8.9
	0.075 mm	6.2	6.2	6.2	6.2
***L-3* Limestone (%)**		15.0	15.0	15.0	15.0
***L-3*b Limestone (%)**		15.0	15.0	15.0	15.0
***L-4* Limestone (%)**		20.0	20.0	20.0	20.0
RAP (%)		50.0	50.0	50.0	50.0
RAP Source		*R-1*	*R-1*	*R-1*	*R-1*
$P_{b(R)}$ (%)		5.5	5.5	5.5	5.5
G_{sb}		2.603	2.603	2.603	2.603
G_{sa}		2.687	2.687	2.687	2.687
***Abs* (%)**		1.23	1.23	1.23	1.23
$P_{AC} = P_b$ (%)		5.9	6.1	6.1	6.1
P_{be} (%)		5.0	5.2	5.2	5.2
$P_{ba(s)}$ (%)		0.9	0.9	0.9	0.9
G_{mm}		2.437	2.430	2.430	2.430
G_{se}		2.665	2.665	2.665	2.665
VMA		15.6	16.5	16.4	15.8
VFA		73.1	70.9	71.8	74.8
P_{200}/P_{be}		1.2	1.2	1.2	1.2

Table 3.11 Properties of 12.5 mm NMAS 50% RAP Crushed Gravel Virgin Aggregate Airfield Mixtures

	Mixture ID	***12.5-0/AM-21***	***12.5-0/AM-22***	***12.5-0/AM-23***	***12.5-0/AM-24***
	N_{des}	75	75	75	75
	Binder Grade	PG 67-22	PG 67-22	PG 67-22	PG 67-22
	WMA	None	Sasobit 1.0%	Evotherm 0.5%	Foam
	Mix Temp (C)	165	130	130	130
	Comp Temp (C)	146	116	116	116
Percent Passing	**25.0 mm**	100.0	100.0	100.0	100.0
	19.0 mm	100.0	100.0	100.0	100.0
	12.5 mm	95.2	95.2	95.2	95.2
	9.5 mm	86.5	86.5	86.5	86.5
	4.75 mm	54.5	54.5	54.5	54.5
	2.36 mm	36.3	36.3	36.3	36.3
	1.18 mm	27.6	27.6	27.6	27.6
	0.60 mm	22.6	22.6	22.6	22.6
	0.30 mm	16.1	16.1	16.1	16.1
	0.15 mm	11.3	11.3	11.3	11.3
	0.075 mm	6.7	6.7	6.7	6.7
	***G-2b* Gravel (%)**	22.0	22.0	22.0	22.0
	***G-4* Gravel (%)**	22.0	22.0	22.0	22.0
	***L-2* Limestone (%)**	5.0	5.0	5.0	5.0
	***S-2* Sand (%)**	0.0	0.0	0.0	0.0
	***HL-1* Lime (%)**	1.0	1.0	1.0	1.0
	RAP (%)	50.0	50.0	50.0	50.0
	RAP Source	*R-1*	*R-1*	*R-1*	*R-1*
	$P_{b(R)}$ **(%)**	5.5	5.5	5.5	5.5
	G_{sb}	2.436	2.436	2.436	2.436
	G_{sa}	2.602	2.602	2.602	2.602
	***Abs* (%)**	2.78	2.78	2.78	2.78
	$P_{AC} = P_b$ **(%)**	6.8	7.0	7.1	6.7
	P_{be} **(%)**	5.5	5.7	5.8	5.5
	$P_{ba(s)}$ **(%)**	1.4	1.3	1.3	1.2
	G_{mm}	2.293	2.286	2.283	2.291
	G_{se}	2.516	2.515	2.5151	2.510
	VMA	15.6	15.8	16.4	15.8
	VFA	75.2	77.5	75.4	73.8
	P_{200}/P_{be}	1.2	1.2	1.2	1.2

3.5.3 Highway Surface Mixtures

3.5.3.1 Control Mixtures

Control highway surface mixture properties are given in Table 3.12. Initially a 0% RAP control mixture was developed having a virgin aggregate gradation matching that of the high RAP mixtures as closely as possible. This mixture is identified in Table 3.12 as *9.5-0/CM-1*. VMA and dust to effective binder ratio values for this mixture are out of MDOT allowable ranges due to the high dust content of the gradation.

Three current practice control mixtures were obtained and tested as part of this experimental program; they were selected to encompass a performance range of current practice rehabilitation mixtures. One 50 design gyration mixture and two 85 design gyration mixtures were selected. One of the 85 gyration mixtures contained neat binder and the other contained polymer-modified binder. Properties of MDOT approved control mixtures *9.5-15/CM-2*, *9.5-15/CM-3*, and *9.5-15/CM-4a* are provided in Table 3.12 and were taken directly from the mix design sheets.

The 50 gyration control mixture (*9.5-15/CM-2*) was obtained from a city street overlay project in Starkville, MS. All mix was sampled on consecutive days in June 2010. The 50 gyration mix was conventional HMA; target overlay thickness was 3.8 cm.

To investigate the properties of current practice 85 gyration mixtures containing polymer-modified binder, control mixture *9.5-15/CM-3* was obtained. This mixture was sampled directly from the asphalt plant in September 2010, placed in metal buckets and returned to the MSU laboratory; the mix was then re-heated and compacted at a later date. The mix with polymer-modified PG 76-22 binder was being produced at the plant as warm mix using foaming technology; the material exited the plant at approximately 132 C and was field compacted at approximately 121 C. Typical MDOT procedure for compaction of reheated mix that was originally produced as foamed warm mix is to compact at a temperature near what would be the hot mix compaction temperature for the mix. This procedure was followed for compaction of reheated *9.5-15/CM-3* mixture; a temperature of 138 C was utilized. The mix was being used for isolated sections of mill and repair near Tupelo MS; the placement thickness was approximately 3.8 cm.

For the second control mixture an 85 gyration mix where PG 67-22 was being substituted for polymer-modified binder was selected. This mix (*9.5-15/CM-4a*) was being used for an overlay project on U.S. Highway 45 near West Point, MS. All the mix was sampled on consecutive days, in November 2009 from the paving location. The 85 gyration mix with neat PG 67-22 binder was conventional hot mix and the target overlay thickness was 5 cm.

At the same time *9.5-15/CM-4a* was being produced, aggregate and RAP was sampled from the stockpiles for later use in producing laboratory specimens of the same mix for comparison to the plant mixed asphalt. Neat PG 67-22 asphalt binder was also sampled from the plant for use in mixture *9.5-15/CM-4b* as previously mentioned in Section 3.3.3. A stock PG 76-22 was used to produce mixture *9.5-15/CM-4c* in the laboratory.

Table 3.12 Properties of 9.5 mm NMAS Control Highway Surface Mixtures

Mixture ID	***9.5-0/ CM-1***	***9.5-15/ CM-2***	***9.5-15/ CM-3***	***9.5-15/ CM-4a***	***9.5-15/ CM-4b***	***9.5-15/ CM-4c***	***9.5-15/ CM-5-28***
Prep Method[a]	1	2	3	2	1	1	2 or 3
Design Traffic	MT	ST	HT	HT	HT	HT	varied
N_{des}	65	50	85	85	85	85	varied
Binder Grade	67-22	67-22	76-22	67-22	67-22	76-22	varied
WMA	none	none	foam[b]	none	none	none	none
Mix Temp (C)	154	157	132	160	154	166	varied
Comp Temp (C)	146	field	138	field	146	154	varied
Percent Passing							
25.0 mm	100	100	100	100	100	100	100
19.0 mm	100	100	100	100	100	100	100
12.5 mm	100	100	100	100	100	100	varied
9.5 mm	94.6	92.4	95.9	96.1	96.1	96.1	varied
4.75 mm	62.2	---	---	---	---	---	varied
2.36 mm	41.6	40.1	41.0	37.1	37.1	37.1	varied
1.18 mm	30.6	---	---	---	---	---	varied
0.60 mm	22.9	---	---	---	---	---	varied
0.30 mm	14.4	---	---	---	---	---	varied
0.15 mm	10.1	---	---	---	---	---	varied
0.075 mm	7.8	5.5	6.1	6.0	6.0	6.0	varied
Gravel (%)	67 (*G-1*)	75	43	37	37	37	varied
Limestone (%)	22 (*L-1*)	0	31	37	37	37	varied
Sand (%)	10 (*S-1*)	9	10	10	10	10	varied
Hyd. Lime (%)	1 (*HL-1*)	1	1	1	1	1	varied
RAP (%)	0	15	15	15	15	15	varied
$P_{b(R)}$ **(%)**	0	4.6	5.5	5.6	5.6	5.6	varied
G_{sb}	2.458	2.533	2.480	2.518	2.518	2.518	varied
G_{sa}	2.642	2.634	2.591	2.658	2.658	2.658	varied
Abs **(%)**	2.82	1.52	1.73	2.08	2.08	2.08	varied
$P_{AC} = P_b$ **(%)**	5.70	6.25	5.80	5.80	5.80	5.80	varied
$P_{b(V)}$ **(%)**	5.70	5.57	4.98	4.96	4.96	4.96	varied
G_{mm}	2.339	2.362	2.332	2.367	2.367	2.367	varied
G_{se}	2.533	2.585	2.526	2.569	2.569	2.569	varied
VMA	14.5	16.0	15.0	15.0	15.0	15.0	varied
VFA	72.4	75.0	73.3	73.3	73.3	73.3	varied
P_{200}/P_{be}	1.7	1.0	1.2	1.2	1.2	1.2	varied

a) Preparation methods were as follows:
   1. Laboratory mixed and short term aged according to standard procedure.
   2. Plant mixed, field sampled, transported in insulated containers, compacted immediately.
   3. Plant mixed, plant sampled, brought to laboratory, reheated prior to compaction.

b) Original mixture was foamed but it was re-heated prior to compaction.

Additional control mixtures were required for Cantabro durability testing. Quality control (QC) specimens of plant produced mixtures were obtained from a local asphalt plant and tested at MSU. Additionally, quality assurance (QA) specimens of a range of asphalt mixture types from around the state prepared at the MDOT central materials laboratory were tested at MDOT. Details of these mixtures are given in Table 3.13. Plant mixtures *9.5-15/CM-5* and *9.5-15/CM-6* were composed of the same 9.5 mm gradation as control mixture *9.5-15/CM-4a*. The only differences were the design compactive efforts and therefore the total asphalt contents of the mixtures. In contrast to the 5.8% total design asphalt content of control mixture *9.5-15/CM-4a*, the design total asphalt content of the 50 gyration mixture *9.5-15/CM-5* was 6.2% and the design total asphalt content of the 65 gyration *9.5-15/CM-6* mixture was 6.0%. Twenty-two mixes were compacted and tested at the MDOT central laboratory, and properties of these mixtures are given in Table 3.13 (control mixtures 7 to 28).

Table 3.13 Properties of Plant Mixed 9.5 mm NMAS Control Highway Surface Mixtures 5 to 28

Mixture ID	Binder Grade	$P_{AC} = P_b$ (%)	N_{des}	Aggregate Components (%) Gravel	Limestone	Sand	RAP
9.5-15/CM-5	67-22	6.2	50	37	37	10	15
9.5-15/CM-6	67-22	6.0	65	37	37	10	15
9.5-15/CM-7	76-22	5.4	85	29	50	5	15
9.5-15/CM-8	76-22	5.1	85	29	50	5	15
9.5-10/CM-9	76-22	5.5	85	79	5	5	10
9.5-15/CM-10	76-22	5.5	85	50	24	10	15
9.5-15/CM-11	76-22	6.2	85	40	34	10	15
9.5-15/CM-12[a]	76-22	5.4	85	75.5	0	7	15
9.5-15/CM-13	76-22	5.8	85	45	7	32[b]	15
9.5-15/CM-14	76-22	5.5	85	61	20	3	15
9.5-15/CM-15	67-22	6.0	85	68	9	7	15
9.5-15/CM-16	67-22	6.1	65	37	37	10	15
9.5-15/CM-17	67-22	5.6	65	52	6	26[b]	15
9.5-15/CM-18	67-22	5.3	65	50	18	16[b]	15
9.5-15/CM-19	67-22	5.5	65	31	50	3	15
9.5-15/CM-20	67-22	6.4	65	40	20	24[b]	15
9.5-10/CM-21	67-22	5.7	65	34	46	9	10
9.5-15/CM-22[a]	67-22	5.8	65	74	0	8	15
9.5-0/CM-23	67-22	5.8	50	40	50	9	0
9.5-10/CM-24	67-22	5.6	50	64	10	10	10
9.5-10/CM-25	67-22	5.4	50	29	45	10	10
9.5-6/CM-26	67-22	5.3	50	28	50	6	6
9.5-10/CM-27	67-22	6.4	50	37	37	10	10
9.5-10/CM-28	67-22	5.2	50	49	25	10	10

Notes: All mixtures contained 1% hydrated lime. CM-5 and CM-6 were prepared with method 2 and mixtures CM-7 to CM-28 with method 3.

a) Contained dust.

b) Contained manufactured sand.

3.5.3.2 25 and 50% RAP Mixtures

Four recycled mixtures containing either 25 or 50% RAP were designed in the laboratory; their properties are given in Table 3.14. Due to difficulty meeting gradation requirements, RAP source *R-3* was not utilized for 25 or 50% RAP mixtures. When determining virgin aggregate gradations, minimum aggregate stockpile percentages of 5% were used to align with cold feed limitations of asphalt plants. For the four recycled mixtures, the same virgin aggregate stockpile percentages were used with each RAP source at the 25% and 50% RAP contents to control the effects of virgin aggregate gradation. Virgin PG 67-22 binder with Sasobit® was used for all mixtures.

Table 3.14 Properties of 9.5 mm NMAS 25 and 50% RAP Recycled Mixtures

	Mixture ID	*9.5-25/RM-1*	*9.5-25/RM-2*	*9.5-50/RM-1*	*9.5-50/RM-2*
	N_{des}	65	65	65	65
	Virgin Binder Grade	PG 67-22	PG 67-22	PG 67-22	PG 67-22
	WMA	Sasobit 1.0%	Sasobit 1.0%	Sasobit 1.0%	Sasobit 1.0%
	Mix Temp (C)	116	116	116	116
	Comp Temp (C)	116	116	116	116
Percent Passing	**25.0 mm**	100	100	100	100
	19.0 mm	100	100	100	100
	12.5 mm	99.1	100	98.3	99.9
	9.5 mm	93.0	95.1	91.8	95.9
	4.75 mm	59.2	62.5	58.4	64.9
	2.36 mm	39.7	42.4	39.7	45.2
	1.18 mm	29.9	31.5	30.9	34.0
	0.60 mm	23.2	24.3	25.0	27.1
	0.30 mm	14.6	15.5	15.8	17.5
	0.15 mm	9.6	9.9	9.9	10.5
	0.075 mm	7.2	7.5	7.3	7.8
	***G-1* Gravel (%)**	56	56	40	40
	***L-1* LST (%)**	8	8	0	0
	***S-1* Sand (%)**	10	10	9	9
	***HL-1* Lime (%)**	1	1	1	1
	RAP (%)	25	25	50	50
	RAP Source	*R-1*	*R-2*	*R-1*	*R-2*
	$P_{b(R)}$ **(%)**	5.5	5.6	5.5	5.6
	G_{sb}	2.450	2.460	2.453	2.473
	G_{sa}	2.624	2.623	2.610	2.609
	***Abs* (%)**	2.70	2.52	2.46	2.10
	$P_{AC} = P_b$ **(%)**	6.1	5.6	6.5	6.2
	$P_{b(V)}$ **(%)**	4.7	4.2	3.7	3.4
	G_{mm}	2.306	2.334	2.311	2.338
	G_{se}	2.508	2.524	2.530	2.552
	VMA	15.2	14.0	15.4	14.9
	VFA	73.6	71.5	74.1	73.1
	P_{200}/P_{be}	1.4	1.6	1.4	1.6

3.5.4 Highway Base Mixtures

3.5.4.1 Control Mixtures

Control highway base mixtures properties are given in Table 3.15. Four current practice control mixtures were obtained and tested; they were selected to encompass a performance range of current practice base mixtures. All four of the mixtures contained 15% RAP. Three 12.5 mm NMAS mixtures and one 19.0 mm NMAS mixture were selected. All the 12.5 mm NMAS mixtures used neat PG 67-22 binder; the 19.0 mm mix used polymer-modified PG 76-22 binder.

The three 12.5 mm NMAS control mixtures represented three levels of design compactive effort (50, 65, and 85 gyrations). They were selected to represent the range of performance of current practice, but their aggregates and gradations all varied so direct comparison is not possible. The 19.0 mm NMAS mixture with polymer-modified binder represents a premium base mixture for applications with high performance standards.

Additional control mixtures were required for Cantabro durability testing. Quality assurance (QA) specimens of a range of asphalt mixture types from around the state prepared at the MDOT central materials laboratory were tested at MDOT. Details of these mixtures are given in Tables 3.16 and 3.17. These mixtures contained varying amounts of RAP up to the 30% maximum allowed by MDOT in base mixtures.

Table 3.15 Properties of 12.5 and 19.0 mm NMAS Control Highway Base Mixtures

Mixture ID	***12.5-15/ CM-1***	***12.5-15/ CM-2***	***12.5-15/ CM-3***	***19.0-15/ CM-4***	***CM-5* to *CM-27***
Prep Method[a]	3	2	1	3	3
N_{des}	50	65	85	85	varied
Binder Grade	67-22	67-22	67-22	76-22	varied
WMA	none	none	none	none	none
Mix Temp (C)	163	163	165	160	varied
Comp Temp (C)	146	146	146	149	varied
Percent Passing					
25.0 mm	100	100	100	100	100
19.0 mm	100	100	100	100	varied
12.5 mm	94.3	95.4	93.1	89.4	varied
9.5 mm	---	---	---	---	varied
4.75 mm	---	---	---	51.0	varied
2.36 mm	50.1	36.2	40.5	---	varied
1.18 mm	---	---	---	---	varied
0.60 mm	---	---	---	---	varied
0.30 mm	---	---	---	---	varied
0.15 mm	---	---	---	---	varied
0.075 mm	5.3	5.9	5.8	5.2	varied
Gravel (%)	52	39	63	73	varied
Limestone (%)	12	35	15	4	varied
Sand (%)	20	10	6	7	varied
Hyd. Lime (%)	1	1	1	1	varied
RAP (%)	15	15	15	15	varied
$P_{b(R)}$ **(%)**	5.0	5.6	6.4	5.6	varied
G_{sb}	2.556	2.515	2.510	2.497	varied
G_{sa}	2.646	2.663	2.628	2.627	varied
***Abs* (%)**	1.33	2.20	1.79	1.99	varied
$P_{AC} = P_b$ **(%)**	5.2	5.40	5.70	4.90	varied
$P_{b(V)}$ **(%)**	4.45	4.56	4.74	4.10	varied
G_{mm}	2.410	2.378	2.350	2.376	varied
G_{se}	2.599	2.567	2.547	2.547	varied
VMA	14.2	14.1	15.2	13.0	varied
VFA	71.8	71.6	73.7	69.2	varied
P_{200}/P_{be}	1.15	1.3	1.1	1.3	varied

a) Preparation methods were as follows:
1. Laboratory mixed and short term conditioned according to standard procedure.
2. Plant mixed, field sampled, transported in insulated containers, compacted immediately.
3. Plant mixed, plant sampled, brought to laboratory, reheated prior to compaction.

Table 3.16 Properties of Plant Mixed 12.5 mm NMAS Control Highway Base Mixtures 5 to 20

Mixture ID	Binder Grade	$P_{AC} = P_b$ (%)	N_{des}	Aggregate Components (%) Gravel	Limestone	Sand	RAP
12.5-12/CM-5	76-22	5.2	85	27	50	10	12
12.5-15/CM-6	76-22	5.5	85	40	34	10	15
12.5-20/CM-7	76-22	3.7	85	0	75	4	20
12.5-14/CM-8	67-22	5.2	85	74	4	7	14
12.5-15/CM-9	67-22	5.2	85	75	0	9	15
12.5-15/CM-10	67-22	5.7	65	73	4	7	15
12.5-15/CM-11	67-22	5.0	65	24	50	10	15
12.5-12/CM-12	67-22	5.2	65	53	26	8	12
12.5-15/CM-13	67-22	5.3	65	69	5	10	15
12.5-15/CM-14	67-22	5.4	50	75	0	9	15
12.5-15/CM-15	67-22	4.7	50	0	61	8	30
12.5-30/CM-16	67-22	5.6	50	64	10	10	15
12.5-12/CM-17	67-22	5.0	50	68	5	8	12
12.5-15/CM-18	67-22	5.7	50	40	34	10	15
12.5-15/CM-19	67-22	6.0	50	52	16	16	15
12.5-15/CM-20	67-22	5.2	50	72	0	12	15

Notes: All mixtures contained 1% hydrated lime and were prepared by method 3.

Table 3.17 Properties of Plant Mixed 19.0 mm NMAS Control Highway Base Mixtures 21 to 37

Mixture ID	Binder Grade	$P_{AC} = P_b$ (%)	N_{des}	Aggregate Components (%) Gravel	Limestone	Sand	RAP
19.0-15/CM-21	76-22	4.7	85	73	4	7	15
19.0-15/CM-22	76-22	4.8	85	48	26	10	15
19.0-20/CM-23	67-22	4.6	85	59	10	10	20
19.0-20/CM-24	67-22	4.9	85	55	14	10	20
19.0-20/CM-25	67-22	5.7	85	68	0	10	20
19.0-12/CM-26	67-22	4.4	85	64	13	10	12
19.0-20/CM-27	67-22	4.5	85	28	41	10	20
19.0-18/CM-28	67-22	5.1	65	71	0	10	18
19.0-25/CM-29	67-22	3.9	65	0	64	10	25
19.0-15/CM-30	67-22	4.9	65	69	5	10	15
19.0-30/CM-31	67-22	4.6	65	0	65	4	30
19.0-15/CM-32	67-22	4.9	65	0	74	10	15
19.0-10/CM-33	67-22	5.7	50	73	6	10	10
19.0-20/CM-34	67-22	4.4	50	35	32	12	20
19.0-15/CM-35	67-22	4.4	50	39	10	35	15
19.0-20/CM-36	67-22	5.3	50	49	20	10	20
19.0-15/CM-37	67-22	4.8	50	0	74	10	15

Notes: All mixtures contained 1% hydrated lime and were prepared by method 3.

3.5.4.2 50 and 75% RAP Mixtures

Four recycled mixtures containing either 50 or 75% RAP were designed in the laboratory; their properties are given in Table 3.18. Based on results of the 100% RAP mixture testing, RAP source *R-3* was not utilized for 50 or 75% RAP mixtures. For the 50% recycled mixtures, the virgin aggregate stockpile percentages were adjusted to match the same overall gradation as closely as possible. The same goal was attempted for the 75% RAP mixtures, however the RAP aggregate gradation was dominate and the overall gradations could not be matched as closely. Virgin PG 67-22 binder with 1.0% Sasobit® was used for all mixtures.

Table 3.18 Properties of 12.5 mm NMAS 50 and 75% RAP Recycled Mixtures

Mixture ID	***12.5-50/RM-1***	***12.5-50/RM-2***	***12.5-75/RM-1***	***12.5-75/RM-2***
N_{des}	50	50	50	50
Binder Grade	PG 67-22	PG 67-22	PG 67-22	PG 67-22
WMA	Sasobit 1.0%	Sasobit 1.0%	Sasobit 1.0%	Sasobit 1.0%
Mix Temp (C)	116	116	116	116
Comp Temp (C)	116	116	116	116
Percent Passing				
25.0 mm	100	100	100	100
19.0 mm	100	100	100	100
12.5 mm	93.0	90.5	91.9	94.3
9.5 mm	83.5	81.2	82.4	88.6
4.75 mm	51.3	53.2	53.0	62.8
2.36 mm	33.7	36.0	36.3	44.4
1.18 mm	25.9	26.9	29.2	33.9
0.60 mm	21.4	22.3	25.1	28.2
0.30 mm	15.0	16.0	17.3	19.9
0.15 mm	10.2	10.1	11.1	12.0
0.075 mm	7.8	7.7	8.3	8.9
***G-1* Gravel (%)**	23	8	0	0
***G-3* Gravel (%)**	23	41	24	24
***L-1* Limestone (%)**	3	0	0	0
***S-1* Sand (%)**	0	0	0	0
***HL-1* Lime (%)**	1	1	1	1
RAP (%)	50	50	75	75
RAP Source	*R-1*	*R-2*	*R-1*	*R-2*
$P_{b(R)}$ **(%)**	5.5	5.6	5.5	5.6
G_{sb}	2.443	2.456	2.458	2.490
G_{sa}	2.608	2.602	2.599	2.597
***Abs* (%)**	2.59	2.28	2.21	1.66
$P_{AC} = P_b$ **(%)**	7.3	6.1	7.3	6.1
$P_{b(V)}$ **(%)**	4.6	3.3	3.3	1.9
G_{mm}	2.281	2.317	2.293	2.332
G_{se}	2.522	2.522	2.538	2.541
VMA	16.9	15.0	17.0	15.6
VFA	76.3	73.3	76.4	74.3
P_{200}/P_{be}	1.3	1.5	1.4	1.7

CHAPTER 4

EXPERIMENTAL PROGRAM

4.1 Overview of Experimental Program

There are four major components of this experimental program corresponding to the four primary objectives of this dissertation. The experimental program organization is shown in Figure 4.1. Prior to development of experimental designs, a description of non standard test methods is given in Section 4.2.

The first component is characterization of RAP properties and evaluation of 100% RAP mixture properties (described in Section 4.3.1). The second component is evaluation of high RAP-WMA mixtures for airfield surfaces (described in Section 4.3.2). For the second component, control mixtures were part of the experimental design. The third component is evaluation of high RAP-WMA mixtures for highway surfaces (described in Section 4.3.3). For the third component, a range of current practice control mixtures were obtained for comparison to high RAP-WMA mixtures. The fourth component is evaluation of high RAP-WMA mixtures for highway bases (described in Section 4.3.4). For the fourth component, a range of current practice control mixtures were obtained for comparison to high RAP-WMA mixtures.

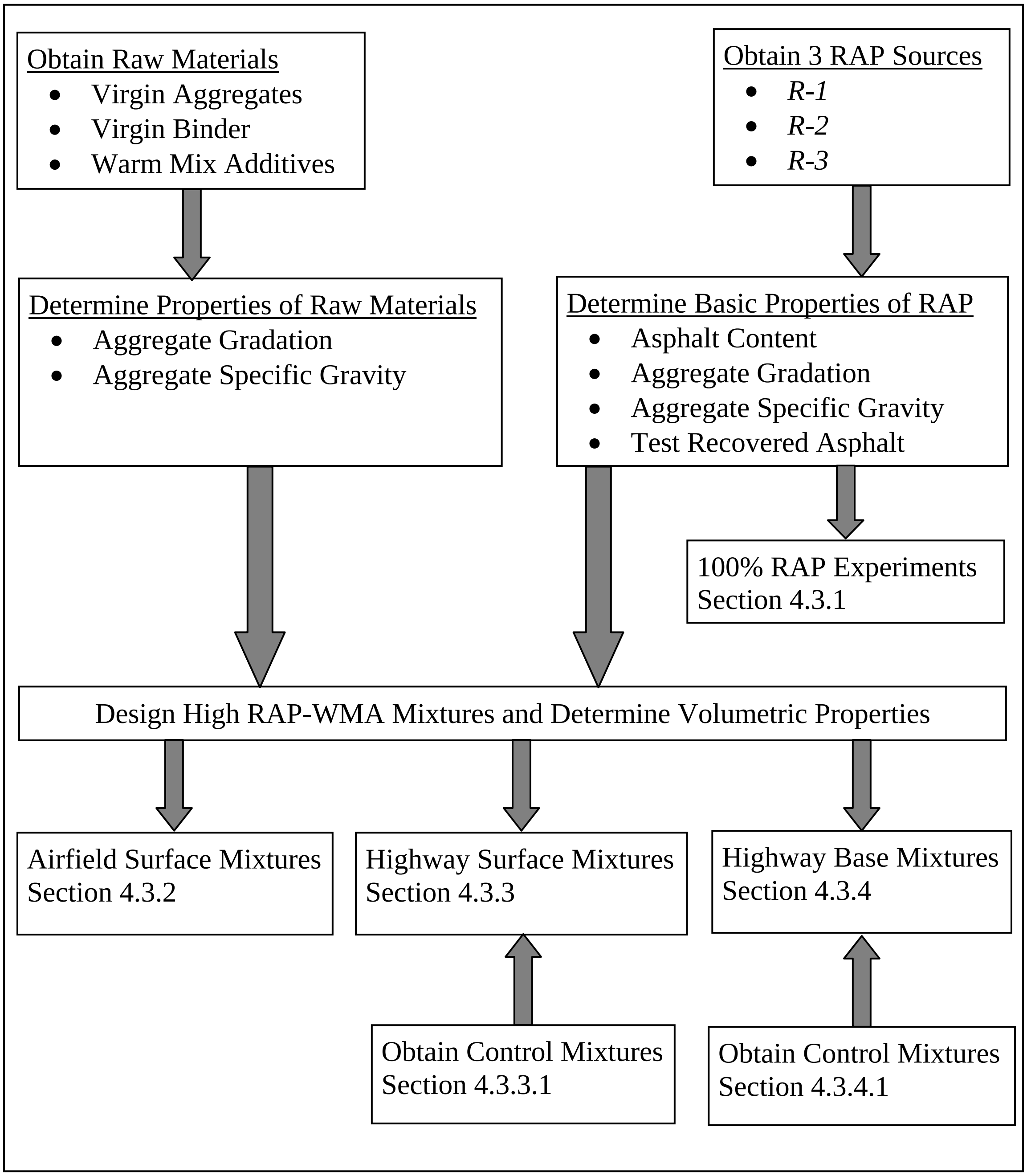


Figure 4.1 Flow Chart of Experimental Program

4.2 Test Methods

4.2.1 Fundamental Properties

Theoretical maximum specific gravity (G_{mm}) was tested according to *AASHTO T 209*. A sample of asphalt mixture was mixed and short term conditioned according to Section 3.4.2 and at the conclusion of the short term aging period, the loose sample was cooled. The sample was divided into two portions with a sample splitter; *AASHTO T 209* was performed on each split portion of the sample and the results were averaged to produce one G_{mm} result.

Bulk specific gravity (G_{mb}) of compacted specimens was measured according to *AASHTO T 331* (Corelok®). An exception was for moisture damage (*TSR*) testing, where G_{mb} was measured according to *ASTM D 2726* (submerged specimen method) in accordance with test method requirements. A second exception was for specimens of airfield mixtures containing primarily limestone aggregate for rut resistance (*APA*) testing, where G_{mb} was measured according to *AASHTO T 166* at a secondary laboratory. It was observed during testing that there were differences between void levels measured by the two methods at the desired *APA* void level. Corelok® was observed to result in higher air voids than *T 166*. The difference is relatively small, but direct comparison of rutting results between limestone and gravel airfield mixtures was avoided due to the observed difference in air voids between the methods.

Density of compacted slabs used for PURWheel and skid resistance testing were estimated by measurement of the slab mass and slab thickness at six locations around the perimeter. A bulk slab density value was computed from this data ($D_{b\text{-}s}$); this density value was used in conjunction with the mixture G_{mm} to compute an estimate of air voids for the slab. To correlate this estimate of air voids with *AASHTO T 331*, a correlation equation from Doyle and Howard (2011) was utilized. Eq. 4.1 is the combined equation relating air voids to bulk slab density and G_{mm}. The equation was developed based on coring of 61 slabs (total of 366 cored specimens) compacted in the *LAC* and measurement of their air voids by *AASHTO T 331*.

$$V_{a(T\,331)} = 89\left(1 - \frac{D_{b-s}}{G_{mm}}\right) \qquad (R^2 = 0.96) \qquad \text{(Eq 4.1)}$$

Where:

$V_{a(T\,331)}$ = air voids measured according to *AASHTO T 331*

D_{b-s} = bulk slab density (g/cm^3)

4.2.2 Indirect Tensile Strength

Indirect tensile strength and time to failure were determined on *SGC* compacted specimens; testing was performed with an Interlaken universal soil and asphalt test system. Before testing, specimens were brought to thermal equilibrium by placing them in the Interlaken environmental chamber where they were ultimately tested (Figure 4.2*a*). A specimen of comparable mass with an embedded thermometer was placed in the chamber with the test specimens to ensure sufficient conditioning had taken place prior to testing. For select 100% RAP mixtures and highway surface mixtures, testing was conducted at low temperatures; details are provided in Sections 4.3.1 and 4.3.3. For highway base mixtures all testing was conducted at 25 C. The length of each specimen was measured, load-time data was recorded from the test at a frequency of 30 Hz; the loading rate was 50 mm/min. This information was used to calculate the indirect tensile strength at failure (S_t). Figure 4.2*b* is a photo of testing.

a) Conditioning Samples *b) Indirect Tensile Strength Testing*

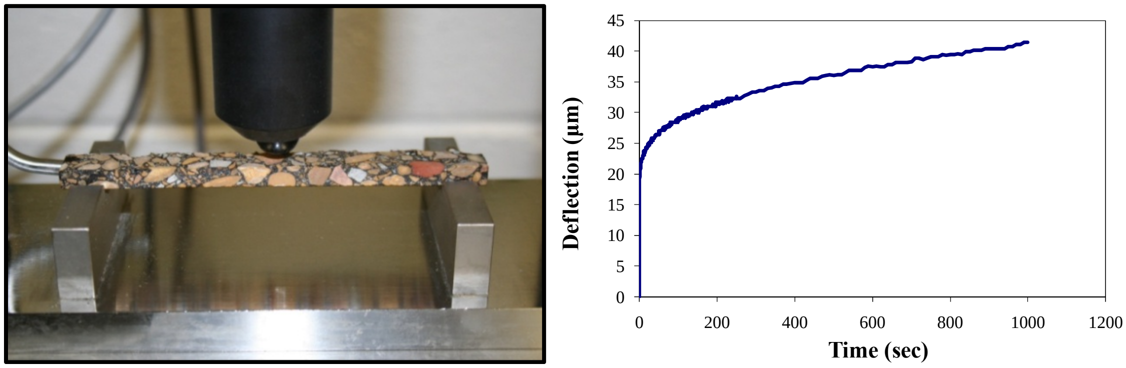
Figure 4.2 Indirect Tensile Strength Testing

4.2.3 Bending Beam Rheometer Mixture Test

To investigate low temperature mechanical properties, flexural creep testing was performed with a *BBR* on mixture beam specimens prepared according to Section 3.4.4. The level of replication varied slightly between different components of this dissertation, partly since the airfield mixture component of this experimental program was conducted for a separate research project and partly because information was identified through literature review that indicated that three replicate specimens was likely adequate.

For the 100% RAP component of this experimental program, five replicate specimens were tested at each temperature. For airfield surface mixtures, two beam replicates were tested from each gyratory specimen at each test temperature. For control highway surface mixtures, five replicate specimens were tested from each gyratory specimen at each temperature. Literature review in combination with analysis of test method variability conducted as part of the highway surface mixture component of the experimental program led to a reduction in the level of replication for 25 and 50% RAP highway surface mixtures; three replicate specimens were tested from each *SGC* specimen for those mixtures.

Dimensions of the beam specimens were measured and recorded prior to testing. A CANNON Thermoelectric *BBR* was used for all testing. Beam specimens were immersed in the cooling bath containing methanol of the *BBR* for 60 ± 5 minutes before testing to ensure they reached thermal equilibrium at the desired test temperature. The test parameters were different than those for the standard binder test; the specific test parameters for the mixture test were a 4.9 N constant load and 1000 second test duration. Specimen deflection at the center of the mixture beam was recorded by the test equipment throughout the test. Figure 4.3*a* shows the test fixture with a mixture beam specimen while removed from the coolant bath. Figure 4.3*b* is an example of deflection data from the *BBR* mixture test. Deflection data obtained during the *BBR* test is used to compute two test parameters: 1) mixture stiffness as a function of time; and 2) instantaneous slope of the mixture stiffness curve (m-value). Values of each parameter are calculated at eight discrete loading times over the period of the test. The time points are 8, 15, 30, 60, 120, 240, and 960 seconds.



a) Test Fixture with Specimen *b) Example Deflection Data*

Figure 4.3 *BBR* Mixture Testing

4.2.4 Cantabro Durability

The Cantabro abrasion loss test procedure used in this study is described as follows. Standard 150 mm diameter gyratory compacted specimens of nominal 115 mm height were compacted of each mixture. Initially, the design compactive effort was utilized for all Cantabro specimens since the goal of this line of testing was to develop a test that could be performed in day to day operations with the types of specimens that are already being made for measurement of volumetric properties. Based on the test data, compaction of specimens to a target density of 4.0 ± 0.5% air voids was also performed for select highway surface mixtures and 100% RAP; details are discussed in Section 4.3.

The level of replication varied slightly between different components of this dissertation. Initially, five replicate specimens were tested for control highway surface mixtures. Analysis of test method variability conducted as part highway mixture component of the experimental program led to a reduction in the level of replication to three replicate specimens per mixture for all other testing.

Prior to testing, specimens were conditioned in an environmental chamber overnight (minimum of twelve hours) at 25 C. A dummy specimen of similar properties with an embedded thermocouple was exposed to the same conditions to verify that the internal temperature of all samples equilibrated to 25 C. The temperature of the LA Abrasion drum was checked before every test and was required to be 25 ± 2 C before testing. A specimen was placed in the drum of an LA Abrasion testing machine without the charge of steel spheres and subjected to 300 revolutions. The mass of the specimen was recorded before and after the test and the loss in specimen mass as a percentage (*ML*) during the test was reported as a percentage of the original mass. All debris leftover from the previous test was removed from the LA Abrasion drum before each test to ensure that there was no variability introduced to the results due to cushioning of the test sample. Figure 4.4 allows for a visual comparison of tested Cantabro specimens with varying binder contents and subsequently varying levels of mass loss (*ML*).

A limited amount of highway surface mixture testing was included in the experimental program to evaluate the effects of laboratory conditioning methods on *ML*. Two laboratory conditioning protocols were selected: 1) the long term conditioning protocol for compacted test specimens of *AASHTO R-30*; and 2) the conditioning protocol specified in Mississippi test method *MT-85*. The *R-30* protocol was 120 ± 0.5 hours (5 days) in a forced draft oven set to 85 ± 3 C. The *MT-85* protocol was 168 hours (7 days) in a forced draft oven set to 64 C. The *MT-85* test method does not specify time and temperature tolerances, so the tolerances from *R-30* were utilized. Specimens were subjected to the desired conditioning protocol then allowed to cool overnight before testing for *ML* according to the same procedure as was used for the un-aged specimens.

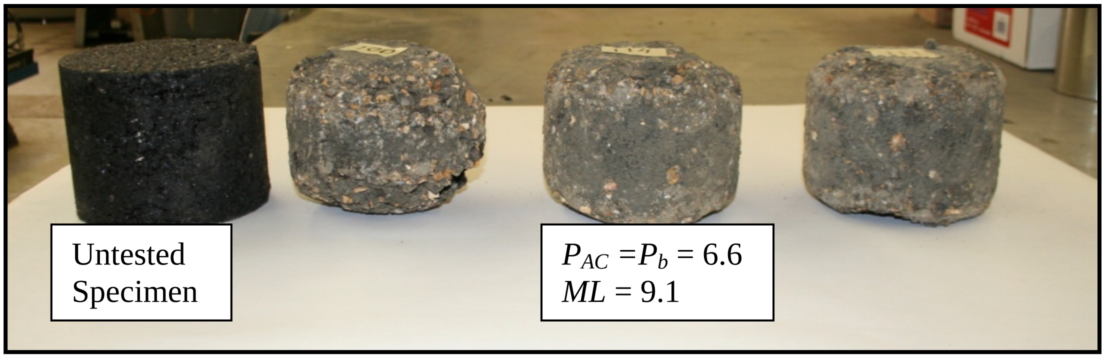


Figure 4.4 Tested Cantabro Specimens with Varying Total Asphalt Content (P_{AC})

4.2.5 Moisture Damage (*TSR*)

Moisture damage testing was performed according to *ASTM D 4867* on 62.5 mm tall by 100 mm diameter gyratory compacted specimens. In accordance with *ASTM D 4867*, target air voids of specimens compacted for moisture damage testing was 7 ± 1% as measured by *ASTM D 2726* (submerged specimen method). A freeze-thaw cycle was not performed as part of the conditioning process. As previously mentioned in Section 3.4, virgin aggregate batches were mixed with approximately 2% water prior to heating to ensure coating of the aggregate by the hydrated lime.

4.2.6 Rutting (*APA*)

For *APA* rut resistance testing, a test temperature of 64 C was used according to MDOT recommendations. The wheel load was 445 N (100 lb) and the hose pressure was 690 kPa (100 psi) according to *AASHTO TP 63*. The wheel load and hose pressure were verified once per day and adjusted if necessary. Automatic measurement of rut depths was used for all data. Specimens were preconditioned at the test temperature prior to testing for a minimum of 6 hours but not more than 24 hours.

APA specimens were created by compaction to a target height and density in the *SGC* as described in Section 3.4.3. The target air voids were 7 ± 0.5% or 10 ± 1.0% and the target height was 75 ± 5 mm. The purpose of testing two different air void levels for *APA* rutting was to evaluate rutting rate of the mixtures independent of the air void level.

4.2.7 PURWheel

PURWheel testing was performed on specimens created by sawing *LAC* compacted slab specimens in half; specimens were approximately 29 cm wide and 31 cm long. Two PURWheel specimens corresponding to the halves of a compacted slab were tested at the same time in the left and right tracks of the PURWheel to be a single replicate PURWheel test. The basic features and test parameters of the PURWheel in use at MSU are given here, additional details can be found in Howard et al. (2010).

Test specimens are grouted in place with Plaster of Paris during testing. The test temperature for the PURWheel is 64 C (same test temperature as the *APA*). Once the test chamber reaches the target temperature, the specimens are conditioned for six hours, not to exceed 24 hours, to ensure the specimen reaches thermal equilibrium. Two independently controlled wheel carriages mounted with 4-ply pneumatic tires are used to load the specimens during the test. The tire inflation pressure is 862 kPa and the wheel load is 178.6 kg, resulting in a gross contact pressure at the beginning of the test of approximately 630 kPa. The travel speed of the wheel over the specimen during testing is 33 cm/sec. A full test consists of 20,000 passes of the wheel over the test specimen or a cumulative rut depth of 15 mm measured by the software (corresponds to a physical specimen deformation of 23 mm), whichever comes first. Eq. 4.2 is the correlation equation used to convert rut depths measured by the LVDT's on the PURWheel to rut depths that would be measured by manual methods.

$$R_M = R_{Adj.} = 0.0153(R_T)^2 + 1.3R_T \qquad (R^2 = 0.96) \qquad \text{(Eq 4.2)}$$

Where:

R_M = total rut depth measured manually (mm)

$R_{Adj.}$ = adjusted rut depth accounting for LVDT & manual measurement difference (mm)

R_T = rut depth measured by PURWheel LVDT's (mm)

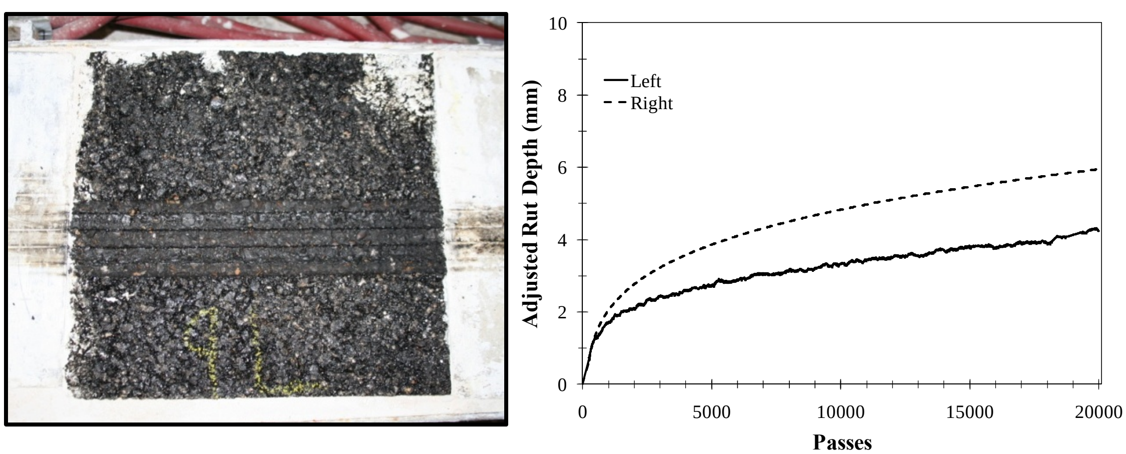
During testing the rut depth of the test specimen is measured over the central 20 cm and recorded by the PURWheel control software. In addition to the electronic measurement of specimen rut depth during the test, manual measurements of the final specimen rut depth are also recorded for each PURWheel test.

Air voids of slabs tested in the PURWheel were on the order of 8 to 10% on a Corelok® (*T 331*) basis or 6.8 to 8.3% on a submerged (*T 166*) basis for most mixtures though the voids varied somewhat in a few cases. The compactive effort of all slabs was constant at 18 passes and 2413 kPa hydraulic system pressure. The void levels experienced by most slabs is in line with the Table 2.3 DOT specifications where average target, full pay maximum, and removal void levels were approximately 7, 8, and 10% measured via *T 166*.

A maximum air void criteria for slabs was established as 10% measured via *T 166* or 12.3% via *T 331* (Eq. 2.3 used to correlate *T 166* to *T 331*). This criteria is in line with the average air void level warranting removal in the southeast US. This criteria allows air voids to be in the range stated by Terrel and Al-Swailmi (1993) to be favorable to moisture damage (7 to 11%). Testing slabs at air void levels representing the higher end of permissible values is more indicative of cases likely to be susceptible to moisture damage in service. If a mixture performs adequately at the upper end of allowable voids, it should in turn perform well at the lower end of allowable voids with the same aggregate blend and asphalt content.

4.2.7.1 PURWheel Dry Protocol

Slabs were tested in a dry condition at 64 C for rutting evaluation. An example of a tested slab from the PURWheel dry protocol test is seen in Figure 4.5*a*. An example set of test data for a PURWheel dry protocol test is seen in Figure 4.5*b*, note the smooth progression of rutting in both left and right test specimens and that Eq. 4.2 was used to determine adjusted rut depths.

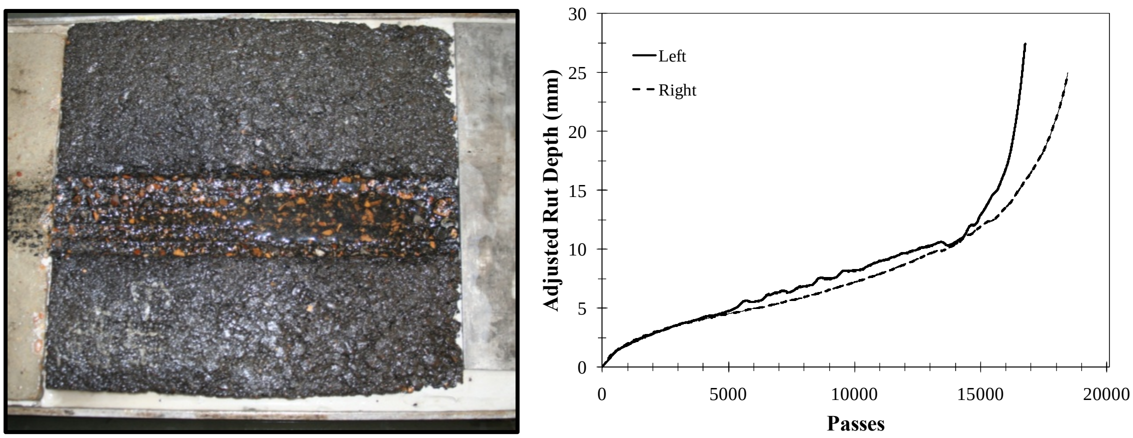


a) Tested PURWheel Dry Test Specimen *b) Example PURWheel Data for Dry Test*

Figure 4.5 PURWheel Dry Protocol Rutting Test

4.2.7.2 PURWheel Wet Protocol

For combined asphalt mixture rutting and moisture damage evaluation using the PURWheel, test specimens were tested underwater at 64 C after 6 hr of conditioning. An example of a tested PURWheel specimen for moisture damage evaluation is seen in Figure 4.6*a*, note the loss of aggregate coating in the wheel path. An example set of test data for a PURWheel wet protocol test is seen in Figure 4.6*b*, note the quick progression of damage and early failure of the test specimens. Early failure occurred more frequently using wet testing but did not occur in all instances.



a) Tested PURWheel Wet Specimen *b) Example PURWheel Data for Wet Test*

Figure 4.6 PURWheel Wet Protocol Moisture Damage Test

4.3 Experimental Designs

4.3.1 100% RAP Experiments

Testing was performed on 100% RAP mixtures with additional virgin asphalt to better understand the effects of RAP when used in recycled mixtures; this was performed in an attempt to reduce the number of variables involved by removing virgin aggregate effects. The purpose was not to design or assess 100% RAP mixtures for use in paving.

From an experimental standpoint, recycled mixtures have four major factors or variables: 1) aggregate contributed from RAP; 2) virgin aggregate; 3) aged asphalt bitumen contributed from RAP; and 4) virgin asphalt. A statistical analysis of an experiment containing four variables has eleven interactions between variables that must be checked and either eliminated or included in the analysis before the four major variables of interest can be evaluated. In the case of a statistical analysis of three major factors there are only four interactions to be checked before the main effects can be evaluated. In the case of a statistical analysis of two major factors there is only one interaction to be checked before main effects can be evaluated. While effects of the four factors concerning recycled mixtures can never be fully isolated and measured separately, the number of total variables can be lowered in an attempt to reduce the overall complexity of the problem. The purpose of this work was to allow for a better fundamental understanding of recycled asphalt mixtures.

Three 100% RAP mixtures were designed with properties were given in Section 3.5.1. The experimental testing performed for 100% RAP mixtures is given in Table 4.1; the values in each row for a mixture indicate the number of replicates tested for each test type and condition. Performance was evaluated in four categories; 1) rutting (*APA* and dry PURWheel); 2) durability (Cantabro); 3) non-load associated cracking (*BBR* mixture test and IDT); and 4) moisture damage (*TSR* and wet PURWheel). The data from 100% RAP testing was analyzed to determine if the performance of 100% RAP mixtures can be used to estimate the performance of a recycled mixture containing a percentage of the same RAP source (e.g. 50% RAP mixture). Results from testing 100% RAP mixtures were used to guide the investigation of high RAP recycled mixtures for airfield and highway applications.

Additionally, four experiments were performed to evaluate the effects on RAP of heating, compaction variables and absorption of asphalt by RAP aggregate. The experiments are described in the following subsections. Analysis of all 100% RAP data is provided in Chapter 5.

Table 4.1 Investigation of 100% RAP Mixtures Compacted with *SGC* Designed Asphalt Content from Table 3.5

Compaction			**Mixture**		
Type	**Test Parameters**		***9.5-100/RM-1***	***9.5-100/RM-2***	***12.5-100/RM-3***
SGC	***APA***	**7% V_a**	2	2	2
	APA	**10% V_a**	2	2	2
	TSR	**7% V_a**	1	1	1
	Cantabro	**Un-aged**	3	3	3
	BBR	**-06 C**	1	1	1
	BBR	**-12 C**	1	1	1
	BBR	**-18 C**	1	1	1
	BBR	**-24 C**	1	1	1
	IDT	**-06 C**	2	2	0
	IDT	**-12 C**	2	2	0
	IDT	**-18 C**	2	2	0
	IDT	**-24 C**	2	2	0
	IDT	**+25 C**	2	2	0
LAC	**PURWheel**	**Dry**	2	2	2
	PURWheel	**Wet**	2	2	2

APA A replicate consisted of a single track in the test equipment composed of two 150 mm diameter specimens.

TSR A replicate consisted of six specimens tested according to *ASTM D 4867.*

Cantabro A replicate consisted of one *SGC* (65 gyrations) compacted specimen.

BBR A replicate consisted of five mixture beams at one temperature from an *SGC* specimen.

IDT A replicate consisted of one 100 mm diameter specimen.

PURWheel A replicate consisted of two specimens cut from a single *LAC* compacted slab and tested side by side at the same time.

4.3.1.1 RAP Relative Heating Experiment

A small experiment was performed with the *9.5-100/RM-1* mixture to investigate the effects of relative heating time on RAP compaction behavior. The RAP was placed in pre-heated steel trays in a layer approximately 5 cm thick and heated in a forced draft oven at 177 C for 15 minutes. The goal was to approximate, albeit crudely, the relatively short period of heating at high temperature that RAP experiences in an asphalt plant during production. The RAP was then removed and mixed with the appropriate amount of virgin binder in the standard manner before being placed in pans for short term aging at 146 C. Five different short term aging times were utilized: 60, 90, 180, 360, and 1440 minutes. At the conclusion of short term aging, specimens were compacted in the *SGC* with the design compactive effort (65 gyrations). Three replicate specimens were produced for each short term aging time and were tested with the Cantabro test. The experimental was then repeated with three more replicates compacted to target 4.0 ± 0.5% air voids. Two G_{mm} replicates were produced as part of this experimental program to evaluate long term asphalt absorption potential of RAP mixture, one with 60 minutes of aging and one with 1440 minutes of aging.

4.3.1.2 RAP Compaction Experiment

To evaluate mixture parameter effects on the compaction behavior of RAP and virgin binder in the absence of virgin aggregate, additional 100% RAP testing was conducted with an expanded number of experimental variables and a reduced level of replication. To accomplish this, a 3^4 factorial designed experiment was created for each RAP source encompassing factors of compaction temperature, compactive effort, warm mix additives, and amount of virgin binder. The data was utilized to investigate quantities of inert and effective RAP asphalt in Chapter 5. Based on results of the testing performed with the *R-1* RAP source at compaction temperatures of 116 and 138 C, the number of factors examined and the amount of testing performed was greatly reduced for the other RAP sources and the 154 C compaction temperature with *R-1* RAP. In all, nearly 400 specimens of 100% RAP with virgin binder were compacted in this portion of the research.

The amount of virgin binder added and total asphalt contents selected for use in this part of the experimental program were based on preliminary work in Howard et al. (2009). Three levels of added virgin binder (low, medium, and high) were investigated corresponding to on the order of 0.5, 1.5, and 2.5% virgin binder. The true amount of virgin binder, RAP asphalt, and total asphalt (P_{AC}) is shown in Table 4.2 on a mix mass basis. The concept was to have three different virgin binder contents that would encompass 4% air voids when compacted.

Table 4.3 is an experimental design detailing the specific factor-level combinations that were tested. Performance testing was performed for specimens from some of the factor-level combinations; however the data is not part of this dissertation. The performance data can be found in Doyle and Howard (2010b).

Table 4.2 Asphalt Contents For 100% RAP at Varying Conditions

RAP Source	Total Asphalt Content (P_{AC}) Range	(%)	$P_{be(V)}$ (%)	$P_{b(R)}$ (%)
R-1	High	8.1	2.8	5.3
	Med	7.1	1.7	5.4
	Low	6.0	0.6	5.4
R-2	High	8.2	2.7	5.5
	Med	7.2	1.7	5.5
	Low	6.2	0.6	5.6
R-3	High	7.4	2.5	4.9
	Med	6.4	1.5	4.9
	Low	5.5	0.5	5.0

Table 4.3 Experimental Design of 100% RAP Mixtures Compacted at Varying Conditions

			Warm Mix Additive and Total Asphalt Content[Table 4.2]								
RAP	Comp		None			Evotherm™ 3G 0.5%			Sasobit® 1.0%		
Source	Temp (C)	N_{des}	High	Med	Low	High	Med	Low	High	Med	Low
R-1	116	50	X	X	X	X	X	X	X	X	X
	116	65	X	X	X	X	X	X	X	X	X
	116	85	X	X	X	X	X	X	X	X	X
	138	50	X	X	X	X	X	X	X	X	X
	138	65	X	X	X	X	X	X	X	X	X
	138	85	X	X	X	X	X	X	X	X	X
	154	65	X	X	X	X	X	X	X	X	X
R-2	116	65	X	X	X	X	X	X	NT[a]	NT[a]	NT[a]
	138	65	X	X	X	X	X	X	X	X	X
	154	65	X	X	X	X	X	X	NT[a]	NT[a]	NT[a]
R-3	116	65	X	X	X	X	X	X	X	X	X
	138	65	X	X	X	X	X	X	X	X	X
	154	65	X	X	X	X	X	X	X	X	X

a) Not tested due to insufficient material.

4.3.1.3 RAP Absorbed Asphalt Experiment 1

To investigate the potential for additional absorption of asphalt by RAP aggregate an experiment was performed that consisted of two factors: 1) additional virgin asphalt content (high, medium, and low); and 2) RAP heating and compaction temperature (116 and 138 C). The factors and levels were the same as in Table 4.3 and 4.4; three G_{mm} replicates were prepared of each factor level combination. The first replicate was only virgin binder, the second replicate contained Sasobit® and the third replicate contained Evotherm™ 3G. Based on results of this experiment with the *R-1* source, only the factor of additional virgin asphalt content was examined for the *R-2* and *R-3* sources.

4.3.1.4 RAP Absorbed Asphalt Experiment 2

Another experiment was performed on the *R-1* and *R-3* RAP sources to investigate absorption of asphalt by RAP aggregate; it consisted of testing four G_{mm} replicates from two samples of RAP. The first sample of RAP was split; one half was used to determine G_{mm} and the other half was heated for 120 minutes at 171 C then cooled and used to determine G_{mm}. The second sample of RAP was heated for 120 minutes at 171 C then mixed with 2% additional virgin binder. The second sample was split; one half was immediately cooled and the other half was placed in an oven at the hot mix compaction temperature (146 C) for four hours before it was removed and cooled. G_{mm} was determined for each half of the second sample. A four hour short term age was chosen as being conducive to producing a maximum potential for asphalt absorption; hot mix temperatures were chosen in favor of warm mix temperatures as they are more favorable to asphalt absorption.

4.3.2 Airfield Surface Mixtures

Current practice airfield surface mixtures typically are not allowed to contain RAP; however up to 30% RAP is allowed in shoulder and intermediated layer (binder and base) mixes (USACE 2010). WMA has not yet seen wide use in airfield mixtures. This component of the experimental program was developed to investigate performance issues related to high RAP-WMA airfield surface mixtures. Table 4.4 summarizes the experimental design for this component of the experimental program; it included factors of aggregate type, RAP content (all *R-1* RAP source) and mixture type. All factor-level combinations of the factorial experiment were tested, encompassing twenty-four asphalt mixtures. The mixture identification nomenclature for airfield surface mixtures is given in Table 4.5. Properties of the airfield surface mixtures were given in Section 3.5.2.

Four performance characteristics were evaluated for all airfield surface mixtures: 1) permanent deformation; 2) durability; 3) non-load associated cracking; and 4) moisture damage. Performance testing details are given in Table 4.5 and were as follows. For permanent deformation (rutting resistance) assessment, the *APA* test was performed. For durability performance the Cantabro test was performed (denoted by *ML* in Table 4.5). The *BBR* mixture test was performed for non-load associated cracking assessment; four beam specimens for each temperature. Moisture damages assessment was performed with the *TSR* test. In addition, binder testing was performed for airfield mixtures 1 to 12.

Table 4.4 Airfield Surface Mixtures Experimental Design

	Virgin Aggregate Type and RAP Content					
	12.5 mm NMAS Limestone			12.5 mm NMAS Crushed Gravel		
Mixture Type	**0% RAP**	**25% RAP**	**50% RAP**	**0% RAP**	**25% RAP**	**50% RAP**
HMA	X	X	X	X	X	X
WMA-Evotherm	X	X	X	X	X	X
WMA-Sasobit	X	X	X	X	X	X
WMA-Foam	X	X	X	X	X	X

Table 4.5 Airfield Surface Mixtures Nomenclature and Performance Testing

Mixture ID	Aggregate Type and Gradation	RAP (%)	Mix Type (---)	*APA* (---)	*ML* (---)	*BBR* -06 C	-12 C	*TSR* (---)
12.5-0/AM-1	Limestone	0	HMA	3	3	2	2	1
12.5-0/AM-2	(LS-1)		Sasobit®	3	3	2	2	1
12.5-0/AM-3			Evotherm™ 3G	3	3	2	2	1
12.5-0/AM-4			Foam	3	3	2	2	1
12.5-25/AM-5	Limestone	25	HMA	3	3	2	2	1
12.5-25/AM-6	(LS-2)		Sasobit®	3	3	2	2	1
12.5-25/AM-7			Evotherm™ 3G	3	3	2	2	1
12.5-25/AM-8			Foam	3	3	2	2	1
12.5-50/AM-9	Limestone	50	HMA	3	3	2	2	1
12.5-50/AM-10	(LS-3)		Sasobit®	3	3	2	2	1
12.5-50/AM-11			Evotherm™ 3G	3	3	2	2	1
12.5-50/AM-12			Foam	3	3	2	2	1
12.5-0/AM-13	Crushed Gravel	0	HMA	3	3	2	2	1
12.5-0/AM-14	(GR-1)		Sasobit®	3	3	2	2	1
12.5-0/AM-15			Evotherm™ 3G	3	3	2	2	1
12.5-0/AM-16			Foam	3	3	2	2	1
12.5-25/AM-17	Crushed Gravel	25	HMA	3	3	2	2	1
12.5-25/AM-18	(GR-2)		Sasobit®	3	3	2	2	1
12.5-25/AM-19			Evotherm™ 3G	3	3	2	2	1
12.5-25/AM-20			Foam	3	3	2	2	1
12.5-50/AM-21	Crushed Gravel	50	HMA	3	3	2	2	1
12.5-50/AM-22	(GR-3)		Sasobit®	3	3	2	2	1
12.5-50/AM-23			Evotherm™ 3G	3	3	2	2	1
12.5-50/AM-24			Foam	3	3	2	2	1

APA A replicate consisted of a single track in the test equipment composed of two 150 mm diameter specimens.

Cantabro A replicate consisted of one *SGC* compacted (75 gyrations) un-aged specimen.

BBR A replicate consisted of 2 mixture beams at one temperature from an *SGC* specimen.

TSR A replicate consisted of six specimens tested according to *ASTM D 4867.*

4.3.3 Highway Surface Mixtures

The purpose of this component of the experimental program was to investigate the feasibility of using 25 to 50% RAP in WMA highway surface mixtures. All mixtures studied were 9.5 mm NMAS and met MDOT requirements for surface mixtures. Control mixtures used in this part of the experimental program were MDOT approved mixtures from current practice. Current practice mixtures, especially plant-mixed material, provide the most realistic reference for the mixes under investigation. Since no previous data or experience with high RAP-WMA in Mississippi was available, comparison to currently acceptable mixes was appropriate. The following subsections describe testing of the control mixtures and the high RAP-WMA mixtures.

4.3.3.1 Control Mixtures

A decision was made to use current practice MDOT approved mixtures as the primary control; the majority of 9.5 mm MDOT approved surface mixtures contain 10 to 15% RAP. Using current practice mixtures containing 15% RAP for performance comparisons instead of producing 0% RAP mixtures in the laboratory was a pragmatic compromise between the experimental rigor of a 0% RAP control mixture and the realism of current practice 15% RAP mixtures for control comparison.

Generally speaking, the relative performance boundaries of asphalt mixtures in Mississippi are represented by 50 design gyration and 85 gyration mixtures. 50 gyration mixtures have the highest effective asphalt content for a particular gradation; this results in a flexible pavement that is resistant to cracking but also results in a pavement that can be susceptible to rutting under heavy traffic. 85 gyration mixtures have the lowest effective asphalt content for a particular gradation; this results in stiff pavements that are resistant to rutting but can also result in an increased potential for cracking. The goal when selecting control mixtures was to encompass the range of potential cracking and rutting resistance of current practice mixtures to the best extent possible. This allowed evaluation of the recycled mixtures in terms of the range of current practice. Properties of the control highway surface mixtures were given in Section 3.5.3.1

The experimental testing performed on control highway surface mixtures as part of this component of the experimental program is provided in Table 4.6; the values in each row for a mixture indicate the number of replicates tested for each test type and condition. Performance was evaluated in four categories; 1) rutting (*APA* and dry PURWheel); 2) durability (Cantabro); 3) non-load associated cracking (*BBR* mixture test and IDT); and 4) moisture damage (*TSR* and wet PURWheel). For PURWheel testing, four tests (two wet and two dry) were performed on each plant mixed material and two tests (one wet and one dry) were performed on each of the laboratory mixed versions of the *9.5-15/CM-4* control mixture.

BBR testing of each *SGC* compacted specimen produced one replicate of five beams at each of the four test temperatures. At any temperature one replicate was the average of five beams. The baseline replication level was two, which required two *SGC* specimens. Additional replicates were tested for control mixes 1, 4*a* and 4*b* to evaluate variability.

Table 4.6 Performance Testing and Replication for 9.5 mm NMAS Control Highway Surface Mixtures

Compaction Type	Test Parameters		Mixture 9.5-0/ CM-1	9.5-15/ CM-2	9.5-15/ CM-3	9.5-15/ CM-4a	9.5-15/ CM-4b	9.5-15/ CM-4c	9.5-15/ CM-5 to 28
SGC	***APA***	**7% V_a**	0	3	3	3	3	2	0
	APA	**10% V_a**	0	3	3	3	0	0	0
	TSR	**7% V_a**	1	1 MDOT	1 MDOT	1 MDOT	0	0	0
	Cantabro	**Un-aged, N_{des}**	5	5	5	5	5	3	116 Table 4.7
	Cantabro	**Un-aged, 4% V_a**	0	0	3	0	0	0	30 Table 4.7
	Cantabro	***R-30*, N_{des}**	0	3	3	0	0	0	30 Table 4.7
	Cantabro	***MT-85*, N_{des}**	0	3	3	0	0	0	3 Table 4.7
	BBR	**-06 C**	3	2	2	4	3	2	0
	BBR	**-12 C**	3	2	2	4	3	2	0
	BBR	**-18 C**	3	2	2	4	3	2	0
	BBR	**-24 C**	3	2	2	4	3	2	0
	IDT	**-06 C**	0	2	2	0	0	0	0
	IDT	**-12 C**	0	2	2	0	0	0	0
	IDT	**-18 C**	0	2	2	0	0	0	0
	IDT	**-24 C**	0	2	2	0	0	0	0
LAC	**PURWheel**	**Dry**	0	2	2	2	1	1	0
	PURWheel	**Wet**	0	2	2	2	1	1	0

APA A replicate consisted of a single track in the test equipment composed of two 150 mm diameter specimens.
TSR A replicate consisted of six specimens tested according to *ASTM D 4867* or MDOT mix design information.
Cantabro A replicate consisted of one *SGC* compacted specimen.
BBR A replicate consisted of five mixture beams at one temperature from an *SGC* specimen.
IDT A replicate consisted of one 100 mm diameter specimen.
PURWheel A replicate consisted of two specimens cut from a single *LAC* compacted slab and tested side by side at the same time.

For the Cantabro durability test, additional data was required to establish both the expected variability of the test method and a range of test results representative of current practice Mississippi mixtures. To achieve this goal, quality control (QC) specimens of plant produced mixtures were obtained from a local asphalt plant and tested at MSU. Additionally, quality assurance (QA) specimens of a range of asphalt mixture types from around the state prepared at the MDOT central materials laboratory were tested at MDOT. Details of the testing performed with these mixtures are given in Table 4.7.

To measure test method variability, plant mixed QC specimens of two mixtures were obtained and tested (*9.5-15/CM-5* and *9.5-15/CM-6*). Thirty un-aged specimens each of *9.5-15/CM-5* and *9.5-15/CM-6* were tested to establish variability of the test method. Based on the investigation of test variability, the baseline number of replicates for Cantabro testing was reduced from five to three for all additional testing (the only control mixture affected was *9.5-15/CM-4c*).

For mixture *9.5-15/CM-5*, three aged Cantabro specimens were tested per aging protocol. Based on the results, data for the aging protocol resulting in the greatest *ML* increase (*R-30*) was then supplemented to reach 30 total replicates. The data was used to evaluate variability of aged specimens and to establish a baseline of aged test results.

To evaluate the range of expected performance of typical Mississippi mixtures with respect to un-aged Cantabro results, QA specimens were tested. Twenty-two mixes were compacted and tested at the MDOT central laboratory, and properties of these mixtures are given in Table 4.7 (control mixtures 7 to 28). Two replicates were typically tested per mix based on availability, though in a few cases multiple sets of the same mix were available from different projects and were tested resulting in four to eight replicates.

Based on results of the Cantabro testing with specimens compacted with design compactive effort, additional investigation was performed of specimens compacted to target 4% air voids for select mixtures. Three specimens of mixture *9.5-15/CM-3* were compacted to target voids. Thirty specimens of mixture *9.5-15/CM-6* were compacted to target air voids to assess variability.

Table 4.7 Replication Details for Cantabro Plant Mixed Control Mixtures 5 to 28

	Cantabro Specimens Tested			
Mixture ID	**Un-aged, N_{des}**	**Un-aged, 4% V_a**	***R-30*, N_{des}**	***MT-85*, N_{des}**
9.5-15/CM-5	30	0	30	3
9.5-15/CM-6	30	30	0	0
9.5-15/CM-7	2	0	0	0
9.5-15/CM-8	2	0	0	0
9.5-10/CM-9	4	0	0	0
9.5-15/CM-10	2	0	0	0
9.5-15/CM-11	4	0	0	0
9.5-15/CM-12	2	0	0	0
9.5-15/CM-13	2	0	0	0
9.5-15/CM-14	2	0	0	0
9.5-15/CM-15	2	0	0	0
9.5-15/CM-16	2	0	0	0
9.5-15/CM-17	2	0	0	0
9.5-15/CM-18	2	0	0	0
9.5-15/CM-19	2	0	0	0
9.5-15/CM-20	2	0	0	0
9.5-10/CM-21	2	0	0	0
9.5-15/CM-22	2	0	0	0
9.5-0/CM-23	2	0	0	0
9.5-10/CM-24	2	0	0	0
9.5-10/CM-25	2	0	0	0
9.5-6/CM-26	8	0	0	0
9.5-10/CM-27	4	0	0	0
9.5-10/CM-28	2	0	0	0

4.3.3.2 25 and 50% RAP Mixtures

To investigate performance of 25 to 50% RAP in WMA for highway surface applications, an experimental design was created to include factors of RAP content and RAP source. Based on results of 100% RAP testing, the *R-3* RAP source was not included. The experimental design and mixture identification is provided in Table 4.8. All the mixtures were 9.5 mm NMAS, designed with 65 gyrations compactive effort, and contained Sasobit®. Properties of the mixtures were given in Section 3.5.3.2.

Table 4.8 Highway Surface Mixtures Experimental Design

	RAP Source	
RAP Content	***R-1***	***R-2***
25%	*9.5-25/RM-1*	*9.5-25/RM-2*
50%	*9.5-50/RM-1*	*9.5-50/RM-2*

For the recycled mixtures, a suite of testing was performed as detailed in Table 4.9; the values in each row for a mixture indicate the number of replicates tested for each test type and condition. Performance was evaluated in four categories; 1) rutting (*APA* and dry PURWheel); 2) durability (Cantabro); 3) non-load associated cracking (*BBR* mixture test and IDT); and 4) moisture damage (*TSR* and wet PURWheel). Cantabro testing of specimens to target air voids was performed for all mixtures. Cantabro testing of aged specimens was only performed for mixtures containing RAP source *R-1*.

Table 4.9 Performance Testing and Replication of 25 and 50% RAP Recycled Mixtures

Compaction Type	**Test Parameters**		**Mixture** *9.5-25/ RM-1*	*9.5-25/ RM-2*	*9.5-50/ RM-1*	*9.5-50/ RM-2*
SGC	***APA***	**7% V_a**	2	2	2	2
	APA	**10% V_a**	2	2	2	2
	TSR	**7% V_a**	1	1	1	1
	Cantabro	**Un-aged, N_{des}**	3	3	3	3
	Cantabro	**Un-aged, 4% V_a**	3	3	3	3
	Cantabro	***R-30*, N_{des}**	3	0	3	0
	BBR	**-06 C**	2	2	2	2
	BBR	**-12 C**	2	2	2	2
	BBR	**-18 C**	2	2	2	2
	BBR	**-24 C**	2	2	2	2
	IDT	**-06 C**	2	2	2	2
	IDT	**-12 C**	2	2	2	2
	IDT	**-18 C**	2	2	2	2
	IDT	**-24 C**	2	2	2	2
LAC	**PURWheel**	**Dry**	2	2	2	2
	PURWheel	**Wet**	2	2	2	2

APA	A replicate consisted of a single track in the test equipment composed of two 150 mm diameter specimens.
TSR	A replicate consisted of six specimens tested according to *ASTM D 4867*.
Cantabro	A replicate consisted of one *SGC* compacted specimen.
BBR	A replicate consisted of five mixture beams at one temperature from an *SGC* specimen.
IDT	A replicate consisted of one 100 mm diameter specimen.
PURWheel	A replicate consisted of two specimens cut from a single *LAC* compacted slab and tested side by side at the same time.

4.3.4 Highway Base Mixtures

The purpose of this experimental program component was to investigate feasibility of using 50 to 75% RAP in WMA highway bases. In keeping with the philosophy for highway surface mixtures, control mixes were MDOT approved and from current practice. Control mixtures studied were 12.5 mm or 19.0 mm NMAS and met MDOT base requirements. High RAP mixtures studied were 12.5 mm NMAS. The following subsections describe testing of the control mixes and high RAP-WMA mixes.

4.3.4.1 Control Mixtures

Properties of the control highway base mixtures were given in Section 3.5.4.1. The four main control mixtures all contained 15% RAP. The experimental testing performed on control highway base mixtures as part of this component of the experimental program is provided in Table 4.10; the values in each row for a mixture indicate the number of replicates tested for each test type and condition. Performance was evaluated in four categories; 1) rutting (*APA* and dry PURWheel); 2) durability (Cantabro); 3) tensile strength (IDT); and 4) moisture damage (*TSR* and wet PURWheel).

For PURWheel testing a common test temperature was utilized throughout this study to provide a relative comparison of properties for high RAP mixtures. However, another approach with merit in loaded wheel tracker testing is to adjust the test temperature based on anticipated temperature at desired location within the pavement structure (i.e. lower test temperature for base layers). Other researchers have successfully taken that approach (e.g. Nielson 2010).

Additional Cantabro testing was performed to establish a range of test results representative of current practice Mississippi mixtures. Plant produced QA specimens of a range of asphalt mixture types from around the state prepared at the MDOT central materials laboratory were tested at MDOT. Details of the testing performed with these mixtures are given in Table 4.11.

Table 4.10 Performance Testing and Replication for 12.5 and 19.0 mm NMAS Control Highway Base Mixtures

Compaction Type	Test Parameters		Mixture 12.5-15/ CM-1	12.5-15/ CM-2	12.5-15/ CM-3	19.0-15/ CM-4	12.5-15/ CM-5 to CM-20	19.0-15/ CM-21 to CM-37
SGC	***APA***	**7% V_a**	3	3	1	3	0	0
	APA	**10% V_a**	3	3	1	3	0	0
	TSR	**7% V_a**	1[MDOT]	1[MDOT]	1[MDOT]	1[MDOT]	0	0
	Cantabro	**Un-aged, N_{des}**	3	3	3	3	54[Table 4.11]	42[Table 4.12]
	IDT	**25 C**	2	2	2	2	0	0
LAC	**PURWheel**	**Dry**	2	1	2	2	0	0
	PURWheel	**Wet**	2	1	3	2	0	0

APA A replicate consisted of a single track in the test equipment composed of two 150 mm diameter specimens.
TSR A replicate consisted of six specimens tested according to *ASTM D 4867* or MDOT mix design information.
Cantabro A replicate consisted of one *SGC* compacted (design compactive effort) specimen.
IDT A replicate consisted of one *SGC* compacted (design compactive effort) specimen.
PURWheel A replicate consisted of two specimens cut from a single *LAC* compacted slab and tested side by side at the same time.

Table 4.11 Replication Details for Cantabro Plant Mixed 12.5 mm and 19.0 mm NMAS Control Mixtures 5 to 37

	Cantabro Specimens Tested
Mixture ID	**Un-aged, N_{des}**
12.5-12/CM-5	2
12.5-15/CM-6	2
12.5-20/CM-7	2
12.5-14/CM-8	2
12.5-15/CM-9	8
12.5-15/CM-10	2
12.5-15/CM-11	10
12.5-12/CM-12	4
12.5-15/CM-13	2
12.5-15/CM-14	4
12.5-15/CM-15	2
12.5-30/CM-16	2
12.5-12/CM-17	4
12.5-15/CM-18	2
12.5-15/CM-19	2
12.5-15/CM-20	4
19.0-15/CM-21	2
19.0-15/CM-22	2
19.0-20/CM-23	2
19.0-20/CM-24	2
19.0-20/CM-25	2
19.0-12/CM-26	2
19.0-20/CM-27	2
19.0-18/CM-28	2
19.0-25/CM-29	6
19.0-15/CM-30	2
19.0-30/CM-31	4
19.0-15/CM-32	4
19.0-10/CM-33	2
19.0-20/CM-34	2
19.0-15/CM-35	2
19.0-20/CM-36	2
19.0-15/CM-37	2

4.3.4.2 50 and 75% RAP Mixtures

Initially, an experimental design was created to investigate the volumetric properties of high RAP-WMA for base mixtures. Factors of RAP source, RAP content and warm mix additive dosage rate were investigated. The experimental design encompassing twelve factor-level combinations is provided in Table 4.12. All the mixtures were 12.5 mm NMAS, designed with 50 gyrations compactive effort, and mixing and compaction temperatures were 116 C. Optimum design asphalt contents were estimated for each experimental treatment as well as tensile strengths with IDT test at 25 C. The results were used to guide additional testing and the only four of the mixtures were selected for performance testing as discussed in the following paragraphs.

To investigate performance of 50 to 75% RAP in WMA for highway surface applications, an experimental design was created to include factors of RAP content and RAP source; the *R-3* RAP source was not included. The experimental design and mixture identification is provided in Table 4.13. All the mixtures were 12.5 mm NMAS, designed with 50 gyrations compactive effort, and contained 1.0% Sasobit®. Properties of the mixtures were given in Section 3.5.4.2.

Table 4.12 Highway Base Mixtures Volumetric Experimental Design

RAP Content	**Warm Mix Additive**	**RAP Source**		
		R-1	***R-2***	***R-3***
50%	1.0 % Sasobit®	X	X	X
	1.5% Sasobit®	X	X	X
75%	1.0 % Sasobit®	X	X	X
	1.5% Sasobit®	X	X	X

Table 4.13 Highway Base Mixtures Performance Experimental Design

	RAP Source	
RAP Content	***R-1***	***R-2***
50%	*12.5-50/RM-1*	*12.5-50/RM-2*
75%	*12.5-75/RM-1*	*12.5-75/RM-2*

For the recycled mixtures, a suite of testing was performed as detailed in Table 4.14; the values in each row for a mixture indicate the number of replicates tested for each test type and condition. Performance was evaluated in four categories; 1) rutting (*APA* and dry PURWheel); 2) durability (Cantabro); 3) tensile strength (IDT); and 4) moisture damage (*TSR* and wet PURWheel).

Table 4.14 Performance Testing and Replication of 50 and 75% RAP Recycled Mixtures

			Mixture			
Compaction Type	**Test Parameters**		***12.5-50/ RM-1***	***12.5-50/ RM-2***	***12.5-75/ RM-1***	***12.5-75/ RM-2***
SGC	***APA***	**7% V_a**	2	2	2	2
	APA	**10% V_a**	2	2	2	2
	TSR	**7% V_a**	1	1	1	1
	Cantabro	**Un-aged, N_{des}**	3	3	3	3
	IDT	**25 C**	2	2	2	2
LAC	**PURWheel**	**Dry**	2	2	2	2
	PURWheel	**Wet**	2	2	2	2

APA A replicate consisted of a single track in the test equipment composed of two 150 mm diameter specimens.
TSR A replicate consisted of six specimens tested according to *ASTM D 4867*.
Cantabro A replicate consisted of one *SGC* compacted specimen.
IDT A replicate consisted of one *SGC* compacted specimen.
PURWheel A replicate consisted of two specimens cut from a single *LAC* compacted slab and tested side by side at the same time.

CHAPTER 5

CHARACTERIZATION OF RAP PROPERTIES

5.1 Overview of RAP Characterization

A rational yet practical approach to mix design incorporating reclaimed asphalt pavement (RAP) in high quantities (e.g. over 25%) needs methods that can account for more than just the total asphalt content and gradation of the RAP. Two RAP sources with the same total asphalt content and gradation could perform very differently in a mixture depending on factors including the amount of absorbed bituminous material and the condition of the bituminous material on the surface of the aggregate. Initially the bitumen of these two sources could have been very different, moderately different, or the same depending on factors including the application and mix design method (Superpave, Marshall, or Hveem). In service, the aging of these two sources could have been very different, moderately different, or the same depending on factors including compaction, traffic, distresses, and environmental conditions. These two RAP sources should not be treated equally in a new mixture unless they are characterized such that it is justifiable to do so, and current practice does not have methods in place to make such an assessment.

In current practice, none of the factors that led to the amount of bituminous material or its condition (e.g. mix design method, field aging) for a given RAP source would be known to the designer of the new mixture. The amount of absorbed bitumen relative to the amount of total bitumen would be a function of the mix design method, and the grade of the bitumen on the surface of the aggregates would be a function of many variables. A method that focuses on the current properties is appropriate, as what led to the current properties of the bitumen is secondary to the properties themselves.

The debate over RAP properties has intensified in recent years due to decreasing budgets coupled with rising raw material prices. Key aspects of RAP behavior that have been debated include if and to what extent the bituminous material within RAP re-livens and contributes to compaction and performance of the new mixture. Figure 5.1 provides evidence that two of the key aspects of the RAP debate (heating temperature and heating time) affect the extent the bituminous material re-livens in a new mixture. Warm mix technologies are the key issue related to heating temperature, and asphalt production methods are the key issue related to heating time.

Figure 5.1*a* investigates the effect of heating temperature by compacting 100% *R-1* RAP without virgin binder at varying temperatures. Raw data is found in Howard et al. (2009). Better compaction occurred as the temperature was increased.

Figure 5.1*b* was intended to crudely approximate RAP heating during plant production (i.e. investigate effects of heating time) using *9.5-100/RM-1* (Raw data provided in Doyle and Howard 2010b). Typical methods of introducing RAP during plant production result in a short but intense level of heating; arguably this will heat the surface of RAP particles but may not fully heat the RAP before virgin binder is added. After mixing and before compaction, RAP has time to absorb additional heat from the virgin aggregate while in storage silos or during transport. Two G_{mm} samples were prepared according to the same procedure; one was aged for 60 minutes and the other was aged for 1440 minutes. Results of testing the two G_{mm} samples were nearly identical and were averaged for calculation of air voids.

Average air voids decreased as the short term aging time increased up to 180 minutes and thereafter average air voids increased. The maximum compaction occurred at 180 minutes of short term aging time (195 minutes total heating time). These results indicate that the addition of heat to the RAP after coating with virgin binder is beneficial to compaction, but that longer aging times are likely stiffening the virgin binder coating the RAP and ultimately hindering compaction relative to lesser aged virgin binder. It is unclear what effects longer storage times might have on high RAP mixes during plant production since asphalt storage silos limit exposure to oxygen in contrast to the forced-draft oven aging performed herein. The total heating time that resulted in optimum compaction for this experiment was approximately 195 minutes; this is close to the total heating time for RAP used for the rest of this study of 210 minutes (see Section 3.4).

The Figure 5.1 data shows that RAP bitumen on the aggregate surface is affected by the conditions encountered and that a portion of the bitumen remains inert (i.e. acts as aggregate) while the rest is effective and re-livens (i.e. facilitates compaction and then acts as binder though perhaps differently than when originally used). The remainder of the RAP bitumen is absorbed in the aggregate pores. A total of three types of bituminous materials are present within RAP: effective surface, inert surface, and absorbed.

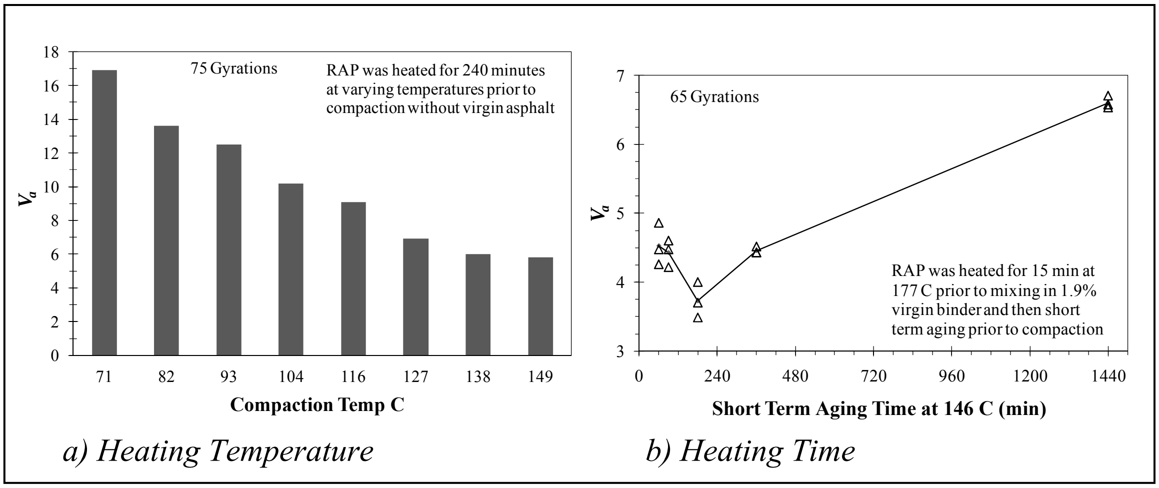


Figure 5.1 Effect of Heating Temperature and Heating Time on RAP

Compaction is arguably the key characteristic of mix design and the lubrication provided by bituminous material is arguably the key characteristic that facilitates compaction of a given aggregate structure. The approach taken in this paper is not able to consider the relative effects of compaction between bituminous material with different lubrication characteristics. This is a limitation as Figure 5.1*b* shows the effects of different amounts of binder aging on compaction.

A meaningful discussion related to lubrication effects of aged RAP bituminous material and virgin asphalt is premature until an estimate of the quantity of aged bituminous material is available. Bituminous material that was originally absorbed into the aggregate pores is not available to lubricate aggregates during compaction. Some of the bituminous material that was originally part of the lubricating material is believed to be inert in many conditions when used as RAP in a new mixture. The reminder of the bituminous material that was part of the original lubricating material would aid in lubrication in the new mixture but would be stiffer and as a result would not lubricate as much in the new mixture.

The component diagram of RAP provided in Figure 5.2 builds on the results of Figure 5.1 and is the focus of the rest of the analysis. The first issue addressed was prediction of absorbed bitumen, $P_{ba(R)}$, within RAP pores as prediction of this parameter has not previously been possible on a large scale such as within the operations of a state DOT. The second issue was characterization of the RAP surface asphalt and decoupling ineffective and effective surface asphalt ($P_{bi(R)}$ and $P_{be(R)}$, respectively). All terms are defined on a mixture mass basis; as a result do not necessarily have the same numerical value. For example, adding virgin binder ($P_{be(V)}$) would change the numerical value of absorbed bitumen ($P_{ba(R)}$) even though the mass of bitumen absorbed has not changed.

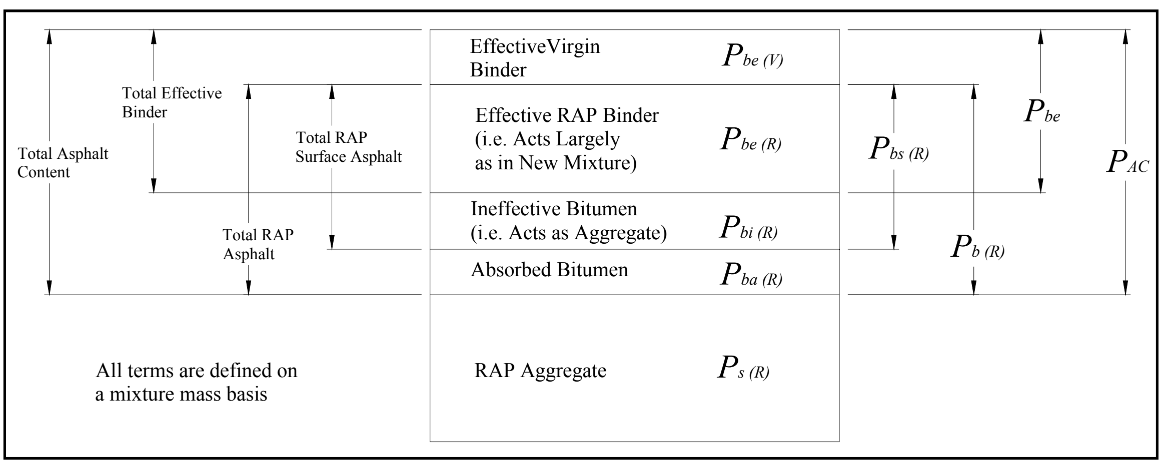


Figure 5.2 Component Diagram for RAP Mixtures

Many approaches have been taken with regard to RAP characterization, but an approach relying on a large mixture database coupled with testing of 100% RAP has not been attempted to the knowledge of the authors. This chapter presents the results of such an analysis using data from all MDOT approved mix designs over the past several years alongside testing of 100% RAP with added virgin binder. The database of mix designs was used to develop regression equations for prediction of properties that have previously posed difficulty. The approach was developed in a manner that does not require inputs that have been shown problematic to measure on extracted RAP aggregates (e.g. G_{sb}). The analysis assumes RAP in Mississippi is fully represented by the database used to develop the regression equations. Testing of 100% RAP was used to compliment the database regression in some instances, while other 100% RAP testing was used to verify quality of the regression; the analysis uses extraction only to measure total asphalt content.

The analysis was developed in a manner focused on practical implementation. Key items of consideration were that: 1) it is difficult to accurately measure G_{sb} on RAP aggregates after bitumen is extracted; 2) it is difficult to accurately measure G_{mm} of RAP as received from a producer stockpile due to dust on the surface of the particles, micro cracks in aged bitumen allowing water absorption, and similar; and 3) it is not difficult to accurately measure G_{mm} of RAP when coated with sufficient virgin asphalt. Justification of this approach is provided throughout the chapter.

5.2 MDOT Asphalt Mixture Database

Properties of all mix designs approved by MDOT between January 2005 and March 2010 were provided by the Materials Division and used for analysis. Data obtained for each asphalt mixture included combined properties of the aggregate blend, compactive effort, asphalt binder grade, and mixture volumetric properties. Also included were individual aggregate stockpile proportions, aggregate types, aggregate water absorptions, and stockpile gradations. For mixtures that contained RAP, the RAP total asphalt content, extracted aggregate gradation, and extracted aggregate water absorption were included. The data needed for the analysis is maintained by MDOT in a database where all approved mix designs are in a standard format, making the approach feasible. The approach could probably be implemented by other state DOT's as they likely maintain similar information in some type of organized fashion.

The raw data was arranged by nominal maximum aggregate size (NMAS) and design compactive effort (i.e. of 50, 65, and 85 gyrations). The database contained a total of 837 entries; 369 were 9.5 mm, 244 were 12.5 mm, and 224 were 19.0 mm NMAS.

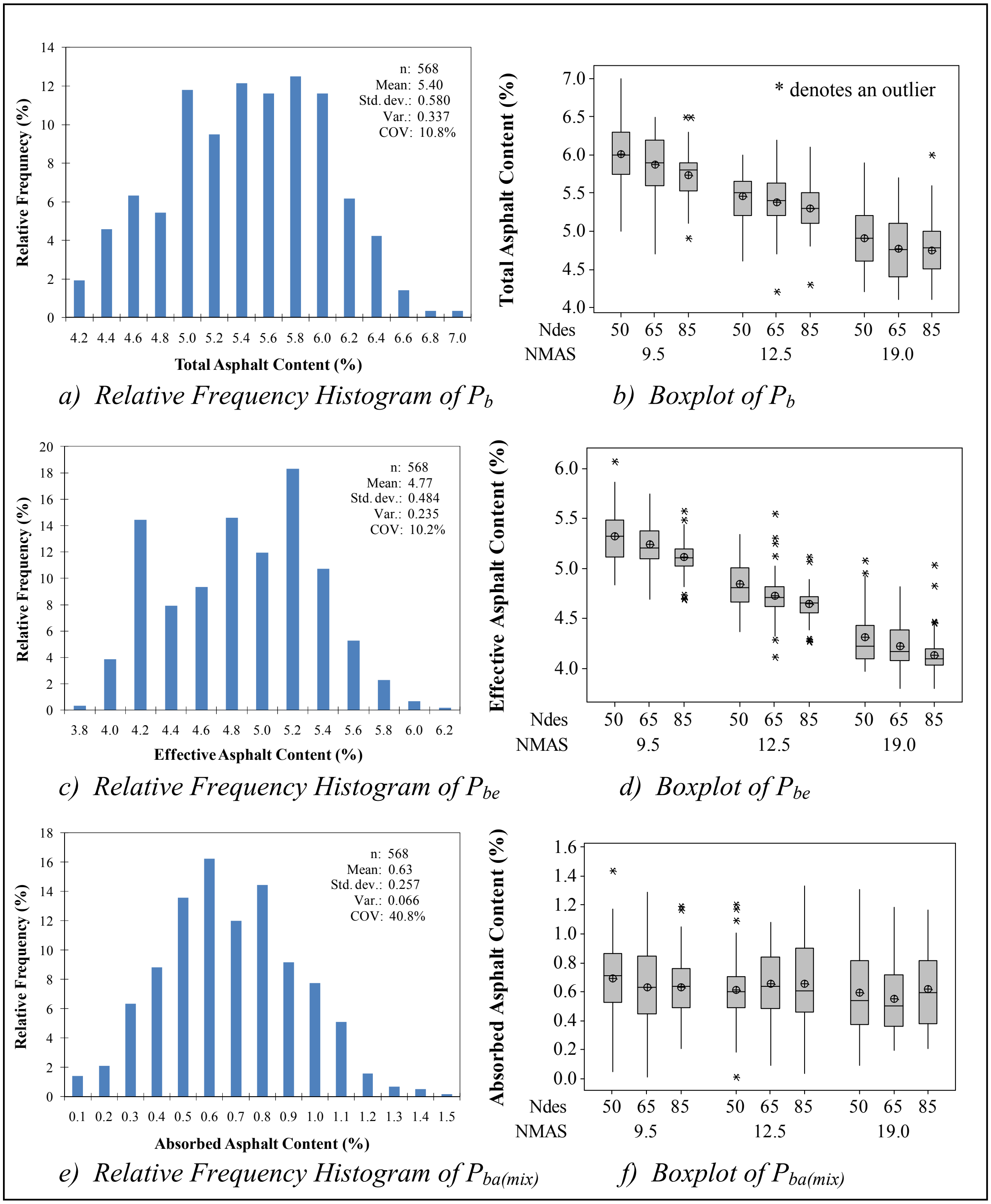
Not all 837 database entries were unique in terms of volumetric properties. In a number of cases there were two mixes with identical aggregate and volumetric properties. In most instances these duplicate cases resulted from re-approvals of existing mix designs with different binder grades or different binder sources. The duplicate cases were removed from the dataset as they do not represent unique volumetric mixture combinations, which reduced the number of mixes to 590.

The overwhelming majority of mixtures contained combinations of gravel, limestone, sand, and RAP though not all mixtures contained all these aggregate types. Twenty-two mixtures (3.7% of the total) were removed from the data set since they contained other aggregate types. The unusual aggregate types removed were: granite (19 mixes), slag (1 mix), sandstone (1 mix), and crushed concrete (1 mix). Removal of these 22 mixes left 568 for use in analysis. Of the 568 mixes, 93% or 529 contained RAP.

The dataset was considered to be the population of asphalt properties in Mississippi. This is a reasonable approach with all the approved mixtures statewide over a period in excess of five years. A key component of the investigation is the assumption that asphalt placed within the past five years represents the RAP being used in present day. This is a reasonable assumption for Mississippi within the jurisdiction of the Mississippi DOT.

5.2.1 Asphalt Contents of Mississippi Mixtures

Figure 5.3 presents relative frequency histograms and boxplots of total, effective, and absorbed asphalt contents for the mixtures. Examination of the relative frequency histograms of total and effective asphalt content (Figure 5.3*a* and Figure 5.3*c*) reveals a relatively wide spread of values and no clearly defined peak. The effective asphalt content standard deviation is lower than the total asphalt content standard deviation. The coefficients of variation (COV) for the two populations are nearly the same (approximately 10%). From the boxplot of total asphalt content (Figure 5.3*b*) it can be observed that as the NMAS of the aggregate gradation increases, the total asphalt content decreases as expected.



a) Relative Frequency Histogram of P_b

b) Boxplot of P_b

c) Relative Frequency Histogram of P_{be}

d) Boxplot of P_{be}

e) Relative Frequency Histogram of $P_{ba(mix)}$

f) Boxplot of $P_{ba(mix)}$

Figure 5.3 Summary Asphalt Content Results

The same observation can be made from the effective asphalt boxplot (Figure 5.3*d*). It is evident that an increase in compactive effort during mix design results in a decrease in effective asphalt content. This is expected since a greater compactive effort during mix design requires less effective asphalt to achieve a target level of air voids.

Examination of the absorbed asphalt relative frequency histogram (Figure 5.3*e*) indicates a general peak and an approximately normal distribution that is in contrast to the distributions of total and effective asphalt. The COV is approximately four times higher for the absorbed asphalt distribution. The boxplot of effective asphalt contents (Figure 5.3*f*) reveals little difference in mean absorption values with changes in compactive effort. The lack of change in mean absorbed asphalt content for changing compactive effort is expected since compactive effort does not affect aggregate absorptive capacity. It is interesting to note that the differences in absorbed asphalt content for different NMAS are quite small. One reason could be crushing the same base aggregate source to produce different gradations, since absorption is a general characteristic of the gravel or stone source.

Table 5.1 summarizes the mixtures contained in the dataset as well as the ranges of their total, effective, and absorbed asphalt contents. A few observations are indicated in the boxplots of total and effective asphalt content as potential outliers. While unusual, these observations were left in the dataset because they represent real mixtures and are part of the population of asphalt mixtures in Mississippi. These mixtures, however, were not shown in Table 5.1 as they detract from the point of the table.

The total asphalt content range over the five year period covered by this dataset was 4.1 to 7.0%. Note the wide range in absorbed asphalt in Table 5.1; absorbed asphalt content is seen to range from 0.03 to 1.33%. By defining $P_{ba(mix)}$ as a percentage of the total mixture, total asphalt content is the sum of absorbed and effective asphalt contents. MDOT uses this definition of absorbed asphalt for their mix designs.

Table 5.1 Summary of 568 Unique Mixtures in MDOT Mixture Dataset

NMAS	N_{des}	No. of Mixtures	Range of P_b max	Range of P_b min	Range of P_{be} max	Range of P_{be} min	Range of $P_{ba(mix)}$ max	Range of $P_{ba(mix)}$ min
9.5 mm	85	80	6.30	5.10	5.57	4.69	1.05	0.21
	65	75	6.50	4.70	5.74	4.69	1.29	0.06
	50	73	7.00	5.00	6.07	4.83	1.17	0.04
9.5 mm Mixes		**228**	**7.00**	**4.70**	**6.07**	**4.69**	**1.29**	**0.04**
12.5 mm	85	73	6.10	4.80	5.11	4.27	1.33	0.03
	65	49	6.20	4.70	5.13	4.28	1.08	0.09
	50	45	6.00	4.60	5.34	4.36	0.94	0.18
12.5 mm Mixes		**167**	**6.20**	**4.70**	**5.34**	**4.27**	**1.33**	**0.03**
19.0 mm	85	68	5.60	4.10	4.94	3.80	1.17	0.20
	65	54	5.70	4.10	4.81	3.80	1.19	0.19
	50	51	5.90	4.20	4.46	3.97	1.31	0.08
19.0 mm Mixes		**173**	**5.9**	**4.10**	**4.94**	**3.80**	**1.31**	**0.08**
All Mixes		**568**	**7.00**	**4.10**	**6.07**	**3.80**	**1.33**	**0.03**

Notes: The term $P_{ba(mix)}$ is defined on the basis of total mass of asphalt mixture and not on aggregate mass. Outliers have been removed from the data in this table.

5.2.2 Water Absorption of Mississippi Aggregate Sources

Aggregate stockpile data was sorted into limestone, sand, and gravel categories based on identifying information in the database. Water absorption relative frequency histograms for the three aggregate categories as well as the combined aggregate blends are provided in Figure 5.4. Discussion of each of the aggregate types and the aggregate blends follows.

The limestone histogram (Figure 5.4*a*) has a distribution with a mean of 0.91%, no clear peak value, and a slight right skew. A possible explanation is that Mississippi has no substantial native sources of limestone so essentially all limestone aggregate is imported from areas such as Kentucky and Alabama. The data could be a reflection of important quantities from different locations as they likely have different absorption properties. Overall, 80% of the limestone water absorption values fall in a range of about 0.35 to 1.75%.

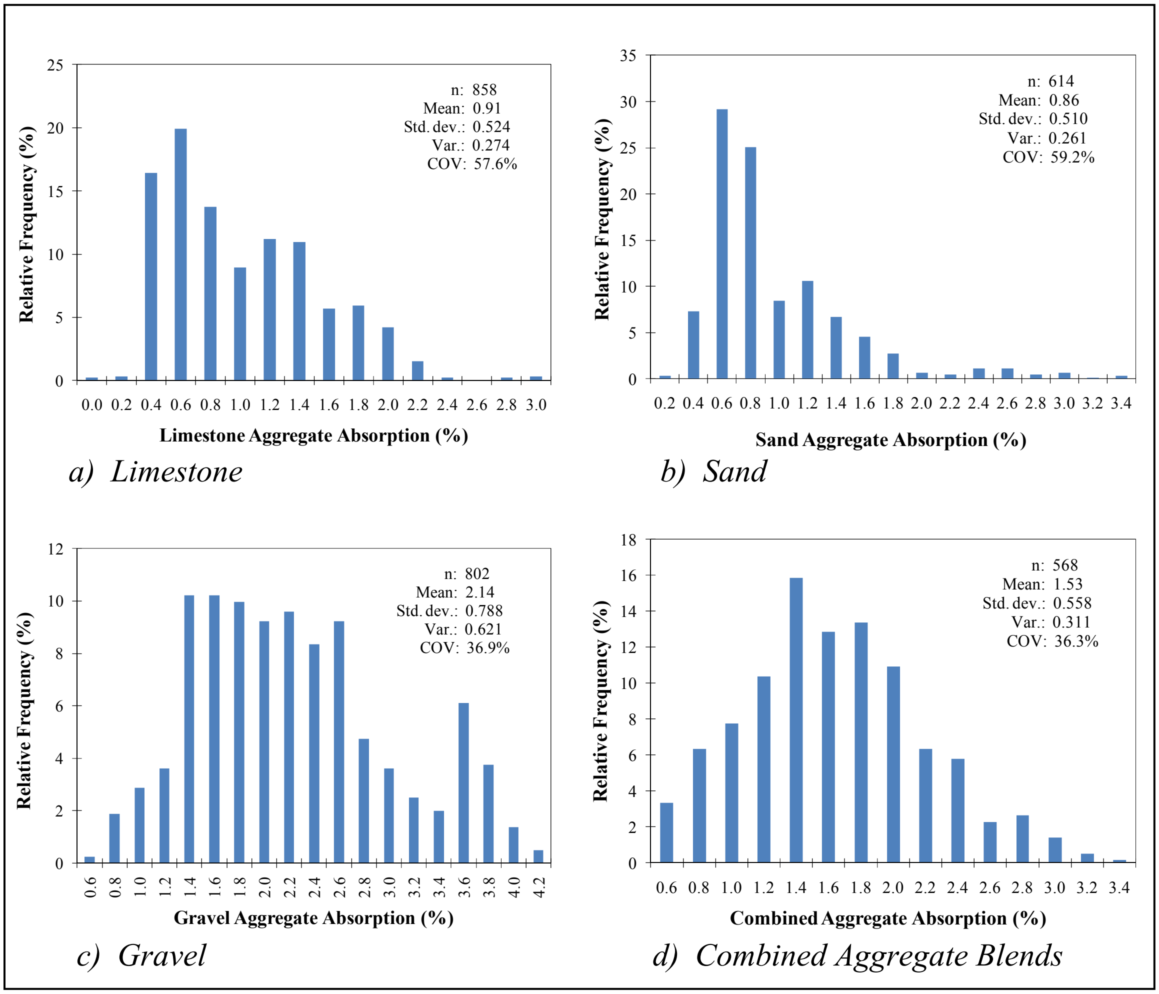


Figure 5.4 Summary Water Absorption Results

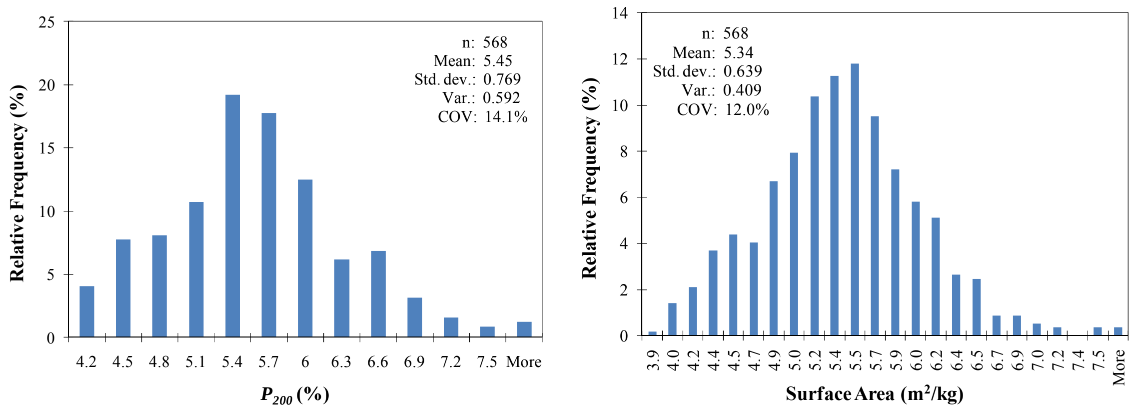
The Figure 5.4*b* sand distribution has a peak that is near the mean value of 0.86% but is severely right skewed. The MDOT database does not have a clear definition of what constitutes sand, which could explain some of the skew. The aggregate identified as sand could be naturally occurring aggregate (i.e. clean but un-crushed) or contain manufactured materials (i.e. crushed aggregate) that could have very different water absorption values. Overall, 80% of the sand water absorption values fall in a range of about 0.40 to 1.55%, which is similar to the limestone data.

The crushed gravel histogram (Figure 5.4*c*) reveals a wide distribution with no clear peak. This is likely due to variations in geology between aggregate sources from around the state. Overall, 80% of the gravel water absorptions fall in a range of about 1.25 to 3.45%.

Figure 5.2*d* plots composite aggregate blend water absorption results. In contrast to the individual aggregate sources, the distribution is approximately normal with a peak near the mean value of 1.53%. Overall, 80% of the aggregate blend water absorption values fall in a range of about 0.80 to 2.25%.

5.2.3 Gradation of Mississippi Aggregate Sources

Figure 5.5 plots percent fines and surface area (*SA*) for all aggregate blends. The percent fines distribution appears generally normal in shape with a mean of 5.45% but with a slightly higher proportion of values below the mean than above. The surface area distribution appears normal in shape with a mean of 5.34 m^2/kg and a few extreme values to the far right of the distribution.



a) Gradation Passing 0.075 mm Sieve *b) Surface Area*

Figure 5.5 Summary Aggregate Blend Results

5.2.4 Mississippi RAP Properties

Figure 5.6 plots RAP properties used in new mixtures. The total RAP asphalt content distribution (Figure 5.6*a*) is generally normal in shape but contains several values that are much higher and lower than the central distribution. The high values are likely due to testing error since they are above the highest total asphalt content of 7.0% contained in Table 5.1. Potential causes of error that would over estimate RAP asphalt content include aggregate degradation in an ignition test, loss of fine material, or incomplete recovery of mineral fines from extraction solvent. The low values may be due to testing error, be from RAP sources with stripped aggregate, or be from RAP mixed with base material during reclaiming.

The water absorption histogram (Figure 5.6*b*) is fairly normal aside from one abnormally low value that is likely testing error. The RAP fines histogram (Figure 5.6*c*) is also fairly normal and has a wide distribution with a range of 3 to 13% and a relatively high standard deviation. The extracted RAP aggregate surface area distribution (Figure

5.6*d*) has a mean value of 7.86 m^2/kg and two peaks on either side of the mean value. In a few cases, surface area exceeded 10.0 m^2/kg, and these cases were those in Figure 5.6*c* with a high fines (i.e. passing 0.075 mm sieve) content.

Since RAP was formerly new asphalt mixture it is informative to compare the distributions of RAP and current MDOT mixture properties. Table 5.2 presents the results of unequal variance *t*-test comparisons between RAP and current mixture properties. The mean RAP total asphalt content is significantly lower than the mean total asphalt content for MDOT mixtures; the difference is 0.21%. The variances of the two distributions are nearly identical which suggests that the distributions are quite similar except for their mean values. Possible reasons for the lower RAP asphalt contents include testing error resulting in lower total asphalt contents (e.g. incomplete extraction of RAP asphalt), loss of asphalt volatiles during service life, or actual loss of asphalt during the reclaiming process (e.g. during milling and handling). Another potential explanation for this result is that mixtures designed according to earlier versions of MDOT specifications (i.e. higher design compactive effort) resulted in generally lower asphalt contents than the current mix design specifications.

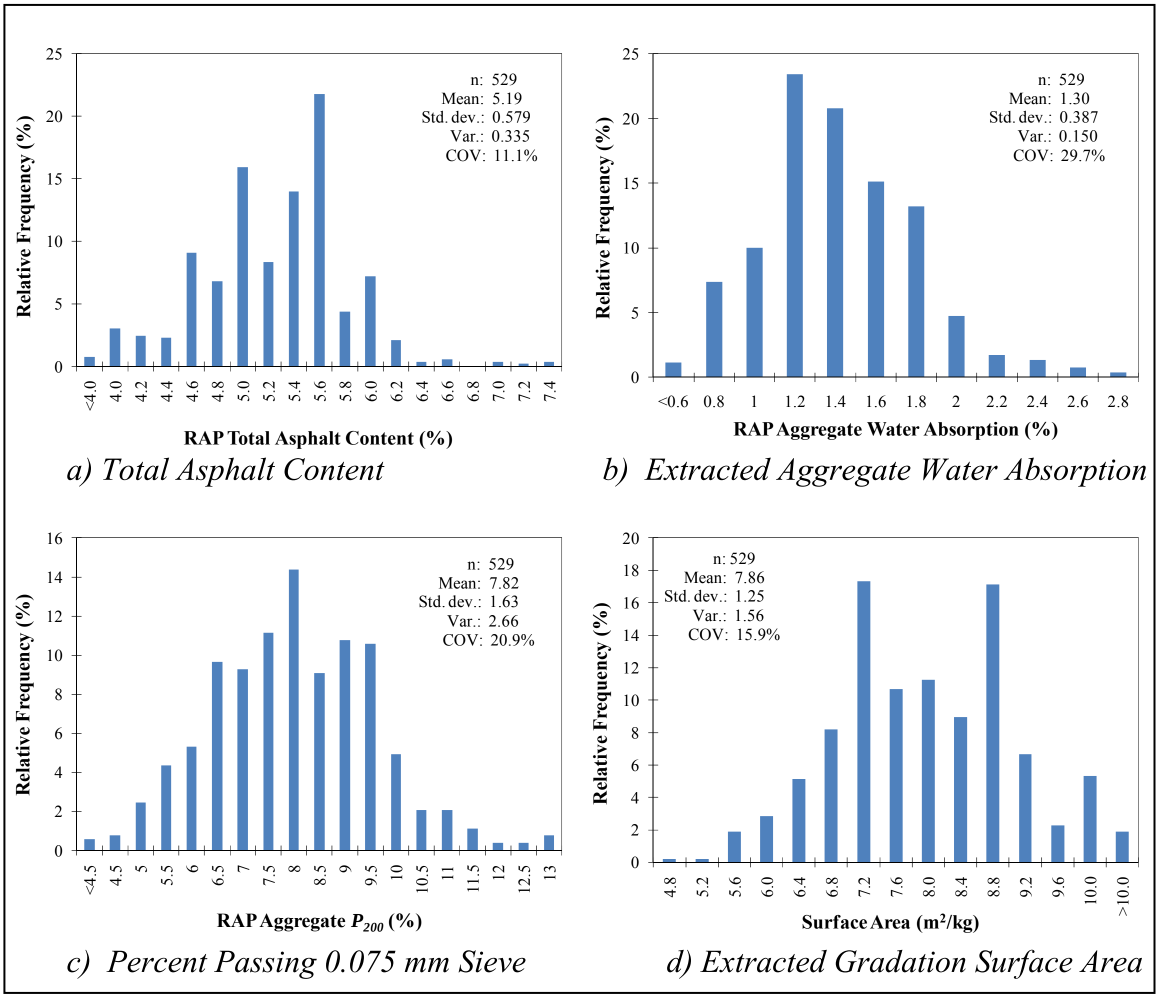


Figure 5.6 Summary RAP Results

The mean water absorption of extracted RAP aggregate is significantly lower than the mean combined aggregate blend water absorption values of current MDOT mixtures; the difference is 0.24%. Lower aggregate absorption values for RAP imply that values of RAP aggregate G_{sb} values are also lower. This aligns with the results found in several sources cited in the literature review that aggregate G_{sb} is often lower for extracted aggregate than for virgin aggregate. The mean percent passing the 0.075 mm sieve (i.e. fines) for RAP aggregate is significantly higher than for current MDOT mixtures; the difference is about 2.4%. The increased fines are likely due to aggregate degradation (milling in particular).

Table 5.2 Unequal Variance *t*-test Test Comparison of RAP to Mixture Properties

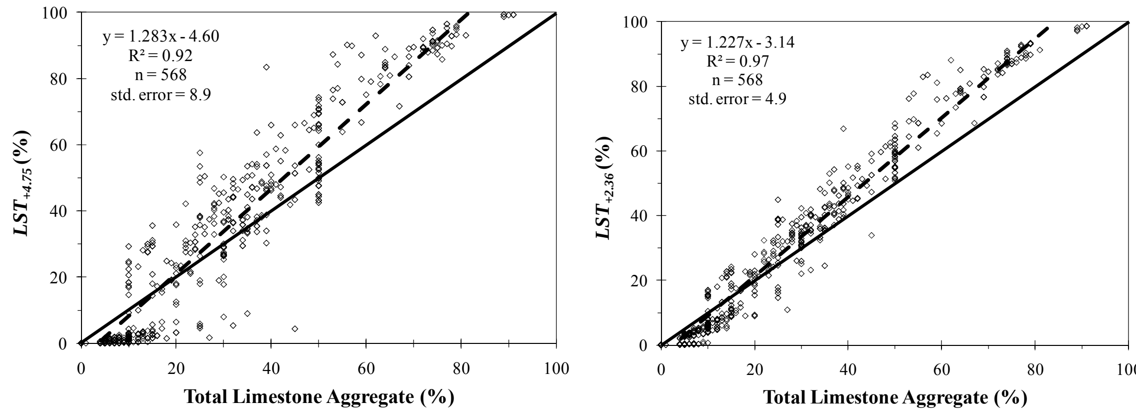
Category	Material	*n*	Mean	Var.	*t*-stat	*t*-crit	Significantly Different?
Asphalt Content	RAP	529	5.19	0.335	5.82	±1.96	Yes
	Mixtures	568	5.40	0.337			
Abs	RAP	529	1.30	0.150	-8.19	±1.96	Yes
	Mixtures	568	1.54	0.311			
P_{200}	RAP	529	7.82	2.662	30.49	±1.96	Yes
	Mixtures	568	5.45	0.592			

Note: Significance testing performed at the 95% confidence level.

5.3 Results of RAP Aggregate Sorting Procedure

To evaluate the usefulness of the aggregate sorting procedure for extracted RAP aggregate described in the experimental program, two regression equations were developed using the aggregate data in the mixture database. The first regression was of $LST_{+4.75}$ to total limestone aggregate in the mixture (Figure 5.7*a*). The correlation is reasonable ($R^2 = 0.92$) but there is some scatter in the data.

The second regression was of $LST_{+2.36}$ to total limestone aggregate in the mixture; this is shown in Figure 5.7*b*. The correlation is better ($R^2 = 0.97$) than for the regression developed for coarse aggregate retained on the 4.75 mm sieve. A very reasonable estimation of the percentage of total limestone aggregate in an aggregate blend can be determined by using the aggregate sorting procedure developed as part of this research project. This procedure is used later in the chapter as an input for regression equations for prediction of RAP properties.



a) Retained on *4.75 mm Sieve* *b) Retained on 2.36 mm Sieve*

Figure 5.7 Correlation of Coarse Limestone Aggregate to Total Limestone Aggregate

5.4 Correlation of Water and Asphalt Absorption

A regression equation was developed relating absorbed asphalt to aggregate water absorption, which is shown in Figure 5.8. A correlation does exist as asphalt absorption is on the order of one third of aggregate water absorption. However, there is a noticeable amount of scatter in the data (R^2 = 0.51). The relationship is similar to that found by Kandhal and Khatri (1992) discussed in the literature review. Prediction of absorbed asphalt using water absorption would require measurement of water absorption on aggregates extracted from RAP, which was found to be variable during literature review. This does not appear to be the optimal approach to estimate absorbed asphalt in RAP.

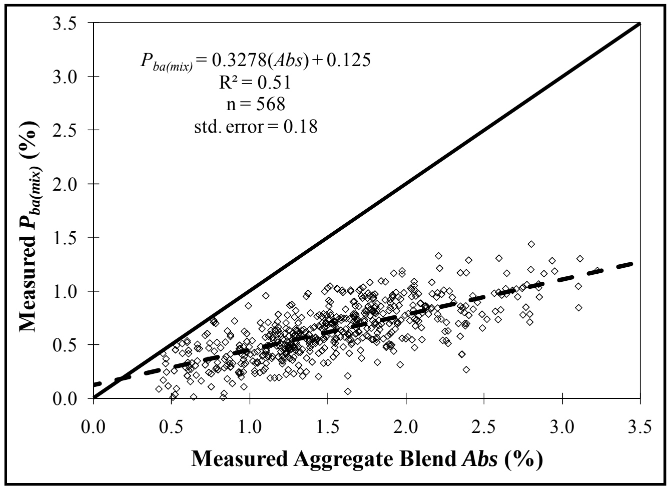


Figure 5.8 Use of Aggregate Water Absorption to Estimate Asphalt Absorption

5.5 RAP Absorbed Asphalt

Evidence is provided in this section that G_{se} can be reliably and efficiently determined by measuring G_{mm} on RAP coated with additional virgin binder. The evidence is supported by data showing RAP does not absorb noticeable amounts of virgin asphalt. The evidence is also supported by discussion related to the difficulty of conducting G_{mm} on RAP versus the ease of determining G_{mm} on RAP coated with virgin binder.

5.5.1 RAP Absorbed Asphalt Experiment 1

The propensity of RAP to absorb additional virgin asphalt was investigated using the methods described in Section 4.3.1.3. *R-1* RAP was heated and short term aged at 116 C and 138 C in conjunction with three total asphalt contents (P_{AC}) as given in Table 4.2. Raw data is provided in Doyle and Howard (2010b), and the raw G_{mm} values were used to calculate G_{se} values. Asphalt binder specific gravity (G_b) of 1.03 was assumed for all calculations.

A pooled variance *t*-test was used to compare replicates containing Sasobit® and Evotherm™ 3G to the replicates without warm mix additives (Table 5.3). Results indicated no significant difference in the mean values for either comparison. In that there were no statistical differences in the data, all the data with and without warm mix additives at a given temperature and asphalt content was grouped together for the next step in the analysis.

An ANOVA test was then performed on the G_{se} data and the results are provided in Table 5.4. The interaction of temperature and total asphalt content was not significant. Also, RAP heating and short term aging temperature were not found to be significant parameters. Likewise, RAP total asphalt content was not found to be a significant parameter.

Temperature and warm mix additives were not considered for the two remaining RAP sources. The three total asphalt contents tested are given in Table 4.2. The raw data is provided in Doyle and Howard (2010b); the data was used to calculate G_{se}. Tables 5.5 and 5.6 provide results of ANOVA analyses and based on the results, RAP total asphalt content was not found to be a significant parameter for G_{se} results for the *R-2* and *R-3* RAP sources.

The results indicate that the warm mix temperatures tested did not induce any additional asphalt absorption for the *R-1* RAP source. Warm mix additives did not induce any additional absorption of asphalt for the *R-1* RAP source. The amount of virgin asphalt added did not affect determination of aggregate G_{se} for the *R-1*, *R-2*, or *R-3* RAP sources.

Table 5.3 Pooled Variance *t*-test Test for G_{se} of *R-1* RAP

Comparison	*n*	Mean	Var.	*t*-stat	*t*-crit	Significantly Different?
None	12	2.581	1.02×10^{-4}	1.56	±2.30	No
Sasobit®	12	2.575	4.36×10^{-5}			
None	12	2.581	1.02×10^{-4}	1.48	±2.30	No
Evotherm 3G™	12	2.574	1.39×10^{-4}			

Note: Significance testing performed at the 95% confidence level.

Table 5.4 ANOVA Test for G_{se} of *R-1* RAP

Source	df	SS	MS	F_{calc}	P_{value}	Significant?
Temp	1	0.0002609	0.0002609	3.04	0.091	No
P_{AC}	2	0.0005409	0.0002705	3.15	0.057	No
Temp * P_{AC}	2	0.0000590	0.0000295	0.34	0.712	No
Error	30	0.0025735	0.0000858			
Total	35	0.0034344				

Note: Significance testing performed at the 95% confidence level.

Table 5.5 ANOVA Test for G_{se} of *R-2* RAP

Source	df	SS	MS	F_{calc}	P_{value}	Significant?
P_{AC}	2	0.0000418	0.0000209	0.80	0.525	No
Error	3	0.0000779	0.0000260			
Total	5	0.0001197				

Note: Significance testing performed at the 95% confidence level.

Table 5.6 ANOVA Test for G_{se} of *R-3* RAP

Source	df	SS	MS	F_{calc}	P_{value}	Significant?
P_{AC}	2	0.0001409	0.0000705	3.92	0.145	No
Error	3	0.0000539	0.0000180			
Total	5	0.0001948				

Note: Significance testing performed at the 95% confidence level.

5.5.2 RAP Absorbed Asphalt Experiment 2

The potential for asphalt absorption to be affected in RAP under varying conditions was investigated using the *R-1* and *R-3* RAP sources. Details are provided in Section 4.3.1.4. Doyle and Howard (2010b) contains the raw data and Table 5.7 provides the results of this experiment.

The as-received (un-heated) data provided a baseline measurement of the RAP aggregate absorbed asphalt. The data after 120 minutes of heating provided a measurement of whether any additional RAP asphalt was absorbed by the RAP aggregate. The sample without short term aging provided a baseline measurement of new asphalt absorption for the mixture. The 4 hour short term aging period at standard hot mix temperature (146 C) was selected to be very favorable to new asphalt absorption and to represent the best possible opportunity for additional asphalt absorption by the RAP aggregate.

Table 5.7 Results of Absorbed Asphalt Experiment 2

Material Tested	**Condition**	P_{AC} **(%)**	G_{mm} [a]	G_{se}
R-1 + 0% $P_{be(V)}$	As received	5.5	2.382	2.579
R-1 + 0% $P_{be(V)}$	2 hr heat at 171 C	5.5	2.373	2.567
R-1 + 2% $P_{be(V)}$	2 hr heat at 171 C, no aging	7.4	2.315	2.571
R-1 + 2% $P_{be(V)}$	2 hr heat at 171 C, 4 hr aging at 146 C	7.4	2.319	2.577
		_R-1_ G_{se} Summary: Average 2.574		**Range 0.012**
R-3 + 0% $P_{be(V)}$	As received	5.0	2.415	2.599
R-3 + 0% $P_{be(V)}$	2 hr heat at 171 C	5.0	2.422	2.608
R-3 + 2% $P_{be(V)}$	2 hr heat at 171 C, no aging	6.9	2.351	2.598
R-3 + 2% $P_{be(V)}$	2 hr heat at 171 C, 4 hr aging at 146 C	6.9	2.358	2.608
		_R-3_ G_{se} Summary: Average 2.603		**Range 0.010**

a) Average of two measurements.

The difference in G_{se} results for the two tested conditions is 0.012 for *R-1* and 0.010 for *R-3*. Both differences are less than the allowable range of 0.014 for four determinations of G_{mm} by a single operator. The results indicate that a negligible amount of additional asphalt (aged or virgin), if any, is absorbed by the *R-1* or *R-3* RAP aggregates during laboratory heating and short term aging.

For uncoated RAP there is a tendency for fine material to be lost during the test as evidenced by the dark cloud that appears in the water bath while obtaining the submerged mass of the sample. Also, broken RAP aggregate surfaces produced during the milling process could affect test results. It is much easier to obtain accurate G_{mm} measurements for G_{se} calculation with RAP coated with an additional 2% virgin asphalt on a mixture mass basis than with only the RAP. Table 5.7 provides evidence the approach is also accurate. Figure 5.9 illustrates the differences between as received RAP and that coated with virgin asphalt. *R-3* RAP was shown as it had the most uncoated aggregates of the sources tested. Some aggregate had stripped during service, but test data showed the asphalt remained in the aggregate pores leading to consistent G_{se} measurements. RAP that has been contaminated with base material that has never been coated with asphalt would cause difficulty, whereas stripped aggregate does not appear to cause difficulty.

The data presented in this section shows that the asphalt absorbed and measured during the mix design process ($P_{ba(mix)}$) is equivalent to absorbed RAP asphalt ($P_{ba(R)}$) for practical purposes. The numerical value of either $P_{ba(mix)}$ or $P_{ba(R)}$ will vary depending on the amount of $P_{be(V)}$ being considered since they are defined on a mixture mass basis. As $P_{be(V)}$ increases $P_{ba(R)}$ decreases. Numerically, $P_{ba(R)}$ is a maximum when $P_{be(V)}$ is zero, but it should be understood that the amount of asphalt absorbed in the RAP is not changing rather the total mix mass is increasing and making the amount of absorbed asphalt less of the total mix mass. The remainder of the chapter uses $P_{ba(R)}$ to define asphalt absorbed into RAP pores since it is one of the key parameters under investigation.

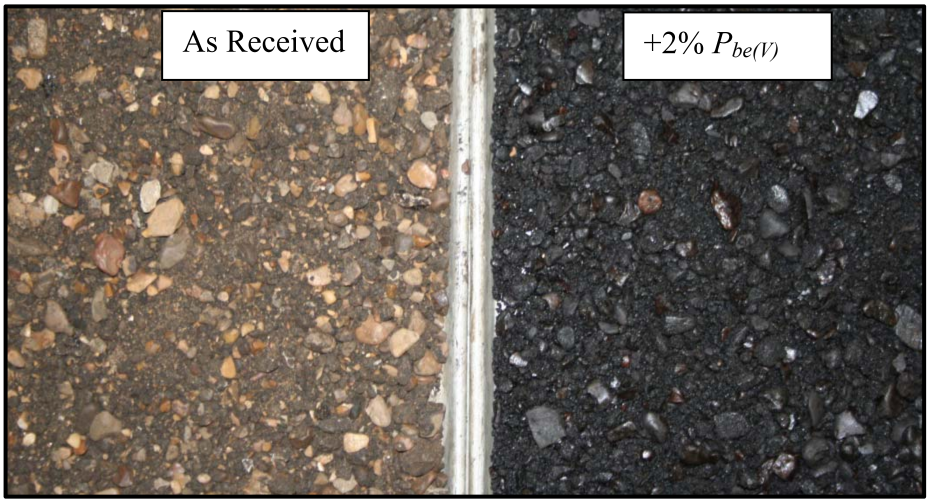


Figure 5.9 Loose *R-3* RAP Samples With and Without Virgin Binder

5.6 Prediction of RAP Absorbed Asphalt

Regression equations were developed to relate the measurable properties of a RAP source (either directly or indirectly) to absorbed asphalt. The measurable properties of a RAP source for purposes of this analysis were: G_{se}, $P_{b(R)}$, SA, $LST_{+2.36}$, and similar properties that can be determined knowing only the total asphalt content and having extracted RAP aggregate. Surface area (*SA*) can be computed based on the percent passing each sieve size and standard surface area factors according to the method presented in Roberts et al. (1991). Figure 5.7 was used to select $LST_{+2.36}$ to represent the percentage of limestone in the mixture. The remaining terms are conventional. Development of regression equations that required inputs not available within current practice or that required inputs shown problematic in literature (e.g. G_{sb}) were not attempted since they are less practical than equations that can be developed with practically measured inputs.

The most desirable approach was to be able to predict $P_{ba(R)}$ directly from regression, and the next most desirable approach was to be able to predict G_{sb} and use volumetric relationships to calculate $P_{ba(R)}$. Regression equations were developed in a step-wise fashion where all input variables under consideration were used to predict the output of interest. Input variables that did not affect the prediction were removed until all variables remaining affected the calculated output variable. Direct $P_{ba(R)}$ calculation could not produce R^2 values greater than approximately 0.6. The best regression equation for calculation of G_{sb} is provided in Eq. 5.1. Eq. 5.2 is the standard volumetric equation used in conjunction with the values calculated in Eq. 5.1 to determine $P_{ba(R)}$.

$$G_{sb\,\min,\max} = \left[1.111G_{se} - 0.329\right] \pm z_{\alpha/2}\left(0.0156\right) \qquad R^2 = 0.94 \qquad n = 568 \qquad \text{(Eq 5.1)}$$

$$P_{ba(R)} = P_{ba(mix)} = \left(100 - P_{b(R)}\right)\frac{\left(G_{se} - G_{sb}\right)}{G_{sb}G_{se}}G_b \qquad \text{(Eq 5.2)}$$

Where:

G_{sb} = oven dry bulk specific gravity of RAP aggregate from Eq. 5.1

G_{se} = effective specific gravity of RAP aggregate measured on coated particles

G_b = specific gravity of asphalt binder (assumed to be 1.03)

$P_{b(R)}$ = total RAP asphalt content measured by ignition or extraction methods (%)

$P_{ba(R)}$ = absorbed asphalt in the RAP source by mixture mass (%)

$P_{ba(mix)}$ = absorbed asphalt by mixture mass from the MDOT database (%)

$z_{\alpha/2}$ = statistical coefficient accounting for variability in the prediction of G_{sb}

The prediction method does not require sophisticated inputs, and its correlation is very reasonable. Figure 5.10 provides a visual representation of the prediction ability of Eq. 5.1 within the activities of MDOT; values were computed with $z_{\alpha/2}$ equal to zero. The data in Figure 5.10 is distributed closely and evenly around the line of equality indicating no consistent errors associated with the prediction. A $z_{\alpha/2}$ value of 1.96 representing a 95% confidence level was used to compute the prediction interval band shown in Figure 5.10.

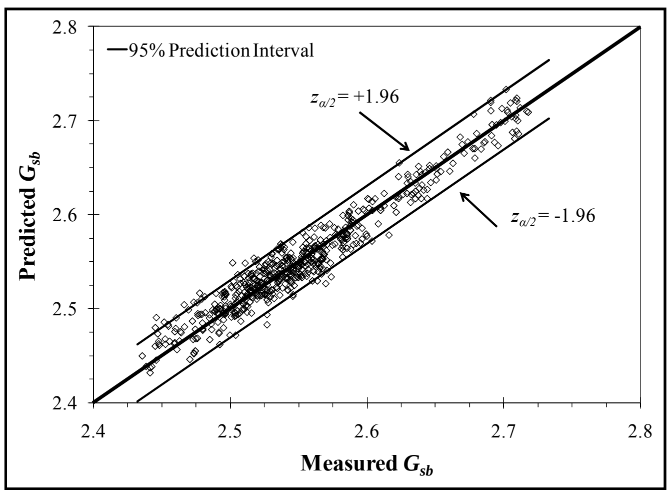


Figure 5.10 Comparison of Measured and Predicted G_{sb}

The uncertainty term in Eq. 5.1 represents variability in the fundamental properties of the asphalt mixtures that cannot be accounted for by any statistically significant and physically meaningful properties that can be readily and reliably measured for RAP in current practice. For practical purposes the 95% prediction interval of Figure 5.10 is equivalent to the single operator repeatability (d2s) index for measurement of G_{sb} on fine aggregate as given in both *AASHTO T 84* and *ASTM* C 128

(0.031 vs. 0.032). The prediction interval range of 0.031 is slightly wider than the coarse aggregate (d2s) repeatability index of 0.025 given in both *AASHTO T 85* and *ASTM* C 127. Overall the ability of the equation to explain the relationship of G_{se} to G_{sb} is on the order of the accuracy of G_{sb} measurement.

Figure 5.11 provides pertinent data in terms of the increase in G_{se} relative to G_{sb} and how that behavior equates to measured $P_{ba(R)}$ values in the database. Interestingly G_{se} never exceeds G_{sb} by more than 0.10. Based on Figure 5.11 increasing G_{se} by 0.01 corresponds to an approximate increase in $P_{ba(R)}$ of 0.15 to 0.20% indicating small errors in G_{sb} or G_{se} result in considerable errors in $P_{ba(R)}$.

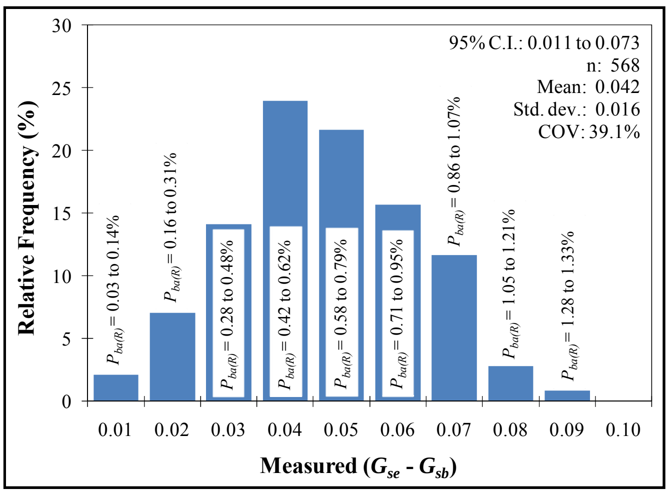


Figure 5.11 Relative Frequency Histogram of Measured (G_{se} - G_{sb})

The error associated with the Eq. 5.1 prediction can be seen using Figure 5.11 and Figure 5.12. Figure 5.12 plots the data used to develop Eq. 5.1, and shows the best fit trend line (i.e. $z_{\alpha/2} = 0$) and the 95% prediction interval (i.e. $z_{\alpha/2} = \pm 1.96$). The minimum, median, and maximum G_{se} values are 2.485, 2.584, and 2.756 respectively. Using Eq.

5.1 with $z_{\alpha/2}$ of zero, G_{se} minus G_{sb} terms are 0.053, 0.042, 0.023 respectively. As seen in Figure 5.11, Eq. 5.1 does not encompass the 95% confidence interval of measured G_{se} minus G_{sb} which is 0.011 to 0.073. Eq. 5.1 and Eq. 5.2 with $z_{\alpha/2}$ of zero are only capable of predicting $P_{ba(R)}$ of 0.30 to 0.86% while the 95% confidence interval of $P_{ba(R)}$ was 0.13 to 1.13% and the total interval with outliers removed was 0.03 to 1.33%.

Error in the prediction using 95% confidence interval data coupled with the approximate increases of $P_{ba(R)}$ with increases of G_{se} minus G_{sb} results in approximately 0.4% increase in $P_{ba(R)}$ that cannot be explained by the Figure 5.12 trend line. Likewise, approximately 0.25% decrease in $P_{ba(R)}$ cannot be explained by the Figure 5.12 trend line. When error in the prediction is considered the distribution of $P_{ba(R)}$ is fully encompassed. Error of 0.25% below the interval and 0.40% above the interval is not out of line with the differences that would occur in calculation of absorbed asphalt due to measurement error of G_{sb} according to (d2s) limits.

The approach provided in this section is not capable of predicting an exact value of $P_{ba(R)}$, though it can provide a reasonable value and a range that is very unlikely to be exceeded. The approach also bounds the problem and does not allow $P_{ba(R)}$ values to be used that cannot be correct. The next section provides verification information that shows this approach is reasonable to predict $P_{ba(R)}$ and that the values predicted are better than current practice for many situations. The next section also shows that current practice reports $P_{ba(R)}$ values that are almost certainly incorrect.

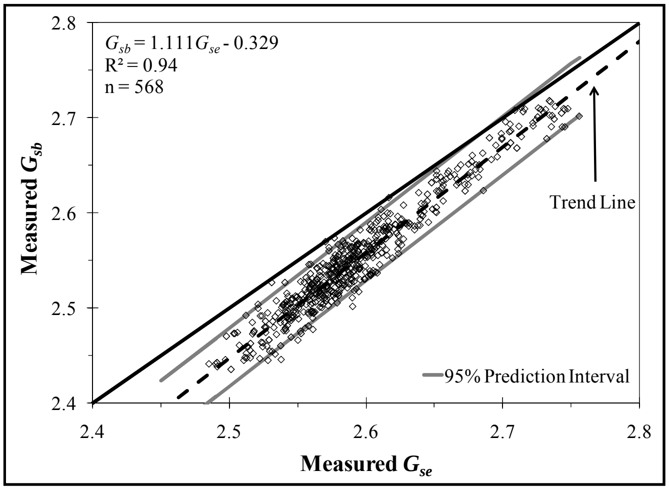


Figure 5.12 Comparison of G_{se} and G_{sb} Values From Database

5.7 Evaluation of RAP Absorbed Asphalt Prediction

The approach developed from the MDOT mixture database for determining G_{sb} and $P_{ba(R)}$ are evaluated in this section. Five RAP sources were used in the evaluation, and the input values and resulting outputs from Eq. 5.1 and 5.2 are provided in Table 5.8. Conventionally measured G_{sb} via *AASHTO T 84* and *T 85* on extracted RAP aggregate was also used to calculate $P_{ba(R)}$ according to standard protocol.

Table 5.8 G_{sb} values differ substantially between Eq. 5.1 and *T 84*, *T 85* measured values, with Eq. 5.1 predicting higher values in all cases. This observation is supported by literature. The G_{sb} values from conventionally measured techniques are very likely too low based on the database information as $P_{ba(R)}$ values were 1.13% or less for the 95% confidence interval for all Mississippi mixes over the last five years. The likelihood of three RAP sources known to come from Mississippi and known to be from different pavements exceeding the 95% confidence interval while no data was within the 95% confidence interval of such a comprehensive data set is near impossible.

Table 5.8 Evaluation of G_{sb} and $P_{ba(R)}$ Prediction Equations

RAP Source	G_{se} Type	n	Value	Eq. 4.1, 4.2 Predicted[a] G_{sb}	G_{se} - G_{sb}	$P_{ba(R)}$	Conventionally Measured G_{sb}	$P_{ba(R)}$
R-1	Avg.	48	2.577	2.534	0.043	0.18, 0.64, 1.11	2.483	1.43
	Max	48	2.599	2.559	0.040	0.14, 0.59, 1.05		
	Min	48	2.557	2.512	0.045	0.21, 0.68, 1.16		
R-2	Avg.	6	2.605	2.565	0.040	0.13, 0.58, 1.03	2.526	1.17
	Max	6	2.608	2.569	0.039	0.12, 0.57, 1.03		
	Min	6	2.596	2.555	0.041	0.14, 0.59, 1.06		
R-3	Avg.	14	2.608	2.569	0.039	0.12, 0.57, 1.03	2.504	1.66
	Max	14	2.626	2.589	0.037	0.09, 0.54, 0.99		
	Min	14	2.596	2.555	0.041	0.15, 0.60, 1.06		
R-4	Avg.	2	2.596	2.555	0.041	0.14, 0.59, 1.05	---	---
R-5	Avg.	2	2.620	2.582	0.038	0.10, 0.54, 0.99	---	---

Note: $P_{ba(R)}$ values shown in this table coincide with $P_{be(v)}$ of zero.

a) G_{sb} and G_{se} - G_{sb} values shown are for $z_{\alpha/2}$ of 0 and $P_{ba(R)}$ is for $z_{\alpha/2}$ of -1.96, 0, and 1.96, respectively.

The variability of the method described by Eq. 5.1 and 5.2 can be seen in Table 5.8. *R-1* RAP with average G_{se} values has been used for the purposes of discussion, though the same concept applies to all RAP sources. It should be understood that the most likely $P_{ba(R)}$ with Mississippi materials for G_{se} equal to 2.577 is 0.64% and that values as low as 0.18% and as high as 1.11% are possible but unlikely. In a good number of cases the actual value for a randomly sampled RAP source would be say 0.52% if lower or 0.77% if higher. In a smaller number of cases, the actual value would be say 0.38% if lower or 0.83% if higher. In a fairly small number of cases, the actual value would be say 0.21% if lower or 1.06% if higher. In any instance, the maximum error in the prediction would be either 0.64 minus 0.18, or 0.46% if lower or 1.11 minus 0.64, or 0.47% if higher. The maximum error is not 1.11 minus 0.18, or 0.93%.

The method to predict $P_{ba(R)}$ using G_{se} on RAP coated with virgin binder was shown to be stable using the Table 5.8 data. *R-1* was measured 48 times and the $P_{ba(R)}$ value predicted with $z_{\alpha/2}$ equal to 0 varied at most 0.09%. Figure 5.11 indicated this level of $P_{ba(R)}$ variation could occur with less than 0.01 difference in measurement of G_{se} minus G_{sb}, which is well within between operator precision in standard test protocols.

The G_{se} minus G_{sb} values shown are reasonable when viewed in terms of the relative frequency histogram provided in Figure 5.11. No RAP was tested with G_{se} values in the upper or lower portions of the Figure 5.11 distribution. *R-4* and *R-5* were on hand in the laboratory and used for verification, but upon testing it was observed they too fall in the central portion of the distribution. Ideally, dozens of RAP sources could have been obtained throughout Mississippi and tested with the proposed and conventional methods for comparison.

5.8 Effect of Additives and Temperature on RAP Volumetrics

Three hundred ninety-four 100% RAP specimens were mixed with virgin binder and compacted according to the procedures described in Section 3.4; bulk density and air voids were determined as described in Section 4.4.1. The experimental design is discussed in Section 4.3.1.2. The compaction data is presented in terms of P_{AC} and each data point represents the average of all replicates for that experimental treatment combination. All of the raw data can be found in Doyle and Howard (2010b).

Results of the *R-1* RAP compaction data at 116 and 138 C are shown in Figure 5.13 organized by compaction temperature and warm mix additive. The effect of compactive effort is observed in all cases. As the compactive effort is increased, the air voids generally decrease for a given asphalt binder content. This result is expected in new mixtures and was also observed in the 100% RAP mixtures.

Figure 5.14 presents the *R-1* data for 65 gyrations organized by compaction temperature and warm mix additive. For a specific compaction temperature and compactive effort combination the effect of warm mix additives is minimal although minor differences are observable at the 138 C temperature. Note how an increase in compaction temperature noticeably reduces the air voids level for the same asphalt content and combination of compactive effort and warm mix additive. Also note that the lowest total asphalt content (highest contribution of RAP bitumen) in combination with 154 C temperature resulted in air voids near 4%.

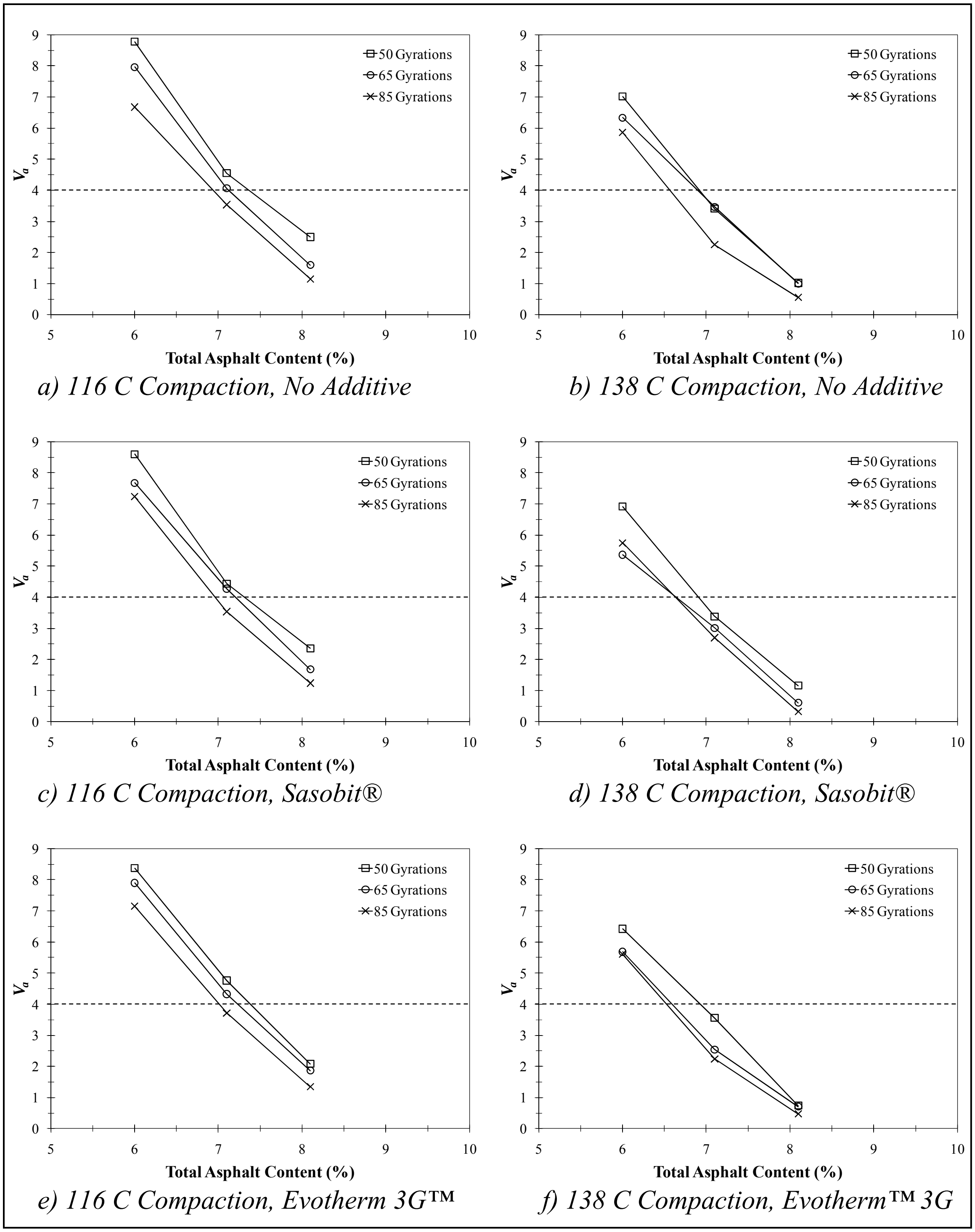
Results of the *R-2* RAP compaction data with 65 gyrations are presented in Figure 5.15 organized by compaction temperature. The data exhibits similar trends to the *R-1* compaction data. Almost no difference is seen in air voids with the addition of warm mix additives except at the 154 C compaction temperature where Evotherm™ 3G is observed to improve compaction somewhat. Increasing the compaction temperature reduces the air voids. Note that the lowest total asphalt content resulted in air voids near 4% at compaction temperatures of 138 C and 154 C.

Results of the *R-3* RAP compaction data with 65 gyrations are presented in Figure 5.16 organized by compaction temperature. The addition of warm mix additives is again seen to have little effect on compaction.

The effects of warm mix additives on *R-1* RAP with 50 and 85 gyrations are observed to be minimal based on Figure 5.17. In general warm mix additives are observed to have very little effect on compaction of 100% RAP. This could be due to the difficulty of mixing small dosage levels of warm mix additives into the thin film of aged RAP bitumen already coating the RAP aggregates.

Linear regression was performed using the data in Figures 5.13 through 5.17 using the average V_a values for each of the three total asphalt contents. The results are provided in Table 5.9. For each combination of gyrations, warm mix additive, and compaction temperature, the total asphalt content (P_{AC}) that would produce 4% air voids in a compacted specimen was calculated from the regressions and tabulated in Table 5.9.

In most cases the addition of warm mix additives resulted in small or no changes in the P_{AC} estimates for the combination of RAP mixture and compaction parameters. No consistent trends of either additive with respect to the control were apparent. In some cases Evotherm™ 3G resulted in the lowest P_{AC}, in other cases Sasobit® resulted in the lowest P_{AC}, and in other cases the control treatment resulted in the lowest P_{AC} estimate but the overall the differences were generally small. The average P_{AC} value in Table 5.9 is recommended for use.



a) 116 C Compaction, No Additive

b) 138 C Compaction, No Additive

c) 116 C Compaction, Sasobit®

d) 138 C Compaction, Sasobit®

e) 116 C Compaction, Evotherm 3G™

f) 138 C Compaction, Evotherm™ 3G

Figure 5.13 Effect of Compaction on *R-1* RAP

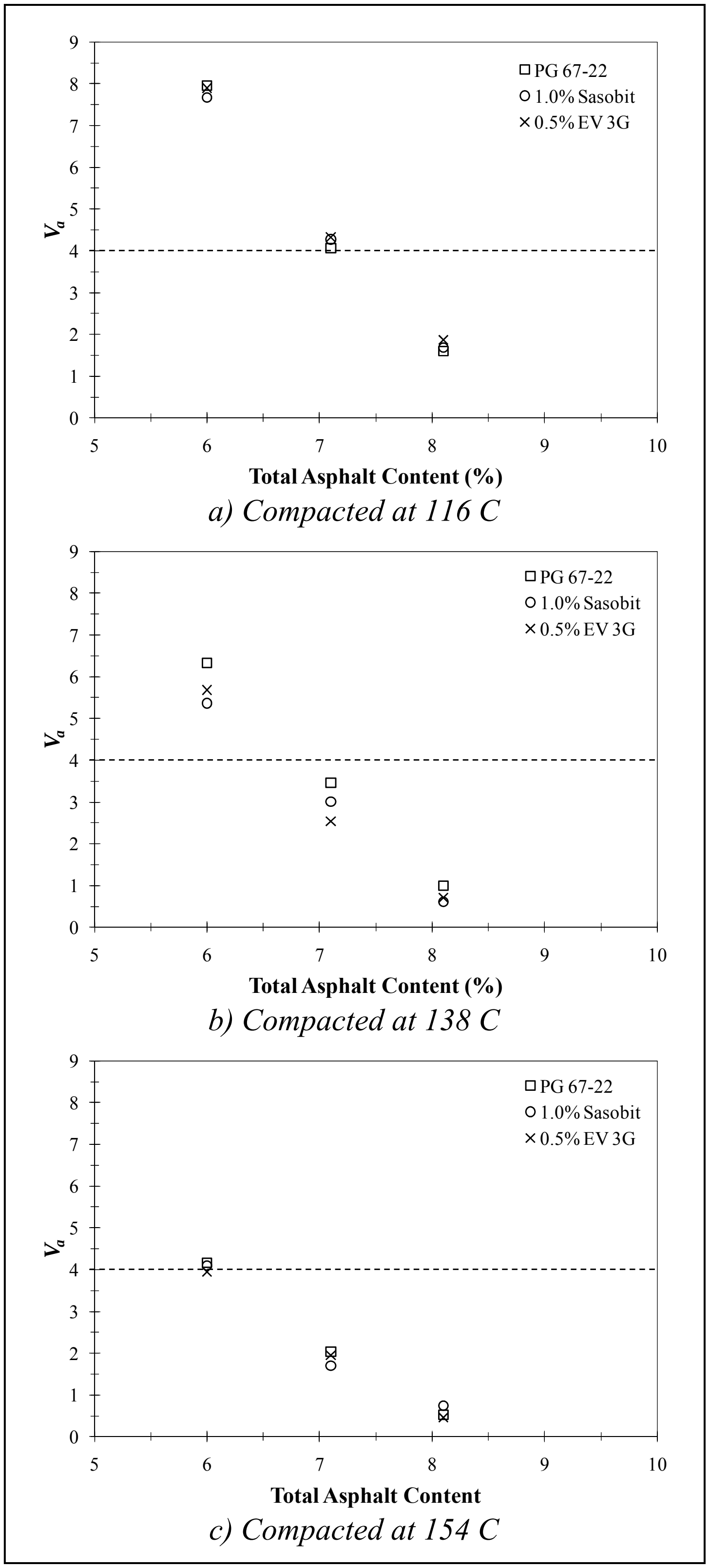


Figure 5.14 Results of *R-1* RAP Compacted to 65 Gyrations

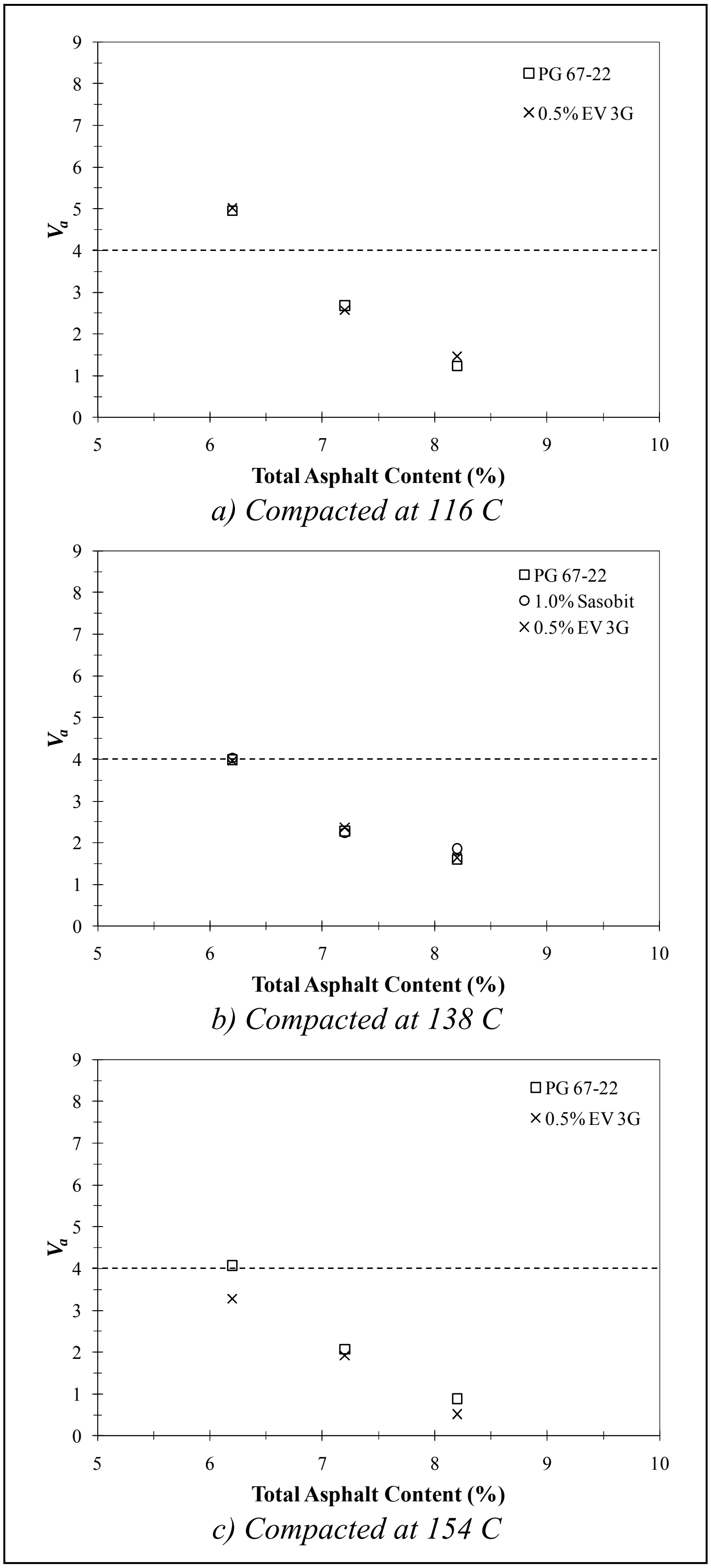
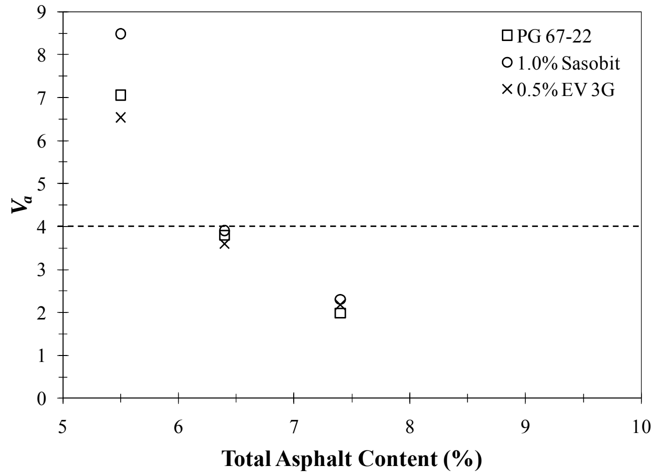
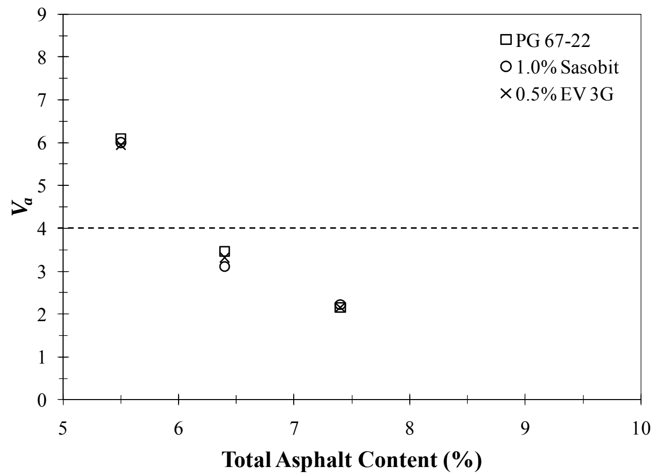


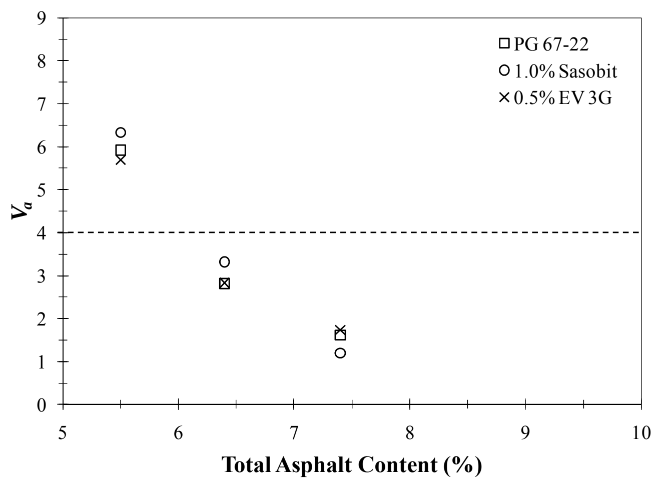
Figure 5.15 Results of *R-2* RAP Compacted to 65 Gyrations



a) Compacted at 116 C

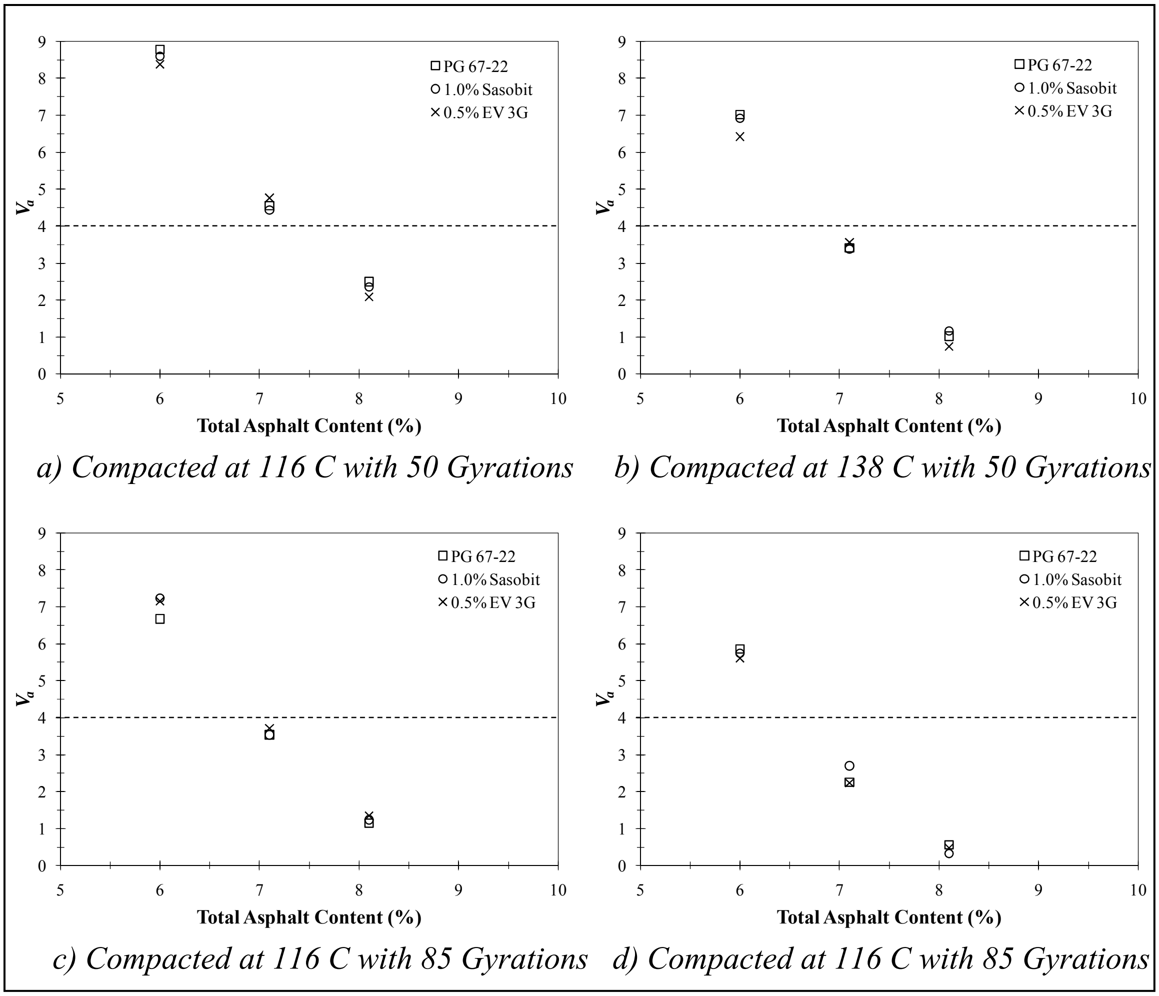


b) Compacted at 138 C



c) Compacted at 154 C

Figure 5.16 Results of *R-3* RAP Compacted to 65 Gyrations



a) Compacted at 116 C with 50 Gyrations *b) Compacted at 138 C with 50 Gyrations*

c) Compacted at 116 C with 85 Gyrations *d) Compacted at 116 C with 85 Gyrations*

Figure 5.17 Results of *R-1* RAP Compacted to 50 and 85 Gyrations

The data in Figures 5.13 through 5.17 is also useful for predicting the effect temperature has on any RAP source. Figure 5.18 illustrates the effect reducing the temperature has on the need for virgin asphalt; 65 gyration data used. The total amount of asphalt needed at 154 C was taken as zero to provide a clearer picture of the additional virgin asphalt demand that could be attributed to temperature reduction. *R-1* had the stiffest binder so it would be expected that more virgin asphalt would be needed as the temperature decreased, which did happen. Interestingly, *R-2* had a higher virgin asphalt demand at 116 C than *R-3*. The key observation is that all three RAP sources were affected by temperature and to different extents. Using RAP in WMA should consider trends of this nature as behaviors at hot mix temperatures (e.g. 154 C) probably won't translate to warm mix temperatures in a consistent fashion over a range of RAP sources.

An extra virgin binder to temperature curve can be generated for a RAP source by compacting eighteen specimens (3 virgin binder contents [0.5%, 1.5%, 2.5%], three temperatures [116, 138, 154 C], and two replicates). For RAP sources available in large quantities, this level of effort would allow much more informed decisions such as whether to use the material in hot mix or warm mix asphalt. RAP sources that have high extra virgin binder at low temperatures would be more appealing in hot mix asphalt, and vice versa, all other factors being equal.

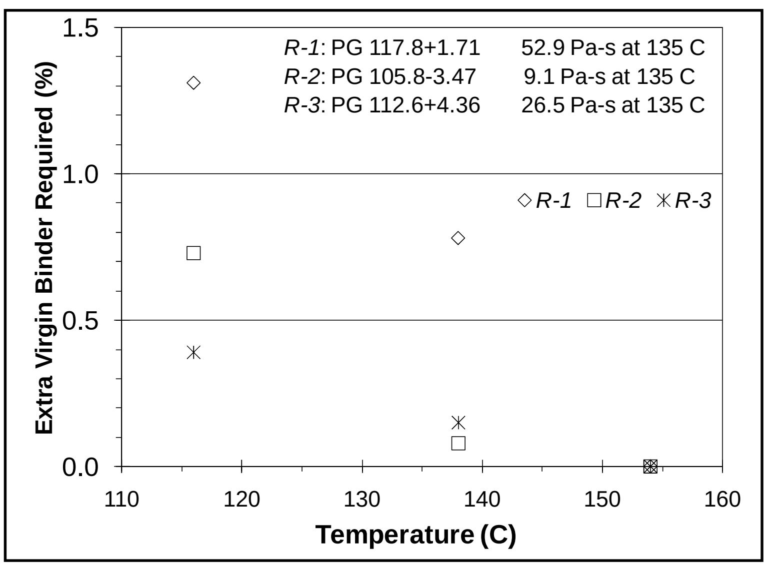


Figure 5.18 Extra Virgin Binder for RAP as a Function of Temperature

Table 5.9 Regression Results From 100% RAP Compacted Testing

RAP ID	Comp Temp (C)	N_{des}	Additive	*n*	Linear Regression[a] Slope	Intercept	R^2	P_{AC}	Avg. P_{AC}
R-1	116	50	None	18	-3.01	26.5	0.97	7.48	
			Sasobit®	15	-2.99	26.2	0.97	7.42	**7.44**
			Evotherm™ 3G	16	-3.00	26.3	0.99	7.43	
		65	None	15	-3.04	26.0	0.99	7.24	
			Sasobit®	16	-2.86	24.7	0.99	7.24	**7.26**
			Evotherm™ 3G	15	-2.88	25.0	0.99	7.29	
		85	None	15	-2.63	22.4	0.99	7.00	
			Sasobit®	15	-2.86	24.2	0.99	7.06	**7.04**
			Evotherm™ 3G	16	-2.77	23.6	0.99	7.08	
	138	50	None	15	-2.86	24.0	0.99	6.99	
			Sasobit®	17	-2.75	23.2	0.99	6.98	**6.97**
			Evotherm™ 3G	16	-2.70	22.7	0.99	6.93	
		65	None	18	-2.54	21.6	0.99	6.93	
			Sasobit®	18	-2.27	19.0	0.99	6.61	**6.73**
			Evotherm™ 3G	15	-2.38	19.8	0.99	6.64	
		85	None	16	-2.54	20.8	0.97	6.61	
			Sasobit®	17	-2.59	21.2	0.99	6.64	**6.60**
			Evotherm™ 3G	18	-2.46	20.1	0.98	6.54	
	154	65	None	6	-1.74	14.5	0.99	6.03	
			Sasobit®	6	-1.61	13.5	0.95	5.90	**5.95**
			Evotherm™ 3G	6	-1.67	13.9	0.99	5.93	
R-2	116	65	None	6	-1.86	16.4	0.98	6.67	**6.67**
			Evotherm™ 3G	6	-1.77	15.8	0.95	6.67	
	138	65	None	6	-1.19	11.2	0.94	6.05	
			Sasobit®	6	-1.09	10.5	0.88	5.96	**6.02**
			Evotherm™ 3G	6	-1.16	11.0	0.95	6.03	
	154	65	None	6	-1.59	13.8	0.98	6.16	**5.94**
			Evotherm™ 3G	3	-1.40	12.0	0.99	5.71	
R-3	116	65	None	6	-2.65	21.4	0.96	6.57	
			Sasobit®	3	-3.22	25.6	0.91	6.71	**6.59**
			Evotherm™ 3G	6	-2.28	18.8	0.95	6.49	
	138	65	None	6	-2.05	17.1	0.95	6.39	
			Sasobit®	6	-1.97	16.5	0.90	6.35	**6.35**
			Evotherm™ 3G	6	-1.98	16.5	0.94	6.31	
	154	65	None	6	-2.24	17.9	0.92	6.21	
			Sasobit®	3	-2.69	20.8	0.97	6.25	**6.20**
			Evotherm™ 3G	6	-2.07	16.7	0.92	6.14	

a) $V_a = m(P_{AC}) + b$

5.9 Prediction of RAP Effective Asphalt

Effective asphalt content is even more problematic than absorbed asphalt content because it is not a constant for a given aggregate blend. Mix design establishes the effective asphalt content, which for a RAP source is more appropriately referred to as surface asphalt since it may not all be effective in a new mixture. The first step in establishing the amount of effective asphalt contributed by the RAP is to be able to decouple surface and absorbed asphalt, which was demonstrated in Sections 5.6 and 5.7.

Ideally, the second step would be to develop a method that could predict the amount of effective asphalt a new mixture would require knowing the aggregate blend and design compactive effort. The MDOT database was used to develop regression equations to estimate the effective asphalt content for a given aggregate blend and design compactive effort. Near perfect correlations were produced when all predictive factors were included, though this is not of practical usefulness since not all predictive factors are known for RAP. Accurate regression equations could not be developed that utilized only known RAP aggregate and asphalt properties. Coefficients of determination (R^2) for the regression equations developed for effective asphalt with only known predictive factors were on the order of 0.30 to 0.35, which isn't useful.

As an alternative to regression equations for effective asphalt prediction, analysis of RAP surface asphalt was conducted in terms of confidence intervals for effective asphalt at each NMAS and level of compactive effort. Population parameters for effective asphalt were determined from the mixture dataset that are provided in Table 5.10. A normal population distribution provided a good fit of the data in most cases. In two cases a few data points were removed to improve the quality of the fit; these cases are described in the Table 5.10 notes. The normal distribution provided a very poor fit for the 50 gyration 19.0 mm NMAS data subset. The poor fit was caused by the existence of two groups of data in the distribution and not by a few extreme values. The mean and standard deviation of the 50 gyration 19.0 mm NMAS data subset are included in Table 5.10 but should be used with caution.

Table 5.10 Effective Asphalt Population Distributions from Database

			Range		**Normal Distribution**		
NMAS	N_{des}	n	**Max**	**Min**	**Fit**	**μ**	**σ**
9.5 mm	85	80	5.57	4.69	Good	5.110	0.1639
	65	75	5.74	4.69	Good	5.238	0.2241
	50	73	6.07	4.83	Good	5.323	0.2751
12.5 mm	85	73	5.11	4.27	Good	4.644	0.1533
	65	45	5.13	4.28	Good[a]	4.696	0.1736
	50	45	5.34	4.36	Excellent	4.844	0.2482
19.0 mm	85	63	4.36	3.80	Good[b]	4.092	0.1285
	65	54	4.81	3.80	Good	4.223	0.2532
	50	51	5.07	3.97	Very Poor	4.313	0.2885

a) Four data points were removed from the MT 12.5 mm NMAS data subset to provide a better fit of the normal probability distribution. The mean value for the data subset was reduced from 4.725 to 4.696 and the standard deviation was reduced from 0.2512 to 0.1736 by this action.

b) Five data points were removed from the HT 19.0 mm NMAS data subset to provide a better fit of the normal probability distribution. The mean value for the data subset was reduced from 4.132 to 4.092 and the standard deviation was reduced from 0.2013 to 0.1285 by this action.

A statistical approach was developed with Table 5.10 as the basis to estimate the amount of RAP surface asphalt that is effective under particular conditions. This approach does not provide a precise estimate of RAP effective asphalt content but it does bound the upper and lower limits of the solution; Figure 5.19 illustrates the approach. For particular combinations of NMAS and level of design compactive effort, the populations of effective asphalt for new mixtures from the MDOT database were assigned normal probability distributions. With the normal population distribution parameters of mean and standard deviation, a 95% confidence interval (C.I.) was constructed for the distribution of effective asphalt contents (Figure 5.19). The upper and lower limits of the confidence interval represent the expected maximum and minimum effective asphalt content possible for a new mixture of a particular type.

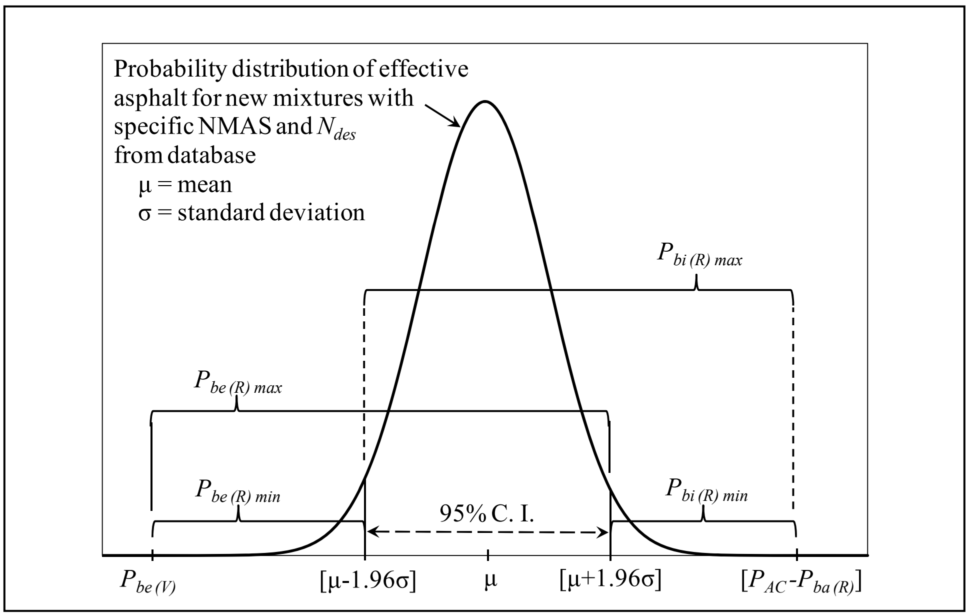


Figure 5.19 Estimation of $P_{be(R)}$ Range

The total amount of surface asphalt in the RAP is defined by Eq. 5.3, where all terms are defined in Figure 5.2. For a given aggregate structure and design compactive effort the amount of effective asphalt can be bounded using Eq. 5.4 using inputs from Table 5.10. The assumption is made that the grade of virgin binder does not appreciably affect compaction characteristics of the aggregate blend; this assumption may not be without consequence but it is routinely made in current practice (e.g. substituting PG 76-22 for PG 67-22 for the same aggregate blend and compactive effort without changing the asphalt content). This assumption allows one to take the distribution of effective asphalt in the database and use the information to make qualitative assessments of RAP surface binder characteristics.

$$P_{bs(R)} = P_{AC} - P_{ba(R)} - P_{be(V)} \quad \text{(Eq 5.3)}$$

$$P_{be\,\min,\max} = \mu \pm 1.96(\sigma) \quad \text{(Eq 5.4)}$$

The amount of effective asphalt in the RAP can be bounded by utilizing Eq. 5.4 and knowing the amount of effective virgin binder added to the RAP source to achieve adequate compaction; the result is Eq. 5.5. By having an estimate of the total RAP surface asphalt from Eq. 5.3 and the boundaries of the effective RAP surface asphalt from Eq. 5.5, the amount of ineffective RAP surface asphalt can also be bounded as shown in Eq. 5.6.

$$P_{be(R)\min,\max} = \left[\mu \pm 1.96(\sigma)\right] - P_{be(V)} \quad \text{(Eq 5.5)}$$

$$P_{bi(R)\min,\max} = P_{bs(R)} - P_{be(R)\min,\max} \quad \text{(Eq 5.6)}$$

Prior to using these equations it should be understood that minimum values can be calculated to be negative and those cases should be interpreted as zero. Negative values can occur because distributions are part of the calculations. An example of a condition that would lead to a negative value is a RAP source with a relatively high total asphalt content where nearly all of the surface asphalt is effective.

5.10 Evaluation of RAP Effective Asphalt Prediction

Estimates of $P_{ba(R)}$ determined in Table 5.8 with average G_{se} values and $z_{\alpha/2}$ of zero, effective asphalt population parameters from Table 5.10, and average estimated P_{AC} values determined in Table 5.9 were used to calculate ranges of $P_{be(R)}$ and $P_{bi(R)}$. Results are given in Table 5.11 for each combination of RAP source, compaction temperature, and compactive effort. The $P_{bs(R)}$ values given in Table 5.11 for each RAP source increase slightly as $P_{be(V)}$ decreases because all terms are defined on a mixture mass basis; the mass of RAP surface asphalt does not change. The Table 5.11 data shows that some of the RAP surface asphalt is very likely ineffective in some conditions and that the behavior is condition dependent.

Table 5.11 Summary of RAP Effective Asphalt Calculations

RAP ID	**Compaction Temp (C)**	N_{des}	P_{AC}	$P_{be(V)}$	$P_{bs(R)}$	**Range of $P_{be(R)}$** Min	Max	**Range of $P_{bi(R)}$** Min	Max
R-1	116	50	7.44	2.05	4.76	2.73	3.81	0.95	2.03
		65	7.26	1.86	4.77	2.94	3.82	0.95	1.83
		85	7.04	1.63	4.78	3.16	3.80	0.98	1.62
	138	50	6.97	1.56	4.78	3.22	4.30	0.48	1.56
		65	6.73	1.30	4.80	3.50	4.38	0.42	1.30
		85	6.60	1.16	4.81	3.63	4.27	0.54	1.18
	154	65	5.95	0.48	4.83	4.32	5.20	0.00	0.51
R-2	116	65	6.67	1.13	4.97	3.67	4.55	0.42	1.30
	138	65	6.02	0.44	5.00	4.36	5.24	0.00	0.64
	154	65	5.94	0.36	5.00	4.44	5.32	0.00	0.56
R-3	116	65	6.59	1.67	4.36	2.69	3.37	0.99	1.67
	138	65	6.35	1.42	4.37	2.94	3.62	0.75	1.43
	154	65	6.20	1.26	4.38	3.10	3.78	0.60	1.28

It should be understood that the maximum and minimum values of $P_{be(R)}$ and $P_{bi(R)}$ given in Table 5.11 estimate the range of possible values with a 95% level of confidence but that there is a fairly high probability that the actual value is near the middle of the range. For example, *R-1* at 116 C and 50 gyrations has a range of 2.73 to 3.81% for $P_{be(R)}$ but the actual value is fairly likely to be between 3.0 and 3.5%. Likewise the range of $P_{be(R)}$ is 0.95 to 2.03 but the actual value is fairly likely to be between 1.2 and 1.8%. For a particular level of compactive effort both the minimum and maximum estimates of $P_{be(R)}$ increase as the compaction temperature is increased indicating that a greater proportion of RAP asphalt is contributing to compaction as the temperature increases.

R-1 had a considerable amount of ineffective surface asphalt at 116 C, though no ineffective asphalt could be detected at 154 C. *R-2* had a moderate amount of ineffective asphalt at 116 C, though no ineffective asphalt could be detected at 138 or 154 C. *R-3* showed ineffective surface asphalt at all temperatures, which was somewhat surprising relative to *R-1* and *R-2*. *R-3* had the lowest RAP asphalt content and its binder properties were intermediate compared to *R-1* and *R-3*. A likely cause of the differing behavior for *R-3* is that it is a multiple source sample so the Table 5.10 12.5 mm NMAS population parameters may not be applicable as the material could be a combination of different mixtures which would affect the effective asphalt content. Additionally, the gradation of a multiple source sample wouldn't necessarily be representative, which is evidenced by the gradation of *R-3* (e.g. sand ratio of 71).

The analysis presented in this section has shown the database approach to evaluating RAP surface asphalt has some appealing characteristics, but that it also has some limitations. Estimates of this nature are valuable for determining the best use of any given RAP source. They also provide estimates of effective RAP asphalt that haven't been available in literature. Limitations are described in the following paragraph.

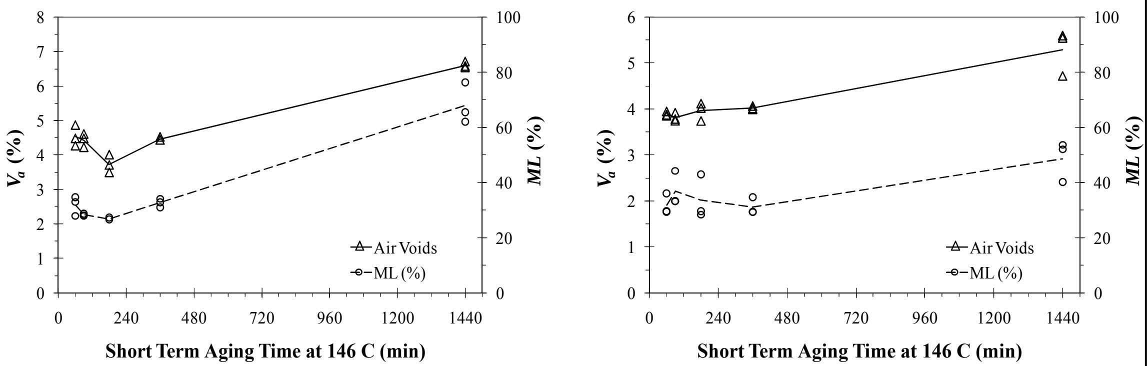
The approach appears to work reasonably well for single source RAP samples (i.e. *R-1* and *R-2*) in terms of the ability to estimate the amount of effective and ineffective surface asphalt and to characterize the effect of temperature on the RAP surface asphalt. Based on the data available, the effectiveness of the approach to estimate effective and ineffective surface asphalt for a multiple source RAP sample (i.e. *R-3*) is questionable, though the approach was able to capture the effect of temperature on the multiple source sample. At present, it is not recommended to use the effective asphalt estimation approach in this section unless the RAP sample was obtained from a single source. Another limitation to the database approach is that gradation changes due to milling are not represented in a direct manner. The fines content of RAP exceeds that of new mixtures (Figure 5.5*a* and 5.6*c*). Gradation changes prevent the Table 5.10 distributions from fully representing the distribution of RAP properties.

5.11 Performance Results for 100% RAP Mixtures

This section presents 100% RAP mixture performance test results. Properties of the mixtures were given in Table 3.5. The raw data is found in Doyle and Howard (2010b). The data in this section is used to provide a perspective of RAP only mix properties relative to high RAP mixes, not for considering 100% RAP use in service.

Relative effects of different heating times were investigated with *9.5-100/RM-1* as discussed in Section 4.3.1.1. The data is summarized in Figure 5.20. For specimens compacted with same compactive effort, air voids and mass loss follow the same trend.

For specimens compacted to target air voids, mass loss does not vary much except to increase for the longest aging time of 1440 minutes; however it was not possible to fully compact those specimens to the target air voids. Results appear to indicate the very high asphalt stiffness of the RAP overwhelms any moderate increase in virgin binder stiffness due to short term laboratory conditioning.



a) Specimens Compacted to N_{des} b) Specimens Compacted to Target V_a

Figure 5.20 Results for *9.5-100/RM-1* Mixture Relative Heating Experiment

Cantabro testing was performed on *SGC* compacted specimens as described in Section 4.2.4 for designed 100% RAP mixes (Table 5.12). *R-3* had the highest *ML* followed by *R-1* and then by *R-2*. *ML* for the designed 100% RAP mixtures was noticeably higher than the 4 to 7% loss observed by Celauro et al. (2010) for 0% RAP.

Table 5.12 Cantabro Results for 100% RAP Mixtures

Mixture ID	***n***	**Avg. Air Voids (%)**	**Avg. *ML* (%)**
9.5-100/RM-1	3	4.6	31.8
9.5-100/RM-2	3	4.1	17.0
12.5-100/RM-3	3	4.6	33.7

BBR testing of the three designed 100% RAP mixes was performed at four test temperatures, test results are presented in Figure 5.21. The isotherms are generally rather flat which indicates potentially poor relaxation properties compared to mixes with softer binder. Somewhat surprisingly, stiffness of the different RAP sources at these temperatures is not as different as might be expected given their variation in total asphalt content and observed differences in compaction behavior. At -24 C and -18 C temperatures the mixture with *R-1* RAP is slightly stiffer than the mixture with *R-2* RAP; stiffness of the mixture with the *R-3* RAP source was variable at these test temperatures. At -12 C and -06 C test temperatures the opposite trend is observed with the *R-1* and *R-2* RAP sources (i.e. *R-2* was stiffer than *R-1*); the *R-3* RAP mixture had similar stiffness to the *R-1* mixture at these test temperatures.

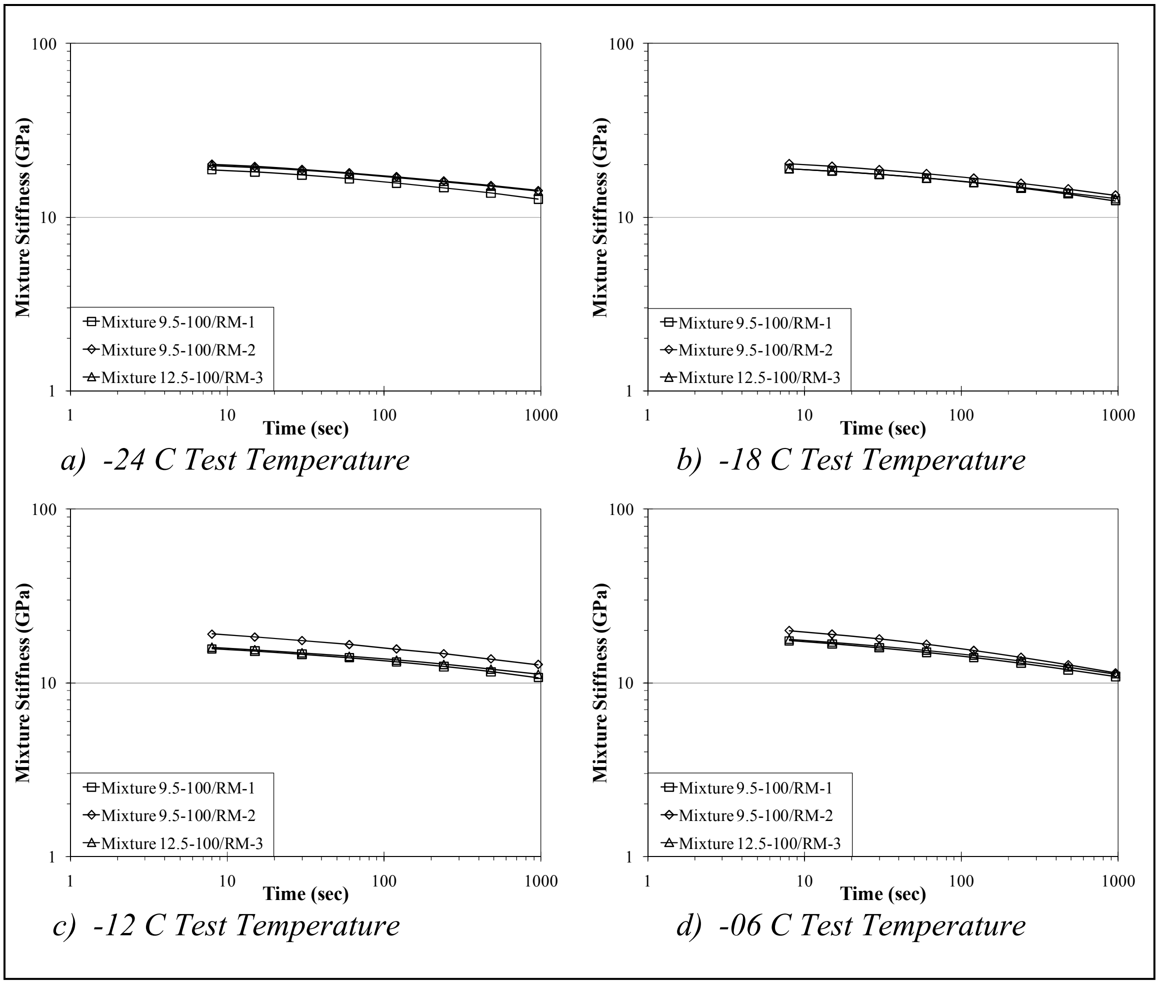


Figure 5.21 *BBR* Data for 100% RAP Mixtures

Tensile strength properties of 100% RAP mixtures at low and moderate temperatures were determined according to Section 4.2.2. Results are presented in Table 5.13. *R-2* RAP source is much stronger than *R-1* RAP source at low temperatures.

Table 5.13 IDT Results for 100% RAP Mixtures

	Test Temperature and Average Tensile Strength (kPa)				
Mixture ID	**-06 C**	**-12 C**	**-18 C**	**-24 C**	**+25 C**
9.5-100/RM-1	2430	1808	1549	1952	2751
9.5-100/RM-2	3597	4660	2874	3476	2990

Note: Each value is average of two test results.

Rutting resistance of 100% RAP was evaluated by *APA* and PURWheel dry protocol testing. *APA* rut testing was performed on *SGC* compacted specimens as described in Section 4.2.6 at nominal air void levels of 7 and 10%; Table 5.14 summarizes the results. In all cases total rutting was very low; less than 1.5 mm for 7% air voids and less than 2.5 mm for 10% air voids.

PURWheel dry protocol rut testing was performed on *LAC* compacted specimens of designed 100% RAP mixtures as described in Section 4.2.7; raw data is located in Tables A.1, A.3 and A.5. The data is summarized in Table 5.15. PURWheel dry protocol average total rut depths for 100% RAP mixtures were generally low. For mixtures *9.5-100/RM-1* and *12.5-100/RM-3* the average total rut depths were the same, about 4.5 mm. Mixture *9.5-100/RM-2* exhibited an average total rut depth of about 6.5 mm, which is slightly higher than results for the other 100% RAP mixtures. Both *APA* and PURWheel test results indicate that 100% RAP mixes with virgin binder are rut resistant but they do rut a measurable amount.

Table 5.14 *APA* Results for 100% RAP Mixtures

Mixture	**Avg.**	**Rut Depth (mm)**		**Linear Rutting Rate**[a]			**Power Law**[b]		
ID	**V_a (%)**	**2000**	**8000**	**Slope (10^{-6})**	**Intercept**	**R^2**	***a***	***b***	**R^2**
9.5-100/RM-1	6.7	0.8	1.1	57	0.69	0.98	0.082	0.293	0.95
	9.6	1.2	2.0	128	0.96	0.99	0.059	0.389	0.98
9.5-100/RM-2	7.1	0.8	1.2	132	0.75	0.98	0.029	0.458	0.99
	10.1	1.2	2.3	184	0.85	0.99	0.025	0.504	0.99
12.5-100/RM-3	7.0	1.0	1.5	89	0.81	0.98	0.065	0.351	0.97
	9.8	0.8	1.5	100	0.68	0.99	0.047	0.381	0.99

a) Linear rutting rate regression analysis is based on averaged data between 2000 and 8000 cycles.
b) Power law regression analysis is based on averaged data and Eq. 2.4.

Table 5.15 PURWheel Dry Protocol Test Results for 100% RAP Mixtures

Mixture ID	V_a (%)[a]	Rep	Rut Depth Pass	Rut Depth mm	Linear Rutting Rate[b] Slope (10^{-4})	Linear Rutting Rate[b] Intercept	Linear Rutting Rate[b] R^2	Power Law[c] *a*	Power Law[c] *b*	Power Law[c] R^2
9.5-100/RM-1	9.5	1-L	20 k	4.4	100	1.91	0.98	0.102	0.380	0.98
		1-R	20 k	5.0	100	2.31	0.97	0.173	0.334	0.86
	9.8	2-L	20 k	3.3	90	1.55	0.97	0.087	0.367	0.99
		2-R	20 k	5.3	200	2.42	0.98	0.134	0.371	0.99
Average	**9.7**	**---**	**20 k**	**4.5**	**123**	**2.05**	**---**	**0.124**	**0.363**	**---**
9.5-100/RM-2	10.0	1-L	20 k	8.7	300	2.84	0.98	0.073	0.475	0.95
		1-R	20 k	7.1	200	2.84	0.96	0.052	0.494	0.92
	9.6	2-L	20 k	5.2	200	1.79	0.97	0.039	0.500	0.96
		2-R	20 k	5.4	200	1.85	0.97	0.031	0.524	0.96
Average	**9.8**	**---**	**20 k**	**6.6**	**225**	**2.33**	**---**	**0.049**	**0.498**	**---**
12.5-100/RM-3	9.5	1-L	20 k	4.1	100	1.84	0.97	0.099	0.376	0.91
		1-R	20 k	5.7	200	2.78	0.95	0.091	0.423	0.92
	11.2	2-L	20 k	3.4	100	1.40	0.97	0.029	0.490	0.88
		2-R	20 k	5.3	200	1.73	0.97	0.029	0.527	0.96
Average	**10.4**	**---**	**20 k**	**4.6**	**150**	**1.94**	**---**	**0.062**	**0.454**	**---**

a) Specimen air voids correlated to *AASHTO T 331*.
b) Linear rutting rate regression analysis is based on averaged data between 2000 and 20000 passes.
c) Power law regression analysis is based on averaged data and Eq. 2.4.

Moisture damage resistance of 100% RAP was evaluated by *TSR* and PURWheel wet protocol testing. For designed 100% RAP mixtures (*9.5-100/RM-1, 9.5-100/RM-2,* and *9.5-100/RM-3*) *TSR* moisture susceptibility testing was performed on *SGC* compacted specimens as described in Section 4.2.5. The results are summarized in Table 5.16. The *R-1* RAP source had an acceptable *TSR* value (i.e. >80%), while the *R-2* and *R-3* RAP sources did not. Air voids for mixture *9.5-100/RM-2* were higher than specified by the test method, but the mixture was not re-tested.

PURWheel wet protocol testing was performed as described in Section 4.2.7 for all designed 100% RAP mixtures; raw data is located in Tables A.2, A.4 and A.6. Figure

5.22 presents results of wet and dry protocol PURWheel tests for mixture with *R-1*; three of the four wet test specimens exhibited evidence of moisture damage in the data and early test termination. The wet test specimen that did not terminate early exhibited a higher rate of rutting than dry test specimens.

Table 5.16 *TSR* Results for 100% RAP Mixtures

Mixture	Conditioned Set			Un-Conditioned Set		
ID	**Avg. V_a (%)**	**Sat (%)**	**S_t (kPa)**	**Avg. V_a (%)**	**S_t (kPa)**	***TSR* (%)**
9.5-100/RM-1	6.4	68	2008	6.5	2229	90
9.5-100/RM-2	8.4[a]	62	1680	8.4	2603	65
12.5-100/RM-3	7.4	62	1383	7.5	1959	71

a) Air voids slightly high but not re-tested.

Figure 5.23 presents results of wet and dry protocol PURWheel testing for mixture with *R-2*; only one of the four wet test specimens exhibited evidence of moisture damage and premature test termination. The other wet test specimens did exhibit somewhat higher rates of rutting than did the dry test specimens. Figure 5.24 presents wet and dry protocol PURWheel test results of mixture with *R-3*. Overall three of the four wet test specimens exhibited moisture damage and early test termination. The two replicates with the worst performance came from the same *LAC* compacted slab.

Table 5.17 summarizes PURWheel wet protocol results for designed 100% RAP mixes. The *LAC* compacted replicate slab specimen seen in Figure 5.24 that performed poorly had higher air voids than its companion slab. Examination of the results for mixture *9.5-100/RM-1* in Table 5.17 and Figure 5.22 reveals that the *LAC* slab in that test set with higher air voids outperformed the companion slab with lower voids. The results bring into question the impact of air voids in evaluating performance of *LAC* slab specimens in the PURWheel wet protocol test. Several tests of one mixture with varying air voids is needed before specific statements could be made. Interestingly, results of *TSR* testing on designed 100% RAP mixtures do not agree with PURWheel wet protocol test results for the same mixture.

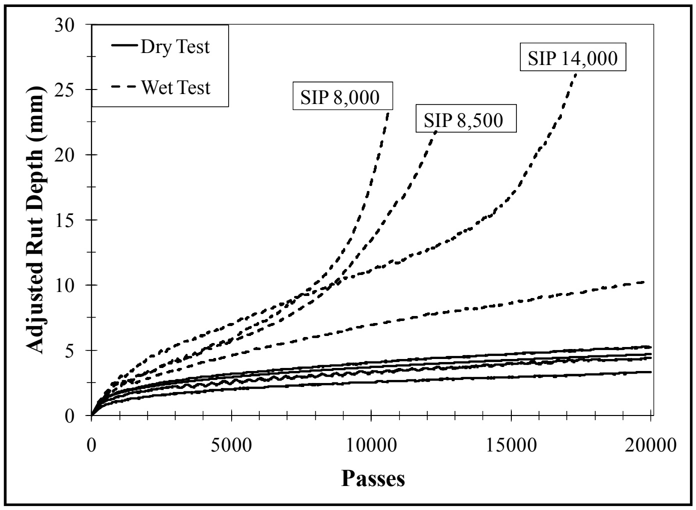


Figure 5.22 PURWheel Test Results for Mixture *9.5-100/RM-1*

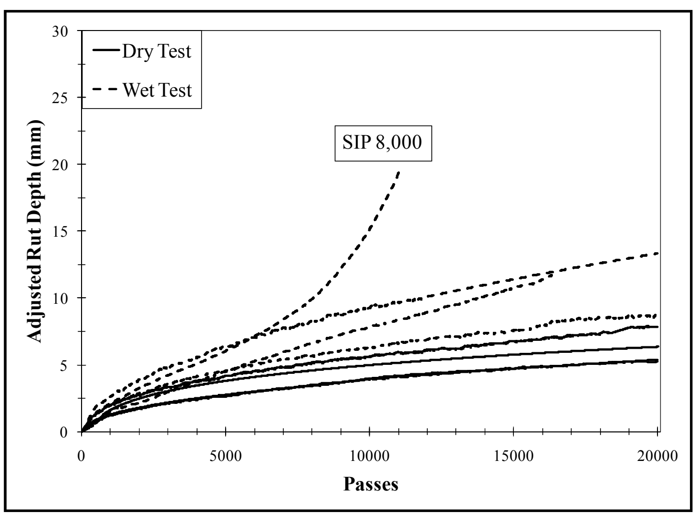


Figure 5.23 PURWheel Test Results for Mixture *9.5-100/RM-2*

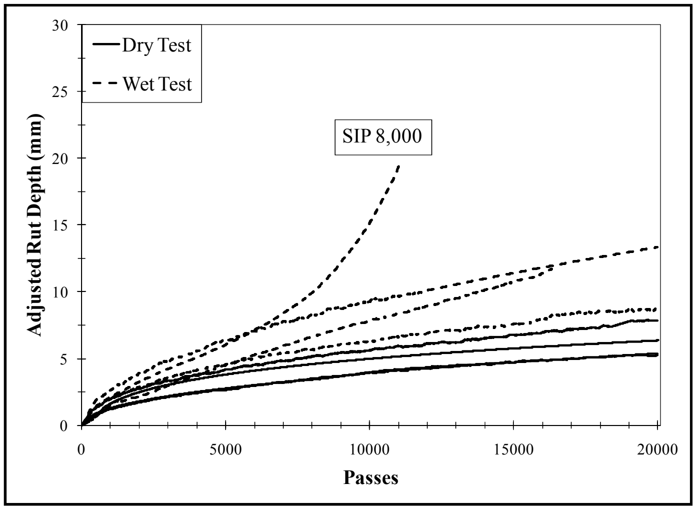


Figure 5.24 PURWheel Test Results for Mixture *12.5-100/RM-3*

Table 5.17 Summary of PURWheel Wet Test Results for 100% RAP Mixtures

				Termination		Visual Assessment		
Mixture ID	**V_a (%)[a]**	**Rep**	**SIP**	**Pass**	**(mm)**	**Bare Agg.**	**Loose Agg.**	**Crack**
9.5-100/RM-1	9.8	1-L	8,500	10,620	23.5	Yes	Yes	Yes
		1-R	8,000	12,314	21.8	Yes	Yes	Yes
	11.3	2-L	14,000	17,312	26.1	Yes	Yes	Yes
		2-R	None	20 k	10.3	Yes	No	Yes
Average	**10.6**	**---**	**12,625**	**15,062**	**20.4**	---	---	---
9.5-100/RM-2	8.8	1-L	None	20 k	8.8	Yes	No	No
		1-R	None	16,412	11.7	Yes	No	No
	10.6	2-L	None	20 k	13.3	Yes	No	No
		2-R	8,000	11,490	19.8	Yes	Yes	Yes
Average	**9.7**	**---**	**17,000**	**16,976**	**13.4**	---	---	---
12.5-100/RM-3	8.7	1-L	16,000	18,130	16.2	Yes	Yes	No
		1-R	None	20,000	6.2	Yes	No	No
	11.5	2-L	3,000	3,800	29.5	Yes	Yes	Yes
		2-R	4,000	4,174	18.1	Yes	Yes	Yes
Average	**10.1**	**---**	**10,750**	**11,526**	**17.5**	---	---	---

Note: When no SIP was observed, a value of 20,000 passes was used to calculate the average SIP.

a) Specimen air voids correlated to *AASHTO T 331*.

5.12 Summary of RAP Characterization

Test data was presented in this chapter that provided means to characterize RAP in the context of its temperature dependency. Testing eighteen compacted 100% RAP specimens using the method presented could provide the effect temperature has on virgin binder demand. Test data also showed that RAP does not absorb additional virgin binder and that measurement of G_{se} on RAP coated with virgin binder is an effective approach.

Use of conventionally measured G_{sb} values for extracted RAP aggregate to calculate absorbed RAP asphalt was shown to yield values that were almost certainly incorrect for the RAP sources evaluated. An approach was developed to estimate $P_{ba(R)}$ of Mississippi RAP sources that does not require measurement of G_{sb} on extracted RAP aggregate. The approach cannot pick exact values but does result in estimates of $P_{ba(R)}$ believed to be more reasonable than current practice in certain situations. Use of a large data set encompassing all agency activities for a period of more than five years makes this approach unique. The methodology should be easily implementable by any state DOT or governing entity since the effort to sort the historical data and use it to develop regression equations is reasonable and this chapter can be used as a guide.

The relative effectiveness of RAP surface asphalt was evaluated for a variety of compaction conditions. The approach coupled distributions of effective asphalt contents Mississippi determined from recent historical practice with compaction of RAP and added virgin binder. Effective binder replacement is believed to be a better way to view RAP than total binder replacement and the approach utilized herein allowed for estimation of amounts of effective binder for RAP to be made at varying compaction conditions. The estimates of effective RAP binder are not without flaws but appear to be a reasonable technique to address a major problem with the use of high RAP quantities.

Performance testing of 100% RAP mixes provides a baseline for comparison to control and high RAP WMA mixes in later chapters. The data highlights differences between RAP sources. For example, current practice would treat *R-1* and *R-2* RAP sources nearly the same (nearly same asphalt content, same NMAS and comparable gradations), yet their performance is notably different especially durability and moisture susceptibility.

CHAPTER 6

AIRFIELD SURFACE MIXTURES

6.1 Overview of Airfield Surface Mixtures

This chapter presents test results for airfield surface mixtures. Mixture properties were provided in Section 3.5.2. The experimental design was discussed in Section 4.3.2.

6.2 Recovered Binder Properties

Table 6.1 provides asphalt content and PG pass/fail temperatures of recovered binder for mixtures *12.5-0/AM-1* to *12.5-50/AM-12*. Binder properties were only evaluated for the limestone aggregate (lowest absorption) mixes since the same RAP was used for all mixes and the effects of virgin aggregate type were assumed to be minor. Testing on recovered asphalt was performed assuming the recovered asphalt was already aged and hence there was no need to test after running the RTFO or the PAV tests.

Going from 0% RAP to 25% RAP shows that the high temperature property increased by approximately 8 degrees. This increase in RAP only changed the low temperature properties by approximately 3 degrees. Going from 0% RAP to 50% RAP changed the high temperature properties by approximately 20 degrees whereas it only changed the low temperature properties by approximately 8 degrees. Hence, adding RAP favorably affects the high temperature properties (provides more rut resistance) much more than it adversely affects the low temperature properties (provides less resistance to thermal cracking). Potentially the aged binder may have reduced the temperature susceptibility resulting in an increase in PG grade at high temperatures and less change at low temperatures; however specific conclusions cannot be made with the available data and further research would be needed to investigate this issue.

Table 6.1 Extraction and Recovered Asphalt Data for Mixtures *12.5-0/AM-1* to *12.5-50/AM-12*

RAP (%)	**Mixture ID**	**Total Asphalt Content (%)**	**Pass / Fail Temperature (C)** High	Intermediate	Low
0	*12.5-0/AM-1*	4.7	73.0	19.3	-27.5
	12.5-0/AM-2	4.8	70.3	20.7	-27.0
	12.5-0/AM-3	4.7	66.0	16.4	-30.1
	12.5-0/AM-4	5.2	68.8	18.3	-27.7
		Range of Temperatures (C)	7.0	4.3	3.1
25	*12.5-25/AM-5*	5.1	80.5	23.3	-24.9
	12.5-25/AM-6	5.2	78.0	24.7	-23.7
	12.5-25/AM-7	5.0	73.4	17.9	-28.2
	12.5-25/AM-8	5.2	78.8	24.0	-25.4
		Range of Temperatures (C)	7.1	6.8	4.5
50	*12.5-50/AM-9*	Not Available	85.8	28.7	-21.9
	12.5-50/AM-10	5.7	88.2	29.1	-22.0
	12.5-50/AM-11	6.1	96.0	31.5	-14.3
	12.5-50/AM-12	5.7	88.2	29.2	-20.7
		Range of Temperatures (C)	10.2	2.8	7.7

Note: The high pass / fail temperature was determined based on a 2.20 kPa min criteria.

WMA mixes with 0% and 25% RAP have decreased high temperature properties relative to the HMA mixes with 0% and 25% RAP; among the WMA mixes the Evotherm™ mixes (mixes *AM-3* and *AM-7*) have values approximately 4 degrees lower than the Sasobit® or foamed WMA mixes. Surprisingly the exact opposite trend is seen with 50% RAP mixes; the WMA mixes have increased high temperature properties relative to the HMA mixes and the Evotherm™ mix (mix *AM-11*) has a value approximately 4 degrees higher than the Sasobit® and foamed WMA mixes. The trend observed for the 0 and 25% RAP mixes is likely due to reduced binder aging at lower mix aging temperature; however that does not explain the 50% RAP properties. The more dramatic changes in asphalt properties of the Evotherm™ 3G mixes relative to other WMA mixes seen with 0 and 25% RAP could potentially be explained by the chemistry of the Evotherm™ 3G additive itself which may have a softening effect on the asphalt but no specific conclusions can be drawn with the available data.

In general the spread of values of high temperature properties for a given amount of RAP is seven degrees or more and would seem to be adequate to reasonably predict the best and worst performing mixes for a given gradation and RAP content. For 0 and 25% RAP, binder data predicts that HMA should rut the least and Evotherm™ 3G the most; for 50% RAP Evotherm™ 3G is predicted to rut the least and HMA the most.

Low temperature binder property values for 0, 25, and 50% RAP WMA mixes with Sasobit® and foam are within about 1 degree of the 0, 25, and 50% RAP HMA mixes which indicates that the Sasobit® and foam had only a slight effect on low temperature properties for a given amount of RAP in the mix. The Evotherm™ 3G WMA mixes with 0 and 25% RAP reduced the low temperature property by approximately 3 degrees relative to the 0 and 25% RAP HMA mixes indicating that the low temperature properties are possibly improved relative to the HMA. For the 50% RAP mixes the exact opposite trend is observed for the Evotherm™ 3G mix; the low temperature property of the Evotherm™ 3G mixes is almost 8 degrees higher than the HMA mix indicating that low temperature properties may be adversely affected.

The unusual results for WMA mixes with 50% RAP, especially the Evotherm™ 3G mix, are thought to be at least partly due to difficulty in fully extracting the asphalt from these mixes. Note in Table 3.10 that all of the asphalt was extracted for the Evotherm™ 3G mix but that not all of the asphalt was successfully extracted for Sasobit® and foam WMA mixes (5.7% asphalt extracted for mixes *AM-10* and *AM-12* and their design total asphalt contents are 6.1%). The asphalt that could not be extracted is most likely aged asphalt from the RAP; if all of the RAP asphalt had been extracted for the Sasobit® and foam mixes their stiffness would likely be increased. Furthermore, the effects of the solvent extraction and recovery process cannot be fully quantified and may also have contributed to the unusual results observed.

6.3 Cantabro Durability Data

Results of durability testing for the twenty-four airfield mixes are presented in Figure 6.1. Cantabro testing was performed according to Section 4.2.4. The effect of RAP addition to the mixes is apparent; the mass loss is increased (i.e. durability is decreased) as additional RAP is incorporated into the mixes. No statistically significant differences were found between the hot mix control and the three warm mix technologies or between the warm mix technologies for any of the six gradations. The effect of RAP on mass loss was found to be statistically significant.

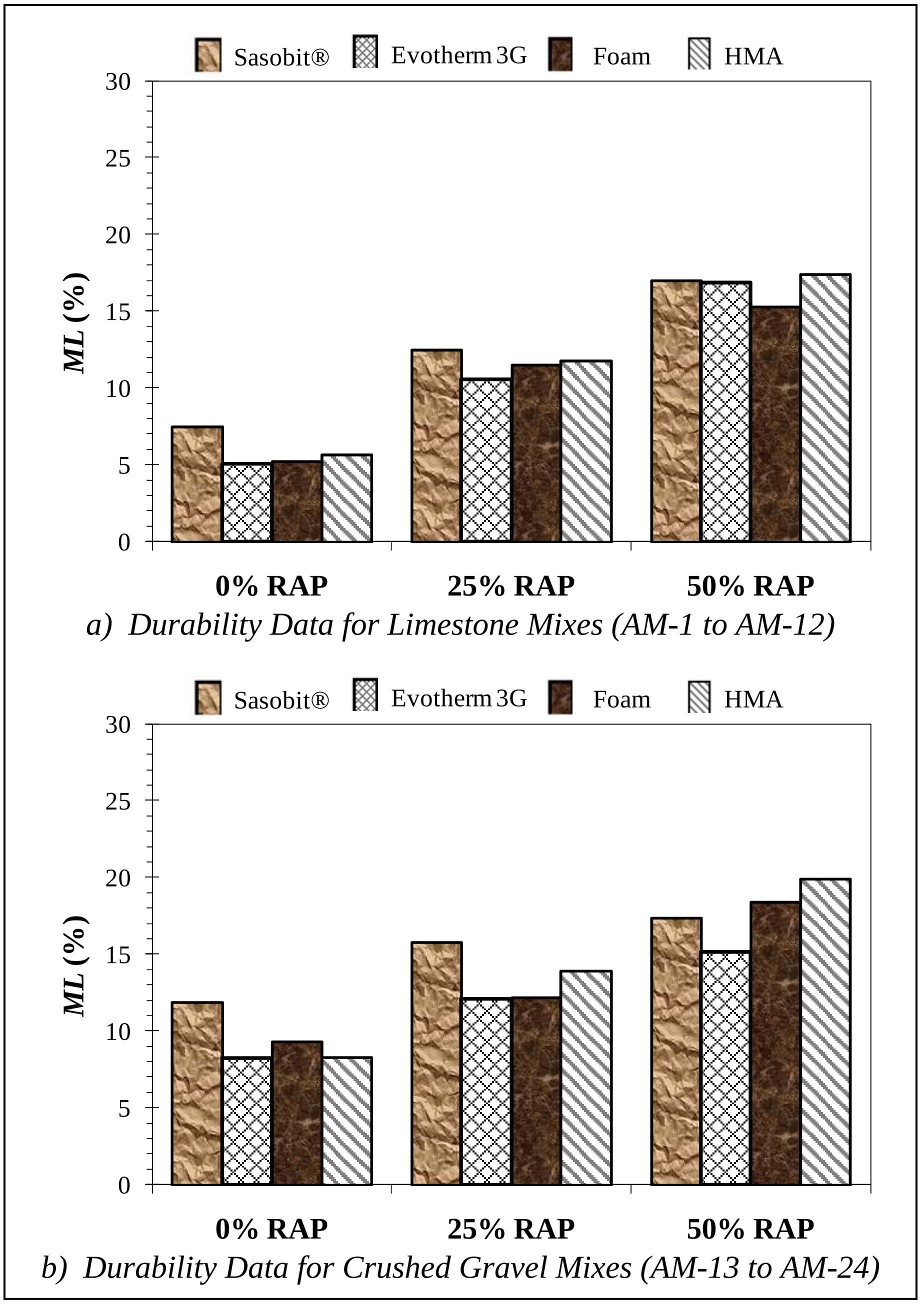


Figure 6.1 Airfield Surface Mixture Durability Results

To better interpret the effects of RAP on durability, statistical comparisons were made using the Tukey multiple comparison procedure; the results are summarized in Table 6.2. Gradations with the same Tukey letter grouping are not significantly different from each other; all gradations in a letter grouping are significantly different than those in other letter groupings. For limestone 0% RAP (LS-1), 25% RAP (LS-2, and 50% RAP (LS-3) gradations are all three significantly different from one another. The same trend held true for the gravel gradations. The limestone and gravel 0% RAP gradations (LS-1 and GR-1) were significantly different from one another; this implies that in the absence of RAP, virgin aggregate type has a significant effect on durability results. However the limestone and gravel 25% RAP gradations (LS-2 and GR-2) were not significantly different from one another. The limestone and gravel 50% RAP gradations (LS-3 and GR-3) were also not significantly different from one another. Based on the data, the presence of 25% and 50% RAP in the mixture appears to overwhelm effects due to virgin aggregate type. As the gradations are all quite similar, it is likely that the contribution of RAP asphalt to the recycled mixes is the dominating factor leading to this result.

To investigate durability test sensitivity to asphalt content changes, testing was performed on specimens from the mix design process with a variety of trial asphalt contents (Figure 6.2). The x-axis (AC ratio) is the asphalt content of the specimen divided by the design asphalt content for the mixture. The y-axis (*ML* ratio) is the mass loss result of the specimen divided by the average mass loss of the mix at the design asphalt content. No discernable differences are seen between gradations or virgin aggregate types. For asphalt contents higher than the design, durability resistance is not adversely affected. However at asphalt contents lower than design, the durability resistance of the mixes begins to decrease noticeably. While this test has not been proven to be related to durability, it is believed that it is a good test for ranking the mixtures.

Table 6.2 Tukey Multiple Comparison Test of Mass Loss for Airfield Mixtures

Gradation Number	**Mean Mass Loss (%)**	**Tukey Grouping**			
LS-1	5.8	A			
LS-2	11.6			C	
LS-3	16.7				D
GR-1	9.4		B		
GR-2	13.5			C	
GR-3	17.7				D

Note: Experimental treatments with the same letter grouping are not statistically significantly different at the 5% significance level.

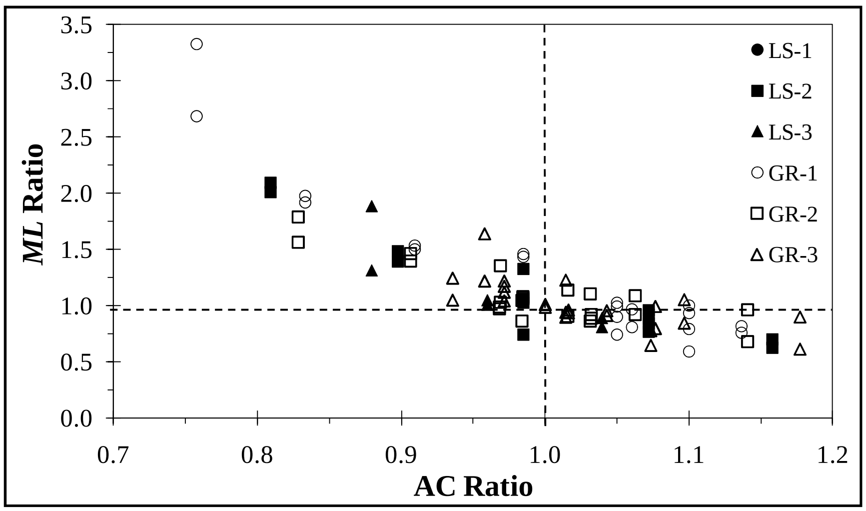


Figure 6.2 Relationship of Durability Results to Design Asphalt Content

6.4 *BBR* Data

BBR mixture beam testing was performed as described in Section 4.2.3. Figures 6.3 and 6.4 present mixture stiffness results at 60 seconds when tested at -6 C and -12 C respectively. Statistically, no significant differences were found between the hot mix control and the three warm mix technologies or between the warm mix technologies for any of the six gradations at either test temperature. However when RAP was included as part of the mixture, in nearly all cases at both test temperatures the warm mix technologies were less stiff than the hot mix. This is likely due to the decreased amount of aging of the binder when mixing using the WMA temperatures.

The effect of RAP on low temperature stiffness values was found to be statistically significant. To better interpret the data two Tukey multiple comparison tests were performed, one for each test temperature; all the results are given in Table 6.3. For a specific temperature, gradations with the same Tukey letter grouping are not significantly different from each other; all gradations in a letter grouping are significantly different from those in other letter groupings. For both test temperatures the two 0% RAP gradations are not significantly different indicating that in the absence of RAP, virgin aggregate type did not appear to affect mixture stiffness. When RAP is included in the mixtures, the results are chained together and no specific conclusions can be drawn; the only significant differences (at either test temperature) are between LS-2 and GR-3. Based on the results, the increase in mixture stiffness from 0% to 25% RAP is generally significant however the subsequent increase in stiffness from 25% to 50% RAP is generally not significant or as large. This is in contrast to the results reported by Li et al. (2008) who found little difference between 0% and 20% RAP mixtures but a large reduction in low temperature fracture resistance for 40% RAP mixtures.

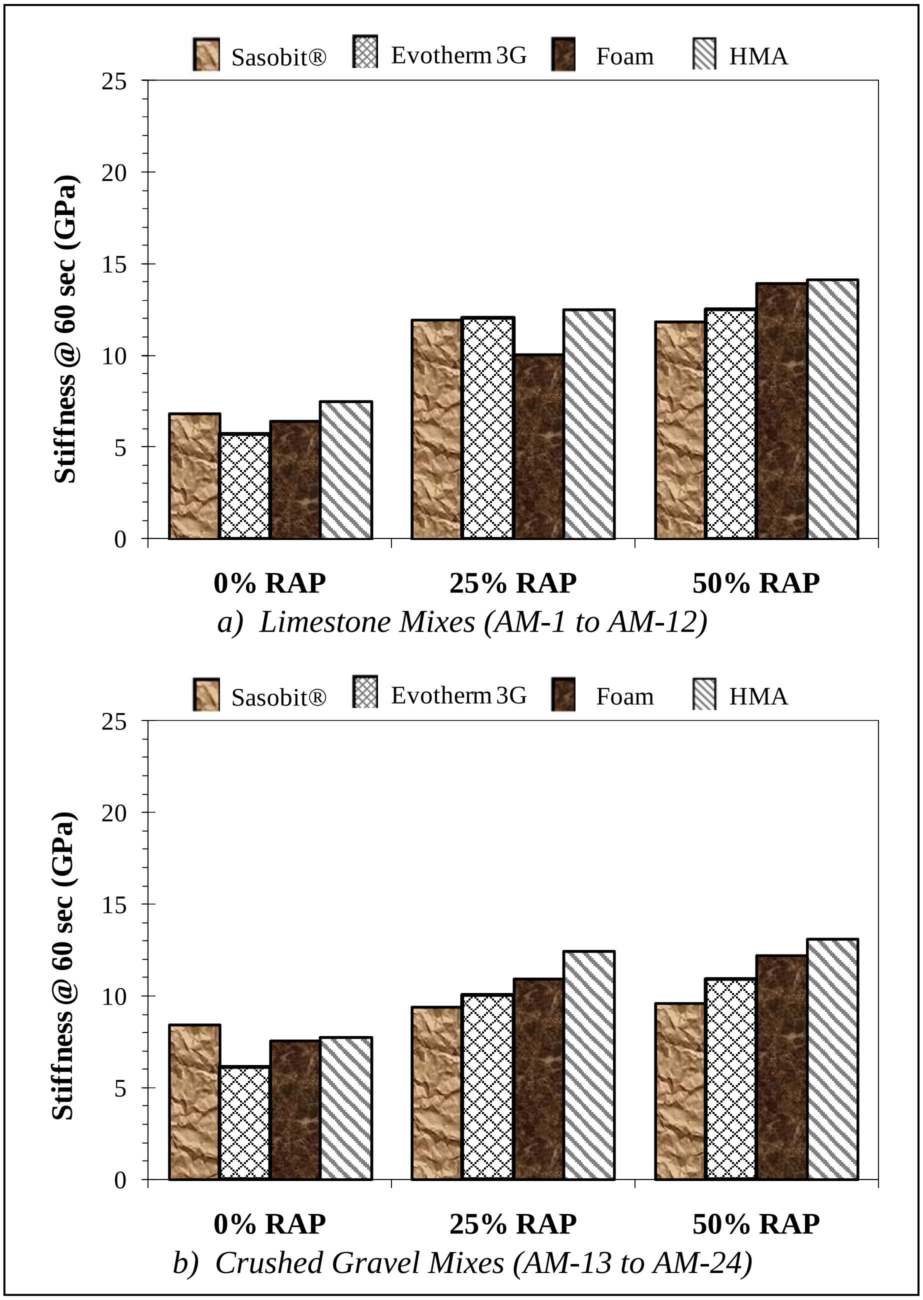


Figure 6.3 Airfield Surface Mixture Stiffness at 60 sec Tested at -6 C

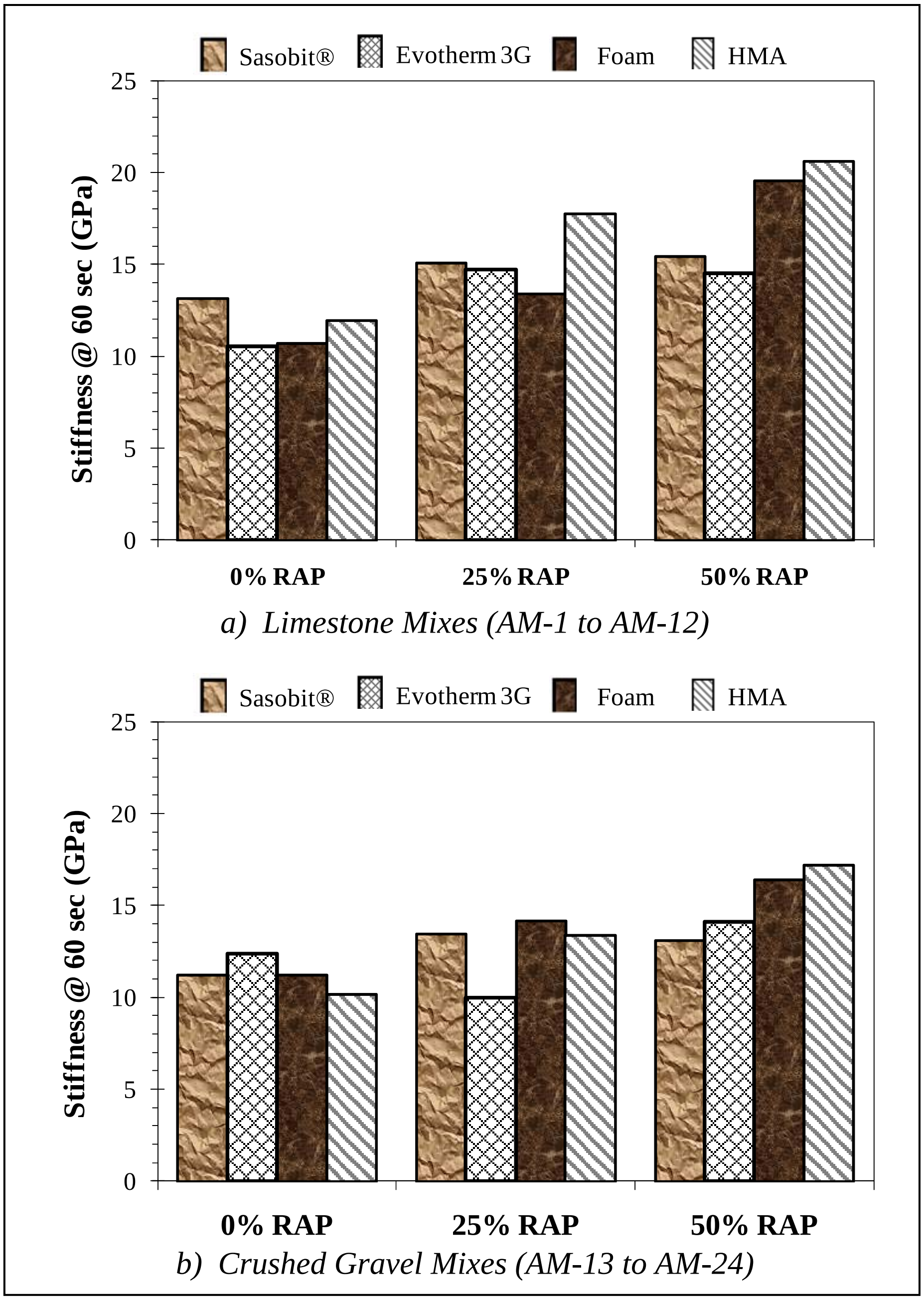


Figure 6.4 Airfield Surface Mixture Stiffness at 60 sec Tested at -12 C

Table 6.3 Tukey Multiple Comparison Test of Airfield Mixture Stiffness at 60 sec

Test Temperature	**Gradation Number**	**Mean Stiffness (GPa)**	**Tukey Grouping**		
-6 C	LS-1	6.6	A		
	LS-2	11.6		B	C
	LS-3	13.1			C
	GR-1	7.5	A		
	GR-2	10.7		B	
	GR-3	11.4		B	C
-12 C	LS-1	11.6	D		
	LS-2	15.2		E	F
	LS-3	17.5			F
	GR-1	11.2	D		
	GR-2	12.8	D	E	
	GR-3	15.2		E	F

Note: Experimental treatments with the same letter grouping are not statistically significantly different at the 5% significance level.

For the limestone mixtures, correlations between mixture stiffness and low temperature binder grade (from Table 6.1) were generally poor as shown in Figure 6.5*a* although the -6 C test data was slightly better than the -12 C test data. Correlations between mixture stiffness and binder stiffness at -12 C test temperature were also generally poor (Figure 6.5*b*). This result is aligned with the evidence presented by Huang et al. (2005) for limited mechanical blending of RAP asphalt and virgin binder.

In addition, the Table 6.1 binder data seems to indicate that the 50% RAP Evotherm™ 3G mixes would be much stiffer than the other 50% RAP mixes; this is not the case for the mixture data presented in Figures 6.3 and 6.4. These poor correlations of mixture properties and binder properties provide evidence of the problems that can be associated with using only binder data to study mixtures with high RAP contents.

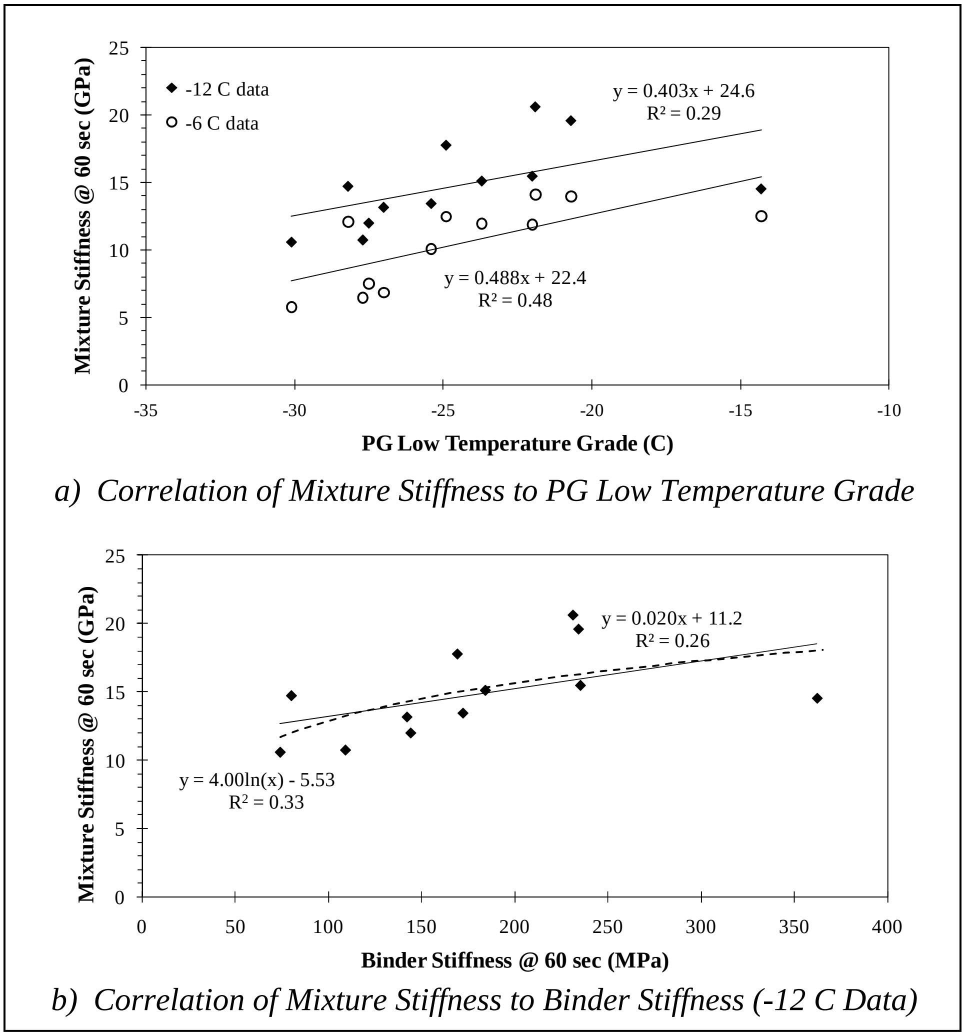
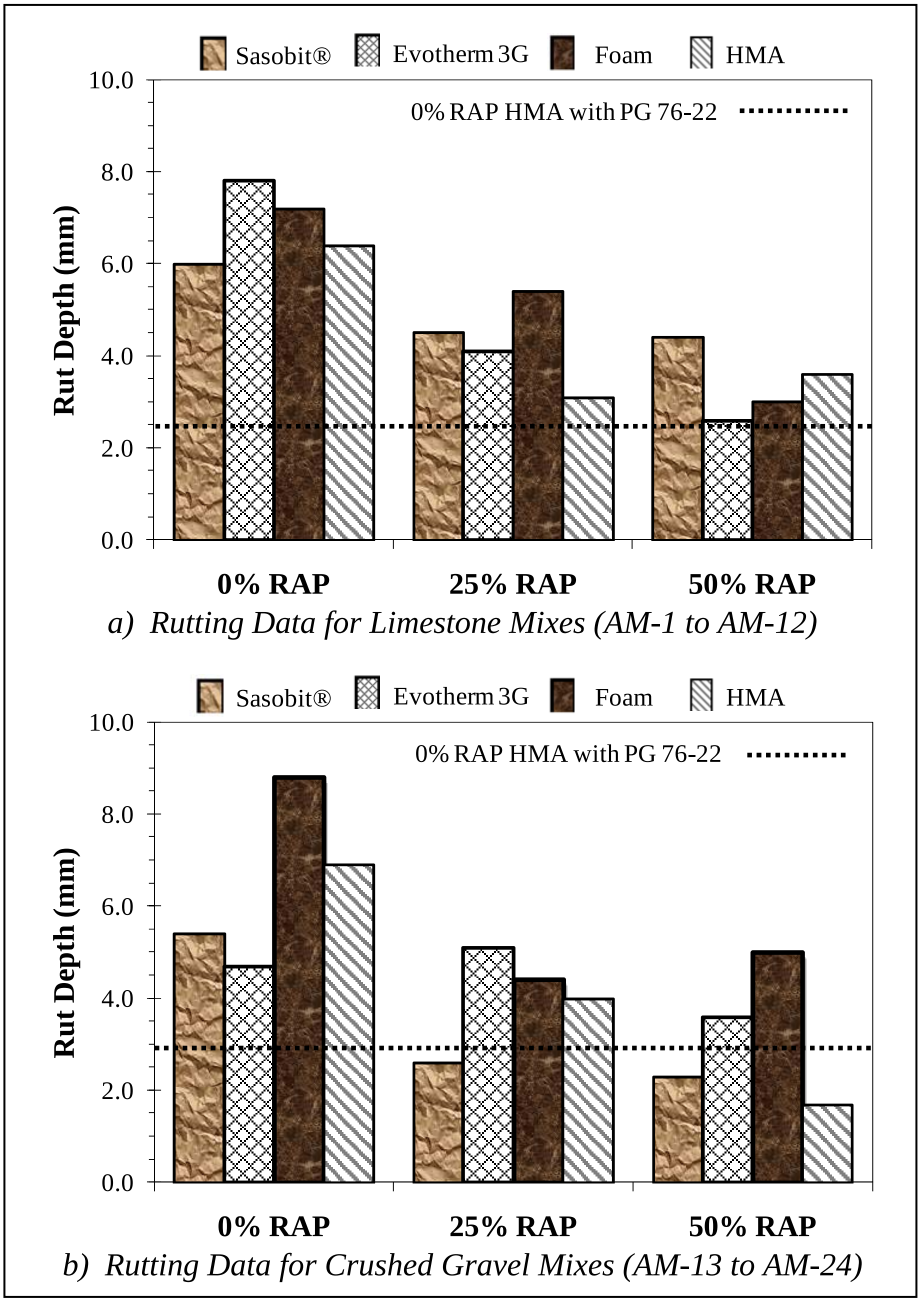


Figure 6.5 Correlations of Low Temperature Mixture Stiffness

6.5 Rutting Data

Figure 6.6 provides rut depth test results for all twenty-four mixtures. An 8 mm pass/fail criteria has been suggested by Brown et al. (2001). All limestone mixtures rutted less than 8 mm, indicating they should have adequate rutting resistance. All gravel mixes with exception of the 0% RAP foam rutted less than 8 mm; the foam specimen rutted 8.8 mm which does not greatly exceed the pass/fail criteria. One possible explanation for the rutting behavior of the 0% RAP foamed gravel is the design effective asphalt content is 0.3% higher than the other two 0% RAP WMA mixes. Overall, no rutting problems were observed for the high RAP with WMA.



a) *Rutting Data for Limestone Mixes (AM-1 to AM-12)*

b) *Rutting Data for Crushed Gravel Mixes (AM-13 to AM-24)*

Figure 6.6 Airfield Surface Mixture *APA* Rutting Test Results

PG 76-22 (modified with radial SBS polymer) binder was used in place of PG 67-22 binder for mixtures *AM-1* and *AM-13* (0% RAP HMA controls) to provide a reference for comparison to the effects from inclusion of RAP. A 0% RAP aggregate with a polymer modified binder is considered a premium mixture that would be relatively expensive and is a good reference to compare with other mixtures. The primary purpose of using polymer modified binder is for rutting resistance. This reference using a mixture with modified asphalt is provided in Figure 6.6 with a horizontal line.

The data generally showed that the mixtures being evaluated had higher rutting than the modified asphalt mixture used for comparison. However, at 50% RAP, the amount of rutting with the mixtures is approximately equal to that for the control modified asphalt mixture. Generally there is less rutting as the amount of RAP increased. There is no mix type that clearly has a higher degree of rutting. In some cases the HMA ruts more and in other cases one of the WMA mixes has more rutting.

The binder data provided in Table 6.1 was used to determine if a correlation existed with limestone specimen rut data. Binder data suggests that HMA and Sasobit® mixtures should rut less with 0% RAP than Evotherm™ 3G and foam, which they did. Figure 6.7 plots the PG high temperature grade of mixes from Table 6.1 versus total rut depth. This correlation clearly shows that the resulting recovered binder grade had a reasonably high correlation (R^2 = 0.73) with the amount of rutting in the APA.

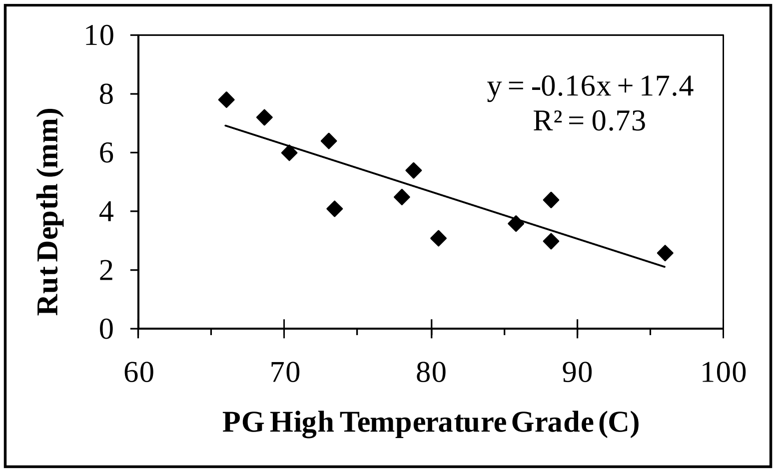


Figure 6.7 Correlation of PG High Temperature Grade to Rut Depth

Table 6.1 binder data correctly predicted which mixture would rut the least for a given amount of RAP in the mixture in three of the six possible cases (only the three limestone mixes were correctly predicted) and only correctly predicted which mixture would rut the most for a given amount of RAP in the mixture in one of the six possible cases (gravel mixture with 25% RAP). The results indicate that while the high temperature binder property was be able to discern general rutting trends as the amount of RAP is increased it was a relatively poor predictor of specific best or worst performing mixes for a given level of RAP. This is evidence of the problems that can be associated with using only binder data to study mixtures with high RAP contents.

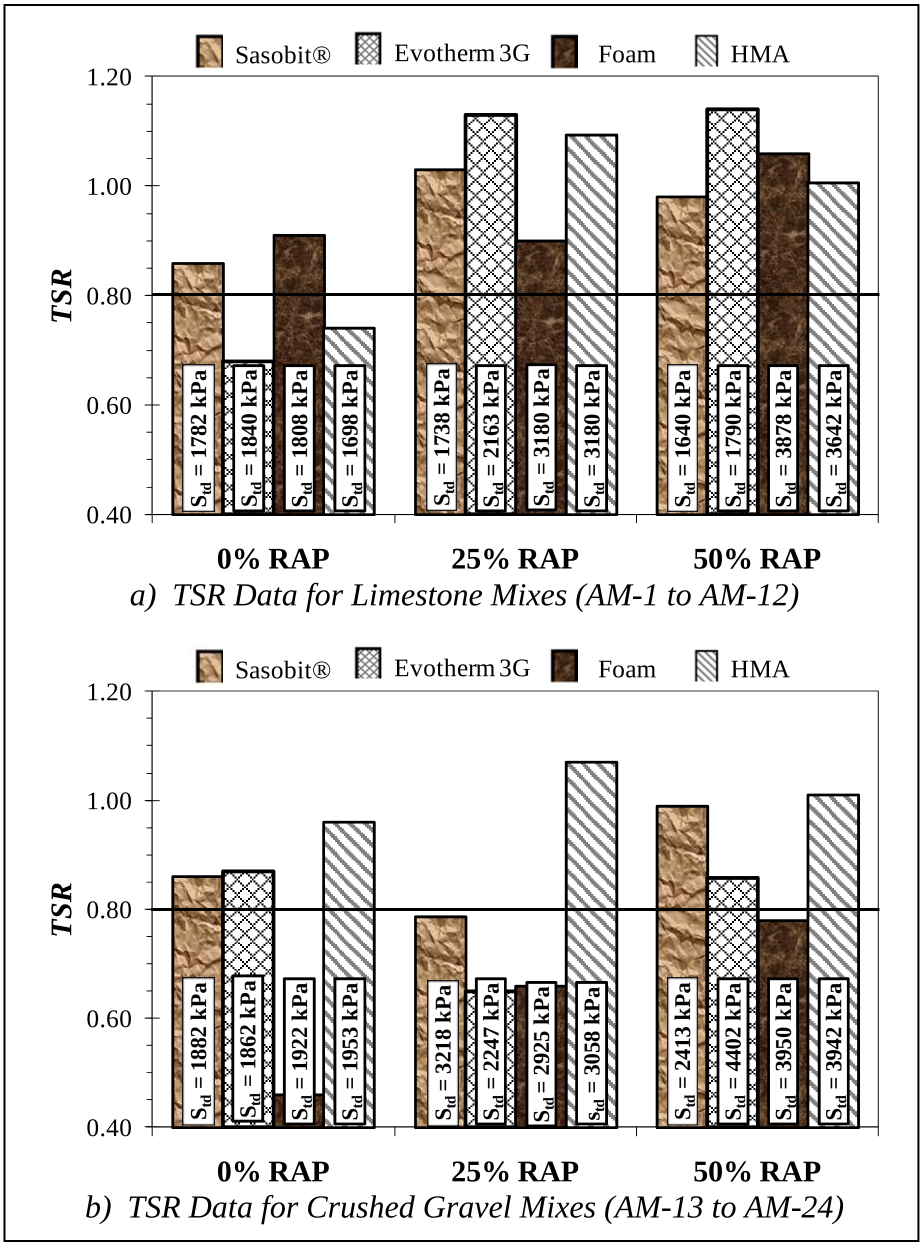
6.6 Moisture Damage Data

Figure 6.8 presents *TSR* test results. Note dry tensile strength (S_{td}) is provided for each mixture in the figure. There is some scatter in the *TSR* data but some general observations can be made. Increasing RAP contents generally provided a higher *TSR* value and higher tensile strengths, although not substantially in some cases. Inclusion of 25% RAP improved the moisture resistance compared to 0% RAP for six of the eight cases. Inclusion of 50% RAP improved the moisture resistance compared to 0% RAP in seven of the eight cases (the gravel with Evotherm™ 3G decreased slightly but *TSR* was still above 0.80). The increased tensile strength results align with the results of Li et al. (2008); however the generally increased *TSR* values do not.

For 0% RAP mixes the gravel aggregate generally performed better than the limestone aggregate with exception of the foamed gravel mix. For 25% RAP mixes the limestone aggregate performed much better than the gravel aggregate; all the 25% RAP WMA mixes with gravel aggregate performed poorly. Mixes with 50% RAP all performed acceptably (*TSR* > 0.80 was considered acceptable) except for the foamed gravel mix.

The HMA mixes and the Sasobit® WMA mixes both generally performed well; five of the six HMA mixes performed acceptably as did five of the six Sasobit® mixes. Four of the six Evotherm™ mixes performed acceptably. The foamed asphalt mixtures often had lower *TSR* values, especially for the gravel; only the three limestone mixes of the six foamed mixes performed acceptably and all of the foamed gravel mixes had a *TSR* value less than 0.80.

Some gravel mixtures have had a history of stripping; all the gravel mixes tested contained 1% hydrated lime to prevent stripping. Visible stripping still occurred in some cases. The mix containing the lowest retained stability was the virgin gravel mix with foamed asphalt shown in Figure 6.9 (*TSR* = 0.46). However, when RAP was added to this mixture stripping was reduced notably for the foamed mix (foamed gravel with 50% RAP had *TSR* = 0.78).



a) TSR Data for Limestone Mixes (AM-1 to AM-12)

b) TSR Data for Crushed Gravel Mixes (AM-13 to AM-24)

Figure 6.8 Airfield Surface Mixture *TSR* Test Results

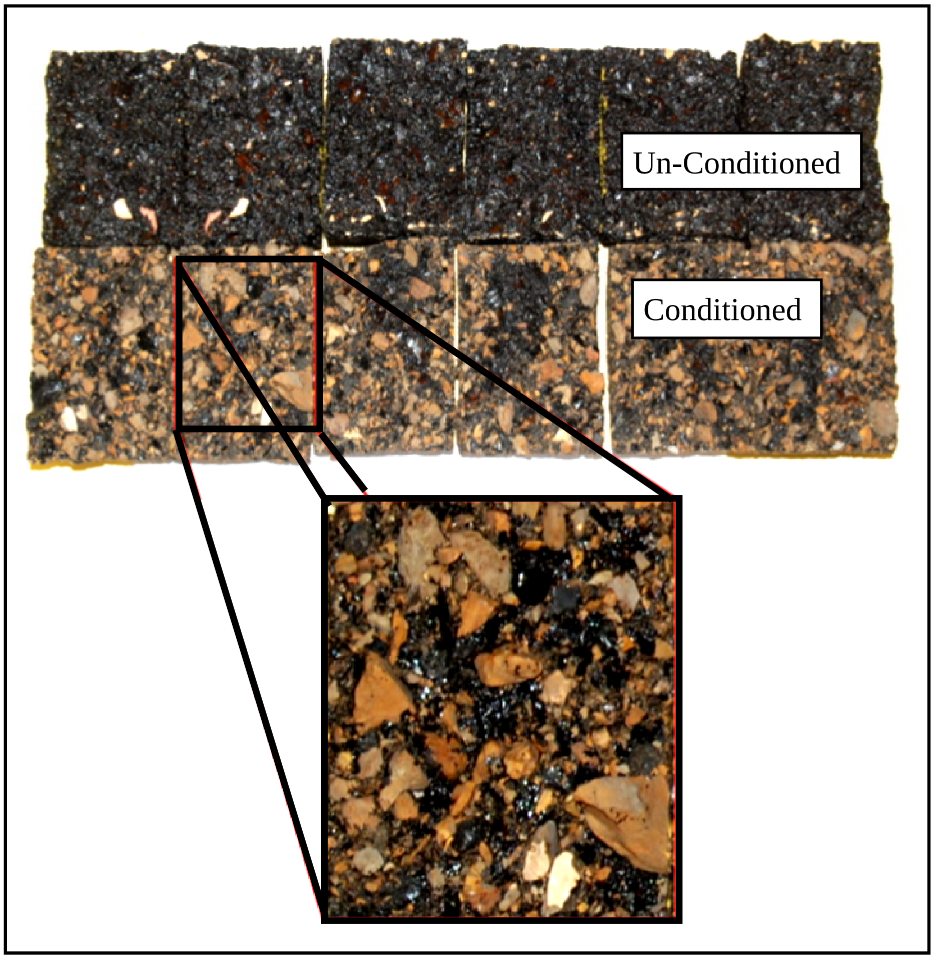


Figure 6.9 Aggregate Stripping on Mixture *12.5-0/AM-16* Conditioned Specimens

CHAPTER 7

HIGHWAY SURFACE MIXTURES

7.1 Overview of Highway Surface Mixtures

This chapter presents results from investigating highway surface mixtures. Properties of mixtures tested are located in Section 3.5.3, and experimental program details are located in Section 4.3.3. The raw data for this chapter is located in Doyle and Howard (2010b). The results are organized in two broad categories, 0 and 15% RAP control mixture results in Section 7.2 and 25 and 50% RAP recycled mixture results in Section 7.3. Subsections within each category organize data by mixture performance and analysis category. Discussion and results interpretation is provided in Chapter 10.

7.2 Control Highway Surface Mixture Results

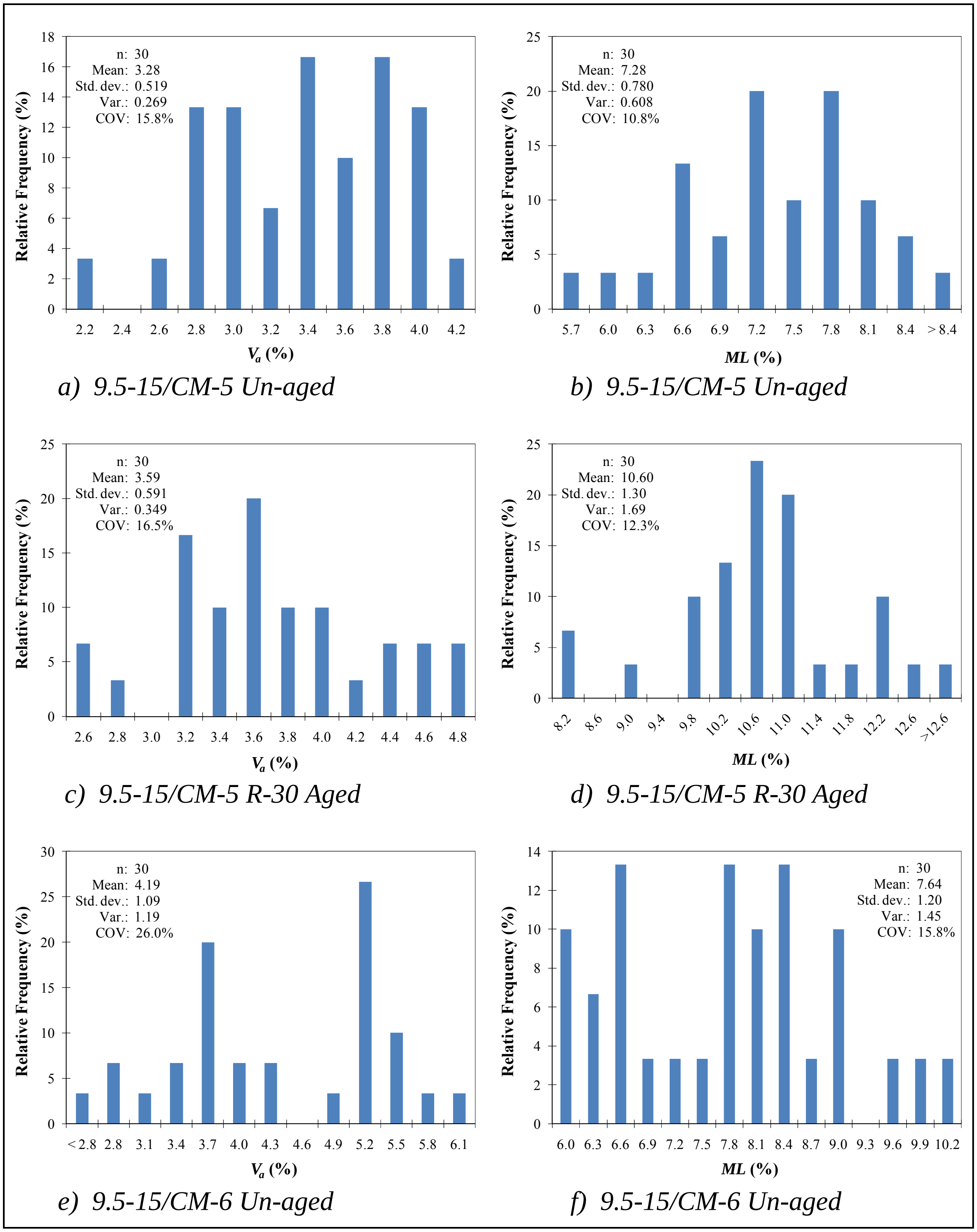
7.2.1 Cantabro Durability Data

7.2.1.1 Single Aggregate Blend

To investigate variability of the Cantabro test method, three sets of thirty plant mixed QC specimens were tested: 1) *9.5-15/CM-5* un-aged; 2) *9.5-15/CM-5* aged according to *R-30*; and 3) *9.5-15/CM-6* un-aged. All specimens were tested for G_{mb} before aging, and the mix design G_{mm} was used to calculate air voids. The mixtures had identical aggregate blends from the same asphalt plant, and the only difference was the design compactive effort (*9.5-15/CM-5* was 50 gyrations and *9.5-15/CM-6* was 65 gyrations) which caused the design asphalt contents to differ by 0.2%.

Figure 7.1 presents relative frequency histograms for air voids and mass loss. To evaluate mass loss effects due to minor variation in design asphalt content an un-equal variance *t*-test was performed for un-aged mixtures *9.5-15/CM-5* and *9.5-15/CM-6* (Table 7.1). The analysis indicated no significant difference in mean mass loss between the two mixtures. To evaluate the effects of aging on mass loss an un-equal variance *t*-test was performed on the data for mixture *9.5-15/CM-5*. The analysis indicated a significant difference in mean mass loss due to *R-30* aging (Table 7.1). *R-30* aging was chosen in favor of *MT-85* aging after testing three replicates of *9.5-15/CM-5* with both protocols and observing a higher mass loss with *R-30* (10.5% loss) than with *MT-85* (9.6% loss).

The data collected seems to indicate variation in air voids even within a moderate range affects mass loss. In Figure 7.1, lower air voids variation corresponded with lower mass loss variation for all three specimen sets as evidenced by the COV data presented. Specimens with lower air voids have correspondingly higher VFA for given mixture proportions which would seem to result in a specimen that is more resistant to mass loss.



a) 9.5-15/CM-5 Un-aged *b) 9.5-15/CM-5 Un-aged*

c) 9.5-15/CM-5 R-30 Aged *d) 9.5-15/CM-5 R-30 Aged*

e) 9.5-15/CM-6 Un-aged *f) 9.5-15/CM-6 Un-aged*

Figure 7.1 Relative Frequency Histograms of Air Voids and Mass Loss

Table 7.1 Un-Equal Variance *t*-test Comparisons of Mass Loss Results

Condition	Mixture ID	*n*	Mean	Var.	*t*-stat	*t*-crit	Significantly Different?
Un-aged	*9.5-15/CM-5*	30	7.23	0.608	0.122	±2.01	No
Un-aged	*9.5-15/CM-6*	30	7.64	1.450			
Un-aged	*9.5-15/CM-5*	30	7.23	0.583	-12.3	±2.01	Yes
R-30	*9.5-15/CM-5*	30	10.60	1.680			

Note: Significance testing performed at the 95% confidence level.

Figure 7.2 plots mass loss and air voids for the three thirty specimen sets. The linear regression equations show at least some correlation of decreased mass loss for decreasing air voids. The slope of the equation for the aged specimen set is higher than the un-aged specimen sets. This is reasonable since specimens with higher air voids have greater potential exposure to oxygen during the oven aging period resulting in greater binder aging and a more brittle mixture with higher mass loss.

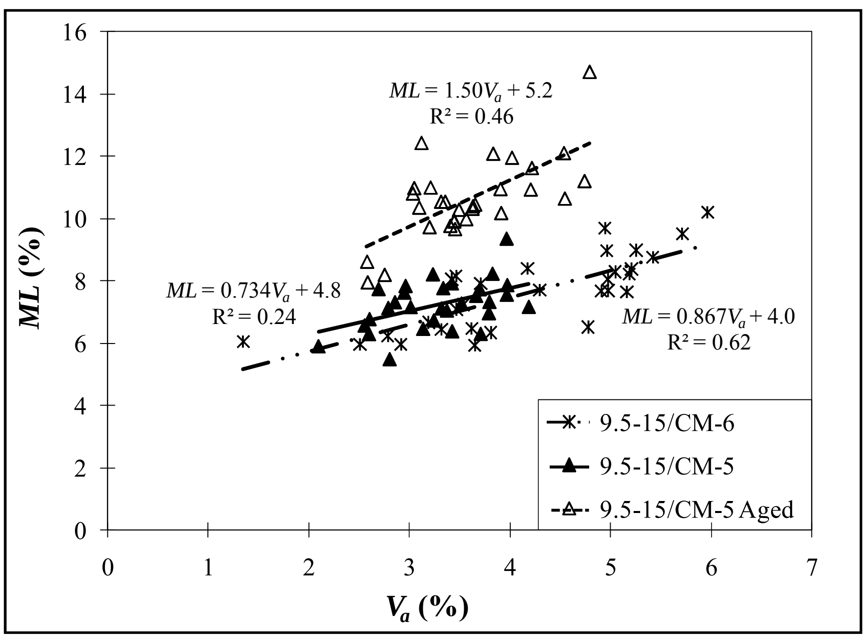
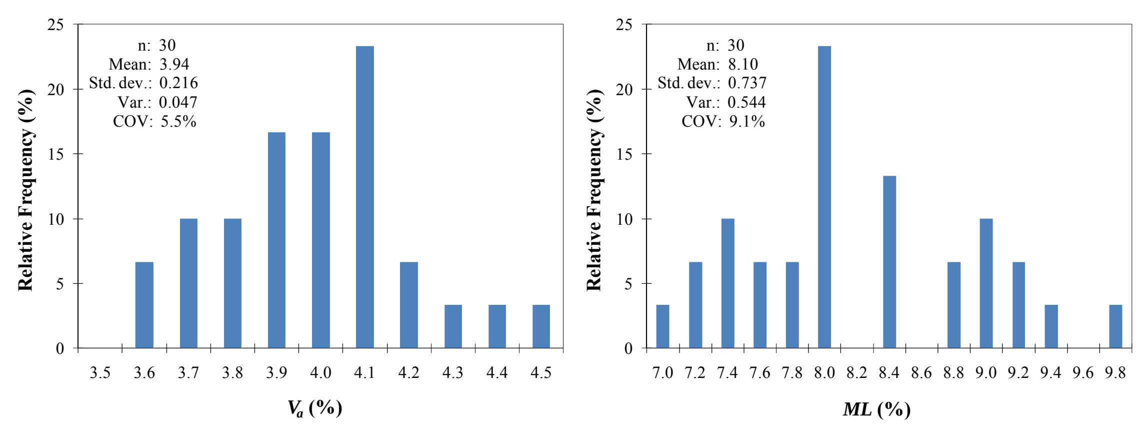


Figure 7.2 Correlation of Air Voids and Mass Loss

One source of variation for specimens from the three sets tested could be the normal variation of plant produced mixture throughout the paving season. Differences in gradation and asphalt content could explain some of the variability as they would also change the air voids. Interpretation of Cantabro results of specimens compacted to N_{des} presented in this report should be tempered by the observation that air voids and mass loss are collinear to some extent.

To investigate mass loss variability at a specific air void level, an additional thirty specimens of mixture *9.5-15/CM-6* were compacted to 4.0 ± 0.5% air voids and tested (Figure 7.3). Variability of V_a is obviously reduced compared to N_{des} compacted specimens (COV of 5.5% compared to 26.0%); however variability of *ML* is only slightly reduced (Figure 7.1*e* compared to Figure 7.3*b*). Standard deviation was reduced from 1.20 to 0.737 and COV was reduced from 15.8% to 9.1%. Mean mass loss of N_{des} specimens was 7.64% and about the same for controlled density specimens (8.10%).



a) 9.5-15/CM-6 Un-aged *b) 9.5-15/CM-6 Un-aged*

Figure 7.3 Relative Frequency Histograms of Air Voids and Mass Loss

7.2.1.2 Random QA Specimens

To establish a durability results range for conventional 9.5 mm dense-graded Mississippi mixtures, plant mixed QA specimens were tested from twenty two mixtures. Basic mixture properties were provided in Table 3.13. The range of mean mass loss for the dense-graded asphalt mixtures presented in Table 7.2 is 2.8 to 11.7%. The mixture with the highest mass loss (*9.5-0/CM-23*) also had the highest air voids. A linear regression relating air voids to mass loss is provided in the notes of Table 7.2; the correlation is noticeable but is not strong. Figure 7.4 plots the Table 7.2 data sorted by binder grade. In general, specimens with higher air voids have higher mass loss. No specific trends are observed for PG 76-22 binder compared to PG 67-22 binder.

A stepwise multiple linear regression was performed with the data to relate mixture parameters to mass loss. Fifteen parameters were considered during the regression: 1) compactive effort (design gyrations): 2) total asphalt content (P_b); 3) effective asphalt content; 4) absorbed asphalt content; 5) mean air voids; 6) voids in mineral aggregate; 7) voids filled with asphalt; 8) film thickness; 9) dust to effective binder ratio; 10) percentage of gravel aggregate; 11) percentage of limestone aggregate; 12) percentage of sand aggregate; 13) percentage of RAP; 14) sand ratio of the aggregate blend (*SR*); and 15) surface area of the aggregate blend. The best regression equation is given in the notes of Table 7.2, which is reasonable for the data available.

Table 7.2 Mass Loss Results for Control Mixtures 7 to 28

Mixture ID	P_b **(%)**	**Gravel (%)**	***SR***	***n***	**Avg.** V_a **(%)**	**Avg.** ***ML*** **(%)**
9.5-15/CM-7	5.4	29	40.7	2	3.3	7.3
9.5-15/CM-8	5.1	29	37.9	2	4.3	2.8
9.5-10/CM-9	5.5	79	43.0	4	4.6	8.2
9.5-15/CM-10	5.5	50	40.4	2	4.5	7.2
9.5-15/CM-11	6.2	40	42.9	4	4.1	7.6
9.5-15/CM-12	5.4	76	40.6	2	3.4	7.2
9.5-15/CM-13	5.8	45	40.9	2	4.1	6.0
9.5-15/CM-14	5.5	61	38.7	2	4.5	5.3
9.5-15/CM-15	6.0	68	49.8	2	4.4	10.5
9.5-15/CM-16	6.1	37	43.5	2	5.9	10.6
9.5-15/CM-17	5.6	52	39.3	2	4.6	8.5
9.5-15/CM-18	5.3	50	37.4	2	4.2	5.2
9.5-15/CM-19	5.5	31	38.4	2	3.5	5.4
9.5-15/CM-20	6.4	40	42.9	2	2.3	3.9
9.5-10/CM-21	5.7	34	53.0	2	4.8	6.8
9.5-15/CM-22	5.8	74	46.5	2	4.4	10.7
9.5-0/CM-23	5.8	40	46.3	2	7.1	11.7
9.5-10/CM-24	5.6	64	41.4	2	4.1	9.4
9.5-10/CM-25	5.4	29	42.6	2	3.1	4.8
9.5-6/CM-26	5.3	28	42.2	8	4.6	7.6
9.5-10/CM-27	6.4	37	43.5	4	6.2	10.7
9.5-10/CM-28	5.2	49	43.1	2	5.8	6.6

Note: $ML = 1.44V_a + 1.06$; $R^2 = 0.46$; n = 22

$ML = -15.2 + 1.10V_a + 1.63P_b + 0.0408\,(\text{Gravel }\%) + 0.157SR$; $R^2 = 0.64$; n = 22

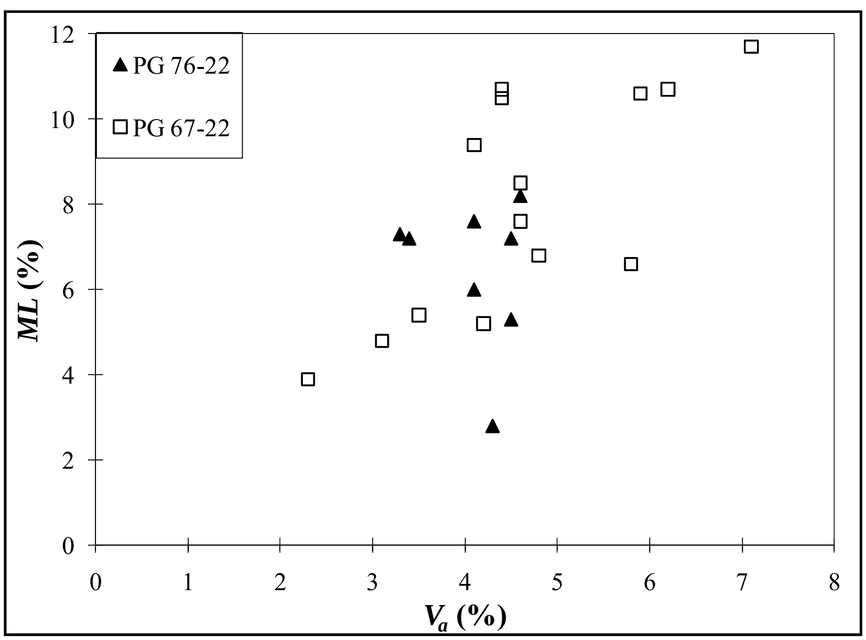


Figure 7.4 Correlation of Mass Loss and Air Voids for Mississippi Mixtures

7.2.1.3 Specific Control Mixtures

The control mixtures used as a comparison for several other properties throughout the report were tested and results are provided in this section. Testing was performed as described in Section 4.2.4. Table 7.3 provides un-aged mass loss results, which ranged from 4.7 to 11.8%. Large replicate testing shown in Section 7.2.1.1 resulted in mass loss values within this range for un-aged testing of 7.3 and 7.6%. The 0% RAP mixture had higher mass loss than most of the 15% RAP mixtures; conventional wisdom would predict a mixture without RAP would have lower mass loss than mixtures with 15% RAP. A variety of factors including mixture composition could explain the behavior. The 0% RAP control mixture had a dust to effective binder ratio of 1.7 (above recommended tolerances) whereas the other control mixtures are 1.0 to 1.2. The 0% RAP mixture results could be due to the high dust content. The lowest mass loss was observed for the plant-warm-mixed PG 76-22 mixture (*9.5-15/CM-3*).

Table 7.4 provides aged mass loss results, which ranged from 7.6 to 10.6%. Large replicate testing shown in Section 7.2.1.1 resulted in mass loss values within this range for aged testing of 10.6%. Aging with *R-30* produced greater mass loss for *9.5-15/CM-2* and the same mass loss for *9.5-15/CM-3* when compared to *MT-85*. *R-30* aging produced greater mass loss in Section 7.2.1.1 when compared to *MT-85*.

Table 7.5 provides mass loss results for un-aged target density specimens of *9.5-15/CM-3*. Mass loss for the mixture is increased 3.1% relative to the N_{des} compacted specimens (average 3.0% air voids). This is noticeable but not unreasonable.

Table 7.3 Mass Loss Results for Un-Aged Control Mixtures

Mixture ID	*n*	Avg. Air Voids (%)	Avg. *ML* (%)
9.5-0/CM-1	5	5.0	11.5
9.5-15/CM-2	5	5.9	8.0
9.5-15/CM-3	5	3.0	4.7
9.5-15/CM-4a	5	7.5	11.8
9.5-15/CM-4b	5	6.0	11.0
9.5-15/CM-4c	3	5.4	9.9

Table 7.4 Mass Loss Results for Aged Control Mixtures

Mixture ID	Aging Protocol	*n*	Avg. Air Voids (%)	Avg. *ML* (%)
9.5-15/CM-2	*R-30*	3	5.7	10.6
	MT-85	3	6.1	9.5
9.5-15/CM-3	*R-30*	3	2.8	7.6
	MT-85	3	3.0	7.6

Table 7.5 Mass Loss Results for Target Density Un-Aged Control Mixtures

Mixture ID	*n*	Avg. Air Voids (%)	Avg. *ML* (%)
9.5-15/CM-3	3	4.0	7.8

Table 7.6 presents un-equal variance *t*-tests of mass loss differences between aging protocols and between aged and un-aged testing. The *R-30* aging protocol resulted in higher mass loss than the *MT-85* aging protocol but the difference was only statistically significant with *9.5-15/CM-2*. Results indicate aged specimens from either aging protocol exhibited significantly higher mass loss than the un-aged specimens for a comparable level of air voids. In general, aged specimens exhibited mass loss on the order of 2 to 4% higher than un-aged specimens of the same mixture.

Table 7.6 Un-Equal Variance *t*-test Comparisons of Mass Loss Results

Mixture ID	Condition	*n*	Mean	Var.	*t*-stat	*t*-crit	Significantly Different?
9.5-15/CM-2	*MT-85*	3	9.47	0.013	6.88	±3.18	Yes
	R-30	3	10.60	0.063			
	Un-aged	5	8.02	0.752	-3.68	±2.78	Yes
	MT-85	3	9.47	0.013			
	Un-aged	5	8.02	0.752	-6.15	±2.57	Yes
	R-30	3	10.06	0.063			
9.5-15/CM-3	*MT-85*	3	7.57	0.243	0.07	±2.78	No
	R-30	3	7.60	0.430			
	Un-aged	5	4.66	0.488	-6.88	±2.45	Yes
	MT-85	3	7.57	0.243			
	Un-aged	5	4.66	0.488	-5.99	±2.57	Yes
	R-30	3	7.60	0.430			

Note: Significance testing performed at the 95% confidence level.

7.2.2 *BBR* and IDT Data

Prior to full analysis, *BBR* mixture testing data was evaluated for reasonableness and outliers. Reasonableness was evaluated by two checks: 1) deflection of the mixture beam increased (and the corresponding calculated stiffness decreased) over the entire duration of the test: and 2) the slope of the stiffness curve increased over the entire duration of the test (concept identified in literature review). Any data points that failed the two checks was omitted from analysis (very small percentage of the data).

Occasional outliers were observed in the *BBR* data that passed the reasonableness checks where the stiffness was less than half that of other replicates of the same mixture and were not representative of the mixture. Information found during literature review revealed that when mixture beams are sawn from a compacted asphalt specimen, the orientation of aggregate particles is essentially random and in most cases a representative cross section of the asphalt mixture is obtained in a sawn mixture beam (Marasteanu et al.

2009). However in some cases the mastic film between aggregates may be oriented in such as way that it in a localized area a large portion of the beam cross-section is composed of the mastic film. This results in a reduction in the measured stiffness of the beam compared to a beam of representative cross-section.

A consistent method to identify these occurrences of mixture stiffness data well below other replicate measurements for the mixture was used. For cases where five replicates were tested, if the standard deviation of stiffness measured at 60 seconds was higher than 4.0 then the data point which was farthest from the mean value for the mix (i.e. very low stiffness) was removed. For cases where three replicates were tested, a cutoff value for standard deviation of 5.0 was used to perform the same data evaluation. All remaining replicates were averaged and used for analysis.

7.2.2.1 Test Method Variability

To evaluate variability of the *BBR* mixture test method, multiple gyratory compacted specimens of *9.5-15/CM-4a*, *9.5-15/CM-4b*, and *9.5-15/CM-1* were tested. Four *SGC* specimens were prepared of mixture *9.5-15/CM-4a* and three gyratory specimens each were prepared of mixtures *9.5-15/CM-4b*, and *9.5-15/CM-1*. Five beam specimens were tested at each of four test temperatures from each *SGC* specimen.

Mixture *9.5-15/CM-4a* the test data was evaluated for reasonableness and outliers; three outliers at -24 C were omitted from analysis. For control mixture *9.5-15/CM-4b*, the data was evaluated for reasonableness and outliers; no outliers found. For control mixture *9.5-15/CM-1*, the data was evaluated for reasonableness and outliers; two outliers at -24 C and one at -06 C were omitted from analysis.

Figure 7.5 presents *BBR* stiffness test data for mixture *9.5-15/CM-4a* organized by test temperature. Error bars at a 95% confidence interval (± 1.96 standard deviations) are shown for the test data at each test time; the actual standard deviations of test data at a particular test time are shown next to the respective error bars. For *BBR* stiffness measurements at all four temperatures the standard deviations decrease as the test time increases. The same trend is seen in the data for mixtures *9.5-15/CM-4b* and *9.5-15/CM-1*; figures are omitted in the interest of brevity. This result indicates that the stiffness values of replicate beam specimens of a mixture tend to converge at longer test times. Therefore better statistical comparisons of stiffness between different mixtures can be made by using data at longer test times.

At -24 C test temperature the standard deviations range from 3.2 at 8 second test time to 2.09 at 960 second test time. Standard deviations of the test data at -18 C test temperature are higher and range from 4.21 at 8 second test time to 2.47 at 960 second test time. Standard deviations of test data at both -12 C and -06 C test temperatures are much smaller and are all less than 2. Data for mixtures *9.5-15/CM-4b* and *9.5-15/CM-1* have the highest standard deviations at the -24 C test temperature and the lowest standard deviations at either the -12 C or the -06 C test temperature. These results indicate that variability of the *BBR* mixture test method tends to be higher at lower test temperatures.

Variability of the *BBR* mixture stiffness test method is within a reasonable range provided the data is first examined and any outlying data omitted from analysis. For the control mixture data used to evaluate variability, only six outliers were identified among the 200 data points (3% of the data). Repeatable results can be obtained with the test method.

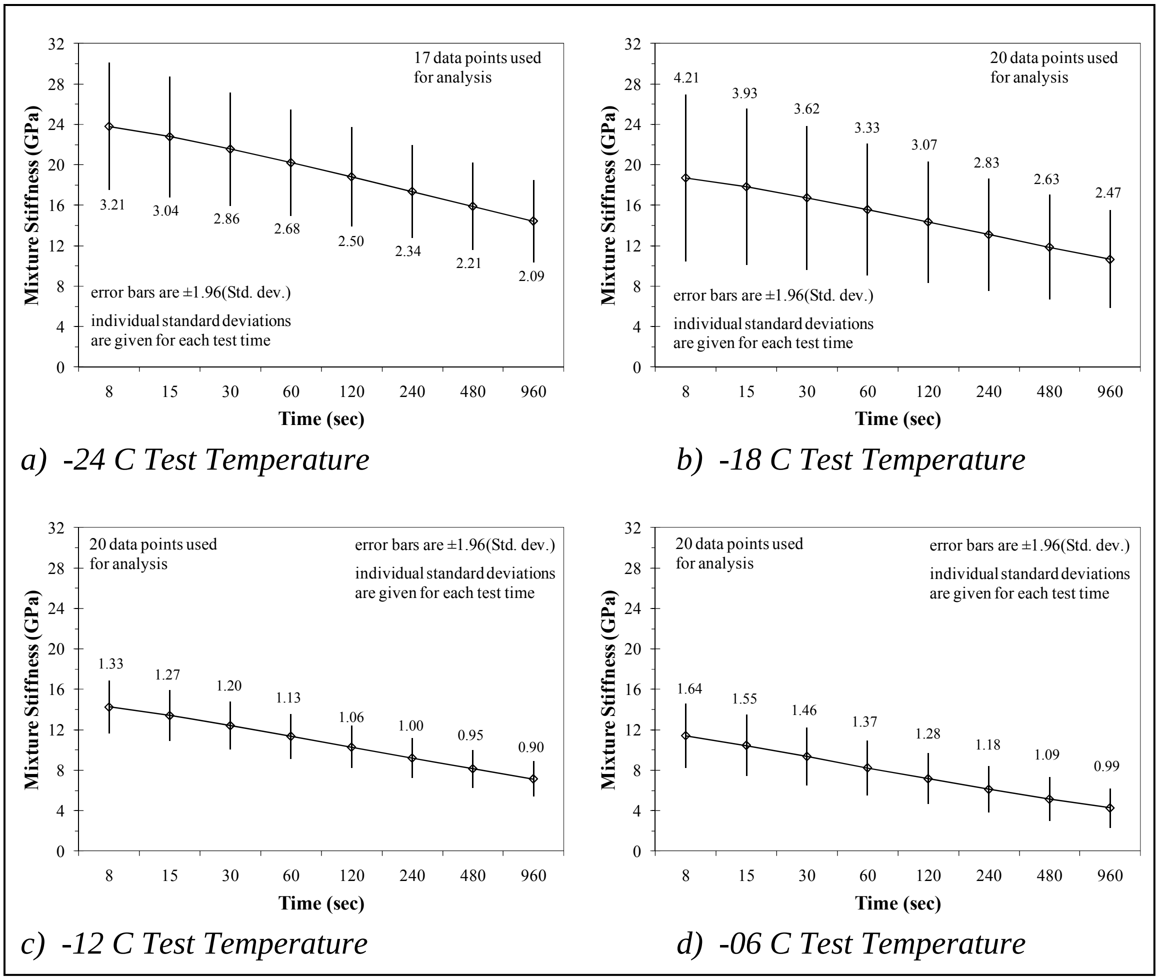


Figure 7.5 *BBR* Stiffness Test Data Variability for Mixture *9.5-15/CM-4a*

7.2.2.2 Control Mixture Data

Figures 7.6 and 7.7 present isotherms of mixture stiffness from averaged test data for control mixtures 1, 2, and 3 and 4a, 4b, and 4c, respectively. The data shows the same general trends of behavior that are observed in typical *BBR* binder testing. Isotherms at the lowest test temperature yield the highest stiffness and generally have the flattest slope. Isotherms at increasing temperatures have lower stiffness and generally have a steeper and gradually increasing slope.

In general the stiffness isotherms for control mixture 1 are as high as or higher than all the other control mixtures and also tend to have flatter curves at all test temperatures. Also the -06 C isotherm is quite close the -12 C isotherm whereas for most of the other control mixtures the -06 C isotherm tends to be noticeably less stiff than the data at colder temperatures. These results for control mixture 1 are hypothesized to be due to the high dust to effective binder ratio of this mixture (value of 1.7 was out of MDOT specification) which will likely result in stiffening of the mixture.

Figure 7.8 presents results for the three versions of control mixture 4 (plant mixed PG 67, lab mixed PG 67, lab mixed PG 76) organized by test temperature to allow for assessment of the effects of different binder grades and mixing preparation methods. Stiffness isotherms of the plant mixed version of control mixture 4 (*9.5-15/CM-4a*) and the laboratory mixed version with PG 67-22 (*9.5-15/CM-4b*) are nearly identical at the -18 and -06 C test temperatures. In contrast the stiffness of the laboratory mixed version is lower than that of the plant mixed version at the -24 and -12 C test temperatures but the difference is relatively small. There is no conclusive evidence that preparation method (plant compared to laboratory) produces any meaningful differences in mixture stiffness at low temperatures as measured by this test method.

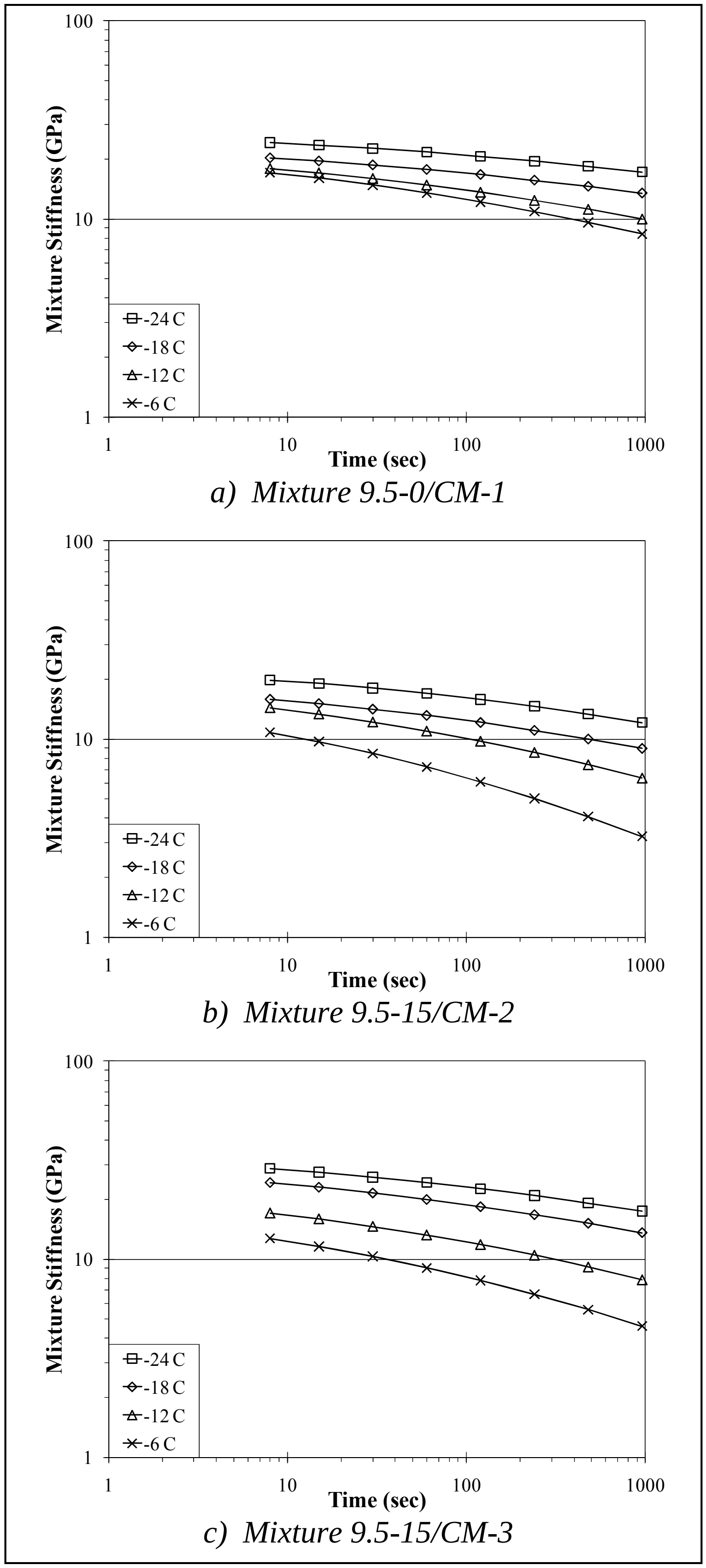
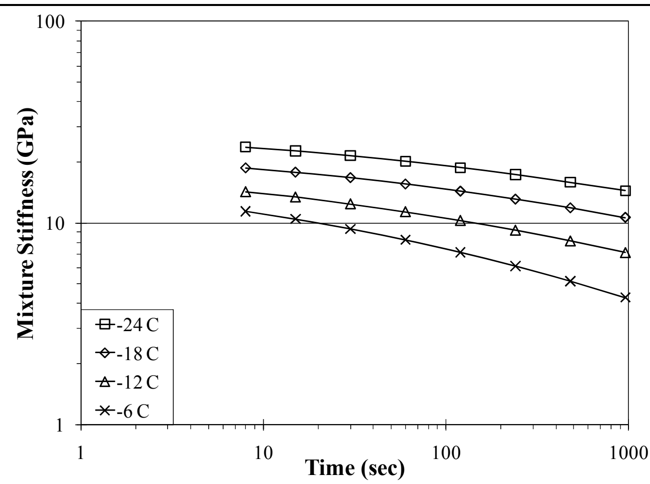
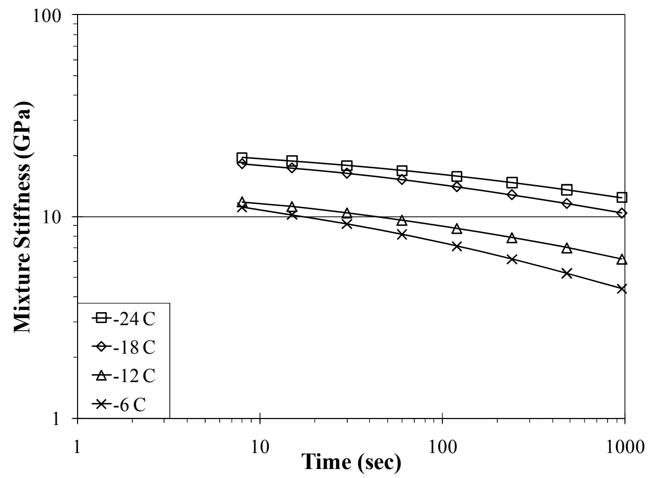


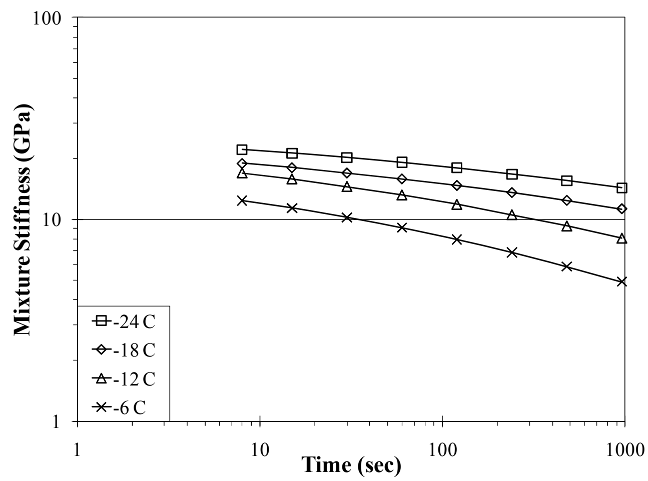
Figure 7.6 *BBR* Stiffness Data for Control Mixtures 1 to 3



a) Mixture 9.5-15/CM-4a



b) Mixture 9.5-15/CM-4b



c) Mixture 9.5-15/CM-4c

Figure 7.7 *BBR* Stiffness Data for Control Mixture 4

The stiffness isotherm of the mixture with polymer modified binder (PG 76-22) at -24 C lies between the isotherms for mixtures with neat binder (PG 67-22) and at -18 C all three isotherms are indistinguishable. This result is reasonable since these test temperatures bracket the low temperature performance grade of the binders. Interestingly, at -12 C and -06 C the PG 76-22 mixture stiffness isotherms are slightly higher than the isotherms for neat binder mixtures although not dramatically so.

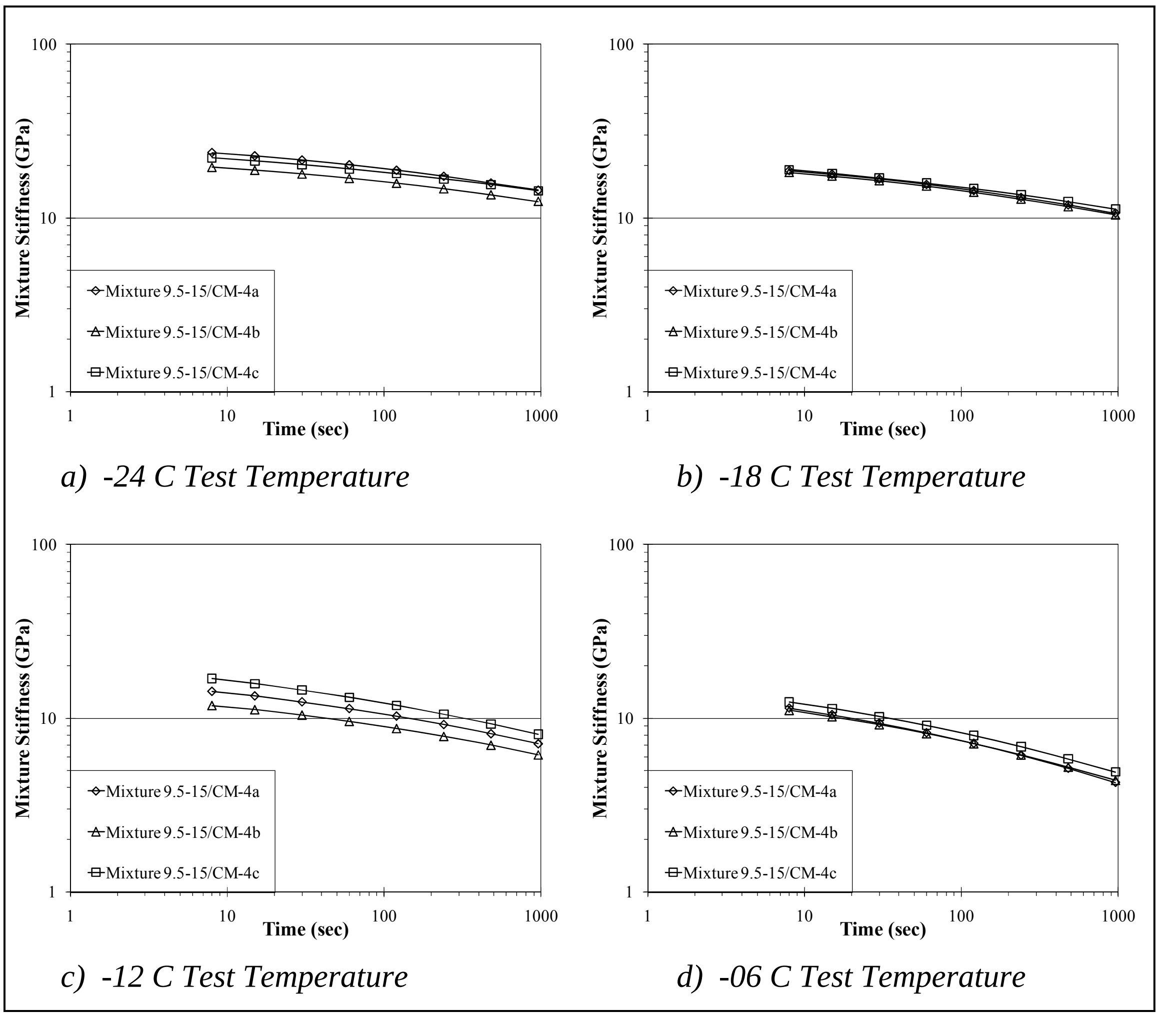


Figure 7.8 *BBR* Stiffness Data for Control Mixture 4 by Test Temperature

Figure 7.9 presents the test data from plant produced control mixtures 2, 3, and 4a organized by test temperature for comparison between plant produced mixtures. Control mixture 2 has the lowest stiffness for each test temperature. This is desirable from the standpoint of susceptibility to thermal cracking (i.e. a less stiff mixture results in lower thermal stress and reduced potential for thermal cracks provided strengths are equivalent to stiffer mixes). This is also reasonable since it is a low traffic (50 design gyration) mixture which results in a greater effective binder content relative to mixtures with higher compactive efforts (65 or 85 gyration mixtures).

Stiffness of both the high traffic mixtures is higher than the low traffic mixture. Control mixture 3 has the highest stiffness for each test temperature. Control mixture 4a has an intermediate level of stiffness at each test temperature. Control mixture 3 being stiffer is not thought to be due to the polymer modified binder but rather to some other mixture parameter (e.g. aggregate properties, effective binder content, etc.) based on testing control mixture 4 with different binder grades. The range of mixture stiffness in Figure 7.9 was taken to represent a reasonable expected range of low temperature mixture stiffness for Mississippi mixtures.

Tensile strength testing at low temperatures was performed for control mixtures 2 and 3 which represented the lowest and highest stiffness control mixtures in *BBR* testing. Properties were determined according to Section 4.2.2 and results are presented in Table 7.7. At the lowest test temperatures (-18 and -24 C), control mixture 3 is stiffer than control mixture 2; however at -12 C the strengths are the same, and at -06 C, control mixture 3 is less stiff than control mixture 2.

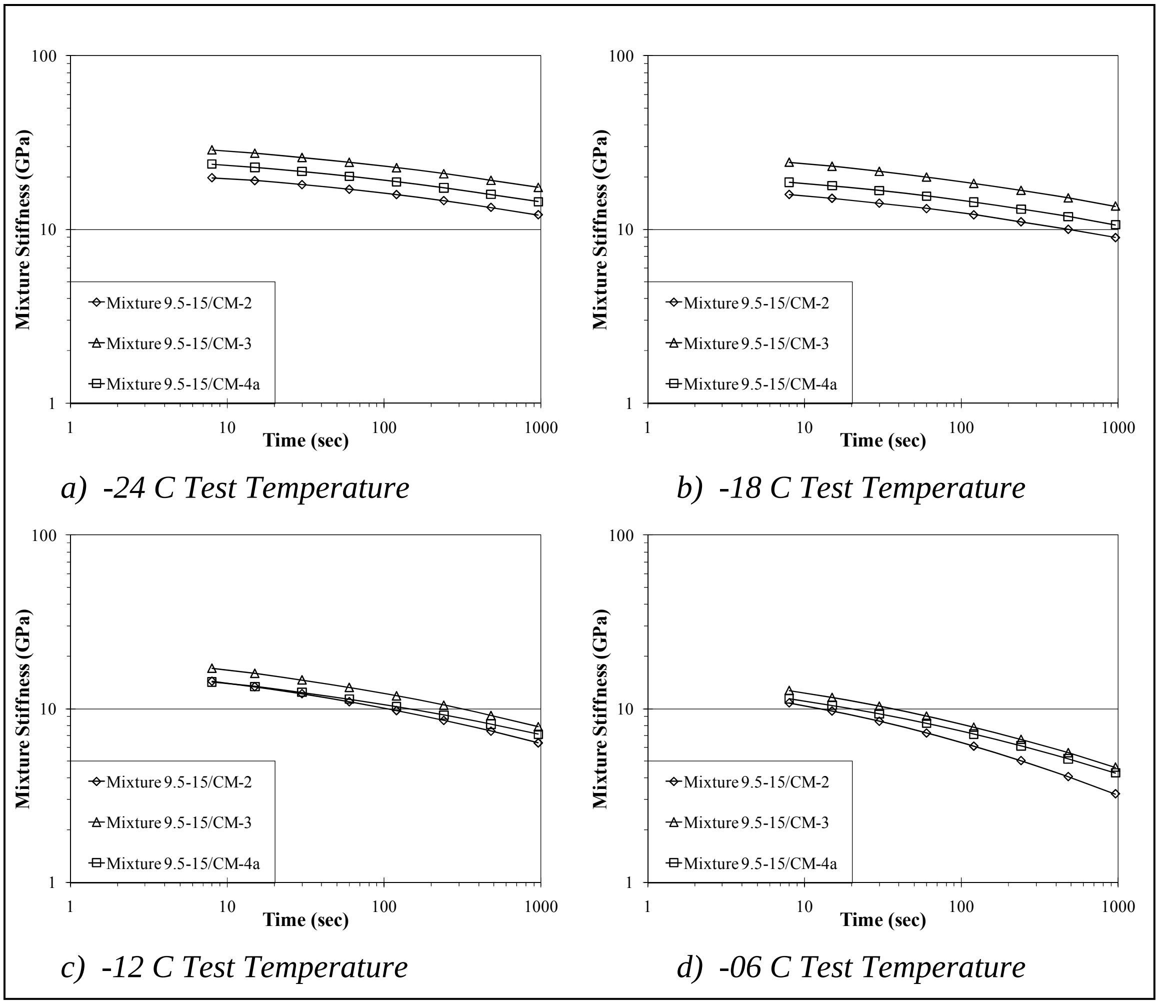


Figure 7.9 *BBR* Stiffness Data for Control Mixtures 2, 3, and 4 by Test Temperature

Table 7.7 Low Temperature IDT Results for Control Mixtures 2 and 3

	Test Temperature and Average Tensile Strength (kPa)			
Mixture ID	**-06 C**	**-12 C**	**-18 C**	**-24 C**
9.5-15/CM-2	4554	3567	3994	3786
9.5-15/CM-3	4291	3547	4507	4116

Note: Each value is average of two test results.

7.2.2.3 Thermal Cracking Analysis

Thermal cracking analysis was conducted generally according to the method in *AASHTO R 49* (AASHTO 2009) though several important modifications to the *AASHTO* standard practice were necessary since mixture test data was utilized instead of binder data. The analysis was performed with commercially available software (TSAR™ Version 0.9.15). The analysis procedure is described briefly in the following paragraphs.

Three mixture parameters are needed as inputs for thermal cracking analysis: 1) density; 2) linear coefficient of thermal contraction (B_{mix}); and 3) glass transition temperature (T_g). Mean mixture bulk specific gravity (G_{mb}) was used for mixture density. Thermal contraction coefficients were estimated for each mixture using Eq. 2.2. The thermal contraction coefficients of each aggregate blend needed for Eq. 2.2 were estimated as an average weighted by the relative proportion of each aggregate type (limestone or gravel) in the mix. It was observed that the estimated B_{mix} values from Eq. 2.2 did not vary much between all the different mixes (including the 25 and 50% RAP mixes discussed later in this chapter). As a result, the average value of B_{mix} for all highway surface mixtures of 2.0×10^{-5} (1/C) was utilized for all thermal cracking analysis performed in this chapter. The work of Nam and Bahia (2004) would tend to indicate that mixture thermal expansion coefficients may vary on either side of the glass transition temperature. However no data was collected as part of this study to evaluate that behavior for the mixes studied; therefore the same value of B_{mix} was used for temperatures above and below T_g. Nam and Bahia (2004) found that T_g values varied based somewhat based on aggregate type and gradation but were within a relatively small range for a particular binder low PG temperature. A T_g of -30 C was selected for all mixes in this study based on the work of Nam and Bahia (2004).

The first analysis step was to plot the four *BBR* isotherms on a log-log scale for each mixture (Figure 7.6 and Figure 7.7). The isotherms were shifted horizontally to produce a single curve with reasonable overlap between isotherms. The linear relationship between shift factors is represented by the Arrhenius equation (Eq. 7.1).

$$\ln(a_T) = a_1\left(\frac{1}{T} - \frac{1}{T_{ref}}\right) \qquad \text{(Eq. 7.1)}$$

Where:

a_T = shift factor

a_1 = mixture dependent constant

T = temperature of isotherm (Kelvin)

T_{ref} = reference temperature (Kelvin)

For the selected reference temperature, the time component of test data from other test temperatures was converted to reduced time (ξ) using Eq. 7.2. The test data was then used to generate a plot of stiffness modulus with reduced time as the x-axis. This results in a single master curve of stiffness at the selected reference temperature.

$$\xi = \frac{t}{a_T} \qquad \text{(Eq. 7.2)}$$

Where:

ξ = reduced time (e.g. computed loading time at the reference temperature)

t = physical loading time (sec)

The resulting master curve of stiffness modulus (S) is a function of reference temperature and reduced time. Eq. 7.3 is the Christensen-Anderson-Sharrock (CAS) master curve model for stiffness modulus (Rowe et al. 2001). For each mixture, the three parameters of S_{glassy}, λ and β were fitted to the shifted test data using a modified non-linear least squares optimization method (Abatech, Inc. 2000, Rowe et al. 2001).

$$S\left(T_{ref},\xi\right)=S_{glassy}\left[1+\left(\frac{\xi}{\lambda}\right)^{\beta}\right]^{-1/\beta} \qquad \text{(Eq. 7.3)}$$

Where:

S_{glassy} = glassy modulus (GPa)

λ = Christensen-Anderson critical time (sec)

β = Christensen-Anderson exponent

Results of the stiffness master curve fitting procedure for the surface highway control mixtures are given in Table 7.8. The fit of Eq 7.1 to the shifted *BBR* isotherms is generally excellent as evidenced by the high R^2 values. The fit of the CAS master curve to the data is also very reasonable as evidenced by the low RMS error values.

Table 7.8 Stiffness Modulus Master Curve Parameters

Mixture	**Arrhenius Equation**			**CAS Parameters for Stiffness Master Curve**			
ID	T_{ref} **(C)**	a_1 **(---)**	R^2	**Error (%)**[a]	S_{glassy} **(GPa)**	**λ (sec)**	**β (---)**
9.5-0/CM-1	-06	20662.4	0.95	1.0	35.5	$59.744\text{x}10^6$	0.139857
9.5-15/CM-2	-06	23578.9	0.99	1.5	28.6	$53.111\text{x}10^3$	0.184053
9.5-15/CM-3	-06	30154.1	0.99	0.99	66.0	$102.82\text{x}10^6$	0.103023
9.5-15/CM-4a	-06	28605.2	0.99	1.7	54.9	$1.0084\text{x}10^9$	0.096452
9.5-15/CM-4b	-06	26199.2	0.94	2.0	44.7	$2.0355\text{x}10^9$	0.098132
9.5-15/CM-4c	-06	24192.6	0.99	0.81	36.1	$1.2386\text{x}10^6$	0.148669

a) Root mean square (RMS) error of the three parameter curve fitting.

For the next analysis step, six analysis parameters were chosen:

- Start temperature: 0 C
- End temperature: -50 C
- Cooling rate: 1 C/hr
- Temperature increment: 0.2 C
- Time increment: 720 sec
- Pavement constant: 1

In *AASHTO R 49* binder analysis, the pavement constant "serves as a damage transfer function to convert the thermal stresses calculated from laboratory [binder] data to thermal stresses generated in the pavement" (AASHTO 2009). Since asphalt mixture was evaluated in this study and not asphalt binder, a value of one was chosen for the pavement constant.

Determination of thermal tensile stress in the asphalt mixture then proceeded according to the procedure given in *AASHTO R 49* (Section 7.3) with the appropriate mixture and analysis parameters. The only deviation from the procedure was use of the CAS form of master curve fitting function previously discussed instead of the specified Christensen-Anderson-Marasteanu (CAM) form of master curve fitting function.

Once data for thermal stress as a function of temperature was obtained from the analysis software, the data was plotted and estimates of critical cracking temperatures (T_{cr}) were made. The two asymptote procedure (TAP) of Shenoy (2002) was utilized to estimate T_{cr} for all mixtures as described in the following paragraph. Additionally, T_{cr} was estimated as the temperature at which thermal stress and tensile strength intersected for all mixtures where tensile strength data at various temperatures was available.

Figure 7.10 plots the development of thermal stress as temperature drops for mixture *9.5-0/CM-1* data. The first asymptote was fitted to the first six data points of the thermal stress curve (i.e. 0 to -5 C). The second asymptote was fitted to the last six data points of the thermal stress curve (i.e. -45 to -50 C). The temperature at which the two asymptotes intersect was determined mathematically. It is interpreted as T_{cr} according to the TAP method of Shenoy (2002). For mixture *9.5-0/CM-1* the T_{cr} temperature is estimated to be -27.3 C.

T_{cr} analysis for control mixtures *9.5-15/CM-2* and *9.5-15/CM-3* are presented in Figure 7.11*a*. These mixtures represented the relative highest and lowest stiffness values in Figure 7.9. Tensile strength data for these mixtures from Table 7.7 was also plotted in Figure 7.11*a*. For both mixtures the estimated thermal stress at -24 C is still less than the measured strength; the intersection of the stress and strength was estimated by extrapolation. For *CM-2*, the TAP method has estimated T_{cr} of -28.1 C and the intersection of stress and strength is estimated to be -31 C. For *CM-3*, the TAP method has estimated T_{cr} of -29.8 C and stress and strength intersection is estimated to be -27 C.

Figure 7.11*b* presents data for all three versions of mixture *CM-4*. Very little difference is observed between the thermal stress curves and the estimated T_{cr} values. This result also supports a conclusion that preparation method (plant compared to laboratory) produces no meaningful differences in low temperature properties.

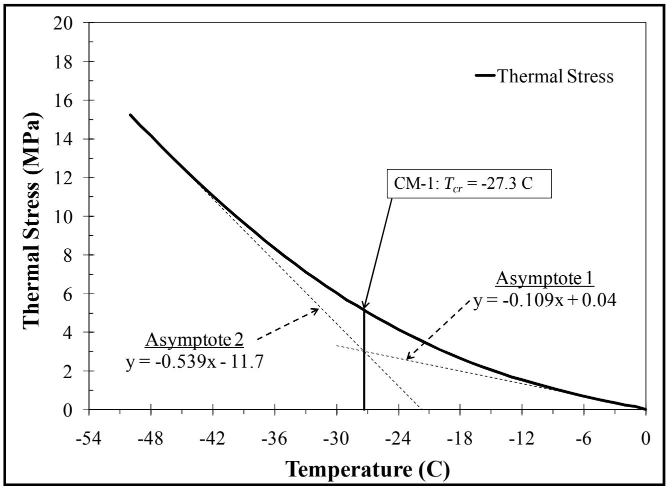


Figure 7.10 Two Asymptote Procedure to Estimate T_{cr} for *9.5-0/CM-1* Data

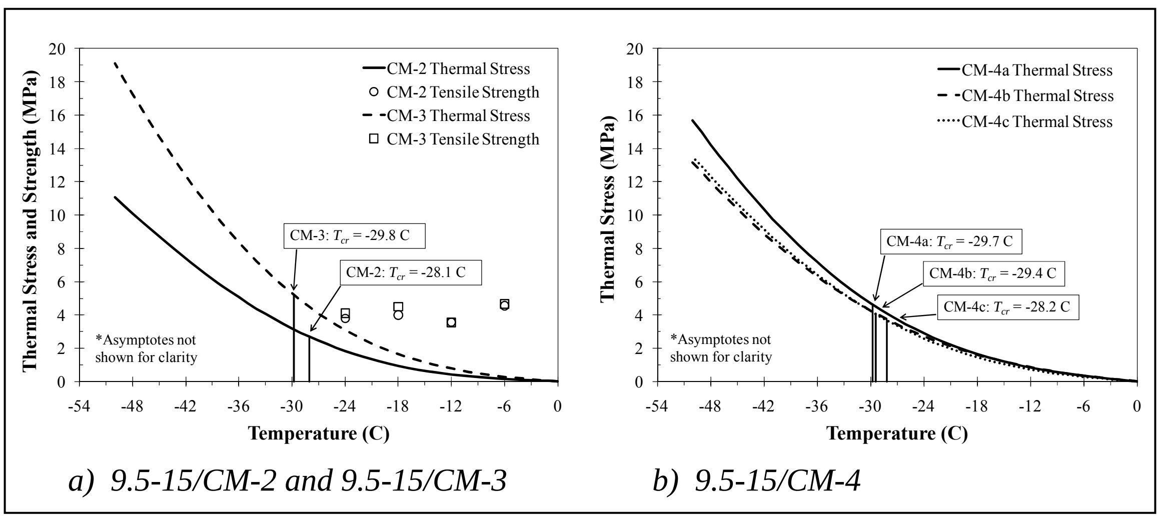


Figure 7.11 T_{cr} Analysis for Highway Control Mixtures *CM-2* to *CM-4*

7.2.3 Rutting Data

Two test methods were utilized to evaluate rutting in a hot-dry condition: 1) *APA* rut testing; and 2) PURWheel dry protocol testing. *APA* and PURWheel testing were performed at 64 C for control mixtures 2, 3, and 4.

7.2.3.1 *APA*

For 15% RAP control mixtures (*9.5-15/CM-2*, *9.5-15/CM-3*, and *9.5-15/CM-4*) *APA* rut testing was performed on *SGC* compacted specimens as described in Section 4.2.6; Table 7.9 summarizes the data. Average air voids are provided as well as total rut depths at 2,000 and 8,000 cycles.

Two types of regression equations were fitted to the data to provide quantitative parameters for comparison: 1) linear regression between 2,000 and 8,000 cycles; and 2) power law regression of data between 0 and 8,000 cycles. Linear regression of data between 2,000 and 8,000 cycles was chosen to represent the rutting data in the secondary flow region after initial densification. Power law regression was chosen to provide a fit of all the rutting data including initial densification and secondary flow. Fitted regression constants and corresponding coefficients of determination are provided in Table 7.9; the regression equations generally provided a very good fit as evidenced by the R^2 values of 0.90 or greater.

Control mixture 2 performed rather poorly with respect to *APA* rutting; total rut depths were over 11 mm regardless of air void level. Control mixture three performed well with total rut depths of 3.5 mm for nominal 7% air voids and 6.3 mm at nominal 10% air voids. With nominal 7% air voids, control mixture 4 with neat binder rutted on the order of 5 mm for both plant and laboratory mixed versions; control mixture 4 with polymer modified binder rutted 2.1 mm.

Table 7.9 *APA* Results for Control Mixtures 2, 3 and 4

Mixture ID	Avg. V_a (%)	Rut Depth (mm) 2000	8000	Linear Rutting Rate[a] Slope (10^{-6})	Intercept	R^2	Power Law[b] *a*	*b*	R^2
9.5-15/CM-2	5.8	9.0	12.2	496	8.56	0.94	0.574	0.350	0.93
	9.6	9.0	11.6	381	8.86	0.91	0.654	0.332	0.90
9.5-15/CM-3	6.8	2.0	3.5	243	1.64	0.97	0.059	0.458	0.99
	9.5	4.4	6.3	299	4.04	0.97	0.181	0.405	0.92
9.5-15/CM-4a	6.8	3.6	5.0	228	3.32	0.96	0.102	0.448	0.91
	9.4	2.5	3.7	182	2.27	0.98	0.112	0.396	0.94
9.5-15/CM-4b	7.1	3.1	4.7	238	2.91	0.95	0.072	0.474	0.92
9.5-15/CM-4c	6.9	1.4	2.1	113	1.23	0.99	0.113	0.327	0.95

a) Linear rutting rate regression analysis is based on averaged data between 2,000 and 8,000 cycles.
b) Power law regression analysis is based on averaged data and Eq. 2.4.

7.2.3.2 PURWheel Dry Protocol

For 15% RAP control mixtures (*9.5-15/CM-2, 9.5-15/CM-3,* and *9.5-15/CM-4*) PURWheel dry rut testing was performed on *LAC* compacted specimens as described in Section 4.2.7.1; the data is located in Tables A.7, A.9, A.11, A.13, and A.15. Table 7.10 summarizes the data. Two types of regression equations were fitted to the data to provide quantitative parameters for comparison: 1) linear regression of the data between 2,000 and 20,000 passes; and 2) power law regression of data between 0 and 20,000 passes. The regression equations generally provided a very good fit of the data for control mixtures 3 and 4 as evidenced by the R^2 values of 0.88 or greater. Linear regression equations could not be fitted to data from control mixture 2 since all specimens of that mixture failed before 2,000 passes. Power law regression equations were fitted to the control mixture 2 rutting data and resulted in reasonable R^2 values; however the R^2 values should be interpreted in light of the limited amount of data used to perform regression.

Similar to *APA* results, control mixture 2 performed poorly in the PURWheel dry protocol rutting test; all specimens exhibited an excessive level of rutting prior to 2,000 passes which resulted in early termination of the test to prevent equipment damage. Post-test visual observations of the specimens revealed failure of the mix in shear as evidenced by the sharp vertical edges of the wheel path (seen in Figure A.7). The PURWheel tires were coated with a film of binder once testing was complete.

Table 7.10 PURWheel Dry Test Results for Control Mixtures 2, 3, and 4

			Rut Depth		Linear Rutting Rate[b]			Power Law[c]		
Mixture ID	V_a **(%)**[a]	**Rep**	**Pass**	**mm**	**Slope (10^{-6})**	**Intercept**	R^2	***a***	***b***	R^2
9.5-15/CM-2	9.2	1-L	1134	24.5	---	---	---	0.001	1.50	0.86
		1-R	800	29.0	---	---	---	1.0 E-4	1.88	0.87
	9.1	2-L	1230	27.5	---	---	---	0.001	1.45	0.86
		2-R	272	12.5	---	---	---	4.0 E-6	2.83	0.95
Average	**9.2**	**---**	**859**	**23.4**	**---**	**---**	**---**	**0.001**	**1.92**	**---**
9.5-15/CM-3	6.9	1-L	20 k	7.0	200	2.92	0.96	0.095	0.438	0.94
		1-R	20 k	4.0	100	2.16	0.98	0.143	0.343	0.89
	8.8	2-L	20 k	5.6	200	2.21	0.95	0.027	0.547	0.94
		2-R	20 k	7.1	200	2.79	0.98	0.078	0.457	0.94
Average	**7.9**	**---**	**20 k**	**5.9**	**175**	**2.52**	**---**	**0.086**	**0.446**	**---**
9.5-15/CM-4a	8.0	1-L	20 k	4.2	100	2.23	0.97	0.191	0.312	0.91
		1-R	20 k	6.0	200	3.07	0.96	0.148	0.378	0.90
	11.5	2-L	20 k	7.3	200	3.07	0.95	0.120	0.416	0.92
		2-R	20 k	11.0	400	3.85	0.96	0.068	0.518	0.96
Average	**10.3**	**---**	**20 k**	**7.1**	**225**	**3.06**	**---**	**0.132**	**0.406**	**---**
9.5-15/CM-4b	10.8	1-L	20 k	11.7	400	4.02	0.99	0.099	0.480	0.95
		1-R	20 k	17.8	700	4.78	0.99	0.216	0.555	0.96
Average	**10.8**	**---**	**20 k**	**14.8**	**550**	**3.06**	**---**	**0.158**	**0.518**	**---**
9.5-15/CM-4c	11.2	1-L	20 k	4.8	100	2.63	0.95	0.185	0.332	0.88
		1-R	20 k	5.5	100	3.07	0.94	0.216	0.331	0.88
Average	**11.2**	**---**	**20 k**	**5.2**	**100**	**2.85**	**---**	**0.201**	**0.332**	**---**

a) Specimen air voids correlated to *AASHTO T 331*.
b) Linear rutting rate regression analysis is based on averaged data between 2000 and 20,000 passes.
c) Power law regression analysis is based on averaged data and Eq. 2.4.

Control mixture 3 performed very well in PURWheel dry rut testing; average rut depths at 20,000 passes were about 6 mm. The other control mixture with polymer-modified binder (*9.5-15/CM-4c*) also performed very well with an average total rut depth of about 5 mm. These two mixtures had the lowest values of slope and intercept parameters for linear rutting rate regression performed data from 2,000 to 20,000 passes.

Control mixture 4*a* (plant mixed with PG 67-22 binder) performed well; average rut depths were about 7 mm, not greatly higher than those for polymer-modified binder mixtures. Control mixture 4*b* (also with PG 67-22 binder) did not perform as well as its plant mixed counterpart; average total rut depths were about 15 mm. This result is unexpected. Variations in mean air voids of the specimens do not fully explain the difference in results since mean specimen air voids for the laboratory mixed specimens are within the range of air voids of plant mixed specimens.

In general, the PURWheel dry protocol test results provided the same relative ranking of rutting performance of control mixtures as did the *APA* test results. Control mixture 2 was observed to have the worst performance, and both control mixtures with polymer-modified binder performed similarly and very well. Notable differences in rutting performance between field and laboratory mixed versions of *CM-4* with neat PG 67-22 binder were observed in PURWheel results that were not seen in *APA* results.

7.2.4 Moisture Damage Data

Two test methods were utilized to evaluate susceptibility of the mixtures to moisture damage: 1) *TSR*; and 2) PURWheel wet protocol. The *TSR* test is a standard moisture susceptibility test currently utilized as a screening tool by many agencies including MDOT. The PURWheel is a research grade loaded wheel tracking test that is similar in some respects to the Hamburg wheel tracking test which is used by a few agencies (e.g. Texas DOT).

7.2.4.1 *TSR*

TSR moisture susceptibility testing was performed on *SGC* compacted specimens as described in Section 4.2.5 for control mixture *9.5-0/CM-1*. For control mixtures *9.5-15/CM-2*, *9.5-15/CM-3,* and *9.5-15/CM-4* the *TSR* values reported on the MDOT mix design sheet were utilized. Control mixture *TSR* results are summarized in Table 7.11. All mixtures have acceptable *TSR* results (i.e. greater than 80%).

Table 7.11 *TSR* Results for Control Mixtures

Mixture	Conditioned Set			Un-Conditioned Set		
ID	Avg. V_a (%)	Sat (%)	S_t (kPa)	Avg. V_a (%)	S_t (kPa)	*TSR* (%)
9.5-0/CM-1	7.5	62.0	1111	7.6	1208	92.0
9.5-15/CM-2	---	---	---	---	---	93.6
9.5-15/CM-3	---	---	---	---	---	94.5
9.5-15/CM-4a	---	---	---	---	---	94.5

Note: Data for mixtures 2, 3, and 4*a* was taken from MDOT mix design sheets.

7.2.4.2 PURWheel Wet Protocol

PURWheel wet protocol testing was performed on *LAC* compacted specimens of all control mixtures except *9.5-0/CM-1* as described in section 4.2.7.2 of the experimental program. Control mixture PURWheel wet test data is found in Tables A.8, A.10, A.12, A.14, and A.16. Analysis of the wet test data was performed as described in the following paragraph.

The data was first plotted and examined for evidence of moisture induced damage. Figure 7.12 provides two example sets of PURWheel wet test data from two different mixtures. Test Data 1 does not provide any evidence of moisture induced damage; the curve resembles a curve from the PURWheel dry test and continues smoothly all the way to 20,000 passes. Test Data 2 has the same general shape as dry test data up to about 5,000 passes. Beginning at approximately 5,000 passes the slope of the curve gradually starts to steepen. Eventually the slope of the curve becomes close to vertical in the vicinity of 7,000 passes. This is evidence of moisture induced damage. Visual observations of specimens at this stage typically reveal bare aggregate surfaces, cracks in the wheel path, and sometimes dislodged and uncoated aggregate. Photographs of test specimens are provided in Appendix A that show these behaviors for moisture damaged specimens.

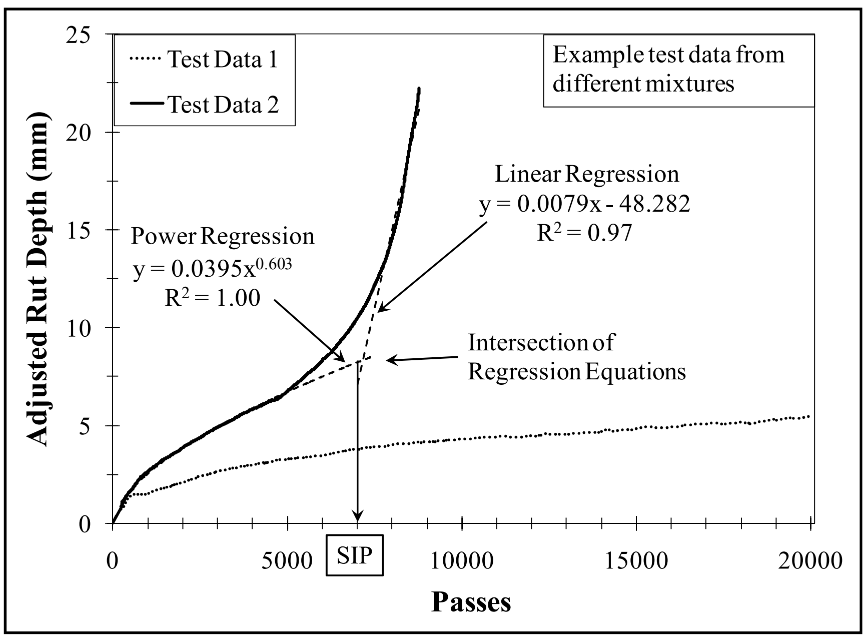


Figure 7.12 Example of PURWheel Moisture Damage Data Analysis Technique

For test data that shows evidence of moisture damage a power law regression equation was fitted to the data from the beginning of the test up to the point where the slope of the data begins to steepen. In the example test data given in Figure 7.12 the power law equation was fitted to the test data between 0 and 5000 passes. Assessment of where the curve began to steepen and what portion of the data provided the best fit of the power law regression equation was accomplished by incrementally increasing the amount of data used for the regression until the R^2 value began to decrease. Next a linear regression equation was fitted to the portion of the data with the steepest slope beginning at the end of the test and progressing backward. The amount of data used for the regression was determined by incrementally increasing the amount of data included until the R^2 value was maximized.

Coefficients of the two regression equations were utilized to determine the intersection point of the equations. The number of passes where the fitted regression equations intersected was considered to be the stripping inflection point (SIP) as shown in Figure 7.12. Calculation of the SIP was rounded to the nearest 500 passes. The power regression equation was extended forward and the linear regression equation was extended backward in Figure 7.12 to demonstrate where the two curves intersect. For the example data in Figure 7.12 the SIP was 7,000 passes.

Summary plots of PURWheel test results for control mixtures are presented in Figures 7.13 to 7.17; data from both wet and dry PURWheel test protocols are presented in the figures to facilitate discussion of mixture relative performance. As seen in Figure 7.13, all of the *9.5-15/CM-2* test specimens failed in less than 1,500 passes. A slight amount of binder was observed to be removed from the aggregate surface in wet test specimens. As discussed in Section 7.2.3.2, control mixture 2 performed very poorly in PURWheel dry testing; this is also observed in the wet test data. No differences are observed between wet or dry test data but this result is not informative since the specimens all failed so quickly.

For control mixture *9.5-15/CM-3*, two of the four wet specimens exhibited moisture damage as shown in Figure 7.14. Some binder was observed to be removed from the aggregate surface in moisture damaged specimens but no wheel path cracking or loose aggregate was seen. The two specimens without moisture damage exhibited deformation behavior similar to the dry test results although for one of them it appeared that moisture damage may have initiated near the end of the test but it did not lead to failure before the test was over.

For control mixture *9.5-15/CM-4a* only one of the four wet test specimens exhibited moisture damage as shown in Figure 7.15; a small amount of binder was removed from the aggregate surface but no loose aggregate or wheel path cracking was observed. The wet test specimens without moisture damage behaved much the same as dry test specimens. Both wet specimens of the laboratory mixed version of control mixture four (*9.5-15/CM-4b*) exhibited evidence of moisture damage as shown in Figure 7.16; noticeable cracking both within and beside the wheel path and minimal loose aggregate was observed for the specimens. One of the two wet specimens of laboratory mixed *9.5-15/CM-4c* with polymer modified binder shown in Figure 7.17 exhibited moisture damage and the other did not.

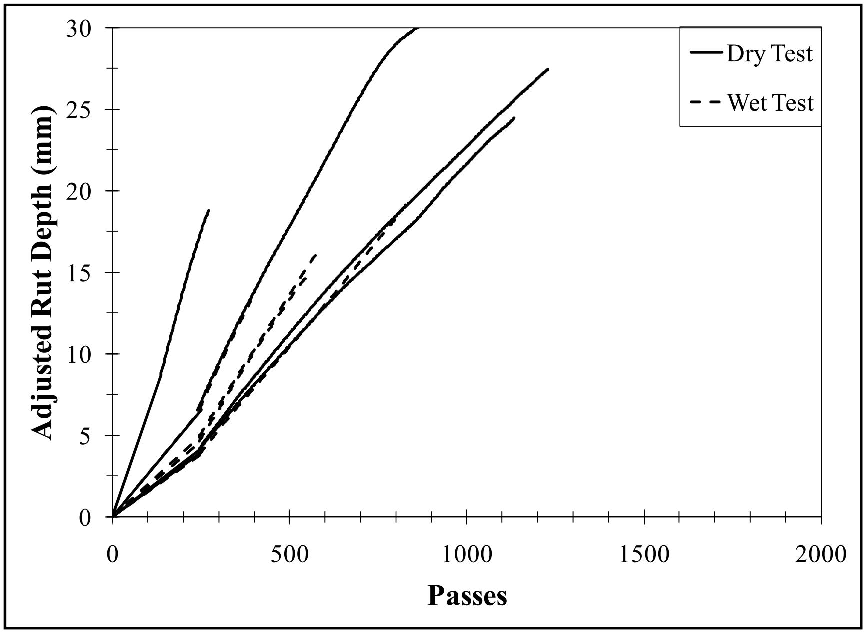


Figure 7.13 PURWheel Test Results for Mixture *9.5-15/CM-2*

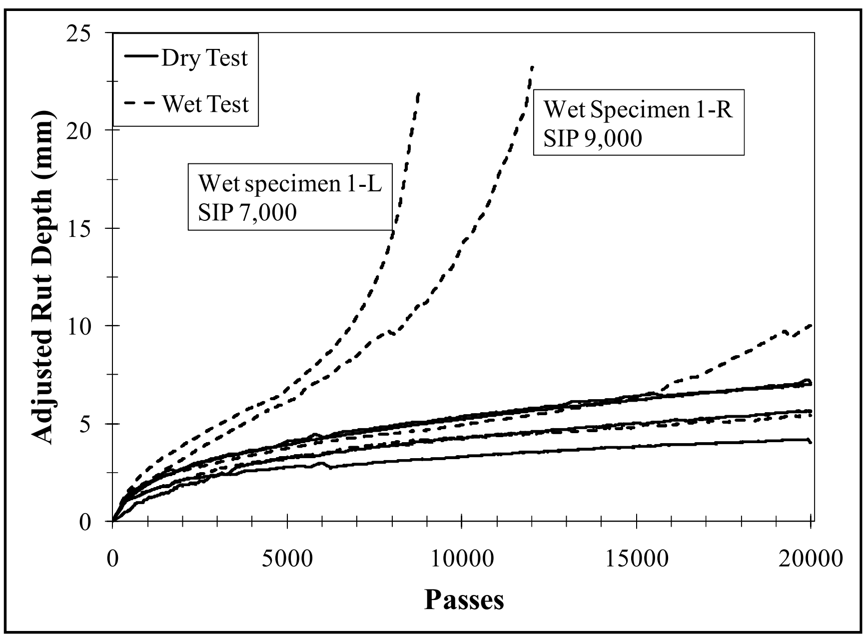


Figure 7.14 PURWheel Test Results for Mixture *9.5-15/CM-3*

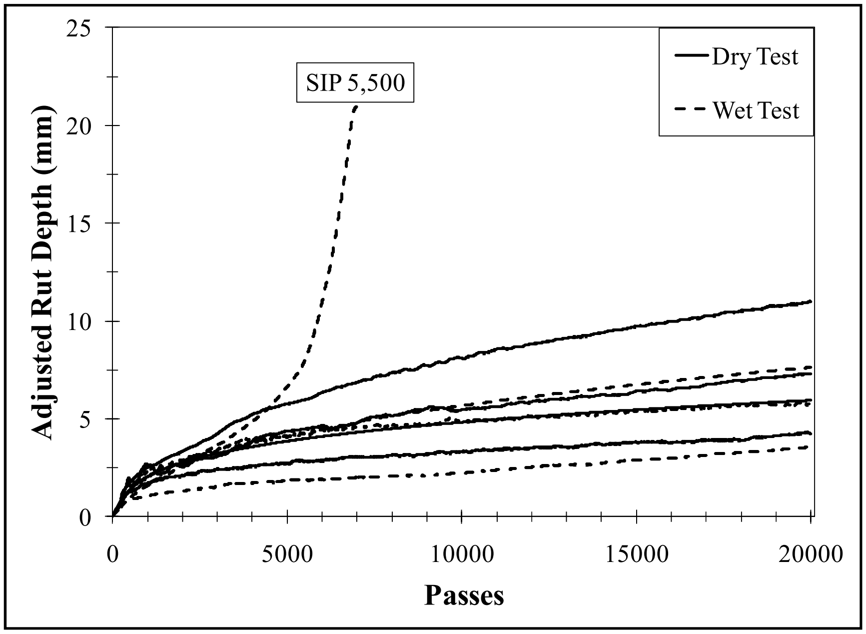


Figure 7.15 PURWheel Test Results for Mixture *9.5-15/CM-4a*

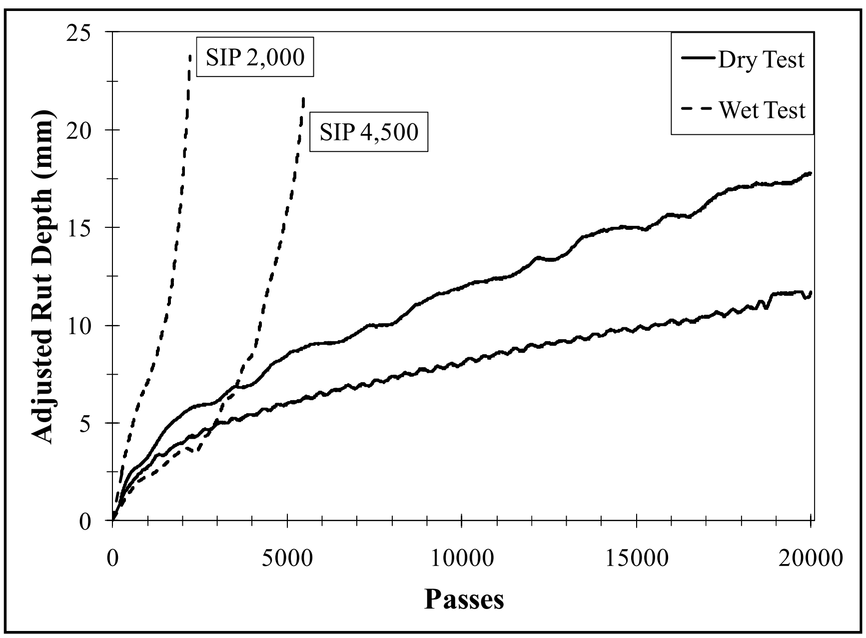


Figure 7.16 PURWheel Test Results for Mixture *9.5-15/CM-4b*

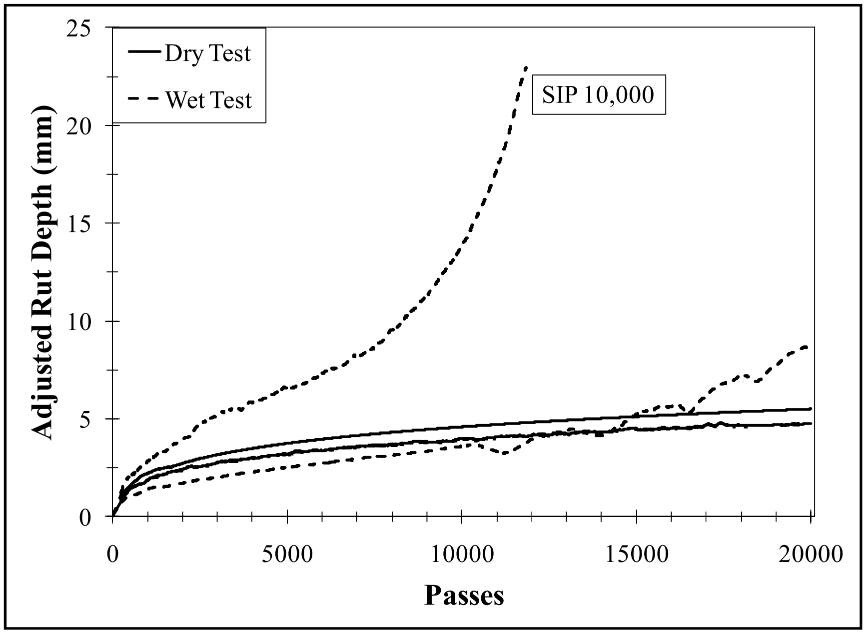


Figure 7.17 PURWheel Test Results for Mixture *9.5-15/CM-4c*

Table 7.12 summarizes PURWheel wet protocol test results for control mixtures. An average SIP value was calculated for each mixture; when no SIP was observed for a specimen a value of 20,000 passes was used to calculate the average SIP. The number of passes at failure and the terminal rut depth of each specimen are given as well as the results of the visual assessment.

Based on performance of the control mixtures, the wet protocol test data is used to suggest a mixture performance classification for this report which is given in Table 7.13. The classification system is based on the average SIP for a mixture. According to the classification system, control mixtures *9.5-15/CM-3*, *9.5-15/CM-4a* and *9.5-15/CM-4c* would all be considered performance classification 4.

Control mixture *9.5-15/CM-4b* would be considered performance classification 1 but in general it performed poorly in PURWheel testing by either the wet or dry test protocols compared to the plant mixed version. Control mixture *9.5-15/CM-2* would also be considered performance classification 1 but since in general the PURWheel overwhelmed the mixture in both wet and dry tests this does not necessarily provide meaningful information about the mixture's performance related to moisture damage.

Table 7.12 Summary of PURWheel Wet Test Results for Control Mixtures

				Failure		Visual Assessment		
Mixture ID	**V_a (%)[a]**	**Rep**	**SIP**	**Pass**	**(mm)**	**Bare Agg.**	**Loose Agg.**	**Crack**
9.5-15/CM-2	9.1	1-L	None	828	19.1	Yes	No	No
		1-R	None	572	16.1	Yes	No	No
	9.2	2-L	None	550	14.8	Yes	No	No
		2-R	None	390	13.2	Yes	No	No
Average	**9.2**	**---**	**---**	**585**	**15.8**	**---**	**---**	**---**
9.5-15/CM-3	7.0	1-L	7,000	8782	22.2	Yes	No	No
		1-R	9,000	12,020	23.2	Yes	No	No
	7.3	2-L	None	20 k	9.7	No	No	No
		2-R	None	20 k	10.0	No	No	No
Average	**7.2**	**---**	**14,000**	**15,200**	**16.3**	**---**	**---**	**---**
9.5-15/CM-4a	7.4	1-L	None	20 k	5.8	No	No	No
		1-R	None	20 k	3.6	No	No	No
	4.8	2-L	None	20 k	7.9	No	No	No
		2-R	5,500	6978	21.0	Yes	No	No
Average	**6.1**	**---**	**16,375**	**16745**	**9.6**	**---**	**---**	**---**
9.5-15/CM-4b	10.7	1-L	2,000	2,214	23.8	Yes	No	Yes
		1-R	4,500	5,490	22.0	Yes	Yes	Yes
	10.7	**---**	**3,250**	**3,852**	**22.9**	**---**	**---**	**---**
9.5-15/CM-4c	10.7	1-L	10,000	11,842	23.0	Yes	No	Yes
		1-R	None	20 k	8.7	Yes	No	No
Average	**10.7**	**---**	**15,000**	**15,921**	**15.9**	**---**	**---**	**---**

Note: When no SIP was observed, a value of 20,000 passes was used to calculate the average SIP.

a) Specimen air voids correlated to *AASHTO T 331*.

Table 7.13 Proposed PURWheel Wet Protocol Mixture Classification System

Performance Classification	**Average SIP**
1	<5,000 passes
2	5,000 to 9,999 passes
3	10,000 to 14,999 passes
4	15,000 to 20,000 passes

7.3 Highway Surface 25 and 50% RAP Mixture Results

7.3.1 Cantabro Durability Data

Cantabro testing (aged and un-aged) was performed as described in Section 4.2.4. The results are summarized in Table 7.14. Mass loss was 11.9 to 13.2% for the 25% RAP mixtures. These results were similar to or only slightly higher than the range of mass loss observed for control mixtures of 12% or less. Mass loss was 14.1 to 16.7% for the 50% RAP mixtures. These results were slightly higher than results for the 25% RAP mixtures and somewhat higher than results observed for control mixtures.

Increasing the amount of *R-1* RAP from 25 to 50% resulted in a *ML* increase of about 5% for both N_{des} specimens and for target density specimens. The same increase in *R-2* RAP only resulted in a *ML* increase of about 1% for N_{des} specimens and actually resulted in decreased *ML* of about 2% for the target density specimens. Cantabro testing of 100% RAP mixtures indicated *R-1* would be more susceptible to mass loss than *R-2* when incorporated into a recycled mixture. This was observed in the 50% RAP mixtures but not in the 25% RAP mixtures. The difference in average specimen air voids between 25% RAP mixtures might account for the observed difference in mass loss since the difference in mass loss between the 25% RAP mixtures was only 1.3%.

Aged Cantabaro test results are summarized in Table 7.15. Mixtures with *R-1* RAP and *R-30* aging were utilized based on previous test results that they produced higher mass loss. With 25% RAP the aged specimens had an increase in mass loss compared to the un-aged specimens of about 6%, which is slightly higher than the 2 to

4% increase observed for control mixtures after aging. For the 50% RAP mixture the mass loss after *R-30* aging was about 9% higher than *ML* for the un-aged mixture.

Table 7.14 Cantabro Data for Un-Aged 25 and 50% Recycled Mixtures

		Compacted to N_{des}		Compacted to Target Air Voids	
Mixture ID	***n***	**Avg. Air Voids (%)**	**Avg. *ML* (%)**	**Avg. Air Voids (%)**	**Avg. *ML* (%)**
9.5-25/RM-1	3	4.6	11.9	3.8	11.4
9.5-25/RM-2	3	5.2	13.2	3.9	11.7
9.5-50/RM-1	3	5.2	16.7	3.8	16.5
9.5-50/RM-2	3	5.2	14.1	4.0	9.8

Table 7.15 Cantabro Data for Aged 25 and 50% Recycled Mixtures with *R-1* RAP

Mixture ID	Aging Protocol	*n*	Avg. Air Voids (%)	Avg. *ML* (%)
9.5-25/RM-1	*R-30*	3	4.7	17.8
9.5-50/RM-1	*R-30*	3	5.7	25.6

7.3.2 *BBR* and IDT Data

BBR testing of recycled mixtures was performed at four test temperatures as per Section 3.5.3. Figure 7.18 presents stiffness isotherms of the *BBR* data at -24 C. Dashed lines in Figure 7.18 are the upper and lower stiffness isotherms for control mixtures tested in Chapter 5 at -24 C. Stiffness isotherms for all four mixtures with 25 or 50% RAP fall within or overlap the band of control mixture (CM) stiffness. Figure 7.19 presents data at -18 C; stiffness isotherms for all four mixtures with 25 or 50% RAP again fall within or overlap the band of control mixture stiffness. In general, the mixes tested at -18 C are less stiff than when tested at -24 C which is a reasonable result. In general, at -24 C and -18 C the mixtures with 50% RAP are slightly stiffer than or of similar stiffness to the mixtures with 25% of the same RAP source.

Figures 7.20 and 7.21 present stiffness isotherms of 25 and 50 % RAP mixes alongside control mixture bands collected at -12 C and -06 C, respectively. All 25 and 50% RAP mixes at -12 C and -06 C are of similar or higher stiffness than the upper band of control mixture stiffness. In general, the mixes tested at -06 C are less stiff than when tested at -12 C which is a reasonable result. In general, at -12 C and -06 C the mixes with 50% RAP are slightly stiffer than or of similar stiffness to the mixes with 25% of the same RAP source. Tensile strength testing at low temperatures was performed for 25 and 50% RAP mixtures. Properties were determined according to Section 4.2.2 and results are presented in Table 7.16.

Table 7.16 Low Temperature IDT Results for 25 and 50% RAP Mixtures

	Test Temperature and Average Tensile Strength (kPa)			
Mixture ID	**-06 C**	**-12 C**	**-18 C**	**-24 C**
9.5-25/RM-1	4019	4735	3950	3886
9.5-25/RM-2	4424	5099	2906	3063
9.5-50/RM-1	3189	3538	3310	2904
9.5-50/RM-2	4609	3925	3055	2986

Note: Each value is average of two test results.

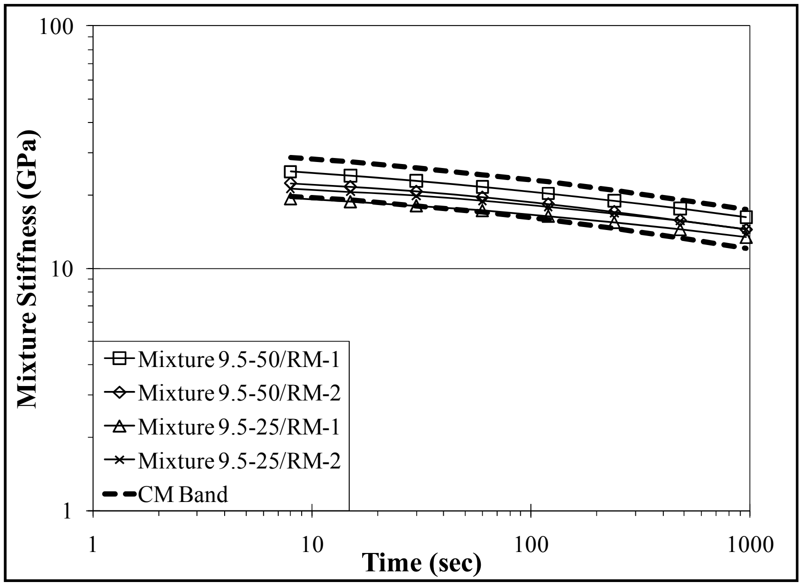


Figure 7.18 *BBR* Stiffness Data for 25 and 50% RAP Mixtures at -24 C

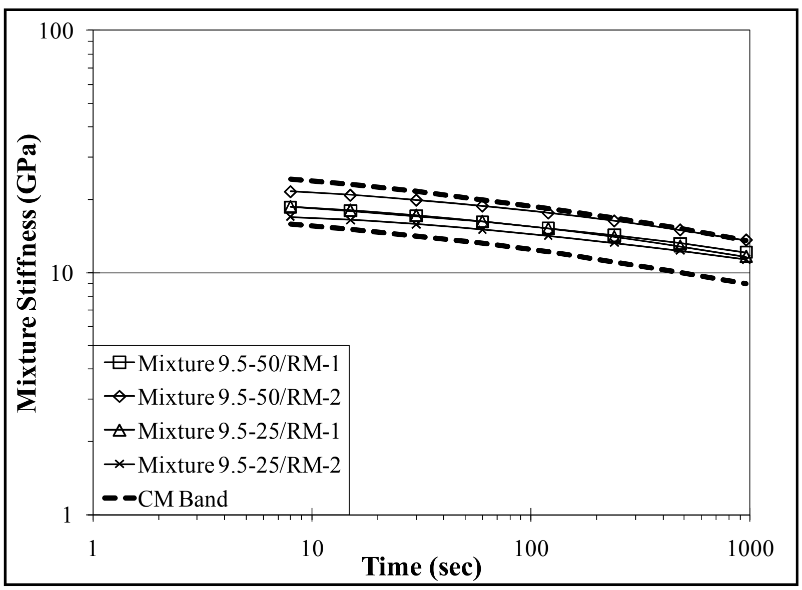


Figure 7.19 *BBR* Stiffness Data for 25 and 50% RAP Mixtures at -18 C

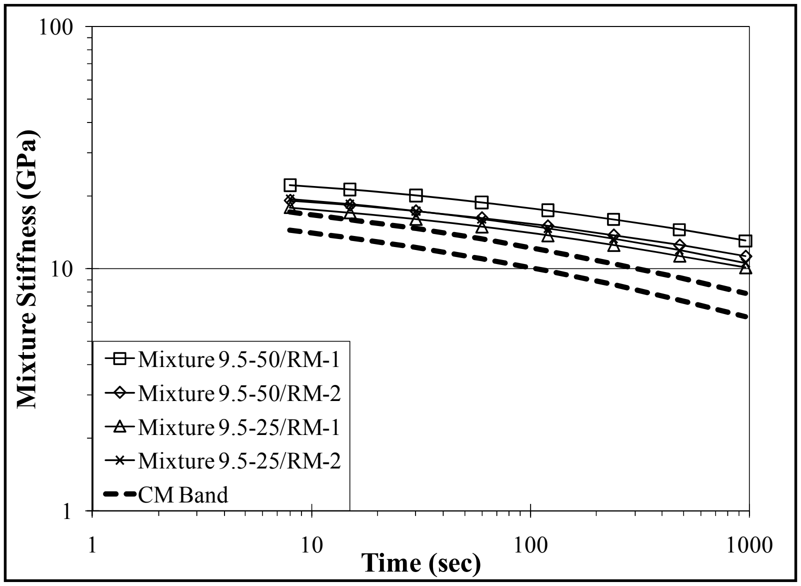


Figure 7.20 *BBR* Stiffness Data for 25 and 50% RAP Mixtures at -12 C

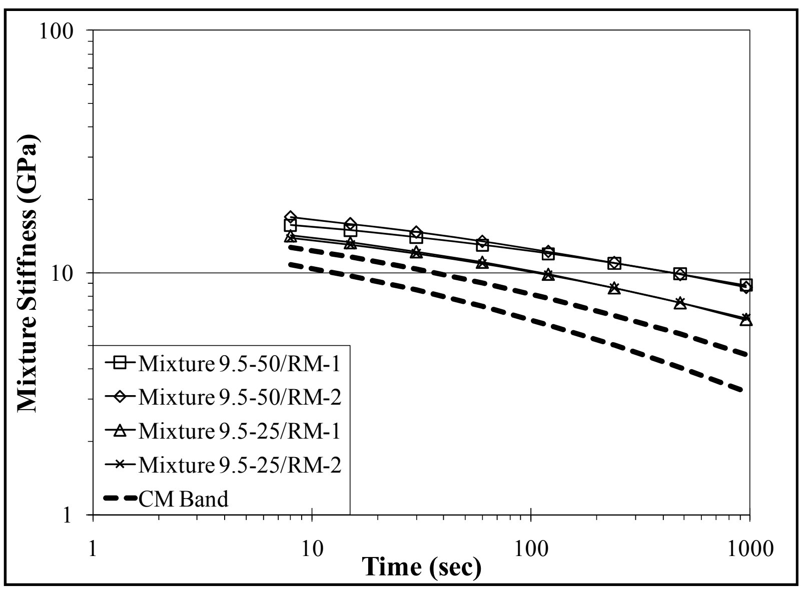


Figure 7.21 *BBR* Stiffness Data for 25 and 50% RAP Mixtures at -06 C

Thermal cracking analysis was performed with the data for 25 and 50% RAP mixtures as described in Section 7.2.2.3. Table 7.17 presents details of the stiffness master curve fitting process. The Arrhenius equation R^2 values are very reasonable and the RMS error for the master curve fitting process is also very reasonable.

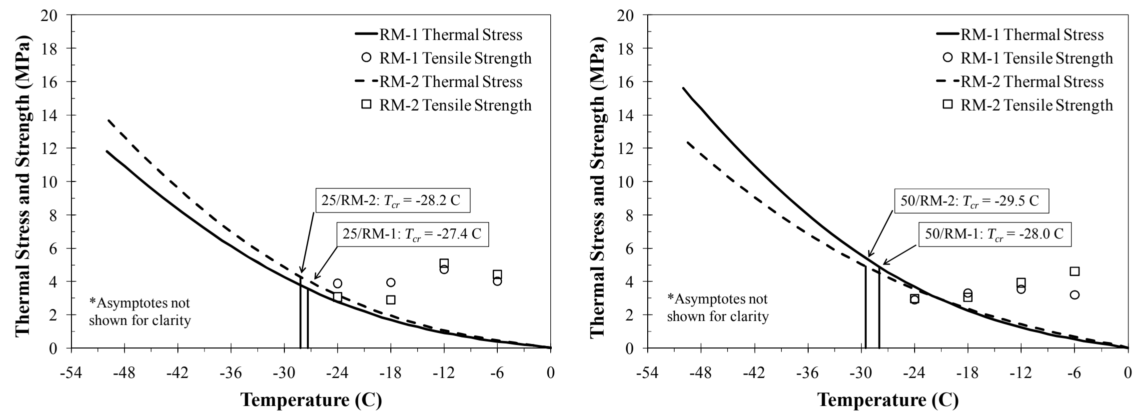
Figure 7.22*a* presents the final results for 25% RAP mixtures. For *9.5-25/RM-1*, the TAP method has estimated T_{cr} of -27.4 C and the intersection of stress and strength is estimated to be -28 C. For *9.5-25/RM-2*, the TAP method has estimated T_{cr} of -28.2 C and stress and strength intersection is approximately -24 C.

Figure 7.23*b* presents the final results for 50% RAP mixtures. For *9.5-50/RM-1*, the TAP method has estimated T_{cr} of -28.0 C and the intersection of stress and strength is approximately -22 C. For *9.5-50/RM-2*, the TAP method has estimated T_{cr} of -29.5 C and stress and strength intersection is approximately -22 C.

Table 7.17 Stiffness Modulus Master Curve Parameters

Mixture	**Arrhenius Equation**			**CAS Master Curve**			
ID	T_{ref} **(C)**	a_1 **(---)**	R^2	**Error (%)**[a]	S_{glassy} **(GPa)**	**λ (sec)**	**β (---)**
9.5-25/RM-1	-06	19133.6	0.94	0.54	28.0	594.83×10^3	0.182443
9.5-25/RM-2	-06	22072.7	0.91	1.2	37.0	17.446×10^6	0.135558
9.5-50/RM-1	-11.6	27190.6	0.93	2.3	40.9	10.000×10^9	0.108532
9.5-50/RM-2	-06	15457.0	0.99	0.47	41.1	142.51×10^6	0.127632

a) Root mean square (RMS) error of the three parameter curve fitting.



a) 9.5-25/RM-1 and 9.5-25/RM-2 *b) 9.5-50/RM-1 and 9.5-50/RM-2*

Figure 7.22 T_{cr} Analysis for Highway 25 and 50% RAP Mixture Data

7.3.3 Rutting Data

7.3.3.1 *APA*

For 25 and 50% RAP control mixtures *APA* testing was performed on *SGC* compacted specimens as described in Section 4.2.6. Specimens with 7 and 10% nominal air voids were tested; Table 7.18 summarizes the data. Linear regression equations were fitted to the data between 2,000 and 8,000 passes and power law regression equations were fitted to the full range of test data. The regression equations generally provided a good fit of the data as evidenced by R^2 values of 0.94 or greater. Total rut depths for 25% RAP mixtures were less than 3 mm for nominal 7% air void specimens, and were less than 5 mm for nominal 10% air void specimens. *APA* rutting performance of the 25% RAP mixtures was comparable to that of 85 gyration control mixtures, yet they were designed as 65 gyration mixtures.

The total rut depth for the 50% *R-1* mixture was 1.9 mm for nominal 7% air voids and 2.9 mm for nominal 10% air voids. For 50% *R-2* the total rut depths were on the order of 4 mm for 7% air void specimens and less than 6 mm for 10% air void specimens. The *R-2* RAP source had slightly higher rutting than the *R-1* RAP source. Overall, *APA* rutting performance of the 25 and 50% RAP mixtures was good and comparable to 85 gyration control mixtures, which is the best reference for rut resistance.

Table 7.18 *APA* Results for 25 and 50% RAP Mixtures

Mixture	**Avg.**	**Rut Depth (mm)**		**Linear Rutting Rate**[a]			**Power Law**[b]		
ID	V_a **(%)**	**2000**	**8000**	**Slope (10^{-6})**	**Intercept**	R^2	***a***	***b***	R^2
9.5-25/RM-1	6.9	1.3	2.4	180	1.04	0.99	0.047	0.439	0.99
	9.5	3.3	5.0	266	3.04	0.94	0.071	0.488	0.95
9.5-25/RM-2	6.9	1.5	3.0	251	1.03	1.00	0.045	0.462	0.99
	9.9	3.3	5.0	268	2.96	0.97	0.093	0.454	0.94
9.5-50/RM-1	7.0	1.2	1.9	107	1.08	0.99	0.083	0.352	0.97
	9.9	1.5	2.9	212	1.21	0.98	0.056	0.438	1.00
9.5-50/RM-2	7.0	2.2	4.3	319	1.80	0.98	0.036	0.537	0.99
	10.0	3.3	5.6	369	2.86	0.96	0.054	0.527	0.97

a) Linear rutting rate regression analysis is based on averaged data between 2000 and 8000 cycles.
b) Power law regression analysis is based on averaged data and Eq. 2.4.

7.3.3.2 PURWheel Dry Protocol

For 25 and 50% RAP mixtures PURWheel dry rut testing was performed on *LAC* compacted specimens as described in Section 4.2.7. Raw data is located in Tables A.17, A.19, A.21, and A.23. Table 7.19 summarizes the data. Linear regression equations were fitted to the data between 2,000 and 20,000 passes and power law regression equations were fitted to the full range of test data. The regression equations generally provided a good fit of the data for the data as evidenced by the R^2 values of 0.85 or greater.

Average total rut depth for mixture *9.5-25/RM-1* was about 10 mm, and average total rut depth for mixture *9.5-25/RM-2* was about 8 mm. Considering variability between specimens, RAP sources *R-1* and *R-2* had similar rut levels when they comprised 25% of the total mixture. For 25% RAP mixtures average total rut depths were higher than those observed for polymer-modified control mixtures and somewhat higher than those observed for the field mixed version of control mixture 4 with PG 67-22. Total rut depths for 25% RAP mixes were less than those for 50 gyration control mixture.

Table 7.19 PURWheel Dry Test Results for 25 and 50% RAP Mixtures

			Rut Depth		Linear Rutting Rate[b]			Power Law[c]		
Mixture ID	**V_a (%)[a]**	**Rep**	**Pass**	**mm**	**Slope (10^{-6})**	**Intercept**	**R^2**	***a***	***b***	**R^2**
9.5-25/RM-1	10.3	1-L	20 k	14.3	600	3.11	1.00	0.035	0.605	0.97
		1-R	20 k	9.1	400	2.34	0.99	0.030	0.576	0.97
	9.0	2-L	20 k	8.5	300	2.88	0.99	0.079	0.473	0.95
		2-R	20 k	6.0	200	2.05	0.99	0.043	0.500	0.96
Average	**9.7**	**---**	**20 k**	**9.5**	**375**	**2.60**	**---**	**0.047**	**0.539**	**---**
9.5-25/RM-2	7.0	1-L	20 k	15.7	600	4.31	0.99	0.069	0.546	0.96
		1-R	20 k	7.6	200	3.36	0.98	0.173	0.381	0.99
	10.4	2-L	20 k	5.4	200	2.45	0.97	0.102	0.403	0.92
		2-R	20 k	11.0	400	4.45	0.98	0.152	0.434	0.93
Average	**8.7**	**---**	**20 k**	**9.9**	**700**	**3.64**	**---**	**0.124**	**0.441**	**---**
9.5-50/RM-1	8.1	1-L	20 k	2.7	60	1.69	0.95	0.135	0.309	0.86
		1-R	20 k	2.4	50	1.47	0.95	0.138	0.294	0.85
	8.7	2-L	20 k	3.9	90	2.36	0.90	0.155	0.334	0.87
		2-R	20 k	2.9	90	1.37	0.92	0.072	0.381	0.92
Average	**8.4**	**---**	**20 k**	**3.0**	**72.5**	**1.72**	**---**	**0.125**	**0.330**	**---**
9.5-50/RM-2	6.4	1-L	20 k	7.6	300	2.78	0.98	0.087	0.452	0.94
		1-R	20 k	5.5	200	2.22	0.98	0.086	0.419	0.94
	8.0	2-L	20 k	6.1	200	2.35	0.98	0.074	0.448	0.94
		2-R	20 k	9.8	300	3.23	0.99	0.086	0.477	0.95
Average	**7.2**	**---**	**20 k**	**7.3**	**250**	**2.65**	**---**	**0.083**	**0.449**	**---**

a) Specimen air voids correlated to *AASHTO T 331*.
b) Linear rutting rate regression analysis is based on averaged data between 2,000 and 20,000 passes.
c) Power law regression analysis is based on averaged data and Eq. 2.4.

For 50% RAP mixtures, rutting was less than was observed for 25% RAP mixtures. Average total rut depth for *9.5-50/RM-1* was 3 mm and average total rut depth for mixture *9.5-50/RM-2* was about 7 mm. This result agrees with the higher viscosity and PG grade of *R-1* RAP asphalt compared to *R-2 RAP* asphalt which would indicate better rutting performance of *R-1*. PURWheel dry protocol test results for 50% RAP mixtures provided the same ranking of rutting performance as *APA* results. Rutting performance of the mixture with 50% *R-1* RAP was comparably or slightly better than that observed for polymer modified control mixtures; rutting performance of the mixture with 50% *R-2* RAP was similar to that observed for the field mixed version of control mixture 4 with PG 67-22. Total rut depths for 50% RAP mixes were less than those for 50 gyration control mixture.

7.3.4 Moisture Damage Data

7.3.4.1 *TSR*

For 25 and 50% RAP mixtures *TSR* testing was performed on *SGC* compacted specimens as described in Section 4.2.5. The results are summarized in Table 7.20. All mixtures had acceptable *TSR* results (i.e. >80%). In general, the 50% RAP mixtures had slightly higher tensile strengths than the 25% RAP mixtures. *TSR* testing of designed 100% RAP mixtures indicated that *R-2* RAP might be more prone to moisture susceptibility than *R-1* RAP but that trend is not observed in the 25 and 50% RAP mixture *TSR* data.

Table 7.20 *TSR* Results for 25 and 50% RAP Mixtures

Mixture ID	Conditioned Set			Un-Conditioned Set		
	Avg. V_a (%)	Sat (%)	S_t (kPa)	Avg. V_a (%)	S_t (kPa)	*TSR* (%)
9.5-25/RM-1	7.5	59	1407	7.4	1447	97
9.5-25/RM-2	7.9	64	1571	7.8	1614	97
9.5-50/RM-1	7.3	64	2053	7.2	2091	98
9.5-50/RM-2	8.0	62	1798	8.0	1942	93

7.3.4.2 PURWheel Wet Protocol

PURWheel wet protocol testing was performed as described in Section 4.2.7 for all 25 and 50% RAP mixtures. The data is located in Tables A.18, A.20, A.22, and A.24. Analysis of the data was performed in the manner described in Section 7.2.4.2. Figure 7.23 presents results of wet and dry protocol PURWheel testing of mixture *9.5-25/RM-1* containing 25% *R-1* RAP. All four PURWheel wet protocol specimens exhibited evidence of moisture damage and early test termination.

Figure 7.24 presents PURWheel wet and dry protocol results for mixture *9.5-25/RM-2* containing 25% *R-2* RAP. Three of the four wet protocol specimens exhibited evidence of moisture damage. The specimen that did not exhibit moisture damage had rutting performance similar to that of dry specimens.

Figure 7.25 presents wet and dry protocol PURWheel results for mixture *9.5-50/RM-1* containing 50% *R-1* RAP. Three of the four wet protocol specimens exhibited evidence of moisture damage. The wet protocol specimen that did not exhibit evidence of moisture damage performed similarly to specimens tested according to PURWheel dry protocol.

Figure 7.26 presents wet and dry PURWheel test results for mixture *9.5-50/RM-2* containing 50% *R-2* RAP. Three of the four wet test specimens exhibited evidence of moisture damage. The wet test specimen that did not exhibit definitive evidence of moisture damage did have an increased rate of rutting compared to dry test specimens in the second half of the test; a mechanical malfunction caused premature termination of the test at 18,000 passes for the wet test specimen without definitive evidence of moisture damage.

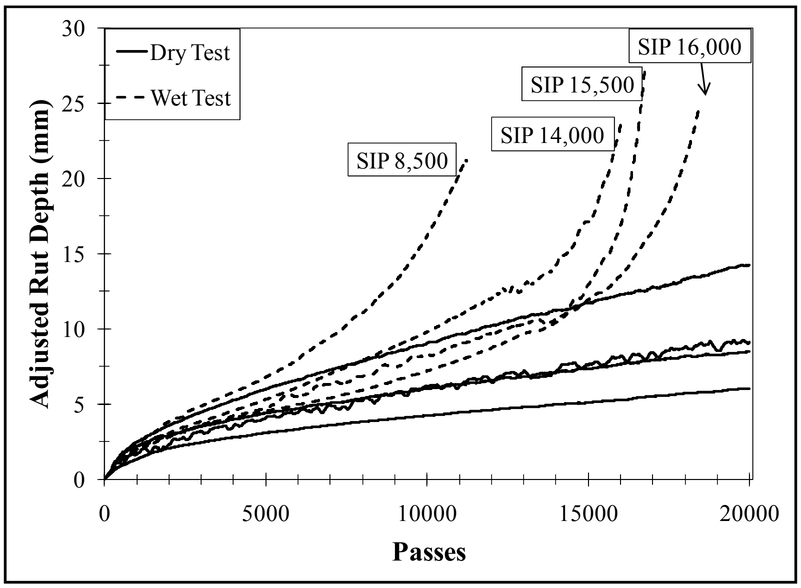


Figure 7.23 PURWheel Test Results for Mixture *9.5-25/RM-1*

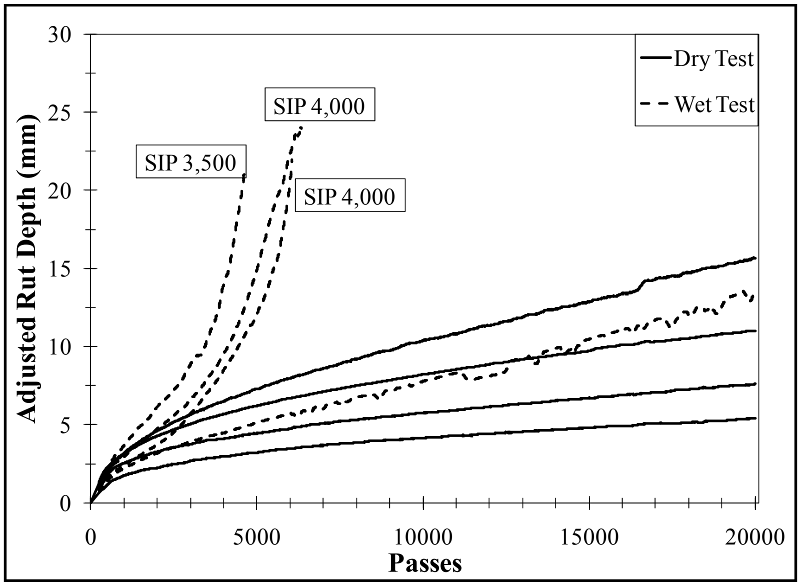


Figure 7.24 PURWheel Test Results for Mixture *9.5-25/RM-2*

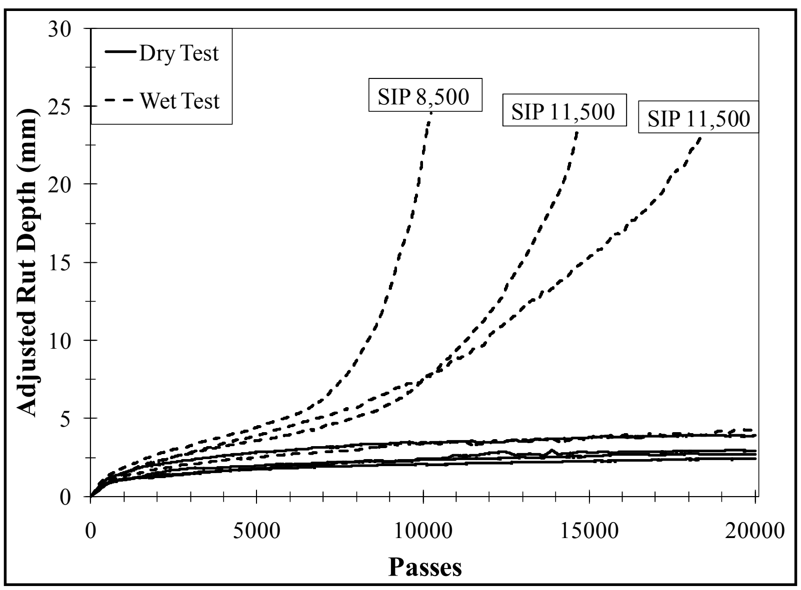


Figure 7.25 PURWheel Test Results for Mixture *9.5-50/RM-1*

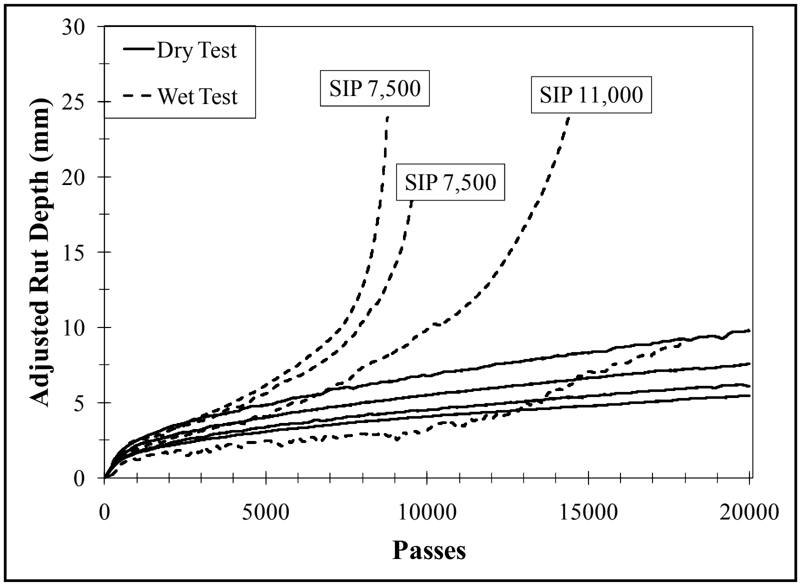


Figure 7.26 PURWheel Test Results for Mixture *9.5-50/RM-2*

Table 7.21 has results of 25 and 50% RAP mixes in the PURWheel wet protocol test. An average SIP was calculated for each mix; when no SIP was observed for a wet test specimen, a value of 20,000 passes was used to calculate an average SIP. Mixtures *9.5-25/RM-1*, *9.5-50/RM-1*, and *9.5-50/RM-2* are performance classification 3 according to Table 7.13. Mixture *9.5-25/RM-2* is a performance classification of 2 in the same proposed system.

Overall, mixtures containing *R-1* RAP as either 25 or 50% of the total mixture performed somewhat better than mixtures containing *R-2* RAP in the same percentage. This result aligns with the results of *TSR* testing of designed 100% RAP mixtures. The result does not agree with results of PURWheel wet protocol testing of designed 100% RAP mixtures in terms of relative ranking.

Table 7.21 Summary of PURWheel Wet Test Results for 25 and 50% RAP Mixtures

				Termination		Visual Assessment		
Mixture ID	**V_a (%)[a]**	**Rep**	**SIP**	**Pass**	**(mm)**	**Bare Agg.**	**Loose Agg.**	**Crack**
9.5-25/RM-1	9.5	1-L	8,500	11,232	21.2	Yes	No	Yes
		1-R	14,000	16,022	23.7	Yes	No	No
	9.1	2-L	15,500	16,766	27.4	Yes	No	No
		2-R	16,000	18,452	25.0	Yes	No	No
Average	**9.3**	**---**	**13,500**	**15,618**	**24.3**	**---**	**---**	**---**
9.5-25/RM-2	9.1	1-L	3,500	4,660	21.5	Yes	No	Yes
		1-R	4,500	6,066	22.0	Yes	No	No
	8.9	2-L	4,000	6,342	24.0	Yes	No	No
		2-R	None	20 k	13.6	Yes	No	No
Average	**9.0**	**---**	**8,000**	**9,267**	**20.3**	**---**	**---**	**---**
9.5-50/RM-1	8.2	1-L	11,500	14,690	23.7	Yes	No	Yes
		1-R	11,500	18,360	23.0	Yes	No	No
	8.3	2-L	8,500	10,238	24.6	Yes	No	Yes
		2-R	None	20 k	4.3	Yes	No	No
Average	**8.3**	**---**	**12,875**	**15,822**	**18.9**	**---**	**---**	**---**
9.5-50/RM-2	6.4	1-L	11,000	14,406	24.1	Yes	No	No
		1-R	7,500	9,526	18.4	Yes	No	No
	8.0	2-L	7,500	8,774	24.0	Yes	Yes	Yes
		2-R	None	18,012[b]	9.2	Yes	No	No
Average	**7.4**	**---**	**11,500**	**12,680**	**18.9**	**---**	**---**	**---**

Note: When no SIP was observed, a value of 20,000 passes was used to calculate the average SIP.

a) Air voids correlated to *AASHTO T 331* values.

b) Premature termination caused by mechanical malfunction; not by excessive specimen deformation.

CHAPTER 8

HIGHWAY BASE MIXTURES

8.1 Overview of Highway Base Mixtures

This chapter presents results from investigation of highway base mixtures. Properties of all mixtures tested are located in Section 3.5.4. Details of the experimental program are located in Section 4.3.4. The results are organized in two broad categories, 15% RAP controls (Section 8.2) and 50 and 75% RAP recycled mixtures (Section 8.3). Subsections organize the data by mixture performance type and analysis category. Discussion and interpretation of all the results is provided in Chapter 9.

8.2 Control Highway Base Mixture Results

8.2.1 Cantabro Durability Data

8.2.1.1 Testing of Random QA Specimens

QA specimens of thirty-three mixtures (12.5 mm and 19.0 mm) were tested for durability (Table 8.1). For 12.5 mm NMAS mixtures, mass loss was observed to generally vary from 5 to 13%; however there is one mixture with mass loss of about 16%. For 19.0 mm NMAS mixtures, mass loss was somewhat higher than for the smaller aggregate gradation and ranged from approximately 7 to 15% with one mixture having

16.5% mass loss. Figure 8.1 plots the mass loss data in terms of mixture air voids. A slight trend of increasing mass loss with increasing air voids is observed in the 12.5 mm NMAS data; however the trend is not observed in 19.0 mm NMAS data.

Table 8.1 Mass Loss Results for Control Mixtures 5 to 37

Mixture ID	**P_b (%)**	***n***	**Avg. V_a (%)**	**Avg. *ML* (%)**
12.5-12/CM-5	5.2	2	3.3	6.7
12.5-15/CM-6	5.5	2	2.9	4.9
12.5-20/CM-7	3.7	2	3.8	12.7
12.5-14/CM-8	5.2	2	3.6	7.8
12.5-15/CM-9	5.2	8	4.0	10.9
12.5-15/CM-10	5.7	2	3.4	12.9
12.5-15/CM-11	5.0	10	4.1	10.3
12.5-12/CM-12	5.2	4	2.8	6.7
12.5-15/CM-13	5.3	2	3.7	7.2
12.5-15/CM-14	5.4	4	4.3	10.7
12.5-15/CM-15	4.7	2	1.9	5.8
12.5-30/CM-16	5.6	2	1.7	8.4
12.5-12/CM-17	5.0	4	3.6	12.8
12.5-15/CM-18	5.7	2	4.1	5.9
12.5-15/CM-19	6.0	2	0.6	5.0
12.5-15/CM-20	5.2	4	5.5	16.1
19.0-15/CM-21	4.7	2	3.9	9.8
19.0-15/CM-22	4.8	2	5.2	12.3
19.0-20/CM-23	4.6	2	1.4	7.7
19.0-20/CM-24	4.9	2	3.8	12.8
19.0-20/CM-25	5.7	2	1.3	9.4
19.0-12/CM-26	4.4	2	3.2	16.5
19.0-20/CM-27	4.5	2	2.2	9.7
19.0-18/CM-28	5.1	2	1.7	12.4
19.0-25/CM-29	3.9	6	3.8	9.2
19.0-15/CM-30	4.9	2	2.4	10.9
19.0-30/CM-31	4.6	4	5.4	9.2
19.0-15/CM-32	4.9	4	3.2	8.2
19.0-10/CM-33	5.7	2	4.6	14.7
19.0-20/CM-34	4.4	2	5.4	14.3
19.0-15/CM-35	4.4	2	5.4	14.1
19.0-20/CM-36	5.3	2	5.1	8.7
19.0-15/CM-37	4.8	2	5.0	6.7

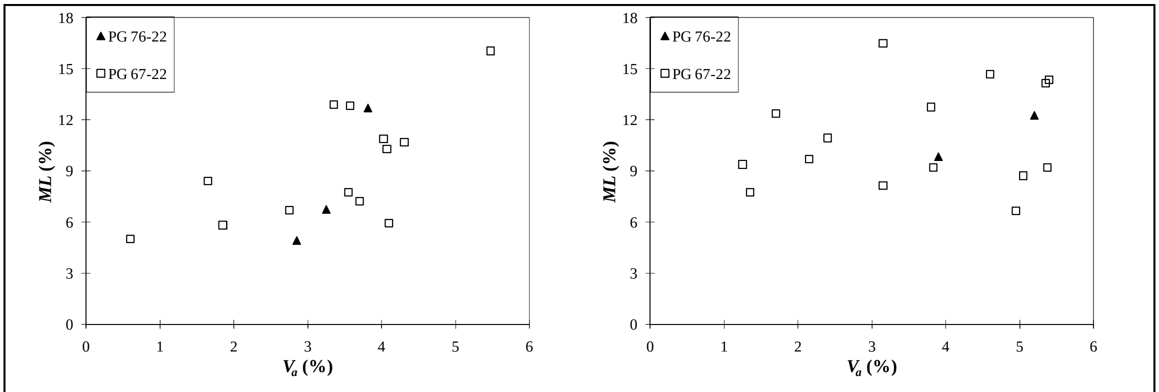


Figure 8.1 Mass Loss Results for Mississippi Mixtures

8.2.1.2 Testing of Specific Control Mixtures

Mass loss results for the specific control highway base mixtures are provided in Table 8.2. Mass loss is seen to range from 11.3 to 14.8% for the 12.5 mm NMAS mixtures. Similar to the random QA specimens, the mass loss of the 19.0 mm NMAS mixture is somewhat higher at 15.5%.

Table 8.2 Mass Loss Results for Control Base Mixtures 1 to 4

Specimen ID	**Rep**	**V_a (%)**	***ML* (%)**
12.5-15/CM-1	1	5.6	9.9
	2	5.0	11.9
	3	4.6	12.7
	Avg.	**5.1**	**11.5**
12.5-15/CM-2	1	5.3	11.5
	2	5.2	10.7
	3	5.2	11.6
	Avg.	**5.2**	**11.3**
12.5-15/CM-3	1	5.3	15.5
	2	4.9	15.3
	3	4.9	13.7
	Avg.	**5.0**	**14.8**
19.0-15/CM-4	1	4.0	14.3
	2	4.8	14.9
	3	4.7	17.3
	Avg.	**4.5**	**15.5**

8.2.2 Tensile Strength Data

Tensile strength results for the control highway base mixtures are provided in Table 8.3. The 19.0 mm NMAS mixture (*19.0-15/CM-4*) had the highest S_t and the 85 design gyration 12.5 mm NMAS mixture (*12.5-15/CM-3*) had the lowest S_t. Overall, tensile strength of the control mixtures ranged from approximately 1100 to 2000 kPa.

Table 8.3 Tensile Strength Results for Control Base Mixtures

Specimen ID	Rep	V_a (%)	S_t (kPa)
12.5-15/CM-1	1	6.6	1277
	2	5.5	1341
	Avg.	**6.0**	**1309**
12.5-15/CM-2	1	4.5	1869
	2	5.1	1917
	Avg.	**4.8**	**1893**
12.5-15/CM-3	1	5.8	1119
	2	5.0	1147
	Avg.	**5.4**	**1133**
19.0-15/CM-4	1	3.2	1843
	2	3.3	2028
	Avg.	**3.3**	**1935**

8.2.3 Rutting Data

8.2.3.1 *APA*

APA rut testing was performed on control highway base mixtures at two air void levels (Table 8.4). Regression equations were fitted to the data as was done in Chapter 7; the linear regression of data between 2,000 and 8,000 passes generally provided very good fit of the data as evidenced by the high R^2 values. Power law regression of the data also generally provided very reasonable fit of the data.

For nominal 7% air voids, the three 12.5 mm NMAS control mixtures ranked as expected in terms of total rut depths, with the 50 design gyration mixture rutting the most (7.9 mm) and the 85 gyration mixture rutting least (3.4 mm). The polymer-modified 19.0 mm NMAS mixture also had low rutting with 7% nominal air void specimens (4.0 mm).

For nominal 10% air voids, the 50 gyration 12.5 mm NMAS mixture had the overall highest total depth as expected (10.2 mm). However, the 65 and 85 gyration mixes had nearly the same total rut depths (lowest rut depth was 6.1 mm). The 19.0 mm NMAS mixture did not perform as well at the higher air void level (total rut depth of 8.4 mm); this may potentially be due to difficulty achieving adequate aggregate interlock.

Table 8.4 *APA* Results for Control Base Mixtures

Mixture	**Avg.**	**Rut Depth (mm)**		**Linear Rutting Rate**[a]			**Power Law**[b]		
ID	V_a **(%)**	**2000**	**8000**	**Slope (10^{-6})**	**Intercept**	R^2	***a***	***b***	R^2
12.5-15/CM-1	7.3	5.1	7.9	456	4.42	0.99	0.187	0.424	0.94
	9.7	6.9	10.2	537	6.14	0.98	0.413	0.363	0.94
12.5-15/CM-2	7.2	3.7	5.7	322	3.18	0.99	0.145	0.413	0.94
	9.9	4.5	6.1	256	4.20	0.98	0.301	0.344	0.89
12.5-15/CM-3[c]	6.9	1.4	3.4	---	---	---	---	---	---
	9.9	3.7	6.3	---	---	---	---	---	---
19.0-15/CM-4	7.0	2.5	4.0	225	2.29	0.96	0.080	0.445	0.97
	10.1	6.0	8.4	384	5.48	0.98	0.323	0.372	0.90

a) Linear rutting rate regression analysis is based on averaged data between 2,000 and 8,000 cycles.
b) Power law regression analysis is based on averaged data and Eq. 2.4.
c) Regression analysis data not available.

8.2.3.2 PURWheel Dry Protocol

PURWheel dry protocol testing was performed on control mixtures one to four. Raw data is located in Tables A.25, A.27, A.29 and A.32. Table 8.5 summarizes PURWheel dry test results for control mixtures.

For 12.5 mm NMAS mixtures, the mixtures rank as expected and in the same order as *APA* results. All specimens of the 50 gyration mixture (*12.5-15/CM-1*) terminated before 20,000 passes due to excessive rutting; average termination was about 8,300 passes. The 65 and 85 gyration mixtures performed similarly, with the 85 gyration mixture having slightly lower average total rutting. The 19.0 mm NMAS mixture ranked the same as the high target air voids *APA* data, namely much better than *CM-1* but not quite as good as *CM-2* or *CM-3*. Overall, control mixtures two to four performed well in PURWheel dry testing with total rut depths on the order of 6 to 8 mm.

Table 8.5 PURWheel Dry Test Results for Control Base Mixtures

			Rut Depth		Linear Rutting Rate[b]			Power Law[c]		
Mixture ID	**V_a (%)[a]**	**Rep**	**Pass**	**mm**	**Slope (10^{-6})**	**Intercept**	**R^2**	***a***	***b***	**R^2**
12.5-15/CM-1	7.0	1-L	8,084	21.9	2600	0.45	0.99	0.012	0.829	0.99
		1-R	7,760	18.5	2200	1.08	0.99	0.019	0.758	0.99
	7.1	2-L	6,618	23.4	3500	0.00	0.99	0.008	0.897	0.96
		2-R	10,982	18.5	1500	2.26	0.99	0.018	0.744	0.99
Average	**7.1**	**---**	**8,361**	**20.6**	**2450**	**0.95**	**---**	**0.014**	**0.807**	**---**
12.5-15/CM-2	5.9	1-L	20 k	7.1	200	2.90	0.98	0.102	0.431	0.94
		1-R	20 k	5.4	200	2.36	0.97	0.126	0.378	0.96
Average	**5.9**	**---**	**20 k**	**6.3**	**200**	**2.63**	**---**	**0.114**	**0.405**	**---**
12.5-15/CM-3	6.8	1-L	20 k	7.0	200	3.42	0.97	0.212	0.354	0.89
		1-R	20 k	3.7	100	1.63	0.96	0.075	0.390	0.92
	7.1	2-L	20 k	6.7	200	2.33	0.99	0.060	0.475	0.95
		2-R	20 k	6.1	200	2.12	0.99	0.088	0.423	0.93
Average	**7.0**	**---**	**20 k**	**5.9**	**175**	**2.38**	**---**	**0.109**	**0.411**	**---**
19.0-15/CM-4	6.3	1-L	20 k	11.6	400	3.53	0.99	0.067	0.519	0.96
		1-R	20 k	7.0	200	3.42	0.97	0.183	0.371	0.91
	6.8	2-L	20 k	7.8	300	3.01	0.98	0.089	0.454	0.94
		2-R	20 k	5.3	200	2.26	0.97	0.077	0.429	0.94
Average	**6.6**	**---**	**20 k**	**7.9**	**275**	**3.06**	**---**	**0.104**	**0.443**	**---**

a) Specimen air voids correlated to *AASHTO T 331*.
b) Linear rutting rate regression analysis is based on averaged data between 2000 and 20,000 passes.
c) Power law regression analysis is based on averaged data and Eq. 2.4.

8.2.4 Moisture Damage Data

8.2.4.1 *TSR*

TSR data for control base mixtures was taken from MDOT mix designs. *TSR* values for control mixtures *CM-1* to *CM-4* were 101.0, 98.1, 98.1 and 92,9% respectively. All four mixtures performed adequately with *TSR* values greater than 90% indicating that moisture damage would likely not be a major concern for these mixtures according to this test method.

8.2.4.2 PURWheel Wet Protocol

PURWheel wet protocol testing was performed on control mixtures one to four. Raw data is found in Tables A.26, A.28, A.30, A.31 and A.33. Table 8.6 summarizes the PURWheel wet test results for control mixtures.

Figure 8.2 presents all PURWheel test data (both wet and dry) for mixture *12.5-15/CM-1*. The wet tests failed slightly sooner than the dry tests with gradual stripping inflection points (SIPs). No loose aggregate or cracking was observed in the visual assessment (Table 8.6).

Figure 8.3 presents all PURWheel test data for mixture *12.5-15/CM-2*. The left wet test specimen immediately began to rut at a faster rate than the dry test and failed relatively quickly, while the right specimen rutted similarly to the dry test for more than half the test before exhibiting moisture damage including cracks and loose aggregate.

Figure 8.4 presents all PURWheel test data for mixture *12.5-15/CM-3*. An additional wet PURWheel test was performed of this mixture (total 3 wet tests and 2 dry tests). Two of the wet test replicates did not exhibit any evidence of moisture damage and their rutting performance was indistinguishable from the dry tests. The other four wet test specimens all exhibited evidence of moisture damage in the data but visually exhibited only bare polished aggregate and minimal evidence of moisture damage.

Figure 8.5 presents all PURWheel test data for mixture *19.0-15/CM-4*. Three of the four wet test specimens exhibited evidence of moisture damage in the rut data. Minimal visual evidence of moisture damage was present.

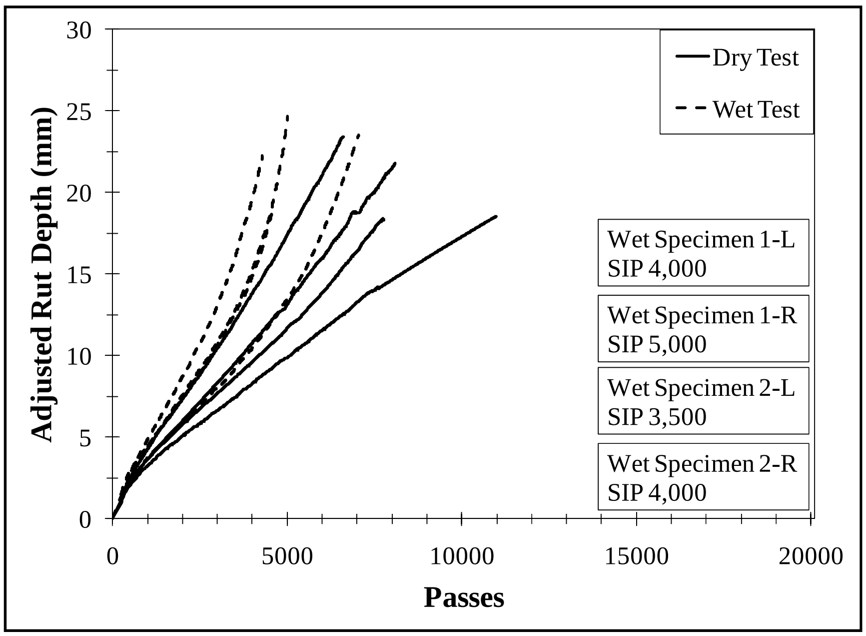


Figure 8.2 PURWheel Test Results for Mixture *12.5-15/CM-1*

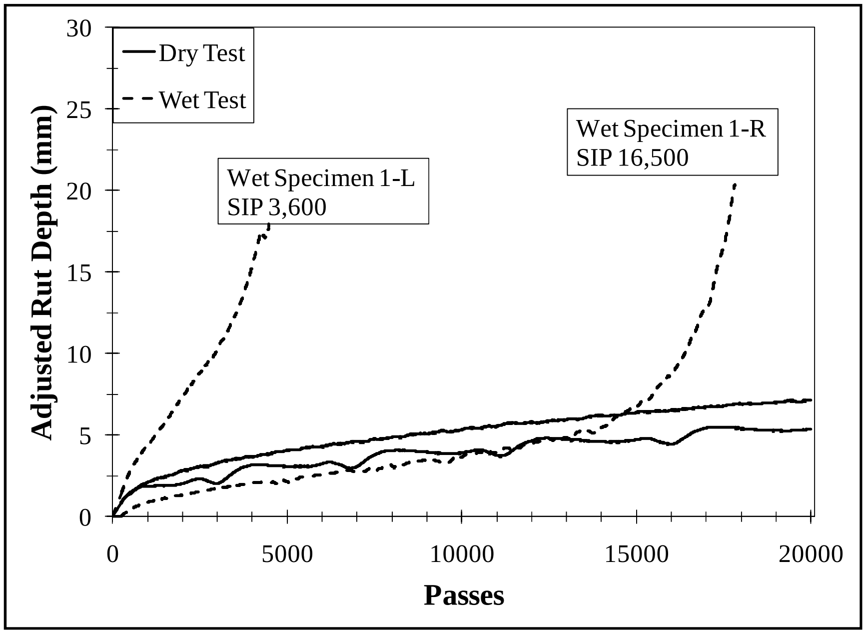


Figure 8.3 PURWheel Test Results for Mixture *12.5-15/CM-2*

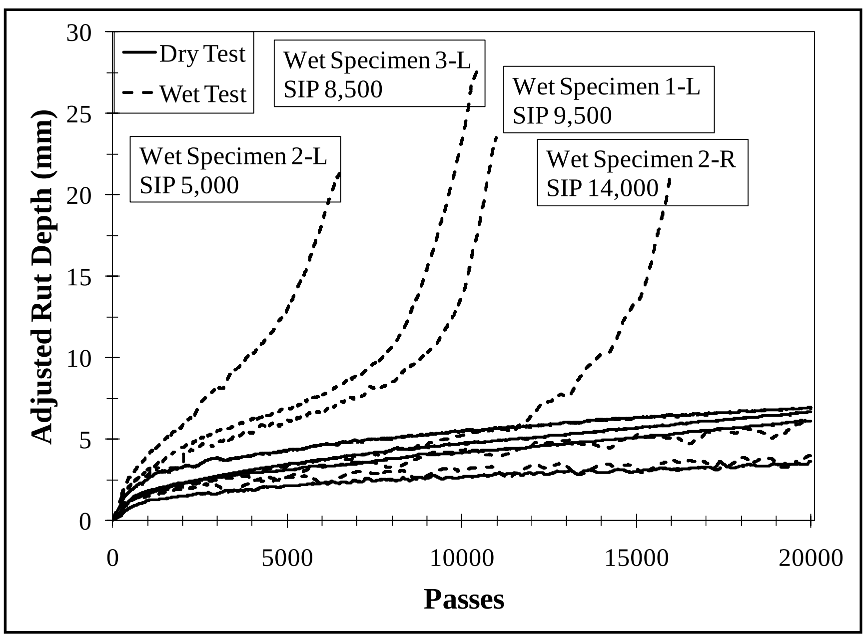


Figure 8.4 PURWheel Test Results for Mixture *12.5-15/CM-3*

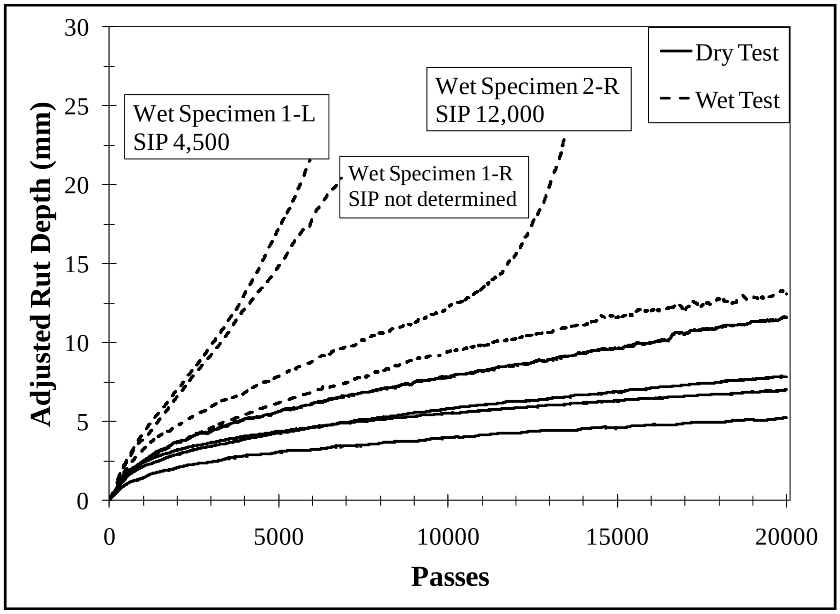


Figure 8.5 PURWheel Test Results for Mixture *19.0-15/CM-4*

Table 8.6 Summary of PURWheel Wet Test Results for Control Base Mixtures

				Failure		Visual Assessment		
Mixture ID	**V_a (%)[a]**	**Rep**	**SIP**	**Pass**	**(mm)**	**Bare Agg.**	**Loose Agg.**	**Crack**
12.5-15/CM-1	6.6	1-L	4,000	5,018	24.7	Yes	No	No
		1-R	5,000	7,046	23.6	Yes	No	No
	6.6	2-L	3,500	4,294	22.3	Yes	No	No
		2-R	4,000	4,560	18.8	Yes	No	No
Average	**6.6**	**---**	**4,125**	**5,230**	**22.4**	---	---	---
12.5-15/CM-2	10.9	1-L	3,600	4,476	18.0	Yes	No	No
		1-R	16,500	17,816	20.4	Yes	Yes	Yes
Average	**10.9**	**---**	**10,050**	**11,146**	**19.2**	---	---	---
12.5-15/CM-3	6.4	1-L	9,500	10,992	23.5	Yes	No	No
		1-R	None	20 k	4.0	Yes	No	No
	6.2	2-L	5,000	6,594	21.6	Yes	No	No
		2-R	14,000	15,976	21.3	Yes	No	Yes
	7.8	3-L	8,500	10,474	28.1	Yes	No	Yes
		3-R	None	20 k	6.1	Yes	No	No
Average	**6.8**	**---**	**12,833**	**14,006**	**17.4**	---	---	---
19.0-15/CM-4	9.4	1-L	4,500	5,920	21.5	Yes	No	No
		1-R	---[b]	6,854	20.5	Yes	No	No
	5.1	2-L	None	20 k	13.1	Yes	No	No
		2-R	12,000	13,426	23.0	Yes	No	Yes
Average	**7.3**	**---**	**12,167**	**11,550**	**19.5**	---	---	---

Note: When no SIP was observed, 20,000 passes was used to calculate an average SIP.

a) Specimen air voids correlated to *AASHTO T 331*.

b) Specimen terminated prematurely but a SIP could not be determined.

8.3 Highway Base 50 and 75% RAP Mixture Results

8.3.1 Volumetric Experiment

Results of the designed experiment to investigate effects of RAP source, RAP content and Sasobit® dosage rate on volumetric properties are given in Table 8.7. The most visible result was Sasobit® dosage rate had almost no effect on design total asphalt content for these high RAP content mixtures. Interestingly, the total asphalt contents for a particular RAP source did not change much if at all between 50 and 75% RAP contents; this may be due to the differences in gradation necessitated by the very high RAP contents. However for a given RAP content, the *R-1* RAP source had noticeably higher total asphalt contents than either other RAP source; the difference is about 1%. This result coincides with the results presented in Chapter 5 concerning differences in effective RAP asphalt contents for different RAP sources at warm mix temperatures.

Table 8.7 Results of Highway Base Mixtures Volumetric Experiment

RAP Content	**RAP Source**	**Sasobit® Content (%)**	P_{AC} **(%)**	$P_{b(V)}$ **(%)**	S_t **(kPa)**
50%	*R-1*	1.0	7.3	4.6	1922
		1.5	7.3	4.6	1947
	R-2	1.0	6.1	3.3	2116
		1.5	6.2	3.4	2167
	R-3	1.0	6.3	3.8	1942
		1.5	6.4	3.9	1960
75%	*R-1*	1.0	7.3	3.3	2535
		1.5	7.2	3.1	2409
	R-2	1.0	6.1	1.9	2845
		1.5	6.1	1.9	2750
	R-3	1.0	6.2	2.5	2138
		1.5	6.2	2.5	2493

Note: All mixtures designed with 50 gyrations compactive effort.
Mixing and compaction temperatures were both 116 C.

8.3.2 Cantabro Durability Data

Durability results for the mixtures are presented in Table 8.8. 50% RAP mixtures had lower mass loss than 75% RAP mixtures and similar levels of mass loss to the upper end of the range observed for control mixtures. 75% RAP mixtures had mass loss slightly above the range observed for control mixtures but not dramatically so.

The *R-1* RAP source outperforms the *R-2* RAP source in both 50 and 75% RAP mixtures. This is an interesting result since the *R-1* RAP source contains stiffer asphalt than the *R-2* source and had higher mass loss in 100% RAP testing. The difference in performance is most likely due to the much higher amount of virgin asphalt in the mixtures made with *R-1* RAP as seen in Table 8.7.

Table 8.8 Mass Loss Results for 50 and 75% RAP Recycled Mixtures

Specimen ID	Rep	V_a (%)	*ML* (%)
12.5-50/RM-1	1	3.9	14.8
	2	4.6	12.2
	3	4.3	13.2
	Avg.	**4.3**	**13.4**
12.5-50/RM-2	1	4.6	14.8
	2	4.9	14.7
	3	4.8	17.9
	Avg.	**4.7**	**15.8**
12.5-75/RM-1	1	3.8	18.6
	2	3.7	19.2
	3	3.2	16.7
	Avg.	**3.5**	**18.1**
12.5-75/RM-2	1	4.8	21.4
	2	4.6	19.9
	3	4.6	22.9
	Avg.	**4.7**	**21.4**

8.3.3 Tensile Strength Data

Indirect tensile strength of the 50 and 75% RAP mixtures was given in Table 8.7. The results were generally similar to those observed for Cantabro testing. 50% RAP mixtures had lower tensile strengths than 75% RAP mixtures (i.e. less brittle) and were near the upper end of tensile strengths observed for control mixtures. 75% RAP mixtures had tensile strengths slightly above that of control mixtures. Mixtures with *R-1* RAP had lower tensile strengths than those with *R-2* RAP.

8.3.4 Rutting Data

8.3.4.1 *APA*

Results of *APA* rut testing on the 50 and 75% RAP mixtures at two target air void levels are given in Table 8.9. Total rut depths were generally very small for all mixtures and comparable to or better than the best performing control mixtures. At low voids, rut depths were 4 mm or less and at 10% target voids rutting was about 5 mm or less.

Table 8.9 *APA* Results for 50 and 75% RAP Recycled Mixtures

Mixture	Avg.	Rut Depth (mm)		Linear Rutting Rate[a]			Power Law[b]		
ID	V_a (%)	2000	8000	Slope (10^{-6})	Intercept	R^2	*a*	*b*	R^2
12.5-50/RM-1	6.1	2.2	3.9	268	1.97	0.94	0.047	0.502	0.99
	9.0	3.5	5.2	270	3.21	0.95	0.106	0.445	0.95
12.5-50/RM-2	6.5	1.8	3.5	277	1.31	0.99	0.061	0.448	0.99
	9.1	2.0	3.6	269	1.56	0.99	0.068	0.442	0.99
12.5-75/RM-2	6.5	1.0	1.7	107	0.85	0.99	0.086	0.328	0.98
	9.1	1.2	2.0	112	1.07	0.99	0.070	0.372	0.98
12.5-75/RM-2	6.8	1.4	2.4	170	1.03	0.98	0.060	0.408	0.98
	9.2	1.9	3.7	274	1.58	0.99	0.055	0.471	0.99

a) Linear rutting rate regression analysis is based on averaged data between 2,000 and 8,000 cycles.
b) Power law regression analysis is based on averaged data and Eq. 2.4.

8.3.4.2 PURWheel Dry Protocol

Summary results of PURWheel dry protocol testing on 50 and 75% RAP mixtures are presented in Table 8.10. Raw data is provided in Tables A.34, A.36, A.38 and A.40. Mixtures containing *R-1* RAP generally rutted more than those containing *R-2* RAP. This is likely due to the higher virgin asphalt contents in the *R-1* mixtures. Mixtures with *R-2* RAP had total rut depths comparable to the best performing control mixtures. Mixtures with *R-1* RAP had much better rut performance than control mixture one but not quite as good as the other control mixtures or the mixtures with *R-2* RAP.

Table 8.10 PURWheel Dry Test Results for 50 and 75% RAP Recycled Mixtures

			Rut Depth		Linear Rutting Rate[b]			Power Law[c]		
Mixture ID	**V_a (%)[a]**	**Rep**	**Pass**	**mm**	**Slope (10^{-6})**	**Intercept**	**R^2**	***a***	***b***	**R^2**
12.5-50/RM-1	7.6	1-L	20 k	12.0	400	3.83	0.99	0.113	0.466	0.94
		1-R	20 k	5.7	200	2.28	0.98	0.083	0.426	0.94
	5.8	2-L	20 k	9.1	300	2.90	0.99	0.069	0.492	0.95
		2-R	20 k	6.3	200	2.61	0.98	0.085	0.437	0.94
Average	**6.7**	**---**	**20 k**	**8.3**	**275**	**2.91**	**---**	**0.088**	**0.455**	**---**
12.5-50/RM-2	4.3	1-L	20 k	6.5	100	2.93	0.96	0.192	0.343	0.88
		1-R	20 k	7.3	200	3.50	0.95	0.126	0.415	0.92
	9.5	2-L	20 k	5.9	200	2.37	0.97	0.070	0.450	0.94
		2-R	20 k	5.9	200	2.28	0.98	0.087	0.425	0.94
Average	**6.9**	**---**	**20 k**	**6.4**	**175**	**2.77**	**---**	**0.119**	**0.408**	**---**
12.5-75/RM-1	9.5	1-L	20 k	8.1	300	2.25	0.99	0.040	0.535	0.96
		1-R	20 k	10.4	400	2.94	0.99	0.046	0.550	0.96
	11.8	2-L	20 k	10.5	400	3.13	0.99	0.058	0.525	0.96
		2-R	20 k	17.6	700	3.07	0.99	0.041	0.605	0.97
Average	**10.7**	**---**	**20 k**	**11.7**	**450**	**2.85**	**---**	**0.046**	**0.554**	**---**
12.5-75/RM-2	8.1	1-L	20 k	4.3	200	1.79	0.97	0.072	0.418	0.94
		1-R	20 k	4.9	200	1.93	0.98	0.067	0.434	0.94
	9.1	2-L	20 k	5.8	200	2.20	0.98	0.052	0.478	0.95
		2-R	20 k	6.1	200	2.22	0.98	0.056	0.475	0.95
Average	**8.6**	**---**	**20 k**	**5.3**	**200**	**2.03**	**---**	**0.062**	**0.451**	**---**

a) Specimen air voids correlated to *AASHTO T 331*.
b) Linear rutting rate regression analysis is based on averaged data between 2000 and 20,000 passes.
c) Power law regression analysis is based on averaged data and Eq. 2.4.

8.3.5 Moisture Damage Data

8.3.5.1 *TSR*

Results of *TSR* moisture damage testing performed for 50 and 75% RAP mixtures are provided in Table 8.11. All the mixtures pass the commonly utilized criterion of *TSR* value of 80% or greater. The *12.5-50/RM-1* mixture was borderline.

Table 8.11 *TSR* Results for 50 and 75% RAP Recycled Mixtures

Mixture	Conditioned Set			Un-Conditioned Set		
ID	Avg. V_a (%)	Sat (%)	S_t (kPa)	Avg. V_a (%)	S_t (kPa)	*TSR* (%)
12.5-50/RM-1	6.2	59.6	1629	6.2	2036	80.0
12.5-50/RM-2	6.1	60.9	2130	6.4	2351	90.6
12.5-75/RM-1	5.9	58.8	2374	5.9	2361	100.6
12.5-75/RM-2	7.1	65.9	2091	6.9	2370	88.2

8.3.5.2 PURWheel Wet Protocol

PURWheel wet protocol testing was performed for 50 and 75% RAP mixtures. The raw data is located in Tables A.35, A.37, A.39 and A.41. Results of the wet PURWheel testing are summarized in Table 8.12. Figures 8.6 to 8.9 present all PURWheel testing (both wet and dry) for 50 and 75% RAP mixtures.

Figure 8.6 presents PURWheel results for mixture *12.5-50/RM-1*. Three of the four wet test replicates exhibited evidence of moisture damage in the rut data; average SIP was nearly 9,500 passes. There was no visual evidence of stripping.

Figure 8.7 presents PURWheel results for mixture *12.5-50/RM-2*. Three of the four wet test replicates exhibited evidence of moisture damage in the rut data and the fourth replicate rutted faster than the dry tests; average SIP was nearly 10,500 passes. There was no visual evidence of stripping.

Figure 8.8 presents PURWheel results for mixture *12.5-75/RM-1*. Three of the four wet test replicates exhibited evidence of moisture damage in the rut data; average SIP was about 13,500 passes. There was minimal visual evidence of stripping.

Figure 8.9 presents PURWheel results for mixture *12.5-75/RM-2*. Two of the four wet test replicates exhibited evidence of moisture damage in the rut data while the other two did not; average SIP was about 16,500 passes. There was no visual evidence of stripping.

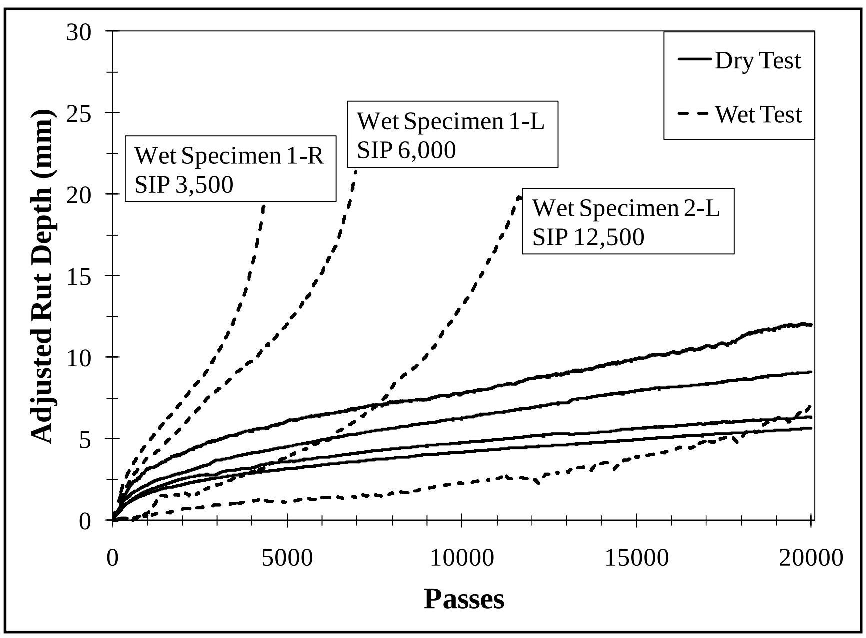


Figure 8.6 PURWheel Test Results for Mixture *12.5-50/RM-1*

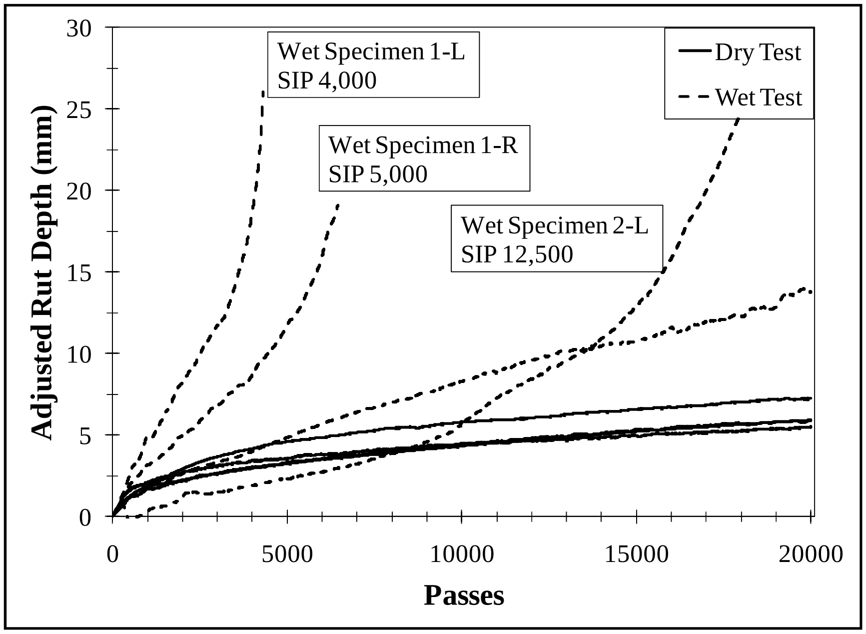


Figure 8.7 PURWheel Test Results for Mixture *12.5-50/RM-2*

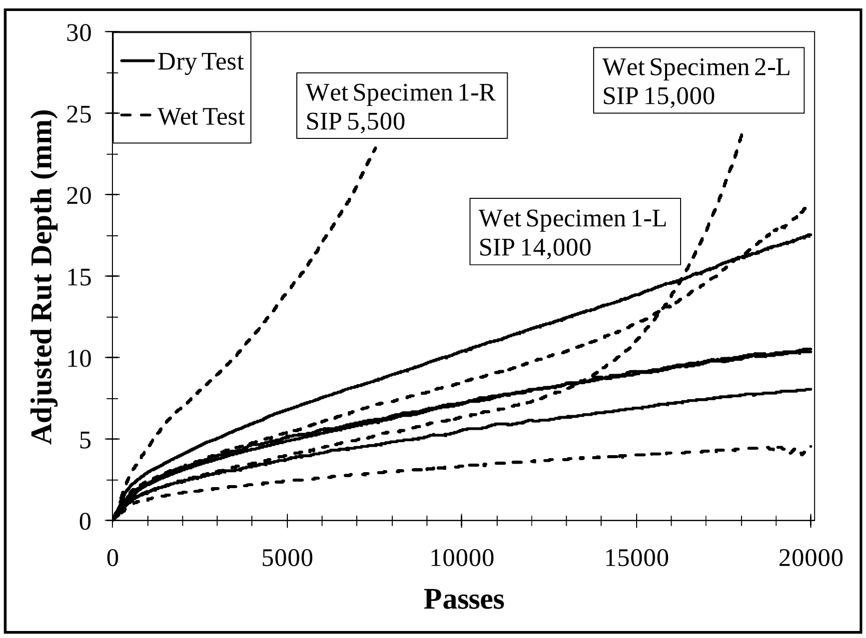


Figure 8.8 PURWheel Test Results for Mixture *12.5-75/RM-1*

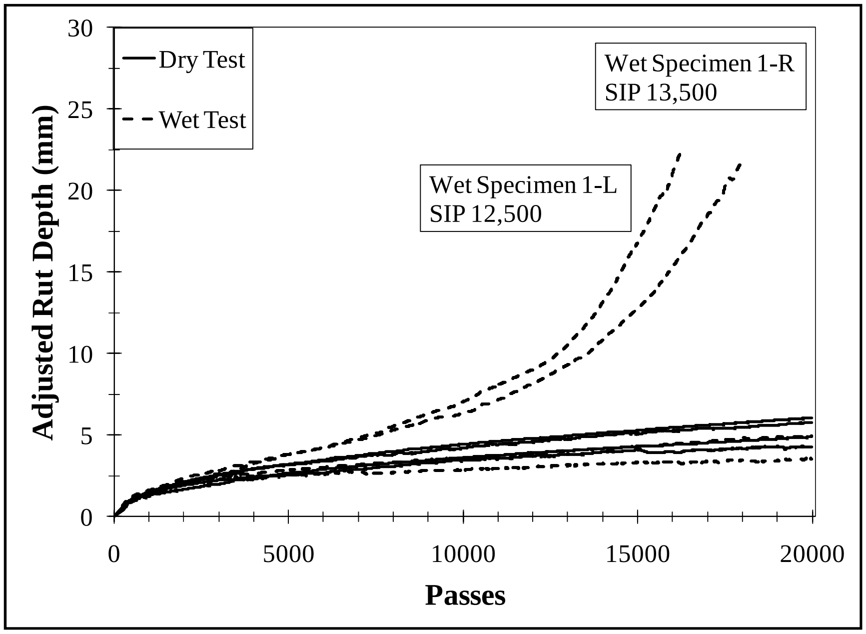


Figure 8.9 PURWheel Test Results for Mixture *12.5-75/RM-2*

Table 8.12 Summary of PURWheel Wet Test Results for 50 and 75% RAP Mixtures

				Failure		Visual Assessment		
Mixture ID	**V_a (%)[a]**	**Rep**	**SIP**	**Pass**	**(mm)**	**Bare Agg.**	**Loose Agg.**	**Crack**
12.5-50/RM-1	11.1	1-L	6,000	6,956	21.4	Yes	No	No
		1-R	3,500	4,370	19.2	Yes	No	No
	7.0	2-L	8,000	11,608	19.8	Yes	No	No
		2-R	None	20 k	7.2	Yes	No	No
Average	**9.1**	**---**	**9,375**	**10,734**	**16.9**	**---**	**---**	**---**
12.5-50/RM-2	11.1	1-L	4,000	4,318	26.1	Yes	No	No
		1-R	5,000	6,454	19.1	Yes	No	No
	6.5	2-L	12,500	17,960	24.4	Yes	No	No
		2-R	None	20 k	13.8	Yes	No	No
Average	**8.8**	**---**	**10,375**	**12,183**	**20.9**	**---**	**---**	**---**
12.5-75/RM-1	10.9	1-L	14,000	20 k	19.5	Yes	No	No
		1-R	5,500	7,848	22.4	Yes	No	No
	10.0	2-L	15,000	18,098	24.1	Yes	No	Yes
		2-R	None	20 k	4.6	Yes	No	No
Average	**10.5**	**---**	**13,625**	**15,315**	**17.7**	**---**	**---**	**---**
12.5-75/RM-2	7.6	1-L	12,500	16,248	22.5	Yes	No	No
		1-R	13,500	17,940	21.6	Yes	No	No
	6.6	1-L	None	20 k	3.6	Yes	No	No
		1-R	None	20 k	4.4	Yes	No	No
Average	**7.1**	**---**	**16,500**	**18,547**	**13.0**	**---**	**---**	**---**

Note: When no SIP was observed, 20,000 passes was used to calculate an average SIP.
a) Specimen air voids correlated to *AASHTO T 331*.

CHAPTER 9

HIGH RAP-WMA COMPACTABILITY

9.1 High RAP-WMA Compactability Overview

This chapter contains analysis of relative compactability for highway mixtures using laboratory gyratory compaction data. After review of literature, six parameters were selected for the compactability analysis: 1) percent of G_{mm} at 0 gyrations ($\%N_0$); 2) percent of G_{mm} at N_{ini} ($\%N_{ini}$); 3) number of gyrations to 92% of G_{mm} ($N_{92\%}$); 4) number of gyrations to mixture locking point (N_{LP}); 5) gyratory compaction slope (m_G); and 6) gyratory compaction intercept (b_G).

Research by Leiva and West (2008a, 2008b) showed that several parameters can be used to assess relative compactability of mixtures in the laboratory. Leiva and West (2008a) found gradation type to be the principal factor affecting laboratory compaction characteristics. Fine graded mixtures were the easiest to compact and SMA mixes were the toughest (Leiva and West 2008a). Values for $\%N_{ini}$ ranged from 85.0 to 89.4 for the data. Values for $N_{92\%}$ ranged from 17.6 to 37.5. Values for N_{LP} were as low as 35 for fine graded mixes and as high as 60 for SMA. Values for m_G were observed below 7 for fine graded mixes and as high as 11.6 for SMA. In total, 81 mixes were evaluated and in general it was observed that limestone aggregate was somewhat more difficult to compact than gravel aggregate. The three parameters $N_{92\%}$, N_{LP} and m_G were mutually well correlated but not well correlated to field compaction data. The best correlation between laboratory and field compaction parameters was for $N_{92\%}$ with $R^2 = 0.45$ (Leiva and West 2008b). The intercept parameter b_G may potentially be affected by binder content and viscosity (Çelik and Atiş 2008).

9.2 Compactability Analysis

To calculate the six compactability parameters for each specimen, height data collected during gyratory compaction was first converted to percentage of G_{mm} using a correction factor calculated according to the procedure of Vavrick and Carpenter (1998). The first four parameters were observed directly from the data. Mixture locking point was defined as the first instance of two consecutive gyrations with the same specimen height (Leiva and West 2008a). Data was then plotted with number of gyrations on the horizontal axis with a semi logarithmic scale and linear regression performed of the data between N_{ini} and N_{des}; slope and intercept were the fifth and sixth parameters. Compaction data for three replicate specimens of each mixture (except *12.5-15/CM-3* for which data was not available) was evaluated.

Lower values of the parameter $\%N_0$ are interpreted to be mixtures that are potentially more difficult to place or initially compact. Lower values of $\%N_{ini}$ represent mixes that are tougher to compact (Leiva and West 2008a). Mixes with high values of $N_{92\%}$, N_{LP} or m_G are considered to be tougher to compact in the laboratory (Leiva and West 2008a). Lower values for b_G are also considered to represent tough to compact mixes (Chadbourn et al. 1998).

9.2.1 Airfield Surface Mixtures

Results of compactability analysis for airfield surface mixtures are presented in Table 9.1. It is apparent that that warm mix additive does not produce much change in compactability. ANOVA statistical analysis was performed of the data for each response parameter. Results were the same for each response variable and indicated that factors of RAP content and aggregate type were statistically significant while warm mix was not. In addition, the interaction between RAP content and aggregate type was a significant effect for all response variables.

Table 9.1 Compactability Data for Airfield Surface Mixtures

Mixture	**Average Compactability Parameters**					
ID	**$\%N_0$**	**$\%N_{ini}$**	**$N_{92\%}$**	**N_{LP}**	**m_G**	**b_G**
12.5-0/AM-1	74.9	84.8	33	59	10.7	76
12.5-0/AM-2	74.7	84.8	32	60	10.7	76
12.5-0/AM-3	74.6	84.6	34	58	10.6	76
12.5-0/AM-4	75.1	85.1	30	58	10.8	76
12.5-25/AM-5	76.9	86.5	25	55	9.6	78
12.5-25/AM-6	76.9	86.5	25	58	9.6	79
12.5-25/AM-7	76.7	86.3	26	52	9.6	78
12.5-25/AM-8	77.0	86.5	26	53	9.5	79
12.5-50/AM-9	78.5	87.3	24	50	8.4	80
12.5-50/AM-10	78.0	87.0	25	51	8.6	80
12.5-50/AM-11	78.0	87.1	24	51	8.6	80
12.5-50/AM-12	78.5	87.3	23	49	8.6	80
12.5-0/AM-13	77.8	87.1	25	52	8.6	80
12.5-0/AM-14	77.4	86.7	27	49	8.5	80
12.5-0/AM-15	77.5	86.9	26	48	8.4	80
12.5-0/AM-16	77.7	87.0	25	50	8.6	80
12.5-25/AM-17	77.8	87.2	24	49	8.6	80
12.5-25/AM-18	77.9	87.2	24	48	8.6	80
12.5-25/AM-19	77.9	87.2	24	51	8.6	80
12.5-25/AM-20	77.6	86.8	25	50	8.8	80
12.5-50/AM-21	78.3	87.6	22	48	8.3	81
12.5-50/AM-22	78.5	87.5	22	50	8.4	81
12.5-50/AM-23	78.1	87.2	24	49	8.5	80
12.5-50/AM-24	78.2	87.0	25	50	8.5	80

9.2.2 Highway Surface Mixtures

Results of compactability analysis for highway surface mixtures are presented in Table 9.2. Control mixture 4 appears to be noticeably more difficult to compact than the other control mixtures according to $\%N_0$, $\%N_{ini}$, $N_{92\%}$ and b_G parameters. Results for the 25 and 50% RAP mixtures fall generally within the range of control mixtures.

Table 9.2 Compactability Data for Highway Surface Mixtures

Mixture ID	**Average Compactability Parameters**					
	$\%N_0$	**$\%N_{ini}$**	**$N_{92\%}$**	**N_{LP}**	**m_G**	**b_G**
9.5-0/CM-1	77.6	87.2	22	49	8.8	80
9.5-15/CM-2	79.1	87.5	21	46	7.9	82
9.5-15/CM-3	77.4	87.3	21	50	8.6	80
9.5-15/CM-4a	73.8	83.6	49	55	9.5	76
9.5-15/CM-4b	75.1	84.6	47	51	8.6	78
9.5-15/CM-4c	75.5	85.3	36	49	8.8	78
9.5-25/RM-1	78.4	87.7	21	47	8.4	81
9.5-25/RM-2	78.2	87.4	23	45	8.2	81
9.5-50/RM-1	78.9	87.6	23	45	7.9	81
9.5-50/RM-2	79.4	88.1	21	46	7.6	82

9.2.3 Highway Base Mixtures

Results of compactability analysis for highway base mixtures are presented in Table 9.3. Compaction data was not available for control mixture 3. Control mixture 1 appears to be somewhat easier to compact than the other control mixtures according to all six parameters. This result is thought likely due to *CM-1* being the only fine-graded highway base mixture evaluated. Results for 50 and 75% RAP mixtures generally fall within the range of results observed for control mixtures.

Table 9.3 Compactability Data for Highway Base Mixtures

Mixture ID	Average Compactability Parameters					
	$\%N_0$	**$\%N_{ini}$**	**$N_{92\%}$**	**N_{LP}**	**m_G**	**b_G**
12.5-15/CM-1	81.7	89.3	16	34	5.9	85
12.5-15/CM-2	77.4	86.9	26	49	8.5	80
19.0-15/CM-4	78.9	87.6	26	44	7.3	82
12.5-50/RM-1	78.9	87.6	26	44	7.3	82
12.5-50/RM-2	79.1	87.3	20	44	8.6	81
12.5-75/RM-1	80.7	88.6	15	44	8.5	82
12.5-75/RM-2	80.1	87.8	19	43	8.1	82

Note: Compaction data was not available for mixture *12.5-15/CM-3*.

9.2.4 100% RAP Mixtures

Results of compactability analysis for 100% RAP mixtures are presented in Table 9.4. All three RAP sources gave generally comparable performance in terms of compactability. All six parameters indicate that *R-1* RAP source was toughest to compact. Data for 100% RAP mixtures will be used for results discussion in Chapter 10.

Table 9.4 Compactability Data for 100% RAP Mixtures

Mixture ID	Average Compactability Parameters					
	$\%N_0$	**$\%N_{ini}$**	**$N_{92\%}$**	**N_{LP}**	**m_G**	**b_G**
9.5-100/RM-1	79.5	87.6	23	45	8.1	81
9.5-100/RM-2	80.7	88.9	17	39	7.1	83
12.5-100/RM-3	81.2	88.7	20	38	6.5	84

9.3 Summary of Compactability Analysis

Six parameters were chosen for the compactability analysis that are easy to compute from gyratory compaction data. Previous research by Leiva and West (2008a) has indicated that some of the parameters may be strongly correlated. The current data set was investigated for correlations between parameters using compaction data for all 43 mixtures. Results are presented in Table 9.5 in terms of Pearson correlation coefficients for each combination of variables. It is observed that very strong correlations are present between most of the variables. This result aligns with that of Leiva and West (2008a).

Table 9.5 Pearson Correlation Coefficient Matrix for Compactability Parameters

	$\%N_0$	$\%N_{ini}$	$N_{92\%}$	N_{LP}	m_G
$\%N_{ini}$	0.967				
$N_{92\%}$	-0.851	-0.913			
N_{LP}	-0.818	-0.757	0.540		
m_G	-0.828	-0.788	0.496	0.882	
b_G	0.966	0.961	-0.786	-0.866	-0.921

CHAPTER 10

DISCUSSION OF RESULTS

10.1 Results Discussion Overview

This chapter presents discussion of mixture volumetrics and performance testing results for the high RAP-WMA mixtures that were presented in Chapters 6, 7, 8 and 9. Performance of the recycled mixtures is discussed in context of performance of the low RAP content control mixtures and also performance of 100% RAP mixtures that was presented in Chapter 5. Four major categories of mixture performance were investigated in this study: 1) durability; 2) crack resistance; 3) rut resistance; and 4) moisture susceptibility. Discussion of each performance category is divided into subsections for each category of intended pavement application (i.e. airfield or highway mixtures and surface or base mixtures) as well as overall discussion of high RAP mixtures.

10.2 Volumetrics

10.2.1 Airfield Surface Mixtures

In general the total and asphalt contents of the mixtures containing limestone virgin aggregate are lower than those containing crushed gravel aggregate; this is reasonable due to the higher specific gravity and lower asphalt absorption of the limestone aggregate relative to the gravel aggregate. For the 25% RAP limestone mixes the ratio of virgin asphalt to asphalt contributed by the RAP is roughly 75:25. For the 50% RAP limestone mixes the ratio is roughly 55:45. For the gravel mixes the ratios are roughly 80:20 and 60:40 for the 25% and 50% RAP mixes, respectively.

There is a large reduction in total asphalt content in the 0% RAP gravel mixes from the hot mix (*12.5-0/AM-13*) to the warm mixes with additives (*12.5-0/AM-14* and *12.5-0/AM-15*). A portion of this reduction is thought to be due to reduced absorption of asphalt binder by the gravel aggregate at lower short term aging temperatures. P_{ba} for mixtures *AM-14* and *AM-15* is 0.2% less than for mixture *AM-13*. A 0.4% reduction in P_{be} is also seen for mixtures *AM-14* and *AM-15* compared to mixture *AM-13*.

For the 50% RAP crushed gravel mixtures, an increase in total asphalt content is noted in mixtures *12.5-50/AM-22* (Sasobit®) and *12.5-50/AM-23* (Evotherm™ 3G) compared to mixture *12.5-50/AM-21* (hot mix). This is thought to be partially due to reduced rejuvenating of the RAP surface asphalt at the lower temperature compared to the hot mix; additional virgin binder is therefore required to achieve compaction. For example the total asphalt content of mix *AM-14* (0% RAP with Sasobit®) is 6.0% and the total asphalt content of mix *AM-22* (50% RAP with Sasobit®) is 7.0%. The two aggregate gradations are of nearly identical shape and mix *AM-22* contains just over half the amount of virgin gravel aggregate that mix *AM-14* does. It was shown that RAP aggregate does not absorb additional virgin asphalt and that the virgin aggregate absorbs less asphalt at a lower short term aging temperature. The additional 1.0% of total asphalt can therefore be at least partly explained by a reduction in rejuvenation of the RAP surface asphalt. Chapter 5 RAP characterization results support this conclusion.

10.2.2 Highway Surface Mixtures

Characterization of RAP volumetrics showed that the virgin asphalt demand of the *R-1* and *R-2* RAP sources varied depending on compaction temperature and other factors. This difference is evident in volumetric properties of the 25 and 50% RAP recycled surface mixtures produced at a warm mix temperature of 116 C. The *9.5-25/RM-1* RAP recycled mixture required 0.5% more virgin asphalt than did the *R-2* RAP mixture (*9.5-25/RM-2*) with the same virgin aggregate proportions and comparable total gradation. The *9.5-50/RM-1* RAP recycled mixture required 0.3% more virgin asphalt than did the *R-2* RAP mixture (*9.5-50/RM-2*) with the same virgin aggregate proportions and comparable total gradation. Testing a single source RAP with added virgin binder has promise in detecting asphalt demand in a new mixture as the data in Chapter 5 showed *R-1* bitumen was more difficult to re-liven than *R-2* bitumen, which agrees with the mixture data. Effects of parameters such as heating temperature and heating time can also be detected on 100% RAP, at least to some extent.

10.2.3 Highway Base Mixtures

Differences in aggregate gradations make the volumetrics of high RAP highway base mixtures more difficult to compare. For both 50 and 75% RAP, mixtures containing *R-1* RAP source required at least 1% more virgin binder than those containing *R-2* RAP source. This result agrees with Chapter 5 results.

10.2.4 All Mixtures

The results of Chapter 5 indicated that varying temperatures would change the amount of RAP bitumen that will re-liven and that the temperature dependent behaviors were different for each RAP source. The temperature dependence of *R-1* RAP source was observed in the volumetric data for airfield mixtures. Differences in RAP sources were observed in the volumetric data for highway mixtures.

10.3 Durability

Selecting and using a good durability test is difficult since there is not general agreement on what test is useful for predicting durability. For this study the Cantabro test was used to assess durability. It has shown some potential to be useful for porous mixtures and it seems to have some potential for use with dense graded mixes. It should at least be capable of providing relative rankings of the mixtures.

10.3.1 Airfield Surface Mixtures

Durability is a major concern for the surface of airfield pavements due to the potential for foreign object debris (FOD) causing damage to aircraft. Specimens were compacted with design compactive effort. Only the *R-1* RAP source was investigated for this component of the research. Results of the designed experiment indicated that use of warm mix was not a statistically significant factor on mass loss, but that RAP content was; virgin aggregate type was only significant for 0% RAP mixtures. Figure 10.1 presents the results organized by RAP content and virgin aggregate type. Mass loss increases as the RAP content increases.

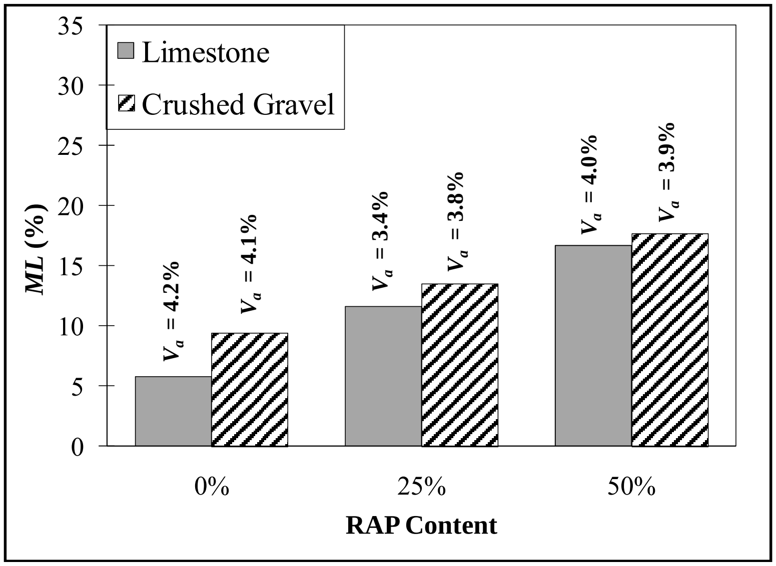


Figure 10.1 Cantabro Mass Loss for Airfield Surface Mixtures

10.3.2 Highway Surface Mixtures

Durability is also a concern for highway surface mixtures. Since the Cantabro test has seldom been used for dense graded mixtures, a random selection of QA specimens was tested to establish a baseline of expected performance of conventional practice Mississippi surface mixtures. Data in previous chapters showed a considerable effect of air voids, which should be considered when interpreting the data presented in this section.

Figure 10.2*a* presents test results of un-aged Cantabro durability specimens compacted to N_{des} as this would be most desirable for use as a routine quality control tool. The band of results determined for control mixtures (2.8 to 11.7%) is represented by horizontal dashed lines. Mass loss for 25% RAP recycled mixtures was on the order of 12 to 13% and comparable to the upper end of Cantabro performance observed for control mixtures. Mass loss for 50% RAP mixtures was on the order of 14 to 17% which was slightly higher than the observed range of performance for control mixtures. For the

R-1 RAP source, increasing the amount of RAP from 25 to 50% caused an increase in mass loss of about 5%. For the *R-2* RAP source, increasing the amount of RAP from 25 to 50% caused an increase in mass loss of about 1%. Mass loss of all control, 25 and 50% RAP mixtures was less than the 20% upper limit for mass loss recommended in literature for OGFC and PFC mixtures.

Figure 10.2*b* presents test results of un-aged Cantabro specimens compacted to target air voids. Mass loss of both the control mixtures was about 8%. Mass loss of 25% RAP mixtures was about 11.5% for both RAP sources. For 50% RAP mixtures, the *R-1* RAP source mixture had higher mass loss (16.5%) than the *R-2* RAP mixture (10%).

Testing of 100% RAP indicated that mixtures containing *R-1* RAP would likely have higher mass loss than mixtures containing *R-2* RAP. Cantabro testing of 100% RAP mixtures successfully predicted the relative performance of the different RAP sources in 50% RAP mixtures. For 25% RAP mixtures, Cantabro performance was similar regardless of RAP source.

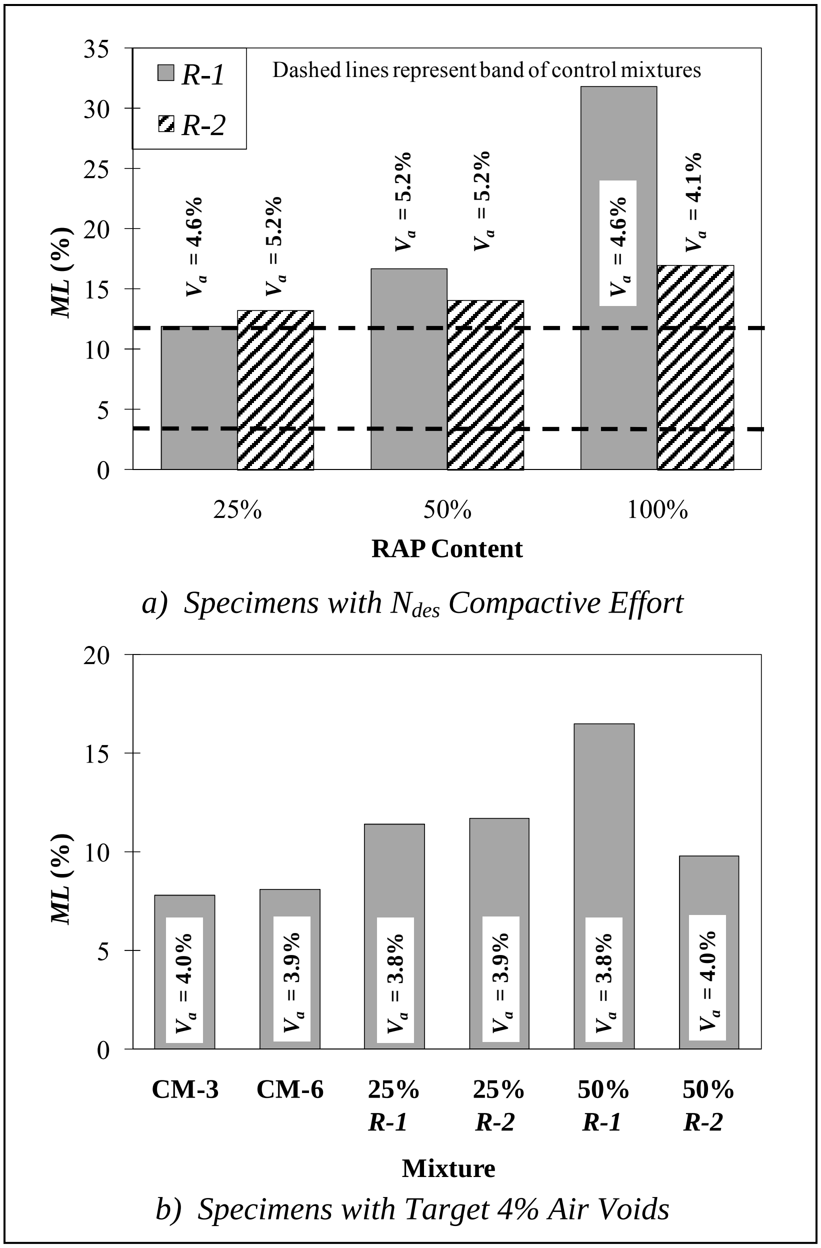


Figure 10.2 Effect of RAP Source on Cantabro Mass Loss for Un-Aged Specimens

The effects of *R-30* aging on mass loss are presented in Figure 10.3. Control mixtures designed according to current practice experienced an increase in mass loss of 2 to 4% compared to un-aged specimens. For 25% and 50% RAP, the increase in mass loss with aging was about 6% and 9% above un-aged results, respectively. The data indicates that specimens with more RAP may become more prone to durability problems over time than would conventional mixtures; more investigation is needed to fully explain this.

Overall, results indicated that mixtures with high RAP may be somewhat more prone to durability issues than current practice mixtures, but the data did not indicate that durability problems would prohibit their use. At higher RAP contents not all RAP sources will give the same level of performance at a particular percentage of total mixture. None of the results indicated that use of high RAP in surface mixtures would not be feasible.

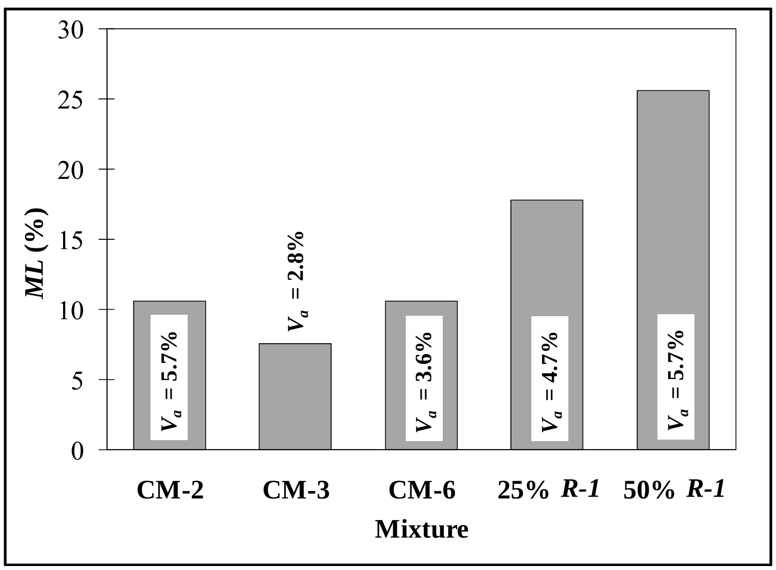


Figure 10.3 Effect of Laboratory Aging on Cantabro Mass Loss for Select Mixtures

10.3.3 Highway Base Mixtures

Durability is less of a concern for base mixtures than for surface mixtures due to the lack of direct exposure to traffic; however the Cantabro test can still provide an indication of mixture performance when compared to conventional practice mixtures.

Figure 10.4 presents test results of un-aged Cantabro durability specimens compacted to N_{des}. The 50% RAP mixtures are within the range of *ML* observed for control mixtures. The 75% RAP mixtures have somewhat higher *ML* than the control mixtures. Testing of 100% RAP was unable to predict 50 and 75% RAP mixture performance; likely due to the large differences in virgin asphalt contents between mixtures with each RAP source.

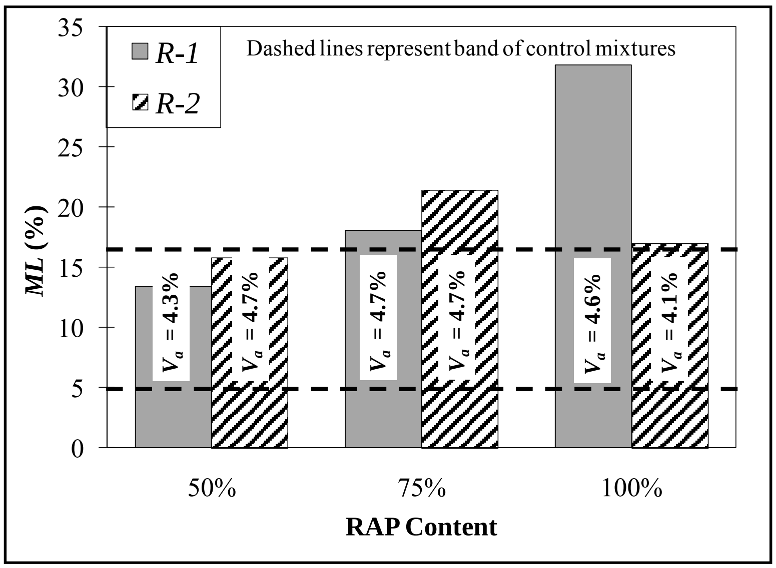


Figure 10.4 Effect of RAP Source on Cantabro Mass Loss for Highway Base Mixtures

10.3.4 All Mixtures

More information is needed to make informed decisions about suitability of high RAP-WMA mixtures for airfields in terms of durability. For highway surface applications, data indicates that mixtures containing up to 25% RAP would have initial Cantabro mass loss comparable to existing current practice mixtures but that additional investigation is needed into effects of aging. For highway base applications, data indicates that mixtures containing 50% RAP would have initial Cantabro mass loss comparable to existing current practice mixtures.

All the test results indicated that increasing the RAP content will tend to increase mass loss. This may translate to decreased durability of the mixture in practice, but this is very difficult to quantify with the data available. Further research is needed to determine the relationship between laboratory performance in the Cantabro test and performance of field mixtures with respect to durability. Results of test sections containing 45% RAP at the NCAT test track led West et al. (2009) to observe that resistance to raveling was likely to be very good for high RAP mixes.

10.4 Crack Resistance

Resistance to cracking is an important quality for all asphalt pavement applications. For surface mixtures, low temperatures are a frequent cause of thermal cracking. For base mixtures, repeated loading is a frequent cause of fatigue cracking.

10.4.1 Airfield Surface Mixtures

The potential for thermal cracking in airfield surface mixtures was evaluated with both binder testing and with *BBR* mixture beam testing. Results of the binder testing indicated that high RAP mixtures did not affect the low temperature properties as much as the high temperature properties. In fact the low temperature grade was only raised by approximately 3 degrees when going from 0% RAP to 25% RAP. While this increase is not good, it is much less than would be expected. When using 50% RAP the low temperature grade was increased approximately 8 degrees. Again, this is considerable but much less than expected based on its effect on the high temperature properties. The final low temperature grading for the asphalt at 50% RAP is almost a -20 which is very close to the -22 that the new binder is classified. The lower mixing temperature is thought to minimize any change in this low temperature grade.

Results of the *BBR* mixture beam testing indicated that use of warm mix was not a statistically significant factor on mixture stiffness. Figure 10.5 presents the results organized by RAP content and virgin aggregate type and test temperature. In some cases at the -12 C test temperature (Figure 10.5*a*) there were differences between virgin aggregate types but the differences were not statistically significant. RAP content was determined to be a statistically significant factor for both test temperatures. For the -06 C test temperature (Figure 10.5*b*), the increase in mixture stiffness when RAP content increases from 25 to 50% was not as large as when RAP content increased from 0 to 25%. For the -12 C test temperature, the increase in mixture stiffness was relatively consistent for both increases in RAP content. Further testing is needed to make an informed decision about suitability of high RAP contents for airfield surface mixtures with respect to thermal cracking.

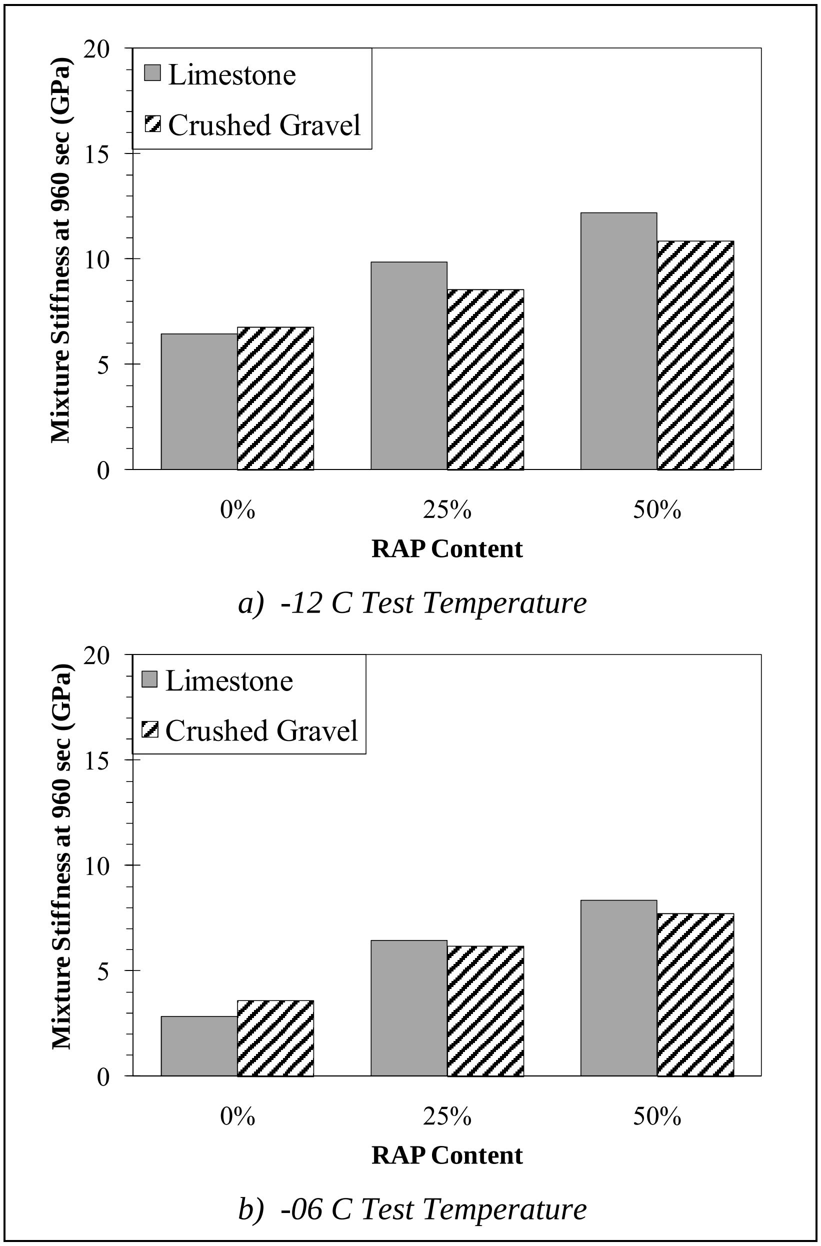


Figure 10.5 Mixture Stiffness for Airfield Surface Mixtures

10.4.2 Highway Surface Mixtures

BBR mixture testing was performed at -24 C and -18 C to bracket the low temperature performance grade of the virgin binders used in this study. Additional testing was performed at -12 C and -06 C to provide a measurement of mixture stiffness at temperatures that can occur in the field slightly above the low temperature performance grade of the virgin binder. Results of control mixture testing established that control mixtures two and three represented the lowest and highest stiffnesses for plant produced control mixtures at all loading times and test temperatures.

Figures 10.6 and 10.7 present the effects of RAP source on mixture stiffness at 960 seconds for -24 C and -18 C test temperatures. Stiffness of all the 25 and 50% RAP mixtures was within the range of stiffness observed for plant produced control mixtures. Designed 100% RAP mixtures also fell within the range of control mixtures. Figures 10.8 and 10.9 present the effects of RAP source on mixture stiffness at 960 seconds for -12 C and -06 C test temperatures. Stiffness of all the 25 and 50% RAP mixtures was higher than the range of stiffness observed for plant produced control mixtures. Designed 100% RAP mixtures also fell above the range of control mixtures. This increased stiffness may suggest that the mixes are more susceptible to cracking. Further testing is needed to make definitive statements as to why the mixtures perform within the control bands at temperatures bracketing the low temperature grade of virgin binder used in Mississippi, but are above the control bands at higher temperatures.

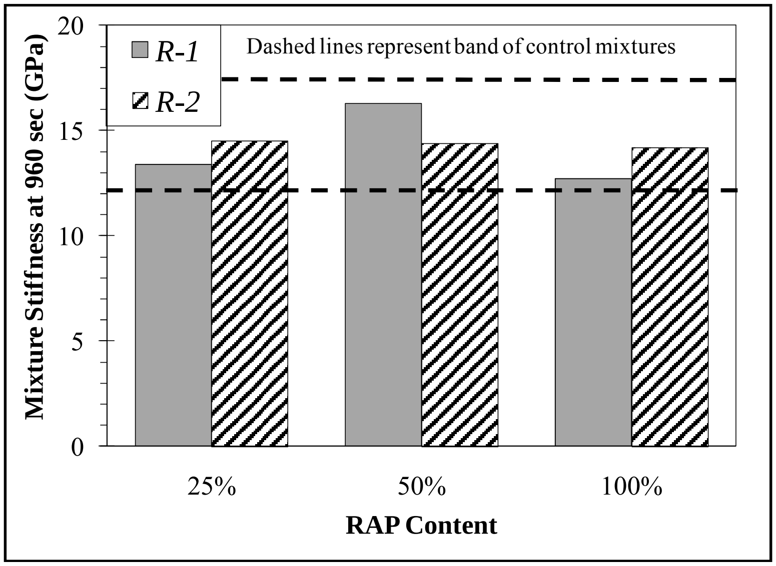


Figure 10.6 Effect of RAP Source on Mixture Stiffness at -24 C

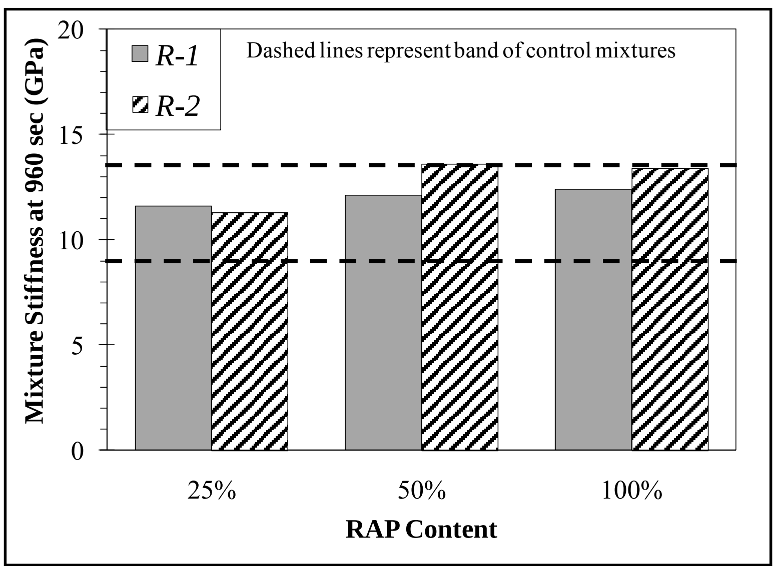


Figure 10.7 Effect of RAP Source on Mixture Stiffness at -18 C

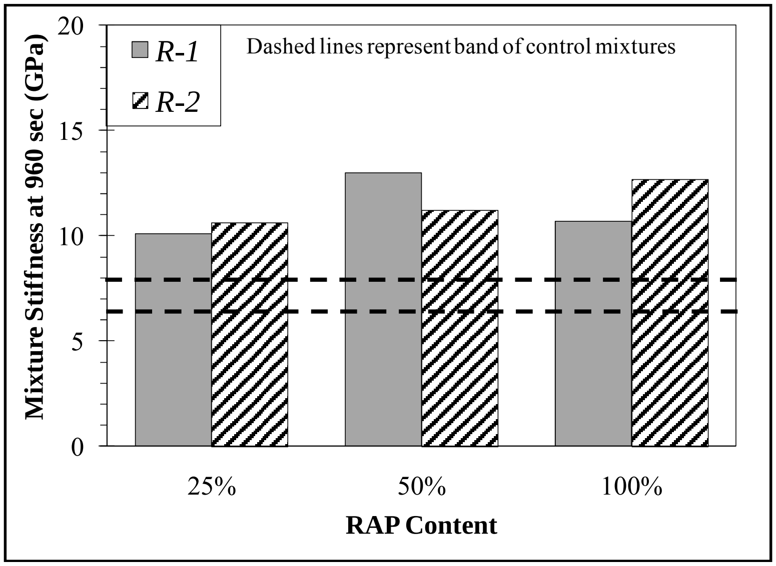


Figure 10.8 Effect of RAP Source on Mixture Stiffness at -12 C

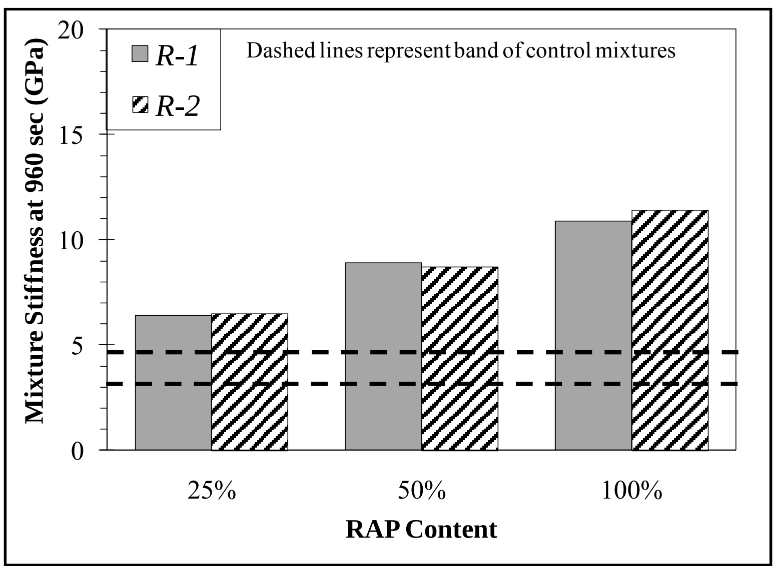


Figure 10.9 Effect of RAP Source on Mixture Stiffness at -06 C

A 50 gyration control mixture produced the lowest stiffness, while an 85 gyration control mixture produced the highest stiffness. At -24 C and -18 C, the 25 and 50% RAP mixes performed worse than the 50 gyration mixture (most crack resistant in conventional practice) but better than the 85 gyration mixture (least crack resistant in conventional practice) in terms of stiffness. This provides some evidence that a 65 gyration designed mixture with 25 to 50% RAP can perform in a comparable manner in the context of cracking relative to a control mixture. At -12 C and -06 C, this behavior was not observed.

Testing of 100% RAP indicated mixtures containing *R-2* RAP source would likely have higher stiffness than mixtures containing *R-1* RAP. This result is counterintuitive given the higher low temperature PG grade for the *R-1* asphalt (+1.7) compared to the *R-2* asphalt (-3.5). Of the eight cases where *R-1* and *R-2* RAP sources were tested for the same conditions (two RAP levels and four test temperatures) only three followed the prediction. In another three of the eight cases the stiffness of *R-1* and *R-2* mixtures was about the same. For the last two cases the observed results were reverse of the prediction. Testing of 100% RAP only correctly predicted the relative ranking of mixture stiffness for the *R-1* and *R-2* RAP sources in 25 and 50% RAP mixture in three of eight cases. Low temperature binder grades of the *R-1* and *R-2* RAP sources only correctly predicted the relative ranking of mixture stiffness in 25 and 50% RAP mixture in two of eight cases.

Data from *BBR* mixture testing was utilized to estimate the critical cracking temperature (T_{cr}) for the 25 and 50% RAP mixtures and the relative highest and lowest stiffness control mixtures. Estimation of the temperature where thermal stress and tensile strength intersect yielded the most reasonable estimates of T_{cr}; Figure 10.10 summarizes the estimated T_{cr} temperatures. Mixtures with 25% RAP performed similarly to control mixtures. Mixtures with 50% RAP had higher T_{cr} temperatures (i.e. higher likelihood of cracking) than control mixtures and 25% RAP mixtures.

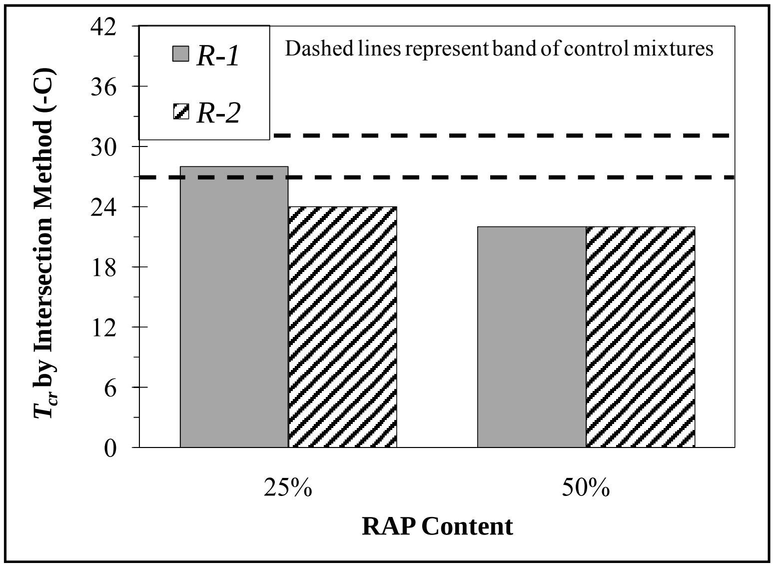


Figure 10.10 Effect of RAP Source on T_{cr} Estimates for Highway Surface Mixtures

Overall, mixtures with 25% RAP did not exhibit characteristics to prohibit recommendation of use on the surface. Mixtures with 50% RAP exhibited potentially problematic behavior in thermal cracking analysis and at *BBR* test temperatures somewhat above the low temperature binder grade in Mississippi. Results indicated that *BBR* stiffness of mixtures with high RAP at temperatures near the low temperature performance grade of virgin binder (-22 C) is within the range of stiffness results observed for current practice mixtures. Results of testing at temperatures slightly above the low temperature performance grade indicated that high RAP mixtures may be somewhat stiffer than conventional practice mixtures.

10.4.3 Highway Base Mixtures

Highway base mixtures were tested for indirect tensile strength at 25 C to provide an assessment of mixture brittleness and fatigue crack potential. However, it must be noted that increased mixture tensile strength alone is not sufficient to indicate higher potential for fatigue cracking. Several sources cited in literature review did not observe any higher incidence for fatigue cracking in high RAP mixtures than in conventional low RAP mixtures. Figure 10.11 presents the tensile strength data for base mixtures. 50% RAP mixtures had similar or slightly higher tensile strength than control mixtures. 75% RAP mixtures had higher tensile strength than controls and only slightly lower than 100% RAP mixtures. Higher tensile strengths of mixtures with *R-2* RAP than those with *R-1* RAP are thought to likely due to higher virgin asphalt content in mixtures with *R-1* RAP even though *R-2* RAP asphalt is less stiff than *R-1* RAP.

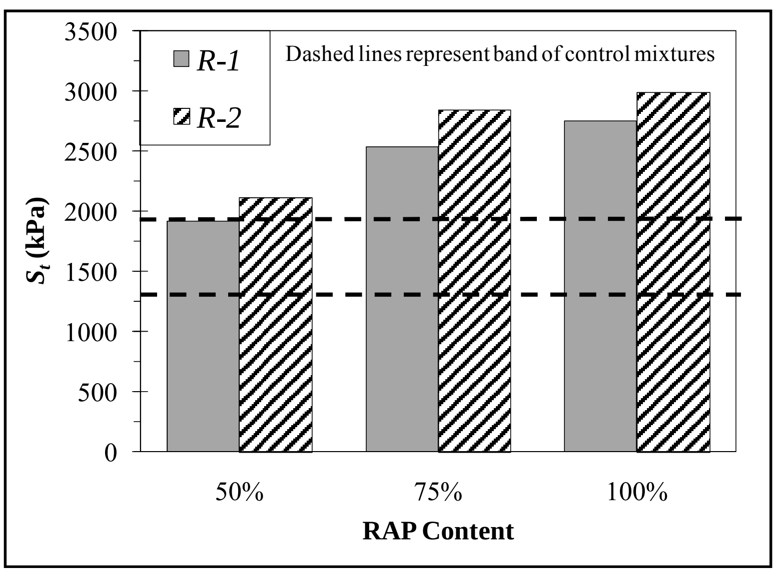


Figure 10.11 Effect of RAP Source on Tensile Strength for Highway Base Mixtures

10.4.4 All Mixtures

Determining the relative potential of a given asphalt mixture to develop cracks is a difficult task given the many factors of mixture properties, environmental distresses and pavement structure that can contribute to cracking performance. For airfield surface mixtures the data indicates a somewhat increased potential for cracking; however the increase in cracking potential is not a great as might be expected with high RAP contents. For highway surface mixtures the cracking potential of 25% RAP mixtures is quite comparable to that of conventional practice low RAP mixtures; 50% RAP mixtures may have a moderately increased cracking potential relative to current practice mixtures. For highway base mixtures, the data indicates that 50% RAP mixtures have similar tensile strengths to current practice mixtures and 75% RAP mixtures have higher tensile strengths than control mixtures.

10.5 Rut Resistance

Rut resistance is an important quality for mixtures in all pavement layers. The temperatures and contact pressures experienced by mixtures on the pavement surface will generally be higher than those of deeper pavement layers. Binder testing and *APA* testing were used to evaluate rut resistance for airfield surface mixtures. *APA* testing and the PURWheel dry protocol testing were utilized to evaluate mixture rut resistance for highway mixtures.

10.5.1 Airfield Surface Mixtures

For airfield mixtures, the data clearly showed that adding RAP reduced the amount of rutting. The primary reason for this is the stiffer asphalt in high RAP content mixtures. Rutting is reduced significantly with higher RAP content due to its effect on the grade of asphalt. Adding 25% RAP appeared to result in approximately an 8 degree increase in the high PG temperature. Adding 50% RAP increased the high temperature grade of the asphalt by approximately 20 degrees. This increase in high temperature grade is useful to resist rutting but should not adversely affect other properties. *APA* rut results also showed that increasing RAP resulted in less rutting. The amount of rutting in the field is anticipated to be low in these high RAP-WMA mixtures.

10.5.2 Highway Surface Mixtures

Two test methods were used to evaluate rut resistance: 1) *APA*; and 2) PURWheel dry protocol. *APA* testing was selected as a conventional test method; specimens were tested at nominal air void levels of 7 and 10%. PURWheel dry protocol testing was selected as a more simulative wheel tracking test method to evaluate rutting as well as being a complement to PURWheel wet protocol testing.

The 50 gyration control mixture exhibited the highest *APA* total rut depths (≈12 mm), and the 85 gyration mixture exhibited the lowest total rut depths for both nominal air void levels. Control mixtures with polymer-modified binder had total rut depths on the order of 2 to 3.5 mm for nominal 7% air voids and on the order of 6 mm for nominal 10% air voids. PURWheel dry protocol results confirmed that the 50 gyration control mixture performed poorly and that mixtures containing polymer-modified binder performed well. PURWheel testing indicated a difference in rutting between the field and laboratory mixed version of the 85 gyration control mixture with PG 67-22 binder that was not observed in *APA* results.

Figure 10.12 presents results of *APA* testing in terms of RAP source. The range of results from testing control mixtures with PG 67-22 and PG 76-22 binder grades are represented with horizontal lines in Figure 10.12. For specimens of 25 and 50% RAP mixtures with 7% nominal air voids (Figure 10.12*a*) the total rut depths are less than the lowest measured for PG 67-22 control mixtures and are comparable to results for polymer modified control mixtures. For specimens of 25 and 50% RAP mixtures with 10% nominal air voids (Figure 10.12*b*) the total rut depths are also comparable with the best performing control mixtures.

Figure 10.13 presents of PURWheel dry protocol testing in terms of RAP source. Mixtures containing 25% RAP had a level of rutting well within and generally at the lower end of the range observed for control mixtures. Mixtures containing 50% RAP had total rut depths either at the lower end of or less than the range observed for control mixtures.

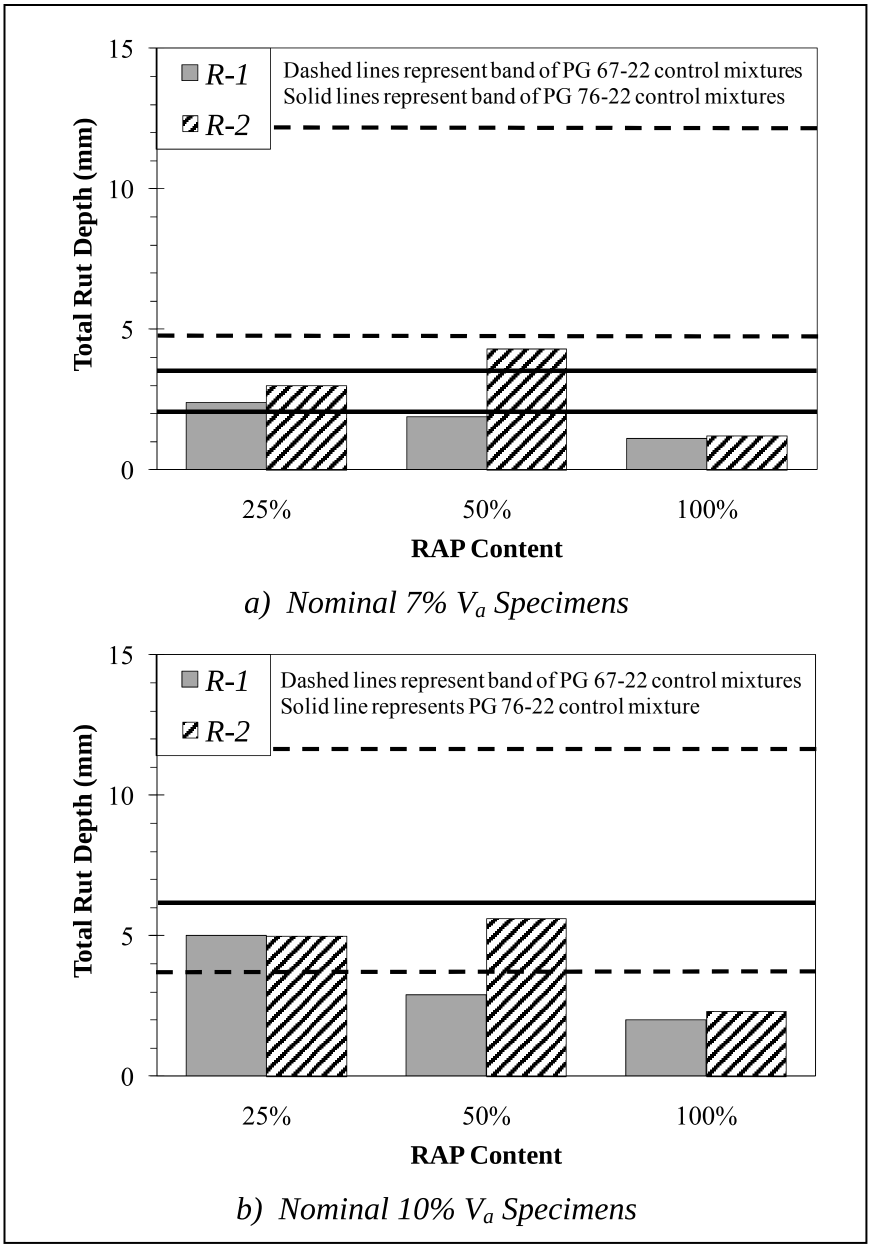


Figure 10.12 Effect of RAP Source on *APA* Rutting of Surface Mixtures

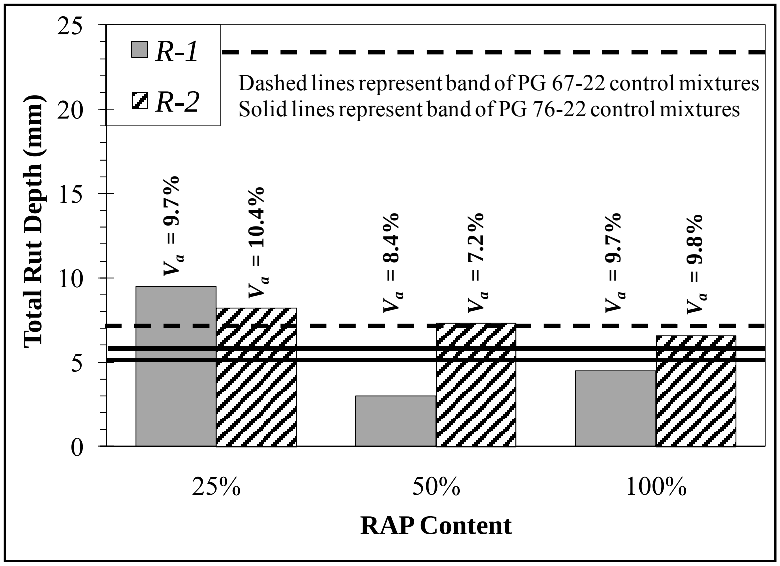


Figure 10.13 Effect of RAP Source on PURWheel Dry Rutting of Surface Mixtures

Overall, results indicated that 65 gyration mixtures with high RAP are likely to be highly rut resistant, although at higher RAP contents not all RAP sources will give equivalent performance for a particular percentage of total mixture. At RAP contents on the order of 50% the data shows some sources of RAP can give performance comparable to that of 85 gyration mixtures with polymer-modified PG 76-22 binder. However, it must be stressed that not every source of RAP is necessarily capable of that level of performance. In general, properly designed high RAP mixtures should be feasible in terms of rut resistance for high traffic (85 gyration) applications. The ability of a 65 gyration PG 67-22 design to be comparable to a 85 gyration PG 76-22 design in terms of rut resistance is significant as it allows a balance of rut and crack resistance.

10.5.3 Highway Base Mixtures

The same test methods used to evaluate rutting for highway surface mixtures were used to test rut resistance of highway base mixtures. Figure 10.14 presents results of PURWheel dry protocol testing for highway base mixtures. Rutting of the 50 and 75% RAP mixtures was within the range of control mixture performance or better than control mixtures. The differences are small but *R-1* mixtures rutted more than *R-2* mixtures; this is likely due to the greater virgin asphalt content in *R-1* mixtures.

Results of *APA* testing are presented in Figure 10.15. For nominal 7% air voids specimens, the 50% RAP mixtures had similar or lower total rut depths to control mixtures; 75 % RAP mixtures had lower rutting than the controls. For nominal 10% air voids specimens, both the 50 and 75% RAP specimens had lower rutting than the control mixtures. Overall, dry rutting is unlikely to be a major concern for high RAP base mixtures.

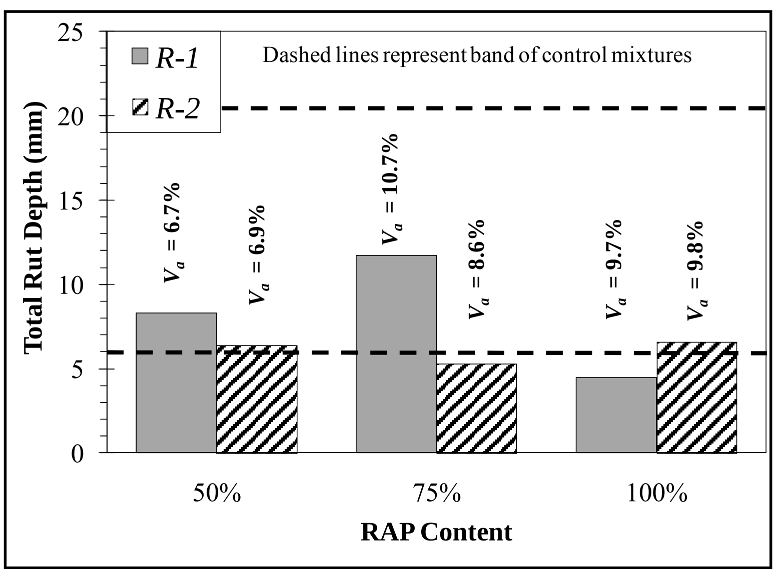


Figure 10.14 Effect of RAP Source on PURWheel Dry Rutting of Base Mixtures

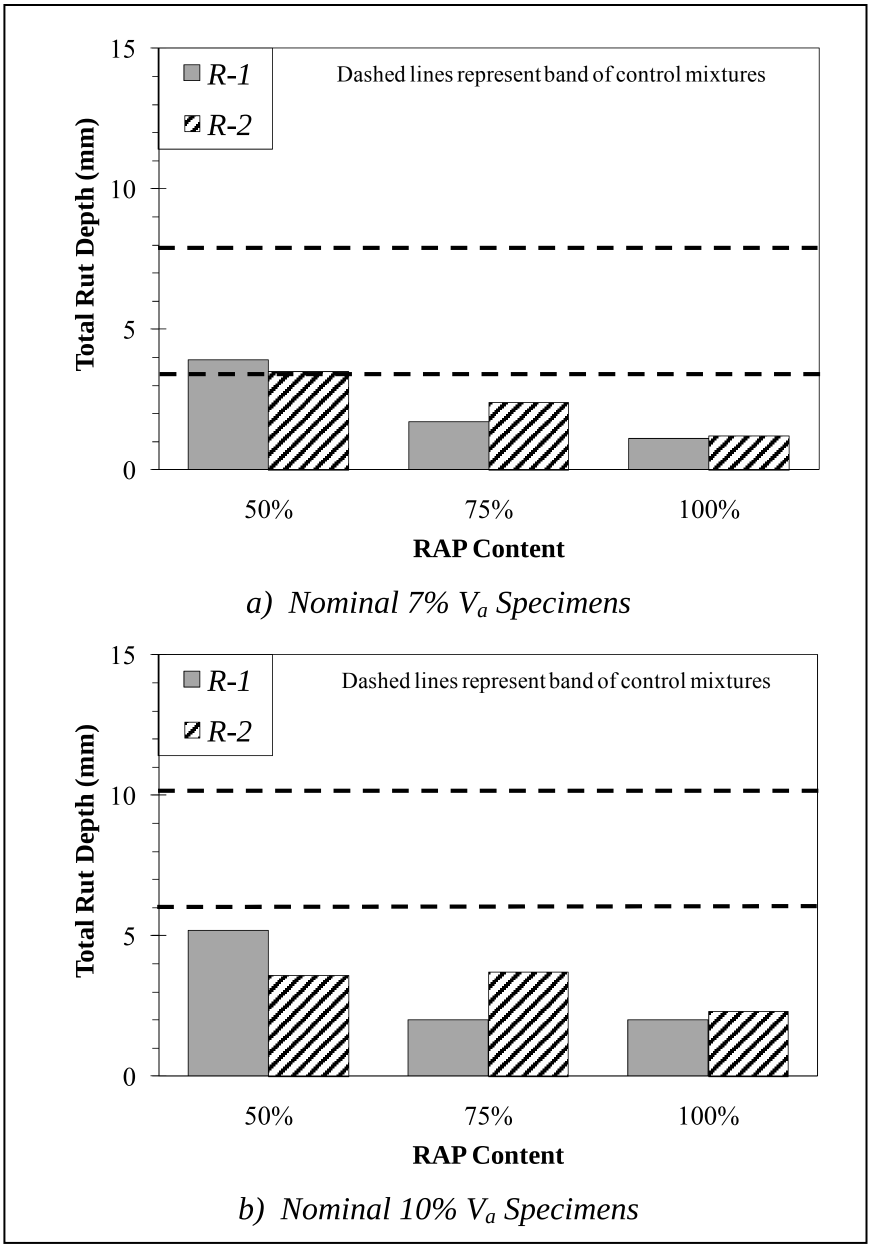


Figure 10.15 Effect of RAP Source on *APA* Rutting of Base Mixtures

10.5.4 All Mixtures

All results appear to indicate that high RAP-WMA is likely to have good rut resistance in service when utilized for either airfield or highway applications in surface and base pavement layers. Designing high RAP mixes with 65 gyrations compactive effort appeared to effectively balance rut and crack resistance for highway mixtures.

10.6 Moisture Damage Susceptibility

The conditions which can lead to moisture damage in asphalt mixtures susceptible to moisture can occur in any layer of a pavement. As a consequence, assessment of the susceptibility of asphalt mixtures to moisture damage is an important consideration. Moisture susceptibility must be carefully evaluated for high RAP-WMA due to the unknown potential for moisture damage from the interaction of lower warm mix production temperatures and high RAP contents.

10.6.1 Airfield Surface Mixtures

TSR testing was used to evaluate moisture resistance for airfield surface mixtures. Generally, increasing RAP content increased moisture damage resistance of airfield mixes. Inclusion of 25% RAP improved moisture resistance compared to 0% RAP in six of eight cases. Inclusion of 50% RAP either improved or did not noticeably decrease moisture resistance compared to 0% RAP in all eight cases. While WMA mixes often tend to have lower resistance to moisture than HMA mixes, the addition of RAP may be a reasonable solution to this problem. The asphalt coating the RAP is very hard and tightly bonded to the aggregate which makes it very difficult to strip when the RAP is used in a recycled mixture. Using RAP and WMA has the potential to alleviate stripping problems.

10.6.2 Highway Surface Mixtures

Two test methods were selected to evaluate moisture susceptibility of asphalt mixtures for highway surface mixtures: 1) *TSR*; and 2) PURWheel wet protocol test. Results of *TSR* moisture susceptibility testing shown in Figure 10.16*a* indicated that all control, 25% RAP, and 50% RAP mixtures should provide acceptable performance (i.e. *TSR* greater than 80%) with regards to potential for moisture damage. Results of PURWheel wet protocol testing shown in Figure 10.16*b* indicated that 25 and 50% warm mixed RAP mixtures generally did not perform quite as well as plant mixed HMA controls. For control mixture wet testing, 50% of the specimens exhibited evidence of moisture damage. For 25 and 50% RAP mixture wet testing, on the order of 80% of the specimens exhibited evidence of moisture damage (7 of 8 for 25% RAP and 6 of 8 for 50% RAP). The data provided does not allow definitive statements as to whether high RAP content, warm mix temperatures, or other factors made the mixes in this study perform worse in the PURWheel wet protocol test than the control mixtures.

The data presented in this study indicates that *TSR* testing and PURWheel wet protocol testing did not provide the same relative results for 25 and 50% RAP mixes in terms of potential for moisture damage. Performance of the *R-1* RAP source in PURWheel wet testing did not vary much regardless of its proportion in the mixture being tested. Performance of mixtures containing *R-2* RAP improved as the amount of RAP was increased; this result coincides with the results observed for airfield surface mixtures albeit for different test methods.

It is not known if the 64 C submerged specimen high pressure loaded wheel PURWheel wet protocol test is overly aggressive in relation to potential conditions experienced by Mississippi mixtures in the field. However, any mixture that can reliably survive the PURWheel wet protocol test (Howard et al. 2010) without exhibiting evidence of moisture damage is thought likely to give good performance in the field. The lower limit of PURWheel wet test results that correlates to acceptable performance in the field has not yet been established.

TSR results for 100% RAP indicated some potential for moisture sensitivity might exist with the recycled mixtures containing *R-2* RAP but no such problems were observed in the *TSR* test results for 25 and 50% RAP mixtures. Testing of designed 100% RAP mixtures in the PURWheel wet test indicated that *R-2* RAP would likely give better moisture damage resistance in recycled mixtures. However, PURWheel wet testing of 25 and 50% RAP mixtures indicated that the *R-1* RAP source provided better moisture damage resistance than the *R-2* RAP source. Overall, results of PURWheel testing indicated that high RAP-WMA mixtures are somewhat more prone to moisture damage than the current practice HMA mixtures tested.

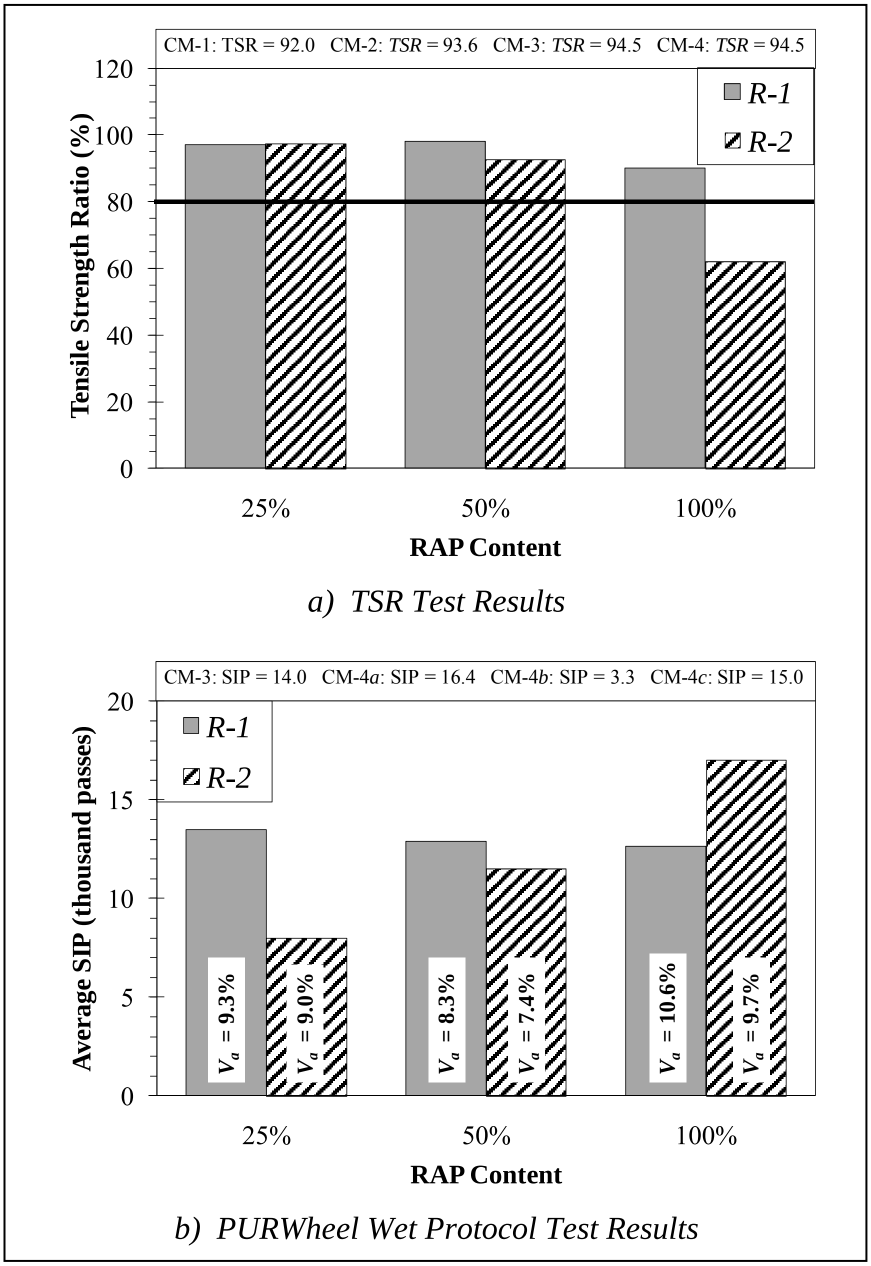


Figure 10.16 Effect of RAP Source on Moisture Susceptibility of Surface Mixtures

10.6.3 Highway Base Mixtures

The *TSR* test and the PURWheel wet protocol test were utilized to evaluate moisture resistance for highway base mixtures. Results of *TSR* testing are provided in

Figure 10.17*a*. The results indicate that all the 50 and 75% RAP mixtures tested pass the commonly accepted pass/fail criteria of 80% *TSR*.

Results of the PURWheel wet protocol testing are provided in Figure 10.17*b*. Average SIPs for the 50% RAP mixtures were slightly less than those for control mixtures two to four. Mixtures with 75% RAP outperformed control mixtures somewhat with respect to average SIPs. Mixtures containing *R-2* RAP performed slightly better than those with *R-1* RAP source for both 50 and 75% RAP mixtures; this was also observed in the 100% RAP testing.

A crucial issue with respect to moisture susceptibility of high RAP mixtures that is not fully evident from the test data is the difficulty in achieving adequate coating of coarse virgin aggregate particles. This is especially difficult for 75% RAP mixtures, where the virgin aggregate component is composed mostly of coarse aggregate. Figure 10.18 illustrates this for compacted slab specimens ready for PURWheel testing; mixtures with *R-2* RAP source are shown but similar results were observed for *R-1* RAP mixtures. Inadequately coated coarse aggregate particles are readily visually apparent in the compacted 75% RAP slab. Mixtures unable to achieve adequate aggregate coating in the closely controlled environment of a laboratory mixing process may also be difficult to coat in the more variable environment of plant mixing process. The 75% RAP mixtures held up relatively well in *TSR* and PURWheel wet testing due to the very high binder stiffness of the mixtures contributed from the RAP; however uncoated aggregate may lead to moisture damage problems in service.

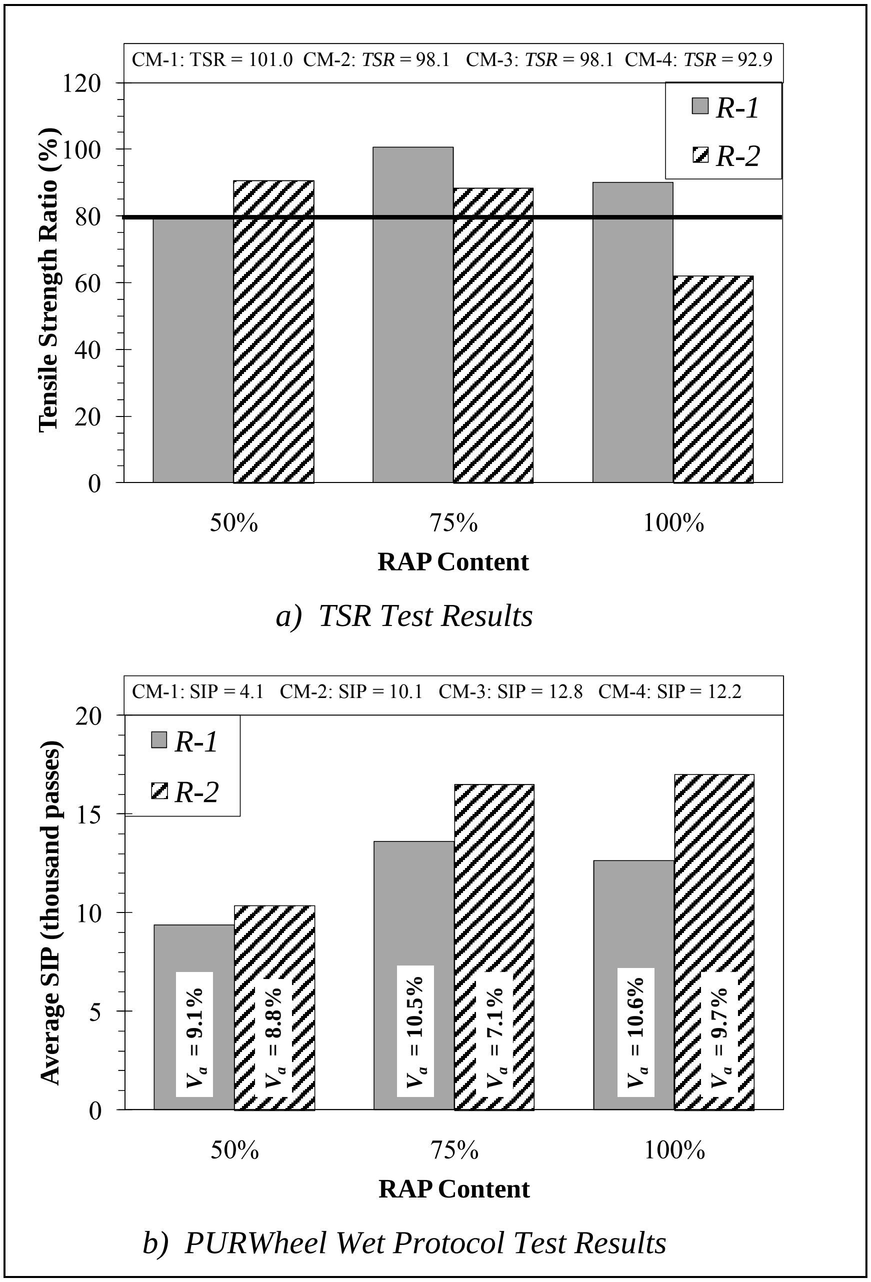


Figure 10.17 Effect of RAP Source on Moisture Susceptibility of Base Mixtures

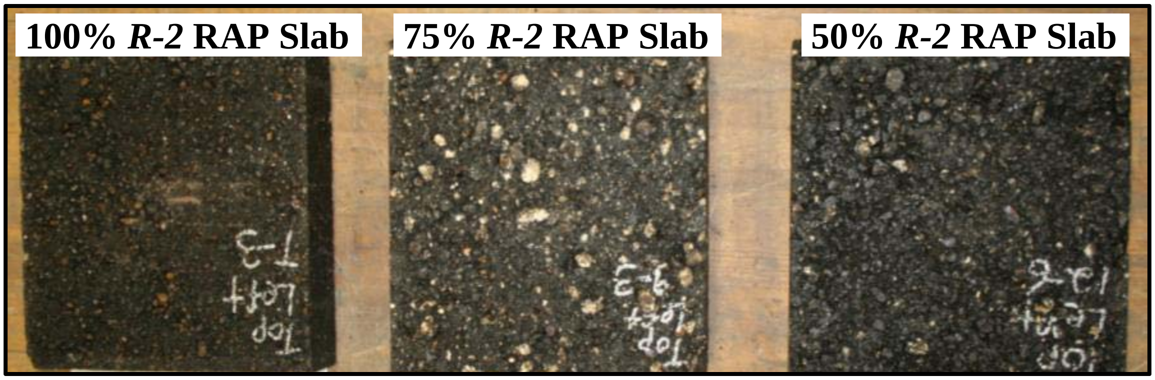


Figure 10.18 Laboratory Mixing Efficiency for High RAP Contents (*R-2* RAP Shown)

10.6.4 All Mixtures

TSR testing for airfield mixtures indicated that moisture damage would likely not be a problem with high RAP-WMA; however additional moisture damage wheel tracking testing should be performed to verify this result. Results for highway surface and base mixtures indicated slightly increased moisture damage susceptibility for mixtures with up to 50% RAP compared to current practice mixtures. Mixtures with 75% RAP are not recommended for use due to inadequate coating of coarse aggregate.

10.7 Compactability Analysis

10.7.1 Airfield Surface Mixtures

Statistical analysis of airfield mixture results indicated that warm mix was not a significant factor but that RAP content and virgin aggregate type were. Results for airfield mixtures are plotted in Figure 10.19. Consistent trends are observed for all six parameters that limestone mixtures were tougher to compact than gravel mixtures.

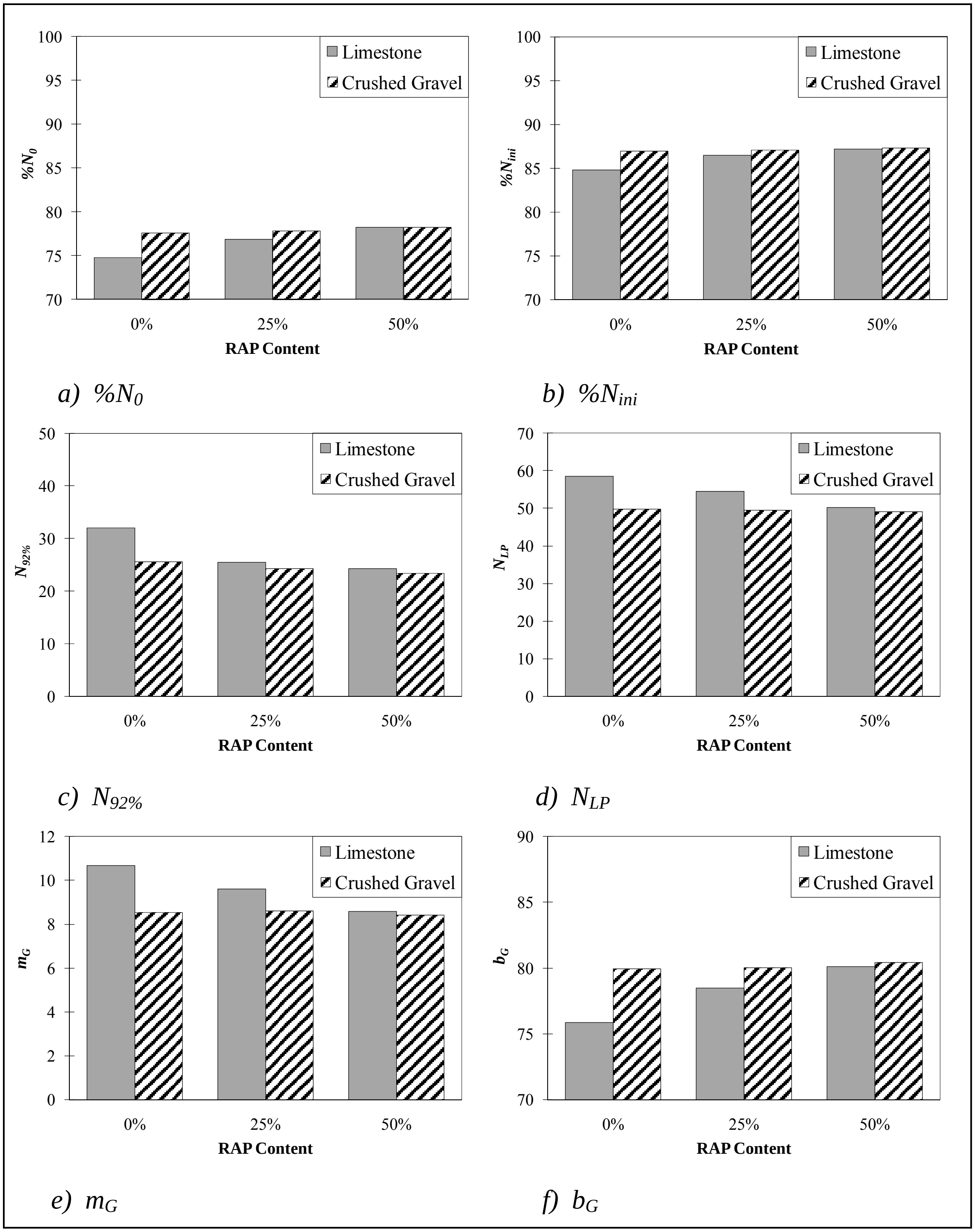


Figure 10.19 Compactability Results for Airfield Surface Mixtures

All levels of RAP content for crushed gravel virgin aggregate had similar levels of compactability while the addition of RAP to limestone aggregate mixtures tended to improve compactability. This may be because limestone aggregate is more difficult to compact than gravel aggregate (Leiva and West 2008a) and that the RAP was primarily composed of gravel aggregate. Values for $\%N_{ini}$, $N_{92\%}$, N_{LP} and m_G for airfield mixtures were all within the ranges of values reported by Leiva and West (2008a).

10.7.2 Highway Surface Mixtures

Compactability data for highway surface mixtures is summarized in Figure 10.20. Dashed lines represent the highest and lowest values observed for control mixtures. 25% RAP mixtures fall within the range of control mixtures in most cases. In general, 25 and 50% RAP mixtures appear to have similar laboratory compactability to current practice mixtures. Little difference is observed between RAP sources when used at 25 and 50%, although some differences are observed for 100% RAP with *R-1* RAP source being somewhat tougher to compact than *R-2*.

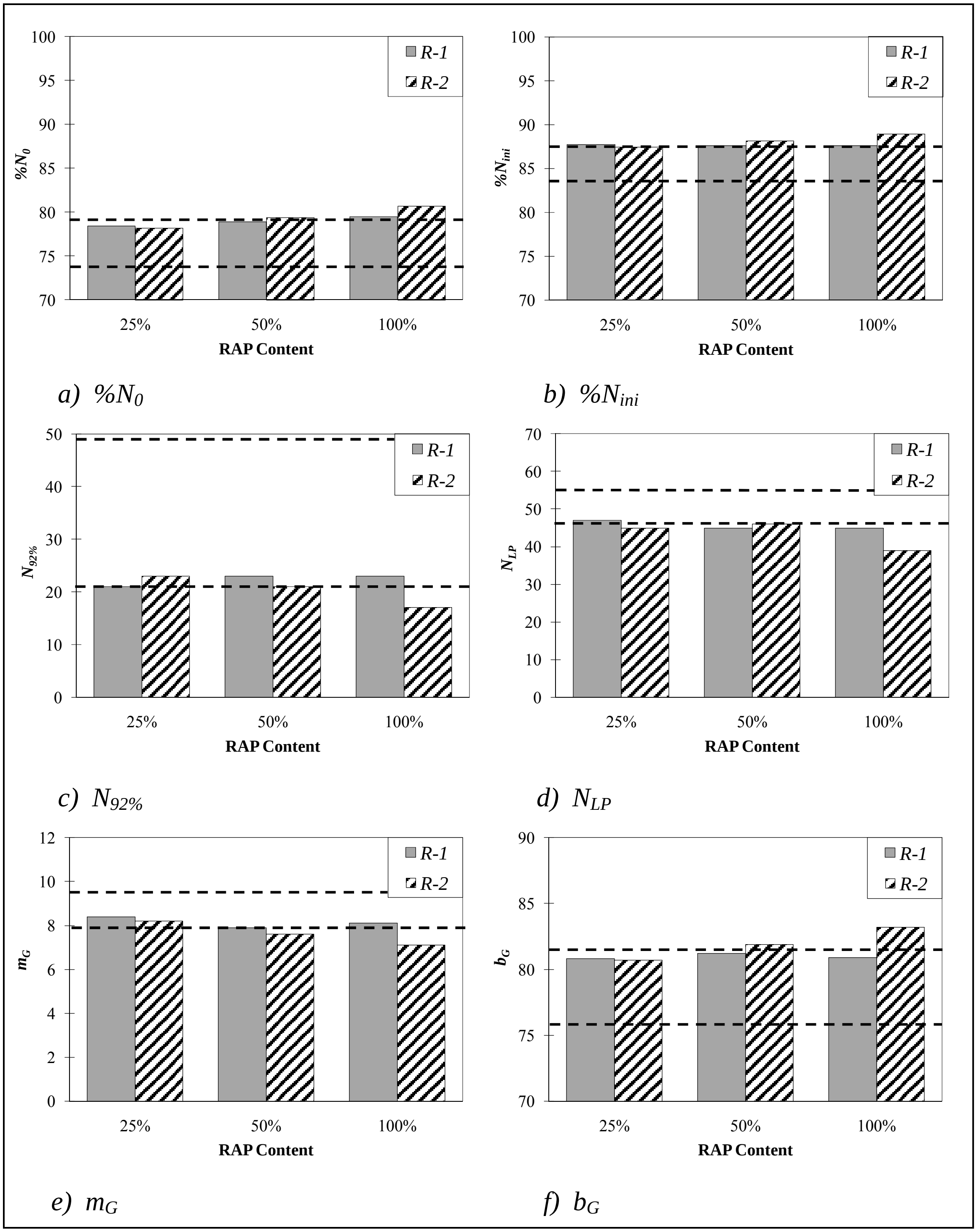


Figure 10.20 Compactability Results for Highway Surface Mixtures

10.7.3 Highway Base Mixtures

Compactability data for highway base mixtures is summarized in Figure 10.21. Dashed lines represent band of control mixture results. Compactability of 50% RAP falls within the range of control mixtures in nearly all cases. 75% RAP mixtures also have reasonable levels of laboratory compactability. Parameters of $\%N_0$, $\%N_{ini}$ and N_{LP} predict similar compactability of both RAP sources at the 50% level, while m_G and b_G predict *R-2* RAP source is tougher to compact and $N_{92\%}$ predicts that *R-1* RAP source is tougher to compact. These results do not align well with results of 100% RAP testing where all six parameters predicted that *R-1* RAP source was more difficult to compact.

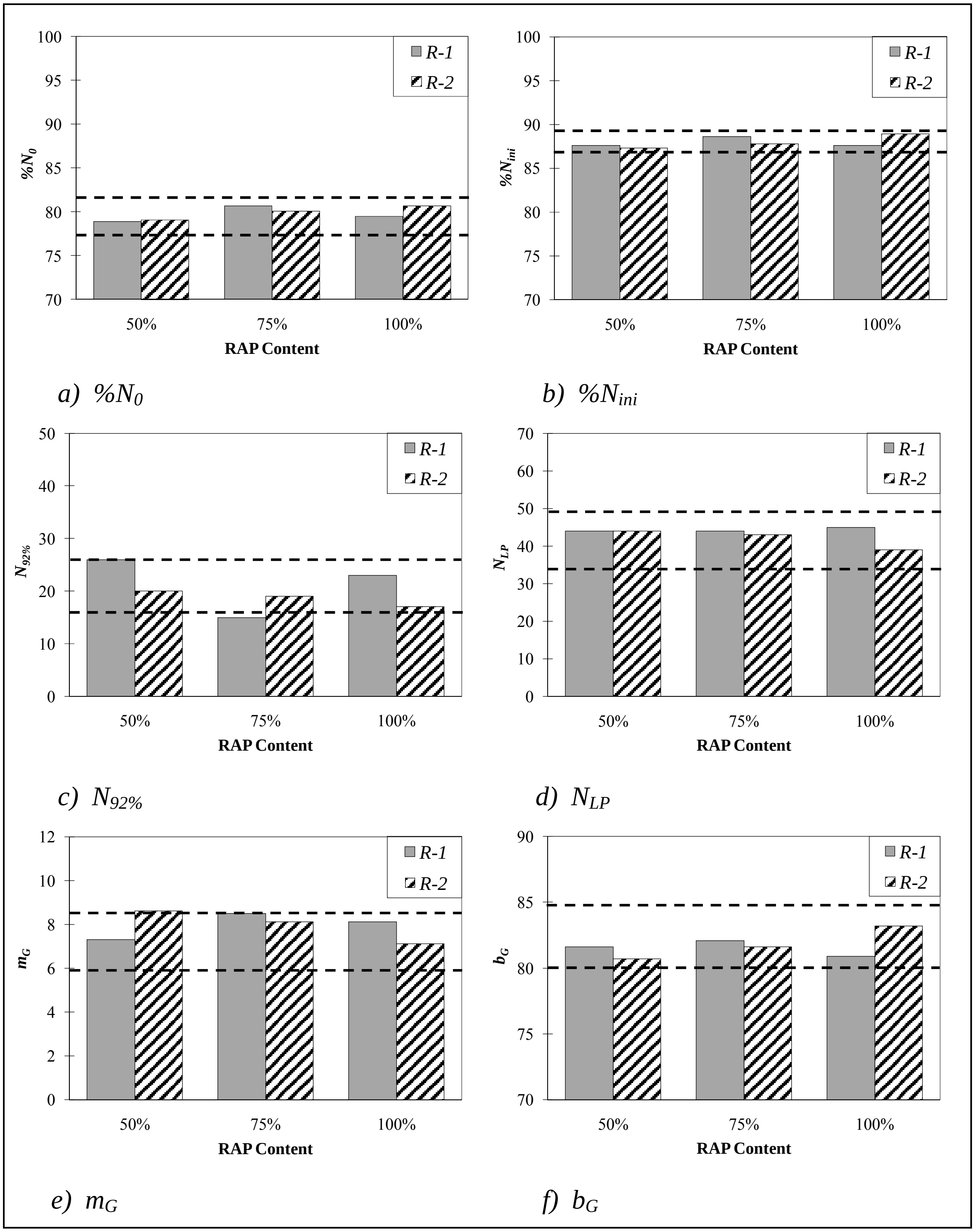


Figure 10.21 Compactability Results for Highway Base Mixtures

10.7.4 All Mixtures

Overall, results of the laboratory compactability analysis for high RAP-WMA indicate that high RAP mixtures are generally comparable to current practice low RAP mixtures. This is encouraging in terms of ease of construction for these mixtures; however analysis of laboratory compactability has been unable to provide accurate prediction of field compactability for other researchers (Leiva and West 2008b). All six compactability parameters evaluated were generally strongly correlated and yielded the same trends in terms of predicted compactability for high RAP mixtures.

CHAPTER 11

CONCLUSIONS AND RECOMMENDATIONS

11.1 Summary

This dissertation focused on four major areas: 1) characterization of RAP properties; 2) high RAP-WMA for airfield surface mixtures; 3) high RAP-WMA for highway surface mixtures; and 4) high RAP-WMA for highway base mixtures. To characterize RAP properties a unique approach was taken that coupled a dataset of properties for 568 asphalt mix designs spanning five years of practice and testing of 100% RAP with added virgin binder; 394 compacted specimens and 68 loose specimens of 100% RAP were tested. A method to predict RAP absorbed asphalt was developed requiring two inputs: 1) total asphalt content; and 2) RAP G_{se} when coated with virgin binder. The method was shown to yield more reasonable results than conventional methods which were shown very likely to give incorrect absorbed asphalt estimates in some conditions. The relative effectiveness of RAP surface asphalt was evaluated and estimates of inert and effective RAP asphalt were made for a variety of temperature, compactive effort, and warm mix additive conditions. Results showed different behaviors between RAP sources and between hot and warm mix temperatures. These results were also observed in volumetrics of high RAP mixtures.

Performance evaluation of airfield surface mixtures considered durability, non-load associated cracking, rutting resistance and moisture susceptibility. Crushed gravel and limestone aggregate mixtures were tested with 0 to 50% RAP in conjunction with three warm mix asphalts. Test results indicated high RAP-WMA is a potentially viable product for surface mixtures. WMA was shown capable of producing rut resistant mixtures with high RAP contents. A more intriguing finding was that while increasing rut resistance the high RAP mixtures did not affect the low temperature properties as much as the high temperature properties. Mixtures with high RAP content appear to be only slightly more susceptible to thermal cracking. Based on *TSR* testing alone, it was shown that, in general, WMA technology can be used with high RAP content to produce mixtures that are more resistant to moisture damage. Testing indicated high RAP content WMA mixes may be more susceptible to durability issues than low RAP mixes.

Performance evaluation of highway surface mixtures considered durability, non-load associated cracking, rutting resistance and moisture susceptibility. Testing indicated that high RAP mixtures containing 25% RAP would likely have comparable durability performance and thermal crack resistance to current practice low RAP content highway surface mixtures; 50% RAP mixes may have slightly higher potential for durability and thermal cracking problems. Rut resistance of high RAP mixtures was found to be similar to or better than current practice mixtures with PG 67-22; in some cases high RAP mixtures had performance similar to current practice PG 76-22 mixtures. PURWheel wet testing indicated a slightly higher potential for moisture damage than current mixtures; *TSR* testing did not.

Performance evaluation of highway base mixtures considered durability, crack resistance, rutting resistance and moisture susceptibility. Testing indicated high RAP mixtures containing 50% RAP would likely have comparable durability performance to current practice low RAP content highway surface mixtures; 75% RAP mixtures may have slightly higher potential for durability problems. Tensile strength of 50% RAP mixes was comparable to current practice mixtures, but tensile strength of 75% RAP mixtures was higher than current practice. High RAP mixtures had similar or better rut resistance than current practice. PURWheel wet test data for 50% RAP mixtures indicated a slightly higher potential for moisture damage than current mixtures. Mixtures with 75% RAP had poor laboratory mixing efficiency as many un-coated coarse aggregate particles were seen; this may lead to moisture damage.

11.2 Conclusions

Based on laboratory testing, the overall conclusions of this research are that: 1) RAP can be characterized to better understand its absorbed, inert and effective bituminous components; 2) use of moderate to high RAP contents (up to 25%) with warm mix technologies in airfield surface mixtures is likely feasible; 3) use of 25% RAP contents with warm mix technologies in highway surface mixtures is feasible and recommended for immediate implementation in Mississippi but 50% RAP requires further investigation; and 4) use of 50% RAP contents in highway base mixtures is to be feasible from a performance standpoint but use of 75% RAP is not recommended. Specific conclusions are presented in the following list.

- RAP aggregate does not absorb additional asphalt. Measurement of RAP G_{se} can be effectively performed using RAP coated with 2% virgin binder.
- Determination of RAP absorbed asphalt by extracted aggregate G_{sb} with current techniques was shown to give unreasonable results. More reasonable estimates for

RAP sources from a single pavement can be made using measured RAP G_{se} and the regression equation developed in this dissertation.

- Compaction temperature can have a relatively large effect on RAP virgin asphalt demand as considerably more virgin asphalt is required at WMA temperatures than at HMA temperatures in some instances; the effect magnitude is dependent on RAP source. Varying compactive effort causes approximately the same relative change in virgin asphalt demand by RAP as it does in conventional mixtures. RAP bitumen on the surface of RAP aggregate is not all effective under some conditions.
- For airfield surface mixtures, the measured increase in high temperature grade of the binder when RAP was added to the mixes was noticeably higher than the measured increase in low temperature grade. This indicates generally improved rut resistance of high RAP content mixes but only a limited increase in thermal cracking susceptibility. WMA technology can be used with high RAP content to produce mixtures that will be resistant to rutting. The binder will be stiffer for the high RAP contents greatly reducing the potential for rutting. Mixtures with high RAP content appear to be slightly more susceptible to thermal cracking. The high RAP content stiffens the asphalt binder, slightly increasing the low temperature grade of the asphalt.
- The Cantabro test was found useful for relative performance measurement of mixture durability. Air voids were observed to have an effect on mass loss; controlling specimen air voids reduced test method variability but did not change the conclusions. Durability testing of aged specimens indicated high RAP mixes may be somewhat more prone to durability issues over time than control mixtures.

- For high RAP airfield surface mixtures the Cantabro test indicated a potential for decreased durability but further testing is needed. For highway surface mixtures, durability testing without aging indicated that 25% RAP mixes were comparable to current practice and performance of 50% RAP mixes was not dramatically worse than control mixes. Durability testing of highway base mixtures indicated that 50% RAP mixes were comparable to current practice.
- The *BBR* mixture test performed on small specimens was found useful for assessment of mixture stiffness at low temperatures. *BBR* data and indirect tensile testing were successfully used to conduct thermal cracking analysis.
- For highway surface mixtures, *BBR* mixture stiffness testing at temperatures bracketing the low PG temperature of virgin Mississippi materials (-22 C) indicated high RAP-WMA mixes performed within the range of 50 and 85 gyration control mixes. Mixture stiffness testing at temperatures slightly above the low PG temperature showed a relatively small increase in stiffness for 25% RAP mixes compared to controls and moderate increase in stiffness for 50% RAP mixes. Thermal cracking analysis indicated similar performance of 25% RAP mixes to current practice.
- PURWheel dry protocol testing at 64 C provided the same relative ranking of mixtures as did the *APA* test method although the magnitude of rutting in the PURWheel was higher.
- Rut testing in both the APA and PURWheel dry protocol indicated that high RAP-WMA mixes are highly rut resistant and were comparable to 85 gyration control mixes in most cases. For highway surface mixtures, 50% of some RAP sources can

provide rut resistance equal to 85 gyration PG 76-22 mixtures; however not all RAP sources can necessarily provide this level of performance.

- 65 design gyration high RAP mixes balanced rut and crack resistance effectively.
- *TSR* testing of high RAP mixes did not indicate any potential for moisture damage in highway mixes and also indicated good performance of high RAP airfield mixes.
- Testing of submerged specimens at 64 C in the PURWheel wet protocol test was an aggressive test method that allowed relative moisture damage performance ranking.
- PURWheel wet protocol testing indicated the potential for moisture damage in control and high RAP-WMA highway mixes, whereas *TSR* testing did not.
- Laboratory compactability analysis did not indicate problems with high RAP mixes.

11.3 Recommendations for Implementation

Until further information becomes available, it is recommended to design high RAP with warm mix technologies as moderate compactive effort (65 gyration) mixes. This report indicated that this type of design should balance rut and crack resistance as the stiff RAP binder can offset the additional virgin binder in terms of rut resistance and the additional virgin binder can help to offset the stiff RAP binder in terms of crack resistance. Use of 75% RAP base mixtures is not recommended due to inability to adequately coat virgin coarse aggregate.

11.4 Recommendations for Further Research

- Test multiple RAP sources in the manner described in Chapter 5 and compare the results to conventional methods. A key component of the investigation should be absorbed asphalt and G_{sb}.
- The use of high RAP in airfield surface mixtures needs further investigation due to higher tire pressures and FOD associated durability problems. Durability and moisture damage wheel tracking should be performed.
- Produce 25% RAP mix at full scale and place on surface of low volume roadway for monitoring. Samples of the raw materials and plant produced mix should be sampled for laboratory testing similar to that conducted in this study. Properties of the pavement sampled mixture and should be tested as a function of time.
- Compare moisture resistance in a wheel tracking test for aggregate gradations designed with hot and warm mixed protocols.
- Moisture damage of warm mixed RAP should be should be compared to hot mixed RAP after multiple aging durations.
- Use of 50% RAP in base mixtures needs further investigation; especially fatigue crack resistance. Limited data in this study and several literature sources indicated that crack resistance may not be a major problem but further study is needed.

REFERENCES

Abatech, Inc. (2000). *Thermal Stress Analysis Routine (TSAR™): Software Users Manual*. Abatech, Inc., Doylestown, PA, pp. 31.

Abbas, A.R., and Ali, A. (2011). *Mechanical Properties of Warm Mix Asphalt Prepared Using Foamed Asphalt Binders*. Report No. FHWA/OH-2011/6, Ohio Department of Transportation, pp. 132.

Aguiar-Moya, J.P., Hong, F., and Prozzi, J.A. (2011). "RAP: Save Today, Pay Later?." *Proc., Transportation Research Board 90th Annual Meeting*, Paper No. 11-1017, Washington D.C., pp.16.

Alabama Department of Transportation. (2008). *Standard Specifications for Highway Construction*. Alabama Department of Transportation, Montgomery, AL.

Al-Qadi, I.L., Elseifi, M., and Carpenter, S.H. (2007). *Reclaimed Asphalt Pavement-A Literature Review*. Report FHWA-ICT-07-001, Illinois Department of Transportation, pp. 23.

Al-Qadi, I.L., Carpenter, S.H., Roberts, G., Ozer, H., Aurangzeb, Q., Elseifi, M., and Trepanier, J. (2009). *Determination of Usable Residual Asphalt Binder in RAP*. Report No. FHWA-ICT-09-031, Illinois Department of Transportation, pp. 94.

Alvarez, A.E., Martin, A.E., Estakhri, C., and Izzo, R. (2008). "Evaluation of Durability Tools for Porous Friction Courses." *Proc., Transportation Research Board 87th Annual Meeting*, Paper No. 08-2370, Washington D.C., pp.15.

American Association of State Highway and Transportation Officials (AASHTO). (2007). "M 323-07: Standard Specification for Superpave Volumetric Mix Design." *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, 29th Ed., Washington, D.C.

AASHTO. (2009). "R 49-09: Standard Practice for Determination of Low Temperature Performance Grade (PG) of Asphalt Binders." *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, 29th Ed., Washington, D.C.

Amirkhanian, S.N., and Williams, B. (1993). "Recyclability of Moisture Damaged Flexible Pavements." *Journal of Materials in Civil Engineering*, Vol. 5, No. 4, pp. 510-530.

Aravind, K., and Das, A. (2007). “Pavement Design with Central Plant Hot-Mix Recycled Asphalt Mixes.” *Construction and Building Materials*, Vol. 21, pp. 928-936.

Arkansas Department of Transportation. (2003). *Standard Specifications for Highway Construction*. Arkansas Department of Transportation, Little Rock, AR.

Aschenbrener, T., and Currier, G. (1993). *Influence of Testing Variables on the Results From the Hamburg Wheel-Tracking Device*. Report No. CDOT-DTD-R-93-22, Colorado Department of Transportation, Denver, CO.

Ashenbrener, T. (1995). “Evaluation of Hamburg Wheel-Tracking Device to Predict Moisture Damage in Hot-Mix Asphalt.” *Transportation Research Record: Journal of the Transportation Research Board,* No. 1492, pp. 193-201.

Austerman, A.J., Mogawer, W.S., and Bonaquist, R. (2009). “Evaluating the Effects of Warm Mix Asphalt Technology Additive Dosages on the Workability and Durability of Asphalt Mixtures Containing Recycled Asphalt Pavement.” *Proc., Transportation Research Board 88th Annual Meeting*, Paper No. 09-1279, Washington D.C., pp.18.

Azari, H. (2010). *Precision Estimates of AASHTO T 283: Resistance of Compacted Hot Mix Asphalt (HMA) to Moisture-Induced Damage*. NCHRP Web-Only Document 166, National Cooperative Highway Research Program, Transportation Research Board of the National Academies, Washington D.C.

Badaruddin, S.R.B., and White, T.D. (1994). *Asphalt Mix Design and Performance*. Report No. FHWA/IN/JHRP-94/03, Joint Highway Research Project, Indiana Department of Transportation, pp. 179.

Bagampadde, U., Isacsson, U., and Kiggundu, B.M. (2006). “Impact of Bitumen and Aggregate Composition on Stripping in Bituminous Mixtures.” *Materials and Structures,* Vol. 39, No. 3, pp. 303-315.

Bautista, E.G., Mangiafico, S., and Bahia, H.U. (2009). “Evaluation of Rheological Properties of Binders in RAP Without Extraction and Recovery.” *Proc., Transportation Research Board 88th Annual Meeting*, Paper No. 09-3076, Washington D.C., pp. 16.

Bennert, T., and Dongré, R. (2010). “Backcalculation Method to Determine Effective Asphalt Binder Properties of Recycled Asphalt Pavement Mixtures.” *Transportation Research Record: Journal of the Transportation Research Board,* No. 2179, pp. 75-84.

Betenson, W. D. (1979). “Recycled Asphalt Concrete in Utah.” *Proc., Association of Asphalt Paving Technologists*, Vol. 48, pp. 272-295.

Bistor, B. (2008). “Florida Contractor Used Warm Mix in challenging Applications.” *Warm-Mix Asphalt: Contractor’s Experiences*, Information Series 134, National Asphalt Pavement Association, Lanham, MD, pp. 18-19.

Boggs, W. (2008). "Customers Warm up to Green System." *Warm-Mix Asphalt: Contractor's Experiences*, Information Series 134, National Asphalt Pavement Association, Lanham, MD, pp. 16-17.

Bonaquist, R. (2007). "Can I Run More RAP?." *Hot Mix Asphalt Technology,* Vol. 12, No. 5, pp. 11-13.

Bonaquist, R. (2011). *Mix Design Practice for Warm Mix Asphalt*. National Cooperative Highway Research Program, Report 691, Transportation Research Board, Washington D.C., pp. 101.

Bouldin, M.G., Dongré, R., Rowe, G.R., Sharrock, M.J., and Anderson, D.A. (2000). "Predicting Thermal Cracking of Pavements from Binder Properties: Theoretical Basis and Field Validation." *Asphalt Paving Technology: Journal of the Association of Asphalt Paving Technologists*, Vol. 69, pp. 455-496.

Brock, J. D., and Richmond, J.L. (2007). *Milling and Recycling.* Technical Paper T-127, ASTEC Industries Inc., Chattanooga, TN, pp. 33.

Brown, E.R., and Cross, S.A. (1989). "A Study of In-Place Rutting of Asphalt Pavements." *Proc., Association of Asphalt Paving Technologists,* Vol. 58, pp. 1-39.

Brown, E.R. (1990). "Density of Asphalt Concrete-How Much is Needed?." *Transportation Research Record: Journal of the Transportation Research Board,* No. 1282, pp. 27-32.

Brown, E.R., Kandhal, P.S., and Zhang, J. (2001). *Performance Testing for Hot Mix Asphalt*. NCAT Report No. 01-05, National Center for Asphalt Technology, Auburn, AL, pp. 72.

Buchanan, S. (2000). "An Evaluation of Selected Methods for Measuring the Bulk Specific Gravity of Compacted Hot Mix Asphalt (HMA) Samples." *Asphalt Paving Technology: Journal of the Association of Asphalt Paving Technologists,* Vol. 69, pp. 608-634.

Buchanan, M.S., and Smith, B.J. (2005). "Performance Evaluation of Hot-Mix Asphalt Using Rotary Loaded-Wheel Testing." *Transportation Research Record: Journal of the Transportation Research Board,* No. 1929, pp. 157-164.

Carpenter, S.H., and Wolosick, J.R. (1980). "Modifier Influence in the Characterization of Hot-Mix Recycled Material." *Transportation Research Record: Journal of the Transportation Research Board,* No. 777, pp. 15-22.

Carvalho, R.L., Shirazi, H., Ayres Jr., M., and Selezneva, O. (2010). " Performance of Recycled Hot Mix Asphalt Overlays in Rehabilitation of Flexible Pavements." *Proc., Transportation Research Board 89th Annual Meeting*, Paper No. 10-2209, Washington D.C., pp. 15.

Cascione, A.A., Williams, R.C., Buttlar, W.G., Ahmed, S., Hill, B., Haugen, D.S., and Gillen, S. (2011). "Laboratory Evaluation of Field Produced Hot Mix Asphalt Containing Post-Consumer Recycled Asphalt Shingles and Fractionated Recycled Asphalt Pavement." *Asphalt Paving Technology: Journal of the Association of Asphalt Paving Technologists,* Vol. 80, In Press.

Castro-Fernandez, P.L. (1996). *Evaluation of Hot Asphalt Concrete Mixes Containing Recycled Materials as an Alternative for Pavement Rehabilitation*. MS Thesis, University of Nevada, Reno.

Celauro, C., Bernardo, C., and Gabriele, B. (2010). "Production of Innovative, Recycled and High-Performance Asphalt for Road Pavements." *Resources, Conservation and Recycling*, Vol. 54, pp. 337-347.

Çelik, O.N., and Atiş, C.D. (2008). "Compactability of Hot Bituminous Mixture made with Crumb Rubber-Modified Binders." *Construction and Building Materials*, Vol. 22, pp. 1143-1147.

Chadbourn, B.A., Newcomb, D.E., Voller, V.R., De Sombre, R.A., Luoma, J.A., and Timm, D.H. (1998.) *An Asphalt Paving Tool for Adverse Conditions*. Report No. MN/RC-1998-18, Minnesota Department of Transportation, pp. 145.

Chehab, G.R., and Daniel, J.S. (2006). "Evaluating Recycled Asphalt Pavement Mixtures with Mechanistic-Empirical Pavement Design Guide Level 3 Analysis." *Transportation Research Record: Journal of the Transportation Research Board,* No. 1962, pp. 90-100.

Chen, D-H., and Daleiden, J. (2005). "Lessons Learned from the Long-Term Pavement Performance Program and Several Recycled Sections in Texas." *Roadway Pavement Preservation:2005,* Transportation Research Circular No. E-C078, Transportation Research Board, Washington, D.C., pp. 70-84.

Chen, J.S., Chu, P.Y., Lin, Y.Y., and Lin, K.Y. (2007). "Characterization of Binder and Mix Properties to Detect Reclaimed Asphalt Pavement Content in Bituminous Mixtures." *Canadian Journal of Civil Engineering,* Vol. 34, No. 5, pp. 581-588.

Cooley Jr., L.A., Brown, E.R., and Maghsoodloo, S. (2001). "Developing Critical Field Permeability and Pavement Density Values for Coarse-Graded Superpave Pavements." *Transportation Research Record: Journal of the Transportation Research Board,* No. 1761, pp. 41-49.

Cooley Jr., L.A. (2003). *Evaluation of Pavement Permeability in Mississippi*. Report to Mississippi Department of Transportation Research Division, pp. 51.

Cooper, S.B. (2009). *Evaluation of HMA Mixtures Containing Sasobit®*. Report No. 06-1TA, Louisiana Transportation Research Center, pp. 36.

Copas, T.L., and Pennock, H.A. (1978). *Recycling Materials for Highways.* National Cooperative Highway Research Program Synthesis of Highway Practice, No. 54, Washington, D.C., pp. 53.

Copeland, A. (2011). *Reclaimed Asphalt Pavement in Asphalt Mixtures: State-of-the-Practice*. Report No. FHWA-HRT-11-021, Federal Highway Administration, pp. 55.

Copeland, A., D'Angelo, J., Dongré, R., Belagutti, S., and Sholar, G. (2010). "Field Evaluation of High Reclaimed Asphalt Pavement-Warm Mix Asphalt Project in Florida: Case Study." *Transportation Research Record: Journal of the Transportation Research Board,* No. 2179, pp. 93-101.

Cross, S.A., Voth, M.D., and Fager, G.A. (2000). "Effects of Sample Preconditioning on Asphalt Pavement Analyzer Wet Rut Depths." *Proc., Mid-Continent Transportation Symposium 2000,* May 15-16, Ames, IA.

D'Angelo, J.D., Harm, E., Bartoszek, J., Baumgardner, G., Corrigan, M., Cowsert, J., Harman, T., Jamshidi, M., Jones, W., Newcomb, D., Prowell, B., Sines, R., and Yeaton, B. (2008). *Warm-Mix Asphalt: European Practice.* FHWA Report No. FHWA-PL-08-007, Federal Highway Administration, Washington, D.C., pp. 68.

Daniel, J.S., and Lachance, A. (2005). "Mechanistic and Volumetric Properties of Asphalt Mixtures with Recycled Asphalt Pavement." *Transportation Research Record: Journal of the Transportation Research Board,* No. 1929, pp. 28-36.

Daniel, J.S., Chehab, G.R., and Ayyala, D. (2009). "Sensitivity of RAP Binder Grade on Performance Prediction in the MEPDG." *Asphalt Paving Technology: Journal of the Association of Asphalt Paving Technologists*, Vol. 78, pp. 352-376.

Daniel, J.S., Pochilly, J.L., and Boisvert, D.N. (2010). "Can More Reclaimed Asphalt Pavement be Added?: Study of Extracted Binder Properties from Plant-Produced Mixtures with up to 25% Reclaimed Asphalt Pavement." *Transportation Research Record: Journal of the Transportation Research Board,* No. 2180, pp. 19-29.

Davidson, D.D., Canessa, W., and Escobar, S.J. (1977). "Recycling of Substandard or Deteriorated Asphalt Pavements-A Guideline for Design Procedures." *Proc., Association of Asphalt Paving Technologists,* Vol. 46, pp. 496-525.

Diefenderfer, S.D., McGhee, K.K., and Donaldson, B.M. (2007). *Installation of Warm Mix Asphalt Projects in Virginia*. Report No. FHWA/VTCR 07-R2, Virginia Department of Transportation, pp. 32.

Doyle, J.D., and Howard, I.L. (2010a). "Compactability and Bitumen Utilization of 100% Warm Mixed RAP." *Proc., Transportation Research Board 89th Annual Meeting*, Paper No. 10-0279, Washington D.C., pp. 21.

Doyle, J.D., and Howard, I.L. (2010b). *Laboratory Investigation of High RAP Content Pavement Surface Layers*. Report No. FHWA/MS-DOT-RD-10-212, Mississippi Department of Transportation, pp. 310.

Doyle, J.D., and Howard, I.L. (2011). *Linear Asphalt Compactor Operator's Manual*. Manual No. CMRC M 10-2 Version 2, Construction Materials Research Center, Mississippi State University, pp. 16.

Druta, C., Wang, W., and Zhu, T. (2009). "Laboratory Investigation of Reclaimed Asphalt Pavement Mixed with Pure Binder Using X-ray CT Scanner." *Proc., Ninth International Conference on Chinese Transportation Professionals: Critical Issues in Transportation Systems Planning, Development, and Management*, Aug 5-9, Harbin, China, pp. 2611-2620.

Dunning, R.L., Mendenhall, R.L., and Tischer, L.K. (1975). "Recycling of Asphalt Concrete Description of Process and Test Sections." *Proc., Association of Asphalt Paving Technologists,* Vol. 44, pp. 537-562.

Dunning, R.L. (1983). "Construction Considerations for Pavement Recycling." *Proc. New Mexico Paving Conference,* Albuquerque, NM, pp. 6-26.

Epps, J.A., O'Neal, R.J., and Gallaway, B.M. (1976). "A Review of Pavement Materials Recycling Techniques and Associated Laboratory Tests and Evaluation." *Proc., Association of Asphalt Paving Technologists,* Vol. 45, pp. 304-329.

Epps, J.A., Little, D.N., Holmgreen, R.J., and Terrel, R.L. (1980). *Guidelines for Recycling Pavement Materials.* National Cooperative Highway Research Program, Report No. 224, Transportation Research Board, pp. 137.

Federal Highway Administration (FHWA). (1979). *Hot Recycling of Asphalt Pavement Materials*. FHWA Technical Advisory T5040.9. <http://www.fhwa.dot.gov/legsregs/directives/notices/n5080-93.htm> (Accessed Sept 10, 2007).

Florida Department of Transportation. (2010). *Standard Specifications for Road and Bridge Construction*. Florida Department of Transportation, Tallahassee, FL.

Georgia Department of Transportation. (2001). *Standard Specifications: Construction of Roads and Bridges*. Georgia Department of Transportation, Atlanta, GA.

Hajj, E.Y., Sebaaly, P.E., and Kandiah, P. (2010). "Evaluation of the Use of Reclaimed Asphalt Pavement in Airfields HMA Pavements." *Journal of Transportation Engineering*, Vol. 136, No. 3, pp. 181-189.

Hall, K.D., and Williams, S.G. (1999a). "Effects of the Ignition Method on Aggregate Properties." *Asphalt Paving Technology: Journal of the Association of Asphalt Paving Technologists,* Vol. 68, pp. 574-588.

Hall, K.D., and Williams, S.G. (1999b). *Acquisition and Evaluation of Hamburg Wheel-Tracking Device*. Report No. FR 1044, Mack-Blackwell Transportation Center, University of Arkansas, pp. 18.

Hicks, R.G. (1991). *Moisture Damage in Asphalt Concrete*. National Cooperative Highway Research Program Synthesis of Highway Practice, No. 175, Washington, D.C., pp. 90.

Hills, J. F., and Brien, D. (1966). "The Fracture of Bitumens and Asphalt Mixes by Temperature Induced Stresses: Prepared Discussion." *Proc., Association of Asphalt Paving Technologists*, Vol. 35, pp. 292-309.

Hodo, W.D., Kvasnak, A., and Brown, E.R. (2009). "Investigation of Foamed Asphalt (Warm Mix Asphalt) with High Reclaimed Asphalt Pavement (RAP) Content for Sustainment and Rehabilitation of Asphalt Pavement." *Proc., Transportation Research Board 88th Annual Meeting*, Paper No. 09-3789, Washington, D.C., pp. 15.

Hossain, M., Metcalf, D.G., and Scofield, L.A. (1993). "Performance of Recycled Asphalt Concrete Overlays in Southwestern Arizona." *Transportation Research Record: Journal of the Transportation Research Board*, No. 1427, pp. 30-37.

Howard, I.L., Cooley Jr., L.A., and Doyle, J.D. (2009). *Laboratory Testing and Economic Analysis of High RAP Warm Mixed Asphalt.* Report No. FHWA/MS-DOT-RD-09-200, Mississippi Department of Transportation, pp. 104.

Howard, I.L., Doyle, J.D., White, T.D., Ivy, J., and Booth, O. (2010). *PURWheel Laboratory Wheel Tracker Operator's Manual*. Manual No. CMRC M 10-2 Version 1, Construction Materials Research Center, Mississippi State University, pp. 65.

Huang, B., Li, G., Vukosavljevic, D., Shu, X., and Egan, B.K. (2005). "Laboratory Investigation of Mixing Hot-Mix Asphalt with Reclaimed Asphalt Pavement." *Transportation Research Record: Journal of the Transportation Research Board,* No. 1929, pp. 37-45.

Huber, G.A., and Heiman, G.H. (1987). "Effect of Asphalt Concrete Parameters on Rutting Performance: A Field Investigation." *Proc., Association of Asphalt Paving Technologists,* Vol. 56, pp. 33-61.

Hughes, C.S. (1989). *Compaction of Asphalt Pavement.* National Cooperative Highway Research Program Synthesis of Highway Practice, No. 152, Washington, D.C., pp. 42.

Hunter, E.R., and Ksaibati, K. (2002). *Evaluating Moisture Susceptibility of Asphalt Mixes*. MPC Report No. 02-138, University of Wyoming, pp. 159.

Hurley, G.C., and Prowell, B.D. (2005a). "Refinement of the Hot-Mix Asphalt Ignition Method for High-Loss Aggregates." *Transportation Research Record: Journal of the Transportation Research Board,* No. 1907, pp. 128-134.

Hurley, G.C., and Prowell, B.D. (2005b). *Evaluation of Sasobit® for use in Warm Mix Asphalt.* NCAT Report No. 05-06, National Center for Asphalt Technology, Auburn, AL, pp. 27.

Hurley, G.C., and Prowell, B.D. (2006). *Evaluation of Evotherm® for use in Warm Mix Asphalt.* NCAT Report No. 06-02, National Center for Asphalt Technology, Auburn, AL, pp. 44.

Hurley, G.C., and Prowell, B.D. (2008). "Field Performance of Warm Mix Asphalt." *Proc., Transportation Research Board 87th Annual Meeting*, Paper No. 08-2999, Washington, D.C., pp. 17.

Izzo, R.P., and Tahmoressi, M. (1999). "Testing Repeatability of the Hamburg Wheel-Tracking Device and Replicating Wheel-Tracking Devices Among Different Laboratories." *Asphalt Paving Technology: Journal of the Association of Asphalt Paving Technologists*, Vol. 68, pp. 589-611.

Jones, G.M., Darter, M.I., and Littlefield, G. (1968). "Thermal Expansion-Contraction of Asphaltic Concrete." *Proc., Association of Asphalt Paving Technologists*, Vol. 37, pp. 56-100.

Kallas, B.F. (1984). *Flexible Pavement Mixture Design Using Reclaimed Asphalt Concrete.* Research Report No. 84-2, Asphalt Institute, Lexington, KY, pp. 15.

Kandhal, P.S. (1992). *Moisture Susceptibility of HMA mixes: Identification of Problem and Recommended Solutions.* NCAT Report No. 92-01, National Center for Asphalt Technology, Auburn, AL, pp. 35.

Kandhal, P.S., and Khatri, M.A. (1992). "Relating Asphalt Absorption to Properties of Asphalt Cement and Aggregates." *Transportation Research Record: Journal of the Transportation Research Board*, No. 1342, pp. 76-84.

Kandhal, P.S., Rao, S.S., Watson, D.E., and Young, B. (1995). *Performance of Recycled Hot Mix Asphalt Mixtures.* NCAT Report No. 95-01, National Center for Asphalt Technology, Auburn, AL, pp. 19.

Kari, W.J., Santucci, L.E., and Coyne, L.D. (1979). "Hot Mix Recycling of Asphalt Pavements." *Proc., Association of Asphalt Paving Technologists,* Vol. 48, pp. 192-220.

Khedaywi, T.S., and White, T.D. (1995). "Development and Analysis of Laboratory Techniques for Simulating Segregation." *Transportation Research Record: Journal of the Transportation Research Board,* No. 1492, pp. 36-45.

Kiggundu, B.N., and Roberts, F.L. (1988*a*). *Stripping in HMA Mixtures: State-of-the-art and Critical Review of Test Methods*. NCAT Report No. 88-02, National Center for Asphalt Technology, Auburn, AL, pp. 42.

Kiggundu, B.N., and Roberts, F.L. (1988*b*). *The Success/Failure of the Test Methods*. NCAT Report No. 88-03, National Center for Asphalt Technology, Auburn, AL, pp. 14.

Kim, Y.R., and Lutif, J.E.S. (2006). *Material Selection and Design Consideration for Moisture Damage of Asphalt Pavement*. Report No. P564, Nebraska Department of Roads, Lincoln, NE, pp. 68.

Kim, K.W., Amirkhanian, S.N., Jung, J., and Doh, Y.S. (2007). "Reducing Stiffness of Oxidized Binder in Recycled Asphalt Mixture by Stage Mixing." *Asphalt Paving Technology: Journal of the Association of Asphalt Paving Technologists,* Vol. 76, pp. 201-234.

Kraus, Z.R. (2008). *The Morphology of Polymer Modified Asphalt and its Relationship to Rheology and Durability*. MS Thesis, Texas A&M University.

Kristjansdottir, O., Muench, S.T., Michael, L., and Burke, G., (2007). "Assessing the Potential for Warm Mix Asphalt Technology Adoption." *Transportation Research Record: Journal of the Transportation Research Board*, No. 2040, pp. 91-99.

Kvasnak, A., West, R., Moore, J., Nelson, J., Turner, P., and Tran, N. (2009). "Case Study of Warm Mix Asphalt Moisture Susceptibility in Birmingham." *Proc., Transportation Research Board 88th Annual Meeting*, Paper No. 09-3703, Washington D.C., pp. 13.

Kvasnak, A., West, R., Michael, J., Loria, L., Hajj, E.Y., and Tran, N. (2010). "Bulk Specific Gravity of Reclaimed Asphalt Pavement Aggregate: Evaluating the Effect on Voids in Mineral Aggregate." *Transportation Research Record: Journal of the Transportation Research Board*, No. 2180, pp. 30-35.

Lee, T-C., Terrel, R.L., and Mahoney, J.P. (1983). "Measurement of Mixing Efficiency in Pavement Recycling." *Proc., Association of Asphalt Paving Technologists,* Vol. 52, pp. 61-87.

Leiva, F., and West, R.C. (2008a). "Analysis of Hot-Mix Asphalt Lab Compactability Using Lab Compaction Parameters and Mix Characteristics." *Transportation Research Record: Journal of the Transportation Research Board*, No. 2057, pp. 89-98.

Leiva, F, and West, R.C. (2008b). "Relationships between Laboratory Measured Characteristics of HMA and Field Compactability." *Asphalt Paving Technology: Journal of the Association of Asphalt Paving Technologists*, Vol. 77, pp. 183-219.

Li, X., Marasteanu, M.O., Williams, R.C., and Clyne, T.R. (2008). "Effect of Reclaimed Asphalt Pavement (Proportion and Type) and Binder Grade on Asphalt Mixtures." *Transportation Research Record: Journal of the Transportation Research Board*, No. 2051, pp. 90-97.

Linden, R.N., Mahoney, J.P., and Jackson, N.C. (1989). "Effect of Compaction on Asphalt Concrete Performance." *Transportation Research Record: Journal of the Transportation Research Board,* No. 1217, pp. 20-28.

Little, D.N., and Epps, J.A. (1980). "Evaluation of Certain Structural Characteristics of Recycled Pavement Materials." *Proc., Association of Asphalt Paving Technologists*, Vol. 49, pp. 219-249.

Louisiana Department of Transportation and Development. (2006). *Standard Specifications for Roads and Bridges*. Louisiana Department of Transportation and Development, Baton Rouge, LA.

Lu, Q. (2005). *Investigation of Conditions for Moisture Damage in Asphalt Concrete and Appropriate Laboratory Test Methods.* Ph.D. Dissertation, University of California at Berkley.

Lytton, R.L, Uzan, J., Fernando, E.G., Roque, R., Hiltunen, D., and Stoffels, S.M. (1993). *Development and Validation of Performance Prediction Models and Specifications for Asphalt Binders and Paving Mixes*. Strategic Highway Research Program Report A-357, National Research Council, Washington, D. C.

Ma, T., Mahmoud, E., and Bahia, H.U. (2010). "Estimation of Reclaimed Asphalt Pavement Binder Low-Temperature Properties Without Extraction." *Transportation Research Record: Journal of the Transportation Research Board*, No. 2179. pp. 58-65.

Mallick, R.B., Bradley, J.E., Bradbury, R.L. (2007). "An Evaluation of Heated Reclaimed Asphalt Pavement Material and Wax-Modified Asphalt for Use in Recycled Hot Mix Asphalt." *Transportation Research Record: Journal of the Transportation Research Board*, No. 1998. pp. 112-122.

Mallick, R.B., Kandhal, P.S., Bradbury, R.L. (2008). "Using Warm Mix Asphalt Technology to Incorporate High Percentage of Reclaimed Asphalt Pavement Material in Asphalt Mixes." *Transportation Research Record: Journal of the Transportation Research Board*, No. 2051. pp. 71-79.

Marasteanu, M.O., and Anderson, D.A. (2001). "Techniques for Determining Errors in Asphalt Binder Rheological Data." *Transportation Research Record: Journal of the Transportation Research Board*, No. 1766. pp. 32-39.

Marasteanu, M., Basu, A., Hesp, S.A.M., and Vollor, V. (2004). "Time-Temperature Superposition and AASHTO MP1a Critical Temperature for Low-Temperature Cracking." *International Journal of Pavement Engineering*, Vol. 5, No. 1, pp. 31-38.

Marasteanu, M., Velasquez, R., Flachetto, A.C., and Zofka, A. (2009). *Development of a Simple Test to Determine the Low Temperature Creep Compliance of Asphalt Mixtures*. IDEA Program Final Report for Project 133, Transportation Research Board, Washington D.C., pp. 51.

Maupin, G.W., Diefenderfer, S.D., and Gillespie, J.S. (2009). "Performance and Economic Evaluation of Virginia's Higher RAP Specification." *Proc., Transportation Research Board 88th Annual Meeting*, Paper No. 09-3425, Washington D.C., pp. 17.

MeadWestvaco. (2011). *Evotherm Warm Mix Asphalt Product Brochure.* <http://www.meadwestvaco.com/mwv/groups/content/documents/document/mwv006575.pdf> (Accessed March 26, 2011).

McDaniel, R.S., Soleymani, H., Anderson, R.M., Turner, P., and Peterson, R. (2000). *Recommended Use of Reclaimed Asphalt Pavement in the Superpave Mix Design Method.* NCHRP Web Document 30 (Project D9-12): Contractor's Final Report, pp. 351.

McDaniel, R., and Anderson, R.M. (2001). *Recommended Use of Reclaimed Asphalt Pavement in the Superpave Mix Design Method: Technician's Manual*. National Cooperative Highway Research Program, Report 452, Transportation Research Board, Washington D.C., pp. 49.

McDaniel, R.S., Soleymani, H., and Shah, A. (2002). *Use of Reclaimed Asphalt Pavement (RAP) Under Superpave Specifications: A Regional Pooled Fund Study.* Report No. FHWA/IN/JTRP-2002/6, Indiana Department of Transportation, pp. 69.

McGee, J.A., and Judd, A.J. (1978). "Recycling Asphaltic Concrete: Arizona's First Project." *Transportation Research Record: Journal of the Transportation Research Board,* No. 695, pp. 47-49.

Middleton, B. and Forfylow, R.W. (2009). "Evaluation of Warm-Mix Asphalt Produced with the Double Barrel Green Process." *Transportation Research Record: Journal of the Transportation Research Board,* No. 2126, pp. 19-26.

Mississippi Department of Transportation (MDOT). (2004). *Mississippi Standard Specifications for Road and Bridge Construction*. Mississippi Department of Transportation, Jackson, MS.

MDOT. (2006). *Field Manual For Hot Mix Asphalt (HMA)*. Mississippi Department of Transportation Materials Division, Jackson, MS.

Mogawer, W.S., Austerman, A.J., Engstrom, B., and Bonaquist, R. (2009). "Incorporating High Percentages of Recycled Asphalt Pavement (RAP) and Warm Mix Asphalt (WMA) Technology into Thin Hot Mix Asphalt Overlays to be Utilized as a Pavement Preservation Strategy." *Proc., Transportation Research Board 88th Annual Meeting*, Paper No. 09-1275, Washington D.C., pp. 21.

Mogawer, W.S., Austerman, A.J., Bonaquist, R., and Roussel, M. (2011a). "Performance Characteristics of Thin Lift Overlay Mixtures Containing High RAP Content, RAS, and Warm Mix Asphalt Technology." *Proc., Transportation Research Board 90th Annual Meeting*, Paper No. 11-1859, Washington D.C., pp.15.

Mogawer, W.S., Austerman, A.J., Kassem, E., and Masad, E. (2011b). "Moisture Damage Characteristics of Warm Mix Asphalt Mixtures." *Asphalt Paving Technology: Journal of the Association of Asphalt Paving Technologists,* Vol. 80, In Press.

Nam, K., and Bahia, H.U. (2004). "Effect of Binder and Mixture Variables on Glass Transition Behavior of Asphalt Mixtures." *Asphalt Paving Technology: Journal of the Association of Asphalt Paving Technologists*, Vol. 73, pp. 89-120.

Nelson, P. (2008). "Lehman-Roberts Debuts Terex Warm-Mix System." *Warm-Mix Asphalt: Contractor's Experiences*, Information Series 134, National Asphalt Pavement Association, Lanham, MD, pp. 20-21.

Newcomb, D.E., Brown, E.R., and Epps, J.A. (2007). *Designing HMA Mixtures with High RAP Content: A Practical Guide.* Quality Improvement Series 124, National Asphalt Pavement Association, Lanham, MD, pp. 36.

Nielson, J. (2010). *Development of a Testing Temperature to be used with the Hamburg Wheel Tracking Device on Asphalt Mixtures that Utilize Performance Grade Binders*. MS Thesis, University of Utah.

North Carolina Department of Transportation. (2002). *Standard Specifications for Roads and Structures.* North Carolina Department of Transportation, Raleigh, NC.

Noureldin, A.S., and Wood, L.E. (1987). "Rejuvenator Diffusion in Binder Film for Hot-Mix Recycled Asphalt Pavement." *Transportation Research Record: Journal of the Transportation Research Board*, No. 1115, pp. 51-60.

Ozer, H, Al-Qadi, I.L., Carpenter, S., Aurangzeb, Q., Roberts, G.L., and Trepanier, J. (2009). "Evaluation of RAP Impact on Hot-Mix Asphalt Design and Performance." *Asphalt Paving Technology: Journal of the Association of Asphalt Paving Technologists,* Vol. 78, pp. 317-351.

Pan, C., and White, T.D. (1999). *Conditions for Stripping Using Accelerated Testing*. Report No. FHWA/IN/JTRP/97-13, Joint Transportation Research Program, Indiana Department of Transportation, pp. 190.

Paul, H.R. (1996). "Evaluation of Recycled Projects for Performance." *Asphalt Paving Technology: Journal of the Association of Asphalt Paving Technologists,* Vol. 65, pp. 231-254.

Peterson, R.L., Soleymani, H.R., Anderson, R.M., and McDaniel, R.S. (2000). "Recovery and Testing of RAP Binders from Recycled Asphalt Pavement." *Asphalt Paving Technology: Journal of the Association of Asphalt Paving Technologists,* Vol. 69, pp. 72-88.

Potter, J.F., and Mercer, J. (1997). "Full-Scale Performance Trials and Accelerated Testing of Hot-Mix Recycling in the UK." *Proc., Eighth International Conference on Asphalt Pavements*, Vol. 1, August 10-14, Seattle, WA, pp. 593-607.

Prowell, B.D., and Carter, C.B. (2000). *Evaluation of the Effect on Extracted Aggregate Properties of Samples Extracted Using the Ignition Furnace*. Interim Report No. VTRC 00-IR1, Virginia Transportation Research Council, pp. 18.

Prowell, B.D., and Hurley, G.C. (2005). *Refinement of the Hot Mix Asphalt Ignition Method for High Loss Aggregates*. NCAT Report No. 05-05, National Center for Asphalt Technology, Auburn, AL, pp. 34.

Prowell, B.D., and Brown, E.R. (2007). *Superpave Mix Design: Verifying Gyration Levels in the N_{design} Table*. National Cooperative Highway Research Program, Report 573, Transportation Research Board, Washington D.C., pp. 62.

Prowell, B.D., and Hurley, G.C. (2007). *Warm Mix Asphalt: Best Practices.* Quality Improvement Series 125, National Asphalt Pavement Association, Lanham, MD, pp. 44.

Prowell, B.D., Hurley, G.C., and Crews, E. (2007). "Field Performance of Warm-Mix Asphalt at National Center for Asphalt Technology Test Track." *Transportation Research Record: Journal of the Transportation Research Board,* No. 1998, pp. 96-102.

Roberts, F.L., Kandhal, P.S., Brown, E.R., Lee, D.Y., and Kennedy, T.W. (1991). *Hot mix Asphalt Materials, Mixture Design and Construction*. NAPA Education Foundation, Lanham, MD, pp. 585.

Rowe, G.M., Sharrock, M.J., Bouldin, M.G., and Dongré, R.N. (2001). "Advanced Techniques to Develop Asphalt Master Curves from the Bending Beam Rheometer." *Petroleum and Coal*, Vol. 43, No. 1, pp. 54-59.

Santos, L.G.P., Baptista, A.M.C., and Capitão, S.D. (2010). "Assessment of the Use of Hot-Mix Recycled Asphalt Concrete in Plant." *Journal of Transportation Engineering*, Vol. 136, No. 12, pp. 1159-1164.

Sasol Wax. (2004). *The Bitumen Additive for Highly Stable Easily Compactable Asphalts*. Product Information 124, Sasol Wax GmbH. <http://www.sasolwax.com/ data/sasolwax_/Bitumen%20Modification/Sasobit%20since%201997.pdf> (Accessed Nov 01, 2007).

Seo, Y., El-Haggan, O., King, M., Lee, S.J., and Kim, Y.R. (2007). "Air Void Models for the Dynamic Modulus, Fatigue Cracking, and Rutting of Asphalt Concrete." *Journal of Materials in Civil Engineering,* Vol. 19, No. 10, pp. 874-883.

Shah, A., McDaniel, R.S., Huber, G.A., and Gallivan, V.L. (2007). "Investigation of Properties of Plant-Produced Reclaimed Asphalt Pavement Mixtures." *Transportation Research Record: Journal of the Transportation Research Board,* No. 1998, pp. 103-111.

Shenoy, A. (2002). "Single-Event Cracking Temperature of Asphalt Pavements Directly from Bending Beam Rheometer Data." *Journal of Transportation Engineering*, Vol. 128, No. 5, pp. 465-471.

Shirodkar, P., Mehta, Y.A., Nolan, A., Sonpal, K., Norton, A., Tomlinson, C., Sauber, R., and DuBois, E. (2010). "A Study to Determine the Degree of Partial Blending of Reclaimed Asphalt Pavement (RAP) Binder for High RAP Hot Mix Asphalt." *Proc., Transportation Research Board 89th Annual Meeting*, Paper No. 10-0917, Washington D.C., pp. 15.

Shiwakoti, H. (2007). *Development of a Rapid Test to Determine Moisture Sensitivity of HMA (Superpave) Mixtures*. MS Thesis, University of Kansas.

Shoenberger, J.E., and Demoss, T.A. (2009). "Hot-Mix Recycling of Asphalt Concrete Airfield Pavements." *International Journal of Pavement Engineering*, Vol. 6, No. 1, pp. 17-26.

Shu, X., Huang, B., and Vukosavljevic, D. (2008). "Laboratory Evaluation of Fatigue Characteristics of Recycled Asphalt Mixture." *Construction and Building Materials*, Vol. 22, pp. 1323-1330.

Sivasubramaniam, S., Galal, K.A., Noureldin, A.S., White, T.D., and Haddock, J.E. (2004). "Laboratory, Prototype, and In-Service Accelerated Pavement Testing to Model Permanent Deformation." *Transportation Research Record: Journal of the Transportation Research Board,* No. 1896, pp. 189-198.

Smith, R.W. (1980). "State-of-the-Art Hot Recycling." *Transportation Research Record: Journal of the Transportation Research Board*, No. 780, pp. 115-124.

Solaimanian, M., and Tahmoressi, M. (1996). "Variability Analysis of Hot-Mix Asphalt Concrete Containing High Percentage of Reclaimed Asphalt Pavement." *Transportation Research Record: Journal of the Transportation Research Board,* No. 1543, pp 89-96.

Soleymani, H.R., Anderson, M., McDaniel, R., and Abdelrahman, M. (2000). "Investigation of the Black Rock Issue for Recycled Asphalt Mixtures." *Asphalt Paving Technology: Journal of the Association of Asphalt Paving Technologists,* Vol. 69, pp. 366-390.

South Carolina Department of Transportation. (2010). *Supplemental Technical Specification for Hot Mix Asphalt (HMA) Quality Assurance*. South Carolina Department of Transportation, Columbia, SC.

Stephens, J.E., Mahoney, J., and Dippold, C. (2001). *Determination of the PG Binder Grade to Use in a RAP Mix.* Report No. JHR 00-278, University of Connecticut-Connecticut Transportation Institute, pp. 38.

Su., K., Hachiya, Y., and Maekawa, R. (2009). "Study on Recycled Asphalt Concrete for use in Surface Course in Airport Pavement." *Resources, Conservation and Recycling*, Vol. 54, pp. 37-44.

Sullivan, J. (1996). "Pavement Recycling Executive Summary and Report." *Final Report No. FHWA-SA-95-060*, Federal Highway Administration, Washington, D.C., pp. 119.

Tabaković, A., Gibney, A., McNally, C., and Gilchrist, M.D. (2010). "Influence of Recycled Asphalt Pavement on Fatigue Performance of Asphalt Concrete Base Courses." *Journal of Materials in Civil Engineering*, Vol. 22, No. 6, pp. 643-650.

Tao, M., and Mallick, R.B. (2009). "Effects of Warm-Mix Asphalt Additives on Workability and Mechanical Properties of Reclaimed Asphalt Pavement Material." *Transportation Research Record: Journal of the Transportation Research Board*, No. 2126. pp. 151-160.

Taylor, N.H. (1975). "Prepared Discussion in Response to Recycling of Asphalt Concrete Description of Process and Test Sections." *Proc., Association of Asphalt Paving Technologists,* Vol. 44, pp. 561-562.

Terrel, R.L., and Al-Swailmi, S. (1993). "Role of Pessium Voids Concept in Understanding Moisture Damage to Asphalt Concrete Mixtures." *Transportation Research Record: Journal of the Transportation Research Board*, No. 1386. pp. 31-37.

Texas Department of Transportation. (2004). *Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges*. Texas Department of Transportation. Austin, TX.

Thakur, S.C., Han, J., Chong, W.K., and Parsons, R.L. (2011). "Comparison of Properties of RAP Aggregates Extracted by Ignition and Centrifuge Methods." *Proceedings of Geo-Frontiers 2011*, Advances in Geotechnical Engineering-Geotechnical Special Publication No. 211 (CD-ROM), ASCE, Mar 13-16, Dallas, TX, pp. 4525-4534.

Thompson, G. (2003). *Determining Asphalt Content for Recycled Asphalt Pavement (RAP) Materials.* Final Report No. FHWA-OR-DF-04-02, Oregon Department of Transportation, pp. 32.

Tia, M., Wood, L.E., and Hancher, D.E. (1980). "The Effects of Added Virgin Binders on the Properties of Cold-mix Recycled Asphalt Paving Mixtures." *Proc., Association of Asphalt Paving Technologists*, Vol. 49, pp. 199-218.

Timm, D.H., Willis, J.R., and Kvasnak, A. (2011). "Full-Scale Structural Evaluation of Fatigue Characteristics in High RAP and Warm Mix Asphalt." *Proc., Transportation Research Board 90th Annual Meeting*, Paper No. 11-2509, Washington D.C., pp. 15.

United States Army Corps of Engineers (USACE). (2010). *Hot-Mix Asphalt (HMA) for Airfields*. Unified Facilities Guide Specification UFGS-32 12 15, pp. 40.

Vavrick, W.R., and Carpenter, S.H. (2008). "Calculating Air Voids at Specified Number of Gyrations in Superpave Gyratory Compactor." *Transportation Research Record: Journal of the Transportation Research Board*, No. 1630, pp. 117-125.

Vavrik, W.R., Carpenter, S.H., Gillen, S., Behnke, J., and Garrott, F. (2008). *Evaluation of Field-Produced Hot Mix Asphalt (HMA) Mixtures with Fractionated Recycled Asphalt Pavement (RAP).* Report No. ICT-08-030, Illinois State Toll Highway Authority, pp. 56.

Velasquez, R.A. (2009). *On the Representative Volume Element of Asphalt Concrete with Applications to Low Temperature*. Ph.D. Dissertation, University of Minnesota.

Vinson, T.S., Janoo, V.C., and Haas, R.C.G. (1989). *Summary Report: Low Temperature and Thermal Fatigue Cracking*. Strategic Highway Research Program Report A-306, National Research Council, Washington, D.C., pp. 83.

Watson, D.E., Moore, K.A., Williams, K., and Cooley Jr., L.A. (2003). "Refinement of New-Generation Open-Graded Friction Course Mix Design." *Transportation Research Record: Journal of the Transportation Research Board,* No. 1832, pp. 78-85.

Watson, D.E., Cooley Jr., L.A., Moore, K.A., and Williams, K. (2004). "Laboratory Performance Testing of Open-Graded Friction Course Mixtures." *Transportation Research Record: Journal of the Transportation Research Board,* No. 1891, pp. 40-47.

Watson, D.E., Moore, J., Jared, D., and Wu, P. (2008). "Evaluation of the Use of Reclaimed Asphalt Pavement in Stone Matrix Asphalt Mixtures." *Transportation Research Record: Journal of the Transportation Research Board,* No. 2051, pp. 64-70.

West, R.C., Zhang, J., and Cooley Jr., L.A. (2004). *Evaluation of the Asphalt Pavement Analyzer for Moisture Sensitivity Testing*. NCAT Report No. 04-04, National Center for Asphalt Technology, Auburn, AL, pp. 37.

West, R., Kvasnak, A., Tran, N., Powell, B., and Turner, P. (2009). "Testing of Moderate and High Reclaimed Asphalt Pavement Content Mixes: Laboratory and Accelerated Field Performance Testing at the National Center for Asphalt Technology Test Track." *Transportation Research Record: Journal of the Transportation Research Board,* No. 2126, pp. 100-108.

West, R., Michael, J., Turochy, R., and Maghsoodloo, S. (2011). "A Comparison of Virgin and Recycled Asphalt Pavements Using Long-Term Pavement Performance SPS-5 Data." *Proc., Transportation Research Board 90th Annual Meeting*, Paper No. 11-2864, Washington D.C., pp. 17.

White, T.D. (1977). "Recycling Asphalt Pavements and Hot-Mix Recycling Mix Design Evaluation." *Proc., New Mexico Pavement Conference,* Albuquerque, NM, pp. 16-35.

Zaghloul, S., and Holland, T.J. (2008). "Comparative Analysis of Long-Term Field Performance of Recycled Asphalt in California Environmental Zones." *Transportation Research Record: Journal of the Transportation Research Board,* No. 2084, pp. 83-99.

Zaniewski, J., and Viswanathan, A.G. (2006). *Investigation of Moisture Sensitivity of Hot Mix Asphalt Concrete*. Report Submitted to West Virginia Division of Highways, pp. 87.

Zearly, L. (1979). "Symposium-Recycling of Asphalt Mixtures: Prepared Discussion." *Proc., Association of Asphalt Paving Technologists,* Vol. 48, pp. 339-341.

Zofka, A., Marasteanu, M., Li, X., Clyne, T., and McGraw, J. (2005). "Simple Method to Obtain Asphalt Binder Low Temperatures from Asphalt Mixture Properties." *Asphalt Paving Technology: Journal of the Association of Asphalt Paving Technologists*, Vol. 74, pp. 255-282.

Zofka, A., Marasteanu, M., and Turos, M. (2008). "Determination of Asphalt Mixture Creep Compliance at Low Temperatures Using Thin Beam Specimens." *Transportation Research Record: Journal of the Transportation Research Board,* No. 2057, pp. 134-139.

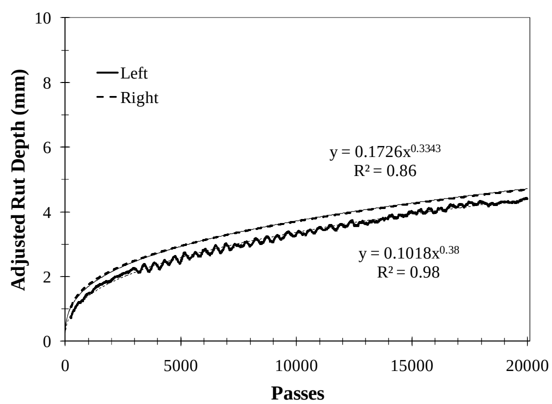
APPENDIX A

PURWHEEL TEST RESULTS

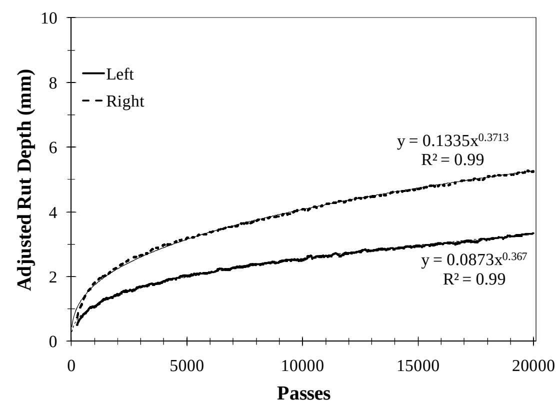
Table A.1 PURWheel Dry Test Results for Mixture *9.5-100/RM-1*

Replicate 1 (Air Voids 9.5%)				**Replicate 2 (Air Voids 9.8%)**			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	0.7	250	1.1	250	0.5	250	0.7
500	1.1	500	1.2	500	0.8	500	1.2
1000	1.5	1000	2.0	1000	1.1	1000	1.8
2000	1.9	2000	2.3	2000	1.5	2000	2.3
4000	2.3	4000	3.0	4000	1.9	4000	3.0
8000	2.9	8000	3.5	8000	2.4	8000	3.7
12000	3.6	12000	3.9	12000	2.7	12000	4.4
16000	4.0	16000	4.1	16000	3.0	16000	4.8
20000	4.4 **(6.3)**[1]	20000	5.0 **(5.0)**[1]	20000	3.3 **(4.4)**[1]	20000	5.3 **(4.3)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



a) Data Replicate 1



b) Data Replicate 2

c) Photo Replicate 1

d) Photo Replicate 2

Figure A.1 PURWheel Dry Test Results for Mixture *9.5-100/RM-1*

Table A.2 PURWheel Wet Test Results for Mixture *9.5-100/RM-1*

Replicate 1 (Air Voids 9.8%)				Replicate 2 (Air Voids 11.3%)			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	0.5	250	0.9	250	0.8	250	0.7
500	1.7	500	1.7	500	1.6	500	1.4
1000	2.6	1000	2.6	1000	2.9	1000	2.2
2000	3.3	2000	3.2	2000	4.4	2000	2.8
4000	5.0	4000	5.0	4000	6.1	4000	4.1
8000	10.1	8000	8.9	8000	9.5	8000	6.0
10620	23.5 **(---)**[1]	12000	20.3	12000	12.7	12000	7.8
---	---	12314	21.8 **(---)**[1]	16000	20.4	16000	9.0
---	---	---	---	17312	26.1 **(20.0)**[1]	20000	10.3 **(9.7)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg

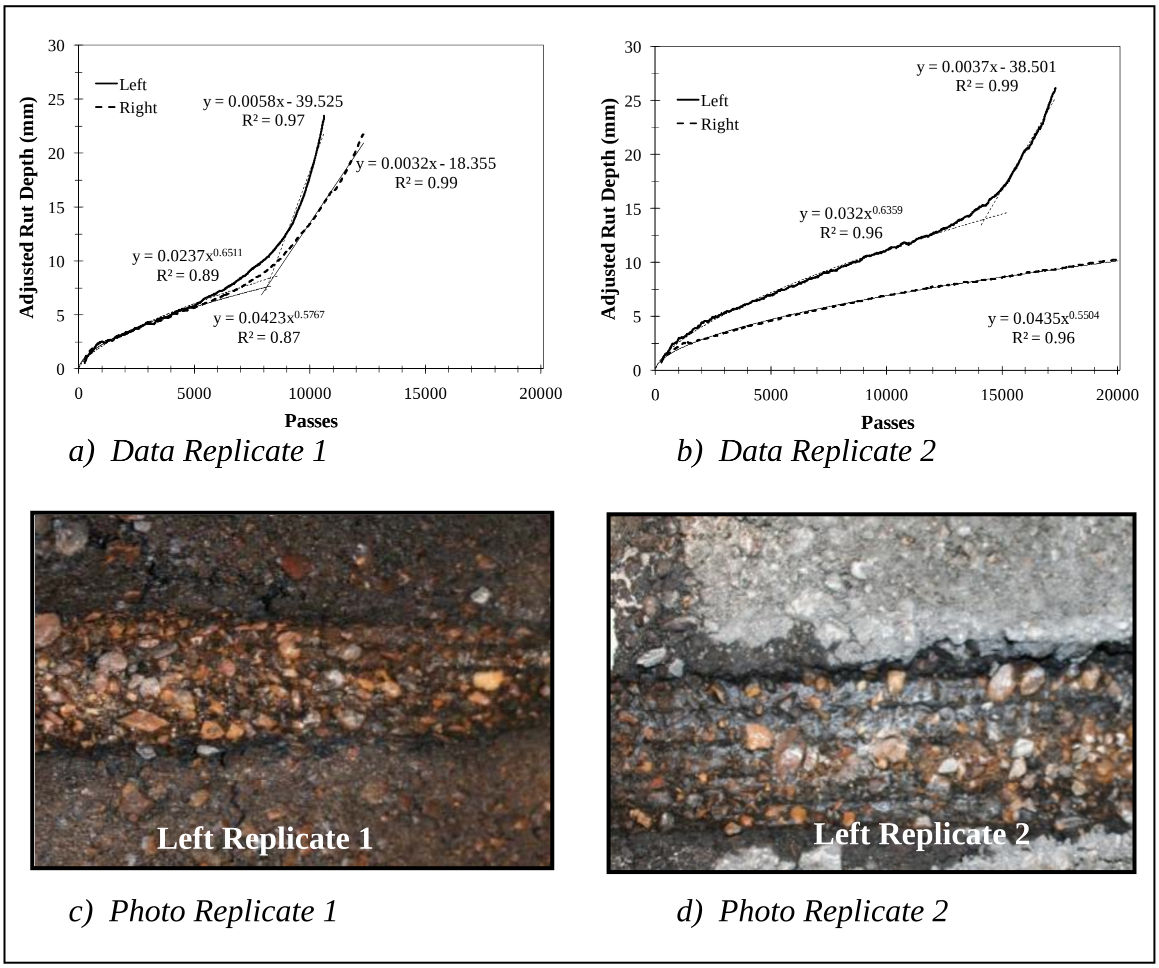
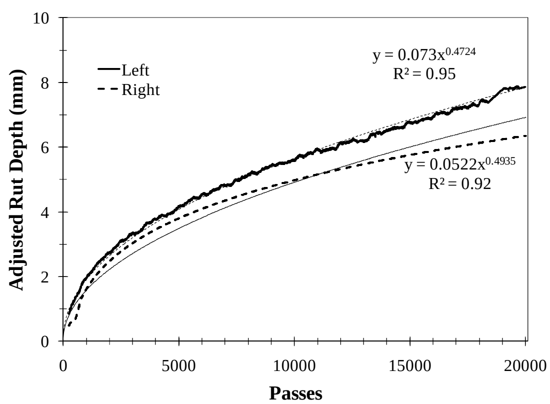


Figure A.2 PURWheel Wet Test Results for Mixture *9.5-100/RM-1*

Table A.3 PURWheel Dry Test Results for Mixture *9.5-100/RM-2*

Replicate 1 (Air Voids 10.0%)				**Replicate 2 (Air Voids 9.6%)**			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	0.8	250	0.5	250	0.3	250	0.5
500	1.3	500	0.6	500	0.8	500	0.9
1000	2.0	1000	1.8	1000	1.2	1000	1.3
2000	2.7	2000	2.2	2000	1.7	2000	1.8
4000	3.8	4000	3.5	4000	2.4	4000	2.5
8000	5.2	8000	4.1	8000	3.5	8000	3.4
12000	6.1	12000	5.5	12000	4.3	12000	4.3
16000	6.9	16000	5.7	16000	4.9	16000	4.9
20000	8.7 **(8.8)**[1]	20000	7.1 **(6.7)**[1]	20000	5.2 **(6.4)**[1]	20000	5.4 **(7.9)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



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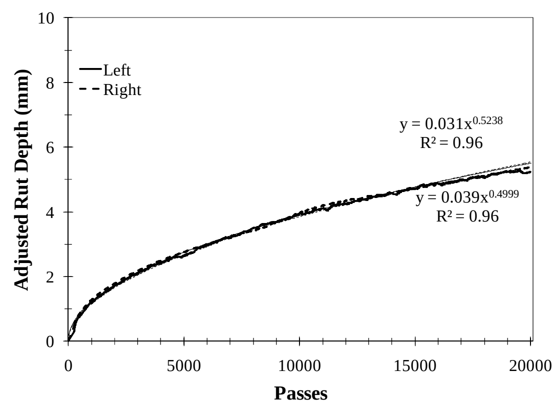


b) Data Replicate 2

c) Photo Replicate 1

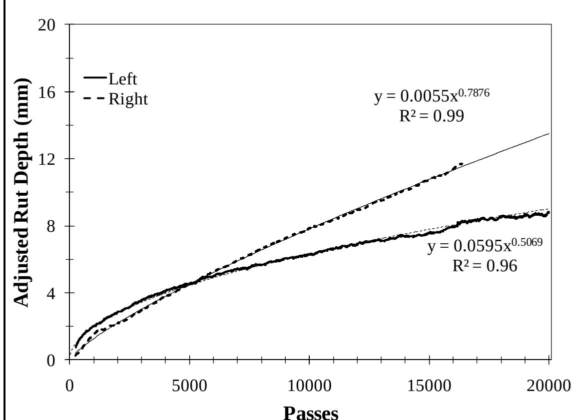
d) Photo Replicate 2

Figure A.3 PURWheel Dry Test Results for Mixture *9.5-100/RM-2*

Table A.4 PURWheel Wet Test Results for Mixture *9.5-100/RM-2*

Replicate 1 (Air Voids 8.8%)				**Replicate 2 (Air Voids 10.6%)**			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	0.7	250	0.3	250	0.9	250	0.7
500	1.4	500	0.7	500	1.9	500	1.4
1000	2.0	1000	1.5	1000	2.6	1000	2.1
2000	2.9	2000	2.2	2000	3.9	2000	3.3
4000	4.1	4000	3.8	4000	5.6	4000	5.1
8000	5.7	8000	6.6	8000	8.3	8000	9.9
12000	6.9	12000	8.9	12000	10.1	11490	19.8 **(---)**[1]
16000	7.9	16000	11.4	16000	11.8	---	---
20000	8.8 **(8.3)**[1]	16412	11.7 **(11.5)**[1]	20000	13.3 **(9.5)**[1]	---	---

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



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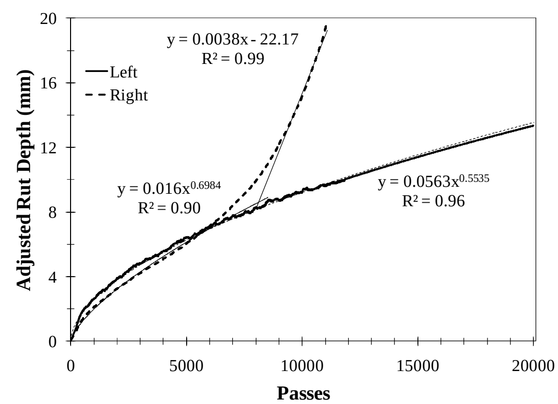

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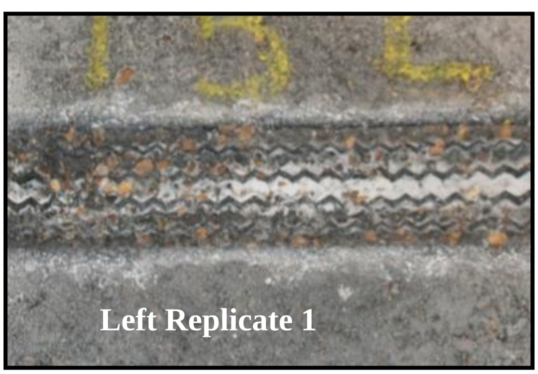


c) Photo Replicate 1

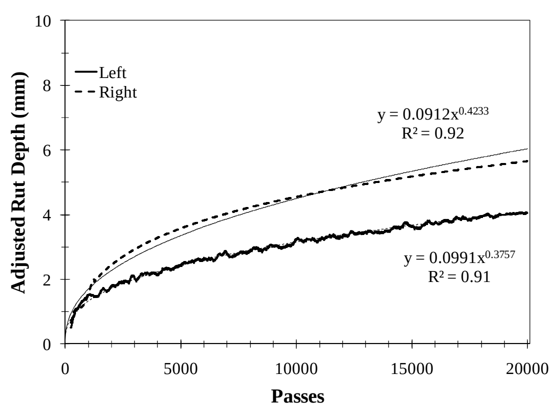
d) Photo Replicate 2

Figure A.4 PURWheel Wet Test Results for Mixture *9.5-100/RM-2*

Table A.5 PURWheel Dry Test Results for Mixture *12.5-100/RM-3*

Replicate 1 (Air Voids 9.5%)				**Replicate 2 (Air Voids 11.2%)**			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	0.5	250	0.7	250	0.1	250	0.4
500	1.1	500	1.1	500	0.4	500	0.8
1000	1.5	1000	1.6	1000	1.1	1000	1.2
2000	1.8	2000	2.5	2000	1.3	2000	1.7
4000	2.2	4000	3.2	4000	1.8	4000	2.4
8000	2.9	8000	4.0	8000	2.4	8000	2.4
12000	3.3	12000	5.0	12000	2.8	12000	3.3
16000	3.7	16000	5.4	16000	3.1	16000	4.2
20000	4.1 **(5.2)**[1]	20000	5.7 **(6.1)**[1]	20000	3.4 **(4.5)**[1]	20000	5.3 **(6.4)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



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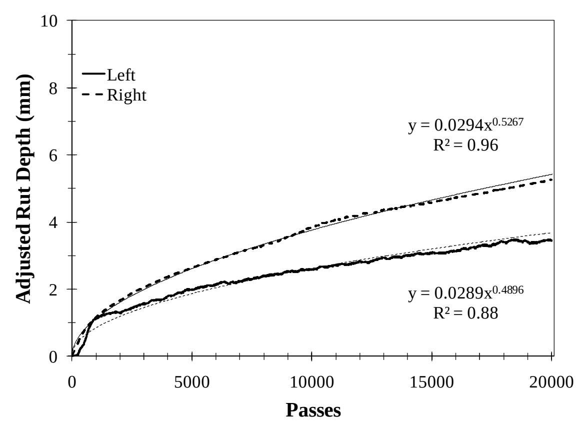


b) Data Replicate 2

c) Photo Replicate 1

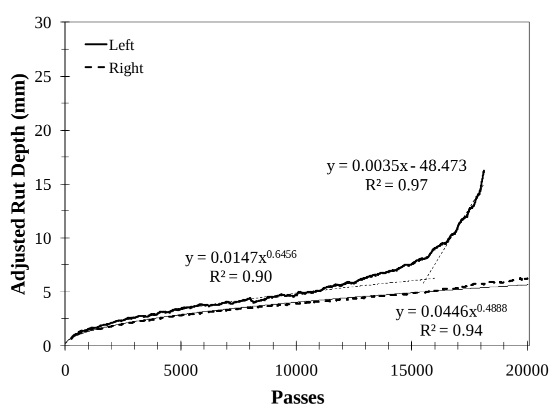
d) Photo Replicate 2

Figure A.5 PURWheel Dry Test Results for Mixture *12.5-100/RM-3*

Table A.6 PURWheel Wet Test Results for Mixture *12.5-100/RM-3*

Replicate 1 (Air Voids 8.7%)				**Replicate 2 (Air Voids 11.5%)**			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	0.7	250	0.6	250	1.0	250	0.8
500	1.1	500	1.1	500	1.9	500	1.2
1000	1.5	1000	1.5	1000	4.0	1000	1.8
2000	2.1	2000	1.8	2000	7.0	2000	2.8
4000	2.9	4000	2.5	3800	29.5 **(---)**[1]	4000	10.8
8000	4.4	8000	3.5	---	---	4174	18.1 **(---)**[1]
12000	5.7	12000	4.3	---	---	---	---
16000	9.0	16000	5.1	---	---	---	---
18130	16.2 **(13.1)**[1]	20000	6.2 **(5.4)**[1]	---	---	---	---

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C   Tire Pressure: 862 kPa   Wheel Load: 178.6 kg



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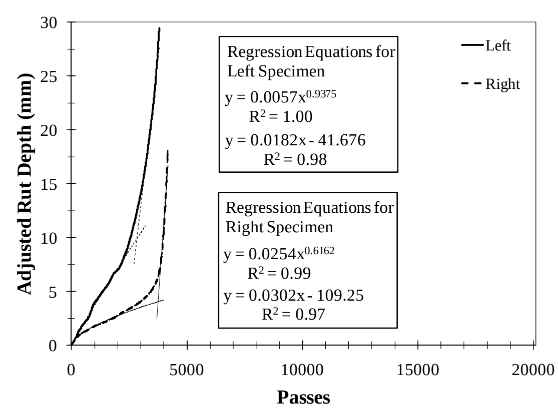


b) Data Replicate 2

c) Photo Replicate 1

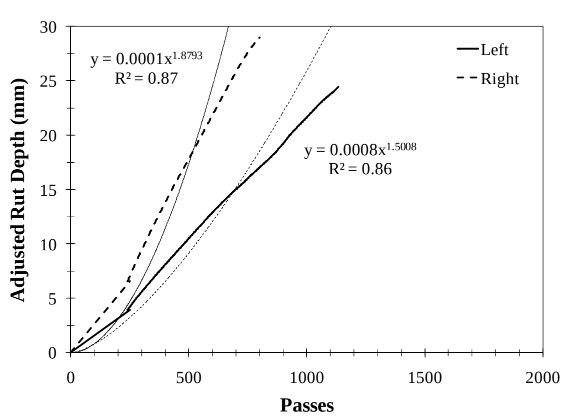
d) Photo Replicate 2

Figure A.6 PURWheel Wet Test Results for Mixture *12.5-100/RM-3*

Table A.7 PURWheel Dry Test Results for Mixture *9.5-15/CM-2*

Replicate 1 (Air Voids 9.2%)				Replicate 2 (Air Voids 9.1%)			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	4.2	250	7.0	250	4.3	250	10.9
500	10.5	500	17.8	500	11.3	272	12.5 **(18.6)**[1]
1000	21.7	800	29.0 **(---)**[1]	1000	22.7	---	---
1134	24.5 **(22.9)**[1]	---	---	1230	27.5 **(---)**[1]	---	---
---	---	---	---	---	---	---	---
---	---	---	---	---	---	---	---
---	---	---	---	---	---	---	---
---	---	---	---	---	---	---	---
---	---	---	---	---	---	---	---

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



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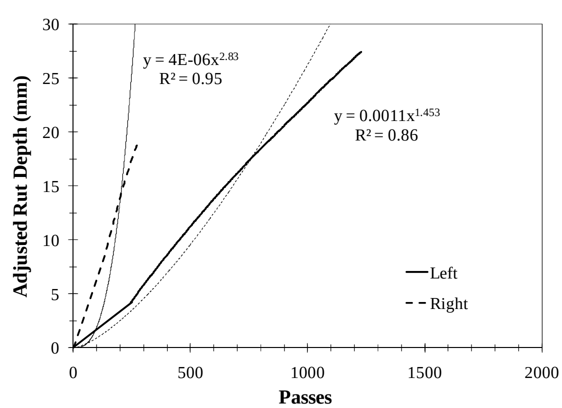

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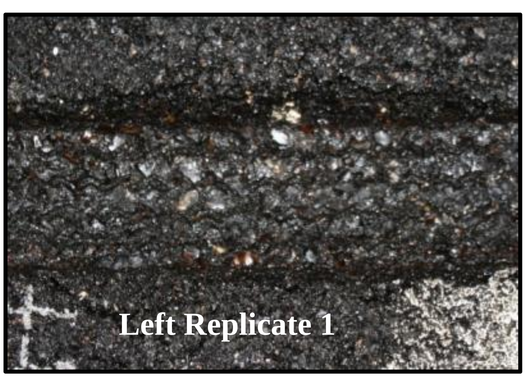


c) Photo Replicate 1

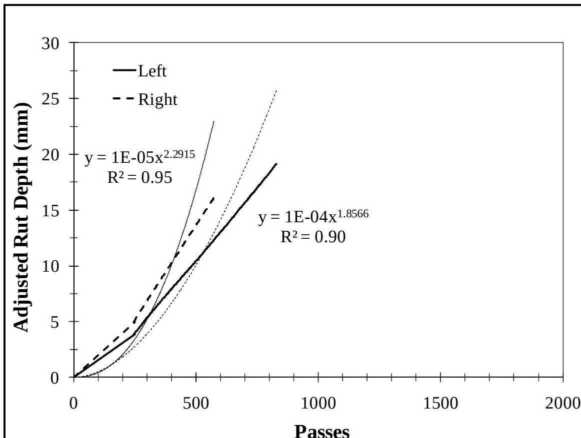
d) Photo Replicate 2

Figure A.7 PURWheel Dry Test Results for Mixture *9.5-15/CM-2*

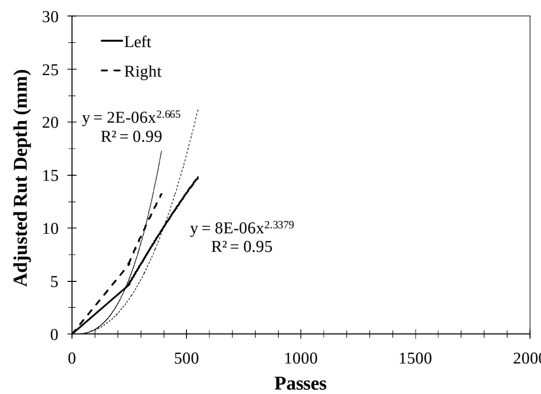
Table A.8 PURWheel Wet Test Results for Mixture *9.5-15/CM-2*

Replicate 1 (Air Voids 9.1%)				**Replicate 2 (Air Voids 9.2%)**			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	4.0	250	5.1	250	4.8	250	6.8
500	10.4	500	13.6	500	13.3	390	13.2 **(17.5)**[1]
828	19.1 **(18.7)**[1]	572	16.1 **(17.3)**[1]	550	14.8 **(17.6)**[1]	---	---
---	---	---	---	---	---	---	---
---	---	---	---	---	---	---	---
---	---	---	---	---	---	---	---
---	---	---	---	---	---	---	---
---	---	---	---	---	---	---	---
---	---	---	---	---	---	---	---

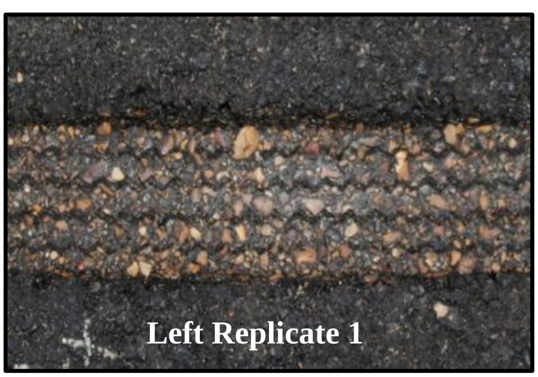
1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



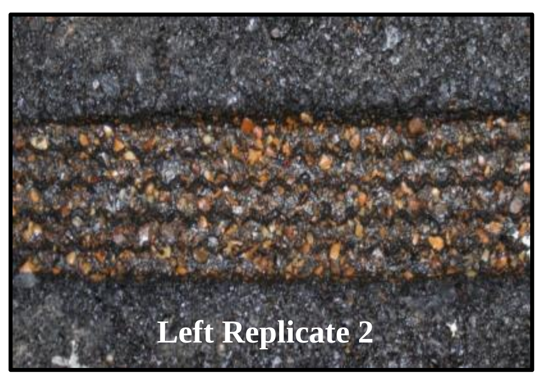
a) Data Replicate 1



b) Data Replicate 2



c) Photo Replicate 1



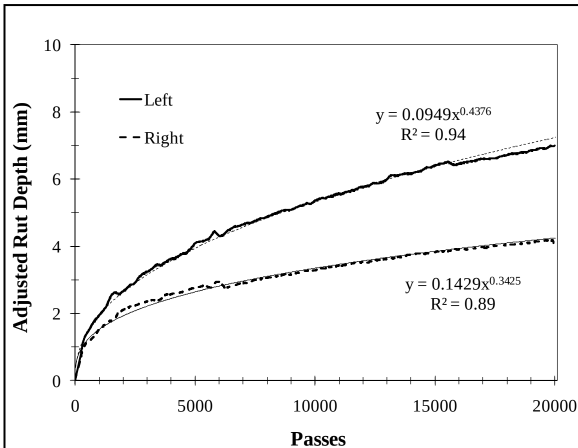
d) Photo Replicate 2

Figure A.8 PURWheel Wet Test Results for Mixture *9.5-15/CM-2*

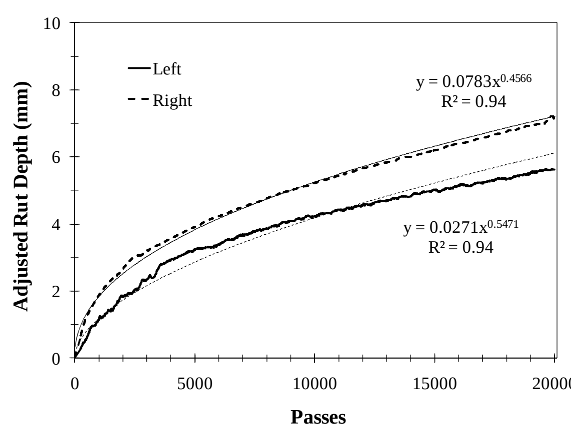
Table A.9 PURWheel Dry Test Results for Mixture *9.5-15/CM-3*

Replicate 1 (Air Voids 6.9%)				**Replicate 2 (Air Voids 8.8%)**			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	0.9	250	0.7	250	0.3	250	0.7
500	1.4	500	1.1	500	0.6	500	1.2
1000	1.9	1000	1.5	1000	1.1	1000	1.9
2000	2.7	2000	2.1	2000	1.8	2000	2.7
4000	3.6	4000	2.6	4000	2.9	4000	3.6
8000	4.9	8000	3.1	8000	3.9	8000	4.8
12000	5.8	12000	3.5	12000	4.6	12000	5.7
16000	6.5	16000	3.9	16000	5.2	16000	6.4
20000	7.0 **(8.6)**[1]	20000	4.0 **(5.8)**[1]	20000	5.6 **(6.9)**[1]	20000	7.1 **(9.2)**[1]

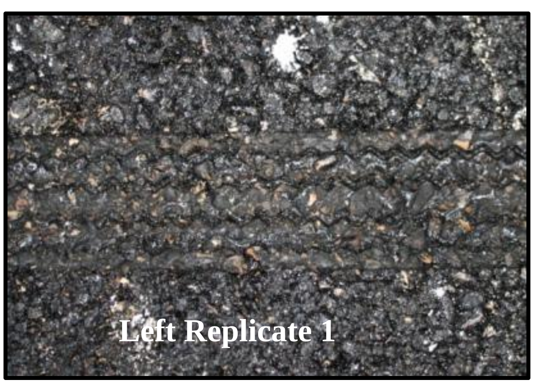
1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C   Tire Pressure: 862 kPa   Wheel Load: 178.6 kg



a) *Data Replicate 1*



b) *Data Replicate 2*



c) *Photo Replicate 1*

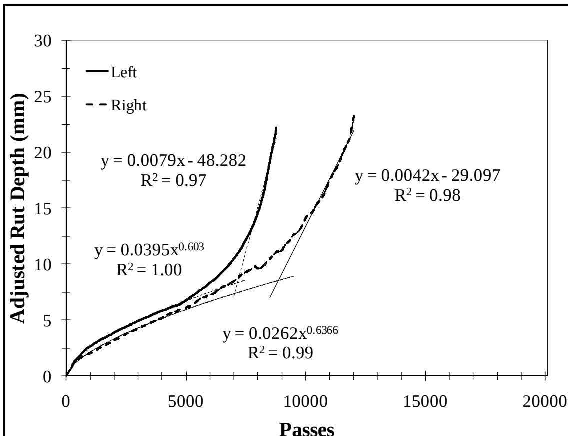
d) *Photo Replicate 2*

Figure A.9 PURWheel Dry Test Results for Mixture *9.5-15/CM-3*

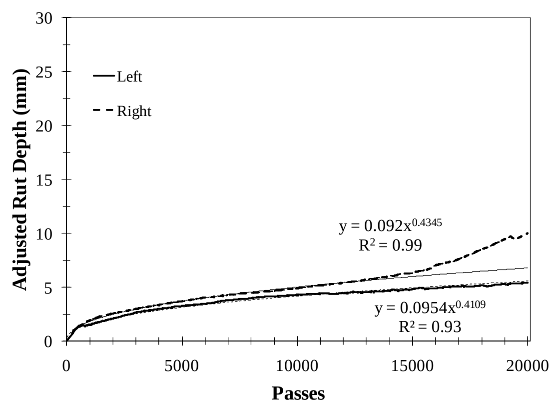
Table A.10 PURWheel Wet Test Results for Mixture *9.5-15/CM-3*

Replicate 1 (Air Voids 7.0%)				Replicate 2 (Air Voids 7.3%)			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	0.9	250	0.9	250	0.7	250	0.8
500	1.7	500	1.5	500	1.4	500	1.3
1000	2.6	1000	2.0	1000	1.5	1000	1.9
2000	3.9	2000	3.2	2000	2.1	2000	2.6
4000	5.8	4000	5.2	4000	3.0	4000	3.4
8000	14.5	8000	9.6	8000	8.2	8000	4.5
8782	22.2 **(---)**[1]	12000	23.0	12000	8.7	12000	5.4
---	---	12020	23.2 **(---)**[1]	16000	9.2	16000	7.0
---	---	---	---	20000	9.7 **(6.1)**[1]	20000	10.0 **(11.7)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



a) Data Replicate 1



b) Data Replicate 2

c) Photo Replicate 1

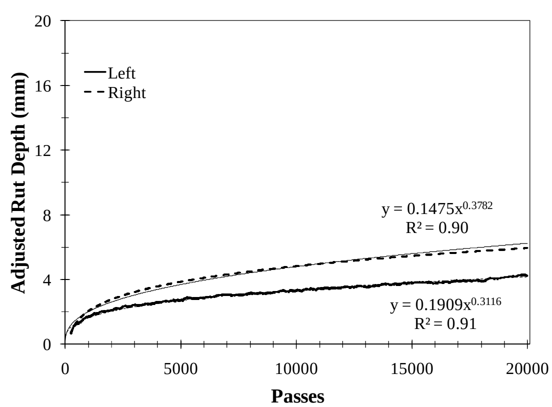
d) Photo Replicate 2

Figure A.10 PURWheel Wet Test Results for Mixture *9.5-15/CM-3*

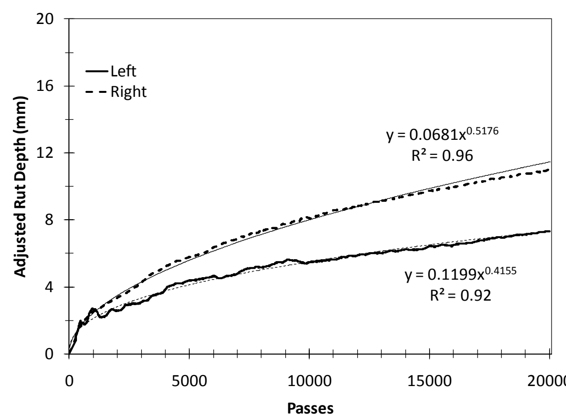
Table A.11 PURWheel Dry Test Results for Mixture *9.5-15/CM-4a*

Replicate 1 (Air Voids 8.0%)				**Replicate 2 (Air Voids 11.5%)**			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	0.7	250	0.7	250	0.8	250	1.0
500	1.3	500	1.4	500	1.8	500	1.7
1000	1.7	1000	2.4	1000	2.7	1000	2.5
2000	2.1	2000	2.8	2000	2.6	2000	3.3
4000	2.6	4000	3.0	4000	3.9	4000	5.2
8000	3.2	8000	3.9	8000	5.2	8000	7.4
12000	3.5	12000	4.6	12000	5.9	12000	8.8
16000	3.8	16000	5.4	16000	6.5	16000	10.0
20000	4.2 **(5.2)**[1]	20000	6.0 **(5.9)**[1]	20000	7.3 **(10.4)**[1]	20000	11.0 **(10.2)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



a) Data Replicate 1



b) Data Replicate 2

c) Photo Replicate 1

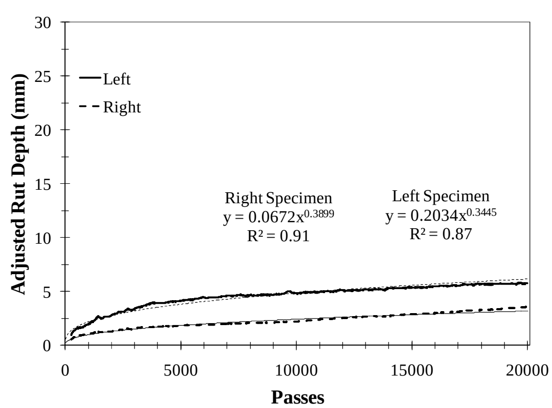
d) Photo Replicate 2

Figure A.11 PURWheel Dry Test Results for Mixture *9.5-15/CM-4a*

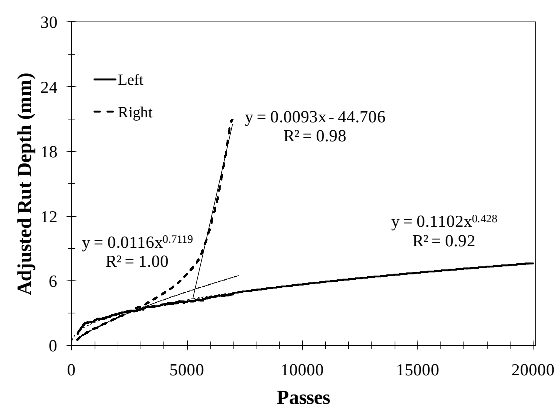
Table A.12 PURWheel Wet Test Results for Mixture *9.5-15/CM-4a*

Replicate 1 (Air Voids 7.4%)				Replicate 2 (Air Voids 4.8%)			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	1.0	250	0.5	250	1.0	250	0.5
500	1.6	500	0.9	500	1.9	500	1.0
1000	2.0	1000	1.1	1000	2.3	1000	1.6
2000	2.8	2000	1.3	2000	2.9	2000	2.0
4000	3.9	4000	1.7	4000	3.8	4000	4.9
8000	4.7	8000	2.1	8000	4.8	6978	21.0 **(---)**[1]
12000	5.1	12000	2.5	12000	5.7	---	---
16000	5.5	16000	3.0	16000	7.3	---	---
20000	5.8 **(5.1)**[1]	20000	3.6 **(6.3)**[1]	20000	7.9 **(6.1)**[1]	---	---

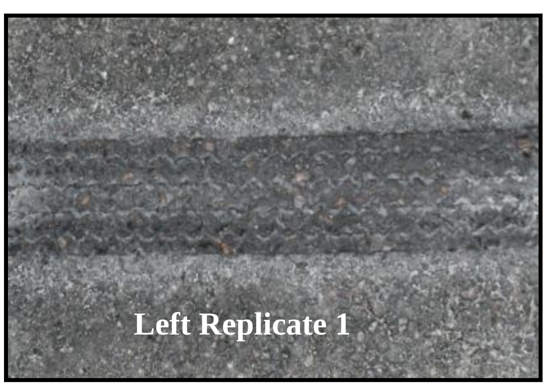
1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



a) Data Replicate 1



b) Data Replicate 2



c) Photo Replicate 1

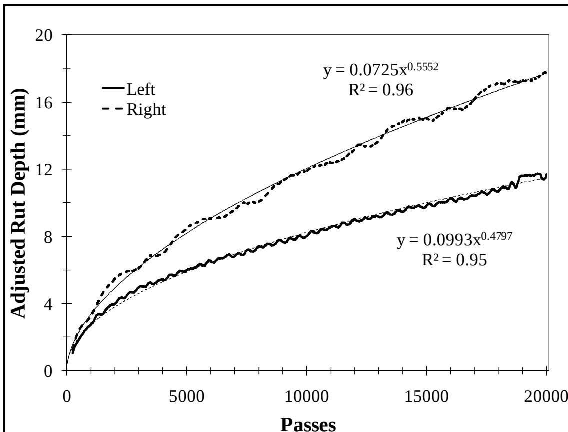
d) Photo Replicate 2

Figure A.12 PURWheel Wet Test Results for Mixture *9.5-15/CM-4a*

Table A.13 PURWheel Dry Test Results for Mixture *9.5-15/CM-4b*

Replicate 1 (Air Voids 10.8%)			
Left Specimen (mm)		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**
250	1.1	250	1.3
500	1.9	500	2.4
1000	2.8	1000	3.3
2000	4.0	2000	5.5
4000	5.4	4000	7.0
8000	7.4	8000	10.1
12000	9.0	12000	13.2
16000	10.3	16000	15.7
20000	11.7 **(13.1)**[1]	20000	17.8 **(15.1)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



a) Data Replicate 1

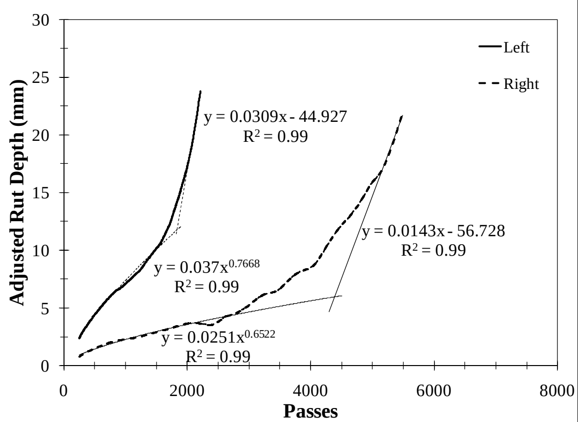
b) Photo Replicate 1

Figure A.13 PURWheel Dry Test Results for Mixture *9.5-15/CM-4b*

Table A.14 PURWheel Wet Test Results for Mixture *9.5-15/CM-4b*

Replicate 1 (Air Voids 10.7%)			
Left Specimen (mm)		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**
250	2.3	250	0.8
500	4.4	500	1.5
1000	7.1	1000	2.3
2000	17.3	2000	3.6
2214	23.8 **(16.3)**[1]	4000	8.5
---	---	5490	22.0 **(---)**[1]
---	---	---	---
---	---	---	---
---	---	---	---

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



a) Data Replicate 1

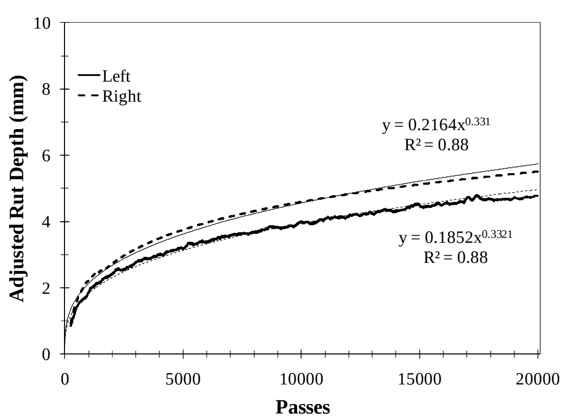
b) Photo Replicate 1

Figure A.14 PURWheel Wet Test Results for Mixture *9.5-15/CM-4b*

Table A.15 PURWheel Dry Test Results for Mixture *9.5-15/CM-4c*

Replicate 1 (Air Voids 11.2%)			
Left Specimen (mm)		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**
250	0.8	250	0.9
500	1.4	500	1.6
1000	1.9	1000	2.2
2000	2.4	2000	2.7
4000	3.0	4000	3.5
8000	3.7	8000	4.3
12000	4.2	12000	4.8
16000	4.5	16000	5.2
20000	4.8 **(5.5)**[1]	20000	5.5 **(6.3)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



a) Data Replicate 1

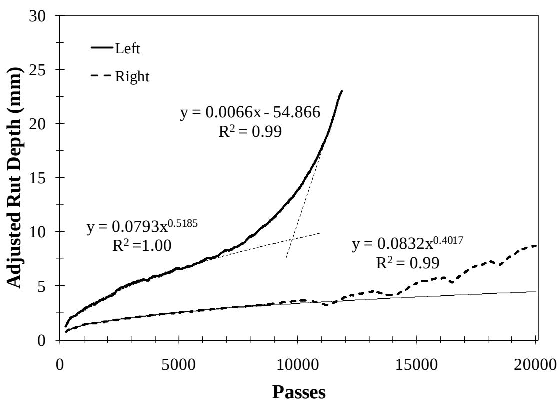
b) Photo Replicate 1

Figure A.15 PURWheel Dry Test Results for Mixture *9.5-15/CM-4c*

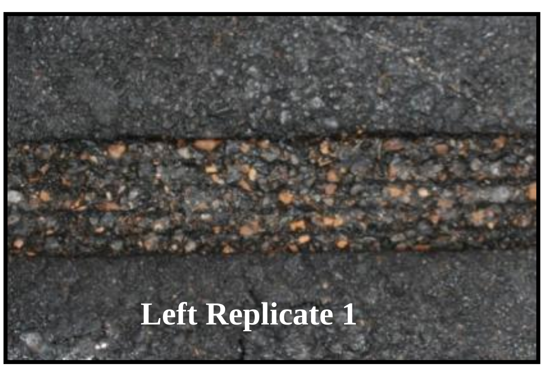
Table A.16 PURWheel Wet Test Results for Mixture *9.5-15/CM-4c*

Replicate 1 (Air Voids 10.7%)			
Left Specimen (mm)		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**
250	1.3	250	0.7
500	2.1	500	1.0
1000	2.8	1000	1.4
2000	4.0	2000	1.7
4000	5.8	4000	2.3
8000	9.6	8000	3.1
11842	23.0 **(---)**[1]	12000	3.9
---	---	16000	5.6
---	---	20000	8.7 **(13.7)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



a) Data Replicate 1



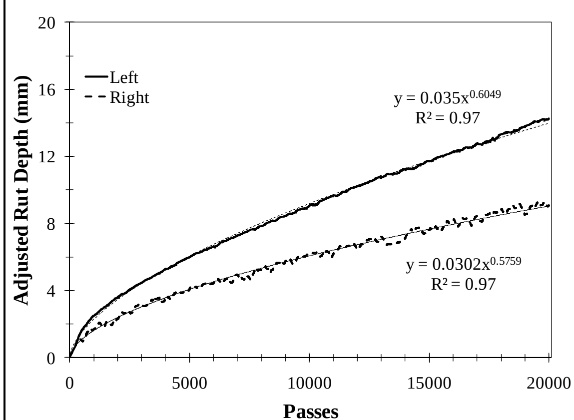
b) Photo Replicate 1

Figure A.16 PURWheel Wet Test Results for Mixture *9.5-15/CM-4c*

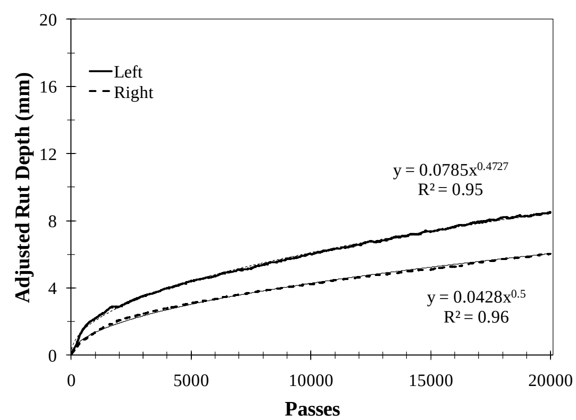
Table A.17 PURWheel Dry Test Results for Mixture *9.5-25/RM-1*

Replicate 1 (Air Voids 10.3%)				**Replicate 2 (Air Voids 9.0%)**			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	0.8	250	0.8	250	0.8	250	0.5
500	1.6	500	1.0	500	1.6	500	0.9
1000	2.5	1000	1.7	1000	2.2	1000	1.3
2000	3.6	2000	2.3	2000	2.9	2000	2.1
4000	5.3	4000	3.5	4000	4.0	4000	2.8
8000	7.8	8000	5.2	8000	5.4	6978	3.8
12000	10.2	12000	6.5	12000	6.6	12000	4.6
16000	12.3	16000	8.2	16000	7.7	16000	5.3
20000	14.3**(13.9)**[1]	20000	9.1 **(9.5)**[1]	20000	8.5 **(10.2)**[1]	20000	6.0 **(7.2)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



a) Data Replicate 1



b) Data Replicate 2

c) Photo Replicate 1

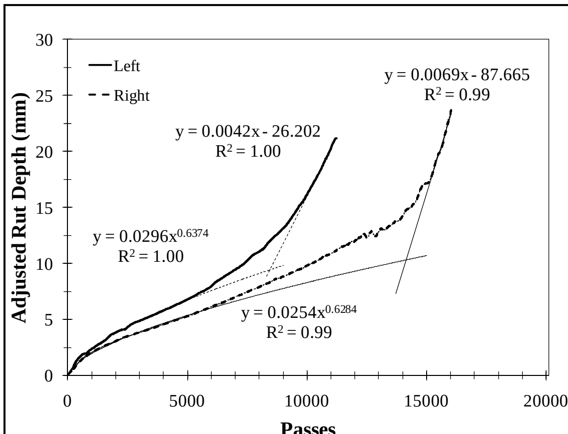
d) Photo Replicate 2

Figure A.17 PURWheel Dry Test Results for Mixture *9.5-25/RM-1*

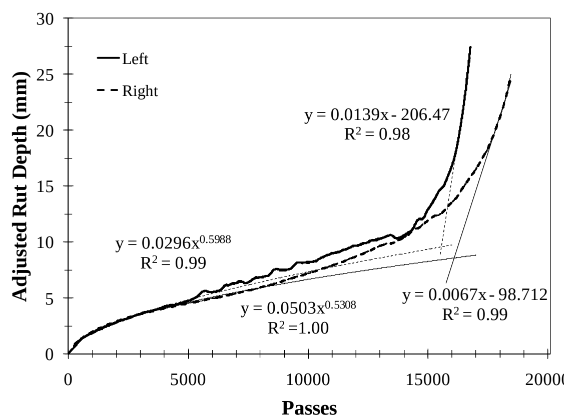
Table A.18 PURWheel Wet Test Results for Mixture *9.5-25/RM-1*

Replicate 1 (Air Voids 9.5%)				**Replicate 2 (Air Voids 9.1%)**			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	0.9	250	0.6	250	0.6	250	0.8
500	1.7	500	1.2	500	1.2	500	1.3
1000	2.4	1000	2.1	1000	1.9	1000	2.0
2000	3.8	2000	3.1	2000	2.8	2000	2.9
4000	5.8	4000	4.6	4000	4.2	4000	4.1
8000	11.1	8000	7.9	8000	6.8	6978	5.9
11232	21.2**(18.2)**[1]	12000	12.0	12000	9.7	12000	8.7
---	---	16000	23.5	16000	16.8	16000	13.5
---	---	16022	23.7 **(22.1)**[1]	16766	27.4 **(---)**[1]	18452	25.0 **(---)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C   Tire Pressure: 862 kPa   Wheel Load: 178.6 kg

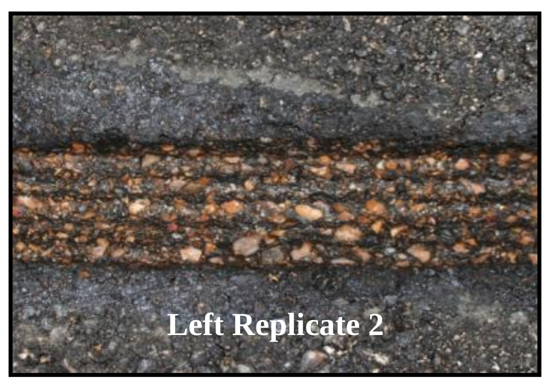


a) Data Replicate 1



b) Data Replicate 2

c) Photo Replicate 1



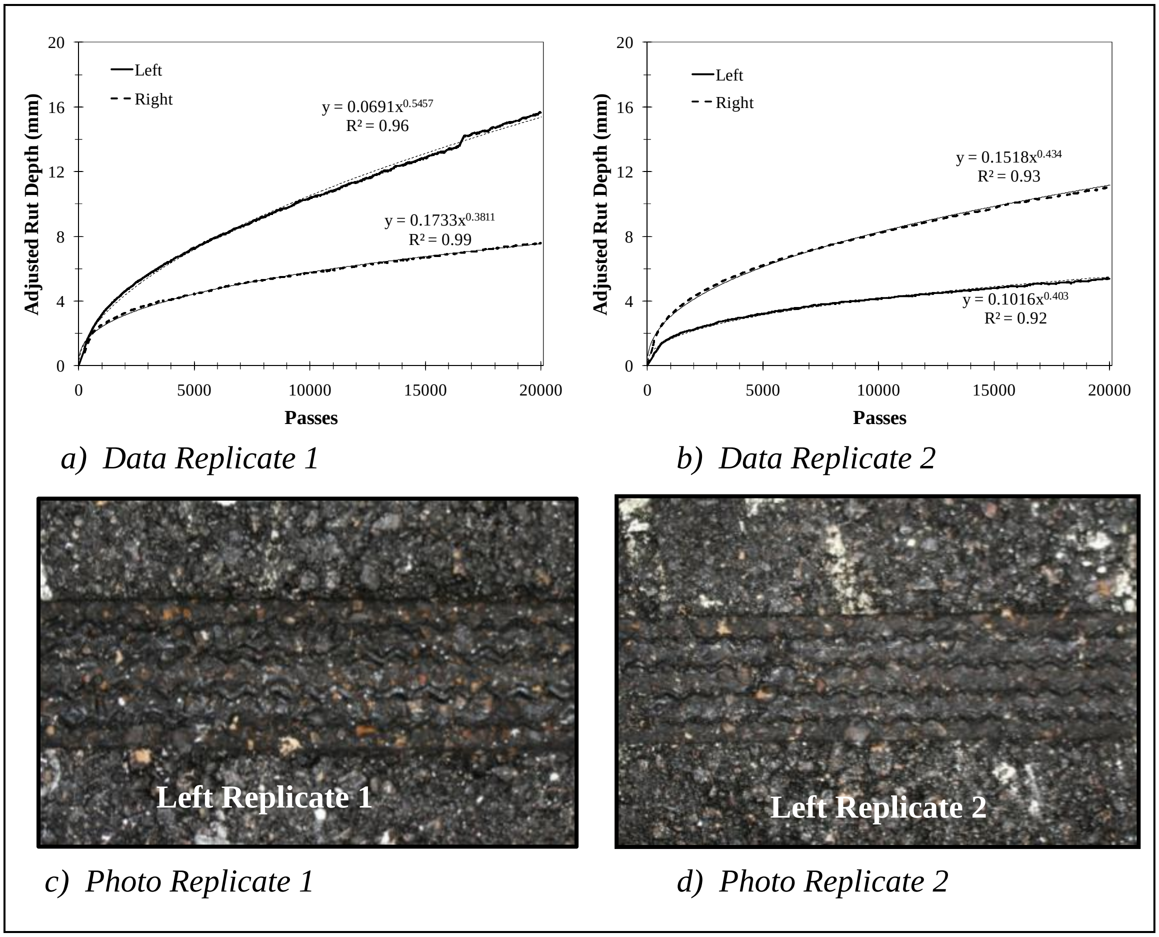
d) Photo Replicate 2

Figure A.18 PURWheel Wet Test Results for Mixture *9.5-25/RM-1*

Table A.19 PURWheel Dry Test Results for Mixture *9.5-25/RM-2*

Replicate 1 (Air Voids 7.0%)				**Replicate 2 (Air Voids 10.4%)**			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	1.1	250	0.8	250	0.6	250	1.3
500	2.0	500	1.7	500	1.2	500	2.2
1000	3.2	1000	2.5	1000	1.7	1000	3.2
2000	4.6	2000	3.3	2000	2.2	2000	4.3
4000	6.5	4000	4.1	4000	3.0	4000	5.7
8000	9.2	8000	5.3	8000	3.9	6978	7.5
12000	11.3	12000	6.2	12000	4.4	12000	8.9
16000	13.4	16000	6.9	16000	4.9	16000	10.1
20000	15.7 **(15.9)**[1]	20000	7.6 **(7.9)**[1]	20000	5.4 **(5.8)**[1]	20000	11.0 **(---)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was not measured.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



a) Data Replicate 1 *b) Data Replicate 2*

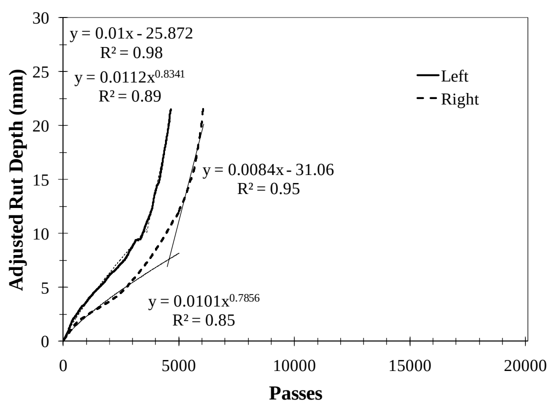
c) Photo Replicate 1 *d) Photo Replicate 2*

Figure A.19 PURWheel Dry Test Results for Mixture *9.5-25/RM-2*

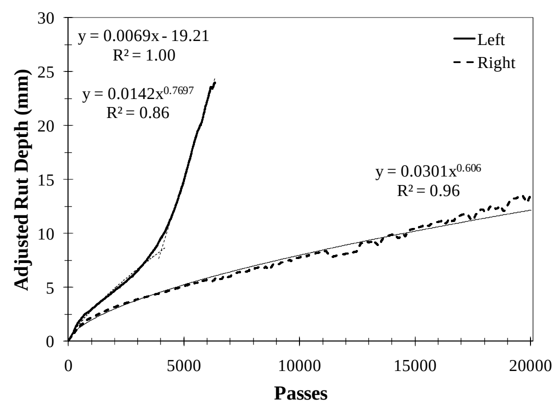
Table A.20 PURWheel Wet Test Results for Mixture *9.5-25/RM-2*

Replicate 1 (Air Voids 9.1%)				**Replicate 2 (Air Voids 8.9%)**			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	1.2	250	0.8	250	1.0	250	0.8
500	2.3	500	1.5	500	2.0	500	1.4
1000	3.7	1000	2.4	1000	2.9	1000	2.2
2000	6.1	2000	3.7	2000	4.7	2000	3.2
4000	14.0	4000	8.5	4000	9.5	4000	4.5
4660	21.5 **(20.8)**[1]	6066	22.0 **(---)**[1]	6342	24.0 **(---)**[1]	6978	6.7
---	---	---	---	---	---	12000	8.1
---	---	---	---	---	---	16000	11.1
---	---	---	---	---	---	20000	13.6 **(13.8)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



a) *Data Replicate 1*



b) *Data Replicate 2*

c) *Photo Replicate 1*

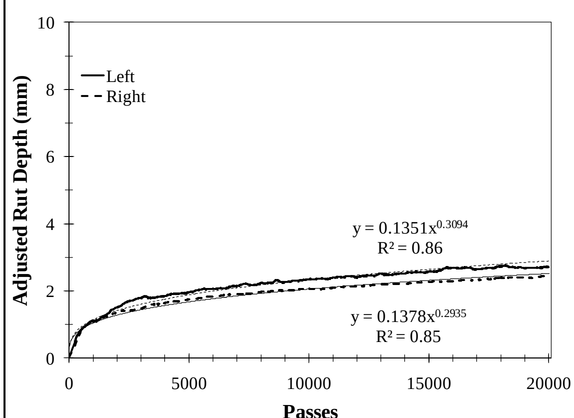
d) *Photo Replicate 2*

Figure A.20 PURWheel Wet Test Results for Mixture *9.5-25/RM-2*

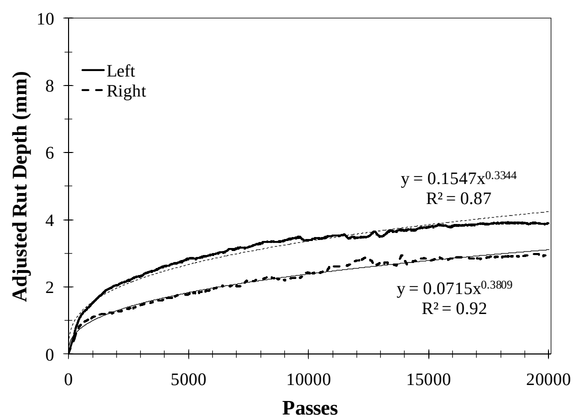
Table A.21 PURWheel Dry Test Results for Mixture *9.5-50/RM-1*

Replicate 1 (Air Voids 8.1%)				Replicate 2 (Air Voids 8.7%)			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	0.5	250	0.4	250	0.6	250	0.5
500	0.8	500	0.8	500	1.1	500	0.9
1000	1.1	1000	1.1	1000	1.5	1000	1.1
2000	1.5	2000	1.4	2000	2.1	2000	1.2
4000	1.9	4000	1.6	4000	2.6	4000	1.6
8000	2.2	8000	2.0	8000	3.3	6978	2.2
12000	2.4	12000	2.1	12000	3.5	12000	2.8
16000	2.7	16000	2.3	16000	3.8	16000	2.9
20000	2.7**(3.3)**[1]	20000	2.4 **(3.4)**[1]	20000	3.9 **(5.3)**[1]	20000	2.9 **(4.0)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



a) Data Replicate 1



b) Data Replicate 2

c) Photo Replicate 1

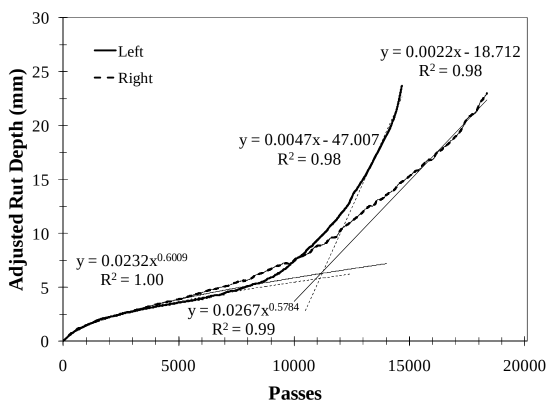
d) Photo Replicate 2

Figure A.21 PURWheel Dry Test Results for Mixture *9.5-50/RM-1*

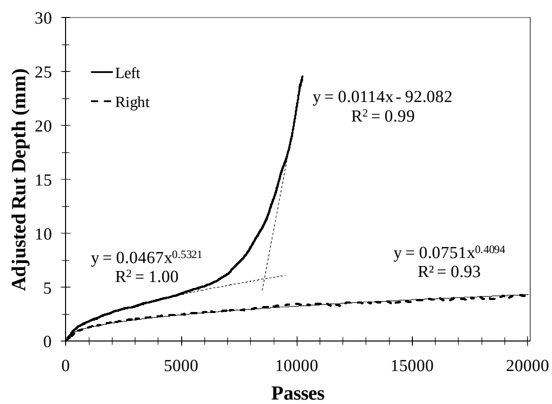
Table A.22 PURWheel Wet Test Results for Mixture *9.5-50/RM-1*

Replicate 1 (Air Voids 8.2%)				**Replicate 2 (Air Voids 8.3%)**			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	0.5	250	0.5	250	0.8	250	0.5
500	0.9	500	1.0	500	1.3	500	0.9
1000	1.5	1000	1.5	1000	1.9	1000	1.3
2000	2.3	2000	2.2	2000	2.7	2000	1.7
4000	3.2	4000	3.4	4000	3.8	4000	2.3
8000	5.1	8000	5.7	8000	8.7	6978	3.0
12000	11.7	12000	10.3	10238	24.6 **(---)**[1]	12000	3.4
14690	23.7**(---)**[1]	16000	16.8	---	---	16000	3.9
---	---	18360	23.0 **(20.2)**[1]	---	---	20000	4.3 **(4.8)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



a) *Data Replicate 1*



b) *Data Replicate 2*

c) *Photo Replicate 1*

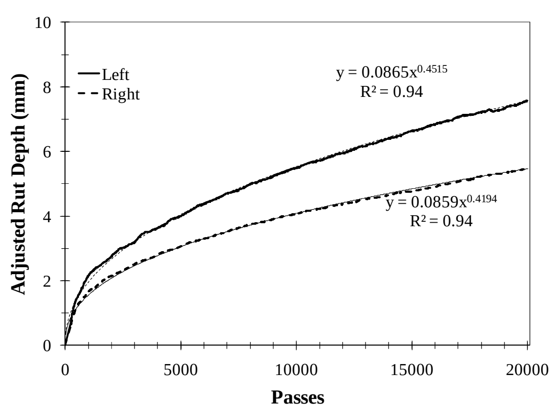
d) *Photo Replicate 2*

Figure A.22 PURWheel Wet Test Results for Mixture *9.5-50/RM-1*

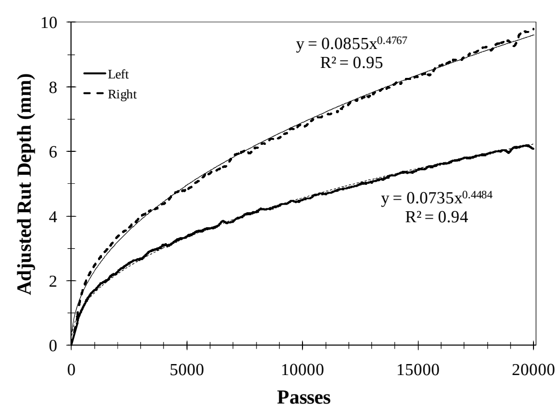
Table A.23 PURWheel Dry Test Results for Mixture *9.5-50/RM-2*

Replicate 1 (Air Voids 6.4%)				**Replicate 2 (Air Voids 8.0%)**			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	0.8	250	0.7	250	0.7	250	0.9
500	1.5	500	1.2	500	1.2	500	1.7
1000	2.2	1000	1.6	1000	1.7	1000	2.5
2000	2.8	2000	2.1	2000	2.3	2000	3.4
4000	3.6	4000	2.8	4000	3.1	4000	4.4
8000	5.0	8000	3.7	8000	4.1	6978	6.1
12000	6.0	12000	4.4	12000	4.9	12000	7.5
16000	6.8	16000	4.9	16000	5.6	16000	8.7
20000	7.6 **(9.1)**[1]	20000	5.5 **(6.3)**[1]	20000	6.1 **(7.5)**[1]	20000	9.8 **(10.1)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



a) Data Replicate 1



b) Data Replicate 2

c) Photo Replicate 1

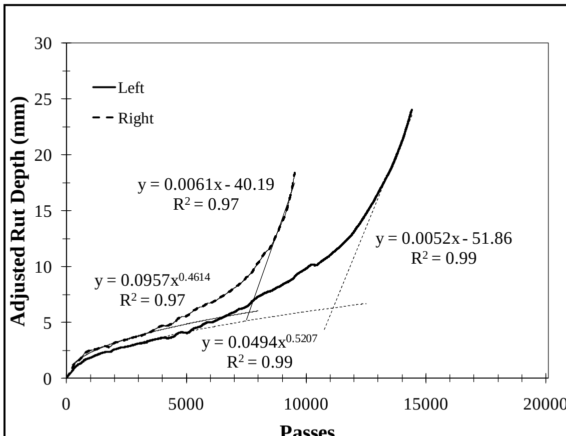
d) Photo Replicate 2

Figure A.23 PURWheel Dry Test Results for Mixture *9.5-50/RM-2*

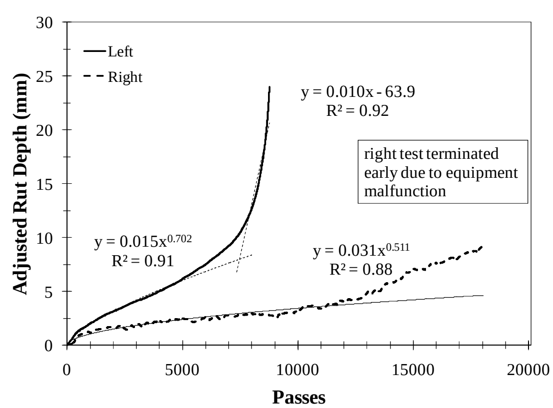
Table A.24 PURWheel Wet Test Results for Mixture *9.5-50/RM-2*

Replicate 1 (Air Voids 6.7%)				**Replicate 2 (Air Voids 7.6%)**			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	0.7	250	1.0	250	0.7	250	0.2
500	1.3	500	1.6	500	1.4	500	0.9
1000	1.9	1000	2.5	1000	2.0	1000	1.2
2000	2.6	2000	3.2	2000	3.1	2000	1.6
4000	3.6	4000	4.7	4000	5.0	4000	2.3
8000	7.3	8000	10.3	8000	12.7	6978	2.9
12000	13.1	9526	18.4 **(---)**[1]	8774	24.0 **(---)**[1]	12000	4.0
14406	24.1**(22.4)**[1]	---	---	---	---	16000	7.6
---	---	---	---	---	---	18012	9.2 **(10.7)**[1]

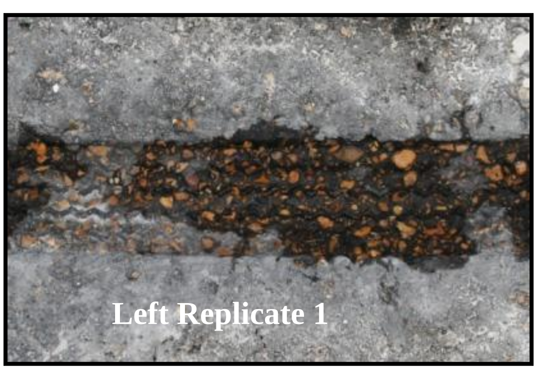
1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



a) *Data Replicate 1*



b) *Data Replicate 2*



c) *Photo Replicate 1*

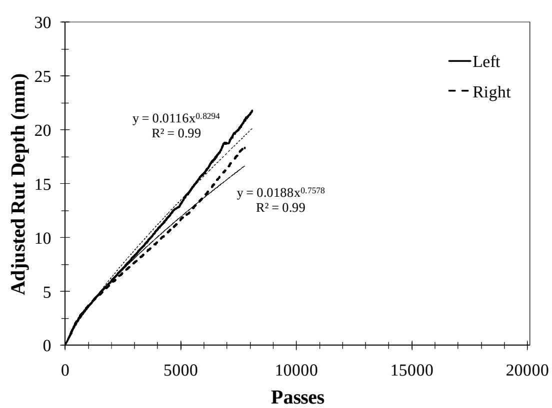
d) *Photo Replicate 2*

Figure A.24 PURWheel Wet Test Results for Mixture *9.5-50/RM-2*

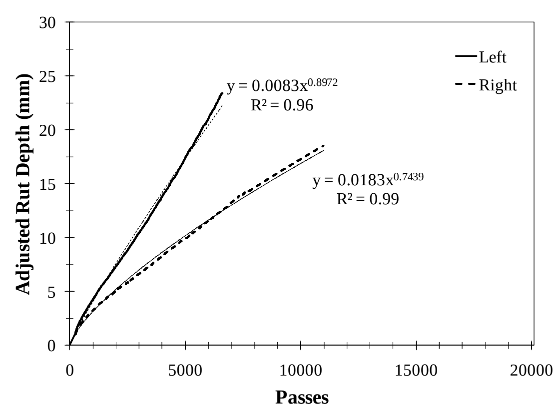
Table A.25 PURWheel Dry Test Results for Mixture *12.5-15/CM-1*

Replicate 1 (Air Voids 7.0%)				**Replicate 2 (Air Voids 7.1%)**			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	1.3	250	1.3	250	1.2	250	1.1
500	2.3	500	2.5	500	2.5	500	2.1
1000	3.8	1000	3.9	1000	4.3	1000	3.3
2000	6.1	2000	6.0	2000	7.3	2000	5.1
4000	10.9	4000	9.8	4000	13.8	4000	8.3
8000	21.6	7760	18.5 **(16.1)**[1]	6618	23.4 **(22.5)**[1]	8000	14.7
8084	21.9 **(20.9)**[1]	---	---	---	---	10982	18.5 **(18.8)**[1]
---	---	---	---	---	---	---	---
---	---	---	---	---	---	---	---

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



a) Data Replicate 1



b) Data Replicate 2

c) Photo Replicate 1

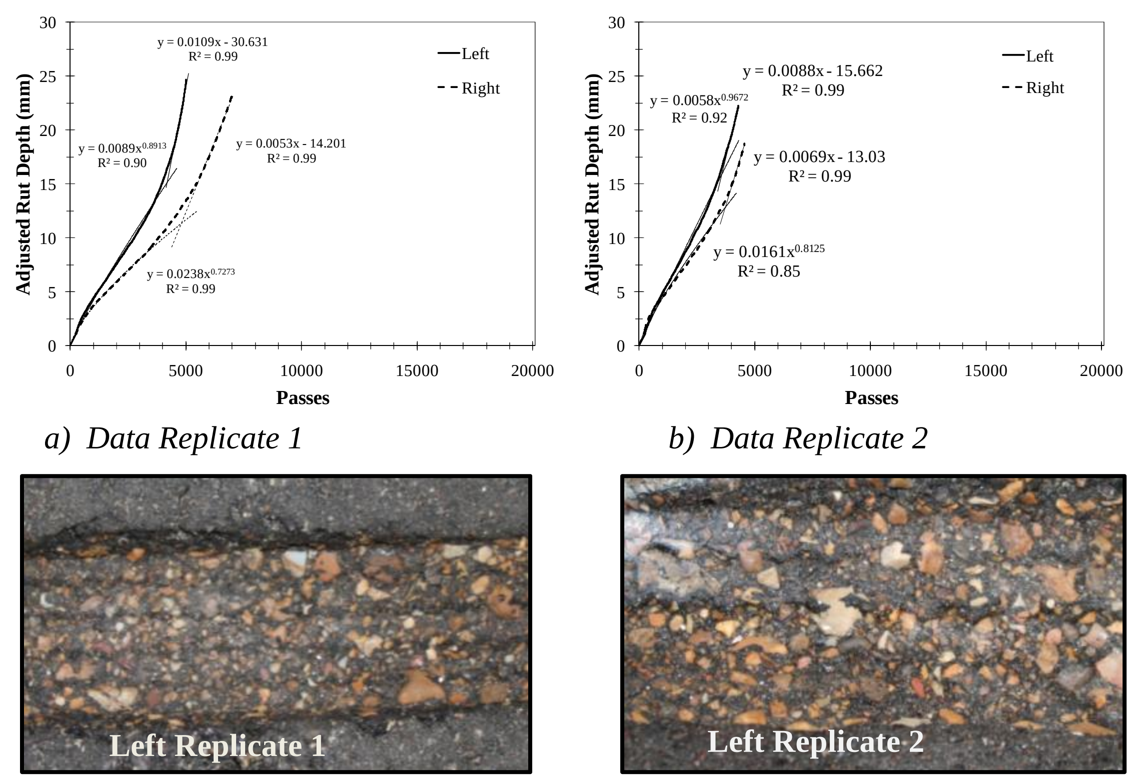
d) Photo Replicate 2

Figure A.25 PURWheel Dry Test Results for Mixture *12.5-15/CM-1*

Table A.26 PURWheel Wet Test Results for Mixture *12.5-15/CM-1*

Replicate 1 (Air Voids 6.6%)				**Replicate 2 (Air Voids 6.6%)**			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	1.2	250	1.1	250	1.2	250	1.5
500	2.6	500	2.2	500	2.7	500	2.9
1000	4.4	1000	3.7	1000	4.9	1000	4.5
2000	7.6	2000	5.9	2000	8.7	2000	7.4
4000	15.3	4000	10.5	4000	19.6	4000	14.9
5018	24.7 **(22.6)**[1]	7046	23.6 **(20.7)**[1]	4294	22.3 **(20.1)**[1]	4560	18.8 **(17.6)**[1]
---	---	---	---	---	---	---	---
---	---	---	---	---	---	---	---
---	---	---	---	---	---	---	---

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



a) Data Replicate 1 *b) Data Replicate 2*

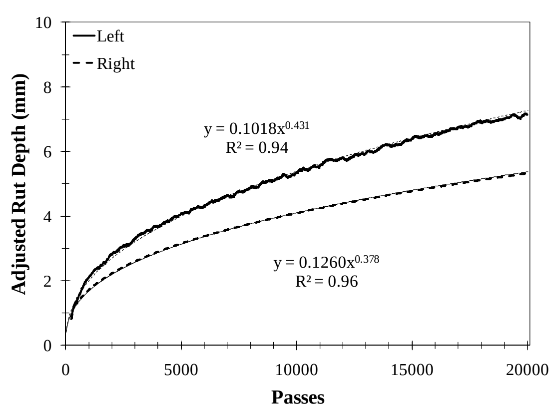
c) Photo Replicate 1 *d) Photo Replicate 2*

Figure A.26 PURWheel Wet Test Results for Mixture *12.5-15/CM-1*

Table A.27 PURWheel Dry Test Results for Mixture *12.5-15/CM-2*

Replicate 1 (Air Voids 5.9%)			
Left Specimen (mm)		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**
250	0.9	250	0.8
500	1.4	500	1.5
1000	2.1	1000	1.9
2000	2.8	2000	2.0
4000	3.7	4000	3.2
8000	4.9	8000	4.1
12000	5.8	12000	4.7
16000	6.5	16000	4.5
20000	7.1 **(8.6)**[1]	20000	5.4 **(6.8)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



a) Data Replicate 1

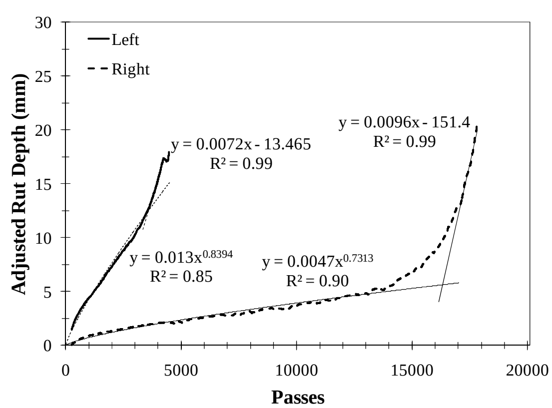
b) Photo Replicate 1

Figure A.27 PURWheel Dry Test Results for Mixture *12.5-15/CM-2*

Table A.28 PURWheel Wet Test Results for Mixture *12.5-15/CM-2*

Replicate 1 (Air Voids 10.9%)			
Left Specimen (mm)		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**
250	1.5	250	0.0
500	2.8	500	0.5
1000	4.4	1000	0.9
2000	7.3	2000	1.3
4000	15.4	4000	2.1
4476	18.0 **(14.9)**[1]	8000	3.1
---	---	12000	4.5
---	---	16000	8.8
---	---	17816	20.4 **(---)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



a) Data Replicate 1

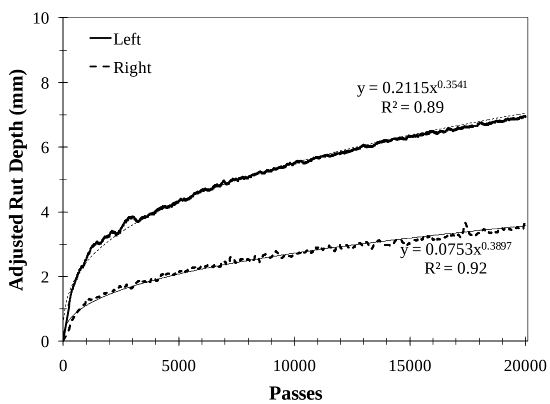
b) Photo Replicate 1

Figure A.28 PURWheel Wet Test Results for Mixture *12.5-15/CM-2*

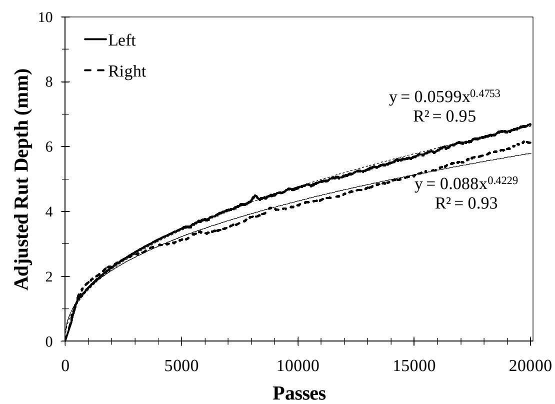
Table A.29 PURWheel Dry Test Results for Mixture *12.5-15/CM-3*

Replicate 1 (Air Voids 6.8%)				Replicate 2 (Air Voids 7.1%)			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	1.1	250	0.4	250	0.6	250	0.7
500	1.8	500	0.8	500	1.2	500	1.3
1000	2.5	1000	1.2	1000	1.9	1000	1.8
2000	3.3	2000	1.5	2000	2.7	2000	2.3
4000	4.0	4000	1.9	4000	3.1	4000	3.0
8000	5.1	8000	2.5	8000	4.3	8000	3.8
12000	5.8	12000	2.9	12000	5.1	12000	4.5
16000	6.5	16000	3.1	16000	5.9	16000	5.3
20000	7.0 **(5.8)**[1]	20000	3.7 **(5.1)**[1]	20000	6.7 **(4.6)**[1]	20000	6.1 **(9.8)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



a) Data Replicate 1



b) Data Replicate 2

c) Photo Replicate 1

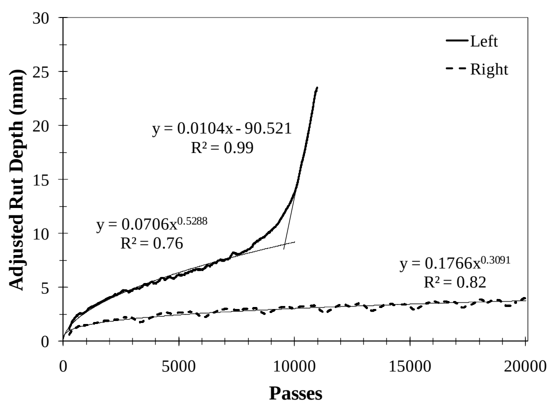
d) Photo Replicate 2

Figure A.29 PURWheel Dry Test Results for Mixture *12.5-15/CM-3*

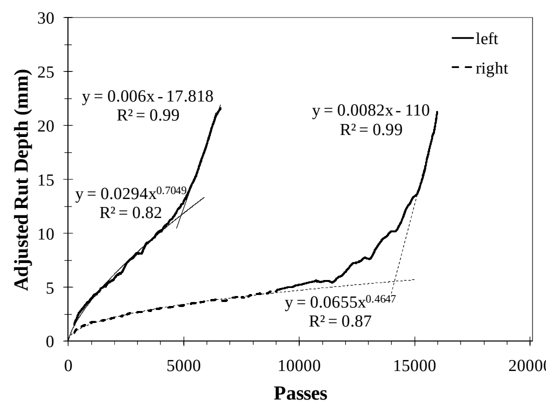
Table A.30 PURWheel Wet Test Results for Mixture *12.5-15/CM-3*

Replicate 1 (Air Voids 6.4%)				**Replicate 2 (Air Voids 6.2%)**			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	1.1	250	0.6	250	1.5	250	0.8
500	2.2	500	1.2	500	2.7	500	1.3
1000	2.8	1000	1.5	1000	4.0	1000	1.8
2000	4.1	2000	1.9	2000	5.8	2000	2.2
4000	5.4	4000	2.4	4000	10.3	4000	3.1
8000	8.5	8000	3.0	6594	21.6 **(14.5)**[1]	8000	4.3
10992	23.5 **(---)**[1]	12000	3.4	---	---	12000	6.4
---	---	16000	3.7	---	---	15976	21.3 **(---)**[1]
---	---	20000	4.0 **(5.7)**[1]	---	---	---	---

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



a) Data Replicate 1



b) Data Replicate 2

c) Photo Replicate 1

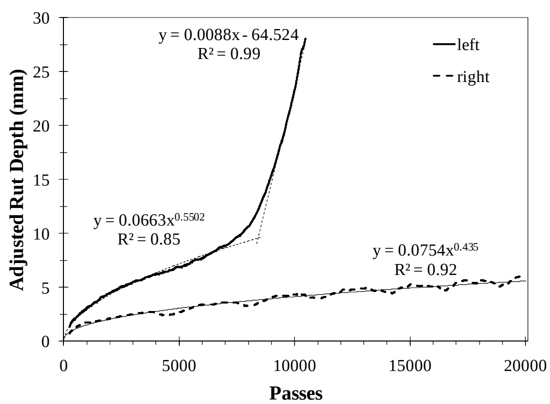
d) Photo Replicate 2

Figure A.30 PURWheel Wet Test Results for Mixture *12.5-15/CM-3*

Table A.31 PURWheel Wet Test Results for Mixture *12.5-15/CM-3*

Replicate 3 (Air Voids 7.8%)			
Left Specimen (mm)		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**
250	1.4	250	0.7
500	2.1	500	1.3
1000	3.1	1000	1.8
2000	4.5	2000	2.1
4000	6.3	4000	2.6
8000	10.6	8000	3.3
10474	28.1 **(---)**[1]	12000	4.6
---	---	16000	5.1
---	---	20000	6.1 **(9.5)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



a) Replicate 3

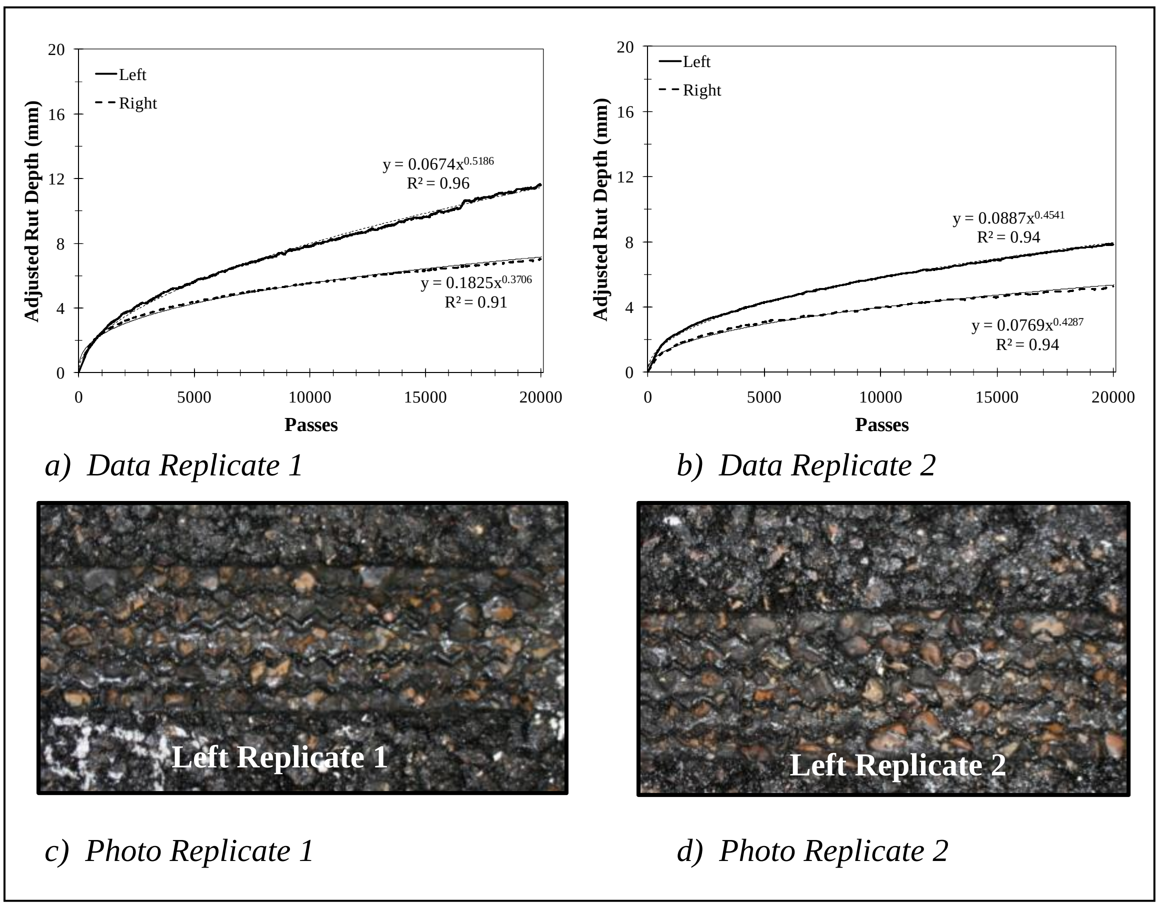
b) Photo Replicate 1

Figure A.31 PURWheel Wet Test Results for Mixture *12.5-15/CM-3*

Table A.32 PURWheel Dry Test Results for Mixture *19.0-15/CM-4*

Replicate 1 (Air Voids 6.3%)				**Replicate 2 (Air Voids 6.8%)**			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	0.9	250	1.0	250	0.8	250	0.6
500	1.6	500	1.8	500	1.5	500	1.0
1000	2.5	1000	2.4	1000	2.2	1000	1.4
2000	3.7	2000	3.2	2000	2.9	2000	2.1
4000	5.2	4000	4.1	4000	3.9	4000	2.8
8000	7.1	8000	5.1	8000	5.3	8000	3.6
12000	8.6	12000	5.9	12000	6.3	12000	4.3
16000	10.0	16000	6.5	16000	7.1	16000	4.8
20000	11.6 **(14.3)**[1]	20000	7.0 **(7.4)**[1]	20000	7.8 **(7.2)**[1]	20000	5.3 **(6.9)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



a) Data Replicate 1

b) Data Replicate 2

c) Photo Replicate 1

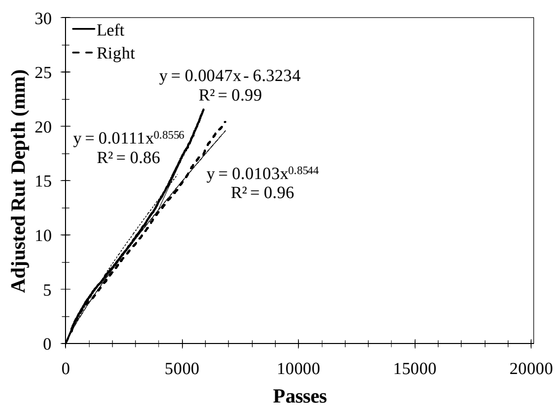
d) Photo Replicate 2

Figure A.32 PURWheel Dry Test Results for Mixture *19.0-15/CM-4*

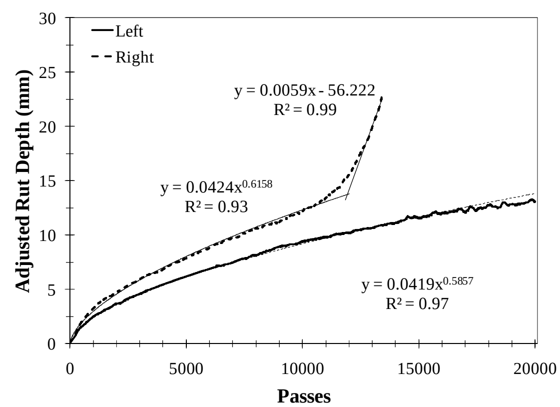
Table A.33 PURWheel Wet Test Results for Mixture *19.0-15/CM-4*

Replicate 1 (Air Voids 9.4%)				**Replicate 2 (Air Voids 5.1%)**			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	1.4	250	1.2	250	0.9	250	0.9
500	2.5	500	2.4	500	1.6	500	2.0
1000	4.3	1000	3.9	1000	2.5	1000	3.3
2000	7.0	2000	6.6	2000	3.7	2000	4.7
4000	13.1	4000	12.2	4000	5.7	4000	6.8
5920	21.5 **(18.9)**[1]	6854	20.5 **(18.2)**[1]	8000	8.2	8000	10.6
---	---	---	---	12000	10.2	12000	15.6
---	---	---	---	16000	12.0	13426	23.0 **(26.3)**[1]
---	---	---	---	20000	13.1 **(13.3)**[1]	---	---

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



a) Data Replicate 1



b) Data Replicate 2

c) Photo Replicate 1

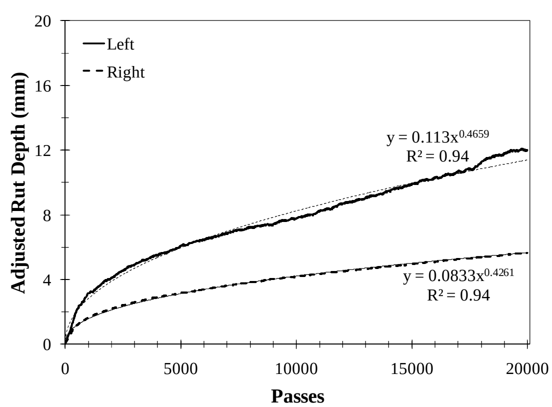
d) Photo Replicate 2

Figure A.33 PURWheel Wet Test Results for Mixture *19.0-15/CM-4*

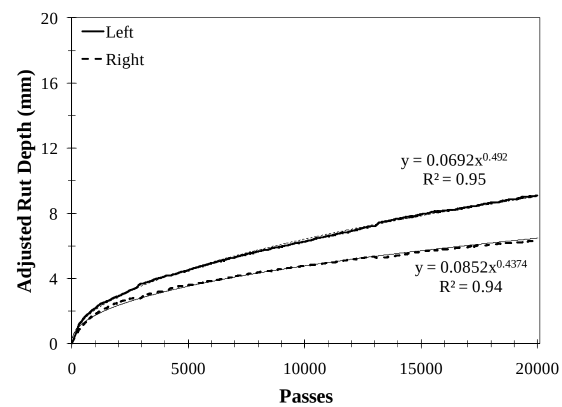
Table A.34 PURWheel Dry Test Results for Mixture *12.5-50/RM-1*

Replicate 1 (Air Voids 7.6%)				**Replicate 2 (Air Voids 5.8%)**			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	1.0	250	0.7	250	1.0	250	0.7
500	2.1	500	1.2	500	1.6	500	1.2
1000	3.2	1000	1.6	1000	2.2	1000	1.8
2000	4.1	2000	2.2	2000	2.9	2000	2.5
4000	5.5	4000	2.9	4000	4.2	4000	3.2
8000	7.2	8000	3.8	8000	5.7	8000	4.4
12000	8.7	12000	4.5	12000	6.9	12000	5.2
16000	10.3	16000	5.1	16000	8.2	16000	5.8
20000	12.0 **(13.1)**[1]	20000	5.7 **(7.3)**[1]	20000	9.1 **(9.2)**[1]	20000	6.3 **(7.2)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C   Tire Pressure: 862 kPa   Wheel Load: 178.6 kg



a) Data Replicate 1



b) Data Replicate 2

c) Photo Replicate 1

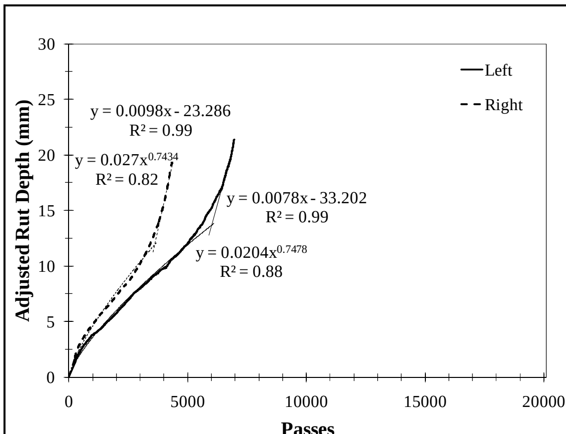
d) Photo Replicate 2

Figure A.34 PURWheel Dry Test Results for Mixture *12.5-50/RM-1*

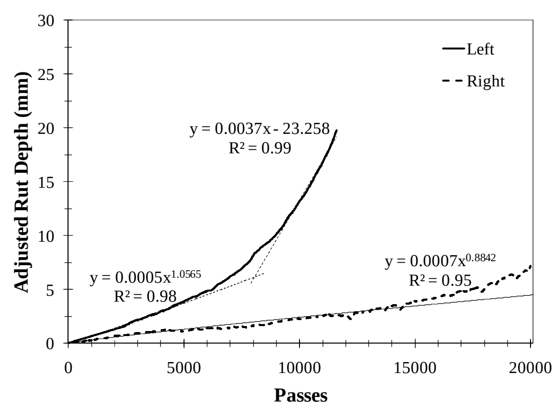
Table A.35 PURWheel Wet Test Results for Mixture *12.5-50/RM-1*

Replicate 1 (Air Voids 11.1%)				**Replicate 2 (Air Voids 7.0%)**			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	1.3	250	1.8	250	0.2	250	0.1
500	2.4	500	3.1	500	0.4	500	0.1
1000	3.8	1000	4.7	1000	0.7	1000	0.3
2000	5.8	2000	7.3	2000	1.3	2000	0.7
4000	9.8	4000	15.6	4000	3.0	4000	1.2
6956	21.4 **(20.4)**[1]	4370	19.2 **(16.2)**[1]	8000	8.2	8000	1.7
---		---		11608	19.8 **(---)**[1]	12000	2.7
---		---		---	---	16000	4.3
---		---		---	---	20000	7.2 **(13.1)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



a) *Data Replicate 1*



b) *Data Replicate 2*

c) *Photo Replicate 1*

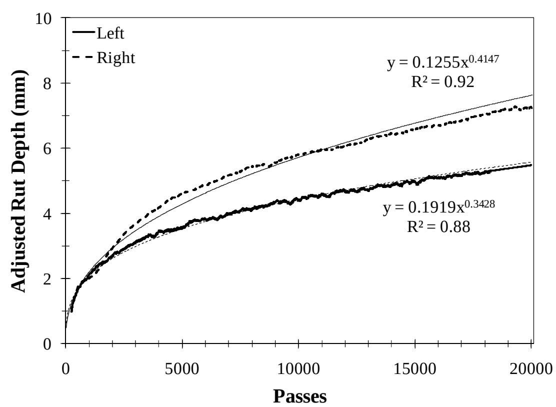
d) *Photo Replicate 2*

Figure A.35 PURWheel Wet Test Results for Mixture *12.5-50/RM-1*

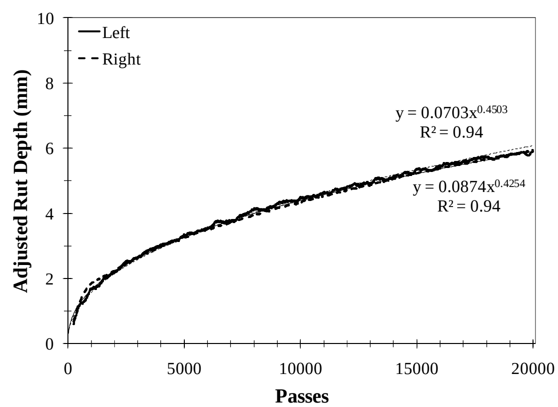
Table A.36 PURWheel Dry Test Results for Mixture *12.5-50/RM-2*

Replicate 1 (Air Voids 4.3%)				**Replicate 2 (Air Voids 9.5%)**			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	1.1	250	1.0	250	0.6	250	0.7
500	1.6	500	1.7	500	1.2	500	1.3
1000	2.1	1000	2.0	1000	1.7	1000	1.9
2000	2.7	2000	2.9	2000	2.2	2000	2.2
4000	3.5	4000	4.2	4000	3.0	4000	3.0
8000	4.1	8000	5.4	8000	4.1	8000	4.0
12000	4.7	12000	6.1	12000	4.8	12000	4.7
16000	5.1	16000	6.7	16000	5.5	16000	5.4
20000	6.5 **(6.8)**[1]	20000	7.3 **(7.4)**[1]	20000	5.9 **(7.2)**[1]	20000	5.9 **(6.3)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



a) Data Replicate 1



b) Data Replicate 2

c) Photo Replicate 1

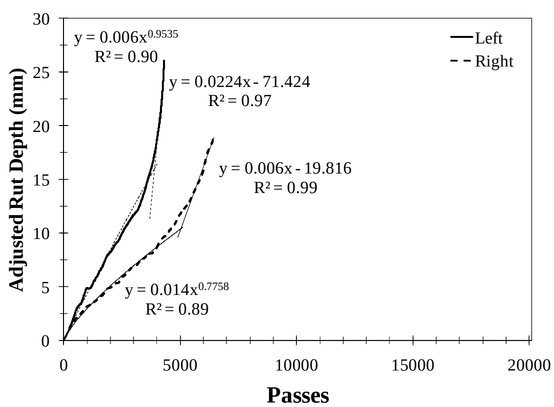
d) Photo Replicate 2

Figure A.36 PURWheel Dry Test Results for Mixture *12.5-50/RM-2*

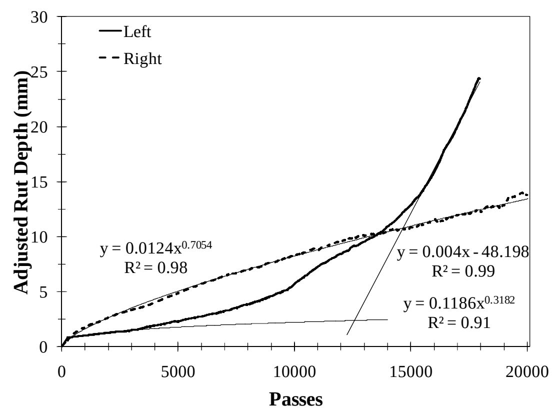
Table A.37 PURWheel Wet Test Results for Mixture *12.5-50/RM-2*

Replicate 1 (Air Voids 11.1%)				**Replicate 2 (Air Voids 6.5%)**			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	1.2	250	1.0	250	0.8	250	0.6
500	2.7	500	2.0	500	0.9	500	1.2
1000	4.9	1000	3.2	1000	1.0	1000	1.8
2000	8.2	2000	4.9	2000	1.3	2000	2.7
4000	18.7	4000	8.7	4000	1.9	4000	4.0
4318	26.1 **(---)**[1]	6454	19.1 **(21.8)**[1]	8000	3.9	8000	7.0
---	---	---	---	12000	8.4	12000	9.6
---	---	---	---	16000	15.9	16000	11.6
---	---	---	---	17960	24.4 **(26.5)**[1]	20000	13.8 **(14.0)**[1]

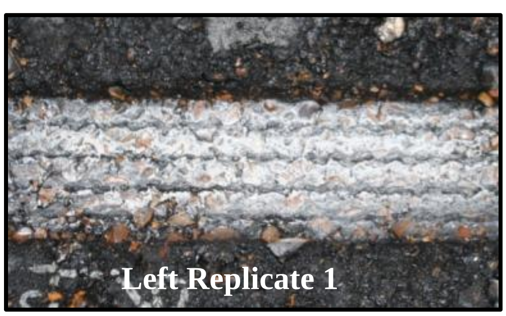
1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



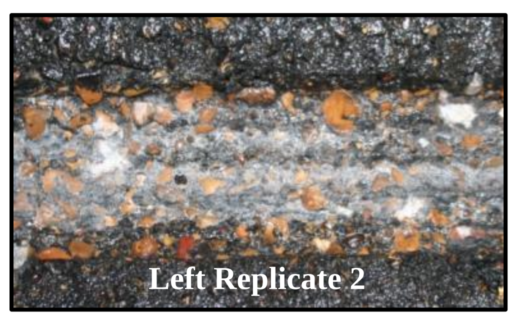
a) Data Replicate 1



b) Data Replicate 2



c) Photo Replicate 1



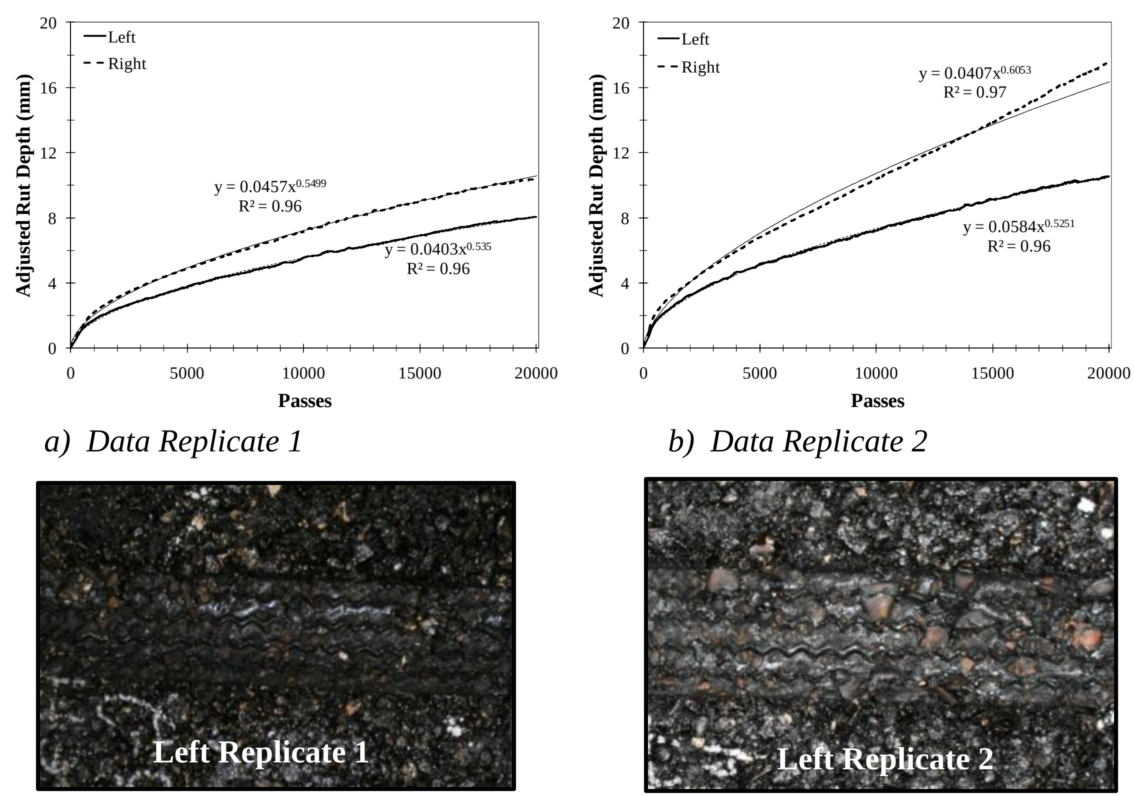
d) Photo Replicate 2

Figure A.37 PURWheel Wet Test Results for Mixture *12.5-50/RM-2*

Table A.38 PURWheel Dry Test Results for Mixture *12.5-75/RM-1*

Replicate 1 (Air Voids 9.5%)				**Replicate 2 (Air Voids 11.8%)**			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	0.6	250	0.8	250	0.9	250	1.2
500	1.2	500	1.4	500	1.6	500	2.1
1000	1.7	1000	2.2	1000	2.3	1000	3.0
2000	2.4	2000	3.1	2000	3.3	2000	4.1
4000	3.3	4000	4.4	4000	4.7	4000	6.0
8000	4.8	8000	6.3	8000	6.4	8000	9.0
12000	6.2	12000	8.1	12000	8.0	12000	11.8
16000	7.2	16000	9.4	16000	9.5	16000	14.6
20000	8.1 **(9.2)**[1]	20000	10.4 **(10.9)**[1]	20000	10.5 **(9.2)**[1]	20000	17.6 **(15.7)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



a) Data Replicate 1 *b) Data Replicate 2*

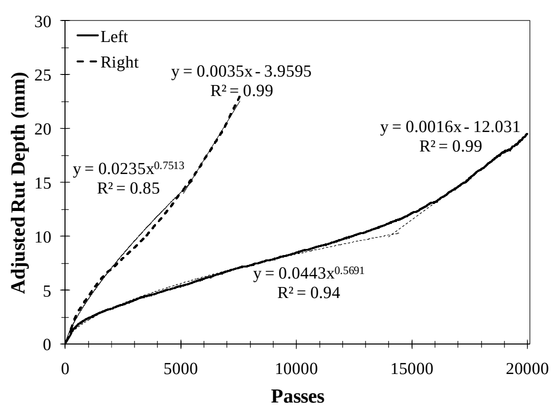
c) Photo Replicate 1 *d) Photo Replicate 2*

Figure A.38 PURWheel Dry Test Results for Mixture *12.5-75/RM-1*

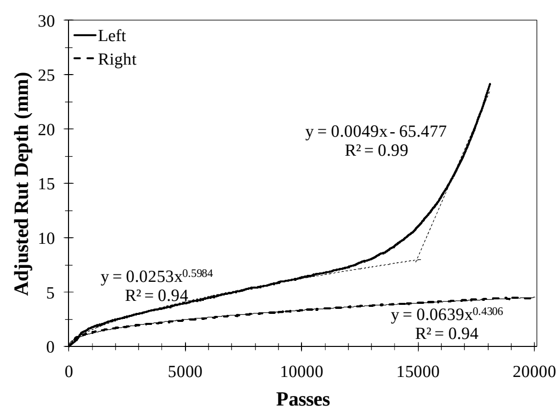
Table A.39 PURWheel Wet Test Results for Mixture *12.5-75/RM-1*

Replicate 1 (Air Voids 10.9%)				**Replicate 2 (Air Voids 10.0%)**			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	1.1	250	1.4	250	0.5	250	0.7
500	1.7	500	2.8	500	1.1	500	1.0
1000	2.4	1000	4.5	1000	1.8	1000	1.3
2000	3.3	2000	7.0	2000	2.5	2000	1.7
4000	4.8	4000	11.3	4000	3.5	4000	2.2
8000	7.3	7848	22.4 **(20.4)**[1]	8000	5.4	8000	3.0
12000	9.8	---	---	12000	7.3	12000	3.7
16000	13.2	---	---	16000	13.8	16000	4.2
20000	19.5 **(15.5)**[1]	---	---	18098	24.1 **(21.3)**[1]	20000	4.6 **(5.0)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg

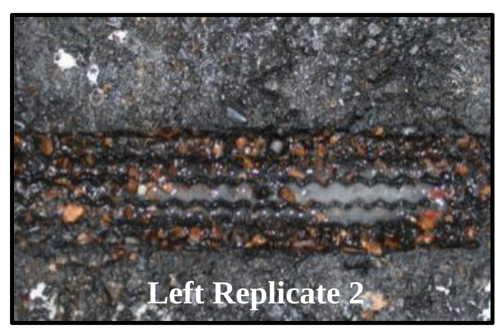


a) Data Replicate 1



b) Data Replicate 2

c) Photo Replicate 1



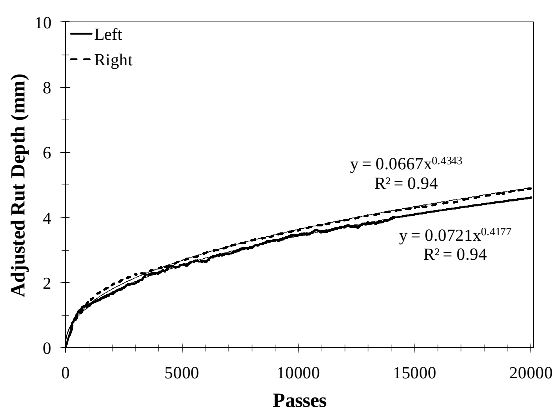
d) Photo Replicate 2

Figure A.39 PURWheel Wet Test Results for Mixture *12.5-75/RM-1*

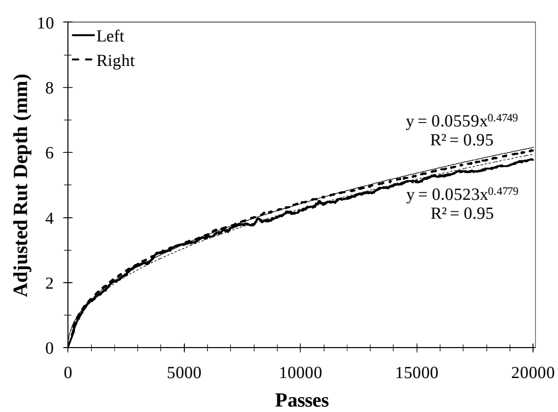
Table A.40 PURWheel Dry Test Results for Mixture *12.5-75/RM-2*

Replicate 1 (Air Voids 8.1%)				**Replicate 2 (Air Voids 9.1%)**			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	0.6	250	0.6	250	0.5	250	0.6
500	1.0	500	1.0	500	1.0	500	1.1
1000	1.3	1000	1.4	1000	1.4	1000	1.5
2000	1.7	2000	1.9	2000	2.0	2000	2.1
4000	2.3	4000	2.4	4000	2.9	4000	2.9
8000	3.1	8000	3.3	8000	3.8	8000	4.0
12000	3.7	12000	3.9	12000	4.6	12000	4.8
16000	4.0	16000	4.4	16000	5.3	16000	5.5
20000	4.3 **(4.8)**[1]	20000	4.9 **(6.0)**[1]	20000	5.8 **(7.0)**[1]	20000	6.1 **(6.2)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



a) Data Replicate 1



b) Data Replicate 2

c) Photo Replicate 1

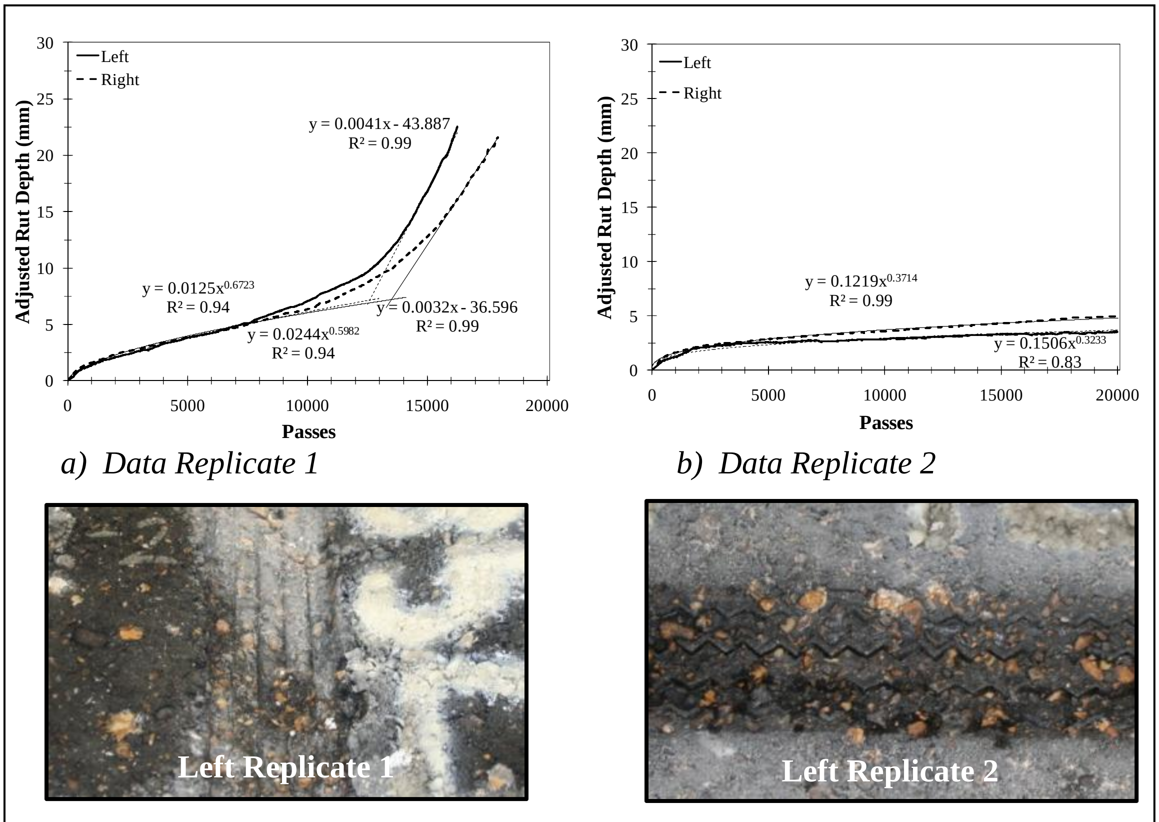
d) Photo Replicate 2

Figure A.40 PURWheel Dry Test Results for Mixture *12.5-75/RM-2*

Table A.41 PURWheel Wet Test Results for Mixture *12.5-75/RM-2*

Replicate 1 (Air Voids 7.6%)				**Replicate 2 (Air Voids 6.6%)**			
Left Specimen (mm)		**Right Specimen (mm)**		**Left Specimen (mm)**		**Right Specimen (mm)**	
Pass	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**	**Pass**	**Adj. Rut**
250	0.4	250	0.6	250	0.5	250	0.6
500	0.9	500	1.1	500	0.9	500	1.2
1000	1.4	1000	1.6	1000	1.2	1000	1.6
2000	2.1	2000	2.3	2000	2.0	2000	2.2
4000	3.3	4000	3.4	4000	2.5	4000	2.7
8000	5.5	8000	5.3	8000	2.7	8000	3.4
12000	9.0	12000	8.2	12000	3.1	12000	3.9
16000	21.0	16000	15.3	16000	3.3	16000	4.5
16248	22.5 **(---)**[1]	17940	21.6 **(17.7)**[1]	20000	3.6 **(3.5)**[1]	20000	4.4 **(5.9)**[1]

1: Value in bold in parentheses is manual measurement. If dashes are present rut was too deep to measure.
Test Temperature: 64 C Tire Pressure: 862 kPa Wheel Load: 178.6 kg



a) Data Replicate 1 *b) Data Replicate 2*

c) Photo Replicate 1 *d) Photo Replicate 2*

Figure A.41 PURWheel Wet Test Results for Mixture *12.5-75/RM-2*