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Samuel B. Cooper, Jr.
Louisiana State University and Agricultural and Mechanical College, scoope7@lsu.edu

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CHARACTERIZATION OF HMA MIXTURES CONTAINING HIGH RECYCLED ASPHALT PAVEMENT CONTENT WITH CRUMB RUBBER ADDITIVES

A Thesis

Submitted to the Graduate Faculty of the Louisiana State University and Agricultural and Mechanical College in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering

in

The Department of Civil and Environmental Engineering

By
Samuel B. Cooper, Jr.
B.S.C.E. Louisiana State University, Baton Rouge, Louisiana 1980
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ABSTRACT

As the price of petroleum and material costs escalate and pressures of maintaining the sustainability of our environment, owners must continually find methods to decrease material costs and maximize their benefits. One such method is to increase and/or begin using readily available recycled materials like reclaimed asphalt pavements (RAP) and crumb rubber (CR).

The objective of this study was to fundamentally characterize the laboratory performance of conventional HMA mixtures and mixtures containing high RAP content and waste tire crumb rubber/additives through their fundamental engineering properties.

A Superpave l9-mm nominal maximum aggregate size (NMAS) Level 2 HMA mixture meeting LADOTD specification was designed and examined. Siliceous limestone aggregates and coarse natural sand that are commonly used in Louisiana were included in this study. Comparative laboratory evaluations of a total of six mixtures were examined in this study. Three conventional mixtures that contain an unmodified asphalt cement binder and styrene-butadiene-styrene polymer modified asphalt cement meeting Louisiana specifications for PG 64-22, PG 76-22M, and PG 76-22M respectively. The fourth mixture contains no RAP, 30 mesh CR plus additives blended (wet process) with a PG 64-22 yielding a PG76-22. The fifth mixture contains 15 percent RAP and PG 76-22M asphalt cement binder. The final mixture contains 40 percent RAP, 30 mesh crumb rubber and additives blended (dry process) with a PG 64-22 asphalt cement binder. The CR and additives were introduced to the mixture at a rate of ten percent by total weight of asphalt cement binder. To evaluate performance, physical and rheological tests were evaluated on asphalt binders. In addition, hot mix asphalt mixture performance and characterization tests namely, Semi-Circular Bend, Dissipated Creep Strain Energy, Dynamic Modulus, Flow Number, and Modified Lottman test were conducted to define permanent deformation (stability) and the fatigue life (durability) of HMA mixtures considered in this study.
For the mixtures evaluated, results indicate that the addition of CR additives as a dry feed to carry rejuvenating agents is promising. The HMA mixture containing 40 percent RAP, PG 64-22, and CR additives performed similar to conventional mixtures containing PG 76-22M asphalt cement binder.
CHAPTER 1 INTRODUCTION

1.1 Background

One of the issues concerning environmental sustainability is determining how to make production, distribution, and consumption of goods and services last longer and have less impact on our ecological systems consisting of all plants, animals and micro-organisms in an area functioning together with all of the non-living physical factors of the environment. One such method of sustainability in the hot mix asphalt (HMA) industry is using recycled materials to replace a percentage of virgin materials used in the manufacturing process such as aggregates and asphalt cement binder which has a direct impact on cost and the environment.

Agencies and owners must continually find methods to decrease material costs and maximize their benefits as the price of HMA mixtures continually raise because of the increase in material costs such as aggregates and petroleum products. One such method is to increase and/or begin using readily available recycled materials like reclaimed asphalt pavements (RAP) and crumb rubber (CR).

Asphalt pavements are the most recycled product in America. A reclaimed asphalt pavement, which is commonly called RAP, is a HMA mixture containing aggregates and asphalt cement binder which has been removed and reclaimed from an existing pavement. Properly processed RAP consists of well-graded aggregates coated with asphalt cement binder.

Reports from the Federal Highway Administration (FHWA) and the United States Environmental Protection Agency (U.S. EPA) state that approximately 80 percent of removed asphalt pavements are reused as part of new roads, roadbeds, shoulders, and embankments.

Another available recycled material is crumb rubber. Crumb rubber or ground rubber is typically defined as scrap tire rubber that has been reduced to a particle size of 3/8-inch or less. There are approximately 290 million scrap tires generated per year in the United States. In 2004...
there were approximately 275 million scrap tires in stockpiles in the United States. About 27 million scrap tires are estimated to be disposed in landfills annually resulting in major disposal costs, environmental risks related to pests and insect growths that promote the outbreak of diseases, and fires that are hard to distinguish and cause contamination of the soil. The three largest markets for the use of recycled scrap tires are tire derived fuel, civil engineering applications (subgrade fill, embankments, septic system drain fields, etc.), and ground rubber i.e. crumb rubber applications/rubberized asphalt. Currently there are 30 million tons of scrap tires that are recycled into crumb rubber each year.

The use of crumb-rubber modifier (CRM) in hot-mix asphalt mixtures can be traced back to the 1840s when natural rubber was introduced into bitumen to increase its engineering performance. Since the 1960s, researchers and engineers have used shredded automobile tires in hot-mix asphalt (HMA) mixtures for pavements. The processes of applying crumb-rubber in asphalt mixtures can be divided into two broad categories: a dry process and a wet process. In the dry process, crumb rubber is added to the aggregate before the asphalt binder is charged into the mixture. In the wet process, asphalt cement is pre-blended with the rubber at high temperature (177 – 210 °C) and specific blending conditions.

The Louisiana Department of Transportation and Development (LADOTD) initiated a research project to evaluate different procedures of CRM applications used in HMA mixtures in 1994 in which the long-term pavement performance of the CRM asphalt pavements was compared to that of the control sections built with conventional asphalt mixtures. It is reported that the conventional mixtures exhibited higher laboratory strength characteristics (indirect tensile strength) than the CRM mixtures. Also, the pavement sections constructed with CRM asphalt mixtures showed overall better performance indices (rut depth, fatigue cracks, and international roughness index numbers) than the corresponding control sections.
In the 1970s, states and paving contractors began making extensive use of RAP in HMA pavements. The use of RAP results in cost savings and an environmentally positive method of recycling. From 1987 through 1993, several research projects were carried out to develop the Superpave method of design based on performance based HMA designs under the Strategic Highway Research Program (SHRP). One of the distinct shortcomings of this mix design method was that there was no provision for the use of RAP in the mix design process. It was noted that the effect of aged binder from RAP on the performance properties of the virgin binder depends upon the level of RAP used in the HMA mixture. When the percentage of RAP used in the HMA is low (10 – 20 percent) the effect on the asphalt binder properties is minimal. As RAP percentage is increased (greater than 20 percent) in the HMA the aged binder from RAP blends with the virgin asphalt binder in sufficient quantity to significantly affect the asphalt binder performance. The blending of old, hardened asphalt binders from RAP with a virgin asphalt binder will typically result in an asphalt binder that is harder than the virgin asphalt binder properties used. Usually this binder hardening is counteracted by adding a softer virgin asphalt binder and letting the RAP asphalt binder stiffen the softer binder to achieve a blended asphalt binder of desired properties. In addition to the use of softer asphalt binders, recycling agents or rejuvenators are also used to soften the hardened RAP asphalt binders.

Shen et al. [2007] studied the effects of rejuvenating agents on Superpave HMA mixtures containing RAP in South Carolina. There were three objectives of this study: first, to evaluate the properties of Superpave mixtures containing various RAP sources and a rejuvenator and then comparing to those of the recycled Superpave mixtures utilizing a softer asphalt cement binder; second, to investigate the use of blending charts of aged asphalt cement binders and a rejuvenator for determining the rejuvenator contents for the design of Superpave mixtures containing RAP; and third, to evaluate the properties of Superpave mixtures, virgin mixes and mixtures containing
RAP. The HMA mixtures were evaluated in terms of volumetrics, indirect tensile strength (ITS), and rutting potential using the asphalt pavement analyzer (APA). It was reported that for the mixtures tested, ITS and APA properties of the RAP HMA mixtures containing rejuvenator were better than those that contained only the softer binder. The use of a rejuvenator in lieu of a softer binder would allow 10 percent more RAP in the HMA mixture to be used and there was good relationships between the measured performance parameters and rejuvenator contents which were determined by the blending charts developed from the extracted aged binders.

This study explored the use of the absorption properties of crumb rubber to carry asphalt cement binder components (light ends) that are typically lost during oxidation of HMA pavements as a dry feed component in the making of hot mix asphalt mixtures. No available literature was found indicating that this method has been evaluated. Laboratory mechanistic performance and mixture characterization evaluations and analysis were performed to determine the effects of crumb rubber additives, and RAP on the HMA mixtures’ performance.

1.2 Problem Statement

Asphalt cement prices, like gasoline and crude oil, are at an all time high with no relief in-site. With the hot mix asphalt (HMA) mixtures prices continuously climbing, highway agencies and owners are continually searching for methods to decrease material costs and maximize their benefits with no compromise in performance. One such method is to develop innovative technology to incorporate waste and recycled materials, such as crumb rubber from waste tires and RAP, in HMA mixtures. RAP is currently allowed for use in limited percentages within HMA layers. As HMA pavements age over time the asphalt binders become hardened and oxidized causing premature cracking in pavements. Thus, the current limiting factor in increasing the percentages of RAP is the excessive stiffness of the resulting HMA mixture. Rejuvenating additives are often used to “soften” the asphalt cement binder of RAP materials.
Therefore, the incorporation of these additives into the HMA mixture will enable the use of higher percentages of RAP in the finished product. Furthermore, the absorption properties of waste tire crumb rubber can be used to carry those additives to revitalize the properties of the aged binders.

A limited comparative laboratory mechanistic performance evaluation of conventional HMA mixtures and mixtures that contain waste tire crumb rubber, additives, and RAP was conducted. HMA mixture characterization in terms of fatigue cracking, moisture susceptibility, and rutting were analyzed and evaluated to determine the effects of the crumb rubber, additives, and RAP on the HMA mixtures’ performance.

1.3 Objectives

The objective of this study was to fundamentally characterize the laboratory performance of conventional HMA mixtures and mixtures containing high RAP content and waste tire crumb rubber/additives through their fundamental engineering properties. The aforementioned mixtures can be used in either wearing or binder course layers.

1.4 Scope

A Superpave 19-mm nominal maximum aggregate size (NMAS) Level 2 HMA mixture meeting LADOTD specification was designed and examined. Siliceous limestone aggregates and coarse natural sand that are commonly used in Louisiana were included in this study. Comparative laboratory evaluations of a total of six mixtures were examined in this study. Three mixtures are classified as conventional mixtures that contain an unmodified asphalt cement binder and mixtures containing styrene-butadiene-styrene polymer modified asphalt cement meeting Louisiana specifications for PG 64-22, PG 76-22M, and PG 76-22M respectively. The fourth mixture contains no RAP, 30 mesh crumb rubber (CR) plus additives blended (wet process) with a PG 64-22 asphalt cement binder which yields a PG 76-22. The fifth mixture
contains 15 percent RAP and PG 76-22M asphalt cement binder. The final mixture contains 40 percent RAP, 30 mesh crumb rubber and additives blended (dry process) with a PG 64-22 asphalt cement binder. The CR and additives were introduced to the mixture at a rate of ten percent by total weight of asphalt cement binder. To evaluate performance, physical and rheological tests were evaluated on asphalt binders and hot mix asphalt mixtures (HMA). In addition to asphalt cement rheology characterization, HMA mixture performance and characterization tests namely, Semi-Circular Bend (SCB) test, Dissipated Creep Strain Energy (DCSE) test, Simple Performance Tests (Dynamic Modulus, E*, Flow Number, F_N), and Modified Lottman test were conducted to define permanent deformation (stability) and the fatigue life (durability) of HMA mixtures considered in this study. Triplicate samples were used for each test.

1.5 Outline

This thesis is divided into five distinct chapters including this introductory chapter (Chapter 1). Chapter summaries of contents of the remaining chapters are as provided below:

Chapter 2 provides a literature review on the Superpave mix design method and its distress criterion, use of Reclaimed Asphalt Pavements (RAP) and Crumb Rubber (CR), and subsequent research studies on the effect of RAP and CR on HMA.

Chapter 3 describes the materials and material properties evaluated in this study. In addition, the experimental laboratory performance test methodologies used to characterize and analyze HMA mixture performance are discussed.

Chapter 4 discusses the HMA characterization test results and related statistical analysis of the mixtures and asphalt binders evaluated in this study.

Finally, Chapter 5 is the summary and conclusion section for the research work conducted under this study.
CHAPTER 2 LITERATURE REVIEW

2.1 State of the Knowledge

The term "sustainability" is relatively new concept which has already proved useful. Sustainability relates to “how to make human economic systems (production, distribution and consumption of goods and services in a particular society) last longer and have less impact on ecological systems consisting of all plants, animals and micro-organisms in an area functioning together with all of the non-living physical factors of the environment, and particularly relates to concern over major global problems such as climate change and oil depletion” [Wikipedia, 2008]. One such method of sustainability in the HMA industry is using recycled materials to replace a percentage of virgin materials used in the manufacturing process such as aggregates and asphalt cement binder which has a direct impact on cost and the environment.

Agencies and owners must continually find methods to decrease material costs and maximize their benefits as the price of hot mix asphalt (HMA) mixtures continually rise because of the increase in material costs such as aggregates and petroleum products. One such method is to increase and/or begin using readily available recycled materials like RAP and crumb rubber. Therefore it is only logical to try to devise methods to increase the usage of these type products without sacrificing HMA mixture performance. Recycled materials such as crumb rubber made from scrap tires and reclaimed asphalt pavements (RAP) are available to the HMA industry.

Reclaimed Asphalt Pavements, which is commonly called RAP, is a HMA mixture containing aggregates and asphalt cement binder which has been removed and reclaimed from an existing roadway. RAP is generated during rehabilitation/reconstruction of existing HMA roadways, or from utility cuts across an existing HMA roadway which was necessary to obtain access to underground utilities. When RAP is properly processed, such as crushed and screened, the RAP will consist of well-graded aggregates coated with asphalt cement binder. During
reconstruction and/or rehabilitation HMA pavements are typically removed by milling machines. This process is commonly referred to as cold planning. The depth of HMA removal by milling varies by the type of reconstruction required. The reconstruction/rehabilitation process may require the removal of an existing wearing course mixture or may require full-depth removal of the entire HMA structure. As the existing HMA pavement is being milled, the RAP is deposited directly into haul trucks and then delivered to a HMA hot mix plant for processing. Full-depth removal involves milling the existing HMA structure in several passes depending on the existing depth of the structure or by ripping and breaking the pavement into large pieces using rippers on a bull dozer or by use of a backhoe. When the RAP is broken in large pieces, the broken material is picked up by a front-end loader or backhoe and then loaded into haul trucks and is usually transported to a HMA hot mix plant for processing. At the HMA hot mix plant, the RAP is processed by crushing, screening and then conveyed and stockpiled [Turner-Fairbank, 2006].

It is reported that asphalt pavements are America’s most recycled product. More than 73 million tons of reclaimed asphalt pavements are recycled each year as compared to the combined total of 40 million tons of recycled paper, glass, aluminum, and plastic. Reports from the Federal Highway Administration (FHWA) and the United States Environmental Protection Agency (U.S. EPA) state that approximately 80 percent of removed asphalt pavements are reused as part of new roads, roadbeds, shoulders, and embankments [NAPA, 2008].

In 1994 there were approximately 800 million scrap tires disposed of in stockpiles. Since then there has been millions of scrap tires removed by aggressive cleanup by state scrap tire management programs. It has been reported that in 2004 there were approximately 275 million scrap tires remaining in stockpiles in the United States. There were approximately 290 million scrap tires generated in 2003, which is the typical yearly rate seen in the United States. About 27
millions scrap tires are estimated to be disposed in landfills annually resulting in major disposal costs, environmental risks related to pests and insect growths that promote the outbreak of diseases, and fires that are hard to distinguish and cause contamination of the soil. As of 2003, there existed markets for the use of 80 percent of the scrap tires which relates to 233 million scrap tires out of 290 million scrap tires available. The three largest markets for the use of recycled scrap tires are tire derived fuel, civil engineering applications (subgrade fill, embankments, septic system drain fields, etc.), and ground rubber i.e. crumb rubber applications/rubberized asphalt. Currently there are 30 million tons of scrap tires that are recycled into crumb rubber each year [US EPA, 2008]. The transportation industry still has the potential to escalate its use of disposed scrap tires by increasing the use of crumb rubber in specialty mixes such as crumb rubber modified (CRM) HMA mixtures.

Crumb rubber or ground rubber is typically defined as scrap tire rubber that has been reduced to a particle size of 3/8-inch or less. Crumb rubber is described or measured by the mesh screen or sieve size through which it passes in the production process. A 30 mesh means there are 30 openings, per linear inch of screen. There are three processes that are typically used in the making the crumb rubber from scrap tires. “First, the scrap tire is reduced to 2 ½-inch to 4-inch size shreds by a slow speed “shear” shredder or shredders. Second, the shreds go through two or three successively narrower blade shredders to further reduce the shreds to 3/8-inch or less. Finally, the particles are processed to even smaller mesh sizes by using cracking or grinding rolling mills.” The final mesh size of the crumb rubber product is determined by the number of passes through the mill. Other than shredding, there are other methods for processing scrap tires into crumb rubber: First there are cryogenic systems which utilize sub-zero temperatures to freeze the tires. Then the frozen tires are shattered using a hammer mill which makes it easy to separate the rubber from the steel and fabric. A second alternative method is to use ambient
systems which operate at room temperature and literally tear the tire material apart. During the
process, screens and gravity separators are used to remove steel, non-ferrous metals, sand and
other unwanted materials, and aspiration equipment is used to remove fibers. One scrap
passenger tire can yield between ten to twelve pounds of crumb rubber product [TNRCC, 1999].

The processes of applying crumb-rubber in asphalt mixtures can be divided into two
broad categories: a dry process and a wet process. In the dry process, crumb rubber is added to
the aggregate before the asphalt binder is charged into the mixture. In the wet process, asphalt
cement is pre-blended with the rubber at high temperature (177 – 210 °C) and specific blending
conditions. Crumb rubber particles in the dry process are normally coarser than those in the wet
process and are considered as part of the aggregate gradations (called “rubber-filler”) whereas, in
the wet process, crumb rubber is reacted with asphalt binders (called “asphalt-rubber”). In the
wet process, crumb rubber is mixed with asphalt binder at high temperature and is allowed to
swell by absorption of the asphalt oil components to form a gel-like material [Heitzman, 1992].
The extent of the swelling process depends on the mixing temperature, the size of the crumb
rubber particles, and the concentration of rubber in the blend [Jensen et al., 2006]. Researchers
have noted that if these variables are not selected properly, the rubber may depolymerize causing
a negative impact on the properties of the blend [Chehovits et al., 1993]. Common dry process
methods include the PlusRide™, chunk rubber, and generic dry. Common wet process methods
include the Arizona, McDonald, Ecoflex, and Rouse continuous blending methods [Heitzman,
1992].

The use of crumb-rubber modifier (CRM) in hot-mix asphalt mixtures can be traced back
to the 1840s when natural rubber was introduced into bitumen to increase its engineering
performance [Heitzman, 1992]. The use of ground rubber from scrap tires has long been
supported by environmental and government agencies to reduce the disposal problem associated
with waste tires. Since the 1960s, researchers and engineers have used shredded automobile tires in hot-mix asphalt (HMA) mixtures for pavements.

In the 1960s, Charles H. McDonald pioneered the development of the wet process (or reacted) crumb rubber modified asphalt cement binders in the United States. In 1963, McDonald first used CRM asphalt cement binders for a patching material in which he termed the operation as a "band-aid" repair technique in Phoenix, Arizona. The CRM asphalt binder was spray applied using an asphalt distributor and then covered with a "localized chip seal" placed by hand over a small pavement area. The first "large area" spray application was performed in 1967 which became known as stress-absorbing membranes (SAM). In 1972, Arizona DOT placed its first stress–absorbing membrane interlayer (SAMI) as part of a project to evaluate techniques to reduce reflection cracking. Arizona placed its first HMA mixture containing CRM asphalt cement in 1975. Arizona DOT currently uses CRM asphalt binders in SAMIs, gap-graded HMA mixtures, and in open-graded friction courses which is now the most popular use of CRM binders [Hicks et al., 2000].

Not until the late 1980s did the use of recycled tire crumb rubber in HMA mixtures become popular. In 1991, the Intermodal Surface Transportation Efficiency Act (ISTEA) specified that all asphalt pavement projects funded by federal agencies must use certain percentages of scrap tires [Public Law 1991, FHWA 1993]. Although this mandate was later suspended from the ISTEA legislation, it has greatly encouraged the research and application of CRM asphalt in HMA pavement.

The National Cooperative Highway Research Programs (NCHRP) “Synthesis of Highway Practice 198 – Uses of Recycled Rubber Tires in Highways” provides a comprehensive review of the use of recycled rubber tires in highways based on a review of nearly 500 references
and on information recorded from state highway agencies’ responses to a 1991 survey of current practices [Epps, 1994].

The Florida Department of Transportation (FLDOT) constructed three HMA mixture demonstration projects that utilized CRM wet processes in 1989 for the purpose of evaluating the short term field performance and constructability of these mixtures. It was necessary to construct these projects so that the FLDOT could develop specifications and procedures for CRM use. The mixtures evaluated were two fine-graded and an open-graded Friction Course mixture type. For this study minus No. 80 mesh crumb rubber was pre-blended (“reacted” or “digested”) with the asphalt cement binder prior to its incorporation with the aggregates. They concluded that the addition of CRM would increase asphalt film thickness, binder resiliency, viscosity, and shear strength. It was further reported that with the use of CRM the FLDOT was able to increase the asphalt binder content of the mixtures because of the stiffening effect it had on the asphalt cement binder. By increasing the asphalt content Florida DOT anticipates increased durability of these type mixtures [Page, 1989].

From 1990 to 1993 Virginia DOT constructed pavements containing CRM asphalt mixtures. The objective was to familiarize the Virginia Department of Transportation and contractors personnel with the construction process and to compare the performance of different types of mixes containing ground tire rubber. Four test sections (Dense graded surface mixes, a gap-graded surface mix, and a base mix, stress-absorbing membrane interlayer) using asphalt rubber hot mix were placed in Virginia utilizing two wet processes, McDonald and Rouse, and then pavement performance was compared to that of conventional asphalt mixtures [Maupin, 1996]. The McDonald process focuses on reacted asphalt cement/CRM binder in which the time required to “react” these materials is dependent on the size of the crumb rubber particles used in the blending process. The Rouse process blends 180-micron (80 mesh) sieve CRM with an
asphalt cement binder utilizing continuous blending procedures [Heitzman et al., 1992]. It is reported that the mixes containing asphalt rubber performed at least as well as conventional mixes. In Virginia mixes, the inclusion of asphalt rubber in HMA pavements increased construction costs by 50 to 100 percent as compared to the cost of conventional mixes [Maupin, 1996].

Troy et al. [1996] conducted research on CRM pavements in Nevada. The objective of the study was to test and evaluate CRM binders blended by the wet process using the Superpave performance grading system binder protocols and its applicability to CRM binders. In addition the CRM HMA mix design was conducted using the Hveem procedure. They concluded that the conventional sample geometry in Superpave binder test protocols cannot be used to test the CRM binders and that the Hveem compaction is inadequate for mixtures containing CRM binders. It was further concluded that the Superpave binder testing protocols would not work for CRM binders containing coarse rubber particles. It was recommended that the plate and cup system be used for asphalt cement binders blended with crumb rubber. It was further concluded that the plate and cup system could not replace the bending beam rheometer for low-temperature testing. In addition, a modified Hveem mix design procedure was developed when CRM mixtures are used.

The Louisiana Department of Transportation and Development (LADOTD) initiated a research project to evaluate different procedures of CRM applications used in HMA mixtures in 1994 in which the long-term pavement performance of the CRM asphalt pavements was compared to that of the control sections built with conventional asphalt mixtures [LTRC, 1996]. There were eight CRM applications evaluated in this study as follows:

- Arizona wet process incorporated into a gap-graded mixture;
- Arizona wet process incorporated into a stress absorbing membrane interlayer (SAMI);
• Arizona wet process incorporated into an open-graded friction course (OGFC);
• PlusRide™ dry process utilizing a gap-graded aggregate structure;
• Rouse powdered rubber wet process incorporated into a typical dense-graded mixture;
• A terminal-blended material formulated by Neste Wright in a dense-graded mixture;
• Rouse dry-powdered rubber process blended into a dense-graded aggregate structure;
• Generic CRM dry process incorporated into a gap-graded mixture.

Huang et al. [2002] evaluated conventional and CRM asphalt mixtures through laboratory engineering performance tests such as indirect tensile strength (ITS) and indirect tensile resilient modulus (\(M_R\)) tests. Marshall Stability and Flow tests were also conducted during the mixture design. Huang et al. also compared field performance through the pavement structural non-destructive test using DYNAFLECT and long-term pavement performance measurement, such as roadway core density, International Roughness Index (IRI), rutting, and fatigue cracking. The conventional mixtures exhibited higher laboratory strength characteristics than the CRM mixtures. However, the pavement sections constructed with CRM asphalt mixtures showed overall better performance indices (rut depth, fatigue cracks, and international roughness index numbers) than the corresponding control sections [Huang et al., 2002]. In addition, Cooper et al. [2007] evaluated the long-term field performance (10 years) as it relates to random cracking, International Roughness Index, and rutting of asphalt pavements constructed with these eight different CRM applications as opposed to the control sections built with conventional HMA mixtures. It is reported that the “pavement sections constructed with CRM asphalt mixtures showed overall better field performance indices (rut depth, random cracks, and IRI numbers) than corresponding control sections. Both CRM modified, wet and dry, hot mix asphalt (HMA) mix types are performing equally well, if not better, than the conventional mix types evaluated”.

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LADOTD conducted a study in 2004 to evaluate and characterize HMA mixtures that used recycled polymer-modified asphalt pavements as one of the mixture components [Mohammad et al. 2004]. “The objectives of this research were to (1) analyze the properties of field-aged polymer modified asphalt cement (PMAC) relative to Pressure Aging Vessel (PAV) with aged PMAC; (2) examine the compatibility and feasibility of blending reclaimed PMAC with virgin PMAC based on chemical component analysis methods and Superpave binder specification; and (3) evaluate the fatigue and permanent deformation properties of asphalt mixtures containing various percentages of laboratory-aged and/or field-extracted PMACs based on laboratory fundamental engineering tests.” The scope of this study was to develop extraction techniques necessary for the removal of the aged asphalt cement binder from the aggregate components of the HMA mixture. Also the extraction technique would allow for the separation of the polymer additive component from the asphalt cement binder. Asphalt cement binder testing, analysis and Superpave characterization included (1) differential scanning calorimetric (DSC) measurement, (2) Fourier transform infrared (FTIR) measurement, (3) gel permeation chromatograph (GPC) measurement, (4) rotational viscosity measurement, (5) dynamic shear modulus and phase angle measurement, (6) beam stiffness and creep slope measurement. In addition, a 19 mm nominal maximum aggregate size (NMAS) high volume HMA mixture that is commonly used by LADOTD was designed using virgin PMAC, meeting LADOTD PAC-40HG and PG 70-22M specifications, and then blended with varying percentages (0, 20, 40 and 60 percent) of Reclaimed Polymer Modified Asphalt Cement (RPMAC) and virgin aggregates. To characterize the HMA mixtures on both lab-aged and field-aged RPMAC mixtures being evaluated a series of fundamental engineering tests were utilized. These tests included the frequency sweep at constant height (FSCH), repeated shear at constant height (RSCH), simple shear at constant height (SSCH), indirect tensile strength and strain (ITS), indirect tensile
modulus ($M_r$), semi-circular fracture, beam fatigue, and asphalt pavement analyzer (APA) tests. It is reported that as the percentage of RPMAC binder in mixtures increased, the rutting resistance increased and the fatigue resistance decreased. The asphalt cement binder that was extracted from field cores revealed that the binder was quite brittle at low temperatures as measured by the force ductility and bending beam tests. In addition, extracted RPMAC binder was blended with the virgin PMAC and analyzed. It is reported that the resultant blends had much stiffer properties than those of lab-aged PMAC, which indicates that the Pressure Aging Vessel (PAV) procedure did not predict the field aging of PMAC binders. It was stated that the HMA mixture containing 60 percent RPMAC exhibited better fatigue life than those mixtures with 20 and 40 percent RPMAC [Mohammad et al. 2004].

In the 1970s, states and paving contractors began making extensive use of RAP in HMA pavements. The use of RAP results in cost savings and an environmentally positive method of recycling. Properly designed HMA containing RAP can perform as well as HMA prepared with 100 percent virgin materials [McDaniel et al., 2001]. From 1987 through 1993, several research projects were carried out to develop the Superpave method of design based on performance based HMA designs under the Strategic Highway Research Program (SHRP). One of the distinct shortcomings of this mix design method was no provision for the use of RAP in the mix design process. This shortcoming hindered the use of RAP in HMA mixtures by agencies that had adopted the Superpave mix design process. In order to temporarily remedy this situation, interim guidelines were developed by a Superpave Mixtures Expert Task Group based on their experience. It was noted that the effect of aged binder from RAP on the performance properties of the virgin binder depends upon the level of RAP used in the HMA mixture. When the percentage of RAP used in the HMA is low (10 – 20 percent) the effect on the asphalt binder properties is minimal. At these low percentages, RAP affects the mix volumetrics and
performance through gradation because RAP acts like a “black rock”. As RAP percentage in the
HMA is increased (greater than 20 percent) the aged binder from RAP blends with the virgin
asphalt binder in sufficient quantity to significantly affect the asphalt binder performance
[McDaniel et al., 2001]. McDaniel et al. [2001], as part of NCHRP Project 9-12, were given the
task of developing guidelines for the use of RAP in HMA mixtures. RAP materials from three
states (Florida, Connecticut, and Arizona) yielded recovered RAP asphalt binders of different
stiffness properties in combination with two virgin asphalt binders at RAP contents of 10 and 40
percent. Mixtures properties were evaluated using the Superpave shear tests (AASHTO TP7 -
Simple Shear Test at Constant Height) at high temperatures and indirect tensile creep and
strength tests (AASHTO TP9 - Standard Test Method for Determining the Creep Compliance
and Strength of Hot Mix Asphalt (HMA) Using the Indirect Tensile Test Device) for low
temperature properties. The findings confirmed current practice that low amounts of RAP,
typically 10 to 20 percent, can be used without determining the recovered asphalt binder
properties. This is because there is not enough of the old, hardened RAP asphalt binder
contribution to the final asphalt cement binder blend to change the properties of the asphalt
binder, and the RAP accounts as an aggregate component of the aggregate. When more than 20
percent RAP is used in a HMA mixture, recovery and testing of its binder is recommended,
along with blending charts to determine what performance grade of virgin asphalt binder should
be used in the HMA mixture design. The blending of old, hardened asphalt binders from RAP
with a virgin asphalt binder will typically result in an asphalt binder that is harder than the virgin
asphalt binder properties used. Usually this binder hardening is counteracted by adding a softer
virgin asphalt binder and letting the RAP asphalt binder stiffen the softer binder to achieve a
blended asphalt binder of desired properties. In addition to the use of softer asphalt binders,
recycling agents are also used to soften the hardened RAP asphalt binders. The recommended binder selection guidelines for RAP mixtures are as follows [McDaniel et al., 2001]:

- Less than 20 percent RAP used – no change in asphalt binder selection.
- Between 20 – 30 percent RAP used – select one grade softer virgin asphalt binder than normally used (e.g. select a Performance Grade (PG) 58-28 in lieu of a PG 64-22).
- Greater than 30 percent RAP – Develop and use recommendations from blending charts.

Softening of hardened RAP binders when high percentages of RAP content (greater than 20 percent) are used in a HMA mixture is typically achieved by adding rejuvenating agents. The use of rejuvenators changes the composition, physical properties, and performance properties of the rejuvenated aged asphalt binders in RAP [Shen et al. 2007]. Rejuvenators are used to recover the original properties of the aged binders and then reconstitute the chemical compositions of the aged binders that were lost due to the aging and oxidation process over time. An asphalt binder that experiences aging of oxidation has a lower concentration of more reactive components, nitrogen base plus first acidaffins and a higher concentration of less reactive components such as paraffines plus second acidaffins [Shen et al., 2007].

Many crumb rubber modified (CRM) asphalt pavements used in the past are becoming prime candidates for recycling. Shen et.al. [2007] studied the effects of rejuvenating agents on CRM modified binders by characterizing blended laboratory-aged CRM asphalt binders and rejuvenating agents using gel permeation chromatography (GPC). Results of the study indicated that the compositional changes of the asphalt binder blends with varying percentages of RAP or rejuvenating agents is reflected in the GPC test results. It was shown that the large molecular size (LMS) of the blends decreases as the small molecular size (SMS) increases as the percentage of rejuvenators used increased regardless of the type of aged binders or rejuvenating agents. As a result, empirical prediction models were developed for Superpave binder properties...
for viscosity and high-failure temperature using LMS and SMS. It is stated that the predicted values from these models show a high correlation with viscosity and the high-failure temperature of asphalt binders [Shen et al., 2007].

Shen et al. [2007] studied the effects of rejuvenating agents on Superpave HMA mixtures containing RAP in South Carolina. There were three objectives of this study: first, to evaluate the properties of Superpave mixtures containing various RAP sources and a rejuvenator and then comparing to those of the recycled Superpave mixtures utilizing a softer asphalt cement binder; second, to investigate the use of blending charts of aged asphalt cement binders and a rejuvenator for determining the rejuvenator contents for the design of Superpave mixtures containing RAP; and third, to evaluate the properties of virgin Superpave mixtures and Superpave mixtures containing RAP to ascertain the possibility of incorporating RAP into Superpave mixtures. Two RAPs typically used in South Carolina were incorporated into a 9.5 mm nominal maximum size Superpave mixtures containing either a rejuvenator or a softer binder (control mixture). The HMA mixtures were evaluated in terms of volumetrics, indirect tensile strength (ITS), and rutting potential using the asphalt pavement analyzer (APA). The rejuvenator content was determined from the blending charts of RAP binders containing the rejuvenator. Twelve Superpave mixtures were designed, 10 containing RAP and two with virgin materials. It was reported that for the mixtures tested, ITS and APA properties of the RAP HMA mixtures containing rejuvenator were better than those that contained only the softer binder. In addition, by using a rejuvenator in lieu of a softer binder you could use 10 percent more RAP in the HMA mixture. It was further reported that there were good relationships between the measured performance parameters and rejuvenator contents utilized which were determined by the blending charts developed from the extracted aged binders making it possible to determine the design rejuvenator contents necessary for recycling RAP [Shen et al., 2007].
CHAPTER 3 METHODOLOGY

3.1 Test Factorial Design

Six HMA mixtures were considered in this study. Table 3.1 presents a summary of the test factorial considered.

Table 3.1 Test Factorial

<table>
<thead>
<tr>
<th>MIX TYPE</th>
<th>Mixture Variables</th>
<th>Modified Lottman</th>
<th>DCSE</th>
<th>E*</th>
<th>Fn</th>
<th>Jc</th>
</tr>
</thead>
<tbody>
<tr>
<td>19 mm NMAS Superpave Level 2</td>
<td>% RAP CRM/Additives Uncond. Cond. Aged Unaged</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Conventional PG 64-22</td>
<td>0 ---- 3 3 3 3 3 9</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Conventional PG 70-22M</td>
<td>0 ---- 3 3 3 3 3 9</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Conventional PG 76-22M</td>
<td>0 ---- 3 3 3 3 3 9</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CRM/additives</td>
<td>0 9% 3 3 3 3 3 9</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RAP</td>
<td>15 ---- 3 3 3 3 3 9</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>RAP, CRM/additives</td>
<td>40 10% 3 3 3 3 3 9</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TOTAL</td>
<td>18 18 18 18 18 54</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

For this study, mixture designations and their descriptions are as follows:

- 64CO: HMA Mixture/PG 64-22, Conventional
- 70CO: HMA Mixture/PG 70-22M, Conventional
- 76CO: HMA Mixture/PG 76-22M, Conventional
- 76CRM: HMA Mixture/PG 76-22, Crumb Rubber Modified (Wet Blend) PG 64-22
- 76RAP15: HMA Mixture/PG 76-22M + 15% RAP (No CR Additive)
- 64RAP40: HMA Mixture/PG 64-22 +40% RAP + CR Additives

3.2 Hot Mix Asphalt Mixture Design Development

A Superpave 19-mm nominal maximum aggregate size (NMAS) Level 2 HMA mixture meeting LADOTD specification ($N_{\text{initial}} = 8\text{-}, N_{\text{design}} = 100\text{-}, N_{\text{final}} = 160\text{-gyrations}$), was designed according to AASHTO TP28, “Standard Practice for Designing Superpave HMA” and
Section 502 of the 2006 Louisiana Standard Specifications for Roads and Bridges [Louisiana, 2000]. Specifically, the optimum asphalt cement content was determined based on volumetric (VTM = 2.5 - 4.5 percent, VMA ≥ 12%, VFA = 68% -78%) and densification (%G_{mm} at N_{initial} ≤ 89, %G_{mm} at N_{final} ≤ 98) requirements. It is noted that the aggregate structure for all the mixtures considered are similar (i.e., the aggregate proportions for the blend selected will be adjusted to allow for the addition of RAP). Siliceous limestone aggregates and coarse natural sand that are commonly used in Louisiana were included in this study. The aggregate gradation for mixtures evaluated in this study is represented graphically in the curves shown in figure 3.1 where, control mix represents conventional mixtures that did not contain RAP, whereas, 76RAP15 and 64RAP 40 mixtures contained RAP.

![Figure 3.1 Aggregate Gradation Curves](image)

The job mix formula for all mixtures considered in this study is summarized in table 3.2. The design optimum asphalt cement binder content for the mixtures indicated is similar.
<table>
<thead>
<tr>
<th>Mixture Designation</th>
<th>64CO</th>
<th>70CO</th>
<th>76CO</th>
<th>76CRM</th>
<th>76RAP15</th>
<th>64RAP40</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mix Type</td>
<td>19.0 mm (3/4 in.) Superpave</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Aggregate Blend</td>
<td>#67 LS</td>
<td>37%</td>
<td>37%</td>
<td>37%</td>
<td>37%</td>
<td>38.5%</td>
</tr>
<tr>
<td></td>
<td>#78 LS</td>
<td>25%</td>
<td>25%</td>
<td>25%</td>
<td>25%</td>
<td>24.5%</td>
</tr>
<tr>
<td></td>
<td>#11 LS</td>
<td>29%</td>
<td>29%</td>
<td>29%</td>
<td>29%</td>
<td>14%</td>
</tr>
<tr>
<td></td>
<td>CS</td>
<td>9%</td>
<td>9%</td>
<td>9%</td>
<td>9%</td>
<td>8%</td>
</tr>
<tr>
<td></td>
<td>RAP</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>15%</td>
</tr>
<tr>
<td></td>
<td>CR</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Binder type</td>
<td>PG 64-22</td>
<td>PG 70-22M</td>
<td>PG 76-22M</td>
<td>PG 76-22 CRM</td>
<td>PG 76-22M</td>
<td>PG 64-22</td>
</tr>
<tr>
<td>% G_{\text{m}}$ at N_{\text{ini}}$</td>
<td>87.0</td>
<td>87.0</td>
<td>87.0</td>
<td>86.9</td>
<td>87.7</td>
<td>87.6</td>
</tr>
<tr>
<td>% G_{\text{m}}$ at N_{\text{Max}}$</td>
<td>97.6</td>
<td>97.6</td>
<td>97.6</td>
<td>97.5</td>
<td>97.3</td>
<td>98.0</td>
</tr>
<tr>
<td>Binder content, %</td>
<td>4.0</td>
<td>4.0</td>
<td>4.0</td>
<td>4.0</td>
<td>4.1</td>
<td>4.0</td>
</tr>
<tr>
<td>Design air void, %</td>
<td>3.7</td>
<td>3.7</td>
<td>3.7</td>
<td>4.2</td>
<td>3.9</td>
<td>3.4</td>
</tr>
<tr>
<td>VMA, %</td>
<td>13</td>
<td>13</td>
<td>13</td>
<td>12</td>
<td>13</td>
<td>12</td>
</tr>
<tr>
<td>VFA, %</td>
<td>68</td>
<td>68</td>
<td>68</td>
<td>66</td>
<td>71</td>
<td>72</td>
</tr>
<tr>
<td>Metric (U. S.) Sieve</td>
<td>Composite Gradation Blend</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>37.5 mm (1½ in.)</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>25.0 mm (1 in.)</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>19.0 mm (3/4 in.)</td>
<td>98</td>
<td>98</td>
<td>98</td>
<td>98</td>
<td>95</td>
<td>95</td>
</tr>
<tr>
<td>12.5 mm (1/2 in.)</td>
<td>77</td>
<td>77</td>
<td>77</td>
<td>77</td>
<td>77</td>
<td>79</td>
</tr>
<tr>
<td>9.5 mm (3/8 in.)</td>
<td>61</td>
<td>61</td>
<td>61</td>
<td>61</td>
<td>60</td>
<td>61</td>
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<tr>
<td>4.75 mm (No. 4)</td>
<td>41</td>
<td>41</td>
<td>41</td>
<td>41</td>
<td>37</td>
<td>37</td>
</tr>
<tr>
<td>2.36 mm (No. 8)</td>
<td>29</td>
<td>29</td>
<td>29</td>
<td>29</td>
<td>28</td>
<td>27</td>
</tr>
<tr>
<td>1.18 mm (No. 16)</td>
<td>21</td>
<td>21</td>
<td>21</td>
<td>21</td>
<td>21</td>
<td>19</td>
</tr>
<tr>
<td>0.600 mm (No. 30)</td>
<td>15</td>
<td>15</td>
<td>15</td>
<td>15</td>
<td>16</td>
<td>15</td>
</tr>
<tr>
<td>0.300 mm (No. 50)</td>
<td>8</td>
<td>8</td>
<td>8</td>
<td>8</td>
<td>9</td>
<td>9</td>
</tr>
<tr>
<td>0.150 mm (No. 100)</td>
<td>6</td>
<td>6</td>
<td>6</td>
<td>6</td>
<td>6</td>
<td>6</td>
</tr>
<tr>
<td>0.075 mm (No. 200)</td>
<td>4.6</td>
<td>4.6</td>
<td>4.6</td>
<td>4.6</td>
<td>4.6</td>
<td>4.5</td>
</tr>
</tbody>
</table>

Note: N/A: Not Applicable, LS: Limestone, CR: Crumb Rubber, CS: Coarse Sand
64CO: HMA Mixture/PG 64-22, Conventional
70CO: HMA Mixture/PG 70-22M, Conventional
76CO: HMA Mixture/PG 76-22M, Conventional
76CRM: HMA Mixture/PG 76-22, Crumb Rubber Modified (Wet Blend) PG 64-22
76RAP15: HMA Mixture/PG 76-22M + 15% RAP (NO CR Additive)
64RAP40: HMA Mixture/PG 64-22 +40% RAP + CR Additives
3.3 Aggregate Tests

Aggregates from each source were tested to determine aggregate properties. The test items include coarse aggregate angularity, fine aggregate angularity (FAA), flat and elongated particles, gradation analysis, and sand equivalency.

For the mixtures considered in this study, reclaimed asphalt pavement (RAP), siliceous limestone aggregates (#67 Limestone, #78 Limestone, and #11 Limestone), and coarse sand typically used in Louisiana were included in this study. To determine the aggregate gradation from each source a washed sieve analysis was performed on aggregates in accordance with AASHTO T 27 “Standard Method of Test for Sieve Analysis of Fine and Coarse Aggregates”. The gradation analysis results of these aggregates obtained from sieve analysis are presented in Appendix A of this document. In addition the measured aggregate consensus properties for the materials used in this study can be found in Appendix A.

Additionally, the #67 Limestone and #78 Limestone aggregates were sieved and materials retained on the 3/4”, 1/2”, 3/8”, No. 4 sieves and passing No. 4 sieves were stored in separate containers. For blending the high RAP content (40 percent) HMA mixture the RAP aggregate was fractionated between the plus 8 and minus 8 sieves and stored in separate containers. The RAP did not require fractionation at the lower percentage (15 percent) evaluated in this study. Separating the aggregates into various sizes was needed so that the required aggregate blend gradations could be batched directly from individual sized fractions for the desired HMA mix design. This method allowed for consistent replication of the HMA mixtures’ composite aggregate gradation because each sieve size batch weight were mixed at the exact proportions needed for the hot mix job mix formula.
3.4 Asphalt Binder Tests

Asphalt cement binders are one of man’s oldest known engineering materials. The rheological properties of an asphalt cement binder can affect an HMA pavement's performance. An asphalt cement binder’s rheological properties change during the production of an HMA mixture and as the AC ages over time due to oxidation and environmental influences. Pavement distresses may result if these changes are not properly addressed before production of a HMA mixture. Some of the specific types of pavement distresses that are contributed to by the rheological properties of an asphalt cement binder are raveling, cracking, stripping, and rutting. To assure that an asphalt cement binder meets criteria to reduce and/or prevent pavement distresses due to changes in its rheological properties necessitates testing of the asphalt cements binder properties. Therefore specifications were developed to minimize an asphalt cement binder’s contribution for durability, rutting, fatigue cracking, and low temperature cracking.

Asphalt cement binders (virgin binder, RAP binder, and RAP with CR additives) were tested and characterized according to AASHTO PP6, “Practice for Grading or Verifying the Performance Grade of an Asphalt Binder” in order to determine the effect of the CRM/additives on asphalt cements considered in this study.

The asphalt binders included in this study (PG 64-22, PG 70-22M, PG 76-22M, and PG76-22CRM) were tested and characterized according to the “Louisiana Department of Transportation and Development Performance Graded Asphalt Cement” specification [LADOTD, 2006], table 3.3. The asphalt cement binders rheological properties were measured on unaged binders in accordance with the American Association of State Highway Transportation Officials (AASHTO) test methods. The Rolling Thin Film Oven (RTFO) test was performed in accordance with AASHTO T 240-06 “Standard Method of Test for Effect of Heat and Air on a Moving Film of Asphalt (Rolling Thin-Film Oven Test)” to simulate the
binder aging that occurs during HMA mixture production and construction operations. The RTFO measures an asphalt cement binder's resistance to aging (durability) during construction. In addition, to determine the effect of long-term aging the Pressure Aging Vessel (PAV) test was conducted in accordance with AASHTO R 28 “Standard Practice for Accelerated Aging of Asphalt Binder Using a Pressurized Aging Vessel (PAV)” to simulate binder aging (hardening) that takes place during a HMA mixtures service life. The PAV test is used to measure the resistance to aging (durability). The test purpose of the Rotational Viscometer (RV) is to measure the binder properties at high construction temperatures to assure pumping and handling during production. This test was conducted in accordance with AASHTO T 316-06 “Standard Method of Test for Viscosity Determination of Asphalt Binder Using Rotational Viscometer” for determining the viscosity of the asphalt binder at 135°C. The Dynamic Shear Rheometer (DSR) test measures the binder properties at high and intermediate service temperatures to determine its resistance to permanent deformation (rutting) and fatigue cracking. The Dynamic Shear Rheometer test was conducted in accordance with AASHTO T 315-06 “Standard Method of Test for Determining the Rheological Properties of Asphalt Binder Using a Dynamic Shear Rheometer (DSR)” method. In addition, the Bending Beam Rheometer (BBR) test is used to measure the asphalt cement binder properties at low service temperature to determine its resistance to thermal cracking. This test was performed in accordance with AASHTO T 313-06 “Standard Method of Test for Determining the Flexural Creep Stiffness of Asphalt Binder Using the Bending Beam Rheometer (BBR)”. Also, additional tests were conducted to determine the elastic properties of the asphalt cements considered in this study utilizing the force ductility and elastic recovery tests in accordance with AASHTO T 300 “Standard Method of Test for Force Ductility Test of Asphalt Materials” and AASHTO T 301 “Standard Method of Test for Elastic Recovery Test of Asphalt Materials by Means of a Ductilometer” respectively.
Table 3. LADOTD Performance Graded Asphalt Cement Specification

<table>
<thead>
<tr>
<th>Property</th>
<th>AASHTO Test Method</th>
<th>Specification</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>PG 64-22</td>
</tr>
<tr>
<td><strong>Tests on Original Binder</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rotational Viscosity @ 135°C, Pa.s</td>
<td>T 316</td>
<td>3.0-</td>
</tr>
<tr>
<td>Dynamic Shear, 10 rad/s, G*/Sin Delta, kPa</td>
<td>T 315</td>
<td>1.30+ @ 64°C</td>
</tr>
<tr>
<td>Force Ductility Ratio (F2/F1, 4°C, 5 cm/min, F2 @ 30 cm elongation)</td>
<td>T 300</td>
<td>N/A</td>
</tr>
<tr>
<td>Force Ductility, (4°C, 5 cm/min, 30 cm elongation, kg)</td>
<td>T 300</td>
<td>N/A</td>
</tr>
<tr>
<td><strong>Tests on RTFO Residue</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dynamic Shear, 10 rad/s, G*/Sin Delta, kPa</td>
<td>T 315</td>
<td>2.20+ @ 64°C</td>
</tr>
<tr>
<td>Elastic Recovery, 25°C, 10 cm elongation, %</td>
<td>T 301</td>
<td>N/A</td>
</tr>
<tr>
<td>% Mass Loss</td>
<td>T 240</td>
<td>1.00-</td>
</tr>
<tr>
<td><strong>Tests on PAV Residue</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dynamic Shear, @ 25°C, 10 rad/s, G*Sin δ, kPa</td>
<td>T 315</td>
<td>5000-</td>
</tr>
<tr>
<td>Bending beam Creep Stiffness, S, Mpa</td>
<td>T 313</td>
<td>300-</td>
</tr>
<tr>
<td>Bending beam Creep Slope, m value</td>
<td>T 313</td>
<td>0.300+</td>
</tr>
</tbody>
</table>

Note: N/A: Not Applicable
"M" designation indicates modified

3.5 HMA Mixture Blending

Upon the completion of the design phase of this study, aggregate blending calculations were performed to determine the weight of each dry aggregate component for a specific batch weight. After determination of each aggregate batch weight, aggregates were weighed and placed in flat pan. After batching, the aggregates were placed in a force draft oven at 163 °C until such time that they reached this temperature. Approximately 1 hour before blending of the aggregate with the asphalt cement (AC) binder, the AC is placed in a force draft oven at 163 °C.
To assure uniform mixing all mixing equipment were also placed in the force draft oven at 163 °C prior to blending of aggregate and AC components. After all components reached the temperature of 163 °C, these materials were placed in a mixing bucket. A crater in the center of the blended aggregate was formed for placement of the AC binder component at the specified batch weight. The mixing operation followed immediately after the AC binder component was added to the aggregate to ensure uniform blending of the materials. After mixing the final HMA mixture was distributed in a flat pan and then placed back in a force draft oven at 163 °C for 1 hour for short term aging. Upon completion of this step, the samples were prepared using the Superpave gyratory compactor to the specified dimensions for each particular test procedure.

When blending RAP as an aggregate component, it was important to add moisture to the pre-dried RAP. For this study 5 percent moisture was added to the dried RAP and then sealed prior to use. The virgin aggregates were placed in a force draft oven at 204 °C to superheat the aggregate. The superheated aggregate is needed to cause steaming of the RAP (figure 3.2) which also helps in the distribution of heat and activation of the RAP binder. The superheated aggregate components and moisture laden RAP was placed in the mixing bucket as follows: first the RAP was placed in the heated mixing bucket on the bottom then the superheated aggregate was placed on top of the RAP. The aggregates were then blended until there were no visible signs of steaming. After mixing the blended aggregates were distributed in a flat plan and placed in the oven at 163 °C to remove any remaining moisture and bring the aggregate blend to the temperature of 163 °C for required incorporation of the asphalt cement. The remaining blending steps were followed as previously described.

It is noted that the addition of crumb rubber at 10 percent by weight of total asphalt cement binder occurred after placement of the RAP and prior to placement of the superheated aggregate in the mixing bucket.
Figure 3.2 is a pictorial representation of the HMA mixture blending procedure.

3.6 Fabrications of Mixture Specimens

Laboratory mix specimens were prepared according to the specific requirements of each individual test. According to the test factorials described, cylindrical samples were fabricated. A Superpave gyratory compactor (SGC) as shown in figure 3.3 was used to compact all cylindrical specimens.
3.7 Laboratory Tests on HMA Mixtures

Laboratory mechanistic performance and material characterization tests were conducted to evaluate the laboratory performance of conventional HMA mixtures and mixtures containing high RAP content and waste tire crumb rubber/additives through their fundamental engineering properties. HMA mixture characterization in terms of fatigue cracking, moisture susceptibility, and rutting were analyzed and evaluated to determine the effects of the crumb rubber, additives, and RAP on the HMA mixtures’ performance. Specimens fabricated through various methods at the target air voids (7 ± ½%) were used to conduct laboratory mixture performance tests as outlined in table 3.4. A brief description of each test is provided below. Triplicate samples were used for each test.
Table 3. 4 Mixture Performance Tests

<table>
<thead>
<tr>
<th>Performance Characteristics</th>
<th>Test</th>
<th>Specimen Details</th>
<th>Test Temp.</th>
<th>Protocol</th>
</tr>
</thead>
<tbody>
<tr>
<td>Durability</td>
<td>Modified Lottman*</td>
<td>φ150x95-mm</td>
<td>----</td>
<td>AASHTO T 283</td>
</tr>
<tr>
<td>Permanent Deformation</td>
<td>Complex Modulus</td>
<td>φ150x100-mm</td>
<td>54 °C</td>
<td>AASHTO TP7</td>
</tr>
<tr>
<td></td>
<td>Flow Number</td>
<td>φ150x100-mm</td>
<td>54 °C</td>
<td>AASHTO TP7</td>
</tr>
<tr>
<td>Fatigue Cracking</td>
<td>DCSE</td>
<td>φ150x50-mm</td>
<td>10 °C</td>
<td>Roque [2002]</td>
</tr>
<tr>
<td></td>
<td>Semi Circular Bend</td>
<td>φ150x57-mm</td>
<td>25 °C</td>
<td>Mohammad [2005]</td>
</tr>
</tbody>
</table>

*One freeze/thaw cycle only.

3.7.1 Modified Lottman Test [AASHTO, 2003]

This test method evaluates the effect of saturation and accelerated water conditioning on compacted HMA samples utilizing freeze-thaw cycles. This method quantifies HMA mixtures sensitivity to moisture damage which is necessary to assure durability and long lasting hot mix asphalt. Numerical values of retained indirect-tensile properties are obtained by comparing conditioned samples, samples subjected to saturation and freeze-thaw cycles, to unconditioned samples. “Unconditioned” samples are samples that are not subjected to freeze-thaw cycles whereas the “Conditioned” samples are. Six – 150 x 95-mm diameter samples are compacted with a Superpave Gyratory Compactor (SGC) to an air void content of 7 ± 0.5 percent. After compaction and air void determination, the six SGC samples are subdivided into two groups of three samples so that the average air void contents of the two subsets are approximately equivalent. The “unconditioned” sample subset was stored at room temperature for 24 ± 3 hours. Afterwards the “unconditioned” specimens were wrapped or placed in a heavy duty, leak proof plastic bag and then conditioned for 2 hours ± 10 minutes in a 25 ± 0.5 °C (77 ± 1°F) water bath. After conditioning, the “unconditioned” specimens were tested to determine the tensile strength for each specimen and then the values of each were summed to determine the average tensile
strength. To calculate the tensile strength of the “unconditioned” and “conditioned” specimens the following formulas are used:

\[
S_t = \frac{2000P}{\pi(t)(D)} \quad \text{(SI Units)} \hspace{1cm} \text{.................................................................(1)}
\]

\[
S_t = \frac{2P}{\pi(t)(D)} \quad \text{(U.S. Customary units)} \hspace{1cm} \text{.................................................................(2)}
\]

where:

- \(S_t\) = tensile strength, kPa (psi);
- \(P\) = maximum load, N (lbs);
- \(t\) = specimen thickness, mm (inches);
- \(D\) = specimen diameter, mm (inches).

The second subset, termed “conditioned” samples are vacuum saturated to a degree of 70 percent to 80 percent saturation by placing the samples in a vacuum container and applying a vacuum of 13 – 67 kPa absolute pressure (10 -26 inches Hg partial pressure) for approximately 5 to 10 minutes. After saturation, the volume of absorbed water is determined by the following formula:

\[
J' = B' - A \hspace{1cm} \text{...............................................................................................................(3)}
\]

where:

- \(J'\) = volume of absorbed water, cubic centimeters
- \(B'\) = mass of saturated, surface-dry specimen after partial vacuum saturation
  
  \(\text{(AASHTO T 166 – Method A), g}\)
- \(A\) = mass of the dry specimen in air, g

The degree of saturation, \(S'\), is determined by comparing the volume of absorbed water, \(J'\), with the volume of air voids \((V_a)\) previously computed using the following formula.
After the degree of saturation for each specimen has been verified and meets test protocol, the “conditioned” samples are individually wrapped with a plastic film and then placed and sealed in a plastic bag containing 10 ± 0.5 mL of water. Then the samples are placed in a freezer at a temperature of -18 ± 3 °C (0 ± 5 °F) for a minimum of 16 hours. After freezing the samples the samples are then thawed by placing them in a water bath at a temperature of 60 ± 1°C (140 ± 2 °F) for 24 ± 1 hour and then placed in another water bath with a temperature of 25 ± 0.5 °C (77 ± 1 °C) for 2 hours ± 10 minutes. After the thawing process the “conditioned” samples are tested to determine their tensile strength and subsequent average tensile strength for the subset.

The tensile strength ratio (TSR) is the numerical value of the HMAs resistance to the detrimental effects of moisture. It is defined as the ratio of the original tensile strength that is retained after the moisture and freeze thaw conditioning (average tensile strength of “conditioned” specimens) to the average tensile strength of the “unconditioned” samples as shown by the following formula:

\[
\text{Tensile Strength Ratio (TSR)} = \frac{S_2}{S_1}
\]

where:

\( S_1 \) = average tensile strength of “unconditioned” specimens, kPa (psi); and

\( S_2 \) = average tensile strength of “conditioned” specimens, kPa (psi).

3.7.2 Dissipated Creep Strain Energy Test

Fatigue cracking is a major asphalt pavement distress that concerns the owners of asphalt pavement highways. Fatigue cracking begins as microcracks that later coalesce to form
macrocracks that propagate due to either tensile or shear stress or a combination of both. Research has indicated that a threshold concept is a good indicator of the cracking mechanism of asphalt pavements and Dissipated Creep Strain Energy (DCSE) is the most reliable criterion to be used as this threshold [Mull et al., 2002]. The DCSE threshold represents the energy that the mixture can tolerate before it fractures. Two laboratory tests, the indirect resilient modulus ($M_R$) test [Witczak, 2004] and the indirect tensile strength (ITS) tests [AASHTO, 2006] were conducted at 10°C on the same specimen to calculate the Dissipated Strain Energy. Triplicate specimens of 150 mm in diameter and 50 mm in thickness were used. Sample instrumentations as shown in figure 3.4 were used in order to accurately capture the small deformations resulting from the repeated load applied in the $M_R$ test. Two units of single integral, bi-axial extensometers model 3910 from epsilon technology that measure both lateral and vertical deformations were clipped onto gage points mounted on each face of the specimen. The gage length (i.e. the distance between two gage points) was maintained at 3 inches which is one half of the sample diameter [Witczak, 2004]. The test specimens were conditioned at 10°C for four hours before a 200-cycle haversine load with 0.1 second loading period and 0.4 second rest period in each loading cycle was applied along the diametrical plane on the specimen. A conditioning loading sequence was applied before the starting of the actual test in order to obtain uniform measurements in load and deformation. Then, a four-cycle haversine compressive load was applied and load and deformation data was recorded continuously. The magnitude of the applied load should be such that it results in a deformation as close as possible to 100 microstrains. After one test is completed, the specimen was rotated 90 degrees and tested again. The resilient modulus was then calculated from the average value of the two test results. Once the $M_R$ test is finished, the ITS test was performed on the same specimen. Both tests, $M_R$ and
ITS, will be performed using an MTS hydraulic loading system which will also be the same system that will be used for the SCB test.

The DCSE calculation used in this study was introduced by Roque et al. [2002 and 2004] and later used by Alshamsi [2006]. As indicated in figure 3.5, DCSE is defined as the Fracture Energy (FE) minus the Elastic Energy (EE). The Fracture Energy is defined as the area under the stress-strain curve up to the point where the specimen begins to fracture. As shown in figure 3.6 the area within the curve OA and X-axis (i.e. Area OAB) is the fracture energy. The Elastic Energy is the energy resulting in elastic deformation. Therefore, $M_R$, calculated from Resilient Modulus test, is selected as the slope of the line AC and the area of triangle ABC is taken as the Elastic Energy (EE). The failure strain ($\varepsilon_f$), Peak tensile strength ($S_t$) and fracture energy are determined from the ITS test. A rather clear picture of DCSE calculation is described below:

$$M_r = \frac{S_t}{\varepsilon_f - \varepsilon_0}$$

(6)
Therefore, $\varepsilon_0 = \frac{(M_R \times \varepsilon_f - S_i)}{M_R}$ ................................................................. (7)

$EE = \frac{1}{2} \times S_i \times (\varepsilon_f - \varepsilon_0)$ ................................................................................................................. (8)

$DCSE = FE - EE$ .................................................................................................................................................. (9)

Figure 3. 5 Dissipated Creep Strain Energy Determination

3.7.3 Semi-Circular Bend (SCB) Test

This test characterizes the fracture resistance of asphalt mixtures [Mohammad et al., 1992 and 2004, Mull et al., 2002] based on a fracture mechanics concept, the critical strain energy release rate, also called the critical value of J-integral, or Jc. To determine the critical value of J-integral, semi-circular specimens with three notch depths (25.4-, 31.8- and 38.0 mm) were tested. The test will be conducted at 25 °C. A semi-circular specimen was loaded monotonically till
fracture under a constant cross-head deformation rate of 0.5 mm/min in a three-point bend load configuration (figure 3.6).

The load and deformation are continuously recorded and the critical value of J-integral is determined based on the following equation:

$$JC = - \left( \frac{1}{b} \right) \frac{dU}{da}$$  

where:

- $b$ = sample thickness
- $a$ = the notch depth
- $U$ = the strain energy to failure.

Aged samples were prepared and tested to examine the influence of CR additives and high RAP contents mixtures performance. Mixture aging was performed according to AASHTO PP2 [AASHTO, 1994] by placing compacted specimens in a forced draft oven for five days at 85°C.

![Figure 3.6 Set-up of Semi-Circular Bending Test](image)

$2r_0=152\text{mm}, 2s=127\text{mm}, b=57\text{mm}$
3.7.4 Simple Performance Tests (SPTs)

Simple SPT tests were performed to characterize the laboratory performance of mixtures evaluated in this study with respect to resistance to permanent deformation as measured by the Dynamic Modulus and Flow number tests. Using the measured Dynamic Modulus and Phase Angles obtained from the Simple Performance Tests a rutting factor and a fatigue factor can be developed which is an indication of a HMA mixtures ability to resist permanent deformation (i.e. rutting).

3.7.4.1 Dynamic Modulus, $|E^*|$

The dynamic modulus test is a triaxial compression test, which was standardized in 1979 as ASTM D3497, “Standard Test Method for Dynamic Modulus of Asphalt Concrete Mixtures” [ASTM, 1979]. This test consists of applying a uniaxial sinusoidal (i.e., haversine) compressive stress to an unconfined or confined HMA cylindrical test specimen as shown in figure 3.7. The stress to strain relationship under a continuous sinusoidal loading for linear viscoelastic materials is defined by a complex number called the “complex modulus” ($E^*$). The absolute value of the complex modulus $|E^*|$, is defined as the dynamic modulus. The dynamic modulus is mathematically defined as the maximum (i.e., peak) dynamic stress ($\sigma_o$) divided by the peak recoverable strain ($\varepsilon_o$).

\[
|E^*| = \frac{\sigma_o}{\varepsilon_o}
\]

(11)

This test is conducted at -10, 4, 20, 38.8 and 54.4C at loading frequencies of 0.1, 0.5, 1.0, 5, 10, 25 Hz at each temperature [Witczak et al., 2002].
3.7.4.2 Repeated Loading Test/Flow Number Test

The flow number test is used to determine the permanent deformation characteristic of hot mix asphalt mixtures by applying a repeated haversine load for several thousand cycles on a cylindrical asphalt sample. The load is applied for 0.1 second with a rest period of 0.9 second in one cycle as shown in figure 3.8.

In this study, the test is conducted for 10,000 cycles at 54°C, and a stress level of 30 psi is used. This test is conducted on specimens 100mm in diameter and 150mm tall for mixtures with nominal maximum size aggregates less than or equal to 37.5mm (1.5 in). The flow number is defined as the number of repetitions corresponding to the minimum rate of change in permanent strain under repeated loading conditions. It is determined by differentiation of the permanent strain versus the number of load cycles curve. Figure 3.8 represents an example of a typical permanent axial strain response and the computation of flow number.
3.7.5 Load Wheel Tracking (LWT) Test

One of the major distresses in asphalt pavements is its inability to resist permanent deformation due to traffic loading. To determine the rutting characteristics of the HMA mixtures considered in this study a loaded wheel tracking test was conducted in accordance with AASHTO T 324-04 “Standard Method of Test for Hamburg Wheel-Track Testing of Compacted Hot-Mix Asphalt (HMA)”. In this test specimens are subjected to a steel wheel weighing 703 N (158 pounds) which are repeatedly rolling across its surface while being submerged in 50 °C hot water. The test completion time is predicated upon test specimens being subjected to a maximum of 20,000 cycles or attainment of 20 mm deformation, whichever is reached first. Upon completion of the test the average rut depth for the samples tested are recorded.
The Hamburg type LWT manufactured by PMW, Inc of Salina, Kansas was used in this study (figure 3.9). The Hamburg LWT can test two specimens simultaneously at a time. The test specimens are subject to two reciprocating solid-steel wheels of 203.5 mm (8 inch) in diameter and 47 mm (1.85 inch) in width while being submerged in hot water at the specified temperature of 50 °C which was utilized in this study. Before actual testing of the laboratory specimens they were conditioned at 50 °C for 90 minutes. After conditioning a fixed load of 703 N (158 lb) with a rolling speed of 1.1 km/h (0.68 mi/h) at the rate of 56 passes /min was implied. Each wheel rolls 230 mm (9.1 inch) before reversing direction.

In order to accurately measure permanent deformation two Linear Variable Displacement Transducers (LVDT’s) were utilized and the subsequent test results (rut depths, number of passes, water bath temperature) are collected and recorded in an automatic data recording system associated with the Hamburg Wheel Tracking Device used in this study. Figure 3.10 represents a typical LWT test output.
3.8 Conduct Data Analysis

Laboratory test data were statistically analyzed using the analysis of variance (ANOVA) procedure provided in the Statistical Analysis System (SAS) program from SAS Institute, Inc. A multiple comparison procedure with a risk level of 5 percent was performed on the means. The groupings will represent the mean for the test results reported by mixture type. The results of the statistical grouping were reported with the letters A, B, C, D, and so forth. The letter A was assigned to the highest mean followed by the other letters in appropriate order. A double (or more) letter designation, such as A/B (or A/B/C), will indicate that in the analysis the difference in the means is not clear-cut, and that the mean is close to either group.
CHAPTER 4 DISCUSSION OF TEST RESULTS

4.1 Asphalt Binder Test Results

It is important to realize that an asphalt cement binder’s rheological properties have an effect on the performance of a HMA pavement. Changes in the AC rheological properties due to production and aging which result from oxidation and environmental influences must be addressed to reduce asphalt binder related pavement distresses such as raveling, cracking, stripping, and rutting. It is essential that the asphalt cement binders are tested to assure that the binder rheology meets specified criteria necessary to reduce pavement distresses. Therefore specifications were developed to characterize an asphalt cement binders rheology which is necessary to minimize the ACs contribution to durability issues, rutting, fatigue cracking, and low temperature cracking.

To assure that an asphalt cement binder meets criteria to reduce and/or prevent pavement distresses due to changes of its rheological properties necessitates testing of the asphalt cements binder properties. Therefore specifications were developed to minimize an asphalt cement binder’s contribution for durability, rutting, fatigue cracking, and low temperature cracking.

Table 4.1 presents the physical and rheological asphalt cement binder test results for the asphalt cement binders considered in this study. The PG 76-22 CRM designated material as shown in table 4.1 utilized unmodified asphalt cement (PG 64-22) that had been wet blended with CR to yield PG 76-22 asphalt cement binder. The PG 76-22 CRM asphalt cement binder had a CR total content of 9 percent crumb rubber additive, 8 percent 30 mesh crumb rubber and 1 percent Gilsonite. The 64RAP40 Extraction sample is the extracted asphalt cement binder taken from the 64RAP40 HMA mixtures and subsequently tested for specification compliance. In the making of the 64RAP40 HMA mixture, crumb rubber additives were introduced as a dry feed at 10 percent by total weight of asphalt cement binder. This study explored the use of the
absorption properties of crumb rubber to carry asphalt cement binder components that are typically lost during oxidation of HMA pavements as a dry feed component in the making of hot mix asphalt mixtures. There were two distinct CR additive components used as a dry feed in the 64RAP 40 HMA mixtures. The first CR component was comprised of 70 percent 30 mesh crumb rubber that had been pre-swelled, 10 percent long-chain wax, and 20 percent asphaltenes. The second component contained 70 percent 30 mesh pre-swelled crumb rubber, 10 percent long-chain wax, and 20 percent de-metalized motor oil. The two components were blended at a 50/50 ratio before being introduced into the HMA mixture at the specified rate of 10 percent by total weight of binder. Table 4.1 shows the final test results for the conventional and crumb rubber modified (wet and dry blend) asphalt cement binders used in this study. The Rotational Viscosity measured at 135 °C for all ACs considered in this study passed the specified criteria of 3.0 Pa·s (maximum value) with the exception of the PG 76-22 CRM binder, 3.1 Pa·s. The conventional asphalt cement binders (PG 64-22, PG 70-22M, PG 76-22M) utilized in this study passed all specification requirements for their appropriate grading as observed in table 4.1. In regards to the extracted 64RAP40 binder, research has shown that when high percentages of RAP are incorporated (i.e. 40 percent as in the 64RAP40 HMA mixture) into a HMA mixture, the blended asphalt cement (RAP AC plus virgin AC) will be stiffer and will grade out as high as three temperature grades, high and low temperature specification parameters, above the original virgin AC used [McDaniel et al., 2001]. For example, the virgin AC grading is PG 64-22, then 40 percent RAP is added to the mixture and RAP AC blends with the virgin AC during production. The asphalt cement is extracted from the HMA mixture and then tested to determine it grading. The final grading could be as much as three temperature grades higher than original grading, i.e. PG 82-4. Table 4.1 shows that this was not the case in the 64RAP40 extraction. The actual final performance grade of this material was a PG 70-28. The addition of the crumb
rubber modifiers softened the RAP binder such that the final blended material stiffness was not increased. In fact, on the high temperature side there was an increase in one temperature grade from PG 64 to PG 70 and there was a significant decrease in the low temperature properties of one grade, from -22 to -28. It must be noted that the extracted binder material did not go through the PAV process that provides for long-term service aging. Aging the 64RAP40 extracted binder with this process would have additionally stiffened the G*Sin\(\delta\) and the Bending Beam results. However, in doing so the worst case scenario would have been that the final asphalt cement blend would have graded out as a PG 70-22. Never the less, the 64RAP40 extracted binder will be more rut and fatigue resistant than the PG 64-22 while also being more resistant to low temperature cracking (thermal cracking). In addition, in regards to pavement performance based on the asphalt cement binder rheology presented in table 4.1, the 64RAP40 HMA mixture should be more rut resistant than the PG70-22M conventional mixture (70CO) and possibly as comparable to the conventional PG 76-22M HMA mixture (76CO) especially since the G*/Sin\(\delta\) rutting factor on the original binder test parameter passed at the temperature of 76 °C. Also the 64RAP40 HMA mixture resistance to cracking should be better than the conventional mixture (64CO) utilizing the PG 64-22 and comparable to the 70CO HMA mixture that had the PG 70-22M asphalt cement binder. Table 4.1 indicates that the addition of a crumb rubber as a dry feed for the purpose of carrying rejuvenating type additives without sacrificing performance is viable. It is shown in table 4.1 that the addition of crumb rubber as a wet blend (PG 76-22CRM) increased the dynamic shear G*/Sin\(\delta\) rutting factor properties and rotational viscosity of the asphalt cement binder while improving the fatigue rutting factor G*(Sin\(\delta\)) as indicated by the Dynamic Shear results at the 25 °C testing temperature. The addition of the wet blended crumb rubber appears to have also improved the elastic properties of the asphalt cement binder tested as shown by the Bending Beam test results.
Table 4. 1 LADOTD Performance Graded Asphalt Cement Specification Test Results

<table>
<thead>
<tr>
<th>Spec</th>
<th>PG 64-22</th>
<th>PG 70-22M</th>
<th>PG 76-22M</th>
<th>PG 76-22 CRM (Wet Blend)</th>
<th>64RAP40 Extraction</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Test on Original Binder</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dynamic Shear, G*/Sin(δ), (kPa)</td>
<td>1.30° @ 64°C</td>
<td>1.92</td>
<td>----</td>
<td>----</td>
<td>6.65</td>
</tr>
<tr>
<td>Dynamic Shear, G*/Sin(δ), (kPa)</td>
<td>1.00° @ 70°C</td>
<td>0.88</td>
<td>1.64</td>
<td>----</td>
<td>3.35</td>
</tr>
<tr>
<td>Dynamic Shear, G*/Sin(δ), (kPa)</td>
<td>1.00° @ 76°C</td>
<td>----</td>
<td>----</td>
<td>1.82</td>
<td>2.71</td>
</tr>
<tr>
<td>Dynamic Shear, G*/Sin(δ), (kPa)</td>
<td>1.00° @ 82°C</td>
<td>----</td>
<td>----</td>
<td>1.29</td>
<td>1.54</td>
</tr>
<tr>
<td>Force Ductility Ratio</td>
<td>N/A</td>
<td>N/A</td>
<td>0.49</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Force Ductility, (4°C, 5 cm/min, 30 cm elongation, kg)</td>
<td>N/A</td>
<td>0.31</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Rotational Viscosity @ 135°C (Pa·s)</td>
<td>3.0°</td>
<td>0.5</td>
<td>0.9</td>
<td>1.7</td>
<td>3.1</td>
</tr>
<tr>
<td><strong>Tests on RTFO</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dynamic Shear, G*/Sin(δ), (kPa)</td>
<td>2.20° @ 64°C</td>
<td>3.25</td>
<td>----</td>
<td>----</td>
<td>5.56</td>
</tr>
<tr>
<td>Dynamic Shear, G*/Sin(δ), (kPa)</td>
<td>2.20° @ 70°C</td>
<td>1.61</td>
<td>3.14</td>
<td>----</td>
<td>4.72</td>
</tr>
<tr>
<td>Dynamic Shear, G*/Sin(δ), (kPa)</td>
<td>2.20° @ 76°C</td>
<td>----</td>
<td>1.65</td>
<td>2.48</td>
<td>5.97</td>
</tr>
<tr>
<td>Dynamic Shear, G*/Sin(δ), (kPa)</td>
<td>2.20° @ 82°C</td>
<td>----</td>
<td>----</td>
<td>1.67</td>
<td>3.25</td>
</tr>
<tr>
<td>Elastic Recovery, 25°C, 10 cm elongation, %</td>
<td>N/A</td>
<td>65</td>
<td>70</td>
<td>75</td>
<td>N/A</td>
</tr>
<tr>
<td><strong>Tests on (RTFO+ PAV)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dynamic Shear, @ 25°C, G*Sin(δ), (kPa)</td>
<td>5000°</td>
<td>2774</td>
<td>4615</td>
<td>2297</td>
<td>2166</td>
</tr>
<tr>
<td>Bending Beam Creep Stiffness @ -12°C, (MPa)</td>
<td>300°</td>
<td>234</td>
<td>196</td>
<td>152</td>
<td>104</td>
</tr>
<tr>
<td>Bending Beam m-value@ -12°C</td>
<td>0.300°</td>
<td>0.312</td>
<td>0.317</td>
<td>0.327</td>
<td>0.320</td>
</tr>
<tr>
<td>Bending Beam Creep Stiffness @ -18°C, (MPa)</td>
<td>300°</td>
<td>----</td>
<td>----</td>
<td>----</td>
<td>----</td>
</tr>
<tr>
<td>Bending Beam m-value@ -18°C</td>
<td>0.300°</td>
<td>----</td>
<td>----</td>
<td>----</td>
<td>----</td>
</tr>
</tbody>
</table>

In order to determine if the CR additives had an effect on the recycled asphalt pavement used in this study it was necessary to blend the crumb rubber additives with the RAP. The crumb rubber
additive rate was based on 10 percent of total weight of binder. This percentage was back calculated to determine the weight of crumb rubber based on 100 percent RAP. The calculated rate based on RAP only was determined to be 1.7 percent by total weight of RAP. Once the CR material weight was determined for a given RAP weight the two CR components as previously described were blended at a 50/50 ratio before being introduced into the RAP mixture at the specified rate. After the RAP and CR additives were weighed out for blending the RAP was then heated to 163 °C and then crumb rubber additives were blended as a dry aggregate with the RAP after the RAP reached the required mixing temperature. The RAP blend was then thoroughly mixed using a mixing bucket. After blending the RAP crumb rubber mixture blend was allowed to cool. The asphalt cement from this RAP blend was then extracted and its rheological properties were then determined. In addition the original RAP was extracted and its asphalt cement properties were characterized. This was necessary so that a comparison could be made and to determine if the crumb rubber additives had an effect on the blended asphalt binders.

Table 4.2 presents the extracted RAP binder test results for the 100 percent RAP and the 100 percent RAP and CR additive blend. It is noted that the asphalt cement materials characterized in table 4.2 did not go through the RTFO and PAV aging methods since the RAP utilized in this study was from a roadway previously constructed 15 years earlier and this material had been through the short term and long term aging process naturally throughout its life on the roadway. It is shown that both materials graded out as a PG 82-16, however in review of each test parameter the addition of the crumb rubber to the RAP had a positive effect. The actual test results showed a significant improvement in regards to $G^*/\sin \delta$, rutting factor, tested as the original asphalt cement binder and as tested as a RTFO material. In addition the $G^*(\sin \delta)$, fatigue factor, as tested on the PAV material exhibited a substantial improvement over the extracted 100 percent RAP material.
Table 4. 2 Extracted RAP Binder Test Results

<table>
<thead>
<tr>
<th>Spec</th>
<th>Extracted 100% RAP</th>
<th>Extracted 100% RAP w/ CR</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Test on Original Binder</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dynamic Shear, (G*/\sin(\delta)), (kPa) @ 70°C</td>
<td>1.00(^{+})</td>
<td>32.23</td>
</tr>
<tr>
<td>Dynamic Shear, (G*/\sin(\delta)), (kPa) @ 76°C</td>
<td>1.00(^{+})</td>
<td>14.25</td>
</tr>
<tr>
<td>Dynamic Shear, (G*/\sin(\delta)), (kPa) @ 82°C</td>
<td>1.00(^{+})</td>
<td>6.41</td>
</tr>
<tr>
<td>Dynamic Shear, (G*/\sin(\delta)), (kPa) @ 88°C</td>
<td>1.00(^{+})</td>
<td>3.18</td>
</tr>
<tr>
<td>Rotational Viscosity @ 135°C (Pa·s)</td>
<td>3.0(^{+})</td>
<td>3.9</td>
</tr>
<tr>
<td><strong>Tests on RTFO</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dynamic Shear, (G*/\sin(\delta)), (kPa) @ 70°C</td>
<td>2.20(^{+})</td>
<td>24.94</td>
</tr>
<tr>
<td>Dynamic Shear, (G*/\sin(\delta)), (kPa) @ 76°C</td>
<td>2.20(^{+})</td>
<td>8.53</td>
</tr>
<tr>
<td>Dynamic Shear, (G*/\sin(\delta)), (kPa) @ 82°C</td>
<td>2.20(^{+})</td>
<td>4.07 pass</td>
</tr>
<tr>
<td>Dynamic Shear, (G*/\sin(\delta)), (kPa) @ 88°C</td>
<td>2.20(^{+})</td>
<td>1.90</td>
</tr>
<tr>
<td><strong>Tests on (RTFO+ PAV)</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dynamic Shear, @ 25°C, (G*\sin(\delta)), (kPa)</td>
<td>5000(^{+})</td>
<td>8616</td>
</tr>
<tr>
<td>Dynamic Shear, @ 28°C, (G*\sin(\delta)), (kPa)</td>
<td>5000(^{+})</td>
<td>6135</td>
</tr>
<tr>
<td>Dynamic Shear, @ 31°C, (G*\sin(\delta)), (kPa)</td>
<td>5000(^{+})</td>
<td>4329</td>
</tr>
<tr>
<td>Bending Beam Creep Stiffness @ -12°C, (MPa)</td>
<td>300(^{+})</td>
<td>200</td>
</tr>
<tr>
<td>Bending Beam m-value@ -12°C</td>
<td>0.300(^{+})</td>
<td>0.299</td>
</tr>
<tr>
<td>Bending Beam Creep Stiffness @ -18°C, (MPa)</td>
<td>300(^{+})</td>
<td>397</td>
</tr>
<tr>
<td>Bending Beam m-value@ -18°C</td>
<td>0.300(^{+})</td>
<td>0.236</td>
</tr>
<tr>
<td><strong>Actual PG Grading</strong></td>
<td>PG 82-16</td>
<td>PG 82-16</td>
</tr>
</tbody>
</table>

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Figure 4.1 is a graphical illustration of the Dynamic Shear rutting factor results of the 100 percent RAP and 100 percent RAP and crumb rubber additives. It is shown that throughout the temperature ranges tested the 100 percent RAP and crumb rubbers were substantially improved over the 100 percent RAP only material. It is shown that both materials passed the required specification of 1.1 kPa at 88 °C.

![Graph showing Dynamic Shear results](image)

Figure 4.1 Dynamic Shear, $G^*/\sin(\delta)$ (RAP binder tested as Original)

Figure 4.2 illustrates the rutting factor, $G^*/\sin\delta$, Dynamic Shear test results on both materials tested as a RTFO material. This figure also shows a benefit in the addition of the crumb rubber additives as can be seen by the lower values for each test temperature tested. It is noted that these materials passed the specification requirement of 2.2 kPa at 82 °C yet failed this criteria at the 88 °C test temperature. Therefore these materials grade out as a PG 82 material on the high temperature side of the performance graded binder specification.
Figure 4.3 presents the graphical representation of the fatigue factor, $G^*/\sin(\delta)$, Dynamic Shear test results for the 100 percent RAP and 100 percent RAP and crumb rubber additives tested as a PAV material. It is shown that there was an improvement in the binder characterization when the crumb rubber additives were blended with the 100 percent RAP as indicated by the decrease in actual test values. This improvement can be seen at all test temperatures conducted for fatigue factor determination. The specification criterion for this material is a maximum value of 5000 kPa tested at 25 °C. In table 4.2 it is indicated that the 100 percent RAP and crumb rubber additives passed this criteria at the required specification test temperature of 25 °C and that the 100 percent RAP binder had a failing test value of 8616 kPa at this temperature. It is shown that the 100 percent RAP binder attained a passing test value of 4329 kPa at the test temperature of 31 °C whereas the 100 percent RAP and crumb rubber additive binder decreased to a test value of 1899 kPa.
Figure 4.3 Dynamic Shear, $G*\sin(\delta)$ (RAP binder tested as PAV)

Figure 4.4 shows the Rotational Viscosity test results for both extracted RAP binders. It is indicated that both materials failed specification criteria of 3.0 Pa·s at the test temperature of 135°C. However the addition of the CR clearly illustrates a reduction in viscosity as indicated.

Figure 4.4 RAP Binder Rotational Viscosity
4.2 HMA Mixture Characterization Test Results

Several laboratory tests were conducted and evaluated to measure the performance characteristics of the HMA mixtures considered in this study. The pavement performance characteristics were analyzed for the HMA mixtures durability as measured by the Modified Lottman test. The HMA mixtures performance in terms of resistance to fatigue cracking was evaluated from results obtained from the Semi-Circular Bend (SCB), Dissipated Creep Strain Energy (DCSE) and Dynamic Modulus (i.e. fatigue factor, $E^*(\sin \delta)$) tests. Furthermore, Dynamic Modulus (i.e. rutting factor, $E^*/\sin \delta$) and Flow Number was used to determine the mixtures resistance to permanent deformation. Triplicate samples were prepared and tested for each laboratory test. The detailed analysis for these test results is included in the following sections of this chapter.

4.2.1 Modified Lottman Test Results

The Modified Lottman test evaluates the effect of saturation and accelerated water conditioning on compacted HMA samples utilizing freeze-thaw cycles. This test quantifies the HMA mixtures sensitivity to moisture damage which is necessary to assure durability and long lasting hot mix asphalt pavements. Moisture sensitivity is measured by the percentage of retained tensile strength ratio of the conditioned samples compared to the control samples. The conditioned samples are samples that have been subjected to the required freeze/thaw cycle. Louisiana requires that the retained tensile strength be equal or greater than 80 percent to be considered as a passing result. Table 4.3 presents the measured Modified Lottman test results and figure 4.5 is the graphical presentation of the test results as shown in table 4.3 for the six mixtures evaluated in this study. It is noted that no liquid anti-strips which facilitates adhesion of the asphalt cement binder to the aggregates were used in this study. In doing so, the test
results specifically indicate the asphalt cement binders affect on adhesion to the aggregate by the measure of the percent retained tensile strength ratio. As expected the 64CO HMA mixture utilizing an unmodified asphalt cement binder (PG 64-22) failed the Modified Lottman test. In addition, mixtures 70CO, 76CO, and 76RAP15 which contain SBS modified asphalt cement binders (PG 70-22M, PG 76-22M), had passing results. It is shown in table 4.3 that the HMA mixture 76RAP15 had the highest percent tensile strength ratio followed by the 76CO mixture. In addition, the HMA mixture 64RAP40 which contains and unmodified AC (PG 64-22), 40 percent RAP, and CR additives passed the Modified Lottman test. However the mixture 76CRM failed this test. The AC used in the 76CRM HMA mixture is unmodified asphalt cement (PG 64-22) that has been wet blended with CR to yield PG 76-22 asphalt cement. The CR modified asphalt had a total of 9 percent crumb rubber additive, 8 percent 30 mesh crumb rubber and 1 percent Gilsonite. It is suspected that the percentage of Gilsonite, which is used to increase resistance to water susceptibility (stripping), was insufficient to increase the retained tensile strength to passing level. Table 4.3 shows similar results for the 64CO and 76CRM mixtures therefore it appears that the 76CRM retained the PG 64-22 AC properties in regards to stripping.

<table>
<thead>
<tr>
<th>Mix Type</th>
<th>Tensile Strength (PSI)</th>
<th>Mix Type</th>
<th>Tensile Strength (PSI)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Control</td>
<td>Conditioned</td>
<td></td>
</tr>
<tr>
<td>64CO</td>
<td>Average</td>
<td>166.48</td>
<td>105.40</td>
</tr>
<tr>
<td></td>
<td>Stdev</td>
<td>31.85</td>
<td>13.29</td>
</tr>
<tr>
<td></td>
<td>CV</td>
<td>19.13</td>
<td>12.61</td>
</tr>
<tr>
<td></td>
<td>%TSR</td>
<td>63.3</td>
<td></td>
</tr>
<tr>
<td>76CO</td>
<td>Average</td>
<td>188.44</td>
<td>160.83</td>
</tr>
<tr>
<td></td>
<td>Stdev</td>
<td>9.27</td>
<td>28.11</td>
</tr>
<tr>
<td></td>
<td>CV</td>
<td>4.92</td>
<td>17.48</td>
</tr>
<tr>
<td></td>
<td>%TSR</td>
<td>85.3</td>
<td></td>
</tr>
<tr>
<td>76RAP15</td>
<td>Average</td>
<td>160.64</td>
<td>140.22</td>
</tr>
<tr>
<td></td>
<td>Stdev</td>
<td>4.01</td>
<td>8.35</td>
</tr>
<tr>
<td></td>
<td>CV</td>
<td>2.49</td>
<td>5.95</td>
</tr>
<tr>
<td></td>
<td>%TSR</td>
<td>87.3</td>
<td></td>
</tr>
</tbody>
</table>
4.2.2 Dissipated Creep Strain Energy (DCSE) Test Results

Mull et al. [2002] has indicated Dissipated Creep Strain Energy (DCSE) is a good indicator of the cracking mechanism of asphalt pavements. The calculated DCSE threshold values represent the energy that the mixture can tolerate before it fractures. Roque et al. [2004] reported that a DCSE value of 0.75 KJ/m³ as the limiting criterion. HMA mixtures having DCSE value greater than 0.75 KJ/m³ are not as susceptible to cracking. Mixtures that exhibit lower DCSE values are more susceptible to cracking than HMA mixtures having higher values when mixtures are exposed to similar conditions such as environmental and loading. Figure 4.6 represents the calculated DCSE mean values for the HMA mixtures analyzed in this study. The coefficient of variation (CV) of the samples tested ranged from 6 to 18 percent. It is shown that the 76CO mixture has the highest DCSE values of all mixtures tested and therefore is less prone
to crack. In addition, five out of the six mixtures meet the 0.75 KJ/m³ criteria for resistance to cracking. The 64RAP40 had less than the limiting criterion necessary for fracture resistance.

![Figure 4.6 Dissipated Creep Strain Energy Test Results](image)

**4.2.3 Semi-Circular Bend (SCB) Test Results**

Table 4.4 indicates the average peak load during testing that was required to cause cracking to begin at the tip of the sample notches which were previously cut as part of the sample preparation. It is shown in this table that there is a decrease in peak load as the notch depths are increased for all mix types. This is consistent since it would take lower peak loads to propagate cracking as the effective depth of the sample above the notch decreases due to the increased sample notch depth. In addition, table 4.4 indicates that in general for the mixtures studied, the mixtures containing RAP had the highest SCB peak loads. It is noted that the 76RAP15 has the highest peak load followed by the 64RAP40 HMA mixture.
Figure 4.7 presents the calculated critical fracture resistance ($J_c$) values for the six HMA mixture types evaluated. In terms of fracture resistance, the higher the $J_c$ value the greater the fracture resistance the HMA mixtures possess. It is shown that the 76RAP15 HMA mixture had the highest $J_c$ value and therefore has the greatest fracture resistance of all mixtures evaluated in this study.

<table>
<thead>
<tr>
<th>Mix Type</th>
<th>Peak Load (KN)</th>
<th>Notch Depths (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Aged</td>
<td>25.4</td>
</tr>
<tr>
<td>64CO</td>
<td>0.75</td>
<td>0.62</td>
</tr>
<tr>
<td>70CO</td>
<td>0.93</td>
<td>0.60</td>
</tr>
<tr>
<td>76CO</td>
<td>1.11</td>
<td>0.77</td>
</tr>
<tr>
<td>76CRM</td>
<td>1.18</td>
<td>0.92</td>
</tr>
<tr>
<td>76RAP15</td>
<td>1.77</td>
<td>1.03</td>
</tr>
<tr>
<td>64RAP40</td>
<td>1.17</td>
<td>1.11</td>
</tr>
</tbody>
</table>

It is noted that the 76RAP15 also had the highest peak loads necessary to propagate cracking as shown in table 4.4. In a previous study, Mohammad et al. [2004] indicates that any mixture achieving a $J_c$ value greater than 0.65 KJ/m$^2$ is expected to exhibit good fracture resistance. Figure 4.7 indicates that with the exception of the 64CO HMA mixture all other mixtures passed this criterion.
4.2.4 Dynamic Modulus (E*) Test Results

The purpose of the Dynamic Modulus test is to evaluate the visco-elastic response characteristics of HMA mixtures over a given range of temperatures and frequencies. Figure 4.8 and 4.9 present the Dynamic Modulus Isotherms at the various temperatures and frequencies for all mixtures considered in this study. The values indicated are the average E* results for three laboratory specimens evaluated per HMA mixture type. The CV of the samples tested ranged from 1 to 18 percent. As shown in figure 4.8 and figure 4.9, the E* values increase as the frequency increases. Furthermore, the E* values decrease with increased temperatures. Figure 4.9 indicates that at low temperatures (4.4 °C) the isotherms to be in an inclined straight line direction. This indicates that the HMA mixture behavior is in the visco-elastic region and is predominantly affected by the asphalt cement binders. The E* isotherms became concave at the intermediate and high temperature levels, 25 °C, 37.8 °C, and 54 °C respectively. This change in the isotherm shape represents the non-linear behavior in HMA mixtures during compression. This non-linear behavior reveals the mechanical response which is caused by the aggregate skeleton of the HMA mixture overwhelming the viscous influence of the asphalt cement binder.
materials within the HMA materials at these high temperatures. In figure 4.8, it is shown that at any given temperature for any given HMA mixture that the E* values decrease with decreased frequency.

Analysis of the phase angle results which is determined from the Dynamic Modulus test was performed to further confirm the findings as shown by the E* Isotherms. These results are discussed further within this chapter.

Figure 4.8 Dynamic Modulus Test Results
4.2.5 Dynamic Modulus (E*) Ratio

Figure 4.38 indicates the comparison between all HMA mixtures evaluated in this study based on the dynamic modulus (E*) at the various test temperatures computed at 5 Hertz (Hz). For the purpose of comparison, the E* values calculated at various test temperatures for the 76CO HMA mixture was considered as the unit value (i.e. E* = 1.0). To illustrate this concept, the E* values for the 64RAP40 and 76CO mixtures at 54.4 °C and 5 Hz were 109.5 Ksi and 62.4 Ksi respectively.

![Figure 4.9 Dynamic Modulus Isotherms](image)

The E* ratio is calculated as $109.5 \div 62.4 = 1.75$ for the 64RAP40 HMA mixture. Any mixtures exhibiting an E* ratio greater than 1.0 has greater stiffness than the 76CO mixture. It is
shown in table 4.10 that the E* ratio for the 70CO mixture is less than the 76CO HMA mixture. This indicates that this mixture is more susceptible to permanent deformation than the 76CO HMA mixture at all temperatures tested. As expected the 64CO mixture at the intermediate and high temperatures (37.8 °C and 54.4 °C respectively) is more susceptible to rutting (i.e. permanent deformation) than the 76CO mixture as can be seen by the reported E* ratio. The 76CRM and 64RAP40 mixtures exhibit greater stiffness at all temperature ranges as compared to the 76CO HMA mixture. At the low and service temperature ranges (4.4 °C and 25 °C) the 76RAP 15 has an E* value less than 1.0 and at the high and intermediate temperatures the 76RAP values are greater than the E* values for the 76CO mixture. Figure 4.10 shows that at the intermediate and high temperature levels that the final ranking of the mixtures is as you would expect for the asphalt cement binders used in this study. The rankings indicate that the 64CO mixture which utilizes a PG 64-22 AC is more susceptible to rutting at both temperatures. At these higher temperatures the asphalt cement binders are leaving the visco-elastic range of their respective material and are becoming more viscous. At these high temperatures the final outcome is predominantly based on the stiffness of the asphalt binders at these temperatures as shown by the E* values.
4.2.6 Phase Angle Test Results

Figure 4.11 indicates the graphical representation of the phase angle mean results with respect to the Dynamic Modulus values for all six HMA mixtures considered in this study. The CV of the samples tested ranged from 7 to 18 percent. As shown in this figure the phase angle for the various materials tested are plotted in the arithmetic scales while the E* values are plotted in the logarithmic scale. This figure shows the phase angle for all materials evaluated in this study increases with an increase in temperature and a decrease in frequency. Then at some point in time the phase angle peaks and then declines as the temperature increases further and the frequency continually decreases. It can be noticed from figure 4.11 that the phase angle values initially increased with an increase in the temperature, reached a peak, and afterwards started to decrease as the temperature further increased.
Figure 4.12 presents the phase angle test results at the various temperatures and frequencies for all HMA mixtures considered in this study. This figure indicates that at low temperature (4.4 °C) the phase angle increases as the frequency decreases. In addition as the temperature increases so does the measured phase angle. At 4.4 °C the phase angles are inclined which indicates that the asphalt cement binder is predominately affecting the characteristics of the HMA mixtures. As can be seen for the test temperature of 25 °C, the 64CO and 70CO HMA mixtures starts to increase and reaches a peak and then decreases as the frequency continues to decrease. It appears that the peak phase angle occurs at 5 Hz at this temperature. It is noted that the phase angle for the other HMA mixture types at this temperature is still predominately being affected by the asphalt cement binders used. At the test temperature of 37.8 °C, figure 4.12 shows that all HMA mixtures with the exception of the 64RAP40 mixture has reached a peak.
phase angle at 5Hz before making the downward trend. The 64RAP40 HMA mixtures phase angle is still being affected by the asphalt cement binder. At 54 °C, the phase angle for all mixtures is decreasing with a decrease in frequency. This trend is opposite of the behavior noted at the low temperature of 4.4 °C. This behavior characteristic indicates that the phase angle is predominately affected by the aggregate structure. It is noted that the shift in this behavior was observed at 25 °C.

Figure 4. 12 Phase Angle vs. Mix Type Relationship
4.2.7 Flow Number Test (Repeated Load Permanent Deformation Test) Results

The flow number test is used to determine the permanent deformation characteristic of hot mix asphalt mixtures. The flow number is defined as the number of repetitions corresponding to the minimum rate of change in permanent strain under repeated loading conditions. It is the starting point, or cycle number, at which tertiary flow occurs on a cumulative permanent strain curve generated during the test. Therefore, the higher the flow number value, the better the mixture resists permanent deformation.

Figure 4.13 presents the Flow Number test results for the six HMA mixtures considered in this study. It is shown that the HMA mixtures, 76CO and 76RAP 15, which contain a SBS modified asphalt cement binder had the highest Flow Number values and therefore are the most rut resistant for the mix types evaluated by this test. The 64RAP40 HMA mixture was in the second group and the 64CO mixture containing an unmodified PG64-22 asphalt cement binder was the least resistant to permanent deformation.

![Flow Number vs. Mix Type](image_url)

Figure 4. 13 Flow Number vs. Mix Type
4.2.8 Evaluation of Rutting and Fatigue Factors from E* Tests

A HMA mixture propensity to resist permanent deformation (rutting) and fatigue cracking can be characterized by using the dynamic modulus test results from various temperatures and frequency. The rutting factor is defined as E*/Sinδ, where δ is the phase angle, at a particular temperature and frequency. A loading frequency of 5Hz and test temperature of 54.4 °C was used for computation of the rutting factor, E*/Sinδ in this study [Witczak et al., 2002]. For mixtures to be rut resistant and exhibit higher stiffness necessitates a higher E* value and a lower phase angle. The higher the rutting factor value indicates a mixture greater resistance to permanent deformation.

Figure 4.14 shows the rutting factor values for all mix types evaluated in this study. It clearly indicates that the 64RAP40 mixture has the greatest resistance to rutting. This can be contributed to the high RAP content (40%) used in this mixture type. It is noted that there is a grouping of similar results for the 76CO, 76CRM, and 76RAP15 HMA mixture types. As indicated in figure 4.14 the 64CO mixture has the least resistance to rutting as would be expected because of the asphalt cement binder utilized in this HMA mixture.

To determine a mixtures resistance to fatigue cracking, a parameter termed fatigue factor is calculated from dynamic modulus test results at a given frequency and test temperature. The test temperature of 25 °C and a loading frequency of 5 Hz were selected for this study [Witczak et al., 2002]. By definition the fatigue factor is calculated as E*(Sinδ), where δ is the phase angle, at the selected temperature and frequency. For a mixture to resist fatigue cracking its corresponding E* value should be lower as well as the phase angle at the in-service temperature of 25 °C. The lower the fatigue factor value indicates the mixtures performance against fatigue cracking.
Figure 4.15 indicates the fatigue factor values for all mix types evaluated in this study. There are three distinct groups as shown in figure 4.15. The first grouping is the 70CO HMA mixture that indicates this mixture as being the best in fatigue cracking resistance of the six mixtures evaluated in this study. The second grouping that indicates similar results is the 76CO, 76CRM, and 76RAP15 HMA mixtures. The last groups, which exhibit the highest fatigue factor values and therefore are the least resistant to fatigue cracking, are the 64CO and 64RAP40 mixtures. This can be contributed to the base asphalt binder cement (PG 64-22) being more viscous and less stiff at the elevated temperature of 54.4 °C.

![Graph showing fatigue factor values for different mix types](image)

Figure 4.14 Rutting factor, E*/Sinδ @ 5Hz, 54.4 °C
4.2.9 Loaded Wheel Tracking Test Results

Figure 4.16 indicates the average rut depth of the six mixtures evaluated in this study. The specimens rut depth is continuously measured and recorded for 20,000 passes unless the specimen attains more than 20.0 mm of rutting in which the testing is then terminated. The average rut depth reported in figure 4.16 is the mean rut depth after 20,000 passes of the LWT. Mixtures with an average rut depth less than 6.0 mm after 20,000 passes are considered acceptable. As shown in figure 4.16 the 64CO and 64RAP40 HMA mixtures that utilize PG 64-22 asphalt cement failed the acceptable rutting criterion. However, it is noted that the 64RAP40 mixture was borderline failing with a measured rut depth of 6.1 mm. All other mixtures tested passed the maximum rut depth requirement (6.0 mm).
4.2.10 Correlation between Laboratory Test Results

This section presents the correlation of test results from various laboratory tests evaluated in this study. A linear regression statistical analysis was applied to determine the level of relationships between laboratory test parameters. In addition, the coefficient of determination, $R^2$, was computed to measure the goodness of fit.

4.2.10.1 Correlation between $J_C$ and DCSE Test Results

Figure 4.17 indicates the correlation between the Semi-Circular Bend Test and the Dissipated Creep Strain Energy test results. As expected there is a fair correlation between the Dissipated Creep Strain Energy and Semi-Circular Bend test parameters as noted by the coefficient of determination, $R^2$, value of 0.68. It is also shown in figure 4.17 by the linear regression line that as the DCSE values increase, the $J_C$ values also increase.

Figure 4. 16 LWT Rutting Results
4.2.10.2 Correlation between Modified Lottman and Semi-Circular Bend Test Results

Figure 4.18 illustrates the correlations between the Modified Lottman Test and the Semi-Circular Bend Test as measured by %TSR and \( J_c \) respectively. As shown in figure 4.18 there appears to be a fair correlation as indicated by the R\(^2\) value between the Modified Lottman test results as measured by %TSR and the Semi-Circular Bend Test, \( J_c \) results. It is indicated that as the %TSR increases the \( J_c \) values also increase. This would appear to be logical since the Modified Lottman test is a measure of a mixtures susceptibility to moisture damage which is highly dependent upon the HMA mixtures adhesion and cohesive properties which is also the case of the Semi-Circular Bend Test.
4.2.10.3 Correlation between Fatigue Factor and DCSE Test Results

Figure 4.19 shows the correlation between the Fatigue Factor and Dissipated Creep Strain Energy laboratory test results. This figure indicates that there was not a strong correlation between these performance characteristics for the mixtures evaluated in this study. However figure 4.19 does show a trend in these properties. It is illustrated that as the DCSE increases the Fatigue Factor decreases. This is logical since higher DCSE values are desirable for crack resistance whereas lower Fatigue Factor values are desirable.
4.2.10.3 Correlation between Fatigue Factor and Semi-Circular Bend Test Results

Figure 4.20 indicates the correlation between the Fatigue Factor and the Semi-Circular Bend Test performance characterization laboratory test results for the HMA mixtures evaluated in this study. This figure shows that there was a fair correlation between the Fatigue Factor and SCB test results. It is noted that figure 4.20 does indicate a trend in these parameters. It is illustrated in figure 4.19 that as the $J_C$ increases the Fatigue Factor decreases. This is desirable trend since higher $J_C$ values indicate a HMA mixtures stronger propensity for crack resistance whereas lower Fatigue Factor values are desirable for resistance to cracking.
4.2.11 Comparison of Statistical Ranking of HMA Mixtures

Tables 4.5 summarizes the statistical ranking of several of the laboratory performance test results for the HMA mixture types considered in this study. The evaluation of the HMA mixtures laboratory performance in this study included durability, permanent deformation, and fatigue resistance. Durability performance characteristic was measured by the Modified Lottman test. The mixtures ability to resist deformation was characterized by the Flow Number and dynamic modulus test as measured by the rutting factor. The mixtures fatigue resistance was measured through the DCSE, SCB, and dynamic modulus test as reported by the fatigue factor calculation. However, statistical analysis was based only on two mixture performance criteria namely: 1) fatigue resistance and 2) permanent deformation. In addition the results reported in
this analysis is the DCSE, fatigue factor, rutting factor, and flow number tests because the SCB and Modified Lottman tests numbers were limited and did not lend themselves to statistical analysis for this study.

The laboratory performance data was statistically analyzed using the analysis of variance (ANOVA) procedure. More specifically a multiple comparison procedure with a risk level of 5 percent was performed on the laboratory test results. The statistical results of each grouping is reported with the letters A, B, C, D, and so forth. The letter A is assigned to the best performing HMA mixture followed by the other letters in appropriate order. A double (or more) letter designation, such as A/B (or A/B/C), indicates that in the analysis the difference in the mixture performance is not clear-cut, and that the mixture performance is close to either group.

It is indicated in table 4.5 that HMA mixtures containing unmodified asphalts (64CO and 64RAP40) ranked lowest in regard to resistance to fatigue cracking. Generally, HMA mixtures containing SBS modified asphalt cements (70CO and 76CO), crumb rubber modified (76CRM), and 15 percent RAP (76RAP15), ranked the best in fatigue resistance. In terms of permanent deformation (i.e. rutting), mixtures 76RAP15, 64RAP40, and the 76CO SBS modified HMA mixture performed the best in resistance to permanent deformation as shown in table 4.5. The HMA mixture (64CO) containing the unmodified asphalt cement performed worst and is the most susceptible to permanent deformation for the mixtures evaluated in this study. It is also noted that in general the 76CO HMA mixture ranked highest in all tests evaluated.

The tests evaluated and presented were selected to capture the laboratory performance of the HMA mixtures studied. However the test results were not consistent and did not clearly rank the mixtures. The LWT and FN tests which are used for checking a mixtures resistance for permanent deformation was not clear cut. This may be due to the fact that the LWT samples are tested in confinement whereas the FN test is tested in an unconfined mode. In addition the
Modified Lottman and $J_C$ tests were inconsistent. A mixtures adhesion and cohesive behavior is important in both of these tests. In one test, Modified Lottman, the 64RAP40 shows good properties. However, the $J_C$ test the mean values are low for this mixture type. If the Modified Lottman indicated good adhesive and cohesion properties then the $J_C$ values should have been higher than the reported value.

Table 4.5 Statistical Ranking of Mixtures Fatigue and Rutting Characteristics

<table>
<thead>
<tr>
<th>Property</th>
<th>Fatigue Performance Characteristic</th>
<th>Permanent Deformation Performance Characteristic</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Fatigue Factor ($E^*\sin\delta$)</td>
<td>DCSE</td>
</tr>
<tr>
<td>Aging Criterion</td>
<td>Un-aged</td>
<td>Aged</td>
</tr>
<tr>
<td>Mixture Type</td>
<td>Mean</td>
<td>Rank</td>
</tr>
<tr>
<td>64CO</td>
<td>340.5</td>
<td>B</td>
</tr>
<tr>
<td>70CO</td>
<td>262.9</td>
<td>A</td>
</tr>
<tr>
<td>76CO</td>
<td>296.5</td>
<td>A/B</td>
</tr>
<tr>
<td>76CRM</td>
<td>316.8</td>
<td>B</td>
</tr>
<tr>
<td>76RAP15</td>
<td>300.4</td>
<td>A/B</td>
</tr>
<tr>
<td>64RAP40</td>
<td>330.2</td>
<td>B</td>
</tr>
</tbody>
</table>
CHAPTER 5 CONCLUSIONS AND RECOMMENDATIONS

5.1 Summary and Conclusions

This study characterized the laboratory performance of conventional HMA mixtures and mixtures containing high RAP content and waste tire crumb rubber/additives through their fundamental engineering properties. A comparative laboratory evaluation of six 19-mm nominal maximum aggregate size (NMAS) Level 2 Superpave HMA mixtures meeting LADOTD specification were considered in this study. Three mixtures are classified as conventional mixtures that contain an unmodified asphalt cement binder and mixtures containing styrene-butadiene-styrene polymer modified asphalt cement meeting Louisiana specifications for PG 64-22, PG 76-22M, and PG 76-22M respectively. The fourth mixture contained 30 mesh crumb rubber (CR) plus additives blended (wet process) with a PG 64-22 asphalt cement binder which yielded a PG 76-22 and no RAP. The fifth mixture contained 15 percent RAP and PG 76-22M asphalt cement binder. The final mixture contained 40 percent RAP, 30 mesh crumb rubber and additives blended (dry process) with a PG 64-22 asphalt cement binder. The CR and additives were introduced to the mixture at a rate of ten percent by total weight of asphalt cement binder. To evaluate performance, physical and rheological tests were evaluated on asphalt binders and hot mix asphalt mixtures (HMA). The Rolling Thin Film Oven (RTFO) test, Pressure Aging Vessel (PAV) test, Rotational Viscometer (RV) test, Dynamic Shear Rheometer (DSR) test, and Bending Beam Rheometer (BBR) tests were performed on the asphalt cement binders to characterize their physical and rheological properties. In addition to asphalt cement rheology characterization, HMA mixture performance and characterization tests namely, Semi-Circular Bend (SCB) test, Dissipated Creep Strain Energy (DCSE) test, Simple Performance Tests (Dynamic Modulus, E*, Flow Number, FN), and Modified Lottman test were conducted to define permanent deformation (stability) and the fatigue life (durability) of HMA mixtures considered.
in this study. A statistical analysis was performed on the results of these tests to determine if there were any significant differences in the fundamental material characterization properties of the HMA mixtures considered in this study. Based on the objectives of this study, the following conclusions are drawn:

- The addition of the crumb rubber additives softened the blended AC for the 64RAP40 HMA mixture as determined by Rheology testing of the asphalt cement extracted from the mixture. The blended AC for the 64RAP40 HMA mixture that contained PG 64-22, high RAP content (40 percent), and crumb rubber additives graded as a PG 70-28 asphalt cement.

- It is clearly shown that the addition of the crumb rubber additives with RAP had a positive influence in the asphalt cement binder Rheology. This can be contributed to the use of the absorptive properties of crumb rubber carrying rejuvenating products back into the HMA mixture allowing the RAP binders to be softened in lieu of the original binders being stiffened by the effect of the aged RAP binders.

- The HMA mixtures considered in this study was subjected to the Modified Lottman test which quantifies the HMA mixtures sensitivity to moisture damage. The mixtures containing utilizing unmodified PG 64-22 failed this test whereas the mixes containing polymer modified asphalt cements (70CO, 76CO) passed as expected. The 64RAP40 mixture that contained unmodified PG 64-22 asphalt cement binder passed the Modified Lottman test. This indicates the CR additives had a positive influence in the asphalt cement binder’s ability to increase adherence to the aggregate structure.

- Fracture resistance as measured by the Dynamic Creep Strain Energy test indicates that the 64RAP40 HMA mixture ranked last in its ability to resist fracture while the 76CO mixture had the highest fracture resistance. This is attributable to the type binders
utilized in the respective HMA mixtures. The 76CO HMA mixture utilized PG 76-22M polymer modified asphalt cement whereas the 64RAP40 contained unmodified PG 64-22 AC which is less stiff than the PG 76-22M material.

- Fracture resistance as measured by the Semi-Circular Bend test confirmed the DCSE results in regards to the fracture resistance of the 64RAP40 mixture. However the SCB results ($J_c = 0.65 \text{ Kj/m}^3$) for this mixture should be expected to exhibit good fracture resistance based on a previous study by Mohammad et al., 2004.

- Dynamic Modulus tests used to evaluate the visco-elastic response of HMA mixtures indicate that as the frequency increases the $E^*$ values also increase and as the temperatures increase the $E^*$ values decrease. In addition, at 4.4 °C the $E^*$ isotherms show that the HMA mixtures are in the visco-elastic range and are primarily affected by the asphalt cement. As the temperatures increase the isotherms shape changes to a non-linear shape which that represents the non-linear response which is indicative of the mechanical response caused by the aggregate structure of the HMA mixture overwhelming the viscous influence of the asphalt cement binder.

- Analysis of the phase angle test results as determined from the Dynamic Modulus test confirms the $E^*$ isotherm findings. The phase angle results indicate that at 4.4 °C all mixtures tested were in the visco-elastic range. At 25 °C, the 64CO and 70CO show the non-linear response indicating the aggregate structure has taken control of these mixtures properties. At the test temperature of 37.8 °C all mixtures with the exception of the 64RAP40 mixture exhibit the non-linear response. It was shown that the 64RAP40 HMA mixtures characteristic was still in the visco-elastic range indicating that the asphalt cement binder was still the contributing factor and that the non-linear response did not occur until the 54 °C test temperature.
• The HMA mixtures resistance to permanent deformation (i.e. rutting) as determined by the E* ratio and rutting factor, E*/Sinδ, as measured from the Dynamic Modulus test indicate that the 64RAP40 HMA mixture has the greatest propensity to resist rutting.

• In regard to fatigue resistance as determined from the fatigue factor, G*(Sinδ), the 64RAP40 and 64CO HMA mixtures have the least resistance to fatigue cracking. Both mixtures utilized unmodified PG 64-22 and therefore these results can be contributed to the asphalt cement binder stiffness properties of the PG 64-22 which are less than all other asphalt cement binders utilized in this study.

5.2 Future Research Recommendations

• The results of this study clearly show the benefits of utilizing the absorptive properties of crumb rubber to carry rejuvenating type products into a HMA mixture that contains a high RAP content. The outcome of this study indicates that the crumb rubber additives can be added as part of the aggregate portion (dry feed) during HMA production in lieu of incorporation of the crumb rubber as part of the asphalt cement binder (i.e. wet blending).

• The addition of the crumb rubber additives had a positive influence on the extracted RAP binder properties as determined in this study.

• The use of crumb rubber additives as demonstrated in this study clearly indicates promise. However, since this was a limited study, further investigation utilizing several RAP sources and asphalt cement sources should be conducted. In addition, life cycle cost analysis is to be included to indicate the economic benefit in utilizing high RAP and recycled products such as crumb rubber.
- Optimization of the crumb rubber additives to maximize a mixtures performance, i.e. fatigue resistance or permanent deformation, is necessary.
REFERENCES


Chehovits, J., and Hicks, R. "Mix Design Procedures." Session 10.0, Crumb Rubber Modifier Workshop Notes, Federal Highway Administration, Office of Technology Applications, March 1993.


FHWA, “ISTEA Section 1038 and Section 325 of HR 2750,” Memorandum from Executive Director, Federal Highway Administration, Washington, D.C., November 2, 1993.


“LTRC Annual Research Program”, Louisiana Transportation Research Center (LTRC), Baton Rouge, LA, June, 1996, pp. 23.


APPENDIX A:

AGGREGATE GRADATIONS AND MATERIAL PROPERTIES
### Table A1: Sieve Analysis of Aggregates (percent passing)

<table>
<thead>
<tr>
<th>Metric (U.S.) Sieve</th>
<th>Aggregate Type</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td># 67 LS</td>
</tr>
<tr>
<td>37.5 mm (1½ in)</td>
<td>100.0</td>
</tr>
<tr>
<td>25 mm (1 in)</td>
<td>100.0</td>
</tr>
<tr>
<td>19 mm (¾ in)</td>
<td>85.8</td>
</tr>
<tr>
<td>12.5 mm (½ in)</td>
<td>45.3</td>
</tr>
<tr>
<td>9.5 mm (⅛ in)</td>
<td>24.5</td>
</tr>
<tr>
<td>4.75 mm (No. 4)</td>
<td>7.0</td>
</tr>
<tr>
<td>2.36 mm (No. 8)</td>
<td>4.1</td>
</tr>
<tr>
<td>1.18 mm (No. 16)</td>
<td>3.0</td>
</tr>
<tr>
<td>0.6 mm (No. 30)</td>
<td>2.4</td>
</tr>
<tr>
<td>0.3 mm (No. 50)</td>
<td>2.2</td>
</tr>
<tr>
<td>0.15 mm (No. 100)</td>
<td>1.9</td>
</tr>
<tr>
<td>0.075 mm (No. 200)</td>
<td>1.8</td>
</tr>
</tbody>
</table>

### Table A2: Aggregate Consensus Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Test Protocol</th>
<th>Specification</th>
<th>HMA Mixtures</th>
</tr>
</thead>
<tbody>
<tr>
<td>CAA, %</td>
<td>ASTM D 5821</td>
<td>95+, 2 face</td>
<td>100</td>
</tr>
<tr>
<td>FAA, %</td>
<td>AASHTO T 304</td>
<td>45+</td>
<td>46</td>
</tr>
<tr>
<td>F&amp;E, %</td>
<td>ASTM D 4791</td>
<td>10-, 5:1 ratio</td>
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</tr>
<tr>
<td>SE, %</td>
<td>AASHTO T 176</td>
<td>45+</td>
<td>62</td>
</tr>
</tbody>
</table>

Note: CAA: Coarse Aggregate Angularity, FAA: Fine Aggregate Angularity, F&E: Flat and Elongated Particles, SE: Sand Equivalent
Table A3: Tire Crumb Rubber Certificate of Analysis

<table>
<thead>
<tr>
<th>Screen Size</th>
<th>Sieve Analysis (% passing)</th>
<th>Chemical Analysis</th>
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<tr>
<td>30 mesh*</td>
<td>100.0</td>
<td>Acetone Extract 12.09%</td>
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<tr>
<td>40 mesh</td>
<td>87.0</td>
<td>RHC 49.03%</td>
</tr>
<tr>
<td>50 mesh</td>
<td>45.7</td>
<td>Carbon Black 31.85%</td>
</tr>
<tr>
<td>60 mesh</td>
<td>31.5</td>
<td>Ash 7.021%</td>
</tr>
<tr>
<td>80 mesh</td>
<td>16.2</td>
<td>Moisture Content 0.65%</td>
</tr>
<tr>
<td>Pan</td>
<td>0.0</td>
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</table>

Reference: PolyVulc, Lot 236-A, 6-06-08

*Trace retained on 30 Mesh
APPENDIX B:

SIMPLE PERFORMANCE TEST RESULTS FOR ASPHALT MIXTURES
<table>
<thead>
<tr>
<th>Temperature</th>
<th>Sample ID</th>
<th>Air Voids (%)</th>
<th>E* (Ksi) values at different frequencies (Hz)</th>
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</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>25 Hz</td>
</tr>
<tr>
<td>4.4 °C</td>
<td>5</td>
<td>6.9</td>
<td>2927</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>7.2</td>
<td>3280</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>6.6</td>
<td>3138</td>
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<td>Average</td>
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<td>3115</td>
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<td>Stdev</td>
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<td>CV%</td>
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<td>4.3</td>
<td>5.7</td>
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<tr>
<td>25.0 °C</td>
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<td>6.9</td>
<td>1045</td>
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<td></td>
<td>6</td>
<td>7.2</td>
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<td></td>
<td>8</td>
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<td>Average</td>
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<td>CV%</td>
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<td>Average</td>
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<td>Stdev</td>
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<tr>
<td>CV%</td>
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<tr>
<td>54.4 °C</td>
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<td>6</td>
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<td>8</td>
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<tr>
<td>Stdev</td>
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<tr>
<td>CV%</td>
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<td>4.3</td>
<td>19.5</td>
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Note: Stdev: Standard Deviation
%CV: Coefficient of Variance (%)
Table B2: Dynamic Modulus (E*) Test Results, 70CO HMA

<table>
<thead>
<tr>
<th>Temperature</th>
<th>Sample ID</th>
<th>Air Voids (%)</th>
<th>E* (Ksi) values at different frequencies (Hz)</th>
<th>25 Hz</th>
<th>10 Hz</th>
<th>5 Hz</th>
<th>1 Hz</th>
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<tbody>
<tr>
<td>4.4 °C</td>
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<td>2053</td>
<td>1869</td>
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<td></td>
<td>2</td>
<td>6.4</td>
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<td>1721</td>
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<td></td>
<td>Average</td>
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<tr>
<td></td>
<td>Stdev</td>
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<td></td>
<td>CV%</td>
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<td>7.1</td>
<td>7.5</td>
<td>8.0</td>
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<td>1024</td>
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<tr>
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<td>901</td>
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Note: Stdev: Standard Deviation
%CV: Coefficient of Variance (%)
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Note: Stdev: Standard Deviation
%CV: Coefficient of Variance (%)
Table B4: Dynamic Modulus (E*) Test Results, 76CRM HMA

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Note: Stdev: Standard Deviation
%CV: Coefficient of Variance (%)
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Note: Stdev: Standard Deviation
%CV: Coefficient of Variance (%)
Table B6: Dynamic Modulus (E*) Test Results, 64RAP40 HMA

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| 25.0 °C     | 1         | 7.3           | 1295  | 1103  | 971  | 686  | 587     | 383    |
|             | 3         | 7.4           | 1219  | 1031  | 893  | 621  | 519     | 320    |
|             | 5         | 7.1           | 1278  | 1074  | 948  | 654  | 554     | 327    |
|             | Average   | 7.3           | 1264  | 1070  | 937  | 654  | 553     | 343    |
|             | Stdev     | 0.15          | 39.8  | 36.3  | 40.1 | 32.8 | 33.9    | 34.5   |
|             | CV%       | 2.1           | 3.1   | 3.4   | 4.3  | 5.0  | 6.1     | 10.0   |

| 37.8 °C     | 1         | 7.3           | 605   | 482   | 397  | 227  | 179     | 98     |
|             | 3         | 7.4           | 554   | 448   | 370  | 208  | 157     | 79     |
|             | 5         | 7.1           | 609   | 469   | 379  | 207  | 158     | 80     |
|             | Average   | 7.3           | 589   | 466   | 382  | 214  | 165     | 86     |
|             | Stdev     | 0.15          | 30.6  | 17.1  | 13.6 | 11.5 | 12.4    | 10.9   |
|             | CV%       | 2.1           | 5.2   | 3.7   | 3.6  | 5.4  | 7.5     | 12.7   |

| 54.4 °C     | 1         | 7.3           | 218   | 148   | 105  | 54   | 30      | 19     |
|             | 3         | 7.4           | 243   | 167   | 120  | 55   | 42      | 23     |
|             | 5         | 7.1           | 203   | 136   | 99   | 47   | 35      | 18     |
|             | Average   | 7.3           | 221   | 150   | 109  | 52   | 39      | 20     |
|             | Stdev     | 0.15          | 20.0  | 15.5  | 10.4 | 4.3  | 3.9     | 2.3    |
|             | CV%       | 2.1           | 9.0   | 10.3  | 9.5  | 8.3  | 10.1    | 11.7   |

Note: Stdev: Standard Deviation
%CV: Coefficient of Variance (%)
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Note: Stdev: Standard Deviation
%CV: Coefficient of Variance (%)
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*Note: Stdev: Standard Deviation
%CV: Coefficient of Variance (%)
### Table B9: Phase Angle Test Results, 76CO HMA

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**Note:** Stdev: Standard Deviation  
%CV: Coefficient of Variance (%)
### Table B10: Phase Angle Test Results, 76CRM HMA

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*Note: Stdev: Standard Deviation
%CV: Coefficient of Variance (%)*
Table B11: Phase Angle Test Results, 76RAP15 HMA

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Note: Stdev: Standard Deviation
%CV: Coefficient of Variance (%)
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Note: Stdev: Standard Deviation  
%CV: Coefficient of Variance (%)

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Note: Stdev: Standard Deviation
%CV: Coefficient of Variance (%)
APPENDIX C:

SAMPLE SAS PROGRAM
Sample SAS Program used for Statistical Grouping / Ranking

dm 'log;clear;output;clear';
options nodate nocenter nonumber;
title 'CTB';
data moduli;
  input type sampl_no modulus @@;
cards;
  1 1 310.5624709
  1 2 338.0773422
  1 3 372.8630293
  2 1 251.0052439
  2 2 281.7199082
  2 3 255.9572303
  3 1 307.2291634
  3 2 312.8247739
  3 3 269.5625729
  4 1 317.2276052
  4 2 307.2576006
  4 3 325.9591168
  5 1 246.7
  5 2 275.9
  5 3 258.7489787
  6 1 328.3839201
  6 2 316.1724378
  6 3 329.7549395
;
proc print;
proc glm;
classes type;
model modulus=type;
means type;
means type / lsd;
run;
quit;
VITA

Samuel B. Cooper, Jr., was born in June 1956, in San Antonio, Texas. Mr. Cooper received a Bachelor of Science in Civil Engineering degree from Louisiana State University (LSU), Baton Rouge, Louisiana, in May 1980. He is a licensed Professional Engineer in Louisiana and Florida and obtained licensure in 1984 and 1989 respectively. Cooper has 28 years of experience in public and private industry in the areas of construction, hot mix asphalt mixture research, materials supplier, and asphalt manager for a major road building contractor. Currently Mr. Cooper is the Associate Director of Technology Transfer and Training at the Louisiana Transportation Research Center as well as a student at LSU. In the spring of 2006, Mr. Cooper rejoined the academic ranks to fulfill a lifelong dream of obtaining his advanced degrees in civil engineering. He expects to receive the degree of Master of Science in Civil Engineering (MSCE) in December 2008.