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LABORATORY CHARACTERIZATION OF RECYCLED AND WARM MIX ASPHALT FOR ENHANCED PAVEMENT APPLICATIONS

A DISSERTATION APPROVED FOR THE
SCHOOL OF CIVIL ENGINEERING AND ENVIRONMENTAL SCIENCE

BY

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DEDICATION

"To my parents."

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ABSTRACT

Hot-mix asphalt (HMA) is the most widely used paving material in the U.S. More than 90 percent of U.S. pavements are paved with asphalt. Each year, over 550 million tons of HMA are produced and used for construction of flexible pavements. Over the past two decades many transportation agencies, asphalt producers and pavement construction companies have taken major initiatives to implement green paving technologies. Saving energy during asphalt production and increased use of reclaimed asphalt materials are important elements of such initiatives. Consequently, there is an increasing need for characterization of green pavements in order to address the concerns over their performance. Furthermore, for implementation of Mechanistic-Empirical Pavement Design Guide (M-EPDG) for the aforementioned green pavements, important input parameters are needed to be determined.

This study seeks to advance the knowledge base in two areas of green pavements: characterization of warm mix asphalt (WMA) that uses significantly less energy and produces less emission than HMA, and characterization of HMA containing higher amounts of reclaimed asphalt pavement (RAP) than normally used in Oklahoma. Different types of WMA technologies were evaluated in this study. The reclaimed asphalt materials studied herein consisted of RAP and reclaimed asphalt shingles (RAS). This study also aims to develop important laboratory data that can be used for the local calibration of the new Mechanistic-Empirical Pavement Design Guide (M-EPDG) for the aforementioned green pavements. Another important aspect of this study was to mechanistically evaluate the effect of using different WMA additives, RAP and

RAS on the moisture-induced damage potential, which is known as one of the most common and complex problems in flexible pavements.

Therefore, this study was carried out in two major phases: in Phase 1, the laboratory performance and M-EPDG input parameters of the following mixes were evaluated:

- WMA mixes
    - Advera® WMA surface course mix,
    - HMA mix corresponding to Advera® surface course mix,
    - Evotherm® WMA surface course mix,
    - HMA mix corresponding to Evotherm® surface course mix,
    - Evotherm® WMA base course mix,
    - HMA mix corresponding to Evotherm® base course mix.
- Mixes containing RAP and RAS
    - HMA surface course mix without RAP or RAS (control surface mix),
    - HMA surface course mix with 10% RAP,
    - HMA surface course mix with 25% RAP,
    - HMA surface course mix with 20% RAP and 5% RAS,
    - HMA base course mix without RAP or RAS (control base course mix),
    - HMA base course mix with 25% RAP,
    - HMA base course mix with 40% RAP,
    - HMA base course mix with 20% RAP and 5% RAS.

A wide range of laboratory tests were conducted to evaluate the performance of the abovementioned mixes and to obtain the M-EPDG input parameters. These tests included dynamic modulus, creep compliance, fatigue, moisture damage, and rutting. According to the WMA study, the WMA mixes showed lower dynamic modulus value (lower stiffness) for all combinations of temperatures and frequencies, reduced potential of low temperature cracking, lower fatigue life (a lower number of cycles to fatigue failure), and a higher rutting potential compared with their HMA counterparts. However, a mixed trend of moisture-induced damage potential was observed for WMA and HMA mixes when evaluated using retained tensile strength ratio (TSR) and stripping inflection point (SIP) obtained from the Hamburg wheel tracking test (HWT). In other words, no correlation was found between TSR and SIP values, indicating a mix which passes a TSR test does not guarantee better performance when tested using a HWT. Furthermore, a good correlation was found between inverse rutting rate and dry indirect tensile strength (DITS), indicating a mix with higher DITS would have better rutting resistance. The results from this study reveal that performance of a WMA mix widely depends on the technology and the type of other additives (e.g., anti-stripping agent) used. The study of the mixes containing RAP and/or RAS indicated that the dynamic modulus and creep compliance of the asphalt mixes increase and decrease, respectively, with an increase in amount of RAP and/or RAS used in the mix. Fatigue life was found to increase with increasing RAP content up to 25%, and to decrease when the RAP content exceeded 25%, or when RAS was used in the mix. It should be noted that this conclusion was based on conducting fatigue tests on asphalt mixes with 0%, 25%, and 40% RAP contents. However, the adverse effect of using RAP on fatigue life may start to occur at a RAP content between 25% and 40%. Therefore, for a more accurate determination of the RAP content which maximizes the fatigue life, testing more mixes with smaller increments in RAP content is recommended (e.g., 25%, 30%, 35%, 40%). The Hamburg wheel tracking (HWT) test results showed increased resistance to rutting and moisture-induced damage with an increase in the amount of RAP and/or RAS. However, the TSR test results were not confirmed by the HWT test.

In Phase 2, the moisture-induced damage potential of the asphalt binder-aggregate systems, containing different WMA additives (Advera®, Sasobit®, and Evotherm®) and HMA mixes with different amounts of RAP binder, was evaluated by applying the surface free energy (SFE) method as a mechanism-based approach. In the mechanistic study of the moisture-induced damage potential of WMA, the SFE components of a PG 64-22 asphalt binder with different percentages of WMA-additives, namely Sasobit® (1.0%, 1.5% and 2.0% by the weight of asphalt binder), Advera® (0.25%, 0.30% and 0.35% by the weight of asphalt mix), and Evotherm® (0.25%, 0.50% and 0.75% by the weight of asphalt binder) were measured in the laboratory. The SFE components of the selected aggregates, namely limestone, sandstone, gravel, granite and basalt, were measured in the laboratory, or adopted from literature. The wettability, the work of adhesion, the work of debonding, and energy ratios were estimated to assess the moisture-induced damage potential of combinations of modified asphalt binders and different aggregates. The results indicated that Sasobit® and Advera® are able to reduce the moisture susceptibility of the mixes, but are not recommended to be used with highly acidic aggregates like granite. Evotherm® resulted in the highest increase in wettability, total surface free energy, increased work of adhesion and a reduction in the work of debonding, leading to a better possible aggregate coating with asphalt binder and lower moisture susceptibility with all types of tested aggregates, compared to those of other WMA-additives. Furthermore, TSR tests were conducted on Advera® and Evotherm®-modified and neat (unmodified) asphalt mixes and results were compared with those from the SFE test. It was found that the SFE approach is a better indicator of moisture-induced damage compared to the traditional TSR test. It is expected that the present study will be helpful in understanding the moisture-induced damage potential of the flexible pavements constructed with WMA technologies. In the mechanistic study of moisture-induced damage of mixes containing RAP, the SFE method was applied to evaluate the effects of asphalt binder type, RAP binder, and aggregate type on the moisture-induced damage potential of asphalt mixes. The SFE components (non-polar, acid and base) of a PG 64-22 and a PG 76-28 (polymer-modified) asphalt binder, each blended with different percentages of RAP binder (0%, 10%, 25% and 40%), were measured using a dynamic contact angle (DCA) device. The aggregates included in this study consisted of limestone, rhyolite, sandstone, granite, gravel, and basalt. The SFE components of limestone and rhyolite aggregates were measured using a Universal Sorption Device (USD), while those for the other aggregates (sandstone, granite, gravel, and basalt) were obtained from literature. The energy ratio parameters estimated based on the spreading coefficient, the work of adhesion, and the work of debonding were used to assess the moisture-induced damage potential of different combinations of asphalt binders with different RAP binder contents and aggregates. The results indicated that the acid SFE component of PG 64-22 and PG 76-28 asphalt binders increases with the addition of the RAP binder, while the base SFE component remains almost unchanged with the addition of the RAP binder. Furthermore, the wettability and the work of adhesion of both PG 64-22 and PG 76-28 asphalt binders over different types of aggregates increased with an increase in RAP content (by 25% and more). Based on the energy ratio parameters, it was found that the resistance to moisture-induced damage increased with an increase in RAP content for both PG 64-22 and PG 76-28 asphalt binders and all types of aggregates, specifically at higher RAP amounts. Furthermore, it was found that the higher the total SFE component of the aggregates, the lower the energy ratio parameter values. Therefore, a high total SFE component of aggregate may result in a high moisture-induced damage potential in the mix. It is expected that this study would be helpful in understanding the moisture-induced damage potential of the asphalt mixes, produced with polymer-modified and non-polymer-modified asphalt binders containing RAP. Furthermore, a parameter combining SFE components and mix design proportions was proposed in order to evaluate the moisture-induced damage characteristics of the mixes containing RAP. For this purpose two approaches were pursued: (i) micro-structural analysis of the aggregate-asphalt bond based on the surface energy parameters, and (ii) mechanical testing of asphalt mixes using TSR and HWT. According to approach (i), the SFE (non-polar, acidic and basic) components of the virgin PG 64-22 binder mixed with 0%, 25% and 40% of RAP binder and aggregates, namely limestone, rhyolite, RAP extracted aggregate, were used to determine the composite work of adhesion and composite work of debonding, and composite energy ratios for each combinations of asphalt binder and aggregates. The composite energy ratios were used to assess the moisture-induced damage potential of the mixes containing different percentages of RAP. According to approach (ii), the TSR and HWT test data conducted on asphalt mixes containing different percentages of RAP were used

to evaluate their moisture-induced damage potential. All test methods (SFE, TSR, and HWT) showed that the moisture-induced damage potential decreased with increasing amount of RAP in asphalt mixes. A strong correlation was found to exist between the moisture-induced damage potential predicted using the micro-structural method and laboratory performance tests. It was found that the micro-structural energy approach, as a mechanistic framework, can be successfully used as an indicator of moisture-induced damage potential of the asphalt mixes.

CHAPTER

1

INTRODUCTION

1.1 Green Paving Technologies

Over the past two decades, many transportation agencies, asphalt producers and pavement construction companies have taken major initiatives to implement *green paving technologies* (NAPA, 2011). Saving energy during asphalt production and increased use of reclaimed materials are important elements of these initiatives. Many studies have been conducted and are being conducted in the United States and abroad to find innovative ways to design and construct environmental friendly and durable pavements. Consequently, the hot-mix asphalt (HMA) producers and paving contractors are undergoing phenomenal changes in terms of material characterization, mix design, construction, and maintenance of pavements. The new characterization and test methods are more rigorous, mechanistic, and performance-based. The present study seeks to advance the knowledge base in two areas of green paving: (i) characterization of warm mix asphalt (WMA) that uses significantly less energy than HMA; and (ii) characterization of HMA containing higher amounts of reclaimed materials than normally used in Oklahoma. This study aims to develop important laboratory data that can be used for the local calibration of the new Mechanistic-Empirical Pavement Design Guide (M-EPDG) for the aforementioned green pavements.

HMA is the most widely used paving material in the U.S. More than 90 percent of U.S. pavements are paved with asphalt (NECEPT, 2010). Each year, over 550 million tons of HMA are produced and used for construction of flexible pavements. Rising oil and gas prices spurs development of methods and technologies for reducing fuel consumption and increased use of reclaimed materials. With increased environmental awareness, using WMA and incorporating reclaimed asphalt pavement (RAP) and reclaimed asphalt shingles (RAS) in pavements have been gaining momentum nationally and globally. These efforts are directed at cutting the emissions and making recycling an industry standard. Based on a recent report published by the National Asphalt Pavement Association (NAPA), asphalt is being reclaimed and reused at a rate of over 99 percent (NAPA, 2011). Approximately 20.5 million barrels of asphalt binder were conserved in 2010, by recycling RAP and RAS (NAPA, 2011).

In 2002, NAPA identified a new promising technology, WMA, which was originally developed in Europe. WMA technologies allow a reduction in production and placement temperature; the range of reduction in asphalt temperature may vary from 20 to 55°C depending upon the type of the technology. Lower production and construction temperatures lead to reduced energy costs and greenhouse gas emissions.

In 2009 and 2010, the Federal Highway Administration (FHWA) contracted with NAPA and conducted a survey on the implementation of recycling and energy efficiency techniques in asphalt pavements. This survey introduced RAP, RAS, and WMA as three key areas of implementation by the asphalt paving industry (NAPA, 2011). Use of WMA as a green pavement technology grew by more than 148 percent from 2009 to 2010; a trend which is expected to continue in coming years (NAPA, 2011).

1.1.1 The Research Needs for WMA Mixes

Despite the aforementioned advantages, national concerns focus on durability and performance issues of WMA mixes over time, particularly with respect to their ability to resist moisture-induced damage. Moisture-induced damage has been reported as a major problem for both HMA and WMA in many states, including Oklahoma (Hurley et al., 2010; Wasiuddin et al., 2008; Prowell et al., 2007; Hurley and Prowel, 2006). Moisture-induced damage causes loss of bond between asphalt binder and aggregates in presence of water, namely stripping. Stripping can cause premature failure of asphalt pavements (WSDOT, 2008; Wasiuddin et al., 2007). Another concern over WMA is the possibility of increased rut depths (Hurley et al., 2010) resulting from reduced asphalt binder aging and oxidation due to reduced mix temperature. Since WMA is gaining rapid acceptance many DOTs and highway agencies are motivated to evaluate the performance of WMA and develop relevant specifications. Based on a report published by NAPA (2007), several research needs are identified in the WMA area, the most critical need relates to the development of a protocol for evaluating new WMA technologies. Laboratory performance tests are an important requirement of this protocol (NAPA, 2007). At the local level, WMA technologies should either fit into the local DOTs' specifications or modifications should be made to the current specifications in the light of sufficient laboratory and field performance data, before they can be successfully implemented.

Therefore, there is a need for a comprehensive laboratory study to evaluate the effect of using different WMA technologies on mix performance. Also, in order to implement M-EPDG for WMA mixes produced using local materials, input design parameters need to be calibrated, which require laboratory testing. Furthermore, there is a need for a study to address the moisture-induced damage potential of WMA mixes, using a mechanistic approach, namely the surface free energy (SFE) method.

1.1.2 The Research Needs for Mixes Containing RAP

With increased use of RAP by the asphalt industry, DOTs have realized the necessity of updating their specifications and test protocols, which need more laboratory and field test data on asphalt mixes containing RAP. According to Jones (2008), more than twenty DOTs, such as Texas, Louisiana and Arkansas, allow 30 percent or more RAP in base course and 10 percent or more RAP in surface course. However, many other DOTs including Oklahoma DOT (ODOT) allow up to 25 percent RAP in base course and none in surface course (FHWA, 2009; ODOT, 2009). Based on a national survey conducted by Jones (2008), the major barriers for use of higher percentages of RAP in asphalt mixes include stockpile management issues, binder issues and mix issues. Stockpile management issues include unknown quality of original material, difficulties related to gradation control and processing requirements. Binder issues consist of bumping binder grade, unknown properties of final blend and compaction issues. Mix issues include unknown durability and performance characteristics, additional testing requirements, variability of RAP mixes, and concerns related to early failure. Therefore, extensive laboratory and field studies are needed on the performance of asphalt mixes containing RAP.

1.1.3 The Research Needs for Mixes Containing RAS

The use of RAS in HMA has both economic and environmental benefits. Economically, use of RAS in HMA will reduce the need for the virgin materials, namely asphalt and aggregates (FVD, 2006; Sengoz and Topal, 2005; Foo et al., 1999). RAS contains 19 to 36 percent asphalt binder and 20 to 38 percent ceramic, a source of fine aggregate (CIWMB, 2007; NAHB, 1998). On the environmental side, use of RAS will reduce the consumption of landfill and reduce the use of virgin materials (Sengoz and Topal, 2005). Based on the results of a recent nationwide survey conducted by NAPA (2011), use of RAS (both manufacturers' waste and tear-offs) increased from 702,000 to 1.1 million tons from 2009 to 2010, a 57 percent increase. Assuming that 20 percent binder is contributed by the shingles, this represents 234,000 tons (1.5 million barrels) of asphalt binder conservation (NAPA, 2011).

Several studies show the use of RAS in HMA to be technically feasible (Sengoz and Topal, 2005; Rajib et al., 2000; Foo et al., 1999; NAHB, 1998; Ali et al., 1995; Button et al., 1995; Grzybowski, 1993). In addition to its economic and environmental benefits, other researchers have observed improvement in pavements' mechanical properties with the use of RAS in HMA. Several studies indicate that mixes containing RAS exhibit improvements in rutting resistance, fatigue life, and overall pavement performance compared to conventional asphalt mixes, while the moisture sensitivity of these mixes was not affected (Baaj, 2007; Ali et al., 1995; Grzybowksi, 1993). Considering their potential benefits, use of RAS in asphalt mixes is expected to become an integral part of recycling in the asphalt industry.

Although a majority of researchers report improvements in rutting performance of pavement with increased RAP and RAS content, contradictory results have been reported on the effect of reclaimed asphalt on the fatigue life and thermal cracking of the mixes (Huang et al., 2004; McDaniel and Shah, 2003; McDaniel et al., 2000). Consequently, there is a need to study the effect of using RAS on dynamic modulus, fatigue life and thermal cracking of the mixes with local aggregate origins.

1.2 Objectives

The specific objectives of this study are as following:

- Determine and compare the stiffness of WMA (produced with Advera® and Evotherm® additives) and HMA mixes by conducting dynamic modulus tests at different temperatures and frequencies.
- Assess the low-temperature cracking potential of WMA (produced with Advera® and Evotherm® additives) and HMA mixes with the help of creep compliance test.
- Determine and compare the fatigue life of WMA (produced with Advera® and Evotherm® additives) and HMA mixes using the four- point bending beam fatigue test.
- Evaluate the moisture-induced damage potential of WMA and HMA mixes using retained tensile strength ratio (TSR) and Hamburg wheel tracking (HWT) methods and to rank the mixes based on their performance according to each test method and visual observation.

- Determine and compare the stiffness of asphalt mixes containing RAP and RAS and virgin HMA mixes by conducting dynamic modulus tests at different temperatures and frequencies.
- Assess the low-temperature cracking potential of asphalt mixes containing RAP and RAS and virgin HMA mixes with the help of creep compliance test.
- Determine and compare the fatigue life of asphalt mixes containing RAP and RAS and virgin HMA mixes using the four-point bending beam fatigue test.
- Evaluate the moisture-induced damage potential of asphalt mixes containing RAP and RAS and virgin HMA mixes using TSR and HWT methods and to rank the mixes based on their performance according to each test method and visual observation.
- Determine the SFE components of aggregates and a PG 64-22 asphalt binder with and without different WMA-additives, namely Sasobit®, Advera® and Evotherm®, using a USD and a Dynamic Contact Angle (DCA) DCA tests, respectively.
- Determine wettability, adhesion, debonding and moisture susceptibility potential of PG 64-22 asphalt binder with and without Sasobit®, Advera® and Evotherm® WMA-additives in contact with different types of aggregates.
- Determine the SFE components of aggregates and PG 64-22 and PG 76-28 (polymer-modified) asphalt binders with and without addition of different

amounts of RAP binder (i.e., 0%, 10%, 25%, and 40%), using a USD and a DCA, respectively.

- Evaluate the coating quality and moisture-induced damage potential of mixes containing RAP with different types of aggregates and asphalt binders based on the energy parameters and energy ratios estimated based on wettability, work of adhesion, and work of debonding.
- Determination of micro-structural moisture-induced damage potential of the mixes, accounting for job-mix formula (JMF) of the mixes, RAP content and the SFE components and other interfacial energy parameters of asphalt binder and aggregates.

1.3 Overview of the Current Study

1.3.1 Implementation of M-EPDG for WMA and Mixes Containing RAP and RAS

In recent years, significant efforts have been made by state DOTs to replace empirical pavement designs with the new mechanistic-empirical pavement design guide (M-EPDG). The implementation of the new M-EPDG for green pavements requires mechanistic input parameters for WMA and asphalt mixes containing RAP and RAS. Lack of such data for local materials is a major constraint for DOTs. The new M-EPDG consists of three levels of designs: Level 1, Level 2 and Level 3. Level 1 design provides the highest level of reliability and takes the actual test results as input parameters, while Level 3 uses a number of default values as input and hence has a relatively low level of reliability. The use of a particular hierarchal input level of analysis depends on the amount of information available to the designer and the importance of the project. For designing WMA and asphalt mixes containing RAP and

RAS, attainment of good performance against fatigue, rutting, low temperature cracking, and moisture induced damage, while optimizing the mix proportions is important. In the new M-EPDG, the dynamic modulus of asphalt mixes is a key input parameter which controls the fatigue cracking and rutting resistance of asphalt pavements (Li et al., 2008; AASHTO, 2004). Use of dynamic modulus is recommended at all three levels (Level 1, Level 2, and Level 3) of analysis for predicting performance of flexible pavements. The new M-EPDG models the thermal cracking using the creep and indirect tensile strength test data (Li et al., 2008). Recently, four-point bending beam fatigue test and the Hamburg Wheel Tracking (HWT) test have gained popularity and are turning into standard means for evaluating the fatigue life and rut and moisture damage for asphalt mixes, respectively (Tarefder et al., 2002).

1.3.2 Laboratory Performance Characterization of WMA Mixes

Although there is a wealth of data and information from different studies available in the literature focused on material, constructability and environmental effects of different WMA technologies, the open literature on the effect of WMA technologies on QC/QA-related properties is rather limited (Bistor, 2009; Hossain et al., 2009; Prowell and Hurley, 2007). For example, Hurley et al. (2010) evaluated two types of WMA mixtures produced using Sasobit® and Evotherm® in a field project located in Milwaukee, Wisconsin. Performance of WMA and conventional HMA test sections was compared after these sections were subjected to four months of traffic. Specifically, field performance was compared in terms of volumetric properties of the mixes used, rutting susceptibility, moisture resistance, and dynamic modulus. It was reported that Sasobit® and control HMA mixes performed approximately equally in laboratory testing. Comparatively, Evotherm® mixes resulted in higher rut depths, lower tensile strengths, and lower moduli than the control HMA. Field performance of all three types of test sections constructed using Sasobit®, Evotherm® and control HMA were comparable; no major differences were noticed. In a recent study, Xiao et al. (2010) conducted laboratory tests to compare rut performance of five different types of WMA mixes containing moist aggregates. They used two aggregate moisture contents of 0 and 0.5 percent, two lime contents of 1 and 2 percent, three WMA additives, namely Aspha-min®, Sasobit® and Evotherm®, and three aggregate sources. It was concluded that the WMA mixes with Sasobit® additive exhibit the best rutting resistance. Comparatively, WMA mixes with Aspha-min® and Evotherm® additives generally showed a similar rut resistance as the control HMA, but lower than the WMA mixes with Sasobit®. In a laboratory study, Kvasnak et al. (2009) evaluated the moisture susceptibility of both laboratory and plant produced WMA and HMA mixes. A total of three properties, TSR, absorbed energy ratio, and stripping inflection point were used to assess the moisture damage potential of the mixes. The results indicated that the laboratory produced WMA was more prone to moisture damage than the plant produced mix. Also, it was observed that WMA specimens are more prone to moisture-induced damage than those of the HMA. Most of the WMA samples, however, passed all three moisture susceptibility criteria. A combined field and laboratory study was conducted by Prowell et al. (2007) to evaluate the performance of a WMA mix containing Evotherm®. For this purpose, accelerated test track at the National Center for Asphalt Technology (NCAT) and laboratory rutting-susceptibility tests were conducted using an Asphalt Pavement Analyzer (APA). It was observed that field densities of WMA surface layers were equal to or better than those of HMA layers. TSR tests revealed an increase in moisture damage potential of WMA compared to HMA mixes. Also, field WMA and HMA sections showed excellent rutting performance. APA rutting tests showed similar performance for both mixes. In a similar combined field and laboratory study by Button et al. (2007), supported by the Texas Department of Transportation (TxDOT), a test section was constructed using an Evotherm® mix. HWT tests were conducted on the cores extracted from the WMA pavement after one month of construction. It was observed that all of WMA cores failed the HWT test requirements. However, the control HMA samples generally passed the HWT test requirements. In a laboratory study, Hurley and Prowel (2006) concluded that stiffness, as measured by resilient modulus, of WMA mixes containing Evotherm® does not show any significant difference compared to the control HMA mix. In an earlier laboratory study by Hurley and Prowell (2005), the performance of different warm mix additives, namely Sasobit®, Sasoflex® and Aspha-min® were evaluated. It was found that the use of WMA additives generally improves the rheological properties of modified binders but performance and moisture susceptibility tests on WMA mixes did not produce any consistent conclusion. APA rut tests did not exhibit any significant increase in rutting potential of WMA mixes. However, two other performance tests including HWT and TSR, showed an increase in moisture damage potential.

Different WMA technologies introduced to the pavement industry utilize different physicochemical means to lower the shear resistance of the mix at production and placement temperatures, while maintaining or enhancing the pavement performance. However, some conflicting observations associated with the performance of WMA were reported by Kvasnak et al. (2009) and Button et al. (2007). Therefore, there is a need to develop an approval system and specifications, both at the local and national levels.

In the current study, a wide range of laboratory tests, namely dynamic modulus, creep compliance, fatigue, moisture damage, and rutting was conducted to evaluate the performance of the WMA mixes and to obtain the M-EPDG input parameters. WMA mixes produced using Advera® and Evotherm® additives, and corresponding HMA mixes were tested for this purpose. Specifically, the laboratory performance and M-EPDG input parameters of the following mixes were evaluated:

- Advera® WMA surface course mix,
- HMA mix corresponding to Advera® surface course mix,
- Evotherm® WMA surface course mix,
- HMA mix corresponding to Evotherm® surface course mix,
- Evotherm® WMA base course mix,
- HMA mix corresponding to Evotherm® base course mix.

According to the test results, the WMA mixes showed lower dynamic modulus value (lower stiffness) for all combinations of temperatures and frequencies, reduced potential of low temperature cracking, lower fatigue life (i.e., a lower number of cycles to fatigue failure), and a higher rutting potential compared to their HMA counterparts. However, a mixed trend of moisture-induced damage potential was observed for WMA and HMA mixes, when evaluated using TSR and stripping inflection point (SIP) obtained from a HWT. In other words, no correlation was found between TSR and SIP values, indicating a mix which passes a TSR test does not guarantee better performance when tested using a HWT. Furthermore, a good correlation was found between inverse rutting rate and dry indirect tensile strength (DITS), indicating a mix with higher DITS would have better rutting resistance. The results from this study reveal that performance of a WMA mix widely depends on the technology and the type of other additives (e.g., anti-stripping agent) used.

1.3.3 Laboratory Performance Characterization of Mixes Containing RAP and RAS

Performance characteristics of asphalt mixes containing RAP and RAS have been investigated by several researchers. For example, Mogawer et al. (2011) evaluated the performance of thin-lift mixes incorporating a high RAP content and RAS in the pavement. HMA mixes with 40 percent RAP and 5 percent RAS, and with 35 percent RAP and 5 percent RAS were produced in the laboratory and tested. It was concluded that, based on the dynamic modulus tests, mixes with high RAP content, RAS content, or both exhibited higher stiffness. Also, it was observed that use of RAP, RAS or both reduced the reflective cracking resistance without a negative impact on the resistance to low-temperature cracking. It was concluded that the addition of RAP, RAS, or both improved the mixes' resistance to moisture-induced damage. Johnson et al. (2010) studied the effect of using RAS on the dynamic modulus of pavement. It was concluded that stiffness of the mixes containing RAS was higher as compared to the virgin mixes. Specifically, at low frequencies, stiffness of the mixes containing tear-off RAS was higher as compared to the mixes containing manufacturer's waste RAS at high temperatures. Cascione et al. (2010) studied the effect of addition of RAS in HMA on dynamic modulus, low temperature cracking and rutting. It was concluded that the rutting performance of the mix improved significantly with the addition of 5 percent

RAS without compromising the low temperature performance. It was also observed that addition of RAS increased the stiffness of the mix. Li et al. (2008) investigated the effect of RAP percentage and sources on the properties of the mix. They used asphalt mixes produced at three different RAP contents, namely 0, 20 and 40 percent, from two different RAP sources and two different asphalt binders (PG 58-28 and PG 58-34). It was concluded that a higher RAP content results in dynamic modulus values that are higher than those of control mixes without any RAP. Experimental data from this study reveal that RAP source is not a significant factor affecting the dynamic modulus at low temperatures. A laboratory study was conducted by Huang et al. (2004) to investigate the effect of RAP contents, varying between 0 to 30 percent, on the fatigue performance of the HMA. It was concluded that use of higher RAP contents increase mixes' stiffness, leading to improved rut resistance and higher tensile strength. It was concluded that inclusion of RAP in HMA improves the fatigue life of the pavement. McDaniel and Shah (2003a) conducted a laboratory study with materials obtained from Indiana, Michigan, and Missouri. Field and laboratory produced mixes with a RAP content of up to 50 percent were tested to evaluate the effect of RAP on the mix performance. Tests conducted with a Superpave® shear tester in most cases indicated that plant-produced mixes showed similar stiffness as their laboratory produced counterparts. Also, they concluded that the use of RAP results in stiffening of the asphalt mix as compared to mixes produced with only virgin materials. Improved stiffness is beneficial to rut resistance but may result in an increase in the potential for fatigue and thermal cracking. Adverse effect of increased RAP on the fatigue life of pavements generally begins to show when the RAP content is greater than 20 percent, as reported by McDaniel et al. (2000). Consequently, they recommended the use of virgin binder of a lower grade to address the fatigue performance issue, especially at high RAP contents. This conclusion is contrary to that by Huang et al. (2004), who reported an improvement in fatigue life due to the addition RAP in a mix. Abdulshafi et al. (2002) conducted an experimental program with a focus on the durability of asphalt mixes with different percentages of RAP. Four different percentages of RAP varying between 0 to 30 were used to prepare HMA mixes. To quantify durability of HMA, TSR tests (AASHTO T 283) were used, and absorbed energy at failure for unconditioned and conditioned samples, based on the indirect tensile strength test, was determined. It was concluded that a HMA mix with limestone aggregates exhibited the best performance in terms of absorbed energy at failure when the RAP content was within 30 percent. Button et al. (1995) conducted a laboratory testing on HMA containing RAS. Two types of dense graded and coarse matrix-high binder surface mixes were modified with 5 and 10 percent RAS and tested. It was concluded that the mixes with the higher air voids in minerals and asphalt film thickness can accommodate RAS better than dense-graded mixes. It was observed that the addition of RAS to the dense-graded mixes decreased the tensile strength of the mix and resulted in an improved resistance to moisture damage. The addition of RAS generally decreased the creep stiffness, which was proportional to the amount of RAS added. Ali et al. (1995) studied the feasibility of using RAS in HMA by testing three mixes with 0, 15, and 25 percent RAS content. Resilient modulus, creep, fatigue, and moisture sensitivity tests were conducted. It was found that both the fatigue life and stiffness of mix improved with an increase in RAS content. Also, it was observed that permanent deformation decreased with addition of RAS, while the moisture sensitivity of the mixes was not affected. Although various researchers have investigated different methods associated with the performance of HMA mixes containing RAP and/or RAS, some results are widely mixed and no clear conclusions could be drawn (Al-Qadi et al., 2007). Also, the available information about the effect of RAP and/or RAS content on the mechanical properties of asphalt mixes is still limited (Li et al., 2008). Hence, there is a need for additional study involving both laboratory and field components. Specifically, appropriate laboratory investigation is needed for local calibration of M-EPDG for mixes containing RAP and/or RAS.

In the current study, a wide range of laboratory tests, namely dynamic modulus, creep compliance, fatigue, moisture damage, and rutting was conducted on the mixes containing RAP and/or RAS to evaluate their laboratory performance and M-EPDG input parameters. Specifically, the laboratory performance and M-EPDG input parameters of the following mixes were evaluated:

- HMA surface course mix without RAP or RAS (control surface mix),
- HMA surface course mix with 10% RAP,
- HMA surface course mix with 25% RAP,
- HMA surface course mix with 20% RAP and 5% RAS,
- HMA base course mix without RAP or RAS (control base course mix),
- HMA base course mix with 25% RAP,
- HMA base course mix with 40% RAP,
- HMA base course mix with 20% RAP and 5% RAS.

The test results indicated a reduction in creep compliance of the mix due to an increase in the RAP content. The dynamic modulus test results illustrated that the asphalt mixes containing higher amounts of RAP have higher dynamic modulus values. The increase in RAP content reduced rutting susceptibility. Furthermore, HWT and TSR tests showed improvement in the resistance to moisture-induced damage of both surface and base course mixes, as a result of using more RAP in the mix.

1.3.4 Mechanistic Approach to Adhesion and Moisture Damage Phenomena

With recent developments in testing equipment and studies focused on performance testing, the mix design philosophy is moving from empirical design towards a mechanistic-based approach. Despite these developments, the moisture damage potential of an asphalt mix is generally evaluated using the TSR test (ratio between conditioned and unconditioned indirect tensile strengths) or from the inflection point in the HWT-based rut test according to the AASHTO T 283 and AASHTO T 324 standard test methods, respectively. Both of these tests are widely used as indicators of moisture-induced damage potential but neither directly address the loss of adhesion and cohesive bonding, so called “failure mechanisms,” that governs the stripping in asphalt pavement. Specifically, a TSR test (AASHTO T 283) is mainly based on a very empirical approach and is more of an index type test. The specimen conditioning, which includes freezing the partially saturated test specimens at -18°C for 16 hours and a warm water soaking of 60°C for 24 hours, is not representative of the field environmental condition. Samples are tested using a non-cyclic load of constant rate, at room temperature, which is not representative of actual traffic loads. These shortcomings of AASHTO T 283 test method has led to use of inflection point from

HWT tests (AASHTO T 324), as an indicator of stripping initiation. The results obtained from the tests conducted on a number of mixes show that some mixes with relatively low TSR values perform well when tested using HWT (Ghabchi et al, 2013a). This type of observations raises questions about the reliability of the current practice for evaluation of moisture-induced damage potential of asphalt mixes. Therefore, there is a need to study the moisture damage mechanism using a mechanistic method that addresses this type of shortcomings of empirical methods. Such needs become more important for newer mixes like WMA mixes and mixes containing RAP and RAS. Recent studies show that SFE characteristics of asphalt binders and aggregates can be used in a mechanics-based approach for quantification of moisture damage potential of asphalt mixes (Ghabchi et al., 2013a; Wasiuddin et al., 2008; Wasiuddin et al., 2008; Bhasin et al., 2007; Bhasin and little, 2007; Wasiuddin, 2007; Lytton et al., 2005; Cheng et al., 2002).

The SFE method is a mechanistic approach to investigate the adhesion and cohesion behavior of asphalt mixes. The SFE method has been applied widely to study coating and adhesion mechanisms in surface science and industry (Elphingstone, 1997; Good, 1992). Elphingstone (1997) showed applicability of the SFE measurement to asphalt materials for prediction of their moisture damage potential. Bhasin et al. (2006) examined the moisture damage potential of HMA mixes based on the SFE method. They quantified the adhesive bond energy of the aggregate-binder and the reduction in surface free energy as a result of binder-aggregate debonding in presence of water. It was concluded that bond energies can vary significantly with aggregates from different sources. Kim et al. (2004) applied dynamic mechanical analysis (DMA) to characterize the fatigue damage and fracture of asphalt binders and mastic. They were able to show the consistency of the results from applying the SFE method and the results of the DMA fatigue tests. Based on these studies and observations reported herein, there is a need for studying the effects of the WMA additives and reclaimed asphalt binder (from RAP and RAS) on the moisture-induced damage potential of the new green pavements, using a mechanistic approach.

In the current study the moisture-induced damage potential of the asphalt binder-aggregate systems, containing different WMA additives (Advera®, Sasobit®, and Evotherm®) and HMA mixes with different amounts of RAP binder were evaluated with applying a mechanism-based SFE approach. In mechanistic study of moisture-induced damage potential of WMA, the SFE components of modified PG 64-22 asphalt binder with different percentages of WMA-additives, namely Sasobit® (1.0%, 1.5% and 2.0% by the weight of asphalt binder), Advera® (0.25%, 0.30% and 0.35% by the weight of asphalt mix), and Evotherm® (0.25%, 0.50% and 0.75% by the weight of asphalt binder) were measured in the laboratory. The SFE components of the selected aggregates, namely limestone, sandstone, gravel, granite and basalt were measured in the laboratory, or adopted from the literature. The wettability, the work of adhesion, the work of debonding, and energy ratios were estimated to assess the moisture-induced damage potential of combinations of modified asphalt binders and different aggregates. The results indicated that Sasobit® and Advera® are able to reduce the moisture susceptibility of the mixes, but are not recommended to be used with highly acidic aggregates like granite. Evotherm® resulted in the highest increase in wettability, total surface free energy, increased work of adhesion and a reduction in the work of debonding, leading to a better possible aggregate coating with asphalt binder and lower moisture susceptibility with all types of tested aggregates, compared to those of other WMA-additives. Furthermore, TSR tests were conducted on Advera® and Evotherm®-modified and neat (unmodified) asphalt mixes and results were compared with those from the SFE test. It was found that the SFE approach is a better indicator of moisture-induced damage compared to the traditional TSR test. It is expected that the present study would be helpful in understanding the moisture-induced damage potential of the flexible pavements constructed with WMA technologies.

In mechanistic study of moisture-induced damage of mixes containing RAP, SFE method was applied to evaluate the effects of asphalt binder type, RAP binder, and aggregate type on the moisture-induced damage potential of asphalt mixes. The SFE components (non-polar, acid and base) of a PG 64-22 and a PG 76-28 (polymer-modified) asphalt binder, each blended with different percentages of RAP binder (0%, 10%, 25% and 40%) were measured using a dynamic contact angle (DCA) device. The aggregates included in this study consisted of limestone, rhyolite, sandstone, granite, gravel, and basalt. The SFE components of limestone and rhyolite aggregates were measured using a universal sorption device (USD), while those for the other aggregates (sandstone, granite, gravel, and basalt) were obtained from the literature. The energy ratio parameters estimated based on the spreading coefficient, the work of adhesion, and the work of debonding were used to assess the moisture-induced damage potential of different combinations of asphalt binders with different RAP binder contents and aggregates. The results indicated that the acid SFE component of PG 64-22 and PG 76-28 asphalt binders increases with addition of RAP binder, while the base SFE component remains almost unchanged with addition of RAP binder. Furthermore, the wettability and the work of adhesion of both PG 64-22 and PG 76-28 asphalt binders over different types of aggregates increased with an increase in RAP content (by 25% and more). Based on the energy ratio parameters, it was found that the resistance to moisture-induced damage, increased with an increase in RAP content for both PG 64-22 and PG 76-28 asphalt binders and all types of aggregates, specifically at higher RAP amounts. Furthermore, it was found that the higher the total SFE component of the aggregates, the lower the energy ratio parameter values. Therefore, a high total SFE component of aggregate may result in a high moisture-induced damage potential in the mix. It is expected that this study would be helpful in understanding the moisture-induced damage potential of the asphalt mixes, produced with polymer-modified and non-polymer-modified asphalt binders, containing RAP. Furthermore, a parameter combining SFE components and mix design proportions was proposed in order to evaluate the moisture-induced damage characteristics of the mixes containing RAP. For this purpose, two approaches were pursued: (i) micro-structural analysis of aggregate-asphalt bond based on the surface energy parameters, and (ii) mechanical testing of asphalt mixes using TSR and HWT. According to approach (i), the SFE (non-polar, acidic and basic) components of the virgin PG 64-22 binder mixed with 0%, 25% and 40% of RAP binder and aggregates, namely limestone, rhyolite, and RAP extracted aggregate were used to determine the composite work of adhesion and composite work of debonding, and composite energy ratios for each combinations of asphalt binder and aggregates. The composite energy ratios were used to assess the moisture-induced damage potential of the mixes containing different percentages of RAP. According to approach (ii), the TSR and HWT test data conducted on asphalt mixes containing different percentage of RAP were used to evaluate their moisture-induced damage potential. All test methods (SFE, TSR, and HWT) showed that the moisture-induced damage potential decreased with increasing amount of RAP in asphalt mixes. A strong correlation was found to exist between the moisture-induced damage potential predicted using the micro-structural method and laboratory performance tests. It was found that the micro-structural energy approach, as a mechanistic framework, can be successfully used as an indicator of moisture-induced damage potential of the asphalt mixes.

1.4 Outline of the Dissertation

This dissertation is focused on effects of different WMA additives and different amounts of RAP and RAS used in the asphalt mixes, on the mix performance, measured in the laboratory. Furthermore, the effects of the binder type, WMA additives, amounts of RAP and aggregate types on the moisture-induced damage potential of asphalt mixes were investigated, using the SFE approach. The findings of this study are presented in this dissertation as 5 journal publications (1 published, 3 under review and 1 recently prepared). Except Chapter 1 (introduction) and Chapter 7 (conclusions) each chapter covers one paper.

Chapter 1 presents an introduction to green paving technologies and a short background on WMA and the use of RAP and RAS in HMA. The objectives and the outline of the dissertations are discussed in this chapter.

Chapter 2 presents the outcomes of a study conducted for evaluation of stiffness, low temperature cracking, rutting, moisture damage, and fatigue performances of WMA mixes. A wide range of laboratory tests, namely, dynamic modulus, creep compliance, fatigue, moisture damage, and rutting was conducted to evaluate the performance of different types of WMA mixes. For this purpose, three WMA mixes, consisting of one Advera® and one Evotherm® surface course mix and one Evotherm® base course mix, were collected from different field projects and tested in the laboratory. In addition, three HMA mixes with aggregate gradations similar to the collected mixes were produced and tested in the laboratory to compare the performance of WMA and HMA mixes.

In Chapter 3, the effects of using RAP and RAS on the laboratory-measured performance of the asphalt mixes were evaluated. Laboratory tests, namely, dynamic modulus, creep compliance, fatigue, moisture damage, and rutting were conducted to evaluate the performance of the asphalt mixes. For this purpose three surface course mixes containing 0% RAP, 25% RAP, and 5% RAS + 20% RAP, and four base course mixes containing 0% RAP, 25% RAP, 40% RAP, and 5% RAS + 20% RAP were tested.

In Chapter 4, the SFE method was used to evaluate the moisture susceptibility of WMA with three different WMA-additives, namely Sasobit®, Advera®, and Evotherm®. The SFE components of modified PG 64-22 asphalt binder with different percentages of WMA-additives and selected aggregates were measured in the laboratory. The wettability, the work of adhesion, the work of debonding, and energy ratios were estimated to assess the moisture-induced damage potential of combinations of modified asphalt binders and different aggregates. Furthermore, TSR tests were conducted on Advera® and Evotherm®-modified and neat (unmodified) asphalt mixes and results were compared with those from the SFE test.

In Chapters 5, the SFE method was used to evaluate the effects of asphalt binder type, RAP, and aggregate type on the moisture-induced damage potential of asphalt mixes. The SFE components (non-polar, acid and base) of a PG 64-22 and a PG 76-28 (polymer-modified) asphalt binder, each blended with different percentages of RAP binder (0%, 10%, 25% and 40%) were measured using a DCA analyzer. The aggregates consisted of limestone, rhyolite, sandstone, granite, gravel, and basalt. The SFE components of limestone and rhyolite aggregates were measured using a USD device, while those of the other aggregates (sandstone, granite, gravel, and basalt) were obtained from the literature. The energy ratio parameters estimated based on the spreading coefficient, the work of adhesion, and the work of debonding were used to assess the moisture-induced damage potential of different combinations of asphalt binders and different RAP binder contents and aggregates.

Chapter 6 presents the results of a study undertaken to evaluate the effects of RAP on moisture-induced damage potential of asphalt mixes using two different approaches: (i) micro-structural analysis of aggregate-asphalt bonding based on the SFE and JMF, and (ii) mechanical testing of asphalt mixes using TSR and HWT. This study involved two phases. In the first phase, the SFE (non-polar, acidic and basic) components of a virgin PG 64-22 binder mixed with 0, 25 and 40% of RAP binder and aggregates (limestone, rhyolite, RAP extracted aggregate) were measured using a DCA and a USD, respectively. Thereafter, composite work of adhesion and composite work of debonding, and composite energy ratios for each combinations of asphalt binder and aggregates were determined to assess the moisture-induced damage potential of the mixes containing different percentages of RAP (0, 25 and 40%). In the second phase, the TSR and HWT tests were conducted on asphalt mixes containing different percentage of RAP (0%, 25% and 40%) to evaluate their moisture-induced damage potential. Both the methods showed that the moisture-induced damage potential decreased with increasing amount of RAP in asphalt mixes.

Chapter 7 presents a summary of the important conclusions drawn from this study and recommendations for future works.

CHAPTER

2

LABORATORY EVALUATION OF STIFFNESS, LOW TEMPERATURE CRACKING, RUTTING, MOISTURE DAMAGE, AND FATIGUE PERFORMANCES OF WMA MIXES[†]

ABSTRACT

Despite the environmental and compaction benefits of warm mix asphalt (WMA), several researchers have expressed concerns over laboratory and field performances of WMA mixes. In the present study, a wide range of laboratory tests, namely, dynamic modulus, creep compliance, fatigue, moisture damage, and rutting was conducted to evaluate the performance of different types of WMA mixes. For this purpose, three WMA mixes, consisted of one Advera® and one Evotherm® surface course mix and one Evotherm® base course mix, were collected from different field projects in Texas. In addition, three HMA mixes with aggregate gradations similar to the collected mixes were produced in the laboratory to compare the performance of WMA and HMA mixes. Overall, the WMA mixes showed lower dynamic modulus value (lower stiffness) for all combinations of temperatures and frequencies, reduced potential of low temperature

[†] This chapter has been submitted to the Journal of Road Materials and Pavement Design under the title "Laboratory Evaluation of Stiffness, Low Temperature Cracking, Rutting, Moisture Damage, and Fatigue Performances of WMA Mixes." The current version has been formatted for this dissertation.

cracking, lower fatigue life (a lower number of cycles to fatigue failure), and a higher rutting potential compared with their HMA counterparts. However, a mixed trend of moisture-induced damage potential was observed for WMA and HMA mixes, when evaluated using retained tensile strength ratio (TSR) and stripping inflection point (SIP) obtained from the Hamburg wheel tracking test (HWT). In other words, no correlation was found between TSR and SIP values, indicating a mix which passes a TSR test does not guarantee its better performance when tested using a HWT. Furthermore, a good correlation was found between inverse rutting rate and dry indirect tensile strength (DITS), indicating a mix with a higher DITS would have better rutting resistance. The results from this study reveal that performance of a WMA mix widely depends on the technology and the type of other additives (e.g., anti-stripping agent) used. The findings of this study are expected to be useful for pavement professionals to better understand performance of WMA mixes and to develop a database of input parameters for the Mechanistic-Empirical Pavement Design Guide.

Keywords: warm mix asphalt, dynamic modulus, creep, rut, moisture damage, fatigue.

2.1 Introduction

Recently, using warm mix asphalt (WMA) technologies has been gaining popularity because of their economic, environmental and compaction benefits. The WMA technologies allow a reduction in mixing and placement temperatures, leading to a major saving in fuel cost, cutting in emissions, and achieving better mix workability at a lower temperature. Therefore, many Departments of Transportation (DOTs) and highway agencies are motivated to evaluate the performance of WMA mixes and develop relevant specifications. However, despite its advantages, national concerns focus on moisture-induced damage, fatigue, and rutting performance of WMA mixes (Ghabchi et al., 2013a; Ghabchi et al., 2013b; Ghabchi et al., 2013c; Bonaquist, 2011; Hurley et al., 2010; Kvasnak et al., 2009; Mallick et al., 2008; Wasiuddin et al., 2008; Wasiuddin et al., 2007; Prowell et al., 2007; Hurley and Prowell, 2006).

Several researchers have reported that the partially dry aggregates at reduced mixing temperature in WMA may establish a poor bond with the asphalt binder, which can easily experience damage in presence of water (Kvasnak et al., 2009; Prowell et al., 2007; Wasiuddin et al., 2007; Hurley and Prowell, 2005; Hurley and Prowell, 2006). However, previous studies have shown different moisture damage potential for WMA mixes, depending on the technologies and processes used. For example, Goh and You (2011) found a similar TSR value for Sasobit®-modified WMA mix and HMA mix. Similarly, Hurley et al. (2010) studied moisture damage performance of Evotherm® and Sasobit® WMA and HMA mixes used for construction of field test sections, and reported that both mixes performed equally well.

As far as rutting performance of WMA mixes is concerned, it is expected that reduced asphalt binder oxidation, as a result of lower mixing temperature, may lead to increased rutting (Hurley et al., 2010; Lee et al., 2009; Wielinski and Rausch, 2009; Xiao et al., 2010; Bonaquist, 2011; Hurley et al., 2010; Goh and You, 2011). However, inconsistent rutting performance of WMA mixes have been reported in the literature. For example, WMA mixes with Sasobit® additive exhibited a better resistance to rutting compared to HMA (Bonaquist, 2011; Hurley et al., 2010; Xiao et al., 2010; Button et al., 2007). While, WMA mixes with Aspha-Min®, Evotherm®, Sasoflex® and Advera® additives generally showed a lower and in some cases equal rutting resistance, as compared to the HMA mixes in laboratory (Bonaquist, 2011; Xiao et al., 2010; Hurley and Prowell, 2006; Hurley and Prowell, 2005).

In spite of some moisture and rutting performance issues reported for WMA mixes, some researchers observed that WMA mixes can have a better or equal fatigue life due to reduced oxidation of the asphalt binder (Goh and You, 2011; Diefenderfer and Hearon, 2008; Kvasnak et al., 2010; D'Angelo et al., 2008). Based on the four-point bending beam fatigue test results, it was also concluded that WMA's fatigue lives of Sasobit® and Advera® WMA mixes, in most cases, were similar to (or in some cases higher than) those of the HMA (Goh and You, 2011). However, depending on the aggregate type, gradation and strain level, the fatigue life of Sasobit® WMA has been reported to be less or equal to that of HMA (Diefenderfer and Hearon, 2008). Similarly, Jenkins et al. (2011) found that, in general, the WMA mixes produced using RedisetTM and Sasobit® additives exhibited fatigue lives lower than those of HMA mixes. Currently, limited data are available to draw a clear conclusion on fatigue performance of WMA mixes. Furthermore, a few studies have been conducted to compare performance of WMA and HMA for their low temperature cracking potential and dynamic modulus as a stiffness indicator. Specifically, the results of these two tests are very important input parameters for Mechanistic-Empirical Pavement Design Guide (M-EPDG) and need to be determined for new mixes.

From the abovementioned studies it can be concluded that the advantages and disadvantages associated with WMA highly depend on the type of the WMA technology being used for mix production. Consequently, it is important to study each WMA technology and process separately. The present study compares dynamic modulus, moisture damage, rutting, fatigue and low temperature cracking performances of WMA and HMA mixes. For this purpose, three WMA mixes, consisting of one Advera® and one Evotherm® surface course mix each and one Evotherm® base course mix, were collected from different projects in Texas and their HMA counterparts were prepared in the laboratory. The moisture damage performance was evaluated using two different methods: TSR and HWT. Likewise, rutting performance was evaluated from accelerated rutting test in HWT. The dynamic moduli of mixes were determined at different temperatures and frequencies. The dynamic modulus is a key input parameter for fatigue and rutting performance of mixes in the M-EPDG. The thermal cracking potential of WMA mixes was also evaluated using creep compliance and IDT tests. The master curves for dynamic modulus and creep compliance curves were generated. The data generated under these tests would be helpful for the Level 1 MPEDG.

2.2 Objectives

The overall objective of this study was to compare the laboratory performance of WMA and HMA mixes. The specific objectives of this study were to:

1. Compare stiffness of WMA and HMA mixes by determining their dynamic modulus values at different temperatures and frequencies.
2. Assess the low-temperature cracking potential of WMA and HMA mixes with the help of creep compliance test.
3. Determine and compare the fatigue life of WMA and HMA mixes using the four- point bending beam fatigue test.

4. Evaluate the moisture-induced damage potential of WMA and HMA mixes using TSR and HWT methods and to rank the mixes based on their performance according to each test method and visual observation.

2.3 Materials

WMA is generally produced using two major technologies based on: (i) use of additives such as water vapor releasing admixtures like zeolites, organic additives, surfactants and/or waxes, and (ii) the process driven technologies which tend to be foaming processes including Double Barrel Green plants, Low Energy Asphalt and WMA-Foam. This study was focused on the WMA technologies that can be classified in the first category. These technologies use (a) water vapor releasing additives (zeolites), and (b) chemical additives. For this purpose, three WMA asphalt mixes (one gradation of Advera® and two gradations of Evotherm®) were collected. In addition, HMA mixes were designed and produced in the laboratory. The details of each of the mixes are provided below.

2.4 Warm Mix Asphalt Mixes

2.4.1 Advera® WMA Mix

The Advera® WMA mix (ADWM) was collected from an asphalt production plant located at Bridgeport, TX, on June 30, 2011. The produced WMA mix was being used by a local contractor for construction of an asphalt overlay project located at the southbound lane of US 287 at the south of Rhome, TX. Figure 2.1 and Table 2.1 show mix design gradations and a summary of the WMA and HMA mixes, used in this study, respectively. Based on Figure 2.1 and Table 2.1, the collected ADWM consisted of a fine surface course mix with a nominal maximum aggregate size (NMAS) of 9.5 mm, and 5.0% of PG 64-22 asphalt binder (AC) content by the total weight of mix. Also, 15% of RAP and 2.4% of RAS, by the weight of aggregates, were used. The mixing temperature used for production of ADWM was 130°C. ArrMaz AD-Here® HP PLUS was used as anti-stripping agent, by 1% of AC weight.

2.4.2 Evotherm® WMA Mix Type B

Evotherm® warm mix Type B (EVWM-B) was collected from Century asphalt plant located in San Antonio, TX on October 26, 2011. This mix was being used by a local contractor for construction of the base layer of a city road. EVWM-B consisted of a fine base course mix with an NMAS of 19 mm, and 4.5% of PG 64-22 AC content by the total weight of mix. Also 20% of RAP and 2.5% of RAS, by the weight of aggregates were used. The mixing temperature used in asphalt plant for production of EVWM-B was 135°C.

2.4.3 Evotherm® WMA Mix Type C

Evotherm® warm mix Type C (EVWM-C) was collected from Century asphalt plant located in San Antonio, TX on January 4, 2012. EVWM-C mix was a coarse surface mix with NMAS of 12.5 mm, and 4.8% PG 70-22 AC content. Also 10% RAP, by the weight of aggregates, was used in the mix. The mixing temperature in asphalt plant for production of EVWM-C was 135°C. Lime, 1% by the total aggregate weight, was used as anti-stripping agent.

2.4.4 HMA Mixes

The HMA mixes with the gradations identical to those of ADWM, EVWM-B and EVWM-C, namely Advera® hot mix (ADHM), Evotherm® hot mix Type B (EVHM-B) and Evotherm® hot mix Type C (EVHM-C) were produced in laboratory

(Figure 2.1). The aggregate, asphalt binder, and anti-stripping agents collected from the abovementioned asphalt plants were used for production of HMA mixes in the laboratory. Aggregate gradations used for HMA are known to be adequate for use in WMA (Hurley and Prowell, 2005). Therefore, there was no need to modify the gradation specifications for WMA from those of HMA (Button et al., 2007). HMA mixes were produced at 160°C.

2.5 WMA Additives

2.5.1 Advera®

Advera® is a synthetic zeolite (Sodium Aluminum Silicate), produced by PQ Corporation, Malvern, PA (Anderson et al., 2008). Its crystalline structure contains approximately 20 percent water by weight, which causes foaming of the asphalt binder due to released water vapor at temperatures above 100°C. The small-scale foaming of asphalt binder created by released water vapor results in an improvement in mix workability (Santucci, 2010). This increased workability enables mix production and placement at lower temperatures by 28°C to 39°C compared to those of conventional HMA (Corrigan, 2011). Use of 0.25% Advera® by weight of the mix is recommended by the manufacturer.

2.5.2 Evotherm®

Evotherm® is a product of MeadWestvaco Asphalt Innovations, Charleston, SC (Button et al., 2007). A non-proprietary technology that is based on a chemical package including cationic emulsification agents and additives which improve aggregate coating, adhesion (anti-stripping agents), workability, and compaction of the asphalt mix is used in production of Evotherm®. The product enhances the mix workability at lower temperatures (Prowell and Hurley, 2007), up to 56°C (Corrigan, 2011). A unique chemical compound customized for aggregate compatibility is delivered into an emulsion (dispersed) asphalt phase (Corrigan, 2011). Use of 0.5% Evotherm® by the weight of the asphalt binder, is recommended by the manufacturer.

2.6 Methodology

The study involves various tasks to successfully achieve the objectives outlined in this study. The preparation of samples in the laboratory, and conducting various performance tests, namely, dynamic modulus, creep compliance, four-point bending beam fatigue, Hamburg wheel tracking and retained indirect tensile strength tests are discussed in this section. Figure 2.2 shows a summary of the work flow conducted in this study.

2.6.1 Sample Preparation

All of the samples tested in this study were compacted in the laboratory. The field-collected WMA mixes (ADWM, EVWM-B, and EVWM-C) were prepared for compaction by reheating them in an oven. The mixing and compaction temperatures summarized in Table 2.1 were used for this purpose. Before starting the compaction, the mix in the oven was stirred several times to ensure its consistency and workability. Laboratory-prepared HMA (ADHM, EVHM-B, and EVHM-C) mixes were conditioned for short-term aging in accordance with AASHTO R 30 (AASHTO, 2002), in order to account for the aging process a plant-produced mix undergoes in the mixing process. Sample compaction of WMA and HMA was pursued according to the required sample type and dimensions. Sample air voids of 7.0% ± 0.5% were targeted for all of the specimens compacted for performance tests. These air voids are based on the densities typically obtained in the field compaction. A Superpave gyratory compactor (SGC) was used to prepare samples for dynamic modulus, creep compliance, Hamburg wheel tracking, and indirect tensile strength tests. Since the air voids at certain dimensions were targeted, the SGC was operated in the height mode. In the height mode, the SGC automatically stops the compaction procedure as soon as the desired height is reached. Volumetric analyses were performed after compaction in order to ensure achieving the targeted air voids, in accordance with AASHTO T 166 (AASHTO, 2010). A linear kneading compactor was used to prepare the required slab samples (before cutting the beam samples) for four-point bending beam fatigue test. Details of sample preparation for each performance test are discussed next.

2.6.1.1 Dynamic Modulus

Initially, at least three replicate specimens having 150 mm in diameter by 167.5 mm in height were compacted in the SGC at 7.0% ± 0.5% air voids. Then, the compacted specimens were cored from the center to obtain 100 mm diameter specimens. The cored specimens were then saw-cut from each end to obtain the final specimens with a diameter of 100 mm and a height of 150 mm. This procedure in known to produce specimens with consistent air void distribution (Chehab et al., 2000). A total of 18 specimens (6 mixes x 3 specimens) were compacted for dynamic modulus tests.

2.6.1.2 Creep Compliance

At least three replicate specimens having 150 mm in diameter by 65 mm in height were compacted using a SGC. Then the compacted specimens were saw-cut from each end to obtain the final specimens with a diameter of 150 mm and a height of 45 to

50 mm, in order to achieve consistent air void distribution. A total of 18 specimens (6 mixes x 3 specimens) were prepared for creep compliance tests.

2.6.1.3 Four-Point Bending Beam Fatigue

The slab samples (406 mm (L) by 152 mm (W) by 76 mm (H)) were compacted using a linear kneading compactor. Then two beam specimens (380 mm (L) by 63 mm (W) by 50 mm (H)) were saw-cut from each slab. A total of 18 slabs (6 mixes x 3 slabs) were compacted and saw-cut to obtain 36 beam specimens for fatigue testing.

2.6.1.4 Hamburg Wheel Tracking

At least four replicate specimens having 150 mm in diameter by 60 mm in height were compacted using a SGC. Then each test set of samples, consisting of two specimens, were saw-cut from the side to match the mold dimensions of the device. A total of 24 samples (6 mixes x 4 samples) were compacted and saw-cut to obtain 12 sets of samples for testing.

2.6.1.5 Retained Indirect Tensile Strength Ratio

At least eight replicate specimens having 150 mm in diameter by 95 mm in height were compacted using a SGC. A total of 48 samples (6 mixes x 8 samples) were compacted and were set aside for testing.

2.6.2 Laboratory Testing

2.6.2.1 Dynamic Modulus

The dynamic modulus test, an indicator of stiffness of the asphalt mix, was conducted on cylindrical specimens in accordance with AASHTO TP 62 (2010). A servo-hydraulic universal testing system from MTS was used for conducting the dynamic modulus test. As recommended by AASHTO TP 62 (2010), dynamic modulus tests were conducted at -10, 4.4, 21.1, 37.8, and 54.4°C temperatures with six loading frequencies, namely 25, 10, 5, 1, 0.5 and 0.1 Hz at each temperature. The tests were conducted starting from the lowest to the highest temperature and from the highest to the lowest frequency. Cyclic haversine-shaped load pulse was applied with a load magnitude, adjusted based on the material stiffness, frequency and temperature, to keep the strain response within 50-150 microstrains (Tran and Hall, 2006). A 100 kN load cell was used to measure the applied loads. Vertical deformations were measured by two Linear Variable Differential Transformers (LVDTs), attached on two diametrically opposite sides on the specimen at 100 mm gauge length. The recorded loads and vertical displacements for the last five cycles of each sequence were used to determine the dynamic modulus values. Finally, the dynamic modulus master curves at a reference temperature of 21.1°C were constructed based on the time-temperature superposition principle. A sigmoidal function, as shown in Equation 2.1, was used in fitting the master curve (Singh et al., 2011a; Singh et al., 2011b).

$$log|E^*| = \delta + \frac{\alpha}{1+\exp(\beta+\gamma(\log f_r))} \tag{2.1}$$

where,

$|E^*|$ = dynamic modulus in MPa,

f_r = reduced frequency at reference temperature,

δ = minimum value of |E*|,

$\delta + \alpha$ = maximum value of |E*|, and

β, γ = parameters describing the shape of the sigmoidal function.

General form of the shift factor is also given in Equation 2.2 and Equation 2.3.

$$a(T) = \frac{f_r}{f} \tag{2.2}$$

$$\log(f_r) = \log\big(a(T)\big) + \log(f) \tag{2.3}$$

where,

$a(T)$ = temperature shift factor,

T = temperature in °C, and

f = frequency at a particular temperature.

The shift factor can be expressed in the form of Equation 2.4, using the Arrhenius time–temperature superposition model (Francken and Clauwaert, 1988).

$$\log\big(a(T)\big) = c\left(\frac{1}{T_K} - \frac{1}{T_{ref}}\right) \tag{2.4}$$

where,

T_K = the test temperature of interest in °Kelvin (°K = 273 + °C);

T_{ref} = the reference temperature in °K.

A nonlinear optimization program, namely Solver in MS-Excel, was used for solving the master curve coefficients, namely $\alpha, \beta, \gamma, \delta$ and c. Then, a quadratic polynomial fit, as shown in Equation 2.5, was used to establish the shift factor-temperature relationship.

$$\log\big(a(T)\big) = mT^2 + nT + p \tag{2.5}$$

where,

m, n, p = polynomial fitting curve coefficients.

The goodness-of-fit statistics, according to the criteria presented in Table 2.2 (Witczak, 2005), was used to evaluate the master curve models and shift factor equations developed in this study. In this method, the S_e/S_y (standard error of estimate/standard deviation) and correlation coefficient (R^2) are used to evaluate the strength of the model. According to the goodness-of-fit statistics and the criteria shown in Table 2.2, the performance of a model may be rated in five categories, namely excellent, good, fair, poor, and very poor.

2.6.2.2 Creep Compliance

Creep compliance tests were conducted on cylindrical samples in accordance with the AASHTO T 322 standard test method (AASHTO, 2007), as indicator and input parameter used in the M-EPDG for prediction of the low-temperature cracking potential of the mixes. A servo-hydraulic universal testing system manufactured by MTS was used for testing. Tests were conducted at temperatures of -18, -10, 0, and 10°C. A static load of fixed magnitude was applied to the specimen along its diameter for 100 seconds. A 100 kN load cell was used to measure the applied load. The vertical and horizontal deformations were measured by two LVDTs with a maximum stroke length of 5 mm, attached on the two diametrically perpendicular directions of the specimen. On the flat face of the specimen, two gauge points were placed along the vertical and two along the horizontal axes with a center to center spacing of 38.0 ± 0.2 mm. During the creep test, horizontal and vertical strains were maintained within the linear viscoelastic limit (typically below 500 microstrain in the horizontal direction), by adjusting the applied static load. Creep compliance was then calculated as a function of time at 1, 2, 5, 10, 20, 50, and 100 seconds after the test's initiation, based on the horizontal and vertical deformations recorded at the center of the specimen. After the creep tests had been completed at all temperatures, indirect tensile strength at -10°C, as recommended by the M-EPDG, was determined by applying a load to the specimen at a vertical ram movement rate of 12.7 mm per minute, until failure. Finally, creep compliance master curves were developed by using the time-temperature superposition principle. According to Ferry's law, at a reference temperature (10ºC), the shapes of adjacent creep compliance curves obtained from different temperatures were shifted with respect to time to obtain an exact matching and form a smooth function (Ferry, 1980). This function was written in the form of Equation 2.6.

$$D(t) = D_0 + D_1 t^m \tag{2.6}$$

where,

$D(t)$ = creep compliance in 1/MPa,

t = time in seconds, and

D_0, D_1, m = model constants.

A nonlinear optimization was used for solving the shift factors at different temperatures and master curve coefficients, namely D_0, D_1, m. Then, a linear function fit, as shown in Equation 2.7, was used to establish the shift factor-temperature relationship for creep compliance.

$$\log\big(a(T)\big) = aT + b \tag{2.7}$$

where,

a, b = model constants.

2.6.2.3 Four-Point Bending Beam Fatigue

Fatigue life of the asphalt mixes in this study was evaluated using four-point bending beam fatigue tests, according to the AASHTO T 321 standard test method (AASHTO, 2011). Each beam specimen was subjected to cyclic loading and unloading with a frequency of 10 Hz, inside a temperature chamber at 20°C. Tests were conducted in the displacement-control mode, at 400-microsrain. This strain level was selected based on the past experience of testing the WMA and HMA mixes, in order to have the fatigue lives of the maximum number of the specimens fall in the range of approximately 50,000 and 500,000 loading cycles, as recommended by Harvey et al. (1995). Using the 400-microstrain also helped to keep the time required for testing in a reasonable range. A 5 kN load cell was used to measure the loads applied to the beam specimen. Vertical deflection at the center of the beam was measured using an LVDT mounted on the beam fixture, and a metallic stud glued at the center of the beam. The fatigue life is the total number of load repetitions that causes a 50 percent decrease in initial beam stiffness (Tayebali et al., 1993; Tayebali et al., 1992; Pronk and Hopman, 1990). Initial beam stiffness was determined at 50^{th} load cycle. At least three beam specimens from each mix were tested for the fatigue life.

2.6.2.4 Hamburg Wheel Tracking

Rutting and moisture susceptibility of asphalt mixes were evaluated by conducting Hamburg wheel tracking tests in accordance with the AASHTO T 324 standard test method (AASHTO, 2011). For this purpose, the specimens were submerged in a temperature-controlled water bath at 50°C and repetitively loaded using a reciprocating steel wheel. The wheel load applied to the specimen is equal to 705 N. After 20,000 passes deformation versus number of passes was plotted for determining the creep slope, stripping inflection point (SIP) and stripping slope. A sudden increase in deformation rate coincides with stripping of the asphalt binder from the aggregate, an indication of moisture-induced damage.

2.6.2.5 Retained Indirect Tensile Strength Ratio (TSR)

Moisture-induced damage potential of the mixes were evaluated in accordance with the AASHTO T 283 standard test method (AASHTO, 2011), based on their retained indirect tensile strength ratio (TSR). In this method, moisture-induced damage potential of the mixes were evaluated by measuring the change in diametric tensile strength of compacted asphalt mixes resulting from the effects of water saturation and accelerated water and temperature conditioning, with a freeze-thaw cycle. After compaction, each set of specimens were divided into two subsets. One subset was tested under dry condition at a temperature of 25°C for indirect tensile strength. The other subset was vacuum saturated under a 13 to 67 kPa absolute vacuum pressure. Saturation was maintained between 70 to 80 percent. Each of the vacuum-saturated specimens were then tightly covered with a plastic film and placed in a plastic leak-proof bag containing 10-mL of water. Then, these specimens were subjected to a freeze cycle of -18°C for a minimum of 16 hours, followed by a 60°C warm water soaking cycle for 24 hours. The conditioned specimens were then placed in a water tank at 25°C temperature for another two hours, before testing for indirect tensile strength. Numerical indices of retained indirect tensile strength were calculated from the test data obtained by the two tested dry and conditioned subsets. The results from this test are generally used to predict long-term stripping susceptibility of the tested mixes. In this study, the TSR results were also compared with those from the Hamburg wheel tracking tests.

2.7 Results and Discussion

2.7.1 Dynamic Modulus

Table 2.3 presents the model parameters (Equation 2.1) for the dynamic modulus master curve for WMA and HMA mixes, at 21.1°C reference temperature. Also, rating for each model based on the goodness-of-fit statistics criteria (Table 2.2) is shown in Table 2.3. From Table 2.3, it is evident that the models used for the development of the dynamic modulus master curves can be statistically rated as "excellent." This indicates that the sigmoidal functions used for modeling the dynamic modulus master curves satisfactorily predict the dynamic modulus values at different reduced frequencies. Furthermore, the temperature shift factor parameters, expressed as a quadratic polynomial according to Equation 2.5, are presented in Table 2.4. Similarly, the model rating, based on the goodness of fit statistics displayed in Table 2.4, was found "excellent" for the entire shift factor polynomials.

Figures 2.3-a, 2.3-b and 2.3-c show master curves for WMA and HMA mixes of Advera® (ADWM and ADHM), Evotherm® Type B (EVWM-B and EVHM-B) and Evotherm® Type C (EVWM-C and EVHM-C), respectively. From Figure 2.3, it is evident that for all mixes the dynamic modulus increases with increased frequency and reduced temperature. This trend of dynamic modulus variation with temperature and loading frequency is consistent with the findings of other studies (e.g., Tashman and Elangovan, 2008; Flintsch et al., 2007; Singh et al., 2011a).

Furthermore, from Figure 2.3 it can be observed that the dynamic modulus values of HMA mixes were higher compared to WMA mixes for all combinations of temperatures and frequencies. This difference in dynamic modulus values was more pronounced for the EVHM-B and EVWM-B mixes with a NMAS of 19 mm (the coarsest mix). The higher stiffness in HMA mixes (ADHM, EVHM-B and EVHM-C) was attributed to the higher mixing temperatures used for production of HMA (Table 2.1), compared to the WMA cases. At a higher temperature (160ºC), the asphalt binder used in the HMA mixes experience more aging compared to that of WMA mixes (Hurley and Prowell, 2006) produced under lower mixing temperatures (130ºC – 135ºC). More aging makes the asphalt binder stiffer and results in higher dynamic modulus values. Furthermore, all of the mixes tested herein include recycled asphalt binder in the form of RAP or a combination of RAP and RAS (Table 2.1). It is expected that at a higher temperature used for mixing the HMA mixes more aged and stiffer asphalt binder from RAP and/or RAS might be activated. These stiffer binders would contribute to the total asphalt binder (Al-Qadi et al., 2009; Kvasnak et al., 2009) utilized for aggregate coating, which would lead to a stiffer HMA mix compared to the WMA. A lower dynamic modulus value in WMA may result in a higher susceptibility to rutting compared to HMA mixes. However, a lower stiffness may result in an increased fatigue life of WMA mixes compared to HMA. Therefore, it is important to evaluate the fatigue performance of these mixes using four-point bending beam bending beam method before a conclusion can be drawn on the fatigue life of WMA and HMA mixes. The dynamic modulus values determined for WMA and HMA mixes in this study can be a vital contribution to generate a database for local calibration of the M-EPDG and to study rutting and fatigue performance of WMA pavements using the M-EPDG.

2.7.2 Creep Compliance

The creep compliance (AASHTO T 322, 2007) is used as input parameter for the M-EPDG for predicting the thermal cracking of pavements during service life. The average creep compliance values at different loading times, namely 1 s, 2 s, 5 s, 10 s, 20 s, 50 s, and 100 s, and temperatures namely, -18°C, -10°C, 0°C and 10°C were determined based on the tests conducted on WMA and HMA mixes. Then, these values were used in order to develop creep compliance master curves at 10°C reference temperature (Equation 2.6) and shift factor equation (Equation 2.7) according to the methodology discussed earlier. The Figures 2.4-a, 2.4-b and 2.4-c, show creep compliance master curves for WMA and HMA mixes of Advera® (ADWM and ADHM), Evotherm® Type B (EVWM-B and EVHM-B) and Evotherm® Type C (EVWM-C and EVHM-C, respectively.

From Figure 2.4, it is evident that the creep compliance increases with an increase in loading time and temperature. Also, from Figure 2.4 it was found that in general WMA mixes (ADWM, EVWM-B and EVWM-C) exhibited higher creep compliance values than HMA mixes (ADHM, EVHM-B and EVHM-C). The creep compliance results indicate that the HMA mixes are stiffer compared to their WMA counterparts. This conclusion is consistent with the findings discussed before, based on the dynamic modulus values. A higher creep compliance value means a higher relaxation modulus. This may result in less thermal stress buildup in a pavement as a result of temperature change, and in part, may lead to a better resistance to low-temperature cracking (Lytton et al., 1993). The creep compliance values determined for

WMA and HMA mixes in this study are important input parameters for the M-EPDG for estimation of low temperature cracking potential of WMA pavements.

2.7.3 Fatigue Life

The fatigue life of an asphalt mix is its ability to withstand repeated traffic loads without experiencing failure. The initial stiffness and number of cycles to failure were compared for WMA and HMA mixes. Figure 2.5 presents the initial stiffness and fatigue failure cycles of the WMA and HMA mixes.

From Figure 2.5 it is evident that HMA mixes showed higher fatigue failure cycles compared to their WMA counterparts. For example, the ADHM mix showed average fatigue failure cycles of 404,270, which was 108% higher than that of the ADWM mix, which failed at 193,923 cycles. Similarly, the EVHM-B mix had average fatigue failure cycles of 63,681, which was 249% higher than that of the EVWM-B mix with 18,248 failure cycles. Likewise, the EVHM-C mix exhibited average fatigue failure cycles of 123,671, which is significantly (221%) higher than that of the EVWM-C mix, with failure cycles of 38,473. Furthermore, the initial stiffness values for the ADHM (7445 MPa) and EVHM-C (7396 MPa) mixes, respectively, are 18% and 28% higher than those of ADWM (6112 MPa) and EVWM-C (5290 MPa) mixes. This is consistent with the results of the dynamic modulus tests (Figure 2.3) in which the WMA mixes showed lower dynamic modulus values compared to the HMA mixes. On the contrary, the EVWM-B mix exhibited an initial stiffness value (8675 MPa) which was 20% higher than that of the EVHM-B (7215 MPa) mix. However, according to the dynamic modulus values presented in Figure 2.3, it was expected that the EVHM-B mix would exhibit a higher stiffness compared to the EVWM-B mix. This might be attributed to the aggregate size, segregation, and scale effects on the test results: from Table 2.1 it was observed that the EVWM-B and EVHM-B are the coarsest mixes with a NMAS of 19 mm. Therefore, the ratio of the coarsest aggregate size (19 mm) to the smallest specimen dimension (50 mm) is 0.39. This is a relatively high value and may lead to a considerable scale effect. Furthermore, the scale effect may be combined with the effects due to the inconsistent large particles' arrangement in specimen, and produce initial stiffness values that are inconsistent with the dynamic modulus test results. Figure 2.6 shows the stiffness ratio variations with loading cycles in four-point bending beam fatigue tests, conducted on WMA and HMA mixes in this study.

Figure 2.6-a shows that in spite of a similar trend of stiffness decay with loading cycles for the ADHM and ADWM mixes, a sudden decrease in stiffness, after the approximate 100,000th cycle, was observed for the ADWM mix. This observation is similar to the findings of Goh and You (2011), in which Advera® WMA mixed at 130ºC showed a lower fatigue life compared to HMA mix. Figure 2.6-b shows that the stiffness decay rate for the EVWM-B mix started to increase in a very early stage (after the 10,000th cycle), which caused an early failure. Furthermore, Figure 2.6-c shows that in spite of a similar early stage trend of stiffness decay with loading cycles for the EVHM-C and EVWM-C mixes, stiffness started to decrease with a higher slope after the 10,000th cycle until failure, compared to that of the EVHM-C mix. It can be concluded from the abovementioned observations that in all cases the HMA mixes exhibited a better fatigue performance compared to their WMA counterparts. This was attributed to the fact that all of the mixes tested herein contained RAP or a combination of RAP and RAS (Table 2.1): A higher mixing temperature in HMA mixes may have

caused a more effectively-activated asphalt binder from RAP (and RAS), available for covering aggregates. However, lowering the mixing temperature, as in the WMA case here, may cause some asphalt binder from RAP and RAS to act as “black rock,” and not to participate in the blending process with the virgin binder (Al-Qadi et al., 2009; Kvasnak et al., 2009). Therefore, the HMA will have a higher amount of combined asphalt binder (virgin and reclaimed) available for coating aggregates than that of WMA mixes. A higher asphalt content is known to increase the fatigue life of the asphalt mixes (Harvey et al., 1995). Furthermore, Indirect Tensile Strength (ITS) test conducted on unconditioned specimens, as a part of TSR tests, shows that the HMA control mixes have higher dry ITS values compared with those of the WMA mixes. This supports the hypothesis of a better asphalt binder-aggregate bond in HMA mixes due to more activated reclaimed binder. More in-depth study is needed to investigate the effects of using RAP d RAS in WMA.

2.7.4 Moisture-Induced Damage Potential

2.7.4.1 Retained Indirect Tensile Strength Ratio (TSR)

A summary of the TSR values based on the tests conducted on WMA and HMA mixes is presented in Figure 2.7-a. According to Figure 2.7-a, only three mixes, ADHM, EVHM-C and EVWM-C pass the specification’s minimum TSR requirement of 0.8 with TSR values of 0.93, 0.81 and 0.95, respectively. Thus, based on the TSR results, the EVWM-C mix is expected to be the most resistant mix to moisture-induced damage. Additionally, Figure 2.7-b presents a summary of the average values of the dry and moisture-conditioned tensile strength of the WMA and HMA mixes. From Figure 2.7-b, it is evident that in spite of the highest TSR value obtained for the EVWM-C mix, the moisture-conditioned indirect tensile strength CITS and dry indirect tensile strength (DITS) values for this mix are the lowest among the mixes tested in this study, which does not agree with the findings from the TSR values.

2.7.4.2 Performance Rating Based on Fractured Face Visual Inspection

Furthermore, conditions of the fractured faces of each asphalt sample subjected to TSR test were examined for visual rating of the extent of stripping, according to AASHTO T 283 (AASHTO, 2011). Photographic views of the fractured faces of the representative dry and moisture-conditioned specimens under ITS test (as a part of TSR), are shown in Table 2.5. The visual rating was performed based on a scale of 1 to 4, ranging from no moisture damage (1) to severe moisture damage (4). According to Table 2.4, the ADWM mix had the lowest rating (1), which is an indicator of no moisture-induced damage. The ADHM, EVWM-B, EVHM-B, and EVHM-C mixes were rated as 2, which is the indicator of low moisture damage. However the EVWM-C mix was rated 3, which means high moisture damage was visible on the fractured face. This is consistent with the findings from CITS and DITS values of the EVWM-C mix (Figure 2.7).

2.7.4.3 Stripping Inflection Point using Hamburg Wheel Tracking (HWT)

A summary of the Hamburg Wheel Tracking (HWT) test results for the WMA and HMA mixes is presented in Table 2.6. Also, for further evaluation of the moisture-induced damage potential of the tested mixes, the average rut values for each mix were calculated and plotted. Graphical comparison of rut and resistance to moisture-induced damage of HMA and WMA mixes including ADHM vs. ADWM, EVHM-B vs.

EVWM-B and EVHM-C vs. EVWM-C mixes are shown in in Figures 2.8-a, 2.8-b and 2.8-c, respectively.

According to Table 2.6 and Figure 2.8, it is evident that none of the above mentioned mixes, except the EVWM-C mix, showed an inflection point, which is an indicator of high resistance to moisture-induced damage. It should be noted that ArrMaz® HP plus and lime were used as anti-stripping agents for Advera® (ADWM and ADHM) and Evotherm® Type C (EVWM-C and EVHM-C) mixes, respectively. However no anti-stripping agent was used for Evotherm® Type B (EVWM-B and EVHM-B) mixes. Acceptable resistance to moisture-induced damage of the mixes (except EVWM-C) may be attributed to their aggregate asphalt binder and anti-stripping agent (if used) compatibility. Comparatively, from Figure 2.8-c it was observed that all three characteristic moisture-induced damage regions are evident in the EVWM-C mix. The EVWM-C mix becomes prone to moisture-induced damage and stripping of aggregates from asphalt binder starts at an inflection point after 12,593 wheel passes with an inverse stripping rate of 2,314 pass/mm (Table 2.6). It is worth noting that, since the number of the wheel passes corresponding to the inflection point is greater than 10,000 wheel passes, the EVWM-C mix marginally passes the mix design requirements for resistance against moisture-induced damage. An inflection point below 10,000 wheel passes indicates significant moisture-induced damage potential. Stripping as an indication of moisture-induced damage was observed for the EVWM-C mix, where lime was used as the anti-stripping agent. Since, Figure 2.5-c shows no detectable inflection point associated with the EVHM-C mix, which may be a result of possible incompatibility between the aggregate-asphalt binder-lime and chemical WMA additive used for production of the EVWM-C mix. Similar to the HWT test results, the visual fractured face rating (Table 2.5) as an extent of the moisture-induced damage showed that the EVWM-C mix, with a visual rating of 3, was expected to have a high moisture-induced damage potential. The other five mixes do not show a high rating (ranging from 1 to 2), which are comparatively similar to the findings from HWT tests.

2.7.5 Comparison of Moisture-Induced Damage Potential Evaluated by TSR and HWT

Screening of mixes using the TSR test have shown that the ADWM, EVHM-B and EVWM-B mixes would not pass the requirements for resistance against moisture-induced damage (Figure 2.7-a). However, the HWT results, according to Table 2.6, suggest the EVWM-C mix to be the only mix prone to moisture-induced damage, with a detectable SIP. Also, according to Table 2.6, based on the HWT test results, the ADWM and EVWM-B mixes performed equally well against moisture-induced damage when compared with their HMA control mixes (ADHM and EVHM-B, respectively). To this end, a significant difference between the results from the TSR and the HWT tests in term of moisture-induced damage evaluation of the WMA and HMA mixes is observed. According to Figure 2.7-b, the EVWM-C mix has the lowest average CITS value compared to the other asphalt mixes tested for TSR. The latter observation may suggest the use of (some form of) CITS value instead of TSR as an indicator of moisture-induced damage. This recommendation requires additional studies of a larger number of mixes to be better substantiated. Thus, a minimum CITS value may also be considered as a pass/fail criterion for a mix. Similar to the HWT test results, the visual fractured face rating (Table 2.6) as a measure of the extent of the moisture-induced damage showed that the EVWM-C mix, with a visual rating of 3, was expected to have a high moisture-induced damage potential. The other five mixes do not show a high rating (ranging from 1 to 2), which are comparatively similar to the findings from the HWT tests.

In conclusion, it was observed that only the EVWM-C mix with lime showed significant moisture-induced damage, possibly due to incompatibility of its WMA additive with asphalt binder, aggregate and lime. Therefore, an in-depth study of the compatibility of the different chemicals used in the asphalt mixes is recommended. Also, it was observed that the TSR value by itself was not able to differentiate the mixes that were prone to moisture-induced damage when the results were validated with the HWT data. However, it was observed that information from the DITS and CITS tests data can be used in conjunction with the HWT test results to evaluate the moisture-induced damage potential. Use of a purely mechanistic approach, namely surface free energy method, is known to successfully predict the moisture-induced damage potential of WMA and HMA (Ghabchi et al., 2013a; Ghabchi et al., 2013b; Ghabchi et al., 2013c, Arabani et al., 2012; Wasiuddin et al., 2008; Wasiuddin et al., 2007; Bhasin and Little, 2007; Lytton et al., 2005; Cheng et al., 2002).

2.7.6 Rutting Performance

From Table 2.6 and Figure 2.8-a, it is evident that the ADHM mix showed an average rut depth of 1.4 mm after 20,000 wheel passes, which is very close to that of the ADWM mix that exhibited a rut depth of 1.9 mm due to 20,000 wheel passes. Also, from Table 2.6, the measured average inverse creep rate for the ADHM mix was 47,627 pass/mm, which is 60% higher than that of the ADWM mix (29,683 pass/mm). This can be interpreted as a better long-term rut performance of the ADHM mix compared to the ADWM mix. This was attributed to the lower production temperature of the ADWM mix (130°C) compared to the ADHM mix (160°C). Lower mixing temperature used for production of the ADWM mix may have resulted in less asphalt binder aging, leading to a softer mix compared to the ADHM mix and making the ADWM mix more prone to rutting. Similarly, according to Table 2.6 and Figure 2.8-b, it was found that the EVHM-B mix showed an average rut depth of 1.5 mm after 20,000 wheel passes, which is considered to be negligibly higher than that of the EVWM-B mix, with an average rut depth of 0.9 mm. Also, from Table 2.5, the measured average inverse creep rate for the EVHM-B mix was 30,906 pass/mm, which is 11% higher than that of the EVWM-B mix (27,879 pass/mm). This is an indication of better long term rut performance of the EVHM-B mix compared to the EVWM-B mix, and may be attributed to the lower mix production temperature of the EVWM-B mix (135°C) compared to control EVHM-B mix (160°C). Furthermore, from Table 2.6 and Figure 2.8-c, it was observed that the EVHM-C mix showed an average rut depth of 2.2 mm after 20,000 wheel passes, which is significantly lower than that of the EVWM-C mix, with an average rut depth of 6.4 mm. But, since an SIP was observed for the EVWM-C mix, the rut depth at the end of the test cannot be used to measure the rutting performance of EVWM-C. However, the measured average inverse creep rate (Table 2.6) for the EVHM-C mix was 22,031 pass/mm, which is 68% higher than that of the EVWM-C mix (7,127 pass/mm). This is an indication of better long term rut performance of the EVHM-C mix compared to the

EVWM-C mix. This may be attributed to the lower mix production temperature of the EVWM-C (135°C) compared to the control EVHM-C mix (160°C).

2.7.7 Rutting Performance Relationship with Indirect Tensile Strength (ITS)

Further study of the data from the TSR and HWT tests were carried out in order to investigate possible correlations between the rutting potential and ITS test results. Figure 2.9 shows the variations of rut depth at 20,000 wheel passes with dry DITS values of tested asphalt samples, resulting from HWT and TSR tests, respectively.

As expected, from Figure 2.9, it was observed that the rut depths decreased as the DITS of the tested asphalt samples increased. Also, a regression model in the form of a power function was developed and displayed on the chart. The coefficient of determination calculated for this model (Rut Depth = 9,468,669 x $DITS^{-2.101}$) is 0.88, which is an indication of good correlation between the measured rut depths (mm) and DITS values (kPa). The rut depths shown in Figure 2.9 are the deformations measured on the asphalt samples after 20,000 wheel passes in a HWT test. However, as discussed earlier, for the EVWM-C mix in which the SIP was observed (shown with grey mark), the measured deformation at the end of a HWT test includes the combined effects of rutting and the moisture-induced damage. For this reason, EVWM-C appears to behave differently when it was compared with other mixes without a SIP. This introduces nonlinearity to the regression equation. In order to capture the correlation between rutting and the DITS, while isolating the moisture-induced damage effect, selection of a characteristic factor representing the pure rutting due to the wheel passes is necessary. For this purpose, the variation of the inverse rutting rate (IRR) with respect to DITS values was plotted, and is shown in Figure 2.10.

From Figure 2.10 it was observed that the IRR values of different asphalt samples increased with an increase in DITS. In other words, resistance to rutting increased with increasing tensile strength of the asphalt samples. Also, a linear regression model (IRR = 34.591 x DITS -24,507) was developed and is displayed on the chart. The coefficient of determination calculated for the above mentioned model is 0.89, which shows a good correlation between the measured rutting rate and the DITS values. It was observed that use of the IRR value successfully eliminated the high values of deformation as a result of SIP and moisture-induced damage effect. Hence, IRR is recommended to be used as the indication of rutting in asphalt mixes in which the SIP is observed.

2.8 Conclusions

WMA and HMA mixes, consisting of one gradation of Advera® and two gradations of Evotherm® mixes (a total of 6 mixes), were characterized using laboratory performance tests. Laboratory tests consisted of dynamic modulus, creep compliance, four-point bending beam fatigue, Hamburg wheel tracking and retained indirect tensile strength tests. Based on the results and discussion presented in this study, the following conclusions can be drawn:

1. The dynamic modulus values of the WMA were lower than those of HMA mixes. This may result in a higher rutting susceptibility of WMA in long term. Less asphalt binder aging in the production of WMA due to a lower mixing temperature compared to the HMA case was found to be responsible for this difference. This difference was more pronounced for the coarsest mix (NMAS = 19 mm), Evotherm® Type B, with its HMA control mix.

2. The creep compliance values of the WMA were higher than those of HMA control mixes. This results in a higher relaxation modulus and therefore may contribute to a higher resistance to low-temperature cracking. Also, it was concluded that temperature sensitivity of the creep compliance reduces with an increase in NMAS.
3. It was observed that the fatigue lives of all HMA mixes tested in this study were higher than those of the WMA mixes. The difference between the fatigue life of the WMA and HMA mixes was more pronounced for the Evotherm® Type B mix compared to the HMA control mix.
4. It was concluded that the WMA mixes showed more susceptibility to rutting than the HMA mixes when inverse rutting rate was used to evaluate the rutting potential. However, the WMA and HMA mixes performed almost equally well with respect to rutting when the total rut depth was used as a rutting indicator.
5. It was found that Evotherm® Type C WMA, with lime as anti-stripping agent, exhibited a stripping inflection point in the Hamburg wheel tracking test. The observed moisture-induced damage was attributed to possible incompatibility of the Evotherm® additive and lime with the aggregates used in the mix.
6. According to the retained indirect tensile strength ratio test results, only HMA mixes of Advera® HMA and Evotherm® Type C WMA and HMA mixes passed the minimum TSR requirement (0.8), and other mixes showed lower TSR values and did not pass the TSR requirement.

7. It was concluded that the TSR and Hamburg wheel tracking tests can result in contradictory outcomes on the moisture-induced damage potential of WMA and HMA mixes, as observed in the present study.
8. It was found that excellent correlations exists between the rutting depth, rutting ratio and dry (unconditioned) indirect tensile strength (DITS) of asphalt mixes. Furthermore, obtaining the DITS value through indirect tensile strength test is comparatively quicker and easier than conducting a Hamburg wheel tracking test and/or using an asphalt pavement analyzer on field cores and laboratory-compacted samples. Therefore, the proposed method may be used for quick evaluation of mixes for rutting in addition to other methods.

It is recommended that the effect of using RAP and RAS on mechanical properties of WMA be studied in detail. Furthermore, it is recommended that the compatibility of different additives (i.e., WMA additives, anti-stripping agents, lime) and asphalt binders be studied with the different types of the aggregates used in WMA mixes, against moisture-induced damage.

Table 2.1 Summary of the WMA and HMA Mix Properties

Mix Type	NMAS† (mm)	Pavement Layer	Mix Type	PG Grade	RAP/RAS (%)	Temperature (ºC)		WMA Additive	Anti-Stripping Agent
						Mixing	Placing		
ADHM	9.5	Overly	C-HMA*	64-22	15 / 2.4	160	145	-	AD-Here‡
ADWM	9.5	Overlay	WMA	64-22	15 / 2.4	130	115	Advera®	AD-Here‡
EVHM-B	19.0	Base	C-HMA*	64-22	20 / 2.5	160	145	-	-
EVWM-B	19.0	Base	WMA	64-22	20 / 2.5	135	121	Evotherm®	-
EVHM-C	12.5	Surface	C-HMA*	70-22	10 / 0	160	145	-	Lime
EVWM-C	12.5	Surface	WMA	70-22	10 / 0	135	121	Evotherm®	Lime

* Control HMA

†Nominal maximum aggregate size.

‡ArrMaz AD-Here HP Plus®

Table 2.2 Model Evaluation Criteria (Witczak, 2005)

Rating	R^2	S_e/S_y
Excellent	≥ 0.90	≤ 0.35
Good	0.70 - 0.89	0.36 - 0.55
Fair	0.40 - 0.69	0.56 - 0.75
Poor	0.20 - 0.39	0.76 - 0.90
Very poor	≤ 0.19	≥ 0.90

Table 2.3 Dynamic Modulus Master Curve Model Parameters

Mix Type	|E*| Master Cuve Parameters (MPa)					Goodness-of-fit Statistics		
	α	β	γ	δ	c	R^2	S_e/S_y	Rating
ADHM	3.935	-1.353	-0.259	0.725	9639.3	0.997	0.038	Excellent
ADWM	4.161	-0.734	-0.205	0.873	10058.6	0.991	0.081	Excellent
EVHM-B	3.098	-1.727	-0.390	1.502	10638.2	0.993	0.044	Excellent
EVWM-B	4.697	-1.439	-0.238	0.044	10152.0	0.997	0.045	Excellent
EVHM-C	2.537	-0.658	-0.450	2.165	10444.7	0.997	0.043	Excellent
EVWM-C	2.498	-0.709	-0.474	2.054	10107.1	0.997	0.028	Excellent

Table 2.4 Dynamic Modulus Master Curve Shift Factor Model Parameters

Mix Type	Shift Factor Parameters			Goodness-of-fit	
	m	n	p	R^2	Rating
ADHM	0.0004	-0.127	2.520	1.00	Excellent
ADWM	0.0004	-0.132	2.629	1.00	Excellent
EVHM-B	0.0004	-0.140	2.781	1.00	Excellent
EVWM-B	0.0004	-0.133	2.654	1.00	Excellent
EVHM-C	0.0004	-0.137	2.730	1.00	Excellent
EVWM-C	0.0004	-0.133	2.642	1.00	Excellent

Table 2.5 Fractured Faces of Asphalt Mixes, and Visual Ratings of TSR Test

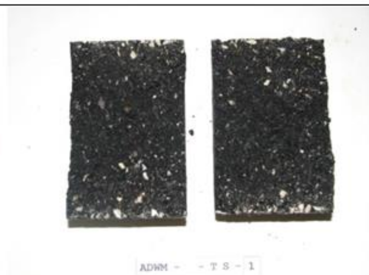
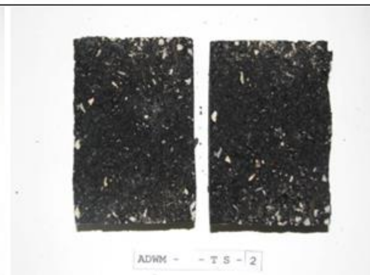
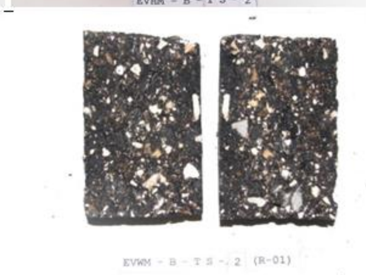
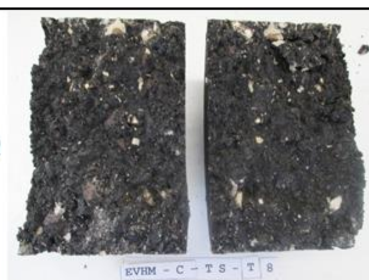
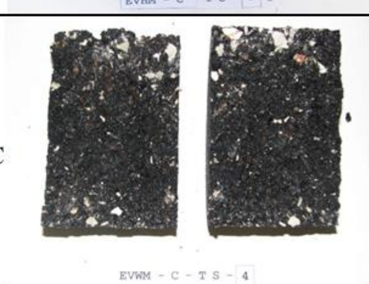
Mix Type	Fractured Section		Visual Inspection
	Tested in Dry Condition	Moisture Conditioned	Rating of Stripping
ADHM	Picture not Available	Picture not Available	2
ADWM			1
EVHM-B			2
EVWM-B			2
EVHM-C			2
EVWM-C			3

Table 2.6 Summary of the HWT tests conducted on WMA and HMA mixes

Mix Type	Max Deformation		Rut Depth (mm)				Rutting Rate		Inflection Point	
	Wheel Passes	Deformation (mm)	5,000 Passes	10,000 Passes	15,000 Passes	20,000 Passes	Creep (pass/mm)	Stripping (pass/mm)	Passes	Deflection (mm)
ADHM	20,000	1.4	1.1	1.2	1.3	1.4	47,627	-	>20,000	-
ADWM	20,000	1.9	1.4	1.6	1.8	1.9	29,683	-	>20,000	-
EVHM-B	20,000	1.5	0.9	1.1	1.3	1.5	30,906	-	>20,000	-
EVWM-B	20,000	0.9	0.6	0.7	0.8	0.9	27,344	-	>20,000	-
EVHM-C	20,000	2.2	1.4	1.7	2.0	2.2	22,031	-	>20,000	-
EVWM-C	20,000	6.4	2.1	2.8	4.3	6.4	7,127	2,314	12,593	3.4

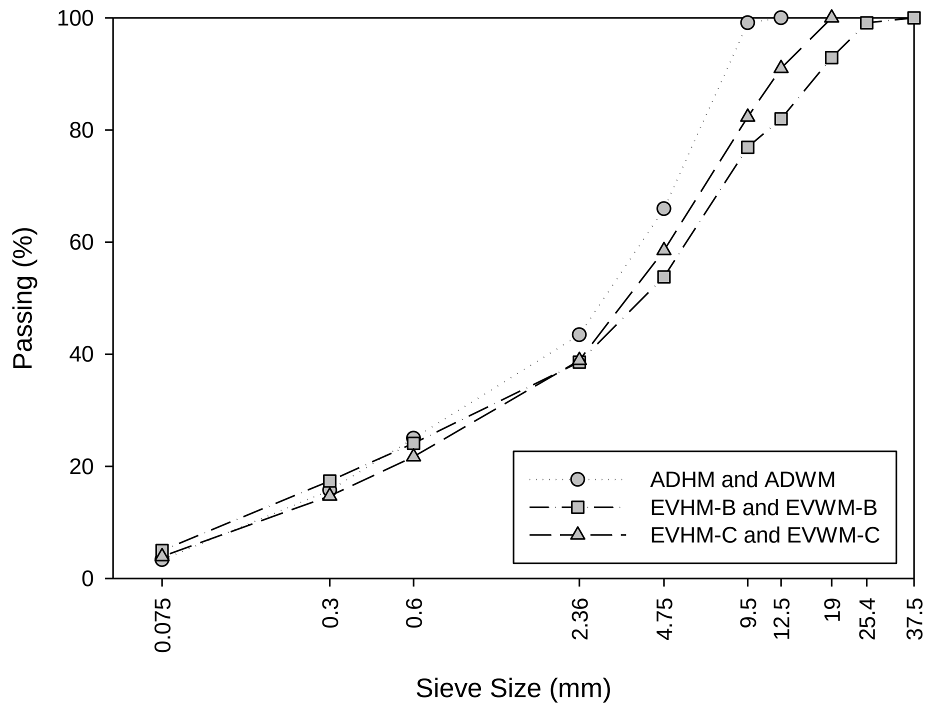


Figure 2.1 Gradations of the WMA and HMA Mixes

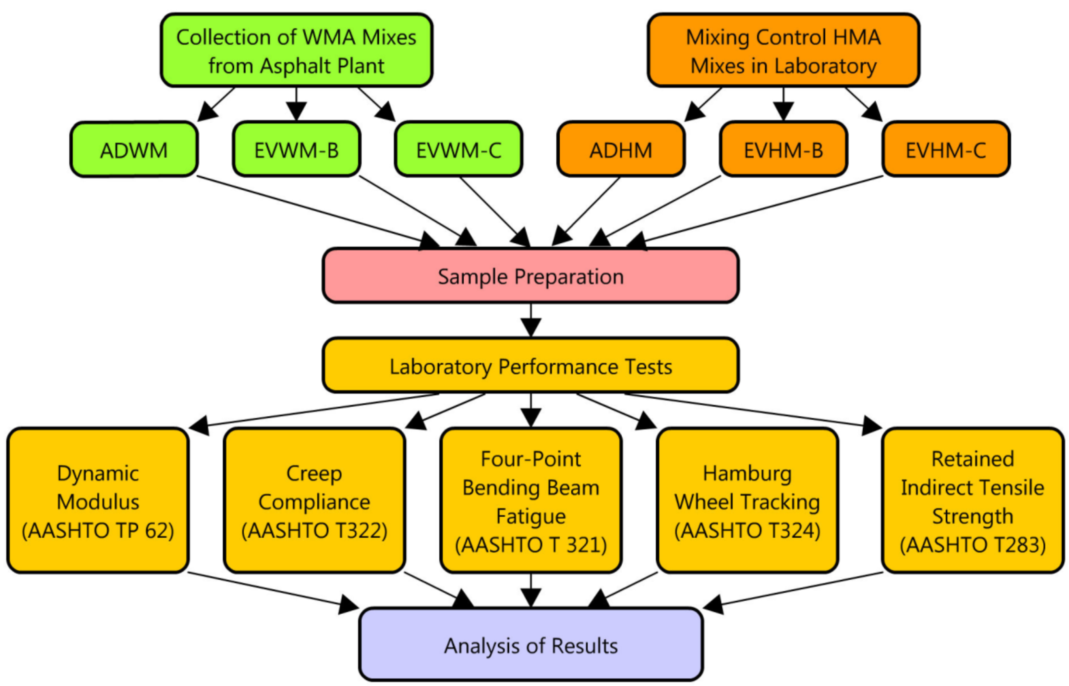


Figure 2.2 Work Flow and Testing Plan

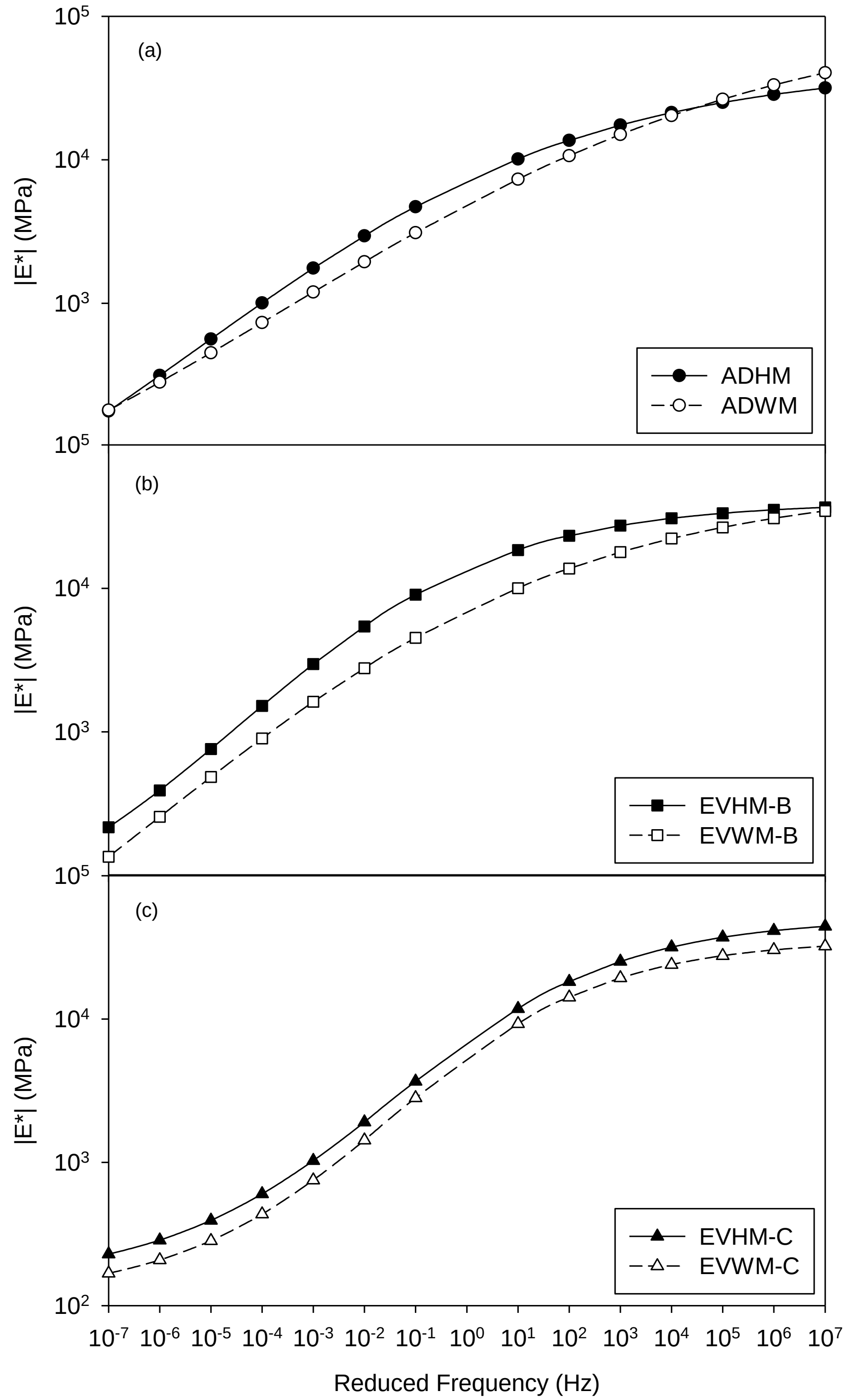


Figure 2.3 Dynamic Modulus Master Curves for (a) ADHM and ADWM, (b) EVHM-B and EVWM-B, and (c) EVHM-C and EVWM-C at 21.1°C Reference Temperature

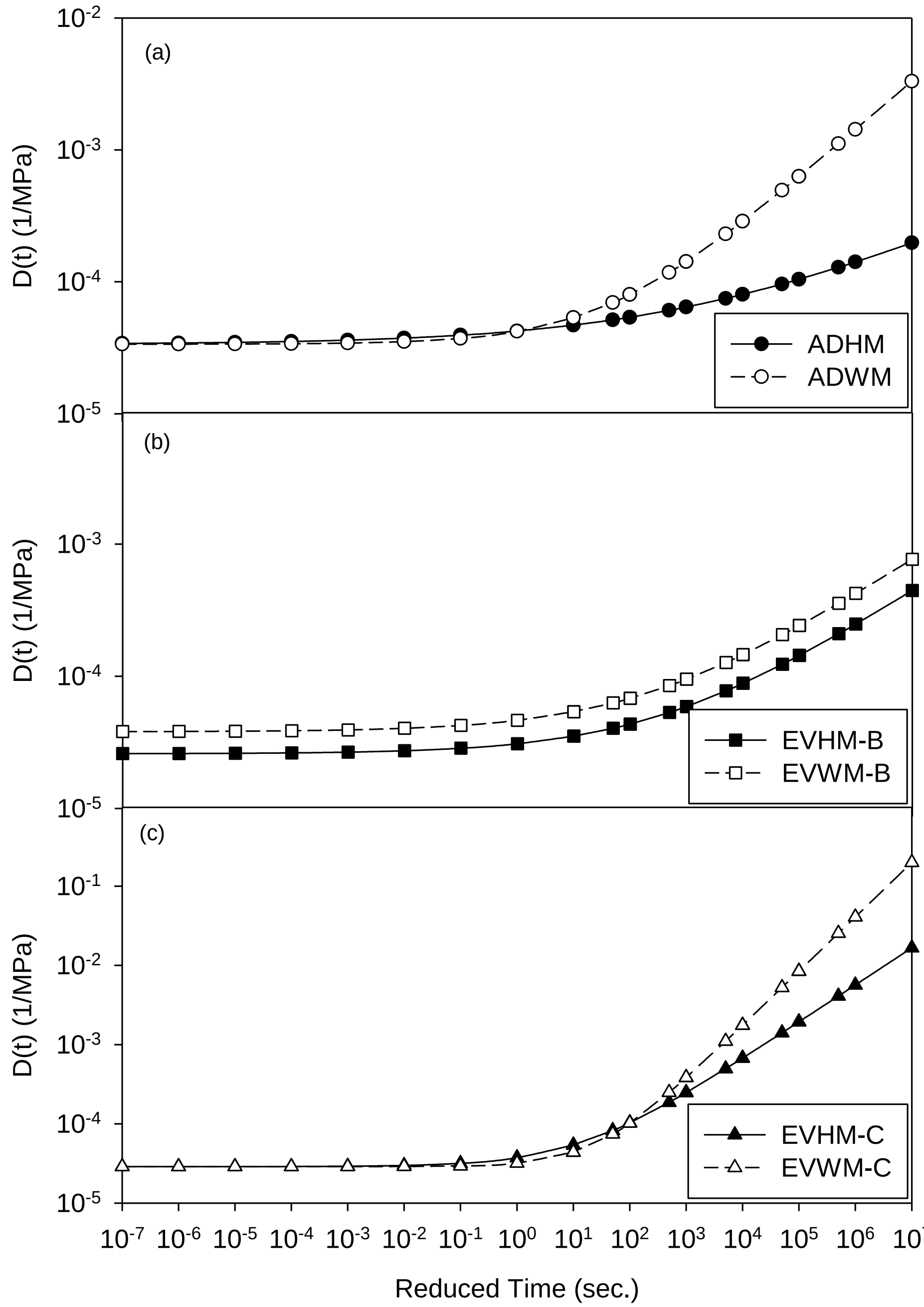


Figure 2.4 Creep Compliance Master Curves at 10°C Reference Temperature for (a) ADHM and ADWM, (b) EVHM-B and EVWM, and (c) EVHM-C and EVWM-C

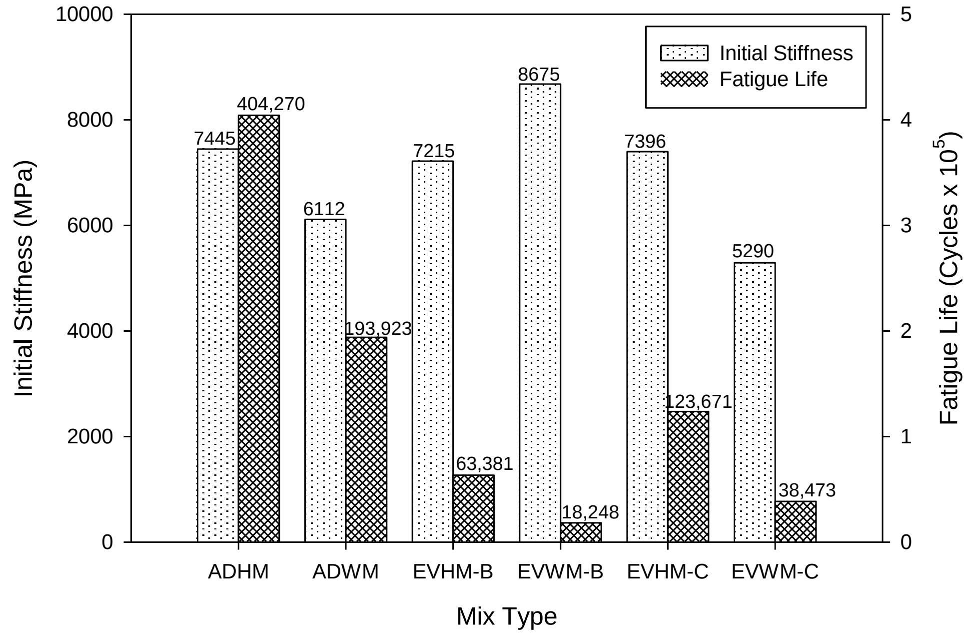


Figure 2.5 Four-Point Bending Beam Fatigue Test Results Conducted on WMA and HMA Mixes

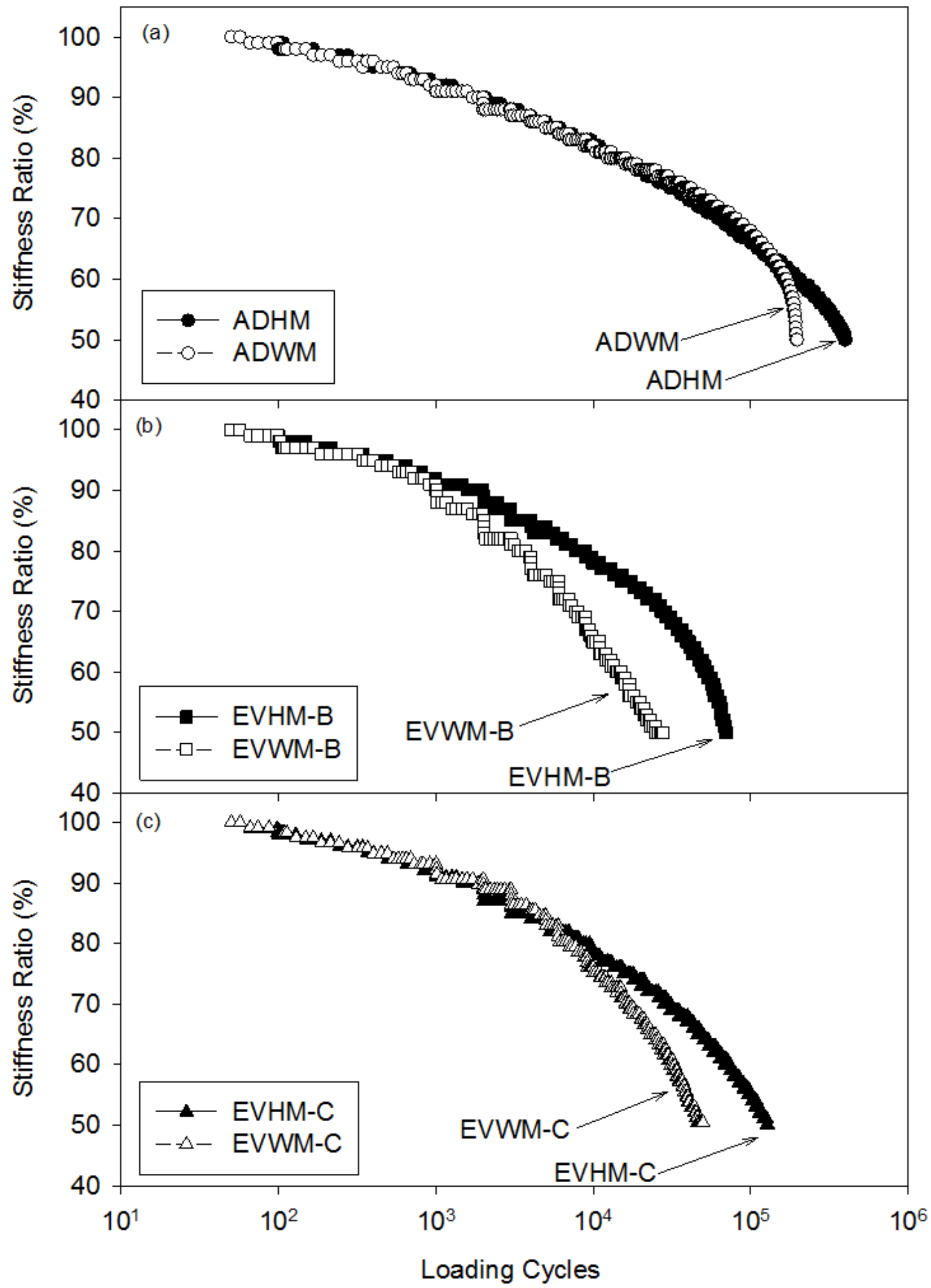


Figure 2.6 Stiffness Ratio Variations with Loading Cycles in Four-Point Bending Beam Fatigue Test Conducted on (a) ADHM and ADWM, (b) EVHM-B and EVWM, and (c) EVHM-C and EVWM-C

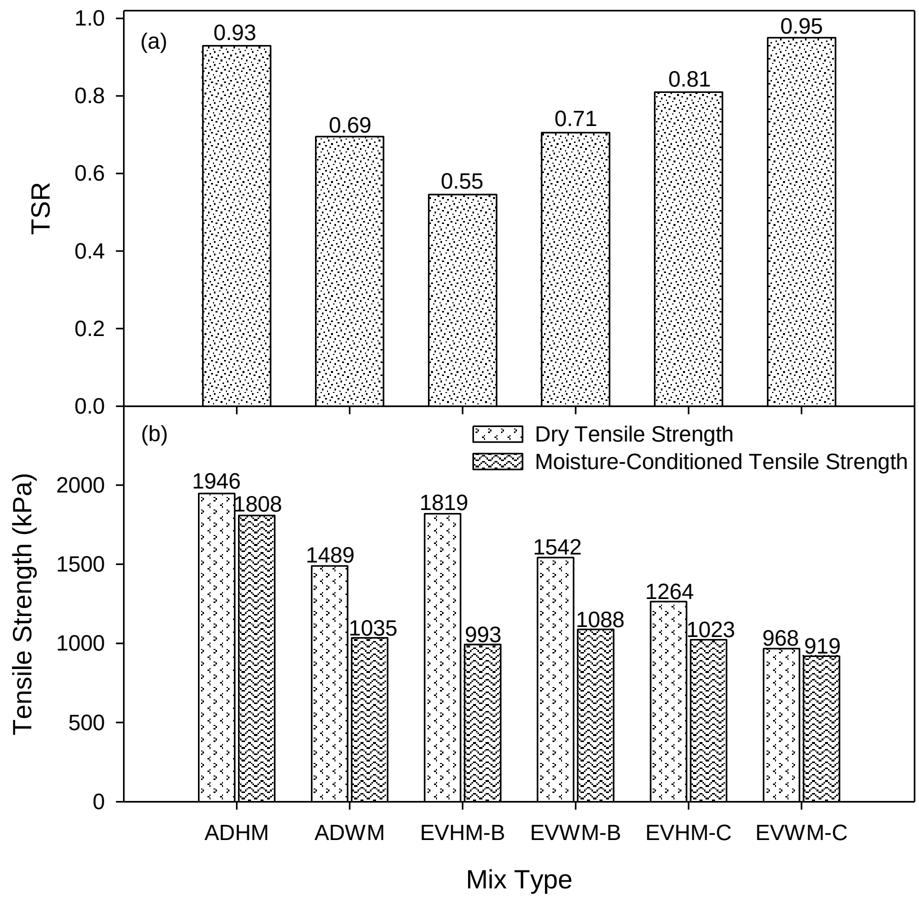


Figure 2.7 (a) Average Dry and Moisture-Conditioned Tensile Strength, and (b) TSR Values of WMA and HMA Mixes

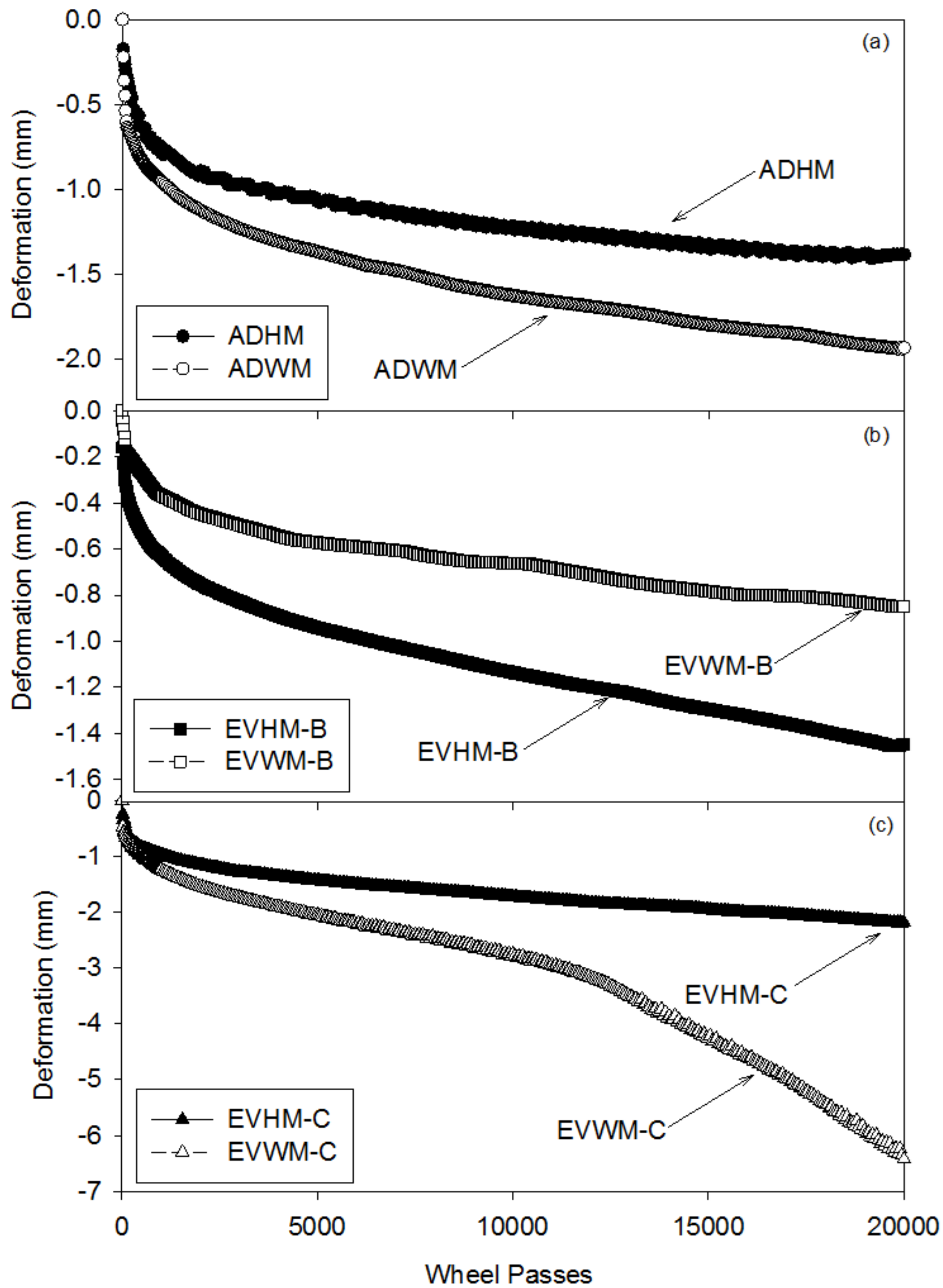


Figure 2.8 Hamburg Wheel Tracking Curves for (a) ADHM and ADWM, (b) EVHM-B and EVWM, and (c) EVHM-C and EVWM-C

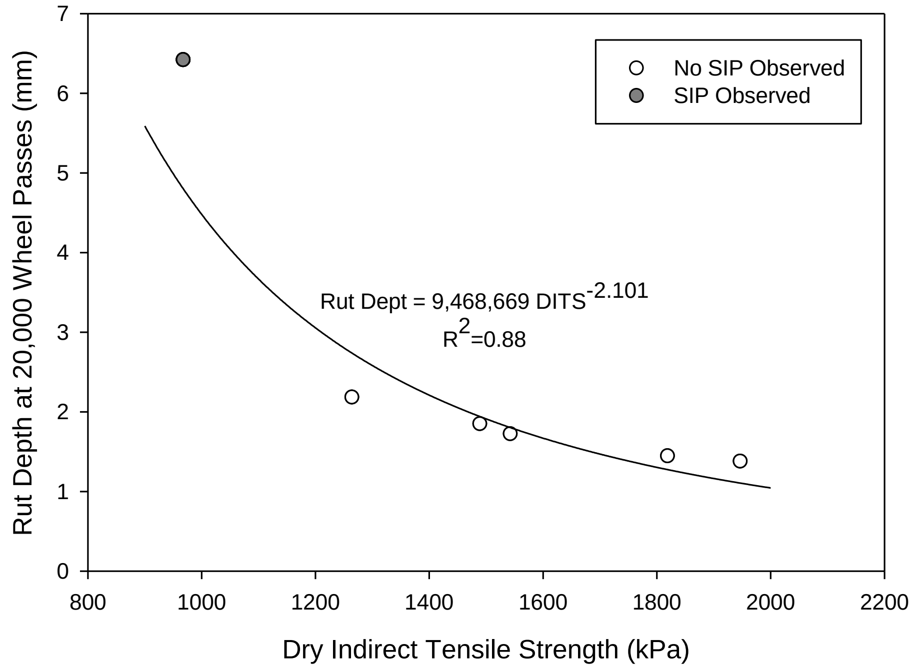


Figure 2.9 Variations of Average Rut Depth with Dry Indirect Tensile Strength of Asphalt Mixes

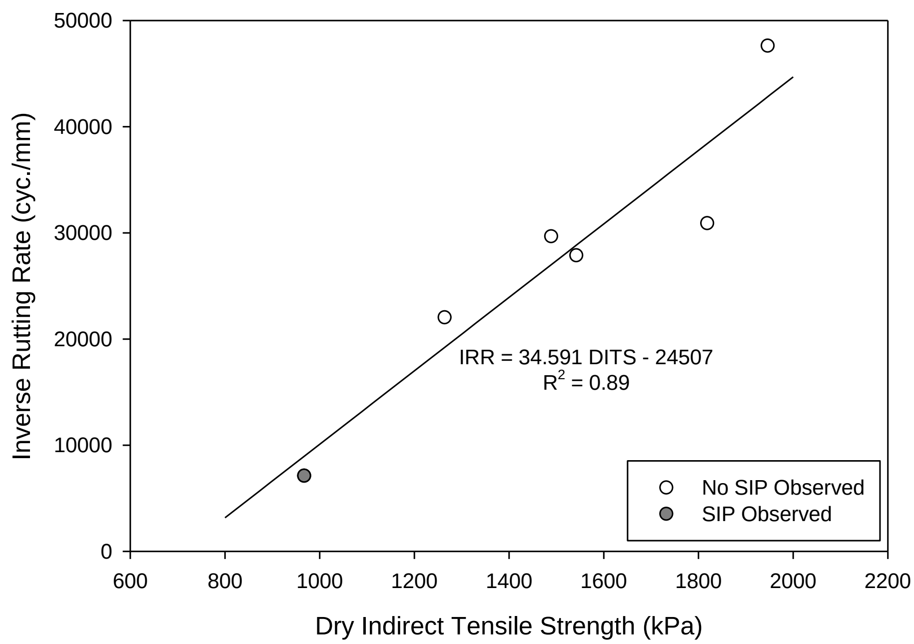


Figure 2.10 Variations of the Average Inverse Rutting Rates with Dry Indirect Tensile Strength

CHAPTER

3

LABORATORY CHARACTERIZATION OF ASPHALT MIXES CONTAINING RAP AND RAS[‡]

ABSTRACT

Due to its economic and environmental benefits, using reclaimed asphalt pavement (RAP) and reclaimed asphalt shingles (RAS) in new hot-mix asphalt (HMA) has become an integral part of today's asphalt industry. The advantages of using RAP and RAS in HMA are not limited to economic and environmental benefits, and may result in improving a number of mix performance characteristics including rutting and resistance to moisture-induced damage. Despite aforementioned benefits, concerns over premature pavement distresses as a result of using RAP and RAS limit their usage in HMA. Furthermore, because of the lack of mechanistic performance data, use of new mixes containing RAP and RAS remains limited. Therefore, the present study was undertaken to investigate the effects of using different amounts of RAP and RAS on laboratory performance of HMA, and to generate valuable input design parameters for implementation of the Mechanistic-Empirical Pavement Design Guide (M-EPDG),

[‡] This chapter has been submitted to the International Journal of Pavement Engineering under the title "Laboratory Characterization of Asphalt Mixes Containing RAP and RAS." The current version has been formatted for this dissertation.

using local materials. Four types of base course mixes containing 0% RAP, 25% RAP, 40% RAP, and 20% RAP+5% RAS, and three types of surface course mixes containing 0% RAP, 25% RAP, and 20% RAP+5% RAS were tested. Laboratory tests were conducted to evaluate stiffness, low-temperature cracking, fatigue life, rut, and moisture-induced damage potential of the mixes. It was found that dynamic modulus and creep compliance of the asphalt mixes increase and decrease, respectively, with an increase in the amount of RAP and/or RAS used in the mix. Fatigue life was found to increase with increasing RAP content up to 25%, and to decrease when the RAP content exceeded 25%, or when RAS was used in the mix. It should be noted that this conclusion was based on conducting fatigue tests on asphalt mixes with 0%, 25%, and 40% RAP contents. Hamburg wheel tracking (HWT) test results showed increased resistance to rutting and moisture-induced damage, with an increase in the amount of RAP and/or RAS. However, the TSR test results were not confirmed by HWT. The findings of this study are expected to be helpful in understanding the effects of using different amounts of RAP and RAS on the performance of asphalt mixes produced using local materials. Furthermore, valuable design input parameters, developed in this study for new mixes containing RAP and RAS, may be used for calibration of the M-EPDG input parameters, with local materials.

Keywords: Reclaimed asphalt pavement (RAP), reclaimed asphalt shingles (RAS), hot-mix asphalt (HMA), dynamic modulus, creep, rut, moisture damage, fatigue.

3.1 Introduction

Re-using reclaimed asphalt materials, namely reclaimed asphalt pavement (RAP) and reclaimed asphalt shingles (RAS), in new pavements has become an integral part of today's asphalt industry. This is due to economic and environmental benefits associated with using RAP and RAS in hot-mix asphalt (HMA). In recent years, with increasing asphalt binder cost (due to global rise of oil price) and scarcity of high quality virgin aggregates, the demand for using RAP and RAS in asphalt mixes has increased steadily. The use of RAP in new pavements was expected to be doubled by 2014 (NAPA, 2009), at the national level. RAP is being reused in new pavements at a rate of over 99 percent (NAPA, 2011). Use of RAP and RAS in 2010 conserved approximately 20.5 million barrels of asphalt binder (NAPA, 2011). Assuming an average of 5 percent asphalt binder in RAP, it is a good source of asphalt binder and high quality aggregates for new HMA. The advantages of using RAP and RAS in HMA are not limited to economic and environmental benefits. It is reported that using RAP may result in an increase in resistance of asphalt mix to rutting (Al-Qadi et al., 2012; Huang et al., 2004; Mohammad et al., 2003; McDaniel and Shah, 2003b). Experimental data revealed that use of asphalt mixes with a higher RAP content results in dynamic modulus values that are higher than those of control mixes without any RAP (Li et al., 2008; Huang et al., 2004; McDaniel and Shah, 2003a). Li et al. (2008) also concluded that the RAP source is not a significant factor affecting the dynamic modulus at low temperatures. On the other hand, Huang et al. (2004) reported an improvement in fatigue life of asphalt mixes, when RAP was incorporated in the mix. Several other studies show the use of RAS in HMA to be technically feasible (Sengoz and Topal, 2005; Rajib et al., 2000; Foo et al., 1999; Ali et al., 1995; Button et al., 1995; Grzybowski, 1993). RAS contains between 19 and 36 percent high viscosity asphalt binder and 20 to 38 percent ceramic fillers and fibers that are potentially desirable components for HMA (CIWMB, 2007). The literature review reveals that the incorporation of RAS results in an improvement in rutting resistance and fatigue life of asphalt mixes, while the moisture-induced damage potential of these mixes remains unaffected (Austin, 2011; McGraw, 2010; Cascione et al., 2010; Baaj, 2007; Ali et al., 1995; Grzybowksi, 1993). In different laboratory studies, Yang et al. (2014), Johnson et al. (2010) and Li et al. (2008) concluded that mixes containing RAS exhibited higher stiffness as compared to virgin mixes. It is also shown that the improvement in HMA properties depends on the amount (up to 5%) and the source of the RAS (McGraw et al., 2007; UL-Islam, 2010; Ddamba, 2011). For example, Krivit (2007) and Lum et al. (2004) reported that asphalt mixes containing up to 5% RAS (by weight) had equal field performance compared with the HMA mixes without RAS. In a laboratory study conducted by Button et al. (1995), it was concluded that the use of RAS may increase the resistance of the asphalt mix to moisture-induced damage. Asphalt producers sometimes prefer to incorporate both RAP and RAS in asphalt mixes: In a study by Mogawer et al. (2011) it was concluded that using RAP, RAS or both in asphalt mixes resulted in a higher mix stiffness and a bump in the performance grade (PG) of the extracted asphalt binder. Furthermore, it was found that mixes containing RAP, RAS, or both have better resistance to moisture-induced damage. In a recent study conducted by Yang et al. (2014), it was found that use of RAP or RAS increases the stiffness of the asphalt mixes in high and low frequencies; however, no differences were found between the field performances of virgin asphalt mix and those containing RAP and/or RAS.

Despite aforementioned benefits, concerns over premature pavement distresses resulting from using RAP and RAS limit their usage in HMA. For example, a reduction in fatigue life due to using more than 20% RAP in asphalt mixes was reported by McDaniel et al. (2000). Consequently, it was recommended that a virgin binder of a lower grade be used to address the fatigue performance issue, especially at high RAP contents. This conclusion is contrary to that drawn by Huang et al. (2004), who reported an improvement in fatigue life due to addition of RAP in asphalt mixes. Similarly, Shu et al. (2008) concluded that using RAP in HMA reduces the fatigue life of asphalt mixes. Another study conducted by You et al. (2011a) showed that the creep stiffness and thermal-cracking potential of the mix increases with an increase in the amounts of RAP and RAS. The concerns associated with the use of RAP and RAS in HMA, as found in the literature, are due to the physical and rheological changes (e.g. bumping in PG grade) in asphalt binder of the final mix, as a result of using RAP and RAS. For example, according to AASHTO standard (AASHTO PP 53, 2012; AASHTO MP 15, 2012), if the percentage of liquid binder contributed by RAS and RAP in the mix exceeds 30 percent of total binder (by weight), the composite binder needs further evaluation. This is in order to ensure the performance grade of the final blended HMA to comply with performance grade requirements set by specifications.

Although several researchers have investigated different performance characteristics of HMA mixes containing RAP and RAS, results are widely mixed and no clear conclusions could be drawn (Al-Qadi et al., 2007). This may be due to the effect of source and local materials used in asphalt mixes. Also, the available information about the effect of RAP content on the mechanical properties of asphalt mixes is still limited (Li et al., 2008). Hence, there is a need for additional study involving both laboratory and field components, in order to examine the effects of using

RAP and RAS and local materials on the mix performance. As of now, all of the state highway agencies allow the use of RAP in base course, but 10 agencies do not allow RAP in surface course mixes. Other agencies permitting the incorporation of RAP in surface course limit its use to certain amounts. Comparatively, 15 states permit limited use of RAS in asphalt mixes (Yang et al., 2014). For example, Oklahoma Department of Transportation (ODOT) allows use of no RAS in surface and base courses, no RAP in surface course, and up to 25% RAP by the total aggregate weight in the base course. This is partly because of the lack of mechanistic performance data on new mixes containing RAP and RAS. Lack of such data for local materials and other long-term performance test results are major constraint for DOTs to develop new specifications and to allow the use of higher amounts of RAP and RAS. Consequently, there is a need to evaluate the performance of different asphalt mixes containing RAP and RAS produced with local aggregates and asphalt binders. This will be instrumental to generate valuable test data in order to help DOTs to develop desired specifications.

Therefore, the present study was undertaken to investigate the effects of using different amounts of RAP and RAS on performance of HMA, in a laboratory setting. Furthermore, important input design parameters for implementation of the Mechanistic-Empirical Pavement Design Guide (M-EPDG), using local materials were developed. For this purpose, four types of base course mixes with a nominal maximum aggregate size (NMAS) of 19.0 mm and three types of surface course mixes with an NMAS of 12.5 mm, were designed and tested in the laboratory. Base course mixes consisted of the following: (i) virgin mix without RAP or RAS (B-0R); (ii) mix containing 25% RAP (B-25R); (iii) mix containing 40% RAP (B-40R); and (iv) mix containing 20%

RAP and 5% RAS (B-20R/5S). Surface course mixes consisted of the following: (i) virgin mix without RAP and RAS (S-0R); (ii) mix containing 25% RAP (S-25R); and (iii) mix containing 20% RAP and 5% RAS (S-20R/5S). For designing asphalt mixes with RAP and RAS, attainment of good performance against fatigue, rutting, low temperature cracking, and moisture induced damage, while optimizing the mix proportions is important. Therefore, a wide range of tests were conducted on these mixes to evaluate their stiffness, low-temperature cracking, fatigue life, rut, and moisture-induced damage potential. In the new M-EPDG, the dynamic modulus of asphalt mixes is a key input parameter which controls the fatigue cracking and rutting resistance of asphalt pavements (Li et al., 2008; AASHTO, 2004). Furthermore, the M-EPDG simulates thermal cracking using the indirect tensile creep test data (Li et al., 2008). Therefore, dynamic modulus and creep compliance tests were conducted in this study to evaluate their stiffness and low-temperature cracking potential as the M-EPDG input parameters for Level 1 pavement design. Fatigue performance of the asphalt mixes was evaluated using the four-point bending beam fatigue test. The effect of using RAP and RAS on the moisture-induced damage potential of the mixes was evaluated using two different methods, namely retained indirect tensile strength ratio (TSR) and the Hamburg wheel tracking (HWT) test. Rutting performance of the mixes was evaluated from accelerated rutting test in HWT. It was found that dynamic modulus and creep compliance of the asphalt mixes increase and decrease, respectively, with an increase in amount of RAP and/or RAS used in the mix. Fatigue life was found to increase with increasing RAP content up to 25%, and to decrease when the RAP content exceeded 25%, or when RAS was used in the mix. It should be noted that this conclusion was based on conducting fatigue tests on asphalt mixes with 0%, 25%, and 40% RAP contents. However, the adverse effect of using RAP on fatigue life may start to occur at a RAP content between 25% and 40%. Therefore, for a more accurate determination of the RAP content which maximizes the fatigue life, testing more mixes with smaller increments in RAP content is recommended (e.g., 25%, 30%, 35%, 40%). The Hamburg wheel tracking (HWT) test results showed increased resistance to rutting and moisture-induced damage with an increase in the amount of RAP and/or RAS. However, the TSR test results were not confirmed by the HWT test. The findings of this study are expected to be helpful in understanding the effects of using different amounts of RAP and RAS on the performance of asphalt mixes produced using local materials. Furthermore, valuable design input parameters developed in this study for new mixes containing RAP and RAS may be used for local calibration of the M-EPDG.

3.2 Objectives

The overall objectives of this study were to evaluate and compare the effects of using RAP and/or RAS in asphalt mixes produced using local materials on their laboratory performance and to generate the M-EPDG input parameters. The specific objectives of this study were to:

1. Compare stiffness of asphalt mixes containing different amounts of RAP and RAS by determining their dynamic modulus values at different temperatures and frequencies.
2. Assess the low-temperature cracking potential of asphalt mixes containing different amounts of RAP and RAS with the help of creep compliance test.

3. Evaluate the fatigue life of these asphalt mixes containing different amounts of RAP and RAS, using the four- point bending beam fatigue test.
4. Evaluate the moisture-induced damage potential of these asphalt mixes containing different amounts of RAP and RAS using TSR and HWT methods and rank the mixes based on their performance according to each test method.
5. Evaluate the rutting performance of these asphalt mixes containing different amounts of RAP and RAS using a HWT test.

3.3 Materials

The Superpave asphalt mixes used in this study were designed in the laboratory in accordance with the AASHTO R 35 (AASHTO, 2012) and AASHTO M 323 specifications (AASHTO, 2013). The aggregates, RAP, RAS, and the PG 64-22 asphalt binder used for mix designs were collected from an asphalt plant located in Oklahoma City, OK. The aggregates were produced in quarries in Oklahoma. The collected RAP was milled from different state highway projects. The RAS used in this study was tear-off materials.

Asphalt mixes were designed for two pavement layers: base course (NMAS = 19.0 mm) and surface course (NMAS = 12.5 mm). The following four types of base course mixes were designed and used in this study: (i) mix with 0% RAP (B-0R); (ii) mix with 25% RAP (B-25R); (iii) mix with 40% RAP (B-40R); and (iv) mix with 20% RAP and 5% RAS (B-20R/5S). B-0R was produced in the laboratory and conditioned in accordance with AASHTO R 30 (AASHTO, 2010) to account for plant aging. B-25R, B-40R, and B-20R/5S mixes were directly collected from an asphalt plant and were used for compaction of the samples produced in the laboratory. Furthermore, the following three types of surface course mixes were used in this study: (i) mix with 0% RAP (S-0R); (ii) mix with 25% RAP (S-25R); and (iii) mix with 20% RAP and 5% RAS (S-20R/5S). The S-0R mix was produced in the laboratory and conditioned in accordance with the AASHTO R 30 method (AASHTO, 2010). The S-25R and S-20R/5S mixes were collected from an asphalt plant and were used for preparing the specimens for testing. Table 3.1 and Figure 3.1 present a summary of the mix properties and gradations used in this study, respectively.

3.4 Methodology

The tasks pursued in this study, including preparation of specimens and methodology used for conducting dynamic modulus, creep compliance, four-point bending beam fatigue, retained indirect tensile strength, and Hamburg wheel tracking tests are discussed in this section. Figure 3.2 presents a summary of the work flow.

3.4.1 Sample Preparation

The mixes collected from the asphalt plant (B-25R, B-40R, B-20R/5S, S-25R, and S20R/5S) and those produced and conditioned in the laboratory (B-0R and S-0R) were reheated in an oven and used for sample compaction. While reheating the mix in the oven, it was stirred occasionally and checked for its consistency and workability. Depending on the standard used for conducting each test, mixes were compacted to the required shape and dimensions. The target air voids of 7.0% ± 0.5% were used for preparation of all of the specimens tested in this study. Dynamic modulus, creep compliance, Hamburg wheel tracking, and indirect tensile strength test specimens were compacted using a Superpave gyratory compactor (SGC). In order to achieve the target air voids for each specimen with given dimensions, the SGC was operated in the height mode. In the height mode, the SGC automatically stops the compaction procedure as soon as the desired height is reached. The compacted specimens were tested for their volumetric properties in accordance with AASHTO T 166 (AASHTO, 2010) to check for their air voids. Slab samples needed for preparing the beam specimens for the four-point bending beam fatigue tests were compacted using a linear kneading compactor. Details of sample preparation for each test are discussed next.

3.4.1.1 Dynamic Modulus

In order to produce dynamic modulus specimens with consistent air void distribution the following procedure was followed (Chehab et al., 2000): a minimum of three replicate specimens having 150 mm in diameter by 167.5 mm in height were compacted using a SGC at 7.0% ± 0.5% air voids. Compacted specimens were vertically cored from the center using a coring machine to obtain 100 mm diameter specimens. Then the cored specimens were saw-cut to bring its height to 150 mm. A total of 21 dynamic modulus specimens (7 mixes x 3 replicates) were prepared using the aforementioned procedure.

3.4.1.2 Creep Compliance

A SGC was used in the height mode to compact the creep compliance specimens having 150 mm in diameter by 65 mm in height. A minimum of three replicate specimens were compacted for each mix. The specimens were then saw-cut from each end to obtain specimens with a diameter of 150 mm and a height of 45 to 50 mm. This procedure is known to produce specimens with consistent air voids distribution (Chehab et al., 2000). A total of 21 specimens (7 mixes x 3 replicates) were prepared for creep compliance test.

3.4.1.3 Four-Point Bending Beam Fatigue

A linear kneading compactor was used for preparing the slab samples having 406 mm in length, 152 mm in width and 76 mm in height. Each slab was then saw-cut to obtain two beam specimens having 380 mm in length, 63 mm in width, and 50 mm in height. A total of 21 slabs (7 mixes x 3 replicates) were compacted and saw-cut to obtain 42 beam specimens for conducting four-point bending beam fatigue tests.

3.4.1.4 Retained Indirect Tensile Strength Ratio

At least eight replicate specimens, having 150 mm in diameter and 95 mm in height, were compacted using a SGC. A total of 56 samples (7 mixes x 8 replicates) were compacted and used for testing.

3.4.1.5 Hamburg Wheel Tracking

A SGC was used for compacting the HWT specimens, having 150 mm in diameter and 60 mm in height. At least four replicate specimens were compacted for each mix. Then each specimen was saw-cut from the side to match the size of the plastic mold used for fixing the samples in the device. A total of 28 specimens (7 mixes x 4 replicates) were compacted and saw-cut to obtain 14 sets of samples for testing.

3.4.2 Laboratory Testing

3.4.2.1 Dynamic Modulus

The dynamic modulus tests were conducted on cylindrical specimens in accordance with AASHTO TP 62 (2010). A servo-hydraulic loading frame from MTS was used for conducting the dynamic modulus test. Dynamic modulus tests were conducted at -10, 4.4, 21.1, 37.8, and 54.4°C temperatures with six loading frequencies, namely 25, 10, 5, 1, 0.5 and 0.1 Hz at each temperature. Cyclic haversine-shaped load pulse magnitude applied to the specimen was adjusted based on the measured stiffness, loading frequency and temperature to keep the vertical strain within 50-150 microstrain (Tran and Hall, 2006). The load applied to the sample was measured using a 100 kN load cell. Vertical deformations of the specimen were measured by two Linear Variable Differential Transformers (LVDTs), attached on two diametrically opposite sides on the specimen at 100 mm gauge length. The measured loads and vertical strains were recorded using a data acquisition system in a computer. The load and strain values recorded for the last five cycles of each sequence were used to determine the dynamic modulus values. Dynamic modulus master curves were constructed based on the time-temperature superposition principle, at a reference temperature of 21.1°C. Equation 3.1, shows a sigmoidal function used for fitting and developing the master curve (Singh et al., 2011a; Singh et al., 2011b).

$$log|E^*| = \delta + \frac{\alpha}{1+\exp(\beta+\gamma(\log f_r))} \tag{3.1}$$

where,

$|E^*|$ = dynamic modulus in MPa,

f_r = reduced frequency at reference temperature,

δ = minimum value of |E*|,

$\delta + \alpha$ = maximum value of |E*|, and

β, γ = parameters describing the shape of the sigmoidal function.

The temperature shift factor function is given in Equation 3.2 and Equation 3.3.

$$a(T) = \frac{f_r}{f} \tag{3.2}$$

$$\log(f_r) = \log\big(a(T)\big) + \log(f) \tag{3.3}$$

where,

$a(T)$ = temperature shift factor,

T = temperature in °C, and

f = frequency at a particular temperature.

Using the Arrhenius time–temperature superposition model (Francken and Clauwaert, 1988), the temperature shift factor function may be written in the form of Equation 3.4.

$$\log\big(a(T)\big) = c\left(\frac{1}{T_K} - \frac{1}{T_{ref}}\right) \quad (3.4)$$

where,

T_K = the test temperature of interest in °Kelvin (°K = 273 + °C);

T_{ref} = the reference temperature in °K.

The Solver, a nonlinear optimization program in MS-Excel, was used for determining the master curve coefficients, namely $\alpha, \beta, \gamma, \delta\ and\ c.$ A quadratic polynomial function shown in Equation 3.5, was used to establish the shift factor-temperature relationship.

$$\log\big(a(T)\big) = mT^2 + nT + p \quad (3.5)$$

where,

m, n, p = polynomial fitting curve coefficients.

In order to evaluate the significance of the models used to fit to the master curve and shift factor functions, the goodness-of-fit statistics, according to the criteria suggested by Witczak (2005) were used (Table 3.2). The goodness-of-fit statistics applies the S_e/S_y (standard error of estimate/standard deviation) and correlation coefficient (R^2) to evaluate the strength of a model. According to the criteria shown in Table 3.2, the goodness-of-fit statistics rates the performance of a model based on its strength in fitting the experimental data in five categories, namely excellent, good, fair, poor, and very poor.

3.4.2.2 Creep Compliance

Creep compliance is used as an important input parameter in the M-EPDG for prediction of low-temperature cracking of asphalt mixes. Creep compliance tests were conducted on asphalt mixes, in accordance with the AASHTO T 322 standard test method (AASHTO, 2007). A servo-hydraulic loading frame and data acquisition system manufactured by MTS was used for testing. Four different temperatures, namely -18, -10, 0, and 10°C, were used for conducting the tests. Creep compliance test consisted of applying a static load of fixed magnitude along the vertical diameter of the cylindrical specimen, for a period of 100 seconds. The applied load was measured by a 100 kN load cell. Two sets of LVDTs, with a maximum stroke length of 5 mm, were used for measuring the vertical and horizontal deformations of the specimen in two diametrically perpendicular directions. LVDTs were attached on the flat face of the specimen, on two gauge points placed along the vertical and horizontal axes with a center to center spacing of 38.0 ± 0.2 mm. The applied static load was adjusted during the test to maintain the specimen deformation within the linear viscoelastic range (typically below 500 microstrain in the horizontal direction). The recorded load and the horizontal and vertical deformations at the center of the specimen were used for calculation of creep compliance as a function of time at 1, 2, 5, 10, 20, 50, and 100 seconds after test's initiation. Indirect tensile strength of the specimen at -10°C, as recommended by the M-EPDG, was determined by applying a load to the specimen at displacement control mode with a vertical ram movement rate of 12.7 mm per minute, until failure. Finally, using the time-temperature superposition principle, the creep compliance master curves were constructed for each mix. At a selected reference temperature (10°C), the shapes of adjacent creep compliance curves obtained from different temperatures were shifted with respect to time to obtain an exact matching and form a smooth function (Ferry, 1980). This function is expressed in the form of Equation 3.6.

$$D(t) = D_0 + D_1 t^m \tag{3.6}$$

where,

$D(t)$ = creep compliance in 1/MPa,

t = time in seconds, and

D_0, D_1, m = model constants.

A nonlinear optimization program (Solver of MS-Excel) was used to solve for the shift factors at different temperatures and master curve coefficients, namely, D_0, D_1, m. Then, a linear function fit, in the form of Equation 3.7, was used to develop the creep compliance shift factor-temperature relationship.

$$\log\big(a(T)\big) = aT + b \tag{3.7}$$

where,

a, b = model constants.

3.4.2.3 Four-Point Bending Beam Fatigue

The four-point bending beam fatigue test, in accordance with the AASHTO T 321 standard test method (AASHTO, 2011), was used to evaluate the fatigue performance of the selected asphalt mixes. Each beam specimen was set inside a temperature chamber at 20°C and subjected to a cyclic loading and unloading regime with a frequency of 10 Hz, using an ATM-100 loading frame manufactured by GCTS.

Loading and unloading was applied to maintain a strain of 400 microsrain in displacement-control operation mode. Selection of 400 microstrain was made based on the past experience of testing the asphalt mixes with and without RAP and RAS, in order to obtain the fatigue lives of the maximum number of the specimens in the approximate range of 50,000 and 500,000 loading cycles, as recommended by Harvey et al. (1995). The strain level for the asphalt mixes used in this study helped to keep the testing time to a reasonable range. The loads applied to the beam specimens were measured using a 5 kN load cell. The vertical deflection measured at the center of the beam was used to control the strain level during the fatigue test. The deflection was measured using an LVDT mounted on the beam fixture, in contact with a metallic stud glued at the center of the beam. The initial beam stiffness was determined at 50^{th} load cycle after the test was initiated. The fatigue life reported in this study is the total number of load cycles to cause a 50 percent decrease in initial beam stiffness (Tayebali et al., 1993; Tayebali et al., 1992; Pronk and Hopman, 1990). A minimum of three replicate beam specimens from each mix were tested for the fatigue life.

3.4.2.4 Retained Indirect Tensile Strength Ratio (TSR)

Retained indirect tensile strength ratio (TSR) tests were conducted, in accordance with the AASHTO T 283 standard test method (AASHTO, 2011), to evaluate the moisture-induced damage potential of the asphalt mixes. In this method, the change in diametric tensile strength of cylindrical specimens due to moisture and temperature conditioning, with a freeze-thaw cycle, is used to evaluate the moisture-induced damage potential of the asphalt mixes. For this purpose, a set of eight SGC-compacted cylindrical specimens were divided into two subsets. One subset was tested for indirect tensile strength (ITS) at a temperature of 25°C, under dry condition. The other subset was subjected to vacuum saturation by water between 70 to 80 percent saturation, under a 13 to 67 kPa absolute vacuum pressure. Each vacuum-saturated specimen was then tightly wrapped with a plastic film and placed in a plastic leak-proof bag containing 10-mL of water. The saturated specimens were subjected to a freeze cycle of -18°C for a minimum time period of 16 hours, followed by a 60°C warm water soaking cycle for 24 hours. The conditioned specimens were placed in a water bath of 25°C temperature for another two hours before testing them for indirect tensile strength. Numerical indices of retained indirect tensile strength ratio (TSR) were determined by dividing the average tensile strength value obtained from testing dry to that of conditioned subsets, respectively. The TSR values along with HWT test results are widely being used for prediction of long-term moisture-induced damage potential of the asphalt mixes.

3.4.2.5 Hamburg Wheel Tracking (HWT)

Hamburg wheel tracking tests, in accordance with the AASHTO T 324 standard test method (AASHTO, 2011), were conducted to evaluate the rutting and moisture-induced damage potential of the asphalt mixes. For this purpose the temperature-controlled water bath of the HWT was was set to 50°C. Then the specimens and the plastic molds were placed and fixed in the metal tray of the HWT. Then specimen setup was submerged in the water bath and fixed to the device. After reaching temperature equilibrium, the test was initiated and the specimen was repetitively loaded using a reciprocating steel wheel of a 705-N weight. Deformations measured on the surface of the specimens were recorded at each wheel pass and were plotted after 20,000 passes.

This plot was used for determining the creep slope, stripping inflection point (SIP) and stripping slope. SIP is an indicator of stripping of the asphalt binder from the aggregates which leads to moisture-induced damage in asphalt. The SIP can be detected by a sudden increase in deformation rate with respect to number of wheel passes.

3.5 Results and Discussion

3.5.1 Dynamic Modulus

The dynamic modulus master curve model parameters (Equation 3.1) developed for different asphalt mixes are presented in Table 3.3. A reference temperature of 21.1°C was used for constructing the master curves. From Table 3.3, and based on the goodness-of-fit statistics, it is evident that the dynamic modulus models used for developing the master curves are all rated as excellent. In other words, the sigmoidal fit functions are able to satisfactorily predict the dynamic modulus values at a reference temperature of 21.1°C. In order to determine the dynamic modulus values at different temperatures, the temperature shift factor quadratic polynomial function as shown in Equation 3.5 was used. The model parameters for the shift factor quadratic polynomials developed for the tested mixes are shown in Table 3.4. Based on the goodness-of-fit statistics, all of the shift factor models were rated as excellent.

The master curves of the base mixes (B-0R, B-25R, B-40R, and B-20R/5S) and surface course mixes (S-0R, S-25R, and S-20R/5S) are presented in Figure 3.3-a and 3.3-b, respectively. From Figures 3.3-a and 3.3-b it was observed that dynamic modulus of all mixes tested herein increase with an increase in the loading frequency and a reduction in temperature. A similar trend of dynamic modulus with temperature and loading frequency is reported in the literature (e.g., Tashman and Elangovan, 2008; Flintsch et al., 2007; Singh et al., 2011a).

From Figure 3.3-a it is evident that, in general, the dynamic modulus values of the B-40R and B-20R/5S mixes are considerably higher than those measured for the B-25R and B-0R mixes. However, the dynamic modulus values of the B-25R mix are slightly higher than those of the B-0R mix. This observation reveals that for the tested base course mixes an addition of 25% RAP slightly increases the dynamic modulus values, compared with those without RAP. However, the addition of 40% RAP (B-40R), or using 20% RAP + 5% RAS (B-20R/5S), considerably increased the dynamic moduli of the asphalt mixes when compared with those with 0% RAP or 25% RAP. Furthermore, the dynamic moduli of the B-20R/5S mix at frequencies less than 1 Hz were higher than those of the B-40R mix. This was attributed to the fact that the B-20R/5S mix contains 20% RAP and 5% RAS by the weight of aggregates, replacing 25.0% and 21.6% of the PG 64-22 asphalt binder, respectively, a total of 46.6% binder replacement (Table 3.1). Furthermore, the binder from RAS is highly aged in the refinery (air-blown) and during its service life as roofing shingles, and therefore has a higher stiffness compared to the virgin asphalt binder and that from RAP. Therefore, it is expected to observe higher moduli for B-20R/5S, specifically at lower frequencies. According to the time-temperature superposition principle, a lower reduced frequency is equivalent to a higher temperature. Therefore, the effect of the highly aged binder of the B-20R/5S mix was more pronounced at lower frequencies, leading to higher moduli when compared to that of B-40R. However, at frequencies greater than 1 Hz the B-20R/5S and B-40R mixes exhibit similar dynamic moduli. This is because of the effect of aggregate structure at higher frequency: at a higher frequency the role of aggregate structure becomes dominant, specifically for coarse mixes.

From Figure 3.3-b, it was observed that the S-20R/5S and S-0R mixes demonstrated the highest and the lowest dynamic modulus values, respectively. Furthermore, the S-25R mix exhibited dynamic moduli less than those of S-20R/5S mix and more than those of the S-0R mix. The S-20R/5S mix has a 39.7% total binder replacement by RAP and RAS, which is 13.1% higher than replaced binder in S-25R by RAP (Table 3.1). This means that the asphalt binder blend of the S-20R/5S mix consisted of a higher portion of more aged and less virgin binder when compared to that of the S-25R mix. More aged binder leads to a stiffer mix and therefore a higher dynamic modulus. Comparatively, the S-0R mix does not contain any aged binder from RAP or RAS, and therefore exhibited the lowest dynamic moduli. It should be noted that the dynamic moduli of the surface course mixes, due to a finer gradation, are more sensitive to binder type, and therefore addition of small quantities of RAP and/or RAS results in a significant change in moduli, as seen in Figure 3.3-b.

Increasing dynamic modulus with an increase in the amounts of reclaimed asphalt materials (RAP and RAS), are in agreement with the results reported in the literature (e.g., Yang et al., 2014; Li et al., 2008; McGraw et al., 2007; Uzarowski, 2006). A low dynamic modulus value in asphalt mixes is known to result in a higher rutting potential compared to stiffer mixes. However, very stiff mix may result in a lower fatigue life compared to those with lower stiffness. Therefore, it is important to evaluate the fatigue and rutting potential of the asphalt mixes through performance tests. Furthermore, the dynamic modulus values determined for the base and surface course asphalt mixes containing different amounts of RAP and/or RAS, can be considered as an important contribution to develop a database for local calibration of the M-EPDG.

3.5.2 Creep Compliance

The M-EPDG uses the creep compliance as an input parameter to predict the thermal cracking of pavements over their service life. The methodology discussed earlier was used to determine the creep compliance master curve model parameters (Equation 3.6). The creep compliance master curve model parameters, goodness-of-fit statistics, and rating of each model are presented in Table 3.5. From Table 3.5, it was observed that, based on the goodness-of-fit statistics, the models used for development of master curves were all rated as “excellent” except those of the B-25R and B-20R/5S mixes, which were rated as “good”. The creep compliance master curves at a reference temperature of 10°C for base and surface course mixes were plotted and presented in Figures 3.4-a and 3.4-b, respectively.

From Figure 3.4-aand 3.4-b, it was observed that the creep compliance increased with an increase in loading time and temperature. This is consistent with the findings reported in the literature (Vargas, 2007).

From Figure 3.4-a it is clear that the B-0R mix shows the highest creep compliance values when compared with the other base course mixes (B-25R, B-40R, and B-25R/5S). The B-0R mix is a base course mix with a PG 64-22 asphalt binder which does not contain RAP in the mix (only virgin binder). However, the other base course mixes (B-25R, B-40R, and B-25R/5S) contain RAP and/or RAS which contribute aged asphalt binder to the binder blend of the mix. Use of aged binder results in a stiffer mix with lower creep compliance. Comparatively, the B-25R mix showed higher creep compliance values than those of B-40R. This is expected, since the B-40R mix contains 29.8% RAP binder in the binder blend which is 13.3% higher than that of B-25R (Table 3.1). More RAP binder results in a stiffer mix (Swiertz et al., 2011) which in turn, leads to lower creep compliance, as expected. On the other hand, the B-20R/5S mix showed the lowest creep compliance compared with all other types of the base course mixes. This is expected due to the fact that according to Table 3.1, the B20R/5S mix has the highest rate of total virgin binder replacement (46.6%) by reclaimed asphalt binder from RAP (25.0%) and RAS (21.6%). Furthermore, the RAS binder (21.6%) in the binder blend of B20R/5S mix, is more aged than that of RAP, which results in a higher stiffness and therefore lower creep compliance.

From Figure 3.4-b, it was observed that the S-0R and S-20R/5S mixes demonstrated the highest and the lowest creep compliance values, respectively, among the surface course mixes. Furthermore, the S-25R mix exhibited creep compliance values less than those of S-0R mix and more than S-20R/5S mix. The S-20R/5S mix has a 39.7% total virgin binder replacement by RAP (21.3) and RAS (18.4), which is 13.1% higher than replaced binder of S-25R by RAP binder (26.6%) (Table 3.1). This means that the asphalt binder blend of S-20R/5S mix consisted of a higher portion of more aged and less virgin binder when compared to that of the S-25R mix. More aged binder leads to a stiffer mix and therefore lower creep compliance. Comparatively, S-0R does not contain any aged binder from RAP or RAS, and therefore exhibited the highest creep compliance. It should be noted that the creep compliance of the surface course mixes, due to a finer gradation, are more sensitive to binder type and therefore addition of small quantities of RAP and/or RAS, resulting in a significant change in stiffness and therefore creep compliance, as seen in Figure 3.4-b.

Decreasing creep compliance values with an increase in the amounts of reclaimed asphalt materials (RAP and/or RAS) is consistent with the observations reported in the literature (e.g. You et al., 2011a, Vargas, 2007). A low creep compliance value of an asphalt mix is known to result in a low relaxation modulus, which may lead to more thermal stress buildup in asphalt pavement as a result of temperature change, and therefore, may lead to a greater low-temperature cracking potential (Lytton et al., 1993). Furthermore, the creep compliance values determined for the base and surface course mixes, containing different amounts of RAP and/or RAS, can be considered as an important contribution to develop a database for local calibration of the M-EPDG.

3.5.3 Fatigue Life

The pavement should be able to withstand repeated traffic loads without a major distress due to fatigue. Therefore, measuring the fatigue life of the asphalt mixes in the laboratory is of vital importance to pavement engineers. For this purpose, after conducting the four-point bending beam fatigue tests on asphalt mixes, the initial flexural stiffness and the number of the cycles to failure of the base and surface course asphalt mixes, containing different amounts of RAP and/or RAS, were compared. Figure 3.5 presents a summary of the initial flexural stiffness and the number of loading cycles to failure of the asphalt mixes tested in this study.

From Figure 3.5, it is was concluded that the flexural initial stiffness of the base course mixes increased with an increase in the amount of RAP and/or RAS content used in each mix. For example, the initial flexural stiffness values measured for the B-25R, B-40-R and B-20R/S mixes were, respectively, 51%, 55%, and 54% higher than that of the B-0R mix. This shows that addition of RAP and/or RAS in base course mix increased the initial flexural stiffness compared to virgin mix. Increasing the mix stiffness with an increase in RAP and/or RAS amounts is consistent with the results obtained from dynamic modulus tests. However, no significance change in initial stiffness with changing the amount of RAP and/or RAS used in the mixes was observed when the flexural stiffness values of the B-25R, B-40R and B-20R/5S mixes were compared. Furthermore, Figure 3.5 reveals that the fatigue life of the base course mixes increased with an increase in the RAP content up to 25% and then started to decrease with further increasing in amount of RAP to 40%, or when RAS was used in the mix. For example, the cycles to failure of the B-0R mix without RAP (119,004) shows a 14% increase when the B-25R mix, containing 25% RAP, was used (135,399 cycles). However, the fatigue life of B-40R, containing 40% RAP, shows a reduction of 11%, compared to that of B-25R mix. Similarly, when the B-20R/5S mix was used the fatigue life showed another 37% decrease when compared with that of B-40R. Therefore, it was concluded that increasing the RAP content up to up to 25% may increase the fatigue life of the base course asphalt mixes. A similar observation is also reported by Huang et al. (2004). However, further increasing RAP content (greater than 25%), or use of RAS, showed an adverse effect on the fatigue life of the asphalt mixes. It should be noted that this conclusion was based on conducting fatigue tests on asphalt mixes with 0%, 25%, and 40% RAP contents. However, the adverse effect of using RAP on fatigue life may start to occur at a RAP content between 25% and 40%. Therefore, for a more accurate determination of the RAP content which maximizes the fatigue life, testing more mixes with smaller increments in RAP content is recommended (e.g., 25%, 30%, 35%, 40%). This finding is considered to be consistent with those reported by McDaniel et al. (2000). McDaniel et al. (2000) reported an adverse effect of increased RAP on the fatigue life of pavements when the RAP content is greater than 20%.

Figure 3.5 reveals that the flexural initial stiffness of the surface course mixes increased with an increase in the amount of RAP and/or RAS content. For example, the initial flexural stiffness of the S-25R and S-20R/S mixes were found to be 101% and 144% higher than that of S-0R. This suggests that addition of RAP and/or RAS in the surface course mix increased the flexural stiffness of the mix compared to virgin mix, which is consistent with findings from the dynamic modulus tests. However, when compared with the base course mixes, it was observed that the addition of RAP and RAS had a more pronounced effect on the initial flexural stiffness of the surface course mixes. This was attributed to the finer gradation of the surface course mixes (NMAS = 12.5 mm) in which the effect of asphalt binder on the stiffness is more dominant. However, mix stiffness was more affected by aggregate structure in base course mixes in with a coarser gradation (NMAS = 19.0 mm). Also, from Figure 3.5 it is clear that the fatigue life of the surface course mixes increased with an increase in the RAP content from 0% (S-0R) to 25% (the S-25R mix with a 26.6% binder replacement) and then decreased when 20% RAP + 5% RAS (the S-20R/5S mix with a 39.7% total binder replacement) was used. For example, the cycles to failure of the S-0R mix without RAP (301,447) showed an 18% increase when the S-25R mix, containing 25% RAP, was used (356,667 cycles). However, the fatigue life of the S-20R/5S mix (226,634 cycles) containing 20% RAP + 5%RAS showed a reduction of 43% compared to that of the S-25R mix. Furthermore, in a comparison between the fatigue lives of the base and surface course mixes it was concluded that the surface course mixes have higher fatigue lives compared to the base course mixes.

Overall, it can be concluded that the fatigue lives of the tested asphalt mixes increased when the RAP content increased up to 25%, and started to decrease when a higher RAP amount was used. Also it was observed that use of RAS and RAP reduced the fatigue life. It should be noted that this conclusion was based on conducting fatigue tests on asphalt mixes with 0%, 25%, and 40% RAP contents. However, the adverse effect of using RAP on fatigue life may start to occur at a RAP content between 25% and 40%. Therefore, for a more accurate determination of the RAP content which maximizes the fatigue life, testing more mixes with smaller increments in RAP content is recommended (e.g., 25%, 30%, 35%, 40%). This was attributed to the type of the binder used in the RAP: The RAP used in this study were obtained from milling the interstate highway projects in Oklahoma, in which polymer- modified binder might have been used to improve their performance. Therefore, the RAP binder contributing in the binder blend of the mixes tested herein can improve the fatigue life. However, addition of excessive amounts of RAP (>25%) or use of RAS can start to increase the brittleness of the mix to an undesirable level, which in turn may reduce the fatigue life. Therefore, using virgin binder of a lower PG grade is recommended in order to address the concerns associated with fatigue life when the RAP amount is more than 25%. Hence, high amounts of RAP and/or RAS in asphalt mixes should be used carefully and with proper laboratory evaluation in order to prevent compromising the fatigue life of the mix.

3.5.4 Retained Indirect Tensile Strength Ratio (TSR)

Figures 3.6-a and 3.6-b present a summary of TSR and the indirect tensile strength (ITS) values of the conditioned and dry specimens, respectively. According to Figure 3.6-a, only two mixes, B-20R/5S and S-20R/5S, with TSR values of 0.63 and 0.68, respectively, do not meet the specification's minimum TSR requirement of 0.8. Based on the TSR results, it was concluded that addition of RAS may result in an increase in moisture-induced damage. However, from Figure 3.6-a it was observed that when only RAP is used the TSR vales generally increased with an increase in amount of RAP. From Figure 3.6-b it is evident that the conditioned and dry ITS values for the B-20R/5S and S-20R/5S mixes are significantly higher than those of the base and surface course mixes, respectively. For example, Dry ITS of the B-20R/5S mix (1,219 kPa) was found to be significantly higher than those of the B-0R (714 kPa), B-25R (740 kPa) and B-40R (630 kPa) mixes. Also, conditioned ITS value of the B-20R/5S mix was higher than those of the B-0R (641 kPa), B-25R (677 kPa) and B-40R (652 kPa) mixes. A similar trend of variation of conditioned and dry ITS was observed for the S-20R/5S mix when compared with the S-0R and S-25R mixes. This means that ITS values (conditioned and dry) increase with incorporation of RAS in asphalt mix. However, TSR values suggest a negative effect on the resistance to moisture-induced damage due to incorporation of RAS in asphalt mix. Therefore, additional test results from HWT are needed to draw a clearer conclusion on the effect of incorporating RAP and/or RAS on the moisture-induced damage potential of the asphalt mixes. Recently, surface free energy method, as a mechanistic approach, was shown to successfully evaluate the mixes for moisture-induced damage potential (Ghabchi et al., 2013a; Arabani et al., 2012; Wasiuddin et al., 2008; Wasiuddin et al., 2007; Bhasin and Little, 2007; Lytton et al., 2005; Cheng et al., 2002).

3.5.5 Hamburg Wheel Tracking (HWT)

A graphical comparison of rut depth with wheel passes of the tested base (B-0R, B-25R, B-40R, and B-20R/5S) and surface course mixes (S-0R, S-25R, and S-20R/0S) are presented in Figures 3.7-a, 3.7-b, respectively. According to Figure 3.7-a, it is evident that the B-0R and B-25R mixes showed stripping inflection point (SIP) of 10,032 and 12,320 passes, respectively. SIP is an indicator of initiation of stripping leading to moisture-induced damage. However, since SIP>10,000 passes both mixes pass the minimum SIP requirement. But since the B-0R demonstrated a lower SIP (10,032 passes) compared with that of the B-25R mix (12,320 passes), it can be concluded that B-0R has a lower resistance to moisture-induced damage than that of the B-25R mix. Comparatively, the B-40R and B-20R/5S mixes did not exhibit a SIP, indicating their high resistance to moisture-induced damage. This finding does not confirm those from TSR tests. According to the TSR test results (Figure 3.6-a) the B-20R/5S mix showed the lowest TSR value (0.63) among other base course mixes, indicating a high moisture-induced damage potential. Therefore, it may be recommended to use dry and conditioned IDT and HWT as additional tools to evaluate the moisture-induced damage potential of asphalt mixes containing RAP and/or RAS. Furthermore, maximum rut depths of 18.1 mm, 19.7 mm, 10.5 mm, and 5.1 mm were observed for the B-0R, B-25R, B-40R, and B-20R/5S mixes, respectively. It can be concluded that in general, the rut depth decreases with an increase in amount of RAP and/or RAS, used in an asphalt mix.

From Figure 3.7-b it is evident that the S-25R mix showed a SIP of 10,978 passes. However, other surface course mixes (S-0R and S-20R/5S) did not exhibit SIP, indicating their high resistance to moisture-induced damage. It means that the S-25R mix has a higher moisture-induced damage potential when compared with the S-0R and S-20R/5S mixes. Similar to base course mixes, the S-20R/5S mix with a TSR value of 0.68 fails to meet the minimum TSR requirement. However, it performs well against moisture-induced damage when tested in HWT (no detectible SIP). On the other hand, the S-0R and S-20R/5S mixes showed rut depths less than 3 mm indicating their high resistance to rutting.

Overall, based on the HWT test results, it can be concluded that incorporation of RAP and/or RAS in asphalt mixes may result in a higher resistance to rutting and moisture-induced damage.

3.6 Conclusions and Recommendations

Four types of base course mixes and three types of surface course mixes containing different amounts of RAP and/or RAS were characterized using laboratory performance tests, namely dynamic modulus, creep compliance, four-point bending beam fatigue, retained indirect tensile strength, and Hamburg wheel tracking. Base course (NMAS = 19.0 mm) consisted of asphalt mixes containing 0% RAP (B-0R), 25% RAP (B-25R), 40% RAP (B-40R), and 20% RAP + 5% RAS (B-20R/5S). Surface course (NMAS = 12.5 mm) consisted of asphalt mixes containing 0% RAP (S-0R), 25% RAP (S-25R), and 20% RAP + 5% RAS (S-20R/5S). Based on the results and discussions presented in this chapter, the following conclusions can be drawn:

1. The dynamic modulus values of the asphalt mixes increased with an increase in amount of replaced virgin asphalt binder by the asphalt binder from RAP and/or RAS. This may result in a better rutting performance of the mixes with a higher percentage of RAP and/or RAS. Aged asphalt binder as found in RAP and RAS was known to be responsible for increasing the stiffness of the mix, resulting in higher dynamic modulus values.
2. The creep compliance values of the asphalt mixes decreased with an increase in amount of replaced virgin asphalt binder by the asphalt binder from RAP and/or RAS. This may result in a higher susceptibility to thermal cracking as a result of decreasing relaxation modulus with a decrease in creep compliance. This was attributed to increasing the mix stiffness with using more aged and therefore stiffer asphalt binder from RAP and RAS.
3. The effect of the amount of RAP and/or RAS on dynamic modulus and creep compliance values was more pronounced for the surface course mixes (NMAS = 12.5 mm) than that of base course mixes (NMAS = 19.0 mm).
4. Fatigue life was found to increase with increasing RAP content up to 25%, and to decrease when the RAP content exceeded 25%, or when RAS was used in the mix. It should be noted that this conclusion was based on conducting fatigue tests on asphalt mixes with 0%, 25%, and 40% RAP contents. However, the adverse effect of using RAP on fatigue life may start to occur at a RAP content between 25% and 40%. Therefore, for a more accurate determination of the RAP content which maximizes the fatigue life, testing more mixes with smaller increments in RAP content is recommended (e.g., 25%, 30%, 35%, 40%).

5. The TSR values of the asphalt mixes tested herein were found to be greater than 0.9, except those containing RAS. This observation was not confirmed by the Hamburg wheel tracking test results, in which mixes containing RAS showed a good performance against rutting and moisture-induced damage.
6. Based on the Hamburg wheel tracking test results, it was found that the resistance of the asphalt mixes to rutting and moisture-induced damage increase with an increase in the amount of RAP and/or RAS used.

In this study a high quality RAP from one source was used, and therefore the effect of variation in RAP source on the mix properties was not studied. Therefore, It is recommended that the effect of RAP and RAS source on the performance of asphalt mixes, specifically that of fatigue and moisture-induced damage, be studied in detail. In the present study, a PG 64-22 binder was used in preparing all mixes involving RAP and/or RAS. A separate study may be undertaken using a softer binder (e.g., PG 58-228) to compensate for the stiffer binders from RAP and/or RAS used to replace the virgin binder.

Table 3.1 A Summary of the Asphalt Mixes Used in this Study

Mix Type	NMAS† (mm)	Pavement Layer	PG Grade	RAP/RAS (%)	Binder Replacement (%)		Total Binder (%)
					RAP	RAS	
B-0R	19.0	Base	64-22	0 / 0	0	0	4.4
B-25R	19.0	Base	64-22	25 / 0	29.8	0	4.1
B-40R	19.0	Base	64-22	40 / 0	43.1	0	5.1
B-20R/5S	19.0	Base	64-22	20 / 5	25.0	21.6	4.0
S-0R	12.5	Surface	64-22	0 / 0	0	0	4.7
S-25R	12.5	Surface	64-22	25 / 0	26.6	0	4.6
S-20R/5S	12.5	Surface	64-22	20 / 5	21.3	18.4	4.7

†Nominal maximum aggregate size.

Table 3.2 Goodness-of-Fit Model Evaluation Criteria (Witczak, 2005)

Rating	R^2	S_e/S_y
Excellent	≥ 0.90	≤ 0.35
Good	0.70 - 0.89	0.36 - 0.55
Fair	0.40 - 0.69	0.56 - 0.75
Poor	0.20 - 0.39	0.76 - 0.90
Very poor	≤ 0.19	≥ 0.90

Table 3.3 Dynamic Modulus Master Curve Model Parameters of Virgin Mixes and Mixes Containing Different Amounts of RAP and/or RAS

Mix Type	|E*| Master Cuve Parameters (MPa)					Goodness-of-fit Statistics		
	α	β	γ	δ	c	R^2	S_e/S_y	Rating
B-0R	2.213	-0.770	-0.554	2.039	11620.4	0.989	0.110	Excellent
B-25R	2.465	-0.957	-0.522	1.860	10100.7	1.000	0.035	Excellent
B-40R	4.558	-1.089	-0.289	0.414	10219.4	0.990	0.074	Excellent
B-20R/5S	3.690	-0.803	-0.256	1.322	10599.4	0.991	0.054	Excellent
S-0R	2.312	-0.699	-0.350	1.495	10302.0	0.993	0.083	Excellent
S-25R	3.738	-0.764	-0.249	1.061	10007.4	0.992	0.081	Excellent
S-20R/5S	3.215	-0.892	-0.400	2.101	10131.4	0.970	0.072	Excellent

Table 3.4 Dynamic Modulus Master Curve Shift Factor Model Parameters of Virgin Mixes and Mixes Containing Different Amounts of RAP and/or RAS

Mix Type	Shift Factor Parameters			Goodness-of-fit	
	m	n	p	R^2	Rating
B-0R	0.0004	-0.153	3.038	1.00	Excellent
B-25R	0.0004	-0.133	2.640	1.00	Excellent
B-40R	0.0004	-0.134	2.671	1.00	Excellent
B-20R/5S	0.0004	-0.139	2.771	1.00	Excellent
S-0R	0.0004	-0.135	2.693	1.00	Excellent
S-25R	0.0004	-0.131	2.616	1.00	Excellent
S-20R/5S	0.0004	-0.133	2.648	1.00	Excellent

Table 3.5Creep Compliance Master Curve Model Parameters of Virgin Mixes and Mixes Containing Different Amounts of RAP and/or RAS

Mix Type	Creep Complinace Master Cuve Parameters (1/MPa)			Goodness-of-fit Statistics		
	D_o	D_1	m	R^2	S_e/S_y	Rating
B-0R	5.11E-05	6.34E-06	6.54E-01	0.99	0.101	Excellent
B-25R	4.60E-05	2.00E-06	7.60E-01	0.99	0.392	Good
B-40R	3.42E-05	6.85E-06	4.70E-01	0.99	0.093	Excellent
B-20R/5S	5.11E-05	4.62E-06	5.40E-01	0.99	0.351	Good
S-0R	5.74E-05	2.59E-05	3.78E-01	0.99	0.110	Excellent
S-25R	2.83E-05	2.12E-05	3.51E-01	0.99	0.110	Excellent
S-20R/5S	2.50E-05	1.40E-06	4.51E-01	0.99	0.282	Excellent

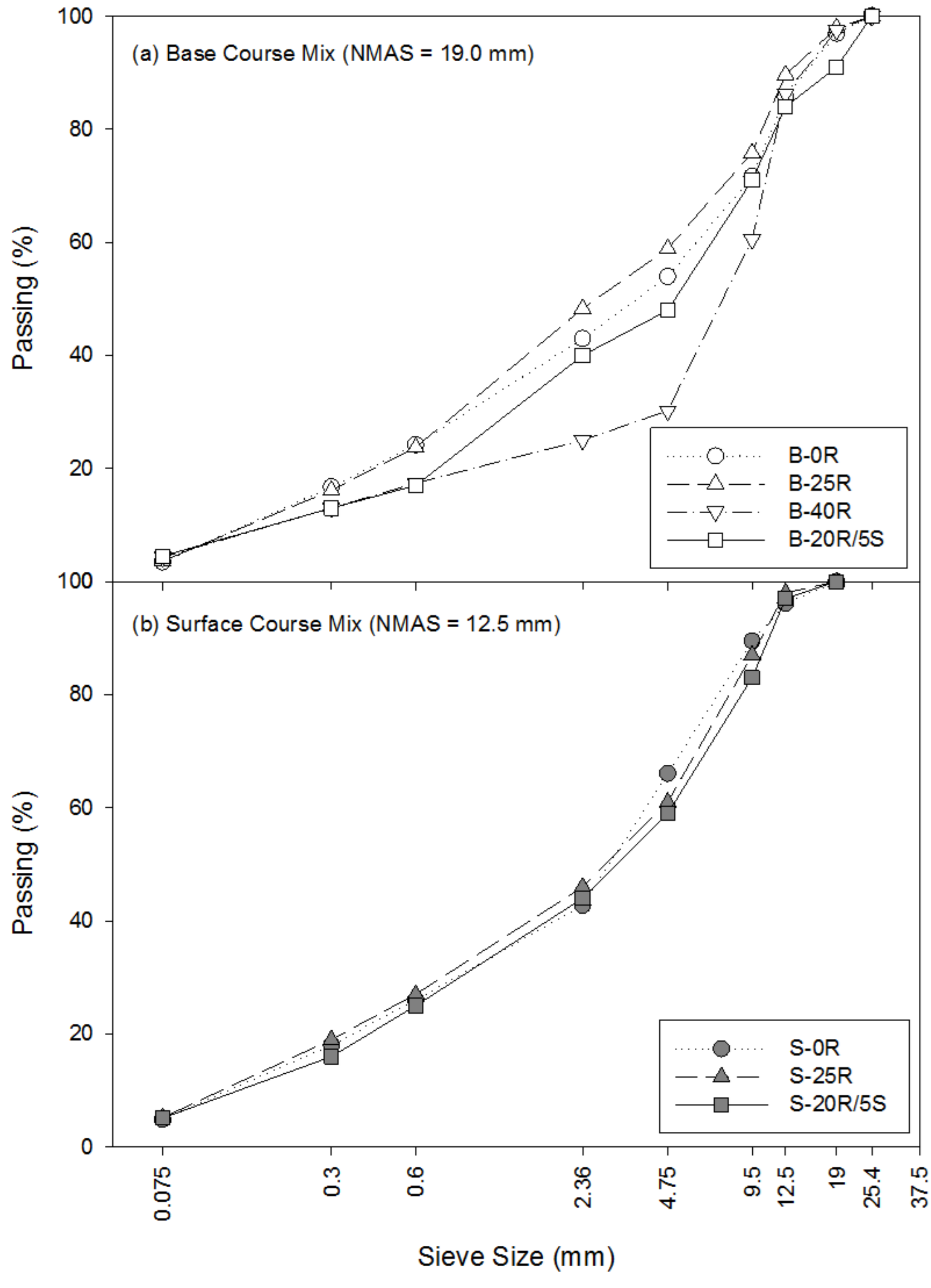


Figure 3.1Asphalt Mix Gradations for (a) Base Course and (b) Surface Course Mixes Used in this Study

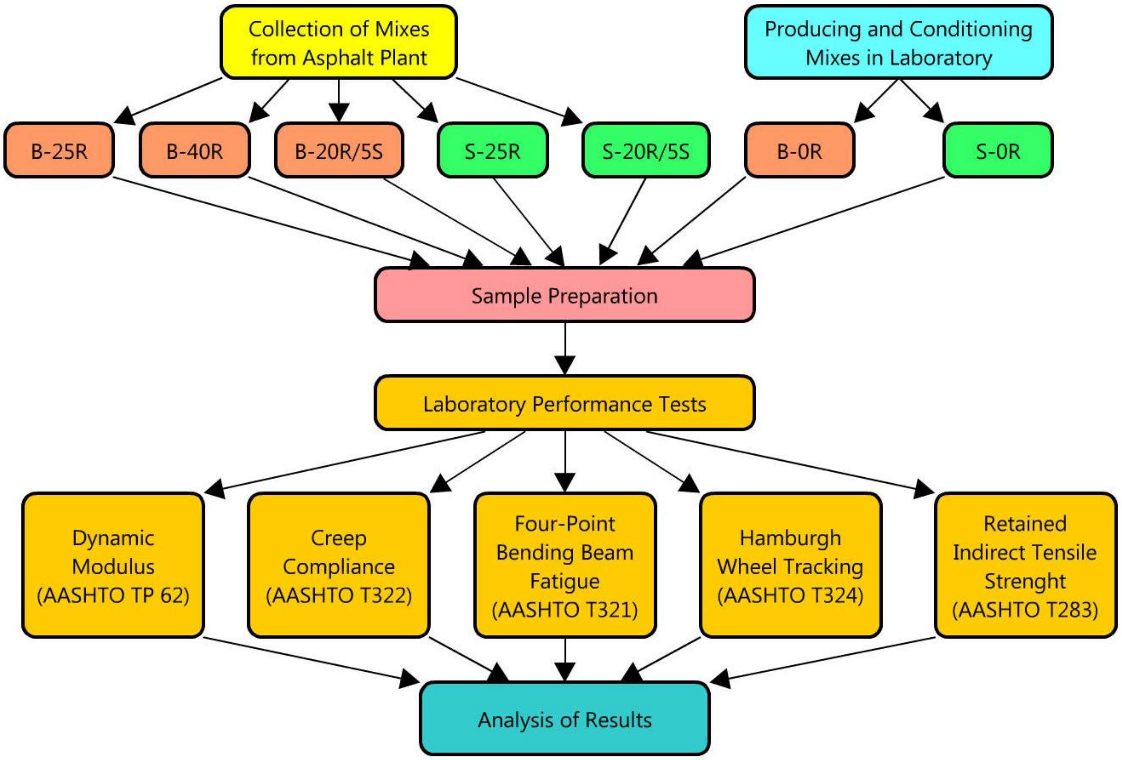


Figure 3.2 Work Flow and Testing Plan

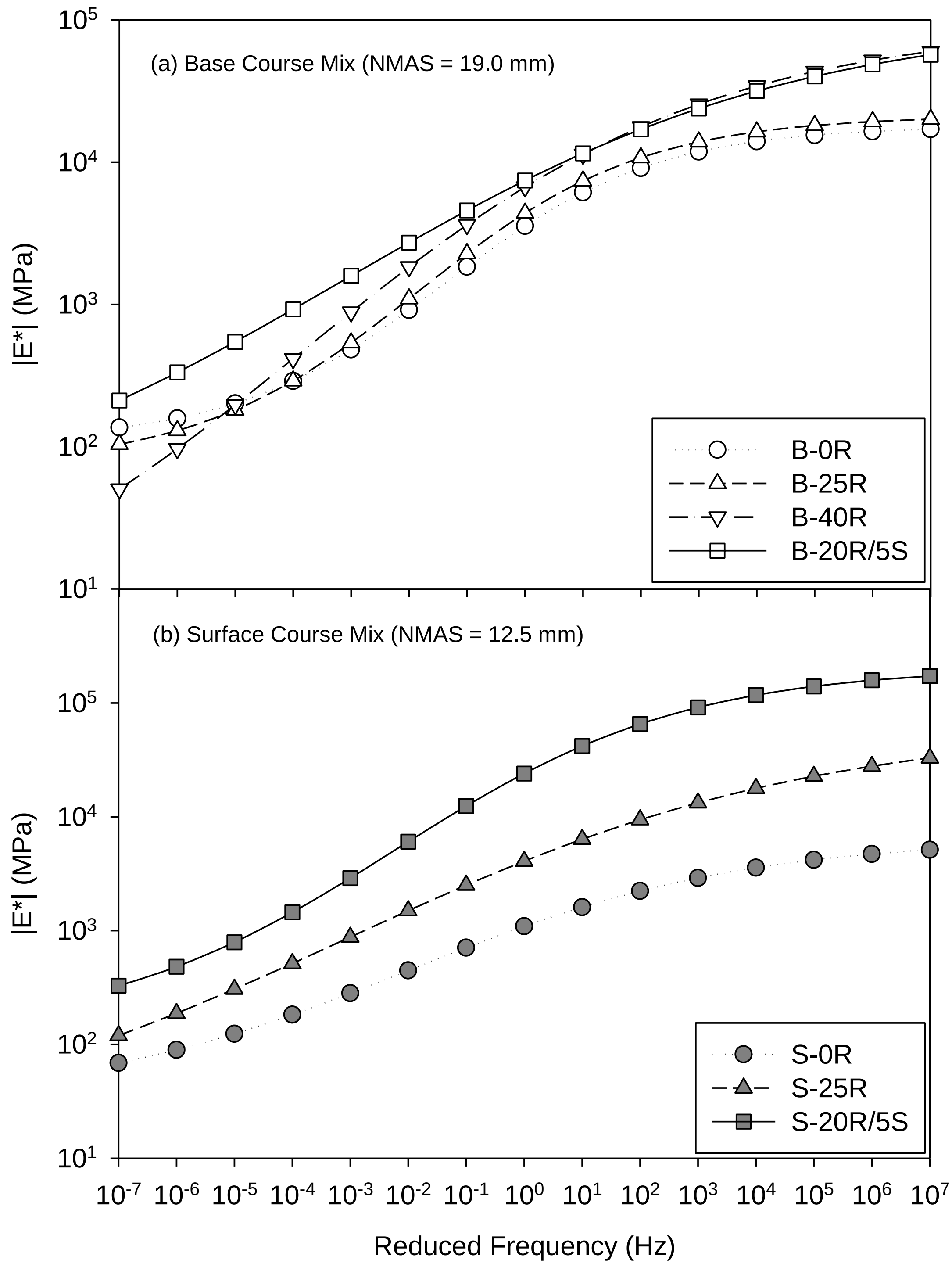


Figure 3.3Dynamic Modulus Master Curves at 21.1°C Reference Temperature for (a) Base course mixes, and (b) Surface Course Mixes

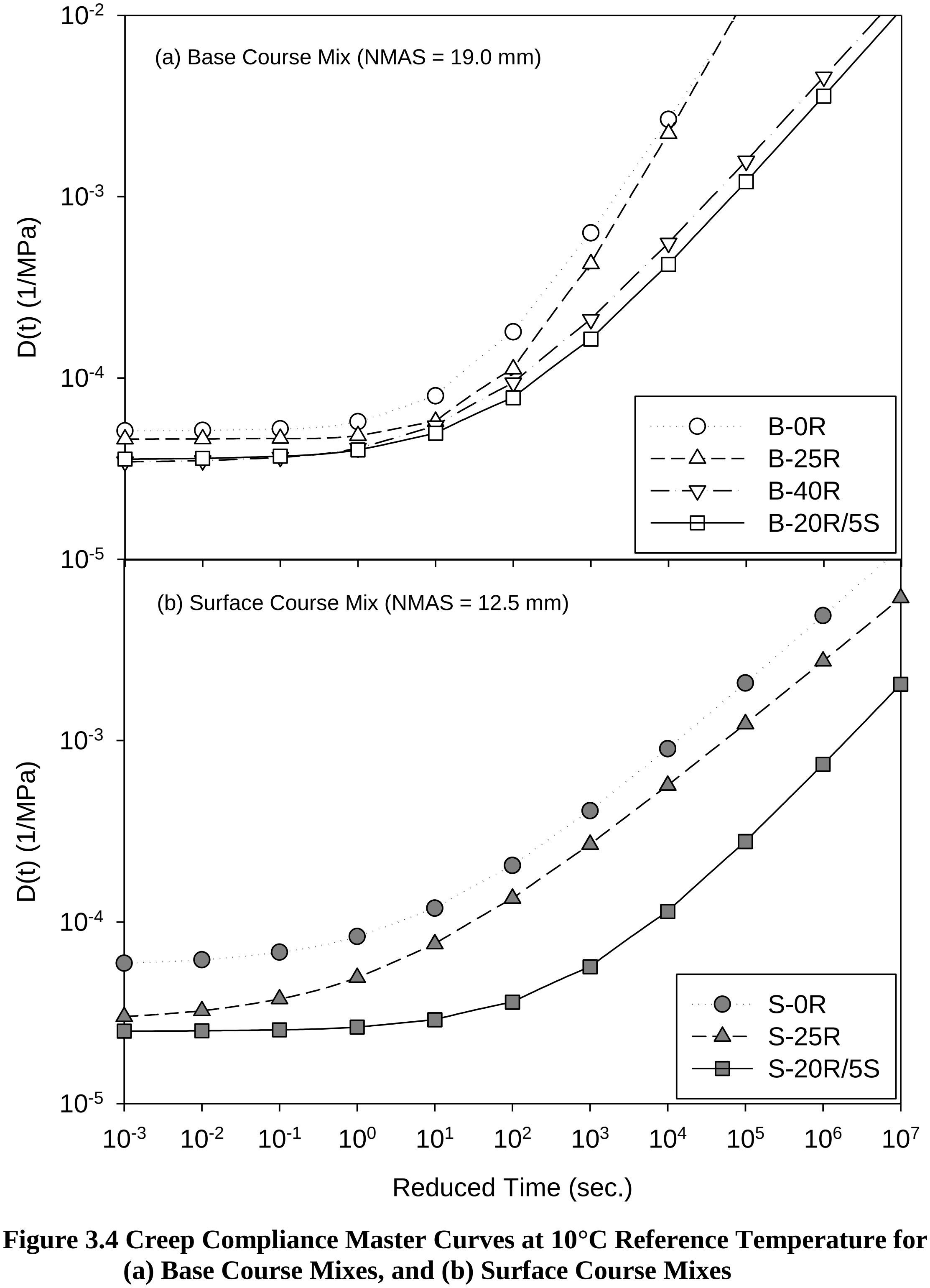


Figure 3.4 Creep Compliance Master Curves at 10°C Reference Temperature for (a) Base Course Mixes, and (b) Surface Course Mixes

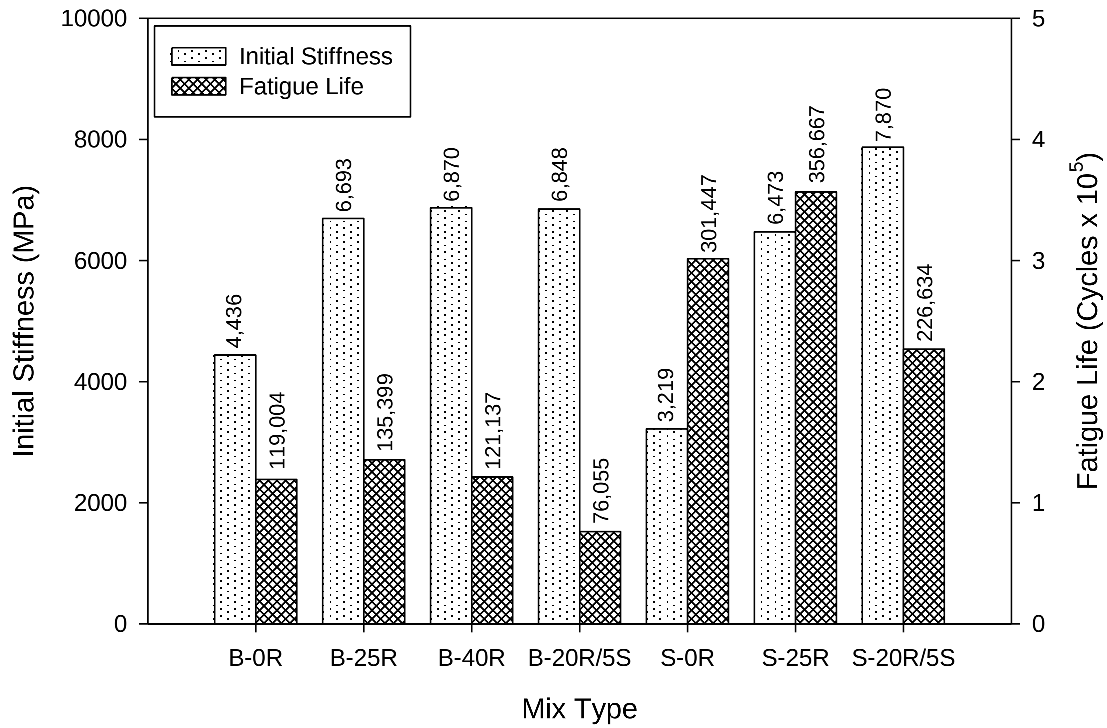


Figure 3.5 Four-Point Bending Beam Fatigue Test Results

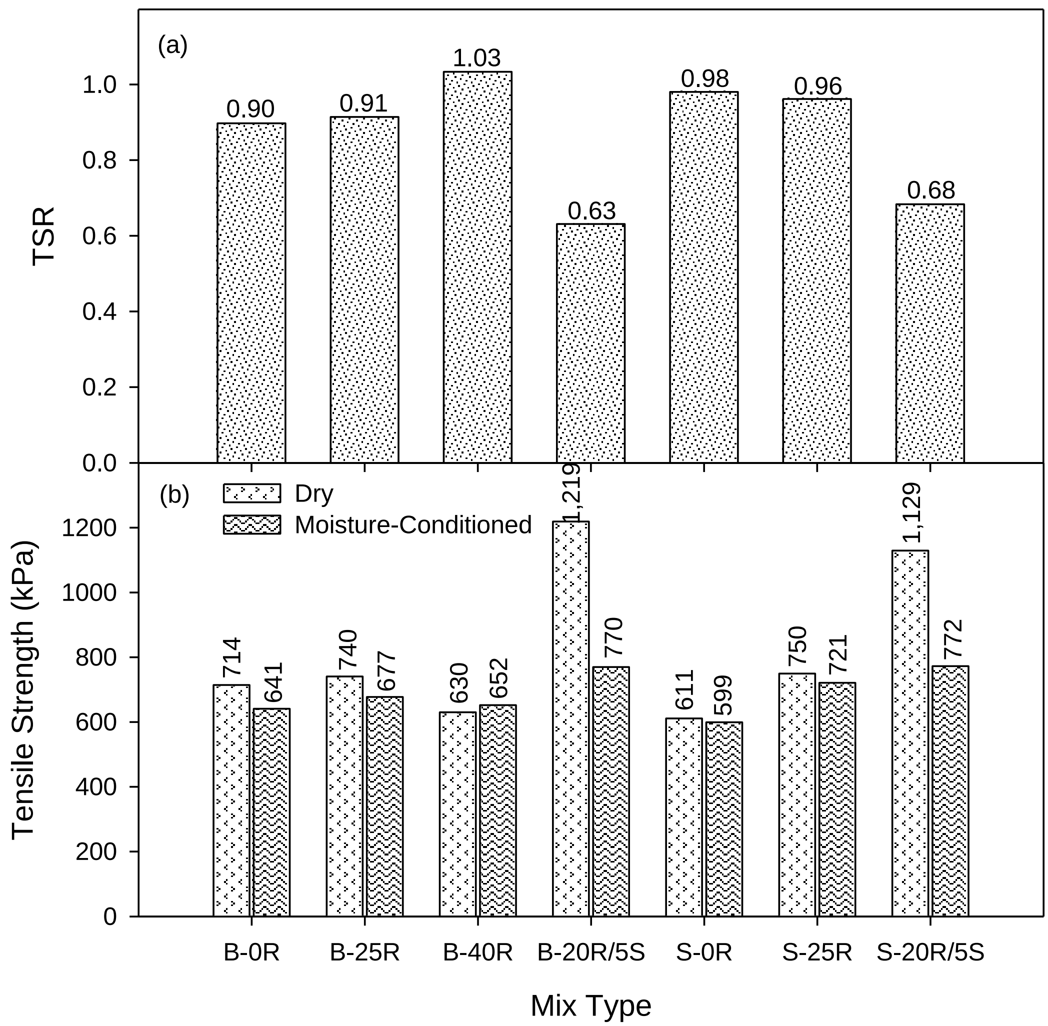


Figure 3.6(a) Average Dry and Moisture-Conditioned Tensile Strength, and (b) TSR Values

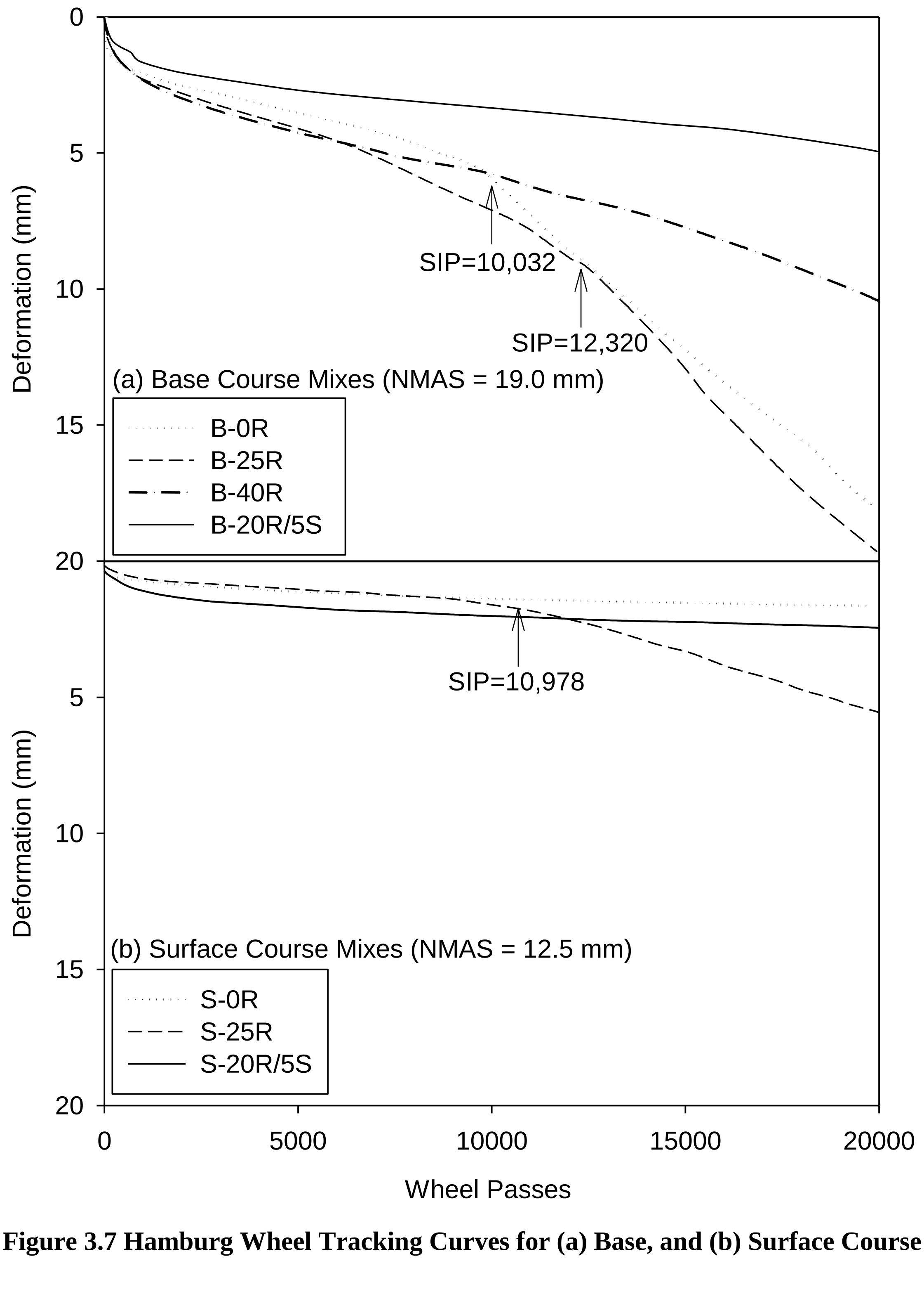


Figure 3.7 Hamburg Wheel Tracking Curves for (a) Base, and (b) Surface Course

CHAPTER

4

MECHANISTIC EVALUATION OF EFFECT OF WMA-ADDITIVES ON WETTABILITY AND MOISTURE SUSCEPTIBILITY PROPERTIES OF ASPHALT MIXES §

ABSTRACT

The present study uses a mechanistic framework (i.e., surface free energy) to evaluate the moisture susceptibility of warm mix asphalt (WMA) with three different WMA-additives, namely Sasobit®, Advera®, and Evotherm®. The surface free energy (SFE) components of modified PG 64-22 asphalt binder with different percentages of WMA-additives and selected aggregates were measured in laboratory. The wettability, the work of adhesion, the work of debonding, and energy ratios were estimated to assess the moisture-induced damage potential of combinations of modified asphalt binders and different aggregates. The results indicated that Sasobit® and Advera® are able to reduce the moisture susceptibility potential of the mixes, but are not recommended to be used with highly acidic aggregates like granite. Evotherm® resulted in the highest increase in wettability, total surface free energy, increased work of adhesion and a reduction in the

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work of debonding, resulting in a better possible aggregate coating with asphalt binder and lower moisture susceptibility with all types of tested aggregates, compared to those of other WMA-additives. Furthermore, Tensile Strength Ratio (TSR) tests were conducted on Advera® and Evotherm®-modified and neat (unmodified) asphalt mixes and results were compared with those from the SFE test. It was found that the SFE approach is a better indicator of moisture susceptibility compared to the traditional TSR test. It is expected that the present study would be helpful in understanding the moisture-damage potential of the flexible pavements constructed with WMA technologies.

Keywords: Warm mix asphalt, moisture susceptibility, surface free energy

4.1 Introduction

Warm mix asphalt (WMA) technologies are capable of significantly reducing the production and placement temperatures of asphalt mixes. This temperature reduction results in saving energy, cutting emissions, extended paving season in cold climates, and significant cuts in production costs (APAO, 2003). Despite WMA's advantages over the conventional hot mix asphalt (HMA), its ability to resist moisture-induced damage is uncertain. Moisture-induced damage is defined as loss of bond within the asphalt binder (cohesive failure), or at asphalt binder-aggregate interface (adhesive failure) due to the presence of moisture (Howson et al., 2009). Lower mixing temperature in WMA results in incomplete drying of the aggregates, and consequently, a poor bond between asphalt binder and aggregate (Hurley and Prowell, 2005). Furthermore, WMA additives like Advera® introduce water into the mix which can reduce the indirect tensile strength and may cause moisture-induced damage (Goh and You, 2012).

A limited number of studies have been conducted to investigate the moisture susceptibility of WMA mixes. Recently, Prowell et al. (2007) studied moisture-induced damage potential of HMA and Evotherm®-modified WMA mixes using tensile strength ratio (TSR) tests. In this test, the ratio of conditioned and unconditioned tensile strengths of compacted samples is used to judge the performance of a mix in terms of moisture susceptibility. A TSR value of greater than 0.8 is required to pass the mix design screening criteria. Despite its popularity, TSR tests sometime fail to correlate with field performance and provide an understanding of the mechanisms of the moisture-induced failures (Bhasin, et al., 2006). Based on the TSR test, Prowell et al. (2007) reported that WMA mixes resulted in an increase in moisture-induced damage potential compared to HMA mixes. In a related study, Hurley and Prowell (2005) found that moisture susceptibility tests on WMA mixes did not produce a solid conclusion. A recent study by Xiao et al. (2009) concluded that the use of moist aggregates increases the moisture-induced damage potential of the asphalt mixes. However, no significant change in indirect tensile strength was reported, as a result of using Aspha-Min® and Sasobit® WMA additives. Kvasnak et al. (2009) reported that WMA mixes produced in laboratory were more prone to moisture-induced damage than the mixes produced in an asphalt plant. A recent study by Kanitpong et al. (2012) revealed that WMA is more prone to moisture-induced damage than HMA. Also, asphalt mixes produced with slag aggregate were reported to be of a lower moisture-induced damage potential compared to those produced using granite aggregates. The TSR test is an empirical test and lacks a mechanistic approach to quantify the moisture-induced damage potential of the mixes. It is evident from the literature that TSR tests alone cannot predict the moisture susceptibility of asphalt mixes. Therefore, the surface free energy (SFE) approach, which gives a mechanistic understanding of moisture-induced damage, has been applied recently to study adhesion and cohesion mechanisms of HMA and WMA mixes (Bhasin, et al., 2006; Xiao et al., 2009; Kvasnak et al., 2009; Kanitpong et al., 2012; Arabani et al., 2012; Hossain et al., 2011; Wasiuddin et al., 2008; Bhasin and Little, 2007; Bhasinet al., 2007; Kim et al., 2004; Cheng, et al., 2002).

Promising results have been reported in the literature about the application of SFE approach to evaluate the moisture-induced damage potential of asphalt mixes. For example, Wasiuddin et al. (2008), using the SFE method, observed that Sasobit® increases the wettability of the aggregates by asphalt binder and reduces the adhesion between aggregates and asphalt binder. Similarly, Bhasin et al. (Bhasin, et al., 2006; Bhasin et al., 2007) suggested different combinations of SFE parameters, including work of adhesion, work of debonding, work of cohesion, and specific surface area of aggregates to describe the moisture susceptibility of an asphalt binder-aggregate system as a single value. They used fatigue and resilient modulus test results in wet and dry conditions as a moisture sensitivity measure, and developed statistically significant correlations between the abovementioned energy parameters and moisture susceptibility indices of tested mixes. In another study, Cheng et al. (2002) utilized the SFE approach to calculate the work of adhesion and free energy of cohesion for different asphalt binders and aggregates with and without the presence of water. Their results were consistent with those obtained from the accelerated moisture-induced damage tests on mixes. In a recent study, Arabani et al. (2012) reported a significant correlation between moisture-induced damage potential of WMA mixes based on SFE and ratio of conditioned to unconditioned dynamic modulus of asphalt mixes. Similarly, Kim et al. (2004) used the SFE approach and dynamic mechanical analysis (DMA) test to characterize fracture of asphalt binders and mastic, and reported that both the methods showed consistent results. According to the foregoing and other studies it is evident that the SFE approach can be used as a reliable mechanistic tool to assess the moisture-induced damage potential of HMA and WMA mixes. Not many studies have investigated the mechanics of the moisture-induced damage potential of the Evotherm® and Advera® WMA mixes in light of the SFE method. In addition, the capability of the current practice of the moisture-induced damage assessment of asphalt mixes, like TSR testing according to AASHTO T 283 (AASHTO, 2011) and its comparison with the SFE-based methods, has not been studied in detail. The present study was undertaken to evaluate the effect of three different WMA-additives (i.e., Sasobit®, Advera® and Evotherm®) on the wettability and moisture susceptibility using the SFE method. For this purpose, the wettability, the work of adhesion and the work of debonding of six types of aggregates and a PG 64-22 asphalt binder modified with different percentages of WMA-additives are evaluated. In addition, TSR tests on control HMA and WMA mixes produced with a PG 64-22 asphalt binder modified by Advera® and Evotherm® are conducted and the results are compared with the SFE parameters.

4.2 Objectives

The objectives of this study were to evaluate the effect of different types of WMA-additives on the surface free energy (SFE) and wettability properties of asphalt binder with different types of aggregates. The specific objectives of this study are listed below:

1. Determination of SFE components of a PG 64-22 asphalt binder with and without different WMA-additives, namely Sasobit®, Advera® and Evotherm®, using the Wilhelmy Plate Test.
2. Determination of SFE parameters of aggregate using the Universal Sorption Device.
3. Determination of wettability, adhesion, debonding and moisture susceptibility potential of PG 64-22 asphalt binder with and without WMA-additives in contact with different types of aggregates.
4. Determination of the moisture susceptibility potential of the HMA and WMA mixes produced with a PG 64-22 asphalt binder with Advera® and Evotherm®, using TSR test (AASHTO, 2004).
5. Comparison of TSR and SFE based approaches to evaluate the moisture susceptibility of asphalt mixes.

4.3 Background on Surface Free Energy

Surface free energy (SFE) of a solid can be defined as the work required to increase the surface of that solid by a unit area under vacuum (Van Oss et al., 1988). Similarly, the free energy required to create two interfaces from one interface consisting of two different phases in contact is called work of adhesion of that material.

According to Van Oss et al. (1988), the total surface energy can be stated in the form of three independent components, based on intermolecular forces: a monopolar acidic component (Γ^{+}), a monopolar basic component (Γ^{-}), and an apolar or Lifshitz-van der Waals component (Γ^{LW}). The total SFE (Γ^{Total}) can be stated based on Lifshitz-van der Waals component (Γ^{LW}) and acid-base component (Γ^{AB}), as shown in Equations 4.1 and 4.2.

$$\Gamma^{Total} = \Gamma^{LW} + \Gamma^{AB} \tag{4.1}$$

where,

$$\Gamma^{AB} = 2\sqrt{\Gamma^{+}\Gamma^{-}} \tag{4.2}$$

With the given SFE components of an asphalt binder and aggregate, the work of adhesion (W_{AS}) between an asphalt binder (subscript *A*) and aggregate or stone (subscript *S*) can be determined from Equation 4.3. The magnitude of work of adhesion indicates the tendency of the two phases of material to bind together (Bhasin et al., 2007).

$$W_{AS} = 2\sqrt{\Gamma_A^{LW}\Gamma_S^{LW}} + 2\sqrt{\Gamma_A^{+}\Gamma_S^{-}} + 2\sqrt{\Gamma_A^{-}\Gamma_S^{+}} \tag{4.3}$$

Similarly, the work of debonding (W_{ASW}^{wet}), as a result of separation of asphalt binder from aggregate surface due to the presence of water (subscript *W*), is determined from Equation 4.4.

$$W_{ASW}^{wet} = \Gamma_{AW} + \Gamma_{SW} - \Gamma_{AS} \tag{4.4}$$

where, Γ_{AW}, Γ_{SW} and Γ_{AS} represent the interfacial energy between asphalt binder and water, aggregate and water and asphalt binder and aggregate, respectively. The interfacial energy by definition is the energy equal to the surface tension at an interface.

The interfacial energy between materials i and j can be determined from Equation 4.5 (Bhasin et al., 2007).

$$\Gamma_{ij} = \Gamma_i + \Gamma_j - 2\sqrt{\Gamma_i^{LW}\Gamma_j^{LW}} - 2\sqrt{\Gamma_i^{+}\Gamma_j^{-}} - 2\sqrt{\Gamma_i^{-}\Gamma_j^{+}} \tag{4.5}$$

In an asphalt-aggregate system in which the debonding occurs due to the presence of water, the work of debonding is negative. This means that energy is released due to debonding, a thermodynamically favorable mechanism. Therefore, the greater the magnitude of W_{ASW}^{wet}, the higher the potential of debonding of asphalt binder from aggregate in the presence of the water (Bhasin et al., 2007).

Wetting is the ability of liquid phase to maintain its contact with the solid surface. It can be a liquid's spreading over a surface which may include penetration into porous medium (Berg, 1993). Therefore, study of the wettability can show the potential of the asphalt binder to coat the aggregates. Wettability is determined by a balance between adhesive and cohesive forces (Berg, 1993). The spreading coefficient of a liquid over a solid is a quantitative measure of wetting and is defined as SFE reduction during the loss of the bare solid surface and formation of a new solid-liquid and liquid-vapor interface (Zettlemoyer, 1968). The spreading coefficient of asphalt binder over the aggregates ($S_{A/S}$) can be determined according to Equation 4.6 (Wasiuddin et al., 2007).

$$S_{A/S} = \Gamma_S - \Gamma_{AS} - \Gamma_A \tag{4.6}$$

where, Γ_S = Surface Free Energy of aggregate, Γ_{AS} = interfacial energy between asphalt binder and aggregate, and Γ_A = free energy of asphalt binder.

Contact angle measurements of the asphalt binder with three different solvents (i.e., one apolar, one monopolar and one bipolar) are used to determine the SFE components of an asphalt binder (Bhasin et al., 2007). With given contact angles, Equation 4.7 is formed for each solvent. The system of three simultaneous equations is solved to obtain the SFE components of the asphalt binder (Wasiuddin et al., 2008).

$$\Gamma_L(1+\cos\theta) = 2\times\sqrt{\Gamma_A^{LW}\Gamma_L^{LW}} + 2\times\sqrt{\Gamma_A^{+}\Gamma_L^{+}} + 2\times\sqrt{\Gamma_A^{-}\Gamma_L^{-}} \quad (4.7)$$

where θ = contact angle, $\Gamma_L^{LW}, \Gamma_L^{+}$ and Γ_L^{-} = SFE components of the liquid solvent.

4.4 Materials

4.4.1 Asphalt Binder and Aggregate

A PG 64-22 asphalt binder and limestone aggregates were collected from the Valero refinery in Muskogee, OK and from the Dolese quarry in Oklahoma, respectively. In addition, the SFE components of commonly used aggregates for pavement construction, including sandstone, gravel, granite and basalt, were adopted from the open literature (Bhasin et al., 2007; Buddhala et al., 2011) to evaluate the effect of aggregates' source and types on energy components and moisture susceptibility of asphalt binder-aggregates systems.

4.4.2 WMA-Additives

Three different WMA-additives, namely Sasobit®, Advera®, and Evotherm®, were selected in the present study. These additives are currently used in practice to produce WMA mixes (Hurley and Prowell, 2005). A brief description of each of the additives is provided in this section.

Sasobit®— Sasobit® is a type of paraffin wax produced by conversion of carbon monoxide into higher hydrocarbons in catalytic hydrogenation, followed by a distillation process called Fischer-Tropsch (FT) synthesis. It has a long molecular chain and fine crystalline structure which results in complete solubility in asphalt binder at temperatures in excess of 240° F (115.6° C). Sasobit® produces a reduction in asphalt binder's viscosity, which in turn makes it possible to drop production temperatures by 18° F (10° C) to 54° F (30° C) (Corrigan, 2012). Use of 0.8% to 3.0% Sasobit® by the weight of the asphalt binder is recommended by the manufacturer (Hurley and Prowell, 2005). For the present study, Sasobit® was collected from Sasol Wax plant in Richmond, CA.

Advera®— Advera® is a product of PQ Corporation in Malvern, PA. It is a synthetic zeolite (Sodium Aluminum Silicate) containing 18 to 21 percent water by mass. This water is entrapped in its crystalline structure and releases at temperature above 210° F (99° C), and creates a foaming of the asphalt binder in the mix. The foaming effect improves the workability of the asphalt mix, which enables production and placement temperatures to be reduced by 50° F (28° C) to 70° F (39° C) compared to conventional HMA (Corrigan, 2012). PQ Corporation recommends use of 0.25% Advera® by weight of the mix to gain desired workability. For the present study, Advera® was collected from asphalt mix production plant located in Bridgeport, TX.

Evotherm®— Evotherm® is a product of MeadWestvaco Asphalt Innovations in Charleston, SC. Chemical additive technology and a "Dispersed Asphalt Technology" (DAT) delivery system are used for the production of Evotherm®. Based on MeadWestvaco reports, field testing of WMA with Evotherm® show a 100° F (55.5° C) reduction in production temperature (MeadWestvaco, 2012). The optimal amount of Evotherm® recommended by Hurley and Prowell (2006) is 0.5% by the weight of the asphalt binder. For the present study, Evotherm® was collected from an asphalt mix production plant located in San Antonio, TX.

4.5 Methodology

The selected asphalt binder (i.e., PG 64-22) was modified with different amounts of Sasobit® (i.e., 1.0%, 1.5% and 2.0% by the weight of asphalt binder), Advera® (i.e., 0.25%, 0.30% and 0.35% by the weight of asphalt mix) and Evotherm® (i.e., 0.25%, 0.50% and 0.75% by the weight of asphalt binder) (Table 4.1). The selection of the amounts of these additives was made based on their optimal dosages as recommended in the literature or by the manufacturer. In the present study, a wide range of dosage was considered to evaluate the effect of these additives on moisture susceptibility of asphalt mixes. The SFE components of modified asphalt binders and neat asphalt binder were determined based on the measurement of the contact angles. Contact angles of asphalt binders were measured in the laboratory using the Wilhelmy Plate Test using three different solvents of known SFE components, namely water, glycerin and formamide, according to the methodology used by Wasiuddin et al. (2007). A total of 108 asphalt binder samples were prepared in the laboratory and tested for contact angles.

Similarly, the SFE components of selected limestone aggregate were determined using a universal sorption device (USD) and applying the methodology discussed by Bhasin and Little (2007) (see Table 4.1). The probe vapors of known SFE components, namely water, n-hexane, and methyl propyl ketone (MPK) were used to determine adsorption isotherms. A total of 9 aggregate samples were tested in the USD. For this purpose a SGA – 100 USD device was used.

The SGA -100 from VTI Corporation is a gravimetric sorption device designed for water and organic vapor sorption studies of materials. This technique is based on the development of a vapor sorption isotherm, i.e. the amount of vapor adsorbed or desorbed on the solid surface at a fixed temperature and partial pressure (Bhasin and Little, 2007). The range of relative pressure (RP) can be varied from 0.02 to 0.98 and temperatures from 5^{o}C to 60°C. At each relative humidity (RH) or pressure step the system controls the RH or RP and monitors sample weight until it reaches equilibrium. Sample weight, temperature, and RH or RP are recorded in a data file at user defined intervals. Identical conditions of temperature and humidity for a sample and a reference are achieved by using a symmetrical, two-chamber aluminum block. The critical components of the system (microbalance, aluminum block, and humidifier) are thermostatically separated. Sample weight changes are recorded using a Cahn D-101 microbalance.

The relative pressure or humidity is determined with a dew point analyzer. To prepare aggregate samples for testing, limestone aggregates were crushed. The portion passing a No.4 sieve and retaining on a No. 8 sieve was selected and washed with distilled water to obtain a dust-free and clean surface. The aggregate was then oven dried at 120°C for 12 hours and allowed to cool to room temperature in a desiccator sealed with silica gel. About 20 grams of aggregate was used to conduct one USD test. At least three replicate samples were tested to ensure consistency and reproducibility of results.

Thereafter, Equations 4.1 through 4.6 were used to determine the spreading coefficient, wettability, work of adhesion, and work of debonding in asphalt binder-aggregate systems.

The effect of using different WMA-additives on moisture susceptibility of selected asphalt mixes was examined in this study. Also, a correlation between the SFE-based approach and laboratory performance was pursued. For this purpose, TSR tests were conducted on HMA (control mix) and WMA (Advera® and Evotherm®-modified) mixes in accordance with the AASHTO T 283 test method (AASHTO, 2011) (Table 4.1).

4.6 Results and Discussion

4.6.1 Contact Angles

The laboratory measured contact angles of modified asphalt binders and neat asphalt binder with water, glycerine, and formamide are presented in Table 4.2. In general, when the contact angle is more than 90° the solvent is unable to wet the surface. When the contact angle is less than 90° the solvent is able to wet the surface. When the contact angle is close to zero spreading of the solvent on the surface can happen. Overall, addition of Sasobit® and Evotherm® resulted in reduced contact angles compared to those of the neat asphalt binder (Table 4.2). Similar trend in contact angle variation with amounts of Sasobit® has also been reported by Buddhala et al. (2011). On the other hand, the addition of Advera® resulted in a mixed trend (i.e., increase or decrease) in contact angle depending upon the amount of Advera® added (Table 4.2). It is expected that Advera® introduces free water to asphalt binder at mixing temperature (Goh and You, 2012) which in turn affects the contact angles. The implications of variations in contact angles on the properties of the asphalt binder are expected to influence the wettability, SFE components, and energy parameters such as work of adhesion and debonding and moisture-induced damage potential, which is discussed later.

4.6.2 SFE Components of Asphalt Binders

The SFE components of PG 64-22 asphalt binder modified with different percentages of Sasobit®, Advera® and Evotherm® are presented in Table 4.3. It was found that the total SFE component (Γ^{Total}) and non-polar Lifshitz-van der Waals component (Γ^{LW}) of asphalt binder decreases with an increase in the amount of Sasobit® and Advera® (Table 4.3). For example, the addition of 2% of Sasobit® and 0.35% of Advera® decreased the total SFE to 10.39 mJ/m^2 and 8.91 mJ/m^2 compared to 11.57 mJ/m^2 for the neat asphalt binder, respectively. Similar observations on reduction in total SFE with an increase in the amount of Sasobit® have been reported by Wasiuddin et al. (2008). The reduction in the total SFE may affect the adhesion of an asphalt binder with the aggregates (Wasiuddin et al., 2008; Arabani and Hamedi, 2011).

Furthermore, Table 4.3 shows that an increase in the amounts of Sasobit® and Advera® increases the ratio of acid to base (Γ^+/Γ^-) component, indicating an increase in acidity of the asphalt binder (Wasiuddin et al., 2008; Buddhala et al., 2011). Highly acidic asphalt binders may not result in a good bond with acidic aggregates such as sandstone, gravel, and specifically granite, since the surface chemistry of Lewis acid and bases do not favor adhesion in this case (Arabani and Hamedi, 2011).

On the other hand, no detectable trend of (Γ^+/Γ^-) was observed for Evotherm®-modified asphalt binder. Asphalt binder modified by 0.25% Evotherm® resulted in a reduction of the total SFE component by 0.58 mJ/m^2 compared to the neat binder. However, when the amount of Evotherm® was increased to 0.5% and 0.75% the total SFE component increased to 12.24 and 13.69 mJ/m^2, respectively.

4.6.3 SFE Components of Aggregates

SFE components of the selected limestone aggregate and other different types of aggregates (i.e., sandstone, gravel, granite and basalt) from literature (Bhasin et al., 2007; Buddhala et al., 2011) are presented in Table 4.3. The SFE components of the limestone aggregate used in the present study are comparable to the results reported for one other limestone aggregate by Buddhala et al. (Buddhala et al., 2011). The Γ^+/Γ^- ratio of different types of aggregates used in this study was found to be in the following order.

$$\Gamma^+/\Gamma^-_{Granite} > \Gamma^+/\Gamma^-_{Sandstone} > \Gamma^+/\Gamma^-_{Gravel} > \Gamma^+/\Gamma^-_{Limestone} > \Gamma^+/\Gamma^-_{Basalt}$$

It was observed that granite is the most acidic aggregate with an acid to base component ratio (Γ^+/Γ^-) of 0.251, and basalt is the most basic aggregate with a Γ^+/Γ^- ratio of 0.004 (Table 4.3). One should be careful using an acidic aggregate such as granite with asphalt binder, which is acidic in nature. This may result in a weak bond between asphalt binder and aggregate (Arabani and Hamedi, 2011), and consequently, high moisture-induced damage potential.

4.6.4 Wettability

Asphalt binder and aggregates pose hydrophobic and hydrophilic natures, respectively (Tarrer and Wagh, 1991). For this reason, wetting and coating aggregates surfaces with the asphalt binder is not easy (Wasiuddin et al., 2008). Hence, there is a need to study the wettability of the liquid asphalt binder over the aggregate surface. The tendency of a liquid to wet a solid surface is expressed in terms of the spreading coefficient (Zettlemoyer, 1968). The spreading coefficient of the liquid asphalt binder over the aggregates ($S_{A/S}$) is the released energy as the liquid asphalt binder readily flows over the aggregates and coat it (Wasiuddin et al., 2008). Therefore, a higher spreading coefficient of an aggregate-asphalt binder system means a higher tendency of the aggregate to be coated by the liquid asphalt binder, which is in favor of better bonding and reduces the possibility of moisture-induced damage. In this study, the spreading coefficient of asphalt binder with and without WMA-additives over different types of aggregates mentioned in Table 4.3, was determined using the Equation 4.6, and the results are presented in Table 4.4.

Table 4.4 shows that the spreading coefficient increases with an increase in amount of Sasobit® for almost all types of the aggregates, compared to that of the neat asphalt binder. A significant improvement in the spreading coefficient (i.e., 25.4%) was found for gravel when 2% Sasobit® was added to asphalt binder (Table 4.4). However, only a 3.1% improvement in the spreading coefficient was observed in the granite case with the addition of 2% Sasobit®. This means that Sasobit®-modified asphalt binders may coat the sandstone, gravel, limestone, and basalt aggregates better than granite aggregates. Therefore, use of Sasobit®-modified asphalt binders with granite aggregates may possibly increase the moisture-induced damage potential of the mixes.

Similarly, the addition of Advera® increased the spreading coefficients over the almost all types of the aggregates. The maximum improvement in the spreading coefficient (i.e., 36.5%) was found for the limestone and gravel aggregates when 0.25% Advera® was added to the asphalt binder (Table 4.4). Use of the same amount of

Advera® improved the spreading coefficient up to 7.2% for granite aggregate (Table 4.4). Consequently, Advera®-modified asphalt binder, when used with granite, may possibly increase the moisture susceptibility of the mix due to a low spreading coefficient (i.e., insufficient coating of aggregates by asphalt binder), compared to the mixes produced with the other types of aggregates. Similar results have been reported by You et al. (2011b).

Furthermore, significant improvement in the spreading coefficient (i.e., 78.9% compared to neat asphalt binder) with the use of the Evotherm® was found for gravel when 0.75% Evotherm® was added to the asphalt binder (Table 4.4). In addition, a 33.9% improvement in the spreading coefficient was observed for granite aggregate with the addition of 0.75% Evotherm®. The results indicate that a better granite aggregate coating is expected when Evotherm® is used with the asphalt binder. In other words, Evotherm® -modified asphalt binder may be used over the different types of aggregate (discussed herein) with less concern over wettability, and therefore a less moisture susceptibility resulting from aggregate coating quality by binder.

4.6.5 Work of Adhesion

Work of adhesion (W_{AS}) is defined as the work required to separate the asphalt binder from aggregate interface (Bhasin et al., 2007). Higher W_{AS} indicates a stronger bond between asphalt mix components, leading to a more durable and a less moisture susceptible mix. Hence, the study of the work of adhesion is very important to gain a better understanding of the moisture-induced damage mechanism (Wasiuddin et al., 2008). Table 4.4 shows the work of adhesion between the aggregates and the PG 64-22 asphalt binder modified with different types and amounts of the WMA-additives.

Table 4.4 shows that the increased amounts of Sasobit®, Advera®, and Evotherm® increased the work of adhesion between asphalt binder and aggregates. However, an increase in the work of adhesion is not significant when Sasobit® and Advera®-modified asphalt binders are used with granite aggregate. As mentioned before, addition of Sasobit® and Advera® increases the acidity of the asphalt binder, and a good adhesion of an acidic asphalt binder with an acidic aggregate (i.e., granite) is very difficult to obtain. Granite aggregate is a highly acidic aggregate, with a Γ^{+}/Γ^{-} ratio of 0.25, significantly higher than that of other aggregates. Based on the work of adhesion, it can be concluded that the use of Sasobit® and Advera®-modified asphalt binders with sandstone, gravel, limestone and basalt aggregates may result in a better adhesion, compared to the mixes containing granite aggregates.

However, the addition of Evotherm® to the selected asphalt binder resulted in a significant improvement in the work of adhesion with all types of aggregates. For example, addition of 0.75% of Evotherm® results in an improvement in the work of adhesion. This improvement is more pronounced with a maximum increasing rate of 67.6% in the gravel case. The least improvement in the work of adhesion, with the use of same amount of Evotherm®, was observed in the granite case. It is desirable for an asphalt mix to have a work of adhesion as high as possible to be durable and less moisture susceptible (Bhasin et al., 2007). Therefore, it is expected that the use of Evotherm®-modified asphalt binder may improve the durability and resistance against moisture-induced damage of the mixes produced with both acidic and basic aggregates.

4.6.6 Work of Debonding

Work of debonding (W_{ASW}^{wet}) is another important energy parameter, defined as the reduction of the free energy of the asphalt binder and aggregate system when asphalt binder gets separated from its interface with aggregate in the presence of the water. Hence, a higher magnitude of the work of debonding implies a higher thermodynamic potential for stripping to occur in the presence of the water (Bhasin et al., 2007). Therefore, a lower work of debonding is more favorable to reduce the moisture susceptibility of the system (Bhasin et al., 2007). Table 4.4 presents the work of debonding between the aggregates and the PG 64-22 asphalt binder modified with different types and amounts of WMA-additives.

Table 4.4 shows that the addition of Sasobit® decreased the work of debonding, except in the granite case. The maximum desirable effect was observed when 2% Sasobit® was added to asphalt binder. Use of 2% Sasobit® with the selected asphalt binder and limestone aggregate resulted in the highest reduction (10.1%) in the work of debonding, compared to that of the neat asphalt binder. However, the use of 2% Sasobit®-modified asphalt binder with granite aggregate increased the work of debonding by 3.7%, which is not desirable when moisture-induced damage resistance is of concern. Asphalt binder modified with Advera® resulted in a decrease in the work of debonding with all types of aggregates. A significant reduction in the work of debonding was obtained when 0.35% Advera® was added to the asphalt binder with basalt. This resulted in a 36.2% reduction in the work of debonding compared to the neat asphalt binder case. However, addition of 0.35% Advera® reduced the work of debonding by 3.6% for granite aggregate, indicating that use of Advera® -modified asphalt binder is not recommended for granite aggregates.

The reduction in work of debonding of the asphalt binder modified with Evotherm® with different aggregates are more significant compared to that of Sasobit® and Advera®. The maximum reduction in the work of debonding for basalt aggregates was observed when 0.75% of Evotherm® was added to the PG 64-22 asphalt binder. Similarly, a reduction of 18.9% was observed in the work of debonding for granite aggregate, which is significantly higher than those for Sasobit® and Advera®. Based on the work of debonding, it can be concluded that Evotherm® might be used with the aggregates discussed in this study, with possibly less concern over the moisture susceptibility of the mix compared to that of other types of additives discussed herein.

4.6.7 Comparison of Moisture Susceptibility Potential Based on SFE and TSR

Based on the definitions of the work of adhesion and work of debonding, it can be concluded that the moisture susceptibility of an asphalt binder-aggregate system decrease with an increase in the work of adhesion (W_{AS}), and increases with an increase in the magnitude of work of debonding ($\left|W_{ASW}^{wet}\right|$) (Bhasin et al., 2007). Consequently, Bhasin et al. (Bhasin et al., 2007) suggested combining W_{AS} and W_{ASW}^{wet} into a single parameter called energy ratio (ER_1), which is directly proportional to the resistance against moisture-induced damage as shown in Equation 4.8.

$$ER_1 = \left|\frac{W_{AS}}{W_{ASW}^{wet}}\right| \tag{4.8}$$

A higher ER_1 value implies a better resistance against the moisture-induced damage, and therefore, a lower moisture susceptibility of the asphalt binder-aggregate system (Bhasin et al., 2007). This value is analogues to the TSR value obtained according to the AASHTO T 283 (AASHTO, 2011) method. The ER_1 values of different combinations of additives and aggregates are presented in Table 4.5.

The ER_1 values in Table 4.5 show that Sasobit® does not significantly increase or decrease the moisture susceptibility of the asphalt binder-aggregate systems. The same trend is observed for combinations of Advera® with different types of aggregates as well, except basalt. The use of Advera® -modified asphalt binder with basalt aggregates increases moisture-induced damage resistance by 68%. The addition of the Evotherm® to the selected asphalt binder results in the highest moisture-induced damage resistance compared to that of other additives, and neat asphalt binder used over different aggregates.

To this end, moisture susceptibility of the asphalt binder-aggregate systems was discussed in light of the SFE method. In order to assess the current practice of the evaluation of the moisture susceptibility used by DOTs and highway agencies, TSR tests were conducted on the asphalt mix samples of surface mixes produced with three types of PG 64-22 asphalt binders namely, control HMA, modified with Advera® and Evotherm® (WMA). The aggregate used for all of the mixes was limestone. TSR samples for all three mixes (HMA and WMA) were compacted in the laboratory at 7±1% target air voids using a superpave gyratory compactor (SGC). The TSR tests were conducted in accordance with AASHTO T 283 (AASHTO, 2011). Wet tensile strength and the TSR values of the above mentioned asphalt mixes are presented in Table 4.6. According to Table 4.6, TSR values of the mixes produced with neat, Advera® and Evotherm®-modified asphalt binders were obtained as 0.93, 0.71, and 0.66, respectively. A higher TSR value (0.93) for HMA, produced with neat asphalt binder, can be attributed to a better drying of aggregate as a result of higher mixing temperatures, compared to those of WMA. As per the standard, a minimum TSR value of 0.80 is required for any mix to pass the mix design phase. Based on this criterion, mix with neat asphalt binder passes the requirement (TSR>0.80). However, mixes prepared with Advera® and Evotherm® did not pass the requirement, and therefore, need to be redesigned. It should be noted that TSR ratio of 0.80 was developed for HMA mixes, not for WMA mixes. Therefore, it is recommended that a research study be conducted to develop a desirable range of TSR value for different WMA mixes.

In this study, the ranking of the mixes was established based on ER_1 and TSR values. The ER_1 values for mixes with neat asphalt binder, Evotherm® and Advera®-modified asphalt binder and limestone aggregate was found to be 1.5,1.0, and 0.6, respectively. Therefore, based on the ER_1 values, the ranking of mixes for resistance to moisture-induced damage can be considered as: Evotherm®, Advera®-modified mixes followed by HMA mix. However, the ranking of the mixes based on TSR value was found to be as follow: HMA, Advera® and Evotherm®-modified mixes. It should be noted here that the ranking of the mixes for resistance to moisture-induced damage based on SFE and TSR approach was found to be in reverse order of each other. In other words, none of these three TSR values follow the ranking of the moisture susceptibility according to the ER_1 values. This trend suggests that TSR test may not be able to capture moisture susceptibility of the tested mixes. Because of limited scope, the results from this study may not be generalized for other mixes. Additional research would be needed to correlate the SFE results with the moisture susceptibility of asphalt mixes through more traditional testing.

4.7 Conclusions and Recommendations

The present study evaluated the effect of three WMA-additives, namely, Sasobit®, Advera®, and Evotherm® on SFE components of the PG 64-22 asphalt binder. The wettability, work of adhesion and work of debonding of the modified and unmodified asphalt binders over different types of the aggregates were determined and moisture susceptibility potential was evaluated. In addition, TSR tests were conducted to evaluate the current practice of the moisture-induced damage compared with the SFE approach. The following conclusions may be drawn from this study.

1. Sasobit® and Advera® additives were found to reduce the total SFE component of the asphalt binder. Evotherm®, on the other hand, increased the total SFE of the asphalt binder.
2. Sasobit®, Advera® and Evotherm® increase the wettability of the asphalt binder over the aggregates, observed as an increase in the spreading coefficient. However, Evotherm® was found to cause a more significant increase in the spreading coefficient for all aggregates, specifically with gravel. This implies a better aggregate coating by asphalt binder.
3. Sasobit®, Advera® and Evotherm® increase the work of adhesion of asphalt binder over the aggregates. Evotherm® was observed to cause a more significant improvement in the work of adhesion for all aggregates, specifically for gravel. This may result in a more durable asphalt mix.

4. Sasobit®, Advera® and Evotherm® reduce the magnitude of work of debonding of the asphalt binders over the aggregates. Addition of Evotherm® was observed to result in a more significant reduction in the magnitude of the work of debonding, and is expected to possibly lower the moisture susceptibility of the mix.
5. Works of adhesion to debonding ratios were used as indicators of the moisture susceptibility of the asphalt binder-aggregate systems. Based on this method, Sasobit® and Advera® do not significantly increase or decrease the moisture susceptibility potential of the asphalt binder, over different aggregates. However use of Advera®-modified asphalt binder with basalt results in a measurable decrease in moisture susceptibility of the mix. Evotherm® was observed to have the maximum effect on the reduction of moisture susceptibility potential.
6. The TSR test was observed (possibly) not to be able to capture moisture susceptibility of the mixes produced using WMA-additives.

According to the methodology and materials used in this study the following recommendations are suggested.

1. The aggregates and asphalt binder from different sources may have different chemical and SFE properties. Therefore, it is recommended that a study be conducted to investigate the influence of source of aggregates and asphalt binder on SFE-based moisture susceptibility of asphalt mixes.

2. Conducting Hamburg wheel tracking and/or rut and moisture susceptibility tests over mixes using asphalt pavement analyzer and comparing their results with TSR and SFE-based moisture susceptibility is recommended.
3. Developing correlations between the SFE-based energy ratio and tensile strength ratio (TSR) is recommended. For developing a significant and valid correlation, a larger database of test results on asphalt binder, aggregate and performance tests on asphalt mixes are required.

Table 4.1 Test Matrix

Material	Types of Additives	Percentage of Additives*	Solvents	No. of Samples
PG 64-22	Sasobit®	0%, 1.0%, 1.5% and 2.0%	Water, Glycerin and Formamide	36
	Advera®	0%, 0.25%, 0.30 and 0.35%	Water, Glycerin and Formamide	36
	Evotherm®	0%, 0.25%, 0.50% and 0.75%	Water, Glycerin and Formamide	36
Limestone Aggregate		Set 1	Water	3
		Set 2	MPK	3
		Set 3	n-Hexane	3
Type of Mix	**Types of Additives**	**Mix Description**	**Type of the Test**	**No. of Samples**
WMA	Advera®	Field Collected Mix	TSR+	4 Conditioned and 4 Unconditioned
WMA	Evotherm®	Field Collected Mix	TSR+	4 Conditioned and 4 Unconditioned
HMA	-	Lab. Produced Mix	TSR+	4 Conditioned and 4 Unconditioned

* The percentages of additives are based on the weight of binder for Sasobit® and Evotherm® and weight of asphalt mix for Advera®

\+ TSR Tests conducted in accordance with AASHTO T283.

Table 4.2 Contact Angles of PG 64-22 Asphalt Binder Modified with WMA-Additives

Type and Amounts of Additives Mixed with PG 64-22 Binder		Advancing Contact Angle (Deg)					
		Water		Glycerine		Formamide	
		Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.
Neat		108.6	0.8	97.0	0.6	92.8	0.2
Sasobit®	1.00%	108.2	0.1	96.6	0.1	92.6	0.4
	1.50%	107.5	0.5	95.5	0.3	92.4	0.1
	2.00%	106.8	0.4	95.0	0.0	92.3	0.5
Advera®	0.25%	106.7	0.4	92.2	0.3	89.4	0.2
	0.30%	109.1	0.5	92.7	0.5	89.7	0.3
	0.35%	110.2	0.6	94.0	0.2	91.1	0.6
Evotherm®	0.25%	104.6	0.2	91.0	0.7	88.6	0.4
	0.50%	101.9	0.1	91.2	0.3	88.9	0.4
	0.75%	100.7	0.6	92.8	0.4	89.2	0.5

Table 4.3 SFE Components of PG 64-22 Asphalt Binder Modified with WMA-Additives and Aggregates

		Surface Free Energy Components (mJ/m^2)					
		Γ^{LW} (Non-polar)	Γ^{-} (Base)	Γ^{+} (Acid)	Γ^{AB}	Γ_{total}	Γ^{+}/Γ^{-}
PG64-22 Binder with Different Types and Amounts of Additives							
Neat	0%	9.44	0.93	1.22	2.13	11.57	1.30
Sasobit®	1.0%	9.09	1.00	1.36	2.34	11.43	1.35
	1.5%	7.44	1.14	2.11	3.10	10.54	1.86
	2.0%	6.78	1.33	2.44	3.61	10.39	1.83
Advera®	0.25%	7.36	0.76	3.08	3.07	10.43	4.03
	0.30%	7.58	0.28	3.14	1.88	9.46	11.14
	0.35%	7.16	0.25	3.04	1.75	8.91	12.12
Evotherm®	0.25%	6.84	1.24	3.45	4.14	10.99	2.77
	0.50%	6.74	2.50	3.03	5.50	12.24	1.21
	0.75%	9.17	3.03	5.50	4.52	13.69	1.82
Aggregates from Testing and Literature							
Limestone (Tested)		51.4	741.4	17.5	227.8	279.2	0.024
Sandstone*		43.5	555.2	28.2	250.3	293.8	0.051
Gravel*		57.5	973	23	299.2	356.7	0.024
Granite*		133.2	96	24.1	96.2	229.4	0.251
Basalt*		52.3	164	0.6	19.8	72.1	0.004

* Adopted from literature [11, 19]

Table 4.4 Energy Parameters of PG 64-22 Asphalt Binder with Additives and Aggregates

WMA Additive Type and Amount		Limestone	Sandstone	Gravel	Granite	Basalt
Spreading Coefficients (mJ/m^2)						
Neat PG64-22		89.0	79.6	101.5	78.9	51.0
Sasobit®	1.0%	92.3	82.5	105.3	79.4	52.2
	1.5%	106.0	94.7	121.1	80.8	57.2
	2.0%	111.3	99.5	127.3	81.3	58.7
Advera®	0.25%	120.9	106.9	138.2	84.7	64.7
	0.30%	121.5	106.6	138.5	84.6	67.1
	0.35%	119.7	105.0	136.4	83.0	66.3
Evotherm®	0.25%	126.0	111.9	144.3	85.8	65.2
	0.50%	120.8	108.6	138.7	85.1	60.1
	0.75%	158.3	141.6	181.6	105.6	79.2
Work of Adhesion (mJ/m^2)						
Neat PG64-22		112.2	102.7	124.6	102.0	74.2
Sasobit®	1.0%	115.2	105.4	128.1	102.3	75.0
	1.5%	127.1	115.7	142.2	101.9	78.3
	2.0%	132.1	120.2	148.0	102.1	79.5
Advera®	0.25%	141.8	127.8	159.0	105.6	85.5
	0.30%	140.5	125.5	157.4	103.5	86.0
	0.35%	137.5	122.8	154.2	100.9	84.2
Evotherm®	0.25%	148.0	133.9	166.3	107.7	87.1
	0.50%	145.2	133.1	163.1	109.6	84.6
	0.75%	185.7	169.0	208.9	133.0	106.6
Work of Debonding (mJ/m^2)						
Neat PG64-22		-176.0	-154.4	-213.6	-58.1	-34.5
Sasobit®	1.0%	-173.5	-152.2	-210.6	-58.2	-34.1
	1.5%	-162.4	-142.8	-197.3	-59.5	-31.7
	2.0%	-158.2	-139.1	-192.4	-60.2	-31.4
Advera®	0.25%	-148.7	-131.7	-181.5	-56.8	-25.4
	0.30%	-147.1	-131.1	-180.2	-56.0	-22.0
	0.35%	-148.7	-132.4	-182.1	-57.3	-22.6
Evotherm®	0.25%	-145.0	-128.2	-176.8	-57.2	-26.4
	0.50%	-151.1	-132.3	-183.3	-58.7	-32.3
	0.75%	-122.4	-108.2	-149.2	-47.1	-22.0

Table 4.5 SFE-Based Moisture Susceptibility Parameters, ER_1

Type and Amount of Additive Mixed with PG 64-22 Binder		ER_1				
		Limestone	Sandstone	Gravel	Granite	Basalt
Neat	0%	0.6	0.7	0.6	1.8	2.2
Sasobit®	1.0%	0.7	0.7	0.6	1.8	2.2
	1.5%	0.8	0.8	0.7	1.7	2.5
	2.0%	0.8	0.9	0.8	1.7	2.5
Advera®	0.25%	1.0	1.0	0.9	1.9	3.4
	0.30%	1.0	1.0	0.9	1.8	3.9
	0.35%	0.9	0.9	0.8	1.8	3.7
Evotherm®	0.25%	1.0	1.0	0.9	1.9	3.3
	0.50%	1.0	1.0	0.9	1.9	2.6
	0.75%	1.5	1.6	1.4	2.8	4.8

Table 4.6 Wet Tensile Strength and TSR Values of Tested Asphalt Mixes

Asphalt Mix Type	Wet Tensile Strength (kPa)						TSR
	No. 1	No. 2	No. 3	No. 4	Average	St. Dev.	
Control HMA	1561.0	1814.0	1784.4	1826.4	1746.4	124.9	0.93
Advera® WMA	994.9	1481.7	1510.0	1475.5	1365.5	247.5	0.71
Evotherm® WMA	900.4	1050.9	1076.1	1136.7	1041.0	100.4	0.66

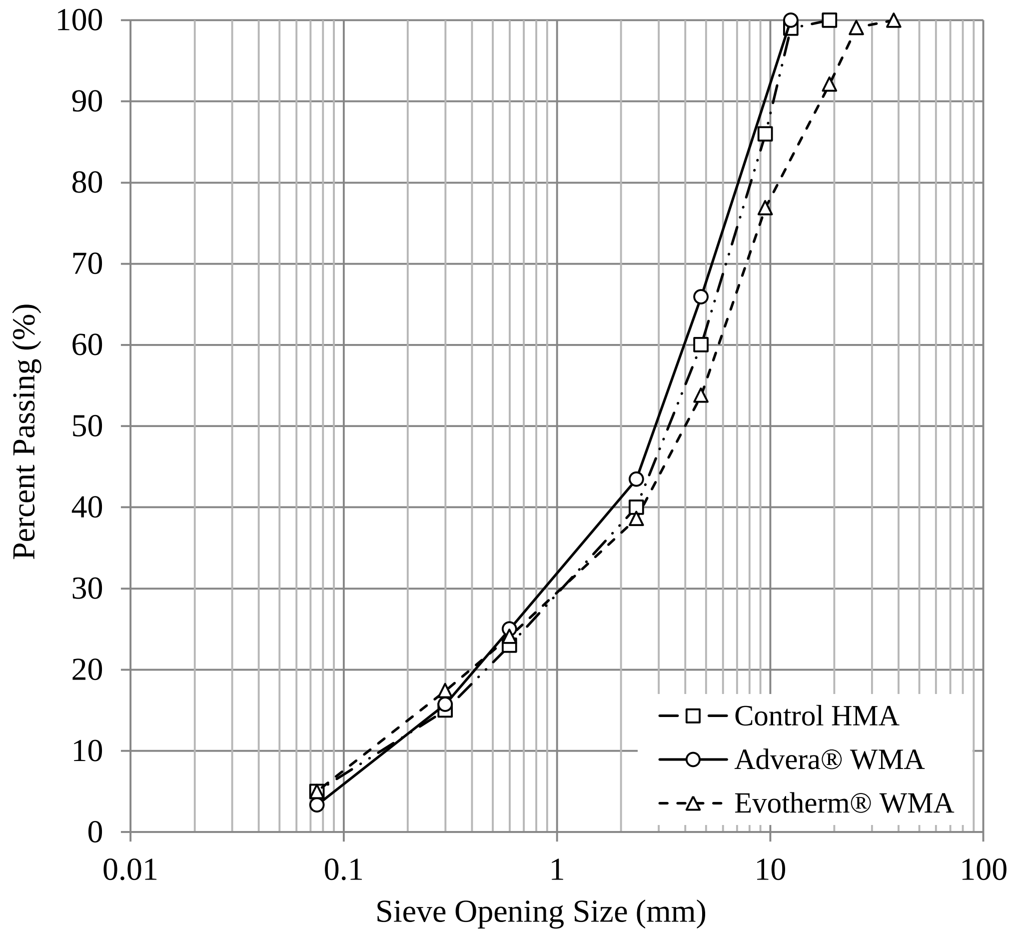

Figure 4.1 Gradation of the Asphalt Mixes used for TSR Tests

CHAPTER

5

EVALUATION OF MOISTURE SUSCEPTIBILITY OF ASPHALT MIXES CONTAINING RAP AND DIFFERENT TYPES OF AGGREGATES AND ASPHALT BINDERS USING THE SURFACE FREE ENERGY METHOD**

ABSTRACT

The Surface Free Energy (SFE) measurement of asphalt binder and aggregate is considered a reliable mechanistic framework for evaluating the moisture-induced damage potential of asphalt mixes. In the present study, the SFE method was used to evaluate the effects of asphalt binder type, reclaimed asphalt pavement (RAP), and aggregate type on the moisture-induced damage potential of asphalt mixes. The SFE components (non-polar, acid and base) of a PG 64-22 and a PG 76-28 (polymer-modified) asphalt binder, each blended with different percentages of RAP binder (0%, 10%, 25% and 40%) were measured using a dynamic contact angle (DCA) analyzer. The aggregates consisted of limestone, rhyolite, sandstone, granite, gravel, and basalt. The SFE components of limestone and rhyolite aggregates were measured using a Universal Sorption Device (USD), while those of the other aggregates (sandstone,

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granite, gravel, and basalt) were obtained from the literature. The energy ratio parameters estimated based on the spreading coefficient, the work of adhesion, and the work of debonding were used to assess the moisture-induced damage potential of different combinations of asphalt binders and different RAP binder contents and aggregates. The results indicate that the acid SFE component of PG 64-22 and PG 76-28 asphalt binders increase with the addition of RAP binder, while the base SFE component remains almost unchanged. Furthermore, the wettability and the work of adhesion of both PG 64-22 and PG 76-28 asphalt binders over different types of aggregates increased with an increase in RAP content (by 25% and more). Based on the energy ratio parameters, it was found that the resistance to moisture-induced damage increased with an increase in RAP content for both PG 64-22 and PG 76-28 asphalt binders and all types of aggregates, specifically at higher RAP contents. Furthermore, it was found that the higher the total SFE of the aggregates, the lower the energy ratio parameter values. Therefore, a high total SFE component of aggregate may result in a high moisture-induced damage potential in the mix. The results presented herein are expected to be helpful in mechanistically assessing the moisture-induced damage potential of asphalt mixes produced with polymer-modified and non-polymer-modified asphalt binders, which contain RAP.

Keywords: surface free energy, asphalt mix, reclaimed asphalt pavement, moisture-induced damage, work of adhesion, work of debonding, wettability

5.1 Introduction

Hot Mix Asphalt (HMA) is the most widely used paving material in the U.S. Nationwide, more than 90 percent of pavements are paved with asphalt (NECEPT, 2010). Each year, over 550 million tons of HMA are produced and used for construction of flexible pavements. Increasing environmental awareness, rising oil prices and scarcity of high quality aggregates in many areas spur development of methods and technologies for reduced use of virgin asphalt binder and aggregates, and increased use of recycled or reclaimed materials. Therefore, over the past two decades many transportation agencies, asphalt producers and pavement construction companies have taken major initiatives to implement *green paving technologies* (NAPA, 2011). Use of Reclaimed Asphalt Pavement (RAP) in asphalt production is an important element of such initiatives. Based on a report published by the National Asphalt Pavement Association (NAPA), asphalt is being recycled and reused at a rate of over 99 percent (NAPA, 2011). Recycling of asphalt pavements and asphalt shingles conserved 20.5 million barrels of asphalt binder in 2010 (NAPA, 2011). Using RAP in HMA not only benefits the economy and environment by cutting costs and preserving natural resources, but also increases the rut resistance of the mix (Al-Qadi et al., 2012; Huang et al., 2004; Mohammead et al., 2003; McDaniel and Shah, 2003b). Concerns over premature pavement distresses due to increased RAP amounts in HMA limit the use of RAP. For example McDaniel et al. (2000) reported that incorporating more than 20% RAP in asphalt mixes resulted in reduced fatigue life when compared with that of the virgin mix. In a similar study, Shu et al. (2008) concluded that using RAP in HMA reduces the fatigue life of asphalt mixes. In addition to rut and fatigue, moisture-induced damage is another important distress in HMA pavements, including those containing RAP.

Moisture-induced damage is defined as the loss of asphalt binder's tensile strength (cohesive failure) or bonding failure at the asphalt binder-aggregate interface (adhesive failure), due to the presence of moisture (Howson et al., 2009). Retained indirect Tensile Strength Ratio (TSR) and Hamburg Wheel Tracking (HWT) tests, in accordance with AASHTO T 283 (AASHTO, 2011) and AASHTO T 324 (AASHTO, 2011), respectively, are used to evaluate moisture-induced damage potential of asphalt mixes. However, it is reported that a TSR test may fail to correlate with field observations due to its empirical nature and lack of mechanistic base (Bhasin et al., 2007). Similarly, HWT test does not directly address the "failure mechanism" that governs the stripping in asphalt pavements. Therefore, using a mechanistic approach to characterize the moisture-induced damage of asphalt mixes containing RAP is important to stripping evaluation.

Recently, the Surface Free Energy (SFE) method has been used successfully, to evaluate the moisture-induced damage potential of asphalt binder-aggregate systems (Ghabchi et al., 2013a; Arabani et al., 2012; Hossain et al., 2009; Howson et al., 2009; Wasiuddin et al., 2008; Wasiuddin et al., 2007; Bhasin et al., 2007; Lytton et al., 2005; Cheng et al., 2002). The SFE method is a mechanistic approach which directly addresses the adhesion and debonding of the asphalt binder and aggregates in presence of moisture. Many studies have applied the SFE approach to assess moisture susceptibility of different asphalt binders, Warm Mix Asphalt (WMA) additives, and anti-stripping agents in contact with different types of aggregates (Ghabchi et al., 2013a; Arabani et al., 2012; Hossain et al., 2009; Howson et al., 2009; Wasiuddin et al., 2008; Wasiuddin et al., 2007; Bhasin et al., 2007; Lytton et al., 2005; Cheng et al., 2002). However, to the authors' knowledge, no study was found in the literature to evaluate the moisture-induced damage potential of mixes containing RAP using the SFE method. RAP contains aged and stiff binder, which may alter the chemical and surface properties of a virgin binder when mixed in different proportions. Hence, the SFE components of an asphalt mix prepared with different amounts of RAP are expected to be different than those for a virgin mix. This in turn may affect the adhesion potential and bond strength of the asphalt binder-aggregate system. Therefore, the current study was undertaken to evaluate the effect of virgin asphalt binders with different Performance Grades (PG), different amounts of RAP, and different types of aggregates on the moisture-induced damage potential of asphalt mixes. The SFE measurements consisted of testing two types of asphalt binders, namely PG 64-22 and PG 76-28, each mixed with different amounts of RAP binder, namely 0%, 10%, 25%, and 40%, using a DCA analyzer. The aggregates included in this study consisted of limestone, rhyolite , sandstone, granite, gravel, and basalt. The SFE components of the limestone and rhyolite aggregates were measured using a USD device, while the SFE components of sandstone, granite, gravel, and basalt aggregates were obtained from the literature. Consequently, a wide range of binder types, RAP amounts, and aggregates were covered in this study. Finally, wettability, work of adhesion, work of debonding, and energy ratio parameters were estimated to mechanistically discuss the effect of binder type, RAP amount and aggregate types on the moisture-induced damage potential of asphalt mixes containing RAP. The results presented herein are expected to be helpful in evaluation of aggregates-asphalt binder-RAP combinations during the material selection for asphalt mixes, in order to minimize the possibility of moisture damage in pavement.

5.2 Objectives

This study aims to evaluate the effect of asphalt binder type, RAP binder, and aggregate type on the moisture-induced damage potential of mixes using the SFE approach. The specific objectives of this study were to:

1. Determine the SFE components of PG 64-22 and PG 76-28 asphalt binders with and without addition of different amounts of RAP binder (i.e., 0%, 10%, 25%, and 40%), using a Dynamic Contact Angle (DCA) analyzer.
2. Determine the SFE components of different types of aggregates using a Universal Sorption Device (USD).
3. Evaluate the asphalt binder’s coating quality with and without addition of RAP binder on different types of aggregates using wettability parameter (spreading coefficient).
4. Evaluate moisture-induced damage potential of mixes containing RAP with different types of aggregates and asphalt binders based on the energy ratio parameters estimated based on wettability, work of adhesion, and work of debonding.

5.3 Background on Surface Free Energy

5.3.1 Surface Free Energy Components

The SFE of a solid (or liquid) is defined as the work required for increasing its surface by a unit area under vacuum (van Oss et al., 1988). Van Oss et al. (1988) proposed a theory (Good-van Oss- Chaudhury theory) which suggests three components of SFE. Based on the Good-van Oss- Chaudhury theory, the total SFE can be expressed in the form of: (i) a monopolar acidic component (Γ^{+}), (ii) a monopolar basic component (Γ^{-}), and (ii) an apolar or Lifshitz-van der Waals component (Γ^{LW}). Also, the total SFE component (Γ^{total}) can be expressed in terms of a Lifshitz-van der Waals (Γ^{LW}) and an acid-base (Γ^{AB}) component, shown in Equation 5.1 and Equation 5.2, respectively.

$$\Gamma^{Total} = \Gamma^{LW} + \Gamma^{AB} \tag{5.1}$$

where,

$$\Gamma^{AB} = 2\sqrt{\Gamma^{+}\Gamma^{-}} \tag{5.2}$$

5.3.2 Surface Free Energy of Asphalt Binder

Thomas Young in 1805, described the occurrence of the wetting and spreading of a liquid over a surface to be directly related to the interaction between the cohesive and adhesive forces. Therefore, contact angle of a liquid over a solid surface determines the wettability of the liquid over a surface. This contact angle is known to be a function of the surface energies of the system. Thus, the SFE components of an asphalt binder can be determined by measuring the contact angles (θ) between the asphalt binder and three different solvents (one apolar, one monopolar and one bipolar solvent) using the Wilhelmy plate test method (Ghabchi et al., 2013b; Ghabchi et al., 2013c; Arabani et al., 2012; Arabani and Hamedi., 2011; Hossain et al., 2009; Howson et al., 2009; Wasiuddin et al., 2008; Wasiuddin et al., 2007). Then Equation 5.3 was solved for three contact angles measured with three solvents (Good and Van Oss, 1991) to determine the three unknown SFE components of the asphalt binder.

$$\Gamma_L(1+\cos\theta) = 2\left(\sqrt{\Gamma_A^{LW}\Gamma_L^{LW}} + \sqrt{\Gamma_A^{+}\Gamma_L^{-}} + \sqrt{\Gamma_A^{-}\Gamma_L^{+}}\right) \tag{5.3}$$

where *A* and *L* subscripts represent the energy parameters associated with asphalt binder and probe liquid, respectively. A description of the method used for conducting the Wilhelmy plate test is provided later in this study.

5.3.3 Surface Free Energy of Aggregates

Adsorption isotherms developed using three different probe vapors are used to determine the SFE components of aggregates using a USD, applying a method suggested by Bhasin et al. (2007). For this purpose Equations 5.4 and 5.5 were used to calculate the spreading pressure and the work of adhesion between probe vapor and aggregates, respectively.

$$\pi_e = \frac{RT}{MA}\int_0^{p_n} \frac{n}{p}\,dp \tag{5.4}$$

$$W_{SV} = \pi_e + 2\Gamma_V^{Total} \tag{5.5}$$

where π_e = equilibrium spreading pressure of the probe vapor on aggregate surface; R = universal gas constant; T = test temperature; M = molecular weight of probe vapor; n = adsorbed mass of probe vapor per unit mass of the aggregate at probe vapor pressure of p; A = specific surface area of aggregate; W_{SV} = work of adhesion between aggregate surface and probe vapor (subscript *V*); and Γ_V^{Total} = total surface free energy of probe vapor. After calculation of W_{SV} for each probe vapor, by Equations 5.5, the SFE components of the aggregate are determined by solving the adhesion equation from the adsorption isotherm results for three solvents. A detailed discussion on the measurement of the SFE components of aggregates, using a USD, is presented subsequently in this chapter.

5.4 Performance Parameters Estimated Using SFE

5.4.1 Wettability

Wetting is a liquid's spreading and maintaining its contact over a surface, which may include penetration into porous medium (Berg, 1993). Therefore, the potential of an asphalt binder to coat an aggregate can be studied by evaluation of wettability. Wettability is determined by the liquid contact angle measured on the surface of the solid phase. The contact angle is formed based on a balance between the adhesive and cohesive forces (Berg, 1993). The spreading coefficient, as a quantitative measure of wettability, is defined as the reduction in SFE during loss of the bare solid surface and formation of a new solid-liquid and liquid-vapor interface (Zettlemoyer, 1968). The spreading coefficient is given in Equation 5.6 (Wasiuddin et al., 2007).

$$S_{A/S} = \Gamma_S - \Gamma_{AS} - \Gamma_A \tag{5.6}$$

where $S_{A/S}$ = spreading coefficient of asphalt binder (subscript *A*) over the aggregate or stone (subscript *S*); Γ_S = total surface free energy of aggregate, Γ_{AS} = interfacial energy between asphalt binder and aggregate, and Γ_A = total surface free energy of asphalt binder.

5.4.2 Work of Adhesion

The free energy required to create two interfaces from one interface, consisting of two different phases in contact, is defined as the work of adhesion. The work of adhesion between an asphalt binder and aggregate can be determined from Equation 5.7.

$$W_{AS} = 2\sqrt{\Gamma_A^{LW}\Gamma_S^{LW}} + 2\sqrt{\Gamma_A^{+}\Gamma_S^{-}} + 2\sqrt{\Gamma_A^{-}\Gamma_S^{+}} \tag{5.7}$$

where W_{AS} = the work of adhesion between an asphalt binder (subscript *A*) and aggregate or stone (subscript *S*). Therefore, the higher the bond strength between the asphalt binder and aggregate in dry condition, the higher the W_{AS} (Bhasin et al., 2007).

5.4.3 Work of Debonding

The energy released as a result of spontaneous separation of the asphalt binder from aggregate surface in the presence of water is called the work of debonding. The work of debonding is determined from Equation 5.8.

$$W_{ASW}^{wet} = \Gamma_{AW} + \Gamma_{SW} - \Gamma_{AS} \tag{5.8}$$

where W_{ASW}^{wet} = the work of debonding of asphalt binder from aggregate in presence of water; Γ_{AW} = interfacial energy between asphalt binder and water (subscript *W*); Γ_{SW} = interfacial energy between aggregate and water; and Γ_{AS} = interfacial energy between asphalt binder and aggregate. Interfacial energy is defined as the energy equal to the surface tension at an interface. Interfacial energy between two materials is determined from Equation 5.9 (Bhasin et al., 2007).

$$\Gamma_{ij} = \Gamma_i + \Gamma_j - 2\sqrt{\Gamma_i^{LW}\Gamma_j^{LW}} - 2\sqrt{\Gamma_i^{+}\Gamma_j^{-}} - 2\sqrt{\Gamma_i^{-}\Gamma_j^{+}} \tag{5.9}$$

where Γ_{ij} = interfacial energy between materials *i* and *j*; Γ_i and Γ_j = total surface free energies of materials *i* and *j*, respectively. Since the spontaneous debonding of the asphalt binder from aggregate due to the presence of water releases energy, the W_{ASW}^{wet} is a negative value. This will cause the total energy level of the system to reduce, a thermodynamically favorable mechanism. It can be concluded that the greater the $|W_{ASW}^{wet}|$, the higher the debonding potential of the asphalt binder from the aggregate in presence of moisture (Bhasin et al., 2007).

5.4.4 Energy Ratio Parameters

To this end, based on the wettability, the work of adhesion and the work of debonding determined from Equations 5.6, 5.7 and 5.8, respectively, it is evident that stripping of the asphalt binder from the aggregate depends on these parameters. The resistance to moisture-induced damage increases with an increase in the wettability ($S_{A/S}$) and work of adhesion (W_{AS}), and decreases with an increase in the magnitude of the work of debonding ($|W_{ASW}^{wet}|$). Therefore, Bhasin et al. (2007) suggested the use of W_{AS} and $|W_{ASW}^{wet}|$ into a single parameter, namely energy ratio (ER_1), shown in Equation 5.10.

$$ER_1 = \left|\frac{W_{AS}}{W_{ASW}^{wet}}\right| \tag{5.10}$$

From Equation 10, it is evident that ER_1 increases with an increase in W_{AS} and decreases with an increase in $|W_{ASW}^{wet}|$. However, Equation 10 does not include the role of wettability as a measure of coating quality and forming bond between aggregate and asphalt binder. Therefore, a second parameter, ER_2, which considers the wettability parameter, was suggested by Bhasin et al. (2007), as shown in Equation 11.

$$ER_2 = \left|\frac{S_{A/S}}{W_{ASW}^{wet}}\right| \tag{5.11}$$

The wettability parameter, the spreading coefficient ($S_{A/S}$), can be estimated from Equation 5.6. The ER_1 and ER_2 were used in this study to evaluate the moisture-induced damage potential of different mixes containing RAP with varieties of aggregates and asphalt binders.

5.5 Materials

5.5.1 Asphalt Binders and Aggregates

The PG 64-22 and PG 76-28 asphalt binders used in this study were obtained from the Valero refinery located in Muskogee, OK. Both of these asphalt binders are commonly used in Oklahoma for construction of pavement. Different types of aggregates tested in this study were collected from different quarries in Oklahoma. The collected aggregates are among the common aggregates used in Oklahoma for production of asphalt mixes.

5.5.2 RAP Binder

The chemicals used in binder extraction methods from RAP may significantly alter the asphalt binders' chemical and surface properties, and may introduce error in measured SFE parameters of the extracted RAP binder. Therefore, in order to produce RAP binder, the Rolling Thin-Film Oven (RTFO) and Pressure Aging Vessel (PAV) methods, in accordance with AASHTO T 240 (AASHTO, 2013) and AASHTO R 28 (AASHTO, 2012) were used, respectively, to long-term age the PG 64-22 binder. The binder prepared using this method was stored in small canisters for further testing.

The asphalt binder aged according to this procedure represents a long-term aging equivalent to seven to ten years of in-service aging (Bahia and Anderson, 1995). This method is used by many researchers to produce simulated RAP binder in the laboratory. It has been reported that the PAV method can simulate both chemical and physical changes of asphalt binders during its service life (Galal et al., 2000). Since PG 64-22 binders are used in a majority of pavements in Oklahoma, a PG 64-22 binder was selected for aging and was used for RAP binder production.

5.6 Methodology

A summary of the work flow and techniques used for evaluation of the moisture-induced damage potential of asphalt binders mixed with RAP and aggregates is presented in Figure 5.1.

5.6.1 Preparation of Asphalt Binder for SFE Testing

The asphalt binder mixes used for the SFE testing consisted of two sets. The following proportions (by weight) were used for each type of virgin binder (PG 64-28 and PG 76-28): (i) 100% virgin binder, (ii) 10% RAP binder + 90% virgin binder, (iii) 25% RAP binder + 75% virgin binder, and (iv) 40% RAP binder + 60% virgin binder. The selection of these proportions was based on the RAP contents commonly used in different types of pavements in Oklahoma. Thus, a total of 8 different asphalt binders and RAP combinations (four for PG 64-22 and four for PG 76-28) were prepared. A summary of the asphalt binder mixes prepared for this study is shown in Table 5.1.

5.6.2 Measurement of Surface Free Energy Components of Asphalt Binders

The SFE components of each asphalt binder were determined by measuring its contact angles with different solvents using the DCA. For this purpose, three different solvents with known SFE components were used, namely, water (bi-polar solvent), glycerin (apolar solvent) and formamide (mono-polar solvent). To prepare DCA samples, standard cover glasses having 25 mm width by 50 mm length were coated with asphalt binder. For this purpose, approximately 100 grams of asphalt binder mix in a canister was kept in an oven at 165°C for two hours to liquefy the binder, and gently mixed a couple of times to ensure the consistency and proper mixing of the RAP binder and virgin binder. Then the glass plate surface was "flamed" by passing it through an oxygen flame for at least three times in less than 5 seconds, in order to obtain a moisture-free and clean surface. Thereafter, the glass plate was carefully dipped at least 20 mm in the asphalt binder mix in the oven, and moved back and forth three times in 5 seconds to ensure proper asphalt coating on the glass plate. The coated plate was then kept in the oven on a stand in vertical position for 2 minutes to drip down the excess binder and to obtain a smooth surface. Finally, the sample was cured for 24 hours in a desiccator before conducting the test. Table 5.1 presents the test matrix for the DCA tests conducted on asphalt binder mixes in this study. As evident from Table 5.1, a total of 120 (2 Binders x 4 RAP percentages x 5 replicates x 3 solvents) asphalt binder samples were tested using the DCA analyzer. After measuring the contact angles, the SFE components of asphalt binder mixes were determined by using Equation 5.3 for each asphalt binder and solvent.

5.6.3 Measurement of Surface Free Energy Components of Aggregates

The SFE components of different aggregates, collected from quarries in Oklahoma, were determined by USD testing. Furthermore, the SFE components of a number of other aggregates were adopted from the literature (Bhasin and Little, 2007; Buddhala et al., 2011). Surface free energies of aggregates were determined from vapor sorption isotherms, i.e. the amount of vapor adsorbed or desorbed on the solid surface at a fixed temperature and partial pressure (Bhasin and Little, 2007). These aggregates consisted of limestone and rhyolite tested in this study, and sandstone, granite, gravel, and basalt, adopted from the literature (Buddhala et al., 2011; Bhasin and Little, 2007). The methodology used by Bhasin and Little (2007) was applied for determination of the SFE components. In this study, three probe vapors, namely, water (bi-polar vapor), methyl propyl ketone or MPK (mono-polar vapor) and n-hexane (apolar vapor) were used to determine adsorption isotherms. For aggregate sample preparation selected aggregates were oven dried at 60°C for 24 hours and cooled down to room temperature. Then, they were crushed in the laboratory and the size fractions with particles larger than 2.36 mm (retaining on a No. 8 sieve) and smaller than 4.75 mm (passing to a No. 4 sieve) were selected. Thereafter, the selected fractions of aggregates were washed several times with distilled water to obtain dust-free clean surfaces. Then, they were oven-dried at 120°C for 12 hours and allowed to cool to room temperature in a desiccator sealed with silica gel before testing. About 20 grams of the prepared aggregate sample was subjected to probe vapors (water, MPK, and n-hexane) in a USD. The recorded changes in sample weight at different relative humidity or pressures were measured using a built-in Cahn D-101 microbalance. A data acquisition system recorded the sample weight, temperature, and relative humidity or pressures in a data file, at user-defined intervals. The collected data was then used for developing the sorption isotherms for each aggregate tested with each probe vapor. Then, Equations 5.4, 5.5 and 5.7 in conjunction with the developed sorption isotherms were used to determine the SFE components of each aggregate (Bhasin and Little, 2007). Table 5.2 presents the test matrix of the USD tests conducted on aggregates tested in this study, and those adopted from the literature. According to Table 5.2, a total of 18 aggregate (2 aggregates x 3 replicates x 3 solvents) samples were tested in the USD device.

5.7 Results and Discussion

5.7.1 SFE Components of Asphalt Binders

The SFE components of asphalt binder play an important role in defining its ability to adhere to aggregates. The SFE components of asphalt binders, along with those obtained from aggregates, are needed for determination of the energy ratio parameters. These parameters are used to evaluate the moisture-induced damage potential of the asphalt binder-aggregate systems. The SFE components of PG 64-22 and PG 76-28 asphalt binders mixed with different amounts of RAP binder are presented in Figure 5.2-a and 5.2-b, respectively. The non-polar SFE component (Γ^{LW}) values of PG 76-28 binder were found to be approximately 2 mJ/m^2 higher than those of PG 64-22 binder for different percentages of RAP binder (Figure 5.2). It is important to note that the PG 76-28 is a polymer-modified asphalt binder; higher Γ^{LW} values for this type of asphalt binder may be attributed to polymer modification. The Γ^{LW} component of both PG 64-22 and PG 76-28 asphalt binders reduced with the addition of 10% and 25% RAP binder. This component increased with the addition of higher amounts of RAP (i.e., 40%). Nonpolar molecules in an asphalt binder are known to work as a matrix for polar molecules. This matrix arrangement is responsible for the elastic properties of asphalt binders (Jones and Kennedy, 1991). Thus, an increase in the Γ^{LW} component may result in an increase in work of adhesion, which is an indicator of a better binder-aggregate bond in dry condition.

Furthermore, an increase in RAP content increased the acid SFE component (Γ^+) and, in general, decreased the base SFE component (Γ^-) of both PG 64-22 and PG 76-28 asphalt binders (Figure 5.2). This increase in Γ^+ and reduction in Γ^- values with increasing RAP amounts are more pronounced for the PG 76-28 binder. Highly acidic binders are known not to produce a strong bond with acidic aggregates. This is due to surface chemistry of Lewis acid and base which does not favor adhesion in this case (Arabani and Hamedi, 2011). It is known that significantly high polar components (Γ^{+} and Γ^{-}) in asphalt binder may result in moisture-induced damage potential, fatigue cracking in thick pavement layers and rutting (Jones and Kennedy, 1991). However, the moisture susceptibility of a mix should be evaluated based on parameters which include wettability, adhesion and debonding properties of an asphalt binder-aggregate system, which is introduced in this study.

5.7.2 SFE Components of Aggregates

The SFE components of the aggregates tested (limestone and rhyolite) and those adopted from literature (sandstone, granite, gravel, and basalt) are presented in Table 5.3. The SFE components of the aggregates tested herein (limestone and rhyolite) were found to be in the range of those reported in the literature (Howson et al., 2009). The gravel and basalt aggregates have the highest and the lowest total SFE components (356.7 and 72.1 mJ/m^2), respectively. Comparatively, gravel and granite were found to have the highest acidic SFE components (24.1 and 23.0 mJ/m^2, respectively) among the other aggregates. A comparison between the asphalt binder and aggregate SFE components (Table 5.3 and Figure 5.2) reveals that the aggregates have relatively higher acid, base and acid-base components, compared to those of asphalt binders. For example, the acid-base SFE components of PG 64-22 and PG 76-28 binders, mixed with different amounts of RAP, are in the range of 1.09 to 1.84 mJ/m^2. These ranges are considerably lower those measured for aggregates (19.8 to 299.2 mJ/m^2). Therefore, water, with a polar molecule, has a higher wetting potential on aggregates than that of asphalt binders (Tarrer and Wagh, 1991).

5.7.3 Wettability

The wettability is defined as the tendency of a liquid to wet a solid surface. Asphalt binder and aggregates are hydrophobic and hydrophilic materials, respectively (Tarrer and Wagh, 1991). Hence, wetting a hydrophilic surface (aggregate) with a hydrophobic material (asphalt binder) is not easy. Therefore, it is important to study the wettability of asphalt binder over the aggregate surface. A higher wettability of asphalt binder over aggregate surface will help the asphalt binder to easily coat the aggregate. The wettability can be expressed by the released energy as the asphalt binder flows over the aggregate to coat it (Wasiuddin et al., 2008; Zettlemoyer, 1968). The released energy, namely, spreading coefficient of the asphalt binder over the aggregates ($S_{A/S}$), can be used to quantify the wettability. In this study, the spreading coefficients of PG 64-22 and PG 76-28 binders mixed with different amounts of RAP binder (0%, 10%, 25% and 40%) over different types of aggregates (limestone, rhyolite, sandstone, granite, gravel and basalt) were determined using Equation 5.6.

5.7.4 Effect of Asphalt Binder Type and RAP Content on Wettability

The spreading coefficients of PG 64-22 and PG 76-28 asphalt binders with different percentages of RAP binder and aggregates are presented in Figure 5.3. It can be seen from Figure 5.3 that the spreading coefficient of PG 64-22 and PG 76-28 asphalt binders with different types of aggregates did not change significantly up to 10% RAP content. However, the spreading coefficients for both PG 64-22 and PG 76-28 asphalt binders increased at higher RAP contents (25% and 40%) for all types of aggregates, except for granite. Therefore, use of higher amounts of RAP will be beneficial for a better aggregate coating with binder, contributing to a better bond. Furthermore, it was observed that the PG 64-22 asphalt binder without RAP and 10% RAP had higher spreading coefficients than those of the PG 76-28 asphalt binder for almost all types of aggregates. For example, spreading coefficients of the PG 64-22 asphalt binder with 0% and 10% RAP was found to be 104.4 and 104.7 mJ/m^2 on gravel aggregate, while they were 92.9 and 92.1 mJ/m^2 for the PG 76-28 asphalt binder (approximately 11% reduction). This means that, when mixed with low amounts of RAP (0% and 10%), a PG 64-22 binder may have a higher tendency to coat the aggregates than a PG 76-28 binder. However, when the amount of RAP increased (25% and 40%), both PG 64-22 and PG 76-28 binders showed similar values of the spreading coefficients with different aggregates, as shown in Figure 5.3. Thus, it can be concluded that the wettability improves and becomes independent of binder type at higher RAP contents (25% and 40%).

5.7.5 Work of Adhesion

The work of adhesion (W_{AS}) under dry condition (without effect of moisture) was determined to evaluate the bond strength between aggregate and asphalt binder (Wasiuddin et al., 2008). The work of adhesion is the energy required for separation of an asphalt binder from the aggregate-binder interface (Bhasin et al., 2007). Therefore, a higher W_{AS} value is desirable for a stronger bond between asphalt binder and aggregate under dry condition. However, the work of adhesion alone cannot rank asphalt mixes based on their moisture-induced damage potentials. This parameter is required to estimate the energy ratio parameters by which the moisture susceptibility is evaluated.

5.7.5.1 Effect of Asphalt Binder Type and RAP Content on Work of Adhesion

Figures 5.4-a and 5.4-b present the work of adhesion between the aggregates and the PG 64-22 and PG 76-28 asphalt binders, respectively, with different amounts of RAP. In general, it can be seen that the work of adhesion for the PG 64-22 and PG 76-28 asphalt binders increases significantly at higher RAP contents (25% and 40%), for all types of aggregates except for granite, when the PG 64-22 binder was used. However, the increase in the work of adhesion was not significant at a lower percentage of RAP (i.e., 10%). Therefore, it can be concluded that addition of low amounts of RAP (10%) may not affect the work of adhesion, while use of higher RAP amounts (25% and more) was found to be beneficial to improve aggregate-asphalt binder adhesion. These observations are consistent with the results reported based on the wettability of asphalt binders for different amounts of RAP binder, discussed in the previous section.

While comparing work of adhesion for both the asphalt binders, it was found that the PG 64-22 asphalt binder without RAP and 10% RAP showed a higher work of adhesion than those of PG 76-28 asphalt binder, for all types of aggregates except for granite. For example, the work of adhesion for the PG 64-22 asphalt binder for gravel aggregates, with 0% and 10% RAP was found to be 128.5 and 128.4 mJ/m^2, while it was found to be 121.0 and 118.0 mJ/m^2 for PG 76-28 asphalt binder on the same aggregate (approximately 6% and 8% reduction, respectively). This means that at low amounts of RAP (0% and 10%) a PG 64-22 binder has higher bonding strength with aggregates than that of the PG 76-28 binder. A similar trend was found for wettability as well. However, it was observed that with further increase in RAP amounts (25% and

40%) both the PG 64-22 and PG 76-28 binders showed close work of adhesion values with different aggregates, indicating that adhesion improves at higher RAP contents (25% and 40%) regardless of the type of virgin asphalt binder used. The gravel and basalt aggregates had the highest and the lowest work of adhesion with both binder types (PG 64-22 and PG 76-28, respectively. This means that the gravel and basalt aggregates have the highest and the lowest dry bond strength by asphalt binder, respectively. Furthermore, it was observed that using RAP with PG 64-22 and PG 76-28 binders on the granite aggregate may not significantly improve the work of adhesion (Figure 5.4).

5.7.6 Work of Debonding

The work of debonding (W_{ASW}^{wet}) is a measure of aggregate-asphalt binder separation potential in presence of moisture. As a result of stripping, a reduction in the free energy of the system occurs when asphalt binder, in presence of water, separates from the aggregate-asphalt binder interface. This reduction in free energy of the system is called the work of debonding and is determined by using Equation 5.8. Therefore, a higher magnitude of the work of debonding (negative number) implies that the occurrence of stripping is thermodynamically more favorable (Bhasin and little, 2007). Therefore, it is essential to evaluate the work of debonding for different combinations of virgin asphalt binder, RAP binder, and aggregates to fully characterize the moisture damage mechanism. However, it should be noted that the work of debonding alone cannot rank moisture damage potentials of asphalt mixture. This parameter is required for estimation of energy ratio parameter, which is crucial in evaluating moisture damage potential.

5.7.6.1 Effect of Asphalt Binder Type and RAP Content on Work of Debonding

Figures 5.5-a and 5.5-b present the work of debonding between the aggregates and the PG 64-22 and PG 76-28 asphalt binders, respectively, with different amounts of RAP. It is evident that the magnitude of the work of debonding for both PG 64-22 and PG 76-28 asphalt binders with different aggregates (except for granite) decrease with an increase in the amount of RAP binder. The reduction in the work of debonding is more pronounced at higher RAP amounts (25% and 40%), while it is not as significant for lower RAP content (10%). Therefore, it can be concluded that the use of higher RAP amounts (25% and more) may lead to less stripping. Furthermore, according to Figures 5.5-a and 5.5-b, comparing the work of debonding of the virgin PG 64-22 and PG 76-28 asphalt binders (0% RAP) reveals that the PG 64-22 has a lower magnitude of the work of debonding for all types of aggregates than that of PG 76-28 asphalt binder. For example, the work of debonding of rhyolite aggregate with virgin PG 64-22 asphalt binder was -176.2 mJ/m^2, which is approximately 7% lower than that of PG 76-28 (-187.6 mJ/m^2). A similar trend also exists for other types of aggregates. As discussed earlier, the PG 76-28 is a polymer-modified binder; and the observations based on the work of adhesion and the work of debonding suggest moisture susceptibility concerns for this polymer-modified asphalt binder when it is compared with non-polymer-modified type. Therefore, more study on the effect of using polymer-modified asphalt binder in asphalt mix on its moisture susceptibility is recommended.

Furthermore, it is evident that the work of debonding changes significantly with the change in aggregate type for both PG 64-22 and PG 76-28 asphalt binders (Figure 5.5). However, the trends of variation in the work of debonding with aggregate type and

RAP content when the PG 64-22 and PG 76-28 asphalt binders were used were found to be similar. Furthermore, it was observed that the work of debonding for granite aggregate with PG 64-22 and PG 76-28 asphalt binders remain unchanged at different RAP contents. In a similar way, basalt aggregate shows very low reduction in work of debonding with an increase in RAP content when PG 64-22 and PG 76-28 asphalt binders were used. However, the work of debonding found for gravel, sandstone, rhyolite and limestone aggregates show similar sensitivities to change in RAP content for each binder type (PG 64-22 and PG 76-28). Therefore, based on the work of debonding values, it can be concluded that when using granite and basalt aggregates it is not expected to gain benefits from using RAP in terms of reducing the work of debonding. However, using gravel, sandstone, rhyolite and limestone aggregates was found to maximize the reduction in the work of debonding as a result of adding RAP. It is important to note that this parameter (work of debonding) should be considered in conjunction with the wettability and work of adhesion to evaluate moisture-induced damage potential, as discussed next.

5.7.7 Energy Ratio Parameters

5.7.7.1 Effect of Asphalt Binder Type and RAP Content on ER_1 and ER_2

The ER_1 and ER_2 values were determined for different combinations of the asphalt binder types (PG 64-22 and PG 76-28), RAP contents (0%, 10%, 25%, and 40%), and different aggregate types (limestone, rhyolite, sandstone, granite, gravel, and basalt) and are presented in Figures 5.6 and 5.7. It is evident from Figures 5.6 and 5.7 that, for both PG 64-22 and PG 76-28 asphalt binders, the ER_1 and ER_2 values increase with an increase in RAP content for all types of aggregates. The addition of low amounts of RAP (10%) does not seem to influence the ER_1 and ER_2 values, while the addition of higher amounts (25% and 40%) of RAP significantly improved the ER_1 and ER_2 values for both types of asphalt binders (Figures 5.6 and 5.7). This means that, in general, the addition of higher percentages of RAP improves the resistance to moisture-induced damage for both PG 64-22 and PG 76-28 asphalt binders.

Furthermore, it was observed that the PG 64-22 asphalt binder without RAP and 10% RAP generally showed higher ER_1 and ER_2 values than those obtained for the PG 76-28 asphalt binder for all types of aggregates, except for granite. For example, the ER_1 values of a PG 64-22 asphalt binder and basalt aggregate with 0% and 10% RAP were found to be 2.70 and 2.78, respectively, while they were 2.27 and 2.28 mJ/m^2, respectively, for the PG 76-28 asphalt binder (approximately 16% and 18% reduction with respect to the PG 64-22 binder). Similarly, the ER_2 values of a PG 64-22 asphalt binder and basalt aggregate with 0% and 10% RAP were found to be 1.87 and 1.94, while they were found to be 1.43 and 1.47 mJ/m^2, respectively, for the PG 76-28 asphalt binder (approximately 25% and 24% reduction). This means that at low amounts of RAP (0% and 10%) the PG 64-22 binder exhibits higher ER_1 and ER_2 values for different types of aggregates than those of the PG 76-28 binders. This can be attributed to the effect of using a polymer-modified asphalt binder (PG 76-28), which increases the moisture induced damage potential compared to non-modified asphalt binder (PG 64-22). However, it was observed that with further increase to the RAP amounts (25% and 40%) both PG64-22 and PG 76-28 asphalt binders with different aggregates, showed similar ER_1 and ER_2 values. Thus, at higher RAP contents type of binder does not have any significant effect on the moisture-induced damage potential.

5.7.7.2 *Effect of Aggregate Type on ER_1 and ER_1*

Figures 5.6 and 5.7 show that the ER_1 and ER_2 values highly depend on the aggregate type. It was observed that the basalt aggregate exhibited the highest ER_1 and ER_2 values, ranging from 2.27 to 3.31, and 1.43 to 2.27, respectively, for different RAP amounts for both PG 64-22 and PG 76-28 asphalt binders. This is due to very low work of debonding found for basalt, compared to its work of adhesion and wettability. Comparatively, the gravel aggregate showed the lowest ER_1 and ER_2 values ranging from 0.55 to 0.73 and 0.41 to 0.59, respectively, for different RAP amounts and asphalt binders. This shows that the mixes with different amounts of RAP in which the basalt and gravel aggregates are used have the highest and the lowest resistance to moisture-induced damage, respectively.

It is worth noting that the ER_1 and ER_2 values were less sensitive to the change in RAP amount for granite aggregates with both PG 64-22 and PG 76-28 asphalt binders. For examples, the ER_1 values for granite aggregate range between 1.91 and 1.97 with the PG 64-22 binder and between 1.96 and 2.25, for the PG 76-28 binder. A similar trend in variation was observed for ER_2. From the above discussion, it may be concluded that ER_1 and ER_2 exhibited similar and consistent trend of variation with change in asphalt binder type, RAP content and aggregate type. Therefore, both of these parameters may be used for the evaluation of moisture-induced damage potential of mixes. Based on the ER_1 and ER_2 values, the aggregates combined with PG 64-22 and PG 76-28 asphalt binders used in this study were ranked based on their resistance to moisture-induced damage, from the highest to the lowest, as: basalt, granite, rhyolite, limestone, sandstone, and gravel. It is interesting to note that the following order was found when the aggregates were ranked with respect to their total SFE from the lowest to the highest, as: basalt, rhyolite, granite, limestone, sandstone, and gravel. This ranking is almost the same as that found from ER_1 and ER_2 values. This is expected since the non-polar components of the aggregates are very low and the most effective SFE component contributing to the total SFE of aggregates is the acid and base components. Therefore, an increase in acid and base components of the aggregates will lead to a high total SFE. As discussed earlier, high acid and base component in aggregates are known to increase moisture-induced damage potential. However, use of ER_1 and ER_2 will produce more accurate evaluation of moisture damage by considering the effects of both aggregate and asphalt binder. The ranking provided herein is expected to be helpful for pavement engineers in selecting aggregates for asphalt mixes containing RAP, so as to minimize the moisture-induced damage potential of the resulting mix.

5.8 Conclusions and Recommendations

The present study evaluated the effects of two types of asphalt binders, namely, PG 64-22 and PG 76-28, four RAP contents, namely, 0%, 10%, 25% and 40%, and six different types of aggregates, namely, limestone, rhyolite, sandstone, granite, gravel, and basalt, on the wettability and moisture-induced damage potential of associated asphalt mixes, applying the Surface Free Energy (SFE) approach. For this purpose, the SFE components of abovementioned asphalt binders and aggregates were used to estimate the wettability, work of adhesion and work of debonding for different combinations of asphalt binders and aggregates. Thereafter, two different energy ratio parameters, namely ER_1 and ER_2, calculated based on wettability, work of adhesion and work of debonding were used to evaluate moisture induced damage potential of the same mixes. The following conclusions may be drawn from the results and discussions presented in the preceding sections.

1. The non-polar SFE component of PG 76-28 asphalt binder was found to be higher than that of PG 64-22 for all RAP contents. This SFE component increases with addition of RAP in higher amounts (i.e., 25% and 40) for both PG 64-22 and PG 76-28 asphalt binders. The acid SFE component (polar component) of both PG 64-22 and PG 76-28 asphalt binders were higher than their base SFE component, and increased with an increase in RAP content; but, the range of variations for acid SFE component was similar for both types of asphalt binders. However, the base SFE component of both PG 64-22 and PG 76-28 asphalt binders did not change significantly with increasing RAP content.
2. Based on the wettability parameter estimated for different combinations of asphalt binder type, RAP contents, and aggregates, the coating quality of both PG 64-22 and PG 76-22 asphalt binders for different types of aggregates increased with an increase in RAP content (25% and more). The gravel and basalt aggregates showed the highest and the lowest coating quality among the tested aggregates, respectively.
3. The bond strength between aggregates and asphalt binder systems under dry condition was evaluated based on the work of adhesion. It was found that the work of adhesion of the PG 64-22 and PG 76-22 asphalt binders with different types of aggregates increases with an increase in RAP content (25% and more).

The improvement in the work of adhesion (under dry condition) was found very low for the granite aggregate. The gravel and basalt aggregates showed the highest and the lowest work of adhesion (in dry condition) among the tested aggregates, respectively, which is consistent with the results obtained for wettability. Use of PG 64-22 asphalt binder resulted in a higher work of adhesion (under dry condition) with different aggregates and RAP amounts. A higher work of adhesion is expected to improve the aggregate-asphalt binder bond strength, under dry condition.

4. The debonding potential of aggregate from asphalt binder under wet condition was evaluated based on the work of debonding. It was found that the work of debonding for both PG 64-22 and PG 76-22 asphalt binders with different types of aggregates decreased with an increase in RAP content (25% and more). The reduction in the work of debonding with increasing RAP content was found to be insignificant for granite and basalt aggregates. Gravel and basalt aggregates showed the highest and the lowest work of debonding among the other aggregates, respectively. A higher work of debonding is expected to increase the separation potential of asphalt binder from aggregate in presence of water. However, it should be discussed in conjunction with the wettability and work of adhesion to make a sound judgment on stripping potential.

5. Overall, the energy ratio parameters (ER_1 and ER_2), as mechanistic indicators of resistance to moisture-induced damage, consistently increased with an increase in RAP content for both PG 64-22 and PG 76-28 asphalt binders and all types of aggregates. Based on the ER_1 and ER_2 values, use of polymer-modified asphalt

binder (PG 76-28) was found to increase the moisture-induced damage potential at lower RAP contents (0% and 10%) compared to non-polymer-modified asphalt binder (PG 64-22). At higher RAP contents (25% and 40%) the improvement in resistance to moisture-induced damage was found to be similar for both types of asphalt binders (PG 64-22 and PG 76-28). For different amounts of RAP and different asphalt binder types (PG 64-22 and PG 76-28) basalt and gravel aggregates showed the highest and the lowest resistance to moisture-induced damage, respectively.

6. It was found that the higher the total SFE of the aggregates, the lower the ER_1 and ER_2 values. Therefore, a high total SFE component of aggregate may result in a high moisture-induced damage potential in the mix.
7. Based on the outcomes of this study, the recommended practice for evaluation of the moisture-induced damage potential of asphalt mixes is a combined use of SFE approach and traditional testing methods (e.g., HWT and TSR).

Based on the methodology and the materials used in this study, the following recommendations are suggested.

1. The SFE components of the asphalt binder are expected to change with changing its source due to variability in chemical composition of crude oil. Therefore, the use of asphalt binders from different sources and adding them to the test matrix is recommended.
2. Additional studies on the effect of polymer-modified asphalt binders with different PG-plus grades on moisture-induced damage potential, using the SFE method, is recommended.

3. Performance tests on the asphalt mixes, using the aggregates and asphalt binders tested herein, are recommended to cross-check the results from the SFE method with those obtained from laboratory mix testing.

Table 5.1 Matrix of the Dynamic Wilhelmy Plate Tests Conducted on Binder Mixes

Asphalt Binder Mix		No. of Samples Tested with Each Solvent			Total Samples
Virgin Binder Type and Amount	RAP Binder Amount	Water	Glycerin	Formamide	
PG 64-22 100%	0%	5	5	5	15
90%	10%	5	5	5	15
75%	25%	5	5	5	15
60%	40%	5	5	5	15
PG 76-28 100%	0%	5	5	5	15
90%	10%	5	5	5	15
75%	25%	5	5	5	15
60%	40%	5	5	5	15
Total Asphalt Binder Samples Tested using DWP Test					120

Table 5.2 Matrix of the Sorption Tests Conducted on Aggregates and those from Literature

Type of Aggregate	Aggregate Code	No. of Samples Tested with Each Solvent			Total Samples
		Water	Glycerin	Formamide	
Limestone	LS	3	3	3	9
Rhyolite	RH	3	3	3	9
Sandstone	SS	Adopted from Bhasin and Little (2007).			
Granite	GN	Adopted from Buddhala et al. (2011).			
Gravel	GV	Adopted from Buddhala et al. (2011).			
Basalt	BS	Adopted from Buddhala et al. (2011).			
Total Aggregate Samples Tested using USD Test					18

Table 5.3 SFE Components of Aggregates

Type of Aggregate	Aggregate Code	Aggregate Surface Free Energy Components (mJ/m^2)				
		Γ^{LW} (Non-polar)	Γ^{-} (Base)	Γ^{+} (Acid)	Γ^{AB} (Acid-Base)	Γ_{total}
Limestone	LS	51.4	741.4	17.5	227.9	279.3
Rhyolite	RH	48.9	877.9	7.5	161.9	210.8
Sandstone[1]	SS	58.3	855.0	14.6	223.5	281.8
Granite[2]	GN	133.2	96.0	24.1	96.2	229.4
Gravel[2]	GV	57.5	973.0	23.0	299.2	356.7
Basalt[2]	BS	52.3	164.0	0.6	19.8	72.1

[1]Adopted from Bhasin and Little (2007)

[2]Adopted from Buddhala et al. (2011)

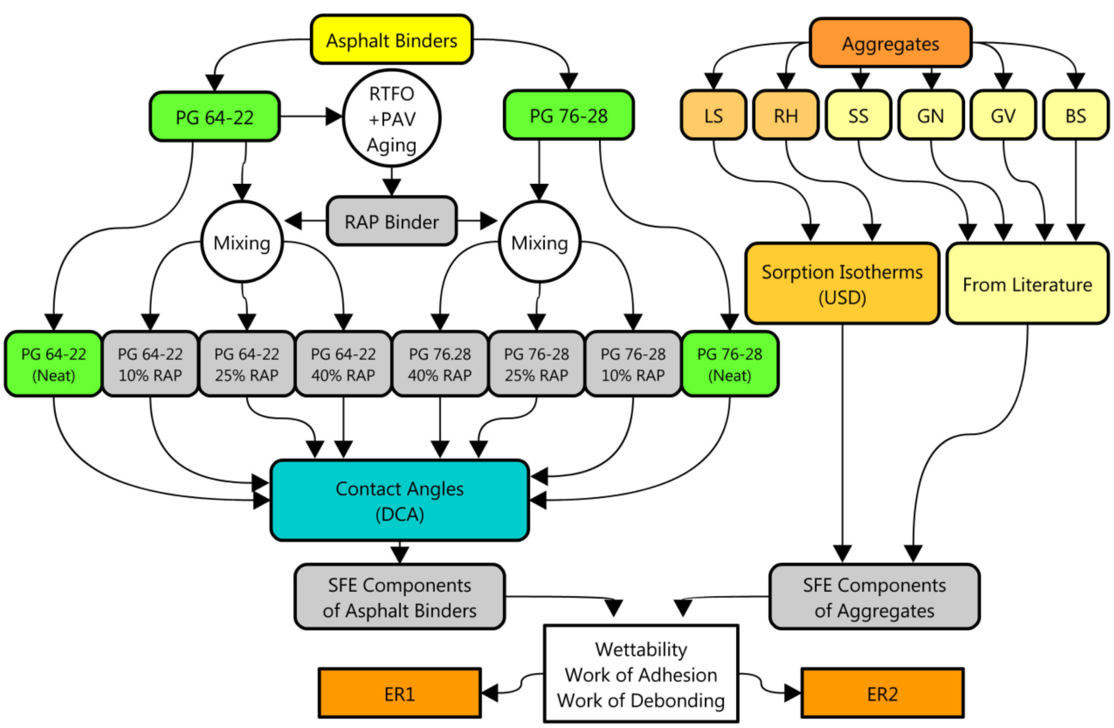

Figure 5.1 Work Flow for Evaluation of Moisture Damage Potential Using SFE Method

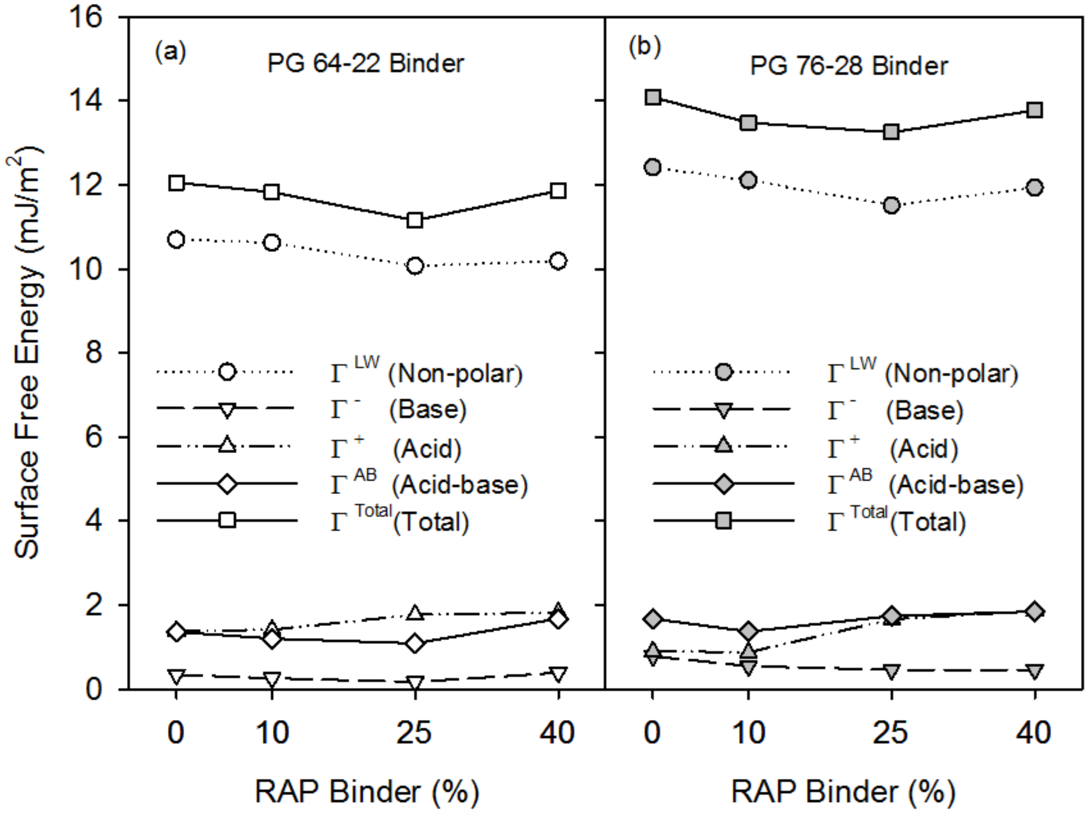

Figure 5.2 SFE Components of (a) PG 64-22 and (b) PG 76-28 Binders Mixed with RAP Binder

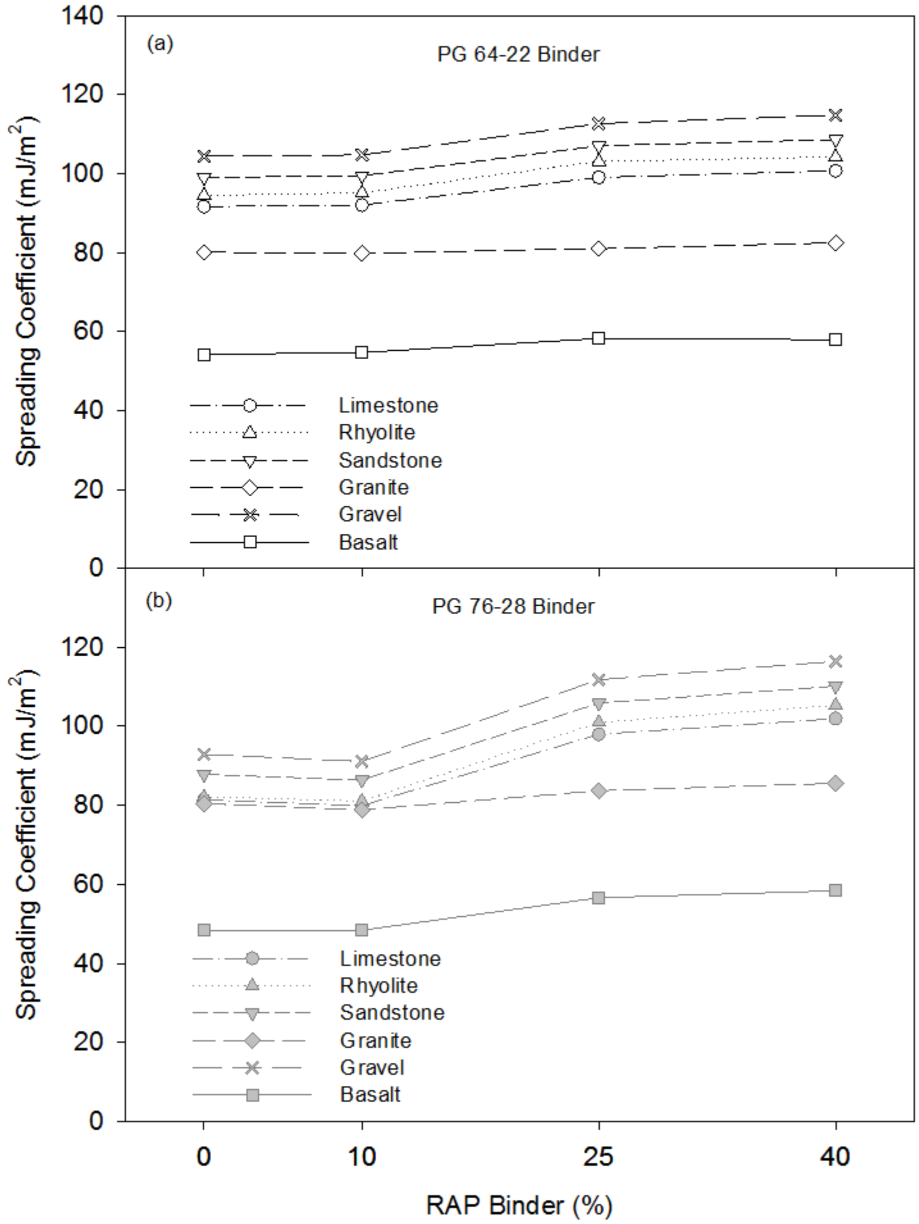

Figure 5.3 Spreading Coefficients of Different Aggregates and (a) PG 64-22 and (b) PG 76-28 Asphalt Binders Mixed with Different RAP Amounts

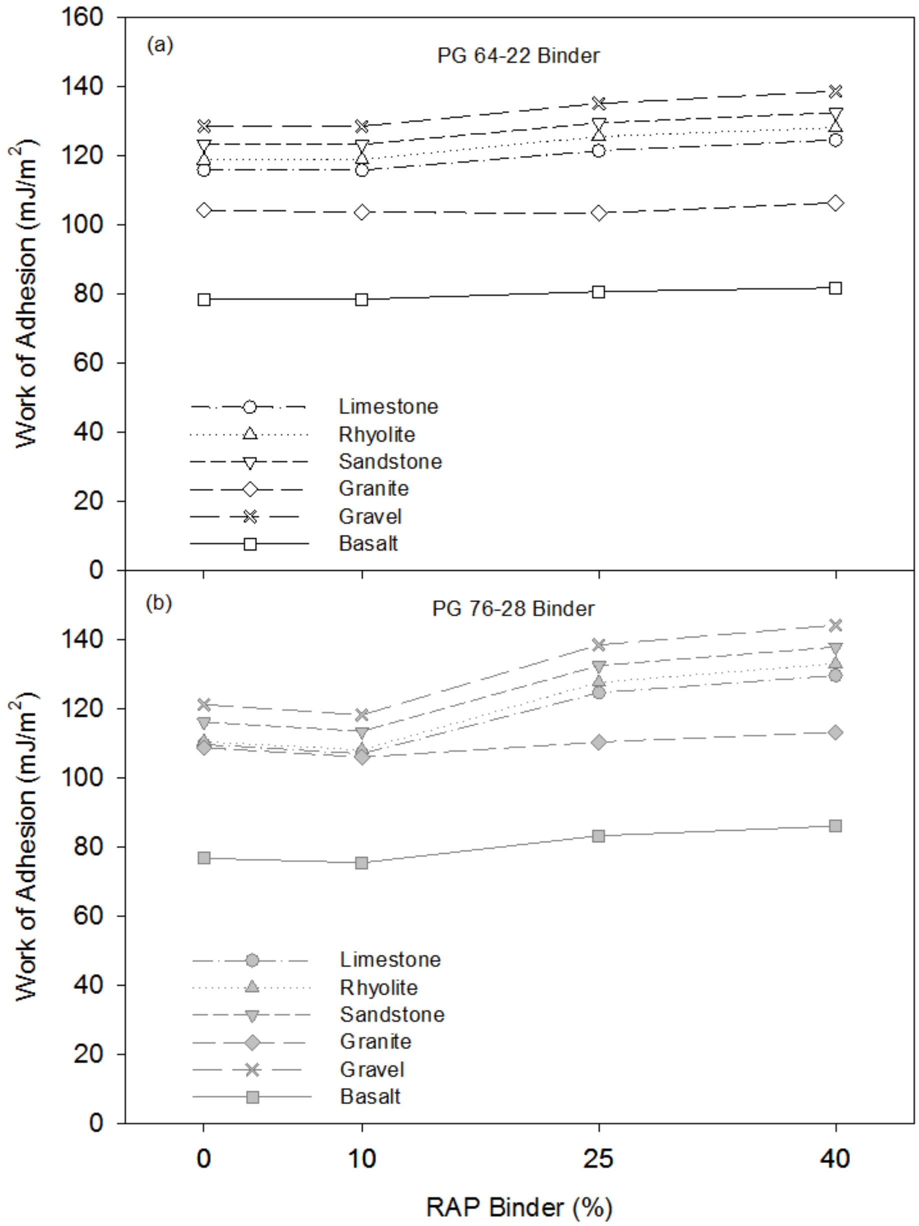

Figure 5.4 Work of Adhesion of Different Aggregates and (a) PG 64-22 and (b) PG 76-28 Asphalt Binders Mixed with Different RAP Amounts

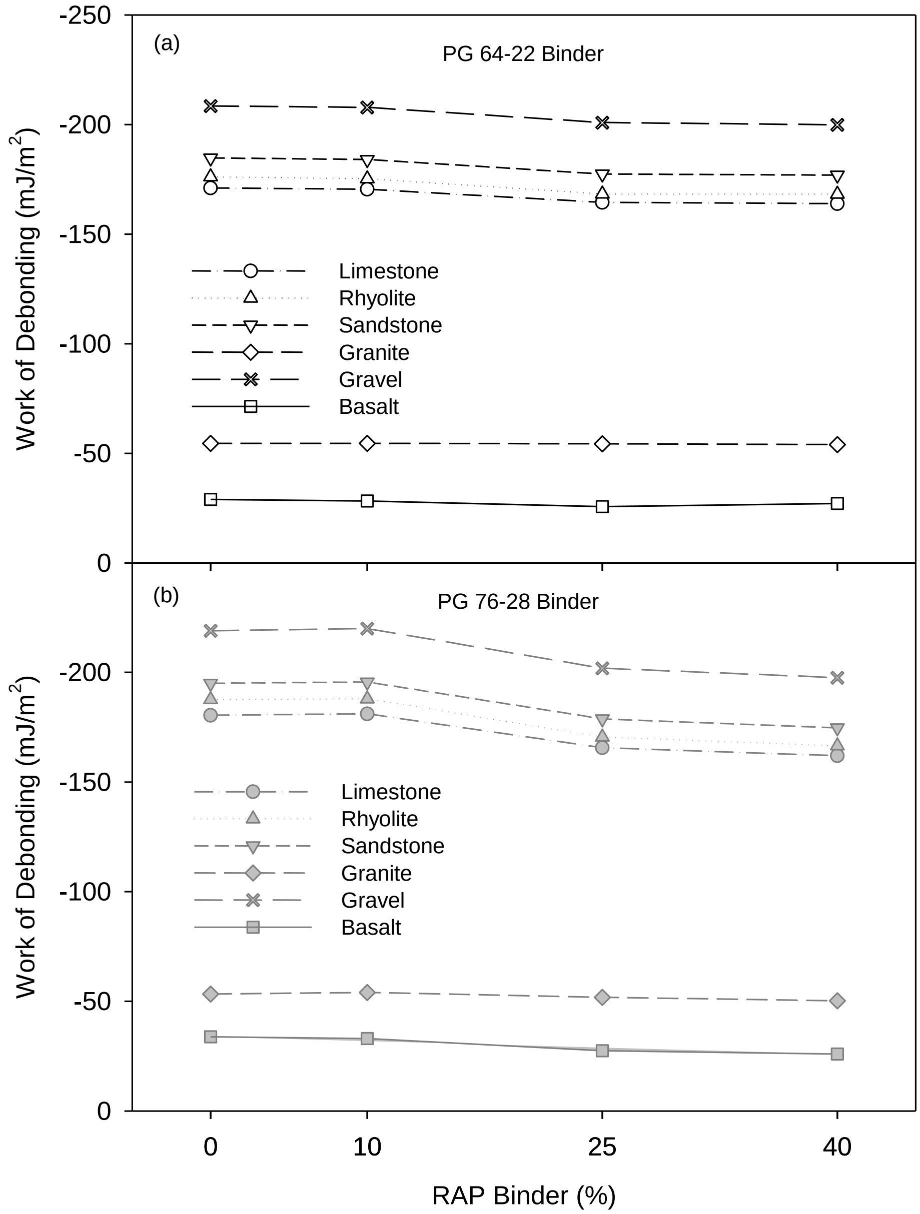

Figure 5.5 Work of Debonding of Different Aggregates and (a) PG 64-22 and (b) PG 76-28 Asphalt Binders Mixed with Different RAP Amounts

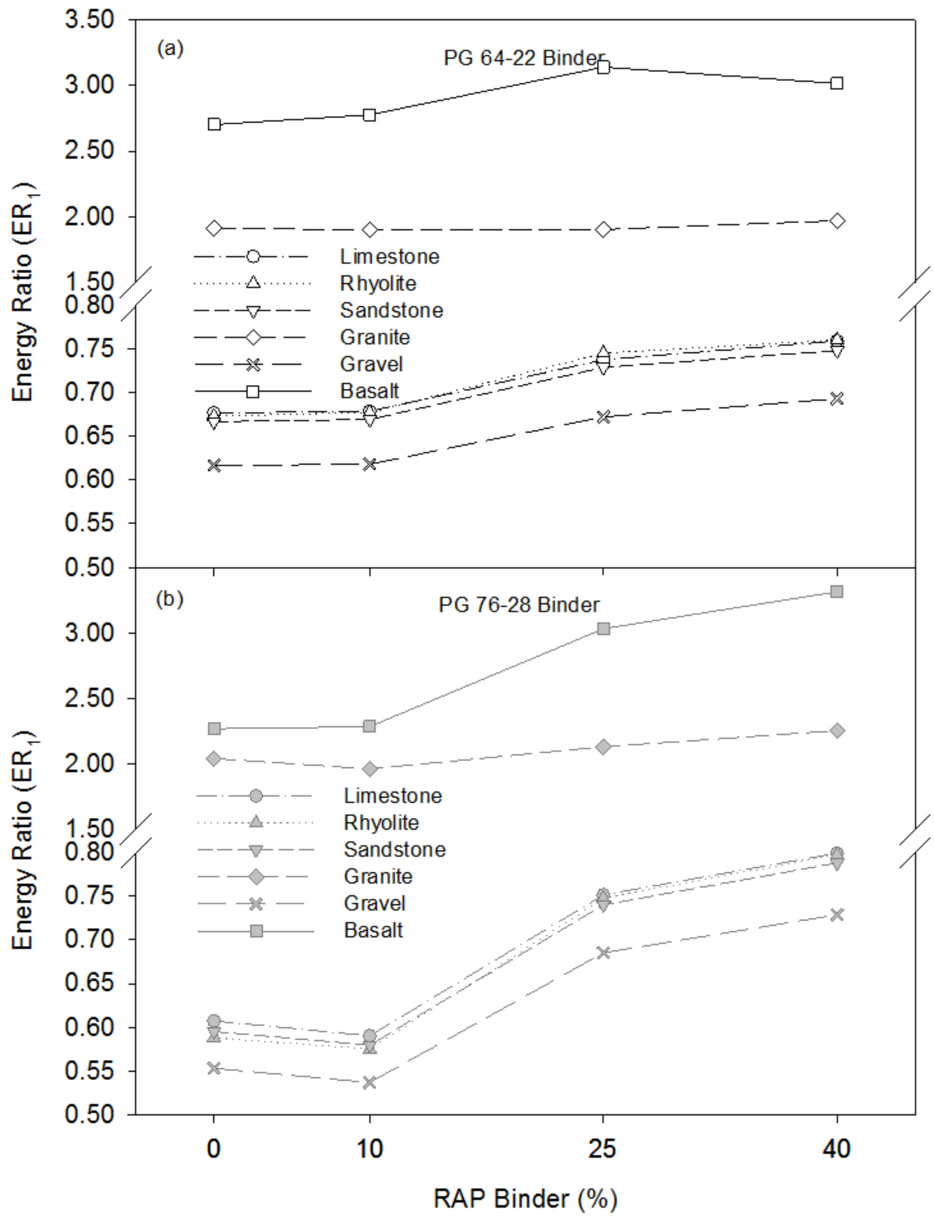

Figure 5.6 ER_1 Values Determined for Different Aggregates and (a) PG 64-22 and (b) PG 76-28 Asphalt Binders Mixed with Different RAP Amounts

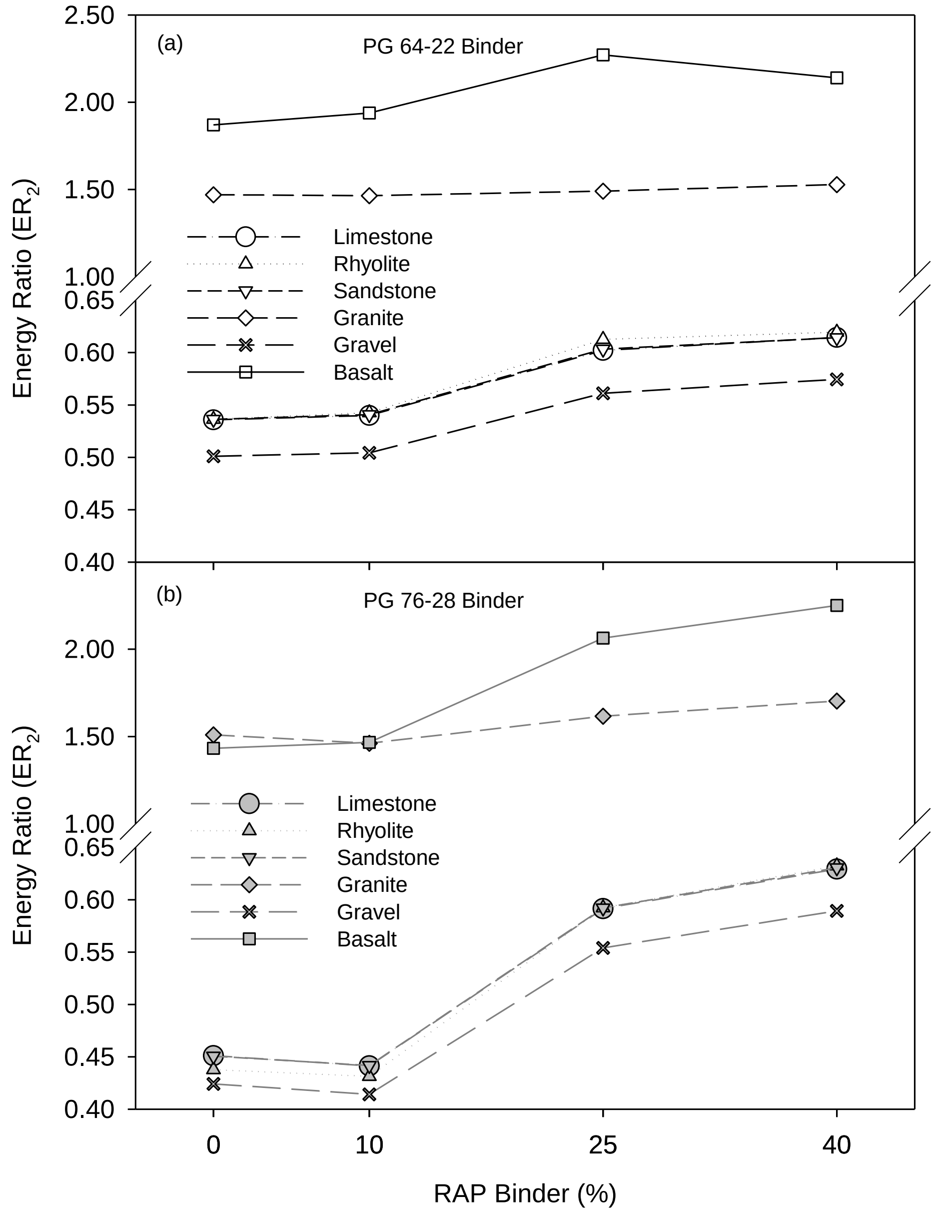

Figure 5.7 ER_2 Values Determined for Different Aggregates and (a) PG 64-22 and (b) PG 76-28 Asphalt Binders Mixed with Different RAP Amounts

CHAPTER

6

MICRO-STRUCTURAL ANALYSIS OF MOISTURE-INDUCED DAMAGE POTENTIAL OF ASPHALT MIXES CONTAINING RAP [††]

ABSTRACT

This study was undertaken to evaluate the effects of reclaimed asphalt pavement (RAP) on moisture-induced damage potential of asphalt mixes using two different approaches: (i) micro-structural analysis of aggregate-asphalt bonding based on the surface free energy (SFE), and (ii) mechanical testing of asphalt mixes using retained indirect tensile strength ratio (TSR) and Hamburg wheel tracking (HWT). This study involved two phases. In the first phase, the SFE (non-polar, acidic and basic) components of a virgin PG 64-22 binder mixed with 0, 25 and 40% of RAP binder and aggregates (limestone, rhyolite, RAP extracted aggregate) were measured using a dynamic contact angle (DCA) device and a universal sorption device (USD), respectively. Thereafter, composite work of adhesion and composite work of debonding, and composite energy ratios for each combinations of asphalt binder and aggregates were determined to assess the moisture-induced damage potential of the

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mixes containing different percentages of RAP (0, 25 and 40%). In the second phase, the TSR and HWT tests were conducted on asphalt mixes containing different percentage of RAP (0, 25 and 40%) to evaluate their moisture-induced damage potential. Both the methods showed that the moisture-induced damage potential decreased with increasing amount of RAP in asphalt mixes. A strong correlation was found to exist between the moisture-induced damage potential predicted using the micro-structural method and laboratory performance tests. It was found that the micro-structural energy approach, as a mechanistic framework, can be successfully used as an indicator of moisture-induced damage potential of the asphalt mixes. It is expected that the present study would be helpful in understanding the moisture-induced damage potential of flexible pavements containing RAP.

Keywords: Reclaimed Asphalt Pavement (RAP), Moisture-Induced Damage, Micro-Structural Analysis, Surface Free Energy.

6.1 Introduction

With increased environmental awareness and focus on recycling, use of Reclaimed Asphalt Pavement (RAP) in pavement construction has become an important topic nationally. Recent studies show that, in addition to preserving the environment, significant savings in cost are realized with increased use of RAP due to reduced requirement of virgin binder and aggregates. Considering huge momentum for using RAP by the asphalt industry, Departments of Transportation (DOTs) have recognized the necessity of updating their specifications and test protocols, which requires laboratory and field test data on asphalt mixes containing RAP.

A large number of studies have indicated that the inclusion of RAP in hot mix asphalt (HMA) alters the mechanical and physical properties of asphalt mixes. For example, many researches have reported an increase in mix stiffness and rut resistance with increasing amount of RAP (e.g. Mogawer et al., 2011; Huang et al., 2004; McDaniel and Shah, 2003a). Despite the wealth of knowledge existing in the literature on the effects of RAP on stiffness and rutting performance of the asphalt mixes, the effects of RAP on HMA's durability is not well understood. A very important distress affecting the durability of the flexible pavements, including those containing RAP, is the moisture-induced damage. By definition, moisture-induced damage is the loss of asphalt binder-asphalt binder tensile strength (cohesive failure) or bonding failure at the asphalt binder-aggregate interface (adhesive failure), due to the presence of moisture (Howson et al., 2009).

A limited number of studies have been conducted to mechanistically investigate the moisture-induced damage potential of asphalt mixes containing RAP. Usually, a Tensile Strength Ratio (TSR) test conducted in accordance with AASHTO T 283 (AASHTO, 2011) is used to evaluate moisture-induced damage potential of asphalt mixes. The TSR is calculated as the ratio of the average tensile strength of moisture-conditioned cylindrical specimens to that of unconditioned specimens. A minimum TSR value of 0.8 is required in order for a mix to pass the mix design requirement (AASHTO M 323). However, despite its popularity, researchers have reported that the TSR test lacks a strong mechanistic basis and in some cases fails to correlate with field observations (Bhasin et al., 2007). In addition, the TSR value of 0.8 set for virgin mixes may not be applicable for recycled mixes.

More recently, the Hamburg Wheel Tracking (HWT) test, conducted according to the AASHTO T 324 (AASHTO, 2011), has been gaining popularity for evaluating rut and moisture-induced damage potential for mixes (Doyle and Howard, 2013; Banerjee et al., 2012; Manandhar et al., 2011; Boyes, 2011; Manandhar et al., 2010; Lu, 2005; Rand, 2002; Tarefder et al., 2002; Aschenbrener et al., 1994). Both of the above mentioned methods (TSR and HWT) are being used widely as indicators of moisture-induced damage potential. Neither of these methods directly addresses the loss of adhesion and cohesion – the "failure mechanisms" that govern the stripping in asphalt pavements. The TSR and HWT test results from a number of mixes show that some mixes with a relatively low TSR value perform well when tested using HWT and vice versa (Ghabchi et al., 2013a). These types of observations raise questions about the reliability of the current practice for evaluation of the moisture-induced damage potential of the mixes. Therefore, there is a need to study the moisture-induced damage mechanism using a mechanistic approach that addresses the shortcomings of empirical methods. Such needs become more important specifically for the mixes containing RAP.

Recent studies show that the Surface Free Energy (SFE) characteristics of asphalt binder and aggregates can be used to quantify moisture-induced damage potential of mixes (Ghabchi et al., 2013a; Ghabchi et al., 2013c; Arabani et al., 2012; Arabani et al., 2011; Hossain et al., 2009; Howson et al., 2009; Wasiuddin et al., 2008; Wasiuddin et al., 2007; Bhasin et al., 2007; Lytton et al., 2005; Cheng et al., 2002). In aforementioned studies, the moisture damage potential of virgin mixes using the SFE approach was investigated, and very limited studies have been carried out to evaluate the SFE components of mixes containing RAP. In addition, no study has been reported, as per the authors' knowledge, where the SFE components of RAP aggregates were determined. Furthermore, the combined SFE components of different types of aggregates available in the asphalt mixes have not been addressed in the literature. The literature is limited to reporting the SFE components of one type of aggregate, which may not be the case for mixes produced in the plant with different types of aggregates such as limestone, granite, sandstone, basalt, and RAP aggregates mixed together. This study focuses on the combined effect of job-mix formula (JMF) and the SFE components of asphalt binder and aggregates referred to as "micro-structure".

Therefore, the current study was undertaken to evaluate the effect of using different amounts of RAP on the moisture-induced damage potential of the mixes by testing asphalt binders and aggregates, applying the micro-structure characterization and mixes using the TSR and HWT tests. For this purpose, the SFE components of a PG 64-22 asphalt binder modified with 0, 25 and 40% RAP binder in contact with the different types of aggregates were determined. The aggregates tested for the SFE components include those collected from different plant stockpiles and extracted from RAP. The contribution of the aggregates from RAP on mix properties becomes more important when the percentage of the RAP is relatively high, compared with the other mix ingredients. The aforementioned asphalt binder and the aggregates were selected from the same materials used for production of the mixes tested using the TSR and HWT methods. The HMA mixes were designed in the laboratory with 0, 25 and 40% RAP and tested using the TSR and HWT methods. The TSR and HWT test results were analyzed to evaluate the moisture-induced damage potential of mixes containing different amounts of RAP. Finally, the results obtained from the SFE method were combined using the JMF of the mixes, according to the method proposed in this study. Then micro-structural energy parameters, as indicators of the moisture-induced damage, were compared with those from testing the mixes using the TSR and HWT tests.

6.2 Objectives

The specific objectives of this study are as follows:

1. Determination of the SFE components of a PG 64-22 asphalt binder with different amounts of RAP binder (0, 25 and 40%) using the Wilhelmy plate test by a Dynamic Contact Angle (DCA) analyzer.
2. Determination of the SFE components of different aggregates used in the mixes tested in this study, including those extracted from RAP using a Universal Sorption Device (USD).
3. Determination of micro-structural moisture-induced damage potential of the mixes, accounting for JMF of the mixes, RAP content and the SFE components and other interfacial energy parameters of asphalt binder and aggregates.
4. Determination of the moisture-induced damage potential of mixes containing different amounts of RAP (0, 25 and 40%) using the TSR and HWT test methods.
5. Ranking the mixes with different percentages of the RAP based on their moisture-induced damage potential, using micro-structural energy-, TSR- and HWT-based approaches.

6.3 Surface Free Energy Theory

By definition, the SFE of a solid is the work required for increasing its surface by a unit area under vacuum (van Oss et al., 1988). Van Oss et al. (1988) proposed a three-component SFE theory, known as Good-van Oss- Chaudhury theory, in which the total surface energy can be expressed in the form of three independent components based on intermolecular forces: (i) a monopolar acidic component (Γ^{+}), (ii) a monopolar basic component (Γ^{-}), and (ii) an apolar or Lifshitz-van der Waals component (Γ^{LW}). According to this theory, as shown in Equations 6.1 and 6.2, the total SFE component (Γ^{total}) can be expressed in terms of a Lifshitz-van der Waals (Γ^{LW}) and an acid-base (Γ^{AB}) component.

$$\Gamma^{Total} = \Gamma^{LW} + \Gamma^{AB} \tag{6.1}$$

where

$$\Gamma^{AB} = 2\sqrt{\Gamma^{+}\Gamma^{-}} \tag{6.2}$$

Recently, several researchers have successfully implemented the surface free energy theory to evaluate adhesion and cohesion behavior of aggregate-asphalt systems (e.g. Howson et al., 2009; Wasiuddin et al., 2008; Wasiuddin et al., 2007; Bhasin et al., 2007; Lytton et al., 2005; Cheng et al., 2002; Elphingstone, 1997; Good, 1992). A discussion on the SFE mechanistic parameters, namely SFE components of asphalt binder and aggregates, work of adhesion, work of debonding, and energy ratio, is provided below for completeness.

6.3.1 SFE Components of Asphalt Binder

One of the test methods which has been successfully used for determination of the SFE components of asphalt binder is measurement of contact angles (θ) between asphalt binder and three different solvents, using the Wilhelmy plate test method (Ghabchi et al., 2013a; Ghabchi et al., 2013c; Arabani et al., 2012; Arabani et al., 2011; Hossain et al., 2009; Howson et al., 2009; Wasiuddin et al., 2008; Wasiuddin et al., 2007). Usually one apolar, one monopolar and one bipolar solvent are used for this purpose. The measured contact angles of asphalt binder with different solvents were used in Equation 6.3 to obtain the SFE components (Good and Van Oss, 1991). A detailed discussion on measurement of the SFE components of asphalt binder selected in this study is provided later in this study.

$$\Gamma_L(1+\cos\theta) = 2\left(\sqrt{\Gamma_A^{LW}\Gamma_L^{LW}} + \sqrt{\Gamma_A^{+}\Gamma_L^{-}} + \sqrt{\Gamma_A^{-}\Gamma_L^{+}}\right) \tag{6.3}$$

where θ represents the contact angle, Γ_L^{LW}, Γ_L^{+} and Γ_L^{-} are Lifshitz-van der Waals, acidic, and base SFE components of the liquid solvent.

6.3.2 SFE Components of Aggregates

The SFE components of aggregates can be determined based on adsorption isotherms of aggregates with three different probe vapors using a USD (Ghabchi et al., 2013a; Ghabchi et al., 2013c; Arabani et al., 2012; Arabani et al., 2011; Hossain et al., 2009; Howson et al., 2009; Wasiuddin et al., 2008; Wasiuddin et al., 2007; Cheng et al., 2002).Three different probe vapors, one apolar, one monopolar and one bipolar, were used in adsorption tests. After obtaining the adsorption isotherms, the methodology used by Bhasin et al. (2007) is applied to determine the aggregates' SFE components. According to this methodology, the work of adhesion between aggregates and probe vapor is given by Equation 6.4.

$$W_{SV} = \pi_e + 2\Gamma_V^{Total} \tag{6.4}$$

where W_{SV} = work of adhesion between aggregate surface and probe vapor; Γ_V^{Total} = total surface free energy of probe vapor; and π_e = equilibrium spreading pressure of the probe vapor on aggregate surface. The spreading pressure is given by Equation 6.5.

$$\pi_e = \frac{RT}{MA}\int_0^{p_n} \frac{n}{p} dp \tag{6.5}$$

where R = universal gas constant; T = test temperature; M = probe vapor molecular weight; n = adsorbed mass of probe vapor per unit mass of the aggregate at probe vapor pressure of p; and A = specific surface area of aggregate. The SFE components of aggregate therefore can be determined by simultaneously solving the Equations 6.4, 6.5 and 6.6. The detailed discussion on measurement of the SFE components of aggregates selected in this study is provided later in this study.

6.3.3 Work of Adhesion

The work of adhesion (W_{AS}) is defined as the free energy required to create two interfaces from one interface, consisting of two different phases in contact. The work of adhesion between an asphalt binder (subscript *A*) and aggregate or stone (subscript *S*) is determined from Equation 6.6. According to the definition of the work of adhesion, the larger the magnitude of W_{AS}, the stronger the bond between the asphalt binder and aggregate (Bhasin et al., 2007). Therefore, a higher W_{AS} may contribute to a mix with a higher resistance to moisture-induced damage.

$$W_{AS} = 2\sqrt{\Gamma_A^{LW}\Gamma_S^{LW}} + 2\sqrt{\Gamma_A^{+}\Gamma_S^{-}} + 2\sqrt{\Gamma_A^{-}\Gamma_S^{+}} \tag{6.6}$$

6.3.4 Work of Debonding

The work of debonding (W_{ASW}^{wet}), is defined as the energy released resulting from separation of asphalt binder from aggregate surface due to presence of water (subscript *W*). W_{ASW}^{wet} is determined from Equation 6.7.

$$W_{ASW}^{wet} = \Gamma_{AW} + \Gamma_{SW} + \Gamma_{AS} \tag{6.7}$$

where, Γ_{AW}, Γ_{SW} and Γ_{AS} stand for the interfacial energies between asphalt binder and water, aggregate and water and asphalt binder and aggregate, respectively. According to its definition, the interfacial (Γ_{ij}) energy is the energy equal to the surface tension at an interface. Γ_{ij} between materials *i* and *j* is determined from Equation 6.8 (Bhasin et al., 2007).

$$\Gamma_{ij} = \Gamma_i + \Gamma_j - 2\sqrt{\Gamma_i^{LW}\Gamma_j^{LW}} - 2\sqrt{\Gamma_i^{+}\Gamma_j^{-}} - 2\sqrt{\Gamma_i^{-}\Gamma_j^{+}} \tag{6.8}$$

where, $\Gamma_i$and Γ_j stand for the total surface free energy of materials *i* and *j*, respectively. Spontaneous debonding between asphalt and aggregate due to the presence of water results in a negative value for W_{ASW}^{wet}. In other words, due to debonding, energy is released and the total energy level of the system is reduced, which is a thermodynamically favorable mechanism. Therefore, a greater $|W_{ASW}^{wet}|$ implies a higher debonding potential between asphalt binder from aggregate in the presence of the water (Bhasin et al., 2007). Therefore, in order to characterize adhesion and debonding of asphalt binder and aggregates, determination of the SFE components of these materials is required.

6.4 Materials

6.4.1 Asphalt Binder and Aggregates

The PG 64-22 asphalt binder used in this study was collected from Schwartz Paving Co. asphalt plant, located in Oklahoma City, OK. The source of the asphalt binder was Valero refinery in Muskogee, OK. This asphalt binder is commonly used in Oklahoma for construction of pavements. Similarly, different types of aggregates, namely, limestone, sandstone and rhyolite used for production of mixes in this study were collected from different quarries in Oklahoma. These are among the most common aggregates used in Oklahoma for production of mixes.

6.4.2 Reclaimed Asphalt Pavement

RAP was collected from Schwartz Paving Co., Oklahoma City, OK. The RAP was milled from the interstate paving projects. This RAP was used for mix design, mix production and aggregate extraction. The collected RAP had an asphalt content of 5.3% by total weight. Also the nominal maximum aggregate size (NMAS) for collected RAP was found to be 12.5 mm.

6.4.3 RAP Binder

In the present study, the pressure aging vessel (PAV) method in accordance with AASHTO R 28 (AASHTO, 2012) was used to produce RAP binder representative of long-term aging of the asphalt binder equivalent to five to ten years of in-service aging. It has been reported in literature that the PAV method can simulate both chemical and physical changes of asphalt binders during its service life (Galal et al., 2000). Sufficient quantity of collected PG 64-22 virgin binder was first short-term aged using a Rolling Thin-Film Oven (RTFO) and then was subjected to long-term aging using a PAV, in accordance with the AASHTO T 240 (AASHTO, 2013) and AASHTO R 28 (AASHTO, 2012) test procedures, respectively. This method has been successfully used by many researchers to produce simulated RAP binder in the laboratory. The PAV method is preferred over the chemical extraction method, in which the chemicals may significantly alter asphalt binders' chemical and surface properties resulting in variability in the SFE parameters.

6.4.4 Asphalt Mixes

Three Superpave mixes with a NMAS of 19 mm and with different percentages of RAP: 0, 25 and 40% were used in this study. These mixes are currently being used in Oklahoma for construction of interstate and state highways and city streets. The control mix with 0% RAP (Mix-1) was designed and produced in the laboratory in accordance with the AASHTO R 35 and AASHTO M 323 standard test methods. While the mixes containing 25% RAP (Mix-2) and 40% RAP (Mix-3) were collected from the asphalt production plant (Schwartz Paving Co.). Details of the aforementioned mixes are presented in Table 6.1. All three mixes were composed of limestone, sandstone and rhyolite aggregates. The gradation of each stockpiles and combined gradation of each mix is also presented in Table 6.1.

As shown in Table 6.1, Mix-1 (0% RAP) consisted of 22% of 38.1 mm rock, 19% of 15.9 mm chips, and 21% of stone sand, all from the limestone source. In addition, it consisted of 16% of natural sand from a sandstone source and 22% of screening from a rhyolite source. The asphalt binder content (AC) of Mix-1 was 4.4% by the weight of the mix.

The Mix-2 (25% RAP) consisted of 15% of 38.1 mm rock, 19% of 15.9 mm chips, and 32% of stone sand, all from the limestone source. In addition, it consisted of 9% of natural sand from a sandstone source and 25% of fine RAP. The total AC content in Mix-2 was 4.1%, out of which 1.3% was added from RAP, indicating approximately 31.7% binder replaced by the RAP binder. The Mix-3 (40% RAP) consisted of 18% of 38.1 mm rock, and 42% of 15.9 mm chips from a limestone source. In addition, it consisted of 40% of fine RAP. The total AC content in Mix-3 was 5.1%, out of which 2.2% was added from RAP, indicating approximately 43.13% binder replaced by RAP binder.

6.5 Experimental Method and Procedure

6.5.1 Preparation of Asphalt Binder

Virgin asphalt binder (PG 64-22) and asphalt binder from RAP are required for the SFE component determination for different combinations of RAP and virgin asphalt binders. The RAP binder and virgin binder were mixed to obtain the desired combinations, according to the mix designs presented in Table 6.1. Therefore, asphalt binder mixes prepared for this study consisted of: (i) 100% virgin binder, (ii) 25% RAP binder + 75% virgin binder, and (iii) 40% RAP binder + 60% virgin binder.

6.5.2 Measurement of Surface Free Energy Components of Asphalt Binders

The SFE components of the virgin asphalt binder and mixes of virgin and RAP binders were determined based on the measurement of their contact angles with different solvents using the dynamic Wilhelmy Plate test (DWP). Three different solvents of known SFE components, namely, water, glycerin and formamide, according to the methodology applied by Wasiuddin et al. (2007), were used in this study.

Samples of cover glasses of 25 mm width by 50 mm length, coated with asphalt binder, were prepared for measurement of contact angles. Before coating the cover glasses with asphalt binder, the plate surface was cleaned by passing it through the oxygen flame at least three times in less than 5 seconds. Then, approximately 100 grams of asphalt binder was placed in a canister and kept in the oven at 165°C for two hours. Thereafter, each glass plate was vertically dipped in the liquefied asphalt binder in the oven and moved back and forth three times in 5 seconds. The plate was placed on a vertical stand in the oven for 2 minutes to obtain a consistent surface. The prepared sample was then moved in a desiccator and cured for 24 hours, before the testing. A DCA device from Cahn was used to conduct DWP tests. The SFE components of asphalt binder were then determined by simultaneously solving the Equation 6.3 written for each contact angle measured for each solvent. A total of 45 asphalt binder samples (5 replicate samples for each binder mix x 3 RAP contents (0, 25 and 40%) for each binder mix x 3 solvents) were prepared and tested in the laboratory using DWP method.

6.5.3 Measurement of Surface Free Energy Components of Aggregates

As seen from Table 6.1, mixes contained different types of aggregates, namely, limestone, sandstone, rhyolite and the aggregates from RAP. The SFE components of these aggregates were measured using a USD as per the methodology discussed by Bhasin and Little (2007). This technique is based on the development of a vapor sorption isotherm, i.e. the amount of vapor adsorbed or desorbed on the solid surface at a fixed temperature and partial pressure (Bhasin and Little, 2007). In this method, the adsorption isotherms of different probe vapors on the aggregates are used to determine the SFE components. For this purpose, the probe vapors of known SFE components, namely, water, n-hexane, and methyl propyl ketone (MPK), were used to determine adsorption isotherms. As recommended, aggregate passing to a No.4 sieve and retaining on a No. 8 sieve were used for USD testing. Sample weight, temperature, and relative humidity or pressures were recorded in a data file at user defined intervals. Sample weight changes were recorded using a Cahn D-101 microbalance. Recorded data were used for calculation of the SFE components using Equation 6.3.

Since the mixes (Mix-2 and Mix-3) were composed of 25% and 40% of RAP, it was planned to determine the SFE components of RAP-extracted aggregates. Usually two methods, namely, ignition oven and chemical methods, are used to extract aggregates from RAP. Since both these methods can change chemical composition and surface properties of aggregates due to application of extreme heat and use of chemicals, another procedure, herein referred to as “cold extraction method,” was used to prepare aggregate specimens from RAP without altering the aggregate properties.

For this purpose, the RAP material was oven dried at 60°C for 24 hours and cooled down to room temperature. Thereafter, it was crushed and the particles passing to a No. 4 sieve and retaining on a No. 8 sieve, without asphalt coating, were selected. The extracted RAP aggregates (EX) were used for testing in a USD.

Before starting the USD test, the selected size of aggregates (passing to a No. 4 sieve and retaining on a No. 8) of limestone aggregates (LS), Rhyolite aggregates (RH), and extracted RAP aggregate (EX), were washed several times with distilled water to obtain a dust-free and clean surface. Thereafter, the aggregates were oven dried at 120°C for 12 hours and allowed to cool to room temperature in a desiccator sealed with silica gel. About 20 grams of each aggregate were used to conduct one USD test with a probe vapor. At least three replicate samples for each probe vapor were tested to ensure consistency and reproducibility of results. A total of 27 (3 types of aggregates (limestone, rhyolite, RAP aggregates) x 3 samples x 3 probe vapors) aggregate samples were tested in the USD device. Thereafter, Equations 6.4, 6.5 and 6.6 were used to determine the SFE components of the LS, RH and EX aggregates.

6.5.4 Asphalt Mix Design

The Superpave volumetric mix designs of three mixes, containing different amounts of RAP (0, 25 and 40%), were established in the laboratory in accordance with the AASHTO M 323 (AASHTO, 2013) standard specification and AASHTO R 35 (AASHTO, 2012) standard practice. Mix designs were carried out for a traffic level of 3-10 EASLs (Equivalent Single Axle Loads). Table 6.1 presents the details of the mix designs developed for this study.

6.5.5 Mechanical Tests to Determine Moisture Damage of Asphalt Mixes in Laboratory

Two different tests, namely, HWT and TSR were used to evaluate the moisture-induced damage potential of mixes in the laboratory. These tests were conducted on virgin and recycled mixes (Mix-1: 0% RAP, Mix-2: 25% RAP, and Mix-3: 40% RAP) as presented in Table 6.1. This section gives a brief overview of both tests.

6.5.5.1 Hamburg Wheel Tracking Test

Hamburg wheel tracking tests (HWT) were conducted on selected mixes (Mix-1: 0% RAP, Mix-2: 25% RAP, and Mix-3: 40% RAP) (Table 6.1) in accordance with AASHTO T 324 (AASHTO, 2011) standard method (AASHTO, 2011). The HWT device consists of a loaded steel wheel of 705 ± 22 N, 204 mm diameter, and 47 mm width, reciprocating over a test specimen. The test stops automatically either at a maximum number of 20,000 wheel passes, or the maximum allowable rut depth. The test specimens of 150 mm diameter and 62 ± 2 mm height were prepared using a Superpave gyratory compactor (SGC) with target air voids of 7 ± 0.5%. The HWT tests were conducted on the specimens submerged in a water bath with a temperature of 50 ± 1°C. The moisture damage potential of the mixes was evaluated from a striping inflection point (SIP).

6.5.5.2 Retained Indirect Tensile Strength Ratio Test

Moisture-induced damage potential of the selected mixes (Mix-1: 0% RAP, Mix-2: 25% RAP, and Mix-3: 40% RAP) was determined based on their retained indirect tensile strength ratio in accordance with the AASHTO T 283 (AASHTO, 2011) standard method. In this method, moisture susceptibility of mixes is evaluated by measuring the tensile strength decay as a result of the accelerated moisture and temperature conditioning. For this purpose a minimum of six cylindrical SGC specimens of 150 mm diameter and 95 mm height were compacted, with 7.0 ± 0.5% target air voids. Specimens were then divided into two subsets of three samples. One subset was tested in dry condition (unconditioned samples) at a temperature of 25°C for indirect tensile strength. The other subset of samples was partially vacuum-saturated (70 to 80 percent) under a 13 to 67 kPa absolute vacuum pressure, called conditioned samples. Then each vacuum-saturated specimen was sealed using a plastic film and placed in a plastic bag containing 10 ml water. Thereafter, the saturated specimens were temperature-conditioned using a freezing cycle of -18°C for a minimum of 16 hours followed by a 60°C hot water conditioning for 24 hours. Before conducting the tensile strength test conditioned specimens were placed in a water tank of 25°C temperature for two hours. The TSR value for each selected mix (Mix-1: 0%RAP, Mix-2: 25% RAP, Mix-3: 40%RAP) was calculated by dividing the average tensile strength of conditioned to that of unconditioned specimen subsets.

6.6 Results and Discussion

6.6.1 SFE Components of Asphalt Binders

The SFE components of the neat and modified PG 64-22 asphalt binder with different amounts of RAP binder (25 and 40%) are presented in Table 6.2.

It can be observed from Table 6.2 that the addition of RAP binder changed the nonpolar SFE component of neat PG 64-22 binder. For example, the non-polar SFE component of neat binder (10.70 mJ/m^2) reduced by 5.9 and 4.8% by addition of 25 and 40% RAP binder, respectively. Furthermore, from Table 6.2 it was observed that the acid SFE component of neat PG 64-22 binder increased from 1.38 mJ/m^2 to 1.77 and 1.82 mJ/m^2, when 25 and 40% RAP binder was added (28.3% and 31.9% increase), respectively. The change in the non-polar and the acid SFE components may be attributed to the change in chemical composition of the asphalt binder due to the addition of aged RAP binder. This change is possibly due to the oxidization as a result of the aging process which the RAP binder had gone through during its life cycle. Additionally, Table 6.2 showed that the total SFE component of the neat asphalt binder generally decreased with the addition of RAP, but a general trend of variation in the total SFE component was not detected. For example, the total SFE component of the tested neat asphalt binder (12.06 mJ/m^2) decreased to 11.16 mJ/m^2 and 11.86 mJ/m^2

with the addition of 25 and 40% RAP binder, respectively. Overall, the use of RAP was shown that will change the SFE components of the asphalt binder mixes.

6.6.2 SFE Components of Aggregates

The SFE components of the aggregates used for mix designs consisting of limestone (LS), rhyolite (RH), and extracted aggregates from RAP (EX) are presented in Table 6.3. Also the SFE components of a typical sandstone (SS) aggregate, adopted from literature (Bhasin and Little, 2007), are shown in Table 6.3.

It can be observed from Table 6.3 that SS and EX aggregates have the highest and lowest non-polar SFE components (58.3 and 33.5 mJ/m^2), respectively. Furthermore the LS and SS aggregates have significantly higher acid SFE components (17.5 and 14.6 mJ/m^2, respectively) compared to those of RH and EX aggregates (7.5 and 2.7 mJ/m^2, respectively). It should be taken into consideration that using an acidic aggregate such as SS with asphalt binder, which is acidic in nature, can cause weak bond resulting in higher moisture-induced damage potential (Arabani and Hamedi, 2011). Therefore, measurement of the aggregate SFE components is helpful for determining the intermolecular forces arising from the surface properties of the aggregates and asphalt binders in contact.

The variations in non-polar, polar, and the total SFE components of the asphalt binder and aggregates, discussed above, are known to be immensely important parameters capable of affecting the adhesion and debonding energies. Therefore the SFE components of these materials, governing the moisture-induced damage mechanism of the mixes with asphalt binder containing different amounts of RAP, and aggregates, are very important to be determined (Bhasin and Little, 2009; Wasiuddin et al., 2008; Masad et al., 2006; Kim et al., 2004; Cheng, 2002; Bhasin, 2006), as carried out in this study.

6.6.3 Effect of RAP Binder and Aggregate Type on Work of Adhesion

As discussed before, bond strength between asphalt binder and aggregate in dry condition can be described by the work of adhesion (W_{AS}). By definition, W_{AS} is work required to separate asphalt binder from aggregate interface (Bhasin et al., 2007). Therefore, it is favorable for asphalt binder-aggregate system to have a higher magnitude of positive W_{AS} value in order to form a stronger bond and therefore a more durable mix. Higher tendency of adhesion also facilitates proper wetting of aggregate by asphalt binder during the mixing process, resulting in a better asphalt coating on aggregates and an improved bond (Bhasin and Little, 2007). Table 6.4 summarizes the W_{AS} of neat PG 64-22 asphalt binder and binder modified with different amounts of RAP binder (0, 25 and 40%), in contact with LS, SS, RH and EX aggregates used in the mixes.

It can be seen from Table 6.4 that W_{AS} increases with an increase in RAP amount. For example, W_{AS} of neat asphalt binder with RH aggregate (118.6 mJ/m^2) increased to 125.4 mJ/m^2 and 128 mJ/m^2 with the addition of 25 and 40% RAP binder, respectively. A similar trend was observed for other types of aggregates, indicating that use of RAP has an improving effect on the bonding characteristics of aggregate-asphalt systems.

6.6.4 Effect of Amounts of RAP Binder and Aggregate Type on Work of Debonding

In the presence of moisture, water displaces the asphalt binder from the aggregate surface and the free energy of the system decreases (Bhasin and Little, 2007).

Therefore, W_{ASW}^{wet} will be a negative value in occurrence of any spontaneous separation. The greater the magnitude of the W_{ASW}^{wet}, the higher the thermodynamic potential of stripping in the presence of water. Therefore, the study of the work of debonding as an energy parameter is immensely important for better understanding the moisture-induced damage mechanism and damage potential quantification. Table 6.5 presents W_{ASW}^{wet}.of the PG 64-22 asphalt binder, modified with different amounts of RAP binder (0, 25 and 40%), in contact with LS, SS, RH and EX aggregates used in mixes.

Table 6.5 shows that the addition of RAP to PG 64-22 binder decreased (the magnitude of) the W_{ASW}^{wet} with all aggregates. For example, the work debonding of the neat PG 64-22 binder with SS aggregate (-184.7 mJ/m^2) reduced addition of 25 and 40% RAP binder (-177.4 and -176.9 mJ/m^2, respectively). A similar trend was observed when PG 64-22 modified with RAP binder was used with LS, RH and EX aggregates, as well. Reduction of the W_{ASW}^{wet} by addition of RAP is implication of the lowered debonding potential between asphalt binder and aggregates. Therefore, based on the W_{ASW}^{wet} values discussed herein, it can be concluded that use of RAP in mixes with different types of aggregates may reduce the moisture-induced damage potential.

It is important to note that the moisture-induced damage potential cannot be evaluated with the magnitude of W_{AS} only or W_{ASW}^{wet}, only. For example, Table 6.4 shows that SS aggregate has the highest W_{AS} value with the virgin PG 64-22 binder, which is indicative of a strong bond between them. However, from Table 6.5 it is evident that the SS aggregate shows the highest magnitude of W_{ASW}^{wet} when used with virgin PG 64-22 binder, an indication of a high potential of debonding in the presence of water. A similar trend is also observed for LS, RH and EX aggregates as well.

Therefore, for evaluation of the moisture-induced damage potential of the mixes, an energy parameter which accounts for both W_{AS} and W_{ASW}^{wet} and the mix design proportions will be used in this study.

6.6.5 Aggregate-Asphalt Binder Micro-Structural Energy Parameters

As reported by many researchers, work of adhesion and the work of debonding are valuable tools to study the asphalt binder-aggregate systems for their adhesive bond strength and the potential of stripping in presence of water, respectively. In order to have a durable mix, it is required to have a higher W_{AS} and a lower magnitude of W_{ASW}^{wet} ($|W_{ASW}^{wet}|$). Therefore, moisture-induced damage potential of the asphalt binder-aggregate system must be studied taking both W_{AS} and $|W_{ASW}^{wet}|$ into account. Based on the above mentioned reasoning, Bhasin et al. (2007) suggested using a single parameter which addresses the effects of the work adhesion and debonding on moisture-induced damage, namely, energy ratio (ER_1). By definition, ER_1 is the absolute ratio of W_{AS} to $|W_{ASW}^{wet}|$. Therefore, greater ER_1 is more desirable for a mix to have a higher resistance to moisture-induced damage. ER_1 is a micro-level parameter directly determined from interfacial energies and molecular forces acting between an aggregate and an asphalt binder interface. However, using this parameter in its current form, for a mix which may consist of different types of aggregates from different sources and different SFE components, is not easily possible. In other words, connecting the "micro-level" energy parameter (ER_1) to "aggregate structure" of mix (amount and type of each stockpile used in the mix) is required to study the mix moisture-induce damage accounting for both "micro-level energy parameters" and "aggregate structure", namely, "micro-structure”. Based on the above mentioned discussion, a “Composite Energy Ratio” (*CER*) is suggested herein and defined as below.

$$CER = \frac{\sum_{i=1}^{n} p_i \ W_{AS_i}}{\left| \sum_{i=1}^{n} p_i \ W_{AS_iW}^{wet} \right|} \tag{9}$$

where *CER* = composite energy ratio; n = the number of aggregate stockpiles used in mix design; p_i = the percentage of aggregate from stockpile i used in the mix; W_{AS_i} = work of adhesion between asphalt binder and aggregate from stockpile i; $W_{AS_iW}^{wet}$ = work of debonding between asphalt binder and aggregate from stockpile i; $\sum_{i=1}^{n} p_i \ W_{AS_i}$ = composite work of adhesion (CWA); and $\sum_{i=1}^{n} p_i \ W_{AS_iW}^{wet}$ = composite work of debonding (CWD). The *CER* value is analogous to TSR value obtained by using AASHTO T 283 (AASHTO, 2011). In other words, a higher *CER* means a higher adhesive bond strength and a lower debonding potential in the presence of water, which is an implication of a mix with a higher resistance to the moisture-induced damage.

Based on the above mentioned definitions, CWA, CWD and *CER* values, associated with Mix-1 (0% RAP), Mix-2 (25% RAP) and Mix-3 (40% RAP), were determined and are presented in Table 6.6.

Table 6.6 reveals that the CWA does not show any detectable trend of variations with increasing amounts of RAP in mixes. For example, the CWA of Mix-1 (0% RAP) was reduced from 117.6 mJ/m^2 to 96.2 mJ/m^2 by using Mix-2 (25% RAP) and increased to 105.9 mJ/m^2 when Mix-3 (40% RAP) was used. This mixed trend of variations in CWA is due to the fact that using Equation 6.9 the works of adhesion between asphalt binders and aggregates are contributing to the CWA based on their amounts used in each mix. On the other hand, from Table 6.6 it was observed that (the magnitude of) the CWD reduces with increasing amounts of RAP in mixes. For example, CWD of Mix-1 containing no RAP (-174.4 mJ/m^2) reduced by approximately 29 and 30%, when 25 and 40% RAP was in Mix-2 and Mix-3, respectively. Although a reduction in the CWD implies a lower stripping potential of the mix in the presence of water, this parameter should be studied along with CWA through *CER* parameter. From Table 6.6 it is evident that the addition of RAP to mixes increases the *CER* values, indicating a better resistance to the moisture-induced damage. As shown in Table 6.6, the *CER* value of Mix-1 containing no RAP (0.67) increased to 0.78 and 0.87 by using 25 and 40% RAP in the Mix-2 and Mix-3 (equivalent to 16 and 30% increase), respectively. Therefore, based on *CER* values, it is evident that the use of RAP reduced the moisture-induced damage potential of the tested mixes in this study. This observation is consistent with the findings of Mogawer et al. (2011), Karlsson and Isacsson (2006), and Abdulshafi et al. (2002) which reported a lower moisture-induced damage potential in HMA mixes with using RAP, by conducting performance tests such as TSR and HWT tests.

6.6.6 Moisture-Induced Damage Potential Based on TSR Tests

The TSR tests were conducted on Mix-1, Mix-2 and Mix-3, containing 0, 25 and 40% RAP, respectively. A summary of the TSR test results are presented in Figure 6.1.

It can be seen from Figure 6.1 that the TSR value of mixes increases with an increase in RAP amounts, indicating a better resistance to the moisture-induced damage with addition of RAP. For example, the *TSR* value of Mix-1 containing no RAP (0.90) increased to 0.91 and 1.03 by addition of 25 and 40% RAP in the Mix-2 and Mix-3, respectively. Therefore, based on the *TSR* values, it is evident that the use of RAP reduced the moisture-induced damage potential of the tested mixes in this study. This observation is consistent with the findings from *CER* values discussed before.

6.6.7 Moisture-Induced Damage Potential Based on HWT Tests

A summary of the results of the HWT tests conducted on different mixes is presented in Figure 6.2. From Figure 6.2, it is evident that the number of the wheel passes corresponding to the stripping inflection point (SIP) increases with increasing the amount of RAP in mixes.

For example, Mix-1 (0% RAP) exhibited SIP at 10,032 wheel passes, while it was obtained as 12,320 and greater than 20,000 for Mix-2 (25% RAP) and Mix-3 (40% RAP), respectively. This is an indicator that the addition of RAP increased the resistance of the tested mixes to moisture-induced damage. These observations indicating the improvement of rut and moisture-induced damage resistance of mixes with addition of RAP are consistent with the reported results in the literature (Doyle and Howard, 2013; Banerjee et al., 2012; Manandhar et al., 2011; Boyes, 2011; Manandhar et al., 2010; Lu, 2005; Rand, 2002; Tarefder et al., 2002; Aschenbrener et al., 1994).

6.6.8 Ranking of Asphalt Mixes Based on TSR, SIP and CER

Ranking of the mixes (Mix-1, Mix-2, and Mix-3) was determined based on their resistance to moisture-induced damage obtained from the *TSR*, *SIP*, and *CER* values. For this purpose *CER*, normalized *SIP* wheel passes (based on 20,000 passes), and *TSR* of each mix, were plotted in Figure 6.1. From Figure 6.1, it was observed that all of the parameters, used for evaluation of moisture-induced damage potential, ranked the mixes at the same order. For example, from Figure 6.1 it is evident that the *CER, TSR, and SIP* values increased with increasing the RAP content in mixes. This finding shows that, for the mixes tested in this study, application of the micro-structural energy parameter and *CER* values was satisfactorily capable of capturing the moisture-induced damage potential of mixes. In other words, based on the outcomes of this study, micro-structural energy of mix which combines the intermolecular forces and interfacial energies of the asphalt ingredients may be used for prediction of the moisture-induced damage in mixes, specifically those containing RAP.

6.7 Conclusions

The moisture-induced damage potential of recycled mixes was evaluated in this study using micro-structural energy approach and mechanical tests (TSR and HWT). Micro-structural approach, which combines the intermolecular forces and interfacial energies of the asphalt ingredients with the JMF properties of HMA, was successfully applied for moisture-induced damage potential evaluation of the mixes containing RAP. Based on the results and discussion presented in this study, the following conclusions can be drawn:

1. A methodology was developed to combine the SFE components and interfacial energy parameters of asphalt mix ingredients with JMF of the mix for moisturize-induced damage evaluation of the mixes containing RAP. It was found that CWA and CWD alone are not capable to identify quality of a mix to resist moisture damage. Therefore, CER was introduced and was found to be capable of predicting the moisture-induced damage potential of the mixes tested.
2. Based on the CER values resistance to moisture-induced damage increased with an increase in amount of RAP used in the mixes tested in this study.

3. The TSR test results showed that the resistance of the mixes to moisture-induced damage increases with an increase in amount of RAP used in the mixes. This improvement of the resistance to moisture-induced damage was shown in the form of increasing the TSR value with increase in amount of RAP. Specifically, the addition of 40% RAP to a mix yielded a TSR value of approximately one, which indicates no tensile strength decay as a result of moisture and temperature conditioning.
4. HWT test results showed improvement in resistance of mixes to moisture-induced damage with the addition of RAP to the mixes.
5. Based on the micro-structural energy approach, the TSR, and HWT test results, all of the mixes tested in this study are ranked at the same order in terms of their resistance to moisture-induced damage: the higher the RAP content, the greater the resistance of mixes to moisture-induced damage.
6. Based on the findings of this study, TSR and HWT show good correlation with CER, however, a detailed study may be conducted to evaluate validity of TSR and HWT for different types of mixes.

Table 6.1 Mix Design Properties of the Asphalt Mixes used in the Study

Bin No.	Aggregate	Aggregate Type					% Used Mix-1 (0% RAP)	% Used Mix-2 (25% RAP)	% Used Mix-3 (40% RAP)
1	38.1 mm Rock	Limestone					22	15	18
2	15.9 mm Chips	Limestone					19	19	42
3	Stone Sand	Limestone					21	32	
4	Natural Sand	Sandstone					16	9	
5	Screenings	Rhyolite					22		
6	Fine RAP							25	40
Sieve Size	Percent Passing (%)						Combined Gradation		
Sieve Size (mm)	Bin No. 1	Bin No. 2	Bin No. 3	Bin No. 4	Bin No. 5	Bin No. 6	Mix-1 (0% RAP)	Mix-2 (25% RAP)	Mix-3 (40% RAP)
25.4	100	100	100	100	100	100	100	100	100
19	86	100	100	100	100	100	97	98	97
12.5	40	94	100	100	100	99	86	90	86
9.5	15	49	100	100	100	93	72	76	60
4.75	1	1	97	100	78	74	54	59	30
2.36	1	1	74	100	50	61	43	48	25
1.18	1	1	27	100	34	51	30	31	21
0.6	1	1	13	97	25	42	24	24	17
0.3	1	1	6	68	19	31	17	16	13
0.15	1	1	3	13	15	18	6	7	8
0.075	0.5	0.8	2.7	1.1	11.1	9.7	3.4	3.6	4.3
Total AC Content						5.3%	4.4%	4.1%	5.1%
Virgin AC PG 64-22 Valero (Muskogee, OK)							4.4%	2.8%	2.9%

Table 6.2 The SFE Components of PG 64-22 Asphalt Binder Modified with Different Amounts of RAP Binder

Virgin Binder Type and Amount		RAP Binder Amount	Γ^{LW} (Non-polar)	Γ^{-} (Base)	Γ^{+} (Acid)	Γ^{AB} (Acid-Base)	Γ_{total}
PG 64-22	100%	0%	10.70	0.33	1.38	1.36	12.06
	75%	25%	10.07	0.17	1.77	1.09	11.16
	60%	40%	10.19	0.39	1.82	1.68	11.86

Asphalt Binder Mix; Surface Free Energy Components (mJ/m^2)

Table 6.3 Surface Energy Characteristics of Aggregates used in Mix Designs

Aggregate Code	Type of Aggregate	Aggregate Surface Free Energy Components (mJ/m^2)				
		Γ^{LW} (Non-polar)	Γ^{-} (Base)	Γ^{+} (Acid)	Γ^{AB} (Acid-Base)	Γ_{total}
LS	Limestone	51.4	741.4	17.5	227.9	279.3
SS	Sandstone*	58.3	855.0	14.6	223.5	281.8
RH	Rhyolite	48.9	877.9	7.5	161.9	210.8
EX	Extracted from RAP	33.5	281.8	2.7	54.7	88.3

* Adopted from literature (Bhasin and Little, 2007)

Table 6.4 Work of Adhesion of Asphalt Binder Modified with RAP and Different Aggregates

Asphalt Binder Mix		Work of Adhesion (mJ/m2)			
Virgin Binder Type and Amount	RAP Binder	LS	SS	RH	EX
100%	0%	115.8	123.1	118.6	79.2
PG 64-22 75%	25%	121.3	129.3	125.4	82.7
60%	40%	124.4	132.4	128.0	84.3

LS: Limestone; SS: Sandstone; RH: Rhyolite; EX: Extracted aggregate from RAP.

Table 6.5 Work of Debonding of Asphalt Binder Modified with RAP and Different Aggregates

Asphalt Binder Mix		Work of Debonding (mJ/m^2)			
Virgin Binder Type and Amount	RAP Binder	LS	SS	RH	EX
100%	0%	-171.1	-184.7	-176.2	-63.5
PG 64-22 75%	25%	-164.4	-177.4	-168.3	-59.0
60%	40%	-163.9	-176.9	-168.3	-59.9

Table 6.6 Composite Works of Adhesion, Debonding and *CER* values of Mix-1, Mix-2 and Mix-3

Asphalt Mix Type	RAP (%)	CWA (mJ/m^2)	CWD (mJ/m^2)	*CER*
Mix-1	0	117.6	-174.4	0.67
Mix-2	25	96.2	-123.3	0.78
Mix-3	40	105.9	-122.3	0.87

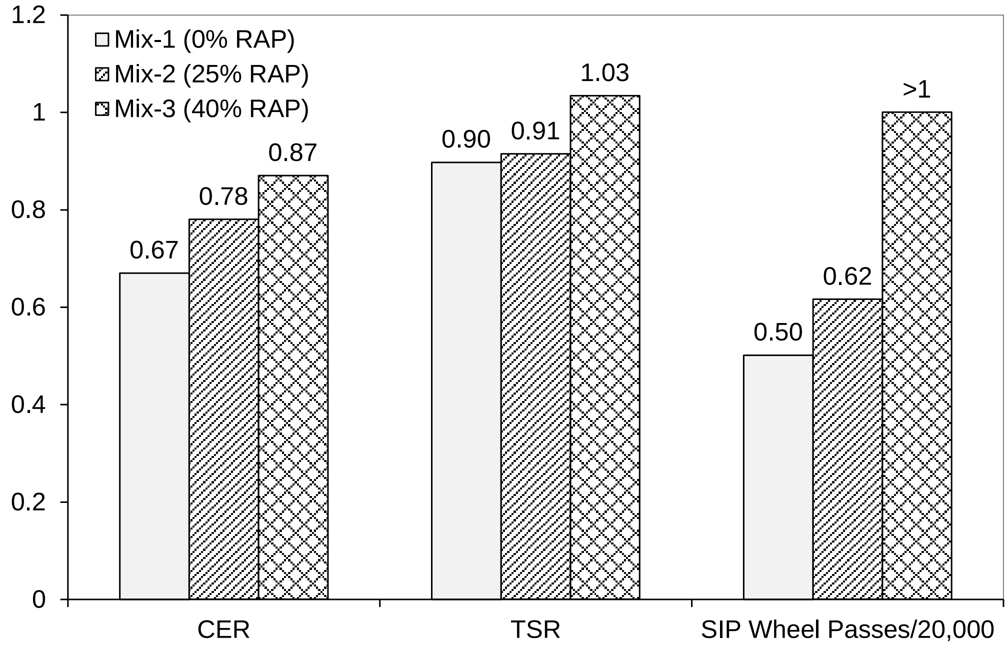

Figure 6.1 Comparison of CER, TSR and Normalized SIP Wheel Passes

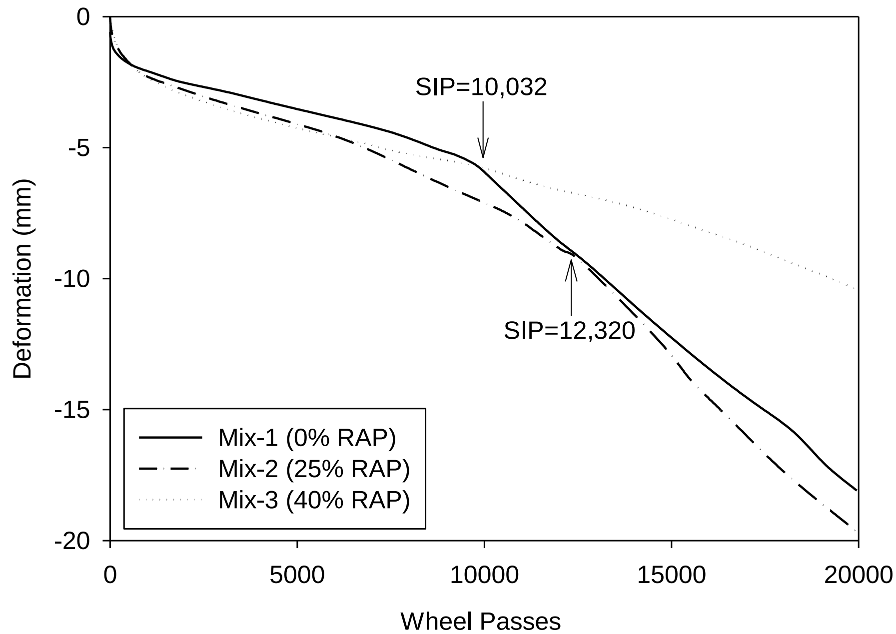

Figure 6.2 Summary of HWT Test Results Conducted on Mix-1, Mix-2 and Mix-3

CHAPTER

7

CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

The major findings of this study are summarized in this chapter. Specific conclusions pertaining to a given topic covered in a given chapter were presented in that chapter. The pertinent conclusions of this study are summarized herein.

7.1.1 Performance of WMA Mixes

1. The dynamic modulus values of the WMA were lower than those of HMA mixes. This may result in a higher rutting susceptibility of WMA in the long term. Less asphalt binder aging in the production of WMA due to a lower mixing temperature, compared to the HMA case, was found to be responsible for this difference. This difference was more pronounced for the coarsest mix (NMAS = 19 mm), Evotherm® Type B, with its HMA control mix.
2. The creep compliance values of the WMA were higher than those of HMA control mixes. This results in a higher relaxation modulus and therefore may contribute to a higher resistance to low-temperature cracking. Also, it was concluded that temperature sensitivity of the creep compliance reduces with an increase in NMAS.

3. It was observed that the fatigue lives of all HMA mixes tested in this study were higher than those of the WMA mixes. The difference between the fatigue life of the WMA and HMA mixes was more pronounced for the Evotherm® Type B mix compared to the HMA control mix.
4. It was concluded that the WMA mixes showed more susceptibility to rutting than the HMA mixes when inverse rutting rate was used to evaluate the rutting potential. However, the WMA and HMA mixes performed almost equally well with respect to rutting when the total rut depth was used as a rutting indicator.
5. It was found that Evotherm® Type C WMA with lime as anti-stripping agent exhibited a stripping inflection point in the Hamaburg wheel tracking test. The observed moisture-induced damage was attributed to possible incompatibility of the Evotherm® additive and lime with the aggregates used in the mix.
6. According to the retained indirect tensile strength ratio test results, only HMA mixes of Advera® HMA and Evotherm® Type C WMA and HMA mixes passed the minimum TSR requirement (0.8), and other mixes showed lower TSR values and did not pass the TSR requirement.
7. It was concluded that the TSR and Hamburg wheel tracking tests can result in contradictory outcomes on the moisture-induced damage potential of WMA and HMA mixes, as observed in the present study.
8. It was found that excellent correlations exists between the rutting depth, rutting ratio and dry (unconditioned) indirect tensile strength (DITS) of asphalt mixes. Furthermore, obtaining the DITS value through indirect tensile strength test is comparatively quicker and easier than conducting a Hamburg wheel tracking

test and/or using an asphalt pavement analyzer on field cores and laboratory-compacted samples. Therefore, the proposed method may be used for quick evaluation of mixes for fatigue in addition to other methods.

7.1.2 Performance of Mixes Containing RAP and/or RAS

1. The dynamic modulus values of the asphalt mixes increased with an increase in amount of replaced virgin asphalt binder by the asphalt binder from RAP and/or RAS. This may result in a better rutting performance of the mixes with a higher percentage of RAP and/or RAS. Aged asphalt binder as found in RAP and RAS was known to be responsible for increasing the stiffness of the mix, resulting in higher dynamic modulus values.
2. The creep compliance values of the asphalt mixes decreased with an increase in the amount of replaced virgin asphalt binder by the asphalt binder from RAP and/or RAS. This may result in a higher susceptibility to thermal cracking as a result of decreasing relaxation modulus with a decrease in creep compliance. This was attributed to increasing the mix stiffness with using more aged and therefore stiffer asphalt binder from RAP and RAS.
3. The effect of the amount of RAP and/or RAS on dynamic modulus and creep compliance values was more pronounced for the surface course mixes (NMAS = 12.5 mm), than that of base course mixes (NMAS = 19.0 mm).
4. Fatigue life was found to increase with increasing RAP content up to 25%, and to decrease when the RAP content exceeded 25%, or when RAS was used in the mix. It should be noted that this conclusion was based on conducting fatigue tests on asphalt mixes with 0%, 25%, and 40% RAP contents. However, the

adverse effect of using RAP on fatigue life may start to occur at a RAP content between 25% and 40%. Therefore, for a more accurate determination of the RAP content which maximizes the fatigue life, testing more mixes with smaller increments in RAP content is recommended (e.g., 25%, 30%, 35%, 40%).

5. The TSR values of the asphalt mixes tested herein were found to be greater than 0.9 except those containing RAS. This observation was not confirmed by the Hamburg wheel tracking test results, in which mixes containing RAS showed a good performance against rutting and moisture-induced damage.
6. Based on the Hamburg wheel tracking test results, it was found that the resistance of the asphalt mixes to rutting and moisture-induced damage increase, with an increase in the amount of RAP and/or RAS used.

7.1.3 Surface Free Energy of WMA Mixes

1. Sasobit® and Advera® additives were found to reduce the total SFE component of the asphalt binder. Evotherm®, on the other hand, increased the total SFE of the asphalt binder.
2. Sasobit®, Advera® and Evotherm® increase the wettability of the asphalt binder over the aggregates, observed as an increase in the spreading coefficient. However, Evotherm® was found to cause a more significant increase in the spreading coefficient for all aggregates, specifically with gravel. This implies a better aggregate coating by asphalt binder.
3. Sasobit®, Advera® and Evotherm® increase the work of adhesion of asphalt binder over the aggregates. Evotherm® was observed to cause a more significant

improvement in the work of adhesion for all aggregates, specifically for gravel. This may result in a more durable asphalt mix.

4. Sasobit®, Advera® and Evotherm® reduce the magnitude of work of debonding of the asphalt binders over the aggregates. Addition of Evotherm® was observed to result in a more significant reduction in the magnitude of the work of debonding, and is expected to possibly lower the moisture susceptibility of the mix.
5. Works of adhesion to debonding ratios were used as indicators of the moisture susceptibility of the asphalt binder-aggregate systems. Based on this method, Sasobit® and Advera® do not significantly increase or decrease the moisture susceptibility potential of the asphalt binder over different aggregates. However use of Advera®-modified asphalt binder with basalt results in a measurable decrease in moisture susceptibility of the mix. Evotherm® was observed to have the maximum effect on the reduction of moisture susceptibility potential.
6. The TSR test was observed (possibly) not to be able to capture moisture susceptibility of the mixes produced using WMA-additives.

7.1.4 Surface Free Energy of Mixes Containing RAP

1. The non-polar SFE component of PG 76-28 asphalt binder was found to be higher than that of PG 64-22 for all RAP contents. This SFE component increases with addition of RAP in higher amounts (i.e., 25% and 40) for both PG 64-22 and PG 76-28 asphalt binders. The acid SFE component (polar component) of both PG 64-22 and PG 76-28 asphalt binders were higher than their base SFE component, and increased with an increase in RAP content; but,

the range of variations for acid SFE component was similar for both types of asphalt binders. However, the base SFE component of both PG 64-22 and PG 76-28 asphalt binders did not change significantly with increasing RAP content.

2. Based on the wettability parameter estimated for different combinations of asphalt binder type, RAP contents, and aggregates, the coating quality of both PG 64-22 and PG 76-22 asphalt binders for different types of aggregates increased with an increase in RAP content (25% and more). The gravel and basalt aggregates showed the highest and the lowest coating quality among the tested aggregates, respectively.
3. The bond strength between aggregates and asphalt binder systems under dry condition was evaluated based on the work of adhesion. It was found that the work of adhesion of the PG 64-22 and PG 76-22 asphalt binders with different types of aggregates increases with an increase in RAP content (25% and more). The improvement in the work of adhesion (under dry condition) was found very low for the granite aggregate. The gravel and basalt aggregates showed the highest and the lowest work of adhesion (in dry condition) among the tested aggregates, respectively, which is consistent with the results obtained for wettability. Use of PG 64-22 asphalt binder resulted in a higher work of adhesion (under dry condition) with different aggregates and RAP amounts. A higher work of adhesion is expected to improve the aggregate-asphalt binder bond strength under dry condition.

4. The debonding potential of aggregate from asphalt binder under wet conditions was evaluated based on the work of debonding. It was found that the work of debonding for both PG 64-22 and PG 76-22 asphalt binders with different types of aggregates decreased with an increase in RAP content (25% and more). The reduction in the work of debonding with increasing RAP content was found to be insignificant for granite and basalt aggregates. Gravel and basalt aggregates showed the highest and the lowest work of debonding among the other aggregates, respectively. A higher work of debonding is expected to increase the separation potential of asphalt binder from aggregate in the presence of water. However, this conclusion should be discussed in conjunction with the wettability and work of adhesion to make a sound judgment on stripping potential.
5. Overall, the energy ratio parameters (ER_1 and ER_2), as mechanistic indicators of resistance to moisture-induced damage, consistently increased with an increase in RAP content for both PG 64-22 and PG 76-28 asphalt binders and all types of aggregates. Based on the ER_1 and ER_2 values, use of polymer-modified asphalt binder (PG 76-28) was found to increase the moisture-induced damage potential at lower RAP contents (0% and 10%) compared to non-polymer-modified asphalt binder (PG 64-22). At higher RAP contents (25% and 40%) the improvement in resistance to moisture-induced damage was found to be similar for both types of asphalt binders (PG 64-22 and PG 76-28). For different amounts of RAP and different asphalt binder types (PG 64-22 and PG 76-28)

basalt and gravel aggregates showed the highest and the lowest resistance to moisture-induced damage, respectively.

6. It was found that the higher the total SFE of the aggregates, the lower the ER_1 and ER_2 values. Therefore, a high total SFE component of aggregate may result in a high moisture-induced damage potential in the mix.

7.1.5 Micro-structural study of Asphalt Mixes Containing RAP

1. Based on the outcomes of this study, the recommended practice for evaluation of the moisture-induced damage potential of asphalt mixes is a combined use of SFE approach and traditional testing methods (e.g., HWT and TSR).
2. A methodology was developed to combine the SFE components and interfacial energy parameters of asphalt mix ingredients with JMF of the mix for moisturize-induced damage evaluation of the mixes containing RAP. It was found that CWA and CWD alone are not capable of identifying the quality of a mix to resist moisture damage. Therefore, CER was introduced and was found to be capable of predicting the moisture-induced damage potential of the mixes tested.
3. Based on the CER values, resistance to moisture-induced damage increased with an increase in amount of RAP used in the mixes tested in this study.
4. The TSR test results showed that the resistance of the mixes to moisture-induced damage increases with an increase in amount of RAP used in the mixes. This improvement of the resistance to moisture-induced damage was shown in the form of increasing the TSR value with increase in the amount of RAP. Specifically, the addition of 40% RAP to mix yielded a TSR value of

approximately one, which indicates no tensile strength decay as a result of moisture and temperature conditioning.

5. HWT test results showed improvement in resistance of mixes to moisture-induced damage with addition of RAP to the mixes.
6. Based on the micro-structural energy approach, the TSR, and HWT test results, all of the mixes containing RAP which were tested in this study are ranked at the same order in terms of their resistance to moisture-induced damage: the higher the RAP content, the greater the resistance of mixes to moisture-induced damage.
7. Based on the findings of this study, TSR and HWT show good correlation with CER, however, a detailed study may be conducted to evaluate validity of TSR and HWT for different types of mixes.

7.2 Recommendations

According to the methodology and materials used in this study the most important recommendations are listed as following:

1. It is recommended that the effect of using RAP and RAS on mechanical properties of WMA be studied in detail.
2. The mechanistic input parameters determined in this study can be used for the prediction of fatigue, rutting and low temperature cracking in asphalt pavements involving similar mixes. Future studies on this topic can involve either M-EPDG or DARWIN-ME software.
3. In this study, a high quality RAP from one source was used, and the effect of variation in RAP source on the mix properties was not addressed. It is

recommended that the effect of RAP and RAS source on the performance of asphalt mixes, specifically that of fatigue and moisture-induced damage, be studied in future.

4. In the present study, a PG 64-22 binder was used in preparing all mixes involving RAP and/or RAS. A separate study may be undertaken using a softer binder (e.g., PG 58-28) to compensate for the stiffer binders from RAP and/or RAS used to replace the virgin binder.
5. In the present study, effect of aggregate source and geology on performance of WMA mixes as well as mixes containing RAP and/or RAS was not addressed. A future study can address these aspects.
6. It is recommended that the compatibility of different additives (i.e., WMA additives, anti-stripping agents, lime) and asphalt binders be studied with the different types of the aggregates used in WMA mixes, against moisture-induced damage.
7. Conducting Hamburg wheel tracking and/or rut and moisture susceptibility tests over mixes using asphalt pavement analyzer and comparing their results with TSR and SFE-based moisture susceptibility is recommended.
8. Developing correlations between the SFE-based energy ratio and tensile strength ratio (TSR) is recommended. For developing a significant and valid correlation, a larger database of test results on asphalt binder, aggregate and performance tests on asphalt mixes are required.
9. SFE components of the asphalt binder are expected to change with changing its source, due to variability in chemical composition of crude oil. Therefore, the

use of asphalt binders from different sources and adding them to the test matrix and developing a database for the local materials is recommended.

10. Additional studies on the effect of polymer-modified asphalt binders with different PG-plus grades on moisture-induced damage potential, using the SFE method, is recommended.
11. Finally, performance tests on the asphalt mixes using the aggregates and asphalt binders tested herein, are recommended to cross-check the results from the SFE method with those obtained from laboratory mix testing.

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APPENDIX

A

LIST OF PUBLICATIONS

Refereed Journal Papers

Ghabchi, R., Singh, D., and Zaman, M. (2014). "Laboratory Evaluation of Stiffness, Low Temperature Cracking, Rutting, Moisture Damage, and Fatigue Performances of WMA Mixes," *Journal of Road Materials and Pavement Design* (Under Review).

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