Capturing Aggregate and Binder Interaction Effects on Aging Via Mixture Testing for Single Aggregate Source Asphalt Mixtures

Bradley Scott Hansen

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Capturing aggregate and binder interaction effects on aging via mixture testing for single aggregate source asphalt mixtures

By

Bradley Hansen

A Thesis
Submitted to the Faculty of
Mississippi State University
in Partial Fulfillment of the Requirements
for the Degree of Master of Science
in Civil Engineering
in the Department of Civil and Environmental Engineering

Mississippi State, Mississippi

December 2017
Capturing aggregate and binder interaction effects on aging via mixture testing for single aggregate source asphalt mixtures

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Pages in Study: 57
Candidate for Degree of Master of Science

This thesis compares asphalt mixture properties before and after one year of field aging. Simple mixtures, one aggregate source for coarse, fine, and dust proportions, were developed and tested using four mixture tests to isolate asphalt-aggregate interaction. The results found asphalt-aggregate interaction effects and mixing temperature had considerable effects on mixture aging after 1 year. Differences in mass loss, rut depth, and indirect tensile strength that existed before aging became more pronounced after aging. This thesis’ results agree with the literature reviewed and generates new knowledge for mixture design and materials selection considerations for improved pavement performance. It is recommended that mixture conditioning be used with an appropriate mixture conditioning protocol to more accurately categorize mixture ingredients effects on aging.
DEDICATION

I would like to dedicate this thesis to my parents Edward and Cindy Hansen. They are the driving force behind my education as well as my success. Without my father displaying the qualities of hard work and dedication, I would not be where I am today. Without my mother always supporting and pushing me, I would have surely quit by now.
ACKNOWLEDGEMENTS

Acknowledgements are due to the Mississippi Department of Transportation for funding the project contained within this thesis. Thanks are due to my major professor and committee Dr. Isaac Howard, Dr. Seamus Freyne, Dr. Robert Moser, and Dr. Tonya Stone. Special thanks are due to Patrick Kuykendall, Alex Middleton, and Braden Smith for items such as testing, data reduction, and overall project support.
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<th>Definition</th>
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<tr>
<td>AASHTO</td>
<td>American Association of State Highway Transportation Officials</td>
</tr>
<tr>
<td>Abs</td>
<td>Water absorption</td>
</tr>
<tr>
<td>AI</td>
<td>Asphalt Institute</td>
</tr>
<tr>
<td>APA</td>
<td>Asphalt Pavement Analyzer</td>
</tr>
<tr>
<td>CAA</td>
<td>Coarse Aggregate Angularity</td>
</tr>
<tr>
<td>CML</td>
<td>Cantabro Mass Loss</td>
</tr>
<tr>
<td>D:B</td>
<td>Dust to Binder ratio</td>
</tr>
<tr>
<td>DGA</td>
<td>Dense Graded Asphalt</td>
</tr>
<tr>
<td>DOT</td>
<td>Department of Transportation</td>
</tr>
<tr>
<td>DSR</td>
<td>Dynamic Shear Rheometer</td>
</tr>
<tr>
<td>FAA</td>
<td>Fine Aggregate Angularity</td>
</tr>
<tr>
<td>G_{mn}</td>
<td>Maximum mixture specific gravity</td>
</tr>
<tr>
<td>G_{sa}</td>
<td>Aggregate apparent specific gravity</td>
</tr>
<tr>
<td>G_{sb}</td>
<td>Aggregate bulk specific gravity</td>
</tr>
<tr>
<td>G_{se}</td>
<td>Aggregate effective specific gravity</td>
</tr>
<tr>
<td>GR</td>
<td>Gravel</td>
</tr>
<tr>
<td>GTR</td>
<td>Ground Tire Rubber</td>
</tr>
<tr>
<td>HLWT</td>
<td>Hamburg Loaded Wheel Tracking</td>
</tr>
<tr>
<td>HMA</td>
<td>Hot Mix Asphalt</td>
</tr>
<tr>
<td>IDT</td>
<td>Indirect Tensile</td>
</tr>
<tr>
<td>IGTC</td>
<td>Inverse Gas Liquid Chromatography</td>
</tr>
<tr>
<td>LPO</td>
<td>Low Pressure Oxidation</td>
</tr>
<tr>
<td>LS</td>
<td>Limestone</td>
</tr>
<tr>
<td>LTOA</td>
<td>Long Term Oven Aging</td>
</tr>
<tr>
<td>M_{R}</td>
<td>Resilient Modulus</td>
</tr>
<tr>
<td>MDOT</td>
<td>Mississippi Department of Transportation</td>
</tr>
<tr>
<td>ΔML</td>
<td>Change in mass loss</td>
</tr>
<tr>
<td>M_{L}</td>
<td>Mass loss</td>
</tr>
<tr>
<td>MRL</td>
<td>Material Reference Library</td>
</tr>
<tr>
<td>NCHRP</td>
<td>National Cooperative Highway Research Program</td>
</tr>
<tr>
<td>NMAS</td>
<td>Nominal Maximum Aggregate Size</td>
</tr>
<tr>
<td>OGFC</td>
<td>Open Graded Friction Course</td>
</tr>
<tr>
<td>P_{12.5-HLWT}</td>
<td>Hamburg Loaded Wheel Tracking passes to reach 12.5 mm rut depth</td>
</tr>
<tr>
<td>P_{b}</td>
<td>Binder percent by mixture mass</td>
</tr>
<tr>
<td>P_{ba}</td>
<td>Percent of absorbed binder</td>
</tr>
<tr>
<td>P_{ba(mix)}</td>
<td>Absorbed binder percent by mixture mass</td>
</tr>
<tr>
<td>P_{be}</td>
<td>Effective binder content</td>
</tr>
<tr>
<td>PAV</td>
<td>Pressure Aging Vessel</td>
</tr>
<tr>
<td>PG</td>
<td>Performance Grade</td>
</tr>
<tr>
<td>Symbol</td>
<td>Definition</td>
</tr>
<tr>
<td>--------</td>
<td>------------------------------------------------</td>
</tr>
<tr>
<td>$R^2$</td>
<td>Coefficient of Determination</td>
</tr>
<tr>
<td>RAP</td>
<td>Reclaimed Asphalt Pavement</td>
</tr>
<tr>
<td>RAS</td>
<td>Reclaimed Asphalt Shingles</td>
</tr>
<tr>
<td>$\Delta RD_{APA}$</td>
<td>Change in Asphalt Pavement Analyzer rut depth</td>
</tr>
<tr>
<td>$RD_{APA}$</td>
<td>Asphalt Pavement Analyzer rut depth</td>
</tr>
<tr>
<td>$\Delta RD_{HLWT}$</td>
<td>Change in Hamburg Loaded Wheel Tracking rut depth</td>
</tr>
<tr>
<td>$RD_{APA}$</td>
<td>Hamburg Loaded Wheel Tracking rut depth</td>
</tr>
<tr>
<td>REOB</td>
<td>Re-refined Engine Oil Bottom</td>
</tr>
<tr>
<td>RTFO</td>
<td>Rolling Thin Film Oven</td>
</tr>
<tr>
<td>$\Delta S_t$</td>
<td>Change in tensile strength</td>
</tr>
<tr>
<td>$S_t$</td>
<td>Tensile strength</td>
</tr>
<tr>
<td>SAS</td>
<td>Single Aggregate Source</td>
</tr>
<tr>
<td>SGC</td>
<td>Superpave Gyratory Compactor</td>
</tr>
<tr>
<td>SHRP</td>
<td>Strategic Highway Research Program</td>
</tr>
<tr>
<td>SIP</td>
<td>Stripping Inflection Point</td>
</tr>
<tr>
<td>T$_{Mix}$</td>
<td>Asphalt mixing temperature</td>
</tr>
<tr>
<td>TFO</td>
<td>Thin Film Oven</td>
</tr>
<tr>
<td>TFOT</td>
<td>Thin Film Oven Test</td>
</tr>
<tr>
<td>WEO</td>
<td>Waste Engine Oil</td>
</tr>
<tr>
<td>WMA</td>
<td>Warm Mix Asphalt</td>
</tr>
<tr>
<td>$V_a$</td>
<td>Air voids</td>
</tr>
<tr>
<td>$V_{be}$</td>
<td>Effective binder volume</td>
</tr>
<tr>
<td>VMA</td>
<td>Voids in Mineral Aggregate</td>
</tr>
<tr>
<td>VFA</td>
<td>Voids Filled with Asphalt</td>
</tr>
</tbody>
</table>
CHAPTER I
INTRODUCTION

1.1 Introduction

Since at least the mid 1900’s, asphalt durability and aging has been studied. Asphalt binder-only research constituted the majority of early efforts to categorize asphalt durability, but it was also known that binder is not the only variable that can cause failure (Hveem 1943). Mainly two types of failure occur cohesive (binder failure) or adhesive (asphalt-aggregate bond failure). Binder only research categorizes cohesive failures while adhesive failures can only be categorized with the usage of binder and aggregates together (a complete asphalt mixture).

Current research, which is presented later in the Literature Review, has consistently found aggregates, hydrated lime, and air voids, to name a few, considerably effect aging and durability. As such, asphalt mixtures can contain the following: limestone, crushed gravel, sand, hydrated lime, warm mix technology, reclaimed asphalt pavement (RAP), reclaimed asphalt shingles (RAS), re-refined engine oil bottom (REOB), sulfur additives, ground tire rubber (GTR), acids, waste engine oil (WEO), etc. The individual effects of these ingredients can be challenging to determine, but adding them all in one mixture makes it even more difficult to determine singular ingredient effects. This is important in situations where a mixture experiences problems since finding the issue in a mixture with
a large number of ingredients becomes especially difficult. After a mixture ages the effects may become even harder to differentiate.

Aging is currently of considerable interest to asphalt paving because of the increase in non-load associated cracking related to Superpave design principals causing lower binder content mixtures (informally called “dry mixes”) (Howard et al. 2016). Once aging occurs, the pavement becomes more brittle (i.e., more susceptible to cracking) so there exists a need for a mixture test(s) to capture aging. There is also a need for mixture conditioning protocols to accurately simulate long term field aging, but this need is not of concern for this thesis. American Association of State Highway Transportation Officials (AASHTO) R30 (2014) is the most common mixture conditioning protocol (long term oven aging) and is said to simulate 7 to 10 years of field aging, but this simulated aging time has been questioned in recent years (e.g. Isola et al. 2014, Yin et al. 2016). Binder aging protocols, such as AASHTO R28 (2014) (pressure aging vessel), are useful but limited because the mixture ingredients can meaningfully affect aging. In other words, binder aging cannot fully capture mixture aging in all situations.

This thesis used mixtures with essentially the same volumetrics and one aggregate source for the coarse, fine, and dust portions. One performance graded PG 67-22 binder and two warm mix additives (Sasobit® and Evotherm 3G™) were used with the aggregates to form ten mixtures (M01-M10) with a maximum of three ingredients. The majority of mixtures contained only binder and aggregates. With such few ingredients, differences between mixtures can be attributed to asphalt-aggregate interaction effects before and after one year of field aging. If meaningful differences are found between mechanical mixture
properties after aging this thesis supports mixture tests being a necessity for fully capturing aging of in service asphalt pavements.

Most of this thesis is, for all practical purposes, the same as a paper submitted for peer-review (Hansen and Howard 2018) with the main difference being an expanded literature review is presented in the thesis. Expansion also exists in the results with a greater number of figures provide more clarity to the project’s findings.

1.2 Objectives and Scope

The objective of this thesis is to determine, in an asphalt mixture with minimum ingredients and constant mixture variables, if differences can be detected in mechanical mixture tests before or after field aging. If differences exist than they can only be attributed to asphalt-aggregate interaction effects. The Literature Review presents relevant studies which give information on asphalt bonding, aging, and asphalt-aggregate interaction effects. By using the literature review findings and thesis results, conclusions can be ascertained which should provide clarity on aggregate effects on mixture aging when there is only aggregate and binder.
CHAPTER II
LITERATURE REVIEW

2.1 Overview of Literature Review

This literature review contains sections on asphalt bonding, simple mixture test methods, binder test methods, and hydrated lime. The literature review is assembled by subject and then referenced source in chronological order. The first purpose of this review is to investigate aggregate bonding and ways to improve bonding between asphalt cement and aggregates. The second purpose is an investigation of asphalt-aggregate interactions with respect to bonding and aging. The third purpose is to review relevant mixture test methods, binder testing methods, and hydrated lime. The information in this review should give the reader background understanding for results interpretations later in the thesis.

2.2 Asphalt Bonding Literature Review

The following discussion of asphalt bonding is divided into 3 sub-sections. The first sub-section discusses historical research on asphalt. The second discusses mixture aging and durability. The last sub-section is the bulk of reviewed literature which discusses asphalt-aggregate interaction. The literature reviewed encompasses binder, mixture, and asphalt-aggregate interaction research along with aging. All topics are of some interest for use in the results section, but asphalt-aggregate interaction is the most pertinent for this thesis.
Asphalt bonding is directly related to adhesion of asphalt binder to the surface of an aggregate. Stripping is the most common issue leading to debonding, or adhesion loss, which only occurs when moisture is present. Stripping is the main mechanism of debonding between asphalt cement or asphalt binder and the aggregate surface via water displacing the weak bonds between the aggregate and binder. A brief introduction to stripping, asphalt-aggregate adhesion, and adhesion promoters is presented in the next paragraph.

According to a technical bulletin by Akzo Nobel (2010) on adhesion, multiple factors contribute to the problem of asphalt stripping which is the main bonding failure mode of asphalt mixtures. Asphalt, in general, has low polarity and chemical affinity for aggregate surfaces, but the aggregate has a high affinity for water which can lead to bonding issues. The aggregate-water attraction is somewhat related to aggregate composition. For example, Akzo Nobel (2010) states acidic aggregates (siliceous) generally display more bonding issues in the presence of moisture than due basic aggregates (carbonic). Pavements at a higher risk for developing bonding problems may have the following characteristics: low binder content, high voids, inadequate drainage, high clay fines content or dusty aggregate surfaces.

To solve adhesion issues in asphalt mixtures, Akzo Nobel (2010) recommends adhesion promoters which can be added directly into the binder, most commonly, or as a precoating to the aggregates. Adhesion promoters work by chemically altering the surface of the aggregates, which displaces weakly bonded components usually water, to form a stronger chemical bond between the aggregate and asphalt binder. If adhesion promoters
are effectively used the strength and durability of asphalt pavements can be sufficiently increased (Akzo Nobel 2010).

2.2.1 **Asphalt Durability Research Before 1980**

Historically asphalt aging has often been viewed as predominately a binder issue, but Hveem (1943) insisted not all asphalt failures are binder related and durability research should strive to identify what part of the asphalt mixture is responsible for the issue (i.e. asphalt binder, asphalt-aggregate interaction, or aggregate). Vallerga et al. (1957) attempted to capture some of the factors that influence aging of asphalt binder. These attempts by Vallerga et al. (1957) found that binder hardens through multiple mechanisms: oxidation, thixotropy, volatilization, syneresis, polymerization, and separation which can be caused by different forms of radiation and the aggregate mineralogy.

In a comprehensive National Cooperative Highway Research Program (NCHRP) report on asphalt pavements, Finn (1967) reported low temperature ductility was a good durability indicator for liquid asphalt. The report found a general consensus that low penetration, low ductility, or high softening point leads to poor performance. Finn (1967) completed a comprehensive literature review on asphalt durability and the findings relevant to this document can be found in Table 2.1.

Finn (1967) explained gradation and particle shape were the drivers of aggregate degradation or durability. The author noted a small amount of literature exists concerning aggregate durability and its effect on asphalt mixture performance; however, the author believed the lack of research implies the test methods, of the time period, were adequate in capturing aggregate effects. Another facet of aggregate effects in asphalt deals with the asphalt-aggregate interaction, specifically adhesion of asphalt to the aggregate’s surface.
Multiple sources reviewed by Finn (1967) detailed moisture as the number one concern with aggregate-asphalt adhesion. Beyond moisture, other aggregate effects are composition and chemical reactivity which can lead to stripping. Zube and Cechetini (1965) reported absorptive aggregate can cause non-load associated cracking in asphalt pavements due to daily expansion and contraction cycles.

Plancher et al. (1976) used an aging index to characterize asphalt hardening, the effect of hydrated lime, and aggregate properties. It was determined hydrated lime treatment reduced aging indices with all 4 aggregate sources (2 limestone, 1 quartzite, and 1 granite); furthermore, one limestone source had a similar effect in reducing asphalt hardening. Investigation of effects on the resilient modulus ($M_R$) showed aggregate type had a significant effect regardless of hydrated lime treatment. The effect was attributed to the physical properties of the aggregate. The softer aggregates were crushed more during compaction to produce more aggregate interlock increasing $M_R$ values. The authors noted aggregate polarity may have contributed to a reversible hardening process on the asphalt which can increase the modulus values, but did not further investigate the reversible hardening. The authors found hydrated lime appeared to remove polar groups from the asphalt which would generally oxidize and harden the asphalt. Also, the hydrated lime treatment reduced the viscosity indicating the hydrated lime removes viscosity building components. The overall conclusions were hydrated lime reduced oxidation by removing oxidation catalysts and polar molecules which can increase viscosity.
Table 2.1  Summary of Literature from Finn (1967)

<table>
<thead>
<tr>
<th>Reference</th>
<th>Variable of Interest</th>
<th>Key Findings</th>
</tr>
</thead>
<tbody>
<tr>
<td>Traxler (1963)</td>
<td>Asphalt Hardening</td>
<td>Added 9 additional aging factors to Vallerga et al. (1957): light (multiple effects), water, chemical reaction with aggregate, microbiological deterioration, asphalt adsorption into aggregate surface.</td>
</tr>
<tr>
<td>Krehma (1958)</td>
<td>Asphalt Hardening</td>
<td>Described 4 mixture properties that drive asphalt weathering: air voids, permeability, film thickness, and aggregate surface effects.</td>
</tr>
<tr>
<td>Doyle (1958)</td>
<td>Asphalt durability</td>
<td>Found low-temperature ductility to be a good indicator for asphalt durability.</td>
</tr>
<tr>
<td>Doyle (1963)</td>
<td>Asphalt durability</td>
<td>Discovered an unnecessary loss in ductility when asphalt is overheated during mixing with aggregate.</td>
</tr>
<tr>
<td>Hveem et al. (1959)</td>
<td>Asphalt durability</td>
<td>Different sources and production can yield varying durability properties. Mix design and construction requirements influence performance, but durable asphalts can perform well in unfavorable conditions.</td>
</tr>
<tr>
<td>White (1961)</td>
<td>Time and Traffic effect on asphalt surface</td>
<td>Higher asphalt content reduces cracks, but rutting is considerable. Little aggregate degradation over 10 years was observed. Aggregate absorption of asphalt lead to dry mixes leading to non-load associated cracking. Concluded a need for increased density requirements to reduce rutting.</td>
</tr>
<tr>
<td>Skog and Zube (1963)</td>
<td>Adhesion</td>
<td>Suggested four tests to measure adhesion: Quantitative dye stripping test, moisture vapor susceptibility test, water susceptibility test, and surface water abrasion test. The results determined aggregate source is the most important factor in the performance of mixtures tested and can override all other efforts to achieve a mix with satisfactory performance.</td>
</tr>
<tr>
<td>Morgan (1957)</td>
<td>Construction effects</td>
<td>Durability and stability of asphalt surface is directly related to densities obtained during construction. No direct relationship existed between stripping and mix instability. No direct relationship was developed between type of aggregate (stone or gravel) and pavement performance.</td>
</tr>
</tbody>
</table>
Copas and Pennock (1979) completed NCHRP Synthesis 59 which investigated asphalt-aggregate interaction and asphalt durability. The authors found asphalt-aggregate interaction can lead to asphalt hardening through adsorption of certain asphalt components. The authors noted to ensure asphalt durability that one needed to select a correct asphalt based on loads and environmental conditions as well as an asphalt minimally susceptible to hardening during plant mixing and construction. The authors provided multiple tests for investigating plant mix hardening and recommended further research using gas-liquid chromatography to further study asphalt-aggregate interaction which can affect pavement durability.

2.2.2 Asphalt Mixture Aging and Durability

Petersen (1984) completed a study on asphalt durability relative to chemical composition. The author found that oxidation and hardening is greatly increased when polar, oxygen containing functional groups are formed. It was concluded that composition information can be used in order to match asphalt-aggregate systems and diagnose potential issues. Welborn (1984) performed a similar study, but on the physical properties of asphalt. The author concluded thin-film oven tests are good as an initial screening of asphalts which show large changes of physical properties at construction temperatures, but more research is needed on physical property changes of the in service pavements.

Kemp and Sherman (1984) investigated asphalt durability in California. The authors found different sources and manufacturing affects asphalt durability within the same climate and load conditions. The authors determined thin film oven tests are good at
showing aging during and shortly after construction, and aged residue viscosity grading systems can differentiate construction aging from long term aging. Air voids and air temperature were determined to be detrimental components to asphalt aging and durability. To combat air voids effect on asphalt durability, the authors recommend adherence to compaction requirements to reduce air voids. The last recommendation given by Kemp and Sherman (1984) was to avoid using absorptive aggregates in hot climates.

Kandhal and Koehler (1984) investigated asphalt durability in Pennsylvania. The authors determined stiffness modulus was a good indicator of low temperature cracking, and that high ductility leads to more durable asphalt pavements. Thin film oven tests were investigated and were found unreliable in determining long term performance. As determined by various other references, Kandhal and Koehler (1984) also noted high air voids can be detrimental to performance and recommended to keep air voids below 5%.

The Strategic Highway Research Program published a study by Anderson et al. (1994) on asphalt aging. Anderson et al. (1994) attempted to find a test that best simulates long term aging. The pressure aging vessel (PAV) was determined to best simulate long term field aging while being simple, efficient, and practical. PAV testing is performed on binders.

Won et al. (2008) indicated aging of an asphalt pavement occurs well below the top inch of the pavement. Furthermore, the authors concluded temperature and oxygen availability do not change as much as previously thought. By investigating accessible air voids (or voids which can be accessed by water), it was determined that the top of the pavement does generally age more, but the differences with depth are not as large as might
be expected; however, accessible air void variability within the pavement can dominate effects of temperature with depth. Won et al. (2008) concluded tight inaccessible air voids leads to slower asphalt hardening.

2.2.3 Aggregate Effects on Asphalt Aging and Durability

2.2.3.1 Bell (1989)

In a Strategic Highway Research Program (SHRP) summary report, Bell (1989) investigated aging of asphalt-aggregate systems. Bell (1989) completed a comprehensive literature review on binder aging studies, mixture aging studies, and some test roads. For binder aging studies it was found that Thin Film Oven Tests (TFOT) are good at capturing aging during mixing and construction, but are not useful for long term aging.

Bell (1989) reported hydrated lime is effective against aging, permeability is the best predictor of resistance to hardening (for dense graded mixtures), and cohesion loss can occur if elevated temperatures are used in an aging procedure. Bell (1989) discussed the findings from the Zaca-Wigmore test road (and other California test roads) as well as test roads in Michigan and Texas. The Zaca-Wigmore test road found asphalt hardening of TFOT to have excellent correlation with plant mixing. In addition, the asphalt hardening was found to be asphalt source dependent. Inverse gas liquid chromatography (IGTC) was used to find phenol retention time. The phenol retention time is a measure of concentration of polar functional groups within the asphalt. The phenol retention time correlated well with performance ratings at 51 months of service. At some other California test roads, it was found that a high average temperature is the most significant factor affecting asphalt
hardening. Void content effect was a contributing factor and was similar between asphalt sources. Aggregate absorption was a factor which was more significant in more volatile asphalts. Recommendations from the California road tests were to follow compaction requirements while avoiding absorptive aggregates.

2.2.3.2 Tarrer and Wagh (1991)

Tarrer and Wagh (1991) completed a SHRP report on physical and chemical characteristics of asphalt-aggregate bonding. The authors detailed five asphalt stripping mechanisms: detachment, displacement, spontaneous emulsification, pore pressure, and hydraulic scouring. Only the first two mechanisms are of interest for this literature review. Detachment is defined as separation of asphalt film from aggregate with no apparent break in asphalt film. A detachment failure indicates complete loss of adhesion between the asphalt and aggregate. Aggregate polarity may be attributed to detachment in circumstances where a highly polar aggregate displays a high affinity for water which could displace the low polarity asphalt causing complete debonding of the asphalt from the aggregate. Displacement is defined as penetration of water to the aggregate surface through a hole in the asphalt film. Displacement failures can be caused by sharp edges from angular aggregates.

Tarrer and Wagh (1991) found multiple aggregate properties affect asphalt moisture susceptibility such as chemical composition, mineralogy, shape, texture, and porosity. Acidic aggregates tend to be hydrophilic (water-loving) which leads to more moisture damage while on the other end basic aggregates tend to be hydrophobic (water-
hating) which leads to less moisture damage. The surface texture of aggregate can affect aggregate compatibility (i.e. smooth aggregates are easier to coat than an angular aggregate). As such Tarrer and Wagh (1991) note lab tests indicate stripping to be more severe with angular aggregates. Aggregate porosity seems to promote adhesion by giving a better mechanical interlock between binder and aggregate. Freshly crushed rock has been shown to perform worse than a crushed rock that has been stockpiled for a period of time. Tarrer and Wagh (1991) reviewed Thelen (1958) which reported aggregates become covered with organic fatty acids and oils which reduces reactive sights on the aggregate surface and leads to less stripping issues. On the other hand, clay, silt, and dust coatings from crushing have been shown to worsen stripping. Specifically, dusty coatings cause less intimate contact between aggregate and asphalt which promotes stripping.

Tarrer and Wagh (1991) recommend preheating aggregate before binder is introduced in order to drive off water vapor from the surface to promote resistance to stripping. Hydrated lime in a slurry form is also an effective antistripping agent after curing. The hydrated lime is thought to absorb carboxylic acids and make calcium salts which results in a water resistant asphalt-aggregate bond.

2.2.3.3 Curtis (1992)

Curtis (1992) investigated asphalt-aggregate interactions with respect to chemistry and physical properties. The chemistry of the interaction causes the polar oxygen containing functional groups of the asphalt to be adsorbed to the aggregate surface more strongly than the liquid asphalt. The polar components that adsorb to the aggregate surface
tend to show the most sensitivity to water which weakens the asphalt-aggregate bond. The author notes natural dust on the surface of the aggregate can also lead to adhesion problems. According to Curtis (1992), the aggregate chemistry is much more influential than asphalt chemistry for adhesion and water sensitivity. In essence it was concluded that aggregate properties are more influential in determining adsorption and stripping behavior. In order to reduce asphalt-aggregate bond issues, Curtis (1992) recommends reducing the water availability by having good pavement drainage and low air voids.

2.2.3.4 Bell and Sosnovske (1994)

Bell and Sosnovske (1994) completed a SHRP report on aging protocols for asphalts. The authors used the following Material Reference Library (MRL) coded binders (the equivalent grades can be found in parentheses): AAA-1 (150/200), AAB-1 (AC-10), AAC-1 (AC-8), AAD-1 (AR-4000), AAF-1 (AC-20), AAG-1 (AR-4000), AAK-1 (AC-30), and AAM-1 (AC-20). The aggregates were MRL coded as the following: RC (high absorption limestone), RD (low absorption limestone), RH (greywacke), and RJ (conglomerate). The different combinations of asphalt and aggregates were subjected to short term aging (4 hours at 135°C) and then one of the four long-term aging protocols: low pressure oxidation (LPO) at 60°C and 85°C for 5 days, and long term oven aging (LTOA) at 85°C for 5 days and 100°C for 2 days. Unaged specimens were compacted directly after mixing for later comparison to the aged specimens. Resilient modulus, dynamic modulus, and tensile strength were used to evaluate the mixtures.
By using statistical groupings, Bell and Sosnovske (1994) ranked the mixtures based on short-term or long-term ratios. Bell and Sosnovske (1994) found short and long term aging is aggregate dependent, but asphalt effects are more significant. With respect to the aggregate dependency, the authors hypothesized that greater adhesion between asphalt and aggregate yields greater mitigation of aging. Bell and Sosnovske (1994) compared rankings of short and long term aged mixtures with asphalt binder rankings. The authors found little relationship between short or long term aged mixture and binder rankings with long term aging showing even less similar rankings than short term aging. The results led Bell and Sosnovske (1994) to conclude aging susceptibility is a mixture problem and not just a binder issue to the effect that binder aging alone does not adequately predict the aging of the mixture. The report also details the aggregate source displays variability with respect to age mitigation effects. The short term aging procedure showed little aggregate differences on aging, but long term aging showed different results indicating short term aging does not predict long term performance. The report gave some recommendations for aging protocols to help adequately predict asphalt aging.

2.2.3.5 Abo-Quadais and Al-Shweily (2007)

Abo-Quadais and Al-Shweily (2007) investigated aggregate effects on stripping and creep behavior of asphalt mixtures by using crushed limestone and basalt as aggregates with two grades of asphalt. The authors did not mention use of sand and said the aggregates were manufactured so it is assumed only one aggregate source is used for this study. Three different gradations were used with both aggregates to test gradation effects. Some creep
specimens were moisture conditioned, before testing, for 10 minutes in a vacuum and then exposed to \(-18 \pm 3^\circ\text{C}\) for 16 hours followed by 24 hours of thawing at 60\(^\circ\text{C}\). From the literature, Abo-Quadais and Al-Shweily (2007) reported antistripping additives significantly reduce stripping, adding portland cement can increase stability and flexural resistance, and chemical additives did not outperform hydrated lime.

Abo-Quadais and Al-Shweily (2007) observed that unconditioned limestone aggregate specimens showed higher creep values with the same trend regardless of asphalt type or gradation. After conditioning, basalt aggregate specimens displayed higher creep values with the same trend regardless of asphalt type or gradation. Regardless of conditioning, limestone mixtures showed higher stability values. The authors concluded the following: stripping resistance is significantly affected by aggregate type, aggregate gradation has a strong effect on stripping (i.e. less dense mixtures are more prone to stripping), and absorbed asphalt could pick up differences within aggregate type, gradation, and type of asphalt. The authors recommended using higher absorption aggregates, all other properties being equal, to improve stripping resistance.

**2.2.3.6 Morian et al. (2011)**

Morian et al (2011) observed mixture characteristics affect aging but are hard to differentiate. Morian et al. (2011) mentions even by 1989 asphalt aging studies were conducted solely on binders. The authors noted in the early studies binder viscosity and penetration did not have a strong enough link into mixture aging with respect to binder aging, so studies were performed on binder or mixtures but not both.
2.2.3.7  Baek et al. (2012)

Baek et al. (2012) found the greater aggregate adhesion the greater the mitigation of aging. Baek et al. (2012) expanded on the reported reasons for the gap between binder and mixture testing by comparing the two approaches. Binder aging protocols have standards, but do not account for aggregate effects. Currently there is no best method for mixture aging which makes it difficult to compare results when different aging methods are used. The binder is the main aging constituent which sheds light on the appeal of only researching binder aging, but mixture aging can account for mixture properties which do affect aging, although there was currently no proven best method available according to the authors as of 2012.

2.2.3.8  Cui et al. (2014)

Cui et al. (2014) investigated the effect of aggregate type and adhesion promoters using two basic aggregates (limestone and marble) and two acidic aggregates (both granite). The four adhesion promoters used were two different silanes, an amine anti-stripping agent, and a polymer modified binder (styrene-butadiene-styrene). The polymer modified binder was only used for the limestone aggregate. The test used to characterize the asphalt-aggregate bond was a peel test which was performed before water submersion and after bonded specimens were submerged under water for varying periods of time. The peel test specimen consisted of an aluminum peel arm on top with the asphalt binder in between the peel arm and the 200mm long, 20mm wide, and 10mm thick slab of different aggregates. To conduct the test, the peel arm was pulled back to a peel angle of 90° and the
peel force was recorded and used to determine adhesive fracture energy. The test was conducted at 20 ± 2°C and 50 ± 5% relative humidity.

Cui et al. (2014) found in the dry condition all asphalt-aggregate combinations displayed cohesive failure (failure within the binder). After submersion in water all but the marble aggregate showed adhesive failure (failure between aggregate and binder). Once adhesion promoters were added, higher adhesive fracture energy and more cohesive failures were observed. For the polymer modified binder, fracture energy increased from 319 J/m² to 1330 J/m² in the dry test and 281 J/m² to 1120 J/m² for the submerged test indicating the modified binder improved adhesion and water resistance. Cui et al. (2014) concluded the following: basic aggregates displayed higher water resistance, aggregate chemistry is more important than aggregate porosity, and all three adhesion promoters as well as the polymer modified binder showed positive effects on adhesive strength.

2.2.3.9 Wu et al (2014)

Wu et al. (2014) studied the aggregate effect on bitumen aging by using combinations of unaged binder, aged binder, limestone aggregate, and granite aggregate to compact dense bitumen macadam specimens. The authors did not indicate the use of sand as fine aggregate so it is assumed the mixtures contained only one source of aggregate. The two variables of interest were stiffness modulus and complex modulus. Two methods of specimen fabrication were slab compaction of unaged asphalt and then slab compaction after short term aging (4 hours at 135°C). After slab compaction, the specimens were cored
and then tested for air voids and stiffness. Once initial measurements were taken the cores were long-term aged via R30 (5 days at 85°C).

After testing for stiffness modulus, limestone aggregate was shown to produce a stiffer mixture than the granite aggregate. One possible reason for higher limestone stiffness values given by Wu et al. (2014) is due to limestone’s slightly higher absorption value which would cause more polar asphalt constituents to be absorbed resulting in a stiffer binder. Lower granite stiffness values were attributed to a less continuous granite gradation and different surface friction values resulting in poor aggregate contact and higher air voids. In addition, thicker binder films on the granite could lead to lower stiffness values.

Wu et al. (2014) observed multiple factors concerning aging. More aging was observed in the limestone mixtures during the SHRP long term aging protocol R30. The unaged binder showed a larger portion of aging during the period before R30. Short term aged mixtures displayed an increase in compaction difficulty. Some factors that can affect aging are different air voids and binder film thicknesses, but the authors believe the aggregate type effects are considerably more dominant with aggregate type significantly affecting binder aging and timing of binder aging. Specifically, aggregate type was seen to affect aging in three ways. The first two ways are catylization of bitumen aging with minerals on the surface of the aggregate as well as adsorption of polar components into the aggregate. The latter results in a decrease in binder aging via protection of polar components within the aggregate from aging. The third aggregate effect results from irreversible adsorption of polar asphalt constituents into the aggregate, not recoverable.
through typical means, which leads to a softer recovered binder for the higher absorption limestone.

2.2.3.10 Aguiar-Moya et al. (2015)

Aguiar-Moya et al. (2015) investigated aging and adhesion properties of asphalt mixtures. The authors stated moisture damage is related to the adhesion of binder to the aggregate as well as the cohesion within the asphalt binder. The oxidation of asphalt leads to microcracks that give a direct pathway for water infiltration which may lead to moisture damage. The authors indicated loss of adhesion to unaged binders is dependent on the aggregate source (possibly due to aggregate polarity), binder, and asphalt-aggregate interaction. Aguiar-Moya et al. (2015) concluded that certain aggregate binder combinations can display adhesion issues even with antistripping agents, and to combat adhesion problems aggregate sources which have displayed moisture susceptibility should be avoided.

2.3 Summary of Asphalt Bonding

From the bonding literature review a few points were of most importance to this study. The first is the chemical composition of the aggregate is important. The aggregate can increase or decrease the bond strength. The second point is physical interaction with the aggregates is also important. The surface roughness or texture of the aggregate can affect the bond strength. In a general case, higher surface roughness leads to better bonding. Other physical characteristics of the aggregate which play a role in bonding are porosity, polarity, and shape. The porosity of aggregates in asphalt may lead to beneficial bond
strength increases through an increased mechanical strength as well as a potential issue when water is captured in the pores. The polarity can cause asphalt binder to strip from the aggregate due to water displacing the asphalt. Shape effects can be seen in asphalt because angular aggregates may puncture the asphalt film causing increased potential for stripping, but if not angularity improves stability. After analyzing asphalt bonding, it is apparent that performance of asphalt, mainly strength and durability, is strongly dependent on physical and chemical properties of aggregates.

2.4 Simple Mixture Tests

2.4.1 Indirect Tensile (Non-Instrumented)

Kennedy (1977) summarized the indirect tensile (IDT) test. The IDT test was originally used for concrete, but The University of Texas at Austin began using the test as part of a sponsor report by the Texas State Department of Highways and Public Transportation. The reports came to the conclusion that IDT test was the best practical test for State DOTs to define asphalt material tensile properties. The reasoning behind the test’s practicality was the IDT test was simple, no new equipment was needed, failure was not seriously affected by surface conditions, the tensile strength which developed was fairly uniform, the coefficient of variation was low compared to other tests, and the test could be used under static or dynamic loads. The test can be used to find tensile strength, modulus of elasticity, and Poisson’s ratio depending on the loading conditions. It also provides information on fatigue and permanent deformation characteristics. On top of its simplicity,
IDT testing can be used to assess moisture effects and stripping potential, though over the past few decades questions have arisen regarding practical limits to use of IDT testing.

Anderson et al. (2003) investigated estimation of rutting potential via supervave gyratory compaction (SGC) properties and IDT strength. Literature reviewed by the authors indicated compaction slope can be found from SGC data. Furthermore, the literature found that aggregate characteristics (mainly gradation, particle shape, and texture) dominated rate of compaction irrespective of binder stiffness or properties. The result indicates that compaction slope can be indicative of internal friction but not cohesion which would mean compaction slope does not necessarily relate to rutting resistance. However, IDT strength has been shown to be a good predictor of cohesive strength, but not internal friction and consequently IDT strength would not necessarily relate to rutting resistance. The authors used the knowledge of IDT strength, compaction slope, voids in mineral aggregate (VMA), and statistical analysis to develop a predictive rut depth model with an adjusted $R^2$ of 0.75.

Christensen and Bonaquist (2002) used the indirect tensile (IDT) test as a simple method to evaluate rutting resistance. The authors state strength testing (IDT in this case) is a simpler way to find mixture strength as opposed to stiffness tests which are not able to fully capture mixture strength. Christensen and Bonaquist (2002) note rut resistance is due to internal friction and cohesion. Furthermore, the authors state internal friction’s contribution to rut resistance is overemphasized and mixture cohesion is at least as important if not more important for rutting resistance of asphalt. The authors were able to predict rutting in a pavement with the use of the IDT test. The advantages of the IDT test
are simple expedient testing which can be conducted on gyratory specimens as well as field cores without the need for sawing.

2.4.2 Asphalt Pavement Analyzer (APA) Rutting Susceptibility

Tran et al. (2009) evaluated AASHTO TP 63 (led to AASHTO 340-12 (2014)) rutting test procedures using a piece of equipment known as the Asphalt Pavement Analyzer (APA). The study had four purposes: determine differences in APA rut depth measurements at two combinations of hose pressure and wheel load, investigate interchangeability of manual and automatic rutting measurements, correlation of lab and field rutting, and determining performance criteria based on lab and field rutting for mix design screening and quality assurance. The study was performed at the National Center for Asphalt Technology (NCAT) Pavement Test Track in Opelika, AL. The authors reviewed literature concerning APA testing and the relevant findings for this thesis include: cylindrical and beam specimens can be used, APA rut depths correlated well with field performance when the right loading and environmental conditions were used, APA and Hamburg Loaded Wheel Tracking (HLWT) rut depths correlated well. The study used plant mixes, field cores, and two combinations of hose pressure and wheel load. The mixtures were sampled in the field and immediately compacted at the test track, to mitigate aging concerns, using a SGC to compact to 7% air voids. The air voids of cores were higher since coring occurred near the edge of the pavement. The APA testing was conducted at 64°C which corresponds to the recommended high PG grade for the location of the study. The tests were performed for 8000 cycles with two hose pressure and wheel load
combinations of 120psi and 120lb and 100psi and 100lb, respectively. Automatic and manual rut depths were measured.

Tran et al. (2009) found manual and automated rut depth measurements to be significantly different. Furthermore, it was found that two combinations of hose pressure and wheel load produced significantly different results with the higher loads showing higher rut depth. The automatic rut depths were not significantly different with respect to pressure and load combination, but the authors felt the manual rut depth measurement was a more rational approach. The authors concluded that the 2 combinations of pressure and load correlated well with field rutting performance. The authors note to use caution when using automatic and manual rut depth measurements interchangeably, but the authors were able to determine reasonable rutting criteria using the APA test which correlated well with field performance.

2.4.3 Hamburg Loaded Wheel Tracking (HLWT)

Ashcenbrener (1995) evaluated the HLWT device for prediction of moisture damage of hot mix asphalt (HMA). The author starts by listing the results which can be gathered from a HLWT test. With one HLWT test the following can be found: creep slope, stripping slope, and stripping inflection point (SIP). The author states the creep slope is the inverse of the rate of deformation of the linear region after post compaction effects have ended and before onset of stripping. The stripping slope is the inverse of the rate of deformation of the linear region after stripping begins until the end of the test. The SIP is where the two slopes meet, or the point at which stripping begins. The author also
investigated material properties that influence HLWT results. The author looked at three aggregate properties, specifically, the presence of clay, high dust to binder ratios, and excessive dust coatings. By using the methylene blue test (for clay presence) AASHTO T53 (for dust to binder ratios), and dry sieving (for aggregate dust coatings) the author was able to show aggregates which failed criteria for 2 of the 3 tests good performance with respect to HLWT was unlikely. Aggregates which passed all 3 tests showed good performance. After investigating asphalt cement stiffness, Ashcenbrener (1995) found differences in high-temperature asphalt properties measured by the HLWT were almost identical to differences measured with Dynamic Shear Rheometer (DSR). Short term aging helped against moisture damage. Antistripping treatment was investigated and it was found that both liquid antistripping agents and hydrated lime increase moisture resistance, but hydrated lime showed better improvements than liquid antistripping agents. Ashcenbrener (1995) concluded that the HLWT device showed potential in discriminating between known field stripping performance and gave two examples in which HLWT was used successfully on field projects.

Izzo and Tahmoressi (1999) completed a study on the repeatability of HLWT testing with respect to different devices and laboratories. Five of the seven participating laboratories used a Helmut Wind manufactured HLWT device while the other two laboratories used similar devices. All devices used steel wheels except for one which used a solid rubber wheel at a higher load and a faster cycle rate. The authors used two different coarse aggregates limestone and gravel. The limestone was 100% crushed and was combined with sand while the gravel was 85% crushed and combined with sand and 1%
hydrated lime. The limestone and gravel mixtures used a AC-20 and AC-30P binder, respectively. Slab and SGC compacted specimens were used for HWLT testing. The results compared well over laboratories with respect to the gravel mixtures with all the gravel mixtures performing well most likely due to the added hydrated lime. The authors found no significant added variation when comparing the SGC specimens vs. slab specimens for either gravel or limestone mixes. The gravel mixtures showed no significant difference when comparing the testing devices with respect to slab specimens or SGC specimens. All of the limestone data could not be statistically compared due to considerable rutting from two laboratories tests where some rut depths were not collected past 20mm. The limestone mixtures showed no significant amount of variation when looking at the Helmut Wind device with standard deviations actually decreasing. The poor results from the limestone cannot be compared to the gravel mixtures because polymer modified binder and hydrated lime was used for the gravel mixtures. The authors note that experience shows limestone generally has better moisture susceptibility than gravel. The authors concluded steel wheel tracking devices showed good repeatability with gravel slab compacted mixtures and poor repeatability with limestone slab compacted mixtures. Furthermore, SGC compacted specimens showed no significant difference when compared to slab compacted specimens.

Lee and Kim (2014) investigated performance based testing for moisture susceptibility of warm mix asphalt (WMA). The authors found it necessary to review adhesive failure or failure between the aggregate and asphalt binder. Lee and Kim (2014) found 6 stripping mechanisms with one addition, pH instability, from previous literature reviewed from Tarrer and Wagh (1991). pH instability is a chemical reaction which affects
the adhesion between the binder and aggregate depending on the pH of the surrounding water. The authors note pH values below 4 have been shown to dissolve amine and lime from aggregate surfaces. The relevant result to this thesis is the HLWT test was the second best indicator of moisture damage for WMA pavements with the SIP showing a strong relationship ($R^2$ of 0.84) with corresponding stripping percentage. Lee and Kim (2014) also recommend HLWT tests in a dry condition to differentiate deformation and moisture effects.

Williams and Prowell (1999) compared multiple wheel tracking tests to WesTrack performance. WesTrack is a 2.9km oval test track used for full scale field testing of asphalt pavements. The two tests of interest to this thesis are the APA test and HLWT test. The results showed rutting correlations of APA and HLWT tests to WesTrack performance were approximately 90%. Williams and Prowell (1999) noted more variability existed with poorer performance. The authors stressed the importance of selecting the correct testing temperature.

2.4.4 Cantabro Mass Loss (CML)

Cox et al. (2017) completed a comprehensive evaluation of Cantabro Mass Loss (CML) testing for dense graded asphalt (DGA). The original use of the CML test was for open graded friction courses (OGFC). For information on OGFC uses see the literature review contained within Cox et al. (2017). The authors stress the need for indices of whether an asphalt mixture is (or may become) excessively brittle. The authors successfully used the CML procedure as a durability index to detect performance differences in binder
grade, presence of polymer, difference between ground tire rubber and SBS modifications, aggregate type, recycled asphalt pavement content, density (air voids), volume of effective binder, and aging. Along with the ability of differentiating between performance characteristics the CML test is a simple and expedient way to assess durability. Specifically, any increase in mass loss indicates an increase in brittleness. Cox et al. (2017) concluded with the following key findings: increasing the high PG binder grade increased mass loss until polymer modification was included, CML differentiated binders with ground tire rubber and SBS, RAP content and aggregate type could be detected, intuitive trends were established between mass loss and air voids, and the volume of effective binder was related to mass loss.

2.5 Binder Testing

2.5.1 Thin Film Oven and Rolling Thin Film Oven Tests

As mentioned earlier, TFOT has shown to correlate with plant based aging, but it is necessary to differentiate between the two main TFOT. Zupanick (1994) compared results from thin film oven (TFO) and rolling thin film oven (RTFO) tests. TFO and RTFO are used to simulate short term aging caused by hot plant mixing. The author reviewed previous literature and found most concluded TFO and RTFO are approximately equivalent at standard conditions with some noting RTFO to be slightly more severe. Zupanick (1994) used approximately 5200 TFO and 1800 RTFO repetitions supplied by the AASHTO MRL to compare the two tests. The results showed statistical difference between the two tests with RTFO being more severe. Furthermore, the results indicated the two tests ranked
asphalts differently. The reasoning behind the differences may be attributed to a skin formation and/or higher initial specimen viscosity in the TFO test which can reduce convection and oxygen diffusion into the sample causing less severe aging. The author concludes contrary to industry belief TFO and RTFO tests cannot be used interchangeably. The author also noted that if TFO were to be specified for use the number of asphalt grades would effectively double which leads to the conclusion that RTFO is the better suited test.

2.5.2 Pressure Aging Vessel (PAV)

Bahia and Anderson (1995) reviewed the pressure aging vessel (PAV). The PAV is used for simulating field aging of asphalt binder. The authors found the PAV highly resembled rheological changes that asphalts endure through in service aging. Precautions should be taken based on climate. For example, a PAV testing temperature of 100°C can lead to errors when comparing to cold climates or hot desert climates. Bahia and Anderson (1995) concluded the PAV can be used effectively to show rheological changes in asphalt which happen after aging.

2.6 Hydrated Lime

As mentioned by Plancher et al. (1976), hydrated lime shows beneficial effects with respect to age hardening. Edler et al. (1985) expanded on this by using aging tests to determine hydrated lime effectiveness in reducing oxidative aging. The authors found a 12% addition of hydrated lime retards oxidation more than 6%, but once physical effects of hydrated lime were accounted for both addition levels performed similarly. The authors concluded hydrated lime is effective in retarding oxidation and formation of viscosity.
building constituents with 12% and 6% hydrated lime additions showing similar results. Lesueur and Little (1999) found hydrated lime the amount of reduction in aging/oxidation and mitigation of moisture damage is bitumen dependent. Huang et al. (2002) found hydrated lime increased binder stiffness with a filler effect, reduced oxidation rate, and helped asphalts retain elasticity during aging. Interesting to note is hydrated lime has shown to be more effective then liquid anti-stripping agents (Souliman et al. 2015).

Little and Petersen (2005) discussed the filler effects of hydrated lime on performance-related properties of asphalt cements. Hydrated lime and limestone were compared as fillers and both fillers stiffened the binder with hydrated lime having a greater stiffening effect. In addition, the authors found hydrated lime has a physiochemical reaction with bitumen which did not occur with a limestone filler. The physiochemical reaction reduces oxidative hardening by irreversibly adsorbing reactive components from the asphalt binder. Little and Petersen (2005) note even though hydrated lime treatment increases binder stiffness the same treated binder was more ductile than untreated binders at low temperatures.

Kim et al. (2008) further investigated hydrated lime effects. Currently 1% hydrated lime by weight of aggregates is a typical application amount in HMA in the US. According to Kim et al. (2008), Hydrated lime can be applied in a dry from to dry or wet aggregates as well as in a slurry form. The results showed mitigation of moisture damage with any form of hydrated lime treatment. The best hydrated lime treatment method was dry hydrated lime added to dry aggregates. In addition to improved asphalt-aggregate bonding, hydrated lime acts as a mineral filler stiffening the binder which helps early stage moisture
damage resistance, but when subjected to multiple freeze-thaw cycles this stiffening effect degrades.

2.7 Literature Review Takeaways

Asphalt bonding is considerably affected by aggregates. The aggregate chemistry and physical properties affect asphalt mixture strength and durability. Aggregates play a major role in bonding and stripping behavior. Poor bonding and stripping are heavily affected by moisture. These moisture susceptibility problems can be mitigated, to some extent, with knowledge of aggregate effects on bonding or additives. Hydrated lime and liquid anti-stripping additives are two common methods used to reduce stripping potential. This information indicates asphalt mixture behavior and performance is dependent on the aggregate.
CHAPTER III
EXPERIMENTAL PROGRAM

3.1 Overview of Experimental Program

This chapter details material properties, specimen preparation, and specimen testing. A section discussing reasons for these Single Aggregate Source (SAS) mixtures having high voids in mineral aggregate (VMA) is presented as well. The 10 SAS mixtures (M01-M10) were composed of one asphalt binder, one aggregate source, and in some cases a warm mix additive. Four test methods were used to measure mechanical properties at intermediate and high temperatures: Cantabro mass loss (CML), non-instrumented indirect tensile strength (IDT), Hamburg Loaded Wheel Tracking (HLWT), and Asphalt Pavement Analyzer (APA).

3.2 Overview of Materials

The following section details the materials used for this thesis. Aggregate, binder, and mixture properties are given. The three aggregates used were Hamilton, MS gravel, Creede, CO gravel, and Tuscaloosa, AL limestone. The aggregates were obtained using standard methods and delivered to campus. Hereafter the three aggregates are abbreviated by state and aggregate type (gravel-GR or limestone-LS), e.g. MS-GR denotes Hamilton, MS gravel. The aggregates were chosen based on market use and their wide range in
properties. One binder and two warm mix additives, Sasobit® and Evotherm 3G™, were used as well.

3.2.1 Aggregate Properties

The three aggregate sources MS-GR, CO-GR, and AL-LS are pictured in Figure 3.1. The aggregates were dried, sieved, and recombined to the desired gradation. Fines on the surface of the aggregates were accounted for via a washed gradation in accordance with AASHTO T11 (2014). At no point was sand, hydrated lime, or other material added to the aggregates. The aggregate gradation was formed with one aggregate source for all proportions (coarse, fine, and dust). A gradation was chosen, within AASHTO M323 (2014) and Mississippi Department of Transportation (MDOT) gradation limitations, based on CO-GR because material quantities were limited. Figure 3.2 shows the aggregate gradations and gives the M323 (2014) and MDOT limits. MDOT limitations were used because these SAS mixtures are a part of a future MDOT report. More discussion on the aggregate gradation and its volumetric effects is given in section 3.3.

Figure 3.1 Photos of each aggregate source (12.5mm Nominal Maximum Aggregate Size)
Figure 3.2  Aggregate gradations

Aggregate angularity, specific gravity, and absorption values are given in Table 3.1. Coarse aggregate angularity (CAA) and fine aggregate angularity (FAA) were performed in accordance with AASHTO T335 (2014) Method A and AASHTO T304 (2014) Method A, respectively. Coarse aggregate is retained on the No. 4 sieve (larger than 4.76 mm) and fine aggregate is passing the No. 4 Sieve (smaller than 4.76mm). Absorption percentage (Abs) is the amount of water absorbed by aggregate into permeable pores relative to the aggregate dry mass. The bulk specific gravity ($G_{sb}$) is based on the oven dry volume of aggregate divided by the total aggregate volume with surface pores included. The apparent specific gravity ($G_{sa}$) is based on the aggregate solid ignoring the surface pores. Specific gravities were measured in accordance with American Society for Testing Materials (ASTM) standard test methods C127 and C128 for coarse and fine aggregate, respectively.

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<td>48</td>
</tr>
<tr>
<td>0.075</td>
<td>6.2</td>
<td>6.0</td>
<td>5.9</td>
</tr>
</tbody>
</table>

Note: The X-axis is the sieve size opening in millimeters raised to the 0.45 power.
Table 3.1  Aggregate properties

<table>
<thead>
<tr>
<th>Source</th>
<th>CAA (%)</th>
<th>FAA (%)</th>
<th>Abs (%)</th>
<th>$G_{sb}$</th>
<th>$G_{sa}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>AL-LS</td>
<td>100</td>
<td>48</td>
<td>0.7</td>
<td>2.694</td>
<td>2.743</td>
</tr>
<tr>
<td>CO-GR</td>
<td>99</td>
<td>47</td>
<td>4.6</td>
<td>2.248</td>
<td>2.507</td>
</tr>
<tr>
<td>MS-GR</td>
<td>96</td>
<td>48</td>
<td>4.2</td>
<td>2.385</td>
<td>2.651</td>
</tr>
</tbody>
</table>

3.2.2  Binder and Additive Properties

A PG 67-22 graded binder from Ergon, Inc. refinery in Vicksburg, MS was used in all mixtures. Sasobit® and Evotherm 3G™ warm mix technology were only used with MS-GR mixtures at typical dosage rates of 1.5% and 0.5%, respectively. Sasobit® comes from Sasol Wax in South Africa, and is obtained from coal gasification (Zhang et al. 2015). Evotherm 3G™ is a chemical package used to improve coating and workability (Hurley and Prowell 2006). Binders containing warm mix technology have shown to age less than binders with no additive (Banerjee et al. 2012). The Sasobit® was added in the laboratory directly to heated binder while being stirred (Figure 3.3). Evotherm 3G™ was received premixed into the binder from Ergon, Inc. For specimen preparation, the binder was stirred and split from 5 gallon buckets into multiple one gallon and one pint metal cans for easier handling.

Figure 3.3  Sasobit® and binder mixing
3.2.3 Mixture Properties and Volumetrics

Mixture volumetrics were determined that correspond to bulk mixture specific gravity ($G_{mb}$) and measured in accordance with AASHTO T166 (2014) to align with most DOT mix designs. The mixture properties and volumetrics can be found in Table 3.2. Table 3.2 denotes aggregate, mixing temperature ($T_{Mix}$), warm mix additive, and other mixture properties. The maximum mixture specific gravity ($G_{mm}$) represents the specific gravity of an asphalt mixture with no air voids. The effective specific gravity ($G_{se}$) of aggregate includes all pore spaces in aggregates except those which absorb asphalt (AI 2001). The percent of binder by mixture mass is denoted by $P_b$ while the percent of asphalt absorbed by aggregate on a mixture mass basis is denoted by $P_{ba(mix)}$. The voids in mineral aggregate (VMA) is the void space in between the aggregates which can be filled with asphalt (VFA) or air voids ($V_a$). The effective binder volume is denoted by $V_{be}$. The dust to binder ratio (D:B) is the percent passing the No. 200 sieve divided by the effective binder content ($P_{be}$).

<table>
<thead>
<tr>
<th>Mix ID</th>
<th>Aggregate</th>
<th>$T_{Mix}$ (°C)</th>
<th>Warm Mix Technology</th>
<th>$G_{mm}$</th>
<th>$G_{se}$</th>
<th>$P_b$ (%)</th>
<th>$P_{ba(mix)}$ (%)</th>
<th>VMA (%)</th>
<th>$V_{be}$ (%)</th>
<th>D:B</th>
</tr>
</thead>
<tbody>
<tr>
<td>M01</td>
<td>MS-GR</td>
<td>163</td>
<td>None</td>
<td>2.250</td>
<td>2.520</td>
<td>8.3</td>
<td>2.3</td>
<td>16.9</td>
<td>12.9</td>
<td>0.97</td>
</tr>
<tr>
<td>M02</td>
<td>MS-GR</td>
<td>163</td>
<td>Evotherm 3G™</td>
<td>2.250</td>
<td>2.520</td>
<td>8.3</td>
<td>2.3</td>
<td>16.9</td>
<td>12.9</td>
<td>0.97</td>
</tr>
<tr>
<td>M03</td>
<td>MS-GR</td>
<td>163</td>
<td>Sasobit®</td>
<td>2.250</td>
<td>2.520</td>
<td>8.3</td>
<td>2.3</td>
<td>16.9</td>
<td>12.9</td>
<td>0.97</td>
</tr>
<tr>
<td>M04</td>
<td>MS-GR</td>
<td>129</td>
<td>None</td>
<td>2.248</td>
<td>2.505</td>
<td>8.0</td>
<td>2.1</td>
<td>16.8</td>
<td>12.8</td>
<td>0.98</td>
</tr>
<tr>
<td>M05</td>
<td>MS-GR</td>
<td>129</td>
<td>Evotherm 3G™</td>
<td>2.248</td>
<td>2.505</td>
<td>8.0</td>
<td>2.1</td>
<td>16.8</td>
<td>12.8</td>
<td>0.98</td>
</tr>
<tr>
<td>M06</td>
<td>MS-GR</td>
<td>129</td>
<td>Sasobit®</td>
<td>2.248</td>
<td>2.505</td>
<td>8.0</td>
<td>2.1</td>
<td>16.8</td>
<td>12.8</td>
<td>0.98</td>
</tr>
<tr>
<td>M07</td>
<td>AL-LS</td>
<td>163</td>
<td>None</td>
<td>2.479</td>
<td>2.733</td>
<td>6.2</td>
<td>0.5</td>
<td>17.2</td>
<td>13.2</td>
<td>1.03</td>
</tr>
<tr>
<td>M08</td>
<td>AL-LS</td>
<td>129</td>
<td>None</td>
<td>2.481</td>
<td>2.735</td>
<td>6.2</td>
<td>0.5</td>
<td>17.0</td>
<td>13.0</td>
<td>1.04</td>
</tr>
<tr>
<td>M09</td>
<td>CO-GR</td>
<td>163</td>
<td>None</td>
<td>2.123</td>
<td>2.362</td>
<td>8.7</td>
<td>2.2</td>
<td>17.2</td>
<td>13.2</td>
<td>0.93</td>
</tr>
<tr>
<td>M10</td>
<td>CO-GR</td>
<td>129</td>
<td>None</td>
<td>2.125</td>
<td>2.351</td>
<td>8.3</td>
<td>2.0</td>
<td>16.8</td>
<td>12.8</td>
<td>0.96</td>
</tr>
</tbody>
</table>

A brief review of mixture volumetrics is needed to explain the abnormally high VMA for the mixtures used in this thesis. The minimum VMA for a typical 12.5 mm Nominal Maximum Aggregate Size (NMAS) is 14% (AI 2001). The VMA for the SAS
mixtures was approximately 17%. Based on CO-GR limited materials a VMA of approximately 17% was required in order to keep other volumetric properties constant, such as effective binder content ($V_{be}$). The VMA for the SAS mixtures had to be adjusted for the other aggregate sources. It must be noted that when the same aggregate gradation and compactive effort are used with different shaped particles differences in VMA can be observed (AI 1997). The differences in VMA were accounted for by adjusting certain sieve size passing percentages until a VMA of 17% was achieved for each aggregate source. Mixtures with a VMA of 17% would not be produced due to costs and unfavorable mixture behavior such as tenderness and rutting. By ensuring the different mixtures had the same volumetric properties the aggregate and binder interaction effects could be isolated.

When this thesis’ mixtures are compared to mixtures already used by MDOT, the differences are clear. Out of 167 12.5 mm NMAS mixtures the maximum $P_b$ and $P_{ba}$ was 6.2% and 1.3%, respectively (Doyle et al. 2012). The MS-GR mixtures (M01-M06) and CO-GR mixtures (M09-M10) are over the max $P_b$ by about 2% while 1% above the max $P_{ba}$. The AL-LS mixtures (M7-M8) are at the maximum $P_b$ while 0.8% below the max $P_{ba}$. Production of mixtures with these binder percentages is not the purpose of this thesis. The purpose is to control as many mixture properties as possible in order to isolate aggregate effects on aging and mechanical performance behavior.

3.3 Specimen Preparation and Compaction

All mixtures were lab mixed and lab compacted. The SAS mixtures had two pairs of mixing and compacting temperatures. Hot mixed asphalt (HMA) mixtures used 163°C for mixing and 149°C for compacting. Warm mixed asphalt (WMA) mixtures used 129°C for mixing and 116°C for compacting. Mixing was conducted in accordance with
AASHTO T312 (2014). The freshly mixed asphalt was subjected to 90 minutes of short term aging at compaction temperature before Superpave Gyratory Compactor (SGC) compaction. All specimens were compacted to height (test dependent) and 7±0.5% air voids.

3.4 Field Aging

The SAS specimens were field aged for one year on a Columbus, MS, non-trafficked asphalt test section between November 1, 2014 and October 31, 2015. The specimens were placed in PVC sleeves to cover the sides. Specimen bottoms were in direct contact with the underlying pavement, while specimen tops were exposed to sunlight (Figure 3.4). Weather data was collected from a nearby weather station and is given in Table 3.3 for a general idea of temperature and precipitation over the aging period.

Figure 3.4 SAS specimens after placement (largest specimens have 15cm diameter and height of 11.5cm)
Table 3.3 Summary of Weather Data (November 1, 2014 to October 31, 2015)

<table>
<thead>
<tr>
<th>Month</th>
<th>Days</th>
<th>Avg. Daily Temp</th>
<th>High Daily Temp</th>
<th>Low Daily Temp</th>
<th>Rainfall</th>
<th>Days of 1.25 cm+</th>
<th>Relative Humidity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mean</td>
<td>St. Dev</td>
<td>Mean</td>
<td>St. Dev</td>
<td>Total (cm)</td>
<td>Mean (%)</td>
<td>St. Dev (%)</td>
</tr>
<tr>
<td>Nov-14</td>
<td>30</td>
<td>14.9</td>
<td>6.0</td>
<td>1.5</td>
<td>10.7</td>
<td>2</td>
<td>70.6</td>
</tr>
<tr>
<td>Dec-14</td>
<td>31</td>
<td>13.3</td>
<td>4.1</td>
<td>3.2</td>
<td>18.2</td>
<td>5</td>
<td>85.0</td>
</tr>
<tr>
<td>Jan-15</td>
<td>31</td>
<td>11.3</td>
<td>5.9</td>
<td>-1.5</td>
<td>12.2</td>
<td>4</td>
<td>72.0</td>
</tr>
<tr>
<td>Feb-15</td>
<td>29</td>
<td>9.1</td>
<td>6.2</td>
<td>-2.2</td>
<td>37.9</td>
<td>3</td>
<td>65.2</td>
</tr>
<tr>
<td>Mar-15</td>
<td>31</td>
<td>18.7</td>
<td>6.3</td>
<td>7.4</td>
<td>15.6</td>
<td>5</td>
<td>82.6</td>
</tr>
<tr>
<td>Apr-15</td>
<td>30</td>
<td>24.1</td>
<td>3.5</td>
<td>12.3</td>
<td>18.9</td>
<td>4</td>
<td>79.2</td>
</tr>
<tr>
<td>May-15</td>
<td>31</td>
<td>29.7</td>
<td>2.9</td>
<td>15.5</td>
<td>11.2</td>
<td>4</td>
<td>73.8</td>
</tr>
<tr>
<td>Jun-15</td>
<td>30</td>
<td>31.7</td>
<td>2.5</td>
<td>20.2</td>
<td>2.0</td>
<td>0</td>
<td>77.2</td>
</tr>
<tr>
<td>Jul-15</td>
<td>31</td>
<td>33.8</td>
<td>2.6</td>
<td>22.2</td>
<td>6.2</td>
<td>3</td>
<td>76.1</td>
</tr>
<tr>
<td>Aug-15</td>
<td>31</td>
<td>31.8</td>
<td>2.7</td>
<td>20.4</td>
<td>12.0</td>
<td>4</td>
<td>77.8</td>
</tr>
<tr>
<td>Sep-15</td>
<td>30</td>
<td>29.9</td>
<td>3.0</td>
<td>17.1</td>
<td>2.2</td>
<td>0</td>
<td>76.9</td>
</tr>
<tr>
<td>Oct-15</td>
<td>31</td>
<td>24.7</td>
<td>4.9</td>
<td>11.2</td>
<td>40.6</td>
<td>1</td>
<td>76.4</td>
</tr>
<tr>
<td>All</td>
<td>366</td>
<td>22.8</td>
<td>9.6</td>
<td>10.7</td>
<td>187.9</td>
<td>35</td>
<td>76.2</td>
</tr>
</tbody>
</table>

3.5 Mixture Test Methods

3.5.1 Cantabro Mass Loss

Cantabro Mass Loss (CML) testing was performed on 15 cm diameter by 11.5 cm tall lab compacted specimens conditioned to 25°C in air before testing. The mass is recorded before and after 300 revolutions in a Los Angeles (LA) abrasion drum (Figure 3.5a) without steel charge. After the specimen reaches 300 revolutions in the LA abrasion drum, it is removed, lightly brushed, and the final mass is recorded. The mass loss (ML) can be calculated by the change in mass divided by the initial mass. Any increase in ML indicates asphalt embrittlement. The LA abrasion drum interior temperature is adjusted to at 25±2°C to begin testing. All specimens were tested within 30 minutes of removal from the environmental conditioning chamber. Figure 3b shows a tested specimen.
3.5.2 **Indirect Tensile (Non-Instrumented)**

Indirect tensile (non-instrumented) (IDT) testing was conducted on 10 cm diameter by 6.3 cm tall lab compacted specimens conditioned to 25°C in air before testing. Testing was performed according to AASHTO T283 (2014) dry protocols on aged and unaged samples. The IDT specimens were loaded diametrically at a rate of 50mm/min until failure.

3.5.3 **Hamburg Loaded Wheel Tracking**

Hamburg Loaded Wheel Tracking (HLWT) was conducted according to AASHTO T324 (2014) on 15 cm diameter by 6.3 cm tall lab compacted specimens. After compaction, the specimens were sliced to fit standard molds. The testing temperature was maintained at 50°C throughout testing. The steel wheel load was maintained at 0.7 kN for 20,000 passes or a max rut depth of 12.5 mm. The main purpose of HLWT is to test moisture sensitivity and stability of a mixture via the presence or absence of a stripping inflection point (SIP) and rutting. Figure 3.6 shows the HLWT testing setup and a tested specimen.
3.5.4 **Asphalt Pavement Analyzer Rut Susceptibility**

Asphalt Pavement Analyzer (APA) testing was conducted according to AASHTO T340 (2014) on 15 cm diameter by 6.3 cm lab compacted specimens. Plaster of Paris was used on the bottom of specimens to make the specimens flush with the top of the molds prior to testing. The test is conducted with a grooved steel wheel moving back and forth over a pressurized hose. The test was performed at 64°C with wheel loads of 0.4 kN for 8000 cycles at a hose pressure of 689 kPa. Rutting is measured over the number of cycles. Figure 3.7 shows the APA test setup a tested specimen.
4.1 Test Results and Discussion

The following sections present the results by testing procedure and then all of the results are discussed together in the last section. Table 4.1 contains all of the results for the SAS mixtures. As mentioned earlier, these results are the same as a paper submitted for peer review (Hansen and Howard 2018). Additional figures are provided herein for clearer visualization of mixture performance changes. The additional figures do not present any new data relative to the submitted paper.

4.2 IDT Results

The tensile strength ($S_t$) and change in tensile strength ($\Delta S_t$), aged minus unaged, are of interest for the IDT test. The $\Delta S_t$ given in Table 4.1 and shown in Figure 4.1b gives the relative change between aggregates after environmental effects. The $\Delta S_t$ appears to be considerably affected by aggregate source. Figure 4.1a shows all of the results and it can be easily seen that MS-GR started out the strongest and gained the most strength, due to hardening, after aging. Figure 4.1c compares the mixtures with warm mix additives and it shows essentially no difference. The max difference in $\Delta S_t$ for HMA and WMA mixtures with additives is 61 kPa (9 psi) and 85 kPa (12 psi), respectively. $T_{Mix}$ considerably affected CO-GR while AL-LS was insensitive to $T_{Mix}$. WMA mixtures showed higher increases in $S_t$ than HMA mixtures in all cases.
<table>
<thead>
<tr>
<th>Mix</th>
<th>Agg.</th>
<th>Add.</th>
<th>$T_{Mix}$ (°C)</th>
<th>Aging</th>
<th>$S_t$ (kPa)</th>
<th>$\Delta S_t$ (kPa)</th>
<th>$ML$ (%)</th>
<th>$\Delta ML$ (%)</th>
<th>$RD_{APA}$ (mm)</th>
<th>$\Delta RD_{APA}$ (mm)</th>
<th>$RD_{HLWT}$ (mm)</th>
<th>$\Delta RD_{HLWT}$ (mm)</th>
<th>$P_{12.5\text{-HLWT}}$</th>
<th>SIP</th>
</tr>
</thead>
<tbody>
<tr>
<td>M01</td>
<td>MS-GR</td>
<td>None</td>
<td>163</td>
<td>1 yr. Field Unaged</td>
<td>1890</td>
<td>1032</td>
<td>858</td>
<td>11.1</td>
<td>5.2</td>
<td>3.5</td>
<td>-3.4</td>
<td>6.6</td>
<td>12.5</td>
<td>-5.9</td>
</tr>
<tr>
<td>M02</td>
<td>MS-GR</td>
<td></td>
<td>163</td>
<td>1 yr. Field Unaged</td>
<td>1879</td>
<td>1082</td>
<td>797</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>M03</td>
<td>MS-GR</td>
<td>Sas.®</td>
<td>163</td>
<td>1 yr. Field Unaged</td>
<td>1804</td>
<td>991</td>
<td>813</td>
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</tr>
<tr>
<td>M04</td>
<td>MS-GR</td>
<td>None</td>
<td>129</td>
<td>1 yr. Field Unaged</td>
<td>2047</td>
<td>1021</td>
<td>1026</td>
<td>12.7</td>
<td>4.8</td>
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<td>7.0</td>
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<td>M05</td>
<td>MS-GR</td>
<td>Evo.3G</td>
<td>129</td>
<td>1 yr. Field Unaged</td>
<td>2024</td>
<td>1045</td>
<td>979</td>
<td>---</td>
<td>---</td>
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<td>---</td>
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<td>---</td>
</tr>
<tr>
<td>M06</td>
<td>MS-GR</td>
<td>Sas.®</td>
<td>129</td>
<td>1 yr. Field Unaged</td>
<td>1936</td>
<td>995</td>
<td>941</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>M07</td>
<td>AL-LS</td>
<td>None</td>
<td>163</td>
<td>1 yr. Field Unaged</td>
<td>1071</td>
<td>719</td>
<td>352</td>
<td>9.2</td>
<td>4.9</td>
<td>4.3</td>
<td>3.3</td>
<td>9.6</td>
<td>-6.3</td>
<td>12.5</td>
</tr>
<tr>
<td>M08</td>
<td>AL-LS</td>
<td>None</td>
<td>129</td>
<td>1 yr. Field Unaged</td>
<td>1065</td>
<td>657</td>
<td>408</td>
<td>9.1</td>
<td>5.2</td>
<td>3.9</td>
<td>4.6</td>
<td>9.9</td>
<td>-5.3</td>
<td>12.5</td>
</tr>
<tr>
<td>M09</td>
<td>CO-GR</td>
<td>None</td>
<td>163</td>
<td>1 yr. Field Unaged</td>
<td>1114</td>
<td>770</td>
<td>344</td>
<td>8.3</td>
<td>4.0</td>
<td>4.3</td>
<td>4.3</td>
<td>8.2</td>
<td>-3.9</td>
<td>12.5</td>
</tr>
<tr>
<td>M10</td>
<td>CO-GR</td>
<td>None</td>
<td>129</td>
<td>1 yr. Field Unaged</td>
<td>1375</td>
<td>703</td>
<td>672</td>
<td>8.6</td>
<td>2.8</td>
<td>5.8</td>
<td>5.2</td>
<td>11.4</td>
<td>-6.4</td>
<td>12.5</td>
</tr>
</tbody>
</table>

Note: IDT and CML results are a 3 specimen average while APA and HLWT are 2 specimens. A total of 144 specimens were tested. 40 were tested for each source with varying aging protocols, mixing/compaction temperatures, and testing procedures. An additional 12 specimens each were IDT tested with 0.5% Evotherm 3G™ and 1.5% Sasobit ®.

* Test shut down early, but specimen exhibited rutting.
a.) All IDT Data

b.) Differences in IDT strength aged – unaged

c.) MS-GR warm mix additive comparison

Figure 4.1   IDT results
4.3 Cantabro Results

Figure 4.2a shows clearly that before aging differences between mixtures already existed. The lowest mass loss (ML) was 2.8% while the max was 5.9%. Once aging occurred these differences became larger, minimum ML was 8.3% while the max was 12.7%, indicating asphalt-aggregate interaction and TMix affects mixture performance after aging. Figure 4.2b shows the change in ML (ΔML) which indicates again that different aggregates and TMix affect asphalt aging differently. The asphalt-aggregate combination which showed the most increase in ML, or most age hardening, at both mixing temperatures was the MS-GR mixtures. WMA mixtures showed higher ΔML in both the gravels when compared to HMA mixtures. From these results it is obvious asphalt-aggregate interaction and TMix affect aging of asphalt mixtures.

![Graph showing mass loss and change in mass loss for different mixtures](image)

a.) All Cantabro Data

![Graph showing change in mass loss for different mixtures](image)

b.) Differences in ML aged - unaged

Figure 4.2 Cantabro results
4.4 HLWT Results

Table 4.1 gives the maximum rut depth ($RD_{HLWT}$), change in rut depth ($\Delta RD_{HLWT}$), and number of passes to reach a rut depth of 12.5 mm ($P_{12.5-HLWT}$). With a VMA of 17% these mixtures should be expected to experience significant rutting. Figure 4.3 plots the rut depth versus number of passes which shows all mixtures except aged MS-GR mixes surpassed a 12.5 mm rut depth before the full 20,000 passes. Reduction in rut depth for aged MS-GR mixtures indicated increased age hardening decreased rutting. The WMA mixtures rutted more quickly according to $P_{12.5-HLWT}$. Besides rutting HLWT gives indications of stripping potential via SIP. The AL-LS mixtures and the unaged warm mixed gravel mixtures all displayed a SIP. One of the AL-LS tests was shut down early, but it is assumed that stripping would most likely have occurred. As mentioned in Literature Review, hydrated lime addition could have helped mitigate some stripping in these mixes.

4.5 APA Results

Table 4.1 gives the maximum rut depth ($RD_{APA}$) and change in rut depth ($\Delta RD_{APA}$) which shows again HMA rutted less than WMA. Figure 4.3 shows once again MS-GR is the stiffest mixture. Mixing temperature did not have much effect on $RD_{APA}$ for unaged MS-GR and AL-LS mixtures. The CO-GR unaged mixtures showed an appreciable difference with respect to $T_{Mix}$. The $\Delta RD_{APA}$ with respect to $T_{Mix}$ was approximately 1 mm for MS-GR and AL-LS mixtures. The CO-GR showed twice as large $\Delta RD_{APA}$ with respect to $T_{Mix}$ than the other mixtures. The APA results showed agreeable results with the other mixtures tests with all tests showing that asphalt-aggregate interaction affects aging.
Figure 4.3  HLWT and APA results
4.6 Discussion of Results

Figures 4.4 and 4.5 compare the four mixture tests to determine if there is a difference between using a combined analysis (multiple tests) or individual analysis (one test). The individual analysis was given in the previous sections with all indications pointing to asphalt-aggregate interaction has meaningful effects after aging. Figure 4.4 shows ML versus Sₜ and then separated by the presence of a HLWT SIP. As ML increased so did Sₜ. One data point was included in the all data plot which is not included in the SIP differentiation due to a testing failure. When only looking at no stripping cases, the R² (coefficient of determination) was 0.94 which improved from a R² of 0.79 in all of the data. The stripping cases experienced a drop in R² to 0.65. The slopes of the trend line are important as well with stripping cases having a slope of roughly 1/3 of the no stripping cases. This means that moisture susceptibility appreciably affected the tensile strength with ML increasing roughly three times as fast per unit than Sₜ when stripping is present. Nothing in the combined analysis indicates anything meaningfully different than the individual analysis.

Figure 4.4 Mass loss versus Tensile strength separated by stripping behavior
Figure 4.5 shows $ML$ versus $RD_{APA}$ and then separated by the presence of a HLWT SIP. The trends follow what is expected with decreased rut depth correlating with increased mass loss since a stiffer mix resists rutting, but becomes more damaged in CML testing due to embrittlement. When there is no presence of a HWLT SIP rut depths decreased at a lower rate than $ML$ on a per unit basis than when a SIP was present. These results are not in conflict with any other of this paper’s assessments. Due to the high VMA, which is known to affect rutting, no more information was drawn from Figure 4.5 results.

Figure 4.5  Mass loss versus APA rut depth separated by stripping behavior
CHAPTER V

CONCLUSIONS AND RECOMMENDATIONS

The objective of this thesis was to determine whether performance differences exist before or after aging in simple mixtures where the only practical difference was aggregate source. This objective was met with all four mixture tests concluding asphalt-aggregate interaction considerably affected mixture aging. Another conclusion was that mixing temperature with certain aggregate sources can also considerably affect mixture aging over a one year period. Warm mix technology was also found to have no measurable effect on tensile strength before and after aging. These conclusions follow what is expected according to Literature Review.

The importance of these findings is with such simple mixes these effects cannot be attributed to anything but asphalt-aggregate interaction and in some cases mixing temperature. Furthermore, in mixtures with more ingredients singular ingredient effects becomes much harder to determine. Although binder testing has value, binder testing alone is unable to fully capture the effects related to different aggregate sources in simple mixtures. These conclusions lead to the recommendation for future work to consider mixture aging coupled with a mixture test(s) to fully capture in service aging.
REFERENCES


