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Investigating the rutting and moisture sensitivity of warm mix asphalt with varying contents of recycled asphalt pavement

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INVESTIGATING THE RUTTING AND MOISTURE SENSITIVITY OF WARM MIX ASPHALT WITH VARYING CONTENTS OF RECYCLED ASPHALT PAVEMENT

by

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CERTIFICATE OF APPROVAL

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PH.D. THESIS

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This is to certify that the Ph.D. thesis of

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ABSTRACT

To evaluate the performance of Warm Mixture Asphalt (WMA) with varying amounts of recycled asphalt pavement (RAP) in comparison with Hot Mix Asphalt (HMA), comprehensive laboratory and field evaluations were conducted. Mix designs were performed for both WMA with a LEADCAP additive and HMA with large amounts of fractionated RAP materials. Hamburg Wheel Tracking (HWT) test was performed to evaluate the rutting and moisture susceptibility of both HMA and WMA laboratory mixtures. HMA mixtures with up to 50% RAP materials by binder replacement exhibited a better performance than WMA mixtures. However, when RAP materials were increased to 75% both WMA and HMA mixtures showed a superior performance. When a specially designed LEADCAP additive for a mixture with a high RAP content called "RAPCAP" was used, the performance was significantly improved. The existing Asphalt Bond Strength (ABS) test (AASHTO TP91-11) was modified to better evaluate the adhesion bond between asphalt binder and aggregate surface. Based on the modified ABS test results, it was found that the asphalt binder type significantly influenced the adhesion bond.

To evaluate the performance of WMA mixtures in the field, test sections were constructed in Iowa, Minnesota and Ohio. The test sections were successfully constructed with less compaction effort than HMA and met the required field densities per each DOT's specification. All HMA and WMA mixtures collected from the test sections passed the HWT and the modified Lottman tests, which indicates high resistance to rutting and moisture damage. The asphalt binders were then extracted and recovered from the field samples then re-graded following AASHTO M320 and AASHTO MP19-10. The recovered asphalt binder grades were found to be higher than the target grades due to the

existence of RAP materials in the mixtures except for asphalt binders extracted from WMA mixtures produced using “RAPCAP” additive.

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CHAPTER 1 INTRODUCTION

Hot mix asphalt (HMA) is a generic term that includes many different types of mixtures of aggregate and asphalt cement (binder). The aggregates and asphalt binder proportions are determined through a mix design procedure such as the Marshall or Superpave Mix Design methods. Overall, the goal of determining such proportions is to establish an HMA mixture that will meet specific performance criteria. In addition, it is very important to ensure that the asphalt binder would fully coat the aggregates and that the resulting mixture is workable and compactable. In order to ensure sufficient aggregate drying and coating, both the asphalt binder and the aggregates are heated to highly elevated temperatures ranging between 275°F and 350°F (135°C and 176°C). The use of such high temperatures would lower the viscosity of the asphalt binder which is the main factor affecting the coating and workability of HMA mixtures.

Recently, new technologies have been introduced to Pavement Industry that allow producing asphalt mixtures at temperatures 30°F to 100°F (15°C to 50°C) lower than what is used for HMA. These new technologies are commonly referred to as Warm Mix Asphalt (WMA). The main goal of WMA is to produce mixtures with similar strength, durability, and performance characteristics as HMA using substantially reduced production temperatures (1). Reducing HMA production and placement temperatures will provide several benefits; including reduced emissions, fumes, odors, and a cooler work environment. An energy saving from lower production temperatures is evident with the use of WMA technology (2). Lower production temperatures can also potentially improve pavement performance by reducing binder aging, which allows for using higher amounts of recycled asphalt pavement (RAP) without adding any asphalt binder rejuvenators.

Some WMA technologies achieve lower mixing and compaction temperatures through altering the viscosity-temperature relationship of the asphalt binder. However, in all WMA technologies, the direct measure of mixture coating, workability and compactability are required to establish the mixing and compaction temperatures (3; 4).

Several research studies reported that WMA may be more susceptible to moisture damage, which can be attributed to the lower production temperature of WMA resulting in residual moisture in aggregates (1).

Therefore, a comprehensive laboratory and field experiments must be completed to investigate the rutting and moisture susceptibility of WMA technology as compared to the conventional HMA technology. This research evaluates field mixtures from three different states with varying amounts of recycled asphalt pavements (RAP).

1.1. Research Objectives

The overall objectives of this research study are listed below:

1. To evaluate WMA paving technologies with various contents of RAP to determine their suitability for use in various applications. This objective was met by conducting an extensive laboratory experiment to assess the durability (i.e. resistance to moisture damage, and rutting) of WMA mixtures produced using different aggregate sources and amounts of RAP materials.
2. To assess the performance of WMA during construction of test sections and over time**.** This objective was met through the construction of test sections using the identified WMA technologies. This effort evaluated the constructability of WMA mixtures. Field produced mixture was also evaluated in the laboratory for their resistance to rutting and moisture damage. After the test sections were constructed,

their short-term performance was monitored to predict the long-term behavior of the WMA mixtures.

1.2. Scope

Two laboratory evaluations were performed in this research effort. First laboratory evaluation included a total of ten mixtures; five HMA as a control mixture and five WMA mixtures prepared using LEADCAP technology 7-1 (liquid) for comparison. The laboratory mixtures were designed for a traffic level of 0.3 million ESAL per Iowa DOT mix design requirements and NCHRP 691 report for WMA mix design. The mixtures used limestone virgin aggregate and a combination of different percentages of fractionated RAP; 20, 30, 40, 50, and 75% RAP by binder replacement.

After analyzing the results from the first laboratory evaluation, one mix design was chosen for further laboratory evaluation. The field implementation included two test sections, one for WMA, and one for HMA in Iowa City, IA in September 2013.

Furthermore, a test section was constructed in Champlin, Minnesota in July 2010, and another one was constructed in Lancaster, Ohio in September 2013 using LEADCAP technologies as a part of the field evaluation of WMA technology.

To investigate the impact of WMA technologies on the asphalt mixtures performance, laboratory testing was conducted to address the following questions:

- The impact of warm-mix additives on the resistance of the mixtures to permanent deformation, or rutting
- The impact of warm-mix additives on the resistance of the mixtures to moisture damage,

The following performance tests were performed in this research study:

- Performance tests on asphalt mixture
    - Hamburg Wheel Track test (AASHTO T324)
    - Modified Lottman test (AASHTO T283)
- Performance tests on asphalt binder
    - Determining the asphalt bond strength (ABS) using adhesion tester device (AASHTO TP 90-11 & ASTM D4541).

CHAPTER 2 LITERATURE REVIEW

2.1. Introduction

Two of the primary failure mechanisms for flexible (asphalt concrete) pavements are rutting, or permanent deformation, and moisture damage in the surface of the traveled roadway. Indeed, the concept of creating hot-mix asphalt (HMA) mixes with increased resistance to permanent deformation was a major driving force behind much of the asphalt-related research performed by many research institutes. With the introduction of the WMA technology into the pavement industry, the need to establish more resistant pavements to those mode of failures became more essential (5).

The use of WMA technology results in less hardening of the asphalt binder, which reflected in an increasing number of road projects that are built using Warm Mix Asphalt (WMA) with high reclaimed asphalt pavement (RAP) contents. However, there was no comprehensive study done to identify the synergistic effects between WMA and high RAP content which will help understand their interactions and their impacts on rutting under a moisture condition, and moisture resistance. Therefore, a comprehensive research study is needed to derive a relationship between fundamental characteristics of WMA and high RAP for varying contents.

The fundamentals of rutting and moisture failure mechanisms will be discussed in the following sections.

2.2. Rutting in Asphalt Pavement

Rutting, also known as permanent deformation, can be defined as the accumulation of small amounts of unrecoverable strains as a result of applied loading to a pavement (6). Figures 2.1 shows different cases of rutted pavements.

Figure 2.1 Sever Mix Rutting (left), Mix Rutting (middle), and Rutting in the Wheel Path (right) (8).

Rutting occurs when the pavement under traffic loading consolidates and/or there is a lateral movement of the hot-mix asphalt (HMA). The lateral movement is a shear failure and generally occurs in the upper portion of the pavement surface. As a result of rutting, the pavement useful service life is reduced. If the rutting depth is significant, water may accumulate in the rutted area, which can lead to vehicle hydroplaning and may create a safety hazard for the traveling public (6; 7). Recently, the potential for rutting has increased rapidly due to the continuous increase in traffic volumes, and the increase use of radial tires, which contain higher inflation pressure than other tire types.

Rutting doesn’t only occur due to the permanent deformation in the surface layer, but it can also happen due to a plastic deformation resulted from over-stressing the base or the subgrade layer during compaction. Figure 2.2 shows an example for rutting in base or subgrade layer. Rutting can be classified into four main types:

- Mechanical deformation or subgrade displacement of the asphalt pavement,
- Plastic deformation of the asphalt mixtures near the pavement surface,
- Consolidation or the continued compaction under the action of traffic, and
- Surface wear, the actual wearing away of surface particles by traffic.

Determining the mode of failure is important in selecting the right correction to the asphalt pavement (9).

Plastic or permanent deformation is a material failure of the asphalt pavement in which the mix is displaced from under the tires and typically humps up outside the wheel tracks. The permanent deformation can happen in the form of shoving or corrugation in the pavement surface as shown in Figure 2.3. Corrugation is a plastic deformation typified by ripples. Shoving is an abrupt wave across the pavement surface. The resulted distortion is usually perpendicular to the traffic direction. Corrugation usually occurs at points where traffic starts and stops. Shoving occurs in the areas where HMA abuts a rigid object (8; 9).

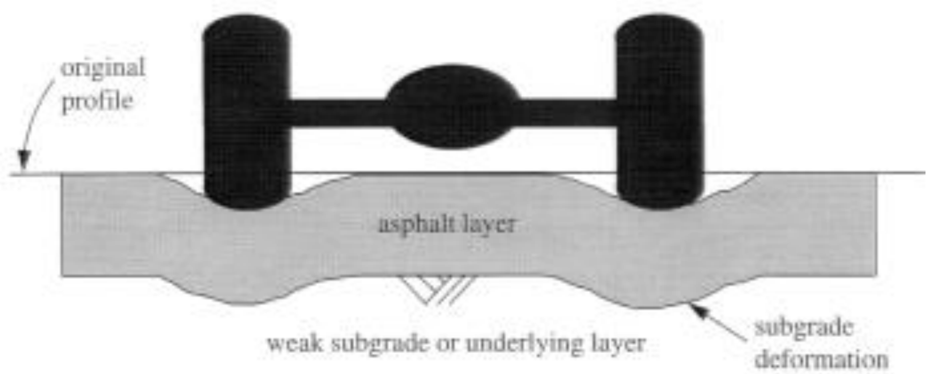

Figure 2.2 Rutting in Subgrade or Base Layer (8).

Figure 2.3 Corrugation (left) and Shoving (right) in the Asphalt Pavement Surface (8).

2.2.1. Factors Affecting the Rutting Resistance

2.2.1.1. Materials Selection

Materials including aggregate and asphalt binder play major role in the potential for permanent deformation of an asphalt mixture. The rutting resistance of an asphalt mix depends on the shear resistance of that mix, which comes mainly from aggregate. If the shear stress created by repeated wheel load applications exceeds the shear strength of the mix, as shown in Figure 2.4, then permanent deformation or rutting will occur. Thus, good aggregate properties and gradation increase the shear strength of the asphalt mixture. Cubical or rough-textured aggregates are more resistant to the shearing action of traffic than rounded, smooth-textured aggregates, as shown in Figure 2.5. Cubical aggregates also tend to interlock better, resulting in a more shear resistant mass of material. Additionally, using higher percentage of coarse aggregate in the aggregate gradation increases the number of contact points, stone-to-stone contact, in the asphalt mixture, which helps in reducing pavement rutting (10).

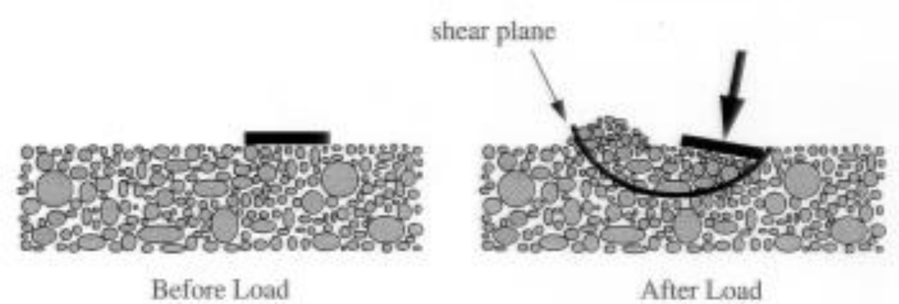

Figure 2.4 Loading Behavior of Aggregate (10).

Similarly, the asphalt binder affects the rut resistance of an asphalt mixtures but not as significant as aggregate. Mixtures produced with soft binder grade (low in viscosity) exhibit less rut resistance than those produced using harder binder grade (high in viscosity) at high temperatures. Rutting usually occurs in the early life of pavement, less than 5 years, when the used binder is still fresh and not aged yet. While the pavement is in service, the asphalt binder oxidized or aged, and leads to harder or more viscous binder, which decreases the potential of rutting in this pavement. Also, modified asphalts can provide higher resistance to rutting at high temperatures than regular asphalts.

Figure 2.5 Contrasting Stone Skeletons (10).

2.2.1.2. Asphalt Mix Type

The asphalt mix type has a significant effect on its resistance to rutting. Increasing the percentage of coarse aggregate in asphalt mixtures can improve the rutting resistance. Certain Superpave and Stone Matrix Asphalt (SMA) mixes are designed with high percentage of coarse aggregates to provide more direct stone-to-stone contact, which significantly improves the rutting resistance of these mixes. In other mixes such as dense graded mixes, good rutting resistance can be achieved through compaction. Good compaction that forces all aggregate particles with high quality rough texture to form better interlock can produce a dense graded mix with reasonable rutting resistance.

Figure 2.6 illustrates the difference in structure between an SMA mix and a dense graded asphalt mix. In SMA mixes, the coarse aggregate particles are intended to carry the load, while the fine aggregate particles are used as a filler. Unlike dense graded mixes, where fine aggregate particles are locked between the coarse aggregate particles, and the load is carried through the entire uniformly graded mix (10).

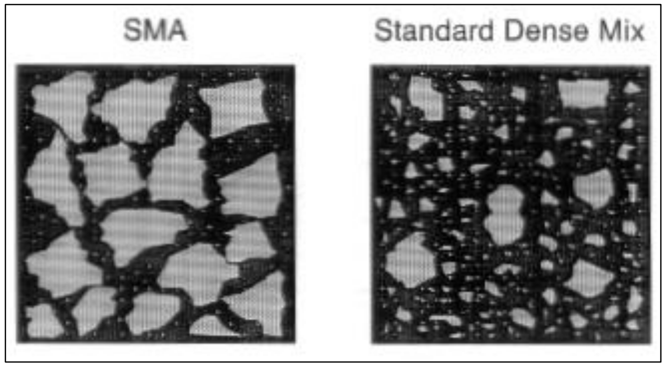

Figure 2.6 SMA Mix Structure versus Dense Graded Mix Structure (10).

2.2.1.3. Compaction Effects

Carefully increased compaction effort can improve the rutting resistance of asphalt mixtures by packing and orienting the aggregate particles into interlocking mass of material that resists shear deformation. The results from WesTrack study showed that lowering the air void content due to compaction increased the rutting resistance of most asphalt pavement sections. Figure 2.7 shows the influence of air void content for the same asphalt mixture on the predicted number of equivalent single axle loads (ESALs) to a 15 mm (0.6 in.) rut depth. Although reducing the air void contents with compaction can be very beneficial to rutting resistance, yet the asphalt mix should not be overcompacted to unstable level. During field compaction, an air void content of 5-6% can be considered as a reasonable target to achieve good rutting resistance and still can protect the mix from the instability associated with lower void content (10).

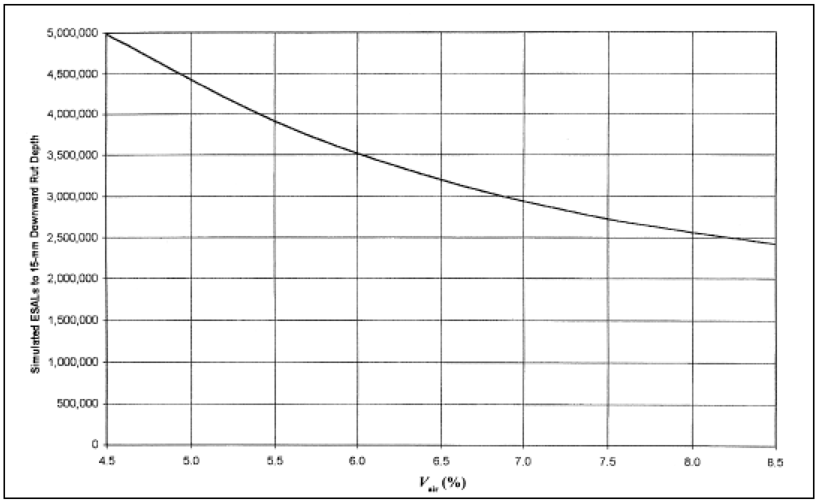

Figure 2.7 Effect of Air Void Content on Estimated ESALs to 15 mm (0.6 in) Rut Depth (10).

2.2.2. Evaluation and Prediction of Rutting

Previous and current research studies related to asphalt permanent deformation are investigating which test method can most accurately evaluate and/or predict the asphalt mixture's potential for rutting. Those methods can be characterized as follow:

2.2.2.1. Static Creep Tests (AASHTO TP9)

In this test, a static load is applied to an asphalt sample then the resulting plastic strain or deformation is measured. Figure 2.8 showed a typical creep stress and strain relationships. The creep test can be conducted confined, unconfined, or diametral. The measured deformation is correlated to the rutting potential, where large number of plastic deformation shows a higher rutting potential. Creep tests were widely used in the past due to their simplicity, however their results didn't accurately correlate with actual in-service pavement rutting (11; 12).

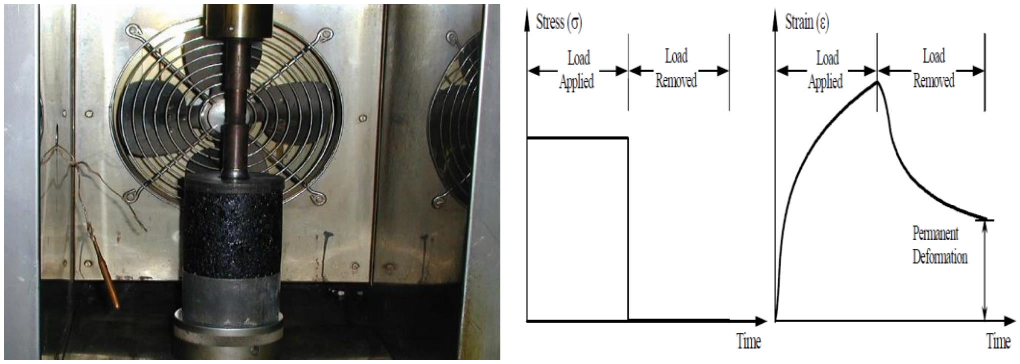

Figure 2.8 Typical Creep Stress and Strain Relationships (11).

2.2.2.2. Repeated Load Tests

During repeated load tests, a repeated load are applied at a specific frequency to an asphalt test sample for many repetitions, usually until the sample fails, then the sample's recoverable and the plastic strains are measured. The test results from repeated load tests correlate with in-service pavement rutting measurements. The most common used repeated load tests for evaluating the rutting potential of asphalt mixtures are the flow number test and the shear repeated load test.

Flow Number Test

During the flow number test, an HMA specimen at a specified temperature is subject to a repeated haversine axial compressive load pulse (deviator stress) of 0.1 second loading and 0.9 second of rest time. The FN test can be run with or without a confining pressure, it is recommended that the test be conducted with a static all around confining pressure using compressed air to simulate field conditions. The resulting cumulative permanent axial strain is measured and plotted versus the number of load cycles (13).

The test specimen is a 4-inch diameter by 6-inch high cylindrical sample. The test is conducted for a certain amount of cycles, usually 12,000 cycles; axial deformations continuously measured over the middle 4 inches of the sample by two independently monitored linear variable differential transducers (LVDT) placed 180° apart. Also, the permanent vertical strain in the sample is measured as a function of load cycles using the Repeated Load Triaxial (RLT) equipment as shown in Figure 2.9. The resulting cumulative permanent strain can be characterized by the primary, secondary, and tertiary zones, as shown in Figure 2.10 (11; 13).

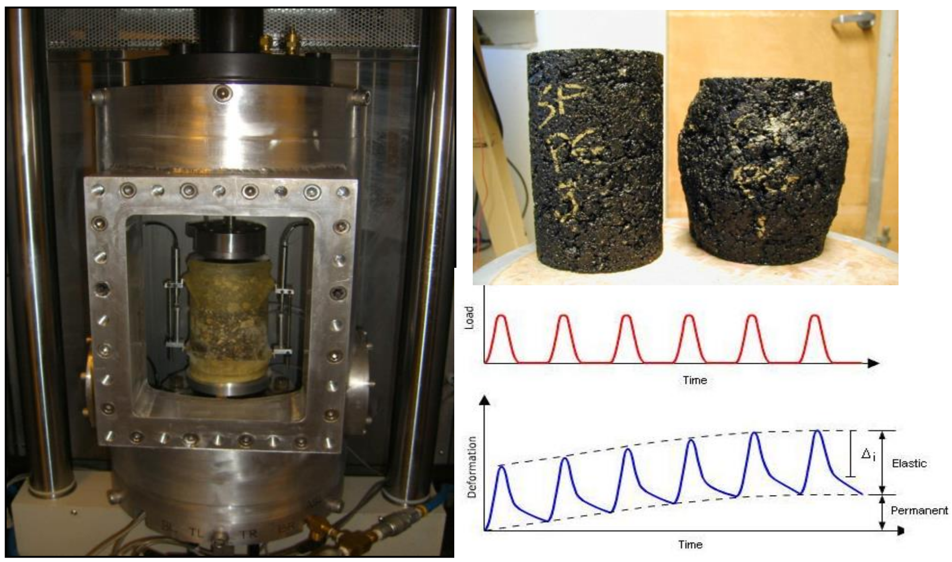

Figure 2.9 Repeated Load Triaxial (RLT) Schematics (13).

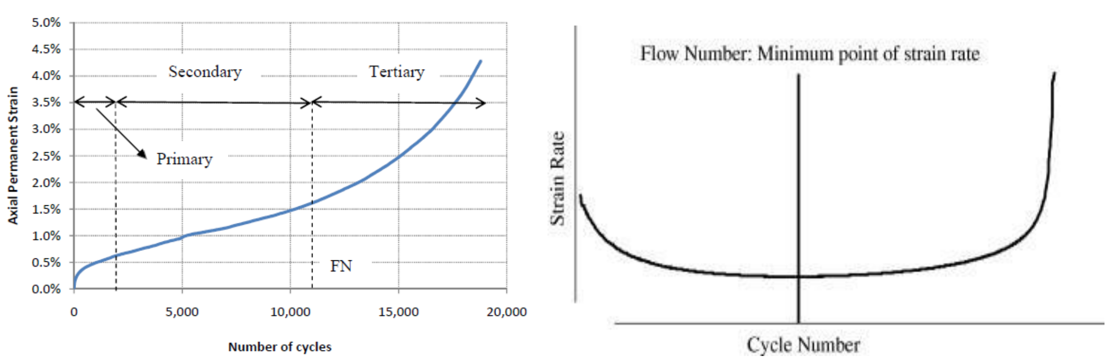

Figure 2.10 Cumulative Permanent Axial Strain vs. Number of Cycles (13; 14).

Under primary flow, there is a decrease in the strain rate with time. Then, with continuous repeated load application, the next phase is the secondary flow state, which is characterized by a relatively constant strain rate. The material enters tertiary flow when the strain rate begins to increase dramatically as the test progresses. Tertiary flow indicates that the specimen begins to deform significantly and the individual aggregates that make up the skeleton of the mix move past each other. The point or cycle number at which pure plastic shear deformation occurs is referred to as the "flow number". Flow number is based on the initiation of tertiary flow or the minimum point of the strain rate curve as shown in Figure 2.10 (3; 11; 13; 14).

The classic power-law model, mathematically expressed by Equation 1, is typically used to analyze the test results:

$\varepsilon_p = aN^b$ Eq. 1

where, ε_p is the plastic/permanent strain, N is the number of load repetitions/cycles, and a and b are the regression constants. Only the secondary stage can be mathematically modeled because of the constant strain rate.

Shear Repeated Load Test

The Superpave Shear Tester (SST), as shown in Figure 2.11, is used to perform the repeated load test in shear or what is called the "Repeated Shear at Constant Height (RSCH) Test". Two main mechanisms were assumed in the development of RSCH test. First mechanism is related to the asphalt binder modulus: stiffer binders increase the mixture resistance to rutting because they minimize shear strains in the aggregate skeleton under each load cycle. The rate of accumulation of permanent deformation is strongly related to the magnitude of the shear strains.

The second mechanism is related to the aggregate structure stability: the axial stresses create a confining pressure which help to stabilize the mixture. A well-compacted mixture with a strong aggregate structure will develop high axial forces at very low shear strain levels. Poorly compacted mixtures can also generate similar levels of axial stresses, but they will require much higher shear strain (12; 15).

During the repeated shear test, those two mechanisms are free to fully develop their relative contribution to the resistance of rutting, because they are not constrained by imposed axial or confining stresses.

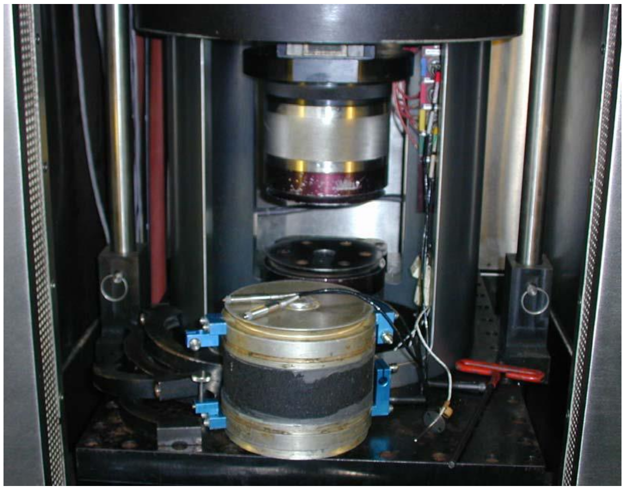

Figure 2.11 Superpave Shear Tester (SST) (15).

Figure 2.12 shows how the amount of permanent shear deformation accumulates with increasing load repetitions (11; 15). The development of permanent shear strain with applied repeated load is similar to the FN test. The specimen deforms quite rapidly during the first stage, the first several hundreds of load cycles. The rate of unrecoverable deformation per cycle decreases and becomes quite steady for many cycles in the secondary stage. At some number of loading cycles, the deformation begins to accelerate dramatically, leading towards failure in the tertiary stage of the curve (11; 15).

The results from the RSHC test showed very good coloration with the in-service pavement rutting. Asphalt Institute set up criteria, as shown in Table 1, for interpreting RSCH maximum permanent shear strain (13).

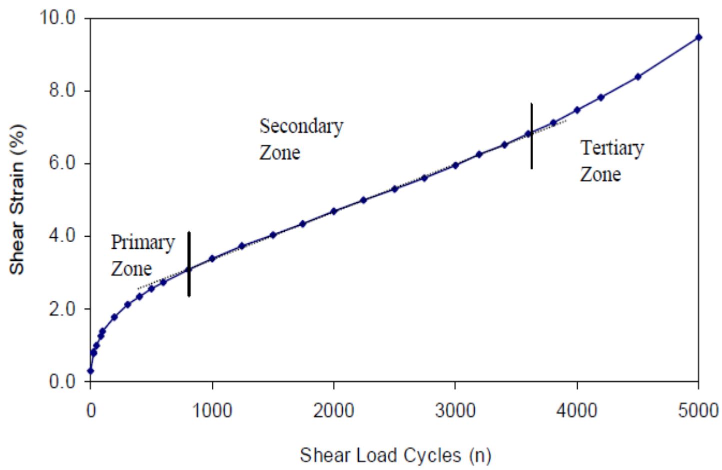

Figure 2.12 Typical Repeated Shear at Constant Height (RSCH) Test Data (13).

Table 2.1 Evaluating of Rut Resistance Using RSCH Permanent Shear Strain (13).

RSCH Maximum Permanent Shear Strain, %	**Rut Resistance**
< 1.0	Excellent
1.0 to < 2.0	Good
2.0 to < 3.0	Fair
> 3.0	Poor

Dynamic Modulus Test (AASHTO TP79)

Dynamic Modulus, also known as |E*|, can also be an indicator of the deformation characteristics of the asphalt mix. The dynamic modulus consists of two main components: the elastic or storage modulus, and the viscous or loss modulus. On the test, a haversine axial compressive stress is applied to a specimen at a given temperature and loading frequency. The test is conducted at a range of frequencies, usually at 25, 10, 5, 0.5, 0.1 Hz, and at a range of temperatures, usually at 40, 70, 100, and 130°F (4.4°C, 21.1°C, 37.8°C and 54°C, respectively). The applied stress and the resulting recoverable and permanent axial strain responses of the specimen are measured and used to calculate the dynamic modulus (14).

Dynamic modulus tests differ from the repeated load tests in their loading cycles and frequencies. While repeated load tests apply the same load several thousand times at the same frequency, dynamic modulus tests apply a load over a range of frequencies. The dynamic modulus test is more difficult to perform than the repeated load test since a much more accurate deformation measuring system is needed (12).

The dynamic modulus test measures a specimen's stress-strain relationship under a continuous sinusoidal loading. For linear (stress-strain ratio is independent of the loading stress applied) viscoelastic materials this relationship is defined by a complex number called the "complex modulus" (E*) (15) as seen in Equation 2:

Dynamic Modulus, $E^* = |E^*|\cos\varphi + i|E^*|\sin\varphi$ Eq. 2

where,

$|E^*|$= Complex/Dynamic modulus,

φ = Phase/Lag angle by which ε_0 lags behind σ_0, for pure elastic material φ = 0°, and for pure viscous material φ = 90°, and

i= imaginary number.

The absolute value of the complex modulus, |E*|, is defined as the dynamic modulus and is calculated as shown in Equation 3:

Dynamic Modulus, $|E^*| = \frac{\sigma_0}{\varepsilon_0}$ Eq. 3

where,

σ_0= peak stress amplitude, and

ε_0= peak recoverable axial strain amplitude.

The dynamic modulus test can be advantageous because it can also measure a specimen's phase angle (φ), which is the lag between peak stress and peak recoverable strain. The complex modulus, E*, is actually the summation of two components: 1) the storage or elastic modulus component and 2) the loss or viscous modulus. It is an indicator of the viscous properties of the material being evaluated (12; 15). Figure 2.13 shows a schematic of a typical dynamic modulus test.

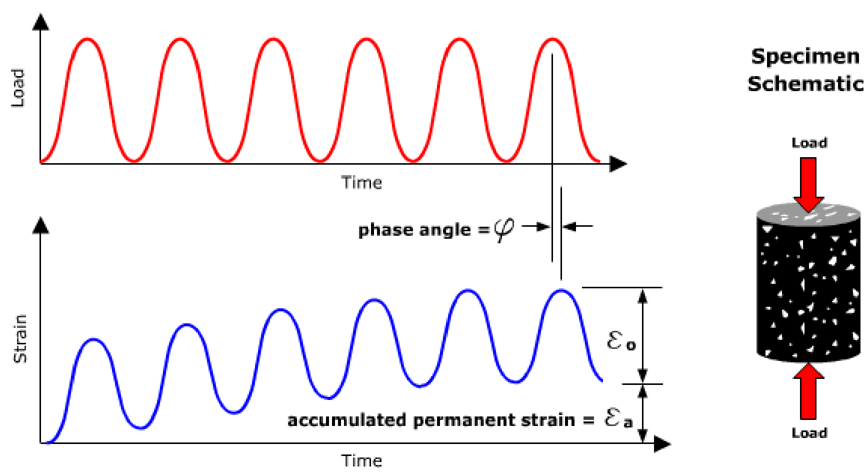

Figure 2.13 Schematic of a Typical Dynamic Modulus Test (12).

The dynamic modulus tests can be conducted confined and unconfined. The unconfined test mode doesn't measure the phase angle. Thus, it is recommended to use the confined test mode to run the dynamic modulus test for permanent deformation evaluation purposes.

Shear Dynamic Modulus (AASHTO T7)

The shear dynamic modulus test, also is known as the frequency sweep at constant height (FSCH) test, uses the same mathematical concepts as those used by dynamic modulus. Similarly, the term E* is replace by G* to denote shear dynamic modulus and σ_0 and ε_0 are replaced by τ_0 and γ_0 to denote shear stress and axial strain respectively (12; 15). The shear dynamic modulus can be done by using weather the Superpave Shear Tester (SST) apparatus or by using Field Shear Tester (FST).

The SST FSCH is done in a constant strain mode. On the SST FSCH test, the sample is prepared with a diameter of 150 mm, and a height of 50 mm, then glued to two plates and inserted to the SST apparatus. A horizontal strain is applied to the sample at a range of frequencies, usually from 10 to 0.1 HZ using a haversine load. The sample's height is kept constant throughout the test by compressing or pulling it vertically as needed. The SST FSCH test is highly sophisticated and needs highly trained operators, which makes it impractical and expensive to run.

The FST FSCH is done in a constant stress mode. The FST FSCH test is a simpler, less expensive, and less complicated version of the SST FSCH test. Unlike the SST FSCH test, the samples height during the FST FSCH is kept constant by using rigid spacers attached to the specimen ends, and shears the sample in the diametral plane. Figure 2.14 shows pictures of the SST apparatus and the samples before and after the test.

Figure 2.14 Superpave Shear Tester, Loading Chamber, Prepared Sample, and Samples after Test (12).

2.2.2.3. Simulative Tests – Laboratory Wheel-Tracking Devices

Recently, laboratory wheel tracking device become more popular among the U.S. transportation agencies. The wheel tracking devices provide reasonable correlated results to the in-service pavement rutting. Wheel tracking devices try to simulate the actual rutting development in the pavement by rolling a small loaded wheel device repeatedly across a prepared asphalt samples, and measure the resulted rut depth. Some newly developed wheel tracking devices can capture the moisture susceptibility effect of the tested samples in addition to measuring the rut depth.

Several laboratory wheel tracking devices currently are being used in the U.S. the most current laboratory wheel tracking devices used in the U.S. include the Georgia Loaded Wheel Tester (GLWT), Asphalt Pavement Analyzer (APA), Hamburg Wheel Tracking Device (HWTD), LCPC (French) Wheel Tracker, Purdue University Laboratory Wheel Tracking Device (PURWheel), and one-third scale Model Mobile Load Simulator (MMLS3). Figures 2.15 through 2.17 show pictures of these devices (16).

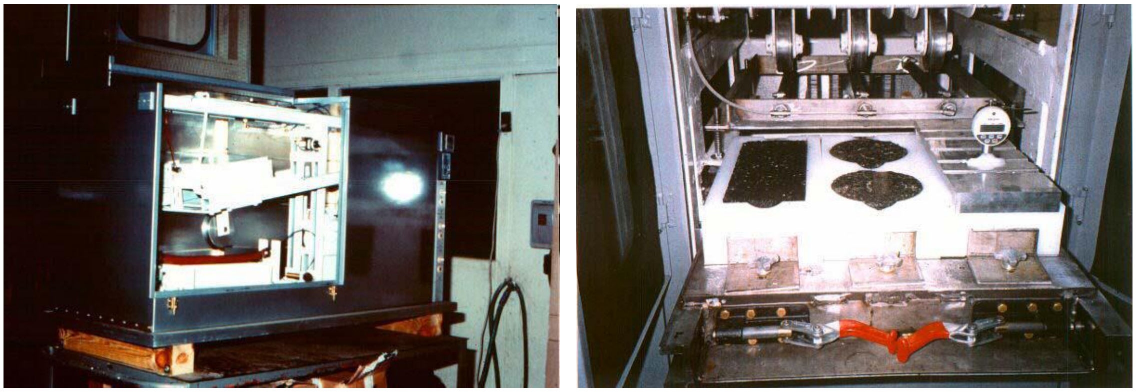

Figure 2.15 Georgia Loaded Wheel Tester (GLWT) (left), and Asphalt Pavement Analyzer (APA) (right) (16).

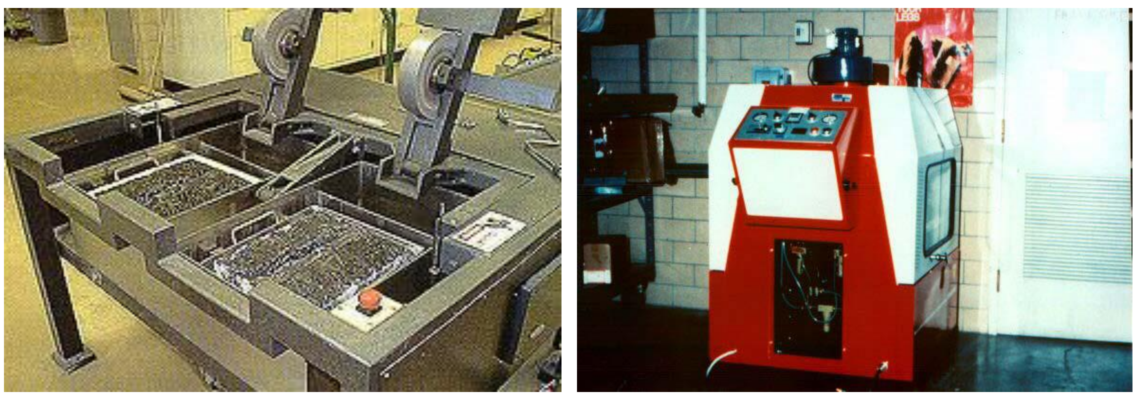

Figure 2.16 Hamburg Wheel Tracking Device (HWTD) (left), and LCPC (French) Wheel Tracker (right) (16).

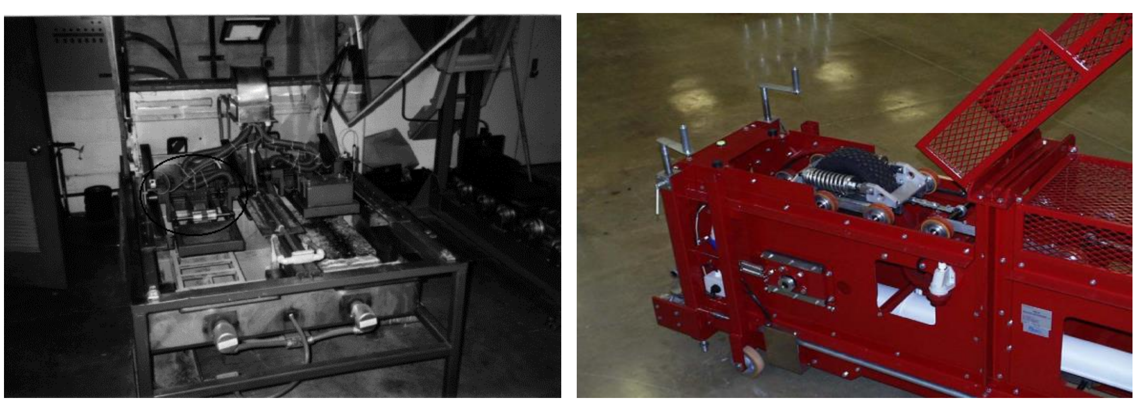

Figure 2.17 Purdue University Laboratory Wheel Tracking Device (PUR Wheel) (left), and one-third scale Model Mobile Load Simulator (MMLS3) (right) (16).

Cooley et al, 2000 (12; 16) reviewed U.S. loaded wheel testers and found:

- Taking the pavement location temperature and environmental conditions into consideration when running the test can provide results that correlate reasonably well to actual field performance.
- Wheel tracking devices can be used to evaluate the influence the effect of asphalt binder type on the rutting resistance of the asphalt pavement.
- Devices with the capability of running tests in air or in submerged mode can provide the user with more pavement evaluation options.

Generally, wheel tracking devices results are affected by the test parameters such as load, number of wheel passes, temperature, or presence of moisture. The users should identify the test parameters that match their pavement condition.

2.3. Moisture Sensitivity in Asphalt Mixtures

Moisture damage can be defined as the loss of strength and durability of asphalt mixtures due to the effects of moisture (17). The lake of enough bond strength between the asphalt binder and the fine aggregate in the asphalt mixture can lead to moisture damage. Moisture damage is the result of moisture interaction with the asphalt binder-aggregate adhesion within the asphalt mixture, making it more susceptible to moisture during cyclic loading (17; 18). This weakening, if severe enough, can result in stripping as shown in Figure 2.18 (18).

Figure 2.18 Fatigue Cracking Caused by Stripping (18).

2.3.1. Asphalt Binder – Aggregate Adhesion Theories

Adhesion bond between the asphalt binder and the aggregate surface is the main contributor to the asphalt mixture moisture resistance. Understanding how the adhesion bond forms between the asphalt binder and the aggregate surface can be very helpful in preventing the moisture damage from occurring in the asphalt mixture.

Previous research showed that there are four adhesion mechanisms that can describe the adhesion between the asphalt binder and aggregate surface: 1) chemical adhesion, 2) surface energy, 3) molecular orientation, and 4) mechanical adhesion.

2.3.1.1. Chemical Adhesion

Chemical adhesion occurs as a result of forming water-insoluble components caused by a chemical reaction between the acidic and basic components of asphalt and aggregate surface. Some research studies suggested that the bond formed by chemical sorption might be necessary in order to minimize stripping potential in asphalt–aggregate mixtures (17). In general, some aggregates with acidic surfaces don't react as strongly with asphalt binders, which may not be enough to counter other moisture damage causing factors (18).

2.3.1.2. Surface Energy

Surface energy can be explained in terms of relative wettability of aggregate surface by water or asphalt. The surface tension between the asphalt binder and the aggregate at the wetting line is less than the surface tension between the water and the aggregate due to its higher viscosity. Thus, if all the three are in contact (water, aggregate, and asphalt binder), water more likely replaces asphalt binder. This will result in less aggregate coating by asphalt binder and eventual striping. The interfacial tension between aggregate and asphalt binder depends on the asphalt type, aggregate type, and the aggregate surface roughness (17; 18).

2.3.1.3. Molecular Orientation

The molecular orientation or the structure of asphalt molecules at the aggregate-asphalt surface is related to the surface energy of the asphalt binder. When the asphalt molecules contacts the aggregate surface, they tend to be oriented relatively to the ions on the aggregate surface, which can cause a weak attraction between the asphalt binder and the aggregate surface. If the water molecules, which are dipolar, are more polar than the asphalt binder molecules, they might be able to satisfy the aggregate surface energy and lead to weak asphalt-aggregate bond ends up with stripping (17; 18).

2.3.1.4. Mechanical Adhesion

Mechanical adhesion depends on the physical properties of the aggregate such as surface texture, porosity or absorption, surface coatings, surface area, and particle size (17). Asphalt binder gets into the aggregate surface pores and irregularities, and when it hardens it causes a mechanical lock. When moisture interferes with the asphalt binder penetration that can reduce the mechanical lock and lead to stripping (18). Good mechanical lock can improve the nature of the chemical bond between the asphalt binder and aggregate surface even in the presence of water (17).

2.3.2. Cohesion Theories

Cohesion is developed in the asphalt mastic, asphalt binder mixed with fine aggregate, and depends on the rheological properties of the asphalt binder. The mastic's resistance to microcrack development is highly influenced by the dispersion of the mineral filler. Therefore, it can be inferred that the cohesive strength is controlled by the combination and the interaction of asphalt binder and the mineral filler (17; 19). Water can affect the cohesion of asphalt mastic in several ways such as, weakening the mastic due to water saturation and void swelling (20).

Schmidt and Graf, 1972 (17; 21), showed that an asphalt mixture will lose about 50 percent of its modulus upon saturation. The loss may continue with time, but upon drying, the modulus can be completely recovered. This is shown graphically in Figure 2.19.

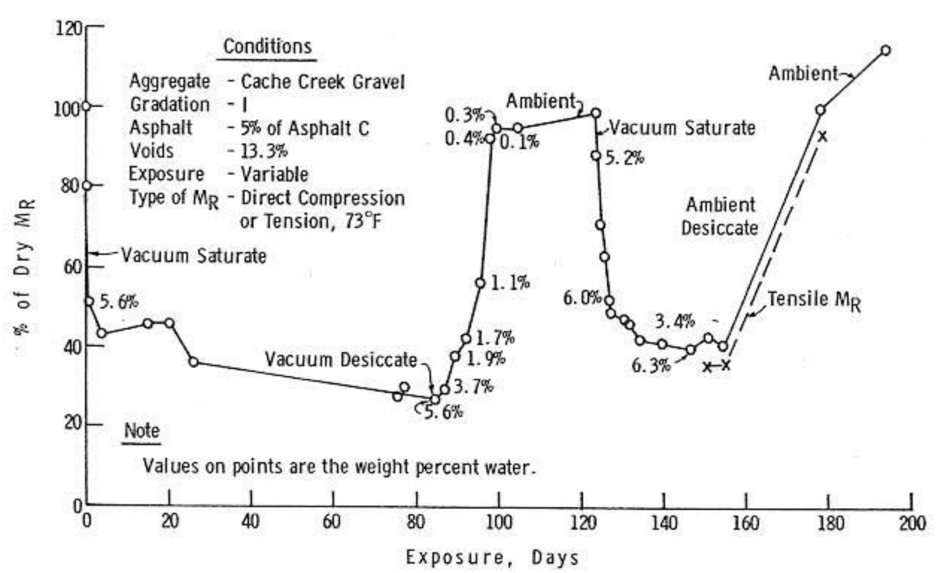

Figure 2.19 Effect of Moisture on Resilient Modulus is Reversible (17; 21).

2.3.3. Adhesive Failure versus Cohesion Failure

The moisture damage in asphalt mixture can be due to a cohesion failure within the mastic, or due adhesion failure at the aggregate-asphalt binder or mastic interface. Both failure modes can be related to the nature of the asphalt mastic and the asphalt binder film thickness around aggregate particles. Thus, it can be said that asphalt mixtures with thin asphalt film tends to fail in tension by adhesive bond rupture, while asphalt mixture with thicker asphalt film thickness tends to fail in a cohesive failure mode due to the damage within mastic. The determination of asphalt film thickness that differentiate the two modes of failure depends on the rheological properties of the used asphalt binder, the amount of damage the asphalt or mastic can withstand prior to failure, the rate of loading, and the temperature at the time of testing (17; 19). Figure 2.20 shows adhesive versus cohesive bond failure based on asphalt film thickness (17).

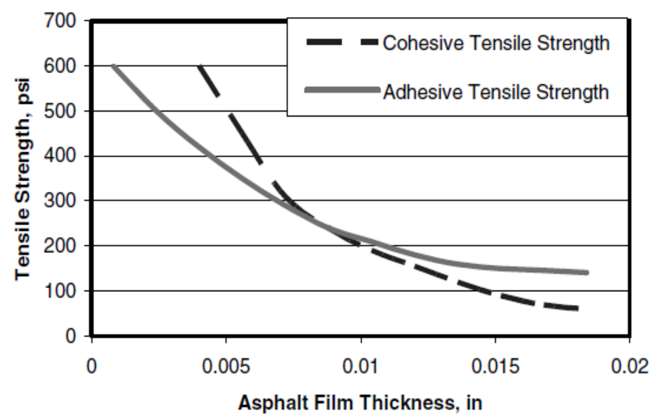

Figure 2.20 Adhesive versus Cohesive Bond Failure based on Asphalt Film Thickness (17).

2.3.4. Factors Affecting the Moisture Resistance

Moisture damage is a complex phenomenon, that can't be attributed to one single failure mechanism, rather it is caused by combination of the previously discussed mechanisms. It can be said that any factor that increases the moisture content in the asphalt mixture, or reduces the adhesion between the asphalt binder and the aggregate surface, can increase the moisture susceptibility of that mixture. Some of the factors that is found to have a great influence on the moisture susceptibility of the asphalt mixtures are discussed in the following section, however none of these factors is proven to be alone the cause of moisture damage in asphalt mixtures (18).

2.3.4.1. Effect of Asphalt Binder Characteristics

The rheological properties of asphalt binder are important factor in shaping the moisture sensitivity of the asphalt mixture. For example, high viscosity can indicate a high concentrations of large polar molecules, or asphaltenes, which can create greater adhesion tension and molecular orientation adhesion. On the other hand low viscosity indicates a low concentration of asphaltenes, which results in higher potential of stripping. Similarly, other asphalt binder components such as sulfoxides, carboxylic acids, phenols and nitrogen bases can influence the moisture susceptibility of asphalt mixtures (17; 18).

2.3.4.2. Effect of Aggregate Characteristics

Generally, acidic aggregates are considered hydrophobic, repulse water, while basic aggregates are hydrophilic, attract water (22). However, it can’t be said that all hydrophobic aggregates are completely resistance to stripping. Other aggregate properties such as surface chemistry and aggregate porosity and pore size also influence the moisture susceptibility of asphalt mixtures. Aggregate surfaces with more acidic chemical nature tend to form asphalt binder-aggregate bond slower than other aggregates. The presence of some chemical elements in the aggregate surface such as iron, magnesium, calcium and perhaps aluminum are considered beneficial, while sodium and potassium are considered harmful (18). Aggregate surfaces with large pores size can increase stripping susceptibility. High porosity leads to high asphalt binder absorption and less available asphalt binder for coating. If high porosity is ignored during the mix design it can lead to thin asphalt binder film and less coating, which can cause faster aging and stripping (17; 18; 20; 22).

2.3.4.3. Effect of Traffic

Continuous increase of traffic loading in the presence of water in the asphalt pavement structure can increase the moisture damage. When the water is trapped in the aggregate pores, the traffic loads will compress the pores and create a buildup pressure. The resulted buildup pressure could remove the asphalt binder from the aggregate surface and lead to stripping. Additionally, the traffic passes tend to move the water in the asphalt pavement surface, which causes a hydraulic scouring action. Hydraulic scouring could contribute to the removal of the asphalt binder from aggregate surface (18; 22).

2.3.4.4. Effect of Air Voids

Air void adjustment is very important aspect in mix design, which indirectly affect the moisture susceptibility of the asphalt mixture. This aspect can be adjusted in the laboratory by selecting the correct asphalt content. Lake of enough field compaction can cause a high undesirable air voids content. Asphalt mixtures with high volume of air voids (usually 8.0% or greater) provide more space for asphalt binder or water molecules to penetrate into the pavement surface. This can cause higher absorption and lake of enough coating problems in case of asphalt binder and pore pressure and ice expansion problems in case of entrapped water (18; 22).

2.3.4.5. Effect of Climate and Construction Weather

Cool weather during construction can prevent enough compaction, which leads to higher air voids, and relatively permeable pavement. This can increase the potential of moisture damage in the asphalt pavement. Similarly the wet climate, freeze-thaw cycles, and temperature overbalancing can allow more moisture in the asphalt pavement, which also can increase probability of moisture damage Incidence (17; 18; 22).

2.3.5. Measuring the Moisture Susceptibility

The test methods used to evaluate the moisture susceptibility of asphalt mixtures can be dived into two main types: tests to be done on loose mixtures (refer to Table 2.2) and tests to be done on compacted specimens (refer to Table 2.3) (22).

Table 2.2 Moisture Susceptibility Tests on Loose Mixtures (22)

Test	ASTM	AASHTO	Other
Methylene Blue			Technical Bulletin 145, International Slurry Seal Association
Film Stripping			(California Test 302)
Static Immersion	D1664*	T182	
Dynamic Immersion			
Chemical Immersion			Standard Method TMH1 (Road Research Laboratory 1986, England)
Surface Reaction			Ford et al. (1974)
Quick Bottle			Virginia Highway and Transportation Research Council (Maupin 1980)
Boiling Test	D3625		Tex 530-C Kennedy et al. 1984
Rolling Bottle			Isacsson and Jorgensen, Sweden, 1987
Net Adsorption			SHRP A-341 (Curtis et al. 1993)
Surface Energy			Thelen 1958, HRB Bulletin 192 Cheng et al., AAPT 2002
Pneumatic Pull-off			Youtcheff and Aurilio (1997)
* No longer available as ASTM standard.			

Table 2.3 Moisture Susceptibility Tests on Compacted Specimens (22)

Test	ASTM	AASHTO	Other
Moisture Vapor Susceptibility			California Test 307 Developed in late 1940s
Immersion–Compression	D1075	T165	ASTM STP 252 (Goode 1959)
Marshal Immersion			Stuart 1986
Freeze–Thaw Pedestal Test			Kennedy et al. 1982
Original Lottman Indirect Tension			NCHRP Report 246 (Lottman 1982); Transportation Research Record 515 (1974)
Modified Lottman Indirect Tension		T283	NCHRP Report 274 (Tunnicliff and Root 1984), Tex 531-C
Tunnicliff–Root	D4867		NCHRP Report 274 (Tunnicliff and Root 1984)
ECS* with Resilient Modulus			SHRP-A-403 (Al-Swailmi and Terrel 1994)
Hamburg Wheel Tracking		T234	1993 Tex-242-F
Asphalt Pavement Analyzer		T312	
ECS*/SPT**			NCHRP 9-34 2002-03
Multiple Freeze–Thaw			
* Environmental Conditioning System			** Simple Performance Tester

Most of the major/famous moisture susceptibility laboratory tests evaluate the moisture resistance of the asphalt mixture as a whole not each individual element of the mixture. These tests can provide comparative results by comparing two different sets of asphalt samples with two different moisture conditions. However, these test can't predict the degree of moisture damage in asphalt mixture. Some of the major tests are discussed in the following section:

2.3.5.1. Boiling Test (ASTM D3625)

The Texas boiling test procedure was developed by Kennedy et al. 1982; 1984 (23; 25). On the test, the loose mix is added to boiling water. After 10 minutes, the mixture is left to cool down while the stripped asphalt binder is skimmed away (24). Then, the percentage of total visible area of aggregate surface that retains its asphalt binder coating is measured.

The test is simple but is subjective, does not involve any strength determination and examining the fine aggregate is difficult (18; 24).

2.3.5.2. Static-Immersion Test (AASHTO T182)

During the test the loose mixture is cured for 2 hours at 60° C then cooled down to room temperature. Then, the sample should be immersed in a glass jar filled with 600 mL of distilled water and kept undisturbed for 16 to 18 hours. A visual observation should be made through the glass to determine the percentage of total visible area of aggregate surface that retains its asphalt binder coating.

Solaimanian et al., 2003, (24) stated that "…..the total visible area of the aggregate is estimated as either less than or greater than 95%. This is a major limitation of the test because the results are decided purely on the basis of a subjective estimate of less than or greater than 95%. Test results have indicated that placing samples at 60° C bath rather than 25° C for 18 hours increases the amount of stripping".

This test is simple but subjective and does not involve any strength determination (18; 24).

2.3.5.3. Modified Lottman Test (AASHTO T283)

The AASHTO T 283 test procedure is used to evaluate the resistance of the asphalt mixture to moisture damage. The test is conducted on dry and wet conditioned specimens measuring 4 inches (100 mm) in diameter and 2.56 inches (65.0 mm) in height. The specimens are loaded until failure at a rate of 2 inches per minute (50.8 mm per minute). Two types of data are obtained from this test. The first is the indirect tensile strength (ITS) of the dry and wet conditioned specimens. The second is the tensile strength ratio (TSR), calculated by dividing the average ITS values of the wet conditioned specimens by the average ITS values of the dry conditioned specimens (refer to Figure 2.21). The ITS value is a measure of the strength and durability of the asphalt mixture, whereas the TSR ratio is a measure of its resistance to damage from freezing and thawing.

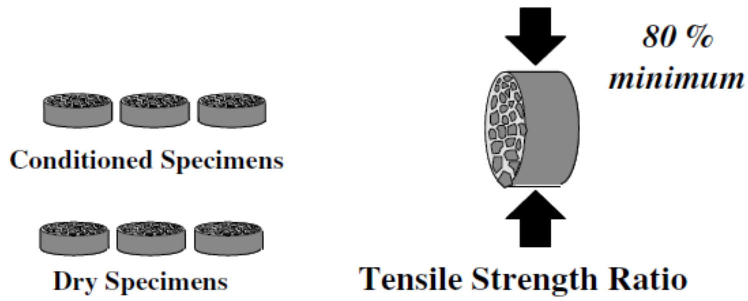

Figure 2.21 Indirect Tensile Test used for Dry and Conditioned Specimens for AASHTO T283 (24).

Although it is expected that the water conditioned samples will have a lower tensile strength, excessively low values indicate the potential for moisture damage (14; 18; 24).

2.3.5.4. Hamburg Wheel-Tracking (HWT) Test (AASHTO T324)

The Hamburg Wheel Tracking device applies a constant load of 685 N through a steel wheel with a diameter of 203.5 mm and a width of 47.0 mm. The tests are run in a water bath that is heated to 50 °C after the test specimens are conditioned for 30 minutes. Figure 2.22 shows the Hamburg Wheel Tracking device and specimens ready for testing. The test is completed when the wheel has passed over the specimens 20,000 times for 6.5 hours or when the rut depth exceeds 20 mm.

The Hamburg Wheel Tracking Device measures rut depth throughout the test and reports four properties: 1) post-compaction consolidation, 2) creep slope, 3) stripping inflection point, and 4) stripping slope. The post-compaction consolidation occurs at around 1,000 wheel passes that is normally caused by the densification of the mixture. The creep slope is used to measure the rutting susceptibility of the mixture that measures the permanent deformation caused by the wheel passes. The stripping inflection point and the stripping slope are used to measure damaged caused by moisture. A mixture with a stripping inflection point less than 10,000 passes should be considered as moisture susceptible.

The specimens had a target air void content of 7.0 ± 2.0 %. Specimens were compacted with a height of 60 mm to fit the mold for the Hamburg Wheel Tracking device. 7.5 mm of material was removed from one side of the specimen so that they fit together in the specimen tray. Figure 2.23 shows the dimensions of the specimen and the mold.


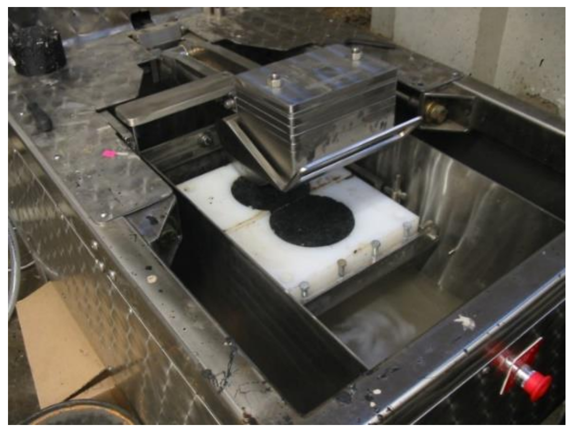

Figure 2.22 Hamburg Wheel Tracking Device (left) and Specimens Ready for testing (right).

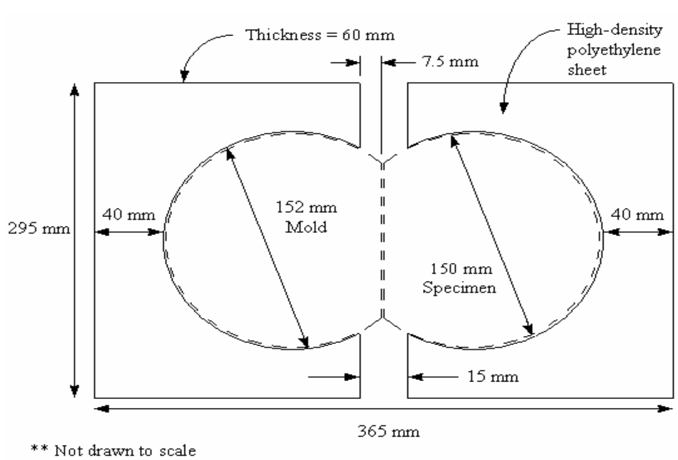

Figure 2.23 Dimensions of the Specimen and the Mold.

Most of the moisture susceptibility tests tend to lake good repeatability and reproducibility of test results. Additionally, small variations in any mix design parameters such as air voids, can substantially influence the test results (18).

2.3.6. Preventive Actions against Moisture Damage

There are several preventive actions that can be taken to prevent or at least reduce the moisture damage of the asphalt mixture. Some of the available preventive actions or technics include material selection, construction practices, mix design, or asphalt mixture additives. Some of the available actions are discussed below;

2.3.6.1. Aggregate Selection

During the design of asphalt mix, the selection of low porosity aggregates with clean surfaces and rough texture is highly recommended.

2.3.6.2. Reduction of Pavement Permeability

As mentioned before, the adjustment of asphalt pavement in place air voids can help in the reduction of pavement permeability, which reduces the impact of moisture penetration and the asphalt pavement surface. This can be achieved during construction by providing enough compaction to achieve good level of air voids, usually 5.0 to 6.0 % (refer to Figure 2.24) (18; 26). Also, lift Thickness, and aggregate gradation can influence the permeability of the asphalt pavement layer (refer to Figure 2.25) (18; 26). Moreover, the preventive maintenance practices such as slurry seal, chip seal, or surface wearing course can waterproof the asphalt pavement surface and then reduce the penetration of water molecules into the surface.

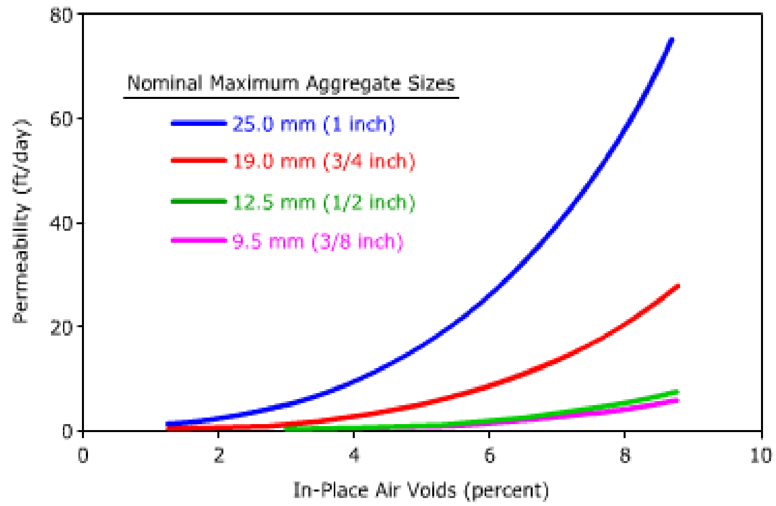

Figure 2.24 In-place air voids vs. permeability for different nominal maximum aggregate sizes (redrawn from Cooley et al., 2002) (18; 26).

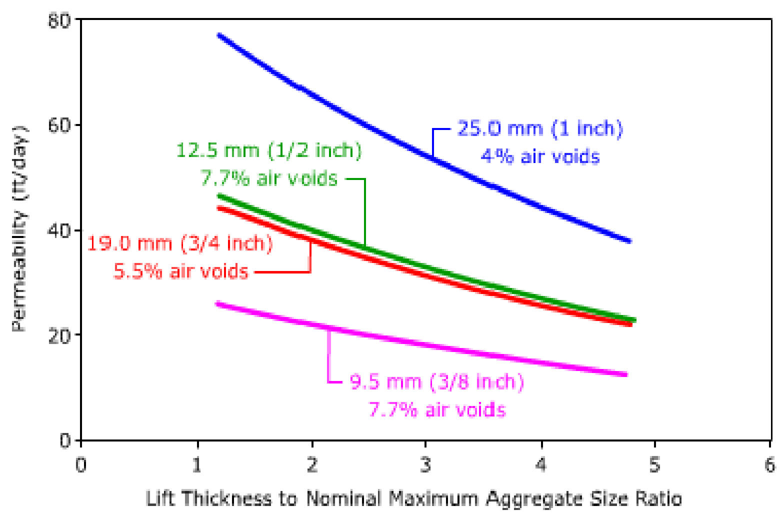

Figure 2.25 Permeability vs. the lift thickness to NMAS ratio (redrawn from Cooley et al., 2002) (18; 26).

2.3.6.3. Anti-Strip Additives

In many cases the addition of an additive is recommend to prevent or reduce the moisture damage potential of the asphalt mixtures. The most common used anti-strip additives include chemicals or hydrated lime.

Chemicals: They basically reduce the surface tension in the asphalt binder as well as impart an opposite electrical charge to the aggregate surface's electrical charge, which allows for better wetting or coating. Most of the available chemicals are added to the asphalt binder at a rate of 0.1 to 1.0 % by weight. The recommended practice by the anti-strip additive manufacture should be carefully followed to guarantee that all the additive reaches the critical asphalt binder-aggregate interface (18).

Hydrated Lime: When hydrated lime is added to the aggregate surface, it replaces the negative ions on the aggregate surface with positive calcium ions, which promotes for better asphalt binder-aggregate adhesion. Moreover, hydrated lime reacts with both asphalt binder molecules (carboxylic acid) and aggregate molecules (acidic OH groups). This reaction results in more readily absorbed molecules on the aggregate surface and less likely to associate with water molecules. Generally, hydrated lime is added to the aggregate at a rate from 0.5 to 1.5 % by dry weight of aggregate. In order to active the hydrated lime, a small percentage of moisture should be existed at the time of addition (18). The national lime association recommends several methods that can be used to add hydrated lime to the aggregate such as dry injection into drum mixers, dry lime on damp aggregate method, or slurry method (27).

2.4. Asphalt Mixtures with Recycled Asphalt Pavement (RAP)

The asphalt pavement industry has always endorse the use of reclaimed asphalt pavement (RAP). The earliest recycling asphalt pavement dates back to 1915 (28). However, significant use of RAP in hot-mix asphalt (HMA) really started in the mid-1970s due to extremely high asphalt binder prices as the result of the oil embargo. Using RAP can significantly reduce the cost of HMA paving, conserve the energy, and help protect the environment (29). Additionally, many studies concluded that properly designed and constructed RAP mixes, exhibit similar performance to HMA mixes (30). Moreover, RAP production and processing technics have greatly enhanced in the past few years. Usually, RAP is processed into smaller sizes through RAP crushing and fractionation. The fractionated material is more uniform and more likely to be used in higher percentages with HMA mixtures. Additionally, fractionated material provides better handling of higher percentages of RAP at the mix plant without negative outcomes. Therefore, the production of quality HMA containing 25 percent RAP or more became more available.

However, a recent FHWA report stated, “Average RAP use is estimated at 12% in HMA…it is unknown why over half of the country uses less than 20 percent RAP in HMA” (31). The most difficult aspect of high-RAP mix design is meeting the volumetric mix design criteria due to the large amount of fine aggregate material introduced to the HMA mix by the RAP materials (32).

High-RAP contents also require changes in the performance grade of the virgin binder used because of the increased stiffness of the aged RAP binder. McDaniel et al. (33) reported that, based on indirect tensile strength, the stiffness of mixtures with a high RAP content (>20%) were so high that they may be susceptible to low temperature cracking. Beeson et al. (34) recommended that up to 22% RAP can be added to the mixture before changing the low temperature grade of a -22 binder and up to 40% RAP can be added to a mixture as long as the virgin binder grade is one grade lower than what is expected.

In a 2009 survey conducted by NCDOT, 9 State DOT's reported requiring fractionation and Wisconsin is reported to allow an increase of 5 percent binder replacement for surface mixes if fractionation is used (31).

2.4.1. RAP Fractionation Methods

There are two main methods of RAP fractionation that are widely used; "Fractionated RAP" method, and "Optimum Fractionated RAP" method. The two methods are discussed below;

2.4.1.1. Fractionated RAP Method

The fractionated RAP method directly targets the fine RAP particles by physically removing the smallest of these materials from the original stockpile during the processing operation. The removed sieve size is determined based on the analysis of the recovered aggregate gradation of RAP materials retained on each sieve size.

2.4.1.2. Optimum Fractionated RAP Method

The optimum fractionated RAP method splits the original RAP stockpile into two separate coarse and fine fractionated RAP (FRAP) stockpiles. The new 'coarse FRAP' and 'fine FRAP' stockpiles can then be re-proportioned to limit the percentage of fine FRAP included in the mix.

2.5. WMA Technologies

WMA technologies were first introduced at the Bitumen Forum of Germany in 1997 as one way to reduce greenhouse gas emissions (35). The National Asphalt Pavement Association (NAPA) has been instrumental in bringing these technologies into the United States with several demonstration projects being constructed since 2004 (1). WMA has been described as "…a group of technologies which allow a reduction in the temperatures at which asphalt mixtures are produced and placed." (36). There are many different processes and products that can be used to achieve this reduction in temperature, but generally WMA technologies can be classified into four main categories:

1. Organic additives – generally wax additives such as Fischer-Tropsch or Montan waxes;
2. Chemical additives – surfactants or other chemical additives;
3. Water-bearing additives – synthetic zeolites; and
4. Water-based processes – non-additive processes based on foaming

2.5.1. Organic Additives

Organic additives are waxes that have been used to reduce the viscosity of the asphalt binder at temperatures higher than their melting points. Figure 2.26 shows the conceptual behavior of an asphalt binder modified with an organic additive versus an unmodified binder. It is evident that for the same temperature beyond water evaporation point, the asphalt binder modified with organic additive has a much lower viscosity than the unmodified binder. An example of organic additives is Sasobit® supplied by Sasol Wax Americas, Inc., which is described as a modifier or an "asphalt flow improver," and is the most often used organic additive in the United States (37). Figure 2.27 shows Sasobit® in Pastille (ca 4 mm diameter) & Prill (ca 1 mm diameter) forms. Another example of organic additives is LEADCAP technology. LEADCAP additive is a new Polyethylene (PE) Wax-based WMA additive developed by the Korean Institute of Technology (KICT) (Figure 2.27). The LEADCAP additive works by reducing the viscosity of the asphalt. Its melting point is 100 °C and crystallization point is 90 °C. The LEADCAP additive controls the crystallization so that it does not become brittle at low temperature. The LEADCAP additive is positively charged to enhance the bonding of asphalt binder to negatively charged aggregate surface (38).

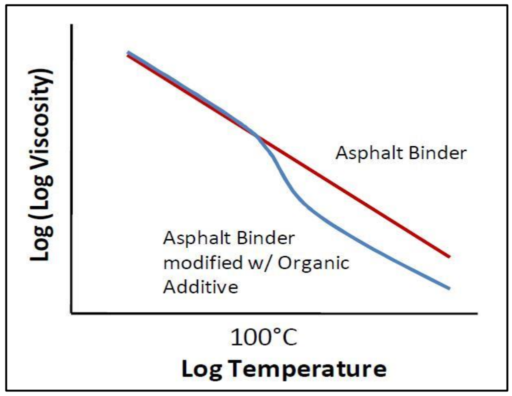

Figure 2.26 Temperature-Viscosity Behavior of Asphalt Binder Modified with Organic Additive (39).

Figure 2.27 Sasobit Pastille (ca 4 mm diameter) & Prill (ca 1 mm diameter) Forms (37).

Figure 2.28 LEADCAP Warm Mix Asphalt Additive (38).

2.5.2. Chemical Additives

Chemical additives, also called surfactants, are a relatively new, emerging group of additives for warm-mix asphalt. These surfactants help the asphalt binder coat the aggregate at lower temperatures. An example of chemical additives is the Evotherm Dispersed Asphalt Technology (DAT), which is produced by MeadWestvaco Asphalt Innovations. The original process was Evotherm™ Emulsion Technology (ET) which consisted of the additive blended in a high asphalt content, water-based asphalt emulsion (~70% solids) that is mixed with hot aggregates to produce asphalt mixtures with a 100°F (55°C) reduction in production temperatures. Evotherm™ requires no plant modifications and simply replaces the liquid asphalt in HMA design. The majority of water in the emulsion flashes off as steam when the emulsion is mixed with the hot aggregates (40). Evotherm is normally added at a rate of 0.5% by weight of binder.

2.5.3. Water-Bearing Additives

In both water-bearing additives and water-based technologies small amounts of water is introduced to asphalt binder to form a controlled foaming effect that leads to a small increase in binder volume and a reduction in its apparent viscosity. Water bearing additives are those synthetic zeolites that have been used to enhance aggregates coating by asphalt at lower production temperatures. Zeolite includes approximately 20% of water trapped in its porous structure. When the zeolite is heated to approximately 85°C, the water is released, and then foamed asphalt is produced (1). An example of water-bearing additives is Advera® (Figure 2.29). Advera® is an aluminosilicate or hydrated zeolite powder, when added to the mixture at a rate up to 0.3% by total weight of mix, asphalt mixtures can be produced at temperatures 50° – 70°F lower than conventional HMA temperatures with no mix design change (41).

Figure 2.29 Close-up picture of Advera® (41).

2.5.4. Water-Based Additives

In Water-Based process foamed asphalt is achieved by adding small amount of water to the original heated asphalt through a mean of nozzle or damped aggregates. The idea of introducing moisture into a stream of hot Asphalt, which causes a spontaneous foaming of the Asphalt (similar to spilling water into hot oil), is to increase the asphalt surface area, which extremely lower its viscosity, and making it ideal for mixing with aggregates; thereby allowing the mix to be handled and worked at lower temperature.

Foaming technology, as shown in Figure 2.30, is believed to be the most cost effective from among the WMA technologies since it does not require any costly additives to be added to the mixtures and more importantly it does not require very expensive plant modifications since the foaming component can be attached to old systems for a reasonable price, without the need for any additional changes (42).

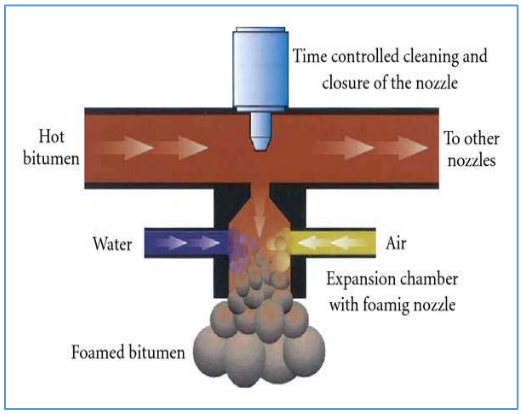

Figure 2.30 Foamed Asphalt Production (43).

2.5.5. Potential Benefits of WMA Technologies

The use of WMA technologies provides a number of potential benefits which may be categorized into three categories: economic, operational and environmental (2). Table 2.4 summarizes a number of the potential benefits of WMA technology which are discussed below:

Table 2.4 Potential Benefits of WMA (2)

Potential Benefit	**Economic**	**Operational**	**Environmental**
Reduced fuel use	X		X
Late season (cool weather) paving		X	
Better workability and compaction	X	X	
Reduced plant emissions of greenhouse gases			X
Increased usage of RAP	X		
Improved working conditions for plant and paving crew			X

2.5.5.1. Reduced Fuel Use

Theoretically, a temperature reduction of 50°F leads to fuel saving of 11 percent. Burner fuel savings with WMA typically range from 11 to 35 percent. It should be known that the range of fuel saving depends on several factors, such as the WMA type, the moisture content of the aggregate, and the plant's design and operation (36).

2.5.5.2. Late Season (cool weather) Paving

In-place WMA production temperatures depends on a number of factors such as haul distance and ambient temperature, but generally are expected to be lower than HMA production temperatures at the same conditions, which means that WMA is compactable at lower temperatures than HMA. Therefore WMA mixture can be produced and will remain compactable for a longer period of time than HMA, which allows paving contractors in most of the United States to extend paving season further into the year (2).

2.5.5.3. Better Workability and Compaction

European practice for WMA showed that WMA can allow contractors to pave in cooler temperatures and still gain paving-related benefits such as obtaining density, hauling the mix longer distances and still have workability to place and compact, and the ability to compact mixture with less effort (35). Better compaction has been listed as a potential benefit of WMA comparing to HMA, which can result in more consistent compaction. WMA users have reported in-place densities comparable to those of HMA (2).

2.5.5.4. Reduced Plant Emissions of Greenhouse Gases

Research studies indicate that the plant emissions are significantly reduced upon the using of WMA technologies. Typical expected reductions are 30 to 40 percent for carbon dioxide (CO_2) and sulfur dioxide (SO_2), 50 percent for volatile organic compounds (VOC), 10 to 30 percent for carbon monoxide (CO), 60 to 70 percent for nitrous oxides (NO_x), and 20 to 25 percent for dust. Actual reductions vary based on a number of factors. Technologies that result in greater temperature reductions are expected to have greater emission reductions (2; 35; 36).

2.5.5.5. Increased Usage of RAP

The usage of RAP in HMA mixture has been limited by many highway agencies because using high amount of RAP will increase the mixture's aging after production, which may lead to early cracking. Lower temperature of production in WMA allows for less aging of the mixture, and thereby increases the usage of RAP in the mixture, which is considered a potential economic benefit for both user and producer.

In Europe, WMA has been successfully produced with up to 50% RAP. In the United States, most of the WMA projects containing RAP have used 20% or less (2; 35).

2.5.5.6. Improved Working Conditions for Plant and Paving Crew

Tests for asphalt aerosols/ fumes and polycyclic aromatic hydrocarbons (PAHs) indicated significant reductions compared to HMA, with results showing a 30 to 50 percent reduction. That means worker exposure to fumes is lesser in WMA when compared to HMA. The lower mix temperature and reduction of visible smoke and odor – or perception thereof – for WMA may contribute to improved working conditions for the paving crew (2; 35).

2.5.6. Mix Design Practices for Warm Mix Asphalt

HMA mixture design and analysis generally consists of five major steps: 1) materials selection, 2) design aggregate structure, 3) design binder content selection, 4) evaluate moisture sensitivity, and 5) performance analysis. Criteria and procedures for steps 1 through 4 for HMA can be obtained from AASHTO M323, Standard Specification for Superpave Volumetric Mix Design, and AASHTO R35, Standard Practice for Superpave Volumetric Design of Hot Mix Asphalt (HMA), respectively (1). Step 5 can be obtained by using standard practices for performance tests that have been developed for

HMA. Performance tests are available for measuring mixture modulus, rutting resistance, and resistance to fatigue cracking and thermal cracking.

The design of WMA requires some changes to the current HMA mix design practices. Those modifications resulted in some differences in mix design between WMA and HMA which are summarized in Table 2.5.

Table 2.5 HMA versus WMA Mix Design Procedure (14)

Criteria		Special Mixture Design Consideration and Methods for WMA	HMA SuperPave Volumetric Mix Design
WMA Process		Producer Selected	N/A
Consensus Aggregate Requirements.		AASHTO M323	AASHTO M323
Gyratory Compactive Effort	$N_{initial}$	AASHTO R35	AASHTO R35
	N_{des}		
	N_{max}		
Mixture Densification	$N_{initial}$	AASHTO M323	AASHTO M323
	N_{des}		
	N_{max}		
Min. VMA at N_{des}		AASHTO M323	AASHTO M323
VFA at N_{des}			
Dust To Binder Ratio at N_{des}			
Short-Term Conditioning for Volumetrics, hours		2 at WMA Compaction Temperature.	2 at HMA Compaction Temperature.
Short-Term Conditioning for Performance, hours		2 at WMA Compaction Temperature.	4 at 135° C (275°F)
Binder Selection		Modified PG Grade	PG Grade
Mixing & Compaction Temperatures.		Coating	Viscosity-Temperature. Chart
		Workability	
		Compactability	
Moisture Sensitivity		AASHTO T283	AASHTO T283
Rutting Resistance		Flow Number Test	N/A

The approach that have been used as a mix design procedure is based on the recommendations from the research conducted under the National Cooperative Highway Research Program (NCHRP) Project 9-43, Mix Design Practices for Warm Mix Asphalt, which concluded that only minor modification of current HMA mix design practice is needed to address WMA. To implement the recommended modifications for WMA, an appendix to the current dense-graded HMA mix design procedure, AASHTO R35, was developed. This appendix titled “Special Mixture Design Considerations and Methods for Warm Mix Asphalt (WMA)” (44; 45).

CHAPTER 3 RESEARCH APPROACH

This chapter describes the research approach to investigate the rutting and moisture sensitivity of warm mix asphalt with varying contents of recycled asphalt pavement.

In order to complete this research effort, laboratory and field evaluations were conducted. A research program consists of four tasks were designed to complete the evaluation studies. The discussion of the research program is presented in the following section.

3.1. Research Program

3.1.1. Task One: Primary Laboratory Evaluation

A laboratory evaluation was conducted to evaluate the rutting and moisture resistance of WMA mixtures with varying contents of RAP using local Iowa materials. Only HWT test was used to complete this task. A total of ten mixes were designed for this task; five WMA mixes prepared using LEADCAP 7-1 liquid additive and another five HMA mixes as control mixes for comparison. The laboratory mixes were designed for a traffic level of 0.3 million ESAL per Iowa DOT mix design requirements and NCHRP 691 report for WMA mix design. The mixtures used limestone virgin aggregate and combination of different percentages of fractionated RAP; 20, 30, 40, 50, and 75% by binder replacement.

3.1.2. Task Two: Secondary Laboratory Evaluation

After analyzing the results from the task one, one mix design was chosen and evaluated in the laboratory in terms of rutting and moisture susceptibilities to be used for field implementation. The field implementation included two test sections, one for WMA, and one for HMA. The test sections were constructed in Iowa City, Iowa in September 2013.

3.1.3. Task Three: Primary Field Evaluation

To assess the performance of WMA during construction and over time, three sets of test sections were evaluated. Each set includes one HMA test section and one WMA test section. All the test sections were constructed as surface layers. The first set was constructed in Champlin, Minnesota in July 2010. The second set was constructed in Lancaster, Ohio in September 2013. The third set was constructed in Iowa City, Iowa in September 2013. The three projects were constructed using LEADCAP additives as part of the field evaluation of WMA technology.

Throughout task three, the construction process of the test sections were monitored and the necessary field data were obtained. Several asphalt cores were extracted to evaluate the densities of the constructed mats. Loose mixtures were collected in order to run the necessary performance tests in the laboratory to evaluate the rutting and moisture resistance of the mixtures used for these test sections.

3.1.4. Task Four: Secondary Field Evaluation

This task includes the assessment of WMA mixture after construction and over time. Condition surveys of the constructed test sections were conducted after 6 months of since construction for Iowa and Ohio test sections and after 18 months since construction for Minnesota test sections in order to evaluate performance of the WMA mixtures.

3.2. Performance Tests

To investigate the impact of WMA technologies on the asphalt mixtures performance, laboratory testing was conducted to address the following questions:

- The impact of warm-mix additives on the resistance of the mixtures to rutting.
- The impact of warm-mix additives on the resistance of the mixtures to moisture damage,

The following performance tests were conducted in order to complete this research study:

- ❖ Performance tests on asphalt mixture
  - ➢ Hamburg Wheel Track test (AASHTO T324)
  - ➢ Modified Lottman test (AASHTO T283)
- ❖ Performance tests on asphalt binder
  - ➢ Determining the asphalt bond strength (ABS) using adhesion tester device (ASTM D4541).

3.2.1. Hamburg Wheel Track (HWT) test (AASHTO T324)

The HWT test procedure was explained earlier in section 2.3.5.4. As explained earlier, the HWT device applies a constant load of 685 N through a steel wheel. The tests are run in a water bath that is heated to 50°C after the test specimens are conditioned for 30 minutes. A test is completed when the wheel has passed over the specimens 20,000 times for 6.5 hours or when the rut depth exceeds 20 mm. Prior to the test, the HMA loose mix was short-term aged for 4 hours at 135°C (275°F) then followed by 2 hours at the compaction temperature and the WMA loose mix was short-term aged for two hours at the compaction temperature then aged for 16 hours at 60°C (140°F) as recommended by NCHRP Report 691 (45).

3.2.2. Modified Lottman Test (AASHTO T283)

The AASHTO T 283 test procedure was used to evaluate the resistance of the HMA and WMA mixtures to moisture damage. As explained earlier in section 2.3.5.5, the test was conducted on dry and wet conditioned specimens measuring 4 inches (100 mm) in diameter and 2.56 inches (65.0 mm) in height. The specimens were loaded until failure at a rate of 2 inches per minute (50.8 mm per minute). Two types of data were obtained from this test. The first was the indirect tensile strength (ITS) of the dry and wet conditioned specimens. The second was the tensile strength ratio (TSR), calculated by dividing the average indirect tensile strength (ITS) values of the wet conditioned specimens by the average indirect tensile strength (ITS) values of the dry conditioned specimens. The indirect tensile strength (ITS) is a measure of the strength and durability of the asphalt mixture, whereas the TSR ratio is a measure of its resistance to damage from freezing and thawing (14; 18; 24).

3.2.3. Determining the Asphalt Bond Strength (ABS) Using Adhesion Tester Device (AASHTO TP 91-11 & ASTM D4541)

This test method is used to measure the pull off force needed to break the bond strength between the asphalt binder and the aggregate surface. In order to prepare the test specimens, the asphalt binder is attached to the aggregate surface by means of adhesion at different controlled environmental (i.e., temperature and humidity) and moisture conditions. To measure the pull off force needed to detach the asphalt binder samples from the aggregate surface, a hydraulic pressure is applied to pullout a stub attached to the asphalt sample as shown in Figure 3.1. The test is conducted using PosiTest AT-A automatic adhesion tester device as shown in Figure 3.2.

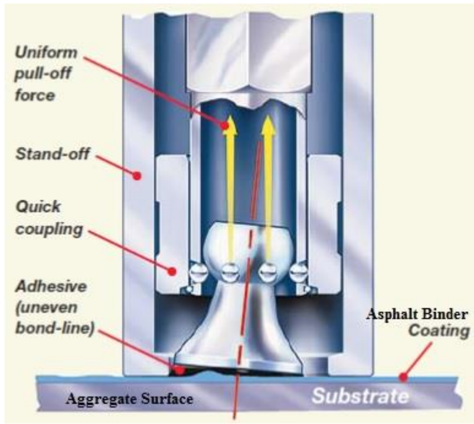

Figure 3.1 Schematic of Pull off Test Method Using a Self-Aligning Tester (46).

Figure 3.2 PosiTest® Pull-Off Adhesion Tester (46).

Measuring the bond strength between the asphalt binder and the aggregate surface can be used to understand the mode of failure during rutting or moisture damage of asphalt samples. Two modes of failures are considered in this test; 1) failure due to lake of enough adhesion between the asphalt binder and the aggregate surface, and 2) failure due to lake of enough cohesion within the asphalt binder or asphalt mastic. As mentioned before, different controlled environmental (i.e., temperature and humidity) and moisture conditions are designed in order to simulate the real conditions of asphalt mixtures during rutting and/or moisture damage process.

3.2.3.1. <u>Aggregate Sample Preparation for the ABS Test</u>

Aggregate samples should be cut in regular/geometric shape, plates for example, with fixed thickness throughout the sample. The top and bottom surfaces of the sample have to be parallel, clean, and smooth. In order to achieve clean and smooth surface, the aggregate samples/plates should be cleaned using a lapping wheel machine or any reasonable way, and then placed in an ultrasonic cleaner for 60 minutes at 60° C to remove any cutting residues. This process should eliminate roughness from the aggregate sample surfaces (47).

3.2.3.2. <u>Test Sample Preparation</u>

Before applying the hot asphalt binder to the aggregate surface, both the aggregate surface and the pull out stubs (dollies) should be degreased with acetone to remove any moisture and dust that can affect the adhesion of the asphalt sample to the aggregate or the pull out stub surfaces. In order to simulate the process of applying the asphalt to the aggregate during mixing, Moraes et al, 2009 (47) recommended the following procedure to prepare the test samples for the bitumen bond strength test:

1. Heat up pull-out stubs and the aggregate plates in an oven at 150° C for a minimum of 30 minutes to remove absorbed water on the aggregate surface and provide a better bond between the asphalt binder and the aggregate surface.

2. Remove the aggregate plates from the oven and place them in a second oven at 60° C for a minimum of 30 minutes to reach the application temperature.
3. Heat up the asphalt binders in the first oven at 150°C for sufficient time.
4. Carefully pour the hot asphalt in a 10.0 mm diameter DSR (Dynamic Shear Rheometer) silicon molds as shown in Figure 3.4. Cool the sample and the mold for 30 minutes at room temperature.
5. Remove the stubs from the first oven and immediately place the hot asphalt binder sample to the surfaces of the stubs for approximately 10 seconds.
6. Remove the aggregate plates from the second oven and then firmly press each stub with the asphalt binder sample into an aggregate plate surface until the stub reaches the surface and no excess of asphalt binder is observed to be flowing. Push down the stubs as straight as possible and avoid any twisting to reduce forming any trap air bubbles inside the samples.
7. Leave prepared dry samples for 24 hours at room temperature before testing.
8. For wet conditioning, samples are left for 1 hour at room temperature then, submerged into water tank at 40°C for the specified conditioning time. Leave the samples at room temperature for 1 hour after conditioning time is complete before testing.

Additionally, Moraes et al, 2009 (47) recommended new geometry and treatment to the stub surface used with PATTI Quantum Gold adhesion tester they used in their study in order to create a rough texture that provides a mechanical interlock and larger contact area between the asphalt binder and stub. This modifications are shown in Figure 3.3. This new modification could be easily applied to the pull-out stubs (dollies) used with the PosiTest® Pull-Off adhesion tester device.

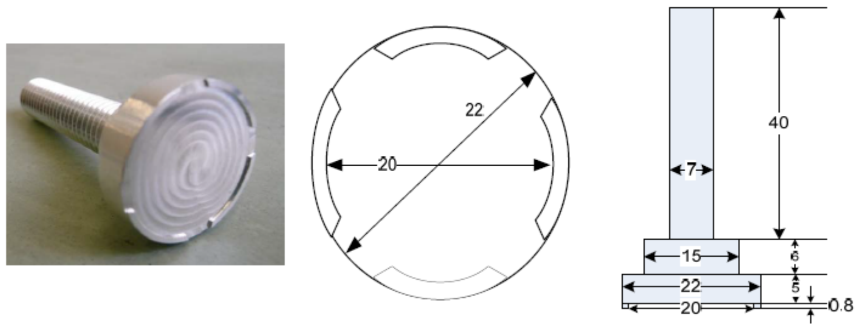

Figure 3.3 Pull-out stub for the Asphalt Bond Strength Test (ABS) (all measurements are in mm) (47).

Similar geometry and treatment to the stub surface to the one recommended in method one was done to the pullout stubs (dollies) used with the PosiTest® Pull-Off adhesion tester device. In order to evaluate the AASHTO TP 91-11 standard procedure to determine the ABS, two asphalt film thicknesses (FT) were studied in addition to the asphalt FT produced by the 0.8 mm pullout stub thickness used by AASHTO TP 91-11standard. The studied FTs were produced by pullout stub thickness values of 0.8 mm, 0.4 mm, and no stub or 0.0 mm.

3.2.3.3. <u>Summary of the Evaluated Materials in the ABS Test</u>

Two ABS studies were conducted to evaluate the bond strength between the asphalt binder and the aggregate surface. The two ABS studies are discussed below;

First Study: Evaluation of the ABS Test Procedure

In this study, un-conditioned samples only were evaluated in order to capture the effect of sample preparation method using different asphalt FT values produced by different pullout stub thicknesses and the asphalt binder grade on the bond strength between the asphalt binder and the aggregate surface. One aggregate source and four different binder grades were tested using three different pullout stub thicknesses (PST). Table 3.1 shows a summary of the evaluated materials.

Table 3.1 Summary of the ABS Test Method Evaluation Study

Asphalt Binder Type	No WMA Additive		
	Limestone Aggregate		
	0.8 mm PST	0.4 mm PST	0.0 mm PST
PG 58-28	X	X	X
PG 64-22	X	X	X
PG 64-28M (Polymer Modified)	X	X	X
PG 70-22M (Polymer Modified)	X	X	X

Based on the obtained results from the ABS test method evaluation, it was found that the no stub or 0.0 mm PST exhibited the most consistent and reasonable ABS values. All the test results obtained from this study are analyzed and discussed in chapter 4 “Laboratory Evaluation’s Test Results and Analysis".

Second Study: ABS Evaluation of Extracted Asphalt Binders

In this study, the asphalt binders of the collected mixtures from the test sections were extracted and then evaluated under both conditioning and un-conditioning situations. This ABS study was designed to evaluate the effects of moisture conditioning, aggregate type, WMA additives, and RAP content on the bond strength between the asphalt binder and the aggregate surface.

A total of three aggregate sources and three WMA additives were evaluated in this research study using the pullout stubs with 0.0 mm thickness. Table 3.2 shows a summary of the evaluated materials. All the test results obtained from this study are analyzed and discussed in chapter 5 “Field Evaluations’ Results and Analysis".

Table 3.2 Summary of the ABS Study of Extracted Asphalt Binder

Project	RAP Content, TWM	Aggregate Type	Mix Type
IA-Surface Layer	38.00%	Limestone-IA	RAPCAP
	38.00%	Limestone-IA	HMA
MN-Surface Layer	25.00%	Granite-MN	LEADCAP 6-8
	25.00%	Granite-MN	HMA
OH-Surface Layer	20.00%	Limestone-OH	LEADCAP 7-1
	20.00%	Limestone-OH	HMA

3.3. Statistical Analysis Techniques

The data obtained from the laboratory tests conducted in the different evaluation studies were statistically analyzed using means of Analysis of Variance (ANOVA). The statistical analysis were done utilizing the Statistical Analysis Software (SAS) to examine the significance of each of the tests' parameters as well as their interactions on the evaluated mechanical properties of the asphalt mixtures.

The most frequently used design with multiple factors is a factorial design. Factorial treatment structure exists when the treatments are the combinations of the levels of two or more factors. These combination treatments are called factor-level combinations or factorial combinations. The main elements included in factorial design are the structure of the statistical model, estimation of the parameters, analysis of variance (ANOVA), and t-procedure (14).

3.4. Pavement Condition Survey Techniques

In order to evaluate the pavement conditions of the test sections, visual inspections of these pavements were conducted after certain time periods. The pavement conditions were determined following the federal guidelines specified in the ASTM D6433-09 "*Standard Practice for Roads and Parking Lots Pavement Condition Index Surveys*". The existed pavement distresses were measured and recorded in a flexible pavement condition survey data sheet as shown in Figure 3.4. Based on the ASTM D6433, the pavement condition indices (PCIs) were calculated for the pavement condition of each test section. The PCI ranges from 0 for a failed pavement to 100 for a pavement in excellent condition (48). Table 3.3 shows an example of PCI rating scale and corresponding general pavement condition description (49). Figure 3.5 shows the pavement condition index (PCI), rating scale and suggested colors according to ASTM D6433-09.

ASPHALT SURFACED ROADS AND PARKING LOTS
CONDITION SURVEY DATA SHEET
FOR SAMPLE UNIT

SKETCH:

BRANCH ____________ SECTION _________ SAMPLE UNIT _________
SURVEYED BY ___________ DATE _________ SAMPLE AREA _________

1. Alligator Cracking
2. Bleeding
3. Block Cracking
4. Bumps and Sags
5. Corrugation
6. Depression
7. Edge Cracking
8. Jt. Reflection Cracking
9. Lane/Shoulder Drop Off
10. Long & Trans Cracking
11. Patching & Util Cut Patching
12. Polished Aggregate
13. Potholes
14. Railroad Crossing
15. Rutting
16. Shoving
17. Slippage Cracking
18. Swell
19. Weathering/Raveling

DISTRESS SEVERITY	QUANTITY									TOTAL	DENSITY %	DEDUCT VALUE

Figure 3.4 Flexible Pavement Condition Survey Data Sheet (Source: ASTM D6433-09).

Table 3.3 Pavement Condition Index (PCI) Rating Scale

Condition	PCI	Description
Excellent	86-100	No significant distress.
Very Good	71-85	Little distress, with the exception of utility patches in good condition, or slight hairline cracks; may be slightly weathered.
Good	56-70	Slight to moderately weathered, slight distress, possibly patching.
Fair	41-55	Severely weathered or slight to moderate levels of distress generally limited to patches and non-load-related cracking.
Poor	26-40	Moderate to severe distresses including load-related type, such as alligator cracking.
Very Poor	11-25	Severely distresses or large quantities of distortion or alligator cracking.
Failed	0-10	Failure of the pavement, distress has surpassed tolerable rehabilitation limits.

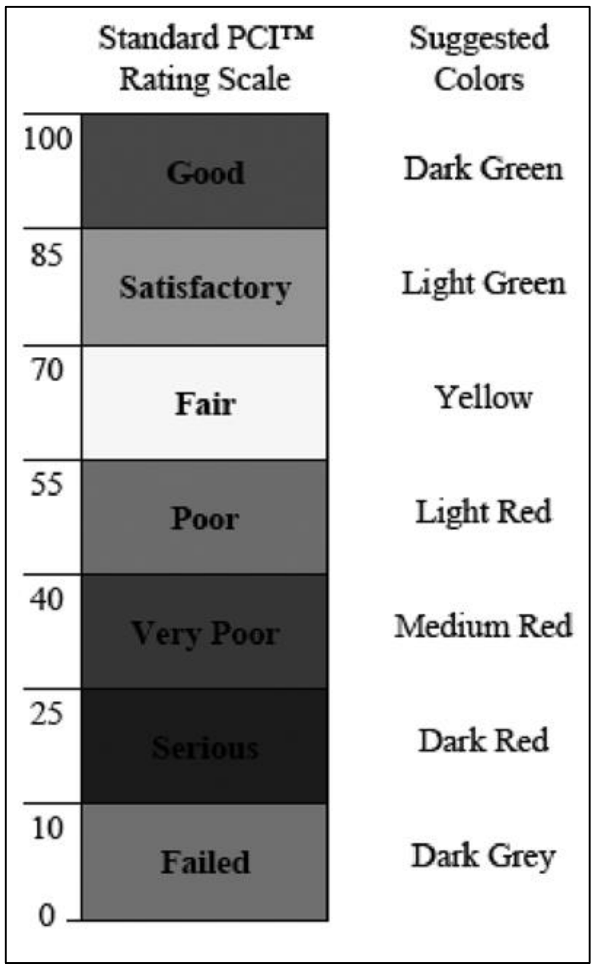

Figure 3.5 Pavement Condition Index (PCI), Rating Scale and Suggested Colors According to ASTM D6433-09

CHAPTER 4 LABORATORY EVALUATIONS' RESULTS AND ANALYSIS

This chapter describes the test results obtained for the primary and secondary laboratory evaluations explained in task one and two and the test results obtained for the asphalt bond strength (ABS) test method evaluation described in section 3.3.3.3. In the primary laboratory evaluation, a total of ten mixtures were designed: five WMA mixtures using LEADCAP 7-1 liquid additive and five HMA mixtures as control mixtures with varying amounts of fractionated RAP materials of 20, 30, 40, 50 and 75% by a binder replacement. The ten laboratory mixtures were designed for a traffic level of 0.3 million ESALs per Iowa DOT mix design requirements and NCHRP 691 Report for WMA mix design. Based on the test results from the primary laboratory evaluation and the recommendations from Iowa DOT, a mix design including 30% fractionated RAP by a binder replacement was chosen for further evaluation in the secondary laboratory study. The new mix was designed for a traffic level of 3 to 10 million EASLs. The RAPCAP (liquid) additive was used for the WMA mixture. Both LEADCAP and RAPCAP additives were added to the mix at a rate of 1.5% by the optimum asphalt content of the mix.

4.1. Primary Laboratory Evaluation

This section describes the mix designs, and performance tests results of the WMA mixtures with fractionated RAP materials described in task one of the research program.

4.1.1. Virgin Aggregate & RAP Material Properties

Limestone virgin aggregates of four different stockpiles collected from River Products quarry located in Coralville, IA along with varying amounts of fractionated RAP from I-80 rehabilitation project were used for designing the mix with ½ inch nominal maximum size. RAP materials passing the No 16 sieve were removed from the RAP stockpile. The Virgin aggregate properties, RAP material properties and the combined gradations for each of five different amounts of RAP materials by a binder replacement of 20, 30, 40, 50 and 75% are summarized in Tables 4.1 through 4.5, respectively. The combined gradations are plotted on the 0.45 power gradation chart in Figure 4.1.

As can be seen from these tables, due to the less amount of binder available from the RAP materials than the optimum binder content, the percentage of RAP materials by weight were significantly higher than the percentage by binder replacement. For example, as shown in Table 4.5, the 75% replacement by binder content was equivalent to 90% replacement by weight. As shown in Figures 4.1 all combined gradations met the Superpave mix design requirements specified by Iowa DOT.

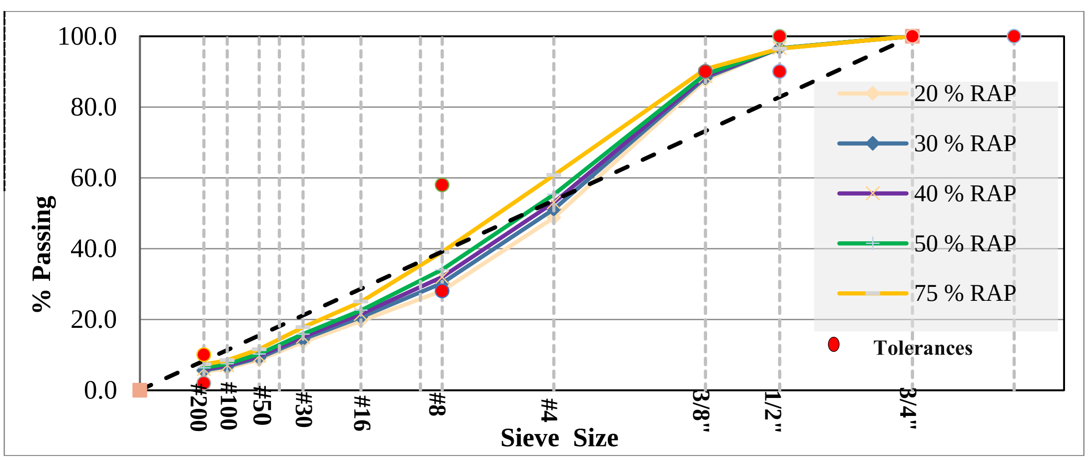

Figure 4.1 Aggregate Gradation Charts for HMA and WMA Mixtures with LEADCAP

Table 4.1 Combined Aggregate Gradation and Mixture Properties with 20% RAP Content.

Mix Design Info.		*Virgin Aggregate Properties*							*RAP Properties*				
Project Name	WMA-RAP	**Virgin Agg. Batch Mix**		**G_{sb}**	**ABS, %**	**G_{sa}**	**RAP (% Binder Replaced)**	**RAP ID**	**% of RAP**	**G_{sb}**	**ABS, %**	**G_{sa}**	**% AC**
Traffic ESAL	300,000	**Sand**	**7.0%**	2.634	0.47	2.667	**20.0%**	**Frac. ≥ #16**	**100.0%**	2.650	1.190	2.736	4.33
Mix Size	1/2"	**Man. Sand**	**13.0%**	2.649	0.84	2.709	**RAP (% Total Mix Weight)**		**0.0%**	0.000	0.000	0.000	0.00
Layer	Surface	**1/2" to Dust**	**65.0%**	2.638	0.98	2.708	**24.3%**	**Combined RAP Mix**		**2.650**	**1.190**	**2.736**	**4.33**
Virgin Binder	PG 64-22	**3/8" Chips**	**15.0%**	2.642	1.04	2.717	**RAP (% Dry Aggregate Weight)**	**Combined Mixture Properties**		**G_{sb}**	**ABS, %**	**G_{sa}**	**% AC**
WMA Additive	LEADCAP 7-1	**Combined Virgin Mix**		**2.640**	**0.900**	**2.707**	**23.5%**			**2.642**	**0.955**	**2.713**	**5.25**

Virgin Aggregate Batch Mix and RAP Material Combined Gradation

Sieve Size		Stockpile and percentage passing				Virgin Agg.	RAP Aggregate Gradation		Recovered	**Comb. Grad.**	Design Spec, 12.5 mm	
ID	mm	**Sand**	**Man. Sand**	**1/2" to Dust**	**3/8" Chips**	Batch Mix	**Frac. ≥ #16**	**0**	Agg. Blend	**w/RAP @ P_{bi}**	Min.	Max.
3/4 in	19.0	100.0	100.0	100.0	100.0	100.0	100	0	100.0	**100.0**	**100.0**	**100.0**
1/2 in	12.5	100.0	100.0	95.0	100.0	96.8	96.5	0.0	96.5	**96.7**	**90.0**	**100.0**
3/8 in	9.5	100.0	100.0	80.0	97.0	86.6	91.1	0.0	91.1	**87.6**	-	**90.0**
#4	4.75	95.0	98.0	29.0	42.0	44.5	62.6	0.0	62.6	**48.8**	-	-
#8	2.36	90.0	76.0	10.0	10.0	24.2	40.7	0.0	40.7	**28.1**	**28.0**	**58.0**
#16	1.18	79.0	43.0	8.0	9.0	17.7	25.9	0.0	25.9	**19.6**	-	-
#30	0.6	53.0	20.0	7.0	8.0	12.1	18.4	0.0	18.4	**13.6**	-	-
#50	0.3	16.0	8.3	6.5	7.5	7.5	12.0	0.0	12.0	**8.6**	-	-
#100	0.15	2.0	2.8	6.0	7.0	5.5	8.8	0.0	8.8	**6.2**	-	-
#200	0.075	1.0	2.5	5.0	5.5	4.5	7.6	0.0	7.6	**5.2**	**2.0**	**10.0**
% Total weight of mixture		**5.3%**	**9.8%**	**49.2%**	**11.4%**	**Surf. Area**	**24.3%**	**0.0%**	**Surf. Area**	**Surf. Area**	Total	
% dry aggregate weight		**5.4%**	**9.9%**	**49.7%**	**11.5%**	**4.00**	**23.5%**	**0.0%**	**6.22**	**4.52**	**OK**	

Table 4.2 Combined Aggregate Gradation and Mixture Properties with 30% RAP Content.

Mix Design Info.		***Virgin Aggregate Properties***						***RAP Properties***					
Project Name	WMA-RAP	**Virgin Agg. Batch Mix**		**G_{sb}**	**ABS, %**	**G_{sa}**	**RAP (% Binder Replaced)**	**RAP ID**	**% of RAP**	**G_{sb}**	**ABS, %**	**G_{sa}**	**% AC**
Traffic ESAL	300,000	**Sand**	**7.0%**	2.634	0.47	2.667	**30.0%**	**Frac. ≥ #16**	**100.0%**	2.650	1.190	2.736	4.33
Mix Size	1/2"	**Man. Sand**	**13.0%**	2.649	0.84	2.709	**RAP (% Dry Mix Weight)**		**0.0%**	0.000	0.000	0.000	0.00
Layer	Surface	**1/2" to Dust**	**65.0%**	2.638	0.98	2.708	**37.7%**	**Combined RAP Mix**		**2.650**	**1.190**	**2.736**	**4.33**
Virgin Binder	PG 58-28	**3/8" Chips**	**15.0%**	2.642	1.04	2.717	**RAP (% Total Aggregate)**	**Combined Mixture Properties**		**G_{sb}**	**ABS, %**	**G_{sa}**	**% AC**
WMA Additive	LEADCAP 7-1	**Combined Virgin Mix**		**2.640**	**0.900**	**2.707**	**36.6%**			**2.643**	**0.988**	**2.717**	**5.42**

Virgin Aggregate Batch Mix and RAP Material Combined Gradation

Sieve Size		Stockpile and percentage passing				Virgin Agg.	RAP Aggregate Gradation		Recovered	**Comb. Grad.**	Design Spec, 12.5 mm	
ID	mm	**Sand**	**Man. Sand**	**1/2" to Dust**	**3/8" Chips**	Batch Mix	**Frac. ≥ #16**	**0**	Agg. Blend	**w/RAP @ P_{bi}**	Min.	Max.
3/4 in	19.0	100.0	100.0	100.0	100.0	100.0	100	0	100.0	**100.0**	**100.0**	**100.0**
1/2 in	12.5	100.0	100.0	95.0	100.0	96.8	96.5	0.0	96.5	**96.6**	**90.0**	**100.0**
3/8 in	9.5	100.0	100.0	80.0	97.0	86.6	91.1	0.0	91.1	**88.2**	-	**90.0**
#4	4.75	95.0	98.0	29.0	42.0	44.5	62.6	0.0	62.6	**51.2**	-	-
#8	2.36	90.0	76.0	10.0	10.0	24.2	40.7	0.0	40.7	**30.2**	**28.0**	**58.0**
#16	1.18	79.0	43.0	8.0	9.0	17.7	25.9	0.0	25.9	**20.7**	-	-
#30	0.6	53.0	20.0	7.0	8.0	12.1	18.4	0.0	18.4	**14.4**	-	-
#50	0.3	16.0	8.3	6.5	7.5	7.5	12.0	0.0	12.0	**9.2**	-	-
#100	0.15	2.0	2.8	6.0	7.0	5.5	8.8	0.0	8.8	**6.7**	-	-
#200	0.075	1.0	2.5	5.0	5.5	4.5	7.6	0.0	7.6	**5.6**	**2.0**	**10.0**
% dry weight of mixture		**4.4%**	**8.1%**	**40.5%**	**9.4%**	**Surf. Area**	**37.7%**	**0.0%**	**Surf. Area**	**Surf. Area**	Total	
% of total aggregate		**4.4%**	**8.2%**	**41.2%**	**9.5%**	**4.00**	**36.6%**	**0.0%**	**6.22**	**4.82**	**OK**	

Table 4.3 Combined Aggregate Gradation and Mixture Properties with 40% RAP Content.

Mix Design Info. ***Virgin Aggregate Properties*** ***RAP Properties***

Project Name	WMA-RAP	**Virgin Agg. Batch Mix**		**G_{sb}**	**ABS, %**	**G_{sa}**	**RAP (% Binder Replaced)**	**RAP ID**	**% of RAP**	**G_{sb}**	**ABS, %**	**G_{sa}**	**% AC**
Traffic ESAL	300,000	**Sand**	**7.0%**	2.634	0.47	2.667	**40.0%**	**Frac. ≥ #16**	**100.0%**	2.650	1.190	2.736	4.33
Mix Size	1/2"	**Man. Sand**	**13.0%**	2.649	0.84	2.709	**RAP (% Dry Mix Weight)**		**0.0%**	0.000	0.000	0.000	0.00
Layer	Surface	**1/2" to Dust**	**65.0%**	2.638	0.98	2.708	**48.2%**	**Combined RAP Mix**		**2.650**	**1.190**	**2.736**	**4.33**
Virgin Binder	PG 58-28	**3/8" Chips**	**15.0%**	2.642	1.04	2.717	**RAP (% Total Aggregate)**	**Combined Mixture Properties**		**G_{sb}**	**ABS, %**	**G_{sa}**	**% AC**
WMA Additive	LEADCAP 7-1	**Combined Virgin Mix**		**2.640**	**0.900**	**2.707**	**47.1%**			**2.645**	**1.017**	**2.720**	**5.20**

Virgin Aggregate Batch Mix and RAP Material Combined Gradation

Sieve Size		Stockpile and percentage passing				Virgin Agg.	RAP Aggregate Gradation		Recovered	**Comb. Grad.**	Design Spec, 12.5 mm	
ID	mm	**Sand**	**Man. Sand**	**1/2" to Dust**	**3/8" Chips**	Batch Mix	**Frac. ≥ #16**	**0**	Agg. Blend	**w/RAP @ P_{bi}**	Min.	Max.
3/4 in	19.0	100.0	100.0	100.0	100.0	100.0	100	0	100.0	**100.0**	**100.0**	**100.0**
1/2 in	12.5	100.0	100.0	95.0	100.0	96.8	96.5	0.0	96.5	**96.6**	**90.0**	**100.0**
3/8 in	9.5	100.0	100.0	80.0	97.0	86.6	91.1	0.0	91.1	**88.7**	**-**	**90.0**
#4	4.75	95.0	98.0	29.0	42.0	44.5	62.6	0.0	62.6	**53.0**	**-**	**-**
#8	2.36	90.0	76.0	10.0	10.0	24.2	40.7	0.0	40.7	**31.9**	**28.0**	**58.0**
#16	1.18	79.0	43.0	8.0	9.0	17.7	25.9	0.0	25.9	**21.5**	**-**	**-**
#30	0.6	53.0	20.0	7.0	8.0	12.1	18.4	0.0	18.4	**15.0**	**-**	**-**
#50	0.3	16.0	8.3	6.5	7.5	7.5	12.0	0.0	12.0	**9.7**	**-**	**-**
#100	0.15	2.0	2.8	6.0	7.0	5.5	8.8	0.0	8.8	**7.0**	**-**	**-**
#200	0.075	1.0	2.5	5.0	5.5	4.5	7.6	0.0	7.6	**5.9**	**2.0**	**10.0**
% dry weight of mixture		**3.6%**	**6.7%**	**33.7%**	**7.8%**	**Surf. Area**	**48.2%**	**0.0%**	**Surf. Area**	**Surf. Area**	Total	
% of total aggregate		**3.7%**	**6.9%**	**34.4%**	**7.9%**	**4.00**	**47.1%**	**0.0%**	**6.22**	**5.05**	**OK**	

Table 4.4 Combined Aggregate Gradation and Mixture Properties with 50% RAP Content.

Mix Design Info.		***Virgin Aggregate Properties***								***RAP Properties***			
Project Name	WMA-RAP	**Virgin Agg. Batch Mix**		**G_{sb}**	**ABS, %**	**G_{sa}**	**RAP (% Binder Replaced)**	**RAP ID**	**% of RAP**	**G_{sb}**	**ABS, %**	**G_{sa}**	**% AC**
Traffic ESAL	300,000	**Sand**	**7.0%**	2.634	0.47	2.667	**50.0%**	**Frac. ≥ #16**	**100.0%**	2.650	1.190	2.736	4.33
Mix Size	1/2"	**Man. Sand**	**13.0%**	2.649	0.84	2.709	**RAP (% Dry Mix Weight)**		**0.0%**	0.000	0.000	0.000	0.00
Layer	Surface	**1/2" to Dust**	**65.0%**	2.638	0.98	2.708	**60.8%**	**Combined RAP Mix**		**2.650**	**1.190**	**2.736**	**4.33**
Virgin Binder	PG 58-28	**3/8" Chips**	**15.0%**	2.642	1.04	2.717	**RAP (% Total Aggregate)**	**Combined Mixture Properties**		**G_{sb}**	**ABS, %**	**G_{sa}**	**% AC**
WMA Additive	LEADCAP 7-1	**Combined Virgin Mix**		**2.640**	**0.900**	**2.707**	**59.7%**			**2.646**	**1.053**	**2.724**	**5.25**

Virgin Aggregate Batch Mix and RAP Material Combined Gradation

Sieve Size		Stockpile and percentage passing				Virgin Agg.	RAP Aggregate Gradation		Recovered	**Comb. Grad.**	Design Spec, 12.5 mm	
ID	mm	**Sand**	**Man. Sand**	**1/2" to Dust**	**3/8" Chips**	Batch Mix	**Frac. ≥ #16**	**0**	Agg. Blend	**w/RAP @ P_{bi}**	Min.	Max.
3/4 in	19.0	100.0	100.0	100.0	100.0	100.0	100	0	100.0	**100.0**	**100.0**	**100.0**
1/2 in	12.5	100.0	100.0	95.0	100.0	96.8	96.5	0.0	96.5	**96.6**	**90.0**	**100.0**
3/8 in	9.5	100.0	100.0	80.0	97.0	86.6	91.1	0.0	91.1	**89.3**	**-**	**90.0**
#4	4.75	95.0	98.0	29.0	42.0	44.5	62.6	0.0	62.6	**55.3**	**-**	**-**
#8	2.36	90.0	76.0	10.0	10.0	24.2	40.7	0.0	40.7	**34.0**	**28.0**	**58.0**
#16	1.18	79.0	43.0	8.0	9.0	17.7	25.9	0.0	25.9	**22.6**	**-**	**-**
#30	0.6	53.0	20.0	7.0	8.0	12.1	18.4	0.0	18.4	**15.9**	**-**	**-**
#50	0.3	16.0	8.3	6.5	7.5	7.5	12.0	0.0	12.0	**10.2**	**-**	**-**
#100	0.15	2.0	2.8	6.0	7.0	5.5	8.8	0.0	8.8	**7.4**	**-**	**-**
#200	0.075	1.0	2.5	5.0	5.5	4.5	7.6	0.0	7.6	**6.3**	**2.0**	**10.0**
% dry weight of mixture		**2.7%**	**5.1%**	**25.5%**	**5.9%**	**Surf. Area**	**60.8%**	**0.0%**	**Surf. Area**	**Surf. Area**	Total	
% of total aggregate		**2.8%**	**5.2%**	**26.2%**	**6.0%**	**4.00**	**59.7%**	**0.0%**	**6.22**	**5.33**	**OK**	

Table 4.5 Combined Aggregate Gradation and Mixture Properties with 75% RAP Content.

Mix Design Info.		*Virgin Aggregate Properties*								*RAP Properties*			
Project Name	WMA-RAP	**Virgin Agg. Batch Mix**		G_{sb}	**ABS, %**	G_{sa}	**RAP (% Binder Replaced)**	**RAP ID**	**% of RAP**	G_{sb}	**ABS, %**	G_{sa}	**% AC**
Traffic ESAL	300,000	**Sand**	**7.0%**	2.634	0.47	2.667	**75.0%**	**Frac. ≥ #16**	**100.0%**	2.650	1.190	2.736	4.33
Mix Size	1/2"	**Man. Sand**	**13.0%**	2.649	0.84	2.709	**RAP (% Total Mix Weight)**		**0.0%**	0.000	0.000	0.000	0.00
Layer	Surface	**1/2" to Dust**	**65.0%**	2.638	0.98	2.708	**90.3%**	**Combined RAP Mix**		**2.650**	**1.190**	**2.736**	**4.33**
Virgin Binder	PG 58-28	**3/8" Chips**	**15.0%**	2.642	1.04	2.717	**RAP (% Dry Aggregate Weight)**	**Combined Mixture Properties**		G_{sb}	**ABS, %**	G_{sa}	**% AC**
WMA Additive	LEADCAP 7-1	**Combined Virgin Mix**		**2.640**	**0.900**	**2.707**	**89.9%**			**2.649**	**1.152**	**2.733**	**5.20**

Virgin Aggregate Batch Mix and RAP Material Combined Gradation

Sieve Size		Stockpile and percentage passing				Virgin Agg.	RAP Aggregate Gradation		Recovered	**Comb. Grad.**	Design Spec, 12.5 mm	
ID	mm	**Sand**	**Man. Sand**	**1/2" to Dust**	**3/8" Chips**	Batch Mix	**Frac. ≥ #16**	**0**	Agg. Blend	**w/RAP @ P_{bi}**	min	max
3/4 in	19.0	100.0	100.0	100.0	100.0	100.0	100	0	100.0	**100.0**	**100.0**	**100.0**
1/2 in	12.5	100.0	100.0	95.0	100.0	96.8	96.5	0.0	96.5	**96.5**	**90.0**	**100.0**
3/8 in	9.5	100.0	100.0	80.0	97.0	86.6	91.1	0.0	91.1	**90.7**	**-**	**90.0**
#4	4.75	95.0	98.0	29.0	42.0	44.5	62.6	0.0	62.6	**60.8**	**-**	**-**
#8	2.36	90.0	76.0	10.0	10.0	24.2	40.7	0.0	40.7	**39.0**	**28.0**	**58.0**
#16	1.18	79.0	43.0	8.0	9.0	17.7	25.9	0.0	25.9	**25.0**	**-**	**-**
#30	0.6	53.0	20.0	7.0	8.0	12.1	18.4	0.0	18.4	**17.8**	**-**	**-**
#50	0.3	16.0	8.3	6.5	7.5	7.5	12.0	0.0	12.0	**11.6**	**-**	**-**
#100	0.15	2.0	2.8	6.0	7.0	5.5	8.8	0.0	8.8	**8.5**	**-**	**-**
#200	0.075	1.0	2.5	5.0	5.5	4.5	7.6	0.0	7.6	**7.3**	**2.0**	**10.0**
% Total weight of mixture		**0.7%**	**1.3%**	**6.3%**	**1.5%**	**Surf. Area**	**90.3%**	**0.0%**	**Surf. Area**	**Surf. Area**	Total	
% dry aggregate weight		**0.7%**	**1.3%**	**6.6%**	**1.5%**	**4.00**	**89.9%**	**0.0%**	**6.22**	**6.00**	**OK**	

4.1.2. Asphalt Binder

The target performance grade (PG) for the binder used for all mixtures is PG 64-22 and Iowa DOT requires lowering the high temperature grade by one level for the HMA mixture that includes more than 20% RAP materials. Therefore, two asphalt binder types were evaluated in this study: PG 64-22 for mixtures with RAP content up to 20% by binder replacement and PG 58-28 for mixtures with RAP content more than 20% by binder replacement. As recommended by the Flint Hills Resources, HMA mixing and compaction temperatures for the PG 64-22 binder were determined as 311° F (155°C) and 293° F (145°C), respectively, and those for the PG58-28 binder were determined as 300° F (150°C), and 275° F (135°C), respectively.

4.1.3. Mix Design

Mixing and compaction temperatures were established based on the current Iowa DOT mix design requirements for HMA mixtures including RAP materials and NCHRP 691 Report for WMA mix design. According to Bonaquist R. (43), WMA minimum mixing and compaction temperature can be indicated using the aging index (ratio of $G^*/\sin\delta$ of the RTFO-aged binder to that of the original binder) of the used asphalt binder without applying any increase to its performance grade. The mix coatability, workability and compactability are then used to finalize the mixing and compaction temperature determination process.

Mixing and compaction temperatures are summarized in Table 4.6 and mix design properties for the ten mixtures are summarized in Tables 4.7. All mixtures met the mix design requirements specified by Iowa DOT. The only exceptions from that were the VMA, dust to binder ration, and the film thickness for the mixtures prepared using 75% RAP.

Table 4.6 Mixing and Compaction Temperatures for the Laboratory Evaluated Mixtures.

Binder Type	Mixture	RAP Heat-Up, Max. 2 hours	Binder Temp.	Mixing Temp.	Comp. Temp.
PG 58-28	HMA	302° F (150° C)	302° F (150° C)	302° F (150° C)	285° F (140° C)
PG 58-28	WMA-LEADCAP	275° F (135° C)	300° F (150° C)	275° F (135° C)	250° F (125° C)
PG 64-22/28	HMA	311° F (155° C)	311° F (155° C)	311° F (155° C)	293° F (145° C)
PG 64-22/28	WMA-LEDACAP/RAPCAP	275° F (135° C)	311° F (155° C)	285° F (140° C)	266° F (130° C)

Table 4.7 Mix Design Summaries

Mix Design Properties	20 % RAP		30 % RAP		40 % RAP		50 % RAP		75 % RAP		Mix Design Criteria
	HMA	WMA	HMA	WMA	HMA	WMA	HMA	WMA	HMA	WMA	
Target Air Voids, %	3.67	3.10	3.99	3.81	3.45	3.50	3.36	3.03	3.11	3.33	3.5 ± 0.5
Optimum Binder Content (%)	5.25	5.25	5.2	5.2	5.3	5.1	5.2	5.2	5.2	5.1	----
Virgin Asphalt Binder Content (ADD AC %)	4.2	4.2	3.64	3.64	3.18	3.06	2.6	2.6	1.3	1.28	----
Virgin Aggregate Content (% Mix Weight)	76	76	64	64	51.7	53.2	40	40	10	11	----
RAP Content (% Mix Weight)	24	24	36	36	49.3	46.8	60	60	90	89	----
RAP Content (% Total Aggregate)	23	23	35	35	48.2	45.7	59	59	89	88	----
Aggregate Bulk Specific Gravity (G_{sb})	2.642	2.642	2.64	2.643	2.65	2.645	2.65	2.646	2.65	2.649	----
Aggregate Effective Specific Gravity (G_{se})	2.669	2.664	2.68	2.666	2.69	2.682	2.71	2.681	2.73	2.724	----
Aggregate Apparent Specific Gravity (G_{sa})	2.713	2.713	2.72	2.717	2.72	2.72	2.72	2.724	2.73	2.732	----
Aggregate Water Absorption (%)	0.955	0.955	0.98	0.984	1.02	1.017	1.05	1.052	1.15	1.146	----
Asphalt Binder Bulk Specific Gravity (G_b)	1.043	1.043	1.04	1.036	1.04	1.036	1.04	1.036	1.04	1.036	----
Bulk Specific Gravity at Optimum AC (G_{mb})	2.376	2.386	2.38	2.37	2.39	2.395	2.41	2.401	2.43	2.431	----
Maximum Theoretical Specific Gravity (G_{mm})	2.467	2.463	2.47	2.464	2.48	2.481	2.5	2.476	2.51	2.515	----
Percent Binder Absorption (P_{ba} %)	0.4	0.32	0.5	0.33	0.62	0.54	0.87	0.51	1.09	1.08	----
Percent Effective Binder (P_{be} %)	4.87	4.95	4.73	4.89	4.75	4.55	4.38	4.72	4.16	4.08	----
Comb. Agg. Surface Area (m^2/ Kg.)	4.52	4.52	4.78	4.78	5.07	5.03	5.32	5.32	6	5.96	----
Comb. Agg. Dust Content ($P_{0.075}$ %)	5.2	5.2	5.57	5.57	5.97	5.91	6.32	6.32	7.28	7.22	Max. 10%
%G_{mm} at N_{ini} (7 gyrations)	84.54	84.27	84.6	84.51	86.8	82.12	86.1	83.41	84.7	82.48	≤ 92.0%
%G_{mm} at N_{des} (68 gyrations)	96.33	96.9	96	96.19	96.6	96.5	96.6	96.97	96.9	96.67	96.5% ± 0.5
%G_{mm} at N_{max} (104 gyrations)	97.8	97.98	97.5	97.77	98	97.67	97.7	97.96	98	97.49	≤ 98.0%
Voids in Mineral Aggregate (VMA, %)	14.78	14.42	14.8	15	14.5	14	13.6	13.96	12.9	12.9	Min.14%
Voids Filled with Asphalt (VFA, %)	75.15	78.52	73.1	74.58	75.8	75	75.2	78.31	75.9	74.19	70% - 80%
Dust to Binder Ratio ($P_{0.075}/P_{be}$)	1.07	1.05	1.18	1.14	1.26	1.3	1.44	1.34	1.75	1.77	0.6 - 1.4
Film Thickness (µm)	10.77	10.94	9.89	10.22	9.36	9.07	8.24	8.87	6.94	6.84	8.0 - 13 µm

4.1.4. Performance Evaluation

4.1.4.1. Hamburg Wheel Track (HWT) Test Results

The HWT Test results for HMA and WMA mixtures are summarized in Tables 4.8 through 4.12 and plotted in Figures 4.2 through 4.6

Mixtures with 20% RAP by Binder Replacement

All HMA specimens successfully passed the test with the average maximum rut depth of 14.33 mm. WMA mixtures didn't pass the test and the average number of passes to failure (20 mm rut depth) was 8,675 passes. The average SIP values were 10,250 passes for HMA specimens and 4,375 for WMA specimens.

Mixtures with 30% RAP by Binder Replacement

All HMA specimens successfully passed the test with the average maximum rut depth of 16.2 mm. WMA mixtures didn't pass the test and the average number of passes to failure (20 mm rut depth) was 11,450 passes. The average SIP values were 8,000 passes for HMA specimens and 6,250 for WMA specimens. It is interesting to note that the SIP value for HMA was lower but the SIP value for WMA was higher when the RAP amount was increased from 20% to 30% by binder replacement.

Mixtures with 40% RAP by Binder Replacement

All HMA specimens successfully passed the test with the average maximum rut depth of 12.0 mm. WMA mixtures didn't pass the test, and the average number of passes to failure (20 mm rut depth) was 13,075 passes. The average SIP were 10,500 passes for HMA specimens, and 6,875 for WMA specimens. It should be noted that the SIP value for HMA was higher but the SIP value for WMA was similar when the RAP amount was increased from 30% to 40% by binder replacement.

Mixtures with 50% RAP by Binder Replacement

Both HMA and WMA specimens successfully passed the test with the average maximum rut depths of 4.2 mm and 19.0 mm, respectively. The average SIP were 15,000 passes for HMA specimens, and 10,750 for WMA specimens. It should be noted that the SIP values for both HMA and WMA were significantly higher when the RAP amount was increased from 40% to 50% by binder replacement.

Mixtures with 75% RAP by Binder Replacement

Both HMA and WMA specimens successfully passed the test with the average maximum rut depths of 2.5 mm for HMA specimens and 3.8 mm for WMA specimens. The average SIP values of both HMA and WMA specimens were greater than 20,000 passes when 75% RAP materials by binder replacement (90% RAP materials by weight) were used. The two way-factor ANOVA analysis, as shown Table 4.13, conducted to evaluate the effects of the mix type, and RAP percentage along with their interaction on the SIP values obtained from the HWT. As seen from Table 4.13, the RAP percentage had a significant effect on the SIP values. The mix type had a very low effect on the SIP values, however the interaction between the RAP percentage and the mix type had no effect on the SIP value, as shown in the interaction plot in Figure 4.7.

Table 4.8 Hamburg Wheel Test Results for Mixtures Including 20% RAP.

Mix Type	Test ID	Air Voids, %	Total Number of Passes	Inverse Creep Slope (Pass/mm)	Inverse Stripping Slope (Pass/mm)	SIP	Max. Rut Depth, mm
HMA	HMA 1	6.0	20000	2200	1837	11000	10.9
	HMA 2	6.1	20000	4211	856	9500	17.8
	Average	**6.0**	**20000**	**3205**	**1347**	**10250**	**14.3**
WMA-LEADCAP7-1	L 1	6.1	8400	938	321	3750	20.0
	L 2	6.1	8950	1250	272	5000	20.0
	Average	**6.1**	**8675**	**1094**	**297**	**4375**	**20.0**

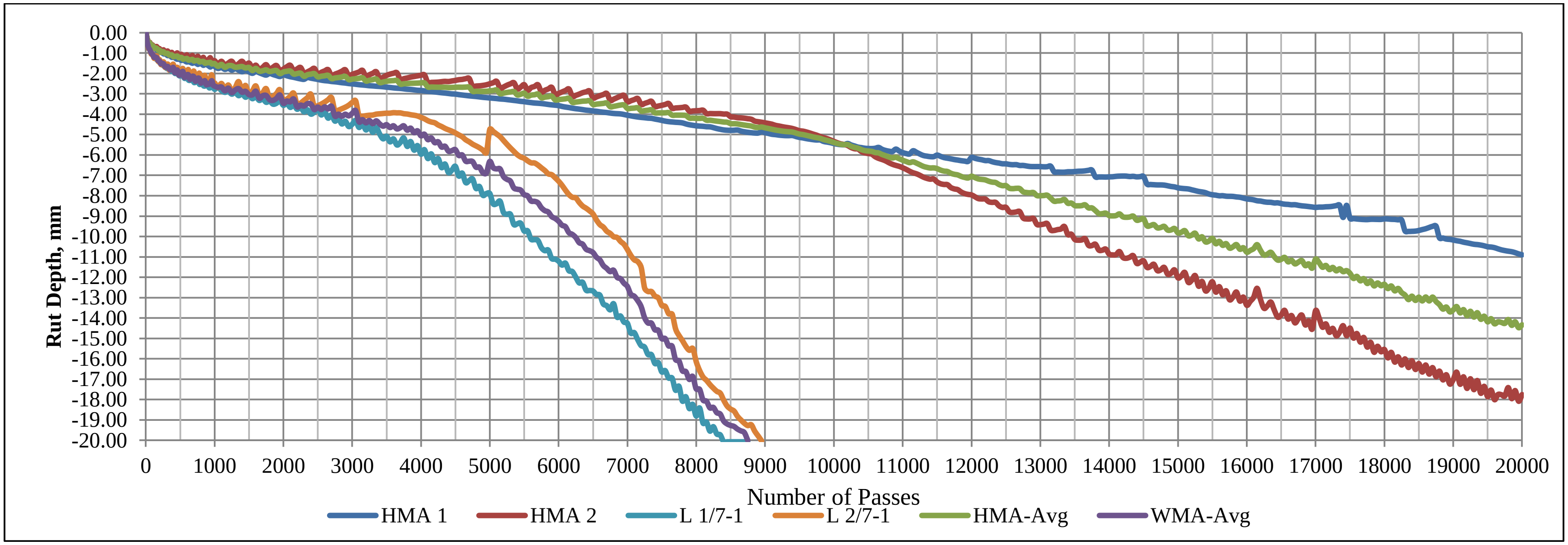

Figure 4.2 Hamburg Wheel Test Results for Mixtures Including 20% RAP.

Table 4.9 Hamburg Wheel Test Results for Mixtures Including 30% RAP.

Mix Type	Test ID	Air Voids %	Total Number of Passes	Inverse Creep Slope (Pass/mm)	Inverse Stripping Slope (Pass/mm)	SIP	Max. Rut Depth, mm
HMA	HMA 1	6.4	20000	3200	1161	8000	14.3
	HMA 2	6.3	20000	3200	853	8000	18.1
	Average	**6.3**	**20000**	**3200**	**1007**	**8000**	**16.2**
WMA-LEADCAP7-1	L 1	6.4	10650	1429	377	5000	20.0
	L 2	6.2	12250	1364	352	7500	20.0
	Average	**6.3**	**11450**	**1396**	**364**	**6250**	**20.0**

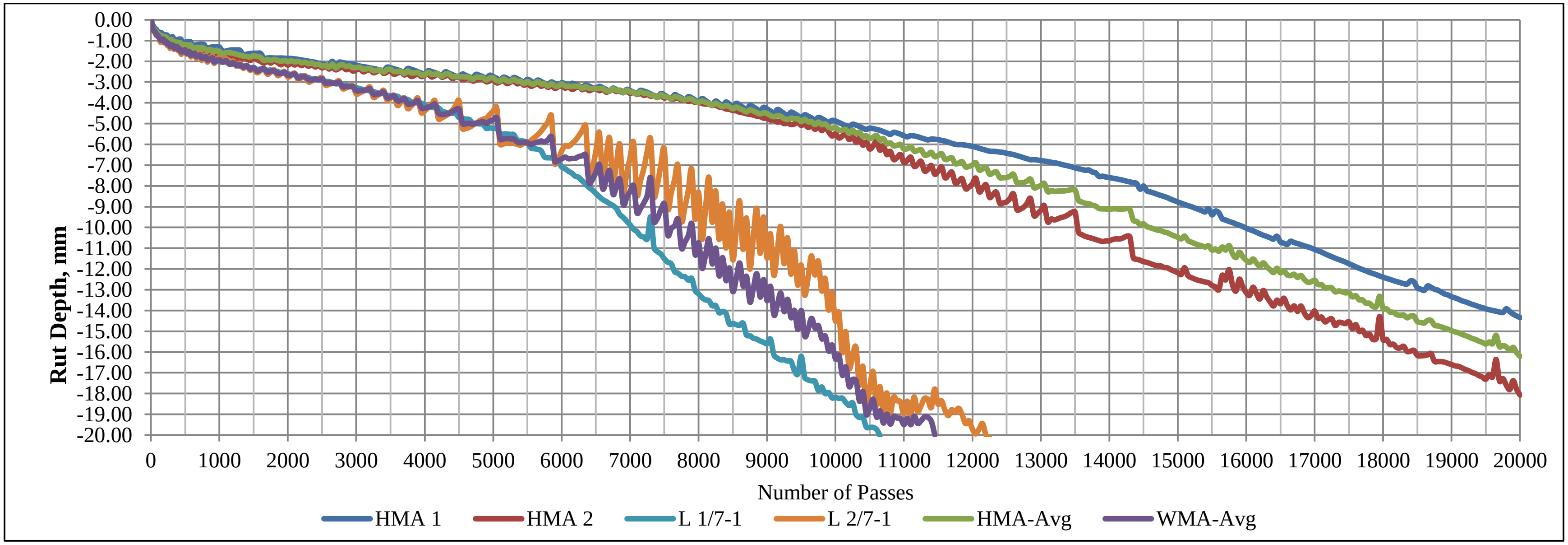

Figure 4.3 Hamburg Wheel Test Results for Mixtures Including 30% RAP.

Table 4.10 Hamburg Wheel Test Results for Mixtures Including 40% RAP.

Mix Type	Test ID	Air Voids %	Total Number of Passes	Inverse Creep Slope (Pass/mm)	Inverse Stripping Slope (Pass/mm)	SIP	Max. Rut Depth, mm
HMA	HMA 1	6.7	20000	4400	1343	11000	9.7
	HMA 2	6.6	20000	1667	1274	10000	14.4
	Average	**6.6**	**20000**	**3033**	**1309**	**10500**	**12.0**
WMA-LEADCAP7-1	L 1	6.8	14650	1938	468	7750	20.0
	L 2	7.35	11500	1304	407	6000	20.0
	Average	**7.06**	**13075**	**1621**	**438**	**6875**	**20.0**

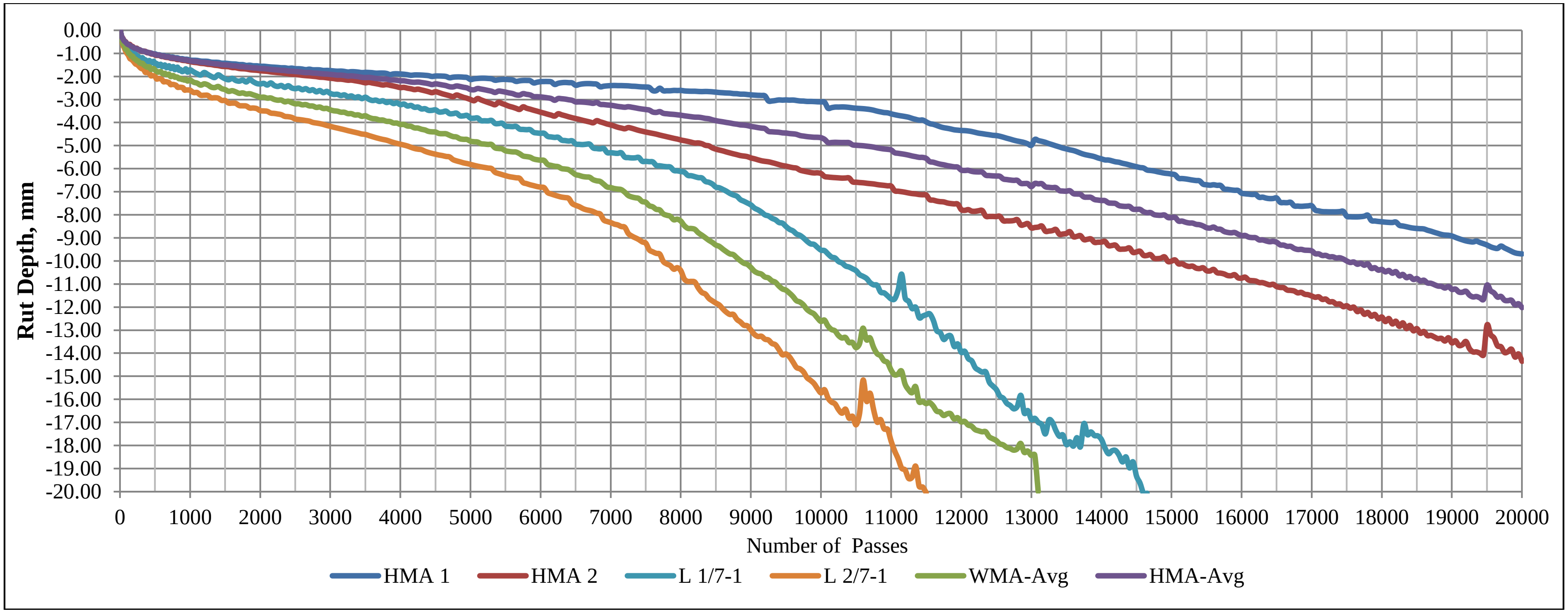

Figure 4.4 Hamburg Wheel Test Results for Mixtures Including 40% RAP.

Table 4.11 Hamburg Wheel Test Results for Mixtures Including 50% RAP.

Mix Type	Test ID	Air Voids %	Total Number of Passes	Inverse Creep Slope (Pass/mm)	Inverse Stripping Slope (Pass/mm)	SIP	Max. Rut Depth, mm
HMA	HMA 1	6.6	20000	7500	4202	15000	3.7
	HMA 2	6.5	20000	5455	3226	15000	4.8
	Average	**6.5**	**20000**	**6477**	**3714**	**15000**	**4.2**
WMA-LEADCAP7-1	L 1	6.2	20000	4500	509	13500	17.0
	L 2	6.1	20000	2462	765	8000	20.0
	Average	**6.2**	**20000**	**3481**	**637**	**10750**	**19.0**

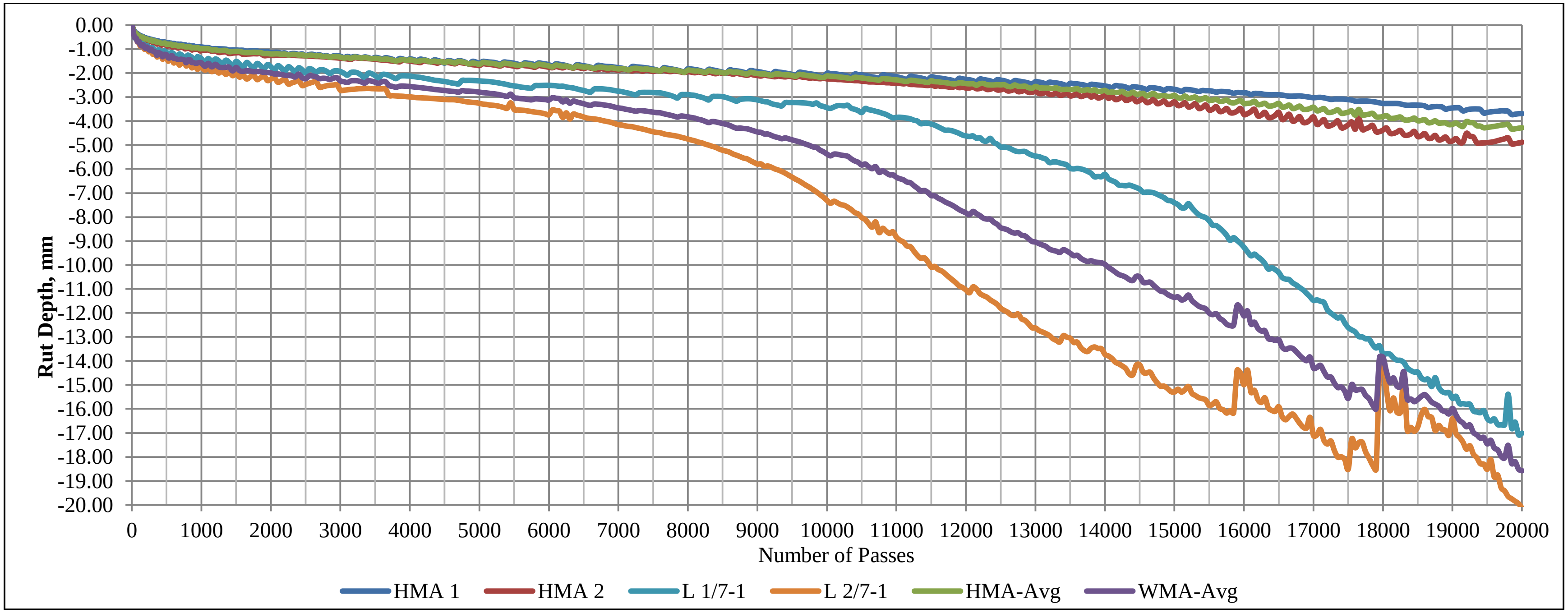

Figure 4.5 Hamburg Wheel Test Results for Mixtures Including 50% RAP.

Table 4.12 Hamburg Wheel Test Results for Mixtures Including 75% RAP.

Mix Type	Test ID	Air Voids %	Total Number of Passes	Inverse Creep Slope (Pass/mm)	Inverse Stripping Slope (Pass/mm)	SIP	Max. Rut Depth, mm
HMA	HMA 1	6.2	20000	8929	N/A	>20000	2.7
	HMA 2	6.1	20000	10870	N/A	>20000	2.3
	Average	**6.1**	**20000**	**9899**	**N/A**	**>20000**	**2.5**
WMA-LEADCAP7-1	L 1	6.5	20000	5970	N/A	>20000	3.9
	L 2	6.2	20000	6309	N/A	>20000	3.7
	Average	**6.3**	**20000**	**6140**	**N/A**	**>20000**	**3.8**

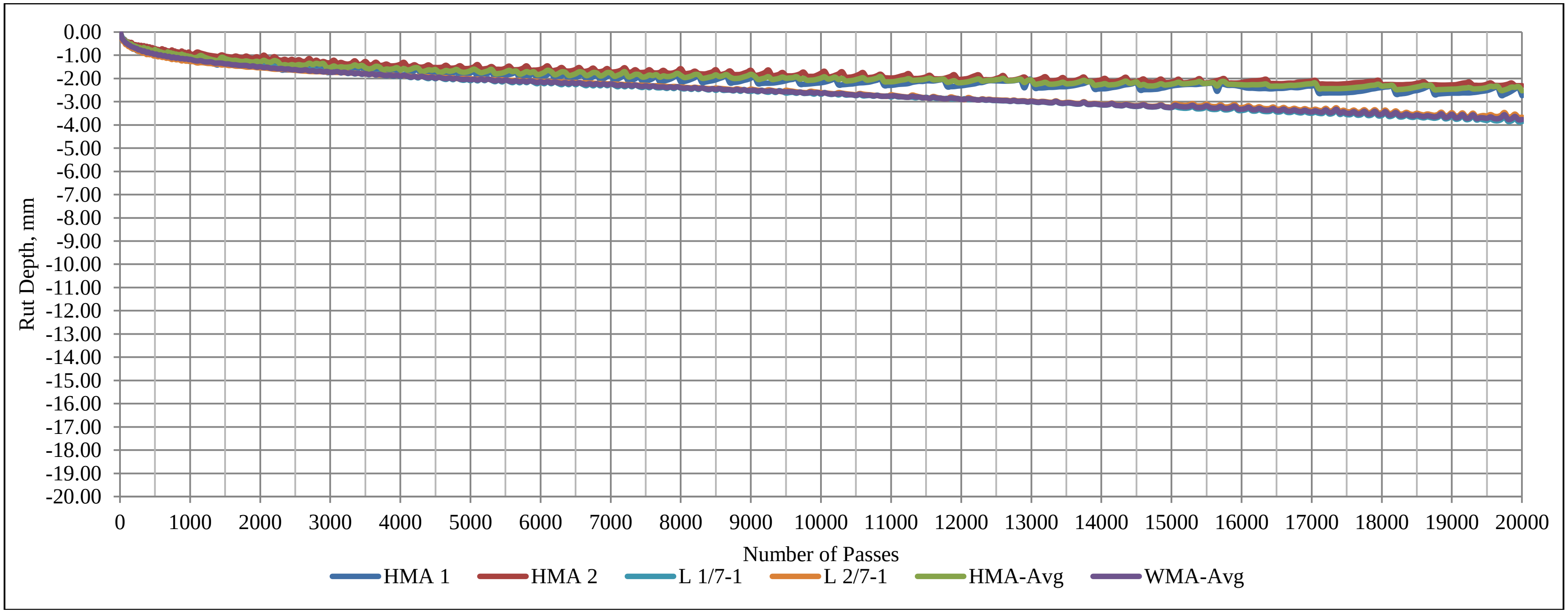

Figure 4.6 Hamburg Wheel Test Results for Mixtures Including 75% RAP.

Table 4.13 ANOVA Table for HWT SIP Values

Source	Degrees of Freedom	Type 1 SS	Mean Square	F Value	P-Value > F
RAP	4	569680750.0	142420187.5	25.28	<.0001
Mix Type	1	22260500.0	22260500.0	3.95	0.0749
RAP*Mix Type	4	12860750.0	3215187.5	0.57	0.6901

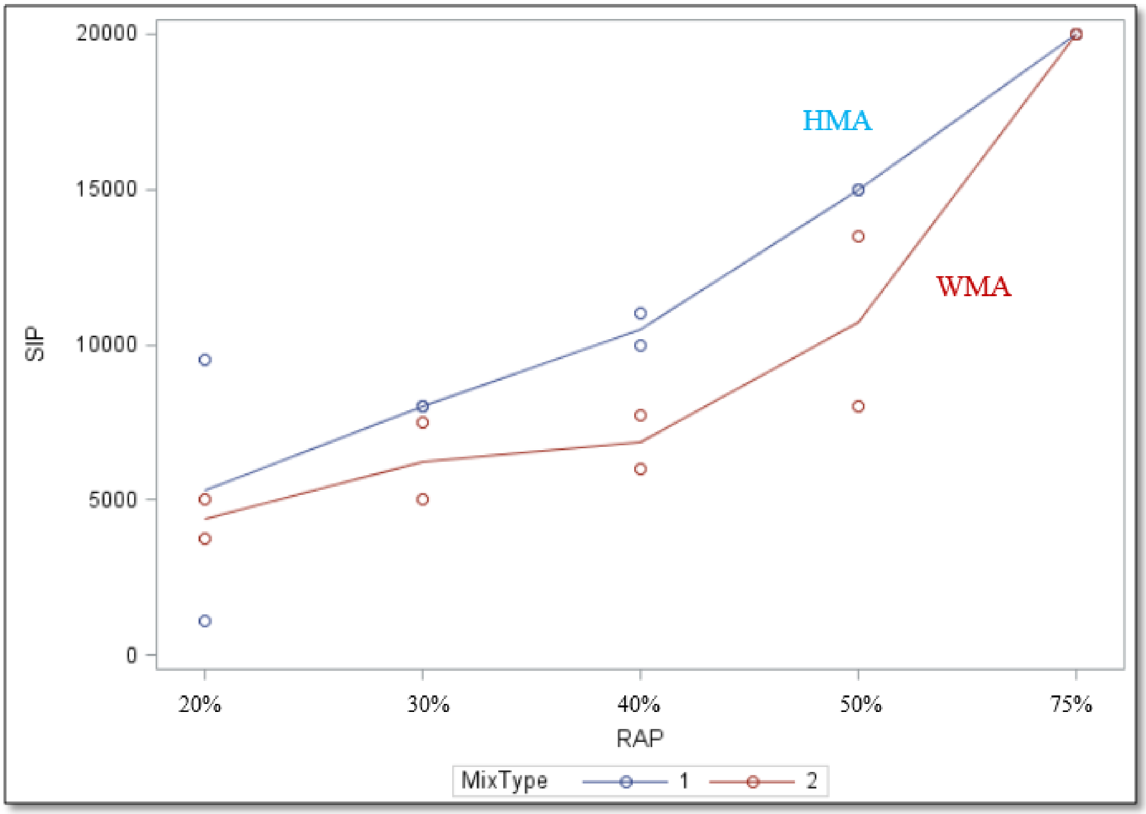

Figure 4.7 Interaction Plot for HWT Test SIP Values.

4.2. Secondary Laboratory Evaluation

This section describes the mix designs, and performance tests' results of the WMA mixtures with fractionated RAP materials described in task two of the research program.

4.2.1. Virgin Aggregate & RAP Material Properties

Limestone virgin aggregates of four different stockpiles and a steel slag stockpile collected from River Products quarry located in Coralville, Iowa along with 30% fractionated RAP by binder replacement from I-80 rehabilitation project were used to design the mix with ½ inch nominal maximum size. In order to meet the aggregate gradation requirements specified by Iowa DOT, RAP materials passing a sieve with a size of 5/16 inch were removed from the RAP stockpile. The Virgin aggregate properties, RAP material properties and the combined aggregate gradation for the designed mixture are summarized in Table 4.14. The combined gradation is plotted on the 0.45 power gradation chart in Figure 4.8. The combined aggregate gradation met all the Superpave mix design requirements specified by Iowa DOT.

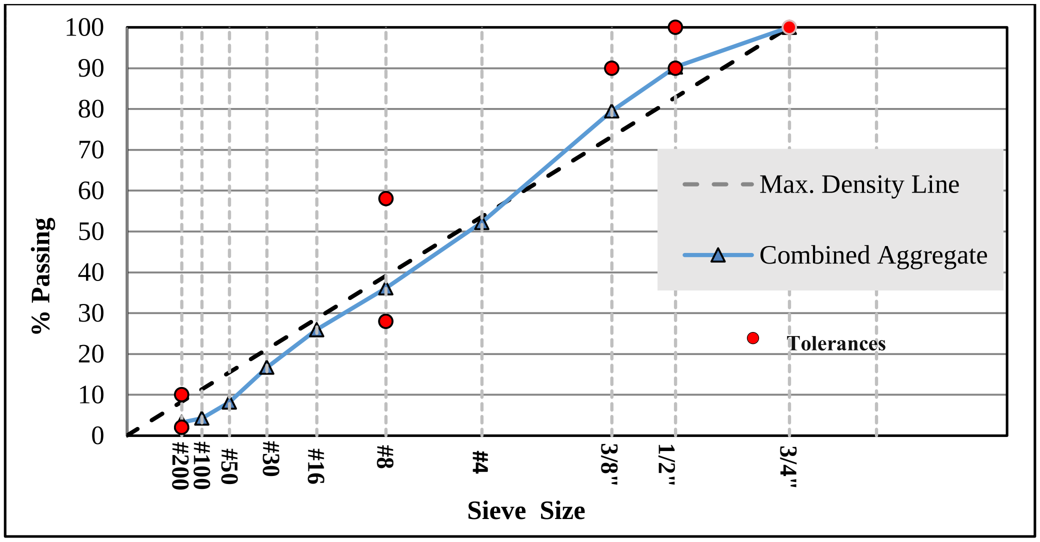

Figure 4.8 Aggregate Gradation Chart for HMA and WMA-RAPCAP Mixtures.

Table 4.14 Combined Aggregate Gradation and Mixture Properties with 30% RAP (>5/16”) Content.

Mix Design Info. ***Virgin Aggregate Properties*** ***RAP Properties***

Project Name	**WMA-RAP**	**Virgin Agg. Batch Mix**		G_{sb}	**ABS, %**	G_{sa}	**RAP (% Binder Replaced)**	**RAP ID**	**% of RAP**	G_{sb}	**ABS, %**	G_{sa}	**% AC**
Traffic ESAL	**10 M**	**Sand**	**20.0%**	2.634	0.47	2.667	**30.0%**	**Coarse > 5/16"**	**100%**	2.650	1.190	2.736	3.83
Mix Size	**1/2"**	**Man. Sand**	**25.0%**	2.649	0.84	2.709	**RAP (% Dry Mix Weight)**	**Fine < 5/16"**	**0.0%**	0.00	0.00	0.00	0.00
Layer	**Surface**	**3/4" A**	**15.0%**	2.652	0.86	2.705	**39.3%**	Combined RAP Mix		2.650	1.190	2.736	3.83
Virgin Binder	**PG 64-28**	**3/8" to Dust**	**25.0%**	2.642	1.04	2.717	**RAP (% Total Aggregate)**	Combined Mixture Properties		G_{sb}	**ABS, %**	G_{sa}	**% AC**
WMA Additive	**RAPCAP**	**3/8" Steel Slag**	**15.0%**	3.709	1.20	3.88				2.719	0.908	2.793	4.6
Combined Virgin Mix				2.763	0.792	2.830	**38.3%**						

Virgin Aggregate Batch Mix and RAP Material Combined Gradation

Sieve Size		**Stockpile and percentage passing**					**Virgin Agg.**	**RAP Gradation**		**Recovered**	**Comb. Grad.**	**Design Spec, 19.0 mm**	
ID	**mm**	**Sand**	**Man. Sand**	**3/4" A**	**3/8" to Dust**	**3/8" Steel Slag**	**Batch Mix**	**Coarse > 5/16"**	**Fine < 5/16"**	**Agg. Blend**	**w/RAP @ P_{bi}**	**Min.**	**Max.**
3/4 in	19.0	100.0	100.0	100.0	100.0	100.0	100.0	100	0	100.0	100.0	**100.0**	**100.0**
1/2 in	12.5	100.0	100.0	55.0	100.0	100.0	93.3	85.6	0.0	85.6	90.3	**90.0**	**100.0**
3/8 in	9.5	100.0	100.0	19.0	97.0	100.0	87.1	67.4	0.0	67.4	79.5	-	**90.0**
#4	4.75	95.0	98.0	4.0	42.0	31.0	59.3	40.9	0.0	40.9	52.2	-	-
#8	2.36	90.0	76.0	3.0	10.0	4.8	40.7	28.9	0.0	28.9	36.2	**28.0**	**58.0**
#16	1.18	79.0	43.0	3.0	9.0	3.0	29.7	19.9	0.0	19.9	25.9	-	-
#30	0.6	53.0	20.0	2.5	8.0	3.0	18.4	14.0	0.0	14.0	16.7	-	**25.0**
#50	0.3	16.0	8.3	2.5	7.5	2.5	7.9	8.7	0.0	8.7	8.2	-	-
#100	0.15	2.0	2.8	2.0	7.0	1.4	3.4	5.8	0.0	5.8	4.3	-	-
#200	0.075	1.0	2.5	2.0	5.0	1.0	2.5	4.5	0.0	4.5	3.3	**2.0**	**10.0**
% dry weight of aggregate		12.1%	15.2%	9.1%	15.2%	9.1%	Surf. Area	39.3%	0.0%	Surf. Area	Surf. Area	Total	
% of total Mixture		12.3%	15.4%	9.2%	15.4%	9.2%	3.72	38.3%	0.0%	4.23	3.91	OK	

4.2.2. Asphalt Binder Properties

The target performance grade (PG) for the binder used for all mixtures is PG 70-22. As it was mentioned earlier, Iowa DOT requires lowering the high temperature grade by one level for the HMA mixture that includes more than 20% RAP materials, therefore, asphalt binder with a performance grade of PG 64-28 was used for this mix design. The asphalt binder was supplied by Bituminous Materials & Supply (BM&S) Inc. in Tama, Iowa. The selected asphalt binder met all specifications as shown in Table 4.15. As recommended by the asphalt binder supplier, HMA mixing and compaction temperatures for the PG 64-28 binder were determined as 311° F (155°C) and 293° F (145°C), respectively.

Table 4.15 Asphalt Binder PG64-28 Test Results (Source: BM&S Co.).

ORIGINAL BINDER				
Test Method			**Test Results**	**Specifications**
Flash Point, ASTM D92-05a/AASHTO T48-04			250+	230ºC Min.
Rotational Vis @ 135C, ASTM D4402/AASHTO T316-04			1.062	3.000 Pa-s Max.
DSR (Dynamic Shear Rheometer), AASHTO T315-05				
Test Temperature, ºC	G*, kPa	Phase Angle, δ, degrees	**G*/sinδ, kPa**	
64	2.281	72.02	2.015	1.000 kPa Min.
Fail Temperature			72.25	report, ºC
Density (Pycnometer) ASTM D70-03/AASHTO T228-04			N/A	report
RTFO (ROLLING THIN FILM OVEN)				
Mass Loss, ASTM D2872-04/AASHTO T240-03			-0.74%	1.000% Max.
DSR (Dynamic Shear Rheometer), AASHTO T315-05				
Test Temperature, ºC	G*, kPa	Phase Angle, δ, degrees	**G*/sinδ, kPa**	
64	4.868	65.77	5.396	2.200 kPa min
Fail Temperature			N/A	report, ºC
PAV (PRESSURE AGING VESSEL), 100º C				
DSR (Dynamic Shear Rheometer), AASHTO T315-05				
Test Temperature, ºC	G*, kPa	Phase Angle, δ, degrees	**G*/sinδ, kPa**	
22	5858	40.41	3797	5000 kPa Max.
BBR (Bending Beam Rheometer), ASTM D6648-01/AASHTO T313-05				
Test Temperature, ºC				
-18	Stiffness, MPa		199.5	300 MPa Max.
	m-value		0.319	0.300 Min.
This binder meets the qualifications of a PG 64-28				

4.2.3. Mix Design

Table 4.6 summarizes the mixing and compaction temperatures for HMA laboratory samples determined following the Iowa DOT RAP mix design procedure. Mix design properties for both HMA and WMA with RAPCAP mixtures using 30% RAP amount by binder replacement are summarized in Table 4.16. Both HMA and WMA-RAPCAP mixtures met all the Iowa DOT mix design requirements. VMA of the HMA mixture barely met the minimum requirement (14 %).

Table 4.16 Mix design Summary for both HMA and WMA-RAPCAP Mixtures

Mix Design Properties	30 % RAP		Mix Design Criteria
	HMA	WMA-RAPCAP	
Target Air Voids, %	4.19	4.39	4.0 ± 0.5
Optimum Binder Content (%)	4.60	4.60	----
Virgin Asphalt Binder Content (ADD AC %)	3.22	3.22	----
Virgin Aggregate Content (% Mix Weight)	64	64	----
RAP Content (% Mix Weight)	36	36	----
RAP Content (% Total Aggregate)	35	35	----
Aggregate Bulk Specific Gravity (Gsb)	2.722	2.722	----
Aggregate Effective Specific Gravity (Gse)	2.767	2.757	----
Aggregate Apparent Specific Gravity (Gsa)	2.796	2.796	----
Aggregate Water Absorption (%)	0.898	0.898	----
Asphalt Binder Bulk Specific Gravity (Gb)	1.036	1.036	----
Bulk Specific Gravity at Optimum AC (Gmb)	2.462	2.449	----
Maximum Theoretical Specific Gravity (Gmm)	2.569	2.561	----
Percent Binder Absorption (Pba %)	0.61	0.48	----
Percent Effective Binder (P_{be} %)	4.01	4.14	----
Comb. Agg. Surface Area (m2/ Kg.)	3.90	3.90	----
Comb. Agg. Dust Content (P0.075 %)	3.22	3.22	Maximum 10%
%Gmm at N_{ini} (7 gyrations)	81.88	82.06	≤ 92.0%
%Gmm at N_{des} (96 gyrations)	95.81	95.61	96.0% ± 0.5
%Gmm at N_{max} (152 gyrations)	96.21	96.24	≤ 98.0%
Voids in Mineral Aggregate (VMA, %)	13.73	14.19	Minimum 14%
Voids Filled with Asphalt (VFA, %)	69.46	69.02	70% - 80%
Dust to Binder Ratio (P0.075/Pbe)	0.80	0.78	0.6 - 1.4
Film Thickness (µm)	10.30	10.63	8.0 - 15 µm

4.2.4. Performance Evaluation

4.2.5.1. Hamburg Wheel Track Test Results

The Hamburg Wheel Track Test results for HMA and WMA-RAPCAP mixtures are summarized in Table 4.17 and plotted in Figure 4.9.

Both HMA and WMA specimens successfully passed the test with average maximum rut depths of 2.8 mm for HMA specimens and 5.4 mm for WMA specimens. The average SIP values were greater than 20,000 passes for HMA specimens and 14167 passes for WMA specimens. Both HMA and WMA mixtures exhibited high rutting and moisture resistance during the HWT test.

4.2.5.2. Modified Lottman Test Results

The moisture sensitivity evaluation of HMA and WMA mixtures was conducted using Modified Lottman test according to “AASHTO T283: Standard Method of Test for Resistance of Compacted Asphalt Mixtures to Moisture-Induced Damage”. Prior to the test, the HMA loose mix was short-term aged for 4 hours at 135°C (275°F) then followed by 2 hours at the compaction temperature and the WMA loose mix was short-term aged for two hours at the compaction temperature then aged for 16 hours at 60°C (140°F) as recommended by NCHRP Report 691 (45). A minimum of 80% for retained tensile strength ratio (TSR) of the mixture is required by Iowa DOT to approve the mix design. Table 4.18 and Figure 4.10 show the results for the indirect tensile strength values (ITS) with the numbers above the bars representing the average values and the whiskers representing the standard deviations. Overlapping of the standard deviations implies the similarity in the measured ITS between the mixtures types.

Table 4.17 Hamburg Wheel Test Results for HMA and WMA-RAPCAP Mixture with 30% RAP.

Mix Type	Test ID	Air Voids %	Total Number of Passes	Inverse Creep Slope (Pass/mm)	Inverse Stripping Slope (Pass/mm)	SIP	Max. Rut Depth, mm
HMA	HMA 1	7.94%	20000	14493	N/A	>20000	2.9
	HMA 2	7.91%	20000	16393	N/A	>20000	3.1
	HMA 3	7.15%	20000	17699	N/A	>20000	2.3
	Average	**7.67%**	**20000**	**16195**	**N/A**	**>20000**	**2.8**
WMA-RAPCAP	R 1	7.57%	20000	8611	3982	15500	3.9
	R 2	7.05%	20000	7500	1259	15000	5.0
	R 3	7.43%	20000	5455	1318	12000	7.4
	Average	**7.35%**	**20000**	**7189**	**2187**	**14167**	**5.4**

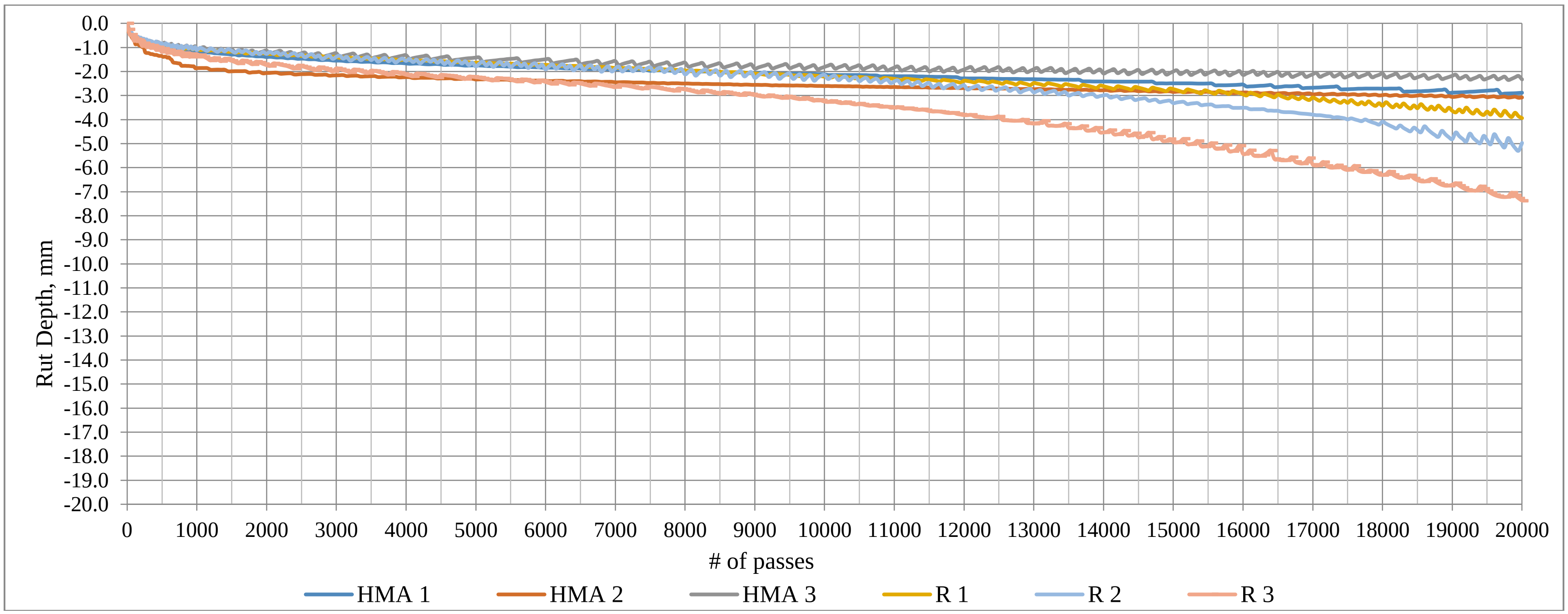

Figure 4.9 Hamburg Wheel Test Results for HMA and WMA-RAPCAP Mixture with 30% RAP.

As can be seen from Table 4.16 and Figure 4.9, the WMA mixture exhibited slightly lower unconditioned ITS values than those obtained for HMA mixture, however HMA and WMA mixtures exhibited similar conditioned ITS values. Both HMA and WMA mixtures met the minimum TSR requirement of 80% specified by the Iowa DOT. The conditioning process during the test, in which the samples were soaked in heated water at 60° C for 24 hours after freezing for 15 hours at -18° C, caused an activation to the limestone fine particles in the mixtures similar to the activation of hydrated lime by adding water to it. It is known that hydrated lime is anti-strip agent, which is used to improve moisture resistance in asphalt mixtures. The activation process resulted in a higher conditioned ITS values than the unconditioned ITS values in case of WMA and similar unconditioned ITS values in case of HMA and lead to TSR values higher than 100%.

Table 4.18 AASHTO T 283 Test Results for HMA and WMA-RAPCAP with 30% RAP.

Mix type	Unconditioned ITS, psi	Conditioned ITS, psi	Average TSR	Standard Deviation	
				Unconditioned ITS	Conditioned ITS
HMA	173	174	**101**	10.27	7.56
WMA-RAPCAP	163	175	**107**	12.14	8.46

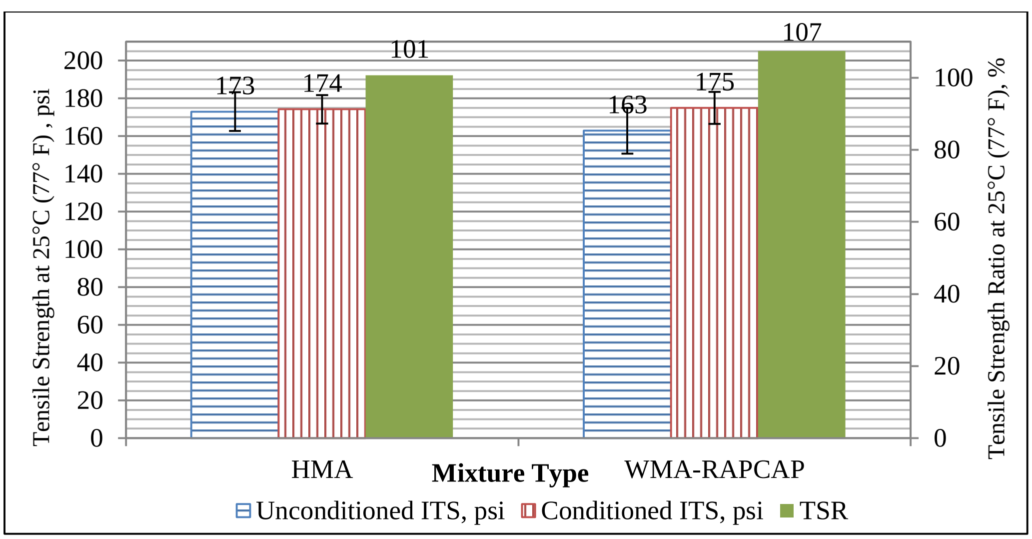

Figure 4.10 AASHTO T 283 Test Results for HMA and WMA-RAPCAP with 30% RAP.

4.3. Evaluation of the Asphalt Bond Strength (ABS) Test Method

As described in section 3.3.3.3, unconditioned samples only were evaluated in order to capture the effect of sample preparation method using different AFT values produced by different pullout stub thicknesses and the asphalt binder grade on the bond strength between the asphalt binder and the aggregate surface. Limestone aggregate plates were prepared following AASHTO TP 91-11 to be tested with four different asphalt binder grades; PG 58-28, PG 64-22, PG 64-28M and PG 70-22M, using 0.8 mm, 0.4 mm and 0.0 mm (no stub) pullout stub thicknesses (PST). Figure 4.11 shows the pullout stubs used for this study. Table 4.19 and Figure 4.12 show the ABS test results with the numbers above the bars representing the average values and the whiskers representing the standard deviations. Overlapping of the standard deviations implies the similarity in the measured ABS values between the different types of pullout stubs.

As can be seen from the test results, the 0.0 mm pullout stub produced significantly higher and more consistent ABS values than the other two stubs. In order to measure the actual film thickness, the aggregate plates were cut across the centerline of the asphalt samples and investigated under Olympus SZ61 microscope equipped with Olympus DP26 digital camera as shown in Figure 4.13.

Figure 4.11 0.8 mm (left), 0.4 mm (middle) and 0.0 mm (right) Pullout Stubs.

Table 4.19 ABS Test Results for Limestone Aggregate Plates.

Asphalt Type	PST, mm	ABS, Psi		Failure Mode
		Average	St. Dev.	
PG 58-28	0.8	135.0	19.5	>50% Cohesion
	0.4	129.0	8.2	>50% Cohesion
	0.0	329.0	27.6	>50% Adhesion
PG 64-22	0.8	307.3	17.2	>50% Cohesion
	0.4	229.0	20.1	>50% Cohesion
	0.0	488.0	35.4	>50% Adhesion
PG 64-28	0.8	233.1	16.8	>50% Cohesion
	0.4	263.0	9.6	>50% Cohesion
	0.0	412.3	17.2	>50% Cohesion
PG 70-22	0.8	259.0	10.5	>50% Cohesion
	0.4	249.7	36.1	>50% Cohesion
	0.0	454.0	29.6	>50% Cohesion

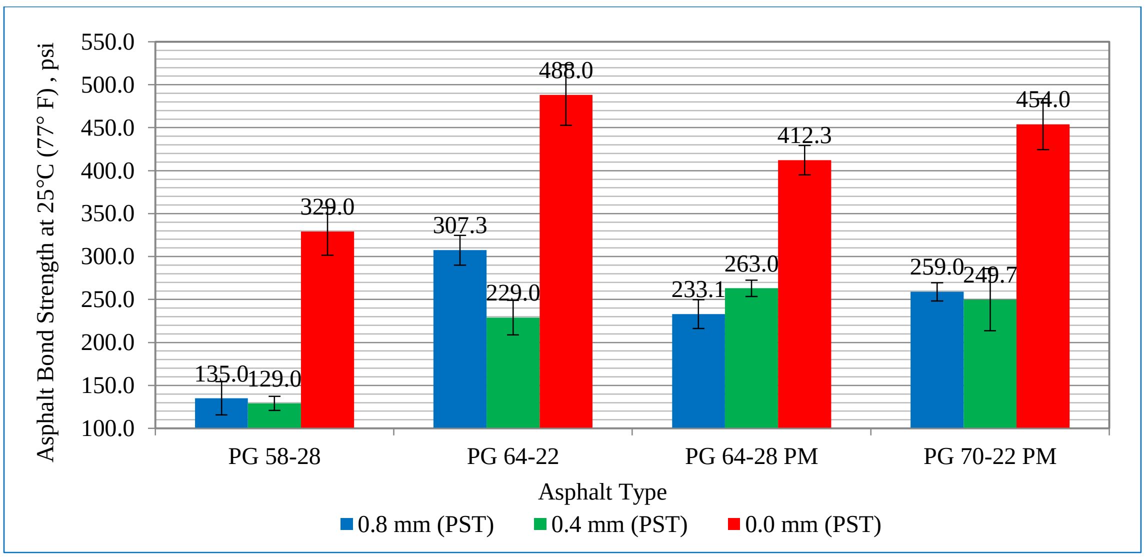

Figure 4.12 ABS Test Results for Limestone Aggregate Plates.

Figure 4.13 Asphalt Sample & Aggregate Plate Cross-sectional Cut under the Microscope

The average values of the measured FTs for 0.8 mm, 0.4 mm and 0.0 mm pullout stubs were 998 µm, 539 µm and 106 µm respectively. Figure 4.14 shows samples of asphalt film thicknesses (FTs) created by the different pullout stubs after running the test. It was noticed that the measured FTs values are slightly higher than the real stubs' thicknesses. This can be attributed to the irregularity and the pores in the aggregate plates' surfaces and the groves made in the stubs' surfaces in order to create interlocking between the pullout stubs and the aggregate plates. Also, the asphalt film thicknesses were measured after the asphalt samples were exposed to a direct tension test during the test, which resulted in a plastic or non-recoverable elongation in the asphalt samples.

Samples fail in cohesion if more than 50% of the binder is removed after the test is performed. Otherwise the failure is cohesive. Figure 4.15 shows a sample picture of a test sample with75% cohesive failure with the yellow areas indicate adhesive failure.

By looking at the measured asphalt FTs and the corresponded mode of failures, it can be seen that the fact that the higher the asphalt FT the more likely the asphalt mixture will fail in cohesive mode is illustrated. This explains why all the asphalt samples created using 0.8 mm, and 0.4 mm stubs failed in cohesive mode. On the other hand, the asphalt samples created by 0.0 mm stub showed both cohesive and adhesive failure modes depending on the asphalt performance grade. The fact that the ABS values obtained for 0.0 mm pullout stubs are significantly higher than others can be related to the more direct contact points between the pullout stub and the aggregate plate created by the 0.0 mm stubs than the other two stubs.

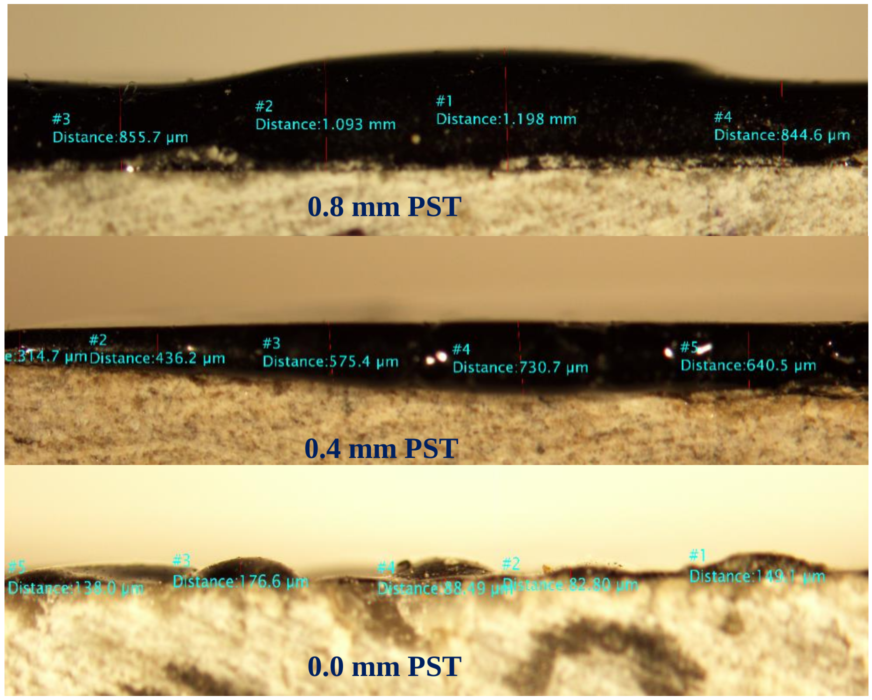

Figure 4.14 Samples of Asphalt Film thicknesses Created by Different Pullout Stubs


Figure 4.15 Test Sample with75% Cohesive Failure.

Tow way ANOVA analysis, as shown Table 4.20, was conducted to evaluate the impact of PST type, the asphalt performance grade and their interaction on the measured ABS values between the asphalt binders and the aggregate surface.

It can be seen from the two way ANOVA analysis that, the PST type, the asphalt performance grade as well as their interaction, as shown in Figure 4.14, had significant impacts on the measure ABS values between the asphalt binders and the aggregate surface

Table 4.20 ANOVA Table for the ABS Values.

Source	Degrees of Freedom	Type 1 SS	Mean Square	F Value	P-Value > F
Asphalt Grade (AC)	3	120048.2222	40016.0741	59.96	<.0001
PST	2	329435.0556	164717.5278	246.83	<.0001
AC*PST	6	16502.2778	2750.3796	4.12	0.0055

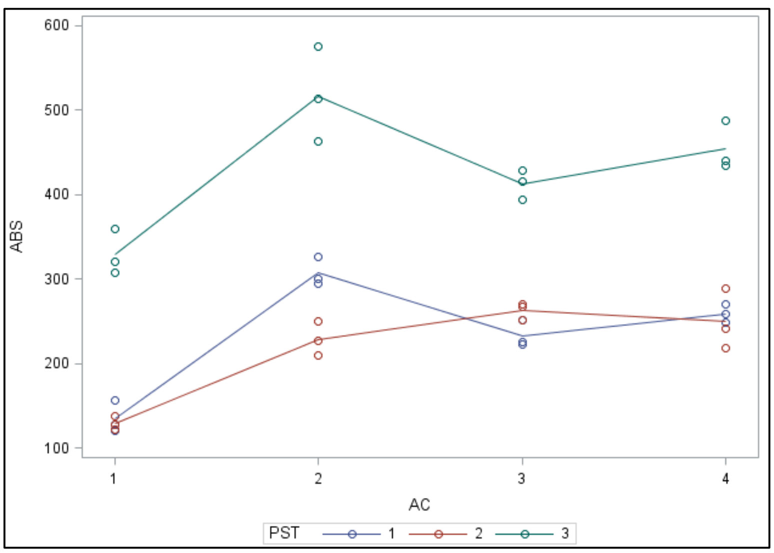

Figure 4.16 Interaction Plot of the ABS Values.

A paired mean comparison analysis at a significance level of 0.05 using t-test procedure, as shown in Table 4.21, was conducted to determine whether there are any statistical differences between the ABS values of the different pullout stubs obtained for the same asphalt type.

Based on the t-test analysis, it can be concluded that there are high significant differences between the 0.0 mm stubs and the other two stubs for all asphalt types. The 0.8

mm and 0.4 mm stubs showed no statistical differences between each other for all the asphalt binder types except for asphalt binder PG 64-28M, the statistical difference was very low.

In general, using the 0.0 mm pullout stubs produced more consistent and realistic ABS values by creating reasonable asphalt film thickness, and providing more direct contact points between the pullout stub and the aggregate surface.

Table 4.21 Summary of t-test Analysis for ABS Data.

Asphalt Type	Treatment, PST	ABS Values	
		Pr > │t│	*Significance
PG 58-28	0.8 mm vs 0.4 mm	0.7269	NS
	0.8 mm vs 0.0 mm	<0.0001	HS
	0.4 mm vs 0.0 mm	<0.0001	HS
PG 64-22	0.8 mm vs 0.4 mm	0.0366	LS
	0.8 mm vs 0.0 mm	0.0004	HS
	0.4 mm vs 0.0 mm	<0.0001	HS
PG 64-28M	0.8 mm vs 0.4 mm	0.048	LS to NS
	0.8 mm vs 0.0 mm	<0.0001	HS
	0.4 mm vs 0.0 mm	<0.0001	HS
PG 70-22M	0.8 mm vs 0.4 mm	0.6935	NS
	0.8 mm vs 0.0 mm	0.0001	HS
	0.4 mm vs 0.0 mm	0.0001	HS
* NS - Not Significant LS - Low Significant HS – High Significant			

CHAPTER 5 FIELD EVALUATIONS'

RESULTS AND ANALYSIS

This chapter describes the test results obtained for the primary and secondary field evaluations explained in task three and four of the research program and the test results obtained for the asphalt bond strength (ABS) values of the extracted asphalt binders described in section 3.3.3.3. Table 5.1 summarizes the properties of the evaluated test sections.

Table 5.1 Summary of the Test Sections Properties.

Test Section	Binder Type	RAP Content, TWM	Aggregate Type	WMA Additive	Layer Thickness	Construction Date
Iowa-Surface	64-28/M	38%	Limestone	RAPCAP	1.5 inch	9/9/2013
Minnesota-Surface	64-28/M	25%	Granit	LEADCAP 6-8	2.0 inch	7/10/2012
Ohio-Surface	70-22M	20%	Limestone	LEADCAP 7-1	1.25 inch	9/16/2013

5.1. State Highway 6 in Iowa City, Iowa

Two test sections were constructed for the field evaluation study; one HMA section with approximately 0.3 mile long, and one WMA section with approximately 0.5 mile long. The two test sections consisted of surface layer with a thickness of 1.5 inch. The HMA mix was placed on the inside lane and the WMA mix was placed on the outside lane. As shown in Figure 5.1, the test sections are located on the west bound of Highway 6 started from south of Lakeside drive towards the downtown of Iowa City, Iowa. The two mixtures were used for comparison: HMA as a control mixture and WMA mixture prepared using LEADCAP additive. The mixtures were designed according to Superpave mix design procedure for a high traffic level of 10 million ESALs per Iowa DOT mix design requirements and NCHRP 691 report "Mix Design Practices for Warm Mix Asphalt". The mixtures used Limestone virgin aggregate and 30% fractionated RAP by binder replacement. The LEADCAP technology used for this research effort was RAPCAP (liquid). RAPCAP (liquid) additive was added to the mix with a rate of 1.5% by optimum asphalt content of the mix.

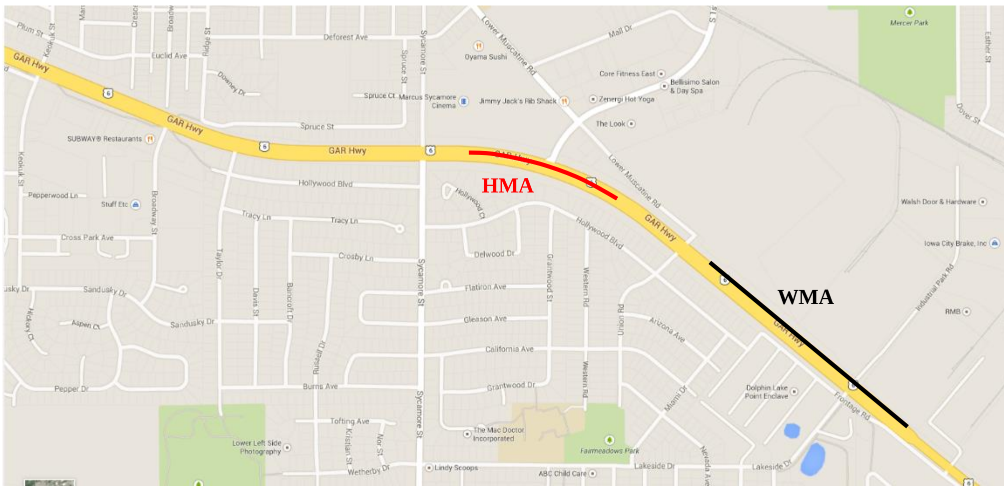

Figure 5.1 Iowa Test Sections Location.

5.1.1. Virgin Aggregate, RAP Material and Asphalt Binder Properties of Iowa Test Sections.

The aggregate gradation for this project was designed with ½ inch nominal maximum size using 6 different stockpiles collected from River Products Quarry in Coralville, Iowa. All virgin aggregate properties met Iowa DOT specifications. Table 5.2 shows the combined aggregate gradation information for the designed mixes and Figure 5.2 shows a plot of gradation on the 0.45 power gradation chart. The Fractionated RAP from I-80 Interstate Highway was used for building the test sections. In order to meet the mix design requirements, all RAP materials smaller than the 5/16-inch size from the RAP stockpile were removed.

The target performance grade (PG) for the binder used for all mixtures was PG70-

22. Iowa DOT requires lowering the PG for the binder used with any mixture includes more 20% RAP by one grade level. Therefore, asphalt binder with PG64-28 was used for the constructed test sections. The asphalt binder properties were evaluated according to AASHTO M320 standard. The selected asphalt binder met all specifications as shown in Table 5.3. Asphalt mixing temperature for the PG 64-28 was determined as 155° C (311° F), as recommended by the binder supplier company.

Table 5.2 Combined Aggregate Gradation and Properties for Iowa Test Sections (Source: LL Pelling).

Material	% in Mix	Producer & Location	Type (A or B)	Friction Type	G_{sb}	%Abs
Sand	11.0%	Williams/S&G Materials Inc.	A	4	2.634	0.47
TAT4 Man. sand	14.0%	Klein/River Products Co	A	4	2.649	0.84
3/4" A	11.0%	Klein/River Products Co	A	4	2.652	0.86
3/8" slag	14.0%	Montpelier/Blackheart Slag	A	2	3.709	1.20
3/8 W. chips	12.0%	Columbus Junction/River Products Co	A	4	2.583	3.23
Classified RAP	38.0%	38% ABC13-0119 (3.38 % AC)	A	2	2.662	1.30

Individual Aggregates Sieve Analysis - % Passing (Target)											
Material	1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200
Sand	100	100	100	100	95	90	79	53	16	2.0	1.0
TAT4 Man. sand	100	100	100	100	98	76	43	20	8.3	2.8	2.5
3/4" A	100	100	55	19	4.0	3.0	3.0	2.5	2.5	2.0	2.0
3/8" slag	100	100	100	100	31	1.8	1.6	1.5	1.5	1.4	1.0
3/8 W. chips	100	100	100	95	50	15	4.0	2.7	2.6	2.5	2.3
Classified RAP	100	100	93	80	51	36	27	20	14	10	8.8
Preliminary Job Mix Formula Target Gradation											
Upper Tolerance	100	100	99	90	61	42		21			6.4
Comb Grading	**100**	**100**	**92**	**83**	**54**	**37**	**26**	**17**	**9.0**	**5.1**	**4.4**
Lower Tolerance	100	100	85	76	47	32		13			2.4
S. A. sq. m/kg	Total	4.47		+0.41	0.22	0.30	0.43	0.49	0.55	0.62	1.44

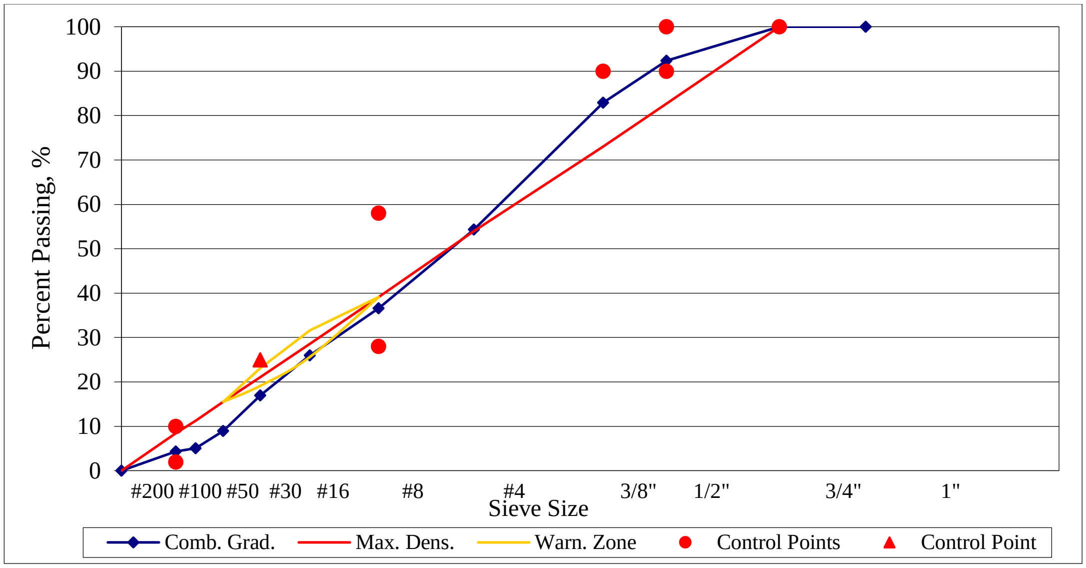

Figure 5.2 Combined Aggregate Gradation Chart for Iowa Test Sections.

Table 5.3 Asphalt Binder PG64-28 Test Results (Source: BM&S Co.).

ORIGINAL BINDER				
Test Method			**Test Results**	**Specifications**
Flash Point, ASTM D92-05a/AASHTO T48-04			250+	230ºC Min.
Rotational Vis @ 135C, ASTM D4402/AASHTO T316-04			1.062	3.000 Pa-s Max.
DSR (Dynamic Shear Rheometer), AASHTO T315-05				
Test Temperature, ºC	G*, kPa	Phase Angle, δ, degrees	**G*/sinδ, kPa**	
64	2.281	72.02	2.015	1.000 kPa Min.
Fail Temperature			72.25	report, ºC
Density (Pycnometer) ASTM D70-03/AASHTO T228-04			n/a	Report
RTFO (ROLLING THIN FILM OVEN)				
Mass Loss, ASTM D2872-04/AASHTO T240-03			-0.74%	1.000% Max.
DSR (Dynamic Shear Rheometer), AASHTO T315-05				
Test Temperature, ºC	G*, kPa	Phase Angle, δ, degrees	**G*/sinδ, kPa**	
64	4.868	65.77	5.396	2.200 kPa min
Fail Temperature			N/A	report, ºC
PAV (PRESSURE AGING VESSEL), 100C				
DSR (Dynamic Shear Rheometer), AASHTO T315-05				
Test Temperature, ºC	G*, kPa	Phase Angle, δ, degrees	**G*/sinδ, kPa**	
22	5858	40.41	3797	5000 kPa Max.
BBR (Bending Beam Rheometer), ASTM D6648-01/AASHTO T313-05				
Test Temperature, ºC				
-18	Stiffness, MPa		199.5	300 MPa Max.
	m-value		0.319	0.300 Min.
This binder meets the qualifications of a PG 64-28				

5.1.2. Mix Design of Iowa Test Sections

Mixing and compaction temperatures for the two mixtures were established following Iowa DOT RAP mix design procedure and they are summarized in Table 5.4. The mix design for the HMA with 30% RAP materials by binder replacement is summarized in Table 5.5. The mix design met all requirements except VMA and the optimum binder content was relatively low (4.33%). Since 30% of the optimum binder content (4.33%) is to be provided by the binder from the RAP materials, only 3.1% virgin asphalt was added.

Table 5.4 Mixing and Compaction Temperatures for Iowa Test Sections.

Binder Type	Mixture	Binder Temp.	Mixing Temp.	Comp. Temp.
PG 64-28	HMA	300° F (150° C)	300° F (150° C)	285° F (140° C)
PG 64-28	WMA	300° F (150° C)	275° F (135° C)	250° F (125° C)

Table 5.5 Mix Design Summary for Iowa Test Sections (source: LL Pelling).

Adjust grade to PG 64-28		Gyratory Data					
% Asphalt Binder		3.9	**4.33**	4.6	4.9		Number of Gyrations
Gmb @ N-Des.		2.491	**2.501**	2.508	2.529		N-Initial
Max. Sp.Gr. (Gmm)		2.625	**2.606**	2.594	2.585		8
% Gmm @ N- Initial		86.7	**87.9**	88.6	89.2		N-Design
%Gmm @ N-Max			**96.5**				96
% Air Voids		5.1	**4**	3.3	2.2		N-Max
% VMA		13.1	**13.2**	13.2	12.7	**OUT**	152
% VFA		61.2	**69.7**	74.9	83		Gsb for Angularity Method A
Film Thickness		7.43	**8.43**	9.08	9.62		2.646
Filler Bit. Ratio		1.33	**1.17**	1.08	1.02		
Gse		2.801	**2.8**	2.799	2.803		Pba / %Abs Ratio
Pbe		3.32	**3.77**	4.06	4.3		0.46
Pba		0.6	**0.59**	0.57	0.63		
% New Asphalt Binder		67.9	**71.3**	73	74.7		Slope of Compaction Curve
Combined Gb @ 25°C		1.0296	**1.0296**	1.0296	1.0296		13.6
Aggregate Type Used	A				Combined	From RAM	Excellent
G_{sb}	2.756	% Friction Type 4 (+4)			62.6	24	Pb Range Check
G_{sa}	2.859	Or Better			89.1	29	1
% Water Abs	1.31	% Friction Type 3 (+4)			0	0	RAM Check
S.A. m^2 / Kg.	4.47	Or Better			26.5	5	OK
Angularity-method A	43	% Friction Type 2 (+4)			26.5	5	
% Flat & Elongated	0.6	% Friction Type 2 (-4)			9	0.9	Specification Check
Sand Equivalent	91	Type 2 Fineness Modulus			2.4	1.6	OUT Does Not Comply
Virgin G_b @ 25°C	1.0294	% Crushed			83	31.6	
Disposition : An asphalt content of			4.30%	is recommended to start this project.			
Data shown in	4.33%	Column is interpolated from test data.		The % ADD AC to start project is 3.10%			

5.1.3. Field Compaction and Mat Densities of Iowa Test Sections

The construction of the HMA, and WMA test sections started at 7 pm on the 8th and the 9th of September, 2013 respectively. The construction process went very well in terms of field compaction. As shown in Figure 5.3, the HMA mixture produced more emission and smoke than the WMA mixture that produced little smoke or emission. Figure 5.4 shows pictures of the test sections after construction is completed.

Figure 5.3 HMA (left), and WMA (right) Emissions during Construction.

Figure 5.4 Iowa Test Sections after Compaction.

The target air voids during construction was 5.0 to 6.0%. A total number of 6 cores were collected from each test section to measure the filed densities of the test sections. The average densities of the HMA and WMA test sections were 94.3%, and 93.9%, respectively. Tables 5.6 and 5.7 show the density data for HMA and WMA, respectively.

Table 5.6 Density Data for Iowa HMA Test Section (Source: LL Pelling).

COMPACTED MAT_HMA TEST SECTION										
Core	Station	CL Reference	W1 Dry (g)	W2 in H20 (g)	W3 Wet (g)	Diff.	G_{mb}	% of G_{mm}	P_a (%)	Thickness (in.)
1	234+65	3.0 S\W Pass	819.5	479.5	820.0	340.5	2.407	92.3	7.7	1.63
2	229+88	4.6 S\W Pass	867.8	516.5	868.3	351.8	2.467	94.6	5.4	1.75
3	229+33	8.0 S\W Pass	814.9	487.5	815.1	327.6	2.487	95.3	4.7	1.50
4	216+40	7.6 S\W Pass	715.3	422.9	715.9	293.0	2.441	93.6	6.4	1.50
5	213+89	1.0 S\W Pass	701.3	417.2	701.9	284.7	2.463	94.4	5.6	1.25
6	209+39	7.2 S\W Pass	650.4	389.9	650.8	260.9	2.493	95.6	4.4	1.25
Course Placed:			Surface (Travel Lane)				Thickness QI:		0.96	
Intended Lift Thickness:			1.50				Avg. Mat Density:		2.460	
Date Placed:			09/08/13				Avg. % of G_{mm}:		94.300	
Test Date/By:			09/08/13				Avg. % Field Voids:		5.70	

Table 5.7 Density Data for Iowa WMA Test Section (Source: LL Pelling).

COMPACTED MAT_WMA TEST SECTION										
Core	Station	CL Reference	W1 Dry (g)	W2 in H20 (g)	W3 Wet (g)	Diff.	G_{mb}	% of G_{mm}	P_a (%)	Thickness (in.)
1	268+00	6.9 S\W Drv	767.2	460.5	776.7	316.2	2.426	92.9	7.1	1.50
2	262+74	9.6 S\W Drv	836.4	496.2	837.0	340.8	2.454	94.0	6.0	1.75
3	260+65	9.8 S\W Drv	793.0	468.2	793.7	325.5	2.436	93.3	6.7	1.63
4	256+63	1.0 S\W Drv	791.0	471.6	791.6	320.0	2.472	94.6	5.4	1.50
5	250+69	2.6 S\W Drv	772.8	460.5	773.4	312.9	2.470	94.6	5.4	1.50
6	244+98	7.6 S\W Drv	832.3	494.2	832.8	338.6	2.458	94.1	5.9	1.75
Course Placed:			Surface (Travel Lane)				Thickness QI:		2.42	
Intended Lift Thickness:			1.50				Avg. Mat Density:		2.453	
Date Placed:			09/09/13				Avg. % of G_{mm}:		93.917	
Test Date/By:			09/09/13				Avg. % Field Voids:		6.08	

5.1.4. Performance Evaluation of Iowa Test Sections

5.1.4.1. Hamburg Wheel Track Test Results

HWT Test results of HMA and WMA mixtures are summarized in Table 5.8 and plotted in Figure 5.5. As it can be seen from the test results, both HMA and WMA specimens successfully passed the test with average maximum rut depths of 2.6 mm for HMA specimens and 4.5 mm for WMA specimens. The average SIP were greater than

20,000 passes for both HMA and WMA specimens. It can be concluded that both HMA and WMA mixtures exhibited similar high resistance to rutting and moisture damage.

5.1.4.2. Modified Lottman Test Results

Table 5.9 and Figure 5.6 show the results for the indirect tensile strength values (ITS) with the numbers above the bars representing the average values and the whiskers representing the standard deviation. Overlapping of the standard deviation implies the similarity in the measured ITS between the mixtures types. Figure 5.7 shows pictures of the HMA and WMA conditioned samples after performing the test.

As can be seen from the AASHTO T 283 test results, the WMA mixture prepared using RAPCAP exhibited significantly lower ITS values than those obtained for HMA mixtures. Also, the WMA mixture showed significantly lower conditioned ITS values than its unconditioned ITS values, which shows less resistance to moisture damage. On the other hand, HMA mixture exhibited similar unconditioned and conditioned ITS values, which, showed no sensitivity to moisture damage in the modified Lottman test. A closer look at the WMA and HMA conditioned samples after the test, we can see the broken aggregate particles with a small amount of stripping in case of HMA samples, and more broken aggregate with higher amount of striping in case of WMA samples. This can be attributed to the softening effect of the RAPCAP additive even with the existence of high RAP amount in the mix, unlike the HMA samples which showed higher stiffness due to the existence of high RAP amount in the mix. Both HMA and WMA mixtures met the minimum TSR requirement of 80% specified by the Iowa DOT.

Table 5.8 Hamburg Wheel Test Results for Iowa Test Sections.

Mix Type	Test ID	Air Voids %	Total Number of Passes	Inverse Creep Slope (Pass/mm)	Inverse Stripping Slope (Pass/mm)	SIP	Max. Rut Depth, mm
HMA	HMA 1	7.94%	20000	13072	N/A	>20000	2.8
	HMA 2	7.91%	20000	9699	N/A	>20000	3.3
	HMA 3	7.15%	20000	18692	N/A	>20000	1.6
	Average	**7.67%**	**20000**	**13821**	**N/A**	**>20000**	**2.6**
WMA-RAPCAP	R 1	7.57%	20000	7117	N/A	>20000	4.1
	R 2	7.05%	20000	4374	N/A	>20000	5.7
	R 3	7.43%	20000	10638	N/A	>20000	3.7
	Average	**7.35%**	**20000**	**7377**	**N/A**	**>20000**	**4.5**

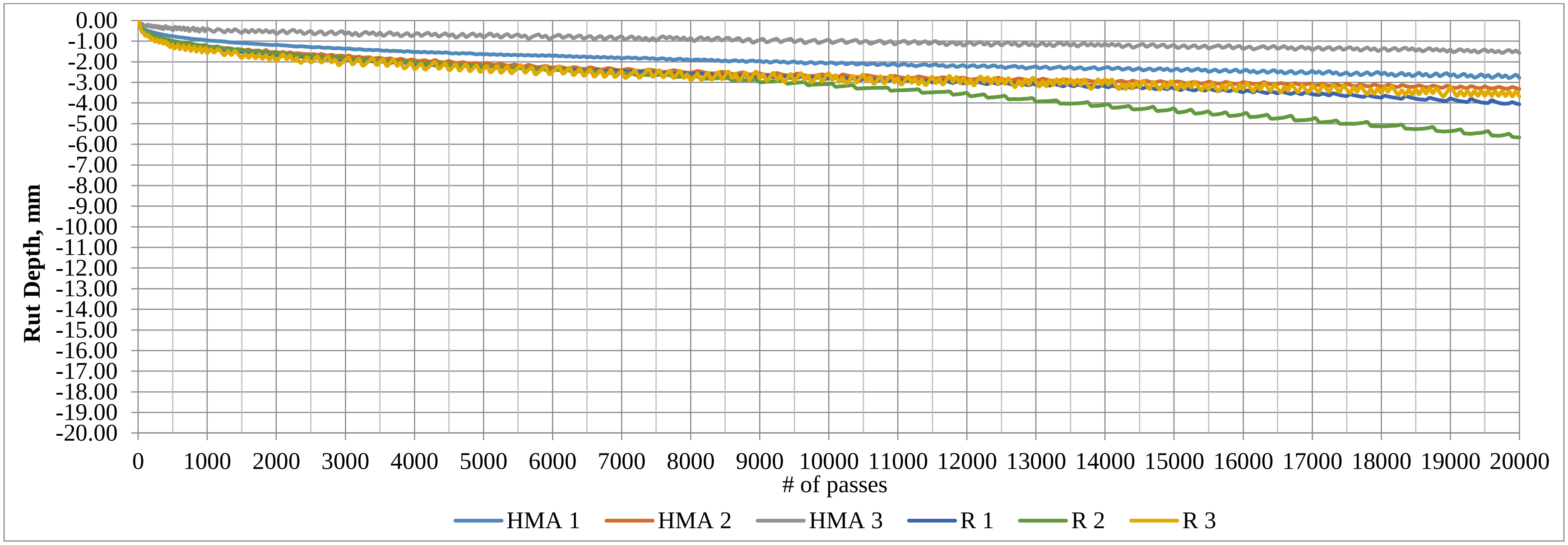

Figure 5.5 Hamburg Wheel Test Results for Iowa Test Sections.

Table 5.9 AASHTO T 283 Test Results for Iowa Test Sections.

Mix type	Unconditioned ITS, psi	Conditioned ITS, psi	Average TSR	Standard Deviation	
				Unconditioned ITS	Conditioned ITS
HMA	208	200	96	24.19	10.42
WMA-RAPCAP	168	142	84	6.14	6.20

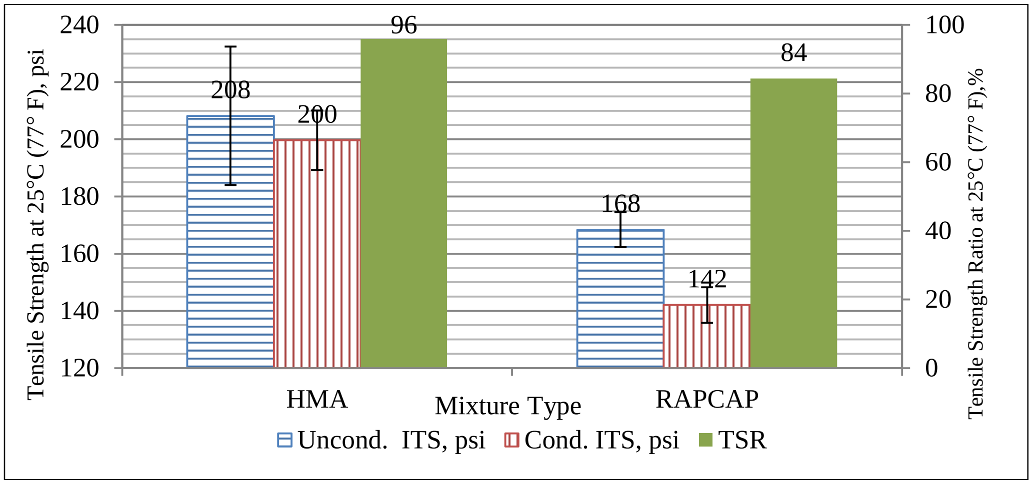

Figure 5.6 AASHTO T 283 Test Results for Iowa Test Sections.

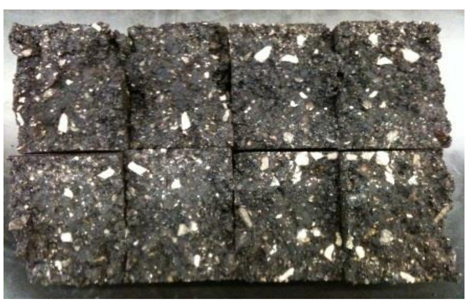


Figure 5.7 HMA (Left) and WMA (Right) Conditioned Sets of Iowa Test Sections.

5.1.5. Assessment of the Iowa Test Sections after Construction and Over Time

A condition survey was conducted in April 2014 in order to evaluate the WMA mixtures performance after 6 months of service and after exposing to a severe winter season. The condition survey was conducted and the pavement condition indices (PCIs) were calculated following the ASTM D6433 standards as described earlier in section 3.5.

The test sections of Iowa were constructed on top of old deteriorated concrete pavement. Both HMA and WMA pavements performed well during the severe winter season of 2014, however medium and high severity joint reflection cracks with densities of 1.83% for High, and 3.67% for medium by surveyed area, were developed from the concrete joints underneath the asphalt pavement layer. The occurrence of joint reflection cracks in the pavement surface dropped its condition for both HMA and WMA test sections from "Good" condition with PCIs of 100 right after construction to "Satisfactory" condition with PCIs of approximately 72 after 6 months since construction. Figure 5.8 and 5.9 show samples of medium and high severities joint reflection cracks from Iowa test sections. Additionally, small amounts of low severity slippage cracks with a density of 0.83% by surveyed area were observed in both HMA and WMA test sections, as shown in Figure 5.10.

Figure 5.8 Medium Severity Joint Reflection Cracks from Iowa Test Sections.

Figure 5.9 High Severity Reflection Joint Cracks from Iowa Test Sections.

Figure 5.10 Low Severity Slippage Cracks from Iowa Test Sections.

5.2. State Highway TH 169 in Champlin, Minnesota

Two test sections were constructed on July 10th, 2012 in Champlin, Minnesota; one for WMA and another one for HMA for comparison. Two-inch mill and overlay was applied on the southbound outside lane of TH 169 state highway in Champlin between the Mississippi River and Hayden Lake Road. Figure 5.11 shows the location of Minnesota test sections.

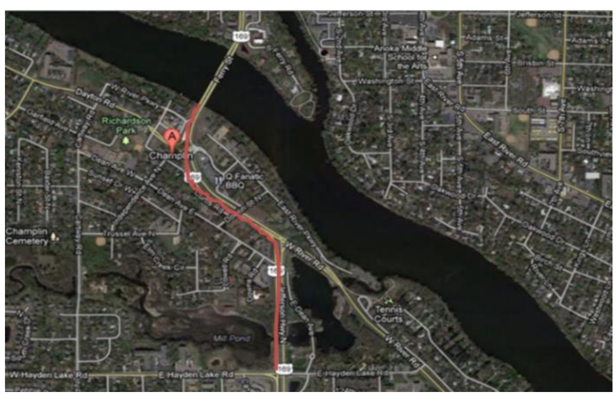

Figure 5.11 Minnesota Test Sections Location.

The HMA and WMA mixtures were designed according to Superpave mix design procedure for a medium traffic level of 3 to 10 million ESALs according to MnROAD mix design requirements. The mixtures used Granite virgin aggregate and 25% traditional RAP by total weight of mix. The LEADCAP technology used for this project was LEADCAP 6-

8. LEADCAP 6-8 additive was added to the mix with a rate of 1.5% by optimum asphalt content of the mix.

5.2.1. Virgin Aggregate, RAP Material, and Asphalt Binder Properties of Minnesota Test Sections

The aggregate gradation for this project was designed with ½ inch nominal maximum size using 5 different stockpiles collected from Valley Paving, Inc. quarry located in Shakopee, MN. The used RAP for this project was collected from different milling projects in Minnesota by Valley Paving Inc. All virgin aggregate and RAP properties met MnROAD specifications. Table 5.10 shows the combined aggregate gradation information for the designed mixes and Figure 5.12 shows a plot of gradation on the 0.45 power gradation chart. A polymer modified asphalt binder with PG 64-28 was used for both WMA and HMA test sections. The asphalt binder was supplied by Flint Hills Resources in Rosemount, Minnesota. The used asphalt binder properties met all the requirements for PG 64-28. Asphalt mixing temperature for the PG 64-28 was determined as 155° C (311° F), as recommended by the binder supplier company. As mentioned before, the LEADCAP technology used for this project was LEADCAP 6-8.

5.2.2. Mix Design of Minnesota Test Sections

Mixing and compaction temperatures for the two mixtures were established following MnROAD SuperPave mix design procedure and they are summarized in Table 5.11 MnROAD. The mix design verification and summary for both HMA and WMA mixtures is shown in Table 5.12. All the mix design requirements for both HMA and WMA mixtures were met.

Table 5.10 Combined Aggregate Properties for Minnesota Test Sections.

%	Pit	Source of Material	Total Sp. G.	Minus # 4 % Passing	Minus # 4 Sp. G.
30.0%	71002	Barton Elk River 9/16 Clear	2.751	6.0%	2.751
24.0%	71002	Barton Elk River Washed Man Sand	2.728	99.0%	2.728
15.0%	19106	Kramer Burnsville Washed Sand (Limestone)	2.700	99.0%	2.700
25.0%		Plant Milling	2.656	76.0%	2.656
6.0%	71059	Barton Elk River #2 Screened Sand	2.639	93.0%	2.639
Mix Aggregate Specific Gravity at the Listed Percentages =			**2.707**		**2.639**

Sieve Size ID	Sieve Size mm	Combined Gradation	Design Spec, NMAS 1/2 inch Min.	Design Spec, NMAS 1/2 inch Max.
3/4 in	19	100	100	100
1/2 in	12.5	94	85	100
3/8 in	9.5	83	35	90
#4	4.75	65	30	80
#8	2.36	48	25	65
#16	1.18	33	-	-
#30	0.6	22	-	-
#50	0.3	11	-	-
#100	0.15	5	-	-
#200	0.075	3.1	2.0	7.0
Spec. Voids		4.0	3.0	5.0
% AC		5.4	5.0	**PG 64-28**

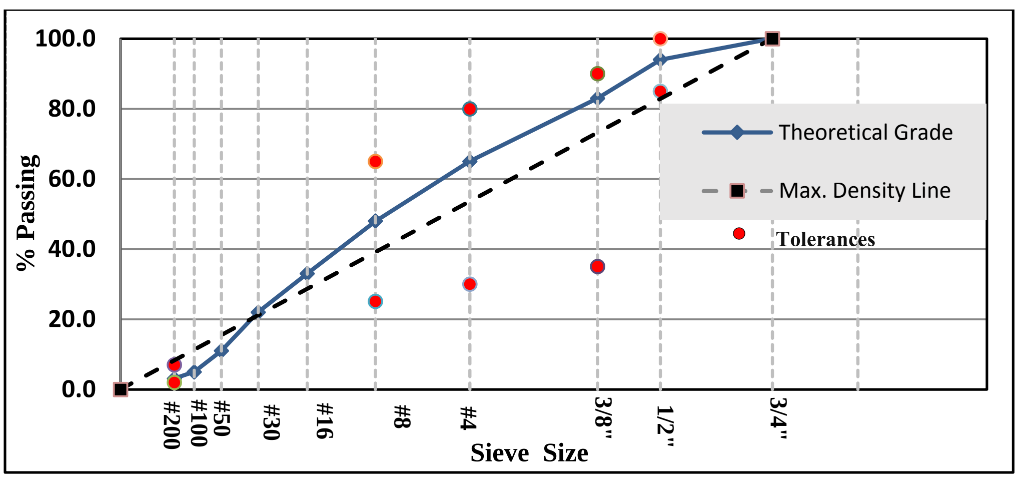

Figure 5.12 Aggregate Gradation Chart for Minnesota Test Sections (Source: MnROAD).

Table 5.11 Mixing and Compaction Temperatures for Minnesota Test Sections (Source: MnROAD).

Binder Type	Mixture	Binder Temp.	Mixing Temp.	Comp. Temp.
PG 64-28	HMA	311° F (155° C)	311° F (155° C)	285° F (140° C)
PG 64-28	WMA	311° F (155° C)	275° F (135° C)	255° F (125° C)

Table 5.12 Mix Design Summary Sheet for Minnesota Test Sections (Source: MnROAD).

Rev.12APR2011

AFT Project

TEST SUMMARY SHEET - GYRATORY DESIGN

Project #'s: CERTIFIED PLANT — Mix Designation: SPWEB440E

Location: VARIOUS — Plant Name: 904 MAPLE GROVE — Course: WE (Wear) - 4% Voids

SPEC. YEAR "2011"			Cont'r	Agency	Cont'r	Agency	Cont'r	Agency	Cont'r	Agency
ACsg	1.035	Test#	429		430	MnDOT	431	MnDOT	432	
		Date/Day:	6/25/2012		7-9-12 NIGHT		7/10/12 NIGHT		7/10/12 NIGHT	
		MDR #	2012-016		2012-016		2012-016		2012-016	
-4 Aggregate Bulk SpG.(Gsb) =			2.694		2.694		2.694		2.694	
Mix Aggregate Bulk SpG.(Gsb) =			2.706		2.706		2.706		2.706	
Project Number =			CTY MPLS		SP 2750-79		SP 2750-79		SP 2750-79	
1"	min.	max.	100		100	100	100	100		
Mov Avg.	100	100	100		100		100		#VALUE!	
Calc. %			100		100		100			
3/4 in.			100		100	99	100	100		
Mov Avg.	100	100	100		100		100		#VALUE!	
Calc. %			100		100		100			
1/2 in.			95		94	93	94	97		
Mov Avg.	85	100	95		95		94		#VALUE!	
Calc. %			95		94		94			
3/8 in.			84		84	83	87	88		
Mov Avg.	35	90	84		84		84		#VALUE!	
Calc. %			84		84		87			
#4			70		68	68	70	71		
Mov Avg.	30	80	68		68		69		#VALUE!	
Calc. %			70		68		70			
#8			55		54	53	55	56		
Mov Avg.	25	65	54		54		54		#VALUE!	
Calc. %			55		54		55			
# 16			39		39	39	40	40		
#16 Calc. %			39		39		40			
# 30			27		28	28	28	28		
#30 Calc. %			27		28		28			
# 50			16		16	16	16	16		
#50 Calc. %			16		16		16			
# 100			7		8	8	7	8		
#100 Calc. %			7		8		7			
#200			4.2		4.2	4.5	3.9	4.2		
Mov Avg.	2.0	7.0	4.0		4.0		4.0		#VALUE!	
#200 Calc. %			4.2		4.2		3.9			
% Asphalt Content		Individual	5.3		5.2	5.5	5.5	5.8		
Design =	5.4	Mov. Avg.	5.3		5.3		5.3		#VALUE!	
		Calc. %AC	5.3		5.2		5.5			
Gmm - Max.SpG		Individual	2.512		2.521	2.514	2.499	2.505		
(Rice Test)		Mov. Avg.	2.515		2.518		2.512		#VALUE!	
		Calc. Gmm	2.512		2.521		2.499			
Gmb - N-design calc.		Individual	2.404		2.426	2.412	2.406	2.396		
90 Gyrations		Calc. Gmb	2.404		2.426		2.406			
		Isolated	4.3		3.8	4.1	3.7	4.4	#VALUE!	
% Air Voids		Individual	4.4		3.7		4.2		#VALUE!	
Design =	4.0	Mov. Avg.	4.2		4.1		4.1		#VALUE!	
% VMA		Individual	15.9		15.0	15.8	16.0	16.6	#VALUE!	
		Calc. VMA	15.9		15.0		16.0		#VALUE!	
Design =	14.0	Mov. Avg.	15.7		15.6		15.6		#VALUE!	
Fines / Effective Asphalt Content			0.8		0.9	0.9	0.7	0.8		
Adjusted AFT		Individual	9.1		8.6	8.9	9.8	9.5	#VALUE!	
		Working AFT	9.1		8.6		9.8		#VALUE!	
		Mov. Avg.	9.2		9.1		9.2		#VALUE!	
% Add AC	66% Min.	Individual	77.4		76.5		72.4			
/Total AC	70% Min.	Mov. Avg.	76.2		76.2		75.7			
Mix Moisture Content										
% Crushing	CAA -1 Face	85			97	96	95	95		
	CAA -2 Face	80			97	91	95	95		
	FAA	44			45	45	45	45		
Sample Ton Number / Tons Represented			89	522	161	642	162	764		31.60+
Daily Project Total / Cummulative Tons			522	21427	748	22069	764	22833		#VALUE!
"NOTES"	Ignition oven Cf		0.69		0.69		0.69		0.69	
Indivual Failures / Tolerance / Note	Quality Control Actions				71.05 CARRYOVER 34.96=7-6-12 BLEND CHANGE		WARM MIX - COMPACTION TEMP: 250-255 DEG F		31.60 7-16-12	
Source #	Aggregate Source		Agg. SpG.	% of mix	Agg. SpG.	% of mix	Agg. SpG.	% of mix	Agg. SpG.	% of mix
1	BARTON E.R. 9/16 CLR		2.751	27	2.751	28	2.751	28	2.751	28
2	BARTON E.R.WSH MN SND		2.728	27	2.728	26	2.728	26	2.728	26
3	KR WSH SND		2.700	15	2.700	15	2.700	15	2.700	15
4	MILLINGS		2.656	25	2.656	25	2.656	25	2.656	25
5	ERSS		2.639	6	2.639	6	2.639	6	2.639	6
6										
7										
8										
9										
10										
Add AC	TARGET AC		5.3	4.1	5.3	4.0	5.3	4.0		

5.2.3. Field Compaction and Mat Densities of Minnesota Test Sections

The construction of the HMA, and WMA test sections started at 7 pm on the 9th and the 10th of July, 2012 respectively. During compaction, the mix temperature measured using PAVE-IR device was more consistent with the WMA than the HMA.

HMA required 6 passes of breakdown roller (vibratory steel double drum), then pneumatic rubber tire and finish rollers, while WMA mixtures required 4 passes of breakdown roller, then pneumatic and finish rollers to achieve the same required density range measured by PQI non-nuclear device. This shows how the addition of LEADCAP 6-8 additive resulted in more workable mixtures and less compaction effort.

The average density of WMA-LEADCAP 6-8 cores was 93.6%., while the average density of HMA cores was 94.5%. Table 5.13 shows the measured cores' densities of both WMA and HAM mixtures for Minnesota test sections.

Table 5.13 Density Data of HMA and WMA mixtures for Minnesota Test Sections

		Core #	Core Thickness (inches)	Bulk Sp. G. (Gmb)	Density (% of Gmm)
HMA	Contractor	1.1	54.0	2.408	95.6
	Contractor	1.2	52.0	2.427	96.4
	Agency	1.1C	54.0	2.393	95.0
	Agency	1.2C	52.0	2.43	96.5
	Contractor	2.1	45.0	2.392	95.0
	Contractor	2.2	52.0	2.324	92.3
	Agency	2.1C	45.0	2.36	93.7
	Agency	2.2C	52.0	2.295	91.1
WMA	Contractor	3.1	54.0	2.361	94.0
	Contractor	3.2	55.0	2.305	91.8
	Agency	3.1C	54.0	2.362	94.0
	Agency	3.2C	55.0	2.279	90.7
	Contractor	4.1	47.0	2.373	94.5
	Contractor	4.2	56.0	2.372	94.4
	Agency	4.1C	47.0	2.377	94.6
	Agency	4.2C	56.0	2.373	94.5

5.2.4. Performance Evaluation of Minnesota Test Sections

5.2.4.1. Hamburg Wheel Track Test Results

HWT Test results of HMA and WMA mixture are summarized in Table 5.14 and plotted in Figure 5.13. As it can be seen from the test results, both HMA and WMA specimens successfully passed the test with average maximum rut depths of 5.4 mm for HMA specimens and 8.2 mm for WMA specimens. The average SIP were greater than 20,000 passes for HMA specimens and 16333 passes for WMA specimens. It can be concluded that both HMA and WMA mixtures exhibited high resistance to rutting and moisture damage, however the HMA mixture shows higher resistance than the WMA mixture prepared using LEADCAP 6-8.

5.2.4.2. Modified Lottman Test Results

Table 5.15 and Figure 5.14 show the results for the indirect tensile strength values (ITS) with the numbers above the bars representing the average values and the whiskers representing the standard deviation. Overlapping of the standard deviation implies the similarity in the measured ITS between the mixtures types.

As can be seen from the AASHTO T 283 test results, both HMA and WMA mixtures exhibited similar low ITS values in both conditioned and unconditioned samples. However, both HMA and WMA mixtures met the minimum TSR requirement of 80% specified by the MnROAD. WMA mixture exhibited higher moisture sensitivity in the modified Lottman test than HMA mixture.

Table 5.14 Hamburg Wheel Test Results for Minnesota Test Sections.

Mix Type	Test ID	Air Voids %	Total Number of Passes	Inverse Creep Slope (Pass/mm)	Inverse Stripping Slope (Pass/mm)	SIP	Max. Rut Depth, mm
HMA	HMA 1	7.21	20000	6061	N/A	>20000	5.3
	HMA 2	7.88	20000	4878	N/A	>20000	6.1
	HMA 3	7.83	20000	6173	N/A	>20000	4.7
	Average	**7.64**	**20000**	**5704**	**N/A**	**>20000**	**5.4**
WMA-LEADCAP	L 1	6.64	20000	8000	N/A	>20000	5.2
	L 2	7.05	20000	6296	N/A	>20000	9.5
	L 3	6.83	20000	6400	N/A	>20000	9.9
	Average	**6.38**	**20000**	**6899**	**N/A**	**>20000**	**8.2**

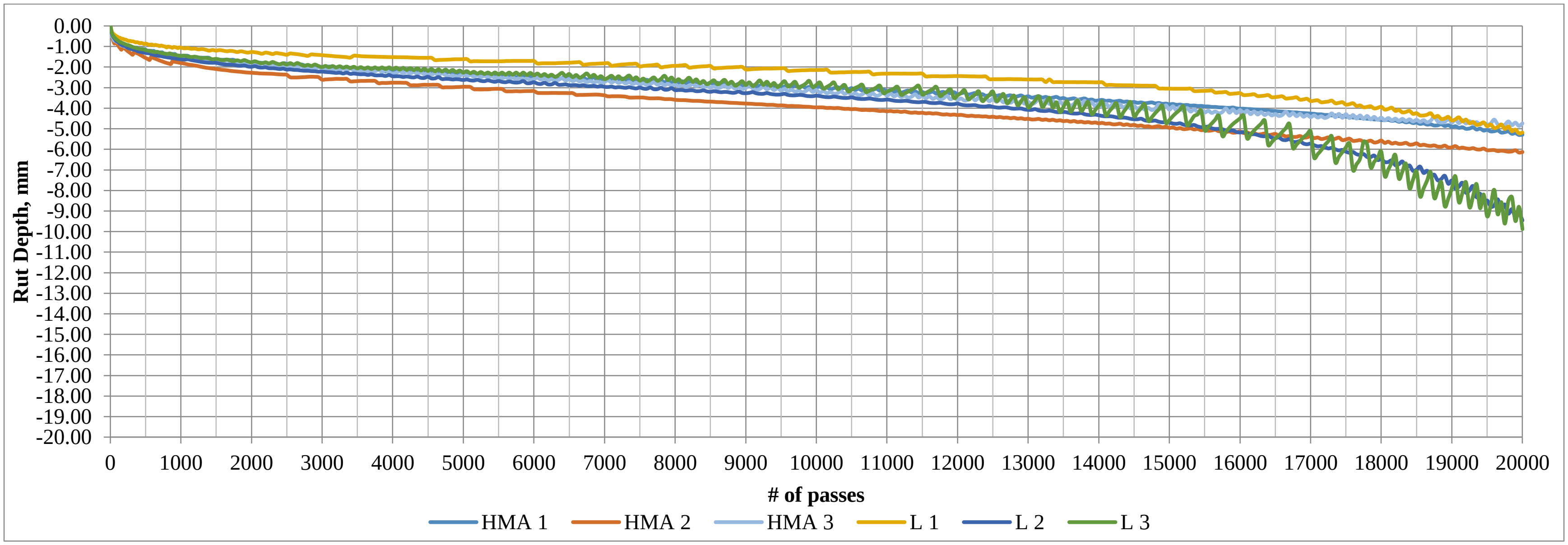

Figure 5.13 Hamburg Wheel Test Results for Minnesota Test Sections.

Table 5.15 AASHTO T 283 Test Results for Minnesota Test Sections.

Mix type	Unconditioned ITS, psi	Conditioned ITS, psi	Average TSR	Standard Deviation	
				Unconditioned ITS	Conditioned ITS
HMA	139	133	95	10.78	2.85
WMA-RAPCAP	130	108	83	4.49	7.12

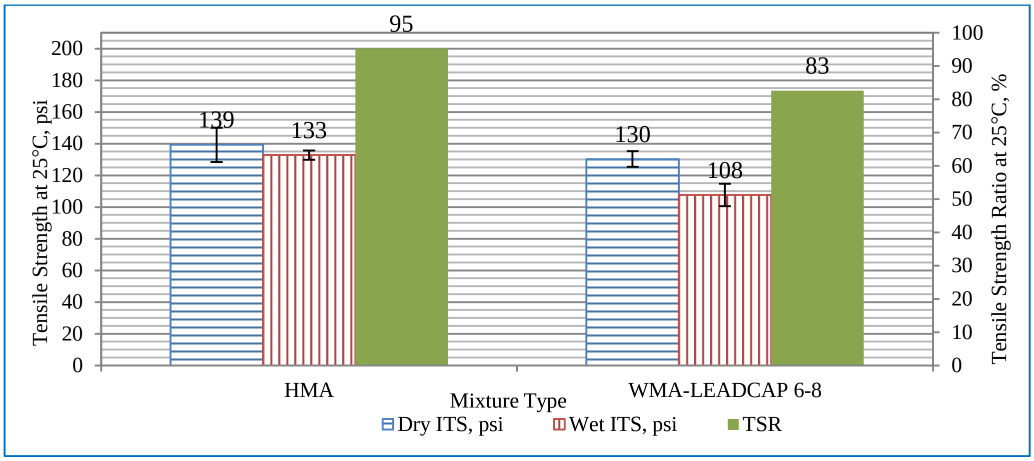

Figure 5.14 AASHTO T 283 Test Results for Minnesota Test Sections.

5.2.5. Assessment of the Minnesota Test Sections after Construction and Over Time

A condition survey was conducted in March 2014 in order to evaluate the WMA mixtures performance after 19 months of service and exposing to a severe winter seasons. The condition survey were conducted and the pavement condition indices (PCIs) were calculated according to ASTM D6433 standards as described earlier in section 3.5. The test sections were constructed on top of an old deteriorated concrete pavement. Both HMA and WMA pavements didn’t perform as good as expected during the severe winter seasons of 2013 and 2014, which resulted in the development of high and medium severities joint reflection cracks, medium severity transverse cracks and medium severity edge cracks. The densities of the joint reflection cracks were 7.3% for high severity and 3.7% for medium severity, respectively. The density of the medium severity transverse cracks was 8.3% and the density of the medium severity edge cracks was 3.7%. The occurrence of these distresses in the pavement surface dropped its condition for both HMA and WMA test sections from “Good” condition with PCIs of 100 right after construction to “Poor” condition with PCIs of approximately 42 after 19 months since construction. Figure 5.15 shows samples of medium and high severities joint reflection cracks, Figure 5.16 shows samples of medium severity transverse cracks, and Figure 5.17 shows samples of medium severity edge cracks from Minnesota test sections.

Figure 5.15 Medium (left) and High (right) Severity Joint Reflection Cracks from Minnesota Test Sections.

Figure 5.16 Medium Severity Transverse Cracks from Minnesota Test Sections.

Figure 5.17 Medium Severity Edge Cracks from Minnesota Test Sections.

5.3. State Highway 158 in Lancaster, Ohio

Two test sections, one for WMA and one for HMA as a control mix, were constructed on State Highway 158 at the milepost of 75.5 in Lancaster, Ohio. The WMA test section has a total length of 1 mile. The HMA mixtures were applied at the section adjacent to the WMA test section at mile post 85.5. A 3.0-inch asphalt layer was constructed that consisted of two layers; the intermediate layer with a thickness of 1.75 inch and the surface layer with a thickness of 1.25 inch.

The two mixtures designed for this research study were: HMA as a control mixture and WMA mixture prepared using LEADCAP 7-1 additive. The LEADCAP 7-1 additive were added to the mix with a rate of 1.5% by optimum asphalt content. The mixtures were designed according to Marshall mix design procedure following Ohio Department of Transportation (ODOT) mix design specifications and NCHRP 691 report “Mix Design Practices for Warm Mix Asphalt”. The mixtures used a blend of Limestone and Natural gravel aggregates and 25% RAP materials by weight for the intermediate layer and 20 % RAP by weight for the surface layer. Only surface layer mixtures were evaluated in this research study.

5.3.1. Virgin Aggregate, RAP Materials and Asphalt Binder Properties of Ohio Test Sections

The aggregate gradation for the surface layer was designed with 1/2 inch nominal maximum size using 4 different stockpiles collected from the Shelly Company Quarry in Lancaster, Ohio. All virgin aggregate properties met ODOT mix design requirements. The combined aggregate gradation is summarized in Table 5.16 and plotted on a 0.45 power gradation chart in Figure 5.18.

Table 5.16 Aggregate Gradation and Properties for Ohio Surface Layer Mixtures (Source: ODOT).

Sieve Size		Stockpile and percentage passing					Design Spec, NMAS 1/2 inch	
ID	mm	Limestone /Gravel	Natural Sand	Limestone Sand	RAP	Combined Gradation	Min.	Max.
1 in	25	100.0	100.0	100.0	100.0	100	100	100
3/4 in	19	100.0	100.0	100.0	100.0	100	100	100
1/2 in	12.5	100.0	100.0	100.0	98.0	100	100	100
3/8 in	9.5	92.0	100.0	100.0	89.5	94	90	100
#4	4.75	20.0	99.0	94.0	65.3	55	45	57
#8	2.36	3.0	87.0	66.0	49.0	38	30	45
#16	1.18	3.0	68.0	40.0	35.8	28	17	35
#30	0.6	3.0	47.0	26.0	26.8	20	12	15
#50	0.3	3.0	16.0	17.0	16.8	10	5	18
#100	0.15	2.0	4.0	10.0	12.0	5	2	10
#200	0.075	2.0	3.0	5.7	9.3	4		
% of Each stockpile		47.0%	23.0%	10.0%	20.0%	100.0%	Check Total	

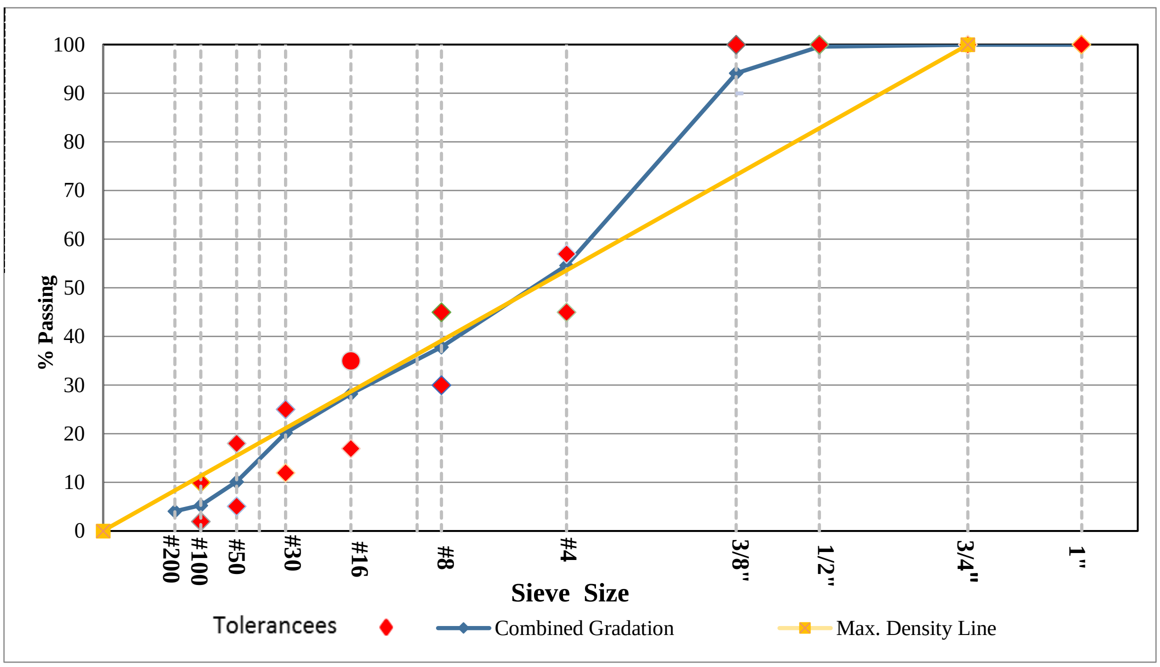

Figure 5.18 Combined Aggregate Gradation Chart for Ohio Surface Layer Mixtures (Source: ODOT).

The asphalt binder used for the surface layer of HMA, and WMA mixtures was PG 70-22M. The asphalt binder properties were evaluated according to AASHTO M320 standard. The results were provided by The Shelly Company. The selected asphalt binder met all specifications as shown in Table 5.17. Asphalt mixing temperature for the PG 70-22M was determined as 160° C (320° F), as recommended by the binder supplier company.

5.3.2. Mix Design Summary of Ohio Test Sections

Mixing and compaction temperatures for the two mixtures were established based on ODOT mix design recommendations. Mix design summary is included in the project reference data sheet. The project reference data sheets for HMA and WMA mixtures of the surface layer are shown in Figure 5.19, and Figure 5.20 respectively.

Table 5.17 Asphalt Binder PG70-22M Test Results (Source: The Shelly Co.).

Quality Control Laboratory					
Date Entered	**Binder Test Results**				**File**
17-Sep-13					603
Binder ID	**Sample Date**	**Date Received**	**Liquid Division Tank**	**Gallons**	**ODOT Sample**
1694	16-Sep-13	16-Sep-13		0	
Project	**Plant**	**Plant Tank #**	**Barge #**	**Sampler**	
293-13	0 62				
Binder Type	**Blend**	**Date tested**	**Tester**		
PG 70-22	1.8% LEADCAP	16-Sep-13			
Original Binder					
Flash point	**Viscosity 135 C CP**	**Dynamic Shear C**	**Original Binder phase Angle**		
0	0	1.61	69		
RTFO Binder					
Mass loss %		**Dynamic Shear C**	**RTFO Binder phase Angle**		
0		3.84	66.1		
PAV Binder					
Dynamic Shear C	**M-Value**	**Stiffness (S)**	**PAV Residue phase Angle**		
1930	0.328	137	49.9		
Additional Tests					
Specific Gravity	**Ductility**	**Force Ratio**	**Penetration**	**Absolute Viscosity**	**Solubility**
0	0	0	68	0	0
Electric Recovery					
0					
Summary					
Pass/Fail temperature			**Binder is A**		
0			PG 70-22		
Comments	**BINDER -TOLEDO (1.8% LEADCAP ADDITIVE)**				
	All required Tests Completed			√	

Project Reference Data Sheet

Producer:	Shelly Materials Plant #62	**Date Submitted:**	
Project:	0293-13 ODOT	**Mix Type:**	T-1-SUR
Spec:	441 **Job #** 830293	**Traffic Volume:**	MEDIUM
Lab No:	20130130 **White Card #**	**JMF:**	B130946
Item Code:	448E46904 ASPHALT CONCRETE SURFACE COURSE, TYPE 1, PG70-22M	**REF:**	015

JMF

SIEVE	% Passing
2":	
1 1/2":	
1":	
3/4":	
1/2":	100
3/8":	94
NO. 4:	54
NO. 8:	38
NO. 16:	28
NO. 30:	20
NO. 50:	10
NO. 100:	5
NO. 200:	4.1

AC CONTENT DETERMINATION

AC Content @ Optimum	6.2
For RAP Design	5.1 **Virgin:** 70-22M
AC in RAP	5.3
Mixing Temp	315
Compaction Temp	295
Conversion Factor	1.952
Max Theo @ Opt	2.399
AC Grade Required by Proposal:	70-22M
Binder Source:	Shelly Liquid - Toledo

F/A Ratio: 0.7 **F/T Value:** 2 **Loaded Wheel Test:** 0

Crushed %: 0.00% **TSR:** 82.9 **Ignition Oven Offset:** 0

	%	SIZE	TYPE	PRODUCER / LOCATION	SPG	CODE
#1 Coarse Agg	47%	#8	GR/LS	Shelly Mtls- (Blended) Columbus, Oh	2.561	04502B-01 8
#1 Fine Agg	23%	SAND	NAT	Mar Zane Logan, Oh	2.56	04959-01 051SD5
#2 Fine Agg	10%	SAND	LS	Shelly Materials Columbus, OH	2.61	4502-01 055SD5
#1 Recycle	20%	RECYCLE	RAP	Composite 13 Columbus, Ohio	2.647	Comp 13 0010137

Gsb: 2.582

Ref #	Item	QTY	UM	JMF	CF	Plt#	PG	Source	% RAP	Whitecard #
015	448 T1 SURFACE	8,294	CY	B130946	1.952	062	70-22M	Shelly Liquid - Toledo	20	

Item Code: 448E46904 Description ASPHALT CONCRETE SURFACE COURSE, TYPE 1, PG70-22M UM: CU YD

Inches 0

Figure 5.19 Project Reference Data Sheet for HMA Mixtures of the Surface Layer (Source: ODOT).

Project Reference Data Sheet

Producer:	Shelly Materials Plant #62			**Date Submitted:**	
Project:	0293-13	ODOT		**Mix Type:**	T-1 LDCAP
Spec:	441	**Job #**	830293	**Traffic Volume:**	MEDIUM
Lab No:	20130133	**White Card #**	41061	**JMF:**	W130959
Item Code:	448E47020	ASPHALT CONCRETE SURFACE COURSE, TYPE 1, PG64-22		**REF:**	017

JMF

SIEVE	% Passing
2":	
1 1/2":	
1":	
3/4":	
1/2":	100
3/8":	94
NO. 4:	54
NO. 8:	38
NO. 16:	28
NO. 30:	20
NO. 50:	10
NO. 100:	5
NO. 200:	4.1

AC CONTENT DETERMINATION

AC Content @ Optimum	6.2
For RAP Design	5.1 **Virgin:** 70-22M
AC in RAP	5.3
Mixing Temp	315
Compaction Temp	257
Conversion Factor	1.958
Max Theo @ Opt	2.405
AC Grade Required by Proposal:	70-22M
Binder Source:	Shelly Liquid Division

F/A Ratio: 2 **F/T Value:** .7 **Loaded Wheel Test:** 0

Crushed %: 0.00% **TSR:** 85.9 **Ignition Oven Offset:** 0

	%	SIZE	TYPE	PRODUCER / LOCATION	SPG	CODE
#1 Coarse Agg	47%	#8	GR/LS	Shelly Mtls- (Blended) Columbus, Oh	2.561	04502B-01 8
#1 Fine Agg	23%	SAND	NAT	Mar Zane Logan, Oh	2.56	04959-01 051SD5
#2 Fine Agg	10%	SAND	LS	Shelly Materials Columbus, OH	2.61	4502-01 055SD5
#1 Recycle	20%	RECYCLE	RAP	Composite 13 Columbus, Ohio	2.647	Comp 13 0010137

Gsb: 2.582

Ref #	Item	QTY	UM	JMF	CF	Plt#	PG	Source	% RAP	Whitecard #
017	448 T1 SURFACE	275	CY	B130928	1.956	062	64-22	Shelly Liquid Division	20	41061

Item Code: 448E47020 Description ASPHALT CONCRETE SURFACE COURSE, TYPE 1, PG64-22 UM: 008

Inches 0

Figure 5.20 Project Reference Data Sheet for WMA Mixtures of the Surface Layer (Source: ODOT).

5.3.3. Field Compaction and Mat Densities of Ohio Test Sections

The construction of the HMA, and WMA test sections started at 7 am on the 14th, and the 16th of September, 2013 respectively. The WMA mixtures showed better workability during construction, which resulted in less compaction effort than the compaction effort needed for HMA mixture. The target field air voids during compaction was 6.0%. The nuclear gauge method was used to measure the mat densities during construction. Three cores were extracted and used to calibrate the nuclear gauge. Four to Six different locations per mile were selected to measure the densities of the asphalt mat. The average densities of the HMA, and WMA test sections were 94.6%, and 95.2% respectively. Table 5.18 shows the density data for HMA and WMA test sections.

Table 5.18 Density Data of HMA and WMA Test Sections for Surface Layer (ODOT).

9/16/13/ LEADCAP	Gauge Reading (Contractor QC)		
Longitudinal Location	Transverse Location	Actual Gauge Reading, pcf	% Density
336100 NB	L	143.2	95.7
326100 NB	C	144.3	96.5
316100 NB	R	140.9	94.2
306100 NB	L	140.1	93.7
296100 NB	C	143.5	95.9
286100 NB	R	142	94.9
ODOT QA Tests	Pcfs L C R Ave	% Density	ODOT Initials
326100 NB	139.2 144.3 140.9 141.5	94.6	
306100 NB	141.7 144.4 141.6 142.6	95.3	
9/14/13/ HMA	Gauge Reading (Contractor QC)		
Longitudinal Location	Transverse Location	Actual Gauge Reading, pcf	% Density
376100 NB	L	141.8	95.6
366100 NB	C	142.6	94.1
356100 NB	R	140.9	95
346100 NB	L	141.8	93.6
ODOT QA Tests	Pcfs L C R Ave	% Density	ODOT Initials
356100 NB	144.7 149.1 140.9 144.9	95.6	
346100 NB	141.8 144.7 141.7 142.7	94.2	

5.3.4. Performance Evaluation

5.3.4.1. Hamburg Wheel Track Test Results of Ohio Test Sections

HWT Test results of HMA and WMA mixture are summarized in Table 5.19 and plotted in Figure 5.21. As it can be seen from the test results, both HMA and WMA specimens successfully passed the test with average maximum rut depths of 3.2 mm for HMA specimens and 8.2 mm for WMA specimens. The average SIP were greater than 20,000 passes for HMA specimens and 12083 passes for WMA specimens. It can be concluded that both HMA and WMA mixtures exhibited high resistance to rutting and moisture damage, however the HMA mixture shows higher resistance than the WMA mixture prepared using LEADCAP 7-1.

5.3.4.2. Modified Lottman Test Results

Table 5.20 and Figure 5.22 show the results for the indirect tensile strength values (ITS) with the numbers above the bars representing the average values and the whiskers representing the standard deviation. Overlapping of the standard deviation implies the similarity in the measured ITS between the mixtures types.

As can be seen from the AASHTO T 283 test results, WMA and HMA mixtures exhibited similar ITS values in both conditioned and unconditioned samples. HMA mixtures exhibited significantly higher ITS values than those obtained for WMA mixtures, which showed less sensitivity to moisture damage than what WMA mixture showed. A closer look at the WMA and HMA conditioned samples after the test, we can see the broken aggregate particles with a small amount of stripping in case of HMA samples, and less broken aggregate with higher amount of striping in case of WMA samples, as shown in Figure 5.23. This can be attributed to the softening effect of the LEADCAP 7-1 additive even with the existence of RAP in the mix, unlike the HMA samples which showed higher stiffness due to the RAP existence in the mix. However, both HMA and WMA mixtures met the minimum TSR requirement of 80% specified by the ODOT.

Table 5.19 Hamburg Wheel Test Results for Ohio Test Sections.

Mix Type	Test ID	Air Voids %	Total Number of Passes	Inverse Creep Slope (Pass/mm)	Inverse Stripping Slope (Pass/mm)	SIP	Max. Rut Depth, mm
HMA	HMA 1	6.79	20000	11173	N/A	>20000	3.4
	HMA 2	6.80	20000	14184	N/A	>20000	2.9
	HMA 3	6.71	20000	10471	N/A	>20000	3.2
	Average	**6.77**	**20000**	**11943**	**N/A**	**>20000**	**3.2**
WMA-LEADCAP	L 1	6.53	20000	4141	2395	13250	7.8
	L 2	6.43	20000	3194	3295	11500	8.1
	L 3	6.61	20000	3286	2566	11500	8.8
	Average	**6.52**	**20000**	**3540**	**2763**	**12083**	**8.2**

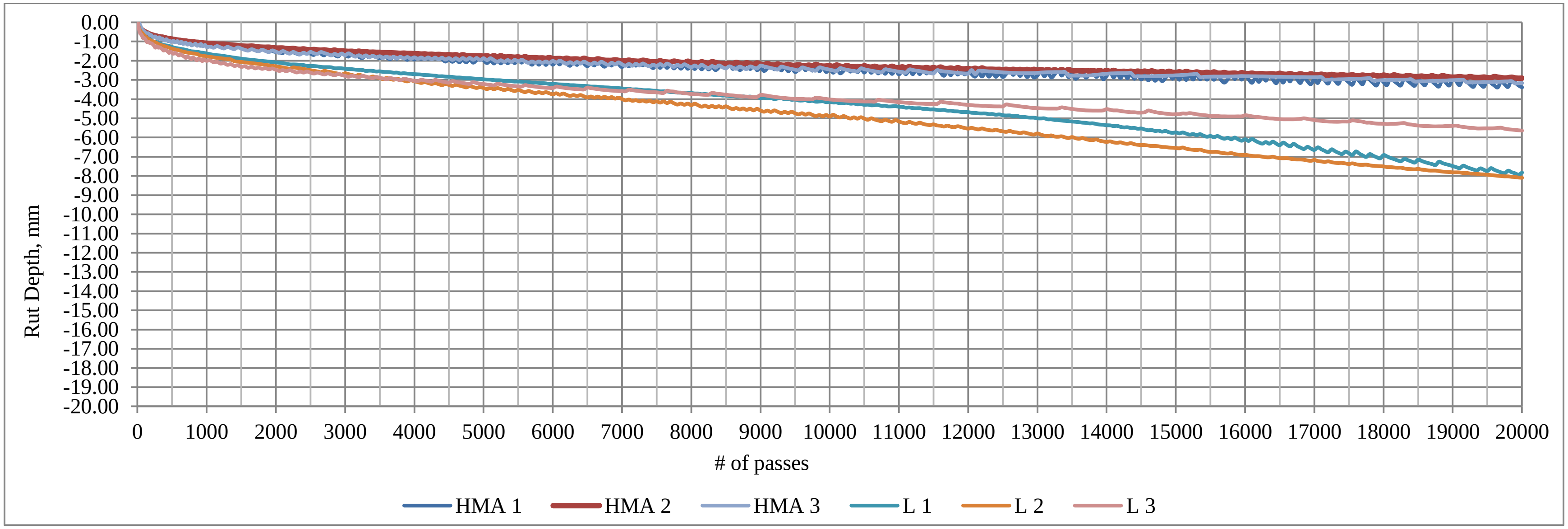

Figure 5.21 Hamburg Wheel Test Results for Ohio Test Sections.

Table 5.20 AASHTO T 283 Test Results for Ohio Test Sections.

Mix type	Unconditioned ITS, psi	Conditioned ITS, psi	Average TSR	Standard Deviation	
				Unconditioned ITS	Conditioned ITS
HMA	187	180	96	5.90	8.11
WMA-LEADCAP	149	151	101	6.49	5.06

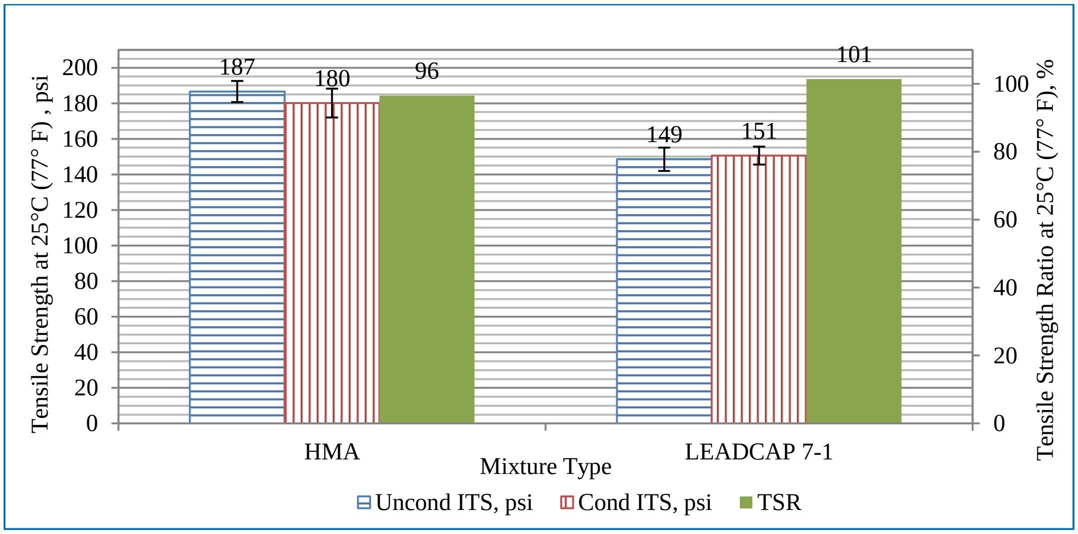

Figure 5.22 AASHTO T 283 Test Results for Ohio Test Sections.

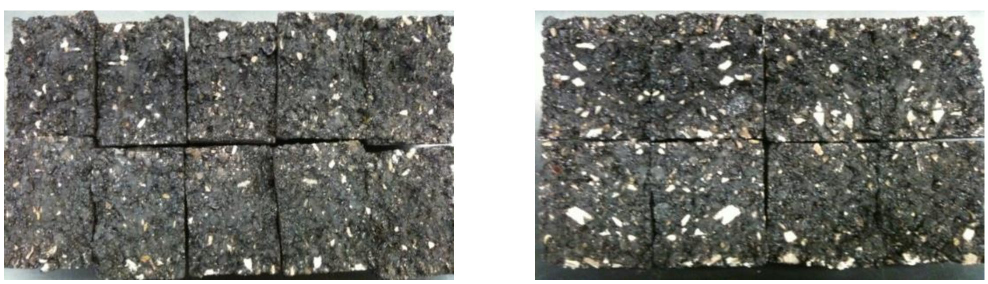

Figure 5.23 HMA (Left) and WMA-LEADCAP (Right) Conditioned Sets of Ohio Mixtures.

5.3.5. Assessment of the Ohio Test Sections after Construction and Over Time

A condition survey was conducted in April 2014 in order to evaluate the WMA mixtures performance after 6 months of service and exposing to a severe winter seasons. The condition survey were conducted and the pavement condition indices (PCIs) were calculated according to ASTM D6433 standards as described earlier in section 3.5. Both HMA and WMA_LEDCAP test sections performed very well during the severe winter season of 2014. No signs of distresses were observed during the condition survey. The pavement condition indices (PCIs) of both HMA and WMA_LEADCAP kept the same values of 100 after 6 months of construction. Figure 5.24 shows sample pictures for the pavement condition of Ohio test sections.

Figure 5.24 Sample Pictures for the Pavement Condition of Ohio Test Sections.

5.4. Asphalt Bond Strength (ABS) Evaluation of Extracted Asphalt Binders

Asphalt bond strength (ABS) was improved by using a 0.0 mm pullout stub when running the test as shown in section 4.3. Thus, the 0.0 mm pullout stubs were used to conduct the ABS evaluation for the extracted asphalt binders from all the HMA and WMA mixtures used for the test sections. The extraction process was done per ASTM D2172/D2172M-11 "Standard Methods for Quantitative Extraction of Bitumen from Bituminous Paving Mixtures", and ASTM D5404/D540M-12 "Standard Practice for Recovery of Asphalt from Solution Using the Rotary Evaporator". After the extraction and recovery of asphalt binders were done, a full performance grading were done following AASHTO M 320 “Standard Specification for Performance-Graded Asphalt Binder” and AASHTO MP19-10 “ Performance Graded Asphalt Binder Using Multiple Stress Creep Recovery (MSCR) Test”.

5.4.1. Rheological Properties and Performance Grades of the Extracted and Recovered Asphalt Binders

Tables 5.21 through 5.26 show summaries of the performance grading results for the different extracted and recovered asphalt binders.

The target performance grade of the asphalt binder for Iowa mixtures was 70-22 with 30% RAP by asphalt binder’s replacement, which required using a lower grade of PG 64-28 to produce the mix. The extracted asphalt binder from the HMA mixtures of Iowa test section was graded as PG 82-16 with a standard traffic “S” level, while the extracted asphalt binder from the WMA mixtures produced using RAPCAP was graded as PG 70-28 with a heavy traffic “H” level. The RAPCAP additive reduced the impact of aging on the asphalt binder during construction. The RAPCAP additive improved the high and low temperature grades of the extracted asphalt binder. Also, the RAPCAP additive improved the asphalt binder characteristics during the MSCR test by decreasing the non-recoverable creep compliance, which resulted in a higher qualified asphalt binder for a higher traffic level “H” instead of a standard traffic level “S”, which was obtained for the extracted asphalt PG 82-16 from the HMA mixture.

The target asphalt binder performance grade for Minnesota mixtures was 70-22 with 25% RAP by total weight of mix, which required using a lower grade of PG 64-28 to produce the mix. The extracted asphalt binder from the HMA and WMA mixtures of

Minnesota test sections were both graded as PG 76-22 with a standard traffic “S” level.

Similarly, the target asphalt binder performance grade for Ohio mixtures was 70-22 with 20% RAP by total weight of mix, which didn’t require lowering the performance grade of asphalt when producing the mix. The extracted asphalt binder from the HMA and WMA mixtures of Ohio test sections were both graded as PG 82-16 with a standard traffic “S” level.

It was expected that the LEADCAP additive would improve the performance grade of the recovered asphalt binder however, this goal was not achieved. This can be attributed to the existence of RAP materials in the asphalt mixtures as the LEADCAP additive was not originally created to improve the aged asphalt binder in the RAP materials.

Table 5.21 Rheological Properties of the Extracted Asphalt Binder from the Iowa HMA Test Section

Property	Test Results	Specification	Test Method
Tests on Original Binder			
Flash Point, °C	NT	230 Min	AASHTO T48
Solubility	NT	99% Min	AASHTO T44
Rotational viscosity at 135°C, Pa-s	NT	3.0 Max	AASHTO T316
Dynamic shear, G*/sin at 82°C (10 rad/s), kPa	1.363	1.00 Min	AASHTO T315
Tests on Residue from RTFO, AASHTO T240			
Mass loss, %	NT	1.00% Max	AASHTO T240
Dynamic shear, G*/sin at 82°C (10 rad/s), kPa	2.3255	2.20 Min	AASHTO T315
Multiple Stress Creep and Recovery (MSCR) of Asphalt Binder at 82°C			AASHTO TP 70-11
Non-recoverable creep compliance, $J_{nr3.2}$, kpa^{-1}	3.95	Max, 4.0 kpa^{-1}	Standard Traffic "S"
Percent difference between $J_{nr3.2}$ and $J_{nr0.1}$, J_{nrdiff}, %	39.7	Max, 75%	
Tests on Residue from Pressure Aging Vessel, AASHTO R28 @ 100°C			
Dynamic shear, G*sin at 28°C (10 rad/s), kPa	3675.5	5000 Max	AASHTO T315
Creep Stiffness at -6°C			AASHTO T313
S-value, MPa	80.95	300 Max	
m-value	0.367	0.300 Min	
The Performance Grade of this Asphalt Binder is **PG 82-16**			

Table 5.22 Rheological Properties of the Extracted Asphalt Binder from the Iowa WMA Test Section

Property	Test Results	Specification	Test Method
Tests on Original Binder			
Flash Point, °C	NT	230 Min	AASHTO T48
Solubility	NT	99% Min	AASHTO T44
Rotational viscosity at 135°C, Pa-s	NT	3.0 Max	AASHTO T316
Dynamic shear, G*/sin at 70°C (10 rad/s), kPa	1.12	1.00 Min	AASHTO T315
Tests on Residue from RTFO, AASHTO T240			
Mass loss, %	NT	1.00% Max	AASHTO T240
Dynamic shear, G*/sin at 70°C (10 rad/s), kPa	2.6875	2.20 Min	AASHTO T315
Multiple Stress Creep and Recovery (MSCR) of Asphalt Binder at 70°C			AASHTO TP 70-11
Non-recoverable creep compliance, $J_{nr3.2}$, kpa^{-1}	1.815	Max, 2.0 kpa^{-1}	Heavy Traffic "H"
Percent difference between $J_{nr3.2}$ and $J_{nr0.1}$, J_{nrdiff}, %	33.7	Max, 75%	
Tests on Residue from Pressure Aging Vessel, AASHTO R28 @ 100°C			
Dynamic shear, G*sin at 22°C (10 rad/s), kPa	4726.5	5000 Max	AASHTO T315
Creep Stiffness at -18°C			AASHTO T313
S-value, MPa	226	300 Max	
m-value	0.3125	0.300 Min	
The Performance Grade of this Asphalt Binder is **PG 70-28**			

Table 5.23 Rheological Properties of the Extracted Asphalt Binder from the Minnesota HMA Test Section

Property	Test Results	Specification	Test Method
Tests on Original Binder			
Flash Point, °C	NT	230 Min	AASHTO T48
Solubility	NT	99% Min	AASHTO T44
Rotational viscosity at 135°C, Pa-s	NT	3.0 Max	AASHTO T316
Dynamic shear, G*/sinδ at 76°C (10 rad/s), kPa	1.914	1.00 Min	AASHTO T315
Tests on Residue from RTFO, AASHTO T240			
Mass loss, %	NT	1.00% Max	AASHTO T240
Dynamic shear, G*/sinδ at 76°C (10 rad/s), kPa	3.26	2.20 Min	AASHTO T315
Multiple Stress Creep and Recovery (MSCR) of Asphalt Binder at 76°C			AASHTO TP 70-11
Non-recoverable creep compliance, $J_{nr3.2}$, kpa^{-1}	2.755	Max, 4.0 kpa^{-1}	Standard Traffic "S"
Percent difference between $J_{nr3.2}$ and $J_{nr0.1}$, J_{nrdiff}, %	34.65	Max, 75%	
Tests on Residue from Pressure Aging Vessel, AASHTO R28 @ 100°C			
Dynamic shear, G*sinδ at 25°C (10 rad/s), kPa	3929.5	5000 Max	AASHTO T315
Creep Stiffness at -12°C			AASHTO T313
S-value, MPa	161	300 Max	
m-value	0.3165	0.300 Min	
The Performance Grade of this Asphalt Binder is **PG 76-22**			

Table 5.24 Rheological Properties of the Extracted Asphalt Binder from the Minnesota WMA Test Section

Property	Test Results	Specification	Test Method
Tests on Original Binder			
Flash Point, °C	NT	230 Min	AASHTO T48
Solubility	NT	99% Min	AASHTO T44
Rotational viscosity at 135°C, Pa-s	NT	3.0 Max	AASHTO T316
Dynamic shear, G*/sinδ at 76°C (10 rad/s), kPa	1.78	1.00 Min	AASHTO T315
Tests on Residue from RTFO, AASHTO T240			
Mass loss, %	NT	1.00% Max	AASHTO T240
Dynamic shear, G*/sinδ at 76°C (10 rad/s), kPa	2.30426	2.20 Min	AASHTO T315
Multiple Stress Creep and Recovery (MSCR) of Asphalt Binder at 76°C			AASHTO TP 70-11
Non-recoverable creep compliance, $J_{nr3.2}$, kpa^{-1}	3.405	Max, 4.0 kpa^{-1}	Standard Traffic "S"
Percent difference between $J_{nr3.2}$ and $J_{nr0.1}$, J_{nrdiff}, %	25.85	Max, 75%	
Tests on Residue from Pressure Aging Vessel, AASHTO R28 @ 100°C			
Dynamic shear, G*sinδ at 28°C (10 rad/s), kPa	3548.5	5000 Max	AASHTO T315
Creep Stiffness at -12°C			AASHTO T313
S-value, MPa	188	300 Max	
m-value	0.3055	0.300 Min	
The Performance Grade of this Asphalt Binder is **PG 76-22**			

Table 5.25 Rheological Properties of the Extracted Asphalt Binder from the Ohio HMA Test Section

Property	Test Results	Specification	Test Method
Tests on Original Binder			
Flash Point, °C	NT	230 Min	AASHTO T48
Solubility	NT	99% Min	AASHTO T44
Rotational viscosity at 135°C, Pa-s	NT	3.0 Max	AASHTO T316
Dynamic shear, G*/sinδ at 82°C (10 rad/s), kPa	2.2219	1.00 Min	AASHTO T315
Tests on Residue from RTFO, AASHTO T240			
Mass loss, %	NT	1.00% Max	AASHTO T240
Dynamic shear, G*/sinδ at 82°C (10 rad/s), kPa	3.3917	2.20 Min	AASHTO T315
Multiple Stress Creep and Recovery (MSCR) of Asphalt Binder at 82°C			AASHTO TP 70-11
Non-recoverable creep compliance, $J_{nr3.2}$, kpa^{-1}	2.87	Max, 4.0 kpa^{-1}	Standard Traffic "S"
Percent difference between $J_{nr3.2}$ and $J_{nr0.1}$, J_{nrdiff}, %	70.4	Max, 75%	
Tests on Residue from Pressure Aging Vessel, AASHTO R28 @ 100°C			
Dynamic shear, G*sin at 28°C (10 rad/s), kPa	4399.5	5000 Max	AASHTO T315
Creep Stiffness at -6°C			AASHTO T313
S-value, MPa	119	300 Max	
m-value	0.372	0.300 Min	
The Performance Grade of this Asphalt Binder is **PG 82-16**			

Table 5.26 Rheological Properties of the Extracted Asphalt Binder from the Ohio WMA Test Section

Property	Test Results	Specification	Test Method
Tests on Original Binder			
Flash Point, °C	NT	230 Min	AASHTO T48
Solubility	NT	99% Min	AASHTO T44
Rotational viscosity at 135°C, Pa-s	NT	3.0 Max	AASHTO T316
Dynamic shear, G*/sinδ at 82°C (10 rad/s), kPa	2.43695	1.00 Min	AASHTO T315
Tests on Residue from RTFO, AASHTO T240			
Mass loss, %	NT	1.00% Max	AASHTO T240
Dynamic shear, G*/sinδ at 82°C (10 rad/s), kPa	3.06645	2.20 Min	AASHTO T315
Multiple Stress Creep and Recovery (MSCR) of Asphalt Binder at 82°C			AASHTO TP 70-11
Non-recoverable creep compliance, $J_{nr3.2}$, kpa^{-1}	3.745	Max, 4.0 kpa^{-1}	Standard Traffic "S"
Percent difference between $J_{nr3.2}$ and $J_{nr0.1}$, J_{nrdiff}, %	62.85	Max, 75%	
Tests on Residue from Pressure Aging Vessel, AASHTO R28 @ 100°C			
Dynamic shear, G*sinδ at 28°C (10 rad/s), kPa	4330	5000 Max	AASHTO T315
Creep Stiffness at -6°C			AASHTO T313
S-value, MPa	109	300 Max	
m-value	0.3835	0.300 Min	
The Performance Grade of this Asphalt Binder is **PG 82-16**			

5.4.2. Asphalt Bond Strength (ABS) Test Results

As described earlier, the test was done on all the extracted asphalt binders from all HMA and WMA mixtures used for test sections. Both unconditioned and moisture conditioned cases were considered during the ABS test. The ABS test results are shown in Table 5.27 and plotted in Figure 5.25 with the numbers above the bars representing the average values and the whiskers representing the standard deviation. Overlapping of the standard deviation implies the similarity in the measured ABS between the asphalt types.

Table 5.27 ABS Test Results of Extracted Asphalt Binders.

Mix Type/Asphalt Grade	Unconditioned Samples			Conditioned Samples		
	Average, psi	St. Dev.	Failure Mode	Average, psi	St. Dev.	Failure Mode
Iowa_HMA/PG82-16	490.0	50.3	76% Cohesion	269.7	38.7	80% Adhesion
Iowa_RAPCAP/PG70-28	519.7	12.6	76% Cohesion	322.3	5.5	65% Cohesion
Minnesota _HMA/PG76-22	488.7	52.3	53% Cohesion	426.0	32.2	85% Cohesion
Minnesota _LEADCAP/PG76-22	490.7	41.0	73% Cohesion	391.3	22.0	73% Cohesion
Ohio_HMA/PG82-16	567.0	16.5	76% Cohesion	200.7	12.5	77% Adhesion
Ohio_LEADCAP/PG82-16	475.3	14.5	76% Cohesion	192.0	12.2	78% Adhesion

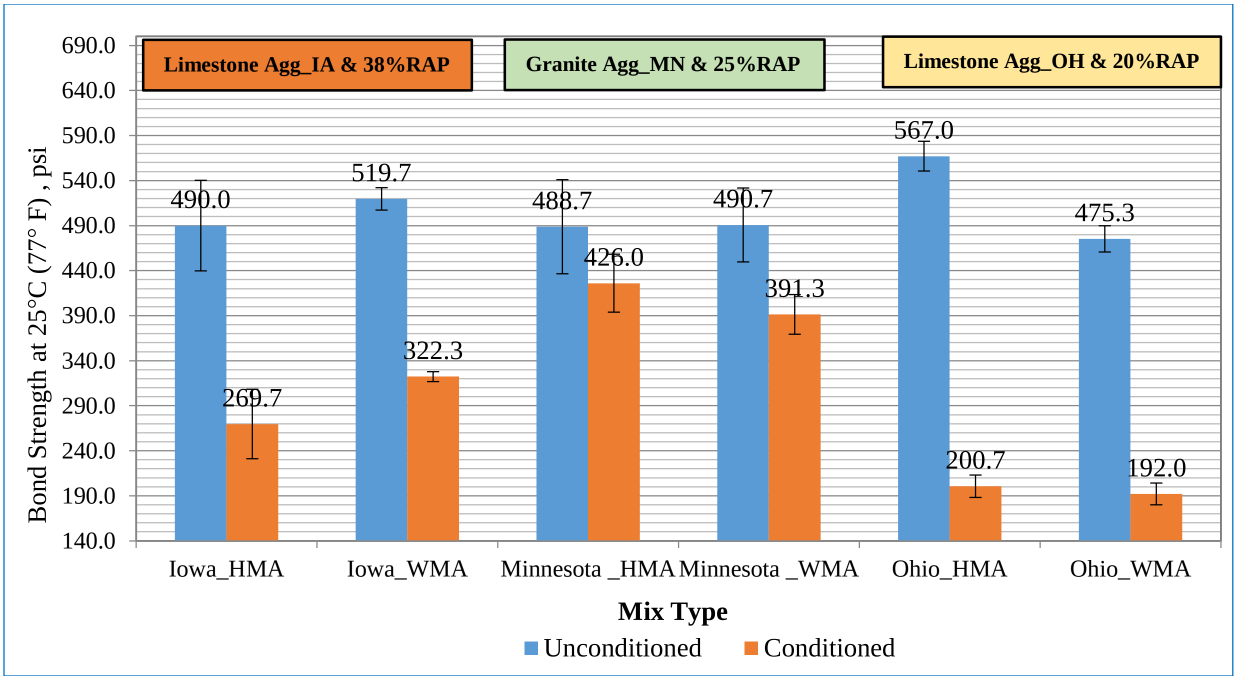

Figure 5.25 ABS Test Results of Extracted Asphalt Binders.

As can be seen from the ABS test results, the following observations can be made;

Iowa Test Sections

Both HMA and WMA samples exhibited similar unconditioned ABS values and significantly lower conditioned ABS values than the unconditioned ABS values. However the WMA samples exhibited significantly higher conditioned ABS values than those obtained for HMA samples. Both HMA and WMA mixtures are susceptible to moisture damage, however the WMA shows less susceptibility to moisture damage, which can be attributed to the RAPCAP additive ability to improve the non-recoverable creep compliance and the percent recovery of the recovered asphalt binder.

Minnesota Test Sections

Both HMA and WMA samples exhibited similar unconditioned ABS values and slightly lower conditioned ABS values comparing to those obtained for unconditioned ABS values. WMA samples exhibited slightly lower conditioned ABS values than those obtained for the HMA samples, however both HMA and WMA samples from Minnesota showed higher conditioned ABS values than Iowa and Ohio test sections samples, which shows their high resistance to moisture damage during the ABS test.

Ohio Project Test Sections

HMA samples from Ohio exhibited the highest unconditioned ABS values among all other samples from other test sections. Also, HMA samples exhibited significantly higher unconditioned ABS values than those obtained for WMA samples. Both HMA and WMA samples exhibited similar conditioned ABS values and significantly lower than unconditioned ABS values. The conditioned ABS values obtained from OHP for both HMA and WMA samples were the least ABS values among all other samples from other test sections, which shows their very low resistance to moisture damage during the ABS test.

All test results obtained for each test section are summarized in Table 5.28

Table 5.28 Summary of All Test Results obtained for Each Test Section

Test Results		Iowa Test Sections		Minnesota Test Sections		Ohio Test Sections	
		RAPCAP	HMA	LEADCAP 6-8	HMA	LEADCAP 7-1	HMA
Field Density, %		93.9	94.3	93.6	94.5	94.6	95.2
HWT	Total Rut Depth, mm	4.5	2.6	8.2	5.4	8.8	3.2
	SIP, Passes	>20000	>20000	>20000	>20000	>20000	>20000
Modified Lottman Test	Unconditioned ITS, psi	168	208	130	139	149	187
	Conditioned ITS, psi	142	200	108	133	151	180
	TSR, %	84	96	83	95	101	96
Asphalt Bond Strength (ABS)	Unconditioned ABS, psi	519.7	490	490.7	488.7	475.3	567
	Unconditioned Mode of Failure, %	76% Cohesion	76% Cohesion	73% Cohesion	53% Cohesion	76% Cohesion	76% Cohesion
	Conditioned ABS, psi	322.3	269.7	391.3	426	192	200.7
	Conditioned Mode of Failure, %	65% Cohesion	80% Adhesion	73% Cohesion	85% Cohesion	78% Adhesion	77% Adhesion
Extracted Asphalt Performance Grade		PG70-28	PG82-16	PG76-22	PG76-22	PG82-16	PG82-16
MSCR Test	Non-recoverable Creep Compliance, $J_{nr3.2}$, kpa^{-1}	1.82	3.95	3.41	2.76	3.75	2.87
	Percent difference between $J_{nr3.2}$ and $J_{nr0.1}$, J_{nrdiff}, %	33.7	39.7	25.85	34.65	62.85	70.4
Pavement Condition, Index (PCI), %		72	72	42	42	100	100

CHAPTER 6 SUMMARY OF FINDININGS

This study presents both laboratory and field evaluation of the rutting and the moisture sensitivity of WMA mixtures, which contain varying amounts of RAP materials. The performance of the WMA mixtures was compared against the conventional HMA mixtures prepared using the same materials with regard to rutting and moisture-induced damage. This study used aggregate and RAP sources from three different states; Iowa, Minnesota and Ohio along with three different WMA technologies; RAPCAP, LEADCAP 6-8 and LEADCAP 7-1, respectively. Combinations of the aggregate types, RAP contents and WMA additives were evaluated in the laboratory and in the field to study their effect on the performance of WMA mixtures compared to HMA. Test sections were constructed in three different geographical locations in Iowa, Minnesota and Ohio. The conclusions and future study recommendations are discussed below.

6.1. Conclusions

Based on the experimental test results, the following conclusions can be made:

- For Hamburg Wheel Tracking (HWT) test, it was found that RAP content had a significant impact on the moisture susceptibility of both HMA and WMA mixtures. With RAP materials less than 50% by binder replacement, WMA mixture with LEADCAP did not perform as well as HMA mixtures. However, with RAP materials of 50% or more, both WMA and HMA performed well with little or no rutting. It can be concluded that RAP materials improve the moisture susceptibility of both WMA and HMA mixtures.
- It was also found that, for the given RAP amount, WMA mixture with a RAPCAP

additive exhibited less moisture susceptibility than the other WMA mixtures.

- Based on the modified Lottman test, it was found that both HMA and WMA mixture with RAPCAP additive had a high resistance to moisture damage.
- Based on the asphalt bond strength (ABS) test following AASHTO TP 91-11, it can be concluded that removing the pullout stub's edge (i.e. 0.0 mm thickness) produced more consistent and reasonable test results by producing a thin layer of asphalt as expected.
- Asphalt binder type had a significant impact on the ABS test results.
- The test sections of WMA using LEADCAP/RAPCAP were successfully constructed in three different states and met field density requirements by each state DOT.
- All WMA mixtures in the three test sections were found to be workable and easy to compact while producing less fumes and emissions during construction.
- All HMA and WMA field samples from the test sections passed the HWT test and exhibited high resistance to rutting and moisture damage.
- Both WMA and HMA Iowa test sections in Iowa exhibited joint reflection cracking after 6 months of construction due to a severe winter season However, the WMA test section exhibited less joint reflection cracking than the HMA test section. Both WMA and HMA test sections in Minnesota exhibited a similar level of distress after 19 months since construction that include joint reflection cracking and edge cracking.
- Both WMA and HMA Ohio test sections exhibited a good condition after 6 months since construction. It should be noted that the Ohio test sections were constructed

on top of an old asphalt pavement layer, which has no joint reflection cracks unlike the other two test sections in Iowa and Minnesota.

- The high and low temperature grades of the recovered asphalts from test locations were higher than the targeted temperature grade due to the presence of RAP in the mixtures except for the asphalt binder extracted from the WMA with a RAPCAP additive.
- It can be concluded that the RAPCAP additive significantly improves the rheological properties of the extracted asphalt binder from RAP materials while lowering the production temperature of the mix.
- RAPCAP additive significantly improved the non-recoverable creep compliance and percent recovery of the asphalt binder during the MSCR test.
- Based on the Asphalt Bond Strength (ABS) test, HMA conditioned samples from Iowa and both HMA and WMA_LEADCAP conditioned samples from Ohio test sections failed in adhesion mode whereas all unconditioned samples failed in cohesion mode.
- The repeatability of the ABS test method was poor due to the wide variability of the test materials such as aggregate plates, asphalt binder and pullout stubs.

6.2. Recommendations for Future Study

Since WMA additive known as "RAPCAP" significantly improved the rheological properties of the aged asphalt binder within the RAP materials, more detailed asphalt binder analysis mixed with extracted asphalt binders from RAP materials should be performed.

In order to predict a long-term performance of WMA with a RAPCAP additive, WMA pavements with varying RAP amounts should be constructed on well-prepared based layer

In order to improve the repeatability of ABS test, more tests should be performed on various aggregate surfaces, different asphalt binders, and different conditioning methods.

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