REDUCING MOISTURE DAMAGE IN ASPHALT MIXES USING RECYCLED WASTE ADDITIVES

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ABSTRACT

Reducing Moisture Damage in Asphalt Mixes Using Recycled Waste Additives

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This thesis has determined that using fly ash as a mineral filler in asphalt pavements can help strengthen and reduce asphalt moisture damage. Also, dynamic shear rheometer tests show that these additives have a stiffening effect on asphalt binder. Moisture related damage is considered one of the main causes of asphalt pavement failure. As water infiltrates a layer of asphalt, it slowly strips away asphalt binder, weakening the aggregate/binder bond. This process, combined with the cyclic loading of traffic, can lead to several different types of asphalt failure including rutting, raveling, bleeding, and cracking. For several decades, research has been conducted to find a solution to this problem. Currently in practice, hydrated lime and a variety of amine-based chemicals are being used as anti-stripping agents. However, as an emphasis towards sustainability has increased, waste products are now being investigated for this purpose. This thesis investigated the anti-stripping effectiveness of two waste products: fly ash and cement kiln dust (CKD), and compared them with hydrated lime and an amine-based chemical additive. The results indicate that class C fly ash can be used as an asphalt anti-stripping additive; however it is more costly than lime or amine chemicals.

Keywords: anti-stripping, asphalt, fly ash, cement kiln dust, CKD, D1075, direct shear rheometer, DSR, HMA, hydrated lime, moisture damage, moisture sensitivity, moisture susceptibility, Superpave, stripping, T283.

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CHAPTER 1: Introduction

1.1 Background

Asphalt pavement is one of the most utilized highway surfaces. According to the National Asphalt Pavement Association, more than 94% of paved roads in the United States are covered with asphalt pavement. Over 4 million miles of roadway and nearly 3 billion vehicle miles are traveled each year in the U.S. In order to expand and maintain this vast network of highways, over 500 million tons of asphalt is produced each year at an estimated cost of \$30 billion. Such a valuable infrastructure supports America's workforce and movement of goods and services.

Most highway pavements are designed for a service life of about 20 to 25 years. However, many sections of highways never last that long without major pavement overhauls. Several damage mechanisms contribute to the shortening on pavement life. One such contributor to asphalt pavement destruction is known as moisture damage. Moisture damage is caused by water infiltrating the pavement surface and weakening bonds that hold the pavements together. Moisture damage provides the opportunity for other forms of pavement distresses as well, further weakening and destroying the pavement.

Decades of research have been conducted to find a solution to this problem. Through the development of laboratory testing procedures and experimenting with different anti-stripping agents, progress has been made in producing more resilient

pavements. With an increasing emphasis towards sustainability, research has become more focused on the use of waste products in asphalt to improve performance.

1.2 Significance and Scope of Research

Moisture damage is one of the major causes of pavement failure and it can lead to other forms of pavement distress. Since asphalt pavement covers nearly all of the interstate highway system, which is depended upon by thousands for transportation, developing asphalt pavements with greater moisture resistance is of the utmost importance. Also, a more resilient pavement would perform better under future traffic increases. If the service life of a pavement can be extended by several years, long-term costs will be greatly reduced, even considering additive costs.

This thesis will investigate the performance of five asphalt additives against moisture damage: class C fly ash, class F fly ash, CKD, lime, and a chemical anti-stripping additive, HP Plus. It also investigates how these additives change asphalt binder properties. Dynamic shear rheometer and rolling thin-film oven tests were conducted on 3 different concentrations of each additive to determine any changes in binder properties. The asphalt mix used for moisture damage was designed from single aggregate and binder sources using the Superpave mix design procedure. For the moisture damage tests, two tests were conducted on compacted specimens: the modified Lottman indirect tensile test and the immersion compression test. For each test, at least 3 concentrations were tested for each additive and 3 different application methods were tested. Each specimen set contained 6 specimens, 3 were left unconditioned and 3 were conditioned by moisture damage.

1.3 List of Acronyms

AASHTO- American Association of State Highway Transportation Officials

ACAA- American Coal Ash Association

ANOVA- Analysis of Variance

ASTM- American Society for Testing and Materials

Caltrans- California Department of Transportation

CKD- Cement Kiln Dust

DSR- Direct Shear Rheometer

FHWA- Federal Highway Administration

HMA- Hot Mix Asphalt

IDT- Indirect Tensile

IRS- Index of Retained Strength

RTFO- Rolling Thin Film Oven

SHRP- Strategic Highway Research Program

Superpave- Superior Performing Asphalt Pavement

SSD- saturated surface dry

TSR- Tensile Strength Ratio

1.4 Organization of Report

This thesis report is organized in the following order: Chapter 1 includes an introduction and general overview of the research conducted. Chapter 2 consists of a literature review discussing asphalt moisture damage causes, evaluation methods, and treatment methods. Chapter 3 describes the experimental work and provides tabulated results. Chapter 4 analyzes and discusses the results of this thesis. Lastly, Chapter 5 presents the conclusions made from the results and provides recommendations for future studies.

CHAPTER 2: Literature Review

2.1 Introduction

Moisture damage in asphalt pavements, also known as stripping or moisture susceptibility, can simply be defined as the breaking of the aggregate-binder bond by the intrusion of water. As water is exposed to asphalt pavements, it seeps through tiny cracks in the asphalt surface. According to Kringos and Scarpas (2008), asphalt pavements that are exposed to water infiltration often begin losing aggregates. Due to the chemical attractiveness aggregates have towards water, the bond between asphalt binder and aggregates weakens, washing away binder. With the continued action of moisture-induced weakening and cyclic traffic loading, progressive dislodgement of aggregates becomes the dominant mode of failure in asphalt pavements. Esarwi et al. (2008) state that this failure can appear in the form of distresses such as rutting, shoving, raveling, or cracking.

For decades, engineers have been studying the effects of asphalt moisture damage and prevention treatments. Several tests have been developed and standardized by organizations such as the Federal Highway Administration (FHWA), the American Association of State Highway Officials (AASHTO), and the American Society of Testing and Materials (ASTM). Also, several additives have been studied and used in asphalt pavements to improve moisture resistance. A discussion on asphalt moisture damage, common testing methods, and additives that prevent moisture damage are presented in this chapter.

2.2 Mechanisms behind Moisture Damage

The reasons behind moisture damage are not quite fully understood, but a few commonly accepted failure mechanisms include pore pressure, hydraulic scour, detachment, displacement, spontaneous emulsification, pH instability, and environmental factors (Santucci, 2010). Each of these mechanisms lead to two major failure types: loss of adhesion or loss of cohesion. Also, several factors are known to contribute to the occurrence and severity of moisture damage. These factors include asphalt binder properties, aggregate properties, asphalt mixture properties, construction practice, quality control during compaction, the nature of water at the interface, the dynamic effect of traffic loading, as well as the type and properties of anti-stripping additives (Huang et al., 2010).

On a microscopic level, the occurrence of moisture damage is described by two failure types according to Hicks (1991). The first failure type is loss of adhesion. This happens when water infiltrates the aggregate-binder bond interface, destroys the bond, and washes away the binder, leaving the aggregate without a binder coating. The second type of moisture damage failure is a loss of cohesion within the asphalt binder itself. This can be described as a softening and weakening effect water has on binder. Both mechanisms are interrelated and work together, slowly destroying the pavement.

Research has shown that these two failures are largely due to interactions at the chemical level (Jahromi, 2009). Aggregates vary greatly in terms of composition, surface chemistry, and morphology. Air sometimes becomes trapped within fine pores, making it difficult for binder to thoroughly cover the aggregate surface. This contributes to various active and inactive areas on the aggregate surface, which affect how asphalt binder

molecules interact. In addition to this, asphalt binder contains some polar hydrocarbons, as well as organometallic constituents that contain nickel, vanadium, and iron. Oxygen containing groups of the asphalt binder are preferentially adsorbed to the aggregate, concluding that the binder-aggregate interface chemistry influences the ultimate adhesive strength. Jahromi (2009) continues to state that water can diffuse through the asphalt film, reaching the aggregate surface and competing for active areas.

Santucci (2010) also discussed the importance of asphalt surface chemistry, describing aggregates ranging from basic (limestone) to acidic (quartzite) in pH and describing asphalt binder as having neutral to acidic tendencies. Because of this, binder would more likely form a stronger bond with limestone. Also, clay present on the surface of aggregates can expand in the presence of water and form a barrier to adhesion, thus weakening the mix.

Two possible failure mechanisms that can aid in pavement moisture damage are pore pressure and hydraulic scouring. According to Lu (2005), the dynamic loading from traffic and vapor from heat can increase the damage done by entrapped water in the pavement through pore pressure. These forces are strong enough to rupture asphalt films especially at aggregate edges where there are higher stress concentrations and thinner asphalt film thicknesses. Hydraulic scouring may occur at the pavement surface or between lifts where there is high saturation or where water sets for extended periods of time. As vehicles roll over the surface, they first apply a compressive force, then a tensile force to the water in the surface pores. These pressures can contribute to the wearing of the asphalt film on the pavement surface.

Detachment is defined as the separation of the asphalt film from the aggregate surface by a thin film of water without an obvious break in the asphalt film. Contrary to detachment, displacement (also known as a film rupture) involves the removal of asphalt from the aggregate surface due to a break in the asphalt film. Spontaneous emulsification is aggravated by the presence of emulsifiers such as clays or asphalt additives resulting in an emulsion of water in asphalt. Changes in pH of the contact water can affect chemical bonds that influence aggregate-binder bond adhesion as well as wetting characteristics of the aggregate. Lastly, environment factors such as air, temperature, and water play a large role in asphalt moisture damage. Premature asphalt failure is more likely to occur when poor quality materials are used in environments that experience excessive rainfall, wide temperature gradients, freeze-thaw conditions, and severe asphalt aging (Lu, 2005).

It is clear that asphalt moisture damage is a complex issue that arises from several different mechanisms. Much of this knowledge is theoretical and only has been demonstrated in laboratory tests. Engineers are merely beginning to understand what actually happens to asphalt pavements on streets, parking lots, and highways.

2.3 Testing for Moisture Damage

There are numerous tests that have been developed for evaluating resistance to moisture damage. Some of these tests evaluate the individual components of the asphalt mix, while others are conducted on loose mixtures and compacted specimens. This section will cover a few of the common tests conducted on loose and compacted specimens, as well as a few new methods in development.

2.3.1 Modified Lottman Indirect Tensile Test (AASHTO T 283)

The modified Lottman indirect tensile test is one of the most common tests conducted on asphalt specimens to detect moisture damage. In fact, it has been adopted by AASHTO and is included as part of the Superpave mix design procedure. This test was derived from the original Lottman test, exchanging 30 minutes of saturation time for quicker vacuum saturation and modifying the test rate and temperature from 0.065 in/min at 10°C to 2 in/min at 25°C (Santucci, 2010). The current test involves curing at least 6 loose specimens for 16 hours at a temperature of 60°C and aging for 2 hours at the compaction temperature. Specimens are then compacted to approximately 7% air voids, removed from the mold, and left to cool. Half the specimens are vacuum saturated to a 70-80% saturation level and placed in a freezer at -18°C for at least 16 hours. Afterwards, these specimens are placed in a 60°C hot water bath for 24 hours and cooled to the 25°C test temperature. The other half of the specimens are left unconditioned. The indirect tensile strength is measured for both conditioned and unconditioned sets. The test is called an indirect tensile test because a compressive force is applied to the sides of the specimen, causing it to split apart (see Chapter 3 for more details). Results from this test are then used to formulate a tensile strength ratio (TSR) between conditioned and unconditioned specimens (AASHTO, 2004). According to Superpave mix design requirements, a set of specimens must retain at least 80% of its tensile strength after moisture conditioning (Asphalt Institute, 2001). Also, a visual inspection is made to the specimens to check for binder stripping and broken aggregates.

2.3.2 Immersion Compression Test (ASTM D 1075)

The immersion compression test is a method which determines the loss of compressive strength resulting from the action of water on compacted asphalt specimens. In this test, 6 specimens are prepared according to Test Method D 1074 with regards to molding and curing. Specimens are compacted to about 6% air voids, although other air void ratios may be used depending on mix design. Specimens are then allowed to cool and are divided into two groups according to average bulk specific gravity. One set is placed in a 60°C water bath for 24 hours and then cooled to a testing temperature of 25°C. Both conditioned and unconditioned sets are tested for compressive strength. The compressive strengths can then be used to calculate an index of retained strength (ASTM, 2002).

2.3.3 Hamburg Wheel Tracking Test

According to the FHWA (2011), the Hamburg wheel tracking test is used to measure the combined effects of rutting and moisture damage by rolling a steel wheel over an asphalt slab immersed in hot water. The testing device was first developed in the 1970's by Esso A.G. of Hamburg, Germany after a similar British device. The City of Hamburg originally used it to measure rutting potential of asphalt mixes by performing 9,540 wheel passes on a specimen immersed in 40-50°C water. The city then increased the number of wheel passes to 19,200 and realized that specimens subjected to over 10,000 wheel passes generally exhibited deterioration from moisture damage.

Today, the wheel tracking device is used to test asphalt slabs 320 mm in length, 260 mm in width, and thicknesses of 40, 80, or 120 mm. The minimum thickness required is three times the nominal aggregate size and pavement cores of at least 250 mm

in diameter can be tested as well. Specimens are molded and compacted to 7% air voids, then placed in steel containers. The specimens are conditioned in 50°C water for at least 30 minutes before testing. During testing, two asphalt slabs are simultaneously tested under solid steel wheels which reciprocate at 53 passes per minute. The wheels have a specified diameter of 203.5 mm, a width of 47.0 mm, and apply a load of 685 N. This configuration is intended to replicate the stress produced by one rear tire of a double-axle truck. In the United States 20,000 passes are being used, although up 100,000 passes may be conducted (FHWA, 2011).

After 6.5 hours (20,000 passes) the test results, as shown in Figure 2.1, can be analyzed to determine post-compaction consolidation, creep slope, stripping inflection point, and stripping slope. Post-compaction consolidation is defined as the deformation at 1,000 wheel passes, while creep slope is used to determine how susceptible the mixture is towards rutting. Both the stripping inflection point and stripping slope can be used to evaluate moisture damage. The stripping inflection point is the point where the creep slope and stripping slope meet. If this point is reached before 10,000 wheel passes, then the specimen is considered to be susceptible to moisture damage. The stripping slope represents permanent deformation due to moisture damage (FHWA, 2011).

Figure 2.1 Typical Hamburg wheel tracking test results (FHWA, 2011)

2.3.4 Tunnickliff-Root Conditioning Test (ASTM D 4867)

The Tunnickliff-Root conditioning test is very similar to the modified-Lottman test. It also uses an indirect tensile test to compare conditioned and unconditioned specimens. Asphalt concrete specimens are prepared between a 6% and 8% air void ratio, half are vacuum saturated to 55-80% saturation. The freezing step for the conditioned specimens is an optional procedure, differing from the modified Lottman test. Specimens are then placed in a 60°C hot water bath for 24 hours and cooled to 25°C to be tested with the unconditioned specimens. A minimum 80% TSR between conditioned and unconditioned specimens must be obtained from indirect tensile tests in order for the batch to pass (Hunter and Ksaibati, 2002).

2.3.5 Boiling Test (ASTM D 3625)

The boiling water test is conducted on loose HMA mixtures and is generally used as an initial screening test for mix quality control. Some agencies even use it to detect the presence of anti-stripping agents during asphalt production. The asphalt mixture is subjected to boiling water for 10 minutes, after which the amount of moisture damage is usually determined by the condition of the loose asphalt in the water. The percentage of asphalt binder visibly retained on the aggregate is rated either above or below 95%. However, it is difficult to determine the amount of stripping on small, fine aggregates with this test (Hunter and Ksaibati, 2002).

2.3.6 Texas Boiling Water Test

The Texas boiling water test (TBWT) is a more standardized version of the original boiling water test. It specifies that the asphalt binder should be heated at 103°C for 25 hours and that 100 or 300 grams of unwashed aggregate be heated to this temperature for 1 to 1.5 hours. Then the aggregate and binder are mixed and cooled for 2 hours before being placed in 1000 mL of boiling distilled water for 10 minutes. Any asphalt binder that floats to the top during boiling is skimmed off. After 10 minutes, the water is left to cool, poured off, and the asphalt mix is dried with a paper towel and graded. A three person panel grades the mixture twice, immediately and the next day after the mix dries. If a mix retains 65% to 75% asphalt binder, it is considered favorable for field use (Hunter and Ksaibati, 2002).

2.3.7 Moisture Induced Stress Tester (MIST)

A moisture induced stress tester as shown in Figure 2.2 is a relatively new device for simulating moisture damage in asphalt pavements. Chen and Huang (2008) cite that this device is designed to simulate stripping by repetitively generating pore pressures experienced by pavements in the field due to temperatures, traffic, and moisture. By using compressed air, pressures and vacuums are applied to force water through the HMA specimen. Within 3 hours a specimen can be conditioned and results can be determined through indirect tensile tests. For Chen's study, the specimen bath water was kept at a constant 40°C and was subjected to a pressure of 40 psi. Up to 1000 MIST cycles were conducted on specimens. Increasing the amount of MIST cycles demonstrated an increase in lab measured moisture damage.

Figure 2.2 A moisture induced stress tester (MIST) (Chen and Huang, 2008)

2.4 Additives for Preventing Moisture Damage

Since moisture damage is such a prominent issue, engineers have been developing and testing various asphalt pavement additives that resist stripping for the past few decades. Perhaps the most commonly used anti-stripping additives are hydrated lime and liquid amine-based chemicals. More recently however, improving pavement performance through the use of recycled waste materials such as fly ash, cement kiln dust, glass, and used tires has gained interest.

2.4.1 Hydrated Lime

Hydrated lime, as shown in Figure 2.3, is perhaps one of the most commonly used asphalt anti-stripping additives. Generally, lime is added as part of the mineral fill portion (less than #200 sieve) consisting of 1% to 2% of the total aggregate weight. It may be added to dry or damp aggregate, combined with water to form a slurry, or mixed in with asphalt binder. Most transportation departments require adding hydrated lime to wet aggregate or in the form of lime slurry (National Lime Association, 2003).

Figure 2.3 Lime

According to Huang et al. (2010), lime is very chemically active in nature, thus making it very effective at reducing moisture damage in asphalt pavements. Huang cites three major mechanisms that contribute to lime's effectiveness. First, lime reacts with the aggregate surface, which forms chemical products that bond strongly with asphalt binder even in the presence of moisture. Secondly, lime consolidates fine coatings on the aggregate surfaces through reactions with calcium and silica, producing calcium rich bonding areas for the acidic polar components of asphalt binder. Third, hydrated lime reacts with carboxylic acids and 2-quinolones allowing less water sensitive polar components to bond with active areas on the aggregate.

In addition to hydrated lime's anti-stripping properties, the additive contributes other benefits to asphalt mixes. Jahromi (2009) cites that lime significantly improves low temperature fracture toughness without inhibiting the asphalt binder's ability to dissipate energy through relaxation. It has also been observed that hydrated lime is capable of forming very strong bonds at all pH levels when compared to liquid anti-stripping agents, which generally form strong bonds at pH levels greater than 8. In creep tests, lime has shown to aid in high temperature stability, which increases resistance to permanent deformation.

All of these properties make hydrated lime an excellent mineral filler for improving asphalt quality. Numerous studies have been conducted on hydrated lime modified asphalt specimens that confirm this. Esarwi et al. (2008) tested two types of asphalt binders, each with and without hydrated lime using both Marshall and Superpave mix design procedures. By adding 2% hydrated lime of the aggregate total weight, they found that lime modified specimens were able to meet acceptable TSR's in all but one

combination. Also, a few mix designs that originally failed to meet TSR requirements without lime were able to meet those requirements with added lime.

Gorkem and Sengoz (2009) conducted a similar study using hydrated lime at 1%, 1.5%, and 2.0% concentrations, as well as testing basalt and limestone aggregate. They observed that by adding lime to both aggregate types, the TSR increased, however a much more significant increase was shown for the basalt aggregate as compared to limestone. Gorkem hypothesized this significant increase could be due to the lime forming insoluble salts on the basalt aggregate, allowing for stronger bonding. Lime content in the mix appeared to have minimal impact on TSR's in the study.

Using three aggregate types and two asphalt binder combinations, Jahromi (2009) measured the adhesive bond energy of mix components and performed permanent deformation tests. The three aggregates tested were river gravel, limestone, and crushed granite. Each was tested with two binder types, as well as with and without hydrated lime. It was found that the wet/dry ratios of the dynamic modulus increased with the addition of lime, indicating reduced moisture damage. Also, a significant difference was observed between the two binder types added to lime-enhanced river gravel.

Haug et al. (2010) studied how the lime particle fineness affected its performance as an asphalt anti-stripping additive. To accomplish this, they placed commercially available lime in a Los Angeles (L.A.) abrasion machine for 0, 500, and 2500 revolutions. This device pulverized the particles into smaller particles with increasing revolutions. Afterward, a 1% concentration of the lime was added to an asphalt mix from which specimens were made and tested for moisture damage. Results showed that increasing the fineness of lime particles increased TSR's.

2.4.2 Liquid Anti-strip Additives

Another common type of additives used to minimize moisture damage in asphalt pavements are liquid chemical anti-stripping additives like to one shown in Figure 2.4. According to Lu (2005), most liquid anti-stripping additives are chemicals containing amines which are strongly basic ammonia-based chemicals. They are generally cationic to promote adhesion between acidic aggregates and acidic asphalt binders, but some contain both cationic and anionic compounds for all aggregate and binder types. When added to asphalt mixes, liquid anti-stripping additives tend to soften the asphalt and affect aging and temperature characteristics. Santucci (2010) explains that liquid anti-stripping additives are added to asphalt binder at the refinery or through in-line blending at hot mix plants. Concentrations of 0.25% to 1% of the asphalt binder weight are used.

Figure 2.4 HP Plus, a liquid anti-stripping additive

A few studies have been conducted on the effectiveness of liquid anti-stripping additives. For example, Aksoy et al. (2005) experimented with two ammine chemicals

(Wetfix I and Lilamin VP 75P), a catalyst (Chemcrete), and a rubber polymer. The chemicals were added to the mix at concentrations of 0.2%, 0.4%, and 0.6% of the asphalt binder and specimens were made according to the Marshall mix design procedure. Specimens were then tested for Marshall stability, flow, and indirect tensile strength. Specimens using chemicals exhibited slightly higher stability and flow ratios than control specimens. There was also a significant difference in TSR's. Wetfix I and Lilamin VP 75P showed the most moisture damage resistance at 0.2% and 0.4% concentrations, respectively. Both chemicals allowed the mix to meet the minimum TSR requirement.

Chen et al. (2008) conditioned asphalt specimens containing amine-based chemicals using a moisture induced stress tester (MIST) and tested them for moisture damage. The liquid anti-stripping additive was added at a concentration of 0.5% of the weight of the asphalt binder. Specimens were then subjected to 0, 1, and 2 freeze-thaw cycles as well as 0, 500, and 1000 cycles of moisture induced stresses. Tensile strength results indicated that the amine-based additives helped decrease moisture damage in the lab specimens.

2.4.3 Fly Ash

According to the American Coal Ash Association (2003), fly ash is fine-grained by-product of burning pulverized coal to generate steam and electricity. At the power plant, coal is usually pulverized and blown into the combustion chamber where it ignites, generating heat and residue. This hot residue cools and forms ash as heat is extracted by the boiler. Part of this residue contains coarser, heavier particles called bottom ash, since it falls to the bottom of the combustion chamber. Smaller particles, called fly ash, become

suspended in the plant's exhaust gases and are extracted from exhaust gases by emission control devices such as electrostatic precipitators and filter fabric baghouses before the gases are released into the atmosphere. Fly ash is then stockpiled in a silo or containment pond until it can be utilized or sent to a landfill.

On a particle level, fly ash contains pozzolans, which are siliceous and aluminous materials that when in the presence of water, react with calcium hydroxide to produce cementitious compounds. Typically finer than Portland cement and lime, fly ash consists of spherical particles typically 10 to 100 microns in size, which aid in its reactivity. Colors range from light tan, to brown, to dark gray, and even black when it contains significant amounts of unburned coal (ACAA, 2003).

Chemically, fly ash primarily contains oxides of silicon, aluminum, iron, and calcium, as well as smaller concentrations of magnesium, potassium, sodium, titanium, and sulfur. It is generally classified as class C or class F, depending on its calcium content. Class C fly ash (Figure 2.5) is produced from sub-bituminous coals and consists of calcium aluminosulfate, quartz, tricalcium aluminate, and at least 20% free lime (CaO). Class F fly ash (Figure 2.6), on the other hand, is produced from bituminous and anthracite coals and consists of aluminosilicate glass, quartz, mullite, magnetite, and less than 10% free lime (ACAA, 2003).

Figure 2.5 Class C fly ash

Figure 2.6 Class F fly ash

When used as mineral filler, several properties allow fly ash to aid in resisting rutting and moisture damage. First, it typically meets mineral filler specifications such as gradation, organic impurities, and plasticity. Fly ash has a lower specific gravity than natural mineral fill ($2.0\text{-}2.6\text{ g/cm}^3$, compared to $2.6\text{-}2.8\text{ g/cm}^3$), reducing HMA weight,

and therefore cost, and may also cost less in areas where it is readily available. Asphalt stripping potential is reduced due to fly ash's hydrophobic (water hating) nature and free lime content. Since asphalt pavement mixes have become coarser to meet the loads of heavy and increasing traffic, mineral fillers such as fly ash are needed to stiffen the mix, as well as reduce asphalt drain down during construction (ACAA, 2003).

A couple of studies have been conducted comparing fly ash to other mineral fillers. Huang et al. (2010) conducted such an experiment by adding 1.0% concentration of class F fly ash to the asphalt mix. Results show a similar resilient modulus ratio compared to the control mixture, but slightly lower than lime. TSR tests show a 15% higher ratio in tensile strength over the control mixture, although variations of hydrated lime increased the ratio by 25%. A field study by Wolfe et al. (2009) showed an increase in the resilient modulus when using class F fly ash with lime as filler in road base. Fly ash with lime performed similarly to cement treated sections and better than emulsion treated sections.

2.4.4 *Cement Kiln Dust*

Cement Kiln Dust (CKD), as shown in Figure 2.7, is a by-product from cement production. It is a fine powder substance similar to Portland cement and is obtained from electrostatic precipitators that trap small airborne particles during production of cement clinker. It may be classified into four categories based on which type of kiln and process it comes from. There are two types of cement kilns: wet kilns and dry kilns and two collection processes: one where dust is separated and returned to the kiln and one where dust can be recycled or discarded (Siddique, 2008).

Figure 2.7 Cement kiln dust

Chemically, CKD has a similar composition to Portland cement and contains mostly lime, iron, silica, and alumina. It also contains traces of cadmium, lead, selenium, and less than 0.05% radio nuclides. It has a pH of around 12, contains a significant amount of alkalies, and is considered to cause corrosion (Siddique, 2008).

CKD has been effectively used as mineral filler in asphalt pavement mixes. The asphalt binder requirements can be significantly reduced by mixing CKD with asphalt binder before it is introduced to the aggregate. It also has the potential to replace hydrated lime and reduce moisture damage in pavements due to its high lime content (Siddique, 2008). Huang et al. (2010) verified this in their study testing various mineral fillers. Adding 1.0% cement kiln dust to the asphalt mix produced a TSR within a few percent of the hydrated lime variations and nearly 25% higher than the untreated control mixture.

2.4.5 *Other Asphalt Additives*

Besides lime, amine chemicals, fly ash, and cement kiln dust, several other products have been tested in asphalt mixes for moisture damage resistance. Aksoy et al. (2005) tested a catalyst called Chemcrete which changes the molecular structure of the asphalt binder through temperature and oxygen dependent reactions. During asphalt mixing and compacting, Chemcrete replaces weak electrostatic forces with strong irreversible chemical bonds. Manufacturer tests report decreased temperature susceptibility, improved deformation resistance, and adhesion. By adding Chemcrete at a 2% concentration of binder weight, Aksoy et al. (2005) were able to significantly increase the tensile strength ratio from 0.45 to 0.80.

Rubber was also tested by Aksoy et al. (2005) for resistance to moisture damage. The main objective in this study was to use rubber to decrease temperature susceptibility and increase the resilient modulus. This reduces pavement cracking and binder oxidation, increasing pavement life. Rubber used in asphalt pavements may be extracted from recycled tires. Results from Aksoy et al. (2005) show increased stability and overall tensile strengths when using rubber, but decreased tensile strength ratios, indicating reduced resistance to moisture damage.

2.5 Summary

This chapter described asphalt moisture damage as a serious issue that affects asphalt pavements in the United States and throughout the world. Research has shown that the repeated cycling of water through pavement structure over time significantly reduces the pavement's life. Due to the vast miles of asphalt highways and limited funding, engineers are continuously studying moisture damage in the field and are

developing more effective laboratory test methods for evaluating asphalt mixes. The ability to realistically simulate moisture damage in the laboratory allows engineers to test a variety of asphalt additives more cost effectively, which will hopefully lead to longer-lasting pavements. Finally, this chapter discussed current and potential remedies for reducing moisture damage.

CHAPTER 3: Materials, Testing, and Results

3.1 Introduction

The goal of this study was to determine the moisture damage resistance of compacted asphalt specimens modified with fly ash and cement kiln dust. The effectiveness of these additives were evaluated by comparing test results conducted on unmodified (virgin), hydrated lime, and amine chemical specimens. The two major tests used in this study to compare the moisture damage resistance were the modified Lottman indirect tensile test (AASHTO T 283) and the immersion compression test (ASTM D 1075). In addition, other performance tests were conducted on the aggregate, asphalt binder, and uncompacted specimens. All tests on aggregate, asphalt binder, loose mixtures, and compacted specimens were conducted according to respective AASHTO, ASTM, Caltrans, or Superpave testing standards.

3.2 Material Selection

Several materials were required for producing asphalt specimens. In addition to aggregate and asphalt binder, several additives were needed. These additives consist of lime, class C fly ash, class F fly ash, and an amine chemical called HP Plus.

The crushed stone aggregate used for laboratory testing was obtained locally from CalPortland Construction's Paso Robles asphalt plant. This aggregate is used locally in asphalt mixes to pave roads such as Highway 101. Over 450 kg (1000 lbs) of aggregate was required to produce over 300 asphalt specimens tested in this study. Aggregate was delivered in pre-sieved sacks which helped significantly with the sieving process, even

though all aggregate had to be sieved to meet gradations used in the lab and for quality control. Four large plastic bins were used to store the aggregate passing $\frac{3}{4}$ ", $\frac{1}{2}$ ", $\frac{3}{8}$ ", #8 sieves, and a smaller container stored dust passing the #200 sieve. These aggregate gradations were used in the formation of the gradation curve mentioned later in this chapter.

The asphalt binder used for testing was obtained from Oxnard Refinery in southern California. Approximately 30 liters (8 gallons) of PG 64-16 binder was required to produce the specimens. The 64-16 performance grade is a common type of asphalt used locally on the central coast of California since the climate is relatively mild. Shipments of asphalt binder were delivered in gallon sized cans from the refinery to CalPortland's Santa Maria office, where they were picked up and brought to the laboratory. The asphalt was then heated and separated into small trays on a weekly basis to allow for quicker heating during asphalt mixing.

The additives used in testing were obtained from a variety of sources. Chemstar high calcium hydrated lime was commercially available at a local farm supply store. The both classes of fly ash were obtained in 5 gallon buckets from Headwater's Resources, but came from different coal power plants. The class C fly ash came from the Dave Johnson Power Plant in Wyoming and contains 24.63% lime, while the class F fly ash originated from the Jim Bridger Power Plant, also in Wyoming, and contains 5.73% lime. The cement kiln dust was obtained by CalPortland from their Colton Cement Plant in southern California and the amine chemical HP Plus was shipped from ArrMaz Custom Chemicals in Florida.

3.3 Aggregate Tests and Preparation

Important properties and performance tests were conducted when selecting aggregates for the asphalt mix design. First, the bulk and apparent specific gravities were calculated for the range of aggregates. Then performance tests for durability, angularity, and clay content were conducted.

3.3.1 *Flat and Elongated Particles in Coarse Aggregate (ASTM D 4791)*

The flat particles, elongated particles, or flat and elongated particles test was used to determine the percentage of flat and elongated particles in the coarse aggregate. This was significant since flat and elongated particles in HMA mixes have difficulty reorienting during compaction and thus have tendency to break along their thin, weak axis. This can cause issues achieving the correct air void ratio in a pavement and lead to pavement degradation.

Figure 3.1: Flat and elongated particle apparatus

To conduct this test, a proportional caliper apparatus, as shown in Figure 3.1, was used. This device has several pivot points which may be adjusted to different ratios. The Superpave mix design specified a coarse aggregate testing ratio of 5:1 for both flatness (width to thickness) and elongation (length to width). None of the particles tested in the batch had a ratio this large, thus easily meeting the maximum batch limitation of 10%.

3.3.2 Specific Gravity and Absorption of Fine Aggregate (AASHTO T 84)

This test was conducted to determine the bulk and apparent specific gravity, as well as the absorption of the fine aggregate (aggregate passing a #4 sieve). In this test, one kg (2.2 lbs) of dry fine aggregate was submerged into water as shown in Figure 3.2 for a period of 15-19 hours. Then, excess water was removed and the specimen was dried to a surface dry condition. A cone tamping test was used to ensure the correct moisture content. Then, half of the specimen was placed into a pycnometer partially filled with water and agitated to remove air bubbles. The total mass was then recorded and used for bulk specific gravity calculations as follows:

$$\text{Bulk Specific Gravity} = A / (B + S - C)$$

Where:

A = mass of oven-dry specimen in air (g);

B = mass of pycnometer filled with water (g);

C = mass of pycnometer with specimen and water (g);

S = mass of saturated-surface-dry specimen (g).

This calculation yielded a fine aggregate specific gravity of 2.47 g/cm^3 (154 lb/ft^3), which was within the typical range of 2.4 and 3.0 g/cm^3 (150 lb/ft^3 and 187 lb/ft^3). Bulk saturated-surface-dry specific gravity and apparent bulk specific gravity were measured using similar equations. The percentage of water absorbed into the aggregate's pores was computed using the following equation:

$$\text{Absorption (\%)} = [(S - A)/A] \times 100$$

This gave a value of 2.5%, which is below the typical maximum acceptable value of 5%.

Figure 3.2 Submerged coarse and fine aggregate for bulk specific gravity test.

3.3.3 *Specific Gravity and Absorption of Course Aggregate (AASHTO T 85)*

The specific gravity and absorption of course aggregate (retained on the #4 sieve) was determined by following a similar procedure to that of the AASHTO T 84. Since the asphalt mix would be designed around a ½” nominal maximum size, 2 kg (4.4 lb) of course aggregate was sampled and immersed in water for 15-19 hours. Then the aggregate was removed from the water and placed on an absorbent cloth where it was dried to a saturated-surface-dry state and weighed. The specimen was placed in a basket and submerged into a water tank to acquire the saturated mass. The bulk specific gravity of the aggregate was then calculated using the following equation:

$$\text{Bulk Specific Gravity} = A/(B - C)$$

Where:

A = mass of oven-dry specimen (g);

B = mass of saturated-surface-dry specimen (g);

C = mass of saturated specimen (g).

The bulk specific gravity of the course aggregate was determined to be 2.52 g/cm^3 (157 lb/ft^3), which was within the acceptable range of 2.4 and 3.0 g/cm^3 (150 lb/ft^3 and 187 lb/ft^3). Absorption was also determined by:

$$\text{Absorption (\%)} = [(B - A)/A] \times 100$$

This yielded a value of 1.4%, which was under the 5% maximum acceptable limit.

3.3.4 Los Angeles Abrasion Test (AASHTO T 96)

The Los Angeles abrasion test was conducted to evaluate the resistance to degradation of small-size coarse aggregate. This test involved placing the course aggregate in a mechanical rotating drum as shown in Figure 3.3 along with steel spheres, which impacted and pulverized the aggregate. Approximately 5 kg (11 lb) of aggregate was needed for this test, the sizes of which were determined from a provided table. For a $\frac{1}{2}$ " nominal size, grading B was used, which consisted of 2.5 kg (5.5 lb) of aggregate retained on a $\frac{1}{2}$ " sieve and 2.5 kg (5.5 lb) of aggregate retained on a $\frac{3}{8}$ " sieve. Also for grading B, 11 spheres 46.8 mm (1.84 in) diameter were required for the impact charge. Both the spheres and aggregate were placed in standardized rotating drum for 500 revolutions at rate of 30 revolutions per minute.

Figure 3.3 Los Angeles abrasion machine

After the test was complete, all material was removed from the drum and aggregate was sieved through a #12 sieve. The remaining coarse material was then washed, oven dried at 110° C, and weighed. Then the percentage of aggregate lost was calculated by subtracting the difference between the original mass and the final mass of the test specimen and dividing by the original mass. For the aggregate tested, this value was 40.5%, which was under the 45% maximum acceptable loss.

3.3.5 Uncompacted Void Content of Fine Aggregate (AASHTO T 304)

Determining the uncompacted void content of fine aggregate helped determine the aggregate's angularity and surface texture in comparison to other aggregates of the same gradation as well as workability in a mix. With this test, there were three gradation options for fine aggregates to choose from. Method A (standard grading) was selected. A 190 g (0.42 lb) specimen of aggregate passing the #8 sieve was tested by being filled into a plugged funnel. Once the funnel was full and leveled on the top, the bottom hole was opened and the aggregate was allowed to pour into the measuring cylinder as shown in

Figure 3.4. The mass of the cylinder was weighed and the uncompacted voids were determined from the following equation:

$$U = [V - (F/G)]/V \times 100$$

Where:

V = volume of cylindrical measure (mL);

F = net mass of fine aggregate in measure (g);

G = bulk dry specific gravity of fine aggregate (g/cm^3);

U = uncompacted voids (%).

The fine aggregate's uncompacted void content was calculated to be 43%, which was close to the recommended value of 45%.

Figure 3.4 Uncompacted Void Apparatus

3.3.6 Sand Equivalency Test (AASHTO T 176)

The sand equivalency test was conducted to determine the amount of dust or clay-like particles in the fine aggregate gradation. To conduct this test, approximately one kg of aggregate passing the #4 sieve was obtained and moistened until it could hold its shape

in a cast. The aggregate was then thoroughly mixed and compacted in a 3 ounce moisture tin. A graduated cylinder was filled to the 102 mm (4 in) mark with a calcium chloride solution and the aggregate was poured in using a funnel. After 10 minutes of standing the cylinder was agitated to remove air bubbles and sides of the cylinder were washed down with solution. Calcium chloride solution was added until it reached the 381 mm (15 in) mark. After 20 minutes of settling, the clay reading was taken, followed by the sand reading. Dividing the sand reading by the clay reading resulted in a sand equivalent of 85% which surpasses the Superpave minimum requirement of 45%.

3.3.7 Gradation

After determining acceptable quality of the aggregates, gradation curves were developed to meet Caltrans standard specifications shown in Table 3.1 (Caltrans Standard Specifications, 2002). These specifications dictated different sieve ranges for different nominal size mixes. Within the ½” nominal range, three mix blends were developed: coarse, intermediate, and fine, as shown in the Figure 3.5. Properties such as specific gravity and air voids of these blends were determined as well.

Table 3.1 Caltrans grading requirements for ½ inch aggregate mixes

Sieve Sizes	Limits of Proposed Gradation	Operating Range	Contract Compliance
3/4"	-	100	100
1/2"	-	95-100	95-100
3/8"	-	80-95	75-100
#4	59-66	X±5	X±8
#8	43-49	X±5	X±8
#30	22-27	X±5	X±8
#200	-	3-8	0-11

Figure 3.5 Aggregate gradation curves

3.4 Asphalt Binder Tests and Results

The PG 64-16 asphalt binder from Oxnard Refinery is commonly used on the central coast of California and was used for the mix design and in HMA specimen preparation. The binder was tested in its virgin and aged states using the dynamic shear rheometer (DSR) test. The test results are presented in the following sections.

3.4.1 *Dynamic Shear Rheometer (DSR) Test (AASHTO T 315)*

The dynamic shear rheometer test as shown in Figure 3.6 was devised to test the various properties of asphalt binder, particularly the binder's dynamic shear modulus and phase angle. Asphalt binder is considered to be a viscoelastic material, which means it displays characteristics of both properties. It behaves like an elastic solid, rebounding after loading and as a viscous liquid. The complex modulus represents the vector component of both the elastic and viscous portions, while the phase angle represents how viscous or elastic the asphalt binder is. To measure these properties, the asphalt binder

was compressed between two parallel plates while the upper plate oscillated, exerting a shear force on the binder. Then, sensitive sensors in the dynamic shear rheometer (DSR) recorded the binder properties.

Figure 3.6 A dynamic shear rheometer

Also, the effect of adding fly ash, CKD, lime, and HP Plus to the PG 64-16 binder was examined by using a DSR. At least two specimens of various additive concentrations were prepared as shown in Table 3.2. For the mineral fillers, percentages in Tables 3.2, 3.3, and 3.4 represent concentrations in an asphalt mix with a typical 5% binder content. This means that having a 3% additive concentration and a 5% binder content in the mix would result in a 0.6:1 additive to binder ratio for DSR testing. A 5% additive concentration in the mix would have a 1:1 additive to binder ratio, and a 7% additive concentration in the mix would have a 1.4:1 additive to binder ratio for DSR testing. The ratios for the liquid anti-stripping additive, HP Plus, were the actual percentages compared to binder weight. For example, the 0.50% concentration only contained 0.5 grams of HP

Plus per every 100 grams of binder used. The binder was heated and additives were thoroughly mixed into asphalt binder before being poured into silicone specimen molds as depicted by Figure 3.7. Proper DSR heating, cooling, and calibration procedures were followed before each specimen was placed between the plates and heated to the 64° C test temperature. Results were then recorded in an Excel spreadsheet and are shown in the following section.

Figure 3.7 DSR specimens in silicone molds

Table 3.2 Additive concentrations

Additive %				
Fly Ash Class C*	Fly Ash Class F*	Cement Kiln Dust*	Hydrated Lime*	HP Plus**
1%	1%	1%	1%	0.50%
3%	3%	3%	2%	0.75%
5%	5%	5%	3%	1.00%
7%	7%	7%		

*= percent of total mix weight (assuming 5% binder content)

**= percent of binder weight

3.4.2 Un-aged Asphalt Binder Results

Table 3.3 shows the DSR test results of un-aged binder for all asphalt additive concentrations tested. The properties of unmodified (control) asphalt binder are also displayed for control purposes.

Table 3.3 DSR test results for un-aged asphalt binder

Additive Concentration	Complex Modulus (psi)	Elastic Modulus (psi)	Viscous Modulus (psi)	Phase Angle °
Control				
0%	0.203	0.006	0.203	88.2
Class C Fly Ash				
1%	0.356	0.019	0.356	87.0
3%	0.338	0.011	0.338	88.2
5%	0.661	0.025	0.661	87.8
7%	1.215	0.059	1.214	87.2
Class F Fly Ash				
1%	0.281	0.012	0.281	87.7
3%	0.452	0.016	0.452	87.9
5%	0.725	0.028	0.725	87.8
7%	1.330	0.065	1.328	87.2
CKD				
1%	0.303	0.013	0.302	87.6
3%	0.746	0.035	0.745	87.4
5%	0.966	0.043	0.965	87.4
7%	1.406	0.070	1.404	87.1
Lime				
1%	0.333	0.011	0.332	88.1
2%	0.702	0.037	0.701	86.9
3%	0.952	0.051	0.950	87.0
HP+				
0.50%	0.048	0.003	0.048	86.8
0.75%	0.030	0.003	0.029	84.5
1.00%	0.021	0.003	0.021	81.3

3.4.3 Rolling Thin-Film Oven Test (AASHTO T 240)

The rolling thin-film oven (RTFO) test was used to measure the effect of heat and air on a moving film of asphalt binder to simulate asphalt aging. This conditioning method was used in conjunction with the DSR test to measure the change in asphalt binder properties.

Figure 3.8 A rolling thin-film oven

Two asphalt specimens of 35 g (0.08 lb) were prepared in standardized glass jars for each additive variation and allowed to cool. Since there were eight spots in the oven's rotating carriage, up to eight specimens were tested at one time. Specimens were placed in a 163° C oven, shown in Figure 3.8, with the carriage rotating, fan rotating, and the air jet on for 85 minutes. After the test, the remaining asphalt residue was quickly scraped out into containers so DSR specimens later be molded. A mass change calculation was also determined at this time. Specimens were then tested in the DSR and results for asphalt binder specimens conditioned by the RTFO test are shown in the following section.

3.4.4 RTFO Aged Asphalt Binder Results

Table 3.4 displays the DSR test results of asphalt binder specimens conditioned in a rolling thin-film oven.

Table 3.4 DSR test results for RTFO aged asphalt binder				
Additive Concentration	Complex Modulus (psi)	Elastic Modulus (psi)	Viscous Modulus (psi)	Phase Angle °
Control				
0%	0.456	0.036	0.455	85.5
Class C Fly Ash				
1%	0.847	0.100	0.841	83.2
3%	1.206	0.116	1.201	84.5
5%	1.766	0.178	1.756	84.2
7%	2.645	0.250	2.632	84.6
Class F Fly Ash				
1%	0.688	0.073	0.684	83.9
3%	1.361	0.159	1.351	83.3
5%	1.855	0.180	1.846	84.4
7%	2.965	0.311	2.949	84.0
CKD				
1%	0.771	0.082	0.767	83.9
3%	1.662	0.200	1.650	83.1
5%	2.458	0.241	2.447	84.4
7%	3.667	0.375	3.648	84.1
Lime				
1%	0.577	0.046	0.576	85.3
2%	1.719	0.188	1.709	83.7
3%	1.994	0.203	1.985	84.2
HP+				
0.50%	0.420	0.033	0.419	85.4
0.75%	0.363	0.025	0.362	86.1
1.00%	0.254	0.013	0.254	87.1

3.5 Uncompacted Asphalt Mix Test

3.5.1 Theoretical Maximum Specific Gravity and Density (AASHTO T 209)

Determining the theoretical maximum specific gravity and density of an asphalt mixture was an important parameter for the overall mix design process. This property was essential in calculating the percent air voids in the compacted asphalt mixture and the amount of binder absorbed by the aggregate particles.

Figure 3.9: Vacuum saturating a loose HMA specimen

For this test, asphalt mix specimens for each mix variation were prepared and cured for two hours. Mixes were then cooled in a loose, uncompacted state and placed in a vacuum container filled with water. A high vacuum pump as shown in Figure 3.9 was attached to the container and activated for at least 15 minutes, removing entrapped air. Shaking of the container was required to remove air bubbles. After vacuum saturation, the container was removed from the pump and filled to the calibrated level with water. Then the mass of the container, specimen, and water was determined. This value, along with the dry mass of the specimen and mass of the container filler with just water, was used to determine the theoretical maximum specific gravity by the following equation:

$$\text{Theoretical Maximum Specific Gravity} = A / (A + D - E)$$

Where:

A = mass of oven dry specimen (g);

D = mass of container filler with 25° C water (g);

E = mass of container filled with specimen and water (g);

The theoretical maximum specific gravities were determined to be 2.40 g/cm^3 (150 lb/ft^3), 2.39 g/cm^3 (149 lb/ft^3), and 2.41 g/cm^3 (150 lb/ft^3) for coarse, intermediate, and fine blends, respectively.

3.6 Asphalt Mix Design

3.6.1 *Superpave Mix Design*

The Superpave mix design procedure was developed by the Strategic Highway Research Program (SHRP) in the early 1990's. Its goal was to develop a standardized method of asphalt mix design that accounts for traffic loading and environmental conditions, can evaluate asphalt binder, and can analyze the final mix design. The Superpave procedure includes several steps: aggregate selection, asphalt binder selection, specimen preparation, performance testing, density and voids analysis, optimum binder content selection, and moisture susceptibility evaluation. Many of these steps incorporate tests mentioned in this chapter.

3.6.2 *Specimen Preparation*

Two 4600g (10 lb) aggregate specimens of each of the three gradation blends were prepared and oven dried. Then, 5% asphalt binder was added to the specimens and thoroughly mixed until all aggregate surfaces were covered with binder. Specimens were placed in a 163°C oven for 2 hours of aging; mixing was conducted every half hour to ensure consistency. After aging, specimens were placed in a 150 mm (6 in) diameter compactor mold and compacted to appropriate parameters based on design equivalent single axle loads (ESALs). An ESAL of 3 to 30 million, which is common for most U.S.

highways, was used for this study. The initial, design, and maximum compaction parameters are 8, 100, and 160 revolutions.

A Rainhart Superpave gyratory compactor, shown in Figure 3.10, was used to compact the asphalt specimens. This device was designed to simulate, in the laboratory, the kneading action of a smooth-wheeled roller used to compact asphalt in the field. It accomplished this by placing a 600kPa (87 psi) vertical pressure on the specimen inside the mold. Rollers were then lifted which helped gyrate the mold at a 1.25° angle for the predetermined number of revolutions. Once the compaction was completed, the angle was removed and the hydraulic ram was retracted. The specimens were then extracted and left to cool as shown in Figure 3.11.

Figure 3.10 Superpave gyratory compactor ready to compact a specimen

Next, the bulk specific gravity and maximum specific gravity of the mix were determined by AASHTO tests T166 and T209 respectively. The compaction data for the three trial blends are shown in Table 3.5. The fine aggregate blend was selected from the three blends due to its compactibility (final height), air voids, and bulk specific gravity.

The values for these properties indicated that this blend would satisfy Superpave 5% air void requirements after determining the optimum binder content.

Table 3.5 Mix design trial specimen results

Trial Specimen		Coarse 1	Coarse 2	Int. 1	Int. 2	Fine 1	Fine 2
% Binder	%	5	5	5	5	5	5
Dry Mass	(g)	4587.2	4582.9	4594.2	4588.4	4591.6	4584.4
Wet Mass	(g)	2578	2574.8	2592.5	2595.7	2613.4	2594.2
SSD Mass	(g)	4626.5	4619.4	4625.8	4620.1	4624.6	4618.8
G _{mm}	g/cm ³	2.401	2.401	2.392	2.392	2.408	2.408
Height @N _{des}	mm	118.19	118.44	117.3	116.64	115.98	116.94
G _{mb} @ N _{des} est.	g/cm ³	2.196	2.190	2.216	2.226	2.240	2.218
Corr G _{mb} @ N _{des} est.	g/cm ³	2.217	2.219	2.232	2.239	2.255	2.235
Corr. Air Voids @ N _{des}	%	7.67	7.58	6.70	6.41	6.37*	7.17*

*Superpave recommends 5.0% air voids which was obtained after determining the optimum binder content.

Figure 3.11 Compacted trial specimen

3.6.3 Optimum Binder Content

With an aggregate blend selected, the optimum binder content needed to be selected. This was done by preparing specimens of varying binder content. Superpave recommends preparing two specimens with a binder content of $\pm 0.5\%$ and $+ 1.0\%$ of the

estimated binder content. After evaluating specimens, a 5.75% optimum binder content was selected since it met the 5% air void requirement. This binder content was used for all subsequent specimen preparations for moisture susceptibility evaluation.

3.6.4 Additive Application Methods

With a completed mix design, ranges of additives to be tested were determined based on previous research and additive lime content. Asphalt additives were introduced to the asphalt mix by three separate methods. The first method consists of adding the mineral filler additives to saturated surface-dry aggregates. This was done by first weighing oven-dry aggregates according to the mix design, but waiting to add material passing the No. 200 sieve. The aggregates were thoroughly mixed with approximately 5% water of their total weight as shown in Figure 3.12. The correct percentage by total mix weight of fly ash, CKD, or lime additive was added and mixed. Lastly, any remaining fine aggregate mineral filler was added and mixed according to the mix design and specimens were placed in the oven.

Figure 3.12 Adding water to aggregates before additive introduction

The second method of additive introduction was the slurry method. This was done according to Caltrans standard procedures in which one part additive is mixed with two parts water, forming a slurry. The slurry was then added to moisture saturated-surface-dry coarse and fine aggregates separately as shown in Figure 3.13 and the specimens were left to marinate for 24 hours. After 24 hours, the specimens were then transferred to the oven to dry before mixing with asphalt binder.

Figure 3.13 Adding slurry to SSD aggregate

The third introduction method investigated in this thesis was adding additives directly to the asphalt binder. This was done simply by adding the correct amount of mineral filler or liquid anti-stripping additive to heated asphalt binder and thoroughly mixing as shown in Figure 3.14. Then the binder/additive mixture was added to the aggregates and mixed as usual.

Figure 3.14 Adding mineral filler additive to asphalt binder

Several different concentrations were tested for each additive, which are displayed in Table 3.6. After the saturated-surface-dry method was conducted, the optimum additive concentration for each additive was selected based its tensile strength ratio and index of retained strength. Optimum concentrations were then tested using the slurry and dry application methods as shown in Table 3.7.

Table 3.6 Additive concentrations tested for the saturated-surface-dry method

Additive % for SSD Method				
Fly Ash Class C	Fly Ash Class F	Cement Kiln Dust	Hydrated Lime	HP Plus
1%	1%	1%	1%	0.25%
3%	3%	3%	1.5%	0.50%
5%	5%	5%	2%	0.75%
7%	7%	7%		

Table 3.7 Additive concentrations tested for slurry and dry methods

Additive % for Slurry and Dry Methods			
Fly Ash Class C	Fly Ash Class F	Cement Kiln Dust	Hydrated Lime
5% Slurry	5% Slurry	3% Slurry	1.5% Slurry
5% Dry	5% Dry	3% Dry	1.5% Dry

3.7 Moisture Sensitivity Tests and Results

3.7.1 *Modified Lottman Indirect Tensile Test (AASHTO T 283)*

As stated in Chapter 2, the modified-Lottman indirect tensile test is an incorporated step in the Superpave mix design. Test specimens were produced and compacted in a 100mm (4 in) diameter mold to air voids of approximately 7% and a height of approximately 63.5 mm (2.5 in) according to AASHTO specifications. Specimens were weighed and separated into unconditioned and conditioned sets according to average air voids. The unconditioned specimens were set aside while the conditioned specimens were vacuum saturated and placed in a freezer for a minimum of 16 hours. After 16 hours, conditioned specimens were placed in a 160° C hot water bath for 24 hours and then in a 25° C bath with the unconditioned specimens for 2 hours as shown in Figure 3.15.

Figure 3.15 Asphalt specimens conditioning in water bath

Once specimens were at testing temperature, they were removed from the water bath and placed in the steel loading apparatus in the hydraulic test machine as shown in Figure 3.16. The indirect tensile strength (ITS) for each specimen was calculated using the following equation:

$$S_t = 2P/(\pi tD)$$

Where:

S_t = tensile strength (psi);

P = maximum force placed on specimen during loading (lbs);

t = specimen thickness (in);

D = specimen diameter (in).

Figure 3.17 illustrates a typical cross-section of a specimen after an indirect tensile test.

Final results are included in the following section.

Figure 3.16 Specimen ready to be tested for tensile strength

Figure 3.17 Specimen broken apart by indirect tensile test

3.7.2 Modified Lottman Indirect Tensile Test Results

Tables 3.8 and 3.9 show the results of compacted asphalt specimens tested for tensile strength. The average strengths for both the unconditioned and moisture-conditioned sets are shown in Tables 3.8 and 3.9 along with the tensile strength ratios (TSR's). Tensile strength ratios represent the proportion of tensile strength retained between the moisture damaged and unconditioned sets of a specific additive concentration. TSR's were calculated using the following equation:

$$\text{Tensile Strength Ratio (TSR)} = S_2/S_1$$

Where:

S_2 = average tensile strength of the conditioned (moisture damaged) set (psi);

S_1 = average tensile strength of the unconditioned set (psi).

The moisture damaged tensile strength of each additive concentration was also compared with the unconditioned control tensile strength using the following equation:

$$\text{Tensile Strength Ratio (TSR)} = S_2/S_{1C}$$

Where:

S_2 = average tensile strength of the conditioned (moisture damaged) set

S_{1C} = average tensile strength of the unconditioned **control** set

All mineral filler additives in Table 3.8 were introduced to saturated surface-dry aggregate and HP Plus was added directly to the binder.

Table 3.8 Modified Lottman IDT test results for SSD prepared specimens

Additive %	Dry Tensile Strength psi	Wet Tensile Strength psi	TSR	TSR compared to control
Control				
0%	372.8	290.3	0.78	0.78
Class C				
1%	337.0	280.0	0.83	0.75
3%	397.7	428.5	1.08	1.15
5%	363.7	404.2	1.11	1.08
7%	364.0	348.5	0.96	0.93
Class F				
1%	352.1	296.2	0.84	0.79
3%	337.1	341.2	1.01	0.92
5%	345.5	378.8	1.10	1.02
7%	321.3	357.3	1.11	0.96
CKD				
1%	358.9	292.6	0.82	0.79
3%	331.1	331.1	1.00	0.89
5%	351.5	338.7	0.96	0.91
7%	329.9	333.9	1.01	0.90
Lime				
1.0%	338.9	363.1	1.07	0.97
1.5%	314.6	355.5	1.13	0.95
2.0%	337.3	387.3	1.15	1.04
HP Plus				
0.25%	334.1	322.1	0.96	0.86
0.50%	316.1	302.0	0.96	0.81
0.75%	335.2	346.1	1.03	0.93

Table 3.9 displays the results from testing two other additive introduction methods; namely slurry marination method and the dry additive to binder method. The percentages tested for these two methods were determined from optimum tensile strength ratios achieved by the saturated-surface-dry method as described in 3.6.4.

Table 3.9 Modified Lottman IDT test results for slurry and dry preparation methods

Additive % Method	Dry Tensile Strength psi	Wet Tensile Strength psi	TSR	TSR compared to control
Class C				
5% Slurry	287.6	278.1	0.97	0.75
5% Dry	272.2	263.5	0.97	0.71
Class F				
5% Slurry	315.9	296.9	0.94	0.80
5% Dry	262.5	254.4	0.97	0.68
CKD				
3% Slurry	286.3	290.2	1.01	0.78
3% Dry	270.2	251.6	0.93	0.67
Lime				
1.5% Slurry	320.0	288.9	0.90	0.78
1.5% Dry	288.8	270.1	0.94	0.72

3.7.3 Immersion Compression Test (ASTM D 1075)

The immersion compression test, while not required by Superpave, was also included in this study for determining the moisture susceptibility of the asphalt mixtures. Slightly taller specimens of about 100 mm (4 in) in height were produced and compacted according to AASHTO specifications. Standard test procedure for this test did not require a freeze thaw cycle, so specimens were conditioned by only placing them in a 160° C bath for 24 hours. After conditioning, they were placed in the 25° C bath with the unconditioned specimens for 4 hours before testing.

Figure 3.18 Specimen being tested for compressive strength

Compression testing simply involved removing the specimens from the cold water bath and placing them between the flat steel plates of the hydraulic test machine as shown in Figure 3.18. Compressive strengths and displacements were recorded for all specimens. Results are included in the following section.

3.7.4 Immersion Compression Test Results

Tables 3.10 and 3.11 present the results of immersion compression test specimens. The average compressive strength for both unconditioned and moisture damaged specimens, along with the index of retained strength (IRS), which is the ratio between moisture damaged and unconditioned specimens, is shown. Also, the adjusted index of retained strength between moisture damaged specimens and unconditioned control, as well as the elastic modulus is included in the table. Concentrations used to test the slurry and dry application methods were determined using the optimum IRS as described in section 3.6.4.

Table 3.10 Immersion compression test results for SSD prepared specimens

Additive % Concentration	Uncond. Comp. Strength (psi)	Conditioned Comp. Strength (psi)	IRS	IRS compared to control	Uncond. Elastic Modulus (psi)	Conditioned Elastic Modulus (psi)
Control						
0%	988.6	963.4	0.97	0.97	62773	63289
Class C						
1%	857.5	862.8	1.01	0.87	50308	47079
3%	1080.8	1080.8	1.01	1.09	76146	76450
5%	1238.1	1238.1	1.03	1.25	85910	102591
7%	1255.9	1197.8	0.95	1.21	98810	76402
Class F						
1%	1066.1	1022.6	0.96	1.03	62880	53968
3%	1153.5	1185.7	1.03	1.20	80416	92014
5%	1037.2	1187.2	1.14	1.20	71643	99310
7%	902.1	1013.4	1.12	1.03	64012	81547
CKD						
1%	1164.5	1110.1	0.95	1.12	86040	77094
3%	923.0	1115.5	1.21	1.13	66122	96269
5%	940.0	1069.7	1.14	1.08	72696	91273
7%	1132.8	1073.1	0.95	1.09	100107	84941
Lime						
1.0%	1169.3	1163.9	1.00	1.18	89366	73049
1.5%	1232.9	1201.1	0.97	1.22	77813	72446
2.0%	1239.3	1158.0	0.93	1.17	82349	75457
HP Plus						
0.25%	964.6	1142.7	1.18	1.16	60206	73306
0.50%	915.7	1007.9	1.10	1.02	62994	70201
0.75%	1026.3	994.7	0.97	1.01	62515	54637

Table 3.11 Immersion compression test results for slurry and dry preparation methods

Additive % Method	Uncond. Comp. Strength (psi)	Conditioned Comp. Strength (psi)	IRS	IRS compared to control	Uncond. Elastic Modulus (psi)	Conditioned Elastic Modulus (psi)
Class C						
5% Slurry	654.6	679.5	1.04	0.69	36321	38950
5% Dry	744.6	731.7	0.98	0.74	45812	42769
Class F						
5% Slurry	802.6	823.9	1.03	0.83	50446	52996
5% Dry	570.0	598.3	1.05	0.61	32496	33611
CKD						
3% Slurry	884.5	946.3	1.07	0.96	55865	63255
3% Dry	831.2	788.0	0.95	0.80	51184	45373
Lime						
1.5% Slurry	927.9	889.3	0.96	0.90	51845	51924
1.5% Dry	932.2	917.0	0.98	0.93	52300	56347

3.8 Summary

This chapter discussed the materials and testing methods involved in this study. After obtaining needed materials from their respective sources, the physical and mechanical properties of aggregates and asphalt binder were evaluated in accordance with AASHTO, ASTM, Caltrans, and Superpave specifications. The asphalt binder was then combined with varying concentrations of each additive and tested using a dynamic shear rheometer (DSR) and a rolling thin-film oven (RTFO). Aggregate gradation curves were established for the mix design following Caltrans standards and an optimum binder content was determined to be 5.75% using the Superpave mix design procedure. Using this mix design, varying amounts of each additive were introduced to specimens following three different application methods. Specimens were compacted and tested for indirect tensile and compressive strengths. Results were then organized and tabulated for moisture sensitivity analysis.

CHAPTER 4: Analysis and Discussion

4.1 Introduction

After testing both asphalt binder and compacted specimens, results were tabulated and organized using Excel. Figures were created in order to graphically represent differences among the different additives, concentrations, and application methods. First, DSR and RTFO tests results were analyzed to determine asphalt binder-additive interactions. Then, moisture sensitivity tests for saturated-surface-dry specimens were analyzed to find the optimum combinations shown in Table 3.7. With these combinations, slurry and dry additive application methods were tested and analyzed.

This chapter intends to visually present and discuss the test results of all binder and moisture sensitivity tests conducted in this study. The binder complex modulus and elastic modulus graphs are shown for all additive concentrations tested, followed by a discussion of the results. Due to the number of variables for compacted specimens, graphical representation is divided per additive groups. Concentrations for fly ash and cement kiln dust are included on the same graph, while lime and HP Plus results are displayed on their own graphs. Additive application methods are shown on subsequent graphs. With regard to tensile and compressive strengths, all specimens are compared to unconditioned (not subjected to moisture damage), and virgin (not modified with any additive) specimens. An analysis of the results is incorporated with each group of graphs, supported by an analysis of variance (ANOVA) using a level of significance (α) of 0.05.

4.2 Asphalt Binder

Results from DSR tests for asphalt binder before and after RTFO aging are discussed and analyzed in this section. The most important properties are the complex modulus (comprised of the elastic modulus and viscous modulus) and phase angle. These binder properties, before and after additive modification, are discussed.

4.2.1 Complex Modulus

The complex modulus represents the total amount of resistance an asphalt binder specimen has against deformation. The complex modulus is simply the vector summation of both the elastic and viscous portions of the binder. Generally, the higher the complex modulus, the stiffer the binder will be against deformation. Figure 4.1 shows the relationship between additive concentration and complex modulus for different types of additives.

Figure 4.1 Complex modulus of asphalt binder specimens before RTFO conditioning

From the DSR tests, the complex modulus for the control specimens (no additives) was 0.203 psi, which when divided by the sine of the phase angle, meets the Superpave minimum requirement of 0.145 psi. The above graph shows the control results as a straight line in order to compare with the varying additive concentrations. It appears the liquid, HP Plus, had a significant softening effect on the asphalt binder with a complex modulus of around 0.030 psi. This explains the difficulty encountered while loading specimens into the DSR, which quickly began melting. It is normal for liquid anti-stripping additives to soften binder; however, HP Plus seemed to have a more pronounced effect.

Alternatively, the mineral fillers had a stiffening effect on asphalt binder, which was confirmed by these results. Also, all mineral fillers tended to further increase binder stiffness as concentration increased. Lime appeared to stiffen binder the most given its concentration, increasing nearly linearly from 0.333 psi to 0.702 psi, and then 0.952 psi for 1%, 2%, and 3% concentrations, respectively. This could be due to fineness of lime particles. All of the mineral fillers tested between 1% and 7% tended to have a more pronounced stiffening effect at higher concentrations, denoted by the increasing slope. Class C fly ash stiffened the binder the least when compared to class F fly ash and CKD, except at the 1% concentration. CKD had the highest overall stiffening effect, having a complex modulus of 1.406 psi at its 7% concentration. The stiffening effect caused by these additives is likely due to their spherical particle shape and fineness, but may also be related to their chemical composition since they contain some amount of lime.

Figure 4.2 Complex modulus of asphalt binder specimens after RTFO conditioning

After conditioning specimens in a RTFO, DSR tests were again conducted, producing the results displayed in Figure 4.2. Some changes were observed in the complex modulus compared to unconditioned specimens. First, the control specimens doubled in stiffness to 0.456 psi, which continues to meet Superpave requirements. This was also the case for binder modified by HP Plus for the additive percentages investigated in this study.

The complex modulus for all mineral fillers also increased after aging. For the lime concentrations, there seemed to be the greatest increase between 1% and 2% concentrations. Class C fly ash, class F fly ash, and CKD specimens exhibited trends similar to that of un-aged (non-RTFO) binder, except all specimens exhibited an increase in stiffness.

4.2.2 Elastic Modulus

The elastic modulus of asphalt binder represents how well the binder can rebound to its original shape after removing a load. Using an asphalt binder with a higher elastic modulus generally means an asphalt mix will be stiffer, more readily rebounding after loading, which helps reduce rutting in the pavement's life. Figures 4.3 and 4.4 present the relationship between additive concentration and elastic modulus for all binder combinations before and after RTFO conditioning, respectively.

Figure 4.3 Elastic modulus of asphalt binder specimens before RTFO conditioning

The DSR results revealed trends for the elastic modulus similar to those of the complex modulus. The control asphalt binder specimens were tested to have an elastic modulus of 0.006 psi. It appears that the elastic modulus increased for all mineral filler specimens, while it decreased for HP Plus specimens. This is again typical for both additive types. It can also be seen that adding higher concentrations of additive increased

the elastic modulus, especially with the mineral fillers, except for the 1% class C fly ash concentration. Class C fly ash specimens seem to exhibit a higher elastic modulus than all other additives at this concentration. The other mineral fillers display changes in elastic modulus several times higher than the control. At 1%, lime begins with an elastic modulus of .011 psi, but rapidly increases to 0.051 psi at 3% concentration. Class F fly ash and CKD specimens exhibit the same trends for elastic modulus as they did for complex modulus with CKD having the largest elastic modulus of 0.070 psi at 7% concentration.

Figure 4.4 Elastic modulus of asphalt binder specimens after RTFO conditioning

After RTFO aging, specimens exhibited similar trends as before, except the elastic moduli increased for all variations. Lime specimens appeared to only increase from 0.188 psi to 0.203 psi between 2% and 3%. Also, 5% class C fly ash and 5% class F fly ash specimens had nearly the same value. At the 1% concentration, class C fly ash

specimens still have the highest elastic modulus. However, for HP Plus specimens, the elastic modulus decreased as the additive concentration increased from 0.5% to 1.0%.

4.2.3 Phase Angle

The phase angle represents how elastic or how viscous an asphalt binder is. A phase angle of 0° represents a purely elastic binder, while a 90° phase angle represents a purely viscous binder. Table 4.3 shows the phase angle of each additive concentration before and after RTFO conditioning.

Table 4.1 Phase angle of specimens before and after RTFO conditioning

Additive Concentration	Phase Angle °	
	Before RTFO	After RTFO
Control		
0%	88.2	85.5
Class C Fly Ash		
1%	87.0	83.2
3%	88.2	84.5
5%	87.8	84.2
7%	87.2	84.6
Class F Fly Ash		
1%	87.7	83.9
3%	87.9	83.3
5%	87.8	84.4
7%	87.2	84.0
CKD		
1%	87.6	83.9
3%	87.4	83.1
5%	87.4	84.4
7%	87.1	84.1
Lime		
1%	88.1	85.3
2%	86.9	83.7
3%	87.0	84.2
HP+		
0.50%	86.8	85.4
0.75%	84.5	86.1
1.00%	81.3	87.1

The phase angle results indicate two important trends. First, the phase angle decreases between 1° and 5° for additives after RTFO aging, except for some

concentrations of HP Plus. This is typical, since binder tends to become less viscous after aging. Also, the phase angle decreases a few degrees when an additive is added to the binder. This indicates that additives tend to make an asphalt binder more elastic and stiffer in behavior which is beneficial for asphalt pavements. The validity of phase angle results before RTFO aging for HP Plus 0.75% and 1% concentrations are questionable, since these concentrations were unstable during testing.

4.3 Analysis of Compacted HMA Test Results

4.3.1 Tensile Strength

4.3.1.1 Saturated-Surface-Dry (SSD) Preparation Method

Tensile strengths for unconditioned and moisture conditioned specimens were averaged and organized to produce the following figures. Also, an analysis of variance was conducted between the mean strength of each additive combination and compared with the control mix using an alpha of 0.05. This helped determine any significant statistical difference among the tested combinations. Figures 4.5, 4.6, and 4.7 compare tensile strength for specimens with different additive concentrations to the control tensile strength. While the graphs show that every combination except for 3% class C fly ash having a lower tensile strength than the control, the ANOVA analysis shown in Table 4.2 revealed no significant difference among the specimen means.

Figure 4.5 Tensile strength for unconditioned HMA treated with different additives

Figure 4.6 Tensile strength for unconditioned HMA treated with lime

Figure 4.7 Tensile strength for unconditioned HMA treated with HP Plus

Table 4.2 Summary of ANOVA for tensile strengths of unconditioned HMA (SSD application)

Additive %	P-value	Significance
$\alpha = 0.05$, No significance if p-value > 0.05		
Class C		
1	0.295	No
3	0.396	No
5	0.833	No
7	0.772	No
Class F		
1	0.519	No
3	0.239	No
5	0.383	No
7	0.143	No
CKD		
1	0.639	No
3	0.243	No
5	0.459	No
7	0.212	No
Lime		
1	0.268	No
1.5	0.183	No
2	0.250	No
HP Plus		
0.25	0.281	No
0.5	0.120	No
0.75	0.288	No

The water-conditioned specimens subjected to a freeze-thaw cycle were also tested for tensile strength. The results reported different findings as compared to unconditioned specimens. The tensile strength of the control batch dropped from 372.8 psi to 290.3 psi after conditioning, indicating an inflicted degree of moisture damage to specimens. When comparing the tensile strength means for all saturated-surface-dry HMA combinations to that for the moisture-conditioned control mix using an ANOVA (Table 4.4), a few combinations showed a significant difference. As Figure 4.10 indicates, both 3% and 5% concentrations of class C fly ash demonstrated tensile strengths of more than 100 psi over control. Also, HMA treated with 5% class F fly ash showed significantly higher strengths. HMA treated with hydrated lime, as presented in Figure 4.11, shows a higher strength over control, however only the 2% concentration was determined to be significant. HMA treated with HP Plus shows higher tensile strengths than control in Figure 4.12; however they were determined to be insignificant.

Table 4.3 Summary of ANOVA for tensile strengths of conditioned HMA (SSD application)

Additive %	P-value	Significance
$\alpha = 0.05$, No significance if p-value > 0.05		
Class C		
1	0.729	No
3	0.010	Yes
5	0.027	Yes
7	0.099	No
Class F		
1	0.823	No
3	0.113	No
5	0.024	Yes
7	0.063	No
CKD		
1	0.932	No
3	0.187	No
5	0.162	No
7	0.162	No
Lime		
1	0.095	No
1.5	0.082	No
2	0.018	Yes
HP Plus		
0.25	0.271	No
0.5	0.690	No
0.75	0.118	No

Figure 4.8 Tensile strength for conditioned HMA treated with different additives

Figure 4.9 Tensile strength for conditioned HMA treated with lime

Figure 4.10 Tensile strength for conditioned HMA treated with HP Plus

4.3.1.2 *Slurry and Dry Application Methods*

When comparing the tensile strength of unconditioned specimens prepared using the slurry and dry application methods with control strengths, there appeared to be a significant difference. The ANOVA shown in Table 4.3 indicates a significant difference in tensile strength for both the slurry and dry application method of class C fly ash and CKD compared to control. HMA treated with class F and hydrated lime also exhibited a significant difference in tensile strength for the dry method. Figures 4.8 and 4.9 demonstrate a reduction in strength for both slurry and dry methods when compared to the tensile strength of control saturated-surface-dry specimens.

Table 4.4 Summary of ANOVA for tensile strengths of unconditioned HMA (slurry & dry methods)

Additive %	P-value $\alpha = 0.05$, No significance if p-value > 0.05	Significance
Class C		
5% Slurry	0.031	Yes
5% Dry	0.019	Yes
Class F		
5% Slurry	0.117	No
5% Dry	0.013	Yes
CKD		
3% Slurry	0.036	Yes
3% Dry	0.019	Yes
Lime		
1.5% Slurry	0.119	No
1.5% Dry	0.033	Yes

Figure 4.11 Tensile strength for unconditioned HMA treated with different additive application methods

Figure 4.12 Tensile strength for unconditioned HMA treated with different lime application methods

For the conditioned tensile strength of HMA specimens subjected different application methods, no significant difference was detected compared to the control tensile strength as indicated by Table 4.5. Figures 4.13 and 4.14 indicate that the tensile strength of specimens subjected to the slurry method maintain strengths similar to that of the control tensile strength. Specimens subjected to the dry method exhibited slightly lower tensile strengths compared to control. However, all slurry and dry method specimens exhibited significantly different tensile strengths when compared to the saturated-surface-dry method as Table 4.6 shows. This agrees with the decrease in strength observed in Figures 4.13 and 4.14 when comparing the saturated-surface-dry strengths with the slurry and dry strengths.

Table 4.5 Summary of ANOVA for tensile strengths of conditioned HMA (slurry & dry methods)

Additive %	P-value $\alpha = 0.05$, No significance if p-value > 0.05	Significance
Class C		
5% Slurry	0.683	No
5% Dry	0.344	No
Class F		
5% Slurry	0.811	No
5% Dry	0.313	No
CKD		
3% Slurry	0.998	No
3% Dry	0.210	No
Lime		
1.5% Slurry	0.958	No
1.5% Dry	0.460	No

Figure 4.13 Tensile strength for conditioned HMA treated with different additive application methods

Figure 4.14 Tensile strength for conditioned HMA treated with different lime application methods and control

Table 4.6 Summary of ANOVA for tensile strengths of slurry and dry HMA specimens compared to SSD specimens

Additive %	P-value $\alpha = 0.05$, No significance if p-value > 0.05	Significance
Class C		
5% Slurry	0.008	Yes
5% Dry	0.004	Yes
Class F		
5% Slurry	0.001	Yes
5% Dry	0.003	Yes
CKD		
3% Slurry	0.014	Yes
3% Dry	0.002	Yes
Lime		
1.5% Slurry	0.010	Yes
1.5% Dry	0.004	Yes

4.3.2 Tensile Strength Ratios (TSR)

Tensile strength ratio was another method used to analyze the indirect tensile strength data. The tensile strength ratio evaluated the amount of tensile strength retained after subjecting specimens to a freeze-thaw cycle as described in chapter 3. While the TSR for control specimens failed to meet the Superpave threshold of 0.80, all additive concentrations for the saturated-surface-dry specimens exceeded the Superpave requirement. An ANOVA was conducted to determine any significant difference in tensile strength after moisture damage conditioning, resulting in Table 4.7. TSR's for HMA treated with class C fly ash, as shown in Figure 4.15, continued to increase up to the 5% concentration before decreasing at 7%. Class F fly ash specimens tended to keep increasing tensile strength, although only a slight increase was seen between 5% and 7%. For CKD treatments, the TSR increased between 1% and 3% concentrations, and then decreased for 5%. For the 7% CKD concentration, additional binder was added during the mixing process to produce compactable specimens, so the increase in the TSR could be due to the added binder. HMA treated with hydrated lime showed a steady increase in its

TSR with an increasing concentration as shown in Figure 4.16, while for HP Plus treatments; TSR's seemed to only increase at 0.75% concentration as shown in Figure 4.17.

Table 4.7 Summary of ANOVA for TSR's of HMA (SSD application)

Additive %	P-value $\alpha = 0.05$, No significance if p-value > 0.05	Significance
Class C		
1	0.045	Yes
3	0.171	No
5	0.353	No
7	0.417	No
Class F		
1	0.017	Yes
3	0.580	No
5	0.048	Yes
7	0.079	No
CKD		
1	0.006	Yes
3	0.999	No
5	0.438	No
7	0.438	No
Lime		
1	0.126	No
1.5	0.005	Yes
2	0.542	No
HP Plus		
0.25	0.542	No
0.5	0.478	No
0.75	0.645	No

Figure 4.15 TSR's for HMA treated with SSD aggregates and different additives

Figure 4.16 TSR's for HMA treated SSD aggregates and lime

Figure 4.17 TSR's for HMA treated with HP Plus

All TSR's for other application methods were also able to exceed the Superpave minimum TSR. Most were able to retain at least 90% of their unconditioned strength, with little difference appearing between slurry and dry methods, as shown in Figures 4.18 and 4.19. According to ANOVA Table 4.8, only the 1.5% lime slurry treated specimens revealed a significant difference in tensile strength after moisture conditioning.

Figure 4.18 TSR's for HMA treated with different additive application methods

Figure 4.19 TSR's for HMA treated with different lime application methods

Table 4.8 Summary of ANOVA for TSR's of HMA (slurry & dry methods)

Additive %	P-value $\alpha = 0.05$, No significance if p-value > 0.05	Significance
Class C		
5% Slurry	0.536	No
5% Dry	0.375	No
Class F		
5% Slurry	0.279	No
5% Dry	0.696	No
CKD		
3% Slurry	0.771	No
3% Dry	0.197	No
Lime		
1.5% Slurry	0.000	Yes
1.5% Dry	0.065	No

While the TSR standard calculation method is the current method used by pavement engineers, it does not accurately represent specimens that might be weaker in strength overall (both conditioned and unconditioned). TSR's were recalculated for all combinations compared to the unconditioned control to offer a more critical analysis of TSR's. Tables 3.8 and 3.9 in the previous chapter display the adjusted TSR's, showing a significant reduction in TSR's for several combinations. Only four combinations were able to retain TSR's higher than 1.00, which were 3% class C fly ash, 5% class C fly ash, 5% class F fly ash, and 2% lime. These combinations were the same combinations which displayed statistical significance for conditioned tensile strength. All of the 1% saturated-surface-dry concentrations, except lime, failed to meet the 0.80 ratio. This is further reinforced by ANOVA Table 4.9, which indicates a significant difference in tensile strength for these combinations. Specimens treated with 5% class C fly ash and 1.5% lime no longer exhibit a significant difference in strength as they did in Table 4.7. Also, 5% class F slurry is the only alternative application method that meets Superpave requirements. ANOVA Table 4.10 indicates a significant difference in strength for all

alternative application specimens. Figures for the adjusted TSR's exactly replicate the relationships shown Figures 4.8, 4.9, 4.10, 4.13, and 4.14; however Figures 4.20 through 4.24 are displayed for visual representation.

Table 4.9 Summary of ANOVA for adjusted TSR's of HMA (SSD application)

Additive %	P-value $\alpha = 0.05$, No significance if p-value > 0.05	Significance
Class C		
1	0.031	Yes
3	0.146	No
5	0.408	No
7	0.438	No
Class F		
1	0.040	Yes
3	0.295	No
5	0.827	No
7	0.599	No
CKD		
1	0.040	Yes
3	0.193	No
5	0.309	No
7	0.309	No
Lime		
1	0.586	No
1.5	0.609	No
2	0.123	No
HP Plus		
0.25	0.123	No
0.5	0.067	No
0.75	0.410	No

Figure 4.20 Adjusted TSR's for HMA treated with different additives

Figure 4.21 Adjusted TSR's for HMA treated with lime

Figure 4.22 Adjusted TSR's for HMA treated with HP Plus

Table 4.10 Summary of ANOVA for adjusted TSR's of HMA (slurry & dry methods)

Additive %	P-value	Significance
$\alpha = 0.05$, No significance if p-value > 0.05		
Class C		
5% Slurry	0.030	Yes
5% Dry	0.014	Yes
Class F		
5% Slurry	0.047	Yes
5% Dry	0.021	Yes
CKD		
3% Slurry	0.035	Yes
3% Dry	0.011	Yes
Lime		
1.5% Slurry	0.031	Yes
1.5% Dry	0.016	Yes

Figure 4.23 Adjusted TSR's for HMA treated with different additive application methods

Figure 4.24 Adjusted TSR's for HMA treated with different lime application methods

4.3.3 Compressive Strength

4.3.1.3 Saturated-Surface-Dry (SSD) Preparation Method

From the ASTM D1075 compression test, results were organized to produce Tables 3.10 and 3.11. An analysis of variance (ANOVA) was also conducted on these test results to help determine statistical significance. Figures 4.25, 4.26, and 4.27 compare the unconditioned compressive strength for HMA specimens with the control using the saturated-surface-dry application method for fly ash, CKD, and lime. It can be observed that HMA with different mineral filler additives achieves the highest compressive strength at different concentrations. The compressive strength for HMA treated with class C fly ash seems to continually increase to 7%, while the maximum compressive strength for HMA treated with Class F fly ash is around 3%. HMA with 5% and 7% class C fly ash have a significantly higher strength when compared to control, as shown in ANOVA Table 4.11. Mixes treated with CKD seem to have the highest compressive strength at the 1% concentration; however strength is less for 3% and 5% but increases again at 7%. Only the strength for HMA treated with 1% CKD was determined to be significant. Hydrated lime appears to have a significantly high compressive strength for 1.5% and 2% concentrations. Lastly, little difference in strength was found for HP Plus as compared to control, which is demonstrated in Figure 4.27.

Figure 4.25 Compressive strength for unconditioned HMA treated with different additives

Figure 4.26 Compressive strength for unconditioned HMA treated with hydrated lime

Figure 4.27 Compressive strength for unconditioned HMA treated with HP Plus

Table 4.11 Summary of ANOVA for compressive strengths of unconditioned HMA (SSD application)

Additive %	P-value	Significance
$\alpha = 0.05$, No significance if p-value > 0.05		
Class C		
1	0.179	No
3	0.246	No
5	0.017	Yes
7	0.036	Yes
Class F		
1	0.344	No
3	0.111	No
5	0.551	No
7	0.406	No
CKD		
1	0.026	Yes
3	0.309	No
5	0.377	No
7	0.217	No
Lime		
1	0.091	No
1.5	0.048	Yes
2	0.010	Yes
HP Plus		
0.25	0.686	No
0.5	0.312	No
0.75	0.483	No

A set of specimens were conditioned in a hot water bath for 24 hours and tested for compressive strength. Comparative results for the saturated-surface-dry application method are shown in Figures 4.28 and 4.29. Figure 4.30 shows comparative results for HP Plus. Trends similar to the unconditioned compressive strength are apparent. HMA specimens treated with Class C fly ash again exhibit a significantly greater compressive strength at 5% and 7% concentrations than control. However, HMA specimens treated with class F fly ash and CKD did not exhibit any significant difference in strength compared to control as indicated by ANOVA Table 4.12. Hydrated lime shows an increase in strength for all concentrations, however only 1.5% and 2% were determined to be significant since 1% results contained more variation. HP Plus shows only a slight increase in tensile strength over control.

Figure 4.28 Compressive strength for conditioned HMA treated with different additives

Figure 4.29 Compressive strength for conditioned HMA treated with hydrated lime

Figure 4.30 Compressive strength for unconditioned HMA treated with HP Plus

Table 4.12 Summary of ANOVA for compressive strengths of conditioned HMA (SSD application)

Additive %	P-value $\alpha = 0.05$, No significance if p-value > 0.05	Significance
Class C		
1	0.192	No
3	0.278	No
5	0.026	Yes
7	0.042	Yes
Class F		
1	0.438	No
3	0.063	No
5	0.228	No
7	0.509	No
CKD		
1	0.103	No
3	0.097	No
5	0.233	No
7	0.206	No
Lime		
1	0.075	No
1.5	0.031	Yes
2	0.044	Yes
HP Plus		
0.25	0.066	No
0.5	0.609	No
0.75	0.695	No

4.3.1.4 Slurry and Dry Application Methods

The unconditioned compressive strength of the different additive application methods are compared in Figures 4.31 and 4.32. Similar to the results for tensile strength, compressive strengths for the slurry and dry methods were less than those for the saturated-surface-dry method. HMA treated with Class C fly ash exhibited the most significant decrease in strength for both methods, along with class F fly ash as indicated in Table 4.13. CKD specimens experienced a significant reduction in compressive strength for the dry method only. Lime was the only additive that didn't experience a significant reduction in strength as compared with control strengths.

Figure 4.31 Compressive strength for unconditioned HMA treated with different additive application methods

Figure 4.32 Compressive strength for unconditioned HMA treated with different lime application methods

Table 4.13 Summary of ANOVA for compressive strengths of unconditioned HMA (slurry & dry methods)

Additive %	P-value $\alpha = 0.05$, No significance if p-value > 0.05	Significance
Class C		
5% Slurry	0.002	Yes
5% Dry	0.007	Yes
Class F		
5% Slurry	0.021	Yes
5% Dry	0.001	Yes
CKD		
3% Slurry	0.120	No
3% Dry	0.035	Yes
Lime		
1.5% Slurry	0.420	No
1.5% Dry	0.359	No

Compressive strengths for conditioned HMA specimens treated with different additives were less than those for control. HMA treated with Class C fly ash showed significantly less strength for both slurry and dry methods as shown in Figure 4.33 and supported by Table 4.14. HMA treated with Class F fly ash also exhibited less compressive strength for the dry method, while CKD and lime specimens (Figure 4.34) experienced no significant reduction as compared to the control mix as indicated in Table 4.10.

Figure 4.33 Compressive strengths for conditioned HMA treated with different additive application methods

Figure 4.34 Compressive strengths for conditioned HMA treated with different lime application methods

Table 4.14 Summary of ANOVA for compressive strengths of conditioned HMA (slurry & dry methods)

Additive %	P-value $\alpha = 0.05$, No significance if p-value > 0.05	Significance
Class C		
5% Slurry	0.011	Yes
5% Dry	0.033	Yes
Class F		
5% Slurry	0.095	No
5% Dry	0.005	Yes
CKD		
3% Slurry	0.802	No
3% Dry	0.101	No
Lime		
1.5% Slurry	0.315	No
1.5% Dry	0.577	No

4.3.4 Index of Retained Strength (IRS)

The index of retained strength of the immersion compression test is similar to the tensile strength ratio of the indirect tensile test. It represents the strength ratio between conditioned and unconditioned specimens of the same additive concentration. While the minimum acceptable index of retained strength is 0.70, a control mix value of 0.97 was achieved from testing. This control value was used to compare compacted HMA specimens treated with different additives. ANOVA Table 4.15 indicates a significant difference in retained strength after moisture conditioning for specimens treated with 3% CKD, 5% CKD, and 2% hydrated lime. Figures 4.35, 4.36, and 4.37 visually compare the IRS values for the saturated-surface-dry application specimens, as well as HP Plus specimens. The IRS is noticeably over 1.00 for specimens treated with 3% and 5% CKD and specimens treated with 5% and 7% class F fly ash. IRS values for specimens treated with hydrated lime tend to show little variation between the different concentrations and the control mix, while HP Plus values decrease with additive concentration.

Table 4.15 Summary of ANOVA for IRS's of HMA (SSD application)

Additive %	P-value $\alpha = 0.05$, No significance if p-value > 0.05	Significance
Class C		
1	0.939	No
3	0.934	No
5	0.675	No
7	0.535	No
Class F		
1	0.507	No
3	0.733	No
5	0.387	No
7	0.256	No
CKD		
1	0.171	No
3	0.010	Yes
5	0.035	Yes
7	0.555	No
Lime		
1	0.712	No
1.5	0.070	No
2	0.013	Yes
HP Plus		
0.25	0.013	Yes
0.5	0.221	No
0.75	0.462	No

Figure 4.35 Index of retained strength for HMA treated with SSD aggregates and different additives

Figure 4.36 Index of retained strength for HMA treated SSD aggregates and lime

Figure 4.37 Index of retained strength for HMA treated with HP Plus

The indices of retained strength for mixes prepared using different application methods are shown in Figures 4.38 and 4.39 indicate little increase or decrease in compressive strengths compared to control. ANOVA Table 4.16 further reinforces this, indicating no significant difference in strength after moisture conditioning. Specimens treated with Class F fly ash and CKD have slightly less compressive strength for slurry and dry methods than the saturated-surface-dry methods.

Figure 4.38 Index of retained strength for HMA treated with different additive application methods

Figure 4.39 Index of retained strength for HMA treated with different lime application methods

Table 4.16 Summary of ANOVA for IRS's of HMA (slurry & dry methods)

Additive %	P-value	Significance
$\alpha = 0.05$, No significance if p-value > 0.05		
Class C		
5% Slurry	0.052	No
5% Dry	0.739	No
Class F		
5% Slurry	0.266	No
5% Dry	0.300	No
CKD		
3% Slurry	0.054	No
3% Dry	0.471	No
Lime		
1.5% Slurry	0.472	No
1.5% Dry	0.774	No

As with tensile strength ratios, the alternate calculation method was conducted for the index of retained strength in an attempt to more accurately compare specimens. This was accomplished by comparing the moisture damaged compressive strength for all specimens to the unconditioned control mix instead of the unconditioned compressive

strength of each respective specimen used in the traditional approach. Tables 3.10 and 3.11 in the previous chapter display the adjusted IRS's. Specimens treated with class C fly ash, class F fly ash, and hydrated lime had a much higher index of retained strength for most concentrations, while values for CKD and HP Plus treated specimens indicated a reduction in IRS for most concentrations. ANOVA Table 4.17 indicates that specimens treated with 5% class C fly ash, 7% class C fly ash, 5% CKD, 1% lime, and 1.5% lime experienced a significant change in retained strength after moisture conditioning when compared to the unconditioned control mix. This contrasts with the significant concentrations revealed by the traditional calculation in Table 4.15. Also, specimens treated with the class C fly ash slurry method and the CKD dry method fail to meet the minimum IRS of 0.70. ANOVA Table 4.18 indicates that all alternative specimens except 3% CKD and lime specimens experienced a significant change in strength. Figures for the adjusted IRS's exactly replicate the relationships shown Figures 4.28, 4.29, 4.30, 4.33, and 4.34; however Figures 4.40 through 4.44 are displayed for visual representation.

Table 4.17 Summary of ANOVA for adjusted IRS's of HMA (SSD application)

Additive %	P-value	Significance
$\alpha = 0.05$, No significance if p-value > 0.05		
Class C		
1	0.062	No
3	0.332	No
5	0.023	Yes
7	0.036	Yes
Class F		
1	0.568	No
3	0.061	No
5	0.261	No
7	0.674	No
CKD		
1	0.095	No
3	0.090	No
5	0.035	Yes
7	0.268	No
Lime		
1	0.024	Yes
1.5	0.033	Yes
2	0.057	No
HP Plus		
0.25	0.057	No
0.5	0.792	No
0.75	0.925	No

Figure 4.40 Adjusted index of retained strength for HMA treated with different additives

Figure 4.41 Adjusted index of retained strength for HMA treated with lime

Figure 4.42 Adjusted index of retained strength for HMA treated with HP Plus

Table 4.18 Summary of ANOVA for adjusted IRS's of HMA (slurry and dry methods)

Additive %	P-value	Significance
$\alpha = 0.05$, No significance if p-value > 0.05		
Class C		
5% Slurry	0.003	Yes
5% Dry	0.013	Yes
Class F		
5% Slurry	0.028	Yes
5% Dry	0.002	Yes
CKD		
3% Slurry	0.435	No
3% Dry	0.048	Yes
Lime		
1.5% Slurry	0.114	No
1.5% Dry	0.328	No

Figure 4.43 Adjusted index of retained strength for HMA treated with different additive application methods

Figure 4.44 Adjusted index of retained strength for HMA treated with different lime application methods

4.3.5 Elastic Modulus

The elastic modulus is an important property that is used in the design and analysis of an HMA layer. This property has a direct impact on how the HMA performs under traffic loading in terms of resistance to permanent deformation and fatigue cracking. Values for the elastic modulus were derived from compression test data spreadsheets. Generally, the elastic modulus values seem to correlate to strength values, as specimens with higher compressive strengths tend to have a higher elastic modulus. Figures 4.45, 4.46, and 4.47 present the elastic modulus for unconditioned specimens treated with various additive concentrations and methods as compared to the control mix elastic modulus. Since the elastic modulus varied greatly, even within the same specimen set, it was more difficult to determine any significant difference between additives as indicated in Table 4.19. Only specimens treated with 7% class C fly ash were determined

to have a significant difference in elastic modulus from the control mix for saturated-surface-dry treatment method.

Figure 4.45 Elastic modulus for unconditioned HMA treated with different additives

Figure 4.46 Elastic modulus for unconditioned HMA treated with lime

Figure 4.47 Elastic modulus for unconditioned HMA treated with HP Plus

Table 4.19 Summary of ANOVA for elastic modulus of unconditioned HMA (SSD application)

Additive %	P-value $\alpha = 0.05$, No significance if p-value > 0.05	Significance
Class C		
1	0.229	No
3	0.265	No
5	0.222	No
7	0.029	Yes
Class F		
1	0.992	No
3	0.210	No
5	0.334	No
7	0.952	No
CKD		
1	0.060	No
3	0.756	No
5	0.371	No
7	0.144	No
Lime		
1	0.134	No
1.5	0.351	No
2	0.194	No
HP Plus		
0.25	0.817	No
0.5	0.980	No
0.75	0.977	No

Just as compressive strength decreased for the slurry and dry alternative treatment methods, the elastic modulus decreased as well as compared to control and saturated-surface-day treatment methods. Specimens treated with class C fly ash for the slurry method and specimens treated with class F fly ash for the dry method had a significantly smaller elastic modulus as shown in Figure 4.48 and Table 4.20. Specimens treated with lime also decreased slightly below the control value for alternative methods, as displayed in Figure 4.49.

Figure 4.48 Elastic modulus for unconditioned HMA treated with different additive application methods

Figure 4.49 Elastic modulus for unconditioned HMA treated with different lime application methods

Table 4.20 Summary of ANOVA for elastic modulus of unconditioned HMA (slurry & dry methods)

Additive %	P-value $\alpha = 0.05$, No significance if p-value > 0.05	Significance
Class C		
5% Slurry	0.040	Yes
5% Dry	0.113	No
Class F		
5% Slurry	0.193	No
5% Dry	0.023	Yes
CKD		
3% Slurry	0.466	No
3% Dry	0.226	No
Lime		
1.5% Slurry	0.360	No
1.5% Dry	0.285	No

The elastic modulus for conditioned specimens showed much variation within the same specimen set as well. Most treated specimens showed an increase in elastic modulus compared to the unconditioned modulus, although some decreased. Figure 4.50 indicates the elastic modulus for conditioned specimens treated with 1% class C fly ash decreased significantly, while 5% class C and 5% CKD treated specimens increased significantly over control values as indicated in Table 4.21. Specimens treated with class F fly ash, lime (Figure 4.51), and HP Plus (Figure 4.52) did not show any significant increase or decrease compared to the control.

Figure 4.50 Elastic modulus for conditioned HMA treated with different additives

Figure 4.51 Elastic modulus for conditioned HMA treated with lime

Figure 4.52 Elastic modulus for conditioned HMA treated with HP Plus

Table 4.21 Summary of ANOVA for elastic modulus of conditioned HMA (SSD application)

Additive %	P-value	Significance
$\alpha = 0.05$, No significance if p-value > 0.05		
Class C		
1	0.049	Yes
3	0.275	No
5	0.037	Yes
7	0.108	No
Class F		
1	0.438	No
3	0.063	No
5	0.228	No
7	0.509	No
CKD		
1	0.092	No
3	0.055	No
5	0.018	Yes
7	0.075	No
Lime		
1	0.229	No
1.5	0.178	No
2	0.420	No
HP Plus		
0.25	0.383	No
0.5	0.418	No
0.75	0.382	No

The alternative application methods again showed a decrease in elastic modulus for the conditioned specimens as compared to control mix and saturated-surface-dry prepared specimens. Both slurry and dry treated class C fly ash specimens showed a decrease in elastic modulus. Dry treated class F fly ash specimens also had a significant decrease as shown in Figure 4.53 and Table 4.22. Lime treated specimens, shown in Figure 4.54, experienced no significant difference for both slurry and dry application methods.

Figure 4.53 Elastic modulus for conditioned HMA treated with different additive application methods

Figure 4.54 Elastic modulus for conditioned HMA treated with different lime application methods

Table 4.22 Summary of ANOVA for elastic modulus of conditioned HMA (slurry & dry methods)

Additive %	P-value $\alpha = 0.05$, No significance if p-value > 0.05	Significance
Class C		
5% Slurry	0.014	Yes
5% Dry	0.029	Yes
Class F		
5% Slurry	0.153	No
5% Dry	0.006	Yes
CKD		
3% Slurry	0.995	No
3% Dry	0.070	No
Lime		
1.5% Slurry	0.109	No
1.5% Dry	0.377	No

4.4 Cost Analysis

Cost is also an important factor in determining which additive to use in HMA preparation. The most cost-effective additive will likely vary from region to region because of availability and transportation costs, as well as binder and aggregate composition. The costs used in this analysis were estimated for asphalt production on the

central coast of California. According to CalPortland Construction, the unit cost of lime is estimated at \$45 per ton, which equates to \$0.023 per pound of lime. Assuming an optimum lime content of 1.5%, the material cost of lime would be \$0.68 per ton of HMA mix. CalPortland also stated that the cost of stockpiling, hydrating, and adding lime to the HMA aggregates would add \$4.00 per ton of HMA. This brings the final cost of adding lime to \$4.68 per ton of HMA (P. Imhoff, personal communication, November 2, 2011).

Headwaters Resources quoted a unit cost of \$72 per ton for class C fly ash, which equates to \$0.036 per pound. Assuming an optimum class C fly ash concentration of 5% in asphalt, the material cost of fly ash would be \$3.60 per ton of HMA mix. After taking aggregate treating costs into consideration, this price increases to \$7.60 per ton of HMA. Headwaters Resources also quoted a class F fly ash unit price of \$62 per ton of class F fly ash, which equates to \$0.031 per pound (J. Seay, personal communication, October 28, 2011). Assuming an optimum class F fly ash concentration of 5% in asphalt, the material cost of fly ash would be \$3.10 per ton of HMA mix. After taking aggregate treating costs into consideration, this price increases to \$7.10 per ton of HMA. CalPortland Construction priced CKD at around \$20 per ton of additive, which assuming an optimum 3% additive concentration, equates to \$0.60 per ton of HMA. Aggregate treating costs increase this price to \$4.60 per ton of HMA.

Lastly for HP Plus, ArrMaz Chemicals quoted price of \$1.55 per pound of additive (P. Whittey, personal communication, October 28, 2011). Using an optimum concentration of 0.75% of the binder weight, the chemical cost comes to \$1.38 per ton of asphalt mix. An in-line system is also required to add liquid anti-stripping agents to the HMA at the plant. These systems typically range between \$10,000 and \$25,000 in initial

cost, which add about \$0.10 to \$0.20 per ton of HMA produced (Epps et al, 2003). This brings the total cost of using HP Plus to \$1.53 per ton of HMA.

According to these values and for the type of aggregate and binder tested in this study, it appears that HP Plus has the least cost of all additives; however it is not as effective as lime or fly ash. CKD is the second least expensive additive at \$4.60 per ton; however it is not very effective and is available in limited quantities. Hydrated lime is the third least expensive additive at \$4.68 per ton of asphalt. Fly ash prices seem to be slightly higher than lime costs; however fly ash may be a viable alternative in areas where lime is more expensive and chemicals like HP Plus are ineffective.

4.5 Chapter Summary

This chapter presented the results from asphalt binder and moisture sensitivity tests. From the DSR and RTFO tests, class C fly ash, class F fly ash, CKD, and hydrated lime all had a stiffening effect on the binder, increasing complex and elastic moduli. Also, with increasing additive concentration, binder stiffness increased further. HP Plus had the opposite effect on the asphalt binder, decreasing both the elastic and complex modulus of the binder. The complex modulus was reduced so much by HP Plus that it fails to meet the minimum Superpave requirement of 0.145 psi. After RTFO aging, similar trends for the additives were observed, except the binder had become much stiffer in all cases, which is typical for this test. The phase angle also decreased for most additive concentrations, making the binder more elastic in nature. Only the 1% HP Plus concentration failed to meet the minimum Superpave requirement of 0.319 psi for RTFO aging.

Compacted specimens were molded and tested using the AASHTO T 283 indirect tensile test. Specimens were compared on the basis of unconditioned tensile strength, conditioned (after moisture damage) tensile strength, and tensile strength ratios. Tensile strength results for unconditioned specimens showed no significant difference in strength, however the ANOVA conducted on tensile strengths for conditioned specimens showed significantly higher strengths for 3% class C fly ash, 5% class C fly ash, 5% class F fly ash, and 2% hydrated lime concentrations. TSR's for every concentration were above the Superpave minimum of 0.80 using the standard calculation method. When calculating TSR's as a ratio of the tensile strength of the unconditioned control mix, only the four combinations mentioned above had TSR's greater than 1.0. A comparison of different additive application methods showed that both slurry and dry methods significantly decreased tensile strength when compared to the saturated-surface-dry method.

The ASTM D 1075 immersion compression test was also conducted on compacted specimens. Specimens were compared using unconditioned compressive strength, compressive strength (after moisture damage), the index of retained strength, and elastic modulus. Specimens treated with 5% class C fly ash, 7% class C fly ash, 1% CKD, 1.5% lime, and 2% lime showed significantly greater unconditioned compressive strengths than control. However after being exposed to moisture damage only 5% class C fly ash, 7% class C fly ash, 1.5% hydrated lime and 2% hydrated lime specimens had significantly higher compressive strength. The compressive strengths for slurry and dry methods seemed to decrease when compared to saturated-surface-dry specimens similar to that of indirect tensile test. The index of retained strength values were over 1.0 for most combinations, such a high index of retained strength may be due to pore water

pressures within the specimens during compression. Trends for the index of retained strength also closely mirrored TSR trends for most additives, showing consistency in the results. Lastly, the elastic moduli for unconditioned and conditioned specimens were compared. Since there was such a large variation of elastic moduli within a specimen set, very few combinations had a significant difference with respect to this property.

After conducting a cost analysis, 0.75% HP Plus had the lowest overall cost of \$1.53 per ton of HMA. While meeting the Superpave minimum TSR of 0.80, it was not as effective as the 1.5% hydrated lime combination of 5% class C fly ash combinations.

CHAPTER 5: Conclusions and Recommendations

5.1 Introduction

This study evaluated the use of different additives to improve the resistance of HMA against moisture-induced damage. Using standardized testing procedures, aggregate was tested for specific gravity, absorption, abrasion resistance, void content, and gradation. Asphalt binder tests were conducted using a dynamic shear rheometer and a rolling thin film oven to first determine if the virgin binder used would meet Superpave requirements. Secondly, these tests were used to determine how each additive interacted with the asphalt binder. For class C fly ash, class F fly ash, and CKD, concentrations of 1%, 3%, 5%, and 7% were tested. Concentrations of 1%, 2%, and 3% lime and 0.5%, 0.75%, and 1.00% HP Plus were also tested. Overall, the mineral fillers had a stiffening effect on binder, while the liquid anti-stripping additive, HP Plus softened the binder.

The HMA was designed according to Caltrans gradation and Superpave mix design requirements. The optimum binder content was determined to be 5.75% of a fine gradation blend, which gave acceptable air void and specific gravity properties.

Moisture sensitivity tests were then conducted on over 350 compacted asphalt specimens. Variations of all the additives were tested for tensile and compressive strength before and after moisture damage conditioning. Fly ash, CKD, and lime additives were added to saturated-surface-dry aggregates during the first round of testing. From these tests, the optimum combination was selected, on which slurry and dry application methods were used. Most additives were able to reduce moisture damage in

the specimens to some degree. However, only a few additives and concentrations were able to significantly reduce moisture damage, as shown by significant differences in tensile and compressive strengths.

5.2 Conclusions

Several conclusions can be drawn from the test results of this study:

- Mineral fillers have a stiffening effect on the asphalt binder according to DSR test results. This was denoted by the increase in both elastic and viscous portions of the complex modulus.
- Higher concentrations of mineral fillers further increase stiffness.
- Of the waste product mineral fillers investigated, CKD produced the greatest overall amount of stiffness, followed by class F fly ash, then class C fly ash.
- Of all the additives tested, lime produced the greatest stiffening effect at most concentrations.
- The liquid anti-stripping additive, HP Plus, had a softening effect on the binder. Generally, increasing the concentration further softened the binder.
- Adding anti-stripping additives to an asphalt mix reduced moisture damage in the laboratory tests. However, certain concentrations of additives had a more significant effect.
- Tensile strength ratios were higher than those for the control for all variations. Lime treated specimens had the highest TSR, followed by fly ash, CKD, and then HP Plus. However, after recomputing TSR's as the ratio of conditioned specimens and unconditioned control, only a few combinations retained more tensile

strength. These are 3% and 5% class C fly ash, 5% class F fly ash, and 2% lime concentrations.

- An ANOVA confirms 3% and 5% class C fly ash, 5% class F fly ash, and 2% lime concentrations as having a statistically significant difference in conditioned tensile strength compared to control.
- No significant difference in tensile strength was determined among the unconditioned specimens.
- The saturated-surface-dry method was determined to produce significantly higher tensile strengths than slurry or dry application methods.
- The index of retained compressive strength was greater than control values in all but a few concentrations. Lime treated specimens had the highest IRS, followed by HP Plus, CKD, class F fly ash, and then class C fly ash specimens. When comparing IRS's of conditioned (moisture damaged) specimens with that of unconditioned control, class C fly ash specimens had the highest IRS, having a higher overall compressive strength, followed by lime, class F fly ash, HP Plus, and then CKD treated specimens.
- Combinations of 5% and 7% class C fly ash and 1.5% and 2% lime were determined to have significantly higher conditioned compressive strengths than control.
- The saturated-surface-dry method had significantly higher compressive strengths than either slurry or dry methods.
- There seemed to be too much variation in elastic modulus to draw a solid conclusion.

- Overall, 5% class C fly ash and 1.5% hydrated lime treated specimens demonstrated the greatest overall resistance to moisture damage.
- Overall, HP Plus was determined to be the least costly additive at \$1.53 per ton of HMA, followed by CKD at \$4.60 per ton of HMA and lime at \$4.68 per ton of HMA. Class F fly ash and class C fly ash were the most expensive, costing \$7.10 and \$7.60 per ton of HMA, respectfully.

5.3 Recommendations

- The scope of this thesis was rather large in terms of the number of tests conducted on additives. Only one set of 6 specimens were produced for each additive concentration. Three specimens were conditioned and three were unconditioned, limiting data analysis only to three specimens of one batch per each variation. Multiple batches of each variation are recommended to ensure batch to batch consistency.
- Tests were only conducted on one mix design, one type of aggregate, and one type of binder. Performance of anti-stripping additives will vary when any one of these mix components are changed. Investigating the effect any of these have on additive performance is recommended.
- Having more aggregate bins, especially a #4 sieve between 3/8" and #8 might better control aggregate gradation as there seemed to be slight proportional inconsistencies within this range when mixing.
- Cost analysis was conducted for central California. It is recommended that another cost analysis be conducted if testing is going to be performed for a different area.

- While HP Plus is the least costly of the additives tested, it is important to note that its effectiveness (as with any other additive) may vary based on aggregate and binder composition. There may be cases where a different chemical additive or lime is more effective.
- Hydrated lime is currently being used in asphalt pavements; however, class C fly ash demonstrated significant performance against moisture damage and could be tested in the field.
- Development of a testing standard or case studies to evaluate the performance of these additives in the field would further benefit asphalt pavement research.
- Evaluation of HMA that includes recycled asphalt pavement is recommended.

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