Study of Anti-Strip Additives on Granite Based FC-5 Asphalt Mixtures

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16. Abstract

This project evaluated the influence of anti-strip additives on the durability and moisture susceptibility of granite-based open-graded friction course, referred to as FC-5 asphalt mixtures. The laboratory testing involved two granite-based FC-5 mixtures containing 1% hydrated lime (by weight of aggregate), 1% hydrated lime plus 0.5% liquid anti-strip (LAS) additive (by weight of asphalt binder), 1.5% hydrated lime, and 1.5% hydrated lime plus 0.5% LAS additive. Two sources of granite aggregates were obtained: one from Junction City, Georgia and the other from a regional supplier with an original source from Nova Scotia, Canada. Four types of LAS additives were collected from Road Science ArrMaz, Inc. and Ingevity, Inc. The binder bond strength test was used to select the LAS agents that provided the best improvement in moisture resistance. The FC-5 mixtures were fabricated in the laboratory using two FC-5 mix designs provided by the Florida Department of Transportation. The specimens were conditioned by the asphalt pavement weathering system to simulate the long-term aging and moisture conditioning in the field. Mixture performance tests, including the Cantabro test, tensile strength ratio test, and Hamburg wheel tracking test, were used to comprehensively evaluate the durability and moisture susceptibility of FC-5 mixtures. Finally, a cost-benefit analysis was performed to determine the cost-effectiveness of the FC-5 mixtures with anti-strip additives. This project found that the addition of LAS additive, extra 0.5% hydrated lime, or both produced longer lasting FC-5 mixtures, and the additional anti-strip additives would improve the cost-effectiveness of FC-5 mixtures.

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Study of Anti-Strip Additives on Granite Based FC-5 Asphalt Mixtures

Final Report

by

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DISCLAIMER

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EXECUTIVE SUMMARY

Most of the Florida Department of Transportation (FDOT)'s open-graded friction courses (OGFC) produced in the northern portions of Florida contain granite aggregate and are referred to as FC-5 mixtures. The FDOT requires the addition of 1% hydrated lime (by weight of aggregate) in granite-based FC-5 mixtures to address the stripping of asphalt binder due to the presence of moisture. This project aimed to determine if liquid anti-strip (LAS) additive, additional 0.5% hydrated lime, or both would produce longer lasting FC-5 mixtures, and if so, determine if the anticipated increased life span of the FC-5 mixtures would more than offset the increased cost of the additional additives using an economic analysis.

First, a literature review was conducted to identify the moisture damage mechanisms and the feasible solutions to improve the moisture resistance and durability of granite-based FC-5 asphalt mixtures. The literature review was focused on the following aspects: moisture damage mechanisms and the factors influencing moisture damage; existing moisture conditioning methods for asphalt mixtures, particularly for OGFC mixtures; and existing performance testing methods for evaluating the moisture damage and durability of asphalt mixtures.

Based on the literature review, a research work plan was developed to ascertain the effects of the additional additives on the performance of FC-5 mixtures. The laboratory testing involved two granite-based FC-5 mixtures, each containing 1% hydrated lime, 1% hydrated lime plus LAS additive, 1.5% hydrated lime, and 1.5% hydrated lime plus LAS additive. The LAS additive was added at a dosage rate of 0.5% by weight of asphalt binder, and hydrated lime was added based on the total aggregate weight.

The research work plan was executed through the following five steps.

1. Material selection: two sources of granite aggregates were obtained: one from Junction City, Georgia and the other from a regional supplier with an original source from Nova Scotia, Canada (a commonly used aggregate in north Florida); a performance graded PG 76-22 (PMA) asphalt binder was received from Mariani Asphalt in Tampa, Florida; four LAS agents were collected from Road Science ArrMaz, Inc. and Ingevity, Inc.; and the hydrated lime was supplied by Carmeuse Lime and Stone (Chatsworth, Georgia).
2. Selection of LAS agents: the binder bond strength (BBS) test per AASHTO T361 was used to select the LAS agents that provided the best moisture resistance (i.e., adhesive bond) when combined with hydrated lime for the asphalt-aggregate (i.e., granite) systems. The dynamic shear rheometer and bending beam rheometer tests were then conducted to determine the impact of LAS agents on the Superpave PG of the asphalt binders.
3. Specimen fabrication and conditioning: FC-5 mixtures were fabricated in the laboratory using two FC-5 mix designs provided by FDOT. The specimens were conditioned in the asphalt pavement weathering system (APWS) to simulate the long-term exposure to water infiltration, vapor diffusion, and thermal and ultraviolet oxidation.
4. Mixture performance tests: the Cantabro test, tensile strength ratio (TSR) test, and Hamburg Wheel Tracking Test (HWTT) were used to comprehensively evaluate the durability and moisture susceptibility of FC-5 mixtures before and after APWS conditioning. The results were used to determine if adding LAS additive, additional 0.5% hydrated lime, or both would produce longer lasting FC-5 mixtures.

5. Economic analysis: a cost-benefit analysis was performed to determine whether the anticipated increased life span of FC-5 mixtures would more than offset the increased cost of the additional additives. The laboratory performance test results were used to predict the life expectancy of FC-5 asphalt mixtures containing additional anti-strip additives.

Major findings of this project are summarized as follows:

- The BBS test was effective in differentiating the moisture susceptibility of the asphalt-aggregate systems, particularly if the asphalt binder was subjected to long-term aging. For the Junction City granite, the LAS1 additive outperformed other LAS additives in terms of the wet bond strength and loss of bond strength parameters. For the Nova Scotia granite, the LAS2 additive showed the best performance against moisture damage.
- The Cantabro mass loss was capable of differentiating the influences of anti-strip additives on the durability of FC-5 mixtures. Overall, after the long-term weathering, adding 1% hydrated lime plus 0.5% LAS and 1.5% hydrated lime plus 0.5% LAS achieved the best durability for Junction City and Nova Scotia FC-5 mixtures, respectively.
- The addition of LAS significantly reduced the moisture susceptibility of asphalt mixtures after 1,000-hour and 2,000-hour APWS conditioning. However, there was no evidence to discriminate the effectiveness of LAS and additional 0.5% hydrated lime in improving the moisture resistance of FC-5 mixtures.
- APWS conditioning enhanced the resistance of asphalt mixtures to stripping. The HWTT results did not demonstrate the influence of anti-strip additives on the moisture susceptibility of Junction City and Nova Scotia FC-5 mixtures.
- In general, the performance test results indicated that the addition of 0.5% LAS additive, extra 0.5% hydrated lime, or both produced longer lasting FC-5 mixtures. The cost-benefit analysis demonstrated that the addition of extra hydrated lime and LAS improved the cost-effectiveness of FC-5 mixtures.

CHAPTER 1. INTRODUCTION

PROBLEM STATEMENT

The Florida Department of Transportation (FDOT) has customarily used open-graded friction courses (OGFC) as the final riding surface on interstates and high speed multilane roadways. The OGFC mixture, referred to as FC-5, provides several benefits due to water drainage, such as reduced hydroplaning, reduced splash and spray, and improved visibility (*Cooley et al., 2000; Kandhal, 2002*). However, some pavement sections surfaced with FC-5 have experienced premature failures in the form of raveling. Raveling typically originates from the top downward and may extend completely through the surface layer to the interface of the underlying layer. Figure 1 shows an example of this type of failure that occurred on one Florida OGFC project.

Figure 1. Raveling of One Florida OGFC Project (*Bennert and Cooley, 2014*)

Generally, raveling is a problem of material damage rather than structural damage (*Arambula-Mercado et al., 2016; Mo, 2010*). The raveling distress is a result of wear from the repeated shearing force between tire and pavement surface, moisture damage, or insufficient asphalt-aggregate bonding (*Qureshi et al., 2015; Mo et al., 2014; Moraes et al., 2011*). Most of the FC-5 mixtures produced in the northern portions of Florida contain granite aggregate and 1% hydrated lime, which is required to address binder stripping due to the presence of moisture. Even with the inclusion of hydrated lime, however, granite FC-5 mixtures exhibit premature raveling in many instances. Consequently, it is urgent to look for solutions to reduce the moisture susceptibility and improve the durability of these mixtures. One possible improvement is to include a liquid anti-strip (LAS) additive in addition to the hydrated lime. Another potential improvement would be to increase the required dosage of hydrated lime to 1.5%.

RESEARCH OBJECTIVE

The objectives of this project were: 1) to determine if LAS, additional hydrated lime, or both would produce longer lasting FC-5 mixtures, 2) if the answer to objective 1 is positive, then determine if the increased life span of the FC-5 mixture more than offsets the increased price of the additional additives using cost-benefit analysis.

RESEARCH BACKGROUND

A considerable amount of research has been conducted for more than 40 years to evaluate the resistance of asphalt mixtures to moisture damage. Moisture damage is defined as the loss of stiffness and strength in asphalt mixtures caused by moisture exposure under mechanical loading (*Little and Jones, 2003*). Moisture damage, usually in the form of stripping, is a worldwide problem causing premature service failure. For example, Figure 2(a) shows a pavement core from Honduras so severely stripped that large portions of the mixture have totally been disintegrated. Figure 2(b) also shows stripping in the pavement layer of a project in the United States that was under construction. Naguno and Tanimoto (*1974*) reported that stripping of asphalt pavements was widespread in Japan after investigating 762 sites. Arambula-Mercado et al. (*2016*) pointed out that more than 90% of the maintenance performed on OGFC mixtures in the Netherlands was due to raveling issues.

(a) (b)

Figure 2. Moisture Damage is a Worldwide Problem: (a) in Honduras; and (b) in the U.S.

Existing research has covered a large variety of interests from determining the mechanisms that lead to moisture damage, laboratory methods for moisture conditioning that simulate actual field conditions, and testing protocols that will evaluate the potential for moisture damage. Over the years, research has been conducted with a large assortment of materials, test equipment, and test procedures all with the goal of predicting the performance of an asphalt pavement concerning resistance to moisture damage.

CHAPTER 2. LITERATURE REVIEW

This chapter provides an overview of the moisture damage mechanisms in asphalt mixtures and the relevant influence factors including hydrated lime and LAS additives. The moisture conditioning methods and performance tests for evaluation of moisture damage of asphalt mixtures are also discussed in detail.

MOISTURE DAMAGE MECHANISMS

Moisture damage significantly influences the durability of asphalt mixtures. It is generally agreed that moisture can degrade the structural integrity of asphalt mixtures through the three mechanisms shown in Figure 3: 1) loss of cohesion within the asphalt binder or mastic (Figure 3a), 2) adhesive failure between aggregate and asphalt (i.e., stripping) (Figure 3b), and 3) degradation of the aggregate (*Copeland et al., 2007*). Cohesive failure occurs due to the rupture of bonds between molecules in the asphalt film while adhesive failure occurs due to the rupture of bonds between molecules of different phases. The effect of moisture on pavement performance can be the result of a combination of both mechanisms (Figure 3c). Failure of the bond between binder and aggregate eventually reduces the raveling resistance of OGFC mixtures.

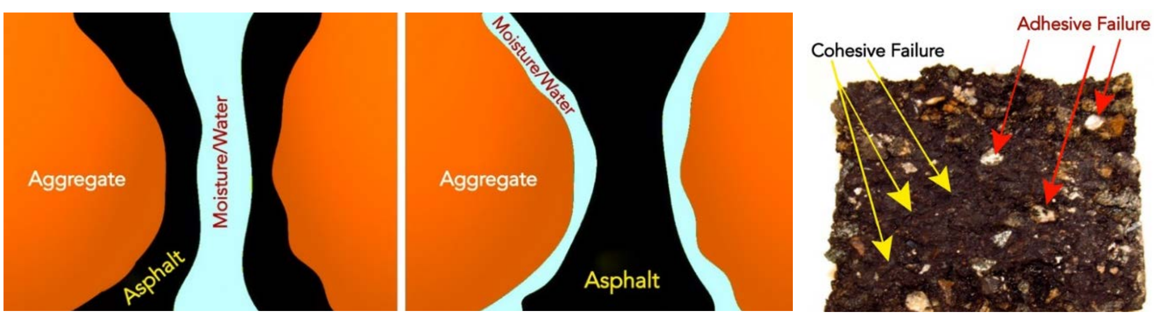


Figure 3. Moisture Damage Mechanisms: (a) Cohesive Failure Mechanism; (b) Adhesive Failure Mechanism; and (c) Observed Cohesive and Adhesive Failure

Stripping can be explained by five different mechanisms: detachment, displacement, spontaneous emulsification, pore pressure, and hydraulic scour (*Birgisson et al., 2005*).

- *Detachment:* Detachment is the separation of the asphalt binder from the aggregate surface with the asphalt coating remaining intact.
- *Displacement:* Stripping by displacement occurs at the three-phase interface between water, asphalt, and aggregate.
- *Spontaneous Emulsification:* Spontaneous emulsification is the formation of an inverted emulsion with water and asphalt, where asphalt is the continuous phase and water the discontinuous phase.
- *Hydraulic Scour and Pore Pressure:* Both are mechanical phenomena that occur when the pavement is saturated. Vehicle tires press water into the pavement in front of the tire and immediately pull it back out behind the tires. This water movement and pore pressure cycling is believed to contribute to the stripping of asphalt films from aggregate.

Table 1 presents the main factors affecting the moisture susceptibility of asphalt mixtures, which are elaborated as follows.

Table 1. Moisture Susceptibility Factors of Asphalt Mixtures (Read and Whiteoak, 2003)

Aggregate Characteristics	Asphalt Binder Characteristics	Asphalt Mixture Properties	External Factors
Mineralogy, Surface Texture, Porosity, Dust, Surface Area, Absorption, Moisture Content, Shape	Rheology, Polarity, Chemical Constitution	Mixture Type, Permeability, Additives, Volumetric Properties, Binder Content, Cohesion and Adhesion of Asphalt-Aggregate System	Rainfall, Humidity, Drainage, Aging, Temperature, Presence of Salt

FACTORS INFLUENCING MOISTURE DAMAGE

Effect of Aggregate Characteristics

The chemistry of aggregate substantially affects asphalt-aggregate adhesion. Various mineral components of aggregate may show a different affinity for asphaltic material. When an aggregate is coated with asphalt binder, the aggregate selectively adsorbs some components of the binder. The general trend is that sulfoxides and carboxylic acids have the greatest affinity for aggregates. It is also apparent that aromatic hydrocarbons have much less affinity for aggregate surfaces than the polar groups. Therefore, the type and quantities of the adsorbed components affect the degree of adhesion and various aggregates develop bonds of different strength (*Robertson, 2000*).

Aggregates are commonly classified as either hydrophilic (i.e., greater natural affinity for water than for asphalt binder) or hydrophobic (i.e., greater natural affinity for asphalt than for water) (*Tarrer and Wagh, 1992*). It is commonly known that acidic aggregates are hydrophobic while basic aggregates are hydrophilic. There are notable exceptions, however, and the general conclusion is that few if any aggregates can completely resist the stripping action of water (*Tarrer and Wagh, 1992*). For example, limestone is classified as hydrophobic aggregate and granite is considered as hydrophilic, however, the level of basic or acidic condition of the limestone and granite aggregates may vary according to their chemical composition.

Aggregates with rougher surfaces and larger surface area are preferred for better adhesive bond. Porosity is another important aggregate characteristic that can affect asphalt physical-mechanical adsorption. For example, when asphalt binder coats a rough aggregate surface with fine pores, air is trapped and the binder has difficulty penetrating the fine pores. Nonetheless, the penetration of asphalt binder into pores is also dependent on the viscosity of asphalt binder at the mixing temperature.

Effect of Asphalt Binder Characteristics

Asphalt binder characteristics can influence both the adhesion of asphalt-aggregate system and the cohesion of asphalt mastic. The properties of asphalt binder affecting the asphalt-aggregate bond are the asphalt chemistry (e.g., polarity and constitution), viscosity, film thickness, and surface energy (*Bahia et al., 2007*). The cohesive strength of asphalt mixture in the presence of moisture is also influenced by the chemical nature of asphalt binder and processing techniques.

The chemical interaction between asphalt binder and aggregate is critical in understanding the capability of asphalt mixtures to resist moisture damage. Robertson (*2000*) describes that carboxylic acids in asphalt binders are polar and adhere strongly to dry aggregate. However, this chemical group tends to be removed easily from aggregate in the presence of water. One reason for this behavior is the fact that sodium and potassium salts of carboxylic acids in asphalt are essentially surfactants or soaps, which are debonded under the action of traffic in the presence of water (*Plancher et al., 1977*). Note that calcium salts from hydrated lime are much more resistant to the action of water. Robertson (*2000*) also suggested that aged asphalts are more prone to moisture damage than unaged asphalts due to the presence of strongly acidic material in oxidized binders. Petersen et al. (*1982*) observed that asphalt binders containing ketones and nitrogen are the least susceptible to moisture damage.

The viscosity of asphalt binder plays a role in the propensity of asphalt mixture to stripping. It has been reported that asphalts with high viscosity resist displacement by moisture better than those with low viscosity (*Bahia et al., 2007*). Asphalts with high viscosity usually carry high concentrations of polar functionalities that provide more resistance to stripping. Since water is a polar molecule, it is readily accepted by the polar asphalt molecules (i.e., resins and asphaltenes). The more polar molecules an asphalt contains, the more readily it will accept water. Oxidation causes aged asphalts to contain more polar molecules. Thus, water should have a greater effect on aged asphalt. It has also been reported that the bond strength is directly related to asphalt film thickness (*Meng, 2010*). Samples with thicker asphalt film tend to have cohesive failure after moisture conditioning. On the other hand, specimens with thinner asphalt film typically have adhesive failure. With respect to surface energy, according to the thermodynamic theory of asphalt-aggregate adhesion, asphalt binder with a lower surface energy is preferable to provide better wetting.

In addition, the changes caused by oxidative aging can potentially affect the chemical components of the asphalt-aggregate interface, particularly with an asphalt-aggregate system that is susceptible to aging. Since several of the functional groups generated during oxidative aging are susceptible to water, the resistivity of the asphalt-aggregate adhesive bond may be weakened by the presence of water, affecting the durability of the asphalt mixture.

Effect of Additives Type and Dosage Rate

In order to improve performance, various antistripping agents have been added to mixtures either in a wet process such as pre-blending with asphalt binder, or in a dry process such as mixing hydrated lime with aggregate. Lee et al. (*1983*) reported that additives may be more effective when applied directly onto the aggregate than when added to the binder. However, it is easier and more economical to blend with the binders and that has become accepted practice when LAS is used. They also stated that chemical additives are typically introduced at a rate of 0.5 to 1.0% of the binder weight. They further mentioned that reclaimed asphalt pavement may be more resistant to moisture damage than virgin aggregate because the aggregate particles have already been precoated, and siliceous and rhyolite aggregates present the most serious moisture damage problems.

Kennedy et al. (*1983*) reported that hydrated lime is effective as an antistripping agent due to its highly alkaline properties that neutralizes organic acids in asphalt binder and replaces the hydrogen, sodium, and potassium on aggregate surfaces. By reducing the acid surface free energy, hydrated lime increases the base surface energy of aggregates. These changes in aggregate surface free energy components lead to a significant improvement in adhesion between asphalt binder and aggregate, which thereby reduces the moisture sensitivity and stripping of asphalt mixtures (*Little and Epps, 2006*).

LAS additives are surface-active agents that decrease the surface tension between asphalt and aggregate surface, thereby allowing aggregate to be more easily wetted by asphalt. Thus, the addition of LAS to asphalt increases the strength of asphalt-aggregate adhesion and reduces the moisture susceptibility of asphalt mixture. The LAS additives are usually blended with asphalt binders at concentrations of up to 0.75% by weight of binder. The use of higher concentrations may result in softening of asphalt mixture, which increases the risk for mix tenderness and permanent deformation.

Most of the research to date has been with dense-graded mixtures; very little has been performed with OGFC such as Florida's FC-5 mixture. Georgia began requiring LAS additive in the late 1960s (*Stapler, 1984*) with only the amount used for dense-graded mixtures as required to meet retained stability results while the amount required for OGFC mixtures was 1.0% based on the weight of asphalt binder. In 1979, the policy was changed to require a minimum of 0.5% LAS additive in all dense-graded mixtures. Stripping continued to be a problem, and in 1982, Georgia began requiring 1.0% hydrated lime as an antistrip agent in asphalt mixtures on all state route and interstate projects instead of LAS additive.

Petersen reported that the standard rate for hydrated lime treatment was 1.0% of the total aggregate, but in special situations the dosage rate was increased to 1.5% (*Petersen, 1988*). He also discovered that hydrated lime was able to reduce age-hardening in asphalt mixtures. A dosage rate of 0.5, 1.0, and 1.5% hydrated lime was used in the research, and he concluded that 1.0% hydrated lime was the minimum needed to capitalize on the effect of reducing age-hardening.

Won and Ho (*1994*) conducted tests with three LAS additives and hydrated lime to evaluate their effects on the properties of asphalt mixtures. The LAS was added at dosage rates of 0, 0.5, and 1.0% by weight of asphalt binder while hydrated lime was added at 0.5 and 1.0% by weight of aggregate. Two grades of modified and unmodified asphalt were used in the study and the aggregate was a sandstone/limestone blend. The results showed that the effect of LAS was additive specific. In other words, additive sources react differently with different binders.

MOISTURE CONDITIONING METHODS FOR ASPHALT MIXTURES

There are currently two types of moisture conditioning methods: water infiltration and vapor diffusion. Water infiltration is directly related to rainfall, drainage conditions, and material properties, while vapor diffusion is dependent on relative humidity, diffusion coefficients, and water-holding potential of material. Water infiltration is recognized as the main source of pavement moisture (*Masad et al., 2007*). However, existing studies found that pavements with severe moisture damage also existed in regions with low levels of annual rainfall, such as Arizona and New Mexico (*Hicks, 1991; Caro et al., 2008*). This indicated that vapor diffusion also contributed to the moisture damage of asphalt mixtures. Table 2 provides a list of moisture conditioning methods commonly used for asphalt mixtures. Each method is discussed in detail in the following sections.

Table 2. List of Moisture Conditioning Methods for Asphalt Mixtures

Conditioning Type	Method	Procedure
Water Infiltration	Hot Water Bath	Submerge specimens in water bath at 40 to 60°C for a specific period.
	Freeze-Thaw Conditioning	1. Vacuum saturate specimens at 26 in. Hg below atmospheric pressure; 2. Freeze specimens at -18°C for a minimum of 16 hours; 3. Submerge specimens in a water bath at 60°C for 24 hours.
	Moisture-Induced Stress Tester (MIST)	1. Apply compressed air to force water into and out of test specimens; 2. Set water temperature at 30 to 60°C and air pressure up to 100 psi (689 kPa); 3. Condition specimens for 3,500 cycles.
	Cyclic Pore Pressure Conditioning (CPPC)	1. Vacuum saturate specimens at 25 in. Hg below atmospheric pressure for 15 minutes; 2. Keep specimens submerged in water for 20 minutes at the normal pressure; 3. Place specimens in the conditioning chamber and set cyclic pore pressure at 5 to 25 psi (34 to 172 kPa) and temperature at 25°C; 4. Condition specimens for 5,800 cycles.
Vapor Diffusion	Relative Humidity Chamber	1. Place specimens in a humidity-controlled chamber (with no direct contact of chemical solutions); 2. Determine the conditioning time.
Water Infiltration + Vapor Diffusion	Asphalt Pavement Weathering System (APWS)	1. Place specimens in a temperature-, ultraviolet-, and humidity-controlled chamber; 2. Adjust temperature, ultraviolet, and water spray amount based on the targeted environment; 3. Determine the conditioning time.

Hot Water Bath

This method requires the use of a manually or automatically controlled water bath. Typically, a set of compacted test specimens are immersed in the water bath at a temperature between 40°C to 60°C, as shown in Figure 4. The amount of time for keeping specimens immersed varies among different test methods. For example, ASTM D1075-11, *Standard Test Method for Effect of Water on Compressive Strength of Compacted Bituminous Mixtures*, requires the specimens to be submerged in water for 24 hours at 60°C. The specimens will then be transferred to another water bath for 2 hours at 25°C prior to being tested for compressive strength. Hamburg wheel tracking test (HWTT) per AASHTO T 324-17, *Standard Method of Test for Hamburg Wheel-Track Testing of Compacted Hot Mix Asphalt (HMA)*, is another test method that utilizes a hot water bath to simulate the water infiltration of asphalt pavements. The test requires preconditioning test specimens in a water bath for 45 minutes at the selected test temperature. In addition, the specimens remain immersed in water for up to 20,000 wheel passes. The test temperature of HWTT varies between 40 and 56°C among state highway agencies depending on their geographic locations and climatic conditions.

Figure 4. Asphalt Mixtures Conditioned in a Hot Water Bath (*Apeagyei et al., 2014*)

Freeze-Thaw Conditioning

Freeze-thaw conditioning is probably the most popular method used by state highway agencies to moisture condition asphalt mixtures. This method requires subjecting test specimens to a freeze cycle at -18°C followed by a warm-water soaking cycle at 60°C. The most notable test method that uses the freeze-thaw conditioning is the modified Lottman test, which is described in AASHTO T 283-14, *Standard Method of Test for Resistance of Compacted Asphalt Mixtures to Moisture-Induced Damage*.

The modified Lottman test consists of measuring the indirect tensile strength for two sets of compacted HMA specimens, with one set tested at the dry condition and the other set after moisture conditioning per the modified Lottman procedure. As illustrated in Figure 5, the conditioning procedure includes partial vacuum saturation followed by a freeze/thaw cycle for a minimum of 16 hours at -18°C in a freezer and then 24 hours at 60°C in a hot water bath. Due to the open void structure of OGFC mixtures, it is difficult to saturate OGFC specimens to a certain degree of saturation (*Watson et al., 2018*). Instead, the specimens are typically saturated at 26 in. Hg below atmospheric pressure for 10 minutes. Additionally, the specimens are required to remain immersed under water during the freeze cycle to maintain saturation.

Multiple freeze-thaw cycles have been used to moisture condition asphalt mixtures. For example, Watson et al. (*2013*) utilized five and ten freeze-thaw conditioning cycles to evaluate the effectiveness of anti-strip agents in asphalt mixtures and concluded that they were more discriminating for moisture susceptibility than one freeze-thaw cycle alone.

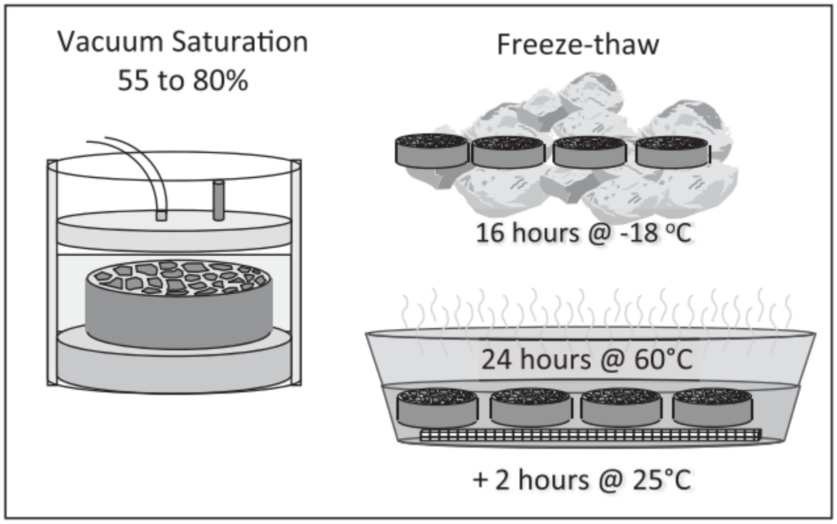


Figure 5. The Modified Lottman Moisture Conditioning Procedure (*Santucci, 2010*)

Moisture-Induced Stress Tester (MIST)

The Moisture-Induced Stress Tester (MIST) is an accelerated cyclic conditioning system that simulates the stripping mechanisms that occur in asphalt pavements. As shown in Figure 6, it consists of a pressurized chamber that applies compressed air to force water into and out of a test specimen, simulating the action of an automobile tire on the road (*InstroTek Inc, 2018*). To simulate different traffic and environmental conditions, the test can be performed at different pressures up to 100 psi (689 kPa) and different temperatures between 30 and 60°C. Typically, the test requires the application of 3,500 pressure cycles. After testing, the specimen is visually inspected for signs of stripping and/or tested to determine the percent reduction in its fundamental mechanical properties, such as indirect tensile strength, dynamic modulus, and resilient modulus (*Chen and Huang, 2008; Yin et al., 2016; DeCarlo et al., 2018*). The MIST test procedure is described in ASTM D7870−13, *Standard Practice for Moisture Conditioning Compacted Asphalt Mixture Specimens by Using Hydrostatic Pore Pressure*.

Figure 6. Moisture-Induced Stress Tester (*InstroTek, 2018*)

However, research by Arambula-Mercado et al. (*2016*) indicated that the MIST device may produce false positives in the case of moisture damage. In a study for FDOT, TSR values were above 100% in five of the six mixtures tested. This means that the MIST conditioning and test procedure actually made the samples stronger. Cantabro test results after MIST conditioning showed similar results in that four of the six mixtures had lower stone loss after conditioning.

Cyclic Pore Pressure Conditioning (CPPC) System

The Cyclic Pore Pressure Conditioning (CPPC) system was developed to simulate the cyclic pore water pressure as a major mechanism of premature moisture damage in asphalt mixtures (*Birgisson et al., 2005; Roque et al., 2012*). As shown in Figure 7, the CPPC system includes a triaxial chamber that allows precise application of uniformly distributed stress in three different directions on test specimens. During conditioning, a set of test specimens are first vacuum saturated at a pressure of 25 in. Hg under atmospheric pressure for 15 minutes and then slightly agitated to remove air bubbles clinging to the surface of the specimens. The specimens are then left submerged in water for 20 minutes at atmospheric pressure. After that, the saturated specimens are placed inside an airtight, water-filled chamber. Birgisson et al. (*2005*) recommended the application of cyclic pore pressure at 5 to 25 psi (34 to 172 kPa) and 25°C. With a waveform frequency of 0.33 Hz, the entire conditioning process of 5,800 cycles typically takes approximately five hours.

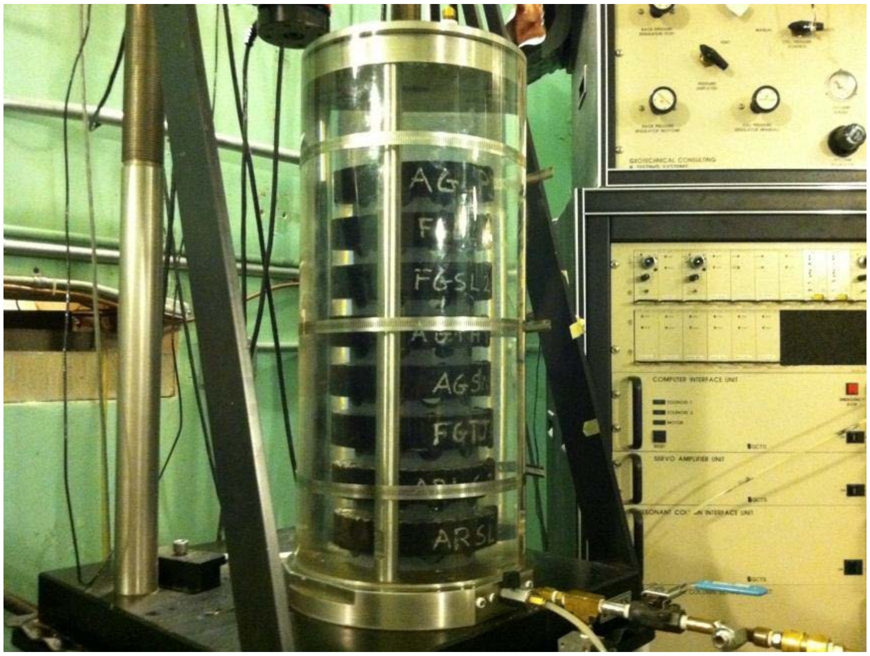


Figure 7. CPPC System and Triaxial Chamber (*Roque et al., 2012*)

Relative Humidity Chamber

Relative humidity chamber has been found effective in simulating the vapor diffusion mechanism, which occurs in asphalt pavements. Tong (*2013*) developed a conditioning system that utilized a vacuum desiccator filled with chemical solutions to condition small-scale fine aggregate mixture (FAM) specimens, as shown in Figure 8. Because the affinity for water of the selected chemical solution regulates the water vapor pressure in the closed system, the relative humidity level in the vacuum desiccator can be controlled by the selected solution in the vacuum desiccator. To simulate zero and 100 percent relative humidity level, Tong (*2013*) used desiccant and distilled water as the chemical solutions, respectively. In his study, the FAM specimens were conditioned in the vacuum desiccator over a period of six months prior to being tested to characterize their fatigue crack growth in the repeated direct tension (RDT) test.

Figure 8. FAM Specimens Conditioned in Vacuum Desiccators (*Tong, 2013*)

Asphalt Pavement Weathering System

The accelerated weathering conditions for asphalt materials was originally designated by ASTM Standard D4798, which is also called Xenon-Arc method (*ASTM, 2016*). This method specifies the requirements of temperature, light, and water that are used for accommodation of asphalt materials in a natural accelerated weathering system. However, due to the limited equipment space, this method is only suitable for asphalt films and small-size asphalt specimens. To condition large-size or full-depth asphalt specimens, PRI Asphalt Technologies, Inc. (PRI) designed the Asphalt Pavement Weathering System (APWS) to simulate natural accelerated pavement weathering, which is shown in Figure 9 (*Grzybowski et al., 2012*).

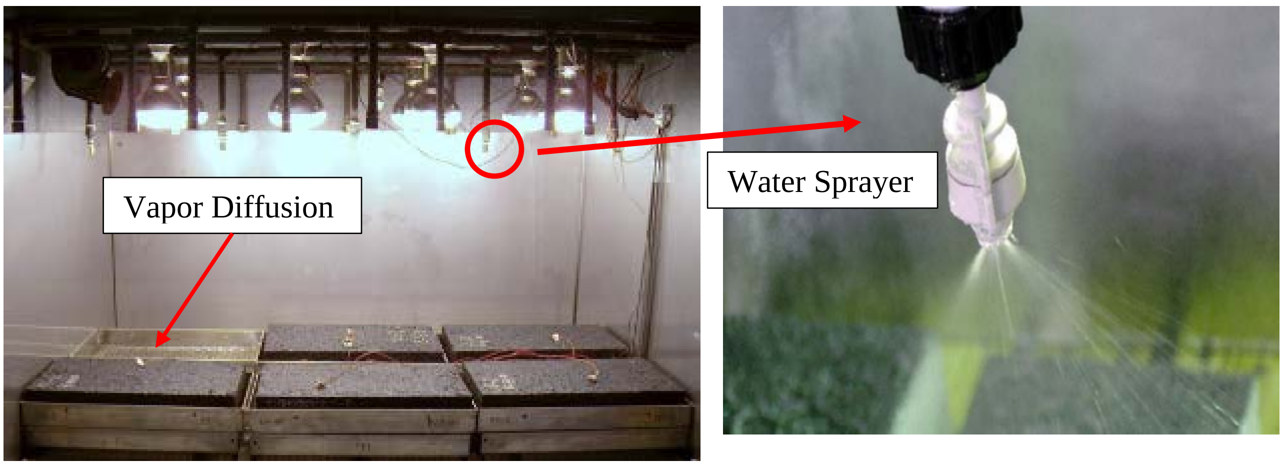


Figure 9. Asphalt Pavement Weathering System

Similar to the Xenon-Arc device, the APWS has controllable cycles to simulate various environmental conditions, including rain, relative humidity, sunlight (UV exposure), and temperature. The APWS adjusts the chamber temperature to represent the thermal-oxidative aging effect and utilizes lamps containing quartz discharge tubes with tungsten filaments to simulate natural sunlight radiation. It also utilizes water spray jets to apply water mist into the chamber, which combines both water infiltration and vapor diffusion modes to simulate the natural moisture condition.

In general, the APWS not only simulates the long-term exposure to moisture, but also simultaneously mimics long-term aging. Thus, this weathering system is able to address the coupling effects of moisture and aging conditions on the durability of asphalt materials. The test variables of APWS are presented as follows (*Grzybowski, 2013*):

- Chamber temperature range: 50-160 °F (10-71 °C);
- Water injection speed: 0.1 – 1.0 inch (2.5 to 25.4 mm) per hour;
- Water type: fresh water, salt water, and deionized water;
- Cycle combination: sunlight only; sunlight + rain; rain only; dark only; dark + rain; sunlight/dark + rain;
- Operation: 24 hours/7 days a week;
- Specimen exposure elapsed time: typically, 1,000 – 5,000 hours; and
- Specimen size: up to 25 ft^2 × 12 inch (2.32 m^2 × 30 cm).

Previous studies by PRI showed that 3,000 hours (four months) in the APWS was equivalent to approximately 12 years of weathering in the field (*Grzybowski*, *2013*).

MOISTURE DAMAGE EVALUATION OF ASPHALT-AGGREGATE SYSTEM

Youtcheff and Aurilio (*1997*) used the Pneumatic Adhesion Tensile Testing Instrument (PATTI), originally developed for the coating industry (*Yoon and Tarrer*, *1988*), to measure the moisture susceptibility of asphalt binders. Kanitpong and Bahia (*2003*) also used the PATTI test to evaluate the moisture susceptibility of asphalt binders modified with polymers and anti-stripping additive. Copeland et al. (*2007*) investigated the effects of aging on the pull-off tensile strength of asphalt binders using the PATTI device. Moraes et al. (*2011*) investigated the feasibility of the Binder Bond Strength Test (BBS) per AASHTO T361 (modified version of PATTI) for moisture damage characterization and indicated that the BBS test is repeatable and reproducible.

The BBS test configuration is illustrated in Figure 10, which is comprised of a portable pneumatic adhesion tester, pressure hose, piston, reaction plate and a metal pull-out stub. To start the test, the piston is placed over the pull-out stub and the reaction plate is screwed onto it. Then, a pressure hose is used to introduce compressed air to the piston. During the test, a pulling force is applied on the specimen by the metal stub. The pull-out stub has a rough surface that can prevent asphalt debonding from the stub surface by providing mechanical interlock and larger contact area between the asphalt binder and stub. Failure occurs when the applied stress exceeds the cohesive strength of the asphalt binder or the bond strength of the asphalt-aggregate interface (i.e., adhesion) (Figure 11). The pull-off tensile strength (POTS) is calculated according to Equation 1.

$$POTS = \frac{(BP \times A_g) - C}{A_{ps}} \qquad (1)$$

where A_g is the contact area of gasket with reaction plate (mm^2), BP is the burst pressure (kPa), A_{ps} is the area of pull stub (mm^2), and C is the piston constant.

(a) (b) (c)

Figure 10. Binder Bond Strength Test Apparatus: (a) Test Configuration; (b) Pull-off Stub; (c) Sample Preparation

Figure 11. Failure in Binder Bond Strength Test: a) Cohesive Failure b) Adhesive Failure

The effect of water on the bond strength of an asphalt-aggregate system can be evaluated by means of POTS due to immersion of the asphalt-aggregate system in water for different conditioning times. The effectiveness of the BBS test has been comprehensively assessed by Moraes et al. (*2011*), which considered asphalt binders with different types and additives, aggregate mineralogy, and conditioning time. A statistical analysis was performed to verify the repeatability and reproducibility of the test. As shown in Figure 12, the BBS test was able to effectively quantify the effects of moisture conditioning time and additional additives on the bond strength of asphalt-aggregate systems. The bonding between asphalt and aggregate under wet conditions is highly dependent on binder, aggregate, and additive types.

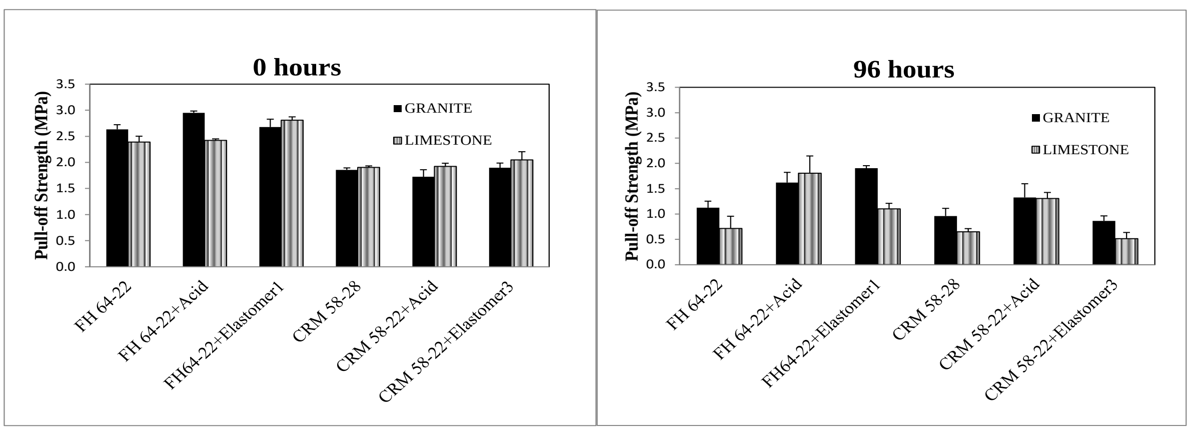


Figure 12. Influence of Conditioning Time and Modification on Binder Bond Strength (*Moraes et al., 2011*)

An important observation highlighted by Moraes (*2011*) is the capability of the BBS test in screening asphalt-aggregate systems that will better perform with a specific additive, resulting in a system less susceptible to moisture damage. As illustrated in Figure 13, the effect of an asphalt additive in reducing a binder moisture susceptibility is highly aggregate specific; for example, a PG 58-28 binder modified with acid showed, after different conditioning times in water, higher pull-off tensile strength values with granite aggregate than with limestone aggregate. Compared to the surface energy test, the BBS was proven to be more efficient and less expensive for moisture damage characterization (*Moraes et al., 2011*).

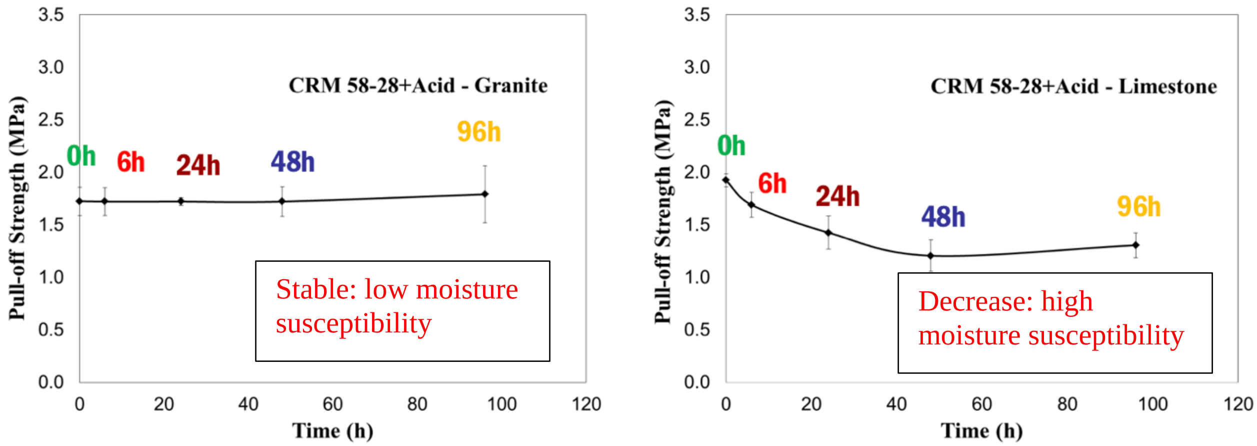


Figure 13. Influence of Aggregate Type on Binder Bond Strength (*Moraes, 2011*)

MOISTURE DAMAGE EVALUATION OF ASPHALT MIXTURES

In general, moisture damage tests can be categorized into three groups: uncompacted loose mixes, comparison of conditioned and unconditioned mixtures, and repetitive loading in presence of water (*Santucci, 2010; Epps Martin et al., 2014*). Some of the most commonly used moisture damage tests are discussed in detail as follows.

Boiling Test

The boiling test requires subjecting asphalt loose mixes to boiling water for 10 minutes and visually observing the percentage of aggregate surface areas that remain coated by asphalt binder. Asphalt mixes with a higher asphalt coating percentage are expected to have better resistance to moisture damage than those with a lower coating percentage. Although Parker and Wilson (*1986*) reported a reasonable correlation between the boiling test results and field performance for Alabama mixtures, the test is subjective and is not recommended for use as a measure of field performance according to ASTM D3625/3625M-12, *Standard Practice for Effect of Water on Bituminous-Coated Aggregate Using Boiling Water*.

The test has several additional limitations; for example, it does not consider the void structure, permeability, and gradation of asphalt mixtures, which are factors affecting the moisture susceptibility. Furthermore, the test is not applicable to fine graded mixtures due to the difficulty of visually observing fine aggregate particles that are not coated by asphalt binders. While the boil test is not considered as accurate as other methods, it is a very quick test to perform and can be used for field quality control during construction as a screening test for moisture susceptible asphalt-aggregate combinations (*Kennedy, 1987*).

Static-Immersion Test

The static-immersion test is another method that requires visual observation of aggregate surfaces that are not coated by asphalt binders due to moisture conditioning. The test procedure is described in the former AASHTO T 182-84, *Standard Method of Test for Coating and Stripping of Bitumen-Aggregate Mixtures*. The test requires submerging asphalt loose mixes in distilled water at room temperature for 16 to 18 hours and then visually estimating the percentage of the total area of the aggregate that remains coated by asphalt binders as either above 95 percent or below 95 percent. Similar to the boiling test, the static-immersion test is subjective and can sometimes provide misleading results.

Modified Lottman Test

The modified Lottman test per AASHTO T 283, *Standard Method of Test for Resistance of Compacted Asphalt Mixtures to Moisture-Induced Damage*, is also known as the tensile strength ratio (TSR) test. It is required in the Superpave mix design procedure to evaluate the moisture susceptibility of asphalt mixtures. The test consists of measuring the indirect tensile strength of two sets of cylinder specimens, with one set tested at the dry condition and the other set after moisture conditioning per the modified Lottman procedure. As discussed previously, the modified Lottman procedure includes partial vacuum saturation followed by one freeze-thaw cycle. The ratio of the conditioned tensile strength to the dry tensile strength yields the TSR value for the mixture. Asphalt mixtures with a higher TSR are expected to have better resistance to moisture damage than those with a lower TSR. Many state highway agencies, including FDOT, require a minimum TSR value of 0.80.

Immersion-Compression Test

The immersion-compression test per ASTM D1075-11, *Standard Test Method for Effect of Water on Compressive Strength of Compacted Bituminous Mixtures*, is similar to the modified Lottman test except that it only requires hot water bath (i.e., no vacuum saturation and freezing) for moisture conditioning and that it measures the reduction of compressive strength instead of indirect tensile strength. The ratio of the conditioned compressive strength to the dry compressive strength yields a test parameter termed index of retained strength. Similar to the TSR value, a higher index of retained strength is desirable for asphalt mixtures and indicates better resistance to moisture damage. Previous experience with the test showed that it did not have a good correlation with field performance. Specifically, asphalt mixtures showing obvious signs of stripping in the field could have an index of retained strength near 1.0. In addition, there was a concern among the asphalt industry that the compressive strength of asphalt mixtures was not sensitive to adhesive and cohesive causes of failure due to moisture damage (*Epps Martin, 2014*).

Hamburg Wheel Tracking Test (HWTT)

HWTT is a laboratory test method for evaluating the resistance of asphalt mixtures to rutting and moisture damage. The test is generally performed in accordance with AASHTO T 324-17, *Standard Method of Test for Hamburg Wheel-Track Testing of Compacted Hot Mix Asphalt (HMA)*. As shown in Figure 14, HWTT specimens are tested under a 158 ± 1 lbs. wheel load for up to 20,000 passes while submerged in a water bath maintained at 40 to 60°C. While being tested, rut depths are measured by two linear variable differential transformers (LVDTs), which record the relative vertical position of the load wheel after each load cycle. After testing, these rut depth data are used to estimate the point at which stripping occurred in the mixture and the rutting susceptibility of those mixtures.

Figure 14. Hamburg Wheel Tracking Test Device

Figure 15 illustrates typical data output from the HWTT test. These data show the progression of rut depth with number of cycles. From this curve two tangents are evident, the steady-state rutting portion of the curve and the portion of the curve after stripping. The intersection of these two curve tangents defines the stripping inflection point (SIP) of the mixture. A higher SIP is desirable for mixtures with better resistance to moisture damage.

In addition to SIP, Yin et al. (*2014*) proposed an alternative HWTT moisture susceptibility parameter termed stripping number (SN). As shown in Figure 16, SN is defined as the inflection point of the rut depth curve where the curvature changes from negative to positive. SN represents the maximum number of load cycles that the mixture can resist before adhesive fracture between the asphalt binder and aggregate occurs. The computation of SN does not require subjective data interpolation of fitting two tangents to the creep phase and stripping phase, and thus, is reported to be more accurate than the SIP parameter. This parameter has been successfully used to assess the moisture susceptibility of over 80 asphalt mixtures with different components and production parameters (*Epps Martin et al., 2014; Newcomb et al., 2015*).

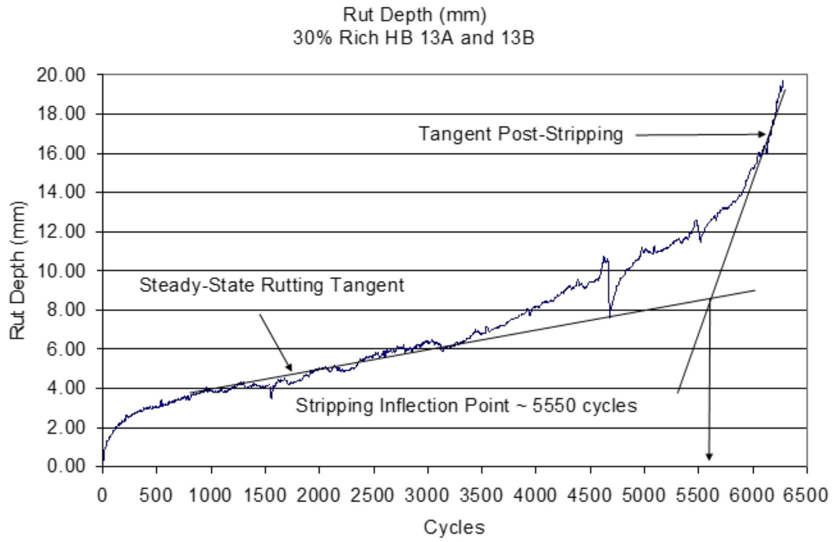


Figure 15. Hamburg Wheel Tracking Data Analysis

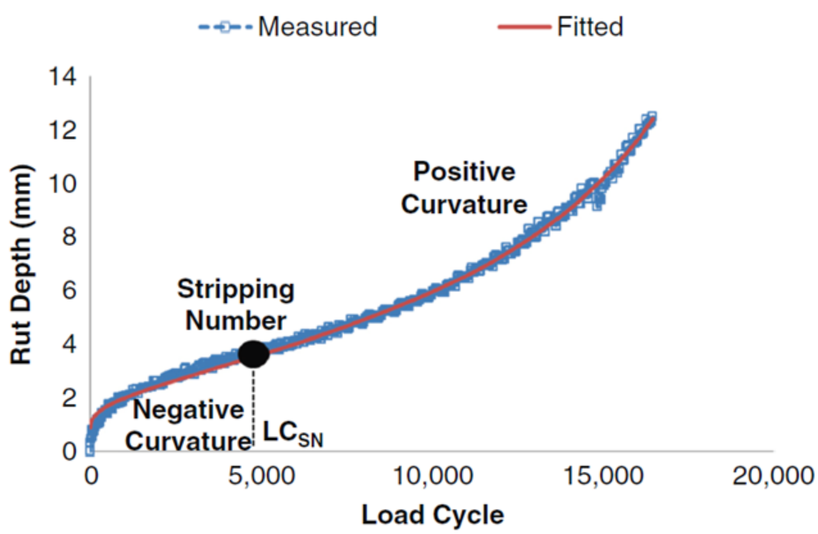


Figure 16. Determination of Alternative HWTT Parameter Stripping Number (SN) (*Yin et al., 2014*)

Lu and Harvey (*2005*) also found that HWTT is effective for determining moisture sensitivity of asphalt mixtures and could identify the effect of antistrip additives. Based on their study, both hydrated lime and LAS provided long-term effectiveness in preventing moisture damage.

Cantabro Test

The Cantabro test, AASHTO TP 108, was developed in Spain and its use has generally been accepted in the United States as an indicator of the cohesion properties of a mixture to resist raveling. The Cantabro stone loss test uses a compacted specimen placed in a Los Angeles abrasion drum without the steel charges. After 300 revolutions, the sample is removed and the mass of the final sample is compared to the initial mass to determine the percent loss (Figure 17).

Figure 17. Cantabro Sample Before and After Testing

Arambula-Mercado et al. (*2016*) reported that the proportion of aggregate loss in the Cantabro test was the best predictor of the durability of OGFC mixtures. They found that when indirect tensile strength, Hamburg Wheel Track, and Cantabro test results were compared, the Cantabro test was the only test able to differentiate the mixtures according to observed field performance. As part of the NCHRP 1-55 study, Watson et al (*2018*) evaluated an OGFC mix that had failed within seven years. The mix was produced at 6.0% optimum binder content. However, the Cantabro test clearly showed the mix would perform poorly (> 20% stone loss) even if an additional 1.0% binder was added (Figure 18).

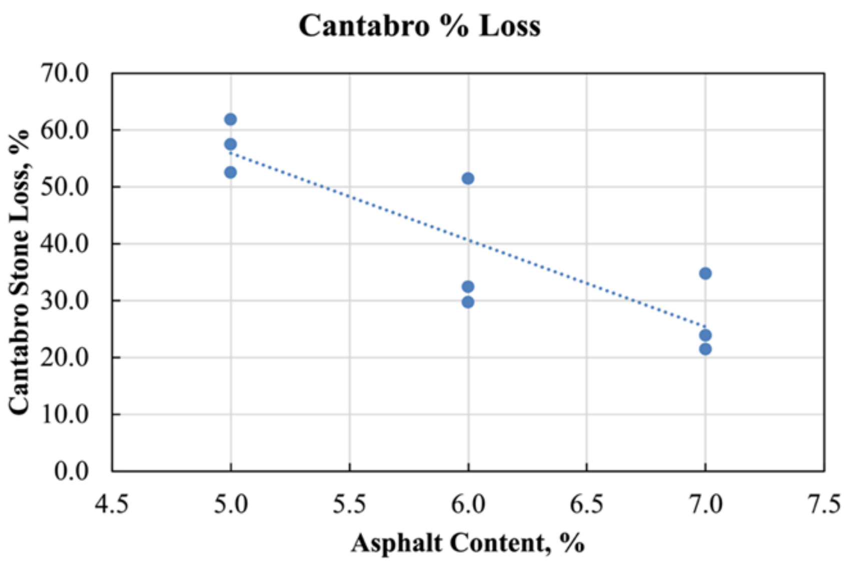


Figure 18. Cantabro Results for Poorly Performing OGFC

Other Moisture Damage Tests

The ultrasonic accelerated moisture conditioning (UAMC) test uses ultrasonic energy to assess displacement and detachment of asphalt binder from the aggregate (*McCann and Sebaaly, 2001; McCann et al., 2006*). Ultrasonic energy is applied to asphalt loose mixes while being immersed in a hot water bath at 60°C for five hours. During the test, the loss of mix weight through a No. 16 sieve is continuously monitored. The rate of material loss due to ultrasonic energy provides an indication regarding the moisture susceptibility of the mixture. According to McCann et al. (*2006*), the UAMC test had a reasonable correlation with the TSR test.

The net adsorption test (NAT) was developed during the Strategic Highway Research Program (SHRP) to evaluate the affinity of asphalt for aggregate and to determine the moisture susceptibility of a given asphalt-aggregate combination (*Curtis et al., 1992*). The test measures the amount of asphalt binder adsorbed onto aggregate from a toluene solution with and without the presence of water. The difference in the amount of adsorbed asphalt binder indicates the moisture susceptibility of the asphalt-aggregate combination. Curtis et al. (*1992*) recommended using the NAT to screen asphalt binders and aggregates for use in asphalt mixtures. Stroup-Gardiner et al. (*1995*) reported a reasonable correlation between the NAT results and field performance for Minnesota mixtures.

Other researchers have explored the reduction in mixture fundamental properties due to moisture conditioning as a moisture damage test for asphalt mixtures. This approach is generally similar to the modified Lottman test except for using different moisture conditioning methods and testing different mixture properties. For example, Birgisson et al. (*2007*) evaluated the effect of moisture damage on the fracture resistance of asphalt mixtures using the Superpave indirect tension test per AASHTO T 322, *Standard Method of Test for Determining the Creep Compliance and Strength of Hot Mix Asphalt (HMA)*. The indirect tension test specimens were moisture-conditioned using cyclic pore pressure. The difference in the energy ratio (ER) before and after moisture conditioning was used to quantify mixture resistance to moisture damage.

In another study by Solaimanian et al. (*2006*), dynamic modulus (E*) test was conducted on unconditioned and moisture-conditioned specimens based on the former AASHTO TP 62, *Standard Method of Test for Determining Dynamic Modulus of Hot Mix Asphalt (HMA)*. Moisture conditioning was simulated using the environmental conditioning system (ECS) developed during SHRP. Asphalt mixtures with an E* stiffness ratio over 0.75 and 0.80 were considered to have adequate moisture resistance. In NCHRP project 09-49, Epps Martin (*2014*) tested the resilient modulus (M_R) ratio with and without the modified Lottman procedure and found the parameter effective in discriminating warm mix asphalt (WMA) mixtures with different moisture susceptibility.

Summary of Moisture Sensitivity Tests

Researchers have used different types of tests to evaluate the moisture sensitivity of asphalt mixtures (*Cho, 2008; Bahia et al., 2007; Bhasin, 2006; Kim et al., 2004; Terrel and Al-Swailmi, 1994*). Conventional laboratory tests for evaluating stripping potential in asphalt mixtures can generally be classified into two types: loose mixture tests and compacted mixture tests. Tests on loose mixtures are conducted on asphalt-coated aggregates in the presence of water. The advantages are cost effectiveness and simplicity. The major disadvantage is the fact that these tests do not consider pore pressure, traffic action, or mechanical properties. Tests on compacted mixtures are conducted on laboratory-compacted specimens, field cores, or slabs. The major advantage of these tests is the fact that the physical and mechanical properties, water and traffic action, and pore pressure effects can be considered. The disadvantage is that more elaborate testing equipment and longer testing times are required. Table 3 and 4 summarize the tests for moisture sensitivity on loose and compacted mixtures, respectively.

Table 3. Moisture Sensitivity Tests on Loose Samples ***(Moraes, 2011)***

Test	Moisture Damage Method
Boiling	Fully coating gravel aggregates with melted asphalt. Boiling adhesion value is obtained from the remaining percentage of coated area (not stripped by water).
Chemical Immersion	Determination of adhesion of asphalt to aggregate by means of boiling loose mix in water. Increasing concentrations of sodium carbonate (Na2CO3), numbered 0 to 9 and referred to as the Riedel and Weber (R&W) number.
Dynamic Immersion	Immerse asphalt loose mix in water followed by agitation. As the period of agitation increases, the degree of stripping increases.
Film Stripping	Immerse asphalt loose mix in water followed by rotation. The results are reported in terms of the percent total aggregate surface stripped.
Methylene Blue	Attempts to identify the harmful clays and dust available in the fine aggregate.
Net Adsorption	Asphalt is adsorbed onto aggregate from a toluene solution. Water is introduced into the system, and amount of asphalt remaining on aggregate surface is calculated.
Pneumatic Pull-Off Strength	Measures bonding strength of asphalt binder applied to aggregate substrate as a function of time while exposed to water.
Rolling Bottle	Aggregate chips are coated with asphalt and covered with water in glass jars under agitation. Periodically, the coating of the stones is estimated visually.
Static Immersion	Immerse loose asphalt mix in water. Total visible area of the aggregate is estimated as either less than or greater than 95%.
Surface Energy	Cohesive bonding within asphalt and adhesive bonding between asphalt and aggregate are related to asphalt and aggregate surface energy.
Surface Reaction	Calcareous or siliceous minerals react with an acid and form a gas to create a pressure proportional to mineral surface area exposed with acid.

Table 4. Moisture Sensitivity Tests on Compacted Mixtures *(Moraes, 2011)*

Test	Moisture Damage Method
Environmental Conditioning System (ECS) with Resilient Modulus	A membrane-encapsulated specimen is subjected to cycles of temperature, repeated loading, and moisture conditioning. The air permeability and dry resilient modulus are determined after it is placed inside the ECS load frame.
Environmental Conditioning System/SPT	The proposed tests are dynamic modulus, repeated axial load, and static axial creep tests.
Freeze-Thaw Pedestal Test	Conducted on an asphalt mix with uniform aggregate sizes. The specimen is placed on a stress pedestal with water. The number of freeze thaw cycles to induce cracking indicates moisture susceptibility.
Hamburg Wheel Tracking	Measures the combined effects of rutting and moisture damage by rolling a steel wheel across the surface of an asphalt concrete specimen immersed in hot water.
Immersion-Compression	Compressive strength is measured on dry and wet specimens. Average strength of wet specimens over that of dry specimens is the moisture sensitivity of the mix.
Original Lottman Indirect Tension	Conditioned and dry specimens are both tested for tensile resilient modulus and tensile strength using indirect tensile equipment. The severity of moisture sensitivity is judged on the basis of the ratio of test values for conditioned and dry specimens.
Modified Lottman Indirect Tension	Compares the indirect tension test results of a dry sample and a sample exposed to water/freezing/thawing. Sample saturation, test temperature, and loading rate are different from the original Lottman test.
Moisture Vapor Susceptibility	Two specimens are prepared and compacted. The compacted surface of each specimen is covered with an aluminum seal cap, and a silicone sealant is applied around the edges. An assembly with a felt pad, seal cap, and strip wick is prepared to make water vapor available to the specimen. The assembly is left in an oven and tested in the Hveem stabilometer.

CHAPTER 3. DEVELOPMENT OF RESEARCH WORK PLAN

To achieve the research objectives, a research work plan was developed as shown in Figure 19. There were four critical steps involved in this project: 1) selecting materials representative of that used on FDOT projects; 2) selecting the laboratory conditioning method, or methods, to simulate long-term exposure to moisture and aging; 3) conducting laboratory evaluation of the effects of LAS and hydrated lime on the performance of FC-5 mixtures; and 4) conducting a cost-benefit analysis to evaluate the cost-effectiveness of using additional additives in FC-5 mixtures.

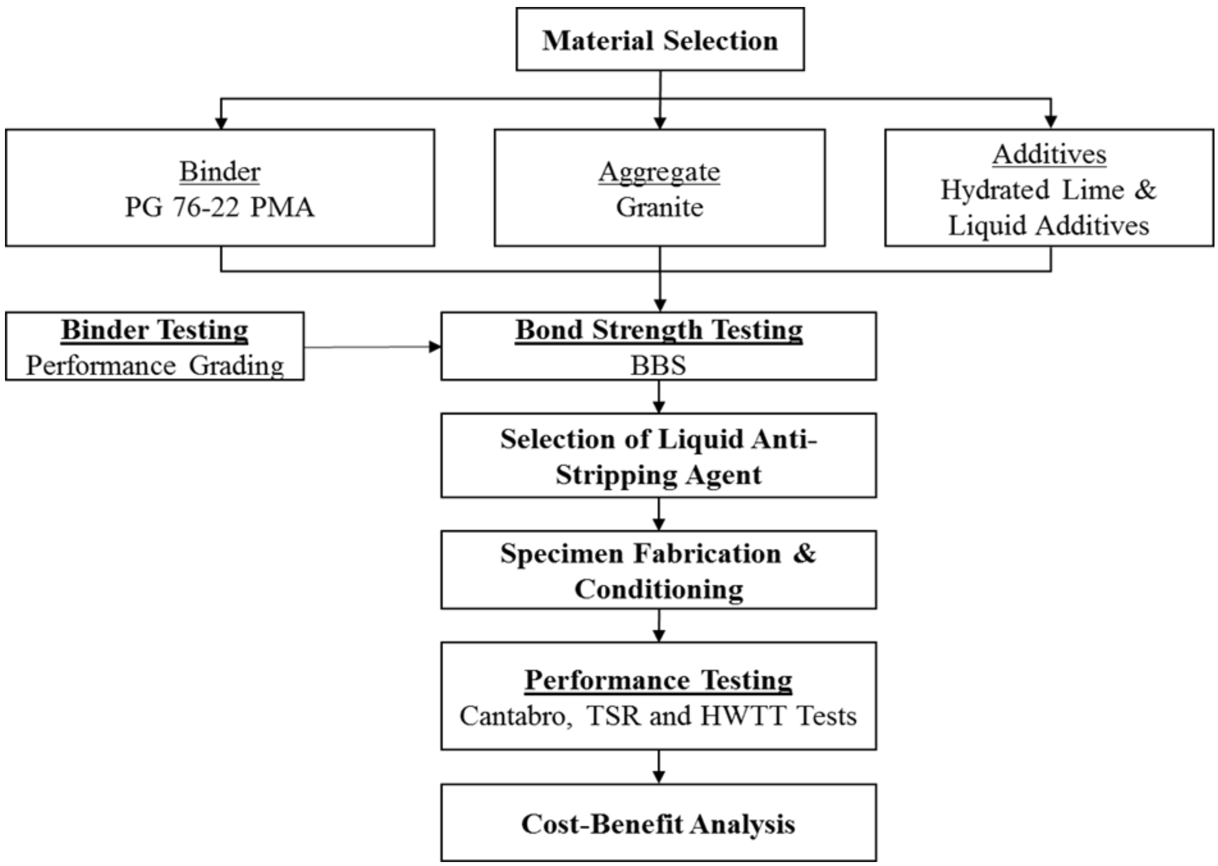


Figure 19. Project Research Methodology

MATERIAL SELECTION

In the first step, it is important to select materials that are commonly used in Florida FC-5 mixtures. The FC-5 mixture is used as a friction surface that also drains water from the surface through the inter-connected air voids. Due to channeling large amounts of water through the open-graded mixture, it is highly susceptible to effects of moisture damage so an anti-stripping agent is needed. To improve resistance to the potential for rutting and raveling, a polymer modified asphalt binder is used, and granite aggregate is used for frictional properties.

Two sources of granite aggregate were obtained for use in this study. The two sources identified are Martin Marietta Materials in which the aggregate is shipped from Nova Scotia, Canada, and Georgia granite from Junction City Mining in Junction City, Georgia. The Junction City aggregate is well-known for its potential for stripping and is used in FDOT testing for approval of anti-stripping additives.

This study used a performance graded (PG) 76-22 (PMA) asphalt binder modified with styrene-butadiene-styrene (SBS) polymer. Mariani Asphalt in Tampa, Florida was selected as the source for the modified binder.

Four different LAS agents, hydrated lime, and the combination of both LAS agent and lime were evaluated in this study. The source of hydrated lime was Carmeuse Lime and Stone. The LAS was blended according to manufacturer recommendations at the specified blending ratios. The four LAS additives proposed for use in this study were the ones most often used in FDOT projects, which were denoted as LAS1, LAS2, LAS3, and LAS4.

SELECTION OF LAS AGENTS

A screening process was conducted among LAS, hydrated lime, and a combination of both in order to verify which additive would better perform under moisture conditioning with the currently used FDOT FC-5 asphalt-aggregate systems. This process was crucial since the effectiveness of an additive, particularly an anti-strip agent, varied with the additive type, as well as with the properties of asphalt binders and aggregates.

Based on the literature review, the moisture susceptibility of an asphalt mixture is primarily dependent on the bond strength between asphalt and aggregate. Thus, the Binder Bond Strength (BBS) test per AASHTO T361 was used to select two LAS agents deemed to provide the best anti-strip performance (i.e., adhesive bond) when combined with hydrated lime for the FDOT asphalt-aggregate (i.e., granite) systems. The test matrix is shown in Table 5.

Table 5. Test Matrix for Selecting Liquid Anti-Strip Additive

Factor	Factor No.	Description
Base Binder	1	SBS polymer modified
Aggregate	2	Nova Scotia and Junction City Granite
Mineral Agent	1	Hydrated Lime (HL) from Carmeuse Lime
Liquid Anti-Strip Additives	4	LAS_1, LAS_2, LAS_3, LAS_4
Blending Ratios	4	1% HL; 1% HL+LAS; 1.5% HL; 1.5% HL+LAS
Moisture Conditioning	2	Unconditioned and tap water submersion for 48 hours @ 40°C
Aging Level	2	Unaged, RTFO+PAV

Since aged asphalt is more prone to moisture damage than unaged asphalt, both standard and blended binders were subjected to aging simulated in the Rolling Thin-Film Oven (RTFO, AASHTO T 240) and the Pressure Aging Vessel (PAV, AASHTO R 28). The Dynamic Shear Rheometer (DSR) and Bending Beam Rheometer (BBR) tests were conducted to determine the Superpave PG of the asphalt binder before and after the inclusion of LAS additives. Subsequently, the BBS test was conducted to evaluate the cohesive bond strength of the binders as well as the adhesive bond strength at the asphalt-aggregate interface.

SPECIMEN FABRICATION AND CONDITIONING

In this step, FC-5 mixtures were fabricated in the laboratory using the two FC-5 mix designs provided by FDOT. Table 6 shows the aggregate gradations of the two mix designs. Both the Junction City and Nova Scotia mixtures had a total binder content of 6.8%. The Junction City mix contained 0.3% cellulose fiber and the Nova Scotia mix had 0.4% mineral fiber. The details of the two FC-5 mix designs are presented in Appendix A of this report. The specimens were compacted by Superpave Gyratory Compactor with 50 gyrations. The specimen height was controlled in a range of 110-120 mm.

Table 6. Aggregate Gradations of FC-5 Mixtures

Sieve Size	Passing Percentage (%)		Specification Range (%)
	Junction City Mix	Nova Scotia Mix	
3/4” (19.0 mm)	100	100	100
1/2” (12.5 mm)	99	95	85-100
3/8” (9.5 mm)	71	75	55-75
No. 4 (4.75 mm)	24	23	15-25
No. 8 (2.36 mm)	9	10	5-10
No. 16 (1.18 mm)	5	6	-
No. 30 (600 μm)	4	4	-
No. 50 (300 μm)	3	3	-
No. 100 (150 μm)	3	3	2-4
No. 200 (75 μm)	2.5	2.1	-

Prior to performance testing, two of three sets of laboratory fabricated FC-5 specimens for each aggregate type were conditioned by the APWS to simulate the long-term exposure to water infiltration, vapor diffusion, and thermal and ultraviolet oxidation. A third set of specimens was not aged. As illustrated in Figure 9, the APWS controls the rainfall (or water spray), relatively humidity, sunlight (UV exposure), and temperature. The water spray speed was determined based on the local weather conditions, which varied from 0.0 to 1.0 inch (25.4 mm) per hour. For example, to simulate a 10-year-old pavement with an average annual rainfall of 50 inches (1270 mm), the water spray speed would be 0.25 inch (6.35 mm) per hour for 2000 hours. This system simulates long-term moisture and aging conditions simultaneously, which provides a significant advantage over other moisture conditioning methods of addressing these coupling effects on the durability of asphalt mixtures. In this study, the FC-5 specimens were conditioned at two levels (i.e., 1,000 and 2,000 hours) to establish the deterioration trend of mixture performance over conditioning time, as compared to the unaged condition.

MOISTURE PERFORMANCE TESTS

Performance tests including Cantabro test, TSR test, and HWTT were used to comprehensively evaluate the durability and moisture susceptibility of the conditioned FC-5 mixtures.

Cantabro Test

The Cantabro test was performed in accordance with provisional standard AASHTO TP 108-14. During the test, OGFC specimens were individually placed inside the Los Angeles Abrasion machine without the steel charges, and then allowed to freely rotate within the drum at a rate of 30 to 33 revolutions per minute for 300 revolutions. The loose particles were then discarded and the final specimen was weighed. The Cantabro loss was calculated as the ratio between the final weight and the initial weight, which is shown in Equation 2. According to ASTM D7064, an acceptable amount of loss is less than 20% for unaged OGFC mixtures and 30% for long-term oven aged mixtures.

$$CL = \frac{A-B}{A} * 100 \tag{2}$$

where CL is the percentage of Cantabro loss; A is the initial weight of the test specimen; and B is the final weight of the test specimen.

Tensile Strength Ratio (TSR) Test

The TSR test was performed in accordance with AASHTO T 283 with a few modifications made to accommodate OGFC mixtures. These modifications include:

- Specimens are compacted to N_{design} rather than to target air voids,
- Specimens are saturated at 26-inch (660 mm) Hg below atmospheric pressure for 10 minutes regardless of the level of saturation, and
- Samples are kept submerged in water during the freeze conditioning cycle.

Both dry and moisture-conditioned specimens were tested to determine their indirect tensile strength using a Marshall Stability press with a loading rate of 2.0 inches (50.8 mm) per minute. Equation 3 is used to calculate the TSR ratio, which is the ratio of the conditioned indirect tensile strength to the dry indirect tensile strength.

$$TSR = \frac{S_2}{S_1} * 100 \tag{3}$$

where S_1 = average tensile strength of the unconditioned specimen; and S_2 = average tensile strength of the conditioned specimen.

Watson et al. (2018) developed a performance-based mix design for OGFC mixtures recommending that TSR should be at least 0.7 for an OGFC mixes with a minimum of 50 psi (345 kPa) conditioned strength. As a comparison, FDOT requires a minimum TSR of 0.80 and a minimum unconditioned tensile strength of 100 psi (689 kPa) for dense-graded Superpave mixtures.

Hamburg Wheel Tracking Test (HWTT)

The HWTT was used in this study to evaluate the moisture susceptibility and rutting resistance of OGFC mixtures using different types of asphalt binders and aggregates. The test was performed by following AASHTO T 324. Test temperature was 50°C. During the test, four cylindrical specimens were placed in a water bath and subjected to a steel wheel load moving at a rate of 52 passes per minute. Rut depths at various positions along the specimens were recorded with each load cycle. Typical HWTT test parameters include stripping inflection point

(SIP) and rut depth at a critical number of load cycles. Watson et al. (*2018*) recommended the following criteria based on asphalt binder grade:

- PG 64 or lower ≥ 10,000 passes before reaching a 12.5 mm rut depth;
- PG 70 ≥ 15,000 passes before reaching a 12.5 mm rut depth; and
- PG 76 or higher ≥ 20,000 passes before reaching a 12.5 mm rut depth.

Table 7 presents the test matrix for laboratory performance testing, which aimed to determine if the additional additives would increase the life span of FC-5 asphalt mixtures.

Table 7. Test Matrix for Laboratory Performance Testing

Factor Type	Factor Name	Factor No.	Description
Mixture Composition	Base Binders	1	SBS polymer modified
	Aggregate	2	Nova Scotia and Georgia granite
	Mineral Agent	1	Hydrated Lime (HL)
	LAS Additive	2	Selected from Step 2
	Blending Ratios	4	1% HL; 1% HL+LAS; 1.5% HL; 1.5% HL+ LAS
Moisture Conditioning	Conditioning Method	1	APWS
	Conditioning Time	3	0, 1000, and 2000 hours
Performance Testing	Test Methods	3	Cantabro, TSR, and HWTT

COST-BENEFIT ANALYSIS

A cost-benefit analysis was performed to determine whether the anticipated increased life span of FC-5 mixtures more than offset the increased price of the additional additives. The laboratory performance test results were used to predict the life expectancy of FC-5 asphalt mixtures containing additional anti-strip additives.

CHAPTER 4. INFLUENCE OF ANTI-STRIP ADDITIVES ON MOISTURE SUSCEPTIBILITY OF ASPHALT-AGGREGATE SYSTEM

In this chapter, the moisture susceptibility of asphalt-aggregate systems was evaluated using the BBS test per AASHTO T361. The granite aggregates were collected from Junction City in Georgia and Nova Scotia in Canada. The aggregate rocks were cut and polished into flat substrates with a uniform texture. An ultrasonic cleaner was used to remove any fine particles embedded into the pores of the aggregate surface. A PG 76-22 (PMA) asphalt binder was received from Mariani Asphalt in Tampa, Florida. Four LAS agents from Road Science and Ingevity were pre-blended into the asphalt binder, respectively, at a dosage rate of 0.5% by weight of binder. In addition to the BBS test, the dynamic shear rheometer and bending beam rheometer tests were conducted to determine the impact of LAS additives on the Superpave PG of the PMA asphalt binder. The laboratory test results are presented as follows.

BINDER BOND STRENGTH TEST

The effect of water (i.e., moisture conditioning) on the bond strength of the asphalt-aggregate system was evaluated via pull-off tensile strength (POTS) of a dry system and wet system after 48-hour water immersion. Figure 20 shows an aggregate substrate with pull-off stubs affixed in dry and wet conditions. The loss of bond strength was calculated by Equation 4.

$$Loss\ of\ Bond\ Strength\ (\%) = \frac{(POTS_{dry} - POTS_{wet})}{POTS_{dry}} * 100 \qquad (4)$$

where $POTS_{dry}$ is the pull-off tensile strength in a dry condition, and $POTS_{wet}$ is the pull-off tensile strength after 48-hour water immersion.

(a) (b)

Figure 20. Binder Bond Strength Test: (a) Aggregate Substrate with Stubs Affixed; and (b) Moisture Conditioning at 40°C

Junction City Aggregate

Figure 21 shows the loss of bond strength of the unaged asphalt-aggregate system, indicating the moisture sensitivity of the asphalt-aggregate system for the Junction City granite. Without the addition of hydrated lime, all samples showed similar sensitivity to moisture damage. The addition of hydrated lime reduced the loss of bond strength of asphalt-aggregate systems except for the one containing LAS3. This demonstrated that the hydrated lime was effective in reducing the moisture susceptibility of the majority of asphalt-aggregate systems. Compared to the systems containing 1.0% hydrated lime, those with 1.5% hydrated lime provided better improvement in resistance to moisture damage.

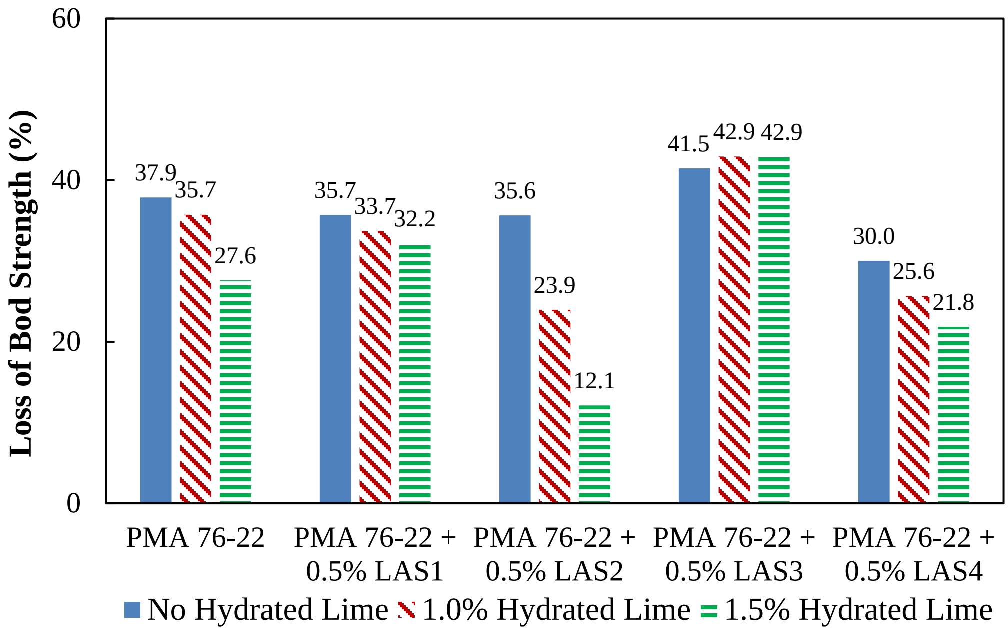


Figure 21. Loss of Bond Strength of Unaged Asphalt – Junction City Aggregate System

Figure 22 shows the wet bond strength of unaged asphalt samples after 48 hours of moisture conditioning. In this study, error bar represents one standard deviation of uncertainty. An analysis of variance (ANOVA) with Tukey honestly significant difference (HSD) test was conducted to statistically rank these results, which is also shown in Figure 22. The confidence level was assigned as 95% (α=0.05). Labels A, A’, and A’’ represent the group of samples that have the statistically highest wet bond strength values with the addition of 0%, 1%, and 1.5% hydrated lime, respectively. As presented, all asphalt samples were grouped as the same level for different dosage of hydrated lime, which demonstrated that no statistical difference was found for the wet bond strength of the evaluated asphalt-aggregate systems. In other words, for unaged asphalt binders, the four investigated LAS additives performed similarly with regard to the resistance to moisture damage. Furthermore, all samples showed cohesive failure (i.e., failure within the asphalt binder) in dry and wet conditions as shown in Figure 23.

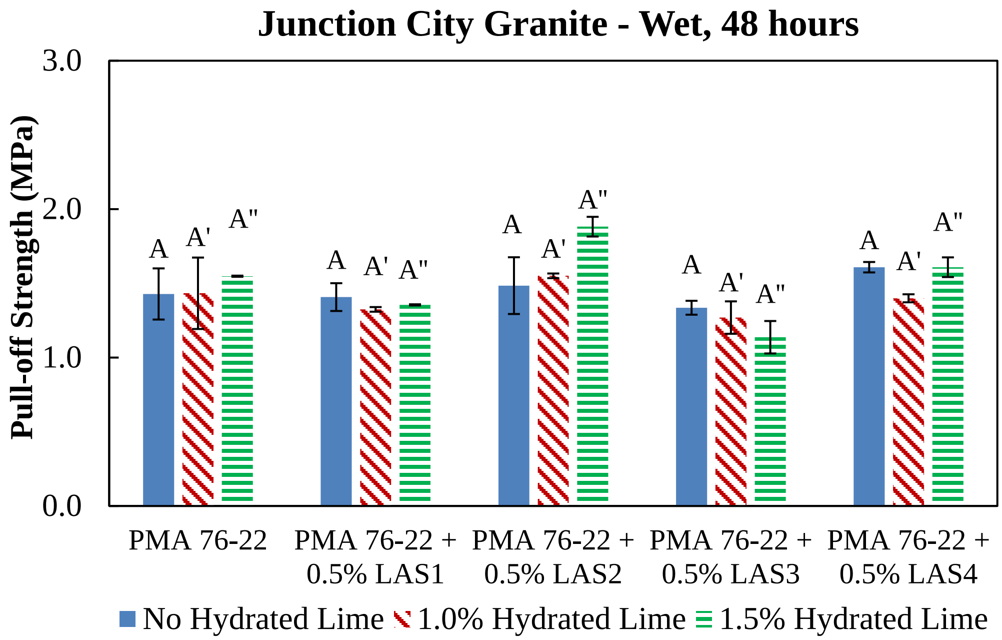


Figure 22. Wet Bond Strength of Unaged Asphalt – Junction City Aggregate System

Figure 23. Cohesive Failure in the BBS Test

To differentiate the performance of LAS additives, the asphalt binders were subjected to long-term aging, including Rolling Thin-Film Oven aging (RTFO, AASHTO T240) followed by 20 hours of aging in the Pressure Aging Vessel (PAV, AASHTO R28) at 100°C. Note that asphalt binders that are more susceptible to oxidative aging are more prone to moisture damage, since the carboxylic acids generated by aging tend to be removed easily from the aggregate in the presence of water (*Robertson, 2000*).

Figure 24 shows the calculated loss of bond strength of the aged asphalt binder samples, indicating the moisture sensitivity of each asphalt-aggregate system. As can be seen, the PG 76-22 (PMA) binder modified with 0.5% of LAS1 showed higher resistance to moisture damage in comparison with all other binders, regardless of the concentration of hydrated lime. Furthermore, the LAS1 additive showed better performance against moisture damage when hydrated lime was not included in the asphalt-aggregate system. This finding differs from the previous results of the unaged binders, where the addition of hydrated lime decreased the moisture resistance of asphalt binder. However, this behavior might be attributed to changes in the composition of the evaluated asphalt binders with oxidative aging.

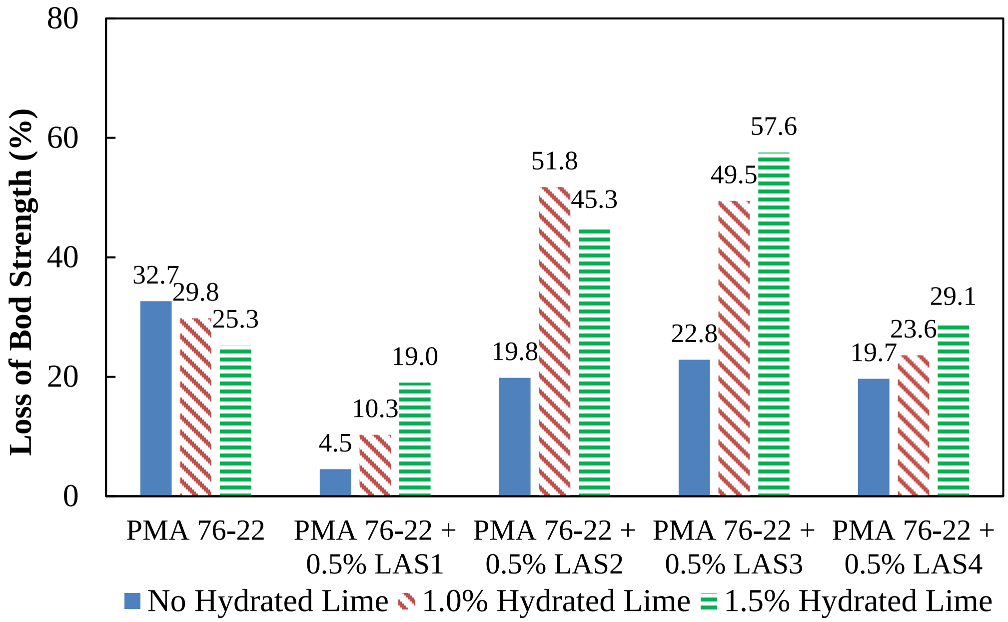


Figure 24. Loss of Bond Strength of Aged Asphalt – Junction City Aggregate System

Figure 25 presents the wet bond strength of aged asphalt samples after 48 hours of moisture conditioning. The ANOVA with Tukey HSD test was also conducted to statistically differentiate these results. As can be seen, no matter what dosage of hydrated lime was used, the LAS1 additive always outperformed other LAS additives in terms of POTS. This confirmed that the LAS1 additive had the best performance against moisture.

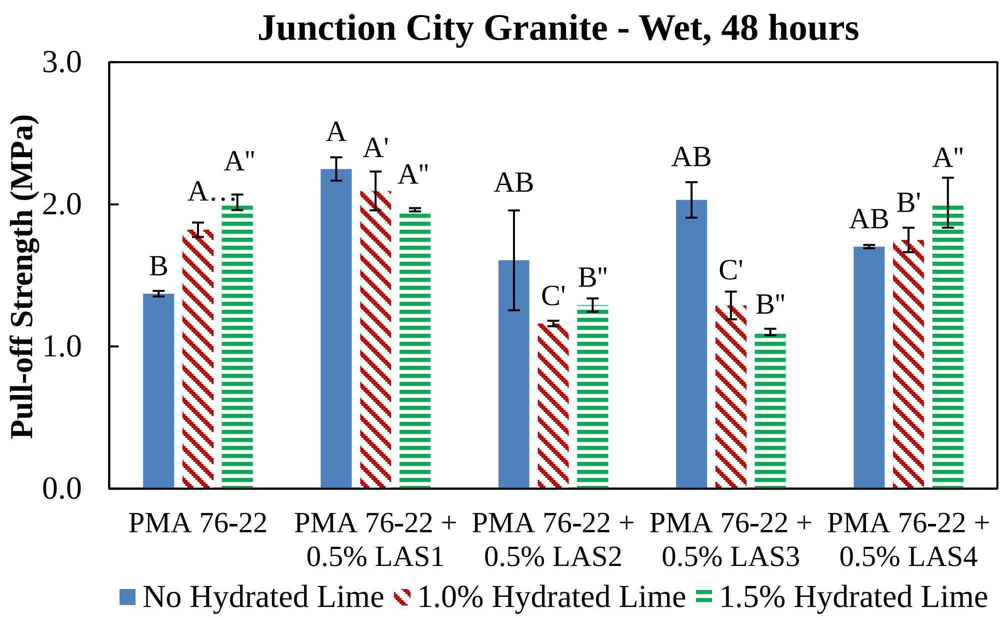
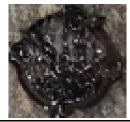
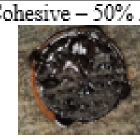


Figure 25. Wet Bond Strength of Aged Asphalt – Junction City Aggregate System

In addition, this study also investigated the failure types of each asphalt-aggregate system, which are summarized in Table 8. The conditioning of specimens in water caused a significant reduction in bond strength and, in some cases, a change in failure mode from cohesive to adhesive at the interface of aged asphalt-aggregate systems. The change in failure mode was expected since water penetrated through the aggregate, which was a porous material, and hence weakened the bond at the asphalt-aggregate interface. The longer the conditioning time in water, the weaker the bond at the interface, which resulted in lower measured pull-off strength. After 48 hours of moisture conditioning, nearly all of the additives demonstrated some degree of adhesive failure. Based on the visual observation, LAS1 additive had slightly less adhesive failure than LAS3. This also indicates that LAS1 additive is better for improving the moisture resistance of granite aggregate from Junction City Mining in Georgia.

Table 8. Failure Mode of Aged Asphalt – Junction City Aggregate System in BBS Test

Sample	Conditioning Time @ 40C	RTFO+20 hrs PAV@100ºC		
		0% Hydrated Lime	1% Hydrated Lime	1.5% Hydrated Lime
PMA 76-22 (SBS)	**0**	Cohesive	Cohesive	Cohesive
	48	50% Cohesive – 50% Adhesive 	50% Cohesive – 50% Adhesive 	Cohesive > Adhesive 
PMA + LAS1	**0**	Cohesive	Cohesive	Cohesive
	48	Cohesive > Adhesive 	50% Cohesive – 50% Adhesive 	Cohesive < Adhesive 
PMA + LAS3	**0**	Cohesive	Cohesive	Cohesive
	48	Cohesive > Adhesive 	Adhesive 	Adhesive 
PMA + LAS4	**0**	Cohesive	Cohesive	Cohesive
	48	50% Cohesive – 50% Adhesive 	50% Cohesive – 50% Adhesive 	50% Cohesive – 50% Adhesive 
PMA + LAS2	**0**	Cohesive	Cohesive	Cohesive
	48	Adhesive 	Adhesive 	Adhesive 

Nova Scotia Aggregate

Considering that the long-term aging of asphalt binder is better to differentiate the performance of LAS additives for the Junction City aggregate, the BBS test was only conducted for the aged asphalt – Nova Scotia aggregate system. Figure 26 shows the calculated loss of bond strength of the aged system, indicating the moisture susceptibility of each binder in the presence of the Nova Scotia granite aggregate. For the PG 76-22 (PMA) binder, the addition of 1% and 1.5% hydrated lime substantially reduced the loss of bond strength of the aged asphalt – Nova Scotia aggregate system. This demonstrated that hydrated lime effectively enhanced the moisture resistance of this specific “base binder + granite aggregate” system. In contrast, the loss of bond strength slightly increased when the PG 76-22 (PMA) binder was modified with LAS additives (with no hydrated lime added). In addition, the combination of hydrated lime and LAS additives did not significantly reduce the moisture susceptibility of the base binder in the presence of the Nova Scotia granite. Accordingly, the sole use of hydrated lime seemed to provide the best performance against moisture to the base binder. Nonetheless, the additive of LAS2 consistently outperformed the other three LAS additives in terms of the loss of bond strength value for the aged asphalt – Nova Scotia aggregate system. This conclusion was drawn on the basis of the BBS test results. Whether the inclusion of LAS additive is detrimental to the evaluated asphalt-aggregate system will be further investigated through mixture performance testing.

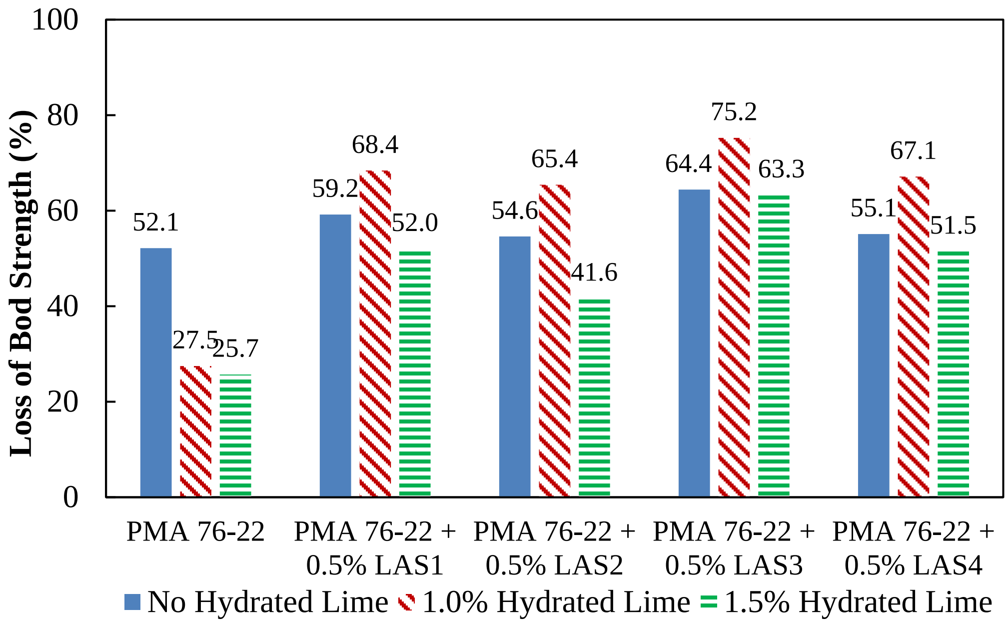


Figure 26. Loss of Bond Strength of Aged Asphalt – Nova Scotia Aggregate System

Figure 27 presents the wet bond strength of aged asphalt – Nova Scotia aggregate system after 48 hours of moisture conditioning. An ANOVA with HSD test was conducted to statistically rank these results, which is also shown in Figure 27. The confidence level was assigned as 95% (α=0.05). Labels A, A', and A'' represent the group of samples that have the statistically highest wet bond strength values with the addition of 0%, 1%, and 1.5% hydrated lime, respectively. As presented, regardless of the dosage of hydrated lime used, the PG 76-22 (PMA) binder without any LAS additive always showed the highest wet bond strength. When comparing among the LAS additives, LAS2 additive had statistically higher wet bond strength at all three hydrated lime contents. This confirmed that LAS2 additive performed better than the other three LAS additives in terms of the reduction of the moisture susceptibility of the base binder in the presence of the Nova Scotia granite.

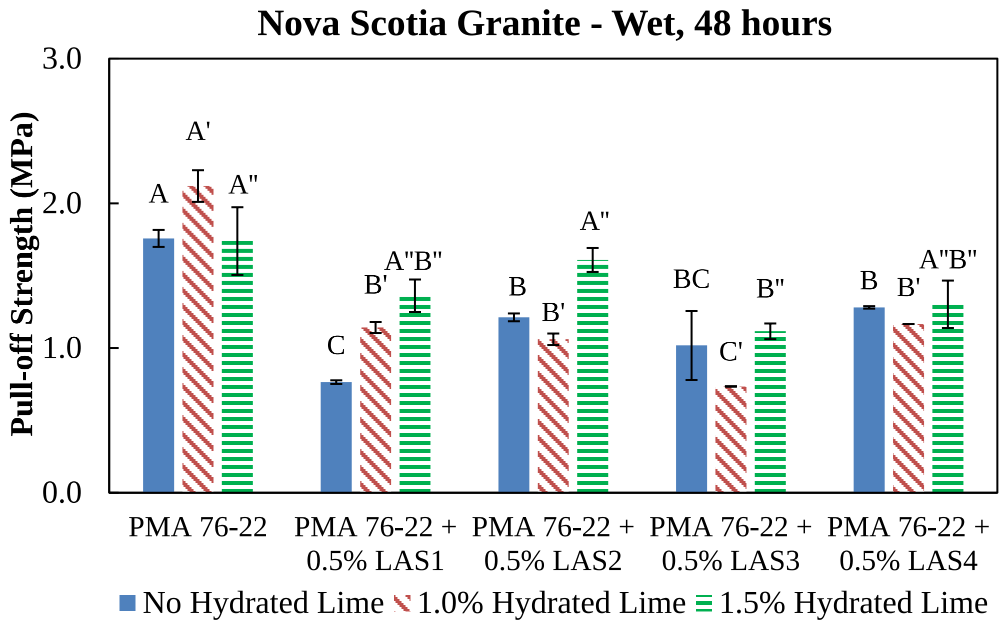
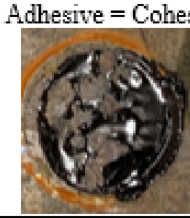
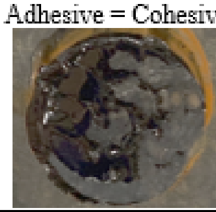


Figure 27. Wet Bond Strength of Aged Asphalt – Nova Scotia Aggregate System

In addition, this study also investigated the failure types of each asphalt-aggregate system, which are summarized in Table 9. As can be seen, the conditioning of specimens in water caused a significant reduction in bond strength and, in most cases, a change in failure mode from cohesive to adhesive at the interface of aged asphalt – Nova Scotia aggregate systems. After 48 hours of moisture conditioning, all of the LAS additives demonstrated adhesive failure. This indicates a weak bond between the binder treated with LAS additives and the aggregate surface. Based on the visual observation, LAS2 additive had slightly less adhesive failure than the other three LAS additives.

Table 9. Failure Mode of Aged Asphalt – Nova Scotia Aggregate System in BBS Test

Sample	Conditioning Time @ 40C	RTFO+20 hrs PAV@100ºC		
		0% Hydrated Lime	1% Hydrated Lime	1.5% Hydrated Lime
PMA 76-22 (SBS)	**0**	Cohesive	Cohesive	Cohesive
	48	Cohesive >>> Adhesive 	Cohesive >>> Adhesive 	Cohesive >>> Adhesive 
PMA + LAS1	**0**	Cohesive	Cohesive	Cohesive
	48	Adhesive >>> Cohesive 	Adhesive >>> Cohesive 	Adhesive = Cohesive 
PMA + LAS3	**0**	Cohesive	Cohesive	Cohesive
	48	Adhesive >>> Cohesive 	Adhesive >> >Cohesive 	Adhesive >>> Cohesive 
PMA + LAS4	**0**	Cohesive	Cohesive	Cohesive
	48	Adhesive >>> Cohesive 	Adhesive 	Adhesive >>> Cohesive 
PMA + LAS2	**0**	Cohesive	Cohesive	Cohesive
	48	Adhesive = Cohesive 	Adhesive > Cohesive 	Adhesive = Cohesive 

BINDER PG TESTS

Binder PG tests including Brookfield viscosity, dynamic shear rheometer (DSR), and bending beam rheometer (BBR) tests were performed to determine the performance grading of the base binder (PG 76-22 [PMA]) and the blended binders with the four LAS additives. Tables 10-14 show the results of binder PG tests.

Table 10. PG Results of PG 76-22 (PMA) Base Binder

Original Binder: Mariani PMA 76-22				
Test Method			**Test Result**	**Specification**
Rotational Viscosity @ 135°C, AASHTO T 316, PaS			1.475	≤ 3 PaS
Dynamic Shear Rheometer, AASHTO T 315				
Test Temperature, °C	G*, kPa	Phase Angle δ, °	G* / sinδ, kPa	≥ 1.00 kPa
76	1.25	67.8	1.35	
82	0.77	68.9	0.82	
Rolling Thin Film (RTFO) Aged Binder, AASHTO T 240				
Mass Change, %			-0.062	≤ 1.00%
Dynamic Shear Rheometer, AASHTO T 315				
Test Temperature, °C	G*, kPa	Phase Angle δ, °	G* / sinδ, kPa	≥ 2.20 kPa
76	2.78	63.4	3.11	
82	1.69	84.7	1.87	
Pressure Aging Vessel (PAV) Aged Binder, AASHTO R28				
Dynamic Shear Rheometer, AASHTO T 315				
Test Temperature, °C	G*, kPa	Phase Angle δ, °	G* sinδ, kPa	≤ 5,000 kPa
22	6848	41.7	4555	
19	10090	39.39	6403	
Bending Beam Rheometer (BBR), AASHTO T313				
Test Temperature, °C				≤ 300 MPa ≥ 0.300
-12	Stiffness, MPa		133	
	m-value		0.320	
-18	Stiffness, MPa		270	
	m-value		0.290	

Table 11. PG Results of PG 76-22 (PMA) Binder with LAS1

Original Binder: PMA 76-22 with 0.5% LAS1				
Test Method			**Test Result**	**Specification**
Rotational Viscosity @ 135°C, AASHTO T 316, PaS			1.49	≤ 3 PaS
Dynamic Shear Rheometer, AASHTO T 315				
Test Temperature, °C	G*, kPa	Phase Angle δ, °	G* / sinδ, kPa	≥ 1.00 kPa
76	1.31	68.6	1.40	
82	0.79	70.2	0.84	
Rolling Thin Film (RTFO) Aged Binder, AASHTO T 240				
Mass Change, %				≤ 1.00%
Dynamic Shear Rheometer, AASHTO T 315				
Test Temperature, °C	G*, kPa	Phase Angle δ, °	G* / sinδ, kPa	≥ 2.20 kPa
76	2.34	64.6	2.59	
82	1.41	66.0	1.54	
Pressure Aging Vessel (PAV) Aged Binder, AASHTO R28				
Dynamic Shear Rheometer, AASHTO T 315				
Test Temperature, °C	G*, kPa	Phase Angle δ, °	G* sinδ, kPa	≤ 5,000 kPa
22	5610	44.8	3950	
19	8560	42.1	5740	
Bending Beam Rheometer (BBR), AASHTO T313				
Test Temperature, °C				≤ 300 MPa ≥ 0.300
-12	Stiffness, MPa		136	
	m-value		0.332	
-18	Stiffness, MPa		275	
	m-value		0.287	

Table 12. PG Results of PG 76-22 (PMA) Binder with LAS2

Original Binder: PMA 76-22 with 0.5% LAS2				
Test Method			**Test Result**	**Specification**
Rotational Viscosity @ 135ºC, AASHTO T 316, PaS			1.45	≤ 3 PaS
Dynamic Shear Rheometer, AASHTO T 315				
Test Temperature, ºC	G*, kPa	Phase Angle δ, º	G* / sinδ, kPa	≥ 1.00 kPa
76	1.34	68.4	1.44	
82	0.75	70.2	0.90	
Rolling Thin Film (RTFO) Aged Binder, AASHTO T 240				
Mass Change, %				≤ 1.00%
Dynamic Shear Rheometer, AASHTO T 315				
Test Temperature, ºC	G*, kPa	Phase Angle δ, º	G* / sinδ, kPa	≥ 2.20 kPa
76	2.30	64.8	2.54	
82	1.39	66.2	1.52	
Pressure Aging Vessel (PAV) Aged Binder, AASHTO R28				
Dynamic Shear Rheometer, AASHTO T 315				
Test Temperature, ºC	G*, kPa	Phase Angle δ, º	G* sinδ, kPa	≤ 5,000 kPa
25	5423	45.4	3860	
22	7949	43.1	5432	
Bending Beam Rheometer (BBR), AASHTO T313				
Test Temperature, ºC				≤ 300 MPa ≥ 0.300
-12	Stiffness, MPa		135	
	m-value		0.331	
-18	Stiffness, MPa		292	
	m-value		0.282	

Table 13. PG Results of PG 76-22 (PMA) Binder with LAS3

Original Binder: PMA 76-22 with 0.5% LAS3				
Test Method			**Test Result**	**Specification**
Rotational Viscosity @ 135°C, AASHTO T 316, PaS			1.44	≤ 3 PaS
Dynamic Shear Rheometer, AASHTO T 315				
Test Temperature, °C	G*, kPa	Phase Angle δ, °	G* / sinδ, kPa	≥ 1.00 kPa
76	1.27	68.3	1.37	
82	0.76	69.9	0.81	
Rolling Thin Film (RTFO) Aged Binder, AASHTO T 240				
Mass Change, %				≤ 1.00%
Dynamic Shear Rheometer, AASHTO T 315				
Test Temperature, °C	G*, kPa	Phase Angle δ, °	G* / sinδ, kPa	≥ 2.20 kPa
76	2.55	63.8	2.85	
82	1.53	65.2	1.68	
Pressure Aging Vessel (PAV) Aged Binder, AASHTO R28				
Dynamic Shear Rheometer, AASHTO T 315				
Test Temperature, °C	G*, kPa	Phase Angle δ, °	G* sinδ, kPa	≤ 5,000 kPa
22	5600	43.3	3840	
19	8410	40.7	5490	
Bending Beam Rheometer (BBR), AASHTO T313				
Test Temperature, °C				
-12	Stiffness, MPa		139	≤ 300 MPa
	m-value		0.323	≥ 0.300
-18	Stiffness, MPa		307	
	m-value		0.279	

Table 14. PG Results of PG 76-22 (PMA) Binder with LAS4

Original Binder: PMA 76-22 with 0.5% LAS4				
Test Method			**Test Results**	**Specification**
Rotational Viscosity @ 135ºC, AASHTO T 316, PaS			1.51	≤ 3 PaS
Dynamic Shear Rheometer AASHTO T 315				
Test Temperature, ºC	G*, kPa	Phase Angle δ, º	G* / sinδ, kPa	≥ 1.00 kPa
76	1.29	68.4	1.39	
82	0.78	69.7	0.83	
Rolling Thin Film (RTFO) Aged Binder, AASHTO T 240				
Mass Change, %				≤ 1.00%
Dynamic Shear Rheometer AASHTO T 315				
Test Temperature, ºC	G*, kPa	Phase Angle δ, º	G* / sinδ, kPa	≥ 2.20 kPa
76	2.54	63.7	2.83	
82	1.54	64.9	1.70	
Pressure Aging Vessel (PAV) Aged Binder, AASHTO R28				
Dynamic Shear Rheometer AASHTO T 315				
Test Temperature, ºC	G*, kPa	Phase Angle δ, º	G* sinδ, kPa	≤ 5,000 kPa
25	4560	45.8	3270	
22	6927	43.1	4730	
Bending Beam Rheometer (BBR) AASHTO T313				
Test Temperature, ºC				≤ 300 MPa ≥ 0.300
-12	Stiffness, MPa		128	
	m-value		0.326	
-18	Stiffness, MPa		301	
	m-value		0.283	

Table 15 summarizes the continuous grade temperatures and the final PG of the binders with and without LAS additives. The addition of 0.5% LAS additives had negligible impacts on the high and low temperature grades of the PG 76-22 (PMA) binder. After addition of each LAS additive, the PG of the asphalt blends was equal to the base binder PG (i.e., 76-22). This confirmed that, at this dosage, none of the LAS additives had a detrimental effect on the PG of PG 76-22 (PMA) binder.

Table 15. Summary of PG Test Results of PMA 76-22 Binders Blended with LAS Additives

Test Parameter	**Binder Type**				
	PMA 76-22	PMA 76-22 + LAS1	PMA 76-22 + LAS2	PMA 76-22 + LAS3	PMA 76-22 + LAS4
T_{crit} @ High Temp. *original*	79.6	80.0	80.7	79.6	79.8
T_{crit} @ High Temp. *after RTFO*	80.1	77.9	77.7	78.9	79.0
T_{crit} @ Intermediate Temp. *after 20 h PAV*	21.2	20.1	22.7	19.8	21.5
T_{crit} @ Low Temp. *Stiffness*	-28.9	-28.7	-28.2	-27.8	-28.0
T_{crit} @ Low Temp. *m-value*	-26.0	-26.3	-25.8	-25.1	-25.6
Delta T_c	-2.9	-2.4	-2.4	-2.7	-2.4
PG	76-22	76-22	76-22	76-22	76-22

CHAPTER 5. INFLUENCE OF ANTI-STRIP ADDITIVES ON MOISTURE SUSCEPTIBILITY OF ASPHALT MIXTURE

This chapter assesses the influence of anti-strip additives on the moisture susceptibility of FC-5 asphalt mixtures using mixture performance tests. The FC-5 mixtures were fabricated in accordance with two mix designs provided by FDOT. One design used the Junction City granite aggregates, while the other used the Nova Scotia granite aggregates. The compacted specimens with 6-inch (150 mm) in diameter and 4.3-4.7 inch (110-120 mm) in height were conditioned in the APWS to simulate the long-term exposure of FC-5 mixtures to water infiltration, vapor diffusion, and thermal and ultraviolet oxidation. The APWS conditioning parameters are presented in Table 16. Figure 28 shows a photo of FC-5 specimens in the APWS. The water spray speed was determined based on the local weather condition in Florida.

Table 16. Conditioning Parameters for Asphalt Pavement Weathering System

Conditioning Parameter	Parameter Value
Conditioning Time	1000 and 2000 hours
Conditioning Cycle	1 hour (51-min UV Sunlight and 9-min Rain and Sunlight)
Rain Water Temperature	50±5°F (10±2.5°C)
Water Spray Rate	0.5 gal/ft^2/hour
Conditioning Temperature	150±5°F (65±2.5°C)
UV Intensity	5.75 nW/cm^3

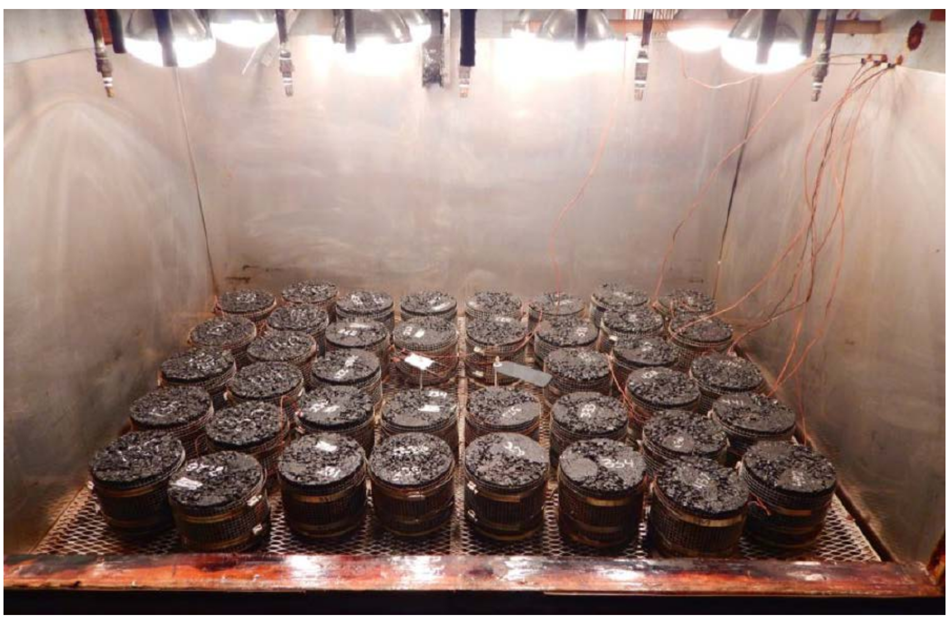


Figure 28. FC-5 Specimens in Asphalt Pavement Weathering System

Mixture performance tests, including the Cantabro, TSR, and HWTT tests, were then conducted on the APWS conditioned and unconditioned FC-5 mixtures to evaluate their moisture susceptibility and durability. Table 17 summarizes the number of replicate specimens used in the performance tests. These tests were performed at the National Center for Asphalt Technology, and the test results are provided in the following sections.

Table 17. Summary of FC-5 Specimen Replicates for Performance Tests

Aggregate Type	Additive Type	Test Method	Number of Specimens			
			Unconditioned	1 F-T Cycle*	1000-hr APWS	2000-hr APWS
Junction City Granite	1% HL**	Cantabro	3	NA	3	3
		IDT***	3	3	3	3
		HWTT	4	NA	4	4
	1% HL + LAS_1	Cantabro	3	NA	3	3
		IDT	3	3	3	3
		HWTT	4	NA	4	4
	1.5% HL	Cantabro	3	NA	3	3
		IDT	3	3	3	3
		HWTT	4	NA	4	4
	1.5% HL + LAS_1	Cantabro	3	NA	3	3
		IDT	3	3	3	3
		HWTT	4	NA	4	4
Nova Scotia Granite	1% HL	Cantabro	3	NA	3	3
		IDT	3	3	3	3
		HWTT	4	NA	4	4
	1% HL + LAS_2	Cantabro	3	NA	3	3
		IDT	3	3	3	3
		HWTT	4	NA	4	4
	1.5% HL	Cantabro	3	NA	3	3
		IDT	3	3	3	3
		HWTT	4	NA	4	4
	1.5% HL + LAS_2	Cantabro	3	NA	3	3
		IDT	3	3	3	3
		HWTT	4	NA	4	4

Note: * F-T = Freeze-Thaw; ** HL = Hydrated Lime; ***IDT = Indirect Tension.

MIX DESIGN VERIFICATION

Prior to performance testing, the two FC-5 mix designs provided by FDOT were verified by comparing against the performance criteria recommended in NCHRP Project 1-55 (*Watson et al., 2018*). Table 18 presents the mix design verification results. Note that this study did not verify the optimum binder content of FC-5 mixtures using the pie plate method. The air voids of FC-5 asphalt mixtures were measured using the CoreLok method. As shown in Table 18, the FC-5 mixtures met all the performance criteria recommended by Watson et al. (*2018*), indicating that they had good durability and moisture susceptibility.

Table 18. FC-5 Mix Design Verification

Mix Performance	Junction City FC-5 Mix	Nova Scotia FC-5 Mix	Performance Criteria
Air Voids (%)	17.5	19.0	≥15
Cantabro loss (%)	10.5	12.5	≤20
Wet Tensile strength (psi)	59.6	61.0	≥50
Tensile strength ratio	0.93	0.94	≥0.70
HWTT Rut depth at 20,000 passes (inch)	0.25	0.23	≤0.5

Note: 1 psi = 6.89 kPa; 1 inch = 25.4 mm.

CANTABRO TEST

Figure 29 shows the Cantabro test results of Junction City FC-5 asphalt mixtures containing four different combinations of anti-strip additives, before and after APWS conditioning for 1,000 and 2,000 hours. According to ASTM D7064, the acceptable criterion for the Cantabro mass loss is maximum 20% for unaged specimens and 30% for long-term oven aged specimens. Therefore, all of the Junction City FC-5 mixtures tested in this study met the minimum requirements for the durability performance of OGFC mixtures. It is clearly seen that the Cantabro mass loss of FC-5 mixtures increased with APWS conditioning time. No significant difference in the Cantabro mass loss was observed for the unconditioned specimens with and without anti-strip agents. However, for the APWS-conditioned specimens, the different combinations of anti-strip additives showed various influences on mixture durability. After 2,000-hour APWS conditioning, the Junction City FC-5 mixtures with 1% hydrated lime plus LAS had much lower Cantabro mass loss than other specimens, indicating much better durability. However, the addition of 1.5% hydrated lime or 1.5% hydrated lime plus LAS did not show any significant enhancement to the durability of Junction City mixtures.

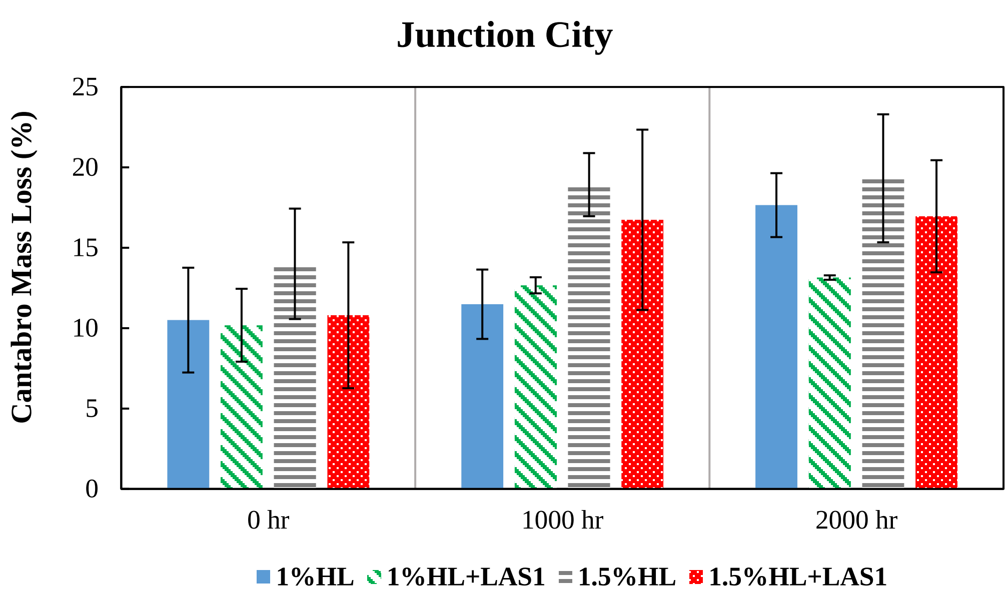


Figure 29. Cantabro Test Results of Junction City FC-5 Asphalt Mixtures Containing Different Combinations of Anti-Strip Agents

To quantify the resistance of asphalt mixtures to weathering, the change in Cantabro loss was calculated as a ratio of the Cantabro loss of mixture after APWS conditioning to that of mixture before conditioning. As shown in Figure 30, increasing the amount of hydrated lime from 1% to 1.5% effectively improved the resistance of Junction City FC-5 mixtures to 2,000-hour of weathering. Overall, the Junction City FC-5 mixtures with 1% hydrated lime and LAS showed the greatest resistance to APWS weathering.

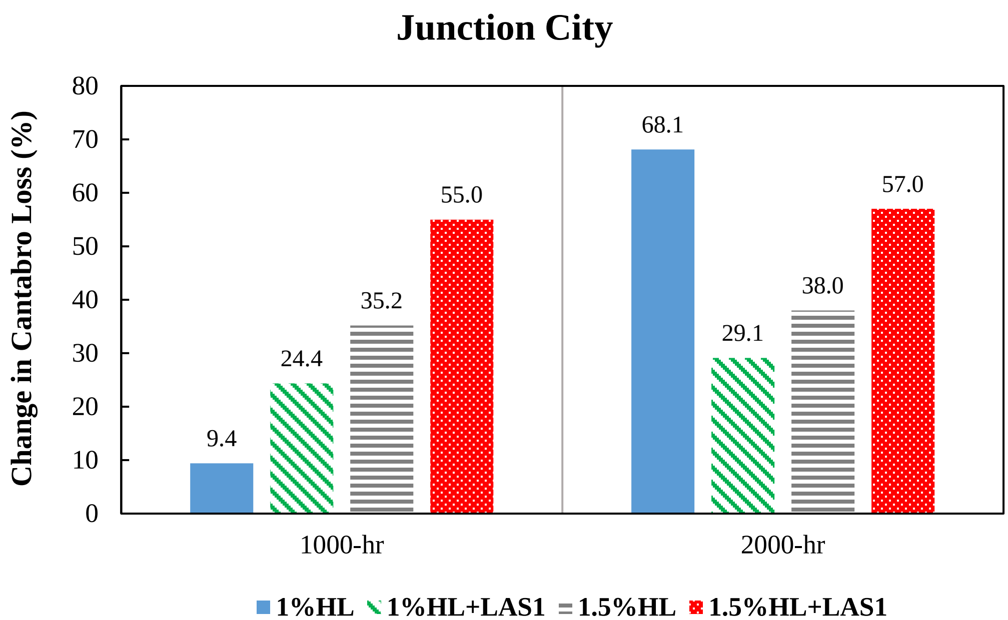


Figure 30. Influence of Anti-Strip Additives on Cantabro Mass Loss of Junction City FC-5 Asphalt Mixtures

Figure 31 presents the Cantabro test results of Nova Scotia FC-5 asphalt mixtures. Compared to the Junction City mixtures, the Nova Scotia specimens initially had comparable Cantabro mass loss without weathering, but had much greater Cantabro mass loss after 2,000-hour weathering. After 2,000-hour weathering, the addition of LAS and extra 0.5% hydrated lime both effectively increased the durability of Nova Scotia FC-5 mixtures. The combination of 1.5% hydrated lime plus LAS provided the best improvement of durability for Nova Scotia FC-5 mixtures.

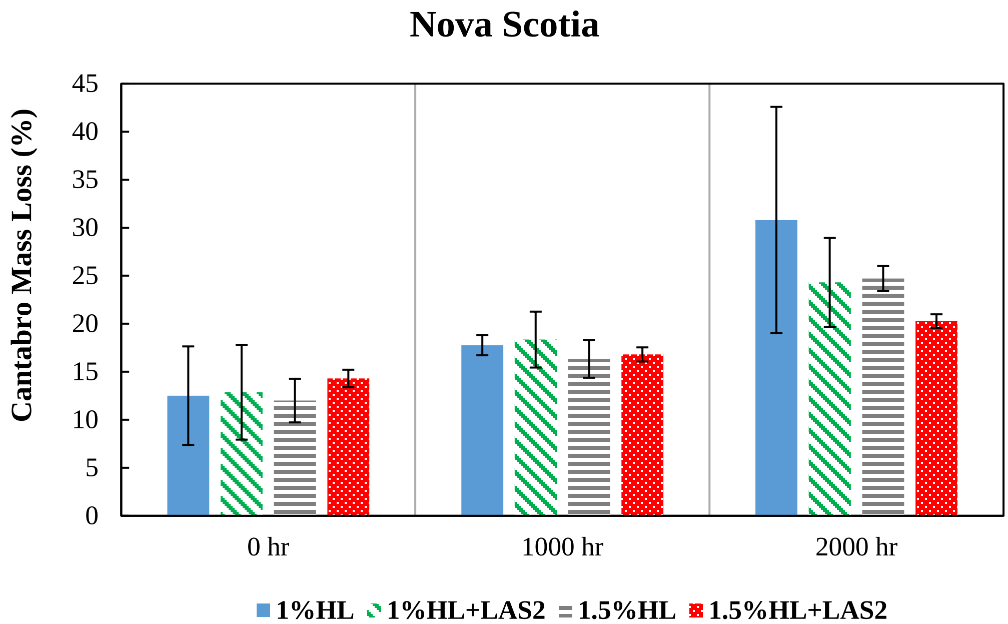


Figure 31. Cantabro Test Results of Nova Scotia FC-5 Asphalt Mixtures Containing Different Combinations of Anti-Strip Agents

As shown in Figure 32, increasing the amount of hydrated lime from 1% to 1.5% also effectively improved the resistance of Nova Scotia FC-5 mixtures to APWS weathering. Overall, the Nova Scotia FC-5 mixtures with 1.5% hydrated lime plus LAS showed the highest resistance to long-term weathering.

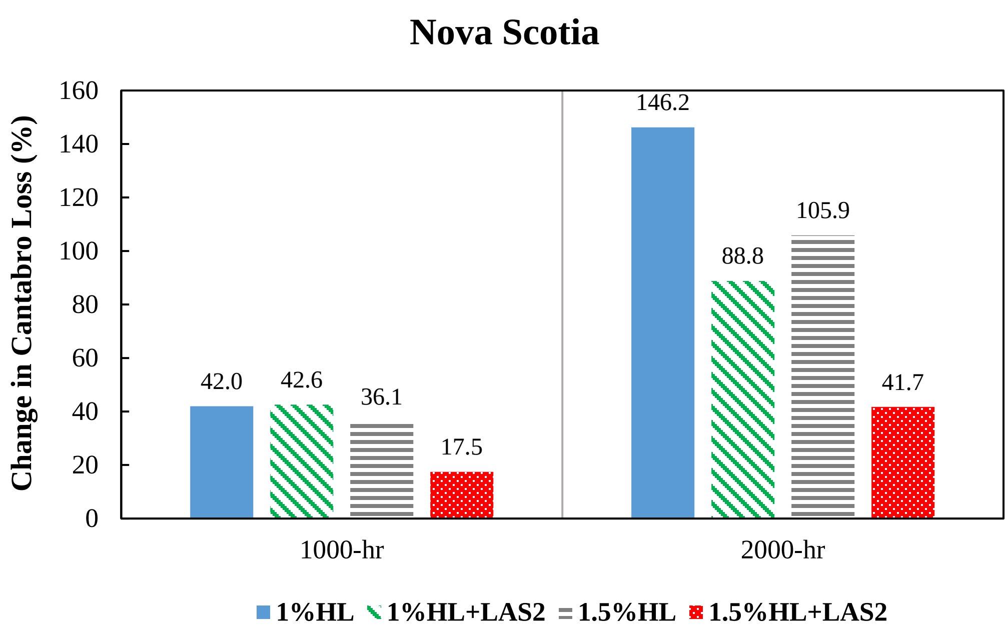
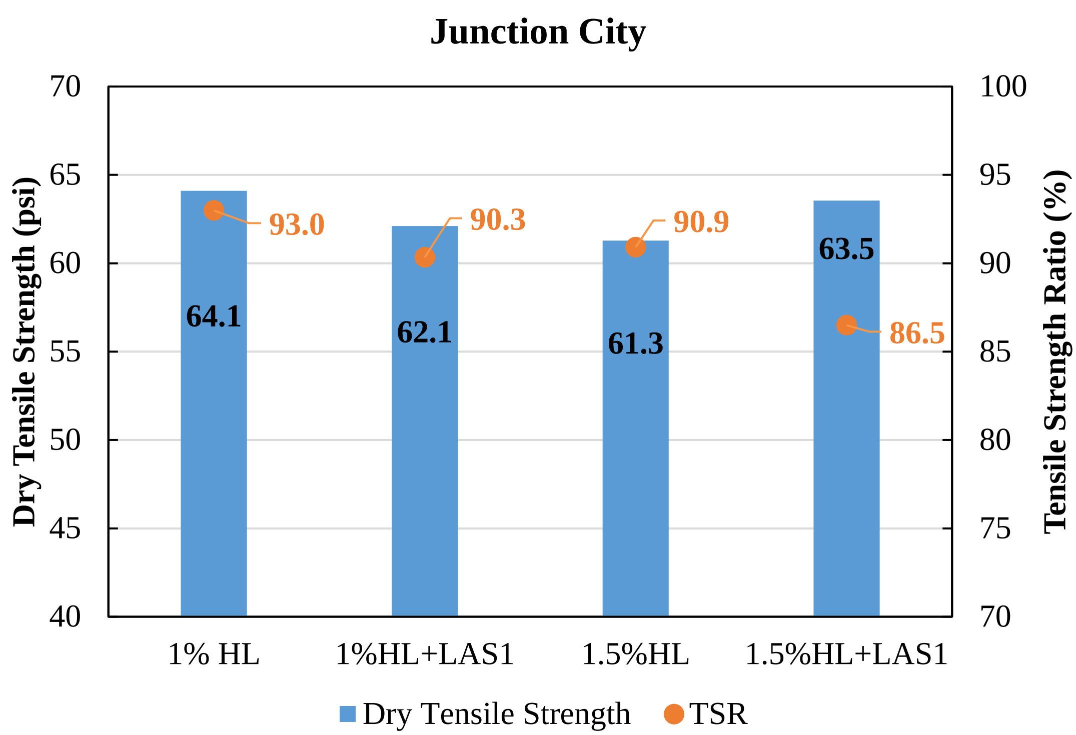


Figure 32. Influence of Anti-Strip Additives on Cantabro Mass Loss of Nova Scotia FC-5 Asphalt Mixtures

The Cantabro test results indicated that the addition of LAS and extra 0.5% hydrated lime could enhance the durability of FC-5 mixtures. However, the effectiveness of LAS and additional hydrated lime in improving the mixture durability was dependent on the source of granite aggregate. In this study, 1% hydrated lime plus LAS and 1.5% hydrated lime plus LAS achieved the best outcomes of durability for Junction City and Nova Scotia FC-5 asphalt mixtures, respectively.

TENSILE STRENGTH RATIO TEST

The indirect tension tests were conducted for the FC-5 asphalt mixtures at dry, one freeze-thaw cycle, and 1,000-hour and 2,000-hour weathering conditions. The TSR is defined as a ratio of the tensile strength of asphalt mixture subjected to one freeze-thaw cycle to that of asphalt mixture at a dry condition. Figure 33 shows the TSR test results for Junction City FC-5 mixtures with various combinations of anti-strip additives. It is notable that the Junction City FC-5 mixtures with 1% hydrated lime showed the highest TSR value. This implied that the addition of LAS or extra 0.5% hydrated lime did not improve the moisture resistance of asphalt mixtures after one freeze-thaw cycle conditioning. One potential explanation is that the control mixture (labeled as 1% HL) had excellent resistance to moisture damage, which might diminish the improvement by the additional anti-strip additives.



Note: 1 psi = 6.89 kPa

Figure 33. Tensile Strength Ratio Test Results for Junction City FC-5 Asphalt Mixtures

Note that the APWS simulated the long-term weathering in the field, which coupled both aging and moisture conditioning. The thermal and UV aging typically stiffens the asphalt mixture, while the moisture conditioning tends to pose an opposite impact. Thus, the key question was how to separate the effect of moisture condition from the weathering process. A modified TSR parameter was proposed to quantify the coupling effects of aging and moisture conditioning on the tensile strength of asphalt mixture, which is defined as a ratio of the average tensile strength of mixture specimens after the APWS conditioning to that of the unconditioned specimens (Equation 5).

$$Modified\ TSR\ (\%) = \frac{Tensile\ Strength_{APWS\ Conditioned}}{Tensile\ Strength_{Unconditioned}} \times 100\% \tag{5}$$

Figure 34 shows the influence of anti-strip additives on the modified TSR of Junction City FC-5 asphalt mixtures. After 1,000-hour and 2,000-hour APWS conditioning, all asphalt mixtures had higher tensile strength, indicating that the strength gain due to aging was more significant than the strength loss due to moisture conditioning. For the mixtures containing 1% hydrated lime, the addition of LAS additive effectively increased their modified TSR values after 1,000-hour and 2,000-hour conditioning. Existing studies indicated that the LAS additives do not accelerate the aging of asphalt mixtures (*Sebaaly et al., 2010; Souliman et al., 2015*). In other words, the LAS additives do not contribute to the strength gain from aging. Thus, the increased tensile strength had to be attributed to the improved moisture resistance of asphalt mixtures. However, for the mixtures containing 1.5% hydrated lime, the LAS additive only slightly improved their modified TSR values. This indicates that the extra hydrated lime diminishes the effectiveness of LAS additives in reducing the moisture susceptibility of asphalt mixtures. It is also shown that increasing the hydrated lime dosage from 1% to 1.5% generally reduced the modified TSR of the Junction City mixtures. This implies that the hydrated lime is an antioxidant that retards the aging of asphalt mixtures. Overall, the Junction City FC-5 mixtures with the addition of 1% hydrated lime and LAS agent exhibited the highest modified TSR after 1,000-hour and 2,000-hour APWS conditioning.

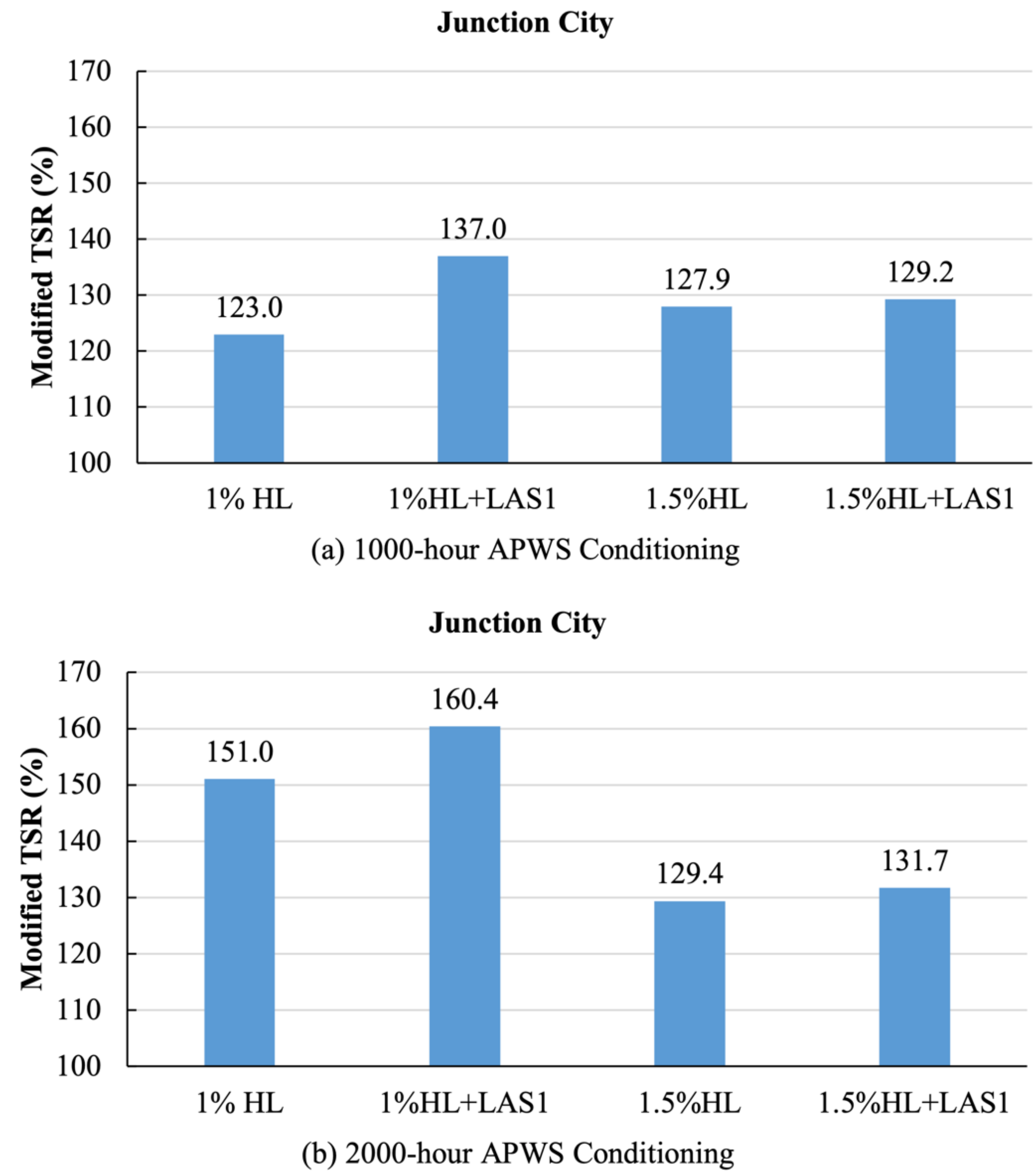
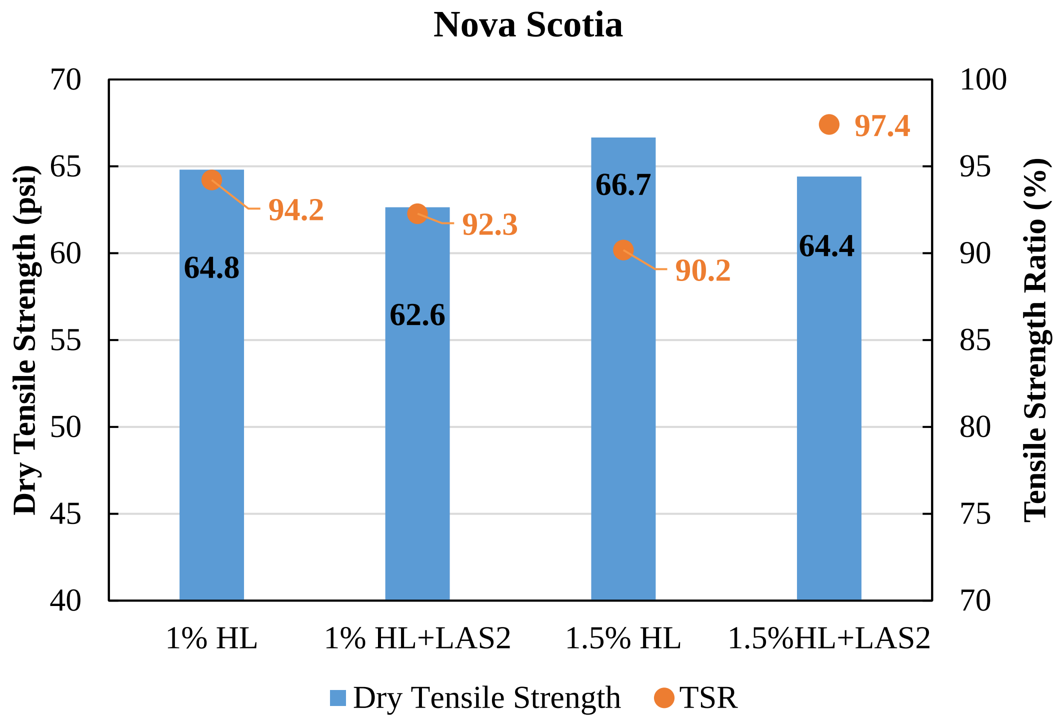


Figure 34. Modified Tensile Strength Ratio for Junction City FC-5 Asphalt Mixtures

Figure 35 shows the TSR test results for Nova Scotia FC-5 asphalt mixtures. Similar to the findings from the Junction City mixtures, the LAS and extra 0.5% hydrated lime showed no significant improvement to the moisture resistance of asphalt mixtures after one freeze-thaw cycle.



Note: 1 psi = 6.89 kPa

Figure 35. Tensile Strength Ratio Test Results for Nova Scotia FC-5 Asphalt Mixtures

Figure 36 presents the effect of anti-strip additives on the modified TSR of Nova Scotia FC-5 asphalt mixtures. The LAS additive was still able to reduce the moisture susceptibility of asphalt mixtures in general. Unlike the Junction City mixtures, the additional 0.5% hydrated lime did not significantly reduce the modified TSR of the Nova Scotia mixtures. This might be because the extra hydrated lime not only mitigated the thermal and UV aging, but also improved the moisture resistance. Under this circumstance, the influence of aging retardation offset that of enhanced moisture stability on tensile strength. Overall, the Nova Scotia FC-5 mixtures with the addition of 1.5% hydrated lime and LAS agent showed the highest modified TSR after 1,000-hour and 2,000-hour APWS conditioning. This was consistent with the findings from the Cantabro test. However, the TSR test results could not discriminate the effectiveness of LAS agent and additional 0.5% hydrated lime in reducing the moisture susceptibility of FC-5 mixtures.

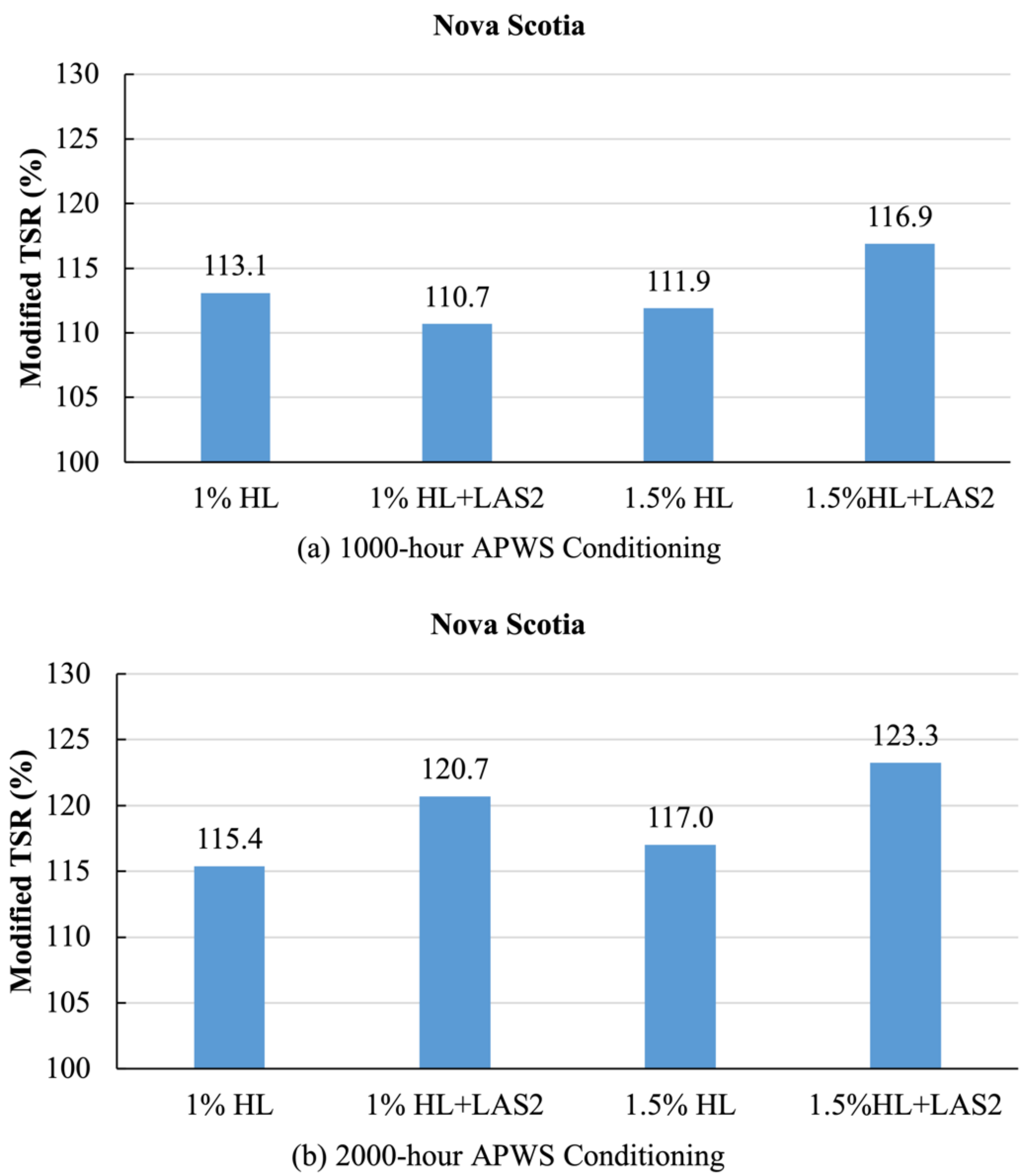


Figure 36. Modified Tensile Strength Ratio for Nova Scotia FC-5 Asphalt Mixtures

HAMBURG WHEEL TRACKING TEST

Figures 37 and 38 show the rut depth of Junction City FC-5 asphalt mixtures after 10,000 and 20,000 wheel passes. These mixtures had average rut depth values, varying from 3 to 5 mm after 10,000 passes and 3 to 6.5 mm after 20,000 passes, indicating that they exhibited acceptable rutting resistance. In general, the APWS conditioning stiffened the asphalt mixtures, and thus improved their rutting performance. After 20,000 wheel passes, none of these mixtures showed any stripping issues, which verifies that the Junction City FC-5 mixtures used in this study had satisfactory durability and excellent resistance to moisture damage. According to the HWTT results shown in Figures 37 and 38, the influence of anti-strip additives did not have a significant impact on the moisture susceptibility and rutting resistance of Junction City FC-5 mixtures.

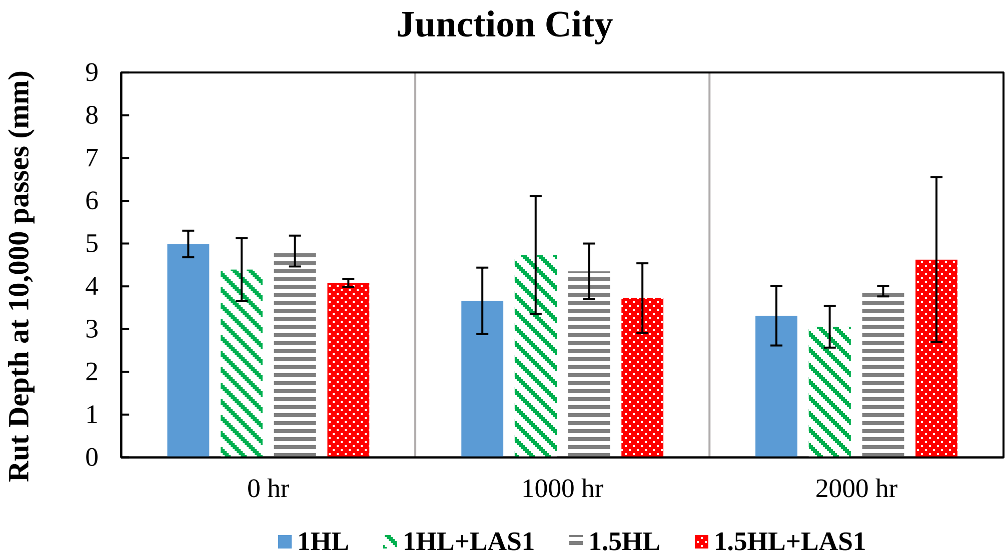


Figure 37. Run Depth of Junction City FC-5 Asphalt Mixtures after 10,000 Passes

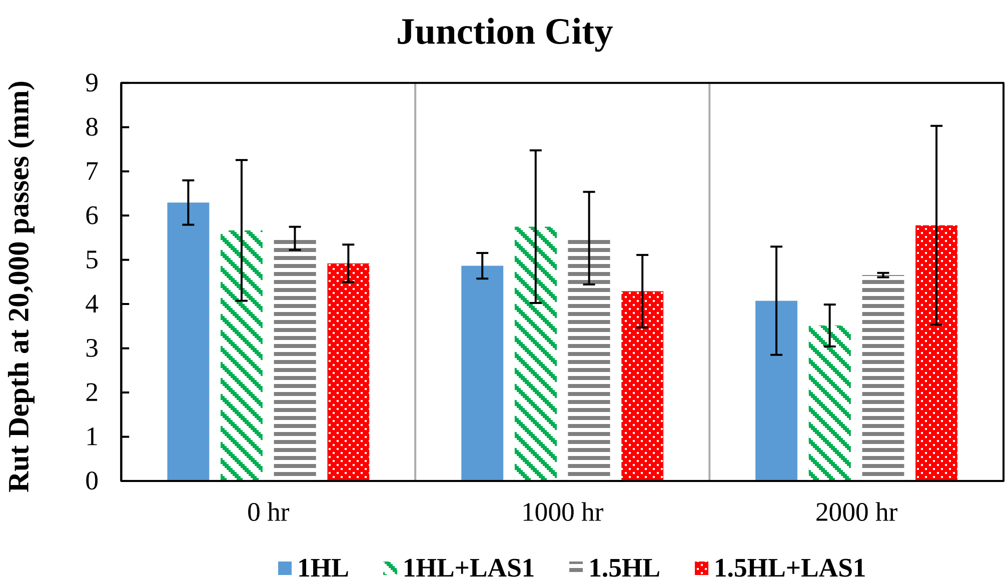


Figure 38. Run Depth of Junction City FC-5 Asphalt Mixtures after 20,000 Passes

Figures 39 and 40 present the rut depth of Nova Scotia FC-5 asphalt mixtures after 10,000 and 20,000 wheel passes. Similarly, the Nova Scotia asphalt mixtures also had adequate resistance to rutting damage, and the long-term exposure to APWS enhanced their rutting resistance as indicated by lower rut depths. It is confirmed that the HWTT results could not reflect the influence of anti-strip additives on the moisture susceptibility of Nova Scotia FC-5 mixtures. After 20,000 wheel passes, only two sets of specimens exhibited late stripping issues. As shown in Figure 41, the unconditioned Nova Scotia asphalt mixtures containing 1.5% hydrated lime had a stripping inflection point (SIP) of 13,888 passes, which increased to 16,963 passes and over 20,000 passes by 1,000-hour and 2,000-hour APWS conditioning, respectively. This may be because aging stiffened the asphalt mixtures, resulting in a delay of stripping.

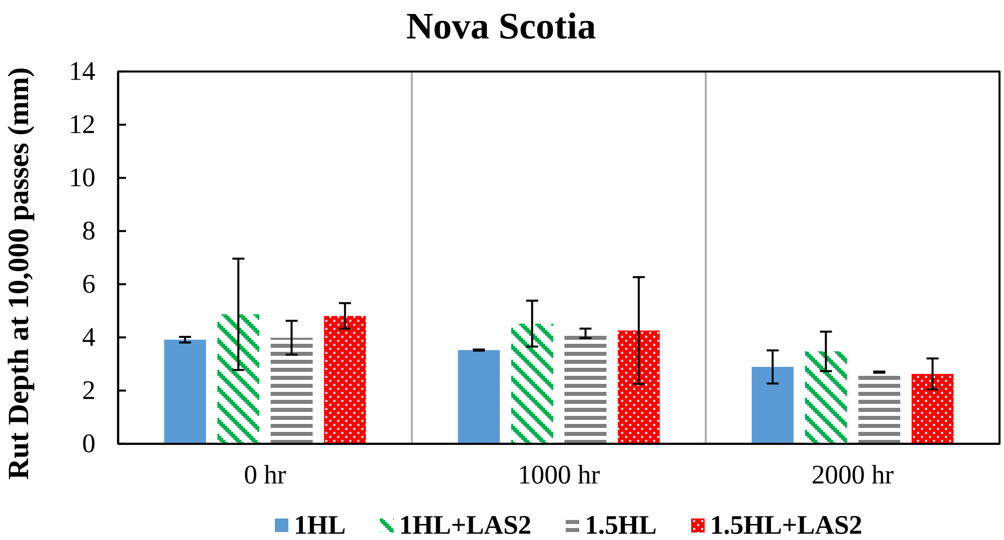


Figure 39. Run Depth of Nova Scotia FC-5 Asphalt Mixtures after 10,000 Passes

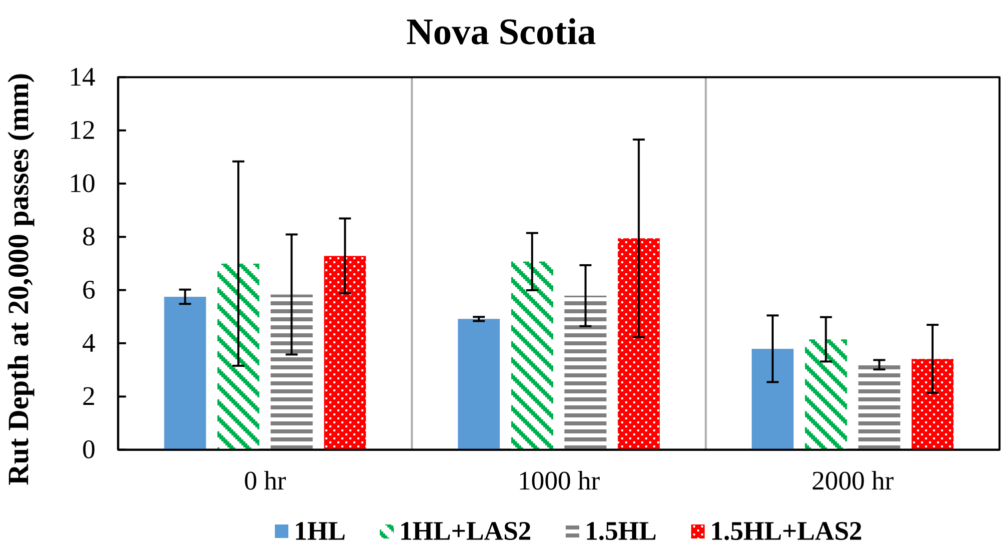
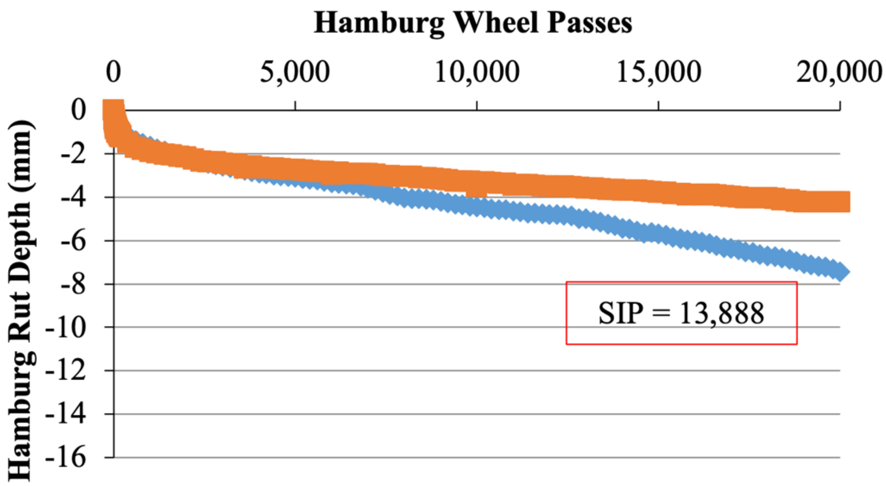
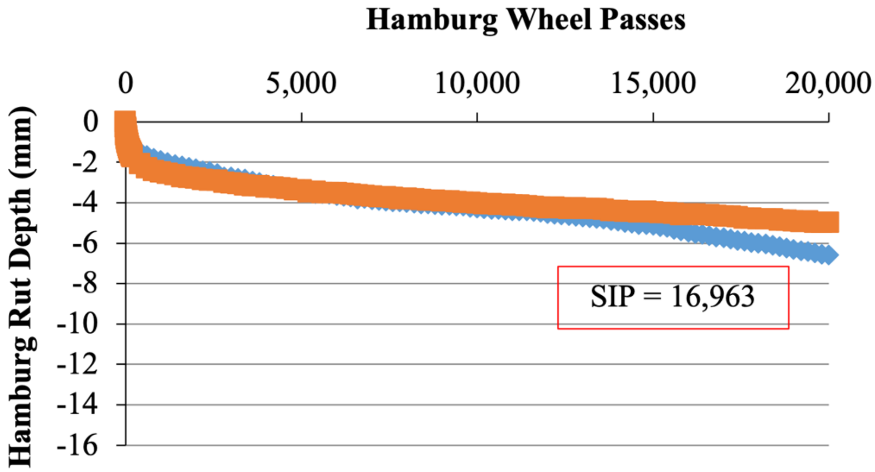


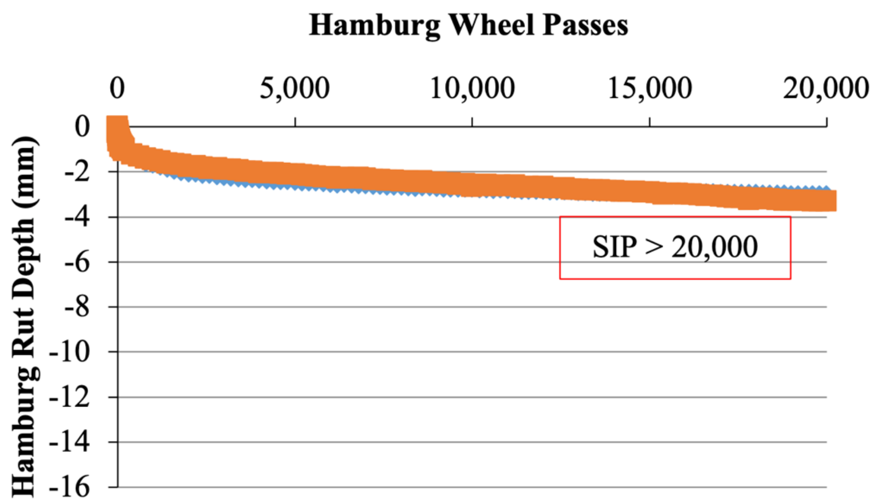
Figure 40. Run Depth of Nova Scotia FC-5 Asphalt Mixtures after 20,000 Passes



(a) Unconditioned



(b) 1000-hour APWS Conditioning



(c) 2000-hour APWS Conditioning

Figure 41. HWTT Results of Nova Scotia FC-5 Asphalt Mixtures Containing 1.5% Hydrated Lime

COST-BENEFIT ANALYSIS

A cost-benefit analysis was conducted for the Junction City and Nova Scotia FC-5 asphalt mixtures that contained different combinations of anti-strip additives. The performance test results demonstrated that only the Cantabro mass loss was capable of differentiating the influences of anti-strip additives on the durability of FC-5 mixtures. According to ASTM D7064, 30% of Cantabro mass loss was determined as the failure threshold for FC-5 mixtures subjected to long-term oven aging.

Figure 42 compares the life span of Junction City FC-5 asphalt mixtures containing different combinations of anti-strip additives. As illustrated, an extrapolation method was used to determine the life span of FC-5 mixtures. According to Grzybowski (*2013*), 3,000-hour APWS conditioning was equivalent to approximately 12 years of weathering in Florida. Thus, based on a linear assumption, 1,000-hour and 2,000-hour APWS conditioning are expected to simulate 4 and 8 years of weathering in the field. The unaged condition represents 0 year of weathering in the field. As shown in Figure 42, the deterioration trend of Cantabro mass loss followed an exponential function with good correlation as shown by the high R^2 values. The maximum service life of FC-5 mixture was assumed as 20 years based on the historical experience with the performance of OGFC mixtures. It was extrapolated that the Junction City mixture with 1% hydrated lime had 17.0 years of service life for reaching a Cantabro mass loss of 30%. The extra 0.5% hydrated lime extended the service life of Junction City FC-5 mixtures by 0.8 year, and the addition of LAS agent prolonged the life span by at least 3 years. However, the addition of 1.5% hydrated lime plus LAS showed no improvement in increasing the life span of Junction City FC-5 mixtures.

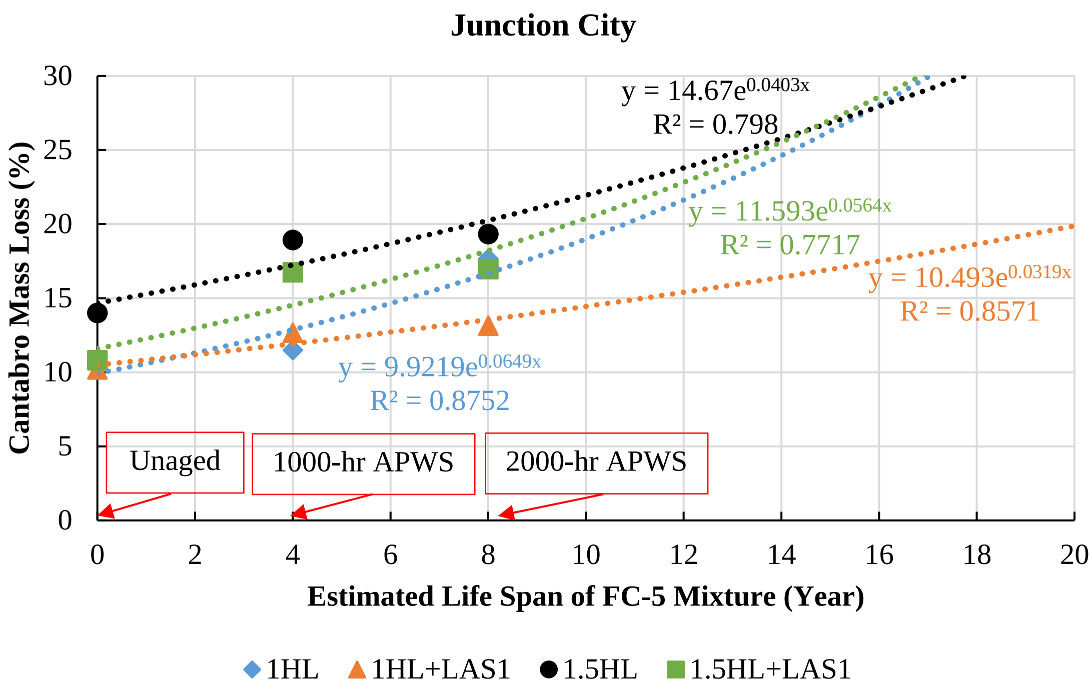


Figure 42. Estimated Life Span of Junction City FC-5 Asphalt Mixtures

Figure 43 presents the life longevity of Nova Scotia FC-5 asphalt mixtures containing different combinations of anti-strip additives. The Nova Scotia mixture with 1% hydrated lime had only 8.0 years of service life. Adding 0.5% more hydrated lime or LAS agent could increase the life span of an asphalt mixture by 2.3-2.5 years, while the combination of both modifications could double its life span. The estimated life span of FC-5 mixtures based on the Cantabro mass loss results are summarized in Table 19.

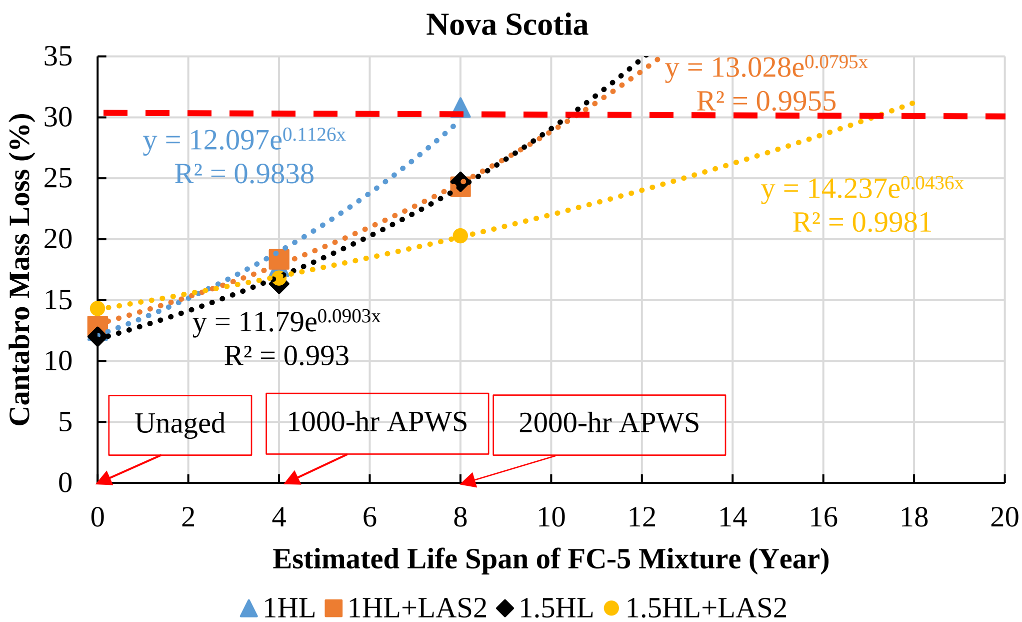


Figure 43. Estimated Life Span of Nova Scotia FC-5 Asphalt Mixtures

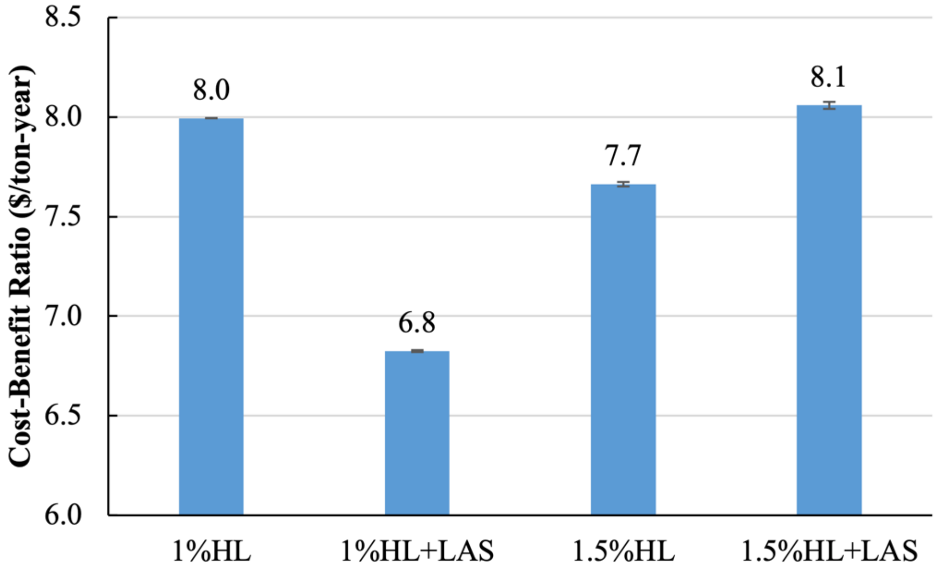
Table 19. Summary of Estimated Life Span of FC-5 Asphalt Mixtures

Additive Type	Estimated Life Span (Years)	
	Junction City FC-5 Mixture	Nova Scotia FC-5 Mixture
1%HL	17.0	8.0
1%HL+LAS	20.0	10.5
1.5%HL	17.8	10.3
1.5%HL+LAS	17.0	17.1

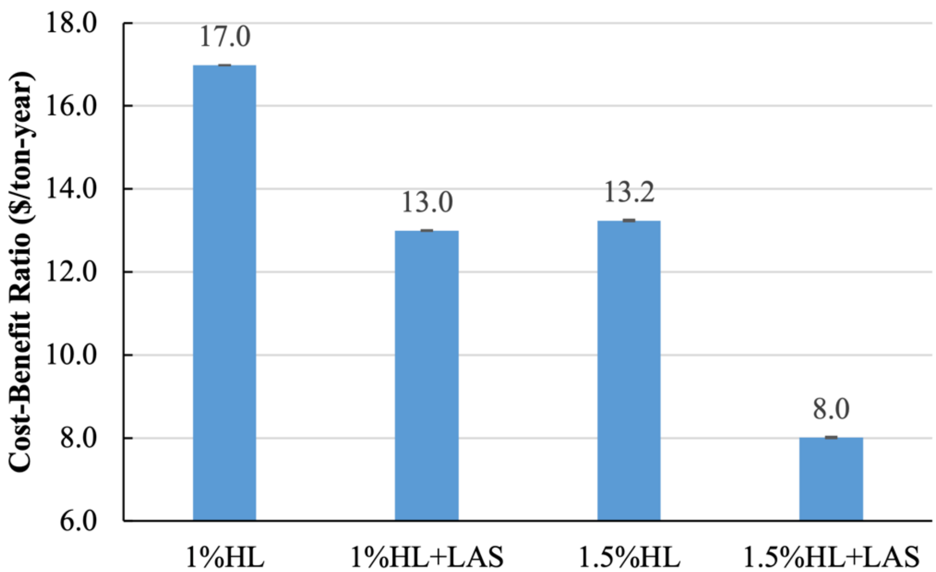
In this study, the unit price of FC-5 mixture was $135.9 per ton, which was obtained from FDOT's online data sheet, entitled "Historical Cost and Other Information" (*FDOT, 2020*). The unit cost for adding 0.5% LAS agent was $0.6 per ton of mixture with a standard deviation of $0.1 (*Sebaaly and Hajj, 2011; Christensen et al., 2015*). The unit cost for adding 0.5% more hydrated lime was $0.5 per ton of mixture with a standard deviation of $0.2 (*Little and Epps, 2006*). The cost-benefit ratio was calculated by Equation 6. A lower cost-benefit ratio represents a better cost-effectiveness.

$$Cost - Benefit\ Ratio = \frac{Material\ Cost}{Life\ Span} \tag{6}$$

Figure 44 shows the cost-benefit ratios for Junction City and Nova Scotia FC-5 asphalt mixtures with different combinations of anti-strip additives. Compared to the unit price of FC-5 mixture, the costs for adding extra 0.5% hydrated lime and LAS agent were insignificant. Therefore, the addition of extra hydrated lime and LAS agent improved the cost-effectiveness of FC-5 mixtures due to improved performance properties and extended life span. For the Junction City mixtures, the addition of 1% hydrated lime and LAS agent achieved the lowest cost-benefit ratio; and for the Nova Scotia mixtures, the addition of 1.5% hydrated lime and LAS agent was the most cost-effective modification.



(a) Junction City FC-5 Asphalt Mixture



(b) Nova Scotia FC-5 Asphalt Mixture

Figure 44. Cost-Benefit Ratio of FC-5 Asphalt Mixtures

CONCLUSIONS

This project evaluated the influences of anti-strip additives on the durability and moisture susceptibility of granite-based FC-5 asphalt mixtures. The laboratory testing involved two granite-based FC-5 mixtures containing 1% hydrated lime, 1% hydrated lime plus 0.5% liquid anti-strip (LAS) additive, 1.5% hydrated lime, and 1.5% hydrated lime plus 0.5% LAS additive. The LAS was added at a dosage rate of 0.5% by weight of asphalt binder, and hydrated lime was added based on the total aggregate weight. Firstly, two sources of granite aggregates were obtained: one from Junction City, Georgia and the other from a regional supplier with an original source from Nova Scotia, Canada. Four types of LAS additives were collected from Road Science and Ingevity. Secondly, the binder bond strength (BBS) test was used to select the LAS agents that provided the best moisture resistance for granite-based FC-5 mixtures. Thirdly, the FC-5 mixtures were fabricated in the laboratory using two FC-5 mix designs provided by FDOT. The specimens were conditioned in the asphalt pavement weathering system (APWS) to simulate the coupling effects of aging and moisture conditioning. Fourthly, mixture performance tests, including Cantabro test, tensile strength ratio (TSR) test, and Hamburg Wheel Tracking Test (HWTT), were conducted to comprehensively evaluate the durability and moisture susceptibility of FC-5 mixtures before and after the APWS conditioning. Finally, a cost-benefit analysis was performed to determine whether the anticipated increased life span of FC-5 mixtures more than offset the increased cost of the additional additives. The major findings of this project are summarized as follows.

- The BBS test was effective in differentiating the moisture susceptibility of an asphalt-aggregate systems, particularly after the asphalt binder was subjected to long-term aging. The additional hydrated lime improved the moisture resistance of the asphalt-aggregate system by increasing the wet bond strength and reducing the loss of bond strength due to moisture conditioning. For the Junction City granite, LAS1 additive outperformed other LAS additives in terms of the wet bond strength and loss of bond strength parameters. For the Nova Scotia granite, LAS2 additive showed the best performance in terms of moisture damage.
- The Cantabro mass loss was capable of differentiating the influences of anti-strip additives on the durability of FC-5 asphalt mixtures. Increasing the APWS conditioning time increased the Cantabro mass loss of FC-5 mixtures. Overall, after long-term weathering, 1% hydrated lime plus LAS, and 1.5% hydrated lime plus LAS achieved the best durability for Junction City granite and Nova Scotia granite FC-5 mixtures, respectively.
- The TSR test results indicated that the addition of LAS or extra 0.5% hydrated lime did not improve the moisture resistance of asphalt mixtures after one freeze-thaw conditioning cycle. A modified TSR parameter was proposed to quantify the coupled effects of aging and moisture conditioning on the tensile strength of an asphalt mixture. The addition of LAS significantly enhanced the moisture resistance of asphalt mixtures after 1,000-hour and 2,000-hour APWS conditioning. However, with respect to TSR results, there was no evidence to discriminate the effectiveness of adding an LAS agent versus additional 0.5% hydrated lime in reducing the moisture susceptibility of FC-5 mixtures. The hydrated lime served as an antioxidant to asphalt binder and retarded the aging of FC-5 mixtures.

- Most of the FC-5 mixtures tested in the study exhibited excellent resistance to rutting and moisture damage. APWS conditioning enhanced the resistance of the FC-5 mixtures to stripping. The HWTT results did not reflect the influence of anti-strip additives on the moisture susceptibility of Junction City granite and Nova Scotia granite FC-5 mixtures.
- In general, the performance test results indicated that the addition of LAS additive, extra 0.5% hydrated lime, or both, produced longer lasting FC-5 mixtures. The cost-benefit analysis demonstrated that the addition of extra 0.5% hydrated lime and LAS agent improved the cost-effectiveness of FC-5 mixtures due to improved performance properties and extended life span. Overall, the combination of 1% hydrated lime and 0.5 % LAS additive significantly improved the cost-effectiveness of Junction City granite and the combination of 1.5% hydrated lime and 0.5% LAS additive maximized the cost-effectiveness of Nova Scotia granite FC-5 mixtures.
- Considering that the two as-provided FC-5 asphalt mixtures used in this study showed good durability and moisture susceptibility, additional FC-5 asphalt mixtures with marginal performance were recommended for further evaluation. Under that circumstance, the additional LAS, extra hydrated lime, or both may further enhance the performance properties and cost-effectiveness of FC-5 mixtures.

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APPENDIX A – FC-5 MIX DESIGNS

STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION

ASPHALT MIX DESIGN

SUBMIT TO THE DIRECTOR, OFFICE OF MATERIALS, CENTRAL ASPHALT LABORATORY, 5007 NE 39TH AVE, GAINESVILLE, FL 32609

Contractor	Anderson Columbia Company, Inc.	Address		DISTRICT 2	
Phone No.		Fax No.		E-mail	ruste.morgan@andersoncolumbia.com
Submitted By	Asphalt Technologies, Inc.	Type Mix	FC-5	Intended Use of Mix	Friction Course

Product Description	Product Code	Producer Name	Product Name	Plant/Pit Number	Terminal
1. S1A Stone	C47	Junction City Mining	S1A Stone	GA553	
2. S1B Stone	C53	Junction City Mining	S1B Stone	GA553	
3. Screenings	F22	Junction City Mining	Screenings	GA553	
4. Hydrated Lime	337-HL	Hydrated Lime	Hydrated Lime		
5.					
6.					
7. PG Binder	916-76PMA		PG 76-22 (PMA)		

PERCENTAGE BY WEIGHT TOTAL AGGREGATE PASSING SIEVES

	Blend	73%	21%	5%	1%			JOB MIX	CONTROL
	Number	1	2	3	4	5	6	FORMULA	POINTS
	3/4" 19.0mm	100	100	100	100			100	100
S	1/2" 12.5mm	98	100	100	100			99	85 - 100
I	3/8" 9.5mm	60	100	100	100			71	55 - 75
Z	No. 4 4.75mm	15	35	100	100			24	15 - 25
E	No. 8 2.36mm	4	5	73	100			9	5 - 10
	No. 16 1.18mm	2	3	47	100			5	
S	No. 30 600μm	1	2	32	100			4	
I	No. 50 300μm	1	2	21	100			3	
E	No. 100 150μm	1	2	13	100			3	
V	No. 200 75μm	1.0	1.0	6.0	100.0			2.5	2 - 4
E S	G_{sb}	2.775	2.764	2.730	2.600			2.769	

The mix properties of the Job Mix Formula have been conditionally verified, pending successful final verification during production at the assigned plant, the mix design is approved subject to F.D.O.T. specifications.

JMF reflects aggregate changes expected during production

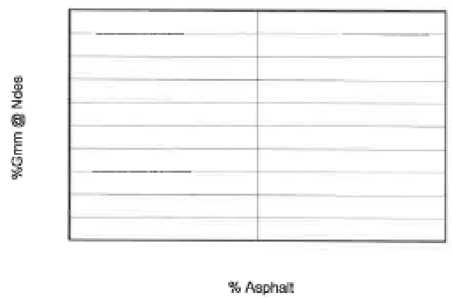
SPM 18-16041A (FC-5)

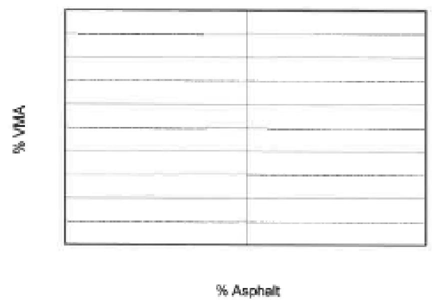
Director, Office of Materials	Timothy J. Ruelke, P.E. Original document retained at the State Materials Office
Effective Date	02 / 13 / 2018
Expiration Date	02 / 13 / 2021

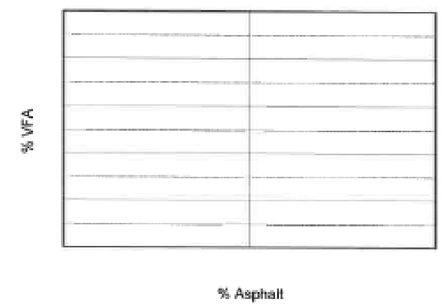
<u>HOT MIX DESIGN DATA SHEET</u>

SPM 18-16041A (FC-5)

P_b	G_{mb} @ N_{des}	G_{mm}	V_a	VMA	VFA	P_{be}	$P_{0.075}$ / P_{be}	%G_{mm} @ N_{ini}	%G_{mm} @ N_{max}







Total Binder Content 6.8 %

Spread Rate @ 1" 112 lbs/yd^2

VMA %

G_{mm} Corr. Factor

FAA %

%G_{mm} @ N_{des}

Ignition Oven Calibration Factor +0.18
(+To Be Added)/(-To Be Subtracted)

(Plant) Mixing Temperature 320 °F 160 °C

(Roadway) Compaction Temperature 320 °F 160 °C

Additives % Cellulose Fiber 0.3 %

STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION
ASPHALT MIX DESIGN
SUBMIT TO THE DIRECTOR, OFFICE OF MATERIALS, CENTRAL ASPHALT LABORATORY, 5007 NE 39TH AVE, GAINESVILLE, FL 32609

Contractor	Duval Asphalt Products	Address	DISTRICT 2
Phone No.	(904) 503-5100	Fax No.	
E-mail	gknagge@ats.consulting		
Submitted By	Gene Knagge	Type Mix	FC-5
Intended Use of Mix	Friction Course		

Product Description	Product Code	Producer Name	Product Name	Plant/Pit Number	Terminal
1. S1A Stone	C44	Martin Marietta Materials	S1A Stone	NS315	
2. S1B Stone	C54	Martin Marietta Materials	S1B Stone	NS315	
3. Screenings	F22	Martin Marietta Materials	Screenings	NS315	
4. Lime	337-HL	Lime	Lime		
5.					
6.					
7. PG Binder	916-76PMA		PG 76-22 (PMA)		

PERCENTAGE BY WEIGHT TOTAL AGGREGATE PASSING SIEVES

	Blend	76%	20%	3%	1%			JOB MIX	CONTROL
	Number	1	2	3	4	5	6	FORMULA	POINTS
	3/4" 19.0mm	100	100	100	100			100	100
S	1/2" 12.5mm	94	100	100	100			95	85 - 100
I	3/8" 9.5mm	69	95	100	100			75	55 - 75
E	No. 4 4.75mm	14	42	97	100			23	15 - 25
V	No. 8 2.36mm	6	13	74	100			10	5 - 10
	No. 16 1.18mm	3	5	46	100			6	
E	No. 30 600μm	2	3	29	100			4	
S	No. 50 300μm	2	2	18	100			3	
	No. 100 150μm	2	2	10	100			3	
	No. 200 75μm	1.0	1.0	6.1	100.0			2.1	2 - 4
	G_{sb}	2.627	2.625	2.580	2.600			2.625	

The mix properties of the Job Mix Formula have been conditionally verified, pending successful final verification during production at the assigned plant, the mix design is approved subject to F.D.O.T. specifications.

SPM 18-16261A (FC-5)

Director, Office of Materials	Timothy J. Ruelke, P.E. Original document retained at the State Materials Office
Effective Date	04 / 20 / 2018
Expiration Date	04 / 20 / 2021

HOT MIX DESIGN DATA SHEET

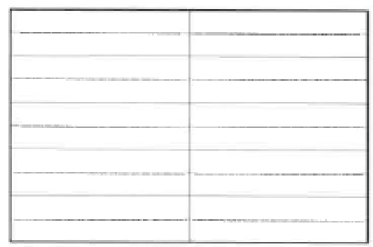
SPM 18-16261A (FC-5)

P_b	G_{mb} @ N_{des}	G_{mm}	V_a	VMA	VFA	P_{be}	$P_{0.075}$ / P_{be}	$\%G_{mm}$ @ N_{ini}	$\%G_{mm}$ @ N_{max}

%Gmm @ Ndes

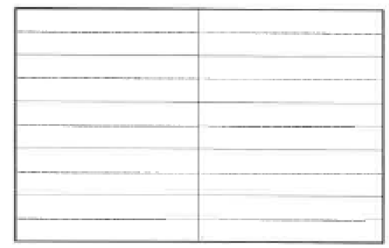
% Asphalt

% VMA



% Asphalt

% VFA



% Asphalt

Total Binder Content 6.6 %

Spread Rate @ 1" 106 lbs/yd^2

VMA %

G_{mm} Corr. Factor

FAA %

$\%G_{mm}$ @ N_{des} #DIV/0!

Ignition Oven Calibration Factor +0.49
(+To Be Added)/(-To Be Subtracted)

(Plant) Mixing Temperature 320 °F 160 °C

(Roadway) Compaction Temperature 320 °F 160 °C

Additives % Mineral Fiber 0.4 %

APPENDIX B – ASPHALT PAVEMENT WEATHERING SYSTEM ACCELERATED AGING

Figure 45. Photo of FC-5 Specimens in APWS Accelerated Aging

100 Series – Core 133	200 Series – Core 229
	
300 Series – Core 333	**400 Series – Core 436**
	

Figure 46. Individual Photos of FC-5 Specimens before Accelerated Aging

APPENDIX C – CANTABRO TEST RESULTS

Aggregate	Anti-strip	Conditioning Time (hr)	Air Voids (%)	Height (mm)	Initial Weight (g)	Final Weight (g)	Mass Loss (%)
Junction City	1% HL	0	19.0	116.8	4,111.5	3,725.1	9.4
Junction City	1% HL	0	19.2	117.3	4,097.7	3,694.6	9.8
Junction City	1% HL	0	18.5	117.1	4,109.0	3,485.0	15.2
Junction City	1% HL	0	17.2	114.5	4,102.1	3,789.2	7.6
Junction City	1% HL + 0.5% LAS1	0	15.1	113.0	4,149.6	3,832.1	7.7
Junction City	1% HL + 0.5% LAS1	0	15.8	115.4	4,173.4	3,719.9	10.9
Junction City	1% HL + 0.5% LAS1	0	17.3	116.8	4,163.4	3,662.7	12.0
Junction City	1.5% HL	0	19.0	116.1	4,041.0	3,377.1	16.4
Junction City	1.5% HL	0	18.7	116.1	4,040.1	3,572.6	11.6
Junction City	1.5% HL+0.5% LAS1	0	18.0	114.2	4,021.2	3,559.7	11.5
Junction City	1.5% HL+0.5% LAS1	0	17.4	115.7	4,058.0	3,454.0	14.9
Junction City	1.5% HL+0.5% LAS1	0	14.7	112.7	4,053.7	3,813.0	5.9
Junction City	1% HL	1,000	18.5	117.9	4,150.0	3,583.3	13.7
Junction City	1% HL	1,000	16.2	114.9	4,140.3	3,665.7	11.5
Junction City	1% HL	1,000	16.8	115.5	4,153.4	3,765.4	9.3
Junction City	1% HL + 0.5% LAS1	1,000	17.1	116.3	4,151.1	3,646.8	12.1
Junction City	1% HL + 0.5% LAS1	1,000	18.5	118.6	4,146.5	3,601.3	13.1
Junction City	1% HL + 0.5% LAS1	1,000	18.1	117.9	4,142.1	3,616.4	12.7
Junction City	1.5% HL	1,000	18.8	118.7	4,144.4	3,427.2	17.3
Junction City	1.5% HL	1,000	18.6	118.3	4,132.4	3,260.1	21.1
Junction City	1.5% HL	1,000	19.0	118.7	4,110.7	3,356.1	18.4
Junction City	1.5% HL+0.5% LAS1	1,000	17.0	115.8	4,128.0	3,669.2	11.1
Junction City	1.5% HL+0.5% LAS1	1,000	17.9	117.2	4,133.1	3,439.9	16.8
Junction City	1.5% HL+0.5% LAS1	1,000	18.7	118.5	4,131.5	3,209.0	22.3
Junction City	1% HL	2,000	18.2	117.3	4,129.8	3,364.0	18.5
Junction City	1% HL	2,000	18.4	117.8	4,134.1	3,347.1	19.0
Junction City	1% HL	2,000	18.5	118.5	4,130.5	3,495.5	15.4
Junction City	1% HL + 0.5% LAS1	2,000	18.7	115.7	4,018.9	3,494.5	13.0
Junction City	1% HL + 0.5% LAS1	2,000	17.7	115.2	4,033.8	3,499.6	13.2
Junction City	1.5% HL	2,000	17.2	116.8	4,163.4	3,476.2	16.5
Junction City	1.5% HL	2,000	18.4	118.8	4,157.4	3,237.2	22.1
Junction City	1.5% HL+0.5% LAS1	2,000	18.8	118.8	4,135.6	3,268.6	21.0
Junction City	1.5% HL+0.5% LAS1	2,000	17.6	117.0	4,142.0	3,508.1	15.3
Junction City	1.5% HL+0.5% LAS1	2,000	17.9	117.6	4,140.8	3,536.0	14.6
Nova Scotia	1% HL	0	21.0	117.8	3,895.4	3,209.4	17.6
Nova Scotia	1% HL	0	17.8	113.6	3,903.9	3,616.9	7.4
Nova Scotia	1% HL	0	18.5	114.5	3,908.8	3,417.8	12.6
Nova Scotia	1% HL + 0.5% LAS2	0	20.7	116.7	3,890.5	3,168.2	18.6
Nova Scotia	1% HL + 0.5% LAS2	0	18.2	114	3,900.7	3,519.7	9.8
Nova Scotia	1% HL + 0.5% LAS2	0	18.4	114.7	3,910.1	3,508.4	10.3
Nova Scotia	1.5% HL	0	20.2	119.3	3,955.5	3,409.8	13.8
Nova Scotia	1.5% HL	0	17.6	115.4	3,962.1	3,587.9	9.4
Nova Scotia	1.5% HL	0	17.9	115.4	3,947.9	3,444.4	12.8
Nova Scotia	1.5% HL+0.5% LAS2	0	21.0	119.6	3,926.9	3,017.0	
Nova Scotia	1.5% HL+0.5% LAS2	0	19.3	117.1	3,935.2	3,346.6	15.0
Nova Scotia	1.5% HL+0.5% LAS2	0	19.2	117.3	3,934.1	3,396.4	13.7
Nova Scotia	1% HL	1,000	18.9	115.5	3,913.9	3,177.0	18.8
Nova Scotia	1% HL	1,000	20.0	117.4	3,920.5	3,226.6	17.7
Nova Scotia	1% HL	1,000	19.4	116.8	3,917.4	3,261.6	16.7

Nova Scotia	1% HL + 0.5% LAS2	1,000	20.4	117.1	3,882.9	3,040.7	21.7
Nova Scotia	1% HL + 0.5% LAS2	1,000	20.8	117.7	3,895.6	3,233.0	17.0
Nova Scotia	1% HL + 0.5% LAS2	1,000	20.4	117.1	3,901.3	3,263.6	16.3
Nova Scotia	1.5% HL	1,000	19.4	117.3	3,957.5	3,306.4	16.5
Nova Scotia	1.5% HL	1,000	19.8	118.3	3,957.9	3,236.2	18.2
Nova Scotia	1.5% HL	1,000	19.1	117.1	3,966.0	3,398.1	14.3
Nova Scotia	1.5% HL+0.5% LAS2	1,000	19.4	117.3	3,941.9	3,114.2	
Nova Scotia	1.5% HL+0.5% LAS2	1,000	20.2	118.7	3,946.0	3,263.6	17.3
Nova Scotia	1.5% HL+0.5% LAS2	1,000	19.0	117.1	3,952.7	3,310.3	16.3
Nova Scotia	1% HL	2,000	17.8	113.3	3,917.1	3,214.2	
Nova Scotia	1% HL	2,000	21.7	118.1	3,888.8	2,366.0	39.2
Nova Scotia	1% HL	2,000	19.4	114.3	3,887.0	3,012.6	22.5
Nova Scotia	1% HL + 0.5% LAS2	2,000	19.7	116.1	3,901.5	2,824.2	27.6
Nova Scotia	1% HL + 0.5% LAS2	2,000	18.1	114.2	3,915.4	3,091.3	21.0
Nova Scotia	1.5% HL	2,000	19.8	118.5	3,996.1	3,047.6	23.7
Nova Scotia	1.5% HL	2,000	18.8	118.0	4,009.3	2,983.1	25.6
Nova Scotia	1.5% HL+0.5% LAS2	2,000	19.1	116.9	3,979.8	3,196.2	19.7
Nova Scotia	1.5% HL+0.5% LAS2	2,000	19.1	117.1	3,970.6	3,174.6	20.0
Nova Scotia	1.5% HL+0.5% LAS2	2,000	19.7	117.4	3,975.6	3,138.2	21.1

APPENDIX D – TENSILE STRENGTH RATIO TEST RESULTS

Aggregate Type	Anti-strip	Conditioning Time (hr)	Air Voids (%)	Gyratory Height (mm)	Trimmed Height (mm)	Peak Load (lb)	ITS (psi)
Junction City	1% HL	0 hr	17.7	112.0	100.77	2,396.4	65.1
Junction City	1% HL	0 hr	17.1	113.7	101.12	2,487.5	67.4
Junction City	1% HL	0 hr	17.6	114.1	100.45	2,194.3	59.8
Junction City	1% HL	1 F/T	15.3	110.2	100.68	2,613.6	71.1
Junction City	1% HL	1 F/T	19.0	116.6	100.89	2,009.6	54.5
Junction City	1% HL	1 F/T	18.7	116.4	101.55	1,972.5	53.2
Junction City	1% HL	1000 hr	19.0	117.9	100.67	2,884.1	78.4
Junction City	1% HL	1000 hr	17.0	115.9	100.04	2,998.3	82.1
Junction City	1% HL	1000 hr	18.0	117.5	99.85	2,768.6	75.9
Junction City	1% HL	2000 hr	15.5	113.7	100.00	3,792.4	103.8
Junction City	1% HL	2000 hr	16.9	116.1	99.87	3,515.1	96.4
Junction City	1% HL	2000 hr	17.9	117.1	100.05	3,295.4	90.2
Junction City	1% HL + 0.5% LAS1	0 hr	19.0	115.6	100.01	1,943.4	53.2
Junction City	1% HL + 0.5% LAS1	0 hr	17.1	113.1	100.15	2,610.6	71.4
Junction City	1% HL + 0.5% LAS1	0 hr	18.0	114.6	99.90	2,251.9	61.7
Junction City	1% HL + 0.5% LAS1	1 F/T	17.8	114.7	99.89	2,036.4	55.8
Junction City	1% HL + 0.5% LAS1	1 F/T	17.7	114.3	99.71	2,053.1	56.4
Junction City	1% HL + 0.5% LAS1	1000 hr	17.1	116.0	100.10	3,249.3	88.9
Junction City	1% HL + 0.5% LAS1	1000 hr	18.8	119.0	100.35	2,786.3	76.0
Junction City	1% HL + 0.5% LAS1	1000 hr	17.5	116.8	100.46	3,311.6	90.3
Junction City	1% HL + 0.5% LAS1	2000 hr	14.3	112.2	99.56	3,795.5	104.4
Junction City	1% HL + 0.5% LAS1	2000 hr	15.0	113.0	99.00	3,428.1	94.8
Junction City	1% HL + 0.5% LAS1	0 hr	17.9	115.2	100.40	2,229.1	60.8
Junction City	1.5% HL	0 hr	18.9	116.0	101.80	2,358.5	63.4
Junction City	1.5% HL	0 hr	19.7	117.4	100.97	2,198.4	59.6
Junction City	1.5% HL	1 F/T	17.3	114.4	101.03	1,999.1	54.2
Junction City	1.5% HL	1 F/T	19.4	117.2	101.57	1,907.0	51.4
Junction City	1.5% HL	1 F/T	18.0	114.9	102.20	2,297.4	61.6
Junction City	1.5% HL	1000 hr	18.2	117.7	100.06	2,942.1	80.5
Junction City	1.5% HL	1000 hr	17.8	117.3	101.08	3,140.3	85.1
Junction City	1.5% HL	1000 hr	19.9	119.4	100.27	2,548.1	69.6
Junction City	1.5% HL	2000 hr	17.7	117.7	100.02	2,763.6	75.7
Junction City	1.5% HL	2000 hr	18.0	118.3	100.85	3,028.9	82.2
Junction City	1.5% HL	2000 hr	18.5	118.7	100.71	2,940.1	79.9
Junction City	1.5% HL	0 hr	18.7	115.8	100.10	2,042.8	55.9
Junction City	1.5% HL+0.5% LAS1	0 hr	18.7	115.8	99.70	2,228.8	61.2
Junction City	1.5% HL+0.5% LAS1	0 hr	15.3	115.1	100.85	2,708.9	73.5
Junction City	1.5% HL+0.5% LAS1	1 F/T	18.2	116.1	100.89	1,870.4	50.8
Junction City	1.5% HL+0.5% LAS1	1 F/T	17.7	111.5	100.97	1,862.7	50.5
Junction City	1.5% HL+0.5% LAS1	1 F/T	15.5	111.8	100.86	2,343.0	63.6
Junction City	1.5% HL+0.5% LAS1	1000 hr	18.1	116.5	100.46	3,030.6	82.6
Junction City	1.5% HL+0.5% LAS1	1000 hr	18.5	117.9	100.21	2,948.4	80.6
Junction City	1.5% HL+0.5% LAS1	1000 hr	18.7	117.3	100.21	3,045.9	83.2
Junction City	1.5% HL+0.5% LAS1	2000 hr	18.5	117.2	99.97	2,884.3	79.0
Junction City	1.5% HL+0.5% LAS1	2000 hr	16.8	115.5	101.01	3,259.0	88.3
Nova Scotia	1% HL	1 F/T	19.5	115.1	98.95	2,125.6	58.8
Nova Scotia	1% HL	1 F/T	18.4	114.4	99.33	2,326.8	64.1
Nova Scotia	1% HL	1 F/T	19.2	115.2	99.78	2,194.4	60.2

Nova Scotia	1% HL	0	18.8	114.6	100.28	2,333.5	63.7
Nova Scotia	1% HL	0	17.5	113.2	99.63	2,648.2	72.8
Nova Scotia	1% HL	0	19.8	116.8	98.92	2,093.0	57.9
Nova Scotia	1% HL	1,000	19.3	116.8	100.52	2,866.5	78.1
Nova Scotia	1% HL	1,000	21.5	119.9	100.66	2,258.1	61.4
Nova Scotia	1% HL	1,000	17.1	112.8	100.52	2,951.0	80.4
Nova Scotia	1% HL	2,000	20.5	117.1	101.39	2,772.1	74.9
Nova Scotia	1% HL	2,000	18.6	115.1	102.71	3,121.5	83.2
Nova Scotia	1% HL	2,000	22.2	118.2	101.55	2,456.5	66.2
Nova Scotia	1% HL	1 F/T	18.2	113.7	101.32	2,193.9	59.3
Nova Scotia	1% HL + 0.5% LAS2	1 F/T	21.2	118.8	100.98	1,792.0	48.6
Nova Scotia	1% HL + 0.5% LAS2	1 F/T	18.8	111.1	99.80	2,388.6	65.5
Nova Scotia	1% HL + 0.5% LAS2	0	19.8	115.6	99.85	2,196.4	60.2
Nova Scotia	1% HL + 0.5% LAS2	0	18.9	115.9	99.78	2,306.5	63.3
Nova Scotia	1% HL + 0.5% LAS2	0	19.8	115.9	100.16	2,356.0	64.4
Nova Scotia	1% HL + 0.5% LAS2	1,000	21.2	117.7	101.18	2,481.8	67.2
Nova Scotia	1% HL + 0.5% LAS2	1,000	20.1	116.1	100.83	2,674.4	72.6
Nova Scotia	1% HL + 0.5% LAS2	1,000	20.8	116.9	99.97	2,491.6	68.2
Nova Scotia	1% HL + 0.5% LAS2	2,000	19.3	115.7	101.19	2,906.8	78.7
Nova Scotia	1% HL + 0.5% LAS2	2,000	20.4	116.9	101.65	2,695.5	72.6
Nova Scotia	1.5% HL	0	17.9	115.2	99.95	2,495.4	68.4
Nova Scotia	1.5% HL	0	19.5	118.5	100.05	2,256.6	61.8
Nova Scotia	1.5% HL	0	18.5	116.4	100.32	2,559.7	69.9
Nova Scotia	1.5% HL	1 F/T	19.0	117.7	101.03	2,052.5	55.6
Nova Scotia	1.5% HL	1 F/T	19.7	118.6	102.00	2,127.7	57.1
Nova Scotia	1.5% HL	1 F/T	17.7	115.9	101.11	2,496.2	67.6
Nova Scotia	1.5% HL	1,000	20.9	120.1	100.25	2,518.6	68.8
Nova Scotia	1.5% HL	1,000	18.9	116.2	100.85	2,962.9	80.4
Nova Scotia	1.5% HL	2,000	20.5	120.3	101.25	2,967.6	80.3
Nova Scotia	1.5% HL	2,000	18.6	117.3	100.89	2,787.3	75.6
Nova Scotia	1.5% HL+0.5% LAS2	1 F/T	20.4	117.8	101.63	2,015.2	54.3
Nova Scotia	1.5% HL+0.5% LAS2	1 F/T	18.1	115.4	99.89	2,468.2	67.7
Nova Scotia	1.5% HL+0.5% LAS2	1 F/T	18.0	115.0	101.32	2,452.9	66.3
Nova Scotia	1.5% HL+0.5% LAS2	0	18.7	115.8	100.17	2,273.7	62.2
Nova Scotia	1.5% HL+0.5% LAS2	0	16.7	112.6	101.80	2,786.4	74.9
Nova Scotia	1.5% HL+0.5% LAS2	0	20.2	118.0	99.81	2,046.8	56.2
Nova Scotia	1.5% HL+0.5% LAS2	1,000	17.9	114.6	101.70	2,873.6	77.4
Nova Scotia	1.5% HL+0.5% LAS2	1,000	19.4	117.8	99.52	2,663.8	73.3
Nova Scotia	1.5% HL+0.5% LAS2	2,000	17.4	113.9	101.17	2,980.8	80.7
Nova Scotia	1.5% HL+0.5% LAS2	2,000	19.3	116.9	101.19	2,888.8	78.2

APPENDIX E – HAMBURG WHEEL TRACKING TEST RESULTS

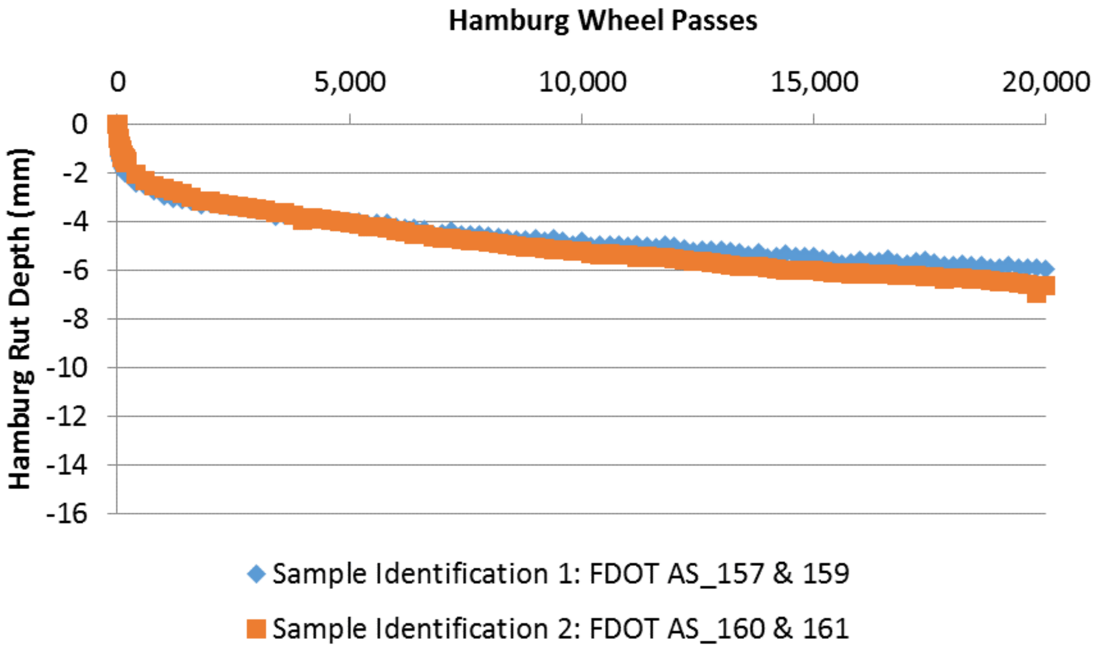


Figure 47. Unconditioned Junction City FC-5 Mixture Containing 1% Hydrated Lime

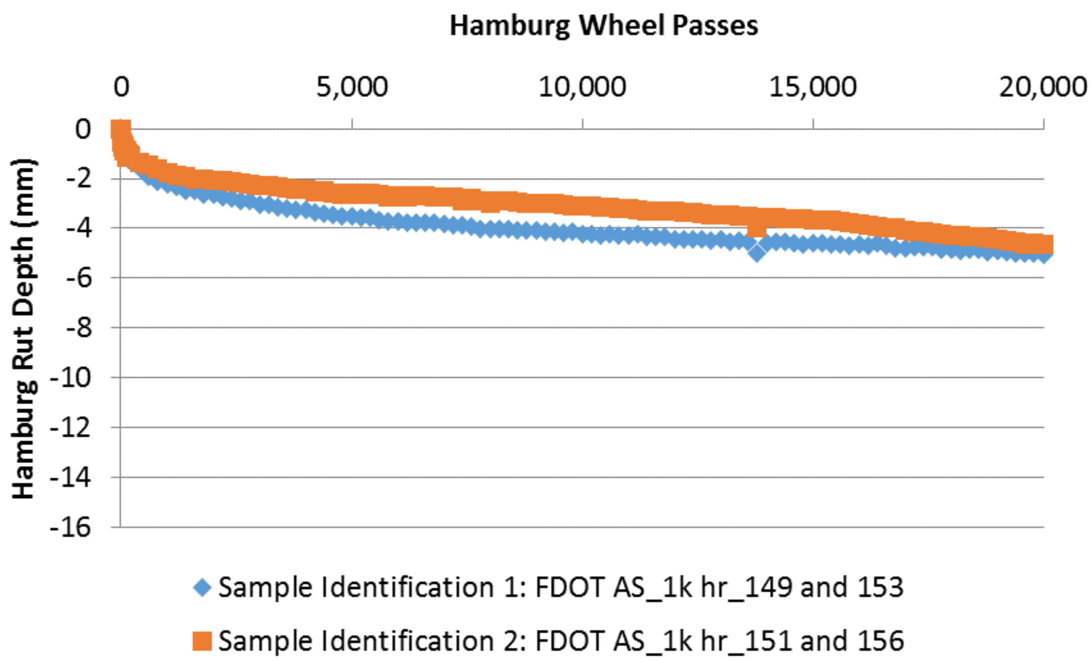


Figure 48. APWS 1000-hour Conditioned Junction City FC-5 Mixture Containing 1% Hydrated Lime

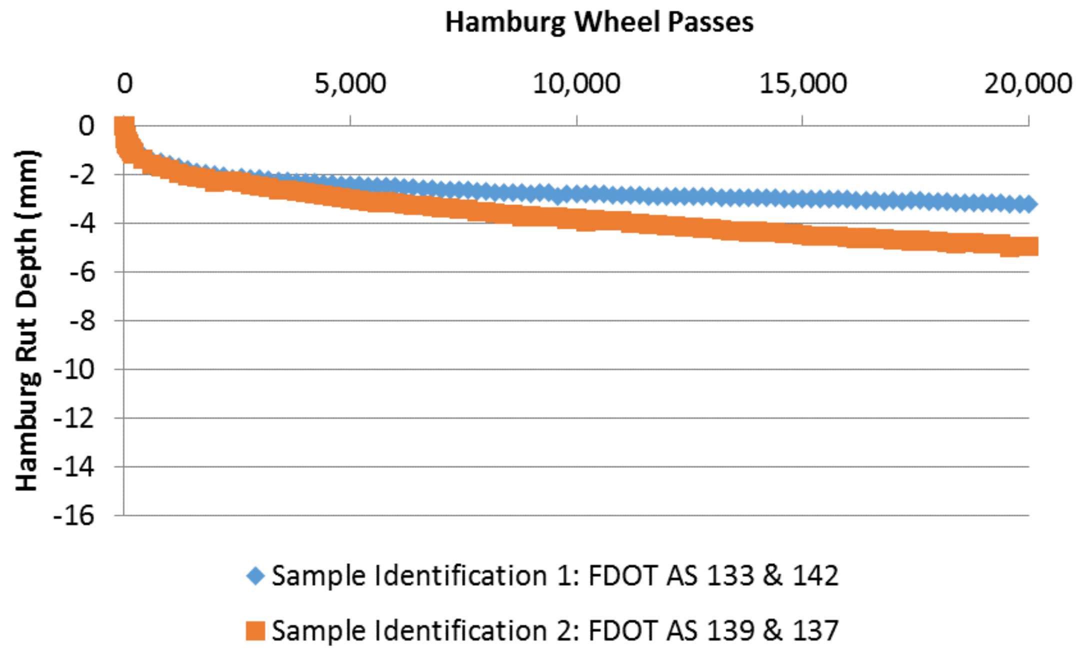


Figure 49. APWS 2000-hour Conditioned Junction City FC-5 Mixture Containing 1% Hydrated Lime

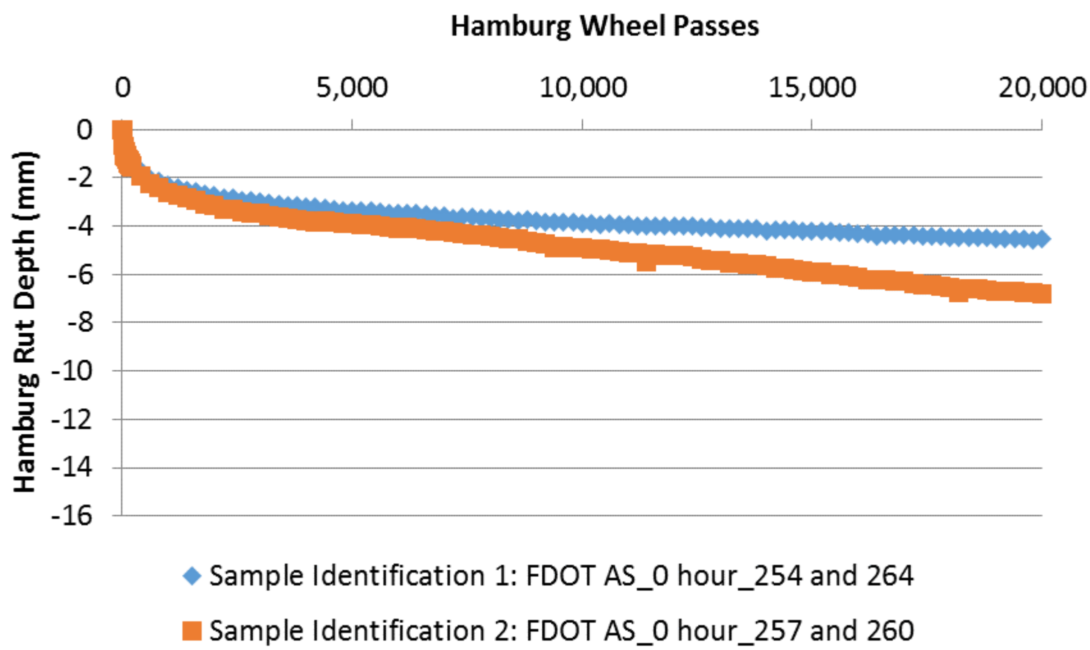


Figure 50. Unconditioned Junction City FC-5 Mixture Containing 1% Hydrated Lime and LAS1

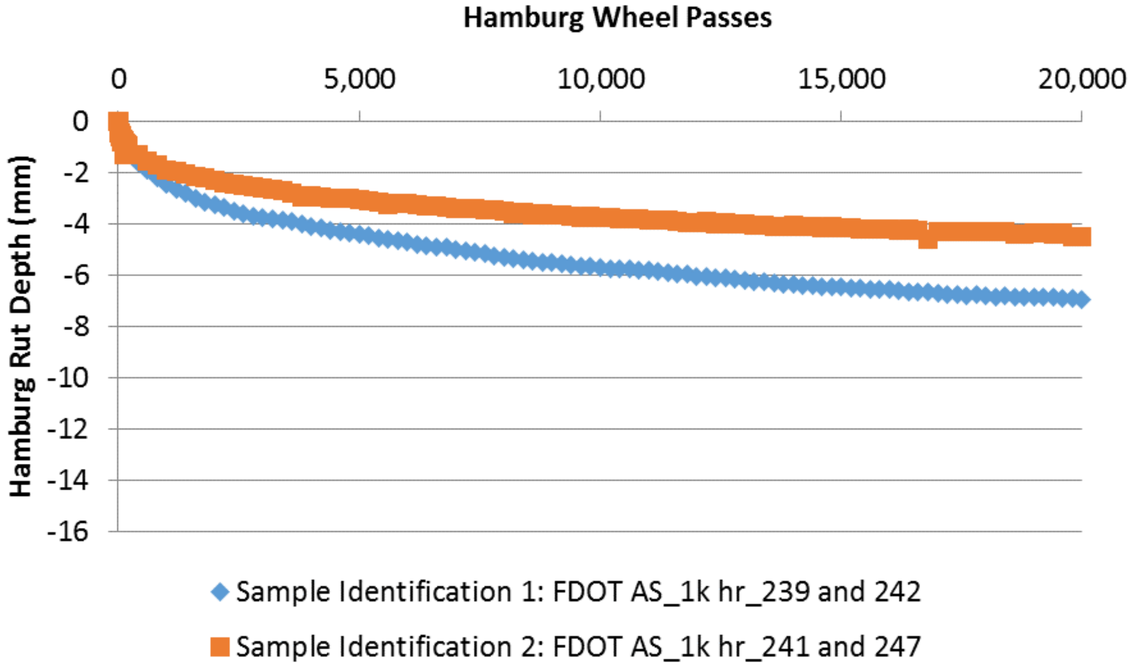


Figure 51. APWS 1000-hour Conditioned Junction City FC-5 Mixture Containing 1% Hydrated Lime and LAS1

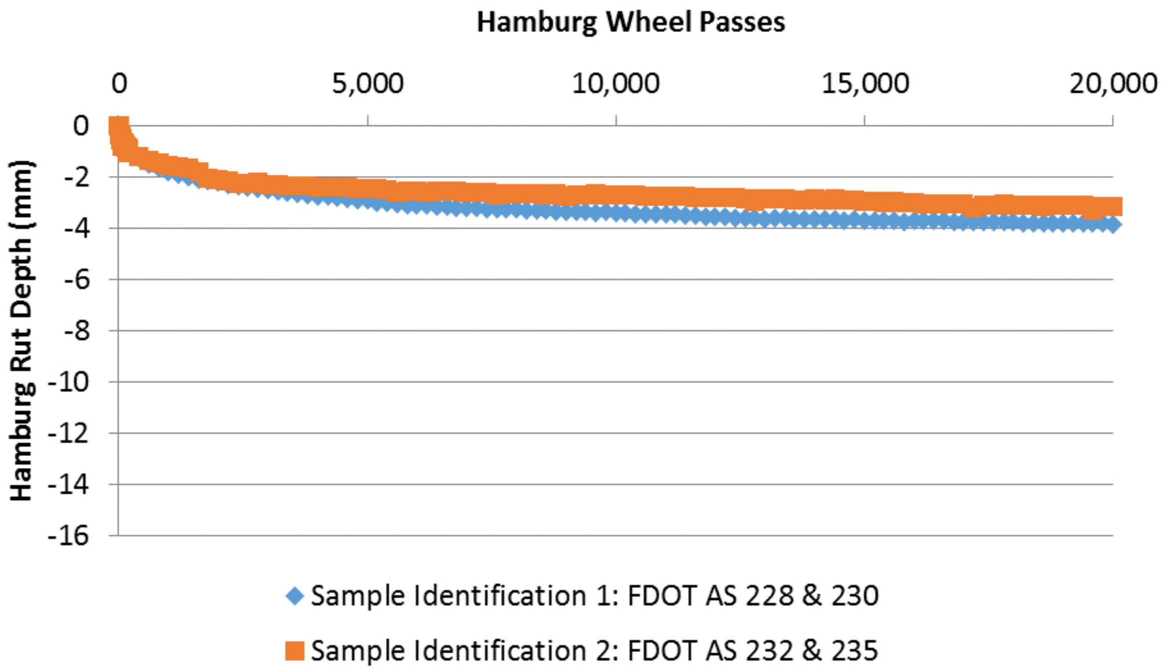


Figure 52. APWS 2000-hour Conditioned Junction City FC-5 Mixture Containing 1% Hydrated Lime and LAS1

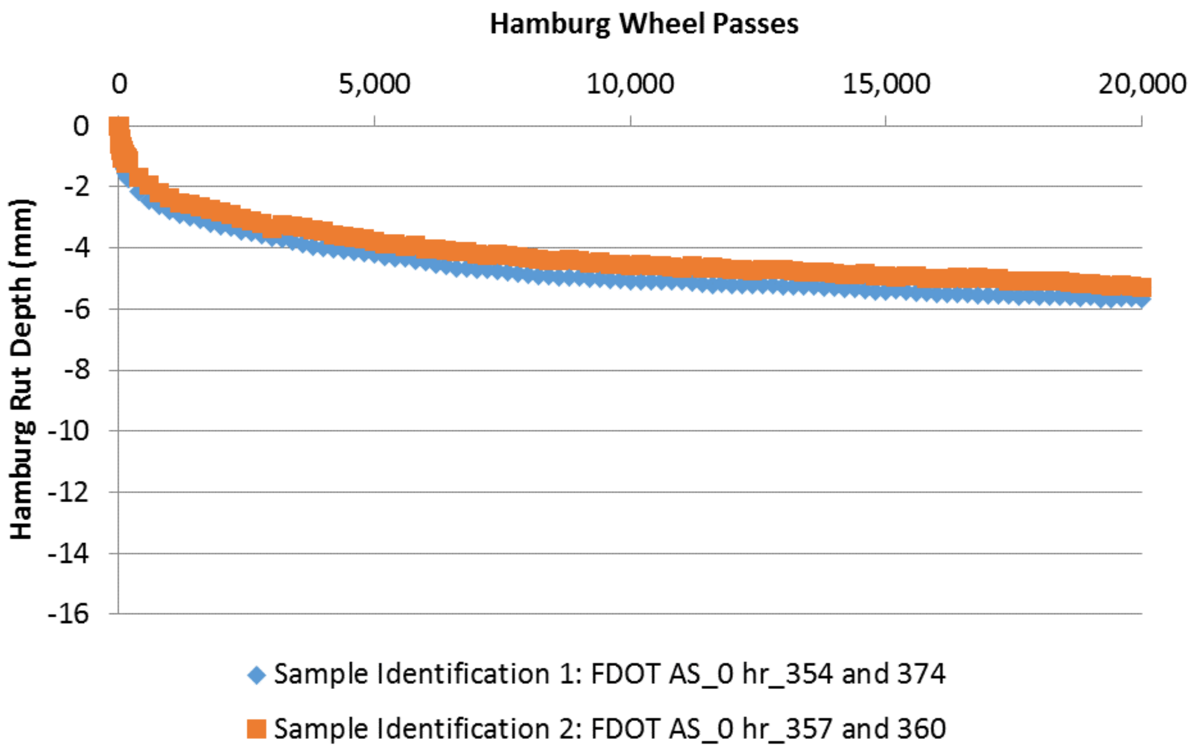


Figure 53. Unconditioned Junction City FC-5 Mixture Containing 1.5% Hydrated Lime

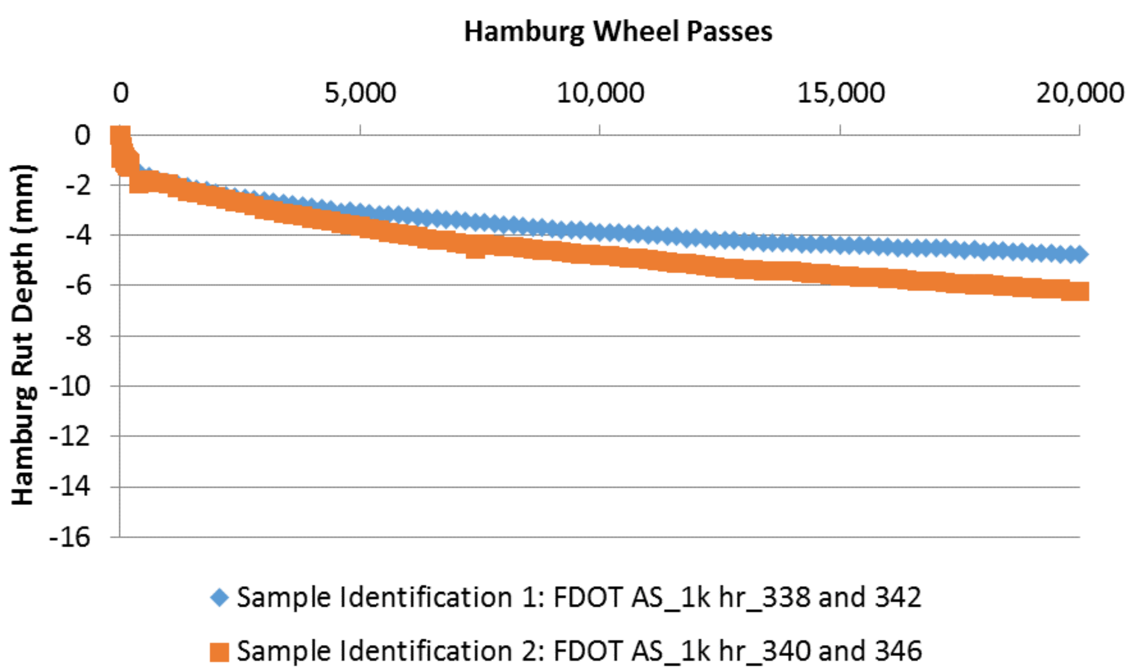


Figure 54. APWS 1000-hour Conditioned Junction City FC-5 Mixture Containing 1.5% Hydrated Lime

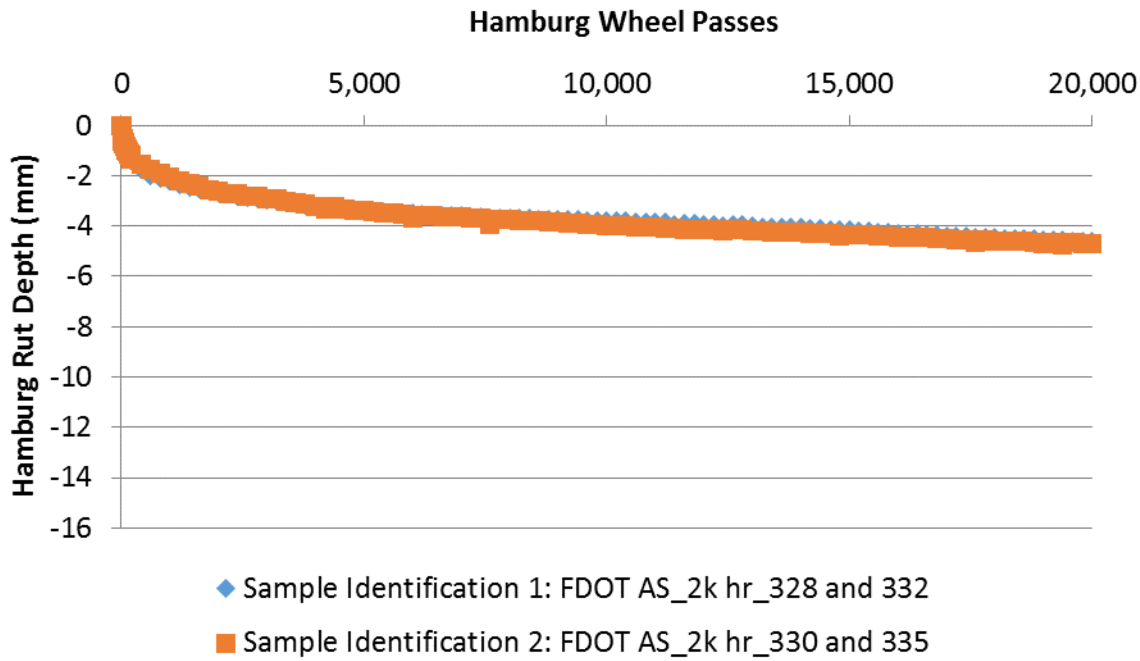


Figure 55. APWS 2000-hour Conditioned Junction City FC-5 Mixture Containing 1.5% Hydrated Lime

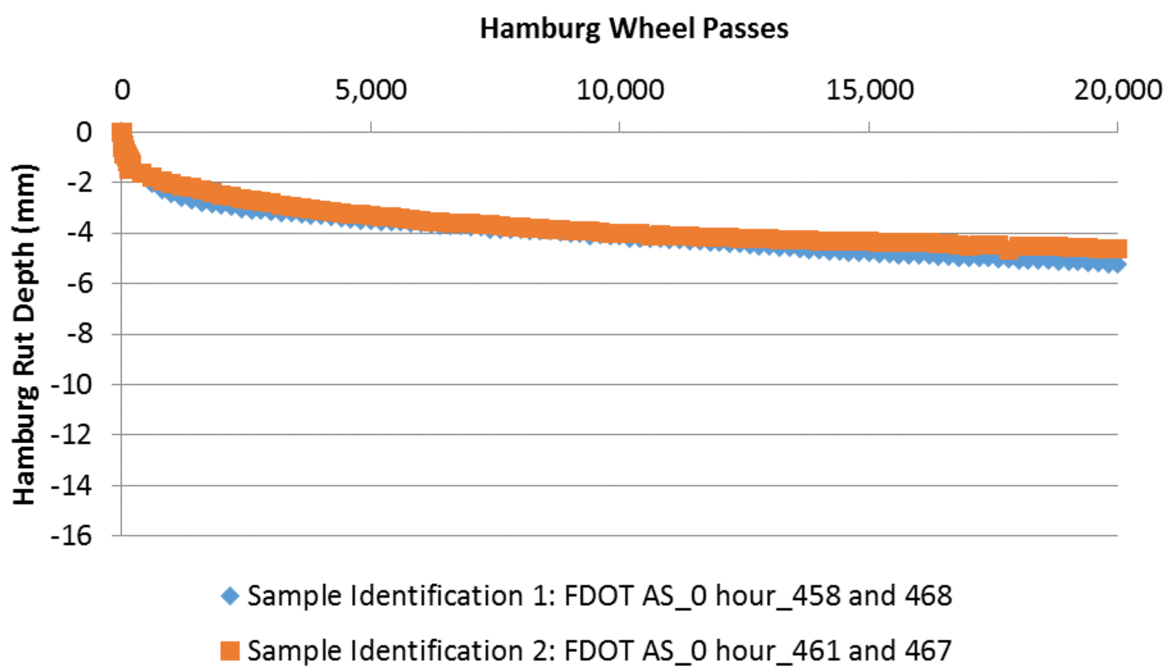


Figure 56. Unconditioned Junction City FC-5 Mixture Containing 1.5% Hydrated Lime and LAS1

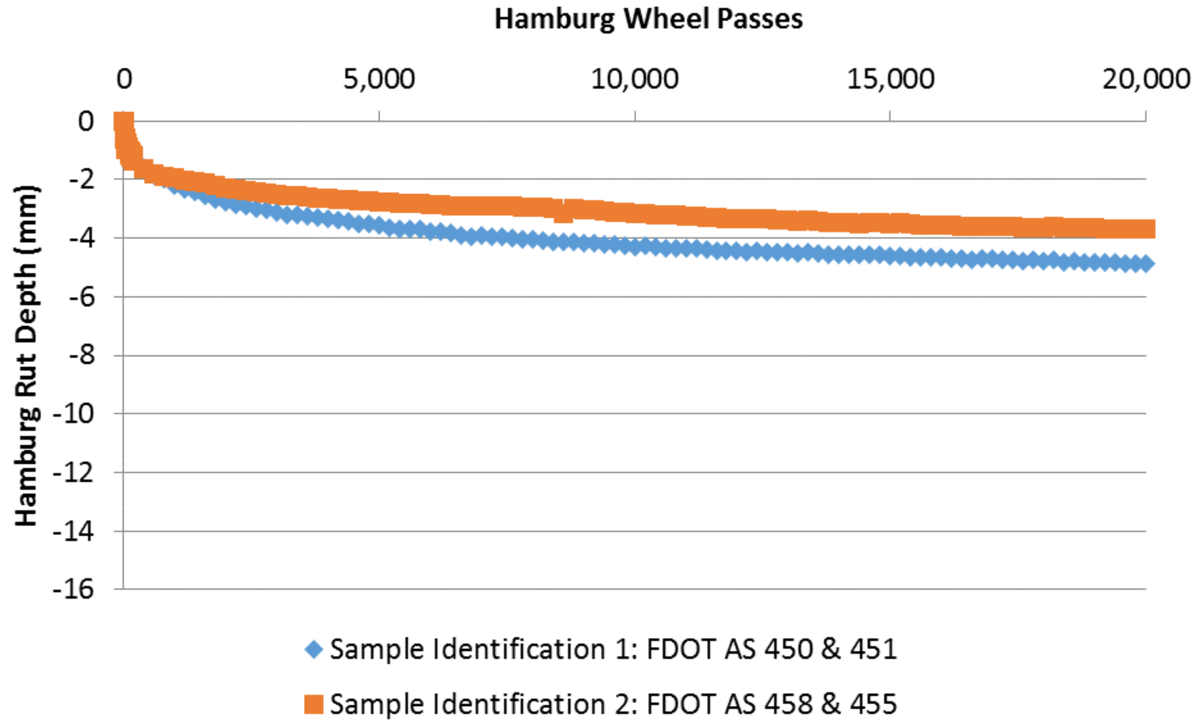


Figure 57. APWS 1000-hour Conditioned Junction City FC-5 Mixture Containing 1.5% Hydrated Lime and LAS1

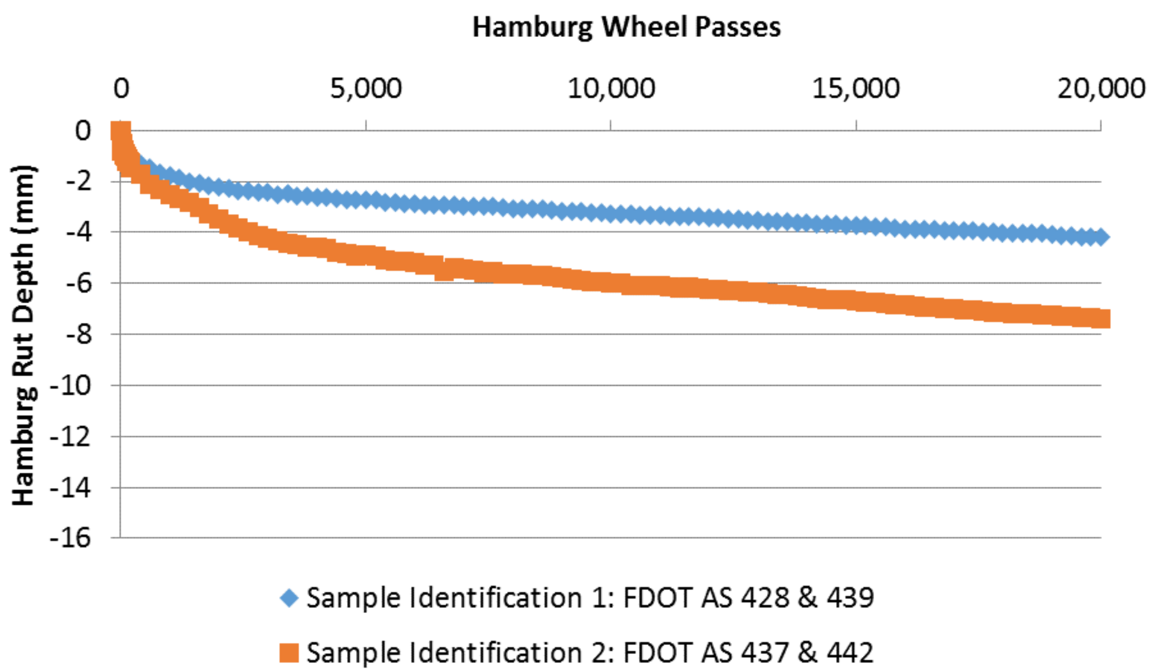


Figure 58. APWS 2000-hour Conditioned Junction City FC-5 Mixture Containing 1.5% Hydrated Lime and LAS1

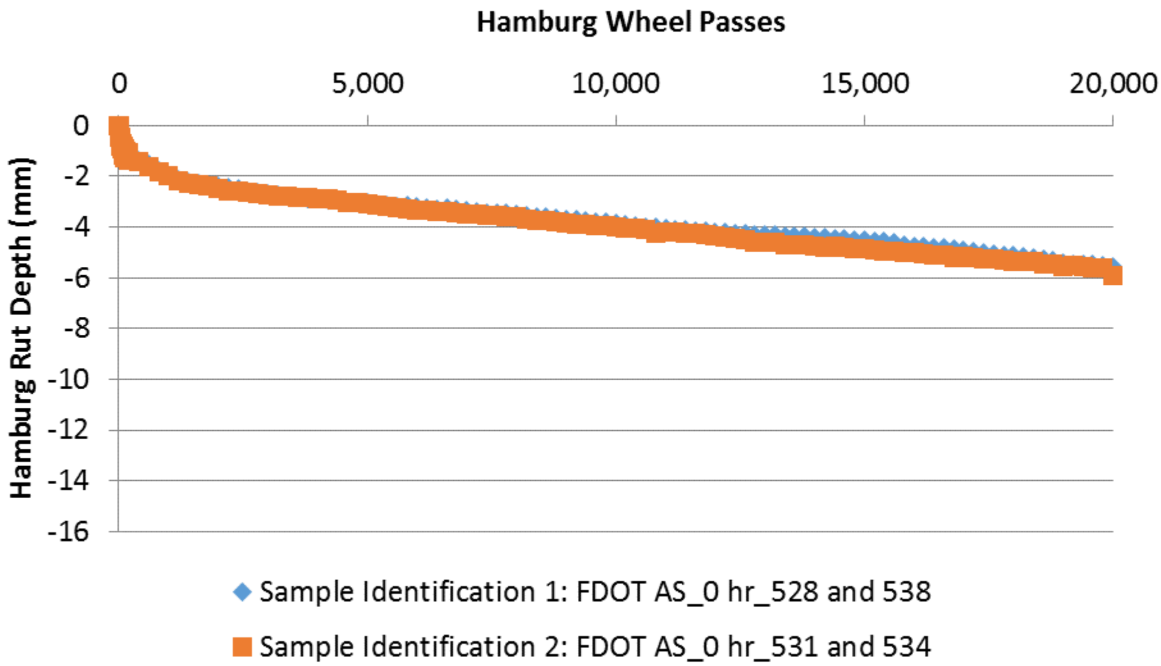


Figure 59. Unconditioned Nova Scotia FC-5 Mixture Containing 1% Hydrated Lime

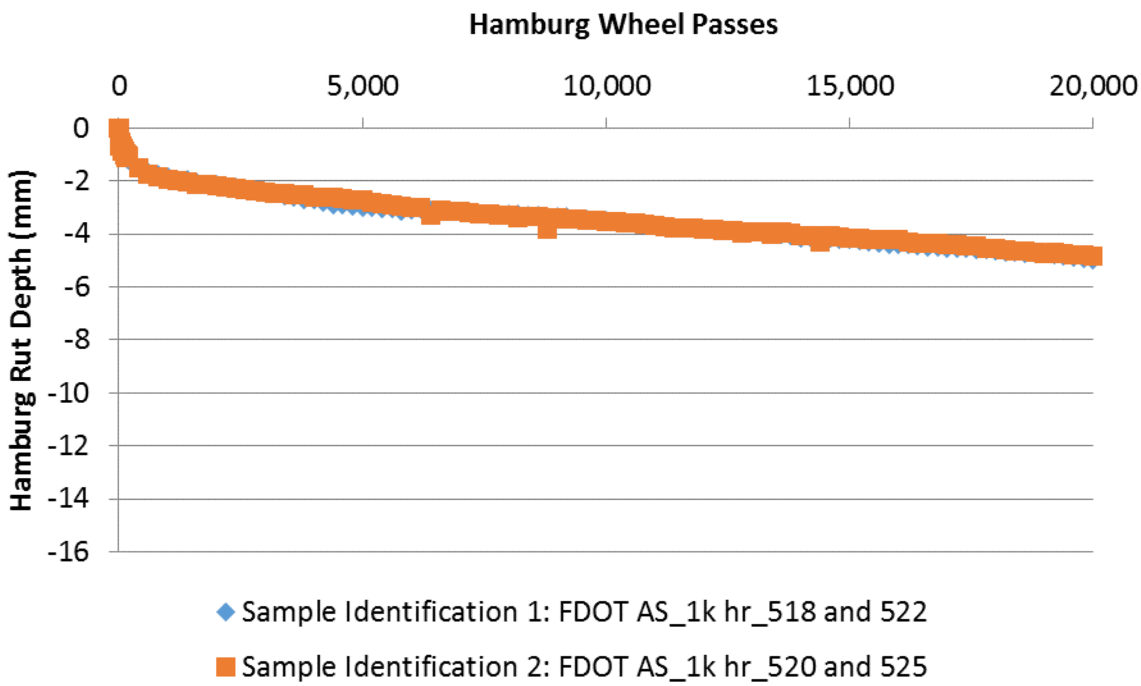


Figure 60. APWS 1000-hour Conditioned Nova Scotia FC-5 Mixture Containing 1% Hydrated Lime

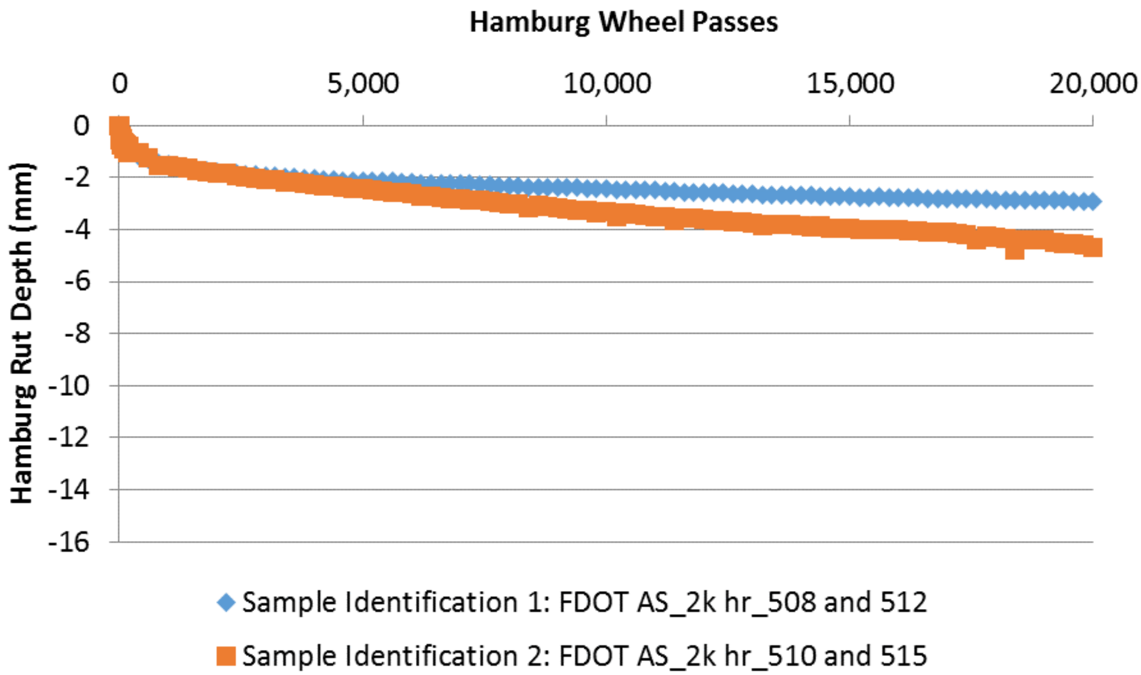


Figure 61. APWS 2000-hour Conditioned Nova Scotia FC-5 Mixture Containing 1% Hydrated Lime

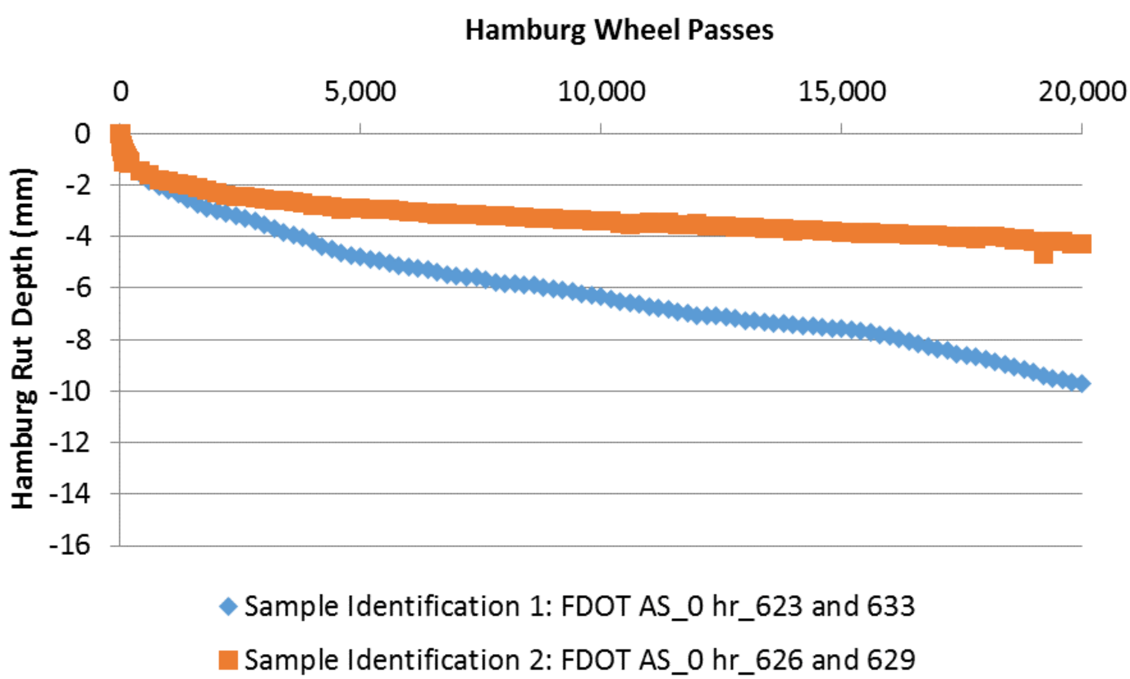


Figure 62. Unconditioned Nova Scotia FC-5 Mixture Containing 1% Hydrated Lime and LAS2

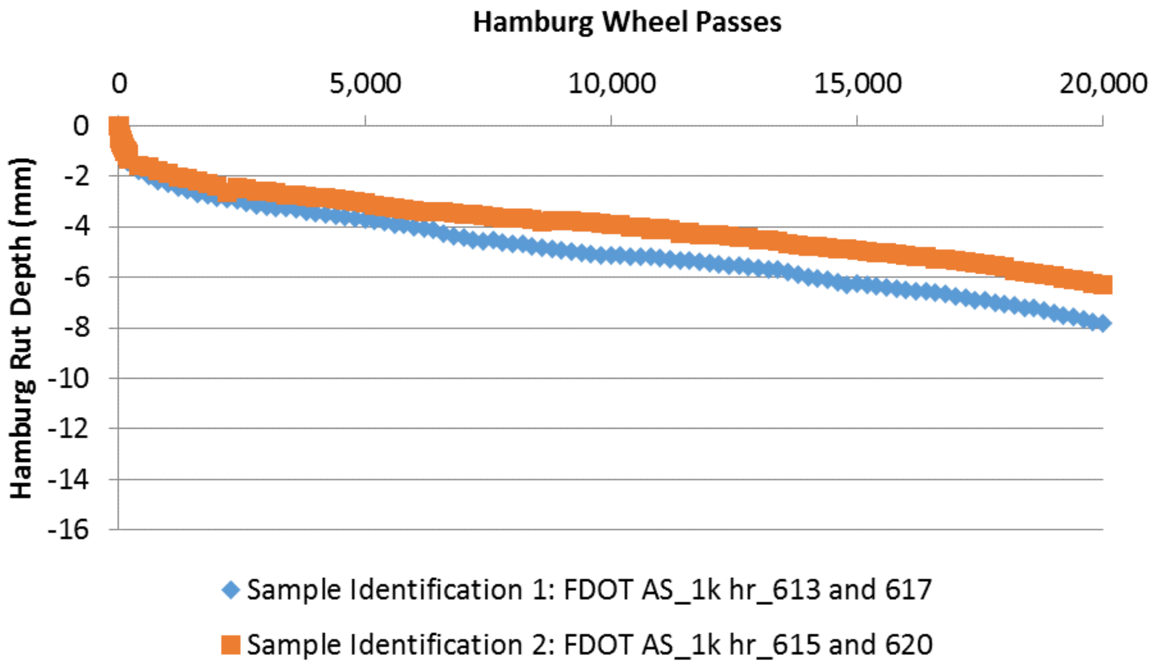


Figure 63. APWS 1000-hour Conditioned Nova Scotia FC-5 Mixture Containing 1% Hydrated Lime and LAS2

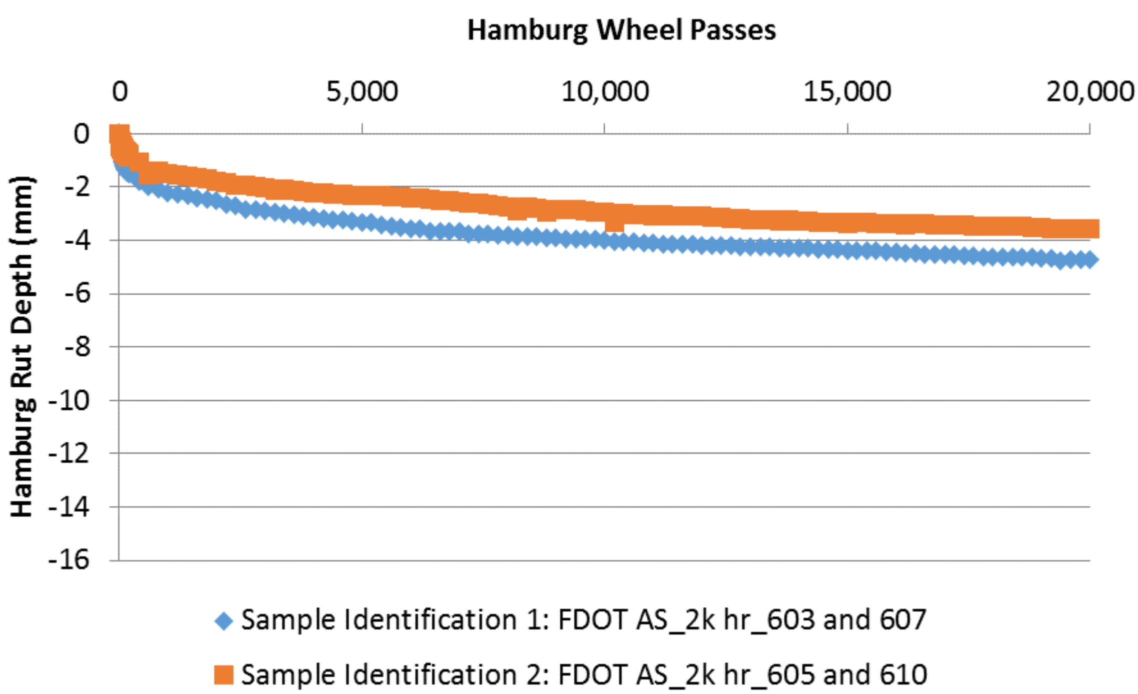


Figure 64. APWS 2000-hour Conditioned Nova Scotia FC-5 Mixture Containing 1% Hydrated Lime and LAS2

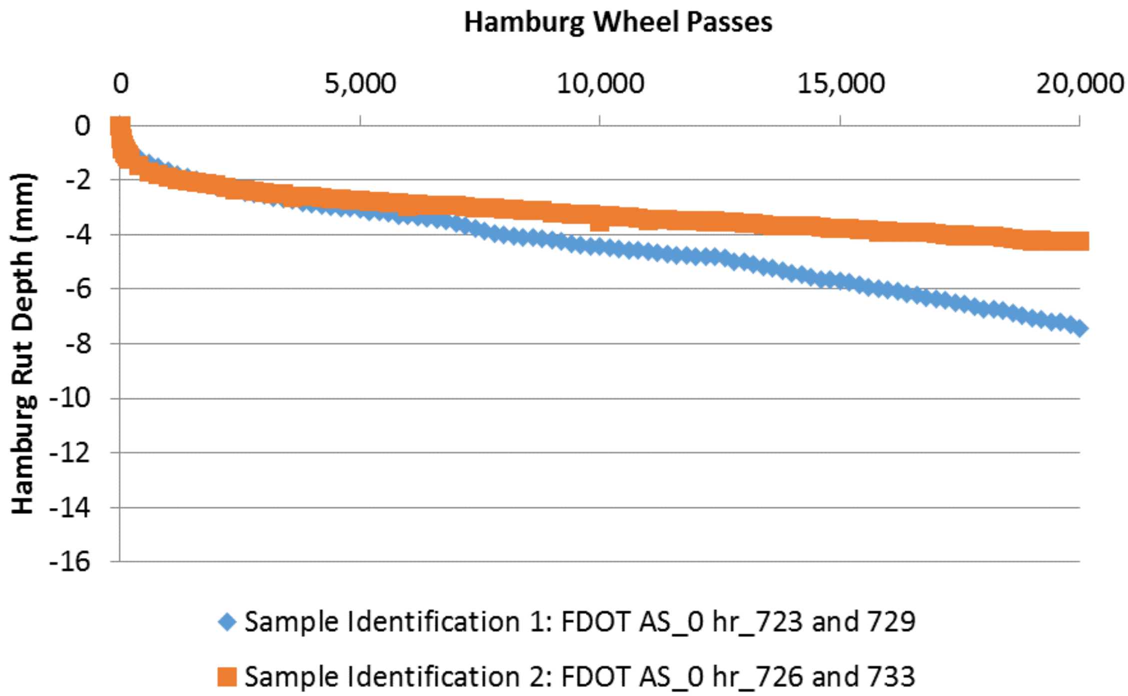


Figure 65. Unconditioned Nova Scotia FC-5 Mixture Containing 1.5% Hydrated Lime

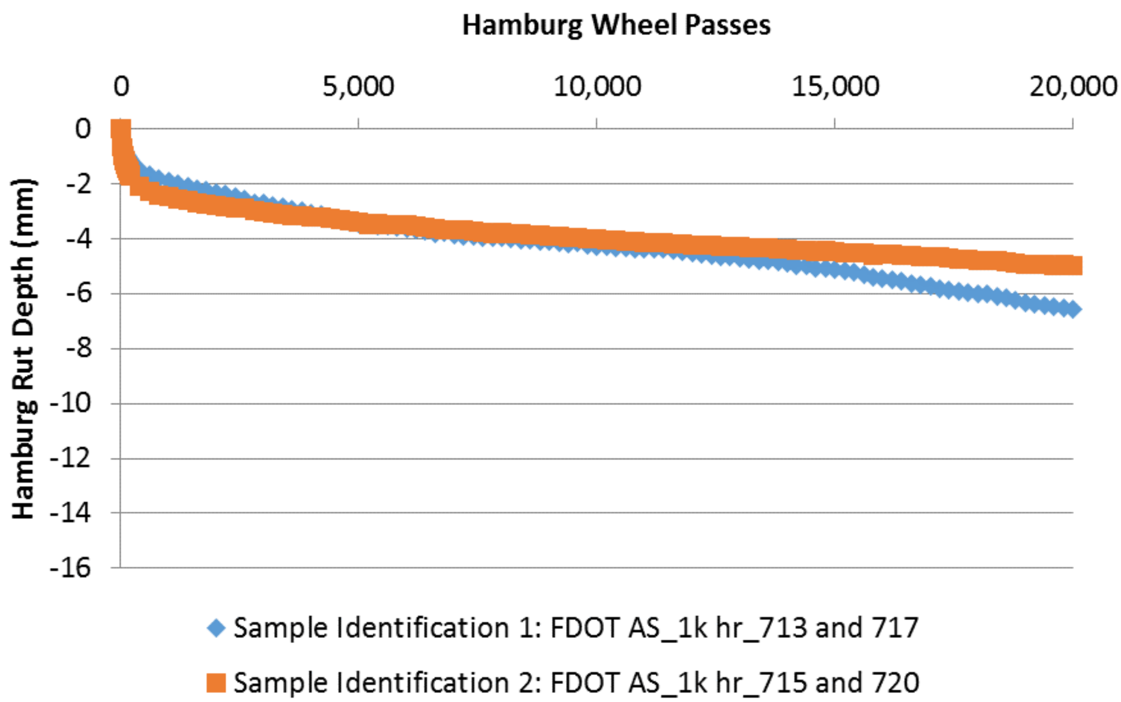


Figure 66. APWS 1000-hour Conditioned Nova Scotia FC-5 Mixture Containing 1.5% Hydrated Lime

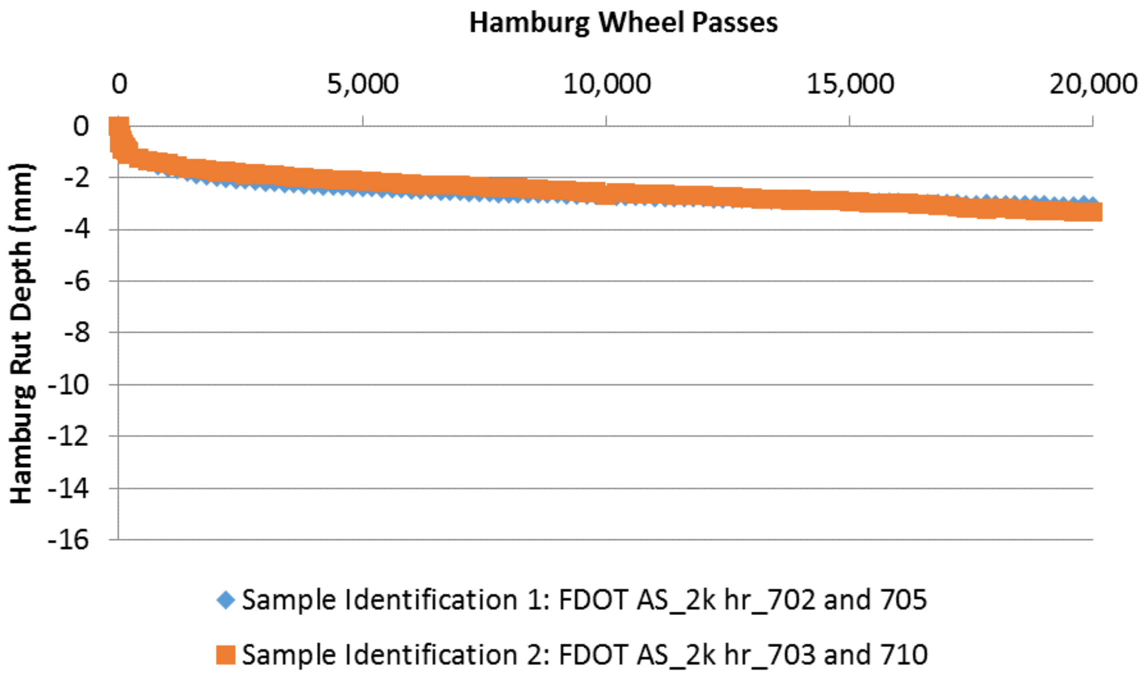


Figure 67. APWS 2000-hour Conditioned Nova Scotia FC-5 Mixture Containing 1.5% Hydrated Lime

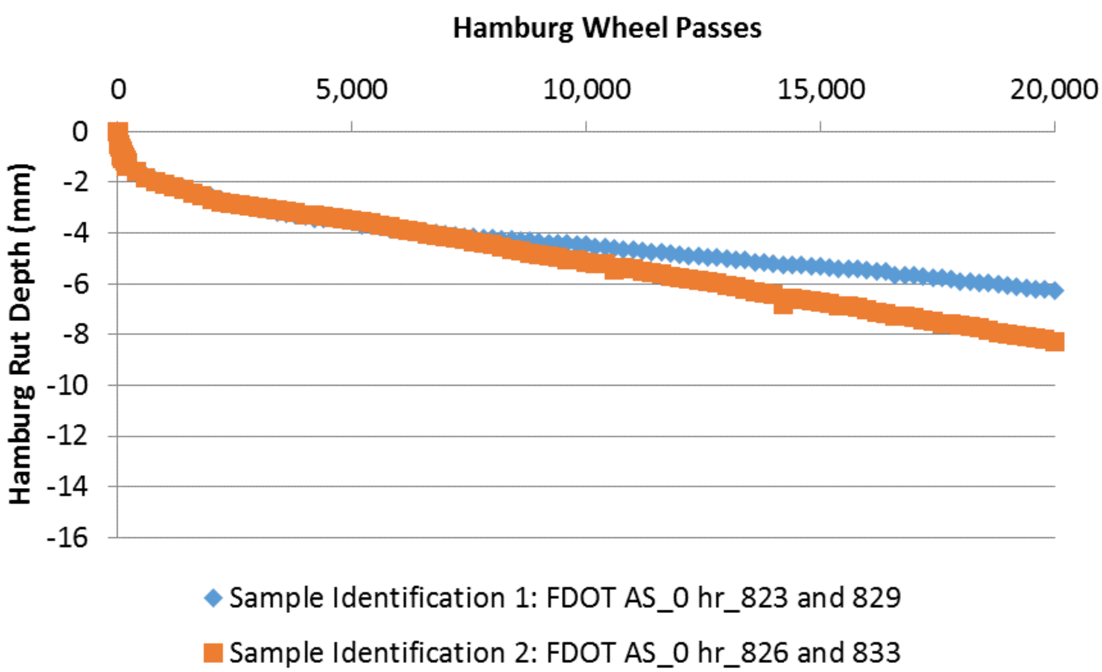


Figure 68. Unconditioned Nova Scotia FC-5 Mixture Containing 1.5% Hydrated Lime and LAS2

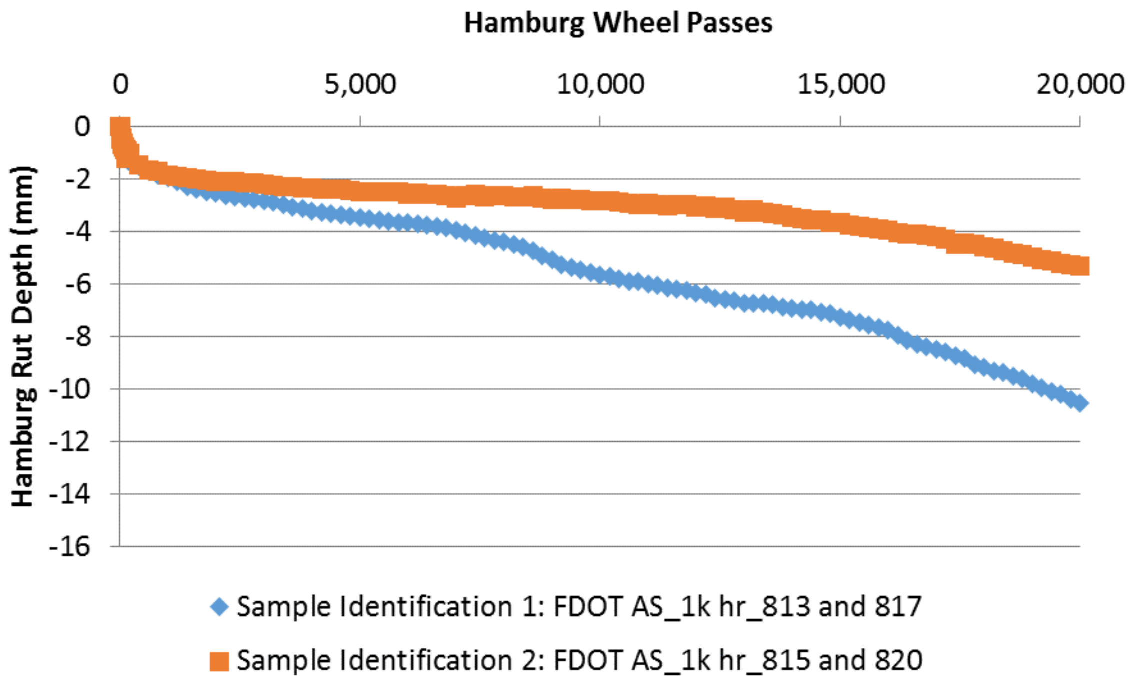


Figure 69. APWS 1000-hour Conditioned Nova Scotia FC-5 Mixture Containing 1.5% Hydrated Lime and LAS2

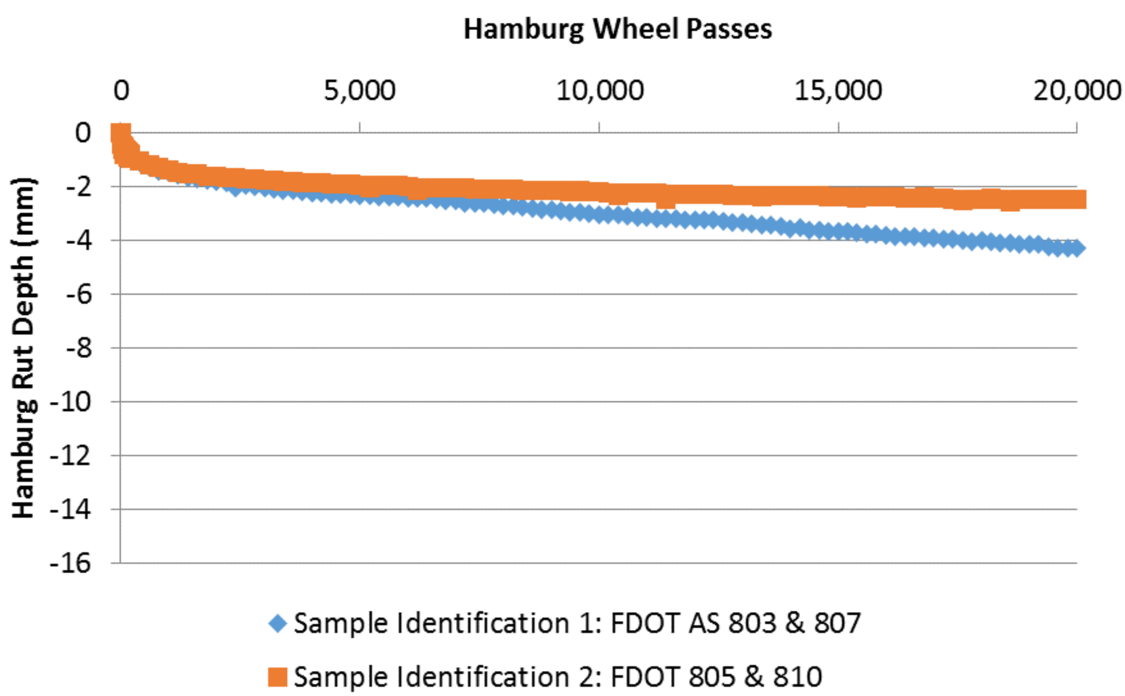


Figure 70. APWS 2000-hour Conditioned Nova Scotia FC-5 Mixture Containing 1.5% Hydrated Lime and LAS2