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FINAL REPORT

EFFECTIVENESS OF ANTI-STRIP AGENTS IN ASPHALT MIXTURES

Georgia Department of Transportation

OFFICE OF RESEARCH
15 Kennedy Drive
Forest Park, GA 30297

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Effectiveness of Anti-Strip Agents in Asphalt Mixtures

Final Report

By
Donald Watson, Lead Research Engineer
Jason R. Moore, Lab Manager
Adam J. Taylor, Assistant Research Engineer
National Center for Asphalt Technology
277 Technology Parkway
Auburn University, AL 36830
Phone: 334-844-6228
E-Mail: watsode@auburn.edu

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16. Abstract:

Since the late 1970s there has been much research performed to better understand the stripping phenomenon in asphalt mixtures. As a result, there have been changes in both materials and technology over the past 30 years to improve the resistance to moisture damage and the ability to test for performance under adverse moisture conditions. Due to the changes in materials and technologies related to the development and improvement of anti-strip agents, this research study was conducted to evaluate the effectiveness of current anti-strip agents used in hot mix asphalt pavements.

One purpose of the project was to evaluate anti-strip agents with a variety of aggregates and mix types. Three granite sources were used: one does not have a history of stripping, one is a known stripping aggregate, and the other has used both hydrated lime and liquid additive in the past in order to meet tensile strength requirements. All granite sources were used in both 12.5 mm surface mixture and 25 mm base mixture. In addition, a limestone source was used in the 25 mm analysis to determine whether liquid antistrips may result in better performance than hydrated lime. Limestone was used only in the 25 mm mix because it is not typically used in surface mixtures due to a tendency to polish under traffic.

Secondly, the past field performance of Georgia's mixes designed with hydrated lime to the performance of Georgia's mixes designed using liquid anti-strip agents was evaluated. Therefore, part of the study involved identifying projects with similar age, aggregate source, and mix type in order to make comparisons of performance for mixtures with liquid anti-strip with similar projects that used hydrated lime.

Thirdly, a field test section was constructed where three different anti-strip agents were used in a conventional Superpave surface mixture. This was done on a typical mill and inlay project.

A final objective involved conducting a series of laboratory performance test comparisons using different aging periods to make long-term comparisons of the effectiveness of hydrated lime to liquid and Warm mix anti-strip additives.

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EXECUTIVE SUMMARY

Due to stripping problems in hot mix asphalt, the Georgia Department of Transportation (GDOT) began in 1969 requiring a liquid anti-strip agent to be mixed with the asphalt cement for hot mix asphalt. Stripping is defined as the loss of bond, or adhesion, between asphalt cement and aggregate particles in the presence of moisture. In 1980, GDOT experienced several pavement failures due to an extremely hot summer. This led to a pavement investigation that included 81 interstate projects. Only 32 percent of those projects did not have visual stripping, and 24 percent of the projects had moderate to severe stripping. As a result, in 1982 GDOT implemented the use of hydrated lime on all “on-system” projects (on the state route system) in order to improve resistance to moisture damage.

Since 1982 there has been much research performed to better understand the stripping phenomenon. As a result, there have been changes in both materials and technology over the past 25 years to improve the resistance to moisture damage and the ability to test for performance under adverse moisture conditions. Due to the changes in materials and technologies related to the development and improvement of liquid anti-strip agents, there is a need to conduct a research study to evaluate the effectiveness of anti-strip agents used in hot mix asphalt pavements.

There were two objectives of this research study. One purpose of the project was to evaluate the past field performance of Georgia’s mixes designed with hydrated lime to the performance of Georgia’s mixes designed using liquid anti-strip agents. The second purpose was to evaluate the long-term effectiveness of anti-strip agents based on laboratory aging procedures and to determine which product is best suited for Georgia.

Since 1982, only “off-system” projects (county and city roads not on the state route system) have been allowed to use liquid anti-strip agents. Therefore, the first objective attempted to identify off-system projects with similar traffic volume, age, aggregate source, and aggregate type as on-system projects. The comparisons were used to determine the in-place strength and performance of mixtures with liquid anti-strip to similar projects that used hydrated lime in the mixtures. Ten projects each were selected for this part of the project scope. The scope was revised to include additional moisture conditioning, including Hamburg Wheel Track testing, for five projects each with lime and liquid additive.

The second objective was to conduct a series of laboratory comparisons using different aging periods and test procedures to make long-term comparisons of the effectiveness of hydrated lime, liquid anti-strip (LAS), and Warm Mix additive (WMX) as agents to improve resistance to moisture damage.

Limestone was used in a 25 mm base course to consider a common assumption that hydrated lime does not perform as well with limestone as with granite due to similar chemical composition of limestone and hydrated lime. An Analysis of Variance (ANOVA)

of 25 mm limestone mix tensile strength showed that performance with hydrated lime was significantly better than test results with other additive types. Limestone mixture treated with hydrated lime averaged 98.7 % TSR while WMX averaged 81.4% and LAS averaged 77.8%. When only granite mixtures are considered, TSR results for the 25 mm mix averaged 115.0% for the hydrated lime treatment, 92.8 % for LAS and 82.4 % for WMX.

Moisture susceptibility was conducted at 0, 1, 5, and 10 freeze/thaw cycles. Results for 0 and 1 cycle were similar and results for 5 and 10 cycles were similar. Both 5 and 10 freeze/thaw cycles were significantly more discriminating than one freeze/thaw cycle alone.

A comparison of test results from project cores after 0, 1, and 3 freeze-thaw cycles shows that subjecting roadway cores to freeze-thaw conditions is more severe than vacuum saturation alone. From the results, the average tensile strength of liquid additive projects was reduced by 50% when comparing results after 3 freeze-thaw cycles to no freeze-thaw cycles. After 3 freeze-thaw cycles, the cores treated with hydrated lime had 50 percent higher tensile strength than the cores treated with liquid additive.

Conclusions from this research study are summarized as follows:

- Hydrated lime maintained the best TSR results for both granite and limestone 25 mm mixtures. The LAS additive performed better than WMX for granite mixtures, but WMX performed better for limestone mixtures.
- For 12.5 mm mixes, the most significant variable considered was additive type. Hydrated lime had the highest TSR results (107.4%), while LAS and WMX results were similar at 92.3 and 92.7%, respectively. Only the Lithia Springs aggregate treated with WMX and subjected to three freeze/thaw cycles failed to meet the average 80% TRS requirement.
- Multiple freeze/thaw cycles of 0, 1, 5, and 10 cycles were used for a portion of the research study. Hydrated lime had the highest tensile strength and highest TSR values and was the only additive treatment to meet the minimum of 80% TSR for all freeze/thaw cycle combinations. Both 5 and 10 freeze/thaw cycles were significantly more discriminating in regard to moisture susceptibility than one freeze/thaw cycle alone.
- Dynamic Modulus and Flow Number tests do not appear to be practical for use as moisture susceptibility tests. Dynamic Modulus results were not sufficiently discriminating, and Flow Number tests produced opposite results from other testing and conditioning methods.
- WMX treated mixtures produced low initial tensile strengths, but the strength of these mixtures improved with time.

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LIST OF ABBREVIATIONS

AASHTO: American Association of State Highway and Transportation Officials

AMPT: Asphalt Mixture Performance Tester

APA: Asphalt Pavement Analyzer

E*: Dynamic Modulus

ESAL: Equivalent Single Axle Load (18 kip load)

FHWA: Federal Highway Administration

FN: Flow Number

GDOT: Georgia Department of Transportation

GLM: General Linear Model

HMA: Hot-Mix Asphalt

HWT: Hamburg Wheel Tracker

IDT: Indirect Tensile

LAS: Liquid Anti-Strip

MEPDG: Mechanistic-Empirical Pavement Design Guide

NCAT: National Center for Asphalt Technology

NCHRP: National Cooperative Highway Research Program

NMAS: Nominal Maximum Aggregate Size

SGC: Superpave Gyratory Compactor

TSR: Tensile Strength Ratio

WMX: Warm Mix Additive

CHAPTER 1: INTRODUCTION

Background

In 1969, due to stripping problems in asphalt pavements, the Georgia Department of Transportation (GDOT) began requiring a liquid anti-strip agent to be mixed with the asphalt cement when producing hot mix asphalt. Stripping is defined as the loss of bond, or adhesion, between asphalt cement and aggregate particles in the presence of moisture (*1*). The liquid anti-strip agent was used in asphalt mixtures for all routes until an extensive stripping study NCHRP Project 4-8 (3), "Predicting Moisture-Induced Damage to Asphaltic Concrete" (*2*) was conducted in 1978. The NCHRP 4-8 (3) project began the development of a laboratory procedure for evaluating the moisture susceptibility of asphalt mixtures often referred to as the Lottman procedure. GDOT participated in this research study and became involved in other research that considered the effectiveness of anti-strip additives. This Lottman procedure was a forerunner of the currently used AASHTO T 283 test procedure for moisture susceptibility.

In 1980, GDOT experienced several pavement failures due to an extremely hot summer. This led to a pavement investigation that included 81 interstate projects. Only 32 percent of those projects did not have visual stripping, and 24 percent of the projects had moderate to severe stripping. As a result, GDOT began implementing the use of other alternative products to reduce moisture susceptibility. One of the most promising alternatives was hydrated lime. The average tensile strength of several mix designs representing use on a statewide basis were compared with lime and liquid anti-strip for each of six different mix types. In each case, the mixtures treated with hydrated lime had the highest tensile strength after moisture conditioning as compared to mixtures with liquid anti-strip additive. In 1982, GDOT implemented the use of hydrated lime on all "on-system" projects (on the state route system) in order to improve resistance to moisture damage.

Since 1982, there has been much research performed to better understand the stripping phenomenon. As a result, there have been changes in both materials and technology over the past 25 years to improve the resistance to moisture damage and the ability to test for performance under adverse moisture conditions. Due to the changes in materials and technologies related to the development and improvement of liquid anti-strip agents, there is a need to conduct a research study to evaluate the effectiveness of anti-strip agents used in hot mix asphalt pavements.

Project Objectives

There were four objectives of this research study. One purpose of the project was to evaluate the past field performance of antistrip additives with several aggregate sources including both granite and limestone. The limestone was used only in a 25 mm base mix since limestone is not typically used in surface mixtures due to a tendency to polish under traffic. Three granite sources were used in all mixtures. A second objective evaluated performance of Georgia's mixes designed with hydrated lime to the performance of Georgia's mixes designed using liquid anti-strip agents. Since 1982, only "off-system" projects (county and city roads not on the state route system) have been allowed to use liquid anti-strip agents. Therefore, off-system projects were identified with similar age, aggregate source, and aggregate type as on-system projects. Another objective of the project was to construct field test sections where three different anti-strip agents were used in a conventional Superpave surface mixture. However, only one such test section was constructed due to difficulty in negotiating with contractors for a change order at no increase in cost. As a result, the research effort was modified during the course of the contract to include a final objective which involved conducting a series of laboratory performance tests using different aging periods to make long-term comparisons of the effectiveness of different types of anti-strip.

Scope

In order to accomplish this research, aggregate was obtained from three granite sources and one limestone source. The granite sources involved one that is not typically identified as a stripping aggregate, one that has stripped in the past and required both hydrated lime and liquid antistrip to meet tensile strength requirements, and a third granite material that is well-known for its stripping potential. Since limestone is primarily available only in northwest Georgia, a source from that geographical area was used for the 25 mm base mixture. All mixtures were subjected to one and three freeze-thaw cycles. Secondly, a review of GDOT archival project records was made to select a total of 20 projects statewide with asphalt mixes for 10 projects being treated with liquid anti-strip additive and compared to 10 paired projects treated with hydrated lime. An effort was made to select projects which have similar age and environmental conditions. Cores were then taken from each project and tested for tensile strength.

Additionally, a test section was placed on State Route (SR) 319 in Tift County using three different anti-strip agents: liquid additive, hydrated lime, and a new warm mix additive (WMX) that is also used as an antistripping additive. Samples were taken immediately after construction and after two years of service.

To accomplish the final objective, laboratory samples from two aggregate sources as well as samples from the Tift County project were tested after multiple freeze-thaw cycles to determine how well the mixtures would resist moisture damage under severe conditions. Samples were also tested with the HWT to evaluate both rutting and stripping resistance. Laboratory samples were tested for dynamic modulus (E*) and flow number (FN) as well.

CHAPTER 2: LITERATURE REVIEW

Moisture Susceptibility Testing

The effect of moisture damage on performance of asphalt pavements has been a concern and a subject of research for more than three-quarters of a century with documented research reports as far back as 1932 (*3*). The common form of distress, known as stripping, is defined as the displacement of asphalt cement films from aggregate surfaces by loss of adhesion primarily due to the action of water or water vapor (*1*, *4*). A study of the mechanisms of stripping by Taylor and Khosla (*5*) identified at least five different processes by which stripping of asphalt from aggregate particles may occur.

1. Detachment - Separation of the asphalt film from the aggregate surface even when there is no break in the asphalt film. Detachment is related to the surface energy that exists at the aggregate/asphalt film interface. Since asphalt is primarily composed of hydrocarbons, it has very little polar activity. On the other hand, both aggregate and water are highly polar and are then attracted by much stronger forces. As a result, the asphalt film peels cleanly from the aggregate surface in a complete loss of adhesion.
2. Displacement - Penetration of water into the aggregate surface through breaks in the asphalt film. The breaks in film may be due to lack of complete coating during mixture production, or may have resulted from rupture of the film particularly at sharp, angular edges of aggregate particles (or even fracture of particles) during compaction or under the loading effect of traffic.
3. Spontaneous Emulsification - Combination of water and asphalt to form an inverted emulsion which results in total loss of adhesion. Fromm (*6*) found that the rate of emulsification depends on the nature of the asphalt and the presence of additives, with some additives actually accelerating the process. Fromm also found that the emulsification process could be reversible; because as water evaporated the asphalt would convert back to its original condition.
4. Pore Pressure - Development of hydraulic pressures that develop under traffic when water is allowed to flow freely through interconnected voids, or when water becomes trapped in impermeable voids of a densified pavement. The end result is stripping of the asphalt film from the aggregate.
5. Hydraulic Scouring - Results when water is pressed down into the surface layer by the action of traffic. As a vehicle travels over the pavement surface, water is pressed into the surface voids just ahead of the tires and is quickly sucked back out as the tire rotates off the point of surface contact.

Fromm (*6*) also indicated that stripping of the pavement structure typically begins with the coarse aggregate in the bottom layers and gradually migrates upward.

There have been many tests developed that try to simulate the stripping effect by one or more of the mechanisms described above. Various early studies have considered saturation, aging and environmental effects, sonic testing, immersion-compression, tensile strength, resilient modulus, and analyses using scanning electron microscopy (*7, 8, 9, 10, 11*). However, no procedure was widely accepted due to lack of reliability and that lab conditioning was not correlated to field environments. Then in the mid-1970s, NCHRP 4-8(3), "Predicting Moisture-Induced Damage to Asphaltic Concrete" was conducted by Robert Lottman of the University of Idaho with the objective of developing a laboratory test procedure for predicting the ability of asphalt mixtures to resist the effects of moisture damage (*2*). The study eventually involved analysis of 17 pavements in 14 states by taking cores from existing pavements that had been placed over a range of several years and comparing results to laboratory mixes made from the same aggregate and asphalt binder sources as used during construction. This research led to the development of what is often referred to as the Lottman Procedure.

Background information for the NCHRP 4-8(3) research was based on an earlier three year study sponsored by the Idaho Department of Highways (*12*). This study found that moisture damage was expected to be more severe in the lower pavement courses, but agencies also reported damage occurring in the upper surface layers as well. Vacuum saturation alone was originally used to simulate a saturated field condition. The vacuum process consisted of pulling a 26 in. Hg vacuum for 30 minutes followed by keeping samples submerged for an additional 30 minutes. But it was found in the early work by Lottman and in work by Schmidt et al (*13*) that vacuum saturation alone did not result in strengths as low as what was being obtained from roadway cores. As a result, experiments were performed with a variety of moisture conditioning procedures for laboratory prepared specimens. The best two methods were selected for further study in NCHRP 4-8(3) and the study was expanded to cover 17 pavements (2 to 12 years old) in 14 states including those with a wide range of moisture damage severity as well as those with no moisture damage. The comparison of dry strength to vacuum-saturated specimens was considered relative to a "short-term" ratio that simulated pavement damage when the pavement approached saturation in the field. For "long-term" ratios, a freeze-thaw cycle was required in order to simulate the damage effects of environment and traffic (*14)*. The practice of comparing wet to dry strength is believed to have originated with use of the Immersion-Compression test (*15*).

The initial testing under NCHRP 4-8(3) included thermal cycling at two levels. Samples were cooled to 0^{o}F the heated to 120^{o}F and cooled back to 0^{o}F (0-120-0 cycle) for one complete cycle that took about 8 hours to complete. A thermal cycle of 40-120-40 was also used in the experiment. Conditioning for 12 and 18 cycles was conducted, but it was found that 18 cycles produced about 10 percent more damage and more closely compared to field core results. However, to complete 18 thermal cycles required a period of six days, so an accelerated procedure of using a freeze-thaw cycle (15 hours at 0^{o}F followed by 24 hours at 140^{o}F) after vacuum saturation was implemented. The accelerated procedure had an advantage not only in speeding up the testing time, but also did not require a special thermal conditioning chamber.

Indirect tensile strength tests were performed at 55^{o}F at a loading rate of 0.065 in/min based on the Idaho study (*12*) and 73^{o}F at a loading rate of 0.150 in/min (*16*). Test results of the 55^{o}F test temperature were more closely correlated to overall field core results, but the 73^{o}F temperature was considered more practical as it did not require a special 55^{o}F conditioning water bath or incubator. But, this comparison also showed that there was less variability when the 55^{o}F test temperature was used (*12*).

In a comparison of field core results to those of laboratory prepared specimens, it was found that roadway cores typically had higher strength. The higher strengths were believed to be due to the age-hardening that takes place on the roadway caused by exposure to environmental elements over several years. In order to improve the correlation between lab and field results a strength ratio was used (*12*) in which strength of conditioned samples were compared to dry strengths. It was found that tensile strength ratios (TSR) less than 0.70 resulted from mixtures that could be predicted to have moderate to severe moisture damage with a high degree of accuracy.

Tensile strength tests were performed by placing prepared specimens between two flat metal strips and loading with a compression test machine at the desired loading rate until the maximum load, P_{max}, was obtained. The load was then removed and the average flattened width was measured. Tensile strength was then calculated using Equations 1 and 2.

$$S_t = \frac{s_{10}}{10{,}000} \times \frac{P_{max}}{t}$$ **Equation 1**

$$s_{10} = 1591 + 437a - 1889a^2 + 2854a^3 - 2474a^4 + 885a^5$$ **Equation 2**

Where,

S_t = tensile strength in psi
s_{10} = maximum tensile stress, in psi, in a 4 in. diameter sample by a load of P = 10,000 lb. per inch of thickness
a = amount of specimen flattening, in inches, under P_{max}
P_{max} = maximum compressive load in lb.
t = thickness of specimen in inches

For convenience, a table of S_{10} values was calculated from Equation 2 based on flattened width for use on a routine basis.

Phase II of the NCHRP 4-8(3) study consisted of field validation of the developed test procedure by seven agencies on 8 test sections of new pavement constructed between 1975 and 1977 with aggregates that had a history of moisture damage (*14*). Two test sections from Georgia were used in this study (on U.S. 78 in Walton County) with the difference being that for one section 0.5% liquid antistrip was used in all layers, and for the other section 0.5% antistrip was used in all layers except the bottom 4 in. of asphalt base (3/4 in. maximum aggregate size). For each mixture 9 specimens were prepared: 3 for control, dry strength; 3 for vacuum saturation only; and 3 for vacuum saturation plus freeze-thaw. The long-term ratio for the Georgia mixes was 0, indicating a total reduction in mechanical properties due to stripping (*14*). The high traffic volume (101,000 one-way ESALs annually) likely contributed to the accelerated moisture damage on the Georgia test sections. In 6 of the 8 test sections, stripping was observed when the TSR dropped below 0.80. The study found no particular bias when testing field cores from either the wheelpath or between the wheelpath. It was also found that “short-term” ratios may exceed 1.0, and this was believed to be caused by the stiffening and early aging of the pavement due to environmental effects. However, the mechanical properties of field cores began to decline after about 1 to 4 years of pavement age (*14*).

There were, however, some concerns that the Lottman procedure may be too severe in terms of moisture conditioning (*5*). Tunnicliff and Root modified the Lottman procedure in an effort to evaluate the effectiveness of antistripping additives as well as specific anti-stripping additive-aggregate-asphalt combinations (*4*). In their study, NCHRP 10-17, the primary objective was to provide guidelines for incorporating anti-stripping additives into asphaltic concrete mixtures. The study found that there were at least 14 test procedures being used with additives to measure various properties and there were at least 100 known additives at the time of the 1984 report. Of those procedures, only three were AASHTO or ASTM standards (*4*). The study found that aggregate type, gradation, source and grade of asphalt cement, and source and dosage rate of additives had a significant effect on resistance to moisture damage.

By contrast, in NCHRP 10-17, Tunnicliff and Root used samples of the original asphalt and additive with Georgia aggregate from the same source as was used in the latter Lottman study, but markedly different results were obtained. In the Lottman study, the Georgia mixtures were the most severely stripped of all; but with the Tunnicliff-Root procedure, the Georgia mixture with additive was ranked best (Table 1). A subsequent study of the same materials by the Laramie Energy Center (*17*) indicated that a dosage rate of 0.25 percent antistrip, as opposed to the required 0.5 percent, provided better performance. Tunnicliff -Root concluded that there was as much evidence that the additive used in Georgia was effective as there was that it was ineffective and suggested a possibility that an incorrect dosage rate was used in the Lottman study.

TABLE 1: Comparison of TSR Test Results (*4*)**.**

Material	**Lottman Study**		**Tunnicliff-Root Study**	
	TSR, %	**Rank**	**TSR, %**	**Rank**
ID w/lime	82	1	81	2
FHWA	63	2	41	7
MT	62	3	81	2
VA	35	4	81	2
CO	22	5	64	5
AZ	21	6	45	6
GA	0	7	37	8
GA w/additive	0	7	83	1

The Tunnicliff-Root procedure differed from the original Lottman procedure in three primary ways.

1. Vacuum saturation is limited to 55-80 percent with the idea that further saturation may cause damage to the specimen other than stripping. The Tunnicliff-Root study (*4*) also showed a correlation between specimen air voids and degree of saturation. It was recommended that specimens with average air voids closer to 6 percent than 7 percent should be saturated to a level above 70 percent.

2. A loading rate of 2 in./min. was used so that typical Marshall loading apparatus could be used and a 77°F water bath was used for final conditioning rather than the 55°F water bath required for the Lottman Procedure. The 77°F bath did not require ice or special equipment for cooling specimens. A study of the 77°F test temperature and 2 in./min. loading rate was compared to Lottman's 55°F temperature and 0.065 in./min. loading rate and was found to have an excellent correlation (*18*).
3. Loading strips specified in ASTM D 4123 (*19*), and used in prior research by Anagnos and Kennedy (*20*), which were 0.5 in. wide for use with 4 in. diameter specimens and 0.75 in. wide for 6 in. diameter samples were used. The tensile strength was then calculated by Equation 3.

$$S_t = \frac{2P}{\pi D t} \qquad \textbf{Equation 3}$$

Where,

S_t = tensile strength in psi
P = maximum compressive load in lb.
t = thickness of specimen in inches
D = Diameter of the specimen in inches

An FHWA-sponsored study at Iowa State University compared TSR results of 4 inch Marshall specimens to 4 inch and 6 inch gyratory compacted samples (*21*). The study found that it would take 3 freeze-thaw cycles to achieve an equivalent level of moisture damage for 6 inch gyratory samples as the Marshall specimens. However, Epps, et al (*22*) found no statistically significant differences in dry tensile strength in 18 of 24 comparisons between 6 in. gyratory-compacted samples and 4 in. Marshall specimens. Wet tensile strengths after one freeze-thaw cycle were compared for 5 mixes. In one case, the Marshall results were higher, in 2 of 5 tests results were the same, and in 2 of 5 tests the gyratory-compacted samples produced higher strengths. Based on these results, a full factorial was conducted with Nevada mixes and it was found that freeze-thaw tensile strengths of 6 in. gyratory samples were statistically the same as for 4 in. Marshall specimens in 22 of 24 possible comparisons (*22*). The study also found that the level of saturation had little effect on tensile strength.

Effect of Antistrip Agents

Another study by Anderson et al, indicated that there were significant differences and unique effects of different additives with different asphalt sources such that the effectiveness of liquid antistrip is asphalt source specific (*23*). Further, the compatibility between aggregate and asphalt binder source is critical to the prevention of moisture damage.

Liquid antistrip additives have sometimes been found to be more effective by using as a pre-treatment for the aggregate prior to adding the bitumen. This is because the additive is a surface active agent, or surfactant, that allows the asphalt to coat the aggregate surface more evenly by reducing surface tension, and at the same time, displaces adsorbed water on or near the aggregate surface (*1*). However, blending the additive with the binder is more economical and practical and is the generally accepted method.

Hydrated lime has been used successfully as an antistrip agent as well. Its use in asphalt mixtures can be traced back to the Warren Brothers patent for asphalt paving materials in 1910 (*24*) although it was mainly used as a filler at the time, and was eventually replaced with more economical local mineral fillers. Schmidt and Graf (*13*) reported that hydrated lime was the most effective additive in reducing moisture damage because the calcium from the lime replaces hydrogen, sodium, and potassium and other acidic components on the aggregate surface and helps neutralize acids of the asphalt cement. As the calcium rich aggregate surface reacts with the organic acids in the asphalt binder, a water-resistant surface is formed (*1*).

A study at the Western Research Institute determined that the addition of hydrated lime benefited the pavement in several ways: reduced asphalt age-hardening, increased high-temperature stiffness of unaged asphalt, increased tensile elongation of asphalt at low temperatures, and improved resistance to moisture damage. The effect of these benefits results in increased durability, reduced rutting, shoving, and other forms of deformation, improved fatigue resistance in aged pavements, and improved resistance to low-temperature transverse cracking (*25*). The study included a granite gneiss aggregate from Grayson, Georgia with which TSR values ranged from 25% with no treatment, 93% when lime was introduced into dry aggregate, and 102% when lime was added to moist aggregate.

When using antistrip additives, NCHRP Report 274 (*4*) gave several thoughts or guidelines to consider.

- Stripping characteristics of aggregate from a single source may change when operations move from one strata to another. No aggregate type always strips, and no aggregate type never strips.
- The source of asphalt cement is a significant variable.
- There seems to be no such thing as a foolproof antistripping additive.
- Neither aggregate source nor asphalt source can be assumed to be constant and always maintain the same stripping potential.
- Different additives yield different test results in the same mixture.
- There are several moisture damage test procedures and various agency modifications of those so that it is unlikely that the various procedures can produce the same results.
- Aggregates should be used with the proposed additive in a condition as close as possible to what is expected in actual field application.
- Decisions regarding aggregate, asphalt, and additive performance at the time of mix design should be considered tentative as the combinations and material properties may vary during actual construction.
- Maintaining a list of approved additives is an ineffective means of ensuring additive effectiveness.
- No basis has been found for choosing the best time or point to introduce additives

into asphalt cement.

- The Boiling Water Test (such as GDT 56 - Heat Stable Anti-strip Additive; formerly ASTM D 3625; sometimes referred to as the Texas Boil Test) may be used as a rapid quality control field test to provide early information regarding potential performance. Unsatisfactory results should be followed with more intense testing of mixtures.
- Current moisture susceptibility tests only indicate relatively short-term performance. There is no known test procedure that reliably predicts long-term performance of additives in pavements.

A Texas study by Liu and Kennedy in 1991 (*26*) involved test sections across the state to evaluate the effect of various additives. Projects included a control section with no treatment, a section treated with hydrated lime, and various other sections treated with liquid antistrip. In all, a total of 14 different liquid antistrip additives were used. The study showed that hydrated lime was effective in most, but not all, districts; and that some districts benefitted from all liquid antistrips, while others showed no benefit from antistrip.

A study by Sebaaly, et al, (*27*) compared performance of laboratory tests for 15 mixtures using aggregate sources from 5 states with 3 additive treatments- no additive, 0.5% liquid additive, and 1% hydrated lime. Up to 15 freeze-thaw cycles were used for accelerated conditioning. Performance properties were then used in the AASHTO MEPDG software to conduct 20-year structural designs to perform a cost analysis for the three treatments. Both lime and liquid antistrip were found to improve resistance to moisture susceptibility based on TSR results. However, the untreated and liquid additive treated mixtures had significantly reduced strength after multiple freeze-thaw cycles whereas the hydrated lime treatment maintained high strength values for the entire 15 cycles for all five aggregate sources (*27*). Life cycle cost data for hydrated lime treated mixes showed significant savings (up to 45%) based on MEPDG performance predictions, and four of the five aggregate sources showed lime treatment would always result in savings on the order of 13 - 34% and that performance would always outweigh its cost. Meanwhile, the study showed that liquid antistrip may even result in additional cost due to materials cost and reduced benefits.

An earlier study of field projects in Nevada compared projects with lime-treated mixtures to projects constructed at the same time and with the same aggregate sources with no treatment. Project specimens were subjected to as many as 18 freeze-thaw cycles. Mixtures with no treatment failed completely after 10 cycles while the lime-treated mixtures still maintained resistance to moisture damage after 18 freeze-thaw cycles. The study found that for environmental and traffic conditions in Nevada, the lime-treated mixtures provided an average of 3 additional years of performance life (*28*). The study further showed that the sixth freeze-thaw cycle represented the critical stage for damage to asphalt pavements. It also verified that there was generally no significant difference in strength properties for cores taken from the wheelpath as compared to cores taken from between the wheelpath.

Long-Term Performance

Georgia began using liquid antistripping additives in the late 1960s due to concerns with poor performance of asphalt mixtures relative to resistance to moisture damage. Initially, only the minimum amount of antistrip needed based on the boil test was used. In 1979, a survey of Interstate projects found early to severe moisture damage on 11 of 29 projects prompting a change to require at least 0.5% liquid antistrip on all dense-graded mixes and 1.0% in all open-graded mixtures (*29*). The decision was also based on laboratory results that compared effectiveness of 0.5, 0.75, and 1.0 percent liquid antistrip. While 0.5% antistrip showed considerable improvement over no treatment, the 0.75% rate did not provide a magnitude of improvement over the 0.5% rate that would justify the additional cost. A follow-up survey in 1981 of 81 projects in Georgia that used liquid antistrip found that more than two-thirds of the projects had slight to severe stripping. From NCHRP 4-8(3), it was found that agencies had concern about the long-term cost effectiveness liquid antistrip additives because of possible construction and aging factors in the field which reduce their effectiveness (*12*). The two Georgia test sections in the Lottman study, for example, yielded acceptable results for about 30 months after initial construction. After that period, some of the cores began to totally disintegrate during the coring operation.

Georgia first began experimenting with the use of hydrated lime in 1972, but it was decided at that time to continue using liquid antistrip due to the additional cost and additional equipment that would be required for introduction of lime into the asphalt plant. After the 1981 survey, GDOT began using hydrated lime on high traffic volume routes. A limited laboratory study was conducted to consider the best method of introducing the lime onto the aggregate: add in dry powder form, or add as a water/lime slurry. Figure 1 shows a comparison of two methods. Although the slurry method was believed to result in better coating, it was decided there was not sufficient difference in results to justify extra handling of aggregate and additional drying costs. A third method, adding to dampened aggregate (as would be done if added to aggregate on the cold feed belt) was also considered, but results were not significantly different than the dry method. A limited study was also conducted in which treatment with both liquid antistrip and hydrated lime was compared to performance of using hydrated lime alone. TSR results for mixes with both lime and additive were essentially the same as for mixes which used lime treatment only so the additional cost of additive was not justified. In 1982, GDOT began requiring hydrated lime as the preferred antistrip agent in all "on-system" state route projects (*29*).

After implementing the use of hydrated lime, GDOT began monitoring projects on an annual basis to determine the effectiveness of hydrated lime over time (*30*). The evaluation included 24 projects, and after five years of satisfactory performance the investigation was limited only to projects which had an OGFC surface since those projects had shown the most severe potential for stripping in the past. After 10 years, average tensile strength values were still slightly more than 100 percent of the initial field core strength (Figure 2).

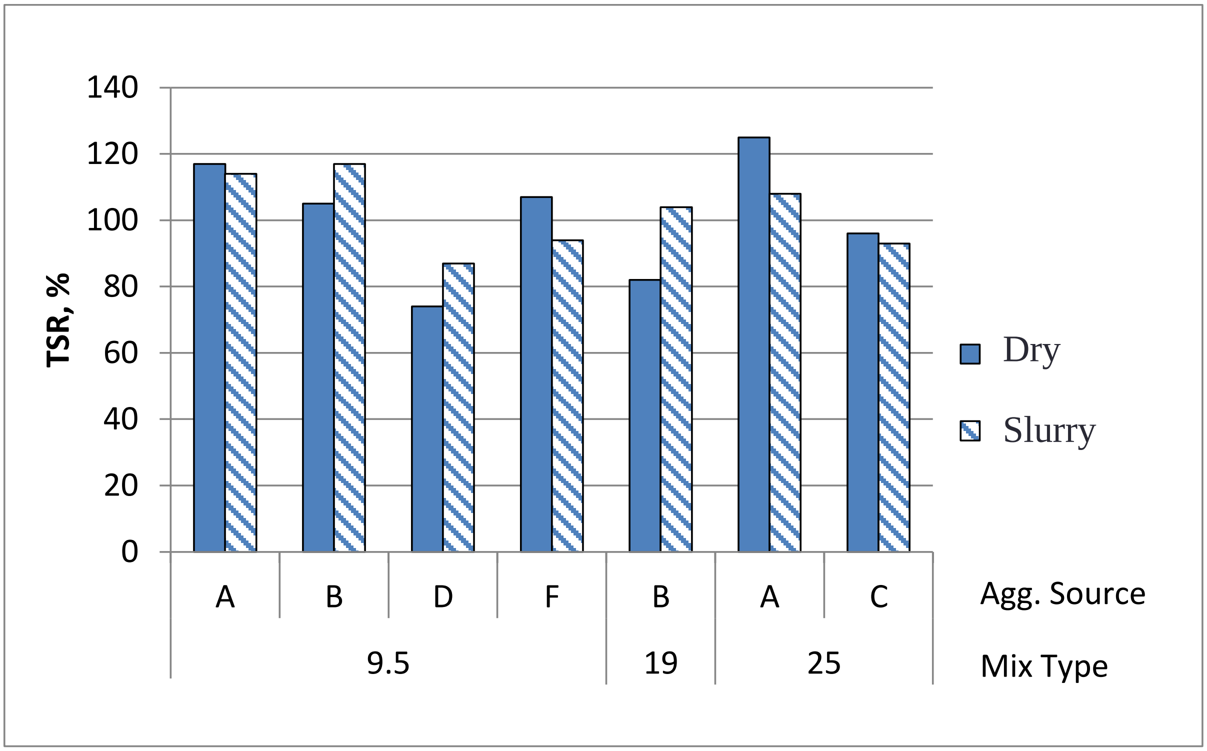

FIGURE 1: TSR Results for Dry vs Slurry Lime Treatment.

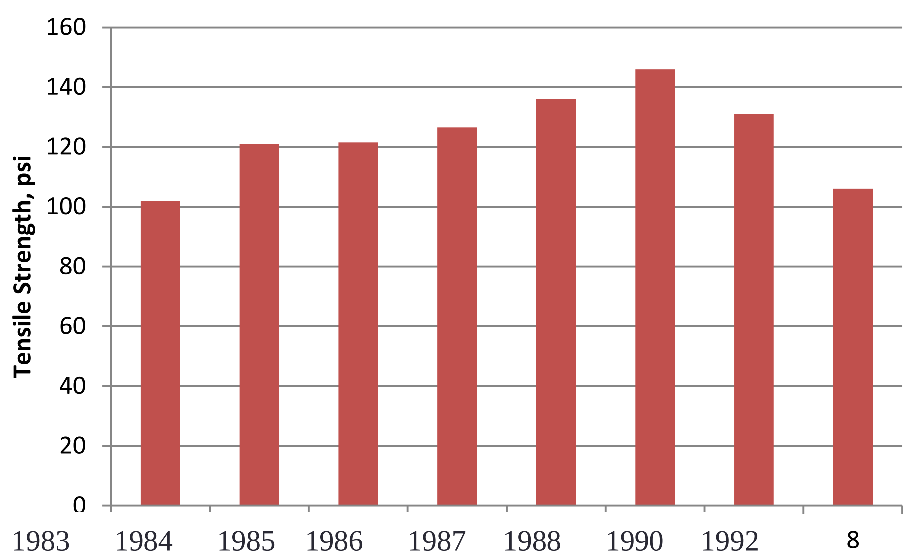

FIGURE 2: Tensile Strength Results of Lime-Treated Projects over Ten Year Period.

CHAPTER 3: LABORATORY TSR COMPARISONS

Vulcan at Lithia Springs (Source A) is a granite source that is not typically associated with severe stripping and was used as the standard baseline for comparisons. A second granite source was Vulcan at Kennesaw (Source B) and it was selected due to its potential for stripping in the past. An aggregate source that is well-known history of stripping (Hanson at Lithonia- Source C) was obtained to prepare mixtures for the laboratory research. Limestone aggregate from Dalton (Source D) was also used since neither lime nor liquid anti-strip seems as effective for limestone mixtures as for granite aggregate. Only 25 mm Superpave mixes were used for the tests with limestone materials since limestone is not used for surface mixes except on low traffic volume projects (<800 ADT). For high traffic volumes, limestone may be used in base and intermediate asphalt courses, but granite materials would be used in the surface mixes.

Three anti-strip agents were used in this study. Hydrated lime at a dosage rate of 1.0 percent of dry aggregate weight was used as the standard material. One liquid anti-strip agent (designated as LAS) selected from Qualified Products List (QPL) 26 (*31*) was used, and a third anti-strip (WMX) used in the laboratory research was a promising new product that was developed as both warm mix and antistrip additive.

Since asphalt content and aggregate gradation may also affect moisture susceptibility, two Nominal Maximum Aggregate Size mixtures, 25 mm and 12.5 mm were used for the laboratory evaluation. It was required to determine optimum asphalt content and moisture susceptibility testing for 21 mix designs [(3 aggregates x 3 additives x 2 mix types) + (1 aggregate x 3 additives x 1 mix type)]. In addition, three conditioning procedures were used.

Control specimens were subjected to tensile strength testing without a freeze-thaw cycle. Results from one and three freeze-thaw cycles were compared by using Georgia's moisture susceptibility test GDT-66. One freeze-thaw cycle is typically used, but some recent research has indicated that one freeze-thaw cycle may not be severe enough to adequately distinguish between moisture susceptibility of mixtures (*21*). Gyratory compaction at 65 gyrations was used to determine the optimum asphalt content at 4.0 percent air voids.

Tensile Strength Ratio (TSR) Testing Methodology

Tensile Strength Ratio (TSR) moisture susceptibility testing was performed for this project in accordance with GDT 66 (*32*). The GDT methodology uses 95 mm samples compacted in a Superpave Gyratory Compactor in accordance with AASHTO T312. These samples were mixed in the laboratory at a target mixing temperature of 300°F and aged two hours at the compaction temperature of 280°F prior to compaction. The target air void level for these samples was 7.0 ± 1.0%.

A set of three specimens were vacuum saturated at 26 inches of mercury below atmospheric pressure for 30 minutes. The samples were then allowed to rest for 30 minutes prior to their SSD weight being determined. This allowed the percent saturation (or percentage of internal voids filled with water) of each specimen to be determined (Equation 4). The samples were then placed in a freezer for a minimum of 15 hours prior to being placed in a warm water bath at 140°F (60°C) for 24 hours. This process constitutes one ‘freeze-thaw’ cycle. These ‘conditioned’ specimens, along with a group of three unconditioned specimens that had not been saturated, were then tested for indirect tensile strength using a Geotest® load frame (Figure 5). The conditioned specimens were removed from the water bath and allowed to come to ambient temperature for one hour prior to pre-test conditioning. All samples are placed in a 55°F (12.8°C ± 2°C) water bath for two hours to equilibrate testing. The samples are tested at a loading rate of 0.065 inches/minute. The peak load for each specimen is recorded and used (along with the specimen dimensions) to calculate the splitting tensile strength for each individual specimen (Equation 3).

The ratio of the average indirect tensile strengths of the conditioned and unconditioned samples is recorded as the tensile-strength ratio (TSR) (Equation 5). In accordance with Georgia Special Provision 828, the minimum TSR requirement for an asphalt mixture is 0.8 (indicating a 20% reduction in indirect tensile strength) with no individual specimen being allowed to have a splitting tensile strength lower than 60 psi. Additionally, a mix may be said to pass the TSR criteria with a TSR greater than 0.7 so long as the individual splitting tensile strength test values all exceed 100 psi.

FIGURE 3: GEOTEST® Load Frame.

$$Percent\ Saturation = \frac{100*(D-A)}{(C-B)*E} \qquad \textbf{Equation 4}$$

Where: A = Dry Weight in Air
B = Weight in water before vacuum saturation
C = SSD Weight before vacuum saturation
D = SSD Weight after vacuum saturation
E = Specimen Air Voids (%)

$$TSR = \frac{Average\ St-Conditioned}{Average\ St-Unconditioned} \qquad \textbf{Equation 5}$$

TSR Results

The early portion of the laboratory study was to look at the effects of aggregate type, additive type, and mix type. Mix type was a consideration for two reasons: to determine if the same performance of additives in surface mixtures was consistent with subsurface mixtures, and to evaluate the effectiveness of additives with both limestone and granite aggregate mixtures. A total of 189 tensile strength tests were required for this portion of the study. Test results are included in the appendix. A comparison of TSR results for each aggregate source and mix type is shown in Figure 4.

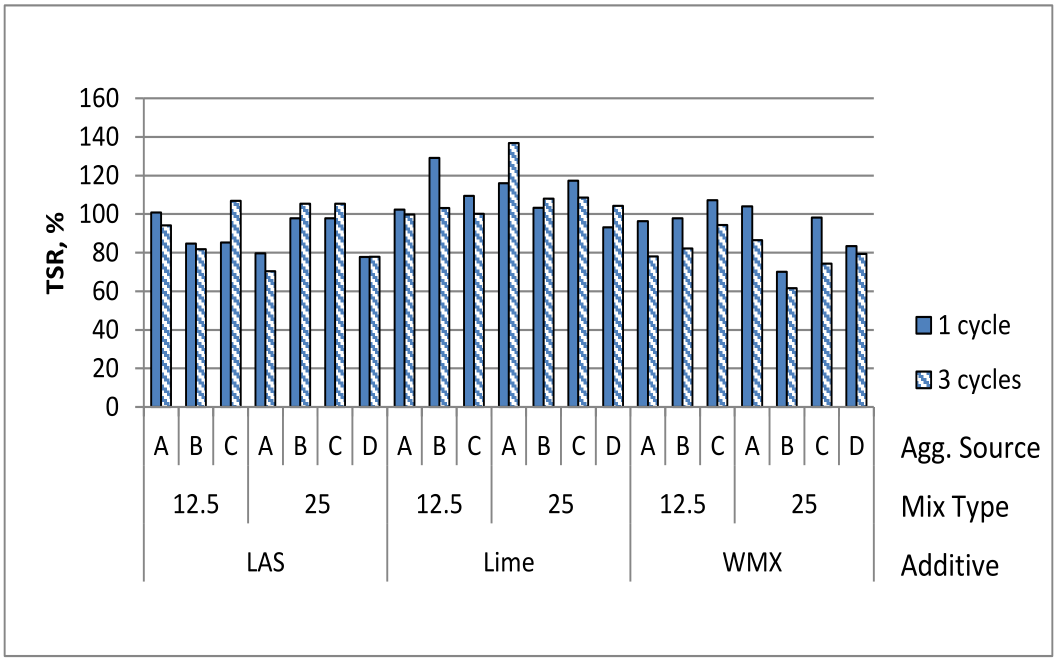

FIGURE 4: TSR Results by Aggregate Source, Mix Type, and Additive Type.

One of the concerns with limestone aggregate is a common perception that hydrated lime may not be as effective with limestone as with granite materials. Since limestone is typically used in base courses in northwest Georgia, aggregate from a source within that area was used. From Figure 4, it is apparent that retained tensile strength was higher for the limestone mixtures treated with hydrated lime (Additive 2) than for the same mixtures treated with other additives. This was also confirmed with an ANOVA that considered the statistical significance of the effect of additive and number of freeze-thaw cycles (Table 2). The ANOVA shows additive type to be highly significant (*P*-value of 0.000) and a Tukey pairwise comparison shows that the hydrated lime is in a statistically different grouping than the other additives at a 95% level of confidence. The results also show that the number of freeze-thaw cycles was not a significant factor in the results. The average TSR for limestone mixture including both 1 and 3 freeze-thaw cycles treated with hydrated lime ranged from 93.1 to

104.3; the WMX additive treatment ranged from 79.5 to 83.4, and the TSR for LAS treated mix ranged from 77.7 to 77.8. Average TSR values are shown in Table 2.

TABLE 2: General Linear Model: TSR versus Additive, Cycles for 25 mm Mix with Limestone Aggregate.

Additive: 1=LAS, 2=Lime, 3=WMX

```
Factor    Type   Levels  Values
Additive  fixed       3  1, 2, 3
Cycles    fixed       2  1, 3

Analysis of Variance for TSR, using Adjusted SS for Tests

Source           DF   Seq SS   Adj SS  Adj MS      F      P
Additive          2  1493.46  1493.46  746.73  21.33  0.000
Cycles            1    27.23    27.23   27.23   0.78  0.395
Additive*Cycles   2   182.03   182.03   91.01   2.60  0.115
Error            12   420.15   420.15   35.01
Total            17  2122.87

S = 5.91711   R-Sq = 80.21%   R-Sq(adj) = 71.96%
Unusual Observations for TSR

Obs     TSR     Fit  SE Fit  Residual  St Resid
  6  67.131  77.843   3.416   -10.712     -2.22 R

R denotes an observation with a large standardized residual.

Grouping Information Using Tukey Method and 95.0% Confidence

Additive  N   Mean  Grouping
2         6  98.67  A
3         6  81.43    B
1         6  77.79    B

Means that do not share a letter are significantly different.
```

A comparison of all TSR results for 25 mm mixtures shows that the LAS treated mix failed to meet the minimum requirement of 80% TSR for both Lithia Springs and Dalton aggregate with both one and three freeze-thaw cycles. However, LAS with Lithia Springs aggregate only marginally failed with an average TSR value of 79.7%. The WMX treated mixture also failed to meet the minimum requirement of 80% TSR for Kennesaw, Lithonia, and Dalton aggregate at three freeze-thaw cycles and for Kennesaw aggregate with one freeze-thaw cycle. When only 25 mm limestone aggregate is considered, mixture treated with hydrated lime averaged 98.7 % TSR while WMX averaged 81.4% and LAS averaged 77.8%. When only granite mixtures are considered, TSR results for the 25 mm mix averaged 115.0 % for the hydrated lime treatment, 92.8 % for LAS and 82.4 % for WMX; therefore, hydrated lime maintained the best TSR results for both granite and limestone 25 mm mixtures. The LAS additive performed better than WMX for granite mixtures, but WMX performed better for limestone mixtures.

In addition to additive source being a significant factor, an ANOVA of results (Table 3) for all 25 mm mixes shows that aggregate source is also significant (*P*-value = 0.000) as well as the interaction of aggregate and additive sources. The interaction of additive type and number of freeze-thaw cycles was also significant. Surprisingly however, TSR results were highest with Lithonia aggregate which is known to be susceptible to stripping. Results with the Dalton limestone aggregate were lower than for all three of the granite sources. The Tukey comparison also showed a significant difference for additive type with hydrated lime, LAS, and WMX each being in a separate grouping category. This means there is a significant difference in TSR results for each of the additive treatments.

TABLE 3: General Linear Model: TSR versus Agg, Additive, Cycles for all 25 mm Mix.

Agg1=Lithia Springs
Agg2=Kennesaw
Agg3=Lithonia
Agg4=Dalton
Cycles= 1 and 3
Additive- 1=LAS, 2=Lime, and 3=WMX

```
Factor    Type   Levels  Values
Agg       fixed       4  1, 2, 3, 4
Additive  fixed       3  1, 2, 3
Cycles    fixed       2  1, 3


Analysis of Variance for TSR, using Adjusted SS for Tests

Source               DF    Seq SS    Adj SS   Adj MS      F      P
Agg                   3   2462.31   2462.31   820.77   9.31  0.000
Additive              2  10854.00  10854.00  5427.00  61.56  0.000
Cycles                1     47.81     47.81    47.81   0.54  0.465
Agg*Additive          6   6670.13   6670.13  1111.69  12.61  0.000
Agg*Cycles            3    318.37    318.37   106.12   1.20  0.319
Additive*Cycles       2   1340.10   1340.10   670.05   7.60  0.001
Agg*Additive*Cycles   6   1036.91   1036.91   172.82   1.96  0.090
Error                48   4231.90   4231.90    88.16
Total                71  26961.53
S = 9.38960   R-Sq = 84.30%   R-Sq(adj) = 76.78%

Unusual Observations for TSR

Obs      TSR      Fit  SE Fit  Residual  St Resid
 38  133.639  117.369   5.421    16.269      2.12 R
 39  100.092  117.369   5.421   -17.278     -2.25 R
 41  129.927  108.570   5.421    21.357      2.79 R
 50  119.429  103.959   5.421    15.470      2.02 R
 56   52.804   70.005   5.421   -17.201     -2.24 R

R denotes an observation with a large standardized residual.
Grouping Information Using Tukey Method and 95.0% Confidence
```

Agg	N	Mean	Grouping
3	18	100.28	A
1	18	98.90	A B
2	18	91.04	B C
4	18	85.96	C

Means that do not share a letter are significantly different.

Grouping Information Using Tukey Method and 95.0% Confidence

Additive	N	Mean	Grouping
2	24	110.95	A
1	24	89.02	B
3	24	82.17	C

Means that do not share a letter are significantly different.

Grouping Information Using Tukey Method and 95.0% Confidence

Cycles	N	Mean	Grouping
1	36	94.86	A
3	36	93.23	A

Means that do not share a letter are significantly different.

Grouping Information Using Tukey Method and 95.0% Confidence

Agg	Additive	N	Mean	Grouping
1	2	6	126.51	A
3	2	6	112.97	A B
2	2	6	105.66	B
2	1	6	101.66	B C
3	1	6	101.66	B C
4	2	6	98.67	B C D
1	3	6	95.21	B C D E
3	3	6	86.21	C D E F
4	3	6	81.43	D E F G
4	1	6	77.79	E F G
1	1	6	74.99	F G
2	3	6	65.81	G

Means that do not share a letter are significantly different.

Grouping Information Using Tukey Method and 95.0% Confidence

Agg	Cycles	N	Mean	Grouping
3	1	9	104.45	A
1	1	9	99.88	A B
1	3	9	97.92	A B C
3	3	9	96.11	A B C
2	3	9	91.71	A B C
2	1	9	90.38	B C
4	3	9	87.19	B C
4	1	9	84.73	C

Means that do not share a letter are significantly different.

Grouping Information Using Tukey Method and 95.0% Confidence

Additive	Cycles	N	Mean	Grouping
2	3	12	114.46	A
2	1	12	107.45	A
1	3	12	89.78	B
3	1	12	88.87	B
1	1	12	88.27	B
3	3	12	75.46	C

Means that do not share a letter are significantly different.

Table 3 also shows that the number of freeze-thaw cycles was not significant. The interaction of cycles and additive type was significant with hydrated lime having the highest TSR values and followed by results with LAS. The WMX additive at 3 cycles produced the lowest results.

Figure 4 also shows visually that hydrated lime generally performed better with the use of granite materials in all of the 12.5 mm mixes. This is also verified with ANOVA (Table 4) in which all of the factors, as well as the interaction between these factors, were significantly important with the most significant variable being additive type. Again, Lithonia aggregate produced the highest results with an average TSR of 100.6 % while Lithia Springs and Kennesaw aggregates averaged 95.3% and 96.5 % respectively.

When comparing additive type, hydrated lime had the highest TSR with an average of 107.4%; but there was not a significant difference between aggregates treated with LAS and WMX with average TSR results of 92.3 % and 92.7 % respectively. The Tukey comparison also showed a significant difference in number of cycles with TSR results averaging 101.5 % for one freeze-thaw cycle and 93.4 % for three freeze-thaw cycles. All mixtures met the minimum requirement of 80 % TSR with the exception of the Lithia Springs aggregate treated with WMX after three freeze-thaw cycles, and it only marginally failed with an average TSR of 78.0%. GDOT allows TSR values less than 80% so long as the TSR is at least 70% and all tensile strength values exceed 100 psi. However, the Lithia Springs/WMX combination tensile strength values ranged from 83.2 to 87.3 psi.

TABLE 4: General Linear Model: TSR versus Agg, Additive, Cycles for 12.5 mm Mix.

Agg1=Lithia Springs
Agg2=Kennesaw
Agg3=Lithonia
Agg4=Dalton
Additive- 1=LAS, 2=Lime, and 3=WMX

Factor	Type	Levels	Values
Agg	fixed	3	1, 2, 3
Additive	fixed	3	1, 2, 3
Cycles	fixed	2	1, 3

Analysis of Variance for TSR, using Adjusted SS for Tests

Source	DF	Seq SS	Adj SS	Adj MS	F	P
Agg	2	278.85	278.85	139.42	6.94	0.003
Additive	2	2671.86	2671.86	1335.93	66.52	0.000
Cycles	1	879.59	879.59	879.59	43.80	0.000
Agg*Additive	4	1826.89	1826.89	456.72	22.74	0.000
Agg*Cycles	2	484.71	484.71	242.35	12.07	0.000
Additive*Cycles	2	1008.18	1008.18	504.09	25.10	0.000
Agg*Additive*Cycles	4	671.68	671.68	167.92	8.36	0.000
Error	36	723.01	723.01	20.08		
Total	53	8544.77				

S = 4.48147   R-Sq = 91.54%   R-Sq(adj) = 87.54%

Unusual Observations for TSR

Obs	TSR	Fit	SE Fit	Residual	St Resid
6	86.300	94.242	2.587	-7.941	-2.17 R
11	89.560	81.790	2.587	7.770	2.12 R
12	73.473	81.790	2.587	-8.317	-2.27 R
31	117.547	109.540	2.587	8.007	2.19 R

R denotes an observation with a large standardized residual.

Grouping Information Using Tukey Method and 95.0% Confidence

Agg	N	Mean	Grouping
3	18	100.59	A
2	18	96.50	B
1	18	95.27	B

Means that do not share a letter are significantly different.

Grouping Information Using Tukey Method and 95.0% Confidence

Additive	N	Mean	Grouping
2	18	107.40	A
3	18	92.67	B
1	18	92.29	B

Means that do not share a letter are significantly different.

Grouping Information Using Tukey Method and 95.0% Confidence

```
Cycles   N    Mean  Grouping
1       27  101.49  A
3       27   93.42    B

Means that do not share a letter are significantly different.

Grouping Information Using Tukey Method and 95.0% Confidence

Agg  Additive  N    Mean  Grouping
2    2         6  116.23  A
3    2         6  104.83    B
1    2         6  101.14    B C
3    3         6  100.82    B C
1    1         6   97.50    B C D
3    1         6   96.11      C D
2    3         6   90.02        D E
1    3         6   87.17          E
2    1         6   83.26          E

Means that do not share a letter are significantly different.

Grouping Information Using Tukey Method and 95.0% Confidence

Agg  Cycles  N    Mean  Grouping
2    1       9  103.92  A
3    1       9  100.72  A
3    3       9  100.45  A
1    1       9   99.83  A
1    3       9   90.71    B
2    3       9   89.09    B

Means that do not share a letter are significantly different.
```

The Tukey pairwise comparison shows the interaction of aggregate and cycles was only significant due to results after three cycles for both Lithia Springs and Kennesaw aggregate. Only the Lithia Springs 12.5 mm mix treated with WMX additive and subjected to three freeze-thaw cycles failed to meet the average 80% TSR requirement.

GDOT has a specification provision that the average tensile strength results for both control and conditioned specimens must be at least 60 psi. All aggregate/additive/mix type combinations met this criterion.

Comparisons were also made of actual tensile strength values for both limestone and granite 25 mm mixes as well as for 12.5 mm granite mixtures. From the summary of results in Figure 5, it is evident that the tensile strength for all three additives used in this study was lower for the limestone aggregate. Tensile strength was also consistent for all three additives regardless of the number of freeze-thaw cycles. An ANOVA of test results (Table 5) confirms what Figure 5 shows graphically, that the tensile strength of limestone 25 mm mixtures was basically unaffected by additive type or number of freeze-thaw cycles (P-value > 0.05).

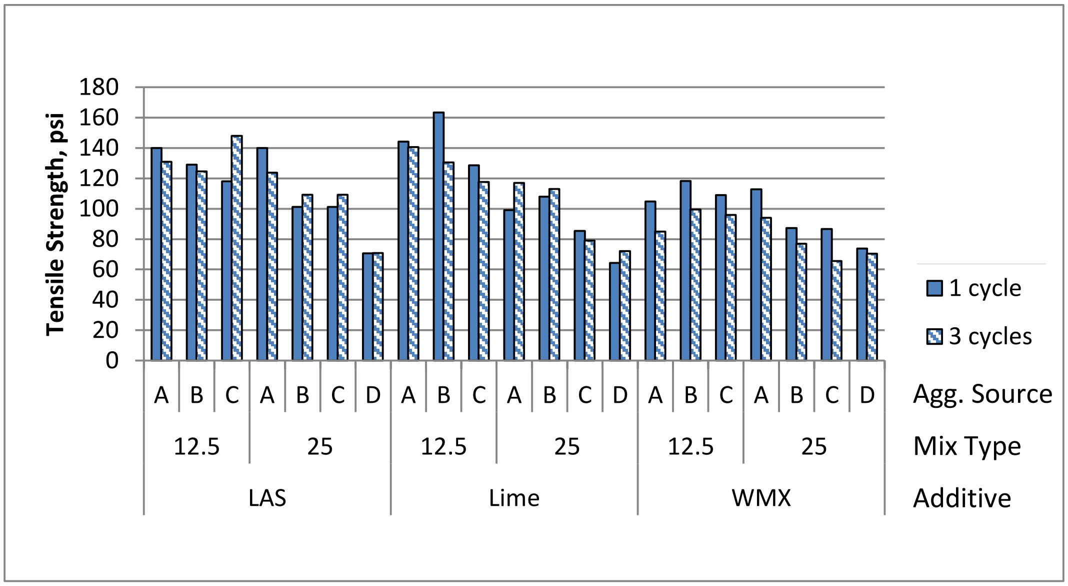

FIGURE 5: Tensile Strength Results by Aggregate Source, Mix Type, and Additive Type.

TABLE 5: General Linear Model: Tensile Strength versus Additive, Cycles for 25 mm Mix with Limestone Aggregate.

Additive- 1=LAS, 2=Lime, and 3=WMX

Factor	Type	Levels	Values
Additive	fixed	3	1, 2, 3
Cycles	fixed	2	1, 3

Analysis of Variance for Tensile Strength, using Adjusted SS for Tests

Source	DF	Seq SS	Adj SS	Adj MS	F	P
Additive	2	46.46	46.46	23.23	0.92	0.424
Cycles	1	9.53	9.53	9.53	0.38	0.550
Additive*Cycles	2	97.10	97.10	48.55	1.93	0.188
Error	12	301.75	301.75	25.15		
Total	17	454.84				

S = 5.01459   R-Sq = 33.66%   R-Sq(adj) = 6.02%

Grouping Information Using Tukey Method and 95.0% Confidence

Interestingly, when a comparison was made of all 25 mm mixes (Table 6), aggregate source and additive type as well as the interaction of those materials was significant. The results followed the expected trend that Lithia Springs aggregate produced the highest strength followed by Kennesaw, Lithonia, and Dalton.

TABLE 6: General Linear Model: Tensile Strength versus Agg, Additive, Cycles for all 25 mm Mixes.

Agg1=Lithia Springs
Agg2=Kennesaw
Agg3=Lithonia
Agg4=Dalton
Additive- 1=LAS, 2=Lime, and 3=WMX

Factor	Type	Levels	Values
Agg	fixed	4	1, 2, 3, 4
Additive	fixed	3	1, 2, 3
Cycles	fixed	2	1, 3

Analysis of Variance for Tensile Strength, using Adjusted SS for Tests

Source	DF	Seq SS	Adj SS	Adj MS	F	P
Agg	3	18772.8	18772.8	6257.6	75.67	0.000
Additive	2	4754.7	4754.7	2377.4	28.75	0.000
Cycles	1	115.0	115.0	115.0	1.39	0.244
Agg*Additive	6	3649.6	3649.6	608.3	7.36	0.000
Agg*Cycles	3	242.5	242.5	80.8	0.98	0.411
Additive*Cycles	2	1196.3	1196.3	598.1	7.23	0.002
Agg*Additive*Cycles	6	1095.7	1095.7	182.6	2.21	0.058
Error	48	3969.3	3969.3	82.7		
Total	71	33795.9				

S = 9.09356   R-Sq = 88.26%   R-Sq(adj) = 82.63%

Unusual Observations for Tensile Strength

Obs	Tensile Strength	Fit	SE Fit	Residual	St Resid	
41	94.500	78.967	5.250	15.533	2.09	R
50	129.700	112.900	5.250	16.800	2.26	R
53	109.100	93.900	5.250	15.200	2.05	R
54	78.600	93.900	5.250	-15.300	-2.06	R
56	65.900	87.367	5.250	-21.467	-2.89	R

R denotes an observation with a large standardized residual.

Grouping Information Using Tukey Method and 95.0% Confidence

Agg	N	Mean	Grouping
1	18	114.44	A
2	18	99.27	B
3	18	87.83	C
4	18	70.26	D

Means that do not share a letter are significantly different.

Grouping Information Using Tukey Method and 95.0% Confidence

Additive	N	Mean	Grouping
1	24	103.26	A
2	24	92.20	B
3	24	83.40	C

```
Means that do not share a letter are significantly different.

Grouping Information Using Tukey Method and 95.0% Confidence

Cycles   N   Mean  Grouping
1       36  94.21  A
3       36  91.69  A

Means that do not share a letter are significantly different.
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The Tukey grouping also shows that the LAS treatment resulted in the highest strengths, followed by hydrated lime and WMX. The analysis also shows that tensile strength was not significantly affected by the number of freeze-thaw cycles.

Additional Freeze-Thaw Cycles

The scope of the project was modified during the course of the research and additional lab testing was performed. Two aggregate sources were tested with the three additives used in earlier testing and tensile strength testing was performed after 1, 5, and 10 freeze-thaw cycles using GDT 66.

Evaluating the effect of an even greater number of freeze-thaw cycles on aggregate and anti-strip combinations was desired based on recent work by Sebaaly, et al, (27) that advocated additional freeze-thaw cycles (up to 15 cycles) to produce discriminating results in regard to moisture susceptibility. Therefore, the Lithia Springs and Lithonia aggregate mixtures were selected for further study. These two aggregates were chosen since the earlier lab study of this project indicated Lithonia 12.5 mm mix performed best and Lithia Springs aggregate had the lowest TSR results and those results were contrary to anticipated performance. For each of these combinations, a set of 12.5 mm specimens was tested using 1, 5, and 10 freeze-thaw cycles. Three replicates were tested per data point.

Table 7 lists a summary of the TSR results for this portion of the project. Included are the average conditioned and unconditioned splitting tensile strengths for each combination of aggregate, anti-strip, and number of freeze-thaw cycles. A TSR value was then calculated from these averages. The "Pass/Fail" column indicates whether each set passed or failed the Georgia TSR criterion. The hydrated lime had both the highest tensile strength and the highest TSR values and was the only additive treatment to meet 80% TSR for all freeze-thaw cycle combinations. In fact, the tensile strength of conditioned specimens treated with hydrated lime after five and ten freeze-thaw cycles was even higher than the unconditioned strengths for both the Lithia Springs and Lithonia aggregate. Mix treated with LAS failed to maintain at least 80% TSR after five cycles for both Lithia Springs and Lithonia aggregates and after ten cycles with Lithonia Aggregate. The WMX mixes failed after both five and ten freeze-thaw cycles for each aggregate source and failed even after only one cycle with Lithonia aggregate.

Table 8 tabulates the data from all the individual samples that were tested during this project. Figures 6 and 7 plot the TSR versus the number of freeze-thaw cycles applied for the Lithia Springs and Lithonia aggregate, respectively. Figures 8 and 9 show the average and standard deviation of the splitting tensile strength versus the number of freeze-thaw cycles for the Lithia Springs and Lithonia aggregates, respectively.

TABLE 7: Summary of Average TSR Results.

Aggregate	Anti-strip	F/T Cycles	Conditioned Tensile Strength (psi)	Unconditioned Tensile Strength (psi)	TSR, %	Pass/ Fail
Lithia Springs	HL	1	161.2	124.7	129.2	Pass
Lithia Springs	HL	5	120.4	124.7	96.5	Pass
Lithia Springs	HL	10	140.8	124.7	112.9	Pass
Lithia Springs	LAS	1	116.7	118.3	98.6	Pass
Lithia Springs	LAS	5	93.4	118.3	79.0	Fail
Lithia Springs	LAS	10	99.9	118.3	84.5	Pass
Lithia Springs	WMX	1	135.9	121.3	112.0	Pass
Lithia Springs	WMX	5	90.8	121.3	74.9	Fail
Lithia Springs	WMX	10	82.3	121.3	67.8	Fail
Lithonia	HL	1	120.9	122.1	99.1	Pass
Lithonia	HL	5	122.9	122.1	100.7	Pass
Lithonia	HL	10	132.3	122.1	108.4	Pass
Lithonia	LAS	1	103.1	116.0	88.9	Pass
Lithonia	LAS	5	87.5	116.0	75.4	Fail
Lithonia	LAS	10	87.4	116.0	75.3	Fail
Lithonia	WMX	1	94.4	120.6	78.3	Fail
Lithonia	WMX	5	72.5	120.6	60.1	Fail
Lithonia	WMX	10	67.2	120.6	55.7	Fail

TABLE 8: Individual Values – TSR Testing.

Aggregate Type	Anti-strip	Sample ID	Sample Air Voids	F/T Cycles	Saturation Level (%)	Splitting Tensile Strength(psi)
Lithia Springs	HL	6	6.8	0	N_A	132.5
Lithia Springs	HL	7	6.5	0	N_A	121.8
Lithia Springs	HL	12	6.6	0	N_A	119.8
Lithia Springs	HL	15	7.1	1	82.2	185.8
Lithia Springs	HL	16	7.1	1	82.8	149.2
Lithia Springs	HL	17	7.1	1	81.0	148.4
Lithia Springs	HL	2	7.2	5	80.3	116.9
Lithia Springs	HL	3	6.8	5	80.0	121.2
Lithia Springs	HL	14	6.6	5	82.4	123.0
Lithia Springs	HL	1	6.2	10	79.4	157.8
Lithia Springs	HL	8	6.9	10	78.4	133.7

Aggregate Type	Anti-strip	Sample ID	Sample Air Voids	F/T Cycles	Saturation Level (%)	Splitting Tensile Strength(psi)
Lithia Springs	HL	13	6.7	10	81.1	131.1
Lithia Springs	LAS	106	7.5	0	N_A	118.8
Lithia Springs	LAS	109	7.0	0	N_A	113.9
Lithia Springs	LAS	112	7.0	0	N_A	122.2
Lithia Springs	LAS	104	7.3	1	85.3	117.8
Lithia Springs	LAS	105	7.2	1	80.5	112.9
Lithia Springs	LAS	110	6.8	1	80.9	119.3
Lithia Springs	LAS	101	7.1	5	83.7	100.1
Lithia Springs	LAS	102	6.8	5	84.4	94.6
Lithia Springs	LAS	103	7.3	5	83.3	85.6
Lithia Springs	LAS	107	6.7	10	80.6	98.9
Lithia Springs	LAS	108	7.1	10	83.9	102.9
Lithia Springs	LAS	111	7.2	10	83.7	97.8
Lithia Springs	WMX	205	7.3	0	N_A	124.2
Lithia Springs	WMX	208	6.8	0	N_A	111.9
Lithia Springs	WMX	210	7.1	0	N_A	127.8
Lithia Springs	WMX	215	7.2	1	83.7	137.1
Lithia Springs	WMX	217	7.4	1	85.3	146.7
Lithia Springs	WMX	218	7.2	1	86.0	123.8
Lithia Springs	WMX	202	6.9	5	81.7	87.2
Lithia Springs	WMX	207	7.4	5	82.9	91.0
Lithia Springs	WMX	213	7.6	5	90.6	94.3
Lithia Springs	WMX	201	6.1	10	78.1	73.1
Lithia Springs	WMX	204	7.4	10	83.4	88.7
Lithia Springs	WMX	206	7.1	10	82.3	85.1
Lithonia	HL	405	6.9	0	N_A	119.0
Lithonia	HL	408	6.9	0	N_A	123.9
Lithonia	HL	411	7.3	0	N_A	123.3
Lithonia	HL	402	6.8	1	82.7	126.0
Lithonia	HL	409	7.0	1	81.6	121.6
Lithonia	HL	414	7.3	1	81.9	115.2
Lithonia	HL	403	6.6	5	82.9	122.2
Lithonia	HL	410	7.2	5	83.5	122.8
Lithonia	HL	413	7.2	5	82.0	123.6
Lithonia	HL	404	7.2	10	82.5	140.6
Lithonia	HL	406	7.1	10	81.4	136.9
Lithonia	HL	412	6.8	10	79.9	119.4
Lithonia	LAS	513	7.9	0	N_A	109.8
Lithonia	LAS	514	7.2	0	N_A	122.9
Lithonia	LAS	520	7.4	0	N_A	115.4
Lithonia	LAS	504	6.9	1	91.4	113.1
Lithonia	LAS	516	7.7	1	86.8	100.4
Lithonia	LAS	517	7.5	1	85.7	95.8

Aggregate Type	Anti-strip	Sample ID	Sample Air Voids	F/T Cycles	Saturation Level (%)	Splitting Tensile Strength(psi)
Lithonia	LAS	502	7.7	5	87.9	105.0
Lithonia	LAS	515	6.9	5	82.3	93.7
Lithonia	LAS	518	7.6	5	83.0	63.8
Lithonia	LAS	510	7.6	10	82.6	93.2
Lithonia	LAS	511	7.6	10	84.8	93.3
Lithonia	LAS	519	7.3	10	85.3	75.6
Lithonia	WMX	604	7.5	0	N_A	121.7
Lithonia	WMX	610	7.5	0	N_A	123.6
Lithonia	WMX	611	7.8	0	N_A	116.6
Lithonia	WMX	601	7.7	1	85.1	98.2
Lithonia	WMX	605	7.4	1	86.8	89.5
Lithonia	WMX	608	7.6	1	86.1	95.6
Lithonia	WMX	603	7.3	5	82.7	66.3
Lithonia	WMX	606	7.6	5	85.0	64.1
Lithonia	WMX	612	7.7	5	86.5	87.0
Lithonia	WMX	602	7.1	10	91.2	52.5
Lithonia	WMX	607	7.6	10	88.4	62.6
Lithonia	WMX	609	7.2	10	85.5	86.6

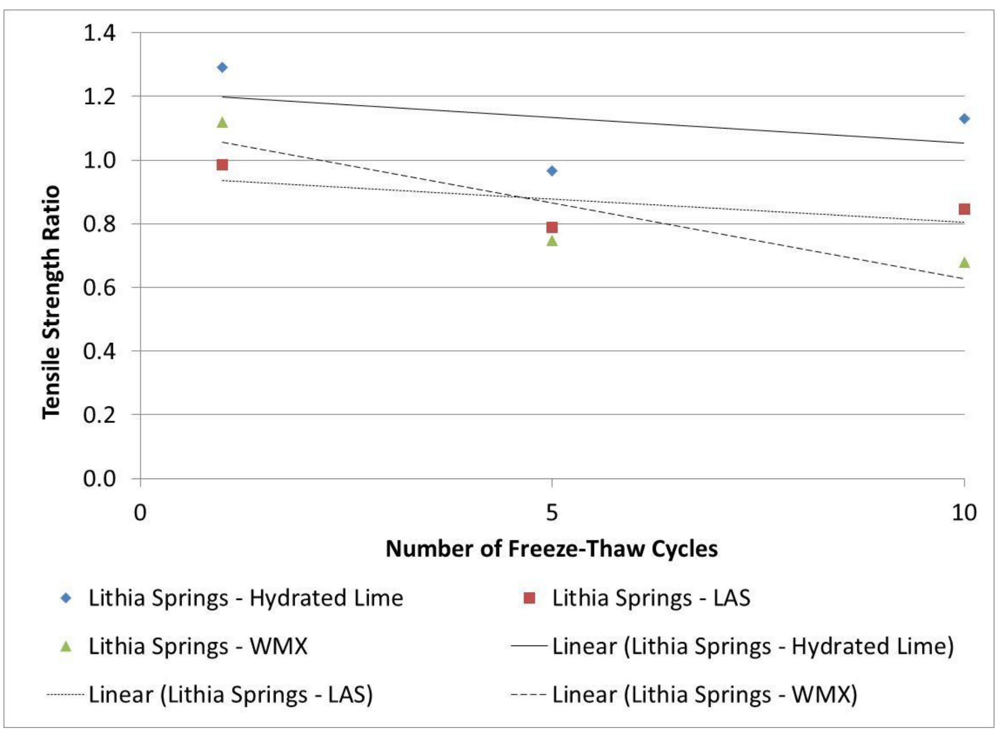

FIGURE 6: TSR versus Number of Freeze-Thaw Cycles – Lithia Springs Aggregate.

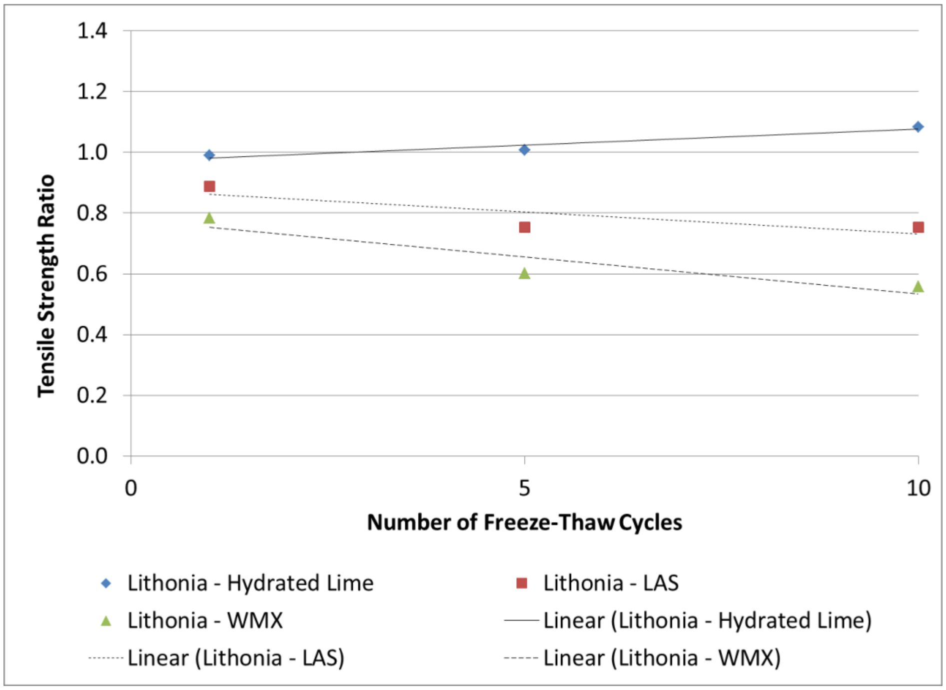

FIGURE 7: TSR versus Number of Freeze-Thaw Cycles – Lithonia Aggregate.

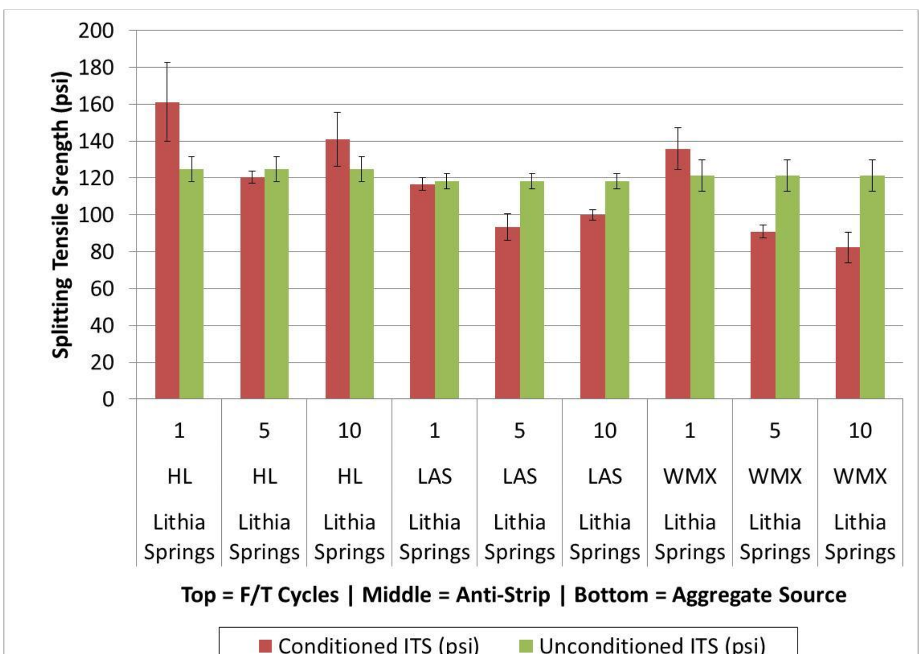

FIGURE 8: Splitting Tensile Strengths versus Number of Freeze-Thaw Cycles – Lithia Springs Aggregate.

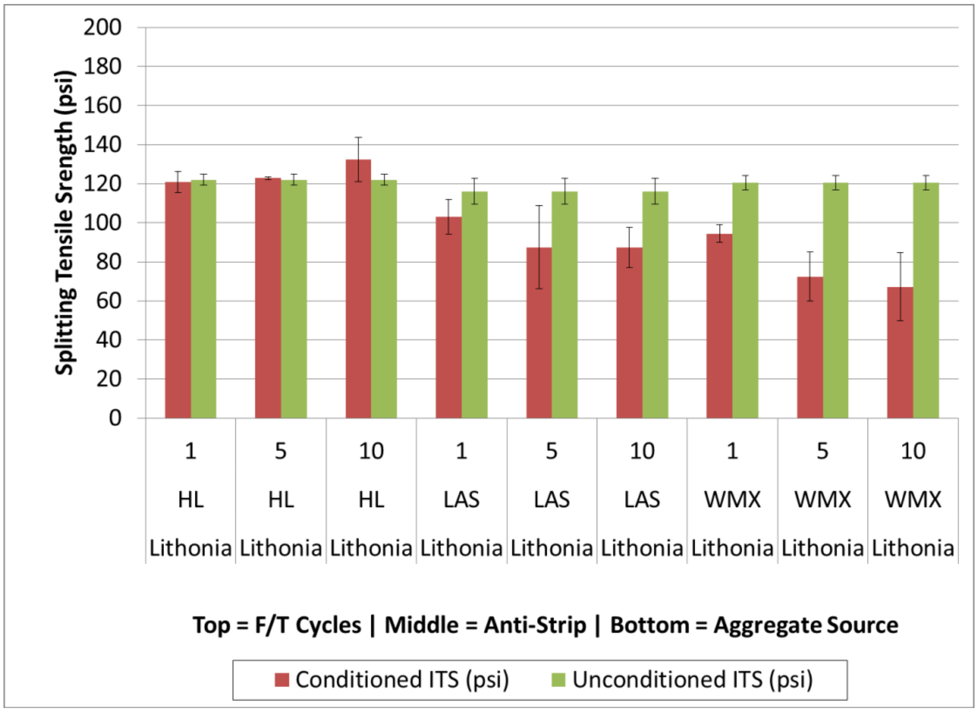

FIGURE 9: Splitting Tensile Strengths versus Number of Freeze-Thaw Cycles – Lithonia Aggregate.

Based on a review of the data, the following conclusions are drawn:

- The Lithia Springs aggregate with hydrated lime appeared to have an atypically high TSR value (129.2%). Inspection of the individual data points in Table 8 indicates this was due to one positive outlier. After removal of the outlier, TSR results were still 119.0%.
- The unconditioned splitting tensile strengths did not appear to vary much (typically around 120 psi) regardless of the aggregate source and anti-stripping agent utilized. An ANOVA (p-value = 0.55 > α = 0.05) confirmed this observation.
- A total of 18 sets of TSR values (one set = average of three samples) were calculated (2 aggregate sources x 3 antistripping agents x 3 freeze-thaw periods).
    - Of these TSR values, 8 sets failed the Georgia TSR criterion. Three sets contained the Lithia Springs aggregate and 5 sets contained the Lithonia aggregate.
    - None of the sets that failed TSR contained Hydrated Lime.
    - Only one set having less than 5 freeze-thaw cycles failed the TSR criterion (Lithonia WMX with one freeze-thaw cycle). This mixture had a TSR greater than 0.7, but did not have individual splitting tensile strengths above 100 psi.
    - All sets containing WMX with 5 or more freeze-thaw cycles failed the Georgia TSR criterion.
    - Three of the mixtures containing LAS failed the TSR criterion. These mixtures had a TSR greater than 0.7, but did not have individual splitting tensile strengths above 100 psi.

- In general, as additional freeze-thaw cycles were added to sample conditioning, the samples with hydrated lime saw little or no reduction in TSR. The samples with LAS outperformed the samples with WMX but did not perform as well as the samples with Hydrated Lime as additional freeze-thaw cycles were added to the material. Figures 6 and 7 illustrate this trend.

A General Linear Model (GLM) ($\alpha = 0.05$) statistical analysis was performed in MINITAB® to determine the relevant statistical factors impacting the overall TSR dataset. A Tukey-Kramer statistical analysis was performed within the GLM to determine which variables were statistically similar or different with the dataset as well. A summary of this analysis is shown in Table 9 with the complete analysis attached in APPENDIX A.

TABLE 9: General Linear Model: TS versus Additives, Cycles, Agg.

```
Factor     Type   Levels  Values
Additives  fixed       3  HL, LAS, WMX
Cycles     fixed       4  0, 1, 5, 10
Agg        fixed       2  LS, Lithonia

Analysis of Variance for TS, using Adjusted SS for Tests

Source                DF   Seq SS   Adj SS  Adj MS      F      P
Additives              2  14844.7  14844.7  7422.4  72.82  0.000
Cycles                 3   8452.8   8452.8  2817.6  27.65  0.000
Agg                    1   3149.5   3149.5  3149.5  30.90  0.000
Additives*Cycles       6   5939.3   5939.3   989.9   9.71  0.000
Additives*Agg          2    327.7    327.7   163.9   1.61  0.211
Cycles*Agg             3   2290.2   2290.2   763.4   7.49  0.000
Additives*Cycles*Agg   6    782.8    782.8   130.5   1.28  0.284
Error                 48   4892.2   4892.2   101.9
Total                 71  40679.3

S = 10.0956   R-Sq = 87.97%   R-Sq(adj) = 82.21%

Grouping Information Using Tukey Method and 95.0% Confidence

Additives   N    Mean  Grouping
HL         24  130.65  A
LAS        24  102.79    B
WMX        24   98.13    B

Cycles   N    Mean  Grouping
 1      18  122.03  A
 0      18  120.51  A
10      18  101.66    B
 5      18   97.91    B

Agg         N    Mean  Grouping
LS         36  117.14  A
Lithonia   36  103.91    B

Means that do not share a letter are significantly different.
```

Based on the results of the statistical analysis, the following conclusions were drawn:

- The type of anti-strip additive, aggregate source, and number of freeze-thaw cycles were statistically significant in the context of the full dataset. The interactions between anti-strip and number of freeze thaw cycles as well as the interactions between number of freeze-thaw cycles and aggregate source were significant as well.

- The samples containing Hydrated Lime had a statistically higher splitting tensile strength than the samples containing LAS and WMX. The difference in the data sets was approximately 30 psi.
- Samples undergoing 0 and 1 freeze-thaw cycles had a statistically higher splitting tensile strength than samples undergoing 5 and 10 freeze-thaw cycles. The average difference in the strengths of the two groupings is about 20 psi. This result is intuitive in the sense that more freeze-thaw cycles should cause more damage to the test specimens. However, it should be noted that additional freeze-thaw cycles beyond 5 freeze-thaw cycles did not cause a significant reduction in sample splitting tensile strength.
- Both five and ten freeze-thaw cycles were significantly more discriminating in regard to moisture susceptibility than one freeze-thaw cycle alone.
- The samples with the Lithia Springs aggregate had a statistically higher splitting tensile strength than the samples with the Lithonia aggregate. The average difference in the two data sets was approximately 14 psi.

CHAPTER 4: PROJECT REVIEW

Project Selection

Projects for the field evaluations were selected and information obtained with the assistance of the Georgia Department of Transportation. Paired projects for hydrated lime and for liquid anti-strip treatment with surface mixes of the same nominal maximum aggregate size, the same aggregate source, similar traffic conditions and age were selected. An extensive search of project records was needed to determine the best project comparisons. One of the difficulties in accomplishing this selection was that the project files are often archived soon after the project is completed, and those files had to be retrieved from archival storage. In order to accomplish this, the exact location of those files in the archive had to be known. Once project files were retrieved, it was found that many times contractors used hydrated lime in mixtures even for off-system projects. Since hydrated lime is required for state route projects, contractors often used hydrated lime in all mixes rather than switching back and forth. Such comparisons also proved to be difficult since traffic data is often not available for off-system projects and off-system projects generally have lower traffic volumes than on-system routes. The search for comparable projects was further complicated by requiring that the surface layer be a 12.5 mm NMAS mix. This was done to ensure that the surface layer had sufficient thickness for tensile strength testing. However, 9.5 mm mix is the most prevalent mixture on local road systems. Any known differences in the project variables will be considered in the analysis as much as possible. Ten state route projects and 10 off-system projects were used for this comparison. Project locations are identified in Figure 10.

Testing for Resistance to Stripping

For each project selected, the sites were visited and cores were taken and evaluated for any evidence of moisture damage according to GDT-66 (*32*). Cores were also evaluated for in-place density to determine if density may be a factor in performance results. For example, the density of mixtures on the state route system may be higher due to more stringent air void limitations.

Tables 10 and 11 provide the tensile strength results for the projects after initial coring. Since the mixtures had been in place for several years and had likely gone through several freeze-thaw cycles, no additional freeze-thaw testing was performed and cores were vacuum saturated and placed directly into a hot water bath at 140^{o}F as described in GDT-66. Those results showed little difference in performance between the liquid additive projects and those treated with hydrated lime after an average service life of more than 8 years. In order to provide more discriminating results for distress from freeze-thaw conditions, additional cores were taken from 5 projects of each treatment and were tested after one and three freeze-thaw cycles. For the LAS projects, tensile strength dropped significantly after both 5 and 10 freeze-thaw cycles. For the lime projects, results decreased after one cycle but not significantly at 10 cycles.

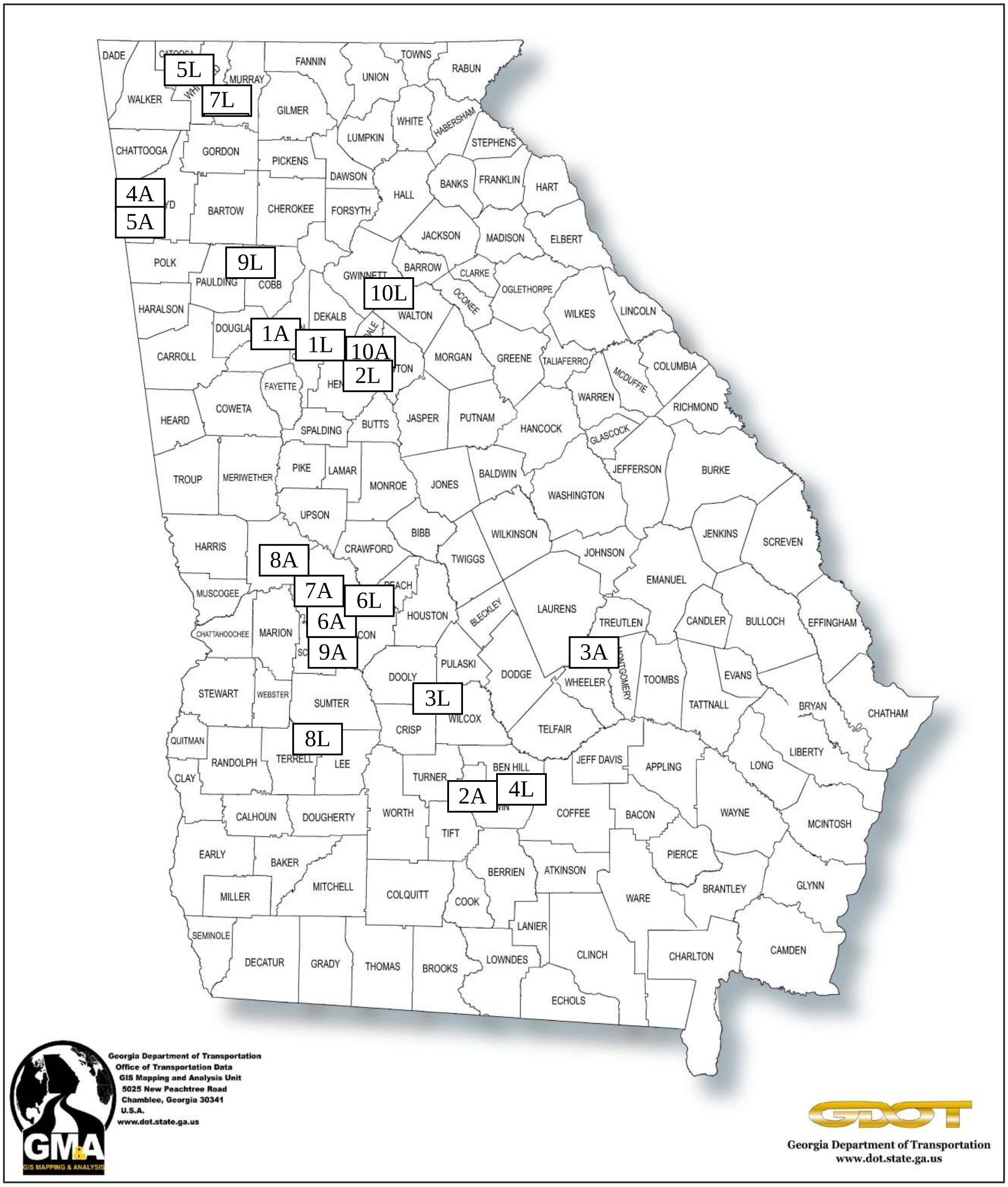

FIGURE 10: Location of Selected Field Projects.

TABLE 10: Summary of Tensile Strength Results from Liquid Additive Projects.

Project	Location	County	Age, yrs.	Avg. Tensile Strength, psi (No freeze-thaw)	Average Tensile Strength, psi (1 freeze-thaw)	Average Tensile Strength, psi (3 freeze-thaw)
1A	Forest Park Rd near S. River Ind. Blvd.	Fulton	8	221.4		
2A	CR 228 beginning at SR 35	Irwin	8	198.7	136	102.800
3A	CR 13 near Higgston	Montgomery	9	241.7	145.1	68.100
4A	Nat. Guard Armory Rd. at SR 1	Floyd	7	193.8	126.5	93.600
5A	Garden Lakes Pkwy extension at Woods Rd.	Floyd	6	191.5	130.7	141.600
6A	Old VFW Rd. at SR 49 south of Oglethorpe	Macon	10	135.2		
7A	Access to Industrial Park from CR 258	Taylor	8	198.3		
8A	Industrial Park Rd. beginning at SR 96	Taylor	8	200.7		
9A	Griffin Rd. at SR 90 north of Oglethorpe	Macon	10	121.7	100.4	50.500
10A	City Pond Rd. at Alcovy Rd. in Covington	Newton	8	122.5		
	Avg. Additive Projects		8.2	182.5	127.7	91.3
	Standard Deviation			41.6	16.8	34.9

TABLE 11: Summary of Tensile Strength Results from Hydrated Lime Projects.

Project	Location	County	Age, yrs.	Avg. Tensile Strength, psi (No freeze-thaw)	Average Tensile Strength, psi (1 freeze-thaw)	Average Tensile Strength, psi (3 freeze-thaw)
1L	Cascade Rd.east of I-285	Fulton	8	166.5	137.4	126.4
2L	Hazelbrand Rd. beginning at US 278	Newton	9	134		
3L	Pine St. at Bowen St. in Abbeville	Wilcox	8	95.6		
4L	SR 32 from Mystic to Douglas	Irwin/Coffee	9	249.8	158	116.9
5L	41 Connector south of SR 71 in Dalton	Whitfield	11	108.6		
6L	SR 49 at Thomas to Randolph St. in Oglethorpe	Macon	9	245.2	170.3	173.6
7L	Manley Rd. at Coniston Rd near US 411	Murray	8	210.3	174.9	170.0
8L	Helen St near Stocks Dairy Rd north of Albany	Lee	9	160.4		
9L	Shadowwood Pkwy at Powers Ferry Rd.	Cobb	6	159.5	94.3	89.1
10L	SR 81 at Youth Jersey Rd. south of Loganville	Walton	7	193.8		
	Avg. Lime Projects		8.4	172.4	147.0	135.2
	Standard Deviation			52.7	32.8	36.1

An ANOVA was performed using the General Linear Model to evaluate the significance of treatment on test results when no freeze-thaw cycle was used. The analysis showed that treatment type was most significant and accounted for the largest portion of the test variability. However, as shown in the Tukey comparison, there was no significant difference between liquid additive and lime treatment for the large majority of projects (Table 12).

TABLE 12: General Linear Model: TS versus Project, Treat.

Factor	Type	Levels	Values
Project	fixed	10	1, 2, 3, 4, 5, 6, 7, 8, 9, 10
Additive	fixed	20	HL, LAS, HL, LAS, HL, LAS, HL, LAS, HL, LAS, HL, LAS, HL, LAS, HL, LAS, HL, LAS, HL, LAS

Analysis of Variance for TS, using Adjusted SS for Tests

Source	DF	Seq SS	Adj SS	Adj MS	F	P
Project	9	34872	34872	3875	2.18	0.044
Additive	10	88390	88390	8839	4.98	0.000
Error	40	70956	70956	1774		
Total	59	194219				

S = 42.1178   R-Sq = 63.47%   R-Sq(adj) = 46.11%

Grouping Information Using Tukey Method and 95.0% Confidence

Project	Treat	N	Mean	Grouping
4	HL	3	249.81	A
6	HL	3	245.17	A
3	LAS	3	241.69	A
1	LAS	3	221.43	A B
7	HL	3	210.27	A B
8	LAS	3	200.66	A B
2	LAS	3	198.70	A B
7	LAS	3	198.30	A B
4	LAS	3	193.78	A B
10	HL	3	193.76	A B
5	LAS	3	191.50	A B
1	HL	3	166.55	A B
8	HL	3	160.44	A B
9	HL	3	159.53	A B
6	LAS	3	135.16	A B
2	HL	3	133.96	A B
10	LAS	3	122.53	A B
9	LAS	3	121.74	A B
5	HL	3	108.56	B
3	HL	3	95.63	B

Means that do not share a letter are significantly different.

Figure 11 shows a comparison of test results from project cores after 0, 1, and 3 freeze-thaw cycles. The results show that subjecting roadway cores to freeze-thaw conditions is more severe than vacuum saturation alone. From the results, the average tensile strength of liquid additive projects was reduced by 50% when comparing results after 3 freeze-thaw cycles to no freeze-thaw cycles. In contrast, the hydrated lime average tensile strength was reduced by 22% from no freeze-thaw to 3 freeze-thaw cycles. A comparison after one and three freeze-thaw cycles indicates the liquid additive mixtures had reduced strength of 29% and the hydrated lime mixtures had a reduced strength of 8%. After 3 freeze-thaw cycles, the cores treated with hydrated lime had 50 percent higher tensile strength than the cores treated with liquid additive.

An ANOVA of the test results (Table 13) for 0, 1, and 3 freeze-thaw cycles shows that the type treatment was not statistically significant at a 95% confidence level (where *P*-value = 0.05).

However, the P-value of 0.0506 is very close to 0.05 and does indicate that there is a practical difference in the results between use of liquid additive and hydrated lime and the difference is almost statistically significant.

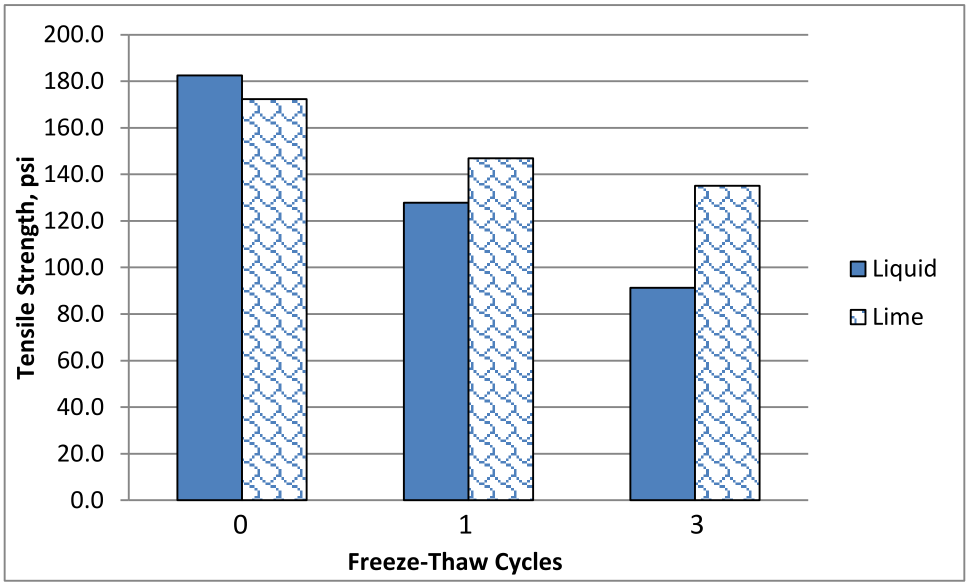

FIGURE 11: Project Core Tensile Strength After Multiple Freeze-Thaw Cycles.

TABLE 13: General Linear Model: TS versus Treatment, Cycles.

```
Factor  Type   Levels  Values
Treat   fixed       2  LAS, Lime
Cycles  fixed       3  0, 1, 3

Analysis of Variance for TS, using Adjusted SS for Tests

Source        DF  Seq SS  Adj SS  Adj MS      F      P
Treat          1    5320    5320    5320   4.23  0.051
Cycles         2   38005   38005   19003  15.13  0.000
Treat*Cycles   2    1123    1123     562   0.45  0.645
Error         24   30151   30151    1256
Total         29   74599

S = 35.4442   R-Sq = 59.58%   R-Sq(adj) = 51.16%

Unusual Observations for TS

Obs       TS      Fit  SE Fit  Residual  St Resid
  5  121.741  189.483  15.851   -67.742     -2.14 R

R denotes an observation with a large standardized residual.

Grouping Information Using Tukey Method and 95.0% Confidence

Treat   N   Mean  Grouping
```

```
Lime   15  162.8  A
LAS    15  136.2  A

Means that do not share a letter are significantly different.


Tukey Simultaneous Tests
Response Variable TS
All Pairwise Comparisons among Levels of Treat
Treat = LAS  subtracted from:

       Difference       SE of           Adjusted
Treat    of Means  Difference  T-Value   P-Value
Lime        26.63       12.94    2.058    0.0506

Grouping Information Using Tukey Method and 95.0% Confidence

Cycles   N   Mean  Grouping
0       10  197.9  A
1       10  137.4    B
3       10  113.3    B

Means that do not share a letter are significantly different.

Tukey Simultaneous Tests
Response Variable TS
All Pairwise Comparisons among Levels of Cycles
Cycles = 0  subtracted from:

        Difference       SE of           Adjusted
Cycles    of Means  Difference  T-Value   P-Value
1           -60.51       15.85   -3.818    0.0023
3           -84.61       15.85   -5.338    0.0001

Cycles = 1  subtracted from:

        Difference       SE of           Adjusted
Cycles    of Means  Difference  T-Value   P-Value
3           -24.10       15.85   -1.520    0.2994
```

The Tukey pair-wise comparison between number of freeze-thaw cycles, shows a statistically significant difference exists between no freeze-thaw cycles and both the 1 and 3 freeze-thaw cycles. There was not, however, a statistically significant difference between 1 and 3 freeze-thaw cycles, although there is an observable difference as shown in Figure 11. These results indicate that including freeze-thaw cycles on aged pavement cores will better discriminate between treatments. However, the study does not show that additional freeze-thaw cycles have a direct correlation with field performance.

Hamburg Testing

Additional moisture susceptibility testing was performed on project cores averaging 8 years of age with the Hamburg Wheel Tracker. Hamburg wheel-track (Hamburg) testing, shown in Figure 12, was performed to determine both the rutting and stripping susceptibility of the mixtures tested for this project. Testing was performed in accordance with AASHTO T 324-04 with the exception that half of the cores were first vacuum saturated for 30 minutes and subjected to one freeze-thaw cycle, at the request of GDOT, to evaluate performance over more severe conditions than typical Hamburg testing.

FIGURE 12: Hamburg Wheel-Tracking Device.

The specimens were tested under a 158 ± 1 lbs wheel load for 10,000 cycles (20,000 passes) while submerged in a water bath which was maintained at a temperature of 50°C (122°F). While being tested, rut depths were measured by an LVDT which recorded the relative vertical position of the load wheel after each load cycle. After testing, these data were used to determine the point at which stripping occurred in the mixture under loading and the relative rutting susceptibility of those mixtures. Testing would normally be terminated after the samples reached a total rut depth of 0.5 inches (12.5 mm) or after 20,000 passes, whichever occurred first.

Figure 13 illustrates typical data output from the Hamburg device. 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 of the mixture. The stripping inflection point can give an indication of the moisture susceptibility of that particular mixture. A stripping inflection point of greater than 5,000 cycles (10,000 passes) has been shown to be a good indicator of a moisture-resistant mix (*33*).

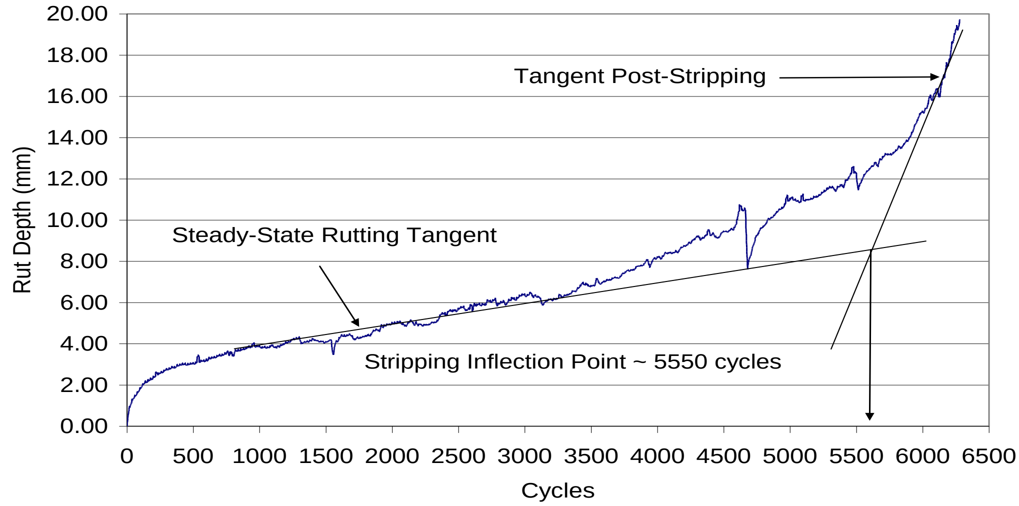

FIGURE 13: Example of Hamburg Data Analysis

The data were also analyzed to determine the number of cycles at which the mixture reached 0.5 inches (12.5 mm) of rutting. This gives an indicator of the relative rutting susceptibility of these mixtures. The State of Texas has a set of minimum requirements for the number of passes a mixture can last in the Hamburg test until that mixture ruts 0.5 inches (*34*). For these requirements, the minimum number of passes varies based on the high PG grade of the mix binder. These requirements are listed in Table 15.

TABLE 14: Texas Hamburg Requirements *(34).*

High PG Grade	**Minimum Number of Passes at 0.5 inch Rut Depth – Tested at 122°F**
64 or lower	10,000
70	15,000
76 or higher	20,000

Tables 16 and 17 show the average rutting versus number of cycles for the individual Hamburg tests performed for this project as well as the stripping inflection point. The data shows the number of cycles at which each sample reached 0.5 inches of rutting. Where N/A is shown, it means the samples never reached 0.5 inches of rutting.

SIP values for two of the five additive projects fell below the 5,000 cycle (10,000 passes) threshold that is desired for a moisture resistant mix with a PG 64-XX binder while samples from all of the lime-treated projects met that criteria.

TABLE 15: Hamburg Results from LAS Projects.

Sample ID	Rutting @ 20,000 Passes (inches)	Number of Passes to 0.5" Rut Depth	Stripping Inflection Point (passes)	Acceptable SIP?
Conway-McDonald - 1 & 3	0.39	N/A	20,000 +	Yes
Conway-McDonald - 2 & 4	0.41	N/A	16,000	Yes
Pleasure Lakes - 1 & 4	0.78	16,952	6400	No
Pleasure Lakes - 2 & 3	0.70	17,672	6425	No
National Guard Armory Road - 1 & 4	0.33	N/A	20,000 +	Yes
National Guard Armory Road - 2 & 3	0.30	N/A	20,000 +	Yes
Garden Lakes Parkway Extension - 1 & 2	0.30	N/A	20,000 +	Yes
Garden Lakes Parkway Extension - 3 & 4	0.20	N/A	16,000	Yes
Griffin Road - 1 & 4	4.62	9,698	9300	No
Griffin Road - 2 & 3	3.28	6,832	9800	No

TABLE 16: Hamburg Results from Hydrated Lime Projects.

Sample ID	Rutting @ 20,000 Passes (inches)	Number of Passes to 0.5" Rut Depth	Stripping Inflection Point (passes)	Acceptable SIP?
SR 49 / SR 90 - 1 & 2	0.62	6394	20,000 +	Yes
SR 49 / SR 90 - 3 & 4	0.14	N/A	20,000 +	Yes
SR 32 - 1 & 2	0.39	N/A	20,000 +	Yes
SR 32 - 3 & 4	0.40	N/A	20,000 +	Yes
Cascade RD – 5 & 6	0.17	N/A	20,000 +	Yes
Cascade RD – 1 & 2	0.21	N/A	20,000 +	Yes
Cascade RD – Between Wheelpath – 3 & 4 - No F/T	0.21	N/A	20,000 +	Yes
Cascade RD – Between Wheelpath – 7 & 8 - No F/T	0.16	N/A	20,000 +	Yes
Manley Road - 1 & 3	0.29	N/A	12,700	Yes
Manley Road - 2 & 4	0.28	N/A	20,000+	Yes
Shadowwood Pkwy – 5 & 6	0.62	12,444	20,000 +	Yes
Shadowwood Pkwy – 7 & 8*	NA	7,468	15,000 +	Yes
Shadowwood Pkwy – 1 & 3 - No F/T	0.50	N/A	20,000 +	Yes
Shadowwood Pkwy – 2 & 4 - No F/T	0.43	N/A	20,000 +	Yes

*Did not complete 20,000 passes.

If materials fail to meet Hamburg requirements, the asphalt mixture may be improved in the following ways (*35*):

- Use hydrated lime or liquid antistrip if it is not already being used.
- Use a different PG binder. Sometimes PG binders of the same grade, but different source can make a difference.
- Use a different source of aggregate with higher quality

CHAPTER 5: FIELD TEST SECTIONS

Test sections were placed on SR 319 in Tift County in order to evaluate three additives under actual field conditions. Hydrated lime was used as the standard anti-strip agent for control purposes. A liquid anti-strip additive from a supplier on the GDOT Qualified Products List (QPL-26) was selected for comparison since the additive is commonly used on off-system projects. The QPL list is available on the GDOT website (*31*). A third additive was a recently developed product that is publicized as being suitable for both an anti-strip additive as well as a Warm Mix additive. This additive is referred to as WMX for this study. The sites were monitored for two years after construction. Cores from each site were taken after initial construction and after two years to determine performance trends for each material.

Dynamic modulus was also performed on samples by the normal dynamic modulus procedure, AASHTO PP 60, and after one freeze-thaw cycle. Samples were tested at 4, 20, and 40°C and at the frequencies recommended in AASHTO PP61. The Flow Number test (AASHTO TP 79) was then run on the dynamic modulus samples.

The project (Figure 14) was a mill/overlay project just south of Tifton, and the route carries 9,340 vehicles per day. The asphalt mixture was a 12.5 mm Superpave mix which had 24% RAP and used virgin aggregate from the USA quarry at Postell, Ga. The mixture was produced from the plant

FIGURE 14: Construction of SR 319 Test Sections.

of Reeves Construction Company in Chula, GA. The liquid additive was added at the rate of 0.5% by weight of asphalt cement and was pre-blended at Reeves' asphalt terminal in Perry Ga. The additive was added at a dosage rate of 2% by weight of asphalt cement. The WMX additive was added to the tanker at the Perry terminal and material was circulated overnight in a dedicated storage tank at the plant facility. The mix was produced at temperatures between 260-290°F in an effort to take advantage of the warm mix properties of the additive.

During construction, samples of loose plant mix were taken and transported to the NCAT laboratory for further testing. Immediately after construction, roadway cores of in-place material were also taken for short-term analysis of strength. The sampling and testing plan for the roadway test sections was as follows:

- Saw cores and save top layer (about 1.5"); determine bulk specific gravity.
- Group in sets of 3 control and 3 conditioned for TSR testing using GDT 166 procedure (30 minute vacuum saturation, freeze-thaw, and 0.065 in./min. loading rate).
- From the buckets of material, prepare 6 specimens from each set for Hamburg testing. Conduct Hamburg testing by AASHTO T 324.

Tensile Strength Testing

Both roadway cores and plant-mix samples were taken from each test section material. The roadway cores were tested for retained tensile strength according to the GDOT procedure, GDT-66, for determining moisture susceptibility. Plant mix samples were transported to the NCAT laboratory and also tested for moisture susceptibility. A comparison of results from both the cores and plant mix for each of the sections is shown in the following table.

TABLE 17: Initial Tensile Strength Results from Field Project Samples.

	Roadway Cores			Plant Mix		
Additive	Control	Cond.	TSR,%	Control	Cond.	TSR,%
LAS	97.1	86.6	89.2	139.8	137.1	98.0
WMX	94.3	67.6	71.7	125.2	108.2	86.4
HL	100.7	104.2	103.5	142.6	156.2	109.5

Both hydrated lime and liquid additive treatments provided acceptable and similar results. Initial results showed the mixture treated with hydrated lime produced the highest tensile strength values for both roadway cores and from plant-produced lab-compacted samples. Mixture treated with liquid additive ranked second in tensile strength followed by the mixture with WMX additive. All plant-produced lab-compacted samples met the required tensile strength ratio of 80% and all tensile strength values exceeded 100 psi. However, only the lime and liquid additive sections met the required TSR based on roadway cores, and only the lime treatment yielded average tensile strength values for roadway cores in excess of 100 psi. It is expected that the WMX additive may have lower strength because the lower production temperatures with WMX does not age the asphalt binder as much as conventional treatments and results in a slightly softer binder.

Hamburg Wheel Track Testing

As shown in Table 19, all initial samples prepared from plant-produced mix met the criteria of no rutting greater than 0.5 inches after 10,000 passes, but the WMX samples failed to achieve at least 10,000 passes for stripping inflection point. This indicates the WMX treated mix may be more susceptible to stripping than the other treatments. Again, the unsatisfactory results may be caused by the WMX mixture not being aged as much due to lower production temperatures.

TABLE 18: Initial Hamburg Testing of Plant Produced Mix.

Sample	PG	Anti-Strip	Air Voids	Total Rutting @ 10,000, in	Stripping Inflection Point, cycles
15B	67-22	LAS	7.1	0.187	N/A
16A	67-22	LAS	6.5		
16B	67-22	LAS	7.4	0.130	N/A
15A	67-22	LAS	7.5		
17A	67-22	LAS	7.1	0.168	N/A
18B	67-22	LAS	6.6		
		Avg.	7.0	0.162	
		Std. Dev.	0.4	0.029	
20	67-22	Lime	7.6	0.055	N/A
21	67-22	Lime	7.5		
22	67-22	Lime	6.9	0.062	N/A
41	67-22	Lime	7.5		
43	67-22	Lime	7.2	0.101	N/A
44	67-22	Lime	7.0		
		Avg.	7.3	0.073	
		Std. Dev.	0.3	0.025	
49	67-22	WMX	7.7	0.394	5300
24	67-22	WMX	7.9		
26	67-22	WMX	7.8	0.292	N/A
47	67-22	WMX	8.2		
50	67-22	WMX	7.2	0.404	6700
48	67-22	WMX	8.1		
		Avg.	7.8	0.363	
		Std. Dev.	0.4	0.062	

Two-Year Evaluation

In order to evaluate performance over time, samples from the project were taken after two years for evaluation. During the year two evaluation, 14 cores from each of the 3 sections (total of 42 cores) were taken for performance analysis according to the following procedures:

- Tensile strength with no freeze-thaw cycle (vacuum saturate 30 minutes and then put into hot water bath)
- Tensile strength with one freeze-thaw cycle (vacuum saturate 30 minutes and put into freezer)
- Hamburg according to AASHTO T 324 except samples subjected to one freeze-thaw
- Hamburg according to AASHTO T 324 except samples subjected to 3 freeze-thaw cycles

Tensile Strength Testing

Field cores were taken after 2 years of service and compared to the original results immediately after construction. For the two-year evaluation, samples were tested with no freeze/thaw cycle as is customarily done, but additional samples were also tested with one freeze/thaw cycle. From Figure 15 one can see that the hydrated lime resulted in much higher conditioned strength initially and after two years. It is typical to test project samples without a freeze/thaw cycle after being in place over a winter season because they are assumed to have already been through a freeze/thaw cycle. However, for this study, field samples that had been in place for two years were also subjected to one freeze/thaw cycle to determine the effect of additional conditioning.

The results show that both the liquid additive and WMX additive had higher strengths than the hydrated lime after the additional conditioning. It is of special interest to note that cores from the WMX section dramatically increased in strength over time. This implies that, although WMX materials may have low tensile strength initially due to a softer binder, the WMX binder ages over time and eventually can be expected to have strength comparable to other additive treatments. Based on the exceptional results with hydrated lime after two years, it was surprising to see tensile strengths reduced significantly after one additional freeze/thaw cycle. The LAS treatment provided very consistent results with and without additional freeze/thaw conditioning.

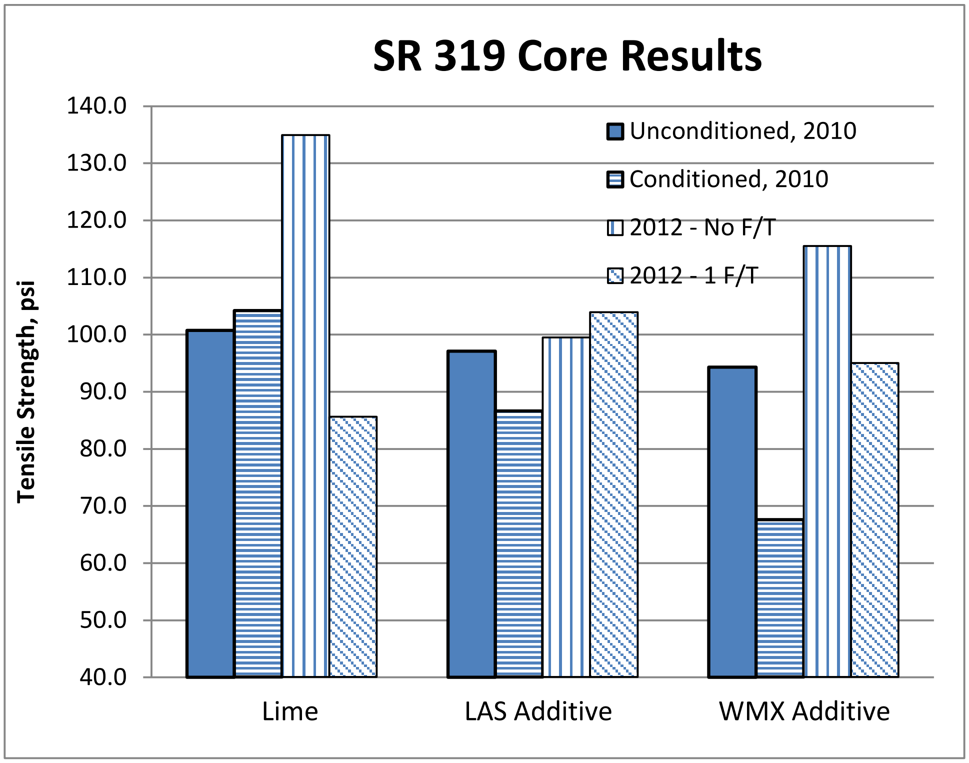

FIGURE 15: Field Core Tensile Strength.

Hamburg Wheel Track Testing

Hamburg Wheel testing was also conducted on roadway cores from the project after two years of service. Testing was performed according to AASHTO T 324 except for the additional conditioning described above. For this analysis, the LAS cores did not perform as well as the other treatments; but the lime and WMX cores performed equally well after both one and three freeze/thaw cycles. For the LAS treatment, samples after one freeze/thaw cycle failed to meet the criteria of at least 10,000 passes before exceeding one-half inch of rutting (Table 20). The same samples were terminated before they reached a stripping inflection point. Testing was stopped after 7,700 passes due to one inch rutting. Samples with three freeze/thaw cycles did not rut as severely, but did have ruts of one-half inch depth after 15,476 passes.

Both hydrated lime and WMX treated sections produced core results that exceeded 20,000 passes for rutting and stripping inflection point for both one and three freeze/thaw cycles. These results again tend to confirm that although mix from the WMX treated section did not perform well initially, it gains strength over time so that it eventually performs comparable to that of other treatments. The poorer performance of WMX indicated in original sample testing did not materialize based of field results after two years of aging.

TABLE 19: Hamburg Results of Conditioned Two Year Old Field Cores.

Sample ID	Rutting @ 20,000 Passes (inches)	Number of Passes to 0.5" Rut Depth	Stripping Inflection Point (passes)	Acceptable SIP?
LAS - 1 F/T *	NA	5,936	7,700 +	?
LAS - 3 F/T *	NA	15,476	12,000	Yes
WMX - 1 F/T	0.28	N/A	20,000 +	Yes
WMX - 3 F/T	0.33	N/A	20,000 +	Yes
Lime - 1 F/T	0.35	N/A	20,000 +	Yes
Lime - 3 F/T	0.33	N/A	20,000 +	Yes

* Did not complete 20,000 passes

Dynamic Modulus

AMPT Sample Fabrication

The samples for this testing were prepared in accordance with AASHTO PP60-09. Samples were mixed at a target temperature of 300°F and then aged for four hours at 275°F. This is in accordance with the short-term mechanical aging procedure outlined in AASHTO R 30-02. The samples were compacted to a height of 175 mm and a diameter of 150 mm and prepared to meet the tolerances outlined in Table 21. The tolerances in Table 21 represent tolerances on the final sample that had been cut and cored from the interior of the larger SGC sample. Three samples were prepared for testing from each mix. The target air void content was 7.0 ± 0.5 percent. The samples prepared for dynamic modulus testing were also used to perform the flow number test. Testing was performed on mixtures containing each combination of aggregate, anti-stripping agent, and either zero or one freeze-thaw cycles (12 mixtures total). The GDT 66 procedure was used to apply one freeze-thaw cycle to a set of samples from each combination of aggregate source and anti-stripping agent.

TABLE 20: Production Tolerances for Dynamic Modulus and Flow Number Specimens.

Parameter	Tolerance
Average Diameter	100 to 104 mm
Standard Deviation of Diameter	≤ 0.5 mm
Height	147.5 mm to 152.5 mm
End Flatness	≤ 0.5 mm
End Perpendicularity	≤ 1.0 mm
Sample Air Voids	7 ± 0.5%

Dynamic Modulus Testing Methodology

Dynamic Modulus testing was performed in an IPC Global Asphalt Mixture Performance Tester (AMPT), shown in Figure 16. Dynamic Modulus testing is performed in order to quantify the stiffness behavior of the asphalt mixture over a wide range of testing temperatures and loading rates (or frequencies). The temperatures and frequencies used for testing are those recommended by AASHTO PP61-10. For this methodology, the high test temperature is dependent on the high PG grade of the base binder utilized in the mix being tested. For this project, a PG 67-22 binder was utilized. Therefore, a 40°C high test temperature for dynamic modulus was selected. Table 22 shows the general outline of temperatures and frequencies used, while Table 23 shows the selection criteria for the high testing temperature. It should be noted, however, that this high test temperature could be reduced in the event of poor quality test data being collected. Quality of data will be better defined later.

FIGURE 16: IPC Global Asphalt Mixture Performance Tester (AMPT).

TABLE 21: Temperatures and Frequencies used for Dynamic Modulus Testing.

Test Temperature (^{o}C)	Loading Frequencies Hz)
4.0	10, 1, 0.1
20.0	10, 1, 0.1
High Testing Temperature	10, 1, 0.1, 0.01

TABLE 22: High Test Temperature for Dynamic Modulus Testing.

High Test Temperature (^oC)	High PG Grade of Base Binder
35	PG 58-XX and softer
40	PG 64-XX and PG 70-XX
45	PG 76-XX and stiffer

Dynamic Modulus testing was performed in accordance with AASHTO TP 79-11 using unconfined samples. Test data was screened for data quality in accordance with the limits set in AASHTO TP79-11. A summary of these data quality statistics is given in Table 24. Variability of Dynamic Modulus values at specific temperatures and frequencies were checked to have a coefficient of variation (COV) at or below 13%. All data were checked for reasonableness as well (reduction in moduli with increasing temperature, slower loading). Data with borderline data quality statistics were evaluated on a case by case basis.

TABLE 23: Dynamic Modulus Data Quality Threshold Values.

Data Quality Statistic	Limit
Deformation Drift	No Limit in Direction of Applied Load
Peak-to-Peak Strain	75 to 125 microstrain (unconfined tests) 85 to 115 microstrain (confined tests)
Load Standard Error	< 10%
Deformation Standard Error	< 10%
Deformation Uniformity	< 30%
Load Drift	< 2%
Phase Angle Uniformity	< 3^o

The collected data were then analyzed for two specific purposes. First, the data were used to generate a dynamic modulus Master Curve. The Master Curve uses the principle of time-temperature superposition to correct collected data at multiple temperatures and frequencies to a reference temperature so that the stiffness data can be viewed without temperature as a variable. This method of analysis allows for visual relative comparisons to be made between multiple mixes. A visual example of using the time-temperature superposition principle to generate a Master Curve is shown in Figure 17.

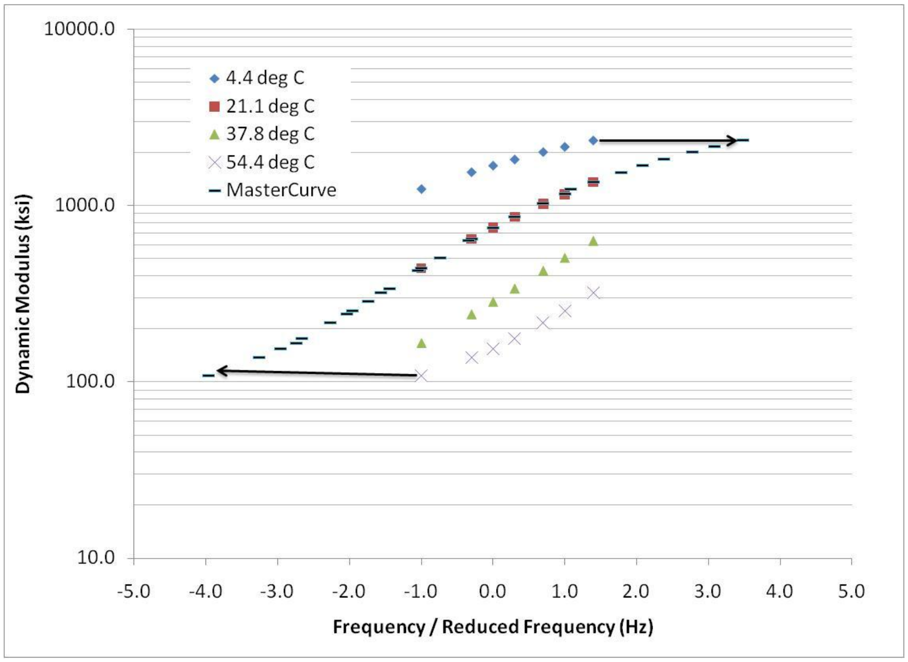

FIGURE 17: Use of Time-Temperature Shift Factors to Generate Dynamic Modulus Master Curve.

Generation of the Master Curve also allows for generation of the dynamic modulus data over the entire range of temperatures and frequencies required for mechanistic-empirical pavement design using the MEPDG. By having an equation for the curve describing the stiffness behavior of the asphalt mix, both interpolated and extrapolated data points at various points along the curve can then be calculated. The temperatures and frequencies needed as an input for the MEPDG are listed in Section 10.6.1 of AASHTO TP 61-09.

Data analysis was conducted per the methodology in AASHTO PP 61-10. The general form of the Master Curve equation is shown as Equation 6. As mentioned, the dynamic modulus data are shifted to a reference temperature. This is done by converting testing frequency to a reduced frequency using the Arrhenius equation (Equation 7). Substituting Equation 2 into Equation 1 yields the final form of the Master Curve equation, shown as Equation 8. The shift factors required at each temperature are given in Equation 9 (the right-hand portion of Equation 7). The limiting maximum modulus in Equation 8 is calculated using the Hirsch Model, shown as Equation 10. The Pc term, Equation 11, is simply a variable required for Equation 10. A limiting binder modulus of 1 GPa is assumed for this equation. Non-linear regression is then conducted using MasterSolver.xls program (developed under NCHRP 09-29) to develop the coefficients for the Master Curve equation. Typically, these curves have an Se/Sy term of less than 0.05 and an R^2 value of greater than 0.99. Definitions for the variables in Equations 6-11 are given in Table 25.

$$Log|E^*| = \partial + \frac{(Max-\partial)}{1+e^{\beta+\gamma \log f_r}} \qquad \textbf{(6)}$$

$$log f_r = log\ f + \frac{\Delta E_a}{19.14714}\left[\frac{1}{T} - \frac{1}{T_r}\right] \quad \mathbf{(7)}$$

$$log|E^*| = \partial + \frac{(Max-\partial)}{1+e^{\beta+\gamma\left\{log f+\frac{\Delta E_a}{19.14714}\left[\frac{1}{T}-\frac{1}{T_r}\right]\right\}}} \quad \mathbf{(8)}$$

$$\log[a(T)] = \frac{\Delta E_a}{19.14714}\left[\frac{1}{T} - \frac{1}{T_r}\right] \quad \mathbf{(9)}$$

$$|E^*|_{max} = P_c\left[4{,}200{,}000\left(1 - \frac{VMA}{100}\right) + 435{,}000\left(\frac{VFA*VMA}{10{,}000}\right) + \frac{1-P_c}{\frac{\left(1-\frac{VMA}{100}\right)}{4{,}200{,}000}+\frac{VMA}{435{,}000(VFA)}}\right] \quad \mathbf{(10)}$$

$$P_c = \frac{\left(20+\frac{435{,}000(VFA)}{VMA}\right)^{0.58}}{650+\left(\frac{435{,}000(VFA)}{VMA}\right)^{0.58}} \quad \mathbf{(11)}$$

TABLE 24: Master Curve Equation Variable Descriptions.

Variable	Definition
|E*|	Dynamic Modulus, psi
δ,β, and γ	Fitting Parameters
Max	Limiting Maximum Modulus, psi
$\mathbf{f_r}$	Reduced frequency at the reference temperature, Hz
f	The loading frequency at the test temperature
$\mathbf{\Delta E_a}$	Activation Energy (treated as a fitting parameter)
T	Test Temperature, ^{o}K
$\mathbf{T_r}$	Reference Temperature, ^{o}K
a(T)	The shift factor at Temperature T
$\mathbf{\|E^*\|_{max}}$	The limiting maximum HMA dynamic modulus, psi (Max)
VMA	Voids in Mineral Aggregate (%)
VFA	Voids filled with asphalt (%)

Dynamic Modulus Testing Results

The following tables and figures give the results of the dynamic modulus testing for the 12 mixtures tested:

- Table 26 lists the Master Curve Coefficients (defined in Table 25) for each mixture.
- Figure 18 plots the dynamic modulus Master Curves for each mixture tested with and without a freeze-thaw cycle. For example, the mixture with the Lithia Springs aggregate and Hydrated Lime as an additive was tested in dynamic modulus both with and without a freeze-

thaw cycle. These two Master Curves are shown on one plot to see the effect of the freeze-thaw cycle on the mixture dynamic modulus. Six plots are shown in Figure 18.

- Figure 19 plots the dynamic modulus Master Curves looking at the effect of the anti-stripping agent on mixtures where the aggregate source and number of freeze-thaw cycles are held constant. Four plots are shown in Figure 19.
- Figure 20 plots the dynamic modulus Master Curves looking at the effect of the aggregate source on mixtures where the anti-stripping agent and number of freeze-thaw cycles are held constant. Six plots are shown in Figure 20.
- A General Linear Model (GLM) ($\alpha = 0.05$) statistical analysis was performed in Minitab® to quantify the factors that statistically impact the dynamic modulus at two points on the Master Curve.
    - Table 27 shows the GLM output for 4°C test temperature and 10 Hz frequency testing condition. This represents a data point on the right-hand portion of the curve. Practically, this analysis shows the impact of the test variables in a cold temperature environment with high traffic speeds.
    - Table 28 shows the GLM output for 40°C test temperature and 0.01Hz frequency testing condition. This represents a data point on the left-hand portion of the curve. Practically, this analysis shows the impact of the test variables in a warm temperature environment with slow traffic speeds.
- The MEPDG dynamic modulus input data was generated as a product of this testing. This information has been provided for each of the 12 tested mixes in Appendix A.

TABLE 25: Master Curve Coefficients – All Mixtures.

Aggregate	**Anti-Strip**	**F/T Cycles**	**Max E* (Ksi)**	**Min E* (Ksi)**	**Beta**	**Gamma**	**E_A**	**R^2**	**S_e/S_y**
Lithia Springs	HL	0	3142.1	5.48	-1.173	-0.503	198534.0	0.991	0.07
Lithia Springs	LAS	0	3166.3	3.64	-1.165	-0.506	202523.9	0.997	0.04
Lithia Springs	WMX	0	3160.0	4.49	-1.139	-0.476	201358.3	0.997	0.04
Lithia Springs	HL	1	3139.7	9.33	-1.034	-0.536	183912.9	0.979	0.10
Lithia Springs	LAS	1	3157.0	5.05	-1.056	-0.534	195875.7	0.994	0.05
Lithia Springs	WMX	1	3170.1	4.01	-1.072	-0.464	198016.3	0.996	0.05
Lithonia	HL	0	3177.3	5.49	-1.157	-0.523	198793.6	0.995	0.05
Lithonia	LAS	0	3177.4	4.65	-1.008	-0.476	198826.0	0.996	0.04
Lithonia	WMX	0	3184.5	4.09	-1.120	-0.472	184876.2	0.993	0.06
Lithonia	HL	1	3185.2	4.88	-1.132	-0.515	193761.4	0.996	0.05
Lithonia	LAS	1	3175.0	4.51	-0.972	-0.477	199578.9	0.997	0.04
Lithonia	WMX	1	3184.5	5.49	-1.009	-0.467	197784.4	0.997	0.04

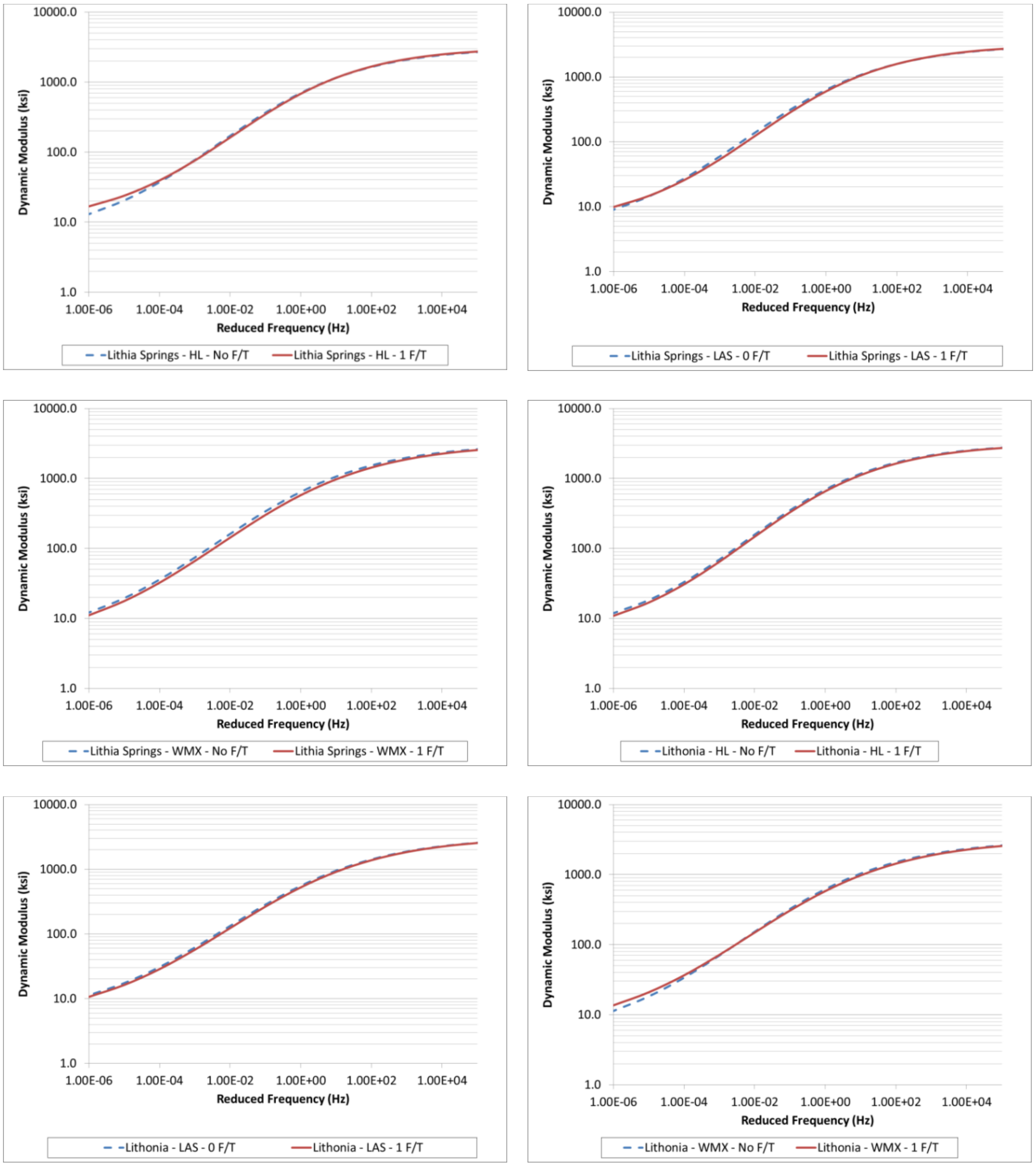

FIGURE 18: Dynamic Modulus Master Curves – with and without a freeze-thaw cycle – Constant Aggregate Source and Additive Type.

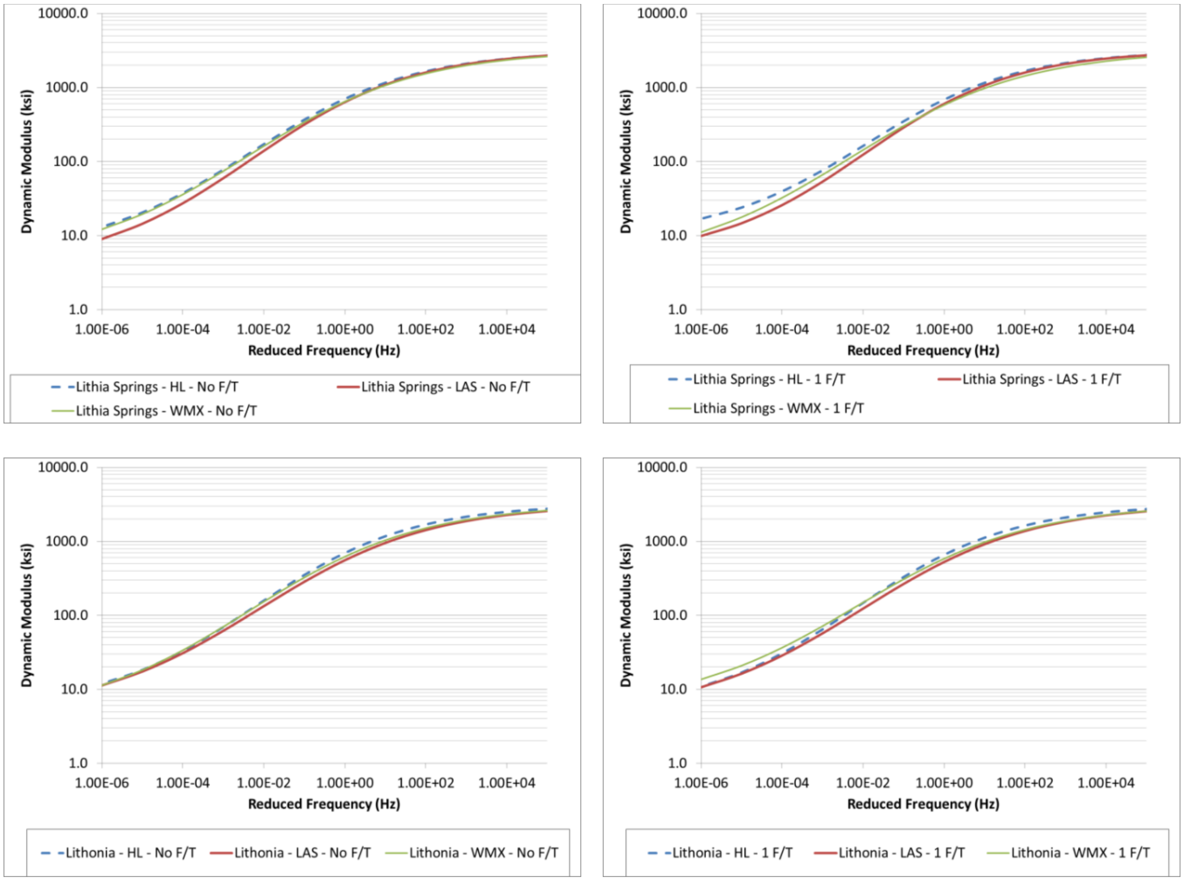

FIGURE 19: Dynamic Modulus Master Curves – Additive as a Variable – Constant Aggregate Type and Number of Freeze-Thaw Cycles.

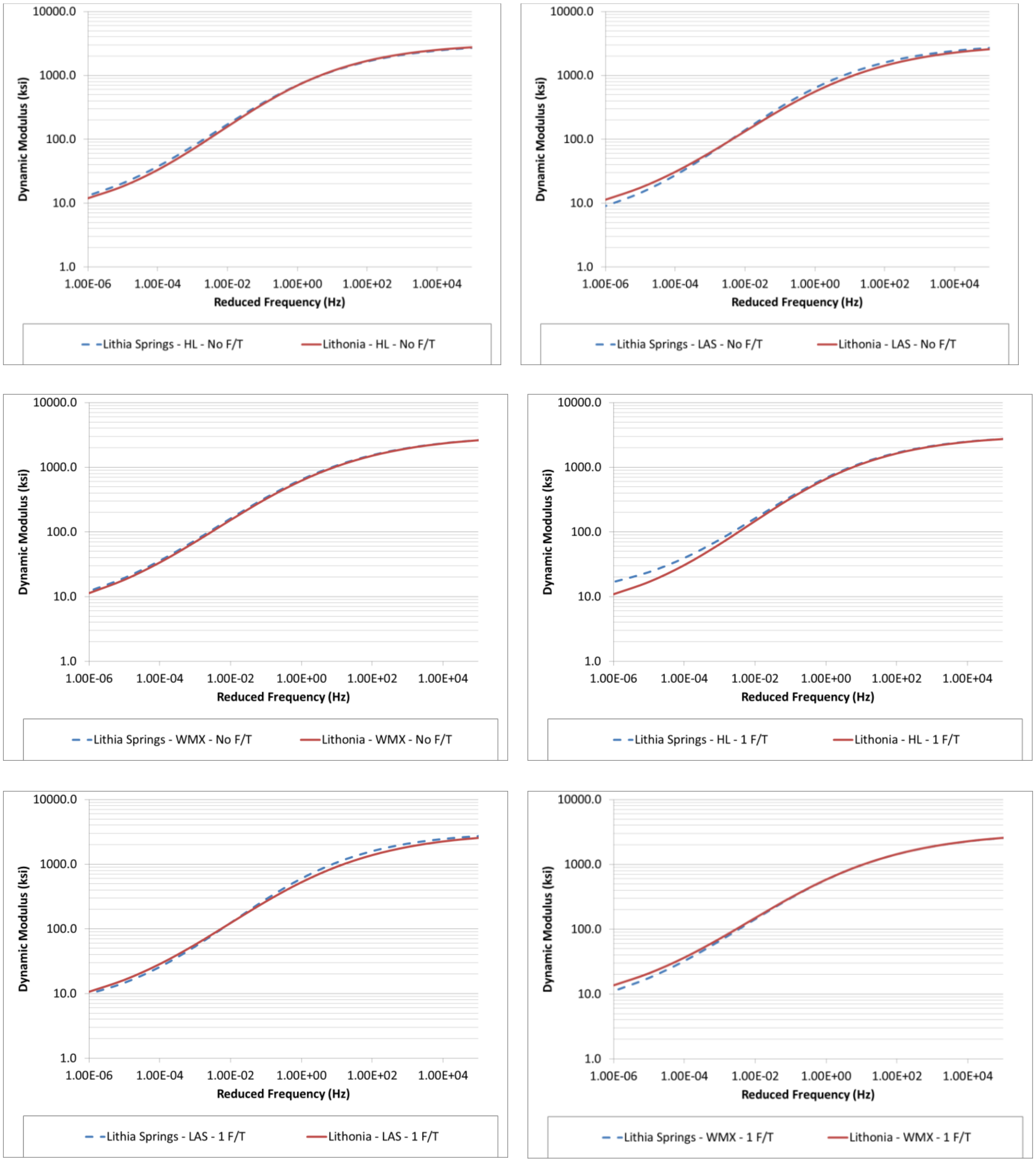

FIGURE 20: Dynamic Modulus Master Curves – Aggregate Source as a Variable – Constant Additive Type and Number of Freeze-Thaw Cycles.

TABLE 26: General Linear Model (α = 0.05) Results – Dynamic Modulus tested at 4°C Temperature and 10 Hz Frequency.

Factor	Type	Levels	Values
Aggregate Source	fixed	2	Lithia Springs, Lithonia
Additive	fixed	3	HL, LAS, WMX
F/T Cycles	fixed	2	0, 1

Analysis of Variance for E*, ksi, using Adjusted SS for Tests

Source	DF	Seq SS	Adj SS	Adj MS	F	P
Aggregate Source	1	96967	96967	96967	6.51	**0.017**
Additive	2	512092	512092	256046	17.20	**0.000**
F/T Cycles	1	32432	32432	32432	2.18	0.153
Aggregate Source*Additive	2	87316	87316	43658	2.93	0.073
Aggregate Source*F/T Cycles	1	704	704	704	0.05	0.830
Additive*F/T Cycles	2	10437	10437	5219	0.35	0.708
Aggregate Source*Additive*F/T Cycles	2	16832	16832	8416	0.57	0.576
Error	24	357304	357304	14888		
Total	35	1114085				

S = 122.015   R-Sq = 67.93%   R-Sq(adj) = 53.23%

Unusual Observations for E*, ksi

Obs	E*, ksi	Fit	SE Fit	Residual	St Resid
19	2033.28	2329.21	70.45	-295.93	-2.97 R
20	2596.32	2329.21	70.45	267.11	2.68 R

R denotes an observation with a large standardized residual.

Grouping Information Using Tukey Method and 95.0% Confidence

Aggregate Source	Additive	F/T Cycles	N	Mean	Grouping
Lithonia	HL	0	3	2329	A
Lithia Springs	HL	1	3	2306	A B
Lithia Springs	HL	0	3	2300	A B
Lithonia	HL	1	3	2217	A B C
Lithia Springs	LAS	0	3	2205	A B C
Lithia Springs	LAS	1	3	2196	A B C
Lithia Springs	WMX	0	3	2101	A B C
Lithonia	WMX	0	3	2017	A B C
Lithonia	LAS	0	3	1975	A B C
Lithonia	WMX	1	3	1958	B C
Lithia Springs	WMX	1	3	1950	B C
Lithonia	LAS	1	3	1940	C

Means that do not share a letter are significantly different.

TABLE 27: General Linear Model (α = 0.05) Results – Dynamic Modulus tested at 40°C Temperature and 0.01 Hz Frequency.

Factor	Type	Levels	Values
Aggregate Source	fixed	2	Lithia Springs, Lithonia
Additive	fixed	3	HL, LAS, WMX
F/T Cycles	fixed	2	0, 1

Analysis of Variance for E*, ksi, using Adjusted SS for Tests

Source	DF	Seq SS	Adj SS	Adj MS	F	P
Aggregate Source	1	0.362	0.362	0.362	0.05	0.834
Additive	2	295.782	295.782	147.891	18.42	**0.000**
F/T Cycles	1	0.163	0.163	0.163	0.02	0.888
Aggregate Source*Additive	2	158.924	158.924	79.462	9.90	**0.001**
Aggregate Source*F/T Cycles	1	7.985	7.985	7.985	0.99	0.329
Additive*F/T Cycles	2	13.494	13.494	6.747	0.84	0.444
Aggregate Source*Additive*F/T Cycles	2	27.532	27.532	13.766	1.71	0.201
Error	24	192.664	192.664	8.028		
Total	35	696.905				

S = 2.83331   R-Sq = 72.35%   R-Sq(adj) = 59.68%

Unusual Observations for E*, ksi

Obs	E*, ksi	Fit	SE Fit	Residual	St Resid
10	41.1762	35.5633	1.6358	5.6130	2.43 R

R denotes an observation with a large standardized residual.

Grouping Information Using Tukey Method and 95.0% Confidence

Aggregate Source	Additive	F/T Cycles	N	Mean	Grouping
Lithia Springs	HL	1	3	35.56	A
Lithonia	WMX	1	3	31.26	A B
Lithia Springs	HL	0	3	30.70	A B C
Lithonia	WMX	0	3	30.69	A B C
Lithia Springs	WMX	0	3	29.48	A B C D
Lithonia	HL	0	3	27.56	A B C D
Lithia Springs	WMX	1	3	27.48	A B C D
Lithonia	HL	1	3	26.44	B C D
Lithonia	LAS	0	3	26.27	B C D
Lithonia	LAS	1	3	24.41	B C D
Lithia Springs	LAS	1	3	22.49	C D
Lithia Springs	LAS	0	3	22.12	D

Means that do not share a letter are significantly different.

As a result of this investigation, the following conclusions were drawn…

- At the 4°C temperature and 10 Hz frequency, the only statistically significant variables were the aggregate source and additive type. However, 10 of the 12 mixtures were considered statistically equivalent. Although the interaction of aggregate source and additive type was not found to be statistically significant at a 95% confidence level, it was deemed to be practically significant base on the *P*-value of only 0.073. Based on these data, it can be concluded that the test variables (aggregate source, anti-stripping agent, and presence or absence of a freeze-thaw cycle) had minimal impact on the dynamic modulus at the cold temperature and fast loading testing condition. Visual inspection of Figures 18 through 20 help validate this finding.

- At the 40°C temperature at 0.01 Hz frequency, the only statistically significant variables were the additive type and the interaction between the aggregate source and additive type. The mixtures with the LAS were statistically the softest mixtures at this testing condition. Visual inspection of Figure 19 reinforces this finding. However, 9 of the 12 mixtures had statistically equivalent dynamic moduli at this testing condition. Practically, this means the test variables did not have a large impact on the dynamic modulus at this testing condition.
- The presence or absence of a freeze-thaw cycle did not impact the dynamic modulus either statistically or visually. This was expected given the fact that none of these mixtures failed the TSR test with only one freeze-thaw cycle. This finding is further reinforced by visual inspection of Figure 18.

Flow Number

Flow Number Testing Methodology

The Flow Number test is a rutting resistance test that is performed using the Asphalt Mixture Performance Tester (AMPT). It applies a repeated compressive loading to an asphalt specimen while the AMPT records the deformation of the specimen with each additional loading cycle. The user defines the temperature, applied stress state (deviator stress and confining stress), and number of cycles at which the test is performed. The loading is applied for a duration of 0.1 seconds followed by a 0.9 second rest period every 1 second cycle. Flow number data is commonly modeled with the Francken model, shown as Equation 12 (36). An example of flow number test data is shown as Figure 21.

$$\varepsilon_p(N) = aN^b + c(e^{dN} - 1) \quad \textbf{(12)}$$

The flow number is defined as the number of cycles at which the sample begins to rapidly fail. This is more properly defined as the breakpoint between steady-state rutting (secondary rutting) and the more rapid failure of the specimen (tertiary flow). Figure 21 demonstrates this concept graphically.

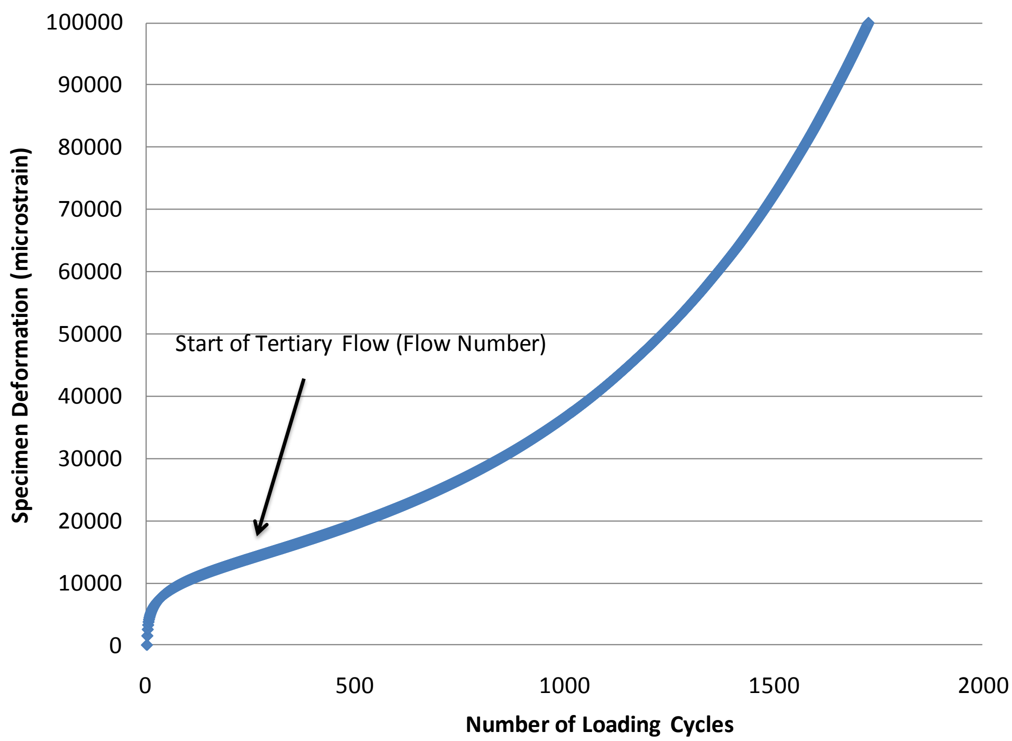

FIGURE 21: Typical Flow Number Test Data.

Flow number testing for this project was performed in accordance with AASHTO PP79-11. The samples were tested unconfined with a deviator stress of 87 psi. Samples were tested unconfined so that tertiary flow would be achieved during testing. A temperature of 62°C was selected for testing because it is approximately the LTPPBind (v3.1) 50% Reliability temperature at a depth of 20mm in an asphalt layer in the southern Georgia geographical region. This temperature region in Georgia was selected due to it being the warmest climate in the state. As such, these testing parameters would be conservative in testing for rutting susceptibility, especially in cooler regions of the state.

Adjustments to temperature are not made based on traffic level using this procedure, since the final flow number is used to determine the traffic level the mix should be able to withstand. These testing parameters are those recommended by NCHRP 09-33 for evaluating the rutting resistance of HMA technologies using the flow number test (*37*). The same flow number testing parameters are also recommended for WMA by NCHRP Project 09-43 (*38*). However, while the testing parameters are the same, the recommended minimum flow numbers for a given traffic level are different for HMA and WMA. These criteria are summarized in Table 29.

TABLE 28: Flow Number Criteria from NCHRP 09-33 (HMA) (Bonaquist 2011) and 09-43 (WMA) (Bonaquist 2011).

Traffic Level (Million ESAL)	NCHRP Report 673 (HMA)	NCHRP Report 691 (WMA)
< 3	---	---
3 to < 10	53	30
10 to < 30	190	105
≥ 30	740	415

For this study, a set of three samples from each combination of aggregate, anti-stripping agent, and freeze-thaw cycles were tested for flow number (a total of 12 sets of flow number). The samples tested were the same samples used for dynamic modulus testing. These samples were fabricated according to the tolerances listed in AASHTO PP 60-09 and were fabricated to 7 ± 0.5% air voids.

Flow Number Results

Table 30 shows the data summary from the flow number testing for this project. This data summary shows the average and standard deviation of the flow number (Francken model) for the tested mixture. Table 30 also shows the recommended traffic level each mix could withstand based on the previously listed recommendations. Figure 22 plots the average and standard deviation for each set of flow number samples tested. The raw data for this testing is given in Table 31. Finally, Table 32 shows a summary of the statistical analysis performed on this data set. A General Linear Model (GLM) analysis ($\alpha = 0.05$) was performed in Minitab to determine the significant statistical factors with respect to rutting susceptibility in the flow number test.

AASHTO TP79-11 states that the coefficient of variation for the flow number should be no more than 20 percent. Table 30 shows the variability for this data set falls beneath that threshold for all but one mixture (Lithia Springs aggregate with WMX additive and no freeze-thaw cycles). This data set contained one sample with a very high flow number (double the flow number of the other samples in the data set). For the purposes of statistical analysis, this data point was removed from the data set.

TABLE 29: Summary of Flow Number Results (Francken Model).

Aggregate	Additive	Freeze-Thaw Cycles	Average Flow Number	Flow Number Std. Deviation	COV (%)	Recommended Traffic Level (MESAL)
Lithia Springs	HL	0	72.7	13.2	18.2	3 to <10
Lithia Springs	LAS	0	30.7	2.5	8.2	<3
Lithia Springs	WMX	0	97.3	44.0	45.2	3 to <10
Lithia Springs	HL	1	61.3	1.5	2.5	3 to <10
Lithia Springs	LAS	1	24.7	4.0	16.4	<3
Lithia Springs	WMX	1	73.3	5.5	7.5	3 to <10
Lithonia	HL	0	41.3	3.1	7.4	<3
Lithonia	LAS	0	39.3	5.5	14.0	<3
Lithonia	WMX	0	57.7	2.5	4.4	3 to <10
Lithonia	HL	1	38.7	7.0	18.2	<3
Lithonia	LAS	1	30.3	3.2	10.6	<3
Lithonia	WMX	1	57.7	6.4	11.1	3 to <10

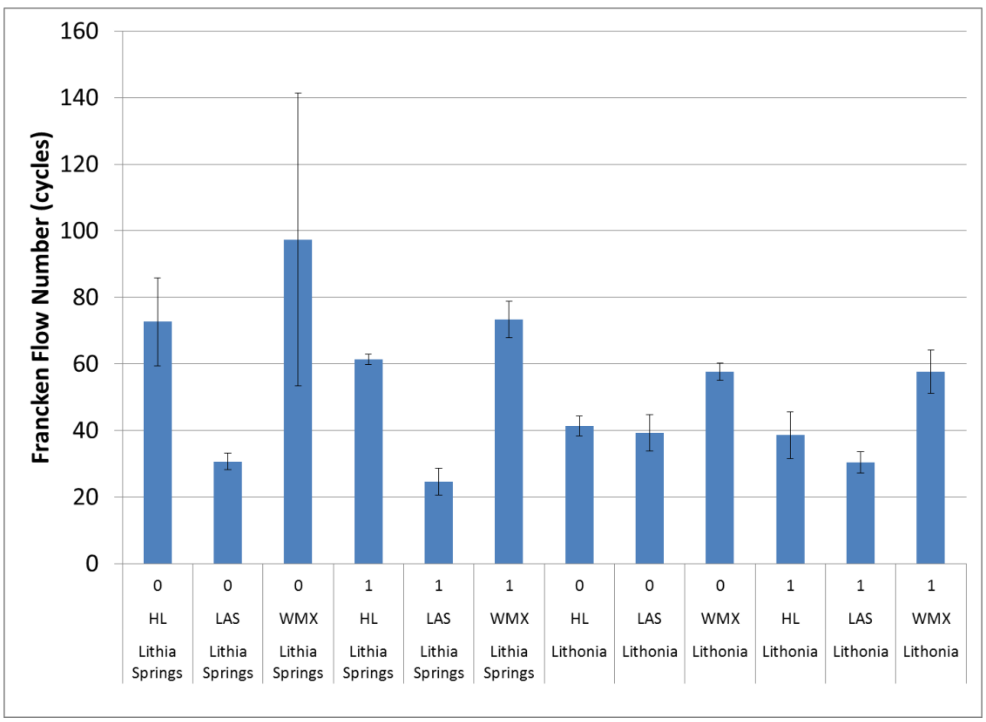

FIGURE 22: Average and Standard Deviation Plot of All Tested Flow Number Samples.

TABLE 30: Raw Flow Number Data.

Aggregate	Additive	Freeze-Thaw Cycles	Specimen ID	Specimen Air Voids (%)	Francken Flow Number	Specimen Microstrain at Flow Number
Lithia Springs	HL	0	10	7.1	87	16948
Lithia Springs	HL	0	11	7.2	61	18234
Lithia Springs	HL	0	12	7.3	70	17114
Lithia Springs	LAS	0	101	6.7	31	13388
Lithia Springs	LAS	0	103	6.9	33	14166
Lithia Springs	LAS	0	105	6.9	28	13871
Lithia Springs	WMX	0	203	6.8	148	16145
Lithia Springs	WMX	0	205	6.8	69	16185
Lithia Springs	WMX	0	207	7.1	75	16726
Lithia Springs	HL	1	9	7.2	61	19061
Lithia Springs	HL	1	13	7.5	60	19826
Lithia Springs	HL	1	14	7.0	63	18660
Lithia Springs	LAS	1	104	6.7	29	14115
Lithia Springs	LAS	1	106	6.9	24	13733
Lithia Springs	LAS	1	108	7.3	21	13979
Lithia Springs	WMX	1	204	6.7	79	18267
Lithia Springs	WMX	1	206	6.7	73	16991
Lithia Springs	WMX	1	208	6.8	68	17066
Lithonia	HL	0	401	6.7	44	15714
Lithonia	HL	0	403	7.1	38	15252
Lithonia	HL	0	405	6.6	42	14892
Lithonia	LAS	0	503	6.8	34	13836
Lithonia	LAS	0	504	7.0	45	13857
Lithonia	LAS	0	505	6.6	39	14306
Lithonia	WMX	0	603	6.6	60	15679
Lithonia	WMX	0	605	6.7	55	15692
Lithonia	WMX	0	607	6.8	58	15618
Lithonia	HL	1	404	6.8	46	15719
Lithonia	HL	1	406	6.5	32	15694
Lithonia	HL	1	407	6.7	38	15803
Lithonia	LAS	1	5.6	6.7	29	13701
Lithonia	LAS	1	5.7	6.6	28	13980
Lithonia	LAS	1	5.8	7.2	34	14619
Lithonia	WMX	1	6.4	6.6	65	16784
Lithonia	WMX	1	6.6	6.6	55	16548
Lithonia	WMX	1	6.8	6.9	53	16367

TABLE 31: General Linear Model (α = 0.05) Results on Flow Number Data Set – Minus Outlier.

General Linear Model: Flow Number versus Aggregate, Additive, F_T

Factor	Type	Levels	Values
Aggregate	fixed	2	Lithia Springs, Lithonia
Additive	fixed	3	HL, LAS, WMX
F_T	fixed	2	0, 1

Analysis of Variance for Flow Number, using Adjusted SS for Tests

Source	DF	Seq SS	Adj SS	Adj MS	F	P
Aggregate	1	992.92	1164.83	1164.83	34.73	0.000
Additive	2	6848.60	6806.39	3403.19	101.48	0.000
F_T	1	192.07	183.71	183.71	5.48	0.028
Aggregate*Additive	2	1811.79	1798.75	899.37	26.82	0.000
Aggregate*F_T	1	8.01	4.51	4.51	0.13	0.717
Additive*F_T	2	112.10	116.08	58.04	1.73	0.199
Aggregate*Additive*F_T	2	59.05	59.05	29.53	0.88	0.428
Error	23	771.33	771.33	33.54		
Total	34	10795.89				

S = 5.79105   R-Sq = 92.86%   R-Sq(adj) = 89.44%

Unusual Observations for Flow Number

Obs	Flow Number	Fit	SE Fit	Residual	St Resid	
1	87.0000	72.6667	3.3435	14.3333	3.03	R
2	61.0000	72.6667	3.3435	-11.6667	-2.47	R

R denotes an observation with a large standardized residual.

Grouping Information Using Tukey Method and 95.0% Confidence

Aggregate	Additive	F_T	N	Mean	Grouping
Lithia Springs	WMX	1	3	73.33	A
Lithia Springs	HL	0	3	72.67	A
Lithia Springs	WMX	0	**2**	72.00	A
Lithia Springs	HL	1	3	61.33	A
Lithonia	WMX	0	3	57.67	A B
Lithonia	WMX	1	3	57.67	A B
Lithonia	HL	0	3	41.33	B C
Lithonia	LAS	0	3	39.33	C
Lithonia	HL	1	3	38.67	C
Lithia Springs	LAS	0	3	30.67	C
Lithonia	LAS	1	3	30.33	C
Lithia Springs	LAS	1	3	24.67	C

Means that do not share a letter are significantly different.

Based on these analysis results, the following conclusions can be drawn…

- Six of the twelve mix designs were suitable for a 3 to 10 million ESAL road while the remaining mix designs were only suitable for a lower volume road (less than 3 million ESALs).
- The GLM results (Table 32) showed the aggregate source, anti-stripping agent, and number of freeze-thaw cycles to be statistically significant (p-value $< \alpha = 0.05$) in the context of impacting the mixture flow number. The interaction between the aggregate source and anti-stripping agent was also shown to be significant. All other interactions were not statistically significant.
- The best mix designs in terms of rutting resistance were the mix designs with the following aggregate and additive combinations
    - Lithia Springs with WMX
    - Lithia Springs with HL
    - Lithonia with WMX
- While the statistical analysis showed the presence or absence of a freeze-thaw cycle to be a significant factor (p-value $= 0.028 < \alpha = 0.05$), this variable did not have an impact on the traffic level a particular mix could withstand. For example, the mixture with Lithia Springs aggregate and the WMX additive was suitable for a 3 to 10 million ESAL road both with and without a freeze-thaw cycle.
- Figure 22 shows the mixtures with WMX to have the highest flow numbers with respect to the other additives. This was confirmed by the statistical analysis.
- Figure 22 shows the mixtures with LAS to have the lowest flow numbers with respect to the other additives. This was confirmed by the statistical analysis.
- With the exception of the mixtures with LAS, the mixtures with the Lithia Springs aggregate source had superior rutting resistance to the mixtures using the Lithonia aggregate source.

CHAPTER 6: RESULTS AND CONCLUSIONS

Discussion of Results

This research consisted of laboratory testing of both lab-prepared and field-produced mix. Field cores of in-place pavements were also taken from initial construction on a test section project, and on field projects that were several years old. A variety of tests such as tensile strength, Hamburg Wheel Test, Dynamic Modulus, and Flow Number were performed on various samples and various levels of conditioning by use of freeze/thaw cycles.

Laboratory Tensile Strength

GDT-66 was used for moisture susceptibility testing. That test is similar to AASHTO T 283 except that vacuum saturation is for a 30 minute period, the conditioning temperature immediately before testing is 55°F, and the loading rate is 0.065 in/min. Three additives were used to represent use of hydrated lime, liquid anti-strip, and WMX additives for improving resistance to moisture damage. Up to four different aggregate sources were used for the study including both granite and limestone materials commonly used in Georgia. However, the limestone was only used in a 25 mm base course design due to potential polishing characteristics under heavy traffic. A total of 189 tensile strength tests were performed during this portion of the research alone.

One reason limestone was used was to consider a common assumption that hydrated lime does not perform as well with limestone as with granite due to similar chemical composition of limestone and hydrated lime. An ANOVA of 25 mm limestone mix tensile strength showed that additive type was the only significant variable in the testing that also included multiple freeze/thaw cycles, and that performance with hydrated lime was significantly better than test results with other additive types.

When all 25 mm mixes are considered, there was a strong relationship between aggregate type, additive type, and the interaction of those variables. When only 25 mm limestone aggregate is considered, mixture treated with hydrated lime averaged 98.7 % TSR while WMX averaged 81.4% and LAS averaged 77.8%. When only granite mixtures are considered, TSR results for the 25 mm mix averaged 115.0 % for the hydrated lime treatment, 92.8 % for LAS and 82.4 % for WMX.

Moisture susceptibility was conducted at 0, 1, 5, and 10 freeze/thaw cycles. Results for 0 and 1 cycle were similar and results for 5 and 10 cycles were similar with the lower number of cycles having the highest strength. Both 5 and 10 freeze/thaw cycles were significantly more discriminating than one freeze/thaw cycle alone.

Field Projects

A total of 20 projects, 10 with lime and 10 with liquid additive treatment were, were selected to evaluate long-term performance of additive type. The average age for each set of projects was 8.2 years for liquid anti-strip treatment and 8.4 years for hydrated lime treatment. A comparison of traffic loading could not be accomplished because that information is often not kept for projects not on the state route system. The selection of projects was also limited due to the need to have 12.5 mm surface courses to ensure cores were thick enough for tensile strength testing.

A comparison of test results from project cores after 0, 1, and 3 freeze-thaw cycles shows that subjecting roadway cores to freeze-thaw conditions is more severe than vacuum saturation alone. From the results, the average tensile strength of liquid additive projects was reduced by 50% when comparing results after 3 freeze-thaw cycles to no freeze-thaw cycles. After 3 freeze-thaw cycles, the cores treated with hydrated lime had 50 percent higher tensile strength than the cores treated with liquid additive.

Additional moisture susceptibility testing was performed on project cores averaging 8 years of age with the Hamburg Wheel Tracker. Five projects each, with lime and with liquid anti-strip treatment, were selected for the additional study. Testing was performed in accordance with AASHTO T 324-04 with the exception that half of the cores were first vacuum saturated for 30 minutes and subjected to one freeze-thaw cycle at the request of GDOT to evaluate performance over more severe conditions than typical Hamburg testing. Two of the five liquid anti-strip projects failed to meet general requirements for an acceptable stripping inflection point; all of the hydrated lime treated projects met this requirement.

Dynamic Modulus testing on mixtures from two aggregate sources was relatively inconclusive. Although hydrated lime treatment produced slightly higher results, 10 of 12 mixtures tested at 4°C and a frequency of 10 Hz were considered statistically equivalent. When tested at slow speed and high temperatures (0.01 Hz and 40°C) as would be typical of a rutting scenario, the hydrated lime again had slightly higher values but 9 of 12 mixes were statistically similar.

Dynamic Modulus samples were also used for Flow Number testing according to AASHTO PP79-11. Results for the mixture treated with WMX were the highest regardless of the number of freeze/thaw cycles. Flow Number with Lithia Springs aggregate was only about one-third the number of cycles to failure when LAS treatment was used as when lime or WMX additives were used. With Lithonia aggregate, both lime and LAS produced similar results which were only about two-thirds the number of cycles to failure as for WMX treatment.

Conclusions

Samples from aged field projects were tested after vacuum saturation without a freeze-thaw cycle as described in GDT-66 due to an assumption that in-place pavement mixtures have already gone through environmental aging conditions. However, testing in this study showed that subjecting samples to one and three freeze-thaw cycles resulted in more severe conditioning and provided greater discrimination between test results for the different treatment.

- Hydrated lime maintained the best TSR results for both granite and limestone 25 mm mixtures. The LAS additive performed better than WMX for granite mixtures, but WMX performed better for limestone mixtures.
- For all 25 mm mixes, the number of freeze/thaw cycles was not significant, but the

interaction of cycles and additive type was significant. Hydrated lime produced the highest TSR results followed by LAS treatment. WMX additive at 3 freeze/thaw cycles produced the lowest results.

- For 12.5 mm mixes, all variables were considered significant with the greatest significance being the additive type. Hydrated lime had the highest TSR results (107.4%), while LAS and WMX results were similar at 92.3 and 92.7%, respectively. Only the Lithia Springs aggregate treated with WMX and subjected to three freeze/thaw cycles failed to meet the average 80% TRS requirement.
- For both 12.5 mm and 25 mm mixes, the limestone aggregate generally produced the lowest tensile strength.
- Multiple freeze/thaw cycles of 0, 1, 5, and 10 cycles were used for a portion of the research study. Hydrated lime had the highest tensile strength and highest TSR values and was the only additive treatment to meet the minimum of 80% TSR for all freeze/thaw cycle combinations. In contrast, the average of all WMX sets with 5 or more freeze/thaw cycles failed to meet the criterion.
- Both 5 and 10 freeze/thaw cycles were significantly more discriminating in regard to moisture susceptibility than one freeze/thaw cycle alone.
- After roadway cores from 8 year old projects were subjected to 3 freeze-thaw cycles, the cores treated with hydrated lime had 50 percent higher tensile strength than the cores treated with liquid additive.
- Hamburg tests of field projects 8 years old showed the hydrated lime treatment met criteria for an acceptable stripping inflection point; in contrast, only 60% of the projects treated with liquid anti-strip met that same criterion. However, these results were based on only five projects each.
- Dynamic Modulus testing showed the mixtures treated with hydrated lime had slightly better results, but for the most part test results were statistically equivalent.
- Flow Number testing showed better results when WMX additive was used. In all cases, LAS treatment produced the lowest results.
- Dynamic Modulus and Flow Number tests do not appear to be practical for use as moisture susceptibility tests. Dynamic Modulus results were not sufficiently discriminating, and Flow Number tests produced opposite results from other testing and conditioning methods.
- WMX treated mixtures produced low initial tensile strengths, but the strength of these mixtures improved with time.

Recommendations

As a result of the research conducted, and the conclusions drawn from the research, the following recommendations are made:

- Continue to require hydrated lime treatment on all state route projects. Mixtures treated with hydrated lime consistently produced higher tensile strength and higher retained tensile strength values even for limestone mixtures. Hydrated lime was the only treatment to maintain at least 80% TSR for up to 10 freeze/thaw combinations.

- Based on results from field cores with an average age of more than 8 years, it is recommended that GDT-66 be revised to include at least one freeze-thaw cycle after vacuum saturation for roadway cores even if they have gone through field environmental conditioning.
- Consideration should be given to use of 5 freeze/thaw cycles for moisture susceptibility testing, especially when the aggregate source has a history of being susceptible to stripping. From the research, 5 and 10 freeze/thaw cycles were significantly more discriminating in regard to stripping than no freeze/thaw cycle or only one freeze/thaw cycle. The results did not show a statistical difference in increasing the number of cycles from 5 to 10 cycles.
- Continue to use tensile strength testing after freeze/thaw conditioning for determining susceptibility to moisture damage. The tensile strength results were more consistent and more definitive for establishing acceptance than the Hamburg Wheel Test results.

Future Research

It is suggested that a follow-up research project be conducted to continue monitoring the SR 319 test sections for long-term performance. Coring and testing the mixtures of a controlled test section after initial construction and again after two years of service produced somewhat conflicting results. Initially, the section treated with hydrated lime produced the highest strength, but the additive treated section also met specification requirements. After two years of service, the WMX section was stronger than the LAS section when no freeze/thaw cycle was used. After subjecting the two-year old project samples to one freeze/thaw cycle, the LAS and WMX samples averaged slightly higher tensile strength than the lime treated section. These results indicate that WMX mixture may have low strength initially, but the strength generally improves with time. The project should continue to be monitored for several years to help better determine effectiveness of the additives for long-term performance.

REFERENCES

1. Kennedy, T., F. Roberts, and K. Lee, “Evaluation of Moisture Effects on Asphalt Concrete Mixtures,” Transportation Research Record 911, TRB/NAS, Washington, D. C., 1983. pp 134-143.
2. Lottman, R., “Predicting Moisture-Induced Damage to Asphaltic Concrete,” NCHRP Report 192, TRB/NAS, Washington, D. C., 1978.
3. Nicholson, V., “Adhesion Tension in Asphalt Pavements, Its Significance and Methods Applicable in Its Determination,” Proceedings, Association of Asphalt Paving Technologists, Vol. 3, 1932.
4. Tunnicliff, D., and R. Root, “Use of Anti-Stripping Additives in Asphaltic Concrete Mixtures: Laboratory Phase,” NCHRP Report 274, TRB/NAS, Washington, D. C., 1984.
5. Taylor, M., and P. Khosla, “Stripping of Asphalt Pavements: State of the Art,” Transportation Research Record 911, TRB/NAS, Washington, D. C., 1983. pp 150-158.
6. Fromm, H., “The Mechanisms of Aasphalt Stripping from Aggregate Surfaces,” Proceedings, Association of Asphalt Paving Technologists, Vol. 43, 1974. pp 191-223.
7. Andersland, O. B., and W. Goetz, “Sonic Test for Evaluation of Stripping Resistance in Compacted Bituminous Mixtures,” Proceedings, Association of Asphalt Paving Technologists, Vol. 25, 1956.
8. Skog, J.,and E. Zube, “New Test Methods for Studying the Effects of Water Action on Bituminous Mixtures,” Proceedings, Association of Asphalt Paving Technologists, Vol. 32, 1963.
9. Field, F., and W. Phang, “Stripping in Asphaltic Concrete Mixes: Observations and Test Procedures,” Proceedings, Canadian Technical Asphalt Association, Vol. 12, 1967.
10. Brown, E., and R. Kuntze, “A Study of Stripping in Asphalt Pavements,” Report RR 177, Department of Transportation and Communications, Ontario, Canada, 1972.
11. Jimenez, R., “Testing for Debonding of Asphalts from Aggregates,” Report AHD-RD-10-123, Research Division of Arizona Highway Department, 1973.
12. Lottman, R., “The Moisture Mechanism that Causes Asphalt Stripping in Asphaltic Concrete Mixtures,” Research Project R-47, Idaho Department of Highways and University of Idaho, Moscow, 1971.
13. Schmidt, R. and P. Graf, “The Effect of Water on the Resilient Modulus of Asphalt Treated Mixes,” Proceedings, Association of Asphalt Paving Technologists, Vol. 41, 1972. pp 118-162.
14. Lottman, R., “Predicting Moisture-Induced Damage to Asphaltic Concrete: Field Evaluation,” NCHRP Report 246, TRB/NAS, Washington, D. C., 1982.
15. Pauls, J. and J. Goode, “Application and Present Status of the Immersion-Compression Test,” Proceedings, Association of Asphalt Paving Technologists, Vol. 16, 1947.
16. Gangopadhyay, N, ”Development of the Split Cylinder Test for the Measurement of Debonding in Water-Saturated Asphalt Concrete,” MSCE Thesis, University of Idaho, Moscow, Idaho, 1971.

17. Petersen, J., H. Plancher, E. Ensley, and G. Miyake, "Chemistry of Asphalt Aggregate Interaction: Relationship with Pavement Moisture-Damage Prediction Test," Transportation Research Record 843, TRB/NAS, Washington, D. C., 1982.

18. Maupin, G., Jr., "Implementation of Stripping Test for Asphaltic Concrete," Transportation Research Record 712, TRB/NAS, Washington, D. C., 1979.

19. ASTM D 4123, "Test Method for Indirect Tension Test for Resilient Modulus of Bituminous Mixtures," ASTM, West Conshohocken, PA, (withdrawn in 2003).

20. Anagnos, J., and T. Kennedy, "Practical Method for Conducting the Indirect Tensile Test," Center for Highway Research, University of Texas at Austin, Research Report 98-10, 1972.

21. Bausano, J., A. Kvasnak, C. Williams, "Development of Simple Performance Test Using Laboratory Test Procedures to Illustrate the Effects of Moisture Damage on Hot Mix Asphalt," Iowa State University. 2006.

22. Epps, J., P. Sebaaly, J. Penaranda, M. Maher, M. McCann, and A. Hand, "Compatibility of a Test for Moisture-Induced Damage with Superpave Volumetric Mix Design," NCHRP Report 444, TRB/NAS, Washington, D. C., 2000.

23. Anderson, D., E. Dukatz, and J. Petersen, "The Effect of Antistrip Additives on the Properties of Asphalt Cement," Proceedings, Association of Asphalt Paving Technologists, Vol. 51, 1982.

24. Kennedy, T., "Use of Hydrated Lime in Asphalt Paving Mixtures," University of Texas at Austin, Prepared for National Lime Association: Bulletin No. 325, 1984, Reprinted in 1987.

25. Petersen, J., H. Plancher, and P. Harnsberger, "Lime Treatment of Asphalt," Western Research Institute, Wyoming, Prepared for National Lime Association. 1987.

26. Liu, M.-J., and T. Kennedy, "Field Evaluation of Stripping and Moisture Damage in Asphalt Pavements Treated with Lime and Antistripping Agents,"Research Report 441-2F, Center for Transportation Research, University of Texas at Austin. 1991.

27. Sebaaly, P., E. Hajj, D. Little, S. Shivakolunthar, T. Sathanathan, and K. Vasconcelos, "Evaluating the Impact of Lime on Pavement Performance," University of Nevada at Reno. 2010.

28. Sebaaly, P., E. Hitti, and D. Weitzel, "Effectiveness of Lime in Hot Mix Asphalt Pavements," Transportation Research Record 1832, TRB/NAS, Washington, D. C., 2003. pp 34-41.

29. Stapler, T., "Update on Georgia's Experience with the Use of Hydrated Lime in Asphaltic Concrete Mixtures," Presented at 43rd Annual SASHTO Meeting, 1984.

30. Watson, D., "Status Report on the Use of Hydrated Lime in Asphaltic Concrete Mixtures in Georgia," Interdepartment Correspondence, GDOT, 1992.

31. "QPL-26, Heat Stable Anti-Stripping Additives," Available at website: http://dot.ga.gov/doingbusiness/Materials/qpl/Documents/qpl26.pdf , June, 2012.

32. "GDT 66 - Evaluating the Moisture Susceptibility of Bituminous Mixtures by Diametral Tensile Splitting," Available at website: http://dot.ga.gov/doingbusiness/TheSource/gdt/gdt066.pdf , June, 2012.

33. Kvasnak, A. , J. Moore, A. Taylor, and B. Prowell, "Preliminary Evaluation of Warm Mix Asphalt Field Demonstration: Franklin, Tennessee," NCAT Report 10-01, Auburn, AL: NCAT, 2010.

34. Texas DOT Standard Specifications, Section 340.4.A. Table 6. Available at http://ftp.dot.state.tx.us/pub/txdot-info/des/specs/specbook.pdf. July, 2012.
35. Texas DOT Construction and Materials Tips, Technical Advisory dated August 16, 2006. Available at http://ftp.dot.state.tx.us/pub/txdot-info/cst/tips/faq_mix_designpt2.pdf. July, 2012.
36. Biligiri, K., K. Kaloush, M. Mamlouk, and M. Witczak. "Rational Modeling of Tertiary Flow for Asphalt Mixtures." Transportation Research Record: Journal of the Transportation Research Baord, 2007: pp. 63-72.
37. Advanced Asphalt Technologies, LLC. NCHRP Report 673: "A Manual for Design of Hot Mix Asphalt with Commentary," Washington, DC: NCHRP, 2011.
38. Bonaquist, R. NCHRP Report 691: "Mix Design Practices for Warm Mix Asphalt," Washington, DC: NCHRP, 2011.

APPENDIX A

Flow Number: Raw Data

Aggregate	Additive	F_T	Flow Number	MS at Flow	Avg. FN	Std. Dev.
Lithia Springs	HL	0	87	16948	72.7	13.2
Lithia Springs	HL	0	61	18234		
Lithia Springs	HL	0	70	17114		
Lithia Springs	LAS	0	31	13388	30.7	2.5
Lithia Springs	LAS	0	33	14166		
Lithia Springs	LAS	0	28	13871		
Lithia Springs	WMX	0	148	16145	97.3	44.0
Lithia Springs	WMX	0	69	16185		
Lithia Springs	WMX	0	75	16726		
Lithia Springs	HL	1	61	19061	61.3	1.5
Lithia Springs	HL	1	60	19826		
Lithia Springs	HL	1	63	18660		
Lithia Springs	LAS	1	29	14115	24.7	4.0
Lithia Springs	LAS	1	24	13733		
Lithia Springs	LAS	1	21	13979		
Lithia Springs	WMX	1	79	18267	73.3	5.5
Lithia Springs	WMX	1	73	16991		
Lithia Springs	WMX	1	68	17066		
Lithonia	HL	0	44	15714	41.3	3.1
Lithonia	HL	0	38	15252		
Lithonia	HL	0	42	14892		
Lithonia	LAS	0	34	13836	39.3	5.5
Lithonia	LAS	0	45	13857		
Lithonia	LAS	0	39	14306		
Lithonia	WMX	0	60	15679	57.7	2.5
Lithonia	WMX	0	55	15692		
Lithonia	WMX	0	58	15618		
Lithonia	HL	1	46	15719	38.7	7.0
Lithonia	HL	1	32	15694		
Lithonia	HL	1	38	15803		
Lithonia	LAS	1	29	13701	30.3	3.2
Lithonia	LAS	1	28	13980		
Lithonia	LAS	1	34	14619		
Lithonia	WMX	1	65	16784	57.7	6.4
Lithonia	WMX	1	55	16548		
Lithonia	WMX	1	53	16367		

Flow Number Summary

Aggregate	Additive	F_T	Avg. Flow Number	St Dev Flow No.	COV (%)
Lithia Springs	HL	0	72.7	13.2	18.2
Lithia Springs	LAS	0	30.7	2.5	8.2
Lithia Springs	WMX	0	97.3	44.0	45.2
Lithia Springs	HL	1	61.3	1.5	2.5
Lithia Springs	LAS	1	24.7	4.0	16.4
Lithia Springs	WMX	1	73.3	5.5	7.5
Lithonia	HL	0	41.3	3.1	7.4
Lithonia	LAS	0	39.3	5.5	14.0
Lithonia	WMX	0	57.7	2.5	4.4
Lithonia	HL	1	38.7	7.0	18.2
Lithonia	LAS	1	30.3	3.2	10.6
Lithonia	WMX	1	57.7	6.4	11.1