COMPARISON OF PELLETIZED LIME WITH OTHER ANTI-STRIPPING ADDITIVES

Final Report

SPR 732

Oregon Department of Transportation

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by
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16. Abstract

Stripping is a common problem in HMA pavements in Oregon, especially in Eastern Oregon. Stripping is the degradation of the bond between the aggregate and the asphalt binder due to the presence of water – this mechanism of degradation can lead to loss of capacity and cracking in the pavement. A common additive used in the industry to mitigate stripping damage is powdered lime. However, challenges with air-borne powdered lime have SHAs investigating alternatives to powdered lime. The purpose of this study was to determine the effectiveness of potential alternatives to powdered lime additive in preventing stripping.

This research evaluated the moisture susceptibility of five anti-stripping additives with three separate aggregates. The aggregates exhibited a range of potential stripping from not susceptible to susceptible. Results indicate that Aggregates 1 and 3 are likely susceptible to stripping, with Aggregate 3 likely being the most susceptible. Powdered lime increased the TSR and ECS ratios for the susceptible aggregates. Mixtures with Additive 4 exhibited similar performance to mixes containing powdered lime. Additive 2 exhibited improved performance compared to the control but TSR and ECS ratios were lower than the specimens with powdered lime. Results from mixtures with Additive 3 exhibited limited improvements in TSR and ECS ratios. Additives 4 and 2 should be considered for future use in HMA when stripping could be an issue.

One practice in ODOT is to inlay HMA pavements 15 years after construction. If the pavement is exhibiting damage resulting from stripping, the inlay can be specified to be 4 inches (102 mm) deep. If the pavement is not exhibiting damage from stripping, the inlay can be specified at 2 inches (51 mm) deep. Using this information, an economic analysis was performed. Other options are available but these were not included in the analysis. The economic analysis indicates that when a reduction in inlay thickness is realized, there is significant value in using additives. The sensitivity analyses indicated that large changes in the input variables do not make the cost of using additive cost ineffective – that is, there is significant value in using additives even when input variables (rate of return, number of future inlays, inlay depth, cost of inlay HMA, original construction cost, and additive cost) change significantly.

17. Key Words	18. Distribution Statement
ANTI-STRIPPING ADDITIVE, HOT-MIX ASPHALT (HMA), POWDERED LIME, TENSILE STRENGTH RATIO (TSR), ENVIRONMENTAL CONDITIONING SYSTEM (ECS), ECONOMIC ANALYSIS	Copies available from NTIS, and online at http://www.oregon.gov/ODOT/TD/TP_RES/

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	millimeters squared	mm^2
ft^2	square feet	0.093	meters squared	m^2
yd^2	square yards	0.836	meters squared	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	kilometers squared	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	ml
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	meters cubed	m^3
yd^3	cubic yards	0.765	meters cubed	m^3
NOTE: Volumes greater than 1000 L shall be shown in m^3.				
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
		TEMPERATURE (exact)		
°F	Fahrenheit	(F-32)/1.8	Celsius	°C

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	millimeters squared	0.0016	square inches	in^2
m^2	meters squared	10.764	square feet	ft^2
m^2	meters squared	1.196	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	kilometers squared	0.386	square miles	mi^2
		VOLUME		
ml	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	meters cubed	35.315	cubic feet	ft^3
m^3	meters cubed	1.308	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams	1.102	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celsius	1.8C+32	Fahrenheit	°F

*SI is the symbol for the International System of Measurement

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DISCLAIMER

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1.0 INTRODUCTION

Stripping is a common problem in hot mix asphalt concrete (HMAC). It is defined as the weakening of the bond between the aggregate and the asphalt binder. Stripping commonly occurs due to the presence of water or moisture in the asphalt pavement, therefore the terms “moisture damage” and “moisture susceptibility” are common ways to describe the problem of stripping.

Stripping often leads to strength loss in the mixture. One of the ways to prevent this is through the use of additives. The most common additive is powdered lime. However, there may be potential health issues associated with air-borne powdered, hydrated lime. The purpose of this study was to determine the effectiveness of potential alternatives to powdered lime additive for HMAC.

Epps and Little reported that it is the aggregate properties that determine the level of moisture susceptibility of the mixture. *(Epps and Little 2001)* Therefore, in this study it was determined that the additives would be tested with different types of aggregates. The initial laboratory study included two different aggregates to evaluate in combination with the additives. A field study was proposed to validate the findings of the laboratory study. However, it was determined that additional laboratory studies would be performed to assess an additional aggregate. The experimental program and laboratory study are described in Chapter 2. Chapter 2 will introduce the test methods that were used in the study, as well as providing information on the different additives.

Chapter 3 contains information on the three aggregates used in this study. It also provides a brief description of other materials used, including the asphalt binder and recycled aggregate pavement (RAP). Mix designs are also provided.

The experimental methods are described in Chapter 4. The batching, mixing, and compaction procedures for each additive are described in this chapter. The laboratory study is then described, including detailed sections on the test methods used for the study. The results of these test methods are provided in Chapter 5. Chapter 6 includes three different economic analyses and Chapter 7 provides a summary of the research findings and conclusions.

2.0 EXPERIMENTAL PROGRAM

This research project included a laboratory study to assess the moisture susceptibility of HMAC mixtures containing different anti-stripping additives. Control specimens that contained no anti-stripping agent were also included in the study. This chapter documents the experimental plan devised to conduct the laboratory study. The experimental plan includes the methods, materials, and mix designs that were used in this research project.

2.1 LABORATORY STUDY

The goal of the laboratory study was to investigate the performance, in terms of moisture susceptibility, of five anti-stripping additives in combination with three coarse aggregate types and two binder types and grades. The moisture susceptibility of these mixes was compared with a control mixture. The five anti-striping additives included a hydrated powdered lime, a pelletized lime (EZ-Lime™), two liquid anti-stripping agent (Zycosoil™ and Zycotherm™), and a polymeric aggregate treatment *(Ultrapave 5000).* Testing for ZycoTherm™ specimens included only TSR testing as this additive was added to the experimental program later in the project. All additives were added to the mixtures mostly in accordance with ODOT TM 316. *(ODOT 2012)* Deviations from the specifications can be found in the methods section. The total numbers of mixes investigated in the laboratory study was sixteen.

Because there were no existing standards at the onset of this project for mixing pelletized lime, a preliminary study was conducted to define how to add this additive. The three different methods investigated included:

1. Mixing the pelletized lime with the aggregate prior to the addition of the asphalt binder;

2. Mixing the pelletized lime with the asphalt binder prior to mixing with the aggregate, and;

3. Mixing the pelletized lime with the wet aggregate, drying, and then mixing.

From the preliminary testing program it was determined that mixing the pelletized lime, water, the aggregate and then heating the mix to remove the water was the most practical method for distributing the lime throughout the mix. Subsequent literature by the manufacturer indicates that the EZ-Lime™ can be added directly to the aggregate prior to adding the asphalt binder.

To evaluate the moisture susceptibility of the five anti-stripping additives, AASHTO T 283, *Standard Method of Test for Resistance of Compacted Asphalt Mixtures to Moisture-Induced Damage* and the ECS/Dynamic Modulus (ECS/E*) test outlined in the NCHRP Report 589 were performed. AASHTO T 283 requires six specimens per mixture be tested. Three specimens per mixture were evaluated for the ECS/E* test. Table 2.1 shows the experimental design for the laboratory study listing the number of specimens per test method.

For the laboratory study, the control group was mixed with no additives, Additive #1 was powdered lime, Additive #2 was EZ-Lime™, Additive #3 was Ultrapave-5000, Additive #4 was Zycosoil™, and Additive #5 was ZycoTherm™.

Table 2.1: Experiment Design for the Laboratory Study

	Number of Test Specimens					
	AASHTO T 283			**ECS/E***		
Aggregate Type / Additive	Aggregate #1	Aggregate #2	Aggregate #3	Aggregate #1	Aggregate #2	Aggregate #3
Control	6	6	6	3	3	3
Additive 1	6	6	6	3	3	3
Additive 2	6	6	6	3	3	3
Additive 3	6	6	6	3	3	3
Additive 4	6	6	6	3	3	3
Additive 5	0	0	6	0	0	0

2.1.1 AASHTO T 283 Test

AASHTO T 283 is a combination of both the Lottman and Tunnicliff and Root tests. *(Lottman 1982; Tunnicliff and Root 1984)* It is the most widely used test used to evaluate moisture susceptibility in HMAC. In this test the splitting tensile strength of unconditioned specimens is compared to the splitting tensile strength of partially saturated, conditioned specimens. The conditioned specimens are subjected to a warm water cycle for 24 hours. This test was chosen to evaluate the performance of the four different anti-stripping additives as it is the most widely used test procedure adopted by highway agencies. A brief description of the test can be found in the methods section.

While several studies on the moisture susceptibility of aggregate in HMAC have been conducted, there is no standard on when an aggregate is considered to be non-susceptible. In his research, Lottman *(Lottman 1982*) specifies a minimum tensile strength ratio (TSR) of 0.70 for an aggregate to be considered non-susceptible. Maupin reported that values of 0.70 to 0.75 differentiated non-susceptible and susceptible aggregates. *(Maupin 1982)* Tunnicliff and Root (*Tunnicliff and Root 1984*) found values ranging from 0.70 to 0.80 to be indicators of non-susceptibility. Although the different research used different conditioning, the inconsistency in values makes it difficult to determine if an aggregate is susceptible or non-susceptible to moisture damage.

2.1.2 ECS/Dynamic Modulus (ECS/E*) Test

Epps and Little (2001) reported that AASHTO T 283 contains some drawbacks. This conclusion was reinforced by research that included five different mixtures from various states in which the results of the laboratory study did not match the results of the field study. To achieve a more accurate evaluation of moisture susceptibility, the ECS/E* test was developed. In the NCHRP Project 1-37 A, "*Development of the 2002 Guide for the Design of New and Rehabilitated Pavement Structures: Phase II*," the dynamic modulus (E*) is recommended as the main material characterization test of HMAC as it links both mixture design and structural design. *(NCHRP 2004)* For this purpose the test was chosen to evaluate moisture susceptibility. In this test, E* of the specimens is determined at three different conditions: dry and unconditioned, static saturated, and load conditioned. In the load conditioned phase of the testing, the specimen is exposed to a repeated haversine wave loading for 18 hours as well as a constant flow of water through the actual specimen. This is done to simulate traffic and environmental conditions. The E* is measured under different loads at different frequencies while the dynamic strain is kept within a fairly constant range of 75 to 125 μstrain (με). The E* is then compared between the three conditions of each specimen to evaluate the resistance to moisture damage. A brief description of the test can be found in the methods section.

3.0 MATERIALS AND MIXTURES

The materials used in this research consisted of virgin aggregate, recycled asphalt pavement (RAP), McCall PG 64-22 and PG 64-28 grade binders, and the anti-stripping additives. PG 64-22 binder was used for aggregate #1 mixtures, while PG 64-28 was used for aggregate #2 and aggregate #3 mixtures. The mixing temperature range for the PG 64-28 is 310 to 322 °F (154 to 161°C); the compaction temperature range is 289 to 298 °F (143 to 148°C). The mixing temperature range for the PG 64-22 is 307 to 318 °F (152 to 159°C); the compaction temperature range is 289 to 297 °F (142 to 147°C). The virgin aggregates (aggregates 1, 2, and 3) were procured from three different sources in Oregon. Knife River of Corvallis, OR provided the RAP for all mixes.

3.1 MATERIAL GRADATION

To meet the required mix design criteria, the aggregate was separated from the three stockpile sizes into the following sieve sizes: ½ in. (12.5 mm), 3/8 in. (9.5 mm), ¼ in. (6.3 mm), #4 (4.75 mm), #8 (2.36 mm), #16 (1.18 mm), #30 (0.600 mm), #50 (0.300 mm), #100 (0.150 mm), #200 (0.075 mm), and minus #200 (0.075 mm). The sieving technique used meets the sieving standards of AASHTO T 27, *Sieve Analysis of Fine and Coarse Aggregates*. *(ASSHTO 2012b)* A sieve analysis was performed to calculate the percentage of fines (minus #200) retained in other sieve sizes. This was done according to AASHTO T 11, *Materials Finer than No. 200 (75µm) Sieve in Mineral Aggregates by Washing*. *(AASHTO 2012a)*The information regarding the amount of fines retained on other sieve sizes was provided by Knife River.

3.2 AGGREGATE

The following section provides a description of the aggregates used in this research program. All aggregates were sieved and recombined to meet the target gradations (Tables 3.3-3.5).

3.2.1 Aggregate #1

Aggregate #1 is a crushed river-run rock that is separated into three stockpile sizes: ½ in. to #4 (12.5 mm to 4.75 mm), #4 to #8 (4.75 mm to 2.36 mm), and #8 to 0 (2.36 mm to 0 mm). These materials were procured from a temporary Quarry near Santiam Pass, OR. Table 3.1 shows the specific gravity data for Aggregate #1.

Table 3.1: Specific Gravity Information for Aggregate#1

Aggregate #1	**Stockpile Sizes**		
	½ in. to #4 (12.5 mm to 4.75 mm)	#4 to #8 (4.75 mm to 2.36 mm)	#8 to 0 (2.36 mm to 0 mm)
Bulk Specific Gravity (G_{sb})	2.639	2.637	2.576
Apparent Specific Gravity (G_{sa})	2.731	2.739	2.755

3.2.2 Aggregate #2

Aggregate #2 was procured from a site in Oregon and stockpiled in three different piles: ½ in. to #4 (12.5 mm to 4.75 mm), #4 to #8 (4.75 mm to 2.36 mm), and #8 to 0 (2.36 mm to 0 mm). The aggregate information is shown in Table 3.2.

Table 3.2: Specific Gravity Information for Aggregate #2

Aggregate #1	**Stockpile Sizes**		
	½ in. to #4 (12.5 mm to 4.75 mm)	#4 to #8 (4.75 mm to 2.36 mm)	#8 to 0 (2.36 mm to 0 mm)
Bulk Specific Gravity (G_{sb})	2.608	2.555	2.579
Apparent Specific Gravity (G_{sa})	2.747	2.728	2.733

3.2.3 Aggregate #3

This material was provided by the same quarry as Aggregate #2. The aggregate came from one single pile and had a bulk specific gravity (G_{sb}) of 2.456. This aggregate is typically not used for HMA and was considered to be highly susceptible to moisture damage and was chosen as a comparison to the two other aggregates evaluated.

3.2.4 Recycled Asphalt Pavement (RAP)

The recycled asphalt pavement (RAP) used for this project was provided by Knife River of Corvallis, OR. For all three mixes, the binder in the RAP was subtracted from the total asphalt content. The RAP was sieved and separated into ½ in. (12.5 mm), 3/8 in. (9.5 mm), ¼ in. (6.3 mm), #4 (4.75 mm), #8 (2.36 mm), #16 (1.18 mm), #30 (0.600 mm), #50 (0.300 mm), #100 (0.150 mm), #200 (0.075 mm), and minus #200 (0.075 mm) sizes and recombined to meet the required gradation. The G_{sb} for the RAP was reported to be 2.713 and the G_{sa} was reported to be 2.790.

3.2.5 Mix Designs

The mix design for the aggregates was provided by the suppliers and approved by ODOT personnel. Each mixture consisted of approximately 70 percent virgin aggregate and 30 percent RAP. Table 3.3 shows the gradation targets for Aggregate #1, Table 3.4 shows the gradation targets for Aggregate #2 and #3. The aggregate gradation for the RAP is shown in Table 3.5.

Table 3.3: Mix Design for Aggregate #1

Aggregate Gradation and Asphalt Content Targets	
Sieve Sizes	**Percent Passing**
(3/4 in.) 19.0 mm	100%
(1/2 in.) 12.5 mm	95%
(3/8 in.) 9.5 mm	81%
(1/4 in.) 6.3 mm	59%
(#4) 4.75 mm	46%
(#8) 2.36 mm	28%
(#16) 1.18 mm	19%
(#30) 0.600 mm	14%
(#50) 0.300 mm	11%
(#100) 0.150 mm	9%
(#200) 0.075 mm	6.9%
Total Asphalt Content (%)	5.8%

Table 3.4: Mix Design for Aggregate #2 and 3

Aggregate Gradation and Asphalt Content Targets	
Sieve Sizes	**Percent Passing**
(3/4 in.) 19.0 mm	100%
(1/2 in.) 12.5 mm	98%
(3/8 in.) 9.5 mm	85%
(1/4 in.) 6.3 mm	66%
(#4) 4.75 mm	58%
(#8) 2.36 mm	40%
(#16) 1.18 mm	26%
(#30) 0.600 mm	18%
(#50) 0.300 mm	12%
(#100) 0.150 mm	9%
(#200) 0.075 mm	6.8%
Total Asphalt Content (%)	5.8%

Table 3.5: Aggregate Gradation for RAP

Aggregate Gradation and Asphalt Content Targets	
Sieve Sizes	**Percent Passing**
(1/2") 12.5 mm	100%
(3/8") 9.5 mm	95%
(1/4") 6.3 mm	80%
(#4) 4.75 mm	69%
(#8) 2.36 mm	47%
(#16) 1.18 mm	32%
(#30) 0.600 mm	23%
(#50) 0.300 mm	18%
(#100) 0.150 mm	14%
(#200) 0.075 mm	10%

4.0 EXPERIMENTAL METHODS

This section provides details on fabrication and test methods. Also included in this section is the methodology for the batching process.

4.1 BATCHING PROCEDURE

Batching for each aggregate mixture was the same for each additive. Additives #1 and #2 were mixed according to the ODOT TM 316 procedures with minor modifications. *(ODOT 2012)* The procedures call for the additives to be mixed with the dry aggregate, but for this study the additives were mixed with aggregate that was already moist. Additives #3, #4, and #5 were mixed according to the manufacturer's instructions. RAP was batched separately from the virgin aggregate according to the mix design. The virgin aggregate batching procedure for each additive is described next.

Additive #1

1. The virgin aggregate was batched according to the mix design, excluding pan material. Additive #1 was measured to 1% of the aggregate weight, and replaced on a 1 to 1 basis with the pan material;

2. Water was weighed to 3% of virgin aggregate weight;

3. The virgin aggregate and water were then placed in a mixing container and mixed for 2 minutes in a rotating mixer;

4. Additive #1 was weighed to 1% of aggregate weight;

5. Additional water was weighed to 2% of aggregate weight;

6. The additive and water were then added to the mixing container and mixed for an additional 2 minutes;

7. The aggregate and Additive #1 combinations were heated to 320 °F (160 °C) until the water evaporated and the aggregate came to mixing temperature.

Additive #2

The batching procedure for Additive #2 followed the same procedure as Additive #1.

Additive #3

1. The virgin aggregate was batched according to the mix design;

2. Water was added to the aggregate (approximately 3% of the aggregate weight);

3. The aggregate and water mixture was placed in a mixing container and mixed for 2 minutes in a rotating mixer;

4. The additive was weighed to 1% of virgin aggregate weight;

5. As the mixer was rotating, the additive was added slowly to the virgin aggregate over a period of 2 minutes;

6. The material was placed in a furnace at 320 °F (160 °C) until the aggregate was dry and at mixing temperature.

Additive #4

1. The virgin aggregate was batched according to the mix design;

2. The additive was diluted by making a solution of 1 part additive to 10 parts ethanol by weight;

3. The binder used for one day of mixing was weighed in a glass beaker. The additive and ethanol solution was added at 1% of the binder weight;

4. After the solution was fully incorporated into the binder, the binder was placed back in the can and placed in a furnace at 320 °F (160 °C) until the binder-additive achieved mixing temperature. Any binder left after mixing for the day was discarded.

Additive #5

1. A solution of 2.2 lbs. (1 kg) Additive #5 to 400 liters of water is mixed;

2. The solution is weighed to 5 percent of the virgin aggregate weight;

3. The solution and aggregate are mixed for 2 minutes in a mixing container.

4. After mixing, the aggregate is placed in a 320 °F (160 °C) furnace until it reaches mixing temperature.

4.2 MIXING PROCEDURE

The first step in the mixing procedure was to bring all the tools used for mixing to the mixing temperature of 320 °F (160 °C). The tools included a metal mixing container, a spoon, a spatula, a blade to scrape the side of the mixing container while spinning, a scale with a resolution to 0.04 oz. (1 gram). All tools except the scale were kept in the same furnace as the batched aggregate and RAP.

To begin the mixing process, the virgin aggregate was carefully placed in the mixing container, followed by the RAP. The spatula was used to scrape off any aggregate or RAP stuck to the pan. The container was then placed back on the scale and the weight of the combined virgin aggregate and RAP was recorded. Based on the asphalt content of the RAP used in this project (6%), and actual asphalt binder content of the mix design, an asphalt binder content of 4.3 percent was used for all mixtures. The container and aggregate mixture was tared on the scale and a spoon was used to create a well in the aggregate. The binder was poured into the well to the required weight. The container was then placed in the rotating mixer and mixed for three minutes. While mixing, the blade was used to scrape the sides of the container preventing the mixture from collecting on the sides. Additionally, the spoon was used to optimize homogeneity. After three minutes of mixing, a pan was tared on the scale and the asphalt mixture was placed into the pan while using the spoon to ensure the mixture was spread evenly (i.e., to prevent segregation). The weight of the mixture was recorded and the mixture was placed in a furnace to cure for a period of two hours at 297 °F (147 °C). The necessary compaction tools (steel compaction molds and lids, a spatula, and a funnel) were also placed in the furnace.

4.3 COMPACTION PROCEDURE

Compaction was performed using a gyratory compactor. After the asphalt mixture had cured for the required two hours, the steel mold was removed from the furnace. A circular sheet of laminated paper was placed in the mold to prevent the mixture from sticking. The mold was then placed on the scale and tared. The mold was removed from the scale and the funnel was inserted into the mold. Using the spatula, the asphalt mixture was carefully placed into the funnel (this was done to prevent segregation and to reduce variability in air content). The weights of the mold and mixture were then recorded. Another sheet of paper was then placed on top, followed by the steel mold lid. At this point the compaction height was determined using the formula

$$\textbf{Compaction ht.} = \frac{\text{HMA weight} \times (1 + \%air)}{0.997 * 17.67 * Gmm} \qquad \textbf{(Eq. 1)}$$

where *Gmm* is the theoretical maximum specific gravity determined according to AASHTO T 209, *Theoretical Maximum Specific Gravity and Density of Bituminous Paving Mixtures.* (*AASHTO 2012d)* The required percent air content for the ECS specimens is 7 ±1 percent according to the NCHRP Report 589, and 7 ±0.5 percent for AASHTO T 283 specimens. *(AASHTO 2012e;NCHRP 2007)* The actual air content in

the specimens is lower than the number that is inputted into the compaction height equation and correlations needed to be developed. The actual air percentages were estimated by mixing several test samples with varying air content inputs. After mixing, the actual air content was determined for each specimen and plotted against the input air content. From here it was determined which input for the compaction height equation would result in an actual air content that met the requirements. Figure 4.1 shows an example plot used to achieve proper air content.

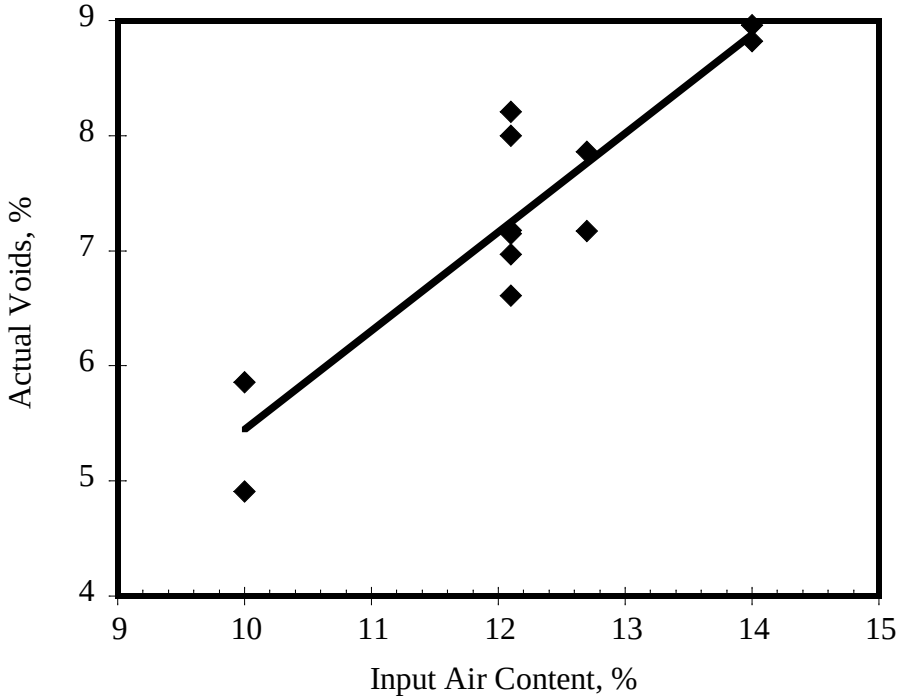

Figure 4.1: Sample Graph Used for Determining Actual Air Voids

This process was repeated for all mixtures. Once the input air content was determined, the same value was used to determine the compaction height for all specimens using that particular aggregate (i.e., additive type did not affect compaction and result air content). After determining the compaction height, the sample was placed in a gyratory compactor. An output of the compactor is the specimen height. Once the reading reached the calculated height, the machine was stopped.

4.4 VOLUMETRIC PROPERTIES

After compaction, the specimens were left in the lab to cool for approximately two hours. The specimens were then removed from the molds using a hydraulic jack. The specimens were then left in the lab overnight before beginning the cutting and coring process. The coring for all specimens was performed using a 4 in. (101.6 mm) diameter coring bit. After they were cored, the specimens were cut to the required length using a diamond saw. The length of specimens used for ECS testing was 6 in. (152.4 mm), while the

specimens for AASHTO T 283 testing were cut to a length of 2.5 in. (63.5 mm). After the specimens were cored and cut they were placed in a 70 °F (21 °C) furnace to dry for at least four hours and then left in the lab overnight.

All samples that were cut and cored were evaluated for bulk specific gravity. This was completed according to AASHTO T 166, *Bulk Specific Gravity of Compacted Asphalt Mixtures Using Saturated Surface-Dry Specimens. (AASHTO 2012c)* The theoretical maximum specific gravity was determined for the control mixture and this value was used for all mixtures of that aggregate. This was performed to determine the air voids in the sample and was completed according to AASHTO T 209. The actual air voids were calculated using the following:

$$\textbf{Percent air void} = \left[1 - \left(\frac{A}{B}\right)\right] \times 100 \qquad \textbf{(Eq. 2)}$$

where *A* is the bulk specific gravity of each specimen and *B* is the theoretical maximum specific gravity of the mixture determined for the control mixture. Table 4.1 shows the theoretical maximum specific gravity for each mixture.

Table 4.1: Theoretical Maximum Specific Gravity of Aggregate Mixtures

Mixture with:	**Gmm**
Aggregate #1	2.469
Aggregate #2	2.437
Aggregate #3	2.456

4.5 LABORATORY STUDY

The laboratory study consisted of evaluating the performance of four anti-stripping additives in conjunction with three different coarse aggregates. The performance of mixtures containing these additives and the control mixtures were assessed using two tests. The first test provided information on the tensile strength such that the tensile strength ratio (TSR) could be determined. The second test assessed the dynamic modulus (ECS/E*) and the assessment included using dynamic modulus ratios to determine moisture susceptibility. This test is outlined in the NCHRP Report 589. This section of the report describes the testing procedures, while Chapter 5 will provide test results.

A fifth additive was added late into the study and was limited to the use of Aggregate #3 and AASHTO T 283.

4.6 AASHTO T 283 TESTING

This test requires six 2.5 in. (63.5 mm) specimens for each additive aggregate combination. Three specimens from each aggregate-additive combination were tested in an unconditioned state while three were tested after being conditioned. Specimens were

selected so the average air void content of the unconditioned and conditioned groups was roughly equal.

4.6.1 Specimen Conditioning

To begin the conditioning process, the unconditioned specimens were set aside while the conditioned group was prepared. The first step in the conditioning procedure was to saturate each of the specimens to a saturation of 70 to 80 percent. This was achieved by using a vacuum to apply 10 to 26 in. Hg (254 to 660 mm Hg) of pressure to the specimen submerged in distilled water at 77 °F (25 °C). The saturation percentage was calculated using the following formula:

$$\mathbf{S = \frac{X}{Y}} \qquad \textbf{(Eq. 3)}$$

where

$$\mathbf{X = \frac{Saturated\ Wt.\ of\ specimen - Dry\ wt.\ of\ specimen}{\rho}} \qquad \textbf{(Eq. 4)}$$

and

$$\mathbf{Y = \frac{(Dry\ Wt.\ of\ specimen - Submerged\ wt.\ of\ specimen) * C}{\rho}} \qquad \textbf{(Eq. 5)}$$

where ρ is the density of water at 77 °F (25 °C), in lbs/in^3 (g/cm^3), and C is the air content of the specimen using Equation 2. The specimen's dry weight, saturated weight, and submerged weight were determined as part of AASHTO T 166 testing.

Following ODOT's test modification, once the specimens were 70 to 80% saturated they were put in a 140 °F (60 °C) water bath for 24 ±1 hour. After 24 hours in the 140 °F (60 °C) water bath the specimens were moved to a water bath at 77 °F (25 °C) for 2 hours. At this time the unconditioned specimens were sealed in a water tight container and placed in the same 77 °F (25 °C) water bath for 2 hours.

4.6.2 Test Procedure

To determine the tensile strength, each specimen was removed from the 77 °F (25 °C) water bath and placed between the steel loading strips with the load applied along the longitudinal axis of the specimen. The load was applied at a constant rate of 2 in. (5.08 cm) per minute in accordance with the AASHTO T 283 testing instructions. The maximum compressive load was recorded and the load was continued to be applied until a vertical crack appeared.

4.6.3 Data Calculation

The tensile strength was calculated according to the following equation as outlined in AASHTO T 283:

$$S_t \text{ (psi)} = \frac{2 \times P}{\pi \times t \times D} \quad \text{(Eq. 6)}$$

and

$$S_t \text{ (kPa)} = \frac{2000 \times P}{\pi \times t \times D} \quad \text{(Eq. 7)}$$

where P is the maximum compressive load (pounds, Newtons), t is the specimen thickness (2.5 in. [63.5 mm]), and D is the specimen diameter (4 in. [101.6 mm]). After all specimens were tested for the dry (unconditioned) and conditioned states, the tensile strength ratio (TSR) was calculated as follows:

$$\text{TSR} = \frac{S_2}{S_1} \quad \text{(Eq. 8)}$$

with S_1 being the average tensile strength of the unconditioned group and S_2 the average tensile strength of the conditioned group.

4.7 ECS/DYNAMIC MODULUS (ECS/E*) TESTING

ECS/E* testing was performed according to the procedure outlined in the NCHRP Report 589. Three specimens for each aggregate-additive combination were tested and assessed. Each specimen was tested under three conditions: dry, after static saturation, and after water and load conditioning in the ECS chamber. For each condition, E* was measured at four frequencies: 5, 2, 1, and 0.5 Hz. In total, 540 data files were collected and assessed. All testing was conducted at 77 °F (25 °C) as per the NCHRP report. After testing, E* was calculated and the ratios between the three conditions were used to assess the degree of moisture damage and aggregate susceptibility within the specimen. Table 4.2 shows the parameters used for each phase.

Table 4.2: Parameters for ECS Testing

Phase		Temperature	Duration	Load/Vacuum	Loading
Unconditioned		77 °F (25 °C)	-	-	-
Static Saturation		77 °F (25 °C)	30 min	25 in. (65 mm) Hg	-
Conditioned	Temperature	140 °F (60 °C)	18 h	-	-
	Load	-	18 h	115 lbs.	Haversine (0.1 sec loading, 0.9 sec rest
	Vacuum	-	18 h	5 in. (100 mm) Hg	-
	Water flow	140 °F (60 °C)	18 h	0.5 $in.^3$ (8 cm^3) /min	-

4.7.1 Test Setup

The test setup consisted of the ECS testing machine and two Linear Variable Differential Transducers (LVDTs). The ECS machine is comprised of a temperature controlled chamber, a loading system consisting of a pneumatic pressurized system with a servo valve controlling the amount of load, a load cell, and a vacuum and water system used to attain a continuous flow of water through the specimen during the conditioning phase. The two LVDTs measured the deformation of the specimen during loading across a gage length of 2 in. (50.8 mm). During testing, the specimen was encased in a rubber membrane. Two perforated, plastic circular plates were placed between the specimen and the base plate on the bottom, and between the specimen and load cell on top. These plates ensured a proper seal when the vacuum was applied. In addition, for the conditioned portion of the testing a thin ring of silicone caulking material was placed on the outside of the specimen near the top and bottom. These measures were implemented to seal the specimen, and ensure the water flowed through the specimen during the conditioning phase. Figure 4.2 shows a sample specimen set-up in the ECS chamber.

Figure 4.2: Set-up for ECS Testing

4.7.2 Unconditioned Testing

Prior to loading, the specimen was put through a preliminary test to estimate the load that would result in a strain of 75 to 125 με. In the preliminary test, a small load was applied for all four frequencies and the strain was documented. From these loads and their respective strain, linear extrapolation was used to determine the load that would result in the recommended strain (75 to 125 με). Once the required loads were determined, the specimen was tested.

4.7.3 Static Saturation

After the dry, unconditioned testing was complete, the specimen was removed from the testing chamber. The specimen was then submerged in a 77 °F (25 °C) water bath. A vacuum was applied to the specimen at a pressure of 25 in. (625 mm) of Hg for 30 minutes. After 30 minutes the specimen surface was dried, silicone was placed at the top and bottom side edges and the membrane was placed over the specimen. The specimen was then set-up in the testing chamber. Once again a preliminary estimate was performed to ensure the proper strain was achieved. The specimen was then tested using the same procedure.

4.7.4 Conditioned Testing

Following testing for static saturation, the specimen was removed from the testing chamber. A thin layer of vacuum grease was applied between the specimen and the

perforated plates at the bottom and top. The specimen was placed back in the ECS chamber and the chamber door was closed. The chamber was set to 140 °F (60 °C), with the vacuum set at 5 in. (100 mm) of Hg of pressure. A water valve was opened to attain a constant flow of 0.5 in.^{3} (8 cm^3) per minute through the specimen, and the loading program was started. The loading program applied a haversine load to the specimen. Each pulse had a duration of 0.1 seconds followed by a rest period of 0.9 seconds. After 18 hours of testing, the load was discontinued, the water valve was closed, and the vacuum pressure turned off. The ECS chamber temperature was lowered to 59 °F (15 °C) for 1 hour and then to 77 °F (25 °C) for another 1.5 hours following the recommendations outlined in the NCHRP Report 589 (note that the surface temperature reached 77 °F (25 °C) in a relatively short period and the temperature at the specimen center was not determined; it was assumed that the center of the specimen reached 77 °F (25 °C) as noted in the NCHRP Report 589).

4.7.5 Data Analysis

To begin the data analysis, each file was formatted so that the stress and strain for each data point was displayed. The loading program recorded one data point every millisecond. The strain was determined by averaging the two deformations measured by the LVDTs and dividing by the gage length of 2 in. (50.8 mm). The data were then graphed. Figure 4.3 shows a sample test at a frequency of 1 Hz.

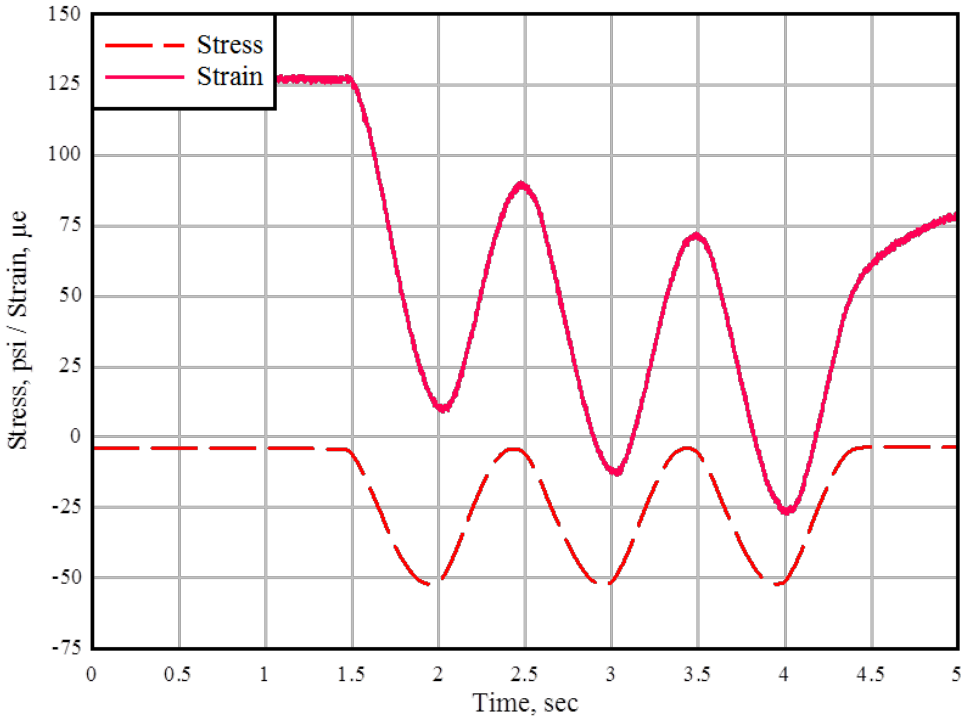

Figure 4.3: Sample Stress-Strain Graph for 1 Hz Testing

To determine the E* for each frequency, the average stress and strain were first determined. This was accomplished by finding the peak to peak stress and strain values of each cycle for that frequency. To do this, the minimum peak value of each wave was subtracted from the prior maximum peak value (see Figure 4.4). An average E* was determined from all data points from each specimen and frequency.

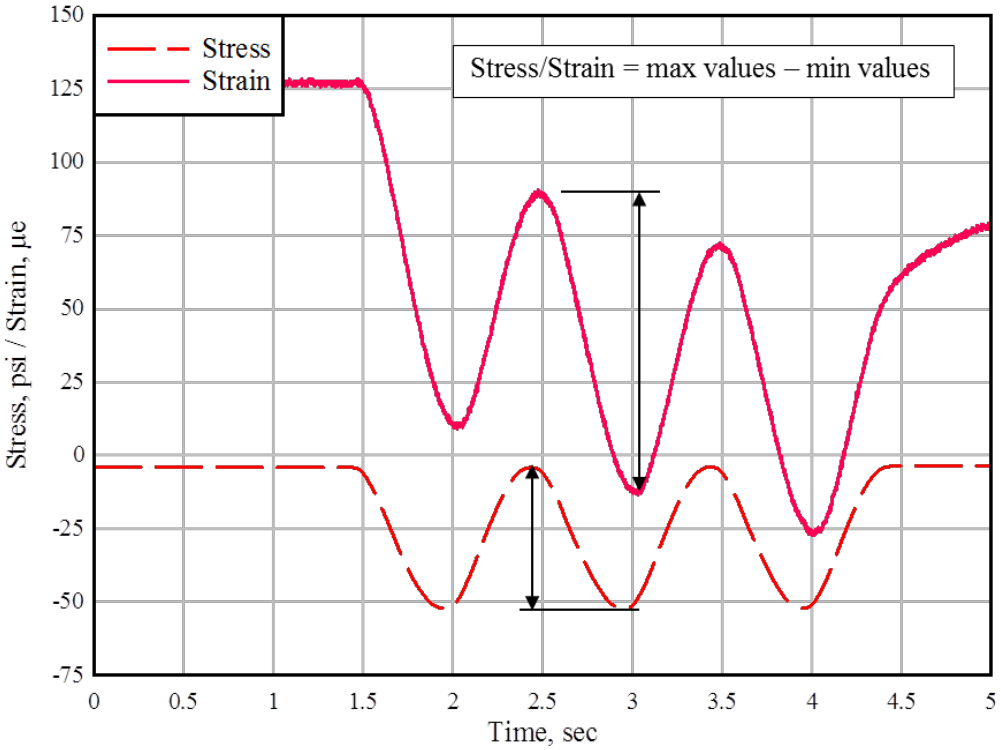

Figure 4.4: Determining Stress and Strain Values

5.0 RESULTS

5.1 AASHTO T 283

The results of AASHTO T 283 testing were used to compare each additive's average TSR to the average TSR of the control group. An increase in average TSR implies that the additive is effective in decreasing the moisture susceptibility of the specimens. In addition, the average TSR of the control group can be an indicator of the general moisture susceptibility of that aggregate. A higher TSR value for the control group means the aggregate has low moisture susceptibility, while a lower TSR value indicates the aggregate is prone to moisture damage (i.e., stripping).

5.1.1 Mixture with Aggregate #1

Table 5.1 shows the results of AASHTO T 283 testing for the specimens containing Aggregate #1. The initial TSR value of the specimen without additives was 0.78. According to the ranges provided by Lottman (*Lottman 1982*) and Maupin (*Maupin 1982*) this mixture would be considered non-susceptible. The values reported by Tunnicliff and Root (*Tunnicliff and Root 1982*), as typically used more by ODOT, indicates the mixture is possibly susceptible. The two additives that exhibited the largest increase in TSR values were Additive #1 and Additive #4. Both additives increased the average TSR value of the control group by about 15 percent. Additive #2 produced a small increase in performance, while Additive #3 slightly decreased the average TSR value, although statistically insignificant. Figure 5.1 shows the TSR values for the mixtures containing Aggregate #1, and Figure 5.2 shows the TSR values of each additive normalized to the control group.

Table 5.1: AASHTO T 283 Testing Results for Mixture #1

Aggregate Name	Additive	Dry	Conditioned	TSR	Normalized to Control
		Avg. Tensile Strength, psi (kPa)	Avg. Tensile Strength, psi (kPa)		
Aggregate #1	Control (C)	228 (1572)	176 (1213)	0.78	1.00
	Additive 1	189 (1303)	169 (1165)	0.89	1.15
	Additive 2	223 (1538)	175 (1207)	0.79	1.01
	Additive 3	247 (1703)	190 (1310)	0.77	0.99
	Additive 4	228 (1572)	202 (1393)	0.89	1.14

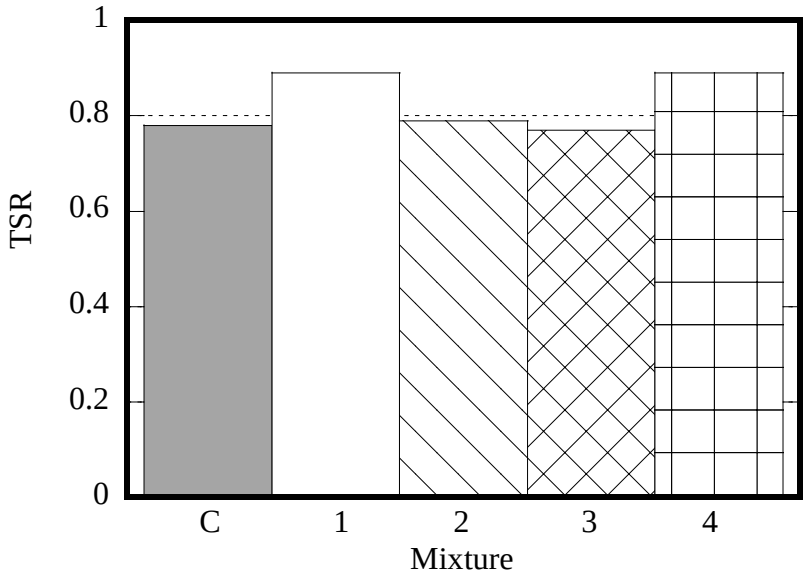

Figure 5.1: TSR Values for Mixtures Containing Aggregate #1

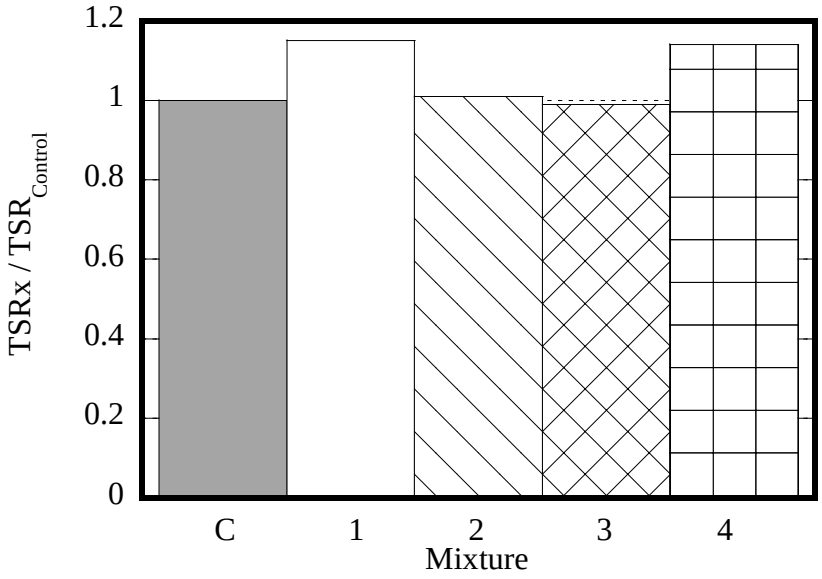

Figure 5.2: Normalized TSR Values for Mixtures Containing Aggregate #1

5.1.2 Mixture with Aggregate #2

Table 5.2 shows the results for specimens mixed with Aggregate #2. The average TSR for the control specimens was 0.87. This indicates that the aggregate is not susceptible to moisture and stripping according to Lottman (*Lottman 1982*), Maupin (*Maupin 1982*), and Tunnicliff and Root (*Tunnicliff and Root 1984*). Additive #1 also shows the highest increase in TSR values. Additive #2 also proved to be effective in increasing the TSR,

while both Additive #3 and Additive #4 exhibited a decrease in TSR values. Figure 5.3 shows the TSR values for the mixtures containing Aggregate #2. Figure 5.4 shows the normalized control comparison.

Table 5.2: AASHTO T 283 Testing Results for Mixture #2

Aggregate Name	Additive	Dry	Conditioned	TSR	Normalized to Control
		Avg. Tensile Strength, psi (kPa)	Avg. Tensile Strength, psi (kPa)		
Aggregate #2	Control (C)	212 (1462)	185 (1276)	0.87	1.00
	Additive 1	206 (1420)	202 (1393)	0.98	1.12
	Additive 2	207 (1427)	222 (1531)	0.93	1.07
	Additive 3	186 (1282)	234 (1613)	0.79	0.91
	Additive 4	212 (1462)	182 (1255)	0.86	0.98

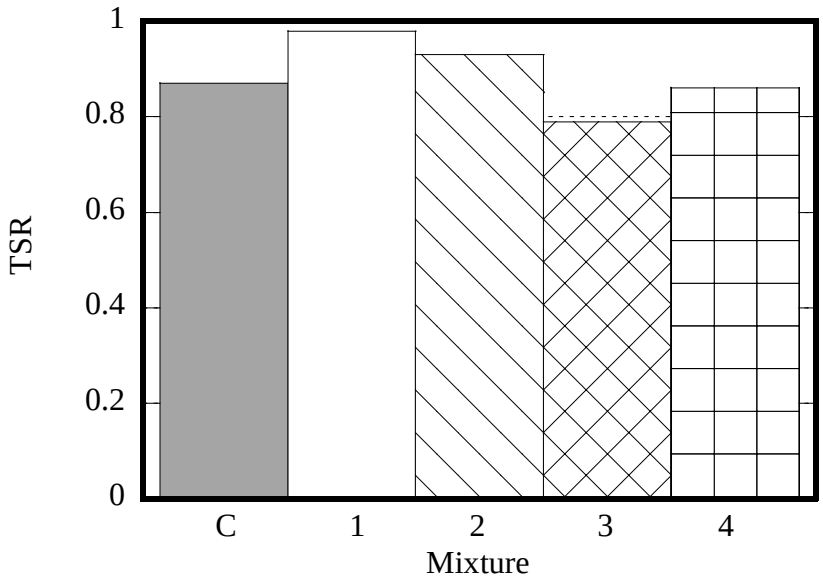

Figure 5.3: TSR Values for Mixtures Containing Aggregate #2

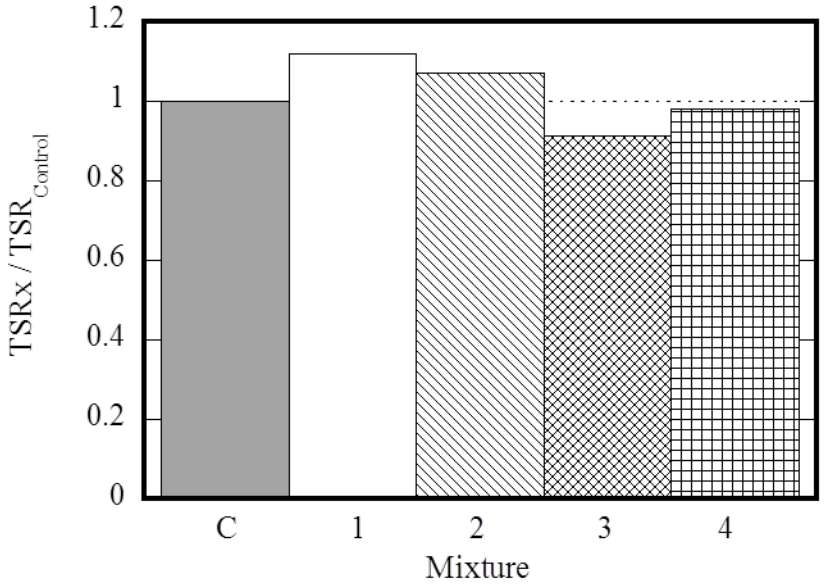

Figure 5.4: Normalized TSR Values for Mixtures Containing Aggregate #2

5.1.3 Mixture with Aggregate #3

The results in Table 5.3 show that the TSR value is lower for the control group of Mixture #3. This aggregate was specifically chosen as a way to gauge the performance of the additives when mixed with an aggregate that potentially exhibits high moisture susceptibility. For this mixture, Additive #2 appeared to be the most effective in increasing the TSR value (increase of 28 percent), followed by Additive #1 with a 27 percent increase. Additive #4 provided similar results at a 23 percent increase in TSR. Additive #3 only provided a slight increase. Figure 5.5 shows the TSR results for Mixture #3. Figure 5.6 shows all values normalized to the control group.

Table 5.3: AASHTO T 283 Testing Results for Mixture #3

Aggregate Name	Additive	Dry	Conditioned	TSR	Normalized to Control
		Avg. Tensile Strength, psi (kPa)	Avg. Tensile Strength, psi (kPa)		
Aggregate #3	Control (C)	230 (1586)	131 (903)	0.57	1.00
	Additive 1	259 (1786)	187 (1289)	0.72	1.27
	Additive 2	268 (1848)	196 (1351)	0.73	1.28
	Additive 3	244 (1682)	143 (986)	0.59	1.03
	Additive 4	244 (1682)	172 (1186)	0.70	1.23
	Additive 5	227 (1565)	112 (772)	0.49	0.86

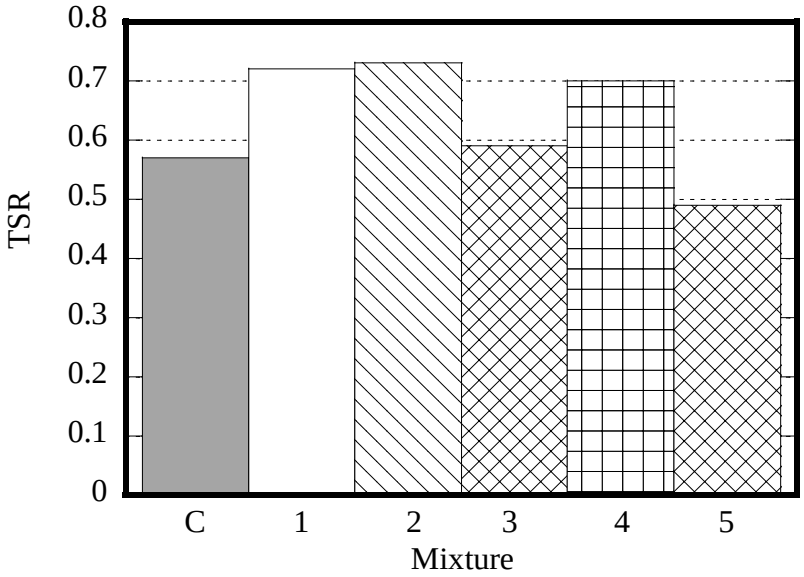

Figure 5.5: TSR Values for Mixtures Containing Aggregate #3

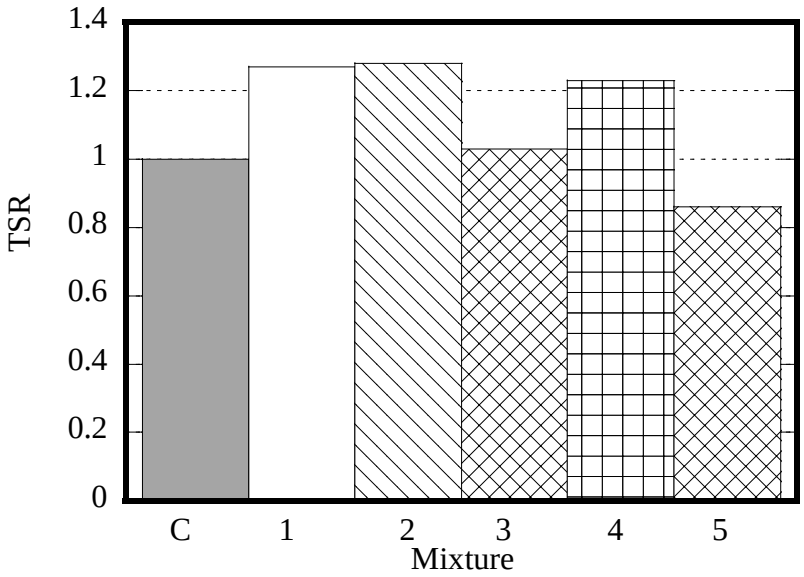

Figure 5.6: Normalized TSR Values for Mixtures Containing Aggregate #3

5.1.4 Summary for AASHTO T 283 Testing

Table 5.4 shows a summary of the TSR testing. The data indicates that Aggregate #1 and Aggregate #2 may not necessarily be susceptible to stripping. Aggregate #3 seems to be more susceptible to stripping. TSR results indicate Additive #1 was the most effective additive for improving the TSR value of the three aggregates. The TSR values for Additive #1 for all three aggregates were on average 18 percent higher than the values from the control specimens. Additive #2 and Additive #4 also provided higher TSR values, around a 12 percent increase over the control specimens. Additive #3, however, did not exhibit a significant increase or improvement. On average, specimens mixed with Additive #3 had TSR values about 2 percent lower than the control specimens. Only the mixture containing Aggregate #3 exhibited an increase in its TSR values when mixed with Additive #3. Figure 5.7 shows a box plot of the average normalized values of each additive. Table 5.5 shows the classification of the mixtures based on the values and ranges determined by Lottman (*Lottman 1982*), Maupin (*Maupin 1982*), and Tunnicliff and Root (*Tunnicliff and Root 1984*).

Table 5.4: Summary of TSR Values

Mix Type	Aggregate #1		Aggregate #2		Aggregate #3		AVG Normalized Values
	AVG TSR	Normalized to Control	AVG TSR	Normalized to Control	AVG TSR	Normalized to Control	
Control	0.78	1.00	0.87	1.00	0.57	1.00	1.00
Additive 1	0.89	1.15	0.98	1.12	0.72	1.27	1.18
Additive 2	0.79	1.01	0.93	1.07	0.73	1.28	1.12
Additive 3	0.77	0.990	0.79	0.91	0.59	1.03	0.98
Additive 4	0.89	1.14	0.86	0.98	0.70	1.23	1.12
Additive 5	N/A	N/A	N/A	N/A	0.49	0.86	**

** data only available for Aggregate #3.

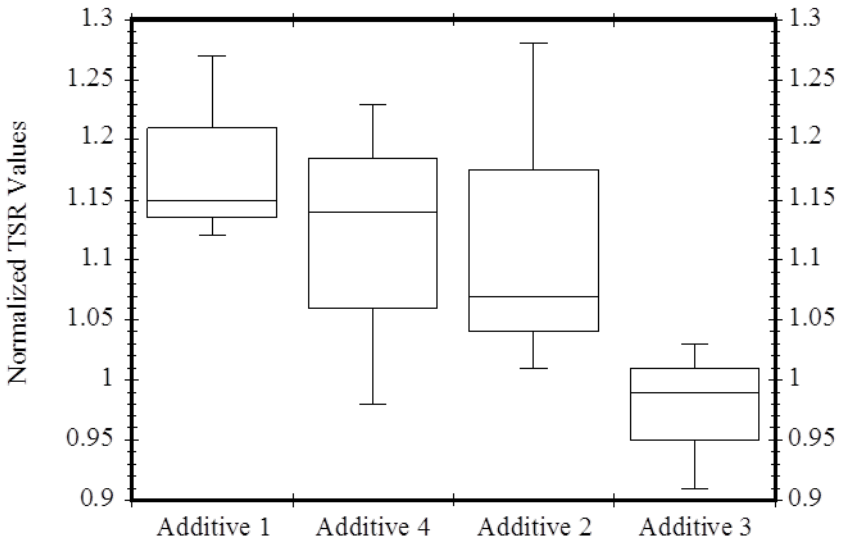

Figure 5.7: Summary of Average Normalized Values (additive 5 not shown)

Table 5.5: Classification of Mixtures Based on TSR Test Results

Mixture	Aggregate #1			Aggregate #2			Aggregate #3		
	Lottman	Maupin	Tunnicliff/Root	Lottman	Maupin	Tunnicliff/Root	Lottman	Maupin	Tunnicliff/Root
Control	NS	NS	PS	NS	NS	NS	S	S	S
Additive 1	NS	NS	NS	NS	NS	NS	NS	PS	PS
Additive 2	NS	NS	PS	NS	NS	NS	NS	PS	PS
Additive 3	NS	NS	PS	NS	NS	PS	S	S	S
Additive 4	NS	NS	NS	NS	NS	NS	PS	PS	PS
Additive 5	N/A	N/A	N/A	N/A	N/A	N/A	S	S	S

NS: Non-susceptible
PS: Possibly susceptible
S: Likely Susceptible

5.2 ECS/DYNAMIC MODULUS

Three ratios were calculated from the ECS testing data. The ratios included the conditioned state with the unconditioned state (dry), the saturated state and dry state, and the conditioned and saturated state. The ratio of the dynamic moduli for the conditioned state and unconditioned state is commonly used to assess susceptibility. According to the NCHRP Report 589, a minimum conditioned to unconditioned ratio ranging between 0.75 and 0.80 must be attained for a mixture to be considered non-susceptible to moisture damage. Values below 0.75 are considered susceptible.

5.2.1 Mixture with Aggregate #1

Table 5.6 shows a summary of the ratios for the specimens containing Aggregate #1, including their average ratios and normalized values. Table 5.7 shows all ECS ratio values obtained for the mix containing Aggregate 1. The conditioned to unconditioned ratio for the control group averages about 0.733 using the data from all four frequencies—this is considered susceptible per NCHRP Report 589. This indicates the specimens had approximately 70 percent of their original stiffness after conditioning. The average conditioned to unconditioned E* ratio for the mixture with Additive #1 was 1.004, indicating the specimens had similar E* values before and after they were conditioned, which is positive. Specimens mixed with Additive #2 exhibited similar results, with a conditioned to unconditioned ratio of 0.955. This indicates only a slight decrease in E* after conditioning, and a significant increase when compared with the control specimens. The specimens containing Additive #3 had a lower conditioned to unconditioned ratio of 0.852. This indicates that specimens containing Additive #3 may not perform as well as specimens mixed with Additives #1 and #2. However, the ratio is still higher than the ratio from the control group, indicating that specimens mixed with Additive #3 may improve longer-term performance when compared with control. Finally, specimens mixed with Additive #4 exhibited the second highest conditioned to unconditioned ratio of 0.985.

The results indicate that all additives had E* ratios above 0.80 which indicates that the mixtures containing these additives are not susceptible to stripping according to NCHRP Report 589.

Table 5.6: E* Ratios and Normalized Values for Mixture with Aggregate #1

Mixture with Aggregate #1	Conditioned to Unconditioned E* Ratio				Average Cond./Dry Ratio	Normalized to Control
	Frequency (Hz)					
	5	2	1	0.5		
Control	0.761	0.737	0.712	0.723	0.733	1.00
Additive 1	1.041	0.995	0.978	1.003	1.004	1.37
Additive 2	0.959	0.938	0.960	0.963	0.955	1.30
Additive 3	0.826	0.865	0.857	0.859	0.852	1.16
Additive 4	1.008	0.952	0.984	0.995	0.985	1.34

Table 5.7: ECS Data for Mixture with Aggregate #1

Mixture Containing Aggregate #1		Frequency (Hz)											
		5			2			1			0.5		
		Modulus Ratio			Modulus Ratio			Modulus Ratio			Modulus Ratio		
		Cond/Dry	Sat/Dry	Cond/Sat	Cond/Dry	Sat/Dry	Cond/Sat	Cond/Dry	Sat/Dry	Cond/Sat	Cond/Dry	Sat/Dry	Cond/Sat
Control	1	0.562	0.905	0.622	0.559	0.925	0.605	0.497	0.929	0.535	0.511	0.951	0.537
	2	0.859	0.946	0.908	0.849	0.940	0.903	0.835	0.926	0.902	0.845	0.963	0.877
	3	0.861	1.010	0.852	0.803	0.973	0.825	0.803	0.978	0.820	0.812	1.001	0.811
Additive #1	1	0.851	0.974	0.874	0.887	0.966	0.919	0.886	0.994	0.892	0.937	1.009	0.929
	2	1.065	1.102	0.967	0.938	1.023	0.917	0.903	1.009	0.895	0.905	1.009	0.896
	3	1.206	1.163	1.037	1.160	1.145	1.014	1.145	1.158	0.989	1.168	1.154	1.013
Additive #2	1	0.878	1.046	0.840	0.865	1.024	0.845	0.848	0.836	1.006	0.847	0.994	0.852
	2	0.950	1.069	0.889	0.878	1.025	0.856	0.934	1.035	0.903	0.928	1.032	0.900
	3	1.047	1.041	1.006	1.071	1.023	1.047	1.098	1.023	1.073	1.115	1.017	1.096
Additive #3	1	0.851	0.984	0.864	0.955	0.966	0.988	0.956	0.966	0.990	0.972	0.972	1.000
	2	0.867	0.956	0.907	0.870	0.944	0.922	0.869	0.951	0.914	0.875	0.954	0.917
	3	0.761	0.949	0.802	0.768	0.981	0.783	0.746	0.954	0.781	0.729	0.950	0.767
Additive #4	1	1.135	1.138	0.998	1.105	1.170	0.944	1.186	1.193	0.994	1.231	1.187	1.037
	2	0.719	0.737	0.975	0.740	0.764	0.968	0.746	0.772	0.967	0.737	0.765	0.964
	3	1.171	1.262	0.928	1.013	1.175	0.862	1.019	1.196	0.852	1.016	1.206	0.843

5.2.2 Mixture with Aggregate #2

The control specimens containing Aggregate #2 produced an average conditioned to unconditioned E* ratio of 0.911, which is higher than the average ratio from specimens containing Aggregate #1. This matches the results from AASHTO T 283 testing, which produced higher control TSR values for Aggregate #2. Table 5.8 shows the average conditioned to unconditioned E* ratios and normalized values for Mixture #2.

Specimens mixed with Additive #1 had a conditioned to unconditioned ratio of 0.987. Specimens containing Additive #2 and Additive #3 had lower values, 0.855 and 0.852, respectively. Specimens mixed with Additives #2 and #3 had lower conditioned to unconditioned E* ratios than the control specimens, indicating that these additive may not improve performance for an aggregate that is not considered to be susceptible (per NCHRP Report 589). Specimens mixed with Additive #4 had a conditioned to unconditioned ratio of 0.910, similar to control specimens. Table 5.9 shows a summary of ECS testing data mixtures containing Aggregate #2.

Table 5.8: E* Ratios and Normalized Values for Mixture #2

Mixture with Aggregate #2	Conditioned to Unconditioned E* Ratio Frequency (Hz)				Average Cond./Dry Ratio	Normalized to Control
	5	2	1	0.5		
Control	0.927	0.918	0.900	0.900	0.911	1.00
Additive 1	1.006	0.975	0.981	0.984	0.987	1.08
Additive 2	0.892	0.849	0.838	0.839	0.855	0.94
Additive 3	0.898	0.845	0.839	0.826	0.852	0.94
Additive 4	0.926	0.922	0.902	0.892	0.910	1.00

Table 5.9: ECS Data for Mixture #2

Mixture Containing Aggregate #2		Frequency (Hz)											
		5			2			1			0.5		
		Modulus Ratio			Modulus Ratio			Modulus Ratio			Modulus Ratio		
		Cond/Dry	Sat/Dry	Cond/Sat	Cond/Dry	Sat/Dry	Cond/Sat	Cond/Dry	Sat/Dry	Cond/Sat	Cond/Dry	Sat/Dry	Cond/Sat
Control	1	0.885	1.002	0.883	0.837	0.964	0.868	0.817	0.966	0.846	0.815	0.971	0.839
	2	0.836	1.001	0.835	0.809	1.015	0.797	0.799	1.001	0.798	0.781	0.986	0.792
	3	1.060	1.195	0.887	1.108	1.251	0.885	1.085	1.260	0.861	1.105	1.290	0.856
Additive #1	1	1.081	1.135	0.952	1.005	1.066	0.942	1.033	1.089	0.949	1.021	1.081	0.945
	2	0.958	0.968	0.990	0.941	0.959	0.981	0.919	0.963	0.954	0.930	0.968	0.960
	3	0.980	0.990	0.991	0.980	0.985	0.995	0.992	0.990	1.002	1.000	0.994	1.006
Additive #2	1	0.824	0.935	0.881	0.787	0.935	0.842	0.779	0.916	0.851	0.777	0.893	0.870
	2	0.976	1.125	0.868	0.912	1.030	0.885	0.902	1.030	0.876	0.921	1.026	0.898
	3	0.877	1.021	0.859	0.849	1.041	0.815	0.834	1.050	0.794	0.819	1.011	0.810
Additive #3	1	0.877	0.985	0.891	0.847	0.972	0.871	0.846	0.984	0.860	0.830	0.964	0.860
	2	0.926	1.105	0.838	0.890	1.109	0.803	0.881	1.110	0.794	0.865	1.116	0.775
	3	0.892	1.081	0.825	0.798	1.003	0.795	0.789	1.009	0.782	0.782	0.986	0.793
Additive #4	1	0.930	0.975	0.954	0.917	0.969	0.946	0.894	0.950	0.941	0.893	0.917	0.973
	2	0.917	0.920	0.997	0.915	0.937	0.977	0.907	0.933	0.972	0.902	0.934	0.965
	3	0.930	0.986	0.943	0.934	0.976	0.957	0.905	0.956	0.947	0.882	0.950	0.928

5.2.3 Mixture with Aggregate #3

Table 5.10 shows the conditioned to unconditioned ratios and normalized values of mixes containing Aggregate #3. The control specimens containing Aggregate #3 had an average conditioned to unconditioned E* ratio of 1.03. Specimens containing Additive #1 had a conditioned to unconditioned ratio of 0.77. The normalized ratio for Additive #1 is greater than unity, indicating that Additive #1 is effective in decreasing moisture susceptibility for Aggregate #3. Specimens mixed with Additive #2 exhibited a normalized ratio of 0.90. This value is lower than the other additives and less than unity, indicating that this additive may not be effective in reducing moisture susceptibility of Aggregate #3. The normalized ratio for Additive #3 is near unity, indicating that although there does seem to be some improvement, this improvement is limited for this aggregate. Results from Additive #4 indicate that this additive exhibits the second highest normalized ratio. Table 5.11 shows ECS testing data for specimens containing Aggregate #3.

Table 5.10: E* Ratios and Normalized Values for Mix with Aggregate #3

Mixture with Aggregate #3	Conditioned to Unconditioned E* Ratio				Average Cond./Dry Ratio	Normalized to Control
	Frequency (Hz)					
	5	2	1	0.5		
Control	0.69	0.70	0.69	0.67	0.68	1.00
Additive 1	0.779	0.78	0.77	0.76	0.77	1.13
Additive 2	0.570	0.63	0.63	0.63	0.62	0.90
Additive 3	0.723	0.71	0.70	0.71	0.71	1.04
Additive 4	0.729	0.74	0.75	0.76	0.74	1.09

Table 5.11: ECS Data for Mixture #3

Mix Containing Aggregate #3		Frequency (Hz)											
		5			2			1			0.5		
		Modulus Ratio			Modulus Ratio			Modulus Ratio			Modulus Ratio		
		Cond/ Dry	Sat/ Dry	Cond/ Sat	Cond/ Dry	Sat/ Dry	Cond/ Sat	Cond/ Dry	Sat/ Dry	Cond/ Sat	Cond/ Dry	Sat/ Dry	Cond/ Sat
Control	1	0.708	0.893	0.793	0.716	0.902	0.793	0.702	0.887	0.791	0.670	0.892	0.752
	2	0.678	0.984	0.689	0.696	0.954	0.730	0.689	0.974	0.707	0.681	0.964	0.706
	3	0.692	0.983	0.705	0.675	0.962	0.701	0.667	0.965	0.692	0.643	0.975	0.659
Additive #1	1	0.809	0.992	0.816	0.783	0.977	0.802	0.775	0.990	0.783	0.765	0.993	0.770
	2	0.708	0.900	0.787	0.698	0.907	0.769	0.685	0.900	0.761	0.669	0.899	0.744
	3	0.818	0.914	0.896	0.858	0.958	0.895	0.857	0.950	0.902	0.855	0.945	0.904
Additive #2	1	0.586	0.681	0.860	0.641	0.722	0.888	0.644	0.745	0.865	0.633	0.754	0.840
	2	0.562	0.604	0.930	0.667	0.682	0.978	0.675	0.687	0.983	0.678	0.669	1.014
	3	0.561	0.522	1.075	0.585	0.567	1.032	0.575	0.554	1.038	0.586	0.561	1.044
Additive #3	1	0.615	0.438	1.404	0.585	0.473	1.235	0.587	0.459	1.279	0.583	0.439	1.327
	2	0.850	0.448	1.897	0.852	0.512	1.666	0.846	0.491	1.721	0.860	0.482	1.784
	3	0.706	0.894	0.790	0.698	0.900	0.776	0.679	0.895	0.760	0.680	0.908	0.749
Additive #4	1	0.706	0.871	0.811	0.715	0.899	0.796	0.700	0.906	0.772	0.733	0.920	0.798
	2	0.698	0.882	0.792	0.719	0.898	0.801	0.738	0.934	0.790	0.758	0.965	0.785
	3	0.784	0.922	0.850	0.795	0.932	0.852	0.798	0.949	0.841	0.792	0.963	0.822

5.2.4 Summary of ECS/E* Testing

Table 5.12 shows a summary of the conditioned to unconditioned E* ratios and the normalized values for all mixture types. Table 5.13 shows the classification of mixtures based on the NCHRP report. The ECS results indicate Aggregates 1 and 3 are susceptible to stripping and that Additives #1 and #4 are the most effective at improving performance, while Additives #2 and #3 only showed slight improvements in performance for all aggregate types tested.

Table 5.12: Summary of E* Ratios and Normalized Values

Mixture	Average Conditioned to Unconditioned E* Ratio						AVG Normalized Values
	Aggregate #1		Aggregate #2		Aggregate #3		
	Ratio	Normalized to Control	Ratio	Normalized to Control	Ratio	Normalized to Control	
Control	0.733	1.00	0.911	1.00	0.68	1.00	1.00
Additive 1	1.004	1.37	0.987	1.08	0.77	1.13	1.19
Additive 2	0.955	1.30	0.855	0.94	0.62	0.90	1.05
Additive 3	0.852	1.16	0.852	0.94	0.71	1.04	1.05
Additive 4	0.985	1.34	0.910	1.00	0.74	1.09	1.14

Table 5.13: Classification of Mixtures Based on ECS Test Results

Mixture	Aggregate #1	Aggregate #2	Aggregate #3
Control	S	NS	S
Additive 1	NS	NS	PS
Additive 2	NS	NS	S
Additive 3	NS	NS	S
Additive 4	NS	NS	S

NS: Non-susceptible; PS: Possibly susceptible; S: Likely Susceptible

5.2.5 Summary of TSR and ECS/E* Testing

Results from both TSR and ECS testing indicate that Aggregate 3 is susceptible to stripping, based on limits reported in the literature. TSR results indicate that neither Aggregate 1 nor 2 are susceptible to stripping while ECS results indicate that Aggregate 1 is susceptible and Aggregate 2 is not susceptible to stripping. These overall result indicate that Aggregate 3 is susceptible to stripping, Aggregate 1 may be susceptible to stripping, and Aggregate 2 is likely not susceptible to stripping.

Because additives would likely not be used with mixtures containing non-susceptible aggregates, a more reasonable quantitative assessment would include only the aggregates that are

susceptible. Table 5.14 shows the mean ratios for TSR and ECS testing based on data from mixtures containing Aggregates 1 and 3 only. To assess potential improvement from the additives, the data can be normalized and average improvement values from both tests can be determined for the different additives – Table 5.15 shows these results. These results indicate that the mean normalized ratio from TSR and ECS testing for aggregates that are potentially susceptible and susceptible can be ranked as follows (highest to lowest): Additive 1, Additive 4, Additive 2, and Additive 3. Insufficient data are available to rank Additive 5.

Table 5.14: Classification of Mixtures Based on ECS Test Results

Mixture	TSR Results		ECS Results	
	Aggregate 1	Aggregate 3	Aggregate 1	Aggregate 3
Control	0.78	0.57	0.73	0.69
Additive 1	0.89	0.72	1.00	0.78
Additive 2	0.79	0.73	0.96	0.57
Additive 3	0.77	0.59	0.85	0.72
Additive 4	0.89	0.70	0.99	0.73
Additive 5		0.49		

Table 5.15: Assessment of Additives Based on TSR and ECS Test Results

Mixture	TSR Results			ECS Results			Averaged Normalized Improvement (both tests/both aggregates)
	Aggregate 1	Aggregate 3	Mean	Aggregate 1	Aggregate 3	Mean	
Control	1.00	1.00	1.00	1.00	1.00	1.00	
Additive 1	1.15	1.27	1.21	1.37	1.12	1.25	1.23
Additive 2	1.01	1.28	1.15	1.30	0.90	1.10	1.12
Additive 3	0.99	1.03	1.02	1.16	1.04	1.10	1.06
Additive 4	1.14	1.23	1.19	1.34	1.08	1.21	1.21
Additive 5		0.86	0.86*				

* two samples

6.0 ECONOMIC ASSESSMENT

Several additives evaluated indicate that they can reduce the stripping susceptibility of susceptible aggregates in HMA systems. Although results indicate improvement, the results do not provide information on the potential value of each additive. To provide better insight on value, this chapter provides an overview of the economics of using each additive. It should be noted that economic analyses can be dependent on dynamic variables. The information provided in this chapter use available data or "best guess" static values and the assessment is deterministic.

To assess the value of the different additives evaluated in this study, a wide range of information is needed. Cost information on each product is needed. In addition, any costs associated with storing or adding the additive to the HMA at the plant, cost associated with changes in construction processes (transporting, placing, or compacting), and costs benefits from extending the service life of the pavement must be realized.

6.1 COST VARIABLES

The research has shown than some additives can reduce the likelihood of stripping in HMA pavements. Although several different scenarios could occur, inputs from ODOT personnel indicate that the most likely difference between pavements that exhibit stripping and pavement that do not exhibit stripping would be the depth of the inlay. It should be noted that many variables could influence the economics and that this assessment is a simplification. A typical service life of a pavement in Oregon is 15 years. The end of service life is dependent on many factors, including deterioration or from the use of studded tires. ODOT tries to resurface pavements every 15 years. If a pavement is experiencing stripping, the depth of the future inlay is increased. Table 6.1 shows the base case values used for the economic analyses for both the stripping and non-stripping conditions. The first economic analysis includes a cost benefit analysis following the Life-Cycle Benefit-Cost Analysis Model procedure used by the California Department of Transportation (CalTrans) (accessed September 2013 at http://www.dot.ca.gov/hq/tpp/offices/eab/LCBC_Analysis_Model.html). The second economic analysis includes a procedure outlined in Trejo and Reinschmidt. *(Trejo and Reinschmidt 2007)*

Table 6.1: Base Case Values for Economic Analysis

Variable	Alternative 1 (New HMA pavement without Additive)	Alternative 2 (New HMA pavement with Additive)
Cost of Original HMA	$75/ton[A] ($68.18/tonne)	$75/ton[A] ($68.18/tonne)
Depth of Original HMA	9 inches (229 mm)[D]	9 inches (229 mm)[D]
Time to First Inlay, *z*	15 years[C]	15 years[C]
Cost of Inlay HMA	$75/ton[A] ($68.18/tonne)	$75/ton[A] ($68.18/tonne)
Cost of Additive	$0/ton	$1.50/ton[C] ($1.36/tonne)
Cost to Grind Existing HMA for Inlay	$1.20/yd^2 [B]	$0.85/yd^2 [B]
Depth of Inlay	4 inches (51 mm)[C]	2 inches (102 mm)[C]
Discount Rate, *r*	4%[D]	4%[D]

A-Source: Item #390131 in Caltrans (2012); B-Source: ODOT (ODOT 2013); C-Source: ODOT personnel; D-Source: ODOT (2010)

6.2 BENEFIT COST AND LIFE-CYCLE COST ANALYSES

This section summarizes the results of a comparative life-cycle cost (LCC) analysis aimed at determining the financial benefit of using additives in HMA pavements. The assessment measures the benefit in terms of a benefit cost ratio (BCR) and LCC benefit by comparing alternatives. Accordingly, the analysis presented herein involves a systematic LCC analysis procedure as follows:

1. Determine a set of baseline parameters (already shown in Table 6.1);
2. Determine a BCR and LCC benefit, and;
3. Perform a sensitivity analysis based on varying key parameters.

The following two alternatives are compared in this LCC analysis to estimate a BCR and LCC benefit of using an anti-stripping additive in HMA for pavements in Oregon:

- Alternative 1: Construction of a new pavement without the anti-stripping additive
- Alternative 2: Construction of a new pavement with the anti-stripping additive

It should be noted that the result of not using an additive when an aggregate exhibits stripping is that the depth of the inlay in future years will have to be deeper than if an additive were used. Simply, ODOT practice is to require thicker inlays for pavements that are exhibiting degradation due to stripping.

6.2.1 Baseline Parameters

As shown in Table 6.1, Alternative 2 assumes that a HMA pavement is constructed with an anti-stripping additive. Accordingly, $1.50 per ton ($0.0016/kg) is added to the paving costs at years 0, 15, and 30. Because Alternative 2 includes the additive, the inlay thickness in future years will be less than if the HMA additives were not used. Alternative 1 does not use an additive and the

depth of the inlay in future years will be greater. As inflation and escalation are highly variable, factors related to inflation and/or escalation of construction costs are not considered in this analysis.

This LCC analysis will determine the life-cycle benefit per unit measure (i.e., per ton (or kg) of newly placed pavement), so as to provide a convenient multiplier for future analysis. Assuming a typical roadway is 12 feet (3.67 meters) wide and using the values in Table 6.1, the length of roadway that can be paved with one ton (907 kg) of HMA can be determined as follows:

$$\text{Lane Length} = \frac{2000\ \frac{lb}{ton}\left(907\ \frac{kg}{ton}\right)}{\text{HMA Unit Wt}\left(\frac{mass\ (weight)}{length^3}\right) \times \text{Lane Width}\ (length)\ \times \text{Pavement Depth}\ (length)} \quad \textbf{(Eq. 9)}$$

For a new 9-inch (0.23-m) pavement, the lane length would be 1.53 lane-feet (0.47 lane-m). If all future calculations are performed for this standard lane length or lane area (1.53 feet×12 feet =18.39 feet^2), cost comparisons can then be performed. The repaving cost at years 15 and 30 years can be determined as follows:

$$\text{Inlay Paving Cost} = \frac{\text{Inlay Thickness}}{\text{Original Pavement Thickness}} \times (\text{Demo \& Paving Cost per Ton}) \quad \textbf{(Eq. 10)}$$

Using Equations 9 and 10, Alternative 1 would result in an inlay cost of $40.00 (note that additive is not used in this inlay) and Alternative 2 would result in an inlay cost of $20.33 (additive is used in this inlay). These costs would recur at 15 and 30 years (separate analyses are performed for these different time periods). Figure 6.1 shows the cash flow diagram for each alternative over the 30-year analysis period (the 15 year option would not consider the values at 30 years).

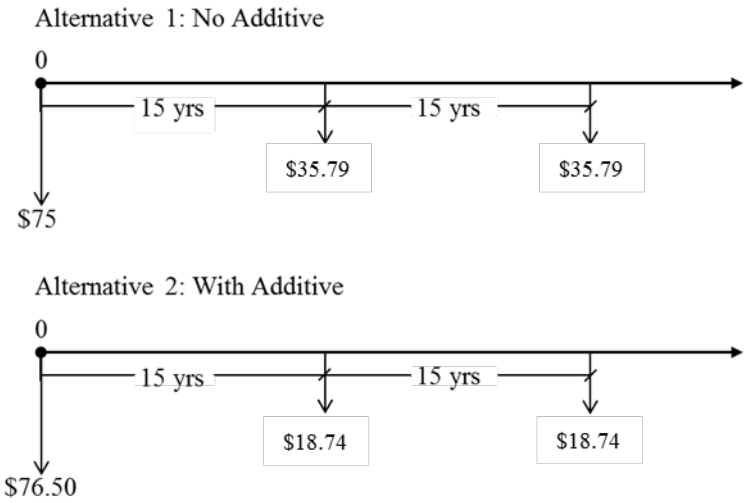

Figure 6.1: Cash Flow Diagram Comparing Two Alternatives for BCR

6.2.2 Benefit Cost Ratio and LCC Benefit

The **BCR** is defined as the value of the future benefit of using the additive divided by the initial additive cost after all values are discounted and expressed in present values. This ratio represents how many dollars can be gained for each dollar spent for the additive over the life of the project. Using basic economic analyses, a future value can be discounted to time 0 by using the following equation:

$$PW = \frac{F}{(1+r)^n} \qquad \textbf{(Eq. 11)}$$

where F is the future value ($40 or 23.33 shown in Figure 6.1), r is the rate of return, and n is the number of compounding periods (for this case, years). Using Equation 11, the benefits can be determined as follows:

$$Benefits = \sum_{i=1}^{x} \frac{F_{i,Alt1}}{(1+r)^n} - \sum_{i=1}^{x} \frac{F_{i,Alt2}}{(1+r)^n} \qquad \textbf{(Eq. 12)}$$

where $F_{i,Alt1}$ is the future value of the i^{th} inlay for Alternative 1 (no additive) and $F_{i,Alt2}$ is the future value of the i^{th} inlay for Alternative 2 (with additive). After determining the benefits, the BCR is simply:

$$BCR = \frac{Benefits}{Cost\ of\ \ Additive\ (per\ ton)} \qquad \textbf{(Eq. 13)}$$

For example, the BCR of using the additive for a 15-year analysis period is:

$$BCR_{15yrs} = \frac{\$19.87 - \$10.40}{\$1.50} = 6.31$$

Table 6.2 summarizes the results of the BCR analysis. For a 15 year assessment period, the BCR indicates that for every $1 spent on additives, $6.31 in benefits can be realized by ODOT. If a pavement experiences 2 inlays, for every $1 spent on additives, ODOT would realize a benefit of $9.82. The BCR indicates that there is significant value from using additives that will result in thinner inlays.

Table 6.2: Summary of BCR Results

Alternative	Initial Paving Costs	Present Value of 1st Repaving Costs at Year 15*	Present Value of 2nd Repaving Costs at Year 30*	Present Value of 1st and 2nd Repaving Costs
1	$75.00	$19.87	$11.03	$30.90
2	$76.50	$10.40	$5.78	$16.18
Δ (Alt. 1 – Alt. 2)		$9.47		$14.72
BCR (Δ/$1.50)		6.31		9.82

r=4% per annum

A similar analysis can be performed using the overall life-cycle costs (including original construction costs). Table 6.3 shows the results from the LCC analysis. The LCC analysis indicates that over a 15 year life (one inlay), the benefit of using an additive that results in a thinner inlay is $7.97/ton ($0.009/kg). For the 30 year life (two inlays), the LCC benefit is $13.22/ton ($0.016/kg). Both cases exhibit significant value.

In 2012, ODOT placed 78,806 tons of new asphalt pavement (ODOT 2012). If additives were added to one-half of this HMA and this resulted in a thinner overlay, ODOT would realize a benefit of $313,902 if each pavement experienced one inlay or $521,019 if each pavement experienced two inlays. The LCC analysis indicates that additives in HMA that result in thinner inlays have significant benefits.

Table 6.3: LCC Results

Analysis Period	15 Years	30 Years
LCC of Alternative 1 (A)	$94.87	$105.90
LCC of Alternative 2 (B)	$86.90	$92.68
LCC Benefit (A - B)	$7.97	$13.22

r=4% per annum

6.2.3 Sensitivity Analysis for LCC

A sensitivity analysis assesses how changes in input variables influence the outcome of the analysis (*Trejo and Reinschmidt 2007b*). As an example, it was assumed in this analysis that the rate of return, *r*, was 4% (Table 6.1). If economic conditions change and *r* became 6% (i.e., a 50% increase in the original *r*), how would this effect the economics of using HMA additives to prevent stripping? Performing sensitivity analyses allows the user to assess the robustness of their outcome, identify sensitive variables, and can help minimize the subjectivity of LCC analyses and hence reduce biases in the analysis.

Six key parameters are selected for the sensitivity analysis, and the impacts of changing these variables are examined. The six parameters are:

a. Discount rate (%)

b. Inlay thickness required for repaving Alternative 2 at year 15 and year 30 (inch)

c. Initial paving cost at year 0 ($ per ton)

d. Inlay paving cost at year 15 and year 30 ($ per ton)

e. Cost to remove existing pavement for repaving, i.e., grinding ($ per ton)

f. Additive cost ($ per ton)

For this sensitivity analysis, the range of the percent change in variables is set from -50% to 50% at 10% increment. Note that some variables, such as construction costs, would likely not vary by ±50% and all ranges may not be realistic under current conditions. However, the analysis includes all variables ranging from ±50% of the base case condition. Table 6.4 shows the input values for the percent change in input value for each variable. These values are used for the sensitivity analysis.

Table 6.4: Varying Inputs for Sensitivity Analysis

Input % Change	Discount Rate (%)	Inlay Thickness (inch)	Initial Paving Cost ($)	Inlay Paving Cost ($)	2-inch Demo Cost ($)	4-inch Demo Cost ($)	Additive Cost ($)
-50%	2.0%	1.0	$38	$38	$0.43	$0.60	$0.8
-40%	2.4%	1.2	$45	$45	$0.51	$0.72	$0.9
-30%	2.8%	1.4	$53	$53	$0.60	$0.84	$1.1
-20%	3.2%	1.6	$60	$60	$0.68	$0.96	$1.2
-10%	3.6%	1.8	$68	$68	$0.77	$1.08	$1.4
Base Case 0%	4.0%	2.0	$75	$75	$0.85	$1.20	$1.5
+10%	4.4%	2.2	$83	$83	$0.94	$1.32	$1.7
+20%	4.8%	2.4	$90	$90	$1.02	$1.44	$1.8
+30%	5.2%	2.6	$98	$98	$1.11	$1.56	$2.0
+40%	5.6%	2.8	$105	$105	$1.19	$1.68	$2.1
+50%	6.0%	3.0	$113	$113	$1.28	$1.80	$2.3

Because the LCC analysis assessed two separate analysis periods, 15 and 30 years, sensitivity analyses will be performed for each condition separately. The output values used for creating the sensitivity charts are provided in Appendix A.

Figure 6.2 shows the rate of change (in percent) of the LCC benefits with respect to changes in the six key parameters for the 15-year analysis period (one inlay). In the sensitivity analysis, larger slopes indicate variables that are more sensitive. Figure 6.2 indicates that the most sensitive variable in the LCC analysis is inlay depth and that the LCC is fairly sensitive to rate of

return (*r*) and the cost of the inlay paving. Figure 6.2 also shows that the LCC is insensitive to the original paving cost, the cost of the additive, and the cost of the pavement grinding for the inlay.

As an example, if the inlay thickness of the inlay at year 15 is reduced by 50% (i.e., from 2 inches (51 mm) to 1 inch (25 mm)), the financial benefit of using the additive (realized by the LCC difference) increases by 65% (1.65 × \$7.97 = \$13.15). Similarly, if the inlay paving cost at year 15 increases by 50% from \$75 to \$113 per ton, the LCC benefit is increased by 58% (1.58 × \$7.97 = \$12.59). Although a significant increase in inlay depth has a significant reduction in the LCC benefit, the analysis indicates that with even large changes, the LCC benefit is still positive. For the ranges assessed, the LCC benefit is always positive, indicating that if additives do result in reduced inlay thicknesses, no input variables would make the investment in additives not cost effective.

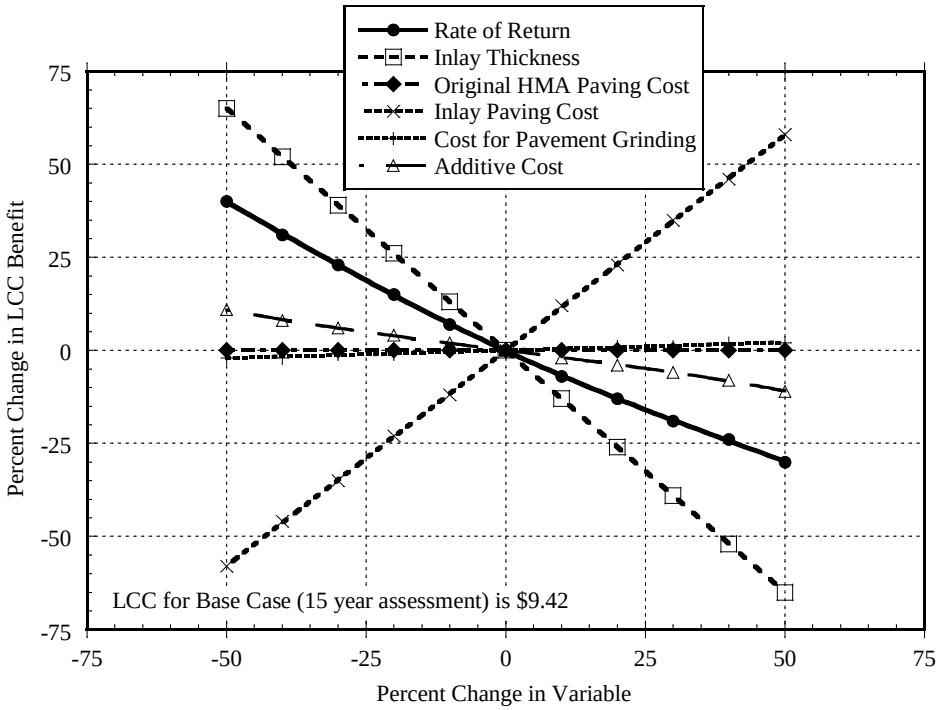

Figure 6.2: Sensitivity Analysis Results – 15 Years

Figure 6.3 shows the sensitivity analysis for the 30-year analysis period where inlays are placed at years 15 and 30. There are no notable changes in the 30-year sensitivity analysis and the 15-year sensitivity analysis except for the effect of discount rate. The sensitivity analysis indicates that as the analysis period increases, the impact of the discount rate on the LCC benefit increases.

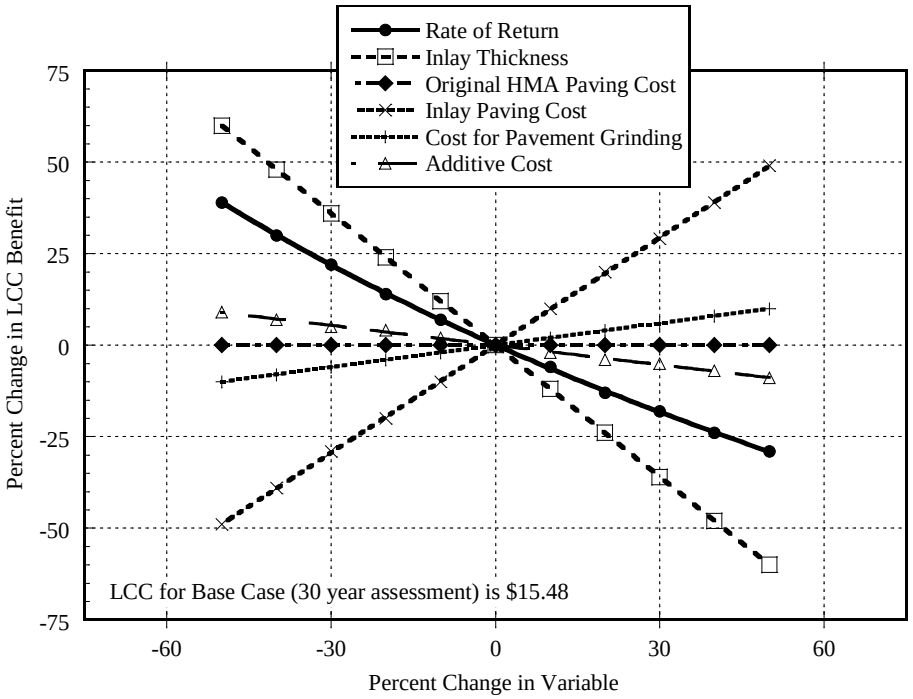

Figure 6.3: Sensitivity Analysis Results – 30 Years

6.3 COMPARISON OF RELATIVE LIFE-CYCLE COSTING

Decisions regarding additives for flexible pavements are typically made during construction planning. An engineer typically enters a decision making process during the design process and bases decisions on availability of materials, susceptibility of materials, site conditions, and other factors. The decision making process should include information on the longer-term performance of the pavement. For example, an engineer should not choose an option that has a higher initial cost and shorter service life. The challenge is that information on service life is not always easy to determine. In addition, some life-cycle models used for life-cycle economic comparisons require the input of detailed information that is not readily available, is highly uncertain during the decision making (design) process, or only assesses for discrete time periods (as for the BCR and LCC benefits). Such models can be difficult for engineers to use. In addition, engineers often do not have the time to perform several alternate options with different materials or inlay depths. Trejo and Rienschmidt developed a model to assess the *relative* economic advantages of using different materials for concrete systems. *(Trejo and Rienschmidt 2007)* Because ODOT personnel believe the most significant maintenance difference for pavements exhibiting stripping and pavements not exhibiting stripping are the depth of the inlay, the relative economic analysis will be used considering this difference. To be useful for designing pavement engineers, a model only has to be accurate enough to support the best option regarding whether additives should be used (which will result in different inlay options). In addition to not only being useful, the model should also be based on the information that is readily available to the engineer. It will be shown that the relative model provides sufficient accuracy and requires limited information. The outcome can provide the engineer with needed information to make decisions regarding the use

of additives in HMAs. Following is a brief description of the derivation of the relative economic analysis.

6.3.1 Derivation of Relative Economic Analysis

Suppose that it is desired to compare the economics of a pavement that contains HMA with and without an additive. The additive would be used because the aggregate could be susceptible to stripping. The consequences of using a susceptible aggregate without additive are that in 15 years the pavement will require an inlay, typically 4 inches (102 mm) deep. If the additive is used, an inlay is still required (due to studded tire use) but the required depth of the inlay will only be 2 inches (51 mm).

Suppose that for some pavement, the benefits of the new pavement constructed with the more durable HMA (i.e., containing additive) are to be compared with a HMA constructed with conventional materials (i.e., no additives). As the economic benefits of durable HMA are obtained over the life-time of the pavement, it is necessary to discount future costs and benefits to the present value. This is simply conventional engineering economics. Simplifying, suppose that the construction time for the durable pavement and the conventional pavement are identical; say T0. The total initial construction cost using the conventional HMA is estimated to be $CC and the total cost of the initial construction using the more durable HMA materials is estimated to be $CD.

It is assumed that both the conventional and durable HMA materials result in a project with exactly the same flow of benefits (e.g., benefits to the public of using the pavement). The difference between the two alternate HMA materials is that, after some time, say *z*, the conventional construction must be inlayed at a cost of $\$C_{I,C}$ (this cost would include the cost of the deep inlay). The project with the durable HMA material, however, would need to be inlayed at a cost of $\$C_{I,D}$ (this cost would include the cost of the shallow inlay).

The cost for repairs, $CI,X, is assumed to include the imputed costs to the users due to the reduction in benefits while the performance of the facility is degenerating (for example, damage or loss of service due to potholes, cracking, detours, lane restrictions, and other effects associated with degraded pavement performance), plus the value of the loss of service of the facility while the inlays are being made.

The benefits to the users are assumed to be the same in both scenarios, except for the value of the loss of service due to deterioration and repair of the pavement, which is assumed, as discussed above, to be captured in the inlay cost $CI,X. Therefore, the actual benefits stream is unknown but irrelevant to this analysis, because it is identical for both alternatives. It is assumed here that the project with conventional HMA materials is cost-justified; that is, the net present worth of all the costs (including both the initial costs and the periodic repair costs) is equal to the net present value of the flow of future benefits at some discount rate r, which is acceptable to the owner and at least as large as the minimum discount rate for this type of project. Simply, at the discount rate r, the project with conventional materials was at some point economically justified and a decision has been made that it will move forward to construction.

The issue that needs to be answered is whether the project should go forward with durable HMA materials rather than the conventional HMA materials. In general, projects typically use conventional HMA materials, as more durable HMA materials are more expensive than conventional HMA materials (i.e., \$CD > \$CC), unless there are some offsetting savings in construction using the durable materials (which is likely not the case for HMA with additives). If \$CD ≤ \$CC, then the project should use the durable HMA material and no further analysis is needed. However, in the more typical case, the durable HMA material should be used only if the additional costs for the future repair for the conventional HMA material are greater than the additional initial cost of the construction using the durable HMA materials, on a present value basis.

Instead of estimating the costs of the more durable HMA option, the present analysis examines the inverse question: How much more could the durable HMA material cost be, relative to the conventional HMA material cost, and still be more economical? This is likely just the cost of the additive as it is assumed that conventional HMA and durable HMA have the same construction costs. Here, a critical value of \$CD/\$CC is defined such that if the ratio of construction cost using durable HMA material to the construction cost using conventional HMA material is less than this critical value, then the durable HMA material is more economical and should be used. To do this, the cost of construction using the durable HMA material, \$CD, will be treated as a variable and the value of \$CD will be determined that makes the present worth of the pavement using durable HMA just equal to the present worth of the pavement using conventional HMA materials.

Assume for simplicity that the initial cost of construction in either scenario can be approximated by a discrete cash flow at the midpoint of the construction duration as shown in Figure 6.4; that is, at time $T_0/2$. Then, the present worth discounted to time 0 of each initial construction alternative, using the discount rate r and standard engineering economics notation, is:

$$PW_{CC} = \frac{\$C_C}{(1+r)^{T_0/2}} \qquad \textbf{(Eq. 14)}$$

for initial construction cost using the conventional HMA material, and:

$$PW_{DC} = \frac{\$C_D}{(1+r)^{T_0/2}} \qquad \textbf{(Eq. 15)}$$

for initial construction cost using the durable HMA material.

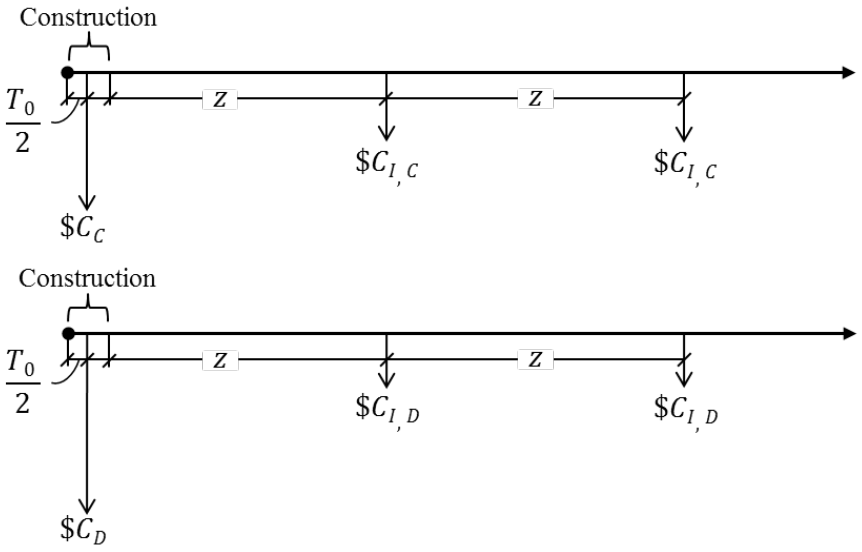

Figure 6.4: Cash flow diagram for relative economic analysis

The present worth of all inlays for the pavement constructed with conventional HMA materials is:

$$PW_{IC} = \frac{1}{(1+r)^{T_0}}\left(\sum_{i=1}^{n}\frac{\$C_{I,C}}{(1+r)^{nz}}\right) \qquad \textbf{(Eq. 16)}$$

The present worth of all inlays for the pavements constructed with durable HMA materials is:

$$PW_{ID} = \frac{1}{(1+r)^{T_0}}\left(\sum_{i=1}^{n}\frac{\$C_{I,D}}{(1+r)^{nz}}\right) \qquad \textbf{(Eq. 17)}$$

The total present value of the costs, including initial construction plus the future repairs, for the pavement using conventional HMA materials is $PW_{CC} + PW_{IC}$, and the corresponding present worth for the construction using more durable HMA materials is $PW_{DC} + PW_{ID}$. The durable construction option (i.e., using HMA material with an additive) should only be chosen if its present value is less than the present value of the conventional materials; that is, if:

$$PW_{DC} + PW_{ID} \le PW_{CC} + PW_{IC} \qquad \textbf{(Eq. 18)}$$

Substituting Equations 14, 15, 16 and 17 into Equation 18 and dividing by PW_{CC} results in the following:

$$\frac{\$C_D}{\$C_C} \le 1 + \frac{\sum_{i=1}^{n}\frac{1}{(1+r)^{nz}}}{(1+r)^{T_0/2}} \cdot \frac{1}{\$C_C} \cdot \left(\$C_{CI} - \$C_{DI}\right) \qquad \textbf{(Eq. 19)}$$

Here, the cost of the inlay for the pavement with the additive, $\$C_{ID}$, can be expressed as a function of the cost of the inlay for the pavement without the additive, $\$C_{CI}$, as follows:

$$\$C_{DI} = K \cdot \$C_{CI} \qquad \textbf{(Eq. 20)}$$

where K is less than unity and represents the ratio of the depth of the inlay for the inlay on the durable pavement and the depth of inlay on the conventional pavement (i.e., without additive). ODOT personnel indicated that this value is likely ½ (2-inch (51 mm) inlay depth for pavements initially constructed with additives and 4-inch inlay depth for pavements initially constructed without additives). Substituting Equation 20 into Equation 19 results in the following:

$$\frac{\$C_D}{\$C_C} \le 1 + \frac{\sum_{i=1}^{n} \frac{1}{(1+r)^{nz}}}{(1+r)^{T_0/2}} \cdot \frac{\$C_{CI}}{\$C_C} \cdot (1-K) \qquad \textbf{(Eq. 21)}$$

This can be rewritten as:

$$\frac{\$C_D}{\$C_C} \le 1 + f(n,z,r,T_0) \cdot \frac{\$C_{CI}}{\$C_C} \cdot (1-K) \qquad \textbf{(Eq. 22)}$$

Equation 22 can be changed to an equality and then be used to determine whether there is value to using additives that result in reduced inlay thickness. Values of $\$C_D/\$C_C$ that are greater than unity indicate an increase in value over the conventional cost (in this case Alternative 1).

6.3.2 Results of Relative Economic Analysis

Initial values of z (time between original construction and inlays and time between inlays) and r (rate of return) are 15 years and 4%, as shown in Table 6.1. The variable n, as shown in Equation 22, represents the number of inlays and T_0 represents the time of initial construction (in years). Initial estimates for n and T_0 for this analysis will be 2 inlays and 0.5 years (although any values could be assumed). Using these values $f(n,z,r,T_0) = 0.428$. This will be assumed to be the base case for this analysis. Assuming K = ½, Equation 22 results in the following:

$$\frac{\$C_D}{\$C_C} = 1 + 0.428 \cdot (0.5) \cdot \frac{\$C_{CI}}{\$C_C} \qquad \textbf{(Eq. 23)}$$

An estimate of the ratio of the cost of the inlay for the pavement without additives ($\$C_{IC}$) and the initial construction costs for the condition without additives ($\$C_C$) can be initially assumed to be the ratio of the depths. From Table 6.1 the depth of the inlay for the pavement without additives is 4 inches (102 mm) and the depth of the original pavement is 9 inches (229 mm), resulting in a ratio of 4/9 or 0.444. Using Equation 23 and the assumptions noted, the value of $\$C_D/\$C_C$ can be determined as follows:

$$\frac{\$C_D}{\$C_C} = 1 + 0.428 \cdot (0.5) \cdot 0.444 = 1.095 \qquad \textbf{(Eq. 24)}$$

This indicates that the additive can cost up to 9.5% of the original construction cost and still add value. Table 6.1 indicates that the cost to construct a new HMA pavement is \$75/ton. The cost for the additive should then be less than or equal to 0.095·\$75/ton = \$7.13/ton. Table 6.1 indicates that the additive cost is \$1.50/ton, which is significantly lower than the threshold value of \$7.13/ton. This indicates that the additive option, that is, using the additive in the original HMA material, provides significant benefits and this option should be selected. It should be noted that the assumed values for the variables may vary. To assess how changes in the variables values affect $\$C_D/\$C_C$, sensitivity analyses will be performed.

6.3.3 Sensitivity Analysis for Relative Economic Analysis

A thorough sensitivity analysis should evaluate the influence of all variables on the output. The influence of each variable on the ratio of the construction cost with additive (aka the durable option) and the construction cost without additive (aka conventional option) is shown in Figure 6.5. The figure shows that $\$C_D/\$C_C$ is most sensitive to changes in the rate of return (r), K, and $\$C_{IC}/\$C_C$. The figure also shows that $\$C_D/\$C_C$ is less sensitive to the number of inlays and that the sensitivity decreases with increasing number of inlays. $\$C_D/\$C_C$ is insensitive to original construction duration. Most importantly, the sensitivity plot shows that a 100% increase in the rate of return (from 4% to 8%) only results in an approximate 5% reduction in $\$C_D/\$C_C$ (from 1.095 to 1.045), an 80% increase in K results in a 7% reduction in $\$C_D/\$C_C$ (from 1.095 to 1.02), and a 68% reduction in $\$C_{IC}/\$C_C$ results in nearly a 6% reduction in $\$C_D/\$C_C$ (from 1.095 to 1.03). Large changes in the input variables do not make the option of using additives in the HMA cost ineffective.

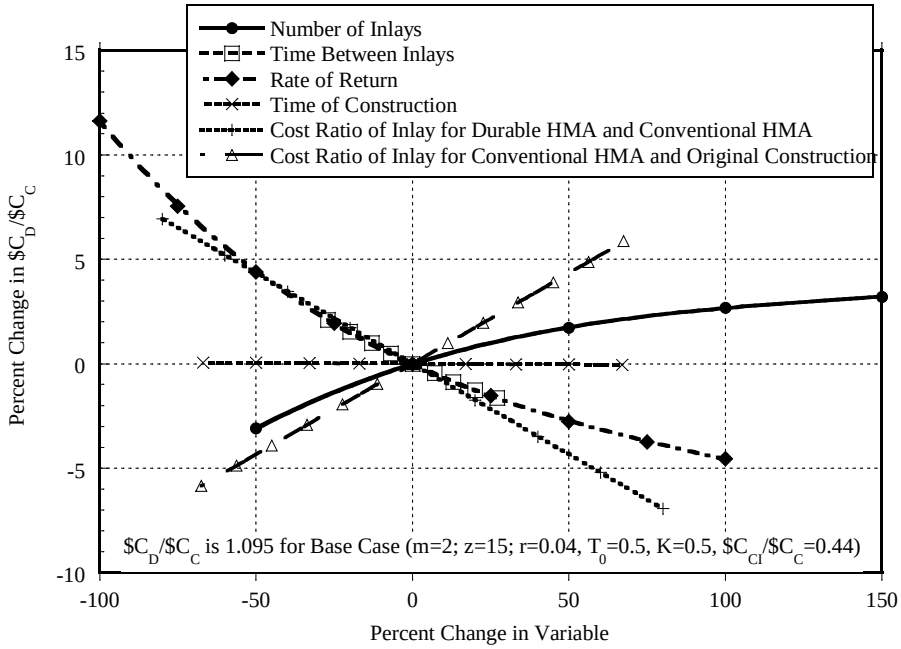

Figure 6.5: Sensitivity plot – effect of change in variables on change in $\$C_D/\$C_C$.

7.0 SUMMARY AND CONCLUSIONS

Stripping is a common problem in HMA in Oregon. Stripping is the degradation of the bond between the aggregate and the asphalt binder due to the presence of water. Stripping often leads to loss of capacity and cracking in the pavement. Additives are available to reduce the potential for stripping. The most common additive used in the industry is powdered lime. However, challenges with air-borne powdered lime have SHAs investigating alternatives to powdered lime. The purpose of this study was to determine the effectiveness of potential alternatives to powdered lime additive for HMAC.

This research evaluated the moisture susceptibility of five anti-stripping additives in HMA. To assess how the anti-stripping additives performed with different aggregate types, three different HMA mixtures with different coarse aggregate types were evaluated. Specimens were evaluated following AASHTO T 283 and ECS/Dynamic Modulus testing. Results indicate that Aggregates 1 and 3 are likely susceptible to stripping, with Aggregate 3 likely being more susceptible. In all cases, the powdered lime showed increases in the TSR and ECS values, indicating improved performance. With the exception of ECS results from the mixture with Aggregate 3, mixtures with Additive 2 also exhibited increases in TSR and ECS values. Mixtures containing Additive 3 exhibited limited increases in TSR and ECS values and under the conditions of this research program showed limited potential to reduce stripping within the HMA. HMA mixtures containing Additive 4 exhibited similar results as the specimens containing powdered lime. TSR results of mixtures containing Aggregate 3 (the most susceptible aggregate) and Additive 5 indicated no improvement. It should be noted that the study evaluating the mixture containing Aggregate 3 and Additive 5 was limited and additional testing is likely needed. Based on the results of the testing program, Additive 4 seems to resist stripping susceptibility at a similar level as powdered lime. Additive 2 showed improvement in performance and Additive 3 showed only modest improvement.

The economic analyses indicate that significant value can be realized if additives result in inlays with decreased depths. The economic analyses indicate that a ~7:1 benefit can be realized by using additives for the assumptions used. However, the sensitivity analyses indicate that large changes in initial assumptions still result in a significant benefit.

Based on the constraints and results of this research, the following conclusions can be drawn:

- Additive 2 and 4 likely reduce stripping, while Additive 3 may not;

- Mixtures containing Additive 4 exhibited similar results to mixtures containing powdered lime and ODOT should consider using this as an anti-stripping additive;

- Mixtures containing Additive 2 exhibited improved results but improvements were not as significant as the powdered lime results. However, this product is a relatively new product and an optimal methodology for adding this product to the mix was not defined at the beginning of the research. Different methods for adding the additive may result in different results and further testing is likely needed.

- Economic analyses for the construction of new asphalt concrete roadways indicates that there can be significant value from using additives to reduce or prevent stripping when the additive results in decreased depths of future inlays. Because of this, additives should likely be used in new pavements when aggregates are suspected or known to be susceptible to stripping.

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