FINAL REPORT

EVALUATE THE CONTRIBUTION OF THE MIXTURE COMPONENTS ON THE LONGEVITY AND PERFORMANCE OF FC-5

FDOT Contract No. BDS15 977-01

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SI (MODERN METRIC) CONVERSION FACTORS (FROM FHWA)

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
SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
AREA				
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
VOLUME				
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: volumes greater than 1000 L shall be shown in m^3				
SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
MASS				
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
TEMPERATURE (exact degrees)				
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
ILLUMINATION				
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
FORCE and PRESSURE or STRESS				
lbf	pound force	4.45	newtons	N
lbf/in^2	pound force per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS TO ENGLISH 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
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	$candela/m^2$	0.2919	foot-Lamberts	fl
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	pound force	lbf
kPa	kilopascals	0.145	pound force per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380 (Revised March 2003)

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16. Abstract

The focus of the project was to evaluate how to improve the longevity of FDOT's FC-5 mixtures. In particular, what FC-5 mixture components have the greatest impact on improving the cracking and durability of the FC-5 mixture. The data mining of FDOT Pavement Management System (PMS) database, provided valuable information regarding typical service life of FC-5 mixtures. Combined with material datasheets from production, the fatigue and durability of the FC-5 mixtures was found to be a function of the effective asphalt content of the mix, with little to no influence from traffic and pavement structure. This was further validated during an extensive field visit. Laboratory testing consisted evaluating current mix design procedures, finer nominal aggregate size (NMAS) FC-5 mixtures, and the influence of production tolerances on the durability, cracking, and rutting performance. The research conducted showed that asphalt contents could be increased by up to 0.6% if Pie-Plate testing was conducted using the PG76-22 and ARB-12 proposed for the mix. A 9.5 mm NMAS FC-5 mixture improves durability and fatigue performance over the current 12.5 mm NMAS FC-5, but rutting issues were observed in the Hamburg test, especially with the ARB-12 asphalt binder. Current production tolerances for FC-5 mixtures seem appropriate, except for the 0.6% reduction in binder content and the finer side of the gradation tolerance for ARB-12 binders. The 0.6% below optimum asphalt content tolerance resulted in mixtures with poor Overlay Tester fatigue performance and higher Cantabro Abrasion Loss values. ARB-12 asphalt binders may create stability issues when FC-5 gradations are on the finer side of the production tolerance, as it is hypothesized that the residual crumb rubber was pushing the aggregate skeleton apart. Stone-on-stone contact verification using the Voids in Coarse Aggregate (VCA) approach would help to rectify this potential issue.

17. Key Word	18. Distribution Statement
FC-5, Durability, Fatigue Cracking, Overlay Tester, Cantabro Abrasion Loss, Indirect Tensile Strength, Hamburg Wheel Tracking Test	

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EXECUTIVE SUMMARY

Like many state transportation agencies in the United States, the Florida Department of Transportation (FDOT) is utilizing open graded friction course (OGFC) mixtures because of the many benefits they provide. Cooley et al (2009) has described the benefits in three different categories: safety, driving comfort and environmental. Benefits of OGFCs related to safety include reduced potential for hydroplaning, improved skid resistance (especially during wet weather), reduced splash and spray, and reduced light reflection (especially at night). Because of the safety benefits of OGFCs, drivers feel an increased confidence when driving during rain events. This increased confidence leads to increased driving speeds (thus less congestion) during wet weather and thus less wet weather accidents (when combined with the improved wet weather friction characteristics). Environmental benefits related to the use of OGFCs include a reduction in tire/pavement noise, increased pavement smoothness (thus, improved fuel economy), and improved quality of stormwater runoff. However, FDOT has discovered that the in-service life of their OGFC mixtures (called FC-5) is less than their dense graded friction course mixtures. The primary distresses observed by FDOT on their FC-5 mixtures are raveling and top-down cracking Improving the life expectancy of these mixtures will provide tangible benefits with regard to the overall performance of asphalt pavements in Florida.

A research effort was conducted to evaluate the field performance and mixture components of FDOT's FC-5 mixtures in an effort to improve their longevity. Data mining of the FDOT's Pavement Management System (PMS) and Laboratory Information Management System (LIMS) yielded valuable information regarding the relationship between the durability of FDOT's FC-5 mixtures and FC-5 volumetric parameters at which the mixtures where designed and produced at. A resultant field visit to FC-5 field sections selected based on their respective PMS field performance showed that many of the FC-5 field sections visited illustrated signs of raveling, but that cracking was limited and classified on longitudinal in nature. Therefore, based on the Pavement and Laboratory Information Management data analyzed, it would appear that increasing the effective asphalt content of the FC-5 mixtures would improve their durability performance and extend their in-service life.

Currently, FDOT utilizes the pie-plate procedure (FM 5-588, *Determining the Optimum Asphalt Binder Content of an Open-Graded Friction Course Mixture Using the Pie Plate Method*) to determine the optimum asphalt content of their FC-5 asphalt mixtures. To help achieve increased effective asphalt contents, slight modifications to the FM 5-588 test procedure, as well as other durability related performance tests, were evaluated with various FC-5 mixtures comprised of aggregates and asphalt binders from different sources.

Since the PMS analysis indicated that effective asphalt content was highly responsible for the durability and fatigue performance of the FC-5 mixtures in the field, the laboratory experiment focused their efforts on how effective asphalt could possibly be increased while not being detrimental to the mixture's stability. During the FC-5 mixture design phase of determining optimum asphalt cthe Optimum Asphalt content using the pie-plate method, it was found that by using an asphalt binder that would be utilized during actual field production (i.e. – PG 76-22 or ARB-12), instead of the currently used PG 67-22, higher optimum asphalt contents could be determined. This is primarily due to the increased viscosity of the binders. When the increased binder content was evaluated, it was determined that durability of the mixtures in the Cantabro Abrasion Loss test increased while having no detrimental impact on the draindown of the mixture. The possible use of a 9.5 mm NMAS FC-5 mixture was evaluated using current

FDOT practices and a recommendation aggregate gradation based on previous NCHRP studies. The laboratory evaluation found that although the 9.5 mm NMAS mixtures did help to improve the general durability and fatigue resistance of the FC-5 mixtures, rutting/stability issues may exist as measured by the Hamburg Wheel Track test. Additional aging using the long-term oven aging procedure in AASHTO R 30 also showed that an additional 0.6% asphalt binder, previously determined as “Optimum” for the PG 76-22 and ARB-12 asphalt binders using the pie-plate test, helped to improve both durability and fatigue cracking even after additional oxidative aging. The influence of production tolerances were also evaluated and determined to be more of an issue with FC-5 mixtures containing ARB-12 asphalt binder. When gradations ran towards the fine side of the production tolerance, it is hypothesized that the residual crumb rubber in the ARB-12 asphalt binder is limiting the stone-on-stone contact of the FC-5 mixture and creating both durability (Cantabro Abrasion Loss) and rutting (Hamburg Wheel Track test) issues.

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CHAPTER 1 – INTRODUCTION

Like many state transportation agencies in the United States, the Florida Department of Transportation (FDOT) is utilizing open graded friction course (OGFC) mixtures because of the many benefits they provide. Cooley et al (2009) has described the benefits in three different categories: safety, driving comfort and environmental. Benefits of OGFCs related to safety include reduced potential for hydroplaning, improved skid resistance (especially during wet weather), reduced splash and spray, and reduced light reflection (especially at night). Because of the safety benefits of OGFCs, drivers feel an increased confidence when driving during rain events. This increased confidence leads to increased driving speeds (thus less congestion) during wet weather and thus less wet weather accidents (when combined with the improved wet weather friction characteristics). Environmental benefits related to the use of OGFCs include a reduction in tire/pavement noise, increased pavement smoothness (thus, improved fuel economy), and improved quality of stormwater runoff. However, FDOT is discovering that the in-service life of their OGFC mixtures (called FC-5) is less than their dense graded friction course mixtures. The primary distresses observed by FDOT on their FC-5 mixtures are raveling and top-down cracking.

1.1 Raveling Distress

Raveling has been reported as a common distress found in OGFC mixtures. Huber (2000) states that OGFC typically end up failing due to raveling. Molenaar and Molenaar (2000) have described two forms of raveling; short-term and long-term. Short-term raveling is caused by intense shearing forces at the tire/pavement interface that occurs within newly placed OGFC mixtures. Conditions that enhance the potential for short-term raveling include placing the OGFC at too low a temperature, incomplete seating of aggregates during compaction and draindown segregation (areas where asphalt content is low). Long-term raveling is typically caused by long-term segregation of the asphalt binder from the aggregates due to gravity (Molenaar and Molenaar, 2000). As the asphalt binder drains from the coarse aggregate structure due to gravity, the aggregates near the surface of the layer are underasphalted. The action of traffic, combined with a greater propensity to oxidized and become brittle, result in raveling. It should be stated that the long-term draindown of the asphalt is encountered more when unmodified, lower viscosity asphalt binders.

Raveling may also be caused by poor bonding to the underlying surface. Because of the open grading in the aggregate skeleton, there is very little aggregate surface area, which results in a relatively thick film of asphalt on the OGFC stone, but also reduces the contact surfaces between the OGFC mixture and the underlying surface. Poor seating during construction and inadequate tacking conditions (i.e. – low tack coat rates, dirty/dusty surfaces, etc.) are primary reasons bonding strength is reduced.

Traditional hot mix asphalt segregation (thermal and aggregate) also plays a role in the development of raveling in OGFC mixtures. These scenarios lend to the theory that raveling of OGFC mixtures may be more related to the production/construction of OGFC mixtures than the mixture components themselves.

1.2 Top-Down Cracking Distress

Top-down (or surface initiated) cracking is not a new phenomenon. However, the amount of research that has been conducted on this subject has steadily increased over the last ten or so years. Currently, there are no universally accepted models that capture the initiation and propagation of surface initiated cracks. However, most research indicates that surface initiated cracks occur when the stresses near the pavement surface exceed the strength of the material. From a practical standpoint, the factors contributing to top down cracking can be categorized as mixture properties, load related properties, environmental properties, and structural factors.

According to the 2004 NCHRP 1-42 report (2004), mixture properties that are important in resistance to surface initiated cracking are stiffness and moisture resistance. From the standpoint of stiffness, Myers et al (2002) indicate that the stiffness gradient is important in the initiation and propagation of surface initiated cracks. Pavement structures exposed to the environmental effects will tend to oxidize near the pavement surface more so than at depth. It has been hypothesized that this gradient affects the development of surface initiated cracks. Intuitively, though it is not reported this way, the volume of effective asphalt within the mix at the pavement surface should have a significant influence on whether surface initiated cracks occur. Higher volumes of effective asphalt (that don't cause stability problems) will help resist oxidation due to environmental effects. Thicker asphalt films should retain tensile strength by resisting oxidative hardening.

Load related properties include the amount and type of traffic. Specifically, the tire-pavement contact stresses have been found important in the occurrence of top-down cracking. Environmental factors include age hardening, thermal stresses, and moisture damage. According to the NCHRP 1-42 report (2004), thinner pavements result in higher tensile stresses at the pavement surface than thicker pavements. Therefore, the pavement structure may also influence the development of surface initiated cracks.

1.3 Current FDOT Mix Design Procedure for FC-5 Mixtures

An FC-5 mixture is typically composed of two to three virgin aggregates, fibers, asphalt binder (asphalt rubber binder for lower traffic levels or PG 76-22 for higher traffic levels), and an anti-stripping agent. Section 337 of the Florida Standard Specifications currently specifies asphalt concrete friction courses used in Florida, including the FC-5. The method for designing FC-5 mixes is contained in FM 5-588, *Determining the Optimum Asphalt Binder Content of an Open-Graded Friction Course Mixture Using the Pie Plate Method*. This method entails the use of a pie plate (generally a nine inch round Pyrex brand pie plate) to visually determine the amount of asphalt binder that drains from an OGFC mixture at elevated temperatures. Three 1,200 gram aggregate batches are prepared to which hydrated lime (if needed) and fiber are added to the aggregate batches and mixed with a PG 67-22 asphalt binder at three asphalt binder contents. The actual asphalt binder contents are dependent upon the mineralogy of the aggregates used in the design. These three mixes are then immediately placed into a pie plate which is placed into a forced draft oven at a temperature of 320±5ºF for one hour. After the one hour, the pie plates are removed from the oven, allowed to cool, and visually evaluated to determine the amount of asphalt binder that has drained from the aggregate structure. Optimum asphalt binder content is determined visually using example pictures contained within FM 5-588. It should be noted that FDOT currently conducts all FC-5 mix designs and thus has extensive experience in the visual ratings.

Besides the selection of optimum asphalt binder content using the pie plate method, no other performance tests or indicators are required during the design of FC-5 mixes. According to Cooley et al (2009), there are a number of performance indicators and/or performance tests that have been used for OGFC mixes. Cooley et al (2009) stated that the most common test used for durability in the world is the Cantabro Abrasion Loss test. This test is used throughout Europe (where the current high air void content OGFCs originated) as well as by many DOTs in the US. Another test method that is used extensively with both OGFC and stone matrix asphalt is the draindown test. Several versions of the draindown test are available, including the pie plate method. Likely the most common draindown test in the US is the draindown basket test described in AASHTO T 305. Another test that has been used in designing OGFC mixes is permeability. Permeability testing has generally been combined with volumetric properties, namely air void content, to ensure that designed OGFC mixes have sufficient draining capacity. To ensure the stability of OGFC layers, the use of the voids in coarse aggregate (VCA) concept has been purported to require the existence of stone on stone contact. Cooley et al (2009) also reported that several types of loaded wheel testers have been utilized to evaluate OGFC mixes. The Asphalt Pavement Analyzer and the Hamburg Wheel Tracking Device have both been used in the US. Another wheel tracking device has been used in Europe to evaluate the raveling potential of OGFCs. This device is called the Wheel Fretting Test.

1.4 Recent Research by FDOT Regarding FC-5 Mixtures

In 2006, researchers at the University of Florida completed a study that evaluated thick open graded and bonded friction courses for Florida (Birgisson, et al., 2006). The study encompassed the development of a new mixture design procedure for “porous friction courses” in Florida that mirrored current in-place procedures specified for Porous European Mixes (PEM) and Georgia DOT’s PEM mixtures. Based on the results generated during the study, the researchers recommended the following and Table 1;

- Compacting specimens to 50 gyrations in the gyratory compactor to air void levels between 18 and 22%;
- Determine optimum asphalt content at the minimum VMA;
- Evaluate draindown using AASHTO T 305
- Determine effective film thickness (recommend > 34 microns)
- Evaluate moisture sensitivity using AASHTO T 283.

The above mixture design parameters are consistent with what has been recommended under NCHRP Project 9-41, *Performance and Maintenance of Permeable Friction Courses* (Cooley et al., 2009).

Table 1.1 – Proposed Gradation and Design Specifications for Florida Porous Friction Courses (after Birgisson et al., 2006)

Asphalt Concrete	12.5 mm PFC
Gradation Requirement	
3/4 in (19 mm) Sieve	100
1/2 in (12.5 mm) Sieve	80 - 100
3/8 in (9.5 mm) Sieve	35 - 60
No. 4 (4.75 mm) Sieve	10 - 25
No. 8 (2.36 mm) Sieve	5 - 10
No. 200 (75 μ) Sieve	1 - 4
Design Requirements	
Range for % AC	5.5 - 7.0
AASHTO T-283 (TSR)	80
Drain-down, AASHTO T305 (%)	< 0.3

Along with the mixture design recommendations, Birgisson et al. (2006) also recommended general ranges of asphalt content based on typical absorptions of predominant aggregate mineralogy used by FDOT for FC-5 mixtures (Table 2).

Table 1.2 – Recommended Asphalt Binder Content Ranges for Typical Aggregates Used in FC-5 Mixtures (after Birgisson et al., 2006)

Aggregate Type	Binder Content
Crushed Granite	5.5 - 7.0
Crushed Limestone (Oolitic)	6.5 - 8.0

In 2009, researchers at the University of Florida conducted another study on FDOT's FC-5 mixtures in an attempt to evaluate the fracture resistance of FDOT's open graded friction course materials (Roque et al., 2009). Roque et al., (2009) utilized continuum damage theory and micro-mechanics based finite element modeling, along with an in-house fracture test to evaluate FE properties and cracking potential of FC-5 mixtures. The researchers indicated that while evaluating the fracture resistance of OGFC and HMA mixtures, OGFC mixtures had lower resilient modulus and failure limits than dense graded asphalt mixtures which resulted in lower fracture energies in the OGFC mixes. This is not really surprising since an OGFC generates it's "strength" from the stone-on-stone contact and the confinement encountered in the field. Additionally, OGFCs do not have true mortars/mastics. Strength and stiffness of OGFC mixtures in a tensile mode of failure will come almost entirely from the binder properties with some influence of the fiber and limited fines. Additional testing on composite mixtures by Roque et al (2009) indicated that the fracture resistance of OGFC mixtures improves in a pavement system when using a polymer modified bonding agent between the OGFC and binder course.

Again in 2009, FDOT sponsored two pavement test sections on the National Center for Asphalt Technology (NCAT) Test Track to examine the field performance of an FC-5 mixture with two different bonding materials; 1) Test Section N1: Polymer modified tack (CRS-2P (SBS)) at 0.21 gal/sy and 2) Test Section N2: Neat tack (NTSS-1HM) at 0.05 gal/sy.

Construction of the FC-5 mixes were completed in July of 2009 and placed over a 100 gyration Superpave mixture that was known to be prone to top-down cracking. Approximately seven months later, longitudinal cracking was observed in both test sections (Figure 1.1). Cores eventually extracted through the surface cracks indicated that the cracks had propagated completely through the FC-5 mixtures and into the binder course below (in set picture of Figure 1.1). Conclusions on the performance of the FC-5 mixtures on the NCAT Test Track noted that:

1. The OGFC layer on Section N1, in which a heavier tack coast was applied, performed better than that of Section N2, in which a conventional tack coast was used.
2. While both sections had cracking, the level of severity and the area of severe cracks were greater in Section N2 than Section N1.
3. It was recommended that to help resist cracking in OGFC layers, heavier tack coats be used.

Figure 1.1 - Top-down Cracking at the NCAT Test Track for FDOT's FC-5 Mixtures

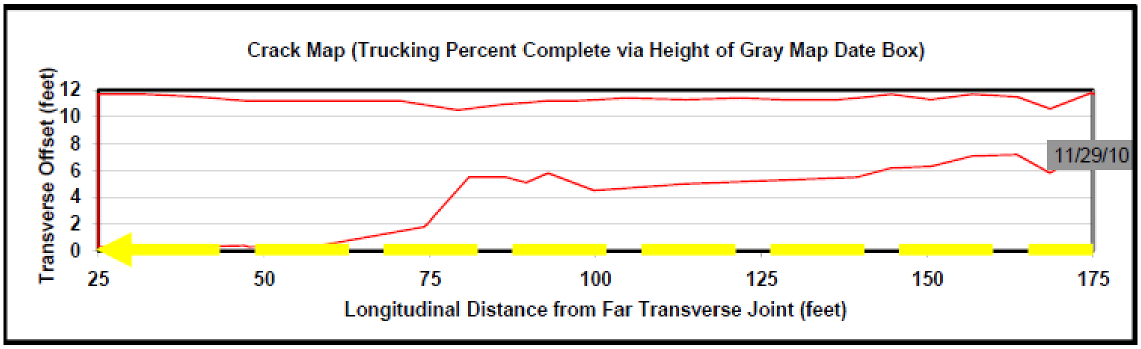
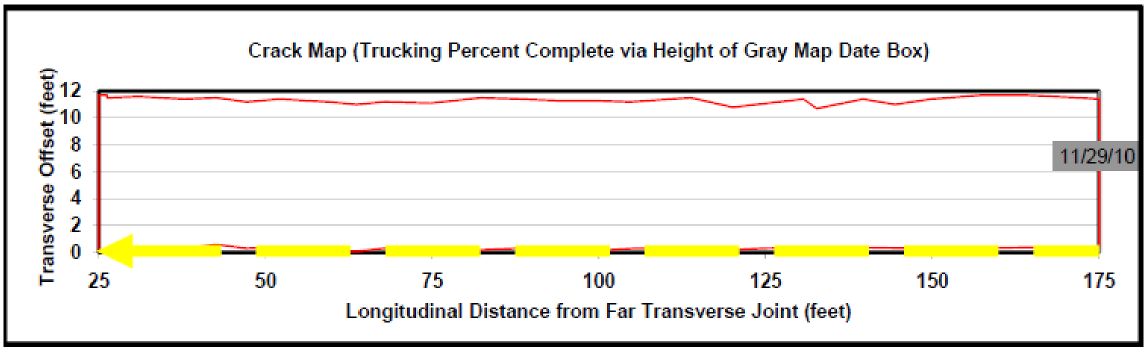


Figure 1.2 - Crack Mapping at the NCAT Test Track - Test Section N1



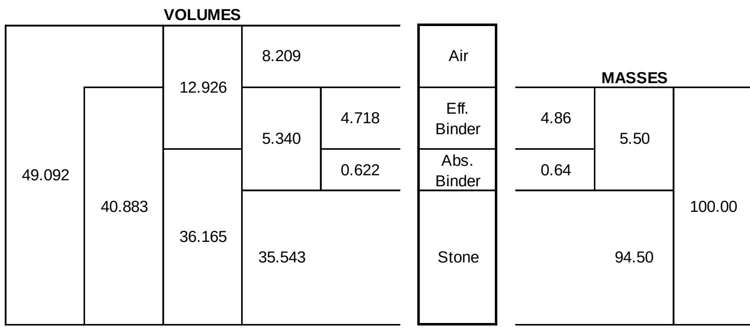
(b)

Figure 1.3 – Crack Mapping at the NCAT Test Track – Test Section N2

A further evaluation of the volumetrics of the FC-5 mixture placed on sections N1 and N2 was also conducted to help further understand the relationship between the field performance at the NCAT Test Track and the volumetrics of the FC-5 mixtures. Figure 1.4 contains the resultant phase diagram and volumetrics determined from the data provided on the NCAT Test Track website (www.PaveTrack.com, 2010). The information contained in the phase diagram is interesting. First, the design effective asphalt binder content is 4.86 percent (by total mix mass) for both sections. Additionally, the volume of effective binder (VMA minus air voids) was 9.3 percent. Compare these values to Section S3 of the NCAT Test Track (Figure 1.5). Section S3 of the NCAT Test Track was placed in 2006 and has been through one complete cycle of traffic and was in its second cycle. Aggregates used in Section S3 were chert gravel aggregates from Mississippi, which some deem inferior to the granites used in Sections N1 and N2. Halfway through the second cycle, no cracking was observed in Section S3 (Figure 1.6). Two observations are provided comparing the Florida sections and Section S3. First, the effective asphalt binder content was 5.65 percent in Section S3, approximately 1 percent higher than the Florida sections. Secondly, based upon volume, there was approximately 5 percent more volume of effective asphalt binder in Section S3 than in the Florida sections. Another observation that may explain why the Florida sections have experienced cracking was the existence of 15 percent RAP in the Florida sections. Depending upon whether one believes the "total blending" or "black rock" theory for the use of RAP within HMA, the reality is that RAP truly is somewhere between the two theories. If this is the case, then the "true" effective volume of asphalt binder within the

Florida sections is likely less than that shown on the phase diagram. This discussion on the different sections brings forth the probability that a major cause of cracking in FC-5 mixes was insufficient volume of effective asphalt binder. It should also be noted that the underlying layer below sections N1 & N2 and the chert gravel OGFC (S3) were different.

Date	15-Jul-09	Notes	Design Data for FC-5 (No QC data presented) - Granite/RAP Aggregates
Lab	NCAT		
Layer	Section N1 and N2		



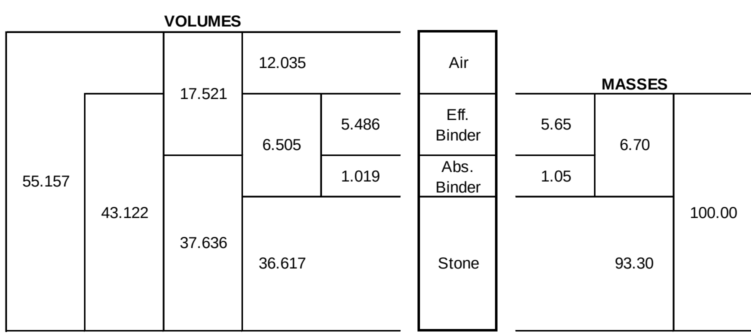
Sieve Size, mm	Passing	S.A.
25	100.0	200
19	100.0	
12.5	95.0	
9.5	64.0	
4.75	15.0	30
2.36	9.0	36
1.18	8.0	64
0.6	6.0	84
0.3	5.0	150
0.15	4.0	240
0.075	3.7	592

13.960	ft^2/lb
2.859	m^2/kg
4.718	V_{be} (cm^3)
0.286	S.A (m^2)
2859	S.A. (cm^2)
16.50	T_F

Knowns		*Calculated*			*Calculated*	
P_b =	5.50	G_{se} =	2.659		VMA =	26.33
G_{sb} =	2.613	% AC	10.88	(volume %)	VTM =	16.72
G_{mm} =	2.446	% AC eff.	9.61	(volume %)	VFA =	36.50
G_b =	1.030	P_{be} =	4.86		P_{200}/P_{be} =	0.76
G_{mb} =	2.037	P_{ba} =	0.68		T_F =	16.50

Figure 1.4 – Phase Diagram and Volumetrics of FC-5 Mixture at the NCAT Test Track Sections N1 and N2

Date	4-Oct-06	Notes	QC Data at Time of Construction - Mississippi Gravel PFC
Lab	NCAT		
Layer	Section S3		



Sieve Size, mm	Passing	S.A.
25	100.0	200
19	100.0	
12.5	100.0	
9.5	92.0	
4.75	31.0	62
2.36	12.0	48
1.18	9.0	72
0.6	8.0	112
0.3	6.0	180
0.15	5.0	300
0.075	3.8	608

15.820	ft^2/lb
3.240	m^2/kg
5.486	V_{be} (cm^3)
0.324	S.A (m^2)
3240	S.A. (cm^2)
16.93	T_F

Knowns		*Calculated*			*Calculated*	
P_b =	6.70	G_{se} =	2.548		VMA =	31.77
G_{sb} =	2.479	% AC	11.79	(volume %)	VTM =	21.82
G_{mm} =	2.319	% AC eff.	9.95	(volume %)	VFA =	31.31
G_b =	1.030	P_{be} =	5.65		P_{200}/P_{be} =	0.67
G_{mb} =	1.813	P_{ba} =	1.12		T_F =	16.93

Figure 1.5 – Phase Diagram and Volumetric for Section S3 (Chert Gravel OGFC) of NCAT Test Track

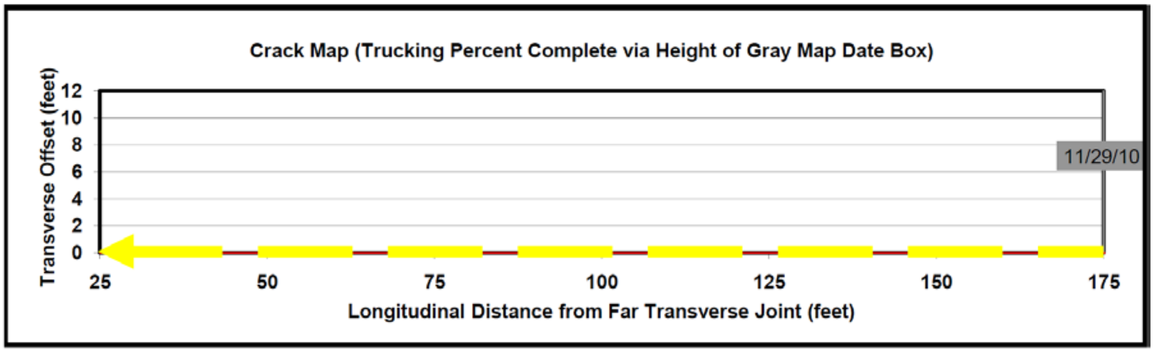


Figure 1.6 – Crack Mapping at the NCAT Test Track – Test Section S3 (Chert Gravel OGFC)

1.5 Research Project Workplan

In an effort to evaluate how the durability and fatigue performance of Florida's FC-5 mixtures can be improved, the Research Team developed and executed an extensive workplan that contained the following components that are described in detail further in the report.

- Comprehensive literature review pertaining to performance of porous friction course mixtures;
- Data mining of FDOT's Pavement Management System (PMS) to determine general performance of FC-5 mixtures;
- Data mining of FC-5 mixture components from the Laboratory Information Management System (LIMS) to determine if a relationship exists between field performance and mixture components;
- Develop a laboratory workplan to;
  - Evaluate how the critical FC-5 mixture components can be modified to improve mixture performance;
  - Evaluate if a possible change in FC-5 gradation can improve mixture performance;
  - Evaluate how changes during production influence the performance of the FC-5 mixtures.
- Generate a final report summarizing all information gathered and generated.

CHAPTER 2 - LITERATURE REVIEW

2.1 Design of OGFC Mixes

In a recent research report, Kline (2010) described three different categories of selecting the optimum asphalt binder content of OGFC mixes used within the US. The first category included methodologies that utilize compaction of OGFC mixes in the laboratory. The second category included methods that use the oil absorption test. Lastly, the final category included methods that use visual observation of loose OGFC mix for draindown properties.

Cooley et al (2009) stated that the design of OGFC mixes was similar to the design of dense-graded mixes in that four steps are involved. The first step is to select appropriate materials. Materials needing selection include coarse aggregates, fine aggregates, asphalt binder and stabilizing additives. The second step is to blend the selected aggregates to select a design gradation. The third step involves selecting the optimum asphalt binder content for the design gradation. Finally, any performance testing is conducted. This section of the literature review is structured to provide information on each of these four steps.

2.1.1 Materials Selection

As stated previously, materials needing selection include coarse aggregates, fine aggregates, mineral fillers, asphalt binders and stabilizing additives. Following are the current state of practice for selection of these materials.

Aggregate Characteristics

A survey of agencies conducted by Cooley et al (2009) included a request for respondents to rank various aggregate characteristics for use in OGFCs. Aggregate characteristics included within the survey were abrasion resistance, durability, polish resistance, angularity, shape, cleanliness and absorption. Results from the survey are illustrated in Figure 2.1. Respondents were requested to rank the various aggregate characteristics on a scale of 1 to 7, with 1 being the most important property and 7 being the least important. Based upon the results of the survey, there appeared to be three levels of importance. Polish resistance and durability were the most important properties as both of these had the lowest average rankings. The next level of importance included angularity, abrasion resistance, particle shape and cleanliness. All four of these characteristics had reasonably similar average ratings. The final level of importance was aggregate absorption. The average rating of this characteristic was much higher than the other six characteristics.

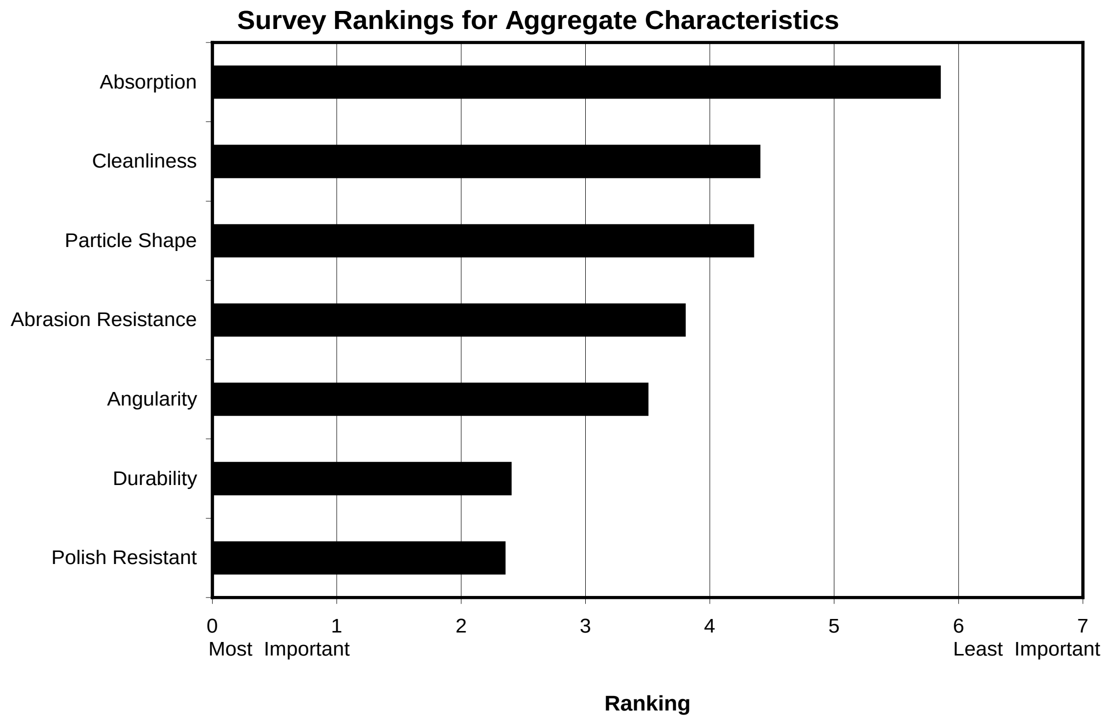


Figure 2.1 - Ranking of Aggregate Characteristics from Agency Survey (Cooley et al., 2009)

As highlighted above, the two most important aggregate characteristics based upon the survey were durability and polish resistance. In Europe, polish resistance is also considered one of the most important aggregate characteristics (Lefebvre, 1993). The Polish Stone Value is the most common requirement specified for ensuring polish resistant aggregate (Ruiz et al, 1990; Lefebvre, 1993).

The predominant test used to evaluate the durability of aggregates is sulfate soundness. Georgia has a maximum loss of 15 percent when determined using magnesium sulfate (Watson et al, 1998). Oregon utilizes a maximum loss of 12 percent when using magnesium sulfate (Huber, 2000).

The Los Angeles Abrasion test is the most common test to evaluate aggregate abrasion/degradation resistance. It is specified both in the U.S. and internationally. Maximum loss values encountered in the literature ranged from a low requirement of 12 percent (Alvarez et al, 2006) to a high of 50 percent loss (Watson et al, 1998). Within the U.S., current recommendations are generally a maximum Los Angeles Abrasion loss of 30 percent (Kandhal, 2002).

Coarse aggregate angularity is most often specified as a minimum number of fractured faces. Generally, specifications are for the percentage of aggregates with two or more fractured faces. Specification values range from a low of 90 percent of the coarse aggregates with two or more fractured faces (Mallick et al, 2000) to a high of 100 percent (Huber, 2000; Lefebvre, 1993).

Cooley et al (2009) recommended use of the uncompacted voids of coarse aggregate test for specifying the angularity of coarse aggregates. A maximum percent voids of 45 was recommended when utilizing Method A.

For fine aggregates, most references simply stated that the fine aggregate fraction should be crushed indicating an angular material. Kandhal (2002) and Cooley et al (2009) recommended using the uncompacted void content of fine aggregate with a specification minimum of 45 percent.

Two tests are generally utilized to specify the desired shape of coarse aggregates, the flakiness index and the flat and elongated test. The flakiness index is generally specified in Europe with a maximum requirement of 25 percent (Huber, 2000; Poulikakos et al, 2006). Arizona has also utilized this specification for the flakiness index (Huber, 2000). Within the US, the flat and elongated test is the most common test to define coarse aggregate particle shape. Requirements for flat and elongated are generally based upon a ratio of 5:1 (Watson et al, 1998) though some guidance specifies ratios of 3:1 and 2:1 (Kandhal, 2002). When a 5:1 ratio is specified, a maximum percentage of flat and elongated particles requirement of 10 percent is common (Watson et al, 1998; Mallick et al, 2000), though some specify a maximum of 5 percent (Kandhal, 2002). When a 3:1 ratio is specified, a maximum requirement of 20 percent is used (Kandhal, 2002; Mallick et al, 2000). Cooley et al (2009) recommended a maximum of 50 percent flat and elongated when using a 2:1 ratio.

Aggregate cleanliness is most often specified based upon the sand equivalent test. Specification values for the sand equivalent test range from a low of 45 (Huber, 2000; Mallick et al, 2000) to a high of 55 (Huber, 2000).

Asphalt Binders

A wide range of asphalt binders have been used in OGFC mixes. Both unmodified and modified asphalt binders have been used with success. In the NCHRP synthesis by Huber (2000), he reported many different types of asphalt binders. These binders were graded in accordance with the Superpave Performance Grading (PG) system, viscosity grading procedure and penetration grading system. Within Europe, the asphalt binders were predominantly graded using the penetration grading system. Huber (2000) reported on material requirements from Britain, Spain, Italy and South Africa. At the time, Britain utilized a 100 pen asphalt binder with and without polymer modification. Spain utilized either a 60/70 or an 80/100 pen asphalt binder with polymer modification. Italy also used an 80/100 pen asphalt binder with polymer modification. Each of these three countries specify either a styrene butadiene styrene (SBS) or ethylene vinyl acetate (EVA) when using polymer modification. Huber (2000) indicated that South Africa allows both polymer modification and modification with rubber.

Within the US, Huber (2000) reported a wide range of asphalt binders being used. Both PG and viscosity graded binders were reported. Some US agencies utilized unmodified asphalt binders. For instance, Arizona was specifying a PG 64-16 and Georgia was specifying a PG 67-22 for some OGFC mixes. However, most agencies specified modified asphalt binders.

Alvarez et al (2006) also provided a synthesis on mix design criteria for PFCs. This work was published in 2006, six years after Huber's synthesis (Huber, 2000). Alvarez et al (2006) reported that asphalt binders used in OGFCs are generally modified.

Proper selection of the asphalt binder to be used within OGFCs should be based upon a number of factors. Ruiz et al (1990) stated that selection of the asphalt binder should be based upon the weather at the project site and the anticipated traffic volume the roadway will carry. Kandhal (2002) also provided similar factors for selection of asphalt binders for PFCs.

Generally, the literature indicates that binders with a high stiffness are needed for OGFCs, hence, most agencies requiring modified asphalt binders. High stiffness binders are needed to help prevent draindown which promotes thick films of asphalt binder coating the aggregates. Molenaar and Molenaar (2000) indicated that stiff, polymer-modified binders also help prevent short-term raveling. Short-term raveling was defined as raveling caused by intense shearing forces at the tire/pavement interface that occurs within newly placed porous asphalt. Ruiz et al (1990) stated that asphalt binders that are too soft may tend to bleed during hot weather and, therefore, lead to rutting problems. Even though stiff binders are desirable, Ruiz et al (1990) also suggested that binders that are too stiff can be detrimental. Asphalt binders that are too stiff may reach a critical hardness earlier which could lead to long-term raveling problems.

Stabilizing Additives

According to the survey conducted by Cooley et al (2009) and literature, one of the primary concerns with OGFCs is draindown during construction. Open-graded mixes have an open aggregate grading with a relatively low percentage of material passing the No. 200 (0.075 mm) sieve. Because of the open grading, the surface area of the aggregate blend is much lower than typical dense-graded mixes, the low aggregate surface area results in relatively thick asphalt binder films coating the aggregates. According to Watson et al (2004a), typical asphalt binder film thicknesses for PFCs are approximately 30 microns compared to approximately 8 microns for dense-graded HMA.

At typical production/construction temperatures, the thick film of asphalt binder common to OGFCs has a propensity to drain from the aggregate structure, termed draindown (Huber, 2000). In order to reduce the potential for draindown, stabilizing additives are generally incorporated into OGFCs. Two types of stabilizing additives can generally be utilized within OGFCs: fibers and asphalt binder modifiers. Many different types of fibers have been used including mineral, cellulose, asbestos, polypropylene, polyacrylonitrile, glass, and acrylic fibers. According to the results of the agency survey conducted by Cooley et al (2009), 85 percent of the responding agencies specified the use of fibers within OGFCs. This value is significantly higher than the 19 percent of agencies reporting the use of fibers within OGFC mixes in the 1998 survey by Kandhal and Mallick (1998).

The increase in the percentage of agencies specifying the use of fibers within open-graded mixes is likely an indication of the effectiveness of fibers in reducing draindown potential. Figure 2.2 illustrates the effect of fiber addition on draindown potential. Data used to create Figure 2.2 is from research conducted by the NCAT on OGFCs and was published by Watson et al (2003) in a slightly different form. Figure 2.2 clearly illustrates that the addition of fiber significantly reduces draindown potential. Similarly, other research projects have shown that the use of fibers significantly reduces the potential for draindown (Mallick et al, 2000). According to Pasetto (2000), additional benefits can be realized from the addition of fibers within OGFC mixes. Pasetto (2000) showed that the addition of fibers increased the strength of OGFC mixes as measured by Marshall stability and indirect tensile testing. Additionally, the use of fibers improved the durability of PFC mixes as measured by the Cantabro Abrasion test.

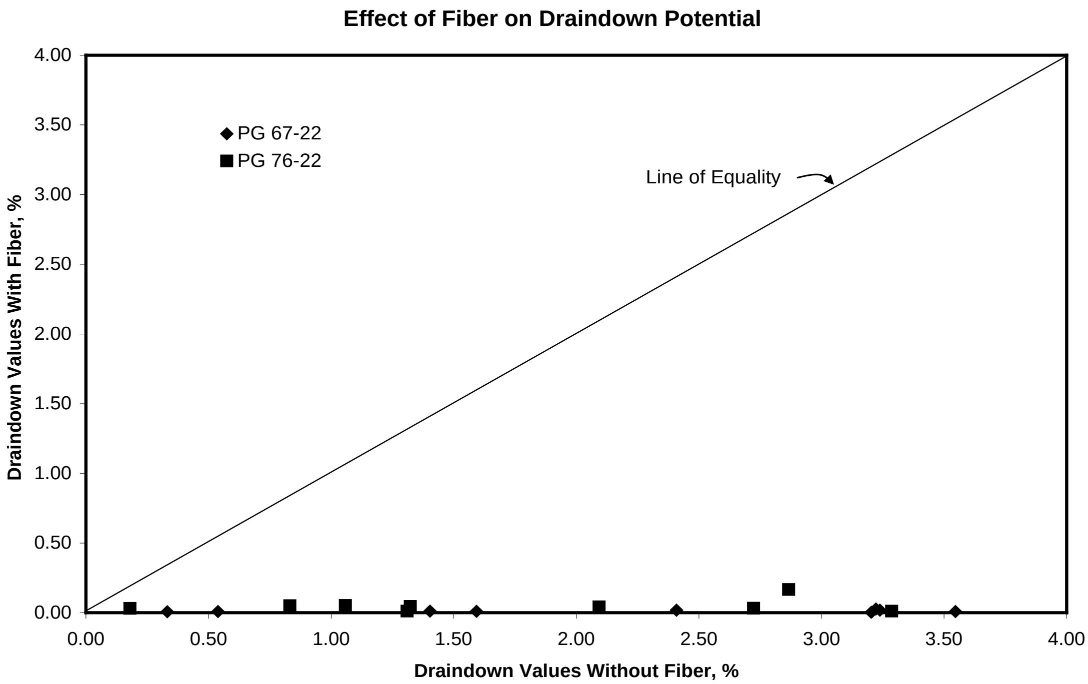


Figure 2.2 - Effect of Fibers on the Draindown Potential of PFCs (Watson et al, 2003)

As stated previously, a wide range of fiber types have been used in OGFCs. Within the US, the most common fiber types used are cellulose and mineral fibers. These two fiber types are also common in Europe (Lefebvre, 1993) and Australia (Alderson, 1996). Addition of fibers is generally at a dosage rate between 0.1 and 0.5 percent, by total mix mass. An important point made by Decoene (1990) is that the selected fibers must be resistant to temperatures above typical production temperatures. This is especially true when using organic fibers.

The other type of stabilizing additive that is commonly used in OGFCs is asphalt binder modifiers. These modifiers are generally polymers or rubber particles. With respect to draindown, these modifiers serve to increase the viscosity (stiffness) of the asphalt binder, thus, helping to maintain the asphalt binder within the aggregate structure. The benefits of modified asphalt binders are not limited to helping prevent draindown. A series of reports and papers from the NCAT (Watson et al, 2003; Watson et al 2004a; Cooley et al, 2000) have shown that the use of modified asphalt binders that provide higher stiffness at typical in-service temperatures help provide increased durability in the laboratory. These results match field experiences described by Huber (2000) and the results of the 1998 survey of US agencies by Kandhal and Mallick (1998). Huber (2000) indicated that in the past, thick films of unmodified asphalt binder tended to drain downward during hot summer weather due to gravitational forces. The remaining thin films of asphalt binder coating the aggregates would age more rapidly becoming brittle, which resulted in raveling. Use of modified asphalt binders helped to retain the thick asphalt binder film, thus improving durability. In addition, research has shown that the use of modified asphalt binders improves the short-term performance of OGFCs. The increased stiffness of the asphalt binder reduces the potential for traffic dislodging aggregate particles shortly after construction. This early age dislodging of aggregate particles has been termed short-term raveling (Molenaar and Molenaar, 2000).

Fillers/Adhesion Agents

A number of agencies from around the world specify the use of fillers or other adhesion agents to improve the bond between aggregates and the asphalt binder. Van Der Zwan et al (1990) state that limestone filler is added during the production process to improve bonding in the Netherlands. The limestone filler must have a hydrated lime content of at least 25 percent. Australia also requires the addition of a filler to OGFC mixes (Alderson, 1996). Hydrated lime is the preferred type of filler in Australia; however, Portland cement and ground limestone are also allowed. Similarly, Watson et al (1998) stated that hydrated lime is required in OGFC mixes in Georgia as an anti-stripping agent.

In their 1998 survey of US agencies, Kandhal and Mallick (1998) evaluated the reported performance of open-graded mixes with various mix design practices. One of the mix design items included within the evaluation was whether the agency specified fillers/adhesion agents. In order to better evaluate the information, Kandhal and Mallick (1998) divided the various agencies by the Strategic Highway Research Program climatic zones in which each resided. These climatic zones included Wet-Freeze, Wet-No Freeze, Dry-Freeze, and Dry-No Freeze. Collectively, of the 19 agencies reporting good performance, 53 percent added some type of filler/adhesion agent whether the material was hydrated lime or liquid antistrip materials. Conversely, only 21 percent of the agencies reporting bad performance with open-graded mixes specified the use of fillers/adhesion agents. Interestingly, all of the agencies reporting good performance within the Dry-Freeze climatic zone specified hydrated lime, while 75 percent of the agencies reporting bad performance in this climatic zone did not specify fillers/adhesion agents.

2.1.2 Selection of Design Gradation

The next step within the design of OGFC mixes is to utilize the selected aggregates to develop a design gradation (or design aggregate structure). Within a typical mix design, this step may include developing several trial gradations and using mix design criteria to select the most appropriate of the trial gradations. Within this section, only typical OGFC gradations will be discussed because the following section will provide the different mix design criteria encountered within the literature.

The literature review for this project and survey of agencies conducted by Cooley et al (2009) resulted in a wide range of gradations encountered for OGFC mixes. The majority of the gradation requirements encountered throughout the world could potentially be characterized by multiple nominal maximum aggregate sizes (as defined in Superpave) depending upon the actual blended gradation. Therefore, within this document gradations will only be discussed by the maximum aggregate size (finest sieve with 100 percent passing).

Oregon was the only US agency that had gradation criteria for a 1 in (25 mm) maximum aggregate size gradation. This gradation is illustrated in Figure 2.3. This figure shows that the gradation is gapped on the No. 4 sieve. The allowable filler content for this gradation band is 1 to 6 percent.

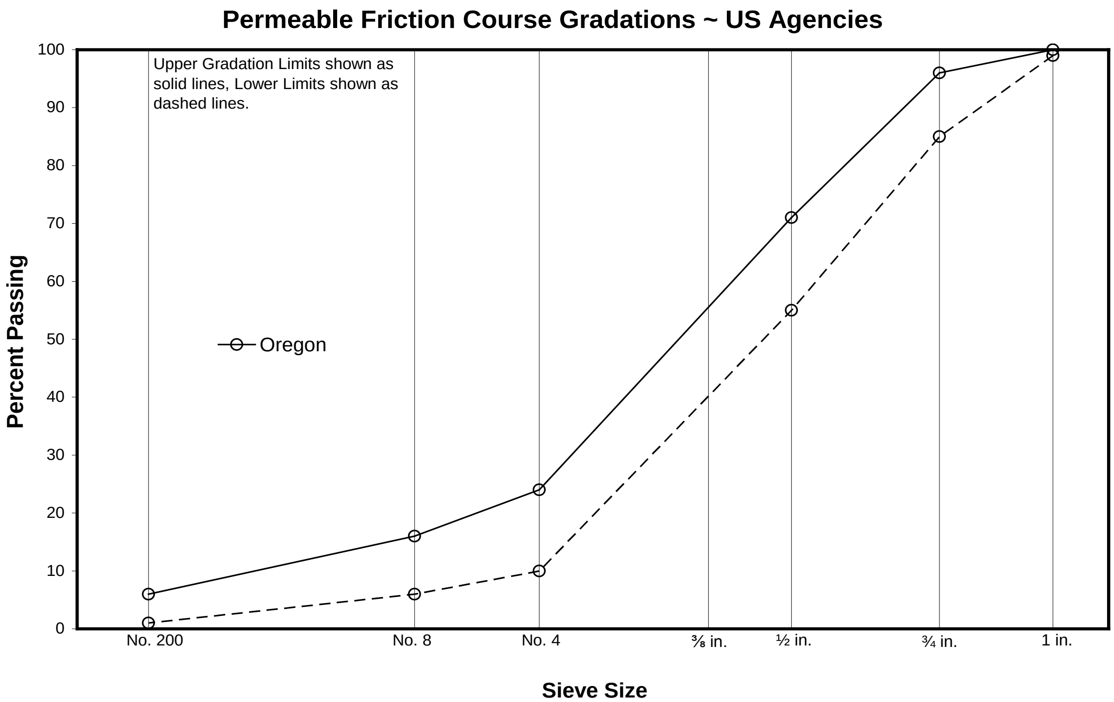


Figure 2.3 - 1 inch PFC Gradation Requirements from US Agencies

A number of agencies provide gradation requirements for a ¾ in (19 mm) maximum aggregate size. Figure 2.4 illustrates the various PFC gradation bands specified in the US. Gradations shown within this figure typically are gapped on the No. 4 sieve; however, some allow for gapping the gradation on the No. 8 sieve. Allowable filler contents range from a low of 1 percent to a high of 5 percent. Interestingly, several agencies have identical (or almost identical) gradation requirements for ¾ in. maximum aggregate size OGFCs. Alabama, Georgia, Louisiana, and South Carolina all specify essentially the same gradation requirements. These specifications can be traced to the original PEM utilized by Georgia (Watson et al, 1998) and the research conducted by the National Center for Asphalt Technology (Watson et al, 2003).

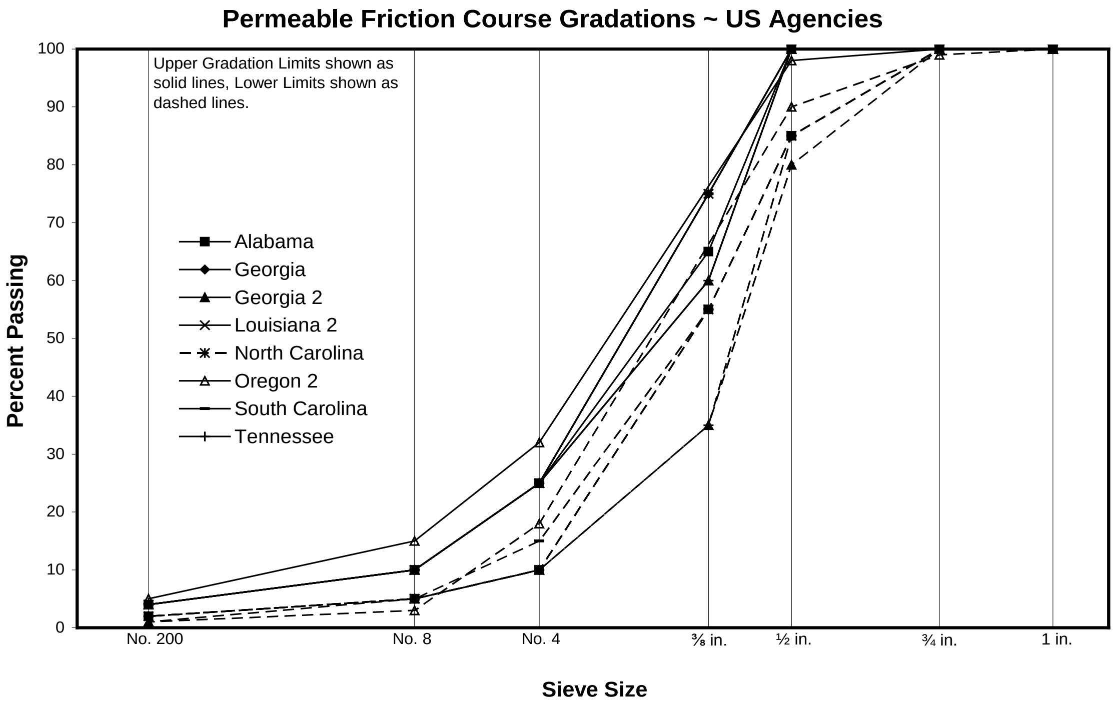


Figure 2.4 - ¾ inch PFC Gradation Requirements from US Agencies

Louisiana was the sole agency that provided gradation requirements for a ½ in (12.5 mm) maximum aggregate size OGFC which is illustrated in Figure 2.5. For this gradation band, the aggregates are gapped on the No. 8 sieve. Filler criteria include a minimum of 2 percent and a maximum of 4 percent.

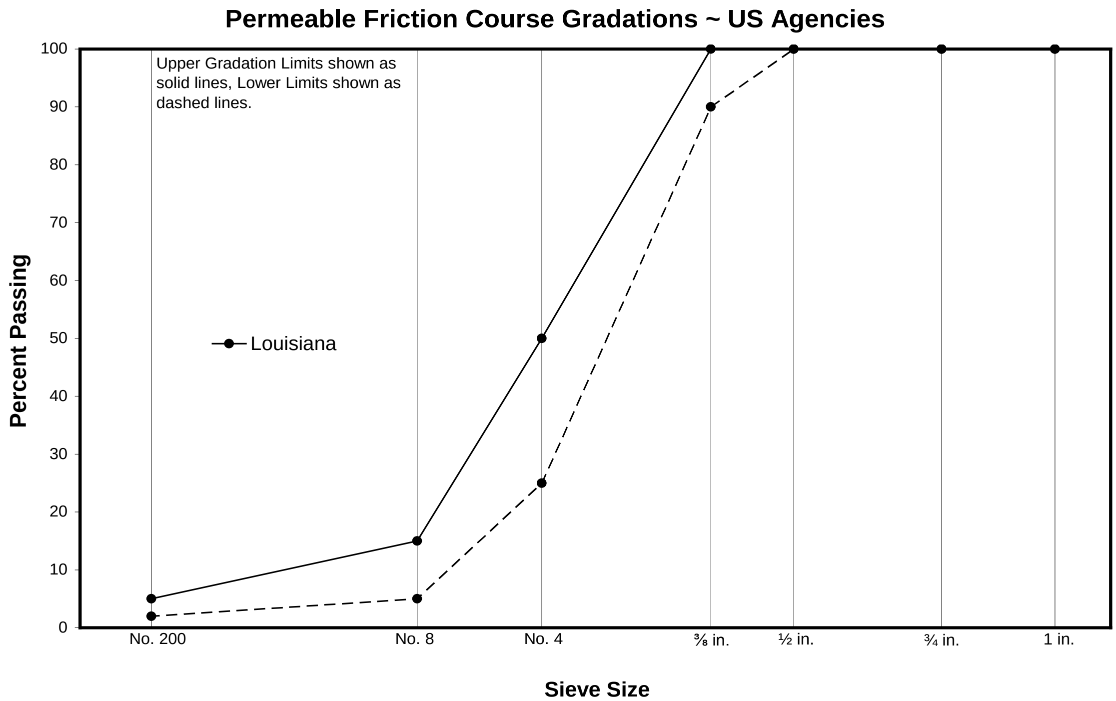


Figure 2.5 - ½ inch PFC Gradation Requirements from US Agencies

From an international standpoint, there are a number of agencies that specify open-graded mixes. A single 1 in. (25 mm) maximum aggregate size gradation was encountered in the literature which was from Britain. This gradation is illustrated in Figure 2.6. According to the gradation band, the gradation is gapped near the No. 4 sieve and the allowable filler content is between 3.5 and 5.5 percent.

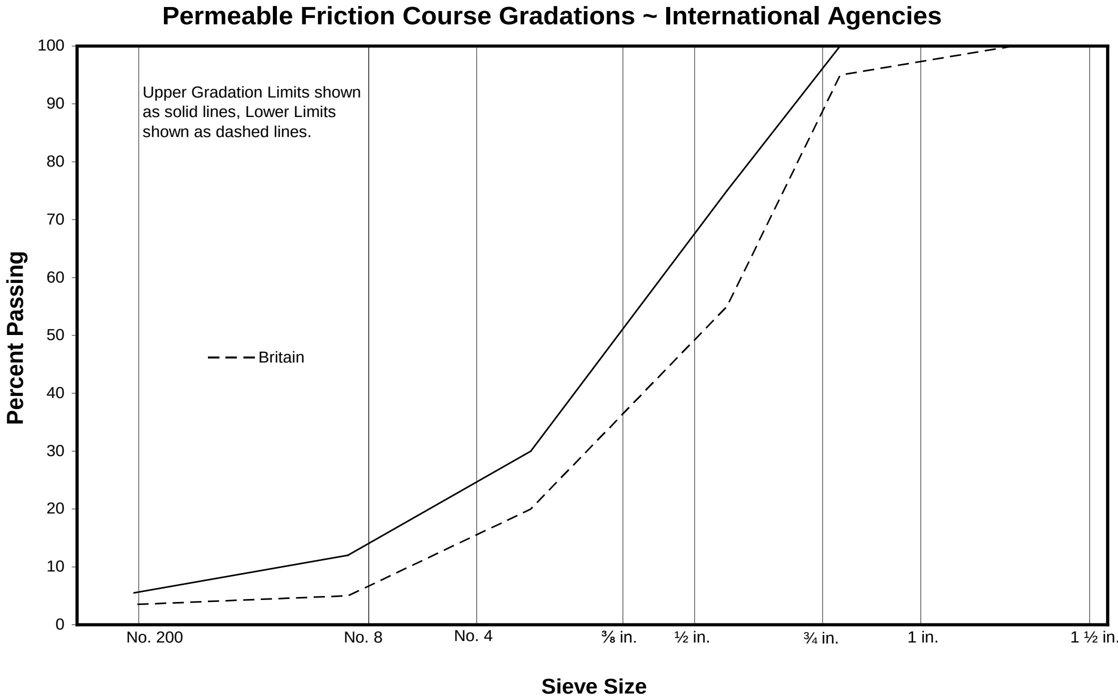


Figure 2.6 - 1 inch PFC Gradation Requirements from International Agencies

Most of the gradation requirements specified by international agencies are for a ¾ in. (19mm) maximum aggregate size gradation. Figure 2.7 shows the various gradation bands for ¾ in. maximum aggregate size OGFC mixes. Again, customary US sieves are shown on the figure. Also shown on this figure is the gradation band recommended by the National Center for Asphalt Technology (Watson et al, 2003). This gradation band is shown to provide a comparison between the typical gradation used in the US (as described above) and those used in other countries. As shown on this figure, there is a wide range of allowable gradations. For instance, on the ⅜ in. (9.5 mm) sieve, gradation requirements range from a high of approximately 75 percent passing (Spain) to a low of approximately 10 percent passing (Italy). The majority of gradation bands would force the aggregate blend to be gapped somewhere between the ⅜ in. (9.5 mm) sieve and the No. 4 (4.75mm) sieve. Filler contents encountered in the various gradation bands also vary significantly. Italy provides a lower limit of 0 percent passing the No. 200 (0.075mm) sieve while South Africa allows as much as 8 percent passing the No. 200 sieve.

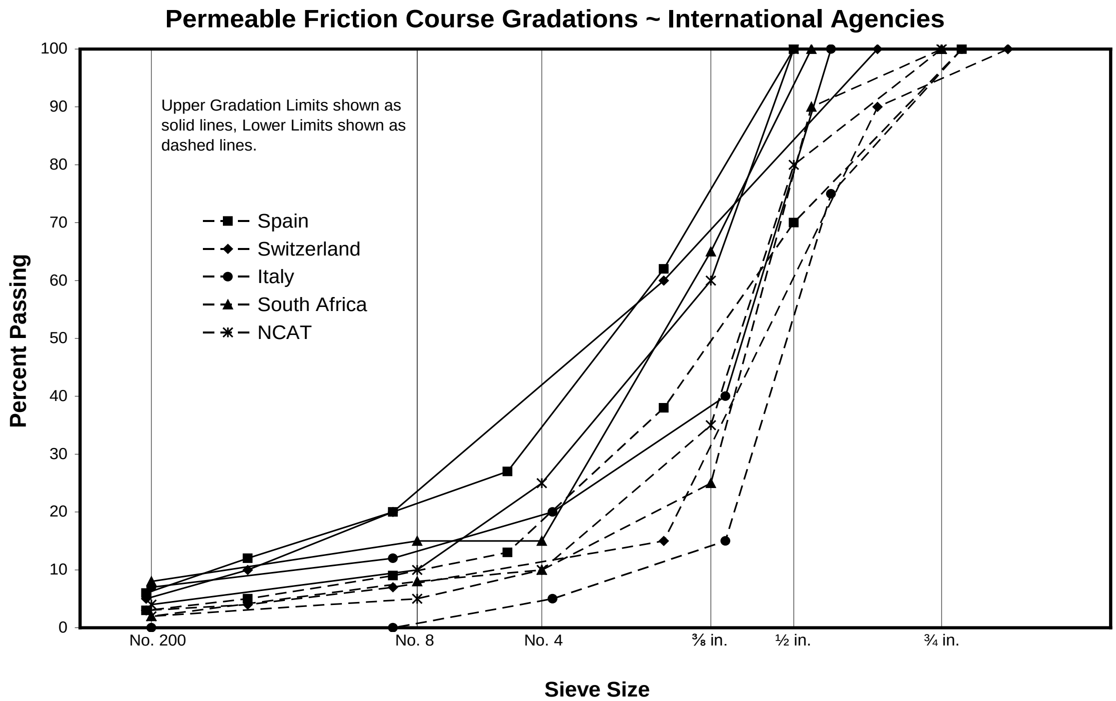


Figure 2.7 - 3/4 inch PFC Gradation Requirements from International Agencies

Figure 2.8 illustrates the single ½ in. (12.5mm) maximum aggregate size gradation encountered in the literature. This gradation band is specified in Britain. According to the figure, this ½ in. maximum aggregate size gradation would be gapped on either the No. 4 (4.75mm) or No. 8 (2.36mm) sieve. The percentage of filler allowed within this gradation band is between 3 and 6 percent.

Several authors indicated that the maximum aggregate size selected for PFC will have an effect on permeability. Ruiz et al (1990) indicated that larger maximum aggregate size gradations result in more permeability.

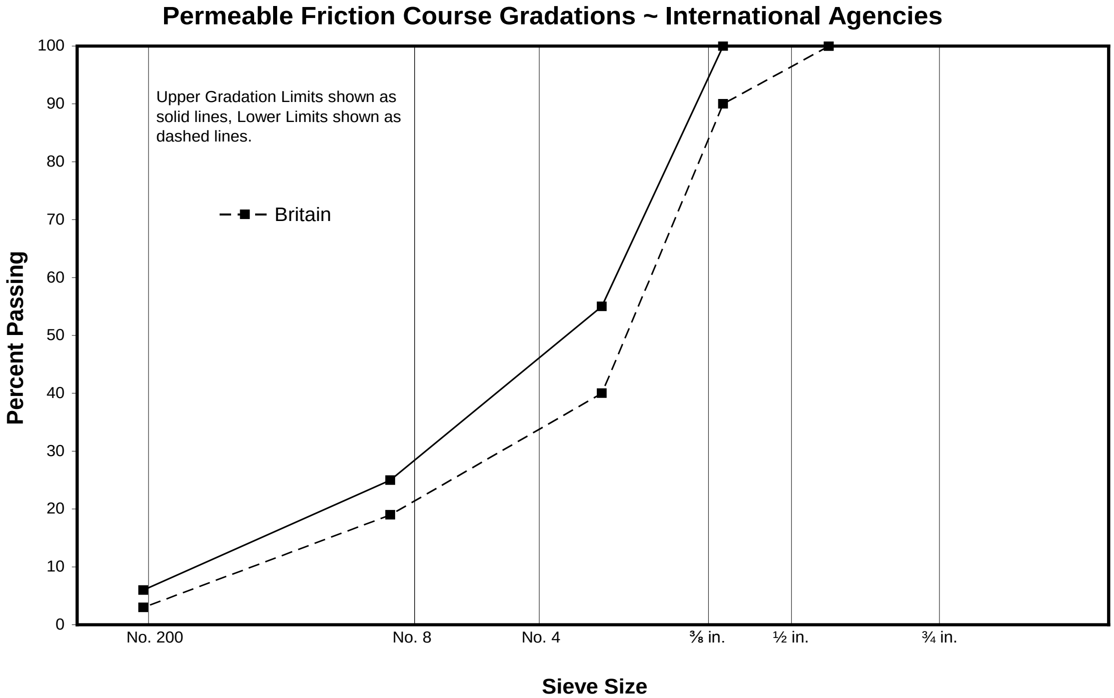


Figure 2.8 - 1/2 inch PFC Gradation Requirements from International Agencies

2.1.3 Selection of Optimum Binder Content

The philosophy of selecting the optimum binder content for OGFC mixes is relatively uniform around the world. However, no specific process or procedure was identified that provided an absolute optimum asphalt binder content. Rather, mix design methods generally identify a range of allowable asphalt binder contents from which optimum can be selected. This fact was discussed by Kline (2010) for mix design methods utilized in the US. Two properties are generally utilized to define the range of allowable binder contents: durability and draindown potential. It should be stated, however, that some mix design methods also require a minimum air void content.

Figure 2.9 illustrates the general concept for selecting the allowable range of asphalt binder contents from which optimum is selected. Within this figure, durability is defined as the amount of loss from the Cantabro Abrasion test. This test evaluates the resistance of compacted open-graded specimens to abrasion. The test method entails compacting mix to the laboratory standard compactive effort, allowing the specimen to cool to room temperature, weighing the specimen to the nearest 0.1 g and then placing the specimen into a Los Angeles Abrasion machine without the charge of steel spheres. The Los Angeles Abrasion machine is then operated for 300 revolutions at a rate of 30 to 33 rpm. After the 300 revolutions, the specimen is removed and again weighed to the nearest 0.1 g and the percent mass loss determined based upon the original specimen mass. This test method was developed in Spain during the 1980's (Lefebvre, 1993). Within the literature, this is the most common test utilized to evaluate the durability of OGFCs.

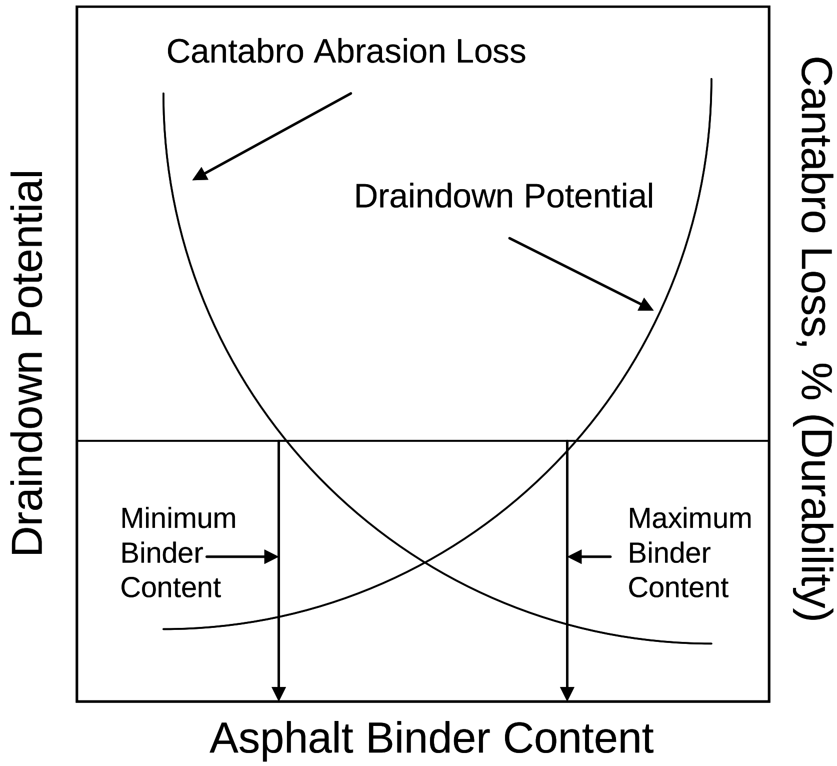


Figure 2.9 - Philosophy of Designing PFC Mixes

As shown in Figure 2.9, the Cantabro Abrasion test is used to identify a minimum asphalt binder content. As asphalt binder content increases, durability is improved. A maximum asphalt binder content is identified by conducting some type of draindown potential test. The concept being that more asphalt binder improves durability, but too much asphalt binder leads to draindown.

As stated above, the Cantabro Abrasion test is the most common test utilized worldwide to evaluate the durability of OGFC mixes. Each country specifying the Cantabro Abrasion test utilizes the same test method with regards to the number of revolutions and rate of revolution within the Los Angeles Abrasion machine. The only variable identified within the Cantabro Abrasion test is the temperature at which the test is conducted. Spain and Belgium utilize a test temperature of 64°F (18°C) (Lefebvre, 1993) and a test temperature of 68°F (20°C) is used in France (Fortes and Merighi, 2004). The remaining countries specify a test temperature of 77°F (25°C).

Criteria for the Cantabro Abrasion test are specified based upon the type of conditioning the samples are subjected. There are three different conditions in which samples are tested: unaged, aged and moisture conditioned. Specification values for the Cantabro Abrasion test conducted on unaged samples are predominantly a maximum percent loss of 25 percent. However, the Texas Department of Transportation specifies a maximum of 20 percent loss as does Belgium (Alvarez et al, 2006). All other agencies specify a maximum of 25 percent loss on unaged specimens. In order to reach the aged condition, samples are placed within a forced draft oven at a given temperature for a specified amount of time. Mallick et al (2000) recommended aging samples at 140°F (60°C) for 7 days prior to testing. After aging, the sample was allowed to cool to the Cantabro Abrasion test temperature of 77°F (25°C). Criteria developed by Mallick et al

(2000) for samples aged in this manner are a maximum of 30 percent loss. The final conditioned state is moisture conditioning. This is practiced in South Africa (Huber, 2000), Italy (Huber, 2000), Britain (Bolzan et al, 2001), and Australia (Alderson, 2006). To moisture condition samples, the specimens are submerged in water for a specified amount of time. The only provided conditions provided in the literature were from Britain where specimens are submerged for 24 hours at 140°F (60°C) water bath (Bolzan et al, 2001). It should be stated that all of the references above utilized test samples that were compacted with a Marshall hammer. Watson et al (2003) developed recommendations for Cantabro Abrasion loss values for samples compacted in a Superpave gyratory compactor. In an unaged condition, abrasion loss should be less than 15 percent.

There are a number of methods for evaluating the draindown potential of OGFC mixes. Decoene (1990) described two methods utilized in Belgium: a basket drainage test and the Schellenberger Drainage Test. During the basket drainage test, OGFC mixes are first compacted in Duriez molds under a pressure of 435 psi (30 bars). The molds containing the compacted OGFC are then placed into an oven maintained at 356°F (180°C). Samples are held at this temperature for 7.5 hours. At the conclusion of this test, the percent asphalt binder lost from the samples is calculated as a percent of the initial binder content (Decoene, 1990).

The Schellenberger Drainage Test begins be placing 1,000 to 1,100 grams of loose OGFC into a glass beaker. The beaker is then placed into an oven maintained at 338°F (170°C) for 1 hour. After the allotted time, the loose OGFC is removed from the beaker and the amount of asphalt binder remaining in the beaker is determined. Draindown potential is described as the binder remaining in the beaker and is expressed as a percentage of the initial asphalt binder content (Decoene, 1990).

Santha (1997) of the Georgia DOT described the Pyrex Pie Plate method for evaluating draindown potential. For this method, mix is prepared and placed into a clear glass Pyrex bowl. The bowl is then placed in an oven set at 250°F (121°C) for 1 hour. A visual examination of the bowl is conducted after the 1 hour to qualify the amount of asphalt binder left on the Pyrex bowl. Santha (1997) also states that the Schellenberg Drainage has been utilized by the Georgia DOT.

In a subsequent paper to Santha's (1997), Watson et al (1998) indicated that the Georgia DOT had adopted the draindown test developed at the National Center for Asphalt Technology. Mallick et al (2000) describe this method as placement of loose mix into a wire basket. The mix and basket are placed into an oven set at the specified temperature. Mallick et al (2000) used test temperatures of 320 and 338°F (160 and 170°C, respectively) though later recommendations from the same authors was to conduct testing 27°F (15°C) higher than anticipated production temperatures (Kandhal, 2002; Watson et al, 2004a). Within the oven and underneath the wire basket, a suitable container of known mass is placed. The mix was then held at the elevated temperature for 1 hour. At the end of 1 hour, the basket was removed from the oven and the mass of the container is determined. Draindown was then calculated based on the mass of binder that drains from the mix through the basket into the container, expressed as a percentage of the total mix mass.

In a later research project, Watson et al (2003) conducted draindown tests of various OGFC mixes using the draindown basket, but with different size wire mesh to fabricate the baskets. The two mesh sizes represented a No. 4 (4.75 mm) screen mesh and a No. 8 (2.36 mm) screen mesh. The standard mesh size was the No.4 screen. The smaller mesh size was investigated because Watson et al (2003) believed that some intermediate sized aggregates could pass through the No. 4 sized screen. Another modification made to standard procedure described above was that asphalt binder remaining on the basket after the 1 hour was considered as part of draindown. Results of comparisons between the standard draindown test and the modified versions showed very strong correlations. However, Watson et al (2003) stated that tests conducted with the No. 8 (2.36 mm) mesh sized basket resulted in more repeatable test results. They did not recommend changes to how draindown was determined.

Some agencies specify air void contents during mix design. As such, this implies that a standard laboratory design compactive effort is needed during mix design. The literature presented two laboratory compaction methods that are prevalent in designing OGFC: the Marshall hammer and Superpave gyratory compactor. Historically, the Marshall hammer has been used to design OGFC mixes. The Marshall hammer has been utilized in Belgium (Decoene, 1990), Georgia (Santha, 1997), United Kingdom (Huber, 2000), Spain (Huber, 2000), Italy (Huber, 2000), South Africa (Huber, 2000), and Switzerland (Alvarez et al, 2006). Not all references reported the compactive effort when using the Marshall hammer; however, all that did report the design compactive effort reported 50 blows per face, except one. Santha (1997) indicated that 25 blows per face were utilized by the Georgia DOT during design (in 1997).

Most of the US agencies that require specific air void contents are currently utilizing a Superpave gyratory compactor. The most common design compactive effort with the Superpave gyratory compactor is 50 gyrations; however, McDaniel and Thornton (2005), utilized 20 gyrations in Indiana. The 50 gyrations was selected during research that compared densities achieved by 50 blows per face of the Marshall hammer and various design gyration levels (Watson et al, 2003). Subsequent work by Watson et al (2004a) conducted a more comprehensive evaluation to determine the appropriate design compactive effort. Within this research, the effect of aggregate breakdown was also evaluated. Watson et al (2004a) concluded that the design compactive effort of 50 gyrations was appropriate.

Though having different operational characteristics than the Superpave gyratory compactor, Alderson (1996) reported that Australia also uses a gyratory compactor to design OGFCs. In Australia, 80 gyrations of the Australia gyratory compactor are used to design OGFCs.

Mallick et al (2000) utilized a laboratory permeability test during mix design. The permeability device was described as a falling-head permeameter that was based on an apparatus developed by the Florida DOT. Mallick et al (2000) stated that the laboratory test was optional during mix design, but indicated that a minimum value of 330 ft/day (100 m/day) should be utilized. Faghriand and Sadd (2002) also utilized permeability testing to evaluate OGFC mixes.

A final test that has been recommended during the design of OGFC mixes is the dry-rodded test to evaluate the existence of stone-on-stone contact. The concept is similar to that used in the design of SMA and is called voids in coarse aggregate (VCA). Kandhal (2002) and Watson et al

(2004b) have recommended the VCA concept in designing PFCs. The method entails first measuring the VCA of the coarse aggregate only using AASHTO T19, Unit Weight and Voids in Aggregates. There is, however, a difference between the two references on the definition of coarse aggregate. Kandhal (2002) defines the coarse aggregates as those aggregates coarser than the No. 4 (4.75 mm) sieve while Watson et al (2004b) utilize the break point sieve as differing between fine and coarse aggregate. Watson et al (2004b) defined the break point sieve as the finest sieve to retain 10 percent or more of the aggregate blend. The next step in evaluating stone-on-stone contact is to calculate the VCA of compacted samples. If the VCA of the compacted OGFC is less than the VCA of the dry-rodded aggregates, then stone-on-stone contact is achieved (Kandhal, 2002; Watson et al, 2004b). Watson et al (2004b) further verified the existence of stone-on-stone contact using x-ray Computed Tomography.

2.1.4 Performance Testing

The predominant type of performance testing conducted during OGFC designs is moisture sensitivity testing. As mentioned under the Cantabro Abrasion Loss discussion above, moisture conditioning of OGFC samples prior to testing has been utilized (Huber, 2000; Alderson, 1996; Bolzan et al, 2001). To moisture condition samples prior to Cantabro testing, samples are submerged in a heated water bath for a specified amount of time.

The most predominant method found in the literature for conducting moisture susceptibility testing on OGFCs is to use indirect tensile strength testing and tensile strength ratios (TSR). The conditioning of samples prior to determining TSRs varies within practice. Some have recommended the use of five freeze-thaw cycles prior to testing (Kandhal, 2002; Spillemaeker and Bauer, 2000), while some use one freeze-thaw cycle. In 2004, Watson et al (2004a) compared TSR results after 1, 3 and 5 freeze-thaw tests. Results from comparisons showed no significant difference in TSRs after 1, 3 and 5 freeze-thaw cycles.

The next most common moisture susceptibility test was the boil test. Santha (1997) utilized this test method. This test method essentially entails placing loose mix into boiling water for a specified time. After boiling, a qualitative evaluation of the amount of binder that has stripped from the aggregates is made.

The final test identified for evaluating moisture susceptibility was a loaded-wheel tester. Cooley et al (2004) loaded samples submerged under water to evaluate moisture susceptibility. The loaded-wheel tester used was an Asphalt Pavement Analyzer.

A number of other tests were identified in the literature to evaluate designed OGFC mixes. In the Netherlands, a dynamic bending test was used to evaluate the stiffness of OGFC mixes (Van Der Zwan et al, 1990). No specifics were provided on the test, but it is assumed to be similar to the four-point bending beam fatigue test. Additionally, the Netherlands have used a wheel-tracking device to evaluate the rutting performance of OGFC mixes (Van Der Zwan et al, 1990). Similarly, Mallick et al (2000) used the Asphalt Pavement Analyzer to evaluate the stability of PFC mixes. Spillemaeker and Bauer (2000) discussed a rotary shearing press to evaluate rutting potential. No specifics were provided for this test other than providing the French Standard (NF P 98-252). Another method of evaluating the potential for rutting potential was described by

Fortes and Merighi (2004). These authors described results from a static, unconfined creep test. Again, specific test conditions were not given.

The final two performance tests identified in the literature were reported by Molenaar and Molenaar (2000). Both of these tests were designed to evaluate the potential for short-term raveling. The first test was called the Wheel Fretting Test (WFT). For the WFT, a treaded tire inflated to 87 psi (600 kPa) and loaded to 675 lb (3kN) was run in a circular path on top of PFC test specimens. The loaded tire had an inclination angle of 2 to 5 degrees. A total of 3 million revolutions were applied to the test samples at a test temperature of approximately 68°F (20°C). The fretting performance was characterized as a mass loss after the wheel passes.

The second short-term raveling test described by Molenaar and Molenaar (2000) was called the California Abrasion Test. The California Abrasion Test utilizes a mechanical shaker that is operated at 20 cycles per second with a specified vibration amplitude. A sample of PFC was placed into a container along with water and steel spheres and subjected to the vibration action for 15 minutes at a test temperature of 39°F (4°C). Again, test results are reported as a percent mass loss after the 15 minutes of abrasive action.

2.2 Performance of Open-Graded Friction Courses

Throughout the history of using open-graded mixes as wearing layers, there have been two predominant performance related problems: raveling and delamination. These two problems led to moratoriums on the use of OGFCs by many state highway agencies during the 1980's (Cooper et al, 2004; Huber, 2000). These problems have been noted not only in the US, but also in Europe (Van Der Zwan et al, 1990; Lefebvre, 1993). This section describes the performance of OGFCs from around the world. The first part of this section describes the distresses that are encountered with OGFC layers. The second part describes the performance life of OGFCs.

2.2.1 Typical Distresses with Permeable Friction Courses

The Long Term Pavement Performance Program (LTPP) has identified a number of distresses related to HMA layers (LTPP, 1993). Within this document, distresses are categorized according to the following general distress types: cracking, patching/potholes, surface deformation, surface defects and miscellaneous distresses. Table 2.1 lists the distresses defined by LTPP within each of these categories.

Table 2.1 - LTPP Defined Distresses for HMA Pavements (LTPP, 1993)

Cracking	Patching and Potholes	Surface Deformation	Surface Defects	Miscellaneous
Fatigue	Patch	Rutting	Bleeding	Lane-to-shoulder
Block	Deterioration	Shoving	Polished	drop-off
Edge	Potholes		Agg.	Water Bleeding
Longitudinal			Raveling	and Pumping
Reflection				
Transverse				

Of the distresses listed in Table 2.1, only raveling has been reported as common to PFCs. Huber (2000) states that OGFCs typically fail by raveling. Molenaar and Molenaar (2000) have described two forms of raveling: short-term and long-term raveling. Short-term raveling is caused by intense shearing forces at the tire/pavement interface that occurs within newly placed PFCs. Pucher et al (2004) state that short-term raveling generally occurs quickly once the flow of traffic begins. Conditions that enhance the potential for short-term raveling include placing the PFC at too low of a temperature, incomplete seating of the aggregates during compaction and draindown (areas lean in asphalt binder). Long-term raveling was described by Molenaar and Molenaar (2000) as being caused by long-term segregation of the asphalt binder from the aggregates due to gravity. As the asphalt binder drains from the coarse aggregate structure due to gravity, the aggregates near the surface of the layer become under asphalted. The action of traffic can dislodge the aggregates, resulting in raveling. It should be stated that the long-term draindown of the asphalt binder due to gravity was mostly encountered in OGFC mixes that did not include modified asphalt binders.

Pucher et al (2004) states that up to 5 to 10 years, OGFCs deteriorate slowly. After this time, the rate of deterioration increases. Raveling is the distress most commonly observed due to this increase in deterioration.

As stated above, delamination is the other distress most commonly associated with OGFCs. Delamination of OGFC layers could, however, be construed as potholes once the layer has been removed by traffic.

The raveling and delamination problems that have plagued OGFC mixes in the past can likely be traced back to mix design, specifically materials selection, and construction problems. Open-graded friction courses have an open gradation with a relatively low percentage of material passing the No. 200 (0.075mm) sieve. Because of the open grading, there is very little aggregate surface area which results in a relatively thick film of asphalt binder coating the aggregates. At typical HMA production/construction temperatures, the heavy film of asphalt binder had a propensity to drain from the aggregate skeleton (Huber, 2000). Because of the draindown issues, a typical remedy was to reduce either the asphalt binder content or the mixing and compaction temperatures during production/construction (Kandhal, 2002). Reduced asphalt binder contents meant that the OGFC mixes were under-asphalted which would increase the potential for raveling. The reduction in temperature increased the viscosity of the asphalt binder which assisted in preventing the asphalt binder from draining from the aggregate skeleton.

When production temperatures of the OGFC are reduced, all of the internal moisture within the aggregates is not removed. Moisture remaining within the aggregates after plant mixing increases the potential for the asphalt binder stripping from the aggregates leading to the aggregates being raveled out due to traffic (Huber, 2000). Reduced mixing temperatures also resulted in reduced compaction temperatures. Mixture delivered to the roadway that was not at an appropriate compaction temperature had difficulty bonding to the tack coat placed on the existing roadway surface. This resulted in an inadequate bond between the OGFC and underlying layer. The lack of an adequate bond increased the potential for delamination problems (Kandhal, 2002).

Evidence that raveling and delamination problems of the past were related to mix design and production/construction practices was provided by Kandhal and Mallick (1998). Based upon a 1998 survey, they stated that highway agencies that had experienced good performance with OGFCs were utilizing polymer-modified asphalt binders and relatively high asphalt binder contents (by using fibers and/or relatively open gradations). The combination of modified asphalt binders and fibers helped hold the asphalt binder on the OGFC's aggregate skeleton, minimizing the potential for draindown. Without the potential for draindown (and, with relatively high asphalt binder contents), there was no need to lower mixing and compaction temperatures which minimized the potential for both raveling and delamination.

Though raveling and delamination are the two most common distresses listed in the literature, other distresses have been mentioned. Rogge (2002) conducted a survey of Maintenance supervisors from the Oregon Department of Transportation (ODOT). One question within the survey concerned typical distresses encountered on PFC pavements. Figure 2.10 illustrates the typical distresses encountered in Oregon on OGFC pavements as reported by 78 respondents to the survey. Within Figure 2.10, the maintenance engineers were requested to rank the various distresses by their frequency using a ranking system of 1 to 4. The higher the ranking, the more frequent the distress encountered. Based upon the survey, tire stud rutting was considered the most common distress. Raveling was the second highest rated distress (icing problems is considered a winter maintenance issue). Other distresses that rank closer to scattered than rare included gouging/scarring, deformation rutting, and potholes (clogging is considered a general maintenance issue).

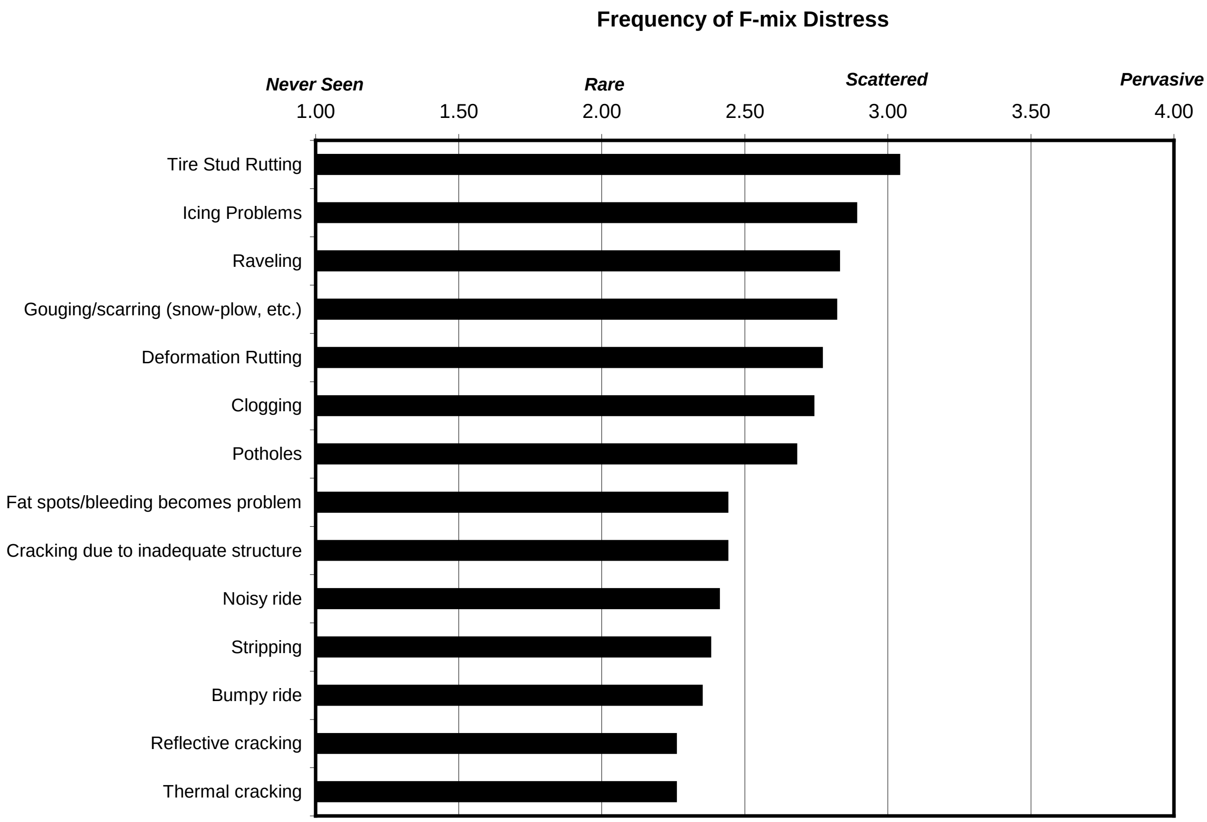


Figure 2.10 - Results of 2001 Survey of ODOT Maintenance Supervisors (Rogge, 2002)

Because of the environment in Oregon, the existence of tire-stud rutting is not unexpected. However, tire-stud rutting should not be considered the same as the traditional rutting seen on typical dense-graded HMA pavements (plastic deformation). Tire-stud rutting as described by Rogge (2002) is likely raveling within the wheel paths. Studded tires can dislodge aggregate particles in the wheel path giving the appearance of classical rutting.

Rogge's report (2002) was the only reference found in the literature that listed rutting as a distress on PFC pavements. Several papers/reports from Europe list resistance to permanent deformation as a benefit of OGFC pavement layers (Greibe, 2002; Lefebvre, 1993; Pucher et al, 2004). Open-graded friction courses should generally not be associated with plastic deformation rutting. Similar to SMAs, OGFCs have a very coarse gradation that results in stone-on-stone contact (Watson, 2004b). Because of the stone-on-stone contact, PFCs should not rut due to plastic deformation unless there are mix design or construction problems.

The only other distress found in the literature was not specifically related to OGFC mixtures; rather, it was the occurrence of stripping in layers underlying the OGFC surface. Huber (Huber, 2000) states that open-graded mixes can change the moisture balance within a pavement structure. Open-graded friction courses can create a moist "microenvironment" at the surface of the underlying layer. When this exists, the increased humidity created by the moist microenvironment can retard evaporation of water from the underlying layer. This, in essence, traps water within the underlying layer. When OGFCs become clogged, the underlying layer may even become wetter. Therefore, if the HMA mixture underlying the OGFC layer contains materials susceptible to moisture, then stripping of the underlying layers may occur.

Though no specific instances were reported, Roque et al (2009) state the OGFCs may play a key role in top-down cracking performance of pavements. They suggest that there is limited evidence that well designed and constructed OGFC layers may minimize the potential for top-down cracking.

2.2.2 Performance of Open Graded Friction Courses

According to Huber (2000), the performance of OGFC pavements in general can be put into one of two categories: performance life and service life. The category of performance life is used to describe the length of time an OGFC pavement maintains its beneficial characteristics. These characteristics would generally include permeability (reduction in potential for hydroplaning and splash/spray and improvement in pavement marking visibility) and the ability to reduce tire/pavement noise. Service life describes the length of time than an OGFC pavement maintains its frictional properties and smoothness. Structural failure of the OGFC would also be included in service life.

Service Life

Of the two categories of performance, the service life will generally be longer. Service life generally relates to the time that an OGFC layer needs to be rehabilitated. The vast majority of reports/papers suggest that OGFC pavement layers will have an average service life of about 10 years, though longer periods have been cited. A number of European countries, including the Netherlands (Van Der Zwan et al, 1990), Switzerland (Lefebvre, 1993), and Spain (Ruiz et al, 1990), indicate that the service life of OGFC pavements is approximately 10 years. Similarly, Australia has also indicated 8 to 10 years of service life (Alderson, 1996). In the United States, a survey of state highway agencies on OGFCs conducted in 1998 by Kandhal and Mallick (1998) showed that 73 percent of the state agencies obtained an estimated average service life of greater than 8 years (Figure 2.11). Forty-three percent of the state agencies estimated an average service life of greater than 10 years.

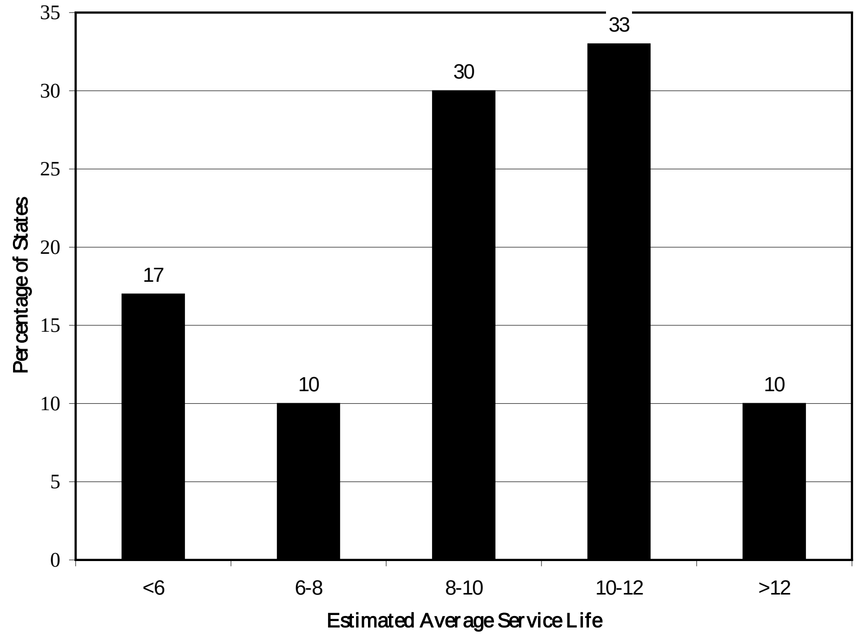


Figure 2.11 - Reported Estimated Average Service Lives for OGFC Layers (Kandhal and Mallick, 1998)

No specific literature was found that presented a research approach that followed the frictional properties or smoothness of an OGFC layer until the end of the service life. Survey results depicted in Figure 2.11 likely reflect more of an issue with smoothness than friction. Smoothness would be affected by raveling problems associated with OGFC pavements and raveling was cited by the vast majority of papers/reports reviewed as the primary performance problem with OGFC layers. Additionally, delamination, which has also been labeled as a major problem with OGFC layers (Kandhal and Mallick, 1998), would also negatively affect smoothness.

Figure 2.10 showed distresses observed in Oregon on OGFC pavement layers as well as the frequency in which those distresses are encountered (Rogge, 2002). Based on this figure, most of the distresses that had a frequency closer to scattered than rare would affect smoothness. As tire-stud rutting, raveling, gouging/scarring, deformation rutting and potholes increase, smoothness would decrease.

As stated previously, Pucher et al (2004) indicated that up to 5 to 10 years, OGFCs deteriorate slowly. After this time, the rate of deterioration increases. Raveling is the distress most commonly observed due to this increase in deterioration.

Similar to smoothness, no specific literature was found that followed the frictional properties of an OGFC pavement layer from construction till the end of the service life. The literature does suggest that the frictional characteristics of OGFC layers are relatively low (but acceptable) immediately after construction (Lefebvre, 1993; Santha, 1997; Padmos, 2002). Open-graded friction courses are intentionally designed to include a relatively high asphalt binder content. After production and placement, aggregates within the OGFC layer will be coated with a thick film of asphalt binder. This thick film of asphalt binder prevents a vehicle tire from adhering to the aggregates (microtexture) at the surface of the layer (Lefebvre, 1993). Griebe (2002) stated that when the wheels lock during a braking action, the friction created between the tire and pavement surface begins to melt the asphalt binder coating the aggregates which hinders friction. This is only true when wheels are locked. When an Anti-lock Braking System is used, the braking distance on OGFC is similar to that of dense-graded HMA. Some literature indicates that it can take 3 to 6 months for the asphalt binder film to wear from the aggregates at the surface of the layer (Heystraeten and Moraux, 1990; Padmas, 2002). However, a research study in Georgia indicated that the asphalt binder layer wore off within 2 weeks (Santha, 1997). Table 2.2 illustrates the results of skid trailer friction testing conducted on six OGFC test sections over a 3.5 year time period just south of Atlanta, Georgia (Santha, 1997). Within this table, the first friction tests were conducted the day after construction. These measurements were all relatively low compared to the subsequent test dates. The data clearly shows that once the asphalt binder film has worn from the aggregates, friction will increase.

Table 2.2 - Average Friction Test Results for Six PFC Test Sections (Santha, 1997)

Test Section Designation	Friction Number (ASTM E274)			
	10/27/92	11/11/92	4/12/93	2/6/96
Std. OGFC (D)	42	53	52	50
Coarse OGFC (D)	41	50	52	51
D + Mineral Fibers (DM)	39	50	53	49
D + Cellulose Fibers (DC)	37	47	53	49
DC + SB Polymer (DCP)	35	46	52	50
D + SB Polymer (DP)	32	47	51	51
D + 16% Crum Rubber (D16R)	37	48	53	51

There are two primary reasons for the good frictional properties of an OGFC layer: permeability and macrotexture. Because of the high percentage of air voids associated with OGFC layers, water will readily drain from the pavement surface into the interstitial voids layer. Water that drains into the layer is not available to be trapped between the vehicle tire and pavement surface in the form of water films, thus improving wet weather friction (Ruiz et al 1990; Van Der Zwan et al 1990; Lefebvre, 1993). Because of the open grading, these mix types result in a relatively high amount of macrotexture (McDaniel and Thornton, 2005; Ruiz et al, 1990; Flintsch, 2003). Table 2.3 presents macrotexture measurements from a research study conducted in Indiana that compares the texture of an OGFC test section to other types of hot mix asphalt. Results shown in Table 2.3, expressed as mean profile depth (MPD), were obtained using a Circular Texture Meter and show that porous friction courses (an OGFC designed to have 18 percent or more air void contents) have significantly more surface texture than dense-graded HMA layers and markedly more surface texture than SMA layers. McDaniel and Thornton (2005) also used results of friction testing with the Dynamic Friction Tester to determine the International Friction Index (IFI) for the three mix types shown in Table 2.3. The IFI utilizes the results of friction measurements along with MPD data to provide a harmonized frictional characteristic measure that is independent of the equipment used. Results, shown in Table 2.4, indicate that the OGFC had the highest IFI followed by the SMA and dense-graded HMA, respectively. These results show the significant influence of surface texture on the International Friction Index.

Table 2.3 - Results of Surface Texture Measurements using Circular Texture Meter (McDaniel and Thornton, 2005)

Mix Type	Mean Profile Depth, mm (Standard Deviation)
Porous Friction Course	1.37 (0.13)
Stone Matrix Asphalt	1.17 (0.14)
Dense-Graded HMA	0.30 (0.05)

Table 2.4 - International Friction Index Data (McDaniel and Thornton, 2005)

Mix	Average Dynamic Friction Tester (DFT) Number (Standard Deviation)			International Friction Index (F_{60})
	20 kph	40 kph	60 kph	
PFC	0.51 (0.03)	0.45 (0.03)	0.42 (0.03)	0.36
SMA	0.37 (0.01)	0.31 (0.01)	0.29 (0.01)	0.28
HMA	0.52 (0.01)	0.47 (0.01)	0.44 (0.01)	0.19

Because of the significant amount of macrotexture produced within OGFC pavement surfaces, these layers will maintain adequate frictional characteristics even after becoming clogged (Isenring et al 1990). The macrotexture will allow water films to be dissipated under tires during rain events.

Performance Life

Similar to smoothness and friction, no specific references were identified that followed the permeability and noise reducing characteristics of OGFC layers over time. Generally, the performance life will be shorter than the service life. This will especially be true in areas that do not employ a general maintenance program for cleaning clogged OGFC layers. Isenring et al (1990) listed a number of causes for reduction in permeability within OGFC layers. First, dust and debris can fill the void structure causing the layer to become clogged. Secondly, slight densification of the layer under traffic will reduce permeability from initial values. Other factors that can lead to reduced permeability include environment (amount of rain) and type of traffic volume. Isenring et al (1990) state that permeability will generally be maintained within wheelpaths. Wheelpaths will maintain permeability longer because of the cleaning pressure/suction action caused by tires traveling over the layer. Van Heystracten and Moraux (1990) also reported that clogging potential is reduced with intense traffic. Isenring et al (1990) state that some OGFC layers will maintain permeability for more than 5 years without maintenance while some will become almost impermeable within one year. In order to maintain permeability through general maintenance, maintenance should take place while the layer is still permeable (Isenring et al, 1990).

Isenring et al (1990) listed a number of favorable conditions for maintaining permeability including: areas with reduced amounts of dirt and debris; good drainage (daylighted edge and sufficient cross slope in underlying layer); high air void contents within the OGFC; and the cleaning action of rapid and intense traffic. Additionally, they stated that larger maximum aggregate size gradations maintained permeability longer than smaller maximum aggregate size gradations. Ruiz et al (1990) reported less clogging in OGFC mixes having more than 20 percent air voids. British Columbia indicated that no clogging or reduction of permeability had been observed (Bishop and Oliver, 2001).

Isenring et al (1990) conducted a number of noise measurements to compare OGFC and dense-grade surfaces. They evaluated sound absorption, tire/pavement noise (using a trailer) and wayside measurements. For sound absorption, their research showed that OGFC layers that are in good functional condition (permeability has been maintained) are capable of absorbing sound. Layers thicker than 2 in. (50 mm) had the potential for absorbing more sound. Isenring et al (1990) also showed a relationship between permeability and sound absorption. As permeability increased, sound absorption also increased. However, the surface texture (macrotexture) seemed to be more important than permeability. Several pavements exhibiting low permeability values (clogged) still had the ability to absorb sound.

When measuring the tire/pavement noise, Isenring et al (1990) found that OGFCs that were in good functional condition had lower noise levels than typical dense-graded layers. At speeds above 30 to 35 mph (50 to 60 km/hr) the difference between the two wearing layers became larger. Testing with the noise trailer also resulted in a relationship between permeability and noise levels. As permeability increased, noise levels generally decreased. Also, OGFCs having a smaller maximum aggregate size generally resulted in lower noise levels than coarser gradations. McDaniel and Thornton (2005) also used a noise trailer to show a 4 to 5 dB(A) reduction in noise levels when comparing OGFC to SMA and dense-graded layers.

Isenring et al (1990) also reported on wayside measurements. The evaluations were conducted where a comparison in noise levels between OGFC and dense-graded layers could be made. For single vehicle cars, a level in noise reduction of between 1 and 5 dB(A) was observed when testing OGFCs. Noise levels for a traffic stream showed reductions between 0 and 3.5 dB(A). Brousseaud et al (2005) used wayside measurements to show a 3 to 5 dB(A) reduction in noise levels when using the Statistical Pass-By Method.

The Danish government has an initiative to reduce the number of dwellings exposed to a noise level of 65 dB(A) by two-thirds (Larsen and Bendtsen, 2002). A two-layer permeable friction course (PFC) system was identified as potentially the most effective means of achieving this goal. According to Dutch experience two-layer PFC systems have good noise-reducing characteristics compared to dense-graded layers. The reason for this is the structure of the system contains a large number of interconnected voids. Tires rolling on the surface result in air pumping as the tire pushes air into the layer and then the air is sucked out as the tire passes. This pumping action generates a high frequency noise. On PFCs, the pumping is reduced because the air is pumped into the interconnected voids of the layer.

Similar to the work of Isenring et al (1990), the Dutch state that OGFC layers also reduce noise levels by absorbing some of the noise emitted by vehicles (Larsen and Bendtsen, 2002). On dense-graded layers, noise emitted towards the pavement is reflected to the surroundings; however, on OGFC some of this noise is absorbed by the pavement through the interconnected void structure.

Huber (2000) cited a number of references that indicated OGFCs maintain their sound attenuation for five years or more as long as their design air voids are above 18 percent.

2.3 OGFC Specific to the Southeast States

This section describes the various mix design methods for OGFC used by DOTs from the Southeast. Evaluations of mix design methods of OGFC mixes were limited to the Southeast because these states should have somewhat similar environmental conditions. Mix design methods from Georgia, Alabama, Mississippi, Florida, South Carolina, Tennessee and Texas were obtained. Similar to the literature review, this section is structured to provide information on each of the four steps in the design of OGFC: materials selection, selection of design aggregate structure, selection of optimum asphalt content, and performance testing.

2.3.1 Materials Selection

As stated previously, materials needing selection include coarse aggregates, fine aggregates, mineral fillers, asphalt binders and stabilizing additives. Following are the current requirements utilized by the Southeastern DOTs.

Aggregate Characteristics

For coarse aggregates, each of the seven DOTs surveyed have a requirement for Los Angeles Abrasion and Impact loss. Maximum loss values range from a low of 30 percent to a high of 52 percent. A caveat to the higher maximum loss value of 52 is that any coarse aggregate having a Los Angeles Abrasion loss of greater than 42 must have a Micro-Duval loss less than 15 percent.

Six of the seven Southeastern states have requirements for coarse aggregate particle shape using flat and elongated requirements. Ratios of five to one and three to one are used. Alabama was the only DOT that has requirements using both ratios with a maximum percent flat and elongated of 10 percent at 5:1 and 20 percent at 3:1. Mississippi's sole requirement was a maximum of 20 percent flat and elongated particles at a 3:1. All of the other states utilize a 5:1 with a maximum requirement of 10 percent except for Tennessee which has a maximum percentage of 20 percent. South Carolina was the lone DOT without a flat and elongated requirement.

Five of the seven DOTs surveyed have requirements for coarse aggregate particle angularity using the percentage of fracture faces as the requirement. The two states not including this criterion, Florida and Georgia, require quarried stone and, therefore, ensure good particle angularity. Four of the five states only have requirements for the minimum percentage of coarse aggregates with two or more fractured faces. These requirements range from a low of 80 percent to a high of 95 percent. Tennessee is the only state that has requirements for both one face and two or more faces (100 and 90 percent, respectively).

All seven of the DOT's have requirements for aggregate durability using the sulfate soundness test. Four of the seven require the sodium sulfate soundness method with maximum requirements ranging from a low of 9 percent to a high of 15 percent. The remaining three DOTs specify magnesium sulfate soundness with maximum percent loss requirements of 15 or 20 percent.

Besides the aggregate requirements described above that were listed by the majority of the states, there were two other requirements specific to only one or two DOTs. Two DOTs specify that any fine aggregates within OGFC mixes must be non-plastic. The Tennessee DOT requires that the water absorption of the combined aggregate blend must be less than 3.0 percent.

Asphalt Binders

Six of the seven states surveyed specified the type(s) of asphalt binder to be used within OGFC mixes (the seventh DOT states that the asphalt binder must be polymer modified but no specific grade is provided). Four of these six specify that a polymer modified asphalt binder meeting the requirements of a PG 76-22 be used within OGFC. Both Florida and Texas allow PG 76-22 asphalt binders, but also allow rubber-modified asphalt binders. The Florida DOT specifies an ARB-12 (now specified as "PG76-22 (ARB)" as of July 2013), while the Texas DOT specifies 15 percent of a crumb rubber modifier.

Stabilizing Additives

As stated above, six of the seven states have specifications on specific types of asphalt binder modifiers which help stabilize the asphalt binder within an OGFC mixture. Additionally, six of the seven states have specifications on the use of fibers. Three types of stabilizing fibers were encountered; cellulose, mineral and recycled polyester fibers. The general percentages of each fiber type range from 0.2 to 0.4 percent by total mix mass.

Other Materials

The only other materials included within the specifications are anti-stripping agents. Some DOTs specify that 1.0 percent hydrated lime be added to the aggregate blend. The Florida DOT requires the use of liquid anti-stripping agents when the aggregate type is oolitic limestone. Other DOTs allow liquid anti-stripping agents, if needed.

2.3.2 Selection of Design Aggregate Structure

Table 2.5 presents the gradation requirements specified by the seven Southeastern DOTs. This table shows that Georgia, Texas and Mississippi have more than one gradation band for OGFCs. Generally, all of the gradations have a maximum aggregate size of 12.5 mm; however, Georgia and Mississippi also have a gradation band for a 9.5 mm maximum aggregate size OGFC. An interesting observation is that most of the gradation bands are very similar below the No. 4 sieve except for the 9.5 mm maximum aggregate size OGFC from Georgia and the asphalt-rubber gradation from Texas. Additionally, the difference in gradation bands between OGFC mixes using a PG-76 asphalt binder and asphalt-rubber from Texas is interesting. The difference in the gradations is because of differences in the minimum asphalt binder contents between the two OGFC mixes. The asphalt-rubber mixes have a minimum asphalt binder content 2.5 percent

Table 2.5 - Gradation Bands Encountered within Southeast

Sieve	FDOT FC-5	Georgia DOT 9.5 mm	Georgia DOT 12.5 mm	Georgia DOT PEM	South Carolina DOT	Texas DOT PG 76	Texas DOT A-R	Alabama DOT	Mississippi DOT 9.5 mm	Mississippi DOT 12.5 mm	Tennessee DOT
3/4 in.	100	100	100	100	100	100	100	100	100	100	100
1/2 in.	85-100	100	85-100	80-100	85-100	80-100	95-100	85-100	100	100	85-100
3/8 in.	55-75	85-100	55-75	35-60	55-75	35-60	50-80	55-75	90-100	80-89	55-75
No. 4	15-25	20-40	15-25	10-25	15-25	1-20	0-8	15-25	15-30	15-30	10-25
No. 8	5-10	5-10	5-10	5-10	5-10	1-10	0-4	5-10	10-20	10-20	5-10
No. 200	2-4	2-4	2-4	1-4	0-4	1-4	0-4	1-4	2-5	2-5	2-4

above the PG-76 mixes. The "coarseness" of the asphalt-rubber gradation band below the No. 4 sieve provides more voids for the extra asphalt binder.

2.3.3 Selection of Optimum Asphalt Binder Content by Southeast States

Based upon the survey of the seven Southeastern states, each DOT has a slightly different method of selecting optimum asphalt binder content for OGFC mixes. Because of the differences, the method for selecting optimum asphalt binder content is discussed individually.

<u>Alabama</u>

The Alabama DOT bases the selection of optimum asphalt binder content on the absorption characteristics of the aggregates. Aggregates passing the 3/8 in. sieve and retained on the No. 4 sieve are immersed in S.A.E. No. 10 lubricating oil. Using the amount of oil absorbed by the aggregates after immersion, a surface content is determined which is used to calculate a target asphalt binder content.

Following determination of the target asphalt binder content, laboratory compaction using either a vibratory compactor or vibrating table, is used to determine the void capacity of the coarse aggregate fraction of the aggregate blend. Next, the optimum content of fine aggregate is calculated and compared to the void capacity of the coarse aggregate. If the optimum fine aggregate content meets the selected design gradation on the No. 8 sieve, then the calculated asphalt binder content from the surface constant is considered optimum asphalt content.

The final step of the method entails evaluating the draindown characteristics of the mixture. Draindown is conducted using a draindown basket and results must be less than 0.3 percent by total mix mass. Draindown testing is conducted at a temperature 27°F above the anticipated plant production temperature.

Alabama provides a range in which the optimum asphalt binder content most fall. Optimum asphalt binder content must be above 4.7 percent and below 9.0 percent.

<u>Florida</u>

The basis of the Florida DOT method for selecting optimum asphalt binder content is draindown. Loose OGFC mixture is placed into a clean, 9 in., flat-bottomed heat resistant pie-plate. The mix and pie-plate are then placed into an oven set at 320 $\pm$ 5° F for 1 hour. After 1 hour, the pie-plate is allowed to cool to room temperature. Once at room temperature, the pie-plate is inverted and inspected. Inspection involves evaluating whether sufficient bonding occurs between the OGFC mix and pie-plate without excessive amounts of asphalt binder draining from the aggregates. Generally, this method will be conducted at three different asphalt contents and an optimum asphalt binder content selected based upon the visual inspection of the pie-plates.

Florida provides ranges of allowable optimum asphalt binder content depending upon the aggregate type used within the OGFC. When crushed granites are utilized, the optimum asphalt binder content must be between 5.5 and 7.0 percent. For oolitic limestone, optimum asphalt binder content must be between 6.5 and 8.0 percent.

Georgia

Georgia uses three tests during the selection of optimum asphalt binder content. First, coarse aggregates are submerged in S.A.E. No. 10 oil and the required asphalt binder content calculated based upon the surface capacity. Secondly, OGFC mixture is compacted utilizing 25 blows per face of a Marshall hammer. Samples are compacted at 0.5 percent asphalt binder content intervals. The bulk specific gravity of each compacted specimen based upon the calculated volume from dimensional analyses. A graph of volume in mineral aggregate (VMA) versus asphalt binder content is then developed. An asphalt binder content corresponding to the minimum VMA is determined. The final test involves evaluating the binder/draindown potential of loose OGFC mix using the pie-plate method described above. Optimum asphalt binder content is selected as the average from the three tests.

Table 2.5 showed that Georgia has three different gradation bands. Each gradation band has a range that optimum asphalt binder content must fall. For the 9.5 mm OGFC, optimum asphalt binder content must be between 6.0 and 7.25 percent. The 12.5 mm OGFC mixes must be between 5.75 and 7.25 percent, while optimum asphalt binder content must be between 5.5 and 7.0 percent for the 12.5 mm PEM mixes.

Mississippi

Selection of optimum asphalt binder content in Mississippi requires an OGFC mix to meet four different criteria. First, the OGFC mixture must have at least 15 percent air voids after laboratory compaction utilizing 50 gyrations of the Superpave gyratory compactor (SGC). Next, the OGFC mix must have a minimum laboratory permeability of 30 m/day. Thirdly, draindown, using a draindown basket, must be less than 0.3 percent by total mix mass. The draindown test temperature is 27°F above anticipated plant mixing temperature. The final criteria is for Cantabro Abrasion Loss. Cantabro Abrasion Loss is determined on OGFC specimens compacted using 50 gyrations of the SGC. A compacted sample is placed in a Los Angeles Abrasion drum without the charge of steel spheres. The drum is then rotated for 300 revolutions at a rate of 30 revolutions per minute. Using the sample mass before and after the test, a percent loss is calculated. Mississippi has requirements for both aged and unaged specimens. Unaged specimens cannot exceed 30 percent loss. For aging, compacted specimens are placed in a forced draft oven set at 147°F for seven days. The maximum Cantabro Abrasion Loss for aged samples is 40 percent. Optimum asphalt binder content is selected as an asphalt binder content that meets all four of the requirements.

Mississippi has a table that provides a minimum optimum asphalt binder content based upon the bulk specific gravity of the aggregate blend. The minimum asphalt binder content applies to both Mississippi gradation bands (Table 2.5).

South Carolina

South Carolina also uses the pie-plate method for determining optimum asphalt binder content. However, the OGFC mix is held for 2 hours at mixing temperature during testing. In addition, to the pie-plate method, South Carolina also verifies minimal draindown potential at the selected optimum asphalt binder content using the draindown basket. For this testing, the sample is held at 350±5°F for 1 hour during testing.

South Carolina has a range in which optimum asphalt binder content must fall. Optimum asphalt binder content must be between 5.5 and 7.0 percent.

Tennessee

Tennessee's Special Provision on OGFC states that OGFCs are designed in accordance with NAPA's Information Series 115 (Kandhal, 2002). Similar to Mississippi, optimum asphalt binder content is selected as an asphalt content that meets several requirements. A minimum of 20 percent air voids is required in OGFC compacted using 50 blows per face of the Marshall hammer. Tennessee requires that the voids in coarse aggregate (VCA) of a compacted mix (VCA_{MIX}) must be less than the VCA of the dry-rodded coarse aggregate (VCA_{DRC}). The VCA_{MIX} is calculated on specimens compacted using 50 blows per face of the Marshall hammer. A maximum draindown of 0.3 percent is specified when utilizing the draindown basket method and test temperature 27°F above anticipated plant mixing temperature. Unaged Marshall hammer compacted specimens are tested for percent loss using the Cantabro Abrasion Loss test with a maximum loss of 20 percent. Again, optimum asphalt binder content is selected as an asphalt binder content that meets all of the above criteria.

Tennessee is the only DOT that does not provide an optimum asphalt binder content range. Rather, optimum asphalt binder content must be higher than 6.0 percent.

Texas

Texas selects optimum asphalt binder content based upon two criteria. First, OGFC mixes compacted using 50 gyrations of a SGC must have between 18 and 22 percent air voids. Secondly, draindown, when tested using the draindown basket method, must be less than 0.2 percent by total mix mass. A test temperature of 350±5°F is used for the draindown testing.

The allowable range in optimum asphalt binder content in Texas is based upon the type of asphalt binder used in the OGFC. Mixes utilizing a PG-76 binder must have an optimum asphalt binder content between 5.5 and 7.0 percent. Mixes utilizing an asphalt-rubber binder must have an optimum asphalt binder content between 8.0 and 10.0 percent.

2.3.4 Performance Testing by Southeast States

Performance testing conducted by the different states is limited to moisture susceptibility testing. Two types of moisture susceptibility tests were encountered: tensile strength ratios (TSRs) and a boil test. For those states requiring TSR testing, minimum TSR results ranged from 80 to 85 percent. For those agencies requiring the boil test, percent retention requirements range from 95 to 100 percent.

CHAPTER 3 – REVIEW AND ANALYSIS OF FLORIDA DOT FC-5 FIELD SECTIONS

An FC-5 pavement evaluation was conducted to identify good and poor performing FC-5 pavement sections. The Florida Department of Transportation's (FDOT) Pavement Management System (PMS) was queried to select only FC-5 pavement surfaces. The resultant Excel spreadsheet was provided to the Research Team to evaluate general performance of the FC-5 pavement surfaces with respect to cracking and raveling. As per RFRP #10/11-003;

> "FC-5 is the open graded friction course (OGFC) used on all FDOT's high speed multi-lane facilities. This mixture type is advantageous compared to dense graded friction courses in that is reduces road spray and hydroplaning potential. The drawback is that its life span is less than dense graded friction courses. Primary distresses are raveling and top-down cracking."

3.1 Florida DOT FC-5 Wearing Course Cracking – PMS Query Results

To help possibly understand any similarity between FC-5 mixture design properties and performance, the FC-5 PMS data was queried in a manner to measure performance life. The PMS database used dated back to 1999, which is around the inception of FC-5 mixtures in Florida. Along with the construction date, two other critical factors were queried for comparison; 1) cracking and 2) raveling. Cracking was established using FDOT's PMS crack rating parameters (10.0 = Best Condition; 0.0 = Worst Possible Condition). It should be noted that FDOT utilizes a rating value of 6.4 to determine when the distress condition has caused the pavement to fall into a "Deficient" classification (FDOT Work Plan Instructions, 2009). Once in this condition, the pavement section qualifies for rehabilitation. Raveling severity information was also identified for the different FC-5 pavement sections. Raveling is indicated in the PMS data using the following classifications shown in Table 3.1 (FDOT – Flexible Pavement Condition Survey Handbook, 2009) and results in the reduction of the crack rating.

Table 3.1 – FDOT's Condition Survey Raveling Codes

PERCENT OF PAVEMENT AREA AFFECTED BY RAVELING	RAVELING SEVERITY LEVEL AND CODE		
	LIGHT	MODERATE	SEVERE
01 -- 05	1	1	1
06 -- 25	2	2	2
26 -- 50	3	3	3
51+	4	4	4
Note: Code the Predominant severity level only			

As shown in Table 3.1, raveling is defined with "severity" level and percent of pavement area affected. For example, a value of "L2" would mean light raveling affecting 6 to 15% of the pavement area.

During the data collection, an attempt was also made to try and collect FC-5 performance information from various Florida districts (Figures 3.1 and 3.2) to help include different virgin materials/job mix formulas (JMF). Different performance life was also taken into consideration as the research team wanted to evaluate if there were any glaring JMF parameters that may correlate to the measured field performance. Figures 3.3 through 3.8 are the resultant of the PMS query with a tabular form of the information shown in Tables 3.2 to 3.7.

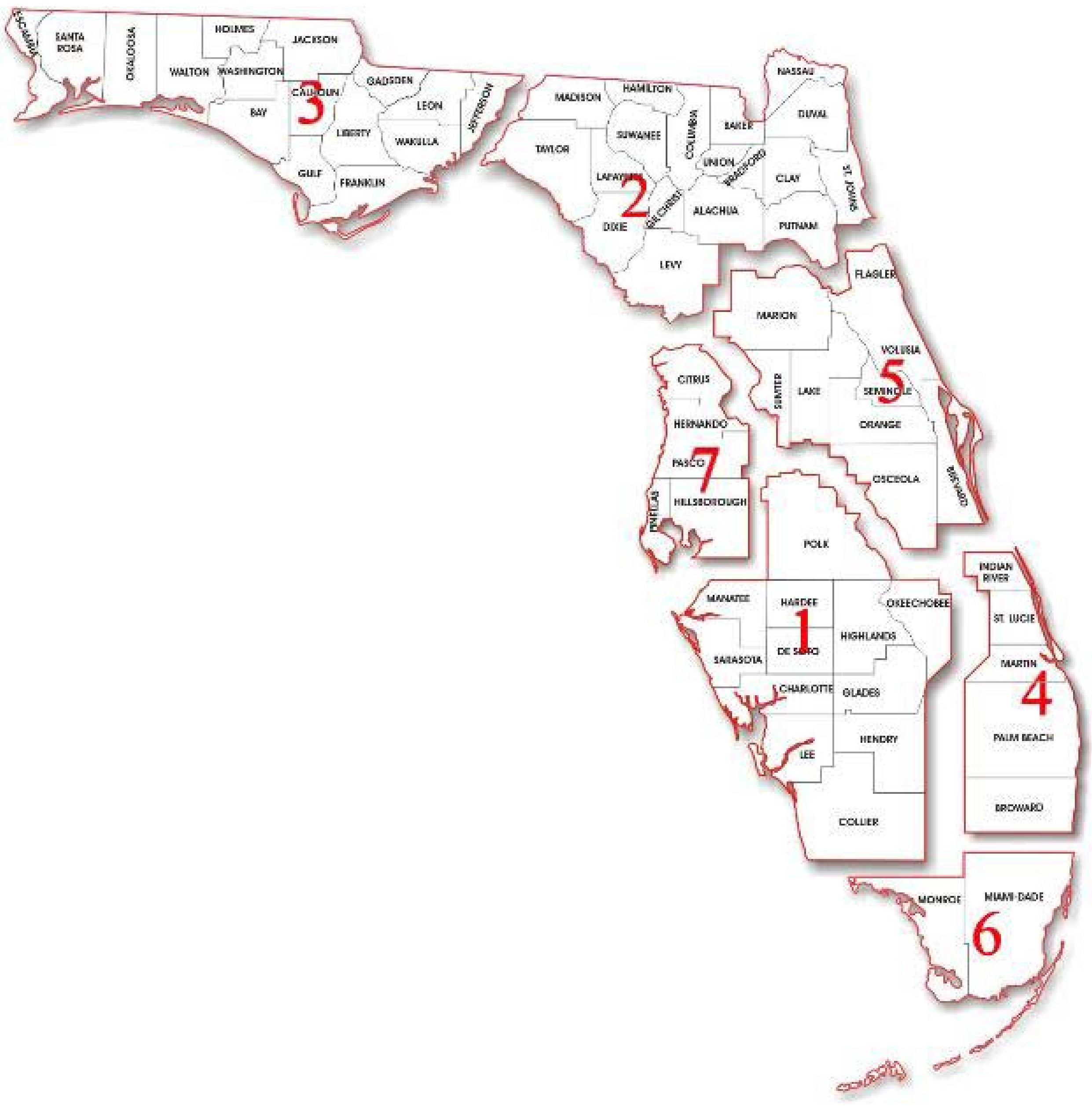


Figure 3.1 - Districts and Associated Counties in Florida

COUNTY CODES

Code	County	Code	County	Code	County	Code	County
26	Alachua	73	Flagler	11	Lake	15	Pinellas
27	Baker	49	Franklin	12	Lee	16	Polk
46	Bay	50	Gadsden	55	Leon	76	Putnam
28	Bradford	31	Gilchrist	34	Levy	58	Santa Rosa
70	Brevard	05	Glades	56	Liberty	17	Sarasota
86	Broward	51	Gulf	35	Madison	77	Seminole
47	Calhoun	32	Hamilton	13	Manatee	18	Sumter
01	Charlotte	06	Hardee	36	Marion	37	Suwannee
02	Citrus	07	Hendry	89	Martin	78	St. Johns
71	Clay	08	Hernando	90	Monroe	94	St. Lucie
03	Collier	09	Highlands	74	Nassau	38	Taylor
29	Columbia	10	Hillsborough	57	Okaloosa	39	Union
87	Miami-Dade	52	Holmes	91	Okeechobee	79	Volusia
30	Dixie	88	Indian River	75	Orange	59	Wakulla
04	DeSoto	53	Jackson	92	Osceola	60	Walton
72	Duval	54	Jefferson	93	Palm Beach	61	Washington
48	Escambia	33	Lafayette	14	Pasco	97	Turnpike
						99	Statewide

Figure 3.2 - Florida State County Codes

Each figure shows a plot of the PMS crack rating vs. year. In all cases, the time period shown in the graphs starts as a new pavement section and dates until the last PMS data collection cycle in the PMS database. Along with the Crack Rating, the level of Raveling is also shown, corresponding to the codes in Table 3.1. For example, County ID: 01075000, SR-93, MP 17.295 – 22.008 was constructed in 2004, where the initial crack rating was a 10.0. Figure 3.1 continues to show this section decrease to a crack rating of 7.0 in year 2010. It also shows that raveling in this section was first noted in 2010 with a level of “L2”.

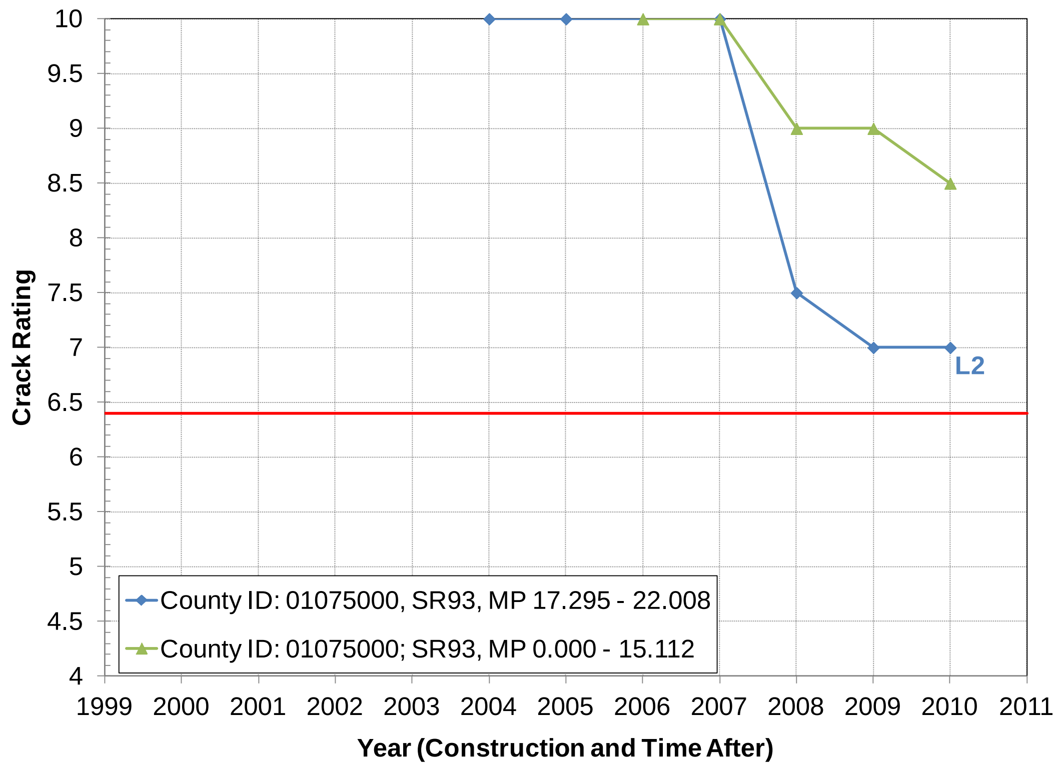


Figure 3.3 – FC-5 Cracking and Raveling Performance in District 1

Table 3.2 - FC-5 Cracking and Raveling Performance in District 1

County ID	SR	US	BMP	EMP	Sys	Rdwy	Type	Lane	Yr	Crk Rat	District
10750000	93	75	17.301	22.008	4	3	7	2	2004	10	1
	93	75	17.301	22.008	4	3	1	2	2005	10	1
	93	75	17.301	22.008	4	3	1	2	2007	10	1
	93	75	17.301	22.008	4	3	1	2	2008	7.5	1
	93	75	17.301	22.008	4	3	1	2	2009	7	1
	93	75	17.301	22.008	4	3	1	2	2010	7	1
County ID	SR	US	BMP	EMP	Sys	Rdwy	Type	Lane	Yr	Crk Rat	District
10750000	93	75	0	15.112	4	3	7	2	2006	10	1
	93	75	0	15.112	4	3	1	2	2007	10	1
	93	75	0	15.112	4	3	1	2	2008	9	1
	93	75	0	15.112	4	3	1	2	2009	9	1
	93	75	0	15.112	4	3	1	2	2010	8.5	1

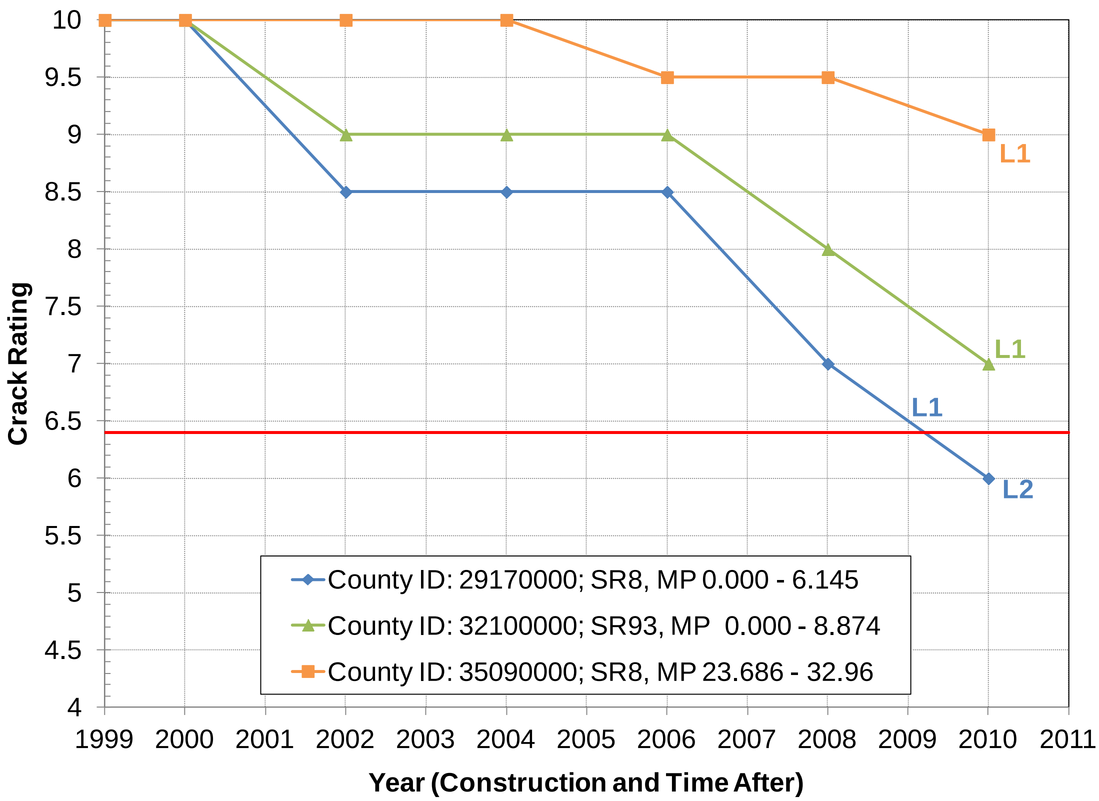


Figure 3.4 – FC-5 Cracking and Raveling Performance in District 2

Table 3.3 – FC-5 Cracking and Raveling Performance in District 2

County ID	SR	US	BMP	EMP	Sys	Rdwy	Type	Lane	Yr	Crk Rat	District
29170000	4008	4010	0	6.145	4	3	7	2	1999	10	2
	4008	4010	0	6.145	4	3	1	2	2000	10	2
	8	10	0	6.145	4	3	1	2	2002	8.5	2
	8	10	0	6.145	4	3	1	2	2004	8.5	2
	8	10	0	6.145	4	3	1	2	2006	8.5	2
	8	10	0	6.145	4	3	1	2	2008	7	2
	8	10	0	6.145	4	3	1	2	2010	6	2

County ID	SR	US	BMP	EMP	Sys	Rdwy	Type	Lane	Yr	Crk Rat	District
32100000	93	4075	0	8.874	4	3	7	3	1999	10	2
	93	4075	0	8.874	4	2	1	2	2000	10	2
	93	4075	0	8.874	4	2	1	2	2002	9	2
	93	4075	0	8.874	4	2	1	2	2004	9	2
	93	4075	0	8.874	4	2	1	2	2006	9	2
	93	4075	0	8.874	4	2	1	2	2008	8	2
	93	4075	0	8.874	4	2	1	2	2010	7	2

County ID	SR	US	BMP	EMP	Sys	Rdwy	Type	Lane	Yr	Crk Rat	District
35090000	8	4010	23.686	32.96	4	3	7	2	1999	10	2
	8	4010	23.686	32.96	4	3	1	2	2000	10	2
	8	4010	23.686	32.96	4	3	1	2	2002	10	2
	8	4010	23.686	32.96	4	3	1	2	2004	10	2
	8	4010	23.686	32.96	4	3	1	2	2006	9.5	2
	8	4010	23.686	32.96	4	3	1	2	2008	9.5	2
	8	4010	23.686	32.96	4	3	1	2	2010	9	2

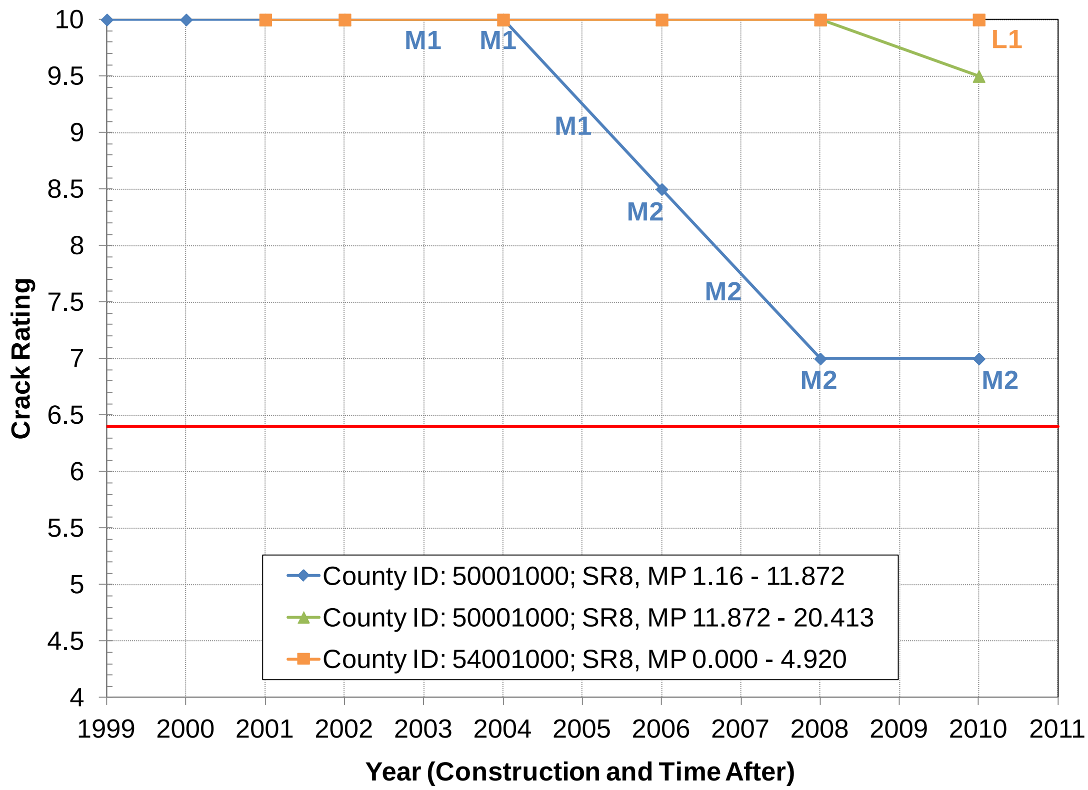


Figure 3.5 - FC-5 Cracking and Raveling Performance in District 3

Table 3.4 – FC-5 Cracking and Raveling Performance in District 3

County ID	SR	US	BMP	EMP	Sys	Rdwy	Type	Lane	Yr	Crk Rat	District
50001000	8	10	1.16	11.872	4	3	7	2	1999	10	3
	8	10	1.16	11.872	4	3	1	2	2000	10	3
	8	10	1.16	11.872	4	3	1	2	2002	10	3
	8	10	1.16	11.872	4	3	1	2	2004	10	3
	8	10	1.16	11.872	4	3	1	2	2006	8.5	3
	8	10	1.16	11.872	4	3	1	2	2008	7	3
	8	10	1.16	11.872	4	3	1	2	2010	7	3

County ID	SR	US	BMP	EMP	Sys	Rdwy	Type	Lane	Yr	Crk Rat	District
50001000	8	10	11.872	20.413	4	3	7	2	2001	10	3
	8	10	11.872	20.413	4	3	1	2	2002	10	3
	8	10	11.872	20.413	4	3	1	2	2004	10	3
	8	10	11.872	20.413	4	3	1	2	2006	10	3
	8	10	11.872	20.413	4	3	1	2	2008	10	3
	8	10	11.872	20.413	4	3	1	2	2010	9.5	3

County ID	SR	US	BMP	EMP	Sys	Rdwy	Type	Lane	Yr	Crk Rat	District
54001000	8	4010	0	4.92	4	3	7	2	2001	10	3
	8	4010	0	4.92	4	3	1	2	2002	10	3
	8	4010	0	4.92	4	3	1	2	2004	10	3
	8	4010	0	4.92	4	3	1	2	2006	10	3
	8	4010	0	4.92	4	3	1	2	2008	10	3
	8	4010	0	4.92	4	3	1	2	2010	10	3

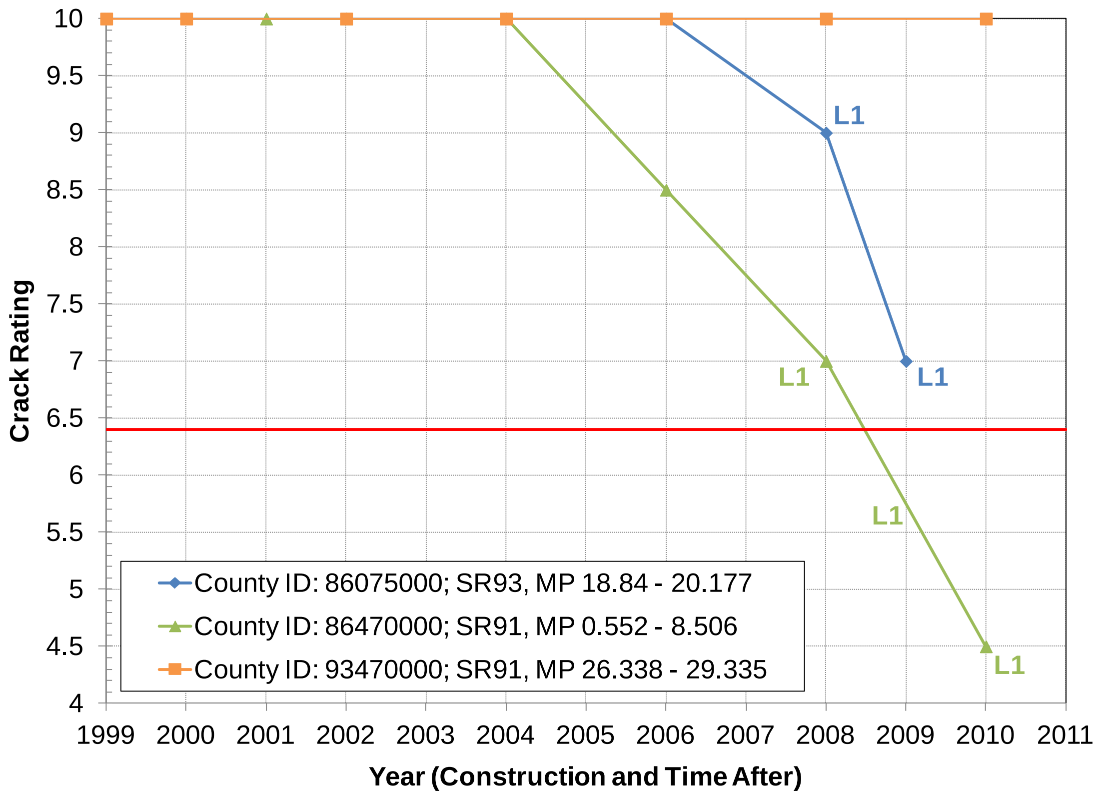


Figure 3.6 - FC-5 Cracking and Raveling Performance in District 4

Table 3.5 – FC-5 Cracking and Raveling Performance in District 4

County ID	SR	US	BMP	EMP	Sys	Rdwy	Type	Lane	Yr	Crk Rat	District
86075000	4093	4075	18.84	20.177	4	3	7	2	2000	10	4
	93	75	18.84	20.177	4	3	1	2	2002	10	4
	93	75	18.84	20.177	4	3	1	2	2004	10	4
	93	75	18.84	20.177	4	3	1	2	2006	10	4
	93	75	18.84	20.177	4	3	1	2	2008	9	4
	93	75	18.84	20.177	4	3	1	2	2009	7	4
County ID	SR	US	BMP	EMP	Sys	Rdwy	Type	Lane	Yr	Crk Rat	District
86470000	91	---	0.552	8.506	5	3	7	3	2001	10	4
	91	---	0.552	8.506	5	3	1	3	2002	10	4
	91	---	0.552	8.506	5	3	1	3	2004	10	4
	91	---	0.552	8.506	5	3	1	3	2006	8.5	4
	91	---	0.552	8.506	5	3	1	3	2008	7	4
	91	---	0.552	8.506	5	3	1	3	2010	4.5	4
County ID	SR	US	BMP	EMP	Sys	Rdwy	Type	Lane	Yr	Crk Rat	District
93470000	91	---	26.338	29.335	5	3	7	2	1999	10	4
	91	---	26.338	29.335	5	3	1	2	2000	10	4
	91	---	26.338	29.335	5	3	1	2	2002	10	4
	91	---	26.338	29.335	5	3	1	2	2004	10	4
	91	---	26.338	29.335	5	3	1	2	2006	10	4
	91	---	26.338	29.335	5	3	1	2	2008	10	4
	91	---	26.338	29.335	5	3	1	2	2010	10	4

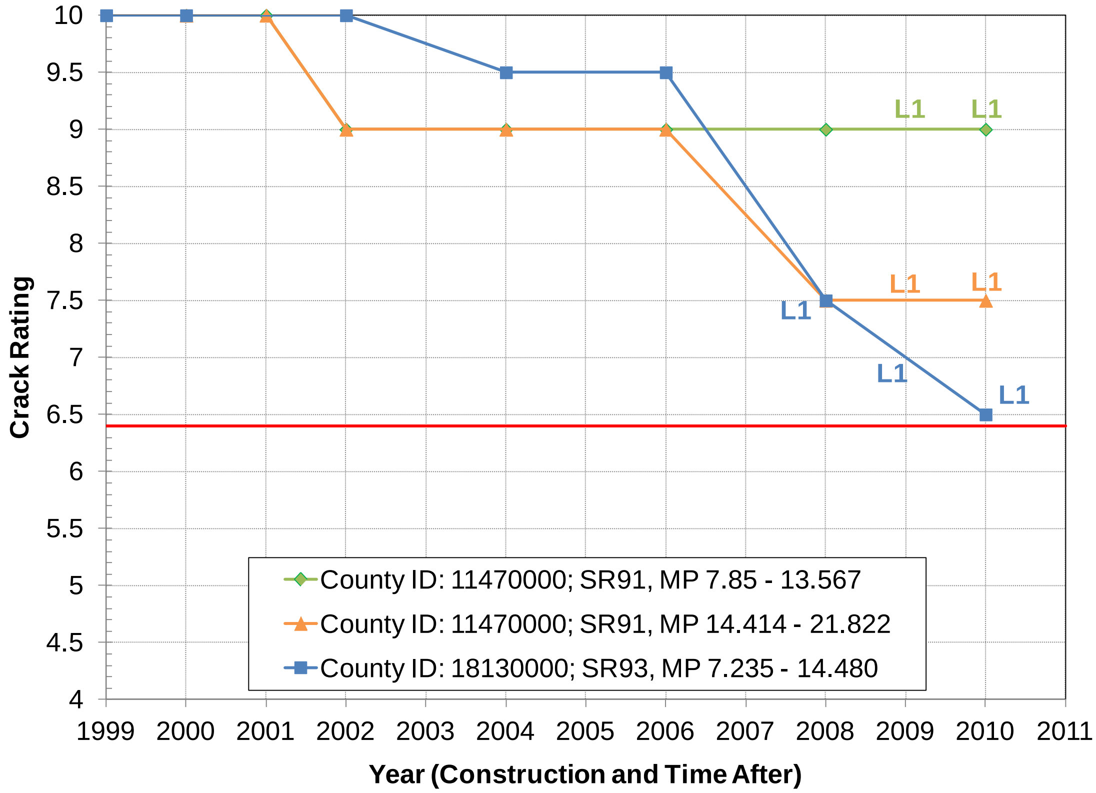


Figure 3.7 – FC-5 Cracking and Raveling Performance in District 5

Table 3.6 – FC-5 Cracking and Raveling Performance in District 5

County ID	SR	US	BMP	EMP	Sys	Rdwy	Type	Lane	Yr	Crk Rat	District
11470000	91	---	7.782	13.567	5	2	7	2	2000	10	5
	91	---	7.782	13.567	5	2	1	2	2001	10	5
	91	---	7.782	13.567	5	2	1	2	2002	9	5
	91	---	7.782	13.567	5	2	1	2	2004	9	5
	91	---	7.782	13.567	5	2	1	2	2006	9	5
	91	---	7.782	13.567	5	2	1	2	2008	9	5
	91	---	7.782	13.567	5	2	1	2	2010	9	5
	91	---	14.414	21.822	5	2	7	2	2000	10	5
	91	---	14.414	21.822	5	2	1	2	2001	10	5
	91	---	14.414	21.822	5	2	1	2	2002	9	5
	91	---	14.414	21.822	5	2	1	2	2004	9	5
	91	---	14.414	21.822	5	2	1	2	2006	9	5
	91	---	14.414	21.822	5	2	1	2	2008	7.5	5
	91	---	14.414	21.822	5	2	1	2	2010	7.5	5
County ID	SR	US	BMP	EMP	Sys	Rdwy	Type	Lane	Yr	Crk Rat	District
18130000	93	4075	7.235	14.48	4	2	7	2	1999	10	5
	93	4075	7.235	14.48	4	2	1	2	2000	10	5
	93	4075	7.235	14.48	4	2	1	2	2002	10	5
	93	4075	7.235	14.48	4	2	1	2	2004	9.5	5
	93	4075	7.235	14.48	4	2	1	2	2006	9.5	5
	93	4075	7.235	14.48	4	2	1	2	2008	7.5	5
	93	4075	7.235	14.48	4	2	1	2	2010	6.5	5

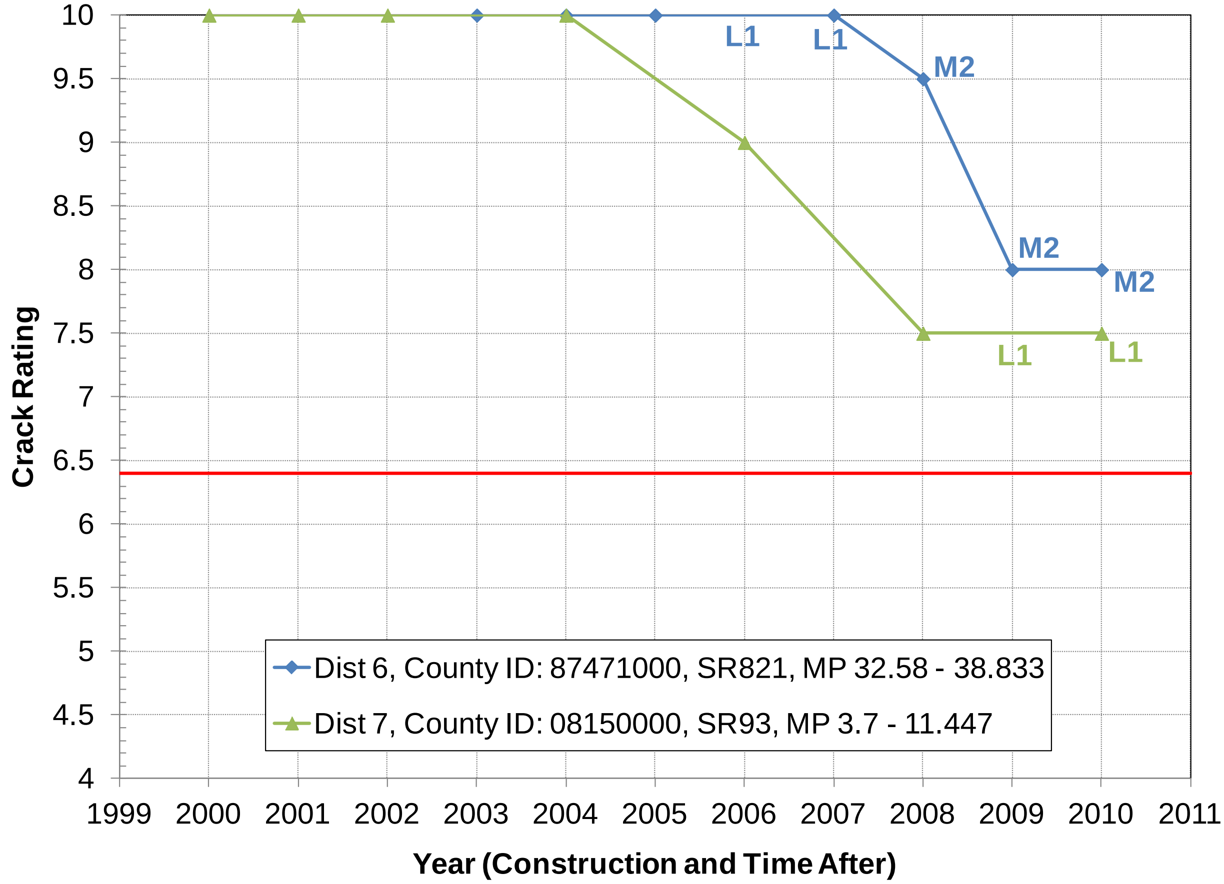


Figure 3.8 - FC-5 Cracking and Raveling Performance in District 6 and 7

Table 3.7 – FC-5 Cracking and Raveling Performance in District 6 and 7

County ID	SR	US	BMP	EMP	Sys	Rdwy	Type	Lane	Yr	Crk Rat	District
87471000	821	---	32.58	38.833	5	2	7	3	2003	10	6
	821	---	32.58	38.833	5	2	1	3	2004	10	6
	822	---	32.58	38.833	5	2	1	3	2005	10	6
	823	---	32.58	38.833	5	2	1	3	2007	10	6
	824	---	32.58	38.833	5	2	1	3	2008	9.5	6
	824	---	32.58	38.833	5	2	1	3	2009	8	6
	825	---	32.58	38.833	5	2	1	3	2010	8	6

County ID	SR	US	BMP	EMP	Sys	Rdwy	Type	Lane	Yr	Crk Rat	District
8150000	93	4075	3.7	11.447	4	3	7	2	2000	10	7
	93	4075	3.7	11.447	4	3	1	2	2001	10	7
	93	4075	3.7	11.447	4	3	1	2	2002	10	7
	93	4075	3.7	11.447	4	3	1	2	2004	10	7
	93	4075	3.7	11.447	4	3	1	2	2006	9	7
	93	4075	3.7	11.447	4	3	1	2	2008	7.5	7
	93	4075	3.7	11.447	4	3	1	2	2010	7.5	7

The PMS cracking and raveling data showed that District location was not a direct influence on the overall durability performance of the FC-5 mixtures. In all cases evaluated, there were FC-5 sections with longer and shorter performance curves. For example, District 4 had 3 FC-5 pavement sections evaluated. Two of the sections on SR-91 (Florida Turnpike), had different levels of performance. The FC-5 placed between mileposts 0.552 to 8.506 lasted 9 years before it reached a terminal crack rating. However, milepost 26.338 to 29.335 was still rated as a 10 (no cracking present) at over 10 years of service life (Figure 3.4 and Table 3.5). Information like this would indicate that although traffic conditions may be aiding in the durability issues, mixture and construction parameters may have a greater influence. Therefore, further analysis of more detailed traffic conditions and mixture components were conducted. Unfortunately, information regarding construction (i.e. – laydown temperature, tack coat conditions, etc.) were not available.

3.2 PMS Crack Analysis - Parameters

A more detailed analysis of the rates and timing of the cracking performance rates of the different FC-5 pavement sections, using the four different performance indicators below, was conducted to help better understand the field performance of the FC-5 mixtures. It should be noted that information regarding the condition of the underlying pavement surface was not provided or found in the database.

1. Time After Construction Until Cracking Started – this parameter was defined as the time from initial construction to the time cracking was first observed in the PMS Crack Rating (in Years).
2. Total Reduction in Crack Rating – this parameter was defined as the total reduction in Crack Rating over the time interval measured in the PMS database (in Crack Rating)
3. Crack Acceleration – this parameter was defined as the rate of change in the Crack Rating once cracking was first observed (in Crack Rating/Year)
4. Crack Rating Per Year – defined as the yearly decrease in Crack Rating measured immediately after construction was completed until the last PMS time interval measured (in Crack Rating Per Year).

The Research Team also asked for, and was provided, the FC-5 mixture production data. Utilizing the FC-5 pavement section information provided by FDOT, the Research Team filtered out various parameters which are believed to be influential to asphalt material cracking performance and conducted an analysis to determine if these parameters were correlated to the observed cracking in the FC-5 pavement sections. Based on the information provided, the Research Team looked at the following parameters;

1. Average Annual Daily Traffic, AADT
2. Average Annual Daily Truck Traffic, AADTT
3. Accumulated AADT (AADT x PMS Performance Time Interval)
4. Accumulated AADTT (AADTT x PMS Performance Time Interval)
5. Design Asphalt Content
6. Actual Asphalt Content (from QC data)
7. Estimated Aggregate Blend Absorption
8. Estimated Effective Asphalt Content (Actual Asphalt Content – Estimated Aggregate Blend Absorption)

9. Estimated Asphalt Film Thickness
10. Estimated Dust/Effective Asphalt Ratio

It should be noted that the term "estimated" is used as limited volumetric and aggregate properties were provided thereby requiring some engineering judgment in effective asphalt content calculations.

3.3 PMS Crack Analysis – Results

The first set of mixture parameters to be compared to the pavement cracking performance was the asphalt binder contents; Actual Total AC (from construction QC data) and Estimated Effective AC. A comparison of these parameters to the "Time After Construction Until Cracking Started" is shown in Figure 3.9. The results in Figure 3.9 show a relatively good correlation between the Estimated Effective AC and the Time After Construction Until Cracking Started. In fact, the trendline seems to plateau at 6% Estimated Effective AC, indicating that at effective asphalt contents of this level, superior cracking performance in the FC-5 pavement sections would be observed.

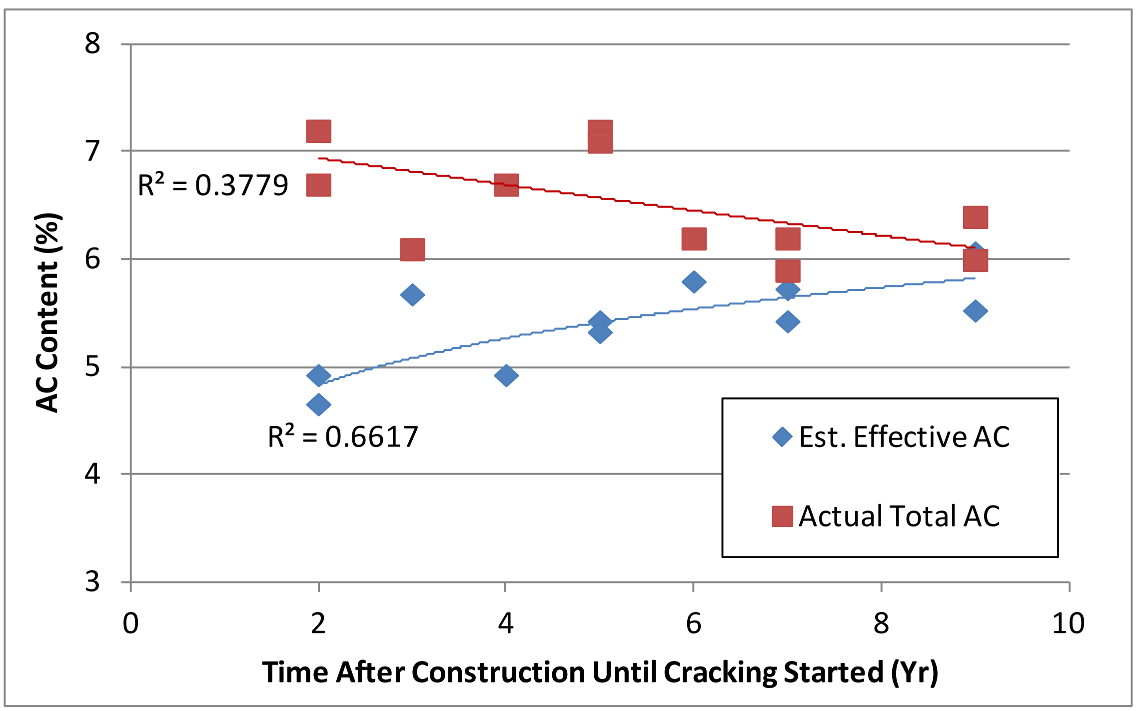


Figure 3.9 - Asphalt Binder Parameters vs Time After Construction Until Cracking Started

Figure 3.9 also indicates that the Actual Total AC did not correlate to cracking performance as one would expect (i.e. – as Total AC increases, fatigue life increases). This clearly indicates that the absorption properties of the aggregates used in the FC-5 mixtures plays a significant role in the cracking performance.

It should be noted that the only pavement performance characteristic which showed any relationship to the material/traffic parameters was the "Time After Construction Until Cracking Started". Therefore, for the sake of brevity, the other parameters are not shown in this report.

Figure 3.10 shows the pavement cracking performance compared to the Estimated Aggregate Blend Absorption. The aggregate blend absorption was estimated using the AASHTO T 84 and T 85 "water absorption" properties of the source aggregates provided by the FDOT. The resultant comparison showed to have a moderate correlation to one another, as indicated in Figure 3.10.

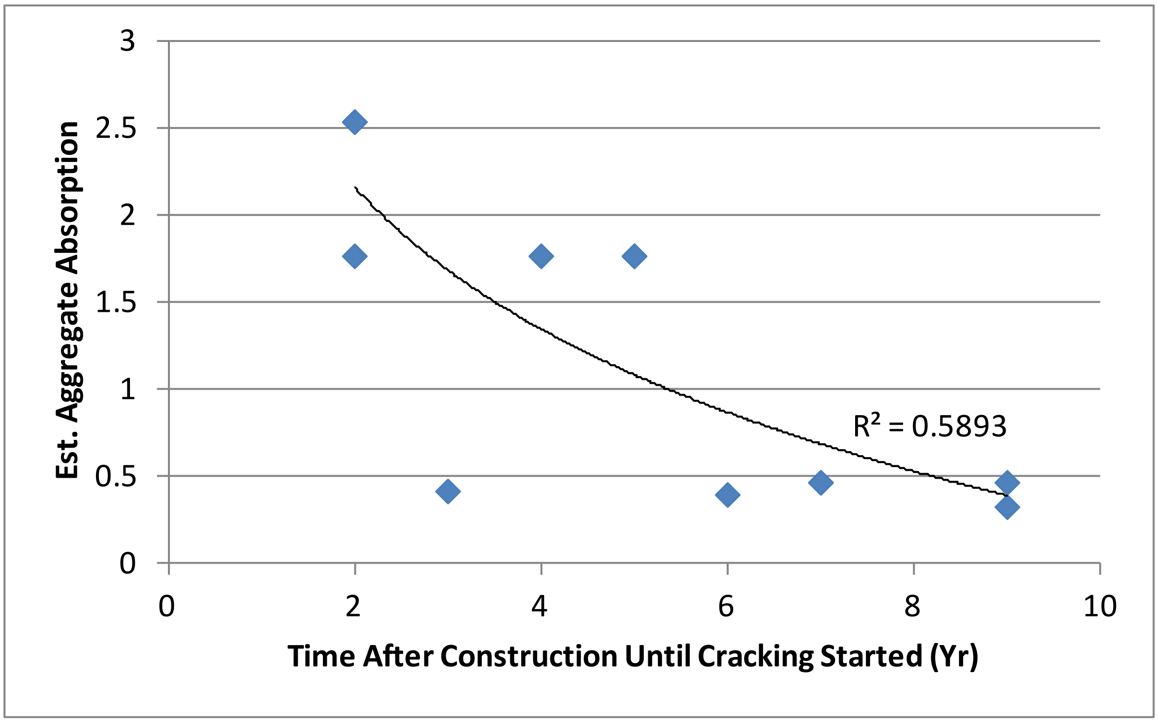


Figure 3.10 - Estimated Aggregate Blend Absorption vs Time After Construction Until Cracking Started

Figure 3.11 contains the results of the comparison between Asphalt Film Thickness and Time After Construction Until Cracking Started. A moderate correlation exists indicating that as the asphalt film thickness increases, an increase in FC-5 cracking resistance increases. As mentioned earlier, asphalt film thickness required more engineering judgment than the remaining parameters due to the lack of volumetric properties commonly used in the calculations.

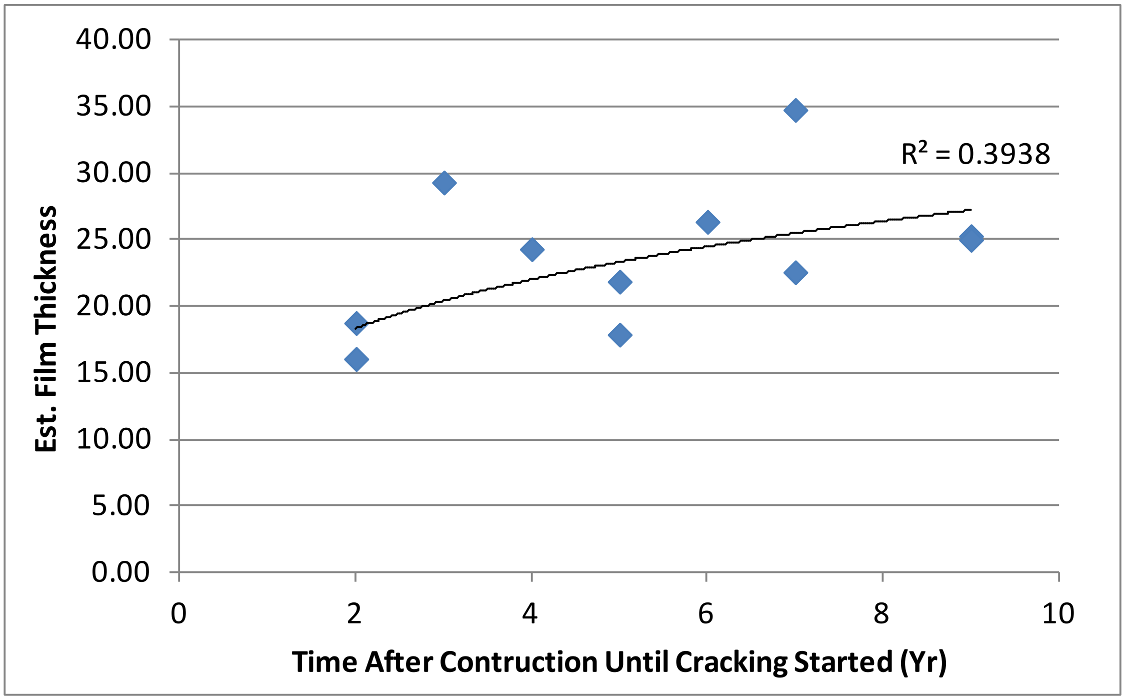


Figure 3.11 - Estimated Asphalt Film Thickness vs Time After Construction Until Cracking Started

The last FC-5 material parameter that indicated any type of correlation to the FC-5 pavement cracking performance was the dust to effective asphalt binder ratio (Figure 3.12). Of the FC-5 material parameters indicating a correlation, the dust to effective asphalt binder content ratio was the weakest. However, the figure does indicate that as the dust to effective asphalt binder content decreases, the FC-5 pavement life, with respect to fatigue cracking, increases. This again illustrates the importance of achieving a higher effective asphalt content in the FC-5 mixtures.

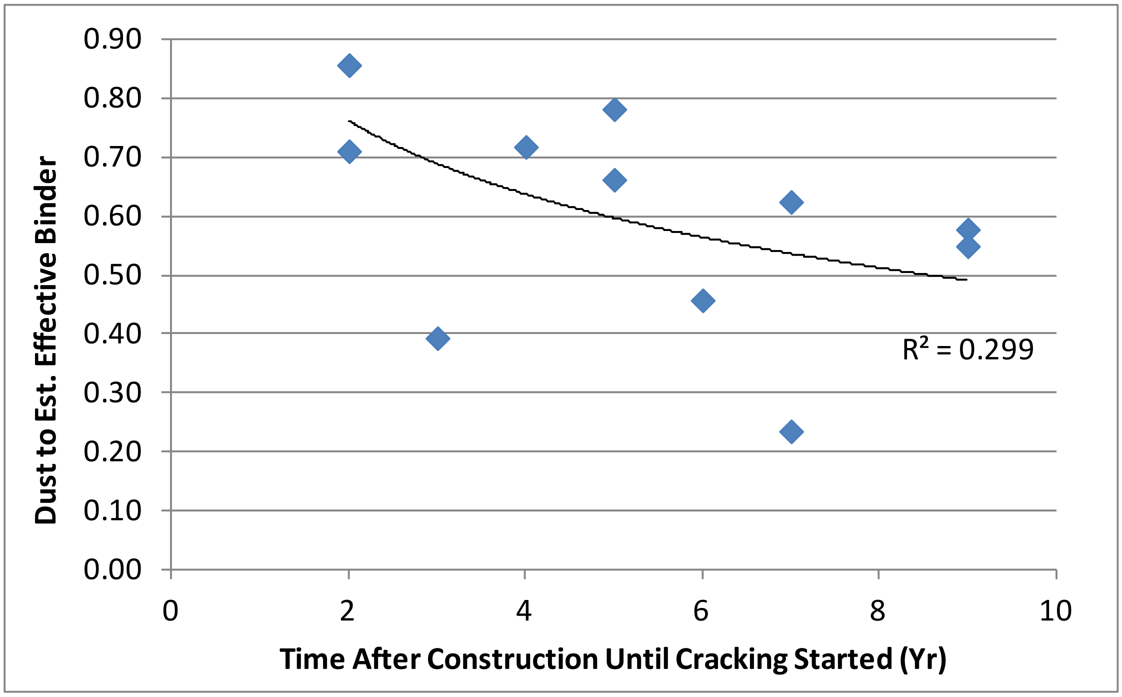


Figure 3.12 - Dust to Effective Asphalt Binder Content vs Time After Construction Until Cracking Started

The final parameter compared to the FC-5 pavement cracking performance was traffic. Originally, the Research Team was concerned that the variable, applied traffic loading of the different pavement sections may confuse any found relationship between the FC-5 material characteristic and the pavement cracking performance. However, there were no moderate to strong correlations found between traffic and FC-5 pavement cracking performance. The only mentionable correlation found is shown in Figure 3.13, and surprisingly, it is indicating that as the cumulative AADTT increases over the performance time interval, the FC-5 pavement cracking performance increases. Obviously, this is opposite of what would be expected and it should be noted that only a minor correlation is found. However, the fact that little to no correlation existed between traffic conditions and the FC-5 pavement cracking performance is encouraging as this would indicate that an increase in cracking performance may be achievable by modifying the material parameters of the FC-5 mixtures (i.e. – effective asphalt content).

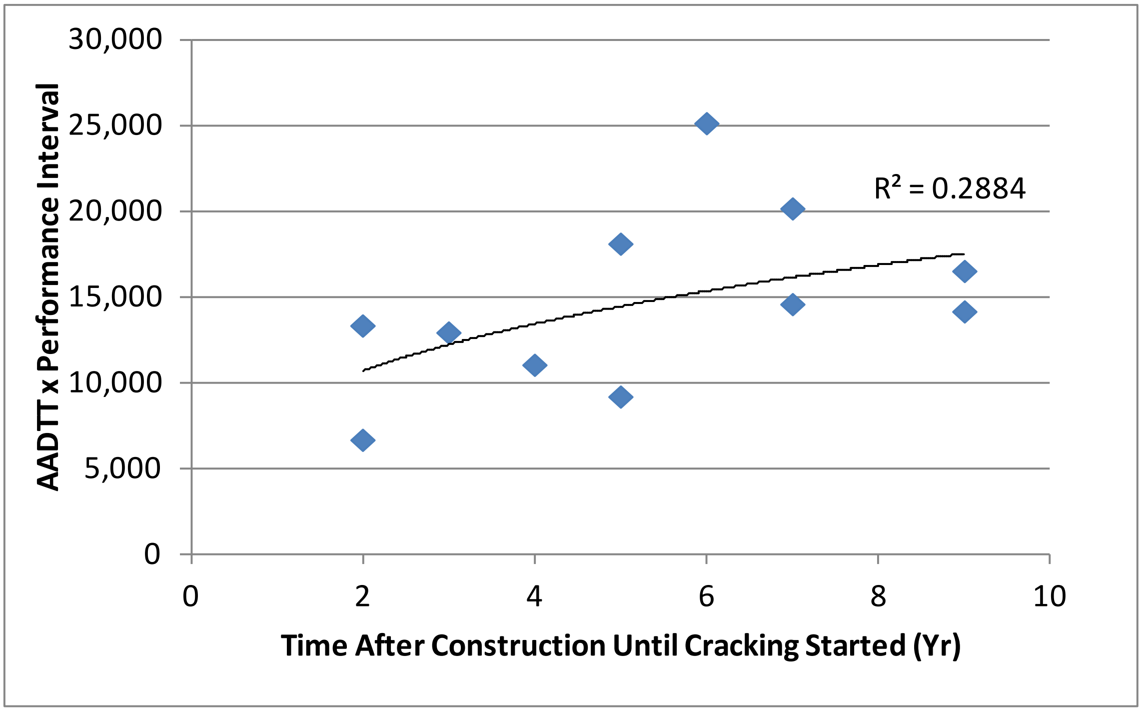


Figure 3.13 - AADTT x Pavement Performance Interval vs Time After Construction Until Cracking Started

3.4 Summary of PMS Query Analysis

The FDOT PMS database was queried to collect pavement cracking performance of FC-5 pavement sections using FDOT's pavement Crack Rating. After selecting various FC-5 pavement sections to evaluate, FDOT procured and supplied the Research Team with mixture design and production QC information pertaining to the material properties and traffic data commonly collected under current FDOT protocols. The Research Team analyzed the provided data and was able to come up with the following conclusions from the experiment:

- The effective asphalt binder properties of the FC-5 mixtures had a strong relationship to the FC-5 pavement cracking performance. This relationship clearly indicated that as the effective asphalt content of the FC-5 mixture increased, the fatigue life of the FC-5 pavement section increased. And based on the shape of the trendline, it would indicate that effective asphalt contents reaching levels of 6% would have superior fatigue cracking resistance.
- Other FC-5 material parameters, involving the aggregate absorption and effective asphalt binder properties of the FC-5 mixtures all showed some kind of relationship. These predominantly included; the Estimated Aggregate Absorption and the Estimated Film Thickness.
- Little to no correlation was found between the provided traffic information and the FC-5 pavement cracking performance. This would indicate that it may possible to increase the fatigue resistance of the FC-5 pavement sections through positive changes in the FC-5 materials and mixture design properties.

3.5 FC-5 Wearing Course Site Visit

One issue that always arises as researchers trying to improve a mix design procedure is visualizing the types and severities of distresses that are encountered. The data developed during the evaluation of the PMS data provided valuable insight on how the different distresses developed over time. However, using the data within the PMS did not provide an indication as to whether the distresses were developed because of material problems, production problems, or construction problems. In order to achieve a better understanding of the causes of the distresses, a site visit to various FC-5 field sections was conducted to visually inspect pavements having typical durability distresses.

The pavement sections identified within the evaluation of the PMS data were selected for the site visit. In evaluating the actual geographical locations of the different pavement sections discussed earlier, the general locations of the sections were found to be within three distinct areas of the state. One geographical area was within the northern portion of the state and encompassed areas of I-10 within Districts 2 and 3 as well as I-75 within District 2. Another geographical area included I-75 and the Florida Turnpike within Districts 5 and 7. These sections were within the central portion of the state. The final geographical area was the I-75 and the Florida Turnpike within District 4 which is in the southern portion of the state.

Dr. Allen Cooley visited Florida during the week of February 27 to March 2, 2012. The visit was conducted in order to visually see the types of distresses typical of FC-5 mixes. During the course of the visit, Dr. Cooley, with the valuable assistance of FDOT personnel, traveled to the three geographical areas described above to see the types and severities of distresses common to FDOT's FC-5 mixes. The intent was not to specifically evaluate the sections highlighted earlier in the report; rather, the intent was to evaluate as many sections as possible within the geographical areas highlighted. Though an attempt was made to target the specific sections noted earlier, there were instances where these sections had already been replaced. As such, many times the evaluations basically occurred because something changed within the FC-5's appearance while driving over a section. Cracks, raveling, or visual changes in the FC-5 surface would cause an evaluation of the section. In each instance, the potential cause of the distress or visual appearance was identified. This potential cause was then deemed to be either caused by material problems, production problems, or construction problems.

3.5.1 FC-5 Site Visit – Day 1

On the first day of the visit, Dr. Cooley and Scott Ellis and Ken Green of FDOT traveled to evaluate FC-5 mixes that were placed on local, lower volume roadways around Gainesville.

The first FC-5 section evaluated was on SR-20 just west of Hawthorne, Florida. This section was last resurfaced in 2000. Granite was the predominant aggregate of this FC-5. Two distresses were noted within the section evaluated: raveling and cracking. Of these two distresses, raveling was more prevalent. Figure 3.14 shows a typical aggregate pop-out from SR-20. Besides these types of aggregate pop-outs, additional raveling was noticed. Coated aggregate particles were observed on the pavement shoulder. In addition to the raveling, several longitudinal cracks were observed within this section. The cracks were not continuous and were observed sporadically over the entire project. Typically, the longitudinal cracks were 4 to 6 ft in length (Figure 3.15) and observed within the wheelpath. There did not appear to be any production or construction issues that caused the distresses. Visually (and this can be observed in Figure 3.14), the FC-5 appeared to be slightly under-asphalted. Not only had the asphalt binder been worn away from the pavement surface by the action of tires, but the aggregates visible under the surface appeared to have very thin asphalt films.

Figure 3.14 - Aggregate Pop-out on SR-20

Figure 3.15 - Longitudinal Crack on SR-20

The second lower volume FC-5 roadway evaluated was US-441 just south of Paynes Prairie in Alachua County. This pavement evaluated was 6.5 years old at the time. The only distress noted in this section of roadway was a low severity of raveling. Again, visually, the FC-5 had the appearance of being under-asphalted or having the appearance where the asphalt film-thickness has worn away.

3.5.2 FC-5 Site Visit – Day 2

Sections of FC-5 within the northern portion of the state were evaluated; specifically, I-75 from Lake City to the Georgia state line and I-10 from Lake City to west of Tallahassee.
The first section of pavement evaluated was on I-75 north of Lake City. This section was 8.5 years old at the time of the site visit. The predominant aggregate in the mix seemed to be an Oolitic limestone and may have been an FC-2 according to the accompanying FDOT engineer. This section showed longitudinal cracking running between the wheelpaths of the middle lane for the entire length of the project. Because of the location of the longitudinal crack, it was hypothesized that the crack was likely caused by some underlying construction joint. However, based on conversations with FDOT engineers, there actually is a longitudinal joint between the wheelpaths on this pavement section. Therefore, the crack noted appears to be longitudinal cracking, most likely top-down, as is typically observed in Florida.

The second section evaluated was an FC-5 mix comprised of a granite aggregate located on I-75 in Hamilton County, between Mile Posts 454 and 455. The pavement section was 2.5 years old at the time of the site visit but was already experiencing low to moderate severity raveling. Most of the raveling appeared to be occurring within the middle and outside lanes. Asphalt binder coated aggregates were observed on the pavement shoulder. A close inspection of the in-place FC-5 mix indicated that the asphalt binder had a dull, almost brown color. It was hypothesized that mix temperature during production was likely higher than desired resulting in the dull color of the asphalt binder.

Another section evaluated on I-75 in Hamilton County was in the northbound lanes between Mile Posts 461 and 462, which was last resurfaced in 1998. It was evaluated because of rutting, cracking and raveling. As shown in Figure 3.16, the outside lane was experiencing rutting. It was not discernible whether the rutting was limited to the FC-5 layer or whether the rutting resulted from issues in an underlying layer. Cracking was observed within the wheelpaths. These cracks had an appearance similar to fatigue cracks and were low severity. A significant amount of raveling had also occurred within the wheelpaths (Figures 3.17 and 3.18). It was unclear from the visual examination of the pavement what caused the issues; however, the probable cause was from an underlying layer. It did not appear to be a materials issue with the FC-5. One possibility is that stripping had occurred in an underlying layer.

Figure 3.16 - Rutting on I-75 in Hamilton County (MP461-462)

Figure 3.17 - Raveling on I-75 in Hamilton County (MP 461-462)

Figure 3.18 - Raveled Aggregates on Shoulder, I-75 Hamilton County (MP 461-462)

On I-10 in Madison County, west of Lake City near Mile Post 263, a hairline crack was observed about 12 inches off the white skip striping in the outside lane. It was deemed that this hairline crack was likely caused by an underlying longitudinal joint. This section was previously resurfaced in 1999.

The final section evaluated on Day 2 was on I-10 in Jefferson County, w between Mile Posts 229 and 230. Within this section of roadway, which was last resurfaced in 1999, transverse cracks were observed within the FC-5 mix (Figure 3.19). The spacing and shape of the transverse cracks indicate that they were likely reflected upwards from an underlying concrete pavement. Additionally, in some locations, diagonal cracks had reflected to the surface from broken slabs (Figure 3.20). These distresses were not deemed to be FC-5 material related distresses.

Figure 3.19 - Transverse Crack on I-10 (MP 229-230)

Figure 3.20 - Transverse Cracking on I-10 (MP 229-230)

3.5.3 FC-5 Site Visit – Day 3

On the third from the area between Gainesville to Fort Lauderdale was evaluated taking a route of I-75 south to SR-50 continuing on to the Turnpike heading south to Fort Lauderdale. The first section of FC-5 evaluated was in the southbound lane of I-75 in Marion County. This section was between Mile Posts 363 and 364. Within this section, last resurfaced in 2005, there was longitudinal cracking at the edge of the wheelpath in the outside lane (Figure 3.21). The cracks were approximately ½ inch wide at some locations and extended down into the underlying layer (Figure 3.22). In some of the crack locations, 2 to 3 inches below the pavement surface could be seen within the cracks.

In addition to the longitudinal cracks, transverse cracks were observed in some locations. These transverse cracks appeared to begin at the longitudinal crack and extend into the shoulder (Figure 3.23). Raveling was observed within the section. Within some areas, the raveling would be considered moderate severity, especially within the wheelpath. It was unclear if the longitudinal or transverse cracking was material related. However, it is assumed that the cracking was caused by an issue within underlying layers. This was assumed because of the transverse cracks that extended into the shoulder.

Figure 3.21 - Longitudinal Crack on I-75 in Marion County

Figure 3.22 - Close-up of Longitudinal Crack on I-75 in Marion County


Figure 3.23 - Transverse Crack on I-75 in Marion County

The second and last section evaluated on the third day was just north of Exit 133 on the Florida Turnpike. This section was in Martin County, and was previously resurfaced in 2008. Flushing was observed on the inside lane (Figure 3.24). The flushing was observed within the wheelpath. Typically, the flushed areas were about 8 to 10 ft long and as wide as the wheelpath. Additionally, flushed spots were observed at approximately equal spacing down the pavement that suggested that their occurrence was construction related. Maurice McReynolds indicated that the haul time from the nearest asphalt plant was approximately 1 hour. The flushed areas could have been caused by draindown occurring during transportation of the mix. Raveling was also observed, but was low severity.

Figure 3.24 - Flushed Area on Florida Turnpike near MP 133

3.5.4 FC-5 Site Visit – Day 4

On the fourth day of the visit, Dr. Cooley, Maurice McReynolds and Scott Ellis initially traveled north on I-75 across Alligator Alley in Broward County. Next, they returned to the Florida Turnpike and traveled north to I-75 and onto Gainesville.

The first section evaluated was in the northbound direction of I-75 in Broward County near Mile Post 30, which was resurfaced in 2011. Within this section, raveling was observed between the wheelpaths (Figure 3.25). Occurrence of the raveling was cyclical and may have been construction related.

Figure 3.25 - Raveling on I-75 in Broward County near MP 30

Another section of southbound I-75, near Mile Post 9, was evaluated because of raveling. . Raveling was observed throughout the entire pavement surface and was not isolated to certain spots. This section was previously resurfaced in 2007. Raveled aggregates were observed on the inside shoulder of the pavement. Material issues may have been a prevalent cause of the raveling.

3.6 Summary of Field Visit Findings

During the four day time period in which the FC-5 pavement evaluations took place, well over 1,000 miles of Florida roadways were traveled. The vast majority of the pavements driven had an FC-5 wearing course. Even though only twelve (12) specific sections were evaluated (as described previously in detail and shown below in the table), observations were made the entire time of traveling the roadways. Based upon the evaluations and other observations made during the field visit, the following comments are provided:

1. In general, the performance of FC-5 mixes in Florida was good.
2. The most common distress observed with the FC-5 wearing course was raveling. Two issues were observed related to the raveling with the FC-5 sections. First, the most common form of raveling observed was what appeared to be "end of load" issues. These occurrences of raveling were cyclical down the roadway at approximately equal distances apart. This form of raveling is most likely associated with some form of segregation. However, it is unclear whether the segregation is physical or thermal. Raveling of this nature is considered to be a construction related issue and not a material issue. The second issue related to raveling was raveling

across the entire pavement surface. The occurrence of this type of raveling was not as prevalent as the "end of load" form of raveling. However, this type of raveling is likely a material related problem. A number of the FC-5 sections evaluated had the appearance of being under-asphalted (i.e. – gray color and dull appearance). It is common after the construction of an FC-5 mix that asphalt binder will be worn from the pavement surface due to the action of tires. However, visual observation of many of the FC-5 mixes suggested that asphalt binder below the pavement surface was minimal.

3. Cracking was not a predominant distress observed within the FC-5 mixture. These cracks were generally low severity and located between the inside wheel path and lane skip stripe. The next most common type of crack observed was associated with pavement scars. The scars appeared to be caused by vehicles with flat tires passing over the pavement surface, sometimes for great distances. Generally, these types of cracks were also low severity. A very small percentage of pavements had longitudinal cracks that cannot be explained by underlying longitudinal/construction joints or pavement scarring. In these instances, namely SR-20 just west of Hawthorne and I-75 in Marion County, it is unclear whether the cracks are top-down or were reflected upward due to issues within an underlying layer. In the case of I-75, in Marion County, it appeared the cracks were caused by an underlying issue, but that is not 100% certain without forensic investigation. In summary, based upon the roadways travelled, performance of the FC-5 layers with respect to cracking appeared to be good.
4. Flushing of the FC-5 layer was only noticed on two projects. Both of these projects were on the Florida Turnpike. A similarity about both of these sections was that they were located such that any haul times would be long and that the flushing was cyclical. The cyclical nature of the flushed spots indicate that their cause is construction related. A possible cause of these flushed areas is draindown during transportation.
5. An observation at a number of the section evaluated was that the FC-5 appeared to be less than ¾ inches thick.

Table 3.8 summarizes the FC-5 field sections visited, as well as the distress observed and possible cause of the distress.

Table 3.8 – Summary of FC-5 Wearing Course Sections Visited and Observations Made

Highway/Interstate	Location/Mileposts	Distresses Observed	Possible Causes
SR-20	West of Hawthorne	Raveling and Longitudinal Cracking	Low Asphalt Content
US-441	South of Paynes Prairie	Raveling	Low Asphalt Content
I-75 (possible FC-2)	North of Lake City	Longitudinal Cracking	Top-down Cracking
I-75 (possible FC-2)	454 to 455	Raveling	High Mix Production Temperatures
I-75 (possible FC-2)	461 to 462	Rutting, Raveling, and Cracking	Failure in Underlying Layer
I-10	263	Longitudinal Cracking	Underlying Longitudinal Joint
I-10	299 to 230	Transverse Cracking	Reflected from Underlying PCC Pavement
I-75 (possible FC-2)	363 to 364	Longitudinal and Transverse Cracking, Raveling	Unclear, possible issues in Underlying Layers
Florida Turnpike	133	Flushing and Raveling	Long Haul Time During Construction
I-75	30	Raveling	End of Truck Physical or Thermal Segregation
I-75	9	Raveling	Age/Low Asphalt Content
Florida Turnpike	107	Flushing and Raveling	Long Haul Time During Construction

CHAPTER 4 – WORKPLAN FOR LABORATORY STUDY

The information collected regarding the field performance of Florida's FC-5 wearing courses and the corresponding mixture components have led to the belief that some of the issues witnessed regarding the durability of these mixtures are due to selecting an optimum asphalt content, thereby affecting the effective asphalt content of the FC-5 mixture. Lower effective asphalt contents will create issues with raveling and lead to a higher potential for cracking. With the FDOT currently using the pie-plate procedure for determining the optimum asphalt content of FC-5 mixtures, it would appear that an assessment of this procedure is required. And although the goal of the study is not to replace the pie-plate procedure, the addition of a supplemental method or recalibration of Pie-Plate test may be necessary. However, any modifications that may deem to be necessary need thorough evaluations regarding how these changes may influence the overall performance of the FC-5 mixtures, as well as how possible production tolerances may affect performance.

The laboratory approach entailed three separate experiments each designed to build upon the other. The first experiment was designed to determine whether a short-term oven aging (STOA) procedure could improve the performance of FC-5 mixes. Experiment 2 evaluated the effect of gradation on performance, while the third experiment evaluated the influence of typical construction variations on performance. Though these are separate experiments with different objectives, they have been designed to allow the evaluation of many items of interest by building upon one other.

4.1 Experiment 1 – Effect of Aggregate Absorption

Table 4.1 presents the factor-level combinations evaluated within the first experiment. Four aggregates were utilized: two granites and two Oolitic limestones were selected based upon the Phase I research. Using the information collected from FDOT, one low absorption and one high absorption (relative to the mineralogical type) will be selected. A single gradation band was utilized for each of the four aggregates which was based upon the JMF's identified during the PMS Query. Both the PG 76-22 and ARB-12 were also included. Two asphalt binder contents were selected - one that corresponded to the selected JMF (average value determined by FDOT during mixture design) and the second was 0.6 percent above the JMF binder content. The increase of 0.6 percent was selected to correspond to the current FDOT construction tolerance. Finally, two short-term oven aging (STOA) conditions were employed: No STOA and 2 hours STOA.

Table 4.1 - Factor - Level Combinations for Experiment 1

Factor	Levels
Aggregate	4 Aggregates selected on properties, JMFs and Performance
Gradation	1 Gradation for each Aggregate (respective JMF)
Binder	2 Binders: PG 76-22, ARB-12
Binder Content	2 Binder Contents: JMF and 0.6% above
STOA	2 STOA: 0 hours and 2 hours

In review of the Florida Method (FM) 5-588, *Florida Method of Test for Determining the Optimum Asphalt Binder Content of an Open-Graded Friction Course Mixture Using the Pie Plate Method*, the research team noticed that FM 5-588 does not utilize a conditioning time to allow for asphalt absorption prior to placement on the pie plate. As stated in Section 5.7 of FM 5-588;

> "Immediately after mixing, carefully transfer the mixture from the mixing bowl into a pie plate using a method that will evenly distribute the mixture over the entire bottom surface of the pie plate without causing segregation. Care should be taken to ensure that the mixture is not disturbed once it has contacted the pie plate. After placing the mixture in the pie plate, place the pie plate on a level surface in an oven and heat for one hour at $320 \pm 5^{o}F$ ($160 \pm 3^{o}C$). Repeat this step for each of the remaining samples."

Although the research team agrees that the phenomena of draindown can begin immediately after the mixing process has completed, a one hour pie-plate test may not allow for full asphalt binder absorption to take place, thereby selecting an artificially low asphalt binder content (i.e. – allowing more available asphalt binder for draindown). Work conducted by Kandhal and Khatri (1991) shows that depending on the aggregate type and absorptive properties, asphalt absorption can continue for up to 8 hours after mixing has been completed. By ignoring the potential for asphalt absorption to take place, an under-asphalted condition will occur, possibly reducing the effective asphalt content which is known to be directly related to cracking resistance (Christensen and Bonaquist, 2006).

For each of the factor-level combinations shown in Table 4.1, three tests were conducted: FM 5-588, Draindown Test (AASHTO T 305), and the Cantabro Abrasion Loss test described by Cooley et al (2009). These tests were selected for two primary reasons. First, one major cause of cracking in many HMA mixes is absorptive aggregates. Cooley et al (2008) showed that by increasing the STOA length during the volumetric mixture design procedure, the initiation of longitudinal surface-initiated cracks can be postponed. This is mainly due to the fact that currently used conditioning times for highly absorptive aggregates did not sufficiently model the level of asphalt binder absorption taking place. The work by Cooley et al (2008) showed that pavements that had originally cracked after two to three years, after undergoing a longer conditioning time during mixture design, had not shown any cracking at four years. In fact, after recent discussions with the sponsors, cracks still have not been observed six years after the STOA was adopted.

The Pie-Plate and Draindown tests would be able to indicate whether the combination of the increased binder contents and STOA will increase the potential for FC-5 draindown. Secondly, the Cantabro Abrasion Loss test would show whether the increased binder content and STOA procedure will improve durability. Cooley et al, (2009) indicated that the Cantabro Abrasion loss test was the most common durability test for OGFCs worldwide. Additionally, the Cantabro Abrasion Loss testing may identify aggregate sources that are more prone to durability problems. Cantabro Abrasion Loss test specimens were compacted using 50 gyrations of the Superpave gyratory compactor.

During this experiment, the Pie Plate test was conducted with PG 67-22, PG 76-22, and ARB-12 asphalt binder. The PG 67-22 was included because it is the binder required within FM 5-558.

4.2 Experiment 2 – Effect of Gradation on Performance

Table 4.2 presents the factor-level combinations evaluated during the second laboratory experiment. Two aggregate sources, Martin Marietta and White Rock Quarries, were selected based on their predominent use in FDOT FC-5 mixtures. Two gradations were evaluated; a current FC-5 gradation (averaged from several JMFs of the respective aggregate source) and a fine, 9.5 mm nominal maximum aggregate size blend. Cooley et al, (2009) have recommended a 9.5 mm OGFC gradation band as shown in Table 4.3. Again, the PG 76-22 and ARB-12 asphalt binders were included. Two asphalt binder contents were evaluated - one at optimum asphalt binder content determined by the Pie-Plate method and the second at 0.6 percent higher. The optimum asphalt content was based on the average asphalt content of the respective aggregate source. As per FDOT recommendation, 0.6% additional asphalt binder (approximately 12% of the optimum asphalt content weight) was added for the ARB-12 asphalt binder FC-5 mixtures to accommodate for the 12% asphalt binder weight made up of crumb rubber. The final factor was the long-term aging procedure (LTOA). Samples were also prepared with and without using the LTOA procedure. The LTOA consisted of placing compacted samples in a forced-draft oven set at 85°C for 5 days in accordance with AASHTO R 30.

Table 4.2 - Factor - Level Combinations for Experiment 2

Factor	Levels
Aggregate	2 Aggregates selected form Experiment 1
Gradation	2 Gradations: FC-5 and 9.5 mm
Binder	2 Binders: PG 76-22, ARB-12
Binder Content	2 Binder Contents: JMF and 0.6% above
STOA	1 STOA selected from Experiment 1
LTOA	1 LTOA: 5 days at 85°C

Table 4.3 - Gradation Band for 9.5mm OGFC (Cooley et al, 2009)

Sieve	% Passing
½ in. (12.5 mm)	100
3/8 in. (9.5 mm)	85-100
No. 4 (4.75 mm)	20-30
No. 8 (2.36 mm)	5-15
No. 200 (0.075 mm)	0-4

Four performance tests were performed to evaluate each of the factor-level combinations. First, the Cantabro Abrasion Loss test was used to assess the durability of the FC-5 mixture. Secondly, the Overlay Tester (OT) was used to evaluate the cracking potential of the FC-5 mixture. Zhou et al (2006) have shown that the OT has a good relationship between volumetrics (namely asphalt film thickness) and cycles to failure. Since surface initiated cracks have been shown to be initiated by the exceeding of tensile strength at the surface of the pavement, the OT provides a simple, commercially available test method of evaluating the tensile strength of FC-5 materials. Because tensile strength is important in the development of surface initiated cracks, indirect tensile strength testing was also conducted for the various mixtures. The tentative approach was to evaluate the indirect tensile strength at 10°C. Roque et al (2009) appeared to differentiate FC-5 mixtures better at this temperature than other temperatures. The fourth and final test utilized during the second experiment was the Hamburg Wheel Tracking Device (HWTD). This test was selected for two reasons. First, the likely solution to most cracking problems is the addition of asphalt binder to the mix. Any time asphalt binder is added, the stability of the mix must be maintained. The HWTD will allow the stability to be verified. Secondly, the HWTD is a useful tool to evaluate the potential for moisture susceptibility.

Therefore, the goal of Experiment 2 was twofold. First, it would allow the research team to determine whether a finer gradation would improve the durability of OGFC mixes in Florida. Secondly, it would allow the research team to determine whether the asphalt binder content can be increased to improve the durability without increasing the potential for draindown. Data generated in Experiment 2 built upon data developed in the first experiment.

4.3 Experiment 3 – Effects of Construction Variations

The final experiment will evaluate the effect of typical construction variations on the performance of FC-5. Table 4 presents the factor-level combinations to be evaluated during the third experiment. Similar to Experiment 2, two aggregate sources will be included. These two will be identical to those sources used in Experiment 2. Three gradations will be evaluated; the JMF gradation and the JMF gradation plus and minus the construction tolerances. Three asphalt binder contents will also be evaluated, including: JMF, plus construction tolerance, and less construction tolerance.

Table 4.4 - Factor-Level Combinations for Experiment 3

Factor	Levels
Aggregate	2 sources, same as Experiment 2
Gradation	3 Gradations: JMF, ± construction tolerances
Binder	2 Binders: PG 76-22, ARB-12
Binder Content	3 Binder Contents: JMF, ± construction tolerances
Gradation Size	1 Gradation sizes: FC-5 (12.5 mm)
STOA	STOA

Four tests are proposed for each of the factor-level combinations within Experiment 3. The four tests will be identical to those used in Experiment 2 and include the OT, HWTD, indirect tensile strength and Cantabro Abrasion Loss.

Once experiments 1, 2 and 3 have been completed, the data will be analyzed, and conclusions will be made to improve the current mix design method for FC-5 mixes. The final portion of the research will be to design six FC-5 mixes to verify the validity of the revised mix design procedure. 12 mixes (six with revised mix design method and six with FM 5-558) will be subjected to performance tests for comparisons between design methods.

CHAPTER 5 – EFFECT OF AGGREGATE ABSORPTION ON FC-5 MIXTURE DESIGN (EXPERIMENT 1)

Experiment 1 was developed to look at the potential impact of asphalt binder absorption on general performance of FC-5 mixtures. The research team were interested to assess:

1. Would asphalt absorption influence the draindown characteristics of the FC-5 mixtures?
2. Would the asphalt absorption influence the durability of the FC-5 mixtures?
3. Does asphalt binder grade/type change the way the FC-5 mixtures behave in the pie-plate, draindown, and Cantabro Abrasion Loss tests?

Table 5.1 presents the factor-level combinations evaluated within the first experiment. Four aggregates were utilized: two granites and two Oolitic limestones selected based upon the Phase I research. Using the information developed in Task 2, one low absorption and one high absorption (relative to the mineralogical type) was selected. A single gradation was respectively utilized for each of the four aggregates. The gradation was based on the typical FC-5 mixture JMF produced for that selected aggregate source, which was determined from the SMO's records of FC-5 mixture designs. Asphalt binders used for Experiment #1 was a PG 76-22 from NuStar Asphalt (Savannah, GA) and ARB-12 from Blacklidge Emulsions (Pensacola, FL). Two asphalt binder contents were selected for evaluation; one that corresponded to the selected JMF (corresponding to the selected aggregate) and the other was 0.6 percent above the JMF binder content. The increase of 0.6 percent was selected to correspond to the current FDOT construction tolerance. Finally, two asphalt mixture conditioning times were assessed: No STOA and 2 hours STOA (volumetric conditioning) in accordance with AASHTO R 30.

Table 5.1 - Factor - Level Combinations for Experiment 1

Factor	Levels
Aggregate	4 Aggregates selected on properties, JMFs and Performance
Gradation	1 Gradation for each Aggregate (respective JMF)
Binder	2 Binders: PG 76-22, ARB-12
Binder Content	2 Binder Contents: JMF and 0.6% above
STOA	2 STOA: 0 hours and 2 hours

5.1 TEST RESULTS

5.1.1 Aggregate Gradation and Job Mix Formula

The aggregate gradations used for Experiment 1 are shown in Figure 5.1 and Table 5.2. The FC-5 gradations shown in the figure and table represent an average mixture gradation found in the FDOT mix design database. As shown in Table 5.2, two Oolitic Limestone and two Granite aggregate sources were utilized in Experiment 1.

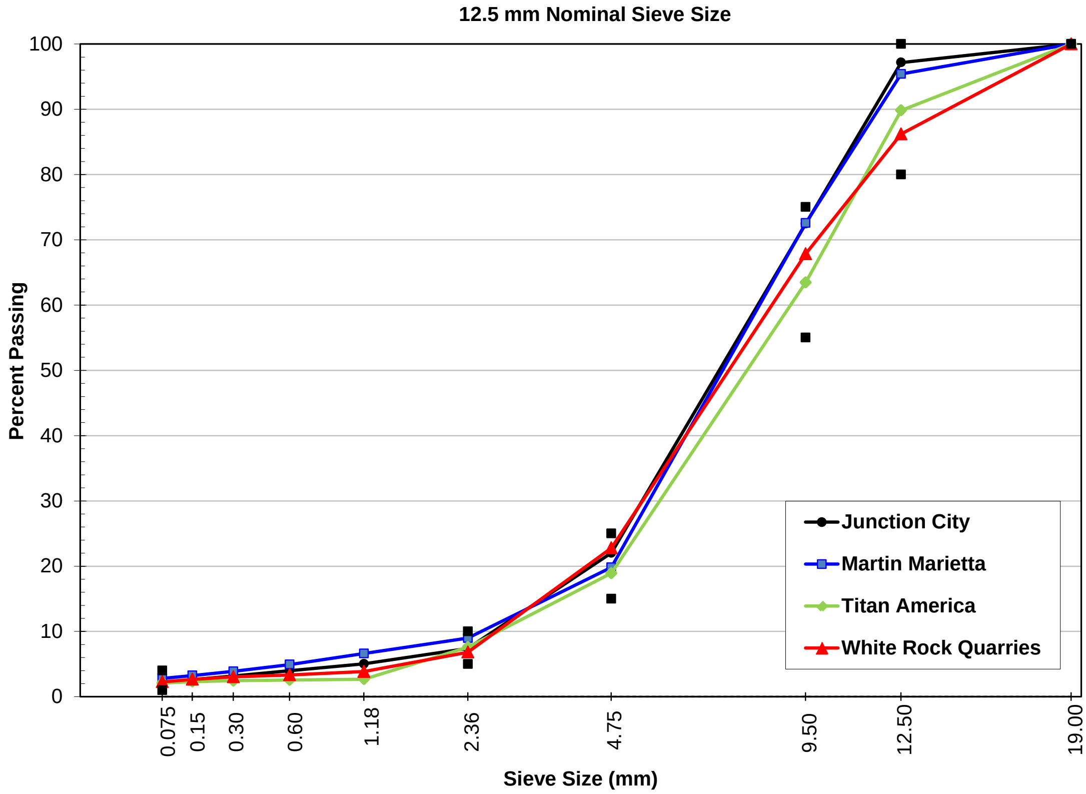


Figure 5.1 - Gradation Used for FC-5 Research Study

Table 5.2 – Gradation Band and Asphalt Content Used in FC-5 Study

Sieve Size (mm)	FDOT FC-5 Specification	Junction City	Martin Marietta	Titan America	White Rock Quarry
3/4" (19.0 mm)	100	100.0	100.0	100.0	100.0
1/2" (12.5 mm)	80 - 100	97.2	95.4	89.8	86.2
3/8" (9.5 mm)	55 - 75	72.5	72.6	63.5	67.8
# 4 (4.75 mm)	15 - 25	22.0	19.8	18.9	22.8
# 8 (2.36 mm)	5 - 10	7.4	9.0	7.6	6.8
# 16 (1.18 mm)		5.0	6.6	2.7	3.9
# 30 (600 µm)		4.0	4.9	2.6	3.3
# 50 (300 µm)		3.2	3.9	2.5	3.1
# 100 (150 µm)		2.6	3.2	2.3	2.7
#200 (75 µm)	1 - 4	2.1	2.8	2.1	2.3
Optimum Asphalt Content (%)		5.9	5.7	7.0	6.6
Primary Mineralogy		Granite	Granite	Oolitic Limestone	Oolitic Limestone

5.1.2 Draindown Test Results

Draindown testing of the FC-5 mixtures was conducted in accordance with AASHTO T 305, *Determination of Draindown Characteristics of Uncompacted Asphalt Mixtures*. The draindown testing was conducted at a test temperature of 345°F, which is approximately 25°F higher than the typical production temperature for the FC-5 mixtures. The draindown testing was conducted using the following test variables:

- Conditioning: 0 and 2 Hour STOA
- Asphalt Binder: PG 76-22 and ARB-12
- Asphalt Contents: Optimum (Opt) and 0.6% Above Optimum (+ Opt)

The 2 Hour STOA conditioning was conducted in accordance with AASHTO R 30, *Mixture Conditioning of Hot Mix Asphalt (HMA).* However, it should be noted that when conducting the 2 hour STOA conditioning of the FC-5 mixtures, the conditioning was actually conducted in the draindown baskets and not a shallow, metal pan. The research team was apprehensive about conditioning in a pan as initial testing showed residual asphalt binder on the pan even after "buttering" the pan prior. The residual asphalt binder would clearly influence the draindown testing, and therefore, the conditioning procedure was modified to eliminate this bias. Detailed test results regarding the influence of the conditioning process is discussed later in this section.

<u>Influence of Asphalt Binder Type and Content on Draindown Testing</u>

FC-5 mixtures were blended with a PG 76-22 and ARB-12 asphalt binder to assess the influence of the asphalt binder type on the draindown characteristics of the mixtures. The PG 76-22 and ARB-12 asphalt binders were selected since they were the typical asphalt binders used to construct the FC-5 mixtures in the field. Tables 5.3 and 5.4 show the test results for the 0 and 2 hour STOA conditioning, respectively.

Table 5.3 – Draindown Test Results (AASHTO T305) for 0 Hour STOA Conditioning

Draindown Test Results (AASHTO T305) Zero Hour STOA Conditioning			
Aggregate Source	Asphalt Content	PG76-22 Asphalt Binder (%)	ARB-12 Asphalt Binder (%)
Martin Marietta	Opt	0.21	0.18
	+Opt	0.19	0.32
Junction City Mines	Opt	0.31	0.22
	+Opt	0.30	0.19
White Rock Quarries	Opt	0.17	0.26
	+Opt	0.18	0.16
Titan America	Opt	0.48	0.18
	+Opt	0.31	0.19

Table 5.4 – Draindown Test Results (AASHTO T305) for 2 Hour STOA Conditioning

Draindown Test Results (AASHTO T305) 2 Hour STOA Conditioning			
Aggregate Source	Asphalt Content	PG76-22 Asphalt Binder (%)	ARB-12 Asphalt Binder (%)
Martin Marietta	Opt	0.44	0.27
	+Opt	0.48	0.24
Junction City Mines	Opt	0.43	0.25
	+Opt	0.49	0.26
White Rock	Opt	0.61	0.29
	+Opt	0.56	0.26
Titan America	Opt	0.34	0.23
	+Opt	0.51	0.26

The test results in Table 5.3 show that for a majority of the draindown testing, the ARB-12 asphalt binder achieved a lower percentage of draindown than the PG 76-22. The test results in Table 5.3 also shows that there does not appear to be a detrimental effect on the draindown performance when the asphalt content is increased 0.6% above the determined optimum asphalt content.

The draindown test results for the 2 hour STOA conditioning is shown in Table 5.4. Similar to the zero hour conditioning, the ARB-12 asphalt binder appears to provide a better resistance to draindown than the PG 76-22. Similar to the results in Table 5.3, the addition of 0.6% more asphalt binder did not appear to be detrimental to the draindown performance.

Influence of Conditioning Time on Draindown Performance

The FC-5 mixtures were exposed to two different conditioning periods to evaluate if volumetric conditioning, which would allow asphalt binder absorption into the aggregates, would influence the draindown performance of the mixtures. Unfortunately, the conditioning period was found to significantly influence the final results of the draindown test. Figures 5.2 and 5.3 show the draindown test results for the PG 76-22 and ARB-12 asphalt binders for the 2 hour STOA conditioning, respectively. The figures were formatted to show two phases of draindown. First, the draindown that occurs during the conditioning phase itself, and second, the draindown that occurs during the test procedure immediately after the conditioning sequence. The test results show that a significant amount of draindown occurs during the 2 hour STOA conditioning that highly biases the final test results. In almost all cases, the draindown taking place during the 2 hour STOA conditioning was 2 to 3 times more than the draindown actually taking place during the draindown test itself (1 hour period after the conditioning time). Therefore, due to the complexities in appropriately conditioning the asphalt mixtures where asphalt binder from the mixture is not lost, it is not recommended to utilize the STOA procedure prior to any test that requires an assessment of asphalt binder loss (i.e. – Draindown or Pie-Plate).

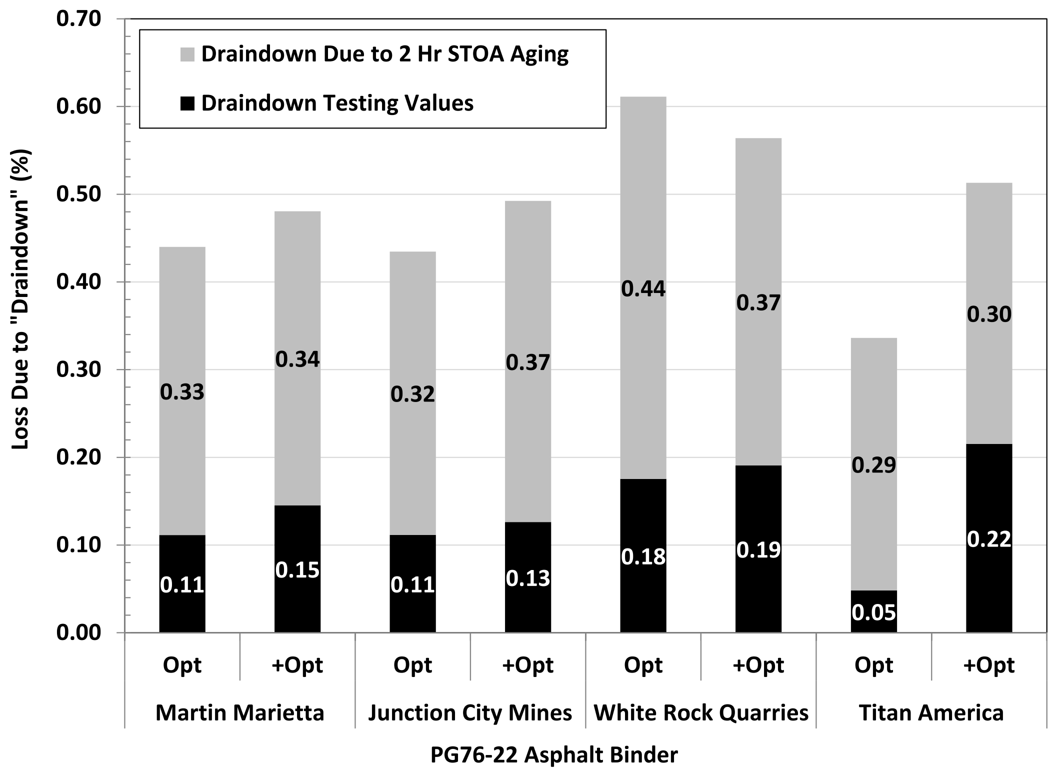


Figure 5.2 - Draindown Test Results for 2 Hour STOA Conditioning – PG 76-22 Asphalt Binder

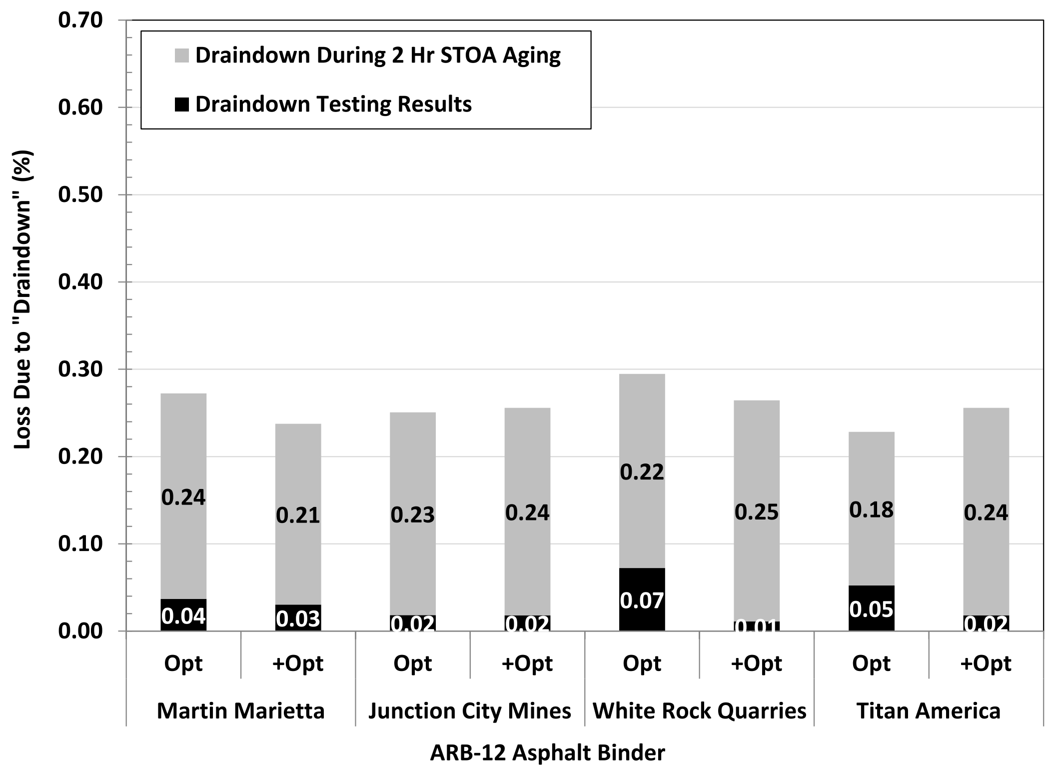


Figure 5.3 - Draindown Test Results for 2 Hour STOA Conditioning – ARB-12 Asphalt Binder

5.1.3 Cantabro Abrasion Loss Test Results

Gyratory compacted FC-5 mixtures were evaluated for their respective durability using the Cantabro Abrasion Loss test. Cooley et al, (2009) indicated that the Cantabro Abrasion loss test was the most common durability test for OGFC's worldwide. Additionally, the Cantabro Abrasion Loss testing may identify aggregate sources that are more prone to durability problems. The Cantabro Abrasion Loss test specimens were compacted using 50 gyrations of the Superpave gyratory compactor.

The Cantabro Abrasion Loss was determined on the FC-5 mixtures containing both the PG 76-22 and ARB-12 asphalt binders at two asphalt contents; optimum and optimum + 0.6%. Additionally, the influence of volumetric conditioning was also evaluated using two conditioning times; zero and 2 hour STOA conditioning.

The test results for the Cantabro Abrasion Loss are shown in Figures 5.4 and 5.5 for the PG 76-22 and ARB-12 asphalt binders, respectively. For the PG 76-22 asphalt binder test results, the following observations can be made:

- Volumetric conditioning does not appear to impact the abrasion loss, either positively or negatively. Only 1 mixture showed to be a statistically detrimentally affected by the conditioning time (Titan America). In almost all cases, there were no statistical differences between the 0 hour and 2 hour STOA conditioned samples.
- In most cases, the addition of 0.6% asphalt binder helped to improve the Cantabro Abrasion Loss. For the 0 hour specimens, it was determined that the Abrasion Loss decreased by approximately 3% when increasing the asphalt content 0.6% above optimum. Little difference was found in the 2 hour STOA conditioned specimens when increasing the asphalt content 0.6% above optimum.

The Cantabro Abrasion Loss test results for the ARB-12 asphalt binder shows an improvement in the abrasion loss properties when compared to the PG 76-22 asphalt binder. In fact, there was a 2.1% reduction and 2.7% reduction in the abrasion loss in the ARB-12 asphalt binder for the 0 hour and 2 hour STOA conditioned specimens, respectively, when compared to the PG 76-22 asphalt binder. The test results for the ARB-12 asphalt binder mixtures are shown in Figure 5.5. Additionally, the following observations can be made regarding the ARB-12 asphalt binder FC-5 mixtures:

- Similar to the PG 76-22 asphalt binder FC-5 mixtures, the addition of 0.6% asphalt binder helped to reduce the abrasion loss, but the improvement was relatively low; 1.7% and 0.4% on average for the 0 hour and 2 hour STOA conditioned specimens, respectively. However, when taking into consideration the relatively low abrasion loss values, these "small" improvements are significant.
- Again, similar to the PG 76-22 asphalt binder FC-5 mixtures, the conditioning of the ARB-12 asphalt binder FC-5 mixtures had negligible effect on the abrasion loss results. For some mixtures, there was an improvement with conditioning time and in some mixtures there was additional abrasion loss.

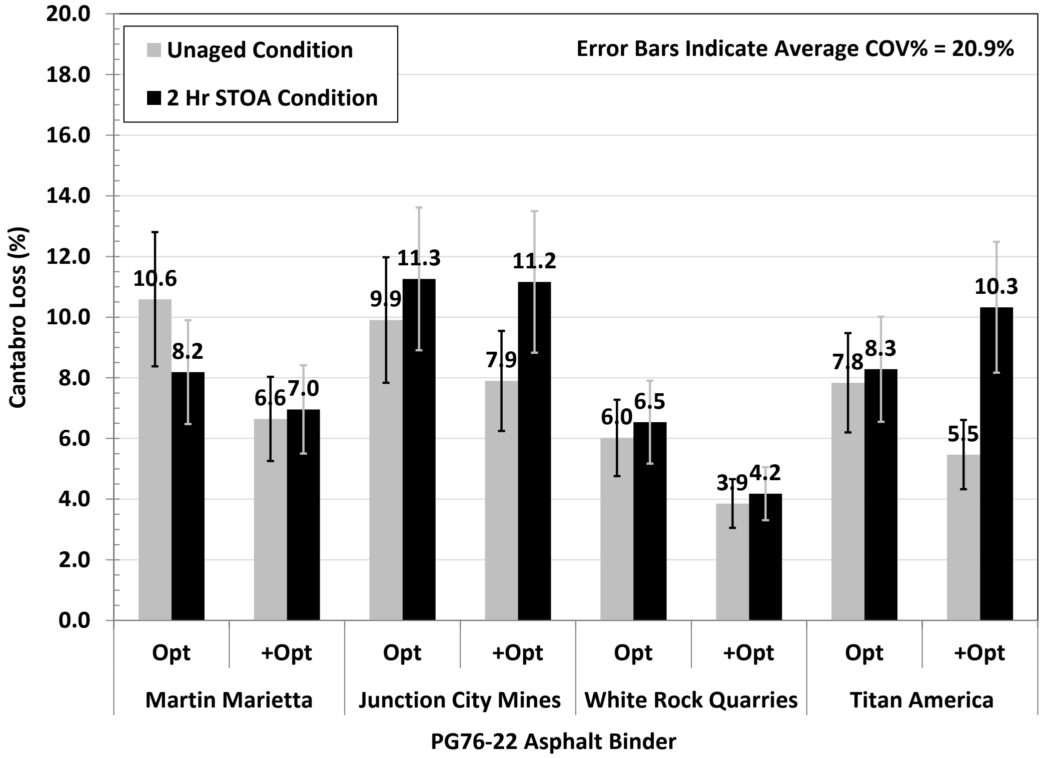


Figure 5.4 - Cantabro Abrasion Loss Results for FC-5 Mixtures with PG 76-22 Asphalt Binder

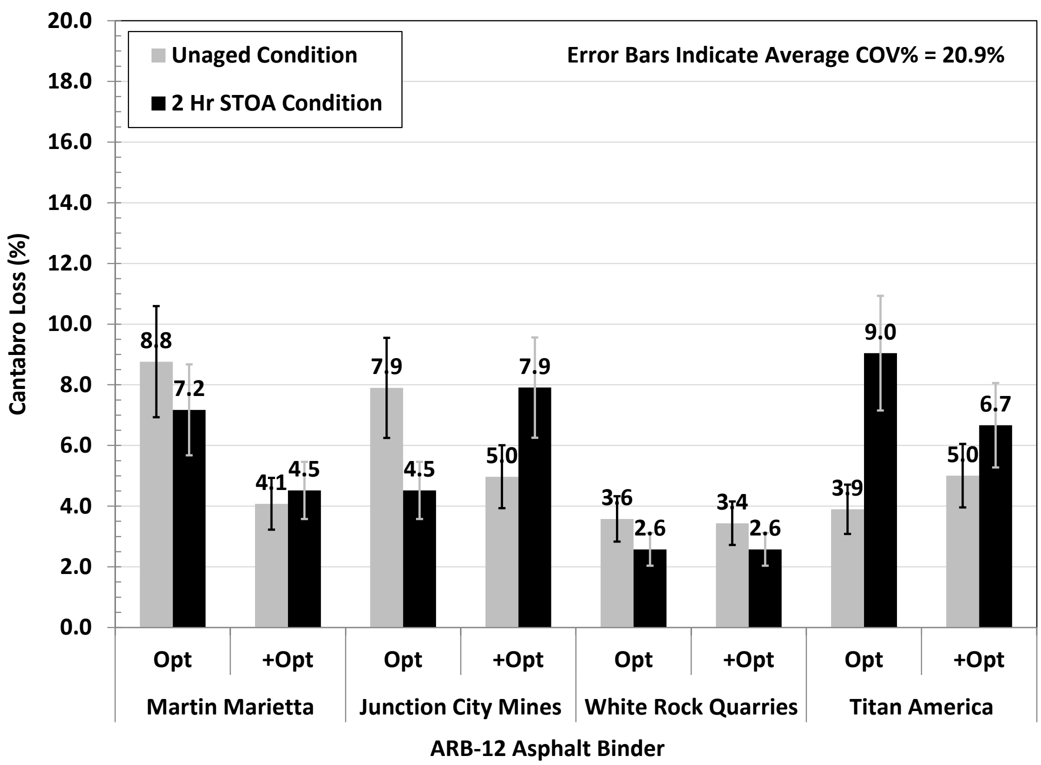


Figure 5.5 - Cantabro Abrasion Loss Results for FC-5 Mixtures with ARB-12 Asphalt Binder Pie-Plate Test Results

The pie-plate test was conducted for the FC-5 mixtures in accordance with FDOT test procedures. Dr. Allen Cooley, who conducted the pie-plate testing, had undergone training at the FDOT State Materials laboratory regarding the proper classification of the pie-plate test results.

The pie-plate testing included three asphalt binders; 1) PG 67-22; 2) PG 76-22; and 3) ARB-12. The asphalt mixtures were produced 325^{o}F. STOA conditioning was not conducted during the Pie-Plate testing to avoid the issues discussed earlier.

The pie-plate test observations are shown in Table 5.5 and photos of some of the Pie-Plate test results are shown in figures at the end of this chapter. The same asphalt binder content (the average optimum asphalt content averaged from the respective JMF's) was used for all mixtures evaluated. The observations made during the Pie-Plate testing show:

- PG 67-22 asphalt binder
    - For all four mixtures at optimum asphalt content, *Sufficient* to *Borderline Sufficient/Excessive* asphalt content was observed. This would be consistent with what would be expected considering this was the asphalt content used for these mixtures.
    - For all four mixtures at "+ Opt", *Excessive* Bonding/Drainage was observed.
- PG 76-22 asphalt binder
    - For all four mixtures at optimum asphalt content, *Sufficient Bonding/Drainage* asphalt content was observed.
    - For all four mixtures at "+ Opt", one mixture was found *Sufficient Bonding/Drainage* (White Rock Quarries), two mixtures were found to have *Borderline Sufficient/Excessive* asphalt content (Martin Marietta and Junction City Mines), and one mixture was observed to have *Excessive Bonding/Drainage* (Titan America). It is interesting to note that both Granite mixtures were found to have borderline conditions.
- ARB-12 asphalt binder
    - All mixtures, except one, were found to have *Sufficient Bonding/Drainage*, regardless of asphalt content (i.e. – optimum or optimum + 0.6%). The one mixture that was not observed to be at *Sufficient Bonding/Drainage* was found to have *Borderline Insufficient/Sufficient* asphalt (Titan America at Optimum Asphalt content, + Opt).

Table 5.5 – Pie-Plate Observations

Pie Plate Evaluations (FM5-558)				
Aggregate Source	Asphalt Content	Pie-Plate Observations		
		PG67-22	PG76-22	ARB-12
Martin Marietta	Opt	***Borderline Sufficient/Excessive***	***Sufficient*** Bonding/Drainage	***Sufficient*** Bonding/Drainage
	+Opt	***Excessive*** Bonding/Drainage	***Borderline Sufficient/Excessive***	***Sufficient*** Bonding/Drainage
Junction City Mines	Opt	***Borderline Sufficient/Excessive***	***Sufficient*** Bonding/Drainage	***Sufficient*** Bonding/Drainage
	+Opt	***Excessive*** Bonding/Drainage	***Borderline Sufficient/Excessive***	***Sufficient*** Bonding/Drainage
White Rock Quarries	Opt	***Sufficient*** Bonding/Drainage	***Sufficient*** Bonding/Drainage	***Sufficient*** Bonding/Drainage
	+Opt	***Excessive*** Bonding/Drainage	***Sufficient*** Bonding/Drainage	***Sufficient*** Bonding/Drainage
Titan America	Opt	***Borderline Sufficient/Excessive***	***Sufficient*** Bonding/Drainage	***Borderline Insufficient/Sufficient***
	+Opt	***Excessive*** Bonding/Drainage	***Excessive*** Bonding/Drainage	***Sufficient*** Bonding/Drainage

Comparisons between the Pie-Plate observations and the Draindown test results for the PG 76-22 and ARB-12 asphalt binders were attempted to see if any trend existed between the two tests. The comparisons are shown in Tables 5.6 and 5.7 for the PG 76-22 and ARB-12 asphalt binders, respectively.

The comparisons appear to indicate that the proposed limit of 0.3% draindown is reasonable when compared to the visual observations of the Pie-Plate test. For the PG 76-22 asphalt binder, there are some fluctuations between results. For example, the Titan America aggregate at optimum asphalt content was shown to have *Sufficient* Bonding/Drainage, but the draindown test results indicated 0.48% loss. Meanwhile, at the Opt+ condition for the Titan America, the pie-plate indicated *Excessive* Bonding/Drainage with the draindown test resulting in only a 0.31% loss.

All pie-plate test results for the ARB-12 asphalt binder showed *Sufficient Bonding/Drainage* or *Borderline Insufficient* conditions with draindown loss values under 0.3%, except for one mixture, Martin Marietta at + Opt.

Table 5.6 – Pie-Plate Observations and Draindown Test Results – PG 76-22 Asphalt Binder

Pie Plate Evaluations (FM 5-558) vs 0 Hr Draindown PG76-22 Asphalt Binder			
Aggregate Source	Asphalt Content	Pie-Plate Observations	Draindown Loss (%)
Martin Marietta	Opt	***Sufficient*** Bonding/Drainage	0.21
	+Opt	***Borderline Sufficient/Excessive***	0.19
Junction City Mines	Opt	***Sufficient*** Bonding/Drainage	0.31
	+Opt	***Borderline Sufficient/Excessive***	0.30
White Rock Quarries	Opt	***Sufficient*** Bonding/Drainage	0.17
	+Opt	***Sufficient*** Bonding/Drainage	0.18
Titan America	Opt	***Sufficient*** Bonding/Drainage	0.48
	+Opt	***Excessive*** Bonding/Drainage	0.31

The variability of the draindown test results, and the sometimes conflicting trend with the Pie-Plate test is most likely a function of the type of draindown physically occurring. In almost all cases, the measured draindown from the test results are due to small mastic/binder coated fine aggregates and not liquid asphalt binder “dripping” off the loose aggregates.

It must be stated that AASHTO T 305 was developed in the late 1990’s during the development of a mix design method for SMA. SMA is a gap-graded mix with a high asphalt binder content and high filler content. During the research on SMA, the mastic was defined as the aggregate fraction passing the break point sieve and asphalt binder. Because of the high filler content (8 to 10 percent), the use of the ¼ in. mesh within the draindown basket was not an issue. The high mortar (filler plus asphalt binder) content would help hold the finer aggregate within the basket. However, with OGFC-type mixtures, there is basically no mortar within the mixture. As such, there is a propensity for the fine aggregates to pass through the ¼ in. mesh (Figure 5.5). The resulting stone loss might indicate an artificially high value for the asphalt binder draindown. This scenario was consistent in all of the FC-5 mixtures evaluated.

Table 5.7 – Pie-Plate Observations and Draindown Test Results – ARB-12 Asphalt Binder

Pie Plate Evaluations (FM 5-558) vs 0 Hr Draindown ARB-12 Asphalt Binder			
Aggregate Source	Asphalt Content	Pie-Plate Observations	Draindown Loss (%)
Martin Marietta	Opt	***Sufficient*** Bonding/Drainage	0.18
	+Opt	***Sufficient*** Bonding/Drainage	0.32
Junction City Mines	Opt	***Sufficient*** Bonding/Drainage	0.22
	+Opt	***Sufficient*** Bonding/Drainage	0.19
White Rock Quarries	Opt	***Sufficient*** Bonding/Drainage	0.26
	+Opt	***Sufficient*** Bonding/Drainage	0.16
Titan America	Opt	***Borderline Insufficient/Sufficient***	0.18
	+Opt	***Sufficient*** Bonding/Drainage	0.19

Figure 5.6 – FC-5 Coated Fine Aggregate “Draindown” During from Draindown Testing (AASHTO T 305)

5.2 OBSERVATIONS FROM EXPERIMENT 1

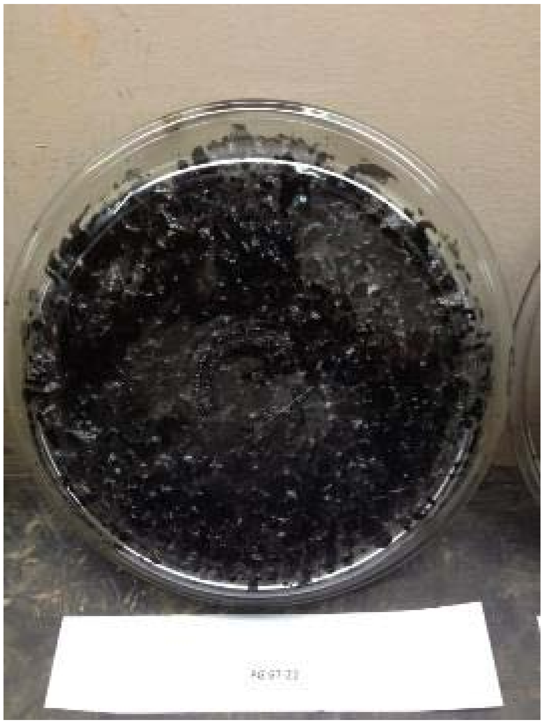
The major premise of experiment 1 was to try to determine if aggregate absorption should be considered during the sample preparation phase prior to the asphalt content determination of FC-5 mixtures. Currently, the FDOT utilizes the Pie-Plate test for the determination of the optimum asphalt content for their FC-5 mixtures. According to the test procedure, the FC-5 mixture is blended and mixed with a PG 67-22 asphalt binder at a temperature of 325^{o}F. Immediately after mixing, the FC-5 mixture is poured into a glass pie-plate. Two immediate questions come to mind when reviewing this procedure:

1. Should asphalt absorption be allowed to occur using a volumetric conditioning time similar to the Superpave Volumetric design procedure? With aggregates of high absorption native to Florida, additional asphalt absorption may be taking place in the field that is not appropriately accounted for during the design phase.
2. A PG 67-22 asphalt binder utilized during the pie-plate test to determine an optimum asphalt content for the FC-5 mixture. Would using the asphalt binder specified for the mixture being placed (i.e. PG76-22 or ARB-12) result in a different optimum asphalt binder content determination?

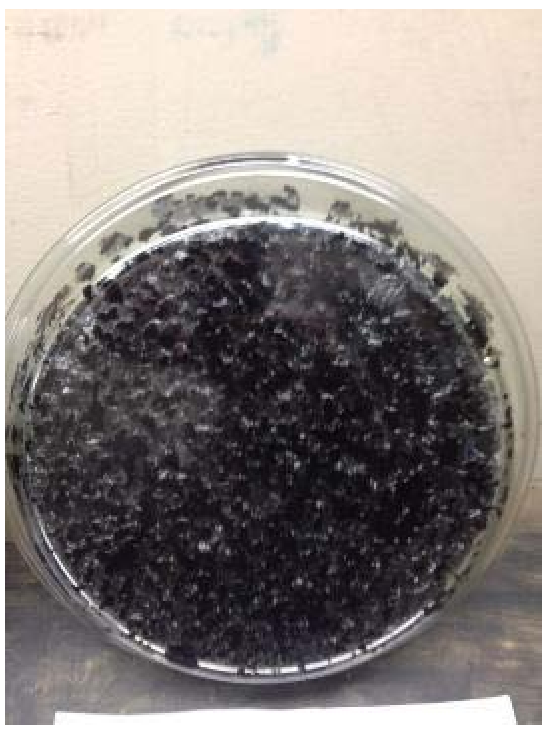
The general work plan developed and conducted in experiment 1 was aimed at answering the above questions. Based on the testing and results found in this phase, the following observations and conclusions can be made:

- STOA proved to be a difficult procedure when conducting prior to draindown testing. Due to residual asphalt binder in the pan, it was determined to condition the loose mix in the draindown baskets. However, it was found that draindown was occurring during the 2 hour conditioning time that was biasing the test results. It should be noted that the draindown was not the classically defined “asphalt binder dripping off the aggregate” draindown. What was being measured was more of mastic/coated fine aggregate that had fallen through the ¼” openings in the mesh basket.
    - *Conclusion:* At this point, due to the issues observed, it is not recommended to utilize a volumetric conditioning phase prior to the draindown and/or pie-plate test procedure. Also, modifications to the draindown basket should be made to decrease the opening size to eliminate mastic falling out of the basket.
- An additional 0.6% asphalt content showed to have minimal to no detrimental effect on the draindown and pie-plate test results for the PG 76-22 and ARB-12 asphalt binders. The additional asphalt binder was also found to help decrease the Cantabro Abrasion Loss, which is an indication that the FC-5 mixture is becoming more durable due to the additional asphalt binder. However, when evaluating the PG 67-22 asphalt binder in the Pie-Plate test, many of the observations for the + Opt condition did indicate *Excessive Bonding/Drainage*.
    - *Conclusion*: In an attempt to achieve a greater effective asphalt content, which would promote greater durability and fatigue resistant FC-5 mixtures in Florida, it may be more beneficial to utilize the asphalt binder specified for that FC-5 mixture. PG 76-22 and ARB-12 asphalt binders have rotational viscosity values 2 to 3 times greater than the PG 67-22 asphalt binder, which results in greater adhesion to the aggregates and less of a chance for draindown-type issues to take place.

- The pie-plate observations are in general agreement with the draindown test results for the PG 76-22 and ARB-12 asphalt binders. It would appear that somewhere in the 0.3 to 0.5% Draindown Loss range, the Pie-Plate observations indicate *Excessive Bonding/Drainage*. Draindown Loss results lower than 0.3% generally resulted in *Sufficient Bonding/Drainage* observations in the Pie-Plate test.



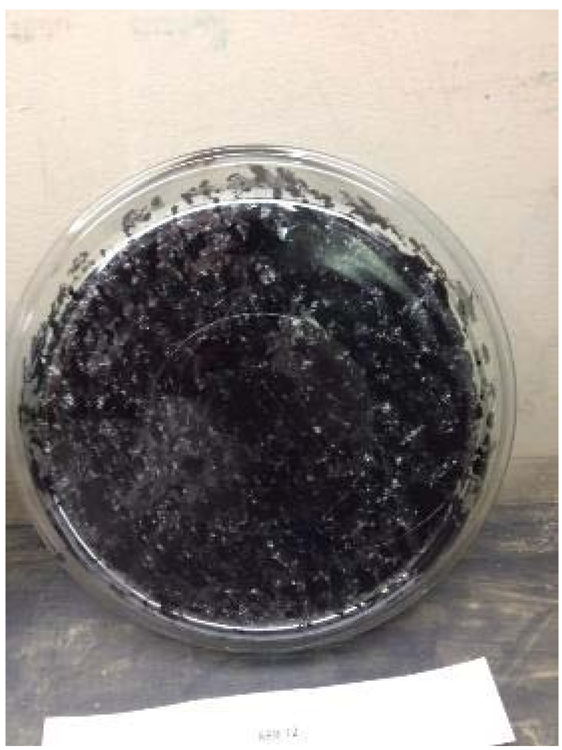
(a) (b)



(c)

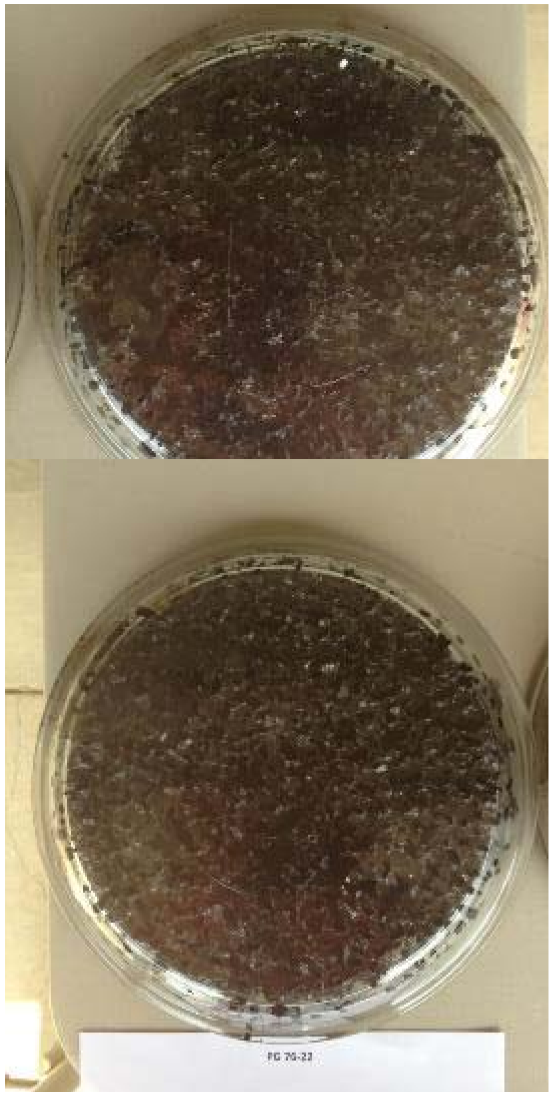
Figure 5.7 - Pie-Plate Test Results for White Rock Quarries FC-5 Mix at Optimum Asphalt Content; a) PG 67-22; b) PG 76-22; c) ARB-12

(a) (b)

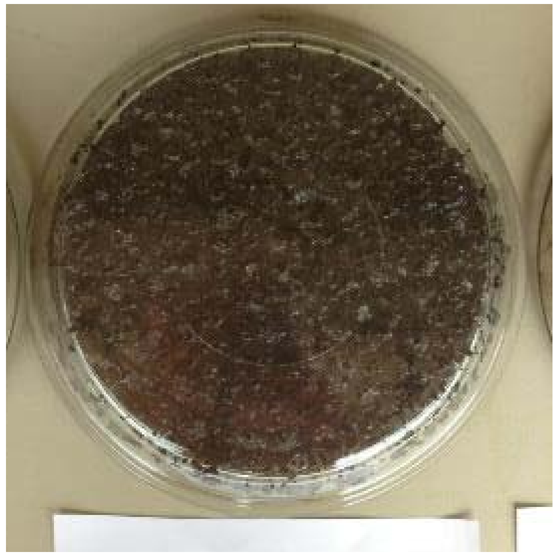


(c)

Figure 5.8 - Pie-Plate Test Results for White Rock Quarries FC-5 Mix at 0.6% Above Optimum Asphalt Content; a) PG 67-22; b) PG 76-22; c) ARB-12

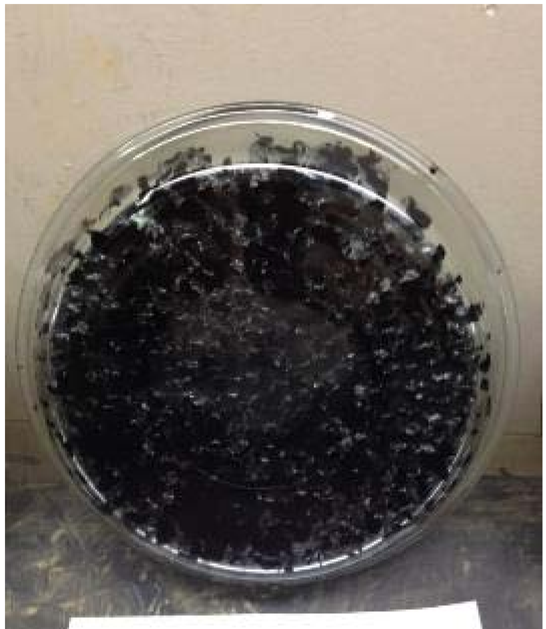


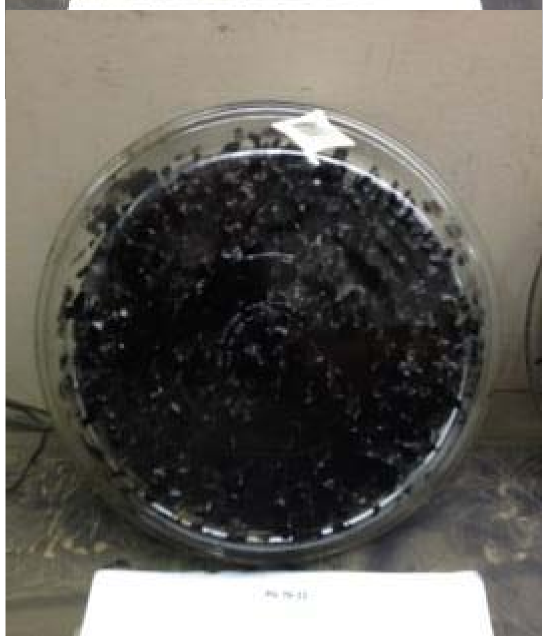
(a) (b)



(c)

Figure 5.9 - Pie-Plate Test Results for Titan America FC-5 Mix at Optimum Asphalt Content; a) PG 67-22; b) PG 76-22; c) ARB-12

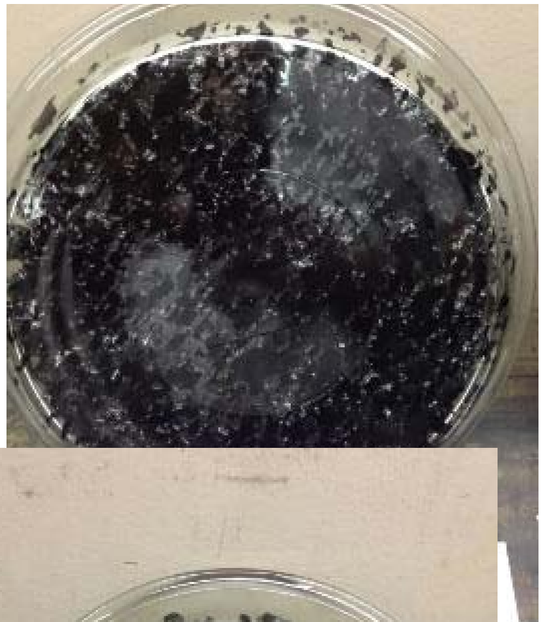




(a) (b)

(c)

Figure 5.10 - Pie-Plate Test Results for Titan America FC-5 Mix at 0.6% Above Optimum Asphalt Content; a) PG 67-22; b) PG 76-22; c) ARB-12



(a) (b)

(c)

Figure 5.11 - Pie-Plate Test Results for Junction City FC-5 Mix at Optimum Asphalt Content; a) PG 67-22; b) PG 76-22; c) ARB-12

CHAPTER 6 – EFFECT OF GRADATION ON THE PERFORMANCE FC-5 MIXTURES (EXPERIMENT 2)

Experiment 2 was developed to look at the difference in FC-5 mixture performance when reducing the nominal maximum aggregate size from FDOT's current 12.5 mm to a proposed 9.5 mm nominal maximum aggregate size. Table 6.1 presents the factor-level combinations evaluated in experiment 2. Two aggregate sources were utilized; Martin Marietta granite and White Rock Quarries Oolitic limestone. These two aggregate sources were selected as they are the two most predominant aggregate sources used for FC-5 mixes in Florida.

Table 6.1 – Factor-Level Combinations for Experiment #2

Factor	Levels
Aggregate	2 Aggregates selected form Experiment 1
Gradation	2 Gradations: FC-5 and 9.5 mm
Binder	2 Binders: PG 76-22, ARB-12
Binder Content	2 Binder Contents: JMF and 0.6% above
STOA	STOA: 2 hours loose at compaction temperature
LTOA	LTOA: 5 days at 85°C

The identical 12.5 mm nominal maximum aggregate size gradation used in Experiment #1 was also used in Experiment #2. The 9.5 mm nominal aggregate size gradations used were based on the recommendations by Cooley et al (2009) and are shown in Table 6.2. The optimum binder content of the two 9.5 mm FC-5 mixtures were determined using the Pie-Plate method in accordance with FDOT 5-588, *Florida Method of Test for Determining the Optimum Asphalt Binder Content of an Open-Graded Friction Course Mixture Using the Pie Plate Method*. The resultant optimum asphalt contents for the respective 9.5 mm FC-5 mixtures are also shown in Table 6.2.

The FC-5 mixtures were short-term and long-term aged in accordance with AASHTO R 30, *Mixture Conditioning of Hot Mix Asphalt (HMA).* In an effort to confine the FC-5 compacted mixtures during the long-term aging, the compacted cylinders were wrapped in "chicken wire" and 6-inch hose clamps (Figure 6.1).

Table 6.2 – 9.5 mm Nominal Maximum Aggregate Size FC-5 Mixtures

Sieve Size	% Passing		NCHRP Report 640 (Cooley et al., 2009)	Previous FDOT FC-2 (No Longer in Use)
	White Rock Quarries	Martin Marietta		
3/4" (19.0 mm)	100.0	100.0		
1/2" (12.5 mm)	100.0	100.0	100	100
3/8" (9.5 mm)	85.4	86.4	85 - 100	85 - 100
# 4 (4.75 mm)	27.5	30.0	20 - 30	10 - 40
# 8 (2.36 mm)	6.7	11.8	5 - 15	
# 16 (1.18 mm)	3.8	7.0		
# 30 (600 µm)	3.1	5.1		
# 50 (300 µm)	2.8	4.0		
# 100 (150 µm)	2.4	3.3		
#200 (75 µm)	2.1	2.8	0 - 4	2 - 5
Optimum AC% for PG 76-22 (FDOT 5-588)	6.0	6.6	N.A.	N.A.
Optimum AC% for ARB-12 (FDOT 5-588)	6.7	7.3	N.A.	N.A.

Figure 6.1 – FC-5 Specimen Confined with "Chicken Wire" Prior to Long-Term Aging

6.1 Performance of 9.5 and 12.5 mm Nominal Aggregate Size FC-5 Mixtures – Short Term Aged

The 12.5 mm nominal maximum aggregate size (NMAS), evaluated in Experiment #1, were further evaluated in Experiment #2. The durability and cracking potential of the FC-5 mixtures were assessed using the Cantabro Abrasion test and the Overlay Tester, respectively. The Indirect Tensile (IDT) Strength test, conducted at 10^{o}C, was also performed based on the experience of previous FDOT studies (Roque et al., 2009). Meanwhile, the rutting performance of the FC-5 mixtures was evaluated using the wet HWTD test (AASHTO T 324).

6.1.1 Cantabro Abrasion Test Results

The Cantabro Abrasion test results for the White Rock Quarries (WRQ) and Martin Marietta (MM) 12.5 mm FC-5 mixtures were shown previously in Chapter 5, are again shown in Figure 6.2 for only the WRQ and MM mixtures. Overall, the WRQ mixtures resulted in lower Cantabro Abrasion Loss than the MM FC-5 mixtures. The test results also indicate that as the asphalt content increases, in this case 0.6%, the durability of the FC-5 mixture, as determined by the Cantabro Abrasion Loss test, improves.

Figure 6.3 shows the test results for the 9.5 mm FC-5 mixtures produced using the WRQ and MM aggregates. The test results show that a reduction in the abrasion loss is found when reducing the NMAS from 12.5 to 9.5 for the MM aggregate source. However, mixed results were for the WRQ aggregate. Reduction in the abrasion loss was found for the PG 76-22 asphalt binder, while an increase in the abrasion loss was found for the ARB-12 asphalt binder. In both 9.5 mm FC-5 mixtures, as the asphalt content increased, the Cantabro Abrasion Loss decreased. Figure 6.4 provides a direct comparison between the 9.5 and 12.5 mm FC-5 mixtures.

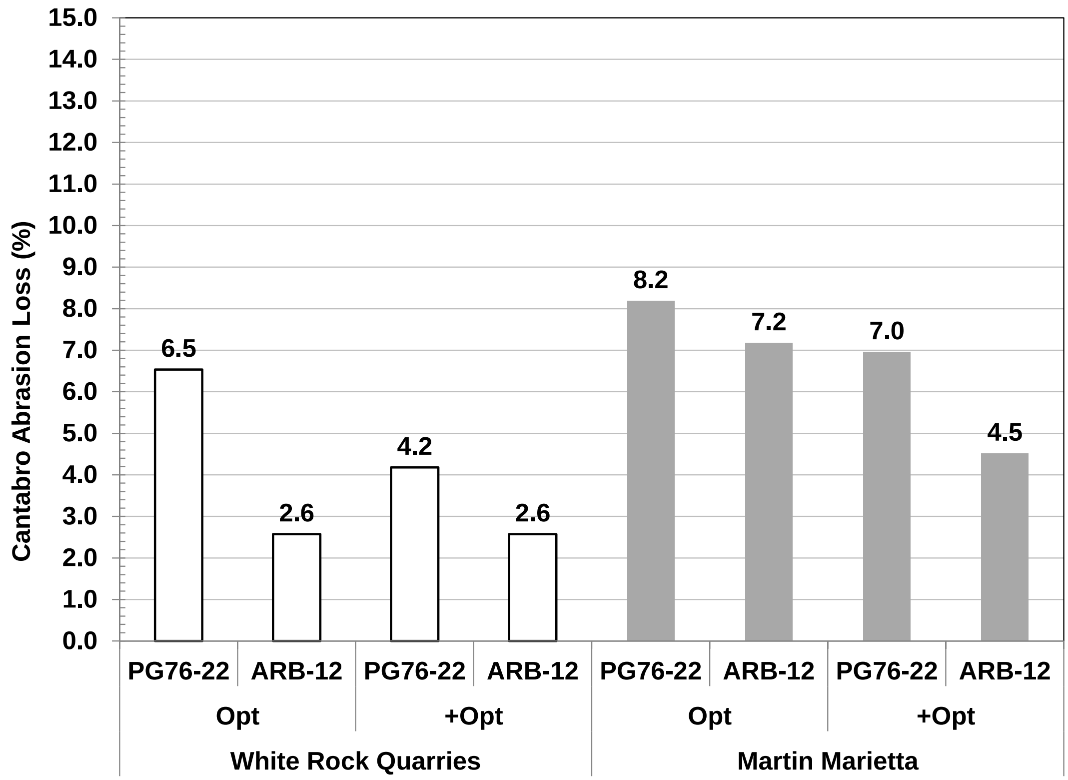


Figure 6.2 – Cantabro Abrasion Loss Results for 12.5 mm FC-5 Mixtures

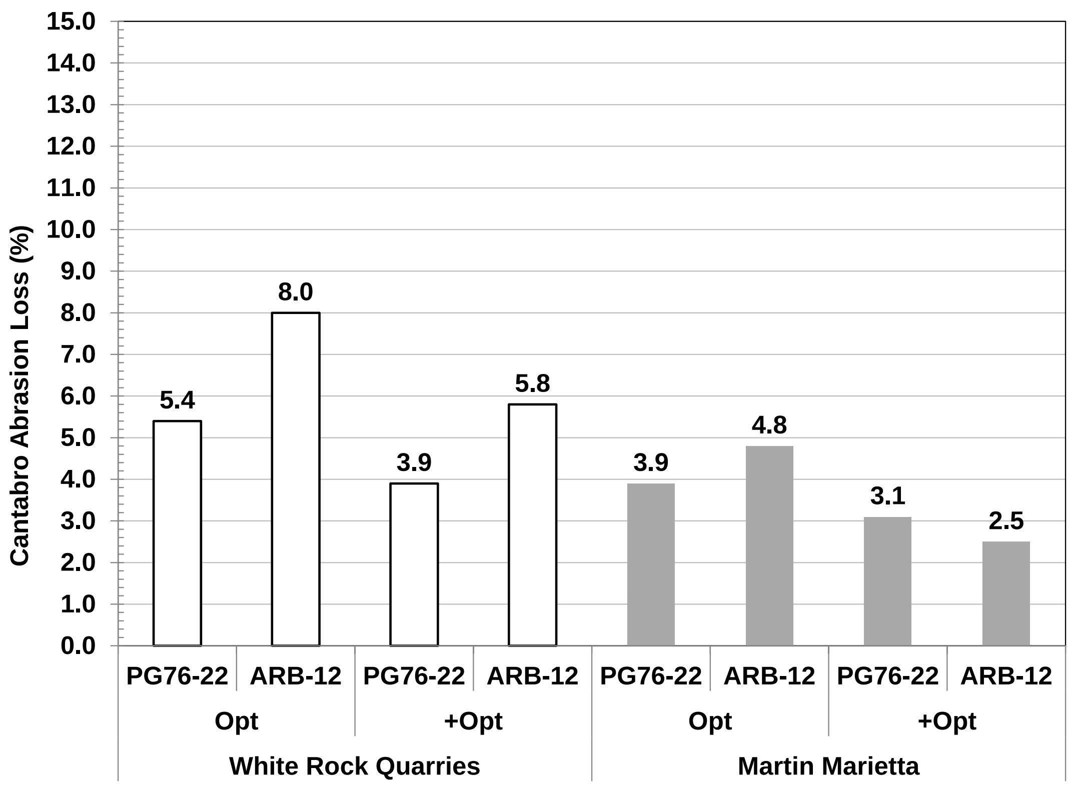


Figure 6.3 – Cantabro Abrasion Loss Results for 9.5 mm FC-5 Mixtures

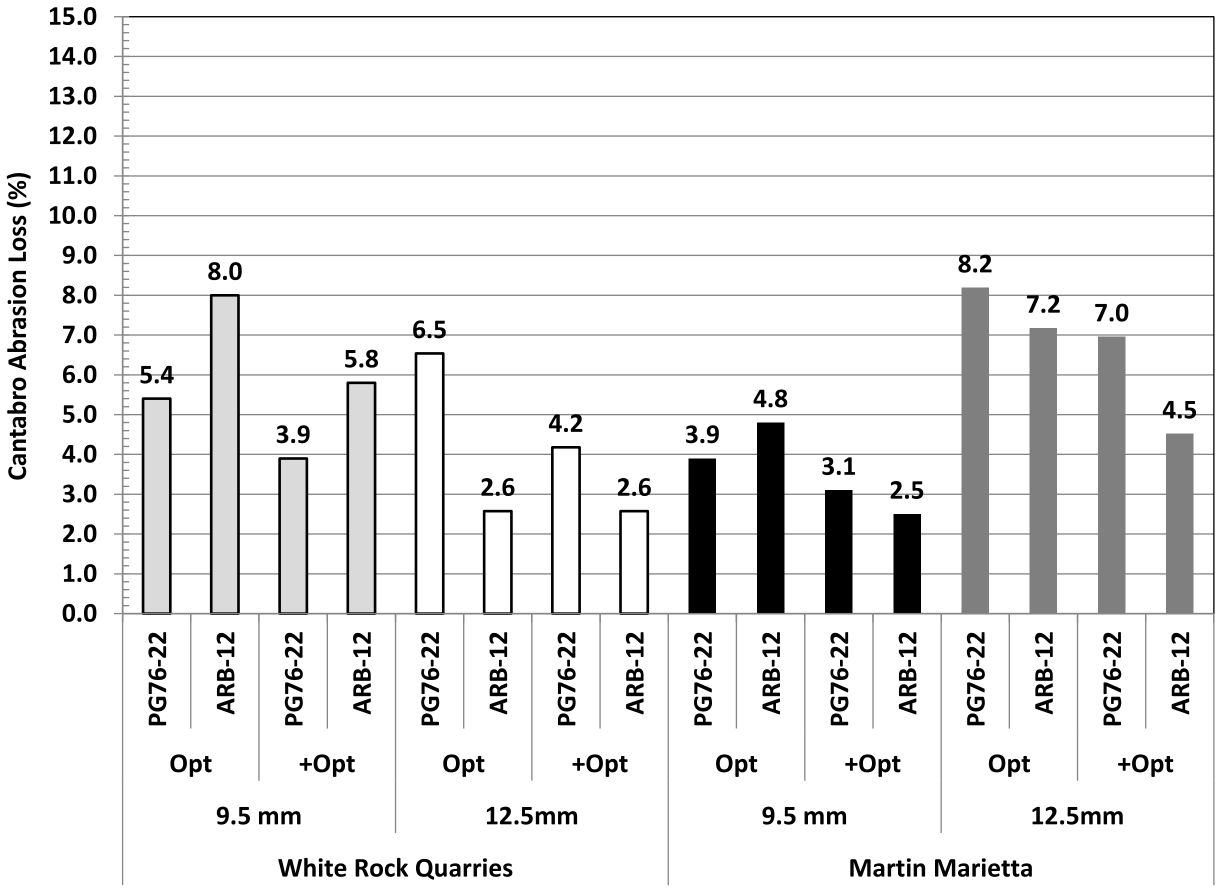


Figure 6.4 – Cantabro Abrasion Loss Comparison – 9.5 and 12.5 mm FC-5 Mixtures

6.1.2 Overlay Tester Results

The Overlay Tester, described by Zhou and Scullion (2005), has shown to provide an excellent correlation to field cracking for both composite pavements (Zhou and Scullion, 2005; Bennert et al., 2009) as well as flexible pavements (Zhou et al., 2007; Bennert and Maher, 2013). The Overlay Tester utilizes a pre-determined location for a crack to initiate. Then, due to the horizontal deformation (tensile strain) applied to the specimen; the crack propagates from the bottom of the specimen to the surface.

Figure 6.5 shows a picture of the Overlay Tester used in this study. Sample preparation and test parameters used in this study followed that of TxDOT TEX-248F, *Overlay Test for Determining Crack Resistance of HMA*. These included:

- 25^oC (77^oF) test temperature;
- Opening width of 0.025 inches;
- Cycle time of 10 seconds (5 seconds loading, 5 seconds unloading); and
- Specimen failure defined as 93% reduction in initial load.

Figure 6.5 – Overlay Tester Device Used for Fatigue Cracking Assessment

Five test specimens were tested for each mixture combination evaluated. The high and low values were eliminated and only the middle three values were averaged and reported (i.e. Trimmed Mean method). Note the error bars associated in the following figures indicate one standard deviation above and below the average.

The Overlay Tester fatigue cracking results for the 12.5 mm FC-5 mixtures are shown in Figure 6.6. The test results clearly indicate that the MM FC-5 mixture is far superior with respect to resisting crack propagation than the WRQ mixture. The results in Figure 6.6 also show that the addition 0.6% of asphalt binder either increased or had no detrimental effect on the Overlay Tester fatigue cracking results. This is expected as it is well known mixtures with higher effective asphalt contents generally achieve better cracking resistance.

The Overlay Tester results for the 9.5 mm FC-5 mixtures are shown in Figure 6.7. The test results indicate that there tends to be a general increase in Overlay Tester results when utilizing a smaller nominal maximum aggregate size FC-5 mixture, even though the same aggregate source was used. This was especially true for the PG 76-22 asphalt binder mixtures. A slight decrease in Overlay Tester results was found for the Martin Marietta 9.5 mm FC-5 mixtures when using the ARB-12 asphalt binder.

Figures 6.8 and 6.9 show a direct comparison between the 9.5 and 12.5 mm NMAS FC-5 mixtures for the WRQ and MM aggregates, respectively.

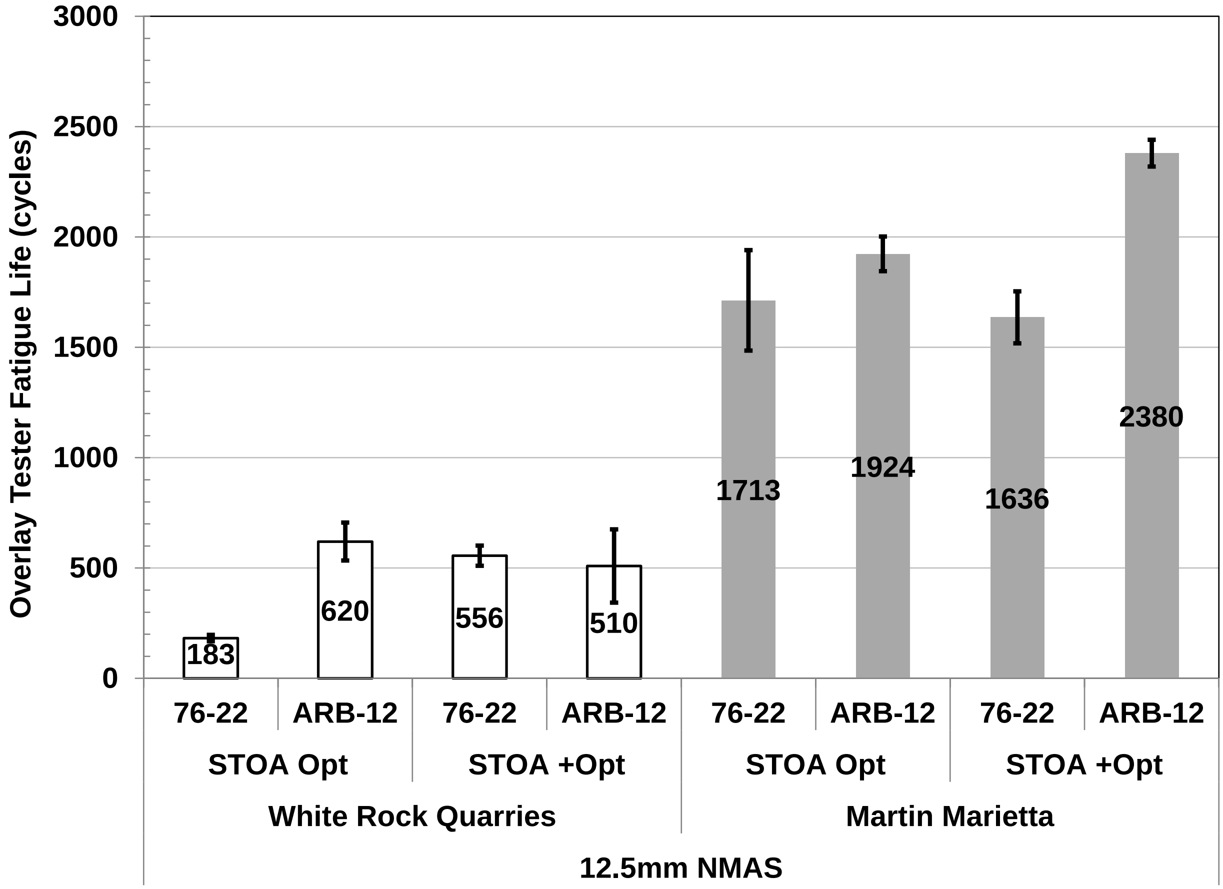


Figure 6.6 – Overlay Tester Results for 12.5 mm FC-5 Mixtures – Short Term Aged

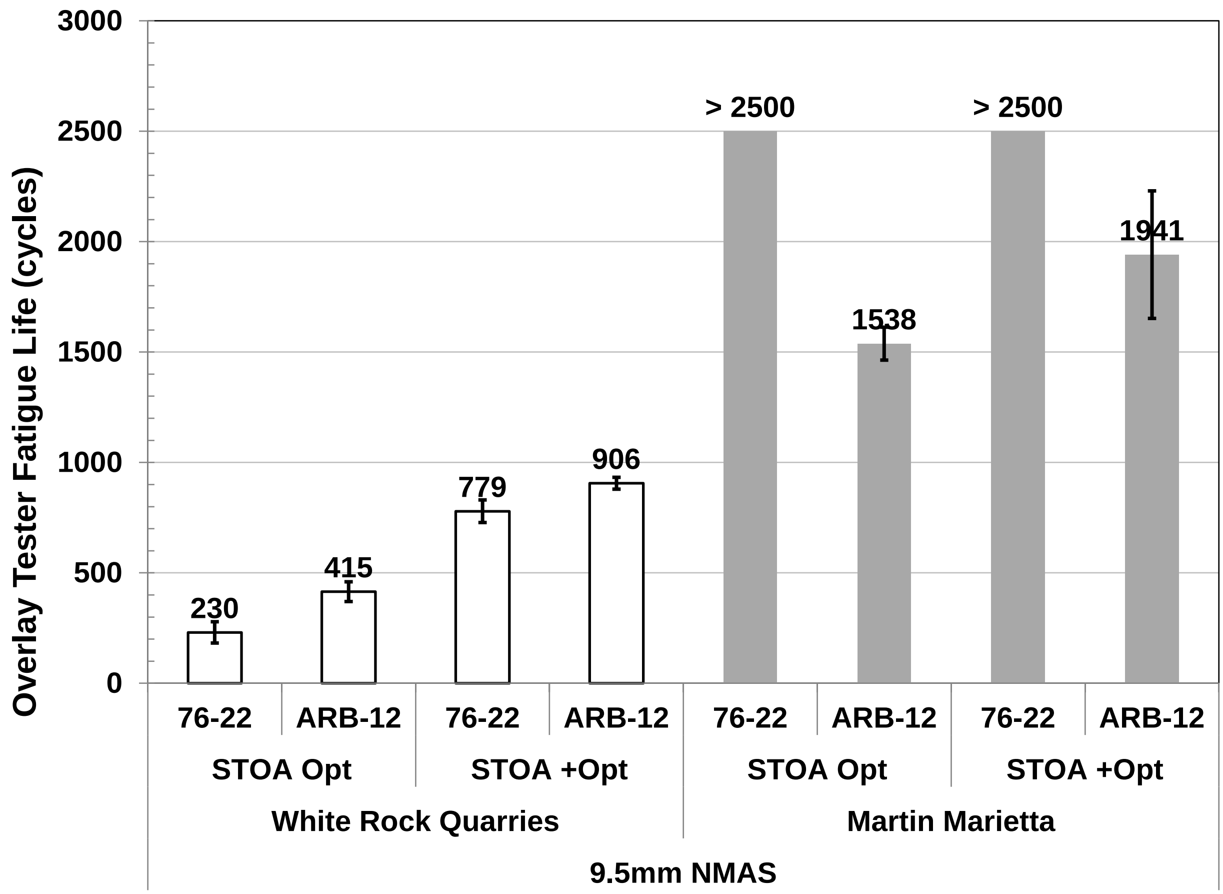


Figure 6.7 – Overlay Tester for 9.5 mm FC-5 Mixtures – Short Term Aged

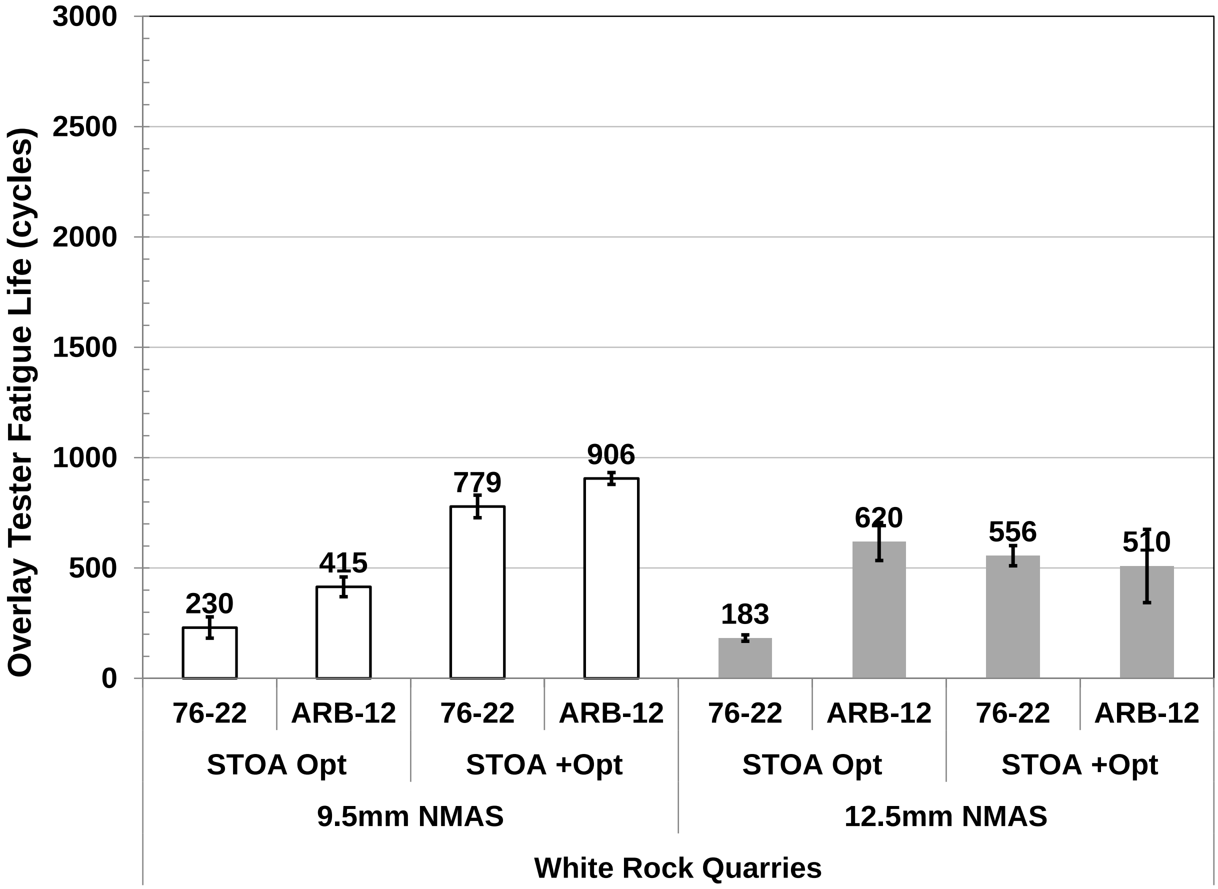


Figure 6.8 – Overlay Tester Results for 9.5 and 12.5 mm NMAS FC-5 Mixtures – White Rock Quarries Short-Term Aged

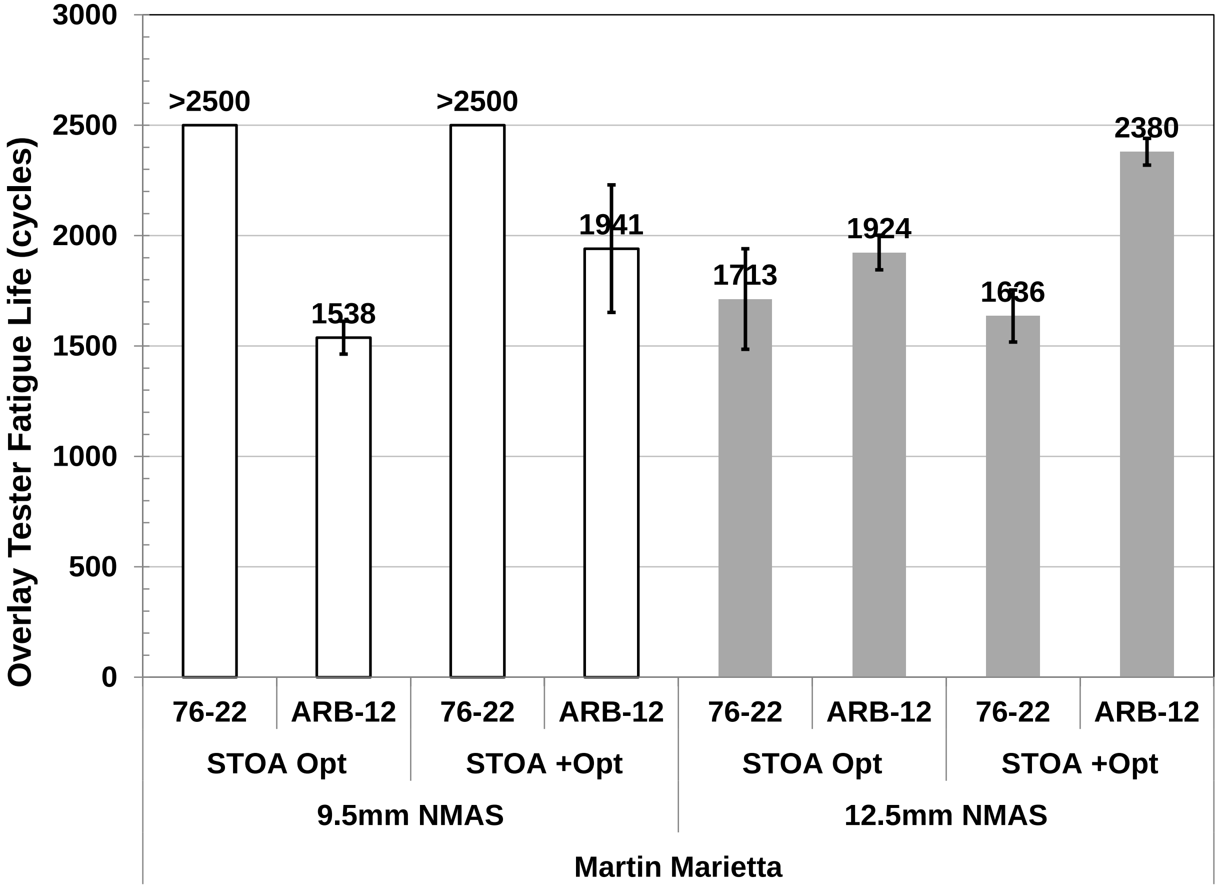


Figure 6.9 – Overlay Tester Results for 9.5 and 12.5 mm NMAS FC-5 Mixtures – Martin Marietta Short-Term Aged

6.1.3 Indirect Tensile Strength Test (IDT)

Because tensile strength is important in the development of surface initiated cracks, the indirect tensile strength test was conducted for the various FC-5 mixtures. The Indirect Tensile Strength (IDT) was determined for the asphalt mixtures at a loading rate of 25 mm (1 inch) per minute (AASHTO T 283) and test temperature of 10^{o} C. Roque et al (2009) appeared to differentiate FC-5 mixtures better at this temperature than other temperatures. The IDT test was conducted by using a 95 mm tall gyratory compacted specimen in the Marshall Compression machine with a modified Lottman fixture. No on-specimen LVDT's or extensometers were used to record deformation or strain, only the cross head deformation was used.

Three specimens were tested in triplicate and averaged for reporting purposes. Along with the IDT strength, the fracture energy (FE) was also determined by calculating the area under the stress-deformation curve up to the maximum IDT strength (Figure 6.10). This approach is not a "classical" fracture energy analyses (Jacobs et al., 1996; Chang et al., 2002; Roque et al., 2004), but more of a simplified approach that possibly could be incorporated at the asphalt plant and not require sophisticated equipment. A 6^{th}-Order polynomial is fitted through the stress-deformation curve and integrated over the boundaries (X_1 and X_2) to determine the area under the curve.

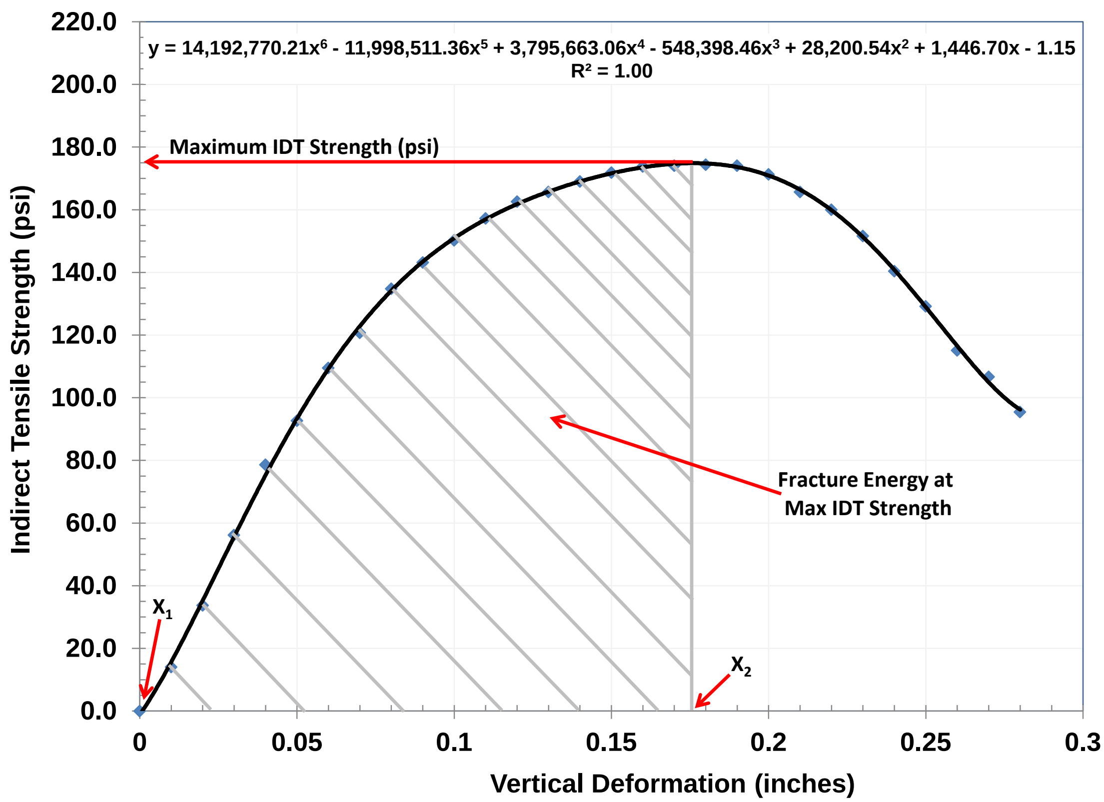


Figure 6.10 – Schematic of Determining FE at Maximum IDT Strength

The IDT strength for the 9.5 and 12.5 mm NMAS FC-5 mixtures are shown in Figures 6.11 and 6.12. The test results for the 12.5 mm NMAS show that the WRQ mixtures achieved a slightly higher IDT strength than the MM mixtures for the same asphalt binder type and asphalt content condition (i.e. Opt or +Opt). However, for the 9.5 mm NMAS mixtures, the IDT strength of the two different aggregate sources were very similar for the same binder type and asphalt content condition.

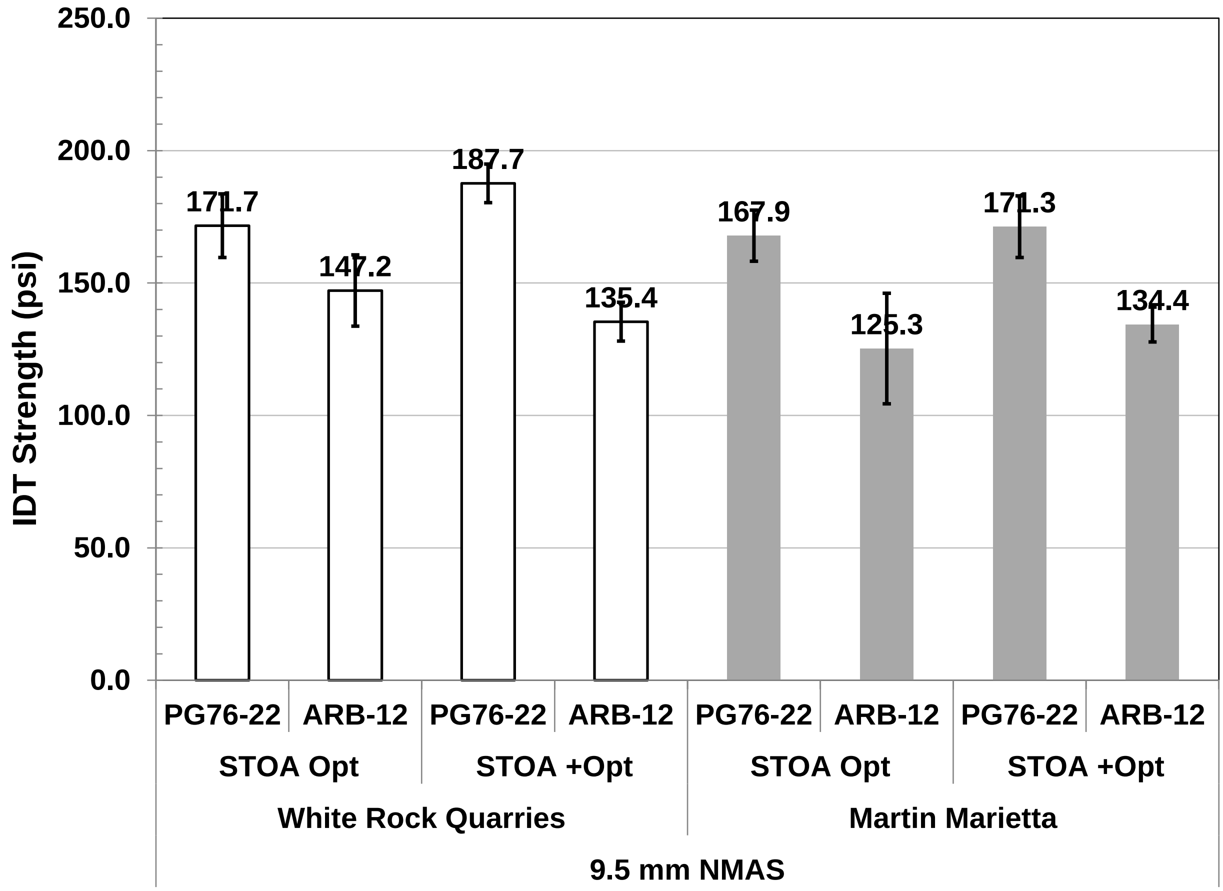


Figure 6.11 – IDT Strength for 9.5 mm NMAS FC-5 Mixtures – Short-Term Aged

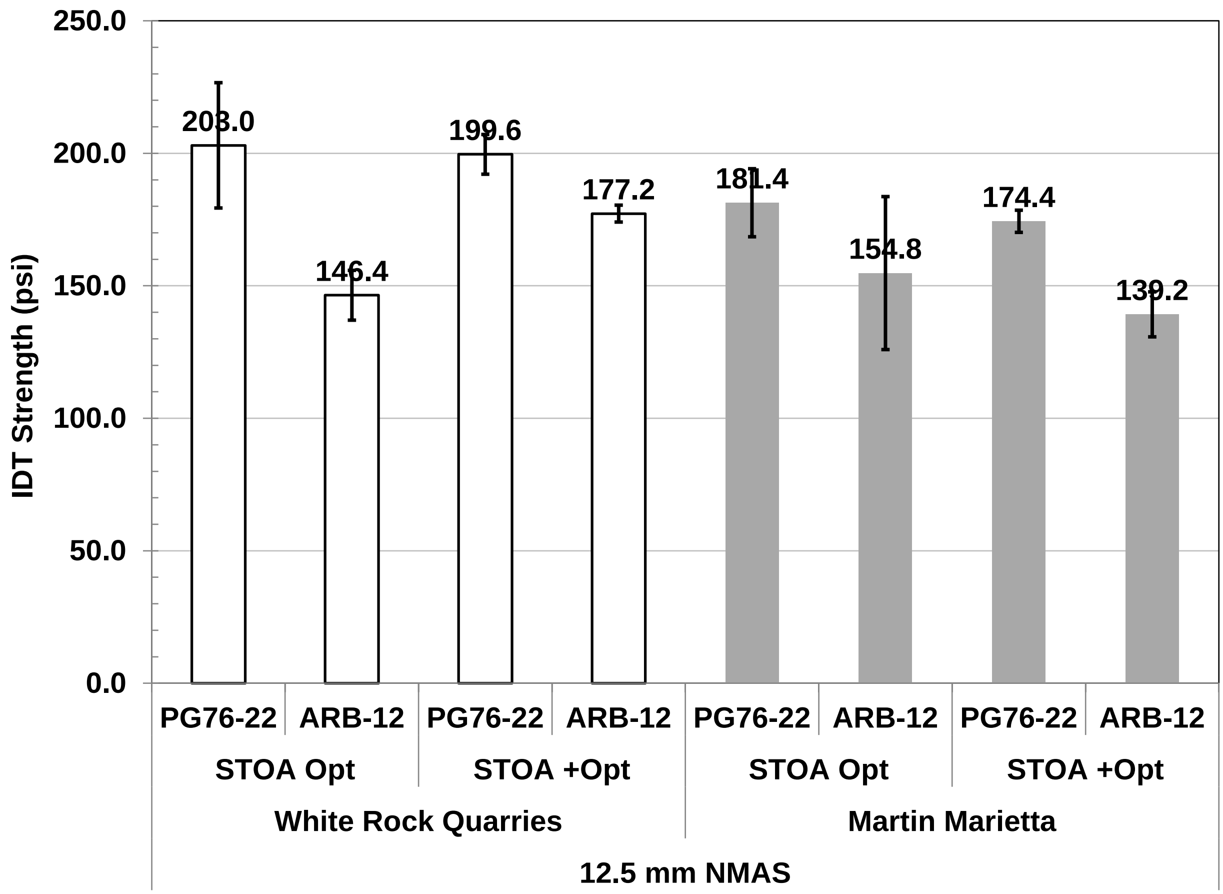


Figure 6.12 – IDT Strength for 12.5 mm NMAS FC-5 Mixtures – Short-Term Aged

The IDT fracture energy results are shown in Figures 6.13 and 6.14. The fracture energy results show a similar to trend to that of the IDT strength where for the PG 76-22 asphalt binder generally achieved slightly higher fracture energy than the ARB-12 asphalt binder for the same aggregate and binder content. Also, the 9.5 mm NMAS mixtures achieved very similar fracture energy for the same binder type and content (Figure 6.13). Meanwhile, for the 12.5 mm NMAS, there is a clear difference in fracture energy between the two aggregate sources where for the same asphalt binder type and content, the WRQ achieved a higher fracture energy.

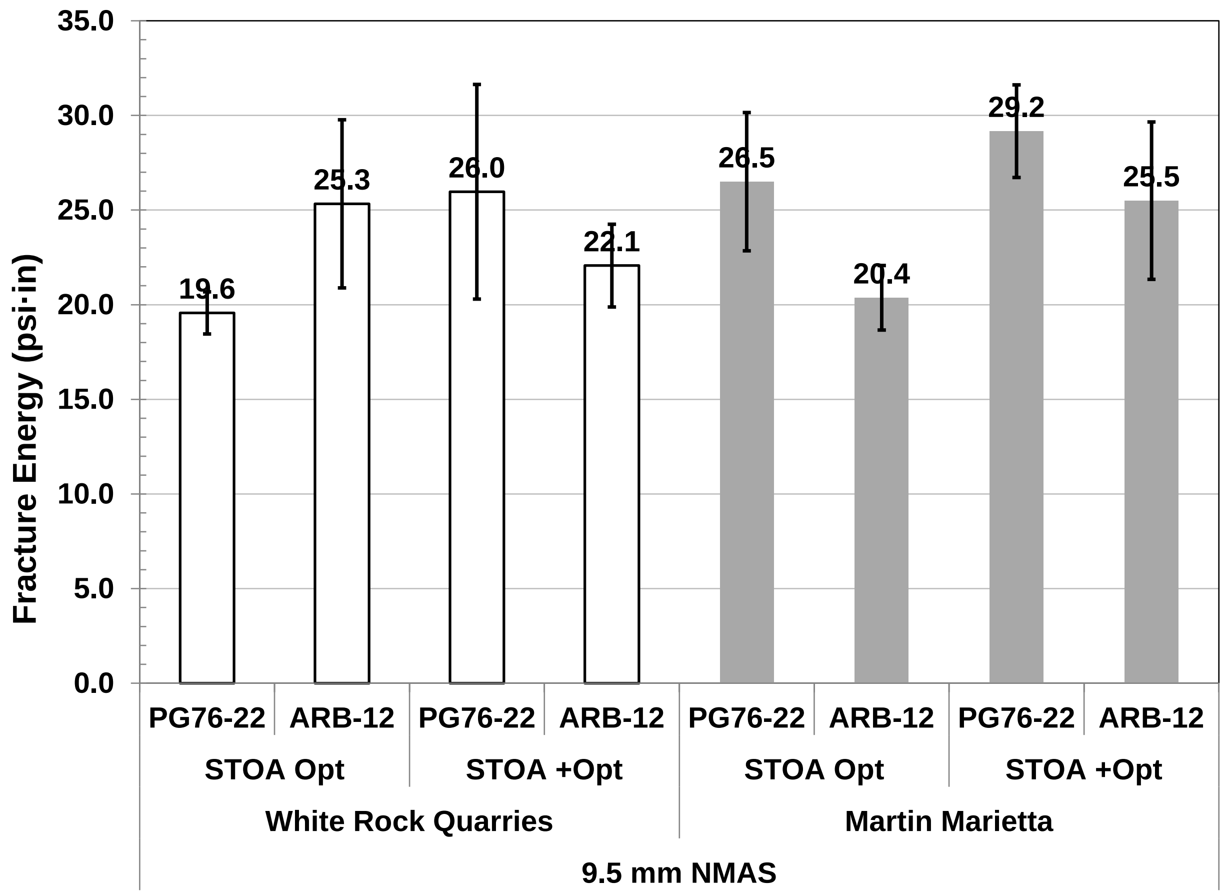


Figure 6.13 - IDT FE for 9.5 mm NMAS FC-5 Mixtures – Short-Term Aged

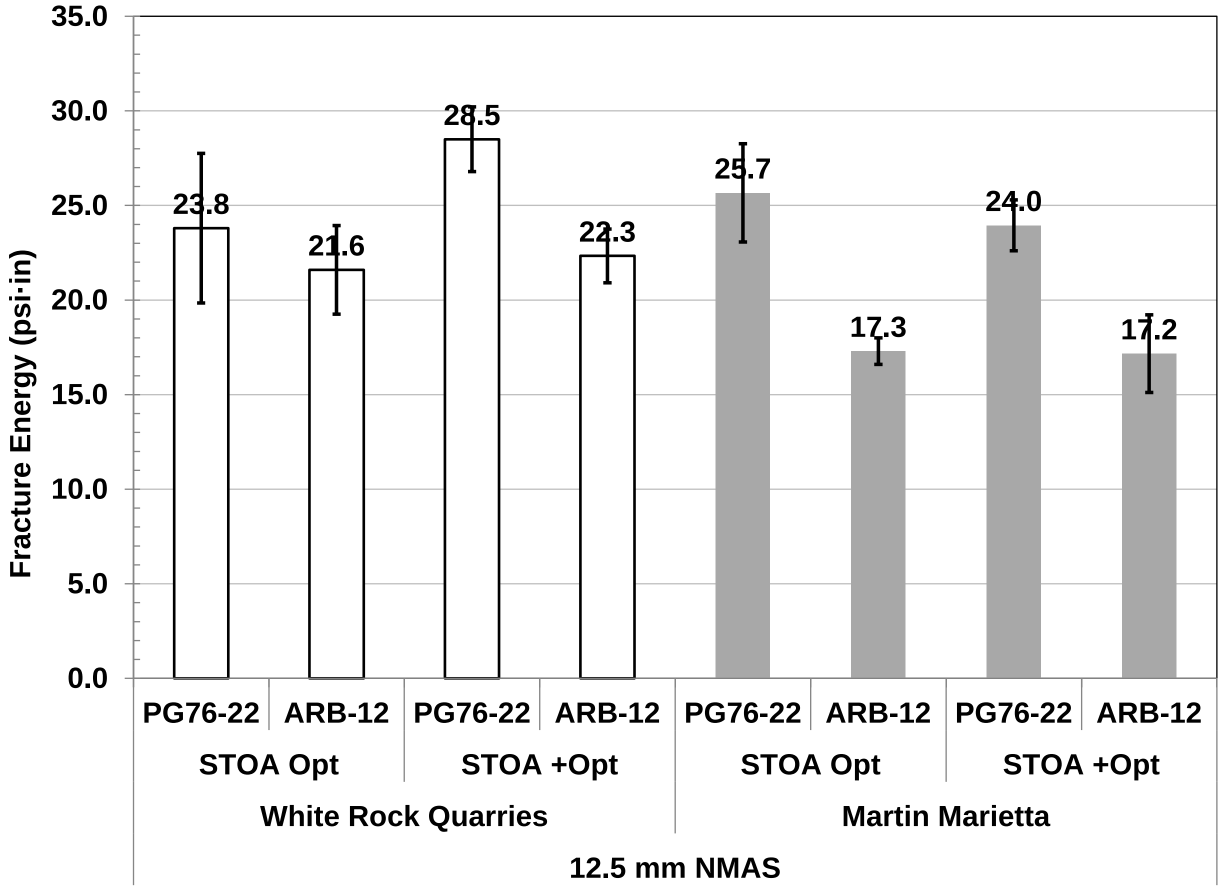


Figure 6.14 – Maximum IDT Strength FE for 12.5 mm NMAS FC-5 Mixtures – Short-Term Aged

6.1.4 Wet Hamburg Wheel Tracking Test

Wet Hamburg Wheel Track tests were conducted in accordance with AASHTO T 324, *Hamburg Wheel-Track Testing of Compacted Hot Mix Asphalt (HMA)*. Test specimens were tested at a water test temperature of 50^{o} C and 158 lb. steel wheel load. The test specimens were loaded at a rate of 52 passes per minute after a minimum soak/conditioning time of 30 minutes at 50^{o} C. Two indices were used to compare the different mixtures/parameters; 1) number of cycles to result in 12.5 mm of rutting; and 2) number of cycles to result in a Stripping Inflection Point (SIP). The SIP is determined by the intersection of the primary and secondary rutting slopes that occur when plotting the Hamburg rutting vs. loading cycles (Figure 6.15).

It should be noted that the SIP is generally used as an indication, or comparison, of moisture damage resistance. Meanwhile, the number of cycles to 12.5 mm of rutting is utilized to compare both the rutting resistance of the asphalt mixtures, as well as the general moisture damage resistance.

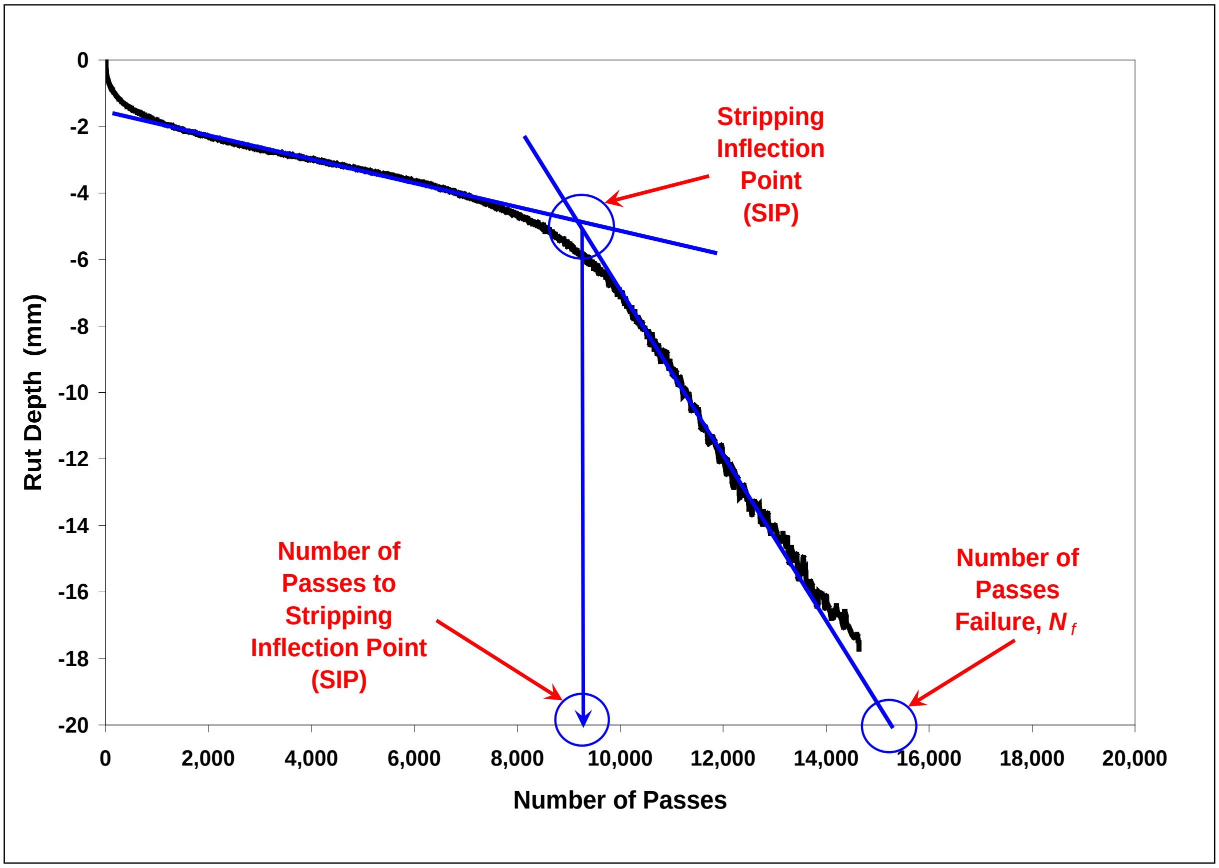


Figure 6.15 – Determination of Stripping Inflection Point (SIP) from Wet Hamburg Wheel Track Test

The HWTD test results for the 9.5 and 12.5 mm NMAS FC-5 mixtures are shown in Figures 6.16 and 6.17. The test results generally indicate that the White Rock Quarries FC-5 mixture was more rut resistant during the Hamburg test. The White Rock Quarries FC-5 averaged about two times greater resistance to rutting when compared to the Martin Marietta FC-5 mixture. This was observed for both the 9.5 mm and 12.5 mm NMAS mixtures. The test results also showed that the PG 76-22 asphalt binder resulted in better resistance to Hamburg rutting than the ARB-12 asphalt binder for the same asphalt binder condition (i.e. Opt or + Opt). The same general trend was found when comparing the Hamburg SIP results. Increasing the asphalt content by 0.6% resulted in slightly more rutting. This was found for the White Rock Quarries and Martin Marietta aggregates, 9.5 mm and 12.5 mm NMAS, and PG 76-22 and ARB-12 asphalt binder types.

Overall, the 12.5 mm NMAS FC-5 mixtures achieved greater performance when comparing the number of cycles to 12.5 mm rutting and SIP. This would indicate that the 12.5 mm mixtures should be more rut resistant in the field than the 9.5 mm NMAS FC-5 mixtures compared in this study.

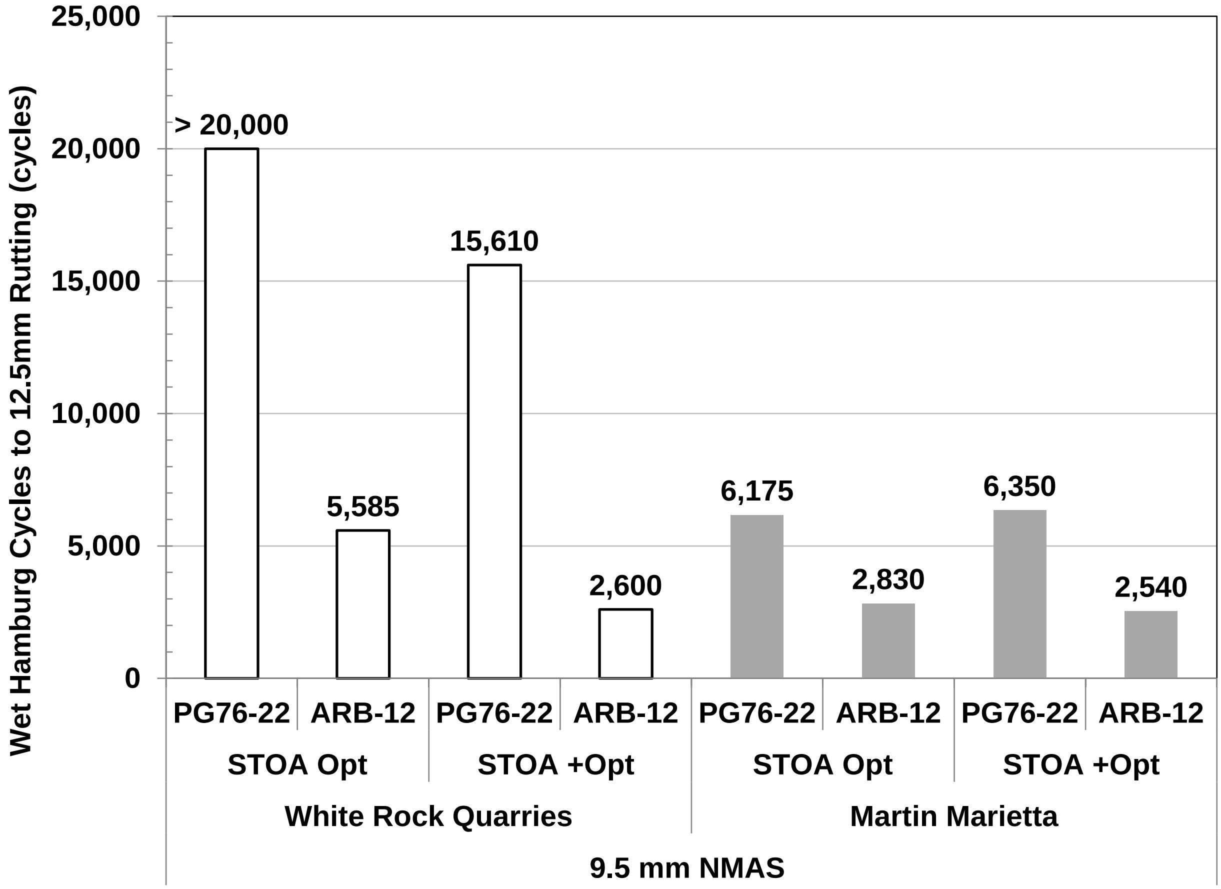


Figure 6.16 – Wet Hamburg Wheel Track Test Results for 9.5 mm NMAS Mixtures – Short-Term Aged – Cycles to 12.5 mm Rutting

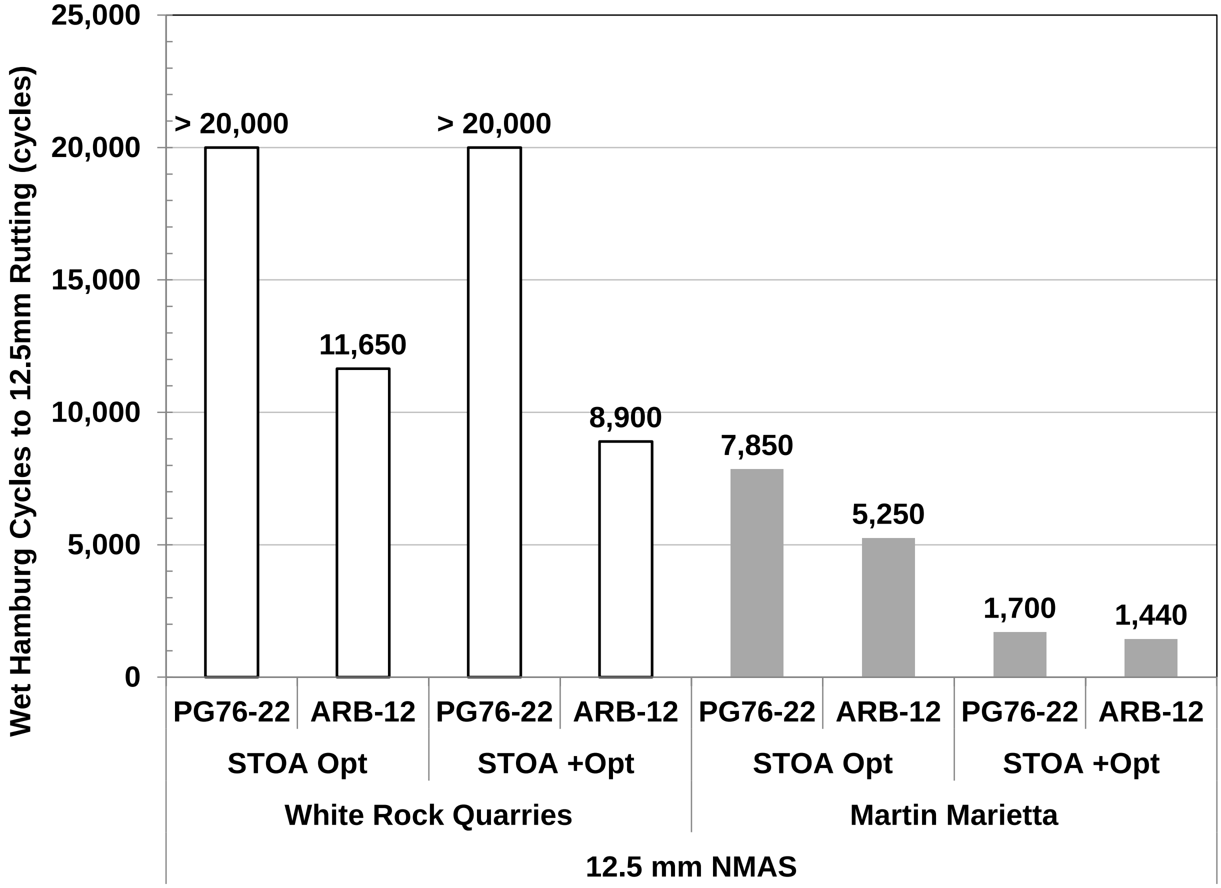


Figure 6.17 - Wet Hamburg Wheel Track Test Results for 12.5 mm NMAS Mixtures – Short-Term Aged – Cycles to 12.5 mm Rutting

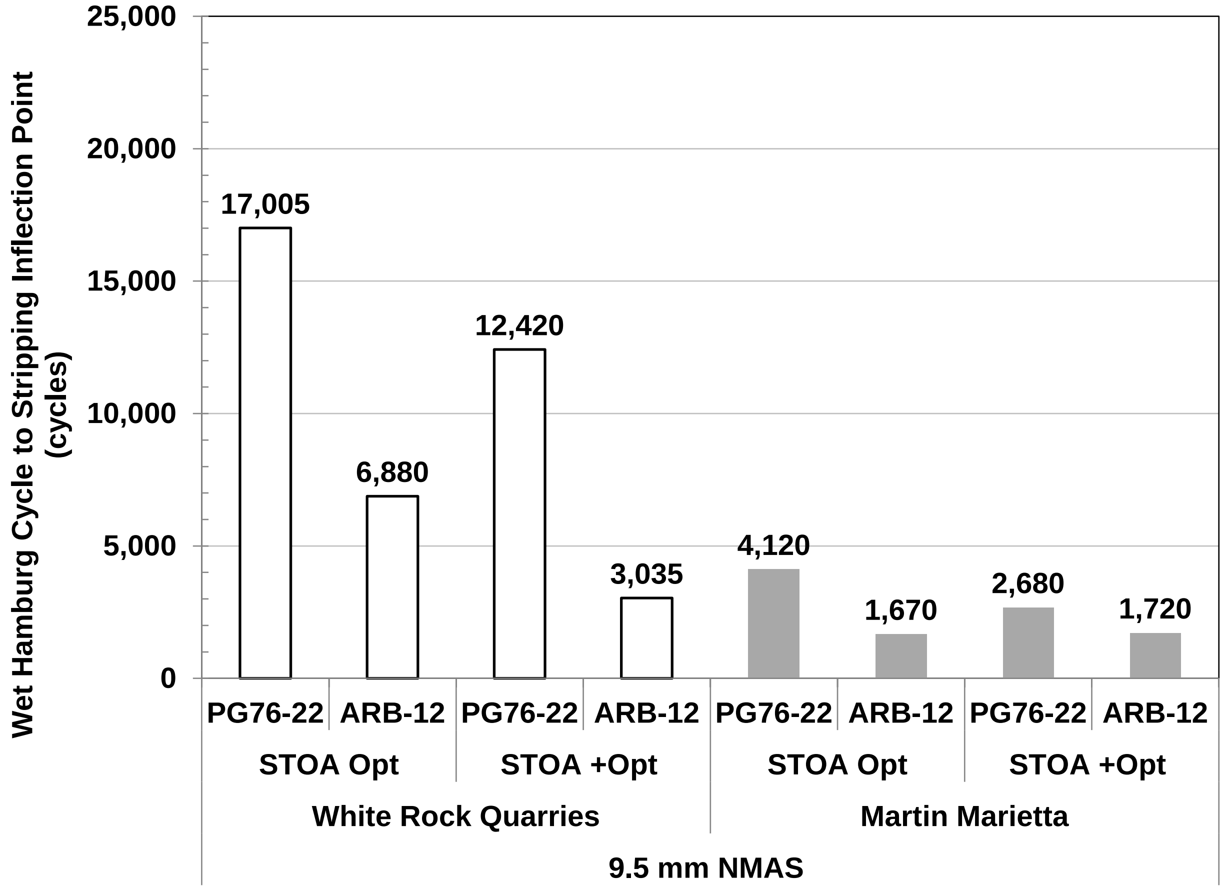


Figure 6.18 - Wet Hamburg Wheel Track Test Results for 9.5 mm NMAS Mixtures – Short-Term Aged – Cycles to Stripping Inflection Point (SIP)

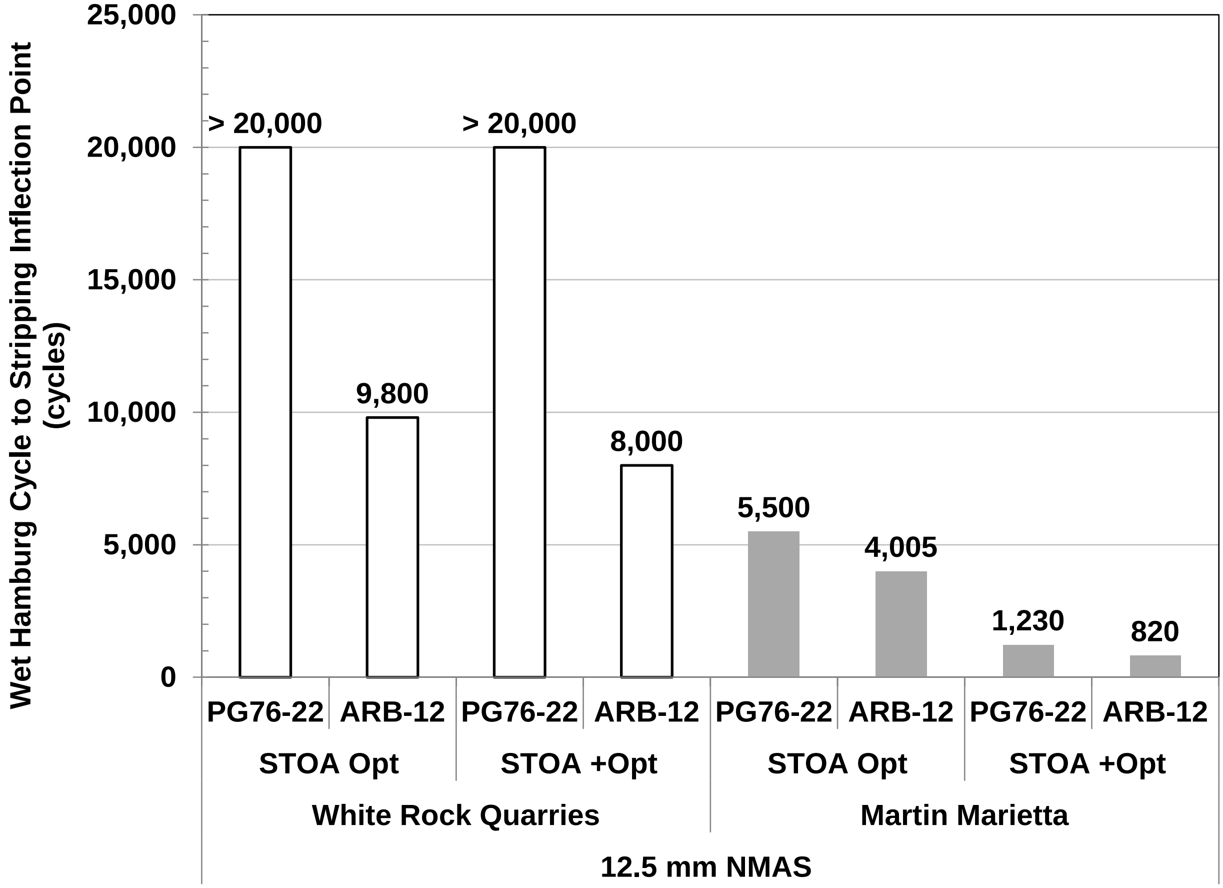


Figure 6.19 - Wet Hamburg Wheel Track Test Results for 12.5 mm NMAS Mixtures – Short-Term Aged – Cycles to Stripping Inflection Point (SIP)

6.1.5 General Conclusions for Short Term Aged FC-5 Mixtures

A series of fatigue and rutting performance tests were conducted on 9.5 mm and 12.5 mm NMAS FC-5 mixtures, using a PG 76-22 and ARB-12 asphalt binder, to evaluate their respective overall performance and assess whether or not a finer, 9.5 mm NMAS FC-5 mixture could lead to an improvement in durability over the traditionally used 12.5 mm NMAS FC-5 mixtures.

The test results showed that:

- FC-5 mixture durability, as measured using the Cantabro Abrasion Loss test resulted in mixed results. For the White Rock Quarries mixtures, on average, the 12.5 mm NMAS mixture resulted in a slightly better durability (i.e. lower Cantabro Abrasion Loss). Meanwhile, for the Martin Marietta aggregates, the 9.5 mm NMAS mixture achieved lower Cantabro Abrasion Loss results than the 12.5 mm NMAS mixtures for the same asphalt binder type and condition. However, it should be noted that all mixtures achieved relatively low abrasion loss values and performed well during the testing.
- Cracking resistance, as evaluated using the Overlay Tester, showed that ultimately the cracking resistance was function of the aggregate source and not the aggregate size. The Martin Marietta FC-5 mixtures were far superior when comparing the fatigue resistance performance in the Overlay Tester (3 to 4 times greater). The FC-5 mixtures with the ARB-12 asphalt binders typically performed better for the identical mixture condition when compared to the PG 76-22 asphalt binder. The exception to this was the 9.5 mm NMAS Martin Marietta mixtures. And although the aggregate source appeared to have the greatest impact on the Overlay Tester results, it was also determined that for each respective aggregate source, the fatigue resistance in the Overlay Tester slightly increased as the NMAS decreased (i.e. going from a 12.5 mm NMAS to a 9.5 mm NMAS).
- The IDT Strength, measured at 10^{o}C, was also used to characterize the fatigue resistance of the FC-5 mixtures. The test results indicated that the IDT strengths were very similar for the White Rock Quarries and Martin Marietta aggregates for the same asphalt binder type and condition. This was observed for both the 9.5 and 12.5 mm NMAS mixtures. The 9.5 mm NMAS mixtures resulted in slightly lower IDT Strengths than the 12.5 mm NMAS mixtures. FC-5 mixtures containing the PG 76-22 asphalt binder generally achieved higher IDT strengths when compared to the ARB-12 asphalt binder. A similar trend in the results was also found when characterizing the test data for fracture energy.
- The Hamburg Wheel Track device was used to assess the rutting potential of the FC-5 mixtures. The White Rock Quarry mixtures drastically outperformed the Martin Marietta mixtures when comparing the 9.5 and 12.5 mm NMAS mixtures. The test results also clearly indicated that the mixtures containing the PG 76-22 outperformed the ARB-12 asphalt. The Wet Hamburg Wheel Track test also determined that the 0.6% increase in asphalt content, which was found to help improve the fatigue cracking resistance in the Overlay Tester, caused the mixtures to have a greater magnitude of rutting when compared to the mixtures produced at optimum asphalt content.

6.2 Performance of 9.5 and 12.5 Nominal Maximum Aggregate Size Mixtures – Long-Term Aged Condition

Compacted FC-5 test specimens were Long-Term Oven Aged (LTOA) in accordance with AASHTO R 30, *Mixture Conditioning of Hot Mix Asphalt (HMA).* The LTOA conditioning required oven aging test specimens for 5 days at 85^{o} C. As shown earlier, all FC-5 test specimens were bound in wire mesh and hose clamps to ensure the aging procedure did not cause sample damage – only oxidation aging of the asphalt binder. According to the work by Kliewer et al. (1995), the LTOA procedure simulates approximately 7 to 12 years of in-place field aging, depending on the regional climatic condition.

To ensure the LTOA procedure applied additional oxidation aging, asphalt binder from the Short-Term and Long-Term aged specimens were extracted and recovered in accordance with AASHTO T 164, *Procedure for Asphalt Extraction and Recovery Process* and ASTM D5404, Standard Practice for Recovery of Asphalt from Solution Using the Rotary Evaporator (Figure 6.20). After the recovery process, the asphalt binder was tested for the respective performance grade (PG), in accordance with AASHTO M 320, *Standard Specification for Performance-Graded Asphalt Binder*, and Multiple Stress Creep Recovery (MSCR) in accordance with AASHTO T 350-14, *Standard Method of Test for Multiple Stress Creep Recovery (MSCR) Test of Asphalt Binder Using a Dynamic Shear Rheometer (DSR).* Master Stiffness (G*) curves of the recovered asphalt binder was also measured and utilized to evaluate the overall stiffness properties of the asphalt binder, as well as the relative aging characteristics of the asphalt binder.

The resultant continuous grade and MSCR properties from the Original, STOA, and LTOA recovered binders are shown in Table 6.3. The test results clearly indicate that stiffening in the asphalt binder occurs, especially when evaluating the high temperature properties (i.e. high temperature continuous grade and MSCR parameters). Table 6.3 indicates that the mixture conditioning procedures used in the study do age the asphalt binder, although there is only a modest change with respect to the STOA and LTOA mixture conditioned recovered asphalt binder. It should be noted that only the PG 76-22 asphalt binder was recovered and tested as the ARB-12 asphalt binder contains crumb rubber, which may not be fully recoverable due to the filtering process used in AASHTO T 164.

Table 6.3 – PG Grade and Multiple Stress Creep Recovery Properties for Sampled Binders

Property		Tank Condition	STOA Condition	LTOA Condition
Continuous PG Grade (^{o}C)	High Temp	77.6	83.6	87.7
	Intermediate Temp	21.3	20.8	24.3
	Low Temp	-27.8	-27.8	-26.2
Multiple Stress Creep Recovery; J_{nr} (% Rec)	58^{o}C	0.103 (68.8%)	0.043 (79%)	0.032 (77.73%)
	64^{o}C	0.267 (58.5%)	0.100 (73.5%)	0.063 (75.5%)
	70^{o}C	0.761 (40.5%)	0.283 (61.1%)	0.167 (65.4%)

Figure 6.20 – Asphalt Binder Recovery Equipment at Rutgers University (Rotavap System)

The shear modulus (G*) master curves, generated by determining the G* properties in the linear-elastic range of the asphalt binder at various temperature and loading frequencies, are shown in Figure 6.21. The G* master curves again indicate that the stiffness of the asphalt binders increase as the magnitude of the conditioning increases. Therefore, the combination of the PG, MSCR and G* Master Curves indicates that the mixture conditioning conducted during the study ages the asphalt binder.

The modest change from STOA to LTOA may be explained by the increased film thickness that is associated with FC-5 type mixtures. It is well recognized that asphalt binder film thickness is much greater than dense-graded mixtures. With a thicker asphalt film on the aggregate, a larger volume of asphalt binder may not be oxidized as much as asphalt mixtures with thinner films. Further testing would need to be conducted to validate this concept, but it is beyond the scope of this study.

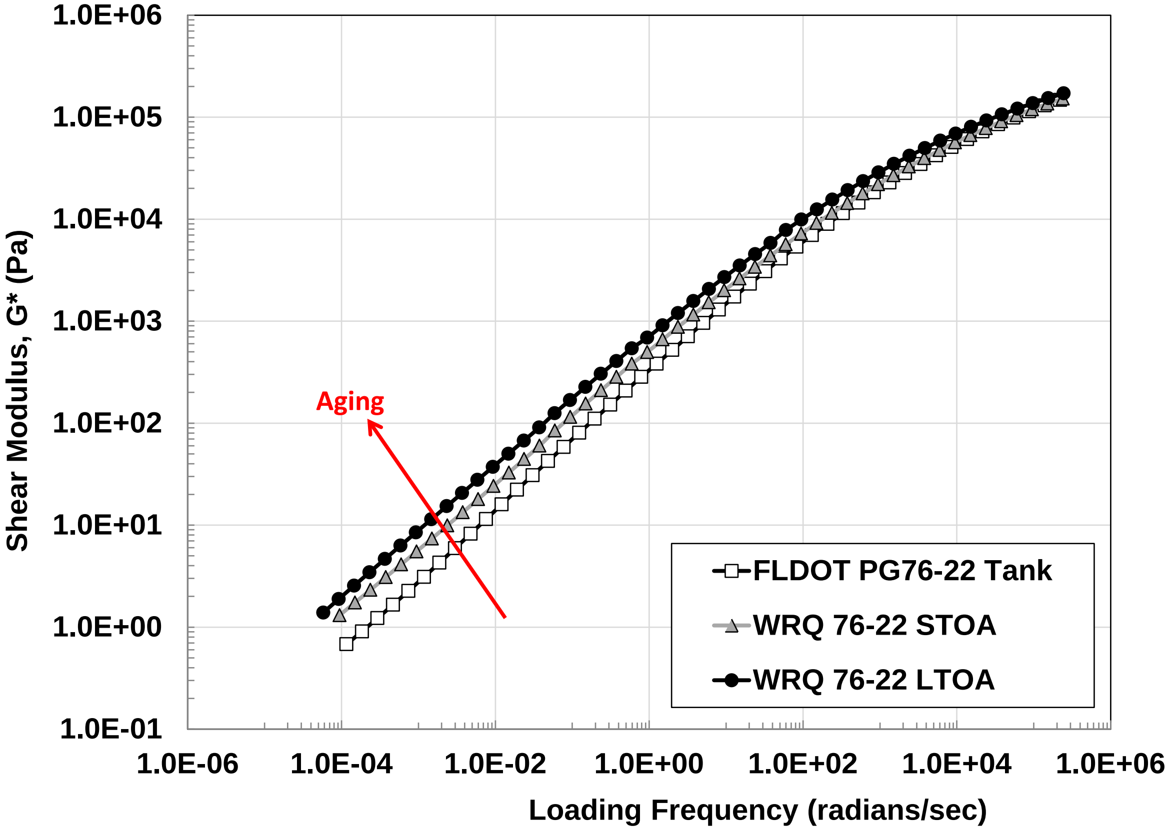


Figure 6.21 – Shear Modulus (G*) Master Stiffness Curves of Tank, STOA, and LTOA Condition PG 76-22 Asphalt Binder

6.2.1 Overlay Tester Results – Long-Term Oven Aged (LTOA)

The Overlay Tester fatigue cracking results for the White Rock Quarries and Martin Marietta aggregates are shown in figure 6.22 and 6.23, respectively. The figures contain the 9.5 mm and 12.5 mm NMAS mixtures for the STOA and LTOA mixture conditions. The results show that there is a clear reduction in fatigue life as evaluated in the Overlay Tester when the mixture aging condition goes from STOA to LTOA. This is expected as the asphalt binder in the mixtures are additionally oxidized and stiffened due to the aging process.

Figures 6.24 and 6.25 show the percent reduction in the Overlay Tester fatigue life when comparing the LTOA to the STOA test results. The figures indicate that a greater reduction in fatigue performance occurs for the PG 76-22 asphalt binder, as opposed to the ARB-12, for the same binder condition (Optimum or Optimum Plus). It was also observed that for most of the mixtures evaluated, the Optimum Plus resulted in a lower percent reduction than the mixtures produced at optimum asphalt content when comparing the same binder source and mixture type (i.e. NMAS and aggregate source). This would indicate that greater film thickness, or higher effective asphalt content, resists aging better than thinner films or lower effective asphalt contents - concepts that are well known already.

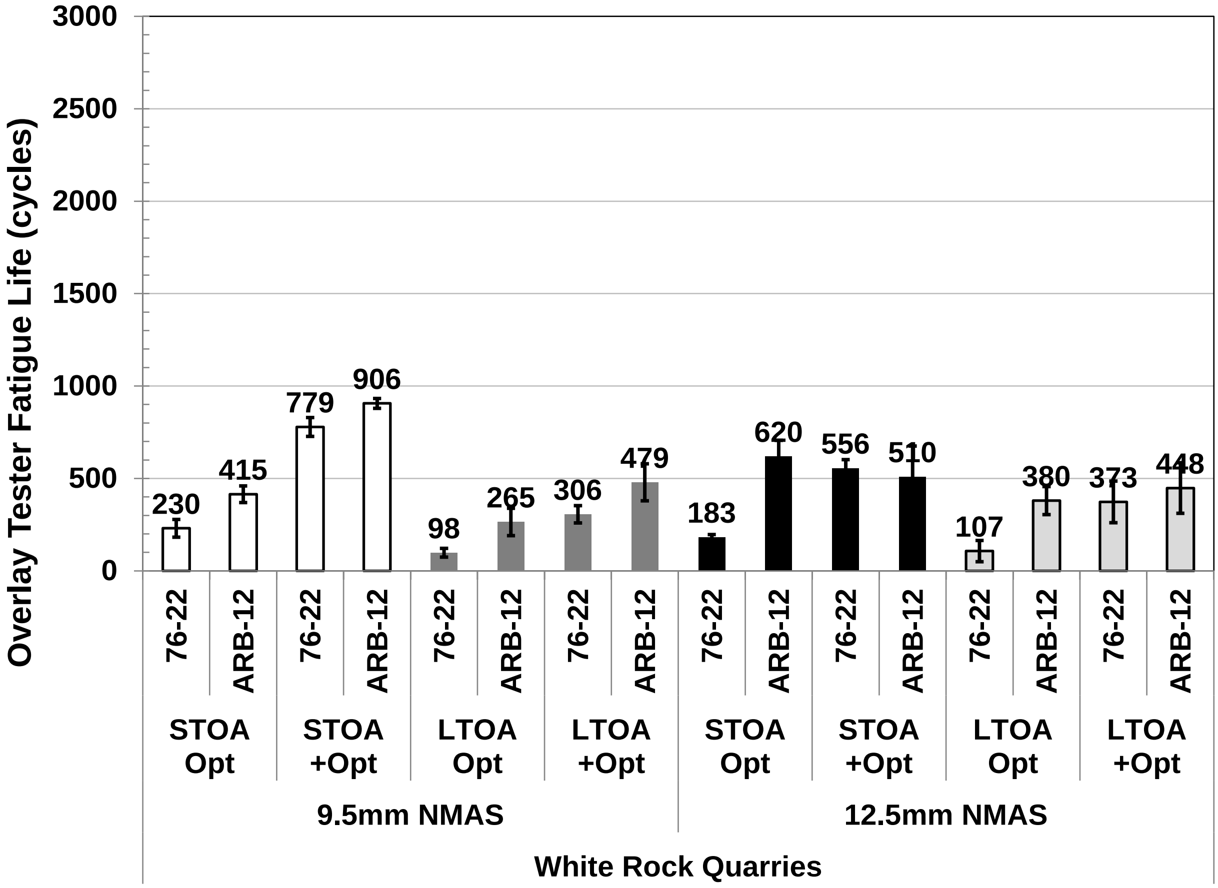


Figure 6.22 – Overlay Tester Results for White Rock Quarries FC-5 Mixtures – Short-Term and Long-Term Oven Aged Conditions

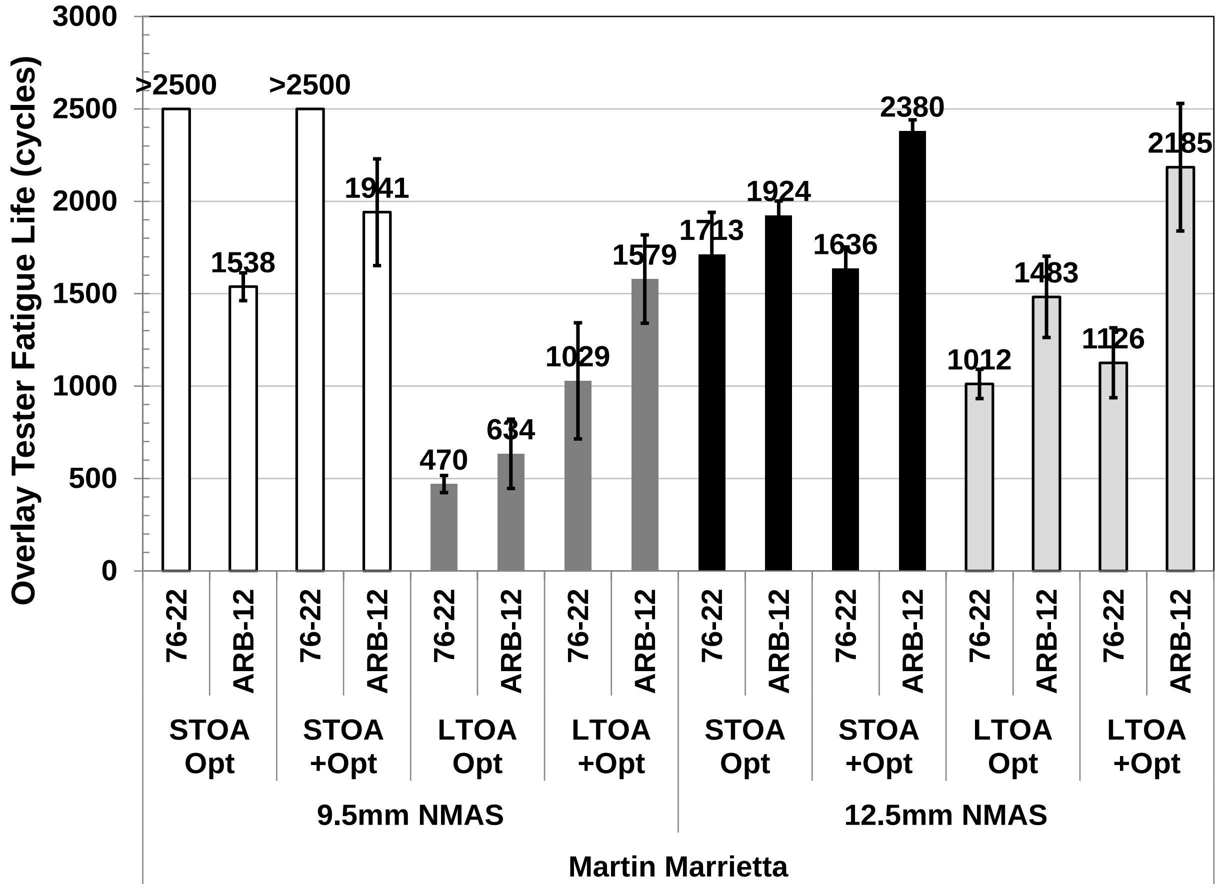


Figure 6.23 – Overlay Tester Results for Martin Marietta FC-5 Mixtures – Short-Term and Long-Term Oven Aged Conditions

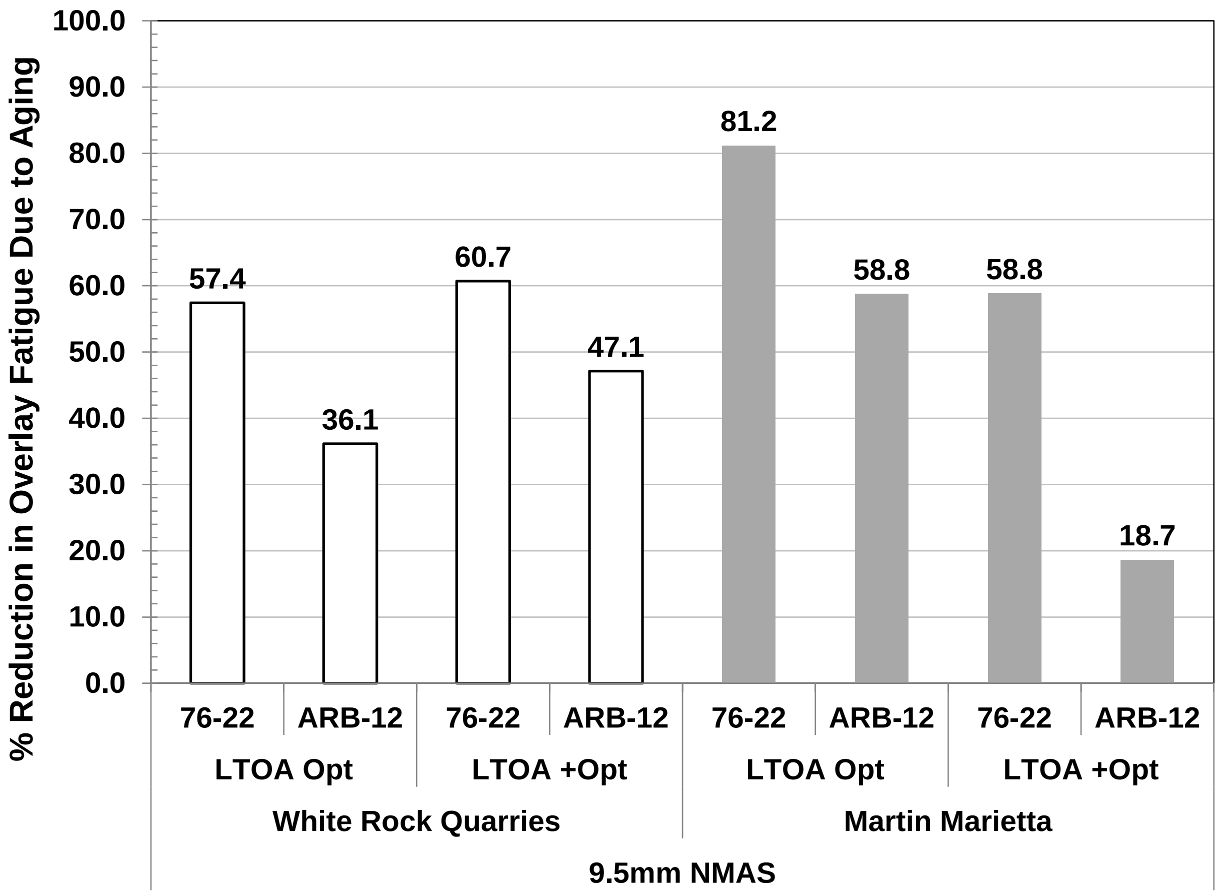


Figure 6.24 – Percent Reduction in Overlay Tester Fatigue Life Due to Laboratory Aging for the 9.5 mm NMAS FC-5 Mixtures

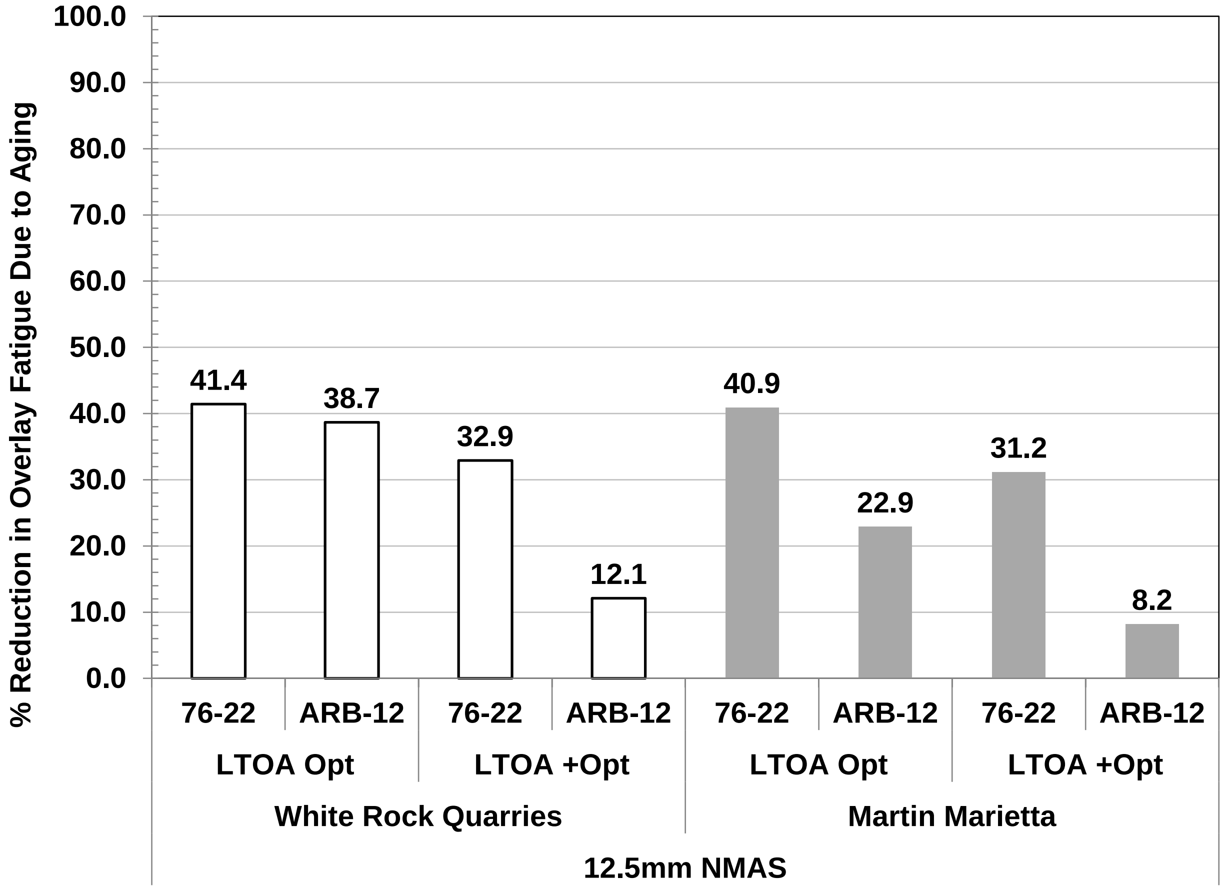


Figure 6.25 – Percent Reduction in Overlay Tester Fatigue Life Due to Laboratory Aging for the 12.5 mm NMAS FC-5 Mixtures

6.2.2 *Indirect Tensile (IDT) Strength – Long-Term Oven Aged (TLOA)*

The IDT strength for the LTOA FC-5 mixtures are shown in Figures 6.26 and 6.27 for the White Rock Quarries and Marin Marietta aggregates, respectively. In general, the IDT strength of the FC-5 mixtures increased with aging. The fact that the IDT strength improved as the mixtures underwent additional oxidation aging would indicate that the IDT strength by itself is not a good indicator of cracking resistance. The test data shown in Figures 6.26 and 6.27 also clearly show that the PG 76-22 asphalt binder resulted in a higher IDT strength when compared to the ARB-12 asphalt binder at the same binder condition.

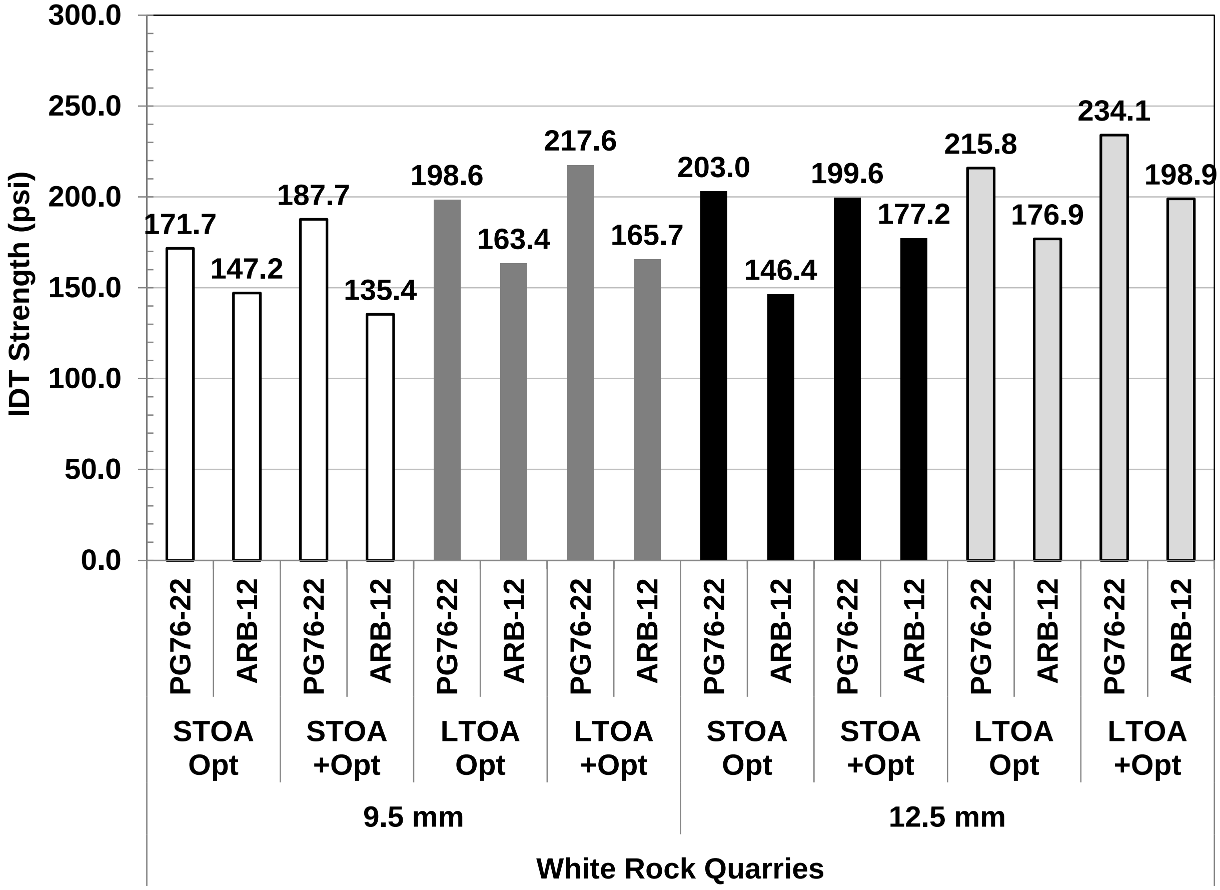


Figure 6.26 – IDT Strength of White Rock Quarries FC-5 Mixtures – Long-Term Oven Aged Condition

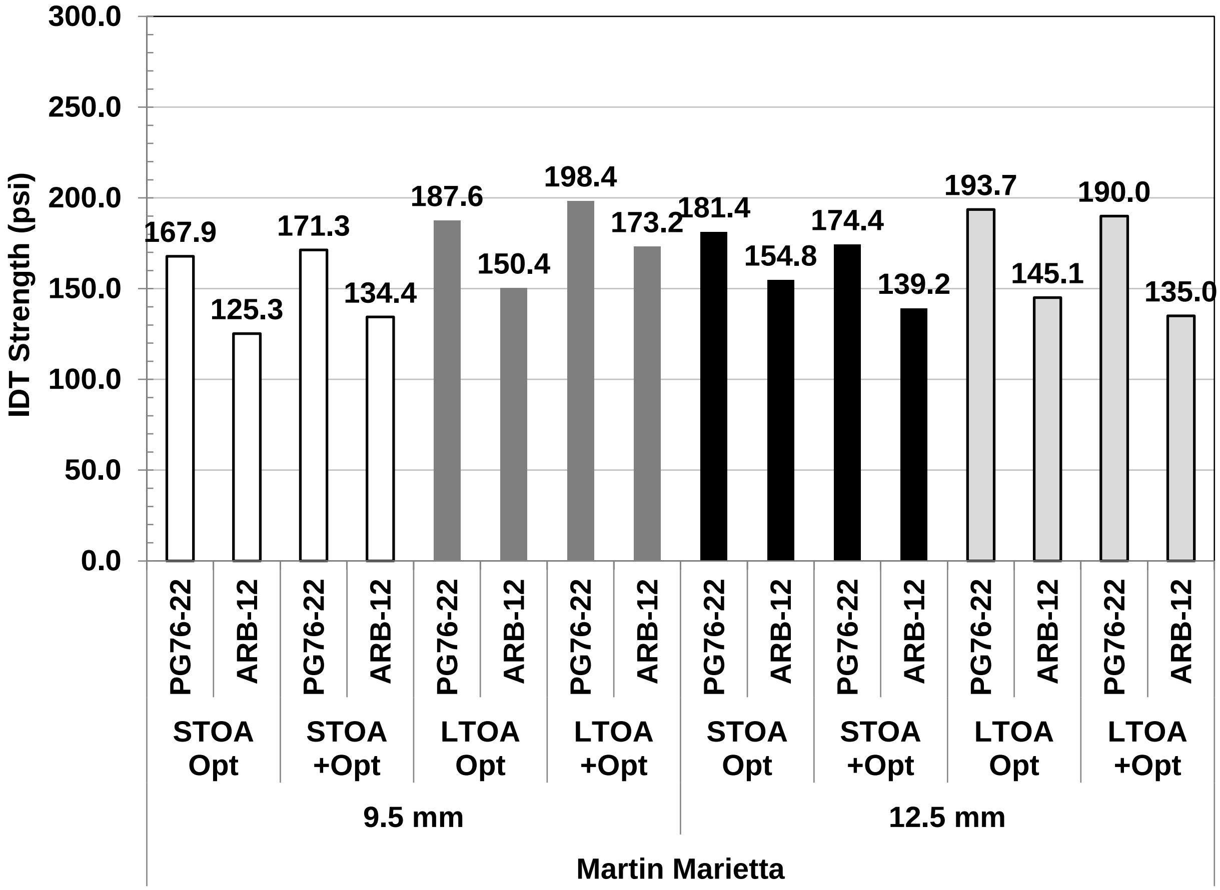


Figure 6.27 – IDT Strength of Martin Marietta FC-5 Mixtures – Long-Term Oven Aged Condition

The IDT FE results are shown in Figures 6.28 and 6.29 for the White Rock Quarries and Martin Marietta FC-5 mixtures, respectively. The FE results are more reasonable with respect to what would be expected, whereas the amount of oxidation aging increases, the FE, or resistance to cracking, decreases. It should be noted that the decrease in FE from the LTOA test specimens were not dramatic and generally within the standard deviation of the test results. This would indicate that the test results between the STOA and LTOA conditioned mixtures were statistically equal.

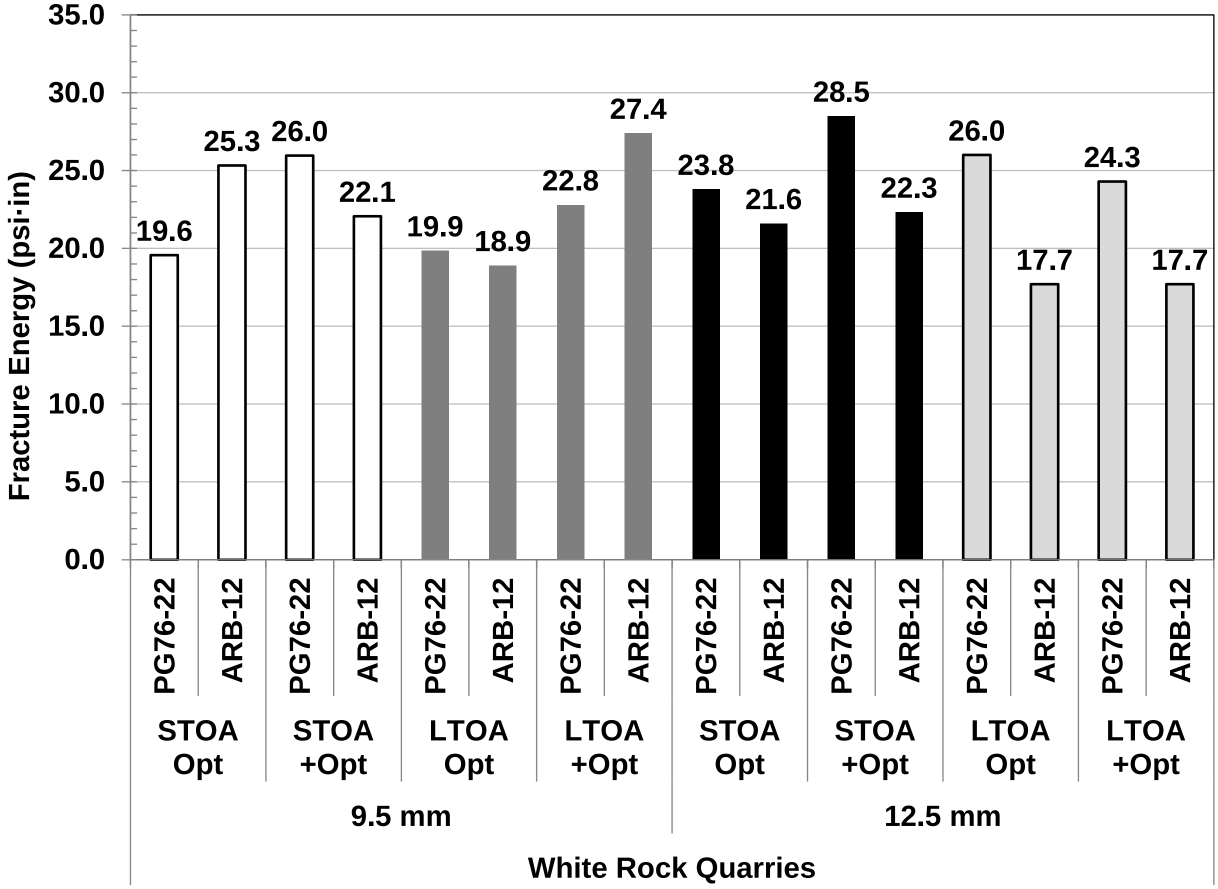


Figure 6.28 – IDT Maximum Strength FE for White Rock Quarries FC-5 Mixtures – Long-Term Oven Aged Condition

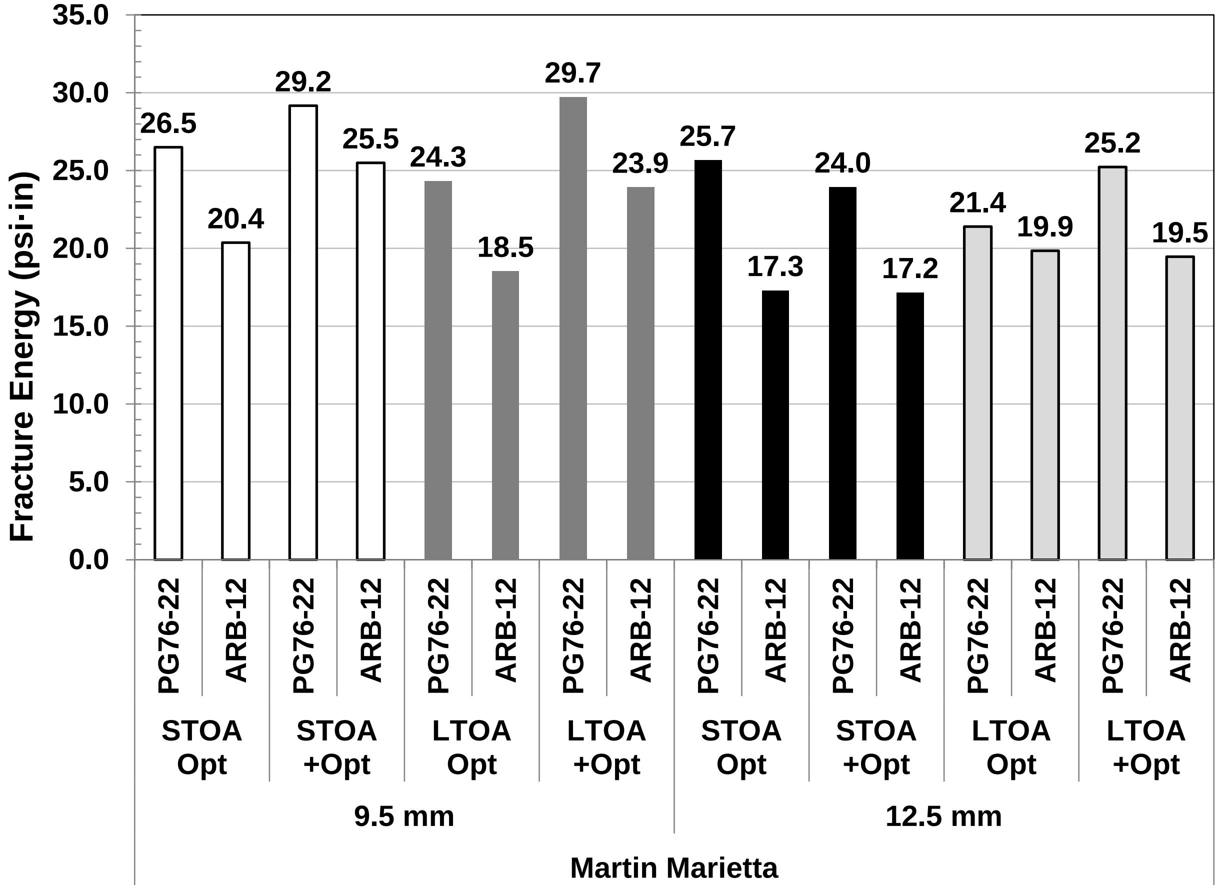


Figure 6.29 – IDT Maximum Strength FE for Martin Marietta FC-5 Mixtures – Long-Term Oven Aged Condition

6.2.3 Hamburg Wheel Track Test – Long-Term Oven Aged Samples

The rutting resistance of the LTOA HWTD samples is shown in Figure 6.30 and 6.31 for the White Rock Quarries and Martin Marietta FC-5 mixtures. The results clearly show that the number of cycles to 12.5 mm rutting increases with aging. This was of no surprise as one would expect rutting resistance to increase as the asphalt mixture stiffens. Also, as indicated in Table 6.3, the high temperature PG and non-recoverable creep compliance (J_{nr}) both improved as the asphalt binders were aged from STOA to LTOA.

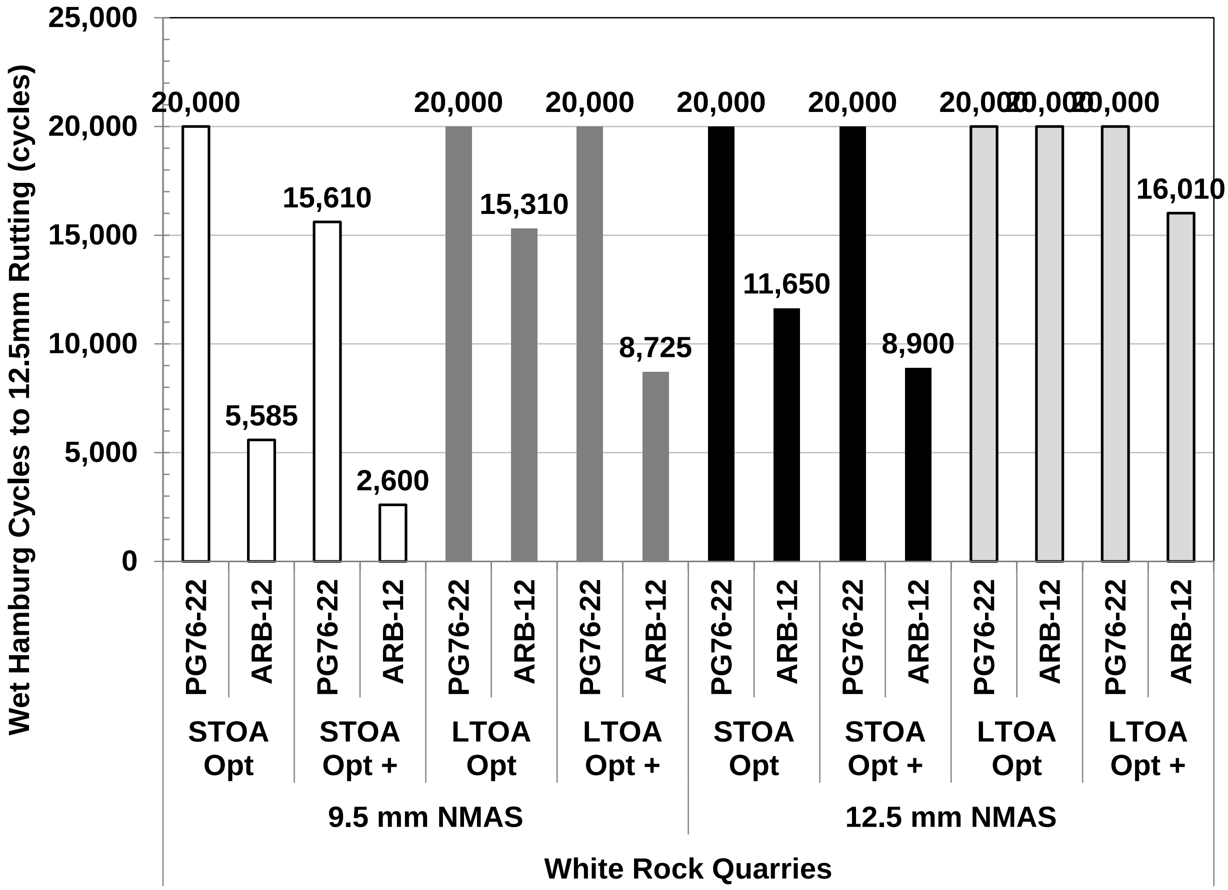


Figure 6.30 – Wet Hamburg Wheel Tracking Test Results for White Rock Quarries FC-5 Mixtures – Long-Term Aged Condition

Similar to the STOA Hamburg test results, the general rutting performance was found to be dominated by the aggregate source, where the White Rock Quarries FC-5 mixtures were more rut resistant than the Martin Marietta mixtures, and the asphalt binder type, where the PG 76-22 was found to be more rut resistant than the ARB-12 asphalt binder. Rutting in the Hamburg was found to increase with the addition of 0.6% asphalt binder (Optimum Plus), even when the mixtures were LTOA. Also similar to the STOA specimens, rutting for the Opt+ specimens were found to be less severe when the PG 76-22 asphalt binder was used.

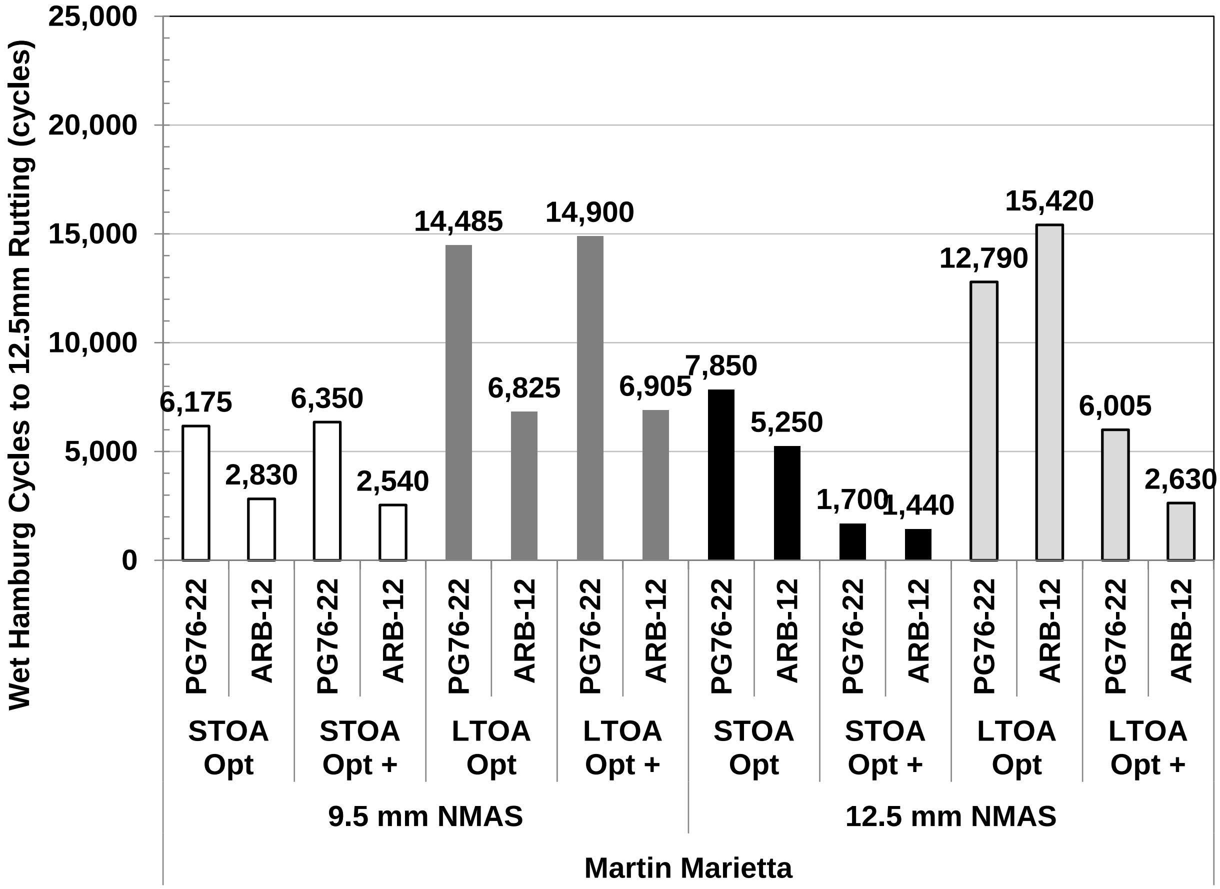


Figure 6.31 – Wet Hamburg Wheel Tracking Test Results for Martin Marietta FC-5 Mixtures – Number of Cycles to 12.5 mm Rutting - Long-Term Aged Condition

The Hamburg SIP results are shown in Figures 6.32 and 6.33. The trend in SIP is similar to that of the number of cycles to 12.5 mm rutting.

- White Rock Quarries FC-5 mixtures outperformed the Martin Marietta mixtures;
- The FC-5 mixtures with PG 76-22 asphalt binder outperformed the ARB-12 asphalt in both the STOA and LTOA conditions;
- The addition of 0.6% asphalt binder resulted in lower SIP than the mixtures produced at optimum asphalt content; and
- The 12.5 mm NMAS mixtures obtained larger SIP values than the 9.5 mm NMAS mixtures of the same asphalt binder type and condition.

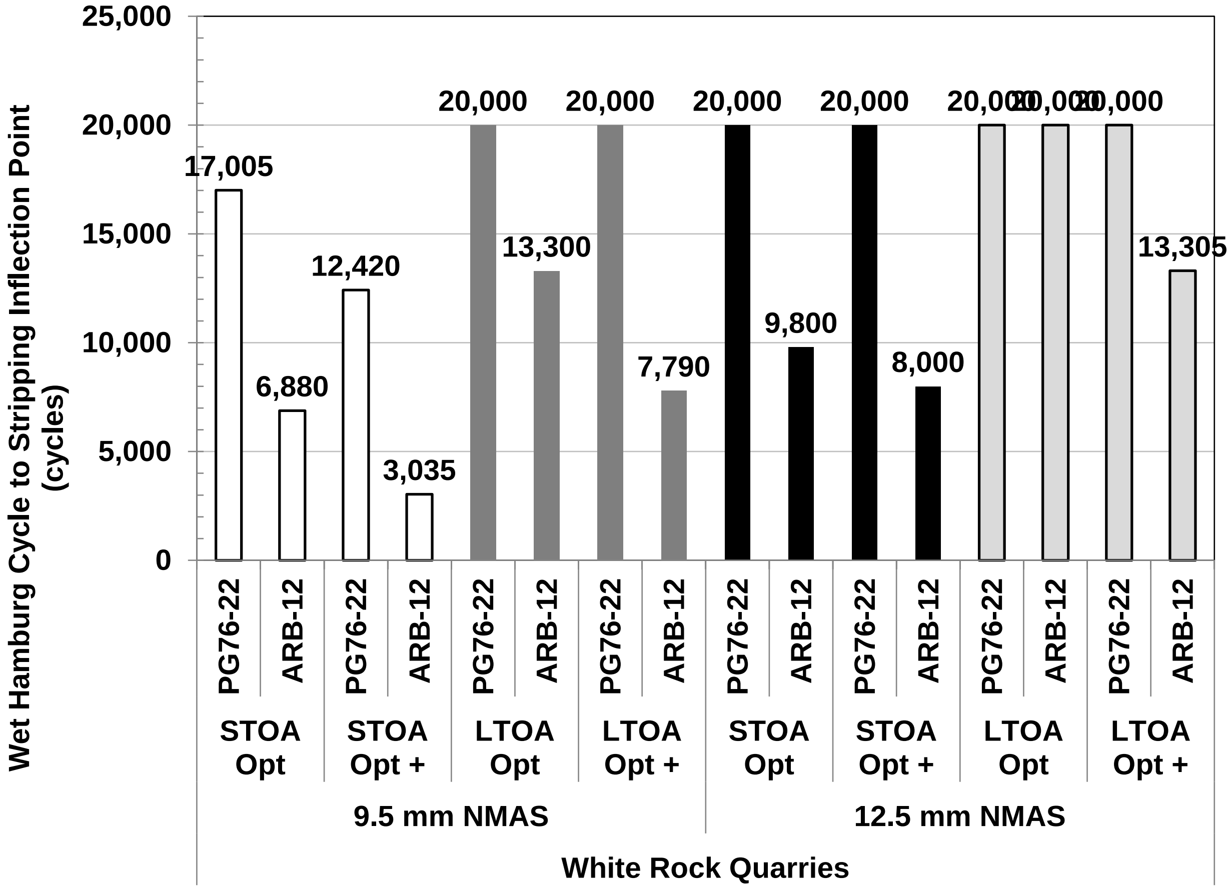


Figure 6.32 - Hamburg Wheel Track Testing for White Rock Quarries FC-5 Mixtures – Stripping Inflection Point (SIP) – Long-Term Aged Condition

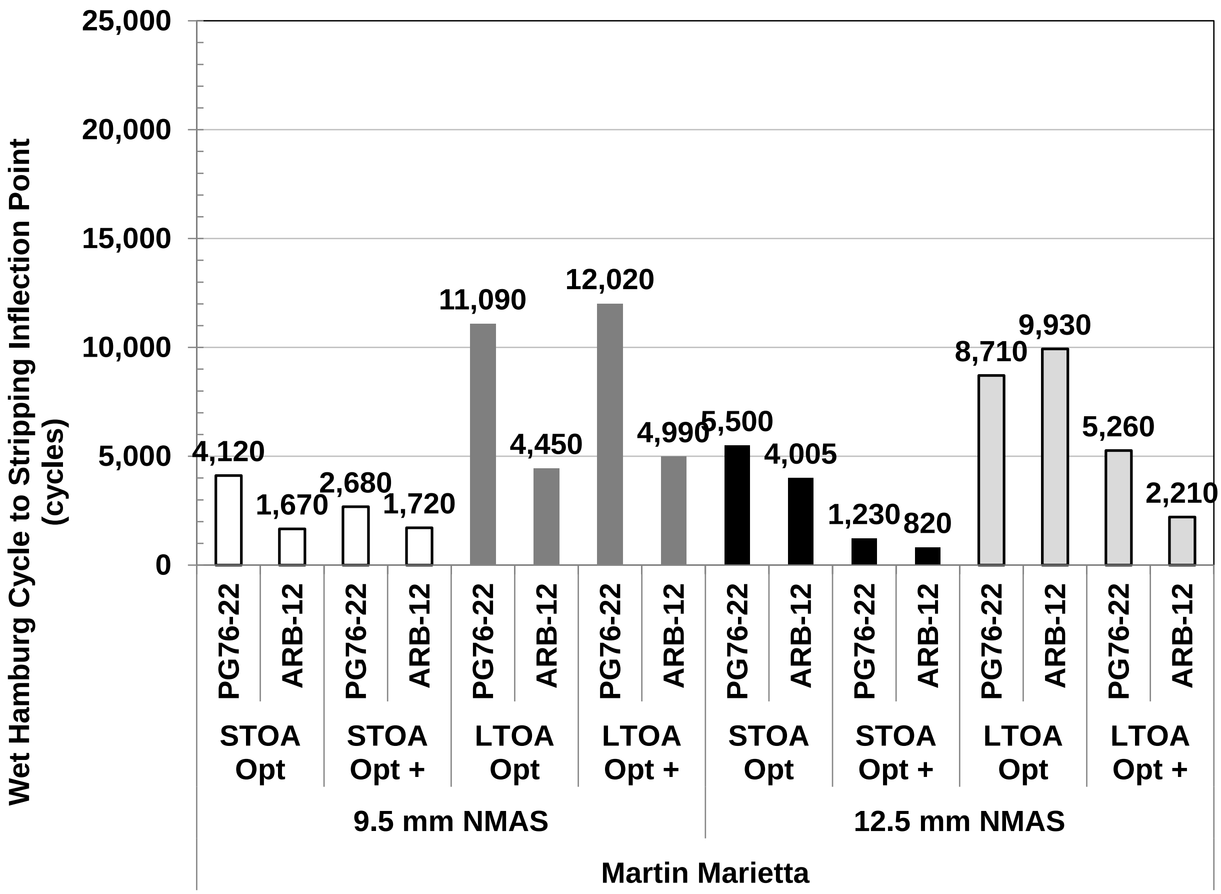


Figure 6.33 – Wet Hamburg Wheel Track Testing for Martin Marietta FC-5 Mixtures – Stripping Inflection Point (SIP) – Long-Term Aged Conditions

6.2.4 General Conclusions of Long-Term Oven Aged (LTOA) Conditioning of FC-5 Mixtures

FC-5 mixtures, produced in the laboratory using different asphalt binder types and contents, were LTOA in accordance with AASHTO R 30 and compared to the performance of the same FC-5 mixtures that were STOA, in an attempt to look at how aging affects the performance of the FC-5 mixtures. Mixtures using both PG 76-22 and ARB-12 binders, different NMAS, and asphalt binder conditions (Opt and Optimum + 0.6%) were evaluated. In general, the following findings were observed from the testing:

- Fatigue resistance, as evaluated in the Overlay Tester, was found to reduce in the FC-5 mixtures after LTOA conditioning. On average, it was found that;
  - When comparing the NMAS, a 28.5% reduction in fatigue life was observed for the 12.5 mm NMAS FC-5 mixtures, while a 52.4% reduction in fatigue life was found in the 9.5 mm NMAS FC-5 mixtures.
  - When comparing asphalt binder type in the 12.5 mm NMAS FC-5 mixtures, the PG 76-22 asphalt mixtures had a 29.3% reduction in fatigue life, while the ARB-12 asphalt binder resulted in a 20.5% reduction in fatigue life.

- When comparing the asphalt binder type in the 9.5 mm NMAS FC-5 mixtures, both the PG 76-22 and ARB-12 asphalt binders resulted in almost the identical reduction in fatigue life, 40.5% and 40.2%, respectively.
- When looking at the influence of adding an additional 0.6% asphalt binder in the FC-5 mixtures, it was found that the mixtures which utilized the ARB-12 asphalt benefitted the most. The ARB-12 mixtures had an average reduction in fatigue life at optimum asphalt content of 39.1%, while the additional of 0.6% ARB-12 asphalt binder reduced that to 21.5%. Meanwhile, when using a PG 76-22 asphalt binder, it was found that mixtures at optimum asphalt content had an average of 55.2% reduction in the fatigue life, while the FC-5 mixtures that had an additional 0.6% PG76-22 asphalt binder only achieved a 45.9% reduction in fatigue life.

- The IDT strength of the LTOA FC-5 mixtures was determined at a test temperature of 10^{o} C using the identical test procedure as the STOA FC-5 mixtures discussed earlier. It was found that the major difference between the IDT strength and FE was the asphalt binder utilized in the respective mixture. The FC-5 mixtures with PG 76-22 obtained higher IDT strengths and FE values than the FC-5 mixtures with ARB-12 asphalt binder. Only slight differences were observed due to the additional oxidation aging that occurred due to LTOA when compared to the STOA test specimens.
    - The IDT strength increased 6.7% and 14.9% for the 12.5 mm and 9.5 mm NMAS FC-5 mixtures, respectively. This indicates that as aging increases, the tensile strength of the FC-5 mixtures also increases. However, when reporting the test data using the concept of FE, it was found that the FE decreased 5.9 and 6.3%, respectively, for the 12.5 mm and 9.5 mm NMAS mixtures.
    - The IDT strength was found to increase in both the PG 76-22 and ARB-12 binders the same, whether the asphalt content was optimum or optimum + 0.6%. At the optimum condition, the IDT strength increased slightly more than 9% while the ARB-12 asphalt binder mixtures recorded a 12% increase. A different trend was found with evaluating the IDT data using FE. At the optimum asphalt content condition, the PG 76-22 and ARB-12 asphalt binder mixtures resulted in a 4.7% and 13.3% reduction in FE, respectively. Meanwhile, the ARB-12 asphalt binder mixtures were found to have a 6% and 0.4% reduction in FE, respectively. Since it is well known that as asphalt binder ages, it is more susceptible to cracking, it appears that the representing IDT strength using FE better represents what is more commonly observed in the field.
- Overall, the same general trend in performance with the STOA Hamburg test specimens was also found with the LTOA conditioned Hamburg test specimens. However, there was an increase in the rutting resistance of the LTOA FC-5 mixtures due to the additional oxidation aging that occurred during the LTOA conditioning. In fact,
    - On average, a 38% increase in Hamburg rutting resistance was found in the 12.5 mm NMAS FC-5 mixtures, while a 49% increase in the Hamburg rutting resistance was found in the 9.5 mm NMAS FC-5 mixtures due to the LTOA conditioning.
    - When comparing asphalt binders within the different NMAS FC-5 mixtures, it was found that a 28% increase in the Hamburg rutting resistance was found for the 12.5 mm NMAS mixtures using PG 76-22, while a 49% increase in Hamburg rutting resistance was observed for the 12.5 mm NMAS FC-5 mixtures with

ARB-12 asphalt binder. For the 9.5 NMAS mixtures, a 34% increase in Hamburg rutting resistance was found when utilizing a PG 76-22 asphalt binder, while a 64% increase in Hamburg rutting resistance was found when incorporating an ARB-12 asphalt binder.

- The addition of 0.6% asphalt binder, above the optimum asphalt content, was shown to influence the FC-5 mixtures with PG 76-22 asphalt more than the ARB-12 asphalt binder when comparing the increase in Hamburg rutting resistance. For the FC-5 mixtures with PG 76-22 at optimum asphalt content, a 24% increase in Hamburg rutting resistance was observed. Meanwhile, the FC-5 mixtures with 0.6% above optimum asphalt content witnessed a 38% increase in Hamburg rutting resistance. The FC-5 mixtures with ARB-12 asphalt binders were found to have similar increases in Hamburg rutting resistance with a 57% and 56% increase for the ARB-12 at optimum and ARB-12 at optimum + 0.6% asphalt binder, respectively.
- It should be noted that an increase in rutting resistance due to additional oxidation aging simply means an increase in the general stiffness of the asphalt mixtures, especially at higher temperature. This is clearly demonstrated in the Shear Modulus, G*, master curves developed using extracted and recovered asphalt binder from tested FC-5 mixtures (Figure 6.21).

CHAPTER 7 – EFFECT OF PRODUCTION VARIATIONS ON FC-5 MIXTURE PERFORMANCE (EXPERIMENT 3)

The final (3^{rd}) laboratory experiment was conducted to evaluate the effect of typical construction variations on the performance of FC-5 mixtures. As per Table 337-2 of the Florida Department of Transportation Standard Specifications for Road and Bridge Construction, Table 7.1 shows the *FC-5 Master Production Ranges*.

Table 7.1 – FC-5 Production Tolerances

Table 337-2 FC-5 Master Production Range	
Characteristic	Tolerance (1)
Asphalt Binder Content (%)	± 0.60 %
Passing 3/8 Inch Sieve (%)	± 7.5 %
Passing No. 4 Sieve (%)	± 6.0 %
Passing No. 8 Sieve (%)	± 3.5 %
(1) Tolerances for sample size n = 1 from the verified mix design	

Table 7.2 presents the factor-level combinations that were evaluated during the third experiment. Similar to Experiment 2, two aggregate sources were utilized; White Rock Quarries and Martin Marietta. Three gradations were evaluated and included the JMF gradation and then plus and minus the construction tolerances. Three asphalt binder contents were also be evaluated; JMF, plus construction tolerance (+0.6%), and less construction tolerance (-0.6%).

Table 7.2 - Factor-Level Combinations for Experiment 3

Factor	Levels
Aggregate	2 sources, same as Experiment 2
Gradation	3 Gradations: JMF, ± construction tolerances
Binder	2 Binders: PG 76-22, ARB-12
Binder Content	3 Binder Contents: JMF, ± construction tolerances
Gradation Size	1 Gradation sizes: FC-5 (12.5mm)
STOA	Short-Term Oven Aged (STOA)

The same four performance tests used in experiment 3 include the Overlay Tester, Hamburg Wheel Track test, Indirect Tensile Strength and the Cantabro Abrasion Loss.

To change the FC-5 gradations so that the effect of production tolerance could be evaluated, a change in the batch percentages was conducted within the mixtures respective aggregate blend. The research team believed that this best represented a potential production issue as opposed to manufacturing a gradation that would most likely not occur unless severe segregation and contamination of the aggregate stockpiles too place. Tables 7.3 and 7.4 show the resultant FC-5 gradations used in the laboratory experiment. It should be noted that the FC-5 mixtures containing the ARB-12 asphalt binder has an additional 0.6% above what is shown in the tables. FDOT generally increases the asphalt binder content by 0.6% for all FC-5 mixtures as currently optimum asphalt content is determined using a PG 67-22 asphalt binder during the Pie-Plate process. The additional 0.6% asphalt binder in FC-5 mixtures containing the ARB-12 is to accommodate for the small percentage of the asphalt binder that is actual crumb rubber and not liquid asphalt.

Table 7.3 – White Rock Quarries FC-5 Production Tolerances

Sieve Size	% Passing - White Rock Quarries			FDOT Production Tolerances
	(-) Tolerances	JMF	(+) Tolerances	
3/4" (19.0 mm)	100.0	100.0	100.0	
1/2" (12.5 mm)	82.7	86.2	89.8	
3/8" (9.5 mm)	60.6	67.8	75.1	±7.5 %
# 4 (4.75 mm)	19.1	22.8	26.4	± 6.0 %
# 8 (2.36 mm)	6.4	6.8	7.2	±3.5 %
# 16 (1.18 mm)	3.9	3.9	3.8	
# 30 (600 μm)	3.5	3.3	3.2	
# 50 (300 μm)	3.2	3.1	2.9	
# 100 (150 μm)	2.8	2.7	2.6	
#200 (75 μm)	2.4	2.3	2.2	
Asphalt Content (%) PG 76-22	5.4	6.0	6.6	± 0.60 %
Asphalt Content (%) ARB-12	6.1	6.7	7.3	± 0.60 %

Table 7.4 – Martin Marietta FC-5 Production Tolerances

Sieve Size	% Passing - Martin Marietta			FDOT Production Tolerances
	(-) Tolerances	JMF	(+) Tolerances	
3/4" (19.0 mm)	100.0	100.0	100.0	
1/2" (12.5 mm)	94.7	95.4	96.3	
3/8" (9.5 mm)	69.2	72.6	76.6	±7.5 %
# 4 (4.75 mm)	13.9	19.8	25.6	± 6.0 %
# 8 (2.36 mm)	6.7	9.0	9.7	±3.5 %
# 16 (1.18 mm)	5.0	6.6	7.0	
# 30 (600 μm)	4.0	4.9	5.1	
# 50 (300 μm)	3.4	3.9	3.9	
# 100 (150 μm)	3.0	3.2	3.3	
#200 (75 μm)	2.6	2.8	2.8	
Asphalt Content (%) PG 76-22	6.0	6.6	7.2	± 0.60 %
Asphalt Content (%) ARB-12	6.6	7.3	7.9	± 0.60 %

As shown in Tables 7.3 and 7.4, the sieve size controlling the production tolerances are actually different when modifying the stockpile blends naturally. For the White Rock Quarries FC-5 mixture, the 3/8 inch sieve almost fails the production tolerance due to changes in the blend proportions. Meanwhile, for the Martin Marietta FC-5 mixture, it is the #4 sieve that is on the verge of failing the production tolerances.

7.1 Cantabro Abrasion Loss Durability Testing

The Cantabro Abrasion Loss test was used to evaluate the durability of the FC-5 mixtures produced at the Job Mix Formula (JMF), as well as plus (+) and minus (-) the gradation and asphalt content production tolerances. A master summary chart showing the test results is shown as Figure 7.1, while the White Rock Quarries and Martin Marietta are shown separately as Figures 7.2 and 7.3, respectively.

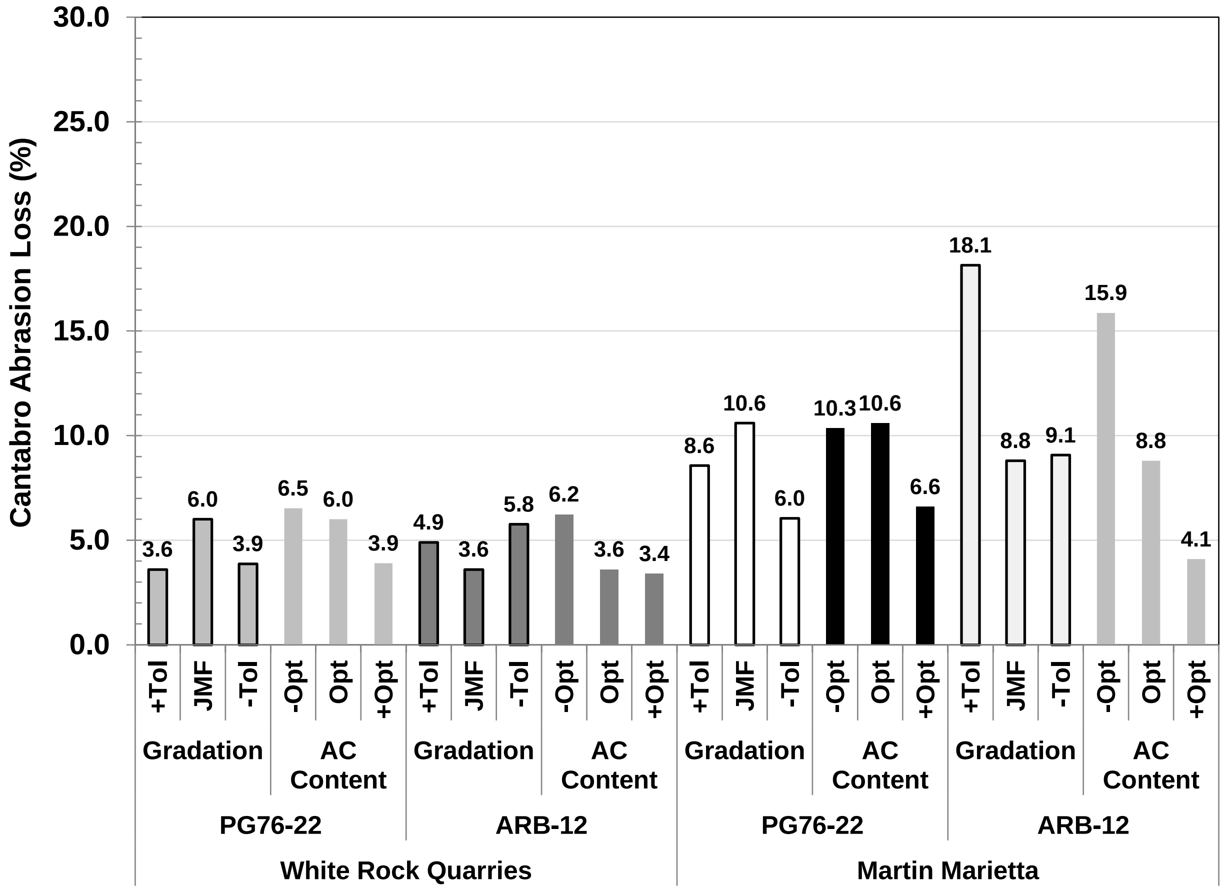


Figure 7.1 – Cantabro Abrasion Loss Results Due to Changes in Production Tolerances

A review of the White Rock Quarries FC-5 mixtures indicates that very little changes occur to the Cantabro Abrasion Loss results due to the changes in the production tolerances. There does appear to be a slightly greater Abrasion Loss when the asphalt content is on the low side of the production tolerance. However, other than that, it does not appear that the FDOT allowable gradation changes have influenced the durability of the White Rock Quarries FC-5 mixtures.

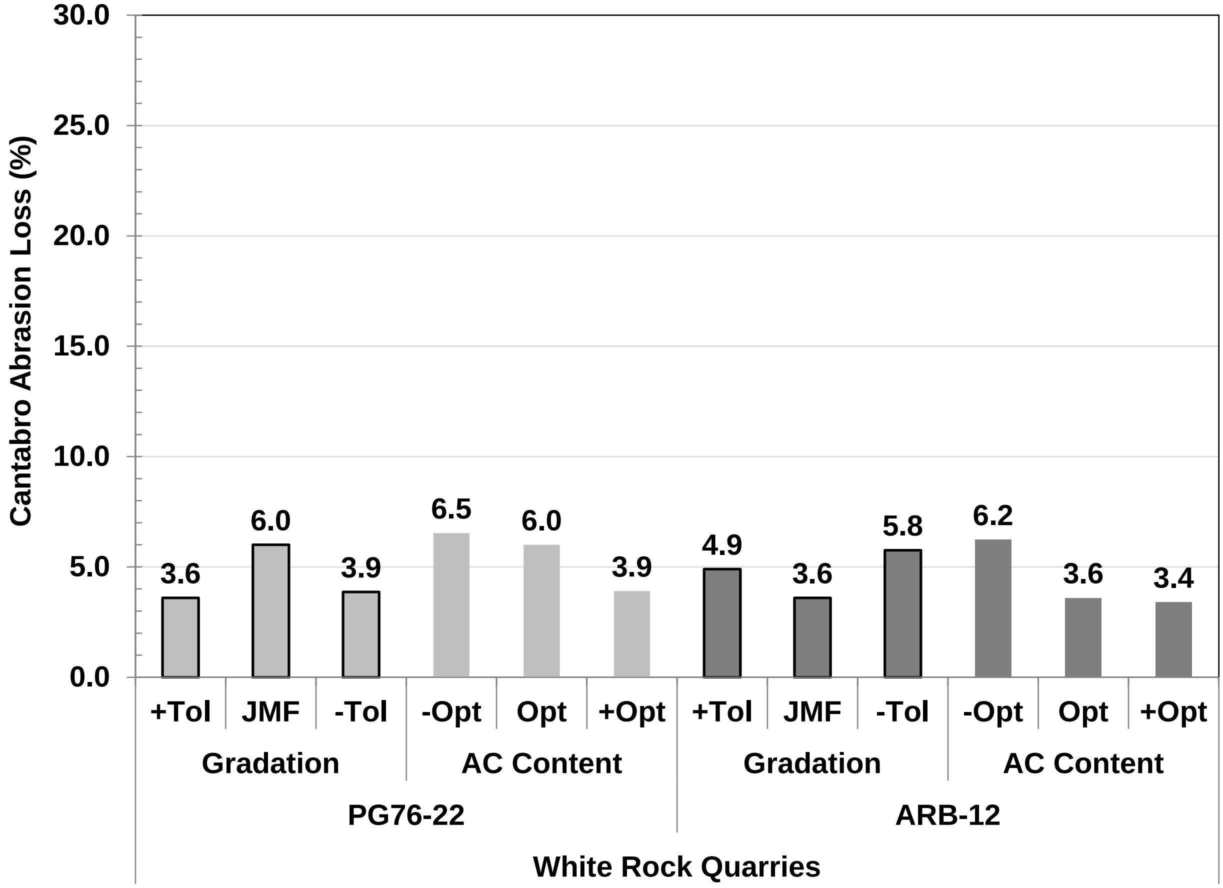


Figure 7.2 – Cantabro Abrasion Loss Results Due to Changes in Production Tolerances – White Rock Quarries

When reviewing the results for the Martin Marietta FC-5 mixtures, there appeared to be more abrasion loss, especially for the ARB-12 asphalt binder mixtures. For the PG 76-22 asphalt binder, the Martin Marietta FC-5 appeared to be insensitive to any production tolerance changes. However, there appeared to be a drastic abrasion loss for the ARB-12 mixtures when the asphalt content was low 0.6% and the aggregate gradation was on the fine side (+ Tol) of the gradation tolerance.

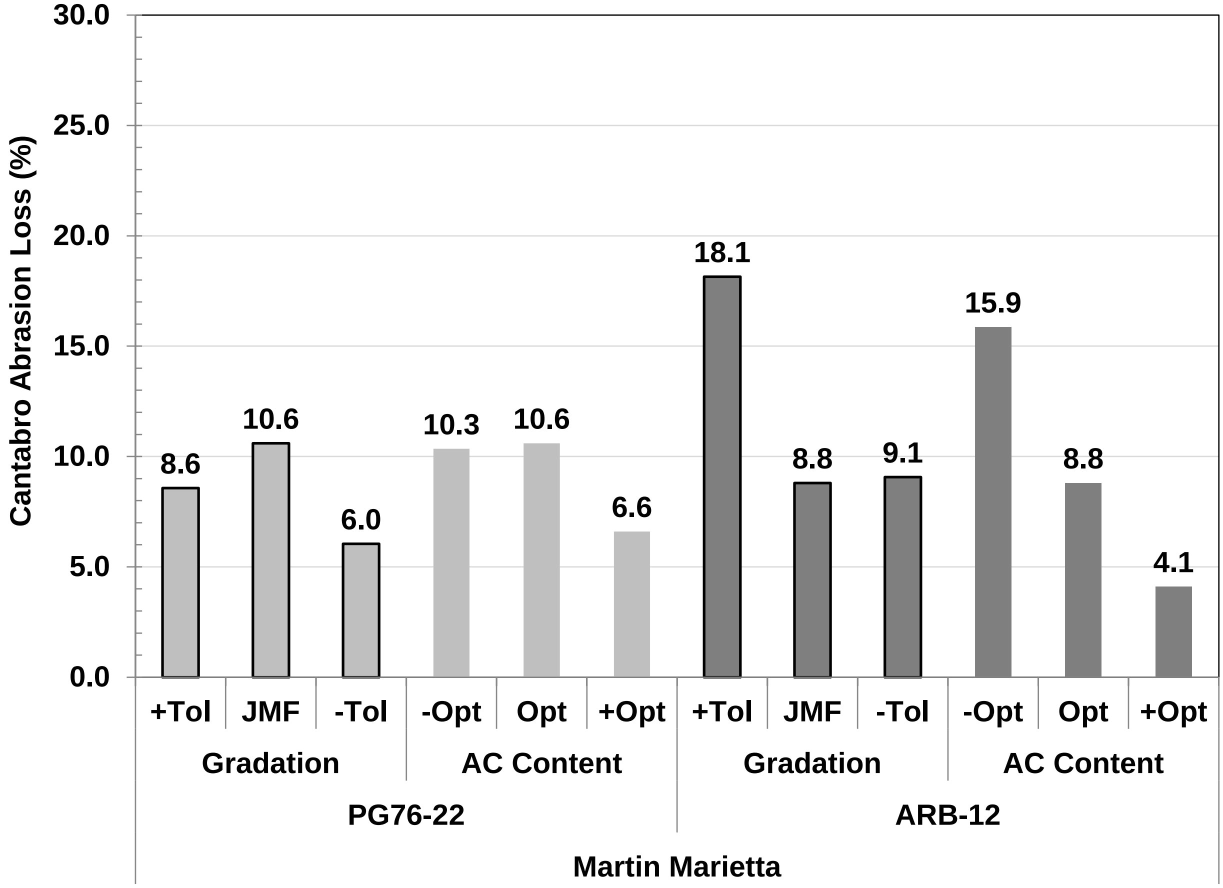


Figure 7.3 – Cantabro Abrasion Loss Results Due to Changes in Production Tolerances – Martin Marietta

7.2 Overlay Tester Fatigue Cracking

A summary of the Overlay Tester results due to changes in the production tolerances are shown in Figure 7.4, while the White Rock Quarries and Martin Marietta FC-5 mixtures are shown separately in Figures 7.5 and 7.6, respectively.

From Figure 7.4, it is quickly noted that the Overlay Tester results in the Martin Marietta FC-5 mixture is far superior to the White Rock Quarries – this was noted earlier in Chapter 6. The White Rock Quarries results, shown in more detail in Figure 7.5, show minimal changes due to the allowable production tolerances except that there was a slight reduction in the fatigue life when the asphalt content dropped 0.6%. The change in gradation production tolerance had minimal effect on the Overlay Tester results.

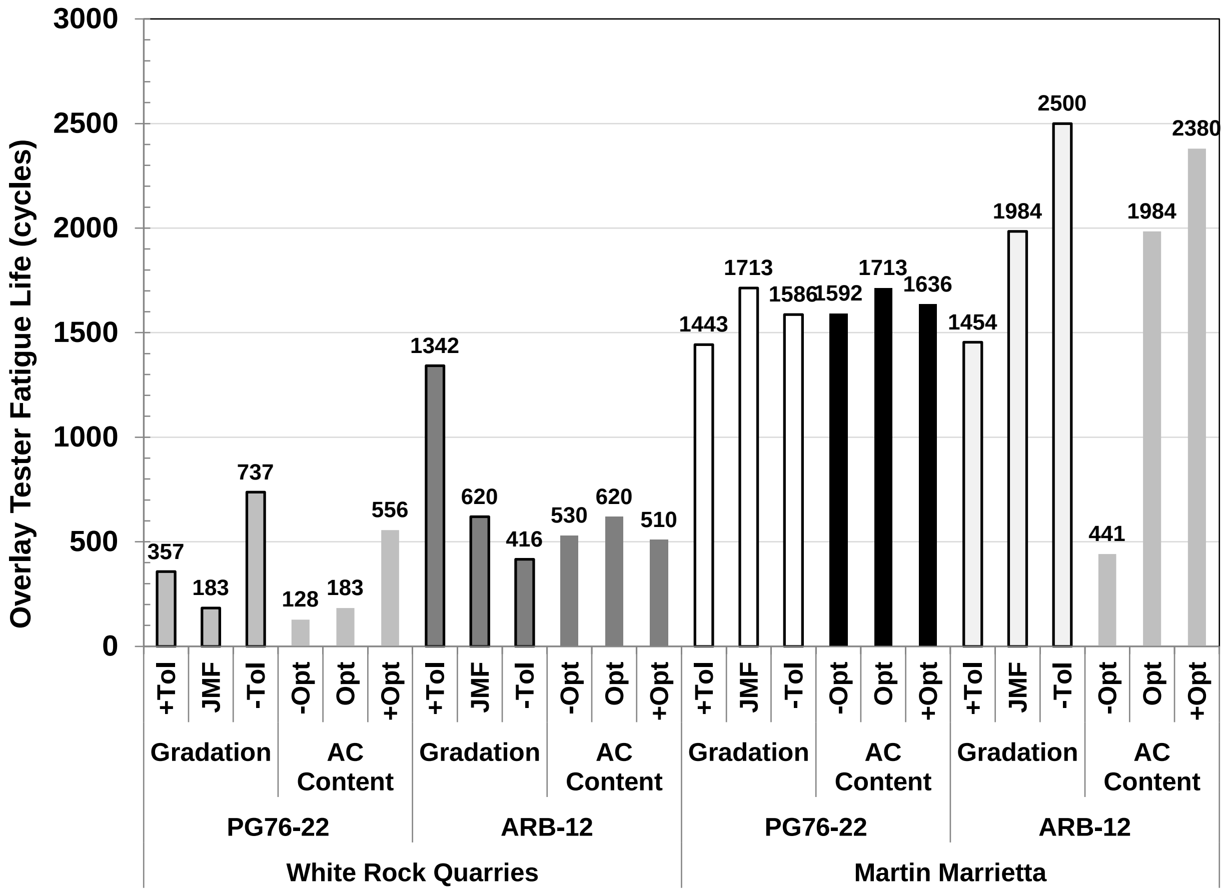


Figure 7.4 – Summary of Overlay Tester Results Due to Changes in Production Tolerances

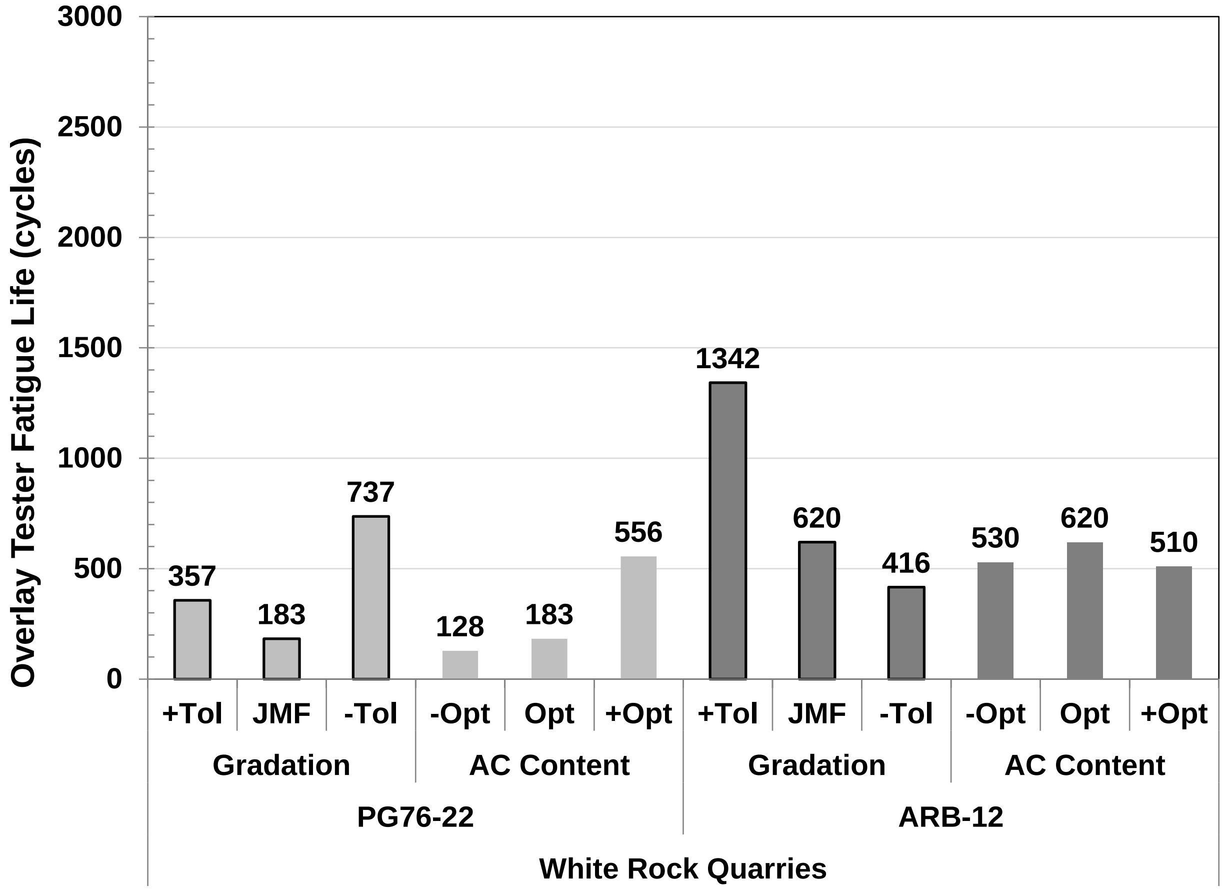


Figure 7.5 – Overlay Tester Results Due to Changes in Production Tolerances – White Rock Quarries

The Martin Marietta Overlay Tester results are highlighted in Figure 7.6. The test results show little to no change with the PG 76-22 asphalt binder. However, there were some noteworthy changes for the ARB-12 asphalt binder. As the aggregate gradation became finer (almost failing the production tolerance on the #4 sieve), there was a drop in the Overlay Tester fatigue results. However, it should be noted that even with the reduction in performance, the mixture still achieved a significantly high Overlay Tester result of 1,454 cycles. Meanwhile, the 0.6% reduction in asphalt content for the Martin Marietta ARB-12 FC-5 mixture resulted in a 75% reduction in the fatigue life.

It should be noted that the Martin Marietta ARB-12 FC-5 mixture that is shown to have experienced reduction in the Fatigue Life also had durability issues in the Cantabro Abrasion Loss test due to the exact production tolerance changes.

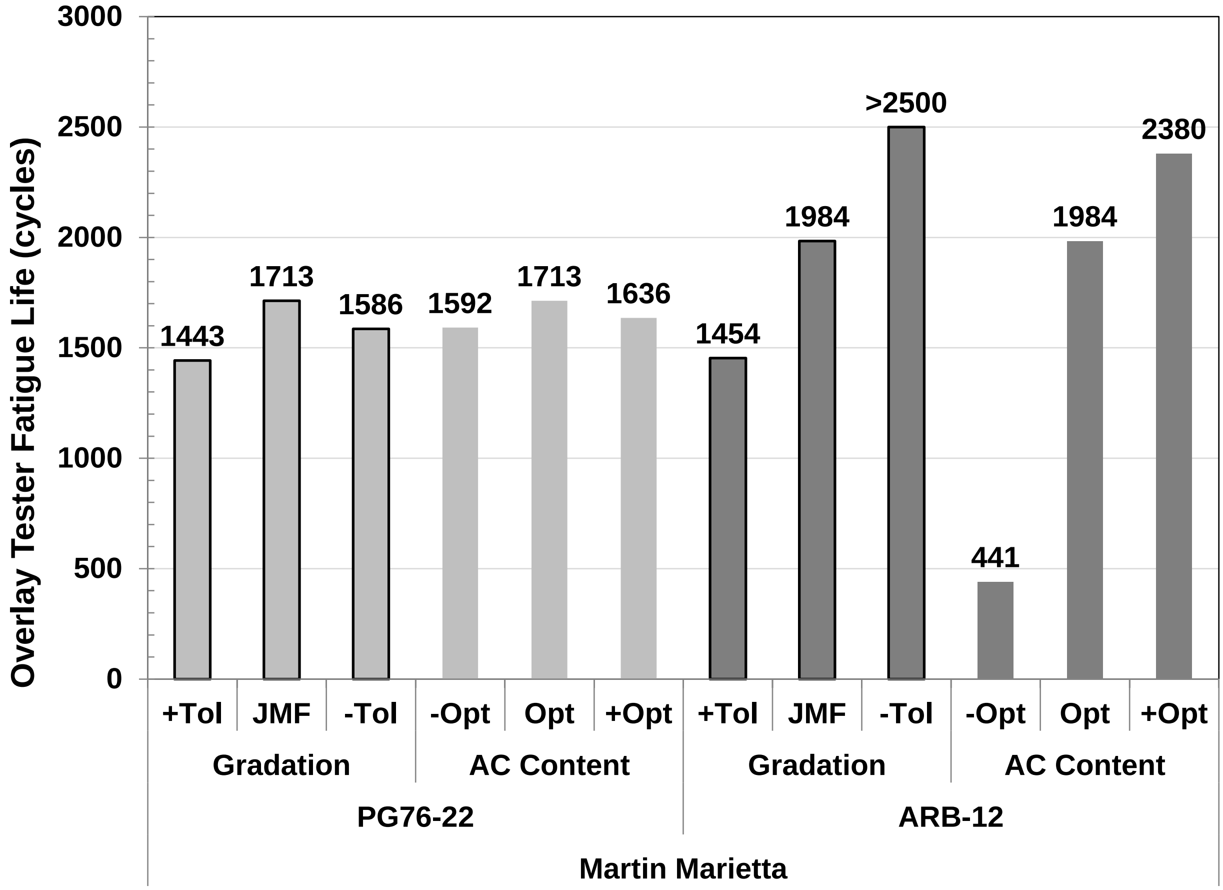


Figure 7.6 – Overlay Tester Results Due to Changes in Production Tolerances – Martin Marietta

7.3 Indirect Tensile (IDT) Strength

The IDT Strength was determined for the White Rock Quarries and Martin Marietta FC-5 mixtures that underwent a series of JMF modifications that were allowed to be within FDOT production tolerances shown earlier in Table 7.1. The master summary of IDT strength and FE are shown as Figures 7.8 and 7.9, respectively.

The changes with the asphalt content and gradations due to allowable production tolerances appear to have a greater influence on the IDT strength properties of the Martin Marietta FC-5 mixture, as opposed to the White Rock Quarries FC-5 mixtures. Overall, it seems clear that a reduction of 0.6% asphalt content reduces the IDT strength – this was found to occur in three of the four mixtures. The IDT Strength also appears to drop with either the change in gradation tolerance; plus (+) or minus (-). The Martin Marietta FC-5 mixture seemed to be more sensitive to the gradation tolerance changes than the White Rock Quarries FC-5 mixture.

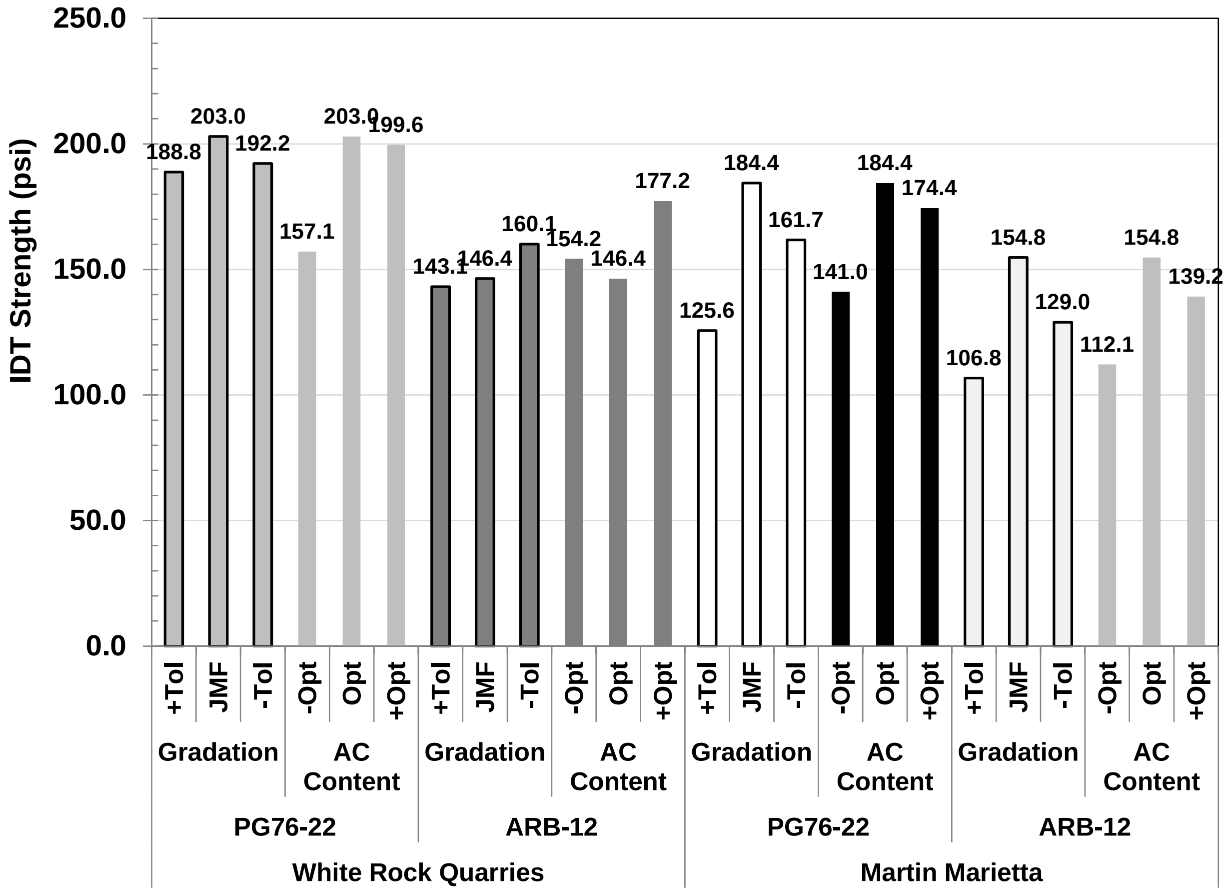


Figure 7.7 – IDT Strength for Due to Changes in Production Tolerances

The FE is shown in Figure 7.8. The calculated FE appeared to be sensitive to changes in the asphalt content as differences between the optimum asphalt content and the +/- 0.6% asphalt content FC-5 mixtures. When the asphalt content was 0.6% lower than the optimum asphalt content, the FE was shown to decrease, while as the asphalt content increased 0.6% above the Optimum asphalt content, the FE increased above the FE for the Optimum asphalt content mixture. Similar to the IDT Strength, the differences in the measured FE were found to be more significant for the Martin Marietta FC-5 mixtures with minor differences found with the White Rock Quarries FC-5 mixtures.

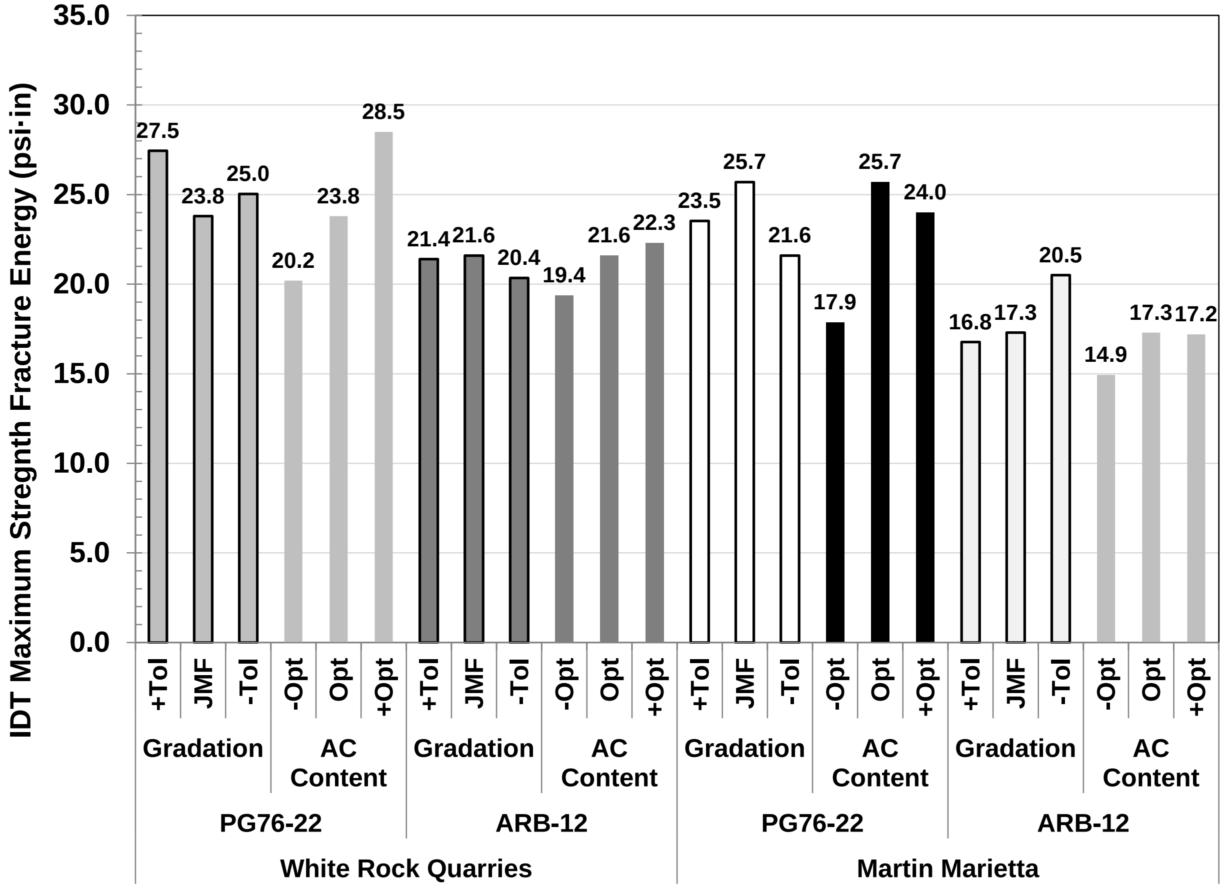


Figure 7.8 – IDT Maximum Strength Fracture Energy Due to Changes in Production Tolerances

7.4 Wet Hamburg Wheel Track Test

The Hamburg Wheel Tracking Device (HWTD) was used to evaluate the rutting potential of the different FC-5 mixtures due to allowable production tolerance gradation and asphalt binder contents. The cycles to 12.5 mm rutting is shown as Figure 7.9, while the cycles to SIP are shown as Figure 7.10. The rutting measured in the HWTD test was found to be very sensitive to the allowable changes in the production tolerances of the FC-5 mixtures. What was quite unusual when analyzing the test data was that the optimum and JMF mixtures, where no changes in production tolerances occurred, always achieved the highest rutting resistance for the respective FC-5 mixture and condition evaluated. Once any of the production tolerances were applied to the FC-5 mixture, clearly stability issues occurred with the Martin Marietta FC-5 mixtures showing more sensitivity to the production tolerance changes and overall poorer rutting resistance.

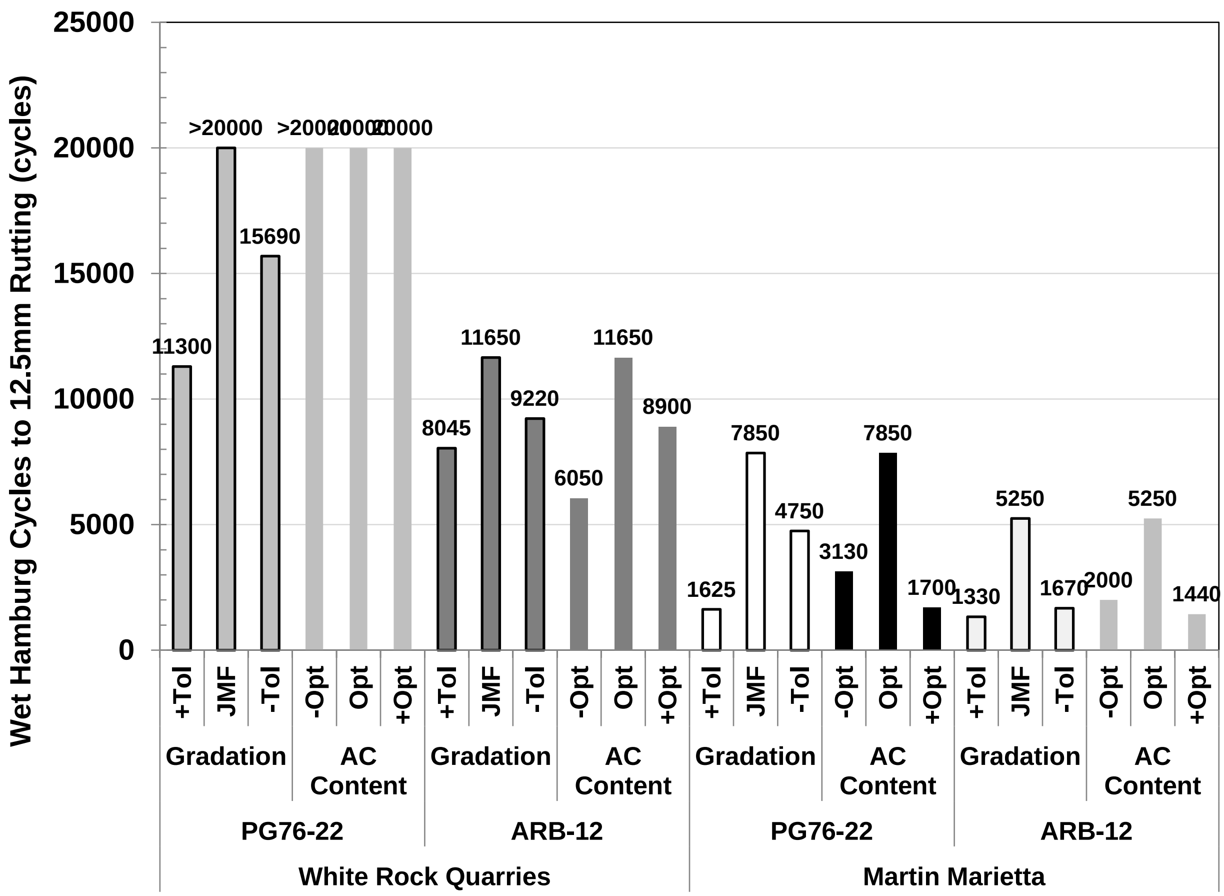


Figure 7.9 – Wet Hamburg Wheel Track Test Results Due to Changes in Production Tolerances – Cycles to 12.5 mm Rutting

The SIP calculated with the Hamburg rutting information is shown in Figure 7.10. Again, similar to the rutting shown earlier in Figure 7.9, the SIP appears to be sensitive to the allowable production tolerances of the FC-5 mixtures. Greater changes in the SIP due to the allowable production tolerances were found in the Martin Marietta FC-5 mixtures than the White Rock Quarries FC-5 mixture. In general, the Martin Marietta mixture performed poorly when compared to the White Rock Quarries FC-5 mixture.

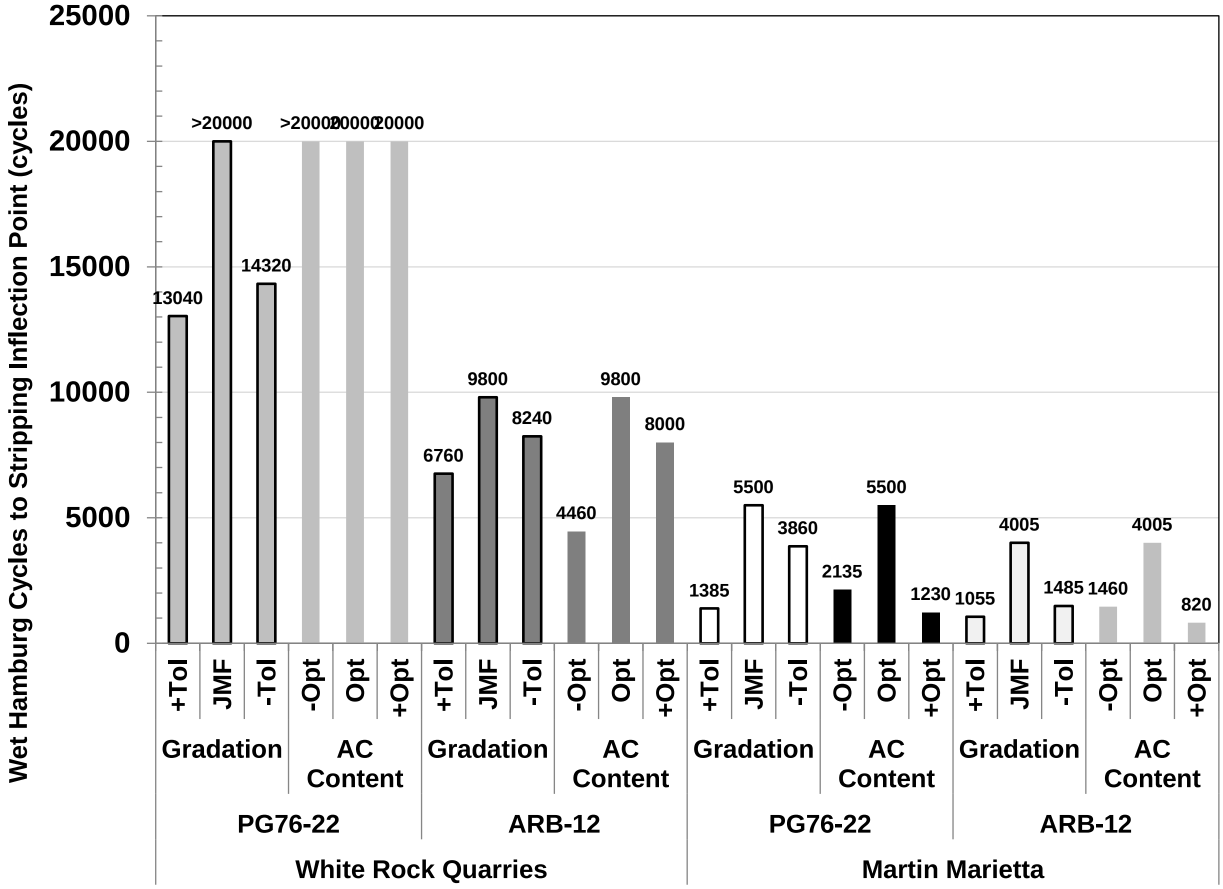


Figure 7.10 – Wet Hamburg Wheel Track Test Results Due to Changes in Production Tolerances – Stripping Inflection Point (SIP)

7.5 Observations from Experiment 3

The White Rock Quarries and Martin Marietta FC-5 mixtures were artificially modified to so that the asphalt content and aggregate gradation reflected allowable production tolerance deviations from the Job Mix Formula. The asphalt content was adjusted to be 0.6% above and below the Optimum asphalt content. Meanwhile, the aggregate gradations were modified by changing the blend percentages until the final aggregate blend almost failed the allowable production tolerances shown in Table 7.1. For the White Rock Quarries aggregate blend, the production tolerances were controlled by the 3/8 inch sieve. For the Martin Marietta aggregate blend, the production tolerances were controlled by the #4 sieve.

The influence of the production tolerances was evaluated using the Cantabro Abrasion Loss test. Minimal durability changes were measured for the White Rock Quarries FC-5 mixtures due to the allowable production tolerances ranges. However, the Martin Marietta FC-5 mixture was found to have a large increase in abrasion loss when asphalt content was 0.6% low and the aggregate gradation shifted towards the finer side of the tolerances (+ Tol).

The Overlay Tester was used to evaluate how the fatigue performance changes with the allowable production tolerances. The Overlay Tester fatigue results did not appear to be significantly affected by the gradation changes. However, FC-5 mixtures produced 0.6% below the optimum were found to decrease in fatigue resistance, while FC-5 mixtures produced 0.6% above the optimum had an increase in fatigue resistance.

The IDT strength and FE was also used to evaluate the tensile strength of the FC-5 mixtures due to production tolerances changes. The IDT strength appeared to be sensitive to the production tolerances changes as mixtures that met the JMF conditions (optimum asphalt content and JMF gradation) achieved the highest IDT strength for that respective mixture condition. When asphalt content was +/- 0.6% and the FC-5 gradation was modified to the allowable production tolerances, the maximum IDT strength dropped.

The Hamburg Wheel Track test was used to evaluate the rutting potential of the FC-5 mixtures after the allowable production tolerances were applied to the FC-5 mixtures. The number of cycles to 12.5 mm of rutting measured in the Hamburg was found to be very sensitive to the allowable production tolerances changes applied to the FC-5 mixtures. Similar to the IDT strength, mixtures produced at the optimum asphalt content and gradations meeting the JMF achieved the highest number of cycles before 12.5 mm of rutting in the Hamburg test. The greatest reduction in rutting resistance was found in the Martin Marietta FC-5 mixtures.

CHAPTER 8 – CONCLUSIONS

The FDOT is utilizing their FC-5 porous friction course asphalt mixtures because of the benefits they provide in enhancing wet weather driving safety. However, FDOT is looking to try and increase the life of their FC-5 asphalt mixtures as their current expected field life is approximate 12 – 14 years. FDOT has reported that the major reason for FC-5 rehabilitation has been primarily due to raveling, with the second major cause being fatigue cracking.

A research study was conducted to evaluate whether or not the FDOT's current FC-5 mixtures could be enhanced to improve their cracking and durability performance, which is hopeful to translate to longer field performance. In an effort to evaluate how the durability and fatigue performance of Florida's FC-5 mixtures can be improved, the research team developed and executed an extensive workplan that contained the following components:

- Comprehensive literature review pertaining to performance of porous friction course mixtures;
- Data mining of FDOT's Pavement Management System (PMS) to determine general performance of FC-5 mixtures;
- Data mining of FDOT's Laboratory Information Management System (LIMS) for FC-5 mixture components to determine if a relationship exists between field performance and mixture components;
- Conducted a laboratory workplan to;
    - Evaluate if asphalt content determined by the Pie-Plate method could be enhanced and how it relates to other accepted practices;
    - Evaluate if a possible change in FC-5 gradation can improve mixture performance;
    - Evaluate how changes to FDOT allowable FC-5 production tolerances influence the performance of the FC-5 mixtures.

The conclusions of the research study are summarized below.

8.1 Pavement Management Data Relating to FC-5 Mixtures and Performance

A data mining exercise was conducted that incorporated the field performance of the FC-5 mixtures with respect to cracking and raveling. Test sections were selected that provided sufficient field performance so degradation curves could be generated. The respective FC-5 mixtures placed on the noted field sections were then "data mined" and their respective mixture components and volumetric properties recorded. Along with the FC-5 mixture properties for the field sections, traffic conditions was also incorporated to see if traffic conditions were able to "normalize" the data so a particular FC-5 volumetric or component could be identified as a controlling factor for cracking and durability performance. The parameters and analysis utilized is noted in detail in Chapter 3.

The results of the PMS data mining exercise clearly identified the effective asphalt content as the main controlling variable that was influencing the measured field cracking and raveling. Figure

8.1 again shows this relationship. Note that it is the effective asphalt content and not the total asphalt content in the figure. Therefore, to help increase the fatigue cracking resistance and overall durability of the FC-5 asphalt mixtures, the effective asphalt content of the FC-5 mixtures must be increased.

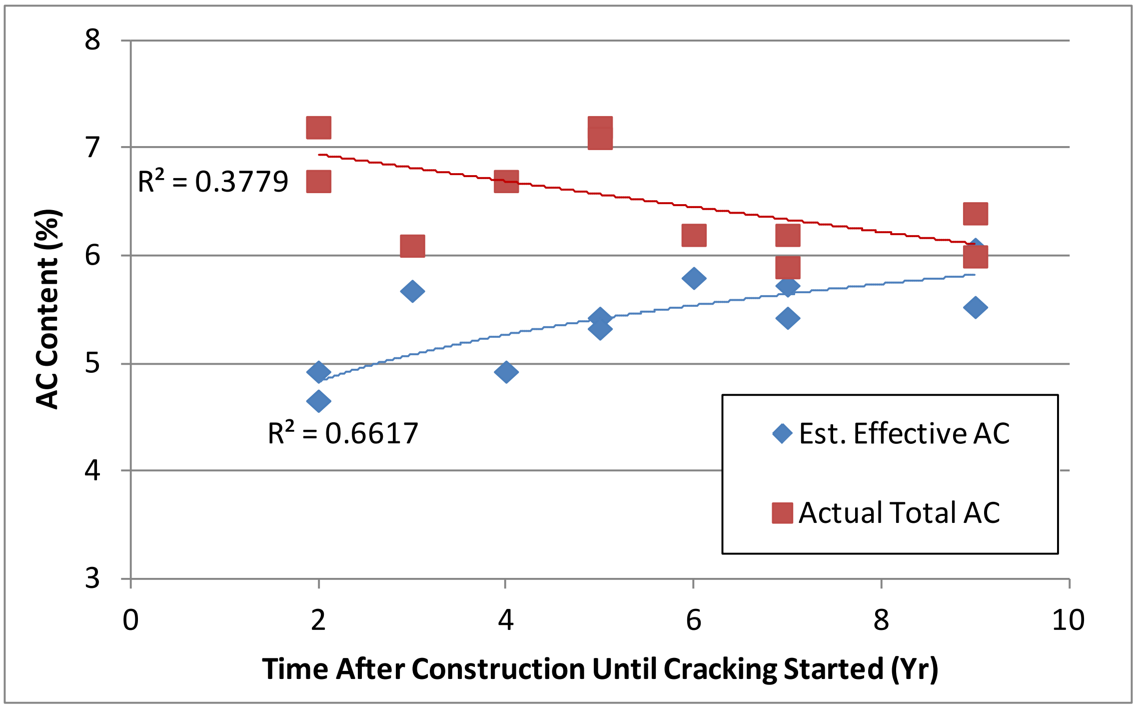


Figure 8.1 – Asphalt Binder Parameters vs Time After Construction Until Cracking Started

8.2 Field Visit to Selected FC-5 Test Sections

A field visit was conducted to FC-5 test sections noted during the PMS data mining exercise to help get a better "feel" to the type and level of cracking and raveling that was being recorded in the PMS system. The field visit was also used to train the Research Team in conducting the Pie-Plate test procedure for later laboratory experiments.

The most common distress observed with the FC-5 wearing course was raveling. Two issues were observed related to the raveling with the FC-5 sections. First, the most common form of raveling observed was what appeared to be "end of load" issues. These occurrences of raveling were cyclical down the roadway at approximately equal distances apart. This form of raveling is most likely associated with some form of segregation. However, it is unclear whether the segregation is physical or thermal. Raveling of this nature is considered to be a construction related issue and not a material issue. The second issue related to raveling was raveling across the entire pavement surface. The occurrence of this type of raveling was not as prevalent as the "end of load" form of raveling. However, this type of raveling is likely a material related problem. A number of the FC-5 sections evaluated had the appearance of being under-asphalted (i.e. – gray color and dull appearance). It is common after the construction of an FC-5 mix that asphalt binder will be worn from the pavement surface due to the action of tires. However, visual observation of many of the FC-5 mixes suggested that asphalt binder below the pavement surface was minimal.

Cracking was not a predominant distress observed within the FC-5 mixture. These cracks were generally low severity and located between the inside wheel path and lane skip stripe. The next most common type of crack observed was associated with pavement scars. The scars appeared to be caused by vehicles with flat tires passing over the pavement surface, sometimes for great distances. Generally, these types of surface abrasions were also low severity. A very small percentage of pavements had longitudinal cracks. In these instances, namely SR-20 just west of Hawthorne and I-75 in Marion County, it is unclear whether the cracks are top-down or were reflected upward due to issues within an underlying layer. In the case of I-75, in Marion County, it appeared the cracks were caused by an underlying issue, but that is not 100% certain without forensic investigation. In summary, based upon the roadways travelled, performance of the FC-5 layers with respect to cracking appeared to be good.

Based on the field visit, it seemed apparent that fatigue cracking associated with traffic loading was not the major issue associated with the field performance of the FC-5 mixtures. This was also noted in Chapter 3 during the data mining exercise. The most prevalent distress was raveling, which may or may not be mixture component issue. In some cases noted, the raveling was due to “end of load” issues that can be addressed and corrected with the use of a Material Transfer Vehicle (MTV). However, the fact that some sections also appeared to be gray or dull in appearance indicates that the effective asphalt, the asphalt not absorbed by the stone, may also be playing a role in the raveling issues.

8.3 FC-5 Mixture Design - Pie Plate Evaluation

As noted earlier, it was evident from the collected PMS, SMO’s mix design database, and LIMS information that as the effective asphalt content increased, the durability and cracking performance increased as well. Current practice by FDOT for determining the Optimum asphalt content of FC-5 mixtures is to utilize the Pie-Plate procedure, FM 5-588, *Determining the Optimum Asphalt Binder Content of an Open-Graded Friction Course Mixture Using the Pie Plate Method*. A 12.5 mm nominal maximum aggregate size gradation conforming to FDOT’s FC-5 gradation band is mixed with a PG 67-22 asphalt binder, along with fibers, is mixed at mixing temperatures and directly placed in a glass pie-plate. Ultimately, the amount of asphalt binder remaining on the pie-plate once the FC-5 mixture is removed dictates the binder condition:

- Insufficient Bonding/Drainage – in this case, more asphalt binder is required
- Sufficient Bonding/Drainage – determined as Optimum Asphalt Content
- Excessive Bonding/Drainage – too much asphalt binder in mixture

The Research Team looked at attempting to enhance the Pie-Plate procedure by looking at two main factors; 1) Allow for absorption to take place through a 2 hour loose mix volumetric conditioning and 2) Utilizing the asphalt binder the mixture is indicated to be produced and placed with (i.e. – PG 76-22 or ARB-12 asphalt binder). The Pie-Plate procedure was also compared with the Draindown Test (AASHTO T 309) and Cantabro Abrasion Loss test to look at a possible combined method of meeting both draindown and mixture durability to help “balance” the mixture design.

The laboratory testing showed that the 2 hour volumetric conditioning resulted in variable Draindown test results due to actual draindown occurring during the conditioning. It was therefore concluded not to include the volumetric conditioning phase during Pie-Plate or Draindown testing.

However, the use of the appropriate binder type (i.e. – PG 76-22 or ARB-12 asphalt binder) was found to improve the Cantabro Abrasion Loss performance at asphalt binder contents above what the pie-plate test procedure determined as “Excessive”. This can be explained by the viscosity of the asphalt binders utilized during pie-plate or draindown testing. As the viscosity of the asphalt binder increases, it will have a tendency to adhere to the aggregate source more and resist draindown. For example, the asphalt binders’ rotational viscosity was measured at the identical temperature used during the pie-plate and draindown testing. The resultant rotational viscosity measurements were recorded:

- ARB-12 Asphalt Binder: 600 cP
- PG 76-22 Asphalt Binder: 477 cP
- PG 67-22 (Currently used during Pie Plate testing): 194 cP

The rotational viscosity measurements show that ARB-12 has a viscosity over 3 times that of the PG 67-22 and the PG 76-22 is almost 2.5 times that of the PG 67-22. Therefore, it is logical that better adhesion, or lower draindown, would occur with the use of a PG 76-22 or ARB-12 asphalt binder.

Therefore, the research in this study suggests that by utilizing the PG 76-22 or ARB-12 asphalt binder during the pie-plate procedure to determine the optimum asphalt binder content of the FC-5 mixture, increased asphalt binder contents would naturally occur, thereby increasing the effective asphalt content of the FC-5 mixtures. And although the addition of asphalt binder may produce a mix more prone to draindown issues in the field, the field visit by the research team noted minimal to no signs of field draindown occurring.

8.4 Possible Use of Finer FC-5 Mixtures – 9.5 mm vs 12.5 mm NMAS

The research team looked at the possible use of a finer FC-5 mixture and whether or not this could enhance the durability and fatigue resistance of the FC-5 mixtures above the current. Based on the previous NCHRP work by Cooley et al (2009), 9.5 mm NMAS mixtures were produced using the White Rock Quarries and Martin Marietta aggregates. The 9.5 mm FC-5 mixtures were evaluated side by side to the 12.5 mm FC-5 mixtures using the same aggregate sources (i.e. – White Rock Quarries and Martin Marietta). The 9.5 mm and 12.5 mm NMAS FC-5 mixtures were evaluated for durability (Cantabro Abrasion Loss), fatigue cracking (Overlay Tester), tensile strength (IDT Strength) and rutting potential/stability (Hamburg Wheel Track test). The test specimens were conditioned using both STOA and LTOA conditions in accordance with AASHTO R 30 to determine if differences existed due to aging condition.

For the STOA aged condition, the durability performance depended on the aggregate source/FC-5 mixture design. The 12.5 mm NMAS mixture for the White Rock Quarries outperformed its 9.5 mm NMAS mixture. Meanwhile, the 9.5 mm NMAS for Martin Marietta was shown to have better durability than the 12.5 mm counterpart. However, it should be noted that all the abrasion loss values were well below the 15% Abrasion Loss typically specified by state agencies. During the fatigue cracking evaluation with the Overlay Tester, it was found that on average, the 9.5 mm NMAS mixtures outperformed their 12.5 mm NMAS comparisons. Meanwhile, the tensile strength parameters were found to be better for the 12.5 mm NMAS than the 9.5 mm mixtures. When comparing the rutting performance of the FC-5 mixtures in the Hamburg Wheel Tracking test, the 12.5 mm NMAS well outperformed the 9.5 mm NMAS mixtures, especially the 9.5 mm NMAS Martin Marietta FC-5 mixtures containing ARB-12. Based on the Hamburg test results, it appears that the 9.5 mm ARB-12 Martin Marietta FC-5 mixtures may have some slight instability issues.

For the LTOA aged conditions, the general trend in results noted for the STOA samples were found. However, the test results did show that FC-5 mixtures produced 0.6% above the optimum asphalt content performed better in the Overlay Tester, indicating they were not as susceptible to accelerated fatigue cracking due to oxidative aging. Mixtures with the ARB-12 asphalt binder were found to be even more resistant to the oxidative aging accelerated fatigue cracking than the PG 76-22 asphalt binder. IDT strength testing showed to improve with aging. However, when analyzing the data using a FE approach, it was shown that the FE of the FC-5 mixtures decreased due to aging – which mirrors what occurs in the field better than the IDT strength. When using the FE to compare the performance, the 9.5 mm and 12.5 mm NMAS FC-5 mixtures performance decreased almost identically due to the additional aging. Once again, the ARB-12 asphalt binders were found to limit the degradation of FE due to additional aging in the FC-5 mixtures evaluated. Meanwhile, due to the additional aging, the rutting resistance in the Hamburg Wheel Tracking test was found to improve. This was expected as the mixtures stiffened during the aging process. However, even with the stiffening due to the additional aging, the 9.5 mm NMAS mixtures still did not perform as well as the 12.5 mm NMAS FC-5 mixtures.

Based on the performance testing comparisons between the 9.5 mm and 12.5 mm NMAS FC-5 mixtures, even though the 9.5 mm NMAS FC-5 mixtures helped to improve the fatigue cracking performance of the FC-5 mixtures, there is some concern over the general stability and rutting potential of these mixtures, especially when incorporating the ARB-12 asphalt binder. Currently in FDOT's FC-5 design procedure, the design engineers are well accustomed to the appropriate aggregate gradations of the 12.5 mm NMAS FC-5 mixtures and stone-on-stone contact is assumed. However, since no field experience exists with the 9.5 mm NMAS FC-5 mixture proposed in this project (although FDOT has had previous experience with a 9.5 mm FC-5 mixutre), it is unclear whether the gradation band selected in this study provides the necessary stone-on-stone contact required in gap-graded mixtures (i.e. – OGFC or SMA). Therefore, the research team believes that possibly the reason for the stability/rutting issues of the 9.5 mm NMAS FC-5 mixtures, especially for the ARB-12 asphalt binder, may be due to not achieving stone-on-stone contact. And considering the 9.5 mm NMAS FC-5 is a finer gradation, residual crumb rubber particles in the ARB-12 asphalt binder may actually be pushing the aggregate skeleton further apart. Figure 8.2 indicates just how severe the Hamburg rutting was at a very low number of loading applications. Therefore, before moving forward with additional 9.5 mm NMAS FC-5 mixture work, further evaluation of approach FC-5 gradations should be conducted with the inclusion of the Voids in Coarse Aggregate (VCA) approach to ensure stone-on-stone contact is being achieved. It should also be noted that the VCA testing should be conducted with the asphalt binder proposed for use during production. This would ensure that if an ARB-12 asphalt binder is to be used, it will not be detrimental to the stone-on-stone FC-5 skeleton. Currently, FDOT does not include a means of designing for and/or verifying stone-on-stone contact in their FC-5 mixtures.

Figure 8.2 – Failed 9.5 mm NMAS FC-5 Specimens with ARB-12 Asphalt Binder

8.5 Influence of FDOT Production Tolerance on FC-5 Mixtures

The FDOT allowable Production Tolerances for FC-5 mixtures was evaluated for the 12.5 mm NMAS White Rock Quarries and Martin Marietta JMF. The allowable production tolerances are controlled by the asphalt content (+/- 0.6% from the optimum), as well as gradations on the 3/8 inch, #4 and #8 sieves. It was decided by the research team to produce specimens that were "naturally" close to the production tolerances by adjusting the blend percentages of the stockpiles, as opposed to manufacturing a gradation at the production tolerance that is unrealistic and would not occur at the plant. By doing so, it was found that the White Rock Quarries FC-5 mixture aggregate gradation was controlled by the production tolerance on the 3/8 inch sieve, while the Martin Marietta FC-5 mixture aggregate gradation was controlled by the #4 sieve.

When evaluating the influence of the production tolerances on the Cantabro Abrasion Loss, there were minimal differences in the White Rock Quarries FC-5 mixtures. However, the Martin Marietta FC-5 mixture was found to have a large increase in abrasion loss when asphalt content was 0.6% low and the aggregate gradation shifted towards the finer side of the tolerances (+ Tol). The Overlay Tester did not appear to be sensitive to any of the gradation production tolerance changes but did show a dramatic decrease in the fatigue performance when that ARB-12 asphalt binder was 0.6% below optimum. The IDT strength appeared to be sensitive to the production tolerance changes as mixtures that met the JMF conditions (optimum asphalt content and JMF gradation) achieved the highest IDT strength for that respective mixture condition. When asphalt content was +/- 0.6% and the FC-5 gradation was modified to the allowable production tolerance, the maximum IDT strength dropped. The number of cycles to 12.5 mm of rutting measured in the Hamburg was found to be very sensitive to the allowable production tolerance changes applied to the FC-5 mixtures. Similar to the IDT strength, mixtures produced at the optimum asphalt content and gradations meeting the JMF achieved the highest number of cycles before 12.5 mm of rutting in the Hamburg test. The greatest reduction in rutting resistance was found in the Martin Marietta FC-5 mixtures.

In almost all performance testing, it is evident that as the asphalt binder decreased to the 0.6% allowable production tolerance, performance suffered. The test results also indicated that when the production tolerance on the aggregate blend went finer (+ Tol), the mixtures containing the ARB-12 asphalt binder dropped in performance, especially the Martin Marietta FC-5 mixtures. Knowing that residual crumb rubber particles still exists in the ARB-12 asphalt binder, there needs to be sufficient void space between the aggregate particles to allow the crumb rubber to reside without possible pushing the aggregate skeleton apart. In the case of the (-) Tolerance aggregate gradation, the aggregate gradation migrates from a more open structure to something that closes up, reducing the void space between the aggregates. This results in the crumb rubber particles “fighting” for space within the aggregate structure. And the fact that the Martin Marietta FC-5 mixture aggregate blend was controlled by the #4 sieve production tolerance may indicate a greater potential for the fine crumb rubber to be affected by the reduction in void space. Since stone-on-stone contact is not verified using a VCA method, it is highly likely that when the aggregate gradation moves towards the finer side of the production tolerance, mixtures containing crumb rubber modified binder may not be achieving stone-on-stone contact, resulting in the instability shown during the Hamburg Wheel Track testing.

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