Final Report

EVALUATION OF THICK OPEN GRADED AND BONDED FRICTION COURSES FOR FLORIDA

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UNIVERSITY OF FLORIDA

Bjorn Birgisson
Reynaldo Roque
Arvind Varadhan
Tung Thai
Lokendra Jaiswal

Department of Civil and Coastal Engineering
College of Engineering
365 Weil Hall, P.O. Box 116580
Gainesville, FL 32611-6580
Tel: (352) 392-9537 SunCom: 622-9537
Fax: (352) 392-3394

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16. Abstract Due to the frequency of high intensity short-duration rainfall events in Florida, vehicular hydroplaning is a serious concern, especially at the Interstate and other limited access facilities. Along with pavement cross-slope and rutting, the surface texture of an asphalt pavement plays a critical role in the prevention of hydroplaning on high-speed, multi-lane facilities. In order to minimize problems of this nature, Florida, in the mid 1970's, developed a friction course (FC-2) which was a 3/8-inch Nominal Maximum Aggregate Size (NMAS) open graded mixture (with polish resistant aggregate) placed approximately ½ inch thick. In the1990's, the FC-2 was eventually replaced by a slightly coarser open graded friction course (FC-5), which is a ½ inch NMAS mixture, placed approximately ¾ inch thick. A number of European countries, as well as several states in the US (Georgia, Oregon, Texas) have developed a "porous friction course" which is an open graded friction course placed in thicknesses ranging from 1¼ to 2 inches thick. The combination of the high in-place air void content of this mixture, coupled with the thickness at which it is placed, gives this type of pavement surface a great deal of potential with regard to storage and drainage of water during a rainstorm. However, there are also a number of questions associated with these types of pavements, such as durability, rutting resistance, and long-term porosity.

This report documents the development of a new mixture design procedure for open graded friction courses and thick porous friction courses in Florida. A new draft Florida method for open graded and porous friction courses is developed, and the groundwork is laid for a new draft specification for porous friction courses in Florida. Porous friction course mixtures prepared according to the new mix design are evaluated for fracture and rutting resistance in the laboratory, as well as on an actual field test project on I-295, in Duval County, Florida. This report also documents the evaluation of "bonded" open graded friction courses from US-27, Highlands County, Florida that are placed with a thick polymer modified tack coat. The performance of these bonded open graded friction courses are compared to open graded friction courses laid with a regular tack coat, as well as to a Novachip mixture, which is basically a stone matrix asphalt mixture with a thick polymer-modified tack coat. The results showed that the introduction of thick polymer-modified tack coat resulted in an increased rutting and cracking resistance, with no adverse changes to pavement friction and noise characteristics.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		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	Mg
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celcius temperature	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	l
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
psi	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	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
		VOLUME		
ml	milliliters	0.034	fluid ounces	fl oz
l	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg	megagrams	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	psi

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

(Revised August 1992)

TABLE OF CONTENTS

LIST OF TABLES

Page

LIST OF FIGURES

Page

EXECUTIVE SUMMARY

Due to the frequency of high intensity short-duration rainfall events in Florida, vehicular hydroplaning is a serious concern, especially on the Interstate and other limited access facilities. Along with pavement cross-slope and rutting, the surface texture of an asphalt pavement plays a critical role in the prevention of hydroplaning on high-speed, multi-lane facilities. In order to minimize problems of this nature, Florida, in the mid 1970's, developed a friction course (FC-2) which was a 3/8-inch Nominal Maximum Aggregate Size (NMAS) open graded mixture (with polish resistant aggregate) placed approximately ½-inch thick. In the1990's, in an effort to improve hydroplaning resistance, the FC-2 was replaced by a slightly coarser open graded friction course (FC-5), which is a ½-inch NMAS mixture, placed approximately ¾-inch thick. However, the additional water storage capacity of FC-5 mixtures has never been quantified. Visual observations of this surface type indicate that the while it is better than the old FC-2, it will still "fill-up" with water – resulting in water ponding on the pavement surface.

A number of European countries, as well as several states in the US (Georgia, Oregon, Texas) have developed a "porous friction course" which is an open graded friction course placed in thicknesses ranging from 1-1/4 to 2 inches thick. The combination of the high in-place air void content of this mixture, coupled with the thickness at which it is placed, gives this type of pavement surface a great deal of potential with regard to storage and drainage of water during a rainstorm. However, there are also a number of questions associated with these types of pavements, such as durability, rutting resistance, and long-term porosity. The first part of this project focuses on the development of thick open graded friction course mixture design for Florida.

The second part of this project focused on the performance evaluation of bonded friction courses that use a special paver to lay a heavy polymer-modified tack coat immediately in front

of the hot mix mat. The ongoing Longitudinal Wheel path Cracking study at UF has confirmed that a significant amount of the distress on high-volume Florida pavements is caused by surface initiated wheel path cracking from lateral stresses generated by radial truck tires. Based on these studies, there is a high likelihood that the bonded friction course process could significantly extend the crack resistance life of open graded mixes in Florida. It is also highly likely that raveling resistance of the friction course will be improved as well.

This report documents the development of a new mixture design procedure for open graded friction courses and thick porous friction courses. A new draft Florida method for open graded and porous friction courses is developed, and the groundwork is laid for a new draft specification for porous friction courses in Florida. Porous friction course mixtures prepared according to the new mix design procedure are evaluated for fracture and rutting resistance in the laboratory, as well as on an actual field test project on I-295, in Duval County, Florida. This report also documents the evaluation of "bonded" open graded friction courses from US-27, Highlands County, Florida that are placed with a thick polymer modified tack coat. The performance of these bonded open graded friction courses are compared to open graded friction courses laid with a regular tack coat, as well as to a Novachip mixture, which is basically a stone matrix asphalt mixture with a thick polymer-modified tack coat.

The primary accomplishments and findings from this work may be summarized as follows:

- A thorough literature review of open graded and porous friction course mixture design approaches was performed. The findings indicated that most design approaches focus on durability through optimizing the mixture for relatively high asphalt content. The results also showed that most mixture designs were somewhat empirical in nature, often relying on torture tests, such as the Cantabro test, to evaluate the adequacy of the design asphalt content.

- A new mixture design method was developed for porous friction courses and open graded friction courses. This design approach uses the Superpave gyratory compactor to compact the mixtures. A modified asphalt film thickness criterion is introduced to ensure mixture durability. The design asphalt content is selected at the lowest percent of voids in the mineral aggregate, meaning that any more or any less asphalt binder will result in a less dense mixture. This new mixture design method led to the development of a Draft Florida Method and Draft Specifications for the Mixture Design of Porous Friction Course Mixtures.
- The introduction of Superpave gyratory compaction to Open Graded and Porous Friction Course Mixture Design led to the development of a modified locking point compaction criterion for these mixtures. This new compaction criterion was used to establish the appropriate number of gyrations for Open Graded and Porous Friction Course Mixtures. The results implied that at about 50 gyrations, most Open Graded and Porous Friction Course Mixtures have reached compaction levels which correspond to those observed in the field after traffic consolidation.
- The new porous friction course mixtures tested had overall a comparable fracture resistance to that of open graded friction courses, despite their higher air void levels. However, both open graded friction course and porous friction course mixtures tend to be sensitive to long term oven aging, meaning that their fracture resistance decreases significantly.
- A field test section on I-295 in Duval County, Florida, was constructed using the new porous friction course mix design. The fracture testing of mixtures obtained from the paver showed a low fracture resistance. Forensic evaluation confirmed field observations of drain down in the mixture, resulting in a low asphalt content. This was likely due to the lack of dispersion of the fiber in the mixture. The part of the test section with draindown problems was repaved to a successful completion of the field testing project.
- The permeability of the new porous friction course mixture placed on I-295, Jacksonville, was similar to that of a nearby open graded friction course (FC-5). However, the added thickness of the porous friction course should help with water storage during rain events.
- The presence of a polymer modified tack coat in open graded friction courses resulted in an increased fracture resistance.
- The rutting resistance of open graded and porous friction courses was evaluated through the use of the Asphalt Pavement Analyzer. The mixtures were tested by two different loading mechanisms: 1) the traditional hose, and the 2) a new rubber strip that was developed to simulate radial tire contact stresses more closely. The mixtures tested included both field cores from the US-27 Highlands County test project and laboratory-prepared specimens. The results showed that both the hose and the strip testing results on newly placed mixtures (field cores) were overly aggressive, resulting in significant APA rutting. However, field cores obtained six months after construction showed little rutting, which is consistent with field observations. Similarly, test results on laboratory-prepared mixtures were similar to those on field cores obtained six months after construction. A forensic evaluation showed that the air voids from laboratory compacted mixtures were

similar to those obtained from field cores six months after construction. This implies that the laboratory compaction level used may be reasonable for the mixtures studied.

- A comparison of APA strip loading rutting results of open graded friction course mixtures with and without a thick modified tack coat showed that the mixtures with the modified tack coat rutted less than the mixtures with the normal tack coat.
- Both open graded and porous granite friction course mixtures tested for water damage susceptibility using the AASHTO T-283 conditioning method and the Superpave IDT (Indirect Tensile) test showed possible susceptibility to moisture damage. In particular, the tensile creep rate for these mixtures increased with moisture damage. Therefore, a modified AASTHTO T-283 based test procedure was developed for evaluating the moisture damage susceptibility of open graded friction courses. This new procedure is described in the report.
- Open graded and porous friction course mixtures made from absorptive limestone aggregate exhibited an increase in fracture resistance due to long term oven aging. However, further evaluation showed that the fracture resistance peaked at about 3-5 days of long term oven aging, after which it started decreasing again. This effect is likely due to the absorption of asphalt into the aggregates during long term oven aging, which is not representative of the field.
- The results of the performance evaluation of the field test sections on US-27 in Highlands County, Florida, showed:
    - ◊ Acceptable friction resistance for all five test sections. However, it is still too early to evaluate the relative long-term friction performance of the different test sections.
    - ◊ All four FC-5 mixtures as well as the Novachip mixture had excellent smoothness. There were no observable differences between granite and limestone FC-5 mixtures, as well as FC-5 mixtures with and without a thick modified tack coat. The Novachip mixture showed an exceptionally low IRI value (46) and a high ride number (greater than 4).
    - ◊ Rutting measured with a roadway profiler and a transverse profilograph was negligible for all test sections. However, back-calculated base course layer moduli for all FC-5 test sections was on the low side (less than 25 ksi). This is a possible concern for the long-term cracking performance of these sections.
    - ◊ All FC-5 sections showed excellent field permeability during the first six months after construction. However, the presence of a thick modified tack coat in FC-5 mixtures resulted in a slightly decreased permeability.
    - ◊ All mixtures showed a similar pavement noise level (~97-99 dB), which corresponds to that of a lawn mover. The presence of a thick modified tack coat does not appear to decrease noise level.

The following conclusions may be derived from the accomplishments and findings summarized above:

- The new porous friction course and open graded friction course mixture design procedure appears to result in reasonable design asphalt contents. The implementation of the research findings should focus on determining how to best ensure durability of this new mixture type for Florida conditions.
- The presence of a bonded polymer-modified tack coat appears to improve both the cracking and rutting resistance of friction course mixtures.
- The Superpave IDT test and the Energy Ratio worked well for evaluating the fracture performance and moisture susceptibility of the mixtures studied. A long-term oven aged mixture or a moisture conditioned mixture with an Energy Ratio less than one should be rejected, or antistripping treatment (such as lime) should be added and the mixture retested.
- The optimal Superpave IDT test conditions for open graded and porous friction courses still need to be finalized. However, based on the results obtained in this study, it was found that if the rate of loading during the tensile strength testing was increased by 50 percent consistent fracture test results were obtained.
- It is important to continue to monitor the long term field performance of the bonded friction course study on US-27, Highlands County. Similarly, it is important to evaluate the long term field performance of the porous friction course test section on I-295, Duval County.

CHAPTER 1
INTRODUCTION

1.1 Background

Due to the frequency of high intensity short-duration rainfall events in Florida, vehicular hydroplaning is a serious concern, especially on Interstate and other limited access facilities. Along with pavement cross-slope and rutting, the surface texture of an asphalt pavement plays a critical role in the prevention of hydroplaning on high-speed, multi-lane facilities. In order to minimize problems of this nature, a number of states (including Florida) have utilized traditional open graded friction courses in order to maximize the pavement's macrotexture, which in turn reduces splash and spray, and minimizes hydroplaning potential.

In the mid-1970's, Florida developed a friction course (FC-2) which was a 3/8-inch Nominal Maximum Aggregate Size (NMAS) open graded mixture (with polish resistant aggregate) placed approximately ½-inch thick. In the1990's, in an effort to improve hydroplaning resistance, the FC-2 was eventually replaced by a slightly coarser open graded friction course (FC-5), which is a ½-inch NMAS mixture, placed approximately ¾ inch thick.

Since the FC-5 mixture is coarser and is placed slightly thicker than the FC-2, it has a greater capacity to store/drain water from the pavement surface during a severe rainstorm. However, this additional storage capacity has never been quantified. Visual observations of this surface type indicate that while it is better than the old FC-2, it will still "fill-up" with water – resulting in water ponding on the pavement surface.

A number of European countries, as well as several states in the US (Georgia, Oregon) have developed a "porous friction course" which is an open graded friction course placed in thicknesses ranging from 11/4 to 2 inches thick. The combination of the high in-place air void content of this mixture, coupled with the thickness at which it is placed, gives this type of pavement surface a great deal of potential with regard to storage and drainage of water during a rainstorm. However, there are also a number of questions associated with these types of pavements, such as durability, rutting resistance, and long-term porosity. The first part of this project focuses on the development of thick open graded friction course mixtures for Florida.

There is also a high priority need to evaluate the estimated performance life and cost effectiveness of a bonded friction course that uses a special paver to lay a heavy polymer-modified tack coat immediately in front of the hot mix mat. This technology is available in Florida and has been demonstrated on several small projects. Other states, including Texas, Pennsylvania, and Alabama have several years experience with this process and indications are that it has great potential.

The ongoing Longitudinal Wheel Path Cracking study at UF has confirmed that a significant amount of the distress on high-volume Florida pavements is caused by surface initiated wheel path cracking from lateral stresses generated by radial truck tires. A recent evaluation of the ten year performance of ground tire rubber in an open graded friction course on SR-16 in Bradford County indicates that increased asphalt content from ground tire rubber modification of the friction course can reduce longitudinal wheel path cracking. Based on these studies, there is a high likelihood that the bonded friction course process could significantly extend the crack resistance life of open graded mixes in Florida by providing more polymer modified asphalt to the friction course. It is also highly likely that raveling resistance of the friction course will be improved as well. The second part of this project focuses on the evaluation of bonded friction courses in Florida.

1.2 Problem Statement

This report evaluates the feasibility of using: 1) thick porous friction courses in Florida, and 2) bonded friction courses in Florida.

1.2.1 Thick Porous Friction Courses

The first part of this report focuses on summarizing the experience from Europe, Georgia, and Oregon and to evaluate the use of thick open graded friction courses (OGFC) with Florida materials. The types of materials used by other countries and states is summarized, and their mixture design guidelines are reviewed. Key issues regarding the construction and laboratory and field evaluation of these mixes are also summarized.

Once basic evaluation methods have been determined, the focus is on the development of a new mixture design procedure for thick open graded friction courses using Florida aggregates as well as granites for durability, long-term permeability, rut resistance and cracking resistance. Finally, based on the findings from the literature review and the new mixture design procedure, basic specification requirements regarding its design are determined for Florida conditions, including: aggregate types, gradations, binder contents, modifiers.

The thick porous friction course study is broken down into several tasks: 1) literature review of porous friction courses, 2) laboratory work involving the development of mixture design of porous friction courses, 3) utilization of the mixtures from actual field test sections to evaluate the rutting and cracking resistance as well as long-term porosity of these pavement types, and 4) overall performance monitoring of actual in-service test sections.

1.2.2 Bonded Friction Courses

Since the use of polymer modified friction courses is a relatively new process and will be more costly than conventional friction courses, a preliminary study is conducted to quantify and estimate the performance differences between bonded and conventional friction courses currently used by the FDOT. A big part of the evaluation focuses on the characterization of the rutting and fracture properties of bonded friction courses. The rutting properties are evaluated using the Asphalt Pavement Analyzer (APA) and the fracture properties are evaluated using the Superpave Indirect Tensile (IDT) test and the Hot Mix Asphalt Fracture Mechanics framework developed at the University of Florida. The use of field test sections for validating the field rutting and cracking performance of bonded friction courses is also explored. This information will be helpful in determining the feasibility of using bonded thick open graded friction courses.

1.3 Objectives

The objectives of the thick Open Graded Friction Course study are:

- To address the suitability of the use of thick open graded friction courses with aggregates typically used in Florida.
- If necessary, to determine whether changes in design procedures are required before suitable thick OGFC mixtures can be evaluated and produced using Florida materials.
- To define the relative performance of thick OGFC under high traffic Interstate conditions compared to FC-5 OGFC using Florida limestone and granite aggregates.
- Develop guidelines for specifications for use of thick open graded friction courses in Florida.

The objectives of the evaluation of Bonded Friction Course Study are:

- Evaluate the feasibility of using bonded friction courses in Florida with aggregates common in Florida.
- Evaluate the rutting and cracking performance differences between bonded and conventional friction courses for Florida conditions based on a detailed laboratory investigation.
- Evaluate the field performance of bonded friction courses using field test sections in Florida.

1.4 Scope

To meet the numerous objectives of this research project, multiple studies were performed. The summary of results for each study is presented at the end of each chapter.

Chapter 1 provides background, objectives, and scope for this research project. Chapter 2 deals with a literature review of bonded and thick OGFC mixtures, with a particular emphasis on mixture design and laboratory performance evaluation techniques. Chapter 3 provides an overview of the development of a new mixture design procedure for thick open graded friction course (OGFC) mixtures. Chapter 4 presents a field evaluation of the porous friction course mixture design, including an evaluation of rutting and fracture resistance of field mixtures obtained from a test project on I-295, Duval County, Florida. Chapter 5 presents a study on the fracture resistance of porous friction course mixtures.

Chapters 6 through 8 specifically address bonded friction courses. Chapter 6 presents an evaluation of APA rutting resistance of FC-5 open graded friction course mixtures from US-27, Highlands County. Chapter 7 presents an evaluation of fracture resistance of FC-5 open graded friction course mixtures from US-27, Highlands County. The FC-5 mixtures on US-27, Highlands County consisted of limestone and granite mixtures, each with and without a thick polymer-modified tack coat. Finally, Chapter 8 presents a summary of field performance monitoring from the US-27 project in Highlands County.

CHAPTER 2
LITERATURE REVIEW

2.1 Overview

This chapter presents a detailed literature review on two different friction course approaches. The first approach consists of thick open graded friction courses, and the second one deals with bonded friction courses, such as Novachip.

A number of European countries, as well as several states in the US (in particular Georgia and Oregon) have developed a "thick porous friction course," which is an open graded friction course placed in thicknesses ranging from 1¼ to 2 inches thick. The combination of the high in-place air void content of these mixtures, coupled with the thickness at which they are placed, gives these types of pavement surface a great deal of potential with regard to storage and drainage of water during a rainstorm. Thus in this literature review the mix design guidelines, construction, and evaluation procedures used by the various different agencies, and in particular Georgia are discussed in detail. The literature review focuses on the possible identification of: 1) preliminary mix design protocol for thick open graded friction courses, and 2) preliminary guidelines on recommended thickness, gradation, binder selection, and other mixture components, using materials common to Florida, including both Florida aggregates and Georgia granites.

There is also a high priority need to evaluate the estimated performance life and effectiveness of a bonded friction course that uses a special paver to place a heavy polymer-modified tack coat just in front of the hot mix mat. This technology is available in Florida and has been demonstrated on several small projects. Other states, including Texas, Pennsylvania, and Alabama have several years experience with this process and indications are that it has great potential. Therefore, in this chapter the types of materials, equipment, mix design and construction guidelines used by the above states are reviewed.

2.2 Bonded Friction Courses

Bonded friction courses are ultra thin friction courses whose primary objective is to restore the skid resistance and surface impermeability of a pavement. Other advantages include improved adhesion, reduced pavement noise, and the reshaping of existing pavements. The advantages of these friction courses (ex. Novachip) over other surface treatments are enumerated by Estakhri (1994) and listed below:

Over Chip Seals

- Excellent chip retention
- Ability to reshape existing pavement for filling ruts and few other minor irregularities
- Less rolling noise

Over Micro Surfacing

- Better adhesion to underlying layer with the use of a heavy tack coat
- Greater surface macrotexture
- Better drainage thus reducing splash and spray

Over Open Graded Friction Course

- Better adhesion to underlying layer with the use of heavy tack coat
- Better protection of the underlying pavement from surface water, a prominent problem in open graded friction course

Over Dense-Graded Thin Overlay

- Better adhesion to underlying layer with the use of heavy tack coat
- Increased rut resistance with use of high quality crushed material
- Greater surface macrotexture
- Improved surface drainage
- Better protection of the underlying pavement from surface water

2.2.1 Novachip Description

Novachip is a proprietary product and consists of a layer of hot pre-coated aggregate over a binder spray application. The tack coat is generally a polymer modified, emulsified asphalt (usually a latex or elastomer modified emulsion). Such a coating offers strong bonding between the chippings. Thus due to the immediate application of the binder, chippings are held in position and whip-off is totally eliminated.

The hot mix material is an SMA type of a gap-graded mixture that includes large proportion (70-80%) of single sized crushed aggregate, bound with mastic composed of sand, filler and binder. The typical binder content varies from 5-6 percent. The course thickness varies from 0.4-0.8 inches (10-20 mm) depending on the maximum size of stone. Layer thickness is usually 1.5 times the diameter of the largest stone (e.g., Serfass, 1991; Estakhri, 1994).

Novachip is placed with a specially designed paving machine that combines the function of an asphalt distributor and a lay down machine (Serfass, 1991; Estakhri, 1994). The paver applies the tack coat and the hot mixture in a single pass. This heavy application of tack helps to ensure adhesion of the friction course to the underlying pavement and to prevent the possibility of surface water from permeating into the underlying pavement. The operation of the paving equipment is described below.

2.2.2 Paving Equipment

The operation involved in the Novachip process is as follows:

- Collection of mixture from the transport truck
- Storage of mixture
- Storage of sufficient tack emulsion for at least 3 hrs of operation
- Distribution of tack coat with a servo-controlled application rate
- Immediate covering of tack with the mixture
- Smoothing of applied mixture

The Novachip paver machine (Serfass, 1991; Estakhri, 1994) includes the following components:

- A receiving hopper for pre-coated chippings (mixture) with a self-locking hook for the supplying truck
- A scraper-type conveyer
- A chipping storage chamber with appropriate thermal insulation and a total capacity of 6.6 cu yd^2 (5 m^3)
- Several binder tanks, thermally insulated, with a capacity of 15.8 yd^2 (12 m^3) (more than half day work)
- A conveyer transferring chippings to the screed unit
- A variable-width spray bar
- A variable-width heating screed unit

The design of the spray bar includes wide-angle nozzles whose delivery depends on the road speed of the machine. The friction course is applied at high speed, i.e., greater than 32.8 ft/min (10 m/min) and reaches 65.6-82.0 ft/min (20-25 m/min).

2.2.3 Post Construction Testing

Novachip is typically evaluated for friction resistance, surface texture, and surface roughness.

2.2.3.1 Surface Roughness – International Roughness Index

The International Roughness Index (IRI) measures pavement roughness (Knoll, 1999). The lower the IRI number, the smoother the ride. The rating system scores a roadway from the criteria listed in Table 2-1.

The test is conducted in accordance with the ASTM E-950 test method.

Table 2-1. International Roughness Index.

<table>
<tr><th>FHWA Highway Statistics Categories (inches per mile)</th><th>Interstate Routes</th><th>National Highway System (NHS) Non Interstate Routes</th><th>Non-NHS Traffic Routes & Other Routes with ADT>=2000</th><th>All Other Routes</th></tr>
<tr><td><60</td><td>Excellent</td><td>Excellent</td><td>Excellent</td><td rowspan="2">Excellent</td></tr>
<tr><td>60-94</td><td>Good</td><td>Good</td><td rowspan="2">Good</td></tr>
<tr><td>95-119</td><td rowspan="3">Fair</td><td rowspan="3">Fair</td><td></td></tr>
<tr><td>120-144</td><td rowspan="3">Fair</td><td rowspan="4">Fair</td></tr>
<tr><td>145-170</td></tr>
<tr><td>171-194</td><td rowspan="3">Poor</td><td rowspan="3">Poor</td></tr>
<tr><td>195-220</td><td rowspan="2">Poor</td></tr>
<tr><td>>220</td><td>Poor</td></tr>
</table>

2.2.3.2 Friction

Pavement friction is tested in accordance with ASTM E-274 test method (locked wheel skid trailer) (Knoll, 1999). The classes of friction number ratings are shown in Table 2-2.

Table 2-2. Friction Number Rating System.

Friction Number	Description
50-100	Very Good
40-49	Good
30-39	Fair
20-29	Poor
1-19	Very Poor

2.2.3.3 Surface Texture

Pavement friction depends on both microtexture and macrotexture. Microtexture refers to detailed surface characteristics of the material. Good microtexture will provide an effective contact area between the tire and the aggregate on the road surface. Macrotexture refers to the general coarseness of the surface, which facilitates the drainage of water from the surface. Pavement macrotexture is defined as the deviation of the pavement from a true planar surface and the average texture depth between the bottom of the pavement surface and the top of surface aggregates.

Surface texture is measured by the Sand Patch Depth (SPD) volumetric technique method in accordance with ASTM E 965-87 (Knoll, 1999).

2.2.4 Ultra-thin Friction Course Mixture Design

The mixture design of ultra-thin friction courses (ex. Novachip) is based on the design developed for Stone Matrix Asphalt (SMA) mixtures by the FHWA Office of Research (Smith, 1974; Asphalt Institute, 1978). Material requirements and aggregate gradations are established, and the bulk and apparent specific gravity of coarse and fine aggregate are determined. The specific gravity of asphalt cement is also obtained. After these preliminary steps, surface capacity (K_c) of the coarse aggregate is calculated and it is used to then calculate the design asphalt content.

2.2.4.1 Gradation

There are three kinds of gradation bands based on the nominal maximum size (Knoll, 1999). These design gradation bands for the various nominal maximum sizes are shown in Table 2-3.

2.2.4.2 Asphalt Content Determination

The surface capacity K_c is determined by the Centrifuge Kerosene Equivalent (C.K.E) test based on the FHWA design procedure (Smith et al. 1974; Asphalt Institute, 1978). The K_c value is a measure of relative roughness and degree of porosity of the aggregate and is used in an experienced based formula to calculate the design asphalt content.

Equipment Needed

a) Pans, 4.49 in. × 0.73 in. (115 mm × 25 mm) deep
b) Hot plate or oven capable of 110º C ± 5º C
c) Beaker, glass 45.0 oz (1500 ml)
d) Balance, 17.50 oz (500 g) capacity,

Table 2-3. JMF Gradation Range for Novachip Mixtures

Sieve Size (mm)	Sieve Size	Percentage Passing					
		Type A 0.25 in. (6.3 mm)		Type B 0.37 in. (9.5 mm)		Type C 0.49 in. (12.5 mm)	
		Max	Min	Max	Min	Max	Min
19	3.76					100	100
12.5	3.12			100	100	100	85
9.5	2.75	100	100	100	75	90	70
6.3	2.29	100	75	45	30	45	30
4.75	2.02	60	40	37	24	40	24
2.36	1.47	24	20	26	21	25	21
1.18	1.08	20	15	23	15	25	18
0.6	0.79	15	10	15	12	20	12
0.3	0.58	12	8	14	8	16	8
0.15	0.43	10	7	10	5	10	5
0.075	0.31	7	5	7	5	7	4
Asphalt Content %			5.3% Min		6% Max		

e) Metal funnels, top dia. 3.47 in. (89 mm), height 4.45 in. (114 mm), Orifice 0.51 in. (13 mm) with piece of 0.08 in. (2 mm) sieve soldered to the bottom of the opening

f) Oil, S.A.E No. 10 lubricating.

Surface Capacity (K_c) Test for Coarse Aggregate

a) Quarter out 3.68 oz (105 g) of aggregate representative of material passing through 0.37 in. (9.5 mm) sieve and retained on 0.19 in. (4.75 mm) (No. 4) sieve

b) Dry sample on hotplate or in110º C ± 5º C (230 º F ± 9º F) oven to a constant mass and allow to cool

c) Weigh out approximately 3.50 oz (100 g) sample to nearest 0.1 g and place it in a funnel

d) Completely immerse sample in S.A.E No. 10 lubricating oil for 5 minutes

e) Drain sample for 2 minutes

f) Place funnel containing sample in 60º C (140º F) oven for 15 minutes of additional draining

g) Pour sample from funnel into a tarred pan, cool, reweigh sample to nearest 0.1 g. Determine the amount of oil retained as percent of dry aggregate mass.

h) Use the chart as shown in Figure 2-1. If the apparent specific gravity for the fraction is greater than 2.70 or less than 2.60 apply correction to percentage oil retained using the formula as shown in Figure 2-1.

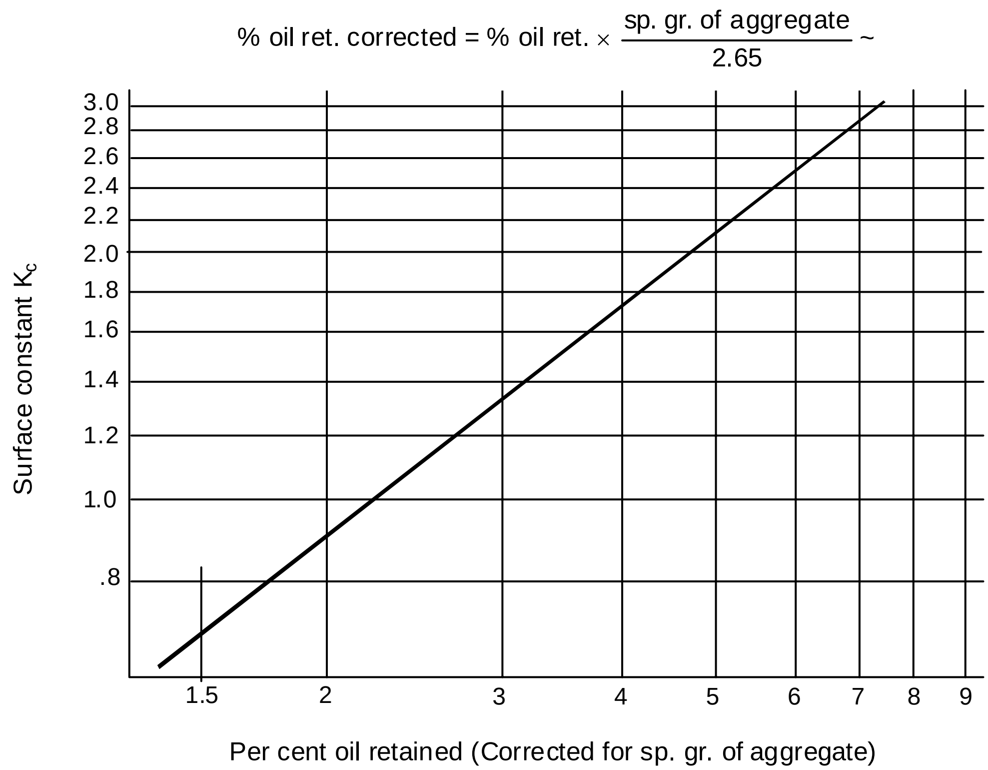

Figure 2-1. Surface Constant K_c vs. Percentage Oil Retained. (Smith et al. 1974)

Design Asphalt Content

a) If the apparent specific gravity of the coarse aggregate fraction is 2.6 or more but not greater than 2.7 then:

$AC = 2\ K_c + 4.0$

b) If the apparent specific gravity of the coarse aggregate fraction is greater than 2.70 or less than 2.60 then:

$AC = (2\ K_c + 4.0) * 2.65/\ S_f$

AC = Design asphalt content, present by mass of aggregate

S_f = Apparent specific gravity of coarse aggregate

Thus this the optimum asphalt content used for final design.

2.2.5 Construction Requirements

The construction details of Novachip as described in the Mississippi DOT specifications (Sheshadri, 1993) are discussed below.

2.2.5.1 Surface Treatment Paving Machine

Screed Unit: The machine shall be equipped with a heated screed. It shall produce finished surface meeting the requirements of the typical cross section. Extensions added to the screed shall be provided with the same heating capability as the main screed unit, except for use on variable width tapered and/or as approved by the engineer. The screed with extensions if necessary shall be of such width as to pave an entire lane in a single pass.

Asphalt Distribution System: A metered mechanical pressure sprayer shall be provided on the machine to accurately apply and monitor the rate of application of the tack coat. The rate shall be uniform across the entire paving width. It shall be applied at a temperature of 160º F ± 20º F. Application can be immediately in front of the screed unit. The application rate shall be 0.22 ± 0.05-gallons/sq. yards (0.83 ± 0.19-liters/sq yd), unless otherwise directed by the engineer. The application rate shall be verified by the carpet tile test before the work commences. At the end of each workday, a check shall be made to determine the quantities of tack coat used. The check shall be made by means of calibrated load cells on the machine.

Tractor Unit: The tractor unit shall be equipped with a hydraulic hitch sufficient in design and capacity to maintain contact between the rear wheels of the hauling equipment and the pusher rollers of the finishing machine while the paving is being unloaded. The machine shall support no portion of the weight of the hauling equipment, other than the connection. No vibrations or other options of the hauling equipment, which could have a detrimental effect on the riding quality of the completed pavement, shall be transmitted to the machine. The use of any vehicle which requires dumping directly into the finishing machine and which the finishing machine cannot push or propel to obtain the desired lines and grades without resorting to hand finishing will not be allowed.

2.2.5.2 Rollers

Steel wheel rollers shall meet be rated at 10 tons and may be three wheel type but the tandem type is preferred.

2.2.5.3 Straightedges and Templates

When directed by the engineer, the contractor shall provide acceptable 10-foot straightedges for surface testing.

2.2.5.4 Weather Limitations

The tack coat and the paving mixture shall be placed only when the temperature of the surface to be overlaid is no less than 50º F and rising, but shall not be placed when the air temperature is below 60º F and falling. It is further understood that the tack coat or paving mixture shall be placed only when the humidity, general weather conditions and moisture conditions of the pavement surface are suitable in the opinion of the engineer.

2.2.5.5 Tack Coat

Before the tack coat and the paving mixture are applied, the surface upon which the tack coat is to be placed shall be cleaned thoroughly to the satisfaction of the engineer. The surface shall be given a uniform application of tack coat using asphaltic materials as specified approximately two seconds prior to the placement of the paving mixture.

2.2.5.6 Hauling Equipment

It shall be required that the truck beds be covered while transporting the paving mixture unless otherwise directed by the engineer. At the discretion of the engineer, the truck beds may be insulated.

2.2.5.7 Spreading and Finishing

The paving mixture shall be delivered at a temperature between 290° F and 330° F unless otherwise directed by the engineer. The paving mixture shall be dumped directly into the surface treatment paving machine and spread on the tack coated surface within two seconds of tack coat having been applied. The paving mixture shall be spread to the depth and width that will provide the specified compacted thickness, grade and cross section. Placing of the paving mixture shall be as continuous as possible. The finished surface shall be smooth and of uniform texture and density.

2.2.5.8 Compaction

Immediately following placement of the paving mixture, the surface shall be rolled to accomplish a good seating without excessive breakage of the aggregate. A minimum of three passes with steel wheel rollers is required. The compaction shall be accomplished prior to the paving mixture cooling below 180° F. The operation of the rollers shall cause no displacement or shoving of the paving mixture. If the displacement occurs, it shall be corrected to the satisfaction of the engineer. Sprinkling of the fresh mat shall be required, when directed by the engineer, to expedite opening the roadway to the traffic. Sprinkling can be with water or limewater solution.

2.2.5.9 Method of Measurement

Paver laid surface treatment, complete in place and accepted will be measured by the ton. Tack coat will be measured by gallons.

2.2.6 Performance Evaluation

The performance of Novachip was evaluated as a part of field test projects by the Alabama Department of Transportation (ALDOT) in 1996 (Kandhal, 1997). The first test section was placed as a wearing surface on a newly overlaid pavement (binder course) of the two eastbound lanes of US 280 (AL 38) in ALDOT Division 4, Tallapoosa County. The test section was placed in October, 1992 from milepost (MP) 63.90 to 66.60 just north of the intersection between US 280 and AL 22 near Alexander City.

The second Novachip project was constructed in November 1992 on AL 21 (an existing 2-lane highway) just north of the town of Talladega from MP 236 to 239 in Talladega County (ALDOT Division 4).

Both Tallapoosa and Talladega projects had been visually inspected annually since construction in 1992. The following observations were made during the last inspection on July 3, 1996, about 3¾ years after construction.

2.2.6.1 Tallapoosa Project

Material Composition

Two different aggregates were used for the gap-graded HMA (Novachip) on this project. Crushed gravel was used from MP 63.90 to MP 64.85. The mix consisted of 70% 3/8-in crushed gravel, 27% sand, and 3% mineral filler.

Crushed granite aggregate was used from MP 64.85 to MP 66.60. The mix consisted of 75% 3/8-in granite coarse aggregate, 20% granite screenings, and 5% mineral filler.

AC-20 asphalt cement was used in preparing the HMA mixtures. The specified mixing temperatures were between 143º C (290º F) and 168º C (335° F). A CRS-2P SBR latex modified asphalt emulsion was used for the tack coat on both test sections.

Performance

The 1996 average daily traffic (AADT) on this road was 13,044 with 10½ % trucks. No significant loss of Novachip or raveling had taken place in both granite and gravel test sections. This indicated very good adhesion between the Novachip and the underlying surface.

Pavement surface friction numbers for gravel Novachip, granite Novachip, and granite control sections were compared. The comparison of granite Novachip section with granite control section indicates that the Novachip surface had higher friction number than the control section in the passing lane, and about equal friction number in the travel lane. Therefore, Novachip surface should exhibit significantly higher friction numbers compared to a dense-graded HMA surface, both using the same aggregate.

2.2.6.2 Talladega Project

Material Composition

The Novachip mixture was made in a batch plant. It consisted of 72% granite coarse aggregate, 22% granite screenings, and 6% aggregate lime mineral filler. AC-20 asphalt cement was used for coating the mixture. A CRS-2P SBR latex modified asphalt emulsion was used for the tack coat material. The existing road surface was slightly raveled and had some transverse cracks, which were partially sealed. The control section in this project consisted of conventional dense-grading wearing course (AL 416 mix) containing granite aggregate.

Performance

The 1996 average daily traffic (AADT) on this road was 7,534 with 11½ % trucks. This project also did not show any loss of Novachip or raveling after about 3¾ years of service. Also the Novachip surface had significantly higher friction numbers compared to dense-graded ALDOT 416 mix.

Apart from the above projects, performance related studies have also been conducted in Mississippi (Sheshadri, 1993), Texas (Estakhri, 1995) and Pennsylvania (Knoll, 1999) which have shown good performance by Novachip in terms of ride quality, friction and noise reduction. Thus based on performance evaluation, Novachip seems like a potential alternate for chip seals, micro surfacing, and open graded friction courses.

2.3 Porous European Mixtures (PEM)

Porous European Mixtures (PEM), also known as porous friction course mixtures, are very coarse open graded mixtures with approximately 4-6% binder and 3% of filler. The binder is typically polymer modified. PEM is designed to contain about 20% air voids to make it very porous (e.g., Heystraeten, 1990; Decoene, 1990; Khalid, 1994; Huber, 2000).

The differences between PEM and conventional OGFC are as follows:

- European permeable mixtures generally allow more gap-graded mixtures as compared to OGFC mixtures. The GDOT specification for PEM is similar to the European mixtures.
- All European PEMs are specified with a minimum air void level. Air voids in OGFC mixtures tend to be significantly lower than PEM (i.e., around 14%).
- European PEMs use polymer modified asphalt binders since modified binders are less susceptible to draindown, both during construction and service.
- European PEMs tend to demand higher LA Abrasion standards for aggregate than OGFC. LA abrasion values are specified from 12-21%. Typical OGFC LA Abrasion requirements are between 35-40%.
- European agencies specify minimum asphalt content based on a durability test (Cantabro test), which is performed on compacted specimen. A maximum limit is based on air voids. In contrast to this, for OGFC mixtures in the U.S., the asphalt content is typically selected on the basis of the FHWA method (Smith et al. 1974). The asphalt content in American OGFC mixtures typically varies from upper five to mid six percent ranges.

2.3.1 Advantages of Porous Asphalt

2.3.1.1 Hydroplaning and Glare Reduction

One of the major hazards while driving in rain is hydroplaning. A layer of water builds between the surface and tire thus causing the vehicle to literally float. As a result it becomes difficult to steer and apply the brakes. Porous asphalt mixtures are designed to have interconnected air voids for high permeability and higher macrotexture, which solves the problem of hydroplaning to a great extent (Khalid, 1994).

With porous asphalt pavement, wet weather visibility is significantly improved since there is little or no free surface water for spraying. Porous asphalt mixtures are also effective in reducing glare. On unlit dense-graded pavements, the nighttime visibility decreases due to a water layer on the road surface, which hampers the performance of reflection devices. Also due to saturation of the pavement surface, mirror reflections are caused by the headlights of oncoming vehicles. However porous asphalt is effective in these conditions in reducing the glare.

2.3.1.2 Noise Reduction

One of the benefits of porous asphalt mixtures is the significant noise reduction compared to the denser-graded mixes, due to their good sound absorption potential. Measurements in most European countries have shown that by using porous asphalt noise levels can be reduced by approximately 3 dB(A) as compared to the conventional dense asphalt concrete surfacing (e.g., Huber, 2000). These figures apply to passenger car vehicles traveling at 45 mph (80 Km/hr) in dry conditions. The noise level is influenced by aggregate size, size distribution, permeability and the condition of the layer. It should be noted that porous asphalt should not be used on high traffic low speed roadways since it has been observed that after few years (typically 3 years) of service all noise reduction benefits are lost as the surface voids get clogged and the mixture in turn becomes dense graded. However at high speeds, tire-pavement interaction creates a hydraulic action which flushes the dirt from the pavement voids and reduces clogging.

Vehicle noise is generated by different mechanisms. At high speed the vibrations in the tire structure and air pumping in the cavities beneath the tire cause tire noise. The tread blocks slip on the road surface mostly at the edges of the contact area where the vertical forces are lower and the tire is deforming as the weight is applied or removed. When the tire rolls forward and the tread blocks snap back to their unloaded shape it generates vibrations on the tire surface, thus generating noise.

Movement of air in the cavities of tire treads causes air pumping. As the tire rolls forward the compressed air tries to escape to the sides. As the voids in the tire tread block are lifted off the pavement, a vacuum is created in the tread cavities and air rushes to fill the voids. If the pavement texture is smooth there is less opportunity for the air to leak from underneath the tread block and the tire noise is accentuated.

2.3.1.3 Friction

Rain may reduce friction of pavements considerably, even when no hydroplaning takes place. Porous asphalt counteracts this effect and even at high speeds the friction with porous asphalt is high on wet pavements (See multiple articles in TRR 1265, 1999; Khalid, 1994; Colwill, 1993). Also on dry pavements porous asphalt gives exceptionally good frictional properties at higher speeds where macrotexture is very important.

2.3.2 Disadvantages of Porous Asphalt

2.3.2.1 Strength

Porous asphalt is generally not considered to contribute to the overall structural integrity of the pavement due to high air voids. In Belgium, it was observed that the moduli of porous asphalt manufactured with 80/100 pen asphalt, was 73-79 percent of that of wearing course using conventional asphalt concrete (Van Heystaeten and Moraux, 1990).

In the Netherlands, the structural behavior of porous asphalt is assessed using the Department of Public Works standard multi layer elastic layer design (Van Der Zwan, 1990). The model assumes that the fatigue resistance of pavements is determined by the lower part of the structure, which ignores the possibility of fatigue cracks being developed in the upper part of the structure. Studies have indicated that the following three aspects need to be specifically addressed to evaluate the bearing capacity of the porous asphalt.

Initial Stiffness Modulus

Using fatigue testing, the initial Young's modulus is determined which is subsequently used in a layered elastic design model to estimate the contribution of porous asphalt to the bearing capacity of the pavement structure. From a study conducted by Van Der Zwan (1990), it was observed that initial effective contribution was about 80 to 90 percent of that attainable with gravel asphalt concrete, depending on the thickness of the structure.

Aging and Stripping

As a result of the open structure of porous asphalt, the binder is likely to undergo oxidation causing accelerated aging which in turn increases the stiffness of the binder considerably. Also water ingress may lead to stripping of the lower part of the surface layer, which affects the cohesive properties as well as the adhesion to the underlying base course thus impairing the load transfer characteristic of the structure. Based on the studies conducted (e.g., Van Der Zwan, 1990) it was observed that the weighted effective contribution to the pavement structural integrity was 35-40 percent of that achieved by using dense-graded asphalt concrete.

Temperature

The suction and pumping action of tires passing over porous asphalt layers, coupled with wind motion, will promote a continuous circulation of air within pores. Consequently, the temperature in a porous asphalt wearing course is likely to remain closer to the prevailing air temperature than with closed surfacing materials. A detailed analysis of the temperature gradient on porous asphalt and dense asphalt concrete showed that the weighted average temperature over a year was found to be about 1º C lower in pavements surfaced with porous asphalt than in comparable structures with dense asphalt concrete wearing course. Thus, the stiffness of porous asphalt is less affected by warm weather.

Thus the combination of the above three factors implies that porous asphalt can be expected to contribute to about 50 percent of the equivalent bearing capacity achievable with dense asphalt concrete (Van Der Zwan, 1990).

2.3.2.2 Clogging

The service life of porous asphalt is generally less than for dense graded mixtures, due to premature clogging of the voids, which leads to ineffective drainage of the surface water. Also when the surface pores become plugged, a pavement might fail by asphalt being stripped from the aggregate surface. However in the Netherlands, a new technique is being explored, where two layers of porous asphalt are used to counter this problem (Huber, 2000). The surface layer uses aggregate that is 0.16 to 0.31 in. (4 to 8 mm) in size. Directly underneath is another porous asphalt layer containing 0.43 to 0.62 in. (11 to 16 mm) aggregate. The surface layer allows water and sound to penetrate to the larger chamber of voids in the lower layer. The surface layer has smaller voids to prevent larger materials from clogging the voids. The smaller debris, which enters the surface voids, can then be suctioned out by the hydraulic action of the tires on the pavements.

2.3.3 Performance Related Laboratory Testing

2.3.3.1 Resistance to Plastic Deformation

Spanish Experience

As a part of laboratory study carried out by University of Cantabria and the ESM Research Centre (Perez-Jimenz, 1990) in Spain, the resistance to deformation was determined using a wheel-tracking test at a temperature of 60º C. The percentage of binder used ranged from 4 to 4.5 for the porous asphalt mixes tested. It was observed that polymer modified asphalt offered greater resistance to plastic deformation than that of ordinary asphalt. Thus the use of polymeric asphalt diminishes the effect of post compaction (consolidation rutting) by traffic, which is sometimes observed in the porous mixes.

2.3.3.2 Tensile Resistance (Indirect Tension Test) – Spanish Experience

The effect of binder on improving tensile resistance was studied using IDT testing on Marshall mixtures as a part of research to study the effect of special binders on porous mixes (Perez, 1990). The test temperatures chosen were 5º C (41º F) and 45º C (113º F) and the load application rate was 1.98 in/min (50.8 mm/min). The samples were compacted with a compaction energy of 50 Marshall blows per face. The test results showed that the performance of mixes made with polymeric asphalt was better than with ordinary asphalt. The difference is more prominent at 45º C (113º F) than at 5º C (41º F), which implies that the low rupture velocity influenced flexibility and toughness of the binder.

2.3.3.3 Resistance to Disintegration

Disintegration resistance is tested in the laboratory through the Cantabro test of wear loss, consisting of testing Marshall specimens in the LA abrasion machine (without the steel balls) and obtaining the weight loss after 300 drum revolutions (Perez, 1990; Ruiz, 1993; Khalid, 1994; Carbrera, 1996). This test is a standard test for porous asphalt prevalent throughout Europe. The test is used to bracket the maximum and minimum asphalt content, however the specifications and the test temperature vary with countries.

2.3.3.4 Adhesiveness

Resistance to stripping is influenced by the adhesiveness between the aggregate and the binder. For studying resistance to stripping, in *Spain* and the *UK*, the Cantabro test is used to determine the loss in test samples that have been submerged in water at a controlled temperature around 49º C (120.2º F) for 4 days (Perez, 1990; Khalid 1994). It is observed that modified asphalt has greater adhesiveness than ordinary asphalt though the losses are higher when immersed in water.

2.3.3.5 Drain-down Test

The Basket Drain-Down test (Decoene, 1990) has been used in Belgium and Spain for evaluating the drain-down of the porous asphalt mixes. The procedure is as follows:

- The mix is manufactured and compacted in Duriez mold (Duriez, 1962) under a pressure of 30 bars
- The molds are then laid on a grid and the set is placed on an oven at 180º C (356º F) for 7½ hours - these severe conditions are chosen to simulate occasional cases when the asphalt is draining through the aggregates
- The asphalt drained through the mix to the grid is recovered and the loss of asphalt is calculated with respect to the initial binder content.

2.3.3.6 Scuff Test

The Scuff Test is a durability test used in the UK (Nicholls, 1997). The procedure for scuffing involves a loaded pneumatic-tired wheel, with its axle set at an angle to the direction of motion, repeatedly passing over the surface of a specimen at an elevated temperature. The standard test scuffs the sample for a period of 12 minutes and an assessment of erosion index is made at the completion of the test. The erosion index gives an indication of the resistance of the sample to the wear by scuffing. The erosion index is the sum of assessment of each 1.95 × 1.95 in. (50 mm × 50 mm) square in a 2 × 5 grid as:

0 (less than 25 percent damage),
1 (25 to 50 percent),
2 (50 to 75 percent), or
3 (over 75 percent damage).

Hence an erosion index of zero indicates no erosion whereas an index of 3 indicates that only 25 percent of the surfacing remains intact over the area covered by the whole grid.

Theoretically this test seems to be more suitable than the particle loss test (Cantabro test), however both tests do not provide any ideas about the failure of the mix and hence it is difficult to assess the suitability of either test.

2.3.3.7 Friction Resistance

Research at the UK Road Research Laboratory showed a significant relationship between polishing of aggregates used in road surfaces and friction resistance. Tests were devised using an Accelerated Polishing Machine and a friction-measuring device, a Skid-Tester, to determine a Polished Stone Value (Nichols, 1998).

The Polished Stone Value (PSV) of aggregate gives a measure of resistance to the polishing action of vehicle tires under conditions similar to those occurring on the surface of a road (Nichols, 1998). The test procedure is given in BS812 Part 1 14-1989 (Nichols, 1998).

The movement of vehicle tires on pavements results in polished surfaces and exposed aggregate surfaces. Resistance to this polishing action is determined by the aggregate properties. The actual relationship between PSV and friction resistance varies with traffic conditions, type of surfacing and other factors. It is calculated by multiplying the coefficient of friction by 100.

2.3.4 European Porous Asphalt Mix Design Approach

The design of porous asphalt (Khalid 1994; Ruiz, 1990) is based on:

- A minimum binder content to ensure resistance against particle losses and thick film on the aggregates
- A maximum binder content to avoid binder runoff and still maintain a permeability in the mix

Using the maximum Cantabro abrasion value, a minimum amount of binder is fixed. The initial selection of the binder type is influenced by the aggregate source and the amount of binder that needs to be carried. The purpose of using modified binders is to improve the resistance against particle losses with very open mixtures through higher cohesion and to obtain a longer durability through thicker binder films because of higher viscosity. A reduction in thermal susceptibility is also sought to get higher consistencies with high temperature and more flexibility with low temperatures (Khalid, 1994). The final paved layer thickness is typically around 4 cm (Huber, 2000).

2.3.4.1 British Design

The mix design approach followed in Britain (Huber, 2000; Khalid, 1994) is as follows:

- Two types of porous asphalt mixes are used; one is 0.39 in. (10 mm) nominal maximum size and the other is 0.78 in. (20 mm) nominal maximum size. The 0.39 in. (10 mm) is more like that of other European countries. But the 0.78 in. (20 mm) mix is considerably coarser. The gradation is shown in Table 2-4.

Table 2-4. British Gradations.

Grading	**% Passing**						
	28 mm	**20 mm**	**14 mm**	**10 mm**	**6.3 mm**	**3.35 mm**	**0.075 mm**
20 mm	100	95 - 100	55 - 80		20 - 30	8 - 14	2 - 7
10 mm	—	—	100	90 - 100	40 - 55	22 - 30	2 - 7

- The aggregates are specified to provide a hard and durable rock. The LA value for aggregates must be low, less than 12% according to Clifford (1996) or 18% according to Nicholls (1997) and Khalid and Walsh (1996).
- The aggregates must have high polishing resistance and all the aggregates should have at least 2 fractured faces.
- Maximum flakiness index of 25 is specified to control aggregate shape degradation.
- Both modified and unmodified asphalt binder used. The binder is modified with styrene-butadiene-styrene (SBS) or ethylene-vinyl acetate (EVA) modifier. Minimum asphalt content is 4.5% and minimum air voids is 20%.
- Specimens are compacted using 50 Marshall Blows. Durability is not tested directly but is controlled through selection of binder grade and minimum asphalt content.

2.3.4.2 Spanish Design

The mix design approach followed in Spain (Huber, 2000; Ruiz, 1990) is summarized as follows:

There are two gradations of porous asphalt used in Spain. Both are 0.49 in. (12.5 mm) nominal maximum size. The P12 has larger gap gradation between the 0.49 and 0.39 in. (12.5 and 10 mm) sieves. The PA12 has larger gap gradation between the 0.39 and 0.20 in. (10 and 5 mm) sieves. The gradations are shown in Table 2-5

Table 2-5. Spanish Gradations.

Grading	**% Passing**						
	20 mm	**12.5 mm**	**10 mm**	**5 mm**	**2.5 mm**	**0.63 mm**	**0.075 mm**
P-12	100	75 - 100	60 - 90	32 - 50	10 - 18	6 - 12	3 - 6
PA-12	100	70 - 100	50 - 80	15 - 30	10 - 22	6 - 13	3 - 6

- A maximum LA abrasion value of 20% is specified for the aggregates.
- A maximum flakiness index of 25% is specified.
- Modified asphalt binder is used (for high traffic and hot climate 60/70-pen asphalt is used. For low traffic and cool temperature, 80/100-pen asphalt is used). The modifier used is SBS or EVA.
- Specimens are compacted using 50 Marshall Blows.
- Minimum air voids of 20% are selected using the Cantabro test with a maximum weight loss of 25% at 25º C.
- Maximum asphalt content is based on the air voids of the compacted specimen. Typical asphalt content is around 4.5%.

2.3.4.3 Italian Design

The mix design approach followed in Italy (Huber, 2000; Khalid, 1994) is as follows:

- A single 0.62 in. (16 mm) nominal maximum size aggregate gradation is used. All aggregate are crushed with no natural sand allowed. The gradation is shown in Table 2-6.

Table 2-6. Italian Gradation Band.

Grading	% Passing					
	20 mm	**14 mm**	**10 mm**	**5 mm**	**2 mm**	**0.075 mm**
16 mm	100	75 - 100	15 - 40	5 - 20	0 - 12	0 - 7

- A maximum LA abrasion of 16% is specified for the aggregates.
- A modified binder is used, with 6-8 percent modifier added to 80/100-penetration asphalt. Mix temperature is 190-200º C. They use SBS as modifier.
- Minimum asphalt content is 4.5 percent based on the Cantabro test. The allowable asphalt content ranges from 4-6 percent. A maximum weight loss of 25 percent is allowed at 20ºC (68º F).
- The target air voids range varies from 18-23 percent.
- Moisture damage is evaluated during drainage using the Cantabro test. Maximum loss allowed is 30 percent.

2.3.4.4 Belgian Design

- The mix design is based on first determining the voids in the coarse aggregate and then measuring at various binder contents the voids and the percentage of wear estimated using the Cantabro test. The design binder content is optimized for air voids and wear.
- Aggregates should contain more than 80 percent greater than 0.08 in. (2 mm) in diameter. The proportion of sand (0.08 – 2 mm) should be around 12 percent and the rest should be filler.
- A gap grading is to be obtained by omitting 2/7 or 2/10 mm fraction from the 0/14mm mixture.
- The target air voids range is around 16-18 percent.
- Asphalt content is around 4-5 percent (unmodified). For modified binders, it should be around 5.5 to 6.5 percent.

2.4 Georgia Open Graded Friction Course Mixture Design and Permeable European Mixture Design

Open Graded Friction Course (OGFC) mixes are typically gap-graded and contain high percentages of single sized coarse aggregate. OGFC typically have high AC content, a thick AC film, low percentage of material passing the 0.0075 mm sieve, high volume of air voids (10-20%) and in-place thickness of up to 1¼ inch (31.75 mm) (Watson, 1998). Georgia has adopted a PEM design for the design of OGFC, thus effectively making the mix design procedures for the PEM and the OGFC the same, with the exception of the gradation, which is coarser for the PEM. Thus a polymer-modified asphalt is used to provide greater film thickness, which safeguards against the weathering problems experienced by earlier OGFC mixes in Georgia. Mineral fibers are also specified, typically around 0.4% of the total mix, although cellulose fiber is also commonly used as a replacement for the mineral fiber. Hydrated lime is added as an antistripping agent to OGFC.

Laboratory tests (APA in particular) have been conducted to evaluate different PEM and OGFC gradations and types of additives. In addition, OGFC specimens are tested for drain down.

2.4.1 Material Composition

The Job Mix Formula (JMF) consists of mainly coarse aggregate, typically granite, with a small amount of fines. The binder used should be very stiff such as PG 76-22 made with polymers (typically SB or SBS). The addition of fibers is desirable since it reduces drain down (Kandhal, 2000; Cooley, 2000).

2.4.1.1 Gradation

The job mix formula used for the Georgia PEM is shown in Table 2-7 (Watson, 1998). The 0.45 power chart is shown in Figure 2-2.

Table 2-7. Georgia PEM Gradations Band.

Sieve Size (mm)	**0.49 in. (12.5 mm) PEM (% Passing)**	
	Max	Min
19	100	100
12.5	100	80
9.5	60	35
4.75	25	10
2.36	10	5
0.075	4	1

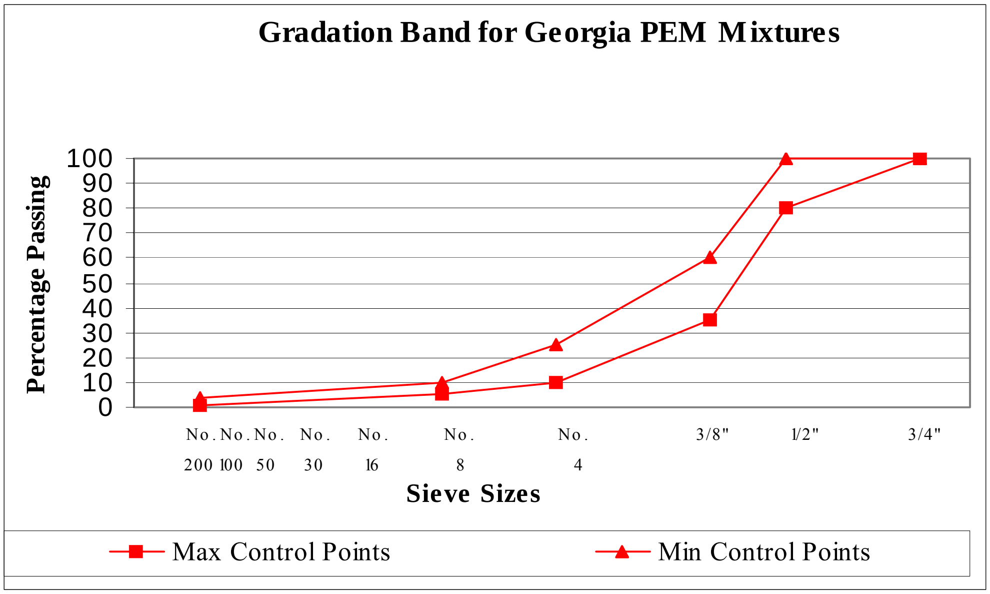

Figure 2-2. Georgia PEM Gradation Band.

2.4.1.2 Aggregate Specification

The specifications for basic aggregate properties are shown in Table 2-8 (Watson, 1998)

Table 2-8. Aggregate Specifications for Georgia PEM.

Parameter	Requirement
LA abrasion Loss (%)	<50
Soundness Loss (%)	<15
Flat and Elongated Particles (5:1 ratio)	<10
Mica Schist Ratio	<10

Watson (1998) notes that only silica rich aggregates can be used (e.g., granite) in Georgia PEM mixtures. Carbonate-rich aggregates are excluded. Soundness loss is measured using magnesium sulfate. Typical loss is less than 2 percent.

2.4.1.3 Polymer Modified Asphalt Cement

GDOT primarily uses two polymers, styrene butadiene (SB) and styrene butadiene styrene (SBS), to modify asphalt cements used in OGFC and PEM mixes (Watson, 1998). The main improvements noted after the inclusion of polymers have been: (1) to increase in binder stiffness by 8-10 times, (2) the softening point of AC has increased by approximately 44º F, and (3) the AC is more ductile and flexible than unmodified AC.

Due to greater viscosity of the polymer blend, temperature requirements in the design procedure have been increased to 325º F. Also a phase angle requirement of less than 75º has been added to help ensure that polymer modification is used to meet binder grade requirements. The base asphalt cement is typically modified with 4.0-4.5 % polymer by weight of AC.

2.4.1.4 Mineral Fibers

Fibers are used in OGFC and PEM mixes to stabilize the AC film surrounding the aggregate particles in order to reduce AC drain down during production and placement. GDOT uses mineral fibers in OGFC and PEM mixes at a dosage rate of 0.4% by weight of total mix (Watson, 1998).

Thus the final design requirements have been tabulated in Table 2-9.

Table 2-9. Design Requirements for Georgia PEM.

Parameter	Requirement
Binder Content	5.5 - 7.0
Polymer SB or SBS (%)	4 - 4.5
Air Voids (%)	20 - 24
Drain Down	< 0.3
PG 76-22	
Fibers (%)	0.2 - 0.4
Anti Stripping (lime) (%)	1.00

2.5 Georgia PEM Mix Design Procedure (GDT – 114)

2.5.1 Scope

The design for the Georgia PEM consists of four steps. The first step is to conduct AASHTO T-245 to determine asphalt cement content then, secondly to determine optimum asphalt content. The third step to perform GDT-127 (Drain-down Test) and final step is performing GDT- 56 (Boil Test). These steps are discussed below.

2.5.2 Apparatus

1. 13 metal pie pans
2. Oven capable of maintaining 250º F ± 3.5º F
3. Oven capable of maintaining 140º F ± 3.5º F
4. Beaker glass, 17 oz (500 ml)
5. Glass funnels, top dia. = 3.5 in. (88.9 mm); height = 4.5 in. (114.3 mm); orifice = 0.5 in. (12.7 mm) with piece of No. 10 sieve positioned at top of funnel neck. Cork stopper to fit the outlet of funnel neck
6. Oil S.A.E. No. 10 lubricating
7. Drain down Equipment as specified in GDT-127
8. Marshall Design equipment as specified in AASHTO T- 245
9. Equipment as specified in GDT – 56
10. Balance 175 oz (5000 gms), 0.1 gm accuracy

2.5.3 STEP 1 – SURFACE CAPACITY (KC)

a) Quarter out 3.68 oz (105 g) of aggregate representative of material passing through 0.37 in. (9.5 mm) sieve and retained on 0.19 in. (4.75 mm) (No. 4) sieve
b) Dry sample 250º F ± 3.5º F oven to a constant mass and allow to cool
c) Weigh out approximately 3.5 oz (100 g) sample to nearest 0.1 g and place it in a funnel
d) Completely immerse sample in S.A.E. No. 10 lubricating oil for 5 minutes by plugging funnel outlet with cork stopper
e) Drain sample for 2 minutes
f) Place funnel containing sample in 140º F (60º C) oven for 15 minutes of additional draining
g) Pour sample from funnel into a tarred pan, cool, reweigh sample to nearest 0.1 g. Determine the amount of oil retained as percent of dry aggregate mass
h) Use the chart as shown in Figure 2-3. If the apparent specific gravity for the fraction is greater than 2.70 or less than 2.60 apply correction to percentage oil retained using the formula shown in Figure 2-3.
i) Determine the required asphalt using the following formula

   $\% AC = 2.0 (K_c) + 3.5$

   (No correction applied for viscosity)

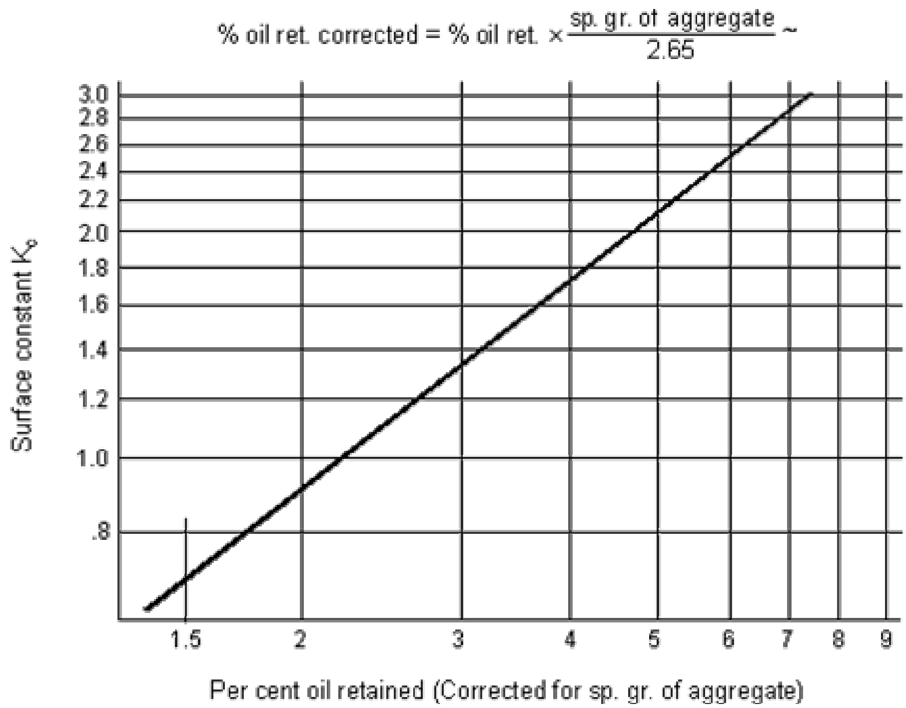

Figure 2-3. K_c vs. Percentage of Oil Retained.

2.5.4 STEP 2 – MODIFIED MARSHALL DESIGN AND OPTIMUM AC

a) Heat the coarse aggregate to 350º F ± 3.5º F, heat the mold to 300º F ± 3.5º F and heat the AC to 330º F ± 3.5º F

b) Mix aggregate with asphalt at three asphalt contents in 0.5% interval nearest to the optimum asphalt content established in Step 1. The three specimens should be compacted at the nearest 0.5% interval to the optimum and three specimens each at 0.5% above and below the mid interval.

c) After mixing, return to oven if necessary and when 320º F ± 3.5º F compact using 25 blows on each side

d) When compacted, cool to the room temperature before removing from the mold

e) Bulk Specific Gravity

1) Determine the density of a regular shaped specimen of compacted mix from its dry mass (in grams) and its volume in cubic centimeters obtained from its dimensions for height and radius. Convert the density to the bulk specific gravity by dividing by 0.99707g/cc, the density of water at 25º C

$$\text{Bulk Sp.Gr} = W / (\pi r^2 h / 0.99707) = \text{Weight (gms)} \times 0.0048417/\text{Height (in)}$$

W = weight of specimen in grams
R = radius in cm
H = height in cm

f) Determine the stability and the flow after 1 hr in 77º F

g) Calculate percent air voids, VMA and voids filled with asphalt based on aggregate specific gravity

h) Plot VMA curve versus AC content

i) Select the optimum asphalt content at the lowest point on VMA curve

2.5.5 STEP 3 - DRAIN-DOWN TEST

Perform the drain test in accordance with the GDT – 127 (Method for Determining Drain Down Characteristics in Uncompacted Bituminous Mixtures). A mix with an optimum AC content as calculated above is placed in a wire basket having 0.25 in. (6.4 mm) ¼ inch mesh openings and heated 25º F (14º C) above the normal production temperature (typically around 350º F (176.7º C) for one hour. The amount of asphalt cement, which drains from the basket, is measured. If the result exceeds the maximum draindown of 0.3 percent, increase the fiber content by 0.1 percent and repeat the test.

2.5.6 STEP 4 - BOIL TEST

Perform the boil test according to GDT – 56 with a complete batch of mix at optimum asphalt content as determined in Step 2 above. If the sample treated with hydrated lime fails to maintain 95% coating, a sample shall be tested in which 0.5% liquid anti stripping additive has been used to treat the asphalt cement in addition to the treatment of aggregate with hydrated lime.

2.6 Production and Construction

The production and construction of Georgia PEM as described in the GDOT guidelines is as follows:

2.6.1 Asphalt Plant Production

Since fibers are used in Georgia PEM, it may be necessary to increase both the dry mixing time and the wet mixing time when using batch HMA facilities. Dry mixing time is generally increased by 5-10 seconds to ensure dispersion of fibers during the mixing cycle. A uniform coating of all aggregate particles by the asphalt binder should be accomplished during the wet mixing cycle. Since OGFC consist of predominantly single size aggregate, the contractor should consider the use of more than one cold feed bin for adding the predominant size aggregate in order to better maintain the mixture quality control and limit variations in mixture aggregate gradation. The screening capacity of the screen deck in the batch plant must also be considered. For batch plants the high percentage of one size aggregate may result in override of the screen deck and hot bins and is likely to occur at lower mix production levels as compared to conventional HMA mixtures.

The OGFC mixture should not be stored in surge bins or silos for extended periods of time due to potential drain down problems. A maximum storage time of 4 hours is recommended.

2.6.2 Hauling

Since the polymer modified asphalt binder in the OGFC has a tendency to bond, it is necessary to apply a heavy and thorough coating of an asphalt release agent to truck beds. Also truck beds should be raised after spraying to drain any puddles of the release agent. Puddled release agent, if not removed, will cool the OGFC and cause cold lumps in the mix. Diesel fuel and other oil-based products should not be used due to their detrimental effects on the hot mix asphalt.

Tarping each load is essential to prevent excessive cooling of the mix during transportation. Tarps should extend over the sides and back of the truck and should be secured. The tarp at the front of the truck should be arranged or protected, in such a manner that air flow is directed over the top of the tarp rather being allowed to flow underneath the tarp during transport of the mix. Haul trucks should be insulated on the front and sides of the vehicle with insulating material that has an R-Factor of at least 4. The insulation material should be able to withstand temperatures up to 400º F (204.4º C) without being damaged, and should be protected from contamination and weathering.

2.6.3 Construction Requirements

The OGFC or PEM should be placed only on an impermeable asphalt course unless it is being applied as the second layer in a two-layer drainage system. A freshly compacted dense – graded HMA course may have as much as 8 percent air voids in the mat and thus may be permeable to water. Therefore, it is essential to provide a uniform tack coat at an adequate application rate to fill and seal the surface voids of the underlying layer. Generally, a tack coat of 0.06-0.08 gal/sq. yd. (0.27-0.36 L/m^2) of asphalt cement (AC-30 or PG 67-22) or an equivalent emulsion tack coat is needed for OGFC. On cracked pavements, the GDOT suggests sealing the underlying pavement by applying a 50 percent diluted slow-setting emulsion tack coat at a rate of 0.05 to 0.1 gallon per square yard. The application rate should be high enough to completely fill the surface voids. A slow-setting emulsion tack coat is likely to penetrate the surface voids more effectively than asphalt cement tack coat.

The OGFC or PEM mat should be daylighted by overlapping at least one foot onto the shoulder so that the rainwater percolating through the OGFC or PEM can drain out freely at its edge. If curb and gutter sections are encountered, the dense graded layer should tie-in at grade level with the face of the curb and the OGFC or PEM should be above grade level. In this case, if the OGFC or PEM is placed more than one inch thick, it should be tapered beginning 12-18 inches from face of the curb so that the thickness at the curb face is not more than one inch. The transverse joints for asphaltic concrete ½-inch and ¾-inch OGFC or PEM mixtures placed at entrance and exit ramps and at end of project joints should be prepared by milling the existing pavement to conform to the following details. The milling should taper in depth as needed for a distance of at least 50 feet from the transverse joint so that milling depth is 1¼ inches (or depth specified by the engineer), for end of project and ramp transverse joints so that the OGFC or PEM ties in at grade level with the existing roadway surface. The milling/inlay operation for entrance and exit ramps should extend to a point specified by the plans or as directed by the engineer. Project limits should be planned so that the transverse joints at the end of the project do not occur near the low point of a vertical curve.

The use of a material transfer device for transferring the OGFC or PEM from the trucks to the paver is optional, but highly recommended. It should have remixing capabilities so that any cold lumps produced during transportation would be remixed into a homogeneous material. The transfer device should be external in order to keep trucks from bumping the paver. The transfer device should have storage and remixing capability and may be used in conjunction with a paver hopper insert to allow continuous operation of the paver. An effective materials transfer device will be able to maintain the temperature so that there is no more than 20º F (11.1º C) difference between the highest and lowest temperatures when measured transversely across the mat.

A hot screed is very important to prevent the pulling of the mat. A propane torch can be used to heat the screed before each use and hot charcoal may be placed on the feet of electronic grade control skis to keep them from causing streaks if a portion of the ski rides on the finished mat.

For compaction, steel wheel rollers are used. It is crucial to keep the breakdown or initial roller within 50 ft of the paver to adequately seat the OGFC or PEM while it is still hot and workable. The breakdown roller usually completes two coverages of the mat in static mode to compact up to 1¼-inch of OGFC or PEM. The breakdown roller may have to be operated in a vibratory mode at transverse joints to eliminate any high spots.

Longitudinal joints in the OGFC or PEM pavement are constructed by placing the mix approximately 1/16-inch above the previously placed and compacted lane. Therefore, it is important for the edge of the screed or extension to follow the joint exactly to prevent excessive overlap. Transverse joints placed against a previously laid OGFC or PEM are constructed by starting with the screed approximately one foot behind the joint. The screed is laid flat on the previously laid OGFC or PEM mat. The hot OGFC or PEM mix is augured into the screed hopper and then dug off the new joint when travel begins. The joint should be cross-rolled with a steel wheel breakdown roller.

2.7 Summary

2.7.1 Novachip

Novachip consists of a layer of hot pre-coated aggregate that is placed over a binder spray application. The tack coat is generally a polymer modified, emulsified asphalt (usually a latex or elastomer modified emulsion). Such a coating offers strong bonding between the aggregates.

The hot mix material is a gap graded mixture that includes a large proportion (70-80%) of single-sized crushed aggregate, bound with a mastic composed of sand, filler and binder. The binder content typically varies from 5-6 percent. The course thickness varies from 0.39-0.78 in. (10-20 mm) depending on maximum aggregate size. Layer thickness is usually 1.5 times the diameter of the largest stone (Serfass, 1991; Estakhri, 1994). The gradations used are shown in Table 2-3.

A special paving machine is used for paving of Novachip as discussed previously. A number of factors are used to evaluate the pavement performance, namely surface roughness, friction, surface macrotexture, ride quality and rolling noise.

The mix design procedure for Novachip is summarized as follows:

- Selecting a gradation based on nominal maximum size falling in the desired gradation band.
- Determining the optimum asphalt content based on the FHWA design procedure (Smith, 1974; Asphalt Institute, 1978) which is summarized as follows:
  - Determine surface capacity K_c by the Centrifuge Kerosene Equivalent (C.K.E.) test in which the K_c value is obtained based on percent oil retained on the coarse aggregate fraction. A correction is applied to percentage oil retained if the apparent specific gravity for this fraction is greater than 2.70 or less than 2.60.
  - The design asphalt content is calculated based on a relationship between K_c and AC.

The construction guidelines for Novachip have been taken from Mississippi DOT specification (Sheshadri, 1993):

- The asphalt paving machine shall equipped with a heated screed unit.
- It should have a metered mechanical pressure sprayer to accurately apply and monitor the rate of application of the tack coat.
- The application temperature shall be around 160º F ± 20º F and the application should be 0.22 ± 0.05-gallons/sq. yards (0.83 ± 0.19-liter/sq. meter), unless otherwise directed by the engineer.
- The tractor unit shall be equipped with a hydraulic hitch sufficient in design and capacity to maintain contact between the rear wheels of the hauling equipment and the pusher rollers of the finishing machine while the paving is being unloaded.
- The rollers used for compacted must be rated at 10 tons and may be three wheeled type.
- The tack coat and the paving mixture shall be placed only when the temperature of the surface to be overlaid is no less than 50° F (10º C) and rising, but shall not be placed when the air temperature is below 60° F (15.5º C) and falling.
- The paving mixture shall be delivered at a temperature between 290° F (143.3º C) and 330° F (165.6º C).
- For compaction a minimum of three passes with steel wheel rollers is required. The compaction shall be accomplished prior to the paving mixture cooling below 180° F (82.2 º C).

Based on studies conducted in Alabama (Kandhal, 1997), Mississippi (Sheshadri, 1993), Texas (Estakhri, 1995) and Pennsylvania (Knoll, 1999), which have shown good performance by Novachip in terms of ride quality, friction and noise reduction, Novachip seems like a potential alternate for chip seals, micro surfacing, and open graded friction courses.

2.7.2 Porous European Mix (PEM)

PEM also known as porous asphalt is a coarse graded mix with around 4-6% binder and 3% of filler. The binder is typically polymer modified. PEM is designed to contain about 20% air voids to make it very porous (e.g., Heystraeten, 1990; Decoene, 1990; Khalid, 1994; Huber, 2000).

The advantages of PEM are a reduction in hydroplaning, glare, and noise; and an improvement in pavement friction. However, the disadvantages include lower structural stability due to the coarse gradation used, along with clogging and oxidation aging over time.

The performance testing performed in Europe includes tensile testing, evaluation of mix disintegration, adhesiveness, drainage test, friction and scuff test.

The design of porous asphalt (Khalid 1994; Ruiz, 1990) is based on:

- A minimum binder content to ensure resistance against particle loss and thick film on the aggregate.
- A maximum binder content to avoid binder runoff and still maintain a good permeability in the mix.

Using the maximum abrasion value, a minimum amount of binder is fixed. The initial selection of the binder type is influenced by the aggregate source and the amount of binder that needs to be carried. The purpose of using modified binders is to improve the resistance against particle loss in very open mixtures through higher cohesion and to obtain longer durability through the thicker asphalt film resulting from the presence of the higher viscosity modified binder (Khalid, 1994).

The mix design for various countries such as Britain, Italy, Spain and Belgium is based on minimum air voids, maximum LA abrasion value, maximum weight loss using the Cantabro test and minimum and maximum asphalt content.

2.7.3 Georgia PEM

OGFC and PEM mixes are typically gap-graded and contain high percentages of single sized coarse aggregate. OGFC and PEM typically have high AC content, a thick AC film, low percentage of material passing the 0.0075 mm sieve, high volume of air voids (10-20%) and in-place thickness of up to 1¼ inch (Watson, 1998). Georgia has adopted PEM design for design of OGFC. Thus, polymer-modified asphalt is used to give greater film thickness, which safeguards against the weathering problems experienced by earlier OGFC mixes in Georgia. Mineral fibers are also used, typically around 0.4% of the total mix. Hydrated lime is added as an antistripping agent to OGFC.

The JMFs consist of mainly coarse aggregate, typically granite, with a small amount of fines. The binder used should be very stiff such as PG 76-22 made with polymers (typically SB or SBS). The addition of fibers is desirable since it reduces drain down (Kandhal, 2000; Cooley, 2000). The gradation band is shown in Figure 2-2.

For aggregate specifications, maximum LA abrasion loss should be 50 % and soundness loss should be limited to 15 percent.

The mix design is according to the procedure described in GDT –114, which is as follows:

- Determine surface capacity K_c by the Centrifuge Kerosene Equivalent (C.K.E.) test in which the K_c value is obtained based on percent oil retained on the coarse aggregate fraction. A correction is applied to percentage oil retained if the apparent specific gravity for this fraction is greater than 2.70 or less than 2.60.
- The design asphalt content is then calculated based on a relationship between K_c and AC.

- The next step is calculating the optimum asphalt content using a modified Marshall Design method in which three specimens are compacted at the nearest 0.5 % interval to the optimum calculated above and three specimens each at 0.5% above and below the optimum. VMA vs. AC content is plotted and the optimum asphalt content is the lowest point of the curve.
- Performing the draindown test according to GDT-127 procedure. The maximum draindown allowed is 0.3%.
- Next is performing the Boil test according to GDT-56 test method. If a sample treated with hydrated lime fails to maintain 95% coating, a sample shall be tested in which 0.5% liquid anti stripping additive has been used to treat the asphalt cement in addition to the treatment of aggregate with hydrated lime.

The construction of Georgia PEM is according to the set of GDOT guidelines as follows:

- Provide a tack coat of 0.06-0.08 gal/sq. yd. (0.27-0.36 L/m^2) of asphalt cement or an equivalent of emulsion tack coat for OGFC/PEM. On a cracked pavement, the GDOT suggests sealing the underlying pavement by applying a 50 percent diluted slow-setting emulsion tack coat at a rate of 0.05 to 0.1 gallon per square yard. The application rate should be high enough to completely fill the surface voids.
- The OGFC/PEM mat should be daylighted by overlapping at least one foot onto the shoulder so that the rainwater percolating through the OGFC/PEM can drain out freely at its edge.
- If OGFC/PEM is placed more than one inch thick, it should be tapered beginning 12-18 inches from the face of the curb so that the thickness at the curb face is not more than one inch.
- The use of a material transfer device for transferring the OGFC/PEM from the trucks to the paver is optional, but highly recommended. It should have remixing capabilities so that any cold lumps produced during transportation would be remixed into a homogeneous material.
- A hot screed is very important to prevent the pulling of the mat. A propane torch can be used to heat the screed before each use and hot charcoal may be placed on the feet of electronic grade control skis to keep them from causing streaks if a portion of the ski rides on the finished mat.
- For compaction, steel wheel rollers are used. It is crucial to keep the breakdown or initial roller within 50 ft of the paver to adequately seat the OGFC/PEM while it is still hot and workable. The breakdown roller usually completes two coverages of the mat in the static mode to compact up to 1¼ inch of OGFC/PEM.
- Longitudinal joints in the OGFC/PEM pavement are constructed by placing the mix approximately 1/16 in. above the previously placed and compacted lane.

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CHAPTER 3

DEVELOPMENT OF MIXTURE DESIGN FOR POROUS FRICTION COURSES IN FLORIDA

3.1 Introduction

The Georgia Department of Transportation started evaluating the use of Porous Friction Courses in 1992 (Watson et al. 1998). These new mixtures were entitled "Georgia Permeable European Mixtures" (Georgia PEM). The new Georgia PEM mixtures proved to be more permeable than conventional OGFC, due to its gap-graded characteristics, with a predominant single size coarse aggregate fraction that contains a high percentage of air voids as described by Watson et al. (1998) and Eason (2004). In this chapter, the Georgia PEM mix design (GDT 114, 1996) was used as a starting point for the new Florida Permeable Friction Course (PFC) mixture design. The proposed Florida PFC mixture design uses the Superpave gyratory compactor as compared to the Marshall Hammer for Georgia PEM (GPEM) mixtures.

Below, the mixture design for thick open graded friction courses (OGFC) used by the Georgia Department of Transportation will be reviewed briefly, followed by the development of Superpave gyratory compaction criteria for PFC mixtures. Finally, a new method for the design of PFC mixtures is proposed that is suitable for Florida materials and uses the Superpave gyratory compactor.

3.1.1 Georgia Permeable European Mixture Design

The main elements of the Georgia PEM mixture design (GDT 114 Test Method B, 1996) are as follows:

- The method specifies the use of modified asphalt cement (PG 76-22) as specified in Section 820 (GDT 114, 1996) and does not require the determination of surface capacity (KC) to determine initial trial asphalt contents.

- The Georgia DOT uses the Marshall Method of compaction during the design of the Georgia PEM mixtures. Specimens are prepared using the Marshall Hammer at 25 blows and 1000 g of mixture.
- A stabilizing fiber is added to the mixture to avoid binder drain down. The fiber meets the requirements of Section 819 (GDT 114, 1996).

Table 3-1 shows gradation limits as defined in GDT 114 (1996). These gradation limits are also used as design limits for the new Florida PFC mixture design.

Table 3-1. Gradation Specifications for GPEM Mixtures According to GDT 114 (1996).

Mixture Control Tolerance	**Asphalt Concrete**	**12.5 mm PEM**
	Grading Requirements	
± 0.0	¾ in. (19 mm) sieve	100
± 6.1	1/2 in. (12.5 mm) sieve	80-100
± 5.6	3/8 in. (9.5 mm) sieve	35-60
± 5.7	No. 4 (4.75 mm) sieve	10-25
± 4.6	No.8 (2.36 mm) sieve	5-10
± 2.0	No. 200 (75 μm) sieve	1-4
	Design Requirement	
± 0.4	Range for % AC	5.5-7.0
	Class of stone (Section 800)	"A" only
	Coating retention (GDT-56)	95
	Drain-down, AASHTO T 305 (%)	<0.3

The specifications for basic aggregate properties are shown in Table 3-2 (Watson et al. 1998). It is of interest to note that only silica-rich aggregates can be used (e.g., granite). Carbonate-rich aggregates are excluded. Soundness loss is measured using magnesium sulfate. Typical loss is less than 2%.

The Georgia DOT method of design for the GPEM mixture consists of three steps. The first is to conduct a modified Marshall mix design (AASHTO T-245) to determine optimum asphalt cement content. The second step is to perform a drain down test, according to GDT-127

Table 3-2. Aggregate Specifications for Georgia PEM.

Parameter	Requirement
LA abrasion Loss (%)	<50
Soundness Loss (%)	<15
Flat and Elongated Particles (5:1 ratio)	<10
Mica Schist Ratio	<10

(2005), or AASHTO T 305-97 (2001). The third step is to perform a boil test, according to GDT-56, or ASTM D 3625.

There are no mixture design guidelines currently available for the determination of trial gradations within the specification limit. Rather, the mixture designer has to use his own judgment to determine a trial gradation within the limits provided.

A. APPARATUS

The apparatus required consist of the following:

1. Drain-Down equipment as specified in GDT-127 (2005) or AASHTO T 305-97 (2001)
2. Marshall design equipment as specified in AASHTO T-245
3. Boil Test Equipment as specified in GDT-56 (2005) or ASTM D 3625
4. Balance, 5000 grams capacity 0.1 g accuracy.

Step 1 – Modified Marshall Design and Determination of Optimum AC

After determining a trial aggregate blend the following steps are required to determine the asphalt content:

1. Heat the coarse aggregate to 350º F ± 3.5º F, heat the mould to 300º F ± 3.5º F and heat the AC to 330º F ± 3.5º F.
2. Mix aggregate with asphalt at three asphalt contents in 0.5% intervals nearest to an estimated optimum asphalt content. The three specimens should be compacted at the nearest 0.5% interval to the estimated optimum and at 0.5% above and below the estimated optimum.
3. After mixing, return to oven if necessary and when 320º F ± 3.5º F compact using 25 blows on each side.

4. When compacted, cool to the room temperature before removing from the mold.
5. Bulk Specific Gravity: Determine the density of a regular shaped specimen of compacted mix from its dry mass (in grams) and its volume in cubic centimeters obtained from its dimensions for height and radius. Convert the density to the bulk specific gravity by dividing by 0.99707 g/cc, the density of water at 77º F (25º C):

   Bulk Sp.Gr = W / (π r2h/ 0.99707) (3.1)

   = Weight (g) × 0.0048417/Height (in.)

   W = Weight of specimen in grams

   r = radius in cm.

   h = height in cm.
6. Calculate percent air voids, VMA and voids filled with asphalt based on aggregate specific gravity.
7. Plot VMA curve versus asphalt content.
8. Select the optimum asphalt content at the lowest point on VMA curve.

Step 2 - Drain-Down Test

Perform the drain test in accordance with the GDT – 127 (2005) Method for determining Drain Down characteristics in Un-compacted Bituminous Mixtures, or AASHTO T 305-97 (2001). A mix with an optimum AC content as determined in Steps 1-2 is placed in a wire basket having (1/4 inch) mesh openings and heated 25º F (13.9º C) above the normal production temperature, which is typically around 350º F (176.7º C) for one hour. The amount of asphalt cement, which drains from the basket, is measured. If the sample fails to meet the requirements of maximum drain down of 0.3%, increase the fiber content by 0.1% and repeat the test.

Step 3 - Boil Test

Perform the boil test according to GDT – 56 (2005) or ASTM D 3625 with complete batch of mix at optimum asphalt content as determined in Step 2 above. If the sample treated with hydrated lime fails to maintain 95% coating, a sample shall be tested in which 0.5% liquid anti stripping additive has been used to treat the asphalt cement in addition to the treatment of aggregate with hydrated lime.

3.1.2 Development of Compaction Criteria for PFC Mixtures Using the Superpave Gyratory Compactor

It is a well-known fact that during compaction in the field, a stage is reached where the aggregate resistance to compaction increases considerably. In other words, there is a great degree of interlocking between the aggregates. Hence, during compaction in the laboratory it becomes important to identify the stage at which the mix exhibits this interlocking. This point of interlocking is called the Locking Point. This concept was first defined by Vavrik and Carpenter (1998) for dense graded HMA. The following section focuses on identification of the locking point for friction course mixtures.

3.1.2.1 Materials

The two mixtures used for establishing the compaction criteria for OGFC mixtures included a Nova Scotia granite FC-5 and an oolitic limestone FC-5 from FDOT test sections on SR-27 in Highlands County. These two mixtures were placed on SR-27 in Highlands County, in July 2003, as part of a friction course study. The gradations for these mixtures are as shown in Table 3-3. The granite was treated with hydrated lime for prevention against stripping.

Table 3-3. Gradations for US-27 (Highlands County) FC-5 Granite and FC-5 Limestone Mixtures.

Sieve Size	**Percentage by Weight Total Aggregate Passing Sieves**	
	FC-5 Granite	**FC-5 Limestone**
19.0 mm	100	100
12.5 mm	96	91
9.5 mm	75	67
4.75 mm	22	23
2.38 mm	10	10
1.18 mm	7	7
600 μ	5	7
300 μ	5	5
150 μ	4	5
75 μ	3.1	4.0

For both FC-5 mixtures, the binder used was PG 67-22 with 12% ground tire rubber. The binder type and binder content used for the field section is as shown in Table 3-4. Mineral fiber was added at 0.4% by weight of the mixture.

Table 3-4. Binder Used in US-27 (Highlands County) Test Section Mixtures.

Type	Binder Type	Binder Content (%)
FC-5 Limestone	ARB-12	6.40%
FC-5 Granite	ARB-12	6.00%

3.1.2.2 Gyratory Compaction of the Friction Course Mixtures

Initially the FC-5 limestone and FC-5 granite mixtures were compacted to 125 gyrations in the Superpave Gyratory Compactor, in order to ensure that a full compaction curve was generated. This was used to evaluate the compaction level for these mixtures. The bulk specific gravity (G_{mb}) of the mixes was obtained from the compaction data through dimensional analysis. This calculated bulk specific gravity was used to calculate air voids. Dimensional analysis was used because of the difficulties encountered with obtaining an accurate value of G_{mb} in the laboratory due to the open texture of the mix (Eason, 2004). The bulk specific gravity, maximum theoretical specific gravity, and the air voids for the mixtures are listed in Table 3-5.

Table 3-5. Air Voids After 125 Gyratory Revolutions.

Type	G_{mb}	G_{mm}	% Air Voids
FC-5 Limestone	2.012	2.336	13.87
FC-5 Granite	1.958	2.441	19.78

The graphs of percentage G_{mm} vs. the number of gyrations (N) for all the compacted specimens are shown in Figures 3-1 and 3-2. The compaction curve (% G_{mm} vs. N) follows closely a logarithmic relationship. Hence, linear regression was performed using a logarithmic relation.

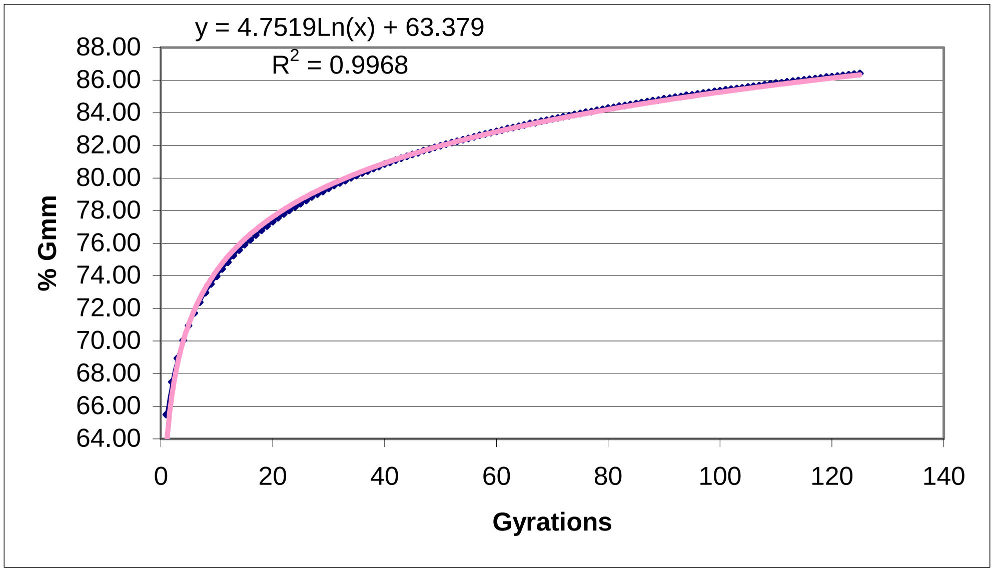

Figure 3-1. Compaction Curve for FC-5 Limestone (Purple line denotes best fit regression; black line denotes compaction data results).

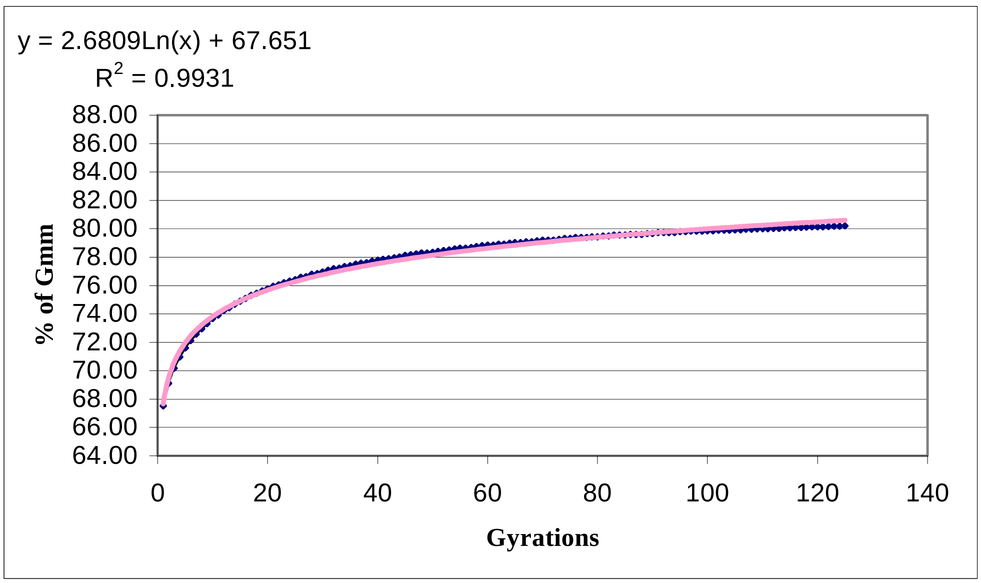

Figure 3-2. Compaction for FC-5 Granite (Purple line denotes best fit regression; black line denotes compaction data results).

The equation of the regression curve was as follows:

$$\%G_{mm} = m * \ln(Ngyr) + c \qquad (3.2)$$

i.e., $N_{gyr} = \exp((\%G_{mm} - c)/m)$

Where m is the slope of the curve at a given gyration and c is a regression constant.

The **locking point** (Vavrik and Carpenter, 1998) is defined as the first three gyrations that are at the same height preceded by two gyrations at the same height (the height is in millimeters and rounded to a single decimal place). Based on this definition, it was found that compaction of FC-5 with limestone did not yield any locking point, which meant that the limestone had probably undergone crushing during compaction. Further, even though the locking point was identified for the FC-5 with granite, the gyrations seemed to be on the higher side since the air voids had more or less reached a constant value by then. The locking point following Vavrik and Carpenter's (1998) definition for the mixtures studied are listed in Table 3-6.

Table 3-6. Locking Point for the Mixtures
Following Vavrik and Carpenter's (1998) Approach

Type	**Locking Point as per Varik and Carpenter (1998) (# of Gyrations)**
FC-5 With Limestone	No Locking Point
FC-5 With Granite	76, 83

To check for the breakdown of the aggregate, the gradations of the mixes were obtained by reflux extraction and compared with the original. Table 3-7 shows a comparison between the gradation from the original (Job Mix Formula) and the extracted gradation. Looking at the gradation, it becomes clear that crushing has taken place in the case of the limestone, whereas for FC-5 with granite there seems to be no significant difference between the gradations. However, the compaction curve indicates that for these mixtures there is no significant change in the air voids at higher gyrations, indicating that the mixture has already "locked up." Thus, the locking point should be below the existing level of gyrations. Hence, instead of looking at the height of compaction, the rate of change of compaction was evaluated, which may be a better indication of resistance to compaction.

Table 3-7. Gradations After Extraction for (a) FC-5 Limestone and (b) FC-5 Granite.

(a)

Gradation	19 mm	12.5 mm	9.5 mm	4.75 mm	2.36 mm	1.18 mm	0.6 mm	0.3 mm	0.15 mm	0.075 mm
Original	100	91	67	23	10	7	7	5	5	4.0
After 125 Gyrations	100	93.77	76.13	31.32	17.29	12.88	10.61	8.95	6.92	5.25
Difference	0	2.77	9.13	8.32	7.29	5.88	3.61	3.95	1.92	1.25

(b)

Gradation	19 mm	12.5 mm	9.5 mm	4.75 mm	2.36 mm	1.18 mm	0.6 mm	0.3 mm	0.15 mm	0.075 mm
Original	100	96	75	22	10	7	5	5	4	3.1
After 125 Gyrations	100	95.60	75.13	22.08	11.42	8.06	6.04	4.71	3.74	3.12
Difference	0	-0.4	0.13	0.08	1.42	1.06	1.04	-0.29	-0.26	0.02

From studying the rate of change of slope of the compaction curve, a modified locking point was identified for these friction course mixtures. Again, locking point being the point beyond which the rate of resistance to compaction increases significantly, it implies that at this stage, the rate of change of compaction decreases significantly, as shown in the following section.

3.1.2.3 Initial Study

Since the focus of the evaluation was on resistance to compaction, the exact nature of the compaction was studied by monitoring the rate of change of compaction. The decrease in the rate of compaction is directly proportional to the increase in resistance to compaction. This premise was used to identify the point of maximum resistance to compaction. It was observed that the compaction curve became linear beyond a certain gyration level. This meant that compaction had reached a stage where no further decrease in rate of compaction was possible and this stage was the stage of maximum resistance to compaction. Hence, the point beyond which the compaction curve became linear was identified and it was observed that beyond 50-60 gyrations the curve more or less became linear in nature. This has been presented in Figures 3-3 and 3-4. Thus, the points from the visual identification served as reference values for identifying the locking points for these mixtures.

It was observed that the compaction curve followed a logarithmic trend. To identify the locking point, the rate of change of slope of compaction curve was used. The stage at which the rate of change of compaction was insignificant, was essentially the point of maximum resistance to compaction.

Using the logarithmic regression of the compaction data, the rate of change of slope can be obtained as:

$$y = m * \ln(x) + c \tag{3.3}$$

Rate of compaction = $dy/dx = m/x$ (at any $x=N$)

Rate of change of slope of compaction curve = $d^2y/dx^2 = -m/x^2$ (at any $x=N$)

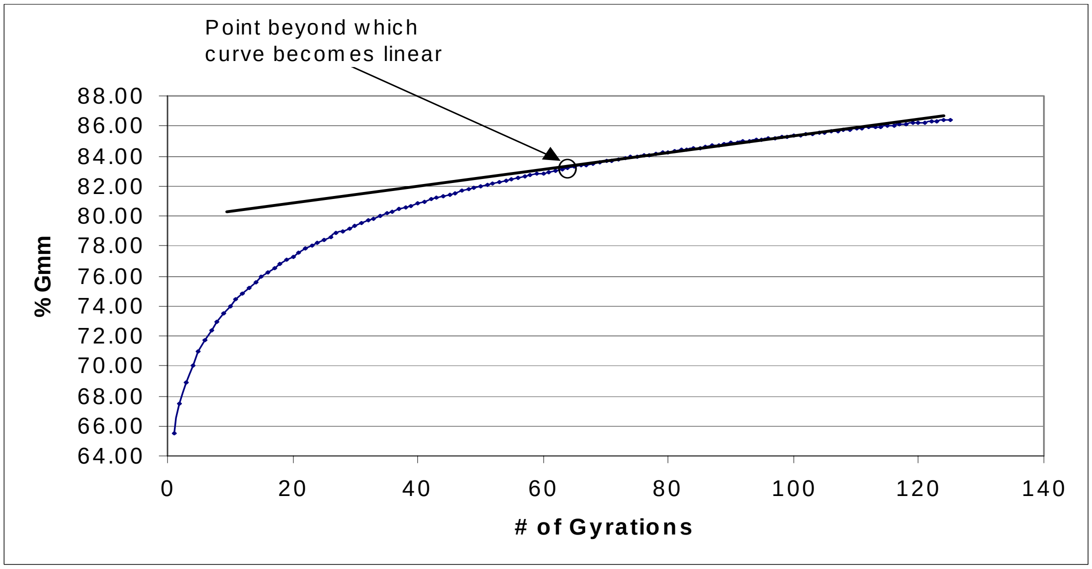

Figure 3-3. Locking Point for FC-5 with Limestone by Visual Observation.

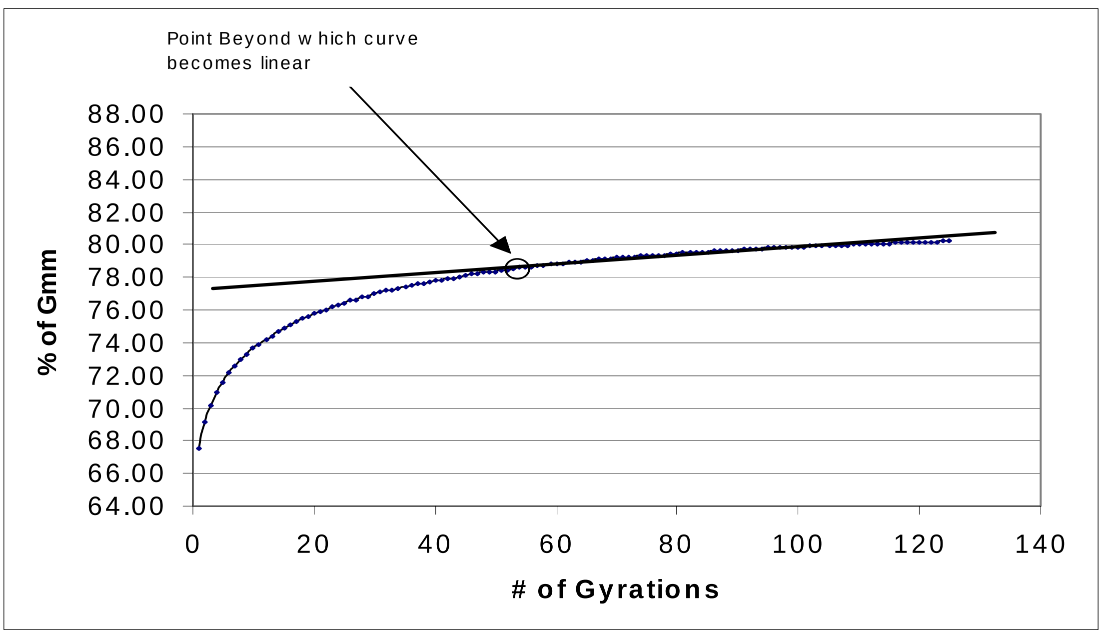

Figure 3-4. Locking Point for FC-5 with Granite by Visual Observation.

Based on the above idea the locking point was identified as the point at which two gyrations at the same gradient of slope were preceded by two gyrations at the same gradient of slope. The gradient was taken up to four decimal places (as shown in Table 3-8 for FC-5 granite). The reason this was chosen as the locking point was based on the fact the change in air voids was

Table 3-8. Locking Point Based on Gradient of Slope.

FC-5 Granite	
# of Gyrations	**Gradient of Slope**
39	0.0018
40	0.0017
41	0.0016
42	0.0015
43	0.0014
44	0.0014
45	0.0013
46 (LP)	0.0013
47	0.0012
48	0.0012
49	0.0011
50	0.0011

insignificant at this stage and that this trend was consistently observed in both mixtures. In addition, the compaction level as identified from visual observation was around 50-60. Thus, based on the above study, the locking points for theses mixtures were identified as shown in Table 3-9.

Table 3-9. Locking Points of All Mixtures Based on Gradient of Slope.

Mixtures	Locking Point
FC-5 limestone	56
FC-5 granite	46

Thus based on above concept the locking points for FC-5 with limestone and the FC-5 with granite were 56 and 46, respectively. The specimens were compacted again to these gyrations and extraction of asphalt was performed to obtain the gradations after compaction. The results of the gradation analysis after extraction are as plotted in Figure 3-5.

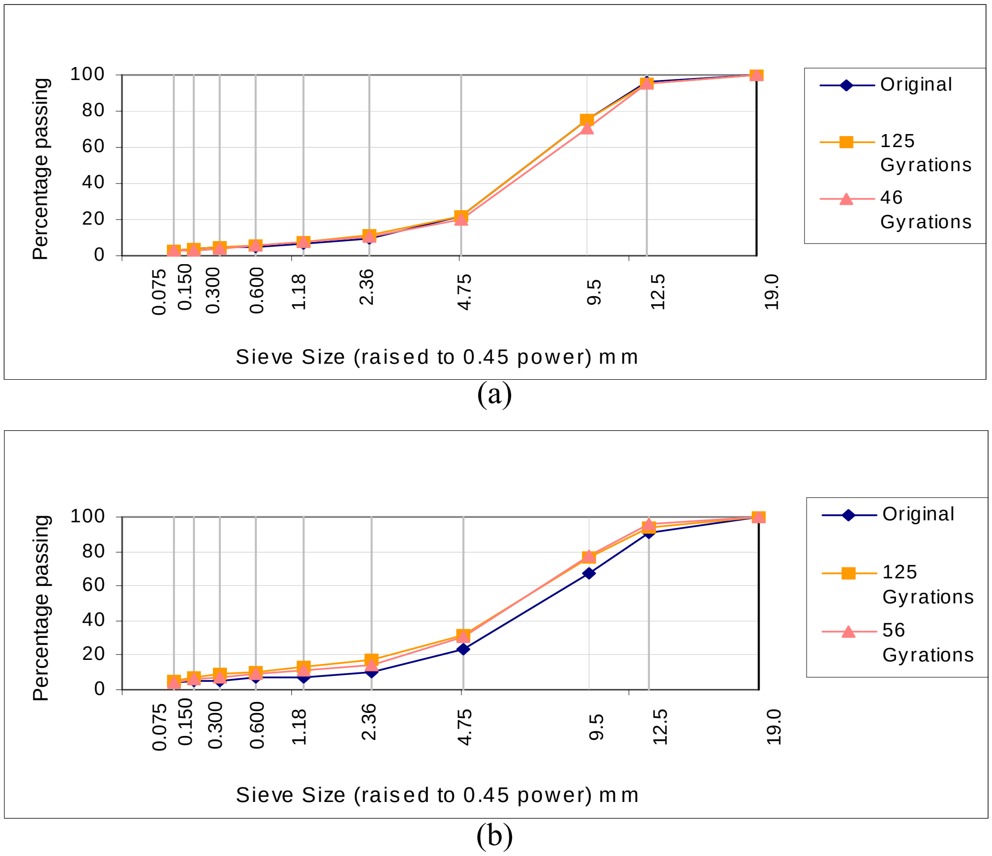

Figure 3-5. Gradations After Extraction for (a) FC-5 Granite and (b) FC-5 Limestone.

Interestingly, even when the gyrations were reduced to 56 from 125 for FC-5 with limestone, the same amount of breakdown was observed. This clearly indicated that in case of the limestone mixture, the breakdown occurred in the initial stages itself i.e., at very low gyrations. Hence, even if the gyrations were to be further reduced, the breakdown was still going to persist. For the FC-5 with granite, the gradation appears to be nearly the same as that of the original gradation.

Thus, from the above the study it is clear that, though the locking point of each of these mixtures differed slightly from each other, it was around 50 gyrations. Thus based on this study, the rate of change of compaction appears to be a suitable criterion for determining the locking point of OGFC mixtures. Furthermore, since it appears that 50 gyrations result in a state that is close to the locking point, it was decided to pick 50 gyrations as the appropriate compaction level for friction course mixes. This compaction level will be compared to densities obtained from field cores from the US-27 Project in Highlands County, Florida, later in this report.

In summary, the steps involved in identifying the locking point based on the above-discussed concepts are as follow:

1. Fitting a Logarithmic curve to the compaction curve obtained from the Superpave Gyratory Compactor.
2. Obtaining the gradient of the slope of the compaction curve by taking the double derivative of the equation of the regressed curve.
3. The locking point is identified as the point at which two gyrations at the same gradient of slope were preceded by two gyrations at the same gradient of slope. This is close to 50 gyrations for the two OGFC mixtures evaluated.

Finally, based on compaction up to 50 gyrations, a comparison is shown in Table 3-10 between measured air voids from laboratory compacted specimens after 50 gyrations and in-place air voids from field cores. The FC-5 layer from the field cores was carefully sliced off the rest of the core sample, and subsequently tested in the Corelok device for determination of bulk specific gravity. The air voids obtained for these mixtures are listed in Table 3-10. The results show that the percent air voids in the field cores immediately after field compaction, are significantly higher than those obtained using the Superpave gyratory compactor. However, after six months of trafficking, the mixtures have consolidated and the aggregate structure has keyed into a more densely packed arrangement, resulting in the percent air voids for both the FC-5 limestone and granite mixtures now being slightly lower than those obtained from the Superpave gyratory compactor. In summary, these results indicate that the selection of 50 gyrations appears to be within the range of air voids obtained after compaction and trafficking in the field for the mixtures tested. However, more work needs to be performed to evaluate the appropriateness of the 50 gyration compaction limit, using FC-5 mixtures of varying gradations and aggregate types before selecting a final design number of gyrations in the laboratory.

Table 3-10. Comparison of Percent Air Voids From Gyratory Compacted Laboratory Specimens and Field Cores.

Mixture Type	Maximum Specific Gravity (Obtained from Field Mixtures)	Laboratory Compacted Specimens (50 gyrations)		Field Compacted Specimens Immediately after construction – 7/1/03)		Field Compacted Specimens (6 months after construction – 12/1/03)	
	G_{mm}	G_{mb}	% Air Voids	G_{mb}	% Air Voids	G_{mb}	% Air Voids
FC-5 Limestone	2.336	1.925	17.60	1.754	24.90	1.964	15.92
FC-5 Granite	2.441	1.916	21.52	1.757	28.03	1.938	20.63

3.2 Verification of Florida Porous Friction Course Mixture Design

3.2.1 Materials

3.2.1.1 Aggregate and Gradation Selection

An existing Georgia PEM gradation obtained from the Georgia DOT was used as a starting point in the PFC mixture design. Figure 3-6 shows the gradation for the Georgia PEM used. Interestingly, the Georgia DOT mixture design follows the middle of the specified gradation band on the coarse side, transitioning to the maximum allowable fines content on the fine

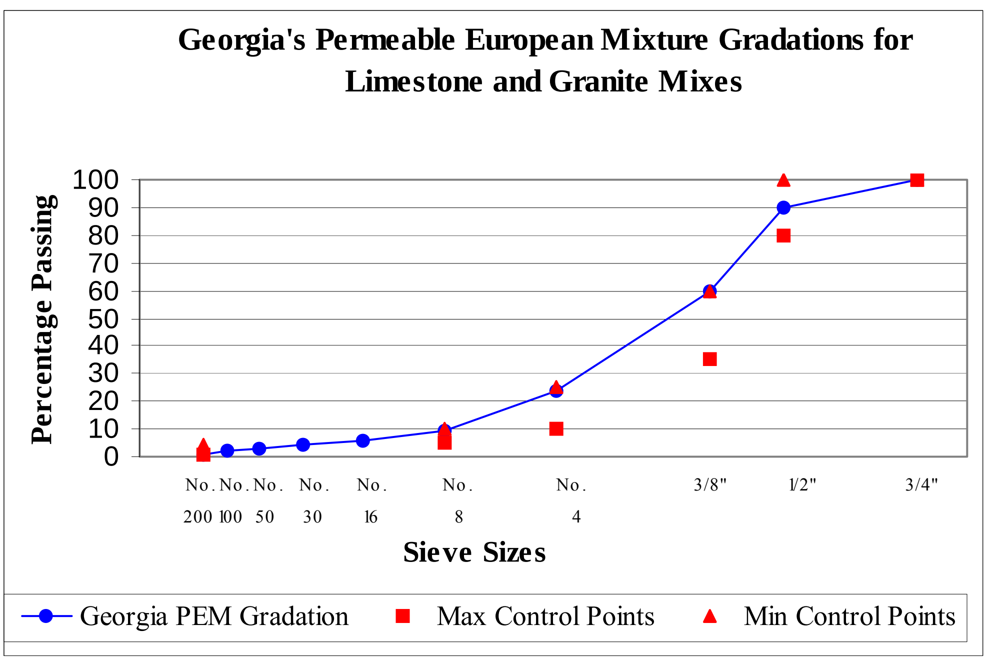

Figure 3-6. Georgia's Permeable European Mixture Gradation Band.

side. This selection of gradation will likely result in a good coarse aggregate to aggregate contact structure, as well as ensuring the highest possible amount of asphalt binder in the mixture. Two types of aggregate are used, namely Nova Scotia granite and oolitic limestone from South Florida (White Rock). The same JMF is used for both granite and limestone mixtures. The Job Mix Formula for the Nova Scotia granite was composed of No. 7, No. 789 stone and granite screenings. The Job Mix Formula for the Whiterock limestone was composed of S-1A stone, S-1B stone and limestone screenings. Hydrated lime (1% by weight of aggregate) was used as anti-stripping agent for the granite aggregates. All aggregates were heated to 350º F ± 3.5º F as specified in GDT 114 Test Method: B Section C (1996). Tables 3-11 and 3-12 show the compositions of the PFC-limestone and PFC-granite job mix formula.

Table 3-11. Composition of PFC-Limestone Gradation JMF.

Type	**S1A**	**S1B**	**Scrns**	**JMF**	**Control Points**	
% Amount	**55.56**	**37.37**	**7.07**	**100**	**Max**	**Min**
Sieve Size						
19	100	100	100	100	100	100
12.5	82	100	100	90	100	80
9.5	28	99	100	60	60	35
4.75	3	39	99	23	25	10
2.36	2	8	70	9	10	5
1.18	2	3	54	6		
0.6	1	1	40	4		
0.3	1	1	30	3		
0.15	1	1	13	2		
0.075	1	1	2	1	4	1

3.2.1.2 Binder and Mineral Fiber

SBS modified PG 76-22 asphalt was used in the mixture design. Mineral fiber 0.4% by weight of total mix, was added to mix in order to avoid binder drain drown. Chemical composition of the mineral fiber is Vitreous Calcium Magnesium Aluminum Silicates. Mineral fibers were pulled apart into fine fragments before adding to the aggregates, to promote even distribution throughout the mixture.

Table 3-12. Composition of PFC-Granite Gradation JMF.

Type	#7	#789	Scrns	Lime	JMF	Control Points	
% Amount	55	37	7	1	100	Max	Min
Sieve Size							
19	100	100	100	100	100	100	100
12.5	82	100	100	100	90	100	80
9.5	28	99	100	100	60	60	35
4.75	2	39	99	100	23	25	10
2.36	2	6	69	100	9	10	5
1.18	2	2	46	100	6		
0.6	1	1	30	100	4		
0.3	1	1	17	100	3		
0.15	0	1	7	100	2		
0.075	0	0	1	100	1	4	1

3.2.2 Selection of Optimum Asphalt Content

Based on experience, the Georgia DOT procedure almost always results in a design asphalt content of 6.0 percent, when Georgia granite aggregates are used (Eason, 2004). However, following the GDOT GDT-114 (1996) procedure, three trial mixtures were prepared at different asphalt contents. The trial asphalt content of 5.5%, 6% and 6.5% were selected for the Nova Scotia granite blend for choosing the asphalt content that results in a minimum VMA. As per GDT 114 (1996), the specified range of percent asphalt content is 5.5%-7.0%.

As a note, based on the early experience with the use of only three trial asphalt contents to obtain an optimal asphalt content, it was observed that it is necessary to use four trial asphalt contents for determining the optimum asphalt content. Choosing only three asphalt contents will always result in one of the chosen asphalt contents to show a minimum, whereas choosing four asphalt contents will result in a true minimum that can be verified. Figure 3-7 shows an example in which determination of an incorrect “optimum” asphalt content is obtained from three trial asphalt contents. In this case, the optimum asphalt content three trials resulted in a design asphalt content of 7.0 percent, compared to the correct optimum asphalt content 6.5 percent.

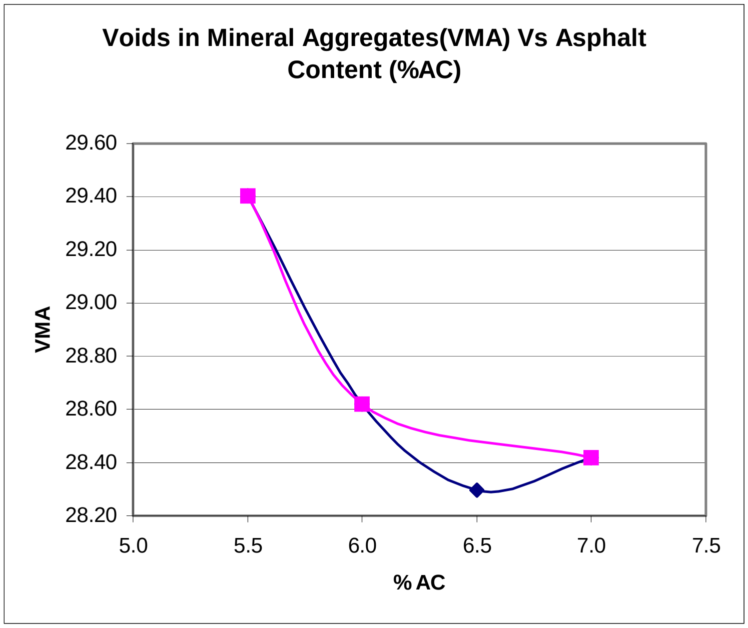

Figure 3-7. Example of Determination of Inconsistent Optimum Asphalt Content.

Because of this reason, a broader range of trial asphalt contents was used for the lime-stone mixture, namely 5.5%, 6.0%, 6.5% and 7%. For each trial asphalt content three specimens were prepared.

<u>3.2.3 Mixing and Compaction of Samples for Determination of Bulk Specific Gravity</u>

Sieved aggregates from each stockpile were batched by weight of 4400 g for each pile. Three specimens were prepared for each trial percentage. Hydrated lime 44 g (1.0% of aggregate weight) is added to batched samples. Aggregates, tools, mixing drum, shredded fibers and the asphalt binder were heated to 330º F ± 3.5° F for a minimum of 3 hours. Aggregates were mixed with asphalt at all trial asphalt contents. Just before mixing, add the required amount of mineral fibers to the aggregate. Once the specimen is mixed it is placed in a clean metal tray. Due to the presence of the SBS in the asphalt binder, these mixtures tend to be "sticky" making the mixing somewhat challenging. In particular, it is important to ensure that there is minimal fines (less than 0.1 percent) while retrieving the mix from the mixing drum. After mixing, mixtures were short-term aged for two hours at 320º F ± 3.5° F as per AASHTO PP2 (1994).

The specimens are compacted to 50 gyrations using the Superpave gyratory compactor. The angle of gyration during compaction is 1.25 degrees. From prior experience, compacted samples should not be retrieved from the molds immediately. They should be allowed to cool for 1 hr and 45 min before extracting the specimens from the molds. Once the specimen is ejected from the mold, it is allowed to cool for another 5 minutes at ambient room temperature before handling. It was found that if a sufficient period for cooling of the specimen after extraction from the mold was not allowed (especially for granite mixtures), small aggregate particles would tend to dislodge and stick to gloves due to the high specimen air voids. Finally, it was found that it was necessary to allow the compacted specimens to cool at ambient room temperatures for another 24 hours before processing them further.

3.2.4. Determination of Optimum Asphalt Content

The determination of bulk specific gravity in accordance with AASHTO T166 (2000) cannot be reliably conducted on the PFC mixtures because of their high air void contents. The determination of Saturated Surface Dry (SSD) weight of the compacted specimens is not reliable for mixtures at these high air void contents (e.g., Cooley et al. 2002). Therefore, the bulk specific gravity (G_{mb}) was determined by dimensional analysis, as described in GDOT-114 (1996). The determination of Theoretical Maximum Specific Gravity (G_{mm}) was performed using the Rice test procedure as per AASHTO T209-99 (2004). For preparation of specimens for determination of Theoretical Maximum Specific Gravity, aggregates are batched by weight of 1100 grams. Two mixes for each trial asphalt percentage are prepared.

Once specimens for all trial asphalt contents had been prepared, the VMA was estimated using dimensional analysis from the Bulk Specific Gravity of the aggregates (G_{sb}). The design asphalt content is selected at the point of minimum VMA. The main purpose of using minimum VMA criterion is to ensure reasonably high asphalt content of the mixture. Secondly, VMA is calculated on a volume basis and is therefore not affected significantly by the specific gravity of aggregate.

Figures 3.8 and 3.9 show a summary of the volumetrics for the limestone and granite mixtures. Optimum asphalt contents of PFC mixtures were found to be 6.5% (VMA of 28.3%) and 6.0% (VMA of 30.2%) for the limestone and granite mixtures, respectively. The porous nature of limestone resulted in a higher optimum asphalt content.

Effective Sp. Grav. of Agg.	% AC	G_{mm}	G_{mb}	VMA (%)	VTM (%)	VFA (%)
2.513	5.5	2.323	1.877	29.40	19.21	34.67
	6.0	2.314	1.908	28.62	17.54	38.71
	6.5	2.298	1.927	28.30	16.16	42.89
	7.0	2.286	1.934	28.42	15.39	45.84

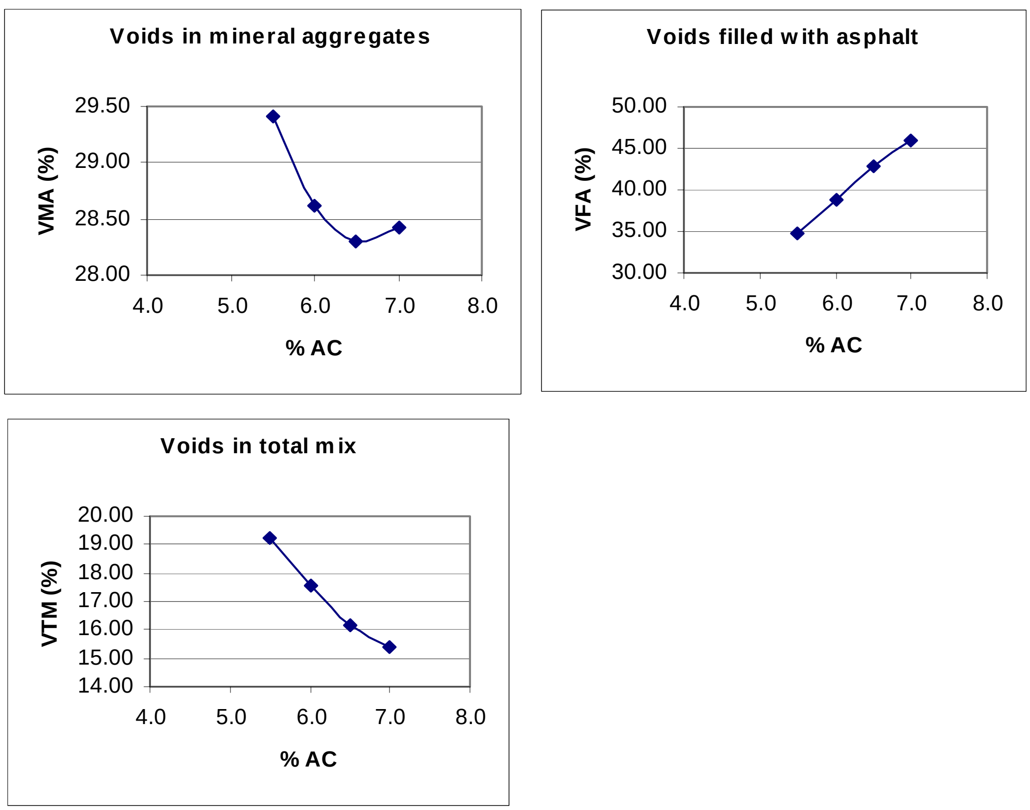

Figure 3-8. Mix Design of Limestone PFC Mixture.

Effective Sp. Grav. of Agg.	% AC	G_{mm}	G_{mb}	VMA (%)	VTM (%)	VFA (%)
2.641	5.5	2.442	1.936	30.74	20.72	32.60
	6	2.414	1.961	30.23	18.78	37.86
	6.5	2.389	1.967	30.38	17.68	41.82

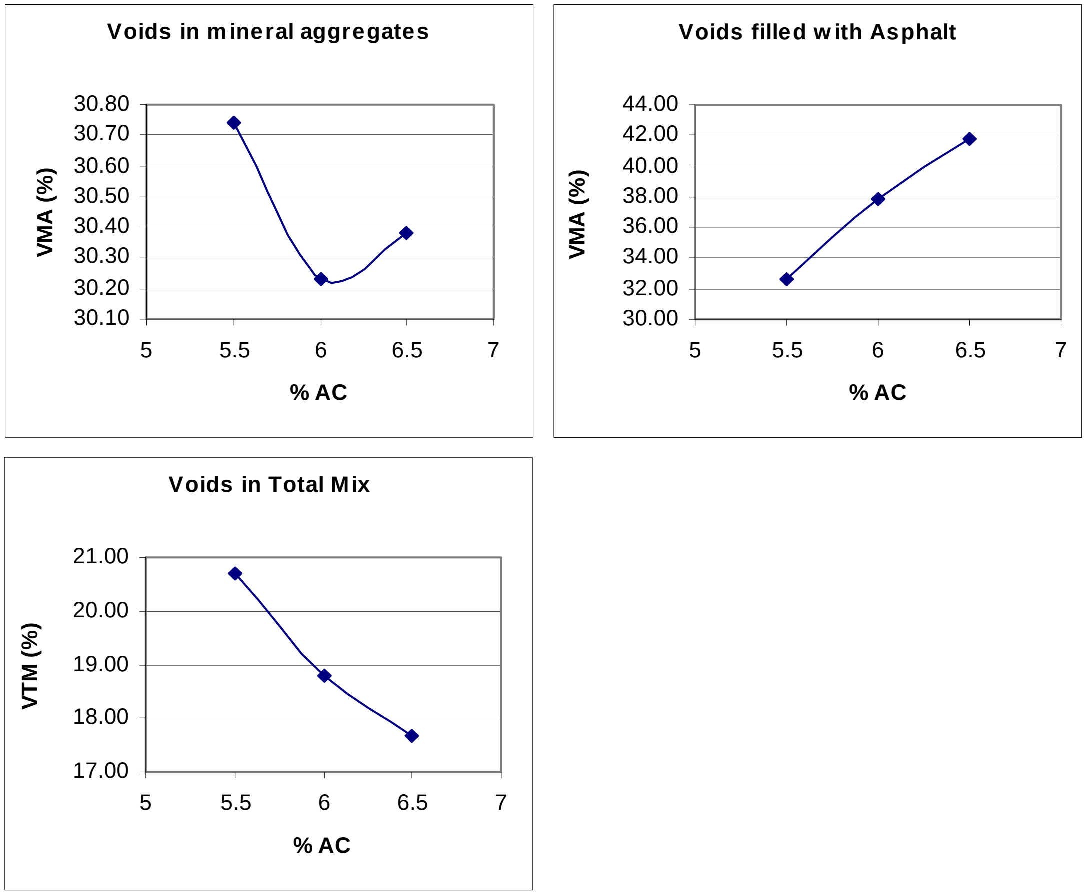

Figure 3-9. Mix Design of Granite PFC Mixture.

3.2.5 Evaluation of Compaction Level for the Porous Friction Course Mixtures

Using the final design asphalt contents for the PFC Limestone and Granite mixtures, the Locking Point was identified as the point at which two gyrations at same gradient of slope were preceded by two gyrations at same gradient of slope. Table 3-13 shows the locking point for GPEM limestone and granite mixtures. Both mixtures had a locking point around 50 gyrations (54 for limestone and 48 for granite). Thus, based on these two PFC mixtures, it appears that the

Table 3-13. Locking Point for the PFC Mixtures.

Mixtures	Locking Point
GPEM Limestone	54
GPEM Granite	48

use of 50 gyrations to determine the design asphalt content is reasonable based on the observed locking points.

3.2.6 Selection of a Criterion for Ensuring PFC Mixture Durability

The Georgia DOT requires a minimum asphalt film thickness for their GPEM mixtures to ensure durability of the mixture. A mixture with a low asphalt film thickness is expected to damage more than a mixture with a thicker film thickness. The other common approach for ensuring durability of OGFC/PFC mixtures includes the use of the Cantabro test (e.g., Huber, 2000). The Cantabro test was developed for use with high-strength silicious aggregates in Europe. The premise of the test is that there will be no aggregate breakdown during the test and thus any loose particles resulting from the test are an indirect measure of the binder content. There is a concern as to the suitability of this test for oolitic limestone aggregates from Florida, since the test would likely result in excessive breakdown of the oolitic limestone aggregate, which may cloud the interpretation of the Cantabro test results. Therefore, the use of a "film thickness criterion" for ensuring durability of OGFC/PFC mixtures appears to be better suited to Florida conditions.

The Georgia DOT does not account for absorption in their calculation of film thickness for GPEM mixtures. This assumption is based on the fact that the granite aggregates that are predominant in Georgia do not have high absorption (Eason, 2004). However, oolitic limestone from Florida has relatively high absorption as compared to silicious granite aggregates.

However, the other argument that can be made is that even though some of the asphalt is absorbed into the aggregates, the absorbed asphalt is still there and may provide a stabilizing bond between the mastic and the aggregates. A suitable analogy may be to think about the absorbed asphalt as "roots" that penetrate the aggregate surface and provide an improved bond between the asphalt mastic and the aggregates. This "root" effect may become more predominant in high absorption aggregates, like oolitic limestone, and may explain in part why oolitic limestone mixtures tend to exhibit higher durability against moisture damage than granite mixtures. In addition, oolitic limestone tends to absorb asphalt into the surface of the aggregate, as compared to granite, thus helping durability. Another consideration is that due to the higher asphalt absorption of oolitic limestone mixtures, their design asphalt content is always higher than for an equivalent (same gradation) granite mixture. However, the calculated film thickness based on effective asphalt content is almost always significantly lower in oolitic limestone mixtures than in granite mixtures. This means that using effective asphalt content for film thickness calculations on Florida oolitic limestone mixtures may overly punish these mixtures, which are less susceptible to durability problems in the first place. Therefore, it was recommended that all film thickness calculations would be based on total asphalt content (by weight) rather than effective asphalt content. However, it is important to note that effective asphalt content could also be used which would imply result in a lower film thickness number. This is the way it is recommended to go based on these assumptions. However, one can also use a lower number based on effective asphalt content.

In summary, the use of asphalt film thickness for ensuring adequate PFC/OGFC mixture durability appears to be reasonable. In particular, silicious aggregate-based mixtures with an adequate film thickness should be expected to be more durable than low film thickness mixtures. High absorption limestone mixtures possess an advantage over silicious aggregate mixtures in that the aggregate surface chemistry tends to favor a stronger bond with asphalt and the absorbed asphalt may provide an added level of stability to the asphalt film.

3.2.7 Review of Asphalt Film Thickness Calculation Methods

The first attempts to calculate minimum asphalt film thicknesses were made by Goode and Lufsey (1965). Their method was based on empirical considerations, leading to the development of the theoretical film thickness (e.g., Roberts et al. 1996), which assumes that all aggregates are rounded spheres, with predefined surface areas, which are coated with an even thickness of asphalt film.

The Georgia DOT introduced a modified film thickness calculation for their PFC/OGFC mixtures (Eason, 2004). The Georgia DOT modified film thickness calculation method is based on empirical considerations and yields similar results to the theoretical film thickness calculations.

Recognizing that these "theoretical film thickness" calculations were developed primarily for fine-graded mixtures with very different aggregate structures from those found in coarse-graded mixtures with predominant coarse aggregate stone-to-stone contact, let alone OGFC and PFC mixtures, Nukunya et al. (2001) developed an effective film thickness concept based on a physical model of coarse-graded mixtures. Nukunya et al. (2001) observed that the aggregate structure for fine- and coarse-graded mixtures is fundamentally different, as shown in Figure 3-10. Fine-graded mixtures tend to have more continuous grading such that the fine-aggregates are an integral part of the stone matrix. Coarse mixtures, on the other hand, tend to have aggregate structures that are dominated by the coarse aggregate portion (i.e., stone-to-stone contact).

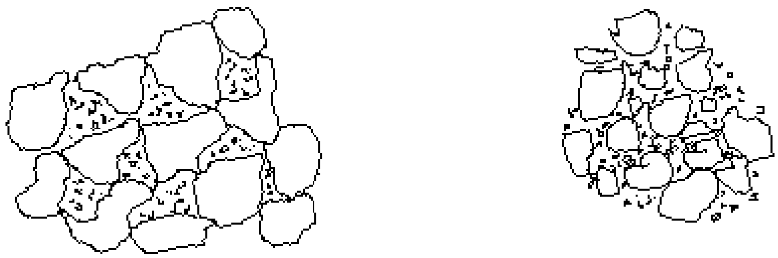

Figure 3-10. Aggregate Structure for Coarse and Fine Mixtures (Nukunya et al. 2001).

Therefore, coarse-graded PFC mixtures are effectively composed of two components: the first one is the interconnected coarse aggregate, and the second component is the fine mixture embedded in between the coarse aggregate particles. The mixture made up of asphalt and fine aggregates coats the coarse aggregate particles, and the fine aggregates within that matrix have access to all the asphalt within the mixture. The result is a film thickness that is greater than that calculated using conventional theoretical film thickness calculation procedures that assume that the asphalt is uniformly distributed over all aggregate particles. To account for the different nature of the aggregate structure in coarse-graded mixtures, a modified film thickness calculation, entitled the "effective film thickness," was developed by Nukunya et al. (2001), in which the asphalt binder is distributed onto the portion of the aggregate structure that is within the mastic.

In the following, the details of the three different approaches to asphalt film thickness calculations will be reviewed (i.e., theoretical film thickness, and effective film thickness, GDOT modified film thickness calculation).

<u>3.2.8 Theoretical Film Thickness Method</u>

This technique for calculating film thickness is based on the calculated surface area factors suggested by Hveem (Roberts et al. 1996), shown in Table 3-14. The film thickness of

Table 3-14. Surface Area Factors by Hveem (Roberts et al. 1996).

Sieve Size	Surface Area Factor
Percentage Passing Maximum Sieve Size	2
Percent Passing No. 4	2
Percent Passing No. 8	4
Percent Passing No. 16	8
Percent Passing No. 30	14
Percent Passing No. 50	30
Percent Passing No. 100	60
Percent Passing No. 200	160

asphalt aggregates is a function of the diameter of particles and the effective asphalt content. The film thickness is directly proportional to the volume of the effective asphalt content and is inversely proportional to diameter of the particle:

$$T_{film} = \frac{V_{eff} \times 1000}{SA \times W_{agg}} \tag{3.4}$$

T_{film} = Film Thickness

SA = Surface Area

W_{agg} = Weight of aggregate

3.2.9 Effective Film Thickness Method (Nukunya et al. 2001)

According to this method only aggregates passing the No. 8 Sieve are taken into account in the calculation of the surface area. Table 3-15 shows the surface area factors used in the

Table 3-15. Surface Area Factor Suggested by Nukunya (2001) for Coarse Aggregate Structure.

Sieve Size	Surface Area Factor
Percent Passing No. 8	4
Percent Passing No. 16	8
Percent Passing No. 30	14
Percent Passing No. 50	30
Percent Passing No. 100	60
Percent Passing No. 200	160

Effective Film Thickness method. Once the surface area is determined, Equation 3.4 is used for calculating Film Thickness.

3.2.10 Modified Film Thickness Method Used by GDOT

Georgia developed this method primarily for PEM mix with granite aggregate. The basic assumption was that the absorption of asphalt is negligible. The method is empirical and assumes that fixed aggregate unit weight per pound of aggregate, based on Georgia aggregates. Hence, the effective film thickness (T_{eff}) is given as:

$$T_{eff} = \frac{[\text{ 453.6 g per lb. divided by \% Aggregate }] - [\text{ 453.6 g per lb.}]}{\text{Surface area in square ft / lb * 0.09290Sq. m per sq. ft. * Sp. gr. of AC}} \quad (3.5)$$

The surface area parameter in Equation (3.5) is calculated following the procedure described previously for the Theoretical Film Thickness.

3.2.11 Establishment of a Film Thickness Criterion for PFC Mixtures

Based on the previous discussion on film thickness calculations, it appears that the effective film thickness (Nukunya et al. 2001) may best capture the behavior of PFC mixtures, as compared to theoretical film thickness or the more empirical GDOT film thickness calculation.

Based on experience, the Georgia DOT requires a minimum film thickness of 27 μ to ensure adequate mixture durability. In order to establish a comparable minimum film thickness criterion based on effective film thickness, a relationship was developed between the two different film thickness calculations, and an equivalent required minimum effective film thickness was established for PFC mixtures.

First, the maximum and minimum specification limits of gradation for GPEM are taken from Table 3-1. Using these upper and lower gradation limits, an “average” gradation band was established. These three hypothetical gradations are provided in Table 3-16. For these three

Table 3-16. Gradations Based on Upper, Lower, and Average Gradation Limits for GPEM Mixtures (GDOT 114, 1996).

GPEM Trial Gradations for Film Thickness Calculations (% Passing)			
Sieve Size	**Max**	**Avg**	**Min**
1½ in. (37.5 mm)	0	0	0
1 in. (25.0 mm)	0	0	0
¾ in. (19.0 mm)	100	100	100
½ in. (12.5 mm)	100	90	80
3/8 in. (9.5 mm)	60	47.5	35
No. 4 (4.75 mm)	25	17.5	10
No. 8 (2.36 mm)	10	7.5	5
No. 16 (1.18 mm)	8.8	6.5	4.2
No. 30 (600 μ)	7.6	5.5	3.4
No. 50 (300 μ)	6.4	4.5	2.6
No. 100 (150 μ)	5.2	3.5	1.8
No. 200 (75 μ)	4	2.5	1

hypothetical gradations, the film thickness is calculated using the GDOT method and the effective film thickness method over the following range of asphalt contents: 5.5%, 5.6%, 5.8%, 6.0%, 6.2%, 6.4%, and 6.5%. The calculated film thickness values for each trial gradation are given in Table 3-17. Using these values, a graph was plotted (see Figure 3-11) showing the relationship between the two different film thickness criteria. From the graph, a polynomial trend line is drawn. Using this equation the for 27 μ GDOT thickness, equivalent Nukunya's effective thickness is calculated to be 33.6 μ, which when rounded up to the nearest whole number is 34 μ.

Table 3-17. Calculated Gradations Based on Upper, Lower, and Average Gradation Limits for GPEM Mixtures (GDOT 114, 1996).

Calculated Film Thickness for Three GPEM Trial Gradations						
	GDOT			**Nukunya (effective)**		
AC (%)	**max**	**avg**	**min**	**max**	**avg**	**min**
5.5	17.1	24.2	41.4	19.73	29.7	60.04
5.6	17.5	24.7	42.1	20.09	30.24	61.13
5.8	18.1	25.6	43.7	20.81	31.32	63.32
6	18.8	26.6	45.4	21.53	32.4	65.5
6.2	19.5	27.5	47	22.24	33.48	67.78
6.4	20.1	28.5	48.6	22.96	34.56	69.87
6.5	20.5	28.9	49.4	23.32	35.1	70.96

3.3 Summary and Discussion

In this chapter, the Georgia DOT mix design for Georgia Permeable European Mixtures was used as a starting point for a new Florida Method for the design of Porous Friction Course Mixtures. The main differences between the two approaches are that gyratory compaction was introduced into the Florida mix design approach and a film thickness criterion for PFC mixtures

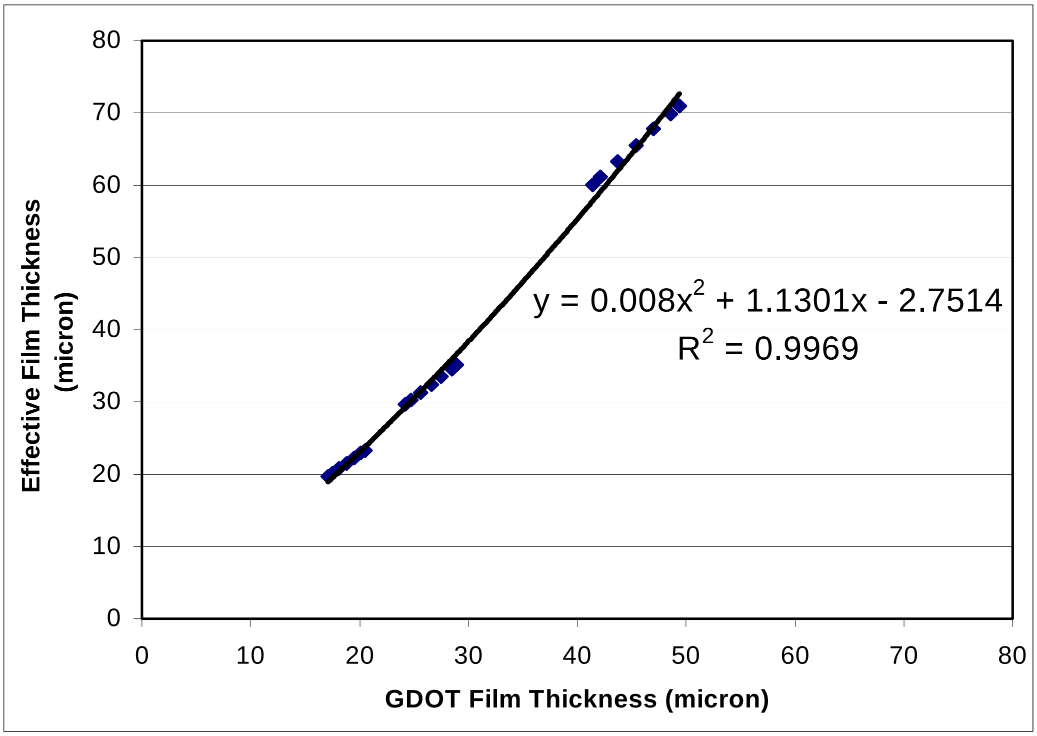

Figure 3-11. Relationship Between GDOT Film Thickness and Effective Film Thickness for PFC Mixtures.

was established using the effective film thickness developed by Nukunya et al. (2001). A modified locking point concept was used as a tool to establish an appropriate gyratory compaction criterion for PFC mixtures. The mixture design approach is otherwise based on the selection of optimal asphalt content at the point of minimum voids in the mineral aggregate (VMA), meaning that any more or any less asphalt binder will result in a less dense mixture. It is not obvious that a VMA-based design criterion necessarily ensures durability. Therefore, an effective film thickness criterion was introduced to ensure adequate mixture durability. This new mixture design method led to the development of a Draft Florida Method and Draft Specifications for the Mixture Design of Porous Friction Course Mixtures.

3.4 References

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CHAPTER 4

FIELD EVALUATION OF NEW FLORIDA POROUS FRICTION COURSE MIXTURE DESIGN

4.1 Project Description

In order to evaluate the performance of porous friction course (PFC) mixtures designed according to the new draft method of mixture design, presented at the end of Chapter 3, a field test section on I-295 was selected by the Florida Department of Transportation (FDOT). Figure 4-1 illustrates the I-295 project location in District 2, in Duval County, Florida. The project starts from the north end of the Trout River Bridge (MP 28.817) and extends to the I-95 intersection (MP 34.507). This highway consists of four 12-foot wide travel lanes, with two lanes in each travel direction (northbound and southbound). In addition to paving both the northbound and southbound lanes with the PFC mixture, a comparison section consisting of a FC-5 open graded friction course was constructed immediately next to the PFC test section.

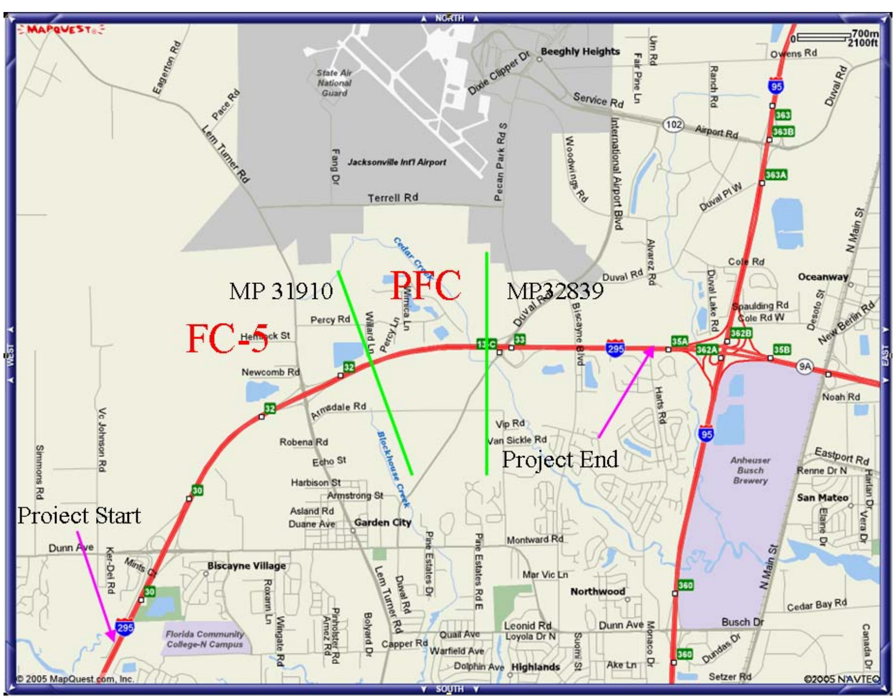

Figure 4-1. I-295 Project Location.

The sections were paved on July 11, 2005. The FC-5 mixture was placed at 0.75 in. compacted thickness (~80 lb/yd^2) and the PFC mixture was placed at approximately 1¼ inch compacted thickness at spread rate of approximately 140 lbs/yd^2.

4.2 Objectives

The objectives of this chapter are:

- To evaluate the rutting resistance of the PFC and FC-5 field mixtures in the laboratory using a modified Asphalt Pavement Analyzer (APA) test with a rubber loading strip in lieu of the traditional hose (Drakos, Roque, and Birgisson, 2005).
- To evaluate the fracture resistance of both field mixtures based on Superpave IDT and the framework of HMA fracture mechanics.

4.3 Porous Friction Course Mix Design

4.3.1 Key Components of the Mixture Design

The FC-5 mixture design followed FDOT specification Section 337-4. The PFC mix design procedure was performed based on FDOT specification Section 337-4, and the Draft Florida Method for PFC Mixture Design, discussed in Chapter 3. Copies of the mix designs are shown in Figures 4-2 and 4-3.

4.3.2 Materials

The aggregate supplier for both the PFC and FC-5 mixtures was Martin Marietta, and the final aggregate blend is composed of the following components:

- #67 Granite Stone from Pit No TM-579/NS-315.
- #78 Granite Stone from Pit No GA-383.
- Granite Screens from Pit No TM-579/NS-315.

Figures 4-2 and 4-3 show the gradation bands and control points used for the I-295 PFC and FC-5 projects as per FDOT specification Section 337. As per the mixture design requirements, hydrated lime was introduced in the mix at the rate of 1% by weight of aggregate. An

STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION
STATEMENT OF SOURCE OF MATERIALS AND JOB MIX FORMULA FOR BITUMINOUS CONCRETE

SUBMIT TO THE STATE MATERIALS ENGINEER, CENTRAL BITUMINOUS LABORATORY, 5007 NORTHEAST 39TH AVENUE, GAINESVILLE, FLA. 32609

Contractor: Atlantic Coast Asphalt Company — Address: 5154 Edwards St., Jacksonville, FL 32254

Phone No.: (904) 268-0274 — Fax No.: (904) 268-0365 — E-mail: dbrown@hubbard.com

Submitted By: Don Brown — Type Mix: PFC — Intended Use of Mix: Structural

TYPE MATERIAL	F.D.O.T. CODE	PRODUCER	PIT NO.	DATE SAMPLED
1. # 67 Stone	54	Martin Marietta Aggregates	TM-579 NS-315	02 / 10 / 2005
2. # 78 Stone	54	Martin Marietta Aggregates	GA-383	02 / 10 / 2005
3. Screenings	23	Martin Marietta Aggregates	TM-579 NS-315	02 / 10 / 2005
4. Hydrated Lime		Global Stone Corp.	Luttrel Co. TN	
5. PG 76-22				
6.				

PERCENTAGE BY WEIGHT TOTAL AGGREGATE PASSING SIEVES

	Blend	20%	70%	9%	1%			JOB MIX	CONTROL	RESTRICTED
	Number	1	2	3	4	5	6	FORMULA	POINTS	ZONE
SIEVE SIZES	3/4" 19.0mm	100	100	100	100			100	100	
	1/2" 12.5mm	60	95	100	100			89	85 - 95	
	3/8" 9.5mm	45	62	100	100			62	55 - 65	
	No. 4 4.75mm	8	6	91	100			15	15 - 25	
	No. 8 2.36mm	4	4	61	100			10	5 - 10	
	No. 16 1.18mm	3	3	38	100			7		
	No. 30 600µm	2	3	22	100			5		
	No. 50 300µm	2	3	15	100			5		
	No. 100 150µm	2	2	7	100			3		
	No. 200 75µm	1.0	1.0	3.5	100.0			2.2	1 - 4	
	G_{sb}	2.625	2.772	2.610	2.600			2.724		

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

LD 05-2545A (PFC)

Director, Office of Materials
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Effective Date: 03 / 30 / 2005

Expiration Date: 03 / 30 / 2008

Figure 4-2. I-295 PFC Construction Mix Design.

STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION
STATEMENT OF SOURCE OF MATERIALS AND JOB MIX FORMULA FOR BITUMINOUS CONCRETE

SUBMIT TO THE STATE MATERIALS ENGINEER, CENTRAL BITUMINOUS LABORATORY, 2006 NORTHEAST WALDO ROAD., GAINESVILLE, FLA. 32609

Contractor: Atlantic Coast Asphalt Company   Address: 5154 Edwards Street, Jacksonville, FL 322254

Phone No.: (904) 268-0274   Fax No.: (904) 268-0365   E-mail: dbrown@hubbard.com

Submitted By: Atlantic Coast Asphalt Co.   Type Mix: FC-5   Intended Use of Mix: Friction Course

TYPE MATERIAL	F.D.O.T. CODE	PRODUCER	PIT NO.	DATE SAMPLED
1. #78 Stone	54	Martin Marietta Aggregates	GA-383	10 / 13 / 2003
2. #89 Stone	54	Martin Marietta Aggregates	TM-579 NS-315	10 / 13 / 2003
3. M-10 Screenings	23	Martin Marietta Aggregates	TM-579 NS-315	10 / 13 / 2003
4. Hydrated Lime		Global Stone Corp.	Luttrel Co. Tenn.	10 / 13 / 2003
5.				
6. ARB-12				

PERCENTAGE BY WEIGHT TOTAL AGGREGATE PASSING SIEVES

Blend	65%	23%	11%	1%			JOB MIX	CONTROL	RESTRICTED
Number	1	2	3	4	5	6	FORMULA	POINTS	ZONE
3/4" 19.0mm	100	100	100	100			100	100	
1/2" 12.5mm	92	100	100	100			95	85 - 100	
3/8" 9.5mm	57	91	100	100			70	55 - 75	
No. 4 4.75mm	8	38	95	100			25	15 - 25	
No. 8 2.36mm	3	10	47	100			10	5 - 10	
No. 16 1.18mm	1	4	29	100			6		
No. 30 600μm	1	3	20	100			5		
No. 50 300μm	1	3	12	100			4		
No. 100 150μm	1	3	6	100			3		
No. 200 75μm	1.0	1.5	3.5	100.0			2.4	2 - 4	
Gsa									

(SIEVES)

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

SP 03-2890A (FC-5)

Transferred from QA 02-10669A (FC-5)

State Materials Engineer

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Effective Date: 10 / 20 / 2003

Expiration Date: 10 / 20 / 2006

http://materials.dot.state.fl.us/Smo/Bituminous/CentralBitLab/CentralBituminousLab.htm

Figure 4-3. I-295 FC-5 Construction Mix Design.

SBS polymer modified asphalt cement PG 76-22 was used, as well as mineral fiber (0.4% by weight of total mix).

4.3.3 Determination of Optimum AC Content

Four trial asphalt contents were used (5.5%, 5.8%, 6.2% and 6.5%) for each gradation. Materials, including batched aggregates, asphalt and mineral fiber were preheated in the oven at 320º F for 3 hours before mixing. Due to the SBS modified asphalt and the addition of mineral fiber, the mixing temperature was selected at 330º F (165º C), according to the experience of the Pavements Group at the University of Florida. Mixing tools and equipment were also preheated to 350º F (176º C) for at least 30 minutes.

Because of the nature of the SBS modified asphalt, special care and handling should be taken in order to minimize the loss of fines. The mixing procedure was the same for both maximum specific gravity samples (Rice test) and the Superpave gyratory compacted specimens. It is important to not overheat the binder during mixing, as it causes aging of binder.

Before compaction, the mixtures were subjected to Short Term Oven Aging (STOA) for two hours, which includes stirring after one hour. The compaction temperature was reduced to 320º F to avoid draindown of binder during compaction. Fifty (50) gyrations were used to achieve compaction level similar to field after traffic consolidation.

4.3.4 Field Data

Dimensional analysis was used to estimate the bulk specific gravity of the mix during design. However, in order to determine more precisely the volumetric property of the loose field mixtures, the optimum PFC asphalt content was determined following the mix design procedure in Appendix A.

The maximum specific gravity (G_{mm}) for loose PFC mixtures was determined by the Rice Test (AASHTO T209) and bulk specific gravities were determined using the CoreLok machine (ASTM D6752, D6857).

The optimum design asphalt content for the PFC mixtures was 6.0 percent. The effective film thickness (Nukunya et al. 2001) for the PFC mixture was determined to be 35.4 μ, which means that it met the new criterion of requiring an effective film thickness of 34 μ. Similarly, the optimum asphalt content for the FC-5 mixture was established by the FDOT as 6.1 percent.

Table 4-1 shows the results of the G_{mm} and G_{mb} tests performed on the FC-5 and PFC field mixtures. The maximum theoretical specific gravity in Table 4-1 was established using the Rice test. The bulk specific gravity was established by using the CoreLok equipment on sliced friction course specimens obtained from test section field cores.

Table 4-1. I-295 CoreLok Mixture Bulk Specific Gravity Test Results.

Mix Type	Location	G_{mm}	G_{mb}	In-Place Air Voids (%)
PFC	Northbound	2.485	2.013	19.0
	Southbound	2.485	2.028	18.4
FC-5	Northbound	2.336	2.061	11.8
	Southbound	2.336	2.089	10.6

4.3.5 Verification of Locking Point of Final JMF for I-295 PFC Project

Using a laboratory-prepared PFC mixture, the modified locking point for the PFC mixture was checked, following the procedure from Chapter 3. The locking point was found to be 49 gyratory revolutions. It is the point at which two gyrations at same gradient of slope were preceded by two gyrations at same gradient of slope. Hence, this again supports the selection of 50 gyrations as the appropriate compaction level to determine the optimum asphalt content in the laboratory.

4.4 Evaluation of Rutting Potential

4.4.1 Review of Methodology Used for Rutting Evaluation

At present, the most commonly utilized laboratory test for rutting susceptibility of HMA mixtures in Florida is the Asphalt Pavement Analyzer (APA) test. The specimen is rutted with a pressurized rubber hose, and the rut depth is measured using a dial gauge, which measures the single lowest point of the specimen. However, Drakos et al. (2005) developed a new method for evaluating the rutting resistance of mixtures with the APA. They introduced a rubber strip in lieu of the hose to better simulate measured contact stresses between radial truck tires and the pavement. Drakos et al. (2005) also introduced a new method for characterizing and analyzing the entire rutting profile rather than just the single lowest point on the specimen. The new method appears to have greater potential of evaluating a mixture's potential for instability rutting than the original hose loading and single rut-depth measurement configuration. Therefore, the rut resistance of PFC and OGFC mixtures can possibly be better determined through the use of the APA strip loading test.

4.4.2 Modified Rut Test Procedure

All specimens are tested by using a standard rubber loading strip under a 150 lbs wheel load (Drakos et al. 2005). The test procedure is summarized as follows:

- Preheat the specimen in the APA test chamber to 64º C (147º F) for a minimum of 6 hours, but not more than 24 hours before the test.
- Calibrate the steel wheel with the load cell to read a load of 150 ± 5 lb.
- Secure the preheated, molded specimen in the APA, close the chamber doors and allow 10 minutes for the temperature to stabilize.
- Apply 25 load cycles and then take initial (datum) measurements.
- Place the specimen back in the APA, close the chamber doors and allow 10 minutes for the temperature to stabilize.

- Restart the APA and continue rut testing.
- Repeat the measurement procedure at 8000 cycles.

4.4.3 Rut Depth Analysis

Each specimen requires two profile measurements at three locations to capture its rutting profile throughout an 8000 cycle test. To proceed with the data analysis, the resulting profiles are scanned as bitmap images (.bmp) and digitized with the help of a program called Grafula 3 (Drakos, 2005). The traditional way of calculating the rut depth for an APA specimen is to take two measurements – the lowest point at the beginning and the lowest point at the end of the test – and report the difference after 8000 cycles. Hereafter, the traditional way of measuring APA rut depth is identified as the Absolute Rut Depth (ARD). The Differential Rut Depth (DRD) is defined as the difference of the lowest point at the beginning of the test and the highest point recorded at the end of the test, as shown in Figure 4-4.

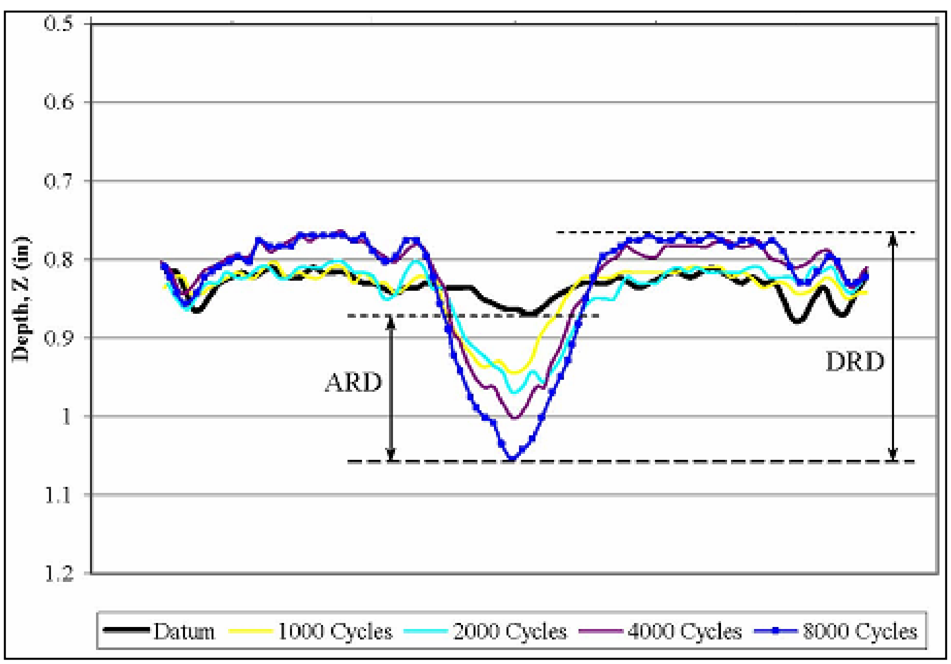

Figure 4-4. A Graphical Illustation of Differential Rut Depth (DRD) and Absolute Rut Depths (ARD).

4.4.4 Rut Mode Identification

The two common mechanisms for permanent deformation of asphalt mixtures in the field are consolidation rutting, which is due to a reduction in air voids due to traffic densification, and instability rutting, which is due to shear-induced deformation within the mixture. Drakos et al. (2005) developed a method to determine whether a mixture is likely to be more susceptible to consolidation rutting or instability rutting.

Using the rut profile measurements, a polymomial is used to fit to the original and deformed surface profile data. The two areas shown in Figure 4-5 are calculated, and used to determine the percentage of area change (ΔA) as:

$$\%\text{Area change} = \frac{A_i - A_f}{A_i} * 100 \tag{4-1}$$

in which the A_i refers to the area from the original profile and A_f refers to the area determined from the final profile. A positive percent area change implies that the mixture may experience instability rutting, whereas a negative ΔA indicates that the mixture is more likely to experience consolidation rutting (Drakos et al. 2005).

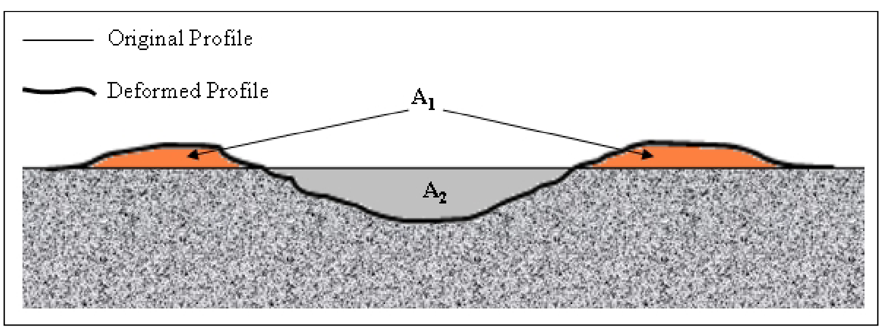

Figure 4-5. Sample of Area Change Analysis.

4.4.5 Results from I-295 APA Strip Rut Testing

The modified APA rut test results are presented in Table 4-2 and Figure 4-6 for the PFC and FC-5 mixtures. The results showed high APA rut depths for both mixtures. These large

Table 4-2. I-295 Rut Results.

Rut Depth (Strip)	FC-5		PFC		FC-5	PFC
	North	South	North	South	Average	Average
DRD (mm)	15	19	17	18	17	18
ARD (mm)	7	8	7	9	7	8
% Area Change	5	6	9	11	6	10

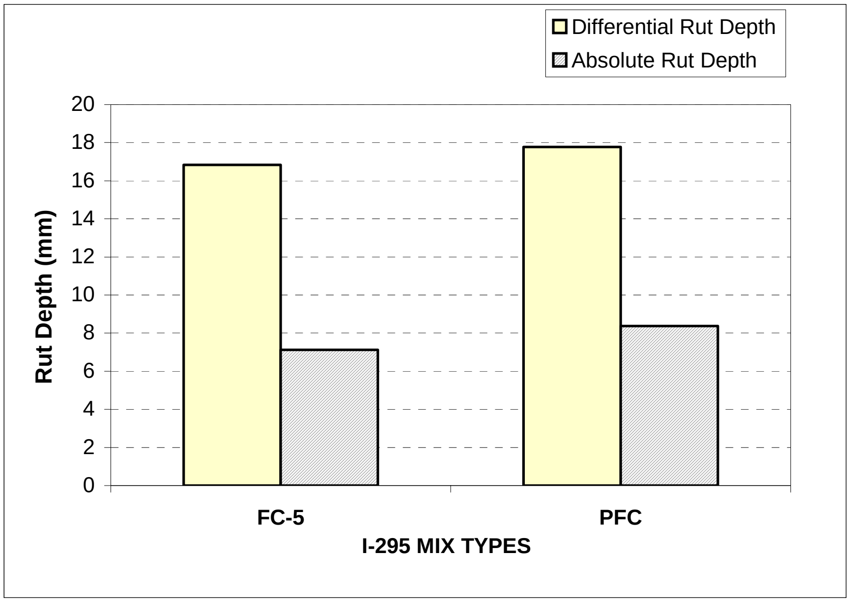

Figure 4-6. I-295 APA Rut Test Results on Field Cores.

numbers are likely due to the mixture aggregates not having had time to "key-in" to form a stable structure, as well as the APA rut test not being fully representative of field loading conditions, since these mixtures did not shown significant rutting in the field after construction. Therefore, the APA test should be at best used as a relative measure of the rutting potential between friction course mixtures, rather than absolute measures of the rutting potential of any given friction course mixture.

4.5 Fracture Evaluation

The fracture evaluation of the PFC and FC-5 mixtures was evaluated using HMA fracture mechanics (Zhang et al. 2001). The basic concept in HMA fracture mechanics is that there exists an energy-based threshold to cracking. There are two energy limits, which define failure, viz., Dissipated Creep Strain Energy (DSCE) and Fracture Energy (FE). DSCE is chosen as criterion under repeated loading condition while FE is selected under critical loading condition as shown in Figure 4-7.

The key points of the HMA Fracture Mechanics Model can be summarized as follows:

- If the threshold is not exceeded, all damage is fully healable micro damage. Once the threshold is exceeded, non-healable macro cracking is initiated.
- Under repeated loading conditions, the DSCE limit at impending fracture can be used as a threshold and it can be easily obtained from strength tests using the Superpave Indirect Tensile test (IDT).
- Asphalt being a viscoelastic material, crack initiation and propagation are inter-related, since cracks grow discontinuously (i.e., crack grows in step wise manner).
- Under critical loading conditions, the FE obtained from the point of impending fracture in a strength test can be used as a threshold.
- The HMA fracture mechanics framework is capable of capturing realistic loading condition and healing effects on asphalt pavements using the DSCE and FE as criterion.
- All parameters needed to describe crack initiation and growth, are obtained from the Superpave IDT test (i.e., resilient modulus, creep response- m-value, fracture energy to failure and tensile strength).

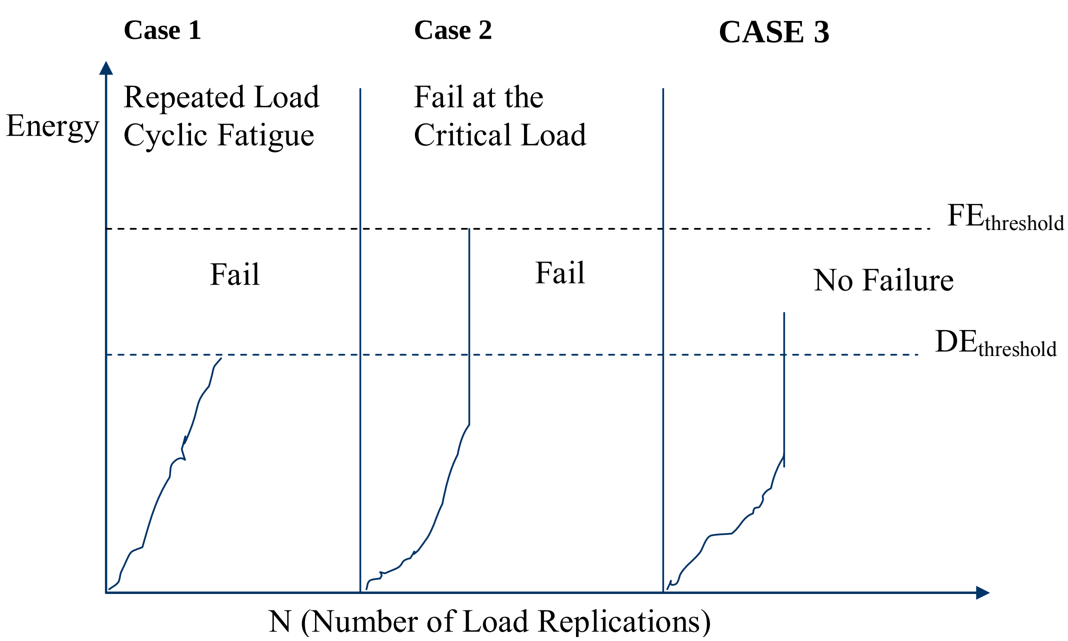

Figure 4-7. HMA Fracture Mechanics Threshold to Cracking and Resulting Effects of Loading Condition.

4.6 Superpave IDT Specimen Preparation

Since open graded mixtures are very porous compared to dense graded mixtures, a specimen thickness of 1.5 to 2.0 inches was used in order to avoid end effects. A specimen cutting device, which has a cutting saw and a special attachment to hold the pills (Figure 4-8), was used to slice the gyratory pill into specimens of desired thickness. Two two-inch thick specimens were obtained from each gyratory compacted specimen. Because the saw used water

Figure 4-8. Cutting Device.

to keep the blade wet, the specimens were dried for one day at room temperature. Before testing, the specimens were placed in a dehumidifying chamber for at least two days to minimize moisture effects during testing.

Gage points were attached to the specimens using a steel template and vacuum pump setup and a strong adhesive (Figure 4-9). Four gage points were placed on each side of the specimens at distance of 19 mm (0.75 in.) from the center, along the vertical and horizontal axes. A steel plate that fits over the attached gage points was used to mark the loading axis with a

Figure 4-9. Gauge Point Attachment.

marker. This helped placing the specimen in the testing chamber assuring proper loading of the specimen (Figure 4-10).

Figure 4-10. Marking Loading Axes.

4.7 Test Procedures

Standard Superpave IDT tests were performed on all mixtures to determine resilient modulus, creep compliance, m-value, D1, tensile strength, failure strain, fracture energy, and dissipated creep strain energy to failure. The tests were performed at 10° C.

4.7.1 Superpave IDT Resilient Modulus Test

The resilient modulus is defined as the ratio of the applied stress to the recoverable strain when repeated loads are applied. The test was conducted according to the system developed by Roque et al. (1992; 1994; 1997) to determine the resilient modulus and the Poisson's ratio. The resilient modulus test was performed in load control mode by applying a repeated haversine waveform load to the specimen for a 0.1 second followed by a rest period of 0.9 seconds. The load was selected to keep the specimen in the linear viscoelastic range, for which the horizontal strain is typically 150 to 350 micro-strains. The procedure for resilient modulus testing is as follows:

- The gyratory compacted pills are cut parallel to the top and bottom faces using a water-cooled masonry saw to produce 2 inch thick specimens having smooth and parallel faces.
- Four aluminum gage points are affixed with epoxy to each trimmed smooth face of the specimen.
- Test specimens are stored in a dehumidifying chamber at a constant relative humidity of 60 percent for at least 2 days. In addition, specimens are cooled at the desired test temperature for at least 3 hours before testing.
- Strain gauge extensometers are mounted and centered on the specimen for the measurement of horizontal and vertical deformation.
- A constant pre-loading of approximately 10 pounds is applied to the test specimens to ensure proper contact with the loading heads before further test loads are applied. The specimen is then tested by applying a repeated haversine waveform load for five seconds to obtain horizontal strain between 150 to 300 micro-strains. If the horizontal strains are higher than 350 micro-strains, the load is immediately removed from the specimen, and the specimen is allowed to recover for a minimum of 3 minutes before reloading at a lower loading level.
- Once the appropriate applied load is determined, the data acquisition program begins recording actual test data. Data are acquired at a rate of 150 points per second.
- The resilient modulus and Poisson's ratio are calculated using the following equations, which were developed based on three dimensional finite element analysis by Roque and Buttlar (1992; 1994):

$$M_R = \frac{P \times GL}{\Delta H \times t \times D \times C_{comp}} \tag{4-2}$$

where:

M_R	=	Resilient modulus
P	=	Maximum load
GL	=	Gauge Length
ΔH	=	Horizontal Deformation
t, D	=	Thickness, Diameter
C_{comp}	=	$0.6354\times (X/Y)-1-0.332$

4.7.2 Superpave IDT Static Creep Test

Creep compliance curve for mixtures is a function of time-dependent strain over stress. The methodology for obtaining creep compliance curves with the Superpave IDT was originally developed to predict thermally-induced stresses in asphalt pavements. However, because it represents the time-dependent behavior of asphalt mixtures, it can be also be used to evaluate the rate of damage accumulation of asphalt mixtures. As shown in Figure 4-11, it is common to assume that the creep compliance curve can be fit with a power law. The power law parameters D_0, D_1, and m-value are mixture parameters obtained from creep compliance tests. Although D_1 and m-value are related to each other, D_1 is more related to the initial portion of the creep compliance curve, while m-value is more related to the longer-term portion of the creep compliance curve. According to HMA fracture mechanics, the rate of tensile creep relates directly to the rate of damage accumulation in asphalt mixtures (e.g., Zhang et al. 2001). This means that the lower the m-value, the lower the rate of damage accumulation. However, mixtures with high m-values typically also have higher DCSE limits.

The Superpave Indirect Tensile Test at Low Temperatures (ITLT) computer program was used to determine the creep properties of the mixtures. The test was conducted in a load control mode by applying a static load. The load was selected to keep the horizontal strain in the linear viscoelastic range throughout testing. The detailed test and interpretation procedures were

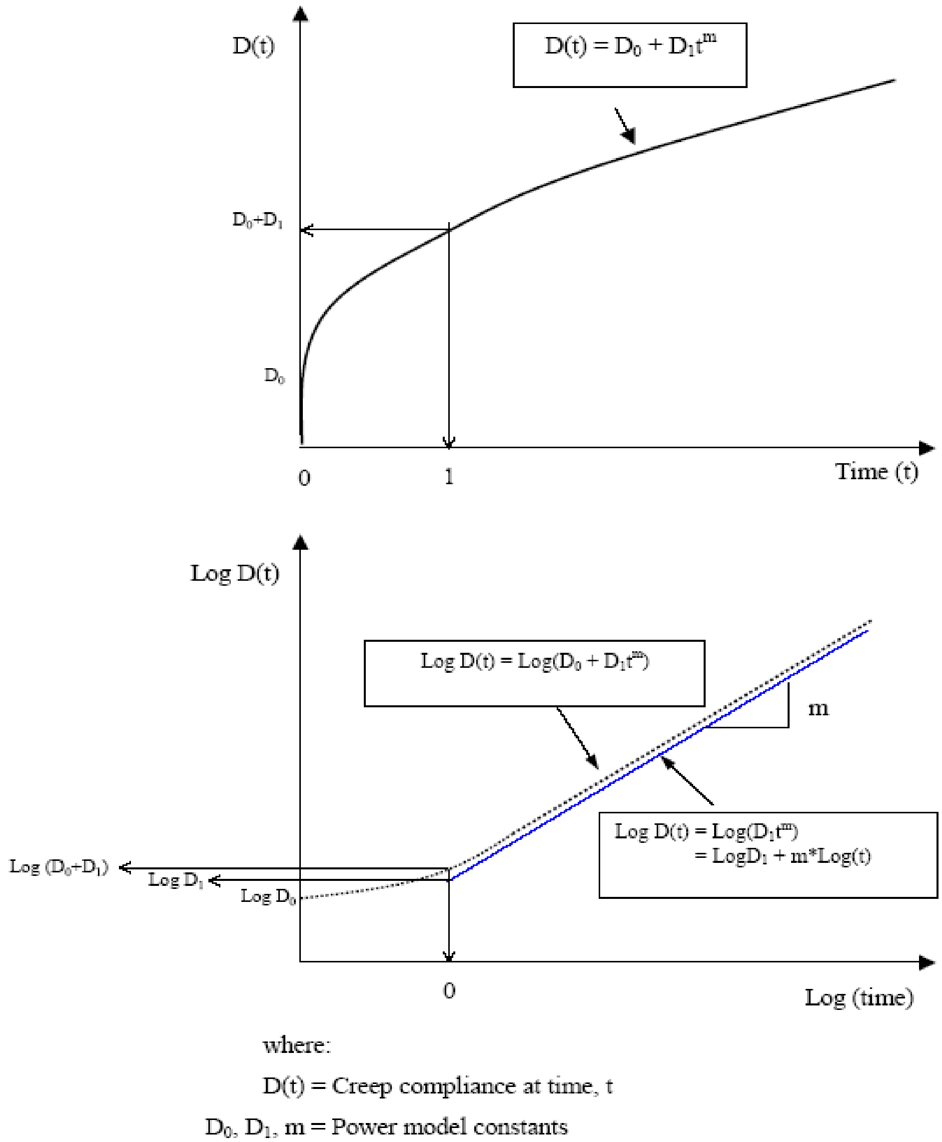

Figure 4-11. Power Model for Mixture Creep Compliance.

presented by Roque et al. (1994; 1997). The procedures for indirect tensile creep test consist of the following steps:

- The preparation of test specimens and the pre-loading are the same as those for resilient modulus test.
- Apply a static load for 1000 seconds. If the horizontal deformation is greater than 180 micro-inches at 100 seconds, the load is immediately removed from the specimen, and specimen is allowed to recover for a minimum 3 minutes, before reloading at a lower load level.

- When the applied load is determined, the data acquisition program records the loads and deflections at a rate of 10 Hz for the first 10 seconds, 1 Hz for the next 290 seconds, and 0.2 Hz for the remaining 700 seconds of the creep test.
- The computer program ITLT was used to analyze the load and deflection data to calculate the creep compliance properties. Creep compliance is computed by the following equation:

$$D(t) = \frac{\Delta H \times t \times D \times C_{comp}}{P \times GL} \tag{4-3}$$

where, D (t) = Creep Compliance.

4.7.3 Superpave IDT Indirect Tensile Strength Test

Failure limits such as tensile strength, failure strain, and fracture energy were determined from strength tests using the Superpave IDT. These properties are used for estimating the cracking resistance of the asphalt mixtures. The strength test was conducted in a displacement control mode by applying a constant rate of displacement of 50 mm/min for field mix and 100 mm/min for saturated mix until the specimens failed. The horizontal and vertical deformation and the applied load are recorded at the rate of 20 Hz during the test. The procedures developed by Roque and Buttlar (1994; 1997) were used to determine the tensile strength (S_t) of the mixtures.

4.8 Energy Ratio Calculation

The Energy Ratio is a dimensionless parameter introduced by Roque et al. (2004) that serves as single criteria for cracking performance of mixtures in pavements. The results obtained from the IDT test were used to determine the Energy Ratio (ER) for the mixtures as:

$$ER = DCSE_f / DCSE_{min} \tag{4-4}$$

where $DCSE_f$ is the dissipated creep strain energy threshold, and
$DCSE_{min}$ is the minimum dissipated creep strain energy

The minimum dissipated creep strain energy is calculated by using Equation 4-5, below:

$$DCSE_{min} = m^{2.98} * D_1 / A \qquad (4\text{-}5)$$

in which:

$$A = 0.0299 * \sigma^{-3.10} * (6.36\text{-}S_t) + 2.46 * 10^{-8}$$

and

m and D_1: power law creep compliance parameters,
σ: applied tensile stress,
S_t: tensile strength.

The creep strain rate from a 1000 sec creep test is calculated as follows:

$$dD(t)/dt = D1 * m * (1000)^{m-1} \qquad (4\text{-}6)$$

Besides the Energy Ratio, the other parameters used for the evaluation of mixture fracture resistance are the Fracture Energy (FE), the Dissipated Creep Strain Energy (DCSE), tensile strength, failure strain, and creep strain rate.

In the Energy Ratio calculation (Eqn. 4-5), the applied tensile stress is taken at the bottom of the AC layer and is very much dependent on the stiffness of the HMA structural layer. For friction courses, the whole pavement structure can be regarded as a composite consisting of the friction course and HMA structural layer beneath it. Thus, the stress at the bottom of the HMA structural layer should be used for friction course ER calculations. Thus, a typical pavement structure, shown in Figure 4-12, with the given loading condition was considered for determining ER for the FC-5 and PFC mixtures.

The thickness for FC-5 layer was taken as 0.75 in. and the thickness of the PFC layer was taken as 1.25 in. The resulting tensile stress at the bottom of the pavement was determined through layered elastic analysis.

4.8.1 Fracture Test Results

The Superpave IDT fracture test results of the laboratory-prepared PFC mixture are summarized in Table 4-3 for Long Term Oven Aging (LTOA) and STOA aging conditions. The

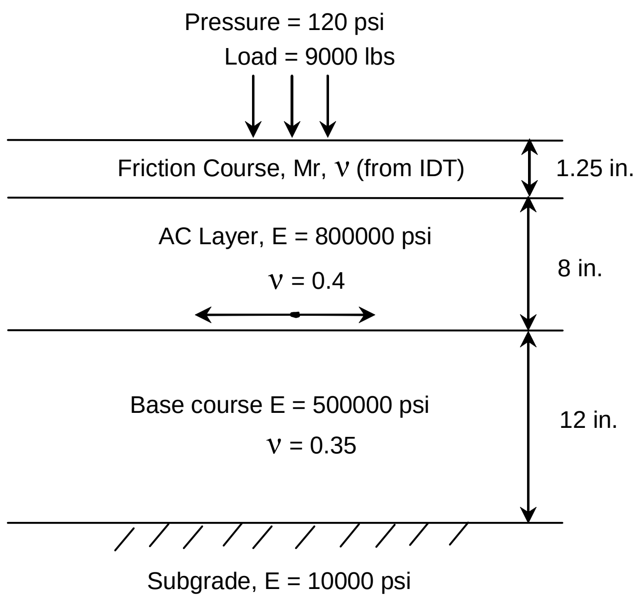

Figure 4-12. Typical Pavement Structure for Stress Calculation.

Table 4-3. Fracture Test Results for Laboratory-Prepared PFC Mixture.

Aging Cond-ition	**m**	**D1 (psi)**	**D′(t) (psi)**	**St (MPa)**	**M_R (GPa)**	**FE (kJ/m^3)**	**Failure Strain (μstrain)**	**DCSE (kJ/m^3)**	**ER@ 88.23 psi**
STOA	0.660	1.20E-6	7.56E-8	1.15	4.41	3.6	3940.1	3.45	1.67
LTOA	0.692	1.56E-06	1.40E-7	0.94	3.28	1.6	2349.0	1.47	0.89

fracture test results for the actual field mixtures for the new FC-5 and PFC test sections are summarized in Table 4-4. The field mixtures were also subjected to LTOA and STOA to study the effects of aging on their fracture resistance. The detail graphical results for each mixture (new FC-5, PFC) by location, i.e., Northbound lane (NB), Southbound lane (SB), are shown in Figures 4-13 through Figure 4-20.

The results in Tables 4-3 and 4-4 show that both the STOA laboratory-prepared PFC mixture and the field FC-5 mixture have ER values in excess of 1.5, which means they have relatively good fracture resistance. For the field FC-5 mixture, the ER values for both STOA and LTOA

Table 4-4. Fracture Test Results for the FC-5 and PFC Field Mixtures.

Mixture Locations		m	D1 (psi)	D'(t) (psi)	St MPa	M_R GPa	FE kJ/m^3	Failure Strain μstrain	DCSE kJ/m^3	ER @ 88.23 psi
FC-5 Mixture										
NB	LTOA	0.537	1.01E-06	2.21E-08	1.24	6.30	2.3	2346.37	2.2	2.32
	STOA	0.507	1.01E-06	1.70E-08	1.4	6.33	3.0	2798.95	2.8	3.41
SB	LTOA	0.570	1.39E-06	4.05E-08	1.14	6.02	2.9	3156.47	2.8	1.83
	STOA	0.570	9.28E-07	2.71E-08	1.04	6.00	2.9	2881.06	2.8	2.78
PFC Mixture										
NB	LTOA	0.594	2.05E-06	7.37E-08	0.73	4.15	1.1	2053.64	1.0	0.42
	STOA	0.538	1.18E-06	2.61E-08	0.98	5.24	1.4	1915.6	1.3	1.22
SB	LTOA	0.648	1.57E-06	8.94E-08	1.17	5.39	4.4	4819.12	4.3	1.69
	STOA	0.691	6.12E-07	5.00E-08	1.26	6.09	2.2	2308.67	2.1	1.72

specimens are high (greater than 1.8), whereas the ER for the laboratory-prepared PFC mixture under LTOA conditions decreased to 0.89, which is slightly below the threshold value of one.

The field PFC mixture from the southbound lane exhibits ER values for both STOA and LTOA conditions that are comparable to the laboratory-prepared mixture (i.e., around 1.7). However, the PFC field mixture from the northbound, outside lane, showed unusually low ER and high creep rate values (1.22 for STOA and 0.42 for LTOA conditions), so they were sent to the FDOT State Materials Office for ignition oven testing and binder analysis.

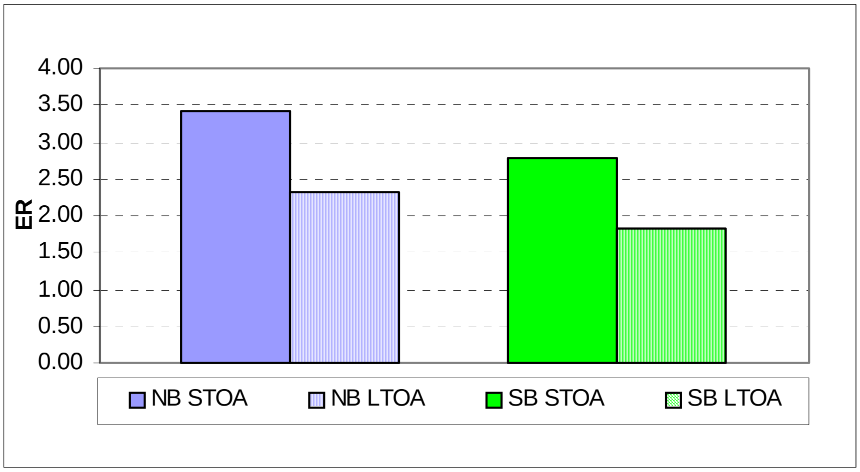

Figure 4-13. Field FC-5 Energy Ratio.

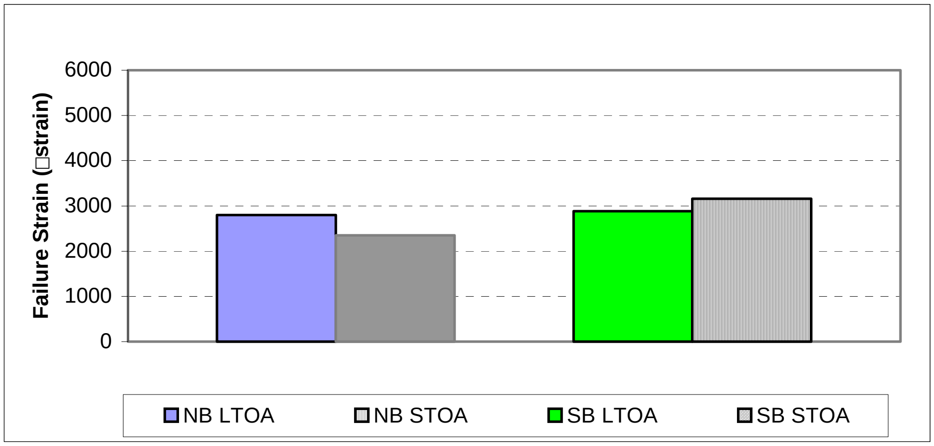

Figure 4-14. Field FC-5 Failure Strain.

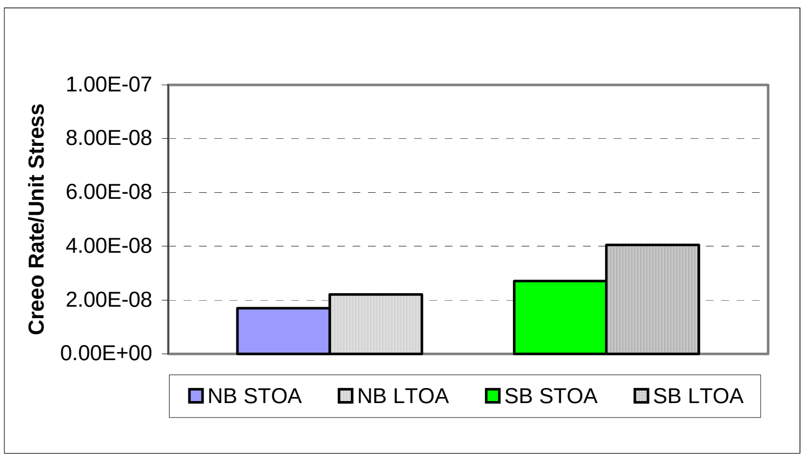

Figure 4-15. Field FC-5 Creep Rate.

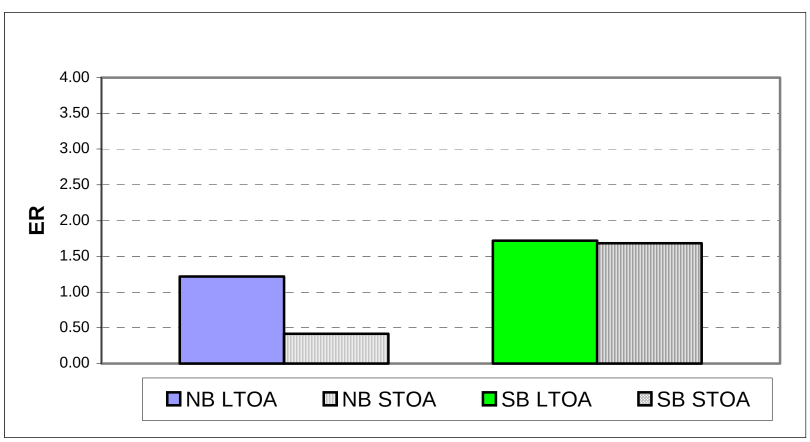

Figure 4-16. Field PFC Energy Ratio.

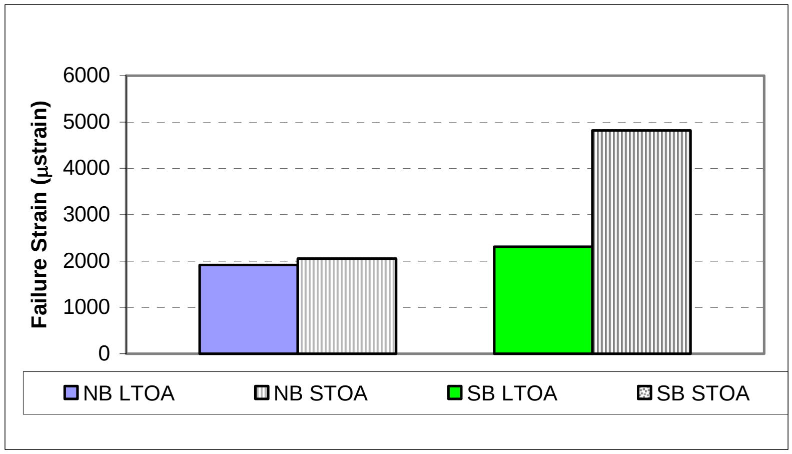

Figure 4-17. Field PFC Failure Strain.

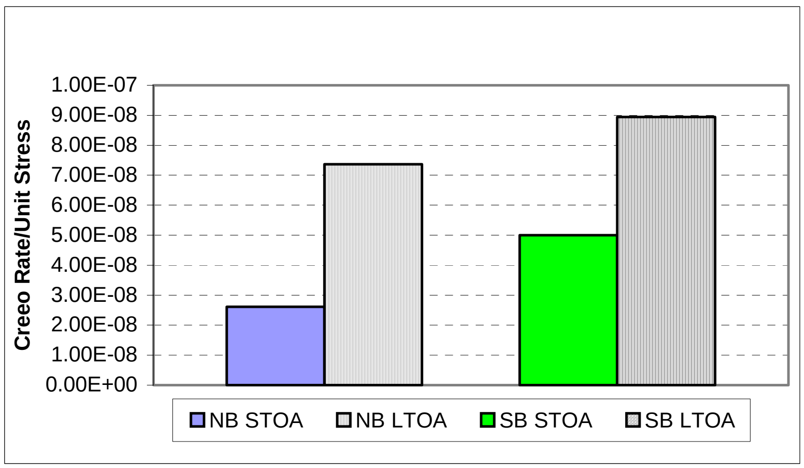

Figure 4-18. Field PFC Creep Rate.

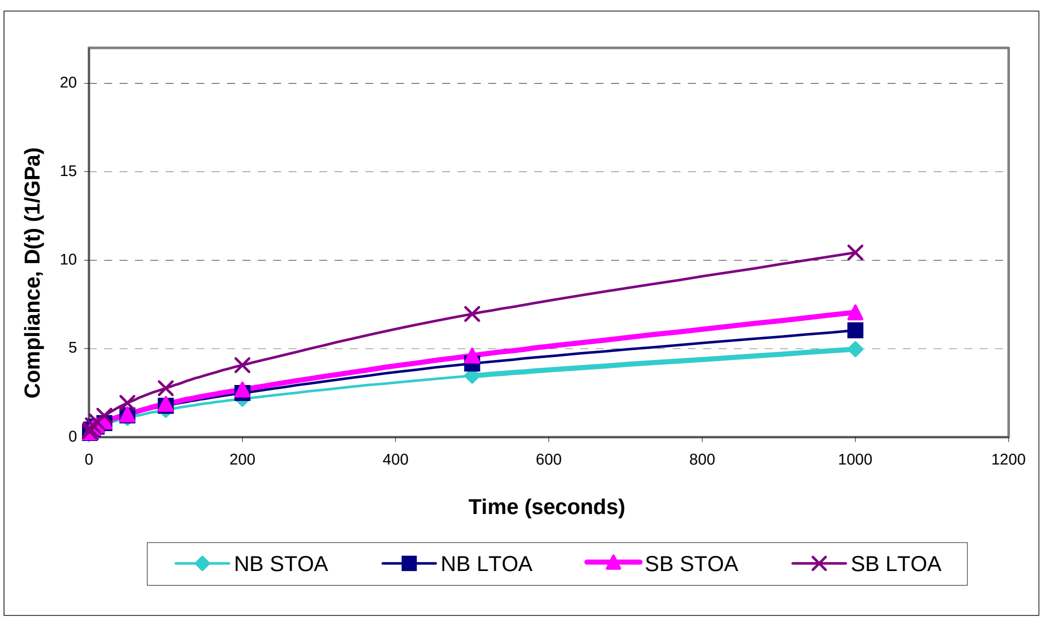

Figure 4-19. Field FC-5 Creep Compliance.

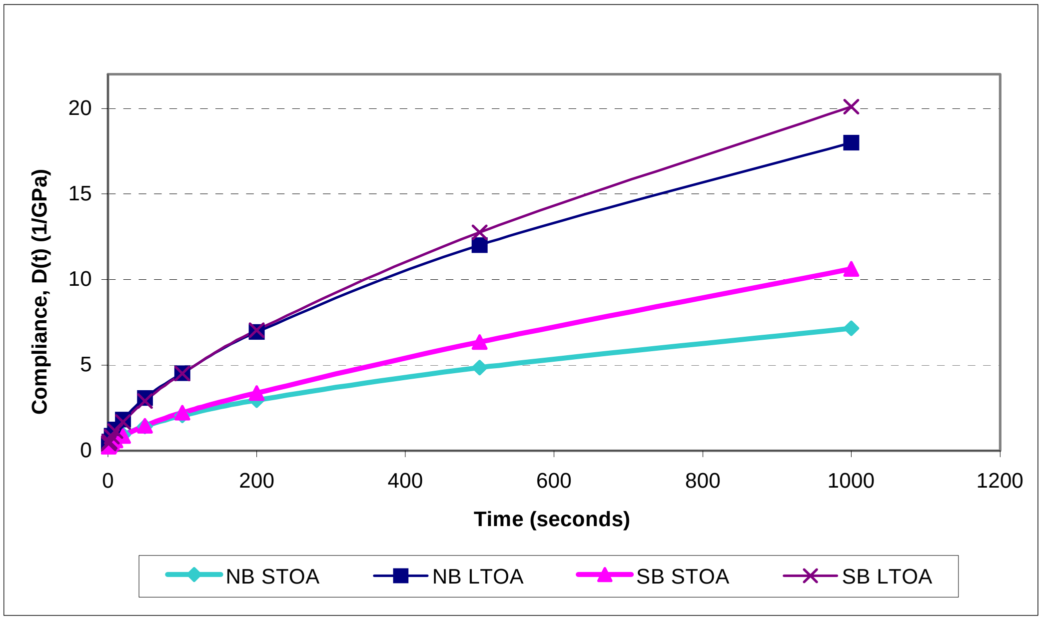

Figure 4-20. Field PFC Creep Compliance.

The results of the analysis performed by FDOT showed that the percent asphalt content was significantly low: 4.58%, 4.73%, 4.69%, 4.94% for the four specimens tested, whereas the design AC content was 6 percent. This could have caused the high creep values observed in the fracture tests. It should be noted that the ignition oven correction factor has been applied to the reported values. It should also be noted that the laboratory samples tested had been trimmed on both sides, thereby possibly causing a slight decrease in asphalt content. Therefore, the percent binder contents from the ignition oven were probably slightly lower than those obtained without cut faces, but that would not explain the larger differences observed.

Another possible reason for the low asphalt binder content is that drain down was occurring in trucks during the construction of the Northbound PFC project. This resulted in a saturated layer of PFC mix at the bottom of the truck beds. When these saturated layers broke off and entered the paver, the result was the formation of high asphalt content spots (bleeding) on the pavement surface, with other parts of the mixture therefore being low on asphalt.

The FDOT visually observed the presence of fiber within the PFC mixtures, but both the quantity of fiber and the dispersion of fiber within the mixture were unknown. The asphalt-binder grading analysis confirmed that the asphalt binder met all requirements for a PG 76-22 binder grading, thus supporting the presence of a polymer within the binder. In addition to extraction of the binder, the FDOT also performed gradation analysis on the field PFC mixture from the northbound lane. Table 4-5 shows a comparison between the gradations for the as-produced

Table 4-5. PFC Oven Ignition Gradation.

Sieve Size		**Ignition Gradation**	**Target JMF Gradation**	**Difference**
3/4"	19.0mm	100.00	100.00	0.00
1/2"	12.5mm	88.74	89.00	0.26
3/8"	9.5mm	63.69	62.00	-1.69
No. 4	4.75mm	16.70	15.00	-1.70
No. 8	2.36mm	8.41	10.00	1.59
No. 16	1.18mm	5.52	7.00	1.48
No. 30	600μ	4.33	5.00	0.67
No. 50	300μ	3.79	5.00	1.21
No. 100	150μ	3.27	3.00	-0.27
No. 200	75μ	3.03	2.20	-0.83

field PFC mixture and the laboratory-based design mixture. The results show that both gradations are close to each other, thus not explaining the observed differences in fracture resistance.

In summary, the most likely remaining explanation for the low fracture resistance is drain down in the field mixture due to either inadequate fiber content or lack of adequate fiber dispersion throughout the mixture. In addition, the QC binder contents during production were all around 6.0%. Therefore, it is possible that enough fiber got weighed into the mix during plant mixing, but the fiber simply did not get dispersed evenly throughout the mixture, resulting in portions of the production mixture with low fiber content. Therefore, the production and construction procedures of PFC mixtures should be approached with special care in order to avoid draindown of asphalt which jeopardize the performance of the mixtures.

4.9 Field Permeability of I-295 Friction Course Mixtures

The permeability of the FC-5 and PFC field test sections is listed in Table 4-6. Interestingly, the FC-5 appears to have closed up quickly (i.e., after about one month after construction). The PFC mixture has overall higher permeability, due to its coarse aggregate gradation, and will have more water storage capacity due to the increased thickness.

Table 4-6. Field Permeability Test Results from I-295 Test Sections (Early September, 2005)

Pavement Test Section	**Permeability (10^{-5} cm/s)**
PFC in Wheelpath	81,000
PFC between Wheelpath	74,000
FC-5 in Wheelpath	22,000
FC-5 between Wheelpath	51,000

Due to the construction problems with the Northbound PFC lane, it was milled and repaved on September 17, 2005. On November 7, the FDOT tested the Northbound lane for field permeability and cut cores for thickness determination. Permeability measurements were obtained at three locations equally spaced within the test section. Measurements were obtained between the wheelpath and in the outside wheelpath. The results are listed in Table 4.7. The thicknesses for each of the locations where the permeability measurements were obtained were fairly consistent and averaged 1.4 inches. It should be noted that the design asphalt content for the new Northbound PFC construction was lowered from 6.0 percent to 5.6 percent. This might explain why the permeability of the new section is higher than the original section that got milled and replaced. It is also possible that the pavement was not exposed to high air temps prior to testing. High pavement temperature leads to binder softening which again may lead to densification.

Table 4-7. Field Permeability Test Results from I-295 PFC Northbound Test Sections (November 7, 2005)

Pavement Test Section	**Permeability (10^{-5} cm/s)**
PFC in Wheelpath	123,098
PFC between Wheelpath	127,821

4.10 Evaluation of Moisture Damage Potential of Laboratory-Prepared PFC Mixture

The AASHTO T283 (2003) moisture conditioning protocol was adopted and modified to evaluate the moisture sensitivity of the laboratory-prepared I-295 PFC mixture. The Superpave IDT test and associated fracture parameters were used to quantify the effects of moisture damage.

Six 6-inch diameter gyratory specimens were compacted to 50 gyratory revolutions in the Superpave gyratory compactor. Three of the specimens were then subjected to a hot water bath at 60° C (140° F) for 24 hours, followed by 2 hours in a bath at 25° C (77° F). Due to the open graded nature of the PFC mixture, it was determined that it was unnecessary to subject the specimens to vacuum saturation prior to the hot water bath. Since the PFC mixture had air voids around 21%, it was possible that the specimens would experience creep or failure during moisture conditioning. In order to eliminate the likelihood of premature specimen failure or damage during conditioning, the specimens were wrapped in 1/8-in wire mesh, with two clamps holding the mesh in place, without exerting pressure onto the specimens. After the conditioning phase, the conditioned specimens were allowed to drain for 36 hours at room temperature before removing the wire mesh.

Once the specimens had drained for 36 hours, both the conditioned and unconditioned specimens were cut by a wet saw into 2-inch thick specimens. The specimens were placed in a dehumidifier chamber for 48 hours, to minimize any effects of moisture during the Superpave

IDT testing. The Superpave IDT test was used to perform Resilient Modulus (M_R), Creep Compliance, and Strength tests from which the following properties were determined: tensile strength, resilient modulus, fracture energy limit (FE), dissipated creep strain energy limit (DCSE), creep compliance, and m-value. Using these mixture properties and the HMA fracture mechanics framework, the Energy Ratio was calculated.

Table 4-8 shows a summary of the fracture testing results of the conditioned and unconditioned specimens. The creep compliance of conditioned mix is 17.66 1/GPa, which is not a significant change as compared to unconditioned sample i.e., 17.53 1/GPa. The strain rate per unit stress of unconditioned mixture, i.e., 7.9 x 10^{-8} 1/psi-sec, remains same after conditioning. However, the DCSE limit was reduced from 3.45 KJ/m^3 to 1.03 KJ/m^3 due to moisture conditioning. Fracture energy also decreased from 3.6 KJ/m^3 to 1.1 KJ/m^3 as result of conditioning. The resilient modulus is increased from 4.41 GPa to 5.25 GPa due to conditioning, but this change is not significant. Finally, the energy ratio of conditioned mixture was 0.6, as compared to 1.67 for the unconditioned mixture. This decrease in energy ratio is significant. However, it is likely that any comparisons to trends observed for dense graded mixtures is not reasonable due to the very easy access of moisture into PFC mixtures. This means that more testing should be performed on PFC mixtures to establish acceptable ranges in ER for conditioned mixtures.

Table 4-8. Results from Evaluation of Moisture Damage Potential for I-295 PFC Laboratory-Prepared Mixture

Sample	Property										
	Resilient Modulus (GPa)	Creep compliance at 1000 seconds (1/GPa)	Tensile Strength (Mpa)	Fracture Energy (kJ/m^3)	Failure Strain (10-6)	m-value	D_1	DCSE (kJ/m^3)	Elastic E. (kJ/m^3)	Energy Ratio	Creep Rate (1/psi-sec)
Unconditioned	4.41	17.53	1.15	3.6	3940	0.66	1.16E-06	3.45	0.150	1.67	8.86E-08
Conditioned	5.25	17.67	0.84	1.1	1827	0.73	7.9E-07	1.03	0.067	0.60	8.91E-08

Another issue to be considered is that the SBS polymer modified PG 76-22 provided by the contractor for the laboratory-based study contained 0.5 percent by weight antistripping agent. The combination of SBS polymer and liquid antistripping agent may have complicated and possibly adversely affected the response of the mixture to moisture conditioning.

4.11 Summary and Conclusions

This chapter documented the PFC mixture design, and performance testing of the test section on I-295 Duval County, Florida. The fracture testing of mixtures obtained from the paver from the Northbound lane showed a low fracture resistance, whereas the other test sections resulted in adequate fracture resistance. Forensic evaluation confirmed field observations of drain down in the mixture, resulting in lowered asphalt content. This was possibly due to the lack of dispersion of the fiber in the mixture. The part of the test section with draindown problems was repaved to a successful completion of the field testing project. The APA rutting performance of field cores was evaluated and found to be high. This is likely due to the mixtures not having had adequate trafficking to allow the aggregate structure to "key in" fully. The field permeability test results showed that both the PFC and FC-5 mixtures had high permeability, with the PFC mixture showing somewhat higher permeabilities as expected due to its coarser aggregate structure. The added benefit of greater PFC mat thickness should also help the water storage capacity of the PFC section with everything else being equal.

4.12 References

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CHAPTER 5
COMPARISON OF FRACTURE RESISTANCE OF PFC MIXTURES

As a final part of the evaluation of PFC mixtures, the fracture resistance of the three PFC mixtures designed as a part of this project are compared in this chapter. These mixtures include the PFC granite and limestone mixtures, discussed previously in Chapter 3, termed GPEM-L and GPEM-G, respectively, since they were based on Georgia GPEM mixture design. The I-295 PFC laboratory-prepared mixture, discussed in Chapter 4 was also included. The objective with this comparison is to establish a baseline for comparison purposes for future PFC mix designs.

5.1 Materials

The PFC mixtures from Chapter 3, included a Nova Scotia granite and south Florida limestone mixture, respectively. The I-295 PFC mixture was composed of Nova Scotia granite, along with granite from Georgia, as discussed in Chapter 4. In the following, the original GPEM limestone and granite mixture designs in Chapter 3 used NS315 aggregate from Nova Scotia and oolitic limestone from South Florida, with an existing gradation obtained from the GDOT. Hence, these two mixtures were entitled “GPEM (Granite) and GPEM (Limestone),” respectively. The mixture entitled PFC-G (granite) is the mixture that was designed for the test section on I-295, Jacksonville.

The binder for all three mixtures was SBS modified PG 76-22 asphalt. All three mixtures used mineral fiber, 0.4% by weight of total mix, to mix in order to avoid binder drain drown. Chemical composition of mineral fiber is Vitreous Calcium Magnesium Aluminum Silicates. Mineral fibers were pulled apart into fine fragments before adding to the aggregates. The granite mixtures contained 1% hydrated lime as an anti-stripping agent.

The design asphalt content for all three mixtures was established using the draft Florida method for the design for PFC mixtures (Appendix A). The optimum asphalt content and the effective film thickness (Nukunya et al. 2001) were:

- GPEM-L mixture: 6.5 percent (Effective film thickness: 50.1 μ).
- GPEM-G mixture: 6.0 percent (Effective film thickness: 54.6 μ).
- PFC-G (I-295) mixture: 6.1 percent (Effective film thickness: 35.4 micron).

The GPEM-L mixture resulted in a higher asphalt content due to absorption of the asphalt into the porous oolitic limestone aggregates. The difference in effective film thickness between the (I-295) PFC mixture and the GPEM-G and GPEM-L mixtures is due to the higher proportion of minus #4 material in the I-295 PFC mixture.

5.2 Superpave IDT Test

The Superpave IDT was used to perform the following tests: Resilient Modulus Test (M_R), Creep test, and indirect tensile strength tests from which the following properties were determined: tensile strength, resilient modulus, fracture energy limit (FE), dissipated creep strain energy limit (DCSE), creep compliance, and m-value. The FE and DCSE values and the modulus can be accurately determined using the Superpave Indirect Tensile Test following the procedures developed by Roque and Buttlar (1992; 1994; 1997). Using these mixture properties and the HMA fracture mechanics framework developed at the University of Florida (Zhang et al. 2001), the Energy Ratio was calculated (Roque et al. 2004), following the same procedure as described in Chapter 4.

5.2.1 Results of Fracture Testing on PFC Mixtures

The short-term oven aged and long-term oven aged test results for the PFC friction course mixtures are presented in Table 5-1. The Energy ratio for short-term oven aged Georgia PEM-G

Table 5-1. Summary of Fracture Test Results on Short-Term and Long-Term Oven Aged Mixtures of Georgia PEM, PFC Project and OGFC Mixture.

Sample	Property										
	Resilient Modulus (GPa)	**Creep Com-pliance at 1000 seconds (1/GPa)**	**Tensile Strength (Mpa)**	**Fracture Energy (kJ/m^3)**	**Failure Strain (10-6)**	**m-value**	**D_1**	**DCSE (kJ/m^3)**	**$e_0(10^{-6})$**	**Energy Ratio**	**Strain Rate per Unit stress (1/psi-sec)**
Short-Term Oven Aged Mixtures											
GPEM-G	4.97	19.93	1.24	4.2	4383	0.74	8.35E-07	4.05	4133.73	1.95	1.061E-07
GPEM-L	5.81	3.54	1.59	3.5	2735	0.51	6.75E-07	3.28	2461.61	3.32	1.161E-08
I-295 PFC	4.41	17.53	1.15	3.6	3940	0.66	1.16E-06	3.45	3679.319	1.67	7.916E-08
Long-Term Oven Aged Mixtures											
GPEM-G	4.9	10.93	0.97	1.1	1552	0.70	5.86E-07	1.00	1354.31	0.86	5.127E-08
GPEM-L	6.27	2.47	1.57	2.3	2026	0.34	1.59E-06	2.10	1775.57	3.09	5.536E-09
I-295 PFC	3.28	27.91	0.94	1.6	2349	0.692	1.56E-06	1.47	2062.21	0.89	1.283E-07

(Granite) and the PFC-G (Granite) ranges from between 1.5 and 2, which indicates that these mixtures should have good fracture performance. In contrast, a mixture with an Energy Ratio well below 1.0 could be of concern. Similarly, for the GPEM (Limestone) mixture, the short-term oven aged energy ratio is around to 3.5, which indicates excellent fracture performance.

The PFC mixtures tested all had high failure strain limits in the tensile strength test. A mixture with a high failure strain can generally deform more in tension before initiating a crack. On the other hand, brittle mixtures tend to show lower failure strains, which could be an indicator of not only low fracture resistance. In the case of GPEM-G mixture, the short-term oven aged failure strain is around 4000 micro strain and the DCSE limit is close to 4 KJ/m^3. In comparison, the short-term oven aged GPEM-L mixture had a DCSE limit of 3.28 KJ/m^3. This lower DCSE limit is primarily due to the low failure strain (2735 micro strain) as compared with granite mixture (4000 micro strain). For the short-term oven aged PFC-G the DCSE limit is 3.5 KJ/m^3 with a failure strain of 3940 micro strains.

In summary, the short-term oven aged PFC mixtures tested all had Energy Ratios greater than 1.0 and all had high failure strains (greater than 1500 micro strains), and thus would be expected to show a good fracture resistance.

Both granite mixtures (GPEM-G and the I-295 PFC mixture) showed a decrease in the energy ratio due to long-term aging. The failure-strain of the granite mixtures was reduced by half, as compared to the short-term oven aged mixtures. This decrease in the failure strain led to a decrease in the DCSE limit. The resilient modulus for the long-term oven aged mixtures is not affected significantly when compared with short-term oven aged mixtures.

Limestone mixtures in general have a rough texture, with a lot of crevices and pores on the aggregate surfaces. During the long-term oven aging, the temperature is around 85° C, and at such a high temperature, the asphalt will flow, further enhancing the absorption of the asphalt into the crevices and the pores in the aggregate. This absorption mechanism may result in a mixture with improved resistance to aging and thus result in less of a decrease in energy ratios after aging than compared to a granite mixture with an identical gradation. This might explain why the GPEM-L, which has a high-energy ratio for short-term oven aged conditions of about 3.3, only showed a slightly reduced energy ratio of 3.09 for the long-term oven aged conditions.

Overall, the three PFC mixtures studied showed adequate fracture resistance as measured by the energy ratio for both STOA and LTOA aging conditions.

5.3 Summary of Findings

The findings from the Superpave IDT fracture test results on the three PFC mixtures tested are as follows:

- All of the mixtures showed energy ratios greater than 1.0 for the short term oven aging, which indicates an adequate fracture resistance.
- Both granite mixtures showed a significant decrease in energy ratio with long term oven aging, resulting in energy ratios slightly below 1.0, which is likely an indicator of the very open graded nature of these mixtures.
- The GPEM-L mixture showed only a slight reduction in energy ratio with long term oven aging, as compared to the granite mixtures which showed a more significant drop in energy ratio. This is possibly due to the effects of absorption of asphalt into the more porous limestone mixture, which may help protect the mixture against excessive aging.
- The limestone short-term oven aged mixture GPEM-L showed higher energy ratio as compared with the granite mixtures. This may indicate that somehow the high surface texture in the oolitic limestone may be helping to provide an added degree of stone-to-stone contact in these very open graded mixtures, which may help the overall fracture resistance of these mixtures.

5.4 References

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Roque, R. and Buttlar, W.G., The Development of a Measurement and Analysis System to Accurately Determine Asphalt Concrete Properties Using the Indirect Tensile Mode. In *Journal of the Association of Asphalt Paving Technologists*, Vol. 61, 1992.

Roque, R. and Buttlar, W.G., Development and Evaluation of the Strategic Highway Research Program Measurement and Analysis System for Indirect Tensile Testing at Low Temperatures. In *Transportation Research Record 1454*, TRB, National Research Council, Washington, D.C., 1994.

Roque, R., Buttlar, W.G., Ruth, B.E., Tia, M., Dickison, S.W., and Reid, B., Evaluation of SHRP Indirect Tension Tester to Mitigate Cracking in Asphalt Pavements and Overlays, Final Report of the Florida Department of Transportation, University of Florida, Gainesville, FL, August 1997.

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CHAPTER 6

EVALUATION OF THE RUTTING RESISTANCE OF BONDED FRICTION COURSES

6.1 Introduction

In the next three chapters, the performance of bonded friction courses will be evaluated. This includes evaluation of rutting and cracking potential, as well as overall field performance. Chapter 6 deals with the evaluation of rutting resistance of bonded friction courses, Chapter 7 deals with the evaluation of their fracture resistance, and Chapter 8 covers their overall field performance.

During the last few years, special pavers that lay a heavy polymer-modified tack coat just in front of the hot mix surface course mat have become more commonplace in the United States. This technology is available in Florida and has been demonstrated on several small projects. Other states, including Texas, Pennsylvania, and Alabama have several years experience with this process and indications are that it has great potential. An example of a mixture that is placed by these pavers is Novachip, which was discussed in Chapter 2. The Longitudinal Wheel Path Cracking study conducted by UF has confirmed that a significant amount of the distress on high-volume Florida pavements is caused by surface initiated wheel path cracking from lateral stresses generated by radial truck tires (e.g., Roque et al. 2004). A recent evaluation of the ten year performance of ground tire rubber in an open graded friction course on SR-16 in Bradford County indicates that increased asphalt content from ground tire rubber modification of the friction course can reduce longitudinal wheel path cracking. Based on these studies, there is a high likelihood that the bonded friction course process could significantly improve the crack resistance of open graded mixes in Florida by providing more polymer modified asphalt to the friction course. It is also highly likely that raveling resistance of the friction course will be improved as well.

Based on recent work by UF on the mechanisms of instability rutting, it appears that instability rutting tends to occur in the top 100 mm (4 inches) of the pavement (Birgisson et al. 2002; Novak et al. 2003). Therefore, it is likely that a stabilizing thick polymer-modified tack coat placed 1-2 inches below the pavement surface will aid in resisting instability rutting, by strengthening the mixture within the critical depth zone at which shear instability is likely to occur (Birgisson et al. 2002; Novak et al. 2003; 2004).

Currently, the optimal ways for testing and evaluating rutting susceptibility of open graded friction courses is still a subject of research. In this chapter, the use of the modified Asphalt Pavement Analyzer for evaluating the rutting resistance of open graded friction courses will be evaluated. The traditional pressurized hose is replaced by a loading strip that better simulates measured contact stresses between radial truck tires and the pavement surface (Drakos et al. 2005). Similarly, the traditional method of measuring rut depth with the Asphalt Pavement Analyzer is to measure the single lowest point of the specimen. Drakos et al. (2005) illustrated that careful measurements of the rut profile result in a more consistent interpretation of APA rut test results. Rut profile measurements also possibly allow for an evaluation of the most likely rutting mechanism, i.e., consolidation rutting versus instability rutting.

In this chapter, the conditions will be examined that allow for the consistent evaluation of the APA rutting potential of 1) open graded friction courses that are bonded to the underlying structural layer with a thick polymer-modified tack coat, and 2) open graded friction courses that are bonded to the underlying structural layer with a normal tack coat. In addition, the relative benefit to rutting resistance of the modified tack coat will be evaluated. The details of the APA test procedures used in this chapter were discussed previously in Chapter 4.

The materials used in this study were obtained from a bonded friction course field project on US-27 in Highlands County. Five test sections were placed in early July 2003. These sections consisted of three types of mixtures: FC-5 with limestone, FC-5 with granite, and Novachip. Each of the FC-5 limestone and granite mixtures were placed with a thick polymer-modified tack coat, as well as with a normal tack coat. The combined length of the five pavement test sections is 6.528 miles, stretching from the Glades County Line and extending to 0.1 miles south of Horn Road in Highlands County.

6.2 Objectives

- To evaluate the rutting potential of the FC-5 field test projects on US-27, in Highlands County, Florida.
- To evaluate the effect of the thick bonding tack coats on rutting performance.
- To evaluate laboratory-based methods to reliably determine rutting susceptibility of OGFC mixtures.

6.3 Scope

This chapter presents the results of a rutting study on both laboratory prepared specimens and field cores. The study was conducted using two different APA loading mechanisms (pressurized hose loading versus strip loading) on the following materials:

- Field cored samples from US-27 (taken immediately after paving).
- Field cored samples from US-27 (five months after construction).
- Laboratory prepared specimens (US-27 materials sampled from the paver and compacted in the gyratory compactor to 50 gyratory revolutions).

6.4 Materials and Methods

6.4.1 Aggregates

The FC-5 limestone mixture consisted of oolitic limestone (White rock S1A, S1B and White rock screenings) from South Florida. The FC-5 granite mixture and the Novachip mixture consisted of Nova Scotia granite (#7 granite, #789 granite, and granite screenings). The gradations for these mixtures are shown in Figures 6-1 through 6-3. The granite FC-5 mixture was pre-

STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION
STATEMENT OF SOURCE OF MATERIALS AND JOB MIX FORMULA FOR BITUMINOUS CONCRETE

SUBMIT TO THE STATE MATERIALS ENGINEER, CENTRAL BITUMINOUS LABORATORY, 2006 NORTHEAST WALDO ROAD., GAINESVILLE, FLA. 32609

Contractor: APAC-Florida, Inc., Central Florida Division   Address: 6701 East Hanna Avenue, Tampa, FL 33610

Phone No.: (813) 623-3622   Fax No.: (813) 620-1918   E-mail: flader@ashland.com

Submitted By: APAC-Florida, Inc.   Type Mix: FC-5 Granite   Intended Use of Mix: Friction Course

TYPE MATERIAL	F.D.O.T. CODE	PRODUCER	PIT NO.	DATE SAMPLED
1. #7 Granite *	44	Martin Marietta Aggregates	TM-322 NS-315	01 / 31 / 2003
2. #789 Granite *	51	Martin Marietta Aggregates	TM-322 NS-315	01 / 31 / 2003
3. Granite Screenings *	22	Martin Marietta Aggregates	TM-322 NS-315	01 / 31 / 2003
~~4. Hydrated Lime (Aggregate Pre-Treat)~~		~~Martin Marietta Aggregates~~	~~TM-322~~	~~01 / 31 / 2003~~
5. ARB-12				
6.				

PERCENTAGE BY WEIGHT TOTAL AGGREGATE PASSING SIEVES

Blend	77%	12%	11%						
Number	1	2	3	4		6	JOB MIX FORMULA	CONTROL POINTS	RESTRICTED ZONE
3/4" 19.0mm	100	100	100	100			100	100	
1/2" 12.5mm	95	100	100	100			96	85 - 100	
3/8" 9.5mm	64	92	100	100			75	55 - 75	
No. 4 4.75mm	11	20	97	100			22	15 - 25	
No. 8 2.36mm	3	5	68	100			10	5 - 10	
No. 16 1.18mm	2	3	43	100			7		
No. 30 600μm	2	3	28	100			5		
No. 50 300μm	2	3	18	100			5		
No. 100 150μm	2	3	11	100			4		
No. 200 75μm	1.1	2.5	8.0	100.0			3.1	2 - 4	
G_{sb}									

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

JMF reflects aggregate changes expected during production.

SP 03-2353C (FC-5)
→ ELIMINATES LIME PRETREATMENT
Components 1 thru 3 are pre-treated w/hydrated lime per 337-10.3 Spec. (01/01/03)

Valid only for these Project Nos.:
403714-1-52-01, 255768-1-52-01,
403740-1-52-01, 403738-1-52-01,
410531-1-52-01 & 194425-1-52-01
3

Revised to reflect change in JMF (-200).

State Materials Engineer: Thomas O. Malerk
Original document retained at the State Materials Office

Effective Date: 03 / 21 / 2003

Expiration Date: 03 / 07 / 2005

http://materials.dot.state.fl.us/Smo/Bituminous/CentralBitLab/CentralBituminousLab.htm

Figure 6-1. US-27 FC-5 Granite Mix Design.

STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION
STATEMENT OF SOURCE OF MATERIALS AND JOB MIX FORMULA FOR BITUMINOUS CONCRETE

SUBMIT TO THE STATE MATERIALS ENGINEER, CENTRAL BITUMINOUS LABORATORY, 2006 NORTHEAST WALDO ROAD., GAINESVILLE, FLA. 32609

Contractor APAC-Southeast, Inc., Central Florida Division Address 1445 N.W. 42nd Street, Winter Haven, FL 33881

Phone No. (941) 483-3329 Fax No. (941) 486-0170 E-mail

Submitted By. APAC-Southeast, Inc. Type Mix FC-5 Intended Use of Mix Friction Course

Limestone

TYPE MATERIAL	F.D.O.T. CODE	PRODUCER	PIT NO.	DATE SAMPLED
1. S-1-A Stone	41	White Rock Quarries	87-339	02 / 07 / 2000
2. S-1-B Stone	53	White Rock Quarries	87-339	02 / 07 / 2000
3. Asphalt Screenings	22	White Rock Quarries	87-339	02 / 07 / 2000
4. PG 67-22 w/ ARB 12				
5.				
6.				

PERCENTAGE BY WEIGHT TOTAL AGGREGATE PASSING SIEVES

Blend	45%	48%	7%				JOB MIX	CONTROL	RESTRICTED
Number	1	2	3	4	5	6	FORMULA	POINTS	ZONE
3/4" 19.0mm	100	100	100				100	100	
1/2" 12.5mm	79	100	100				91	85 - 100	
3/8" 9.5mm	36	92	100				67	55 - 75	
No. 4 4.75mm	7	26	100				23	15 - 25	
No. 8 2.36mm	3	7	68				10	5 - 10	
No. 16 1.18mm	3	3	67				7		
No. 30 600µm	3	3	55				7		
No. 50 300µm	3	2	35				5		
No. 100 150µm	2	2	14				5		
No. 200 75µm	1.0	1.0	3.0				4.0	2 - 4	
G_{sb}	2.4252	2.4509	2.527						

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

JMF reflects aggregate changes expected during production.

SP 03-2606A (FC-5)

Transferred from QA 01-10136A (FC-5)

State Materials Engineer
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Effective Date 05 / 29 / 2003

Expiration Date 05 / 29 / 2005

http://materials.dot.state.fl.us/Smo/Bituminous/CentralBitLab/CentralBituminousLab.htm

Figure 6-2. US-27 FC-5 Limestone Mix Design.

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Contractor APAC-Florida, Inc. Central Florida Division Address 1445 NW 42nd Street, Winter Haven, FL 33881

Phone No. (863) 967-0646 Fax No. (863) 551-1946 E-mail jamie_d_hill@ashland.com

Submitted By· APAC-Florida, Inc. Type Mix FC-C Intended Use of Mix Friction Course

TYPE MATERIAL	F.D.O.T. CODE	PRODUCER	PIT NO.	DATE SAMPLED
1. #7 Granite *	44	Martin Marietta Aggregates	TM-322 NS-315	12 / 01 / 2002
2. #789 Granite *	51	Martin Marietta Aggregates	TM-322 NS-315	12 / 01 / 2002
3. Granite Screenings *	22	Martin Marietta Aggregates	TM-322 NS-315	12 / 01 / 2002
4. ~~Hydrated Lime (aggregate pre-treat)~~		Martin Marietta Aggregates	TM-322 NS-315	12 / 01 / 2002
5.				
6. PG 76-22		Blacklidge Emulsions		

PERCENTAGE BY WEIGHT TOTAL AGGREGATE PASSING SIEVES

	Blend	60%	10%	30%				JOB MIX	Gradation		
	Number	1	2	3	4	5	6	FORMULA	Design Range		
	3/4" 19.0mm	100	100	100	100			100		100	
S	1/2" 12.5mm	95	100	100	100			97	85	-	100
I	3/8" 9.5mm	62	93	100	100			77	60	-	80
E	No. 4 4.75mm	11	20	95	100			37	28	-	38
V	No. 8 2.36mm	6	8	72	100			26	25	-	32
E	No. 16 1.18mm	4	5	50	100			18	15	-	23
	No. 30 600µm	3	4	33	100			12	10	-	18
S	No. 50 300µm	2	3	21	100			8	8	-	13
I	No. 100 150µm	2	2	14	100			6	6	-	10
Z	No. 200 75µm	2.0	2.0	8.5	100.0			4.0	4	-	7
E	G_{sb}	2.627	2.633	2.580				2.613			

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

LD 03-2536A (FC-C)
B→ remove pretreatment.

*Components 1 thru 3 are pretreated w/hydrated lime per 337-10.3 Spec. (01/01/2003).

Valid for Project 194435-2-52-01 only.

State Materials Engineer Thomas O. Malerk
Original document retained at the State Materials Office

Effective Date 04 / 16 / 2003

Expiration Date 04 / 16 / 2005

http://materials.dot.state.fl.us/Bituminous/trade_secret_disclaimer.htm

Figure 6-3. US-27 Novachip (Type C) Mix Design.

treated with hydrated lime per FDOT Specification 337-10.3. The hydrated lime met the requirement of AASSHTO M303 Type 1 and was applied at 1 percent. Both FC-5 mixtures had 0.4 percent mineral fiber by weight of total mix.

<u>6.4.2 Binders</u>

The asphalt rubber binder met the requirements of Sections 336 and 337 of the FDOT Specifications. An ARB-12 (AC-30 with 12% ground tire rubber binder) supplied by Blacklidge Emulsions, Inc. was used for the FC-5 mixtures. The design asphalt content for the granite mixture was 6.0 percent, and the limestone mixture was 6.4 percent. A polymer-modifed SBS binder (PG 76-22) was used for the Novachip mixture. The design asphalt content of the Novachip mixture (prior to the application of the tack coat) was 5.0 percent.

The polymer-modified emulsion tack coat used in paving the Novachip mixture was a styrene-butadiene block co-polymer (SB) modified asphalt emulsion. Polymer modification of the base asphalt was completed prior to emulsification. A summary of the binder type and binder content used in each of the field sections is shown in Table 6-1.

Table 6-1. Field Mixture Binder Content.

Type	Binder Type	Binder Content
FC-5 Limestone	ARB-12	6.4%
FC-5 Limestone With Nova-bond	ARB-12	6.4%
FC-5 Granite	ARB-12	6.0%
FC-5 Granite With Nova-bond	ARB-12	6.0%
Novachip	PG 76-22 (SBS Modified)	5.0%

6.4.3 Preparation of Asphalt Pavement Analyzer Specimens

Field cores from US-27 were used in the APA testing. These field cores were obtained using a Florida Department of Transportation coring rig. The coring was divided into two rounds. The first round of coring was performed by the FDOT immediately after construction. These samples had been minimally affected due to traffic loading and weather at the time of coring. The second round was performed by a contractor six months after construction to evaluate the effects of traffic and weathering.

The preparation of APA standard samples from the field cores was achieved by cutting off the top 75 mm (± 3 mm) of the field cores using a masonry wet saw (Figure 6-4), in order to maintain the interface bond that was established during paving. The samples were then washed and dried carefully before being preheated in the chamber at 64º C (147º F) for a minimum of 6 hours but not more than 24 hours before the actual APA rut testing.

Figure 6-4. Masonry Wet Saw.

In addition, loose field mixtures obtained at the time of construction, were compacted to 50 revolutions in a Superpave gyratory compactor for APA testing. The compaction temperature was set at 255° F. Once the specimens were compacted and had been allowed to cool for 24 hours, they were cut into 75 mm ± 3 mm dia. size specimens for APA testing.

6.5 Results from APA Testing

Cores were obtained from the US-27 test sections for APA testing. These cores were obtained immediately after construction, which took place during July 2003. Two samples were required for each APA test. The APA testing was performed with the traditional hose loading mechanism as well as with the new strip loading mechanism, developed by Drakos et al. (2005). The APA tests and their subsequent interpretation followed the procedures discussed previously in Chapter 4. The rut test results were tabulated in Table 6-2 as well as in Figure 6-5 and Figure 6-6. The five mixture types and interface conditions tested were as follows:

- FC-5 with limestone, denoted as “FC-5 LS,” and normal tack coat.
- FC-5 with limestone and Novabond polymer-modified tack coat, denoted as “FC-5 LS NV.”
- FC-5 with granite, denoted as “FC-5 GRN,” and normal tack coat.
- FC-5 with granite and Novabond polymer-modified tack coat, denoted as “FC-5 GRN NV.”
- Novachip mixture, denoted as “FCC NC.”

In Figures 6-5 and 6-6 the traditional way of measuring APA rut depth is identified as the Absolute Rut Depth (ARD). The Differential Rut Depth (DRD) is defined as the difference of the lowest point at the beginning of the test and the highest point recorded at the end of the test.

Table 6-2. First Round APA Rut Depth (mm) Results on Field Cores from US-27 in Highlands County (Obtained in July 2003).

Rut Depth	Tests	FC-5 LS	FC-5 LS NV	FC-5 GRN	FC-5 GRN NV	Novachip
DRD (mm)	Hose	13	10	18*	24	14
	Strip	11	8	25	19	10
ARD (mm)	Hose	6	3	9	11	6
	Strip	3	3	17	14	5
%Δ Area	Hose	-2.05	4.08	0.54	3.51	4.87
	Strip	1.56	-0.49	9.78	5.79	-1.24

*Note: The test was stopped at 5500 cycles due to excessive rutting.

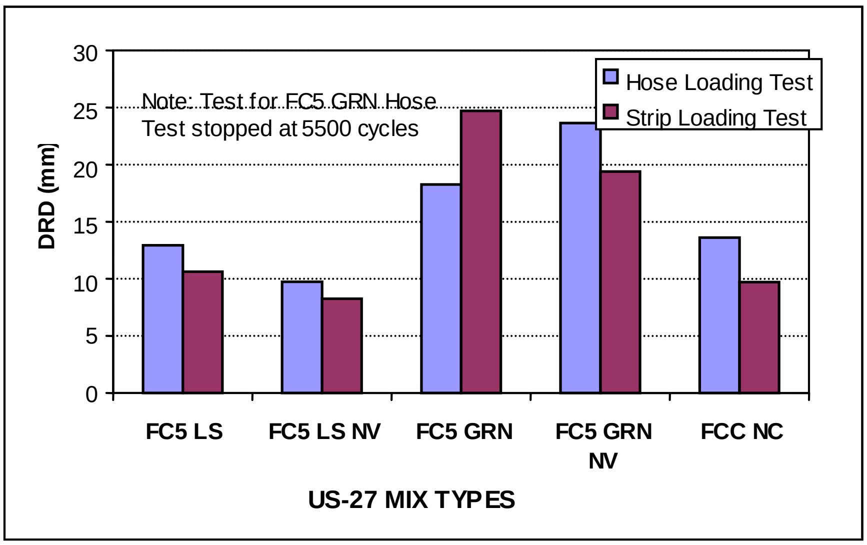

Figure 6-5. First Round Differential Rut Depth.

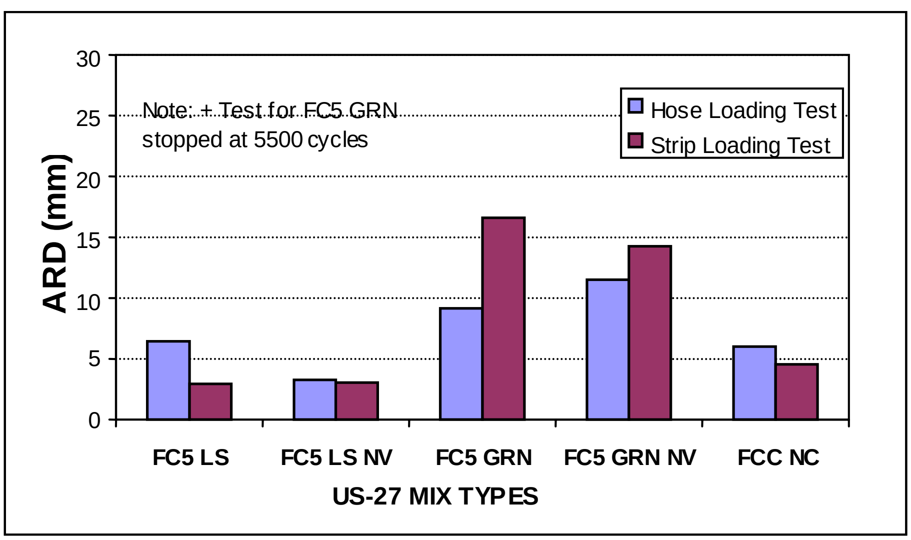

Figure 6-6. First Round Absolute Rut Depth.

The results presented in Table 6-2 and Figures 6-5 and 6-6 show that the Novabond thick polymer modified tack coat helped improve the APA rutting resistance for the FC-5 limestone mixture under both hose and strip loading conditions. Similar results were obtained for the strip loading conditions for the FC-5 granite. However, for hose loading conditions, the testing of the FC-5 granite mixture without Novabond polymer-modified tack coat had to be stopped at 5500 APA cycles, due to excessive rutting of the specimen, thus negating a meaningful comparison between the APA hose testing results for the FC-5 granite with and without Novabond. Overall, the FC-5 limestone and Novachip mixtures showed good rutting resistance, as measured by the Absolute Rut Depth (i.e., the traditional APA rut depth measurement), whereas the FC-5 granite mixture showed excessive rutting. Similarly, the FC-5 granite mixture showed a tendency toward instability rutting, as evidenced by a positive Percent Area Change value shown in Figure 6-7. In contrast, the introduction of Novabond tack coat to the FC-5 limestone mixture appeared to stabilize the mixture, changing the potential rutting mechanism from instability rutting toward consolidation rutting. The granite friction courses (FC-5 granite and FC-5 granite with Novabond) both showed a tendency toward instability rutting.

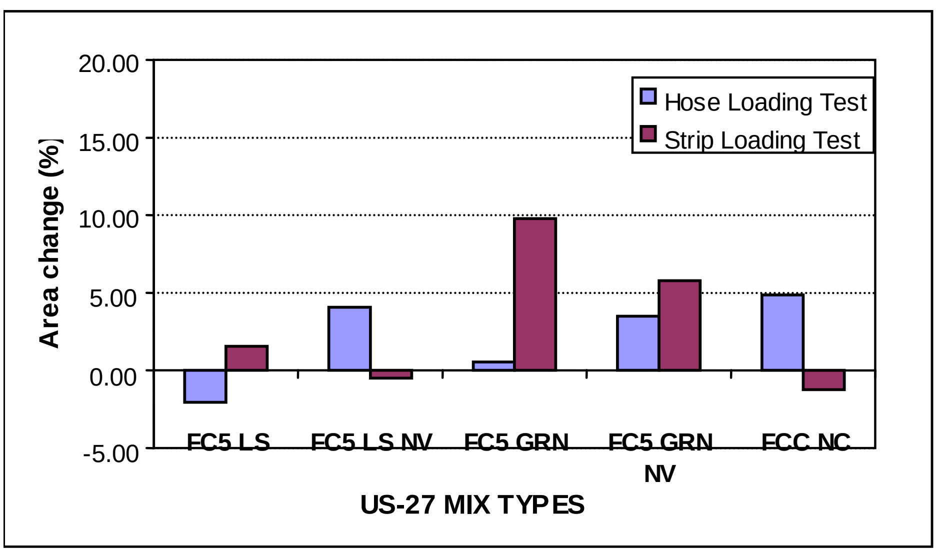

Figure 6-7. First Round Percent Area Change.

In order to examine the behavior of the FC-5 granite mixtures further, cores were again obtained approximately 5 months after the initial construction from the granite FC-5 sections on the US-27 test sections for APA testing. The cores obtained included FC-5 granite and FC-5 granite with a Novabond tack coat. APA testing was performed with both the hose and strip loading on the retrieved cores. In addition, the strip loading was conducted under two different loading conditions, namely at 100 lb load, and at the standard 150 lb load. The APA rut test results are presented in Table 6-3 and in Figures 6-8 through Figure 6-10. The results

Table 6-3. Second Round Field Rut Results from US-27 in Highlands County (Cores Obtained Early December, 2003).

Rut Depth	Tests	FC-5 GRN	FC-5 GRN NV
DRD (mm)	Hose	16	10
	Strip 100 lbs	4	4
	Strip 150 lbs	5	4
ARD (mm)	Hose	6	4
	Strip 100 lbs	0	1
	Strip 150 lbs	0	0
% Area Change	Hose	4.71	0.07
	Strip 100 lbs	1.62	-1.25
	Strip 150 lbs	0.64	-0.11

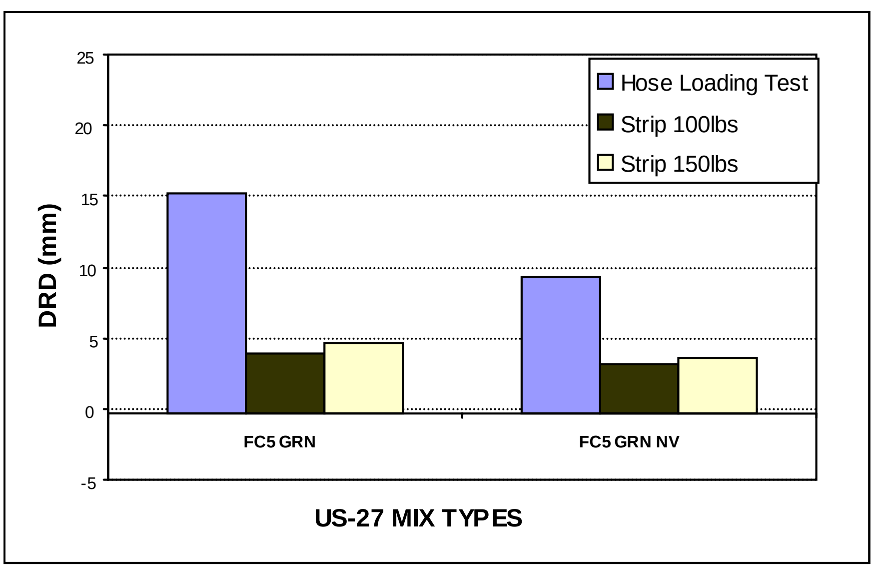

Figure 6-8. Second Round Differential Rut Depth.

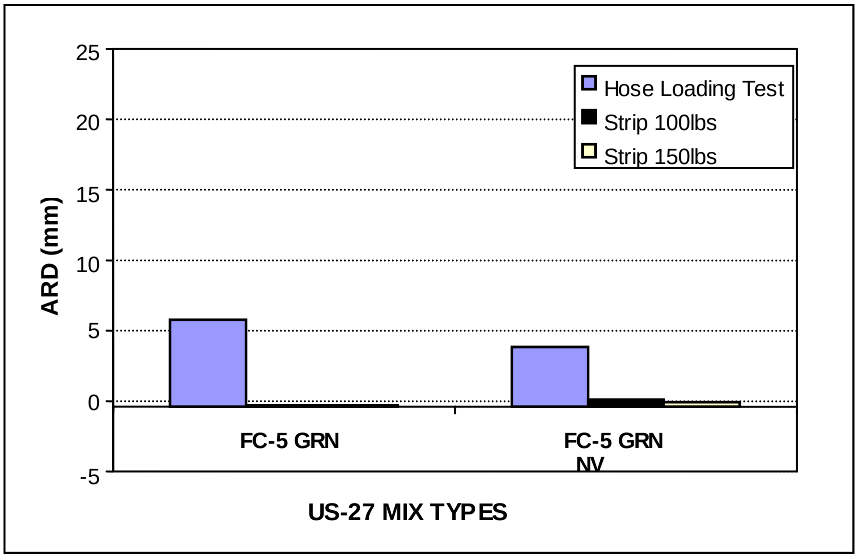

Figure 6-9. Second Round Absolute Rut Depth.

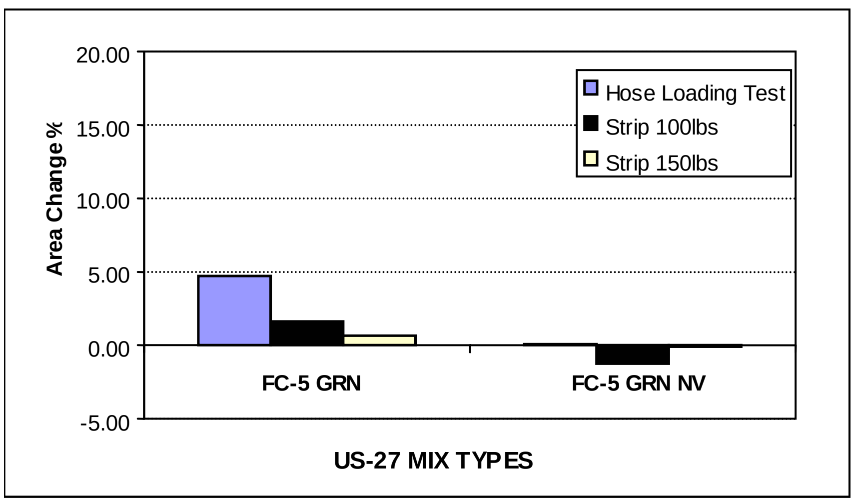

Figure 6-10. Second Round Percent Area Change.

showed significant improvement in rutting resistance over the initial testing results. Again, the FC-5 granite showed a tendency toward instability rutting, whereas the FC-5 granite with Novabond tack coat showed a tendency toward consolidation rutting. Interestingly, there was not a significant difference in the results from the 100 lb and 150 lb strip load testing. However, the hose test results resulted in higher rutting than the strip test results. This is likely due to the fact that the hose applies a high pressure onto the mixture that tends to shear the material apart

(Drakos et al. 2005). This condition is not consistent with actual contact stresses applied from radial truck tires (Drakos et al. 2005). In contrast, the strip loading aims to simulate the actual contact stress states under radial truck tires. Given that virtually no field rutting was observed from the US-27 test sections, it can be reasoned that the APA hose loading mechanism may not be reasonable for use on friction course mixtures.

Another explanation for the large decrease in measured APA rutting is that the aggregate structure in the granite mixtures had not been allowed to "key in" or reorient fully during the initial compaction, but after five months of trafficking, the mix densified adequately to improve rutting resistance. In order to examine this, loose FC-5 granite and FC-5 limestone mixtures obtained from the laboratory were compacted to 50 gyratory revolutions and hose and strip APA testing was performed. In addition, the CoreLok device, shown in Figure 6-11 (ASTM D6752, D6857) was used to determine bulk specific gravity (G_{mb}) of the field and laboratory compacted mixtures which in turn was used along with previous maximum Specific Gravity test results (G_{mm}) to calculate percent air voids.

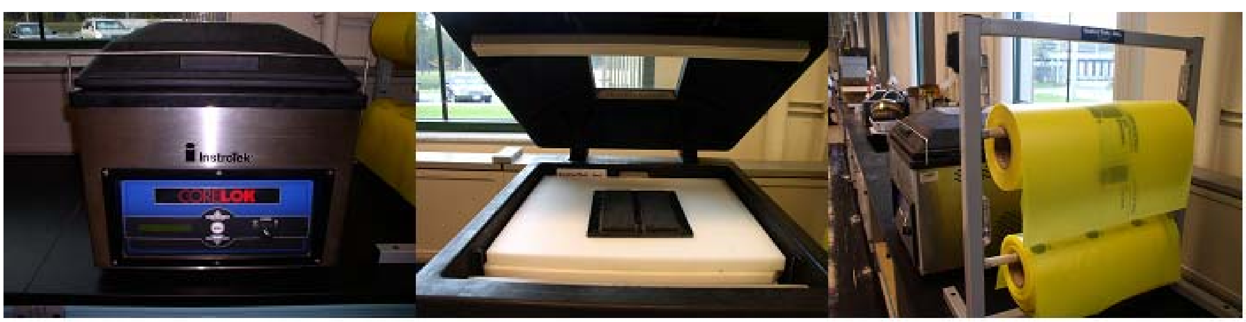

Figure 6-11. CoreLok Device.

The APA test results on laboratory compacted FC-5 granite and FC-5 limestone mixtures without the Novabond tack coat simulation are shown in Table 6-4 and in Figures 6-12 through 6-14. Compared to the testing of the field cores obtained immediately after paving, the laboratory compacted mixtures showed less APA rutting for both the hose and the strip loading conditions.

Table 6-4. APA Rut Test Results on Laboratory Compacted Specimens.

Rut Depth	Tests	FC5 GRN	FC5 LS
ARD (mm)	Hose	3	2
	Strip 150lbs	3	1
DRD (mm)	Hose	10	7
	Strip 150lbs	9	6
% Area Change	Hose	3.43	2.99
	Strip 150lbs	2.16	-2.11

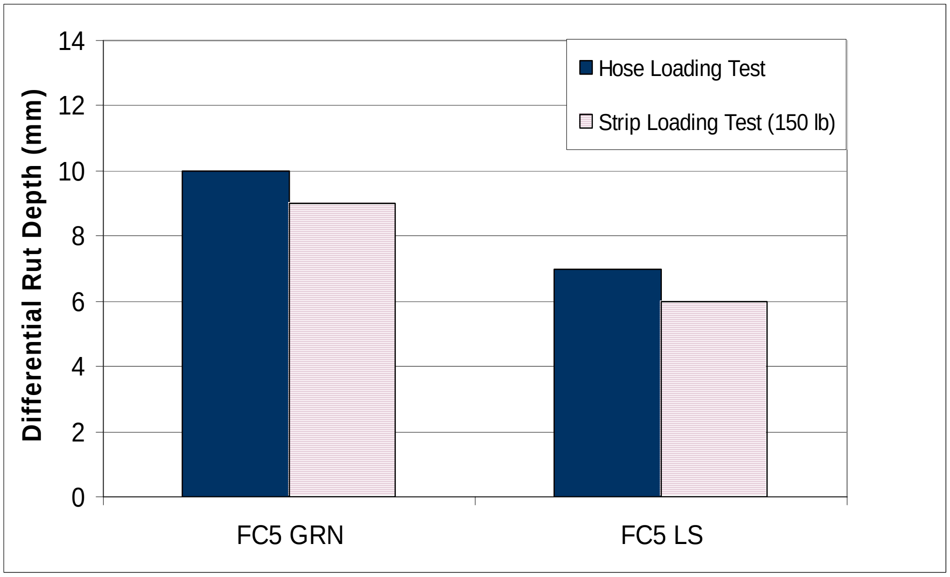

Figure 6-12. APA Differential Rut Depth from Testing on Laboratory Compacted Specimens.

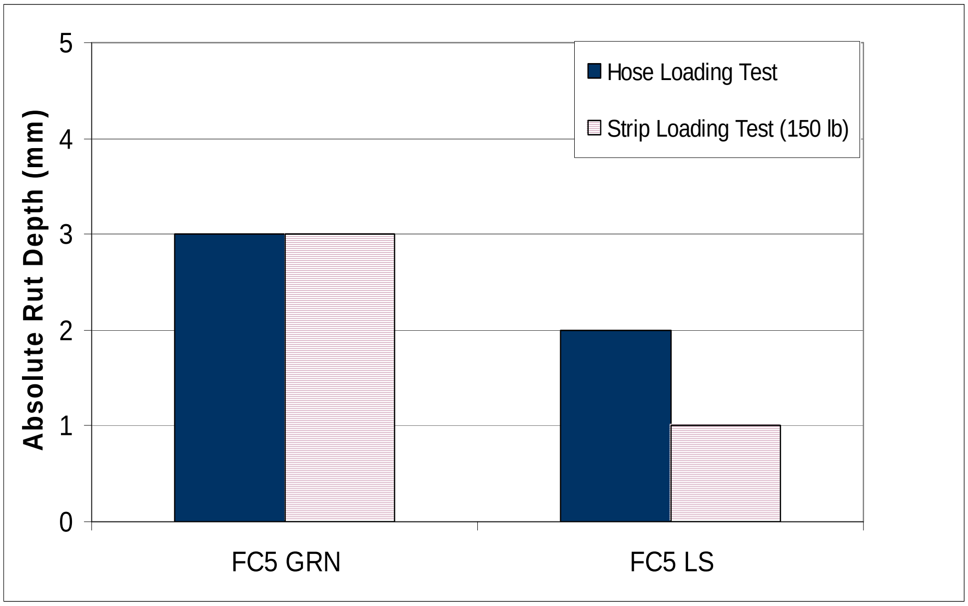

Figure 6-13. APA Absolute Rut Depth from Testing on Laboratory Compacted Specimens.

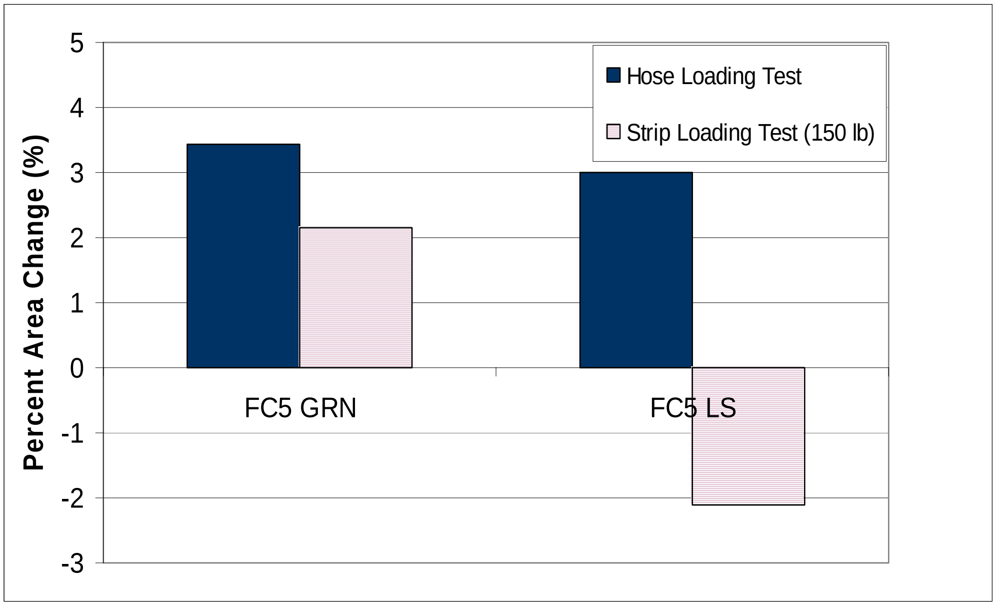

Figure 6-14. APA Percent Area Change Obtained from Laboratory Compacted Specimens.

The FC-5 limestone mixture changed its tendency for instability rutting immediately after paving to consolidation rutting, as indicated by the negative area change in Table 6-4 and Figure 6-14. The FC-5 granite mixture showed less rutting compared to the initial field cores, but slightly more than compared to the 5 month old field core results.

Table 6-5 and Figure 6-15 show the percent air voids for both the field and laboratory compacted mixtures. The initial percent air voids of both the FC-5 granite and FC-5 limestone mixtures were high (28.03 % and 24.9 %, respectively), which likely explains the high APA rutting values obtained from the freshly compacted field cores. However, after 5 months of trafficking, the air voids had decreased down to 20.63 % and 15.92 %, for the FC-5 granite and FC-5 limestone mixtures, respectively. The laboratory compacted FC-5 granite and FC-5 limestone mixtures had air voids of 21.52 % and 17.6 %, respectively. This means that the laboratory compacted mixtures are more consistent with field mixtures that have experienced a few months of trafficking, rather than freshly compacted field mixtures. These results also imply the

Table 6-5. Air Void Content Analysis on Mixtures from the US-27 Field Test Project.

Sample ID and Location		Maximum Specific Gravity	Bulk Specific Gravity (g/cm3)	% Air-void
Field cores 7/1/03	Granite FC-5	2.441	1.757	28.03
	Granite FC-5 w/ Nova-bond	-	-	-
	Limestone FC-5	2.336	1.754	24.90
	Limestone FC-5 w/ Nova-bond	-	-	-
	Novachip	2.474	1.962	20.68
Field cores 12/1/03	Granite FC-5	2.441	1.938	20.63
	Granite FC-5 w/ Nova-bond	-	-	-
	Limestone FC-5	2.336	1.964	15.92
	Limestone FC-5 w/ Nova-bond	-	-	-
	Novachip	2.474	2.100	15.10
Laboratory Samples	FC-5 Granite	2.441	1.916	21.52
	FC-5 Limestone	2.336	1.925	17.60

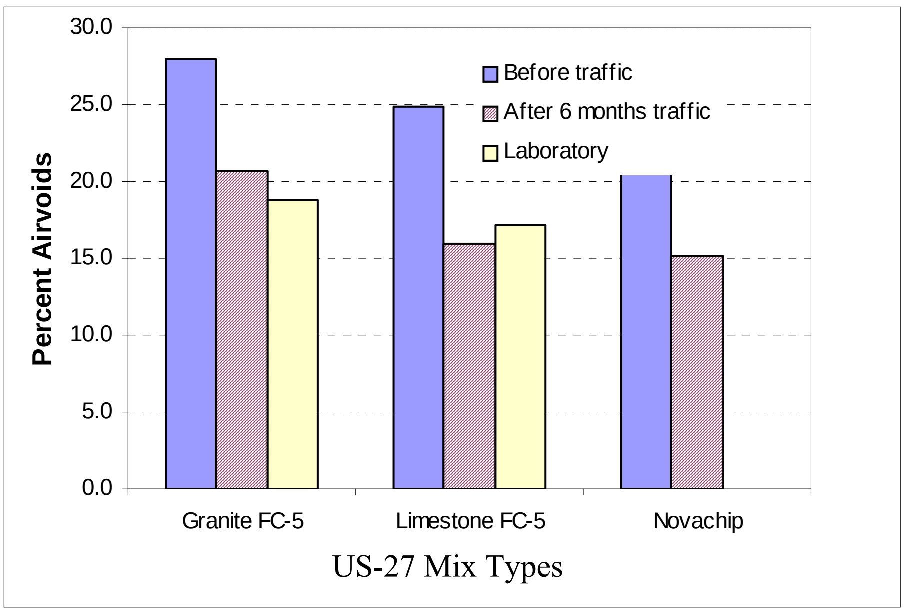

Figure 6-15. Air-Void Comparison.

importance of using representative compaction levels in evaluating the APA rutting potential of open graded friction courses.

6.6 Summary of Findings

The results presented in this chapter show that the presence of a Novabond tack coat appears to help reduce APA rutting. The results also show the importance of testing samples that have air void levels representative of a few months of trafficking, rather than testing fresh field cores. The results also show that 50 gyratory revolutions in the laboratory result in air void levels that correspond to those observed in the field after a few months of trafficking. Thus, the procedure for compacting OGFC mixtures in the laboratory appears to results in mixture densities that are similar to those observed in the field after traffic consolidation. However, more testing needs to be performed to evaluate further the correspondence between percent air voids obtained from field compacted and laboratory compacted specimens. Also, the use of APA hose loading may overpredict the rutting potential of OGFC mixtures, and is thus not recommended. Rather, these mixtures may be tested with the new APA loading strip, in order to obtain a relative measure of rutting resistance. However, it should be noted that field rutting in the US-27 sections was negligible, thus indicating the APA testing in general may not be suitable for evaluating the rutting resistance of open graded surface mixtures.

6.7 References

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CHAPTER 7

EVALUATION OF THE FRACTURE RESISTANCE OF BONDED FRICTION COURSES

The Florida Department of Transportation Longitudinal Wheel Path Cracking study has confirmed that a significant amount of the distress on high-volume Florida pavements is caused by surface initiated wheel path cracking from lateral stresses generated by radial truck tires (e.g., Roque et al. 2004). A recent evaluation of the ten year performance of ground tire rubber in an open graded friction course on SR-16 in Bradford County indicates that increased asphalt content from ground tire rubber modification of the friction course can reduce longitudinal wheel path cracking (Roque et al. 2004). Based on these studies, there is a high likelihood that the bonded friction course process, described previously in Chapter 6, could significantly improve the crack resistance of open graded mixes in Florida by providing more polymer modified asphalt to the friction course and also enhancing the load transfer to the underlying structural layer by providing improved bonding between the friction course and the structural layer.

In the following, the results are presented from the Superpave IDT testing of the mixtures from the US-27 test project in Highlands County, Florida. These mixtures included one granite FC-5 mixture, one oolitic limestone FC-5 mixture, and a Novachip stone matrix asphalt mixture. Since each FC-5 mixture was placed both with a regular tack coat and a thick polymer modified tack coat (Novabond), the overall experiment focuses on the relative performance differences between FC-5 with and without thick polymer modified tack coat, using the Novachip mixture for overall comparison purposes. The details of the materials used and the mix designs are discussed in Chapter 6.

7.1 Preparation of Laboratory-Based Mixtures that Simulate the Effects of a Polymer Modified Tack Coat

The asphalt contents for FC-5 with limestone, granite and Novachip mixtures were 6%, 6.4%, and 5%, respectively. For the friction courses, when placed with the thick polymer modified tack coat, the interface between the fiction course and the underlying layer becomes nearly saturated. The tack coat is heavily polymer modified and essentially contains PG 76-22 binder modified with SBS. Hence, to replicate this field condition in the laboratory, it was essential to nearly saturate the existing mixes with PG 76-22. Even though this tack coat (Novabond) is primarily used in Novachip mixtures, the very same tack coat was also used for FC-5 with limestone and granite on the test sections.

The volumetric calculation for determining the weight of asphalt to be added was based on maintaining the same VMA and reaching the target air void level. The derivation is shown below:

G_{mm1} = initial theoretical maximum specific gravity

G_{mb1} = initial bulk specific gravity of the mixes

G_{mb2} = bulk specific gravity after addition of PG 76-22

W_1 = total weight of mix before addition of PG 76-22

V_t = total volume of mix

V_1 = volume of aggregate and asphalt before addition of PG 76-22

V_{air1} = volume of air voids before asphalt addition

V_{air2} = volume of air voids finally after adding PG 76-22

a_2 = final percentage of air voids desired

δW_{as} = weight of PG 76-22 added.

Thus, $G_{mb1} = W_1/(V_1 + V_{air1})$

$V_1 + V_{air1}$ = volume of compacted specimen = V_t

$$V_t = W_1/G_{mb1} \quad (7\text{-}1)$$

$$V_1 = W_1/G_{mm1}$$

Now, again in this mixture, additional SBS polymer modified PG 76-22 binder was added. The total volume V_t remains the same but the percentage distribution of air voids and asphalt content changes. Thus, the G_{mm} of the mix changes:

Thus G_{mm2} = final maximum theoretical specific gravity

$$= (W_1+\delta W_{as})/(V1+ \delta V_{as}) \quad (7\text{-}2)$$

Now $G_{mb2} = (1-a_2/100) * G_{mm2}$

$$= (1-a_2/100) * (W_1+\delta W_{as})/(V_1+ \delta V_{as}) \quad (7\text{-}3)$$

where δV_{as} = volume of asphalt added = $\delta W_{as}/G_b$; and

G_b = specific gravity of PG 76-22 = 1.028.

Also $G_{mb2} = (W_1+\delta W_{as})/V_t$ (the final volume is same) (7-4)

Thus equating (7-3) and (7-4) and substituting (7-1), the amount of asphalt to be added can be obtained as:

$$\delta W_{as} = ((1-a_2/100) * V_t-V_1) * G_b \quad (7\text{-}5)$$

7.2 Mixing and Compaction of Laboratory Specimens with Excess Modified Asphalt Binder

As already discussed, to replicate the interface consisting of a polymer modified tack coat, a PG 76-22 SBS polymer modified binder was added since the tack coat contained the same binder. Hence, it was also decided to reduce the air voids to 2%. The mixing and compaction temperature for all the three mixtures (FC-5 granite, FC-5 limestone, and Novachip) was 320° F.

During mixing, it was observed that the amount of drain down was excessively high and it was difficult to work with these mixtures at such high asphalt contents. Further, compaction was virtually impossible because of the high drain down.

Hence, it was decided that the asphalt added should be such that air voids were reduced by 50% in the case of Novachip and 25% in case of FC-5 with granite and limestone. Again, initially the mixing and compaction temperature was kept as 320° F. However, the amount of drain down was still significant. Then it was decided that both mixing and compaction temperature be brought down to 255° F. However, because of the polymer-modified asphalt, mixing became very difficult since the asphalt was very viscous at such a low mixing temperatures. Finally, a mixing temperature of 320° F and compaction temperature of 255° F was adopted and the workability of these mixtures improved significantly. However, since the asphalt content was still very high, the loss in asphalt due to drain down was in the range of about 10-15 g.

It was observed that after the addition of asphalt to these mixtures, especially FC-5 with granite, they became very sensitive to the compaction temperature. For the mixtures compacted without the additional modified asphalt, the number of gyrations was determined as 50. However, for the saturated specimens, the gyration level varied from 60-90. This indicates that the rate of reduction in temperature in the case of polymer-modified asphalt is very high.

Finally, after compaction the specimens were not immediately retrieved from the mold. It was allowed to cool down for around 1 hr. 45 min. before extruding the sample out (for unsaturated field mix the specimens were allowed to cool down for 45 minutes before retrieving them). This was because it was observed that these mixes collapsed when they were retrieved from the mold immediately after compaction, due to their high air void levels.

The final air void level and the theoretical maximum specific gravity (G_{mm}) for the mixtures after compaction are shown in Table 7-1.

Table 7-1. Determination of Weight of Added SBS Polymer Modified PG 76-22 Binder.

Type	Initial				Final				
	Weight of Mix (g)	**G_{mb}***	**G_{mm}****	**Air Voids (%)**	**Air Voids (%)**	**% G_{mm}**	**G_{mm}**	**Asphalt Added (g)**	**Compaction Height (mm)**
FC-5 limestone	4800	1.923	2.336	17.68	13.26	86.740	2.269	113.50	141.250
FC-5 granite	4700	1.916	2.441	21.51	16.13	83.869	2.350	135.59	138.813
Novachip	4800	2.089	2.474	15.56	7.78	92.219	2.352	183.79	130.000

* G_{mb} = Bulk Specific Gravity; ** G_{mm} = Maximum Theoretical Specific Gravity

7.3 Long-Term Oven Aging of Friction Course Mixtures

The loose mixtures used in this part of the study were sampled from the paver during construction and hence they had already undergone short-term aging. The mixtures were subjected to long-term aging according to LTOA-AASHTO PP2-94. However, since the mixtures were very coarse and open, there was a possibility of these mixes falling apart or deforming during aging. Hence, a procedure was developed to contain the compacted specimens from falling apart during aging.

A wire mesh with openings of 1/8″ and steel clamp was used. The mesh size was chosen in order to ensure that there is good circulation of air within the sample for oxidation and at the same time, to prevent the smaller aggregate particles from falling through the mesh. The specimen was covered twice by the mesh and two clamps were used to contain the specimen without applying excessive pressure on it. The whole system is shown in Figures 7-1 and 7-2.

Figure 7-1. 1/8″ Mesh Used for Containing the Specimen.

Figure 7-2. Specimen Contained with Mesh.

7.4 Superpave IDT Fracture Test Results

The results obtained from the IDT test were analyzed using ITLT software developed at the University of Florida (Roque and Buttlar, 1992; Buttlar and Roque, 1994; Roque et al. 1997). Using the results from the software, the DCSE limit and Energy Ratio were calculated. The Energy Ratio is a dimensionless parameter that serves as single criteria for cracking performance of mixtures in pavements. It is defined as follows (Roque et al. 2004):

$$ER = DCSE_f / DCSE_{min} \tag{7-6}$$

where $DCSE_f$ is the dissipated creep strain energy threshold, and $DCSE_{min}$ is the minimum dissipated creep strain energy, which is given by:

$$DCSE_{min} = m^{2.98} * D_1 / A, \tag{7-7}$$

where m and D_1 are creep compliance parameters, and A is a parameter dependent on tensile stress at the bottom of the asphalt pavement layer (σ), and tensile strength, S_t:

$$A = 0.0299 * \sigma^{-3.10} * (6.36 - S_t) + 2.46 * 10^{-8} \tag{7-8}$$

Since the applied tensile stress is taken at the bottom of the asphalt concrete layer, it is very much dependent on the stiffness of the asphalt concrete layer. In the case of friction courses, the whole pavement structure can be regarded as a composite consisting of a friction course and the asphalt concrete layer beneath it. Thus, the stress at the bottom of the asphalt concrete (AC) layer should be considered for ER calculation for the friction course. It should also be noted that the ER was determined for dense graded mixtures only and it has not been truly calibrated for friction courses.

Thus, a typical pavement structure with the given loading condition shown in Figure 7-3 was considered for determining ER for all mixes.

The thickness for the FC-5 and Novachip mixtures was taken as 0.75 inches. The stresses were calculated using layered elastic analysis.

The other parameters used for evaluation are the FE, DCSE, tensile strength, failure strain and creep strain rate. The creep strain rate for a 1000 sec creep test is calculated as follows:

$$d\varepsilon(t)/dt/\sigma = dD(t)/dt = D_1 * m * (1000)^{m-1} \tag{7-9}$$

where $d\varepsilon(t)/dt/\sigma$ = strain rate per unit stress; and

D(t) = creep compliance.

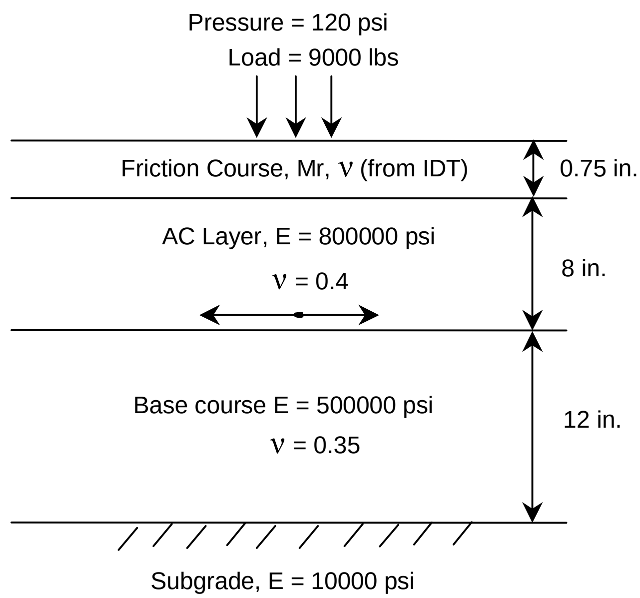


Figure 7-3. Typical Pavement Structure for Stress Calculation.

7.5 Evaluation of FC-5 and Novachip Field Mixtures

The test results for the friction courses are presented in Figures 7-4 through 7-10. The three field mixtures were also subjected to long-term oven aging to study the effects of aging on their fracture resistance. In addition, to replicate the interface between the surface mixture and the underlying HMA, which contains the polymer modified tack coat, additional PG 76-22 was added to the field mix to reach the desired target air void level. The results for the saturated mixtures are presented later in this chapter.

7.5.1 Unaged Field Mixtures

As shown in Figure 7-6 for the FC-5 granite and FC-5 limestone, the ER is around 1.5 which indicates good field cracking resistance. For the Novachip mixture, the ER is close to 3, which indicates that it has excellent fracture resistance. In the case of FC-5 granite, the failure strain is around 3000 micro strain and the DCSE is close to 3 KJ/m^3. For the FC-5 limestone, the

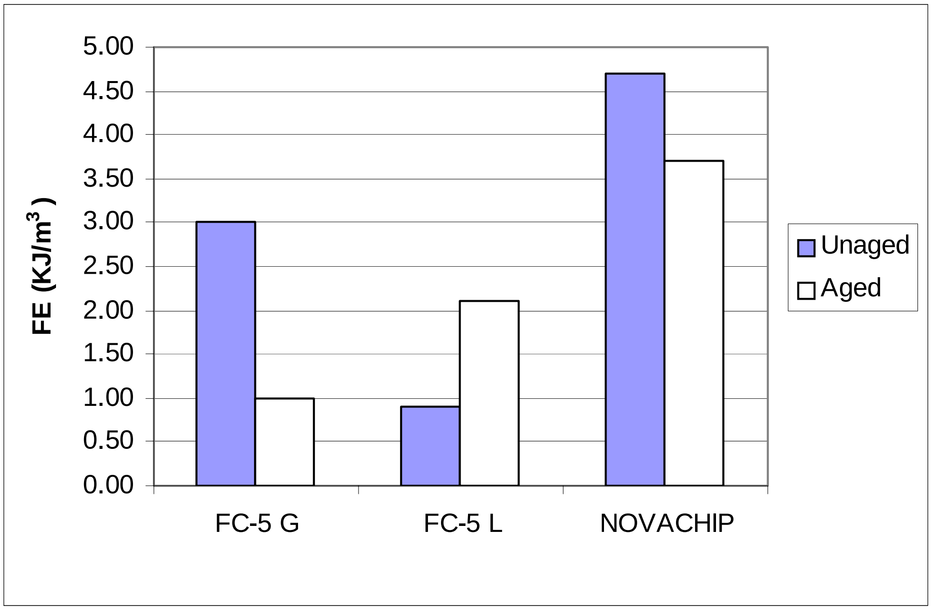


Figure 7-4. Fracture Energy for Field Mixtures.

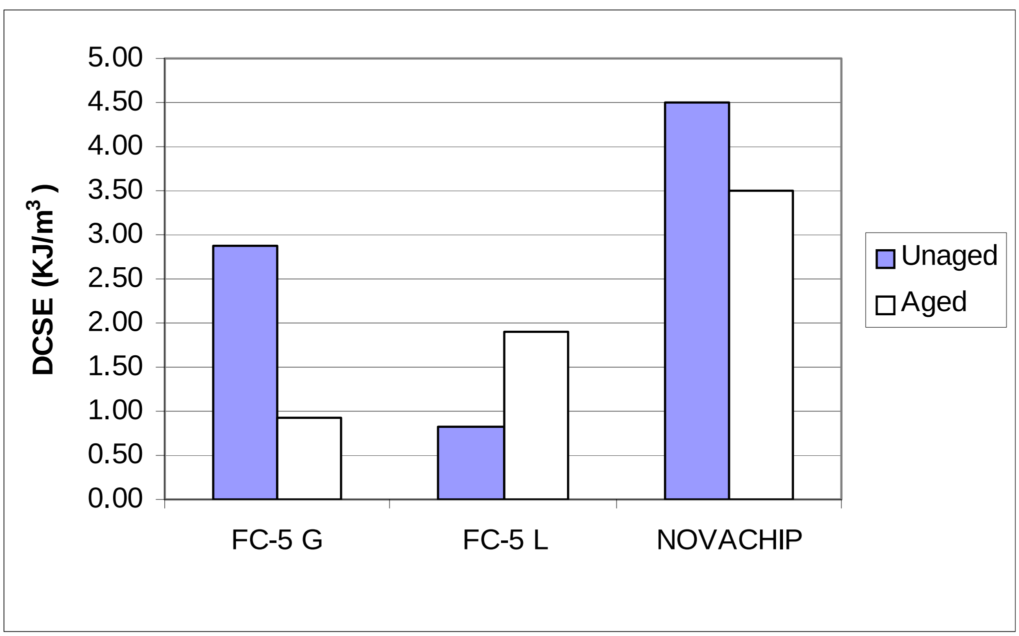


Figure 7-5. Dissipated Creep Strain Energy for Field Mixtures.

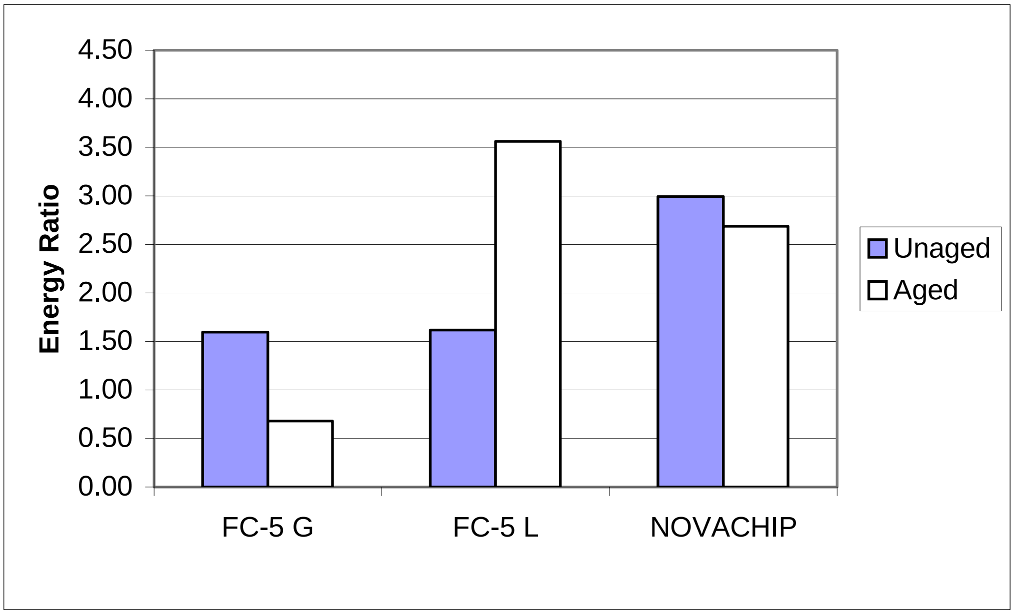


Figure 7-6. Energy Ratios for Field Mixtures.

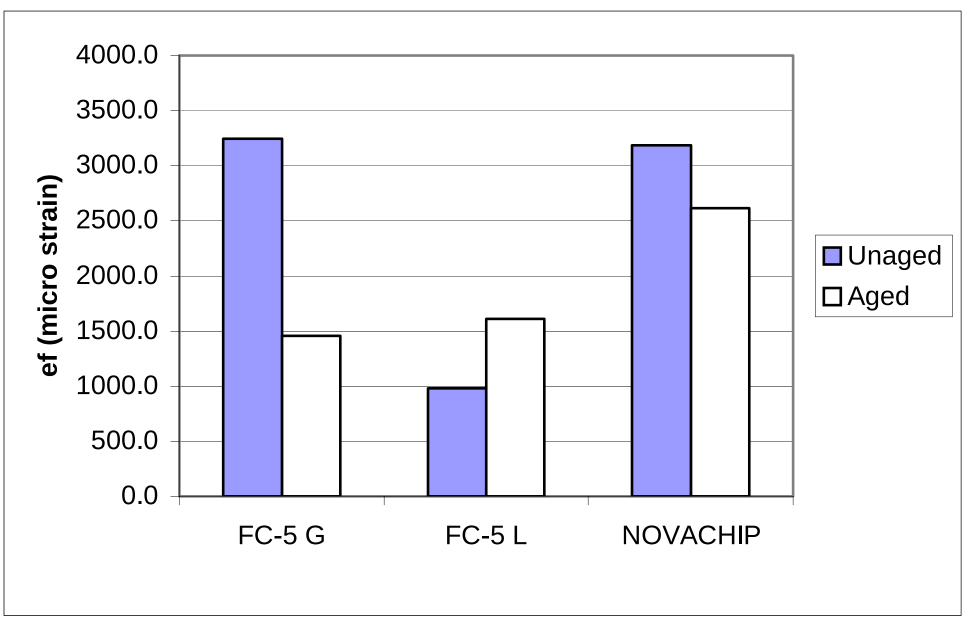


Figure 7-7. Failure Strain for Field Mixtures.

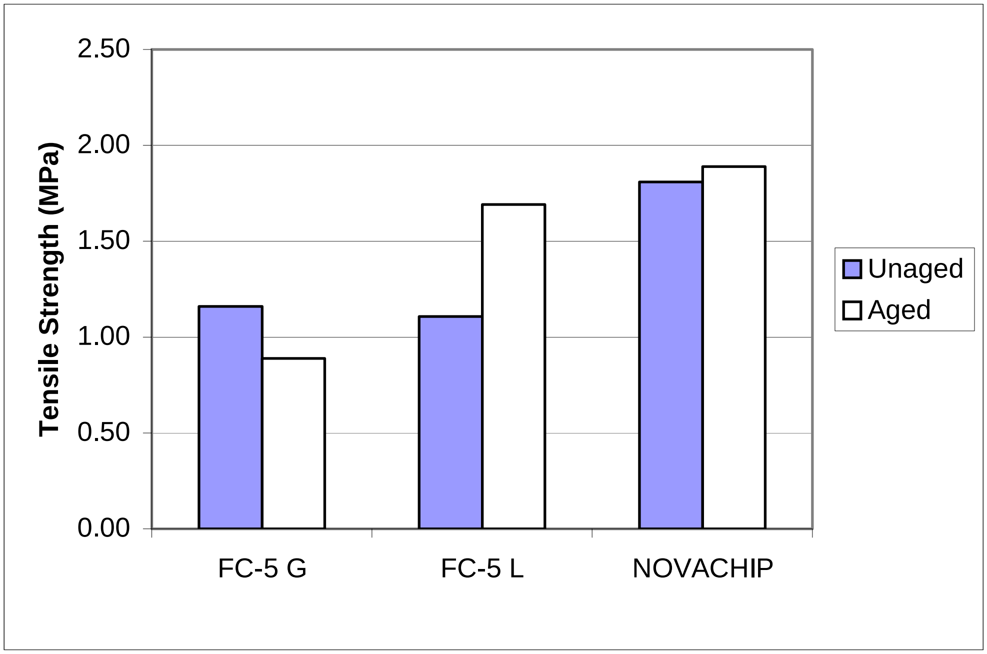


Figure 7-8. Tensile Strength for Field Mixtures.

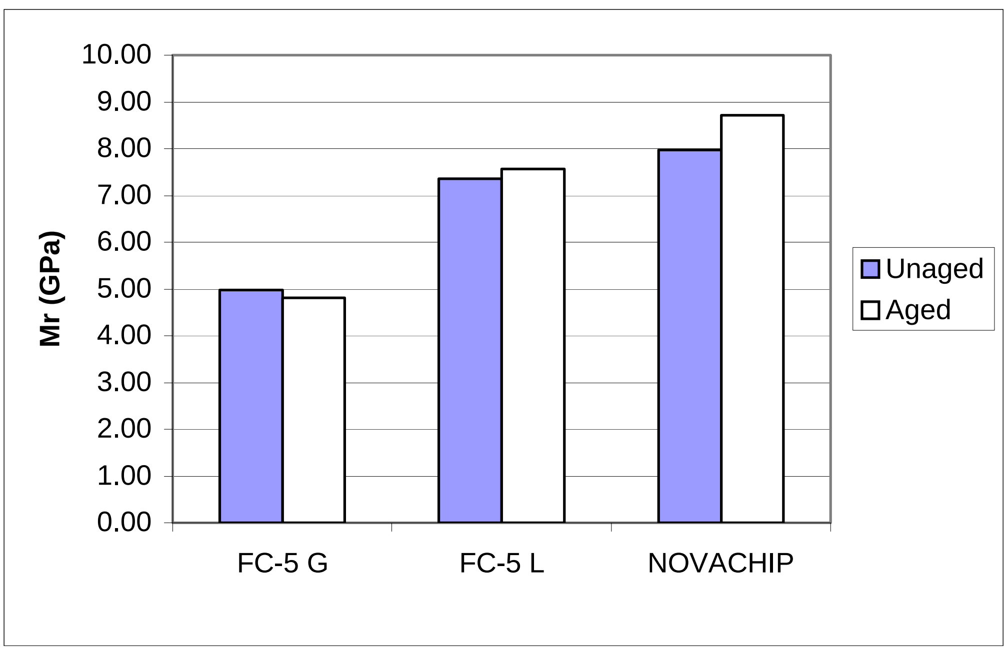


Figure 7-9. Resilient Modulus for Field Mixtures.

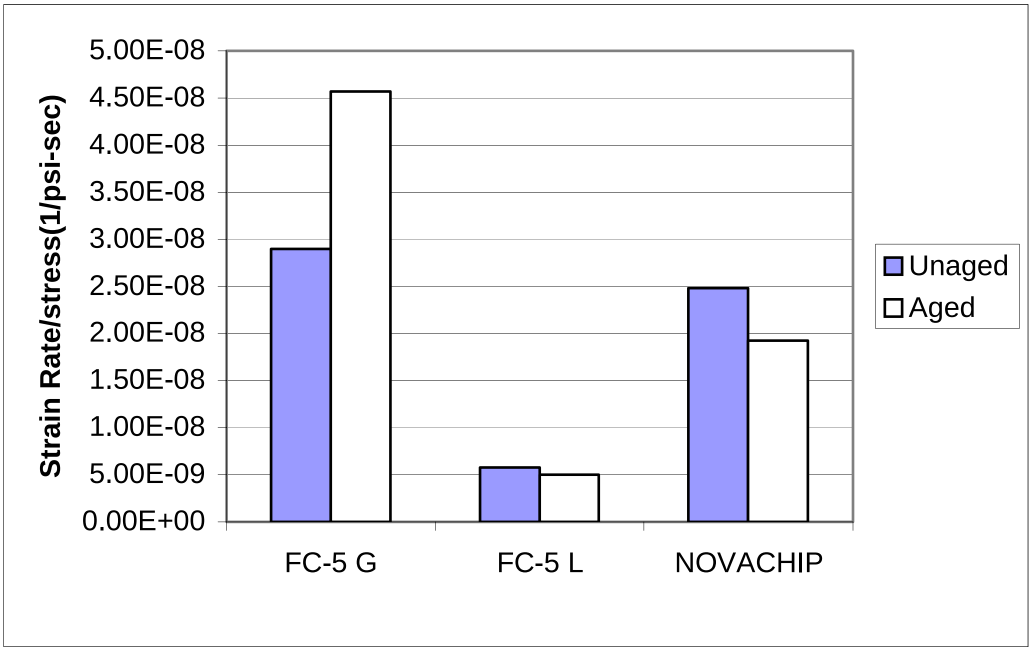


Figure 7-10. Creep Rate for Field Mixtures.

DCSE is very low (just below 1 KJ/m^3). This is primarily because of low failure strain (around 950 micro strain). For the Novachip mixture, the DCSE is around 4.5 KJ/m^3 with a high failure strain of 3200 micro strain. As far as the tensile strength and resilient modulus are concerned, Novachip has higher values as compared to FC-5 granite and limestone.

Finally, the Novachip and FC-5 granite mixtures have significantly higher creep rate (close to 3e-08/ psi-sec) as compared to the FC-5 limestone. For the FC-5 granite, this is one of the reasons for the lower ER value. For the Novachip mixture, the high creep rate is compensated by high DCSE, which in turn is reflected in its high ER value.

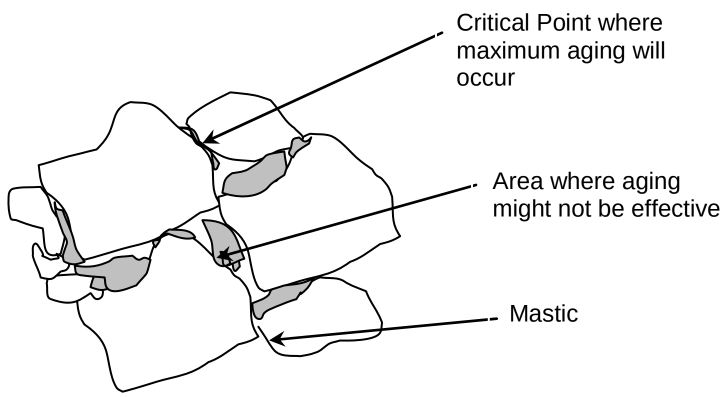
7.5.2 Aged Field Mixtures

In the case of FC-5 granite, there is a substantial decrease in ER due to long-term oven aging. As shown in Figure 7-7, for aged FC-5 granite mixture, the failure strain is reduced by half as compared to the unaged mixture. As a result, the DCSE also plummeted to 1 KJ/m^3 which resulted in low ER value of 0.7.

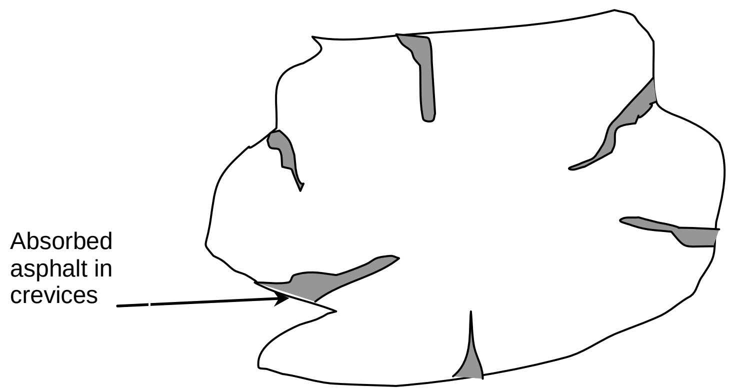
The creep rate however increases with aging for granite. This could possibly be explained with the following hypothesis: Open graded mixtures may have critical points in the matrix consisting of aggregate and mastic, wherein aging takes place locally. These critical points are the points of stone-to-stone contact. Further, there are regions in the matrix where aging is not as effective due to large “globules” of mastic. Hence, due to aging, a structure with varied stiffness is developed and hence these different regions would respond differently to load application. Thus, any kind of damage to these critical points, which govern the creep behavior of the specimens, causes damage in the specimens, resulting in higher creep rate. This concept is illustrated in Figure 7-11(a).

For the Novachip mixture, aging seems to have little effect on FE, DCSE and ER. However, interestingly, for the FC-5 limestone mixture, aging increases ER by factor of two (ER for aged limestone is around 4.2). This is primarily due to higher failure strain, DCSE, and lower creep rate. To explain this phenomenon, a study was done on the effect of aging on these mixes, which is presented below.

The asphalt content of open graded FC-5 limestone friction courses was 6.4%. Further, limestone in general has a very rough texture and has lot of crevices and pores in it. The long-term oven aging temperature is around 85° C and at such a high temperature, the asphalt flows, occupying the crevices and the pores in the aggregates. This is illustrated in Figure 7-11(b). Thus, with aging in the laboratory, the absorption of asphalt by the limestone increases and the aggregates become more ductile. In addition, the bonding between the aggregates also increases. Visual observation of the cracked specimens indicated that the failure happened through the aggregate. Thus, because of increased absorption due to aging, limestone becomes more ductile and hence the failure strain increases resulting in increased DCSE and ER.



(a) FC-5 Granite



(b) FC-5 Limestone

Figure 7-11. Effect of Aging on: a) FC-5 Granite; b) FC-5 Limestone.

It should be noted however, that in the field, the temperature varies from anywhere between 10° and 60° C and that the pavement temperature approaches the higher end only during part of the day in the summer. Thus, it is likely that it is only in laboratory aging conditions that such results can be observed and that it is not a realistic representation of field conditions. Therefore, the AASHTO PP 2-94 procedure for Long Term Oven Aging may not apply to open graded friction courses.

To corroborate the laboratory results further, another set of testing was performed on limestone under the following conditions, i.e., 1) unaged, 2) 2-day aging, 3) 5-day aging (LTOA), and 4) 10-day aging. The results are presented in Figures 7-12 through 7-18.

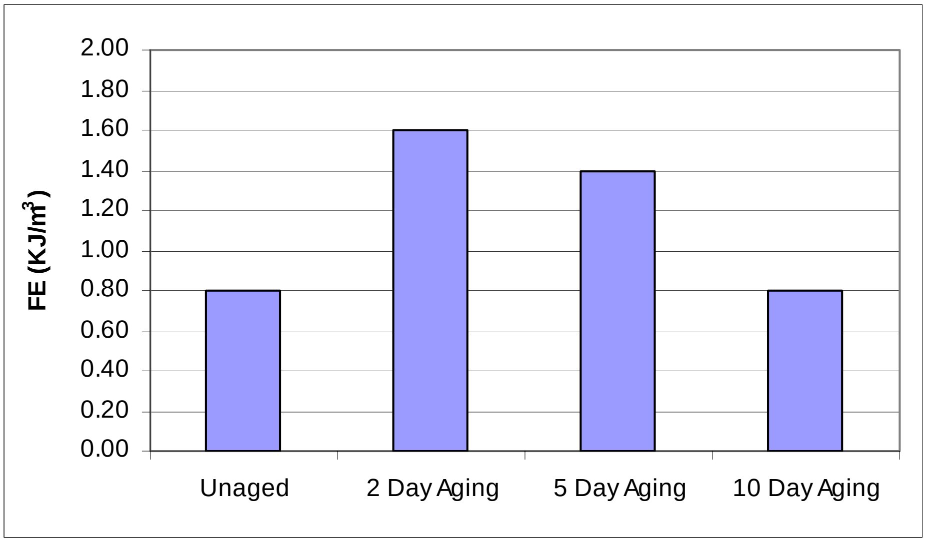


Figure 7-12. FE for FC-5 Limestone at Various Stages of Aging.

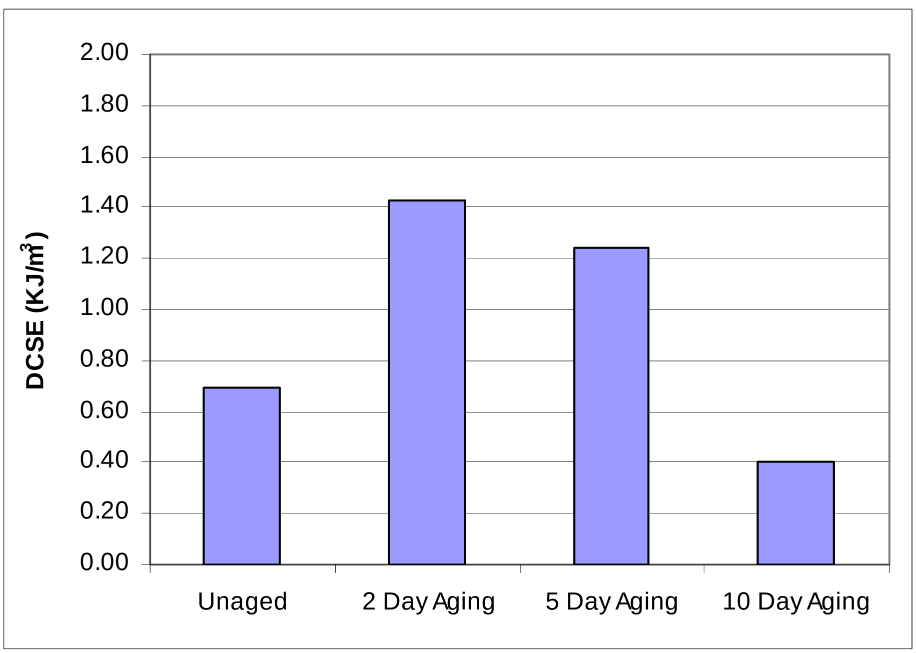


Figure 7-13. DCSE for FC-5 Limestone at Various Stages of Aging.

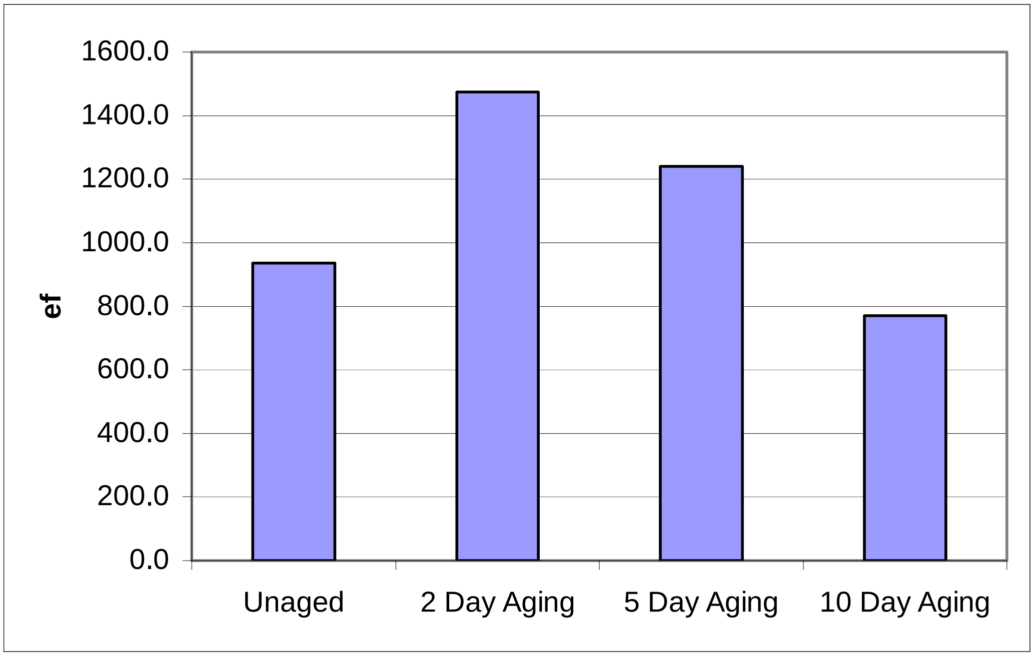


Figure 7-14. Failure Strain for FC-5 Limestone at Various Stages of Aging.

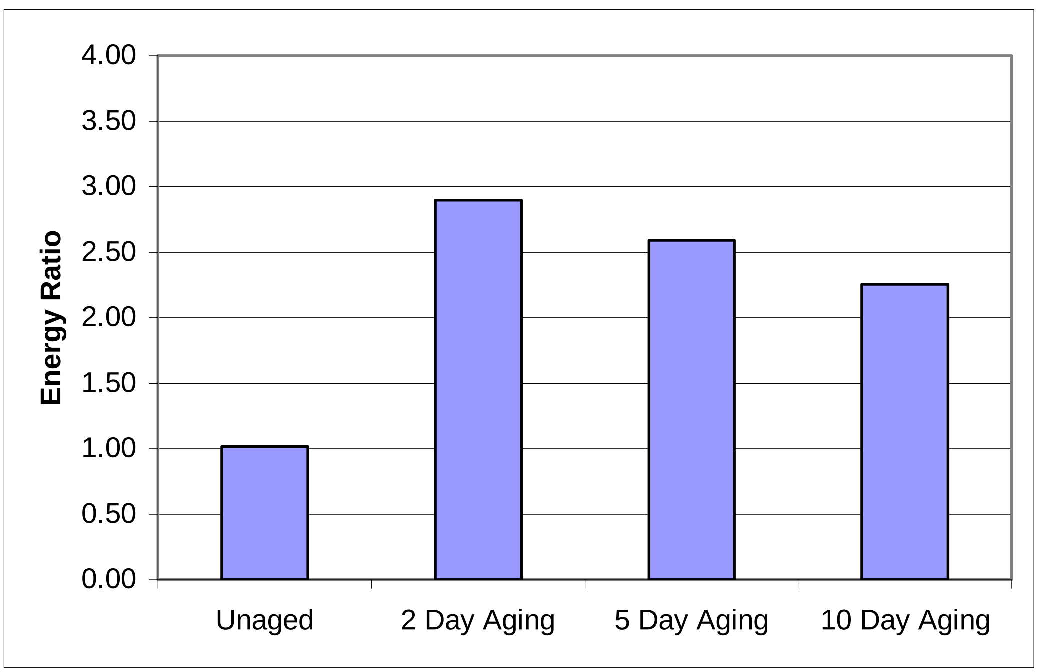


Figure 7-15. Energy Ratio for FC-5 Limestone at Various Stages of Aging.

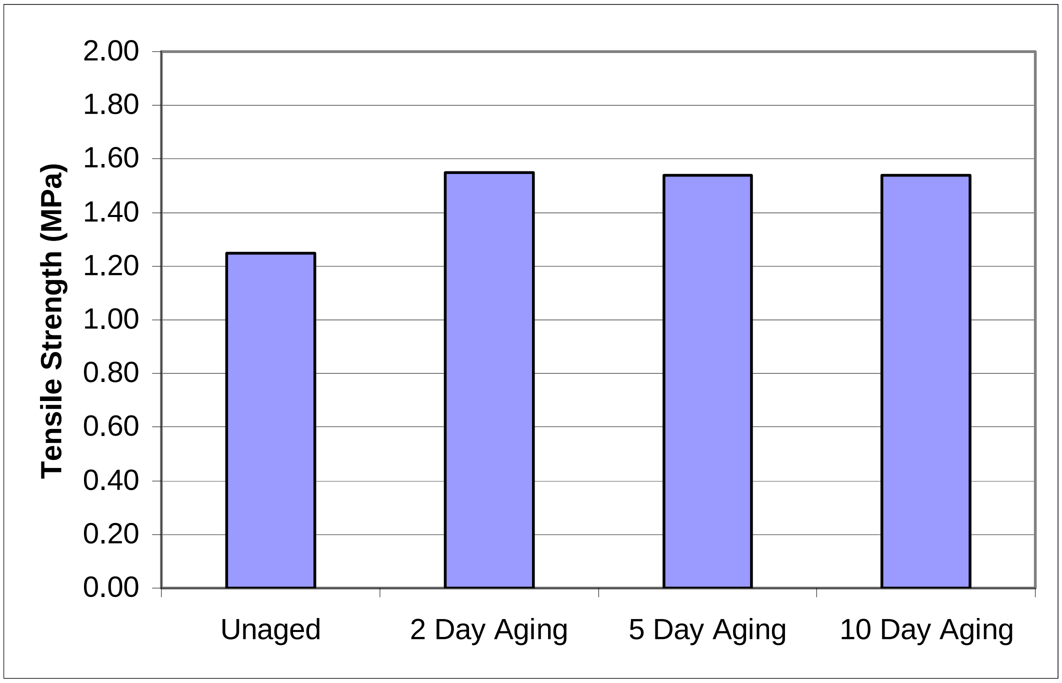


Figure 7-16. Tensile Strength for FC-5 Limestone at Various Stages of Aging.

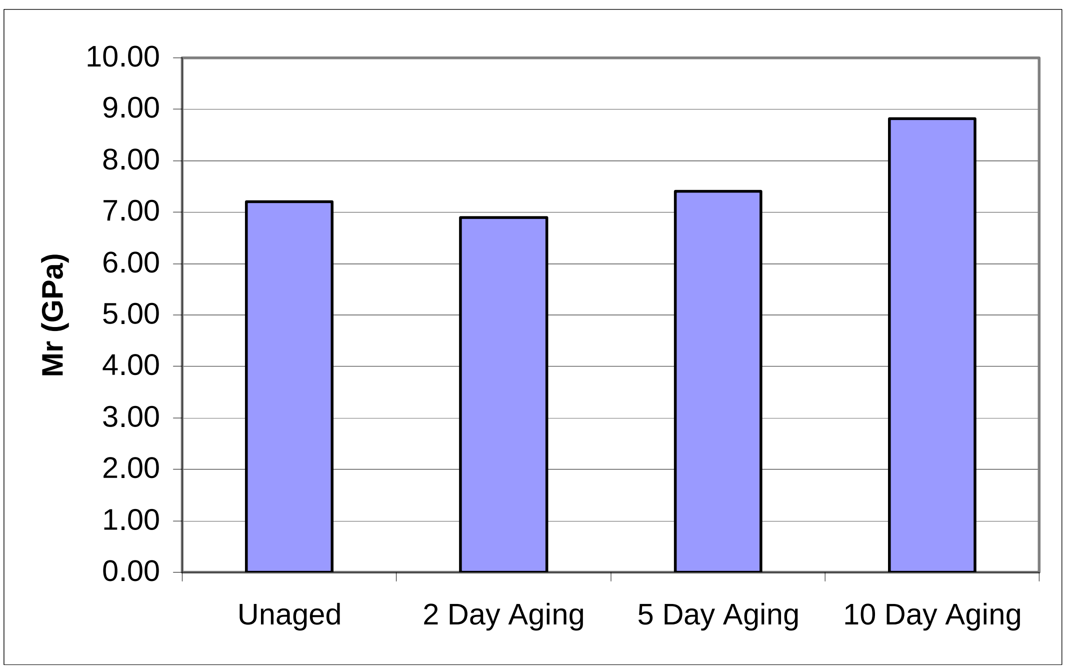


Figure 7-17. Resilient Modulus for FC-5 Limestone at Various Stages of Aging.

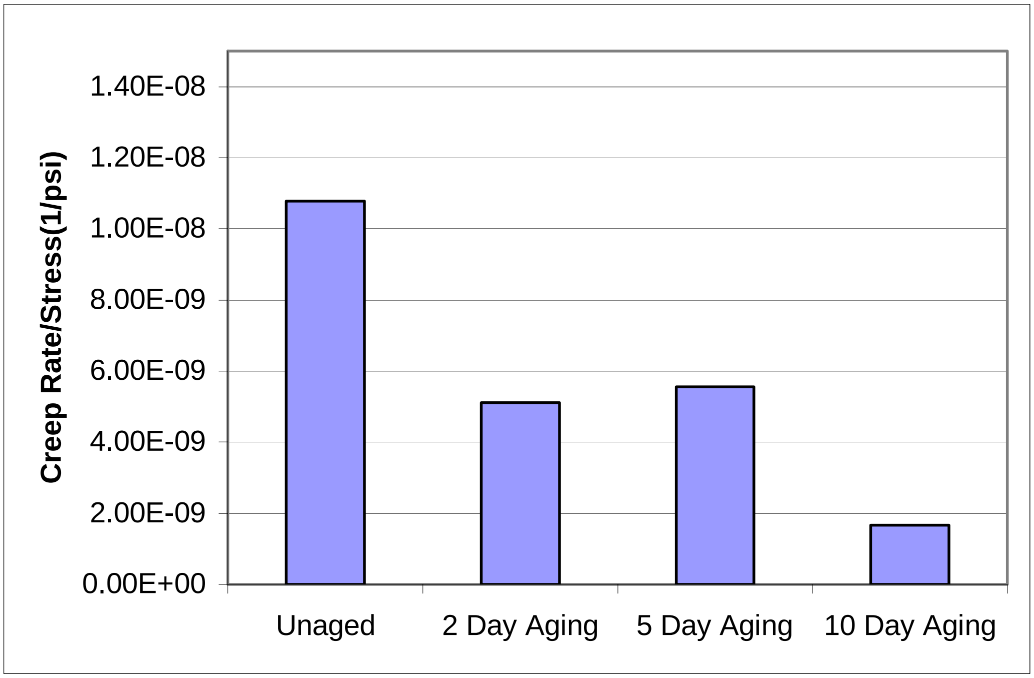


Figure 7-18. Creep Rate for FC-5 Limestone at Various Stages of Aging.

The results for unaged and long-term oven aged specimens showed the same trend as before. Clearly, looking at the ER, it can be seen that the 2-day and 5-day aged mixtures have higher ER as compared to the unaged mix. The same is the case for the DCSE and failure strain. However, for the 10-day aged mix, the DCSE falls below that of the unaged mix, though the ER remains higher.

The 2-day aged mix has the highest ER and then it decreases with aging. The same trend is observed for DCSE and failure strain. Thus, as stated before, during the process of laboratory aging, the asphalt starts flowing and occupying the crevices and pores in the limestone aggregates making the mixture more ductile. However, as aging progresses, the absorbed asphalt also starts aging and the failure strain decreases, and hence the DCSE, for the 5-day and 10-day aged mixtures. This again is reflected in the lower ER values. The resilient modulus increases slightly with aging. The tensile strength increases for the aged mixes. For this new set of testing, the creep rate of the unaged mixtures seems to be on the higher side as compared to field mixtures. However, the 5-day aged mixture seems to have a similar creep rate as obtained before for the aged field mixture. The 10-day aged mix has the lowest creep rate.

7.5.3 Saturated Unaged Mix

As mentioned previously, a SBS modified PG 76-22 was added to the mixtures to evaluate the effect of Novabond on the fracture resistance of the mixtures. Clearly, for FC-5 granite and Novachip mixtures, the polymer modified Novabond increases the FE, DCSE and failure strain (Figures 7-19 through 7-25). In fact, for the Novachip mixture, the FE increases to 9 KJ/m^3, which is twice as much as for the unaged field mix. The ER for granite (around 2) does not increase substantially, possibly because of an increase in creep rate with the addition of asphalt (Figure 7-26). However, for the Novachip mixture, the creep rate remains more or less the same as that of the field mix and the DCSE increases significantly to 9 KJ/m^3 hence a large jump in the ER value (slightly above 6) is observed. For the FC-5 limestone mixture, the failure strain increases to around 1400 micro strain and DCSE increases slightly to around 1.2 KJ/m^3.

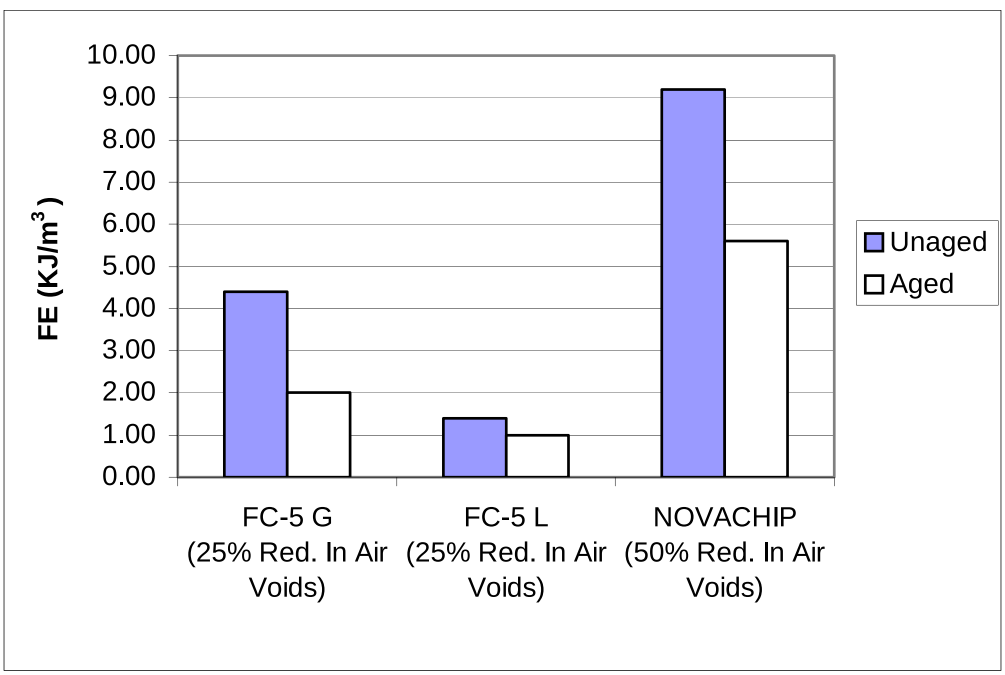


Figure 7-19. Fracture Energy (FE) for Saturated Mix.

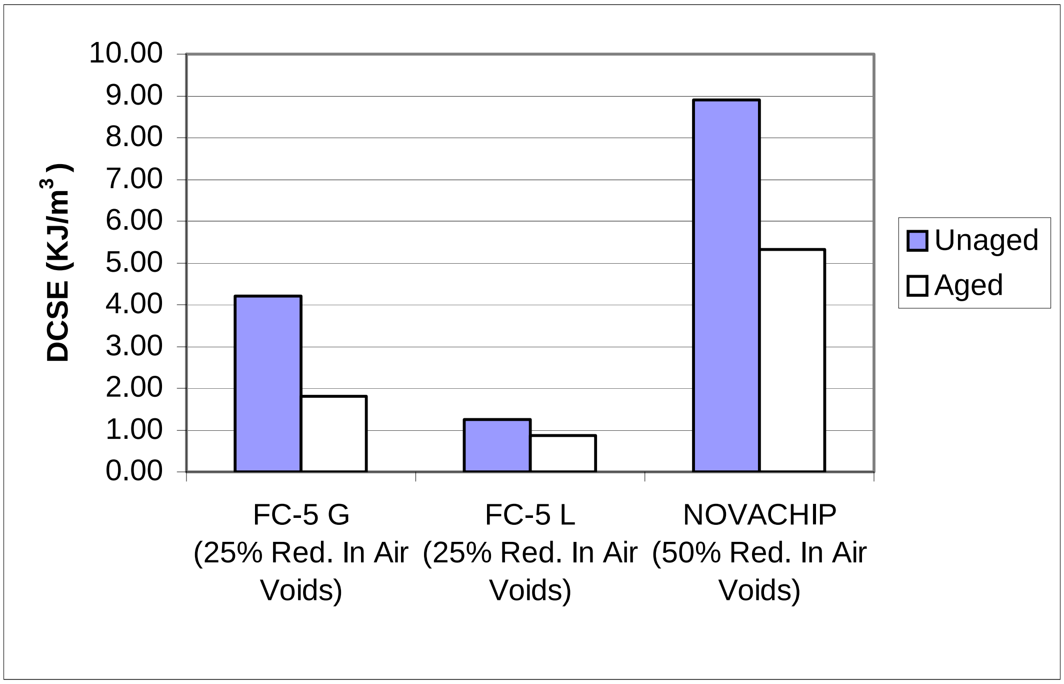


Figure 7-20. Dissipated Creep Strain Energy (DCSE) for Saturated Mix.

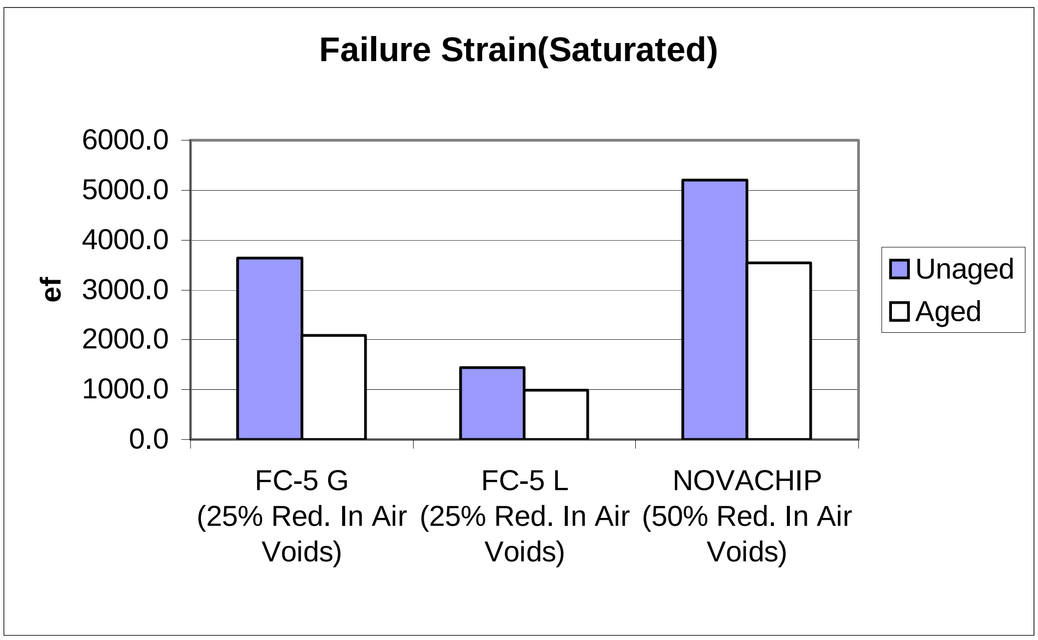


Figure 7-21. Failure Strain (e_f) for Saturated Mix.

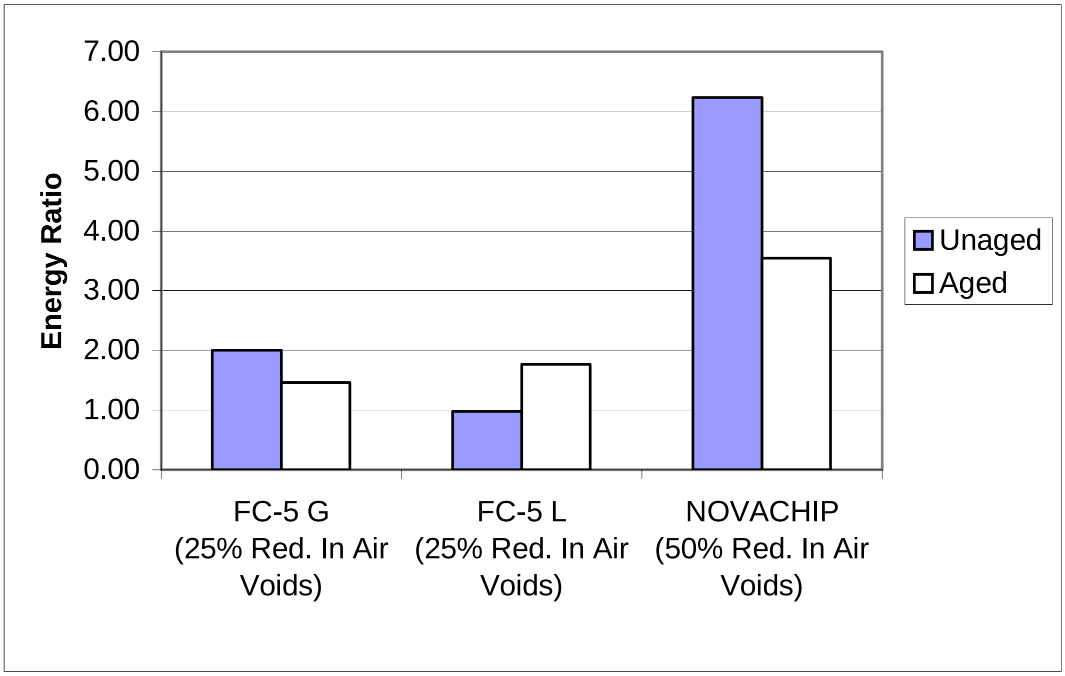


Figure 7-22. Energy Ratio (ER) for Saturated Mix.

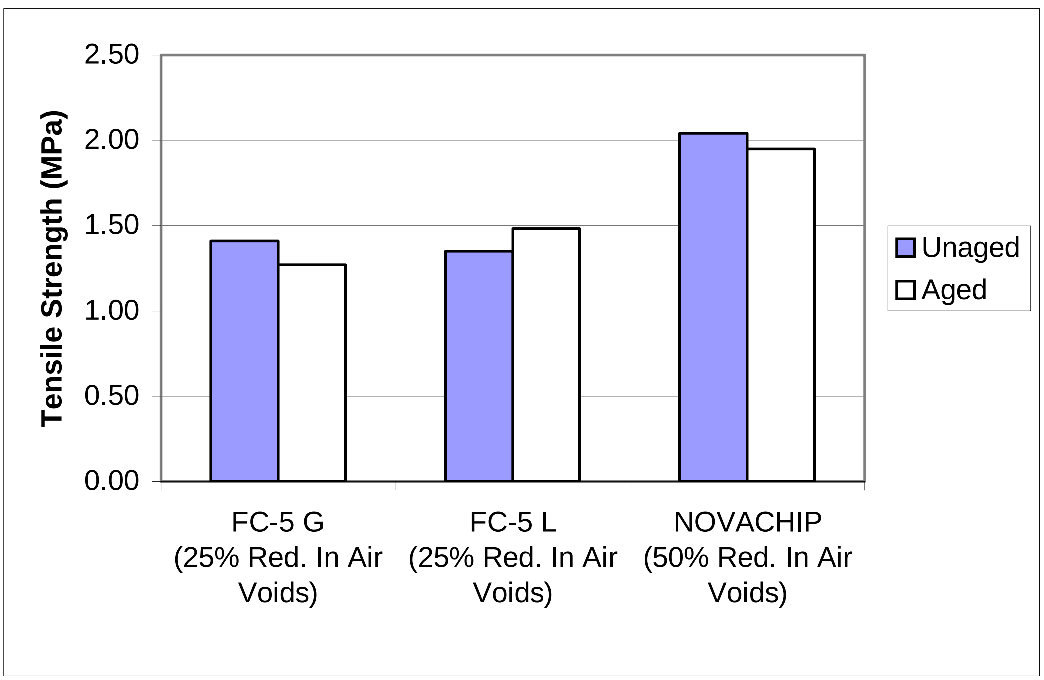


Figure 7-23. Tensile Strength for Saturated Mix.

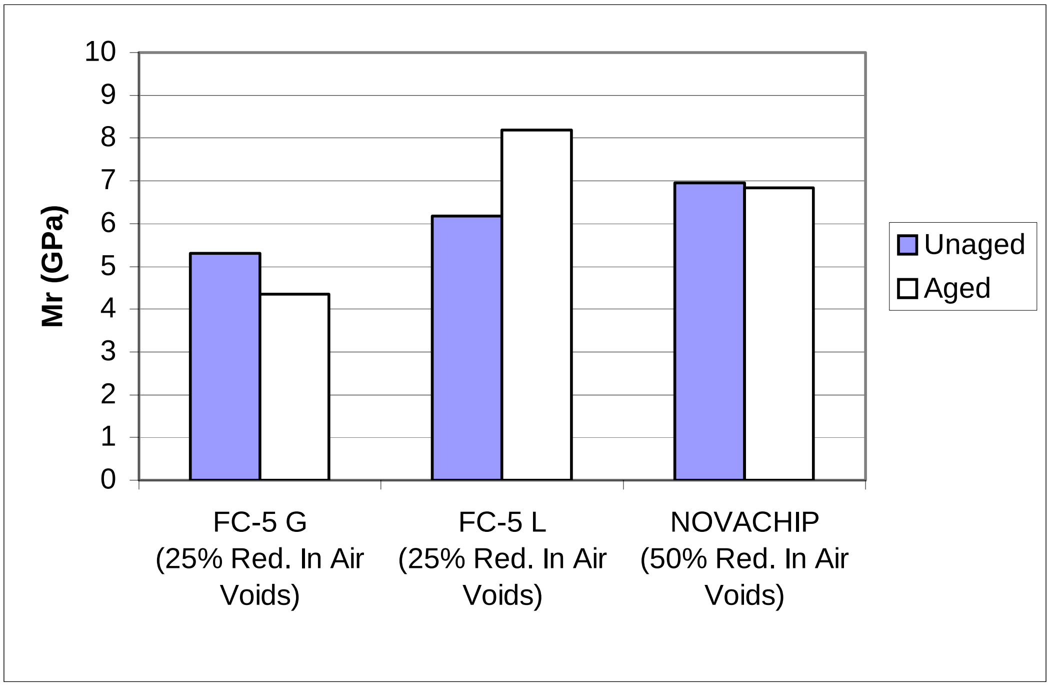


Figure 7-24. Resilient Modulus (M_r) for Saturated Mix.

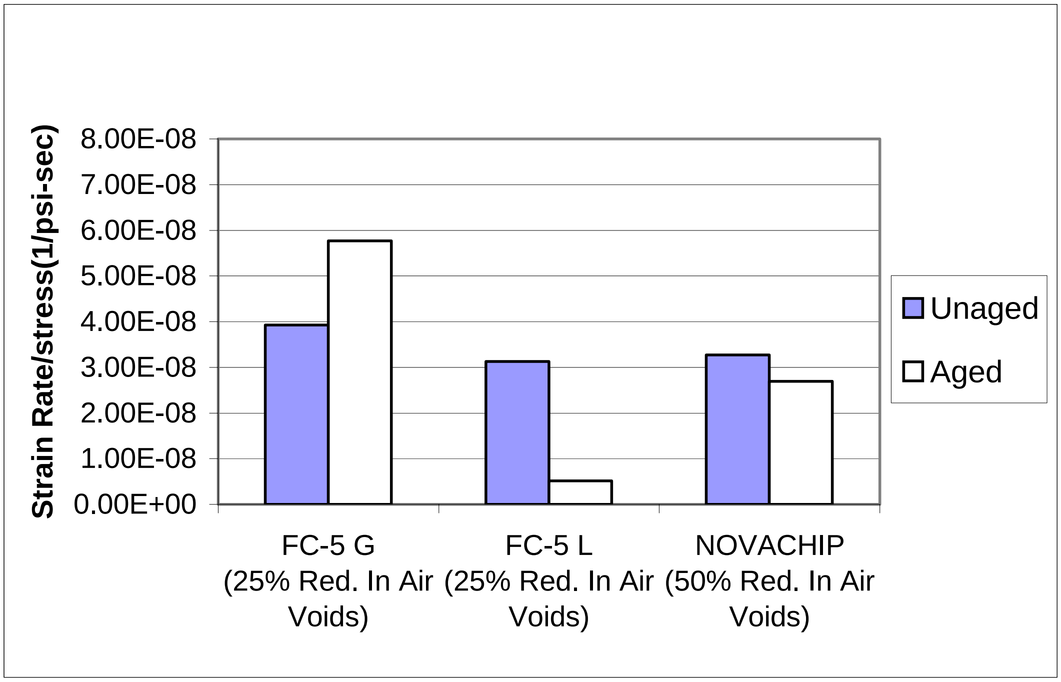


Figure 7-25. Creep Strain Rate for Saturated Mix.

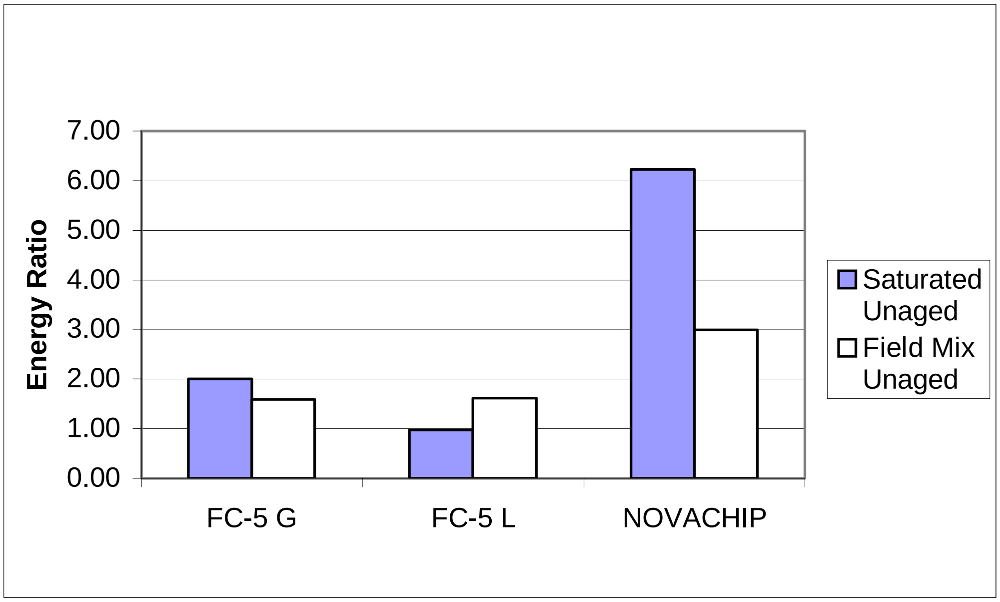


Figure 7-26. Comparison of Energy Ratio Between Saturated Unaged and Unaged Field Mix.

The ER for the FC-5 limestone saturated mixture however decreases as compared to the field mixture. This is primarily because of the high creep rate (around 3E-08/psi-sec) due to additional of asphalt. It should be noted that for the saturated mixtures, the resilient modulus decreases due to increased asphalt content.

7.5.4 Saturated Aged Mix

As in case of field mix, aging decreases ER for FC-5 granite (slightly above 1) and the Novachip mixtures. This is because of a significant decrease in the failure strain and hence in the DCSE limit. In the case of FC-5 limestone, the aged mix has a higher ER than the unaged, as was the case with the field mixture. However, the aged FC-5 limestone mixture has lower DCSE and failure strain than the unaged mixture. The reason for higher ER is because of high creep rate of the unaged saturated mixture. The creep rate of aged saturated FC-5 limestone mixture was the same as for the field mixture. Another interesting thing to note is that, the FE, DCSE and ER of the aged field FC-5 limestone mixture are much greater than for the saturated FC-5 limestone mixture.

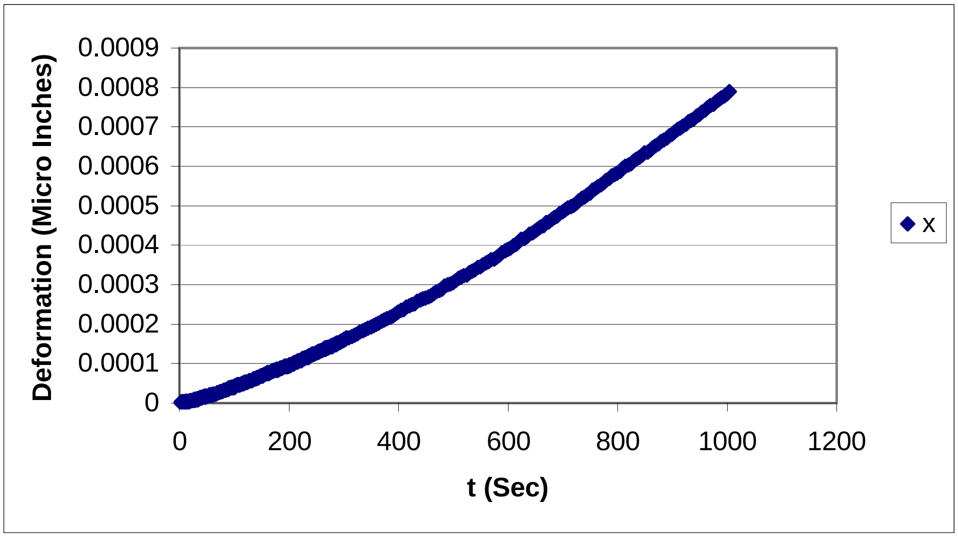
7.5.5 Moisture Conditioning

The FC-5 mixtures were subjected to moisture conditioning as outlined in Chapter 5. The AASHTO T283 (2003) moisture conditioning protocol was adopted and modified. The Superpave IDT test and associated fracture parameters were used to quantify the effects of moisture damage.

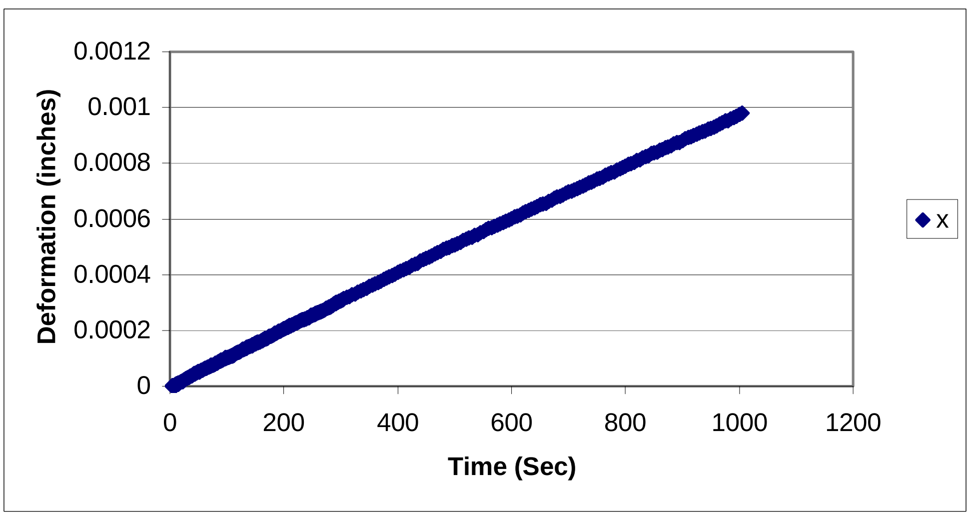
For each of the mixtures, the specimens were compacted to 50 gyrations. The specimens were then placed in a hot water bath at 60° C for 24 hours. After this, the samples were placed in a 25° C water bath for two hours. Then water was allowed to drain for 36 hrs before cutting the samples. As discussed previously in Chapter 5, the specimens were wrapped in a wire mesh during the moisture conditioning to minimize the possibility of specimen damage. Once the specimens were conditioned and drained, they were then tested in the Superpave IDT test. The results are presented below.

In the case of FC-5 granite, two of three specimens seemed to have failed during the creep test itself. The creep test results for the specimens are presented in Figures 7-27a and 7-27b. As evident from the figures, for a load as low as 4 pounds, the samples showed deformation close to 900 micro inches (Figure 7-27a). In fact at that low of a load level, there was no elastic response and the creep curve did not seem to follow the power model. Hence, the creep rate for all the specimens was calculated using the straight-line portion of the creep deformation curve and the results were compared to that of the unconditioned samples.

Clearly from Figure 7-28 it is evident that the creep rate for conditioned samples increased significantly and that the specimens failed during the creep test itself. Hence, the strength test results are not truly representative of the behavior of the mix. Further, the resilient modulus of the conditioned samples was lower as compared to unconditioned samples (close to 3.6 MPa). This clearly indicates that the samples had been damaged during conditioning. Thus FC-5 granite is highly sensitive to moisture conditioning.



(a)



(b)

Figure 7-27. Creep Deformation Curve for Conditioned Samples of FC-5 Granite Mixture.

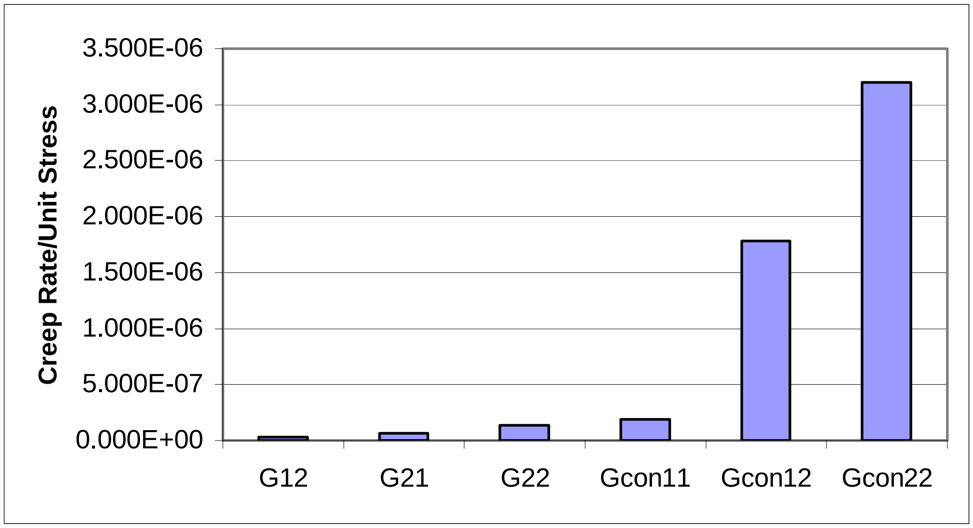


Figure 7-28. Creep Rate for Unconditioned and Conditioned Samples FC-5 Granite Mixture.

Moisture conditioning contributes to aging the mixture and hence an increase in Mr and tensile strength is observed for conditioned FC-5 limestone specimens. Further, due to increased absorption during conditioning, an increase in FE and ER is observed for conditioned samples. In addition, the creep rate decreases significantly for the conditioned specimens. The results are presented in Figures 7-29 through 7-34.

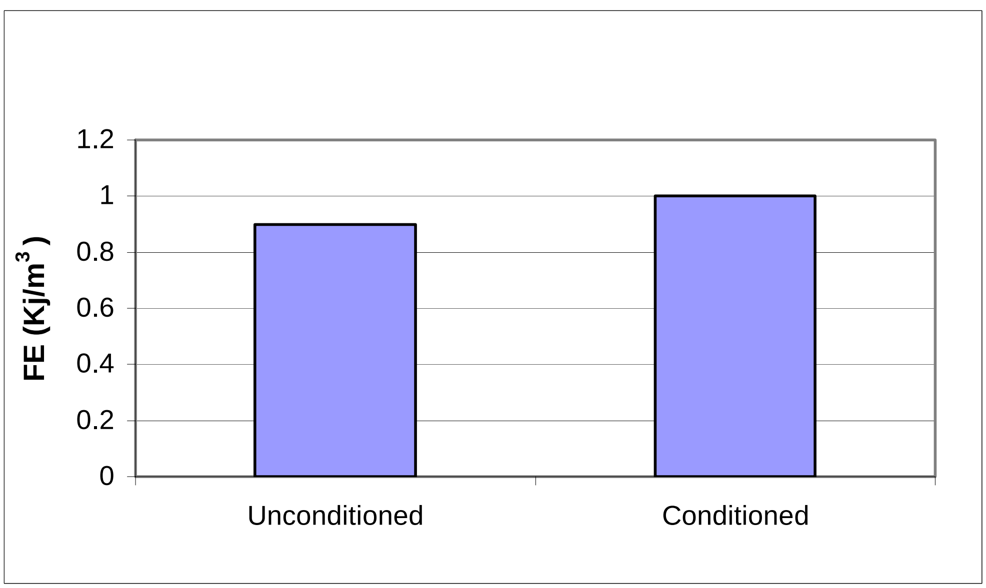


Figure 7-29. Fracture Energy (FE) After Conditioning for FC-5 Limestone Mixture.

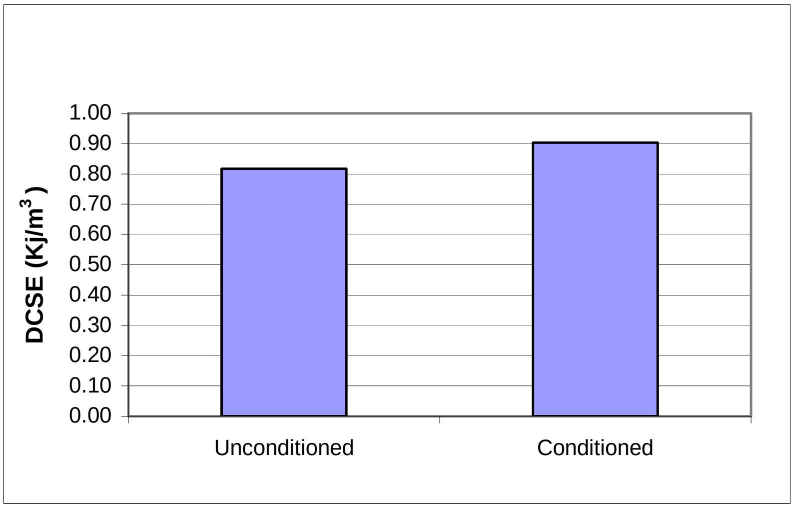


Figure 7-30. Dissipated Creep Strain Energy (DCSE) After Conditioning for FC-5 Limestone Mixture.

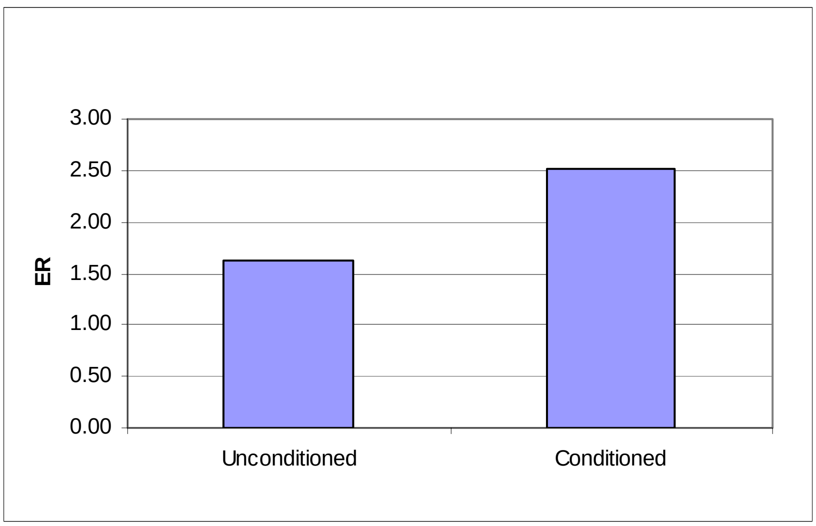


Figure 7-31. Energy Ratio (ER) After Conditioning for FC-5 Limestone Mixture.

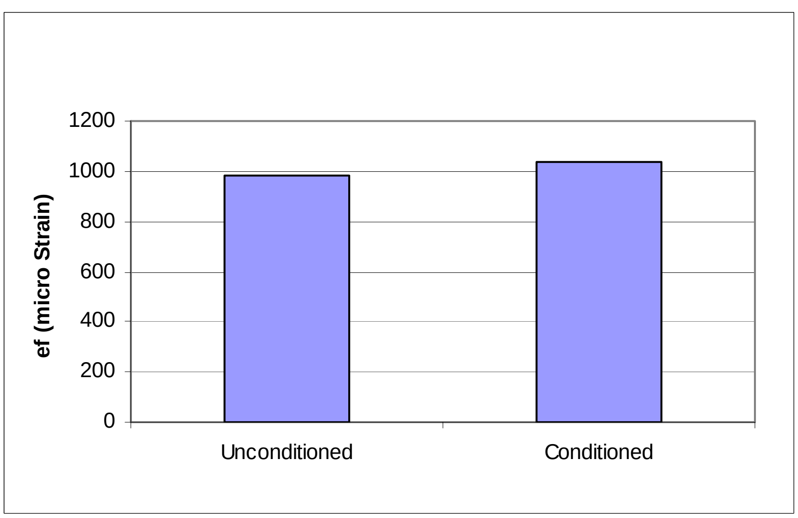


Figure 7-32. Failure Strain After Conditioning for FC-5 Limestone Mixture.

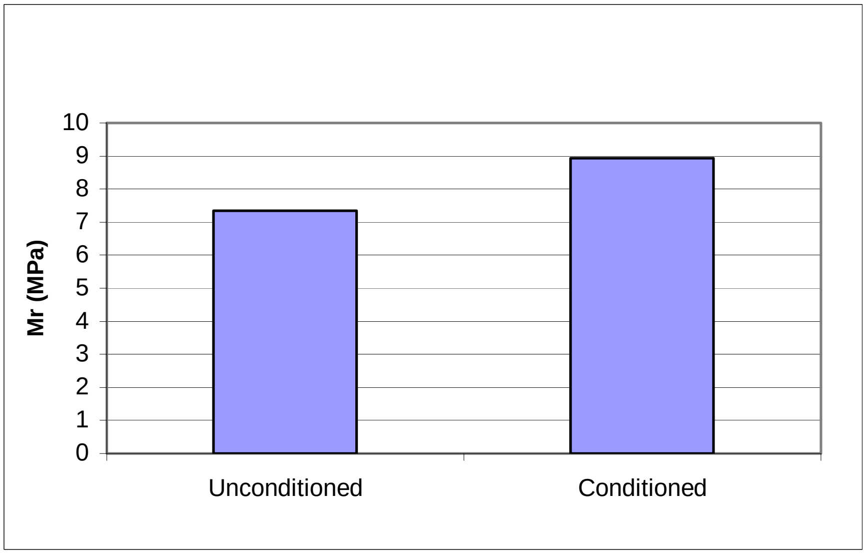


Figure 7-33. Resilient Modulus (M_r) After Conditioning for FC-5 Limestone Mixture.

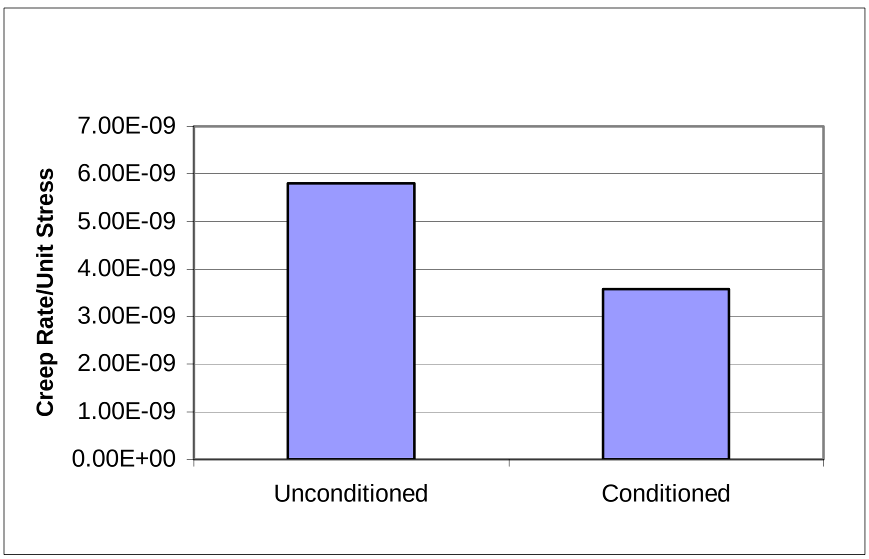


Figure 7-34. Creep Rate After Conditioning for FC-5 Limestone Mixture.

7.6 Summary of Findings

The key findings in this chapter can be summarized as follows:

- In the case of mixtures with additional SBS polymer modified PG 76-22, the mixing temperature was increased to 320º F and the compaction temperature was reduced to 255º F to avoid drain down. The additional asphalt added was such that the air voids were reduced by 50% for Novachip and 25 % for FC-5 limestone and granite.

- The presence of a polymer modified tack coat appears to improve the overall fracture resistance of the Novachip mixture, also slightly benefiting the FC-5 granite and limestone mixtures. In the case of the FC-5 limestone and FC-5 granite mixtures, the ER values may not truly represent the effect of the Novabond tack coat.
- In the case of the FC-5 limestone mixture, laboratory aging seems to increase its fracture resistance. This probably is because of increased absorption during aging, as a result enhancing the fracture properties. FC-5 granite seems to be highly sensitive to aging.
- Open graded and porous friction course mixtures made from absorptive limestone aggregate exhibited an increase in fracture resistance due to long term oven aging. However, further evaluation showed that the fracture resistance peaked at about 3-5 days of long term oven aging, after which it started decreasing again. This effect is likely due to the absorption of asphalt into the aggregates during long term oven aging, which is not representative of the field.
- Open graded friction course mixtures were tested for moisture damage susceptibility using a modified AASHTO T-283 conditioning method and the Superpave IDT test. These mixtures showed possible susceptibility to moisture damage. In particular, the tensile creep rate for the FC-5 granite mixture increases with moisture damage.

7.7 References

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CHAPTER 8
PERFORMANCE EVALUATION OF BONDED FRICTION COURSES

8.1 Introduction

It has been hypothesized that open graded friction courses provide many benefits to the highway user such as reduction in glare, noise, roughness, and increase in friction resistance, water-storage capacity and ride quality (Huber, 2000). However, these potential benefits have never been fully quantified and documented for Florida open graded friction course mixtures, let alone for sections constructed with a thick polymer modified tack coat.

Based on work performed by FDOT State Materials Office personnel and the literature review in Chapter 2, a construction specification for bonded friction courses was developed for Florida. This new draft specification is provided in Appendix B. The pavement test sections on US-27, Highlands County were constructed to evaluate the new specification for bonded friction course construction.

This chapter presents the performance of the bonded friction course pavement test sections on US-27, Highlands County. The test methods and devices currently used by FDOT to evaluate open graded friction courses are also described. The results presented in this chapter and the expected follow-up studies by FDOT will provide an improved understanding of the performance of Florida open graded friction course mixtures that are constructed with a thick polymer modified tack coat.

8.2 Evaluation of Surface Friction Characteristics

8.2.1 Introduction

The problems associated with pavement wear, such as aggregate polishing, low friction and wet weather accidents are well known. Nowadays, higher traffic volumes and speeds have made the consequences of low friction a problem of increasing concern (e.g., Huber, 2000).

The primary purpose of open graded friction course mixtures is to increase friction resistance in wet weather and reduce vehicular hydroplaning (Huber, 2000). Therefore, the friction characteristics of open graded friction courses are one of the main elements that need to be recognized in providing a safe highway. It is of sufficient importance that all practical measures need to be taken to ensure pavement surfaces are constructed and maintained with appropriate friction properties and that sections of pavement with questionable friction resistance properties be identified and corrected (e.g., Huber, 2000).

8.2.2 Surface Friction Testing Method used on US-27 Test Sections in Highlands County, Florida

Tests were conducted in accordance with "Standard Test Method for Friction Resistance of Paved Surfaces a Using Full-Scale Tire" (ASTM E 247-97).

Friction (skid) resistance is defined as the ability of the traveled surface to prevent the loss of tire traction and is generally quantified using some form of friction measurement such as a friction factor or friction number. The number most commonly used to report the results of a pavement friction test is called the friction number (FN) (ASTM E 247-97). The friction of most dry pavements is high. Wet pavements tend to be more problematic. Thus, the friction testing process involves an application of water to the pavement surface prior to the determination of a friction number.

The friction testing unit is composed of a truck with one or more test wheels incorporated into it or forming a part of a suitable trailer towed by a vehicle, water tank, and mobile data recorder, as shown in Figure 8-1. Measured values are taken representing the frictional force on a locked wheel as it is dragged over a wetted pavement surface under a constant load and constant speed. The friction number (FN) at 40 mph using a standard ribbed tire (FN_{40R}) (Holzschuher and Choubane, 2001) is calculated as:

$$FN_{40R} = (F/W) \times 100 \tag{8-1}$$

where F = horizontal force applied to the test tire at the tire-pavement contact patch,

W = dynamic vertical load on test wheel.

Figure 8-1. Friction Unit (Left) and Locked Wheel (Right).

To ensure measurement precision, calibration of the equipment is performed at regular intervals. Distance and speed calibrations are also conducted at regular intervals. In addition, each friction unit is calibrated every two years at the Texas Transportation Institute Field Test Center. The calibration reports received from the Field Test Center on the friction units used in this study showed excellent correlation with the Area Reference Friction Measurement System (Holzschuher and Choubane, 2001).

It is not correct to say a pavement has a certain friction factor because friction involves two bodies, the tires and the pavement, which are extremely variable due to pavement wetness, vehicle speed, temperature, tire wear, tire type, etc. Typical friction tests specify standard tires and environmental conditions to overcome this. In general, the frictional resistance of most dry pavements is relatively high, while wet pavements are lower. The number of accidents on wet pavements is twice as high as for dry pavements, but other factors such as visibility are involved in addition to frictional resistance.

8.2.3 Results of Friction Studies on Test Sections on US-27, Highlands County, Florida

Several friction measurements were performed on the test sections of the US-27 test sections. Measurements were conducted in the left wheel path of the lane tested. The normal testing frequency is three lockups per mile. Test results are presented in Table 8-1 and Figure 8-2.

8.2.4 Summary of Findings

With regard to the testing after 6 months of pavement service, the friction resistance of the test mixtures (Figure 8-2 or Table 8-1) is shown to vary from fair to good and acceptable for heavily traveled roads, as defined by Knoll (1999). However, the FC-5 limestone mixtures showed a slight decrease in frictional resistance as the aggregates became more polished. In

Table 8-1. Friction Number Test Results.

Friction Number (FN)			
Mixture Type	**Round #1**	**Round #2**	**Round #3**
	7/22/2003	**9/8/2003**	**12/1/2003**
FC-5, LS	40.9	39.5	39.0
FC-5, LS, NB	39.4	38.0	37.2
FC-5, GR	41.3	47.2	45.8
FC-5, GR, NB	42.9	47.9	47.2
Novachip, GR	48.9	51.8	50.6

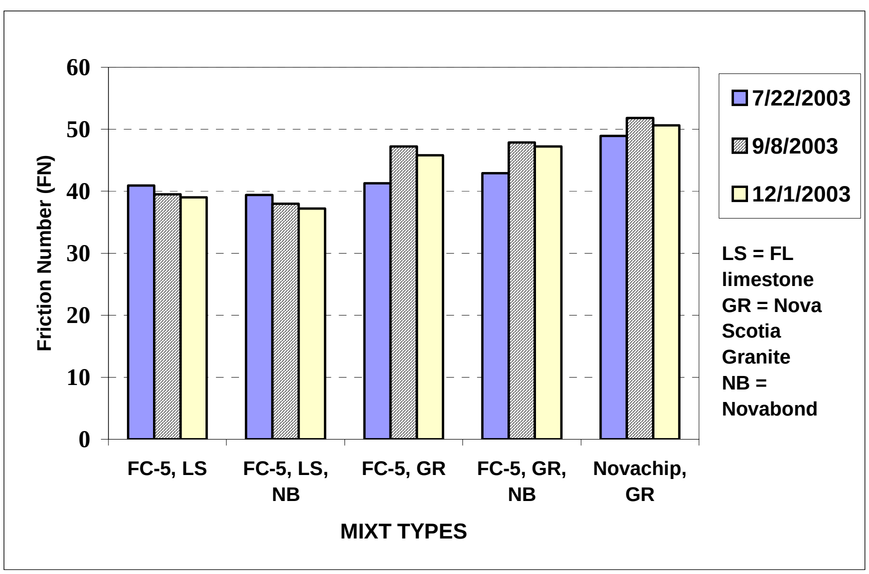

Figure 8-2. US-27 Friction Number Results.

contrast to the FC-5 limestone mixtures, the granite FC-5 and Novachip mixtures showed a slight increase in Friction Number (FN). The increase in friction can possibly be due to seasonal variations, which result in higher fall and winter values and lower spring and summer values (Jayawickrama et al. 1998). As expected, there was no discernable difference in friction numbers between the FC-5 sections with and without a polymer modified tack coat. Further follow-up friction testing is recommended to evaluate the long-term friction resistance of the test sections on US-27, Highlands County.

8.3 Roughness

8.3.1 Testing Methods and Equipment used to Characterize Pavement Roughness

A recent survey indicated that pavement smoothness is the most significant measure motorists use to judge the quality of roadways (Huber, 2000). Pavement smoothness directly relates to driver comfort as well as pavement life expectancy. In order to provide a measure of pavement surface condition that has nationwide consistency and comparability and is as realistic and practical as possible, a uniform, calibrated roughness measurement for paved roadways is required (Huber, 2000).

Roughness is defined as “The deviation of a surface from a true planar surface with characteristic dimensions that affect vehicle dynamics and ride quality” (ASTM E867). After a detailed study of various methodologies and road profiling statistics, the International Roughness Index (IRI) was chosen as the standard reference roughness index by the Federal Highway Administration (see Table 8-2) (e.g., Bertrand et al. 1990). The IRI was developed by the World Bank in the 1980s (e.g., Paterson, 1986). The IRI is used to define a characteristic of the longitudinal profile of a traveled wheel track and constitutes a standardized roughness measurement. It measures pavement roughness in terms of the number of inches per mile that a laser, mounted in a specialized van, jumps as it is driven along the pavement. The IRI is then converted to the Ride Number (RN) which is another method of rating pavement surface roughness based on IRI measurements (see Table 8-3). Smaller IRI or higher RN indicates smoother highways. If all of the elevation values in a measured profile are increased by some percentage, then the IRI increases by exactly the same percentage. An IRI of 0.0 means the profile is perfectly flat. There is no theoretical upper limit to roughness, although pavements with IRI values above 8 m/km (or 8 × 63.36 in/mile = 506.88 in/mile) are nearly impassable except at reduced speeds (Sayers, 1995).

Table 8-2. Federal Highway Association IRI Roughness Classification.

<table>
<tr><th>FHWA Highway Statistics Categories (inches per mile)</th><th>Interstate Routes</th><th>National Highway System (NHS) Non Interstate Routes</th><th>Non-NHS Traffic Routes & Other Routes with ADT ≥ 2000</th><th>All Other Routes</th></tr>
<tr><td>< 60</td><td>Excellent</td><td>Excellent</td><td>Excellent</td><td rowspan="2">Excellent</td></tr>
<tr><td>60 - 94</td><td>Good</td><td>Good</td><td rowspan="2">Good</td></tr>
<tr><td>95 - 119</td><td rowspan="3">Fair</td><td rowspan="3">Fair</td><td rowspan="2">Good</td></tr>
<tr><td>120 - 144</td><td rowspan="3">Fair</td></tr>
<tr><td>145 - 170</td><td rowspan="3">Fair</td></tr>
<tr><td>171 - 194</td><td rowspan="3">Poor</td><td rowspan="3">Poor</td></tr>
<tr><td>195 - 220</td><td rowspan="2">Poor</td></tr>
<tr><td>> 220</td><td>Poor</td></tr>
</table>

Table 8-3. FDOT Ride Number (RN) Rating.

Ride Number (RN)	Method of Acceptance FDOT
≥ = 4	Full Acceptance
3.7 to 3.9	Correct Deficiency
< 3.7	Remove and Replace

8.3.2 Testing Methods and Equipment Used to Characterize Pavement Roughness on the US-27 Test Sections in Highlands County

The advent of non-contact measurements using ultrasonic and laser devices has rendered the manual method obsolete in many Departments of Transportation State Highway agencies around the nation. FDOT has recently optimized the roadway profiler (Figure 8-3) to evaluate several pavement performance characteristics such as roughness and rutting profile (Holzschuher and Choubane, 2001). The roadway profiler consists of three laser sensors mounted in the front of a specially designed bumper of a full-sized van. Two of the sensors are mounted 69 inches apart and equidistant from the centerline to measure pavement profiles in the two wheel paths of the traveled surface. The third sensor is located exactly at the bumper centerline.

Figure 8-3. FDOT Roadway Laser Profiler.

The roadway profiler also uses two accelerometers mounted with the outward sensors to isolate vehicle motion. The vehicle is also equipped with a data acquisition system (Figure 8-4) to collect and store elevation profile data of the traveled surface. A distance-measuring instrument is provided to monitor the traveled distance.

The sampling rate is selected based on the ride parameter requirements for the intended data processing. The vehicle is driven along the wheel paths or in the designed position along the pavement section to be tested. While the vehicle is driven at highway speed, the sensors measure the vertical acceleration of the vehicle and the vertical distance between the accelerometer and the pavement surface as well as the traveled distance. These sensor signals are combined through a computerized process to generate the longitudinal profile of the traveled pavement surface.

Figure 8-4. Data Acquisition System.

The IRI and the RN are determined in accordance with ASTM E-1926 and ASTM E-1489, respectively. In addition, for the purpose of the present procedure, the entire raw ride data used in computing IRI and RN is filtered to a 300 ft wavelength.

8.3.3 Results of Pavement Roughness Studies on Test Sections on US-27, Highlands County, Florida

The FDOT collects raw IRI data measurements of pavement surface roughness on pavements in the State of Florida on an ongoing basis. The IRI results for the US-27 test sections are shown in Figure 8-5. The measurements are then translated from IRI measurements into RN, shown in Figure 8-6. Table 8-4 shows the combined IRI and RN results for the test sections on US-27. Data are collected on an annual, calendar year basis.

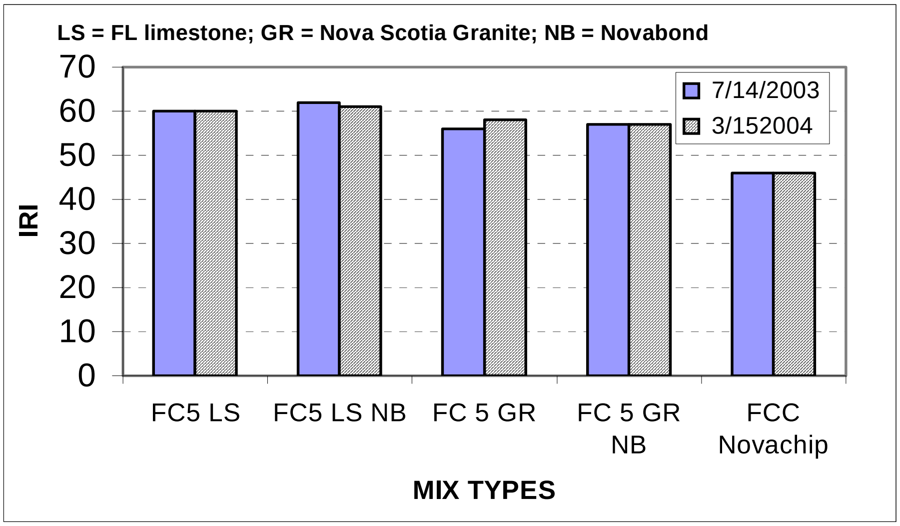

Figure 8-5. International Roughness Index.

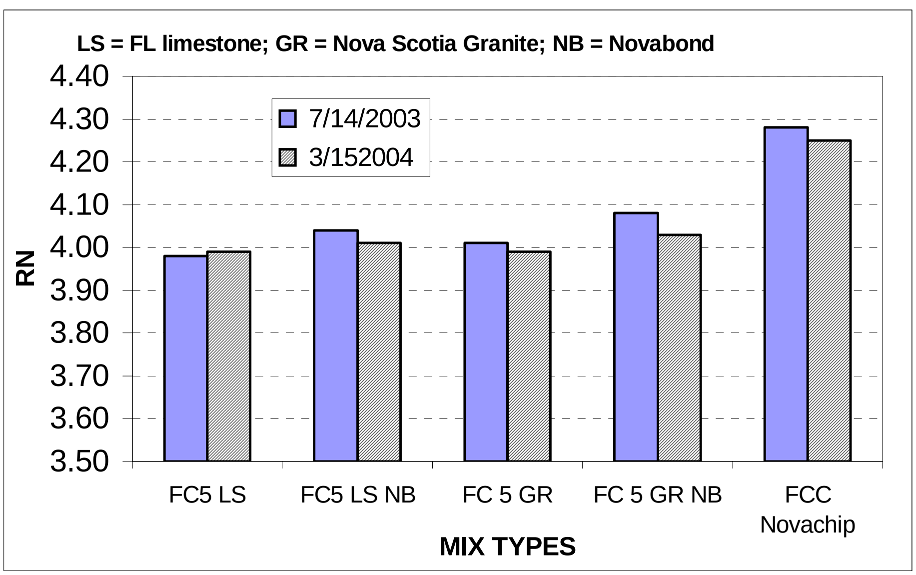

Figure 8-6. Ride Number.

Table 8-4. Roadway Laser Profiler Results.

Date Tested	Mile post	Section	Average (IRI)	Average Ride Number (RN)	Standard Deviation
7/14/2003	0.000 To 1.306	FC-5 LS	60	3.98	0.029
	1.306 To 2.612	FC-5 LS NB	62	4.04	0.035
	2.612 To 3.918	FC-5 GR	56	4.01	0.026
	3.918 To 5.224	FC-5 GR NB	57	4.08	0.025
	5.224 To 6.528	Novachip	46	4.28	0.026
3/15/2004	0.000 To 1.306	FC-5 LS	60	3.99	0.029
	1.306 To 2.612	FC-5 LS NB	61	4.01	0.037
	2.612 To 3.918	FC-5 GR	58	3.99	0.027
	3.918 To 5.224	FC-5 GR NB	57	4.03	0.026
	5.224 To 6.528	Novachip	46	4.25	0.028

8.3.4 Summary of Findings

According to the FHWA method of rating pavement smoothness using the IRI, all the mixtures are rated excellent and satisfy FDOT smoothness requirement. The RN and IRI seem to be comparable among the FC-5 mixtures, with no discernable differences between mixtures with and without a polymer modified tack coat. The Novachip mixture had an exceptionally low IRI value, i.e., the smoothest surface. This is likely due to the fact that the Novachip mixture has more fines than the FC-5 mixtures.

8.4 Field Measured Rutting

8.4.1 Introduction

Rutting of pavements can represent a major hazard to highway users as well as being an early indicator of pavement failure. Therefore, rut depth measurements are usually included in most pavement performance monitoring programs. The FDOT laser profiler is also used to measure transverse profile which in turn gives field rut depth. However, the interpretation of surface transverse profile measurements poses a major challenge in determining the contribution of the different layers to rutting.

8.4.2 Testing and Analysis Methods

8.4.2.1 Falling Weight Deflectometer

The Falling Weight Deflectometer (FWD), shown in Figure 8-7 is a non-destructive testing device that is used for evaluating the layer moduli of the pavement structure. The FWD is a device capable of applying dynamic loads to the pavement surface, similar in magnitude and duration to that of a single heavy moving wheel load. The response of the pavement system is measured in terms of vertical deformation, or deflection, over a given area using accelerometers.

Figure 8-7. Falling Weight Deflectometer.

The use of a FWD enables the FDOT to determine a deflection basin caused by a controlled load. The FWD generated data, combined with layer thickness, can be used to obtain the "in-situ" back-calculated elastic modulus of a pavement structure.

8.4.2.2 Transverse Profile Beam

A transverse profile beam (Figure 8-8) was used to collect data on the transverse road profiles on the pavement test sections on US-27, Highlands County. This instrument consists of a 3.6 m long beam which is fixed at either end. The beam is leveled to give a horizontal datum. A linear vertical displacement transducer is connected to a wheel below the beam. As the wheel is moved along the ground the transducer is moved relative to the beam and the vertical displacement of the wheel is recorded. The data are displayed to an accuracy of 0.1 mm. The beam was then used to record the transverse profile. A tape measure was placed on the pavement surface, and zeroed at the left edge of the pavement. The beam wheel was then moved across to the right hand edge with the values being recorded on data collection form.

Before the beam was operated a calibration test was conducted per the manufacturer's recommendations. This entailed setting up the instrument on a level floor and placing a calibration stand under the wheel. The heights from the Transverse Profiler Beam were verified to be

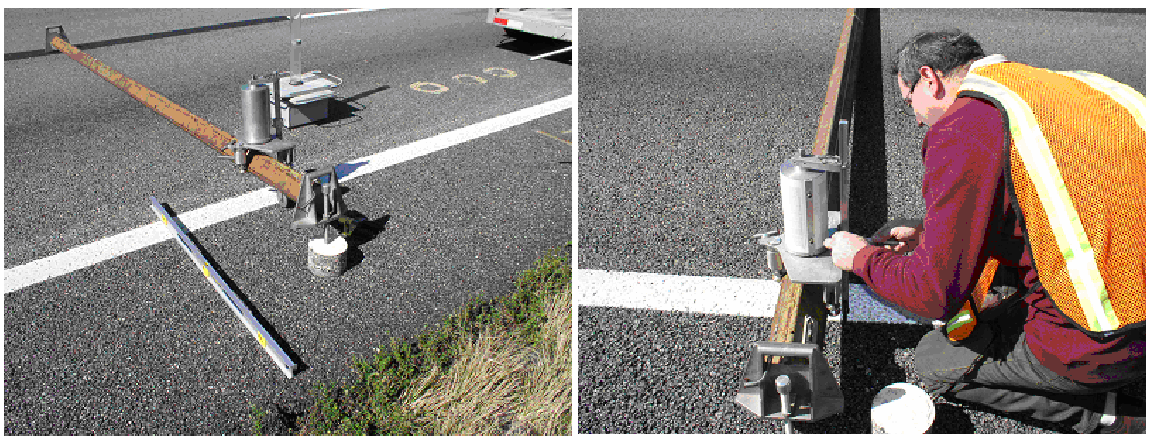

Figure 8-8. Transverse Profile Beam.

the same as those corresponding to the calibration stand. The transverse profile was recorded at 15 locations (3 locations per section) on the US-27 test sections.

8.4.2.3 Modulus Back-Calculation

Modulus Back-calculation is the accepted term used to describe a process whereby the elastic (Young's) moduli of individual pavement layers are estimated based upon measured FWD surface deflection results. As there are no closed-form solutions to accomplish this task, a mathematical model of the pavement system (called a forward model) is constructed and used to compute theoretical surface deflections with assumed initial layer moduli values at the appropriate FWD loads.

Through a series of iterations, the layer moduli are changed, and the calculated deflections are then compared to the measured deflections until a match is obtained within tolerance limits. The BISDEF program, which utilizes an Elastic Layer Program as the forward model, was used in this research to back-calculate the moduli based on surface deflection data. The US-27 pavement structure consists of 4 layers with a description and thickness as follows:

Layer 1: Friction layer (0.8 inches)

Layer 2: Structural layer (4.5 inches)

Layer 3: Lime rock base (12 inches)

Layer 4: Stabilized sub-grade (12 inches)

For convenience of analysis, the friction layer and the structural layer were combined into one layer with the thickness of 5.3 inches.

8.4.3 Results and Findings

Raw data from the transverse profilograph beam were analyzed based on methods developed by (Villiers et al. 2004) and summarized in Table 8-5. The average moduli of the pavement layers based on the back calculation process are presented in Table 8-6.

Table 8-5. Field Rut Result.

Date	Test	FC-5 LS (mm)	FC-5 LS NB (mm)	FC-5 GR (mm)	FC-5 GR NB (mm)	Novachip (mm)
7/14/2003	Roadway Profiler	0.0	2.5	0.0	2.0	2.0
3/15/2004	Roadway Profiler	0.0	2.5	0.0	2.5	2.3
1/18/2005	Transverse Profilograph	2.0	1.2	0.6	0.5	0.3

Table 8-6. US-27 Test Section Layer Moduli.

Base Layer Modulus (psi)					
Test section	**Limestone FC-5**	**Limestone FC-5 NB**	**Granite FC-5**	**Granite FC-5 NB**	**Novachip**
Under wheel path	23,800	24,600	18,500	19,000	30,700
Between wheel path	21,900	20,500	16,600	21,400	30,200
Asphalt Layer Modulus (psi)					
Test section	**Limestone FC-5**	**Limestone FC-5 NB**	**Granite FC-5**	**Granite FC-5 NB**	**Novachip**
Under wheel path	790,000	824,000	711,200	710,000	879,000
Between wheel path	653,000	560,700	560,000	606,700	700,000

In general, all the test sections show negligible rutting. More importantly, the back-calculated modulus results (Table 8-6) show that the test sections have a marginally competent base (low modulus), which may possibly lead to premature top-down cracking due to the high bending stresses in an asphalt layer overlying a soft base.

8.5 Permeability and Drainage

8.5.1 Background and Testing Methods

The surface texture including microtexture and macrotexture of an asphalt pavement plays a critical role in the prevention of hydroplaning on high-speed, multi-lane facilities (Corley-Lay, 1998). Microtexture refers to the small-scale texture of the pavement aggregate component (which controls contact between the tire rubber and the pavement surface) while macrotexture refers to the large-scale texture of the pavement as a whole due to the aggregate particle arrangement (which controls the escape of water from under the tire and hence the loss of friction resistance with increased speed).

Open graded friction courses (OGFC) with their high degree of porosity offer significantly better drainage behavior than normal pavement materials. While standard pavements normally allow runoff only along the road surface, highly porous asphalt mixtures allow for both horizontal and vertical (through pavement) drainage. Such drainage characteristics are very appealing and may greatly increase driver safety by removing water more rapidly from the driving surface thereby creating a dry driving surface even in moderate rainstorms (e.g., Huber, 2000).

Permeability or hydraulic conductivity is a measure of the material's ability to spatially transfer pore fluid. It is related to the material's porosity, but it is most dependent on the void geometry such as the interconnectivity of the void pathways (Hall, 2001). Flow through porous media is normally governed by Darcy's Law, but in some cases a non-Darcy theory is a better predictor of the fluid transport.

With regards to asphalt materials, the mixture characteristics will determine which theory would be the most appropriate. For laminar flow with Reynolds number ranging from 1 to 10, Darcy's law is generally valid, and is given by:

$$v = ki \tag{8-2}$$

where k = coefficient of permeability of material
i = hydraulic gradient across the sample or head loss per unit length; and
v = equivalent average velocity through sample.

When Reynolds number is greater than 10, Darcy's law (Eqn, 8-2) is invalid and the flow behavior is commonly characterized by a "modified" Darcy's law, in which the flow velocity is now proportional to a power of the hydraulic gradient (Isenring, 1990):

$$v = ki^m \tag{8-3}$$

in which m = a power parameter that accounts for the nonlinearity between the hydraulic gradient and the flow velocity. Open graded friction courses are typically at the borderline of the validity of Darcy's law (Egn. 8-2) due to their high air voids, with the fluid flow characteristics of the more porous friction courses, such as porous friction courses, being controlled by Eqn. 8-3 (e.g., Mohammad et al. 2000).

The FDOT falling head permeameter (Figure 8-9), which is still being developed, was used to analyze the field drainage characteristics of the pavement test sections. This apparatus is basically a standpipe attached to a squared flat plastic plate placed on a rubber seal with a specified thickness to prevent water running off on the surface of the pavement. A standard weight is applied on the plastic plate to create pressure on the seal. The procedure for this test is quite simple. Two technicians are required for this test. The timer is started as soon as water is poured into the pipe up to the upper mark (Figure 8-10). Then the timer is stopped when the water level descends to the lower mark on the pipe. The flow quantity is calculated based on Darcy's law (Eqn. 8-2) for falling head flow conditions as:

$$k = \frac{a \times L}{A \times t} \times \ln \frac{h_1}{h_2} \tag{8-4}$$

Figure 8-9. Field Falling Head Permeability Test Device.

Figure 8-10. Timer Recording in Field Permeability Test.

By substituting in known values of a, A, h_1, and h_2, Eqn 8-4 can be written as:

$$k = 0.693 \times \frac{L}{t} \tag{8-5}$$

where k = flow quantity (cm/s)

a = inside cross area of standpipe

L = thickness of the pavement layer

A = cross-sectional area of which water can penetrate the layer

h_1 = initial head

h_2 = final head; and

t = elapsed time to flow water between h_1 and h_2.

To assess the permeability of the pavement sections with regards to the effect of the saturated modified tack coat, field permeability tests were performed. In addition, falling head permeability testing was performed on field cores with the thick polymer modified tack coat to verify that the interface between the friction course and the underlying structural layer was impervious. The FDOT laboratory permeameter, shown in Figure 8-11, was used for the laboratory study (Choubane et al. 1998; 2000).

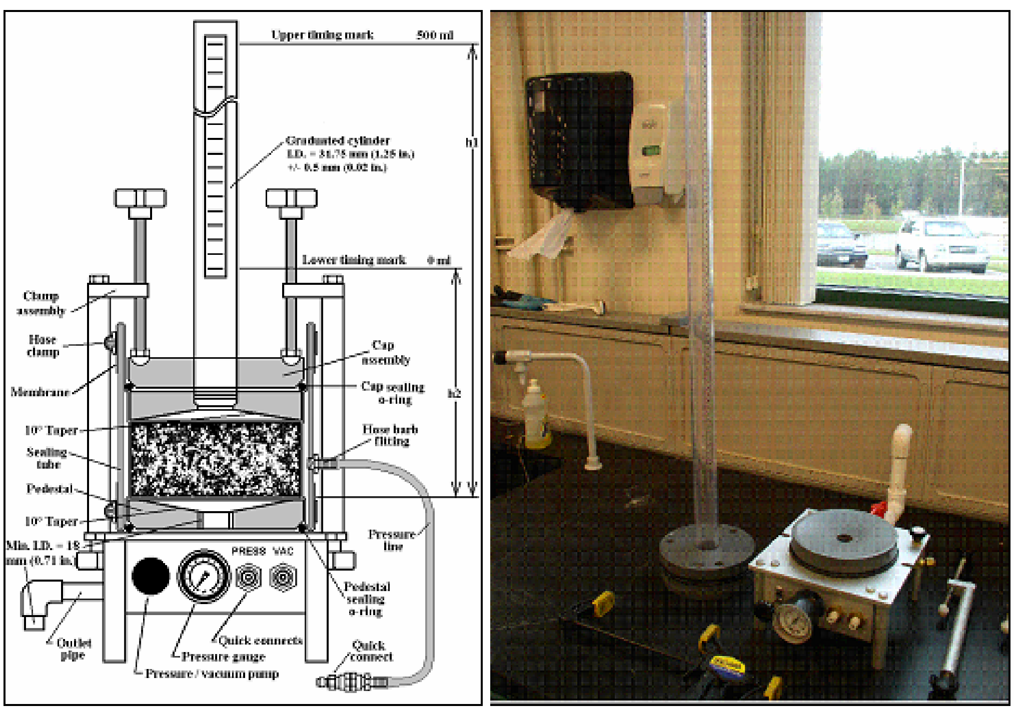

Figure 8-11. FDOT Lab Permeameter.

8.5.2 Results

As expected, the laboratory testing of field cores with a thick polymer modified tack coat showed virtually no permeability, as did the mixtures with normal tack coat. The vertical permeability is governed by the pavement layer with the lowest permeability, which is either the modified tack coat or the underlying structural layer in the case of a normal tack coat. This means that virtually all of the flow in open graded friction courses is confined to the friction course layer, with very little water penetrating into the structural layer.

The field flow quantity results are presented in Figure 8-12. These flow quantities in the field tests are basically from the friction course layers themselves. The polymer modified Novabond tack coat seems to produce significant loss in permeability (more than 50%), which is likely due to the reduction in storage volume, rather than actual reduction in permeability.

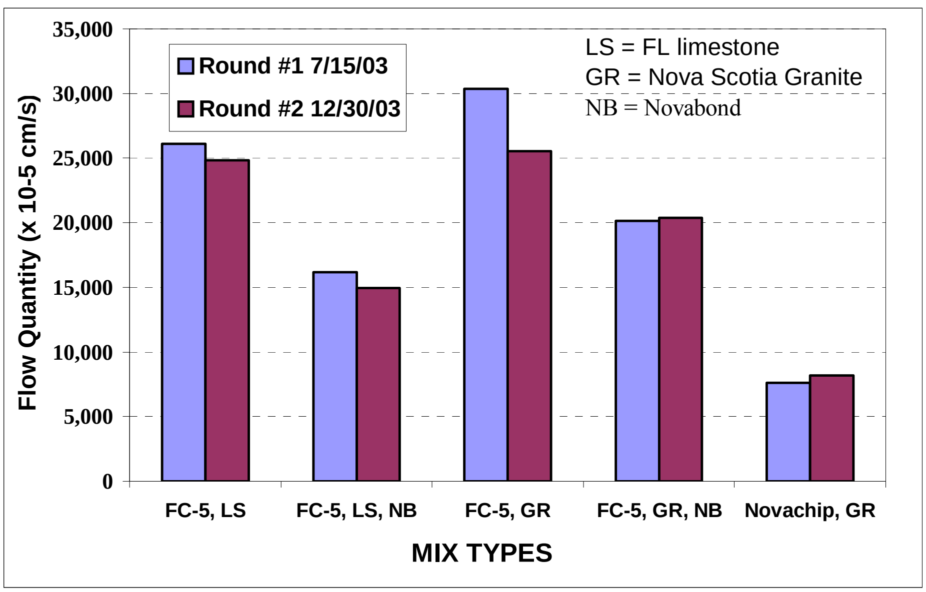

Figure 8-12. Field Permeability Result.

8.6 Pavement Noise Evaluation

8.6.1 Introduction

Traffic noise is becoming an issue of increased concern. In this context, noise is the generation of sounds that are unwanted and affect the people living near roadways. It has been widely observed that OGFC may have tire noise reducing properties (e.g., Huber, 2000). When tires roll over regular asphalt, air within the tread pockets is first compressed between tire and pavement and then, as the tire rolls on, suddenly released, creating a pulse of broadband noise in a manner analogous to a suction cup. However, when tire rolls over an OGFC surface, rather than being compressed, air within the tire treads is able to escape laterally through the interconnecting pores. This translates to a substantial reduction in noise generated by tire-pavement interaction, particularly at mid and high frequencies (Wayson, 1998).

As a secondary effect, tire noise, after being radiated from the tire-pavement interaction zone, is partially absorbed upon encountering the porous OGFC pavement, thereby somewhat reducing the noise reaching the roadside. At highway speeds, tire noise dominates overall traffic noise output, and OGFC can have a significant positive effect on overall traffic noise exposure at locations adjacent to highways (e.g., Huber, 2000).

Instead of a linear scale, a logarithmic scale is used to represent sound levels and the unit is called a decibel (or dB). The A-scale is used to describe noise. The term dB(A) is used when referring to the A-scale. The curve that describes the A-scale roughly corresponds to the response of the human ear to sound.

The decibel scale ranges from 0 dB(A), the threshold of human hearing, to 140 dB(A) where serious hearing damage can occur. Table 8-7 shows this scale and some of the levels associated with various daily activities.

A serene farm setting might have a decibel level of 30 dB(A) while a peaceful subdivision might be at 40 to 50 dB(A). Alongside a freeway the sound level (i.e., noise) might be in the range of 70 to 80 dB(A). The transition from a peaceful environment to a noisy environment is around 50 to 70 dB(A). Sustained exposure to noise levels in excess of 65 dB(A) can have negative health effects. As a general rule of thumb, one can only differentiate between two sound levels that are at least 3 dB(A) different in loudness.

Table 8-7. Noise Levels Associated With Common Activities (Wayson, 1998).

Activities	**Noise Level dB(A)**
Lawnmower	95
Loud Shout	90
Motorcycle passing 50 feet away	85
Blender at 3 feet	85
Car traveling 60 mph passing 50 feet away	80
Normal conversation	60
Quiet Living room	40

In addition to sound level, people hear over a range of frequencies (and this is the reason for the A weighting described earlier). A person with good hearing can typically hear frequencies between 20 Hz and 20,000 Hz. An older person, however, may not be able to hear frequencies above 5,000 Hz. So this indicates, to some extent, some of the reasons why different people hear things somewhat differently.

For the study of roadway noise on the US-27 test sections, the close proximity method CPX (ISO Standard 11819-2, 1997), was used by NCAT to measure noise level. The NCAT noise device, shown in Figure 8-13, consists of a CPX trailer and a haul truck. This device measures near-field tire-pavement noise, and thus measures the sound levels at or near the tire-pavement interface. This method consists of placing microphones near the tire-pavement interface to directly measure tire-pavement noise levels. The mounting of the microphones and the acoustical chamber are shown in Figure 8-14. The near-field test procedures offer the following advantages:

- The ability to determine the noise characteristics of the road surface at almost any arbitrary site.

Figure 8-13. NCAT Close Proximity Trailer.

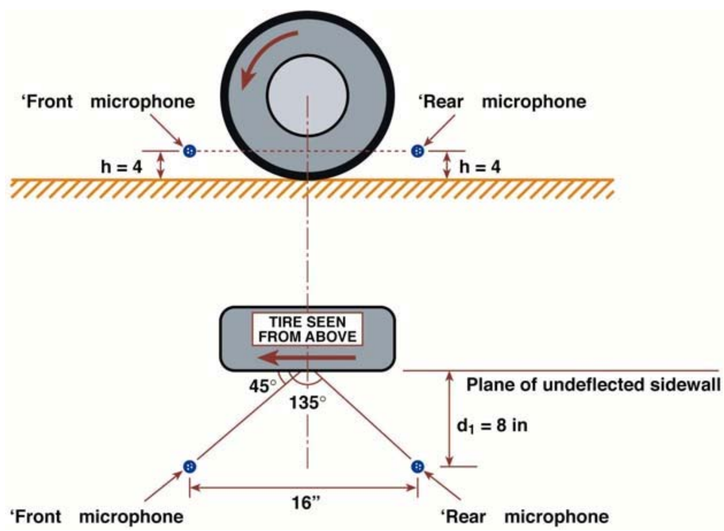

Figure 8-14. Microphone Locations in NCAT CPX Trailer.

- Can be used for checking compliance with a noise specification for a surface.
- Can be used to check the state of maintenance (i.e., the wear or damage to the surface) as well as clogging and the effect of cleaning porous surfaces.
- It is more portable than the pass-by methods, requiring only a quick setup prior to use.

8.6.2 Results

The noise level results conducted by NCAT are shown in Table 8-8 and Figure 8-15. There is no difference in the noise level among the mixtures as one can only differentiate between two sound levels that are at least 3 dB(A) different in loudness. Hence, the introduction of a thick polymer modified tack coat (Novabond) does not appear to change the noise characteristics of the underlying friction course mixtures.

Table 8-8. Summary of Noise Level Result.

Section	Noise Level (dB)
FC-5 LS	99.1
FC-5 LS w/ NB	98.9
FC-5 Granite	98.6
FC-5 Granite w/NB	99.3
Novachip	97.2

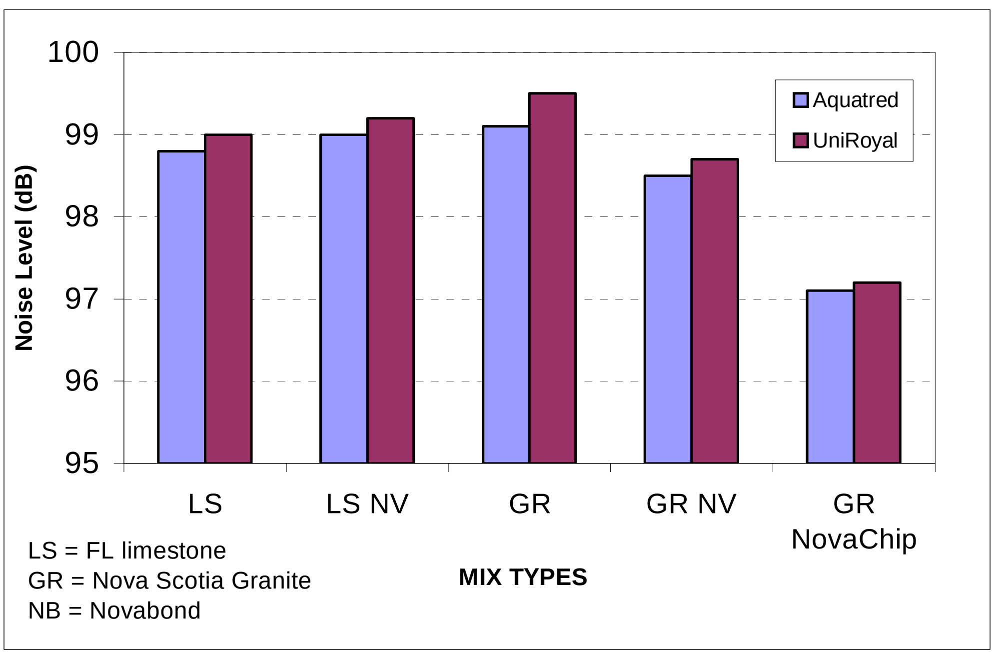

Figure 8-15. Noise Level Comparison.

8.7 Summary of Findings

- The results of the performance evaluation of the field test sections on US-27 in Highlands County, Florida, showed:
  - Acceptable friction resistance for all five test sections. However, it is still too early to evaluate the relative long-term friction performance of the different test sections.
  - All four FC-5 mixtures as well as the Novachip mixture had excellent smoothness. There were no observable differences between granite and limestone FC-5 mixtures, as well as FC-5 mixtures with and without a thick modified tack coat. The Novachip mixture showed an exceptionally low IRI value (46) and a high ride number (greater than 4).
  - Rutting measured with a roadway profiler and a transverse profilograph was negligible for all test sections. However, back-calculated base course layer moduli for all FC-5 test sections were on the low side (less than 25 ksi). This is a possible concern for the long-term cracking performance of these sections, due to the increase in induced bending stresses caused by the soft underlying base layer.
  - All FC-5 sections showed excellent field permeability during the first six months after construction. However, the presence of a thick modified tack coat in FC-5 mixtures resulted in a slightly decreased permeability.
  - All mixtures showed a similar pavement noise level (~97-99 dB), which corresponds to that of a lawn mower. The presence of a thick modified tack coat does not appear to decrease noise level.

In summary, it is important to continue to monitor the long term field performance of the bonded friction course study on US-27, Highlands County.

8.8 References

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Appendix A

DEVELOPMENTAL SPECIFICATION

FOR ASPHALT CONCRETE FRICTION COURSES INCLUDING

POROUS FRICTION COURSE SECTIONS

(REV6-22-04)

SECTION 334 (Pages 266-278) is deleted and the following substituted:

SECTION 337
ASPHALT CONCRETE FRICTION COURSES

337-1 Description.

Construct an asphalt concrete friction course pavement with the type of mixture specified in the Contract, or when offered as alternates, as selected. This Section specifies mixes designated as FC-5 and porous friction course (PFC).

Meet the plant and equipment requirements of Section 320, as modified herein. Meet the general construction requirements of Section 330, as modified herein.

337-2 Materials.

337-2.1 General Requirements: Meet the requirements specified in Division III as modified herein. The Engineer will base continuing approval of material sources on field performance.

337-2.2 Asphalt Binder: Meet the requirements of Section 336, and any additional requirements or modifications specified herein for the various mixtures. When called for in the Contract Documents, use a PG 76-22 asphalt binder meeting the requirements of 916-1.

337-2.3 Coarse Aggregate: Meet the requirements of Section 901, and any additional requirements or modifications specified herein for the various mixtures.

337-2.4 Fine Aggregate: Meet the requirements of Section 902, and any additional requirements or modifications specified herein for the various mixtures.

337-2.5 Hydrated Lime: Meet the requirements of AASHTO M303 Type 1.

Provide certified test results for each shipment of hydrated lime indicating compliance with the specifications.

337-2.6 Fiber Stabilizing Additive: Use either a mineral or cellulose fiber stabilizing additive. Meet the following requirements:

337-2.6.1 Mineral Fibers: Use mineral fibers (made from virgin basalt, diabase, or slag) treated with a cationic sizing agent to enhance the disbursement of the fiber, as well as to increase adhesion of the fiber surface to the bitumen. Meet the following requirements for physical properties:

1. Size Analysis
   Average fiber length ....0.25 inch [6.0 mm] (maximum)
   Average fiber thickness...0.0002 inch [0.005 mm] (maximum)
2. Shot Content (ASTM C612)
   Percent passing No. 60 [250 μm] Sieve...............90 - 100
   Percent passing No.230 [63 μm] Sieve................65 - 100

Provide certified test results for each batch of fiber material indicating compliance with the above tests.

337-2.6.1.1 Notice of Patented Process: Take notice that the use of mineral fibers treated with cationic sizing agent and the size analysis range for average fiber thickness are subject to U.S. Patent No. 4,613,376, held by Fiberand Corporation, 7150 Southwest 62nd Avenue, South Miami, FL 33143. Obtain all mineral fibers required to meet the FC-5 requirements of this Contract only from Fiberand Corporation or a duly authorized licensee of Fiberand. Assume responsibility, pursuant to 7-3, for obtaining any and all necessary rights to use such processes and pay any and all royalties, license fees or other costs incurred in order to meet the FC-5 requirements of this Contract. Include any and all royalties, license fees and other costs arising due to the existence of U.S. Patent No. 4,613,376 in the bid unit price for friction course FC-5.

337-2.6.2 Cellulose Fibers: Use cellulose fibers meeting the following requirements:

1. Fiber length ....0.25 inch [6.0 mm] (maximum)
2. Sieve Analysis
    a. Alpine Sieve Method
        Percent passing No. 100 [150 μm] sieve 60-80
    b. Ro-Tap Sieve Method
        Percent passing No. 20 [850 μm] sieve 80-95
        Percent passing No. 40 [425 μm] sieve 45-85
        Percent passing No. 100 [150 μm] sieve 5-40
3. Ash Content: .... 18% non-volatiles (±5%)
4. pH:..7.5 (±1.0)
5. Oil Absorption: ....5.0 (±1.0) (times fiber weight)
6. Moisture Content: ....5.0 (maximum)

Provide certified test results for each batch of fiber material indicating compliance with the above tests.

337-3 General Composition of Mixes.

337-3.1 General: This project consists of a conventional FC-5 asphalt mixture and one test section (as described in the plans) of a porous friction course (PFC) mixture. Use a bituminous mixture composed of aggregate (coarse, fine, or a mixture thereof), asphalt rubber binder, and in some cases, fibers and/or hydrated lime. Size, uniformly grade and combine the aggregate fractions in such proportions that the resulting mix meets the requirements of this Section. The use of RAP material will not be permitted.

337-3.2 Specific Component Requirements for FC-5 and PFC:

337-3.2.1 Aggregates: Use an aggregate blend which consists of either 100% crushed granite or 100% crushed Oolitic limestone.

In addition to the requirements of Section 901, meet the following coarse aggregate requirements. Use either crushed granite or crushed limestone. Use crushed limestone from the Oolitic formation, which contains a minimum of 12% non-carbonate material (as determined by FM 5-510), and has been approved for this use.

In addition to the requirements of Section 902, meet the following fine aggregate requirements. Use either crushed granite screenings, or crushed Oolitic limestone screenings for the fine aggregate.

337-3.2.2 Asphalt Binder: Use an ARB-12 asphalt rubber binder for the FC-5 mixture. Use a PG 76-22 asphalt binder for the PFC mixture.

337-3.2.3 Hydrated Lime: Add the lime at a dosage rate of 1.0% by weight of the total dry aggregate to mixes containing granite.

337-3.2.4 Fiber Stabilizing Additive: Add either mineral fibers at a dosage rate of 0.4% by weight of the total mix, or cellulose fibers at a dosage rate of 0.3% by weight of total mix.

337-3.3 Grading Requirements:

337-3.3.1 FC-5: Use a mixture having a gradation at design within the ranges shown in Table 337-1.

Table 337-1 FC-5 Gradation Design Range									
¾ inch [19.0 mm]	½ inch [12.50 mm]	3/8 inch [9.50 mm]	No. 4 [4.75 mm]	No. 8 [2.36 mm]	No. 16 [1.18 mm]	No. 30 [600 :m]	No. 50 [300 :m]	No. 100 [150 :m]	No. 200 [75 :m]
100	85-100	55-75	15-25	5-10	--	--	--	--	2-4

337-3.3.2 PFC: Use a mixture having a gradation at design within the ranges shown in Table 337-2.

Table 337-2 PFC Gradation Design Range									
¾ inch [19.0 mm]	½ inch [12.50 mm]	3/8 inch [9.50 mm]	No. 4 [4.75 mm]	No. 8 [2.36 mm]	No. 16 [1.18 mm]	No. 30 [600 :m]	No. 50 [300 :m]	No. 100 [150 :m]	No. 200 [75 :m]
100	85-95	55-65	15-25	5-10	--	--	--	--	1-4

337-4 Mix Design.

337-4.1 FC-5: The Department will design the FC-5 mixture. Furnish the materials and all appropriate information (source, gradation, etc.) as specified in 334-3.2.5. The Department will have two weeks to design the mix.

The FC-5 mix design will use an ARB-12 asphalt binder. The Department will establish the design binder content for FC-5 within the following ranges based on aggregate type:

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

337-4.2 PFC: The Department will design the PFC mixture using the Superpave gyratory compactor using the following criteria: 1) the design number of gyrations is 50, 2) the asphalt binder content will be selected at the minimum voids in the mineral aggregate (VMA) content, 3) the air void content will be between 18 and 22 percent. The PFC mix design will use a PG 76-22 asphalt binder.

337-4.3 Revision of Mix Design: For FC-5 and PFC, meet the requirements of 334-3.3. For FC-5, all revisions must fall within the gradation limits defined in Table 337-1. For PFC, all revisions must fall within the gradation limits defined in Table 337-2.

337-5 Contractor's Process Control.

Provide the necessary process control of the friction course mix and construction in accordance with the applicable provisions of 330-2 and 334-6 for FC-5, and 330-2 and 334-6 for FC-9.5 and FC-12.5.

The Engineer will monitor the spread rate periodically to ensure uniform thickness. Provide quality control procedures for daily monitoring and control of spread rate variability. If the spread rate varies by more than 5% of the spread rate set by the Engineer in accordance with 337-9, immediately make all corrections necessary to bring the spread rate into the acceptable range.

337-6 Acceptance of the Mixture at the Plant.

337-6.1 FC-5 and PFC: The FC-5 and PFC mixtures will be accepted at the plant with respect to 334-4 with the following exceptions:

1. The mixture will be accepted with respect to gradation ($P_{-3/8}$, P_{-4}, and P_{-8}), and asphalt binder content (P_b) only.
2. Testing in accordance with AASHTO TP4-00 and FM 1-T 209 (and conditioning prior to testing) will not be required as part of 334-4.1.
3. The standard lot size of FC-5 and PFC will be 2,000 tons [2,000 metric tons], with each lot subdivided into four equal sublots of 500 tons [500 metric tons] each.
4. Initial production requirements of 334-4.3 do not apply.
5. The Between-Laboratory Precision Values described in Table 334-4 are modified to include ($P_{-3/8}$, P_{-4}, and P_{-8}) with a maximum difference per FM 1-T 030 (Figure 2).
6. Table 334-5 (Master Production Range) is replaced by Table 337-3.

Table 337-3 FC-5 and PFC Master Production Range	
Characteristic	Tolerance (1)
Asphalt Binder Content (%)	Target ± 0.60
Passing 3/8 inch [9.50 mm] Sieve (%)	Target ± 7.50
Passing No. 4 [4.75 mm] Sieve (%)	Target ± 6.00
Passing No. 8 [2.36 mm] Sieve (%)	Target ± 3.50
(1) Tolerances for sample size of n = 1 from the verified mix design	

337-6.2 Individual Test Tolerances for FC-5 and PFC Production: In the event that an individual Quality Control test result of a sublot for gradation ($P_{-3/8}$, P_{-4}, and P_{-8}), does not meet the requirements of Table 337-3, take steps to correct the situation and report them to the Engineer.

In the event that two consecutive individual Quality Control test results for gradation ($P_{-3/8}$, P_{-4}, and P_{-8}) or an individual test result for asphalt binder content do not meet the requirements of Table 337-3, the LOT will be automatically terminated and production of the mixture stopped until the problem is adequately resolved (to the satisfaction of the Engineer), unless it can be demonstrated to the satisfaction of the Engineer that the problem can immediately be (or already has been) resolved. Address any material represented by the failing test result in accordance with 334-9.4.

337-7 Acceptance of the Mixture at the Roadway.

The FC-5 and PFC mixtures will be accepted on the roadway with respect to surface tolerance in accordance with the applicable requirements of 330-12. No density testing will be required for these mixtures.

337-8 Special Construction Requirements.

337-8.1 Hot Storage of FC-5 and PFC Mixtures: When using surge or storage bins in the normal production of FC-5 and PFC, do not leave the mixture in the surge or storage bin for more than one hour.

337-8.2 Longitudinal Grade Controls for FC-5 and PFC: Use either longitudinal grade control (skid, ski or traveling stringline) or a joint matcher.

Non SI Units

$$\text{PLI} = \frac{\text{Total Weight of Roller (lb)}}{\text{Total Width of Drums (in.)}}$$

SI Units

$$\frac{\text{kg}}{\text{mm}} = \frac{\text{Total Weight of Roller (kg)}}{\text{Total Width of Drums (mm)}}$$

337-8.3 Temperature Requirements for FC-5 and PFC:

337-8.3.1 Air Temperature at Laydown: Spread the mixture only when the air temperature (the temperature in the shade away from artificial heat) is at or above 65ºF [18ºC]. As an exception, place the mixture at temperatures lower than 65ºF [18ºC], only when approved by the Engineer based on the Contractor's demonstrated ability to achieve a satisfactory surface texture and appearance of the finished surface. In no case shall the mixture be placed at temperatures lower than 60ºF [16ºC].

337-8.3.2 Temperature of the Mix: Heat and combine the asphalt rubber binder and aggregate in a manner to produce a mix having a temperature, when discharged from the plant, meeting the requirements of 330-6.3. Meet all requirements of 330-9.1.2 at the roadway. The target mixing temperature shall be established at 320ºF [160ºC].

337-8.4 Compaction of FC-5 and PFC: Provide two, static steel-wheeled rollers, with an effective compactive weight in the range of 135 to 200 PLI [2.4 to 3.6 kg/mm], determined as follows:

Non SI Units

$$\text{PLI} = \frac{\text{Total Weight of Roller (lb)}}{\text{Total Width of Drums (in.)}}$$

SI Units

$$\frac{\text{kg}}{\text{mm}} = \frac{\text{Total Weight of Roller (kg)}}{\text{Total Width of Drums (mm)}}$$

(Any variation of this equipment requirement must be approved by the Engineer.) Establish an appropriate rolling pattern for the pavement in order to effectively seat the mixture without crushing the aggregate. In the event that the roller begins to crush the aggregate, reduce the number of coverages or the PLI of the rollers. If the rollers continue to crush the aggregate, use a tandem steel-wheel roller weighing not more than 135 lb/in (PLI) [2.4 kg/mm] of drum width.

337-8.6 Prevention of Adhesion: To minimize adhesion to the drum during the rolling operations, the Contractor may add a small amount of liquid detergent to the water in the roller.

At intersections and in other areas where the pavement may be subjected to cross-traffic before it has cooled, spray the approaches with water to wet the tires of the approaching vehicles before they cross the pavement.

337-8.7 Transportation Requirements of Friction Course Mixtures: Cover all loads of friction course mixtures with a tarpaulin.

337-9 Thickness of Friction Courses.

337-9.1 FC-5: The total thickness of the FC-5 layer will be the plan thickness as shown in the Contract Documents. For construction purposes, the plan thickness will be converted to spread rate based on the combined aggregate bulk specific gravity of the asphalt mix being used as shown in the following equation:

Non-SI Units

Spread rate (lbs/yd^2) $= t \times G_{sb} \times 40.5$

Where: t = Thickness (in.) (Plan thickness)
G_{sb} = Combined aggregate bulk specific gravity from the verified mix design

SI Units

Spread rate (kg/m^2) $= t \times G_{sb} \times 0.83$

Where: t = Thickness (mm) (Plan thickness)
G_{sb} = Combined aggregate bulk specific gravity from the verified mix design
The weight of the mixture shall be determined as provided in 320-2.2.

Plan quantities for FC-5 are based on a G_{sb} of 2.635 and thickness of 0.75 in. [20 mm], corresponding to a spread rate of 80 lbs/yd^2 [44 kg/m^2]. Pay quantities will be based on the actual combined aggregate bulk specific gravity (G_{sb}) of the mix being used.

337-9.2 PFC: The total thickness of the PFC layer will be the plan thickness as shown in the Contract Documents. For construction purposes, the plan thickness will be converted to spread rate based on the combined aggregate bulk specific gravity of the asphalt mix being used as shown in the following equation:

Non-SI Units

Spread rate (lbs/yd^2) $= t \times G_{sb} \times 40.5$

Where: t = Thickness (in.) (Plan thickness)
G_{sb} = Combined aggregate bulk specific gravity from the verified mix design

SI Units

Spread rate (kg/m^2) $= t \times G_{sb} \times 0.83$

Where: t = Thickness (mm) (Plan thickness)
G_{sb} = Combined aggregate bulk specific gravity from the verified mix design

The weight of the mixture shall be determined as provided in 320-2.2.

Plan quantities for PFC are based on a G_{sb} of 2.635 and thickness of 1.25 in. [32 mm], corresponding to a spread rate of 133 lbs/yd^2 [70 kg/m^2]. Pay quantities will be based on the actual combined aggregate bulk specific gravity (G_{sb}) of the mix being used.

337-10 Special Equipment Requirements for FC-5 and PFC.

337-10.1 Fiber Supply System: Use a separate feed system to accurately proportion the required quantity of mineral fibers into the mixture in such a manner that uniform distribution is obtained. Interlock the proportioning device with the aggregate feed or weigh system to maintain the correct proportions for all rates of production and batch sizes. Control the proportion of fibers to within plus or minus 10% of the amount of fibers required. Provide flow indicators or sensing devices for the fiber system, interlocked with plant controls so that the mixture production will be interrupted if introduction of the fiber fails.

When a batch plant is used, add the fiber to the aggregate in the weigh hopper or as approved and directed by the Engineer. Increase the batch dry mixing time by 8 to 12 seconds, or as directed by the Engineer, from the time the aggregate is completely emptied into the pugmill. Ensure that the fibers are uniformly distributed prior to the addition of asphalt rubber into the pugmill.

When a drum-mix plant is used, add and uniformly disperse the fiber with the aggregate prior to the addition of the asphalt rubber. Add the fiber in such a manner that it will not become entrained in the exhaust system of the drier or plant.

337-10.2 Hydrated Lime Supply System: For FC-5 and PFC mixes containing granite, use a separate feed system to accurately proportion the required quantity of hydrated lime into the mixture in such a manner that uniform coating of the aggregate is obtained prior to the addition of the asphalt rubber. Add the hydrated lime in such a manner that it will not become entrained in the exhaust system of the drier or plant. Interlock the proportioning device with the aggregate feed or weigh system to maintain the correct proportions for all rates of production and batch sizes and to ensure that all mixture produced is properly treated with hydrated lime. Control the proportion of hydrated lime to within plus or minus 10% of the amount of hydrated lime required. Provide and interlock flow indicators or sensing devices for the hydrated lime system with plant controls so that the mixture production will be interrupted if introduction of the hydrated lime fails. The addition of the hydrated lime to the aggregate may be accomplished by Method (A) or (B) as follows:

337-10.2.1 Method (A) - Dry Form: Add hydrated lime in a dry form to the mixture according to the type of asphalt plant being used.

When a batch plant is used, add the hydrated lime to the aggregate in the weigh hopper or as approved and directed by the Engineer. Increase the batch dry mixing time by eight to twelve seconds, or as directed by the Engineer, from the time the aggregate is completely emptied into the pugmill. Uniformly distribute the hydrated lime prior to the addition of asphalt rubber into the pugmill.

When a drum-mix plant is used, add and uniformly disperse the hydrated lime to the aggregate prior to the addition of the asphalt rubber. Add the hydrated lime in such a manner that it will not become entrained in the exhaust system of the drier or plant.

337-10.2.2 Method (B) - Hydrated Lime/Water Slurry: Add the required quantity of hydrated lime (based on dry weight) in a hydrated lime/water slurry form to the aggregate. Provide a solution consisting of hydrated lime and water in concentrations as directed by the Engineer. Use a plant equipped to blend and maintain the hydrated lime in suspension and to mix it with the aggregates uniformly in the proportions specified.

337-10.3 Hydrated Lime Pretreatment: For FC-5 and PFC mixes containing granite, as an alternative to 337-10.2, pretreat the aggregate with hydrated lime prior to incorporating the aggregate into the mixture. Use a feed system to accurately proportion the aggregate and required quantity of hydrated lime, and mix them in such a manner that uniform coating of the aggregate is obtained. Control the proportion of hydrated lime to within ± 10% of the amount required. Aggregate pretreated with hydrated lime in this manner shall be incorporated into the asphalt mixture within 45 days of pretreatment.

337-10.3.1 Hydrated Lime Pretreatment Methods: Pretreat the aggregate using one of the following two methods:

Pretreatment Method A – Dry Form: Add the required quantity of hydrated lime in a dry form to the aggregate. Assure that the aggregate at the time of preteatment contains a minimum of 3% moisture over saturated surface dry (SSD) conditions. Utilize equipment to accurately proportion the aggregate and hydrated lime and mix them in such a manner as to provide a uniform coating.

Pretreatment Method B – Hydrated Lime/Water Slurry: Add the required quantity of hydrated lime (based on dry weight) in a hydrated lime/water slurry form to the aggregate. Provide a solution consisting of hydrated lime and water in a concentration to provide effective treatment. Use equipment to blend and maintain the hydrated lime in suspension, to accurately proportion the aggregate and hydrated lime/water slurry, and to mix them to provide a uniform coating.

337-10.3.2 Blending Quality Control Records: Maintain adequate Quality Control records for the Engineer's review for all pretreatment activities. Include as a minimum the following information (for each batch or day's run of pretreatment): pretreatment date, aggregate certification information, certified test results for the hydrated lime, aggregate moisture content prior to blending, as-blended quantities of aggregate and hydrated lime, project number, customer name, and shipping date.

337-10.3.3 Certification: In addition to the aggregate certification, provide a certification with each load of material delivered to the HMA plant, that the material has been pretreated in conformance with these specifications. Include also the date the material was pretreated.

337-11 Compensation.

337-11.1 FC-5 and PFC: Meet the requirements of 334-8 with the following exceptions:

1. Pay factors will be calculated for asphalt binder content and the percentages passing the 3/8 inch [9.50 mm], the No. 4 [4.75 mm], and the No. 8 [2.36 mm] sieves only.
2. Table 337-4 replaces Table 334-6.
3. Table 337-5 replaces Table 334-7.
4. The Composite Pay Factor in 334-8.3 is replaced with the following:

CPF = [(0.20 x PF 3/8 inch [9.50 mm]) + (0.30 x PF No. 4 [4.75 mm]) + (0.10 x PF No. 8 [2.36 mm]) + (0.40 x PF AC)]

337-12 Failing Material.

Meet the requirements of 334-9. For FC-5 and PFC, use the Master Production Range defined in Table 337-3 in lieu of Table 334-5.

337-13 Method of Measurement.

For the work specified under this Section (including the pertinent provisions of Sections 320 and 330), the quantity to be paid for will be the weight of the mixture, in tons [metric tons]. The pay quantity will be based on the average spread rate for the project, limited to a maximum of 105% of the spread rate set by the Engineer in accordance with 337-9.

Table 337-4 Small Quantity Pay Table for FC-5 and PFC		
Pay Factor	1-Test Deviation	2-Test Average Deviation
Asphalt Binder Content (%)		
1.00	0.00-0.50	0.00-0.35
0.90	0.51-0.60	0.36-0.42
0.80	>0.60	>0.42
3/8 inch [9.50 mm] Sieve (%)		
1.00	0.00-6.50	0.00-4.60
0.90	6.51-7.50	4.61-5.30
0.80	>7.50	>5.30
No. 4 [4.75 mm] Sieve (%)		
1.00	0.00-5.00	0.00-3.54
0.90	5.01-6.00	3.55-4.24
0.80	>6.00	>4.24
No. 8 [2.36 mm] Sieve (%)		
1.00	0.00-3.00	0.00-2.12
0.90	3.01-3.50	2.13-2.47
0.80	>3.50	>2.47

Table 337-5 Specification Limits for FC-5 and PFC	
Quality Characteristic	Specification Limits
Asphalt Binder Content (%)	Target ± 0.45
Passing 3/8 inch [9.50 mm] sieve (%)	Target ± 6.00
Passing No. 4 [4.75 mm] sieve (%)	Target ± 4.50
Passing No. 8 [2.36] sieve (%)	Target ± 2.50

The bid price for the asphalt mix will include the cost of the asphalt binder (asphalt rubber (or polymer), asphalt cement, ground tire rubber, anti-stripping agent, blending and handling) and the tack coat application as directed in 300-8, as well as fiber stabilizing additive and hydrated lime (if required). There will be no separate payment or unit price adjustment for the asphalt binder material in the asphalt mix. The weight will be determined as provided in 320-2 (including the provisions for the automatic recordation system).

Prepare a Certification of Quantities, using the Department’s current approved form, for the certified asphalt concrete friction course pay item. Submit this certification to the Engineer no later than Twelve O’clock noon Monday after the estimate cut-off or as directed by the Engineer, based on the quantity of asphalt produced and accepted on the Contract. The certification must include the Contract Number, FPID Number, Certification Number, Certification Date, period represented by Certification and the tons [metric tons] produced for each asphalt pay item.

337-14 Basis of Payment.

Price and payment will be full compensation for all the work specified under this Section (including the applicable requirements of Sections 320 and 330).

Payment will be made under:

Item No. 337- 7- Asphaltic Concrete Friction Course -per ton.
Item No. 2337- 7- Asphaltic Concrete Friction Course -per metric ton.

Draft Florida Method for the Design of Porous Friction Course Asphalt Mixtures

In the following, the new mixture design approach developed for Porous Friction Courses is summarized in terms of a Method for Mixture Design. A draft of the corresponding change in the Florida Specifications for Open Graded Friction Courses (including Porous Friction Courses) is found on pages A-2 through A-10.

Scope

1. This method covers the procedures for the design of a modified open graded bituminous mixture selecting the proper aggregate blend and asphalt binder. The selected blend will then be subjected to a series of tests to ensure the proper amount of asphalt binder is used.
2. This method does not purport to address all of the safety problems, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.

Referenced Documents

AASHTO T 305-97 (2001)

AASHTO T 283 (2001)

Summary of Test Method

The method of design for a modified open graded bituminous mixture consists of four steps. The first step is the selection of a trial aggregate blend and asphalt binder content. The second step involves the determination of the optimum asphalt content and checking for adequate asphalt film thickness to ensure durability. The third step involves the performance of a modified AASHTO T 305-97 (2001) (i.e., an asphalt drain down test), and the fourth step involves the performance of AASTHO T-283 (2001). The details of each step are discussed below.

Apparatus

1. Drain-Down equipment as specified in AASHTO T 305-97 (2001)
2. Superpave Gyratory Compactor

3. Balance, 5000 g Capacity, 0.1 g Accuracy
4. 10 metal pie pans
5. Oven capable of maintaining 330 º F ± 3.5 º F
6. Oven capable of maintaining 350 º F ± 3.5 º F
7. Timer

Procedure

1. The aggregate trial blend should be selected to fit within the gradation limits listed in Table 3-17. The asphalt binder should be a PG 76-22 asphalt binder.

Proposed Gradation and Design Specifications for Florida Porous Friction Courses

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 T 305 (%)	<0.3

2. Heat the coarse aggregate and the mold to 350º F ± 3.5º F and the binder to 330º F ± 3.5º F. Mix aggregate with asphalt to obtain at least four trial asphalt contents, viz., 5.5%, 6%, 6.5% and 7%. Just before mixing, add the required amount of mineral fibers to the aggregate. Prepare three specimens at each of the four asphalt contents.

   After mixing, return the mix to the oven for two hours for STOA (Short Term Oven Aging) at 320º F ± 3.5º F. Then compact to 50 gyrations using the Superpave Gyratory Compactor.

   When compacted, cool down at room temperature for 1 hour 45 minutes before removing the specimens from the compaction mold to avoid damaging the specimens.

3. Determine the density of the compacted specimens from its dry mass (in grams) and its volume in cubic centimeters obtained from its dimensions for height and radius. Convert the density to the bulk specific gravity by dividing by 0.99707 g/cc, the density of water at 25º C.

$$G_{mb} \text{ (Bulk Sp.Gr)} = W / (\pi R^2 H / 0.99707) \qquad (3.6)$$

$$= \text{Weight (g)} \times 0.0048417/\text{Height (in)}$$

where:

W = Weight of Specimen in grams

R = Radius in centimeters

H = Height in centimeters

4. Determine Theoretical Maximum Specific Gravity according to AASHTO T-209-99 (2004).

   Calculate percent air voids, VMA and voids filled with asphalt based on aggregate specific gravity.

   Plot VMA curve versus AC content and determine point of minimum VMA, select corresponding AC as Optimum asphalt content.

5. Calculate the Theoretical Film Thickness at the optimum asphalt content, using the following steps:

   a) Determine volume of asphalt (V_T) (in ml) by using following equation:

$$V_T = \frac{P_b * W_{agg}}{100 * 1.03} \tag{3.7}$$

where:

P_b = Percent Asphalt (by weight of aggregate),

W_{agg} = Weight of aggregate (in grams)

   b) Determine the Theoretical Film Thickness:

$$T_{film} = \frac{V_T \times 1000}{SA \times W_{agg}} \tag{3.8}$$

where:

W_{agg} = Weight of aggregate
SA = Surface area (determined using the Surface Area Factors in Table 3-18).

Surface Area Factors.

Sieve Size	**Surface Area Factor**
Percentage Passing Maximum Sieve Size	0
Percent Passing No. 4	0
Percent Passing No. 8	4
Percent Passing No. 16	8
Percent Passing No. 30	14
Percent Passing No. 50	30
Percent Passing No. 100	60
Percent Passing No. 200	160

Note: The minimum acceptable effective film thickness is 34 micron.

6. Perform the drain down test in accordance with AASHTO T 305-97 (2001). A mix with an optimum AC content as determined previously, is placed in a wire basket consisting of 6.4 mm (¼ in) mesh openings and heated 14º C (25º F) above the normal production temperature (this is typically around 177° C [350º F]) for one hour. The amount of binder, which drains from the basket, is measured. If the specimen exceeds the maximum drain down of 0.3%, increase the fiber content by 0.1% and repeat the test.
7. Perform an evaluation of moisture damage susceptibility in accordance with a modified AASHTO T-283 (2003) on the compacted specimens. Each specimen is placed in 1/8-in wire mesh roll, which is kept in position using two clamps on either edge of the specimen for avoiding mixture damage or breakdown at the conditioning temperature of 60º C (140º F) for 24 hours, and then allowed to cool down in a water bath at 25° C (77° F) for 2 hours. It is not necessary to perform the vacuum saturation procedure in AASHTO T-283, since these mixtures already have high air voids allowing water to penetrate the mixture easily. Once, the moisture conditioning is completed, the conditioned and unconditioned specimens are tested for their indirect tensile strength, as per AASHTO T-283. Minimum requirements should include TSR of 0.8 or greater.

Appendix B

DEVELOPMENTAL SPECIFICATION

FOR OPEN GRADED FRICTION COURSE SECTIONS INCLUDING

POLYMER MODIFIED TACK COAT

SECTION 337
ASPHALT CONCRETE FRICTION COURSES

337-1 Description.

Construct asphalt concrete friction course test sections using the CQC acceptance system as defined in these Specifications.

Meet the plant and equipment requirements of Section 320, as modified herein. Meet the general construction requirements of Section 330, as modified herein.

337-2 Materials.

337-2.1 General Requirements: Meet the requirements specified in Division III as modified herein. The Engineer will base continuing approval of material sources on field performance.

337-2.2 Asphalt Binder: Meet the requirements of Section 336, and any additional requirements or modifications specified herein for the various mixtures. When called for in the Contract Documents, use a PG 76-22 asphalt binder meeting the requirements of 916-1.

337-2.3 Polymer-Modified Emulsion Membrane: The Polymer Modified Emulsion Membrane shall be a styrene-butadiene block co-polymer (SB) modified asphalt emulsion. Polymer modification of the base asphalt shall be completed prior to emulsification. The emulsion shall be smooth and homogeneous and conform to the following requirements:

Test on Emulsion	**Method**	**Min.**	**Max.**
Viscosity @ 77°F SSF	AASHTO T-59	20	100
Sieve Test, %	AASHTO T-59		0.1
24-Hour Storage Stability, % (1)	AASHTO T-59		1
Residue from Distillation @ 400°F %(2)	AASHTO T-59	63	
Oil portion from distillation ml of oil per 100 g emulsion			2
Demulsibility 35 ml 0.02 N $CaCl_2$ or 35 ml, 08% dioctyl sodium sulfosuccinate	AASHTO T-59	60	
Test On Residue from Distillation			
Solubility in TCE, %(3)	AASHTO T-44		97.5
Elastic Recovery, 50° F, 20 cm elongation %(4)	AASHTO T-301	60	
Penetration @ 77° F, 100 g, 5 sec, 0.1 mm	AASHTO T-49	60	150

(Note 1) After standing undisturbed for 24 hours, the surface shall show no white, milky colored substance, but shall be a smooth homogeneous color throughout.
(Note 2) AASHTO T-59 with modifications to include a 400 ± 10 ºF maximum temperature to be held for a period of 15 minutes.
(Note 3) ASTM D5546, Test Method for Solubility of Polymer-Modified Asphalt Materials in 1,1,1-Trichloroethane may be substituted where polymers block the filter in Method D2042.
(Note 4) ASTM D6084, Standard Test Method for Elastic Recovery of Bituminous Materials by Ductilometer with exception that the elongation is 20 cm and the test temperature is 50ºF.

337-2.4 Coarse Aggregate: Meet the requirements of Section 901, and any additional requirements or modifications specified herein for the various mixtures.

337-2.5 Fine Aggregate: Meet the requirements of Section 902, and any additional requirements or modifications specified herein for the various mixtures.

337-2.6 Hydrated Lime: Meet the requirements of AASHTO M303 Type 1. Provide certified test results for each shipment of hydrated lime indicating compliance with the specifications.

337-2.7 Fiber Stabilizing Additive: Use either a mineral or cellulose fiber-stabilizing additive. Meet the following requirements:

337-2.7.1 Mineral Fibers: Use mineral fibers made from virgin basalt, diabase, or slag treated with a cationic sizing agent to enhance the disbursement of the fiber, as well as to increase adhesion of the fiber surface to the bitumen. Meet the following requirements for physical properties:

1. Size Analysis
   - Average fiber length 0.25 inch (maximum)
   - Average fiber thickness 0.0002 inch (maximum)
2. Shot Content (ASTM C612)
   - Percent passing No. 60 Sieve 90 - 100
   - Percent passing No.230 Sieve 65 - 100

Provide certified test results for each batch of fiber material indicating compliance with the above tests.

337-2.7.1.1 Notice of Patented Process: Take notice that the use of mineral fibers treated with cationic sizing agent and the size analysis range for average fiber thickness are subject to U.S. Patent No. 4,613,376, held by Fiberand Corporation, 7150 Southwest 62nd Avenue, South Miami, Fl. 33143. Obtain all mineral fibers required to meet the FC-5 requirements of this Contract only from Fiberand Corporation or a duly authorized licensee of Fiberand. Assume responsibility, pursuant to 7-3, for obtaining any and all necessary rights to use such processes and pay any and all royalties, license fees or other costs incurred in order to meet the FC-5 requirements of this Contract. Include any and all royalties, license fees and other costs arising due to the existence of U.S. Patent No. 4,613,376 in the bid unit price for friction course FC-5.

337-2.7.2 Cellulose Fibers: Use cellulose fibers meeting the following requirements:

1. Fiber length 0.25 inch (maximum)
2. Sieve Analysis
   - a. Alpine Sieve Method
     - Percent passing No. 100 sieve 60-80
   - b. Ro-Tap Sieve Method
     - Percent passing No. 20 sieve 80-95
     - Percent passing No. 40 sieve 45-85
     - Percent passing No. 100 sieve 5-40
3. Ash Content: 18% non-volatiles (±5%)
4. pH: 7.5 (±1.0)
5. Oil Absorption: 5.0 (±1.0) (times fiber weight)
6. Moisture Content: 5.0 (maximum)

Provide certified test results for each batch of fiber material indicating compliance with the above tests.

337-3 General Composition of Mixes.

337-3.1 General: This project consists of conventional FC-5 asphalt mixture and one test section (as described in the plans) of a FC-5 bonded friction course mixture. The FC-5 mixture and the FC-5 bonded friction course mixture will be the same open graded friction course mixture except that the FC-5 bonded friction course mixture will be applied over a thick polymer modified membrane described in this Developmental Specification. In general, use a bituminous mixture composed of aggregate, asphalt binder, fibers and in some cases hydrated lime. Size, uniformly grade and combine the aggregate fractions in such proportions that the resulting mix meets the requirements of this Developmental Specification. The use of RAP material will not be permitted.

337-3.2 Mixture Component Requirements:

337-3.2.1 FC-5 and FC-5 Bonded Friction Course:

337-3.2.1.1 Aggregates: Use an aggregate blend which consists of either 100% crushed granite or 100% crushed Oolitic limestone. Use the same aggregate type, from the same source, for both the conventional FC-5 mixture and the FC-5 bonded friction course mixture.

In addition to the requirements of Section 901, meet the following coarse aggregate requirements. Use either crushed granite or crushed limestone. Use crushed limestone from the Oolitic formation, which contains a minimum of 12% non-carbonate material (as determined by FM 5-510), and has been approved for this use. In addition to the requirements of Section 902, meet the following fine aggregate requirements. Use either crushed granite screenings, or crushed Oolitic limestone screenings for the fine aggregate. Meet the design gradation requirements of Table 337-1.

337-3.2.1.2 Asphalt Binder: Use an ARB-12 asphalt rubber binder. If called for in the Contract Documents, use a PG 76-22 asphalt binder.

337-3.2.1.3 Hydrated Lime: Add the lime at a dosage rate of 1.0% by weight of the total dry aggregate to mixtures containing granite.

337-3.2.1.4 Fiber Stabilizing Additive: Add either mineral fibers at a dosage rate of 0.4% by weight of the total mix, or cellulose fibers at a dosage rate of 0.3% by weight of total mix.

337-3.3 Grading Requirements:

Use a mixture with a gradation from Table 337-1. Use the same aggregate gradation for both the conventional FC-5 mixture and the FC-5 bonded friction course mixture.

Table 337-1
Gradation Design Range

3/4 inch	1/2 inch	3/8 inch	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200
100	85-100	55-75	15-25	5-10	--	--	--	--	2-4

337-4 Mix Design.

337-4.1 FC-5 and FC-5 Bonded Friction Course: The Department will design the FC-5 and FC-5 bonded friction course mixtures. There will be one design used for both the FC-5 and FC-5 bonded friction course. Furnish materials and the appropriate information (source, gradation, etc.) as specified in 334-3.2.5. The Department will have two weeks to design the mixtures.

The Department will establish the design binder content for these mixtures within the following ranges based on aggregate type:

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

337-4.2 Revision of Mix Design: All revisions must fall within the applicable gradation ranges for each test section mixture as defined in Table 337-1.

337-5 Contractor's Process Control.

Provide the necessary process control of the friction course mix, Polymer Modified Emulsion Membrane, and construction in accordance with the applicable provisions of 330-2 and 334-6.

The Engineer will monitor the spread rate periodically to ensure uniform thickness. Provide quality control procedures for daily monitoring and control of spread rate variability. If the spread rate varies by more than 5% of the spread rate set by the Engineer in accordance with 337-9, immediately make all corrections necessary to bring the spread rate into the acceptable range.

337-6 Acceptance of the Mixture At the Plant.

The mixture will be accepted at the plant with respect to 334-4 with the following exceptions:

1. The mixture will be accepted with respect to gradation (P-3/8, P-4, and P-8), and asphalt binder content (Pb) only.
2. Testing in accordance with AASHTO TP4-00 and FM 1-T 209 (and conditioning prior to testing) will not be required as part of 334-4.1.
3. The standard lot size will be 2,000 tons, with each lot subdivided into four equal sublots of 500 tons each.
4. Initial production requirements of 334-4.3 do not apply.
5. The Between-Laboratory Precision Values described in Table 334-4 are modified to include (P–3/8, P-4, and P-8) with a maximum difference per FM 1-T 030 (Figure 2).
6. Table 334-5 (Master Production Range) is replaced by Table 337-2.

Table 337-2
Master Production Range

Characteristic	Tolerance (1)
Asphalt Binder Content (%)	Target ± 0.60
Passing 3/8 inch Sieve (%)	Target ± 7.50
Passing No. 4 Sieve (%)	Target ± 6.00
Passing No. 8 Sieve (%)	Target ± 3.50

(1) Tolerances for sample size of n = 1 from the verified mix design

337-6.1 Individual Test Tolerances for Production: In the event that an individual Quality Control test result of a sublot for gradation ($P_{-3/8}$, P_{-4}, and P_{-8}), does not meet the requirements of Table 337-2, take steps to correct the situation and report them to the Engineer.

In the event that two consecutive individual Quality Control test results for gradation ($P_{-3/8}$, P_{-4}, and P_{-8}) or an individual test result for asphalt binder content do not meet the requirements of

Table 337-2, the LOT will be automatically terminated and production of the mixture stopped until the problem is adequately resolved (to the satisfaction of the Engineer), unless it can be demonstrated to the satisfaction of the Engineer that the problem can immediately be (or already has been) resolved. Address any material represented by the failing test result in accordance with 334-9.4.

337-7 Acceptance of the Mixture at the Roadway.

337-7.1 General: The mixtures will be accepted on the roadway with respect to surface tolerance in accordance with the applicable requirements of 330-12. No density testing will be required for these mixtures.

337-7.2 Additional Tests: The Department reserves the right to run any test at any time for informational purposes and for determining the effectiveness of the Contractor's quality control.

337-7.3 Finished Pavement: The finished pavement at the time of completion shall be free of all types of pavement distresses, including, but not limited to flushing, bleeding, fat spots, raveling, mix delamination, potholes, rutting, and shoving. Areas failing to meet these criteria shall be corrected as approved by the Engineer. In no case shall inspection or acceptance testing by the Department relieve the Contractor of this responsibility.

337-8 Special Construction Requirements.

337-8.1 Hot Storage of Mixtures: When using surge or storage bins in the normal production of mixtures, do not leave the mixture in the surge or storage bin for more than one hour.

337-8.2 Longitudinal Grade Controls for Friction Courses: Use either longitudinal grade control (skid, ski or traveling stringline) or a joint matcher.

337-8.3 Surface Preparation: Manhole covers, drains, grates catch basins and other such utility structures shall be protected and covered with plastic or building felt prior to paving and also shall be clearly referenced for location and adjustment after paving. Thermoplastic traffic markings shall be removed if greater than 0.2” thickness. Pavement cracks and joints greater than 0.25” wide shall be cleaned and filled as approved by the Engineer. The entire pavement surface to be overlaid shall be thoroughly cleaned, giving specific attention to accumulated mud, hot mix asphalt build-up, tack tracking and debris. Milling pressurized water and/or vacuum systems may be required to insure a clean surface.

337-8.4 Bonded Friction Course Paving Equipment: Use a self-priming paver, designed and built for the purpose of applying the bonded asphalt concrete friction course. The self-priming machine shall be capable of spraying the Polymer Modified Emulsion Membrane, applying the hot asphalt concrete overlay and leveling the surface of the mat in one pass at the rate of 30.5 - 92 ft/minute. The self-priming paving machine shall incorporate a receiving hopper, feed conveyor, insulated storage tank for Polymer Modified Emulsion Membrane, Polymer Modified Emulsion Membrane spray bar and a variable width, heated, tamper bar screed. The screed shall have the ability to crown the pavement at the center both positively and negatively and have vertically adjustable extensions to accommodate the desired pavement profile.

337-8.5 Polymer Modified Emulsion Membrane: The Polymer Modified Emulsion Membrane shall be used on test section for the FC-5 bonded friction course. The Polymer Modified Emulsion Membrane shall be sprayed by a metered mechanical pressure spray bar at a temperature of 140 - 180°F. The sprayer shall accurately and continuously monitor the rate of spray and provide a uniform application across the entire width to be overlaid. The target rate of application shall be in the range of 0.15 – 0.30 gal/yd^2 as recommended by the Contractor. Adjustments to the spray rate shall be made based upon the existing pavement surface conditions and recommendations of the Polymer Modified Emulsion Membrane supplier. The actual spray rate shall be verified by the Engineer at the job site during construction.

No wheel or other part of the paving machine shall come in contact with the Polymer Modified Emulsion Membrane before the hot mix asphalt friction course is applied.

The hot mix asphalt concrete shall be applied at a temperature of 320°F and shall be spread over the Polymer Modified Emulsion Membrane immediately after the application of the Polymer Modified Emulsion Membrane. The hot mix asphalt friction course shall be placed over the full width of the Polymer Modified Emulsion Membrane with a heated, combination vibratory-tamping bar screed.

337-8.6 Application: Immediately cease transportation of the asphalt mixture from the plant when rain begins at the roadway. Do not place asphalt mixtures while rain is falling, or when there is water on the surface to be covered. A damp pavement surface is acceptable for placement of Polymer Modified Emulsion Membrane if it is free of standing water and favorable weather conditions are expected to follow.

Allow the pavement temperature to cool to a temperature of not greater than 175°F prior to opening the pavement to traffic.

337-8.7 Temperature Requirements:

337-8.7.1 Air Temperature at Laydown: Spread the mixture only when the air temperature (the temperature in the shade away from artificial heat) is at or above 65ºF. As an exception, place the mixture at temperatures lower than 65ºF, only when approved by the Engineer based on the Contractor's demonstrated ability to achieve a satisfactory surface texture and appearance of the finished surface. In no case shall the mixture be placed at temperatures lower than 60ºF.

337-8.7.2 Temperature of the Mixture for FC-5 and FC-5 Bonded Friction Course: Heat and combine the asphalt rubber binder and aggregate in a manner to produce a mix having a temperature, when discharged from the plant, meeting the requirements of 330-6.3. Meet all requirements of 330-9.1.2 at the roadway. The target mixing temperature shall be established at 320°F.

337-8.8 Compaction Requirements:

337-8.8.1 Compaction of FC-5 and FC-5 Bonded Friction Course: Provide two static steel-wheeled rollers with an effective compactive weight in the range of 135 to 200 lb/in (PLI) as determined below. The Engineer must approve any variation of this equipment requirement. Establish an appropriate rolling pattern for the pavement in order to effectively seat the mixture without crushing the aggregate. In the event that the roller begins to crush the aggregate, reduce the number of coverages or the PLI of the rollers. If the rollers continue to crush the aggregate, use a tandem steel-wheel roller weighing not more than 135 PLI of drum width as determined below.

$$\text{PLI} = \frac{\text{Total Weight of Roller (lb)}}{\text{Total Width of Drums (in.)}}$$

337-8.9 Prevention of Adhesion: To minimize adhesion to the drum during the rolling operations, the Contractor may add a small amount of liquid detergent to the water in the roller.

At intersections and in other areas where the pavement may be subjected to cross-traffic before it has cooled, spray the approaches with water to wet the tires of the approaching vehicles before they cross the pavement.

337-8.10 Transportation Requirements of Friction Course Mixtures: Cover all loads of friction course mixtures with a tarpaulin.

337-9 Thickness of Friction Courses.

The total thickness of the FC-5 layer will be the plan thickness as shown in the Contract Documents. For construction purposes, the plan thickness will be converted to spread rate based on the combined aggregate bulk specific gravity of the asphalt mix being used as shown in the following equation:

$$\text{Spread rate (lbs/yd}^2\text{)} = t \times G_{sb} \times 40.5$$

Where: t = Thickness (in.) (Plan thickness)
G_{sb} = Combined aggregate bulk specific gravity from the verified mix design

The weight of the mixture shall be determined as provided in 320-2.2.

Plan quantities are based on a G_{sb} of 2.635, corresponding to a spread rate of 80 lbs/yd^2. Pay quantities will be based on the actual combined aggregate bulk specific gravity (G_{sb}) of the mix being used.

337-10 Special Equipment Requirements for FC-5.

337-10.1 Fiber Supply System: Use a separate feed system to accurately proportion the required quantity of mineral fibers into the mixture in such a manner that uniform distribution is obtained. Interlock the proportioning device with the aggregate feed or weigh system to maintain the correct proportions for all rates of production and batch sizes. Control the proportion of fibers to within plus or minus 10% of the amount of fibers required. Provide flow indicators or sensing devices for the fiber system, interlocked with plant controls so that the mixture production will be interrupted if introduction of the fiber fails.

When a batch plant is used, add the fiber to the aggregate in the weigh hopper or as approved and directed by the Engineer. Increase the batch dry mixing time by 8 to 12 seconds, or as directed by the Engineer, from the time the aggregate is completely emptied into the pugmill. Ensure that the fibers are uniformly distributed prior to the addition of asphalt rubber into the pugmill.

When a drum-mix plant is used, add and uniformly disperse the fiber with the aggregate prior to the addition of the asphalt rubber. Add the fiber in such a manner that it will not become entrained in the exhaust system of the drier or plant.

337-10.2 Hydrated Lime Supply System: For FC-5 mixes containing granite, use a separate feed system to accurately proportion the required quantity of hydrated lime into the mixture in such a manner that uniform coating of the aggregate is obtained prior to the addition of the asphalt rubber. Add the hydrated lime in such a manner that it will not become entrained in the exhaust system of the drier or plant. Interlock the proportioning device with the aggregate feed or weigh system to maintain the correct proportions for all rates of production and batch sizes and to ensure that all mixture produced is properly treated with hydrated lime. Control the proportion of hydrated lime to within plus or minus 10% of the amount of hydrated lime required. Provide and interlock flow indicators or sensing devices for the hydrated lime system with plant controls so that the mixture production will be interrupted if introduction of the hydrated lime fails. The addition of the hydrated lime to the aggregate may be accomplished by Method (A) or (B) as follows:

337-10.2.1 Method (A) - Dry Form: Add hydrated lime in a dry form to the mixture according to the type of asphalt plant being used.

When a batch plant is used, add the hydrated lime to the aggregate in the weigh hopper or as approved and directed by the Engineer. Increase the batch dry mixing time by eight to twelve seconds, or as directed by the Engineer, from the time the aggregate is completely emptied into the pugmill. Uniformly distribute the hydrated lime prior to the addition of asphalt rubber into the pugmill.

When a drum-mix plant is used, add and uniformly disperse the hydrated lime to the aggregate prior to the addition of the asphalt rubber. Add the hydrated lime in such a manner that it will not become entrained in the exhaust system of the drier or plant.

337-10.2.2 Method (B) - Hydrated Lime/Water Slurry: Add the required quantity of hydrated lime (based on dry weight) in a hydrated lime/water slurry form to the aggregate. Provide a solution consisting of hydrated lime and water in concentrations as directed by the Engineer. Use a plant equipped to blend and maintain the hydrated lime in suspension and to mix it with the aggregates uniformly in the proportions specified.

337-10.3 Hydrated Lime Pretreatment: For FC-5 mixes containing granite, as an alternative to 337-10.2, pretreat the aggregate with hydrated lime prior to incorporating the aggregate into the mixture. Use a feed system to accurately proportion the aggregate and required quantity of hydrated lime, and mix them in such a manner that uniform coating of the aggregate is obtained. Control the proportion of hydrated lime to within ± 10% of the amount required. Aggregate pretreated with hydrated lime in this manner shall be incorporated into the asphalt mixture within 45 days of pretreatment.

337-10.3.1 Hydrated Lime Pretreatment Methods: Pretreat the aggregate using one of the following two methods:

Pretreatment Method A – Dry Form: Add the required quantity of hydrated lime in a dry form to the aggregate. Assure that the aggregate at the time of preteatment contains a minimum of 3% moisture over saturated surface dry (SSD) conditions. Utilize equipment to accurately proportion the aggregate and hydrated lime and mix them in such a manner as to provide a uniform coating.

Pretreatment Method B – Hydrated Lime/Water Slurry: Add the required quantity of hydrated lime (based on dry weight) in a hydrated lime/water slurry form to the aggregate. Provide a solution consisting of hydrated lime and water in a concentration to provide effective treatment. Use equipment to blend and maintain the hydrated lime in suspension, to accurately proportion the aggregate and hydrated lime/water slurry, and to mix them to provide a uniform coating.

337-10.3.2 Blending Quality Control Records: Maintain adequate Quality Control records for the Engineer’s review for all pretreatment activities. Include as a minimum the following information (for each batch or day’s run of pretreatment): pretreatment date, aggregate certification information, certified test results for the hydrated lime, aggregate moisture content prior to blending, as-blended quantities of aggregate and hydrated lime, project number, customer name, and shipping date.

337-10.3.3 Certification: In addition to the aggregate certification, provide a certification with each load of material delivered to the HMA plant, that the material has been pretreated in conformance with these specifications. Include also the date the material was pretreated.

337-11 Compensation.

Meet the requirements of 334-8 with the following exceptions:

1. Pay factors will be calculated for asphalt binder content and the percentages passing the 3/8 inch, the No. 4, and the No. 8 sieves only.
2. Table 337-3 replaces Table 334-6.
3. Table 337-4 replaces Table 334-7.
4. The Composite Pay Factor in 334-8.3 is replaced with the following:

CPF = [(0.20 x PF 3/8 inch) + (0.30 x PF No. 4) + (0.10 x PF No. 8) + (0.40 x PF AC)]

Table 337-3
Small Quantity Pay Table

Pay Factor	**1-Test Deviation**	**2-Test Average Deviation**
Asphalt Binder Content (%)		
1.00	0.00-0.50	0.00-0.35
0.90	0.51-0.60	0.36-0.42
0.80	>0.60	>0.42
3/8 inch Sieve (%)		
1.00	0.00-6.50	0.00-4.60
0.90	6.51-7.50	4.61-5.30
0.80	>7.50	>5.30
No. 4 Sieve (%)		
1.00	0.00-5.00	0.00-3.54
0.90	5.01-6.00	3.55-4.24
0.80	>6.00	>4.24
No. 8 Sieve (%)		
1.00	0.00-3.00	0.00-2.12
0.90	3.01-3.50	2.13-2.47
0.80	>3.50	>2.47

Table 337-4
Specification Limits

Quality Characteristic	**Specification Limits**
Asphalt Binder Content (%)	Target ± 0.45
Passing 3/8 inch sieve (%)	Target ± 6.00
Passing No. 4 sieve (%)	Target ± 4.50
Passing No. 8 sieve (%)	Target ± 2.50

337-12 Failing Material.

Meet the requirements of 334-9. Use the Master Production Range defined in Table 337-2 in lieu of Table 334-5.

337-13 Method of Measurement.

337-13.1 Asphalt Friction Course: For the work specified under this Section (including the pertinent provisions of Sections 320 and 330), the quantity to be paid for will be the weight of the mixture, in tons. The pay quantity will be based on the average spread rate for the project, limited to a maximum of 105% of the spread rate set by the Engineer in accordance with 337-9.

The bid price for the asphalt mix will include the cost of the asphalt binder (asphalt rubber or polymer, asphalt cement, ground tire rubber, anti-stripping agent, blending and handling), tack coat application as directed in 300-8, fiber-stabilizing additive, hydrated lime (if required), and Polymer Modified Emulsion Membrane. There will be no separate payment or unit price adjustment for the asphalt binder material in the asphalt mix. The weight will be determined as provided in 320-2 (including the provisions for the automatic recordation system).

337-14 Basis of Payment.

337-14.1 Asphalt Friction Course: Price and payment will be full compensation for all the work specified under this Section (including the applicable requirements of Sections 320 and 330).

Payment will be made under:
Item No. 901-337-8 Asphaltic Concrete Friction Course - per ton.