NCAT Report 20-07

IMPROVEMENTS OF OPEN-GRADED FRICTION COURSE IN ALABAMA

ALDOT 930-790

Don Watson
Nadeem Anwer Qureshi
Zhaoxing Xie
Nam Tran

December 2020

National Center for Asphalt Technology
NCAT
at AUBURN UNIVERSITY

277 Technology Parkway ■ Auburn, AL 36830

Improvements of Open-Graded Friction Course in Alabama

ALDOT 930-790

NCAT Report 20-07

By

Don Watson, PE
Lead Research Engineer
National Center for Asphalt Technology
Auburn University, Auburn, Alabama

Nadeem Anwer Qureshi
Visiting Scholar
National Center for Asphalt Technology
Auburn University, Auburn, Alabama

Zhaoxing Xie
Doctoral Candidate
National Center for Asphalt Technology
Auburn University, Auburn, Alabama

Nam Tran, PhD, PE
Assistant Director
National Center for Asphalt Technology
Auburn University, Auburn, Alabama

December 2020

DISCLAIMER

ACKNOWLEDGEMENTS

This work was sponsored by the Alabama Department of Transportation.

The authors gratefully acknowledge the following members of the NCAT Applications Steering Committee for their review of this technical report: Heather Hall, Tim Kowalski, Kevin McGhee, Tim Murphy, and Debbie Schwerman.

ABSTRACT

Open-graded friction courses (OGFC) are special purpose surface mixtures that have a wide range of benefits including:

- Reduces the risk of hydroplaning and/or wet skidding
- Reduces the splash and spray of vehicle tires on the pavement
- Reduces pavement noise
- Improves the visibility of pavement markings at night
- Improves the visibility of pavement markings during wet weather

However, performance issues that lead to decreased service life, such as raveling and cracking, have hindered the widespread use of OGFC.

The primary objective of this study was to evaluate the performance of OGFC pavements in Alabama and determine methods for improving durability. The scope of the research included field work and lab investigations. Field work involved a field survey and coring on selected projects for laboratory testing. International roughness index (IRI) data from the pavement management system of the Alabama Department of Transportation (ALDOT) was analyzed to help determine the nature of distresses and identify whether the problem was consistent throughout the pavement section or localized. Review of the mix design, construction data and specifications were completed to compare ALDOT requirements with other state DOT requirements. Laboratory testing included indirect tensile strength (IDT), Cantabro stone loss, air voids, gradation, and asphalt content of the extracted cores. Other methods for improving OGFC performance were considered by comparing the performance of OGFC sections on the NCAT Test Track.

Results of the analyses show that in raveled portions of a project, OGFC pavements tend to have higher air voids and lower IDT strengths. IDT strength can be improved by using a thicker OGFC layer or using a finer gradation. An increase in tack application rates for emulsion should also improve resistance to raveling and OGFC should not be placed in cold weather.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

1 INTRODUCTION

An open-graded friction course (OGFC) asphalt mixture is designed to be water permeable and is normally placed on top of traditional dense-graded pavement. OGFC is a special purpose thin surface mixture (Cooley et al., 2000) used to:

- Reduce the risk of hydroplaning,
- Reduce wet-weather skidding,
- Reduce splash and spray when driving in the rain,
- Reduce pavement noise,
- Improve night and wet-weather visibility of pavement markings, and
- Provide better friction surfaces during wet weather.

OGFC offers numerous benefits to the environment in terms of sound absorption potential and pollution control (Hamzah et al., 2010). OGFC has helped to reduce pollution by enhancing road water surface runoff, thereby reducing harmful metal discharge into the environment (Pagotto et al., 2000; Boheman et al., 2003). OGFC is also effectively used in storm water drainage and parking lots (Boving et al., 2007; Martin et al., 2007).

OGFC mixtures are frequently used as surface layers in countries like the United States, Japan, United Kingdom, Malaysia, Australia, New Zealand, and South Africa on high speed roadways and interstate expressways (Huber, 2000; Alvarez et al., 2006). OGFC is also recommended to be used as surface course on airport pavements (Cooley, 2007).

1.1 Background

The Alabama Department of Transportation (ALDOT) has customarily used OGFC as the final riding surface on interstate and high traffic volume roadways. The OGFC mixture provides several benefits due to water drainage such as reduced hydroplaning, reduced splash and spray, and improved visibility. However, some pavement sections that were surfaced with OGFC have experienced premature failures that may threaten its use as a surface course in Alabama.

ALDOT has experienced premature failures in several of its construction divisions. Mixes on those projects are raveling out after only six to eight years. In contrast, it is reported that ALDOT's spray paver-placed OGFC surfaces are lasting more than 10 years (on extremely high traffic volume roadways). Industry representatives have contended that the problem with OGFC performance in the past was that the former placement rate (70 lbs/yd^2) was not thick enough. However, less than satisfactory performance has also been experienced on some projects with a rate more than 90 lbs/yd^2.

Raveling at transverse joints is an issue on some OGFC projects. At plant start-up, there is usually some time required for all the metal of the plant and roadway equipment to be heated up and production temperatures to be stabilized. One good practice to improve transverse joint construction is to begin with the third or fourth load of mix, which is more likely to be at a consistent temperature than the first load produced. Transverse raveling may also be a result of paver stops waiting on truckloads of mix. To prevent this, make sure mix placement is a continuous operation and require the contractor to take corrective action if paver stops are encountered.

Several pavement sites were visited on I-65 that exhibited severe failures. Although surface distress was evident in all lanes, the severe failures were more predominant in the outside lanes in both the southbound and northbound directions. The types of failure were visually identified and placed in three categories:

- Raveling from the top downward;
- Raveling completely through the layer; and
- Combined failure distresses.

Significant loss of coated aggregate from OGFC (raveling) that appears to be from the top downward is consistent with wear from high traffic volumes, moisture damage, and insufficient asphalt particle coating. An example of this type failure is shown in Figure 1.

Figure 1. Raveling in Southbound Outside Lane at Milepost 93 (near Evergreen)

Raveling that extends completely through the surface layer to the interface of the underlying layer, as shown in Figure 2 and Figure 3, may be associated with a lack of tack bond strength or insufficient internal cohesive strength. Such distress may be due to an insufficient amount of tack coat, improper application of tack, or failure to adequately clean the surface prior to application of tack. The raveling may also be a result of inadequate binder content and use of a coarse gradation with little to no fines.

Figure 4 (a) and 4(b) show pavement failures at milepost 173 northbound, which is next to a bridge at the Alabama River. The distress evident on the pavement surface suggests that the failure is a combination of raveling in the OGFC and cracking caused by insufficient support from the underlying layers. The lack of support may be caused by moisture damage in subsurface layers or structural base failure.

Figure 2. Raveling in Southbound Outside Lane at Milepost 177.9 (near Prattville)

Figure 3. Loose Aggregate on Northbound Shoulder at Milepost 94.1 (near Evergreen)

(a) (b)

Figure 4. Failures on Northbound Outside Lane at Milepost 173 (Alabama River)

The current maintenance practices of OGFC pavements are also not yielding effective results. The application of fog and rejuvenator seals, or the lack thereof, could be one of the possible causes for this issue.

1.2 Objective

The purpose of this research study was to improve the durability of OGFC pavements in Alabama. A three phase research plan was conducted to accomplish the study objectives.

- In the first phase, five OGFC pavements located in Alabama were evaluated using field work and lab investigations.
- In the second phase, the effect of using fog and rejuvenator seals on the surface friction and durability of OGFC was assessed with the aim of recommending an optimal application rate.
- In the third phase, laboratory performance tests were conducted to evaluate the durability of OGFC mixtures.

1.3 Work Plan

This research study was conducted by performing a literature review to determine available OGFC mix design procedures currently used by ALDOT and others, conducting a field survey to determine the cause of failure on selected projects, and evaluating laboratory test procedures that may lead to improved performance by making revisions to the mix design procedure.

Field work involved a field survey and coring on selected projects for forensic investigation. The mix design, construction data, and specifications were reviewed to compare ALDOT requirements with other state DOT requirements. Laboratory investigation of field cores included measurement of layer thickness, indirect tensile (IDT) strength, air voids, gradation, and the asphalt binder content of extracted cores.

OGFC pavement thickness and gradation were believed to be important factors in the long-term performance of OGFC mixtures so comparisons were made of OGFC sections on the NCAT Test Track, which included coarse and fine gradations and different combinations of binder modifier and fiber stabilizer. The performance tests included permeability, Cantabro stone loss, Hamburg wheel tracking tests, and IDT strength tests.

2 LITERATURE REVIEW

2.1 OGFC Mix Design Procedures

OGFC was introduced in the United States in the 1940s, but it was slow to be implemented until the development of an OGFC design method by the Federal Highway Administration (FHWA) in 1974 (Smith et al., 1974). This method was adopted by a number of states for the design of OGFC mixtures; however, problems such as raveling, stripping, draindown, and short service life forced state agencies to abandon their shortly afterward. During the same period, OGFC mixtures were used successfully in Europe and Japan because those were placed in thicker lifts, had coarser gradations, higher air voids, and stabilizers such as fibers were used to control drain down. FHWA incorporated several modifications to the earlier versions of OGFC mix design and revised the mix design method in 1990. Some state agencies developed their own mixture design procedures that included a combination of the 1990 design method and experiences of Europe regarding OGFC mixtures.

When the Superpave mix design method was introduced and state agencies started adopting this technology for design of dense-graded mixtures, there was a need to incorporate the technology into the OGFC mix design method. The National Center for Asphalt Technology (NCAT) developed an OGFC mix design method based on comprehensive research along with European and state agency experiences (Kandhal, 2002). The method incorporated the Superpave specifications into the OGFC mix design method. This method was further refined by NCAT in 2004 and has been adopted by a number of state agencies with some modifications (Watson et al., 2003; Watson et al., 2004).

Some state agencies have sponsored research projects to develop their own OGFC specifications and design methods. For example, the Texas Department of Transportation (TxDOT) developed a procedure for designing OGFC in 2005 and incorporated further refinements for achieving durability in 2008 (Alveraz et al., 2006; Estakhri et al., 2008). ASTM has incorporated an OGFC design procedure in ASTM standard D7064/ D7064M *Standard Practice for Open-Graded Friction Course (OGFC) Mix Design*, which is almost the same as the NCAT method of OGFC design with minor variations.

2.1.1 *FHWA Mixture Design Method*

FHWA developed a mix design method for OGFC in 1974, revised it in 1990, and published a technical advisory T5040.31: *Open Graded Friction Courses* (FHWA, 1990). Both the original and revised design methods are based on the Marshall mix design procedures with a 12.5 mm maximum sieve size and include four design steps:

1. Estimating surface capacity of predominate aggregate fraction and optimum asphalt content;

2. Selecting gradation;
3. Selecting optimum mixing temperature; and
4. Evaluating mixture resistance to moisture susceptibility.

The first step is to estimate the surface capacity of the predominate aggregate fraction (passing the 9.5 mm (3/8 in.) sieve and retained on the 4.75 mm (No. 4) sieve) by immersing and draining the aggregate in S.A.E. No. 10 lubricating oil. The asphalt binder content is calculated using the surface constant value estimated from the percentage retained lubricating oil and the apparent specific gravity of the aggregate fraction.

The second step is to calculate the fine aggregate percentage of total aggregate (by weight) by taking into consideration the asphalt binder content and target air void content (15%). If the air voids content at the selected asphalt binder content is not 15% or higher, the gradation of the predominant aggregate fraction should be modified. The recommended gradation band is given in Table 1. The recommended grade of asphalt binder was an AC-20. Table 2 gives granular material specifications. Specifications regarding mineral filler are flexible and can be modified according to local requirements.

Table 1. FHWA Gradation Band for OGFC (FHWA, 1990)

Sieve Size, mm (in)	Passing by Weight (%)
12.5 (1/2)	100
9.5 (3/8)	95-100
4.75 (No. 4)	30-50
2.36 (No. 8)	5-15
0.075 (No. 200)	2-5

Table 2. FHWA OGFC Specifications for Granular Material (FHWA, 1990)

Parameter	Specified Value
Los Angeles abrasion, %, max	40
Fractured faces, one face, %, min	90
Fractured faces, two faces, %, min	75
Mineral filler's specifications	AASHTO M 17 or state standard

The third step is to establish the optimum mixing temperature by conducting a Pyrex glass plate test and keeping in view the draindown potential. This test can be conducted by spreading approximately 1000 grams of coated aggregate under specified conditions on a Pyrex glass plate (8 to 9 inches in diameter). The bottom of the glass plate is observed after 60 minutes. A slight puddle of asphalt at the points of contact between aggregate and glass plate is desirable, as shown in Figure 5; otherwise, the test is repeated at a higher or lower temperature. Several agencies use this draindown test to establish optimum asphalt content. In that case, the mixing temperature is held constant and the binder content is adjusted until the desired amount of draindown is obtained.

The last step is to evaluate the mixture's resistance to moisture susceptibility by the immersion-compression test (AASHTO T 165, *Effect of Water on Cohesion of Compacted Bituminous Mixtures*, and T 167, *Compressive Strength of Bituminous Mixtures*).

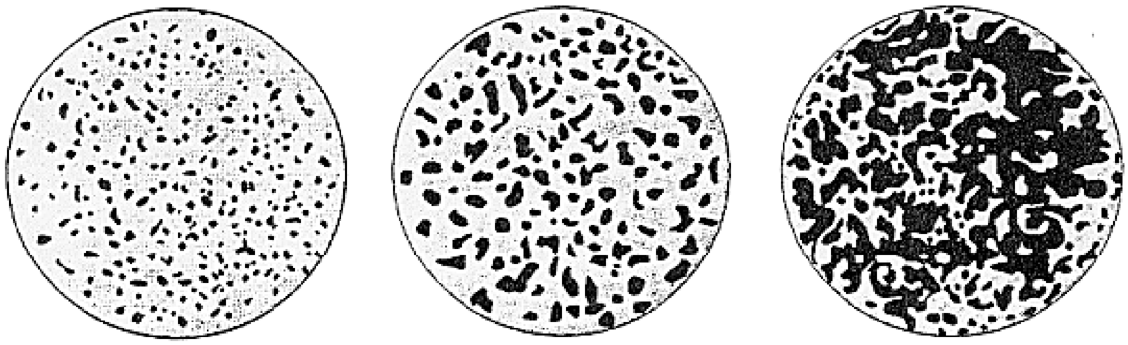

Figure 5. FHWA Test for Draindown Characteristics (FHWA, 1990)

2.1.2 *ALDOT Mixture Design Method*

ALDOT-259-97 "Open-Graded Asphalt Concrete Friction Course Design Method" is based on the FHWA method with differences in gradation band, sample compaction method, and moisture susceptibility testing. This method was last revised in 1999. The recommended gradation band is given in Table 3 with a maximum aggregate size of 19.0 mm. A vibrating table or vibrating rammer operating at 3600 cycles per minute is used to compact samples for determining the void capacity of coarse aggregate. Optimum binder content is based on a surface constant determined using the oil absorption test performed in the FHWA procedure.

Table 3. ALDOT Gradation Band for OGFC (ALDOT, 1999)

Sieve Size, mm (in)	Passing by Weight (%)
19 (3/4)	100
12.5 (1/2)	85-100
9.5 (3/8)	55-65
4.75 (No. 4)	10-25
2.36 (No. 8)	5-10
0.075 (No. 200)	2-4

All moisture susceptibility samples are compacted in a Superpave gyratory compactor (SGC) using 100 gyrations. Moisture susceptibility of the designed mixtures is checked using the modified Lottman method with no vacuum saturation or freeze/thaw cycle conditioning. The retained tensile strength of OGFC samples should be 80% for successful design.

2.1.3 *NCAT Mixture Design Method*

NCAT developed a comprehensive method for designing OGFC mixtures in 2000 (Mallick et al., 2000). This method was based on different approaches used in the United States, European methods, and additional research. This method was refined in 2005 to incorporate additional information. The method includes the following steps for designing an OGFC mixture (Kandhal, 2002):

1. Selection of materials;
2. Selection of design gradation;
3. Determination of optimum asphalt content; and
4. Evaluation for moisture susceptibility.

The first step is to select granular materials and a binder. Table 4 and Table 5 show the criteria for selecting granular materials and binders, respectively. PG 76-22 binder is typically used in southern parts of the U.S. and PG 70-28 used in colder, northern regions of the U.S.

Table 4. NCAT Specifications for Granular Material (Mallick et al., 2000)

Parameter	Specified Value
Los Angeles abrasion, %, max	30
Fractured faces, one face, %, min	100
Fractured faces, two faces, %, min	90
Flat and elongated particles, ratios of 3:1, %, max	5
Flat and elongated particles, ratios of 5:1, %, max	20

Table 5. NCAT Binder Selection Criteria (Kandhal, 2002; Mallick et al., 2000)

Recommended Type of Binder	Traffic Volume
High stiffness binders made with polymers; fiber addition is desirable	Medium to high
Polymer modified binders or fiber addition	Low to medium

The second step is to select a design gradation within the band recommended in Table 6 (Kandhal, 2002). The design gradation is selected to ensure a high air void content (18 to 22%) and provide stone-on-stone contact within the coarse aggregate fraction retained on the 4.75 mm (No. 4) sieve. Stone-on-stone contact is ensured by keeping the voids in the coarse aggregate of the compacted mixture (VCA_{MIX}) less than the voids in the coarse aggregate that is in the dry-rodded condition (VCA_{DRC}). The VCA_{DRC} is determined in accordance with AASHTO T 19: *Bulk Density and Voids in Aggregate*. In order to select the design gradation, three trial blends are developed. Each trial blend is then mixed with 6.0 to 6.5% asphalt binder and compacted with 50 gyrations of the Superpave gyratory compactor (SGC).

Table 6. NCAT Gradation Band for OGFC (Mallick et al., 2000)

Sieve Size, mm (in)	Passing by Weight (%)
19 (3/4)	100
12.5 (1/2)	80-100
9.5 (3/8)	35-60
4.75 (No. 4)	10-25
2.36 (No. 8)	5-10
0.075 (No. 200)	2-4

The third step is to select the optimum asphalt binder content based on laboratory tests conducted on compacted and uncompacted samples with different binder contents, and the last step is to determine the moisture susceptibility of the designed mixtures using the modified Lottman method (AASHTO T 283) with five freeze/thaw cycles. The retained tensile strength of OGFC samples should be 80% for successful design. Table 7 summarizes these tests and specification limits. All compacted samples are produced in the SGC using 50 gyrations.

Table 7. NCAT Criteria for Optimum Binder Content (Mallick et al., 2000)

Test Criteria	Specified Value
Cantabro Abrasion Loss on Unaged Specimens, %, max. (AASHTO TP 108/ASTM D7064)	20
Cantabro Abrasion Loss on Aged Specimens, %, max. (AASHTO TP 108/ASTM D7064)	30
Air Voids (%), min. (ASTM D6752)	18
Binder Draindown at 170°F, max. (AASHTO T 305)	0.3

Since previous mix design compaction in the U.S. and Europe was based on the 50-blow Marshall procedure, there was a need to relate laboratory Marshall results to SGC results. An NCAT study compared 25 and 50-blow Marshall density to SGC specimens compacted to 30, 45, and 60 gyrations (Watson et al., 2003). A ratio of the bulk mix specific gravity for SGC versus the 50 blow Marshall should approach 1.0 at a point where the two methods were equal. The SGC results for the traprock and granite mixes were equivalent to that of the 50 blow Marshall within a range of 45 to 53 gyrations as shown in

Figure 6. Gravel mixes had a higher density with 50-blow Marshall.

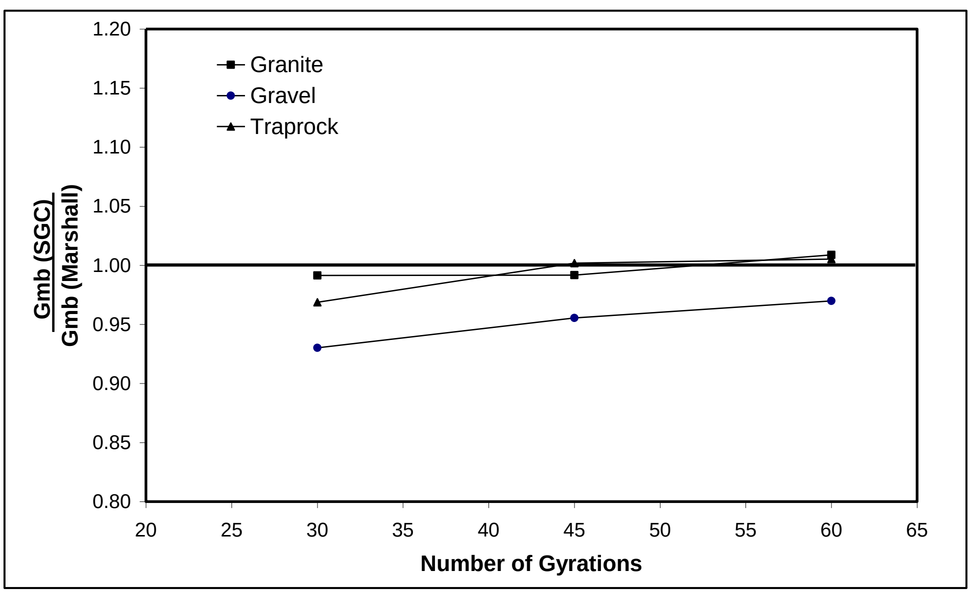

Figure 6. Relationship Between SGC and 50-Blow Marshall Density

The study also evaluated the use of the conventional dimensional method versus the CoreLok vacuum sealing method (with double bag procedure) for determining air voids in OGFC specimens. Double bags were needed when testing gyratory samples to prevent the sample edges from puncturing the plastic bags. The dimensional method resulted in air voids about 2 to 4% higher than the CoreLok method. This is largely due to the dimensional method assumption that the sample has smooth sides and does not consider volume loss due to surface texture.

SGC samples compacted to 50 gyrations were found to almost always have lower Cantabro loss than 50-blow Marshall specimens (Watson et al., 2003). As a result, it was recommended that a maximum of 15% loss be used as the acceptance threshold for unaged Cantabro samples compacted with the SGC. A follow-up study did not find a significant difference between results of conditioned and unconditioned samples and concluded that the conditioning procedure may not be necessary (Watson et al., 2004).

An analysis of the asphalt binder draindown procedure (AASHTO T 305) was also conducted by Watson et al. in 2003 using two basket mesh sizes: 2.36 mm (No. 8) and 4.75 (No. 4). It was believed that the larger openings of the 4.75 mm (No. 4) sieve may allow fine aggregate to fall through and be inappropriately considered as part of the binder draindown. The repeatability standard deviation of test results when the 2.36 mm (No. 8) mesh was used is only 60 percent as much as the standard deviation when the 4.75mm (No. 4) sieve was used. This would indicate that the 2.36 mm (No. 8) mesh size provides more repeatable measurements.

Earlier NCAT research had recommended five freeze-thaw cycles when determining moisture susceptibility of OGFC mixtures using AASHTO T 283. A follow-up study reported by Watson et al. in 2004 conducted moisture sensitivity testing using one, three, and five freeze-thaw cycles. The AASHTO T 283 procedure was modified by using a 10 minute vacuum saturation process followed by keeping samples submerged in water during the freeze period. The results showed that the tensile strength of conditioned samples was relatively unaffected by multiple freeze-thaw cycles (

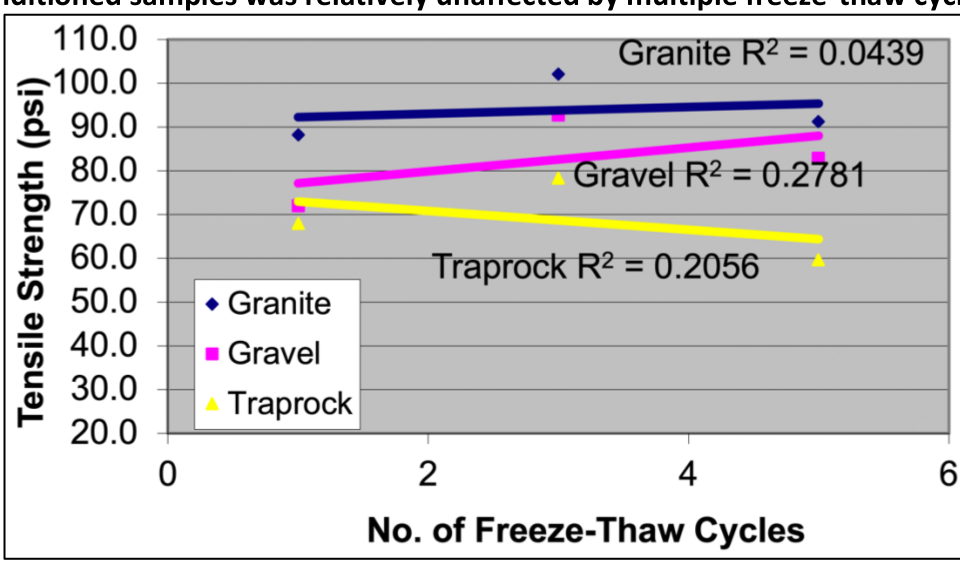

Figure 7) and recommended that only one freeze-thaw cycle was needed.

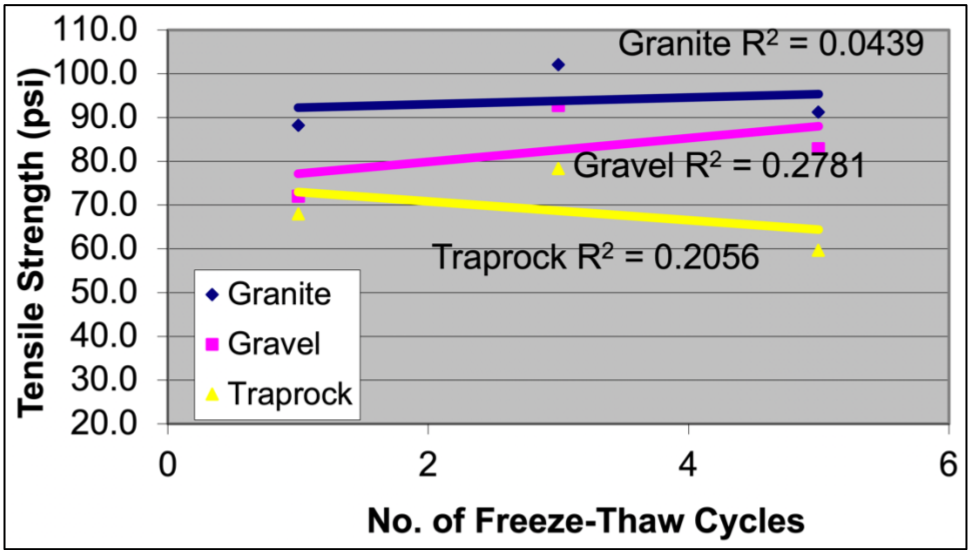

Figure 7. Tensile Strength after Multiple Freeze-Thaw Cycles

2.1.4 *TxDOT Permeable Friction Course Design Method*

TxDOT developed a comprehensive method for designing OGFC mixtures in 2008 and the steps involved in the design process are as follows (Alveraz et al., 2006):

1. Selection of materials;
2. Determination of optimum asphalt content based on target air voids and design gradation; and
3. Evaluation for draindown, moisture susceptibility, and durability.

The first step is to select binder and granular materials. TxDOT has different design criteria for PFC with performance graded (PG) binders and asphalt-rubber (A-R) binders (TxDOT, 2004). The PG binder type includes a polymer modified asphalt binder with a minimum high temperature grade of PG 76-XX and A-R binder type includes a TxDOT Grade C or B with a minimum of 15% crumb-rubber (by mass of asphalt binder). PFC mixes that utilize polymer modified binders include 1 to 2% hydrated lime and 0.2 to 0.5% fibers. Table 8 lists the criteria for selection of granular material.

The second step is to determine optimum asphalt content based on the target air voids and design gradation. The samples of the PFC are compacted in the SGC using 50 gyrations. The optimum binder content is selected based upon the target air voids of 18 to 22% and minimum asphalt binder content requirement of 6%.

Table 8. TxDOT Specifications for Coarse Aggregate (TxDOT, 2004)

Parameter	Specified Value
Deleterious material, %, max	1
Decantation, %, max	1.5
Los Angeles abrasion loss, %, max	30
Magnesium sulfate soundness, 5 cycles, %, max	20
Fractured faces, two faces, %, min	95
Flat and elongated particles, ratios of 5:1, %, max	10

The third step is to evaluate the designed PFC mixtures for draindown, moisture susceptibility, and durability. A maximum draindown of 0.20% is specified using a draindown basket (AASHTO T 305). Moisture susceptibility is evaluated by boiling loose mixture in water for 10 minutes and visually evaluating the percentage of stripping immediately after boiling and again after 24 hours (Tex 530-C). Durability is evaluated using the Cantabro test (AASHTO TP 108) with a maximum loss requirement of 20%.

A research study conducted for TxDOT at Texas Transportation Institute in 2008 made the following recommendations for improvement in mix design of OGFC (FHWA, 2008):

- The dimensional analysis for determining both bulk specific gravity (G_{mb}) based on total air voids and water accessible air voids of PFC mixtures was recommended.
- The calculation of density and total air void content of compacted specimens at trial AC was modified by measuring theoretical maximum specific gravity (G_{mm}) at two low asphalt contents (i.e., 3.5 and 4.5%).
- Field evaluation of initial permeability rate of PFC pavement was recommended due to differences in field and lab permeability values. A maximum water flow value (WFV) of 20 seconds was permitted in the field to ensure required permeability.
- The density specification was reduced by 2% (from 78-82% to 76-80%) to ensure proper drainability.

2.1.5 *Other State Mixture Design Methods*

Although ASTM and AASHTO have developed a standard for the design of OGFC mixtures in the United States, many states have developed their own specifications using a combination of FHWA, NCAT, AASHTO, and/or ASTM guidelines. Most states use a polymer modified binder (PG 76-22) with or without fibers to control draindown (Jackson et al., 2008). Table 9 shows a summary of OGFC mix specifications adopted by different state agencies.

Some states use dual specifications based on whether PG or A-R binder is used. Arizona and California are the exceptions, which restrict themselves to A-R binder in OGFC. The states using A-R binder in OGFC mixtures specify a higher percentage of A-R binder as compared to states using polymer modified binder. Some states, like Arizona, allow cement or lime as modifiers to control draindown. Most states use the OGFC layer as a final wearing surface allowing only virgin and crushed aggregates. Most states specify aggregate type, and some states, like Arizona, also specify percentage of carbon in aggregate to avoid polishing at an early stage. Most of the states use tack rates 50% higher than used for dense graded HMA. Almost all of the states specify air temperature ranging from 60^{o}F to 70^{o}F for placing OGFC.

Table 9. Summary of OGFC Requirements for U.S. Agencies (Jackson et al., 2008)

Grading	WSDOT Class D	WSDOT OGFC Test	WSDOT OGFC-AR Test	Alabama OGFC	Arizona ARFC	California O-G RAC	Florida FC-5	Georgia OGFC 12.5mm	Idaho PMS-OG	Indiana OGFC OG19.0	Nevada OGFC
3/4"				100		100	100	100		70-98	
1/2"	100			85-100		95-100	85-100	85-100	100	40-68	100
3/8"	97-100	100	100	55-65	100	78-89	55-75	55-75	95-100	20-52	90-100
#4	30-50	35-55	30-45	10-25	30-45	28-37	15-25	15-25	30-50	10-30	35-55
#8	5-15	9-14	4-8	5-10	4-8	7-18	5-10	5-10	5-15	7-23	
#16						0-10					5-18
#200	2-5	0-2.5	0-2.5	2-4	0-3	0-3	2-4	2-4	2-5	0-8	0-4
% Asphalt	4-6	9	9	5.6-9			ARB12	5.75-7.25			
Asphalt	PG 58-22	PG 70-22	A-R	PG 76-22	A-R	A-R	PG 76-22	PG 76-22			
Min Air Temp.	55° F	55° F	55° F	40° F	70° F *	70° F	65° F	55° F	60° F	60° F	
					* + 85° F Surface						
Grading	**New Jersey OGFC**	**New Mexico OGFC I & II**	**New Mexico OGFC III**	**North Carolina OGFC FC-1 Mod**	**North Carolina OGFC FC-2 Mod**	**Oklahoma OGFC**	**Oregon OGM 1/2"Open**	**Oregon OGM 3/4" Open**	**South Carolina OGFC**	**Texas PGFC**	**Texas A-R**
3/4"		100	100		100		99-100	85-96	100	100	100
1/2"	100	100	70-90	100	85-100	100	90-98	55-71	85-100	80-100	95-100
3/8"	80-100	90-100	40-65	75-100	55-75	90-100			55-75	35-60	50-80
#4	30-50	30-55	15-25	25-45	15-25	25-45	18-23	10-24	15-25	1-20	0-8
#8	5-15			5-15	5-10		3-15	6-16	5-10	1-10	0-4
#10		0-20	6-12			0-10					
#40		0-12	0-8								
#200	2-5	0-6	0-5	1-3	2-4	0-5	1-5	1-6	0-4	1-4	0-4
% Asphalt				5-8	5 - 8				5 - 7	5.5-7.0	8-10
Asphalt				PG 76-22	PG 76-22				PG 76-22	PG 76-22	A-R
Min Air Temp	60° F	70° F	70° F			60° F		60° F	60° F	70° F	70° F

2.1.6 *European Mixture Design Methods*

A porous European mixture (PEM) is a coarse open-graded mixture with air voids between 18 and 26%. These mixtures typically have 3 to 6% polymer modified binder and 3% mineral filler. The Marshall compactor is used for mix design in most European countries, except in France and Switzerland where both Marshall and Gyratory compactors are used. Different methods have been used to determine the optimum binder content based on noise reduction, permeability, splash and spray reduction, skid resistance, and durability in the Netherlands, whereas it is based on Cantabro test results in Belgium and Spain. In Denmark, optimum binder content is based on a draindown test, mechanical tests such as the Hamburg wheel tracking test and the rotating surface abrasion (RSA) test. In the United Kingdom, Switzerland, Italy, and France, it is based on air voids and durability tests. Some tests, such as indirect tensile (IDT) strength, semi-circular bending (SCB), and splitting tensile (ST) tests have been used for checking the durability of PEM. Stabilizers such as styrene butadiene styrene (SBS), cellulose fibers (CF), hydrated lime (HL), limestone filler (LF), and ethylene-vinyl acetate (EVA) were used to control draindown and enhance performance life. Typical maximum aggregate (MA) size varies (20, 16, 11, and 10 mm).

Table 10 summarizes various important features of PEM used in Europe (Huber, 2000; Alveraz et al., 2006; Brousseaud et al., 2005). Most European countries take into account the laboratory and field permeability in evaluation of PEM.

Table 10. Summary of PEM Design Methods (Huber, 2000; Alveraz et al., 2006; Brousseaud et al., 2005)

Method	Blows Marshall /Gyratory	Air Voids (%)	Binder Content Based On	Binder Type/ Modifications	Test	Aggregate Size/ Gradation
Danish	-	26 (Target)	Draindown test, HWTD and RSA test	Modified (50/100 pen grade with SBS, CF, HL, LF)	Draindown, HWTD and lab and field permeability	
Netherlands	50	20 (Min)	Functional properties (normally 4.5%)	Modified (pen grade with HL)	Retained IDT, RSA, Cantabro test and SCB	PA 0/11 mm and PA 0/16 mm (No. after PA shows max aggregate size in mm), 50 mm thickness
Belgian	-	21 (Min)	Cantabro test	-	Cantabro abrasion (loss < 20%) and field permeability	Stone fraction > 2 mm: 81-85 % Sand fraction (0.063 mm -2 mm) : 11-13% Filler fraction 4-6%
British	50	20 (Min)	Binder drainage test (3.7-4.5%)	Unmodified (pen grade 100/150 or 160/220) and modified (SBS, EVA)	Field Permeability before trafficking	6/20 mm
Spanish	50	20 (Min)	Cantabro test (dry and wet), draindown (typically 4.5%)	Unmodified (80/100 pen grade) and modified (SBS, EVA)	IDT, Wheel tracking and lab permeability	10/12.5 mm and 5/12.5 mm
Swiss	50/40	18-22	3-5	Unmodified (pen grade) and modified (polymer)	Evaluation of mixture drainage capacity and ST for moisture susceptibility	PA 11 and PA 8 (No. after PA shows max aggregate size in mm)
Italian	-	18-23	4-6	Modified (pen grade 80/100 with SBS)	Cantabro abrasion (< 30%)	16 mm NMAS (all aggregate should be crushed, natural sand not allowed)
French	25 of GSC	20-25 for class 1 and 25-30 for class 2	Typical (4.5-6)	Modified (acrylic, glass or cellulose fibers)		10 mm NMAS gradation with gap grading between 2 and 6 mm. Filler fraction 3-4%

2.1.7 *Australian Mixture Design Method*

Mixture design methods used in Australia are comprised of three types of open graded mixes: open graded asphalt (OGA) for light/medium traffic, OGA for heavy/very heavy traffic, and ultra thin asphalt (UTA) (AAPA, 2004). UTA is a thin layer of asphalt (12-15 mm compacted thickness) that uses modified grading for improving the resistance to surface shearing forces. Australian agencies have shifted to the Australian gyratory compactor for the design of porous asphalt along with the Marshall method. The maximum aggregate size used in Australia is 10 mm for UTA, 14 mm for OGA light/medium, and 20 mm for OGA heavy/very heavy. A modified binder used with fibers or hydrated lime is recommended for OGA with heavy/very heavy traffic. The minimum asphalt binder content is established through the Cantabro test while maximum content is determined based on air voids and draindown test. Table 11 summarizes the requirements for designing porous asphalt in Australia (Alderson, 1996).

Table 11. Australian Design Parameters for Porous Pavements (AAPA, 2004)

Test Criteria		OGA Light/ Medium	OGA Heavy/ Very Heavy
CA Unconditioned, %, max		25	20
CA Moisture Conditioned, %, max		30	35
Air Voids, %		18-23	20-25
Binder Draindown at 170 °F, max		0.3	0.3
For Laboratory Compaction	Gyratory (cycles)	80	80
	Marshall (blows)	50	50

*CA (Cantabro Abrasion)

2.2 Laboratory Performance Testing

A graphical description of the research plan for this study is shown in Figure 8.

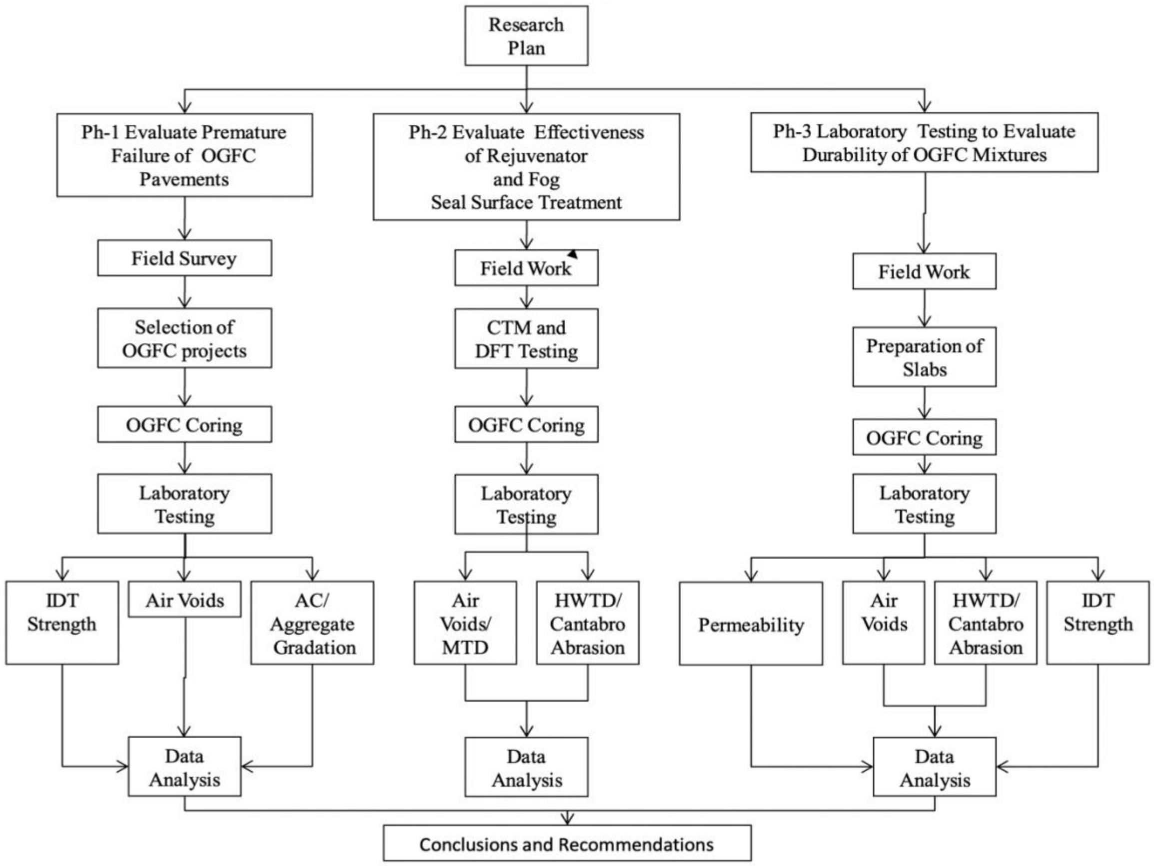

Figure 8. Project Research Plan for Projects Selected for Field Review

During the dense graded HMA mixture design process, strength and durability of the mixture are evaluated. With OGFC mixtures, functionality replaces strength, yet strength still remains important in the mixture design process. The durability of HMA is a measure of how the physical properties of asphalt binder change with age (Pavement Interactive, 2010). It encompasses the ability of an HMA mixture to retain its inherent properties and resist aging caused by four main factors: air, water, temperature, and traffic. These main factors affect binder oxidation, volatization, polymerization, thixotropy, syneresis, and separation, which influence the rheological properties of the binder and durability of HMA pavements over time (Brown et al., 2009). The strength measures the mixture's resistance against permanent deformation (rutting) during construction and over the service life of the pavement. Durability provides the mixture's resistance against cracking (fatigue and thermal) and water damage (Mogawer et al., 2004). The importance of durability is clear for OGFC mixtures as cracking and water related damage normally occur on OGFC pavements.

HMA durability is related to air voids as well as asphalt binder thickness surrounding each particle (Christensen, 2006). Air voids play a key role in long-term durability of HMA. High air voids, in the case of OGFC mixtures, will result in easy access of water and air into the pavement and increase the potential for moisture damage, oxidation, cracking, and raveling (Brown et al., 2004). To ensure durability of the HMA mixture, the asphalt binder needs to have a strong bond with the aggregate. The durability of that bond, affected by the intrusion of water into the HMA mixture, decreases the anti-stripping capability of the asphalt film (Sengoz and Topal, 2007). To enhance the anti-stripping capability of OGFC, fibers and polymers are used. Faghri (2002) pointed out that the strength of porous mixes increased significantly by the addition of polymers and minutely by the addition of fibers. Poulikakos et al. (2003) compared two open graded mixes through various mechanical tests and confirmed that polymer modification increases IDT strength. Hamzah et al. (2010) compared performance of Malaysian porous asphalt mixes incorporating conventional and modified binders in terms of permeability, air voids, abrasion loss, and IDT strength. He used a general linear model (GLM) through analysis of variance (ANOVA) to identify the significant effect of aggregate grading, bitumen type, and bitumen content on performance properties of porous mixes. He reported that modified binder increased the resistance to abrasion and improved IDT strength of the mixes.

The Hamburg Wheel Tracking Test was introduced in the United States as a result of the 1990 European Asphalt Study Tour. The device was manufactured and first used in Hamburg, Germany as a performance test to evaluate both rutting and moisture susceptibility of asphalt mixtures in Europe (Larson, 1991). Initial testing of the device by the Turner Fairbanks Research Laboratory of the FHWA and by the Colorado Department of Transportation gave favorable reviews and showed that the device was sensitive to aggregate properties, asphalt binder source, testing temperature, and short-term aging (Kandhal, 2002). The procedure is described in AASHTO T 324-04 (2008): *Hamburg Wheel-Track Testing of Compacted Hot Mix Asphalt (HMA)* and requires a steel wheel with a 158 lb. load traveling in a reciprocal motion over an immersed sample. The test provides a graph of deformation over 20,000 wheel passes. If a mixture is susceptible to moisture damage, it will have both a rutting slope and a stripping slope as indicated in Figure 9.

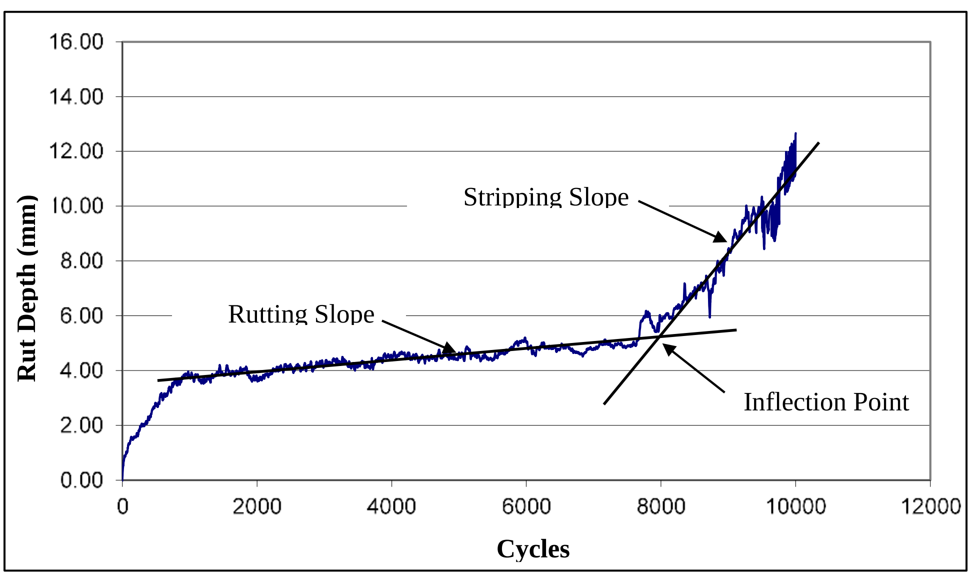

Figure 9. Hamburg Wheel Tracking Test Results

The Cantabro test is a European procedure that indicates mixture resistance to wear and raveling and is described in AASHTO TP 108-14 *Standard Method of Test for Determining the Abrasion loss of Asphalt Mixture Specimens*. It has been recommended as a standard OGFC mix design procedure based on previous NCAT research (Watson et al., 2004), in ASTM D7064/ D7064M-08 *Standard Practice for Open-Graded Friction Course (OGFC) Mix Design,* and in AASHTO PP 77-14 *Standard Practice for Materials Selection and Mixture Design of Permeable Friction Courses (PFCs)*. The procedure requires that compacted specimens be placed in a Los Angeles (LA) abrasion machine without the normal steel charges and then tumbled in the drum for 300 revolutions. Figure 10 shows an example of a specimen before and after the Cantabro test. The European specifications require a maximum stone loss of 20 percent for unaged samples. Samples are also typically aged in a fan forced-draft oven at 60°C for seven days. After this aging process, the stone loss of compacted samples is limited to a maximum of 30 percent. The European procedure was based on compacted Marshall samples. NCAT research using Superpave gyratory-compacted samples recommended a maximum stone loss for unaged samples of 20 percent (Kandhal, 2002). Analyzing the result of core specimens that have been exposed to environmental aging may provide more information as to whether an aging procedure is needed in the laboratory mix design stage.

Figure 10. Before and After Cantabro Test Results

Tensile strength ratio is described in AASHTO 283, *Resistance of Compacted Asphalt Mixtures to Moisture-Induced Damage,* and is used to determine susceptibility to moisture damage. The IDT strength test has been widely used for HMA mixture design and is used by some European countries i.e., The Netherlands and Spain, for porous pavement design. The results are used to obtain the comparative strength of OGFC mixtures and predict rutting and raveling potential.

Since the performance of porous mixtures depends on tensile strength of the binder, IDT strength is also an important test for OGFC mixtures (Cetin, 2013). The IDT test is performed by using Marshall Stability test equipment on a cylindrical specimen. Average compressive deformation rate of the sample should be around 50 mm/min and the test should be conducted at a temperature of 25°C. The specimen is failed by tensile stress developed perpendicular to the direction of the applied load. The ultimate failure load is used to calculate the IDT strength of the specimen.

$$S_t = 2P/\pi Dt \tag{1}$$

Where:

S_t = Tensile Strength (psi);
P = Load (lbf);
D = Sample diameter (inches); and
T = Sample thickness (inches).

For moisture susceptibility testing, AASHTO PP 77-14: *Standard Practice for Materials Selection and Mixture Design of Permeable Friction Courses (PFCs)* recommends one freeze/thaw cycle in accordance with AASHTO 283 as opposed to the recommended five cycles in ASTM D7064-08. A laboratory permeability test has been recommended as an optional test by NCAT and in both the ASTM D7064 and AASHTO PP77-14 procedures. A laboratory falling head permeability test was previously described in ASTM PS 129-01: *Measurement of Permeability of Bituminous Paving Mixtures using a Flexible Wall Perimeter.* The ASTM test has been deleted from the standard ASTM test procedures, but many agencies still use the procedure (or an adaptation) as an optional procedure.

3 PHASE 1 - CONDUCT FIELD SURVEY

3.1 Selection of Projects

IRI data and project level videos from the ALDOT pavement management system were analyzed to determine the nature of distresses on various OGFC projects and determine whether the problem was localized or consistent throughout the section. On the basis of the distress survey and IRI data analysis, 13 projects (Table 12) were selected for review of mix design, specifications, and construction information.

Table 12. Projects Selected for Review

Project No.	Route	Age (Years)	Div.	County	Project No. for Coring
IM-I059(349)	I-59	1 (paver laid tack coat)	1	Etowah	
NHF-0004(513)	US 78	1	2	Marion	
EBF-0004(512)	US 78	2	2	Marion	
I065(315)	I-65	5 (paver laid tack coat)	1	Cullman	5 (B, G)
IM-NHF-I020(320)	I-20	4	3	St. Clair	
IM-1085(302)	I-85	6	6	Macon	2 (T)
ST-013-013-007	US 43	4	8	Clarke	
IM-I059(316)	I-59	8	5	Greene	1 (B, T, G)
IM-85-1(136)	I-85	7	6	Macon	
IM-65-1(257)	I-65	10	9	Conecuh	3 (B, G)
IM-65-1(253)	I-65	10	6	Butler	
IM-59-1(225)	I-59	9	8	Sumter	4 (B, G)
NHF-372(28)	US 82	11	5	Tuscaloosa	

Out of these, five projects were selected for coring to represent the various aggregate types used in Alabama with the exception of project 5 constructed with paver-laid tack coat. The selected projects for coring were numbered from 1 to 5 in the sequence they were cored. Good (G) and bad (B) portions from projects 3, 4, and 5; good (G), transition (T) and bad (B) portions from project 1; and transition (T) portions from project 2 were selected on the basis of a field distress survey and IRI data analysis. A section was termed as a good performing portion based on severe intensity raveling less than 2%, medium intensity raveling between 2 and 10%, and low severity raveling between 10 and 15%. A section was termed as a bad performing portion if any two of the above conditions were not fulfilled. A section was termed a transition performing portion if any one condition was not fulfilled. Raveling was measured according to the distress identification manual (Miller and Bellinger, 2003). The length of each portion for coring was 100 meters. The list of the five projects selected for review of mix design, specifications, construction information, and coring is shown in **Error! Not a valid bookmark self-reference.**. The graphical location of projects to be cored is shown in Figure 11.

Table 13. Materials Used and Condition of Projects for Coring

Project No.	Route	Age (Years)	Aggregate Type	Binder Type	Thickness (mm)	Condition of the Project Portion+
1	I-59	8	Sandstone	PG 76-22	22-40	B, T, G
2	I-85	6	Granite	PG 76-22	20-23	T
3	I-65	10	Slag	PG 76-22	16-23	B, G
4	I-59	9	Slag with sandstone	PG 76-22	18-32	B, G
5	I-65	5*	Slag with sandstone	PG 76-22	15-27	B, G

* Paver laid tack coat + B (bad), T (transition), G (good)

Six cores of 150 mm diameter were each extracted at a spacing of 20 meters from each project/site bad (B), transition (T) or good (G) portion for laboratory testing. One out of six cores for each specific project portion was taken from the wheel path and the other five from the center of the lane. OGFC wearing course was separated/cut from full depth cores with a masonry saw.

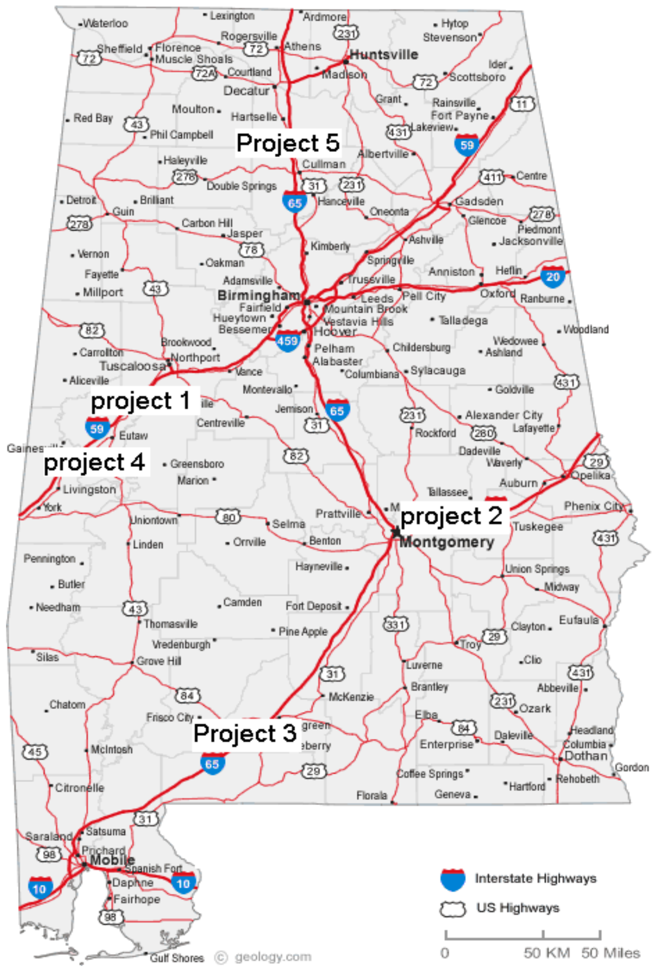

Figure 11. Location of Projects for Coring

3.2 Review of Project Records

3.2.1 *Tack Coat*

One possible cause of raveling is inadequate or improperly applied tack coat. For that reason, project records were reviewed to evaluate reported tack rates. A total of 69 reports were reviewed for projects that used conventional emulsion and 44 reports were reviewed on paver-laid projects that used a spray-paver for applying emulsion tack coat.

ALDOT specifications previously required tack be applied within the range of 0.05-0.10 gal/sy if a conventional emulsion is used. However, these rates include the portion of water included in the emulsion. The residual rate was 0.03-0.07 gal/sy. Figure 12 shows the average residual application rate on the projects reviewed that used conventional emulsion. These applied rates are lower than typically used for OGFC pavements, but

ALDOT specifications allow the engineer to reduce the rate when tacking new, freshly laid pavement. On projects where the spray paver is used, CQS-1HP asphalt emulsion is required and the application rate ranges from 0.18-0.23 gal/sy. This will result in a residual asphalt rate of 0.12-0.15 gal/sy once the water content is accounted for.

Figure 13 shows the range of tack application rates used where spray paver-laid OGFC was placed on the projects reviewed. By comparing Figure 12 and Figure 13, it is easily seen that the tack application for spray paver-laid OGFC is approximately four to five times that of projects that used conventional distributor application. For comparison, Table 14 shows tack application rates used in the southeast for OGFC based on residual asphalt content.

The minimum revised rate was revised with the 2018 specifications and only CQS-1hp and PG trackless asphalt are used as tack coat materials for OGFC. The residual rate for CQS-1hp is 0.12 – 0.15 gal/sy and for trackless tack is 0.13 – 0.18 gal/ sy. CQS-1hp is required when the contractor uses a spray paver for mix placement, and PG trackless tack is required when a conventional paver is used.

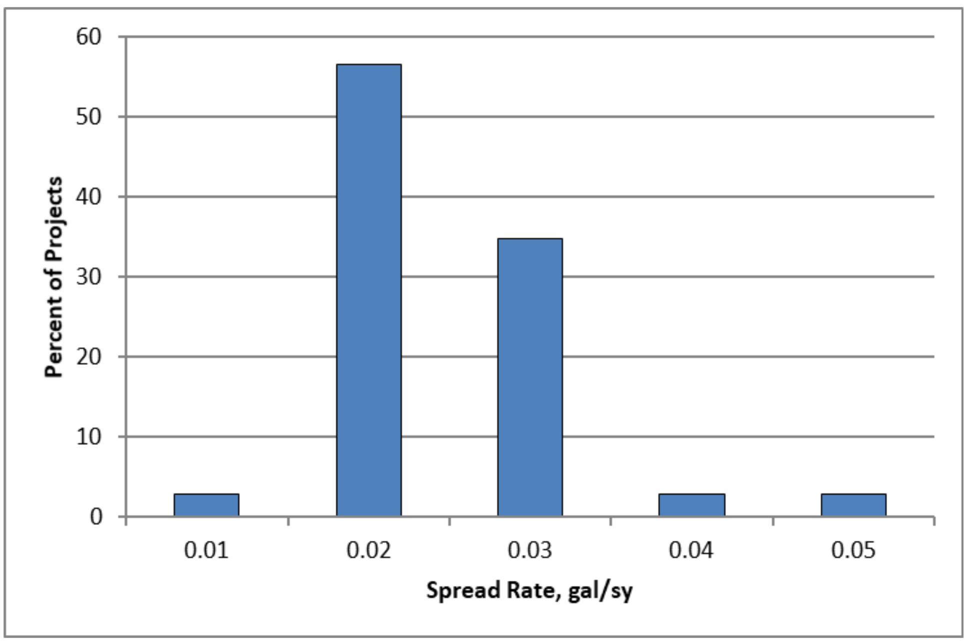

Figure 12. Average Residual Application Rate on Conventional Tacked Projects

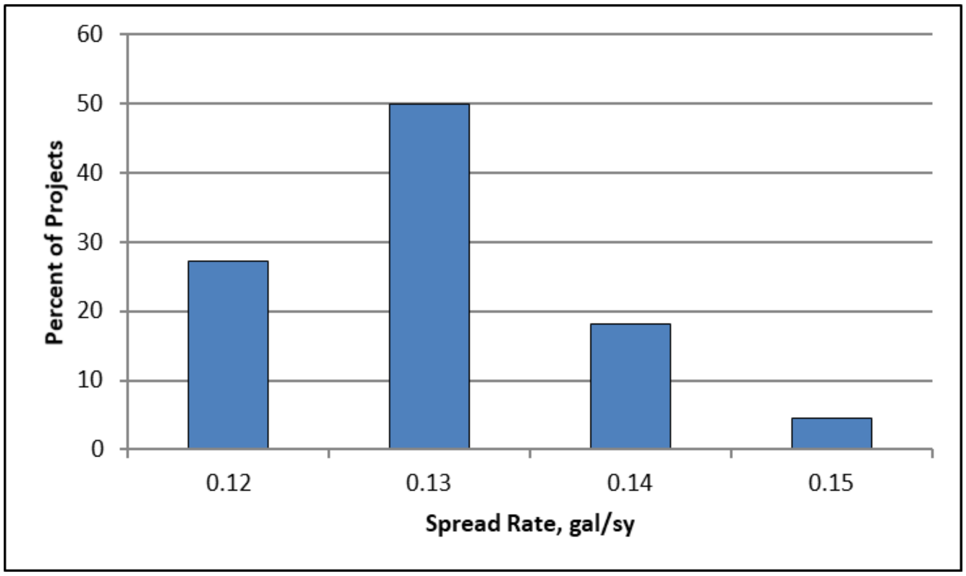

Figure 13. Average Residual Application Rate on Spray Paver-Laid Projects

Table 14. Comparison of Tack Application Ranges for OGFC

State	Tack Rate Range, Residual gal/sy
Alabama	Previously 0.03 – 0.07 (revised 2018 to 0.12 – 0.15 for CQS-1hp and 0.13 – 0.18 for PG Trackless tack)
Georgia	0.06 - 0.08
Florida	0.06 Minimum
Mississippi	0.07 - 0.09
North Carolina	0.06 - 0.08
South Carolina	0.03 - 0.10
Tennessee	0.03 - 0.07
Texas	0.04 - 0.10

3.2.2 *Weather Limitations*

ALDOT specifications limit placement temperatures to no less than 60°F for surface or air temperature when placing polymer modified mixtures at a spread rate of 200 lbs/sy or less. However, reports indicate that OGFC was placed in cold weather on some projects. In at least one case, the daily temperature during placement ranged from 33 to 59°F. Waiving the low temperature requirement was sometimes permitted in order to complete the project so it wouldn't have to be maintained over the winter. In one case, the paver broke down and had to be repaired. By the time placement restarted, truckloads of mixture had been sitting at the project site for three hours. This sometimes led to cold lumps that had to be removed by shovel and replaced with hot mix (Figure 14). In another case, the project was delayed due to construction problems and loads of mixture had been on the site with temperatures in the range of 44 to 57°F for four hours before being unloaded. Some of the mix in this area started to ravel out within 24 hours of placement and had to be patched as shown in Figure 15.

Figure 14. Cold Lumps Removed From OGFC

Figure 15. Patched Area Where Mix Placed in Cold Weather Had Raveled

4 LABORATORY TESTING OF FIELD CORES

All six cores for each specific project portion were tested for bulk specific gravity (G_{mb}), indirect tensile (IDT) strength, G_{mm}, percentage asphalt content, and aggregate gradation. Percentage air voids was calculated using G_{mb} and G_{mm} data. G_{mb} was determined according to AASHTO T 331-11 *Standard Method of Test for Bulk Specific Gravity (G_{mb}) and Density of Compacted Hot Mix Asphalt (HMA) Using Automatic Vacuum Sealing Method*. All cores were then tested for IDT strength of bituminous mixtures according to AASHTO T 283 *Standard Method of Test for Resistance of Compacted Asphalt Mixtures to Moisture Induced Damage* except that samples were kept submerged in water during the freeze cycle. The theoretical maximum specific gravity according to AASHTO T 209-10 *Standard Method of Test for Theoretical Maximum Specific Gravity and Density of Hot-Mix Asphalt (HMA)* was then performed. Asphalt was extracted from samples according to ASTM D2172-05 *Standard Test*

Methods for Quantitative Extraction of Bitumen from Bituminous Paving Mixtures. Gradation of the aggregate was done according to AASHTO T 27-06 *Standard Method of Test for Sieve Analysis of Fine and Coarse Aggregates.*

4.1 Results and Discussion

4.1.1 *Thickness Data*

Analysis of thickness data was conducted to determine the effect of thickness on the performance of OGFC. The cores ranged between 13.2 mm and 44.9 mm thick with a mean value of 24 mm. Figure 16 shows a boxplot of the selected project (B, T, G) core thickness values. The wide variation in thickness provides a chance to differentiate good and bad portions on the basis of pavement thickness. Analysis of variance (ANOVA) tests were run on test data for assessing pavement thickness based on the difference between bad and good performing sections of each project. Important ANOVA estimated statistics are tabulated in

Table 15.

The result of ANOVA tests at the 95% confidence level (*P*-value < 0.05) showed that the variation of thickness on project 3 ($P = 0.034$, $F = 6$), project 4 ($P = 0.003$, $F = 15.63$), and project 5 ($P < 0.001$, $F = 38.86$) was statistically significant. In the case of project 1 (*P*-value > 0.05), variation of thickness was statistically insignificant ($P = 0.114$, $F = 2.79$). Project 2 was not included in the ANOVA because it did not have sections that met both the good and bad criteria. The good portions typically had greater thickness in comparison with bad portions of all projects except project 4.

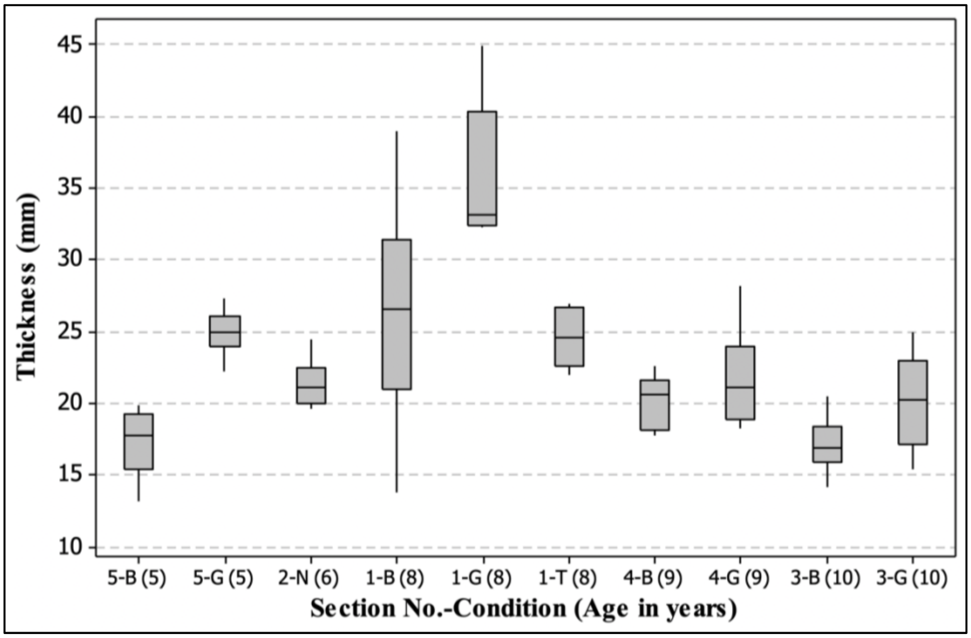

Figure 16. Boxplot of Project Core Thickness

Table 15. Important ANOVA Estimated Statistics of Project Core Thickness

Response	Factors		Sample Size (N)	F-value	p-value
	Project	Condition			
Thickness	5	B, G	12	38.86	0.000
	1	B, G	18	2.79	0.114
	4	B, G	12	15.63	0.003
	3	B, G	12	6.00	0.034

The results indicate that thickness is an important property relating to the cause of premature failure of OGFC pavements. Thickness in the range of 25-33 mm seems to perform better as compared to thicknesses between 15-25 mm. The increase in thickness seems to improve internal cohesion through aggregate particle interlock. Additional thickness, along with other factors, strengthens the bond strength of OGFC mixtures, which is helpful in improving resistance to raveling. The overall thickness/NMAS ratio on all the projects was around 1.4. This is less than the 2.5 ratio recommended to develop sufficient internal cohesion for good performance based on a review of OGFC performance on the NCAT Test Track.

4.1.2 *Air Void Measurements*

An analysis of air voids was conducted to determine their effect on the performance (good or bad) of OGFC pavements. The air voids of the cores are calculated based on the G_{mb} and G_{mm} of specimens and ranged between 13.3% and 25% with a mean value of 19.5%. Figure 17 shows a boxplot of the air voids.

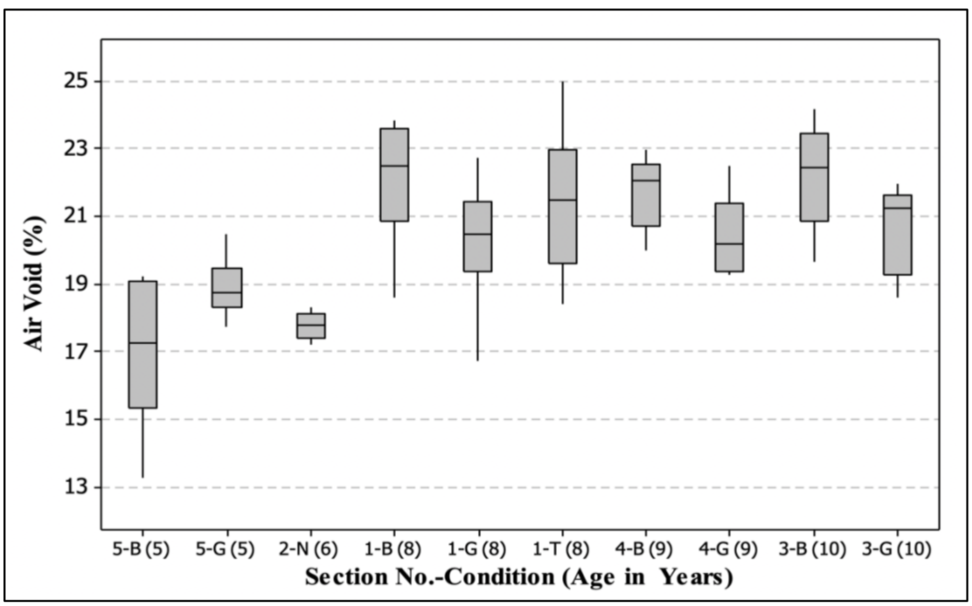

Figure 17. Boxplot of Project Air Voids

All of the projects were designed with target air voids of 18 to 22%. The record of average air voids during construction is not required for OGFC projects, so a possible reduction in air voids cannot be discussed. The air voids of good portions were lower in comparison to bad portions for all the projects except for 5, as illustrated in Figure 17. The higher air voids in projects 1, 3, and 4 can be explained on the basis that the projects are older and higher air voids may be the result of more severe raveling. The age of projects 2 and 5 are less (six and five years, respectively) as compared to other projects in the study and the lower air voids may indicate that raveling is not yet as severe as for the other projects.

An ANOVA was run on project data to differentiate bad and good performing sections on the basis of pavement air voids. In this ANOVA test, the response was air voids and the effects included project and condition. The results of the ANOVA are shown in Table 16. The results at the 95% confidence level (*P*-value < 0.05) showed that the variation of air voids was statistically insignificant in determining the performance (good or bad) of OGFC pavements although there appeared to be a trend of reduced air voids in good performing sections as compared to bad performing sections. The OGFC project portions seem to perform well within the air voids range of 18-21%.

Table 16. Important ANOVA Estimated Statistics of Project Air Voids

Response	Factors		Sample Size (N)	F-value	P-value
	Project	Condition			
Air Voids	5	B, G	12	3.65	0.085
	1	B, G	18	1.46	0.245
	4	B, G	12	1.26	0.288
	3	B, G	12	6.00	0.293

4.1.3 *Indirect Tensile (IDT) Strength*

IDT strength data was analyzed to determine the effect of tensile strength or stiffness on the performance (good or bad) of OGFC pavements. The stiffness of OGFC mixtures is generally 50% of conventional HMA mixtures (stiffness depends upon VMA; the lower the VMA, the higher the stiffness). It was not possible to get initial IDT strength data of the mixes used on the various projects since IDT strength is not a compulsory test in the ALDOT design method. Generally, design IDT strength of OGFC mixtures should lie in the range of 250-400 KPa (36-58 psi) (Cetin, 2013) depending upon air voids.

The IDT strength values of the cores from all project locations ranged between 180 KPa and 1000 KPa (26-145 psi) with an average of 580 KPa (84 psi). The probable cause of variability in IDT strength values within project portions is reduced cohesion in weak spots and slight variability in thicknesses of the cores. Although AASHTO T 283 does not specify a minimum core thickness, ASTM D6931 recommends a minimum thickness of 38 mm (1.5 in) when determining tensile strength. Figure 18 shows the boxplot of project core IDT values.

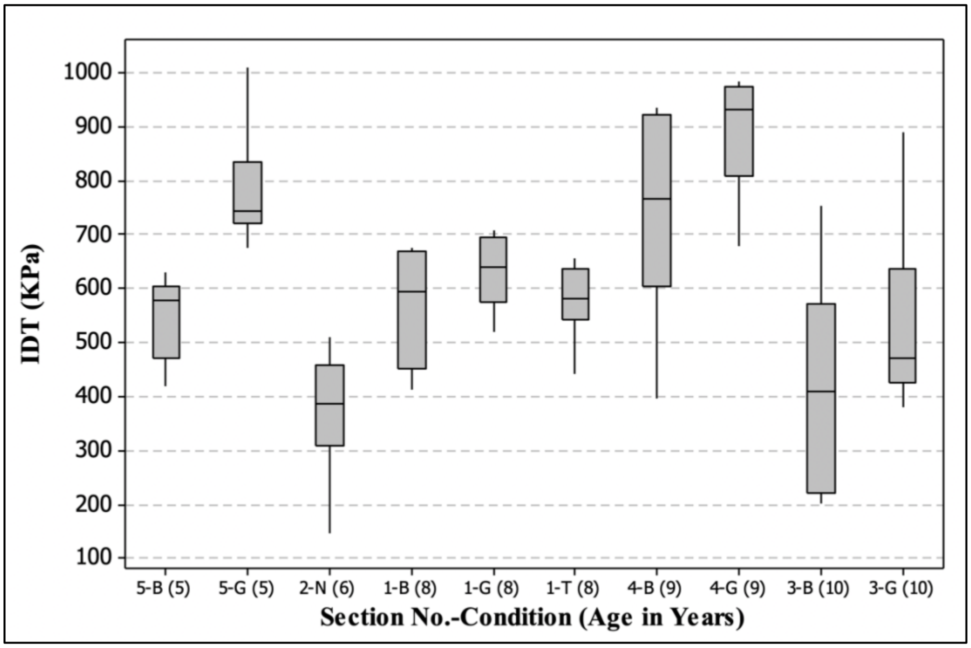

Figure 18. Boxplot of Project IDT Strength

By comparing Figure 16 and Figure 18, the general trend is to have higher tensile strength values with thicker cores. This is expected, as the tensile strength test is sensitive to the thickness of cores. There is no specific trend of tensile strength with the age of the section. Also, good performing portions of a project have higher strength as compared to bad performing portions of the same project. This can be linked to lower air voids, which ultimately tends to increase the tensile strength values. Although air voids are higher in the good performing portion of project 5 as compared to the bad performing portion, tensile strength values are still higher in the good portions. This shows tensile strength values can be considered as a criteria for better performing sections. An interesting observation is made regarding a comparison of project 5 (spray paver applied tack coat) and project 2 (distributor applied tack coat) with almost the same age. Tensile strength values in project 2 are about 25% lower as compared to the good portion of project 5. This shows better strength in spray paver laid tack projects.

ANOVA tests were conducted for all the projects to differentiate bad and good performing sections on the basis of tensile strength. In this ANOVA test, the response was tensile strength and the effects included project and condition. The results of ANOVA statistics are tabulated in Table 17. The results of this ANOVA at the 95% confidence level (P-value < 0.05) showed that the tensile strength on project 5 was statistically significant (P = 0.002, F = 16.58) whereas for projects 1, 3, and 4 (P-value > 0.05), the tensile strength was statistically insignificant. Yet, in all the projects, good performing portions had higher strength as compared to bad performing portions of the same project. As the overall sample size was small, statistical significance was pronounced only in project 5. OGFC pavements having higher IDT values performed well as compared to projects with lower IDT values.

Table 17. Important ANOVA Statistics of Project Tensile Strength

Response	Factors		Sample Size (N)	F-value	P-value
	Project	Condition			
IDT	5	B, G	12	16.58	0.002
	1	B, G	18	0.68	0.422
	4	B, G	12	2.57	0.140
	3	B, G	12	1.03	0.334

4.1.4 *Asphalt Content*

Figure 19 shows the average job mix formula (JMF) asphalt content (AC) and tested AC of the project portions from field cores. The AC of all portions reduced as compared to JMF AC except for the good portion of project 5. It is expected that in-place asphalt content would be lower now than when constructed due to traffic wearing off some of the binder coating on the layer surface. One reason why project 5 has a higher asphalt content may be migration of the thick tack application of the spray paver into the OGFC layer. Asphalt content appears to be insignificant in leading to the cause of premature failure. The results highlight that within a certain range of asphalt content, other factors are more important in contributing to failure.

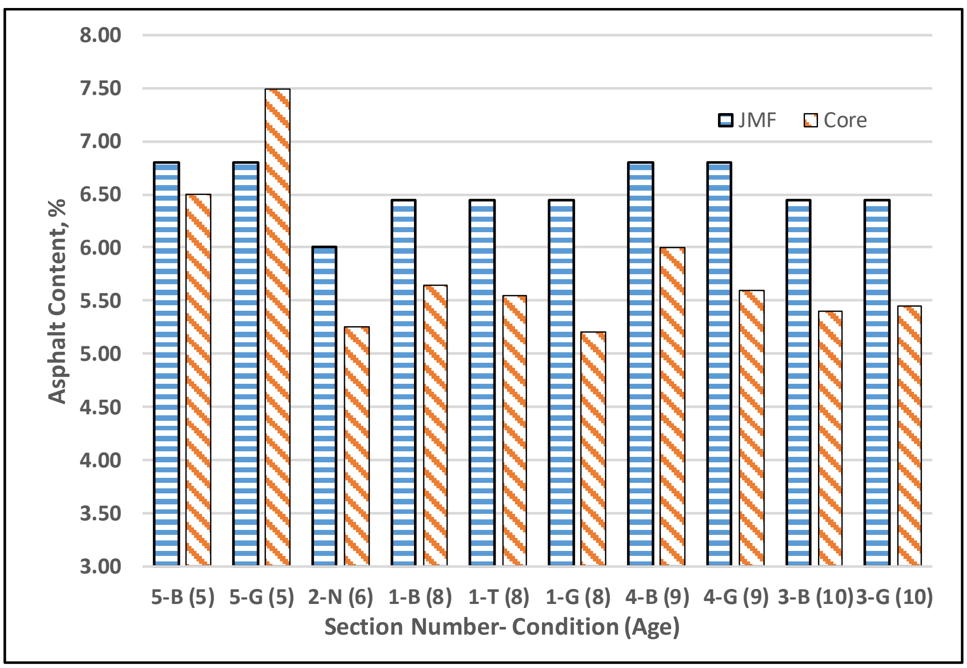

Figure 19. Average JMF Asphalt Content and Asphalt Content from Project Cores

4.1.5 *Aggregate Gradation*

Figure 20 to Figure 22 shows the grading curves of the OGFC extracted cores on all the five selected project sites as compared to the JMF grading curve.

Figure **20** shows the aggregate grading curves of project 2, which is composed mainly of granite. This project is six years old and the pavement was in good condition. The gradation of OGFC on this project was within the gradation envelope of the ALDOT 420 gradation requirements. There was minor degradation on the No. 8 (2.36 mm) sieve.

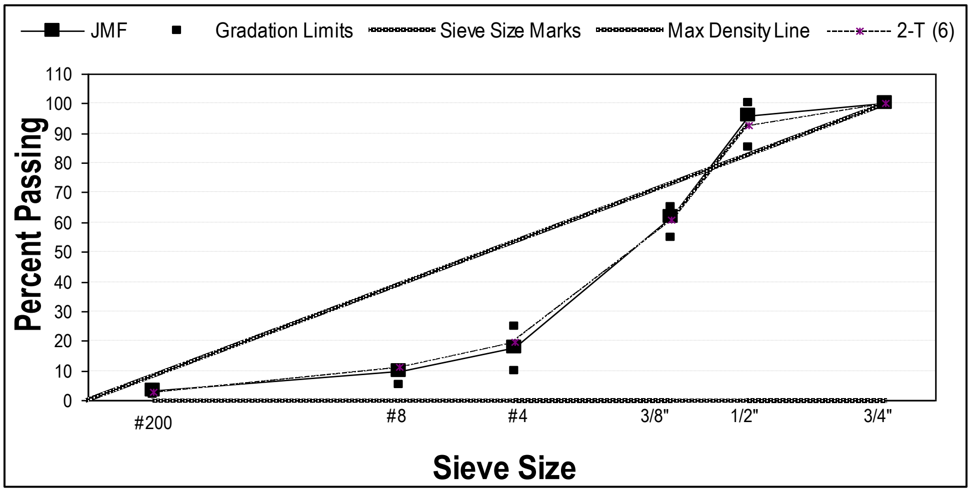

Figure 20. Gradation of Project 2 Mixture

Figure 21 shows the aggregate grading curves of projects 1 and 5. The aggregate composition of project 5 is mainly slag with sandstone, whereas project 1 is mainly sandstone. Projects 5 and 1 were five and seven years old, respectively, and both pavements were in moderate condition. The gradation of OGFC on both projects was on the finer side (on material passing No. 8 sieve and No. 4 sieve) of the gradation envelope.

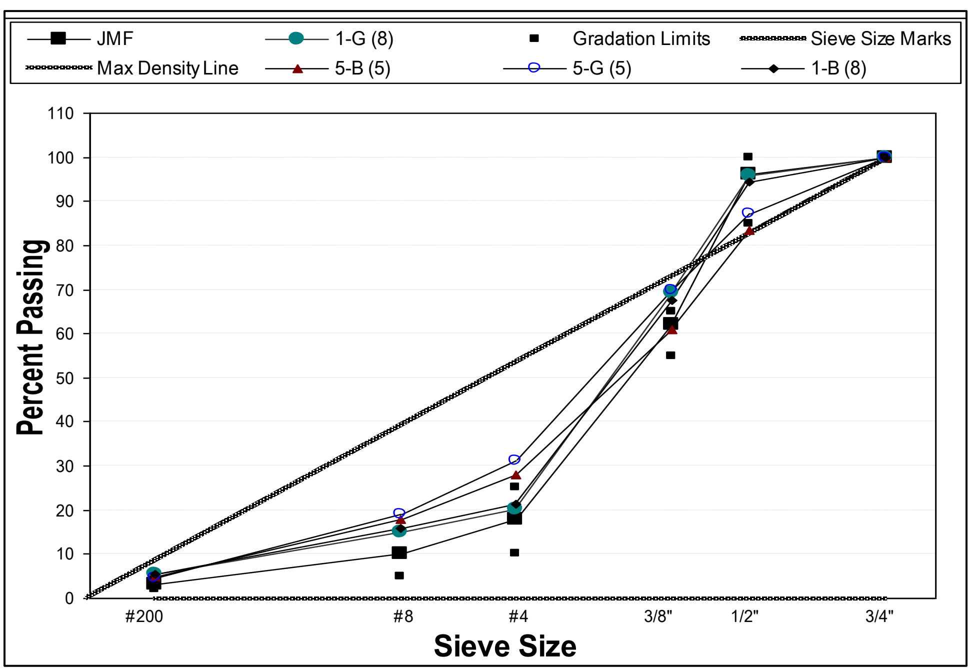

Figure 21. Gradation of Projects 1 and 5

Figure 22 shows the aggregate grading curves of projects 3 and 4. The aggregate composition of project 3 is mainly of slag, whereas project 4 is mainly slag with sandstone. Projects 3 and 4 were 10 and 9 years old, respectively, and both pavements were in moderate condition. The gradation of OGFC on both projects was also on the finer side (on material passing No. 8 sieve and No. 4 sieve) of the gradation envelope.

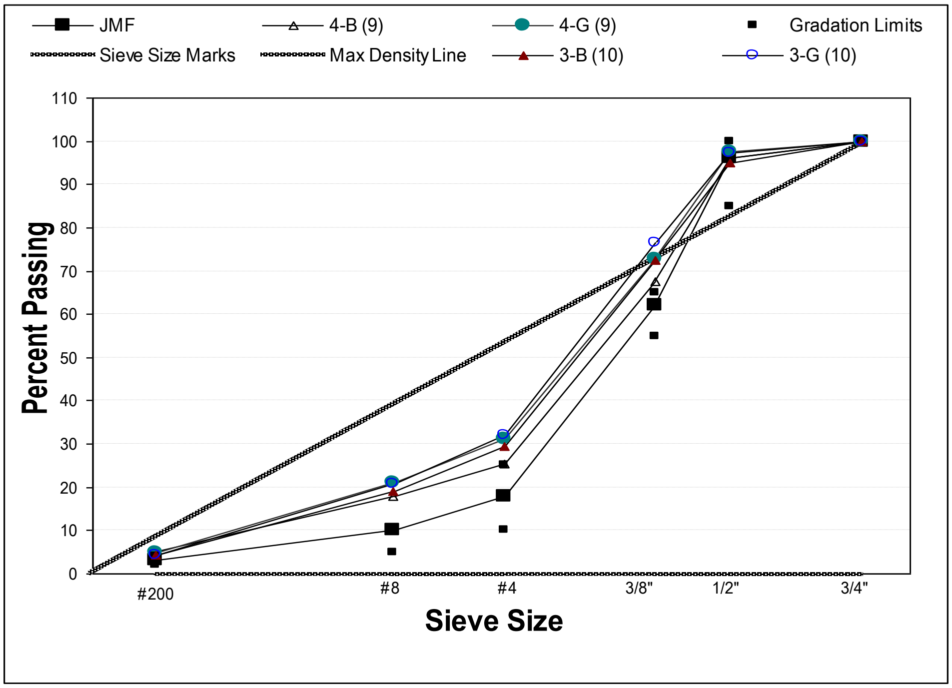

Figure 22. Gradation of Projects 3 and 4

If a comparison is drawn between projects 2 and 5, more aggregate degradation has occurred on project 5, which has slag and sandstone material, although the age is slightly less (five years for project 5 vs. six years for project 2). This trend shows that granite aggregate is less prone to degradation than slag and sandstone. One important observation from gradation data is that all four projects have more fines (passing No. 200 sieve) in bad portions as compared to good portions of the projects. Also, three out of four projects have finer gradation (passing No. 1/2 in., 3/8 in., and No. 4 sieves) in good portions as compared to bad portions of the projects.

5 PHASE 2 - EVALUATION OF REJUVENATOR AND FOG SEAL SURFACE TREATMENT

The practice of fog sealing is used by several agencies as a way to maintain longevity of the OGFC surface course. A fog seal, which typically consists of a diluted tack application, is sprayed on the road surface and allowed to penetrate into the surface voids. This recoats the aggregate particles, and in some cases, may help rejuvenate the aged asphalt binder. This preventive maintenance practice may extend the life of OGFC surface courses. However, there are two trade-offs that must be considered if fog seals are to be applied. First of all, the fog seal will fill surface voids and reduce the drainage capacity of OGFC mixtures. Over time, the mix may become much like a dense-graded mix with little permeability and little reduction in splash and spray. A second disadvantage of fog seals is that the applied material may temporarily reduce surface friction. If there is a significant loss in friction, the potential for reduced safety of travelling motorists outweighs the benefits of applying the fog seal. Blocking the lane from traffic or applying a sand coat to absorb any excess sealer may be required, but both are undesirable.

Pavepreserve (PP) and Pavegaard (PG) were the two rejuvenators used in this study. Pavepreserve uses selected maltene fractions to delay aging, while Pavegaard uses selected resins and asphaltenes to penetrate and rejuvenate aged asphalt. There is no specific timeframe for applying rejuvenators and in some cases they have been applied to relatively new pavements, but the typical pavement age at time of application ranges from three to seven years.

5.1 Methodology

Phase 2 of this research investigated the use of three types of fog seal/rejuvenator material and three application rates on 20 in. by 20 in. pavement sections of W4 and W5 on the inside lane at the NCAT Test Track. The two sections were surfaced with two OGFC mixtures in 2000. The two OGFC mixtures used in Sections W4 and W5 consisted of the same granite aggregate and gradation (Figure 23) and had similar binder contents of 6.1% and 6.2%, respectively.

Friction tests using the dynamic friction tester as described in ASTM E1911-09a *Standard Test Method for Measuring Paved Surface Frictional Properties Using the Dynamic Friction Tester* were used to evaluate friction loss after incremental increases of fog seal rate. Cores were then taken from the slabs and the Cantabro wear test was used to determine effectiveness of the seal to reduce raveling. Comparisons were made to determine the seal type that performs best and the recommended application rate.

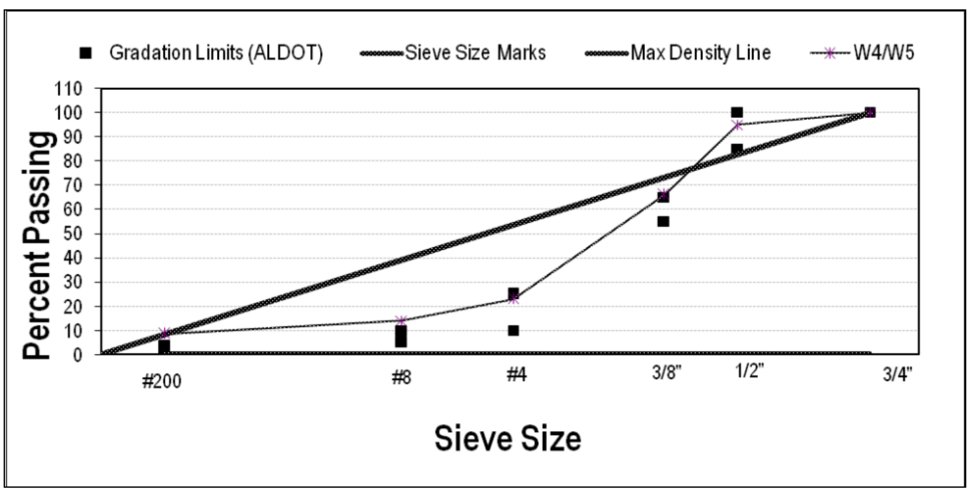

Figure 23. Design Gradation for OGFC Mixture Used in Test Sections W4 and W5

The test site selected for field evaluation consisted of performance grade (PG) 76-22 modified binders containing styrene butadiene rubber (SBR) and styrene butadiene styrene (SBS) to construct Sections W4 and W5, respectively. The average thickness of the OGFC surfaces for Sections W4 and W5 was 1.1 in. and 0.7 in., respectively, based on the thickness measurement of the cores extracted from the two sections. These sections were not trafficked regularly but were used for moving construction equipment during the construction, reconstruction, and maintenance of the outside lane, which was trafficked during each three-year research cycle. Thus, the site selection posed an inherent limitation as the effect of traffic on aging of asphalt binder is lacking.

In each square, a fog/rejuvenator seal material was evenly sprayed (Figure 24) at application rates of 0.03, 0.07, and 0.10 gal/sy (0.16, 0.32 and 0.48 liters/m^2). The surface was not sanded after the application because the fines from the sanding process might fill in the surface voids causing an adverse effect on the drainability of the OGFC and would likely affect friction results.

Table 18 shows the layout of Sections W4 and W5 for evaluating the fog seal and rejuvenator seals as well as the type of fog/rejuvenator seal material and its application rate for each 20 x 20 inch square. In each square, a fog/rejuvenator seal material was evenly sprayed (Figure 24) at application rates of 0.03, 0.07, and 0.10 gal/sy (0.16, 0.32 and 0.48 liters/m^2). The surface was not sanded after the application because the fines from the sanding process might fill in the surface voids causing an adverse effect on the drainability of the OGFC and would likely affect friction results.

Table 18. Layout of Test Sections for Evaluating Fog and Rejuvenator Seals

		PP Rejuvenator Seal			**PG Rejuvenator Seal**			**CSS - 1h Fog Seal**		
Square/Pad No.	Control	1	2	3	4	5	6	7	8	9
Application Rate, gal/sy	0	0.03	0.07	0.1	0.03	0.07	0.1	0.03	0.07	0.1
Section W4 (Existing Binder - SBR Modified)										
Square/Pad No.	Control	10	11	12	13	14	15	16	17	18
Application Rate, gal/sy	0	0.03	0.07	0.1	0.03	0.07	0.1	0.03	0.07	0.1
Section W5 (Existing Binder - SBS Modified)										

PP = Pavepreserve; PG = Pavegaard

To control the application rate, the sprayer was weighed before and during the spray application to determine the amount of fog/rejuvenator seal material applied in each square. In addition, a 2 in. x 2 in. (5 x 5 cm) geosynthetic pad with a predetermined weight was placed at the center of each square during the spray application and then removed and oven-dried to a constant mass to determine the actual application rate of fog/rejuvenator seal material for each square.

After the fog and rejuvenator seals had been cured for one week, the friction and macro texture characteristics of the OGFC surface in each square were measured with a dynamic friction tester (DFT) in accordance with ASTM E1911-09a and with a circular texture meter (CTM) in accordance with ASTM E2157-09. After that, 150 mm (6-in.) diameter cores were taken from each square for laboratory testing.

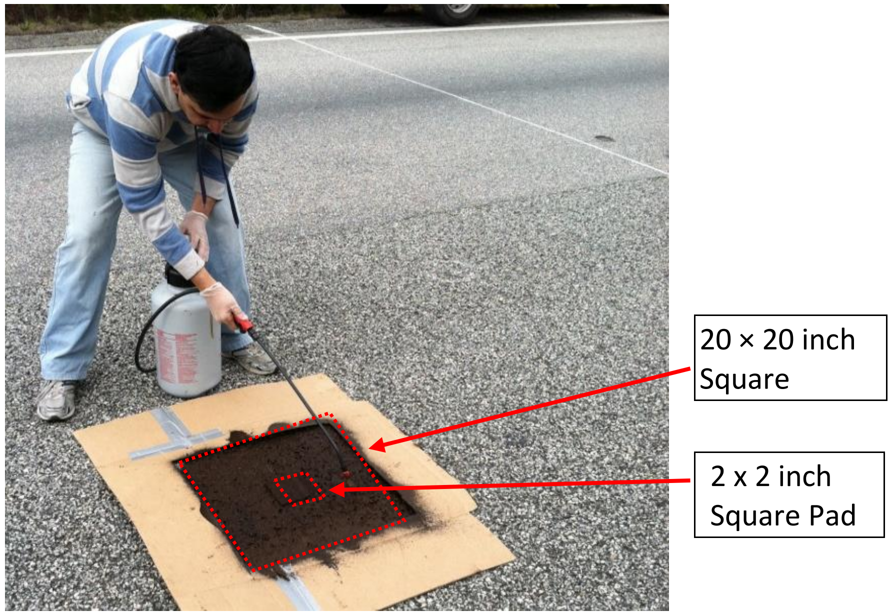

Figure 24. Spray Application of Seal Material on OGFC Pavement

The full depth cores (including the OGFC wearing course and the Superpave mixture in the underlying layer) taken from each square (except squares containing CSS-1H), shown in In each square, a fog/rejuvenator seal material was evenly sprayed (Figure 24) at application rates of 0.03, 0.07, and 0.10 gal/sy (0.16, 0.32 and 0.48 liters/m^2). The surface was not sanded after the application because the fines from the sanding process might fill in the surface voids causing an adverse effect on the drainability of the OGFC and would likely affect friction results.

Table 18, were used for further testing in the laboratory to determine the effects of the rejuvenators on the durability of OGFC. For each set of cores, two cores were used for bulk specific gravity measurement and Cantabro test and two for the HWTD test.

For bulk specific gravity measurement and the Cantabro test, the OGFC surface layers were cut from three full-depth cores to prepare three test specimens. The bulk specific gravity of each OGFC specimen was determined using automatic vacuum sealing method in accordance with AASHTO T 331. Two specimens were then used for the Cantabro test in accordance with AASHTO TP 108.

For the HWTD test, the top 3.75 cm (1.5-in.) layers including the OGFC wearing course and a part of the Superpave mix in the underlying layer were cut from two full-depth cores to prepare two test specimens. The two specimens were then used to run one HWTD test in compliance with AASHTO T 324-08.

5.2 Results and Discussion

5.2.1 *Micro and Macro Surface Friction Characteristics*

Analyses of CTM and DFT testing results were conducted to assess the micro and macro surface friction characteristics following the application of fog and rejuvenator seals.

The macro and micro texture analyses were conducted for the mean profile depth (MPD) obtained from CTM and friction number measured at 20 km/h (DFT_{20}) and at 40 km/h (DFT_{40}) using the DFT. The international friction index (IFI) parameter F_{60} was calculated and analyzed to find the mutual effect of rejuvenators on macrotexture and microtexture of the pavement surface. The IFI consists of two parameters: F_{60} and speed constant (*Sp*). F_{60} is the harmonized estimate of the friction at 60 km/hr computed from both the friction measurement and *Sp*, whereas *Sp* is linearly related to macrotexture measurements. The IFI parameter F_{60} can be estimated based on DFT and CTM results using Equation 1 as given in ASTM E1960 - 07(2011).

$$F_{60} = 0.081 + 0.732 \times DFT_{20} \times e^{\frac{-40}{Sp}} \qquad (2)$$

Where:

F_{60} = International friction index;
DFT_{20} = Friction number obtained at 20 km/h using the DFT; and
Sp = Speed constant = 14.2 + 89.7 MPD (MPD in mm).

Two ANOVA tests for all of the measured MPD and DFT_{20} data were conducted. Tukey's method was employed for multiple comparisons between the means. The results of these ANOVA tests at 95% confidence level ($P < 0.001$) showed that the effect of the existing surfaces in Sections W4 and W5 on the MPD and DFT_{20} measurements were statistically significant. Hence, the effects of applying fog seal on the surface friction characteristics of OGFC were analyzed separately for Sections W4 and W5.

Figure 25 and

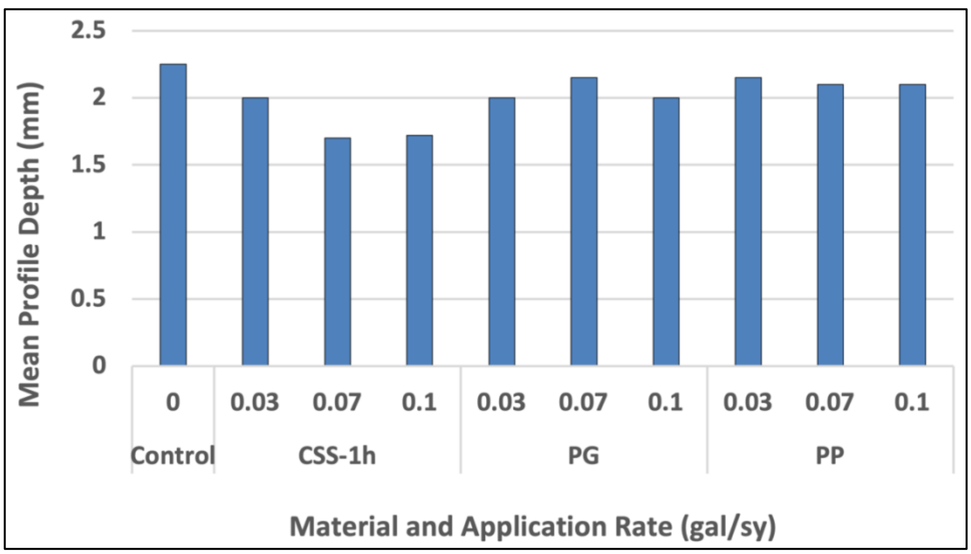

Figure **26** graphically compare the effect of fog and rejuvenator seals on MPD for Sections W4 and W5, respectively.

Table 19 and Table 20 show results of ANOVA statistics of the effect of fog and rejuvenator seals on MPD for Sections W4 and W5, respectively. ANOVA results indicate that the application of fog seal (CSS-1H) significantly affected the MPD of the OGFC surfaces. Lower MPD results of the CSS-1h sections indicates that the fog seal emulsion may have filled more of the surface voids than the rejuvenator treatments.

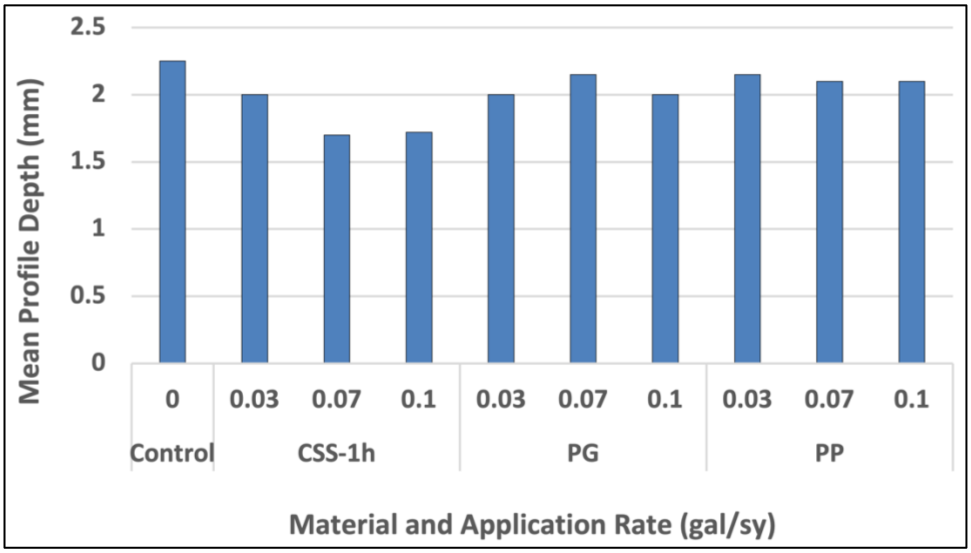

Figure 25. Graphical Comparison of Effect of Fog and Rejuvenator Seals on Mean Profile Depth for Section W4

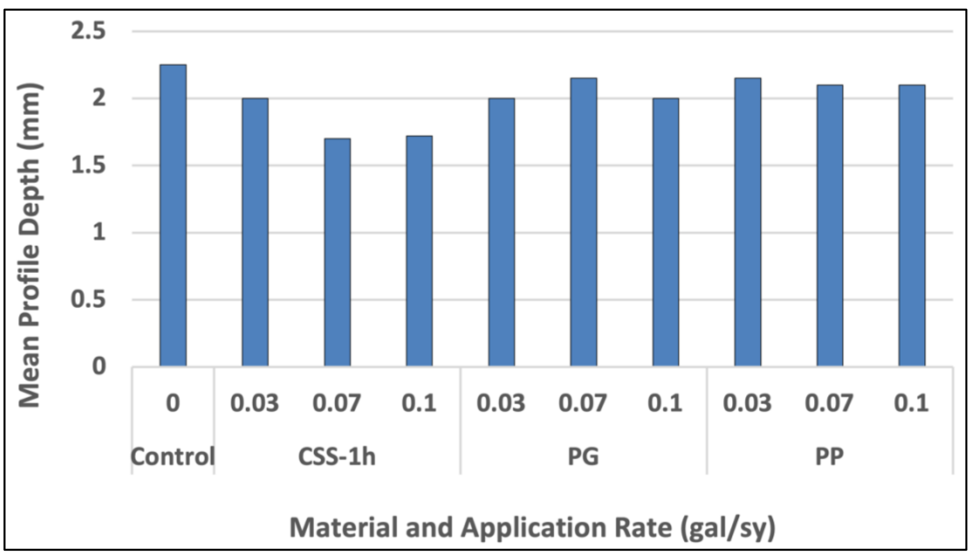

Figure 26. Graphical Comparison of Effect of Fog and Rejuvenator Seals on Mean Profile Depth for Section W5

Table 19. ANOVA Statistics for Analysis of Effect of Fog and Rejuvenator Seals on MPD for Section W4

Response	Factors		N	F-value	P-value
	Material	Application Rate (gal/sy)			
MPD	CSS	0.03	30	56.34	0.000
	PG	0.07			
	PP	0.10			

Table 20. ANOVA Statistics for Analysis of Effect of Fog and Rejuvenator Seals on MPD for Section W5

Response	Factors		N	F-value	P-value
	Material	Application Rate (gal/sy)			
MPD	CSS	0.03	30	112.21	0.000
	PG	0.07			
	PP	0.10			

Figure 27 and Figure 28 show results of graphical comparison for analysis of the effect of fog and rejuvenator seals on friction numbers for Sections W4 and W5, respectively. The results show that the application of all fog and rejuvenator seals, especially PG for Section W4, significantly affected DFT_{20} and DFT_{40}.

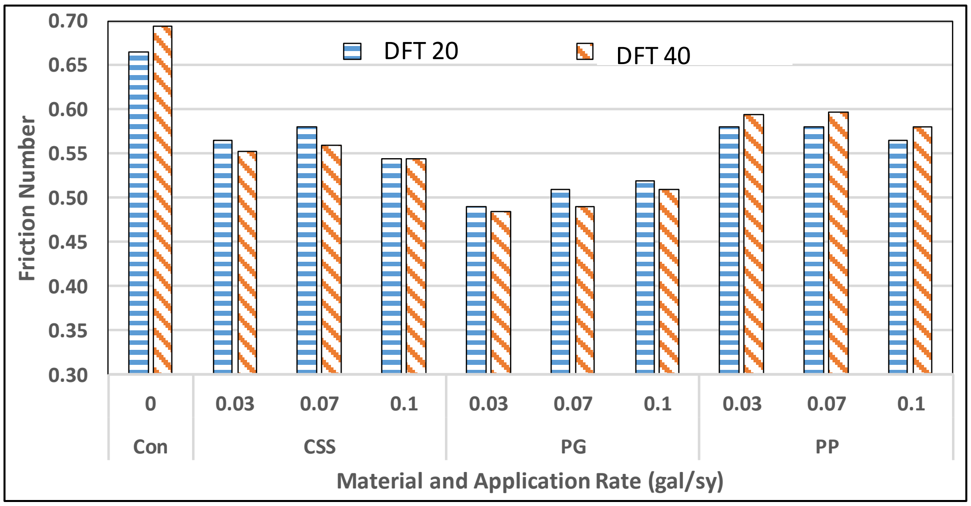

Figure 27. Graphical Comparison for Effect of Fog and Rejuvenator Seals on Friction Number for Section W4

For Section W5, the application of the CSS fog seal and PG rejuvenator significantly affected DFT_{20} and DFT_{40}. In the case of section W5, application of PP rejuvenator at the high application rate significantly affected DFT_{20} and DFT_{40}. For the PG rejuvenator, there is a trend of increase in DFT_{20}, $DFT_{40,}$ and MPD with an increase in application rate, which is contrary to the PP and CSS-1H sections. The probable reason of this behavior lies in the composition of the seals. PP and CSS-1H seals are less viscous (asphaltenes up to 0.75% by weight) as compared to the PG rejuvenator (asphaltenes up to 11% by weight). A liquid emulsion film is formed on the pavement surface after application of fog or rejuvenator seals, causing reduction in Friction Number and MPD. In the case of PG rejuvenator, increase in application rate increases asphaltenes concentration, resulting in increased friction. Application of the PP rejuvenator at the low rate had little effect of friction results in the W5 section. It is notable that the PP rejuvenator had higher friction results than the other treatments in both W4 and W5 sections. This may indicate that the PP rejuvenator penetrated more into the binder film and surface voids whereas the PG and fog seal acted more as a film coating to seal the surface.

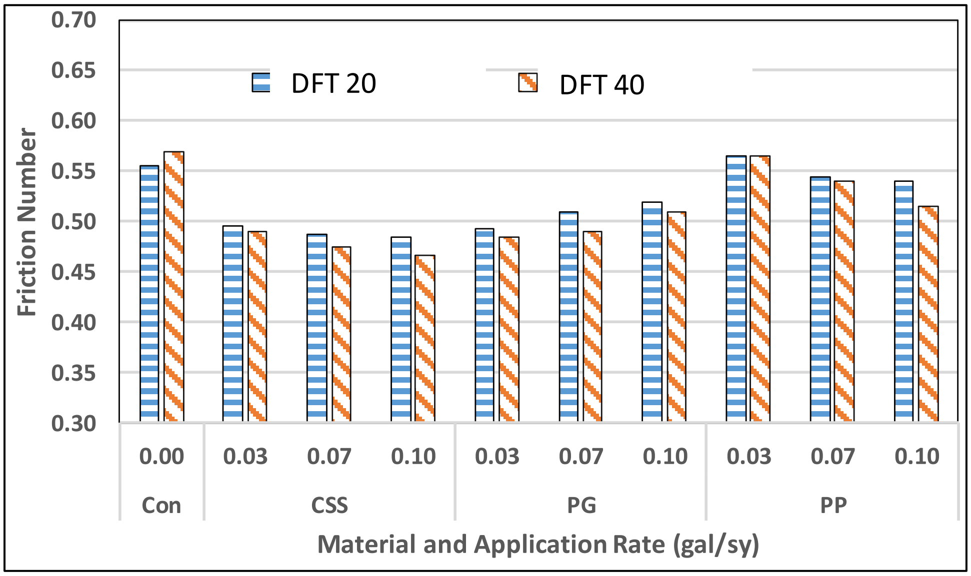

Figure 28. Graphical Comparison for Effect of Fog and Rejuvenator Seals on Friction Number for Section W5

There is a decrease in the IFI parameter F_{60} due to the application of fog and rejuvenator seals for sections W4 and W5 (Figure 29). The decrease in F_{60} depends on the type of modified binder used on the existing surface (W4 versus W5), the type of rejuvenator or fog seal material, and the application rate. The surface friction based on F_{60} was reduced 2-24%. Thus, fog and rejuvenator seals should be used with extreme caution on OGFC as they may cause a temporary loss of friction. The fog seal (CSS-1H) showed similar effects on the surface friction characteristics of the two sections.

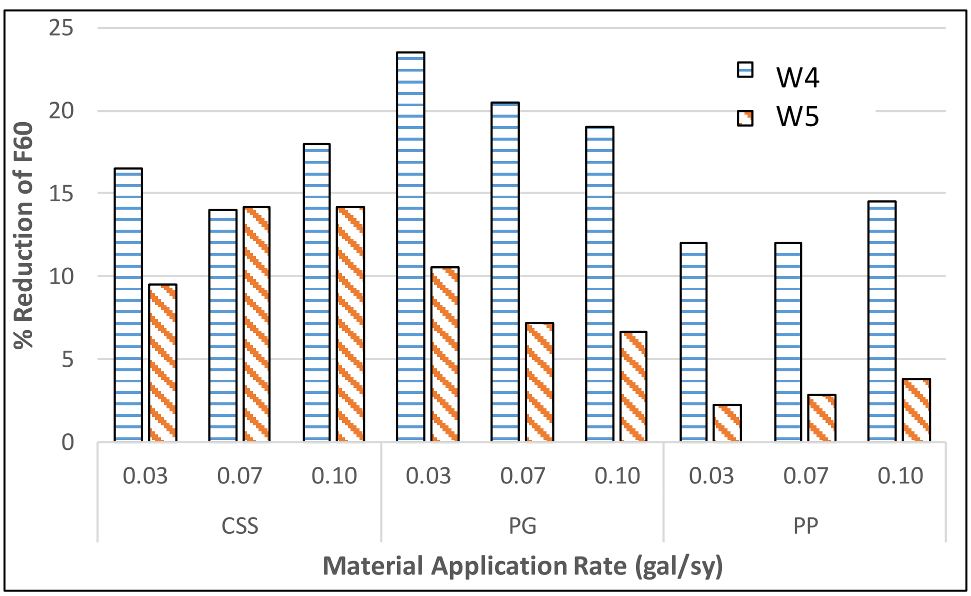

Figure 29. Percent Decrease in F60 on OGFC Sections Due to Fog and Rejuvenator Seals

5.2.2 *Air Void Measurements*

An analysis of air voids was carried out to assess the impact of fog and rejuvenator seals on the functionality of OGFC. Figure 30 shows the air void measurements for the control squares with no treatment and for other squares with the rejuvenator seals sprayed at 0.03, 0.07, and 0.10 gal/sy.

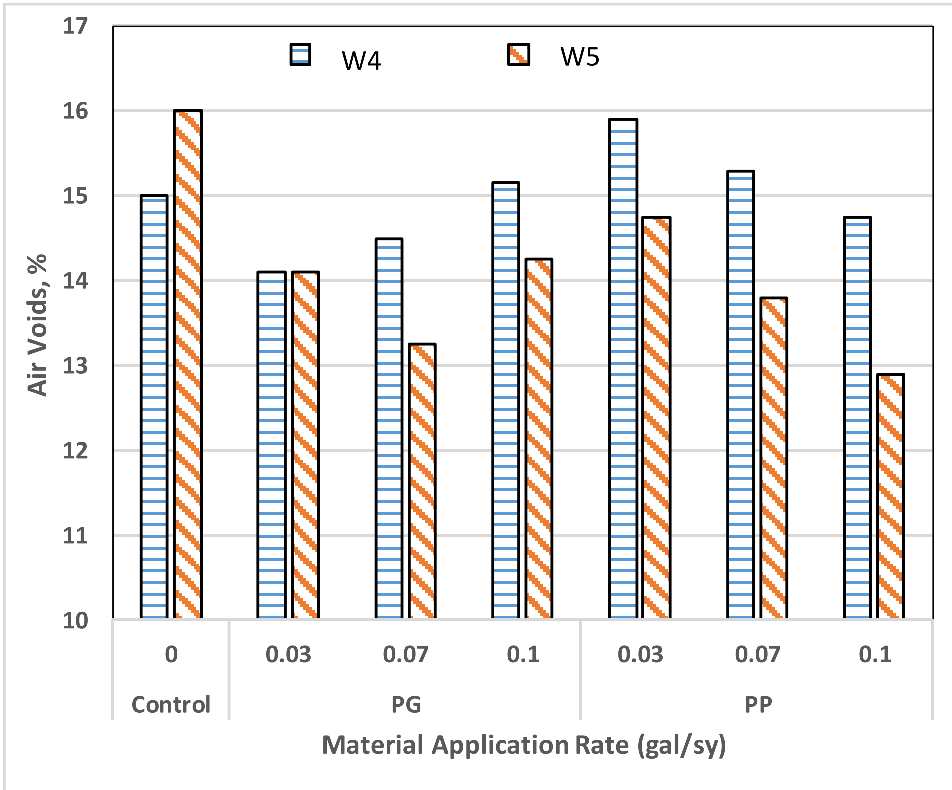

Figure 30. Percent Specimen Air Voids Resulting from Spraying with Rejuvenator Seals

There is a visual trend of reduced air voids with an increase in rejuvenator seal application rate, resulting in a reduction in functionality for Section W5, although there was little difference in Section W4. ANOVA tests for all the measured air voids were conducted separately for Sections W4 and W5. The results of the tests at 95% confidence level show that the differences in air voids were not statistically significant (*P*-value = 0.250) for Section W4, while for Section W5 the results were borderline significant (*P*-value = 0.033). This is a matter of concern, as the capacity to drain water through OGFC pavements would be reduced and the functionality of the mix would be affected. In the case of PG rejuvenator in section W4, an increase in air voids is observed with increases in application rate. One of the causes of this phenomenon can be due to within-section variability.

5.2.3 *Cantabro Tests*

The Cantabro test indicates the mixture's resistance to wear and raveling (Larson, 1991) and is recommended for use in a standard OGFC mix design procedure based on previous NCAT research (Kandhal, 2002; Watson et al., 2004). This test has been used to predict the durability of OGFC pavements during their service life. The maximum Cantabro loss for OGFC mixtures compacted to 50 gyrations is 15% based on previous research at NCAT (Watson, et al, 2003).

Figure 31 shows the Cantabro loss results for OGFC cored specimens extracted from control squares and squares with the two rejuvenator seal products sprayed at the rates of 0.03, 0.07, and 0.10 gal/sy.

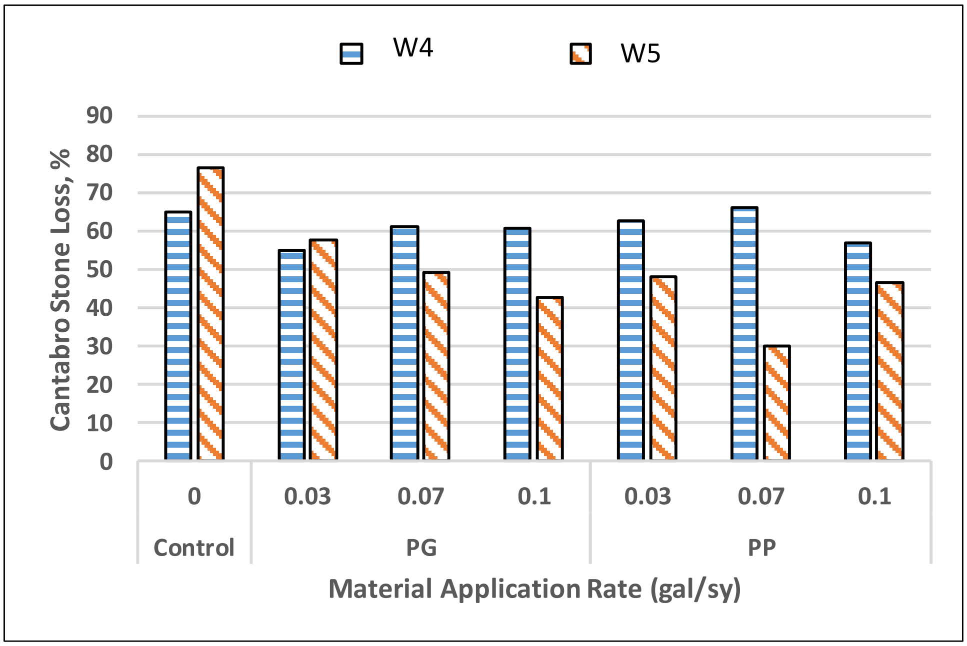

Figure 31. Effects of Treatment Type and Application Rate on Cantabro Loss

The loss values were higher than what would be acceptable during mix design but they were probably due to the age of the pavement and the thinness of the cores used for testing. Resistance to abrasion usually improves with an increase in binder content, and in this case, each of the rejuvenator seal products appeared to improve abrasion resistance. These results indicate that an application rate ranging between 0.07 and 0.10 gal/sy should be suitable depending on the type of rejuvenator seal applied and the type of modified binder used on the existing surface (W4 with SBR versus W5 with SBS).

6 PHASE THREE – INVESTIGATE METHODS TO IMPROVE DURABILITY

The objective of this portion of the study was to evaluate potential changes in ALDOT's OGFC mix design and construction procedures to improve the durability of these mixtures in the field. These potential changes were evaluated in a laboratory experiment and field test sections on the NCAT Test Track. Results of these experiments were analyzed to determine the effect of the potential changes on the laboratory and field performance of OGFC mixtures.

6.1 Comparison of NCAT Test Track Sections

6.1.1 *Experimental Plan*

Once the literature review was complete, ALDOT's standard specifications and procedure for designing OGFC mixtures was reviewed. During the review process, three potential changes were identified that could improve the field performance of OGFC mixtures in Alabama. They included: (1) use of a finer gradation with a 9.5-mm NMAS instead of a currently specified 12.5-mm NMAS; (2) utilizing synthetic fiber instead of cellulose fiber; and (3) using asphalt binder modified with GTR and without fiber stabilizer in place of the standard SBS polymer-modified binder with cellulose fiber stabilizer. These changes were incorporated in three experimental OGFC mix designs paved in Sections E9A, E9B, and E10 on the NCAT Test Track, respectively.

After milling the old surface layer, sections were inlaid with a 1.5 inch dense-graded leveling layer and each test section mixture was then placed as a ¾-inch thick riding surface. Test sections E9A and E9B were each 100 feet long, while E10 was 200 feet long. An emulsified tack coat was applied at a residual rate of 0.05 gal/sy in each test section before paving the OGFC mixtures. All mixtures used the same compaction patterns and the in-place air voids after construction were approximately 20%.

After construction, the three test sections were loaded with heavy triple trailer trains (Figure 32) with an average gross vehicle weight of 77.5 tons (155,000 lbs) driven at a cruise speed of 45 miles per hour. The fleet of triple-trailer trucks operated 16 hours per day, five days per week. During that time, routine weekly performance measurements were made including rut depth, ride quality, surface texture, and visual inspection for cracking. In addition, the permeability of the three test sections was also measured immediately after construction and every quarter thereafter.

Figure 32. Heavily Loaded Triple-Trailer on the NCAT Test Track

The three experimental OGFC mixtures were designed based on a 12.5-mm OGFC mix design previously approved by ALDOT, which had 6% PG 76-22 asphalt binder modified with SBS polymer. ALDOT determines the optimum asphalt binder content based on a surface constant and oil absorption procedure described in ALDOT-259 procedure, "Open-Graded Asphalt Concrete Friction Course Design Method."

The three experimental OGFC mixtures paved on the NCAT Test Track were as follows:

- For Section E9A, the OGFC mixture was designed with a 9.5-mm NMAS gradation instead of a 12.5-mm NMAS gradation and with 6% PG 76-22 asphalt binder modified with polymer.
- For Section E9B, the OGFC mixture was designed with a 12.5-mm NMAS gradation similar to the one utilized in the state approved OGFC mix design. It had 6% PG 76-22 asphalt binder modified with SBS polymer. However, this mix design had 0.3% synthetic fiber instead of the cellulose fiber typically used to prevent draindown of the thick asphalt binder film from aggregate particles.
- The same 12.5-mm NMAS gradation of Section E9B was used in the OGFC mix design for Section E10. However, the mix design had 6.3% GTR modified asphalt, which consisted of 5.8% base asphalt modified to obtain PG 76-22 by adding 12% GTR (by weight of asphalt binder). The GTR was a minus No. 30 mesh size and the base asphalt was a PG 67-22. No fiber was added to the mix in order to determine whether GTR alone could prevent drain-down and provide resistance to raveling.

During mix design, all of the binders were pre-blended with an antistrip agent at a dosage of 0.5% by weight of the base binder. The antistrip was added as a conservative measure but is now required by ALDOT in all OGFC mixes. Samples 150 mm diameter by 115 mm high were prepared in the NCAT laboratory and compacted to 50 gyrations to measure air void content, Cantabro loss, and splitting tensile strengths. Table 21 shows the basic information of three experimental mixes. Table 22 summarizes the design gradations for the 9.5-mm and 12.5-mm OGFC mixtures.

Table 21. Mixes Used in the Study

Section ID	NMAS, mm	Fiber Type and Rate	Binder PG and AC content	Purpose for Selected Mix
Lab mix only	12.5	Cellulose 0.3%	SBS PG 76-22, 6.0%	Control Mix
E9A	9.5	Cellulose 0.3%	SBS PG 76-22, 6.0%	Evaluate effect of aggregate NMAS
E9B	12.5	Synthetic 0.3%	SBS PG 76-22, 6.0%	Evaluate effect of fiber
E10	12.5	No Fiber	GTR PG 76-22, 6.3%	Evaluate effect of binder type without fiber

Table 22. Design Aggregate Gradations for OGFC Mixtures

Sieve Size (mm)	Percent Passing	
	9.5-mm OGFC	12.5-mm OGFC
19.0	100.0	100.0
12.5	98.5	95.7
9.5	87.2	56.1
4.75	32.4	15.7
2.36	9.8	9.5
1.18	5.7	7.1
0.6	4.2	5.7
0.3	3.1	4.5
0.075	1.4	2.6

6.1.2 *Laboratory Test Results*

6.1.2.1 Air Voids and Cantabro Stone Loss

Figure 33 shows the specimen air voids and Cantabro loss results for the control mix and the three modified OGFC mixtures evaluated during the mix design process. The ALDOT typical mix had lower air voids compared to the three modified OGFC mixes placed on the Test Track. These results indicate that the 9.5 mm mix had higher air voids, which may improve the potential for increased permeability while still maintaining raveling resistance. The air voids of the three experimental mixtures were close to or above the minimum air void content of 15%, and the Cantabro loss results were all below the maximum Cantabro loss threshold of 15%. The air voids of the ALDOT typical mix did not meet the air void requirement of 15%.

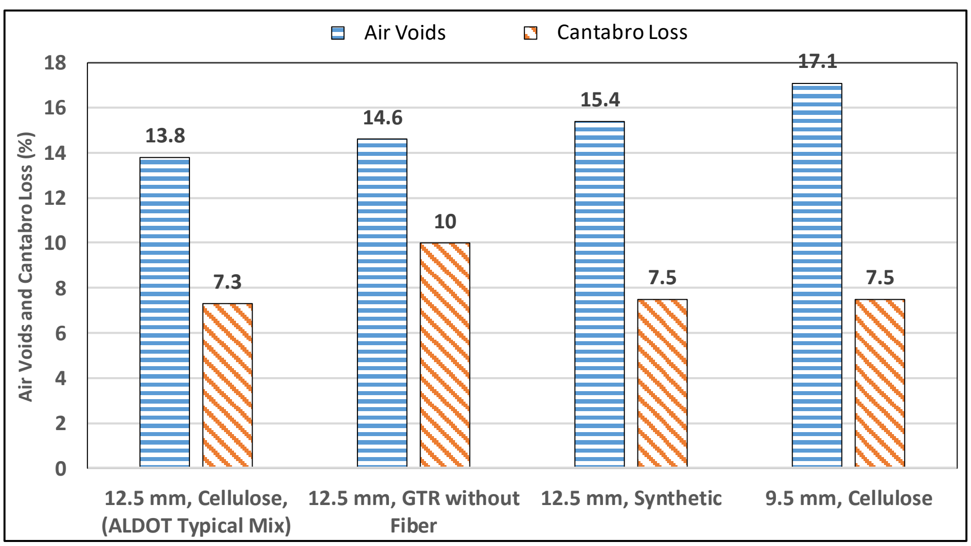

Figure 33. Comparison of Air Voids and Cantaboro Loss for OGFC Mixtures

6.1.2.2 Moisture Susceptibility

Moisture susceptibility is another aspect of OGFC mixes that may lead to reduced performance life, since the high air void content makes them more susceptible to damage from moisture and freeze/thaw action. Moisture susceptibility testing was performed in accordance with AASHTO T 283 with the following modifications:

- Standard 115 mm high specimens were used;
- 50 gyrations were used rather than controlling air voids to a certain level;
- Due to the open void structure, specimens were vacuum saturated for 10 minutes rather than to a specified percent saturation. Saturation is not calculated due to the difficulty in obtaining accurate saturated surface dry measurements; and
- Conditioned specimens were kept submerged during the freeze/thaw process to prevent water in the internal void structure from draining out.

This procedure is different than the ALDOT 259 OGFC mix design procedure, which uses 100 gyrations to compact samples and does not require a freeze/thaw cycle. Indirect tensile strength (ITS) of conditioned and unconditioned specimens was measured using a Marshall press apparatus. The ratio of the indirect tensile strengths of the conditioned and unconditioned samples was recorded as the tensile-strength ratio (TSR). A TSR criterion of 0.80 is required for OGFC mixtures in Alabama.

Figure 34 compares splitting tensile strength test results obtained during mix design for the four OGFC mixes. The 9.5-mm mixture with cellulose fiber provided the highest tensile strengths. Among the 12.5-mm OGFC mixtures, the GTR mix without fiber had the highest tensile strengths but marginally failed the ALDOT TSR requirement of 0.8. The 12.5-mm control mix with cellulose fiber (ALDOT typical OGFC mix) failed this requirement significantly. The moisture conditioning and freeze/thaw cycle in this study had a significant effect on the conditioned splitting tensile strength of the ALDOT typical OGFC mixture. The ITS and TSR results suggest that the potential changes in ALDOT's OGFC mix design improved the moisture susceptibility of OGFC mixes, especially for 9.5 mm OGFC mix with cellulose fiber.

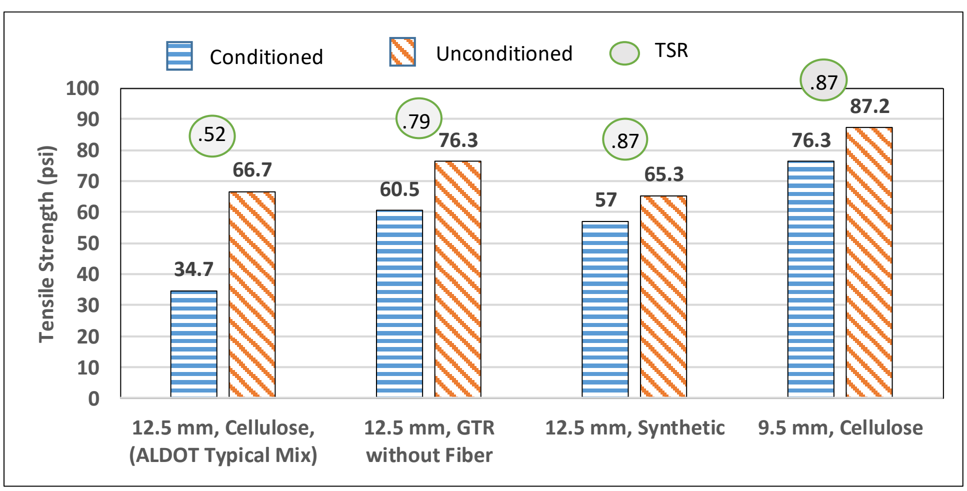

Figure 34. Comparison of Tensile Strength Test Results for OGFC Mixtures

6.1.2.3 Permeability

Field permeability in the wheelpath in the OGFC pavements was measured using the falling-head field permeameter (Figure 35), which was designed using a standpipe of a constant diameter. The standpipe is then sealed to the pavement using a flexible rubber base and metal base plate to force the sealant into the surface voids. Head loss is then recorded based on the amount of water drained from the standpipe over a measured period of time (Cooley, 1999).

Figure 35. Field Permeameter

Field permeability as a function of traffic loading is presented in Figure 36. The permeability of the 9.5-mm mix in Section E9A was always higher than both of the 12.5-mm mixes in Section E9B and Section E10. The slope of the permeability degradation curve for the 9.5-mm mix in Section E9A was also flatter than those for the other two mixes. This indicates that the 9.5-mm mix had a lower rate of permeability degradation over traffic loading and maintained a level of permeability for a longer period of time as compared to both 12.5-mm mixes.

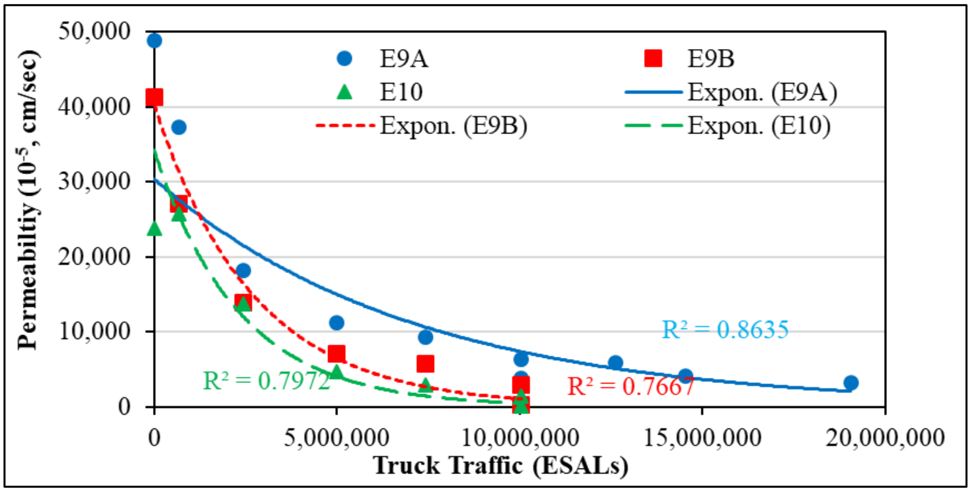

Figure 36. Field Permeability Comparison

For the two 12.5-mm mixes, the 12.5-mm mix with synthetic fiber in Section E9B exhibited a slightly higher permeability than the 12.5-mm GTR modified mix in Section E10. Additionally, the 12.5-mm mixes in Section E9B and Section E10 had significantly lower permeability (0.002 cm/sec for E9B and 0.0008 cm/sec for E10) after the application of 10 million equivalent single axle loads (ESALs) than E9A (0.04 cm/sec). The 9.5-mm mix in Section E9A is still permeable (0.03 cm/sec) after the application of 20 million ESALs.

6.2 Additional Field Sampling and Comparison Lab Testing on U.S. 80 Mixture

With ALDOT's current mix gradation and placement thickness, a thickness/nominal maximum aggregate size (NMAS) ratio of 1.4 is obtained. A review of OGFC performance at the NCAT Test Track showed better performance for OGFC mixtures having thickness/NMAS ratio of about 2.5. To increase thickness/NMAS ratio, either additional thickness will need to be provided or the gradation will need to be made finer. To further evaluate this hypothesis, samples from a current ALDOT construction project on U.S. 80 in Sumter County was compared to the finer gradation mixture used on NCAT Test Track Section E9A.

Loose mix, virgin aggregate, and asphalt binder were collected from the plant providing mix to the OGFC project on U.S. 80 and from the plant providing mix to Section E9A at the NCAT Test Track. From the loose mix samples, slabs were prepared at each of the following approximate thicknesses: 19, 25, 32, and 38 mm (0.75, 1.0, 1.25, and 1.5 inches). From each slab, 10 cores were taken: four 6-inch diameter and six 4-inch diameter specimens. The four 6-inch cores were used to make two replicates for Hamburg testing. Three of the 4-inch cores were tested for Cantabro, and the rest were tested for permeability. The three permeability cores were then conditioned and tested for tensile strength after one freeze/thaw cycle. Permeability testing for this research was accomplished using a falling head test (ASTM PS 129-01). This provisional standard is no longer used by ASTM; however it is similar to the Florida Method (FM 5-565).

For the U.S. 80 project, the mixture consisted of a sandstone aggregate with 6.3% asphalt binder. The loose aggregate, fiber, and binder used were collected from the plant site of the project in Sumter County. The E9A test section used granite aggregate with 6.0% asphalt binder.

6.2.1 *Laboratory Test Results*

Figure 37 shows the average rut depth (using the Hamburg rutting procedure) of cores for U.S. 80 and Section E9A mixtures. HWTD test results clearly showed a trend of reduction in rut depth with increased thickness for both the 12.5 mm OGFC used on U.S. 80 and the 9.5 mm OGFC used on Section E9A. Based on these results, it appears that an increase in OGFC thickness improves performance in resistance to rutting/moisture damage to some extent. The excessive rutting of thin lifts for the U.S. 80 mix during the Hamburg test also indicates that the sandstone mix may be disintegrating due to the aggregate rutting/moisture interaction.

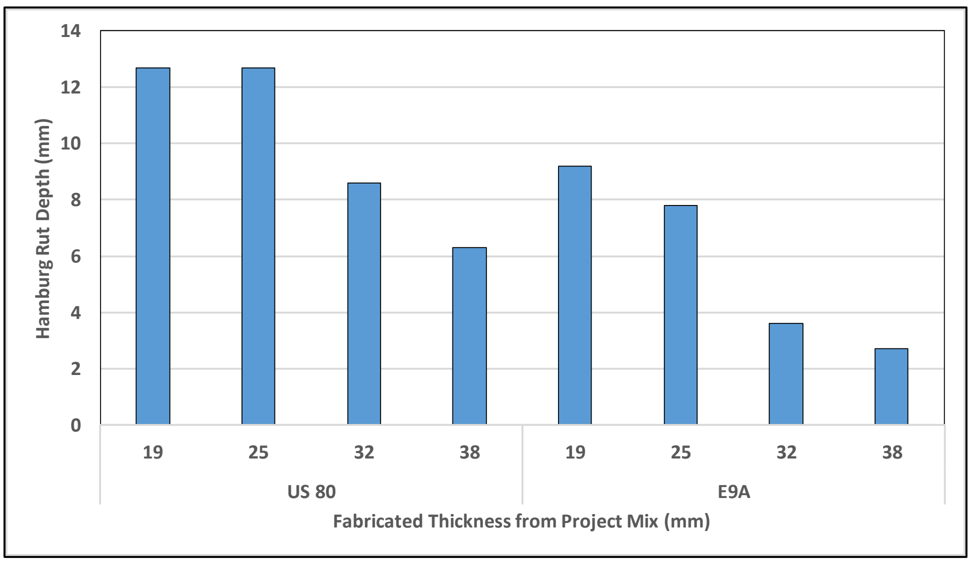

Figure 37. Average Hamburg Rut Depth Based on Layer Thickness

6.2.2 *Cantabro Tests*

Analysis of Cantabro percent loss was conducted to evaluate the improvement in resistance to wear and raveling for different projects and thicknesses. Figure 38 shows the Cantabro percent loss results of OGFC mixes used on U.S. 80 and Section E9A. The test results indicate that the increase in thickness of OGFC pavement significantly reduced the Cantabro stone loss, thereby providing improvement in resistance to wear and raveling.

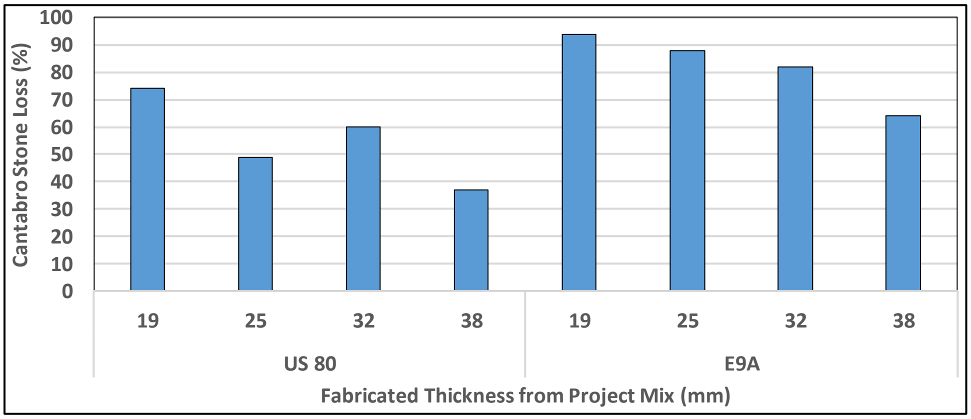

Figure 38. Comparison of OGFC Layer Thickness to Cantabro Stone Loss

For each of the mixes, Cantabro stone loss exceeded the maximum target value of 15%. This is likely due to the thinness of the cores used in the test. The reduction in raveling susceptibility with increase in thickness of OGFC is likely caused by enhanced internal cohesion through aggregate particle interlock as well as an increase in the aspect ratio of specimen size. An increase in thickness of the OGFC layer also increases the thickness/NMAS ratio to provide a better aggregate skeleton. The reduction in stone loss with increase in thickness is greater for the U.S. 80 mixture than for the E9A mixture. There is marked improvement in raveling resistance on thicknesses of 25 mm (1 inch) or greater. These results indicate that an improvement in resistance to wear and raveling is directly proportional to layer thickness. For this reason, a thickness/NMAS ratio of 2.5 is recommended.

6.2.3 *Permeability Tests*

Analysis of permeability was conducted to evaluate the variation in water transmission characteristics for different OGFC mix types and thicknesses. Figure 39 shows the permeability test results of cores for mixtures used on US 80 and E9A. ANOVA tests comparing the permeability to thickness were conducted together for U.S. 80 and E9A projects. In these ANOVA tests, the responses were permeability and the effects included thickness. The results of these ANOVA tests at the 95% confidence level (P = 0.033 and R^2 = 25.03%) showed that the permeability was borderline statistically significant to thickness. The R^2 value of 25.03% for a comparison between the variables of permeability and thickness indicated that a weak relationship existed.

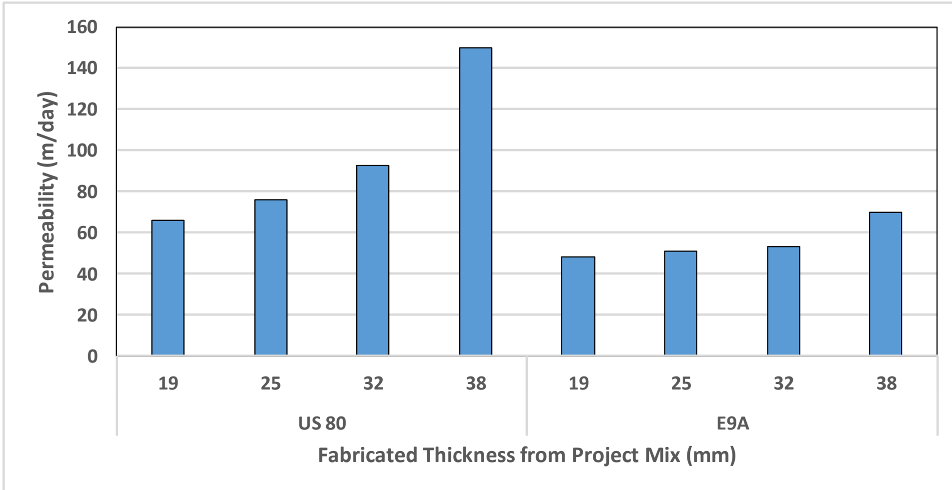

Figure 39. Comparison of OGFC Layer Thickness to Permeability

The coarser U.S. 80 mixture gives better permeability than E9A and only 38 mm (1.50 inch) thickness of the U.S. 80 project met the criteria of 100 m/day recommended by the ASTM OGFC design procedure. Although air voids for both mixtures are generally in the same range (15-18%), the U.S. 80 mixture had higher permeability probably due to the coarser gradation and more interconnected voids. These results indicate that the permeability function of OGFC can be better achieved by using a coarser mix or by increasing the thickness of OGFC layers.

7 CONCLUSIONS

Based on the field and laboratory investigations to identify the possible causes of premature failure of OGFC layers and the experiments and analysis to improve their durability, the following conclusions are drawn.

- The residual tack rate most often used on Alabama projects was 0.02 gal/sy. This is below the minimum application rate. ALDOT specifications allow the engineer to reduce the tack rate when overlaying a recently placed layer, but this practice is more applicable to dense-graded mixes and should not be practiced when placing OGFC pavements.
- Spray paver-laid OGFC projects generally perform better than when conventional emulsion is distributor applied, but the improvement is most likely because the tack rate for spray paver-laid applications is four to five times greater than conventional applications. As a result, ALDOT has increased the residual rate for CQS-1hp modified emulsions to 0.12-0.15 gal/sy.
- OGFC mixtures have been placed when air temperatures were lower than the minimum specified.
- Good performing OGFC sections have better cohesion as evidenced by higher IDT strength values.
- Thicker cores have higher IDT strength values as compared to thinner cores, but this may be due to the sensitivity of the test procedure to core thickness.

- The sandstone OGFC had more degradation than the granite mixture and was more susceptible to rutting and disintegration in the Hamburg test due to moisture interaction.
- OGFC pavements in the thickness range of 25-33 mm tend to perform better as compared to pavements with a thickness in the 15-25 mm range.
- Fog and rejuvenator seals significantly affect the micro and macro texture of OGFC surfaces.
- Rejuvenator seals appear to improve the abrasion resistance of OGFC pavements.
- The Cantabro test indicates that a rejuvenator application rate of 0.07-0.10 gal/sy may be suitable depending on the type of rejuvenator being applied and the type of modified binder used in the pavement.
- The HWTD test results clearly show a trend that the medium application rate (0.07 gal/sy) of rejuvenator is better in improving resistance to moisture susceptibility.
- OGFC pavements show better performance in terms of resistance to wear, raveling, and permeability if the pavement has a thickness/NMAS ratio of about 2.5.
- All of the OGFC mix designs that were part of the durability study in Phase 3 met the Cantabro loss requirement of 15% maximum.
- The state-approved 12.5-mm OGFC mixture had much lower tensile strength after moisture and freeze/thaw conditioning when compared with the 9.5 mm OGFC mixture.
- A 9.5 mm OGFC had higher tensile strength and improved permeability over a 12.5 mm OGFC.
- Adjustment in gradation, increase in layer thickness, and inclusion of performance tests (such as Cantabro stone loss, Hamburg rutting, and moisture susceptibility) in the mix design process can improve the performance of OGFC mixtures.
- Thicker layers reduced rutting and moisture susceptibility, reduced Cantabro stone loss, and improved permeability. However, these results may be affected by the sensitivity of the test procedure to core thickness.

8 RECOMMENDATIONS

Based on the conclusions of this research, the following recommendations are made:

- Apply tack coat toward the high side of specification tolerances. The practice of applying tack for OGFC at less than the minimum specified should be eliminated.
- Avoid placement of OGFC in cold weather. The minimum specification placement air temperature of 60°F should be enforced.
- Use the third or fourth load of mix to construct the beginning transverse joint to reduce the potential for raveling.
- Specify a minimum IDT strength to improve OGFC performance.
- Include a freeze/thaw cycle in the OGFC mix design procedure.

- Apply rejuvenator seals at a rate of 0.07-0.10 gal/yd^2. If rejuvenator seals reduce surface friction, a sand blotter may be needed.
- Use a thickness/NMAS ratio of 2.5 for OGFC pavements in order to improve internal cohesion and resistance to raveling.

REFERENCES

AASHTO T 324-08 (2008). *Standard method of test for Hamburg wheel-track testing of compacted hot mix asphalt*, American Association of State Highway and Transportation Officials, Washington, D.C.

Alabama Department of Transportation (1999). *ALDOT 259, OGFC Design Procedures*. Montgomery, Al.

Alderson, A. (1996). *The Design of Open Graded Asphalt.* Rep. No. CR C5151. Australian Asphalt Pavement Association, Victoria, Australia.

Alvarez, A. E., Epps Martin, A., Estakhri, C., Button, J. W., Glover, C. and Jung, S. H. (2006). *Synthesis of current practice on the design, construction, and maintenance of porous friction courses.* Rep. No. FHWA/TX-06/0-5262-1. Texas Transportation Institute-Texas A&M Univ., College Station, TX.

ASTM Standard D7064 / D7064M (2008). *Standard Practice for Open-Graded Friction Course (OGFC) Mix Design*. ASTM International, West Conshohocken, PA.

ASTM Standard E1911 (2009) AE01. *Standard test method for measuring paved surface frictional properties using the dynamic friction tester*. ASTM International, West Conshohocken, PA.

ASTM Standard E2157 (2009*). Standard test method for measuring pavement macrotexture properties using the circular track meter*. ASTM International, West Conshohocken, PA.

ASTM Standard D6752 / D6752M (2011). *Standard test method for bulk specific gravity and density of compacted bituminous mixtures using automatic vacuum sealing method*. ASTM International, West Conshohocken, PA.

ASTM Standard 1960, 2007 (2011). *Standard Practice for Calculating International Friction Index of a Pavement Surface*. ASTM International, West Conshohocken, PA.

Australian Asphalt Pavement Association (2004). *National Asphalt Specifications*, Edition 2. AAPA, Victoria, Australia.

Boheman, H. D. V. and Van De Laak, W. H. J. (2003). The influence of road infrastructure and traffic on soil, water and air quality. *Journal of Environmental Management*, 31 (1): 60 – 68.

Boving, T. V., Stolt, M. H., Augenstern, J. and Brosnan, B. (2007). Potential for localized ground water contamination in a porous pavement parking lot setting in Rhode Island. *Journal of Environmental Geology*, 55 (3): 571 – 582.

Brousseaud, Y. and Anfosso-Ledee, F. (2005). *Silvia Project Report: Review of Existing Low Noise Pavement Solutions in France*. Rep. No. SILVIA-LCPC-011-01- WP4-310505. European Commission on Sustainable Road Surfaces for Traffic Noise Control.

Brown, E. R., Hainin, M. R., Cooley, A. and Hurley, G. (2004) . *NCHRP Web Document 68 (Project 9-27): Relationships of HMA In-Place Air Voids, Lift Thickness, and Permeability Volume 3*. Transportation Research Board, Washington, D.C.

Brown, E. R., Kandhal, P. S., Roberts, F. L., Kim, Y. R., Lee, D. Y. and Kennedy, T. W. (2009). *Hot Mix Asphalt Materials, Mixture Design, and Construction*. 3rd Ed., National Asphalt Pavement Association Research and Educational Foundation, Lanham, MD.

California Department of Transportation (2006). *Open Graded Friction Course Usage Guideline*. MS # 5, Division of Engineering Services, Materials Engineering and Testing Services, Office of Flexible Pavement Materials, Caltrans, Sacramento, CA.

Cetin, A (2013). Effects of Crumb Rubber Size and Concentration on Performance of Porous Asphalt Mixtures. *International Journal of Polymer Science, Volume 2013, Article ID 789612.*

Christensen, D. W. and Bonaquist, R. F. (2006). *NCHRP Report 567: Volumetric Requirements for Superpave Mix Design*. Transportation Research Board, Washington, D.C.

Choubane, B., Page, G. C. and Musselman, J. A. (1998). Investigation of Water Permeability of Coarse Graded Superpave Pavements. *Journal of the Association of Asphalt Paving Technologists. 67:254-276*.

Cooley, Jr. L. A., Brown, E. R. and Watson, D. E. (2000). *Evaluation of OGFC mixtures containing cellulose fibers.* NCAT Rep. No. 2000-05. National Center for Asphalt Technology, Auburn University, Auburn, Al.

Cooley, Jr. L. A., Ahlrich, R. C., Watson, D. E. and Kandhal, P. S. (2007). *Improved porous friction courses (IPFC) on asphalt airfield pavements*. Final Report. Airfield Asphalt Pavement Technology Program, Auburn University, Auburn, Alabama.

Cooley, Jr. L. A., Brumfield, J. W., Mallick, R. B., Mogawer, W. S., Partl, M., Poulikakos, L. and Kicks, G. (2009). *NCHRP Report 640: Construction and Maintenance Practices for Permeable Friction Courses*. Transportation Research Board, Washington, D.C.

Cooley Jr, L. A. (1999) *Permeability of Superpave mixtures: evaluation of field permeameters*. NCAT Report No. 99-1. National Center for Asphalt Technology.

Estakhri, C. K., Alvarez, A. E. and Martin, A. E. (2008). *Guidelines on Construction and Maintenance of Porous Friction Courses in Texas*. Rep. No. 0-5262-2. Texas Transportation Institute-Texas A&M Univ., College Station, TX.

FABB, T.R.J. (1993). *The Case for the Use of Porous Asphalt in the UK*. The Asphalt Year Book 1993:46-59. Institute of Asphalt Technology, Staines, UK.

Faghri, M. (2002). *Performance improvement of open-graded asphalt mixes*. Report on URI-TC Project No. 536144. University of Rhode Island Transportation Center, Kingston, Rhode Island.

FHWA (1990). *Open-Graded Friction Courses FHWA Mix Design Method*. Technical Advisory T 5040.31. Federal Highway Administration, U.S. Department of Transportation, Washington, D.C.

FHWA (2008). *Evaluation and Recommended Improvements for Mix Design of Permeable Friction Courses*. Rep. No. FHWA/TX-08/0-5262-3. Texas Transportation Institute-Texas A&M Univ., College Station, TX.

Hamzah, M. O., Hasan, M., Che Wan, C. N. and Abdullah, N. H. (2010). A comparative study on performance of Malaysian porous asphalt mixes incorporating conventional and modified binders. *Journal of Applied Sciences*, 10 (20): 2403-2410.

Huber, G. (2000). *NCHRP Synthesis of Highway Practice 284: Performance Survey on Open-Graded Friction Course Mixes*. Transportation Research Board, Washington, D.C.

Jackson, N. C., Vargas, A., and Pucinelli, J. (2008). *Quieter Pavement Survey*. Publication WA-RD 688.1. Washington State Transportation Commission, Department of Transportation, Olympia, WA.

Kandhal, P. S. (2002). *Design, Construction, and Maintenance of Open-Graded Asphalt Friction Courses*. National Asphalt Pavement Association, Lanham, MD.

Larson, T. D. (1991). *Report on the 1990 European asphalt study tour*. American Association of State Highway and Transportation Officials, Washington, D.C.

Mallick, R. B., Kandhal, P. S., Cooley, L. A., and Watson, D. E., Jr. (2000). Design, construction and performance of new-generation open-graded friction courses. *Journal of the Association of Asphalt Paving Technologists.* 69: 391–423.

Martin, C., Ruperd Y. and Legret, M. (2007). Urban storm water drainage management: The development of a multi criteria decision aid approach for best management practices. *European journal of Operational research*, 181 (1): 338-349.

Miller, J. S., and W. Y. Bellinger (2003). *Distress Identification Manual for Long- Term Pavement performance Program (Fourth Revised Edition)*. Publication FHWA-RD-03-031. FHWA, U.S. Department of Transportation.

Pavement Interactive. *Durability*. Pavia Systems, Seattle, WA. http://www.pavementinteractive.org/article/durability/ Accessed April 2010.

Pagotto, C., Legret, M. and Clooirec, P. Le (2000). Comparison of the hydraulic behavior and the quality of highway runoff according to the type of pavement. *Water Research*, 34 (18): 4446-4454.

Poulikakos, L. D., Takahashi, S. and Partl, M. N. (2003). A Comparison of Swiss and Japanese Porous Asphalt through various Mechanical Tests. *Proceedings of the 3rd Swiss Transportation Research Conference (STRC)*, Ascona, Switzerland.

Sengoz, B. and Topal, A. (2007). Minimum voids in mineral aggregate in hot-mix asphalt based on asphalt film thickness. *Building and Environment*, 42 (10).

Smith, R. W., Rice, J. M. and Spelman, S. R. (1974). *Design of Open-Graded Friction Courses*. Report No. FHWA-RD-74-2. Federal Highway Administration, U.S. Department of Transportation, Washington DC.

TxDOT (2004). *Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges*. Texas Department of Transportation. Austin, TX.

Watson, D. E., Moore, K. A., Williams, K. and Cooley, L. A. (2003). Refinement of New-Generation Open-Graded Friction Course Mix Design. *Proceedings of the 82nd Transportation Research board Annual Meeting (CD-ROM)*. Transportation Research Board, Washington, D.C.

Watson, D. E., Cooley, L. A., Jr., Moore, K. A., and Williams, K. (2004). Laboratory Performance Testing of Open-Graded Friction Course Mixtures. *Transportation Research Record 1891: 40-47,* Transportation Research Board, Washington, D.C.