EVALUATION OF OPEN-GRADED FRICTION COURSES: CONSTRUCTION, MAINTENANCE, AND PERFORMANCE

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16. Abstract

The South Carolina Department of Transportation (SCDOT) has been using open graded friction courses (OGFC) since the mid-1970s to reduce accidents on high volume routes. While the permeability of OGFC has several advantages, there are also concerns about the performance and maintenance of these pavement layers. The primary research objective of this study was to identify methods to improve the design, performance, construction, and maintenance of OGFCs in South Carolina. To accomplish this objective, several tasks including literature review, surveys, laboratory investigations, and field evaluations were completed.

The results of this study led to several recommendations for SCDOT to consider in the future design, construction, and maintenance of OGFCs. These included adjustments to the mix design procedure for determining the optimum binder content, consideration of an alternative aggregate gradation to further, a procedure for determining the necessary thickness of OGFC layers, best practices for construction of OGFCs, and potential maintenance solutions.

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CHAPTER 1: INTRODUCTION

An open graded friction course (OGFC) is a thin layer of permeable asphalt which is placed on top of traditional dense graded asphalt pavement. It was created from experimentation with plant seal mixes (PSMs) in the 1940's (Huber 2000). The seal mixes were to provide a better performing alternative to chip seals and their popularity increased throughout the United States in the 1970's. Japan, as well as many European countries also began using OGFCs on their roadways (Kandhal 2002).

California was the first state to begin using OGFCs in the United States. Their plant seal mixes were applied in a thin layer, used a smaller nominal aggregate size, and increased binder content as compared to traditional paving mixes. This provided similar benefits to the chip seals, but also resulted in reduced road noise, increased durability, and a better ride quality (Kandhal 2002).

OGFCs gained popularity across the United States in the 1970's in response to the FHWA's program to increase frictional resistance on roadways (Kandhal 2002). Due to durability problems from the altered mix design of PSMs, many states discontinued use in the 1980s. Some states, however, including Georgia, Texas and Oregon, tried to improve the mix designs and continued its use. The changes included using a polymer modified binder and fiber additives to stabilize the mix and decrease binder draindown; increased the binder content and air voids; and specification of more durable aggregates. The modified binder also produced a thicker film on the aggregate particles which decreased oxidation and raveling (Fitts 2002).

In Europe, this asphalt pavement type is referred to as Porous European Mixes (PEMs). They are similar to the OGFC mixes used in the US, with a few subtle differences. The European mixes tend to have a higher air void content of 18-22% as compared to OGFCs at 15%, and are specified by a minimum air void content. The gradation of PEMs is a bit more gap graded as seen in Table 1.1 (Watson et al. 1998) and polymer modified binders are used almost exclusively. The differences in air content and gradation make PEMs more permeable than OGFCs (Watson et al. 1998). Also, the aggregate standards are higher in Europe than in the United States (Huber 2000).

Table 1.1. Comparison of gradation specifications for OGFC and PEM (Watson et al. 1998)

Sieve Size	Percent Passing	
	12.5mm OGFC	**12.5mm PEM**
3/4 inch (19 mm)	100	100
1/2 inch (12.5 mm)	85-100	90-100
3/8 inch (9.5 mm)	55-75	35-60
#4 (4.75 mm)	15-25	10-25
#8 (2.36 mm)	5-10	5-10
#200 (0.075 mm)	2-4	1-4

In the early 1960's, the United Kingdom began using porous pavement in military airfield runways to avoid hydroplaning and skidding in wet weather (Hwee and Guwe 2004). After research into the advanced aging and hardening was conducted, the mix design changed to use higher binder contents with additives to prevent draindown. This pavement was then allowed on main roadways where the benefits were shown to outweigh the disadvantages (Nielsen 2006).

The use of porous asphalt in France began in 1976 and its use grew through 1990 when winter maintenance recommendations discouraged use. French research studies have determined that modified binder is necessary to help minimize raveling and draindown. It was also found that this pavement type should only be used on roadways with high design speeds (50 mph) (Nielsen 2006).

The Netherlands were introduced to porous asphalt in the early 1980's and by 1990, it was decided that the entire highway network was to be paved with porous asphalt. The OGFC pavements typically lasted 10-12 years with maintenance or rehabilitation being required due to raveling (Nielsen 2006).

The use and performance of open graded friction courses across the US is highly variable. In 1998, a survey of transportation departments was conducted by the National Center for Asphalt Technology (NCAT). The survey evaluated the use, performance, design and construction methods of OGFCs. It was found that 38% of the respondents had discontinued the use of OGFCs on their roadways while only 8% had never used this pavement type at all. The estimated service life was found to be between 8 and 12 years with good to very good durability and surface friction performance. Most respondents had a mix design, while some used a recipe and others used a combination of the two. Also, there was a determination of mix temperature by either the FHWA test (visual inspection of draindown), draindown test, or standard temperature-viscosity charts. The additives used included fiber, silicone, rubber, liquid anti-striping agents, and hydrated lime (Kandhal and Mallick 1998).

Problem Statement

The South Carolina Department of Transportation (SCDOT) has been using OGFCs since the mid-1970s to reduce accidents on high volume routes. OGFC has a permeable aggregate matrix which allows water to flow through the mix and off of the road, thereby reducing the potential for hydroplaning and visible spray from other vehicles. The permeability of the OGFC also creates a quieter riding surface that reduces highway noise as compared to conventional asphalt and concrete mixtures. Since the mix is permeable, it also allows air to circulate through the mix making it more susceptible to "icing" during the winter months. Another concern with OGFC is that asphalt binder will prematurely oxidize which will potentially cause raveling that often leads to noisy or rough riding pavements during its last years of serviceable life. OGFC has safety benefits but at the same time can create issues in the areas of maintenance and long term performance.

Study Objectives and Scope

The primary research objective of this study was to identify methods to improve the design, performance, construction, and maintenance of open graded friction courses in South Carolina. To accomplish the primary objective, there were four specific objectives:

1. Develop improved guidelines for the design of higher quality OGFCs.
2. Develop improved guidelines for the construction of better performing OGFCs.

3. Develop maintenance guidelines for pavements constructed with OGFC wearing courses.
4. Develop guidelines for the use of OGFC mixtures to improve roadway safety and produce quieter roadways.

Organization of Report

To accomplish the objective of this study, the following tasks were completed. The report is organized into chapters dedicated to each of the tasks.

1. Literature Review – An extensive literature review of the design, construction, maintenance, and performance of OGFC mixtures was conducted to learn about the prior research and best practices related to OGFCs in the US and internationally. The literature review is included in the appropriate chapters throughout this report.
2. Survey of OGFC Usage and Specifications – A survey of state DOTs was conducted to gather information related to OGFC in other states. Specific survey topics included maintenance issues, ice and snow management, clogging issues, overlay practices, construction guidelines, mix design procedures, and pavement design considerations. There were several surveys published in the 1990s and early 2000s, and most recently in 2009. These surveys were used as a basis for the survey with a short supplemental survey to gather further information pertinent to this study.
3. Laboratory Evaluation of OGFC Mix Design Procedures – There are several methods to determine the optimum binder content of OGFC mixtures. Some are based on measuring mixture properties, while others are more qualitative. The current method utilized by SCDOT was compared to methods from other states to identify modifications that should be made to the current SCDOT mix design method for OGFC.
4. Laboratory Evaluation of OGFC Aggregate Gradations & Properties – There are several aggregate gradations used for OGFC across the US and worldwide. Additionally, aggregate property requirements vary widely. This study evaluated the properties of OGFC mixtures produced with aggregates of varying properties (e.g., LA abrasion loss and/or Micro-Deval abrasion value) and gradations. The mix properties tested included volumetrics, permeability, moisture susceptibility, rutting resistance, mix durability, and clogging susceptibility.
5. Evaluation of OGFC Construction – OGFC construction practices were observed and evaluated during the project duration. This enabled the investigators to observe paving practices and issues on a first-hand basis.
6. Evaluation of OGFC Pavement Maintenance Procedures – Maintenance practices for OGFC mixtures were investigated. Of specific focus were the issues related to patching OGFC mixtures and pavement preservation treatments. Patching practices for OGFC were evaluated to identify proper patching techniques that will not restrict the flow of water through the void structure in the OGFC layer.

CHAPTER 2: SURVEY OF THE USE OF OGFC IN THE UNITED STATES

OGFC has been used in the US for many years with varying levels of performance. To gain a better understanding of the use of OGFC, its performance, and the state-of-the practice in OGFC construction and maintenance, the research team had planned to conduct a comprehensive survey to obtain the desired information. However, such a survey was conducted by Cooley et al as part of NCHRP Project 09-41 and the results are reported in NCHRP Report 640 (Cooley et al 2009). The survey was distributed to highway agencies in the US and around the world and included questions related to general use, structural design, mix design, construction practices, maintenance and rehabilitation, and performance. Responses were received from 32 states plus four Canadian provinces, Austria, and Japan.

Figure 2.1 summarizes the use of OGFC amongst the agencies that responded to the survey. It was also noted that with the exception of Oregon and California, the use of OGFC in the US was limited to the southeastern states. The volume of OGFC placed per year is relatively low compared to conventional asphalt mixtures. Six states reported usage greater than 100,000 tons per year and five states reported less than 20,000 tons annually. This is to be expected due to the fact that OGFC is most commonly used on higher speed roadways such as urban freeways (75% of respondents) and rural primary highways (50% of respondents). Only three states reported that they use OGFC on the five major classes or roadways (urban freeways, urban arterials, urban collectors, rural interstates, and rural primary highways). Additionally, the low production volumes is also the result of thin lifts of approximately 1-inch thick, with the exception of one state that requires a minimum thickness of 1.5-inches.

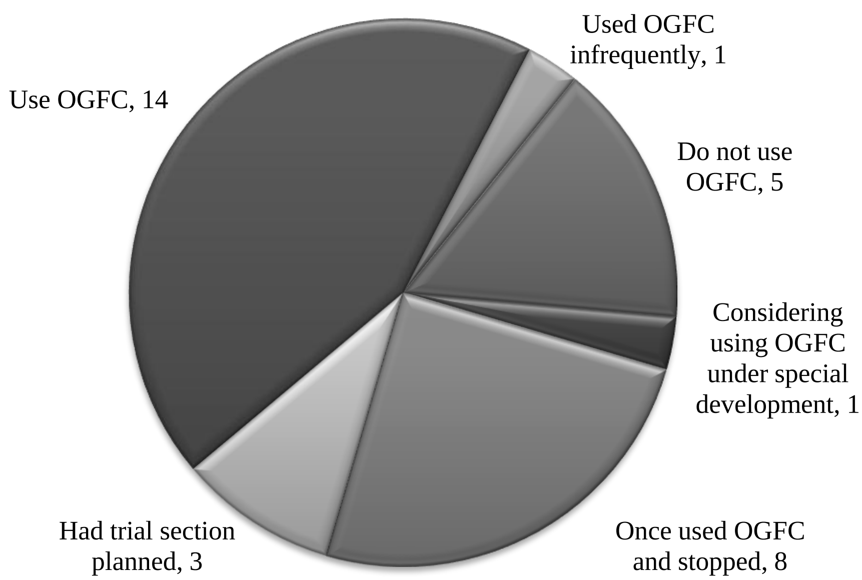


Figure 2.1. Usage of OGFC in the US ((Cooley et al 2009)

In addition to the survey conducted as part of NCHRP Project 09-41, another survey was created as part of this research project to collect information on the use, specifications, and design of OGFCs across the US. The survey was distributed to the 50 US DOTs as well as some Canadian provinces. The questions and response options are listed below. There were 45 total responses to this survey which included 41 different highway agencies. Figure 2.2 shows the distribution of responses to Question 2 regarding the current use of OGFCs (not including Ontario, Canada and the District of Columbia which both indicated that they do not currently use OGFCs). The results show that 61% of respondents are currently using OGFCs on roadways in their jurisdiction.

1. Please provide your contact information.
    - Name
    - Job Title
    - Organization
    - Email
    - Phone

2. Does your organization use open graded friction courses on roadways in your state? If yes, then please answer Questions 3 and 4. If no, then you may skip to the end and submit your survey.
    - Yes
    - No

3. Does your organization have a standard specification, supplemental specification, or neither for OGFC?
    - Standard Specification
    - Supplemental Specification
    - Other (e.g., design memo, etc.)
    - No Specification or Policy

4. Does your organization have a standard method for determining the optimum binder content in OGFC mixtures?
    - Yes
    - No

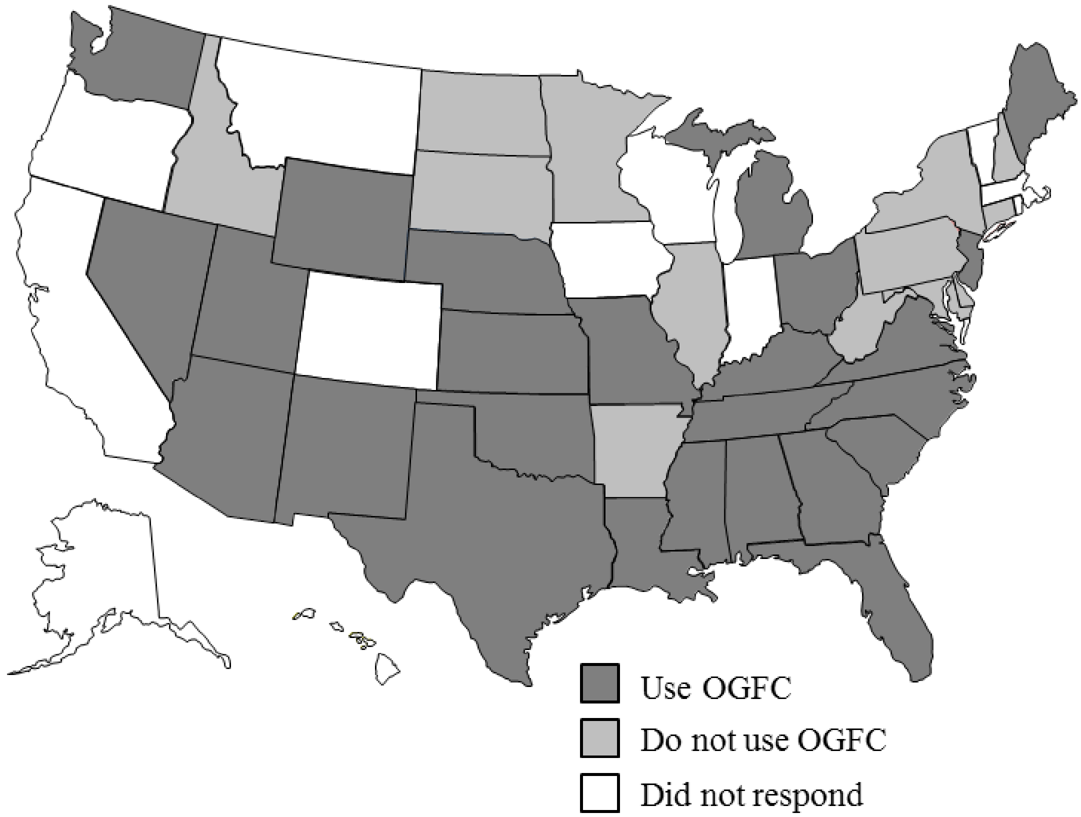


Figure 2.2. Current use of OGFCs on roadways

The next question was regarding the specification type which is used for OGFCs. Figure 2.3 summarizes the responses of the agencies which currently use OGFCs on their roadways. The majority of agencies that responded do have a standard specification for OGFCs, while the remainder of the agencies was split between having a supplemental specification and some other method of specification.

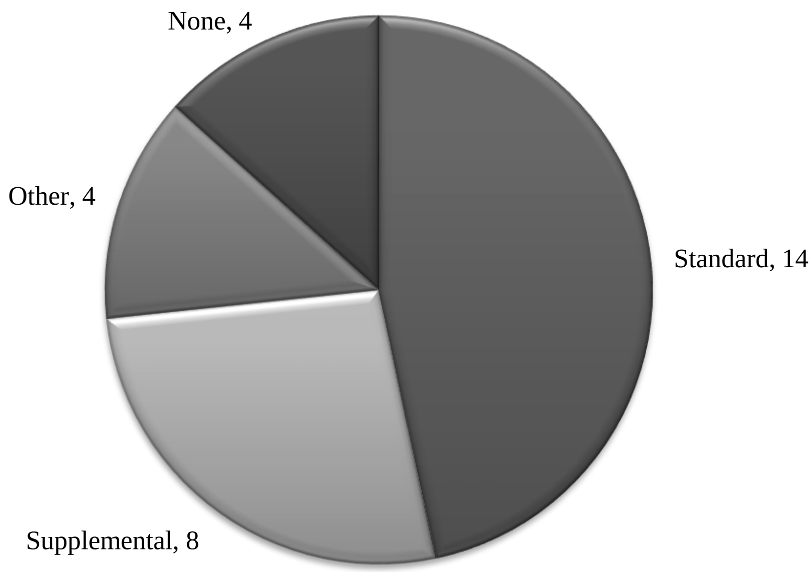


Figure 2.3. Type of specification used with OGFCs

The final question asked whether the agencies had a standard procedure for determining the optimum binder content of an OGFC mixture. The results shown in Figure 2.4 indicate that 20 of agencies who currently use OGFCs on their roadways (and responded to this question of the survey) do have a standard procedure for determining the optimum binder content, which is discussed in further detail in Chapter 4.

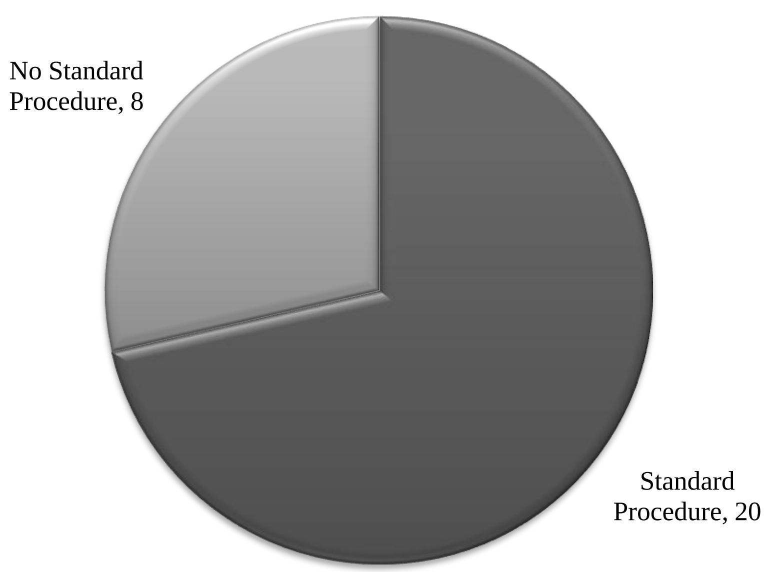


Figure 2.4. Procedure for determining optimum binder content

CHAPTER 3: PERFORMANCE OF OGFC

OGFCs can provide numerous advantages on roadways. The increased air void content of OGFCs leads to interconnected permeable voids, which create permeability in the pavement. The water from storm events infiltrates through the pores and can be removed from the surface immediately. This reduces splash and spray of water during rain events, hydroplaning of vehicles, and glare caused by reflected light, all of which are caused by a film of water on the surface of roadways. Even during a longer storm event which causes the OGFC surface to remain wet, the pressure exhibited on a roadway from vehicle tires will dissipate through additional voids and keep the pavement tire interaction strong (Kandhal 2002). A study conducted in the United Kingdom showed that an OGFC layer can reduce water spray up to 95% when traveling approximately 10 feet behind a truck (Nicholls 1997). The surface texture of OGFCs helps reduce noise, increase friction, and enhance visibility. The porous design can also help improve stormwater runoff quality issues. Additionally, OGFCs are placed in thin layers; therefore, high quality aggregate can be conserved. However, the high air content of OGFCs can lead to durability related problems, increased and specialized winter maintenance, and pore clogging (Table 3.1).

Table 3.1. Problems encountered with open graded friction courses (Nielsen 2006; Kandhal and Mallick 1998)

Agency		Typical Problems Encountered
International	Austria	Raveling
	Germany	Raveling
	France	Raveling
	The Netherlands	Raveling & Rapid Aging
	Spain	Raveling & Pore Clogging
	United Kingdom	Pore Clogging & Rapid Aging
United States	Alaska	Ice Removal
	Colorado	Stripping
	Hawaii	Raveling
	Idaho	Pore Clogging
	Iowa	Ice Removal
	Kansas	Ice Removal
	Louisiana	Raveling
	Maine	Ice Removal
	Maryland	Raveling
	Minnesota	Raveling & Pore Clogging
	Rhode Island	Raveling
	South Dakota	Pore Clogging
	Tennessee	Stripping & Ice Removal
	Virginia	Stripping

In a 2009 report summarizing a survey of OGFC usage and performance in the US and Europe (Cooley et al. 2009), a portion of the survey was dedicated to life expectancy of OGFC surface courses. Responses ranged from less than six years to as high as 15 years. The majority of respondents indicated that the typical service life for OGFC in their jurisdictions was 8 to 10 years. The service life of OGFC pavements has been defined as the length of time that an OGFC pavement maintains its frictional properties and smoothness. The service life is different than the performance life of an OGFC pavement in that the performance life is defined as the length of time the pavement maintains its beneficial characteristics including permeability and tire/pavement noise reduction (Huber 2000). Experience has shown that permeability of an OGFC pavement can remain for 1 to 5 years without any maintenance activities (Isenring et al. 1990). However, with proper maintenance, permeability can be maintained for a longer period. With regard to sound absorption, Graf and Simond reported that sound absorption of OGFCs maintained reduced tire/pavement noise levels for up to 9 years (2005).

Functional Performance

The functional performance of an OGFC refers to the properties related to the performance of the void structure. This includes properties mainly related to water infiltration and noise reduction. OGFCs have a filter-like structure of interconnected voids which allow water to pass through, but are also susceptible to small particles which can get caught in the voids and clog the pores in the pavement. Maintenance practices can help to minimize clogging by cleaning the voids on a regular basis. Clogging can also be decreased by only using OGFCs on high speed roadways as the traffic creates a suction action when the tires roll over the pavement, thus limiting the ability for clogging material to settle in voids in the mix. A 2008 study of clogging characteristics found that the relationship between permeability and air voids follows a power model relationship and pavements which have initial permeability of over 164 in/hr have good drainage potential even after clogged conditions (Suresha et al. 2008).

Noise Reduction

The open aggregate gradation of OGFCs creates a different surface texture than traditional asphalt pavement. This helps trap unwanted noise, increase the friction and skid resistance, and enhance the visibility for drivers on roadways. A 2004 study by the Colorado DOT found that air voids and noise had a linear indirect relationship. After testing 19 sites, the quietest pavement type was determined to be OGFC pavements (Hanson and James 2004). OGFCs can also be considered as an alternative to sound barriers as they may be less expensive and can help decrease noise to even tall structures (Kandhal 2002).

Safety

Frictional resistance is an important safety factor in roadway construction. Low friction roadways have increased potential for accidents, especially in wet weather. The frictional resistance of a pavement is characterized by the friction number. As speed increases, the friction decreases. Table 3.2 shows the results of Pennsylvania DOT pavement friction testing conducted in 1975. The OGFC showed higher friction numbers even with various aggregate types (Brunner 1975). The Virginia DOT also reported friction numbers for OGFC pavements ranging from 51 to 72 (Maupin 1976). A study done in France reported 52 accidents on a roadway between 1979 and 1985. After an OGFC layer was placed, zero accidents occurred in the same section of roadway from 1985 to 1989 (Chaignon 1993). In 2008, the South Carolina DOT also noted the safety benefits of OGFC after an analysis showed that wet weather crashes at interstate locations decreased by 26% per year after paving the surface with OGFC (Werts 2008).

Table 3.2. OGFC pavement friction data (obtained by the Pennsylvania DOT) (Brunner 1975)

Pavement Type	Friction Number 30 mph	Friction Number 40 mph
OGFC with gravel	74	73
OGFC with dolomite	71	70
Dense graded HMA with gravel	68	60
Dense graded HMA with dolomite	65	57

Stormwater Runoff Quality

OGFCs are created by eliminating the majority of fines in the aggregate gradation. This creates a permeable interconnected void structure that can work as a filter for stormwater. As water from stormwater passes through the surface course, fines and other pollutants can be removed. In a University of Texas research project, runoff water from a pavement was collected. The pavement began as traditional dense graded asphalt, but after a few months, was overlaid with an OGFC. Table 3.3 shows the reduction in pollutants that was measured after the OGFC overlay (Barrett et al. 2006).

Table 3.3. Reduction in stormwater runoff pollutants (Barrett et al. 2006)

Pollutant	Reduction (%)
Total Suspended Solids	91
Total Kjeldahl Nitrogen	2
Total Phosphorus	35
Total Copper	47
Total Lead	90
Total Zinc	75
Dissolved Zinc	30

Structural Performance

OGFCs are not typically considered a structural portion of pavement, but some states, including South Carolina, and other countries do consider OGFC layers as providing structural capacity to the pavement structure. Although they provide several benefits, adding them to a pavement system will be an additional cost. Other characteristics which should be considered are durability issues, winter maintenance, and clogging of the surface voids. Table 3.1 shows problems encountered with open graded friction courses (Nielsen 2006; Kandhal and Mallick 1998). In 2004, Pucher et al. reported that OGFCs deteriorate slowly in the first 5 – 10 years of service, but after this initial deterioration phase, the rate of deterioration typically increases (Pucher et al. 2004).

Durability

The durability of an OGFC mixture is different than a traditional HMA mix. The interconnected permeable voids not only allow water to drain, but also allow air to flow through the entire structure, rather than only the top surface. Over time, oxygen reacts with the binder and accelerates the aging, making the binder more brittle. This can lead to decreased performance and an increased level of distress. In a survey of OGFC use and performance in the

US, it was found that the most common problems with OGFCs were stripping of existing underlying pavement, raveling, and winter maintenance issues (Kandhal and Mallick 1998). Stripping of a pavement is when the aggregate and binder become separated. This typically occurs due to fines which surround the aggregate particles and prevent a good bond between the binder and the aggregate during mixing, or water which weakens the bond between the aggregate and binder due to inadequate drainage (Kandhal and Mallick 1998).

Raveling, the most common distress identified in OGFCs (Huber 2000; Cooley et al. 2009), is when particles of aggregate still coated with binder lose adherence to the pavement mix. This typically occurs due to inadequate binder contents, vehicle chemicals disintegrating the binder, or stripping of the asphalt mix. The high air content in OGFC mixtures causes a decreased cohesion of particles and can result in raveling of the pavement if other factors (e.g., low binder content, inadequate compaction, binder draindown) are present (Kandhal and Mallick 1998). Additionally, two types of raveling have been described by Molenaar and Molenaar (2000): short-term and long-term raveling. Short-term raveling, caused by intense shearing forces at the tire pavement interface that occurs within newly placed OGFCs (Molenaar and Molenaar 2000), generally occurs shortly after traffic flow on the pavement begins (Pucher et al. 2004). Short-term raveling can be exacerbated by placing the OGFC mix at too low of a temperature, incomplete seating of aggregates during compaction, and areas having low asphalt binder contents as a result of draindown. Long-term raveling was described as being the result of long-term segregation of the binder from aggregate due to gradual draindown over time. This results in a low binder content of the OGFC mix closest to the wearing surface, which can be dislodged under the action of traffic. It should be noted that long-term draindown has been mostly seen in mixtures that did not include modified binders (Molenaar and Molenaar 2000).

<u>Performance of OGFC in South Carolina</u>

As indicated previously, OGFC has been used on interstate roadways in South Carolina for decades. However, over the years that SCDOT has been using OGFC wearing surfaces, the performance has varied for many reasons. The most common distress in OGFCs identified by SCDOT personnel is raveling, which is representative of the trend throughout the rest of the US. As part of this study, the research team visually inspected interstate routes in South Carolina to identify the relative frequency and potential causes of raveling in OGFCs. Table 3.4 summarizes the locations of the pavements that were evaluated.

During this evaluation, areas of discrete raveling were targeted. These areas were identified as those where the OGFC raveling was confined to a discrete area (i.e., not an entire section of pavement having general surface wear). The reason for limiting the evaluation to these areas is that pavements experience wear over time due to traffic, but areas of severe raveling that are confined to a relatively small area are typically caused by reasons other than routine traffic wear. Additionally, as the evaluation progressed, it was noticed that a majority of the raveled areas occurred at a transverse construction joint (including tie-ins coming off of bridges). The presence of linear transverse construction joints at these areas was easily identified, even while driving at the posted speed limit. It should also be noted that in sections of I-85 and I-20 there were areas that had been patched over the entire lane width with a dense graded mix for extended lengths. Based on the frequency of raveling in these areas, the patched areas were likely areas that had experienced severe raveling. These patched areas were not included in the raveling observations because it was not certain that this was in fact the case.

Table 3.4. Summary of OGFC raveling occurrences on South Carolina interstates

Interstate	Direction	Lane Miles of OGFC*	Raveling Observations by Location		Raveling Observations Occurring at Joints
			Joint	Other	
I-26	East	110.4	28	18	60.9%
	West	122.4	27	15	64.3%
I-126	East	10.5	1	0	100%
	West	10.0	4	0	100%
I-526	East	9.3	0	0	n/a
	West	9.2	0	0	n/a
I-85	North	121.3	22	17	56.4%
	South	131.3	22	11	66.7%
I-185	North	4.0	0	0	n/a
	South	4.0	0	0	n/a
I-385	North	30.1	3	0	100%
	South	30.3	7	0	100%
I-20	East	75.1	6	0	100%
	West	111.0	22	54	40.7%
Total		770.9	142	115	55.3%

** Lane miles are approximate (based on odometer readings).*

Based on the results of this low-level evaluation, it is clear that this specific discrete raveling originates at construction joints more commonly than between joints. There are several potential reasons for this phenomenon, but the most likely reason is due to a reduced level of compaction of the first load of mix that is laid by the paver. As the first load of mix enters the material transfer vehicle (MTV) and then into the paver, it cools down because the equipment is cooler than mix thus causing accelerated cooling of the mixture. When discharged through the paver, the cooler mixture is more difficult to compact and seat the aggregate particles with typical compactive effort. As mix continues to move through the MTV and paver, the mix cools less and is therefore more effectively compacted. For this reason, there is likely a compaction gradient from the start of a construction joint in the direction of paving until the paving equipment heats up and the discharged mix reaches an equilibrium temperature. A reduction in the level of compaction of the pavement mat results in a reduction in cohesion within the mix thus leading to raveling in these areas. This also occurs as the paving crew moves over a bridge and commences paving on the other side. In this case, there are two potential causes of mixture cooling. First, the mix remaining in the paving equipment can cool down while making the move over the bridge. Second, if the equipment is run empty prior to crossing the bridge, the equipment will cool down, which creates similar cooling issues as start-up.

Another potential cause for the occurrence of isolated raveling near transverse joints could be the result of over-compaction. When paving from a transverse joint, there is the possibility of over-compacting the OGFC in an effort to create a smooth joint transition. In this case, the high compactive effort imparted by the rollers could potentially result in breakdown of the aggregate particles, which could cause the fractured particles to ravel prematurely. While this has not been validated, it is something to investigate further.

During the evaluation, other distresses were also identified, however, the frequency or severity were not specifically recorded. In addition to raveling, other common distresses, yet less frequent than raveling are included in Table 3.5.

Table 3.5. Distresses other than raveling observed on South Carolina OGFC sections.

Distress	**Description**	**Potential Causes**
Cuts and gouges	Gouges in the direction of traffic.	Traffic accidents and tire blow-outs where the wheel rim, or other part of the vehicle rides directly on the pavement.
Delamination	Observed in some locations over underlying pavement markings. Underlying pavement markings are clean (no evidence of tack).	Lack of adequate bond between the pavement marking and layer of OGFC.
Cracking	Longitudinal cracking at joints.	Reflective cracking over longitudinal joints.
Raveling	Deeper surface texture in the wheel paths over larger areas than observed in Table 3.4. Typically in the outside pavement lane.	Wearing due to traffic.

CHAPTER 4: OGFC MIX DESIGN PROCEDURES

The first widely used OGFC mix design in the US was created by the Federal Highway Administration (FHWA) in 1974. This was modified twice, and then through research at NCAT, a new generation mix design was created. Meanwhile, state highway departments have altered and adjusted their own mix design procedures as problems were encountered with their pavements.

The first FHWA open graded mix design procedure was published in 1974 (Watson et al. 2002). This design procedure was modified in 1980, and again in 1990. This procedure specifies materials, gradation, optimum binder content, mix temperature, and resistance to effects of water (FHWA 1990). The aggregate should be a high quality aggregate with the gradation included in Table 4.1 while the binder and additives are based on local conditions. The binder content is determined using the predominant aggregate size and oil absorbance testing. A draindown test is then used to determine the mixing temperature and measured by a visual inspection. A moisture resistance test is also required with at least 50% retained strength.

Table 4.1. Recommended OGFC aggregate gradations (FHWA 1990; Kandhal 2002)

Sieve Size	FHWA Gradation Percent Passing	NCAT Gradation Percent Passing
3/4 inch (19 mm)	-	100
1/2 inch (12.5 mm)	100	85-100
3/8 inch (9.5 mm)	95-100	55-75
#4 (4.75 mm)	30-50	10-25
#8 (2.36 mm)	5-15	5-10
#200 (0.075 mm)	2-5	2-4

A 1998 NCAT survey determined the mix design types that are being used in the United States (Kandhal and Mallick 1998). Of the 43 states that responded, 76% had a specification for OGFC mixes. Some states had special provisions while the remaining highway departments had no mix design. The majority of the job mix formulas were determined by mix design, while some states used a recipe and others used a hybrid of the two methods. It was also determined that a range of asphalt content was specified for approximately half of the states. The aggregate gradations used were similar, but there was a wide variation on the 3/8 inch, No. 4, and No.8 sieve sizes. The binder type and additives also varied as 48% of respondents used a polymer modified binder and 46% used additives other than polymer including fiber, silicone, crumb rubber, liquid anti-strip additives, or hydrated lime. Overall, it was suggested that an improved mix design procedure would be useful in helping states develop better OGFC mix design practices (Kandhal and Mallick 1998).

In 2000, NCAT published a new generation OGFC mix design based on research in response to the OGFC experiences in the US and Europe (Mallick et al. 2000). There are three primary components in the mix design. The first characteristic is material selection. A strong and durable aggregate should be chosen with recommended LA abrasion values of 30% or less. The aggregate should also be crushed, have minimal flat and elongated particles, and low absorption values. The binder type recommended is two grades higher than typically used in the area and should be polymer modified. Fibers are also recommended for strength and durability.

The second component is gradation. A recommended gradation is shown in Table 4.1, but is chosen by comparing the voids in coarse aggregate (VCA) of the mix to the VCA of the aggregate alone and the air voids of the mix. This ensures stone-on-stone contact of the aggregate particles and permeability. The final component is choosing the optimum binder content. The optimum binder content is determined by a series of tests on specimens compacted with a gyratory compactor. The mixture properties tested include air voids, abrasion on aged and unaged specimens, binder draindown, and moisture susceptibility (Kandhal 2002). The requirements are summarized in Table 4.2.

Table 4.2. NCAT mix design criteria (Kandhal 2002)

Criteria	**Recommended Value**
Air Voids	Minimum 18%
Unaged Cantabro Abrasion	Maximum 20%
Aged Cantabro Abrasion	Maximum 30%
Asphalt Binder Draindown	Maximum 0.3%
Tensile Strength Ratio	Minimum 80%

As part of this study a survey was administered where agencies were asked to submit mix design procedures. There were 25 agencies which indicated that they currently use OGFC on their roadways. Of those, 20 indicated that they have a standard procedure for determining the optimum binder content (OBC) in the OGFC mixture. These designs, as well as four national agency mix design procedures, were reviewed and categorized based on the determination of the optimum binder content. The four national organizations are the American Society for Testing and Materials (ASTM), the Federal Highway Administration (FHWA), the National Asphalt Pavement Association (NAPA), and the National Center for Asphalt Technology (NCAT). Table 4.3 shows the agencies and categorization of the mix designs. The mix designs for Louisiana DOT, Michigan DOT, and Utah DOT, although considered standard procedures, were not used in this research.

Table 4.3. Categorization of OGFC mix designs

Compacted Specimens	**Absorption Calculation**	**Visual Determination**
ASTM	FHWA	Florida DOT
NAPA	Alabama DOT	Georgia DOT*
NCAT	Arizona DOT	Nevada DOT
Georgia DOT*	Georgia DOT*	New Jersey DOT
Kansas DOT	Kentucky TC	South Carolina DOT
New Mexico DOT	Wyoming DOT	
North Carolina DOT		
Mississippi DOT		
Missouri DOT		
Nebraska DOT		
Tennessee DOT		
Texas DOT		
Virginia DOT		

*Use a combination of mix designs procedures

The first category of mix design is “compacted specimens.” These mix designs determine the optimum binder content by evaluating compacted specimens having a range of binder contents, similar to a typical asphalt mix design procedure. The specimens are evaluated using one or more criteria such as binder content range allowed, required air voids, abrasion resistance, moisture susceptibility, permeability, or other standard propertis or performance measurements. Table 4.4 shows the agencies which use this type of design procedure and which tests are required. It should be noted that the Kansas DOT uses the same procedure as the Texas DOT.

The next category of mix designs calculates the binder content based on an oil absorption value of the aggregate. In the FHWA procedure, the predominant aggregate size is soaked in oil, and then drained under particular conditions. This value of oil absorption is used in calculations to determine the optimum binder content of the mixture. This is followed closely by most of the agencies which fall in this category. Two agencies also factor in the absorption of the fine aggregate using a kerosene absorption value (Arizona DOT and Wyoming DOT). It should be noted that the Wyoming DOT uses the same procedure as the California DOT.

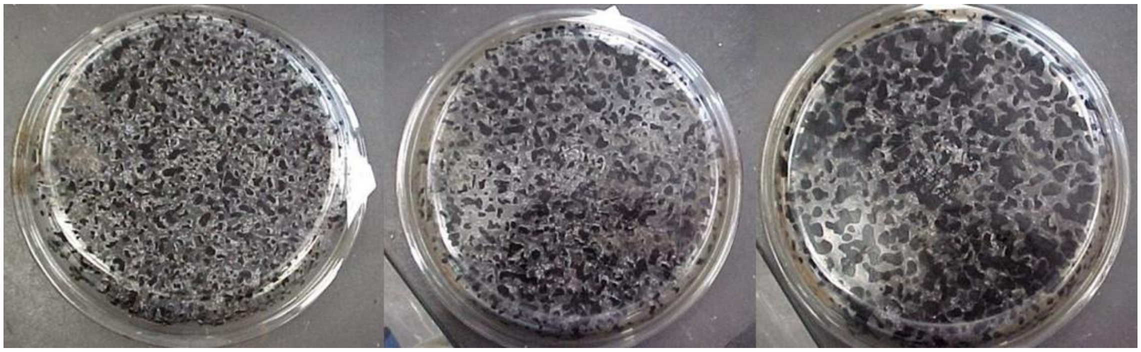
The final category of mix design uses a visual determination method. In this procedure, an uncompacted loose asphalt mixture is placed in a clear container and conditioned. The conditioning entails heating the mixture for a specified period of time. The time and temperature at which the mixture is heated varies by procedure. The optimum binder content shows some drainage where the asphalt mix comes in contact with the container, but not too much, as seen in Figure 4.1 (SCDOT 2010) and Figure 4.2 (FDOT 2009).

Figure 4.1. SCDOT OGFC mix design image reference

Table 4.4. OGFC mix design requirements for design methods in the Compacted Specimen category of mix design methods.

Agency	Mix Properties											Aggregate Properties		
	No. of Trial Binder Contents	Gyrations	$VCA_{mix} < VCA_{drc}$	Asphalt Content (%)	Air Voids (%)	Draindown (%)	Unaged Abrasion (%)	Aged Abrasion (%)	TSR (%)	Permeability (in/hr)	Other	LA Abrasion Loss (%)	Fractured Faces (>1/≥1 %)	Flat & Elongated Particles (%)
ASTM	3	50	X		≥18	≤0.3	≤20	≤30	≥80	≥164		≤30	90/95	≤10[4]
NAPA/NCAT	3	50	X		≥18	≤0.3	≤20	≤30	≥80			≤30	90/100	≤5[4]
GDOT											X[1]	≤45		≤10[4]
NMDOT	3			6.5		≤0.3			≥95[2]				75	
NCDOT	3	50			≥18	≤0.3	≤20			≥164		≤45	100	
MSDOT	4	50	X		≥15	≤0.3	≤30	≤40	X[3]	≥49		≤45	90	≤20[5]
MODOT		50	X	≥6.0	≥18	≤0.3	≤20		≥80			≤50		
NDOR	4	50		5.8-6.8	18±1	≤0.3						≤40	90/95	≤10[4]
TNDOT			X				≤20						70	≤20[5]
VDOT		50	X	5.75-7.25	≥16	≤0.3	≤20		≥80			≤40	90/100	≤10[4]

[1] – The volumetric portion of the Georgia procedure selects a binder content based on the minimum VMA value.
[2] – The New Mexico TSR value is a static immersion moisture susceptibility test, not an indirect tensile strength ratio.
[3] – The Mississippi design procedure requires the moisture susceptibility to be checked, no requirement is given for the value.
[4] – 5:1 ratio.
[5] – 3:1 ratio.



(a) (b) (c)

Figure 4.2. FDOT mix design image references (a) too little binder; (b) just right amount of binder; (c) too much binder

The Georgia DOT uses a combination of the three design categories. The GDOT design procedure has four main steps. The first step is an oil absorption test; an asphalt content is then calculated based on the amount of oil absorbed. Next, the volumetrics of compacted specimens are calculated. A binder content is also chosen from this procedure based on the lowest voids in mineral aggregate (VMA) value. The following step is a visual determination of the binder content. Finally, the three binder contents are averaged together to calculate the optimum binder content of the OGFC mixture (GDOT 2009).

After the survey results were analyzed and categorized, materials were obtained, and specimens were prepared. Various tests were then completed according to the design procedures to determine the optimum binder contents for the 14 different mix design procedures evaluated.

Experimental Materials and Methods

Material Selection

Three different crushed granite aggregate sources were used in this research. Each aggregate source is located in the state of South Carolina and will be referred to as Aggregates A, B, and C. Table 4.5 summarizes the properties of each aggregate. It should be noted that the aggregate requirements of different DOTs may not be met by these aggregate sources and different aggregate sources will result in varied results. However, the primary objective in this particular portion of the study was to evaluate mix design procedures and not aggregate properties.

Table 4.5. Aggregate properties

Property	Aggregate A	Aggregate B	Aggregate C
Aggregate Type	Granite	Granite	Granite
Bulk Specific Gravity	2.66	2.64	2.62
SSD Specific Gravity	2.67	2.65	2.63
Apparent Specific Gravity	2.69	2.68	2.65
Absorption (%)	0.6	0.4	0.5
Micro-Deval (%)	11.3	1.3	–
LA Abrasion (%)	51	49	28

All of the aggregate sources have similar specific gravity and absorption values. The bulk specific gravities of Aggregates A, B, and C are 2.66, 2.64, and 2.62, respectively, which is typical for granite aggregates located in South Carolina. The absorption values are all below 1% which is generally recommended for asphalt mixtures.

The durability of the aggregate sources, however, has some variation. The Los Angeles Abrasion percent loss value for Aggregate C is 28%, while A and B are 51% and 49%, respectively. The LA Abrasion loss values for Aggregates A and B may be high for some areas, but is typical for aggregate located in the upstate of South Carolina. SCDOT specifications have a maximum LA Abrasion loss of 52% for OGFC mixtures (SCDOT 2007). While the LA Abrasion loss values for Aggregates A and B are similar, the micro-deval values are 11.3 and 1.3% loss, respectively.

The aggregate gradation used in the study was based on the SCDOT specifications for OGFC (SCDOT 2007). Table 4.6 shows the SCDOT specifications and research gradation used. The research gradation was achieved by creating a blend of a No. 7 stone and an 89M stone as obtained from the source of Aggregate A. This same gradation was kept for Aggregates B and C to maintain a consistent testing base to determine the optimum binder content. The amount of material passing the No. 4 sieve was also kept under 20% based on findings by Mallick et al. (2000).

Table 4.6. SCDOT OGFC gradation specifications and research gradation

Sieve Size	**Percent Passing**	
	SCDOT Specification	**Research Gradation**
¾ inch (19.0 mm)	100	100
½ inch (12.5 mm)	85-100	94
⅜ inch (9.5 mm)	55-75	63
No 4 (4.75 mm)	15-25	17
No 8 (2.36 mm)	5-10	6
No 200 (0.075 mm)	0-4	1

Two different performance grades of asphalt binders from the same crude source were used in this research. The first was a PG 64-22 (also meets the PG 67-22 requirements) as it is the common binder type used on South Carolina roadways paved with dense graded HMA and was necessary to conduct the mix design specified by the Florida DOT (2009). The second was a PG 76-22, which is two grades higher than typical binder specified by SCDOT (PG 64-22) and several other states for OGFC mixtures. The binder types used for OGFC mixtures vary by agency and are based mainly on climate and precipitation rates of the area. The properties of the binders used in this research are listed in Table 4.7. The mixing and compaction temperature ranges are also included in this table. In this study, the PG 64-22 binder was only used to determine the OBC in the FDOT procedure, but all compacted specimens were produced using PG 76-22 binder.

The additives used in this research were hydrated lime and cellulose fiber. Hydrated lime was added to the mixtures at a rate of 1% of the total aggregate weight as specified by the SCDOT. This helps to prevent stripping of asphalt mixtures. Most of the OGFC mix design procedures recommend that stabilizing fibers also be added to the mixture. Cellulose fiber was added to the mix at a rate of 0.3% of the total mix weight, which helps to strengthen the binder and reduce draindown potential which can lead to pavement deterioration and cause pore clogging.

Table 4.7. Asphalt binder properties (provided by the supplier)

Property	PG 64-22	PG 76-22
Original		
Viscosity (@ 135^{o}C), *Pa·s*	0.542	1.642
Viscosity (@ 165^{o}C), *Pa·s*	0.154	0.415
G*/sinδ (@ test temp), *kPa*	1.81 (64^{o}C)	1.44 (76^{o}C)
δ (@ test temp), ^{o}	83.8 (64^{o}C)	69.8 (76^{o}C)
RTFO aged		
Mass change, *%*	-0.304	-0.317
G*/sinδ (@ test temp), *kPa*	4.52 (64^{o}C)	2.94 (76^{o}C)
δ (@ test temp), ^{o}	78.6 (64^{o}C)	64.9 (76^{o}C)
PAV aged		
G*sinδ (@ test temp), *kPa*	2540 (25^{o}C)	1070 (31^{o}C)
δ (@ test temp), ^{o}	50.8 (25^{o}C)	53.3 (31^{o}C)
Stiffness (60s @ -12^{o}C), *MPa*	138	132
m-value (60s @ -12^{o}C)	0.367	0.366
Mixing temperature, ^{o}*F*	318 – 329	327 – 338
Compaction temperature, ^{o}*F*	297 – 307	304 – 315

Mix Design Procedures

Compacted Specimen Procedures

The mix design procedures based on compacted specimens are all similar. Specimens are mixed and compacted, then tested for particular properties. In this study, thirty specimens were compacted per aggregate source. There were six specimens compacted with 50 gyrations of the Superpave gyratory compactor at each of five different binder contents which ranged from 5.0% to 7.0% (by weight of mixture) in 0.5% increments. This covered the binder content range for most procedures; however, the Virginia DOT recommends a range of 5.75% to 7.25% (VDOT 2007). Table 4.4 provides an overall summary of the agency requirements evaluated in this research which use this type of design procedure. The number of trial binder contents is the number of different binder contents that are required to be tested to determine the OBC. The number of gyrations for compaction is also included. The voids in coarse (VCA) aggregate requirement is to have a VCA of the mixture be less than the VCA of the dry-rodded aggregate. There is no value for this, just a comparison specification. This test was not completed in this phase of the research as it is to ensure the proper gradation, not the optimum binder content. However, VCA is included in the evaluation of aggregate gradation presented in Chapter 5 of this report. The tensile strength ratio (TSR) is a moisture susceptibility test. This test was also not completed as it is a check for stripping potential, not to determine the optimum binder content. The draindown (AASHTO T305), unaged and aged abrasion (ASTM D7064) values are maximum acceptable values. The tensile strength ratio and permeability are both minimum acceptable values.

Absorption Calculation

The absorption calculation design procedures are all similar, but do have a few differences. The FHWA procedure, the Alabama DOT procedure, and the Kentucky Transportation Cabinet use the same steps and calculations (Eq. 4.1-4.3) to determine the OBC (FHWA 1990; ALDOT 1999; KTC 2008). In these procedures, 100 grams of the predominant aggregate size was soaked in oil for five minutes. It was then drained for two minutes at room temperature and an additional fifteen minutes in an oven at 140^{o}F (60°C). The masses of the dry aggregate and the oil soaked aggregate were recorded. The Arizona DOT and Wyoming DOT use the same procedure for determining the oil absorption of the coarse aggregate, but then also account for the fine aggregate absorption and use different calculations (ADOT 2010; WYDOT 2003). The Georgia DOT also uses an oil absorption test as a portion of their mix design (GDOT 2009). This is the same as the above mentioned procedures, but uses different calculations to determine the binder content (Eq. 4.1 and 4.4). All the design procedures use Society of Automotive Engineers (SAE) No. 10 Oil for the coarse aggregate, and the Arizona and Wyoming procedures require kerosene to determine the fine aggregate absorption. Only the FHWA, Alabama DOT, Georgia DOT, and Kentucky TC procedures were used in this research scope. These procedures also check the VCA to ensure that the VCA of the mix is less than the VCA of the dry-rodded aggregate. The moisture susceptibility is also evaluated and the tensile strength ratio (TSR) requirements vary from 50 to 80%.

$$Percent\ Oil\ Retained\ (POR) = \frac{G_{sa}}{2.65} \times \frac{B-A}{A} \times 100 \tag{4.1}$$

Where,

G_{sa} = aggregate apparent specific gravity
A = mass of dry aggregate
B = mass of oil soaked aggregate

$$Surface\ Constant\ (K_{c1}) = 0.1 + 0.4POR \tag{4.2}$$

$$Optimum\ Binder\ Content\ (OBC) = (4 + 2K_{c1}) \times \frac{2.65}{G_{sa}} \tag{4.3}$$

$$Optimum\ Binder\ Content\ (OBC) = 3.5 + 2K_{c2} \tag{4.4}$$

Where K_{c2} is determined from Figure 114-1 in GDT-114 (2009).

Visual Determination

The visual determination of optimum binder content design procedures all have the same general steps. An uncompacted mix sample is placed in a clear glass container and conditioned for a period of time at a specified temperature. The major differences include binder grades and time and temperature of conditioning. The Florida DOT (FM 5-588) and South Carolina DOT (SC-T-91) design procedures were used in the scope of this research. For the Florida DOT procedure, loose mixture is produced at three different binder contents using a PG 67-22 binder. Each mix is placed in a clear glass round pie plate and conditioned in an oven at 320^{o}F (160^{o}C) for one hour before inspection. In the South Carolina DOT procedure, loose mixture is prepared at three binder contents using PG 76-22 binder. Each mix is placed in a clear glass rectangular dish and conditioned in an oven at the binder mixing temperature for two hours prior to inspection. Because this is a subjective determination, the design procedures sometimes come with examples to compare to (Figures 4.1 and 4.2). The SCDOT also requires a draindown test to be completed with a minimum binder retention of 99.5%.

Mixture Testing

There were a series of specimens created for testing in this research. Uncompacted mix specimens were used to determine maximum specific gravity (1500g) and draindown (1000g) characteristics. Compacted specimens (150 mm diameter by 115 ±5 mm tall) were created with a Superpave gyratory compactor at 50 gyrations and tested for specific gravity and porosity (ASTM D7063), permeability (modified ASTM PS129), and abrasion (ASTM D7064).

In the permeability testing, there were slight changes made to the ASTM PS129 procedure. The first change was the size of the standpipe. The procedure calls for a 1.25-in. (31.8-mm) interior diameter pipe, but a 2.5-in. (63.5-mm) diameter was used to account for the high permeability values. The second change was the location of the outlet. The standard calls for this to be placed 2-in. (50-mm) above the bottom of the asphalt specimen. This permeameter was set up with the outlet location below the bottom of the specimen.

Following the volumetric and permeability measurements, the compacted specimens were tested using the Cantabro abrasion test procedure outlined in ASTM D7064 in unaged and aged conditions. The six specimens for each binder content were divided into two groups of three so that the average void content of each group was similar. One group was tested in the unaged condition and the other was aged for 7-days at 140^{o}F (60^{o}C). After 7-days, the aged group was allowed to cool at room temperature for 24 hours prior to testing. To measure the abrasion resistance of the OGFC specimens, the initial mass of a specimen was measured and then the specimen was placed in a clean LA Abrasion drum without any steel charge. The specimen was tumbled in the drum for 300 revolutions at 77^{o}F (25^{o}C). After 300 revolutions, the specimen was removed from the drum, brushed off and then weighed again. The loss due to abrasion was calculated using equation 4.5. Where w_i and w_f are the intial and final masses of the specimen, respectively.

$$Abrasion\ Loss\ (\%) = \frac{w_i - w_f}{w_i} \times 100 \tag{4.5}$$

Results and Discussion

Draindown

Uncompacted mix specimens were tested for draindown testing according to AASHTO T305. This was completed for each aggregate source at the five different research binder contents. The maximum allowable amount of draindown is 0.3% (by weight of total mixture) in most agencies that require this test. The SCDOT specifies a maximum draindown of 0.5% (or 99.5% retention) of the binder mass. The draindown was negligible for all of the binder contents tested for both aggregate sources. This is most likely due to the addition of fibers to the mix as a stabilizing additive.

Mixture Volumetrics

The mix design properties for the OGFC specimens prepared using aggregates A, B, and C are summarized in Figures 4.3, 4.4, and 4.5, respectively. The average bulk specific gravities (G_{mb}) can be seen in Figures 4.3(a), 4.4(a), and 4.5(a) and the average porosities and air voids can be seen in Figures 4.3(b), 4.4(b), and 4.5(b). The expected trend of the bulk specific gravity for a dense graded mixture is a curve with a single point maximum. This is not seen in the research data as the gradation of an OGFC mixture changes the composition of voids within the sample. The expected trend of the porosity and air voids is to decrease with increasing binder content as the voids are being filled with binder as seen in the experimental data. Additionally, the air void contents of the specimens were generally greater than the porosity values as expected because not all of the air voids in a mix are accessible by water. The porosity is a measure of the accessible air voids in a mix.

The other volumetric properties of the asphalt specimens were calculated using the results of the maximum specific gravity (G_{mm}), and specific gravity and porosity testing. Figures 4.3(c), 4.4(c), and 4.5(c) summarize the voids in mineral aggregate (VMA) and Figures 4.3(d), 4.4(d), and 4.5(d) present the voids filled with asphalt (VFA). As the binder content increases, the air voids should decrease as the binder fills the voids. The opposite trend should be seen in the VFA as it is a measure of the binder that is filling the void space. The expected trend of the VMA graph is a curve with a single minimum point. The VMA decreases with increasing binder content at first; the air voids are decreasing more than the amount of binder is increasing. After a certain point, the change in air voids becomes smaller than the increase in binder, and the VMA begins to increase again. The air voids and VFA experimental trends are as expected, but the VMA curve is not. This is due to the high air void content of the OGFC. With the binder contents used in this research, only the bottom portion of the curve is seen, which looks very much like a straight line. If a larger range of binder content was used, the expected trend would likely be seen, but over a much broader range of binder contents.

Permeability

The average permeability values are seen in Figures 4.3(e), 4.4(e), and 4.5(e). As with porosity, the expected trend is decreased permeability with increasing binder content. When the binder content increases, the asphalt binder takes the place of some of the air voids, decreasing the air content. This results in reduced interconnected voids for the water to travel through. This trend is seen in all data sets.

Cantabro Abrasion

Figures 4.3(f), 4.4(f), and 4.5(f) show the abrasion loss of the unaged and aged specimens. The expected trends, which are seen, are decreased abrasion loss with increasing binder content. This occurs as the air voids decrease, and a thicker and stronger film of binder is holding the aggregate together when the binder content increases. In addition, one would expect that the aged specimens would exhibit higher abrasion losses than unaged specimens due to binder oxidation and resulting brittleness. This was not necessarily the case as the abrasion loss was similar for the unaged and aged specimens made with Aggregate A. As for Aggregate B, the aged specimens actually had lower loss values than the unaged specimens. This is an indicator that the Cantabro abrasion test may not be a very discriminating test for evaluating the durability of OGFC mixtures.

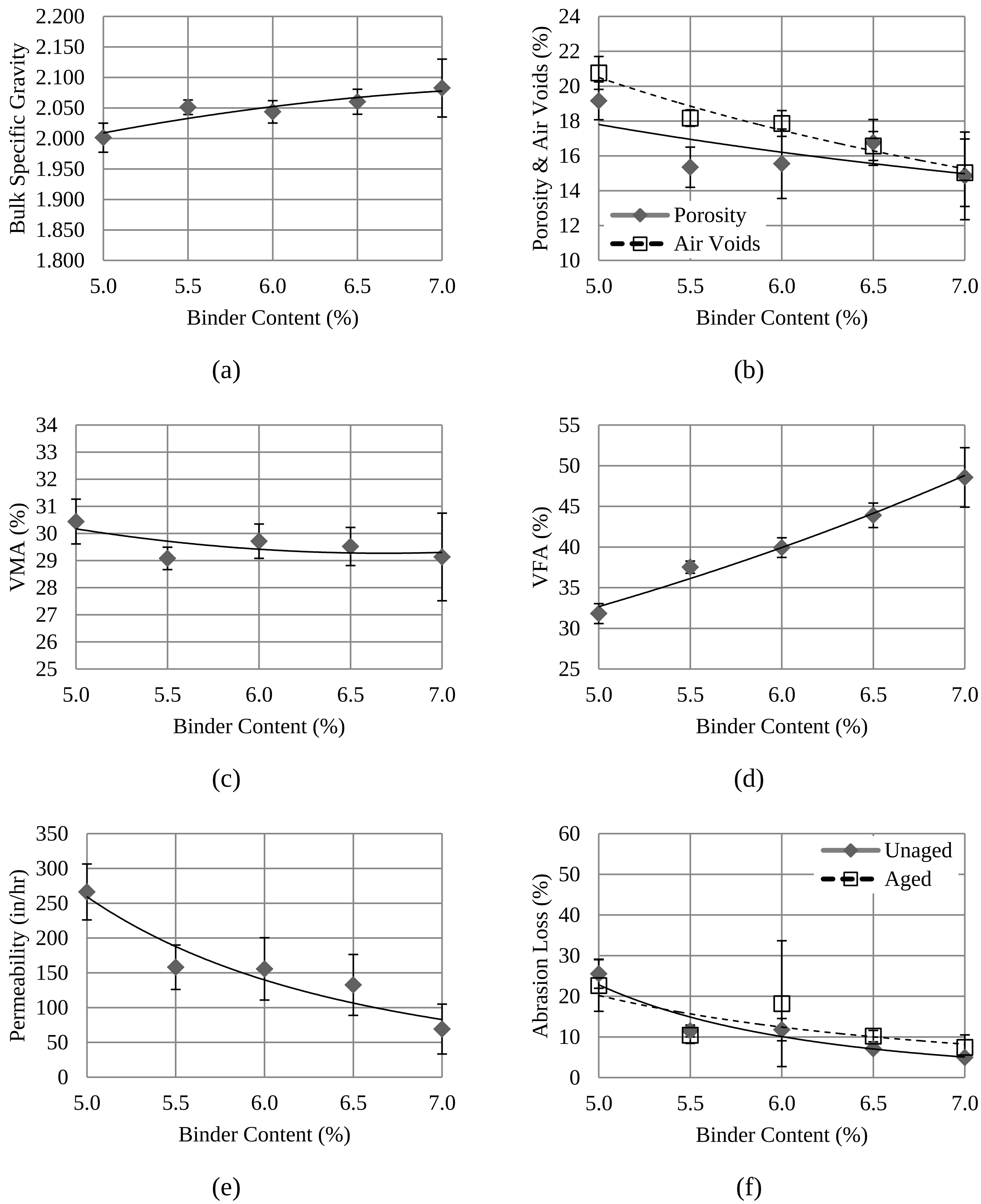


Figure 4.3. Mixture properties for specimens made with Aggregate A: (a) bulk specific gravity; (b) porosity and air voids; (c) VMA; (d) VFA; (e) permeability; (f) Cantabro Abrasion loss. Note: error bars indicate one standard deviation.

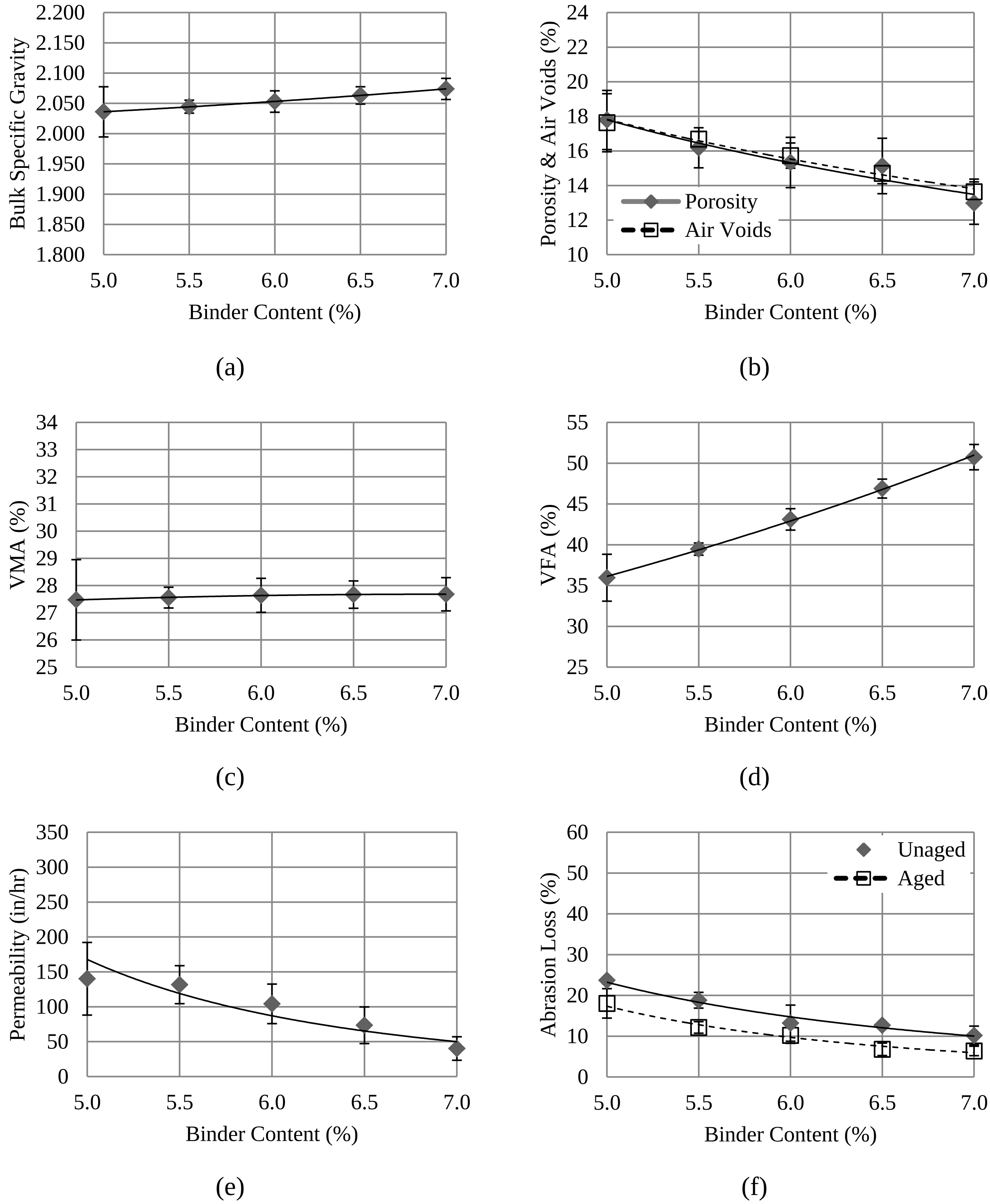


Figure 4.4. Mixture properties for specimens made with Aggregate B: (a) bulk specific gravity; (b) porosity and air voids; (c) VMA; (d) VFA; (e) permeability; (f) Cantabro Abrasion loss. Note: error bars indicate one standard deviation.

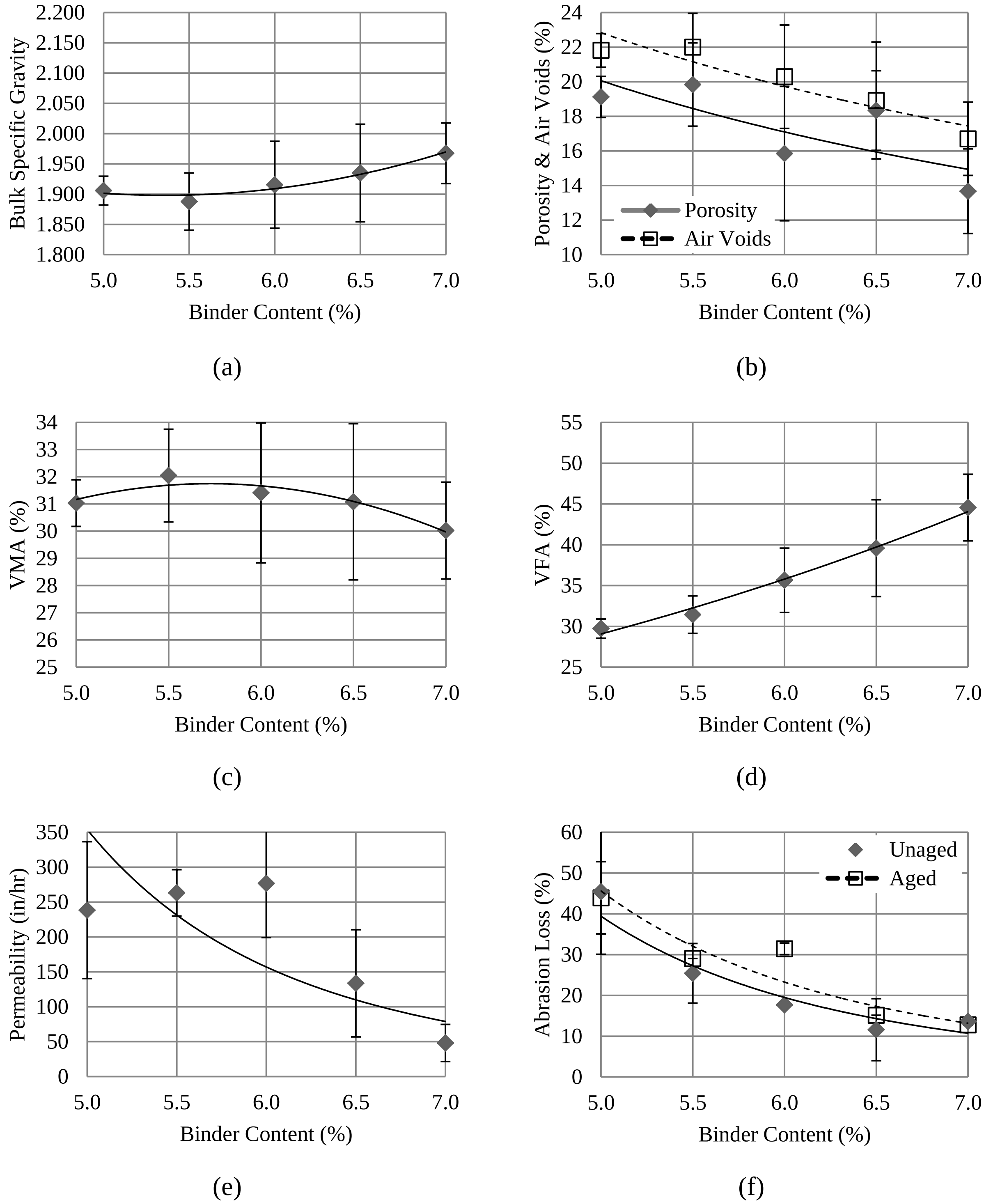


Figure 4.5. Mixture properties for specimens made with Aggregate C: (a) bulk specific gravity; (b) porosity and air voids; (c) VMA; (d) VFA; (e) permeability; (f) Cantabro Abrasion loss. Note: error bars indicate one standard deviation.

Oil Absorption

The oil absorption test is a very simple procedure in which binder is not actually required to determine the optimum binder content. The amount of oil absorbed by the predominant aggregate size is determined and then used in a series of calculations to determine the OBC. Table 4.8 shows the average oil absorbed and the corresponding OBCs.

Table 4.8. Oil absorption results

Aggregate Type	Oil Absorbed (%)	Calculated Binder Content with FHWA Procedure (%)	Calculated Binder Content with Georgia DOT Procedure (%)
A	2.06	6.0	5.4
B	2.42	6.2	5.7
C	2.26	6.0	5.5

Visual Draindown Characteristics

An additional set of uncompacted specimens was used to evaluate the visual draindown testing. This was done with both the SCDOT procedure (SC-T-91) and the FDOT procedure (FM 5-588). In these tests, the amount of binder drained to the bottom of a clear container after a period of heating a sample is visually evaluated. The optimum binder contents for all aggregate sources for the South Carolina and Florida procedures are included in Table 4.9. The expected trend is that more binder will drain as the binder content increases. Although this is not seen in every case, the general trend was seen in all sample sets.

Table 4.9. Visual draindown results

Aggregate Type	OBC Based on SCDOT Procedure (%)	OBC Based on FDOT Procedure (%)
A	6.2	5.8
B	6.0	5.8
C	6.5	6.0

Optimum Binder Content

After the compacted specimen testing was completed, the optimum binder contents for the OGFC mixture could be determined according to the various mix design requirements. Table 4.10 and Figure 4.6 shows the range of optimum binder contents which were determined for Aggregates A, B, and C. The mix designs are sorted by the categories which were summarized in Table 4.3. The average optimum binder content for Aggregate A was 5.9%, Aggregate B was 5.6%, and Aggregate C was 6.2%. Although the average values are similar, the result of each design procedure does vary greatly. The overall range of the OBCs in each aggregate source is 5.0% to 7.0% which is limited by the experimental binder content range.

Table 4.10. Optimum binder contents determined by each mix design procedure

		Optimum Binder Content (%)		
Category	**Procedure**	**Aggregate A**	**Aggregate B**	**Aggregate C**
Compacted Specimens	ASTM	5.2 – 5.7	5.0	6.0
	NAPA/NCAT	5.2 – 5.8	5.0	5.9 – 6.7
	GDOT	7.0	5.0	7.0
	NMDOT	6.5	6.5	6.5
	NCDOT	5.2 – 5.7	5.0	6.0
	MDOT	5.0 – 7.0	5.0 – 6.3	5.4 – 7.0
	MODOT	*	*	6.0 – 6.7
	NDOR	5.8 – 6.2	*	6.3 – 6.8
	TDOT	5.2 – 7.0	5.0 – 7.0	6.0 – 7.0
	VDOT	5.2 – 6.6	5.0 – 5.8	6.0 – 7.0
Oil Absorption	FHWA	5.8	6.1	6.0
	GDOT	5.4	5.7	5.5
Visual Determination	SCDOT	6.2	6.0	6.5
	FDOT	5.8	5.8	6.0

** Did not meet the criteria in the range of binder contents evaluated.*

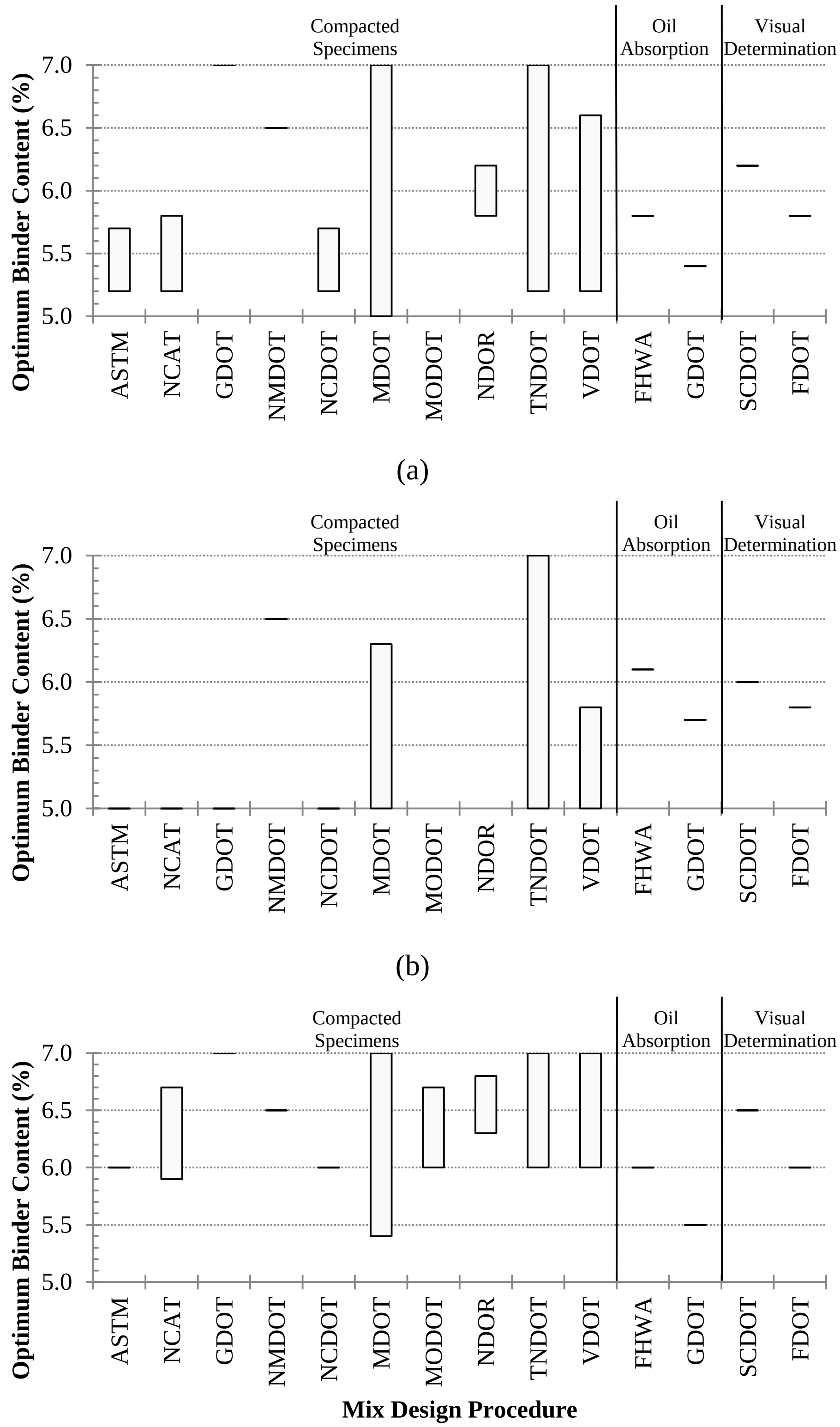


Figure 4.6. Optimum binder content results for (a) Aggregate A, (b) Aggregate B, and (c) Aggregate C

Conclusions

The objective of this portion of the research was to evaluate and compare the different mix design procedures currently in use for OGFC mixtures in the United States and to provide any guidance related to the advantages and disadvantages of different types of design procedures. A survey of OGFC mix design procedures used in the US revealed that there are many different mix design recommendations, but all of the procedures can be grouped into three categories: (1) those that base the optimum binder content on the properties of compacted specimens; (2) those that calculate the optimum binder content based on the absorption capacity of the aggregate; and (3) those that determine the optimum binder content based on visual inspection of loose mix.

The optimum binder contents determined from the fourteen procedures evaluated showed variability between the different procedures for the same aggregate source. The largest variability came within the Compacted Specimens mix design category. For many of the procedures in the Compacted Specimens category, the outcome was a range for the optimum binder content. In some cases this range was as wide as the range of binder contents evaluated in the procedure (5-7%). This does not provide any guidance to the designer as to what the OBC should be for a particular mixture. Additionally, in some cases the OBC was determined to be the lowest binder content evaluated in the range, which is likely too low to ensure long-term durability in the field as evidenced by the relatively high abrasion loss values for mixtures containing lower binder contents. A contributing factor to these mix design methods resulting in a range of acceptable binder contents could be the variability in some of the test methods, specifically, the Cantabro test.

An advantage of these types of methods is that they do evaluate the performance properties (permeability and durability) of the mixtures at the different binder contents, which can provide valuable information to the informed designer.

The procedures in the other two categories (Oil Absorption and Visual Determination) resulted in a single value for optimum binder content for each procedure, which is an advantage depending on the experience of the designer. The OBC values from the Oil Absorption procedures showed higher variability than the Visual Determination values, but not nearly the variability as the Compacted Specimen procedures. The Visual Determination procedures, although subjective in nature, provided the most consistency in OBC between procedures. However, these procedures do not require the designer to evaluate the mixture performance properties at the determined OBC, which presents a downside to these types of design methods. However, when tested at the OBCs, these mixtures demonstrated adequate permeability required for many porous pavement applications as well as abrasion resistance necessary for long-term durability.

Recommendations

Based on the results of this study, it is recommended that the Visual Determination procedure outlined in SC-T-91 continue to be used as long as designers have proper training to determine the OBC of OGFC mixtures. This type of procedure is more repeatable than methods centered around the use of test procedures that have a relatively high level of variability such as the Cantabro test used in the Compacted Specimens category. It should be noted, however, that the designer does need to understand what they are looking for due to the subjectivity of these procedures. This guidance, in the form of visual aids, is typically provided in the specific procedures outlined by DOTs that specify this type of mix design method.

The mixture should also be evaluated for draindown at the OBC to ensure that the mix meets the draindown requirements (99.5% retention when using SC-T-90). It is also recommended to investigate the selection of OBC based on a draindown test (SC-T-90 or AASHTO T305). This test can remove some of the subjectivity from the selection of the binder content. However, when fibers are used, the binder content to reach 0.5% draindown by binder weight or 0.3% by mix weight could be quite high.

In addition to determining the OBC, it is recommended that the mixture properties are verified at the OBC. This should involve an evaluation of the susceptibility of the mixture to moisture induced damage and permeability. There are several methods to evaluate the moisture sensitivity of a mixture including boil tests (SC-T-69) and tensile strength ratio (TSR) of compacted specimens (similar to AASHTO T283 or SC-T-70). It is recommended to evaluate these (or other) performance tests to establish test procedures and minimum criteria.

While it would be advantageous to evaluate the raveling susceptibility of OGFC mixtures at the OBC, studies need to be completed to identify the best methods to evaluate raveling of these types of mixtures. Currently, the Cantabro Abrasion test is used by some for this purpose, but it does not always give sufficient guidance about durability as shown in this study and others (Alvarez et al. 2010).

CHAPTER 5: INFLUENCE OF AGGREGATE GRADATION ON OGFC MIXTURES

There has been a significant amount of research related to OGFC mixtures conducted in the US over the past 50 years with the goal to improve its performance (Huber 2000). Some of the notable improvements that have been made were related to gradation and type of binder used. Studies continue on this topic to enhance the performance of these mixtures and develop a better, more durable mix. A well-designed, well-constructed OGFC should not have raveling problems and should retain adequate permeability during its life.

The aggregate gradation of OGFC mixtures is critical to producing a mixture that will have the necessary structural and functional performance required for a specific application. The aggregate gradation controls the porosity of the aggregate structure, which in turn controls the permeability of the mixture (Cabrera and Hamzah 1996; Suresa et al. 2010). The aggregate gradation should contain a large percentage of coarse aggregate, but the selection of the amount of fine aggregate is important as the fine aggregate content must be low enough to prevent the void structure from closing up (Ruiz et al. 1990).

In addition to permeability, aggregate gradation also affects the ability of an OGFC mixture to withstand traffic loading. Due to the high void content of the mixtures, they are more susceptible to deformation than dense asphalt mixtures. The source of stability of any asphalt mixture is aggregate interlock which is enhanced by the coarse aggregate (Cabrera and Hamzah 1996). When too much fine aggregate is included in an OGFC mixture, the finer particles will separate the coarse particles and increase the rutting potential of the mixture (Ruiz et al. 1990). Recommendations have been made for selecting aggregate gradations based on stone-on-stone contact as determined by comparing the voids in coarse aggregate (VCA) of the mixture to the VCA of the aggregate itself (Watson et al. 2004b; Alvarez et al. 2010b). However, at times, these recommendations have been inconsistent at predicting the rutting resistance of OGFC mixtures as mixtures meeting the criteria have been found to exhibit high rutting susceptibility and those that failed to meet the criteria showed high resistance to rutting (Mallick et al. 2000; Suresa et al. 2010).

As part of NCHRP Project 09-41, a survey was sent to agencies in different parts of the world to determine which agencies use porous asphalt for OGFCs (Cooley et al. 2009). Sixty four percent of the survey recipients responded, including 32 US states along with Canada, Austria, England, Columbia, and Japan. Of the 32 US states that responded to the survey, some states only use one gradation, while others use two gradations. Two states reported that they have specifications for three different gradations (Table 5.1) (Cooley et al. 2009).

Table 5.1. Summary of OGFC gradation specifications for 20 US DOTs (Cooley et al. 2009)

	Percent Passing								
State	**1 in.** *25.0mm*	**3/4 in.** *19.0mm*	**1/2 in.** *12.5mm*	**3/8 in.** *9.5mm*	**No. 4** *4.75mm*	**No. 8** *2.36mm*	**No. 16** *1.18mm*	**No. 30** *0.60mm*	**No. 200** *0.075mm*
AL		100	85-100	55-65	10-25	5-10			2-4
CT			95-100		20-35	5-19			1-5
DE			100	88-98	25-42	5-15			2-5
FL		100	85-100	55-75	15-25	5-10			2-4
IN			100	83	28	13			2-4
KY			100	90-100	25-50	5-15			2-5
MO		100	85-100	55-75	10-25	5-10			2-4
MS			100	80-100	15-30	10-20			2-5
NE		100	95-100	40-80	15-35	5-12			0-3
NY			95-100	40-56	20-30	6-14	4-12	3-9	2-5
TN		100	85-100	35-60	10-25	5-10			2-4
OH			100	85-96	28-45	9-17			2-5
SC		100	85-100	55-75	15-25	5-10			2-4
LA 1			100	90-100	25-50	5-15			2-5
LA 2		100	85-100	55-75	10-25	5-10			2-4
NV 1			100	90-100	35-55		5-18		0-4
NV 2				95-100	40-60		12-22		0-5
CA 1				78-89	28-37	7-18			
CA 2					29-36	7-18			
OR 1	99-100	85-96	55-71		10-24	6-16			1-6
OR 2		99-100	90-98		18-32	3-15			1-5
NC 1			100	75-100	25-45	5-15			1-3
NC 2			100	75-100	25-45	5-15			1-3
NC 3		100	85-100	55-75	15-25	5-10			2-4
GA 1		100	100	85-100	20-40	5-10			2-4
GA 2		100	85-100	55-75	15-25	5-10			2-4
GA 3		100	80-100	35-60	10-25	5-10			1-4
TX 1		100	80-100	35-60	1-20	1-10			1-4
TX 2		100	95-100	50-80	0-8	0-4			0-4

The main objective of this portion of the study was to evaluate the effects of aggregate gradation on the properties of OGFC mixtures. To accomplish this objective, the properties (porosity, permeability, abrasion resistance, rutting resistance, and indirect tensile strength) of OGFC mixtures made with ten different gradations were compared.

Experimental Materials and Methods

To accomplish the objectives of this study, ten different OGFC mixtures—each having a different aggregate gradation—were evaluated. The mixtures were produced using one SBS modified PG 76-22 asphalt binder source having the properties summarized in Table 5.2. One granite aggregate source (Aggregate C from Chapter 4) was also used to prepare the mixtures in this study. Additionally, each mixture included hydrated lime as an anti-stripping additive added at a rate of 1.0% by weight of aggregate. Finally, cellulose fiber was used as a stabilizing additive as required by SCDOT specifications to minimize binder draindown. The fibers were added at a rate of 0.3% by total mixture weight.

Table 5.2. Summary of asphalt binder properties provided by supplier

Property	Value
Original	
Viscosity (@ 135^{o}C), *Pa·s*	1.642
Viscosity (@ 165^{o}C), *Pa·s*	0.415
G*/sinδ (@ 76^{o}C), *kPa*	1.44
δ (@ 76^{o}C), ^{o}	69.8
RTFO aged	
Mass change, *%*	-0.317
G*/sinδ (@ 76^{o}C), *kPa*	2.94
δ (@ 76^{o}C), ^{o}	64.9
PAV aged	
G*sinδ (@ 31^{o}C), *kPa*	1070
δ (@ 31^{o}C), ^{o}	53.3
Stiffness (60s @ -12^{o}C), *MPa*	132
m-value (60s @ -12^{o}C)	0.366
Mixing temperature, ^oF	327 – 338
Compaction temperature, ^oF	304 – 315

To select the research gradations, the survey results from NCHRP Report 640 were used to identify 29 different OGFC gradations included in state DOT specifications (Table 5.1) (Cooley et al. 2009). These gradations were combined into groups of similar gradations using a Cluster Analysis Program (Average Linkage Process) (Figure 5.1) to group samples based on the average distance between the mid-point percent passing of each sieve for each gradation (SAS Institute 2011). An average distance of 0.41 between clusters was chosen to group the 29 gradations into ten gradation groups (Figure 5.2). All states under the largest bracket that falls under 0.41 were grouped together. For example, AL, FL, GA2, NC3, SC, LA2, MO, GA3, TX1, and TN fall under one group.

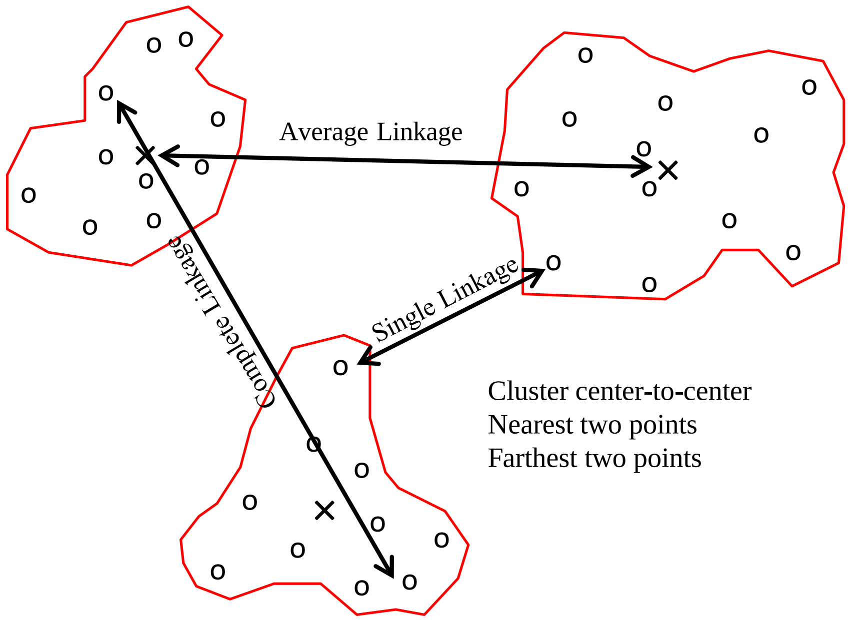


Figure 5.1. Three methods of inter-cluster distances

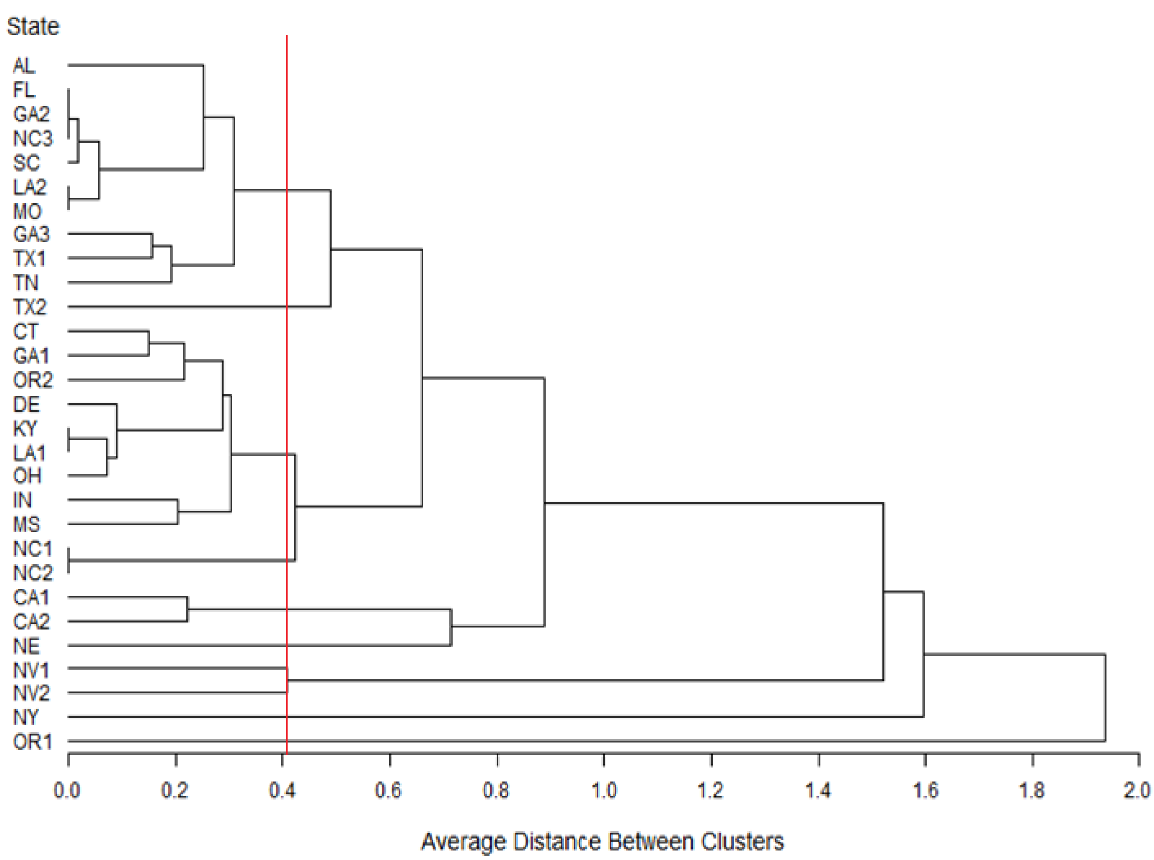


Figure 5.2. Average linkage cluster analysis of states OGFC gradations

Once the ten gradation groups were formed, each group's gradation was then calculated as the average percent passing (upper limits and lower limits) for each sieve size for all states within each group. Table 5.3 shows the ten final gradations used for the study and Figure 5.3 includes the gradation curves for each mix. The current SCDOT gradation for OGFC is included in mix A. Table 5.3 also includes the dry-rodded unit weight (DRUW) and void ratio of each gradation measured per AASHTO T19.

Table 5.3. Compiled research gradations

	Percent Passing (%)									
Sieve Size	**Mix A**	**Mix B**	**Mix C**	**Mix D**	**Mix E**	**Mix F**	**Mix G**	**Mix H**	**Mix I**	**Mix J**
¾ in.	100	100	100	100	100	100	100	100	100	90.5
½ in.	93.0	91.0	97.5	99.0	100	100	97.5	100	97.5	63.0
⅜ in.	66.0	47.5	65.0	95.0	90.0	87.5	60	96.0	48.0	45.0
No. 4	21.0	15.0	8.0	33.0	22.5	35.0	25	48.5	25.0	17.0
No. 8	8.0	7.0	4.0	10.0	15.0	10.0	8.5	25.0	10.0	11.0
No. 16	-	-	-	-	-	-	-	14.0	8.0	-
No. 30	-	-	-	-	-	-	-	-	6.0	-
No. 200	3.0	3.0	4.0	3.5	3.5	2.0	1.5	2.3	3.5	3.5
DRUW *(lb/ft^3)*	101.1	103.0	98.6	102.3	104.2	101.1	103.6	106.1	104.2	109.8
Void Ratio *(%)*	38.8	37.4	40.2	38.0	36.7	38.6	37.2	35.7	36.9	33.4

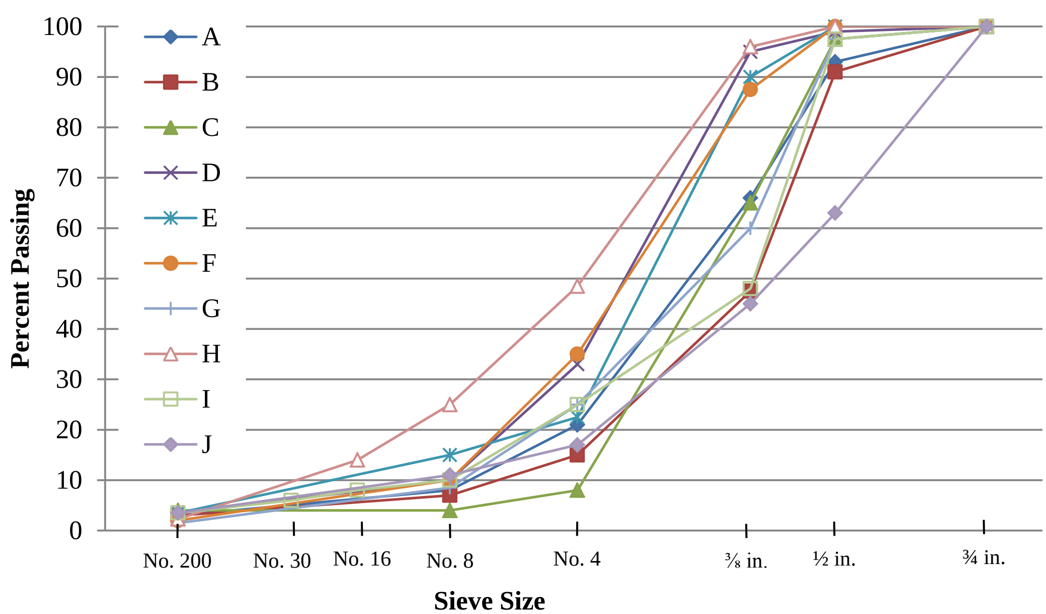


Figure 5.3. Gradation curves for each aggregate gradation curve evaluated

Optimum Binder Content (OBC) and Draindown Determination

Aggregate-asphalt binder blends were prepared with binder contents ranging from 5% to 7% (by weight of total mixture) at 0.5% increments. The SC-T-91 procedure was used for the optimum binder content determination in which the uncompacted mixtures were placed in clear Pyrex dishes. The mixtures were then placed in an oven at the mixing temperature (332^{o}F) for 2 hours. The OBC was subjectively determined by visual inspection of the mixtures using guidance from the published procedure (SCDOT 2010).

AASHTO T305 was used to measure the asphalt draindown at each binder content used in the OBC determination (AASHTO 2010). The only exception to the procedure was that the draindown was only measured at the mixing temperature (332^{o}F). The test is intended to simulate conditions that the mixture is likely to encounter as it is produced, stored, transported, and placed and to ensure that a mixture does not have excessive binder content that would result in binder draindown. A maximum draindown limit of 0.3% by weight of total mix is typically the limiting value for determining acceptable performance (Jackson 2003). This procedure is slightly different from the SC-T-90 procedure used by the SCDOT. In SC-T-90, the draindown is determined after conditioning the specimen in the oven at 350^{o}F for one hour, instead of the mixing temperature. Additionally, the draindown is calculated as a percentage of the binder content instead of the total mixture. Because of the different draindown calculation, the SCDOT draindown limit is 0.5% (by binder weight).

Mixture Performance Tests

A total of 16 specimens were made for each mix at the respective optimum binder content. Cylindrical specimens having a diameter of 150 mm and height of 115 ± 5 mm were compacted using 50 gyrations of a Superpave gyratory compactor. Each compacted specimen was tested for porosity and bulk specific gravity using the CoreLok vacuum-sealing method outlined in ASTM D7063. These results, coupled with the theoretical maximum specific gravity (AASHTO T209) were used to calculate the volumetric properties (porosity, air voids, voids in mineral aggregate, and voids filled with asphalt) of each specimen. Further testing on the compacted specimens included permeability, abrasion resistance, moisture susceptibility, and rutting resistance.

After the volumetric properties were determined, the specimens from a particular mix were divided into six different groups (one group of specimens per test) in such a manner that each group's average porosity was as close as possible to the overall mean porosity of all of the specimens from a particular gradation. Grouping the specimens together in this manner reduced the chance of bias for each test. An ANOVA test was then conducted to ensure that there was no significant difference between the porosity of the different groups (α = 0.05). The group with the highest porosity range was used for permeability testing to investigate how changes in porosity can affect permeability.

The permeability of the mixtures was evaluated using a custom falling-head permeameter constructed for this study illustrated in Figure 5.4. It should be noted that this procedure is different than that used for the testing described in Chapter 4. Each specimen was wrapped securely with a thin plastic film to seal the sides of the specimen and only allow water to exit through the bottom surface when the specimen was fit snuggly into the standpipe. Petroleum jelly was applied onto the plastic wrapped specimen for lubrication before the specimen was inserted into the standpipe. A moldable sealant (plumbers putty) was then applied around the top of each specimen to ensure that no water would drain around the outer edges of the specimen. Once the apparatus was secured and the permeameter was filled with water to a height of 15-in. above the top of the specimen, the permeameter was leveled. The valve was opened and the time (t) required for the water to drop from the initial head (h_1) of 12-in. above the specimen to the final head (h_2) of 3-in. above the specimen was recorded and used to calculate the permeability using Equation 5.1. The permeability (k) was measured three times on each of the 30 specimens (3 specimens for each gradation). In Equation 5.1, a is the cross-sectional area of the standpipe, L is the thickness of the specimen, and A is the cross-sectional area of the specimen.

$$k = \frac{aL}{At} ln \frac{h_1}{h_2} \tag{5.1}$$

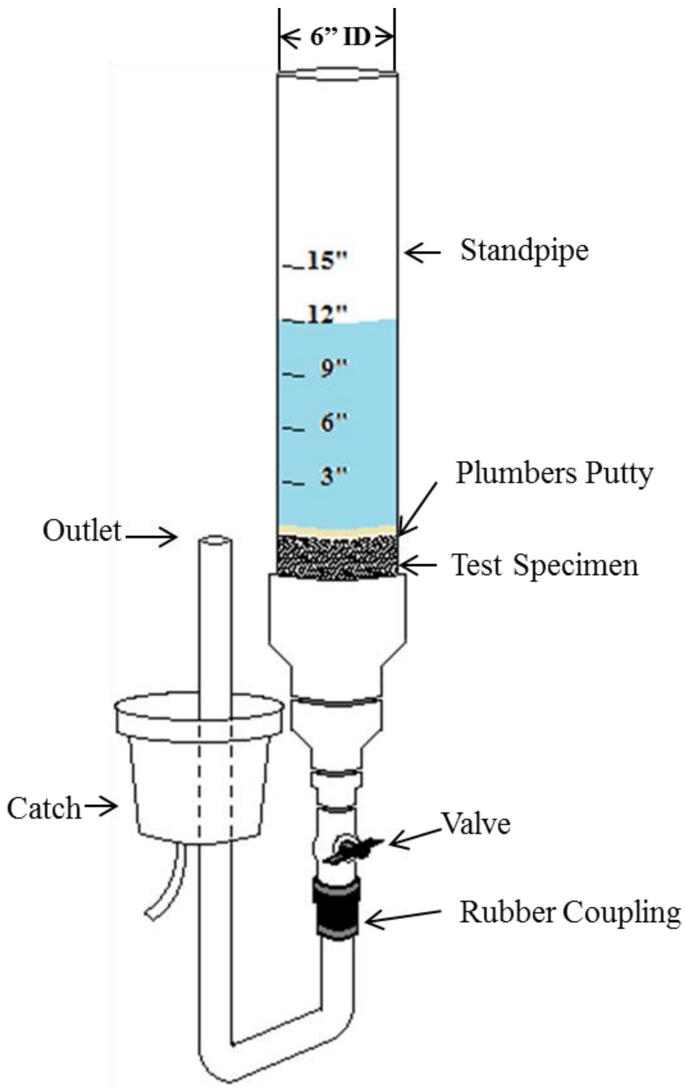


Figure 5.4. Schematic of permeameter apparatus

Following the permeability measurements, the same specimens were tested to evaluate their clogging potential. To accomplish this, a method similar to ASTM E895 *Standard Test Method for Measuring Pavement Macrotexture Depth Using a Volumtric Technique* was employed. ASTM C778 Graded Sand was used as the clogging material and was spread on the top surface of each specimen. The sand was used as received. The mass of the sand required to completely fill the surface voids was recorded, then the volume of sand in the surface voids was calculated based on the unit weight of the sand. Finally, the average macrotexture depth was calculated by dividing the volume of sand by the cross-sectional area of the specimen. Once the macrotexture depth was calculated, the permeability of the clogged specimen was measured and the reduction in permeability was calculated.

The Cantabro method outlined in ASTM D7064 was used to evaluate mix loss of the porous asphalt mixtures due to abrasion. Specimens were placed in a Los Angeles abrasion drum one at a time without the steel charges and the test was run for 300 revolutions. The Cantabro loss, expressed as a percentage, corresponds to the ratio of mass lost during the test to the initial mass of the compacted specimen and has been considered as an index of a mixture's resistance to raveling (Jimenez and Perez 1990). While this test has been shown to have a high degree of variability, it is still commonly used to assess the raveling potential of porous asphalt mixtures (Alvarez et al. 2010a).

A total of 60 specimens were used for this test (3 unaged and 3 aged specimens per mix). To age the specimens, they were placed in a fan forced temperature controlled chamber at 140^{o}F (60^{o}C) for 7 days. Prior to testing, the conditioned specimens were allowed to cool to 77^{o}F (25^{o}C) for approximately 24 hours.

To assess the moisture susceptibility of the OGFC mixtures, the indirect tensile strength (ITS) and tensile strength ratio (TSR) were calculated. The conditioning of specimens prior to determining TSR varies as some agencies recommend the use of five freeze-thaw cycles, while others only require one, and yet others (including SCDOT) only use wet conditioning without a freeze-thaw cycle. Based on research findings from Mallick et al. (2000) and Watson et al. (2004a), a modified version of the AASHTO T 283 procedure was used to determine moisture susceptibility of the OGFC mixtures. The primary changes in the procedure were to use a specific time period of 30 minutes for vacuum saturation under water and one freeze-thaw cycle for the conditioning process. Additionally, the void content was not controlled because some of the gradations studied will produce void contents that will vary greatly from other gradations as seen in Table 5.3. Two specimens were used to test for dry ITS and two more specimens were conditioned for one freeze-thaw cycle and tested to determine the conditioned ITS for each mix. While the procedure outlined in SC-T-70 does not require a freeze-thaw cycle, one was used to evaluate a worst-case scenario for the OGFC mixtures.

The rutting resistance of each mixture was evaluated using the AASHTO T340 procedure with the Asphalt Pavement Analyzer (APA). A total of 30 specimens (3 specimens for each gradation) were evaluated with this test. Compaction to height was neglected to maintain a close range of air voids with the specimens used for the other tests. Also, there are no air void requirements for the porous asphalt mixtures and the void content will change depending on the gradation of the mix. The specimens were then cut to the required height of 75 ± 2 mm. Testing was conducted at a test temperature of 147^{o}F (64^{o}C) using a hose pressure of 100 psi and a vertical wheel load of 100 lbs. Prior to testing, the specimens were conditioned in the APA chamber at 147^{o}F for 4 hours.

Statistical Analysis

Fisher's test for least significant difference (LSD) was used to determine the statistical differences between the studied gradations. The results of this statistical test are expressed as letters in the corresponding tables. The letters obtained from the output of the analysis indicate similarities between the mixtures. Mixtures which have the same letter are not significantly different than each other. Some mixtures can have more than one letter indicating similarities with more than one group.

An analysis of variance (ANOVA) was conducted for the Cantabro abrasion and moisture susceptibility to determine if the results before conditioning were significantly different from the results after conditioning. These results are also included in the respective tables. All of the analyses in this research (LSD and ANOVA) were conducted with a 95% level of significance (α= 0.05).

Results and Discussion

Optimum Binder Content

To determine the OBC, uncompacted samples were placed in a clear glass container and conditioned for a period of time at a specified temperature. Even though the highest tested binder content was 7.0%, 9 out of the 10 mixtures had an OBC greater than 7.0% because a high performance grade binder was used in this research and cellulose fibers were added to all of the mixtures. The OBC and binder film thickness for each gradation is summarized in Table 5.4.

Table 5.4. Mixture optimum binder content and average volumetric properties with LSD test results

Mix	OBC	Film Thickness	Porosity		Air Voids		Permeability	
	(%)	*(μm)*	*(%)*	*LSD*	*(%)*	*LSD*	*(in/hr)*	*LSD*
A	7.2	26.5	15.1	b	17.7	bc	288	bc
B	7.3	29.5	13.4	cd	15.6	d	300	b
C	6.7	22.1	20.0	a	22.1	a	681	a
D	7.2	24.9	14.7	bc	18.9	b	230	bcd
E	7.3	24.0	12.8	d	16.4	cd	140	de
F	7.3	35.4	14.7	bc	17.7	bc	219	bcd
G	7.2	42.9	14.1	bcd	17.5	c	257	bcd
H	7.5	26.8	8.4	f	10.0	f	42	e
I	7.0	24.5	11.2	e	13.2	e	183	d
J	7.0	23.4	10.6	e	12.2	e	199	cd

Binder Draindown

The purpose of the draindown test was to simulate conditions that the mixture is likely to encounter as it is produced, stored, transported and placed. The maximum allowable amount of draindown is 0.3% set by most agencies that require this test (0.5% based on SC-T-90 by SCDOT). All ten gradations used in this research had negligible draindown with the highest draindown being only 0.02%. This negligible draindown is common when using fiber stabilizing additives and polymer modified binders. That is another reason why the expected trend of increasing draindown with increasing binder contents was not noticed during this test.

Air Voids & Effective Porosity

The determination of the air voids in the OGFC specimens was determined based on the ratio of bulk to maximum specific gravity, while the CoreLok vacuum-sealing method was used to determine the porosity of the specimens. The difference between the two lies in the fact that the effective porosity is the percentage of air voids that can be accessed by water through interconnected pores to saturate a compacted specimen (ASTM D 7063). The average porosity and air void contents for each mixture are summarized in Table 5.4 and Figure 5.5 and the relationship between the effective porosity and air voids is illustrated in Figure 5.6. The expected trend of effective porosity increasing as air voids increased can be noticed in the trend. However, the porosity was lower than the air void content, which was expected because the porosity value does not include the air voids that are not accessible by water. This has also been seen in other research (Kline and Putman 2011; Alvarez et al. 2008).

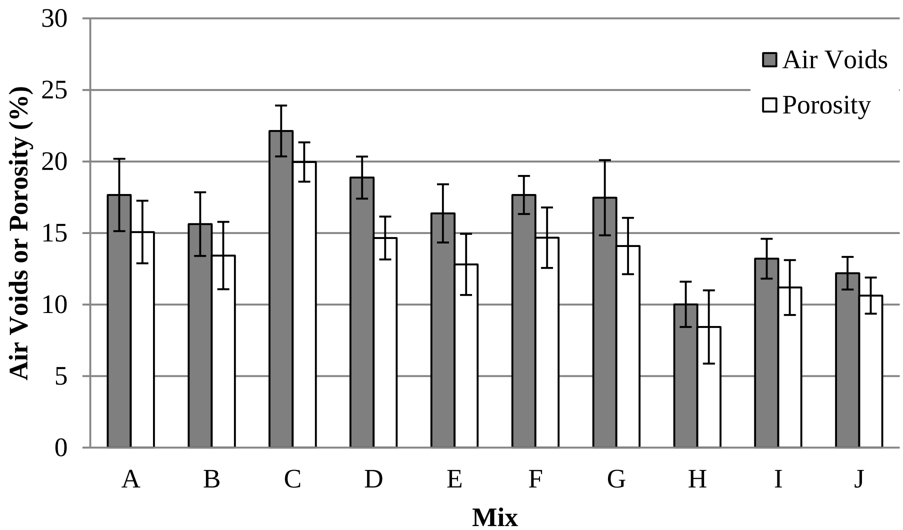


Figure 5.5. Air void content and porosity of OGFC mixtures

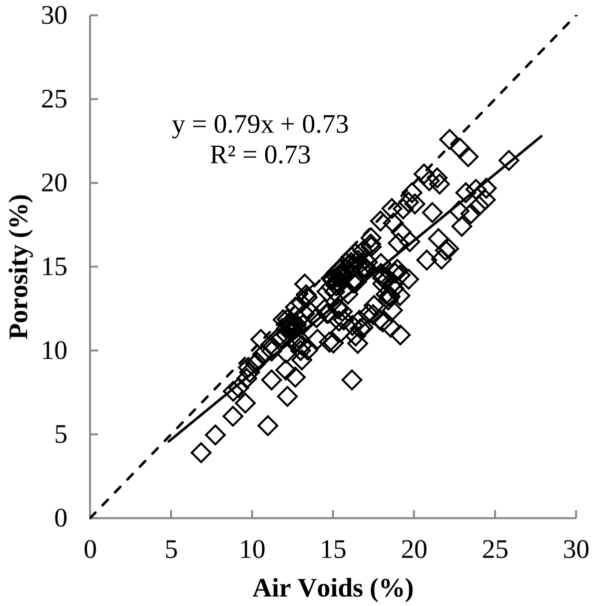


Figure 5.6. Relationship of porosity and air voids

Figure 5.7 shows the correlation between the aggregate void ratio and average porosity/air voids for the different gradations. Both correlations were fairly good with R^2 values of 0.70 and 0.71. This indicates that the correlations potentially allow for a prediction of porosity and air void content based on the void ratio of the aggregate gradation used to produce an OGFC mix. Because the same aggregate source was used in this research, it can be concluded that the gradation was the only factor affecting the aggregate void ratio. A denser gradation (lower void ratio) indicates that the aggregate particles were more tightly packed than other gradations because it leaves less space for air voids to form in the mix.

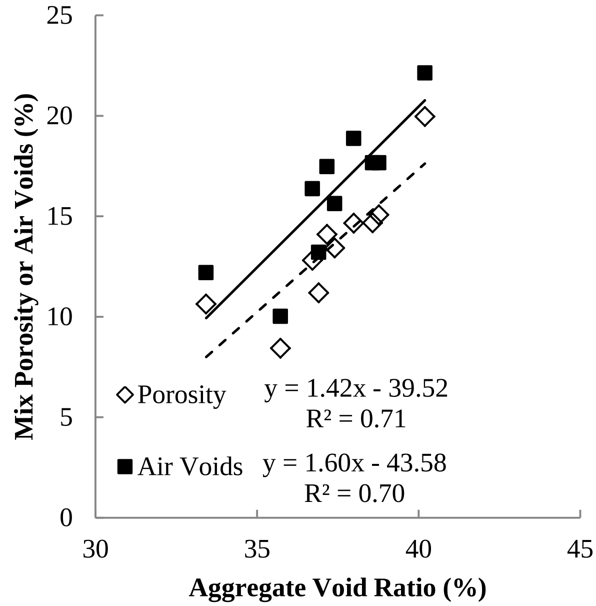


Figure 5.7. Correlation between aggregate voids and porosity and air voids

A summary of the LSD test that was conducted on the porosity and air voids results to identify statistical differences between the different mixtures is also included in Table 5.4. The results indicate that mix C had a significantly higher porosity and air void content than any other mixture and mix H had the lowest porosity and void content.

Permeability

The rate at which water flows through a material is referred to as the permeability or hydraulic conductivity. The falling head permeameter was used to measure the permeability of the OGFC specimens. Because the main purpose of OGFC mixtures is to allow water to drain through and away from the pavement, a proper rate at which water flows through the pores of the structure is essential. According to Mallick et al., a minimum permeability rate of 164 in/hr is recommended to provide acceptable performance (2000). The permeability rates of the ten gradations are included in Table 5.4 and Figure 5.8. Figure 5.8 also includes permeability data after clogging, which is discussed in the next section. Mixes E and H were the only mixes that failed to meet the permeability rate of 164 in/hr. The low permeability result of mix H was expected as it had the lowest porosity. The fine gradation of mix H resulted in more closely packed aggregate particles than coarser gradations, which diminishes the air void content in the mix resulting in reduced permeability. As expected, the gradation with the highest porosity, mix C, resulted in the highest permeability.

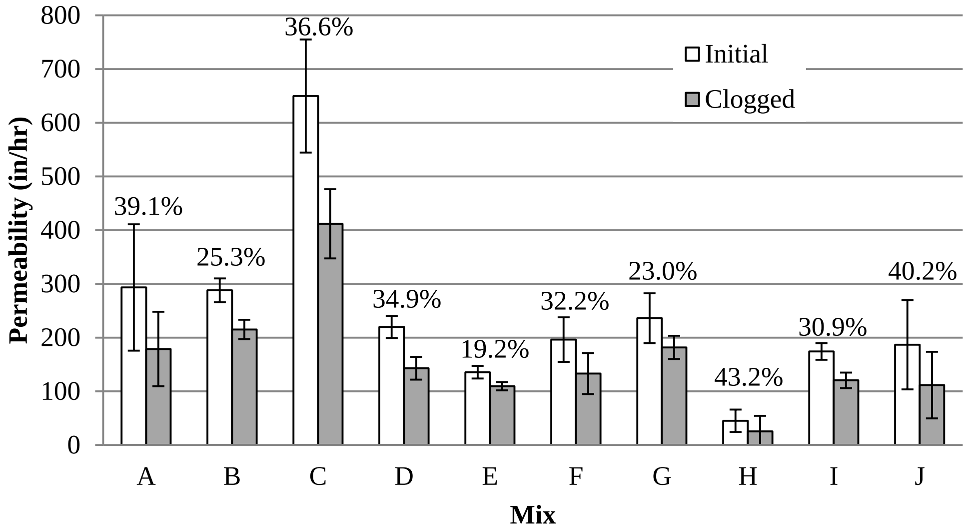


Figure 5.8. Comparison of initial and clogged permeability (values above each pair of bars indicates the permeability reduction due to clogging)

Figure 5.9 shows that a correlation exists between permeability and porosity/air voids. As expected, permeability had a stronger correlation with porosity than it did with air voids because porosity is defined as the interconnected voids that are accessible by water from outside. Based on the relationships between permeability and porosity (Figure 5.9) and between mix porosity and aggregate void ratio (Figure 5.7), it is potentially possible to predict the permeability of an aggregate mixture based on the void ratio of the aggregate gradation using Equation 5.2. However, more research is needed to validate this finding.

$$k = 3.64(1.42V - 39.52)^{1.57} \tag{5.2}$$

Where,

k = permeability (in/hr)
V = void ratio of aggregate per AASHTO T19 (%)

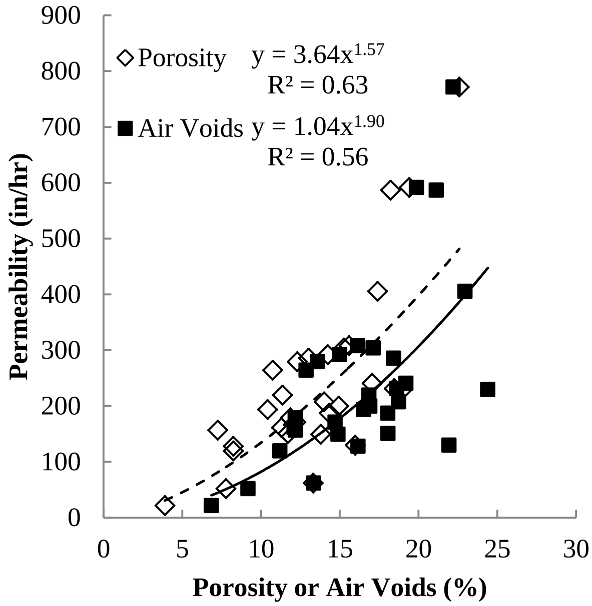


Figure 5.9. Correlation of permeability to porosity and air voids

Macrotexture Depth

The macrotexture depth of each mixture was determined using a sand patch method similar to that described in ASTM E965 and the results are summarized in Figure 5.10. Additionally, Figure 5.11 indicates that there is a strong relationship between the macrotexture depth as measured by the sand patch method and the percent of the aggregate material passing the No. 4 (4.75 mm) sieve. Because frictional resistance is a function of macrotexture, the design of the aggregate gradation can influence the frictional performance of the pavement. However, this must be validated with direct measures of the frictional characteristics of the mixtures.

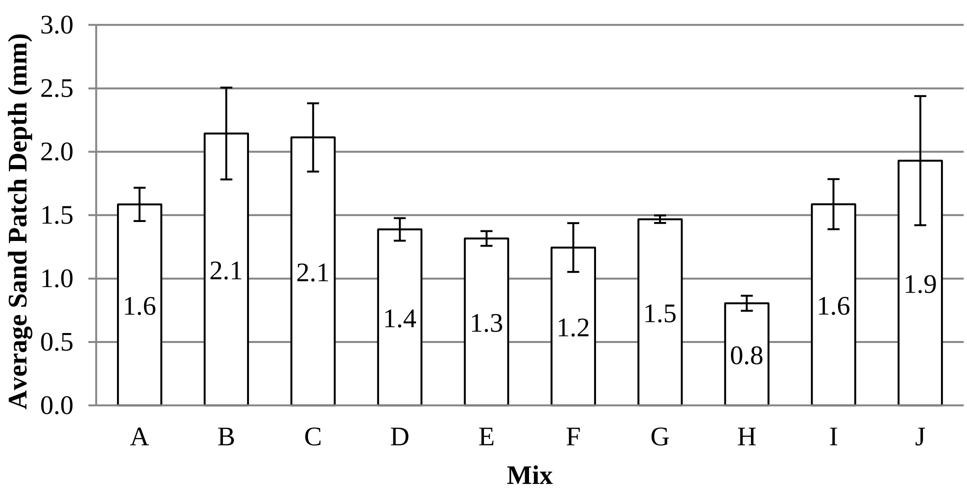


Figure 5.10. Macrotexture of OGFC mixtures measured using a sand patch test

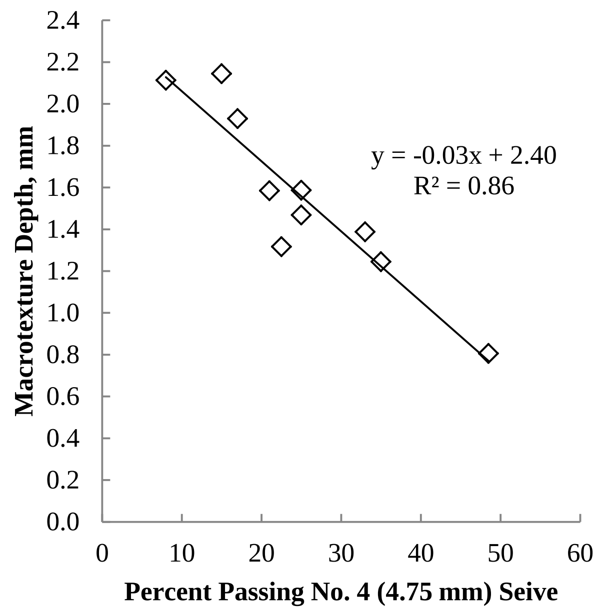


Figure 5.11. Relationship of macrotexture depth with percent passing the No. 4 seive

Clogging Potential

The clogging potential of each mixture was evaluated by comparing the initial permeability previously discussed with the permeability of each specimen after clogging with sand from the sand patch test. Figure 5.8 summarizes the results of the initial and clogged permeability of each mixture. While the permeability of each mixture decreased after sand was applied, it was seen that the reduction in permeability (noted above each pair of bars in Figure 5.8) was somewhat similar for each mixture. It should be noted that the clogging media in this study was sand and the results will likely differ for finer grained clogging material having lower permeability. This does, however, provide a relative comparison of the potential for each mixture gradation to become clogged.

Abrasion Resistance

The Cantabro abrasion test was used to characterize the durability of the compacted specimens. Typically, a maximum loss of 20% is specified for the unaged condition and after aging, the abrasion loss should not exceed 30% (Kandhal 2002). Table 5.5 and Figure 5.12 present the unaged and aged abrasion loss of the mixes for all ten gradations. The results indicate that abrasion resistance was generally influenced by the mixture porosity and air voids, the gradation with the highest porosity (mix C) exhibited the highest abrasion loss and the mix with the lowest porosity (mix H) experienced the lowest abrasion loss. However, when analyzing the entire data set, there was not a strong relationship between the porosity (or air voids) and abrasion loss (Figure 5.13). Results also show that all of the mixes met the recommended criteria for unaged and aged abrasion loss of less than 20% and 30%, respectively (Kandhal 2002).

Having such high resistance to abrasion was not surprising due to the combined use of polymer modified binder and fiber in the mixtures, which resulted in increased binder contents thus increased mix durability. Additionally, the binder contents of the mixtures used in this study were higher than those typically used on SCDOT projects (5.5 – 6.5%). An increase in abrasion loss was expected in the aged specimens when compared to the unaged specimens because binder aging is considered one of the contributing factors in reducing the cohesion and adhesion within the mixture leading to raveling (Mo et al. 2009). Aging, due to oxidation, of asphalt binders increases the stiffness which causes the bond between aggregate particles and the asphalt binder to be more susceptible to breaking. This reduction in abrasion resistance was noticed in six out of the ten gradations. However, in the remaining four mixtures, the aged specimens had lower abrasion values than the unaged specimens. This was also noticed in the results presented in Chapter 4.

Table 5.5. Average mixture performance properties including LSD results between gradations and differences within gradations resulting from conditioning.

Mix	**Abrasion Loss**					**ITS**					**TSR**
	Unaged		*Aged*			*Dry*		*Conditioned*			
	(%)	*LSD*	*(%)*	*LSD*	*Diff.*	*(psi)*	*LSD*	*(psi)*	*LSD*	*Diff.*	*(%)*
A	3.4	c	6.3	abc	N	63.7	d	71.7	b	N	112
B	2.7	c	7.1	ab	N	70.0	cd	67.6	b	N	97
C	16.3	a	6.0	abcd	Y	43.7	e	50.8	c	N	116
D	2.7	c	4.5	bcd	N	80.4	bc	65.4	b	N	81
E	3.4	c	2.6	cd	N	85.1	b	85.6	a	N	101
F	3.5	c	6.1	abcd	N	75.5	bcd	67.8	b	N	90
G	11.5	b	9.4	a	N	71.6	bcd	65.2	b	N	91
H	2.7	c	1.8	d	N	109.5	a	95.0	a	N	87
I	5.5	c	5.3	abcd	N	77.9	bcd	72.2	b	N	93
J	5.6	c	3.3	bcd	Y	81.0	bc	73.8	b	N	91

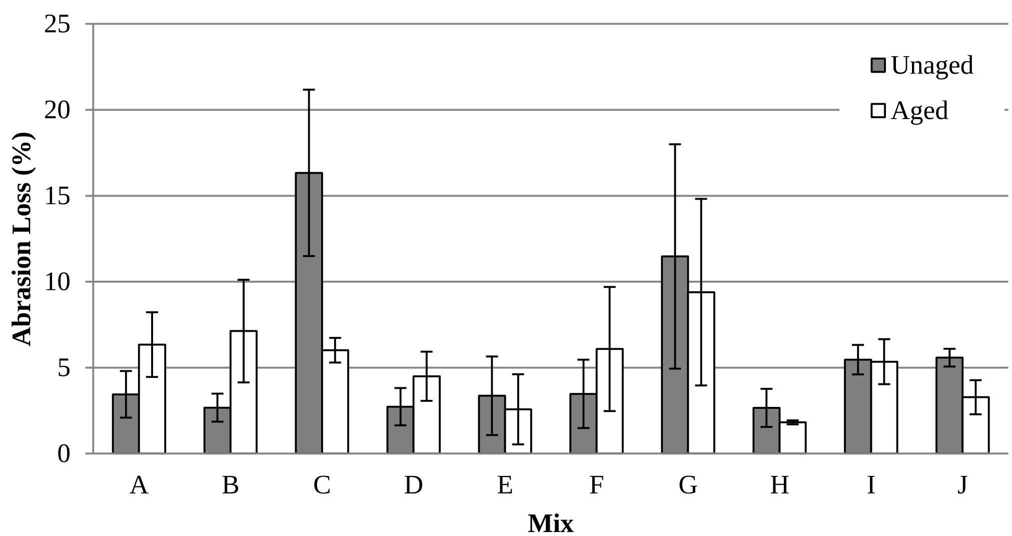


Figure 5.12. Loss due to abrasion for unaged and aged OGFC mixtures

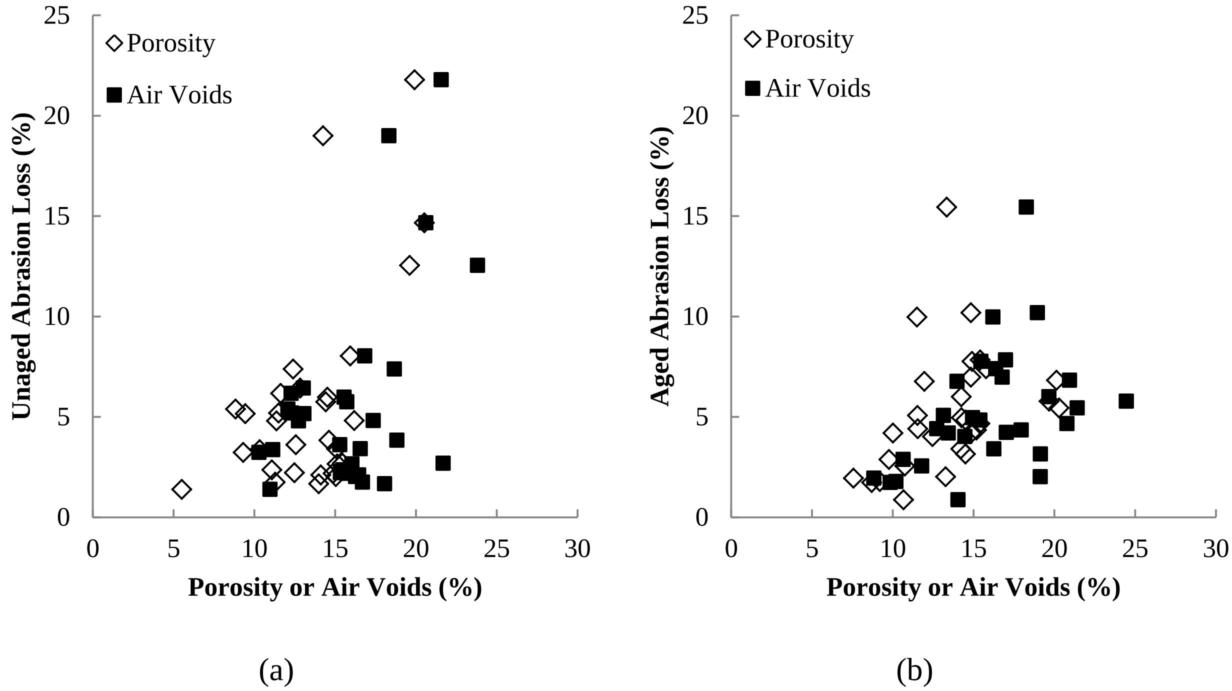


Figure 5.13. Correlation between porosity (and air voids) and (a) unaged abrasion loss and (b) aged abrasion loss

Indirect Tensile Strength and Moisture Susceptibility

The percentage of air in OGFC mixtures is much greater than in conventional dense asphalt, which reduces the support from surrounding particles and thus reduces the strength of the mixtures relative to dense graded mixtures. The indirect tensile strengths of the different mixtures when tested after dry conditioning and after one freeze-thaw cycle are summarized in Table 5.5 and Figure 5.14. Mix H had the highest tensile strength because it is a finer gradation with its particles closely packed together when compared to other gradations. This reduced the percentage of air voids in the mix and increased its tensile strength. In addition, mix C, which had the highest voids and porosity resulted in the lowest tensile strength of all of the mixtures. The same patterns were observed for both the dry and conditioned indirect tensile strengths (Figure 5.15).

The comparison between dry and freeze-thaw conditioned indirect tensile strength was used to examine the resistance of the asphalt mixtures to moisture-induced damage. As shown in Table 5.5, all of the mixtures had a TSR value greater than 80%, which is the minimum recommended by Kandhal (2002) and only one mixture had a value below 85% (mix D), which is the minimum TSR value specified by SCDOT (without a freeze-thaw cycle). Additionally, no signs of stripping were observed upon visual inspection of the fractured specimens. These results indicate that all of the mixtures were relatively resistant to moisture induced damage, which corroborates the historical performance of the aggregate source with respect to stripping. This could also be attributed to the use of hydrated lime as an anti-stripping additive.

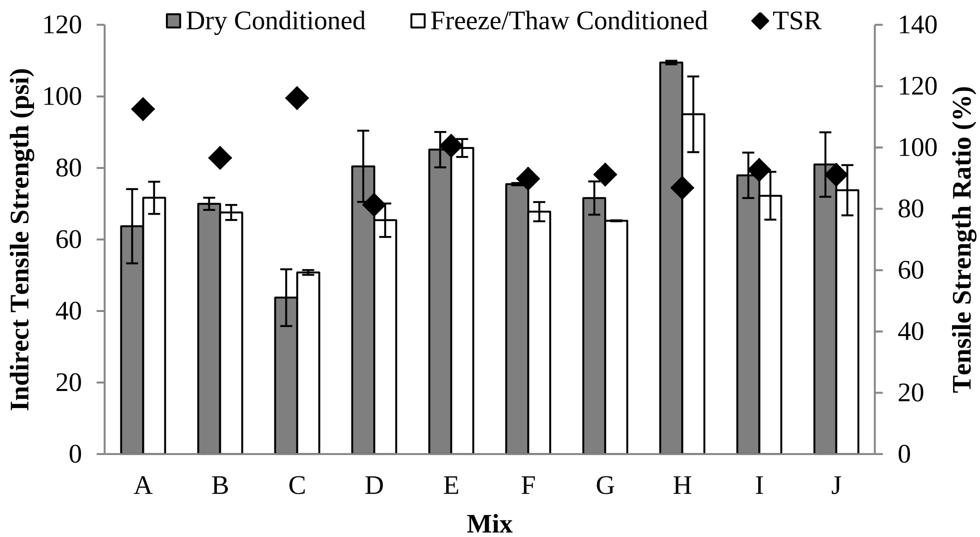
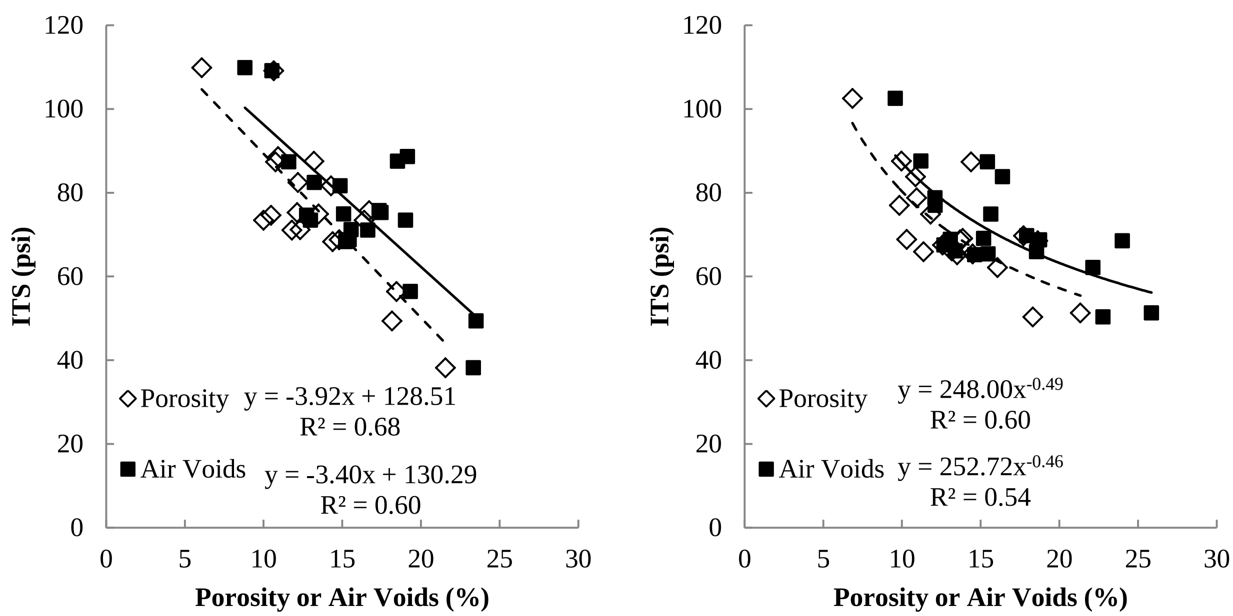


Figure 5.14. Indirect tensile strengths of OGFC mixtures



(a) (b)

Figure 5.15. Relationship between ITS and porosity and air voids for (a) dry conditioned and (b) freeze-thaw conditioned.

Rutting Resistance

One of the disadvantages of porous asphalt mixtures such as OGFC is low rutting resistance compared to gap and dense graded mixtures (Teong 2007). The high air void content in porous mixtures allows aggregate particles to shift their positions when subjected to traffic loading. Rutting is one of the main distresses in asphalt pavements especially in hotter summer temperatures and/or under heavy loads. When considering OGFCs, rutting is not a major concern because they are placed in relatively thin layers (¾ – 1.5 inches). In addition, the void content provides an insulating effect, resulting in lower temperatures within the pavement layer compared with dense asphalt (Van Der Zwan 1990).

Table 5.6 and Figure 5.16 summarize the average rut depths of all mixtures. The results indicate that mix F had the lowest rut resistance compared to the other mixtures. Both mixes I and G showed the highest rut resistance and did show significant differences between each other. The rutting resistance of dense graded asphalt mixtures is influenced by the air content of the mixtures—as the air content increases, the resistance to rutting decreases. Figure 5.17 illustrates that there is not a strong correlation between porosity/air voids and rut depth for the OGFC mixes tested in this study.

Table 5.6. Mixture rutting performance and VCA ratios

Mix	**Rut Depth**		**VCA_{mix}/VCA_{DRC}**		
	(mm)	*LSD*	*Criterion 1*	*Criterion 2*	*Criterion 3*
A	5.7	bc	1.08	1.18	0.86
B	6.0	bc	1.00	1.11	1.00
C	6.4	bc	0.95	1.05	0.95
D	6.9	ab	1.24	1.32	0.87
E	5.2	bc	1.05	1.15	1.06
F	8.8	a	1.31	1.40	0.90
G	4.5	bc	1.13	1.22	0.86
H	5.7	bc	1.42	1.49	1.01
I	4.4	c	1.12	1.21	0.84
J	5.3	bc	0.97	1.08	0.97

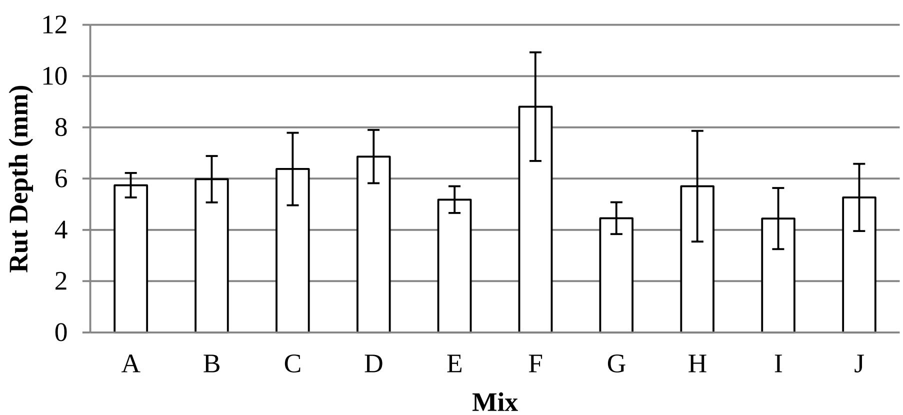


Figure 5.16. Rut depths of OGFC mixtures measured using the APA

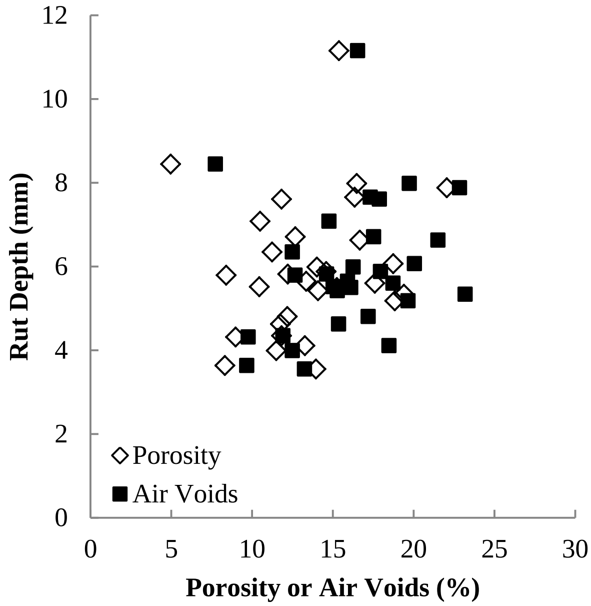


Figure 5.17. Relationship between rut depth and porosity (and air voids)

Another mixture characteristic that is thought to effect permanent deformation of OGFC mixtures is stone-on-stone contact (Mallick et al. 2000; Kandhal 2002; Alvarez et al. 2009). Stone-on-stone contact is evaluated by comparing the voids in coarse aggregate of the mixture (VCA_{mix}) to that of the dry-rodded coarse aggregate (VCA_{drc}). Equations 5.3 and 5.4 are used to calculate VCA_{mix} and VCA_{drc}, respectively. However, to calculate the percent of coarse aggregate, P_{CA}, three different criteria were used to determine the breakpoint sieve for the percent of coarse aggregate in a mix (Equation 5.5):

Criterion 1: The breakpoint sieve is the No. 4 (4.75mm) sieve for all mixtures (Kandhal 2002, ASTM D7064).

Criterion 2: The breakpoint sieve is the sieve below which the slope of the gradation curve begins to flatten out (Watson et al. 2004b).

Criterion 3: The breakpoint sieve is the smallest sieve in which a minimum of 10% of the aggregate is retained (Watson et al. 2004b).

$$VCA_{DRC} = \left[\frac{(G_{CA} \times \gamma_w) - \gamma_s}{G_{CA} \times \gamma_w}\right] \times 100 \tag{5.3}$$

$$VCA_{mix} = \left[1 - \frac{G_{mb} \times P_{CA}}{G_{CA}}\right] \times 100 \tag{5.4}$$

$$P_{CA} = \left(\frac{\%R_{BS}}{100}\right) \times \left(1 - \frac{P_b}{100}\right) \tag{5.5}$$

Where,

G_{CA} = bulk specific gravity of the coarse aggregate
γ_w = unit weight of water
γ_s = dry-rodded unit weight of the coarse aggregate
G_{mb} = bulk specific gravity of the mixture
P_{CA} = percent of coarse aggregate in the mixture
P_b = percent binder in the mixture
$\%R_{BS}$ = percent aggregate retained on the breakpoint sieve

Research has recommended that the ratio of VCA_{mix} / VCA_{DRC} must be less than or equal to 1.0 for there to be stone-on-stone contact when using Criterion 1 (Kandhal 2002) or Criterion 2 (Watson et al. 2004b). Additionally, research by Alvarez et al. recommended that the ratio be less than or equal to 0.9 when using Criterion 3 (2010b). Based on the results obtained from this research presented in Figure 5.18 and in Table 5.6, there was no definitive relationship between stone-on-stone contact and permanent deformation. Additionally, when considering the VCA ratios for the criteria above and the values in Table 5.6, the mixtures with the highest rut depths did not have the highest VCA ratio and those with the lowest rut depth did not have the lowest VCA ratio. This finding was also reported by Mallick et al. (2000). In fact, mix F, which had the highest rut depth (8.8 mm) had a VCA ratio less than 0.9 based on Criterion 3. These findings indicate that stone-on-stone contact alone cannot be used to guarantee resistance to permanent deformation and that more research is needed to develop relationships between stone-on-stone contact and rutting resistance.

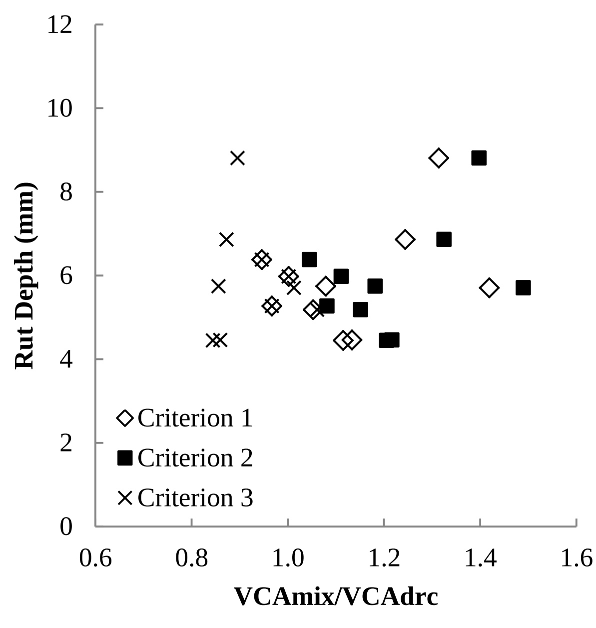


Figure 5.18. Relationship between rut depth and VCA ratio

Conclusions

The objective of this portion of the study was to investigate the influence of aggregate gradation on the properties of OGFC mixtures. Mixtures having ten different gradations that are representative of the gradations included in DOT specifications across the US were designed and tested. The results indicate that aggregate gradation does have an effect on the performance properties of porous asphalt mixtures.

The functional performance of an OGFC mixture, which is measured by permeability is directly related to air voids and effective porosity of the mixture, but showed a slightly better correlation with porosity. This is due to the fact that porosity is a measure of the air voids that are accessible by water. The porosity and air void content are directly related to the void content of the aggregate skeleton, which is dependent on the gradation of the aggregate. These relationships suggest that the permeability of a porous asphalt mixture could potentially be estimated during the design process based on the void content of the aggregate skeleton, but more research is needed to validate this relationship.

The gradation also affected the strength and durability properties of the mixtures to different degrees. The strength property impacted the most by gradation was the indirect tensile strength. In general, as the porosity increased, the indirect tensile strength decreased. While this was the expected trend, it was interesting to note that ranking order of the ITS from highest to lowest, was almost exactly opposite of that for the permeability with a couple of exceptions.

The mixtures also exhibited good durability with respect to raveling. The aged abrasion resistance followed a similar pattern as the dry indirect tensile strength, but the differences in abrasion resistance between gradations were not as statistically significant as with the ITS. This is a result of the variability of the Cantabro abrasion test method, which has also been seen in previous research. The effect of aging on the abrasion loss was not significant in most cases (8 out of 10). In fact, the aged specimens had lower loss values than unaged specimens in some cases. The good durability of the mixtures may be due to the high binder contents used in this study, which could indicate that mixtures currently used having durability issues may have too low a binder content to ensure desired durability.

Finally, there was no definitive relationship between rutting resistance and aggregate gradation based on the gradations and aggregate sources evaluated in this study.

Based on these findings, it is evident that when designing an OGFC mixture, it is important to understand the conditions in which the pavement will be expected to perform as there are trade-offs that need to be considered. The conditions that need to be considered are the required infiltration rate of the pavement, which is directly related to the permeability of the mixture; the traffic loading that the pavement will be exposed to, which will affect the required strength or stability of the mixture; and availability of local aggregate products that can drive the selection of a particular mix gradation and impact the cost of the OGFC.

From the results of this portion of the study, the gradations used for mixes E and H did not have adequate permeability to meet the recommended values of 164 in/hr. After clogging, the permeability of mixtures A, C, D, F, I, and J was reduced by more than 30%, indicating that the functionality of the mixtures with regard to infiltration may be compromised due to the deposition of sediment. Mixes C and G had abrasion loss values greater than 10%, which were significantly greater than the other gradations. While 10% loss due to abrasion is not typically considered high, in this study, it was comparatively high. Finally, gradations F and D exhibited rut depths that were significantly greater than the other mixes. While rutting is typically not an issue in OGFCs because of the thin lifts in which it is placed, it could indicate some potential stability problems that could lead to other problems. Based on this analysis, it appears that the gradation used for mix B has the best balance between functional performance characteristics (permeability related) and structural performance (durability and strength) of the gradations evaluated included in this study. This gradation has more material retained on the ⅜-in. (9.5-mm) sieve compared to the current SCDOT gradation for OGFC and should be studied in greater depth to determine if changes should be made to the specification.

Finally, regardless of the aggregate gradation, the production and installation will have a significant impact on the performance of the finished pavement even if extreme attention to detail is taken when designing the porous asphalt mixture.

CHAPTER 6: THICKNESS DESIGN OF OGFC PAVEMENT LAYERS

The lift thickness of OGFC pavement layers has long been based on experience and is typically standardized by each state DOT. For instance, the Oregon DOT has routinely used 2 inches as a standard OGFC lift thickness for many years. The Georgia DOT specified a thickness of ¾ inches for OGFC layers, but then changed to a thickness of 1 ¼ inches in recent years (Cooley et al. 2009). South Carolina has also adjusted the standard OGFC lift thickness over the years and currently uses typical placement rates of 110 lb/yd^2 (~1 in.) for two lane pavements and 125 lb/yd^2 (~1.2 in.) for three lanes. This adjustment was also based on experience.

Minimum OGFC lift thickness has been recommended to be between 1.5 and 2 times the maximum aggregate size of the OGFC mix. The current OGFC mix specified by SCDOT has a maximum aggregate size of ¾ in (19.0 mm) and a nominal maximum aggregate size (NMAS) of ½ in (12.5 mm). Based on these gradation parameters, the minimum lift thickness in South Carolina would be between 1 ⅛ and 1 ½ inches.

Hydraulic Based OGFC Lift Thickness Design

As part of NCHRP Project 9-41, Cooley et al. set out to recommend a rational method for selecting the lift thickness of an OGFC pavement layer (Cooley et al. 2009). The method that they recommended is based on the flow of water through and unconfined aquifer as suggested by Ranieri (2002). The similarities between an OGFC layer and an unconfined aquifer lie in the fact that there is an impermeable layer beneath a porous layer and a free surface above the porous layer (Figure 6.1). The method for calculating OGFC lift thickness suggested by Cooley et al. is based on Darcy's law for one-dimensional flow and Dupuit's equations resulting in Equation 6.1.

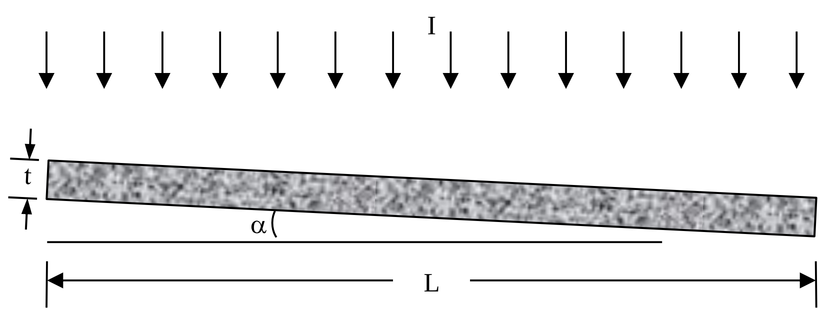


Figure 6.1. Schematic of OGFC layer as an unconfined aquifer with recharge

$$0 = \frac{K}{2L}(L \times \alpha + t)^2 - \frac{IL}{6} \quad (6.1)$$

Where,

K	= permeability of the OGFC mix
L	= length of the flow path
α	= cross slope of the pavement
t	= OGFC layer thickness
I	= rainfall intensity

Equation 6.1 can used to solve for the layer thickness (t) yielding Equation 6.2.

$$t = \sqrt{\frac{IL^2}{3K}} - L\alpha \tag{6.2}$$

The most important variable when calculating the thickness of an OGFC layer is the rainfall intensity (I) because as the intensity of rainfall increases, the thickness of the porous pavement layer must also increase to accommodate the flow of the water within the pavement and without any flow along the pavement surface. Cooley et al, recommend using the 90th percentile rainfall intensity (I_{90}) (2009). That is the rainfall intensity that is not exceeded in 90% of the rain events in a particular area. The I_{90} can be determined by plotting the cumulative distribution function of the rainfall intensity as seen in Figure 6.2. Figure 6.2 includes the cumulative rainfall intensity data from a weather station in Greenville, SC obtained from a database maintained by the National Oceanic and Atmospheric Administration (NOAA), which can be found on the NOAA web page at http://cdo.ncdc.noaa.gov/cgi-bin/HPD/HPDStats.pl. From this figure, the I_{90} is 0.20 in/hr in Greenville, SC. This value indicates that 90% of the rain events in the Greenville area have an intensity of 0.2 in/hr or less.

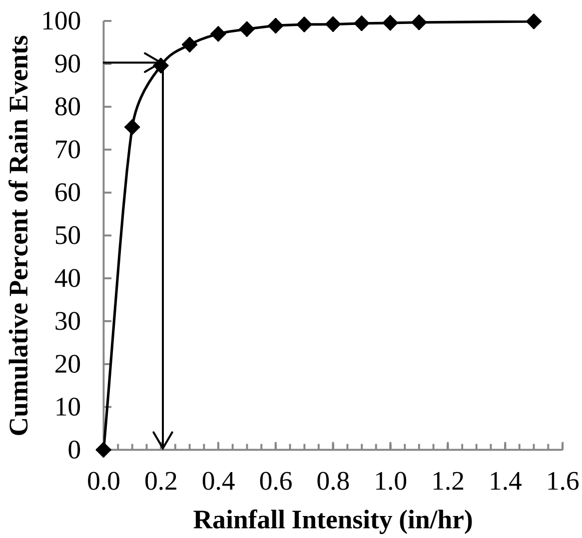


Figure 6.2. Hourly rainfall intensity data for Greenville, SC

The I_{90} values for all of the South Carolina weather stations available in the NOAA database are summarized in Table 6.1. The rainfall data presented in Table 6.1 indicate that I_{90} values range from 0.20 to 0.39 in/hr in South Carolina with an average value of 0.31 in/hr and a standard deviation of 0.05 in/hr. The cumulative frequency of I_{90} values from Table 6.1 is plotted in Figure 6.3. Based on this analysis, the 90th percentile I_{90} value in South Carolina is 0.363 in/hr.

Table 6.1. I_{90} values for South Carolina from NOAA database

Location	**I_{90} (in/hr)**
Belton	0.33
Bishopville	0.32
Calhoun Falls	0.38
Charleston	0.25
Clark Hill	0.29
Clemson	0.34
Columbia	0.23
Georgetown	0.36
Greenville	0.20
Jocassee	0.30
Lancaster	0.26
Laurens	0.31
Lockhart	0.28
Longcreek	0.35
Loris	0.32
Manning	0.32
Moncks Corner	0.25
Mullins	0.36
Newberry	0.32
Pickens	0.27
Saint George	0.34
Santee Cooper Spillway	0.24
St. Matthews	0.32
Travelers Rest	0.39
Wagener	0.31
Ware Shoals	0.38
Winnsboro	0.31
Mean	0.31
Median	0.32
Range	0.20 – 0.39
Standard Deviation	0.05

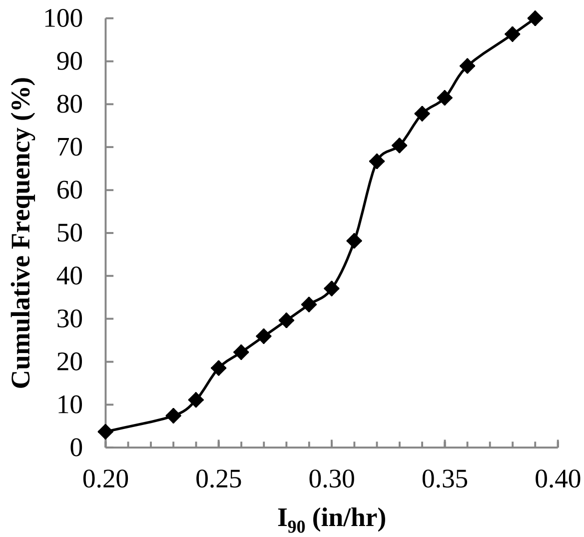


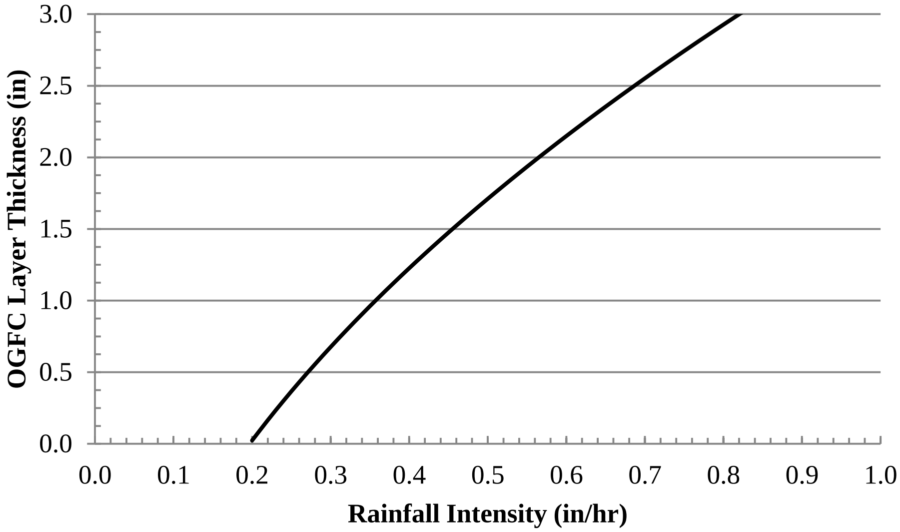
Figure 6.3. Cumulative frequency of I_{90} values in South Carolina

Another critical variable for a proper functioning OGFC is the permeability of the mixture itself. As discussed in Chapter 4, the most commonly recommended value for mixture permeability is 164 in/hr (Kandhal 2002). Based on Equation 6.2, if the permeability (K) increases, the layer thickness can be reduced due to the higher flow capacity through the layer. However, if the permeability is decreased for any reason (compaction, clogging, etc.), the thickness must be increased to accommodate the same flow.

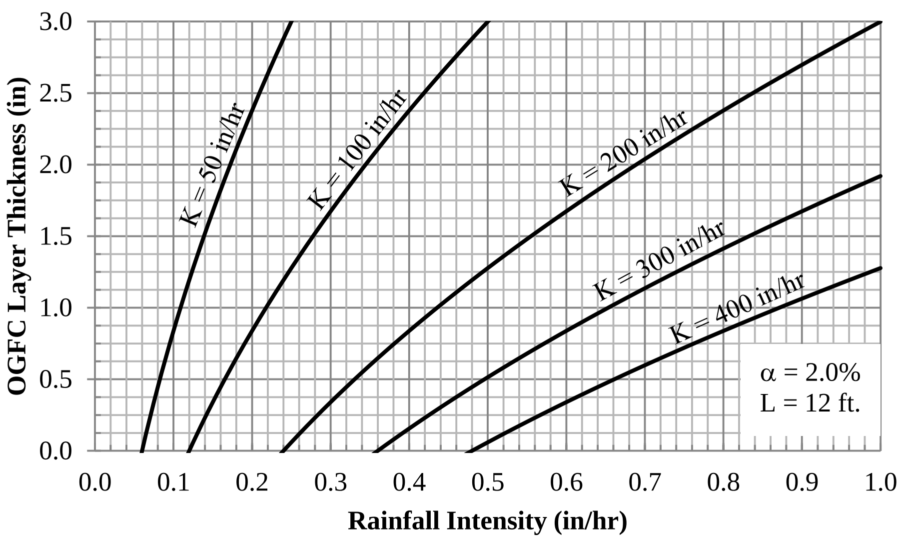
Once the design variables have been identified, it is possible to design the required thickness of an OGFC layer using Equation 6.2. Based on a I of 0.37 in/hr, a permeability (K) of 164 in/hr as recommended, a cross slope (α) of 2.0%, and a pavement width (L) from crown to shoulder of 12 ft, Equation 6.2 yields a required thickness of 1.07 inches, which would likely be rounded up to practical thickness such as 1 ⅛ or 1 ¼ inches.

$$t = \sqrt{\frac{IL^2}{3K}} - L\alpha = \sqrt{\frac{0.37(12\times 12)^2}{3\times 164}} - (12\times 12)0.02 = 1.07in$$

This design methodology also enables the designer to conduct a sensitivity analysis to understand how changes to the input variables can affect the hydraulic behavior of the OGFC layer. Figures 6.4 through 6.7 illustrate the sensitivity of the OGFC layer designed above to changes in the rainfall intensity, pavement width, cross slope, and permeability, respectively. Based on these figures, it is evident that rainfall intensity and permeability have the greatest influence on the required pavement thickness. Cross slope is also important, but cross slope values are typically at least 2.0%. Finally, the flow path length is also a factor, but a majority of the pavements that are OGFC surface are four-lane divided highways. In these roadways, there are two lanes in each direction with a crowned cross-section. For these sections, the pavement width is 12 ft. and the OGFC also overlaps onto the shoulder 2 to 8 ft. This makes the overall flow path length 14 to 20 ft. For roadways having three or four lanes, the lengths can range from 24 to 32 ft. depending on how far the OGFC layer overlaps the shoulder. For these longer distances, either the thickness of the pavement should be increased, the permeability of the mix should be increased, or the cross slope should be increased to ensure that there is no water flow across the pavement surface. Alternatively, a combination of these modifications can be used.



(a)



(b)

Figure 6.4. Effect of rainfall intensity on design lift thickness using (a) the input values of K=164 in/hr., L=12 ft., α=2.0% (b) variable permeability values keeping L and α constant.

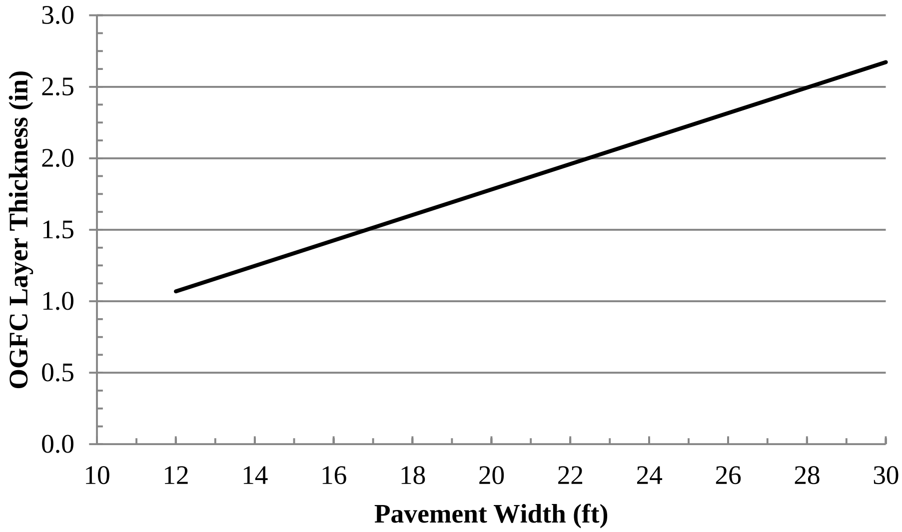


Figure 6.5. Effect of pavement width (flow path length) on design lift thickness (I=0.37 in/hr., K=164 in/hr., α=2.0%)

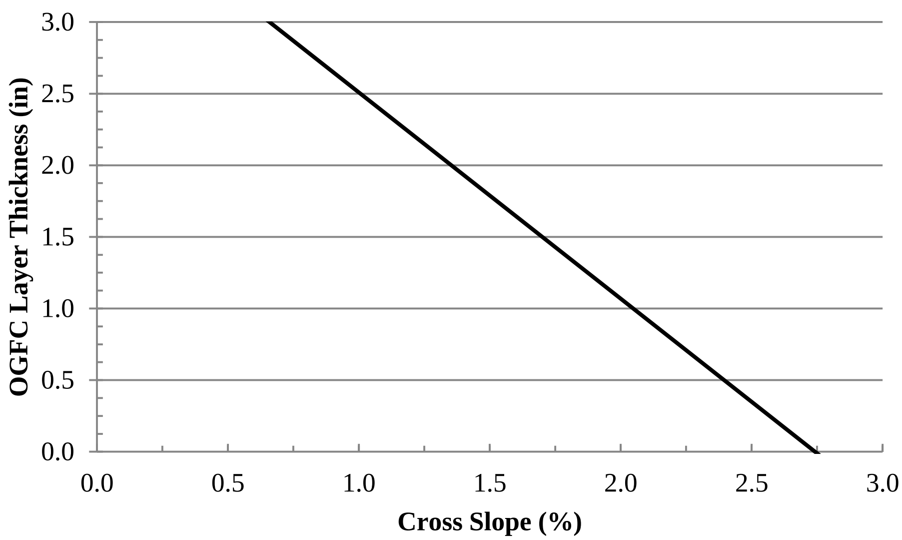
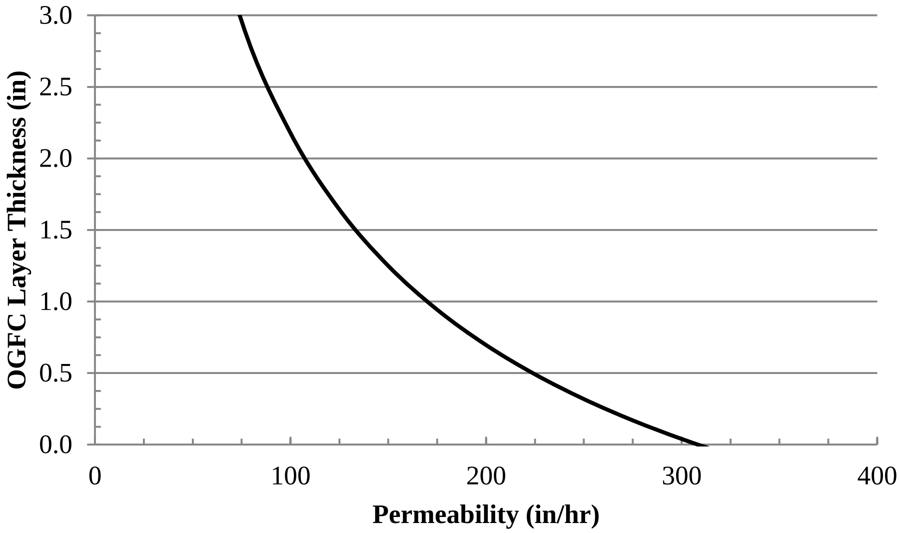
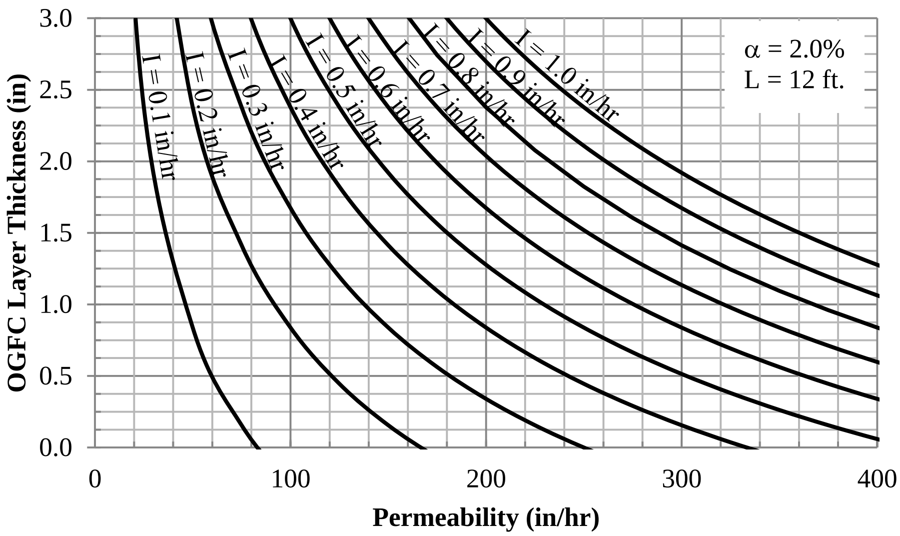


Figure 6.6. Effect of pavement cross slope on design lift thickness (I=0.37 in/hr., K=164 in/hr., L=ft.)



(a)



(b)

Figure 6.7. Effect of permeability on design lift thickness using (a) the input values of I=0.37 in/hr., L=12 ft., α=2.0% and (b) variable rainfall intensity values keeping L and α constant.

Further analysis of the impact of permeability on the required pavement thickness illustrated in Figure 6.7 emphasizes the potential detrimental effect of permeability reduction due to clogging of the OGFC surface. As discussed in Chapters 4 and 5, the mix design and gradation have a major influence on the initial permeability of an OGFC mixture. Additionally, in Chapter 5, the clogging susceptibility was evaluated for OGFC mixtures made with different aggregate gradations. This investigation revealed that the range of permeability reduction due to clogging with sand was between 19 and 43%. Applying this reduction to the design curve in Figure 6.7, would result in a thickness increase of 41 to 120% yielding a thickness ranging from 1.51 to 2.35 inches. OGFC lifts this thick may not be practical in some cases. This suggests that the initial permeability of the OGFC should be designed to account for this potential reduction due to clogging. Applying a "clogging factor" of 1.19 to 1.43 to the design permeability of 164 in/hr results in a mix design permeability ranging from 195 to 235 in/hr. It should be noted that these reduction factors are based on the results of this study (Chapter 5) and are not indicative of actual field permeability reduction factors due to clogging.

Based on the thickness design considerations discussed in this chapter, it is recommended that the SCDOT consider the following when selecting the thickness of an OGFC layer:

- Select lift thicknesses that are at least 2 times the maximum aggregate size of the mixture. This could potentially reduce the chance of progressive raveling in the event that localized raveling occurs at the surface. When the pavement layer is too thin relative to the size of the aggregates in the mix, the loss of one larger particle could conceivably create a void that is more than 50% of the lift thickness.
- Use Equation 6.2 to calculate the minimum lift thickness of the OGFC based on the pavement dimensions, material properties (permeability), and rainfall intensity. A standard lift thickness can be calculated for different scenarios in South Carolina. The main variable that would change for the different roadways, would essentially be the number of lanes.
- Evaluate the performance of OGFC lift thicknesses ranging from 1 ¼ to 1 ½ inches in South Carolina.

CHAPTER 7: CONSTRUCTION OF OGFC

OGFCs have four stages of construction: Production, transportation, placement, and compaction. In addition, QC/QA is vital to successful paving. Many of the best practices from SMA paving can be utilized in OGFC paving since there are similarities between the two such as a coarser gradation. Almost two-thirds of agencies which responded to a National Cooperative Highway Research Program (NCHRP) survey in 2009 indicated that OGFC mixes were part of their standard specifications and paving operations while the other third had special provisions (Cooley et al. 2009).

Plant Production

The majority of typical HMA plants can be used to produce OGFC mixtures. A plant should have well marked and separated aggregate stockpiles and cold feed bins because the aggregate gradation of OGFC mixtures is so different than conventional HMA mixes. The asphalt binder is stored and added like typical HMA, but may be at higher temperatures depending on the binder type selected in design (Cooley et al. 2009). OGFCs usually have additives which a plant may need to make provisions for. The most common additive is stabilizing fibers as they are used by 90% of the respondents to the NCHRP survey (Cooley et al. 2009). These additives also typically increase the mixing time by up to 30 seconds to ensure it is sufficiently distributed in the mixture. The storage time is also important as OGFC mixtures have the potential for draindown. The typical maximum storage time was 2 hours for states which responded to the survey and had standard specifications for OGFCs.

Aggregates

Stockpile management is critical as with any HMA. Making sure stockpiles are on clean, sloped, well-maintained surfaces is vital in keeping the moisture content low and consistent. This makes controlling production temperatures much easier (Huber 2000).

For OGFC mixes, coarse aggregate (retained on the No. 4 sieve) comprises 85 percent of the aggregate gradation. This creates better stone-on-stone contact in the pavement structure and higher air voids providing a durable, permeable surface. Since the coarse aggregate is so controlling in the mix design, more than one cold feed bin should be used to handle and monitor the aggregate to minimize variability and provide better control (Brown and Cooley 1999).

Asphalt Binder

Asphalt binder in OGFC production is handled very similarly to regular HMA production. Plants usually have a second storage tank for modified binders, but most plants that regularly utilize modified binders will already be setup for this. Agitators are also typically necessary to handle modified binder storage (Huber 2000); however, binder should be handled as recommended by manufacturer.

Stabilizing Additives

Because OGFCs utilize higher binder contents and a coarser gradation, the mix is susceptible to binder draindown. A stabilizing additive is required to prevent draindown when producing and transporting the mix. Usually, modified binders and fibers are used to counteract this draindown and improve mix durability. Draindown testing should be performed during the mix design to determine the draindown potential and need for additives, such as stabilizing fibers.

There are two types of stabilizing fibers commonly used in OGFC mix designs, cellulose or mineral fibers, and these can come either loose or pelletized. Both types and configurations can be used in both batch and drum mix plants. Typically, cellulose fibers are added at a rate of 0.3% of the mix by weight, and mineral fibers are typically added at 0.4% of the mix. However, reported rates vary from 0.1 to 0.5% (Kandhal 2002).

To introduce fibers in a batch plant, fibers usually come in bags that melt when heated (Decoene 1990), so whole bags can be introduced directly into the pugmill using a conveyor system. The drawback to this system is that it is manual and therefore labor intensive. Another method used to introduce fibers with the use of a blower. A blower fluffs the fibers with large paddles as it meters the fibers and blows them into the pugmill or the hopper. This method is also used at drum mix plants. Fibers can be introduced into the lime injection point so that it mixes with the aggregate before it enters the drum (Santha 1997); however, the fiber must be captured by binder before it reaches the gas system or the fibers will be collected by the baghouse (Kandhal 2002). Careful monitoring of the fiber introduction system must be performed since variations in the fiber input can be detrimental to the mix. A common method of doing this is to provide a clear section in the hose between the fiber blower and the introduction point (Cooley et al. 2009).

Pelletized fibers are introduced similarly to loose fibers. They are carried on a calibrated conveyor belt and introduced in the pugmill of a batch plant or with the RAP collar in a drum mix plant. Either way the fibers are introduced, they mix with the heated aggregate and the pellets warm allowing them to bond with the aggregate. Some pelletized fibers contain a small amount of asphalt binder, but the manufacturer will recommend how this binder should be accounted for in the mix (Kandhal 2002).

It is important to consult the plant manufacturer to determine the best entry point for fibers into the drum mixer to optimize mix uniformity. Another important consideration when using fibers no matter what type or how they are introduced is careful monitoring of the system to prevent variability in the mix (Huber 2000).

Another method for stabilizing OGFC mixes is the use of asphalt binder modifiers. These modifiers can be introduced at the plant during mix production or at the refinery or terminal. For the plant method, modifiers can be introduced during production or directly placed in the dry aggregate. Addition of the modifier is performed by either in-line blending or by blending the modifier and the binder in a storage tank. If the modifier is incorporated into the aggregates, it can be introduced directly into the pugmill for a batch mix plant or introduced with the RAP system in a drum mix plant (Brown and Cooley 1999). Stabilizer advice and assistance should be sought from the stabilizer provider and plant manufacturer when deciding the best method to introduce the modifier.

Mixture Production

Plant calibration should be performed carefully for the mixing of OGFC. Both the calibration of the aggregate bins and the additive system must be performed with care to limit variability (Cooley et al. 2009).

Careful monitoring of the mixing temperature is necessary for the production of a high quality OGFC in accordance with the properties of the asphalt binder. If the mixing temperature is too hot, the mix can be more susceptible to draindown and premature oxidation which can cause poor performance. Arbitrarily lowering the temperature can prevent the moisture removal from the mix or leave the mix too cool for compaction once it reaches the site (Cooley et al. 2009). Compared to typical HMA pavements, OGFC mixing temperatures should be about the same or a little higher. When using a batch mix plant the screening capacity of the screen deck must be checked because the largely uniformly graded aggregate can create problems (Kandhal 2002). Start-up and clean-out procedures are also crucial when dealing with an OGFC mix (Choubane et al. 1999).

Experience with OGFC pavements has shown that mixing time must be extended slightly due to the introduction of fibers (Kandhal 2002). For batch plants, mixing times in both the wet and dry cycles must be extended 5 to 15 seconds. For drum mix plants, the asphalt binder injection can be relocated or basically extended to allow for more complete mixing of the pellets (Brown and Cooley 1999). Most importantly, the mix should be visually inspected for undistributed fibers, and the mixing times can be adjusted accordingly (Cooley et al. 2009).

In general, OGFC mixes should not be stored for extensive periods of time because this promotes draindown (Kandhal 2002). The recommended maximum storage time for an OGFC mix is 2 hours. Experience has shown that mixes can typically be stored for 2 hours without detriment (Cooley et al. 2009).

Transportation

When transporting OGFCs, the main concern is to reach the construction site with the mix at an appropriate compaction temperature. This can be achieved by limiting haul time, limiting haul distance, or specifying a minimum arrival temperature. Most agencies which responded to the NCHRP survey had a minimum specified temperature, but one had a maximum haul distance of fifty miles and one had a maximum haul time of one hour. The specified temperature varied from 225°F to 300°F depending on climate and binder type (Cooley et al. 2009). The SCDOT requires that OGFC be placed on the roadway at a temperature no less than 320°F (SCDOT 2007). Another concern is temperature variation within the mixture in the trucks. Insulated trucks and tarps to cover the mix help minimize this, but ideally, a material transfer device should be used to re-mix the asphalt before placing it in the paver.

One imperative aspect of the paving process, especially for OGFCs, is having adequate transportation, so the paver does not have to stop to wait for trucks. These trucks must be cleaned thoroughly with a heavy release agent, and any excess release agent must be dumped out because excess can cause cold spots in the mat (Kandhal 2002). Trucks should be tarped to prevent excess crusting on the mix. Moreover, the haul distance should be limited to approximately 50 miles. Some agencies go even further by requiring an insulated truck bed or a heated dump bed (Huber 2000).

The haul time for OGFC should obviously be as short as possible. Haul time always governs over haul distance and should be limited to 2 hours max; however, less than one hour is preferred—SCDOT specifies that the mix be placed within one hour from mixing at the plant (SCDOT 2007). Arbitrarily raising the mix temperature during production to make a longer haul is problematic because this facilitates draindown (Brown and Cooley 1999). Increasing the mix temperature in this manner can also cause premature oxidation of the binder in the mix.

Placement

As with conventional HMA, OGFCs are typically placed with a paver. The same precautions concerning weather and surface preparation are followed such as temperature, precipitation, and existing surface conditions. Edge clearing is another consideration which should be acknowledged in OGFC placement as debris can clog the pores near the pavement edge when not properly cared for (Estakhri et al. 2007). Also, a tack coat is normally required before the placement of OGFCs (Kandhal and Mallick 1998). Because OGFCs are placed in thin lifts (¾ to 2 inches), it is also important not to constantly correct the yield as it can result in unsatisfactory pavement projects. The binder type, often polymer modified, will be sticky and minimum hand work should be performed.

OGFC should not be placed in cold or inclement weather. The minimum paving temperature for OGFC is typically 50°F (10°C) although some agencies specify higher temperatures (Alderson 1996). The SCDOT specifies that the ambient air temperature during placement of OGFC is 60°F or higher when measured in the shade (SCDOT 2007). Other environmental factors such as wind and the existing pavement temperature should also be taken into consideration.

Preparation of the existing surface for the placement of OGFC is similar to the preparation for any HMA layer. Since OGFCs are designed to allow water to infiltrate the pavement structure, the layer below the OGFC must be impermeable. This layer can be an impermeable HMA layer or a Portland cement concrete layer (Lefebvre 1993). Placement on an impermeable layer will ensure that water passing through the OGFC will laterally drain off the surface of the underlying layers. If water is trapped in the pavement structure, stripping will most likely occur. OGFC should never be placed on rutted pavement because ruts will act as troughs catching the water. Any pavement containing large or permanent deformations should be milled and repaved or filled and sealed (Cooley et al. 2009).

OGFCs should be daylighted on the shoulder of the roadway so that water draining through the pavement can escape (Lefebvre 1993). There are two methods for terminating the edge of an OGFC pavement, either the pavement extends the length of the shoulder to the edge or the pavement is terminated on the shoulder (Ruiz et al. 1990). For either case, a strip at least 4 inches wide should be left between an OGFC and any grass area while the OGFC should extend at least 12 to 20 inches onto the shoulder before being tapered off. With either case, the underlying layer must be impervious. Regular HMA can have as high as 8% air voids so most HMA layers must be sealed with a heavy tack coat or fog seal (Alderson 1996). Any cracking or other deformations must be repaired and sealed as well. Application rates must be sufficient to completely seal the surface of the pavement. The FHWA recommends the use of a 50% diluted slow-setting asphalt emulsion applied at a rate of 0.05 to 0.10 gal/yd^2 (FHWA 199). The use of a slow-setting emulsion is more likely to seal the surface voids of the underlying pavement more effectively than a faster setting material. A wide range of tack and seal coats have been used to seal underlying layers (Kandhal 2002; Ruiz et al. 1990; Bishop and Oliver 2001); however, if a pavement is two years old or more the surface is likely already sealed from traffic. Pavements that have been micro-milled will require a higher tack coat application rate to seal the freshly exposed surface, which also has greater surface texture. Kandhal and Mallick reported that the use of tack coats when constructing OGFCs is required by all of the DOT survey respondents and that the application rates vary from 0.022 to 0.11 gal/yd^2 (Kandhal and Mallick 1998). Estakhri et al. recommended an emulsion tack rate of 0.08 to 0.1 gal/yd^2 in Texas (2007); the

New Jersey DOT requires a PG 64-22 binder tack applied at a rate of 0.06 to 0.14 gal/yd^2 (NJDOT 2007); and CalTrans requires a minimum emulsion tack rate of 0.07 gal/yd^2 over new HMA, 0.11 gal/yd^2 over existing HMA and PCC pavements, and 0.12 gal/yd^2 over planed pavement (CalTrans 2009).

Although paver operation for OGFC placement is very similar to that of regular HMA placement, a hot screed is of vital importance to prevent tearing of the mat. Heating the screed can be performed with a torch before paving begins (Cooley et al. 2009).

The OGFC mix is typically delivered to the paver in the same manner as regular HMA. The trucks should not back up all the way to the paver because the resulting depression is more difficult to smooth, which will affect the pavement roughness. Some agencies require material transfer vehicles (MTV) as they improve the temperature consistency of the mix by remixing the colder, crusted top of the mix with the hotter mix. MTVs also help maintain continuous operation of the paver and reduce material segregation. Windrows are allowed in some states when placing OGFCs, but the windrow length and condition should be monitored and will be affected by the weather. Windrows in ideal weather conditions should not exceed 150 feet (Huber 2000).

The paver must be calibrated before the placement of an OGFC. This calibration should include the flow gates, slat conveyors, and augers. Auger extensions should be employed if extendable screeds are used (Huber 2000).

When placing an OGFC, the paver speed is largely governed by the ability to compact the mat. For successful placement, production, delivery, and compaction must all be coordinated to ensure continuous operation. Stops and starts cause significant negative impact as the mat cools and rollers have to sit and wait on the mat (USACE and FHWA 2000). Also, augers should not be run at excessive speeds as starting and stopping could potentially strip the binder film from the aggregate. The paver speed should be calibrated so that the augers run 85 – 90% of the time at a consistent speed (Brown and Cooley 1999). Paver wings should not be lifted unless mix is being discarded.

The minimum lift thickness for an OGFC pavement should be about twice the maximum aggregate size. Typical lift thicknesses for OGFC pavements are between 1.25 and 2 inches. A tolerance of ± 0.25 inches should be allowed (Brown and Cooley 1999).

A minimum amount of hand working should be performed on OGFC pavements since the mix is known to be very sticky and stiff. When necessary, hand placement of material should be performed carefully (Lefebvre 1993). Longitudinal joints are created by placing the mix ⅛-in above the previously placed lane. Excessive overlay should be limited and longitudinal joints should not be tacked except at the crown of the road because tacking will prevent drainage through the pavement (Kandhal 2002). When horizontal (transverse) joints are required, the screed should be started 1 foot behind the joint, and the joint must be cross rolled by a steel wheel roller (Brown and Cooley 1999).

Compaction

The initial breakdown roll of OGFCs should be the same as dense graded HMA so that the compaction can occur at the correct temperature. The high air void content will allow the pavement to cool faster and require different finishing compaction techniques. A pneumatic tire roller is not recommended for OGFCs, only static steel wheel rollers are used (Kandhal 2002).

The main difference between compaction of dense graded mixes and OGFCs is that the compaction goal of dense graded mixes is to create an impermeable pavement while OGFC needs to be compacted enough to seat the mixture to create cohesion within the mat; promote a good bond to the tack coat; and create a smooth riding surface.

Rolling should be performed with the same intensity for OGFC as regular HMA requires. To achieve correct compaction, the breakdown roller should follow the paver closely, no more than 50 feet, depending on the mix temperature. Pneumatic rollers should not be used on OGFC pavements. On thin lift thicknesses of 1 inch, one or two passes in the static mode is enough to achieve adequate compaction. On thicker lifts, 2 to 4 passes are typically necessary using a 10 ton roller at a maximum speed of 3 mph. If rollers must wait or sit idle, they should be removed from the pavement mat. Rolldown for normal HMA is between 20-25% of the lift thickness, but for OGFC, rolldown is approximately 10-15% of the lift thickness (Huber 2000). Vibratory rollers (in vibratory mode) should not be used on OGFCs, but finish rolling to remove marks may be performed by a static steel wheeled roller (Kandhal 2002). A water/release agent mix may be used on rollers to prevent mix from sticking to the roller, but excess water should not get on the mat (Brown and Cooley 1999). The goal of compaction with OGFC mixes is quite different than that of regular HMA mixes. HMA is compacted to 5-7% air voids immediately after compaction to seal the pavement surface. No minimum density is required for OGFC pavements; however, a few agencies require permeability testing (McDaniel and Thronton 2005).

A minimum density is not required for OGFC, since the objective of compacting an OGFC is not to achieve a certain density. Nevertheless, if density measurements are desired, some agencies calculate the bulk specific gravity of pavement cores using the volumetric method while some agencies perform the vacuum seal test. For the vacuum seal test, the double bag method is generally necessary (Watson et al. 2002).

Quality Control/Quality Assurance

Testing for QC/QA should target binder content, gradation, and draindown to ensure that the mix meets the job mix formula. Draindown testing ensures that stabilizing additives are added appropriately. Pavement smoothness should be tested to ensure quality of the finished OGFC wearing course (Cooley et al. 2009). Some countries have also employed permeability testing to determine if the mix has been properly constructed (Decoene 1990; Ruiz et al. 1990; Bolzan et al. 2001; Iwata et al. 2002). Argentina has adopted the Cantabro Abrasion loss test as part of the QC program (Bolzan et al. 2001). The percent within limits (PWL) system was developed similarly to the system used for HMA pavements, but the specifications for materials and other variables are unique to OGFC pavements. The four criterion used to develop the PWL system that are believed to directly relate to performance are asphalt binder content, percent passing the ⅜ in. (9.5 mm) sieve, percent passing the No. 4 (4.75 mm), and percent passing the No. 8 (2.36 mm) sieve (Sholar et al. 2005).

Pavement Markings

Some problems have occurred with thermoplastic markings because this material can heat up the OGFC causing draindown and eventual surface raveling due to the draindown in the areas beneath and surrounding the marking. With this in mind, fully recessed thermoplastic markings have proven to be the most durable and provide the best reflectivity (Corrigan et al. 2001).

CHAPTER 8: MAINTENANCE OF OGFCS

A significant amount of research has been conducted on maintenance for OGFC pavements, both general and winter maintenance.

General Maintenance

General maintenance involves cleaning clogged OGFC, preventative surface maintenance, and corrective surface maintenance.

Clogging

OGFCs may become partially clogged or choked due to dirt and debris deposited into the voids of the pavement structure over time (Rogge and Hunt 1999). Three possible methods for cleaning an OGFC pavement are with a fire hose, a high pressure cleaner, or a specially engineered cleaning vehicle. Vehicles have been developed in numerous regions for this purpose (Hiershe and Freund 1992; Abe and Kishi 2002). By performing permeability tests after cleaning, Hiershe and Freund found the high pressure cleaner to be the most effective (1992). Cleaning an OGFC while the pavement is still permeable is more desirable than trying to unclog a clogged pavement (Isenring et al. 1990).

Preventative Maintenance

Over many years, binder in OGFC pavements can become oxidized and brittle. Some agencies have used a fog seal to rejuvenate an OGFC pavement (Cooley et al. 2009). The seal is a 50:50 mix of emulsion and water with no rejuvenating agent (FHWA 1990). However, some research suggests that fog seals reduce the permeability of the OGFC, and the assumption that the fog seal extends the life of the pavement may be unsubstantiated. The fog seal lowers the surface friction of the pavement initially after placement but does not negatively affect the reduction in hydroplaning potential because the macrotexture is still maintained with the fog seal. Chip seals may also be used to seal the surface of an OGFC when water damage to underlying layers is a problem (Rogge 2002). Raveling is another issue seen in OGFCs due to draindown, excessive cooling, or insufficient compaction (Ruiz et al. 1990). Some agencies employ a seal coat to halt light raveling or repair distressed areas (Wimsatt and Scullion 2003). This practice may be necessary to maintain the pavement serviceability until a contract can be executed to correct a severe raveling issue.

Corrective Maintenance

When an OGFC needs to be repaired, another OGFC mixture is necessary to repair or patch a delaminated or potholed section if the section is large enough to justify the production of OGFC mixes. Regular HMA can be used when the patch size is small and flow through the OGFC layer will not be significantly altered (Rogge 2002). If using a dense graded HMA mix to patch, CalTrans recommends that the patch should be diamond shaped and oriented so that the water can drain along the patch at 45 degree angles (CalTrans 2006). An example of this is illustrated in the sketch in Figure 8.1. However, milling an area of this shape may be difficult depending on the equipment used. Alternatively, the grade and cross-slope of the pavement section should be considered when designing patches for OGFC as the water will flow downhill along the edge of a dense graded patch until it can flow laterally to the edge of pavement. With this, it is important that a patch does not extend continuously through a “valley” in the pavement section. When patching with an OGFC mix, a light emulsion tack coat (as opposed to a heavy tack) should be applied to the edge of the patch to prevent the drainage of the OGFC from being inhibited. Because of the textured surfaced of the OGFC, small cracks are hidden, and any visible cracks are significant and also need to be sealed. Transverse cracks can be sealed using normal methods since the permeability of the pavement will not be affected. Longitudinally cracked areas are more troublesome because sealing the cracks will prevent drainage. One method of attacking this problem is to mill the strip of pavement surrounding the longitudinal crack and replace with OGFC. Rehabilitation is the only other solution if the severity of the crack becomes excessive (Cooley et al. 2007).

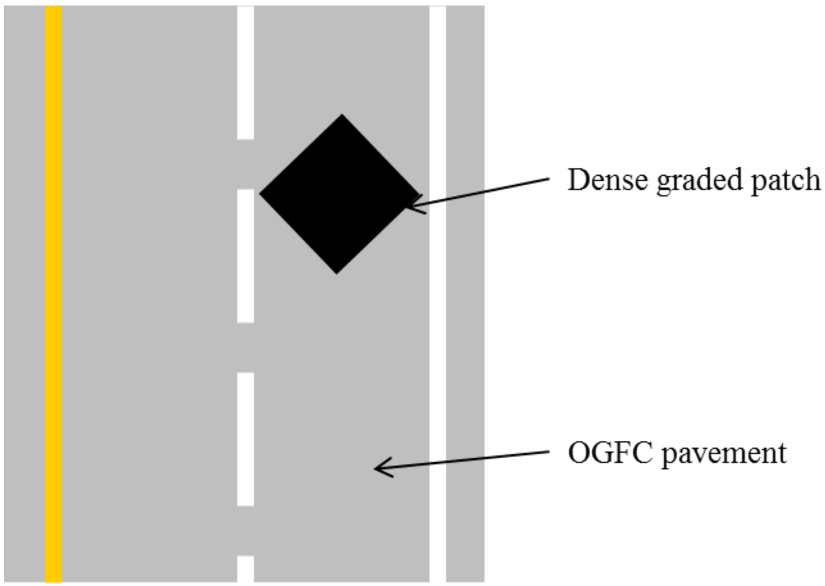


Figure 8.1. Schematic of a diamond shaped patching solution for an OGFC pavement

Winter Maintenance

Winter maintenance of OGFCs needs to be carefully considered before the construction of a pavement. In the United States and Europe, studies have found that snow and ice accumulates differently on OGFC pavements than on traditional pavements (Yildirim, et al. 2006). The porous structure does not insulate like a dense graded structure and an OGFC surface is typically several degrees cooler than a traditional pavement surface. This allows for faster accumulation of snow and ice as well as faster freezing. Additionally, when the temperature of an OGFC drops below freezing, it will stay below freezing longer than regular HMA pavements, thus leading to delayed thawing (Huber 2000; Yildirim et al. 2006).

Some of the advantages for OGFCs in winter conditions include (Isenring et al. 1990):

1. Ice generally will not form on wet OGFC surfaces.
2. The high level of macrotexture is beneficial when snow and slush exist.
3. The tendency for ice formation in the wheel paths is reduced.

Some of the disadvantages for OGFCs in winter conditions include (Isenring et al. 1990):

1. The need for increased quantities of deicing chemicals.
2. The use of sand and small aggregates is not recommended because these will clog the pavement surface pores.
3. Snow and ice tend to accumulate quicker on OGFCs because the mixture cools faster than dense graded mixtures.
4. Snow and ice can form on the surface quicker because deicers do not stay on the pavement surface.
5. Preventive salting is not beneficial because the salt penetrates into the void structure.
6. If the OGFC permeability is reduced as a result of clogging or other reasons, ice will build up in the pavement voids and expand onto the surface.
7. Icing can occur in adjoining dense graded HMA since OGFCs limit salt transfer by cars.

There is a wide range of winter maintenance practices and opinions about winter maintenance for OGFC across the US and Europe, so the following is a summary of the state of the practice for winter maintenance. The reason for the inconsistency is that the behavior of the salt with the OGFC is unpredictable and varies greatly from mix to mix (Greib 2002). Because of this problem, some states have indicated that there is no definitive solution for OGFC winter maintenance (Padmos 2002). Others, however, suggest that solutions can only be found through experience (Brousseaud and Anfosso-Lédée 2005).

Several winter weather conditions cause concern for OGFC pavements. The phenomenon known as freezing fog, rain falling on a frozen OGFC layer, or simple snow or sleet can be problematic as research shows that OGFC layers have 40-70% the thermal conductivity of a regular HMA pavement (Lefebrve 1993). Because of this, OGFCs consume more deicing materials, and OGFCs must be treated with deicers or anti-icers soon after plowing or slush will freeze in the voids of the pavement. Once frozen, the resulting layer of ice is much more difficult to remove from an OGFC layer than from a regular HMA. However, a research study conducted in Japan showed that when snow hit a wet HMA surface it melted while an OGFC surface turned the same snow to slush. Other examples from this study show that OGFC layers did not promote black ice. Precipitation was more likely to remain in the state it fell on an OGFC (Iwata et al. 2002).

While the need for salt is generally higher on OGFCs due to salt solution penetrating the void structure, research has shown that as long as traffic volumes remain high, the salt solution will be pumped in and out of the void structure by the traffic diminishing the need for extra salt (Greibe 2002). This phenomenon has been observed in multiple studies as researchers have noticed higher friction numbers in the travel lane of an OGFC pavement. In fact, friction numbers in winter weather conditions on OGFCs have actually proven to be better than regular HMA surfaces except for the case of compacted snow (Bennert and Cooley 2006; Padmos 2002; Iwata et al. 2002).

Traditional winter maintenance methods include salting, sanding, de-icing chemicals, and snow plowing. Salting can be used, but needs to be done carefully. Small pieces should be used to dissolve quickly and minimize pore clogging. Sanding is not a practical option for OGFCs because it would clog the voids and severely reduce the permeability. Snow plowing needs to be done carefully as an OGFC surface has less resistance to the blade of the snow plow. De-icing chemicals can be used, but they need to be applied more often as the chemicals will drain through the pavement structure (Yildirim et al. 2006).

Some studies conducted in Europe have shown that OGFCs require 25 to 50% more salt than regular HMA, although some researchers report higher increases. Salt is generally placed on pavement at a rate of 0.02 to 0.04 lb/yd^2. Immediately after plowing, the application rate must be raised to about 0.06 lb/yd^2. In Italy, researchers have found that reducing the maximum aggregate size from ¾ to ⅝ inches significantly improved winter weather surface conditions (Litzka 2002). If too much rock salt is placed on roads, conditions can become slippery due to excess salt (Van Doorn 2002).

Anti-icing techniques, which are different than deicing techniques, are also more difficult on an OGFC pavement. Anti-icing is preventing ice or snow from forming on the road surface while deicing is attempting to remove ice or snow already on the road surface. About 30% more anti-icing materials must be used on an OGFC than on a typical asphalt pavement. If anti-icing measures are performed too late, ice or snow will be compacted down into the void structure of an OGFC creating a frozen layer in the pavement (Giuliani 2002). Liquid magnesium chloride has been used as an anti-icing agent effectively by the New Jersey Garden State Parkway (NJGSP) to prevent ice buildup in an OGFC (Bennert et al. 2005).

OGFC Mixtures Used for Maintenance Applications

In 2010, the SCDOT published a Supplemental Specification for the use of maintenance OGFC mixtures for use in limited patching applications. This supplemental specification covers the mixture composition and construction guidelines. The composition of the maintenance OGFC mixture includes the use of crushed stone meeting the gradation requirements listed in Table 8.1. Because this mixture type is to be produced in relatively small quantities for the purpose of limited patching, the binder is specified to meet a PG 64-22 binder instead of a polymer modified binder having a grade of PG 76-22. Additionally, the use of fibers is not required, but hydrated lime (1% by aggregate weight) is required to prevent stripping. The optimum binder content (OBC) is determined using the visual draindown procedure outlined in SC-T-91 (SCDOT 2010), but the binder content is to be between 5 and 6% of the total mixture weight. The production temperature is also lowered to minimize draindown with the absence of stabilizing fibers or polymer modifiers in the mix.

Table 8.1. Gradation requirements for SCDOT maintenance OGFC mixture (SCDOT 2010)

Sieve Size	Percent Passing
3/4 inch (19.0 mm)	100
1/2 inch (12.5 mm)	95 – 100
3/8 inch (9.5 mm)	80 – 100
No. 4 (4.75 mm)	20 – 50
No. 8 (2.36 mm)	5 – 20
No. 200 (0.075 mm)	0 – 3

As part of this study, maintenance OGFC mixtures meeting the SCDOT requirements were evaluated in the laboratory. The objective was to determine the performance properties (volumetrics, permeability, and abrasion resistance) of these mixtures.

Experimental Materials and Methods

Mixtures were prepared using three different aggregate gradations summarized in Table 8.2 and Figure 8.2 (one conventional OGFC gradation and two meeting the maintenance gradation requirements) and two different aggregate sources (A and C from Chapter 4). The binders used in this evaluation were the PG 64-22 and PG 76-22 binders previously described in Chapter 4. Hydrated lime was also included in each mixture at a rate of 1% by weight of aggregate.

Table 8.2. Maintenance OGFC gradations evaluated

Sieve Size	Percent Passing		
	Gradation C	**Gradation M**	**Gradation N**
¾ inch (19.0 mm)	100	100	100
½ inch (12.5 mm)	95.4	100	97.6
⅜ inch (9.5 mm)	72.4	99.8	85.8
No. 4 (4.75 mm)	20.2	36.4	28.1
No. 8 (2.36 mm)	4.4	6.8	5.5
No. 200 (0.075 mm)	1.3	1.3	1.3

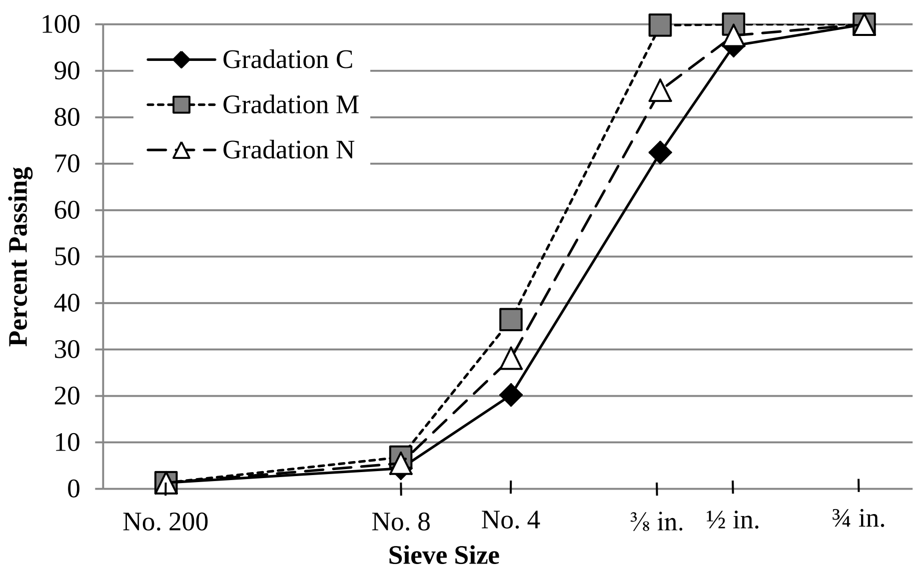


Figure 8.2. Gradation curves of OGFC mixtures evaluated for maintenance applications

Draindown curves were determined for each mixture by measuring the draindown at different binder contents. These results were then plotted to visualize the influence of binder content on the draindown. These curves also provide an indication of how much binder can be added to a mixture while still meeting the maximum draindown limits (typically 0.3% of total mixture weight). As with other portions of this study, a modified AASHTO T305 procedure was used to measure draindown. Draindown testing was performed at the mixing temperature for each binder (325°F for the PG 64-22 and 332°F for the PG 76-22).

Following draindown testing, the OBC for each mixture was determined using the SC-T-91 procedure and four specimens were compacted with 50 gyrations of the Superpave gyratory compactor at the optimum binder content. The compacted specimens were tested to determine the porosity in accordance with ASTM D7063. Porosity was measured instead of air content because of the results presented in previous chapters and the fact that porosity is a better indicator of mixture functionality (i.e., permeability) as seen in Chapter 5. After the volumetric properties of each specimen were calculated, the permeability was measured as described in Chapter 5. Finally, the specimens from each mixture were divided into two groups of two for the Cantabro abrasion test. The two specimens having the highest and lowest air void content were tested in the unaged state and the remaining two specimens were conditioned (140^{o}F for 7 days) and tested in the aged state.

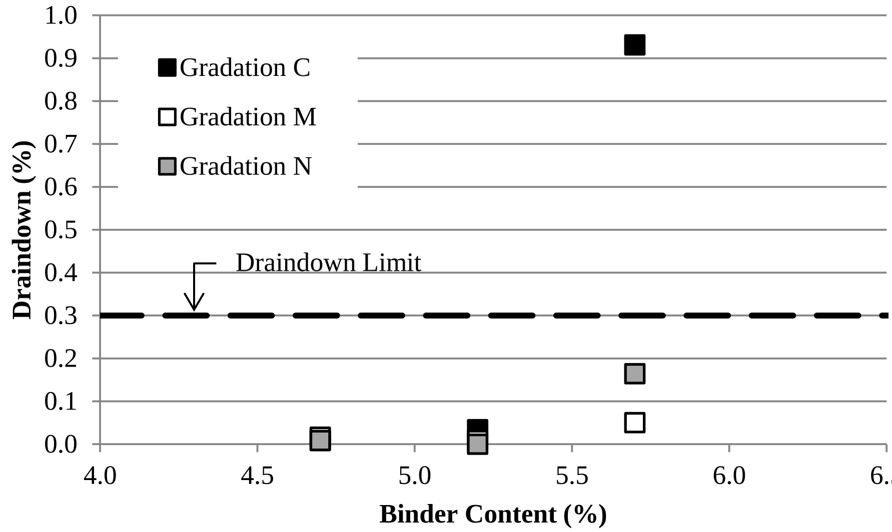
Results and Discussion

Results of the draindown testing are summarized in Figures 8.3 and 8.4 for PG 64-22 and PG 76-22 binders, respectively. The results indicate that the draindown is aggregate source dependent due to the different aggregate absorption values. Additionally, the maintenance gradations (M and N) generally exhibited less draindown than the regular OGFC gradation (gradation C) at the upper end of the evaluated range. This is due to the increased surface area of the finer gradations used for the maintenance mixtures. The draindown results indicate that the need for fibers in these maintenance mixtures is not necessary within reasonable binder content limits, even for mixtures made with PG 64-22 binders.

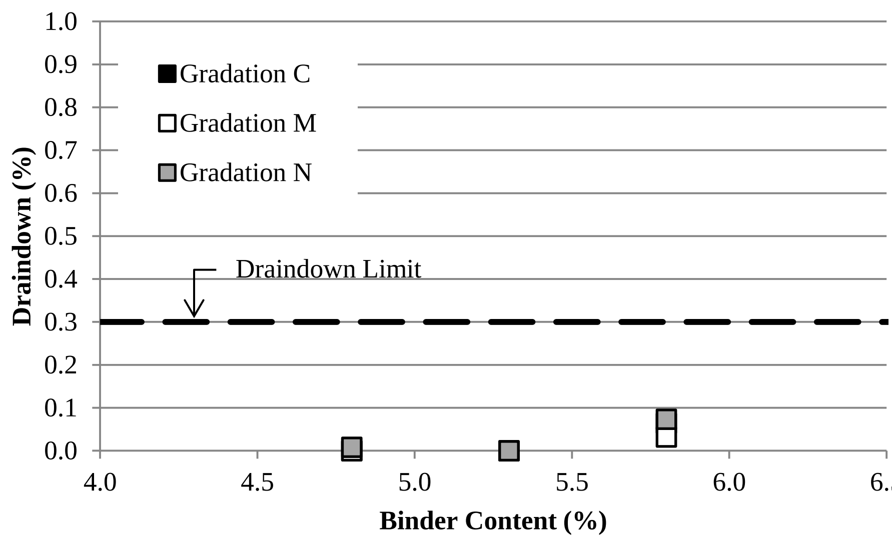
The optimum binder contents determined using the SC-T-91 procedure for each gradation, binder, and fiber combination are included in Table 8.3. The results indicate that the finer maintenance mixtures (M and N) have higher OBCs than reference mixture C. This is in agreement with draindown results previously discussed and is due to the increased surface area of the finer maintenance gradation specification. The addition of cellulose fibers at a rate of 0.3% of the total weight to the mixtures resulted in an increase in the binder content by 1% or more. It is interesting to note that the difference in binder content between the PG 64-22 and PG 76-22 was small for a given aggregate source and gradation.

Table 8.3. Optimum binder contents for different reference and maintenance OGFC mixtures.

		Optimum Binder Content, %			
		Aggregate A		**Aggregate C**	
	Gradation	**PG 64-22**	**PG 76-22**	**PG 64-22**	**PG 76-22**
Without Fibers	C	5.2	5.4	5.5	5.5
	M	5.4	5.5	5.6	5.7
	N	5.3	5.5	5.5	5.6
With Fibers	C	6.2	6.5	6.7	7.0
	M	6.8	6.8	7.0	6.9
	N	6.5	6.8	7.0	7.0

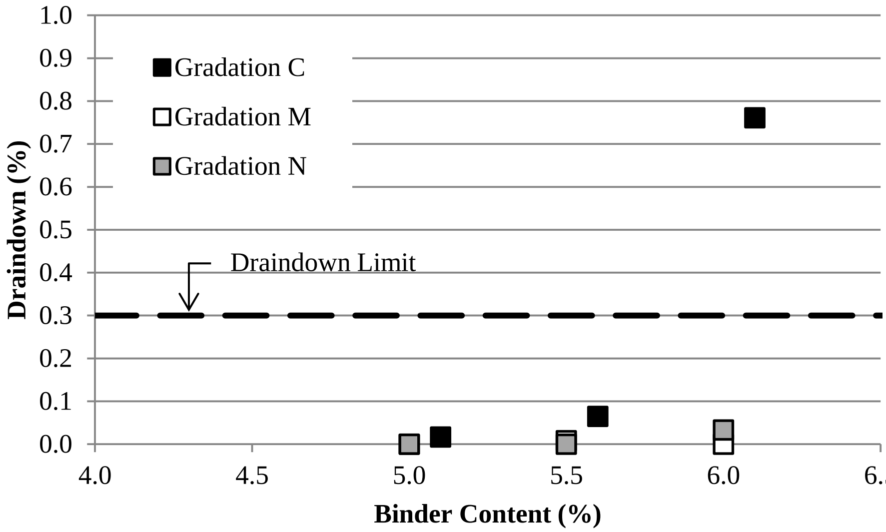


(a)

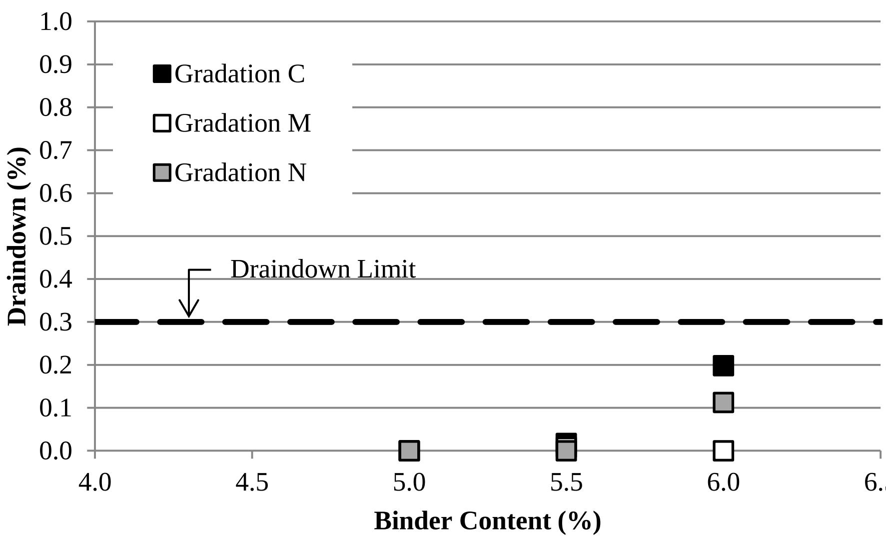


(b)

Figure 8.3. Draindown curves for maintenance OGFC mixtures made with PG 64-22 binder and without fibers for (a) aggregate source A and (b) aggregate source C.



(a)



(b)

Figure 8.4. Draindown curves for maintenance OGFC mixtures made with PG 76-22 binder and without fibers for (a) aggregate source A and (b) aggregate source C.

The average porosity of each mixture is summarized in Figure 8.5. For this evaluation, mixtures were made with and without fibers to quantify the impact that fibers may have on the mixtures. The results indicate that the removal of fibers from the mixtures significantly increased the mix porosity. The effects of fibers can further be seen in permeability results in Figure 8.6. In all cases, the mixtures without fibers exhibited substantially higher permeability values. In some cases the difference was two-fold.   Further analysis of Figures 8.5 and 8.6 indicate that the porosity and permeability of the three different gradations were comparable.

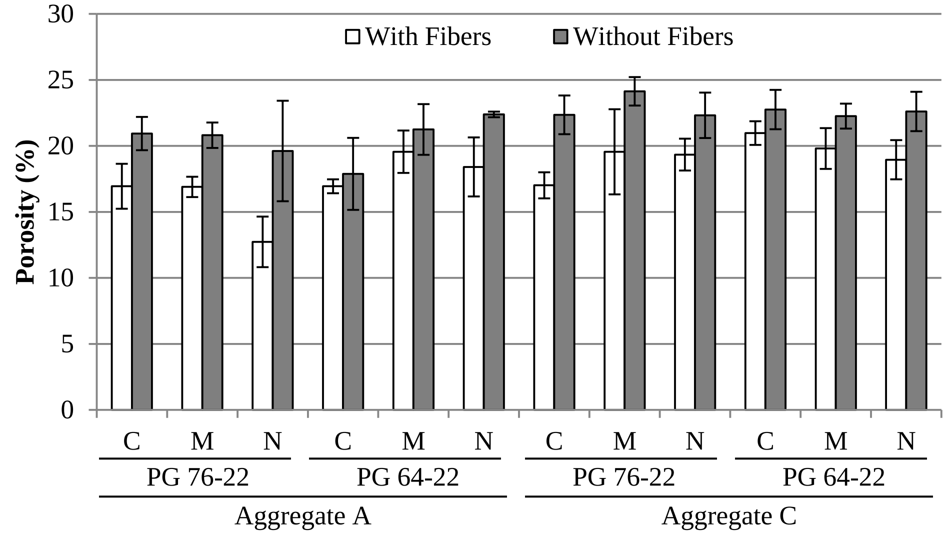


Figure 8.5. Average porosity of maintenance OGFC mixtures. The error bars represent one standard deviation from the average.

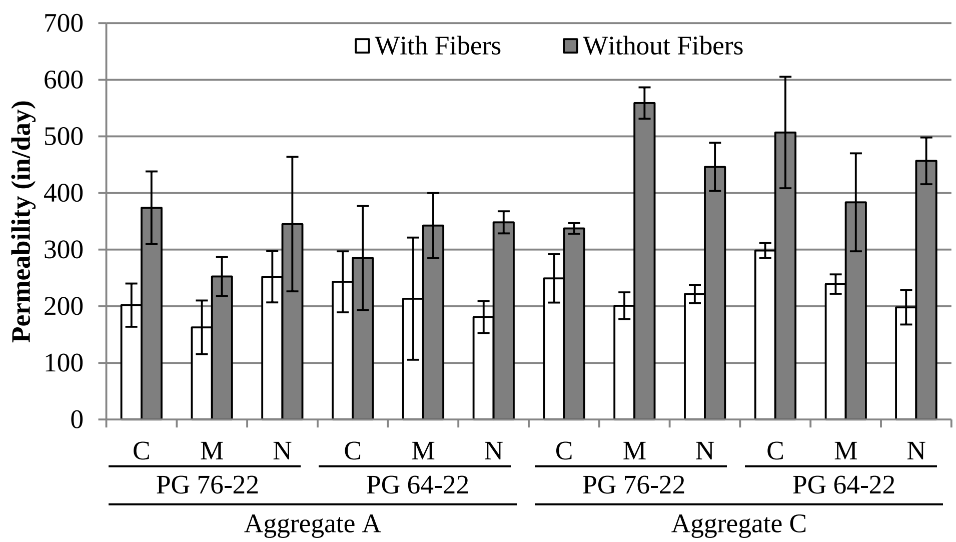


Figure 8.6. Average permeability of maintenance OGFC mixtures. The error bars represent one standard deviation from the average.

The results of the Cantabro abrasion test are summarized in Figures 8.7 and 8.8 for the different mixtures made with and without fibers, respectively. The results indicate that the mixtures made with PG 76-22 generally exhibit lower loss due to abrasion than the PG 64-22 mixes. This is expected due to the enhanced durability characteristics of polymer modified binders and the slightly higher binder contents. Additionally, the use fibers had a significant impact on the durability of the mixes as mixes without fibers exhibited higher abrasion loss than those with fibers. This is also to be expected because the incorporation of fibers increases the binder content and thus the film-thickness surrounding the aggregate particles, which enhances durability. Finally, the gradation did not have a significant impact on the durability measured by the Cantabro abrasion test.

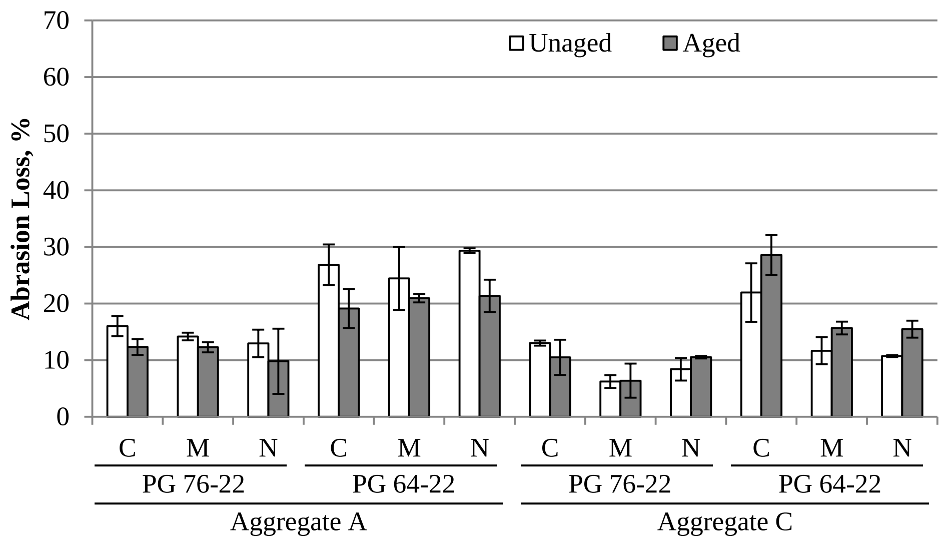


Figure 8.7. Average abrasion loss of maintenance OGFC mixtures made with fibers. The error bars represent the range of the experimental values.

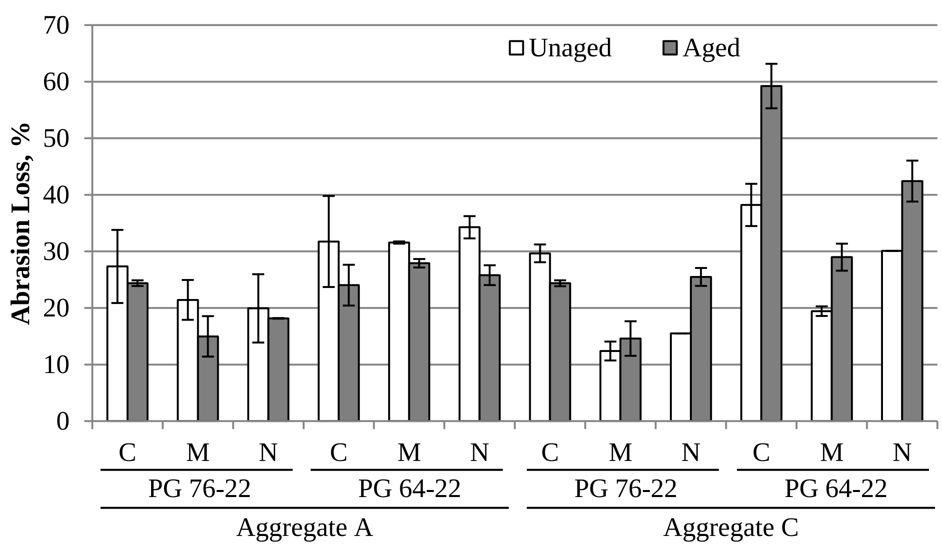


Figure 8.8. Average abrasion loss of maintenance OGFC mixtures made without fibers. The error bars represent the range of the experimental values.

Conclusions

Based on the review of maintenance experience by others, it is evident that maintenance of OGFCs must be considered due to the fact that these mixtures have historically deteriorated prematurely due to raveling and have had their functionality decreased due to clogging. To address the topic of OGFC raveling, the main question is whether it is best to patch the areas, apply fog seals, or do nothing. Patching with a dense graded mixture could inhibit the flow of water depending on the location, which could lead to other issues including stripping and safety concerns. Fog seals could reduce, or eliminate the permeability of the pavement. The do nothing approach will result in a rough pavement.

When patching OGFC pavements with dense graded asphalt, it is important to study the location of the patch so that water will be able to flow around the patched area to the daylighted pavement edge. This would require patches to be located on a pavement grade that will enable water flow. It is not recommended to place patches at the bottom of a vertical curve because water will not be able to flow around the patch.

Alternatively, OGFC pavements can be patched using a maintenance type OGFC as evaluated in this study. This type of patch would be especially attractive when the pavement has a substantial amount of life remaining. While the standard OGFC mixtures have greater performance characteristics with respect to durability, maintenance OGFC mixtures could be produced without the use of polymer modified binders or fibers, which would make them more practical to produce in small quantities. However, attention must be paid to proper selection of optimum binder content especially with regard to draindown and permeability.

Another maintenance topic to consider with OGFCs is winter maintenance. It has been shown that OGFC pavements do freeze faster than dense graded asphalt pavements. As such, maintenance needs to be performed to combat ice formation. This typically requires the use of deicers. When using deicers on OGFC pavements, it will be necessary to increase the application rate to account for the permeability of the pavement surface as deicing salts and chemicals will eventually migrate into the pore structure of the OGFC layer and not remain on the pavement surface. Additionally, it is important that deicing salts not be mixed with sand as the sand will eventually clog the OGFC layer.

Finally, clogging of an OGFC layer will limit and could eventually eliminate the permeability of the pavement. While a clogged pavement will still be serviceable as a wearing course, its functionality and safety benefits can be greatly diminished. To reduce the onset of clogging, it is recommended that OGFC layers not be located in areas prone to sediment deposition on the pavement. In addition it is recommended that the permeability of the pavement layer be routinely assessed and that proactive actions be taken to restore permeability before a pavement becomes clogged beyond restoration. Declogging activities that have been used include high-pressure application of water to the pavement surface to flush out sediment particles and vacuuming the pavement surface with specialty vacuum trucks, among others. It has also been found that high speed traffic limits the degree of sediment deposition in OGFC voids, which can help to maintain the permeability of the driving lanes, but could lead to clogging of the shoulders. If the shoulders become clogged, then water could build up within the travel lanes and not be able to exit. This suggests that routine shoulder maintenance may be more important than travel lane maintenance, which could simplify the maintenance activities.

CHAPTER 9: WARM MIX OGFC

The production of hot mix asphalt (HMA) has long involved the combination of petroleum based asphalt binder and mineral aggregate. Recently, the asphalt paving industry has seen the need to develop more sustainable pavements and is making efforts to cut costs, reduce emissions, and recycle reclaimed asphalt pavement (RAP) into new pavements (Copeland et al. 2010). These trends are now becoming the industry standard with hopes of reducing the need for virgin binder and aggregate, both nonrenewable resources. One of the technologies that has grown out of the need for more sustainable construction is warm mix asphalt (WMA).

The objective of WMA is to produce and construct asphalt pavements at lower temperatures (up to 100^{o}F lower) than conventional hot mix. Some technologies such as MeadWestvaco's (MWV) Evotherm™ directly alter binder properties with the use of a carefully selected chemistry, while foaming technologies employ water to create steam in the binder which provides the desired change in binder properties (Hurley and Prowell 2006; Wielinski et al. 2009). This alteration in binder properties is the primary goal of any warm mix technology and allows for warm mix asphalt to be mixed at much lower temperatures with results similar to HMA paving.

In addition to the environmental benefits of WMA reported by many (Vaitkus et al. 2009; Prowell and Hurley 2007), there are also other benefits of WMA that could benefit OGFC mixtures. Some time related benefits provided by warm mix technologies include the potential for longer haul distances and less time required after paving before opening a road to traffic (Vaitkus et al. 2009; Prowell and Hurley 2007). Another advantage of WMA is that it reduces binder aging during production, which can potentially reduce cracking and early degradation (Hurley and Prowell 2006).

In recent years, warm mix asphalt technologies have been tested in limited performance trials with OGFC indicating promise (Barrows and Dmytrow 2009; Jones et al. 2010). There is potential that warm mix technologies can help improve OGFCs due to reduced binder aging and hardening during the mixing process because of lower production temperatures. With this hypothesis, the reduction in binder aging could inhibit raveling, which is so debilitating to OGFCs proving WMA OGFCs more durable than traditional OGFCs. In addition to reduced binder aging, some WMA technologies have the ability to increase the binder film thickness around the individual aggregate particles. This was realized with research conducted on stone matrix asphalt (SMA) mixtures, where a mix produced with the warm mix additive Evotherm™ and no fibers resulted in fatigue life that was approximately 900% greater than the HMA SMA mixture with fibers (Bennert 2011).

There are currently a number of different warm mix technologies on the market, but they all work for the common goal of providing asphalt binder properties at lower temperatures to match the same binder properties of typical hot mix asphalt binder. There are four basic types of WMA technologies: Chemical packages, water-bearing agents, waxes, and water-injection methods.

With WMA technology and OGFC becoming increasingly popular, the combination of the two could provide a new, successful market for both. The potential for the expansion in the use of OGFCs is dependent on whether or not the most common problems with this mix can be mitigated by WMA: Raveling and draindown. The current solution to OGFC draindown and stability issues is fibers. Although the introduction of fibers has been relatively successful in addressing mix draindown and stability, asphalt plant personnel have indicated that fibers can be a production hassle. The incorporation of fibers into a mix raises the cost (material and labor) and potentially, the variability of a mix as the introduction of fibers into a mix can be difficult to monitor and control at times (Cooley et al. 2009). Due to the aforementioned properties of WMA, there is the potential that raveling and draindown could be decreased dramatically with the use of warm mix OGFC. If WMA technologies can provide this desired effect, contractors could potentially eliminate the need for fibers as stabilizing additives.

Research Objectives

The primary objective of this portion of the study was to evaluate the feasibility of using WMA technologies (Evotherm™ and Foaming) to produce quality OGFC mixtures without the need for stabilizing fibers. This evaluation was based on the comparison of Evotherm™ WMA and foamed WMA mixes with traditional HMA OGFC using three main criteria: Draindown, permeability, and abrasion resistance.

Experimental Materials and Methods

To realize the objectives of this study, the evaluation included five different mix designs (2 HMA, 2 Evotherm™ WMA, and 1 foamed WMA). The primary component of each mix design that was varied was the inclusion of fibers. Initial testing was completed to characterize the mixes based on binder draindown as draindown curves were developed for each mixture. Following the draindown evaluation, the optimum binder content of each mix was determined and specimens were made to test the volumetric and performance properties of each mix, specifically permeability and abrasion resistance.

Materials

For this study, one crushed granite aggregate source was used (Aggregate C from Chapter 4). The aggregate gradation was designed to meet the SCDOT requirements for OGFC (Table 9.1) (SCDOT 2007). This specific gradation has been used for OGFC within the state of South Carolina and is typical for OGFC gradations found around the nation (Kandhal 2002; Cooley et al. 2009).

Table 9.1. OGFC gradation used for WMA evaluation

Sieve Size	Percent Passing
¾-inch (19.0 mm)	100
½-inch (12.5 mm)	93
⅜-inch (9.5 mm)	68
No. 4 (4.75 mm)	20
No. 8 (2.36 mm)	7
No. 200 (0.075 mm)	2

The binder used for this project was a PG 76-22 SBS modified binder as described in Chapter 4 (Table 4.7). PG 76-22 binder is commonly used in OGFC applications across the nation because of its high resistance to permanent deformation as well as its tendency to reduce draindown (Cooley et al. 2009).

As in Chapters 4 and 5, hydrated lime and cellulose fibers were used in this evaluation. When fibers were included in a mixture, they were added at a rate of 0.3% by weight of the entire mix. Hydrated lime was added at 1% by weight of the aggregate for each mix. In this project, the Evotherm™ 3G WMA technology already contains a liquid anti-stripping additive; however, the hydrated lime was still used in all mixes within this study to maintain consistent parameters.

Two WMA technologies were evaluated: Evotherm™ and foaming. When the Evotherm™ was added to the asphalt binder, the manufacturer recommendations were followed for proper incorporation of the additive to the mixture. The binder was first heated to the target mixing temperature of 285°F (141°C). The Evotherm™ 3G additive was then added to the binder at a rate of 0.5% by weight. The WMA binder was then stirred for 1-2 minutes before being placed back in the oven at the mixing temperature for 30 minutes. Once mixed, the binder was added to the heated aggregate and mixed in a mechanical bucket mixer in the same manner as the regular HMA samples.

For the foaming WMA technology, water was injected into the hot asphalt binder at 2% by weight of the asphalt binder by "The Foamer" produced by Pavement Technology, Inc. The binder used for the mix was heated and placed into "The Foamer" at the HMA mix temperature 339°F (171°C) before water was injected into the binder and the foamed binder was emitted and mixed with the hot aggregate at the WMA mix temperature 285°F (141°C). This mixing of the water instigated the foaming action creating the WMA. This foamed WMA binder was then added to the heated aggregate and mixed in a mechanical bucket mixer in the same manner as the regular HMA samples.

Experimental Methods

Draindown testing was performed for all the mixes in accordance with AASHTO T305 with the exception that only the mixing temperature was evaluated. This testing consisted of measuring the binder lost from the mix placed in a draindown basket (No. 4 mesh) and conditioned at the mixing temperature (339°F for HMA and 285°F for WMA) for 1 hour. Two draindown specimens were tested per binder content over a binder content range from 5.0 to 7.5%. This testing provided the rate of binder draindown relative to the binder content of the mix. This test has been shown to be effective in determining the stabilizing capacity of fibers in draindown prone mixes such as OGFC (Putman and Amirkhanian 2004).

Following the draindown testing, the optimum binder content (OBC) of each mix was determined in accordance with the SCDOT procedure for designing OGFC mixtures, SC-T-91 (SCDOT 2010).

Once the optimum binder content for each mix was determined, the moisture susceptibility of each mix was evaluated. The SC-T-69 procedure was used to test the moisture susceptibility of each mix design at the OBC (SCDOT 2010). This test procedure consists of placing a loose asphalt sample (300g) into a beaker of boiling water for 10 minutes before removing the sample and visually determining the percent stripping. Two specimens were tested at the optimum binder content for each mix design.

The temperature reduction for Evotherm™ as recommended by the manufacturer was approximately 54°F (30°C). This reduction was applied to the draindown test temperature as well as the mixing and compaction temperature ranges for making the other specimens. The same temperature reduction was used for both WMA technologies (Evotherm™ 3G and Foaming) to maintain consistent and comparable research parameters. With this reduction, the mixing temperature for the WMA mixtures was 285°F.

After determining the OBC and evaluating the stripping potential, nine compacted specimens were made for each mix design at the OBC for further testing to measure the performance properties (permeability and abrasion resistance). The specimens were 150 mm diameter by 115±5 mm tall and were compacted using a Superpave gyratory compactor at 50 gyrations. Once compacted, the specimens were allowed to remain in the mold to cool in front of a fan for approximately 15 minutes. This cooling period prevented the specimens from falling apart or becoming distorted due to gravity. After a specimen was removed from a mold, it was removed from the compaction area and moved to a cooling station.

All of the compacted specimens were tested for specific gravity and porosity per ASTM D7063 and then the volumetrics (air voids, VMA, VFA) were calculated for each specimen using the maximum theoretical specific gravity measured in accordance with AASHTO T219. Once this initial testing was completed, the nine specimens from each mix design were divided into three groups of three specimens per group for the performance testing (3 for permeability, 3 for unaged Cantabro abrasion, and 3 for aged Cantabro abrasion). The porosity data was used to group the specimens to ensure that each group was representative of the overall mix design properties. Lastly, to verify that the three test groups were similar with respect to porosity, an analysis of variance (ANOVA) was performed using $\alpha = 0.05$.

After completing the volumetric testing, six of the nine specimens were tested for Cantabro abrasion (3 unaged and 3 aged for 7 days at 140°F) as outlined in ASTM D7064. The remaining three specimens were tested for permeability. The permeability was measured using the falling head apparatus described in Chapter 5.

Once tested for initial permeability, the permeability of each specimen was measured after a series of aging cycles. The specimens were aged in a 140°C chamber and tested for permeability after 3, 6, 9, and 14 days of aging. The effect of long-term aging on the OGFC specimens was evaluated because the removal of fibers from the mixture could potentially result in binder draindown over time which could reduce the permeability, thus reducing the effectiveness of the mixture in draining water.

Statistical Analysis

Statistical analyses were performed on the experimental data to determine the statistical differences between the different mixtures with respect to the volumetric and performance properties. The results are included in the respective figures through the use of letters, which indicate similarities between the various mix designs within a specific property and were determined using Fisher's test for least significant difference (LSD). Mix designs that have the same letter indicate similarity for a particular property. Some mix designs have more than one letter indicating similarity with more than one other mix design group. All of the analyses were conducted with a 95% level of significance ($\alpha = 0.05$).

Results and Discussion

The OGFC mix designs completed and tested in this study included five different mix designs (2 HMA, 2 Evotherm™ WMA, and 1 foamed WMA). The primary variables were the inclusion of fibers (with fibers and without fibers) and WMA technology. To begin with, uncompacted specimens were tested for maximum specific gravity, draindown, and optimum binder content (OBC) determination. Then compacted specimens were produced for specific gravity, porosity, permeability, and Cantabro abrasion testing. These results were then analyzed to determine the effect of fibers and warm mix on the mix properties.

Draindown Testing

Uncompacted specimens were tested for draindown for each of the mix designs in accordance with AASHTO T305. For the majority of agencies, the most commonly accepted maximum limit for binder draindown is 0.3% of total mix weight. The draindown curves produced in this study can be seen in Figure 9.1. The initial hypothesis of the study was that the WMA technologies might alleviate excessive draindown and, therefore, eliminate the need for fibers in OGFC mixes. While the draindown curves of the HMA and WMA mixtures including fiber were fairly similar (Figure 9.1(a)), the most significant reduction in draindown using the WMA technologies can be seen in the mixtures that do not contain fibers (Figure 9.1(b)). In these mixes, the Evotherm™ and foamed mixes performed similarly to each other and only exhibited draindown above the 0.3% limit after the binder content reached approximately 7.2%. It should be noted that this binder content is at the high end of typical HMA OGFC mixtures containing fibers. In contrast, the HMA mix without fibers exhibited draindown above the 0.3% limit at approximately 6.2% binder. Such a reduction in draindown in WMA mixes without fibers could potentially lead to the elimination of fibers in OGFC mixes as the primary purpose of including fibers in these mixes is to limit draindown. Additionally, it should be noted that different test temperatures were used in the determination of draindown for the HMA mixtures compared to the WMA mixtures. While the test temperatures differed by 54°F for the WMA mixtures compared to the HMA mixtures, the comparison is valid because the WMA mixtures will be produced at a mixing temperature that is 54°F lower than that of the HMA.

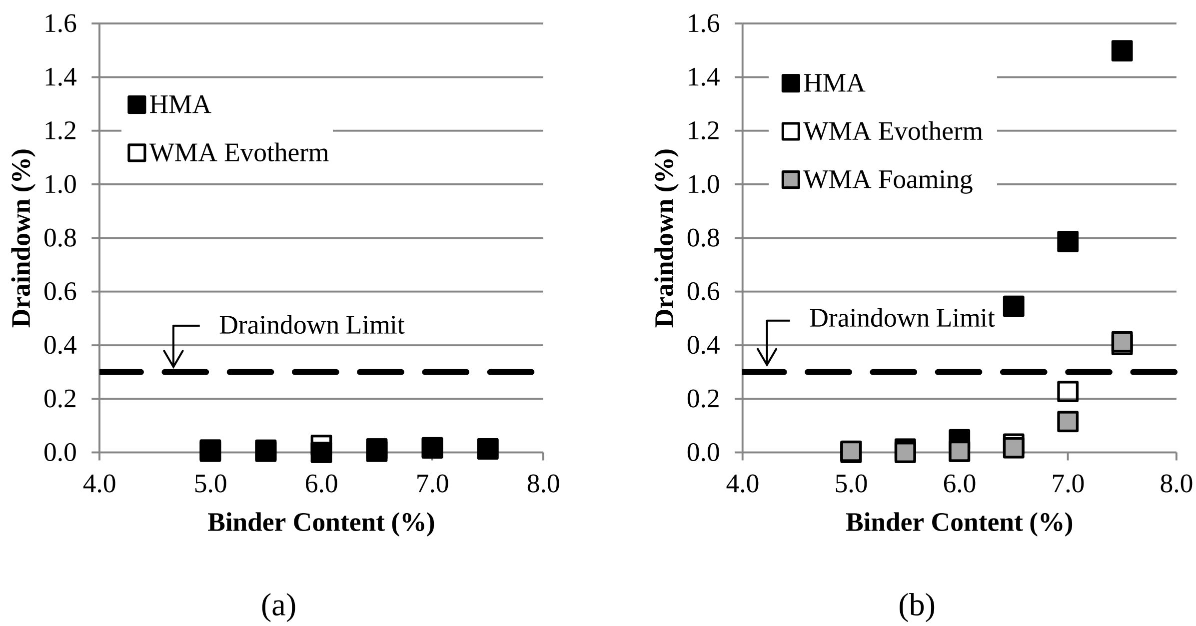


Figure 9.1. Draindown results for HMA and WMA OGFC mixtures (a) with cellulose fibers and (b) without fibers.

Optimum Binder Content Determination

To determine the optimum binder contents, the visual draindown method was used in accordance with SC-T-91 (SCDOT 2010). The OBCs of all the mix designs ranged from 5.0% to 7.5% and can be seen in and in Figure 9.2. However, for the mixes without fibers all the OBCs were between 5.0% and 6.0%. It can be noted from Figure 9.2, that the OBCs for the WMA mixes were greater than the equivalent HMA mixture without fiber, which indicates a thicker binder film compared to the HMA mixes. Asphalt producers typically have increased the binder content and film thickness of OGFC mixes by adding additives such as polymers to the binder and fibers in the mix design. However, as evidenced by this study, WMA technologies have the ability provide this same benefit without the fibers and without increasing binder draindown. For this reason and the fact that the mixes with fibers had similar draindown curves and OBCs, fibers were only included in the HMA mix for further comparison and not in any of the WMA mixtures.

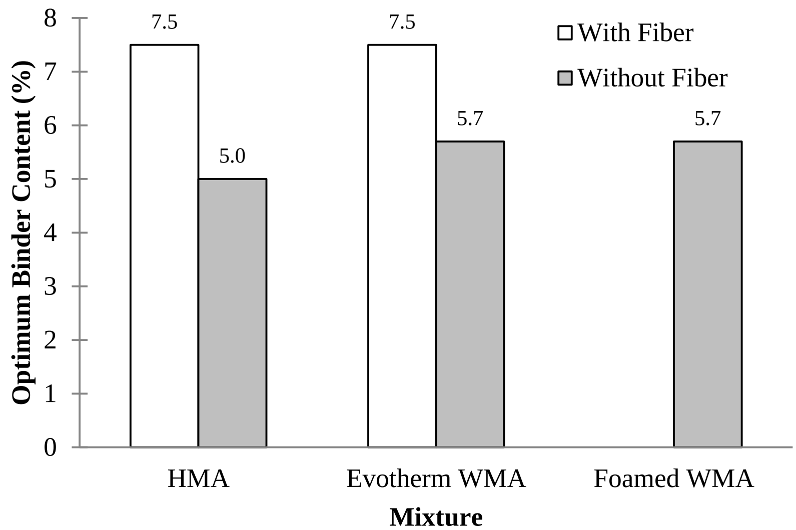


Figure 9.2. Optimum binder contents for HMA and WMA OGFC mixtures with and without fibers.

As seen in the binder draindown curves (Figure 9.1), none of the mix designs exhibited significant draindown at the respective OBC. These curves suggest that the binder contents of WMA mixes could be increased by more than 0.5% without exceeding the maximum draindown limit. By increasing the binder contents, the WMA mixes without fibers would then have nearly the same amount of binder as typical HMA OGFC mixes with fibers. This advantage would then most likely be realized in an increase in the durability of the mixtures. For this reason, properties of mixtures without fiber were further evaluated at the OBC and at a binder content of 0.5% greater than the OBC.

Moisture Susceptibility Testing

Uncompacted specimens were tested for moisture sensitivity using the boil test outlined in SC-T-69 (SCDOT 2010). Although moisture susceptibility is thought to be a possible weakness for some WMA technologies, all mixes in this study performed well under this test, showing no noticeable evidence of stripping. This result was expected as the aggregate source used in this study historically performs well with regard to stripping and hydrated lime was also added as an anti-stripping additive to each mix at a rate of 1%.

Volumetric Properties

The volumetric properties of the specimens were determined based on the maximum specific gravity and bulk specific gravity test results. Figure 9.3 summarizes the voids in total mix (VTM), or air voids and porosity of each mix. Air voids and porosity are important properties of any OGFC mix design, as these properties are indicative of the permeability (or functionality) of a mix.

Figure 9.3 shows that each mixture had similar total air void content, with the exception that the foamed WMA mix had a higher overall void content compared to the other mixtures. The mix with fibers, however, had a significantly lower porosity, which was caused by the fiber/binder mastic clogging void channels thus preventing water from accessing all of the existing voids. Since these mixes are representative of OGFC mix designs used by SCDOT, this is a noteworthy difference which should be expected as two contributing effects of adding cellulose fibers to a mix are reduced air voids and increased binder content. Meanwhile, when the binder contents of the WMA mixtures without fibers were increased by 0.5% above the OBC, there was no significant change in the porosity, although the total air void content of the HMA mixture decreased. It should also be noted that the overall similarity in the data is due to the fact that all mix designs had the same aggregate gradation and were tested at OBC, or 0.5% above OBC.

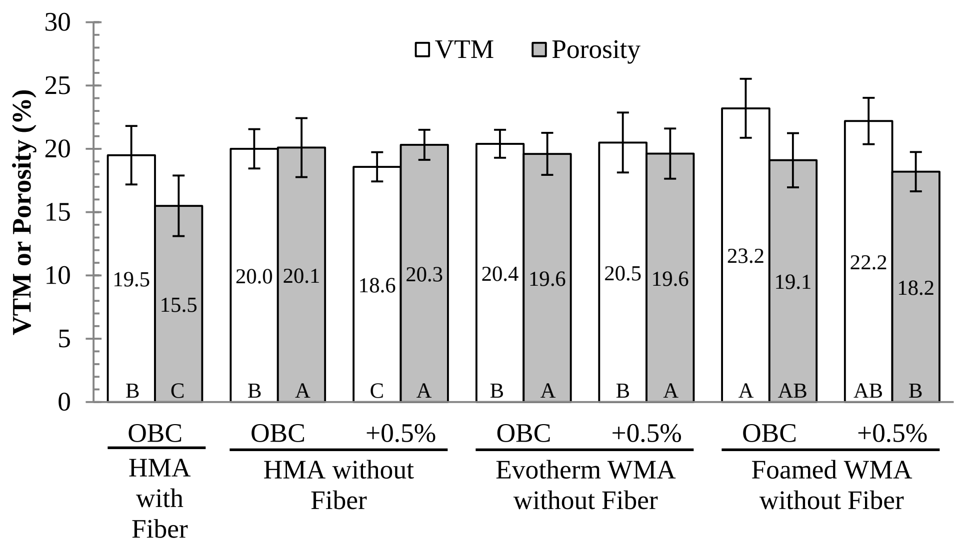


Figure 9.3. Average air voids and porosity for the OBC mixtures at the OBC and 0.5% above OBC.

Permeability

The permeability results of each of the OGFC mixtures are summarized in Figure 9.4. The results indicate that the inclusion of fibers in the mix significantly reduced the permeability compared the mixtures without fibers. Additionally, increasing the binder content of the mixtures without fibers by 0.5% above the OBC did not have a significant effect on the permeability of the mixtures. Finally, while the WMA mixes generally had higher permeability, the differences were not statistically different from the HMA mix without fiber. It should be noted that the permeability of all of the mixes tested far exceed the minimum recommendation of 164 in/hr (ASTM 2010; Kandhal 2002).

After the initial permeability was measured, the specimens were conditioned at 140^{o}F to simulate long-term aging that the mixtures would experience during their service life. The permeability of each specimen was measured after 3, 6, 9, and 14 days of aging to determine if the permeability would reduce over time due to binder draindown. The results indicate that over the conditioning regime adopted in this study, the permeability did not decrease for any mixture. On the contrary, the permeability actually slightly increased over the high-temperature conditioning period, although the increase was not statistically significant for any mixture.

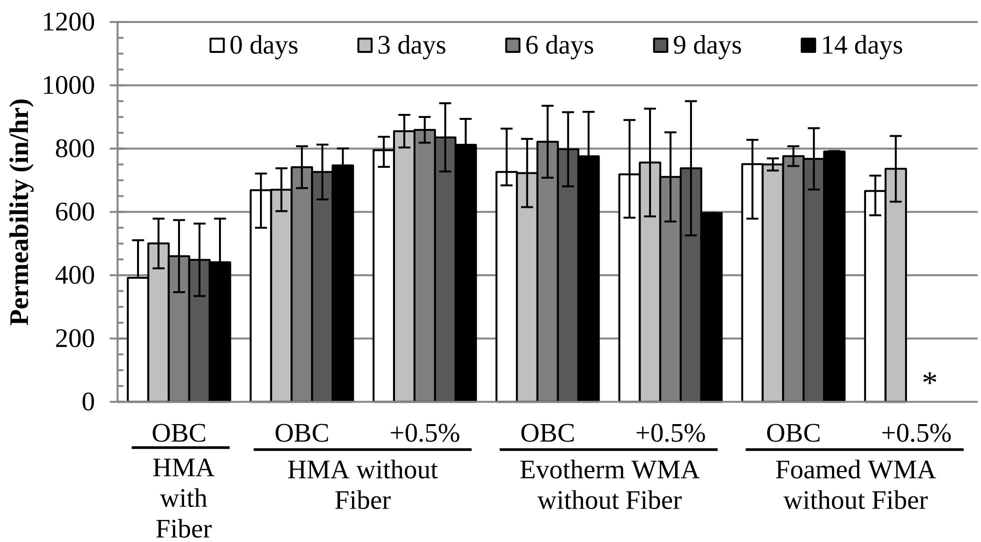


*Figure 9.4. Average permeability of the OFGC mixes over the 14 day conditioning period at 140^{o}F. * indicates that specimens did not survive the conditioning regime.*

To help understand the cause of the difference in permeability from the beginning of the conditioning regime to the end of the conditioning duration, the porosity of each specimen was measured after the 14 day permeability test. The results indicated that the porosity of the specimens generally increased as a result of the conditioning. While the cause of the change in porosity was not part of the scope of this study, it is speculated that one potential cause could be volatilization of the light fractions of the binder may have reduced the thickness of the binder film, thus increasing the volume of the pores within a specimen.

An additional finding from this evaluation of long-term conditioning under high temperatures was that some of the specimens did not survive the entire 14 day conditioning duration. After 3 days of conditioning, one of the Foamed WMA (OBC) specimens collapsed and could not be tested; after 6 days, all three of the Foamed WMA (OBC+0.5%) specimens collapsed; and after 14 days, one of the WMA Evotherm (OBC+0.5%) specimens collapsed. All of the specimens that collapsed due to the heat did not include fiber. This is something that should be investigated further, but this is not necessarily indicative of field performance.

Abrasion Resistance

The abrasion resistance of the OGFC mixtures in this study was evaluated using the Cantabro abrasion test. The unaged and aged abrasion loss results are summarized in Figure 9.5. In nearly every mix design, the aged specimens generally outperformed the unaged specimens; however the difference was only significant for the HMA mix without fibers and the Foamed WMA mixes without fibers. This trend was also observed in Chapters 4 and 5, but was somewhat unexpected since the binder oxidizes during aging becoming stiffer and more brittle. It is speculated that this stiffening was the characteristic of the binder aging that affected the test the most, having a much larger impact than the increased brittleness of the binder. This concept was seen in a study performed by Mo et al. who found that binder oxidation actually improved the abrasion resistance of a sample during warm weather conditions but dramatically decreased the abrasion resistance in cold weather as the elasticity of the binder is compromised (Mo et al. 2010).

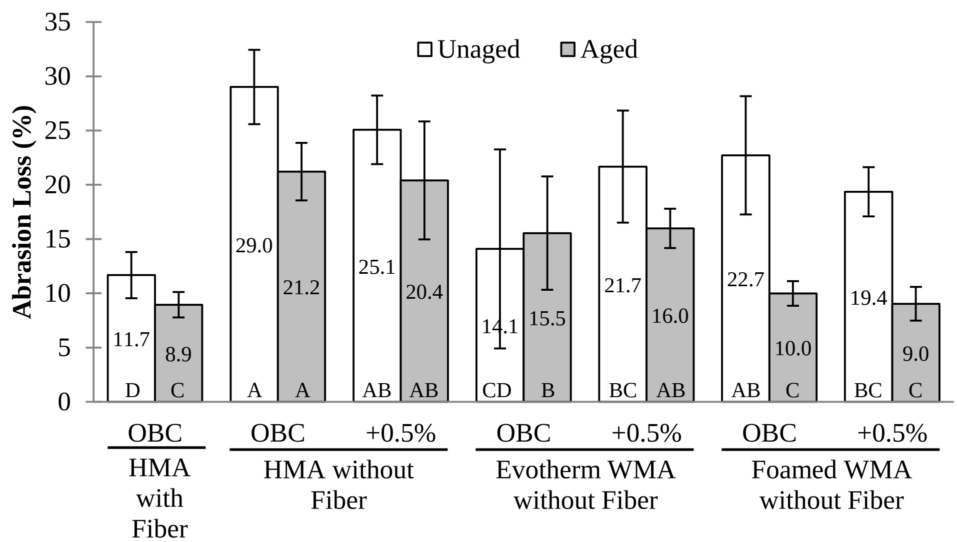


Figure 9.5. Abrasion loss results of the mix designs.

Based on the statistical analysis, after aging, both foamed WMA mixtures (OBC and OBC+0.5%) without fibers performed similarly to the HMA with fiber in the Cantabro abrasion test. Additionally, it can be seen that the WMA mixtures without fibers generally exhibited better abrasion resistance compared to the HMA mixture without fibers. Finally, it should be noted that all of the mixtures met the maximum aged abrasion loss of 30% set by specifications and guidelines used by many agencies as presented in Chapter 4.

Conclusions

The main objective of this portion of the study was to evaluate the feasibility of using WMA technologies to produce quality OGFC mixtures without the need for stabilizing fibers. The study focused on the comparison of Evotherm™ WMA and foamed WMA mixes with traditional HMA OGFC using three main criteria: draindown, permeability, and abrasion resistance. The results of this limited laboratory study indicate that OGFC mixtures made with WMA technologies and without fibers outperformed HMA OGFC mixtures that also did not have fibers with respect to draindown and durability, as long as warm mix temperatures were used for mixtures containing the WMA technologies. Additionally, the permeability was similar to the HMA mixtures without fibers. When compared with HMA OGFC mixtures containing fibers, the WMA mixes without fibers had significantly higher permeability due to the increased mixture porosity. In all cases, the WMA mixtures without fibers exhibited similar or greater durability compared to the HMA mixtures without fibers after long-term aging. Some mixtures also showed similar abrasion resistance to the HMA mixtures that included fibers. With respect to the unaged durability results, the HMA with fibers outperformed all but one of the WMA mixtures without fiber (Evotherm™ OBC).

The binder content of WMA OGFC mixtures without fibers can potentially be increased by at least 0.5% without sacrificing mixture performance with respect to increased draindown or reduced permeability. The draindown results from this study showed that HMA mixtures were more sensitive to this binder content "bumping" without fibers. By increasing the binder contents, within reason, the effect on mixture durability could be positive, but further study is needed to validate this.

The removal of fibers from OGFC mixtures could possibly help contractors produce a more consistent finished pavement, potentially without an increase in cost. However, this conclusion must be validated with field trials. In this study, the permeability nearly doubled when the fibers were removed. This could translate to a safer driving surface as rainwater can be drained more quickly. Additionally, the abrasion resistance did not significantly decrease when the fibers were removed and foaming WMA technology was used to produce the OGFC mixture at WMA temperatures. The durability did decrease for the HMA and Evotherm™ WMA mixes, but the aged abrasion loss was still lower than the maximum recommended value of 30%. This could be due to the increase in the thickness of the binder film for the WMA mixtures compared to the HMA mixture without fibers coupled with the reduced binder oxidation that occurs at the lower WMA mixing and compaction temperatures. This should be evaluated further.

CHAPTER 10: SUMMARY OF FINDINGS, CONCLUSIONS, AND RECOMMENDATIONS

The primary objective of this study was to identify methods to improve the design, performance, construction, and maintenance of open graded friction courses (OGFC) in South Carolina. To accomplish this objective, several tasks were completed to gain as much information about OGFCs. An extensive literature review on all facets of OGFCs was conducted to learn about the state-of-the-practice with respect to OGFC on a national and international scale. The performance of OGFC in South Carolina and other states and countries was surveyed to identify recurring problems with this type of pavement. A laboratory study was conducted to compare different OGFC mix design procedures employed by state DOTs in the US. The influence of aggregate gradation on OGFC properties was also evaluated in the laboratory where the performance properties of OGFC mixtures made using ten gradations representative of those in use throughout the US were evaluated. The study also surveyed best practices for the construction and maintenance of OGFC mixtures. Finally, results of an externally sponsored study on the laboratory performance of OGFC mixtures made with warm mix asphalt technologies were also included in this report.

Due to the extensive scope of this study, the conclusions and recommendations are organized based on specific aspects of OGFC.

Performance

The primary performance concerns with OGFC mixtures include: Raveling, delamination, clogging, and draindown. Each of these issues are affected by either the mix design, the production and construction methods, the environment, or a combination of these factors.

Raveling can be attributed to the following:

- A mixture having too little binder content to provide adequate cohesion within the mix.
- Oxidation of the binder film over time, which can be accelerated due to the increased porosity of the mixture allowing air flow through the pavement layer. The porosity also exposes more surface area of the mix to oxygen.
- Inadequate compaction due to cooling of the mixture during placement. This is likely the cause of isolated raveling at transverse joints.
- Excessive compaction at transverse joints causing aggregate breakdown.
- Excessive use of release agent chemicals in the paving equipment prior to paving that can affect the adhesion of the binder in the mixture.
- Normal wearing due to traffic over time.

Delamination can be attributed to the following:

- Inadequate tack coat application to ensure sufficient bond between the OGFC and the underlying layer.
- Excessive cooling of the mixture prior to compaction. This also limits the bonding capability with the tack coat.
- Excessive use of release agent chemicals in the paving equipment prior to paving that can affect the mix adhesion.

- Paving OGFC over thermoplastic pavement markings. This was observed in some isolated areas in South Carolina.

Clogging can be attributed to the following:

- Sediment deposited in the voids of the OGFC pavement surface. Such clogging material can be deposited by vehicles (e.g., between tire treads, tire wear, break pad wear, accidental dumping, etc.). Sediment can also be transported by stormwater if appropriate measures (e.g., erosion control, geometric design, shoulder design, drainage) are not taken to minimize such transport.
- Fat spots in the pavement caused by clumping of fibers in the mix.
- Inadequate porosity of the mixture resulting from gradation selection or over-compaction.

Draindown can be attributed to the following:

- Excessive binder content.
- Inadequate amount of stabilizing additives such as fibers.
- Excessive production temperatures.
- Long haul distances or queues at the paving location and high temperatures.

Mix Design

Based on the evaluation of the different mix design procedures currently used in the US, it was evident that different methods yield different results for optimum binder content if all other variables were held constant. The most variability was found for the methods based on the properties of compacted specimens similar to conventional asphalt mix design procedures. The oil absorption and visual determination methods were more consistent and repeatable. Based on these findings, it is recommended that SC-T-91 continue to be used to determine the optimum binder contents of OGFC mixtures. However, it is also recommended to measure the performance of the mix design. At a minimum, this evaluation would include measuring the porosity and permeability of specimens compacted using 50 gyrations of the Superpave gyratory compactor. More in-depth mixture analysis could also evaluate the raveling susceptibility of the mixture. Currently, the Cantabro abrasion test is the most common method adopted to assess the durability of OGFC mixtures. However, research (including this study) has found that this method is highly variable. It is also recommended that further research be conducted to determine a more suitable laboratory test to evaluate the raveling susceptibility of OGFC mixtures.

The evaluation of the aggregate gradation indicated that the aggregate gradation specified by SCDOT ranked in the top half of the gradations studied with respect to permeability, texture, abrasion loss, and rut resistance. It is recommended, however, that the gradation be designed, so that the percent passing the No. 4 (4.75 mm) sieve is less than 20%. This recommendation is based on the results of this study and others (Mallick et al. 2000). The current SCDOT specification requires that the percent passing the No. 4 sieve be 15–25%. Additionally, this study revealed that a gradation containing a higher percentage of material retained on the ⅜-inch sieve exhibited the best all-around performance (permeability, clogging, abrasion resistance, indirect tensile strength) of the ten gradations evaluated. It is recommended to investigate this further.

It is possible to design OGFC mixtures that exhibit desired properties (draindown and durability) without the use of fibers. One study on the use of warm mix asphalt (WMA) technologies is presented in this report. In addition to WMA, there are other alternatives including crumb rubber modified binders and activated mineral binder stabilizers. It is recommended that more research be conducted to further evaluate the performance of OGFC mixtures made without fibers. Removal of fibers from these mixtures will improve the functional performance as porosity will increase, thus increasing the permeability of the mixtures. Additionally, this could potentially result in more consistent mix production and construction quality.

Thickness Design

Based on the assessment of the current performance of OGFC layers in South Carolina and a rational lift thickness design methodology based on rainfall intensity, pavement design parameters, and mix design properties, the thickness of OGFC lifts in South Carolina could be increased to potentially enhance the performance of the pavements. The recommended layer thickness should be a minimum of 1 ¼ inches and no less than 2 times the maximum aggregate size of the OGFC mixture. For pavements having a flow path greater than 14 feet (two lanes of traffic), the thickness should be increased to ensure water does not flow over the pavement surface. The thickness of the OGFC lift can be designed using Equation 6.2.

In addition to increasing the minimum lift thickness, the mixture permeability has a major influence on the ability of an OGFC layer to function as intended. This functionality is commonly reduced due to clogging of the voids in the OGFC. To compensate for this a clogging factor can be applied to the permeability of the mixture. It is recommended that the clogging factor range from 1.2 to 1.4. Applying the clogging factor to the recommended design permeability of 164 in/hr results in a mix design permeability of 196 to 230 in/hr.

Construction

Based on the performance of OGFC mixtures in South Carolina, a main concern is raveling at transverse joints and bridge tie-ins. In these areas, raveling can be severe, but isolated. While more observations are needed, it is speculated that the cause of the isolated raveling in these specific areas is the result of inadequate compaction due to lower mixture temperatures. This may be more of an issue when paving is done at cooler ambient temperatures compared to hot summer months, but should be studied further. Potential strategies to minimize this type of distress include:

- Carefully monitor the mat temperature to ensure that compaction occurs within the proper compaction range.
- Use the first load, or portion of the first load of mix to warm up the material transfer vehicle and possibly the paver and then dispose of the mix. This will prevent the mixture that will actually be placed on the pavement from coming in contact with cold equipment that would consequently cool the mixture.
- Provide additional compaction effort (more roller passes) near these joints to ensure proper mix cohesion. However, care should be taken not to close the pore structure of the mix or cause aggregate breakdown.
- Some have proposed that the first load or two of mix be produced at significantly higher temperatures, so when it exits the cold MTV and paver, the temperature is in the ideal compaction range. While this may produce satisfactory results, caution should be

exercised as excessive mix temperatures can result in premature aging and increased draindown. Both of these consequences can lead to reduced durability resulting in raveling.

Additional attention should also be given to the tack coat below an OGFC. The tack coat should provide full coverage of the pavement lane and be thick enough to promote adhesion of the OGFC layer to the underlying pavement layer. This could potentially be addressed with the use of non-tracking tacks that reduce the amount of the tack coat that is picked up by haul trucks or paving equipment during the paving operation. Another potential solution is spray-applied ultra-thin bonded wearing courses, or ultra-thin asphalt concrete surfacings (UTACS). These newer wearing courses consist of a gap- or open-graded wearing course that is bonded to the underlying pavement surface by a thick polymer-modified asphalt emulsion membrane. This needs to be studied further.

In addition to construction practices, methods should be developed to monitor the quality of OGFC pavement during construction. While it is difficult to monitor pavement density using a nuclear density gauge due to the high void content and thin lifts, there are other potential methods that could be adopted. One such method is to measure the in-situ infiltration rate of the OGFC layer as outlined in Appendix E. This is a simple, non-destructive test that can be conducted as soon as the mat cools. The OGFC layer should have a minimum infiltration value to be functional, so this could be used for quality control and quality assurance.

Maintenance

As with any type of pavement, maintenance is important for OGFCs. Often times, DOTs take a “do nothing” approach to OGFC maintenance because distresses are typically isolated and the layers are thin enough that they do not present a safety hazard when there is raveling or delamination. Additionally, if a section of OGFC were to be patched with conventional HMA, then the lateral flow of water through the pavement layer would not occur. As for clogging, it is difficult to prevent clogging, aside from employing appropriate erosion and sediment control measures. It is also costly to restore permeability once clogging occurs.

Based on findings of this research, the following recommendations pertaining to OGFC maintenance are in order:

- When patching must be done, it can be performed with conventional mix as long as a lateral flow path exists for water to exit the pavement from the area surrounding the patch. Attention must be paid to the location of the patch with respect to grade and cross-slope with the pavement to ensure an adequate drainage path for the water within the OGFC layer. Alternatively, patches can be angled such that the water can flow around them. However, depending on the size of the patch, this may be difficult to accomplish with typical milling equipment.
- The SCDOT *Supplemental Specification for Maintenance OGFC* can be used to design and construct patches using OGFC mixtures. These mixtures were evaluated in the laboratory and performed well with respect to permeability and durability. As with any OGFC mix, care should be taken to select a binder content that will not result in excessive draindown, or exhibit durability issues. The performance of patches placed using these mixtures should be evaluated in the field.
- Surface applications, such as fog seals have been used by DOTs on OGFCs in the past to minimize oxidation in an effort to prevent raveling. While such surface treatments can be

beneficial, one must be cautious with respect to permeability. If an emulsion is applied to an OGFC surface that has been partially clogged, then the clogging material will be trapped in the pavement after the treatment is applied. In this case, permeability cannot be restored. If a clogged OGFC is near the end of its serviceable life, then a fog seal would slow down, or prevent raveling until restoration can take place. However, if a pavement has seen little clogging, then this is not an issue and the treatment may perform as desired as long as application rate is such that it does not clog the pores. This needs further study.

- While clogging may not be a significant issue on the high speed travel lane of an interstate pavement, the shoulder could potentially become clogged due to the minimal traffic action. For this reason, the permeability, or infiltration of OGFC shoulders should be monitored to ensure proper functioning of the entire OGFC pavement layer. If the shoulders become clogged, water flowing within the OGFC layer in the travel lanes will not be able to completely drain out of the pavement, which could potentially lead to stripping or freezing depending on the temperature. If shoulders become clogged, then shoulder declogging strategies should be investigated.

<u>Recommendations for Further Evaluation of OGFCs in South Carolina</u>

Based on the findings of this research, the following topics are recommended for further evaluation related to OGFCs in South Carolina:

- Construction and evaluation of OGFC test sections made with the use of alternatives to stabilizing fibers. These alternatives could include the use of crumb rubber modified binders, warm mix asphalt technologies, mineral filler stabilizers, gradation modifications, or lower production temperatures among others. It should be noted that at the time this report was completed, the SCDOT had already awarded a project to construct two OGFC test sections made without fibers on I-20. This project includes control HMA OGFC sections made with fibers as currently required by SCDOT. In addition, OGFC was produced without fibers using two alternatives: (1) crumb rubber modified binder and (2) Evotherm™ WMA additive. The performance of these and other sections should be monitored for short-term and long-term performance.
- The Cantabro abrasion test is currently the most recommended test to evaluate the durability of OGFC mixtures with respect to raveling. However, this test method has proven to have relatively high variability to be a reliable screening tool. It is recommended that an alternative test procedure be developed or adopted for OGFC mixtures.
- This study included a comparison of ten different OGFC gradations from across the US. While the gradation currently used by SCDOT performed well, it did have the second highest reduction in permeability when exposed to a clogging procedure. It is recommended that the current gradation be compared with gradation B evaluated in this study to determine if the gradation specifications should be modified. Previous research also recommends that the percent passing the No. 4 (4.75 mm) sieve be less than or equal to 20%.
- A visual survey of OGFC pavements in South Carolina revealed that isolated raveling was prevalent near transverse cold joints where paving began, or at bridge tie-ins. One potential cause of this could be cooling of the mixture at start-up or when mobilizing over

bridges. When the mix cools, it cannot be adequately consolidated using normal rolling patterns. It is recommended to study the degree that a mixture cools from the time it exits the haul truck to the time it exits the paver as the paving operation progresses from start-up. The respective pavement sections should then be monitored for premature raveling due to mix cooling. This could help provide further guidance for quality OGFC construction. The degree of cooling may also be dependent on ambient temperature, so it should be monitored throughout the paving season to determine if the mix cooling is more of an issue in cooler temperatures than it is in the heat of the summer months.

- Clogging of the voids in OGFC is the limiting factor of the functional life of the pavements. It is recommended that several OGFC sections be identified across South Carolina to monitor the loss in surface infiltration due to clogging. The infiltration rate of the pavements should be measured immediately after construction to establish a baseline. Subsequently, the infiltration should be measured every 6 to 12 months to track the rate of infiltration loss. As clogging occurs in these sections, maintenance alternatives can be evaluated for larger-scale implantation.
- OGFC layer thickness is a major factor contributing to the desired performance of the pavement layer. If the layer is too thin, it will not accommodate the flow of water that the pavement will be exposed to. In this case, water will flow over the pavement surface thus creating a potential safety hazard. From a pavement condition standpoint, a layer that is too thin will be more likely to experience full-depth raveling of the layer, which will not only increase the roughness of the pavement, but can also create safety hazards. With this in mind, it is recommended that the SCDOT construct OGFC test sections having thicknesses of 1 ¼ to 1 ½ inches and evaluate the long-term performance.

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APPENDIX A: PROPOSED OGFC MIX DESIGN GUIDELINES

Standard Method of Test for
Method of Determining the Optimum Binder Content of Open Graded Friction Course (OGFC) Mixtures
SC Designation: SC-T-XX

1. SCOPE

This method outlines the procedure for designing and preparing an uncompacted bituminous mixture (OGFC) composed of crushed mineral aggregate, polymer modified binder, mineral fiber stabilizing additives, and hydrated lime to determine optimum binder content.

2. REFERENCED DOCUMENTS

2.1. AASHTO Standards
T 245
T 312
2.2. SC Test Methods
T-88
T-90
T-91
2.3. SC Lab Forms
269

3. SUMMARY OF TEST METHODS

3.1. None

4. SIGNIFICANCE AND USE

4.1. The purpose of this procedure is to determine the optimum binder content for an uncompacted bituminous mixture (OGFC).

5. APPARATUS

5.1.

6. TEST SPECIMENS

6.1. A SCDOT certified HMA Design Technician (Level II) must prepare the uncompacted mixture design, and submit the appropriate 269 form and all design data, including optimum asphalt content, and drain down test information.
6.2. Six test specimens, three sets of two specimens each should be blended with binder, weighing approximately 1000 grams total. The first set of two OGFC specimens should be mixed at the optimum asphalt binder content, two 0.5 % above, and two 0.5 % below optimum asphalt binder content.
6.3. One asphalt drain down specimen must be blended with binder at optimum binder content, weighing approximately 350 grams total. This will be used to see if there is enough asphalt binder retained.

6.4. One 1000 grams batch, without asphalt binder, to be used for the SCDOT verification-check sample.
6.5. Containers of Polymer Modified Asphalt Binder, and mineral fibers need to be obtained and proportioned to the correct amounts.

7. PROCEDURE

7.1. These steps will be performed by the Contractor's Level II technician.
7.2. Verify and determine the optimum asphalt content of the uncompacted asphalt blend. The mixtures should be placed into clear pyrex type dishes, which have minimum surface areas of at least 100 sq. in., and a minimum of 1 ½ in. of depth. The mix is allowed to stand inside a calibrated oven at mixing temperature for 2 hrs. The optimum asphalt content is determined by judging the appearance of the asphalt through the pyrex dishes. The optimum binder content should be determined by observing the excessive mixture draindown, or filling of uncompacted air voids through the pyrex dish. The technician must be careful not to allow the mixture to slide, or move while observing the uncompacted mixture.

7.3. Perform SC-T-90 to determine the amount of binder retention at optimum asphalt binder content. This will eliminate the use of excessive binder content in the OFGC. Adjustment of optimum binder content or dosage rate of mineral fibers may be required, in order to meet retention coating of the uncompacted mixture.
7.4. If either of the uncompacted blends do not compare, the technician must redesign a new mixture to meet SCDOT specifications.
7.5. Compact two (2) 150 mm diameter by 115±5 mm tall OGFC mix specimens at the binder content selected from steps 7.2 and 7.3. The specimens shall be compacted using 50 gyrations of a Superpave gyratory compactor. Addtionally, two (2) specimens shall be prepared at binder contents that are 0.5% below and above the selected binder content.
7.6. Measure the porosity of each of the compacted specimens in accordance with the procedure outlined in SC-T-XX (proposed porosity procedure included in Appendix B).
7.7. Measure the permeability of each of the compacted specimens in accordance with SC-T-XX (proposed laboratory permeability procedure included in Appendix C).
7.8. Measure the abrasion resistance of each of the compacted specimens in accordance with SC-T-XX (proposed mixture abrasion resistance procedure included in Appendix D).
7.9. Adjust the optimum binder content of the OGFC mixture based on the mixture performance tests and the requirements in Table 1.

Table 1. Required properties for OGFC mixtures at optimum binder content.

Property	Value	Test Procedure
Binder retention	≥ 95.5%	SC-T-90
Porosity	≥ 13%	SC-T-XX
Permeability	≥ 200 in/hr.	SC-T-XX
Abrasion loss	≤ 20%	SC-T-XX

8. CALCULATIONS

8.1. As per AASHTO T-11, AASHTO T-27, SC-T-90, SC-T-91, SC-T-XX, SC-T-XX, SC-T-XX.

9. REPORT

9.1. The contractor must submit a 269 form, along with copies of the mix design results; along with at least one verification sample to the Research & Materials Laboratory for mix verification and approval of asphalt mix design.

Appendix B: OGFC Specimen Porosity Test Procedure

Standard Method of Test for

Porosity of Compacted Open Graded Friction Course (OGFC) Mixture Specimens
SC Designation: SC-T-XX

1. SCOPE

This procedure is to measure the porosity (water accessible voids) of compacted asphalt concrete specimens. This procedure is applicable to OGFC mixtures.

2. REFERENCED DOCUMENTS

2.1. SC Test Procedures
T-XX

3. APPARATUS

3.1. Balance with ample capacity and sufficient sensitivity
3.2. Under water weighing holder
3.3. Thermometer – calibrated liquid-in-glass
3.4. Water bath
3.5. Calipers

4. TEST SPECIMENS

4.1. Specimens may be cored from the roadway or compacted in the laboratory.

5. PROCEDURE

5.1. Record the dry mass of the specimen to the nearest 0.1g (W_{dry}).
5.2. Measure and record the height and diameter of the specimen at three representative locations to the nearest (0.1mm). Calculate the average height (H_{avg}) and diameter (D_{avg}) of the specimen.
5.3. Submerge the specimen in 77^{o}F (25^{o}C) water for 30 minutes.
5.4. After 30 minutes, keeping the specimen submerged, invert the specimen 180^{o} (flip it over) being sure not to remove it from the water at all.
5.5. Keeping the specimen submerged, tap the specimen against the bottom of the tank 5 times without damaging the specimen, then invert it 180^{o} (while fully submerged).
5.6. Measure the submerged mass of the specimen under water without exposing it to air and record (W_{sub}).

6. CALCULATIONS

6.1. Calculate the volume of the specimen using Equation 1.

$$V_T = \frac{(D_{avg})^2 \times \pi \times H_{avg}}{4} \quad (1)$$

6.2. Calculate the porosity of the specimen using Equation 2.

$$P(\%) = \left[1 - \frac{\frac{(W_{dry} - W_{sub})}{\rho_W}}{V_T}\right] \times 100 \qquad (2)$$

7. REPORT

7.1. Average specimen diameter to the nearest 0.01 in.
7.2. Average specimen height to the nearest 0.01 in.
7.3. Porosity to the nearest 0.1%.

APPENDIX C: LABORATORY PERMEABILITY TEST PROCEDURE

Standard Method of Test for
Laboratory Determination of Permeability of Compacted Open Graded Friction Course (OGFC) Mixture Specimens
SC Designation: SC-T-XX

1. SCOPE

This procedure is to measure the permeability of compacted asphalt concrete specimens in the laboratory. This procedure is applicable to OGFC mixtures.

2. REFERENCED DOCUMENTS

2.1. SC Test Procedures
T-XX

3. APPARATUS

3.1. Calipers
3.2. Permeameter
3.3. Moldable sealant such as plumbers putty
3.4. Plastic wrap
3.5. Petroleum jelly

4. TEST SPECIMENS

4.1. Specimens may be cored from the roadway or compacted in the laboratory.

5. PROCEDURE

5.1. Measure the porosity of each specimen in accordance with SC-T-XX (proposed porosity procedure included in Appendix B).
5.2. Measure and record the height and diameter of the specimen at three representative locations to the nearest (0.1mm). Calculate the average height (H_{avg}) and diameter (D_{avg}) of the specimen.
5.3. Tightly wrap plastic film around the circumference of the specimen to ensure water will not flow out of the sides of the specimen during the test.
5.4. Place the specimen in the standpipe of the permeameter so the bottom of the specimen is resting on the bottom of the standpipe. A thin coat of petroleum jelly may be applied to the outside of the wrapped specimen to help facilitate placement in the permeameter.
5.5. Apply plumbers putty around the top edge of the specimen to seal the gap between the specimen and the inner wall of the standpipe.
5.6. Make marks on the standpipe 3, 12, and 15 inches above the top of the specimen.
5.7. Adjust the level of the outlet so it is level with the top of the specimen.
5.8. Open the valve and add 77^{o}F (25^{o}C) water to the permeameter through the standpipe. Once water begins flowing out of the outlet, close the valve and continue to add water until the level reaches the mark at 15 inches above the specimen. Tap the sides of the standpipe to remove air bubbles from the specimen. If needed, add more water until water is at the 15 inch mark.

5.9. Open the valve to begin water flow. Start the timer when the water level reaches the 12 inch mark and stop the timer when the water reaches the 3 inch mark. Record this time (t_i).

5.10. Repeat steps 5.7 and 5.8 a total of three times.

6. CALCULATIONS

6.1. Calculate the permeability of the specimen using Equation 1.

$$k = \frac{aL}{At} ln \frac{h_1}{h_2} \times 3600 \tag{1}$$

Where,

k	= permeability (in/hr)
a	= cross-sectional area of the standpipe (in^2)
L	= specimen height (in.)
A	= cross-sectional area of the specimen (in^2)
t	= time for water to drain from h_1 to h_2 (s)
h_1	= height above specimen when timing starts (12 in.)
h_2	= height above specimen when timing ends (3 in.)

7. REPORT

7.1. Average specimen diameter to the nearest 0.01 in.

7.2. Average specimen height to the nearest 0.01 in.

7.3. Average permeability to the nearest 1 in/hr.

APPENDIX D: ABRASION RESISTANCE OF OGFC MIXTURES

Standard Method of Test for
Abrasion Resistance of Open Graded Friction Course (OGFC) Mixtures
SC Designation: SC-T-XX

1. SCOPE

This procedure is to estimate the abrasion resistance of compacted asphalt concrete specimens. This procedure is applicable to OGFC mixtures.

2. REFERENCED DOCUMENTS

2.1. AASHTO Test Procedures
T96
2.2. SC Test Procedures
T-XX

3. APPARATUS

3.1. Balance with ample capacity and sufficient sensitivity
3.2. LA Abrasion machine
3.3. Thermometer

4. TEST SPECIMENS

4.1. Specimens shall be compacted in the laboratory to a height of 115±5 mm using 50 gyrations of a Superpave gyratory compactor.
4.2. At least two specimens (preferably three) are required per mixture being tested.

5. PROCEDURE

5.1. Measure the porosity of each specimen in accordance with SC-T-XX (proposed porosity procedure included in Appendix B).
5.2. The test procedure is 77°F (25°C).
5.3. Record the weight of the specimen to the nearest 0.1g (W_1).
5.4. Keep the specimen at the test temperature for at least 4 hours prior to testing.
5.5. Place the specimen in the clean LA abrasion drum without any steel spheres, start the machine and allow it to run for 300 revolutions. The drum shall rotate at a rate of 30 to 33 revolutions per minute.
5.6. After 300 revolutions, remove the specimen from the drum and lightly brush it off to remove any loose particles and dust.
5.7. Record the weight of the specimen to the nearest 0.1g (W_2).

6. CALCULATIONS

6.1. Calculate the loss due to abrasion of each specimen using Equation 1.

$$\%\,Loss = \frac{W_1 - W_2}{W_1} \times 100\% \qquad (1)$$

6.2. Calculate the average *% Loss* of all specimens tested.

7. REPORT

7.1. Mass of each specimen before testing (W_1) to the nearest 0.1g.
7.2. Mass of each specimen after testing (W_2) to the nearest 0.1g.
7.3. Test temperature.
7.4. *% Loss* for each specimen and the average *% Loss* for all specimens to the nearest 1%.
7.5. Porosity of each specimen to the nearest 0.1%.

APPENDIX E: INFILTRATION TEST FOR OGFC PAVEMENTS

Standard Method of Test for
In-situ Infiltration Rate of Open Graded Friction Course (OGFC) Pavements
SC Designation: SC-T-XX
(This procedure has been adopted from ASTM C1701)

1. SCOPE

This procedure is to measure the infiltration rate of in place OGFC pavements.

2. REFERENCED DOCUMENTS

2.1.

3. APPARATUS

3.1. Brush or broom – A brush or broom to sweep the area to be tested.
3.2. Infiltration ring – A cylindrical ring, open at both ends that is watertight and sufficiently rigid to retain its shape when filled with water. The ring shall have a diameter of 12 ± 0.5 in. with a minimum height of 2 in. The bottom edge of the ring shall be even. The inner surface of the ring shall be marked or scored with two lines at a distance of 0.4 and 0.6 in. from the bottom of the ring.
3.3. Measuring container – Graduated container capable of measuring 1 gallon of water.
3.4. Container – A plastic 5 gallon bucket to be used to pour water into the infiltration ring.
3.5. Stop watch – Accurate to 0.1s.
3.6. Plumbers putty (non-hardening)
3.7. Water

4. TEST LOCATIONS

4.1. Perform tests at multiple locations at a site.
4.2. Provide at least 3 ft. of clear distance between test locations, unless at least 24 hours have elapsed between tests.
4.3. Do not test if there is standing water on the pavement or within 24 hours of the last precipitation.

5. PROCEDURE

5.1. Clean the pavement surface by brushing loose material from the pavement surface where the test is to be conducted.
5.2. Apply plumbers putty around the bottom of the infiltration ring and place the ring onto the pavement surface. Press the putty into the surface and around the bottom edge of the ring to create a watertight seal. Use additional putty as needed.
5.3. Pour 1 gallon of water into the ring at a sufficient rate to maintain the water level between the two marked lines. Begin timing as soon as the water impacts the pavement surface and stop timing when water is no longer present on the pavement surface. Record the time to the nearest 0.1 s.
5.4. Repeat step 5.3 so the test has been conducted a total of three times.

6. CALCULATIONS

6.1. Calculate the infiltration rate (I) using Equation 1.

$$I = \frac{832000}{A \times t} \tag{1}$$

Where,

A = inside area of the infiltration ring (in^2)
t = time (s)

6.2. Calculate the average infiltration rate of tests 2 and 3.

7. REPORT

7.1. Time elapsed since last rain event, if known.
7.2. Inside diameter of infiltration ring to the nearest 0.01 in.
7.3. Time elapsed for each of the three test runs to the nearest 0.1 s.
7.4. Infiltration rate of each test run to the nearest 1 in/hr.
7.5. Average infiltration rate of test runs 2 and 3 to the nearest 1 in/hr.